THE INDIAN RIVER INLET BRIDGE:

CHANGING FROM A SINGLE RIB TIED ARCH

TO A CABLE-STAYED DESIGN

by

Timothy Jeffrey Stuffle

A thesis submitted to the Faculty of the University of Delaware in partial fulfillment of the requirements for the degree of Master of Civil Engineering

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ABSTRACT

The University of Delaware's Center for Innovative Bridge Engineering (CIBrE) developed an instrumentation and monitoring program for a proposed 1,000 foot concrete tied arch bridge over the Indian River Inlet in Southern Delaware. That bridge will not be built due to budgetary constraints, however, a new bridge is still needed. It is believed that the new bridge, which will be instrumented and monitored by CIBrE, will be a cable-stayed bridge.

This thesis examines the Indian River Inlet site, the proposed arch bridge and instrumentation plan, and cable-stayed bridge related issues. The Indian River Inlet site and the proposed arch bridge are introduced. The expectation of a cable-stayed bridge being built at the inlet is explained. The history and evolution of cable-stayed bridges are examined. Terminology and structural systems relevant to modern cable-stayed bridges are introduced. Behavior of cable-stayed bridges is explored through use of simplified models which represent the limiting conditions of actual bridges. The phenomenon of stay-cable vibration is also discussed briefly. Finally, conclusions about the behavior of cable-stayed bridges and how that relates to the monitoring of a cable-stayed bridge at the Indian River Inlet site are made.

Chapter 1

INTRODUCTION

1.1 The Indian River Inlet Bridge

The Indian River Inlet Bridge is located on Delaware State Route 1 (SR 1) between Dewey Beach and Bethany Beach, Delaware as shown in Figure 1.1. The bridge spans the Indian River Inlet, which connects the Indian River Bay and the Rehoboth Bay to the Atlantic Ocean.

Figure 1.1 Map of Indian River Inlet Area (Modified from mapmachine.nationalgeographic.com)

The inlet functions as a passage for recreational boaters to travel from marinas and boat launches located in the bays to the Atlantic Ocean. SR 1 functions as a vital north-south link for vehicular and pedestrian traffic in the area which is a popular tourist destination during the summer months.

Historically, the inlet moved up and down the coast over approximately a two mile stretch. In 1928 the inlet began to be dredged to ensure it was passable to boat traffic, and in 1938 jetties were built to prevent the inlet from wandering as it had in the past. The jetties were successful at fixing the location of the inlet, however, the depth of the inlet has increased due to scour since the building of the jetties. The depth of the inlet in 1938 was 12 ft, by 1965 the depth at the center of the channel had increased to 28 ft, and in 2005 the average depth had increased to over 52 ft with scour holes having depths of 100 ft or more in several locations (DelDOT May 2004, 5-7).

Several bridges were built over the inlet it in the past century. The first bridge was a timber bridge built in 1934, the second was a swing-span built in 1938, the third, built in 1952, was also a swing-span, and construction of the current bridge started in 1965. From the dates that these bridges were constructed it can be seen that bridges constructed at the inlet tend to be rather short lived. The current bridge consists of twin spans which were constructed at different times. The foundations for both structures and the superstructure for what is currently the northbound bridge were built in 1965. The superstructure of the southbound bridge was not constructed until 1976 (DelDOT May 2004, 6-7). During the late 1980's, scour exposed the pilings of the existing bridge. In 1989, a scour mitigation project was conducted to protect the foundations of the existing bridge. The project involved the placement of large pieces

of rip-rap, some as large as cars, around the foundations to prevent the material surrounding the pilings from washing away (O'Shea and Pate 2003, 16). It was believed that the rip-rap would adequately protect the bridge foundations from scour, however, underwater inspections performed in 1996 and 2001 determined that the rip-rap was moving. The Delaware Department of Transportation (DelDOT) considers the bridge to be safe, but there are concerns about the continued scour and destabilization of the rip-rap. To avoid the possibility of SR 1 being closed to thru traffic if the current bridge needs to be closed, DelDOT has decided to build a new bridge. Because scour is a continuing problem at the site, it was also decided that the new bridge should not have piers in the water (DelDOT December 2004, 8-9). The inlet is currently about 500 ft wide, however, the possibility of the inlet being widened to 800 ft exists (Figg 2005, B-16). Therefore, the main span of the new bridge was chosen to be 1,000 ft.

1.2 Proposed Arch Bridge

The Figg Engineering Group (Figg) was selected by DelDOT to develop a bridge design that would meet the physical constraints of the site, while also pleasing the public. During April and May of 2004 several public design charettes were held to gather public input on the new bridge. The result of these charettes, which is shown in Figure 1.2, was a concrete tied arch bridge which uses cables that hang from the arch to support the roadway.

Figure 1.2 Rendering of the Proposed Arch Bridge (O'Shea and Pate 2003, 16)

The proposed bridge has a 1,000 ft main span and two 150 ft side spans (O'Shea and Pate 2003, 16-17). The bottom of the roadway sections are 45 ft above the inlet, while the top of the arch is 278 ft above the inlet. There are two 12 ft traffic lanes and a 10 ft shoulder for each direction of travel. The northbound side of the bridge also has a 12 ft sidewalk for pedestrian and bicycle traffic (Figg 2005, B-16, B-19). Details of the materials, cross sections, and construction process are detailed in the University of Delaware master's thesis of Julia West which is titled "Structural Health Monitoring of a Signature Bridge."

Because the arch bridge was to be a significant and unusual structure, it was decided that the Center for Innovative Bridge Engineering (CIBrE) at the University of Delaware would develop and implement an instrumentation plan to monitor the bridge during construction and in service. In establishing the monitoring program for the Indian River Inlet Bridge, careful attention was focused on creating a

system that would assist in the construction and maintenance of the bridge, as well as provide design verification for certain aspects of its behavior. The goal of the plan was that the data collected from sensors would be used in conjunction with analytical models to address the specific issues associated with the unique structure (West 2005, 35). Details of the specific issues that the monitoring program was intended to address are given in "Structural Health Monitoring of a Signature Bridge" by Julia West.

The development and organization of the monitoring program was a significant task. To sufficiently plan for the successful implementation of the program a manual titled "Indian River Inlet Bridge Monitoring Program Manual" was developed by Dan Weston and Tim Stuffle. The entire manual is included in Appendix A and an overview of the manual is presented Chapter 2.

Bids for the construction of the arch bridge were solicited by DelDOT in the spring of 2005, however by the October 2005 bid deadline, only one bid was submitted. The bid was substantially above the estimated cost for constructing the bridge. In response to this, DelDOT cancelled the bid opening and abandoned their plans to construct the proposed arch bridge. In a news release dated October 3, 2005, DelDOT stated several factors that led to this outcome:

> …Competition is intense for major infrastructure projects nationwide, due to a significant federal influx of money in response to the hurricanes in the Gulf Coast…
>
> Materials, labor and fuel continue to increase at record levels and contractors anticipate major costs in those areas when bidding on projects…
>
> With DelDOT's current financial issues ($287 million in projects were delayed in fiscal year 2006), approving a bid that comes in almost 100 percent higher than the estimate is not acceptable…

> As the current Indian River Inlet bridge design would be the first of its kind in the U.S., there were few contractors interested. With plenty of major infrastructure work available nationwide, contractors do not appear willing to take on a bridge project that has not been built previously in the U.S.

In the same press release DelDOT Chief Engineer Carolann Wicks is quoted as saying:

> We are reviewing our options right now, but make no mistake that from an engineer's perspective a new bridge must be built, and we will build it. What this process shows us is that the construction industry does not appear ready to build this particular type of bridge at this time.

At the time that the bid was cancelled, a roadway contract which included building approaches, retaining walls, abutments, and roadways associated with the proposed arch bridge had already been awarded. These items are currently being constructed at the site, and will be incorporated, as much possible, into any new bridges designed for the site.

1.3 Anticipated Cable-Stayed Bridge

Because the proposed arch bridge design will not be used, but a new bridge is still needed, there has been speculation about what type of bridge will be built at the Indian River Inlet site. One strong possibility is that a traditional cable-stayed bridge will be built. If this happens, it is likely that CIBrE will develop and implement an instrumentation and monitoring program for the new bridge design. Issues relating to this possibility are discussed in the University of Delaware master's thesis by Dan Weston which is titled "Existing and Future Plans for the Structural Health Monitoring of the Indian River Inlet Bridge."

Chapter 2

MANUAL FOR PROPOSED ARCH BRIDGE

2.1 Purpose of the Manual

The implementation of the monitoring project for the proposed arch bridge would have been logistically challenging and required extensive planning. In recognition of this fact, CIBrE developed a manual to ensure that the project went as smoothly as possible by making all involved parties aware of the necessary steps in completing the project. The manual was written to explain in detail CIBrE's plans to instrument and monitoring the proposed arch bridge. The manual details important information relating to the instrumentation plan in all stages of the project. The manual was intended to serve as an aid to CIBrE, DelDOT, the contractor, the designer of the bridge (Figg), and the independent reviewer of the bridge (T.Y. Lin). It should be noted that DelDOT cancelled plans to build the proposed arch bridge while the manual was still being written. Because much of the material contained in the manual is specific to the proposed arch bridge design, it was determined that continued work on the manual was not productive. As a result, a few sections of the manual are not complete. The manual is included in its entirety in Appendix A.

2.2 Contents of the Manual

The manual for the proposed arch bridge consists of eleven chapters which are described in the following sections.

2.2.1 Chapter 1, Purpose of the Manual

Chapter 1 introduces the proposed arch bridge, the need for DelDOT to efficiently manage the bridge, and a brief description of CIBrE's goals for the monitoring program. This chapter also identifies who will benefit from the monitoring program and the manual.

2.2.2 Chapter 2, Scope and Goals of the Project

Chapter 2 introduces the general goal of collecting data from sensors to be used in conjunction with computer models to address the specific design, construction, and maintenance issues associated with the Indian River Inlet Bridge. The chapter also identifies five stages that the project has been divided into. Each stage is then defined and a brief overview of the goals for each stage is presented. The five stages include: preliminary work, mechanically stabilized earth wall construction, bridge construction, initial condition, and long term condition.

2.2.3 Chapter 3, Responsibilities

Chapter 3 is intended to identify the specific responsibilities of CIBrE and the contractor for each portion of the instrumentation program. Responsibilities included in this chapter are intended to clearly define who is responsible for tasks such as: purchasing gages, installing gages, moving materials within the construction site, providing access to specific locations, and other tasks that must be performed in order to implement the monitoring project. The chapter acts as a supplement to special provision 763621 "University of Delaware Bridge Monitoring Program" by fully explaining the special provision and including details which were not available at the time the special provision was written.

2.2.4 Chapter 4, Monitoring Project Timeline

The implementation of the monitoring project would have been logistically challenging and required extensive planning. Chapter 4 was written to address this fact. The chapter identifies the overall timeline of the entire project and then directs readers to future chapters which address in detail each stage of the project.

2.2.5 Chapters 5 through 9, Project Stages

Chapters 5 through 9 provide an in depth description of the objectives of each stage and list specific tasks that must be accomplished in order to complete the stated objectives. The chapters are titled as follows: Chapter 5, Stage I – Preliminary Work, Chapter 6, Stage II – Mechanically Stabilized Earth Wall Construction, Chapter 7, Stage III – Bridge Construction, Chapter 8, Stage IV – Baseline Behavior, Chapter 9, Stage V – Long-Term Conditions.

2.2.6 Chapter 10, Instrumentation

A variety of different gages and sensors will be used in the instrumentation of the Indian River Inlet Bridge. Chapter 10 is intended to identify these gages and sensors and to give important information relating to their purpose, purchase, installation, setup and use. Currently the chapter includes how each gage is mounted, what each gage measures, what the measurements will be used for, as well as miscellaneous considerations for each sensor. The chapter is incomplete and upon completion it should be expanded to list specific brands and models of sensors to be used, list sources for purchasing the sensors, cite relevant manuals related to the sensors, and give a step by step procedure for the installation and set up of each sensor.

2.2.7 Chapter 11, Reporting

The Indian River Inlet Bridge Monitoring Program involves installing many gages and sensors on and in the bridge. Data from these instruments will be recorded by data acquisition systems. However, the raw data itself will be of little use until it is processed and interpreted. The results of the processing and interpretation must then be put into report form so that the information may be conveyed to interested parties to allow the goals of the instrumentation program to be met. Chapter 11 includes, for each different stage of the project, the purpose of the reports, what should be included in these reports, when the reports should be issued, and what parties may be interested in the information contained in the reports.

2.3 Adapting the Manual for a Cable-Stayed Bridge

At the current time, it is anticipated that the proposed arch bridge will not be built and that a cable-stayed bridge, which has not yet been designed, will be built at the Indian River Inlet site. It is also anticipated that CIBrE will develop and implement an instrumentation and monitoring program for the cable-stayed bridge. If this materializes, a manual similar to the one developed for the proposed arch bridge will be needed. Many aspects of a manual for a cable-stayed bridge will be similar to those of the proposed arch bridge manual. It will therefore be possible to reuse many of the ideas presented in the manual. Unlike the instrumentation program for the arch bridge, instrumentation programs for cable-stayed bridges have already been performed. It will therefore be helpful to research how others have gone about performing this task in order to learn from their successes and failures. These issues are explored in "Existing and Future Plans for the Structural Health Monitoring of the Indian River Inlet Bridge" by Dan Weston.

Chapter 3

CABLE-STAYED BRIDGES: HISTORY AND TERMINOLOGY

3.1 Introduction

It is anticipated that a cable-stayed bridge will be built at the Indian River Inlet site and that CIBrE will develop an instrumentation and monitoring program for the bridge. To successfully accomplish this task, it is necessary to learn as much as possible about cable-stayed bridges. This chapter will introduce the history of cable-stayed bridges as well as terminology used to describe modern cable-stayed bridges.

A cable-stayed structure is a structure that contains a beam which is supported by tension members which are anchored at one end to the beam and at the other end to a pylon. The concept of using stayed structures is not at all new, and throughout history many forms of stayed structures have been used with varying degrees of success.

Ancient Egyptian sailors hung rope stays from the mast of their ship to support the boom. Stayed bridges built during the nineteenth century used iron bars and chains to support their spans, however, several bridges of this era collapsed. A combined suspension and stayed system was used by the famous American bridge designer, John Roebling, in several of his monumental bridges. The most famous of these bridges, the Brooklyn Bridge, still serves as a vital link between Brooklyn and Manhattan 123 years after its construction. The use of stayed structures throughout history has culminated in the development of the modern cable-stayed bridge.

Modern cable-stayed bridges, which use high strength steel wire for the stays, are considered ideal for medium spans, which are too long for girder bridges, but are too short to warrant suspension spans. Modern cable-stayed bridges have been built in a variety of sizes, shapes, and configurations. The configurations can generally be described by six basic characteristics: longitudinal stay layout, pylon type, transverse stay layout, stiffening girder type, anchoring method, and longitudinal pylon fixity. Each configuration has advantages and disadvantages, therefore designers must choose which combination of these characteristics will create a configuration that is appropriate for a given bridge. This selection is based on factors such as span length, environmental conditions, aesthetics, constructability, material availability, cost, strength requirements, and stiffness requirements.

The remainder of this chapter focuses on the history of cable-stayed bridges and the terminology used to describe them. Section 3.2 introduces terminology used to describe the characteristics of modern cable-stayed bridges, and Section 3.3 gives a brief history of the use of cable-stayed structures and the development of the modern cable-stayed bridge.

3.2 Cable-Stayed Bridge Terminology

This section introduces the terminology used to describe cable-stayed bridges. The section focuses on the six main characteristics of cable-stayed bridges which are: longitudinal stay layout, pylon type, transverse stay layout, stiffening girder type, anchoring method, and longitudinal pylon fixity. Examples of commonly used arrangements and component types are given. However, these examples are not intended to form an exhaustive list. It should also be noted that the names of the arrangements and component types are not always uniformly accepted. Different

people often use different terminology for the same arrangement, or the same terminology for different arrangements. This is especially true for the names given to longitudinal stay layouts. To avoid confusion sketches and pictures are included for each arrangement and component, and terminology will be used consistently within this document.

3.2.1 Longitudinal Stay Layout

The first characteristic that is noticed when viewing a cable-stayed bridge is the longitudinal layout of the stays. Many configurations for longitudinal stay layout have been used in modern cable-stayed bridges. Some layouts are used because they are efficient, while others are used for aesthetic purposes. The three most prevalent longitudinal stay layouts used for cable-stayed bridges include the radiating layout, harp layout, and fan layout.

The radiating layout uses stays that converge at the top of the pylon and are more or less evenly spaced when they anchor to the stiffening girder. The name "radiating" comes from the fact that the stays radiate from the top of the pylon. Sketches of radiating layouts with varying numbers of stays are shown in Figure 3.1.

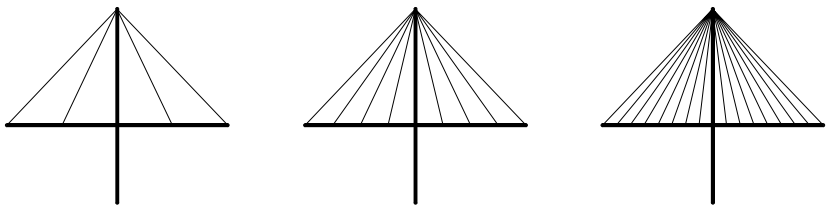

Figure 3.1 Sketches of Radiating Stay Layouts (Adapted from Troitsky 1988, 20)

The harp layout uses stays that are parallel or nearly parallel to one another. The name of the layout comes from the fact that the parallel configuration of the stays resembles the strings of a harp. Sketches of harp layouts with different numbers of stays are shown in Figure 3.2.

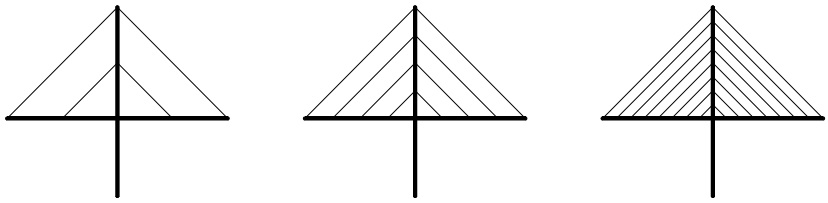

Figure 3.2 Sketches of Harp Stay Layouts (Adapted from Troitsky 1988, 20)

The fan layout is a compromise between the radiating and harp layouts. In the fan layout the stays are evenly spaced where they anchor to the stiffening girder, but they become more closely spaced where they anchor to the pylon. The name of the layout comes from the visual similarity of the system to a fan made of folded paper. The fan layout combines the radiating and harp layouts to overcome the drawbacks of each while retaining each layout's benefits. Sketches of fan layouts with different numbers of stays are shown in Figure 3.3.

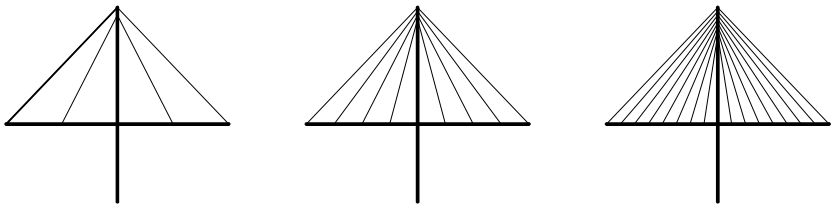

Figure 3.3 Sketches of Fan Stay Layouts (Adapted from Troitsky 1988, 20)

In the prior discussion, the stays were shown to intersect the pylon, but details of these connections were not discussed. However, the method of connection influences the overall behavior of the bridge, so connection methods will now be discussed briefly. The stays of a cable-stayed bridge may be attached to the pylon using two methods: anchors or saddles. When anchors are used, the stays are terminated inside of the pylon. At the location of termination, a mechanical anchorage is used to transfer the cable force into the pylon. When saddles are used, the stays continue through the pylon and are supported through side pressure and side friction. The saddle may be fixed to the pylon by bearings which provide both horizontal and vertical restraint (fixed saddle) or by horizontally moveable bearings which provide only vertical restraint (moveable saddle). Cables are not permitted to move longitudinally over the saddle. This type of movement is resisted by friction between the cables and the saddle. Often a clamping force is required to ensure adequate friction is available. Cable-stayed bridges generally use either anchors or fixed saddles. These two methods allow both vertical and horizontal loads to be transferred to the pylon. A few early cable-stayed bridges utilized moveable saddles. This does not permit horizontal forces to be transferred to the pylon, and was done to limit bending in the pylon. However, moveable saddles result in a less efficient system, and are generally no longer used (Gimsing 1997, 395-399). It will be assumed that all bridges discussed in the remainder of this paper use anchors or fixed saddles to connect the stays to the pylon.

3.2.2 Pylon Type

The second characteristic of cable-stayed bridges that will be discussed is the pylon type. Many different designs have been used in modern cable-stayed

bridges, and the possibility for new designs is bounded only by the imagination of the designer. However, the multitude of pylon types may be classified into two general classes: cantilevered towers and frames. The two classes are differentiated by how transverse loads are transmitted to the foundation. This section will introduce the two classes of pylons as well as several common pylon layouts from each class.

Cantilevered towers are usually in the form of a single central tower, twin towers, or a side tower. In all three cases the towers are cantilevered in the transverse direction to allow the transmission of lateral loads to the foundation. The Sunshine Skyway Bridge shown in Figure 3.4 utilizes single central towers which are located in the median of the roadway. The slender central towers of the Sunshine Skyway anchor to massive piers below the level of the roadway.

Figure 3.4 Sunshine Skyway (Photo by U.S. Department of Interior, Source: en.wikipedia.org, Public Domain Photo)

The Kessock Bridge shown in Figure 3.5 uses twin towers which are located on either side of the roadway. The Kessock Bridge is similar to the Sunshine Skyway in that

the towers are slender above the roadway and anchor to massive pier structures below the elevation of the roadway.

Figure 3.5 Kessock Bridge (Copyright by Toby Speight, Source: geograph.org.uk, Licensed for Reuse under Creative Commons 2.0)

The Sundial Bridge shown in Figure 3.6 is a pedestrian bridge that utilizes a side tower which is located to one side of the walkway. This pylon type is not efficient for the transmission of large loads, so its use is generally confined to pedestrian bridges.

Figure 3.6 Sundial Bridge (Photo by C. Antkowiac, Source: en.wikipedia.org, Licensed for Reuse under Creative Commons 1.0)

Frame pylons come in a variety of shapes, some of the more common frame pylons are the: portal frame, A-frame, and modified A-frames. In frame pylons forces in the transverse direction are transmitted to the foundation by frame action. The New Severn Bridge, shown in Figure 3.7, utilizes a portal frame. The portal frame is similar to the twin tower configuration because of the use of two parallel vertical members. However, unlike a twin tower configuration, portal frames utilize one or more cross-members which are located above the roadway to join the vertical members.

Figure 3.7 New Severn Bridge (Photo by Eligijus Mecionis, Source: en.structurea.de, Reuse Permitted)

The Severins Bridge shown in Figure 3.8 utilizes an A-frame pylon. Pure A-frame pylons are characterized by straight inclined pylon legs that meet at some point above the roadway.

Figure 3.8 Severins Bridge (Photo by Armina, Source: commons.wikimedia.org, Licensed for Reuse under Gnu Free Documentation License Version 1.2)

Many variations of the A-frame have been used in cable-stayed bridges. Modified A-frames are often called diamond, delta, or λ pylons depending on their specific shape. In modified A-frames the pylon legs often change direction at or below the level of the roadway. They may also utilize cross members or have a vertical member extending from the top of the pylon. However different the shapes may be, modified A-frames generally have inclined legs which form symmetric triangles and/or trapezoids. The interesting shapes of these systems are considered to be aesthetically pleasing by many and the geometry often possesses advantages over the more traditional portal frame and A- frame pylon types. The Anzac Bridge in Figure 3.9 uses a type of modified A-frame pylon which is commonly referred to as a diamond pylon. The Normandie Bridge shown in Figure 3.10 uses a type of modified A-frame pylon which is commonly referred to as a λ pylon.

Figure 3.9 Anzac Bridge (Photo by Ian G. Bowie, Source: en.structurae.de, Reuse Permitted)

Figure 3.10 Normandie Bridge (Photo by S. Moeller, Source: commons.wikimedia.org, Public Domain Photo)

3.2.3 Transverse Stay Layout

The third characteristic of a cable-stayed bridge is the transverse layout of the stays. This characteristic may be described by the number and orientation of planes of stays. Figure 3.11 shows three possibilities for a system that contains a single plane of stays. Arrangement A uses a single central vertical plane of stays. This arrangement is used in the Sunshine Skyway Bridge shown in Figure 3.4. Arrangement B uses a single offset vertical plane of stays, and arrangement C uses a single offset inclined plane of stays. Arrangements B and C are generally only used for pedestrian bridges with side towers. The Sundial Bridge, which is shown in Figure 3.6, utilizes Arrangement C.

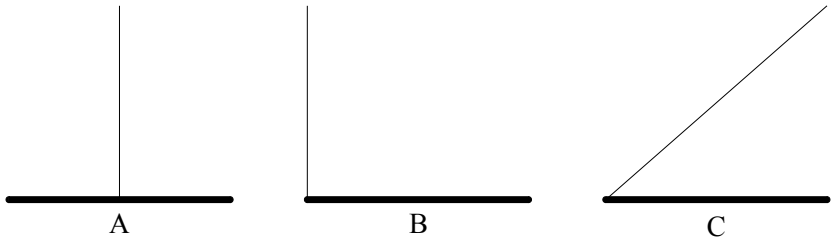

Figure 3.11 Cable-Stayed Systems with a Single Plane of Stays (Adapted from Podolny and Scalzi 1976, 31)

Figure 3.12 shows two possibilities for cable-stayed systems which utilize two planes of stays. Arrangement A uses the double vertical plane stay layout, which was used in the New Severn Bridge of Figure 3.7. Arrangement B uses the double inclined plane stay layout which was used for the Anzac Bridge of Figure 3.9.

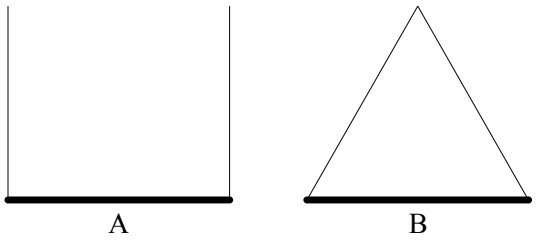

Figure 3.12 Cable-Stayed Systems with two Planes of Stays (Adapted from Podolny and Scalzi 1976, 31)

The use of three or more planes of stays is also possible, especially for wide bridges. However, the use of a narrower bridge that is double decked is often chosen over using a wider deck with only one level.

3.2.4 Stiffening Girder Type

The fourth characteristic of a cable-stayed bridge is the type of stiffening girder. The term stiffening girder encompasses all elements of the roadway structure including the deck, transverse floor beams, and the longitudinal beams or trusses. The deck is usually composite with the longitudinal beams or trusses so that the two function as a unit, which greatly increases the structural capacity. Stiffening girders may be described by their material and cross section. Modern cable-stayed bridges almost exclusively use concrete, steel, or a composite of concrete and steel as the material for the stiffening girder. The cross section of a stiffening girder may be placed into three general categories: solid webbed open cross sections, solid webbed closed cross sections, and open webbed cross sections (trusses).

Two solid webbed open cross sections are shown in Figure 3.13. Cross section A utilizes two longitudinal beams while cross section B utilizes four

longitudinal beams. The beams in these cross sections are typically steel plate girders which act compositely with either an orthotropic steel deck or a concrete deck.

Figure 3.13 Solid Webbed Open Cross Sections (Adapted from Podolny and Scalzi 1976, 46)

Solid webbed closed cross sections are shown in Figure 3.14 and Figure 3.15. In Figure 3.14, cross section A utilizes a single box section, cross section B utilizes two box sections, and cross section C utilizes one multi-celled box section. Figure 3.15 shows solid webbed closed cross sections which utilize trapezoidal sections. In solid webbed closed sections the boxes/trapezoids may be either steel or concrete. If steel is used for the box/trapezoid an orthotropic steel deck or a concrete deck may be used. However, if the box/trapezoid is made of concrete then a concrete deck is typically used.

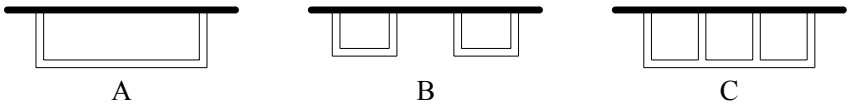

Figure 3.14 Rectangular Solid Webbed Closed Cross Sections (Adapted from Podolny and Scalzi 1976, 46)

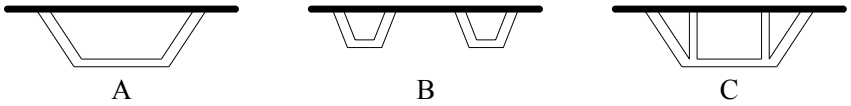

Figure 3.15 Trapezoidal Solid Webbed Closed Cross Sections (Adapted from Podolny and Scalzi 1976, 46)

Three examples of open webbed cross sections, or trusses, are shown in Figure 3.16. Cross section A consists of a deck supported on each edge by a truss, this configuration was used in early suspension bridges and found to be unstable under certain aerodynamic conditions. To stabilize cross section A, a light wind bracing may be added between the bottom chords of the two vertical trusses. This configuration, which is shown as cross section B, has better aerodynamic stability (Gimsing 1997, 305). The double decked truss, shown in cross section C, allows traffic to travel on two levels. Use of double decked trusses is an alternative to using a very wide bridge. Single decked trusses such as those shown in cross sections A and B are generally not used in cable-stayed bridges. However, the double decked truss shown in cross section C has been used in several Japanese cable-stayed bridges such as the Yokohama Bay Bridge, shown in Figure 3.17. The decks used with trusses may be either orthotropic steel or concrete (Gimsing 1997, 63).

Figure 3.16 Open Webbed Cross Sections (Trusses) (Adapted from Gimsing 1997, 305)

Figure 3.17 Yokohama Bay Bridge (Photo by Gleam, Source: commons.wikimedia.org, Licensed for Reuse under Gnu Free Documentation License Version 1.2 and Creative Commons 1.0, 2.0, and 2.5)

3.2.5 Anchoring Method

The fifth characteristic of cable-stayed bridges to be discussed is the anchoring method. The anchoring method describes how the anchor cable and stiffening girder are restrained. The importance and function of the anchor cable will be discussed in detail in Chapter 4. However, for this discussion, it is sufficient to describe the anchor cable as the cable which extends from the top of the pylon to the abutments of the side span of the bridge. Three anchoring methods will be discussed in this section, they are; earth anchored, partially earth anchored, and self anchored. Figure 3.18 shows possible bridge configurations that represent each of the three anchoring methods.

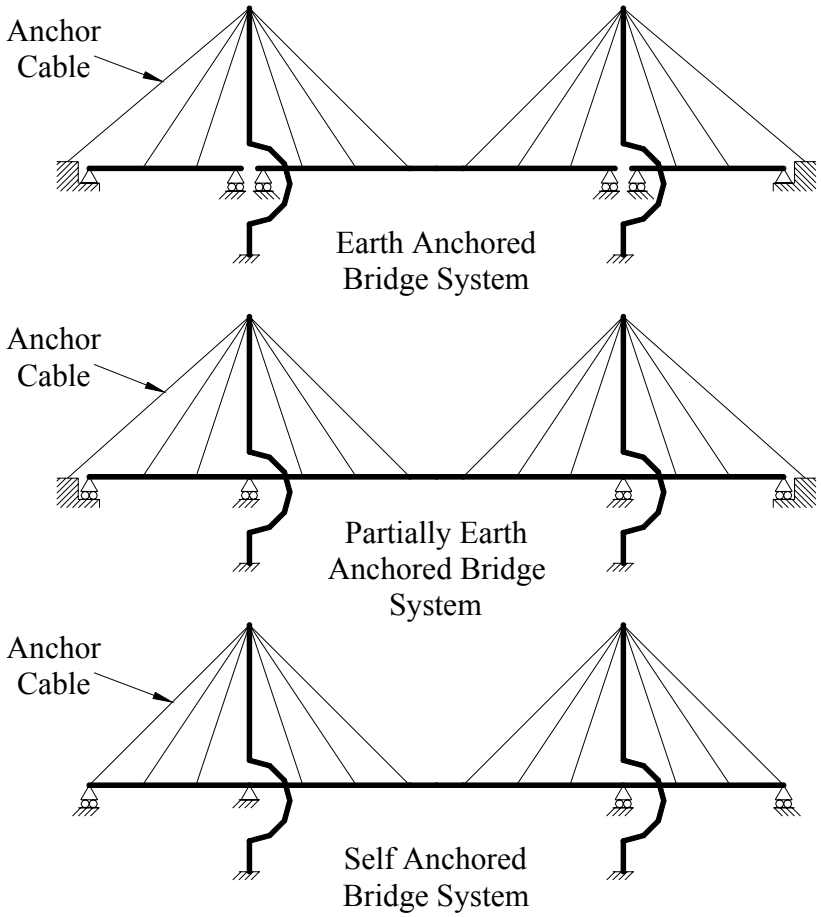

Figure 3.18 Anchoring Methods (Adapted from Gimsing 1997, 284)

In an earth anchored bridge system the anchor cables and stiffening girder are restrained both horizontally and vertically at the abutments. This requires that both the horizontal and vertical components of force in the anchor cable and stiffening girder are restrained entirely by the anchor block at the abutments. At the pylons the stiffening girder has expansion joints, so it is only restrained vertically. Because there are expansion joints in the stiffening girder at the pylons there can not be axial force in

the stiffening girder at the pylons. However, all other locations in the stiffening girder will be subjected to a tensile axial force (Gimsing 1997, 284).

In the partially earth anchored bridge system the anchor cables are restrained both vertically and horizontally at the abutments and the continuous stiffening girder is supported exclusively on movable bearings which provide only vertical support. The stiffening girder in this system will be subjected to: no axial force at the abutments, a compressive axial force in the side span and part of the main span, and a tensile axial force at the center of the main span (Gimsing 1997, 284).

In the self anchored bridge system the anchor cable is supported only in the vertical direction by the abutments. This method requires the vertical component of force in the anchor cables to be restrained by the anchor block and the horizontal component of force to be resisted by the stiffening girder. The stiffening girder is thus subject to compression throughout its length except at the middle of the main span where there will be no axial force (Gimsing 1997, 284).

Generally, suspension bridges use an earth anchored method and cable-stayed bridges use a self anchored method. However, any of the three anchoring methods could be used for either type of bridge (Gimsing 1997, 3). Because most cable-stayed bridges are earth anchored, Chapter 4 will deal with the behavior of bridges that utilize this method.

3.2.6 Longitudinal Pylon Fixity

The final characteristic of cable-stayed bridges to be discussed is longitudinal pylon fixity. The fixity of the pylon has a major impact on how a bridge behaves structurally in the longitudinal direction. The three types of pylon fixity

discussed in this section are: fixed base pylon, pinned base pylon, and pylon fixed to the stiffening girder. Sketches of each scenario are shown in Figure 3.19.

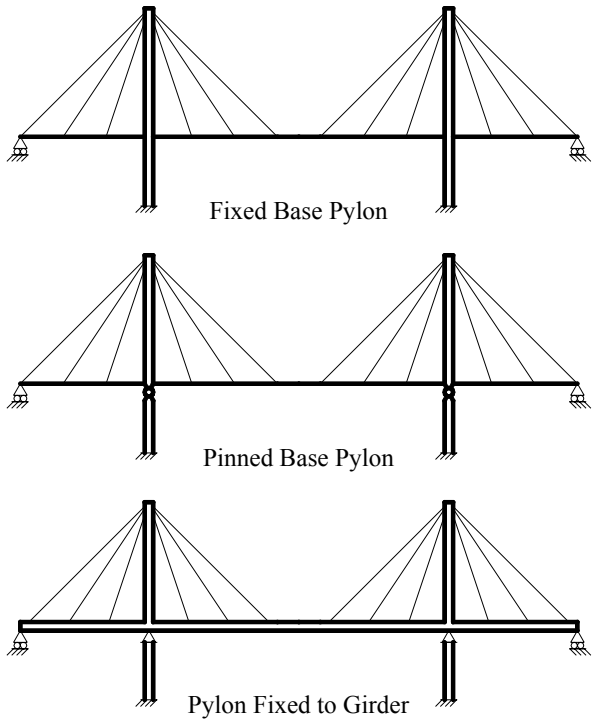

Figure 3.19 Pylon Fixity Conditions (Adapted from Gimsing 1997, 348-349)

Fixed base pylons are rigidly connected to a pier or foundation that is capable of resisting axial loads, shears, and moments. Fixed base pylons are often used for modern cable-stayed bridges and usually contribute substantially to the overall stability of the bridge system. This will be explained further in Chapter 4.

Pinned base pylons are connected to a pier or foundation through a connection that is only capable of transmitting axial loads and shears. The bearings are costly for large structures, so use of this type of pylon is typically limited to

smaller structures (Gimsing 1997, 347). The pylon in this system also has less contribution to the overall stability of a bridge system than does a fixed base pylon. Pylons with pinned bases also require temporary support during construction.

The behavior of a pylon that is fixed to the stiffening girder is largely governed by the stiffness of the stiffening girder. If the stiffening girder is rigid then the pylon will behave like it has a fixed base. If the stiffening girder is flexible, less restraint will be provided and rotational displacement will occur (Gimsing 1997, 349-350).

3.2.7 Bridge Combinations

In the prior section some common arrangements and component types of cable-stayed bridges were introduced. Cable-stayed bridges must combine these arrangements and components into an effective structure in order to be successful. Some combinations may be immediately ruled out because they are simply not compatible. An example would be the use of an A-frame pylon to support a single central vertical plane of stays that is in the harp layout. This configuration is not possible because most of the cables would not intersect the pylon, so they would have no place to anchor. Other systems may be ruled out because they are structurally ineffective. An example of this is the use of a single central vertical plane of stays to support a stiffening girder with a solid webbed open cross section. This combination has generally not been used because the stiffening girder lacks the torsional stiffness that would be required to support unbalanced loads. Once unsuitable combinations have been ruled out, many combinations still remain as feasible options. This fact may be verified by the many different cable-stayed bridge configurations that have been used in the past and will be used in the future.

3.3 History of Cable-Stayed Structures

The basic concept of a cable-stayed structure is the use of tension members which are anchored to a pylon to support a beam. Although the first modern cable-stayed bridge, the Stromsund Bridge in Sweden, was opened to traffic only fifty years ago (Gimsing 1997, 25), the concept is quite old. Figure 3.20 shows an Egyptian sailing ship that uses the same concept as a cable-stayed bridge (Troitsky 1988, 2). On this ship several ropes are anchored to the mast of the ship, these ropes then run diagonally and attach at several points along the boom of the ship. The mast functions as the pylon, the ropes function as the stays, and the boom of the ship is the beam that requires support.

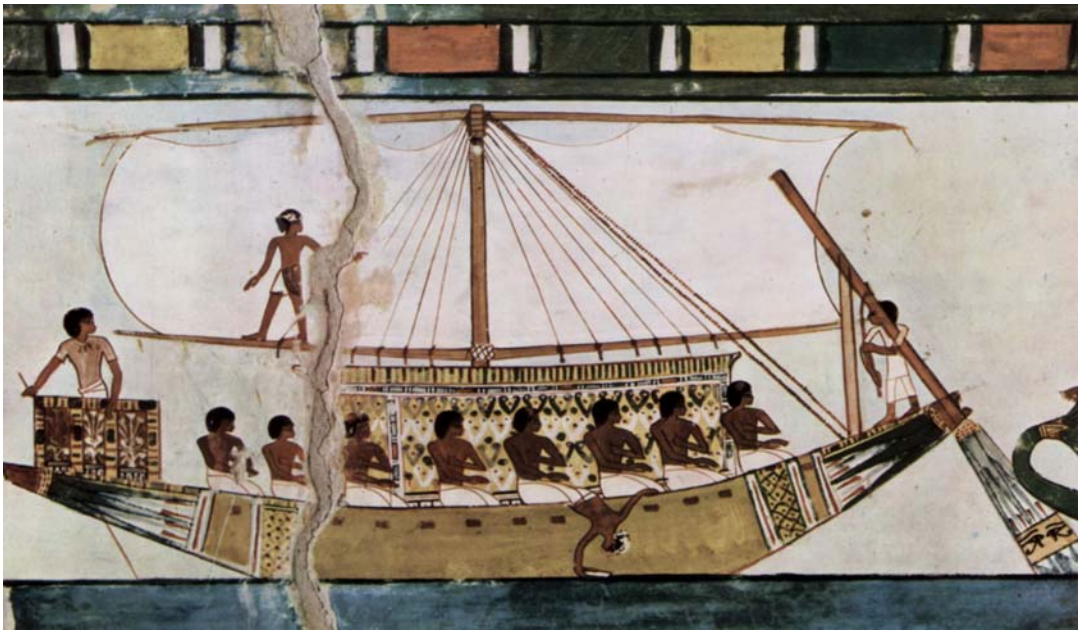

Figure 3.20 Ancient Egyptian Sailing Ship (Copyright by The Yorck Project, Source: commons.wikimedia.org, Licensed for Reuse under Gnu Free Documentation License)

An early example of the use of stays to support a bridge is shown in Figure 3.21. This bridge, which was proposed by Faustus Verantius in 1617, used eye bars to support a wood deck system (Troitsky 1988, 4). The system proposed was not purely a stayed system. It was instead a dual system which employed both a stay system and a suspension system. There are three sets of stays on each side of the bridge. In addition to the stays, a suspension type set of eye bars was used to support a vertical hanger at the center panel point of the bridge. This early bridge may have been the source of Dischinger's bridge system, which is discussed later in this section.

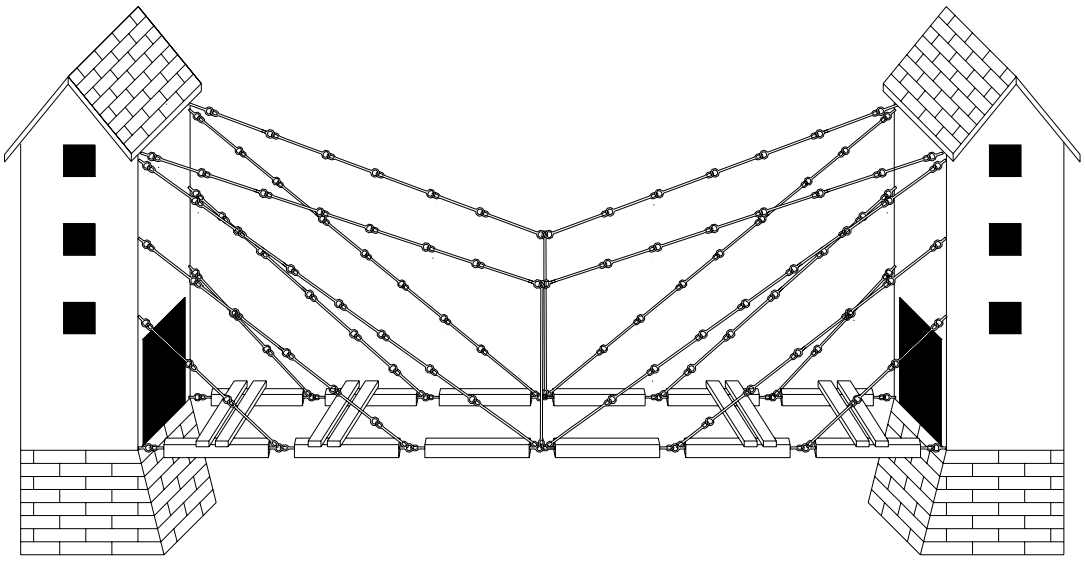

Figure 3.21 Bridge Proposed by Faustus Verantius, Italy, 1617 (Redrawn with Modifications from Troitsky 1988, 4)

Another early example of a bridge employing stays was the timber bridge in Figure 3.22. This bridge was designed and built by Immanuel Loscher in Fribourg Germany in 1784 (Podolny and Scalzi 1976, 5).

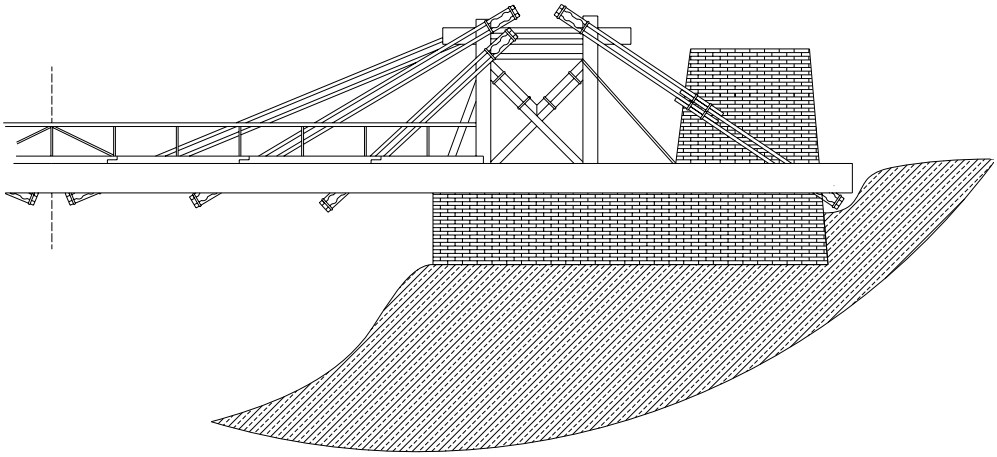

Figure 3.22 Timber Stayed Bridge, Germany, 1784 (Redrawn with Modifications from Troitsky 1988, 5)

The two bridges above represent early attempts to use stays to support bridges. The stays used were not cables, but eye bars and wood. The use of cables as stays was successfully attempted in 1817 in the King's Meadows Bridge, which is shown in Figure 3.23. This footbridge, which spanned about 110 feet, was designed by two British engineers. The bridge utilized cast iron towers and wire cables for the stays (Troitsky 1988, 5).

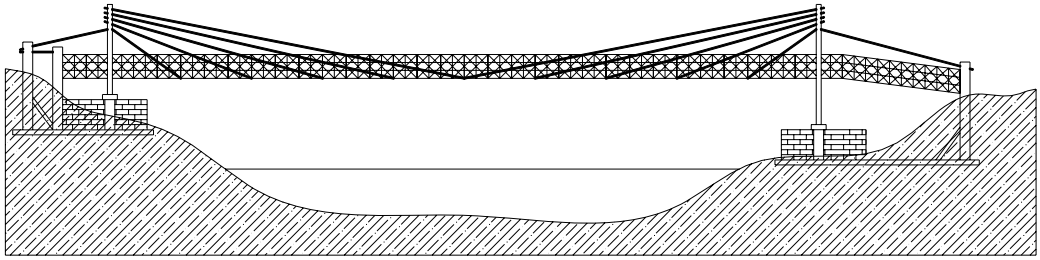

Figure 3.23 King's Meadow Bridge, England, 1817 (Redrawn with Modifications from Troitsky 1988, 5)

The concept of using stays to support bridges seemed to be gaining popularity in the early eighteen-hundreds. In 1821 Poyet, a French Architect, proposed a system in which wrought iron bars were used to support beams from towers. This system, which is shown in Figure 3.24, was expanded upon by Navier, a French Engineer, in 1823. Navier considered the use of chains instead of bars in Poyet's system. Navier determined that the cost of a structure for a given situation was about the same regardless of whether bars or chains were utilized (Troitsky 1988, 6).

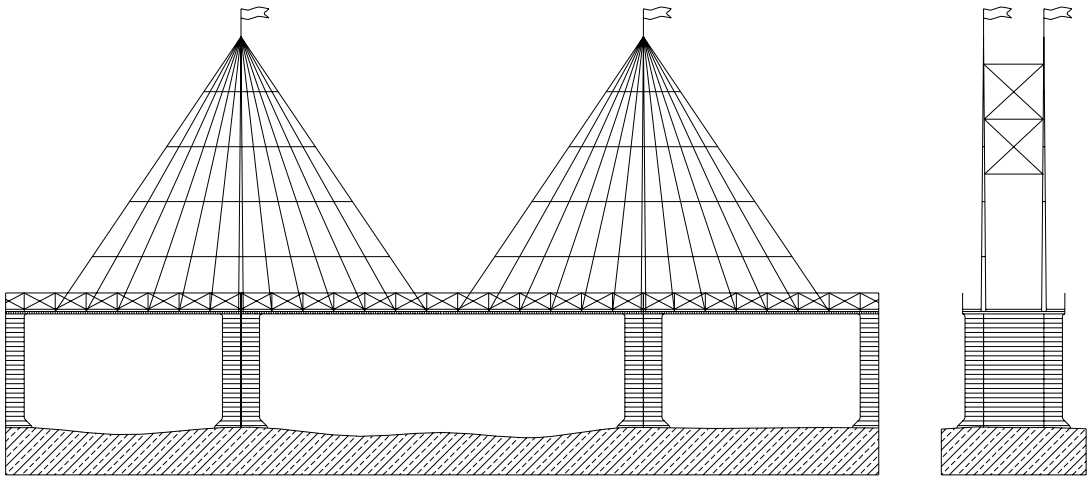

Figure 3.24 Poyet's Proposed System, France, 1821 (Redrawn with Modifications from Troitsky 1988, 6)

The gain in popularity of stayed bridges, however, was halted by the failure of several bridges that utilized the concept. The most notable failure was that of the bridge erected in 1824 at Nienburg Germany to cross the Saale River. This bridge, which is shown in Figure 3.25, utilized chain stays to support its 256 ft span. Failure of the chain stays caused it to collapse while supporting a crowd of people

who had gathered to watch events being held on the river (Troitsky 1988, 9). The failure is attributed to gross overloading of the structure (Walther et al 1999, 8). Following this and other failures, stayed bridges were no longer regarded favorably and suspension bridges became the preferred structure of the time (Podolny and Scalzi 1976, 6)

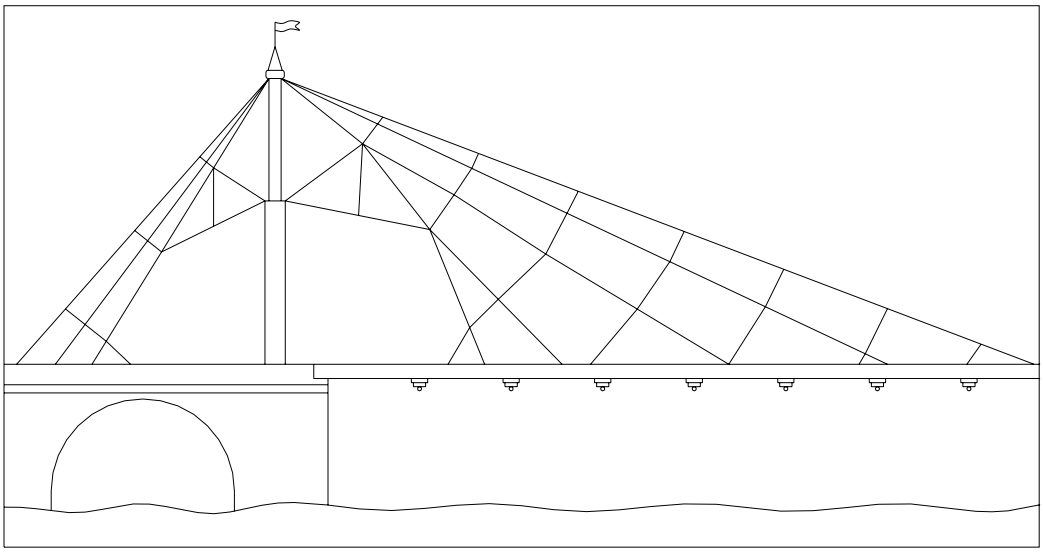

Figure 3.25 Bridge Over the Saale River, Germany, 1824 (Redrawn with Modifications from Troitsky 1988, 8)

Although the concept of using stays to support a bridge fell out of favor, it was not entirely forgotten. American bridge designer John Roebling used a stayed system in conjunction with the suspension system. This may be seen on the Niagara River Bridge (1855), Ohio River Bridge (1867), and Brooklyn Bridge (1883) which is shown in Figure 3.26. The original reason for using the stay system in addition to the suspension system was to stiffen the bridge and guard against wind induced vibration.

However, Roebling recognized and utilized the capacity that the stay system added to the bridge (Troitsky 1988, 11-13). Roebling explains the strength of the stay system on the Brooklyn Bridge in the following quote:

> The floor, in connection with the stays, will support itself without the assistance of the cable, the supporting power of the stays alone will be ample to hold up the floor. If the cables were removed, the bridge would sink in the center, but would not fall. (Troitsky 1988, 13)

Figure 3.26 Brooklyn Bridge, New York, 1883 (Photo by Simone Roda, Source: commons.wikimedia.org, Licensed for Reuse under Creative Commons 2.0)

Roebling was not the only designer to consider combining the suspension system with the cable-stayed system. A mix of the two systems was also proposed by the German engineer Dischinger in 1938. An example of Dischinger's system is presented in Figure 3.27.

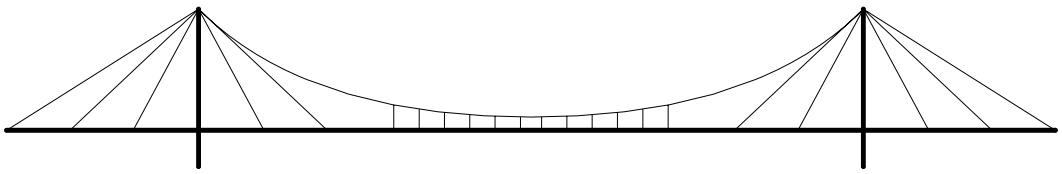

Figure 3.27 Dischinger Bridge System, Germany, 1938 (Redrawn with Modifications from Gimsing 1997, 24)

This system is notably different than Roebling's. Roebling used the suspension and cable-stayed systems in a way that allowed them to both participate in carrying the dead and live load on the bridge. Dischinger's system utilized the cable-stayed system to support the portions of the bridge close to the towers and used the suspension system to support the loads at the center of the main span, away from the towers. Dischinger's system was never actually used in the construction of a bridge, although it was proposed for several bridges in Germany during post World War II reconstruction. Despite the fact that Dischinger's system was never used, ideas that he introduced while developing the system have influenced the evolution of cable-stayed bridges (Gimsing 1997, 24). It is interesting to note that Roebling's system was redundant, in that if the suspension cables were removed the Brooklyn Bridge would still stand. Dischinger's system was not redundant, so if the suspension cables were removed the center of the main span would be left without support.

The Stromsund Bridge in Sweden, which was built in 1955, is considered the first modern cable-stayed bridge. The bridge was constructed by Demag, a German engineering company, in collaboration with Dischinger (Podolny and Scalzi 1976, 2). As can be seen in Figure 3.28 the bridge has two vertical planes of stays, with each set of stays radiating from the top of portal frame pylons which have pinned

bases (Gimsing 1997, 347). The main span of the bridge is 599 ft and the side spans are 245 ft. The bridge utilizes a solid webbed open cross section which consists of two plate girders and an orthotropic steel deck (Gimsing 1997, 25).

Figure 3.28 Swedish Postcard Celebrating Opening of Stromsund Bridge

Following the success of the Stromsund Bridge, many cable-stayed bridges were built in Germany to replace bridges that had been destroyed during World War II. The first of these bridges, the Theodor Heuss Bridge, was opened to traffic in 1957 in the city of Dusseldorf. The bridge crossed the Rhine River using a 853 ft main span and two 354 ft side spans. As can be seen in Figure 3.29, the Theodor Heuss Bridge uses two vertical planes of stays in the harp layout. The stays are supported by twin towers and support a stiffening girder with a solid webbed closed cross section that utilizes two steel box girders and an orthotropic steel deck (Gimsing 1997, 27-28).

Figure 3.29 Theodor Heuss Bridge (Photo by Nicolas Janberg, Source: en.structurae.de, Reuse Permitted)

The Stromsund Bridge and the Theodor Heuss Bridge both used only a few stays to support the stiffening girder. The stays essentially act as intermediate piers, but allow the waterways that the bridges cross to remain unobstructed for marine traffic. However, the large stay spacing utilized on these bridges does not fully exploit the potential advantages of cable-stays. This is because large stay spacing still requires a longitudinally stiff stiffening girder (Walther et al 1999, 11-13). However, the large stay spacing was necessary because of the state of structural analysis at the time these bridges were designed. The Stromsund Bridge was the first cable-stayed bridge to have systematic calculations of cable forces made for each stage of construction. If the Stromsund Bridge is modeled as a plane system, the bridge is statically indeterminate to the eighth degree. However, by use of symmetry and antisymmetry, the number of redundants can be lowered to four. The Theodor Heuss Bridge, when modeled as a plane system, is statically indeterminate to the tenth degree. By use of symmetry, antisymmetry, and splitting the bridge into part systems the number of redundants may be reduced to four as well. These systems were thus

able to be analyzed using the technology of the time, which consisted of slide rules and mechanical calculators (Gimsing 1997, 25-28).

Early cable-stayed bridges utilized few stays which were not closely spaced. This approach allowed the designers to accurately analyze the structures using the technology of the time, but did not utilize the potential efficiency of using closely spaced stays. Cable-stayed bridges with many stays, which will be referred to as multi-cable or multi-stay bridges in the remainder of this document, were introduced during the design competition of 1967 for the bridge across the Great Belt in Denmark. Two German engineering firms, Demag and Dywidag, submitted designs which utilized a large number of stays with close spacing. The designs were thus able to utilize slender stiffening girders which would not have been possible if fewer stays were used. The designs were never built, but they are thought to have influenced the design of the Friedrich Ebert Bridge, which was designed in1967 and constructed shortly after the competition (Gimsing 1997, 40-41).

H. Homberg designed the Friedrich Ebert Bridge, which is shown in Figure 3.30, in Bonn, Germany in 1967. The bridge was the first cable-stayed bridge constructed to utilize a large number of closely spaced stays to support the stiffening girder (Walther et al 1999, 13). The multi-cable design of the Friedrich Ebert Bridge makes it indeterminate to the 82nd degree when it is considered as a plane system. Digital computers were used to analyze the system, which would have been impossible to analyze by hand as was done for the Stromsund Bridge only twelve years earlier (Gimsing 1997, 42).

Figure 3.30 Friedrich Ebert Bridge (Photo by Falk Beckmann, Source: en.structurae.de, Reuse Permitted)

The use of multi-cable bridges is advantageous because it allows the longitudinal stiffness of the stiffening girder to be reduced, facilitates construction by limiting the length of unsupported sections of the stiffening girder during erection, and makes concrete decks and stiffening girders a viable option. These advantages allow bridges with closely spaced stays to be more economical to build than bridges with larger stay spacing. The multi-cable bridge is currently universally accepted as the optimal solution for cable-stayed bridges (Walther et al 1999, 13).

This chapter began by introducing the basic terminology and characteristics of cable-stayed bridges and finished by discussing the history and evolution of these structures. This information will aid in the understanding of the next chapter. Chapter 4 will begin by discussing the structural behavior of several common cable-stayed bridge configurations by utilizing simple models as boundary conditions. The advantages and disadvantages of some of the characteristics discussed in Chapter 3 will then be explored. Chapter 4 will end with a brief discussion of stay-cable vibration issues.

Chapter 4

CABLE-STAYED BRIDGE STRUCTURAL BEHAVIOR

4.1 Introduction

The longitudinal structural behavior of cable-stayed bridge systems is quite complex, especially for multi-cable bridges which are highly redundant. Therefore, analysis by hand is generally not practical except for simple bridges which use very few stays. The use of digital computers is considered necessary to analyze cable-stayed bridges in most cases. However, some insight into the behavior of cable-stayed bridges may be gained by examining simplified models of the actual system.

In the following section several simplified models of self anchored cable-stayed bridges will be examined. Each model examines the potential stabilizing effect that the cables, pylon, or girder may have on a bridge system. These simplified models represent boundary conditions of actual cable-stayed bridge systems and allow insight into the behavior of those systems. These simplified models will be the focus of Sections 4.2 through 4.4. Section 4.5 will discuss the behavior of actual bridge systems and Section 4.6 will revisit the six characteristics used to describe cable-stayed bridges and discus the advantages and disadvantages of different bridge components and configurations. Section 4.7 will introduce the problem of stay-cable vibration.

4.2 Cable Systems

The first structural idealization that will be examined is the "cable system" of a cable-stayed bridge. In the cable system the global bending stiffness of the pylon and the stiffening girder will be neglected. The members in this system will be considered effective for the transmission of axial loads only. More specifically, the pylon and stiffening girder will be considered effective for transmitting compression or tension and the cables will be considered effective for transmitting tension only. The global bending stiffness of the pylon and stiffening girder may be removed from the system by modeling the system with hinges at all locations where members intersect. The bridge system and corresponding cable systems of a typical three span self anchored cable-stayed bridge are shown in Figure 4.1.

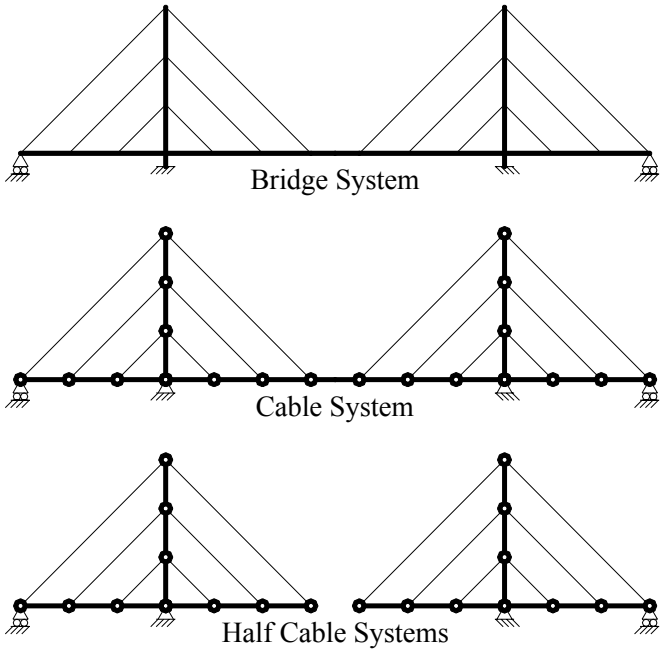

Figure 4.1 Bridge System with Corresponding Cable System Idealization (Adapted from Gimsing 1997, 168)

The bridge system of Figure 4.1 has fixed base pylons. However, because the bending stiffness of the pylon is removed in the cable system, the cable system model will be the same whether the base of the pylon is fixed or pinned. Also, because the stiffening girder of a self anchored system has no axial force at the center of the main span and the stiffening girder in the cable system may only transmit axial loads, the portion of the stiffening girder at the center of the main span may be removed. This will not change the behavior of the cable system, but will allow examination of the half cable system shown in Figure 4.1.

Now that the concept of cable systems has been introduced, the behavior and stability of these systems may be examined. Loads acting on the cable system will be idealized as concentrated loads applied at the nodes in a downward direction, similar to the modeling of gravity loads on a truss. The stability of a cable system may be defined as one of three categories: stable of the first order, stable of the second order, or unstable. A system that is stable of the first order will be in equilibrium without assumed nodal displacement (Gimsing 1997, 167). An example of a cable system which is stable of the first order is shown in Figure 4.2.

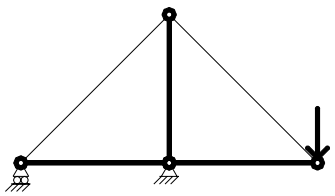

Figure 4.2 Cable System Stable of the First Order (Adapted from Gimsing 1997, 168)

The cable system of Figure 4.2 may be analyzed in a manner similar to a truss, and seen to be stable. It should be noted that the members behave consistently with the definition of a cable system: the cables are in tension and the pylon and stiffening girder are in compression.

A system that is stable of the second order will be in equilibrium only after nodal displacement has occurred due to the external loads (Gimsing 1997, 167). An example of a cable system which is stable of the second order is shown in Figure 4.3. In this figure no stiffening girder is present, and it can be seen that the profile of the cable will change as the magnitude of the two external loads are varied with respect to one another. However, once nodal displacements have occurred the system will achieve equilibrium.

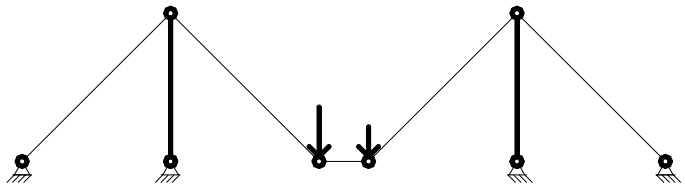

Figure 4.3 Cable System Stable of the Second Order (Adapted from Gimsing 1997, 168)

In an unstable cable system equilibrium can not be achieved by the cable system alone (Gimsing 1997, 167). An example of a cable system which is unstable is shown in Figure 4.4. The system of Figure 4.4 is unstable due to inadequate support conditions. The result of this instability is rotation of the entire structure.

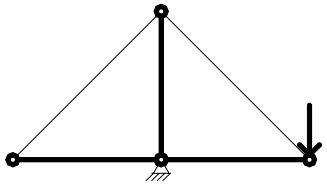

Figure 4.4 Unstable Cable System (Adapted from Gimsing 1997, 168)

The stability of the radiating, harp, and fan cable systems of typical self anchored cable-stayed bridges will be examined in the following sections.

4.2.1 Radiating Layout

The first bridge configuration to be examined will be the self anchored radiating layout. An example of a radiating bridge with its corresponding cable system is shown in Figure 4.5. The span between the pylon and the roller support will be referred to as the backspan and the other span will be referred to as the main span.

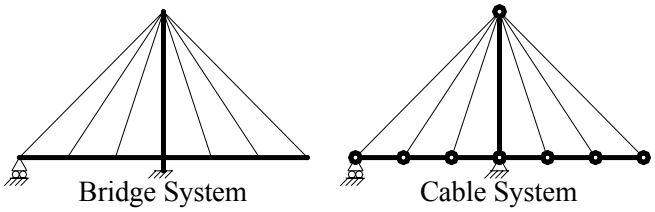

Figure 4.5 Radiating Bridge and Cable System

Temporarily ignoring dead loads on the radiating cable system and applying live load at E, as shown in Figure 4.6, the system will be stable of the first order. This may be proved by examining the load path in the system using the method of joints. Applying a load at E it may be seen that the load is resisted by compression

in the ED section of the stiffening girder and tension in cable EH. The compression in the ED section of the stiffening girder is transferred to the support at D. Cable EH transfers the tension force to the top of the pylon where it is resisted by compression in the pylon and tension in cable AH. The compression in the pylon is transferred into the support at D. The tension in cable AH is restrained at A vertically by the external support, and horizontally by compression in the AD section of the stiffening girder. The stiffening girder then transfers the compression to the support at D. By examining the load path it may be noted that cable AH participates for loads placed at E, F, and G. Cable AH is thus called the anchor cable of the system, and all other cables are referred to as stays. The anchor cable may be seen to be a critical stabilizing element in the system. If the anchor cable does not remain in tension the cable system will become unstable (Gimsing 1997, 169).

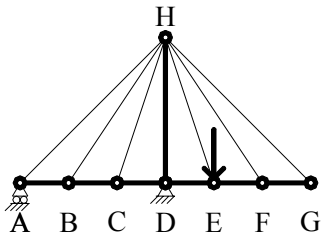

Figure 4.6 Radiating Cable System with Load at E

If the stiffening girder is assumed to have a uniform cross section along its length, then uniform dead load such as that shown in Figure 4.7 will be applied to the system. For stability to be present, the resultant horizontal force applied to the pylon by stays HE, HF, and HG must be greater than that applied by stays HB and HC. This

will assure that anchor cable HA remains in tension. The stability of this system with only dead loads present will thus be determined by the relative lengths of the spans. If the main span is longer than the side span, the anchor cable will remain in tension, and the system will be stable of the first order. However, as the side span becomes longer than the main span, the anchor cable will not remain in tension, and the system will be unstable. This introduces a restraint on the stability of the self anchored radiating cable system. This restraint will be strengthened when considering a system with live load on the side span.

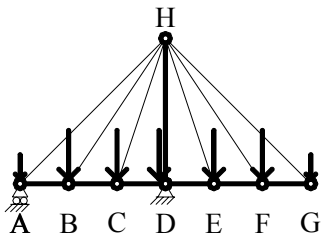

Figure 4.7 Radiating Cable System with Uniform Dead Load

If live load is applied to the side span in addition to uniform dead load from the stiffening girder, as shown in Figure 4.8, the stability of the system will, again, be determined by the relative lengths of the spans. In order to achieve stability of the system, the resultant horizontal force applied to the pylon by stays HE, HF, and HG, which carry dead load only, must be greater than the resultant horizontal force applied by stay HB and HC, which now carry dead load and live load. For this to happen, the main span must generally be longer than the side span. The required ratio of side span

to main span length will be determined by the live load to dead load ratio on the bridge (Gimsing 1997, 196).

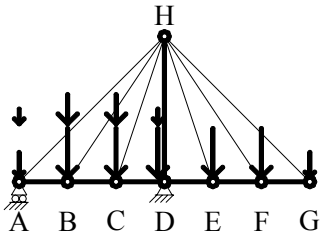

Figure 4.8 Radiating Cable System with Uniform Dead Load and Side Span Live Load

From the above discussion it may be stated that self anchored radiating cable systems will be stable of the first order for all gravity loadings provided that the side span to main span length ratio is correct for the given ratio of live load to dead load. In addition to stability, the allowable stress range in the anchor cable will impose restrictions on the span ratio (Gimsing 1997, 196). Expressions for the required ratio of side span to main span length for given ratios of live load to dead load for the self anchored radiating cable system have been derived in "Cable Supported Bridges: Concept and Design" by Niels J. Gimsing, but will not be discussed here.

4.2.2 Harp Layout with Single Anchor Pier

The second bridge configuration to be examined will be the self anchored harp layout with a single anchor pier. An example of a typical harp bridge with a single anchor pier is shown with its corresponding cable system in Figure 4.9.

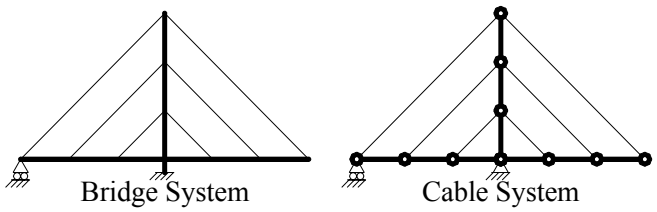

Figure 4.9 Harp Bridge and Cable Systems with Single Anchor Pier

The harp cable system of Figure 4.10 may be seen to be stable for gravity loads applied at A and D, which are supports, and at G which activates the anchor cable. However, loads placed at B, C, E, or F will lead to instability of the cable system.

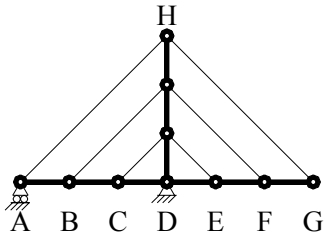

Figure 4.10 Harp Cable System with Single Anchor Pier

By examining the cable system as a truss, one can immediately recognize that the configuration of the harp cable system uses quadrilaterals. As shown in Figure 4.11, quadrilaterals, unlike triangles, are unstable if pinned joints are used to connect the members. This leads to the conclusion that the self anchored harp cable system with a single anchor pier is unstable under gravity loads.

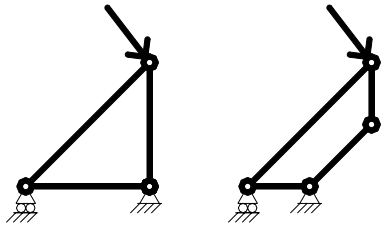

Figure 4.11 Trusses Using Triangles and Quadrilaterals

4.2.3 Harp Layout with Multiple Anchor Piers

The third bridge configuration to be examined will be a self anchored harp layout with multiple anchor piers. An example of a harp cable system with multiple anchor piers is shown in Figure 4.12.

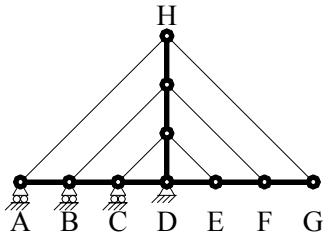

Figure 4.12 Harp Cable System with Multiple Anchor Piers

In this system, any cable that is located at an anchor pier has the ability to act as an anchor cable. The cable system will be stable when any joint located at an anchor pier is loaded because the load will be resisted by the anchor pier. The system will also be stable when a joint that has a stay that joins an anchor cable is loaded. Self anchored harp cable systems with multiple anchor piers are stable under gravity loads if anchor piers are provided for all backspan cables.

4.2.4 Fan Layout

The final bridge configuration to be considered will be the self anchored fan layout. The cable system of this bridge configuration may be seen to act in a similar manner to the harp cable system. This is because the fan cable system still uses quadrilaterals in its geometry. The discussion of self anchored harp cable systems made above may therefore be applied to self anchored fan systems as well.

4.2.5 Cable System Considerations

Cable systems and their stability have been discussed in this section. From this discussion it can be seen that self anchored cable systems are stable of the first order or unstable. Cable systems that are stable of the second order are generally only seen when earth anchoring is used (Gimsing 1997, 173). It must be emphasized that instability of a cable system does not imply instability of the bridge system. It only implies that the cable system alone is unable to achieve equilibrium. In an actual structure the stiffening girder and pylon may have significant bending stiffness which was ignored when examining cable systems. The stiffness of the stiffening girder and pylon are often utilized to stabilize actual bridge systems, and neglecting the stiffness they provide is unrealistic. However, by understanding the behavior of cable systems it will become apparent when the bending stiffness of the pylon and stiffening girder is required to stabilize a bridge system.

Although instability of a cable system does not imply instability of a bridge system, a stable cable system will be stiffer and stronger than an unstable cable system which utilizes similar members and materials. In a parametric study by Gimsing, use of a stable cable system instead of an unstable cable system decreased maximum deflection by more than half and decreased stress as well (Gimsing 1997,

172). Therefore, it may be advantageous to utilize cable systems which are stable of the first order. It should also be noted that minor changes to how a bridge system is supported may change the stability of the cable system, so it is important to examine restrains conditions carefully (Gimsing 1997, 170).

4.3 Pylon Systems

The second idealization that will be examined is the case of having a rigid pylon and a flexible stiffening girder. This system will be referred to as the "pylon system" and may be modeled by using hinges in the stiffening girder where it intersects all other members. Similar to the cable system the stays may only carry tension and loads acting on the system will be idealized as concentrated loads applied at the nodes in a downward direction. In this section, first order analysis will be used and loads will be assumed to be transferred through the centroid of the members so that moments due to eccentricities will not be present. The simplified analysis used in this section does not capture the true structural behavior of cable-stayed bridges because they are flexible structures and loads are often eccentric, however, insight into the behavior of cable-stayed bridges may still be gained.

The bridge system in Figure 4.13 has fixed base pylons. Because the bending stiffness of the pylon is considered in the pylon system, the pylon system model will be different depending on whether the base of the pylon is fixed or pinned. Similar to the cable system, the portion of the stiffening girder at the center of the main span may be removed without affecting the behavior of the pylon system. This allows examination of the half pylon system shown in Figure 4.13.

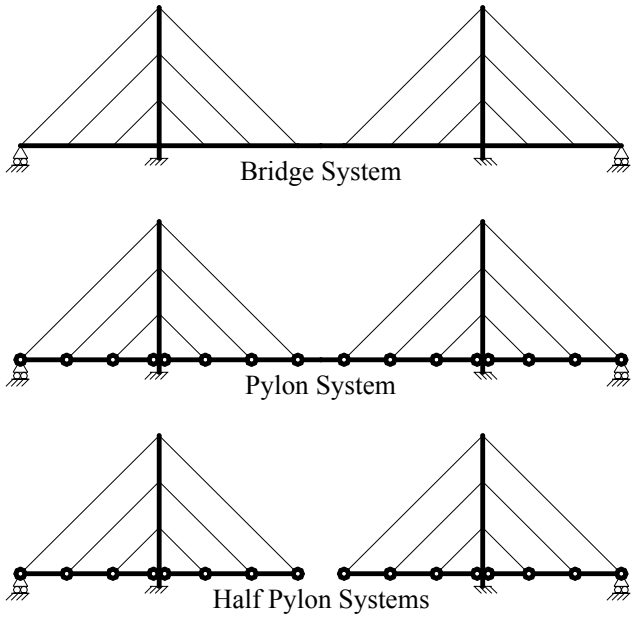

Figure 4.13 Bridge System with Corresponding Pylon System Idealization

The cable systems examined in the previous section were able to be analyzed by the method of joints because the systems behaved as determinate trusses with the stays functioning as tension only members. Parts of the pylon system may be analyzed using the method of joints; however, joints that are positioned on the pylon may not. This is because the pylon in the pylon system is permitted to transmit shear and moment, so joints on the pylon will contain too many unknowns. It should also be noted that the pylon system is statically indeterminate if the pylon is fixed at the base. It therefore becomes necessary to analyze the structure using matrix analysis techniques. Matrix analysis uses equilibrium equations, constitutive relationships, and compatibility to solve for reactions and member forces in an indeterminate structure. The qualitative load, shear, and moment diagrams in the following section were developed by hand and verified using a matrix analysis computer program.

4.3.1 Radiating Layout Fixed Base Pylon

The first bridge configuration to be examined will be the self anchored radiating bridge with the pylon fixed at the base as shown in Figure 4.14.

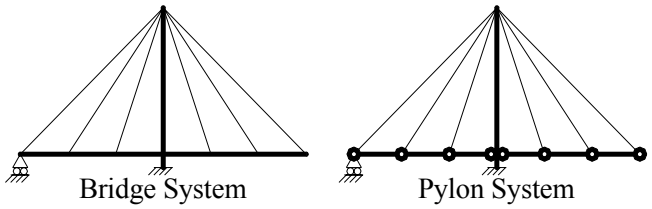

Figure 4.14 Radiating Bridge and Pylon System with Fixed Base Pylon

If the pylon of the bridge in Figure 4.14 is considered to be infinitely rigid, then the top of the pylon will not deflect when loads are placed on the spans. Because the top of the pylon does not deflect the anchor cable does not undergo elongation and will not carry load. The bridge may be considered to consist of the systems in Figure 4.15.

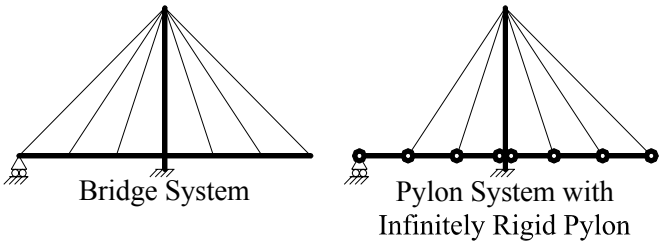

Figure 4.15 Radiating Bridge and Infinitely Rigid Fixed Base Pylon Systems

Qualitative load, shear, and moment diagrams for a radiating pylon system with an infinitely rigid pylon are shown in Figure 4.16.

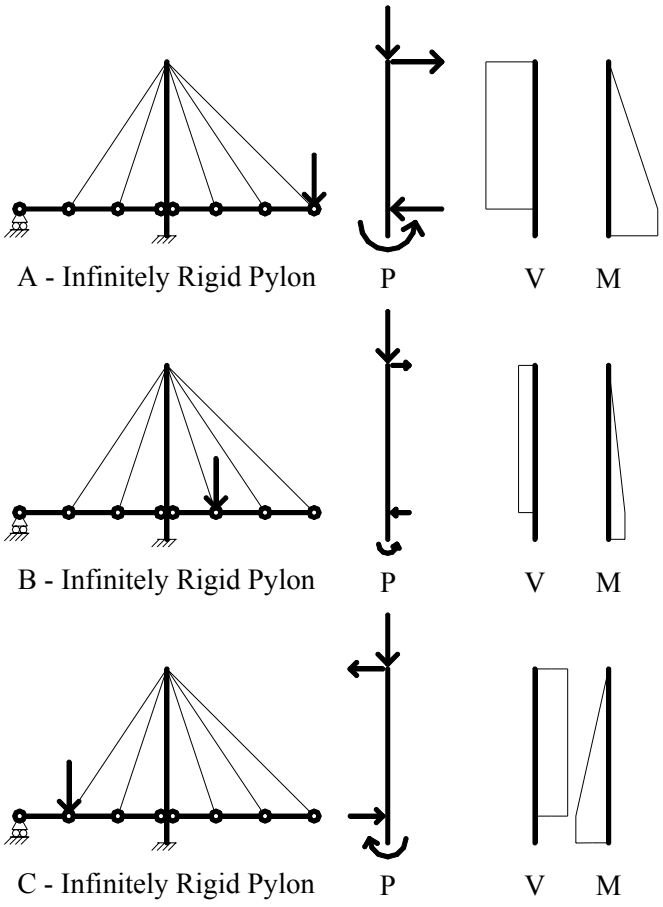

Figure 4.16 Load, Shear, and Moment Diagrams for Radiating Pylon System with Infinitely Rigid Fixed Base Pylon

From Figure 4.16 it may be seen that the axial load in the pylon is equal to the applied load and that the moment diagram in the pylon is the same shape regardless of where the load is applied. The magnitude of the moment diagram is dictated by how far the load is placed from the pylon and the sign of the moment diagram depends on which side of the pylon the load is applied. It may be noted that loads of equal magnitude and direction placed symmetrically about the pylon will result in zero bending in the

pylon (except if one of the loads is placed at the support). The geometry of a bridge may be intentionally chosen so that bending in the pylon is avoided under dead load.

If the pylon is less than infinitely rigid, then it will deform when load is placed on the spans. If load is placed on the main span, horizontal deflection at the top of the pylon will cause elongation of the anchor cable, thus allowing the anchor cable to carry axial load. This concept is explained by Figure 4.17. Sketch A shows a radiating pylon system with a deformable pylon and a load on the main span. The joints along the girder may be examined using the method of joints. This allows several zero force members to be identified. Sketch B shows the pylon system with the zero force members removed. Using the method of joints at the joint where the load is applied allows the tension in the stay and the compression in the main span of the stiffening girder to be determined. The forces transmitted by these members are shown in sketch C. The system in sketch C is indeterminate, so deformations must be examined for the reactions and member forces to be solved. Sketch D shows the deformation of the anchor cable. At the roller support the cable may only displace horizontally, this horizontal displacement will be equal to the sum of the axial shortening in the girder and the horizontal deflection of the pylon at the level of the girder. At the top of the pylon the anchor cable will displace horizontally by the same amount that the pylon deflects due to bending and it will deflect vertically by the same amount that the pylon deflects due to axial shortening. The axial shortening of the girder and the axial shortening of the pylon are likely to be small compared to the horizontal deflections of the pylon. If this is true, then the elongation of the anchor cable will be controlled by the relative horizontal deflection of the pylon between the level of the stiffening girder and the top of the pylon. From this it may be seen that

the distribution of horizontal load being carried by the anchor cable and the pylon depends upon the relative stiffness of the two members to horizontal deflection at the pylon top.

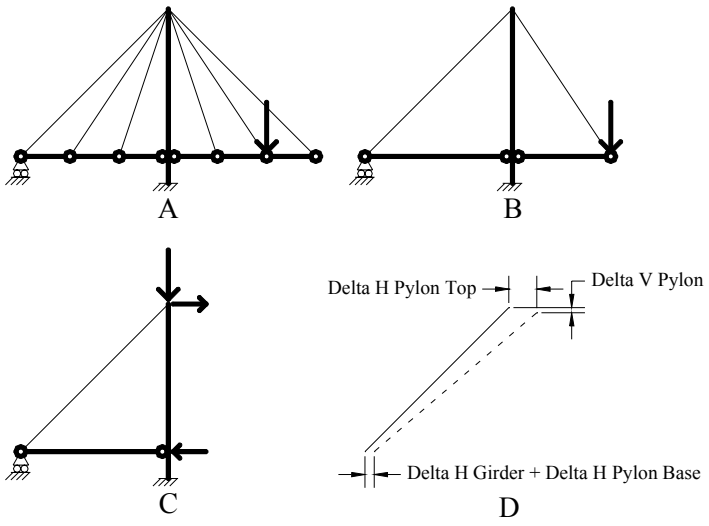

Figure 4.17 Analysis of Radiating Pylon System with Fixed Base Deformable Pylon

Figure 4.18 shows the qualitative load, shear, and moment diagrams for the pylon in the pylon system when the relative stiffness of the pylon and anchor cable are varied. Sketch A shows a system in which the pylon is much stiffer than the anchor cable, Sketch B shows a system where the stiffnesses are equal, and sketch C shows a system where the anchor cable is much stiffer than the pylon. From Figure 4.18 it may be seen that as the flexural stiffness of the pylon is decreased compared to the anchor cable axial stiffness, there will be lower bending moments and higher axial load in the pylon.

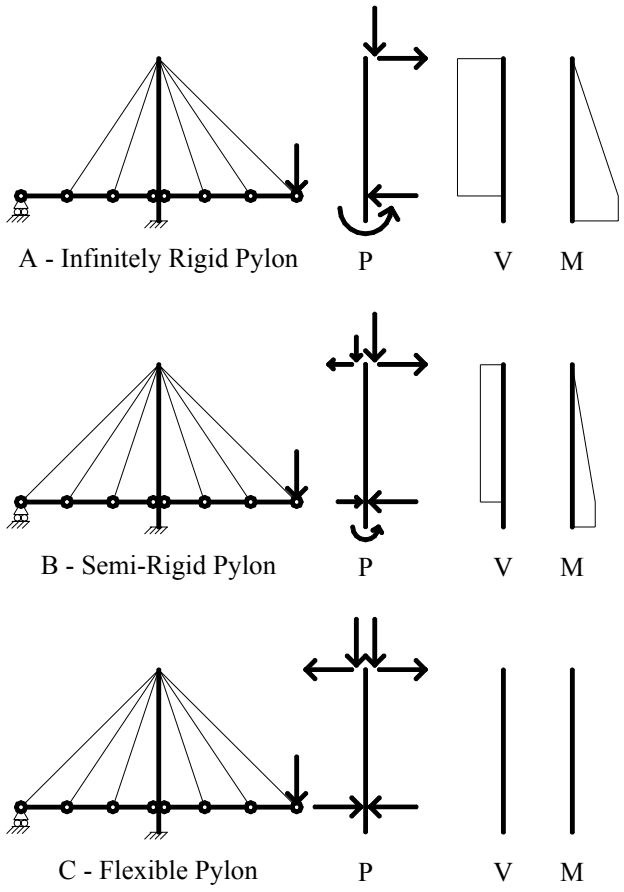

Figure 4.18 Load, Shear, and Moment Diagrams for Radiating Pylon Systems with Fixed Base Pylons of Varied Stiffness

If the side span of a radiating pylon system with a deformable pylon is loaded the anchor cable may not be in tension. If this happens, the pylon will no longer be assisted by the anchor cable. The pylon will instead be subjected to loads similar to those shown in Figure 4.16 C. Therefore it may be advantageous to proportion the spans so that the anchor cable remains in tension.

4.3.2 Radiating Layout Pinned Base Pylon

If a pinned base pylon is used in a bridge with a radiating stay layout the pylon system will be that shown in Figure 4.19. It should be noted that this pylon system is the same as the radiating cable system and may be analyzed as a truss. It will be stable for loads on the main span but will become unstable for loads on the side span if the spans are not proportioned properly.

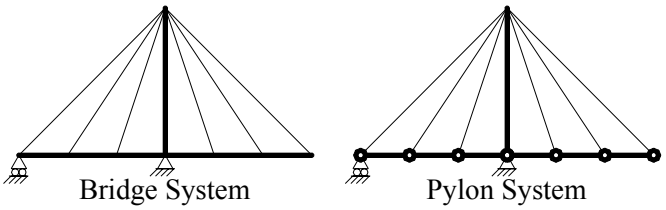

Figure 4.19 Radiating Bridge and Pylon System with Pinned Base Pylon

Figure 4.20 shows the forces applied to the pylon for loads at several locations. The pylon only carries axial load and is not subjected to shear or bending. The magnitude of the axial load in the pylon becomes greater as a load is moved further from the pylon.

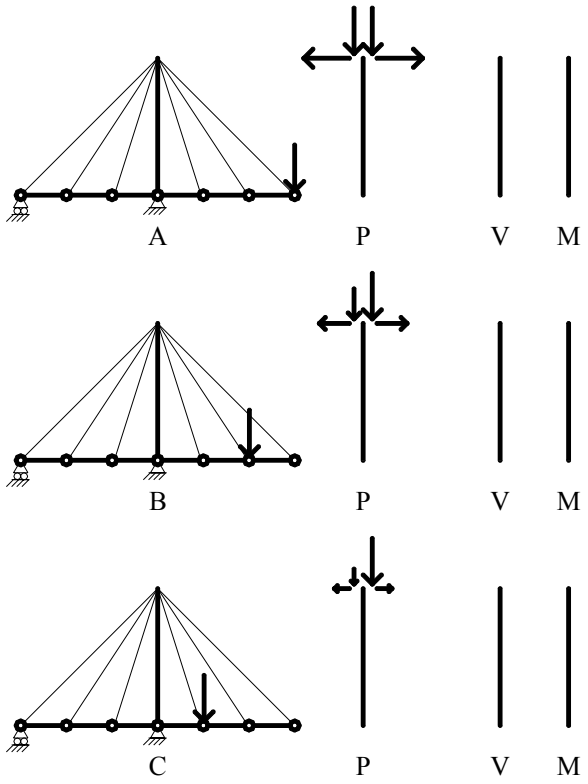

Figure 4.20 Load, Shear, and Moment Diagrams for Radiating Pylon System with Pinned Base Pylon

4.3.4 Harp Layout Fixed Base Pylon

The second bridge configuration to be examined will be the self anchored harp bridge with a fixed base pylon, such as the one shown in Figure 4.21. If the pylon of the bridge in Figure 4.21 is considered to be infinitely rigid, then similar to the radiating system, the anchor cable is ineffective. Qualitative load, shear, and moment diagrams for harp pylon systems with infinitely rigid pylons are shown in Figure 4.22.

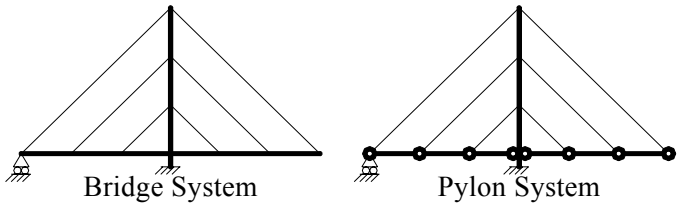

Figure 4.21 Harp Bridge and Pylon System with Fixed Base Pylon

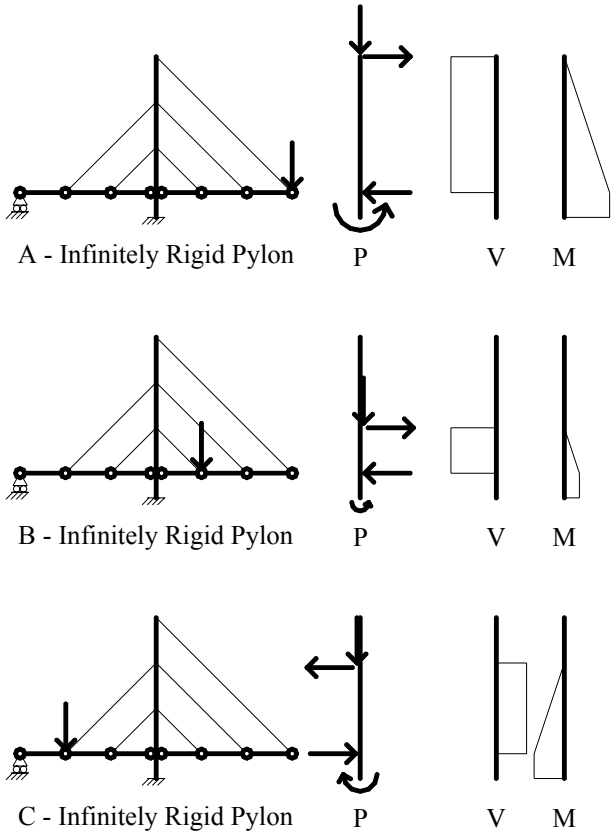

Figure 4.22 Load, Shear, and Moment Diagrams for Harp Pylon System with Infinitely Rigid Fixed Base Pylon

The behavior of the harp pylon system with a rigid pylon is similar to that of the radiating pylon system with a rigid pylon except that, for the harp configuration, portions of the pylon may not be subject to bending.

If the pylon in a self anchored harp pylon system is less than infinitely rigid, then it will deform when load is placed on the spans and the anchor cable will be subjected to deformation. An analysis similar to the one performed for the radiating pylon system may be used to analyze the structure. Qualitative load, shear, and moment diagrams for a harp pylon system with varied ratios of anchor cable to pylon stiffness are shown in Figure 4.23.

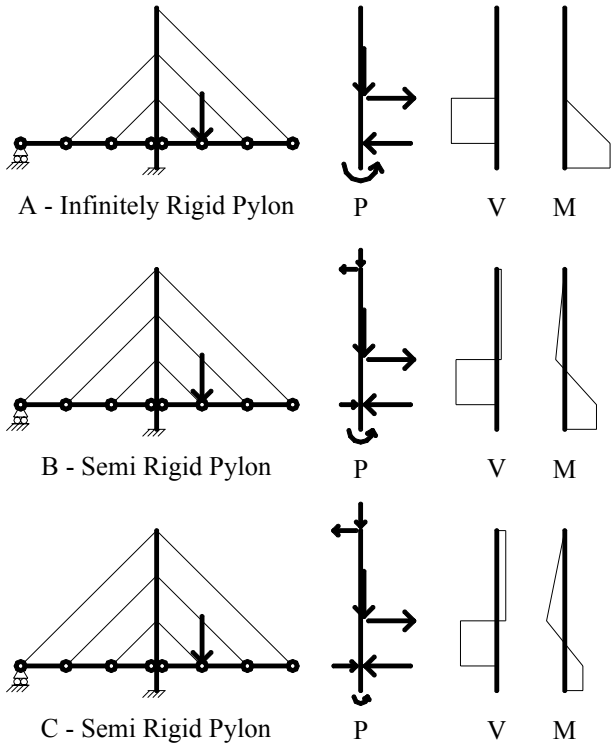

Figure 4.23 Load, Shear, and Moment Diagrams for Harp Pylon Systems with Fixed Base Pylons of Varied Stiffness

The sketches in this figure illustrate the change in the load, shear, and moment applied to the pylon as the pylon becomes more flexible with respect to the anchor cable. As the pylon becomes less rigid the anchor cable takes more load. This causes the shear diagram to shift by the amount of horizontal load carried by the anchor cable. The moment diagram changes both due to the change in the shear diagram and by a change in the moment applied at the fixed base.

When loads are placed on the side span of a harp pylon system with a deformable pylon the anchor cable may cease carrying tension. When this happens the pylon will be unassisted by the anchor cable and be subject to loads similar to those shown in Figure 4.22 C. Therefore, similar to the radiating system, it may be advantageous to proportion the spans so that the anchor cable remains in tension.

4.3.4 Harp Layout Pinned Base Pylon

If a pinned base pylon is used in a bridge with a harp stay layout the pylon system will correspond to the one shown in Figure 4.24. This Pylon System will be stable for loads on the main span but will become unstable for loads on the side span if the spans are not proportioned properly.

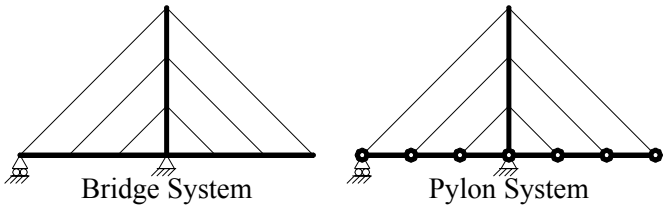

Figure 4.24 Harp Bridge and Pylon System with Pinned Base Pylon

Figure 4.25 shows the forces applied to the pylon for loads at several locations. The anchor cable provides the stabilizing force to the pylon, which behaves like a simply supported beam for the support of horizontal forces applied by the stays. The magnitude of the axial load in the pylon becomes greater as a load is moved further from the pylon.

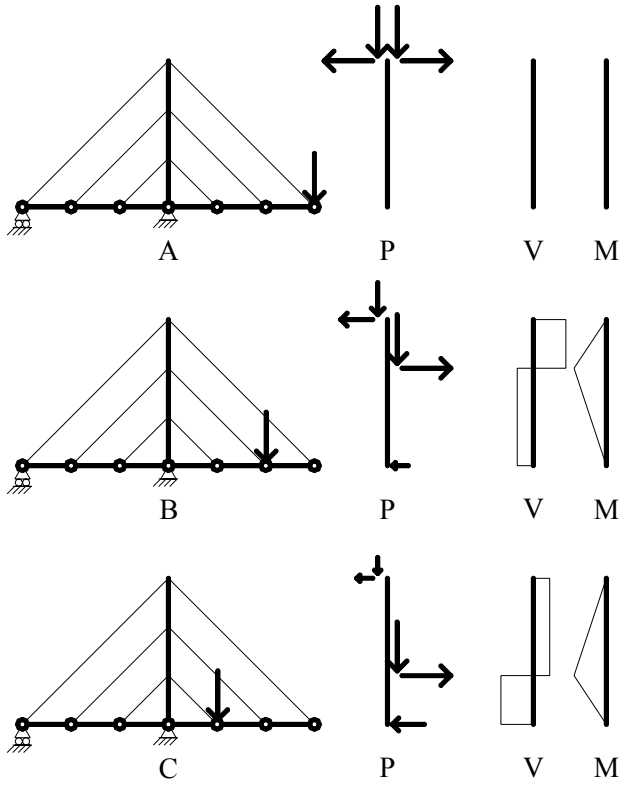

Figure 4.25 Load, Shear, and Moment Diagrams for Harp Pylon System with Pinned Base Pylon

4.3.5 Fan Layout

The third bridge configuration to be considered is the self anchored fan layout. This pylon system behaves similar to the harp pylon system, but because the stays are connected more closely on the pylon, the load, shear, and moment diagrams change slightly. Investigation of this system may be carried out by the reader using the concepts presented previously.

4.3.6 Pylon System Considerations

The pylon system idealization is useful in determining the stabilizing effect of the pylon on a bridge system and the axial loads, shears, and moments in a pylon that functions as a key stabilizing member in a system. It should be noted that instability of a pylon system does not imply instability of a bridge system. If the pylon system is unstable the stability of the bridge system must be achieved through the stiffening girder as will be discussed in Section 4.4. It must also be noted that the simplified analysis used in this section neglects two major contributions to the moments induced in a pylon.

The first factor neglected is the eccentricity of axial forces carried by the undeformed pylon. Often in a pylon, the stays are anchored a significant distance away from the centroid of the member. This is illustrated in the hypothetical pylon shown in Figure 4.26.

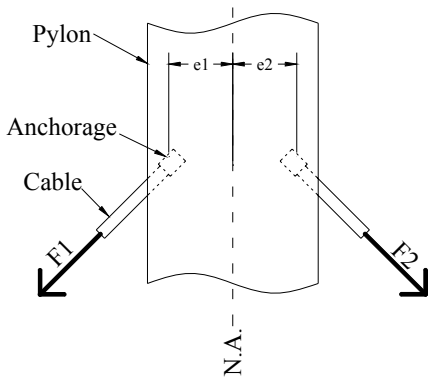

Figure 4.26 Eccentric Loading of a Pylon

If the stays are symmetric about the centroid of the pylon (e1 = e2), each stay carries the same load (F1 = F2), and the stays are at the same angle of inclination, then the resultant force will act at the centroid of the section. However, during construction the anchors might be placed slightly away from their intended locations and symmetry will no longer exist. Construction tolerances should therefore be considered when evaluating eccentricities. Typically an eccentricity of 100 millimeters, which is about 4 inches, is assumed to account for construction tolerances (Gimsing 1997, 346). Another way that eccentricity will be introduced is if the stays are subject to unequal axial forces or if the forces are at different angles of inclination. If this is the case, then the resultant force will not act at the center of the section. Eccentric axial loads will induce a moment of p*e, where p is the axial load and e is the eccentricity. In the case of construction tolerance the direction of the eccentricity will not be known, so the moment caused by the eccentricity must be assumed to increase the magnitude of moment in the pylon. In the case of eccentricity due to unbalanced forces in symmetrically placed stays the direction of eccentricity is known and the resulting moment may either increase or decrease the bending moment in the pylon.

The second factor neglected in the analysis of this section are second order effects, also called geometric nonlinearity. Second order effects are caused by deflection of a system under loads. As discussed above, a deformable pylon is required to activate the anchor cable. As the tip of the pylon deflects horizontally toward the main span, the anchor cable is elongated and thus carries load to assist the pylon. However, as the pylon deflects the vertical forces from the stays, anchor cable, and self weight of the pylon will no longer be acting directly above the base of the pylon as shown in Figure 4.27. This will cause additional moment and additional deflection. Because cable-stayed bridges may be very flexible structures second order effects may be important (Fleming and Egeseli 1978, 148).

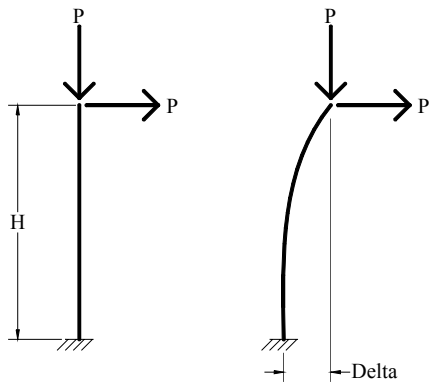

Figure 4.27 Pylon Deflection and Second Order Effects

The contribution of cable forces to second order effects in the longitudinal direction typically result in destabilizing forces. Although transverse behavior of pylons has not been discussed, it is interesting to note that cable forces may act as stabilizing forces when considering transverse deflection of a pylon. This is because

as the top of the pylon deflects transversely, the member that the cables are anchored to may not deflect as much as the pylon top in the transverse direction. If this is true, the cable force will no longer be vertical and the horizontal component of the cable force will act as a stabilizing force on the pylon (Gimsing 1997, 346). Conversely, if the member that the cable anchors to deflects more than the pylon in the transverse direction, then the horizontal component of the cable force will be destabilizing.

4.4 Girder Systems

The third idealization that will be examined is the case of having a rigid stiffening girder and a flexible pylon. This system will be referred to as the "girder system" and may be modeled by using hinges in the pylon where it intersects all other members. Loads acting on the girder system may be idealized as distributed loads or concentrated loads at any point along the girder. A typical three span self anchored cable-stayed bridge with a continuous stiffening girder is shown with its corresponding girder system in Figure 4.28.

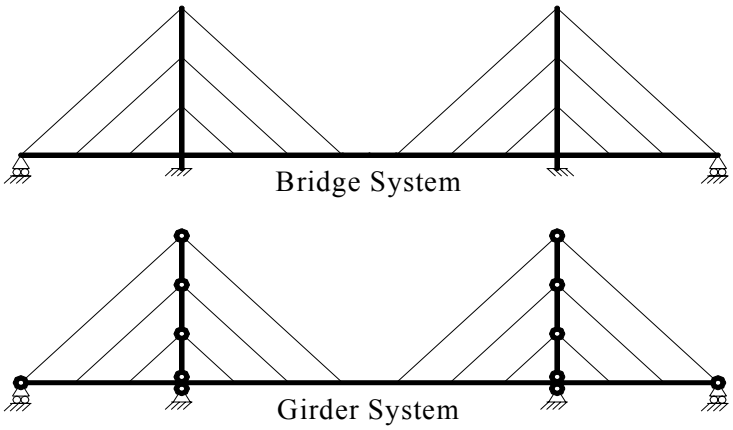

Figure 4.28 Bridge System with Corresponding Girder System Idealization

The bridge system of Figure 4.28 is shown with a fixed base pylon. However, because the bending stiffness of the pylon is neglected in the girder system, the girder system idealization will be the same regardless of the fixity of the pylon base. It should also be noted that a half system is not shown for the girder system. This is because the girder at the center of the main span is permitted to transmit shear and moment, so it can not be removed. If the stiffening girder in the bridge system contained one or more hinges near midspan, which is sometimes the case, these hinges would be modeled in the girder system. However, the use of hinges will not be explored.

The behavior of the stiffening girder in the girder system will be greatly influenced by the relative stiffness of the girder compared to the stays. The concept is similar to the relation between the stiffness of the pylon relative to the anchor cable in a fixed base pylon system. When a downward vertical deflection is imposed on the stiffening girder at a stay supported location, resisting loads will be induced in both the girder and the stay. The girder will resist the deflection by shear and moment and the stay will resist the deflection by axial tension. The amount of load caused in each member by the imposed deformation will be a function of that member's stiffness. For the stay, it will depend on the axial stiffness and the stay's orientation. For the girder, it will depend on flexural stiffness.

4.4.1 Radiating Layout

The bridge and girder system for a self anchored radiating layout bridge are shown in Figure 4.29. This system is indeterminate, so the moment diagram in the stiffening girder will be affected by the stiffness of the members.

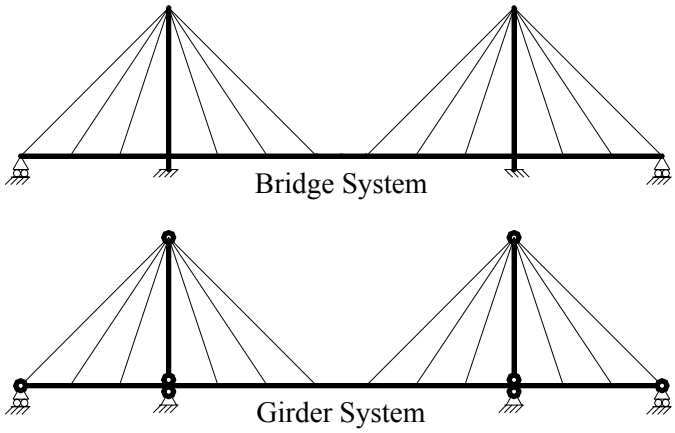

Figure 4.29 Radiating Bridge and Girder System

Figure 4.30 shows several girders with their corresponding moment diagrams, which are drawn to scale. Diagram A represents a girder system with axially inextensible cables. Because the cables are inextensible, they act as rigid supports as long as they are under tensile loading. Diagram D represents an infinitely rigid girder. Because the girder is infinitely rigid the cables are unable to participate in carrying the load, so they may be removed. Diagrams B and C represent cases where both the stiffening girder and cables resist the load. Diagram B represents a system which has a larger ratio of axial cable stiffness to girder flexural stiffness than does diagram C. The maximum moment in Diagram D is more than fourteen times larger than the maximum moment in Diagram A. The advantage of using axially stiff stay-cables to reduce bending moment in the stiffening girder is clear.

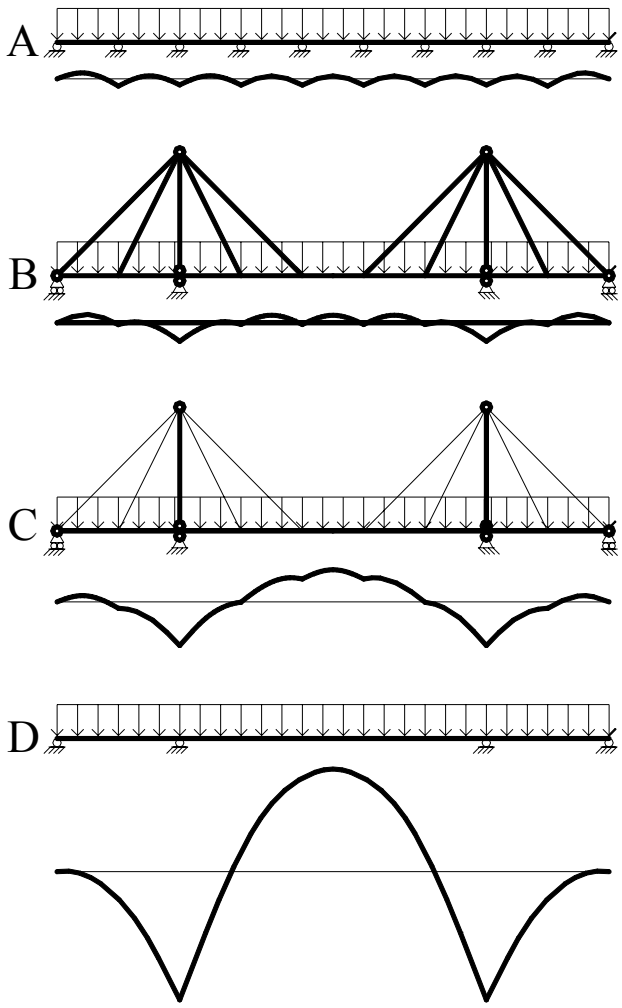

Figure 4.30 Moment Diagrams for Varied Ratios of Cable Axial Stiffness to Girder Flexural Stiffness

If a radiating girder system is subject to side span loads which cause the anchor cable tension to be fully released, then the excess load on the side span must be resisted by the girder if the system is to remain stable. This is illustrated in Figure 4.31. Diagram A is a beam with roller supports at its ends and its ninth points, this corresponds to placing a roller at every cable and at the pylon bases. Diagram C

corresponds to a beam with roller supports only at its ends and at the pylon bases. Diagram B corresponds to the girder system of Figure 4.30 B (stiff cables). It may be seen that for the side span loading the girder system behaves more like the girder with four supports than the girder with 10 supports. This is because the cables are ineffective in compression, once the tension in the anchor cable is released, the load is carried primarily by bending in the girder, however, some of the stays do remain in tension. This illustrates how the stiffening girder may stabilize a system when the cables and pylon are unable to.

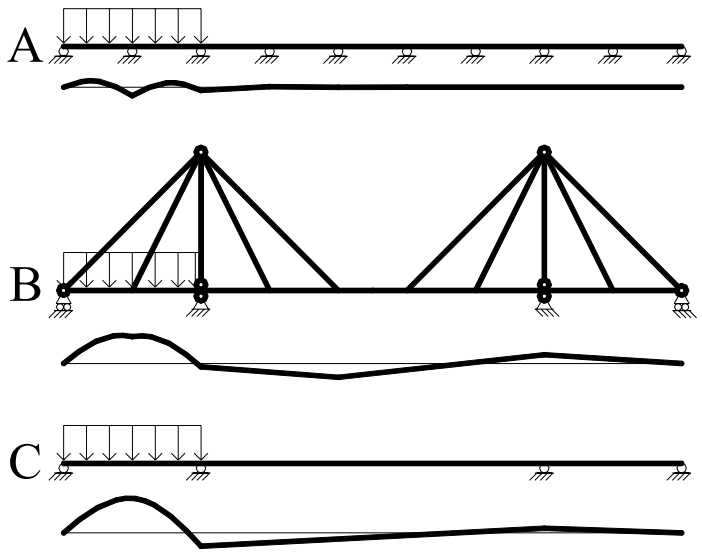

Figure 4.31 Moment Diagrams for Uniform Load on Left Side Span

4.4.2 Harp Layout

The cable system for the harp layout is unstable under gravity loads. One way that the harp bridge system may be stabilized is by utilizing the stiffness of the

girder. Figure 4.32 compares the bending moments caused by a concentrated load for two different girder configurations. Configuration A has no cable support, while configuration B is a girder system with stiff cables in the harp configuration. Although the cable system of the harp bridge is unstable, the cables are still capable of reducing the bending moments in the girder.

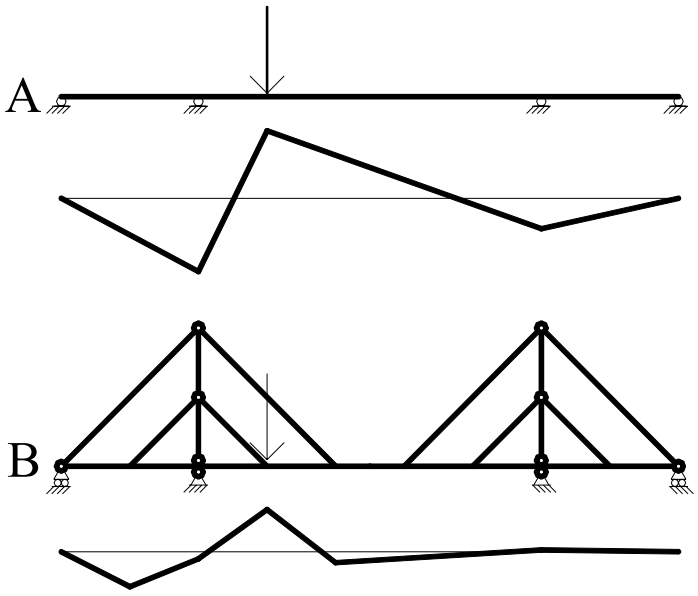

Figure 4.32 Moment Diagrams for Concentrated Load on Harp Girder System

4.4.5 Girder System Considerations

The girder system idealization is useful for understanding the effect that the ratio of cable stiffness to girder stiffness has on the moments induced in the stiffening girder as well as the potential stabilizing effect that the stiffening girder may have on the bridge system. However, the first order analysis used to examine the

girder system neglects three major factors that will influence the behavior of an actual structure.

The first factor that was neglected is the nonlinear axial stiffness of the stays. This nonlinearity, which is of the geometric nature, is caused by the sag in the stays of an actual bridge structure. Sag will increase if stays are: longer, heavier per unit length, oriented closer to horizontal than vertical, and/or lightly loaded. The sag in a stay may be described by the elastic catenary curve, which is roughly parabolic when sag in the cable is small (Podolny and Scalzi 1976, 342). For a given stay that has sag, as the stay is loaded in tension, the sag is reduced and the axial stiffness increases (Podolny and Scalzi 1976, 352). This behavior was not modeled in the girder systems of this section, but must be considered in design of cable-stayed bridges.

The second factor that was neglected is the eccentricity of axial load in the undeformed stiffening girder. The stay anchorages in the stiffening girder will not always be near the centroid, so eccentric axial load will cause additional bending moment in the stiffening girder. This concept is similar to eccentricities in the pylon, that were discussed in Section 4.3.6, except that now only one cable force is present at a section.

The third factor that was neglected is second order effects in the deformed stiffening girder. The stiffening girder in the self anchored systems shown before are subjected to large compression forces, so as the girder deflects second order moments will be developed. This concept is similar to second order effects in the pylon that were discussed in Section 4.3.6.

4.5 Actual Bridge Systems

The behavior of real cable-stayed bridge systems will be bounded by the behavior of the cable systems, pylon systems, and the girder systems described in the prior sections. For many bridges the stability will not be dominated by any one of the three systems. However, some bridges may be seen to have characteristics close to the three systems discussed.

An example of a bridge that likely behaves similar to the radiating girder system (Figure 4.29) is the Stromsund Bridge, which was shown in Figure 3.28. The bridge has a pinned base pylon which corresponds to the model used in the girder system. The girders of the bridge are relatively deep and flexurally stiff. The ratio of axial stiffness of the cables to flexural stiffness of the stiffening girder is not known, but it is likely that the designers used stiff cables to obtain behavior similar to that shown in Figure 4.30 B. It may also be noted that this bridge has a stable cable system for all loads in the main span and for live loads up to a limiting intensity on the side span. Many early cable-stayed bridges utilize deep girders and relatively slender pylons. These bridges are likely to exhibit structural behavior similar to that of the girder system.

The trend of using more stays to support flexible stiffening girders has led to cable-stayed bridges where the stability is dominated by the pylon. Stabilization of systems with a harp stay layout and a slender stiffening girder will be dominated by the rigidity of the pylon because the cable system is unstable and the girder is flexible.

Although some bridges may be seen to exhibit characteristics similar to one of the system idealizations, most bridges will utilize a combination of pylon stiffness, girder stiffness, and cable system stability to create an efficient structure. It

is not possible to easily identify how these structures behave without using computer software, or obtaining field response data.

It must also be noted that all of the system idealizations explored above were analyzed using simplifying assumptions which may not be ignored in design. Complicating factors such as nonlinear axial cable stiffness, eccentricities of load in the pylon and/or girder of the undeformed system, and second order effects in the pylon and/or girder of the deformed system must be considered. To reasonably account for all of these factors nonlinear second order analysis are needed. An additional consideration for analysis is nonlinear material behavior. Under normal loadings materials in a cable-stayed bridge should remain elastic. However, material nonlinearities such as yielding and cracking may occur under extreme event loadings (Ulstrup et al 1992, 21).

4.6 Advantages and Disadvantages of Cable-Stayed Bridge Systems

This section will briefly describe potential advantages and disadvantages of some of the many possible components and configurations that may be utilized in cable-stayed bridges.

4.6.1 Longitudinal Stay Layout

Some of the many considerations to be examined when evaluating the longitudinal stay layout of a bridge include: the weight of cable steel used, the axial force introduced into the stiffening girder, the axial forces and bending moments imposed on the pylon, the stiffness of the bridge system, the ease of anchorage or saddle design, and the aesthetic qualities of the stay layout.

The radiating stay layout is the most structurally advantageous of the longitudinal stay arrangements with respect to the cables and stiffening girder. This is because the stays are all at a maximum inclination for a given pylon height. The system is advantageous because as the inclination of a stay is increased the vertical component of the force in that stay increases while the horizontal component of the force decreases. Therefore, to support a given vertical load, a stay with a steeper inclination will be required to carry less total force and may therefore have a smaller cross section. A stay with a steeper inclination will also introduce less axial force into the stiffening girder and less horizontal force to be resisted by the pylon or anchor cable (Troitsky 1988, 21). The radiating layout has the disadvantage of having all of the cables converge at the top of the pylon. This often presents a challenge when designing an anchoring or saddle system that will accept the large forces being transmitted from all of the cables. The layout of the system also requires the entire pylon height to be subjected to the large axial loads introduced by the cable forces (Podolny and Scalzi 1976, 37-38). The bending moments imposed on the pylon are moderate (Walther et al 1999, 25). The radiating stay layout also has a cable system that is stable of first order, so higher stiffness and strength may be obtained than for other layouts given the same material use. However, the layout is considered by many to be less than ideal with respect to aesthetic appeal.

The harp system is often considered the most aesthetically pleasing stay layout because there are no intersecting cables. The system also offers the advantage of having the anchorages spread out over the height of the pylon. This makes detailing the anchorages or saddles easier because there is less congestion and smaller forces being transferred in a given area. The distribution of the cables along the pylon

also means that the entire tower is not subject to maximum loads allowing an efficient pylon design (Podolny and Scalzi 1976, 37-38). The disadvantage of the harp system is that stays which anchor near the base of the tower are not at a maximum inclination. This means that those stays must carry a greater load to supply the required vertical support and that larger axial forces are introduced into the stiffening girder. On the other hand, the stays which anchor near the base of the pylon are much shorter than they would be if the radiating system were used, so the increase in cross section may be offset by the decrease in length. The harp cable system has the disadvantage of having an unstable cable system, so bending of the pylon or girder is required to stabilize the system, and the stiffness of the system will be less than that of the radiating system.

The fan system combines the radiating and harp systems to overcome the weakness of each while retaining their benefits. The stays in a fan system are close to being at maximum inclination, but when they intersect the pylon they are spread out. The steep inclination of the stays minimizes the force in the stays and the axial force introduced into the stiffening girder, while the distribution of stays as they meet the pylon greatly simplifies the design of the anchorage or saddle system (Podolny and Scalzi 1976, 38). Similar to the radiating system, the fan system's structural weakness lies in the fact that nearly the entire height of the tower is subjected to large loads. The fan system is also considered to be less visually pleasing than either of the two systems that it is adapted from.

4.6.2 Pylon Type

Considerations to be examined when evaluating the pylon type to be used for a bridge include: compatibility with the selected stay layout, efficiency in resisting

transverse loads, size of the area occupied by the base of the pylon, required median width, and constructability.

Cantilevered towers have the disadvantage of being less effective than frame pylons for the transmission of transverse loads. Single central towers require the median of the roadway to be at least the width of the tower and are usually only used with a single central plane of stays. However, the footprint of the tower base is confined to a small area. Twin towers do not require a wide median because the towers are not contained in the roadway. However, the footprint of the towers is spread over a larger area than for a single tower. Twin towers are generally used with two vertical planes of stays. Side towers do not dictate the width of the median and have a small footprint at their base. They may be used with vertical or inclined planes of stays or both. Side towers are generally inefficient, but may possess aesthetically pleasing appearances. Cantilevered towers generally use vertical members which are simple and economical to construct (Walther et al 1999, 25).

Frame pylons are more efficient than cantilevered towers for the transmission of transverse loads and do not dictate median width in most cases. Portal frames are usually only used with two vertical planes of stays and the footprint of the base of portal frames is often significant. Shear and bending in the frame legs at the level of the stiffening girder due to transverse loads may be an issue (Gimsing 1997, 358). A-frame pylons have the advantage of transferring large portions of transverse loads through normal forces in the pylon legs instead of through shear and bending. They are also generally stiffer than portal frames in the transverse direction (Gimsing 1997, 364). The disadvantage of the A-frame is that the area that the foundations are located in is larger than for other types of pylons, especially if the roadway is at a

significant elevation. A-frame pylons are compatible with two inclined planes of stays or with stays in the radiating layout. Diamond and delta modified A-frames have bases which occupy less area than that of a pure A-frame. However, the change of direction of the legs of diamond and delta modified A-frames reduces their structural efficiency and leads to reduced transverse stiffness and large bending moments in the legs (Gimsing 1997, 365-366). Modified A-frames are considered to be quite aesthetically pleasing. All frame pylons utilize horizontal or inclined members to achieve structural efficiency. While this may lead to a savings in material, construction of horizontal or inclined members may be difficult, so construction costs may offset any reduction in material costs (Gimsing 1997, 357).

4.6.3 Transverse Stay Layout

Considerations to be examined when evaluating the transverse stay layout include efficiency in resisting unbalanced loads on the roadway and vertical clearance above the roadway.

Vertical clearance above the roadway will not be restricted by vertical planes of stays, however it may be restricted by inclined planes of stays. Single planes of stays are ineffective for resisting unbalanced loads on the roadway and require a torsionally stiff stiffening girder. Multiple planes of stays which anchor to the stiffening girder on opposite sides of the roadway are effective at resisting unbalanced loads and may be used with torsionally flexible stiffening girders (Walther et al 1999, 24).

4.6.4 Stiffening Girder Type

Considerations to be examined when evaluating the type of stiffening girder to be used include: flexural stiffness, torsional stiffness, aerodynamic stability, dead load, and constructability. Flexural stiffness may be important if the stiffening girder is to play a major role in the global stability of a bridge, or for distribution of concentrated loads to multiple stays (Gimsing 1997, 286). The flexural stiffness of most types of stiffening girders may be varied considerably to obtain the desired behavior. Torsional stiffness of the stiffening girder is required when the transverse stay layout does not resist unbalanced loads. Solid web open cross sections and trusses lack torsional stiffness, while solid web closed sections may possess significant torsional stiffness (Gimsing 1997, 305). Aerodynamic stability of the stiffening girder is a complex topic and requires careful attention for any cross section or material used. The dead load of the stiffening girder is often a significant portion of the overall gravity load on a cable-stayed bridge. Concrete sections will have a larger dead load than steel cross sections. This may be advantageous in certain circumstances and detrimental in others. An example of increased dead load benefiting a structure is the use of concrete backspans to balance the dead load on bridges with short backspans and long steel main spans (Gimsing 1997, 213). Constructability and cost of a cross section or material are based on local material availability and contractor capabilities. Contractor capabilities and material availability are largely regional factors and may also vary over time.

4.6.5 Anchoring Method

The two main considerations when selecting an anchoring method are the normal forces in the stiffening girder and the forces that must be resisted by the

foundations at the abutments. The magnitudes of the axial forces in the stiffening girder will determine if strengthening of the stiffening girder (above what is needed to meet other requirements) is needed. In an earth anchored bridge system, the girder is subjected to tension. In a partially earth anchored bridge system, the girder is subjected to tension and compression. Finally, in a self anchored bridge system, the girder is subjected to compression. The stiffening girder in a self anchored system generally requires the most strengthening to resist the axial loads (Gimsing 1997, 284-286).

The magnitude and direction of forces transmitted to the foundations at the abutments will determine how large the foundations must be. In the earth anchored system, horizontal and vertical forces from both the anchor cable and stiffening girder must be resisted. In partially earth anchored systems, horizontal and vertical forces from the anchor cable and only the vertical forces from the stiffening girder must be resisted. In the self anchored system, only vertical forces from the anchor cable and stiffening girder are resisted by the foundations. The earth anchored system generally requires the largest foundations at the abutments. To determine which system is most economical for a given situation, the tradeoff between the cost of resisting axial forces in the stiffening girder must be compared to the cost of resisting the loads at the foundations (Gimsing 1997, 284-285).

4.6.6 Longitudinal Pylon Fixity

A major consideration when investigating the longitudinal fixity of a pylon is its effective length in the longitudinal direction. If buckling is the controlling factor in the pylon design, then a pylon with a longer effective length will require a larger cross section to carry a given load than a pylon with a shorter effective length.

The fixity of the pylon along with the support conditions of the stiffening girder and the anchorage of the anchor cable will influence the effective length of the pylon in the longitudinal direction (Gimsing 1997, 347).

The bridge system shown in Figure 4.33 has fixed base pylons, a stiffening girder that is supported exclusively on roller supports, and anchor cables that are anchored to the stiffening girder. The pylon of this system has a theoretical effective length of 2L in the longitudinal direction. This is because the base of the pylon is fixed and the pylon top is unrestrained. If the pylon top undergoes a longitudinal displacement, the stiffening girder will displace longitudinally as well, and the anchor cables will be unable to restrain the top of the pylon (Gimsing 1997, 348). If the pylon in Figure 4.33 had a pinned base, the effective length in the longitudinal direction would theoretically be infinite, and the pylon would be unstable.

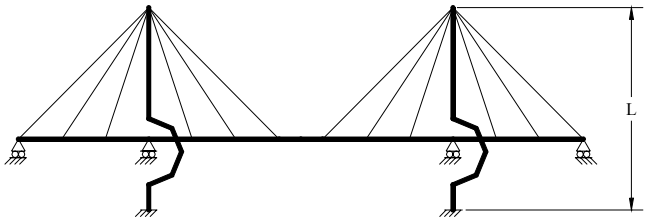

Figure 4.33 Bridge System where Effective Length of Pylon is 2L in the Longitudinal Direction (Adapted from Gimsing 1997, 349)

The bridge system shown in Figure 4.34 has fixed base pylons, a stiffening girder that is supported horizontally and vertically at one end, and anchor cables which are earth anchored. If the anchor cable in this system remains in tension, the pylon will have a theoretical effective length of approximately 0.7L in the

longitudinal direction. This is because the base of the pylon is fixed and the pylon top is restrained, although not perfectly, by the anchor cable. The axial stiffness of the anchor cable will affect the effective length of the pylon. If the cable is inextensible the effective length is 0.7L. As the cable becomes more axially flexibly the top of the pylon becomes less restrained and displacement of the pylon top must be considered (Gimsing 1997, 348). If the pylon in Figure 4.34 had a pinned base, the theoretical effective length in the longitudinal direction would be 1L, assuming an inextensible anchor cable which remains in tension for all loads. If the bridge system in Figure 4.34 used anchor cables which attached to the stiffening girder, the anchor cables would still be effective at restraining the pylon because the girder is not longitudinally movable (Gimsing 1997, 348).

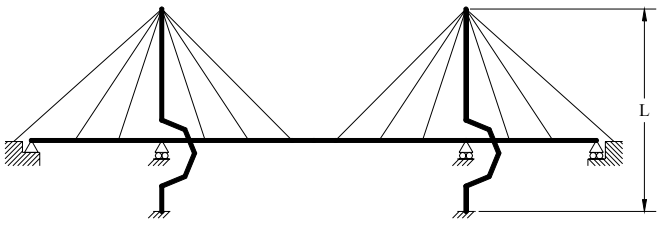

Figure 4.34 Bridge System where Effective Length of Pylon is 0.7L in the Longitudinal Direction

It should also be noted that buckling of the pylon in the transverse direction may control the design of the pylon, especially for cantilevered tower pylons. When considering the effective length in the transverse direction, the pylon will have a theoretical effective length of 2L for pylon self weight and cable self weight. However, the effective length of the pylon in the transverse direction for

cable forces may be less than 2L. If the stiffening girder deflects transversely less than the pylon top, then the cable forces will not remain vertical and will provide a restoring force. An in depth discussion of transverse buckling of pylons will not be presented here. Interested readers should refer to "Cable Supported Bridges: Concept and Design" by Niels J. Gimsing.

4.6.7 Choice of Cable-Stayed Bridge Systems

The preceding sections introduced some of the advantages and disadvantages of cable-stayed bridge system components. Selecting the most efficient combination of system components for a cable-stayed bridge for a given situation requires consideration of the issues presented, as well as many more which were not discussed in this document. There are many systems that can be utilized, and determining the best one for a given situation is an interesting and challenging task.

4.7 Stay-Cable Vibration

Stay-cables are generally light, flexible, and have small amounts of damping. This combination of properties makes stays susceptible to vibration (Gimsing 1997, 271). Not surprisingly, oscillation of individual stays on cable-stayed bridges has occasionally been a problem. An FHWA scanning tour of cable-stayed bridges in Europe found several interesting conclusions regarding cable vibrations. It was found that vibrations are more likely on long stays than on short stays, although what constitutes a "long stay" was not specifically discussed. It was also found that stays with smooth polyethylene (PE) stay sheathing were particularly susceptible to vibration, and installation of certain surface protrusions on the sheathing was capable of mitigating the problem. Danish researchers shared their knowledge that, in some

cases, stays with smooth sheathing, which initially did not experience oscillation, would become prone to oscillation as the sheathing became worn or abraded. The Danes were unable to develop a model to predict rain-wind induced vibration, and believe that movement of the pylon and stiffening girder (sympathetic vibration) may contribute to the initiation of oscillations. It was also discovered that rain-wind induced vibrations do not appear to be a problem when locked wire rope is used for the stays. This may be due to the fact that the sheathing on locked wire rope takes the contour of the underlying wires, thus surface protrusions are built into the sheathing, and the mechanism of rain-wind induced vibration is disrupted. In Europe, stay vibration mitigation was effectively accomplished using three techniques: stay sheathing surface protrusions, dampers, and cross ties (FHWA 2001, 50-51).

In the past it was believed that stay oscillations under wind action were caused solely by vortex shedding (Gimsing 1997, 271). Vortex shedding is a phenomenon experienced by bluff sections immersed in a fluid flow, such as stay-cables subject to wind. Vortices will be shed from the immersed body, alternating between the top and bottom of the section. The vortices cause a change in pressure, which in turn, creates an alternating force on the body. If the magnitude of the force is sufficient, and the frequency of the force is near the natural frequency of the immersed body, then large amplitude oscillations may take place (Ulstrup et al 1992, 27-28).

The natural frequency of a taught cable with rigid supports is a function of the tension, weight per unit length, and total length of the cable. The frequency of the oscillating force caused by vortex shedding is a function of wind speed, diameter of the stay, and the Strouhal number. For the dimensions of typical stays, the Strouhal number may be regarded as a constant. Using formulas for the natural frequency of

the vortex shedding force and the natural frequency of a stay, it is possible to determine the wind speed at which the natural frequencies will be equal. Vortex shedding induced oscillation of the stay may be expected at that wind speed. The stays on a cable-stayed bridge usually have a wide range of natural frequencies, mainly due to the varied length of the stays. Therefore, a range of wind velocities may cause vortex shedding induced stay oscillation (Gimsing 1997, 271).

In the 1980's stay oscillation was observed at wind speeds well below those predicted for vortex shedding. Through several investigations it was determined that the oscillations were caused by a previously unrecognized phenomenon called rain-wind induced vibration. Rain-wind induced vibration occurs on stays with smooth sheathing when both rain and wind are present. It was found that water rivulets on the smooth sheathing may drastically change the aerodynamic behavior of the stay. It was also determined that a fixed rivulet was incapable of inducing vibration. This indicates that the ability of the water rivulet to change both shape and location under wind action is key to rain-wind induced vibration (Gimsing 1997, 271).

The use of a small protruding spiral rib on the stay sheathing has proved effective at disrupting rain-wind induced vibration. The Higashi Kobe Bridge built in the early 1990's utilized longitudinal ribs on the polyethylene stay sheathing (Gimsing 1997, 273). The general shape of the sheathing used on the Higashi Kobe Bridge is shown in Figure 4.35.

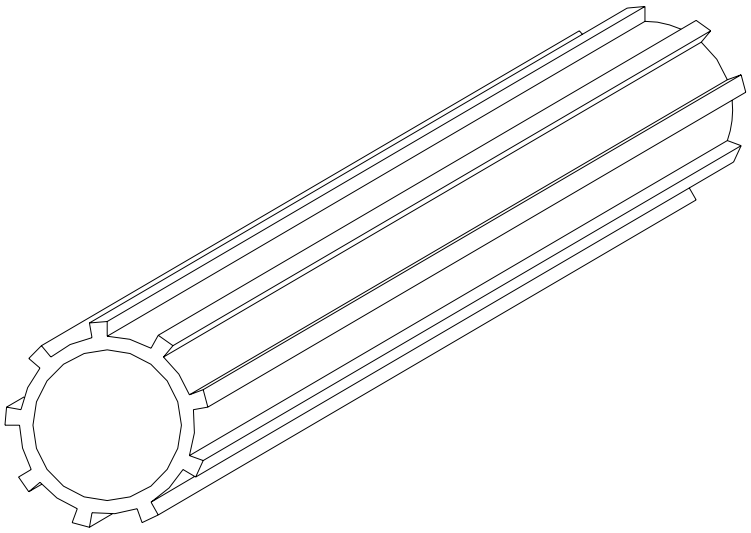

Figure 4.35 Stay Sheathing with Longitudinal Ribs (Redrawn with Modifications from Gimsing 1997, 273)

Other methods of reducing stay vibration include the use of dampers and cross ties (FHWA 2001, 50). Dampers are used to increase the damping of a stay to the point where oscillation is not a problem. Heavy-duty shock absorbers may be used as dampers. The shock absorbers, which are similar to those used on automobiles, are placed between the stiffening girder and the cable as shown in Figure 4.36 (Scanlan and Wardlaw 1977, 180). This approach was implemented as a retrofit on the Brotonne Bridge in France, two years after construction, to mitigate excessive cable oscillations (FHWA 2001, 50). The approach was also used on the Sunshine Skyway, however, in this case the shock absorbers were included in the design as a preventative measure (Gimsing 1997, 273).

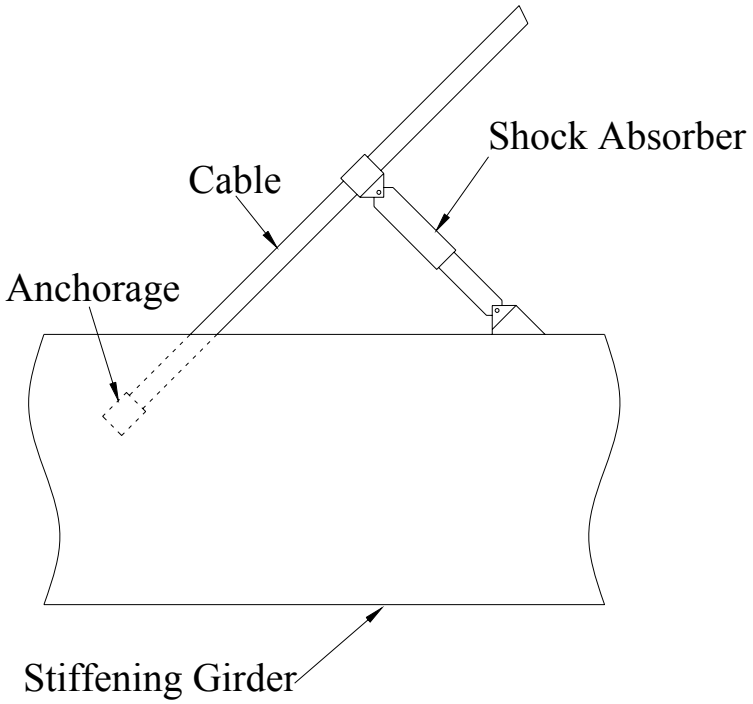

Figure 4.36 Vibration Suppression Using Shock Absorber (Adapted from Scanlan and Wardlaw 1977, 202)

The use of cross ties restricts the vibration of individual cables by attaching several stays together with small diameter cables as shown in Figure 4.37 (Gimsing 1997, 272). This approach was used in conjunction with stay sheathing surface protrusions on the Faroe Bridge in Denmark. Before the retrofit, stays on this bridge experienced vibrations with amplitudes of up to five feet due to rain-wind induced sympathetic vibration (FHWA 2001, 50).

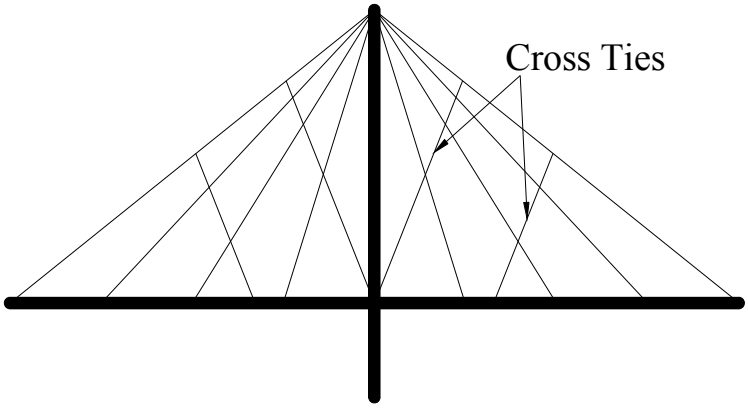

Figure 4.37 Vibration Suppression Using Cross Ties (Adapted from Gimsing 1997, 272)

There are two major problems that result from large stay-cable vibration. First, excessive cable oscillation may cause fatigue in the cables which can lead to strand breakage. Second, large cable oscillation may also be perceived by bridge users and lead to discomfort and alarm (Gimsing 1997, 272). The occurrence of stay vibration is not fully understood and is currently not able to be predicted with certainty. If unacceptable vibrations occur on an existing bridge, retrofits such as the ones introduced above may be used to mitigate the problem. Many new cable-stayed bridges are incorporating preventative measures in their design to prevent oscillations from occurring in the first place.

Chapter 5

CONCLUSIONS

The behavior of cable-stayed bridge systems will be bounded by the behavior of the cable systems, pylon systems, and the girder systems as described in the prior sections. Some bridges have characteristics close to one of the three idealized systems discussed, however, many bridges will not be dominated by characteristics similar to any one of the systems. Early cable-stayed bridges used small numbers of stays and flexible pylons that were often pinned at the base. These bridges have characteristics similar to the girder system, however, it is unlikely that such a system will be used at the Indian River Inlet site. Recent cable-stayed bridges have utilized large numbers of stays in conjunction with flexible stiffening girders. The use of a bridge with these characteristics, which are similar to those of the pylon system, is likely at the Indian River Inlet site.

Although it seems likely that a bridge resembling the pylon system will be used at the Indian River Inlet site, this is only speculation because the design has not yet been developed. Once the design has been developed, an evaluation of the bridge can be performed to determine which of the idealized systems it resembles. Careful attention should be given to the support conditions of the bridge. Many combinations of support conditions may be used to obtain an efficient design, so it is possible that the support conditions of the bridge will not match those used in the cable, pylon, and girder system models discussed herein. In that case, cable, pylon, and girder systems which utilize appropriate support conditions should be developed and investigated.

In the systems presented in this paper, horizontally movable, pinned, and rigidly fixed support conditions were assumed. However, in an actual structure these conditions will not exist. A support modeled as horizontally moveable will have a friction force resisting horizontal movement and will not provide perfectly rigid restraint in the vertical direction. A pinned support will not be perfectly rigid either, and may be modeled using stiff springs in the horizontal and vertical directions. A rigidly fixed support may be modeled using stiff springs in the horizontal and vertical directions and a torsional spring that resists rotation. The support condition idealizations made in the simple conceptual models should be revised to incorporate imperfect support conditions. Investigation of imperfect support conditions in the simple conceptual models will aid in the understanding of the behavior of the actual bridge system.

Once conceptual models have been investigated, a structural model which accounts for the stiffness of the cables, pylon, and stiffening girder should be developed. This model should be based upon stiffness properties and support conditions as determined from the bridge plans. It should be noted that during load tests, and for much of the time that monitor data is collected, the loading on the bridge will be small compared to the capacity of the bridge. This may limit the impact of issues such as second order effects and cable nonlinearity, possibly allowing first order analysis to be utilized. This may simplify the modeling process and allow the use of a relatively simple program such as STAAD Pro. This model, referred to as the "preliminary structural model," should be used to predict the behavior of the bridge under the various loading conditions.

As the bridge is constructed, instrumentation should be installed to allow the axial forces and bending moments at selected locations in the pylon and stiffening girder, as well as axial loads in some of the cables, to be monitored. In any given cross section, the gages and gage locations used will likely be similar to what was planned for the proposed arch bridge.

Upon completion of the bridge, load tests should be performed to determine how live load is carried by the actual bridge structure. The data obtained from these tests should be compared to the predictions made using the preliminary structural model. If the preliminary structural model adequately predicts the obtained responses, then it will not need to be modified. However, if the results do not match well, an investigation should be performed to determine the source of the discrepancy. It may be possible to adjust parameters, such as stiffness or support conditions in the preliminary model to obtain better agreement. However, due to the many parameters that can be adjusted, it is important to rationally determine which parameters to adjust. Special attention should be given to identifying how the fixity of the pylon bases and restraint of the stiffening girder at the abutments and pylons affect the model. Once the preliminary structural model has been calibrated to provide satisfactory results, it should be considered to be the "initial condition" model of the bridge.

Long term monitoring should then be performed by tracking monitor data and repeating the load tests on the bridge at specified time intervals. The results from the load tests and monitor data should be compared to the results from the initial load tests and previous monitor data, as well as to the results predicted by the initial condition model of the bridge. If the results from load tests or monitor data are

substantially different than data collected from similar previous situations, an investigation should be conducted to determine the cause.

Change in the structural response of the bridge may indicate structural deficiencies of bridge components such as cracking of concrete, breaking of strands, or deterioration of the bearings. These events would cause changes in member stiffness, pre-stress force in a section, and restraint conditions at supports. Material phenomena, such as creep and shrinkage, and bridge maintenance, such as stay re-tensioning, may also cause changes in the behavior of the bridge.

If changes in the behavior of the bridge are detected through monitor data or load tests the cause of the behavior change should be identified. The cause should then be incorporated into the initial condition model of the bridge which will then become a "current condition" model. The current condition model, once verified, should then be used to predict the response of the bridge under various loadings as was done using the preliminary and initial condition models. The current condition model should be updated as needed to track the long term performance of the bridge.

The STAAD Pro model described above may be adequate for predicting the static response of the bridge under ordinary loads. However, for more substantial loadings on the bridge, such as a significant wind event, complicating factors such as nonlinear axial cable stiffness and second order effects must be considered. Predicting the dynamic response of the bridge is also likely to be beyond the capability of a STAAD Pro model. To account for nonlinear or dynamic behavior, a finite element model will need to be developed. This will be a complex task. Prior to attempting to create such a model, issues such as support fixity and member stiffness properties should be well understood. However, an understanding of these issues should have

been gained through development of the preliminary, initial condition, and current condition STAAD Pro models. The finite element model may then be calibrated and updated in a manner similar to that used for the STAAD Pro model. The finite element model may then be used to predict dynamic response of the bridge as well as response due to extreme loading events.

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APPENDIX A

INDIAN RIVER INLET BRIDGE MONITORING PROGRAM MANUAL

INDIAN RIVER INLET BRIDGE

MONITORING PROGRAM MANUAL

by

Daniel Frederick Weston

And

Timothy Jeffrey Stuffle

CHAPTER 1

PURPOSE OF THE MANUAL

1.1 Introduction

The Delaware Department of Transportation (DelDOT) is planning to build a 1,000-foot single rib concrete tied arch bridge over the Indian River Inlet. The bridge, which is shown in Figure 1.1, is being designed by the Figg Engineering Group. The contract for construction of the bridge has not yet been awarded. It will be a monumental structure with an intended service life of over 75 years. The inspection and maintenance of the bridge during that time will require a substantial investment of time and resources. Under the direction of DelDOT, the University of Delaware's Center for Innovative Bridge Engineering (CIBrE) will be installing a long-term structural monitoring system on the bridge during construction and monitoring the bridge from the time of construction through the first bi-annual inspection. The system will enable DelDOT and CIBrE to monitor the bridge during construction to ensure safety, understand the initial bridge condition and use it as a baseline for future evaluations, as well as monitor changes in the bridge condition during its life to aid in ongoing management. Finally, as a result of the significance of the bridge to the surrounding community and its potential as a tourist destination, portions of the monitoring information will be available to the public at an information kiosk.

1.2 Purpose of the Manual

This manual has been written to explain and describe CIBrE's plans to instrument and monitor the bridge. It will detail important information relating to the instrumentation plan in the preliminary, construction, initial condition, and long-term condition stages of the project. The manual is intended to serve as an aid to CIBrE, DelDOT, the contractor, the designer of the bridge (Figg), and the independent reviewer of the bridge (T.Y. Lin). It should be noted that although every effort has been made to make the plans presented within this manual practical, they may need to be modified as unexpected circumstances arise during construction. It is also important to recognize that this manual is not part of the contract documents and should not serve as a replacement for those documents.

Figure 1.1 Proposed Indian River Inlet Bridge (http://www.indianriverinletbridge.com)

CHAPTER 2

SCOPE AND GOALS OF THE PROJECT

2.1 Scope and Goals

In establishing a program to monitor the Indian River Inlet Bridge, CIBrE and DelDOT have focused on creating a system that will not only assist in the construction and maintenance of the bridge but also provide design verification for certain aspects of its behavior. It is the goal of this plan that the data collected from sensors will be used in conjunction with computer models to address the specific design, construction, and maintenance issues associated with the Indian River Inlet Bridge. To this end, the monitoring program has been divided into five stages: preliminary, mechanically stabilized earth wall construction, bridge construction, initial condition, and long-term condition, each with its own unique objectives.

2.1.1 Preliminary Stage

Work done during the preliminary stage of the project will prepare for and assist in completing all future stages. Elements of this stage include creating timelines and procedures, delineating expected results, and obtaining, assembling, and testing instrumentation equipment. It is expected that some unexpected circumstances during the project will lead to modification of the plans set forth in this manual; however, a significant effort has been taken to minimize these events.

2.1.2 Mechanically Stabilized Earth Wall Construction Stage

The main goals of the MSE wall construction stage are to install gauges, set up data acquisition equipment, use the data acquisition equipment to obtain data during construction, compare obtained data with expected results, and issue reports stating and explaining the results.

The purpose of monitoring the MSE walls during the construction stage will be to verify the analytical predictions stress in the reinforcement and the predicted settlement of the MSE walls. Verification will consist of comparing measured data to predicted values. Any major discrepancies between the actual and predicted values will be investigated and reported to DelDOT. At the same time, the data will be made available to the contractor to assist in the construction of the MSE walls and approaches.

2.1.3 Bridge Construction Stage

The main goals of the bridge construction stage are to install gauges, set up data acquisition equipment, use the data acquisition equipment to obtain data during construction, compare obtained data with expected results, and issue reports stating and explaining the results.

The purpose of monitoring during the construction stage will be to verify the analytical predictions of the evolution of stresses during construction and to verify that the temporary stays, support cables, and post-tensioning strands are properly tensioned. Verification will consist of comparing measured erection stresses to those calculated by computer models and determining whether discrepancies occur. Any major discrepancies between the actual and predicted stress values will be investigated

and reported to DelDOT. At the same time, the data will be made available to the contractor to assist in the erection of the bridge.

2.1.4 Initial Condition

The initial condition stage will begin as soon as the bridge is completed and may last up to one year. During this time, the main objective will be to establish the baseline behavior of the bridge, so it can be used for design verification and as a reference during the long-term condition stage. Baseline behavior studies will include determining the initial state of stress in the bridge after the construction loads have been applied but before the bridge is opened to traffic. In addition, the baseline response of the bridge to wind, thermal, and traffic loading and the initial settlement of the arch bases will be determined. These measured behaviors will be used to verify the analyses. In each case, collected data will be compared to predicted values to determine whether the computer models used in design accurately predict the actual bridge behavior. Special attention will be paid to aspects of the behavior in which the design team and review team predicted different states of stress, or areas in which stresses are close to the design limits. After this, the baseline responses should be catalogued for future use.

2.1.5 Long-Term Condition

When the initial condition stage ends, the long-term condition stage will begin. The main goals during this stage will be to supplement standard visual inspections, to assist in the maintenance of the bridge, and to determine whether the bridge behavior is changing as it ages. During this time, the condition of the post-tensioning strands, support cables, bearings, and structural members will be assessed

by comparing the baseline behavior to the current behavior. The current behavior of the bridge will be determined by examining data from events such as wind, temperature change, and load tests similar to those conducted in the initial condition stage. Changes in the response will be investigated to determine whether they represent significant maintenance issues.

CHAPTER 3

RESPONSIBILITIES

3.1 Responsibilities of Involved Parties

This chapter is intended to identify the specific responsibilities of CIBrE and the contractor for each portion of the IRIB instrumentation program. The chapter will act as a supplement to special provision 763621 University of Delaware Bridge Monitoring Program by fully explaining the special provision and including details which were not available at the time the special provision was written.

3.2 General Responsibilities

This section identifies responsibilities that are relevant to the entire project. The responsibilities listed here apply to all sections in the remainder of this chapter.

3.2.1 CIBrE

- Provide all instrumentation equipment except support cable load cells. Instrumentation equipment shall include gauges, gauge wire, data acquisition systems, communication cables, and the means of attaching these elements to the bridge.
- Install all instrumentation equipment except support cable load cells.

- Provide the contractor with a contact person who will handle all correspondence between the contractor and the University throughout the project.
- Ensure that all University staff who perform work at the bridge construction site or at casting yards have appropriate safety training for the tasks they are performing. This shall be done independently at no cost to the contractor.
- Notify the contractor when alternative portions of the bridge will be instrumented if such action is determined necessary by DelDOT.

3.2.2 Contractor

- Provide the University access to the bridge construction site, the bridge superstructure, temporary construction towers, temporary stays, pile caps, bearings, and fabrication yards for precast members throughout the construction process as detailed in the remainder of this chapter
    - ❖ "Provide access" means that the contractor will provide the means for University staff to physically get to the needed locations to install and read all instruments. It is expected that the contractor will provide the needed lifts, ladders, or other devices to allow this to happen.
    - ❖ The period of time allotted for the University to access a section shall not overlap with the period of time allotted for access to any other section.

- ❖ During periods of access provided to the University, appropriate lighting shall be provided by the contractor.

- Move bulky equipment, including but not limited to support cable load cells, to required locations.
- Notify the University contact person at least four weeks prior to work on any item listed in this chapter.
- Allow the University to draw power from onsite generators during selected activities.
- Ensure that gauges, wires, blockouts, data acquisition systems, and any other instrumentation equipment installed by the University are not damaged by the contractor or any subcontractor once they are placed.
- Allow the substitution of alternative segments if it is determined necessary by DelDOT.

3.3 Pile Cap Responsibilities

This portion of the project includes the installation of four chloride penetration monitors (embedded) in the pile cap of the southern arch base.

3.3.1 CIBrE

- Provide and install four chloride penetration monitors and corresponding wires.
- Consult the contractor to determine appropriate blockout location as not to interfere with constructability.
- Provide and install the 8″ x 8″ x 6″ foam blockout.

3.3.2 Contractor

- Provide four hours of access to the southern arch base pile cap before it is cast, but after the entire rebar cage is placed (University staff shall have access and means to reach the entire rebar cage) to allow for the installation of chloride penetration sensors and blockout.
- Upon request from the University approve or suggest the location of 8″ x 8″ x 6″ foam blockout.

3.4 Temporary Construction Tower Responsibilities

This portion of the project involves installing five sets of anemometers on the southern temporary construction towers to take wind speed data during construction. It also involves instrumenting the temporary stays on the southern end of the bridge to allow the forces in the stays to be determined. The instrumentation in this portion of the program will send data to a data logger which will be located at the base of the temporary construction towers. This data logger will be referred to as a *base station* for the remainder of the manual. The specifics of the instrumentation for this portion of the project have not been finalized because they are dependent on the final plans for the temporary construction towers, which are not yet available.

3.4.1 CIBrE

- Review the final design of the temporary towers, which will be provided by the contractor, and devise the specifics of the instrumentation plan.
- Provide the contractor with details of anemometer locations and how they will be installed.

- Provide the contractor with details of the stay monitoring scheme, including which stays will be instrumented and how the instrumentation will be installed (it is anticipated that load cells and/or accelerometers will be used).
- Provide details to the contractor about the base station to be set up at the base of the temporary towers that will log data from anemometers and temporary stay instrumentation.
- Provide and install anemometers, mounting brackets, and wires.
- Provide and install stay instrumentation and wires.
- Provide and install base station to collect data.

3.4.2 Contractor

- Provide the University with the final temporary tower design so that anemometer locations may be determined and stay monitoring may be finalized.
- Provide three hours of access per anemometer set to install five sets of anemometers along with their corresponding wires at various heights on the southern temporary towers.
- Provide two hours of access per stay to allow the University to install instrumentation on the southern temporary tower stays.
- Allow a base station to be set up at the base of the temporary towers that will log data from anemometers and temporary stay instrumentation.
- Allow the University to access the base station as needed to retrieve data; this is anticipated to be a weekly event.

- Provide access to the University to read accelerometers on temporary stays each time they are read (this will require lifts to reach the upper stays).

3.5 Bearing Responsibilities

This portion of the instrumentation program involves installing linear potentiometers (surface mounted) on three of the bearings in the southern arch base and on two bearings for each backspan (two at each abutment).

3.5.1 CIBrE

- Provide and install linear potentiometers, wires, and data acquisition equipment.
- Provide and install data loggers to record data during construction.

3.5.2 Contractor

- Provide 8 hours of access to the bearings in the south arch base immediately upon their installation on the lower portion of the arch base, but before the portion of the arch base above them is cast to allow for the installation of three linear potentiometers and a data logger.
- Provide six hours of access to the bearings at the end of each backspan (abutments) upon completion of each backspan to allow for the installation of two linear potentiometers and one data logger per location.

- Provide four hours of access to the bearings in the south arch base immediately upon unblocking for inspection of instrumentation equipment.
- Provide means for gauge wire to be run from the bearings in the south arch base:
    - ❖ Option One – Install communication conduit with pull wire through arch base to tie beam (as per bridge plans), then provide three hours of access for the installation of gauge wires through the conduit.
    - ❖ Option Two – Install communications conduit with pull wire from arch base directly to UD communications enclosure, then provide three hours access for the installation of gauge wires through the conduit.
- Notify the University as to which option for the wire run from the south arch base will be used; this should be done as soon as practicable once the decision has been made.

3.6 Arch Section Responsibilities

This portion of the instrumentation program involves the installation of weldable foil strain gauges (embedded), vibrating wire strain gauges (embedded), accelerometers (surface mounted), and GPS units (surface mounted) in selected arch segments.

- Arch sections to be instrumented include S-C1U, S-C1D, S-T10U, S-T10D, S-T22U, S-T22D, and S-T36U.

- Sections S-C1U, S-C1D, S-T10U, S-T10D, S-T22U, and S-T22D will each be instrumented with four weldable foil strain gauges, four vibrating wire strain gauges, and one accelerometer.
- Section S-T36U will be instrumented with four weldable foil strain gauges, four vibrating wire strain gauges, one accelerometer, and one GPS unit.

3.6.1 CIBrE

- Provide and install all gauges listed above along with corresponding wires.
- Consult the contractor to determine appropriate blockout locations as not to interfere with constructability.
- Provide and install the 8″ x 8″ x 6″ foam blockout.
- Provide and install data loggers to record data during construction.

3.6.2 Contractor

- Provide access for four hours per section to all arch sections listed above before the section is cast, but after the entire rebar cage is placed (University staff shall have access and means to reach the entire rebar cage) to allow for the installation of embedded gauges and blockouts.
- Upon request from the University approve or suggest the location of 8″ x 8″ x 6″ foam blockouts.
- Ensure that gauges, wires, and blockouts are not damaged once they are placed.

- Provide access to the interior of each section for eight hours per section to all arch sections listed above immediately after it has been cured for installation of accelerometers, GPS units (if applicable), and data logger.

3.7 Support Cable Responsibilities

This portion of the instrumentation program involves the installation of load cells on selected support cables. The load cells are anticipated to be hollow cylinders with a height of roughly 36 inches. The load cell will sit upright in the arch with the support cables running through the center of the cylinder and the anchorage resting on top of the cylinder. Each load cell is expected to weigh several hundred pounds. The load cells must accommodate the pressurized nitrogen system in the support cables and include a gauge for monitoring the pressure of the nitrogen.

- Support cables to be instrumented include 5S, 5D,11S, 11D, and 17S.
- The load cells for these cables will be installed in the arch; corresponding arch sections are S-A9U, S-A9D, S-A21U, S-A21D, and S-A33U.

3.7.1 CIBrE

- Provide and install data loggers to record data during construction.

3.7.2 Contractor

- Purchase support cable load cells that meet the specifications provided in Table 3.1 from a qualified source (two potential suppliers are listed in Table 3.2).

- Install support cable load cells in sections S-A9U, S-A9D, S-A21U, S-A21D, and S-A33U.
- Install the nitrogen sensing system (this shall be done in consultation with the load cell provider).
- Provide access to the University for four hours per load cell once load cells have been placed/installed to allow for installation of one data logger per load cell.

Table 3.1 Load Cell Specifications

Capacity	5,000,000 lb range
Environmental	Splash proof
Temperature range	-30˚ to 150˚ F
Non-linearity	1%FS or better
Dynamic reading capability	
Calibration certification required	
Must accommodate pressurized nitrogen system	
Include a pressure gauge with a range from 0 to 10 psig that reads in 0.1 psi or finer increments and is capable of communicating with data collection equipment	

Table 3.2 List of Potential Load Cell Suppliers

Name	Mailing Address	Website
CTL Group	5400 Old Orchard Road Skokie, IL 60077-1030	www.CTLGroup.com
Honeywell Sensotec	2080 Arlingate Lane Columbus, Ohio, 43228	www.sensotec.com

3.8 Tie Beam/Central Beam Responsibilities

This portion of the instrumentation program involves the installation of weldable foil strain gauges (embedded), vibrating wire strain gauges (embedded), accelerometers (surface mounted), and GPS units (surface mounted) in selected segments of the tie beam and central beam.

- Tie Beam/Central Beam sections to be instrumented include TB-T1U, TB-T1D, TB-T19U, TB-T19D, TB-T37U, CENTRAL BEAM STA. 294+12.500, CENTRAL BEAM STA. 294+78.000, and CENTRAL BEAM STA. 305+22.000.
- Tie Beam/Central Beam sections TB-T1U, TB-T1D, TB-T19U, TB-T19D, TB-T37U, and CENTRAL BEAM STA. 294+12.500 will each be instrumented with four weldable foil strain gauges, four vibrating wire strain gauges and one accelerometer.
- Central Beam sections CENTRAL BEAM STA. 294+78.000 and CENTRAL BEAM STA. 305+22.000 will each be instrumented with one GPS unit.

3.8.1 CIBrE

- Provide and install all gauges listed above along with corresponding wires.
- Consult the contractor to determine appropriate blockout locations as not to interfere with constructability.
- Provide and install the 8″ x 8″ x 6″ foam blockout.
- Provide and install data loggers to record data during construction.

3.8.2 Contractor

- Provide access for four hours per section before the section is cast, but after the entire rebar cage is placed to the following Tie Beam/Central Beam sections: TB-T1U, TB-T1D, TB-T19U, TB-T19D, TB-T37U, and CENTRAL BEAM STA. 294+12.500 (University staff shall have access and means to reach the entire rebar cage) to install embedded gauges and blockouts.
- Provide 8 hours of access to the interior of the sections TB-T1U, TB-T1D, TB-T19U, TB-T19D, TB-T37U, and CENTRAL BEAM STA. 294+12.500 immediately after each has been cured to install accelerometers and data loggers.
- Provide four hours of access to the interior and top of the central beam at stations 294+78.000 and 305+22.000 immediately after each has been cured to install GPS units and data loggers.
- Upon request from the University approve or suggest the location of 8″ x 8″ x 6″ foam blockouts.
- Ensure that gauges, wires, and blockouts are not damaged once they are placed.

3.9 Roadway Section Responsibilities

This portion of the instrumentation program involves the installation of weldable foil strain gauges (embedded), vibrating wire strain gauges (embedded), and accelerometers (surface mounted) in selected roadway segments.

- Roadway sections to be instrumented include 1S-8, 1S-17, 2S-3U, 2S-3D, 2S-30U, 2S-30D, 2S-57U, 1N-8, 1N-17, 2N-3U, 2N-3D, 2N-30D, 2N-30U, and 2N-57U.
- Roadway sections 1S-8, 2S-3U, 2S-3D, 2S-30U, 2S-30D, 2S-57U, 1N-8, 2N-3U, 2N-3D, 2N-30D, 2N-30U, and 2N-57U will each be instrumented with four weldable foil strain gauges, four vibrating wire strain gauges and one accelerometer.
- Roadway sections 1S-17 and 1N-17 will each be instrumented with four vibrating wire strain gauges.

3.9.1 CIBrE

- Provide and install all gauges listed above along with appropriate wires.
- Consult the contractor to determine appropriate blockout locations as not to interfere with constructability.
- Provide and install the 8″ x 8″ x 6″ foam blockout .
- Provide and install data loggers to record data during construction.

3.9.2 Contractor

- Provide access for four hours per section to all roadway sections listed above before the section is cast but after the entire rebar cage is placed (University staff shall have access and means to reach the entire rebar cage) to install embedded gauges and blockouts (this will take place at the precasting plant).

- Provide 8 hours of access per section to the interior of all roadway sections listed above immediately after it has been placed for the installation of accelerometers and data loggers.
- Upon request from the University approve or suggest the location of 8″ x 8″ x 6″ foam blockouts.
- Ensure that gauges, wires, and blockouts are not damaged once they are placed.

3.10 Grouted Tendon Responsibilities

This portion of the instrumentation program involves the installation of void detection sensors in selected tendons of the bridge. The sensors will be able to detect voids in the grout and to identify whether the void is due to water or air. The purpose of the sensors will be to allow evaluation of the effectiveness of the grouting techniques used. The specific tendons to be instrumented have not been determined.

3.10.1 CIBrE

- Provide contractor with details of what tendons will be monitored.
- Provide and install all embedded gauges and wires (list types and quantities).

3.10.2 Contractor

- Provide appropriate extra hole in anchorages.
- Allow gauge wire to be pulled through tendon along with post-tensioning strand.
- Provide access to each section (2 hours of access per tendon) once after it has cured.

3.11 Through-Hole Responsibilities

This portion of the instrumentation program involves the installation of through-holes in selected segments of the bridge. Through-holes labeled as "UD Communication" in the bridge plans will be used to run gauge wires from roadway segments into adjacent tie/central beam where data acquisition equipment is located. For wires to be run from the roadway segments into the tie/central beam, there must be a hole in both the roadway segment and the tie/central beam. Furthermore, these holes must align to allow wires to be run through them. The holes also must remain clear once the closure pour between the roadway segment and tie/central beam is completed. It is anticipated that some sort of flexible or rigid conduit will be used to connect the holes in the roadway segments and central/tiebeam so that the closure pour does not fill in the wire way. A summary of the "UD Communication" through-hole locations is provided in Table 3.3.

There are also three through-holes in the bridge plans which are for monitoring equipment but are not labeled as "UD Communication." These holes will be used for the installation of GPS units and wires. The location and purpose of these three holes are given in Table 3.4.

3.11.1 CIBrE

- Communicate with contractor to convey the intent of the holes and what criteria must be met for them to be functional.

3.11.2 Contractor

- Install "UD Communications" through-holes as per the bridge plans.
- Install through-holes for GPS units as per bridge plans.

- Ensure that "UD Communication" through-holes match within 1/8″, are of 2″ diameter, and are clear after the closure pour.

Table 3.3 "UD Communication" Through-Hole Segments

Tie-Beam Section/ Central Beam Station	Adjacent Roadway Sections		Reference Sheets
TB-T1D	NB SB	2N-3U 2S-3U	B-313 B-401 B-416
TB-T19D	NB SB	2N-30D 2S-30D	
TB-T37U	NB SB	2N-57U 2S-57U	
TB-T19U	NB SB	2N-30U 2S-30U	
TB-T1U	NB SB	2N-3D 2S-3D	
STA. 294+12.500	NB SB	1N-8 1S-8	B-334 B-401 B-416

Table 3.4 Other UD Through-Hole Segments

Central Beam Station/ Arch Section	Purpose	Reference Sheets
STA. 294+78.000	Southern Backspan	B-207
STA. 305+22.000	Northern Backspan	B-244
S-T36U	Arch Crown GPS	B-346

3.12 Data Collection During Construction

Data acquisition during construction will be accomplished by using temporary data loggers. The temporary data loggers will be installed once segments

are cured (cast in place) or placed (precast) and will remain until hardwiring is run to the permanent data acquisition stations, which are shown in the electrical portion of the bridge plans.

It is anticipated that one data logger will be used at each instrumented section of the arch, tie beam, central beam, roadway segments, bearings, and at all GPS units and support cable load cells. A summary of all locations where temporary data loggers will be used in the bridge during construction is presented in Table 3.5. The data loggers that will be used are contained in small metal boxes (dimensions to be determined) that will run on internal batteries and record data from the gauges to which they are wired. The data loggers will need to be read (data will be physically retrieved from them by a UD staff member) once per week. Installation of the data loggers has been accomplished for each location in the corresponding chapter sections above. Therefore this section does not describe installation of the data loggers but instead focuses on responsibilities related to retrieving data from the data loggers.

Table 3.5 Location of temporary data loggers during construction*

* Does not include base station

Section/Location Type	Section Names	Channels Per Section
Strain Gauges in Arch Sections	S-C1U, S-C1D, S-T10U, S-T10D, S-T22U, S-T22D, S-T36U	4
GPS in Arch Section	S-T36U	1
Load Cells in Arch Sections	S-A9U, S-A9D, S-A21U, S-A21D, S-A33U	1
Strain Gauges in Tie Beam Sections	TB-T1U, TB-T1D, TB-T19U, TB-T19D, TB-T37U	4
Strain Gauges at Central Beam station	CENTRAL BEAM STA. 294+12.500	4
GPS at Central Beam Stations	CENTRAL BEAM STA. 294+78.000, 305+22.000	1
Southbound Roadway Sections	1S-8, 1S-17, 2S-3U, 2S-3D, 2S-30U, 2S-30D, 2S-57U	4
Northbound Roadway Sections	1N-8, 1N-17, 2N-3U, 2N-3D, 2N-30D, 2N-30U, 2N-57U,	4
Bearing Locations	Southern Arch Base, Southern Abutment, Northern Abutment	3,2,2

3.12.1 CIBrE

- Collect data on a weekly basis.

3.12.2 Contractor

- Provide access (one hour per data logger) to data loggers installed in the arch, central beam, tie-beam, roadway, and bearings in order to download data. It is anticipated that data will be retrieved from all installed data acquisition systems on a weekly basis until installation of the permanent data acquisition systems has been completed.

3.13 Installation of Permanent Data Acquisition Systems

Installation of permanent data acquisition equipment includes the running of gauge wires from gauges to permanent data acquisition locations, the running of communication cables from each data acquisition location to the UD communications enclosure at the southern end of the bridge, and the installation and set up of each data acquisition system. Permanent installation of data acquisition equipment can not take place until all "UD Communication" conduits have been installed, power has been connected to all "UD Receptacles," and the UD communications enclosure has been completed.

3.13.1 CIBrE

- Provide and install gauge wires from gauge locations to data acquisition system locations.
- Provide and install data acquisition systems.
- Provide and install communication cables from data acquisition systems to the UD communications enclosure located at the southern end of the bridge.

3.13.2 Contractor

- Install "UD Communications" conduits as per bridge plans.
- Install wire pulls in "UD Communications" conduit as per bridge plans.
- Install "UD Receptacles" as per bridge plans.
- Install UD communications enclosure as per bridge plans.
- Once the first four items in this section have been completed, provide a total of 56 hours of access to the interior of the entire

arch, southern backspan central beam, southern abutment, and the UD communications enclosure to allow hardwire connections to be run from gauges to data acquisition systems, and from data acquisition systems to the UD communications enclosure.

- Once the first four items in this section have been completed, provide a total of 56 hours of access to the interior of the entire tie beam, both central beams, all northbound roadway segments, all southbound roadway segments, the southern abutment, and the UD communications enclosure to allow hardwire connections to be run from gauges to data acquisition systems and from data acquisition systems to the UD communications enclosure.

CHAPTER 4

MONITORING PROJECT TIMELINE

4.1 Overview

The scope of the monitoring project necessitates a detailed organizational format for efficient management. This chapter outlines CIBrE's proposed organizational scheme and directs the reader to future chapters that specifically outline the stages involved in the project.

The monitoring project is scheduled to last from 09/01/04 to 08/31/10. In this six-year timeframe, the monitoring program will be developed, installed, activated, and continuously utilized.

Once a conceptual version of the monitoring project was developed, the University was in continual contact with Delaware Department of Transportation (owner), Figg Engineering Group (designer), and T.Y. Linn (independent design reviewer). This communication has allowed everyone involved in the project to fully understand the importance of the monitoring project and the overall effect it will have on all stages of design and construction. Additionally, in the near future, the University will be in constant contact with the chosen contractor to insure the correct installation, activation, and collection of monitoring data.

The project schedule is intended to allow CIBrE and its staff to efficiently manage the project, which comprises five distinct stages: Stage I – Preliminary Work, Stage II – Mechanically Stabilized Earth Wall Construction, Stage III – Bridge Construction, Stage IV – Initial Conditions, and Stage V – Long-Term Conditions.

After the five stages of the monitoring project were defined, a detailed flowchart was developed to help illustrate the sequence of events, as shown in Figure 4.1.

It can be seen from the figure that a large portion of the monitoring project takes place during the construction, requiring synchronization of the monitoring project schedule and the construction schedule. The schedule and timeline in this manual were developed from contacts with DelDOT and Figg and from the design drawing (provided by Figg). It is expected that some changes will be necessary once a construction schedule is generated by the contractor and construction begins. Every effort has been taken to minimize these changes through careful development of the monitoring project.

The following sections briefly outline the five main stages of the project. The main objective of each stage is laid out and a brief description of the major tasks that will be accomplished in each stage is given. As a result of the complexity of each of the stages described below, an individual chapter has been devoted to each stage that will discuss, in detail, the objectives and components of each.

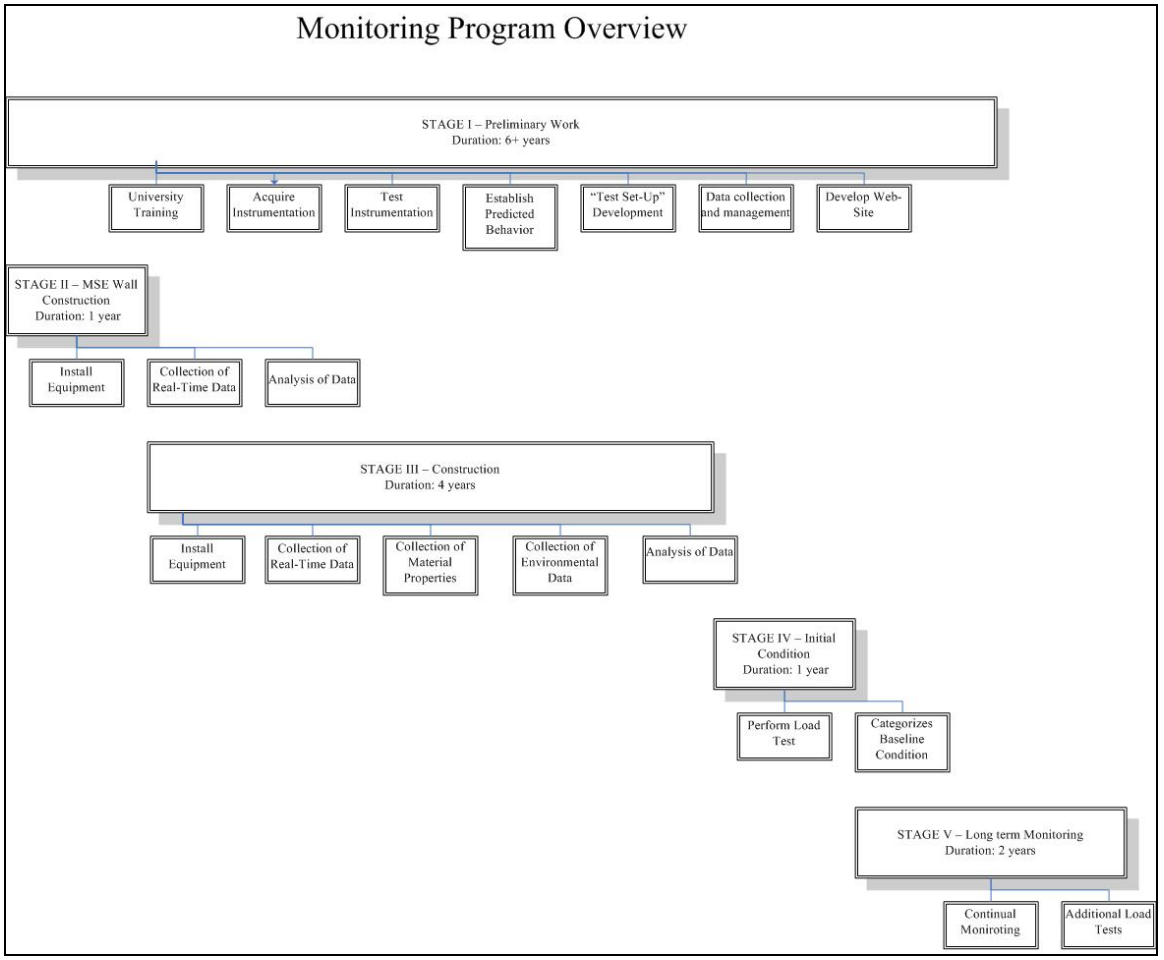

Figure 4.1 Monitoring Project Overview

4.1.1 Stage I - Preliminary Work

The primary objective of the preliminary work stage is to develop an efficiently organized management system that is capable of training University employees, acquiring and testing the instrumentation equipment, integrating and managing the collected data, and communicating the results via reports, web-sites, etc. to the desired persons.

The preliminary work on the monitoring project started when the project was proposed; work done during this stage laid the ground work for the remainder of the project. Stage I tasks include obtaining the required training (software and safety), acquiring all instrumentation equipment, establishing the predicted behavior of the bridge, developing a "test set-up" for the planned instrumentation, developing an efficient data collection and management system, and developing a website that is capable of displaying and archiving real-time monitoring data.

Stage specific objectives and components within Stage I will be discussed in greater detail in Chapter 5. Additionally, detailed schedules will be developed to assist in the completion of all aspects of Stage I.

4.1.2 Stage II – Mechanically Stabilized Earth Wall Construction

The primary objective of Stage II is to install all necessary strain gauges in the mechanically stabilized earth walls. Stage specific objectives and components within Stage II will be discussed in greater detail in Chapter 6. Additionally, detailed schedules will be developed to assist in the completion of all aspects of Stage II.

4.1.3 Stage III – Bridge Construction

The monitoring project will be implemented during construction. Implementation consists of installing and activating all instrumentation; commencing the collection of real-time monitoring data; creating a well-organized network and database of real-time monitoring data, material property data, and environmental conditions data; and verifying the predicted behavior of the bridge during construction.

Construction of the bridge is scheduled to take approximately four years. Major tasks during this stage include installation of all instrumentation equipment (gauges, data acquisition systems, global positioning system, gauge wires, and communication conduit), collection of real-time monitoring data, collection of material property data, documentation of environmental conditions, and analysis of real-time monitoring data.

Stage specific objectives and components within Stage III will be discussed in greater detail in Chapter 7. Additionally, detailed schedules will be developed to assist in the completion of all aspects of Stage III.

4.1.4 Stage IV - Initial Condition

At the completion of construction, important data must be gathered in order to classify the baseline behavior of the bridge. The primary task associated with this stage of the project is characterization of the response of the bridge to live load effects (thermal, wind, traffic) by performing various load tests on the bridge, collecting and correlating environmental data to real-time monitoring data, and characterizing the initial geometry of the bridge.

Stage IV will begin immediately following the completion of construction. It is expected that various tests will be conducted on the bridge before it is opened to traffic. Also, for certain baseline behaviors (ambient wind and traffic), monitoring will need to be conducted overtime. Consequently, Stage IV is predicted to last not more that one year.

Stage specific objectives and components within Stage IV will be discussed in greater detail in Chapter 8. Additionally, detailed schedules will be developed to assist in the completion of all aspects of Stage IV.

4.1.5 Stage V - Long-Term Condition

Once the baseline conditions of the bridge are determined through load testing and short-term monitoring, the long-term monitoring of the bridge will begin. The major objective of the long-term monitoring stage is to develop a long-term database of monitoring data and reports that will allow DelDOT to effectively monitor the structural health of the bridge throughout the service life of the bridge.

To accomplish this objective, the response of the bridge to live loads will be constantly monitored and compared to baseline measurements. The key areas of interest are the effectiveness of the bearings, the condition of selected post-tensioned cables, forces in the support cables, and internal strain in various sections of the bridge. Furthermore, every year the load tests (vehicle, ambient wind, etc.) will be conducted and compared to the initial load tests. It is anticipated that the University will conduct long-term monitoring on the bridge until the first scheduled bi-annual inspection.

Stage specific objectives and components within Stage V will be discussed in greater detail in Chapter 9. Additionally, detailed schedules will be developed to assist in the completion of all aspects of Stage V.

CHAPTER 5

STAGE I – PRELIMINARY WORK

5.1 Introduction

The primary objective of the preliminary work stage is to develop an efficiently organized management system. Given this objective, the primary tasks associated with this stage are obtaining the required training, acquiring all necessary monitoring equipment, developing a method to test and calibrate all monitoring equipment, developing an efficient data collection and management system, and developing a website that is capable of displaying and archiving real-time monitoring data. The following sections outline the necessary steps to accomplishing the objective of Stage I.

5.2 University Employee Training

University employees will need to be trained on how to install each gauge type, collect and read the data from each gauge type, and use the computer software necessary to operate the data acquisition systems. It is suggested that the manufacturers of the gauges and data acquisition systems provide some guidelines (owner's manuals) and possibly training courses that deal with project-specific conditions.

In addition, University employees will have to receive safety training. Most of the monitoring equipment will be installed in the field, which will require University employees to become certified in the required construction safety

procedures as specified by the contractor. The University will independently certify all its employees at no cost to the contractor. The contractor and University will be in close contact once the project begins to ensure that the correct training is given in a timely manner. All training should be complete before any monitoring equipment is installed on the bridge. Decisions will need to be made as to who is going to be trained (graduate students, lab technicians, and/or professors) and what task each party will be responsible for during the length of the project.

5.3 Monitoring Equipment

During the preliminary work stage of the project all necessary monitoring equipment will need to be chosen and purchased. Plans have been laid out in Chapter 7, Stage III – Bridge Construction as to where the monitoring equipment is going to be installed. This should serve as a guide to help the University purchase the correct type and amount of monitoring equipment. In addition, a list of all vendor contact information should be generated. Once the final construction schedule is determined by the contractor the University will determine a final schedule for the acquisition and installation of all monitoring equipment. For additional specific information on the monitoring equipment please refer to Chapter 10 – Instrumentation.

5.4 Calibration and Equipment Testing

CIBrE will be responsible for testing and calibrating all the equipment acquired for the project. University employees will make sure that all gauges are working properly before they are installed in the field. Monitoring equipment should be tested and calibrated in accordance with the manufacturers' recommended methods.

Once the monitoring equipment is tested and calibrated, a plan should be developed for an "in-field" test to be conducted under conditions as close as possible to the actual monitoring set-up on the bridge. This test will help CIBrE determine whether the desired set-up is adequate for the task of gathering data.

5.5 Data Collection and Management System

The data collection and management system will consist of all necessary equipment to collect, store, and report the data, including data acquisition systems, gauge wire, communication cable, and central computers. The system should be able to operate without CIBrE interference for long periods of time. The main output of the system should be in graphical form and very easy for the user (i.e. DelDOT) to read and comprehend.

The data collection and management system should be tested in conjunction with the testing of the monitoring equipment. A complete system of gauges, wires, communication cable, data acquisition systems, central computers, and wireless communication should be set up and tested before any gauges are placed in the field.

A timeline should also be developed once the final construction schedule has been established by the contractor. This timeline should detail the required testing and the time at which testing should take place. This timeline will assist in the overall management of the project and is a must for the preliminary work stage of the project.

5.6 Real-Time Website

To effectively communicate the gathered data, a website that is capable of displaying the data will need to be created. The website designer should communicate

with all involved parties (DelDOT, CIBrE, and the contractor) to determine what material will be included on the website and how the website is to look.

The website will consist of all the gathered data (i.e. strains, temperature, pressures, displacements, video, wind, etc.) in graphical form. The site will also include pictures of the bridge gauge locations. Webcams are also recommended as a feature of the site.

Users will be able to view archived data, load test data, construction documentation, and inspection reports. This data will aid in management of the bridge throughout its service life.

The primary website will be accessible to certain parties (DelDOT, CIBrE, and contractor). It should be password protected and a secure site. It is also believed that the data gathered can be used to educate the public about the Indian River Inlet Bridge. A public link could be added to the site showing basic real-time bridge behavior and other interesting information about the Indian River Inlet Bridge.

CHAPTER 6

STAGE II – MECHANICALLY STABILIZED EARTH WALL CONSTRUCTION

6.1 Overview

The mechanically stabilized earth walls being constructed as a part of the Indian River Inlet Bridge project will be very high, and settlement is expected to be significant. Therefore, the University of Delaware will install stain gauges on specific reinforcement panels to monitor the performance of the mechanically stabilized earth walls both during construction and over the intended service life. A detailed plan and schedule have yet to be developed for this phase of the project. Once a plan has been developed, the University will be in contact with DelDOT and the contractor to ensure the success of the roadway monitoring project.

For more information regarding the roadway monitoring program please see Appendix C – University of Delaware Roadway Monitoring Program Special Provisions.

CHAPTER 7

STAGE III – BRIDGE CONSTRUCTION

7.1 Introduction

During Stage III the instrumentation equipment will be installed and data collection initiated. This chapter first describes the monitoring instrumentation in detail. Plans are laid out for the locations of all instrumentation equipment and required hardwire connections. Additionally, a phased installation schedule, based primarily on the construction schedule, is developed and details are given regarding each instrumented location.

7.2 Instrumentation Program Description

To accomplish the objectives of the monitoring program, more than 240 permanent gauges, eleven data acquisition systems, 39 data loggers, and hundreds of feet of gauge wire and communication cable will be installed during the construction of the bridge. This instrumentation will allow CIBrE to effectively collect and store all the monitoring data necessary to quantify bridge behavior and allow DelDOT to efficiently manage the monumental structure.

Given the extent of the resources being devoted to the monitoring program, careful attention has been given to ensuring that the instrumentation installed will efficiently capture the behavior of the bridge during construction and its intended service life. The bridge behavior that will be captured by the monitoring program can be categorized into two types: event data and monitor data. Event data, defined as

data that is gathered at high frequencies, is designed to document dynamic effects such as live load due to traffic and wind loading. Monitor data, defined as data that is gathered at low frequencies, is designed to document daily or seasonal effects such a thermal loads and settlement of the structure.

To collect all necessary event and monitor data, gauge locations and types have been selected to ensure that critical bridge behavior is quantified with a minimal number of gages. As a result, numerous types of gauges will be installed on and around the bridge, including vibrating wire gauges, weldable foil strain gauges, accelerometers, global positioning systems, load cells, linear potentiometers, chloride penetrations monitors, void detection devices, nitrogen monitoring gauges, a permanent weather station, and temporary anemometers. (For a detailed description of each gauge, its intended purpose, the manufacturer, the installation procedure, scheduling issues for each gauge and other information please refer to Chapter 10 – Instrumentation).

7.2.1 Permanent Instrumentation

Permanent instrumentation is defined as equipment (gauges and data acquisition systems) that will be installed during construction and will remain on the bridge for the duration of the monitoring program. A total of 25 locations on the bridge will be instrumented. The chosen instrumented sections include south abutment, north abutment, south arch base, south pile cap, arch-rib, tie-beam, central beam, support load cables (installed internally to the arch-rib), southbound roadway sections, northbound roadway sections, and post-tensioning tendon ducts. All locations will be instrumented with gauges that will be used to collect data on bridge response. Additionally, several locations will be instrumented with data acquisition

systems for the purpose of collecting, storing and communicating the gathered real-time data.

7.2.1.1 Permanent Gauge Location

As stated before, permanent gauges will be installed at 25 locations on the bridge. Figures 7.1 and 7.2 detail the locations of permanent gauges. Furthermore, Table 7.1 defines the planned permanent gauge locations by describing the locations that will be instrumented and the gauge types that will be installed at each location. Table 7.5, located at the end of Section 7.2, describes the abbreviations used in the tables located in Section 7.2.[1]

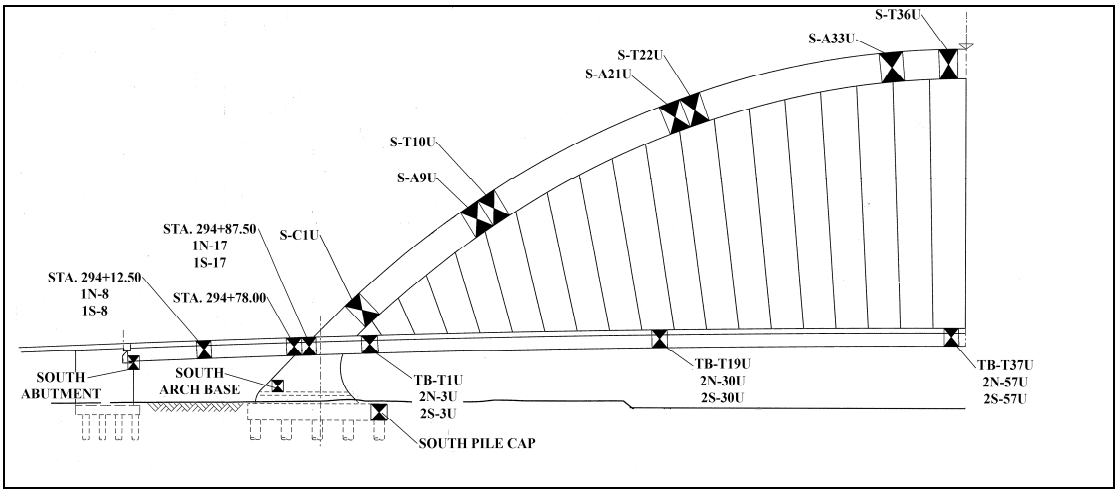

Figure 7.1 South Permanent Gauge Locations (Modified IRIB Specifications and Construction Plans)

[1] The labeling scheme that was used to identify instrumented segments (i.e. arch segment S-A9U) was developed from the IRIB Specifications and Construction Plans. All Figures and Tables throughout the manual use this labeling scheme. Appendix D provides a complete list of design drawings used in the development of the monitoring program. The reference design drawings will provide the complete labeling scheme for all bridge segments.

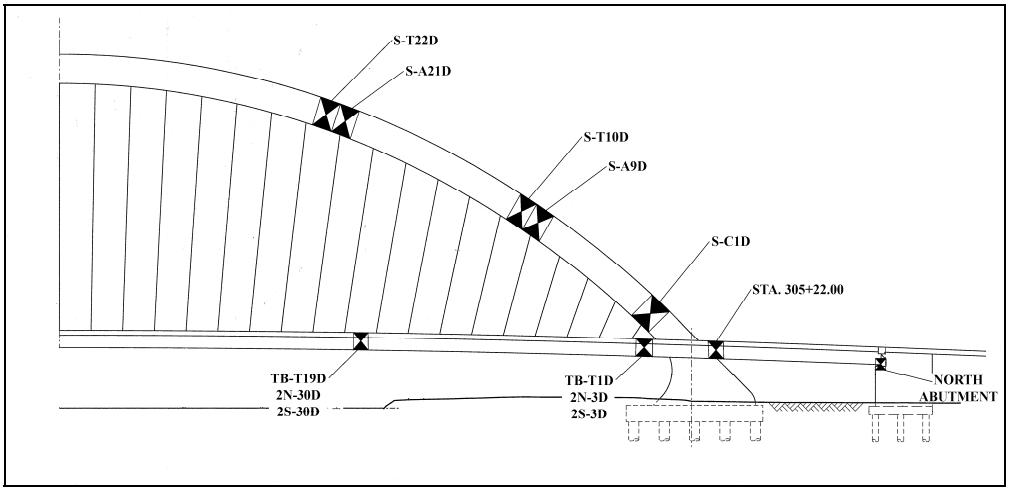

Figure 7.2 North Permanent Gauge Locations (Modified IRIB Specifications and Construction Plans)

Table 7.1 Permanent Gauge Locations and Associated Gauges

Location	Instrumentation
Pile Cap	
South Pile Cap	4 CM
Abutment	
North Abutment	2 LP
South Abutment	2 LP
Arch Base	
South Arch Base	3 LP
Prestressing Tendon Ducts	
Location to be Determined	VD
Arch-Rib Section	
S-C1U	4 VW, 4 FS, 1 ACC
S-T10U	
S-T22U	
S-T22D	
S-T10D	
S-C1D	
S-T36U	4 VW, 4 FS, 1 ACC, 1 GPS
S-A9U	1 LC, NM
S-A21U	
S-A33U	
S-A21D	
S-A9D	

Tie-Beam Section/Central Beam Station	Adjacent Roadway Sections		Instrumentation in Tie-Beam/Central Beam	Instrumentation in Roadway
TB-T1U	NB	2N-3U	4 VW, 4 FS, 1 ACC	4 VW, 4 FS, 1 ACC
	SD	2S-3U		
TB-T19U	NB	2N-30U		
	SB	2S-30U		
TB-T37U	NB	2N-57U		
	SB	2S-57U		
TB-T19D	NB	2N-30D		
	SB	2S-30D		
TB-T1D	NB	2N-3D		
	SB	2S-3D		
294+12.500	NB	1N-8		
	SB	1S-8		
294+87.500	NB	1N-17	N/A	4 VW
	SB	1S-17		
294+78.000	NB	N/A	1 GPS	N/A
	SB	N/A		
305+22.000	NB	N/A		
	SB	N/A		

7.2.1.2 Data Acquisition System Locations

For the purpose of collecting, storing, and communicating real-time data, eleven data acquisition systems will be installed. Figures 7.3 and 7.4 detail the location of data acquisition systems, and Table 7.2 details the gauge distribution to all data acquisition systems.

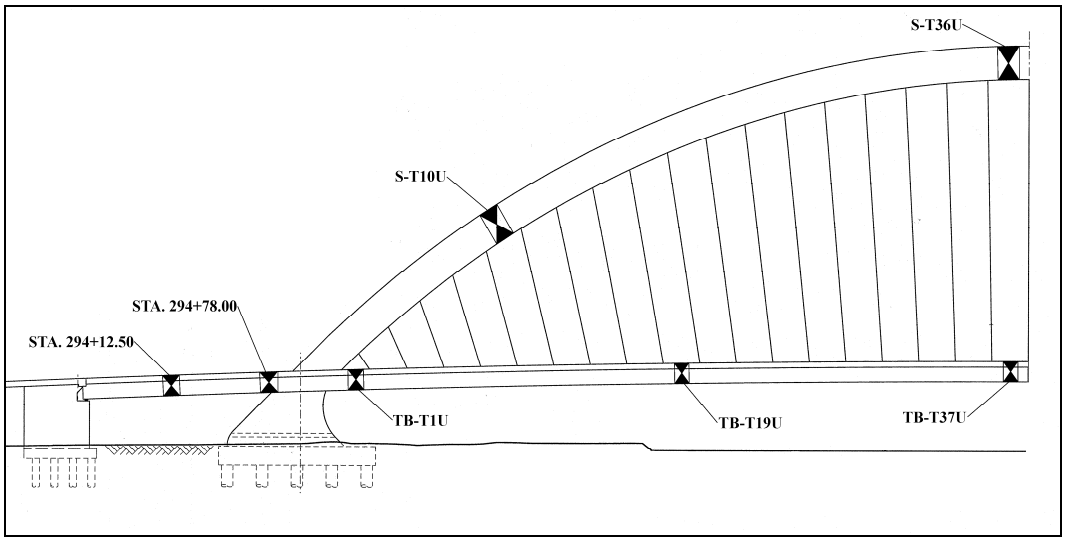

Figure 7.3 South Data Acquisition System Locations (Modified IRIB Specifications and Construction Plans)

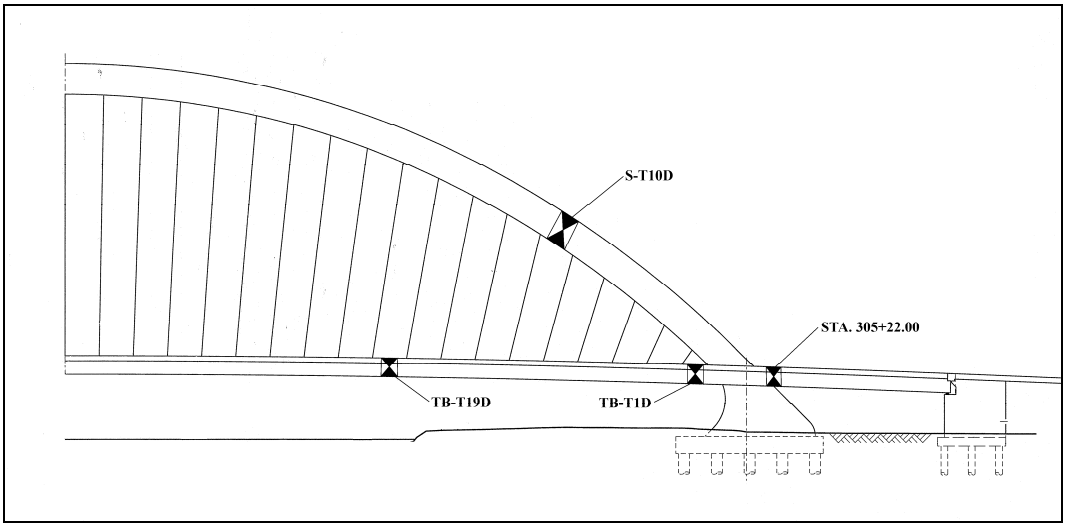

Figure 7.4 North Data Acquisition System Locations (Modified IRIB Specifications and Construction Plans)

Table 7.2 Gauge Distribution to Data Acquisition Systems

Data Acquisition Unit Location	Vibrating Wire	Weldable Foil	Accelerometer	GPS	Load Cell	Linear Potentiometer	Total
S-T10U	12	12	3	0	2	0	29
S-T36U	4	4	1	1	1	0	11
S-T10D	12	12	3	0	2	0	29
TB-T1U	20	12	3	0	0	3*	36
TB-T19U	12	12	3	0	0	0	26
TB-T37U	12	12	3	0	0	0	26
TB-T19D	12	12	3	0	0	0	26
TB-T1D	12	12	3	0	0	0	26
Central Beam 294+12.500	12	12	3	0	0	0	26
Central Beam 294+78.000	0	0	0	1	0	0	1
Central Beam 305+22.000	0	0	0	1	0	0	1
Total Number of Gages	108	100	19	3	5	2	237
* Final sensor routing to be determined by the contractor							

7.2.2 Temporary Instrumentation

Temporary instrumentation is defined as equipment (gauges and data loggers) that will be installed during the construction of the bridge but will not remain on the bridge during its intended service life. All permanent instrumented locations

will be temporarily instrumented with data logger systems in order to collect real-time monitoring data during construction. Also, the south temporary construction tower and its temporary stays will be instrumented with temporary gauges to quantify wind behavior at the site and forces in the temporary stays

7.2.2.1 Temporary Gauge Locations

As stated before, temporary gauges (anemometers and load cells) will be installed on the south temporary construction tower and on the temporary stays. Currently the design of the temporary towers has not been completed. As a result, the CIBrE team has not been able to finalize its intentions with respect to the temporary gauges (anemometers and load cell) and their respective locations. Once the final tower design is complete and the intended construction sequence is finalized, CIBrE will be able to define the exact number and location of the temporary gauges and provide details about how the gauges will be installed. Please see Sections 3.4.1 and 3.4.2 for more details regarding the responsibilities CIBrE and the contractor with regard to the temporary construction towers.

7.2.2.2 Data Logger System Locations

To capture real-time monitoring data at the beginning stages of construction, data logger systems will need to be installed at every permanent instrumentation location (see Figures 7.5 and 7.6). In Figure 7.7 the location of the base station is described. The base station will be installed to collect real-time monitoring data from the gauges located on the temporary tower and temporary stays. Table 7.3 details the gauge distribution to all temporary data logger systems. It is

intended that the temporary data acquisition systems will run off battery power for the length of time that they are needed.

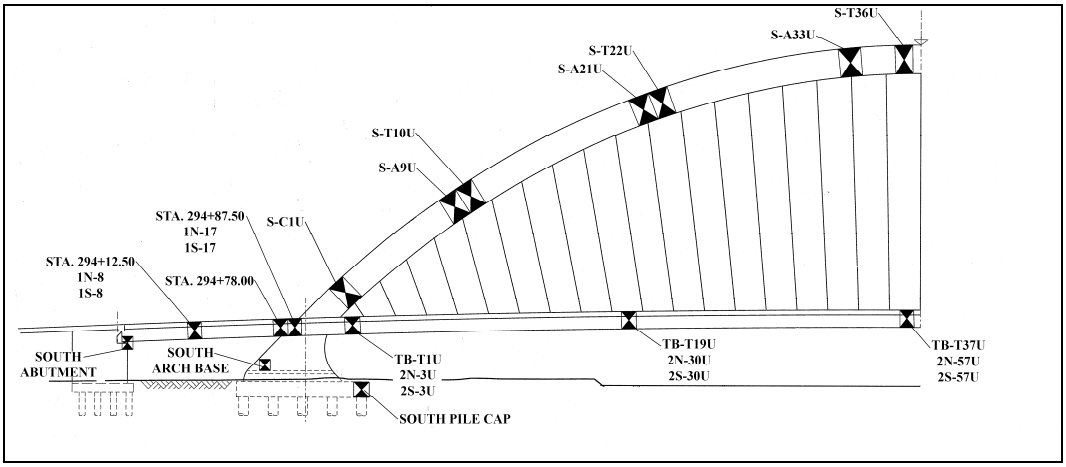

Figure 7.5 South Data Logger System Locations (Modified IRIB Specifications and Construction Plans)

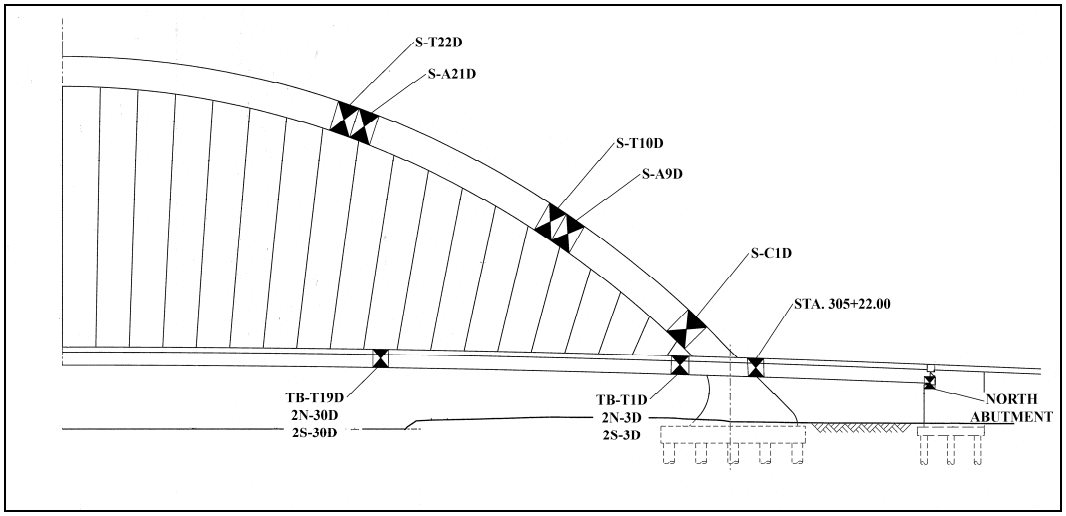

Figure 7.6 North Data Logger Locations (Modified IRIB Specifications and Construction Plans)

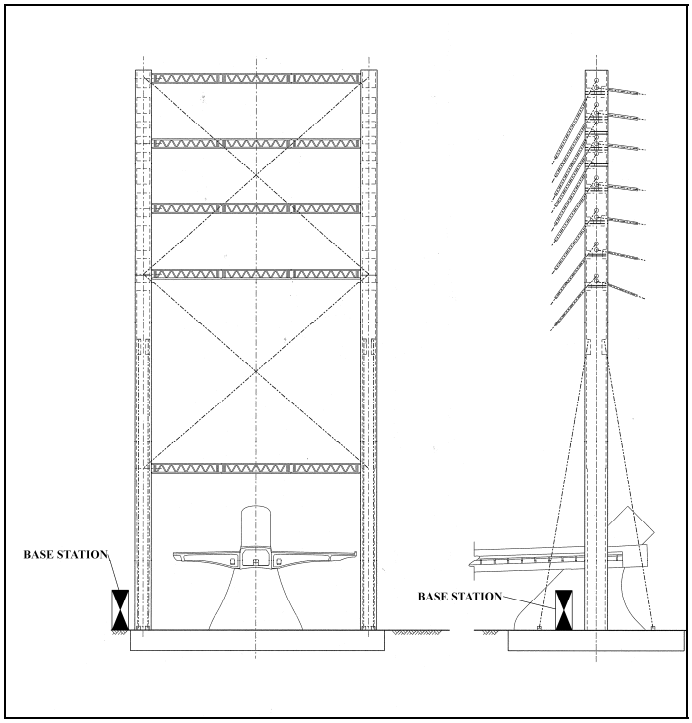

Figure 7.7 Base Station Location (Modified IRIB Specifications and Construction Plans)

Table 7.3 Gauge Distribution for Data Logger Systems

Data Logger Unit Location	Gauge Type	
Arch Base	**Linear Potentiometer**	
South Arch Base	3	
Abutment	**Linear Potentiometer**	
South Abutment	2	
North Abutment	2	
Arch-Rib Section	**Vibrating Wire**	**Load Cell**
S-C1U	4	0
S-A9U	0	1
S-T10U	4	0
S-A21U	0	1
S-T22U	4	0
S-A33U	0	1
S-T36U	4	0
S-T22D	4	0
S-A21D	0	1
S-T10D	4	0
S-A9D	0	1
S-C1D	4	0
Central Beam Station		
294+12.50	4	
Tie-Beam Sections		
TB-T1U	4	
TB-T19U	4	
TB-T37U	4	
TB-T19D	4	
TB-T1D	4	

Data Logger Unit Location	Gauge Type	
Roadway Sections	**Vibrating Wire**	
1N-8	4	
1S-8	4	
1N-17	4	
1S-17	4	
2N-3U	4	
2S-3U	4	
2N-30U	4	
2S-30U	4	
2N-57U	4	
2S-57U	4	
2N-30D	4	
2S-30D	4	
2N-3D	4	
2S-3D	4	
Temporary Tower	**Anemometer**	**Load Cell**
Base Station	XXX	XXX
Global Positioning System Locations		
S-T36U		
294+78.00		
305+22.00		

Total Temporary System Requirements		
Number of Locations	39	
Number of Gauges	#VALUE!	

7.2.3 Hardwire Connections

To communicate the gathered data to a central computing station (UD communication enclosure located just west of the south abutment), hardwire connections must be made between instrumented locations. There are two types of connections that will need to be made, gauge wire connections and communication cable connections. The following sections define and describe the required hardwire connections.

7.2.3.1 Gauge Wire Connections

Gauge wire connections are defined as connections necessary between locations of instrumentation without a data acquisition system (e.g., Arch-Rib Section S-C1U) and locations of instrumentation with a data acquisition system (e.g., Arch-Rib Section S-T10U) so that real-time monitoring data can be collected and stored. Figure 7.8 details the required gauge wire routing.

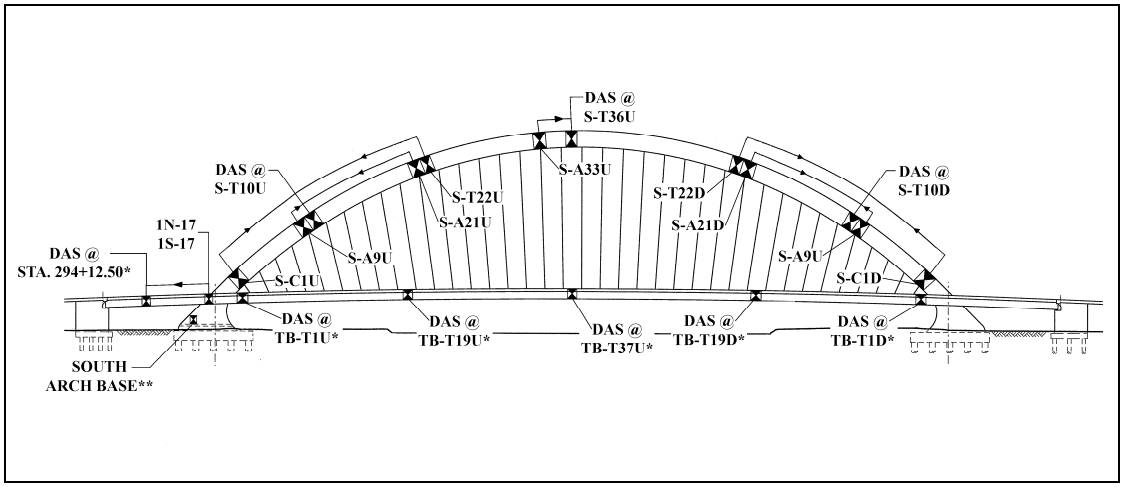

Figure 7.8 Gauge Wire Routing Schematic (elevation view) (Modified IRIB Specifications and Construction Plans)

Three types of gauge wire routing will be necessary. The first type is routing from one location to another via "long" wire runs. This is indicated by the black solid lines in Figure 7.8. There are nine such wire runs in the arch-rib section and one in the south back span. These "long" wire runs will be installed in the local conduit provided by the contractor and specified in the bridge plans. Secondly, gauge wire runs will need to be made from adjacent roadway sections to tie-beam locations. In Figure 7.8 an asterisk (*) denotes this type of connection. These connections will

be made via through-holes located in both the adjacent roadway and tie-beam sections. Figure 7.9 depicts the through holes and required routing of the gauge wires.

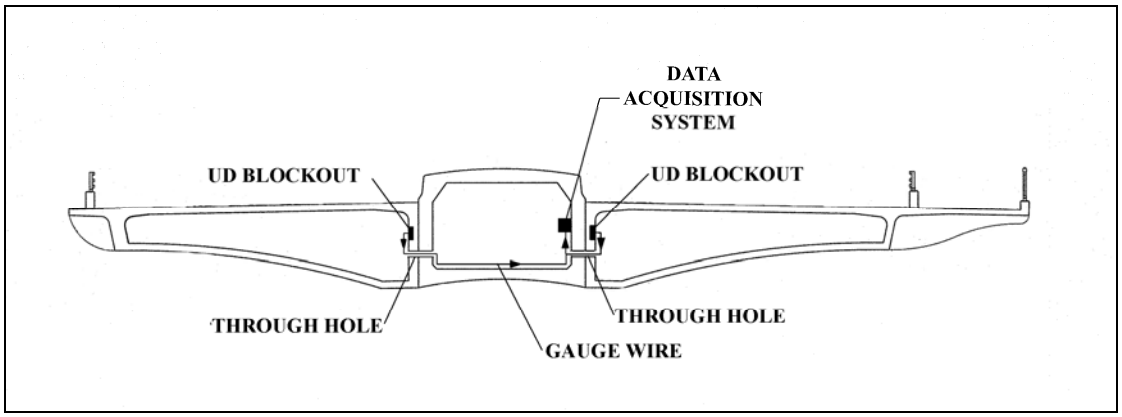

Figure 7.9 Typical Roadway Section to Tie-Beam Gauge Wire Connection (Modified IRIB Specifications and Construction Plans)

Finally, gauge wire will need to be run from the south arch base to a data acquisition location. In Figure 7.8 these locations are denoted with a double asterisk (**). It is up to the contractor to decide how these gauge wires will be routed. The contractor has two options. The first option is to route the wires up through the arch base and into the tie-beam sections (as specified in the bridge plans). The second option is to route the wire underground directly to the UD communication enclose located nearby. CIBrE will work closely with the contractor to determine the best course of action.

Table 7.4 further describes the gauge wire connections by giving a detailed description of the routing schematic, the required distances, and the number of gauges that will be needed to be routed.

Table 7.4 Sensor Wire Routing Connection Details

Data Acqusition System Location	Connecting Segment	Number of Gauges in Connecting Segment*	Distance Between Segments (Center to Center) (ft)
S-T10U	S-C1U	9	131.4
	S-A9U	1	14.6
	S-A21U	1	160.6
	S-T22U	9	175.2
S-T36U	S-A33U	1	43.8
S-10D	S-C1D	9	131.4
	S-A9D	1	14.6
	S-A21D	1	160.6
	S-T22D	9	175.2
TB-T1U	2N-3U	9	Through Hole Access
	2S-3U	9	Through Hole Access
TB-T19U	2N-30U	9	Through Hole Access
	2S-30U	9	Through Hole Access
TB-T37U	2N-57U	9	Through Hole Access
	2S-57U	9	Through Hole Access
TB-T19D	2N-30D	9	Through Hole Access
	2S-30D	9	Through Hole Access
TB-T1D	2N-3D	9	Through Hole Access
	2S-3D	9	Through Hole Access
STA. 294+12.50	1N-8	9	Through Hole Access
	1S-8	9	Through Hole Access
	1N-17	9	75.0
	1S-17	9	75.0
* Number of gauges does not include gauges located in the section with the data acquisition system			

7.2.3.2 Communication Cable Connections

Communication cable connections are defined as connections necessary to transfer collected data from a data acquisition system to the UD communication enclosure located near the south abutment of the bridge. Communication cable will be installed after construction is complete. The cable will be installed in the communication conduit provided by the contactor and detailed in the bridge plans. Figure 7.10 details the communication routing.

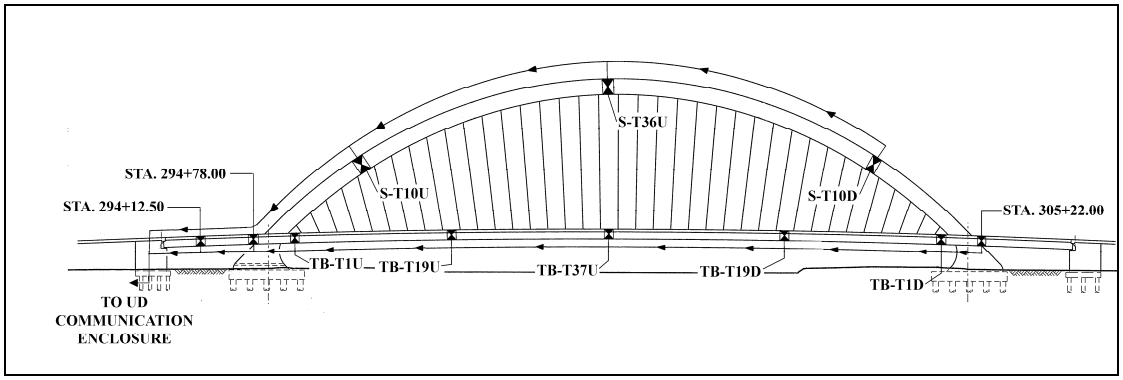

Figure 7.10 Communication Cable Routing Schematic (elevation view) (Modified IRIB Specifications and Construction Plans)

Table 7.5 Legend Abbreviations

Sensor	Abbreviation
Vibrating Wire Gages	VW
Weldable Foil Strain Gages	FS
Accelerometers	ACC
Global Position System	GPS
Load Cells	LC
Nitrogen Monitoring System	NM
Anemometers	A
Linear Potentiometers	LP
Corrosion Monitoring	CM
Void Decection Device	VD

7.3 Phased Implementation of Monitoring Equipment

The task of implementing the monitoring program consists of all activities relating to the installation, activation, and initial data collection of the instrumentation equipment. Additionally, during construction of the bridge CIBrE will monitor the environmental conditions at the site and will work closely with the DelDOT Materials Division to create a database of material properties. The data collected by the

monitoring equipment will enable CIBrE to track both construction loads and the evolution of dead load stresses throughout the structure.

It is intended that as the bridge is being constructed all permanent instrumentation equipment will be installed as soon as possible. This means that both the construction schedule and the installation of the instrumentation equipment schedule will be very similar. As a result, there is a need for continual contact with the chosen contractor to ensure that the instrumentation equipment gets installed in the proper locations and at the correct time.

7.3.1 General Construction Sequence

A schedule for the installation of the monitoring equipment has been developed based on the proposed construction schedule detailed in the final bridge plans (Figg Engineering Group). With regard to the monitoring program, the proposed construction schedule can be broken down into two parts: preliminary work and the general construction sequence. During preliminary work the deep foundation will be constructed and the south arch base pile cap will be instrumented. During the general construction sequence, the majority of the instrumentation will be installed. For the purpose of easily identifying when each gauge will be installed, the general construction sequence has been broken down into six distinct steps, as shown in Figures 7.11 through 7.16. During Step 1 (Figure 7.11) of construction, arch bases, temporary towers, the cast-in-place central beam, and both northbound and southbound roadway segments are erected. In Step 2 (Figure 7.12), the arch-rib is constructed and temporary support cables are installed to support the arch. Next, in Step 3 (Figure 7.13) a closure segment is cast in the arch-rib and the arch-rib is complete. During Step 4 (Figure 7.14) the tie-beam segments are constructed, along

with associated support cables. Step 5 (Figure 7.15) depicts the closure segment casting of the tie-beam. Finally in Step 6 (Figure 7.16) all northbound and southbound roadway segments are erected, temporary supports are removed and the bridge is complete. A more detailed construction schedule will be provided by the contractor, and a review of this schedule is necessary to ensure that proper installation of all monitoring equipment can take place.

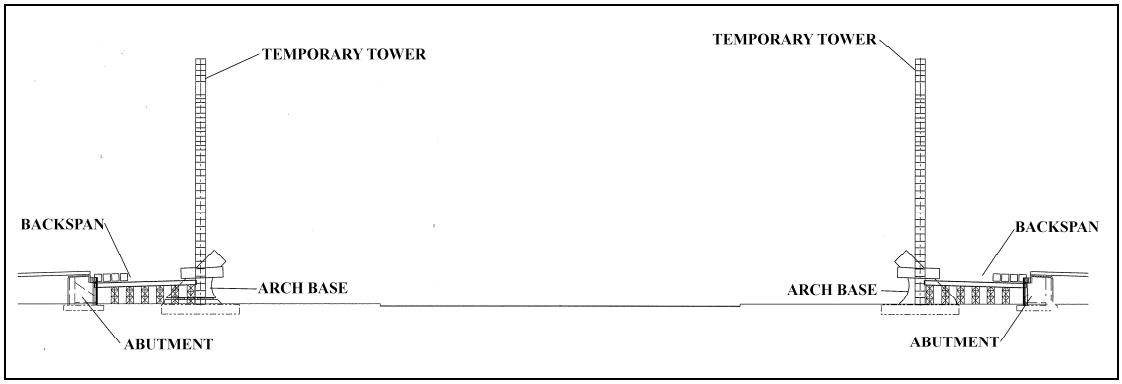

Figure 7.11 Step 1-Construct Approach Spans (Modified IRIB Specifications and Construction Plans)

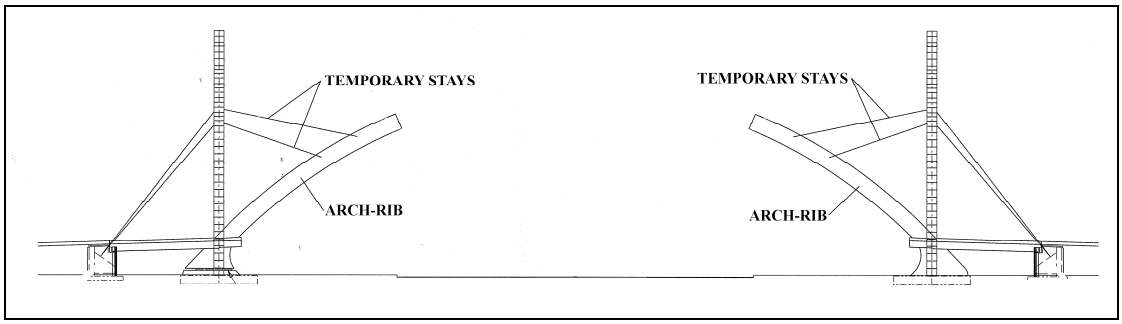

Figure 7.12 Step 2-Construct Arch (Modified IRIB Specifications and Construction Plans)

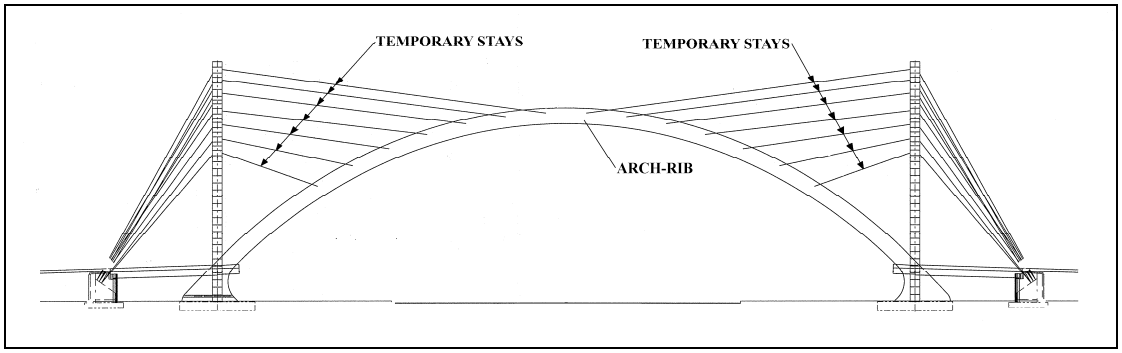

Figure 7.13 Step 3-Complete Arch (Modified IRIB Specifications and Construction Plans)

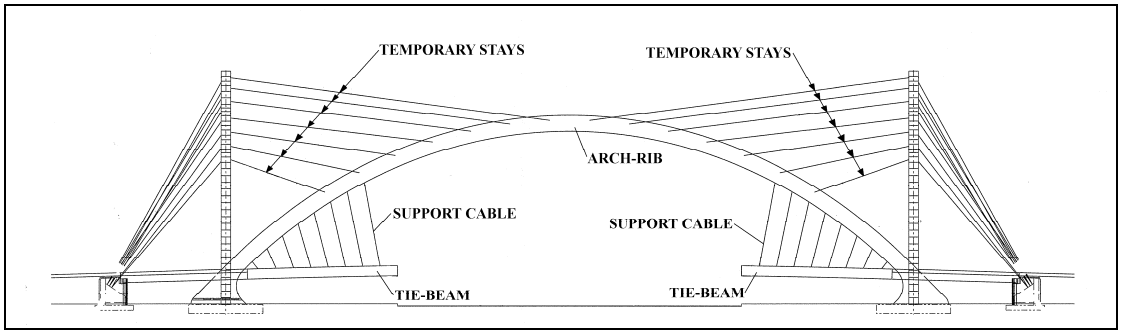

Figure 7.14 Step 4-Construct Tie-Beam (Modified IRIB Specifications and Construction Plans)

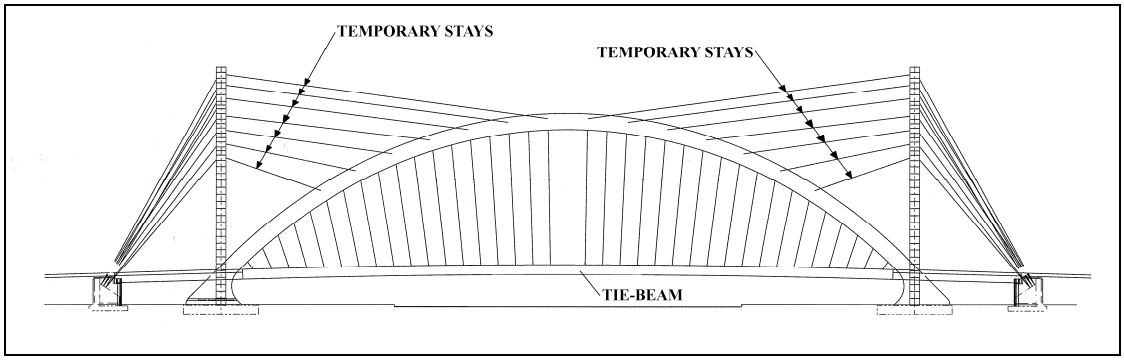

Figure 7.15 Step 5-Complete the Tie-Beam (Modified IRIB Specifications and Construction Plans)

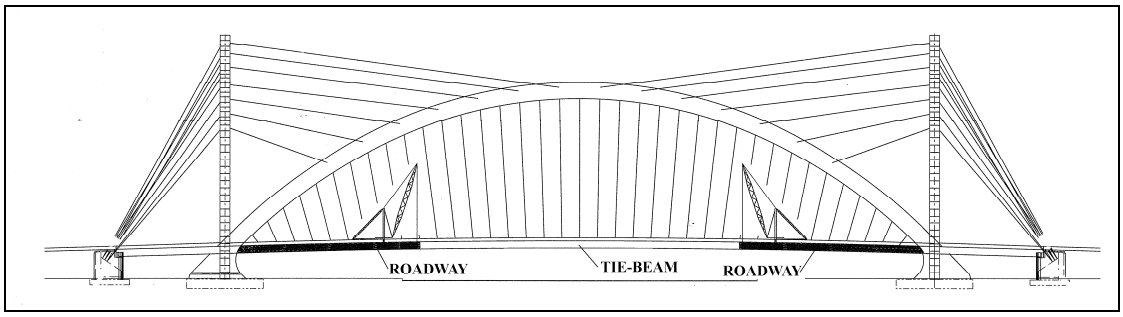

Figure 7.16 Step-6 Erect Main Span Deck (Modified IRIB Specifications and Construction Plans)

7.3.2 General Instrumentation Schedule

Once the proposed construction sequence was determined, a detailed schedule was laid out for the installation of all instrumentation equipment. Based on the construction schedule the monitoring equipment installation can be broken down into seven different phases. Each phase corresponds to a portion of the bridge that will be instrumented with equipment. Table 7.6 describes the specific section of the bridge that will be instrumented in each phase.

Table 7.6 Instrumented Locations

Instrumentation Phase	Instrumented Location
Phase 1	South Arch Base Pile Cap
Phase 2	South Arch Base
Phase 3	South Temporary Tower
	Temporary Stays
Phase 4	Abutment Bearings
	Central Beam
	Roadway
Phase 5	Arch-Rib
Phase 6	Tie-Beam
Phase 7	Roadway

Due to the uncertainty of the construction sequence, it has been possible only to generate idealized flowcharts of the events that will take place during implementation of the monitoring equipment. Figure 7.17 generally describes when and what types of instrumentation will be installed. No overall timelines have been set for the project.

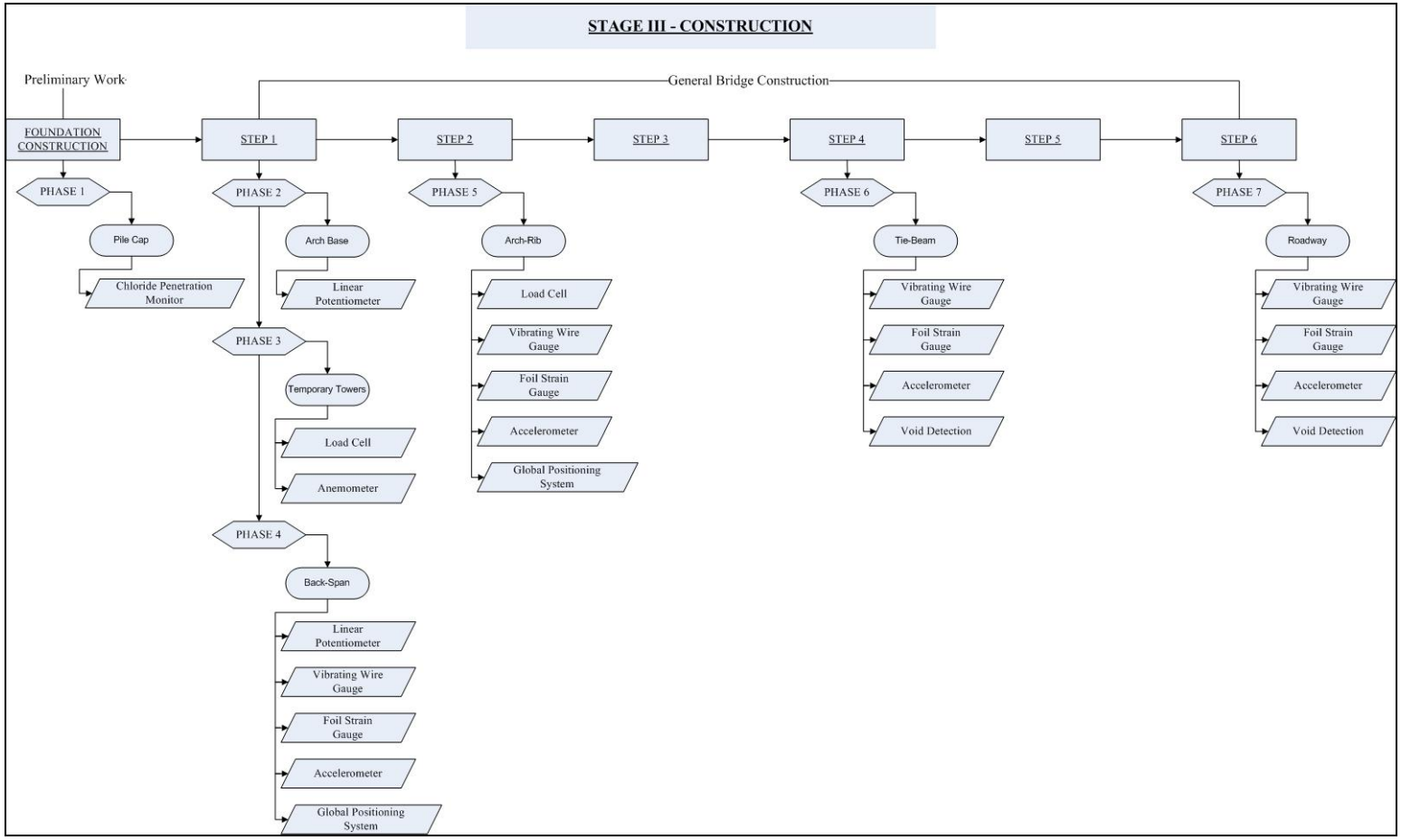

Figure 7.17 Monitoring Equipment Installation Schedule

The following sections outline the proposed monitoring instrumentation in more detail.

7.4 Phase 1 - Pile Cap Instrumentation

Before Step 1 of the general construction sequence (Section 7.3.1), the foundation for the bridge will be constructed. CIBrE plans to instrument the south pile cap with chloride penetration monitors. The following section outlines CIBrE intentions for the instrumentation of the south pile cap.

7.4.1 South Pile Cap

To monitor corrosion of the pile cap, chloride penetration monitors will be installed in the South Arch Pile Cap. It is intended that four units will be installed in the foundation cap, one at each corner of the pile cap. Currently there are two options for the foundation, drilled shafts and driven steel pipe piles. It is up to the contractor to decide which option to use. Figures 7.18 and 7.19 detail the proposed location of the chloride penetration monitors. Exact locations of the monitoring equipment will need to be discussed with the contractor and a final location will need to be decided upon. Also, exact locations of the "block-out" will need to be determined.

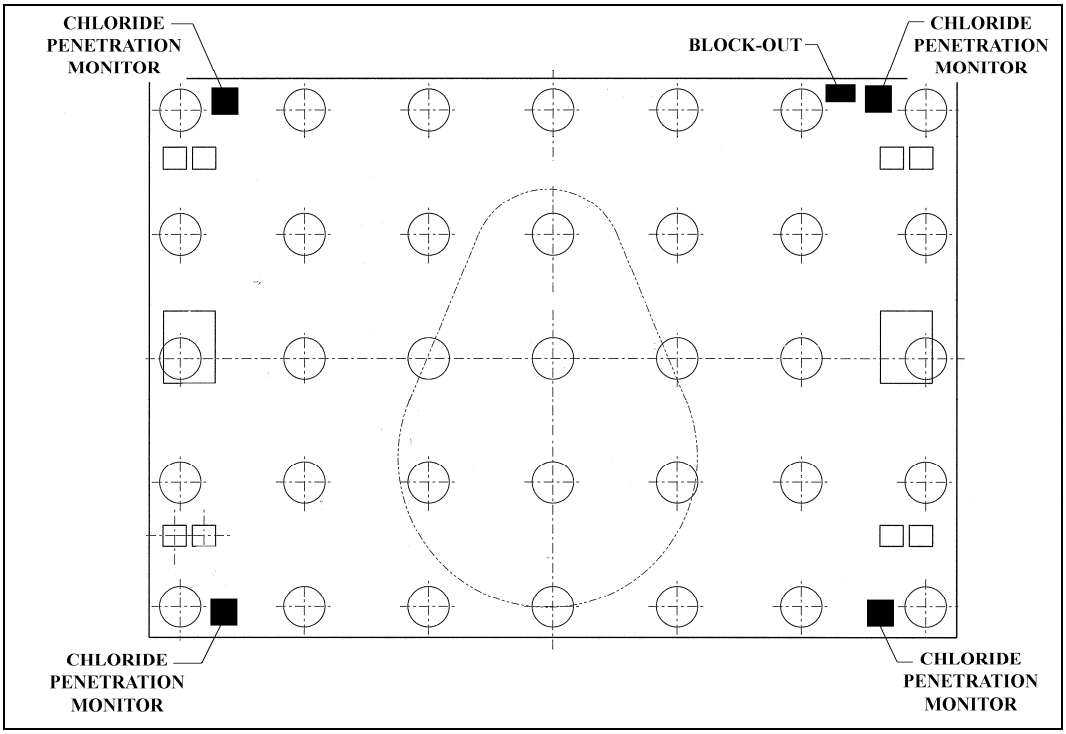

Figure 7.18 Chloride Penetration Monitor Locations (Plan View) (Modified IRIB Specifications and Construction Plans)

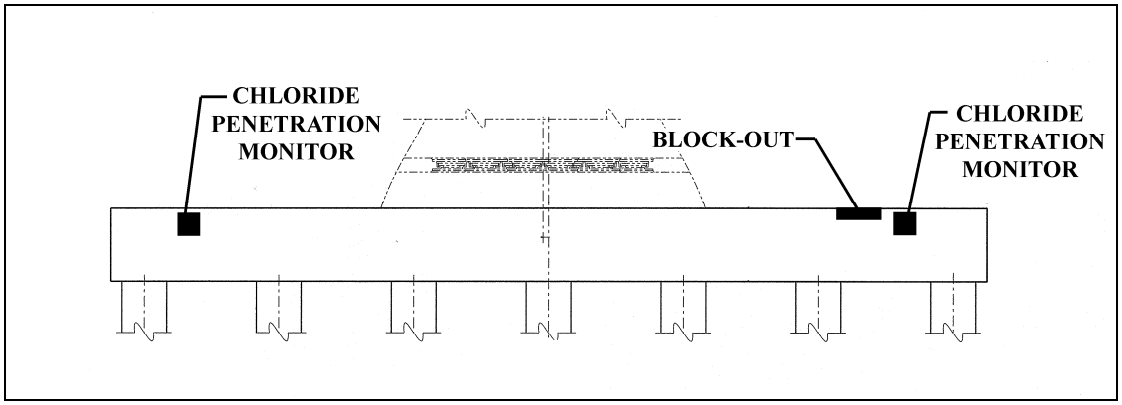

Figure 7.19 Chloride Penetration Monitor Locations (Elevation View) (Modified IRIB Specifications and Construction Plans)

The design of the chloride penetration monitoring equipment has not yet been finalized. It is unclear where the gauge wires will be routed to tie into the monitoring system and how often data will need to be gathered. When more details are obtained about the chloride penetration monitors and the pile cap, CIBrE will provide the contractor and DelDOT with the required information to ensure the proper installation of the gauges and collection of data.

Further installation details and procedure for the chloride penetration monitors can be found in Chapter 10 – Instrumentation.

7.5 Phase 2 - Arch Base Instrumentation

During Step 1 of the general construction sequence (Section 7.3.1), the arch bases will be erected. CIBrE plans to instrument the south arch base with linear potentiometers. The following section outlines CIBrE intentions for the instrumentation of the south arch base.

7.5.1 South Arch Base

To of quantify linear displacement of the bridge due to live load effects, linear potentiometers will be installed in the South Arch Base. It is intended that three bearings will be instrumented with linear potentiometers. Figures 7.20 and 7.21 detail the locations of the three linear potentiometers. Exact locations of the monitoring equipment will need to be discussed with the contractor.

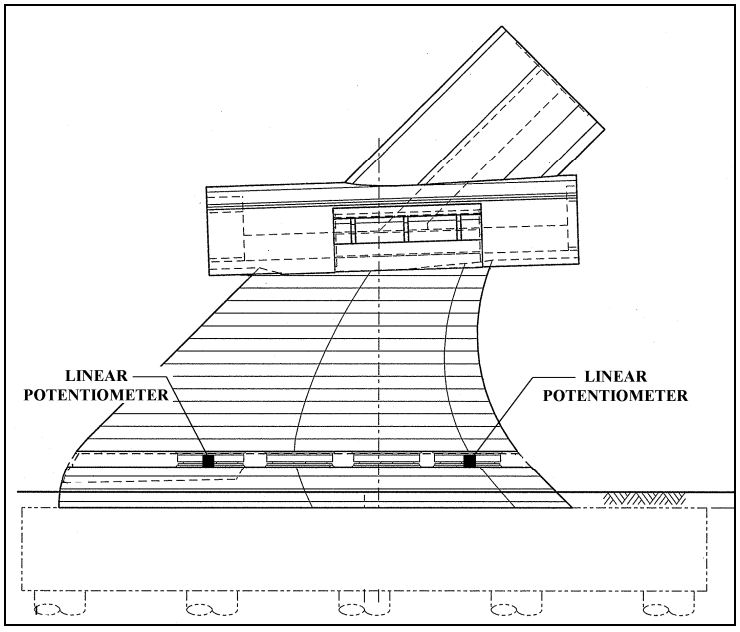

Figure 7.20 Linear Potentiometer Locations (Elevation View) (Modified IRIB Specifications and Construction Plans)

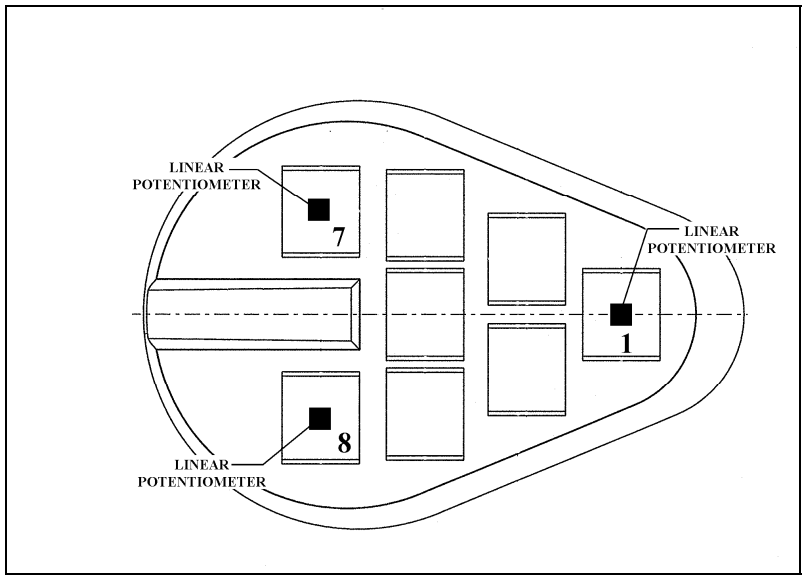

Figure 7.21 Linear Potentiometer Locations (Plan View) (Modified IRIB Specifications and Construction Plans)

The linear potentiometers will be installed as the south arch base is being constructed. It is anticipated that as soon as the bearings are installed, the linear potentiometers will be placed. The linear potentiometers will be installed in such a manner as to allow for data to be gathered as the arch base is being finished, during verification of the bearings; ability to displace, during the bearing blocking procedures, and after the bearings are unblocked at the end of construction.

Data collection for the linear potentiometers will be in two phases. The first phase will be temporary data collection using a data logger system. This system will be installed at the same time the bearing instrumentation is installed and is intended for use throughout construction. Phase two of data collection is permanent data collection using one of two methods. The contractor will determine hoe to hardwire the linear potentiometers into the monitoring system and notify the University as soon as possible after making a decision. One option is to use conduit installed in the arch base (see bridge plans) to route gauge wire to a data acquisition system located in the tie-beam. The second option is to provide access for the gauge wires to be routed directly to UD communication enclosure. Figure 7.22 details the installation scheduled for the three linear potentiometers and describes the intended data acquisition system requirements and sequencing.

Further installation details and procedure for the linear potentiometers can be found in Chapter 10 - Instrumentation.

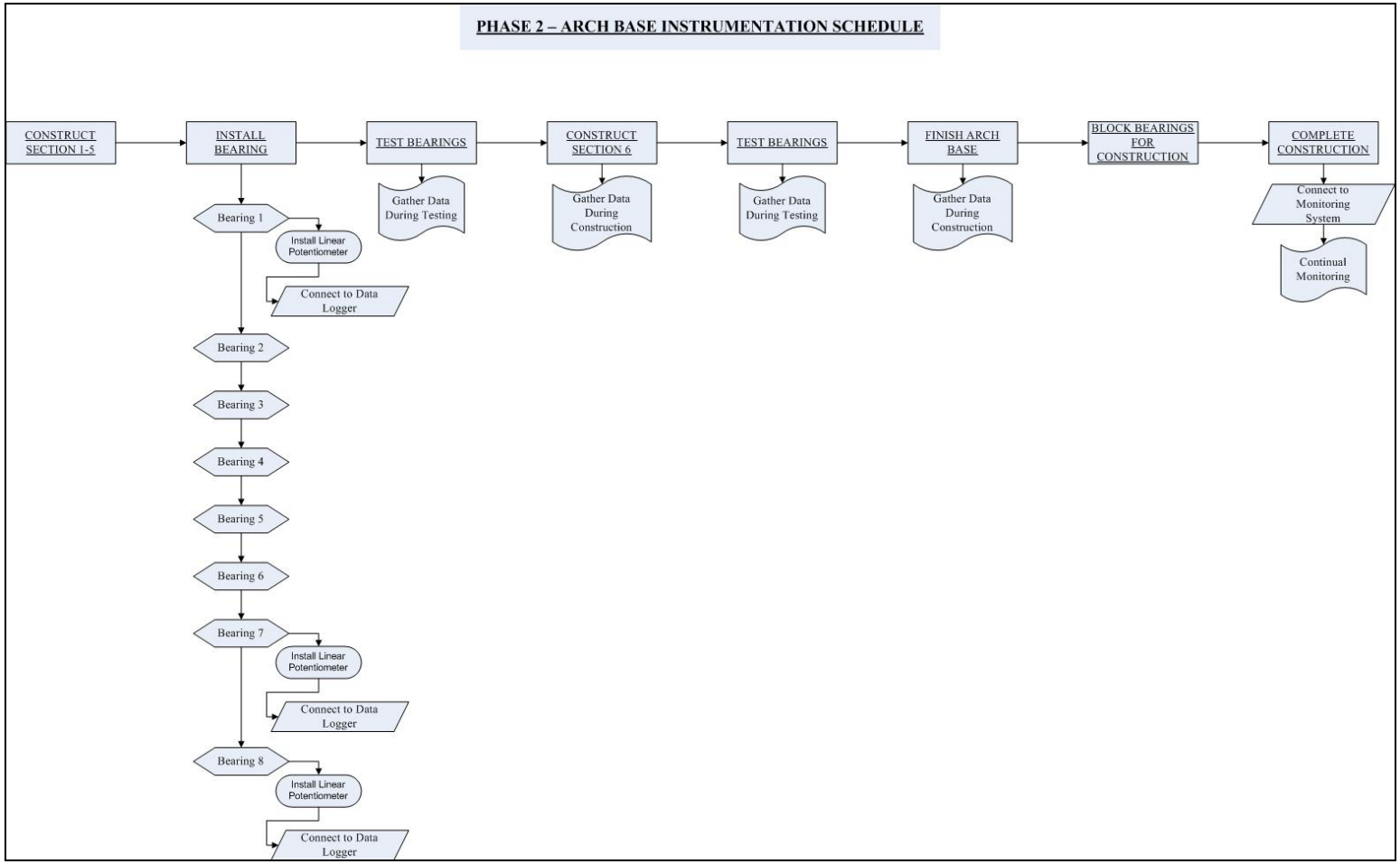

Figure 7.22 Linear Potentiometer Installation Schedule

7.6 Phase 3 - Temporary Tower Instrumentation

As a part of Step 1 of the general construction sequence (Section 7.3.1), temporary towers will be erected. The purpose of the temporary towers is to provide support for the arch-rib as it is being constructed. CIBrE plans to use the temporary towers to mount instrumentation to and intends to instrument several temporary stays. The following sections outline CIBrE intentions for the instrumentation of the temporary towers and temporary stays.

7.6.1 South Temporary Tower

To quantify site-specific wind behavior, CIBrE will instrument the south temporary construction tower with anemometers. The anemometers will be placed at five different heights to help determine a wind velocity profile for the proposed site. CIBrE has not yet determined the final location of the anemometers.

It is intended that as the tower is constructed segmentally during Step 1 of the general construction sequence described in Section 7.3.1, the anemometers will be installed. All hardwire connections for the anemometers will be made by routing the sensor wires down the legs of the tower to the base station located near the base of the tower. The exact location of this base station has not yet been determined.

CIBrE and the contractor will work together to determine a suitable location for the anemometers and the base station enclosure based on the provided final design of the temporary towers.

Further installation details and procedure for anemometers can be found in Chapter 10 – Instrumentation.

7.6.2 Temporary Stays

To monitor the forces in the temporary support cables, CIBrE intends to instrument several temporary stays with load cells. The information gathered from the load cells will help CIBrE track the evolution of stresses in the arch as it is being constructed. The data will also help quantify the dead load stress before the bridge is open to traffic. CIBrE has not yet finalized the details regarding instrumentation of the temporary stays but will do so upon receiving the final design for the temporary tower. CIBrE will work with the contractor to determine which temporary stays will be instrumented and how the installation will be accomplished. Hardwire connections will be achieved similar to the anemometers, and all instrumentation will be connected to the base station located at the base of the temporary towers.

Further installation details and procedure for temporary load cells (and other equipment) can be found in Chapter 10 – Instrumentation.

7.7 Phase 4 - Back-Span Instrumentation

In addition to erecting the temporary towers and constructing the arch bases during Step 1 of the general construction sequence (Section 7.3.1), the back-spans will be erected. Erection of the back spans consists of erecting abutments, casting the central beam, and erecting the precast roadway sections. CIBrE intends to instrument the north and south abutment bearings and predetermined locations in the central beam and precast roadway segments. Due to the complexities of this phase of the project, a schedule was developed to organize the installation of all monitoring equipment. Figure 7.23 details this schedule.

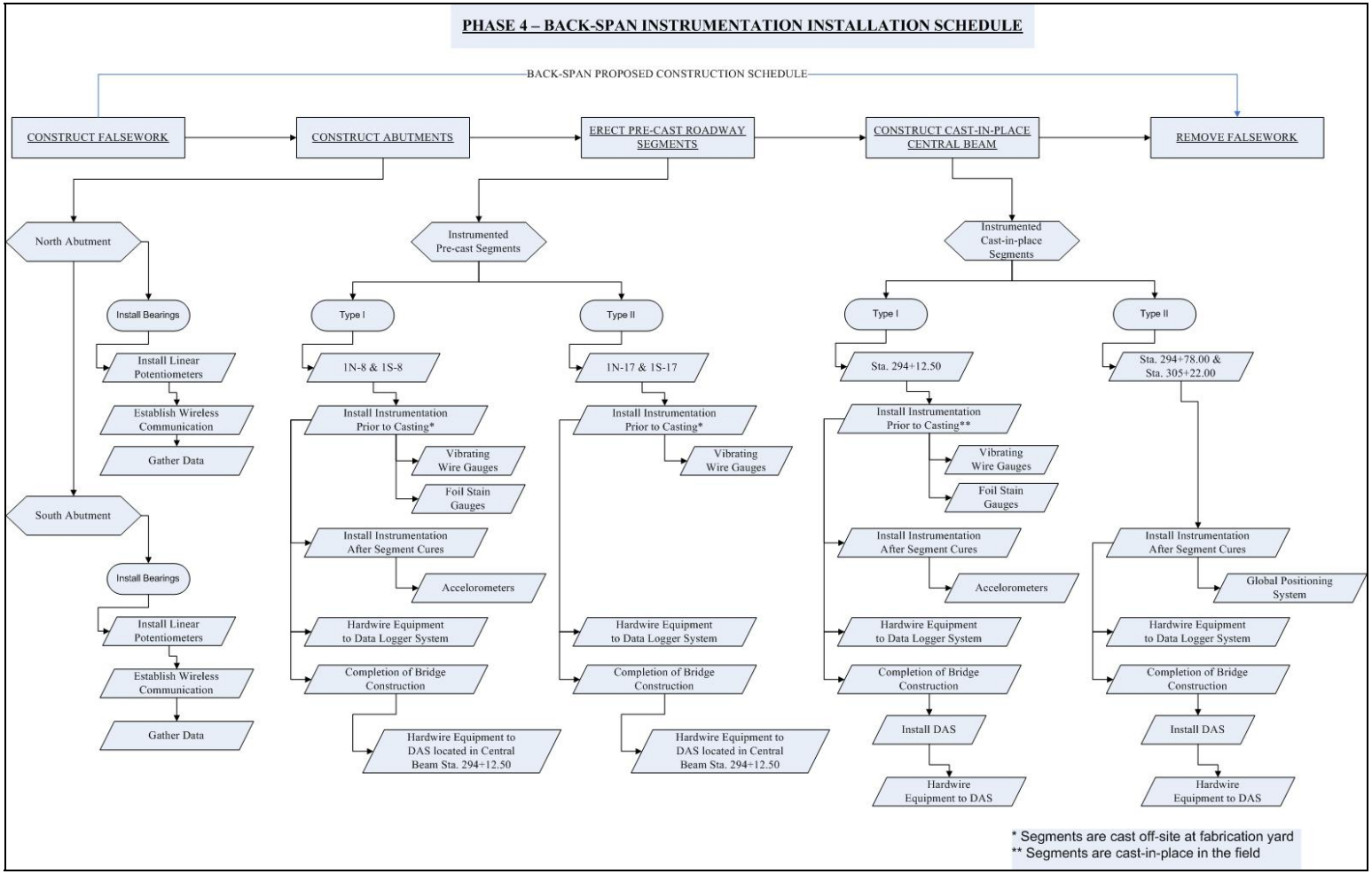

Figure 7.23 Back-Span Instrumentation Installation Schedule

7.7.1 Abutment Bearing Instrumentation

To monitor expansion and contraction of the bridge due to live load effects (thermal, wind, and ambient traffic), linear potentiometers will be installed. As described in Section 7.2.1, the north and south abutment bearings will be instrumented. Two linear potentiometers will be installed at each location. The following section outlines the intended instrumentation plan. Further details about each gauge type can be found in Chapter 10 – Instrumentation.

7.7.1.1 South and North Abutment Bearing Instrumentation

Both the north and south abutment bearings will be instrumented. There are two bearings at each location. Linear potentiometers will be installed at these locations. For the purpose of data communication, a wireless system will be installed when the linear potentiometers are installed. It is intended that no hardwiring will be necessary at these locations. CIBrE has not yet finalized a plan for the wireless communication of the collected data but will contact DelDOT and the contractor as soon as the information becomes available. Figure 7.24 details the location of the linear potentiometers.

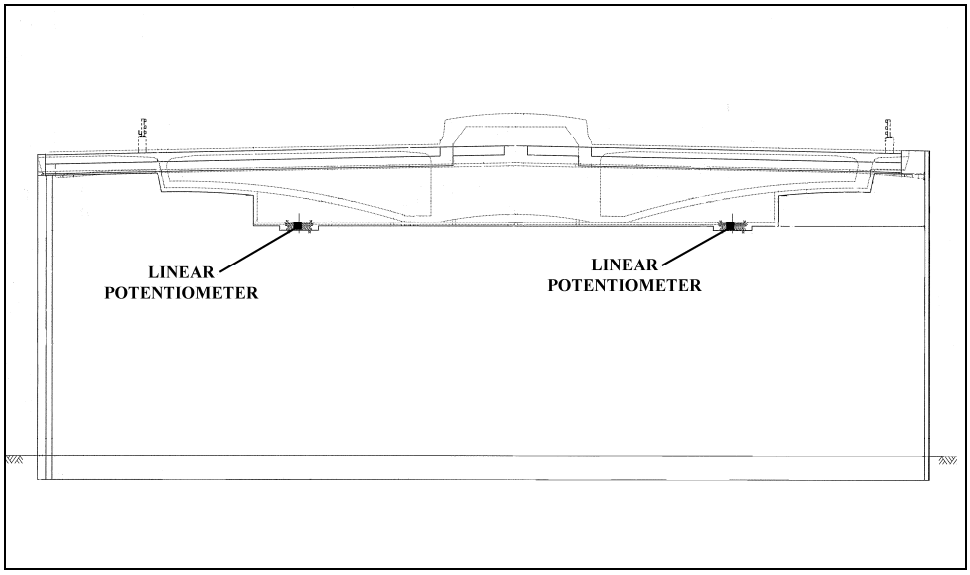

Figure 7.24 Abutment Bearing Linear Potentiometer Locations (North and South Abutment) (Modified IRIB Specifications and Construction Plans)

7.7.2 Precast Northbound and Southbound Roadway Segments

Monitoring equipment will be installed in the northbound and southbound roadway sections to capture various load effects. As described in Section 7.2.1, a total of four segments will be instrumented. All instrumented segments will be located on the south side of the bridge. Various combinations of vibrating wire gauges, foil strain gauges, and accelerometers will be installed. The following sections outline the different types of sections that will be instrumented with the various monitoring equipment. Further details about each gauge type can be found in Chapter 10 – Instrumentation.

7.7.2.1 Type-I

Precast Roadway Type-I segments are located at 1N-8 and 1S-8. There will be four vibrating wire gauges, four foil strain gauges, and one accelerometer

installed in each segment. Additionally, one data logger will be installed during construction to capture monitoring data. To accommodate permanent data acquisition, through-holes have been provided so that gauge wire can be run from the block-out to the data acquisition system located in the central beam. Figure 7.25 details the location of all permanent equipment that will be installed within Type-I. Figure 7.26 further details the schedule for Type-I instrumentation installation and required hardwire connections. Exact locations of the monitoring equipment will need to be discussed with the contractor. Also, exact locations of the "block-out" will need to be determined.

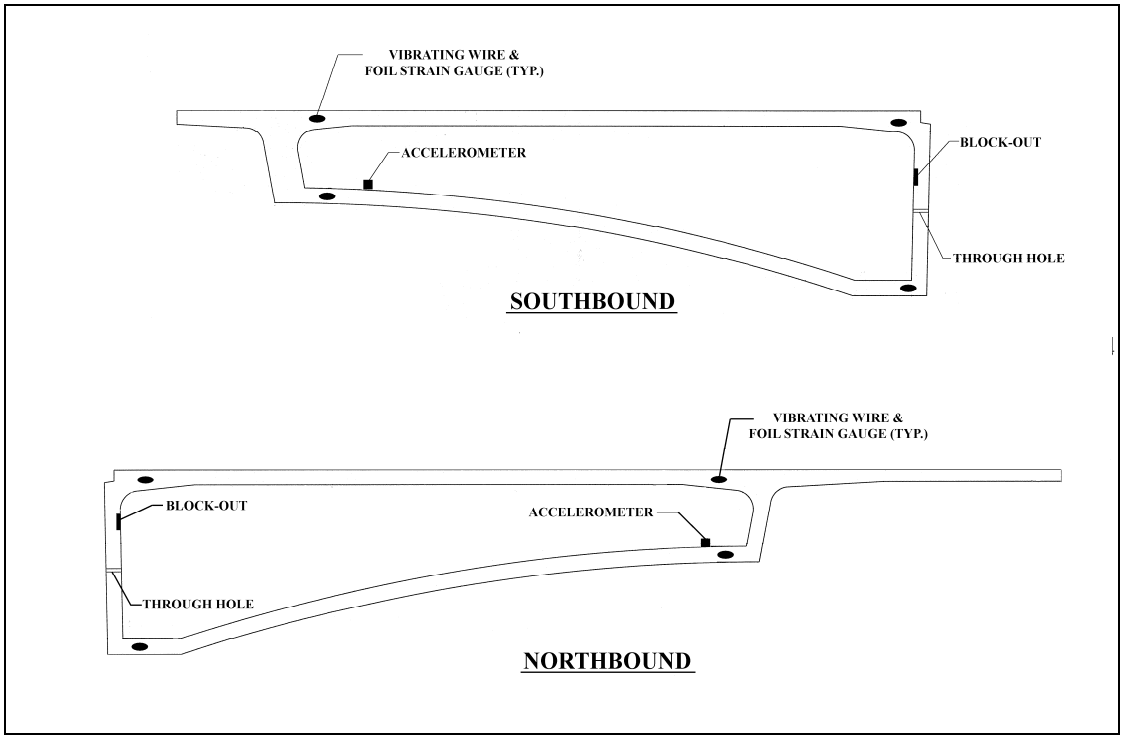

Figure 7.25 Roadway Segments Type-I Gauge Locations (Modified IRIB Specifications and Construction Plans)

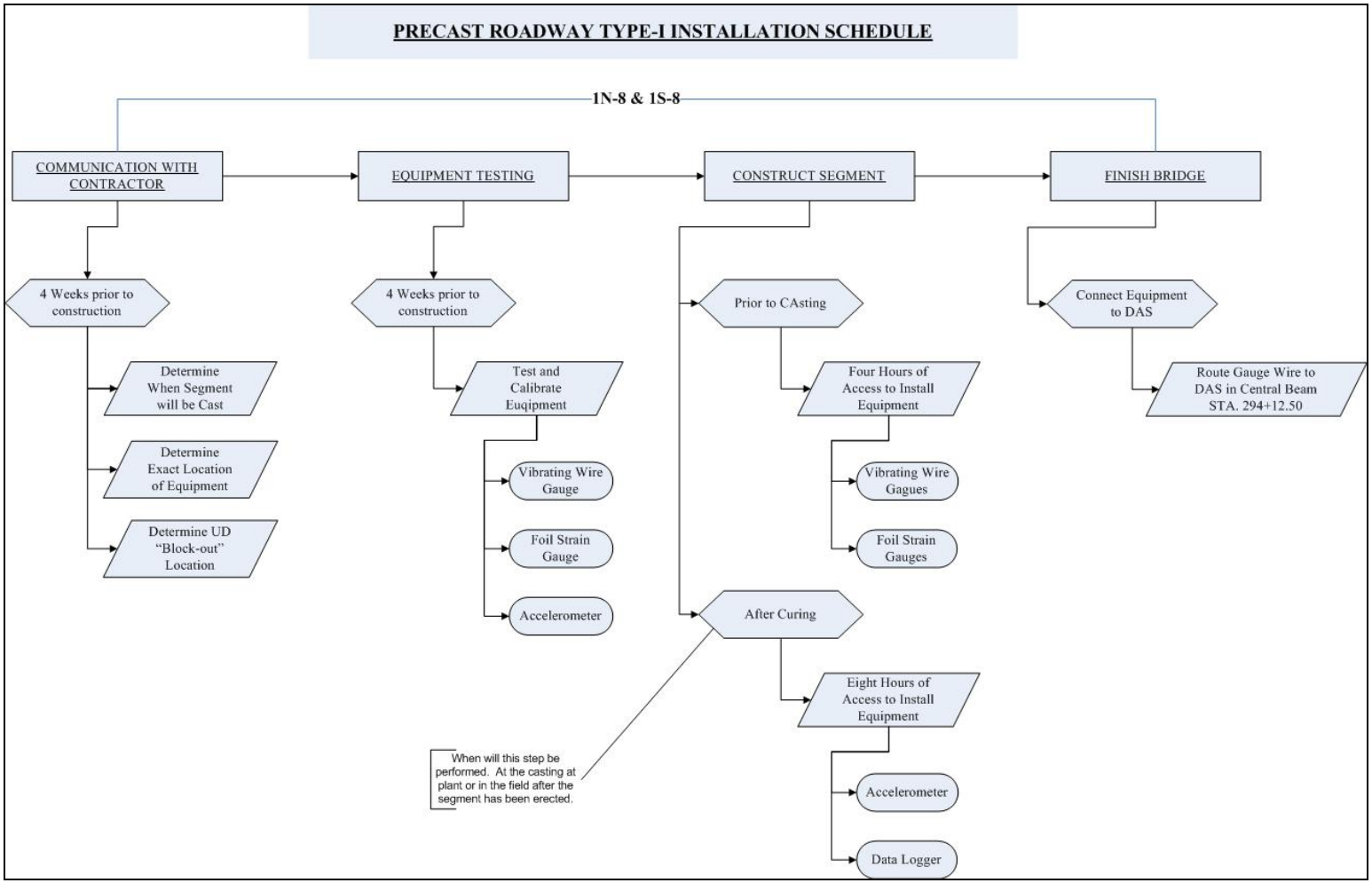

Figure 7.26 Precast Roadway Type-I Installation Schedule

7.7.2.2 Type-II

Precast Roadway Type-II segments are located at 1N-17 and 1S-17. There will be four vibrating wire gauges installed. Additionally, one data logger will be installed during construction to capture monitoring data. To accommodate permanent data acquisition, through-holes have been provided so that gauge wire can be run from the block-out to the data acquisition system located in the central beam. Figure 7.27 details the location of all permanent equipment that will be installed within Type-II. Figure 7.28 further details the schedule for Type-II instrumentation installation and required hardwire connections. Exact locations of the monitoring equipment will need to be discussed with the contractor. Also, exact locations of the "block-out" will need to be determined.

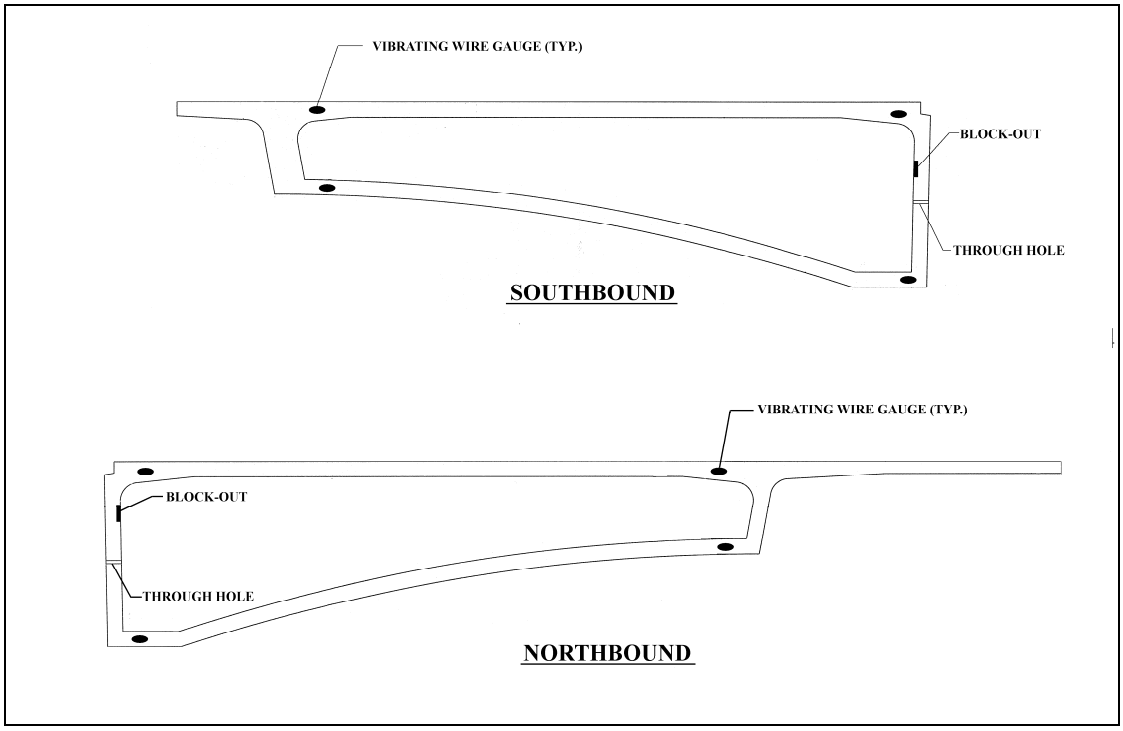

Figure 7.27 Roadway Segments Type-II Gauge Locations (Modified IRIB Specifications and Construction Plans)

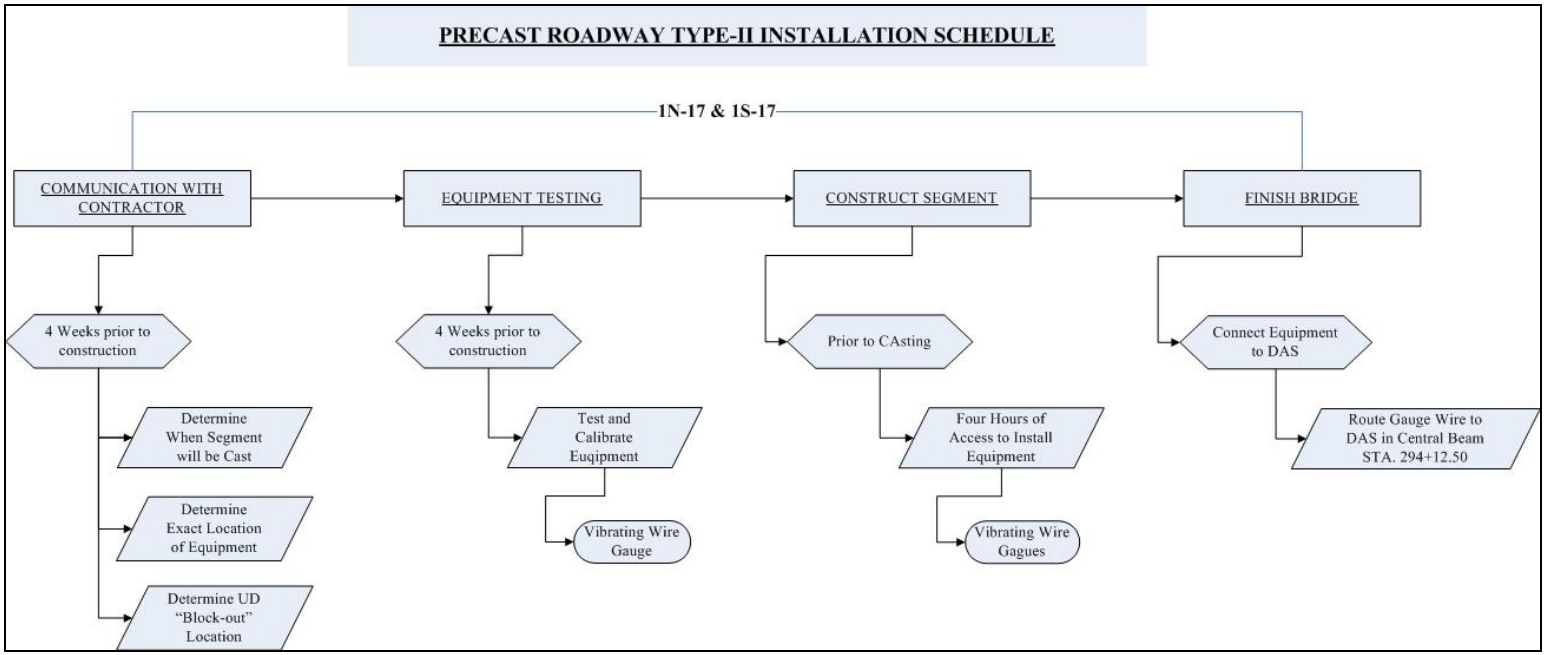

Figure 7.28 Precast Roadway Type-II Installation Schedule

7.7.3 Cast-In-Place Central Beam Segments

To capture load effects in the central beam, monitoring equipment will be installed. As described in Section 7.2.1, there will be a total of three segments instrumented in the central beam. Two of the segments will be located on the south side of the bridge and one on the north side. Various combinations of vibrating wire gauges, foil strain gauges, accelerometers, and global positioning systems will be installed at predetermined locations in the central beam. The following sections outline the various types of sections that will be instrumented with monitoring equipment. Further details about the each gauge type can be found in Chapter 10 – Instrumentation.

7.7.3.1 Type-I

Central Beam Type-I segment is located at Sta. 294+12.50. There will be four vibrating wire gauges, four foil strain gauges, and one accelerometer. Additionally, one data logger will be installed during construction, and one data acquisition system will be installed once construction is complete. Figure 7.29 details the location of all permanent equipment that will be installed. Figure 7.30 further details the schedule for Type-I instrumentation installation and required hardwire connections. Exact locations of the monitoring equipment will need to be discussed with the contractor.

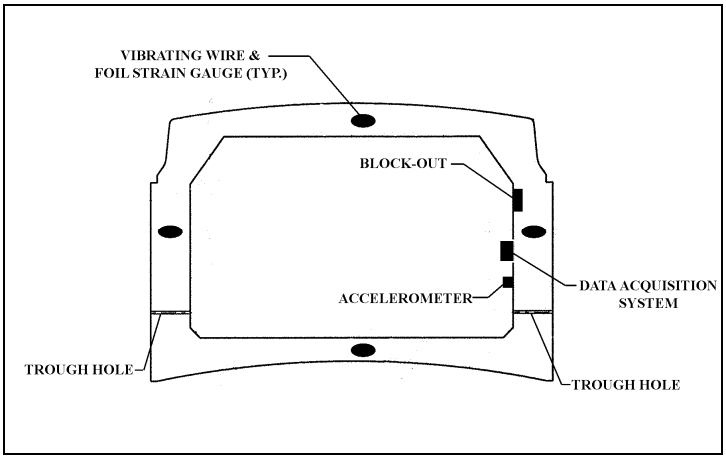

Figure 7.29 Central Beam Type-I Gauge Locations (Modified IRIB Specifications and Construction Plans)

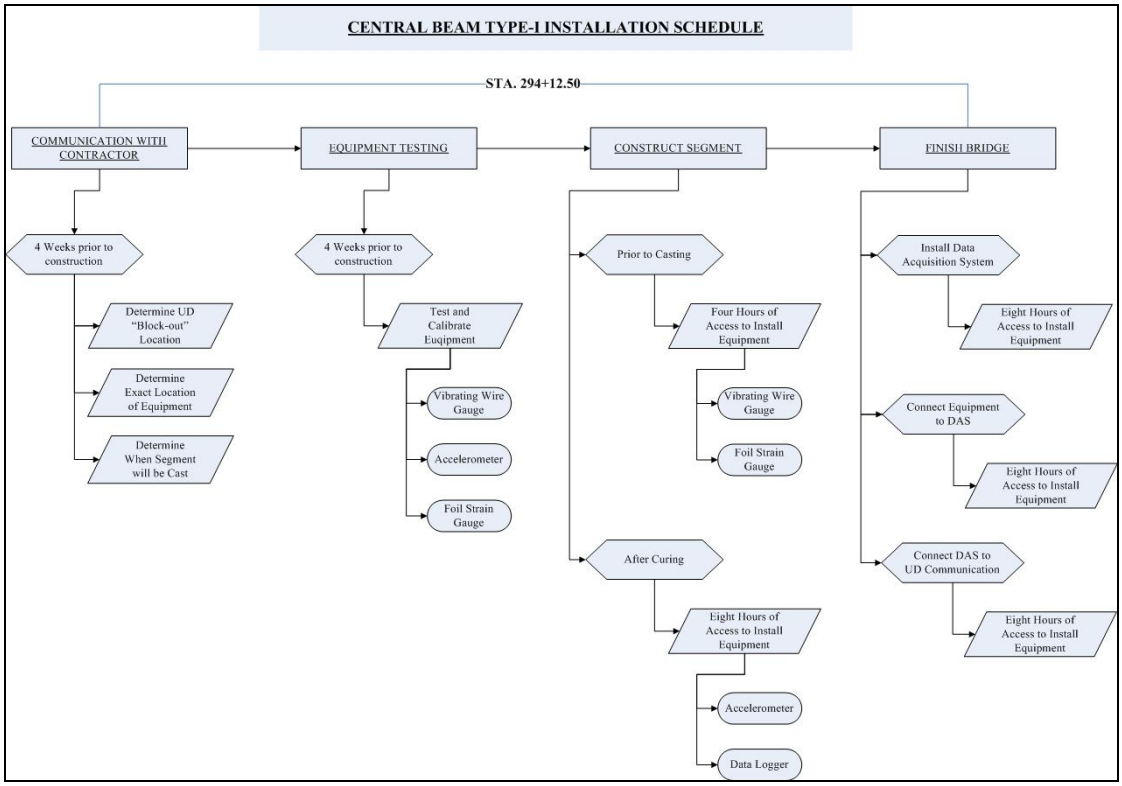

Figure 7.30 Central Beam Type-I Installation Schedule

7.7.3.2 Type-II

Central Beam Type-II segments are located at Sta. 294+78.00 and 305+22.00. There will be one global positioning system installed at these locations. Additionally, one data logger will be installed during construction, and one data acquisition system will be installed once construction is complete. Figure 7.31 details the location of all permanent equipment that will be installed within Type-II. Figure 7.32 further details the schedule for Type-II instrumentation installation and required hardwire connections. Exact locations of the monitoring equipment will need to be discussed with the contractor.

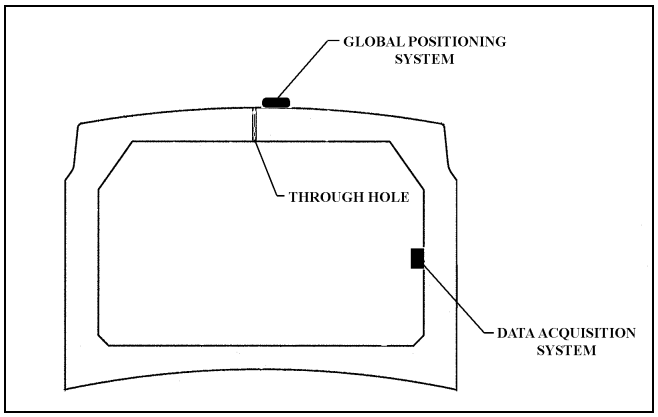

Figure 7.31 Central Beam Type-II Gauge Locations (Modified IRIB Specifications and Construction Plans)

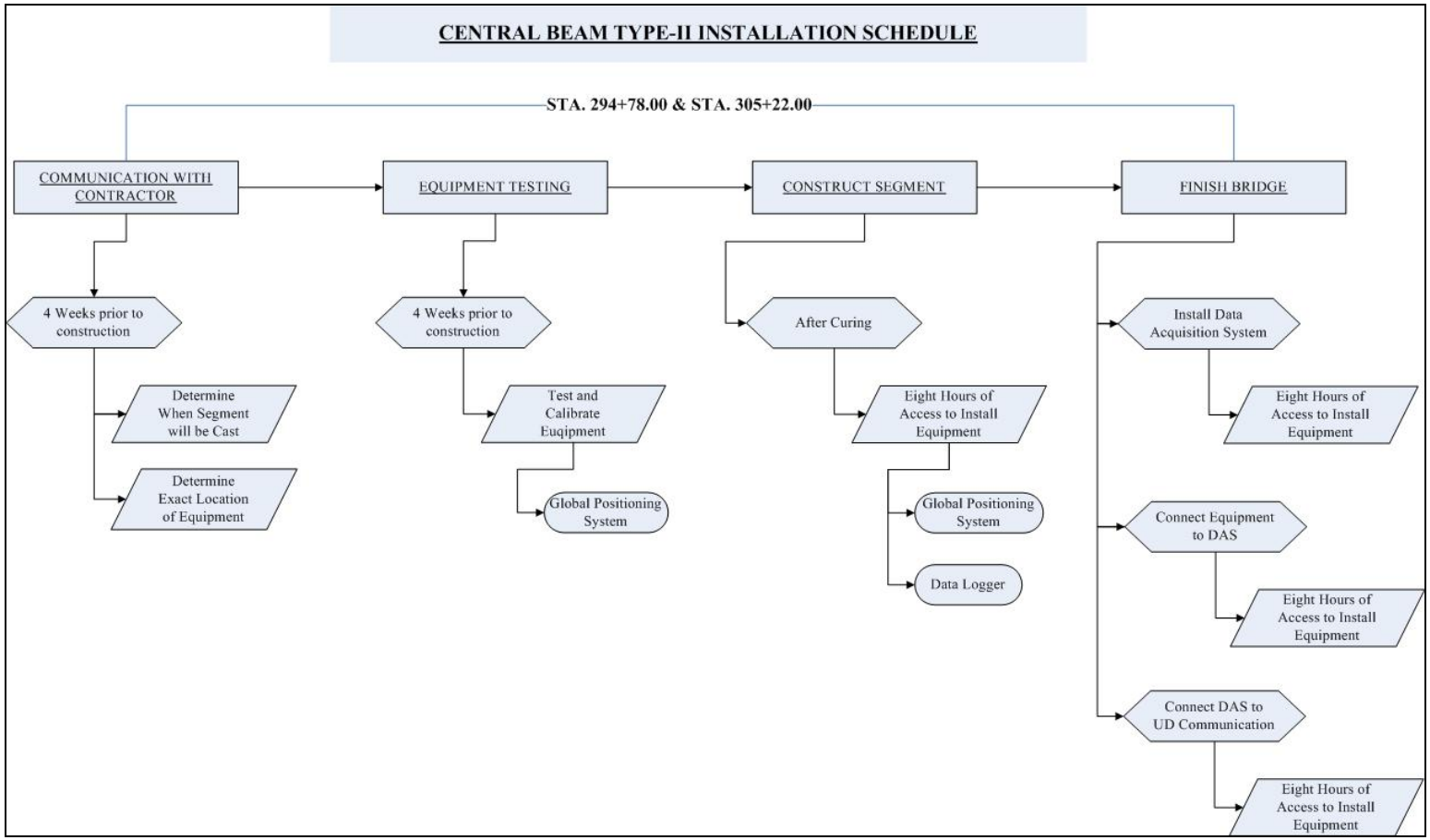

Figure 7.32 Central Beam Type-II Installation Schedule

7.8 Phase 5 - Arch-Rib Instrumentation

During Step 2 of the general construction sequence described in Section 7.3.1, the arch-rib will cast-in-place using a form-traveler and supported throughout construction with temporary stays. Both the Upstation and Downstation portions of the bridge will be constructed at the same time. CIBrE plans to instrument twelve locations in the arch-rib (seven locations Upstation and five locations Downstation). Each location will consist of a variety of gauges. A complete schedule of the arch-rib construction and the intended instrumentation procedures are described in Figures 7.33 and 7.34. Figure 7.33 describes the proposed construction sequence of the arch-rib while, Figure 7.34 gives a general outline of the schedule for each type of arch segment that will be instrumented.

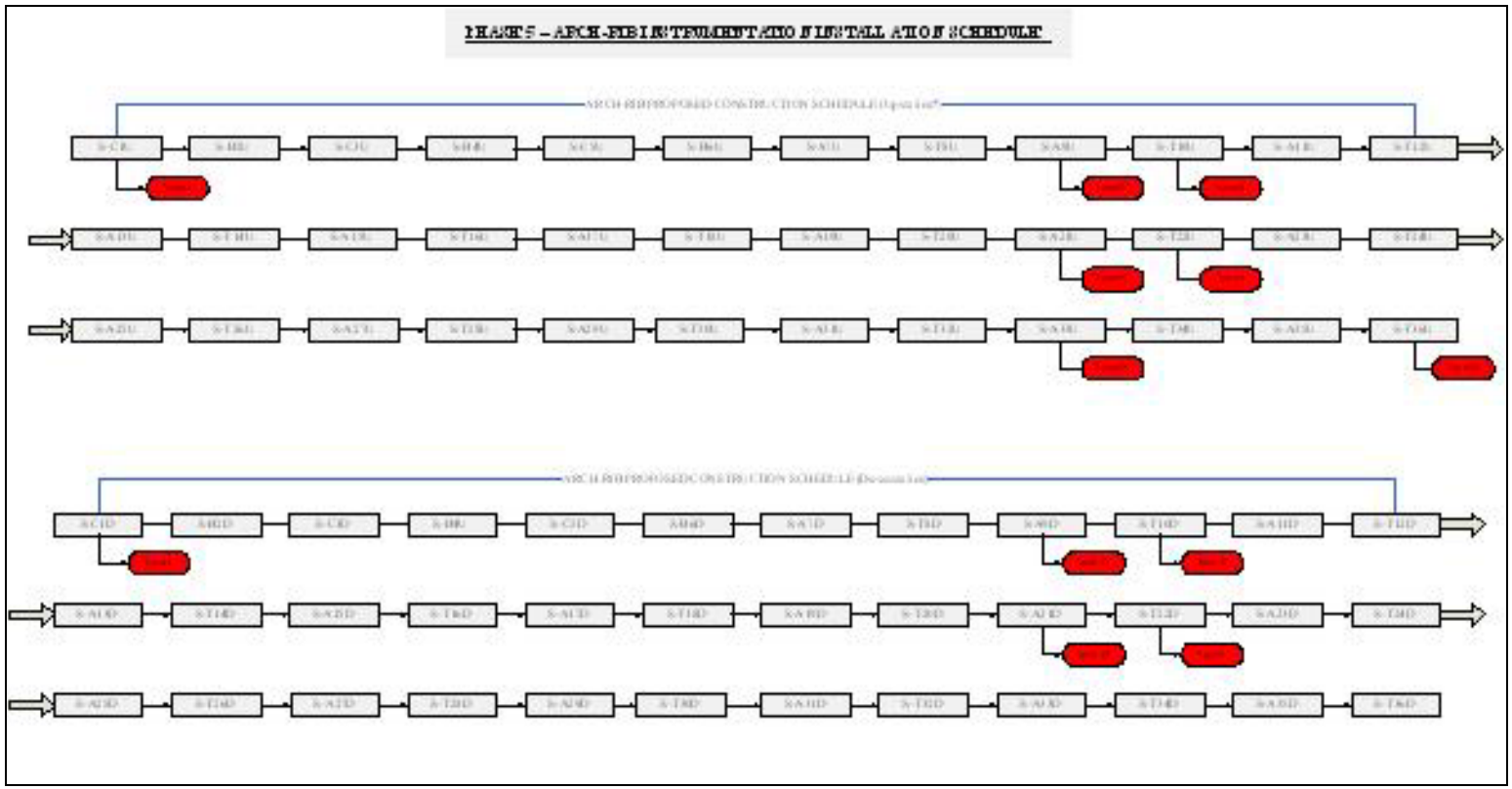

Figure 7.33 Arch-Rib Instrumentation Installation Schedule

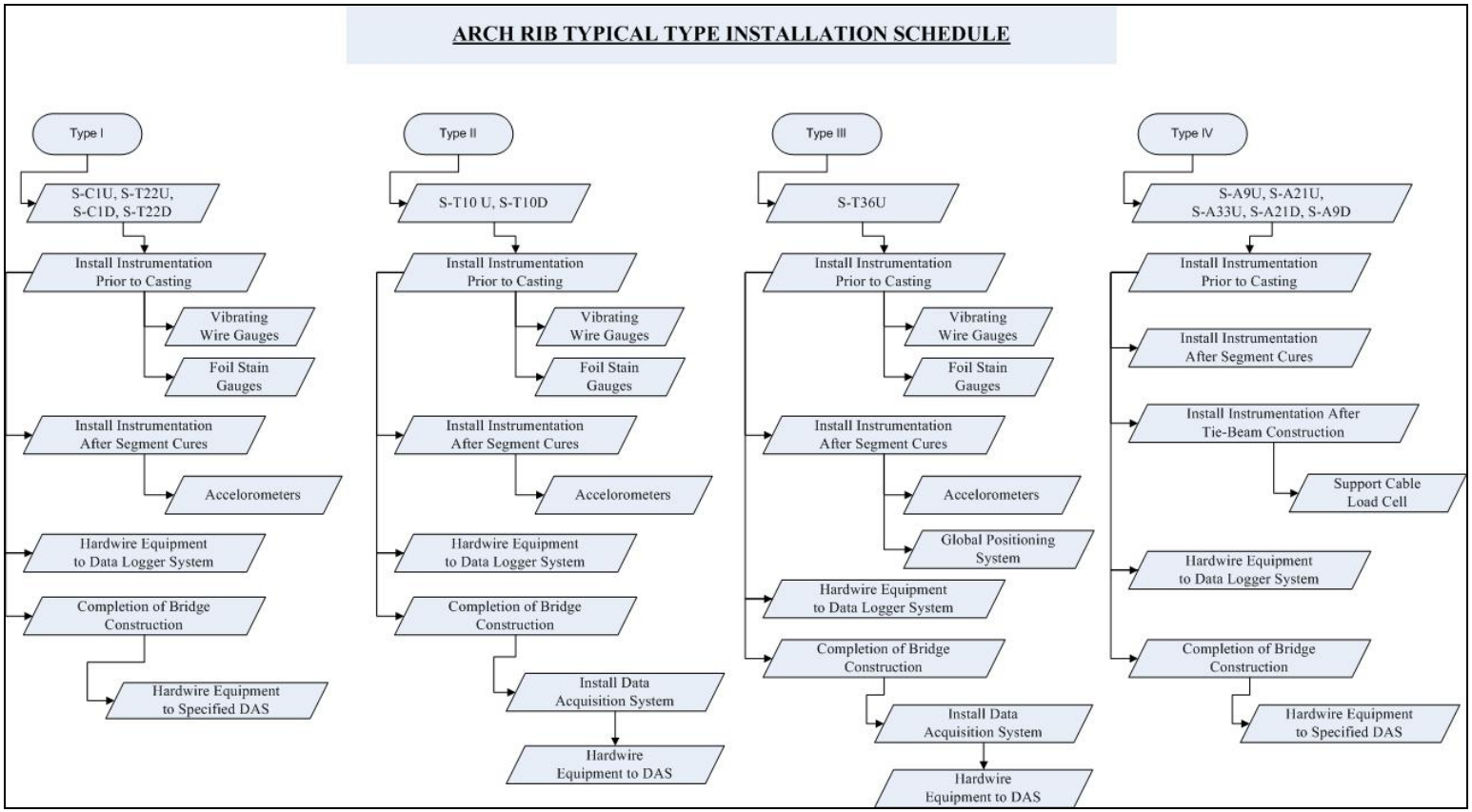

Figure 7.34 Arch-Rib Typical Type Installation Schedule

7.8.1 Cast-In-Place Arch-Rib Segments

Twelve sections along the arch-rib will be instrumented with monitoring equipment to capture load effects. Various combinations of vibrating wire gauges, foil strain gauges, accelerometers, load cells, and global positioning systems will be installed. The following sections outline the various types of sections that will be instrumented with monitoring equipment. Further details about each gauge type can be found in Chapter 10 – Instrumentation.

7.8.1.1 Type-I

Arch-Rib Type-I segments are located at S-C1U, S-T22U, S-C1D, and S-T22D. There will be four vibrating wire gauges, four foil strain gauges, and one accelerometer. Additionally, one data logger will be installed during construction to capture monitoring data. Permanent data acquisition will be accomplished by routing gauge wire from the block-out through contractor-installed local conduit to a location in the arch-rib which contains a data acquisition system. Please see Section 7.2.3.1 for further details. Figure 7.35 details the location of all permanent equipment that will be installed within Type-I. Figure 7.36 further details the schedule for Type-I instrumentation installation and required hardwire connections. Exact locations of the monitoring equipment will need to be discussed with the contractor. Also, exact locations of the "block-out" will need to be determined.

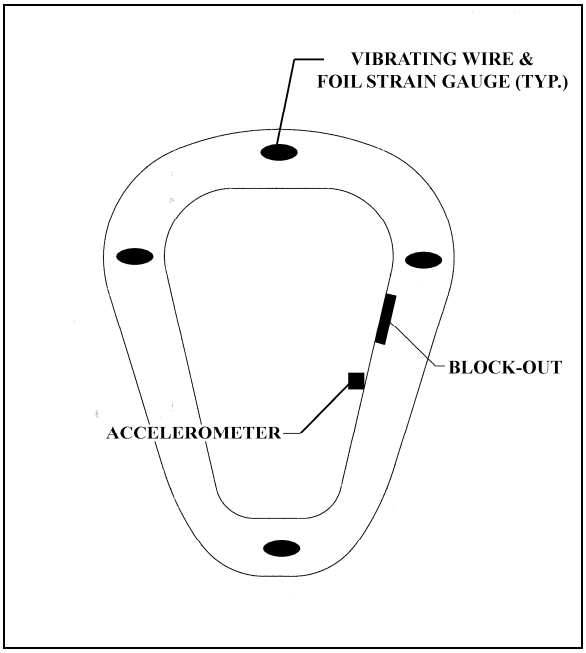

Figure 7.35 Arch-Rib Type-I Gauge Locations (Modified IRIB Specifications and Construction Plans)

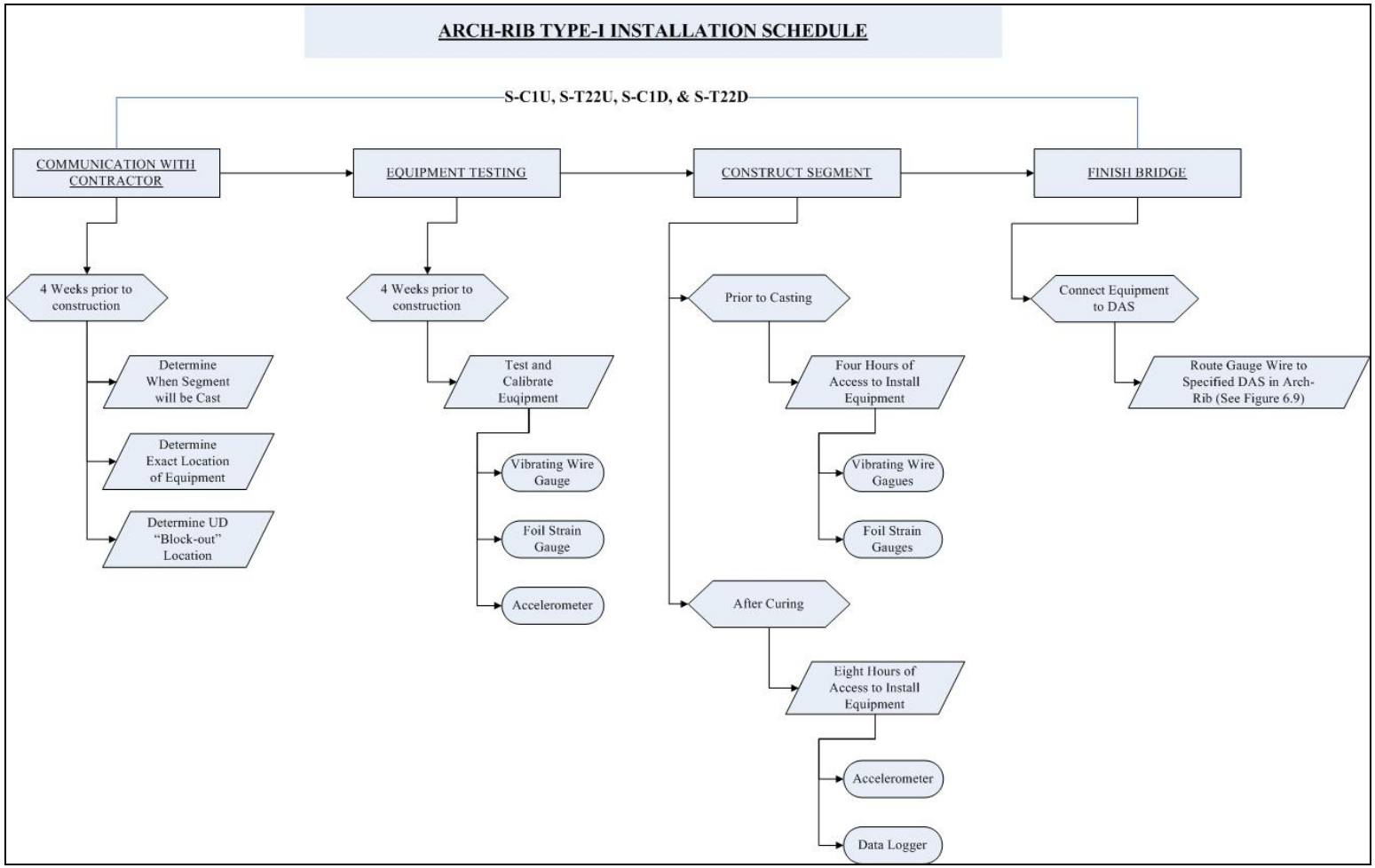

Figure 7.36 Arch-Rib Type-I Installation Schedule

7.8.1.2 Type-II

Arch-Rib Type-II segments are located at S-T10U and S-T10D. There will be four vibrating wire gauges, four foil strain gauges, and one accelerometer. Additionally, one data logger will be installed during construction, and one data acquisition system will be installed once construction is complete. Permanent data acquisition will be accomplished by routing gauge wire from the block-out through contractor-installed local conduit to a location in the arch-rib which contains a data acquisition system. Please see Section 7.2.3.1 for further details. Figure 7.37 details the location of all permanent equipment that will be installed within Type-II. Figure 7.38 further details the schedule for Type-II instrumentation installation and required hardwire connections. Exact locations of the monitoring equipment will need to be discussed with the contractor. Also, exact locations of the "block-out" will need to be determined.

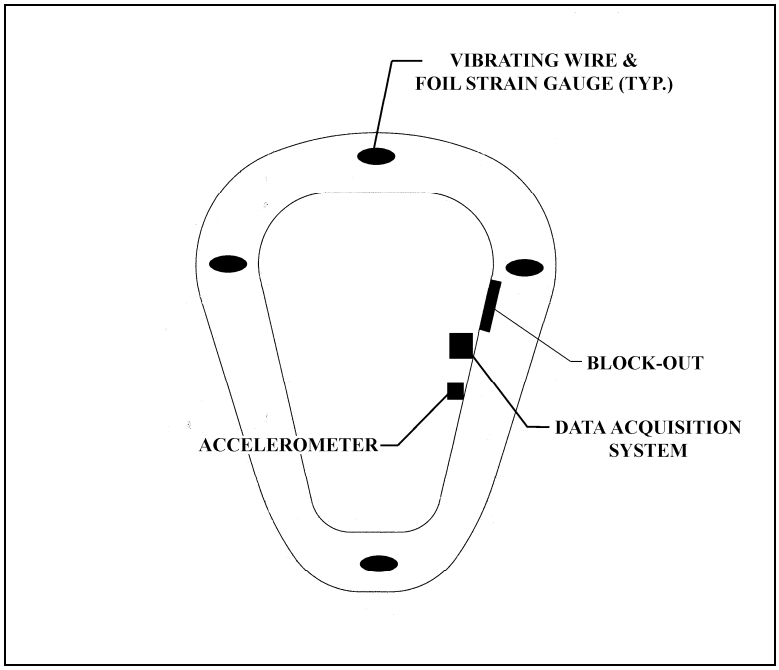

Figure 7.37 Arch-Rib Type-II Gauge Locations (Modified IRIB Specifications and Construction Plans)

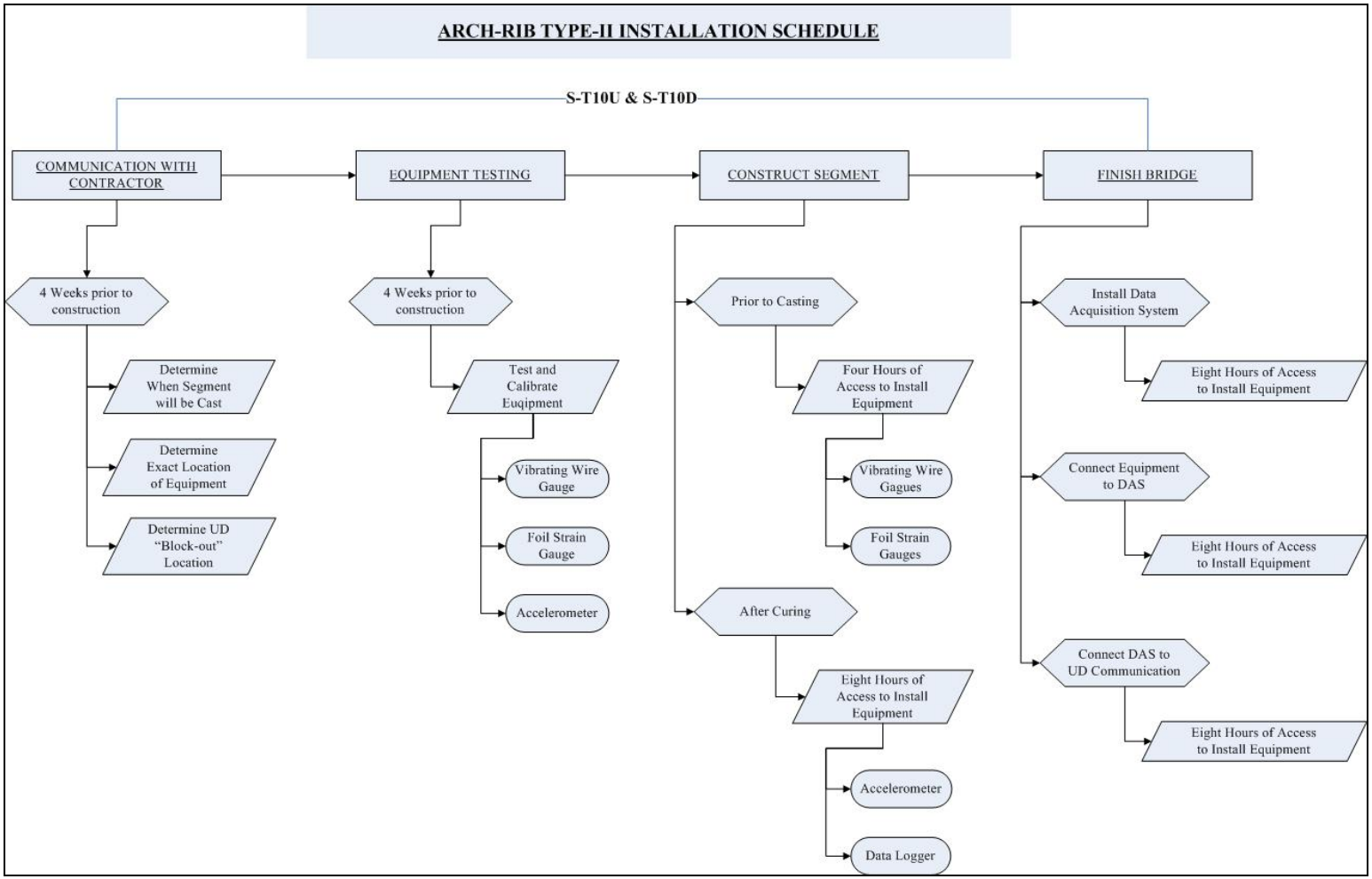

Figure 7.38 Arch-Rib Type-II Installation Schedule

7.8.1.3 Type-III

Arch-Rib Type-III segment is located at S-T36U. There will be four vibrating wire gauges, four foil strain gauges, one accelerometer, and one global positioning system installed. Additionally, one data logger will be installed during construction, and one data acquisition system will be installed once construction is complete. Permanent data acquisition will be accomplished by routing gauge wire from the block-out through contractor-installed local conduit to a location in the arch-rib which contains a data acquisition system. Please see Section 7.2.3.1 for further details. Figure 7.39 details the location of all permanent equipment that will be installed within Type-III. Figure 7.40 further details the schedule for Type-III instrumentation installation and required hardwire connections. Exact locations of the monitoring equipment will need to be discussed with the contractor. Also, exact locations of the "block-out" will need to be determined.

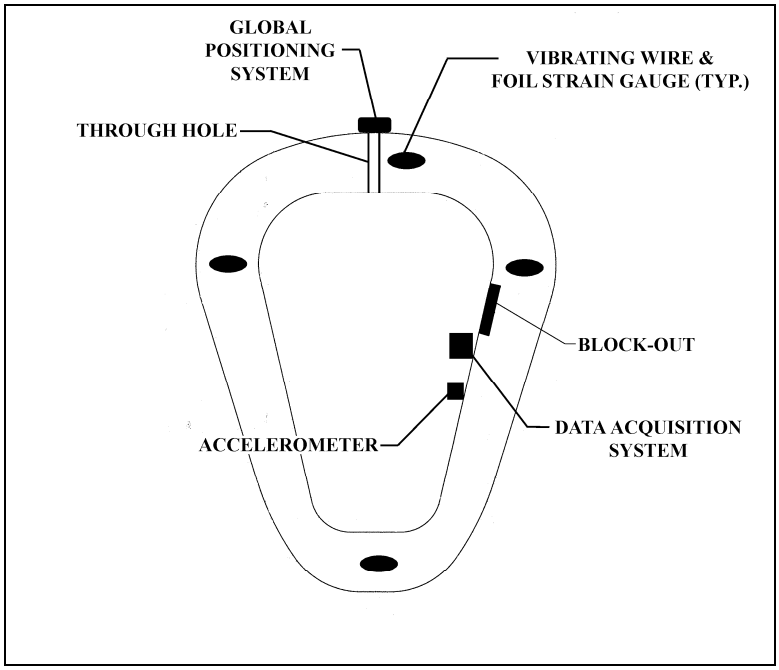

Figure 7.39 Arch-Rib Type-III Gauge Locations (Modified IRIB Specifications and Construction Plans)

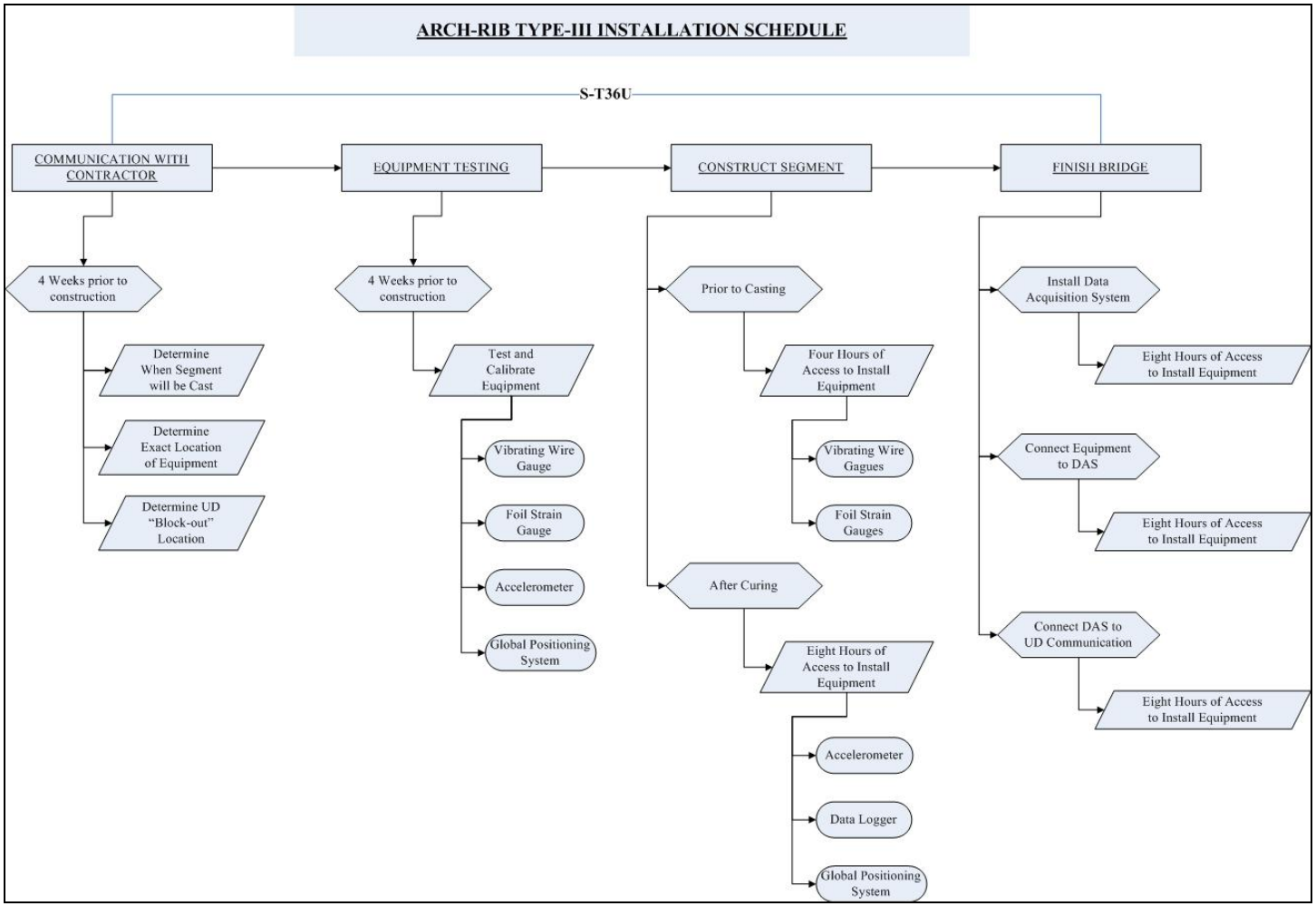

Figure 7.40 Arch-Rib Type-III Installation Schedule

7.8.1.4 Type-IV

Arch-Rib Type-IV segments are located at S-A9U, S-A21U, S-A33U, S-A21D, and S-A9D. There will be one load cell installed. Additionally, one data logger will be installed during construction to capture monitoring data. Permanent data acquisition will be accomplished by routing gauge wire from the block-out through contractor-installed local conduit to a location in the arch-rib which contains a data acquisition system. Please see Section 7.2.3.1 for further details. Figure 7.41 details the location of all permanent equipment that will be installed within Type-IV. Figure 7.42 further details the schedule for Type-IV instrumentation installation and required hardwire connections. Exact locations of the monitoring equipment will need to be discussed with the contractor. Also, exact locations of the "block-out" will need to be determined.

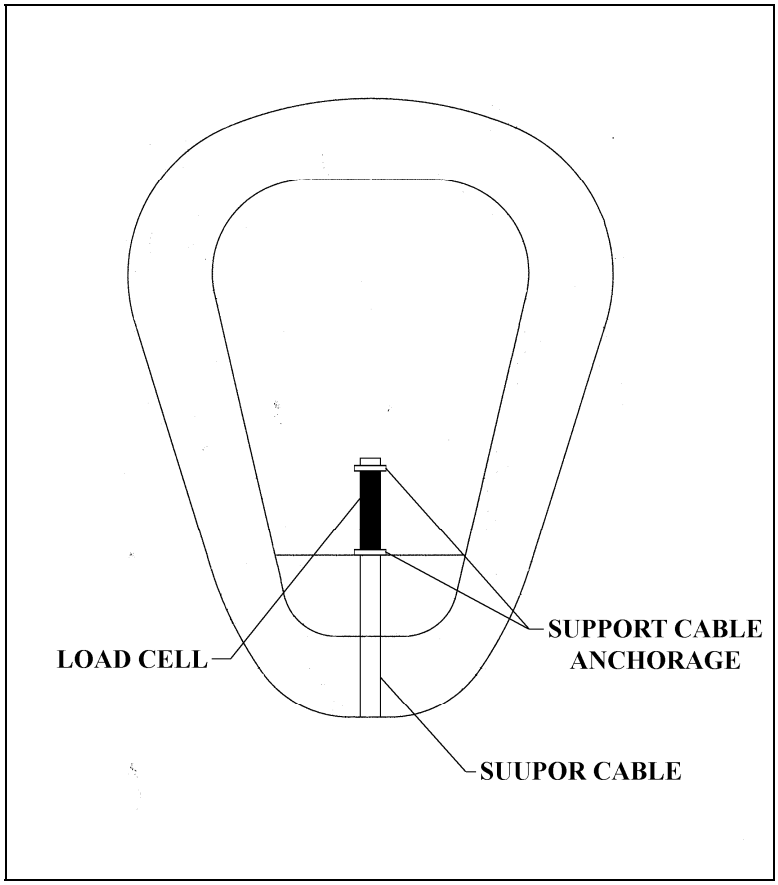

Figure 7.41 Arch-Rib Type-IV Gauge Locations (Modified IRIB Specifications and Construction Plans)

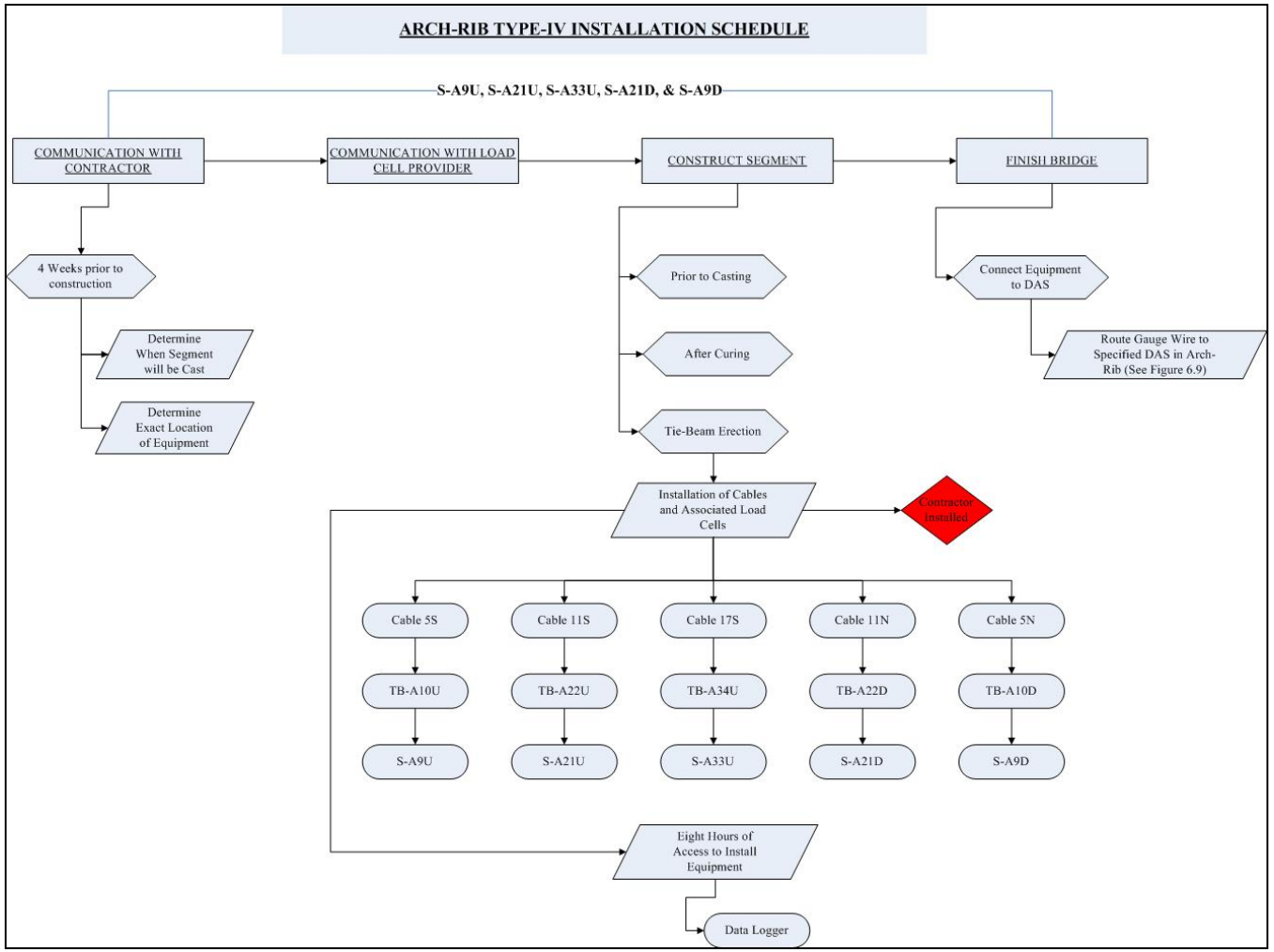

Figure 7.42 Arch-Rib Type-IV Installation Schedule

7.9 Phase 6 - Tie-Beam Instrumentation

During Step 4 of the general construction sequence described in Section 7.3.1 the tie-beam will be cast-in-place using a form-traveler. Both the Upstation and Downstation portions of the bridge will be constructed at the same time. CIBrE plans to instrument five locations in the tie-beam (three locations Upstation and two locations Downstation). Each location will consist of a variety of gauges. A complete schedule of the tie-beam construction and the intended instrumentation procedures is described in Figure 7.43.

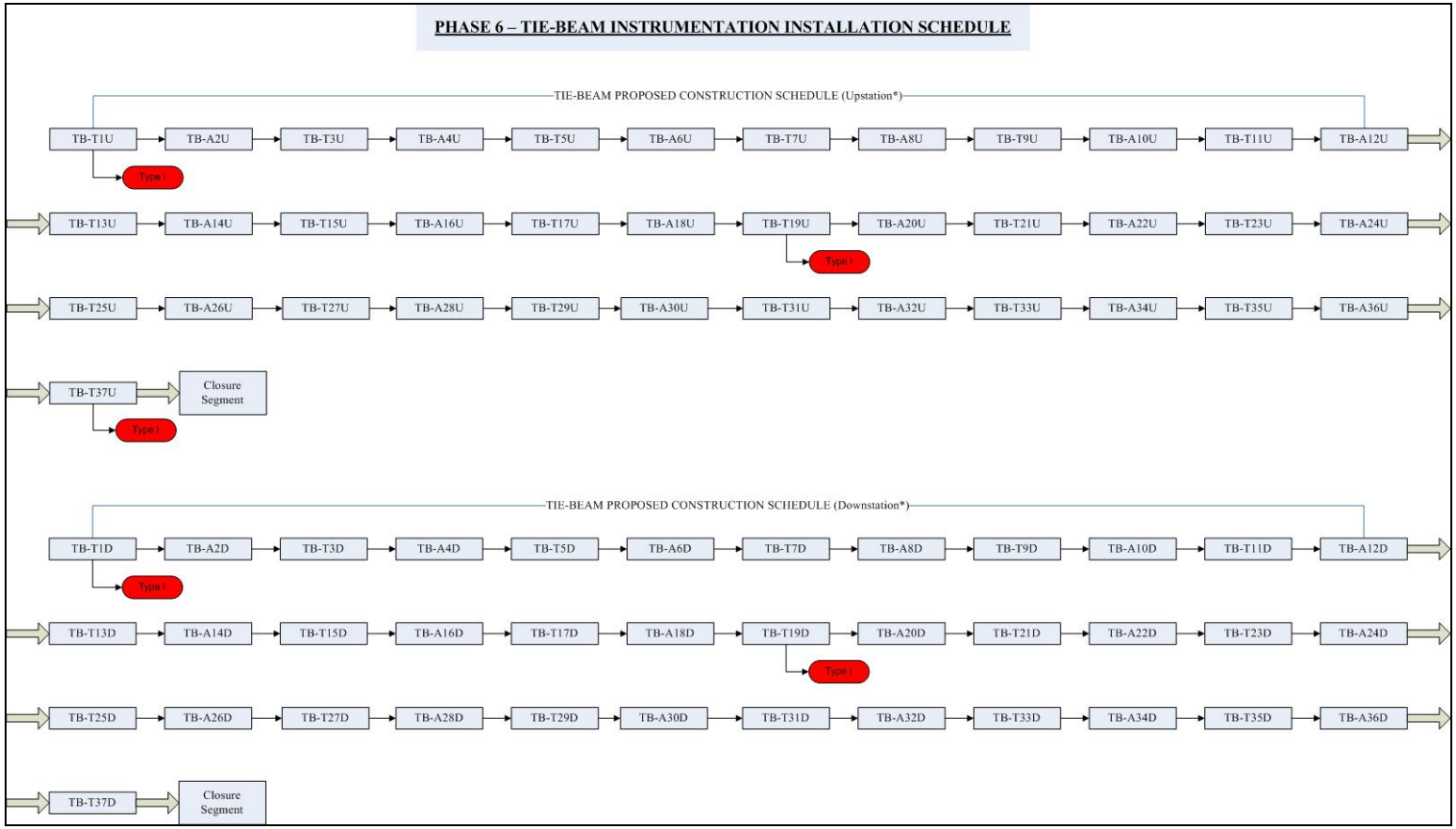

Figure 7.43 Tie-Beam Instrumentation Installation Schedule

7.9.1 Cast-In-Place Tie-Beam Segments

Five sections along the tie-beam will be instrumented with monitoring equipment to capture load effects. Various combinations of vibrating wire gauges, foil strain gauges, accelerometers, load cells, and global positioning systems will be installed. The following sections outline the various types of sections that will be instrumented with monitoring equipment. Further details about each gauge type can be found in Chapter 10 – Instrumentation.

7.9.1.1 Type-I

Tie-Beam Type-I segments are located at TB-T1U, TB-T19U, TB-T37U, TB-T19D, TB-T1D. There will be four vibrating wire gauges, four foil strain gauges, and one accelerometer installed at each location. Additionally, one data logger will be installed during construction, and one data acquisition system will be installed once construction is complete. Figure 7.44 details the location of all permanent equipment that will be installed within Type-I. Figure 7.45 further details the schedule for Type-I instrumentation installation and required hardwire connections. Exact locations of the monitoring equipment will need to be discussed with the contractor. Also, exact locations of the "block-out" will need to be determined.

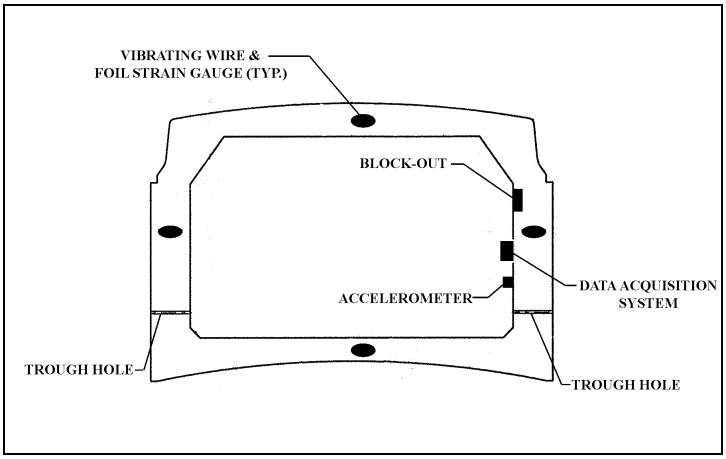

Figure 7.44 Tie-Beam Type-I Gauge Locations (Modified IRIB Specifications and Construction Plans)

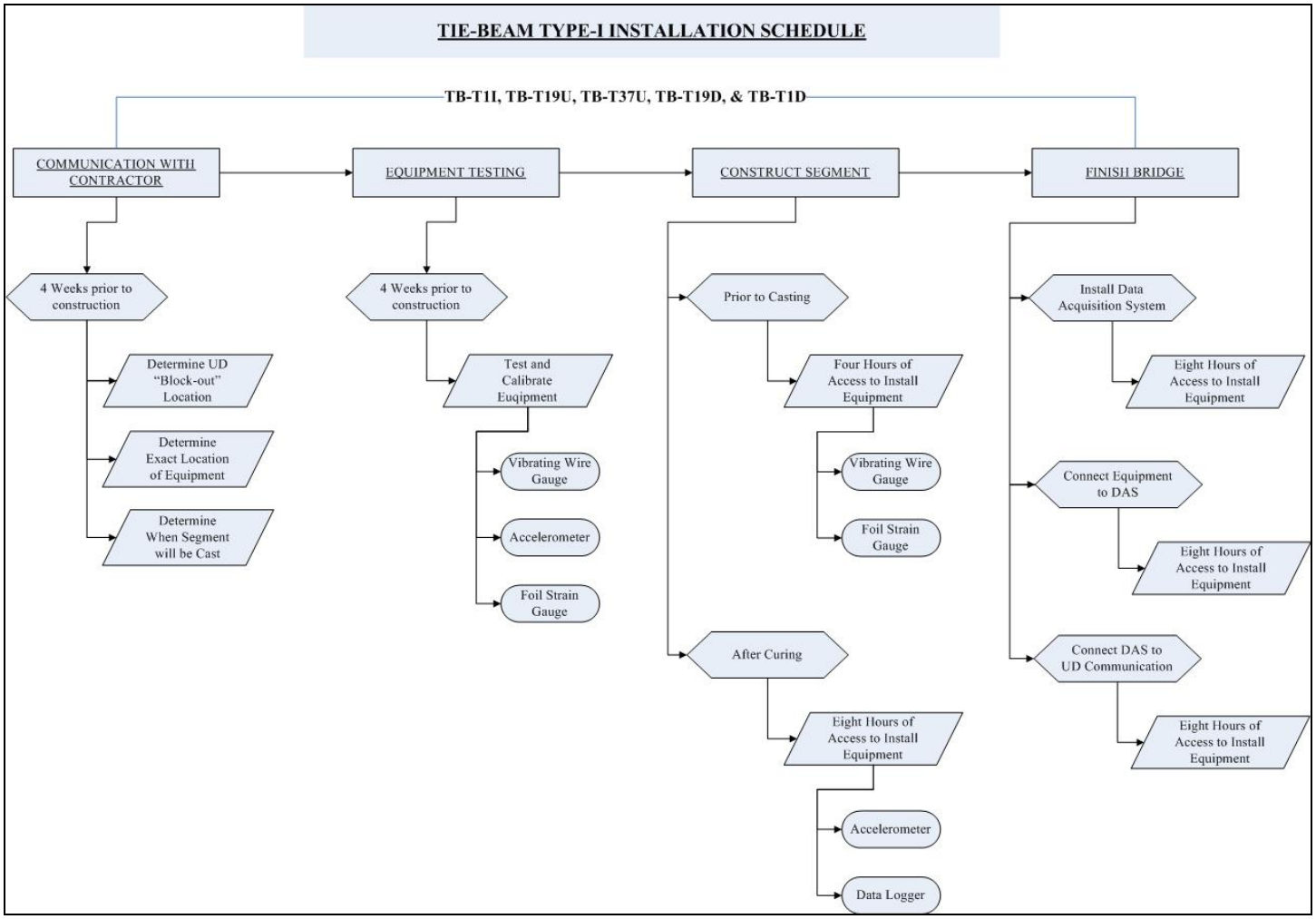

Figure 7.45 Tie-Beam Type-I Installation Schedule

7.9.2 Post-Tensioning Tendons

CIBrE intends to instrument void detection equipment in three post-tensioning tendons located in the tie-beam. CIBrE has not yet determined which post-tensioning tendons will be instrumented with void detection equipment. The instrumentation involves pulling a thin monitoring wire through the ducts along with the post-tensioning strands. The wire will require a separate access hole in the anchorage end plate. To demonstrate the installation of the tendon void detection system, and provide calibration, the same system will be installed in the clear ducts used to demonstrate the grouting techniques. CIBrE will contact the contractor and DelDOT when the all details of the void detection system have been finalized.

7.10 Phase 7 – Northbound and Southbound Roadway Instrumentation

During Step 6 of the general construction sequence described in Section 7.3.1, the northbound and southbound roadway segments will be erected. Both the Upstation and Downstation portions of the bridge will be erected at the same time. CIBrE plans to instrument five locations each in the northbound and southbound roadway segments (three locations Upstation and two locations Downstation). Each location will consist of a variety of gauges. A complete schedule of the roadway erection and the intended instrumentation procedures are described in Figures 7.46 and 7.47.

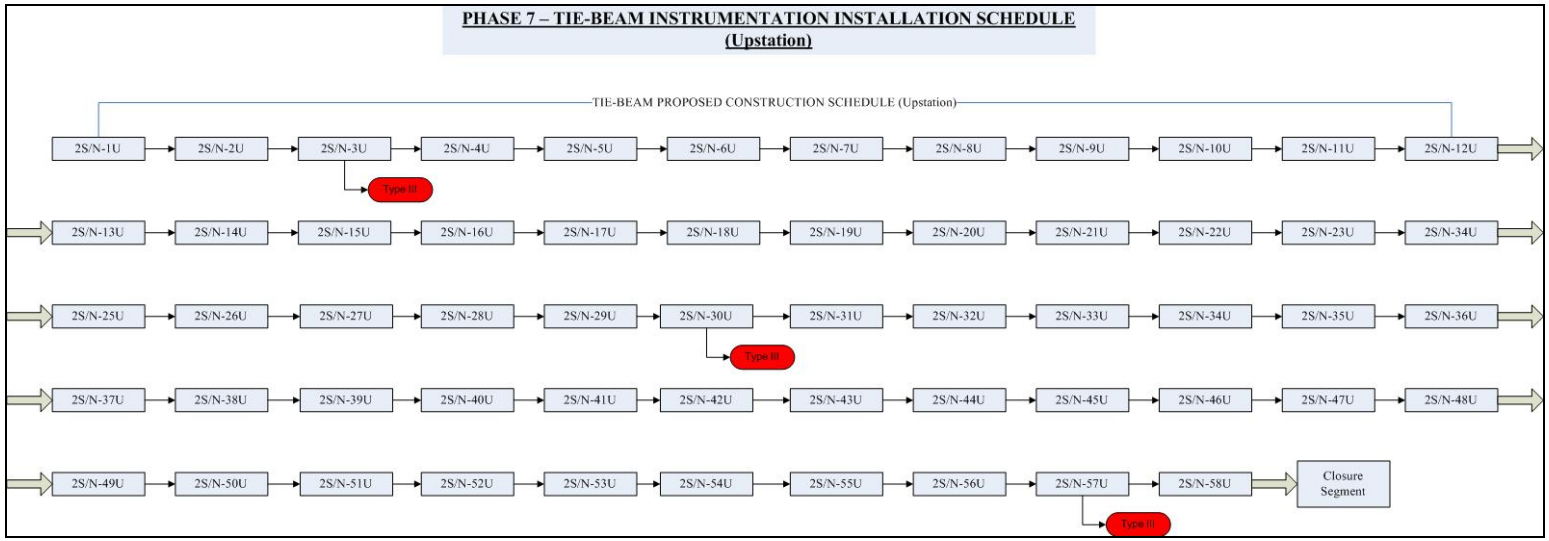

Figure 7.46 Roadway Instrumentation Installation Schedule (Upstation)

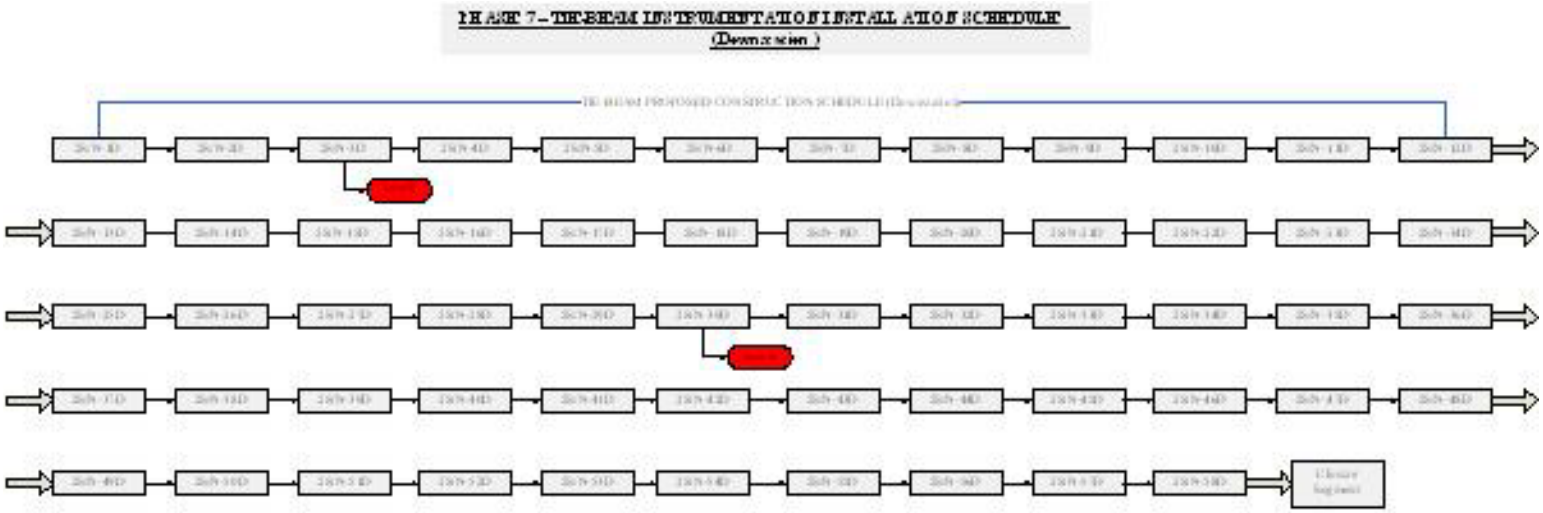

Figure 7.47 Roadway Instrumentation Installation Schedule (Downstation)

7.10.1 Precast Roadway Segments

Five sections each in the north and southbound roadway sections will be instrumented with monitoring equipment to capture load effects. Various combinations of vibrating wire gauges, foil strain gauges, and accelerometers will be installed. The following sections outline the different types of sections that will be instrumented with monitoring equipment. Further details about each gauge type can be found in Chapter 10 – Instrumentation.

7.10.1.1 Type-III

Precast Roadway Type-III segments are located at 2N-3U, 2S-3U, 2N-30U, 2S-30U, 2N-57U, 2S-57U, 2N-30D, 2S-30D, 2N-3D, 2S-3D. Four vibrating wire gauges, four foil strain gauges, and one accelerometer will be installed at each location. Additionally, one data logger will be installed during construction to capture monitoring data. To accommodate permanent data acquisition, through-holes have been provided so that gauge wire can be run from the block-out to the data acquisition system located in the tie-beam. Figure 7.48 details the location of all permanent equipment that will be installed within Type-III. Figure 7.49 further details the schedule for Type-III instrumentation installation and required hardwire connections. Exact locations of the monitoring equipment will need to be discussed with the contractor. Also, exact locations of the "block-out" will need to be determined.

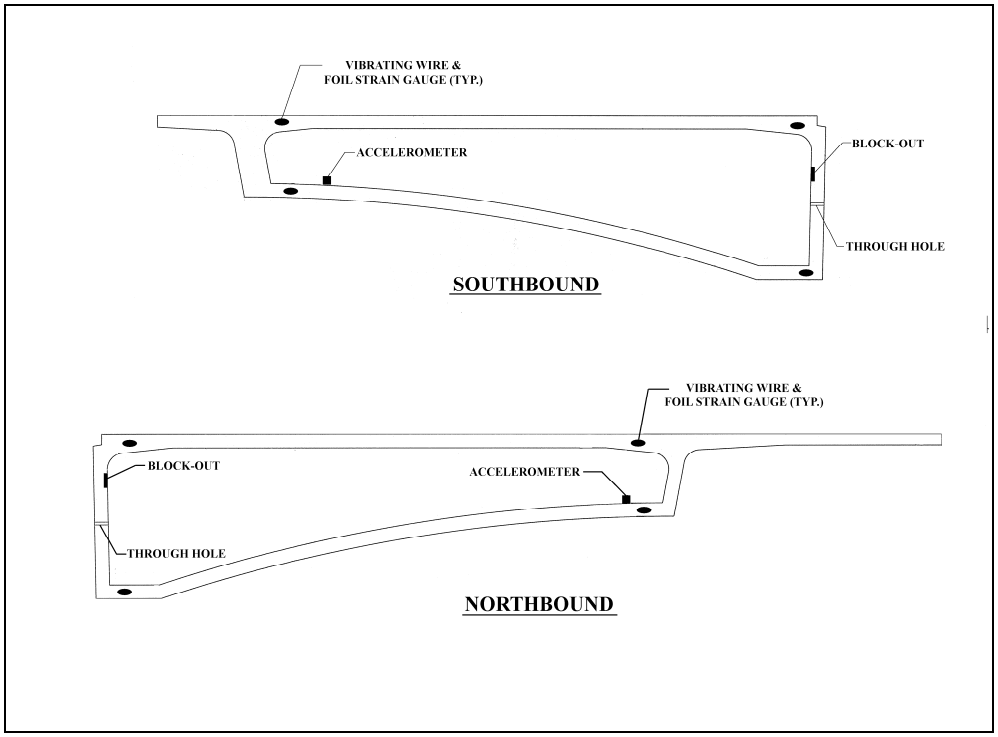

Figure 7.48 Precast Roadway Segments Type-I Gauge Locations (Modified IRIB Specifications and Construction Plans)

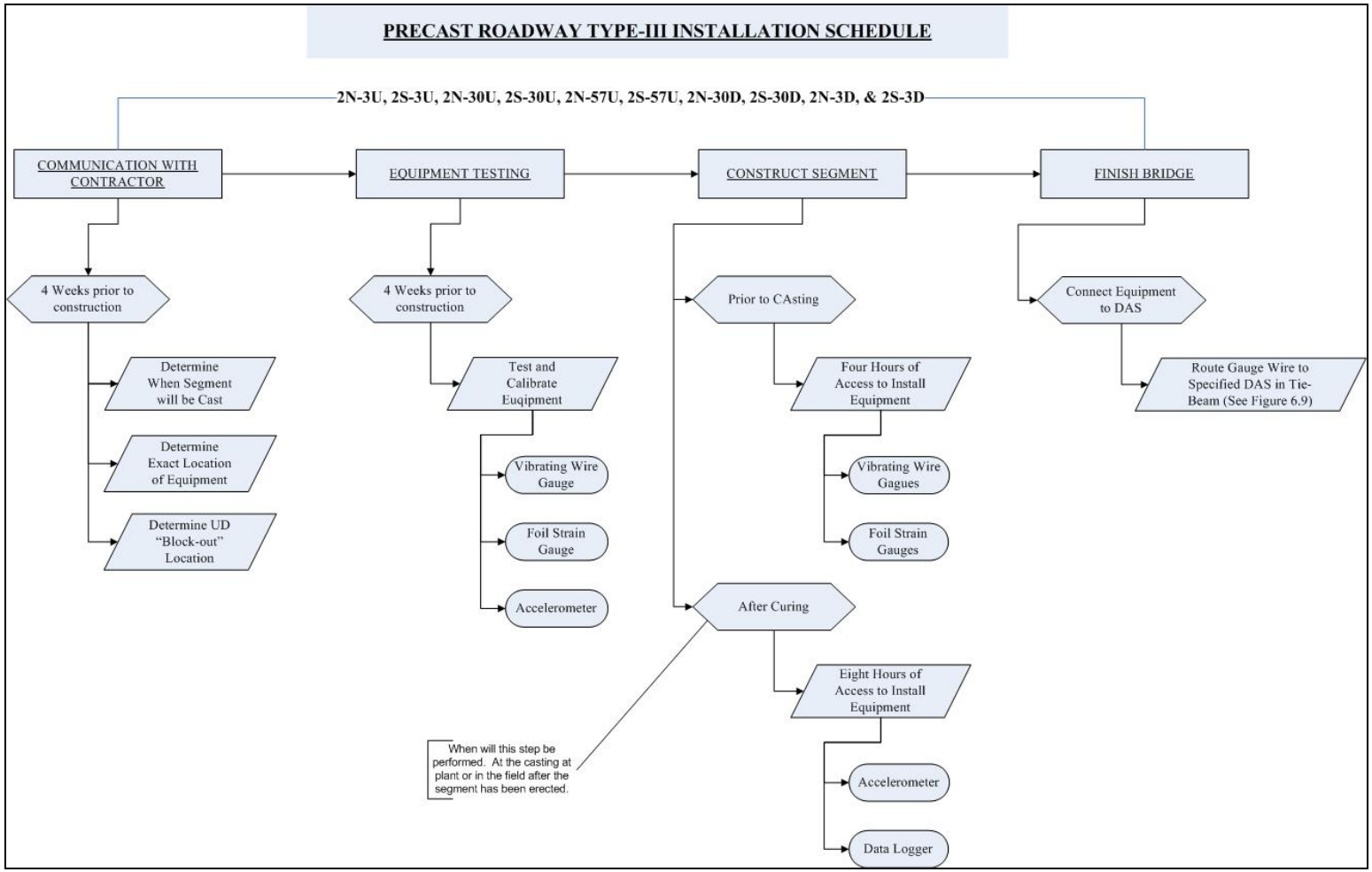

Figure 7.49 Precast Roadway Type-III Installation Schedule

7.10.2 Post-Tensioning Tendons

CIBrE intends to instrument void detection equipment in six post-tensioning tendons in the precast roadway segments; exact locations have not yet been determined. The instrumentation involves pulling a thin monitoring wire through the ducts along with the post-tensioning strands. The wire will require a separate access hole in the anchorage end plate. To demonstrate the installation of the tendon void detection system and provide calibration, the same system will be installed in the clear ducts used to demonstrate the grouting techniques. CIBrE will contact the contractor and DelDOT when the details of the void detection system have been finalized.

CHAPTER 8

STAGE IV – BASELINE BEHAVIOR

8.1 Introduction

During Stage IV of the monitoring project, CIBrE intends to capture the baseline behavior of the bridge, including initial geometry and behavior with respect to traffic, wind, and thermal loading. The information will be used to create a database to serve as a reference for DelDOT throughout the service life of the bridge. It is intended that this stage of the project will last no more that one year after the completion of the bridge. CIBrE will perform tests on the bridge before the bridge is open to construction and during its first year of service. The following sections outline the necessary tests that will be performed to help CIBrE quantify the behavior of the bridge under various loading conditions.

8.2 Initial Geometry

Geometry of the bridge will be continuously monitored through the use of three global positioning systems placed on the bridge (see Section 7.2.1 for exact location of GPS units). The initial geometry will be determined with these GPS units. A baseline temperature should also be determined.

Currently, CIBrE has no plans to capture the geometry in more detail. If CIBrE or DelDOT decides that more geometry data is necessary or desired than a detailed plan and schedule of the required tests should be laid out by CIBrE.

8.3 Traffic Response

The bridge's response to traffic will be determined through static, dynamic, and ambient vibration load tests. CIBrE will work with DelDOT to determine the appropriate tests to conduct and the time of the tests, to coordinate traffic, and to obtain the required loading trucks. Additionally, any instrumentation needed to perform the load tests should be purchased and tested well in advance of the actual tests. Additional instrumentation may include scales, cameras, gauges, and wires. Once all of this information is available, a detailed plan and schedule for the load tests will be developed and available for review. As stated previously all collected data should be stored and made available to DelDOT for future use.

8.4 Wind Response

The bridge's response to wind loading will be determined by gathering bridge response data under ambient wind conditions. Also, CIBrE will measure wind conditions at a weather station located near the bridge (the exact location of the weather station has yet to be determined). It is intended that ambient wind data and response will be taken over a one-year period of time to quantify the baseline behavior of the bridge's response to wind loadings. A site-specific wind history will also be developed for the Indian River Inlet. All collected data should be stored and made available to DelDOT for future use.

8.5 Thermal Response

Seasonal variations in response due to temperature will be tracked, and displacement and internal strains will be monitored. To determine the baseline behavior of the bridge, one year's worth of data will be captured. This data will help to determine the at-rest stresses and strains that were present in the bridge before it

was opened to traffic. All collected data should be stored and made available to DelDOT for future use.

CHAPTER 9

STAGE V – LONG-TERM CONDITIONS

9.1 Overview

During Stage V, the bridge will be continuously monitored. CIBrE will actively take part in the monitoring of the bridge up to the first biannual inspection. CIBrE's main goal for this stage of the project is to create a database of monitoring data and reports that will allow DelDOT to effectively monitor the structural health of the bridge throughout its service life. CIBrE also hopes to use the data collected for research purposes.

9.2 Biannual Inspection

During the first biannual inspection of the bridge, CIBrE intends to conduct load test on the bridge. The load tests will be similar to the tests conducted after the completion of the bridge. The test will serve as a measure of long-term performance. Results will be compared to the initial baseline response determined previously. CIBrE will use the results from the biannual inspection, biannual load tests, initial load tests, and the data that have been continually gathered to create a report. That will describe the bridge's current condition and assess the overall effectiveness of the monitoring program. Once the biannual inspection is completed and the report is finalized, CIBrE intends to turn the monitoring program over to DelDOT.

CHAPTER 10

INSTRUMENTATION

10.1 Instrumentation

A variety of gages and sensors will be used in the instrumentation of the Indian River Inlet Bridge. This chapter is intended to identify these gages and sensors and to give important information relating to their purpose, purchase, installation, setup and use.

Upon completion, this chapter should list specific brands and models of sensors to be used, list sources for purchasing the sensors, cite relevant manuals related to the sensors, and give a step-by-step procedure for the installation and set-up of each sensor.

However, at this time this chapter lists only a few highlights about each sensor type. These highlights include how the gage is mounted, what each gage measures, what the measurements will be used for, and some miscellaneous considerations about each sensor.

10.1.1 Foil Strain Gage

- Mounted on sister bars, then embedded in concrete sections
- Measure strains
- Ideal for high frequency reading
- Allow for the determination of plane strain in a section when used in groups of three (four including a backup)

- Will be used primarily to track event data such as load tests and wind events

10.1.2 Vibrating Wire Strain Gage

- Mounted on sister bars then embedded in concrete sections
- Measure strains and temperature
- Ideal for low frequency reading
- Allow for the determination of plane strain in a section when used in groups of three (four including a backup)
- Will be used primarily to track long-term behavior, creep, shrinkage, and thermal effects

10.1.3 Accelerometer

10.1.3.1 Use in Concrete Sections

- Surface mounted using epoxy
- Measure acceleration of a section
- Allow the dynamic behavior of the bridge to be examined
- Will be used to determine the dynamic response of the bridge to wind and traffic

10.1.3.2 Use On Temporary Stays

- Mounted how?
- Measure acceleration of a cable
- Allow the determination of tension in a cable
- Will be used to track temporary stay forces during construction

10.1.4 Global Positioning System (GPS)

- Surface mounted
- Measures change in position of the arch crown and each arch base
- Allow the determination of absolute movements of the arch crown and arch bases
- Will be used to evaluate settlement, thermal effects, and lateral deflection of the arch crown during wind events
- Accuracy on the order of _____

10.1.5 Linear Potentiometer

- Surface mounted
- Measure relative movements at bearing locations
- Allow a time history of bearing movement to be constructed
- Used to evaluate thermal movement and effectiveness of the bearings

10.1.6 Chloride Penetration Sensor

- Embedded in the reinforced concrete pile caps
- Measure the concentration of chloride particles
- Allow a time history of chloride concentrations in the pile cap to be constructed
- Used to evaluate possible corrosion of reinforcing steel

10.1.7 Void Detection Sensor

- Wire sensors embedded for the entire length of a tendon
- Detect voids in grouted tendons

- Capable of differentiating between air voids and water voids
- Allow the mapping (size, location, and type) of voids in grouted tendons
- Used to evaluate the grouting techniques
- Require special holes in anchorages

10.1.8 Anemometer

- Surface mounted on temporary construction towers
- Measure site specific wind data during construction
- Allow site specific wind data to be compiled for the time that the temporary construction towers are in use
- Used to characterize site specific wind characteristics
- Used to document significant wind events during construction

10.1.9 Load Cell

- Measure force in selected support cables
- Custom built to specifications listed below
- Allow a time history of support cable forces to be constructed
- Used to determine bridge behavior under dead and live loads
- Used to track changes in behavior of the bridge as it ages

Table 10.1 Load Cell Specifications

Capacity	5,000,000 lb range
Environmental	Splash proof
Temperature range	-30 to 150 deg F
Non-linearity	1%FS or better
Dynamic reading capability	
Calibration certification required	
Must accommodate pressurized nitrogen system	
Include a pressure gauge with a range from 0 to 10 psig that reads in 0.1 psi or finer increments and is capable of communicating with data collection equipment	

CHAPTER 11

REPORTING OF DATA

11.1 Reporting of Data

The Indian River Inlet Bridge Monitoring Program involves installing many gages and sensors on and in the bridge to collect data. However, the raw data itself will be of little use until it is processed and interpreted. The results of the processing and interpretation must then be put into report form so that the information may be conveyed to interested parties to allow the goals of the instrumentation program to be met. This chapter includes the following for each stage of the project: the purpose of the reports, what should be included in these reports, when they should be issued, and what parties may be interested in the information contained in the reports.

11.1.1 Construction Stage

During the construction stage, the main purpose of collecting and analyzing data will be to

- Record the evolution of stresses, forces, and moments in members instrumented with strain gages
- Record the evolution of forces in support cables and temporary stays instrumented with load cells or accelerometers

- Record the evolution of settlements, relative movements, and displacements for members instrumented with linear potentiometers and GPS units
- Gather site-specific wind data from anemometers on the temporary towers

During this phase bimonthly reports will be issued by the University of Delaware to DelDOT. Information contained in these reports will include maximum and minimum values for each gage during the report period, as well as maximum and minimum forces, moments, displacements, and relative movements for each given member. Any unexpected values or significant discrepancies from predicted values will be discussed.

A final report will be issued at the end of construction. This report will include plots for each member that show the evolution of forces, moments, and displacements throughout the entire construction process. It will also include information on the wind data collected during construction. A discussion of unexpected results and discrepancies from expected values throughout the construction phase will be included.

The information contained in these reports will be of interest to DelDOT, the designer, the independent reviewer, and the contractor. The reports may be issued by DelDOT to these and other interested parties as deemed necessary.

11.1.2 Initial condition Stage

During the initial condition stage, the main goal of data collection and analysis will be to establish the baseline behavior of the bridge. Specific areas of interest include

- Determining the initial state of stress of the bridge under ambient conditions
- Quantifying the behavior of the bridge due to several load tests
- Quantifying the behavior of the bridge due to ambient traffic
- Quantifying the behavior of the bridge to any significant weather events such as high wind or large temperature changes that may occur

A report will be issued by the University of Delaware to DelDOT for each of the above bulleted areas of interest. Each report will contain maximum values for stress, force, moments, displacements, etc, for each member due to each specific event. Records of actual average responses will also be included and compared to predicted responses.

The information contained in these reports will be of interest to DelDOT, the designer, and the independent reviewer. The reports may be issued by DelDOT to these and other interested parties as deemed necessary.

11.1.3 Long-Term Monitoring

The main goals of the long-term monitoring portion of the program are to supplement standard visual inspection of the bridge as well as to determine whether the current behavior of the bridge differs from the baseline behavior. This will be accomplished by repeating load tests on the bridge and comparing current wind, traffic, and temperature events to similar past events.

A report will be issued at the first biannual inspection of the bridge. It will include data similar to the reports from the initial condition stage. However, the discussion of data will compare current values of response to past values of response

in addition to values of response predicted by design. Unexpected results will be discussed.

The information contained in these reports will be of interest to DelDOT, the designer, and the independent reviewer. The reports may be issued by DelDOT to these and other interested parties as deemed necessary.

The current contract between the University of Delaware and DelDOT ends after the first biannual inspection. Additional reports at biannual inspections may be handled under separate contracts as the need arises.

APPENDIX A

DEFINITIONS

A

B

BASE STATION – The temporary data logger that will be located at the base of the southern temporary tower that will log data from the instrumentation on the temporary tower and temporary stays during construction

C

CIBrE – Acronym for “Center for Innovative Bridge Engineering”

COMMUNICATION CABLE – Cable that connects a data acquisition system to the UD communications enclosure

COMMUNICATIONS CONDUIT – Conduit that runs from data acquisition systems to the UD communications enclosure for the purpose of housing communication cables

CONDUIT – Pipes installed for the purpose of protecting wires or cables

CONTRACTOR -

D

DATA ACQUISITION SYSTEM – Piece of data acquisition equipment housed in a metal box that gathers data from surrounding gauges through gauge wires then transmits that data to the UD communications enclosure through a communication

cable, permanent part of instrumentation program. Locations of data acquisition systems are specified in the bridge electrical plans.

DATA LOGGER – A piece of data acquisition equipment that is connected to gauge(s) by gauge wire and has the ability to communicate with and store data returned by the gauge(s), but does no communicate with the UD communications enclosure. Data loggers will be used to collect data during construction and will be removed upon installation of data acquisition systems.

DelDOT – Acronym for "Delaware Department of Transportation"

E

EMBEDDED GAUGES – Gauges which are encased in concrete (or grout) because they are internal to a segment and concrete has been cast around them

EVENT DATA – Data which is taken at a high frequency to capture effects from live loads such as wind and traffic

F

FIGG -

G

GAUGE – An instrument or means of measuring and/or testing

GAUGE WIRE – Wire which transmits data from a gauge to a data acquisition system or data logger

GPS – Acronym for "Global Positioning System"

H

I

INFORMATION KIOSK -

INSTALLATION EQUIPMENT – Apparatus for installing gauges, data acquisition systems, sister bars, etc. in the bridge, includes but not limited to wire ties, epoxy, bolts and nuts

INSTRUMENTATION PROGRAM - The portion of the monitoring program that is concerned with the purchase, installation, and set up of gauges, gauge wire, communication cable, data loggers, and data acquisition systems

IRIB – Acronym standing for “Indian River Inlet Bridge”

J

K

L

LOCAL CONDUIT - Conduit that runs from gauge locations to data acquisition systems for the purpose of housing gauge wire

M

MONITOR DATA – Data which is taken at a low frequency to capture effects from dead load or temperature

MONITORING PROGRAM – The project being undertaken by the University of Delaware which includes instrumenting the IRIB, collecting the data, interpreting the data, predicting data, and communicating results with concerned parties

N

O

P

PROVIDE ACCESS – Allow the University of Delaware to enter the bridge construction site, the bridge superstructure, temporary construction towers, pile caps, bearings, and fabrication yards for precast members throughout the construction process as detailed in this manual. Providing access means that the contractor will provide the means for University staff to physically get to the needed locations to install and read all instruments. It is expected that the contractor will provide the needed lifts, ladders, or other devices to allow this to happen. The period of time allotted for the University to access a section shall not overlap with the period of time allotted for access to any other section. During periods of access provided to the University appropriate lighting shall be provided by the contractor.

Q

R

S

SENSOR – Synonym of gauge

SURFACE MOUNTED GAUGES – Gauges which are mounted to the surface of a section by epoxy or other mechanical methods

T

TEMPORARY STAYS - The cables which act to temporarily support the arch rib during construction

THROUGH HOLE – Holes in specified segments that will be used to run gauge wire or install GPS units

U

UD COMMUNICATIONS ENCLOSURE – The enclosure to the west of the southern abutment that will house UD Communication equipment, see sheet 578 “Site Electrical Plan Bridge Electrical Service”

V

W

X

Y

Z

APPENDIX B

UNIVERSITY OF DELAWARE BRIDGE MONITORING PROGRAM SPECIAL PROVISIONS

B.1 Description

This work consists of providing the University of Delaware (University) access to the bridge construction site, the bridge superstructure, and temporary construction towers throughout the construction process. University staff will be installing internal and external gages in the bridge (both on site and at fabrication yards) and on the temporary towers for the purpose of quantifying bridge behavior during construction and after its completion. The University will be responsible for supplying and installing all of the sensors except for the support cable load cells (which will be purchased and installed by the Contractor). The Contractor shall provide University staff with access and means to reach the locations where the sensors will be installed and read at the appropriate construction phases. The Contractor will also be responsible for the movement of bulky equipment to required locations. In addition, the Contractor will be responsible for the proper installation of University of Delaware monitoring through holes shown in the plans, as well as communication conduit which are also shown in the plans. The Contractor will also be responsible for installing wire pulls in the communication and sensor conduits to allow sensor and communication wires to be fished through the conduit to needed locations. Details of the instrumentation program will be provided at a pre-bid

meeting. University staff will be independently safety certified and insured at no cost to the Contractor.

B.2 Construction Methods

B.2.1 General Note

University staff shall be notified at least four weeks prior to work on any of the items listed below. The University will provide the Contractor with a contact person at the outset of the project. In the following notes, "provide access" means that the Contractor will provide the means for University staff to physically get to the needed locations to install and read all instruments. It is expected that the Contractor will provide the needed lifts, ladders, or other devices to allow this to happen. The Contractor will also be expected to allow the University to draw power from on-site generators during selected activities.

B.2.2 Temporary Construction Tower

If temporary construction towers are used in the construction of the bridge, the University intends to install five anemometers at various heights on one of the construction towers. This instrumentation will collect wind speed data during construction. The University shall be provided access to the tower to install each of the anemometers (three hours per anemometer). The locations of the anemometers will be provided to the Contractor once the tower design is complete. The University will also instrument selected temporary support cables to monitor cable forces. Once a design of the tower and cables are complete, the University will specify which cables will be monitored and how the instrumentation will be installed. The monitoring will involve a combination of load cells and accelerometers. At locations that

accelerometers are used, they may require the University to have access to the support cable each time they are read. This would require lifts or other means to reach the upper cables. It is anticipated that the installation of instrumentation will require two hours per support cable. Wiring to both the anemometers and load cells/accelerometers will need to be run to a base station located at the base of the temporary tower, and the University shall be allowed access to that base station throughout the time that the instrumented towers are in use.

B.2.3 Bearings

Upon completion of the initial installation of the bridge bearings in the arch base, the University staff shall be provided with access to the bearings for a period of eight hours for the purpose if installing displacement instrumentation. As the south arch base is being constructed, a pull wire should be run through the communication conduit to allow the displacement wiring to be run up into the central beam. Once the arch base is complete, the University shall be provided access to pull the displacement wiring through the conduit and connect it to an appropriate data logger. This will require up to 3 hours. The University shall also be provided with access to the bearings for a period of four hours after they are unblocked at the later stages of the construction. The University shall also be provided with six hours of access each to the bearings at the end of each back span (at the abutments) immediately after their installation to install additional displacement instrumentation. If the contractor prefers, a communication conduit can be run down from the arch base bearing location and then underground directly to the University data kiosk. If this is done, a pull wire will need to be placed in the conduit to allow wiring to be pulled from the kiosk to the bearing location.

B.2.4 Internal Gauges

For the purpose of installing internal gages, the University staff shall be provided with four hours of access to each of the following sections S-C1U, S-C1D, S-T10U, S-T10D, S-T22U, S-T22D, S-T36U, TB-T1U, TB-T1D, TB-T19U, TB-T19D, TB-T37U, CENTRAL BEAM STA. 294+15.583, 1S-8, 1S-17, 2S-3U, 2S-3D, 2S-30U, 2S-30D, 2S-57U, 1N-8, 1N-17, 2N-3U, 2N-3D, 2N-30D, 2N-30U, and 2N-57U before each section is cast, but after the rebar cage is complete. Access and a means to reach any location of entire rebar cage at these sections shall be provided by the Contractor. During the installation, the University will be running wiring through the sections to an 8-inch by 8-inch by 6-inch block-out box that will also be installed by the University. Location and installation of the block-out boxes will be selected in consultation with the Contractor to ensure that it will not interfere with the construction methods. During casting of segments, the Contractor will ensure that the gages, wiring, and block-out boxes are not damaged. The University reserves the right to substitute alternative sections for instrumentation if the Delaware Department of Transportation (DelDOT) determines that such alternate sections need to be instrumented.

B.2.5 External Gauges and Data Acquisition Systems

In order to install external gages and attach wires to local data acquisition systems, an additional eight hours of access shall be provided to the interior of each section listed above (in the “Internal Gages” section) directly after it has cured (cast in place) or been placed (pre-cast). Four hours of access to the top and interior of the central beam at Stations 294+78.000 and 305+22.000 must be provided directly after these segments have cured (cast in place) or been placed (pre-cast).

B.2.6 Through-Holes

In order to run wiring for the instrumentation from the northbound and southbound sections into the tie beam and central beam, two-inch diameter through-holes have been specified in the plans for sections: TB-T1D, TB-T1U, TB-T19D, TB-T19U, TB-T37U, CENTRAL BEAM STA. 294+15.583, 1S-8, 2S-3U, 2S-3D, 2S-30U, 2S-30D, 2S-57U, 1N-8, 2N-3U, 2N-3D, 2N-30D, 2N-30U, and 2N-57U. It is the Contractors responsibility to make sure that the through-holes in tie-beams or central beams match up with the through-holes in the adjacent roadway sections (within a tolerance of 1/8 inch), and that the hole is clear (nominal 2 inch diameter opening) after the closure pour. The following table shows the tie beam/central beam sections and the adjacent roadway sections.

Table B.1 Though-Hole Sections

Tie-Beam Section/Central Beam Station	Adjacent Roadway Sections	
TB-T1D	NB	2N-3U
	SB	2S-3U
TB-T19D	NB	2N-30D
	SB	2S-30D
TB-T37U	NB	2N-57U
	SB	2S-57U
TB-T19U	NB	2N-30U
	SB	2S-30U
TB-T1U	NB	2N-3D
	SB	2S-3D
STA	NB	1N-8
	SB	1S-8

B.2.7 Load Cells

University staff will provide the Contractor with specifications for the load cells that will be installed at five arch anchorages and a list of potential suppliers. It will be the Contractors responsibility to purchase and install the load cells. The load cells are cylindrical donuts that will extend the anchorage zone upwards approximately 36 inches. Each load cell weighs several hundred pounds. The load cell assembly should also include instrumentation to allow the pressure sensing for the nitrogen gas system being used in the hanger cables. The cables to be monitored with the load cells are in sections S-A9U, S-A9D, S-A21U, S-A21D, S-A33U. These sections have been selected to avoid sections where temporary support cables are located. The University reserves the right to substitute alternative sections for installation of load cells if the location of temporary support cables change, or if the DelDOT determines that such alternate sections need to be instrumented. The Contractor will be responsible for getting the load cells from the ground to the appropriate locations as specified by the University, and installing both the load cells and nitrogen pressure sensors. Once installed, the University will need access to the load cell and adjacent instrumented sections to connect the load cell to the data acquisition system. This is anticipated to take four hours.

The required specifications for the load cells are as follows:

Capacity : 5,000,000 lb range

Environmental : Splash proof

Temperature range : -30 to 150 deg F

Non-linearity : 1%FS or better

Dynamic reading capability

Calibration certification required

Two potential suppliers are:

CTL Group

5400 Old Orchard Road

Skokie, IL 60077-1030

www.CTLGroup.com

Honeywell Sensotec

2080 Arlingate Lane

Columbus, Ohio, 43228

www.sensotec.com

B.2.8 Hardwire Connections

After external gages are placed at the sections listed below, an additional eight hours of access shall be provided to run instrumentation wire from that section to the closest data acquisition system installed (this is expected to occur right after the gages are installed):

- Upon completion of external gage installation at S-T10U, access shall be provided through the center of the arch from section S-C1U to S-T10U
- Upon completion of external gage installation at S-T10D, access shall be provided through the center of the arch from section S-C1D to S-T10D

•Upon completion of external gage installation at S-T22U, access shall be provided through the center of the arch from section S-T10U to S-T22U

•Upon completion of external gage installation at S-T22D, access shall be provided through the center of the arch from section S-T10D to S-T22D.

B.2.9 Data Collection

During bridge construction, the University staff will require periodic access to the all data acquisition systems installed in the arch and tie-beam segments in order to download data. It is anticipated that data will be retrieved from all installed data acquisition systems on a weekly basis. It will take no more than one hour per location to download data.

B.2.10 Corrosion Monitoring System

To monitor for future corrosion, instrumentation will be installed into both the arch base support cap and several post-tensioning tendons. The University will install four Corrosion Penetration Monitoring Probe (CPMP) units, one at each corner, of the south arch base cap beam prior to casting. The units will be tied off to the top mat of reinforcing. Monitoring instrumentation will also be installed in selected tendons of both the tie beam and roadway sections. Three post-tensioning tendons in the tie beam (at one section) and six post-tensioning tendons in the roadway sections (three in each of two sections) will be identified for monitoring at the outset of the construction. This instrumentation involves pulling a thin monitoring wire through the ducts along with the post-tensioning strands. The wire will require a separate access hole in the anchorage end plate. To demonstrate the installation of the tendon

corrosion system, and to provide calibration, the same system will be installed in the clear ducts used to demonstrate the grouting technique.

B.3 Basic of Payment

The cost of providing the University access for the purpose of installing monitoring equipment, as well as any labor, equipment, tools, etc. will be paid for in a lump sum.

APPENDIX C

UNIVERSITY OF DELAWARE ROADWAY MONITORING PROGRAM SPECIAL PROVISIONS

C.1 Description

This work consists of providing the University of Delaware (University) access to specific portions of the roadway abutment throughout the construction process. University staff will be installing strain gages on specific reinforcement panels for the purpose of monitoring the performance of an MSE wall both during construction and after its completion. The University will be responsible for supplying all sensors. The Contractor shall coordinate transfer of specified instrumented panels from the University, and provide University staff with access and means to reach the locations where the panels will be installed at the appropriate construction phase, as well as to provide access to read data from the sensors throughout the project. Details of the instrumentation program will be provided at a pre-bid meeting. University staff will be independently safety certified and insured at no cost to the Contractor.

C.2 Construction Methods

C.2.1 General Note

The Contractor should notify the PI about the planned installation of the specified panels at least 3 months in advance. The University will provide the Contractor with a contact person at the outset of the project. In the following notes,

"provide access" means that the Contractor will provide means for the University staff to physically get to the needed locations to install and later read all instruments. It is expected that the Contractor will provide the needed lifts, ladders, or other devices to allow this to happen. The Contractor will also be expected to allow the University to draw power from on-site generators during selected activities, as well as provide lighting if work needs to be conducted during night time hours.

C.3 MSE Wall Instrumentation

C.3.1 Location

The MSE wall being instrumented is very high and its anticipated settlement is most significant on South Bound SR1 (SB) between Station +292.00 and Station +293+00 (South Abutment). To avoid the effects of the abutment and yet obtain significant data, the desired section location for instrumentation would be 100 feet south of the South Abutment, on the SB side (this location coincides with the location for which MACTEC has specified other instruments to be placed).

C.3.2 Strain Gauge Installation and Reading

Strain gages will be attached to every 4^{th} layer of reinforcement at a spacing of 5 feet apart so as to obtain the strain distribution along that layer starting near its front end. Since geosynthetic material is likely to be used, attachment of strain gages for long-term field performance is not well established yet. Hence, this will be done at the University with the help of an experienced technician. The Contractor should notify the PI about the planned installation at least 3 months in advance of the start of work, and at that time provide the University with details of the exact type of reinforcement being used. The University will acquire and instrument the needed

layers of reinforcement. The University will contact the Contractor when the instrumented geosynthetic panels are done (at least 2 weeks prior to being needed for construction), and the Contractor will arrange for them to be taken back to the site for installation. The University will work with the Contractor to make sure that the panels are transported and stored in a manner so as to protect the gages. The Contractor shall provide access to the University at the construction site when each strain gaged panel is placed. The Contractor shall follow instructions provided by the University representative to ensure proper placement of each strain gaged panel. Fill over each strain gaged panel will be placed with care to prevent damaging the strain gages. The University will assist in protecting the gages. Electrical wires connected to the gages will be placed on top of each respective gaged reinforcement layer in a zigzag fashion extending through the face of the wall, and then these wires will be run down the face of the wall to a central box (provided by the University, footprint is approximately 2-feet by 2-feet). The central reading box will be located at least 10 feet away from the wall at the toe grade elevation. The Contractor and University will discuss and agree upon the ultimate location of the central box. The University will fabricate the reading box ensuring it is safe against unauthorized access as well as protected against the environmental degradation. The box will be installed by the University considering possible concerns by the contractor. The Contractor will be provided with instructions as to how to channel all electrical wires to the central box in a 2 foot deep trench (to protect the wires from construction equipment). The trench will be dug in a zigzag fashion to allow for excess wire thus allowing settlement and deformation without stressing the wires. For each strain gaged panel, all wires will be routed down the face of the wall, through the trench, and to the central box. In order to run the wires down

the wall face, a ladder or lift will need to be provided by the Contractor. After placement of all wires in the trench, it will be covered allowing for the normal use over the trench. The wires provided by the University will be shielded and water resistant thus needing no special protection after being embedded in the trench. Reading of data will be performed by the University. The Contractor will provide access to the data acquisition system throughout the project to allow data to be collected.

C.4 Basic of Payment

The cost of providing the University access for the purpose of installing monitoring equipment, as well as any labor, equipment, tools, etc. will be paid for in a lump sum.

APPENDIX D

SELECTED INDIAN RIVER INLET BRIDGE PLANS

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Appendix B

PHOTO RE-USE AGREEMENTS

B.1 Gnu Free Documentation License 1.2

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Version 1.2, November 2002

0. PREAMBLE

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