Full-Scale Implementation and Testing of Non-Prestressed Full-Depth Precast Bridge Deck Panels with Continuous Shear Pockets

by

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Abstract

The Alabama Department of Transportation (ALDOT) currently has over three miles of major interstate bridges near downtown Birmingham involving approximately 600,000 square feet of deck area with significant levels of deterioration. In an effort to minimize the impact of bridge deck replacement projects on the end user, it is necessary to rapidly replace deteriorated bridge decks with new precast concrete deck panels. By exploring new and innovative types of precast deck panel systems, it is possible to expedite deck replacement projects throughout Alabama.

In this study, a replacement bridge deck panel system utilizing non-prestressed full-depth precast concrete bridge deck panels with continuous shear pockets was investigated. First, the research team performed conceptual improvement, design, detailing, and fabrication studies on a specific deck replacement system (system CD-2) proposed by previous researchers. Next, an experimental program was carried out to construct and test a full-size bridge precast deck panel specimen that incorporated the newly refined deck replacement system. Based on the results of the static and cyclic load testing program, it was found that the modified CD-2 type deck panel system performed satisfactorily with regards to AASHTO serviceability requirements.

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Chapter 1: Introduction

1.1 Motivation

In the United States, more than 26% of the nation's bridges are currently categorized as either structurally deficient or functionally obsolete. Although these bridges do not pose immediate life safety risks, these deficiencies limit the functionality of bridges and require substantial repairs and remediation to return bridges to satisfactory operating conditions (American Society of Civil Engineers 2009). Of these deficient bridges, many are deck I-girder type bridges exhibiting substantial cracking and deterioration of their concrete decks.

The Alabama Department of Transportation (ALDOT) is no exception to this national trend and currently has over three miles of major interstate bridges near downtown Birmingham involving approximately 600,000 square feet of deck area with significant levels of deterioration (Oliver 1999). These bridges are approximately 45 years old and serve a large volume of annual average daily traffic (AADT). ALDOT has expressed the need for replacement of these deteriorated bridge decks, but seeks to minimize the potentially tremendous impact of these replacement projects on the end user. Any deck replacement methods utilized for remediation of these spans need to be as rapid as possible in order to minimize user costs associated with the inevitable disruptions of traffic.

Previous research sponsored by the Transportation Research Board has proposed a new type of full-depth precast concrete panel system (system CD-2) for use in both bridge deck replacement and new deck construction projects. This system utilizes multiple innovative concepts and unique details that are intended to reduce the overall duration required for installation (Badie and Tadros 2008). Although such a system offers great promise to reduce the duration of necessary replacement projects throughout the state of Alabama, system CD-2 has not yet been fully developed, fabricated, or tested to prove its viability as a deck replacement option.

1.2 Research Objectives

The objectives of this research project were as follows:

- Perform fabrication and erection studies on a specific precast bridge deck panel system (system CD-2) proposed by previous researchers (Badie and Tadros 2008).
- Perform service-load testing on a full-scale precast deck panel system in the laboratory to evaluate its in-service performance.

1.3 Tasks

In order to achieve the objectives of this research, the following tasks were completed:

1. Review previous research related to full-depth precast concrete bridge deck panels.
2. Perform conceptual improvement and design studies on system CD-2.

3. Perform fabrication and erection studies on a full-scale bridge-type specimen in laboratory.
4. Develop a service-load testing protocol.
5. Perform static and cyclic load testing in accordance with service-load protocol.
6. Analyze test data.
7. Present results regarding system performance.

1.4 Scope and Approach

The research described in this thesis is limited to the full-scale laboratory application of a CD-2 type deck system as applied to highway bridge structures. Accordingly, a system utilizing non-prestressed full-depth precast bridge deck panels with continuous shear pockets was investigated. No effort is made to examine or evaluate alternative bridge deck systems including, but not limited to, the following: prestressed or post-tensioned systems, partial-depth concrete systems, exodermic systems, or steel grid systems.

Results of this investigation exclusively reflect the behavior of the deck system as supported on rigid girders and do not include global superstructure behavioral effects (i.e. deflection of longitudinal supporting girders). This methodology is consistent with common bridge deck design and analysis practices that consider the deck system and supporting superstructure girders as independent structural elements (Barker and Puckett 2007).

1.5 Thesis Organization

This thesis is organized into eight chapters. Chapter 1 gives an introduction, presents the research objectives, and clarifies the scope of the project. Chapter 2 provides a brief explanation of rapid bridge deck replacement principals and how they intrinsically relate to the use of precast concrete panels, introduces limited necessary background information regarding the different types of precast concrete panels, and also includes a review of similar testing programs previously completed by others. Chapter 3 gives details of the conceptual improvement and design study process performed in order to improve and develop the previously proposed CD-2 system to the degree necessary to allow laboratory fabrication. Chapter 4 documents the fabrication and erection of the full-size bridge-type specimen in the laboratory. Chapter 5 provides a description of the load testing program including test setup, loading protocol, and instrumentation schemes. Chapters 6 and 7 present the results of the various load cases and include the analysis and presentation of these results. Finally, Chapter 8 provides a summary of the overall investigation, presents conclusions based on laboratory activities and data analysis, and also includes final recommendations regarding the use of the modified CD-2 type system.

Chapter 2: Background and Literature Review

2.1 Overview

This chapter provides a basic overview of precast concrete panel usage in bridge deck construction projects and also presents a discussion of recent research efforts devoted to this topic. Included is some basic background and terminology necessary for reader understanding, a brief discussion of the two major types of precast concrete panels commonly used in bridge deck construction, and a summary of previous research efforts relevant to the objectives of this investigation.

2.2 Precast Concrete Panels in Bridge Deck Construction

The process of installing a bridge deck is one of the most labor-intensive operations in bridge construction. Prefabrication of any portion of the deck system offers an opportunity to significantly reduce on-site construction time (Culmo 2011). One of the most logical and thus widely-explored methods for prefabricating bridge components is the use of precast concrete deck panels. These concrete panels are typically fabricated off-site at concrete casting facilities and transported to the project location for final installation.

The major challenge to the use of precast concrete bridge deck panels is associated with the complicated connections required after panel placement. These connections include both superstructure-to-precast panel connections and precast panel-to-panel connections. Sufficient superstructure-to-panel connections are required to assure that the bridge resists flexural loads by composite action as typically assumed in design. Adequate panel-to-panel connections are necessary to join adjacent panels together in multiple directions in order to mimic the structural behavior of a conventionally monolithic deck surface. Both types of connections discussed above are commonly further complicated by efforts to minimize the presence of joints on the top surface of the deck in order to improve ride surface quality and system durability.

2.3 Joint Terminology

For the purposes of this investigation, it is necessary to introduce the terminology used to refer to three types of joints. The main types of jointing details associated with the configuration utilized in this project include the following:

- Transverse joints
- Longitudinal joints
- Staged-construction joints.

These different joint types are described below and are also illustrated in Figure 2-1.

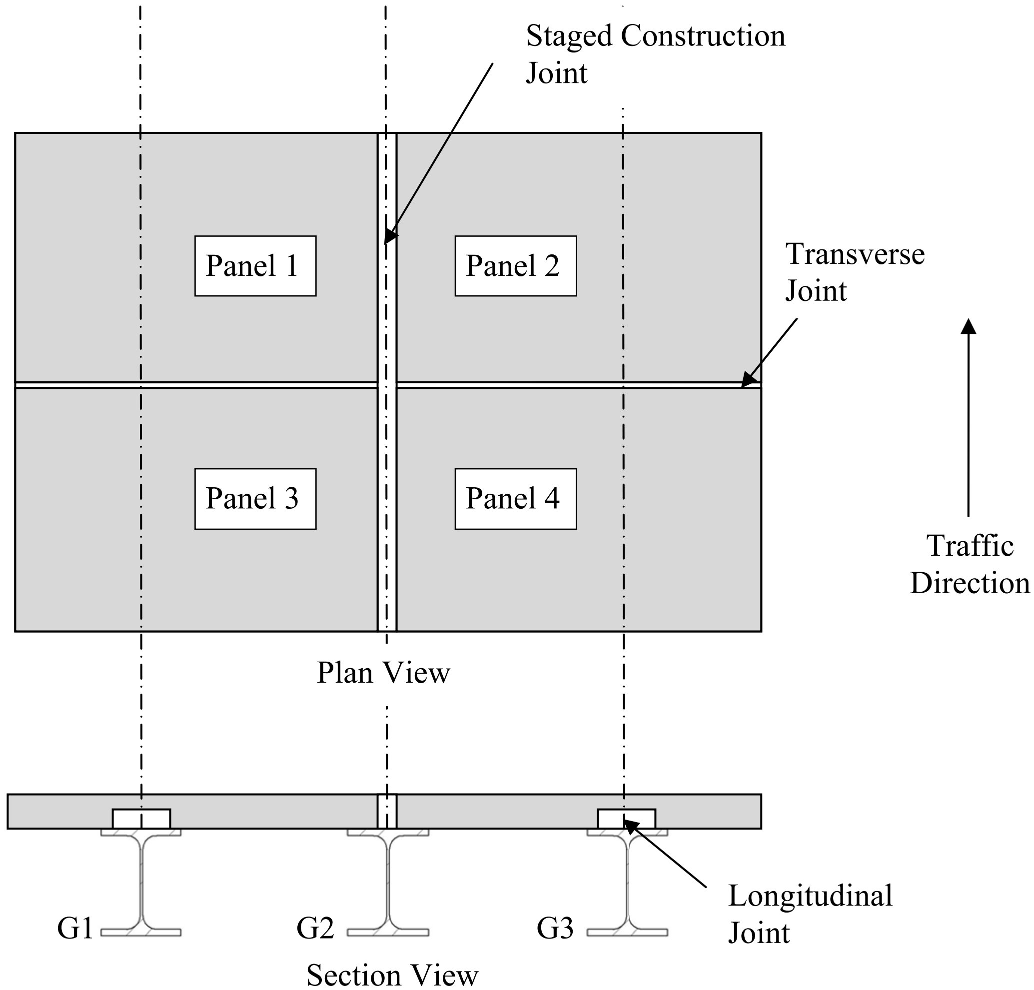


Figure 2-1: Joint Type Illustration

2.3.1 Transverse Joints

Transverse joints serve as panel-to-panel connections and are typically oriented transverse to the direction of superstructure girders and traffic. As such, these joints are typically unsupported from below except where they pass over superstructure girders. These joints typically consist of both (1) a method to connect or splice steel reinforcing across the joint and (2) a shear key intended to transfer shear across the joint.

2.3.2 Longitudinal Joints

Longitudinal joints serve as deck-to-girder joints and are typically oriented in the direction of traffic and occur along the top of girders. These joints are typically designed to provide full-composite action between girders and deck systems. Longitudinal joints are commonly the most complex and time-consuming connections on precast deck installation projects and are the subject of extensive research. A recent trend by designers is to include blind longitudinal joints, or joints that are not visible from the top surface of the deck (Badie and Tadros 2008). Such joints are shown above girders G1 and G3 in Figure 2-1.

2.3.3 Staged Construction Joints

Staged construction joints are a special kind of longitudinal joint. Similar to a typical longitudinal joint, these joints serve as deck-to-girder joints. However, staged construction joints also serve the added purpose of connecting adjacent deck panels. This unique type of joint is common on bridge deck replacement projects where work is sequenced to allow undisturbed traffic in adjacent travel lanes during construction. Such a joint is shown above girder G2 in Figure 2-1. Staged construction joints typically consist of (1) a method to connect or splice steel reinforcing across the panel-to-panel joint and (2) a method to provide “fully composite” action between supporting girders and deck panels. In contrast to the transverse joint discussed above, staged construction joints do not typically include a shear key because the entire length of the joint is supported from below by the girder top flange.

2.4 Partial-Depth Panels

Among the earliest applications of precast deck panels was the use of partial-depth concrete panels in order to expedite bridge deck construction projects. These partial-depth reinforced or prestressed panels are typically four inches thick and are placed on top of the beams on interior bays (Culmo 2011). After multiple panels are set in place, a top layer of conventionally reinforced concrete is installed to finish the composite decking system. These partial-depth panels are advantageous because they serve as stay-in-place formwork for the upper slab and also act as structural members useful in resisting traffic loads. Various municipalities have successfully utilized and demonstrated the merits of partial-depth precast concrete deck panels throughout the past century. For example, 85% of all bridges built in Texas utilize the partial-depth bridge deck panels shown below in Figure 2-2 (Culmo 2011).

Figure 2-2: Partial-Depth Deck System (Culmo 2011)

As a first exploration into the field of rapid bridge deck replacement, the use of partial-depth panel systems was vital. However, in comparison to many alternative systems offered today, partial-depth systems exhibit certain intrinsic shortcomings. For instance, partial-depth systems do not allow for panels that are continuous across multiple girder spans. Also, partial-depth systems still require time-consuming field placement of concrete. As a result, the use of partial-depth panel systems for deck replacement projects has been virtually eclipsed in recent years by the use of newly-developed full-depth panel systems, which promise to further reduce deck replacement project durations.

2.5 Full-Depth Panels

After an initial popularity and positive response to the reduced deck replacement project durations achieved by partial-depth panel systems, substantial research and development efforts began to focus on other methods to further decrease deck replacement timelines. Among the most popular methods was the development of full-depth precast concrete deck panel systems by different municipalities and researchers across the United States (Badie and Tadros 2008). Similar to partial-depth panels, full-depth systems are comprised of precast concrete panels that are fabricated off-site and transported to the project site for installation as shown in Figure 2-3.

Figure 2-3: Full-Depth Deck Panel System (Culmo 2011)

Once panels are set in final position, grout material is installed into small closure ports between panels and into specially-designed deck-to-superstructure joints in order to complete the installation. Substantial time savings are achieved by avoiding the need for a cast-in-place topping slab as required in partial-depth construction.

While these full-depth systems do offer marked reductions in construction timelines, much additional effort and ingenuity is typically required in panel design to provide satisfactory system performance in the final installed condition. This unique challenge, to design viable and innovative full-depth decking solutions, has attracted attention from many leading engineers and researchers in recent years (Badie and Tadros 2008).

2.6 National Cooperative Highway Research Program (NCHRP) Report 584

In order to document the state of the art and to further develop and encourage the use of full-depth precast bridge deck panel systems, a joint research project was sponsored by the National Cooperative Highway Research Program (NCHRP) in 2003. Funding was provided from both the Federal Highway Administration (FHWA) and the American Association of State Highway and Transportation Officials (AASHTO). This joint research project culminated in 2007 with the publication of NCHRP Report 584 – Full-Depth Precast Concrete Bridge Deck Panel Systems. Included in this report are the following:

1. An extensive literature review and survey of transportation professionals;
2. Identification of common weaknesses in current full-depth systems;
3. Development of two new full-depth systems utilizing innovative details to address identified weaknesses;
4. Validation of one newly developed system by an experimental and analytical program;
5. Recommendations for design codes regarding full-depth panel systems.

2.6.1 Relevant Research Results

Two new full-depth precast bridge deck panel systems were developed as a result of the efforts of NCHRP Report 584. The systems were identified as CD-1 and CD-2. The general features of the two systems are summarized in the table below.

Table 2-1: General Features of NCHRP Report Designs (Badie and Tadros 2008)

System Designation	CD-1	CD-2
Reinforcement Type: Transverse Longitudinal	 Pretensioned Conventional	 Conventional Conventional
Supporting girder and construction type: New construction projects Deck replacement projects Alteration to existing shear connectors	 Steel or concrete girders Steel girders High	 Steel or concrete girders Steel or concrete girders Minimum
Made composite with the girder	Yes	Yes
Longitudinal posttensioning	No	No
Use of overlay	No	No
Panel can be crowned to match the bridge profile	No	Yes
Notes	Two panel-to-panel connection details were developed for this system (CD-1A and CD-1B). A full-scale bridge mockup using this system was constructed and tested in this project.	This system was not tested in this project.

Both of these systems satisfied the following required conditions:

- They do not include longitudinal post-tensioning to attach panels.
- They do not use proprietary products.
- The precast panels can be fabricated off the construction site at a precast yard.
- Any grouted areas are minimized and kept as hidden as possible.
- No overlay is required.

System CD-1 was the main focus of the NCHRP Report 584 author's research efforts. The following components for system CD-1 were included in the research: fully developed system details, comprehensive design calculations, and a comprehensive construction and experimental testing program. As part of this testing program, a full-size bridge specimen was constructed and subjected to service-level loadings as shown below in Figure 2-4. This testing program validated system CD-1 as a viable full-depth system, and it has accordingly enjoyed widespread implementation in recent years (Culmo 2011).

Figure 2-4: Constructed System CD-1 Specimen and Test Setup (Badie and Tadros 2008)

Although system CD-2 was also conceptually developed during their investigation, it received limited attention from the study authors. System CD-2 is clearly the more radical of the two newly developed systems and offers a fundamentally different approach to the implementation of full-depth deck panels. Significant advantages of the CD-2 system over the CD-1 system include the following: minimal alteration to existing shear connectors on deck replacement projects, the use of conventional reinforcement, and the capability to crown panels to match the bridge profile. However, full design, detail development, and implementation of this system was not included in the scope of the NCHRP work and Report 584. As such, system CD-2 is the focus of the remainder of the investigations included in this thesis. A fully detailed description of system CD-2 as originally proposed by previous researchers is introduced at the beginning of Chapter 3.

2.7 Accelerated Bridge Construction (ABC) Final Manual

In late 2011, the FHWA released a publication entitled Accelerated Bridge Construction – Experience in Design, Fabrication, and Erection of Prefabricated Bridge Elements and Systems (Culmo 2011). This publication was developed for the purpose of encouraging the use of prefabricated bridge elements and systems (PBES) as part of accelerated construction projects. Sections were included in this manual that provided extensive coverage of precast concrete deck panel systems and served to summarize the state of the art at the time of publication.

Although an extensive discussion regarding the development and field implementation of NCHRP system CD-1 was presented, no mention was made of any efforts to develop or validate the use of the originally-proposed CD-2 system. This lack of information regarding system CD-2 demonstrates that little to no research effort has been devoted to the development and testing of system CD-2 since the original publication of NCHRP Report 584 in 2008.

Chapter 3: Deck System Description, Conceptual Improvement, and Design Study

3.1 Overview

This chapter begins with a detailed description of the CD-2 deck panel system as originally proposed by researchers in NCHRP Report 584. It is necessary for readers to develop a full understanding of the previously proposed CD-2 system in order to enable a thorough understanding of conceptual changes and improvements undertaken as part of this investigation. After a summary of the most significant improvements made to the originally proposed CD-2 system, a discussion regarding the structural design of the modified CD-2 system included in this investigation is presented.

3.2 NCHRP Report System CD-2 Description

The following section is intended to give a detailed description of the CD-2 system as originally proposed in the NCHRP Report 584. For clarity, the geometry and configuration of the installed panels will be introduced first. Next, a discussion of the mild steel reinforcing will be presented. Finally, the two types of connections included in this system will be introduced and reviewed.

3.2.1 Panel Geometry and Configuration

System CD-2 as originally proposed was intended for application as a full bridge width precast deck panel installation method. In this method, precast bridge deck panels are installed perpendicular to the direction of traffic as shown in Figure 3-1 to form the completed bridge deck.

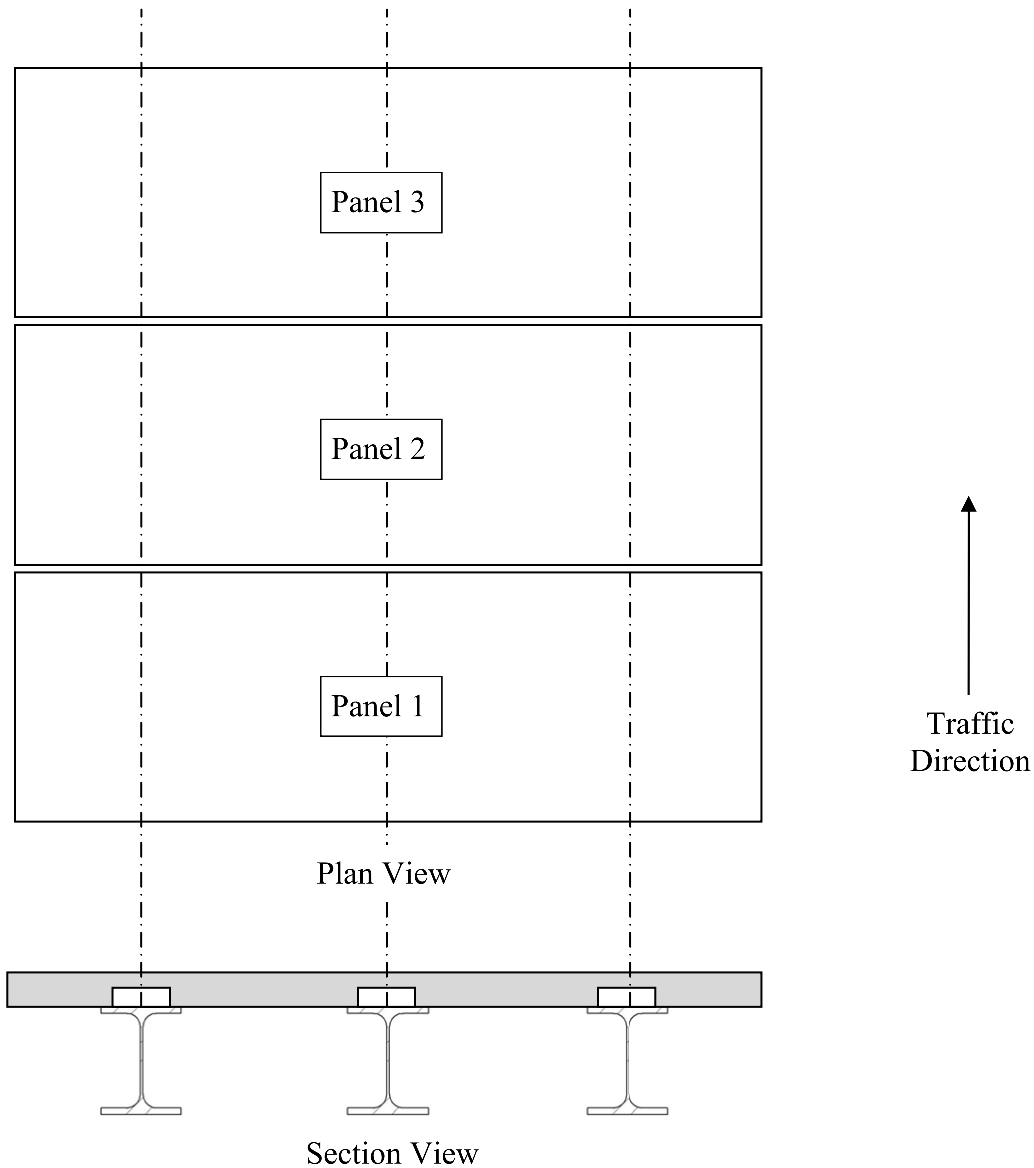


Figure 3-1: Installation Intent of NCHRP System CD-2

A detailed plan and elevation of the originally-proposed system CD-2 is shown in Figure 3-2. In this case, the system is utilized as continuous across four longitudinal girders spaced at 12’-0” with cantilevers of 4’-0” length on each side. The width of each panel in its minor dimension (parallel to traffic) was proposed as 8’-11” in order to allow panel transportation by a typical tractor-trailer truck. The thickness of the deck panels as proposed in the system was 8 ¼” in order to allow for a ¼” sacrificial grinding surface after panel installations.

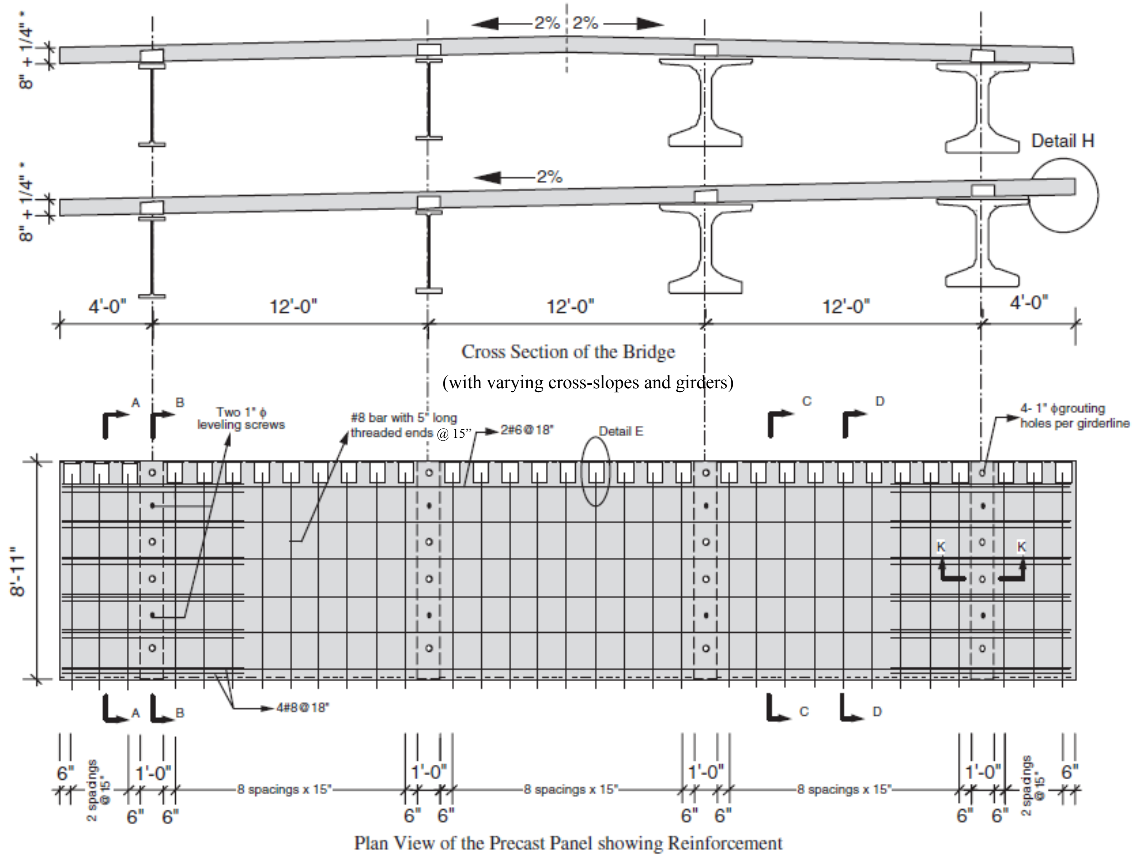


Figure 3-2: NCHRP System CD-2 Plan and Section (Badie and Tadros 2008)

3.2.2 Panel Reinforcing

As originally proposed in the NCHRP Report 584, the full-depth precast panels include three layers of mild reinforcing steel. In the transverse (perpendicular to traffic) direction, both top and bottom layers of reinforcing are present and continuous.

Including both layers of reinforcing in this direction is expected, as bridge deck panels are customarily designed to act in one-way flexure across supporting girders. For areas spanning between girder lines in this direction, the reinforcement configuration proposed is #6 bars at 18” spacing in both the top and bottom layer as seen in Sections C-C and D-D of Figure 3-3. As expected, the cantilevered edge sections of the slab are heavily reinforced in the top reinforcement layer with two additional #8 bars bundled to each previously mentioned #6 bar as shown in Section A-A and B-B of Figure 3-3.

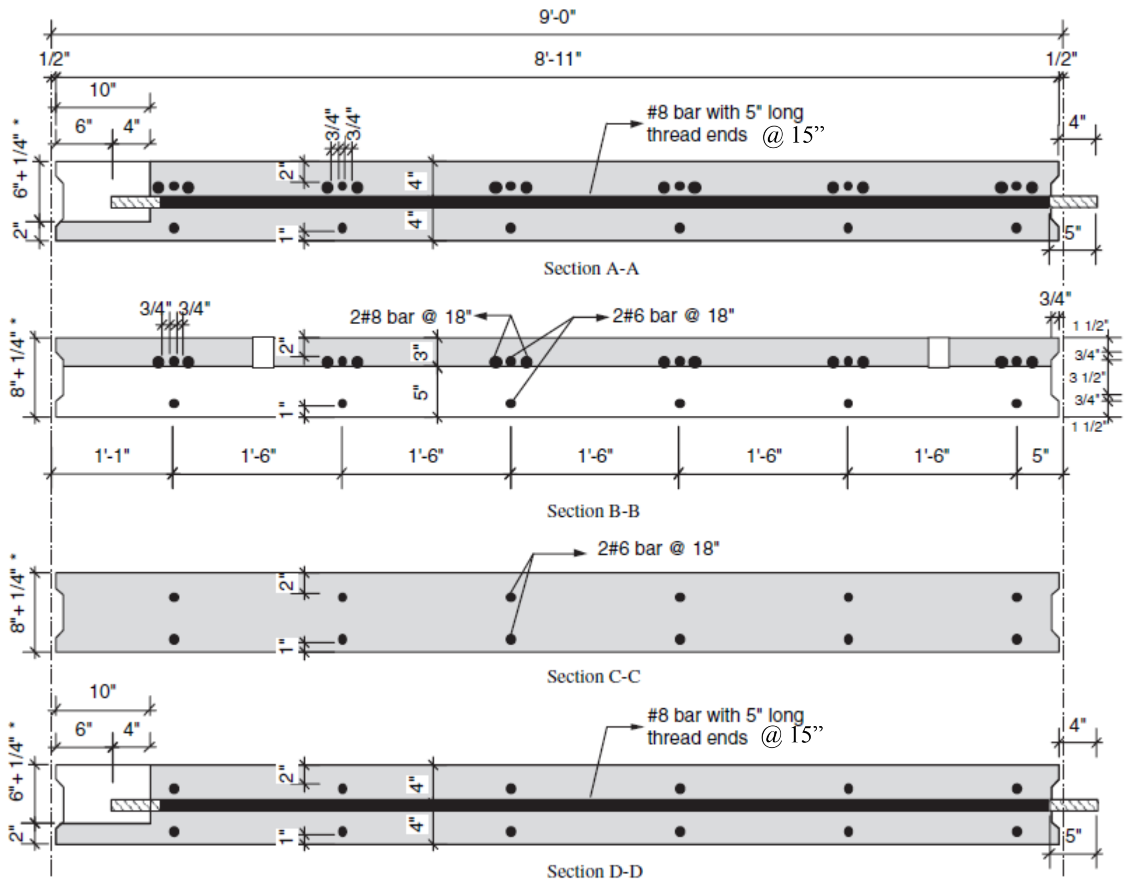


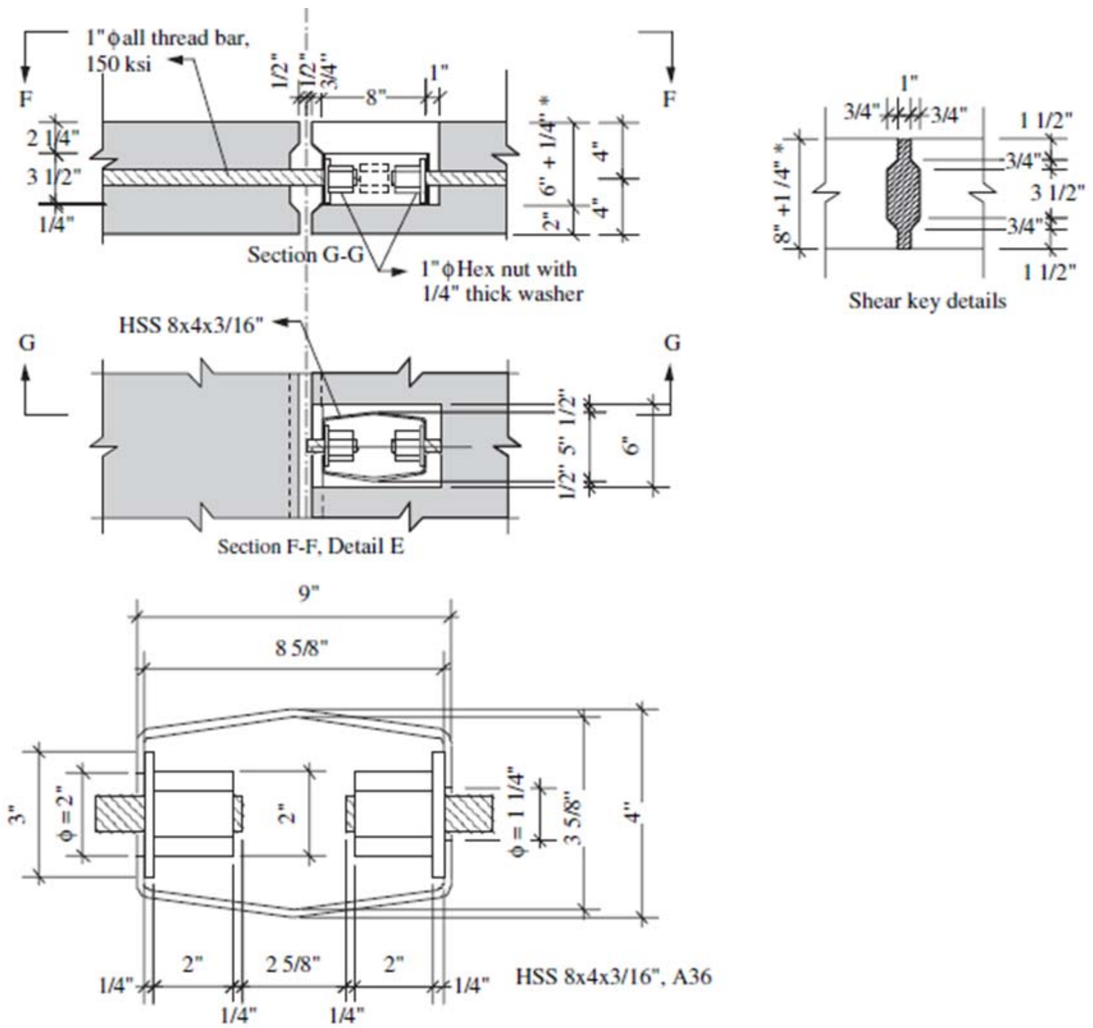
Figure 3-3: NCHRP System CD-2 Cross Sections (Badie and Tadros 2008)

In the longitudinal (parallel to traffic) direction, a single layer of mild reinforcing steel is located at the mid-height of the precast deck panels as visible in Section D-D below. In the originally proposed CD-2 configuration, this reinforcing consists of #8 either partially or fully threaded reinforcing bars spaced at approximately 15” on center.

3.2.3 Panel Transverse Connections

As previously defined in Chapter 2, the transverse joint is the panel-to-panel joint perpendicular to the direction of traffic. As is common for transverse joints, this joint detail is responsible for transmitting both shear and moment effects between adjacent panels. Accordingly, both a shear key detail and a tensile reinforcing steel splice are included in the system CD-2 connection details shown in Figure 3-4.

The grouted female-to-female shear key detail utilized in this system transmits shear forces between adjacent panels. This shear key runs the entire width of the panel, is located at approximately the slab mid-height, and includes panel depressions up to ¾” as shown in Figure 3-4. Extensive research has validated the use of grouted female-to-female shear key joints as being the most practical and durable joint type among available alternatives (Badie and Tadros 2008). In order to achieve best joint performance, it is commonly recommended that the concrete shear key detail be sandblasted to expose aggregate prior to the installation of grout.



HSS 8x4x3/16" Details

Figure 3-4: NCHRP System CD-2 Transverse Connection Details (Badie and Tadros 2008)

The tensile reinforcing splice utilized in system CD-2 consists of a slot-cut structural steel shape, which acts a coupler joining threaded rods protruding from adjacent panels. As originally proposed, a rectangular hollow structural section (HSS) is slot-cut to a desired height and bulged along its major direction to form a coupler. Holes are then drilled in each side of the section to accept threaded bars from each adjacent panel. For clarity, a rendering of the originally proposed HSS coupler is illustrated in Figure 3-5.

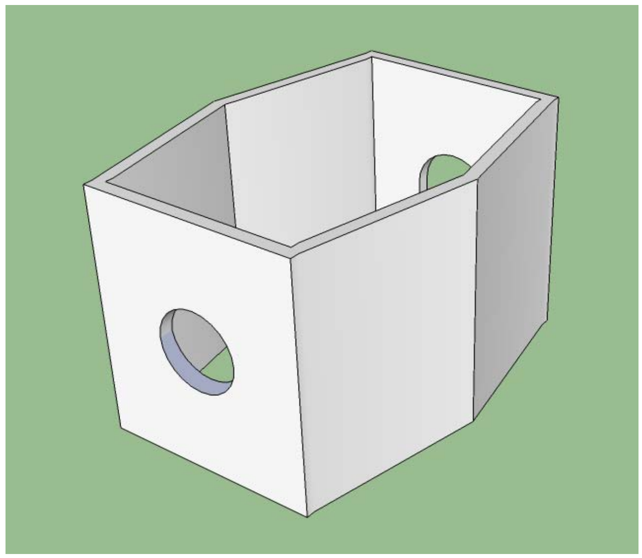


Figure 3-5: NCHRP System CD-2 Bulged HSS Coupler Rendering

After the threaded bars are positioned in the coupler holes, washers and locking nuts are threaded to join the panels together. Finally, a non-shrink grout is installed to fill joint void spaces and complete the installation. As detailed and shown above in Sections G-G and F-F of Figure 3-4, the tensile reinforcing splice is not symmetric across the joint, with the splice being located completely on one side of the transverse joint.

3.2.4 Panel Longitudinal Connections

The longitudinal connection proposed for system CD-2 is intended as a superstructure-to-panel connection that achieves full composite behavior between components via the use of headed shear connectors welded to the steel girder top flanges. Most commonly, this connection type is achieved by the use of "discrete" clusters of shear connectors that fit into preplanned void locations in the precast panel system as shown below in Figure 3-6. This application type is typically called a "discrete" shear pocket connection and is widely utilized.

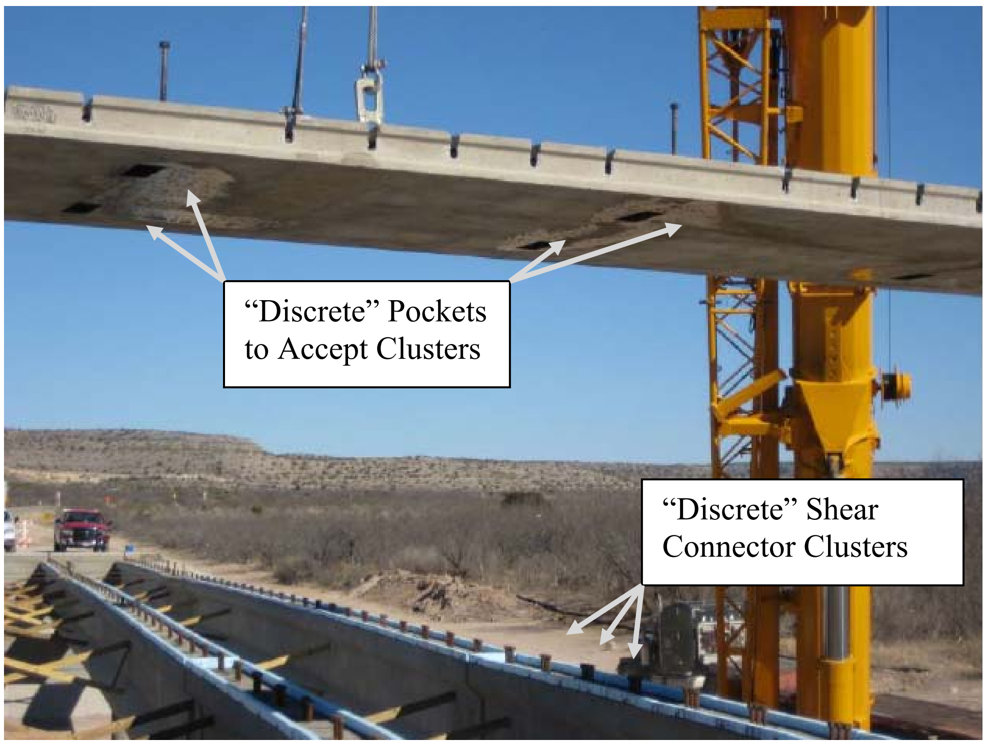


Figure 3-6: Typical "Discrete" Shear Pocket Longitudinal Joint (Culmo 2011)

In contrast to most presently-installed systems, system CD-2 proposes the use of an innovative "continuous" shear pocket detail. Instead of requiring shear connectors to be clustered into "discrete" groups, system CD-2 utilizes a "continuous" partial-depth pocket to accommodate connectors along the entire girder length as shown in Figure 3-7.

As shown in Figure 3-7, only 3 in. of concrete slab remains intact above the “continuous” pocket. This longitudinal joint configuration is not visible from the top surface of the deck panel and is therefore is referred to as a “blind” connection detail. To complete the installation, non-shrink grout is injected into the longitudinal joint through small preplanned ports from the deck surface above.

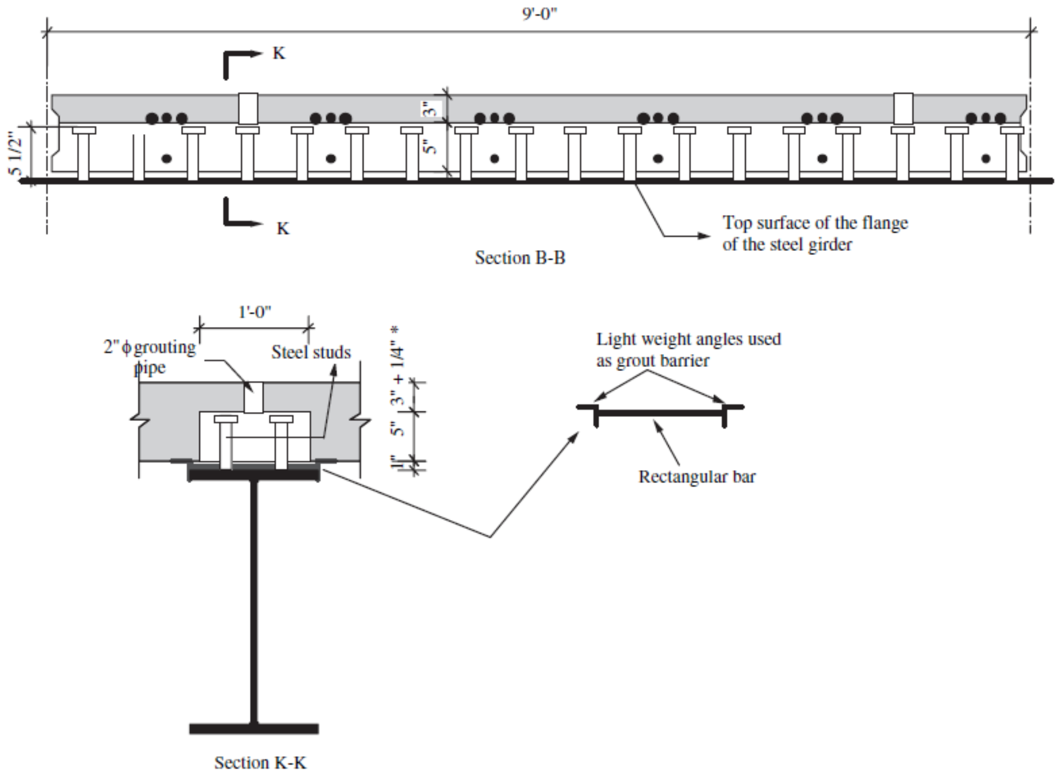


Figure 3-7: NCHRP System CD-2 Longitudinal Connection Details (Badie and Tadros 2008)

This detail is without question the most unique and controversial detail associated with the originally proposed CD-2 system. Although this “continuous” pocket longitudinal joint promises substantial installation time savings if practical, many are skeptical of the durability of panels that include a full-length void through approximately

65% of their thickness. To date, a "continuous" shear pocket system has not been implemented or tested and is therefore not yet accepted as a viable option by the professional engineering community.

3.3 Key Conceptual Changes and Improvements

The following section includes key conceptual changes and improvements made to the originally-proposed CD-2 as a result of a thorough investigation. These revisions reflect an attempt to improve and transform the original CD-2 system into a revised CD-2 system which is practical for application to the aging Alabama bridge infrastructure. The following topics are included in this section: geometric changes made to meet Alabama standard bridge applications, modifications made to the originally-proposed transverse joint, and the development of a new staged construction joint type.

3.3.1 Modifications per ALDOT Standard Practices

The geometries, dimensions, and various material properties proposed in NCHRP Report 584 are different than those typically utilized in highway bridge construction in the state of Alabama. In order to assure that the revised CD-2 system is readily applicable to rapid deck replacement projects on Alabama bridges, various modifications were made to the originally-proposed system. The majority of these changes were aimed at accommodating standard ALDOT practices and details available from the "ALDOT Bridge Bureau Structures Design and Detail Manual" and also from available as-built bridge construction drawings acquired by the research team. Although an effort was made in all cases, some standard ALDOT details could not be accommodated due to the innovative nature and application of a CD-2 type precast deck panel system.

As originally proposed, the CD-2 system spanned transversely between girders spaced at 12’-0” on center. This dimension was reduced to the more commonly used 8’-0” spacing typical of Alabama infrastructure bridges. In addition, the originally proposed 4’-0” deck cantilever length was reduced to a more common 3’-6” length as typical of ALDOT standard details (Alabama Department of Transportation 2008). The originally proposed bridge deck thickness of 8 ¼” was similar to typical ALDOT dimensions and was thus preserved for the revised CD-2 system design. A typical schematic of an ALDOT steel girder bridge is shown in Figure 3-8.

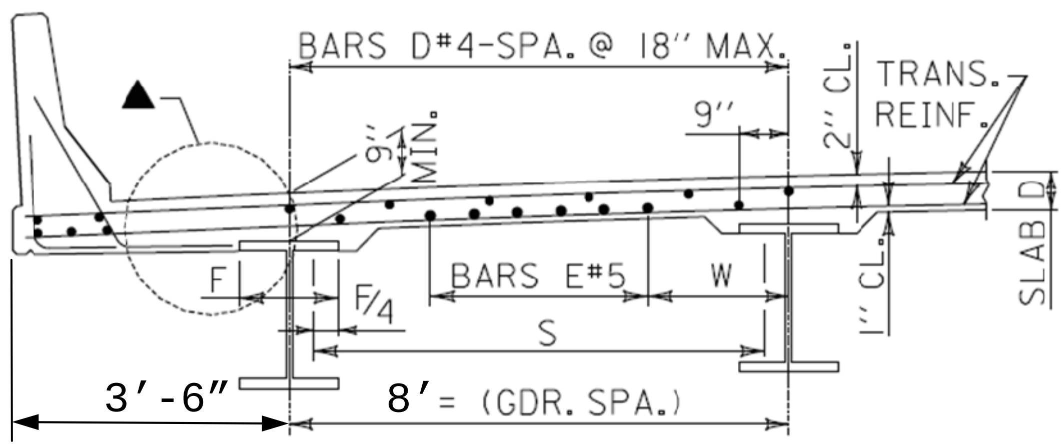


Figure 3-8: Standard ALDOT Steel Girder Bridge Geometry (Alabama Department of Transportation 2008)

In addition to the geometric changes outlined above, an effort was also made to conform to the material and construction specifications included in the ALDOT Bridge

Bureau specifications wherever possible. For instance, concrete strengths, steel reinforcing strengths, standard barrier rail details, and concrete cover requirements per ALDOT specifications were implemented into the revised CD-2 system. An effort was also made to satisfy current ALDOT reinforcing bar spacing limitations wherever possible in the newly developed system.

In choosing the configuration of the revised CD-2 system, it was especially important that the new system be compatible with existing bridges that would be likely candidates for rapid bridge deck replacement in the future. As a result, it was of paramount importance that the revised system be compatible with existing girder and headed shear-connector configurations. In the state of Alabama, girder top flanges can be as narrow as 12" wide and currently require a minimum of 5" headed shear connectors welded to the top flanges (ALDOT 2008). Figure 3-9 illustrates typical configurations utilized on rapid deck replacement candidate bridges throughout the state of Alabama. Although these configurations are efficient for the initial construction of a cast-in-place bridge deck, they pose significant geometric challenges to overcome in the development of a precast deck system.

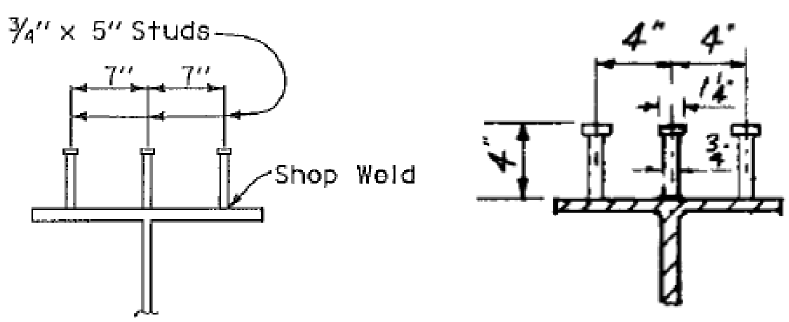


Figure 3-9: Common Shear Connector Configurations (ALDOT 2008)

3.3.2 Transverse Joint Modifications

Modifications to the originally-proposed transverse joint were initially explored as a method to increase placement and connection tolerances for the CD-2 system. However, after a detailed investigation of the original joint details, additional modifications were undertaken in an attempt to improve both the durability and ease of constructability of the joint. These changes included the following: changes to the location of the tension splice coupler, modification of the originally proposed HSS coupler, and revisions to the shape of the HSS coupler.

As outlined in the beginning of this chapter, the originally-proposed transverse joint details as shown in Figure 3-4 position the tension splice coupler asymmetric to the transverse joint on a single panel edge. The research team chose to shift the tension coupler location to a symmetric location about the transverse joint in an attempt to both increase grout continuity across the joint as well as to increase the potential for symmetric moment transfer across the detail. An early concept rendering is shown in Figure 3-10.

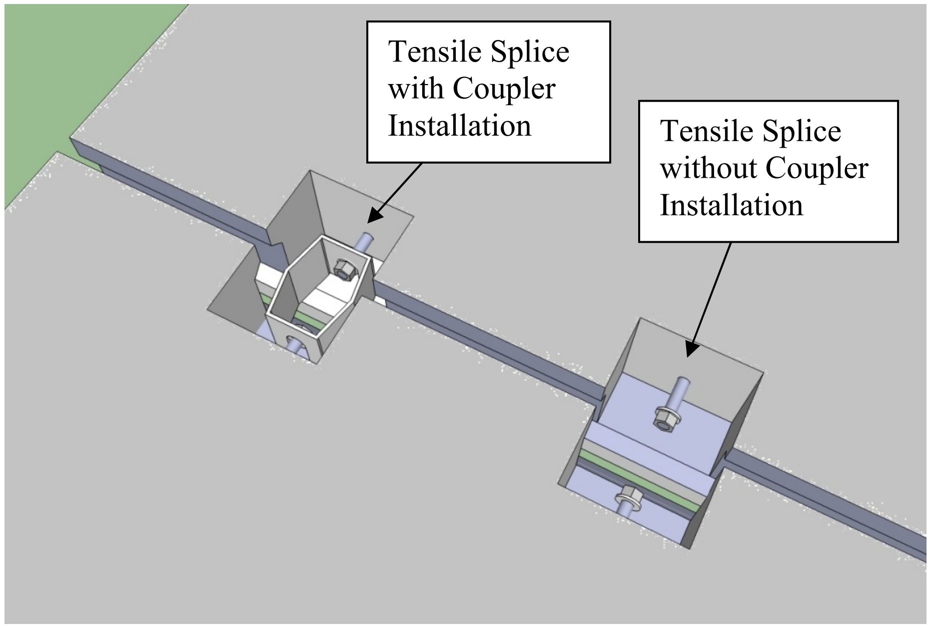


Figure 3-10: Relocated Transverse Joint Tensile Splices

Another major modification to the transverse joint was the modification of the HSS coupler to include a slot to allow installation after panel placement. As previously proposed details for this joint show, the HSS coupler was intended to be installed on a panel prior to the placement of an adjacent panel. As adjacent panels were positioned, protruding threaded rebar were to be aligned and inserted into the small HSS coupler circular holes along the entire panel length. This originally proposed method of installation requires extremely tight handling tolerances and operator skill to accomplish. By the addition of an installation slot in the coupler, a single worker can easily make the tension splice from above after final panel positioning as shown in Figure 3-11.

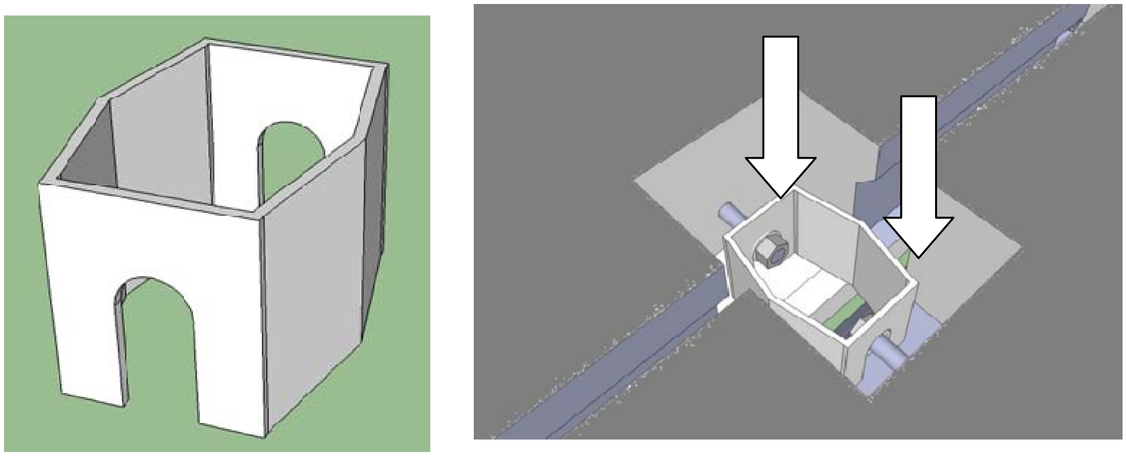


Figure 3-11: Slotted Tensile Coupler Pre- (Left) and Post-Installation (Right)

A final improvement to the transverse joint addressed the bulging of the HSS coupler as suggested by the originally proposed system CD-2 design. The original intent of bulging the straight HSS section was to increase confinement effects inside the grouted coupler area when resisting a tension load. However, previous research results regarding the use of a bulged HSS connection in similar applications suggest that only relatively minimal gains are achieved by the effects of bulging couplers (Badie and Tadros 2008). In addition, despite correspondence with various structural steel fabricators, the research team could not find an efficient and feasible method to successfully bulge the HSS couplers on a large scale. Although this type of bulging is commonly accomplished in larger sections by pneumatic jacks, the reduced size of the CD-2 couplers prohibited the use of this method. In the end, the research team chose to utilize straight HSS couplers for simplicity in design, analysis, and construction.

3.3.3 Addition of Staged Longitudinal Construction Joint

The largest challenge of implementing a CD-2 type system to rapid bridge deck replacement is the requirement to develop a staged construction detail. This joint detail must accommodate all the requirements of the transverse joint, while also acting as a longitudinal joint to attach deck panels to the superstructure. Development of a staged construction joint for the CD-2 system proved to be a significant challenge due to the congestion of reinforcing bars, tensile splices, and headed shear connectors in the vicinity.

When a staged construction joint is located over an interior girder, the joint detail must achieve the following:

1. Provide sufficient bearing area for adjacent precast panel edges despite often narrow girder top flanges.
2. Accommodate sequenced construction demands by minimizing reinforcing bars protruding beyond the end of precast panels.
3. Splice top reinforcing tensile steel across the longitudinal panel-to-panel joint.
4. Anchor both deck panels to the girder below to achieve composite action of the deck-girder system.
5. Accommodate transverse joint intersections at “four corner” panel-to-panel joint locations.
6. Minimize grouted connection areas visible from the top surface of the deck.

The research team successfully developed a staged construction joint concept which achieved the above-referenced goals. Included in the joint were the following features: a tensile steel splice similar to that used in the transverse joint except utilizing confining steel stirrups, a “continuous” pocket detail to accommodate headed shear connectors, and unique details to address conflicts with intersecting transverse joints. Although presented in full detail later in this thesis, a view of the joint prior to placement of confining stirrups and the closure concrete/grout is shown in Figure 3-12.

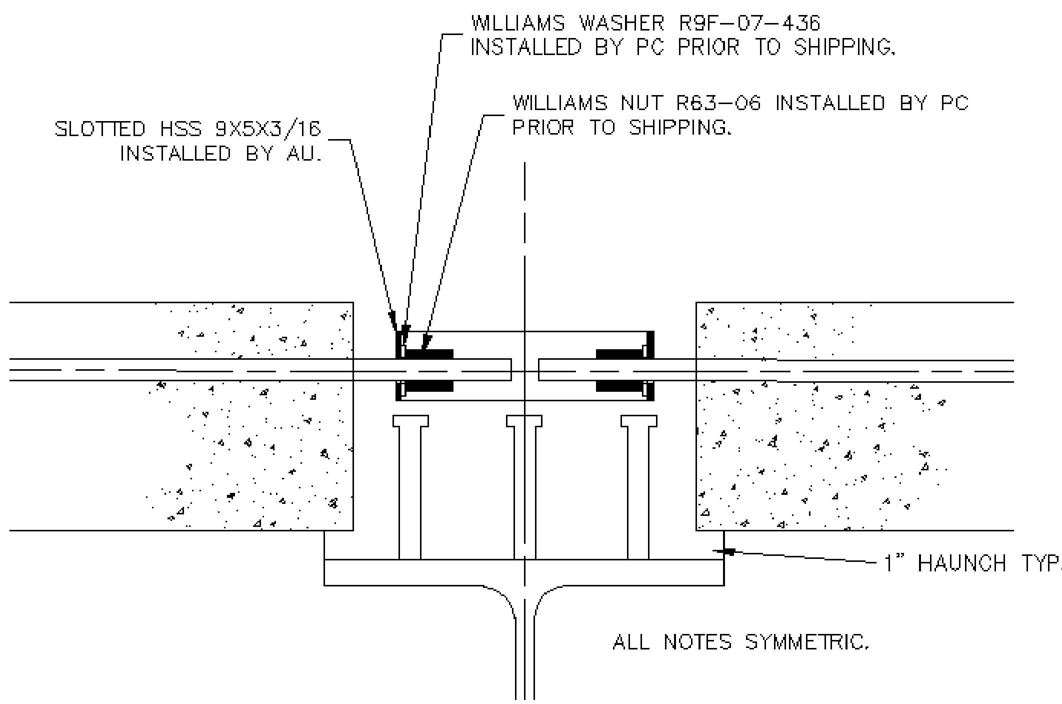


Figure 3-12: Staged Longitudinal Construction Joint Detail Elevation Prior to Placement of Confining Stirrups and Closure Grout

3.4 Design Study

The original details of the CD-2 system included in NCHRP Report 584 did not contain extensive design details or calculations for the proposed CD-2 system. It became evident to the research team that a CD-2 system design study was required in order to design an experimental test specimen and also to provide design guidance to potential future system users. Although various aspects of the structural design are summarized in this section, select design calculations which conform to the 5th Edition of the *AASHTO LRFD Bridge Design Specifications* (AASHTO 2010) are provided in Appendix A of this document.

3.4.1 Material Properties

Material properties as used in the design and construction of the test specimens included in this investigation were selected by a comprehensive review of previous similar research and as governed by regional availability of products. Final material properties of the products chosen for use in this investigation are summarized in Table 3-1.

Table 3-1: Design Material Properties

Material	Description	Strength
Precast Panel Concrete	ALDOT Mix PPM-039-08	f'_c = 4000 psi min
Threaded Reinforcing Steel and Fasteners	Williams Form Proprietary	f_y = 75 ksi
Deformed Bar Reinforcing Steel	ASTM A615	f_y = 60 ksi
Non-Shrink Grout	BASF SS Mortar	f'_m = 6000 psi
Steel Girder	Grade A992	f_y = 65 ksi
Steel HSS Coupler	A500 Grade B	f_y = 46 ksi

3.4.2 Deck Panel Reinforcing Steel Design Approach

The design of typical cast-in-place bridge deck slabs is typically performed assuming continuous one-way behavior of the deck system in a direction perpendicular to the superstructure girders (Barker and Puckett 2007). This design assumption remains valid for the design of precast deck panel systems as long as monolithic slab behavior is simulated by connections in the final installed state. Accordingly, the design of precast concrete deck panel reinforcing is not significantly more complicated than that of a typical cast-in-place bridge deck system. However, minor complications do arise due to the discontinuous reinforcing at panel joints, and thus the majority of additional design effort is often dedicated to connection design. For the CD-2 panel reinforcing in this investigation, various design methods were utilized for different portions of the system.

For interior span locations, the AASHTO Empirical Design Method was utilized to size transverse and longitudinal top and bottom layers of reinforcing. This method is an expedited design method that results in a substantial reduction in reinforcement by accounting for arching action as a load resistance mechanism in one-way slab construction (AASHTO 2010). The arching creates what is best described as an internal compressive dome. Although present design codes do not explicitly permit this method for design of precast panel systems, many researchers have argued it can safely be used for this purpose (Badie and Tadros 2008). After initial sizing of primary direction reinforcing steel perpendicular to the support girders was completed using the empirical design method, the research team realized that an increase in reinforcement quantity was required during the detailing phase to accommodate connection spacing demands and the availability of threaded bar products.

For design of the cantilevered overhang locations, the research team utilized the AASHTO Equivalent Strip Method. This method is among the most widely used for flexural design of one-way slabs and relies on an idealized slab strip to resist applied loads. Included in the design for the cantilevered overhang reinforcement is the accommodation of collision forces associated with a standard ALDOT TL-4 crash barrier. Selected design calculations are included in Appendix A.

3.4.3 Connection Design Approach

A full design of each connection type discussed above was carried out as part of the revised CD-2 system design study. However, it quickly became apparent to the research team that connection details and sizing were most commonly controlled by geometric challenges and constructability concerns, as opposed to sizing of the joint couplers for resistance to force effects. Nonetheless, it remained important to confirm that all final joint and coupler configurations can satisfactorily meet the existing reinforcement splice requirements set forth in the AASHTO 5th Edition Specifications, and that mechanisms for transmission of all force effects between panels are included in the design. Table 3-2 summarizes and clarifies the force effects required to be transmitted across each joint type by the assumptions of typical one-way continuous slab behavior. Note that for the transverse joint, positive and negative moment force effects act locally to transfer applied loadings to the equivalent width of resisting slab. Various joint design factors unique to a CD-2 type system are discussed below.

Table 3-2: Joint Design Methodology

Type	Shear	Positive Moment	Negative Moment
Transverse Joint	X	X	X
Longitudinal Joint	X		X
Staged Construction Joint	X		X

For the transverse panel-to-panel joint, the adequacy of the revised CD-2 type detail was verified by the research team. Per AASHTO tension splice requirements, the coupler was sized to resist at least 1.25 times the yield load of the reinforcing bars being spliced (AASHTO 2010). This was accomplished by considering the axial tension forces as distributed uniformly through the HSS coupler sidewall as shown below in Figure 3-13. For analysis purposes of the transverse joint, the coupler end walls were considered continuously supported by adjacent grout in the final installed state and therefore did not require sizing for bending considerations (Badie and Tadros 2008).

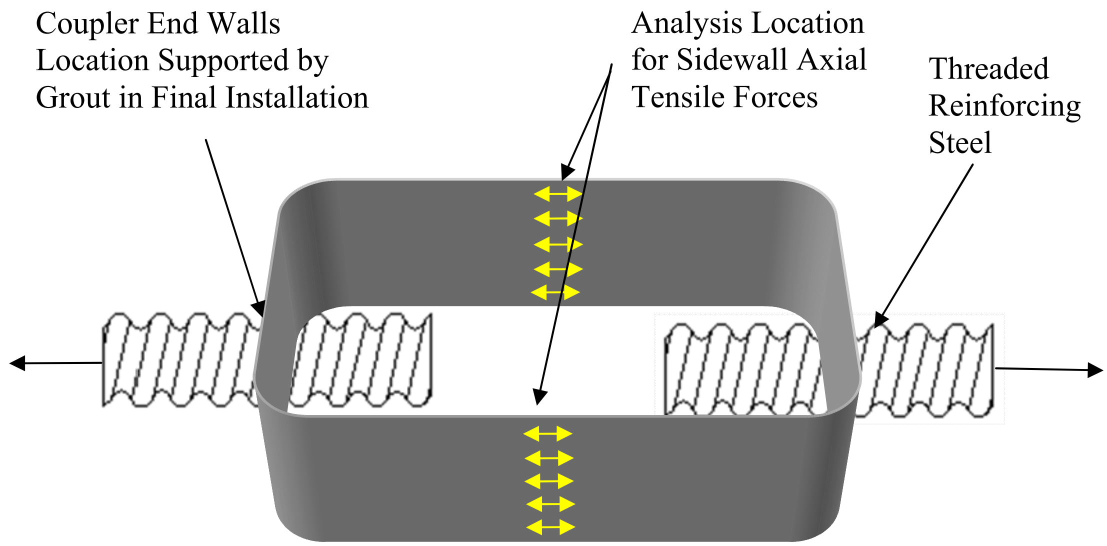


Figure 3-13: HSS Tensile Splice Coupler Design Methodology

The staged construction joint was designed in a manner similar to that for the transverse joint as detailed above. However, in the staged construction joint, the tension splice is located near the top of the slab instead of at its mid-height as in the transverse joint detail. At this location, the research team anticipated there may be cracking that extends through the full depth of the staged construction joint tensile splice as shown in Figure 3-14.

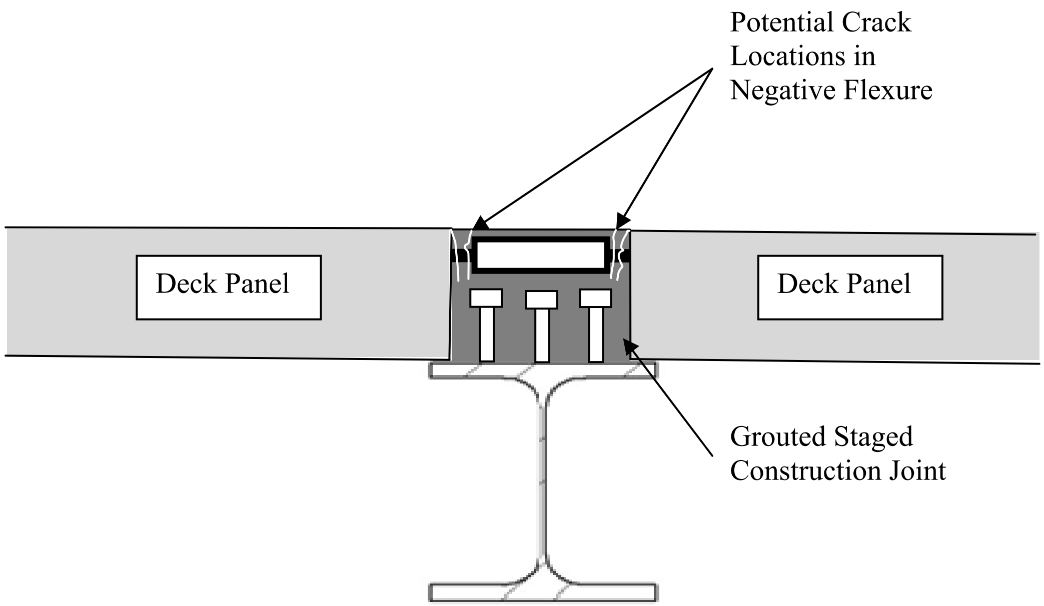


Figure 3-14: Staged Construction Joint Potential Crack Locations

If this cracking occurs as shown above, it may no longer be valid to assume that the HSS coupler end wall is continuously supported by intact grout in service-level conditions. As a result, an additional method to resist endwall bending was introduced to the coupler detail at staged construction joint locations. After much consideration, the research team chose to utilize reinforcing bar stirrups as oriented in Figure 3-15 to provide confinement and continuous support to the HSS endwall and to assure that grout adjacent to the HSS endwalls remained intact.

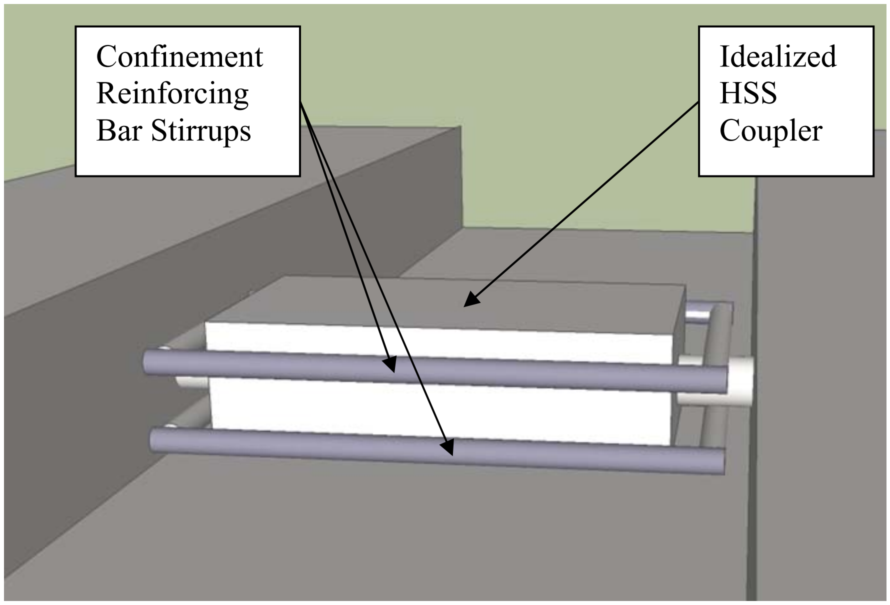


Figure 3-15: Staged Construction Joint Confinement Stirrups

3.4.4 Constructability Checks

Previous limited research regarding the use of continuous shear pocket longitudinal joints has stressed the importance of evaluating panel stresses during all stages of handling and construction (Badie and Tadros 2008). In this investigation, the research team conducted various design checks for constructability during key activities such as lifting panels with cranes, supporting and positively fastening panels to tractor-trailer beds, and unloading by forklift. These activities needed to be carefully pre-planned to minimize the risk of premature cracking in the panels.

A key activity requiring preplanning and design judgment was the location of rigging points for lifting of panels by overhead cranes. As previously discussed, the longitudinal continuous shear pocket detail resists traffic loadings in negative flexure in its final grouted condition. However, prior to installation and grouting, this longitudinal joint detail is oriented such that it is conducive to resisting lifting loads by positive flexure as shown in Figure 3-16. It should be noted, however, that the joint is capable of resisting negative flexure up to that required to buckle or yield the unbraced bottom reinforcing bars.

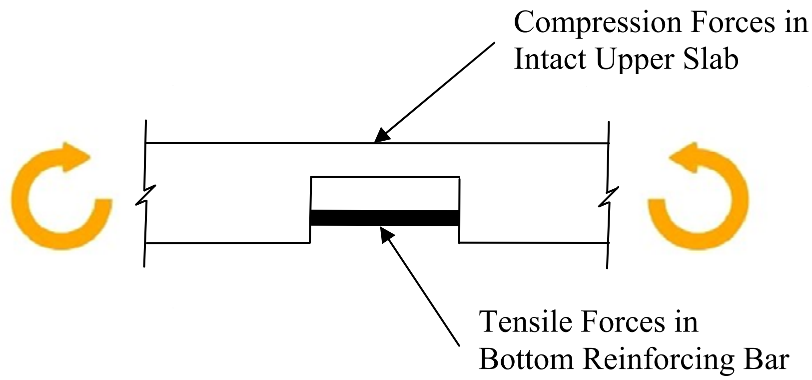


Figure 3-16: Elevation View of Longitudinal Joint in Positive Flexure

The research team chose to induce positive flexure across this joint by locating rigging points at panel corners. Design analyses were performed to verify that the flexural and shear capacity of the panels were not exceeded during any stage of the lifting operations.

Previous researchers have encouraged the use of rigging spreader beams as a means to minimize the possibility of inducing undesirable axial and moment effects during panel handling (Utah Department of Transportation 2010). In this investigation, the research team utilized spreader beams during panel handling operations. The spreader beam was oriented to prevent inducing additional moments in the direction which may tend to “hinge” the pocket location. A schematic of the final lifting configuration is shown in Figure 3-17.

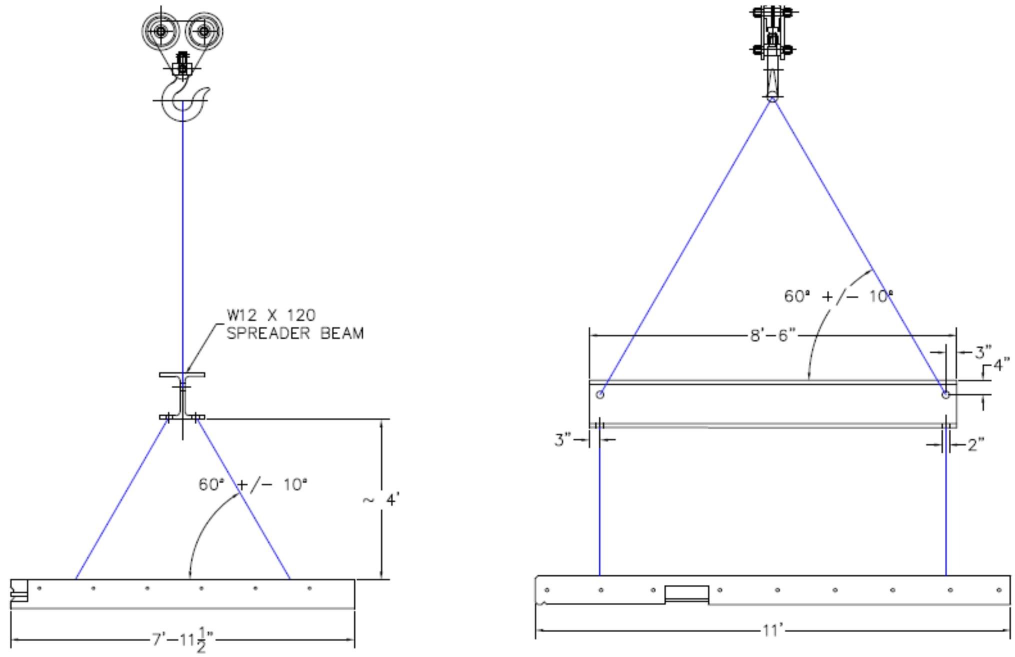


Figure 3-17: Panel Lifting Configuration

3.5 Specimen Configuration and Details

As a result of the conceptual improvement and design study summarized above, a modified CD-2 type full-scale bridge-type laboratory specimen was designed and detailed for construction and structural load testing. In this section, a brief overview of the system configuration and corresponding details will be presented. For reference, a comprehensive set of construction documents utilized for the full-scale bridge-type specimen is located in Appendix B.

3.5.1 Specimen Layout

The full-scale bridge-type deck specimen in this investigation comprises four precast deck panels and three supporting girders as shown in Figures 3-18 and 3-19. This specimen configuration was the most practical to incorporate the three desired joint types previously outlined and discussed. Panels are numbered as shown in Figure 3-19. The final bridge deck specimen is 23'-0" in transverse width and approximately 16'-0" in longitudinal length.

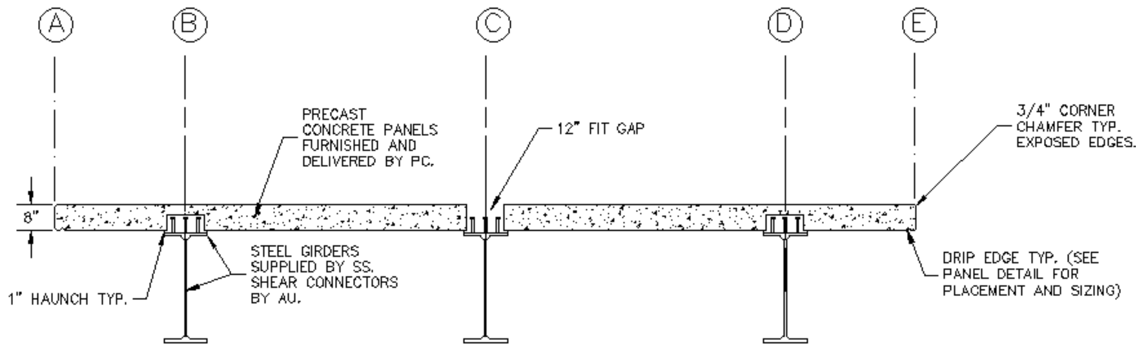


Figure 3-18: General Specimen Elevation

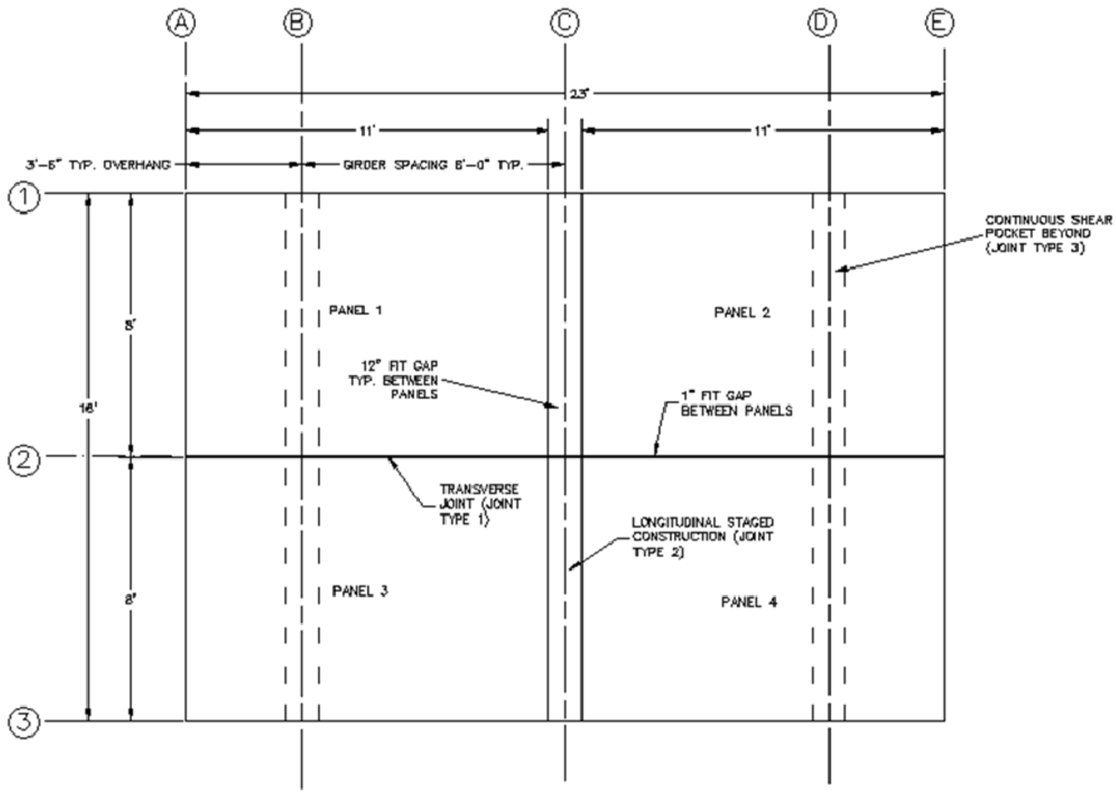


Figure 3-19: General Specimen Plan View Layout and Control

In order to allow expedited reference to various specimen locations and also to assist in construction layout, a control line scheme was used as shown in Figures 3-19 and 3-20. Longitudinal control lines are assigned using alphabetical notation from left to right as shown. Control lines in the transverse direction are assigned using numbers originating at the specimen corner corresponding to longitudinal control line “A” and transverse control line “1.”

3.5.2 Structural Steel Girder Details

The girders utilized in the full-size bridge specimen were W27x178 wide flange sections spaced at 8'-0" on center as shown below in Figure 3-20. Headed shear connectors were utilized to achieve composite action between the girders and full-depth precast panels in the final installed state. The shear connector configuration consisted of three connectors across the flange width at a typical longitudinal center-to-center pitch of five inches as shown in Figure 3-21. Shear connectors were omitted in areas which would conflict with panel reinforcing bars spanning across the longitudinal joint detail during panel installation. These conflicting bar locations are shown as dotted center lines in Figure 3-20, and shear connectors are omitted accordingly.

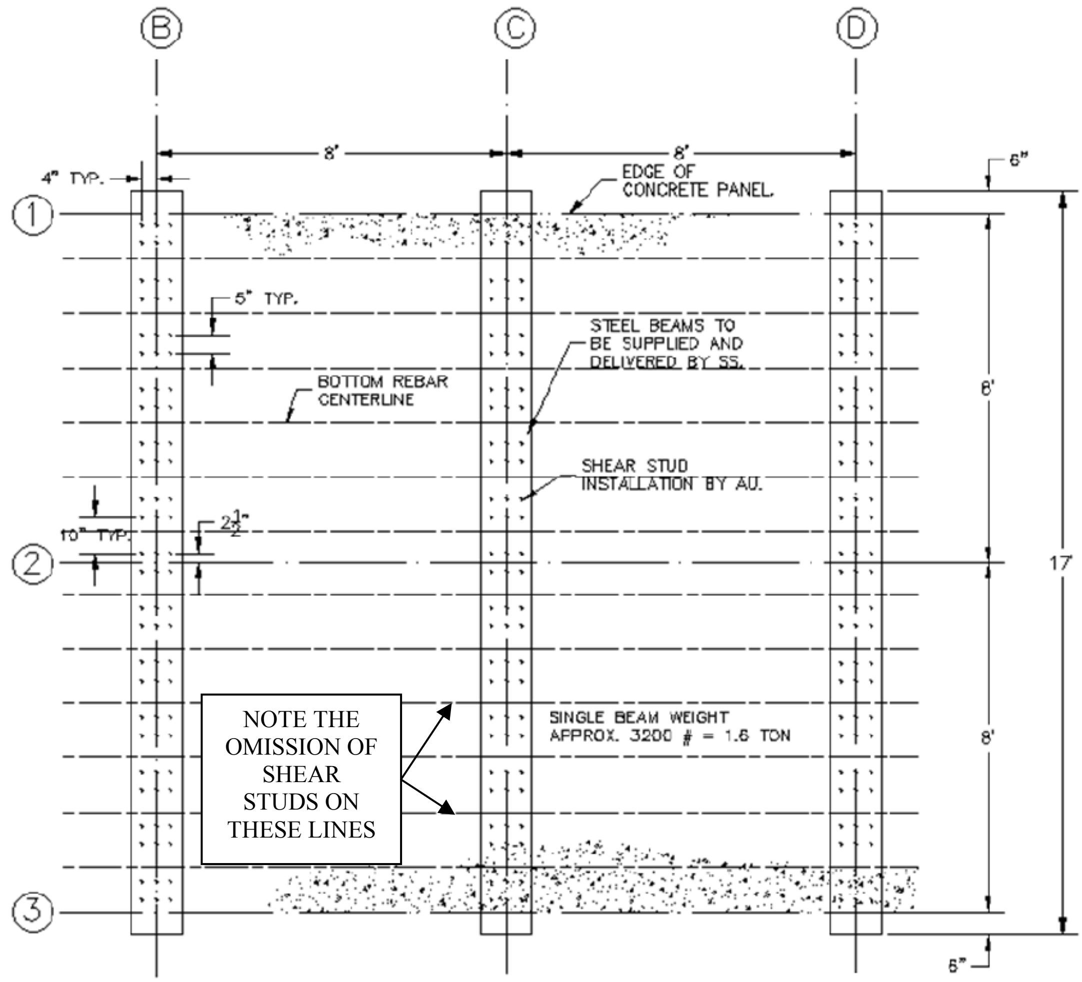


Figure 3-20: Structural Steel Plan View

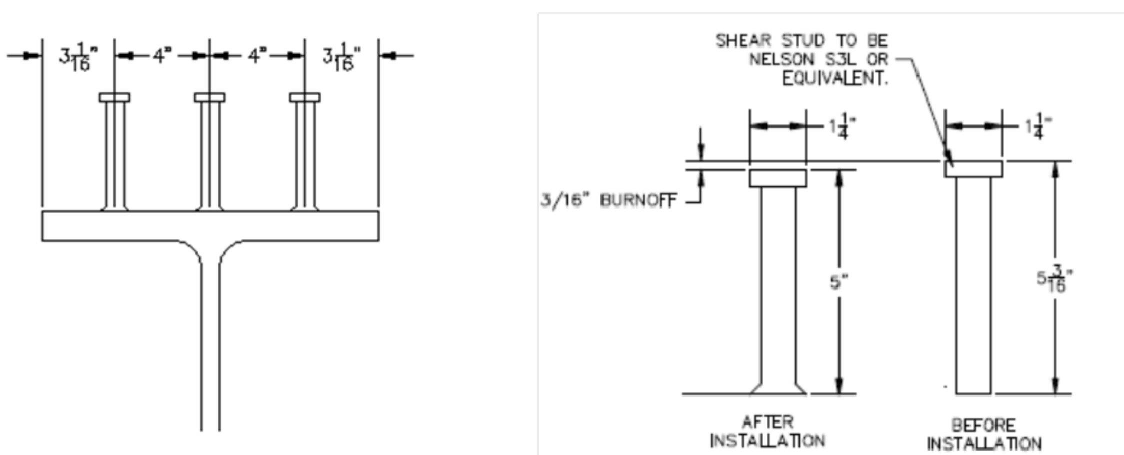


Figure 3-21: Headed Shear Connector Details

As previously stated in the scope details at the beginning of this document, results of this investigation are intended to exclusively reflect the behavior of the deck system as a one-way concrete slab system on rigid supports. As such, the longitudinal girders of the specimen rest continuously on the laboratory floor and are fastened in place to resist potential overturning during panel erection or translation during cyclic load testing. A schematic of the typical girder to laboratory floor connection detail is shown in Figure 3-22.

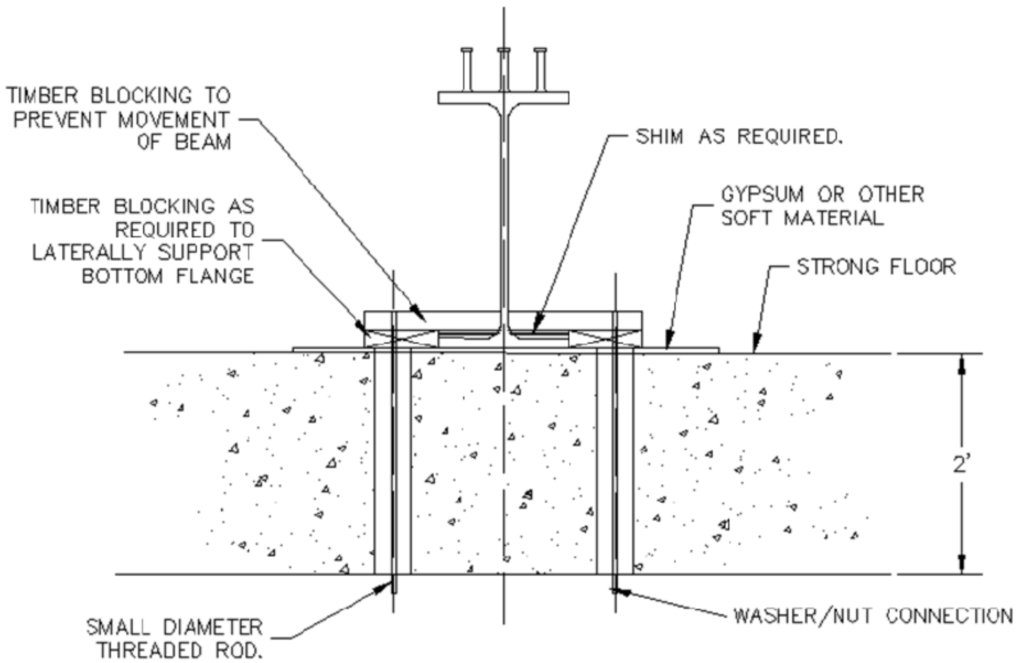


Figure 3-22: Laboratory Floor Girder Connection Detail

3.5.3 Precast Panel Details

The four precast panels included in the experimental specimen were symmetric about the control and layout lines previously discussed. Figure 3-23 shows a rendering of a typical panel to assist the reader in understanding of the complex three-dimensional panel geometry. Geometric details and steel reinforcing details are shown in Figures 3-24 and 3-25 respectively. Refer to Appendix B for full section details and reinforcing schedules.

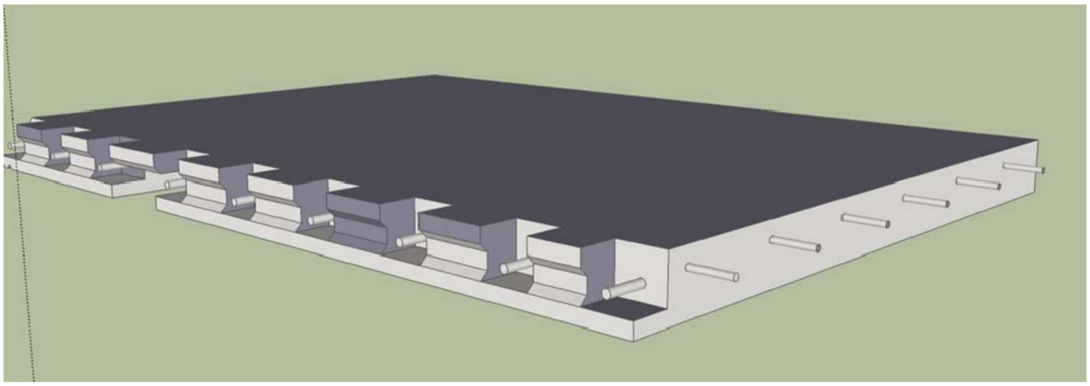


Figure 3-23: Typical Modified CD-2 System Full-Depth Panel Rendering

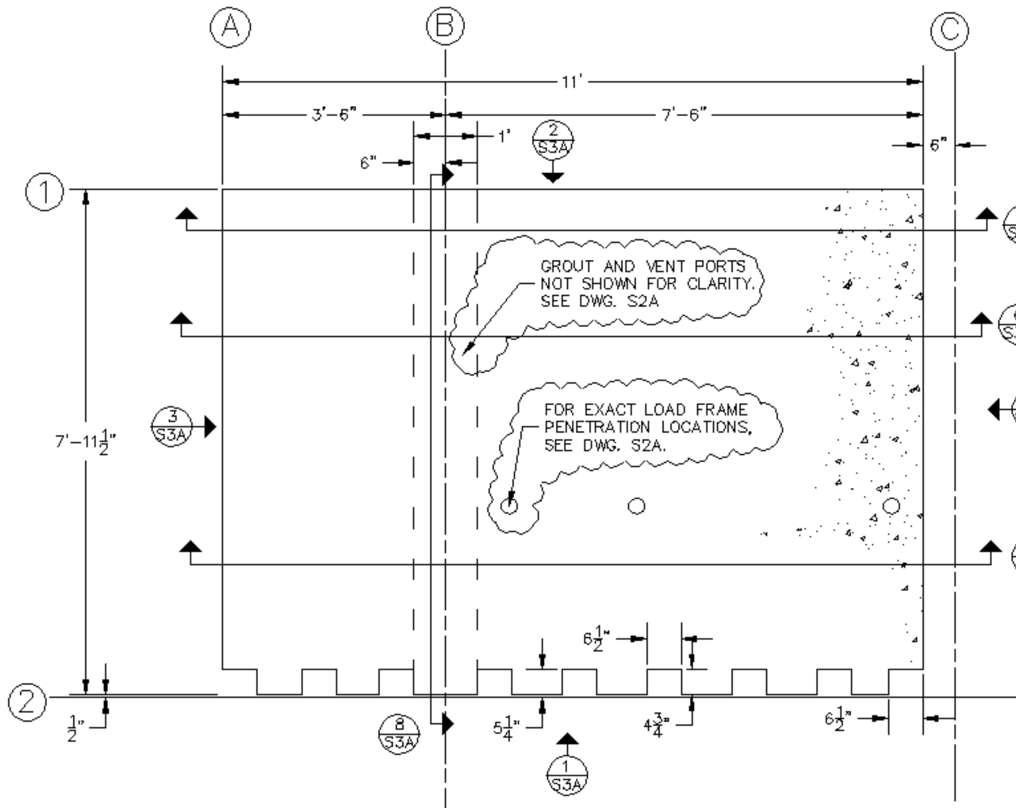


Figure 3-24: Typical Full-Depth Panel Geometry Plan

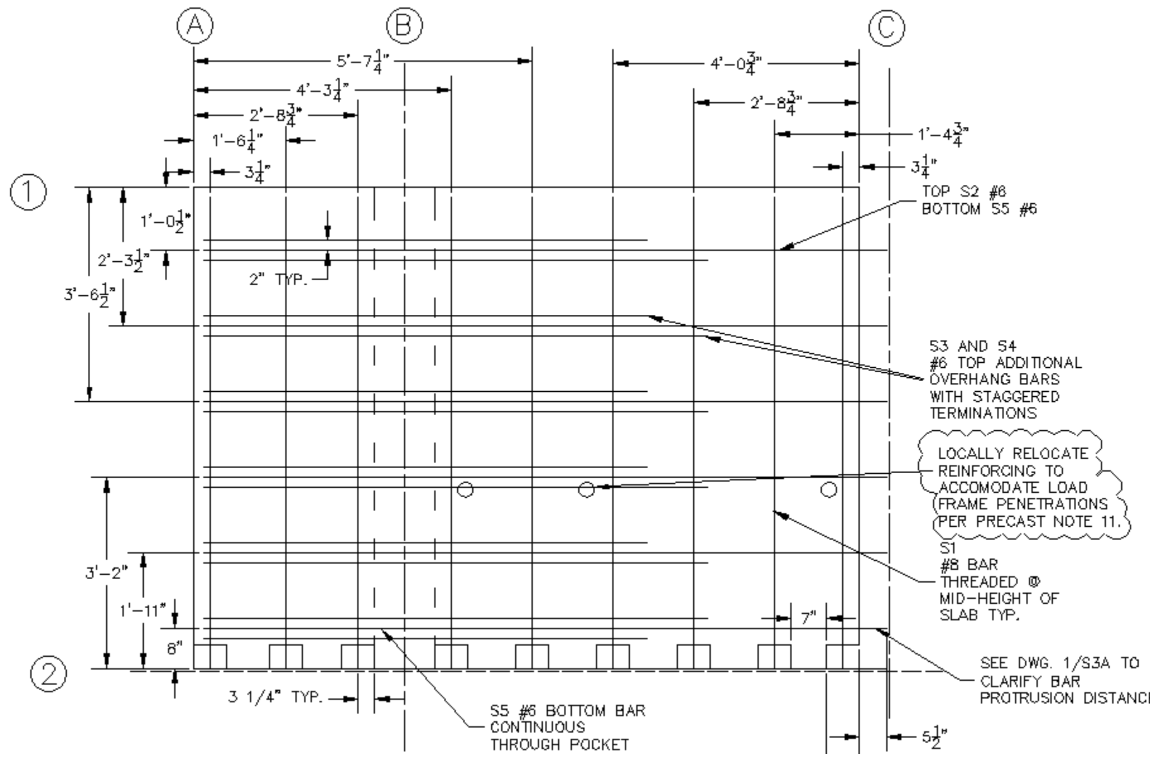


Figure 3-25: Typical Full-Depth Panel Reinforcing Plan

3.5.4 Connection Details

Substantial effort was devoted to the detailing of connections due the complexity of connection types included in the specimen. Figure 3-26 shows a specimen plan view with all panel connections visible. Figures 3-27 and 3-28 illustrate the final configuration of the transverse joints and longitudinal joints, respectively.

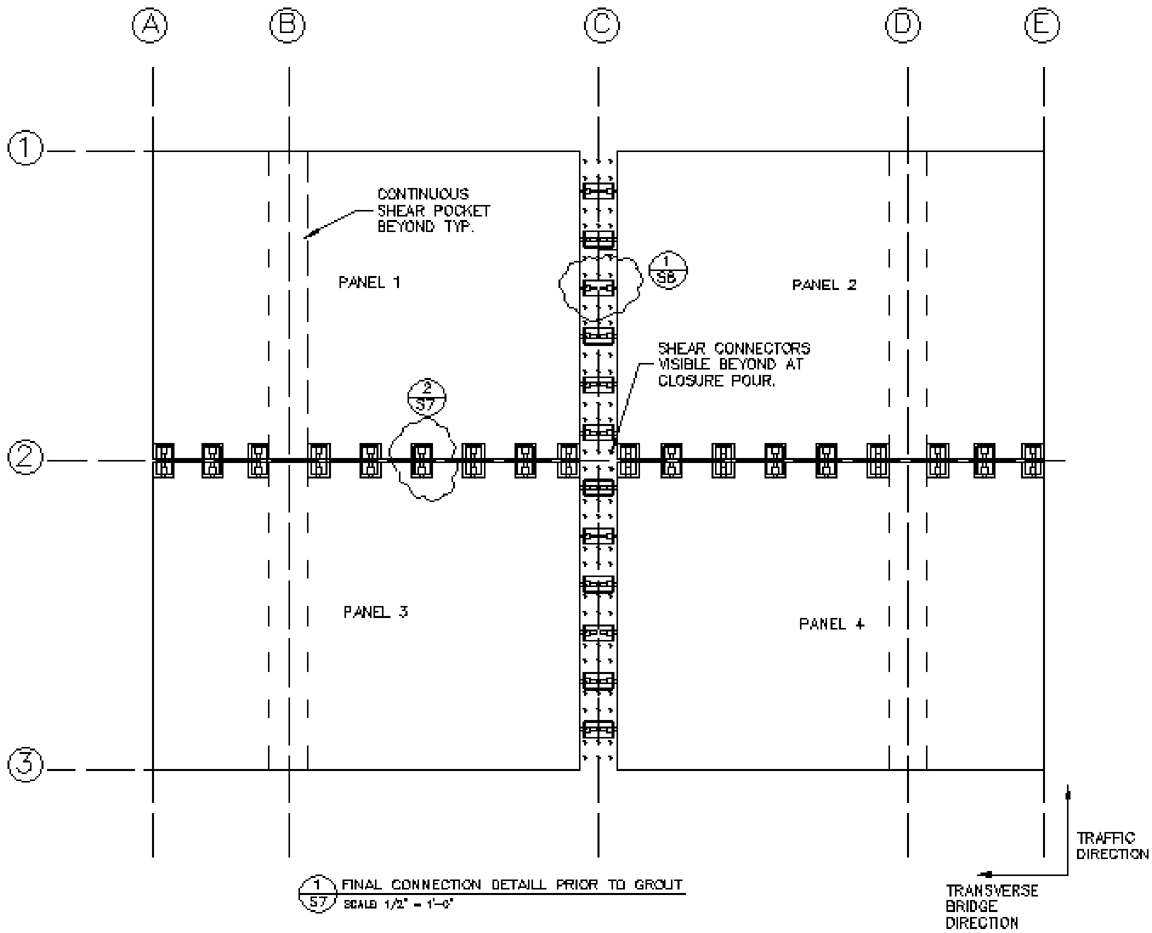


Figure 3-26: Specimen Plan Prior to Grout Installation

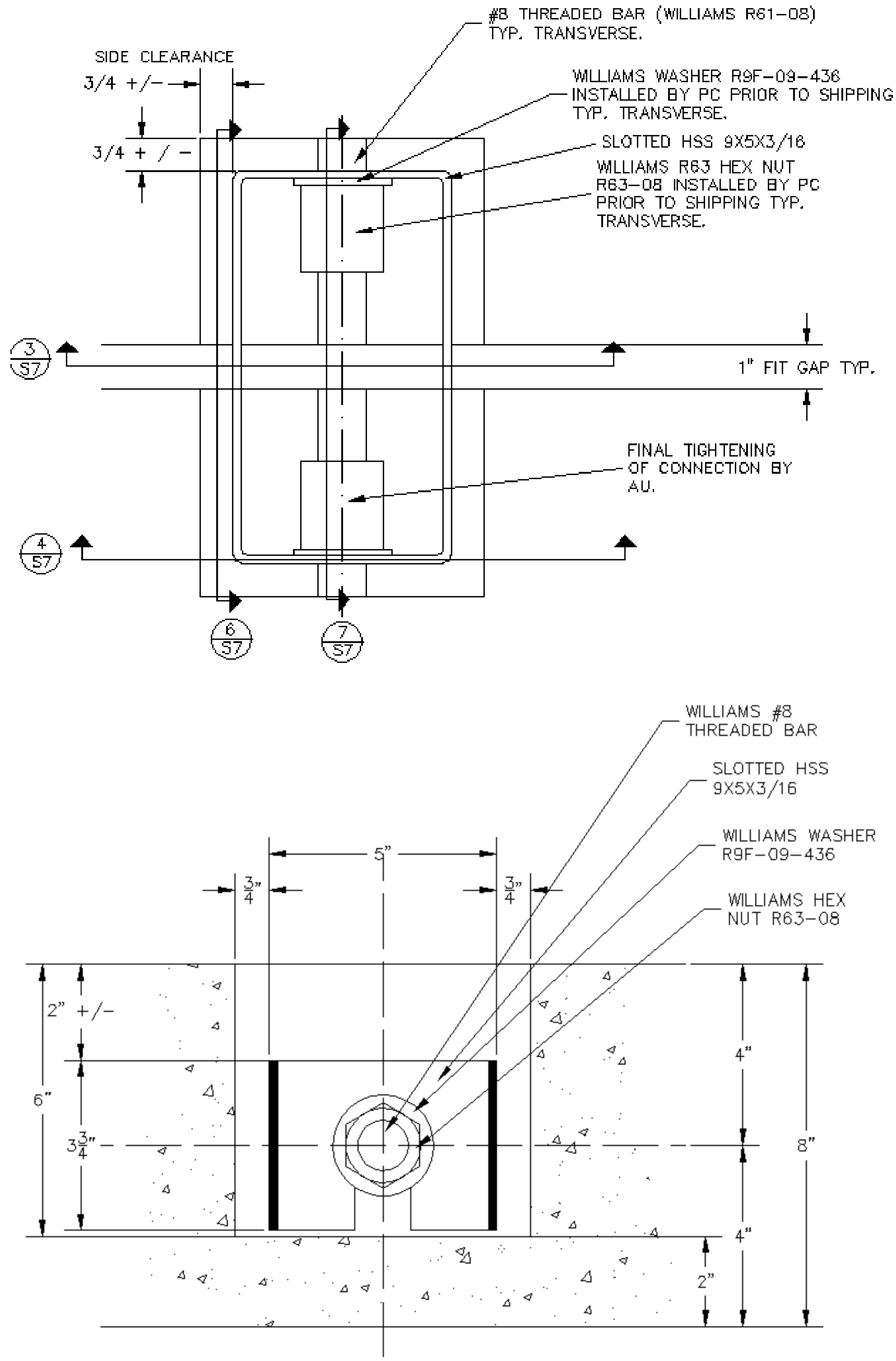


Figure 3-27: Transverse Joint Plan View (Top) and Section View (Bottom)

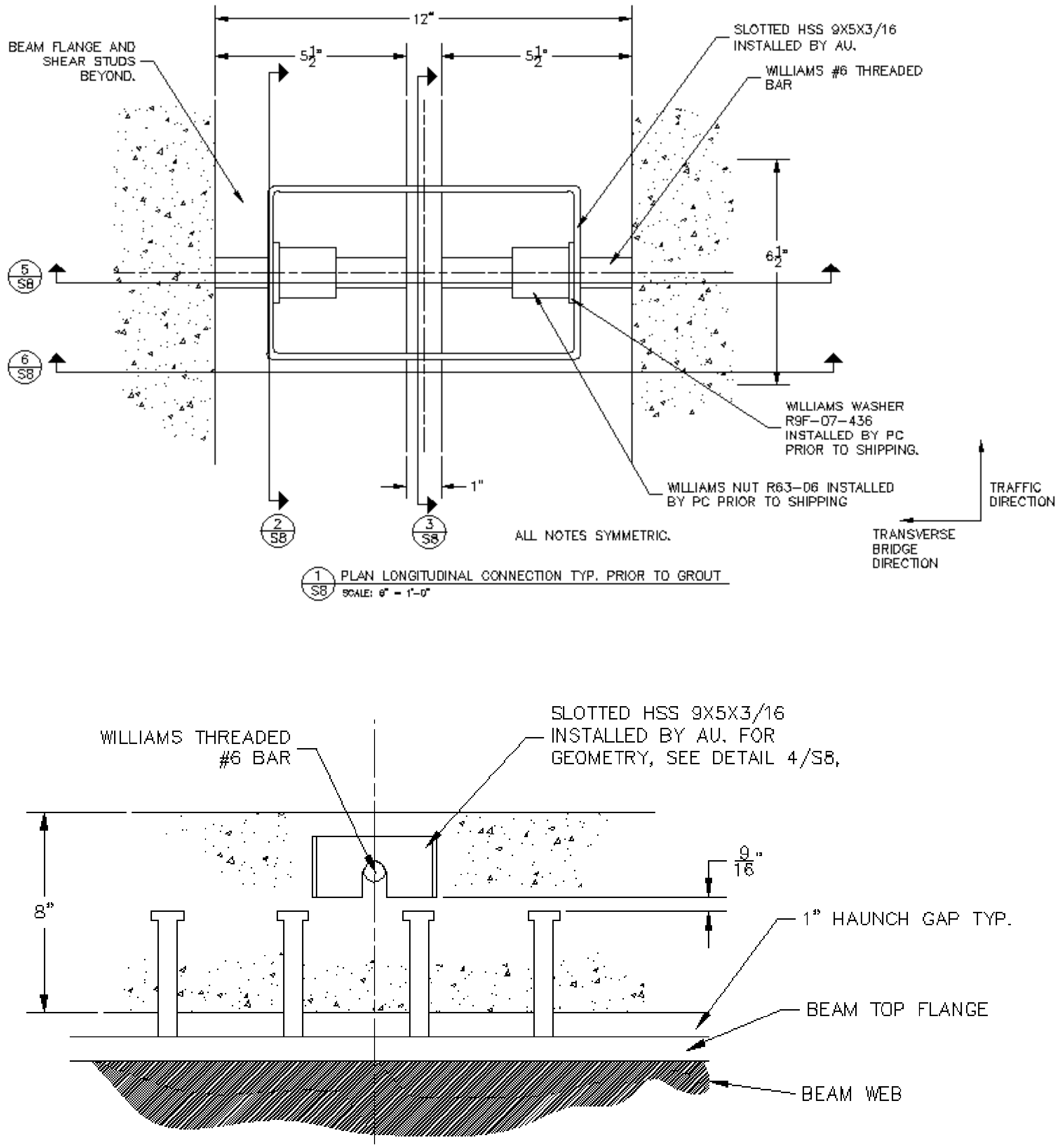


Figure 3-28: Staged Construction Joint Plan View (Top) and Section View (Bottom)

Chapter 4: Deck Panel Specimen Construction

4.1 Overview

This chapter will provide a detailed description of the construction of the modified CD-2 type deck system in the laboratory. With exceptions as noted below, the majority of specimen construction and erection was performed at the structures laboratory located within Harbert Engineering Center on the campus of the Auburn University Samuel Ginn College of Engineering. Constraints of this facility relevant to the sizing of this project specimen include a gantry-type crane capable of lifting vertical loads of 20,000 pounds as well as delivery bay door width and height limitations. Documentation of the construction process by topic is included in this chapter as follows: structural steel fabrication, precast concrete deck panel construction, deck system assembly, and grout installation.

4.2 Structural Steel

The following section details the fabrication and construction of the structural steel aspects of the modified CD-2 type system.

4.2.1 Longitudinal Girders

The longitudinal girders were delivered to the laboratory as W27X178 wide flange steel sections cut to 17'-0" length as shown in Figure 4-1. Each girder was outfitted with lifting holes to assist in handling by laboratory staff. The girder top flanges were machined to remove mill scale and to prepare the surface for shear stud installation.

Figure 4-1: Delivery of Steel Girders

4.2.2 HSS Coupler Fabrication

Fabrication of the HSS couplers for both the transverse and staged construction joint was completed by a local steel fabricator. After manufacture, the couplers were test-fit onto the corresponding threaded bar size to assure intended fit as shown in Figures 4-2 and 4-3. Feedback from the fabricator regarding the prohibitive difficulty of bulging sections of this size was consistent with the preliminary findings of the research team.

Figure 4-2: Test Fit of HSS Coupler: Staged Construction (left) and Transverse (right) End Elevation

Figure 4-3: Test Fit of HSS Coupler: Staged Construction (left) and Transverse (right) Side Elevation

4.2.3 Shear Connector Installation

Installation of the shear connectors was completed by laboratory staff under the supervision of representatives of Nelson Stud Welding Company. Figure 4-4 shows the steel girder top flanges prepared to receive welded shear connectors.

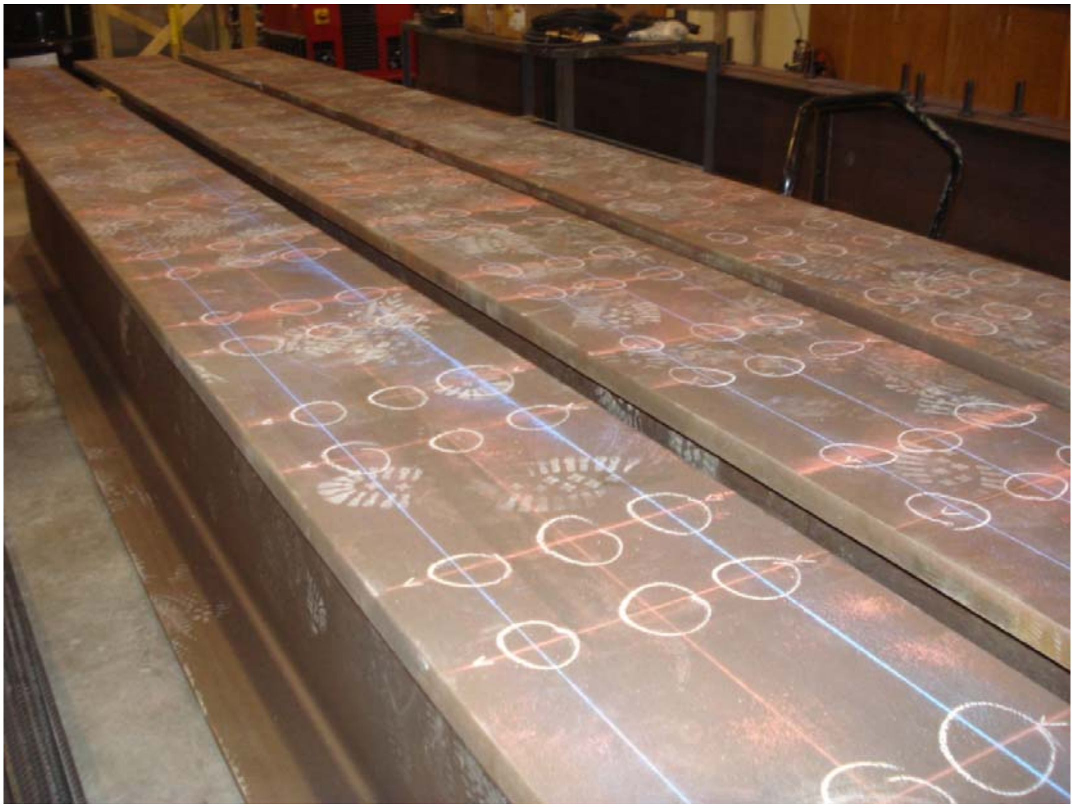


Figure 4-4: Girder Top Flange Prepared for Shear Connector Placement

The laboratory staff utilized a Nelson Nelweld 6000 stud welding machine equipped as shown in Figures 4-5 and 4-6. The setup consists of the stud welding machine, welding leads, and a stud welding gun.

Figure 4-5: Nelson Nelweld 6000

Figure 4-6: Shear Connector Welding Accessories

As recommended by the stud manufacturer for the size utilized in this project, the ¾” diameter shear connectors were welded using a current of 1500 amps applied for 0.9 seconds. Welds produced using this configuration met quality control requirements for stud-type connectors as outlined in the American Welding Society (AWS) Bridge Welding Code (American Welding Society 2008). Among the most critical requirements for satisfactory welds are visual inspections showing full 360-degree flash and bend tests. Figure 4-7 shows a quality control bend test performed in accordance with the provisions of AWS Section 7.6.

Figure 4-7: Shear Stud Quality Control Bend Test

The stud welding process is shown below in Figure 4-8a. The burn-off length, or the shortening of the stud due to the welding process, was consistent with manufacturer predictions and yielded final stud installations with a total height of 5” as intended by specimen design. Figure 4-8b shows the final installation of shear connectors to girder top flanges before clean-up.

Figure 4-8a: Shear Connector Welding Process

Figure 4-8b: Completed Shear Connector Configuration before Cleanup

4.3 Precast Concrete Deck Panels

Fabrication of the precast concrete deck panels was performed by a skilled precast concrete contractor with significant experience in the manufacture of precast concrete elements. Panels for the specimen in this project were fabricated at the contractor's facility in Birmingham, Alabama. The research team made a site visit during panel fabrication to accomplish the following:

- Document the panel construction.
- Confirm adherence to construction drawings and provisions.
- Collect feedback regarding constructability.

4.3.1 Formwork

Due to the relative complexity of the panel geometry, significant effort was devoted to the manufacture of formwork appropriate to panel construction. The contractor noted that the system geometry is conducive to the manufacture of reusable metal formwork for future production of large panel quantities. Dense foam was utilized as a block-out for the continuous shear pocket areas. Use of this foam proved convenient to prevent concrete from entering the continuous shear pocket, while also allowing a reinforcing bar to continue through the pocket area. The formwork is shown in detail in the photos of Figures 4-9 through 4-13. These figures include detailed views for each of the joint types.

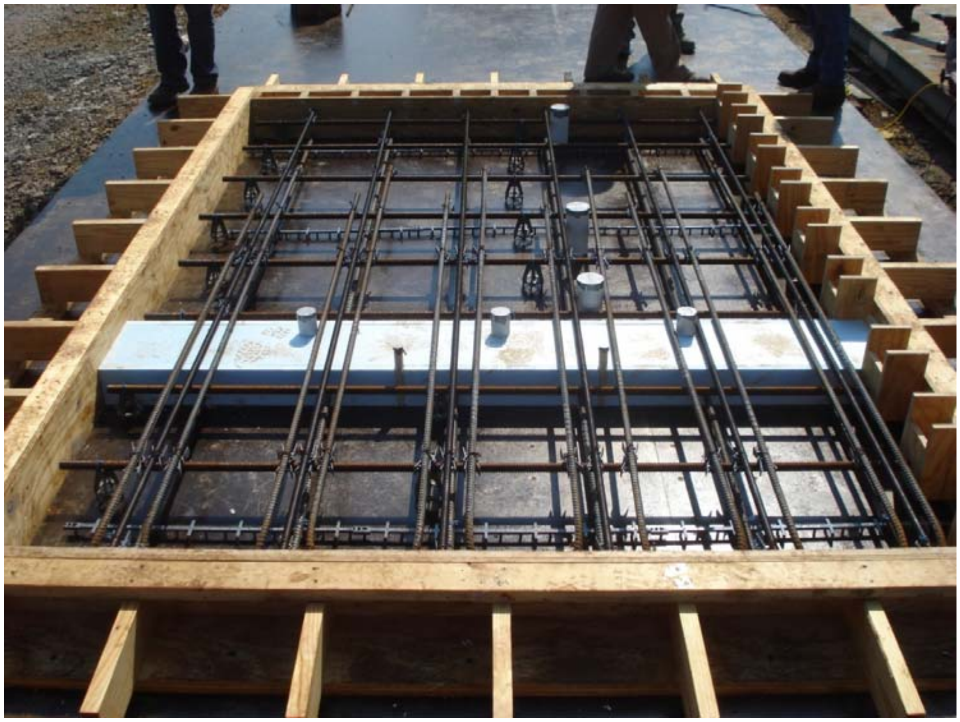


Figure 4-9: Typical Panel Formwork Looking Perpendicular to Girder Line

Figure 4-10: Typical Panel Formwork Looking Parallel to Girder Line

Figure 4-11: Formwork for Staged Construction Joint

Figure 4-12: Formwork for Transverse Joint

Figure 4-13: Continuous Shear Pocket Formwork

4.3.2 Concrete Placement

Concrete was transported from the on-site mixing facility and placed as shown in Figure 4-14 using a concrete buggy. After placement, the concrete was vibrated and received a smooth troweled finish, as shown in Figure 4-15.

Figure 4-14: Placement of Concrete for Typical Deck Panel

Figure 4-15: Finishing of Panel Surface

4.3.3 Curing Procedure

Due to concerns relating to panel durability, the research team required the contractor to moist cure panels in order to reduce the probability of shrinkage cracking. The research team was particularly concerned about potential shrinkage cracking above the continuous shear pocket, which may negatively impact long-term panel durability. Moist curing was accomplished by covering panels with plastic and continual moistening until concrete release strength of 4000 psi was reached. No significant shrinkage cracks were observed during or after the panel fabrication.

4.3.4 Panel Transportation

Transportation of the deck panels from the fabricator's facility in Birmingham, Alabama to the testing facility in Auburn, Alabama was a major concern to the research team. It was vital for the panels to arrive intact and undamaged in order to prove the feasibility of a perceivably "fragile" CD-2 type system. Working together, the research team and the panel fabricator agreed on the following trailer loading requirements to minimize panel damage risks:

1. Provide cribbing at multiple locations along the panel length to avoid inducing larger-than-necessary moments or amplifying road impact movements.
2. Provide cribbing near each side of shear pocket if cribbing is positioned parallel to pocket length.
3. Prohibit placing cribbing within the pocket to support the panel as this may induce undesirable negative curvature in this region.

4. Load straps are to be positioned only in regions where cribbing is present to avoid inducing unnecessary moments.
5. Require the fabricator/transport company to submit a transportation plan and assure compliance with all requirements prior to movement of panels.

The precast panels in this study were successfully transported along 130 miles of highway road with no significant load shifting or damage. Figures 4-16 through 4-18 document the loaded trailer upon arrival at the testing facility.

Figure 4-16: Loaded Trailer upon Arrival at Testing Facility

Figure 4-17: Panels 1 and 2 upon Arrival at Testing Facility

Figure 4-18: Panels 3 and 4 upon Arrival at Testing Facility

4.4 Deck System Assembly

This thesis section is intended to document the placement and connection of the modified CD-2 deck panels to simulated bridge superstructure girders and to adjacent deck panels to form the modified CD-2 type decking planned for this study. The research team attempted to install panels in an accelerated manner as would be required in a rapid bridge deck replacement application. For convenience, this section is subdivided into the following topics: pre-assembly tasks, panel handling and lifting, panel placement, and panel connections.

4.4.1 Pre-Assembly Tasks

Prior to the arrival of the panels to the laboratory, the research team prepared the steel girders to accept the deck panels in order to expedite installation. This preparation consisted of two key activities: positioning girders and forming the girder haunches. It was essential to the project success that these activities be completed within specified geometric tolerances prior to the panel arrival in order to facilitate a timely panel installation.

First, the girder positions were carefully surveyed to assure proper installation. Next, the girders were moved into place by an overhead gantry crane. After positioning, the girders were attached to the laboratory floor utilizing the preplanned details included in the construction drawings. Next, the haunch was formed using one-inch-width spacing bars and plywood as shown below in Figure 4-19. A completed view of the deck support girder setup prior to panel placement is shown in Figure 4-20.

It was important to the research team to form the haunch prior to panel placement in order to minimize the under-slab work required after panel placement. Any construction tasks required to be completed from the underside of newly-installed deck panels are among the most dangerous for construction workers and often require specialized preplanning and safety equipment on bridge replacement projects.

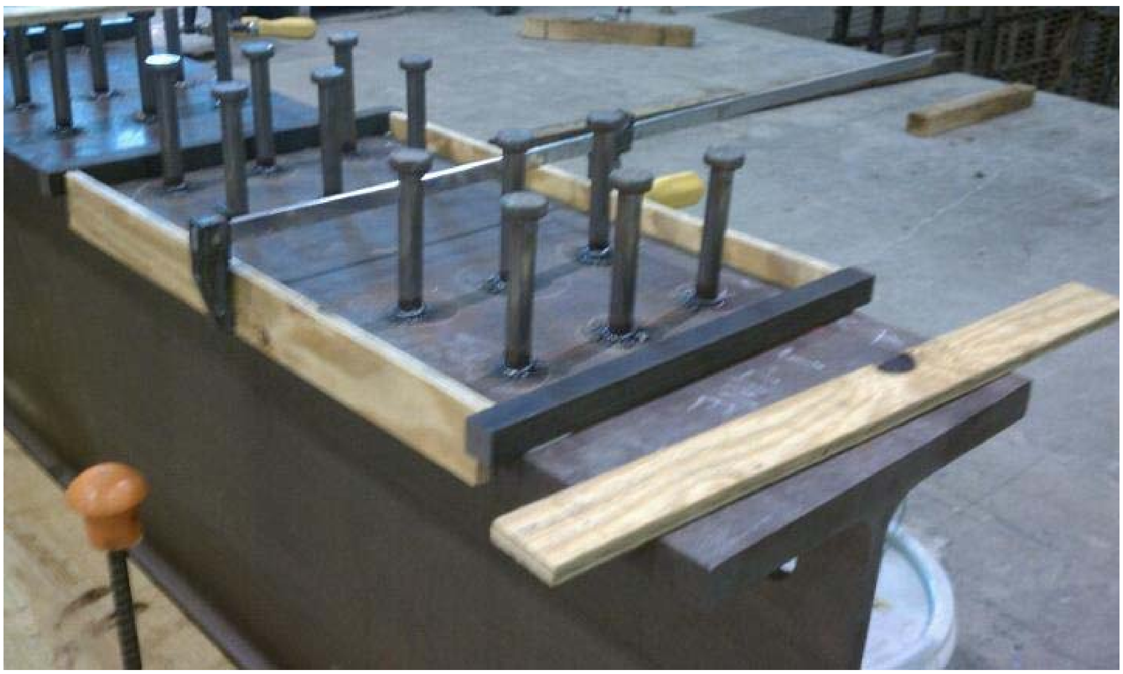


Figure 4-19: Forming of the Bridge Haunch Prior to Panel Arrival

Figure 4-20: Girder Configuration Prior to Deck Panel Placement

4.4.2 Deck Panel Handling and Lifting

In a real-life deck replacement project, deck panels are installed directly from truck trailers into final set position using large overhead cranes. Due to the constraints of the research laboratory used in this investigation, panels had to be offloaded using a forklift prior to lifting by the interior laboratory gantry crane. The research team took careful precautions to evaluate and analyze the deck panels for potential force effects induced by the somewhat unorthodox offloading by forklift. The photos shown in Figures 4-21 and 4-22 document the offloading process.

Figure 4-21: Deck Panel Offloading Using Forklift

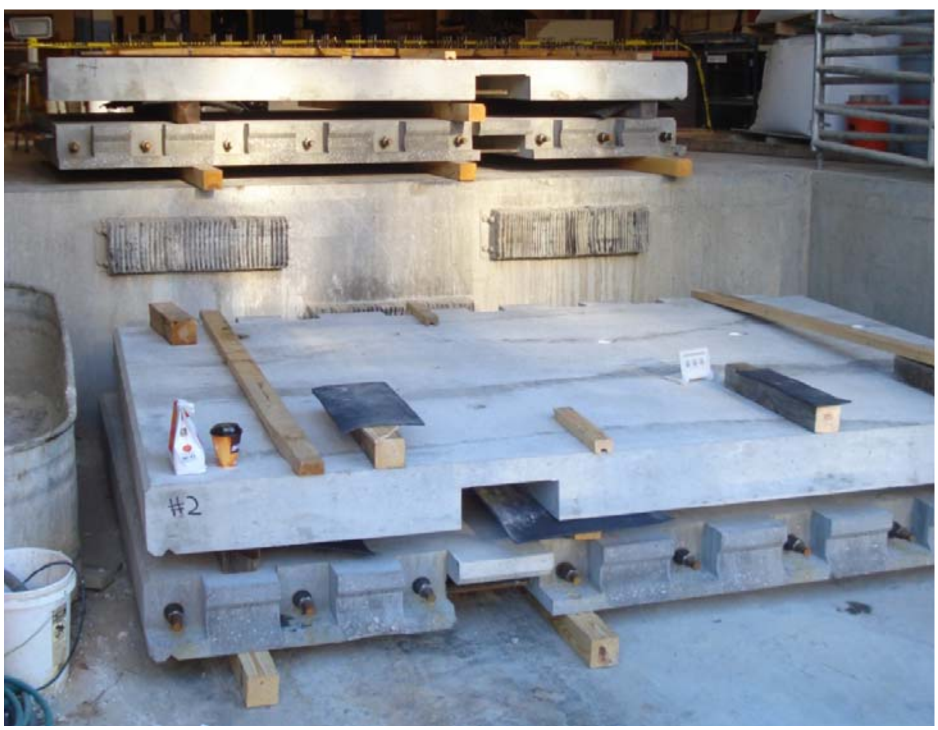


Figure 4-22: Offloaded Deck Panels

After positioning the panels within the delivery bay door, the overhead gantry crane was used to lift the panels. The research team was careful to adhere to the preplanned lifting plan, including the use of a spreader beam, as previously discussed in Chapter 3. Figure 4-23 shows the orientation of the spreader beam used during all deck panel crane lifting operations.

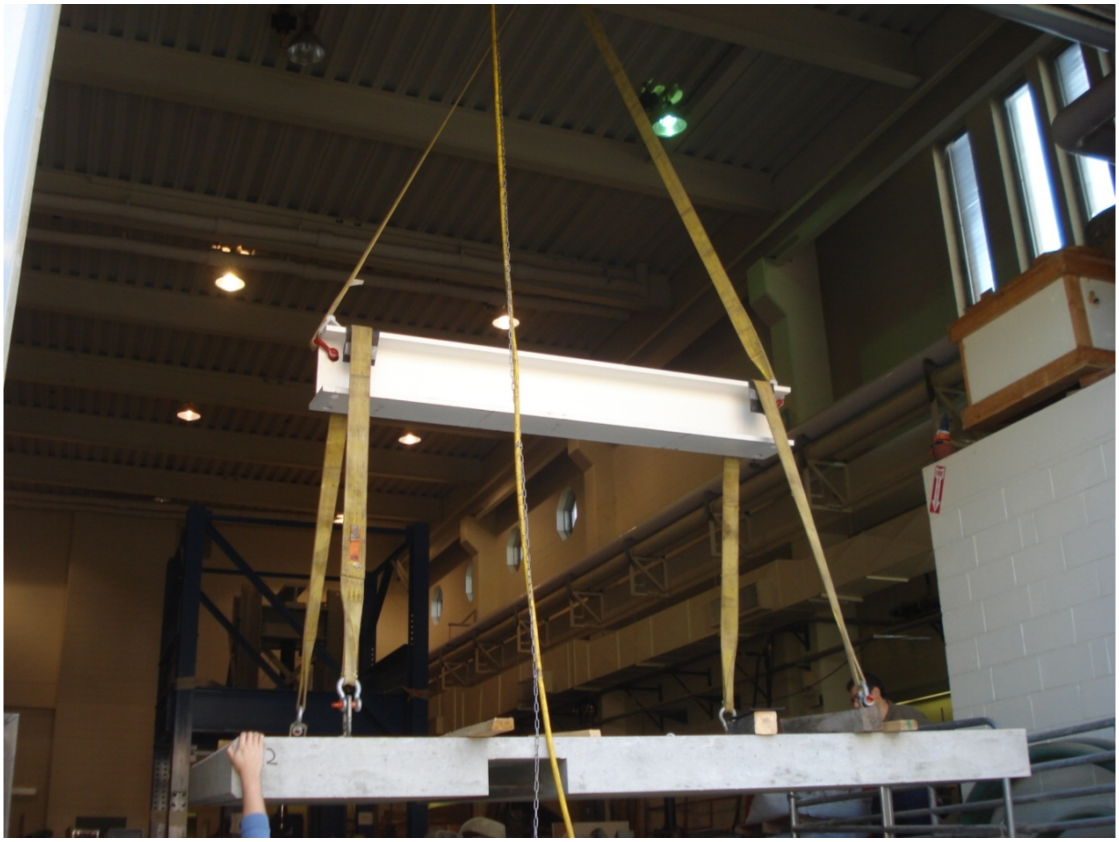


Figure 4-23: Deck Panel Lifting Configuration Including Spreader Beam

4.4.3 Deck Panel Placement

The concrete deck panels were carefully maneuvered into final position using the overhead gantry crane. The laboratory staff was able to successfully locate all four panels into final position without incident in less than 90 minutes. Placement tolerances as reflected on the construction drawings proved sufficient in all cases to avoid conflicts among various construction elements. The photographs in Figures 4-24 and 4-25 show panel placement operations.

Figure 4-24: Positioning of Typical Deck Panel

Figure 4-25: Test Deck Specimen after Panel Placement

4.4.4 Panel-to-Panel Connections

After all four panels were in final position, the transverse and staged construction panel-to-panel connections were made. The laboratory team completed installation of the slotted HSS couplers with ease in both joint types. After the HSS couplers were lowered into place, nuts on the threaded bars were tightened. It was necessary to use small wooden spacing blocks between panels to preserve the required 1 inch grout installation gap and also to allow uniform tightening of connections along the panel length. Finally, the staged construction joint was completed by installation of the confining horizontal steel stirrups around the HSS couplers. Preparation and installation of the stirrups for the staged construction joint proved to be the most time-consuming activity associated with panel connections. Figure 4-26 shows the initial panel connections prior to installation of staged construction joint confining stirrups. Figures 4-27 and 4-28 show detailed views of the completed transverse and staged construction connections, respectively.

Figure 4-26: Initial Panel-to-Panel Connections

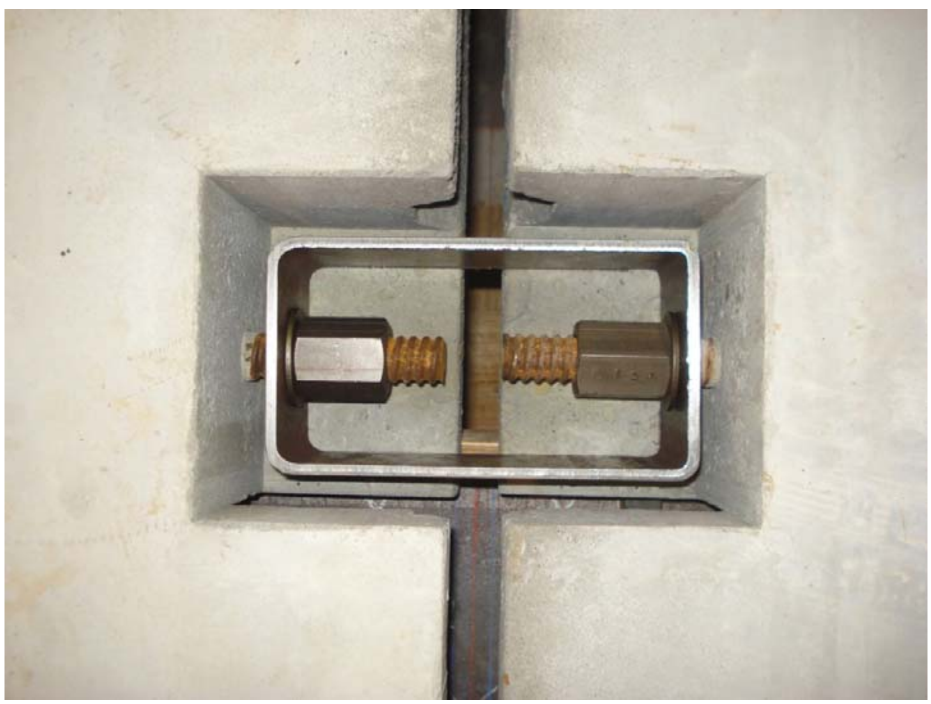


Figure 4-27: Completed Transverse Joint Typical Connection

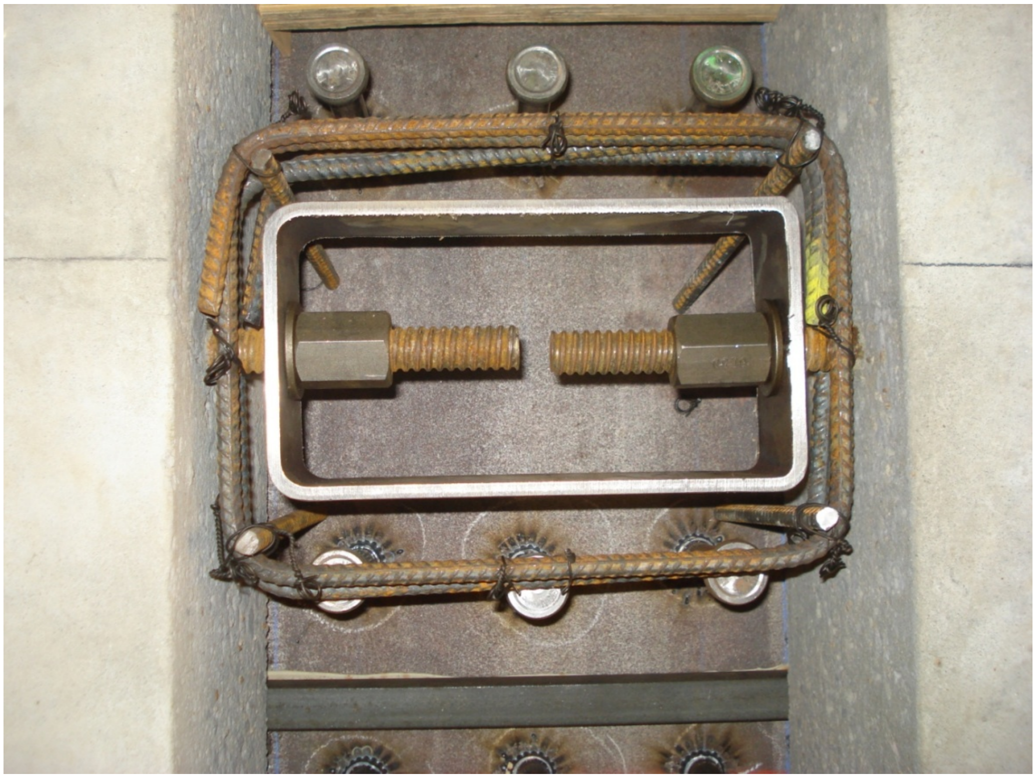


Figure 4-28: Completed Stage Construction Joint Typical Connection

4.5 Grout Installation

In order to complete each deck connection type, non-shrink grout was installed into all voids within the specimen deck. The research team was able to directly install grout into the staged construction and transverse joints from overhead. For the “blind” longitudinal joint, grouting was achieved through a series of pre-planned injection and vent ports on the deck surface. This thesis section is divided into three sub-sections: formwork, placement, and curing procedures.

4.5.1 Formwork

After panel placement and mechanical connections were complete, the laboratory team fabricated formwork to allow grout placement from above. This consisted of end forms on each continuous shear pocket, forms on each end of the transverse joint, and any necessary formwork below the specimen deck. The final formwork configuration is shown in Figure 4-29.

Figure 4-29: Formwork Prior to Grout Installation

4.5.2 Placement

Due to the relatively small volume of grout required for this project, the grout was prepared on-site by a team of laboratory workers. The non-shrink grout utilized in this investigation was batched, mixed, and installed per published manufacturer instructions and also specific recommendations from manufacturer representatives. A team of ten workers was able to mix and place the grout in approximately 5 hours. The grout installation process is documented via photos in Figures 4-30 through 4-35.

Figure 4-30: Grout Mixing Station

Figure 4-31: Dispensing Grout into Overhead Bucket

Figure 4-32: Grout Installation into Staged Construction Joint

Figure 4-33: Grout Installation into Longitudinal Joint through Injection Port

Figure 4-34: Grout Installation at the Panel-to-Panel Joint Intersection

Figure 4-35: Final Leveling and Finishing of Grout Installation

4.5.3 Grout Curing Procedure

The grouted joints received a moist cure per manufacturer recommendations. The research team chose to extend the duration of the moist cure past the required 24 hours in order to reduce the probability of shrinkage cracking of the grout surface. The wet cure was achieved by placing wet burlap and plastic sheathing as shown below in Figure 4-36. The laboratory team continually moistened the burlap for the wet cure duration. Upon stripping of the wet cure, the research team did note minor surface cracking perpendicular to the staged construction joint at various locations along the joint length as shown in Figure 4-37. This shrinkage cracking appeared cosmetic in nature, was documented, and monitored for growth throughout the remainder of the project.

Figure 4-36: Moist Curing of Grouted Joints

Figure 4-37: Shrinkage Cracking across Staged Construction Joint

Chapter 5: Load Testing Program

5.1 Overview

This chapter presents an overview of the load testing program utilized in this investigation. The intent of the testing program was to apply service-level traffic loads to the bridge specimen in order to evaluate the in-service performance of the modified CD-2 type deck system. This chapter contains a general description of the loading program and also includes details regarding the load application apparatus, instrumentation, and data acquisition.

5.2 Loading Program

In developing the loading program for this investigation, the research team relied heavily on published testing standards as well as previous experimental research programs by others. Among the most beneficial resource was the American Society for Testing and Materials (ASTM) Standard D-6275 entitled “Standard Practice for Laboratory Testing of Bridge Decks” (American Society for Testing and Materials 1998). This standard serves as a universal summary of conventional laboratory testing practices as applied to bridge deck testing. The final loading program used in this research project adhered to the requirements of the ASTM D-6275 where possible, while also maintaining similarity with prior experimental tests to facilitate comparison of results.

5.2.1 Load Magnitude

The load magnitude for the specimen laboratory test was selected in accordance with the provisions of the *AASHTO LRFD Bridge Design Specification*s. Per these provisions, a concrete bridge deck must be able to resist the factored forces induced by the design truck pictured below.

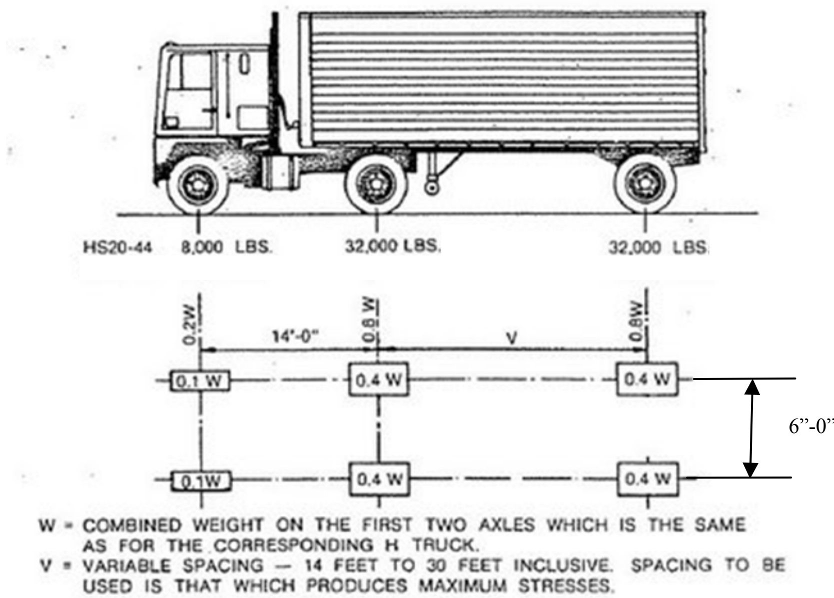


Figure 5-1: Standard HS-20 Truck Loading (AASHTO 2010)

In typical bridge deck testing applications, a single 32,000-pound rear axle load is applied to the intact bridge specimen. This single axle is applied as two 16,000-pound tire contact loads applied directly to the bridge deck surface. These tire loads are applied at a transverse tire spacing of 6'-0" over a 20" x 10" tire contact area as required by AASHTO requirements. To achieve service-level loadings, it is necessary to include the service-level load factor of 1.0, as well as an applicable impact amplification factor. The research team chose to use an impact factor of 33% for the final design. Although some researchers suggest using a 75% impact factor as designated for bridge deck joints, this larger factor is intended only for discontinuous joint types such as bridge expansion joints. After accounting for the impact and service load factors, the final total axle load applied to the specimen was 42,600 pounds. For the stability of the load application frame during cyclic testing, the research team chose to increase the load magnitude as to maintain tension in vertical threaded bars during all load cycling. The final cyclic axle load fluctated between between 4,000 pounds and 46,600 pounds.

5.2.2 Load Locations

In choosing the deck surface locations to apply the tire contact loads, the research team followed guidance from the ASTM C-6275 specification. This specification logically recommends that the truck wheel loads be positioned in locations as to produce the maximum force effects across critical deck sections (ASTM 1998). A summary of the two load cases as selected by influence line diagrams is illustrated below in Figure 5-2 and 5-3, respectively.

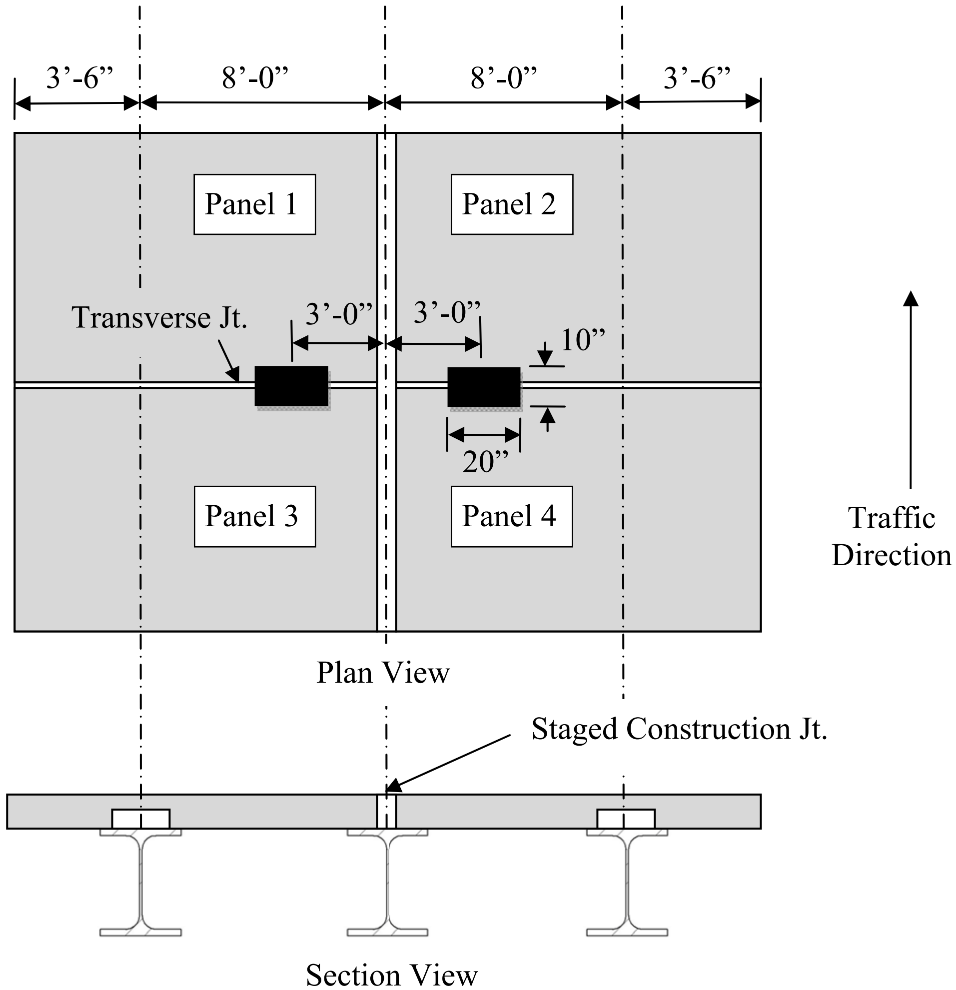


Figure 5-2: Load Positioning for Case I

Load Case I corresponds to the maximum negative moment induced at the middle girder staged construction joint location. As seen in Figure 5-2, the tire contact areas are located symmetrically across the tranverse construction joint in this configuration. The research team chose to orient the loads in this manner in order to evaluate local moment resistance durability of the transverse joint when subject to directly applied loads.

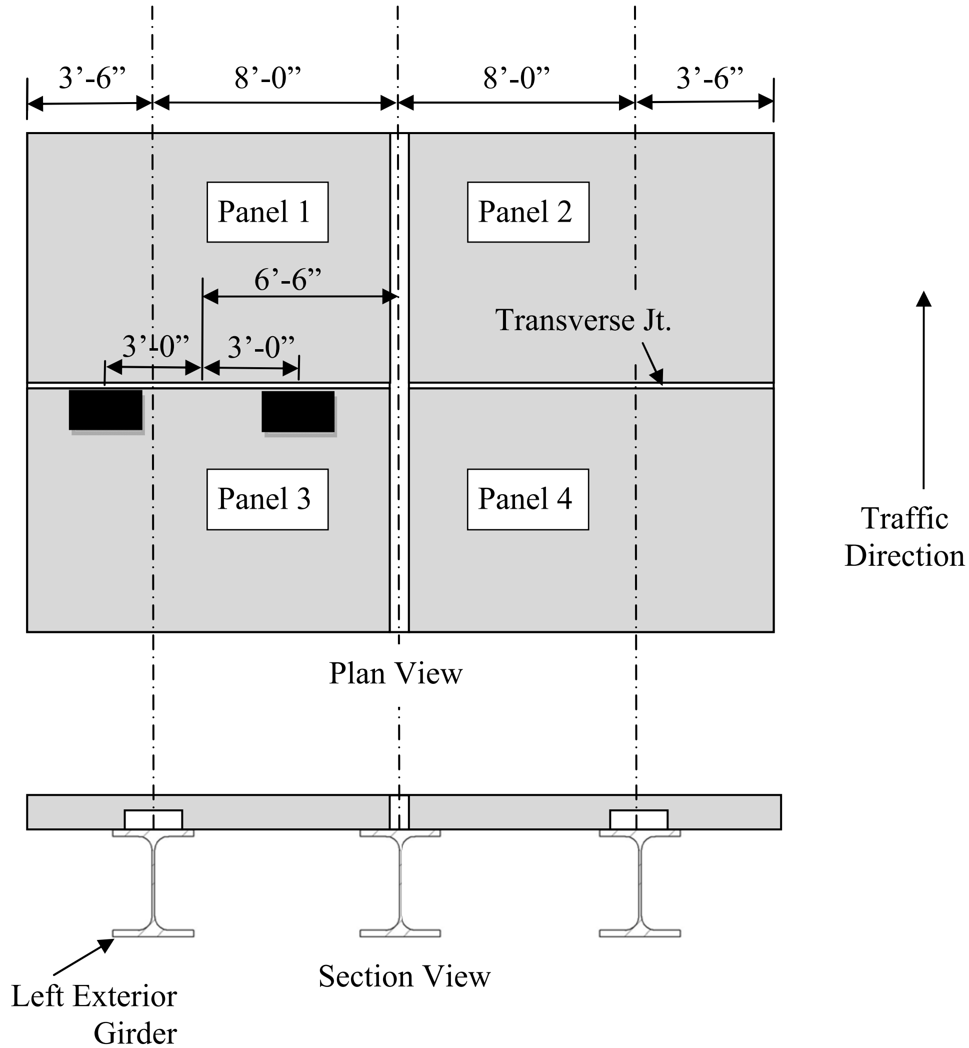


Figure 5-3: Load Positioning for Case II

Load Case II corresponds to the maximum negative moment induced at the continuous longitudinal joint detail located along the left side exterior girder line. As seen in Figure 5-3, the research team chose to locate the tire contact areas asymmetric to the tranverse joint in an effort to evaluate the tranverse joint durability under full local shear effects. It is important to note that the outermost tire contact area utilized in Load Case II is located outside of the permitted loading area per the AASHTO design provisions. For design purposes, an unloaded area is typically preserved along the bridge perimeter consisting of the actual barrier rail width as well as a 12" AASHTO buffer zone. In this investigation, the research team chose to apply the design truck load for Load Case II in order to induce the same maximum negative moment across the longitudinal joint as was induced across the staged construction joint by Load Case I. This unique loading scheme was attempted in order to allow the research team to extrapolate testing results from this study to potential future panel configurations which may utilize multiple longitudinal joints on a single deck panel.

5.2.3 Static Loading Protocol

Static load tests were performed twice at each load location. The first static test was performed prior to cyclic loading, with the final static test being performed after completion of cyclic loading. For each static loading test, the applied load was slowly increased from zero to the maximum load of 46,600 pounds, with momentary pauses at intervals of approximately 10,000 pounds in order to assess load-deflection behavior.

5.2.4 Cyclic Loading Protocol

Per the recommendations of ASTM D-6275 and previous similar research, loads were applied for 2,000,000 cycles in each load case. Loads were applied using a sinusoidal loading function which fluctuated between 4,000 pounds and 46,600 pounds at a frequency of approximately 2.0 Hz. The total duration of a single 2,000,000-cycle test was approximately 11.5 days.

5.3 Loading Apparatus

A brief overview of the loading apparatus utilized by the research team is presented in this section. The loading apparatus consisted of a load frame as well as a hydraulic actuator assembly.

5.3.1 Load Testing Frame

A load testing frame, as shown below in Figures 5-4 and 5-5, was specifically designed for this test program in order to facilitate loading locations as detailed above. The loading frame consisted of four threaded bars, a top spreader beam, a bottom spreader beam, and stabilization straps. These can all be seen in Figures 5-4 and 5-5. Complete details for the load testing frame utilized in this investigation are included in Appendix B.

Figure 5-4: Fully Assembled Load Testing Frame from Above

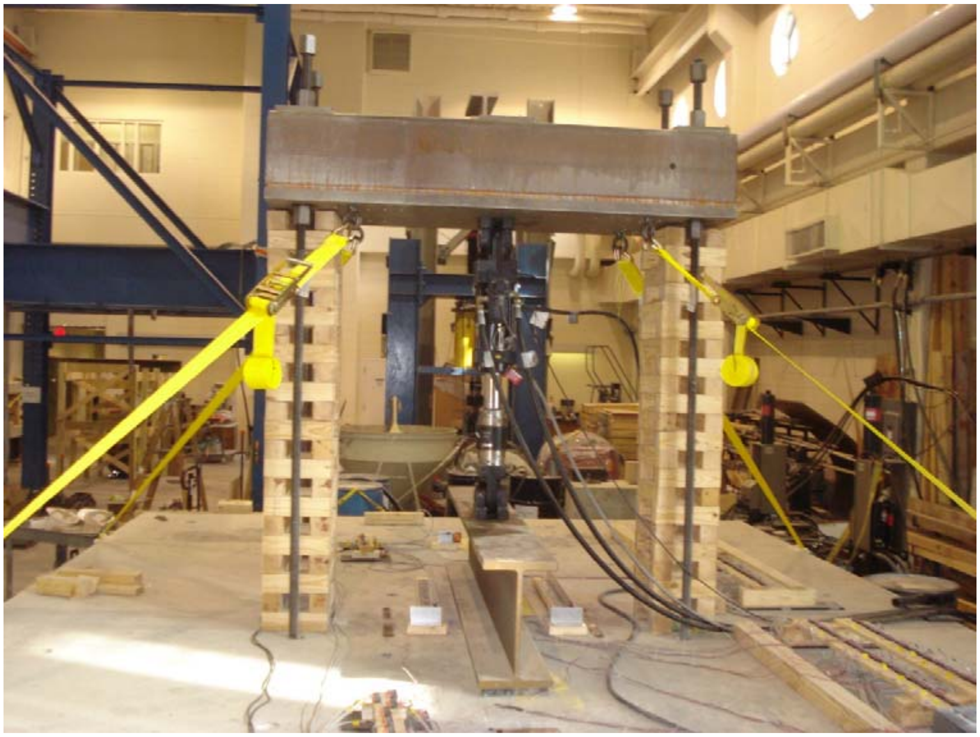


Figure 5-5: Fully Assembled Load Testing Frame Side View

5.3.2 Hydraulic Actuator

An MTS Model 243.35 actuator was utilized for testing in this program. The actuator, capable of utilizing load control as well as displacement control, can range from 54 kips in tension to 82 kips in compression and has a stroke range of 10 inches. An MTS Model 407 controller was used to control the actuator, and load control was used for all testing in this program.

5.4 Deck Instrumentation

The full-size bridge specimen included in this study was instrumented in order to provide information regarding the performance of the modified CD-2 type deck panel system during loading. Instrumentation was provided in order to achieve the following goals:

- Evaluate each joint type's performance under applied loading.
- Measure relative spreading of panels, if any, due to applied loads.
- Explore resistance mechanisms of HSS couplers in both staged construction and transverse joint applications (axial tension component vs. bending component).
- Evaluate degradation of stiffness, if any, through measurements of displacement at key specimen locations.

In order to achieve the above goals, a combination of drawstring potentiometer displacement gages, laser displacement gages, and variously-sized traditional strain gages was used.

5.4.1 Internal Instrumentation

Internal instrumentation in this study consisted of four heavily instrumented HSS couplers. Internal instrumentation was positioned exclusively for use during the heavily instrumented symmetric Load Case I and was not used for later loading configurations. Figure 5-6 shows the locations of the instrumented HSS couplers, as well as the particular locations of the 52 strain gages used for this purpose. Note that the confining stirrups located at the SC1 and SC2 locations were instrumented as well and are labeled accordingly below. Figure 5-7 shows the gages installed on two couplers prior to installation, while Figure 5-8 shows the fully installed gage configuration during grout installation. Note that sidewall gages are installed at third-points along the wall height as shown in Figure 5-7, while endwall gages are installed at coupler mid-height.

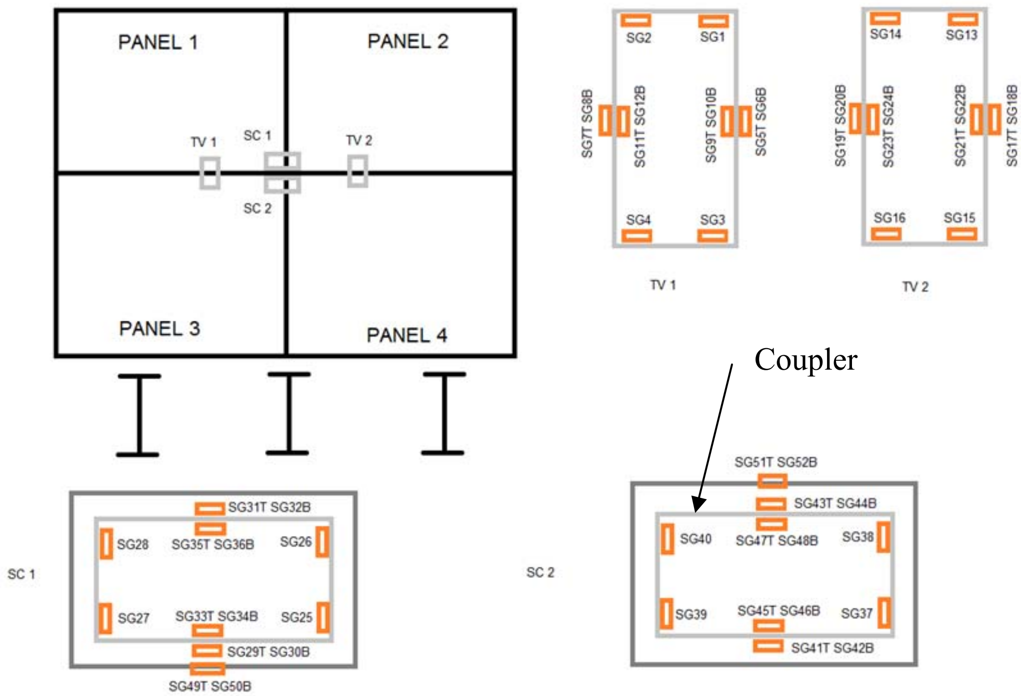


Figure 5-6: Internal Slab Instrumentation Locations

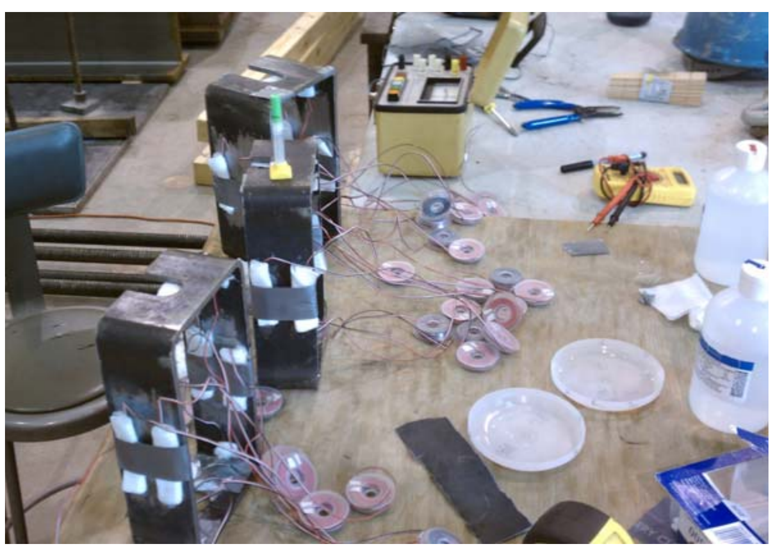


Figure 5-7: HSS Coupler Strain Gage Instrumentation

Figure 5-8: HSS Coupler Instrumentation at Staged Construction Joint

5.4.2 External Instrumentation

External instrumentation utilized for Load Case I of this investigation is shown in Figure 5-9. The staged construction joint was monitored for relative panel spread by both laser displacement gages and concrete strain gages across the closure pour. Similarly, the transverse joint was also monitored for relative panel spread using laser displacement and concrete strain gages. Vertical deflections of the deck system were measured at both exterior overhang locations as well as intermediate locations between interior girders.

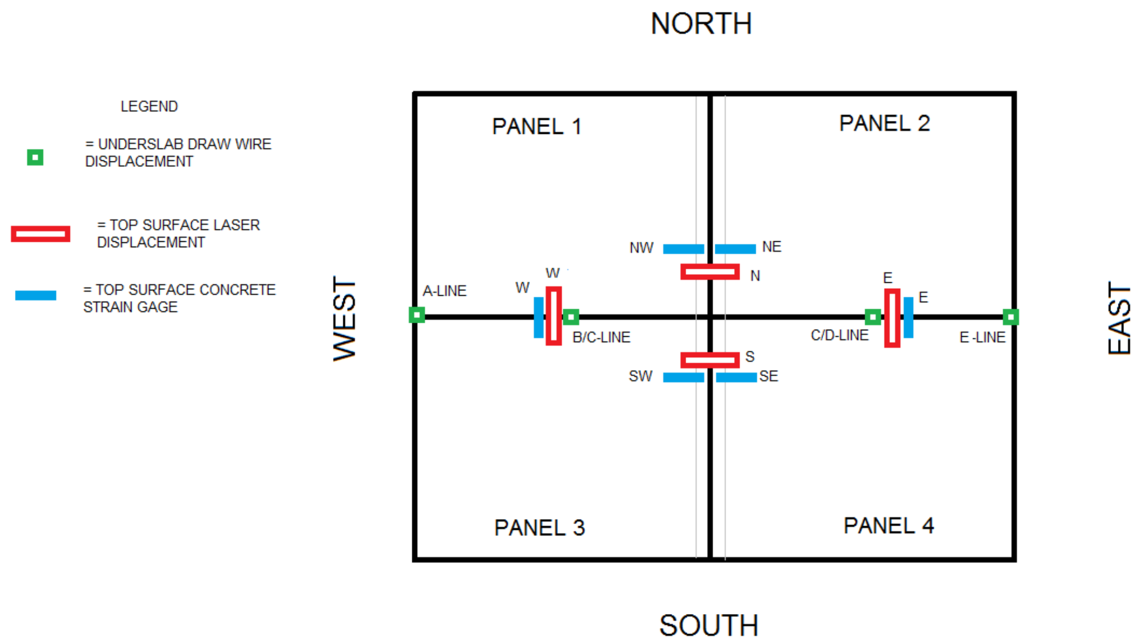


Figure 5-9: External Slab Instrumentation Plan – Load Case I

Figure 5-10 shows a typical drawstring displacement gage application attached to the underside of the concrete deck system. Figure 5-11 shows a typical joint gage configuration consisting of a laser displacement and various concrete strain gages positioned across the staged construction joint.

Figure 5-10: Typical Under-Slab Drawstring Displacement Gage Application

Figure 5-11: Typical Concrete Strain Gage and Laser Displacement Gage

External instrumentation for Load Case II was limited in comparison to Load Case I. Laser displacement sensors were oriented under the slab in order to monitor global deflections of the system at select locations. These laser displacement sensors were positioned at the same locations as the drawstring sensors utilized in Load Case I instrumentation as detailed above. Additional laser sensors were positioned across the transverse joint on each side of the loading frame to measure relative panel spread during applied loading.

5.5 Data Acquisition

For the purposes of this investigation, data acquisition was completed using both a Pacific Instruments 6000 Series Data Acquisition System (DAS) as well as using other manual analog data recording methods. Load Case I was closely monitored by the DAS in order to provide insight into both the global slab behavior and the HSS coupler resistance mechanisms. Load Case II, which served mainly as a validation test for the longitudinal joint, received limited instrumentation, which was manually recorded using analog methods. The DAS utilized in this experiment is shown below in Figure 5-12.

Figure 5-12: Data Acquisition System

5.5.1 Sampling Rate

For portions of the experimentation where the Pacific Instrumentation DAS was used, data was recorded with a sampling rate of 200 samples per second. This sampling rate was appropriate for monitoring instrumentation during cyclic testing performed at a frequency of 2 Hz without causing truncation errors in the recorded data.

Chapter 6: Test Results and Data Presentation

6.1 Overview

This chapter presents the results of the testing program as previously described. Results in this chapter are grouped primarily by load case. For each load case, the static and cyclic test results are presented separately. The static test data consists of the pre-cyclic and post-cyclic static testing data and offers snapshots of the system behavior before and after cyclic loading. Conversely, the cyclic test data monitors progressive changes in system behavior at discrete intervals during cyclic load application. For each test, results are presented in terms of the following three categories: visual inspections, external instrumentation, and internal instrumentation.

6.2 Load Case I Static Test Results

Static load tests were completed by the research team both before and after cyclic loading. As previously mentioned, the bridge specimen was heavily instrumented for Load Case I to provide information about global system behavior as well as to explore local panel-to-panel joint behavior at coupler locations.

6.2.1 Visual Inspection

The research team visually monitored the deck surface of the bridge specimen for cracking during both pre- and post-cyclic static load testing. No discrete cracking was observed in either the concrete of the precast panel or within the grouted areas of the joints. However, the research team did observe minor hairline cracking at certain concrete panel-to-grout interface locations during static load testing at peak loadings. These hairline cracks were only detectable by the research team under two unique circumstances: in areas where grout overspill created a thin membrane of grout across interfaces, and when interfaces were viewed through marking paint. Figure 6-1 shows a hairline crack visible through yellow marking paint at a transverse grout-to-concrete interface.

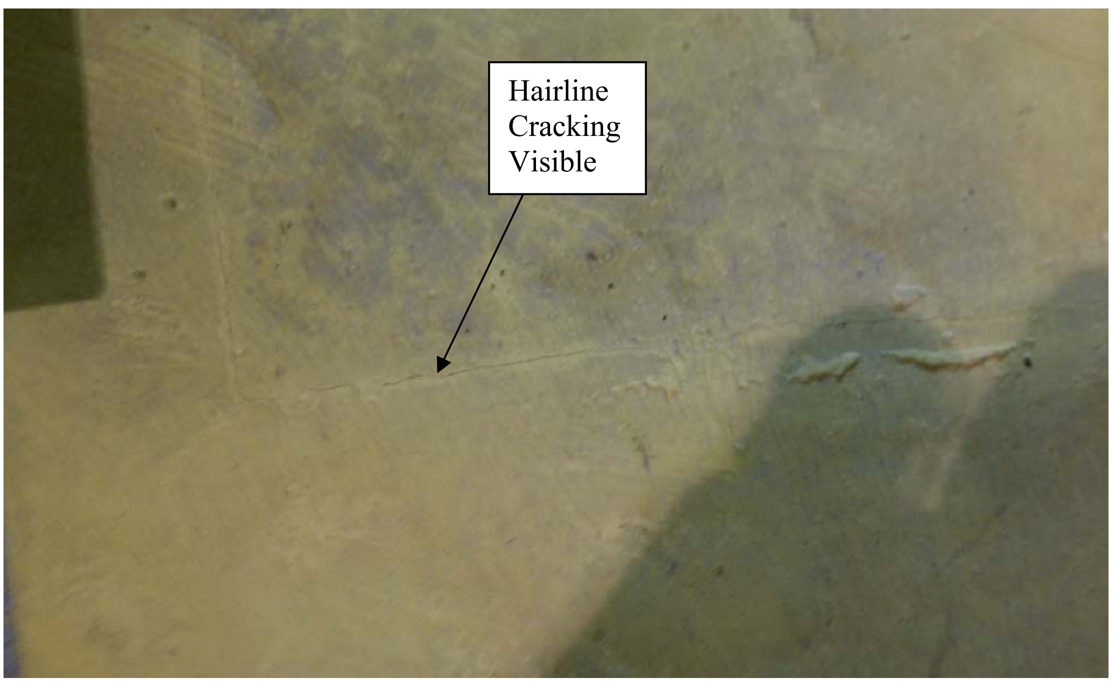


Figure 6-1: Hairline Cracking at Transverse Grout-to-Concrete Interface

6.2.2 External Instrumentation

As previously discussed in Chapter 5, the external instrumentation consisted of top surface concrete strain gages, drawstring displacement gages, and laser displacement gages. Static test results for these gages were extracted for both pre- and post-cyclic static tests and plotted for comparison. Figures 6-2 and 6-3 show pre- and post-cyclic static test results for top surface concrete strain gages, respectively. The results acquired from these surface concrete strain gages were of measureable levels given sensor precision and appear to show consistent and valid trending. Note the shifting up in positive strains in the post-cyclic plots – indicating permanent strains resulting from the cyclic loading. However, slopes of the plots in Figures 6-2 and 6-3 are almost identical – indicating no change in the slab stiffness from the cyclic loading.

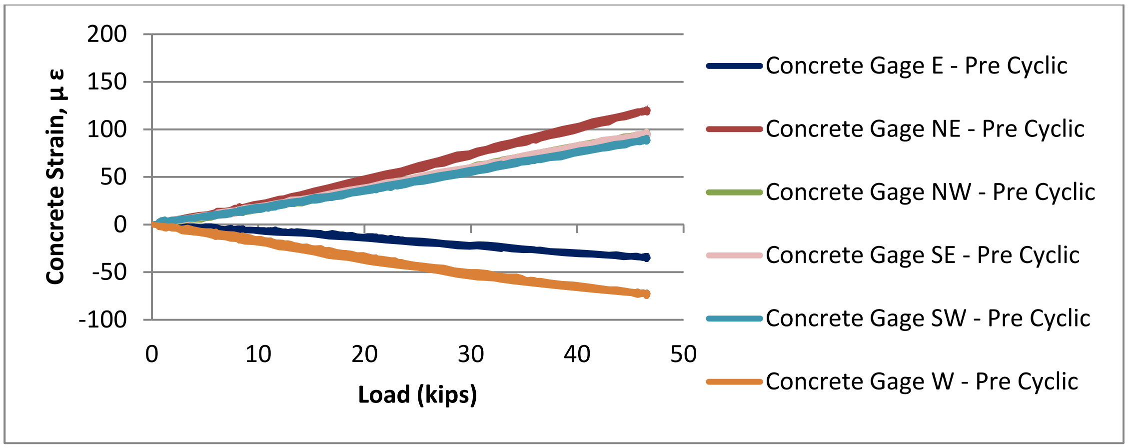


Figure 6-2: Pre-Cyclic Static Concrete Strain Gage Results

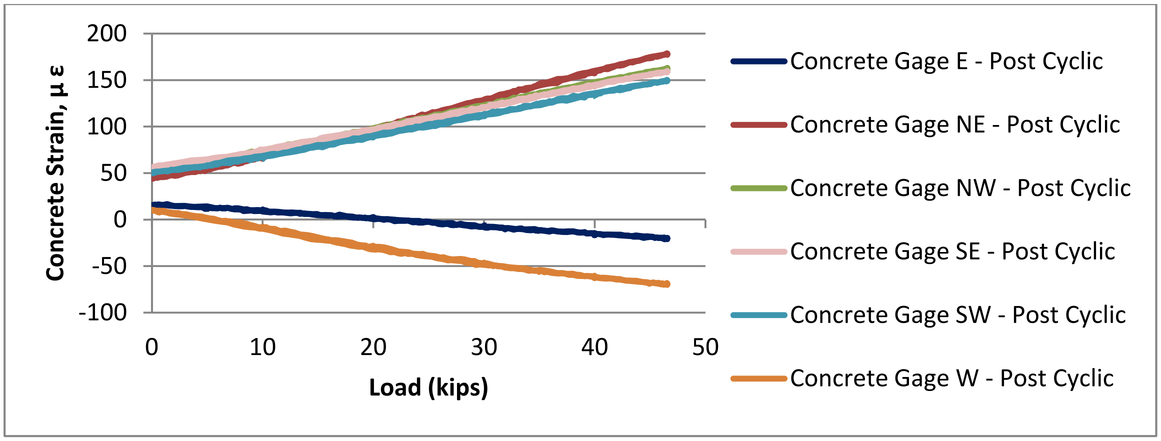


Figure 6-3: Post-Cyclic Static Concrete Strain Gage Results

Figures 6-4 and 6-5 show pre- and post-cyclic static test results for the under-slab drawstring displacement gages, respectively. The results acquired from these drawstring deflection gages appear to approach instrument precision levels as exhibited by the presence of significant electrical noise on the following plots. However, peak results do exceed background electrical noise and reflect small increases in deflections with increasing load and very stiff slab behavior. Also, the plots reflect small permanent deflections resulting from the cyclic loading.

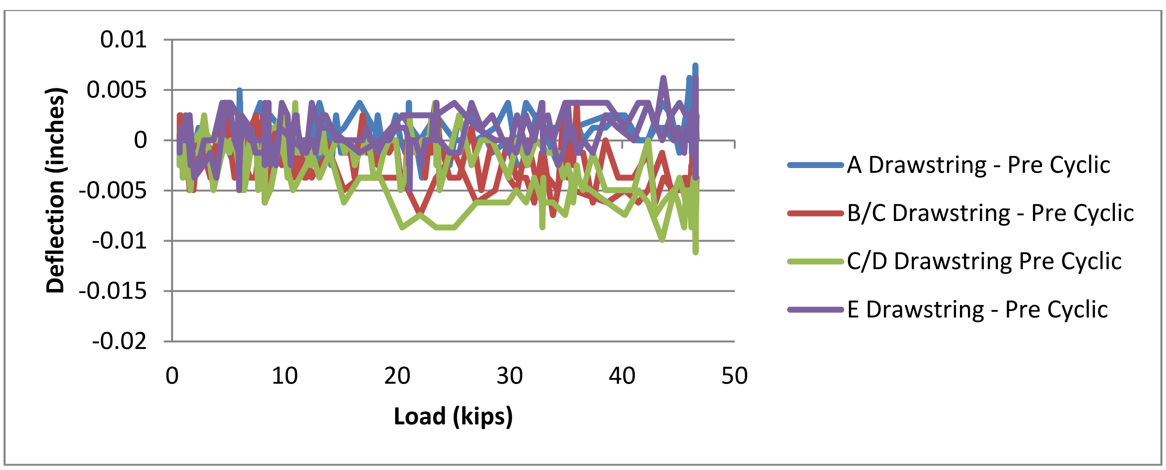


Figure 6-4: Pre-Cyclic Static Under-Slab Displacement Gage Results

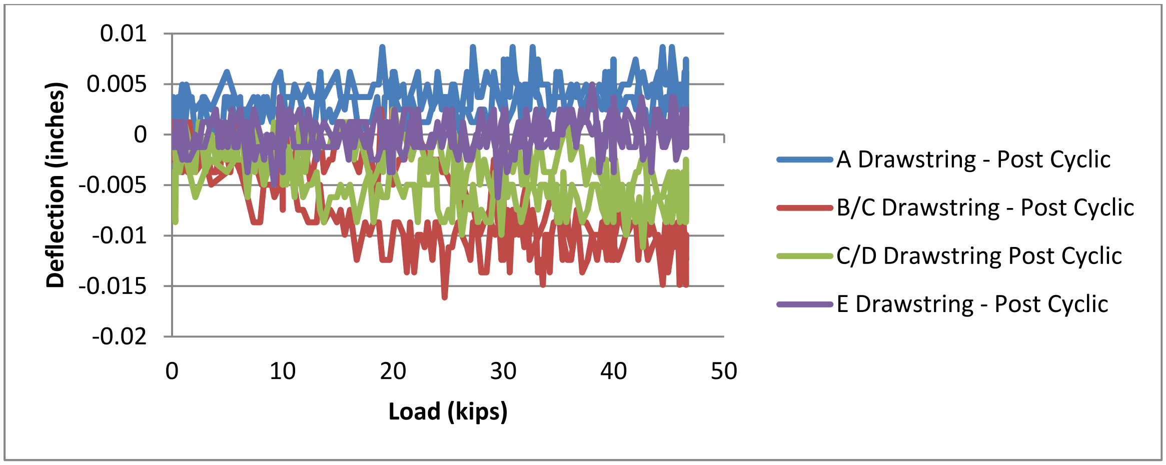


Figure 6-5: Post-Cyclic Static Under-Slab Displacement Gage Results

Figures 6-6 and 6-7 show pre- and post-cyclic static test results for the laser displacement gages, respectively. Again, the results acquired from these displacement gages appear to approach instrument precision levels as exhibited by the presence of significant electrical noise on the following plots. However, peak results do exceed background electrical noise and reflect similar trends as those shown in Figures 6-2 through 6-5.

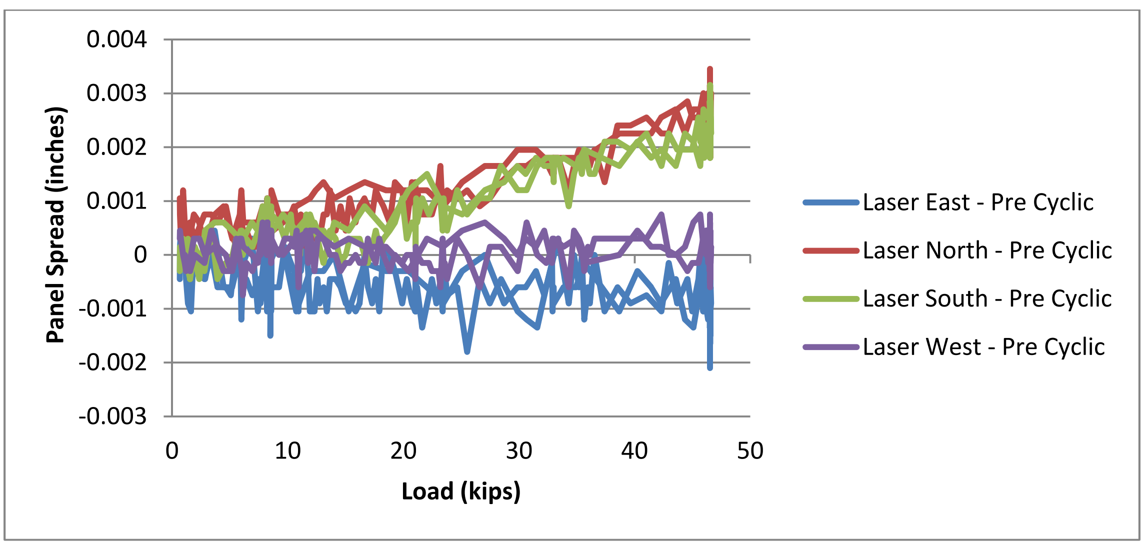


Figure 6-6: Pre-Cyclic Static Laser Displacement Gage Results

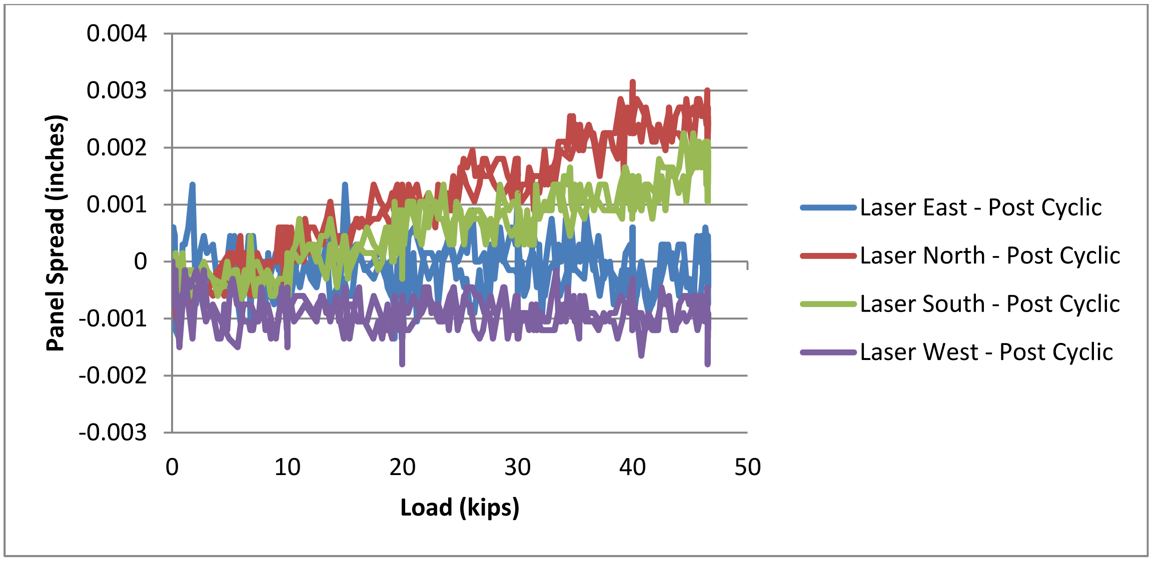


Figure 6-7: Post-Cyclic Static Laser Displacement Gage Results

6.2.3 Internal Instrumentation

As previously discussed in Chapter 5, the internal instrumentation consisted of four heavily instrumented HSS couplers oriented in both the transverse joint and staged construction joint locations. For the purpose of results presentation, each instrumented joint detail will be presented independently. Figures 6-8 and 6-9 show pre- and post-cyclic static test results, respectively, for the transverse joint instrumented HSS couplers. The results acquired from these instrumented HSS couplers were of measureable levels given sensor precision and appear to show consistent and valid trending, with the exception of two apparent strain gage failures, which are omitted from the plots below. The cyclic loading appeared to result in significant permanent strains in the HSS couplers, but rather insignificant changes in stiffness.

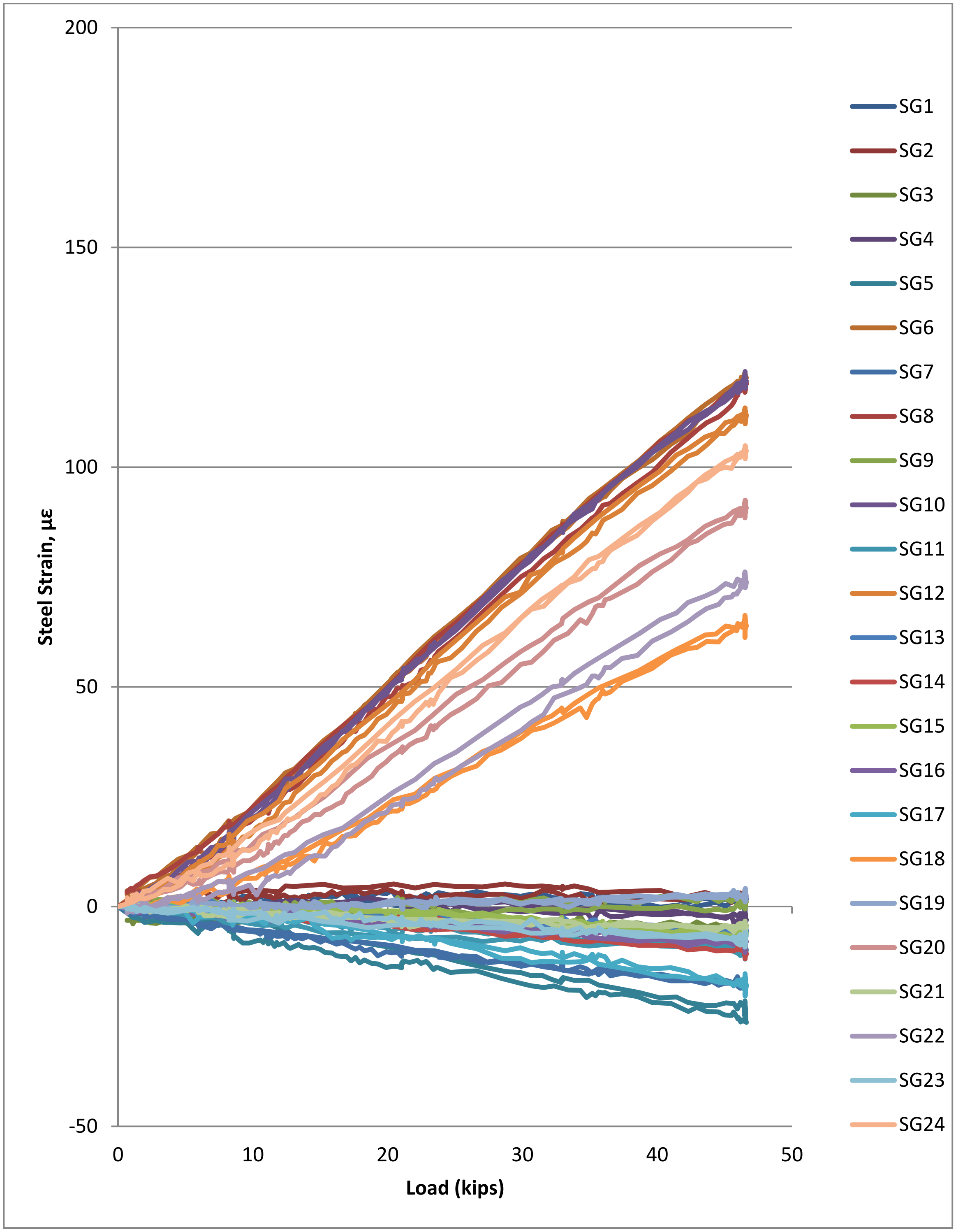


Figure 6-8: Pre-Cyclic Static Transverse Joint HSS Coupler Gage Results

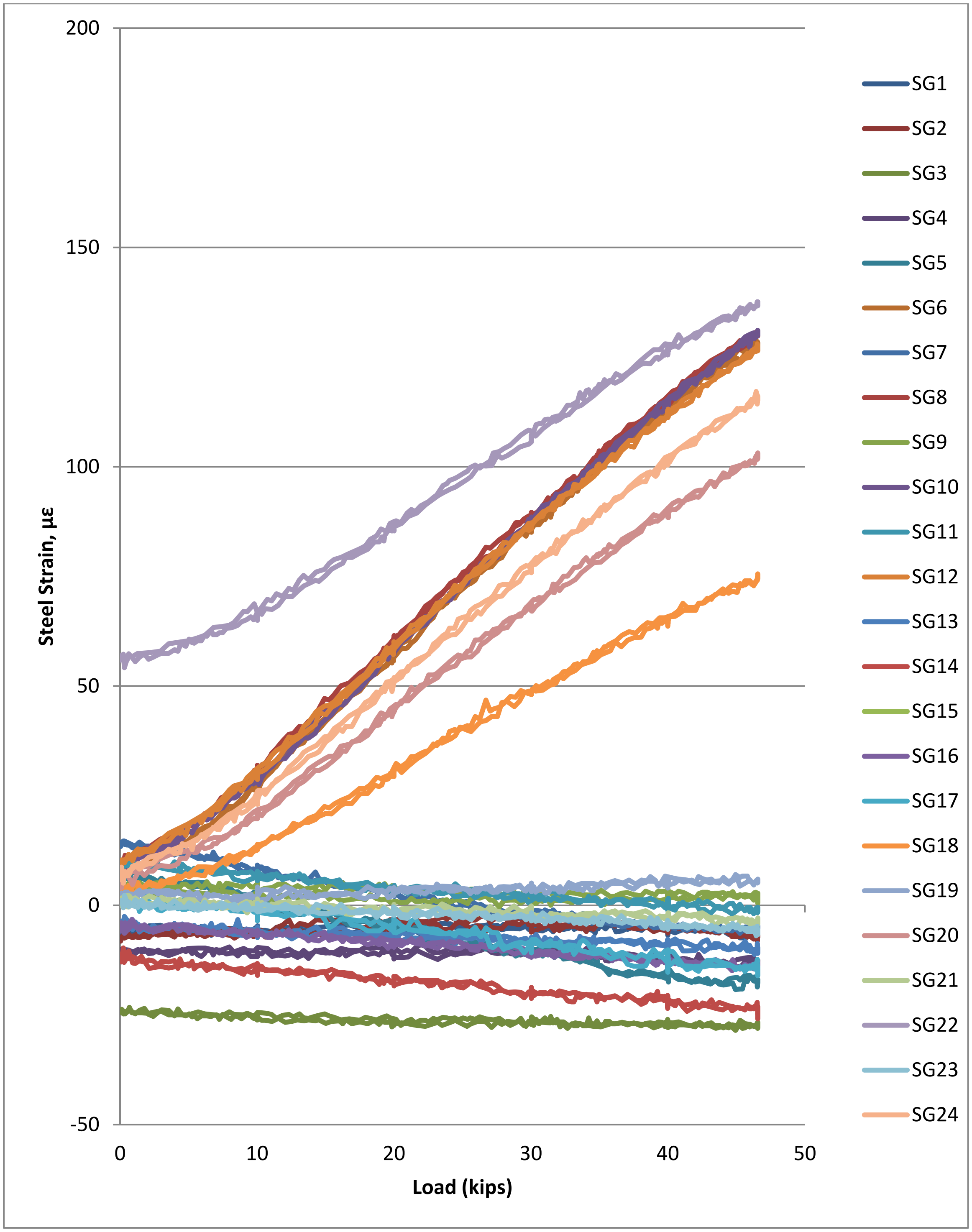


Figure 6-9: Post-Cyclic Static Transverse Joint HSS Coupler Gage Results

Figures 6-10 and 6-11 show pre- and post-cyclic static test results for the staged construction joint instrumented HSS couplers, respectively. The results acquired from these instrumented HSS couplers and stirrups were of measureable levels given sensor precision and appear to show consistent and valid trending results that were similar to those for the transverse joint HSS couplers.

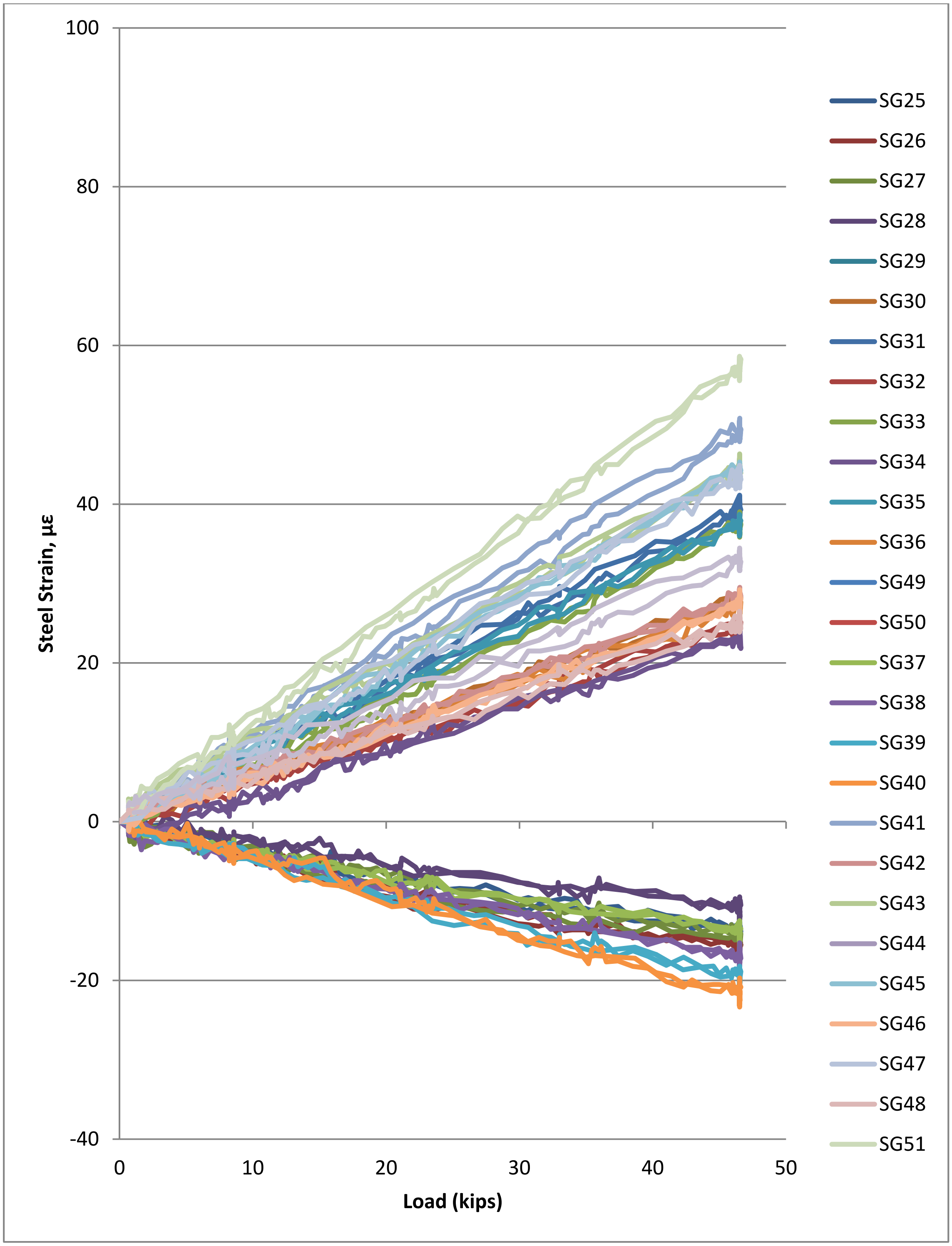


Figure 6-10: Pre-Cyclic Static Staged Construction Joint HSS Coupler Gage Results

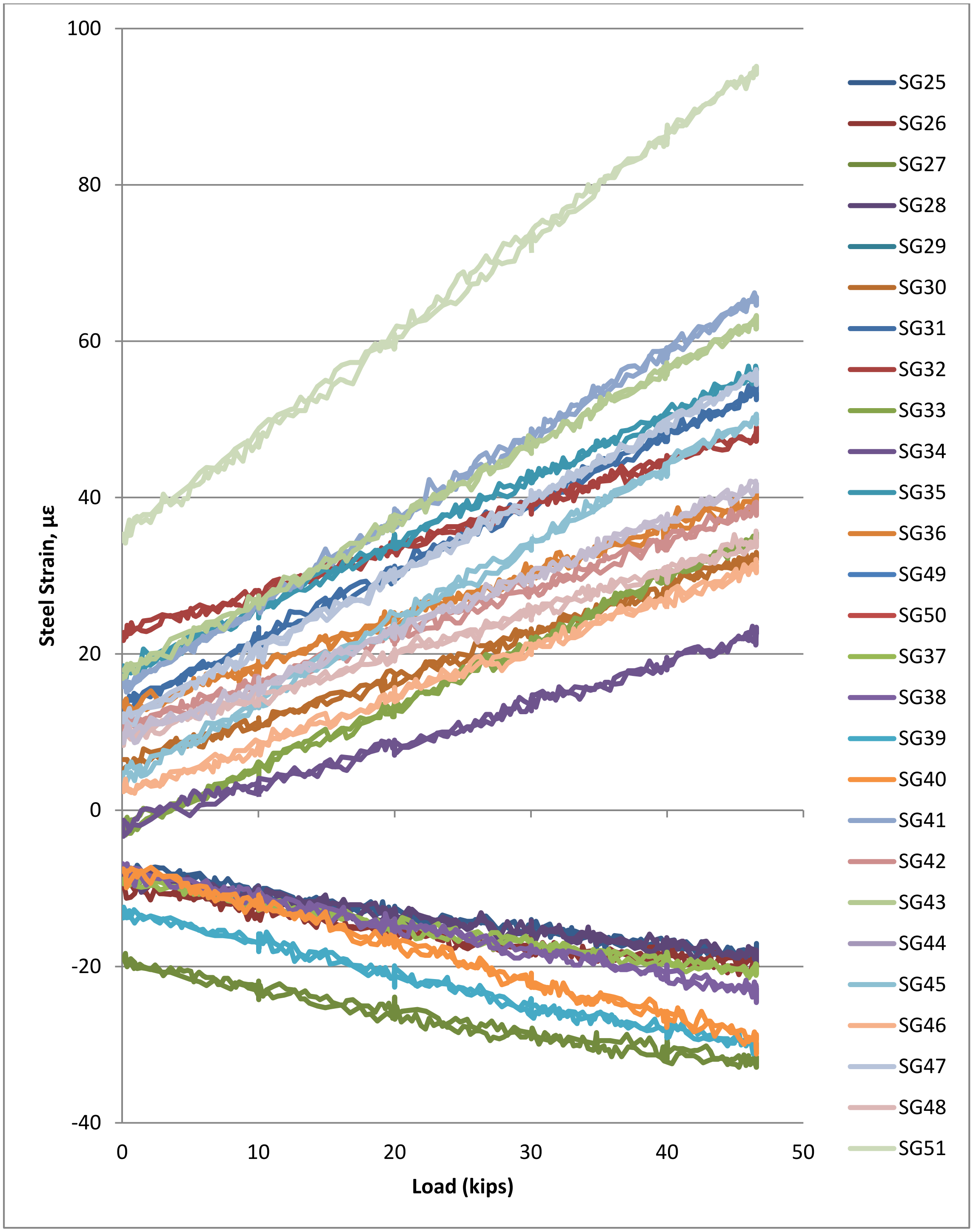


Figure 6-11: Post-Cyclic Static Staged Construction Joint HSS Coupler Gage Results

6.3 Load Case I Cyclic Test Results

Cyclic load testing was completed as planned without interruption. The research team concluded the cyclic load testing at approximately 2,300,000 load cycles. During cyclic load testing, it was noted that the actual applied peak cycle load varied slightly from the specified peak cycle load on the analog controller. Variations such as those noted may be a result of hydraulic system limitations and/or varying dynamic response modes of the loading frame. It is important to note that although the peak load slightly varied throughout testing, the research team assured the desired 42.6 kip load range was preserved for each load cycle. Figure 6-12 shows the actual applied peak cycle loading throughout the test measured at the actuator head, normalized with respect to the initial specified peak load. Note in this plot that the applied load did not deviate from the desired load range by more than +0.15% or less than -0.25%.

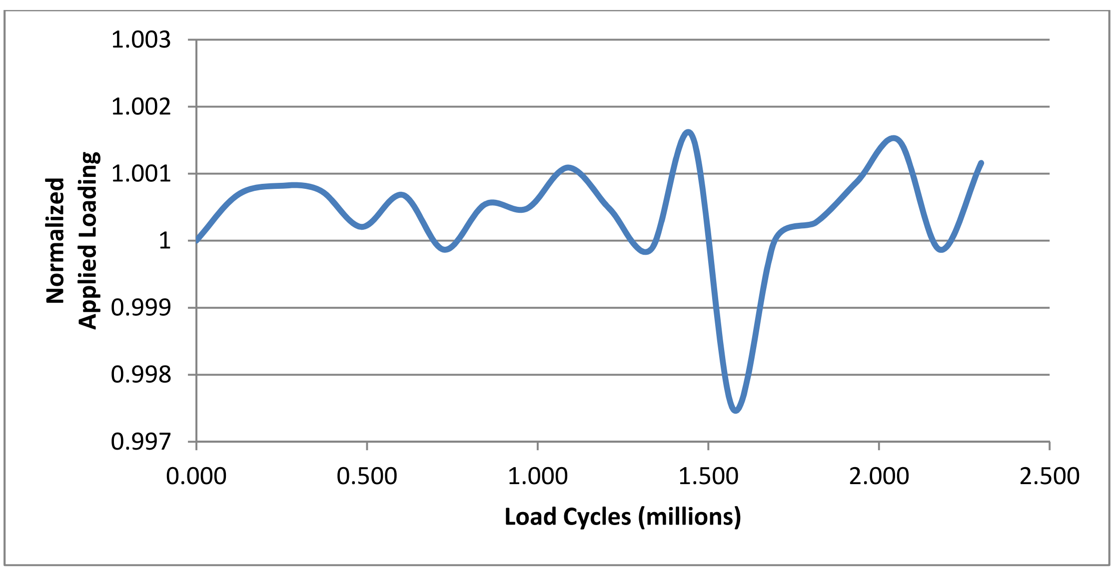


Figure 6-12: Normalized Applied Loading During Cyclic Testing

6.3.1 Visual Inspection

Visual crack inspections of the specimen were conducted throughout the cyclic testing program. No discrete cracking was observed in either the concrete of the precast panel or within any grouted areas. Any hairline interface cracks as previously described remained barely detectable, and the research team did not observe any noticeable increase in the width of these cracks.

6.3.2 External Instrumentation

As previously discussed, external instrumentation consisted of top surface concrete strain gages, drawstring displacement gages, and laser displacement gages. Figure 6-13 shows the response of the top surface concrete strain gages throughout the cyclic testing program. It is evident that the transverse joint gages, concrete gages "E"

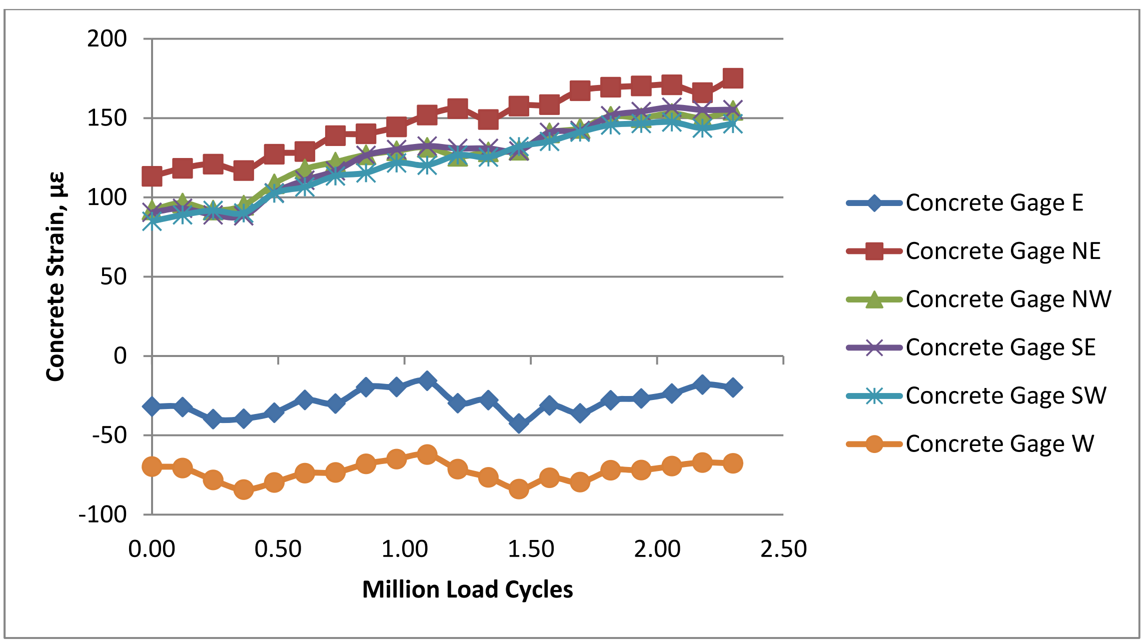


Figure 6-13: Cyclic Concrete Strain Gage Results

and “W,” exhibit consistent strains throughout cyclic testing. Conversely, the concrete strain gages oriented across the transverse joint show a gradual increase in strain throughout testing. This change in strain across the transverse joint is examined more closely in Chapter 7.

As discussed in the previous section, readings taken from the laser displacement and drawstring displacement external slab gages approached instrument precision and provided minimally useful results for the static load tests. When attempting to extract peak load values for cyclic analysis, it became obvious that the additional error introduced by the slightly varying peak loads exceeded the instrument precision and tended to have a masking effect on experimental trends. Therefore, cyclic analysis using these gage types was not feasible.

6.3.3 Internal Instrumentation

As previously discussed, the internal instrumentation consisted of four heavily instrumented HSS couplers oriented in both the transverse joint and staged construction joint locations. For the purpose of results presentation, each instrumented joint detail will be presented independently. Figure 6-14 shows the cyclic response of the transverse joint instrumented HSS couplers throughout the testing program.

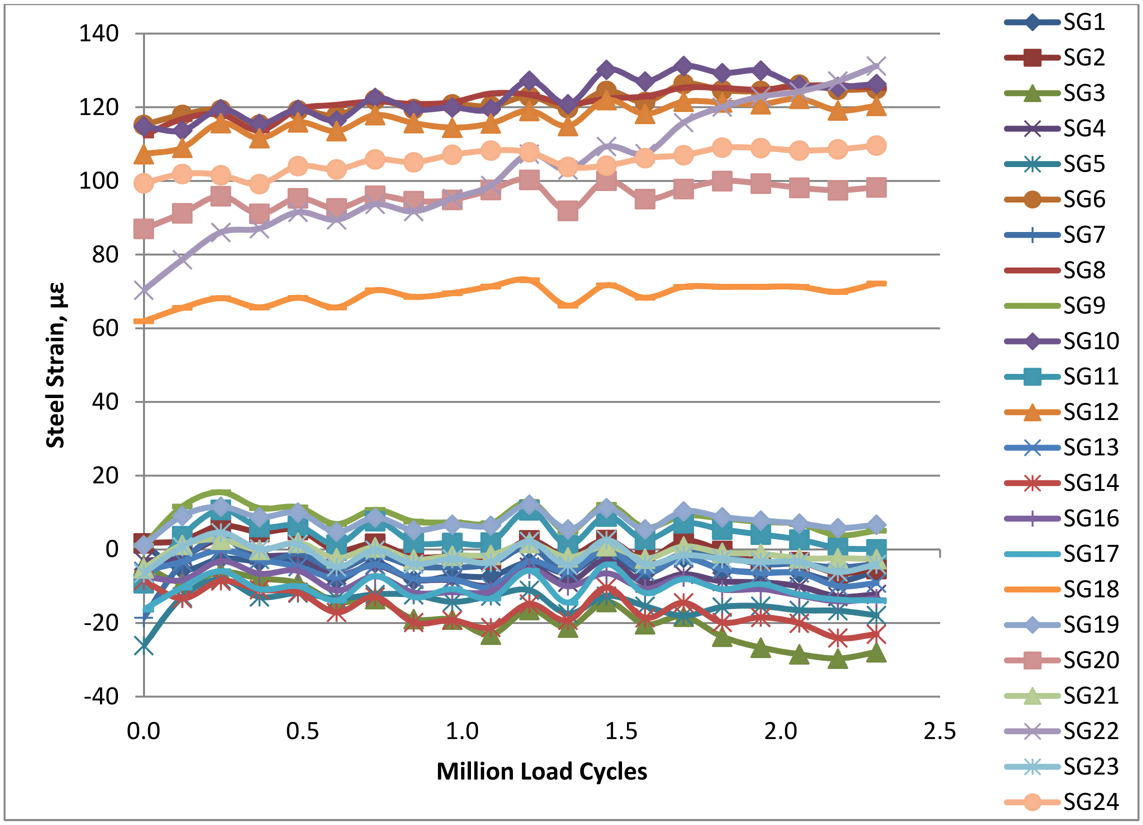


Figure 6-14: Cyclic Response of Transverse Joint HSS Coupler Gage Results

The results in the above plot are of measureable levels given sensor precision and appear to show valid trending consistent with static testing results, with the exception of the two apparent strain gage failures, which are omitted from the plot. Figure 6-15 shows the cyclic response of the staged construction joint instrumented HSS couplers throughout the testing program. The results acquired from these instrumented HSS couplers and stirrups are of measureable levels given sensor precision and appear to show valid trending consistent with static testing results.

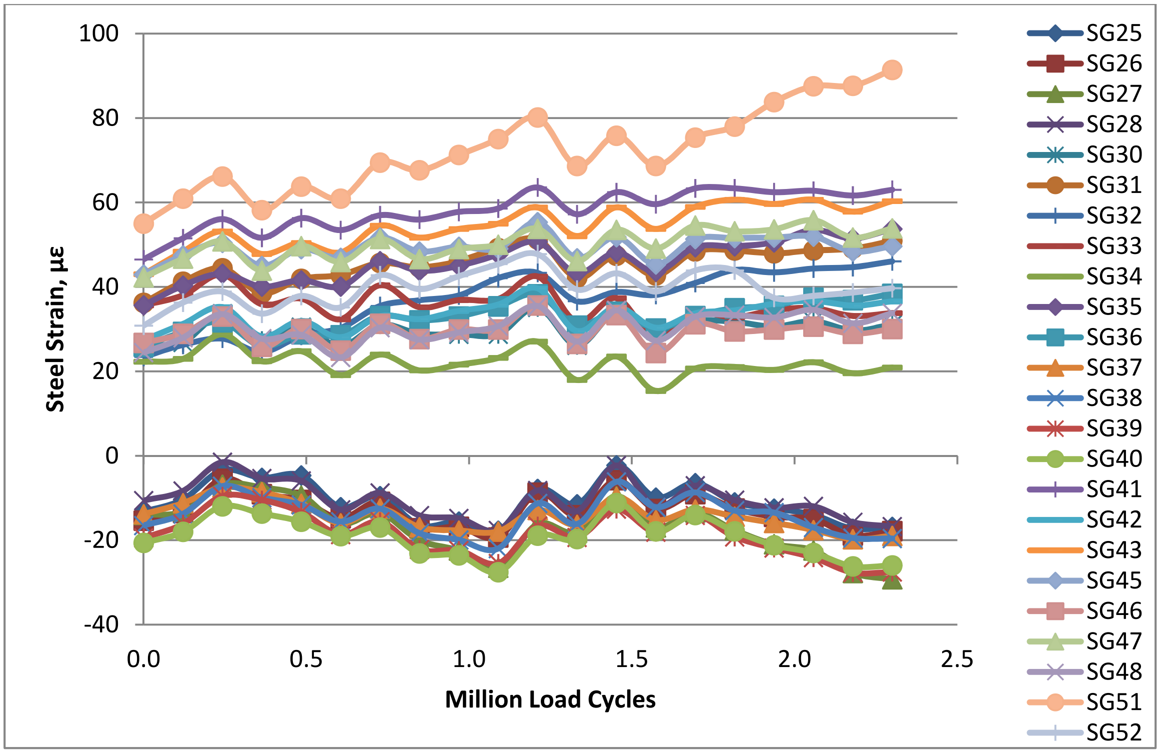


Figure 6-15: Cyclic Response of Staged Construction HSS Coupler Gage Results

6.4 Load Case II Static Test Results

Pre- and post-cyclic static load tests were completed by the research team after repositioning the load frame to the Load Case II location. As previously discussed, Load Case II served primarily as an additional validation of system performance and utilized limited instrumentation monitored by analog methods. Instrumentation consisted of four laser displacement gages intended to monitor global system behavior. The research team chose to conduct each static test twice for Load Case II in order to provide possible insight into global behavioral trends observed earlier during Load Case I static testing.

6.4.1 Visual Inspection

Visual inspections by the research team showed no discrete cracking visible from the top surface of the bridge deck system. Similar to all previous tests, minor hairline cracks were barely detectable at grout-concrete interface locations and were not observed to increase in width throughout testing. The research team also did note the development of minor cracking around the grouted area of the longitudinal joint as visible when viewing the girder ends as shown below in Figure 6-16. These minor cracks developed along the grout-to-concrete interfaces upon initial Load Case II specimen loading, were barely detectable, and did not appear to grow in width during testing.

Figure 6-16: Minor Cracking at Longitudinal Joint Grout-to-Concrete Interface

The research team also noted cracking at the outermost grouted splice location in the transverse joint visible from the specimen side as shown in Figure 6-17. This grout cracking appeared to be isolated and only present on the edge splice location on the side of the applied loading.

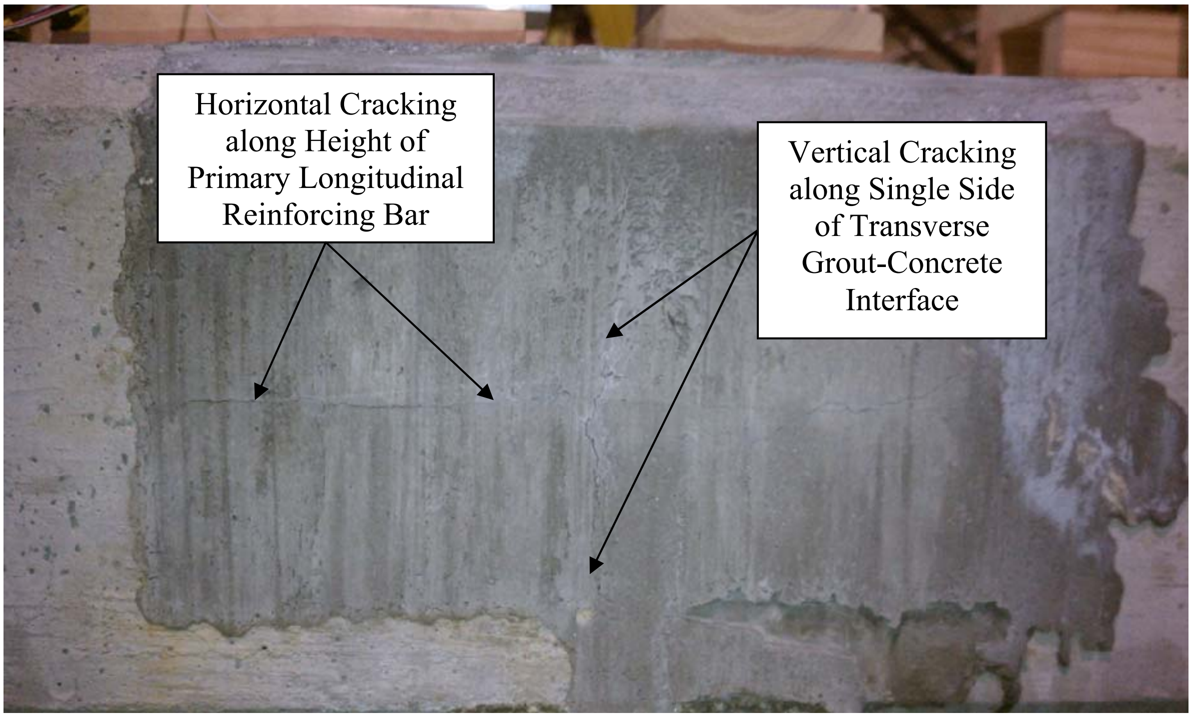


Figure 6-17: Grout Cracking at Transverse Joint Edge Splice Location

Cracking in this area tended to occur vertically along a single side of the transverse grout-concrete interface joint, as well as horizontally at the height of the primary longitudinal reinforcing bar in this area, as indicated in Figure 6-17.

6.4.2 External Instrumentation

As previously discussed in Chapter 5, the external slab instrumentation for Load Case II consisted of four laser displacement gages. Two of these gages were positioned to measure slab deflections from underneath the bridge deck. The remaining two gages were utilized to monitor panel-to-panel joints for panel spread. Figure 6-18 and 6-19 show pre- and post-cyclic static test results for the under-slab deflection gages, respectively. Results of each test are of measureable levels given sensor precision and appear to show consistent and valid trending. Note that the results reflect small permanent slab deflections from the cyclic loading, but no distinct changes in slab stiffness.

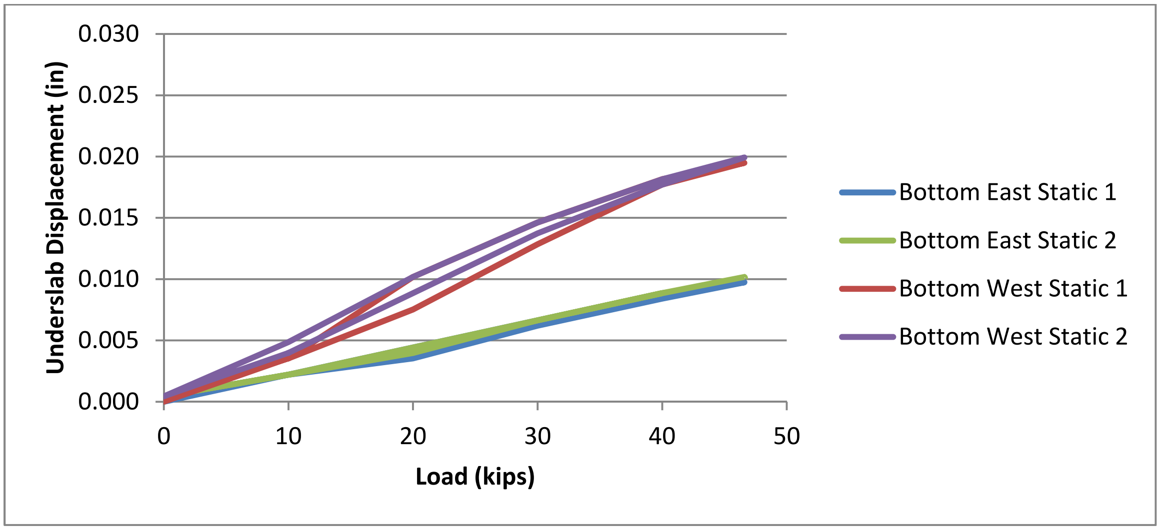


Figure 6-18: Pre-Cyclic Static Load Test Under-Slab Deflection Results

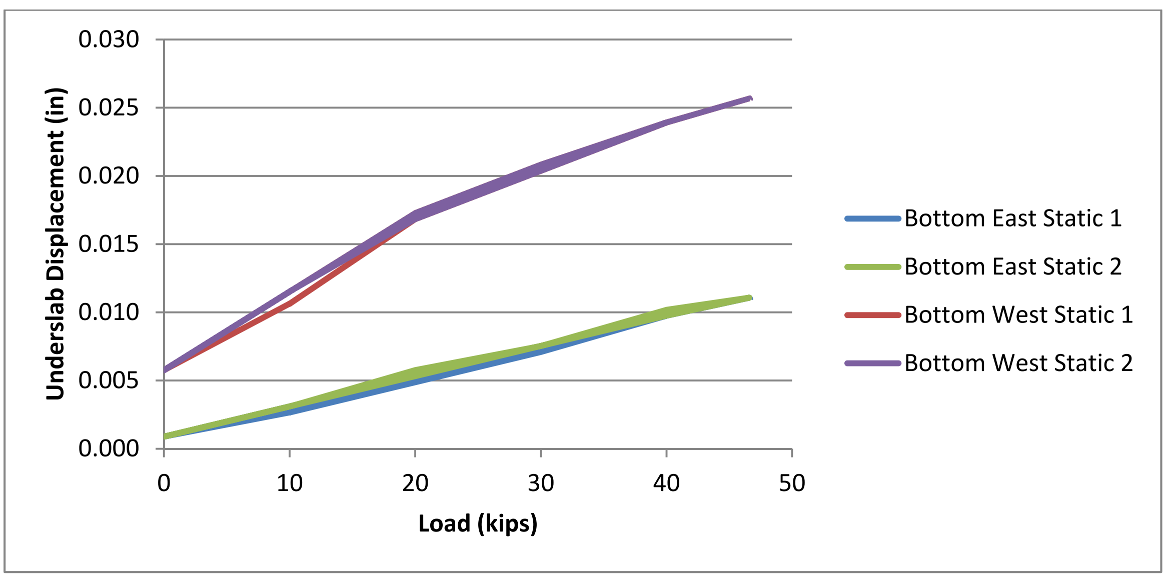


Figure 6-19: Post-Cyclic Static Load Test Under-Slab Deflection Results

The remaining two laser gages, used to monitor panel spread, yielded results approaching analog instrument precision limits and thus, were monitored only for peak values. Table 6-1 presents baseline and peak panel spread measurements on each side of the applied load during static load testing. For reference, a value of zero corresponds to the unloaded transverse joint condition prior to static load testing. Positive values under loading correspond to a spreading tendency of the transverse joint, while negative values correspond to a closing tendency of the transverse joint.

Table 6-1: Load Case II Static Load Panel Spread Results

When Tested	Load Level	East Side* (*in.)*	West Side* (*in.*)
Pre-Cyclic	Zero Load	0.000	0.000
	Peak Load	-0.001	-0.002
Post-Cyclic	Zero Load	-0.004	+0.007
	Peak Load	-0.005	+0.005

* + Indicates spreading
- Indicates closing

6.5 Load Case II Cyclic Test Results

Cyclic load testing was completed as planned with three temporary interruptions for mandatory facility electrical shutdowns. The research team concluded cyclic testing at 2,000,000 cycles without incident.

6.5.1 Visual Inspection

Visual inspection results during the Load Case II cyclic loading were similar to those noted above in the static testing inspection summary.

Chapter 7: Interpretation and Analysis of Test Results

7.1 Overview

Although certain trends may be readily identifiable from the presentation of results in the previous chapter, a more thorough analysis and interpretation of data is necessary in order to sufficiently explore the behavioral response and joint performance of the modified CD-2 type system tested in this investigation. In this chapter, each load case is analyzed separately utilizing the following methodology. First, a brief review of the load case is presented highlighting the locations of maximized force effects. Next, the global behavioral response of the deck system during both static and cyclic testing is explored, referencing applicable test results. Finally, each of the three main joint types included in this investigation, i.e., transverse, staged construction, and longitudinal joints, is similarly explored utilizing relevant experimental results.

7.2 Load Case I

As discussed in Chapter 5, the axle load location chosen for Load Case I corresponds to the condition that induces the maximum negative moment across the staged construction joint. In addition, each tire contact area is positioned symmetrically across the transverse joint in order to evaluate local moment durability of the transverse joint under directly applied loads.

7.2.1 Global System Behavior

Throughout the laboratory specimen design, the research team anticipated that overhang loadings would be resisted by simple flexural mechanisms, while any loadings applied between supporting girders would be resisted by a combination of flexure and arching action. It was anticipated that the deck system would mimic monolithic cast-in-place bridge deck behavior throughout load testing. Prior to testing, anticipated peak live load deflections based on equivalent AASHTO slab strips were calculated and are shown below in Figure 7-1.

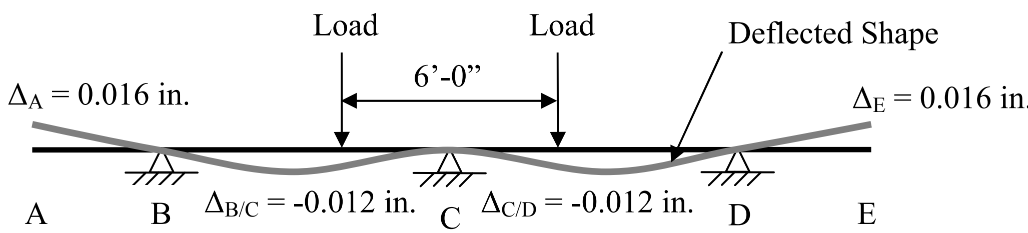


Figure 7-1: Load Case I Anticipated Live Load Deflections

For this investigation, under-slab drawstring displacement gage data provides the most direct information regarding global system behavior. Figure 7-2 shows the under-slab displacements during pre-cyclic static load testing.

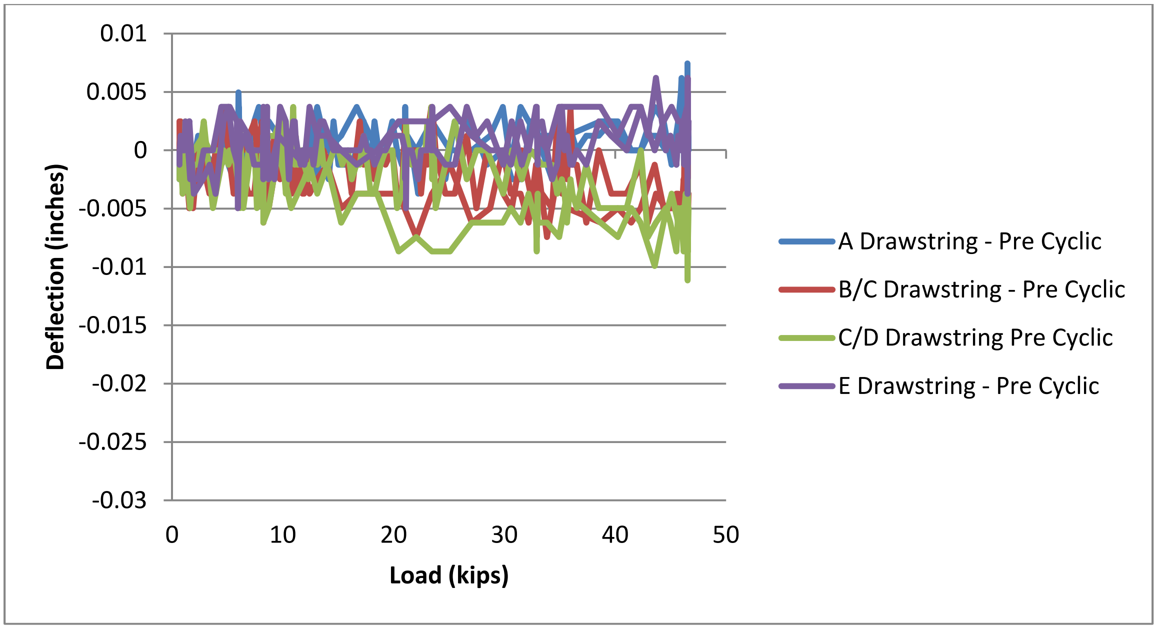


Figure 7-2: Pre-Cyclic Static Load Test Under-Slab Displacement Gage Results – Load Case I

As shown above in Figure 7-2, peak overhang deflections along control lines A and E reach approximately 0.006 +/- 0.002 inches. When compared to the calculated live load deflections shown in Figure 7-1, it appears the overhang portion of the deck system exhibits slightly stiffer behavior than predicted. This disparity may be due to the concrete having a greater stiffness than originally anticipated by design or due to the support girders introducing small amounts of rotational stiffness which were neglected by design analysis. Peak recorded deflections between interior girders (B/C and C/D gages) are approximately -0.007 +/- 0.002 inches. Deflections at these locations show reasonably good agreement with the predicted -0.012 inches.

Next, it is important to compare the pre-cyclic static load test data to the post-cyclic static load test data. Figure 7-3 shows the post-cyclic displacement gage response of the deck system.

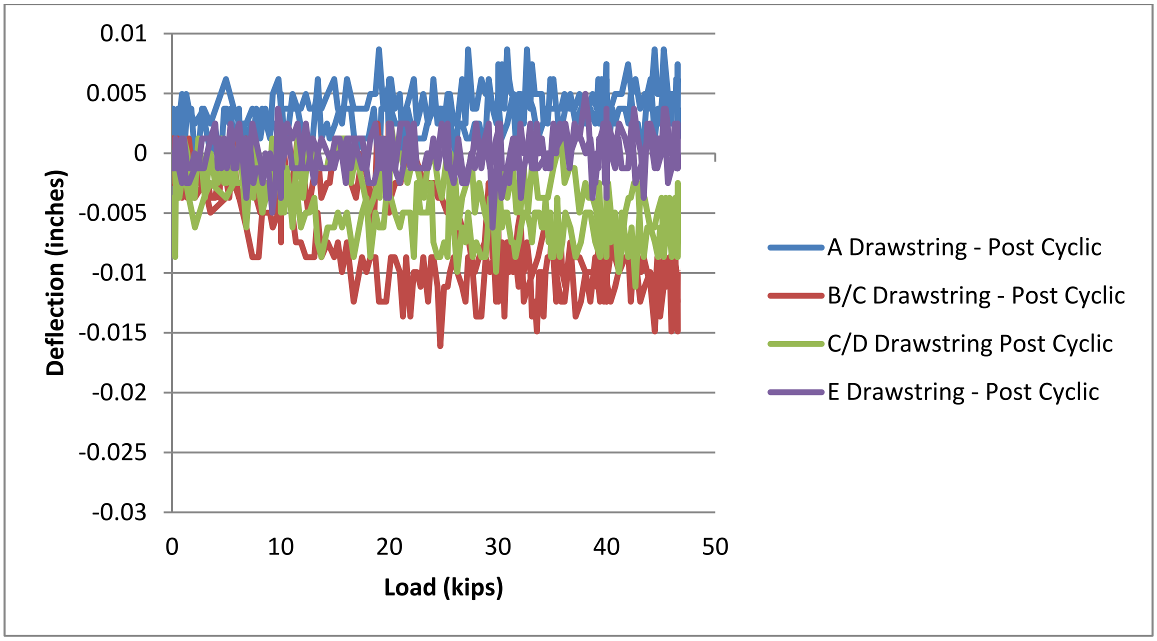


Figure 7-3: Post-Cyclic Static Load Test Under-Slab Displacement Gage Results – Load Case I

It is interesting to note that the post-cyclic test data exhibits both greater spread and larger slab deflections than the pre-cyclic test data. Peak overhang deflections along control lines A and E reach 0.009 inches, approximately 0.001 inches greater than in pre-cyclic testing. In addition, the maximum slab displacement between interior girders increases to peak values approaching -0.015 inches, about 0.005 inches greater than pre-cyclic testing.

The disparity in pre-cyclic and post-cyclic static test peaks may suggest that a minor degradation of stiffness occurred as a result of the cyclic Load Case I testing. Because visual inspections by the research team did not detect cracking within the concrete panels, it is logical to conclude that any degradation of stiffness that was globally observed after cyclic testing was likely concentrated at a joint detail. The following sections will include a thorough exploration of possible stiffness degradation at each joint detail type.

In summary, the modified CD-2 type deck system tested in this investigation exhibited global system behavior similar to that predicted in specimen design. Any minor cracking detected by the research team was limited, well-controlled in all areas, and did not pose serviceability concerns for the deck system. Although a slight degradation of stiffness was observed after cyclic testing, deflections remained considerably below AASHTO service-level deflection limits of L/800.

7.2.2 Transverse Joint Behavior

The transverse joint is designed to transmit both shear and moment effects across the panel-to-panel joint. The forces transferred across this joint are a direct result of applied loading and are intended to distribute loads longitudinally to an equivalent width of resisting slab. It is extremely difficult to quantify the behavioral effects of this distribution steel without advanced finite element modeling of the system. In addition, because this distribution steel is directly prescribed by AASHTO guidelines rather than explicitly designed, the research team was unable to predict anticipated stresses at this location. It was decided that the most effective way to examine joint performance was by comparison of joint behavior before, during, and after applied loading cycles. The research team was confident that as long as no serviceability limits were reached, the joint detail performed satisfactorily.

In order to begin the analysis of the transverse joint, it is most logical to examine the surface concrete strain gages that were positioned across the joint detail. Of the surface strain gages used in this study, both the east and west gages were positioned across the transverse joint detail as can be seen in Figure 7-4.

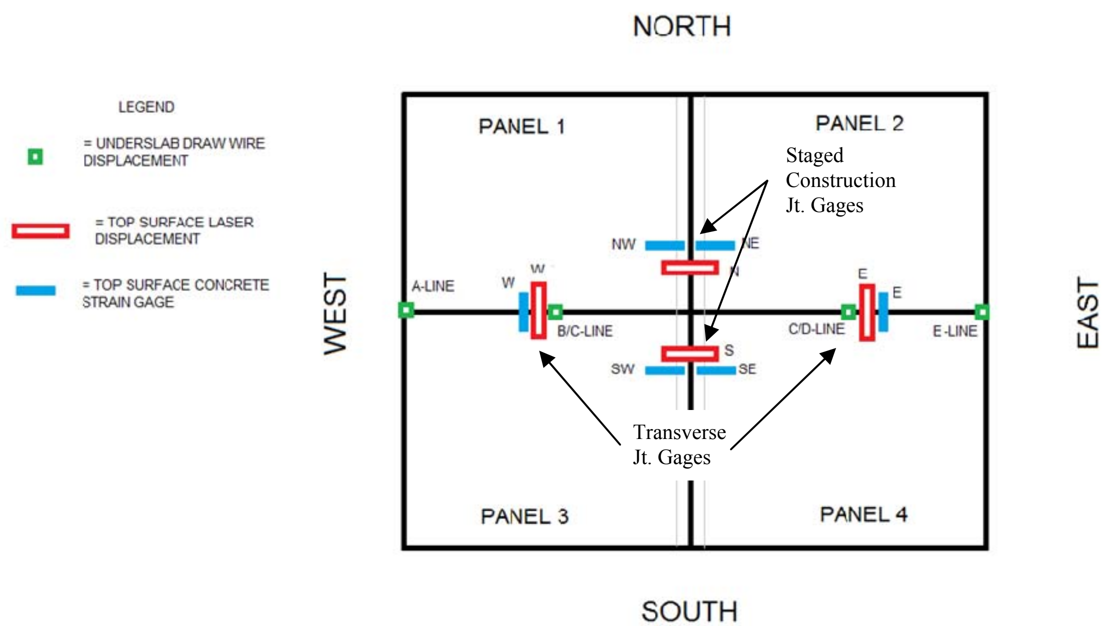


Figure 7-4: Surface Gage Locations and Orientations of Transverse Joint

For reference, the pre- and post-cyclic static load testing results are shown below in Figures 7-5 and 7-6.

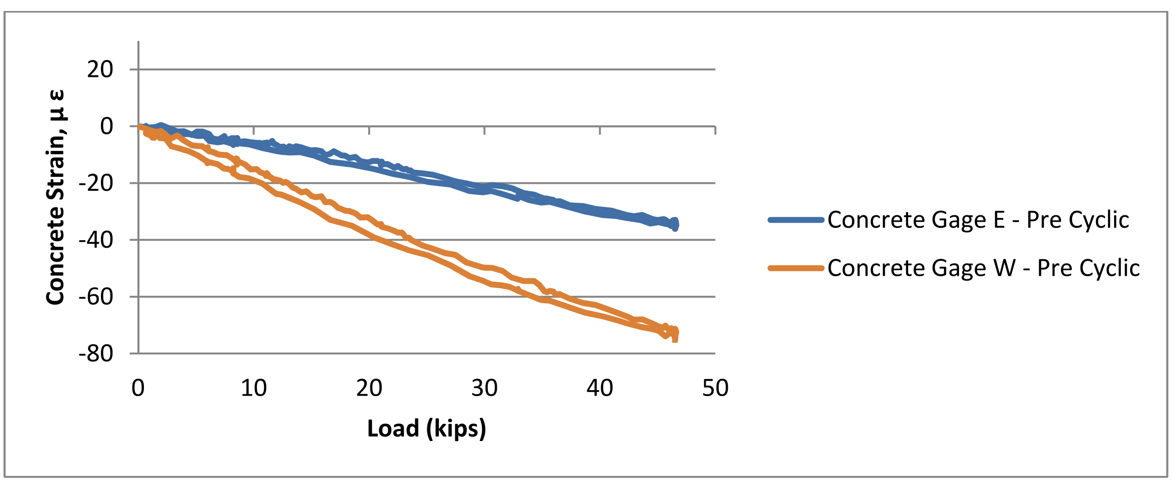


Figure 7-5: Pre-Cyclic Static Surface Strain Gage Readings across Transverse Joint – Load Case I

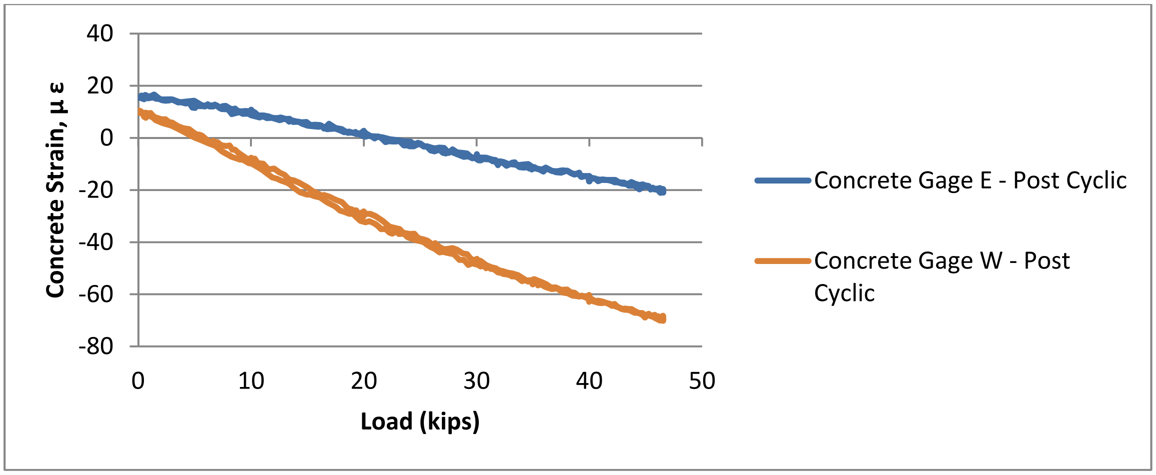


Figure 7-6: Post-Cyclic Static Surface Strain Gage Readings across Transverse Joint – Load Case I

The gages at this location are measuring negative strain values which indicate compression across the top surface of the joint. This is likely due to local bending behavior near the locations of applied loadings. The compressive strains measured in both pre- and post-cyclic testing remain relatively small throughout and do not approach compression limits for concrete. For instance, test results show that maximum compressive strains reach less than 80 microstrain, well below concrete elastic compressive strain limits of approximately 2000 microstrain. It appears that the concrete in compression exhibits relatively linear elastic behavior with consistent load-deflection slopes both before and after cyclic testing. Discrepancies between east and west side gages may have been caused by eccentricities introduced by the loading apparatus or by the relative proximity of each gage to load application points.

In each location discussed above, the post-cyclic testing results appear to be shifted upwards approximately 20 microstrain when compared to pre-cyclic results. This may indicate a slight tendency for panel spread to occur across the transverse joint location as a result of cyclic testing. For reference, if the 20 microstrain offset corresponded to a discrete crack occurrence within the gage length, the width of that crack would be less than 8×10^{-5} inches. As such, this strain offset is more likely due to a material-level phenomenon occurring at the grout-to-concrete panel interface. The research team feels this strain offset, too small to correspond to discrete cracking, may indicate a degradation of the grout-to-concrete interface at this joint location. From the available experimental results, it is difficult to decipher whether the strain shift discussed above occurred at a discrete time or progressively over the duration of cyclic testing.

The research team also examined results from the internal HSS coupler gages at the transverse joint detail in order to better understand joint behavior. For the purposes of this analysis, HSS coupler gages are divided into the following two categories: sidewall gages and endwall gages. Sidewall gages are intended to monitor force effects through the coupler sidewall, while endwall gages monitor for unintended bending of the coupler endwall during loading. Pre- and post-cyclic sidewall gages recorded similar values for couplers on each side of the specimen. Typical sidewall results for a single coupler are shown below in Figures 7-7 and 7-8.

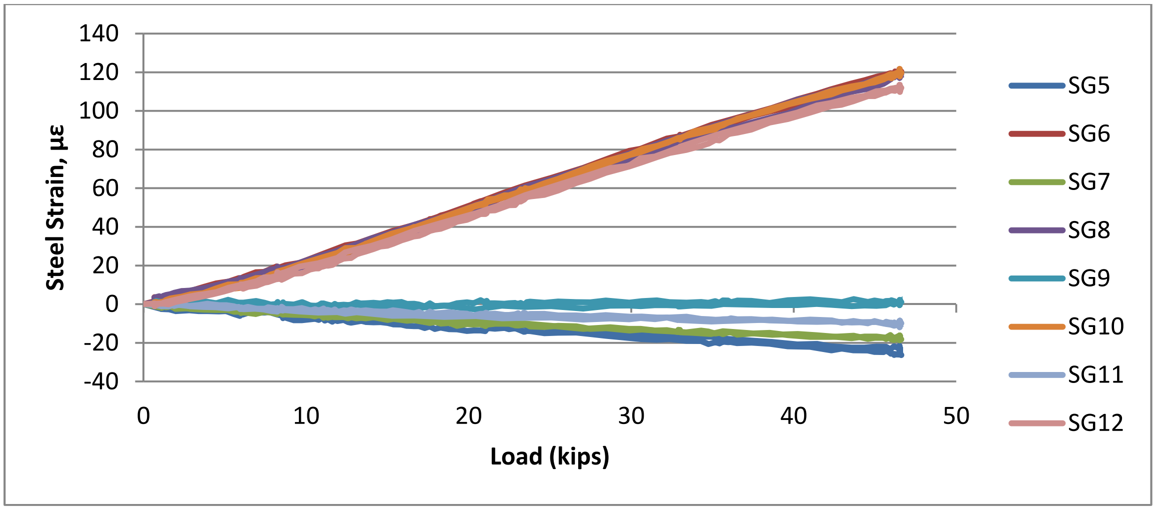


Figure 7-7: Pre-Cyclic Static Load Test Transverse Joint HSS Coupler Sidewall Gage Results – Load Case I

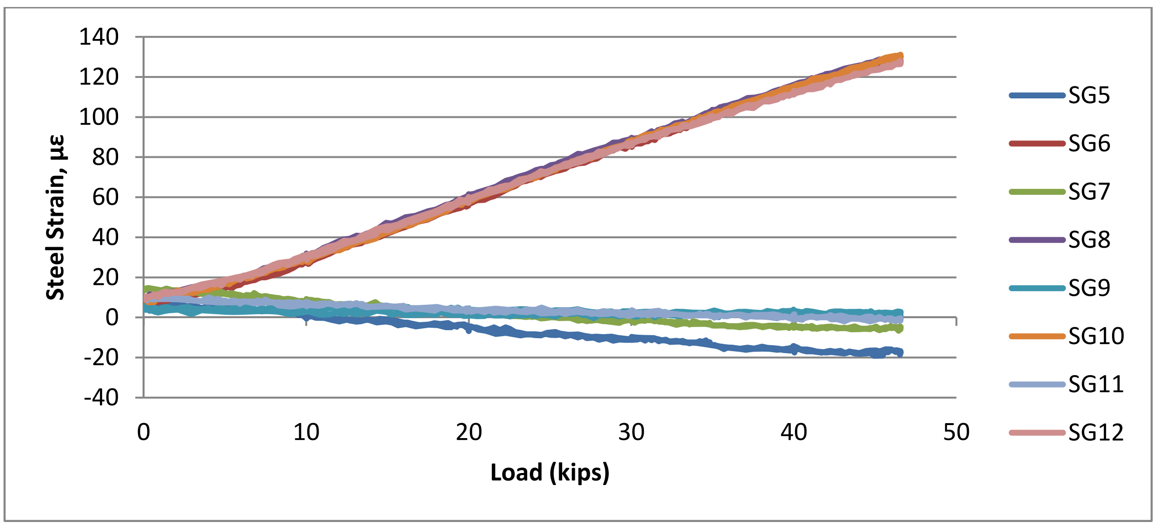


Figure 7-8: Post-Cyclic Static Load Test Transverse Joint HSS Coupler Sidewall Gage Results – Load Case I

Strains recorded in the sidewall remain relatively small, with peaks reaching approximately 120 microstrain. These peak values are far below yield strains of approximately 1600 microstrain. Both tensile and compressive strains are recorded in the sidewalls, which correspond to a successful transfer of local bending effects across the joint detail. While upper sidewall gages show compressive strains, lower sidewall gages show tensile strains.

The research team again noted a minor offset of coupler sidewall gage data in the tensile direction between pre- and post-cyclic loading. This offset may correspond to additional tensile loads travelling through the mechanical coupler detail as the tensile concrete-to-grout interface bond degraded throughout cyclic testing. From the available experimental results, it is difficult to decipher whether this strain shift occurred at a discrete time or progressively over the duration of cyclic testing.

Endwall strains in the transverse HSS couplers also remained relatively consistent between each of the two instrumented couplers. As previously discussed in Chapter 5, gages at these locations were installed on the interior side of the coupler endwall to monitor for unintended endwall bending. Pre- and post-cyclic transverse joint coupler endwall gage results are shown below in Figures 7-9 and 7-10.

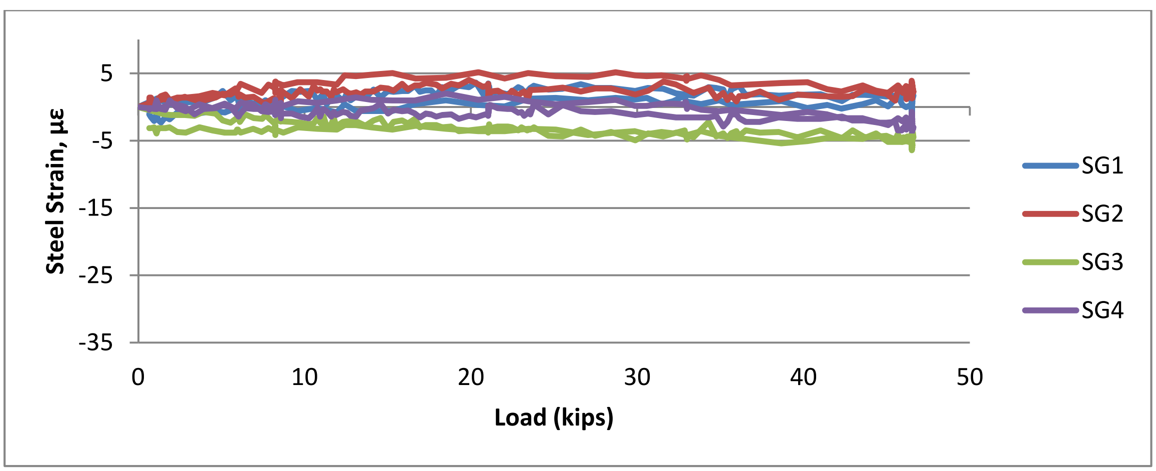


Figure 7-9: Pre-Cyclic Static Transverse Joint HSS Coupler Endwall Gage Results – Load Case I

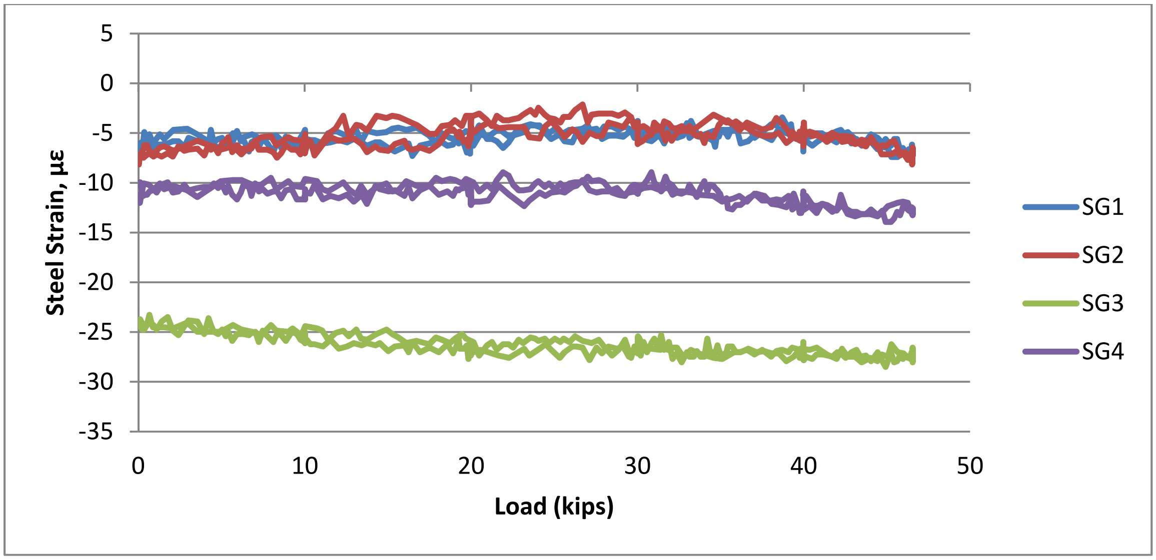


Figure 7-10: Post-Cyclic Static Load Test Transverse Joint HSS Coupler Endwall Gage Results – Load Case I

Compressive strains recorded in this area affirm that only negligible bending of the endwall region occurred during testing. However, strain offsets between pre- and post-cyclic loading suggest minor localized plastic deformations may have occurred in the coupler endwalls either during initial joint seating or as a result of additional load transferred to the HSS coupler throughout testing. The author believes this localized yielding may have occurred at regions of stress concentrations in the HSS coupler, such as at corners, near the endwall installation slot, or in the vicinity of the location where washers transmitted forces from the threaded reinforcing bars to the HSS couplers.

In summary, the transverse joint detail satisfied serviceability requirements for the modified CD-2 type deck system examined in this study. Although the research team did note minor changes in strain levels between pre- and post-cyclic tests, strain levels remained low in all cases. The strain offsets observed in this study may indicate the necessity of a discrete "slip" or seating occurring at mechanical splice locations which fully activates intended resistance mechanisms. This discrete "slip" occurrence may be unique to mechanically-spliced systems, as it is not typically observed in joint details that rely on the development of reinforcing bars to transfer forces such as NCHRP Report 584 System CD-1.

7.2.3 Staged Construction Joint Behavior

The staged construction joint is designed to transmit both shear and moment effects across the longitudinal panel-to-panel joint in the modified CD-2 type system investigated in this study. This joint type is predominately exposed to negative moment and thus, requires a reinforcing steel tension splice near the top of the joint. The performance of this joint type was examined by comparison of joint behavior before, during, and after applied loading cycles. Again, the research team was confident that as long as no serviceability limits were approached, the joint detail performed satisfactorily.

In order to begin analysis of the staged construction joint, it is most logical to examine the surface concrete strain gages that are positioned across this joint detail. Four gages were installed across the grout-to-concrete joints as previously described in Chapter 5. For reference, the pre- and post-cyclic load testing results are shown below in Figures 7-11 and 7-12.

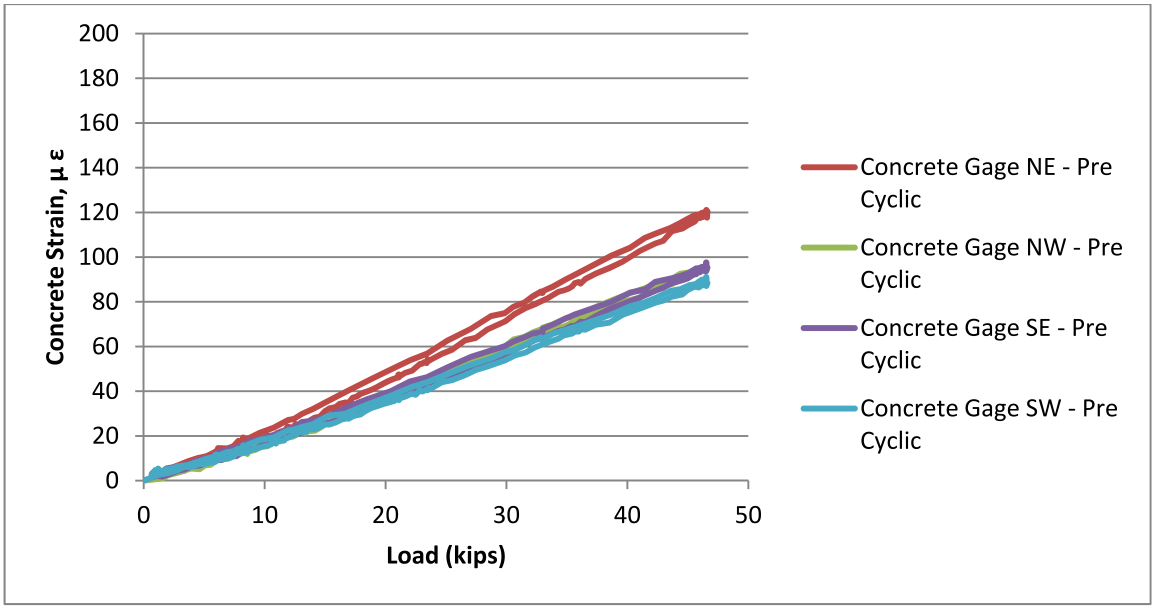


Figure 7-11: Pre-Cyclic Static Load Test Surface Strain Gages across Staged Construction Joint – Load Case I

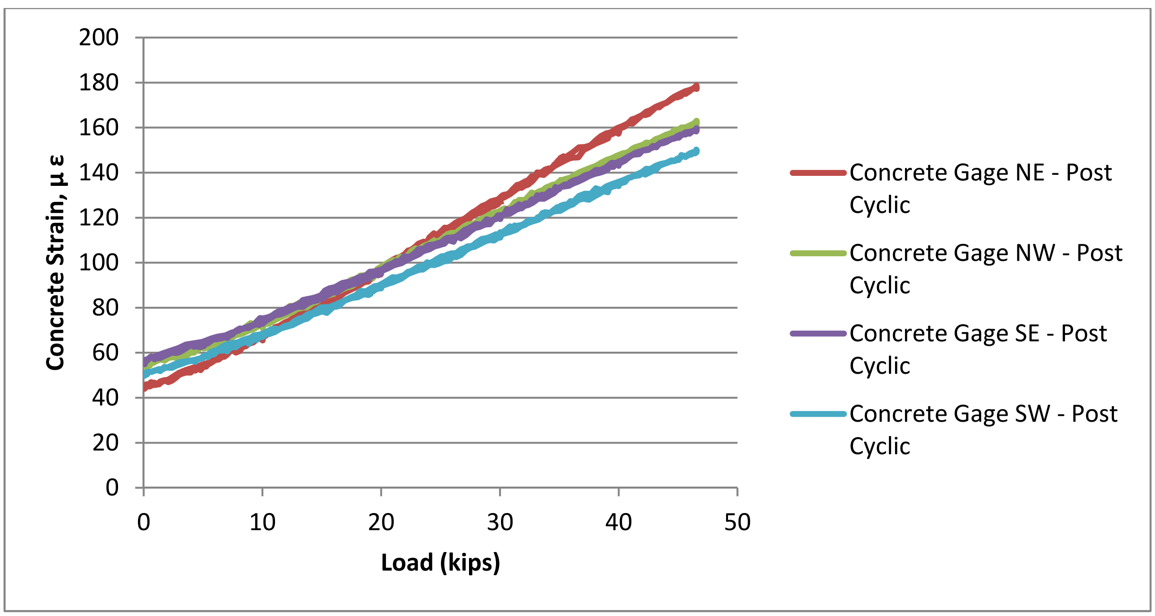


Figure 7-12: Post-Cyclic Static Load Test Surface Strain Gages across Staged Construction Joint – Load Case I

The gages at this location are measuring positive strain values which indicate tension across the top surface of the joint as anticipated at this joint detail. The tensile strains prior to cyclic loading reach peak values of approximately 120 microstrain and appear to exhibit linear elastic material behavior. For post-cyclic results, peak tensile strains again exhibit linear elastic behavior and reach peak values of approximately 180 microstrain. Strains in this range are characteristic of the onset of tensile concrete cracking in bridge decks. However, it is noteworthy that neither of the above figures captures a behavioral transition from uncracked to cracked behavior. The absence of this transition leads the research team to believe that the grout-to-concrete interface at the staged construction joint detail likely acts as an initial crack, prior to any applied loading.

In the gages discussed above, the post-cyclic testing results appear to be shifted upwards approximately 50 microstrain when compared to pre-cyclic results. This may indicate a slight tendency for permanent panel spread to occur across the top of the staged construction joint as a result of cyclic testing. For reference, if the peak tensile strain value of 180 microstrain corresponded to a discrete crack occurrence at each interface location, the width of each crack would be 7×10^{-4} inches. Permissible service-level crack widths for bridge decks are typically regarded as 7×10^{-3} inches or smaller in the tensile face and would correspond to strain readings of greater than 1700 microstrain in this case (ACI Committee 224 2001). As shown below in Figure 7-13, the strain shift appears to occur relatively consistently throughout cyclic testing, but at no time do values approach those corresponding to serviceability issues.

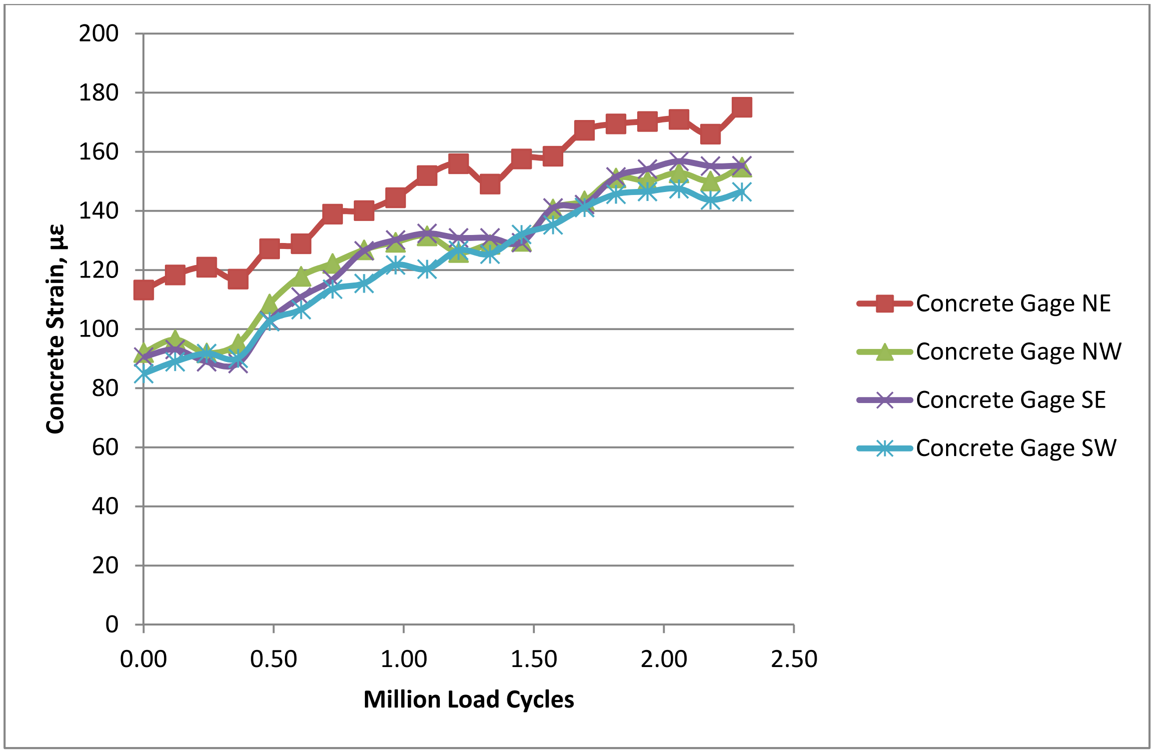


Figure 7-13: Cyclic Staged Construction Strain Gage Results –Load Case I

The research team also examined results from the internal HSS coupler gages and confining stirrups at the staged construction joint detail in order to better understand joint behavior. Similarly to the previous discussion, the coupler gages are grouped into the following two categories: sidewall and endwall gages. Pre- and post cyclic sidewall gage results were similar for each instrumented coupler. Typical sidewall results for a single coupler are shown below in Figures 7-14 and 7-15.

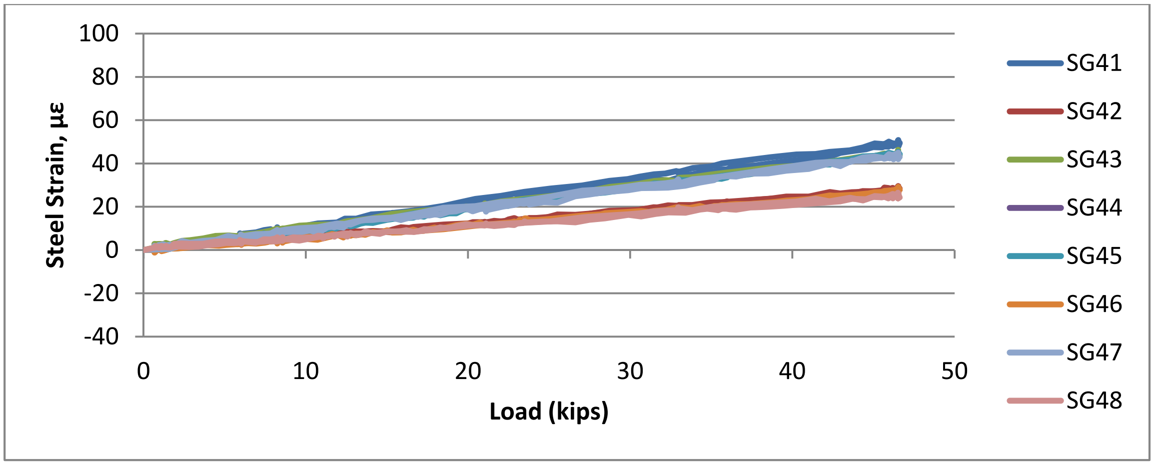


Figure 7-14: Pre-Cyclic Static Load Test Staged Construction Joint HSS Coupler Sidewall Gage Results – Load Case I

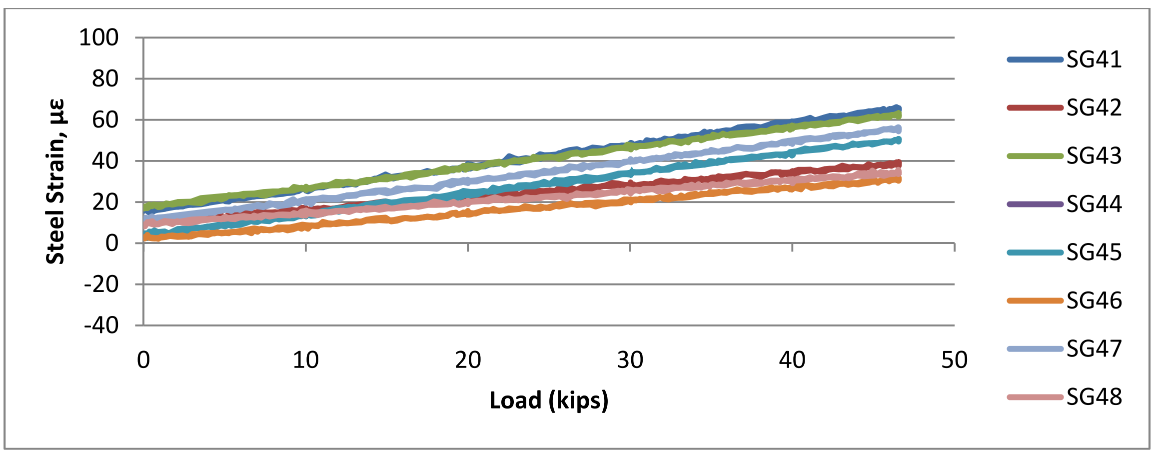


Figure 7-15: Post-Cyclic Static Load Test Staged Construction Joint HSS Coupler Sidewall Gage Results – Load Case I

Strains recorded in the sidewall remained relatively small, with peaks reaching approximately 70 microstrain. These peak values are far below yield strains of approximately 1600 microstrain. Predominately tensile strains are recorded in the coupler sidewalls, which are expected given the proximity of the coupler to the top of the slab. However, it is interesting to note that the sidewall does experience a minor bending component, as exhibited by the two clusters of gage results shown in Figure 7-14. In this case, upper gages tend to experience higher tensile stresses than those gages oriented on the lower half of the coupler sidewall. The gages installed on confining stirrups yielded values very similar to adjacent sidewall gages. The research team did again note a minor offset of coupler sidewall gage data in the tensile direction between pre- and post-cyclic loading. This offset may correspond to a slight degradation in stiffness of the connection type.

Endwall strains in the staged construction joint HSS couplers remained relatively consistent between each of the two instrumented couplers. As previously discussed, these gages were intended to monitor unintended endwall coupler bending during applied loading. Typical pre- and post-cyclic transverse joint coupler endwall gage results are shown below in Figure 7-16 and 7-17.

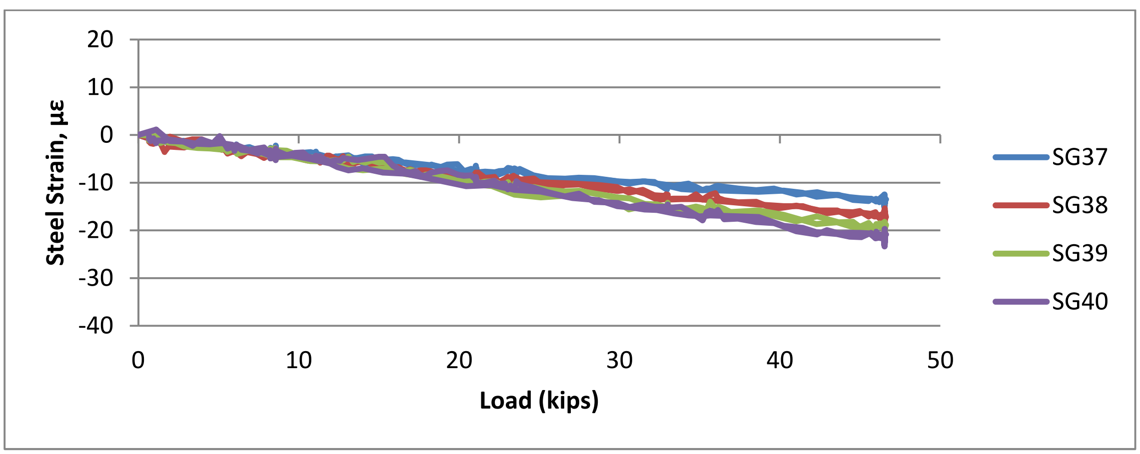


Figure 7-16: Pre-Cyclic Static Load Test Staged Construction Joint HSS Coupler Endwall Gage Results – Load Case I

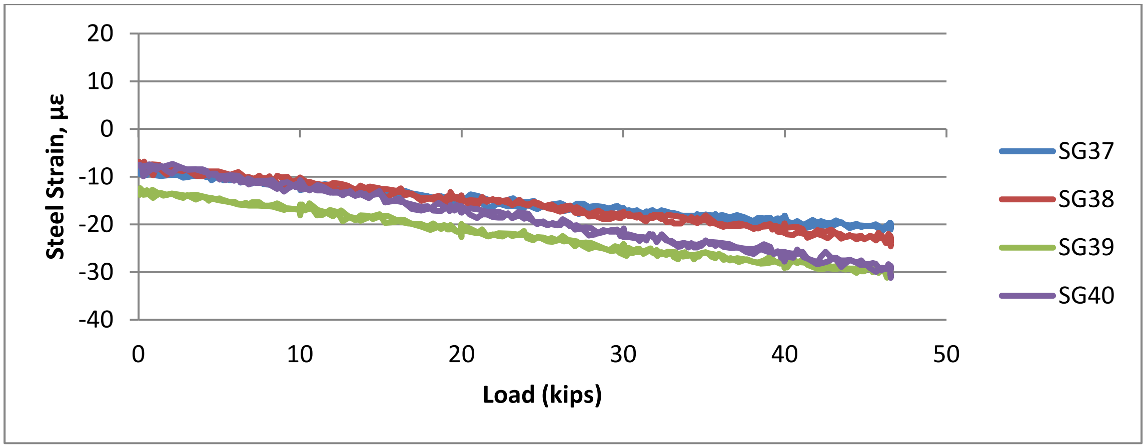


Figure 7-17: Pre-Cyclic Static Load Test Staged Construction Joint HSS Coupler Endwall Gage Results – Load Case I

Compressive strains recorded in this area affirm that only negligible bending of the endwall region occurred during testing. However, strain offsets between pre- and post-cyclic loading suggest minor localized plastic deformations may have occurred in the coupler endwalls either during initial joint seating or throughout cyclic testing. The authors believe this localized yielding may have occurred at regions of stress concentrations in the HSS coupler, such as at corners, near endwall installation slots, or in the vicinity of the location where washers transmitted forces from the threaded reinforcing bars to the HSS couplers.

In summary, the staged construction joint detail included in this investigation satisfied serviceability requirements for the modified CD-2 type deck system. Although the research team did note minor changes in strain levels between pre- and post-cyclic tests, strain levels remained low in all cases. In similar fashion to the previously-examined transverse joint, the research team hypothesizes that a portion of the strain offsets observed in this test may be due to the necessity of a discrete "slip" to engage mechanical reinforcement splices.

7.2.4 Longitudinal Joint Behavior

As previously described, longitudinal joints serve as deck-to-girder joints and are typically oriented in the direction of traffic occurring along the top of girders. In the modified CD-2 type system examined in this study, the longitudinal joint comprised of a continuous pocket in order to accommodate existing girder top flange shear connectors. With regards to this joint type, the primary intent of this investigation was to validate the constructability of continuous shear pocket type joints, while also examining the joint detail for durability under applied service loadings. It is important to note that the investigation of continuous shear pocket details included in this study is limited to their influence and effects on concrete deck panel systems only. No specific provisions were included in this study in order to evaluate the capability of this joint type to achieve composite action with supporting superstructure.

This study successfully demonstrated the fabrication, transportation, and erection of a deck system utilizing seemingly-fragile continuous shear pocket details at the longitudinal joint locations. As documented in Chapter 4, the research team did not observe cracking or other deficiencies as a result of handling, fabrication, or erection of the panel system. The successful construction the deck system included in this study appears to be the first documented implementation of a continuous shear pocket system in engineering literature. The research team hopes the successful demonstration of a continuous shear pocket system will help to reduce skepticism towards such systems throughout the engineering community and will also inspire expanded future research interests in this area.

The research team carefully monitored the longitudinal joint detail for cracking and other signs of deterioration throughout Load Case I service load testing. Two main areas closely monitored during testing were the top of the concrete slab at this joint detail, as well as the edge of the continuous pocket location visible from the side of the bridge specimen. The research team was also especially careful to monitor locations near previous grout injection and vent ports for any cracking caused along these locations. No signs of cracking in the vicinity of the longitudinal joints were observed at any point during Load Case I testing.

In summary, the longitudinal joint detail performed satisfactorily during Load Case I static and cyclic loading. No cracking was detected at any location associated with the continuous shear pocket joint under these loading situations.

7.3 Load Case II

As discussed in Chapter 5, Load Case II served primarily as a validation test and received limited instrumentation during load testing. The axle load location chosen for Load Case II corresponds to the condition that induces the maximum negative moment across the longitudinal joint. In addition, because each tire contact area is positioned at the edge of a single panel, the transverse joint is subject to maximum shear by the applied loading.

7.3.1 Global System Behavior

In order to examine global behavior of the deck system during Load Case II, two under-slab laser displacement gages were utilized. As previously discussed, these gages were located on the east and west side of loading frame. Using similar procedures as those described previously, live-load slab deflections were predicted prior to testing and are shown below in Figure 7-18. Actual experimental pre-cyclic static test results are then shown in Figure 7-19 for comparison.

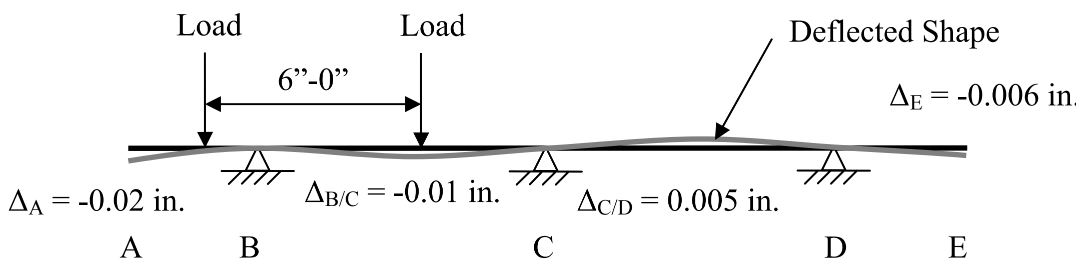


Figure 7-18: Load Case II Anticipated Live Load Deflections

As shown in Figure 7-19, peak displacement values on the cantilevered edge, corresponding to the west gage, reached peak values of approximately 0.02 inches and showed very good agreement with predicted results. Similarly, the gage located between control lines B and C, corresponding to the west gage, showed very good agreement with the predicted peak value of 0.01 inches.

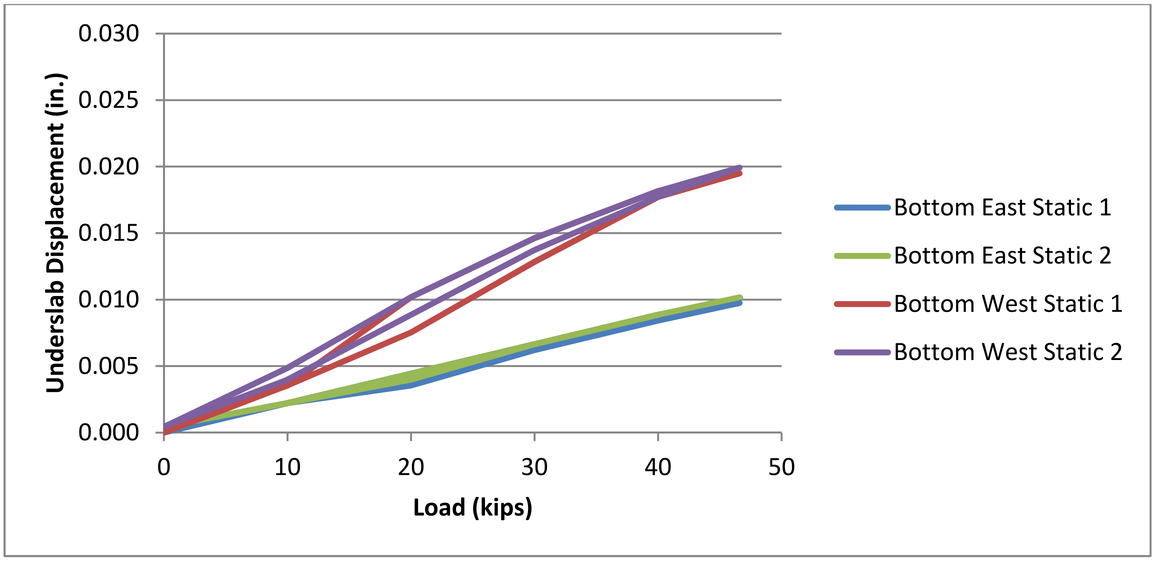


Figure 7-19: Pre-Cyclic Under-Slab Displacement Gage Results – Load Case II

Next, it is important to compare the pre-cyclic static load test data to the post-cyclic static load test data. Figure 7-20 shows the post-cyclic displacement gage response of the deck system.

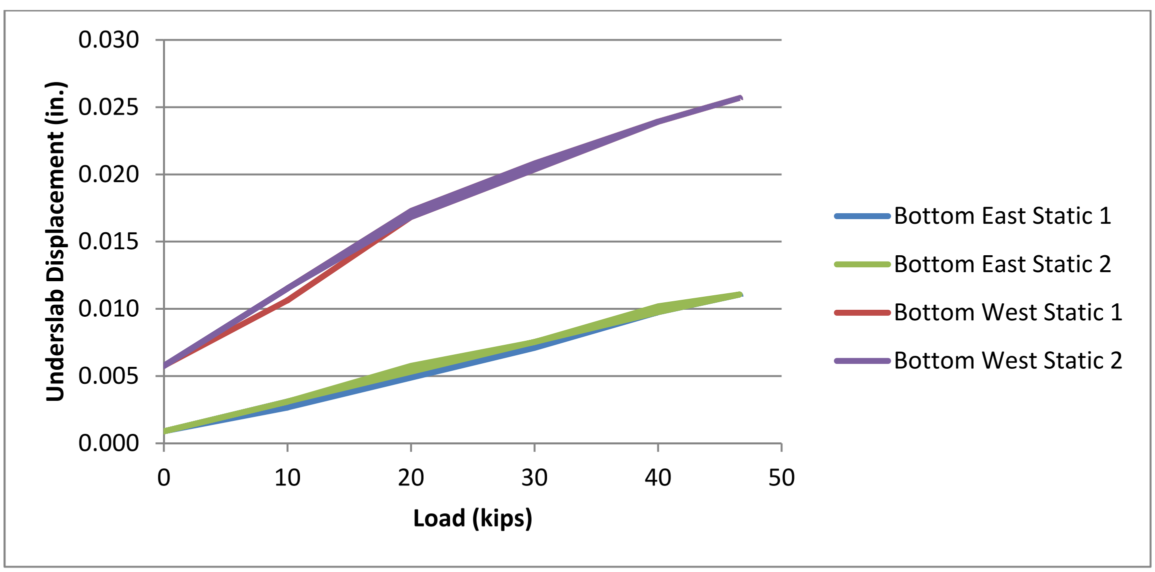


Figure 7-20: Post-Cyclic Under-Slab Displacement Gage Results – Load Case II

As shown, the pre- and post-cyclic values of the east side gage are similar and suggest consistent linear response of the slab system in this area without degradation of stiffness during cyclic loading. However, the west side gage shows additional post-cyclic deflections at the cantilever edge. Although the additional deflection is small, it may suggest a degradation of global stiffness a result of the applied cyclic loading. A possible cause for this stiffness degradation may be the presence of localized cracking at various joint details as reported by the research team during Load Case II cyclic loading. This possibility is explored more closely in the following sections examining local joint behavior.

In summary, the global behavior of the CD-2 type system included in this study exhibited global system behavior similar to that predicted by specimen design. Maximum deflections which occurred at the cantilever edge reached peak values of approximately 0.025 inches, roughly half of the allowable AASHTO deflection limit of L/800. Although a slight degradation in stiffness was observed as a result of cyclic testing, no serviceability limits for the deck system were exceeded.

7.3.2 Transverse Joint Behavior

Behavior of the transverse joint was monitored by laser displacement gages that spanned across the transverse joint on each side of the loading frame. Static load tests were performed both before and after the Load Case II cyclic loading. Test data from these static load tests is shown below in Table 7-1. Panel spread values are presented relative to original panel position prior to applied loading.

Table 7-1: Load Case II Static Load Panel Spread Results

When Tested	Load Level	East Side* (*in.*)	West Side* (*in.*)
Pre-Cyclic	Zero Load	0.000	0.000
	Peak Load	-0.001	-0.002
Post-Cyclic	Zero Load	-0.004	+0.007
	Peak Load	-0.005	+0.005

* + Indicates spreading
- Indicates closing

The pre-cyclic load test data shows a squeezing tendency of the panels at the top of the transverse joint under peak loading. It is possible that this reduced joint width observed under loading may actually be due to the laser displacement sensors having sensitivities large enough to record the effects of the local compression induced at the top of the slab near applied loads. Recall the concrete strain gages located similarly during Load Case I also documented compressive strains near load application points. No cracking was observed at this joint detail during pre-cyclic static testing.

The unloaded post-cyclic static load test data shows slightly different values than the unloaded pre-cyclic load test. On the east side, stiffness of the system seems similar to the pre-cyclic loading, with a change of -0.001 inches under full peak load. Similarly, the west side gages indicate pre- and post-cycle behavior was consistent under loading, with a change of -.002 inches documented under peak load. However, it is interesting to note that a shift in the west gage peak values was recorded during testing. The west side of the specimen corresponds to the cantilever location where cracking was observed during Load Case II as documented in Chapter 6. The research team feels that this cracking may have been responsible for the slight increase in panel spread throughout cyclic testing observed on the cantilever side, as well as a portion of the global stiffness degradation exhibited during cyclic Load Case II testing.

Minor cracking observed at the top of the transverse joint was barely detectable and well below permissible crack widths. However, cracking at the exterior-most joint coupler location on the deck cantilever approached permissible crack widths. As previously discussed in Chapter 5, the research team chose to orient the Load Case II load footprint further towards the cantilevered bridge edge than permitted by AASHTO specifications in order to induce certain desired peak moment effects across the longitudinal joint. As a result of this shifted loading, the cantilever experienced higher peak deflections than it would have with the truck footprint in a permitted location. The research team believes this may have been the cause of localized cracking at the exterior coupler location.

Despite the above explanation for the localized cracking, the research team feels degradation in this area might be remedied by an adjustment to the transverse joint detailing in future CD-2 type deck systems. By locating this coupler directly on the edge of the panel as shown below in Figure 7-21, no adjacent deck panel concrete is present to confine the coupler on the exterior side. As such, this detail may focus undesirable cracking on the exterior of the panel, exposing it to potential future degradation by environmental influences. The research team suggests future work in this area to consider relocating this exterior splice location inward to minimize exposed cracking at this detail.

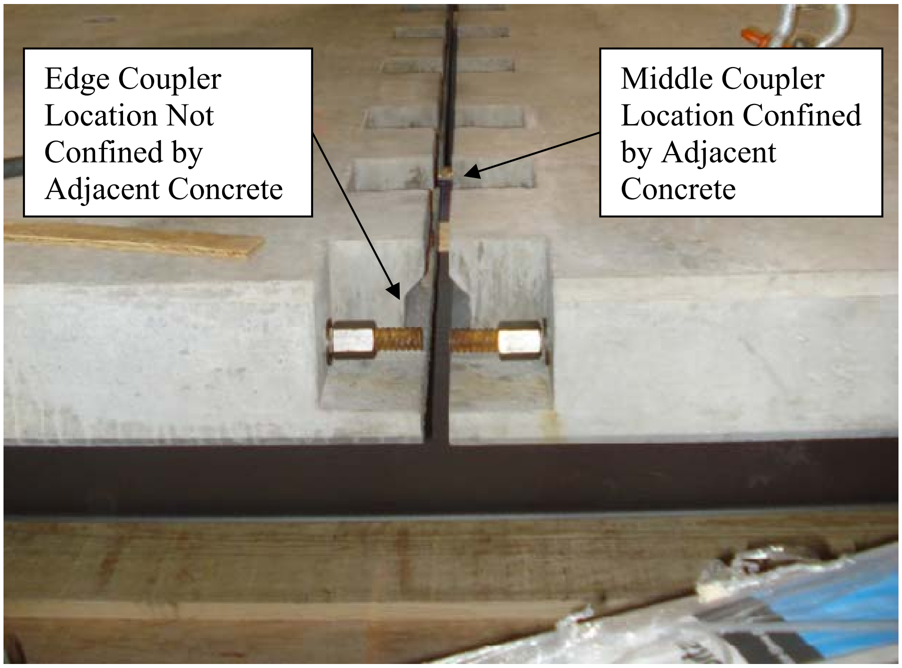


Figure 7-21: Transverse Joint Coupler Locations Before Grout Placement

In summary, the transverse joint detail satisfied serviceability requirements for the modified CD-2 type deck system examined in the study. Despite local cracking observed in the exterior coupler location which may have caused slight degradation of global stiffness, deflections remained well below AASHTO limits. In addition, all crack widths observed during Load Case II testing remained below accepted serviceability crack limits. The research team suggests a minor relocation of the edge coupler in the transverse joint to potentially improve joint durability in future implementations of similar deck systems.

7.3.3 Staged Construction Joint Behavior

Under Load Case II loadings, the staged construction joint detail was not exposed to force effects greater than those induced by Load Case I. The research team chose to visually monitor this joint detail for cracking intermittently throughout testing. No additional cracking or changes to existing cracks were observed at this joint detail throughout the duration of Case II loadings.

7.3.4 Longitudinal Joint Behavior

As previously described, the longitudinal joint serves as a deck-to-girder joint and is oriented in the direction of traffic along the top of the superstructure girders. The primary intent of this investigation with regards to this joint type detail was to both validate the detail's constructability, as well as to investigate the joint durability in regards to a CD-2 type precast concrete deck system.

The continuous pocket longitudinal joint detail performed satisfactorily with regard to the concrete deck system examined in this study. Although minor cracking was documented at the interface between the grouted pocket and the concrete panel when viewed from the end of the specimen, this cracking was limited and did not seem to affect global behavior of the specimen. Future study in this area is recommended to fully understand the behavior of the continuous pocket longitudinal joint detail, especially when superimposing global effects of composite action with supporting superstructure components.

Chapter 8: Summary, Conclusions, and Recommendations

8.1 Summary

The Alabama Department of Transportation (ALDOT) currently has over three miles of major interstate bridges near downtown Birmingham involving approximately 600,000 square feet of deck area with significant levels of deterioration. ALDOT has expressed the need for replacement of these deteriorated bridge decks, but seeks to minimize the potentially tremendous impact of these replacement projects on the end user. For rapid bridge deck replacement projects, it is often beneficial to utilize full-depth precast deck panel replacement systems to reduce project durations. However, many current deck panel replacement systems employ details which are inherently time-consuming. By exploring and developing a replacement deck panel system utilizing innovative concepts, geometries, and details, it is possible to reduce construction durations on rapid bridge deck replacement projects.

In this thesis, a replacement bridge deck panel system utilizing non-prestressed full-depth precast bridge deck panels with continuous shear pockets was investigated. Expanding on previous work by others, the research team performed fabrication and erection studies on a specific precast concrete bridge deck panel system. In addition, service-level load testing on a full-scale deck panel system was performed to evaluate in-service performance.

Based on the research presented in this thesis, it was found that the modified CD-2 deck panel system performed satisfactorily at the service-level loadings utilized in this investigation. By demonstrating the preliminary success of such a system under the constraints of this investigation, the research team is optimistic that continued research in this area will further validate and encourage the implementation of the innovative time-saving details explored in this study.

8.2 Conclusions

Based on the work performed in this investigation, the following conclusions regarding the performance and implementation of the modified CD-2 type precast deck panel system were reached:

- The innovative details of the modified CD-2 type deck panel system were such that they were able to be fabricated, transported, and erected without evidencing premature cracking or other significant durability concerns prior to the introduction of service-level loadings.
- Global behavior of the modified CD-2 type deck panel system under service-level loadings was relatively consistent with predicted values and performed satisfactorily with regards to applicable AASHTO serviceability limits.
- The transverse joint detail included in this investigation performed satisfactorily with regards to serviceability requirements under service loadings. Although minor global stiffness degradation was observed during cyclic testing, strain levels measured across this joint detail remained low in all cases.

- The staged construction joint detail included in this investigation satisfied serviceability requirements under service-level loadings. Although the research team did note minor changes in strain levels during cyclic testing, strain levels remained low in all cases.
- The longitudinal girder-to-deck panel joint detail included in this investigation performed satisfactorily under service-level loadings. Any cracking in these areas was minor and well-controlled. The research team believes the successful implementation of the "continuous shear pocket" detail included in this study is the first of its kind.

8.3 Recommendations

The research team recommends the following areas for continued research efforts:

- Perform strength-level static testing of the modified CD-2 type deck panel system to verify ultimate strength requirements are satisfied.
- Perform a more comprehensive test program that includes provisions to account for global superstructure behavioral effects on the modified CD-2 type deck panel system. This program should further develop and more closely explore the "continuous shear pocket" detail to verify its adequacy in developing full composite action between the bridge deck and superstructure girders.
- Perform an evaluation of various grout products and mixtures with regards to their applicability to rapid bridge deck replacement using the modified CD-2 or other similar deck replacement systems.

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Appendix A: Selected Design Calculations

Selected Design Calculations:
Modifed CD-2 Precast Bridge Deck Panel System:

These calculations are based on AASHTO LRFD Bridge Design Specifications, 5th Edition.

1. Deck Panel Reinforcing Steel Design

1a. Typical Interior

[AASHTO 9.7.2.3] System geometry satisfies requirements necessary to apply Empirical Design Method.

[AASHTO 9.7.2.5] Top Reinforcing Steel

$$A_{topr} := 0.18 \quad \frac{in^2}{ft}$$

Use #5 @ 18" each direction

$$A_{tops} := \left(\frac{5}{9}\right)^2 \cdot \frac{12}{18} = 0.206 \quad \frac{in^2}{ft}$$

Use #5 @ 18" top steel each direction

[AASHTO 9.7.2.5] Bottom Reinforcing Steel

$$A_{bottomr} := 0.27 \quad \frac{in^2}{ft}$$

Use #6 @ 18" each direction

$$A_{bottoms} := \left(\frac{6}{9}\right)^2 \cdot \frac{12}{18} = 0.30 \quad \frac{in^2}{ft}$$

Use #6 @ 18" bottom steel each direction

Selection of Reinforcing Sizes for CD-2 Precast System

Note: Threaded reinforcing bars available in #6 and larger only.

Combine distribution steel (longitudinal steel) top and bottom layer to single layer per NCHRP Report 584 recommendations.

$$A_{longr} := A_{tops} + A_{bottoms} = 0.502 \quad \frac{in^2}{ft}$$

$$A_{longs} := \left(\frac{8}{9}\right)^2 \cdot \frac{12}{18} = 0.527 \quad \frac{in^2}{ft}$$

Use single #8 @ 18" spacing for longitudinal bar @ slab mid-height.

Use #6 bars @ 18" spacing top and bottom for transverse bars

Note: The above bar spacings will be adjusted during system detailing to accomodate equal spacing along panel lengths.

1b. Overhang

Critical design section is at external girder under Extreme Event II Limit State.

Dead Load:

Barrier:

$A_{barrier} := 293$ in^2

$width_{barrier} := 15$ in

$height_{barrier} := 32$ in

Reference: ALDOT Standard Drawings and Design Detail Manual.

$$P_{barrier} := \frac{A_{barrier}}{144} \cdot 0.150 = 0.305 \qquad \frac{kips}{ft}$$

[AASHTO 4.6.3.2.6] Take design section for negative moment at 1/4 flange width from center of support.

Top Flange Width: $btf := 14$ in

Overhang Length: $o := 3.5$ ft

For one foot width strip:

Apply barrier load at 3" from cantilever edge:

$$M_{barrier} := -P_{barrier} \cdot \left(o - \frac{3}{12} - \frac{\frac{btf}{12}}{4} \right) = -0.903 \qquad \frac{kip \cdot ft}{ft}$$

Slab:

$$w_{slab} := \frac{8}{12} \cdot 0.150 = 0.1 \qquad \frac{kips}{ft^2}$$

$$M_{slab} := -w_{slab} \cdot \left(o - \frac{\frac{btf}{12}}{4} \right) \cdot \left(\frac{o - \frac{\frac{btf}{12}}{4}}{2} \right) = -0.515 \qquad \frac{kip \cdot ft}{ft}$$

Live Load:

[AASHTO 3.6.1.3.4] Assure geometric provisions are met for live load reduction.

$$M_{liveload} := -1.0 \cdot \left(o - \frac{width_{barrier}}{12} - 1 - \frac{\frac{btf}{12}}{4} \right) \cdot 1.33 \cdot 1.2 = -1.53 \qquad \frac{kip \cdot ft}{ft}$$

Barrier Rail Collision Loads:

For typical ALDOT barrier rail, collision loadings are as shown below. (Reference: Design and Construction Planning of Rapid Bridge Deck Replacement Systems for I-59 Bridges in Collinsville, AL by Bryan

Edward Harvey 2011 .)

$$M_{collision} := -16.73 \quad \frac{kip \cdot ft}{ft}$$

Note: Detailed calculations regarding force transfer from barrier rail to the deck slab are omitted from this design exercise due to the absence of a barrier rail on the laboratory specimen. However, the research team deemed it important to size overhang steel to realistic levels for potential ALDOT barrier loadings.

Factored Extreme Event II Loads:

[AASHTO 13.4.1] $\gamma := 1.0$ $\quad \eta := 1.05$

$$M_{EEII} := \eta \cdot \left[\gamma \cdot \left(M_{slab} + M_{barrier}\right) + 0.5 \cdot M_{liveload} + 1.0 \cdot M_{collision}\right]$$

$$M_{EEII} = -19.858 \quad \frac{kip \cdot ft}{ft}$$

Overhang Top Steel Design:

Use 3 #6 @ 18" spacing with 2" top cover

$F_y := 60$ ksi

$f_c := 4$ ksi

$$A_s := \left(\frac{6}{9}\right)^2 \cdot 3 \cdot \frac{12}{18} = 0.889 \quad \frac{in^2}{ft}$$

$$a := \frac{F_y \cdot A_s}{0.85 \cdot f_c \cdot 12} = 1.307 \quad in$$

$$\phi M_n := -F_y \cdot A_s \cdot 1.0 \cdot \frac{\left(8 - 2 - \frac{a}{2}\right)}{12} = -23.762 \quad \frac{kip \cdot ft}{ft} \qquad \text{OK}$$

Verify Flexural Requirements of Strength I Limit Case are met.

[AASHTO 5.7.3.3.2] Factored strength I flexural resistance equal to lesser of 1.2 cracking moment or 1.33 factored strength I moment demand.

[AASHTO 3.4.1.2] $\gamma := 1.25$ For permenant loads

Find 1.33 M Strength I

$$M_{StI} := \eta \cdot \left[\gamma \cdot \left(M_{slab} + M_{barrier}\right) + 1.75 \cdot M_{liveload}\right]$$

$$M_{StI} = -4.671 \quad \frac{kip \cdot ft}{ft}$$

$1.33 \cdot M_{StI} = -6.212 \quad \frac{kip \cdot ft}{ft}$

Find 1.2 M crack

[AASHTO 5.4.2.6] $\quad f_r := 0.37 \cdot \sqrt{\frac{4000}{1000}} = 0.74 \quad$ ksi

$I := \frac{12 \cdot 8^3}{12} = 512 \quad in^4$

$$M_{cr} := \frac{\frac{-f_r \cdot I}{\frac{8}{2}}}{12} = -7.893 \quad \frac{kip \cdot ft}{ft}$$

$1.2 \cdot M_{cr} = -9.472 \quad \frac{kip \cdot ft}{ft}$

$\max(1.2 \cdot M_{cr}, 1.33 \cdot M_{StI}) = -6.212 \quad \frac{kip \cdot ft}{ft}$

$\phi M_n = -23.762 \quad$ OK

2. Connection Design

For the purposes of these selected calculations, HSS coupler final sizing will be verified against AASHTO mechanical splice demands. In reality, constructability and practical considerations govern coupler sizing.

2a. Transverse Panel to Panel Joint

[AASHTO 5.11.5.2.2] Mechanical connections between reinforcing steel bars must be designed to resist 125% of the tension yield load.

Check coupler sidewall tension

$$A_s := \left(\frac{8}{9}\right)^2 = 0.79 \qquad in^2$$

$$T_{req} := A_s \cdot 60 \cdot 1.25 = 59.3 \qquad kips$$

$$coupler_{height} := 3 + \frac{3}{4} \qquad inches$$

$$coupler_{thickness} := \frac{3}{16} \qquad inch$$

$$F_{ycoupler} := 46 \qquad ksi \qquad \text{Carbon Steel A500 Grade B}$$

$$T_s := coupler_{thickness} \cdot coupler_{height} \cdot 2 \cdot F_{ycoupler} = 64.7 \qquad kips \qquad OK$$

Check coupler endwall in bearing

$$d_{hole} := \frac{5}{8} \cdot 2 = 1.25 \qquad in \qquad \text{Hole in HSS coupler end}$$

$$d_o := 2.25 \qquad in \qquad \text{Outer diameter of washer}$$

$$d_i := 1.25 \qquad in \qquad \text{Inner diameter of washer}$$

Accounting for installation slot (Assume lose 1/3 contact area):

$$A_{bearing} := \left(\frac{\pi}{4} \cdot d_o^{\,2} - \frac{\pi}{4} \cdot d_{hole}^{\,2}\right) \cdot \frac{2}{3} = 1.833 \qquad in^2$$

$$T_s := A_{bearing} \cdot F_{ycoupler} = 84.3 \qquad kips \qquad OK$$

2b. Staged Construction Joint

[AASHTO 5.11.5.2.2] Mechanical connections between reinforcing steel bars must be designed to resist 125% of the tension yield load.

Check coupler sidewall tension

$A_s := \left(\frac{6}{9}\right)^2 = 0.444$ in^2

$T_{req} := A_s \cdot 60 \cdot 1.25 = 33.3$ kips

$coupler_{height} := 2 + \frac{7}{16}$ inches

$coupler_{thickness} := \frac{3}{16}$ inch

$F_{ycoupler} := 46$ ksi Carbon Steel A500 Grade B

$T_s := coupler_{thickness} \cdot coupler_{height} \cdot 2 \cdot F_{ycoupler} = 42$ kips OK

Check coupler endwall in bearing

$d_{hole} := 1 = 1$ in Hole in HSS coupler end

$d_o := 1.75$ in Outer diameter of washer

$d_i := \frac{13}{16}$ in Inner diameter of washer

Accounting for installation slot (Assume lose 1/3 contact area):

$A_{bearing} := \left(\frac{\pi}{4} \cdot d_o^{\,2} - \frac{\pi}{4} \cdot d_{hole}^{\,2}\right) \cdot \frac{2}{3} = 1.08$ in^2

$T_s := A_{bearing} \cdot F_{ycoupler} = 49.7$ kips OK

Appendix B: Construction Drawings

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S3. Panel 1 Plan and Reinforcing Schedule

S3A. Panel 1 Elevations and Sections

S4. Panel 2 Plan and Reinforcing Schedule

S5. Panel 3 Plan and Reinforcing Schedule

S6. Panel 4 Plan and Reinforcing Schedule

S7. Transverse Joint Details

S8. Longitudinal Joint Details

S9. Structural Steel

S10. Lab Location Plan

S11. Proposed Rigging Plan

S12. Loading/Instrumentation Plan

S13. Load Frame Details

S14. Load Frame Steel Fabrication Details

RIG. Rigging Beam Details

AUBURN UNIVERSITY - CIVIL ENGINEERING DEPARTMENT

FULL SCALE IMPLEMENTATION AND TESTING OF NON-PRESTRESSED FULL-DEPTH PRECAST BRIDGE DECK PANELS WITH CONTINUOUS SHEAR POCKETS

BY: DAVE MANTE, DR. HASSAN ABBAS, DR. GEORGE RAMEY

PANEL FABRICATION, HANDLING, AND STORAGE:

1. FABRICATOR IS RESPONSIBLE FOR FABRICATION, HANDLING, AND STORAGE OF PANELS IN SUCH A MANNER THAT DOES NOT CAUSE UNDUE STRESS OR CRACKING IN THE PANEL.

2. AU WILL INSPECT ALL PANELS AND REJECT ANY DEFECTIVE PANELS. REPLACE ANY REJECTED PANELS AT THE FABRICATOR'S EXPENSE.

3. THE FOLLOWING CRITERIA WILL BE CAUSE FOR REJECTION:
   - BROKEN CORNERS THAT CANNOT BE PROPERLY REPAIRED
   - FULL DEPTH CRACKING IN PANEL
   - SIGNIFICANT DIMENSIONAL DEFORMITIES
   - PANELS THAT ARE FABRICATED OUTSIDE OF SPECIFIED TOLERANCES.
   - BROKEN, DAMAGED, OR DEFORMED THREADED BAR PROTRUDING FROM PANEL.

4. TOLERANCE OF CASTING PANELS AND REINFORCING PLACEMENT IS CRITICAL TO SUCCESSFUL INSTALLATION. WHEREVER POSSIBLE, MEASUREMENTS FOR REINFORCING PLACEMENT ARE GIVEN FROM A CONTROL LINE TO AVOID BUILD-UP OF TOLERANCE ERRORS TYPICAL IN CENTER-TO-CENTER MEASUREMENTS.

5. EVERY EFFORT HAS BEEN MADE TO ASSURE THAT INDIVIDUAL PANEL WEIGHT WILL NOT EXCEED OR APPROACH 8 TONS. PLEASE CONFIRM DURING FABRICATION.

6. DURING STORAGE, PANELS SHOULD BE PROTECTED FROM ENVIRONMENTAL EFFECTS, SUCH AS RAIN AND SNOW TO PROTECT ANY EXPOSED STEEL COMPONENTS FROM RUSTING.

7. DURING LIFTING AND HANDLING, PANELS SHOULD BE HANDLED ACCORDING TO APPROVED RIGGING PLAN DUE TO FRAGILE NATURE OF PANELS.

8. PC TO SUBMIT PANEL TRANSPORTATION PLAN TO AU INDICATING TRAILER SIZE, SHIPPING ORIENTATION OF PANELS, AND MEANS TO POSITION/RESTRAIN PANELS.

9. WASHER AND NUT SHALL BE INSTALLED BY PC PRIOR TO SHIPPING ON ALL EXPOSED THREADED REINFORCING STEEL. WASHER AND NUT ASSEMBLY SHOULD BE THREADED TO APPROXIMATELY HALFWAY POINT OF EXPOSED THREADS.

10. ACCEPTABLE FABRICATION TOLERANCE:
    1/16" – REBAR PLACEMENT
    1/8" – CONCRETE GEOMETRY
    1/8" – LOAD FRAME PENETRATIONS

11. WHERE LOAD FRAME PENETRATIONS (3" DIA. HOLES) INTERFERE WITH REINFORCING, BEND REINFORCING TO ACCOMODATE AT INTERIOR OF PANEL. HOWEVER, IT IS CRITICAL TO MAINTAIN REINFORCING LOCATIONS PROTRUDING FROM PANEL.

STRUCTURAL STEEL NOTES:

1. SS SHALL BURN LIFTING HOLES AS REQUIRED TO SAFETY HANDLE AND RIG BEAMS.

2. FABRICATED HSS WILL BE SHIPPED AND DELIVERED AT A LATER TIME THAN STEEL GIRDER DELIVERY. INCLUDE 5 EXTRA OF EACH TYPE.

3. HSS SIZING IS CRITICAL TO CONNECTION DETAIL. PLEASE CONFIRM DIMENSIONS MATCH (1/8" +/–) THOSE INCLUDED IN CONNECTION DETAILS ON DWG. S7 AND S8.

4. SS TO FURNISH AND DELIVER 30 STEEL SHIM BARS AS SHOWN ON DWG. 7/S9.

5. SS TO FABRICATE AND DELIVER COMPONENTS OF LOADING FRAME AS SHOWN ON DWG. S14.

GENERAL NOTES:

1. PRECAST CONCRETE PANELS DESIGNED IN ACCORDANCE WITH AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, 5TH EDITION WHERE POSSIBLE.

2. PANELS DESIGNED FOR HL–93 LOADING AND LEVEL TL–4 CRASH BARRIER FORCES.

3. PRECAST PANEL CONCRETE: f'c = 4000 PSI MIN (WET CURED)

   REINFORCING STEEL –THREADED fy = 75 KSI
   (PROPRIETARY SPECIFICATION – USE WILLIAMS FORM THREADED BAR, WASHER, AND NUT ASSEMBLIES – 770–949–8300)

   REINFORCING STEEL – NON–THREADED fy = 60 KSI

   NON–SHRINK GROUT f'c = 6000 PSI @ 1–DAY
   (BASF SS MORTAR OR EQUIVALENT INTENDED FOR METALLIC APPLICATION.)

   STRUCTURAL STEEL GIRDER: fy = 50–65 KSI
   A992 HIGH STRENGTH LOW ALLOY

   STEEL FABRICATION (HSS) fy = 46 KSI
   A500 GRADE B

4. EVERY EFFORT HAS BEEN MADE TO SATISFY ALDOT BRIDGE BUREAU STRUCTURES DESIGN AND DETAIL MANUAL DATED 1/1/2008. DUE TO THE NATURE OF INNOVATIVE DECK SYSTEMS, CERTAIN DETAILS CANNOT MEET REQUIREMENTS OF THIS DOCUMENT.

5. WELD ACCORDING TO TO AASHTO/AWS D1.5 BRIDGE WELDING CODE.

6. CHAMFER ALL EXPOSES CORNERS 3/4". PRECAST PANELS ADJACENT TO CLOSURE POURS OR OTHER PANELS ARE NOT CONSIDERED EXPOSED CORNERS.

7. AS INDICATED ON DOCUMENTS, DRIP EDGE SHALL BE ADJACENT 3/4" CHAMFERS.

8. TOP OF PANEL TO RECEIVE SMOOTH FINISH TO ALLOW FOR INSTRUMENTATION.

9. STANDARD CONCRETE COVER (CLEAR) UNLESS INDICATED OTHERWISE
   TOP: 2"
   EXPOSED SIDE (IN FINAL CONDITION): 2"
   BOTTOM: 1"

10. CONCRETE HAUNCH IS 1" IN ALL AREAS AFTER FINAL POSITIONING OF PANELS. SPECIMEN WILL BE LEVEL AND FLAT.

11. KEY FOR RESPONSIBLE PARTIES:

   AU: AUBURN UNIVERSITY
   PC: PRECAST PANEL FABRICATOR
   SS: STRUCTURAL STEEL FABRICATOR/SUPPLIER

12. AT INTENDED JOINT LOCATIONS, PANEL EDGES TO BE ROUGHENED TO EXPOSE AGGREGATE, FOLLOWED BY THOROUGH WASHING. THIS CAN BE ACHIEVED BY EITHER SANDBLASTING OR THE USE OF A RETARDING AGENT ON SIDE FORMS PRIOR TO PLACEMENT.

INDEX OF SHEETS:

FOR CONSTRUCTION
100% DOCUMENTS
FOR CONSTRUCTION

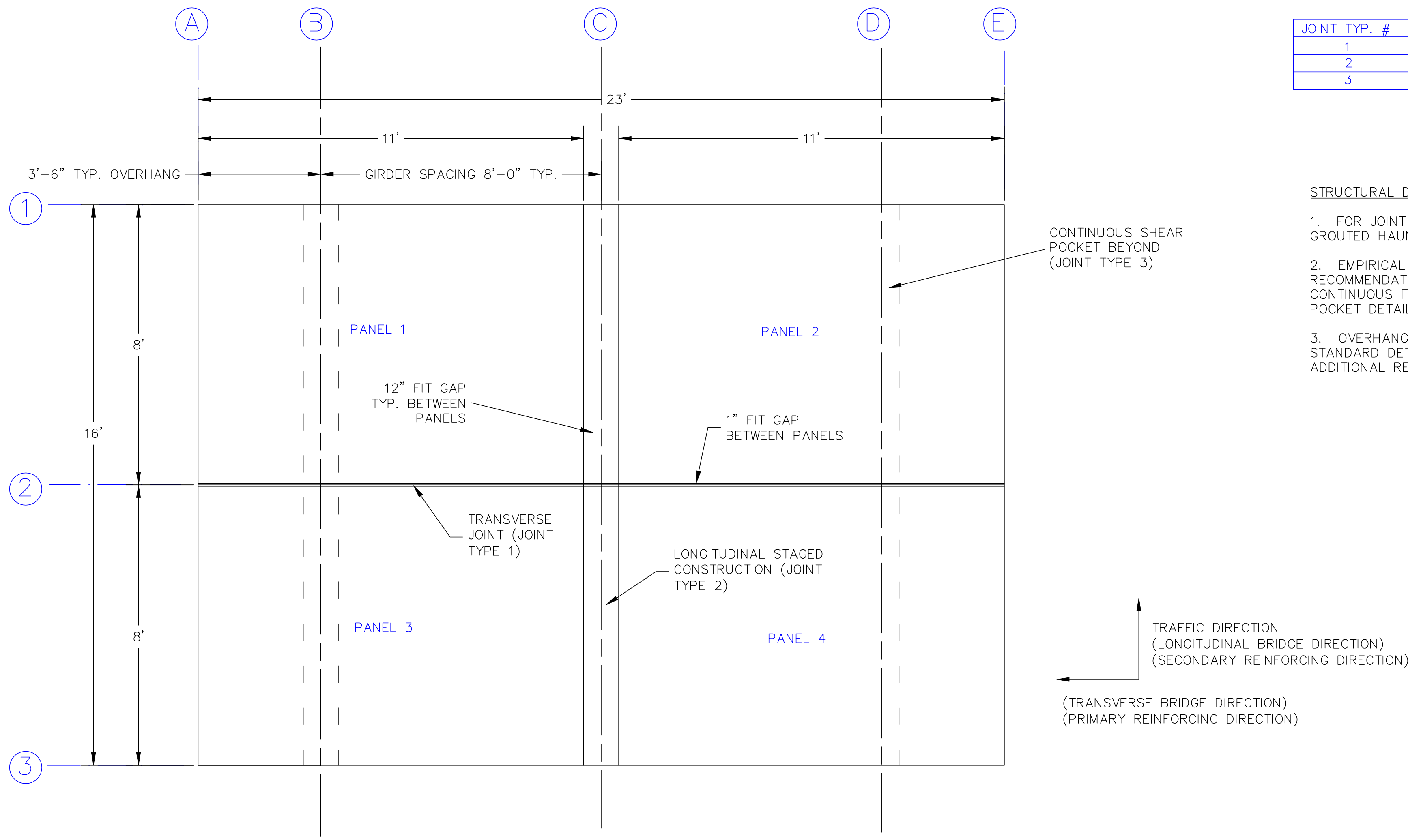
General Notes

No.	Revision/Issue	Date
	100% DOCUMENTS	

Firm Name and Address
AUBURN UNIVERSITY
CIVIL ENGINEERING
238 HARBERT CENTER
AUBURN, AL 36849

Project Name and Address
GENERAL NOTES

Project 220087	Sheet S1
Date 7/19/2011	
Scale AS NOTED	



1/S2 GENERAL SPECIMEN LAYOUT AND CONTROL
SCALE: 1/2" = 1'-0"

JOINT TYP. #	JOINT TYPE	SHEAR	+ MOMENT	– MOMENT
1	TRANSVERSE JOINT	X	X	X
2	LONGITUDINAL STAGED CONSTRUCTION			X
3	CONTINUOUS SHEAR POCKET			X

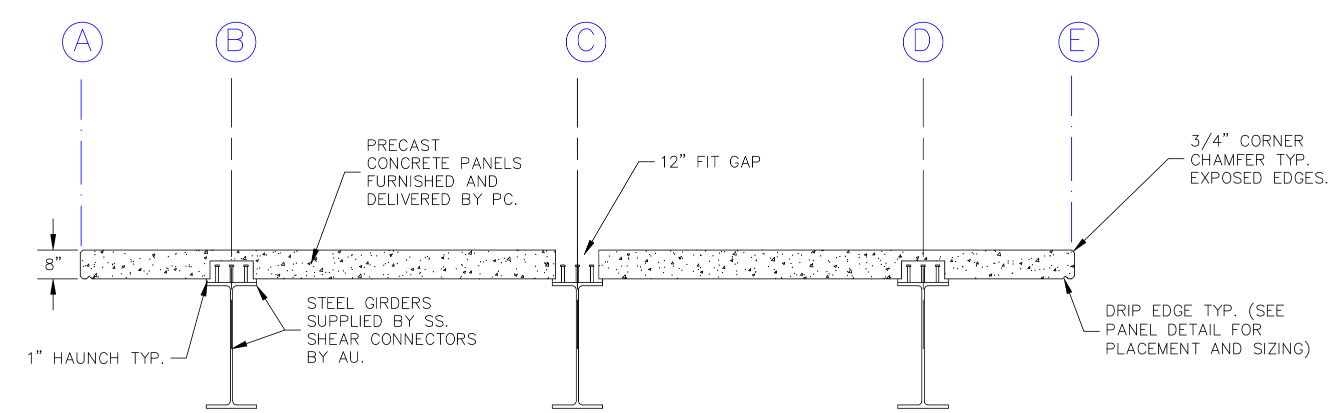
3/S2 JOINT DESIGN FORCE EFFECTS
SCALE: NTS

STRUCTURAL DESIGN NOTES:

1. FOR JOINT CONFIGURATION OVER A GIRDER LINE, SHEAR IS CARRIED THROUGH GROUTED HAUNCH.

2. EMPIRICAL DESIGN METHOD UTILIZED FOR INTERIOR SLAB. PER NCHRP REPORT 584 RECOMMENDATIONS AS WELL AS PAPER BY FANG ET. AL, REINFORCING NEED NOT BE CONTINUOUS FOR POSITIVE MOMENT ACROSS STAGED CONSTRUCTION OR CONTINUOUS POCKET DETAIL.

3. OVERHANG DESIGN TYPICAL FOR TL–4 CRASH BARRIER LOADING USING ALDOT STANDARD DETAIL. BARRIERS ARE NOT INCLUDED IN THIS SPECIMEN FOR SIMPLICITY, BUT ADDITIONAL REINFORCING BAR IN SLAB IS INCLUDED FOR CONSTRUCTIBILITY CONCERNS.



2/S2 GENERAL SPECIMEN ELEVATION
SCALE: 1/2" = 1'-0"

General Notes

1. SEE DWG. S1 FOR TYPICAL NOTES AND REQUIREMENTS.

No.	Revision/Issue	Date

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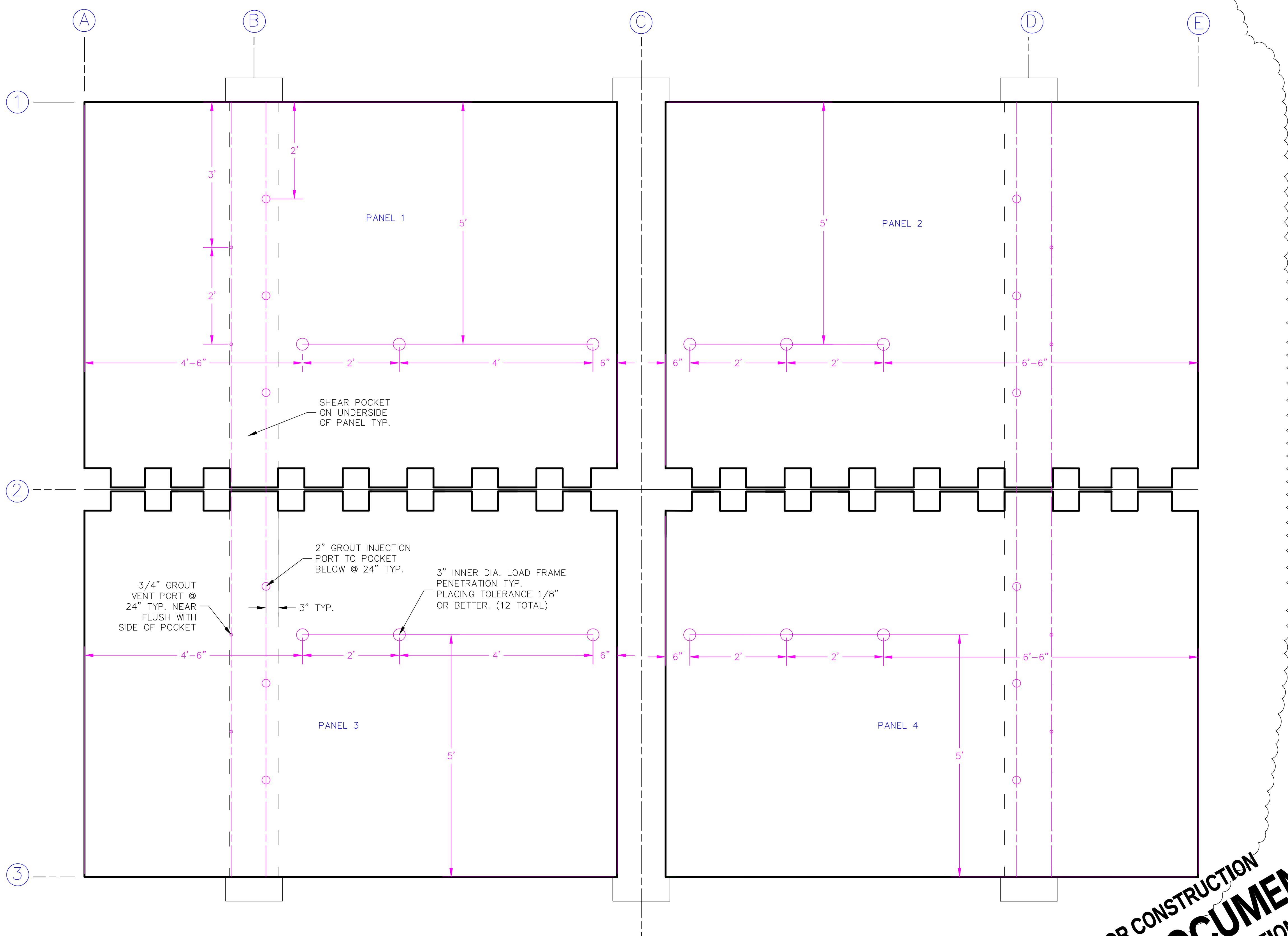
AUBURN, AL 36849

Project Name and Address
GENERAL LAYOUT/CONTROL

Project 220087
Date 7/19/2011
Scale AS NOTED
Sheet S2



GROUT PORT NOTE: THE GOAL OF THE GROUT INJECTION AND VENT PORT SPACING IS TO HAVE A 2" DIA. INJECTION PORT AT LEAST EVERY 24" ALONG THE POCKET WITH A CORRESPONDING VENT PORT WITHIN 24" OF INJECTION LOCATION. THE CONFIGURATION DETAILED ABOVE IS SATISFACTORY.

1 / S2A PRECAST PANEL PENETRATION PLAN
SCALE: 1" = 1'-0"

FOR CONSTRUCTION
100% DOCUMENTS
FOR CONSTRUCTION

General Notes

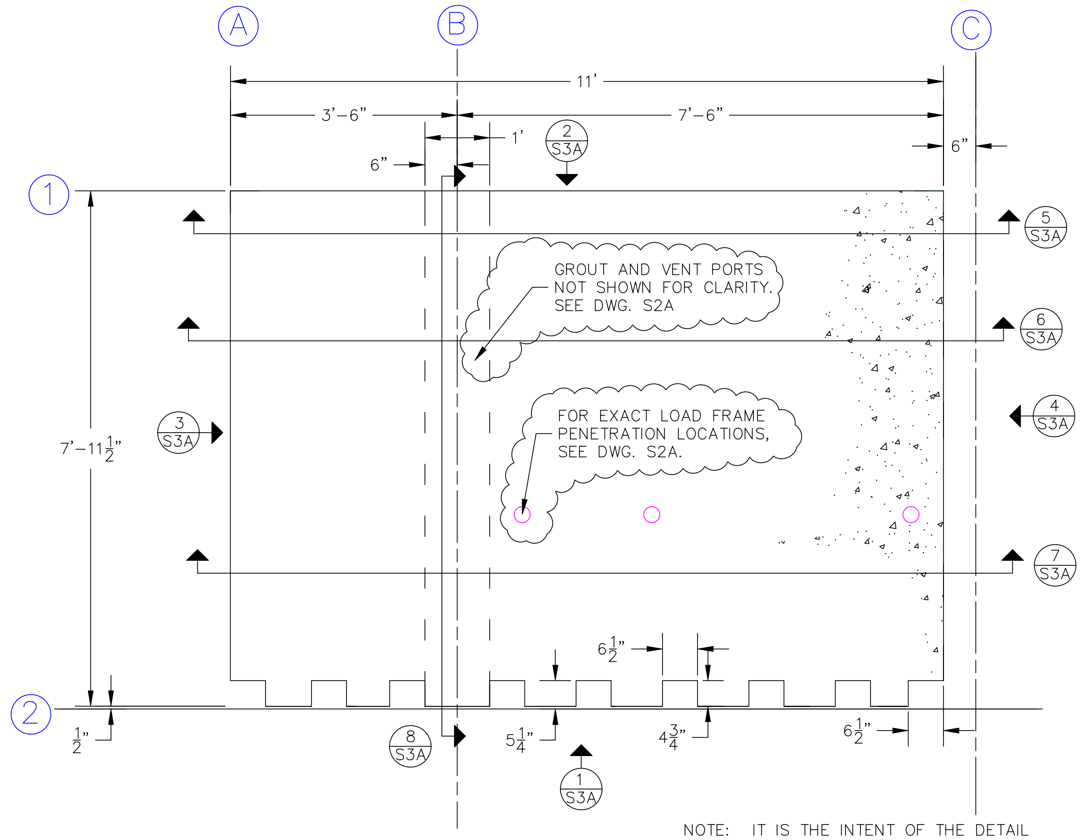
1. SEE DWG. S1 FOR TYPICAL NOTES AND REQUIREMENTS.

No.	Revision/Issue	Date

Firm Name and Address
AUBURN UNIVERSITY
CIVIL ENGINEERING
238 HARBERT CENTER
AUBURN, AL 36849

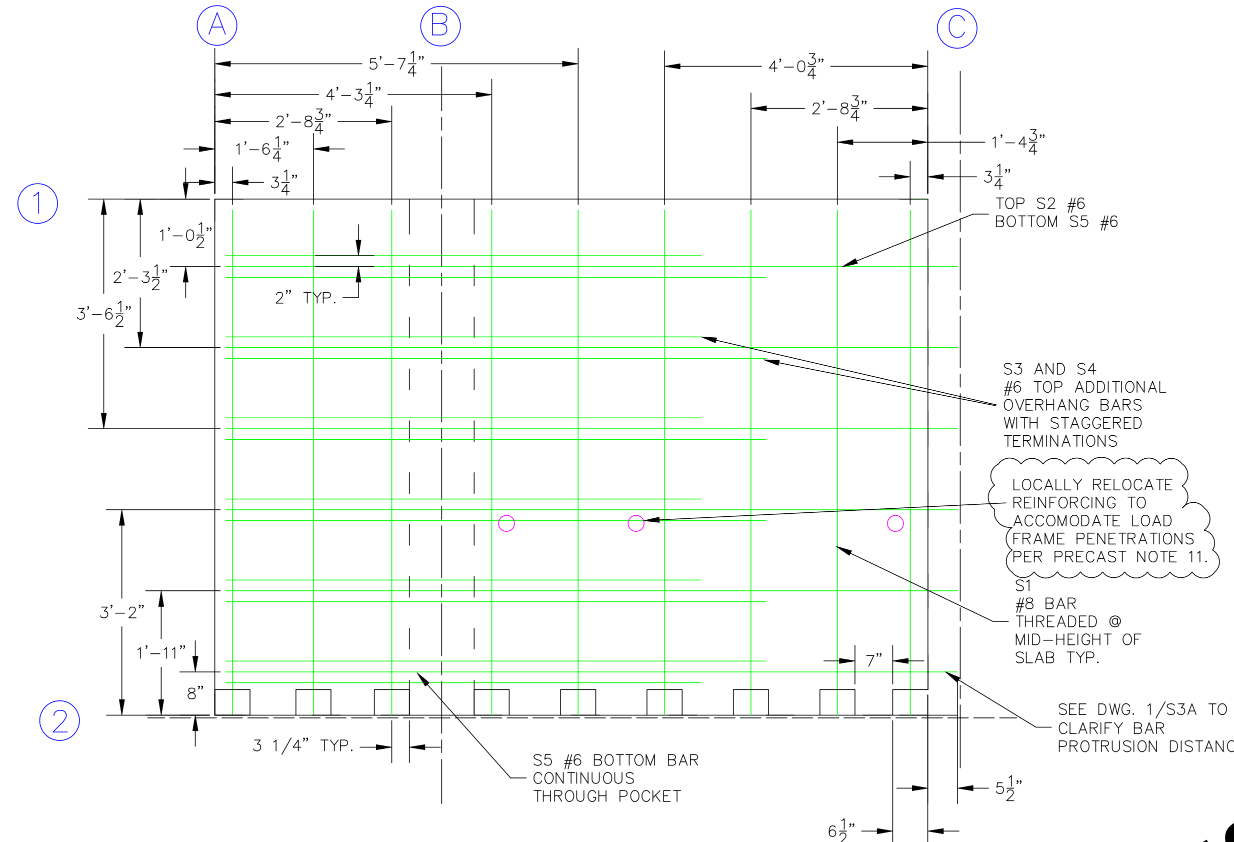
Project Name and Address
PANEL PENETRATION PLAN

Project: 220087
Date: 7/19/2011
Scale: AS NOTED
Sheet: S2A



1 / S3 PANEL 1 GEOMETRY PLAN
SCALE: 3/4" = 1'-0"

NOTE: IT IS THE INTENT OF THE DETAIL THAT POCKET LOCATIONS ALONG 2-LINE ARE CENTERED ON PROTRUDING REINFORCING SHOWN IN DWG. 2/S3.

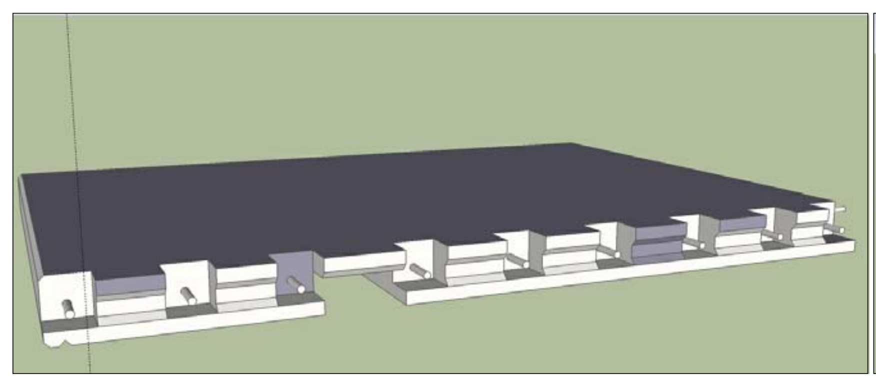


2 / S3 PANEL 1 REINFORCING PLAN
SCALE: 3/4" = 1'-0"

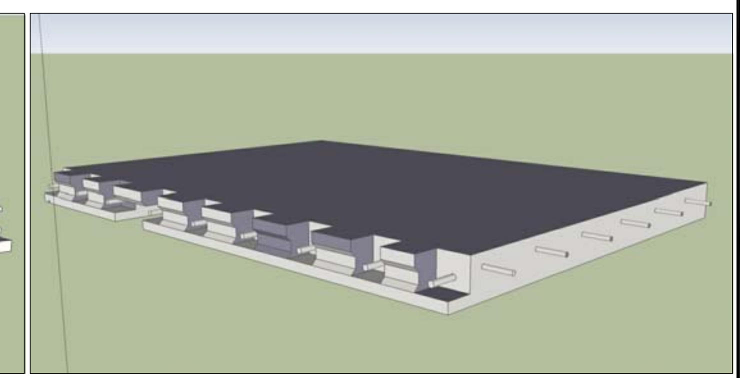
FOR CONSTRUCTION
100% DOCUMENTS
FOR CONSTRUCTION

MARK	LOCATION	SIZE.	THREAD	NO. BARS	RAW LENGTH	SKETCH
S1	DECK	#8	Y	9		7'-9 1/2"
S2	DECK	#6	Y	6		11'-3 1/2"
S3	DECK	#6	N	6		7'-3" R 1.25" 3"
S4	DECK	#6	N	6		8'-3" R 1.25" 3"
S5	DECK	#6	N	6		10'-8"

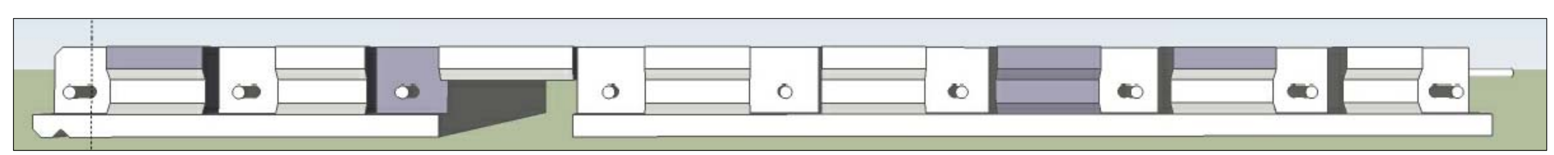
3 / S3 PANEL 1 REINFORCING SCHEDULE
SCALE: NTS



4 / S3 RENDERING PANEL 1 CONCRETE GEOMETRY
SCALE: NTS



5 / S3 RENDERING PANEL 1 CONCRETE GEOMETRY
SCALE: NTS



6 / S3 RENDERING PANEL 1 TRANSVERSE JOINT
SCALE: NTS

NOTE: ABOVE RENDERINGS (DWGS. 4,5, AND 6) ARE FOR CONCRETE GEOMETRY ONLY AND DO NOT SHOW BOTTOM REINFORCING BAR CONTINUOUS THROUGH PCKKET AT BOTTOM OF PANEL.

General Notes

1. SEE DWG. S1 FOR TYPICAL NOTES AND REQUIREMENTS.

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AUBURN UNIVERSITY
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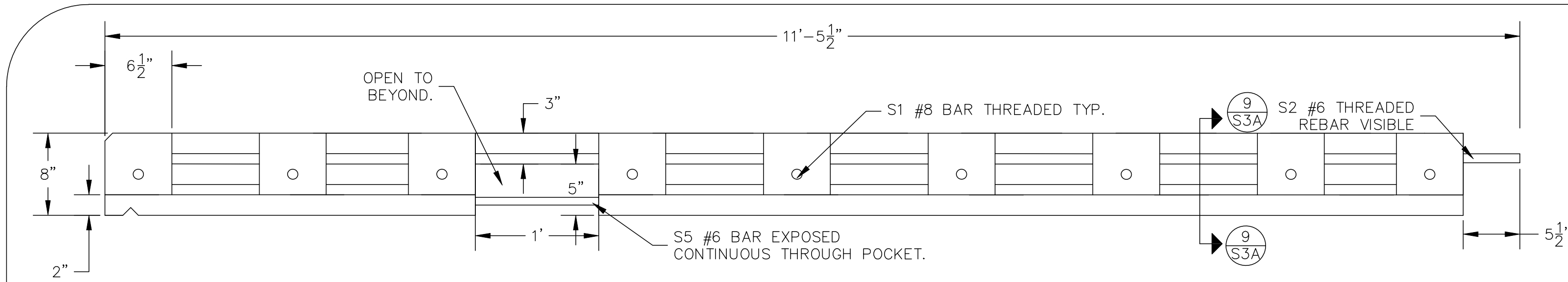
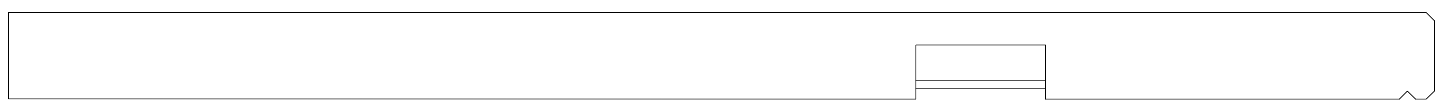
238 HARBERT CENTER
AUBURN, AL 36849

Project Name and Address
PANEL 1 PLAN AND REINFORCING SCHEDULE

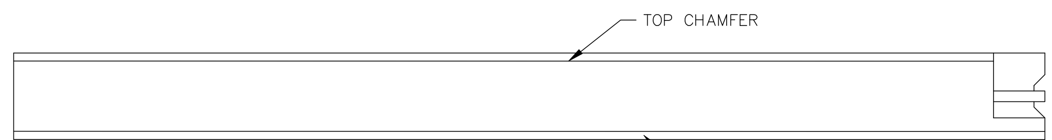
Project: 220087
Date: 7/19/2011
Scale: AS NOTED
Sheet: S3



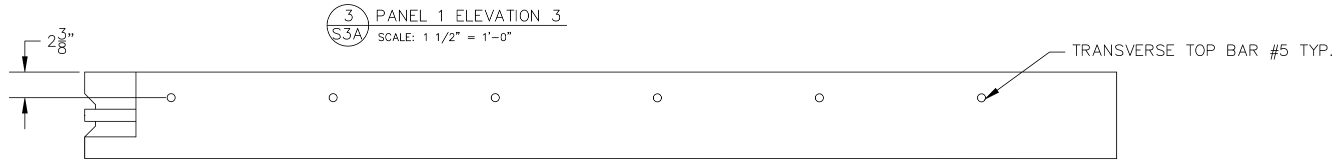
1 S3A PANEL 1 ELEVATION 1
SCALE: 1 1/2" = 1'-0"



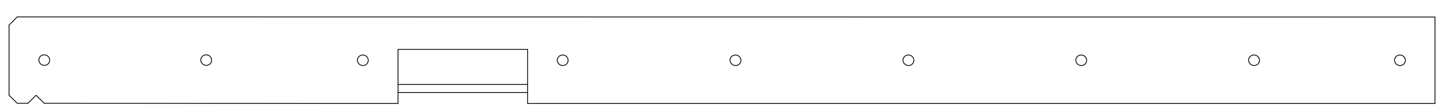
2 S3A PANEL 1 ELEVATION 2
SCALE: 1 1/2" = 1'-0"



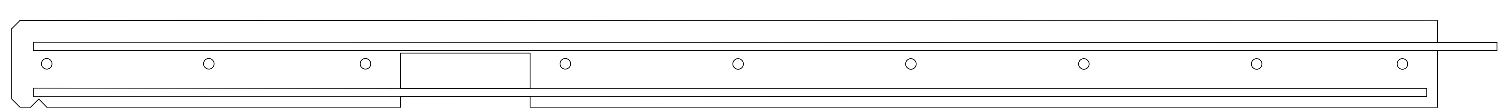
3 S3A PANEL 1 ELEVATION 3
SCALE: 1 1/2" = 1'-0"



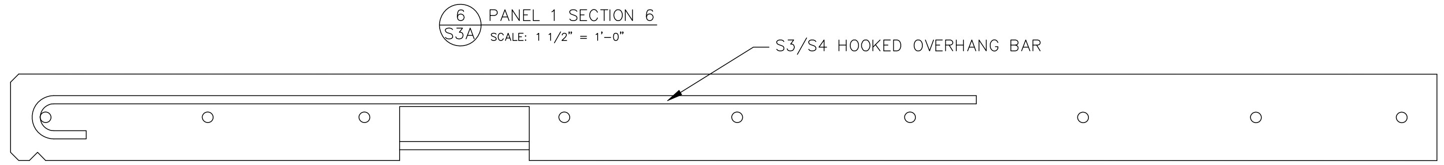
4 S3A PANEL 1 ELEVATION 4
SCALE: 1 1/2" = 1'-0"



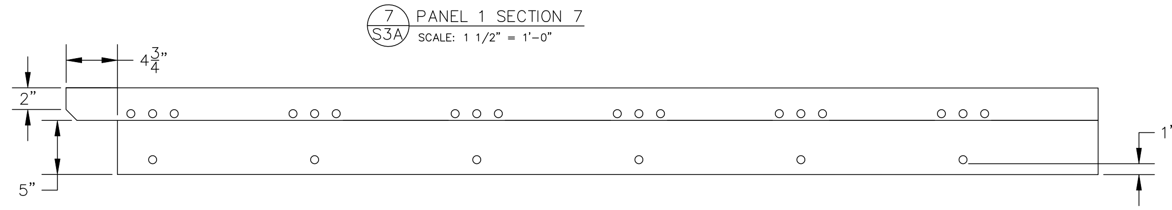
5 S3A PANEL 1 SECTION 5
SCALE: 1 1/2" = 1'-0"



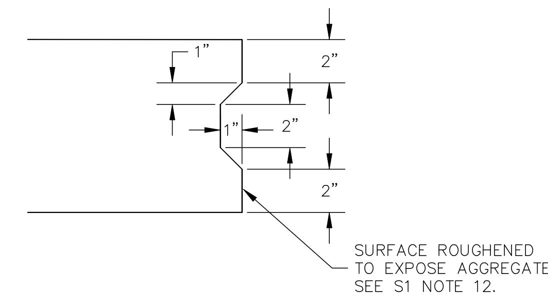
6 S3A PANEL 1 SECTION 6
SCALE: 1 1/2" = 1'-0"



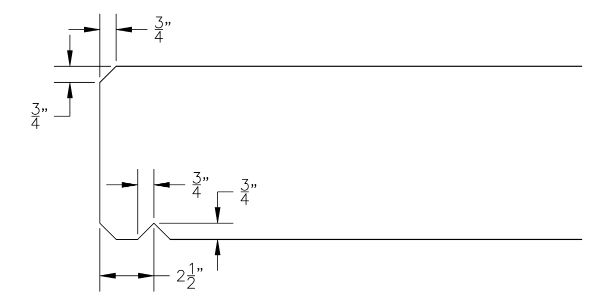
7 S3A PANEL 1 SECTION 7
SCALE: 1 1/2" = 1'-0"



8 S3A PANEL 1 SECTION 8
SCALE: 1 1/2" = 1'-0"



9 S3A SHEAR KEY DETAIL
SCALE: 3" = 1'-0"



9 S3A TYPICAL CHAMFER/ DRIP GROVE DETAIL
SCALE: 3" = 1'-0"

FOR CONSTRUCTION
100% DOCUMENTS
FOR CONSTRUCTION

General Notes

1. SEE DWG. S1 FOR TYPICAL NOTES AND REQUIREMENTS.

No.	Revision/Issue	Date

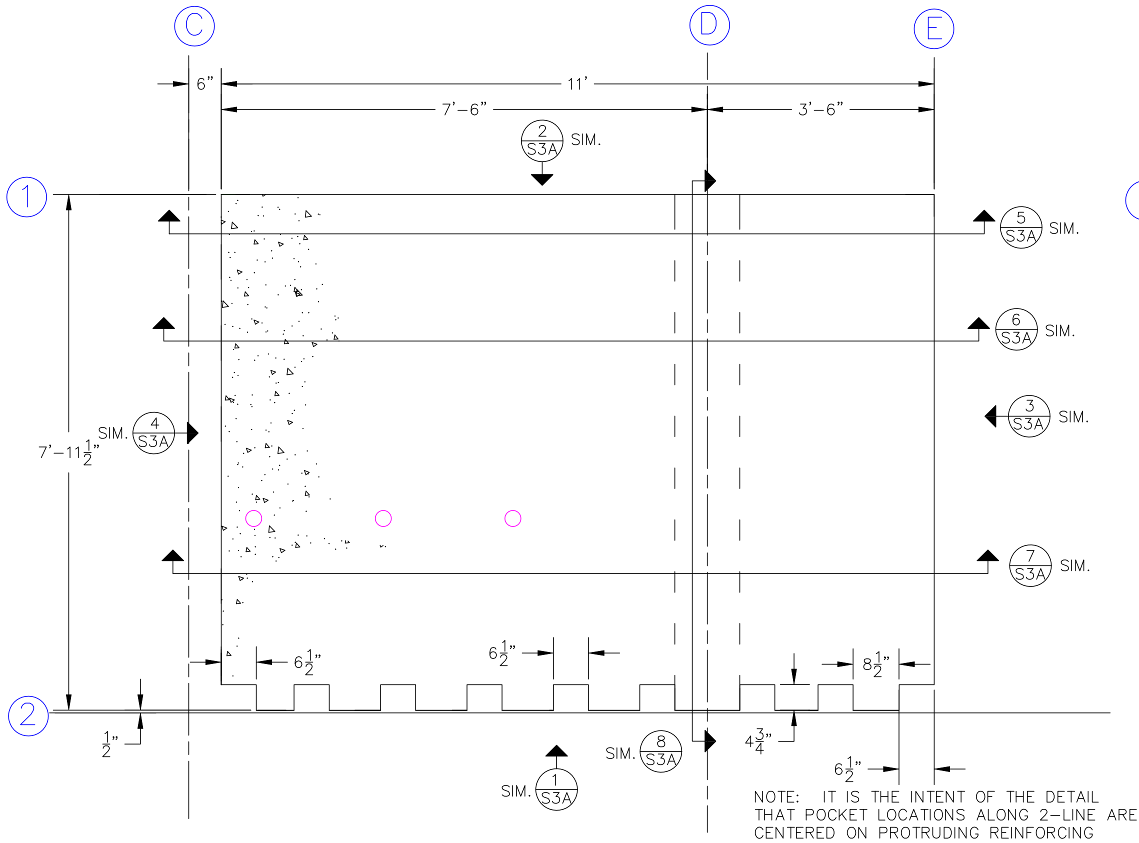
Firm Name and Address

AUBURN UNIVERSITY
CIVIL ENGINEERING
238 HARBERT CENTER
AUBURN, AL 36849

Project Name and Address

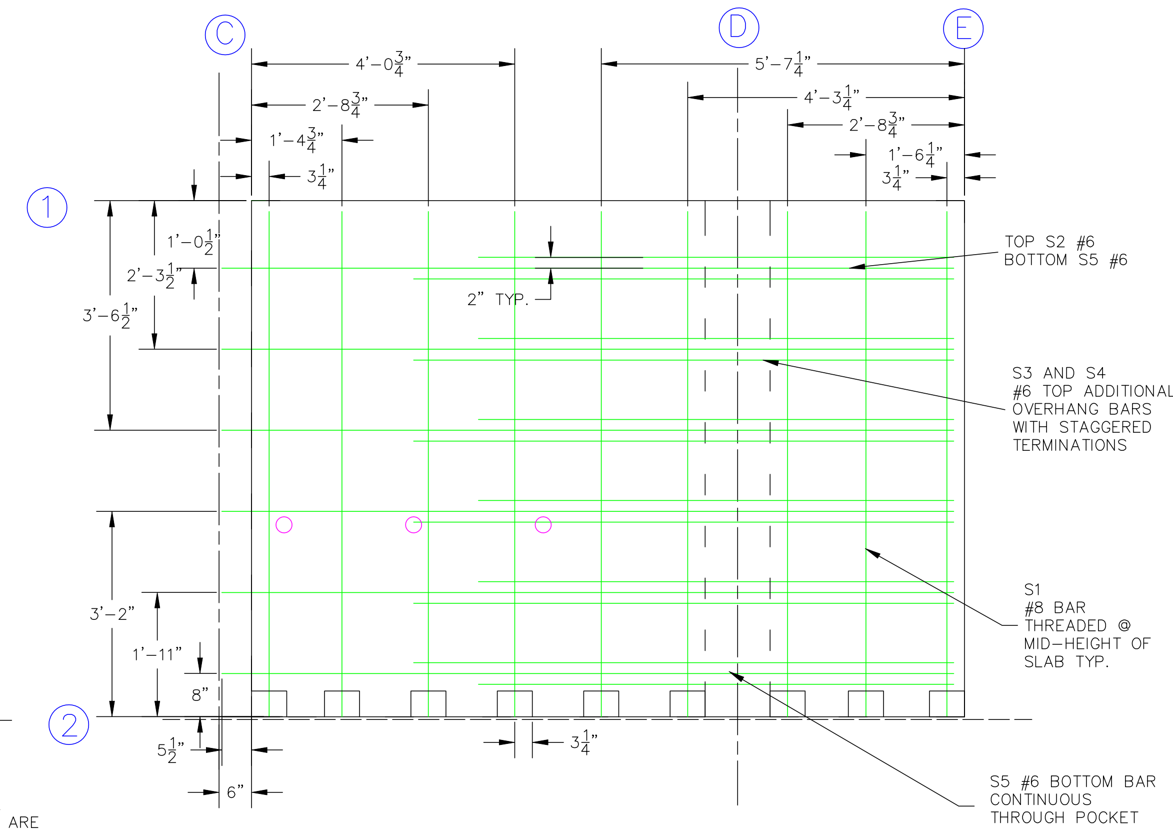
PANEL 1 ELEVATIONS AND SECTIONS

Project: 220087
Date: 7/19/2011
Scale: AS NOTED
Sheet: S3A



NOTE: IT IS THE INTENT OF THE DETAIL THAT POCKET LOCATIONS ALONG 2-LINE ARE CENTERED ON PROTRUDING REINFORCING SHOWN IN DWG. 2/S3.

1/S3 PANEL 2 GEOMETRY PLAN
SCALE: 3/4" = 1'-0"



2/S3 PANEL 2 REINFORCING PLAN
SCALE: 3/4" = 1'-0"

MARK	LOCATION	SIZE.	THREAD	NO. BARS	RAW LENGTH	SKETCH
S1	DECK	#8	Y	9		7'-9 1/2"
S2	DECK	#6	Y	6		11'-3 1/2"
S3	DECK	#6	N	6		7'-3" R 1.25" 3"
S4	DECK	#6	N	6		8'-3" R 1.25" 3"
S5	DECK	#6	N	6		10'-8"

3/S3 PANEL 2 REINFORCING SCHEDULE
SCALE: NTS

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FOR CONSTRUCTION

General Notes

1. SEE DWG. S1 FOR TYPICAL NOTES AND REQUIREMENTS.

No.	Revision/Issue	Date

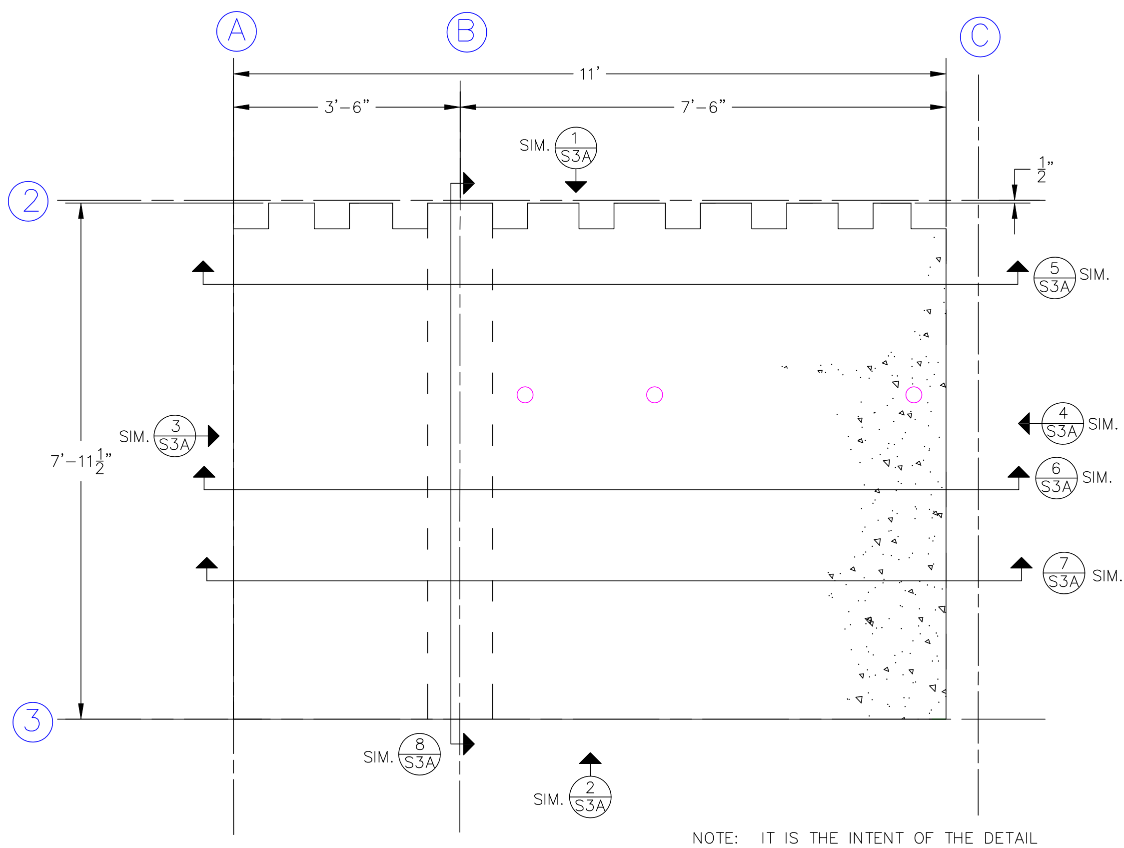
Firm Name and Address

AUBURN UNIVERSITY
CIVIL ENGINEERING
238 HARBERT CENTER
AUBURN, AL 36849

Project Name and Address

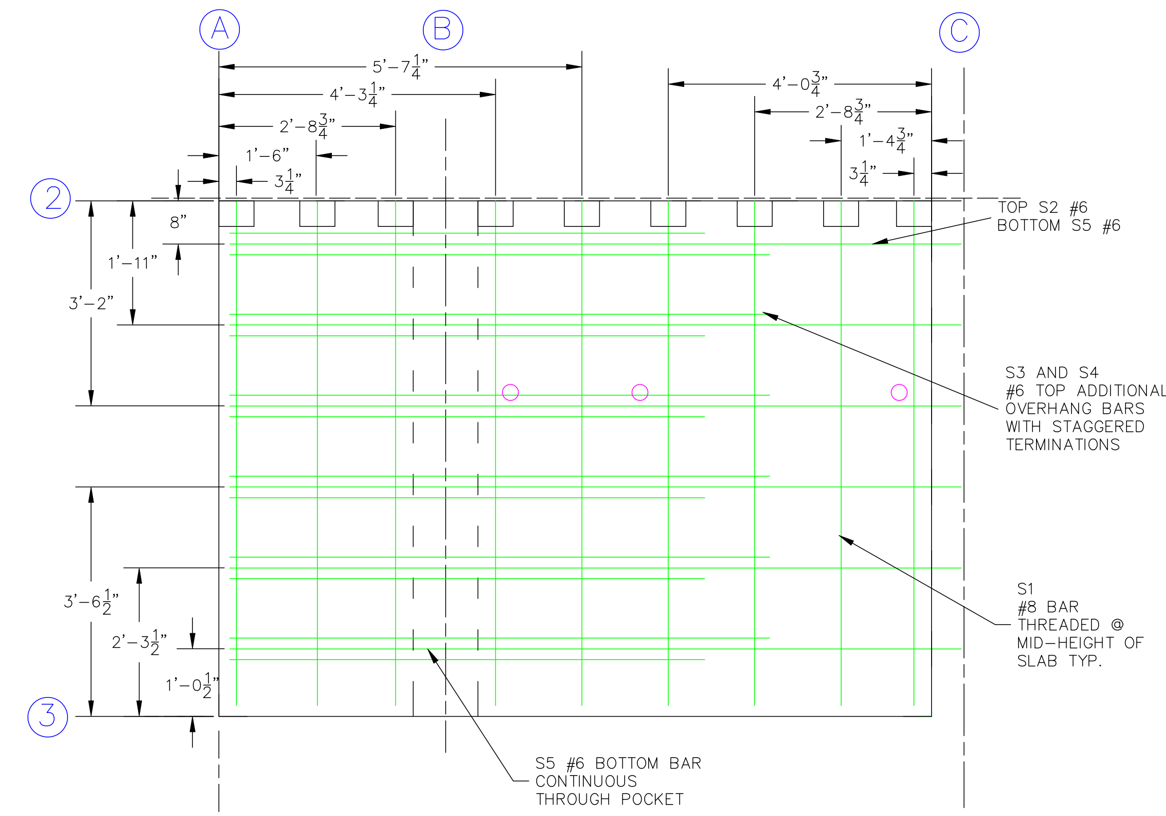
PANEL 2 PLAN AND REINFORCING SCHEDULE

Project 220087
Date 7/19/2011
Scale AS NOTED
Sheet S4



1/S3 PANEL 3 GEOMETRY PLAN
SCALE: 3/4" = 1'-0"

NOTE: IT IS THE INTENT OF THE DETAIL THAT POCKET LOCATIONS ALONG 2-LINE ARE CENTERED ON PROTRUDING REINFORCING SHOWN IN DWG. 2/S3.



2/S3 PANEL 3 REINFORCING PLAN
SCALE: 3/4" = 1'-0"

MARK	LOCATION	SIZE.	THREAD	NO. BARS	RAW LENGTH	SKETCH
S1	DECK	#8	Y	9		7'-9 1/2"
S2	DECK	#6	Y	6		11'-3 1/2"
S3	DECK	#6	N	6		7'-3" R 1.25" 3"
S4	DECK	#6	N	6		8'-3" R 1.25" 3"
S5	DECK	#6	N	6		10'-8"

3/S3 PANEL 3 REINFORCING SCHEDULE
SCALE: NTS

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FOR CONSTRUCTION

General Notes

1. SEE DWG. S1 FOR TYPICAL NOTES AND REQUIREMENTS.

No.	Revision/Issue	Date

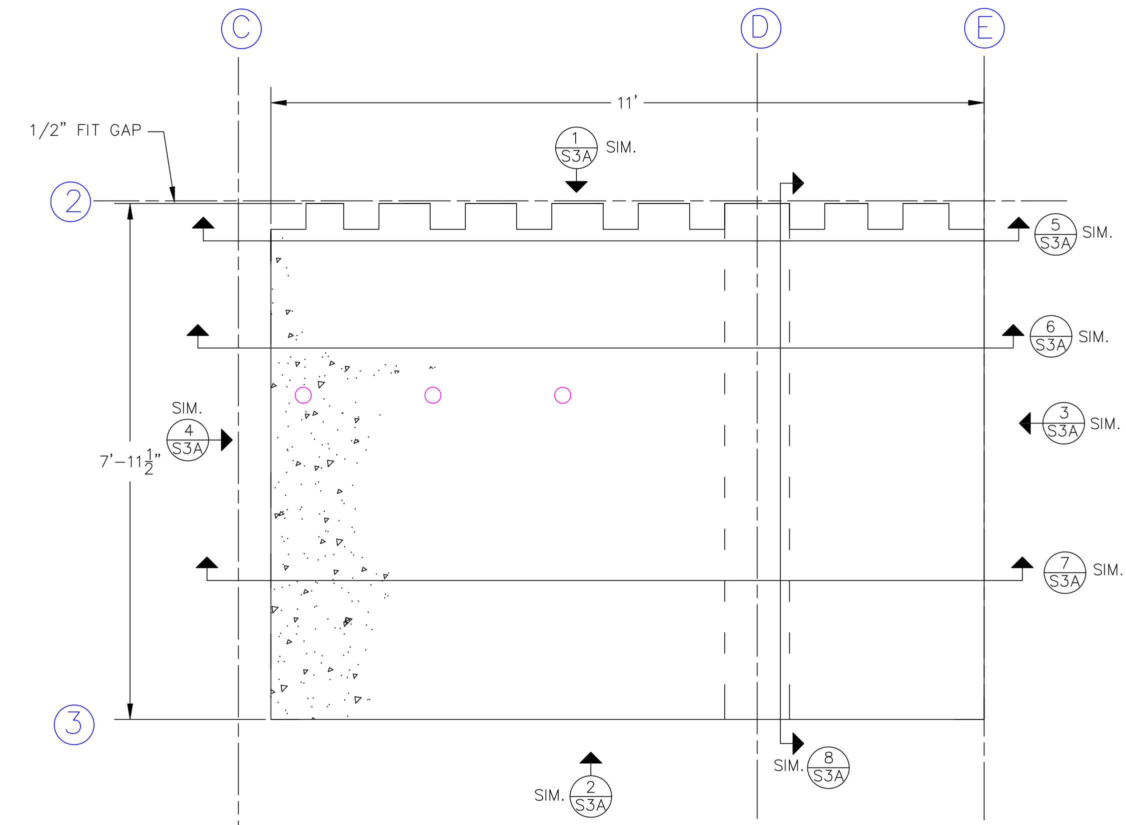
Firm Name and Address

AUBURN UNIVERSITY
CIVIL ENGINEERING
238 HARBERT CENTER
AUBURN, AL 36849

Project Name and Address

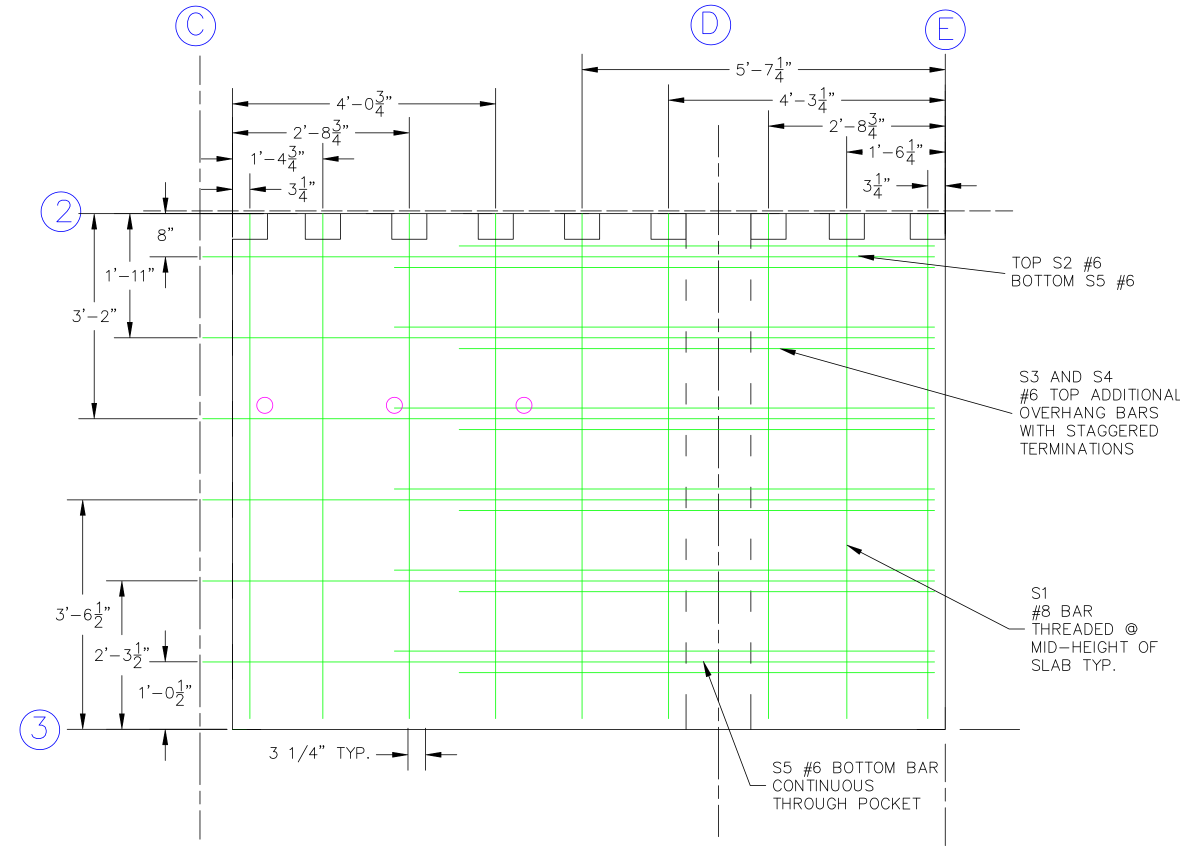
PANEL 3 PLAN AND REINFORCING SCHEDULE

Project 220087
Date 7/19/2011
Scale AS NOTED
Sheet S5



1 S3 PANEL 4 GEOMETRY PLAN
SCALE: 3/4" = 1'-0"

NOTE: IT IS THE INTENT OF THE DETAIL THAT POCKET LOCATIONS ALONG 2-LINE ARE CENTERED ON PROTRUDING REINFORCING SHOWN IN DWG. 2/S3.



2 S3 PANEL 4 REINFORCING PLAN
SCALE: 3/4" = 1'-0"

MARK	LOCATION	SIZE.	THREAD	NO. BARS	RAW LENGTH	SKETCH
S1	DECK	#8	Y	9		7'-9 1/2"
S2	DECK	#6	Y	6		11'-3 1/2"
S3	DECK	#6	N	6		7'-3" R 1.25" 3"
S4	DECK	#6	N	6		8'-3" R 1.25" 3"
S5	DECK	#6	N	6		10'-8"

3 S3 PANEL 4 REINFORCING SCHEDULE
SCALE: NTS

FOR CONSTRUCTION
100% DOCUMENTS
FOR CONSTRUCTION

General Notes

1. SEE DWG. S1 FOR TYPICAL NOTES AND REQUIREMENTS.

No.	Revision/Issue	Date

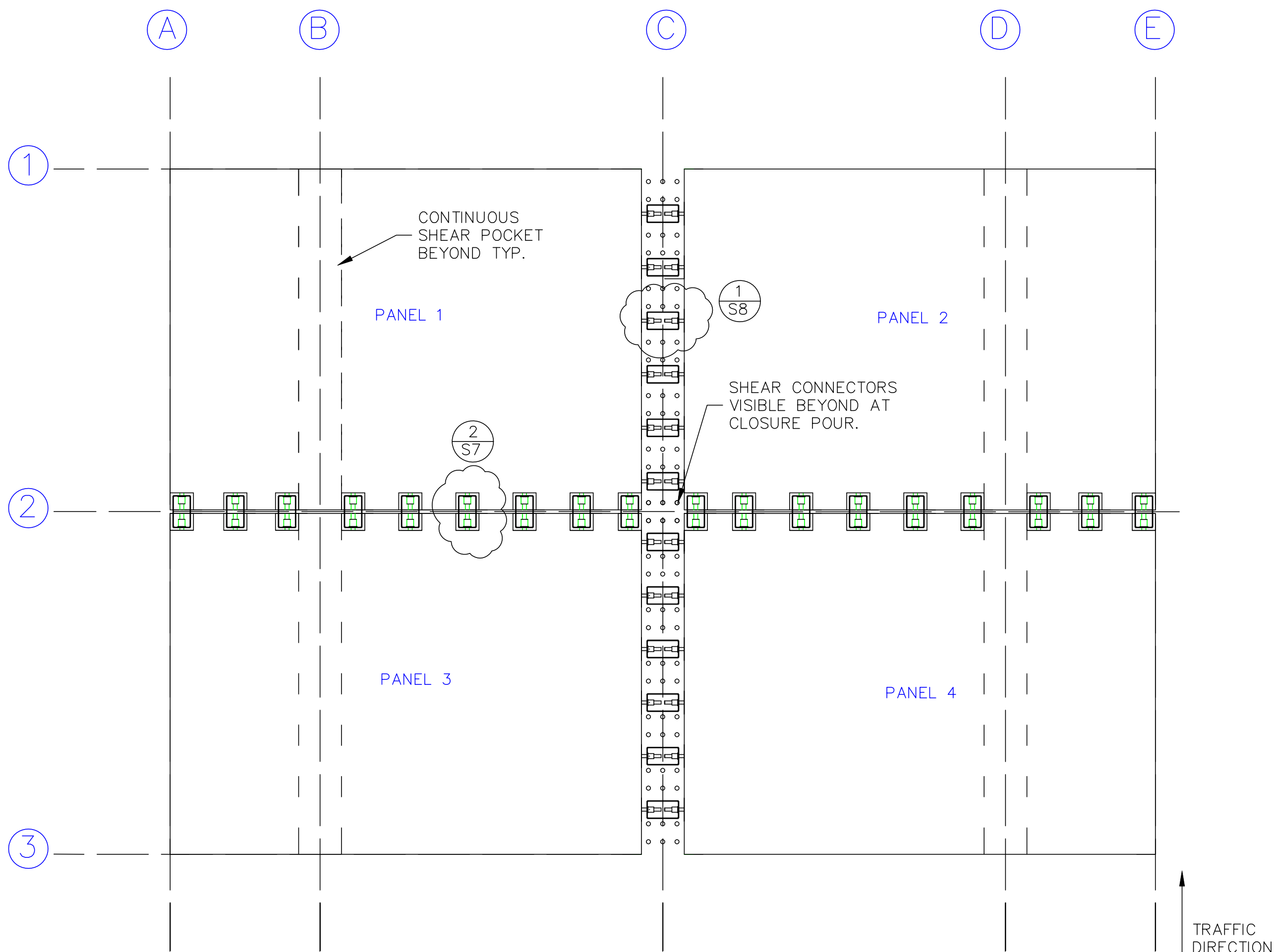
Firm Name and Address

AUBURN UNIVERSITY
CIVIL ENGINEERING
238 HARBERT CENTER
AUBURN, AL 36849

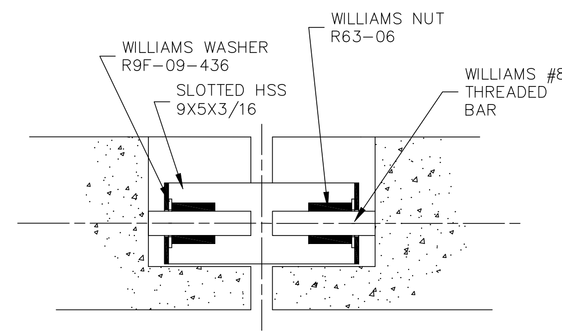
Project Name and Address

PANEL 4 PLAN AND REINFORCING SCHEDULE

Project 220087
Date 7/19/2011
Scale AS NOTED
Sheet S6



1 S7 FINAL CONNECTION DETAILL PRIOR TO GROUT
SCALE: 1/2" = 1'-0"



7 S7 SECTION 7 TRANSVERSE CONNECTION
SCALE: 3" = 1'-0"

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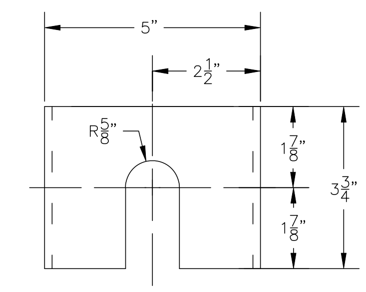
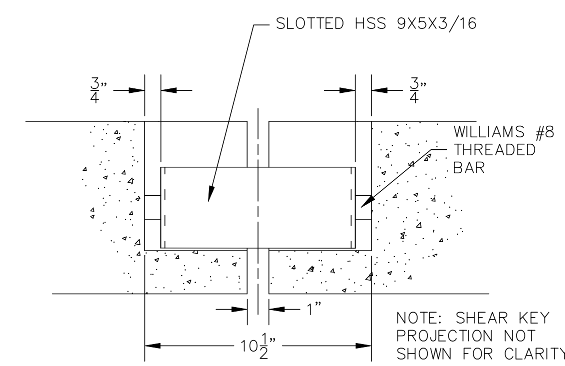
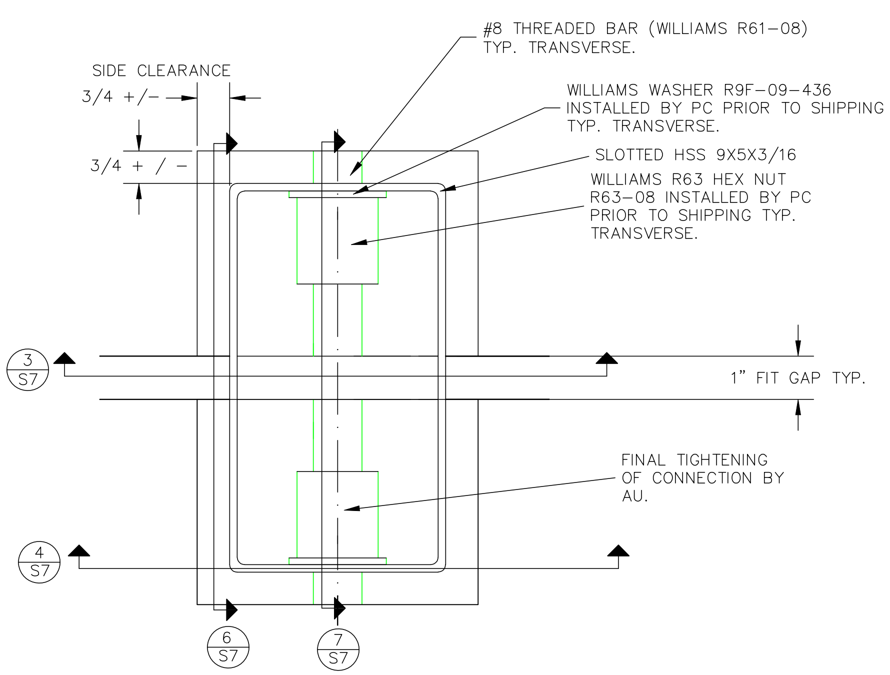


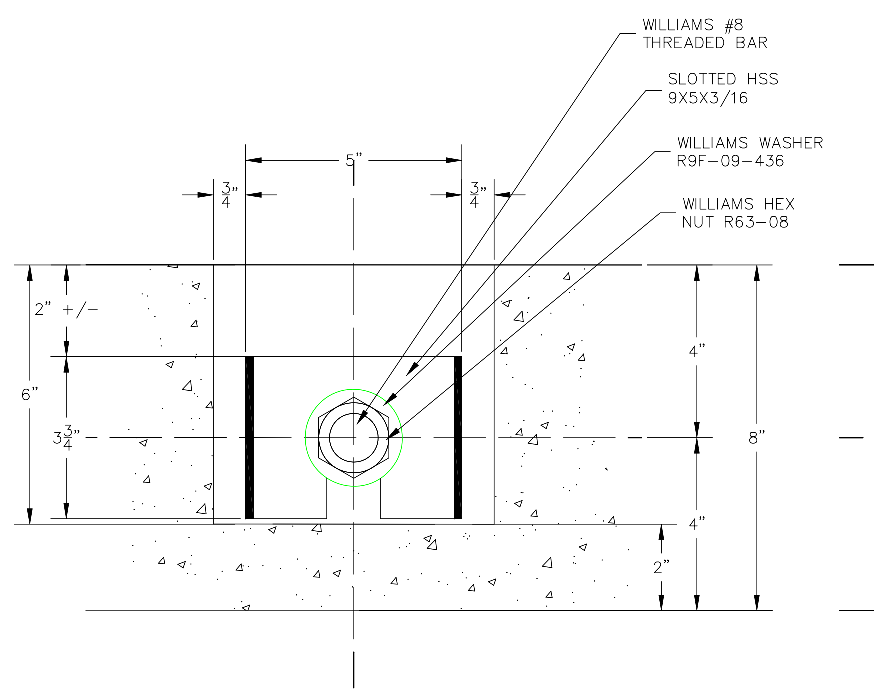
5 S7 SECTION 5 TRANSVERSE CONNECTION HSS GEOMETRY
SCALE: 6" = 1'-0"



6 S7 SECTION 6 TRANSVERSE CONNECTION
SCALE: 3" = 1'-0"

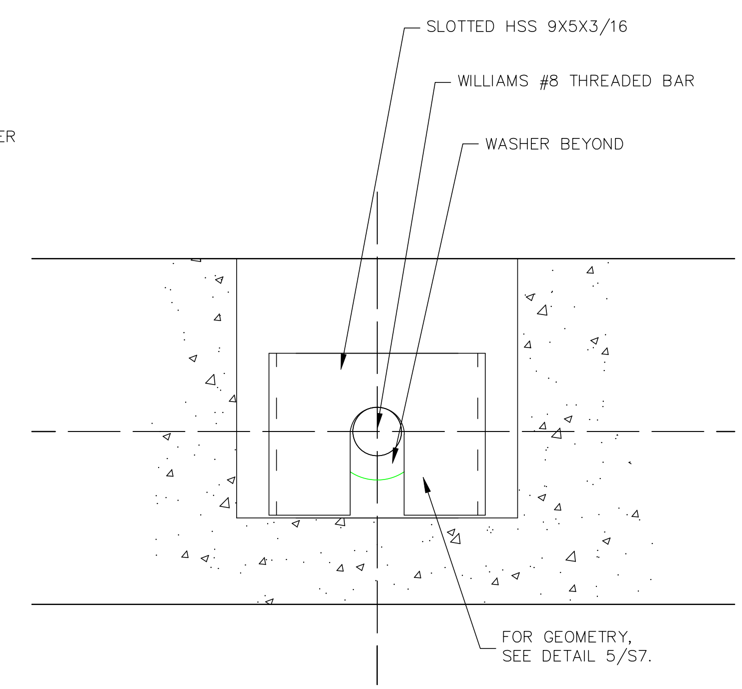


2 S7 PLAN TRANVERSE CONNECTION TYP. PRIOR TO GROUT
SCALE: 6" = 1'-0"



3 S7 SECTION 3 TRANSVERSE CONNECTION
SCALE: 6" = 1'-0"

NOTE: SHEAR KEY PROJECTION NOT SHOWN FOR CLARITY.



4 S7 SECTION 4 TRANSVERSE CONNECTION
SCALE: 6" = 1'-0"

General Notes

1. SEE DWG. S1 FOR TYPICAL NOTES AND REQUIREMENTS.

ESTIMATED QUANTITIES:

TRANSVERSE HSS: 18 + 5

LONGITUDINAL HSS: 12 + 5

No.	Revision/Issue	Date

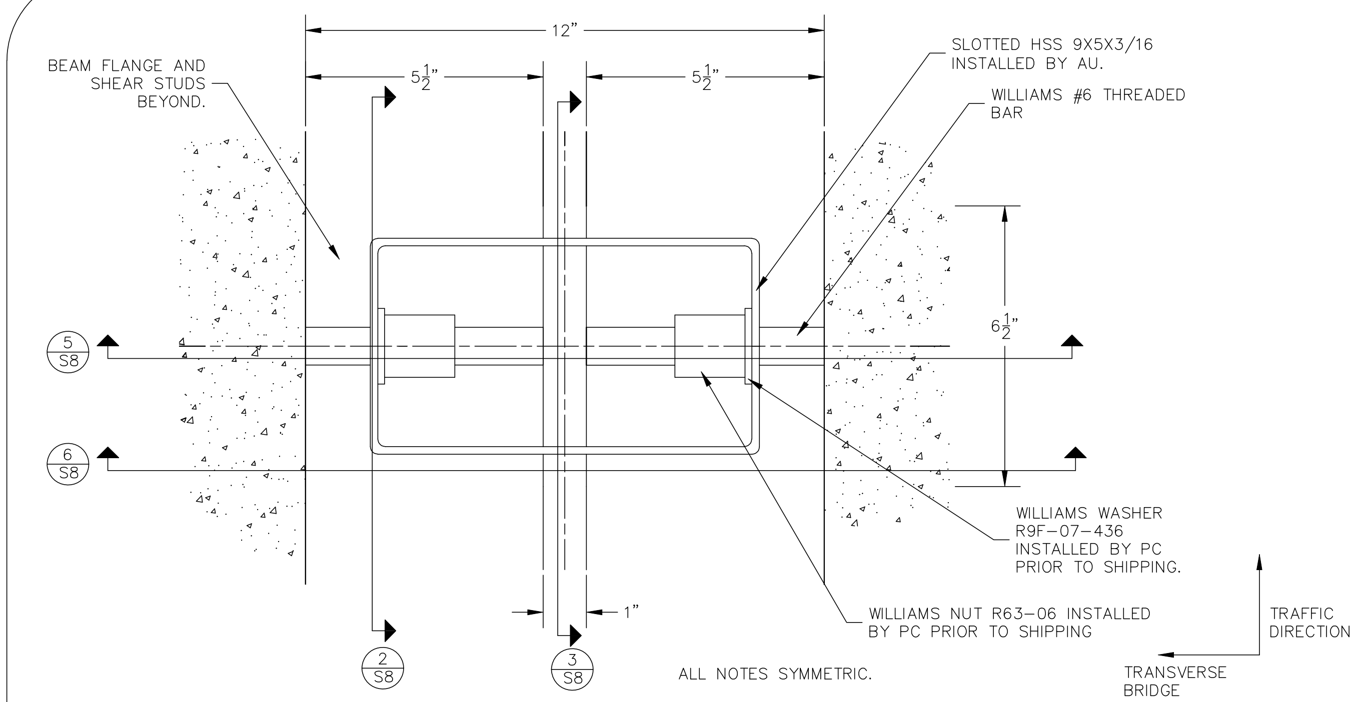
Firm Name and Address

AUBURN UNIVERSITY
CIVIL ENGINEERING
238 HARBERT CENTER
AUBURN, AL 36849

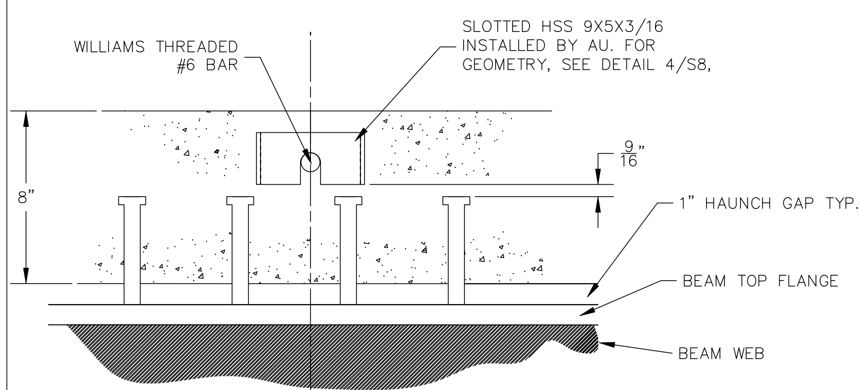
Project Name and Address

TRANSVERSE JOINT DETAILS

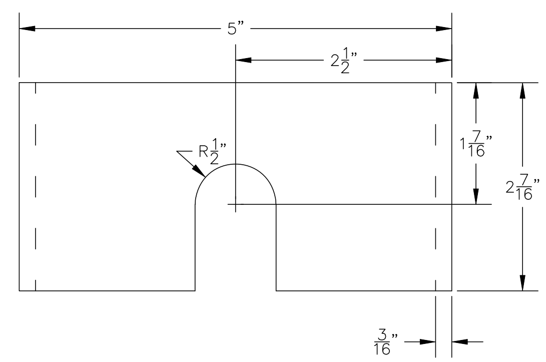
Project 220087
Date 7/19/2011
Scale AS NOTED
Sheet S7



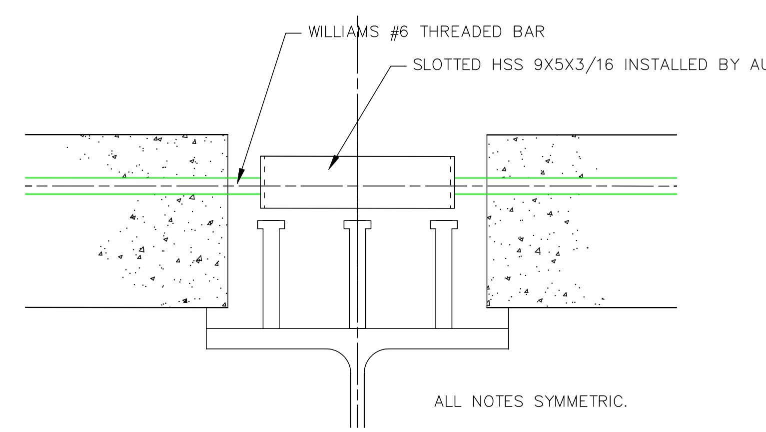
1 S8 PLAN LONGITUDINAL CONNECTION TYP. PRIOR TO GROUT
SCALE: 6" = 1'-0"



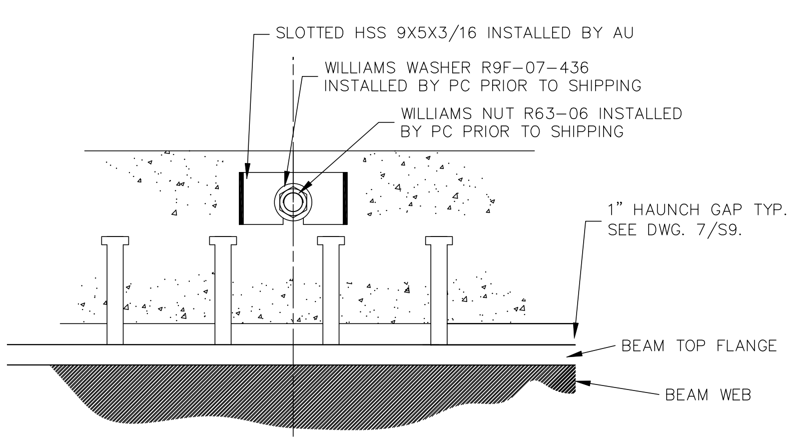
2 S8 SECTION 2 LONGITUDINAL CONNECTION
SCALE: 3" = 1'-0"



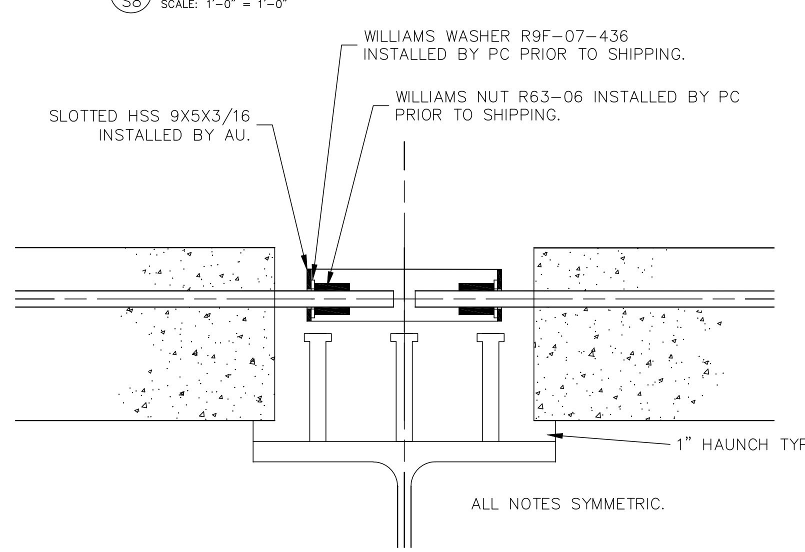
4 S8 SECTION 4 LONGITUDINAL CONNECTION HSS GEOMETRY
SCALE: 1'-0" = 1'-0"



6 S8 SECTION 6 LONGITUDINAL CONNECTION
SCALE: 3" = 1'-0"



3 S8 SECTION 3 LONGITUDINAL CONNECTION
SCALE: 3" = 1'-0"



5 S8 SECTION 5 LONGITUDINAL CONNECTION
SCALE: 3" = 1'-0"

FOR CONSTRUCTION
100% DOCUMENTS
FOR CONSTRUCTION

General Notes

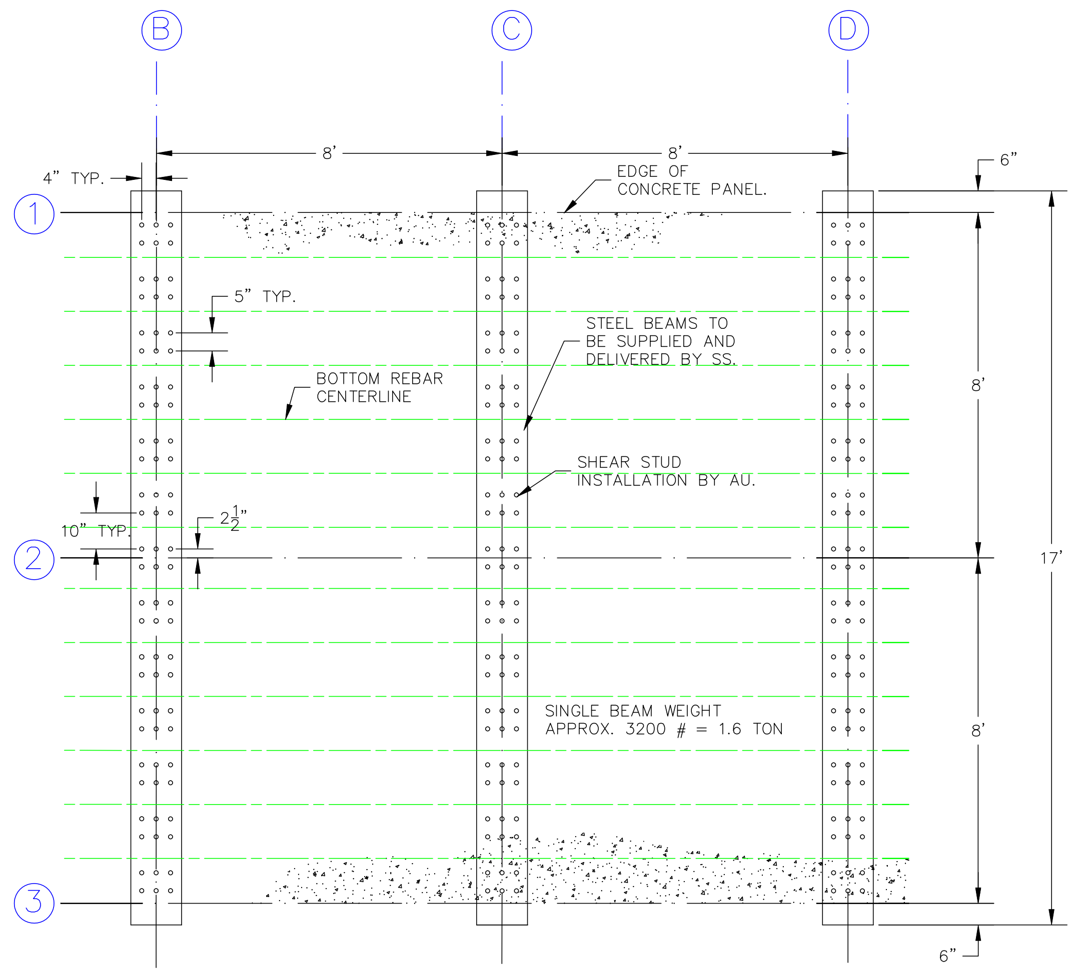
1. SEE DWG. S1 FOR TYPICAL NOTES AND REQUIREMENTS.

No.	Revision/Issue	Date

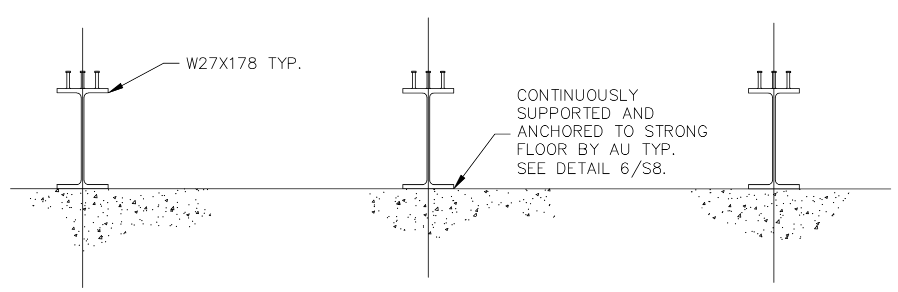
Firm Name and Address
AUBURN UNIVERSITY
CIVIL ENGINEERING
238 HARBERT CENTER
AUBURN, AL 36849

Project Name and Address
LONGITUDINAL JOINT DETAILS

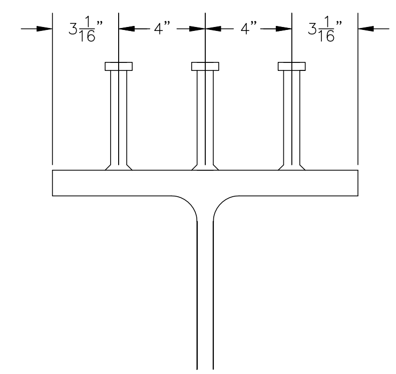
Project 220087
Date 7/19/2011
Scale AS NOTED
Sheet S8



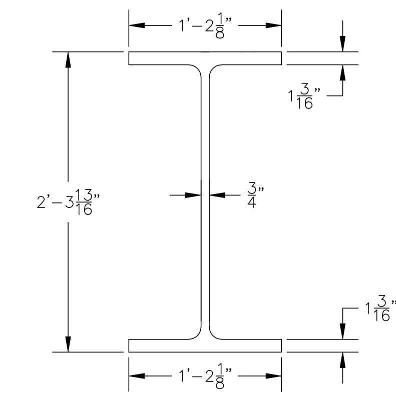
1/S9 STRUCTURAL STEEL PLAN
SCALE: 1/2" = 1'-0"



2/S9 STRUCTURAL STEEL ELEVATION
SCALE: 1/2" = 1'-0"



3/S9 SHEAR CONNECTOR DETAIL
SCALE: 3" = 1'-0"

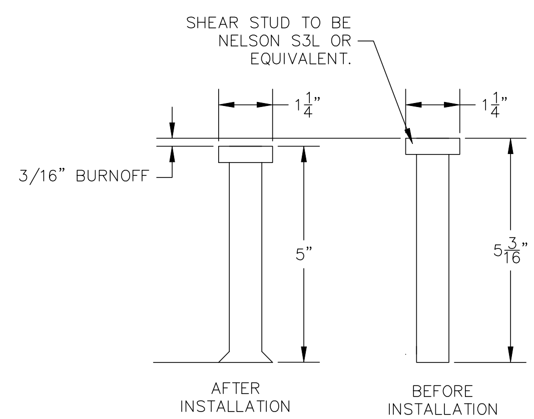


4/S9 W27X178 REFERENCE DIMENSIONS
SCALE: 3" = 1'-0"

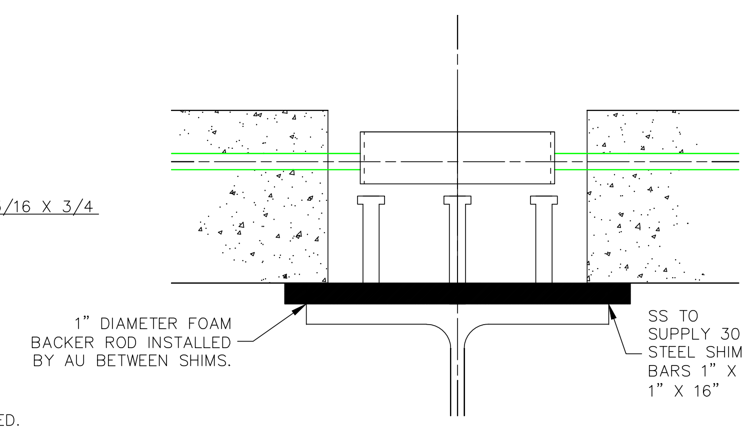
NOTES:

1. SS MAY PROPOSE ALTERNATE SECTION FROM AVAILABLE STOCK SATISFYING THE FOLLOWING REQUIREMENTS:
   - TOP FLANGE >= 14"
   - OVERALL DEPTH > 27"

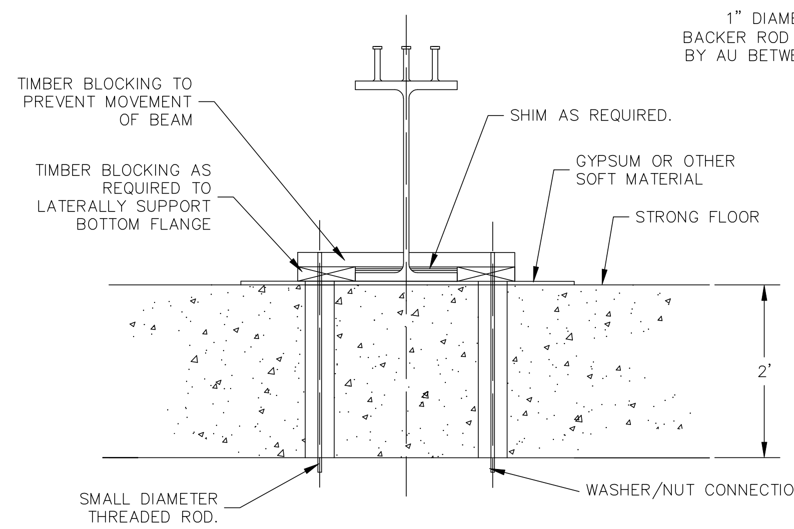
2. ABSOLUTELY NO SUBSTITUTION SHALL BE MADE UNLESS APPROVED BY AU.



5/S9 SHEAR CONNECTOR: NELSON S3L STOCK 5 3/16 X 3/4
SCALE: 6" = 1'-0"



7/S9 HAUNCH SHIM DETAIL
SCALE: 3" = 1'-0"



6/S9 STRONG FLOOR BEARING DETAIL.
SCALE: 1" = 1'-0"

FOR CONSTRUCTION
100% DOCUMENTS
FOR CONSTRUCTION

General Notes

1. SEE DWG. S1 FOR TYPICAL NOTES AND REQUIREMENTS.

ESTIMATED QUANTITIES:

SHEAR CONNECTOR: 250 COUNT

STEEL SHIM BARS: 30 COUNT

STEEL BEAMS: 3 COUNT

1" BACKER ROD: 150 FT

No.	Revision/Issue	Date

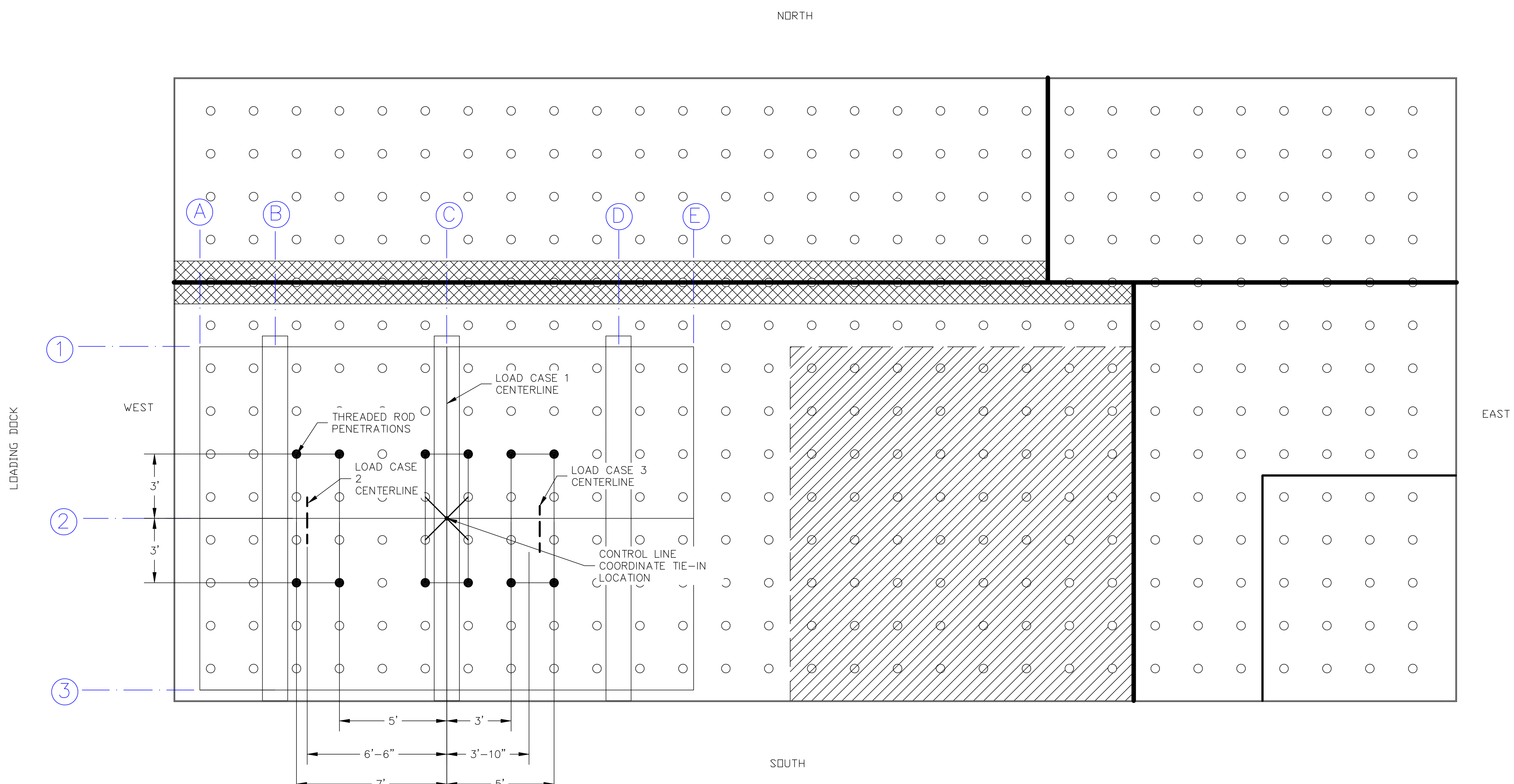
Firm Name and Address

AUBURN UNIVERSITY
CIVIL ENGINEERING
238 HARBERT CENTER
AUBURN, AL 36849

Project Name and Address

STRUCTURAL STEEL

Project: 220087	Sheet: S9
Date: 7/19/2011	
Scale: AS NOTED	



NOTE: THE ABOVE DIMENSIONS ARE FROM CONTROL LINES AND DO NOT ACCOUNT FOR 1" FIT GAP. DIMENSIONS FROM EDGE OF PANEL TO PENETRATION SHOULD BE 1/2" LESS THAN CONTROL DIMENSIONS SHOWN ABOVE. REFERENCE DWG 1/S2A FOR EXACT PANEL PENETRATION SIZING AND LOCATION.

1/S10 HARBERT LAB LOCATION PLAN
SCALE: 3/8" = 1'–0"

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General Notes

1. SEE DWG. S1 FOR TYPICAL NOTES AND REQUIREMENTS.

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LAB LOCATION PLAN

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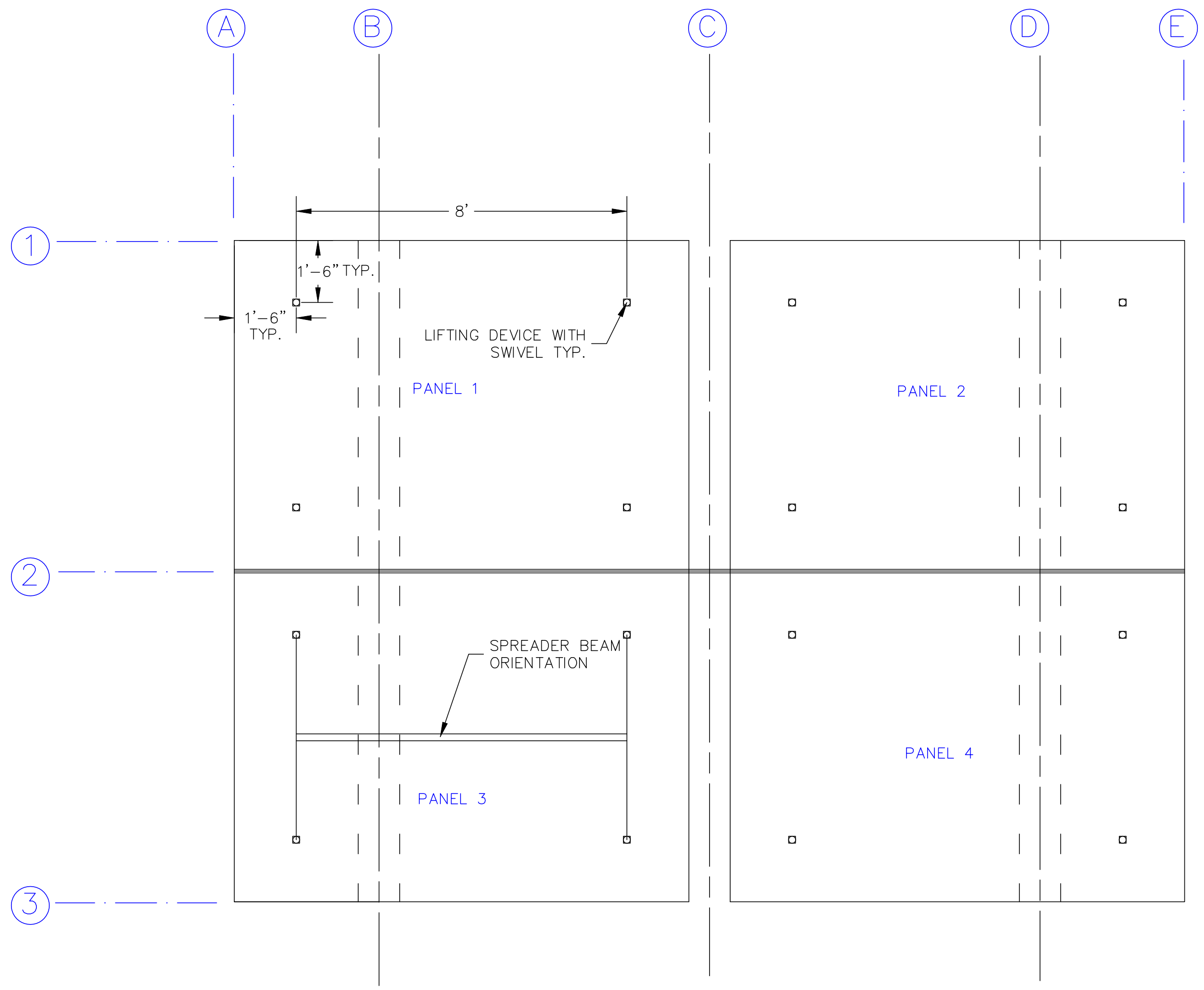
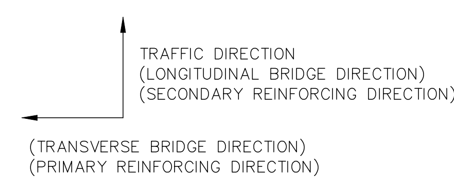


1/S11 PROPOSED RIGGING PLAN
SCALE: 1/2" = 1'-0"



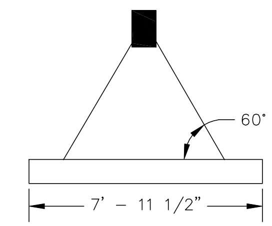
RIGGING NOTES:

1. EACH PANEL SHALL HAVE 4 SWIVEL LIFTING INSERTS WITH SIDE CLEARANCE OF 18" TO CENTER OF DEVICE.

2. ADDITIONAL REINFORCING SHALL BE INCLUDED AT LIFTING POINTS TO DISTRIBUTE LOADS. ADDITIONAL REINFORCING CONFIGURATION TBD BY AU UPON PROPOSAL OF LIFTING INSERTS BY PC.

3. PC SHALL INCLUDE CAST-IN PLACE LIFTING ANCHORS AND CORRESPONDING SWIVEL APPARATUS NECESSARY FOR LIFTING.

4. SPREADER BEAM SHALL BE USED IN THE TRANSVERSE DIRECTION AS SHOWN. SLING ANGLE IN PANEL LONGITUDINAL DIRECTION (MEASURED FROM HORIZONTAL) NOT TO BE LESS THAN 60 DEGREES.



5. PC MAY PROPOSE ALTERNATE RIGGING PLAN AND/OR ANCHOR LOCATIONS FOR APPROVAL BY AU.

6. AU TO FURNISH LIFTING SPREADER BEAM TO PC FOR USE DURING FABRICATION. AFTER FABRICATION, THIS SPREADER BEAM WILL ACCOMPANY PANEL DELIVERY FOR USE IN OFFLOADING BY AU.

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General Notes

1. SEE DWG. S1 FOR TYPICAL NOTES AND REQUIREMENTS.

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PROPOSED RIGGING PLAN

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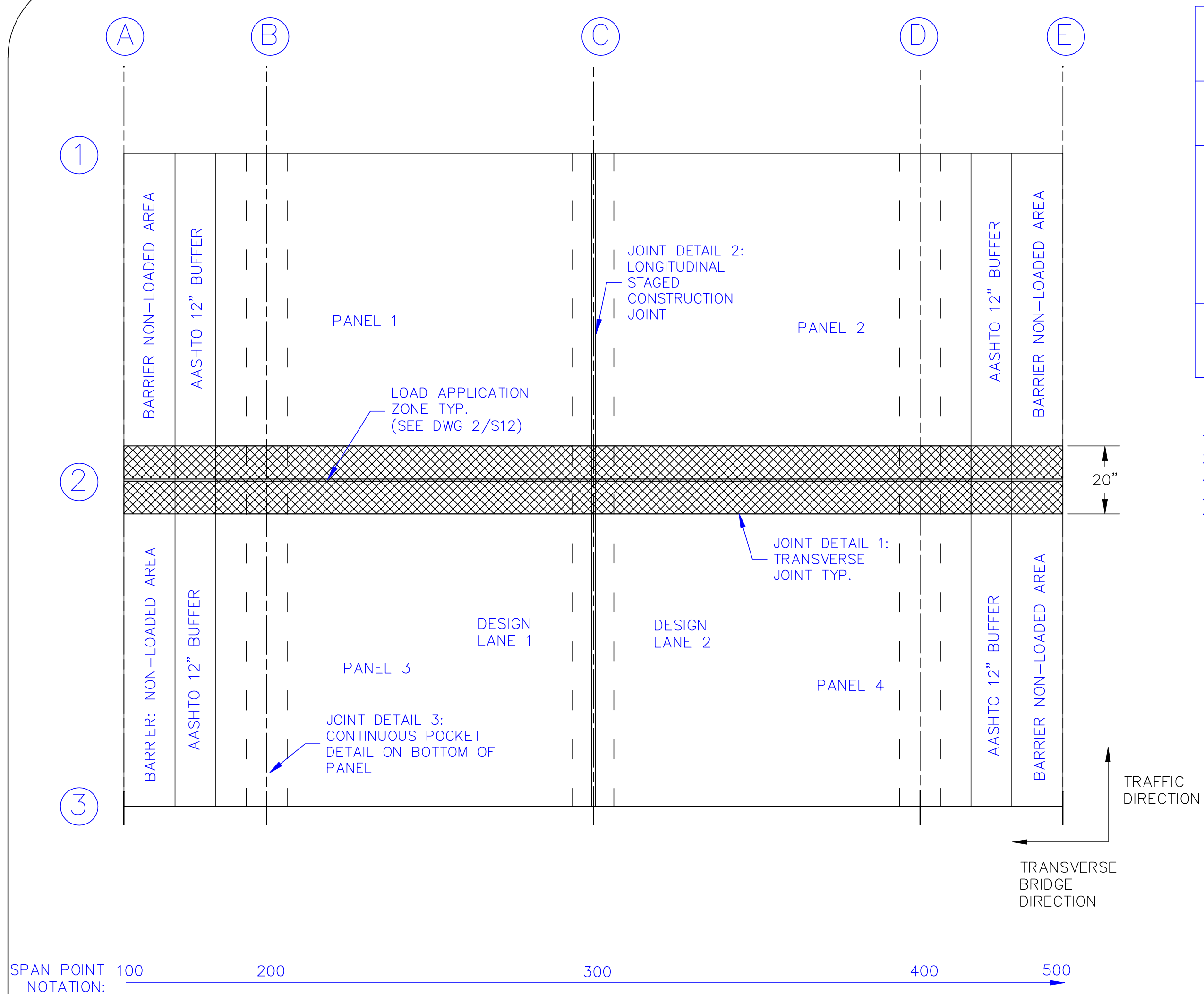
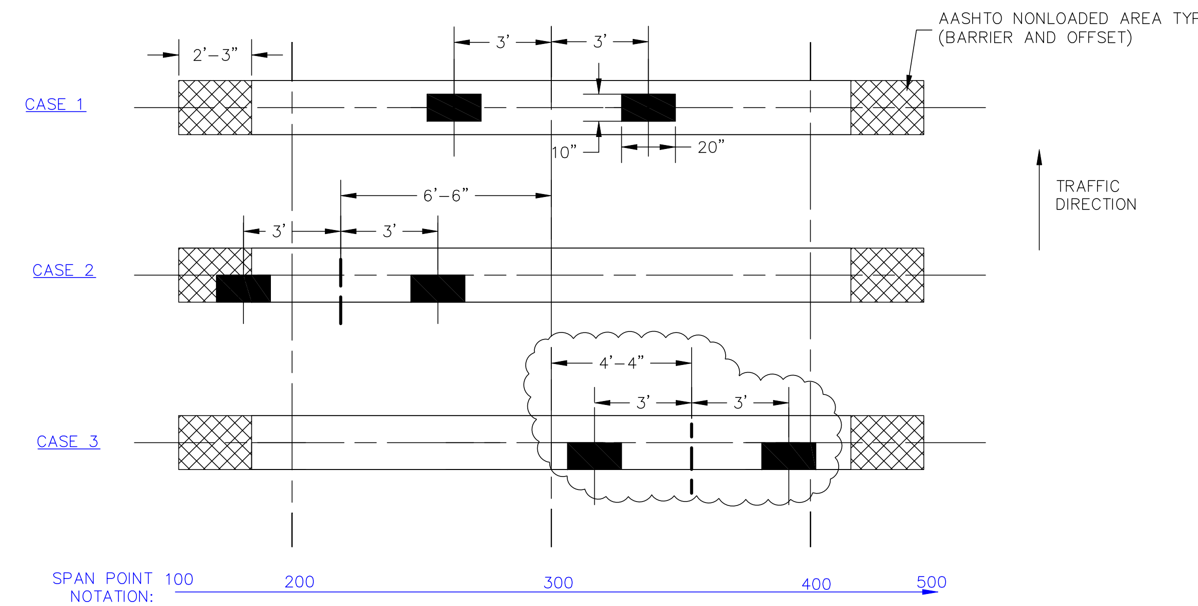
LOAD CASE NUMBER	DESCRIPTION/INTENT:	SINGLE WHEEL LOAD MAGNITUDE:
1	INDUCE MAXIMUM M300 = −42 KIP FT (AT JOINT DETAIL 2) EVALUATE LOCAL DURABILITY OF JOINT DETAIL 1	STATIC: 28 K CYCLIC: 4 K TO 32 K
2	INDUCE MAXIMUM V200 = −28 KIP (AT JOINT DETAIL 3) INDUCE MAXIMUM M200 = −42 KIP FT (AT JOINT DETAIL 3) EVALUATE LOCAL DURABILITY OF JOINT DETAIL 1 (PLEASE NOTE: LOAD APPLICATION IS OUTSIDE OF LOADING LANE, BUT ALLOWS EVALUATION OF DETAIL 3 AS IF INTERIOR GIRDER ON BRIDGE WITH > 3 GIRDERS.)	STATIC: 28 K CYCLIC: 4 K TO 32 K
3	INDUCE MAXIMUM V300 = 33.5 KIPS (AT JOINT DETAIL 2) EVALUATE LOCAL DURABILITY OF JOINT DETAIL 1	STATIC: 28 K CYCLIC: 4 K TO 32 K

LOADING INFORMATION:

1. SINGLE WHEEL LOAD IS APPLIED OVER A TIRE CONTACT AREA 20" X 10"
2. DYNAMIC IMPACT FACTORS (1.75 FOR DECK JOINTS) ARE INCLUDED IN THE LOADING PLAN ABOVE.
3. MULTIPLE PRESENCE FACTORS (m) ARE NOT INCLUDED IN THIS INVESTIGATION.
3. PERMISSIBLE LOAD FREQUENCY: < 5 HZ (2 HZ COMMON IN SIMILAR TESTING)
4. PER WITHDRAWN ASTM STANDARD D6275 AND NCHRP REPORT 584 PROCEDURE: LOAD TO 2,000,000 CYCLES FOR EACH CASE.



1/S12 GENERAL LOADING PLAN
SCALE: 3/8" = 1'-0"



2/S12 LOAD CASE PLAN
SCALE: 3/8" = 1'-0"

NOTE: LOADING SHOWN ABOVE IS IN LOAD APPLICATION ZONE SHOWN IN DWG 1/S12.

DETAIL NUMBER	JOINT TYPE	DESIGNED FOR SHEAR	DESIGNED FOR + MOMENT	DESIGNED FOR − MOMENT
1	TRANSVERSE JOINT	X	X	X
2	LONGITUDINAL STAGED CONSTRUCTION			X
3	CONTINUOUS SHEAR POCKET			X

4/S12 FOR REFERENCE: DESIGN FORCE EFFECTS
SCALE: NTS

LOAD CASE	V200 (KIP)	M200 (KIP FT)	V300 (KIP)	M300 (KIP FT)
1	5.3	0	22.7	−41.8
2	−28.0	−42.0	−11.9	−11.0
3	−2.0	0	33.5	−16.0

NOTE: SHEAR FORCE IN TABLE 3/S12 REPRESENTS MAXIMUM ABSOLUTE SHEAR (EITHER IMMEDIATELY LEFT OR RIGHT OF REACTION POINT.)

3/S12 FOR REFERENCE: DETAILED LIVE LOAD FORCE EFFECTS
SCALE: NTS

General Notes

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LOADING/INSTRUMENTATION PLAN

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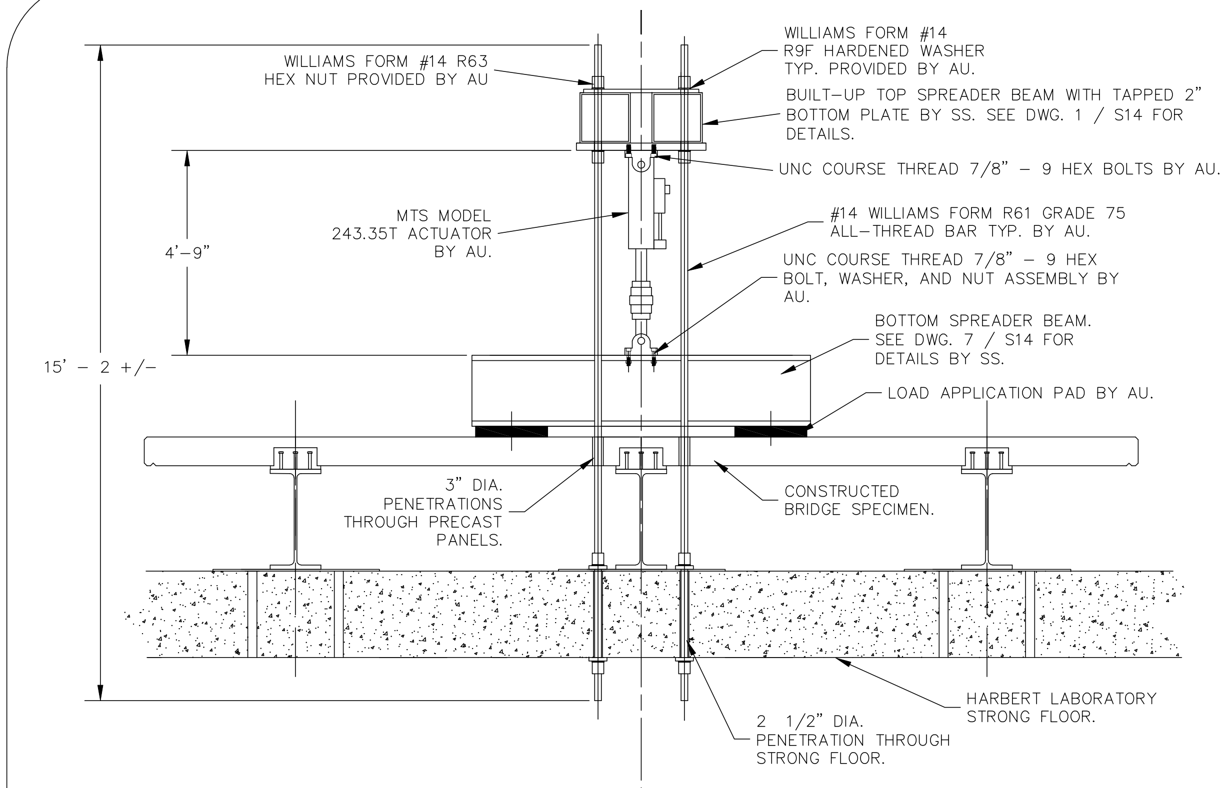
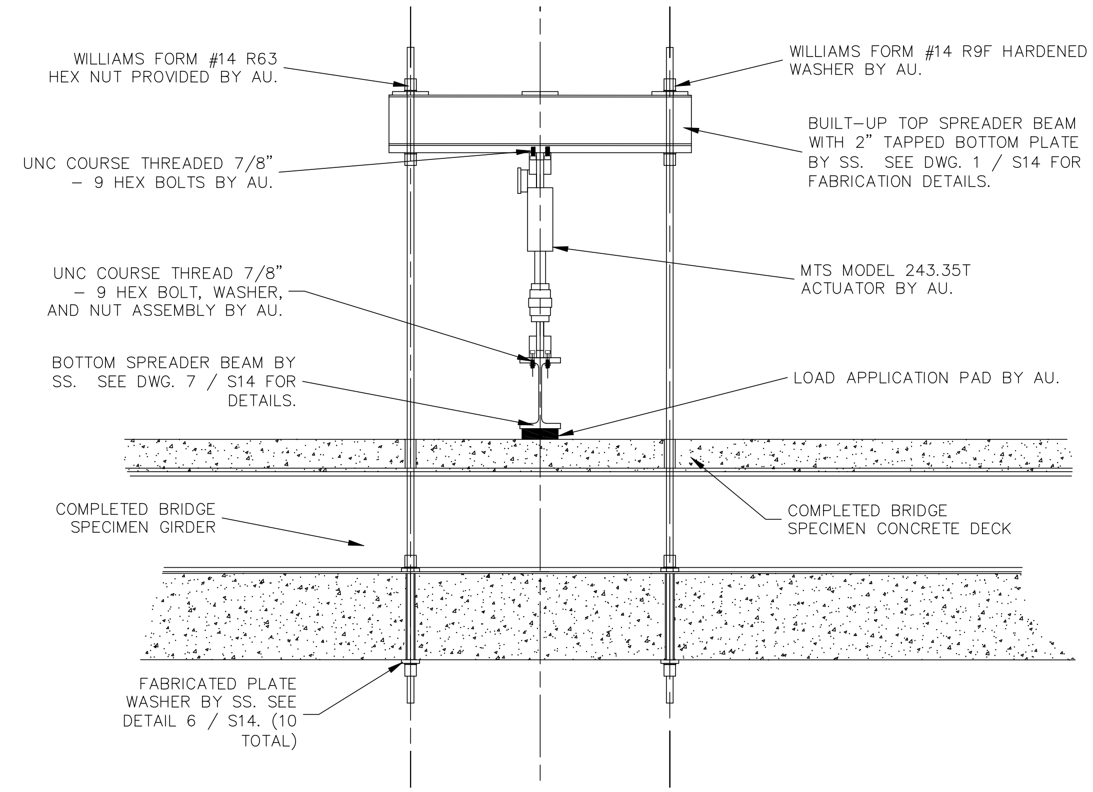
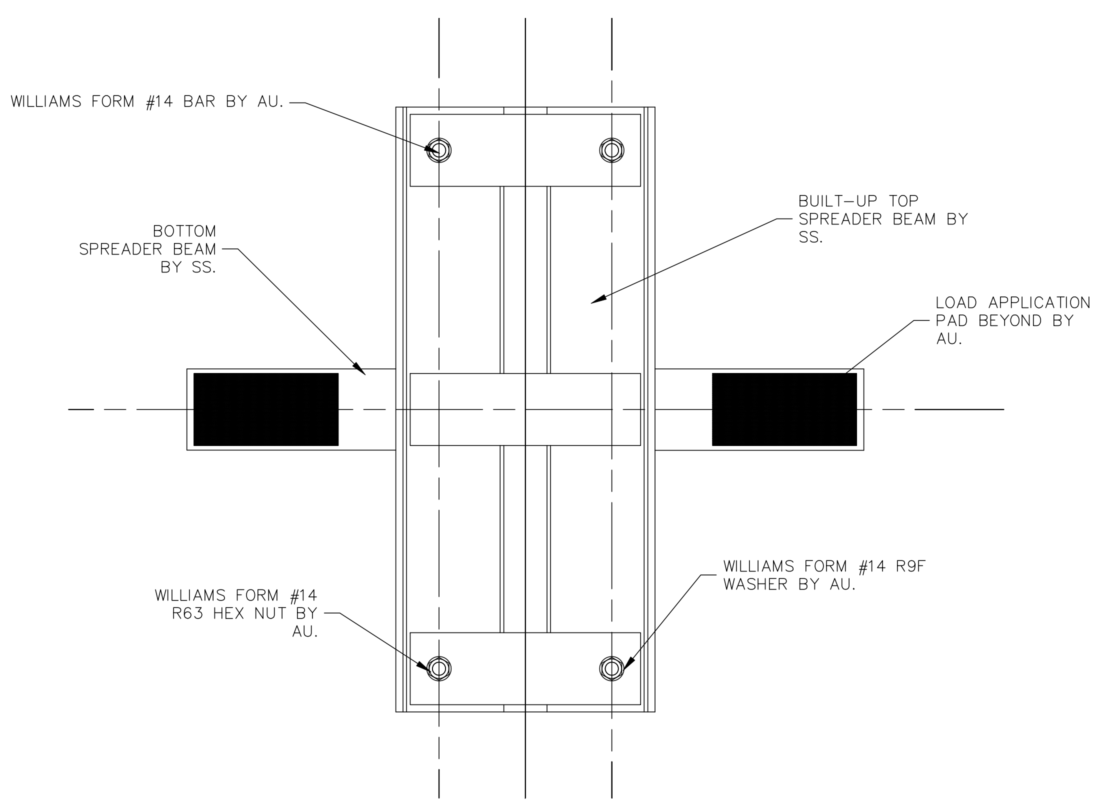


1/S13 ASSEMBLED LOAD FRAME ELEVATION: CASE 1
SCALE: 1/2" = 1'-0"



2/S13 ASSEMBLED LOAD FRAME ELEVATION: CASE 1
SCALE: 1/2" = 1'-0"



3/S13 ASSEMBLED LOAD FRAME PLAN: CASE 1
SCALE: 1" = 1'-0"

LOAD FRAME NOTES:

1. RELATIVE STIFFNESS OF SPECIMEN TO LOAD FRAME GOVERNED MEMBER SIZING. ANTICIPATED MAXIMUM DEFLECTION OF SPECIMEN UNDER LOADING SHOWN ON DWG. S12 = 0.03".

2. BUILT-UP TOP SPREADER BEAM 2" BOTTOM PLATE IS TAPPED FOR 3 DISTINCT ACTUATOR MOUNTING LOCATIONS. SEE DETAILS ON DWG. 4 / S14.

3. MTS MODEL 243.35T ACTUATOR HAS A TOTAL STROKE OF 10" AND IS REPRESENTED IN ALL DETAILS AT MID-STROKE (5").

4. SS PLEASE VERIFY BUILT-UP TOP SPREADER BEAM DOES NOT EXCEED 10,000 LBS.

5. LOAD FRAME MATERIAL PROPERTIES:

HSS SHAPES:	A500 GRADE B Fy = 42 KSI
STEEL PLATE:	A36 STEEL Fy = 36 KSI OR BETTER
WELDED CONNECTIONS:	5/8" PJP (FCAW-G) BTC-P10-GF WITH E70 WELD METAL.

6. DIMENSIONS FOR PENETRATIONS IN TOP SPREADER BEAM 2" PLATE ARE GIVEN FROM CENTERLINES TO ALLOW PLATE TO BE ROUGH TORCH CUT.

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General Notes

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Project Name and Address
LOAD FRAME DETAILS

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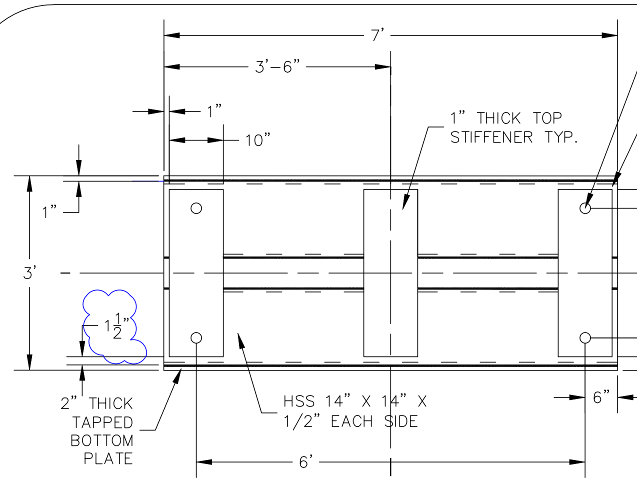
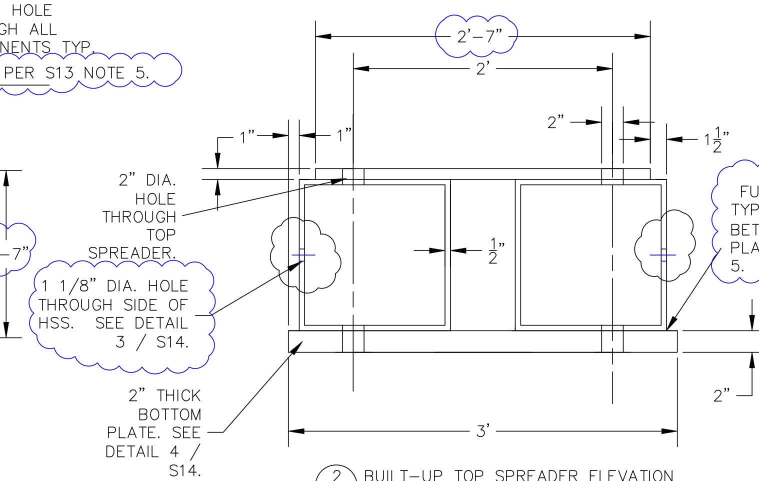
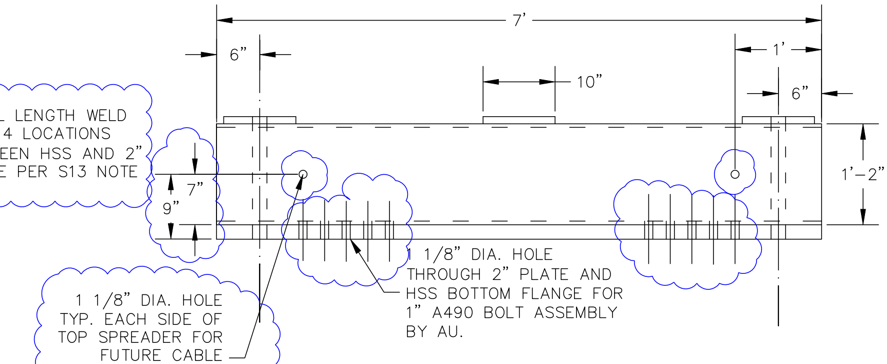


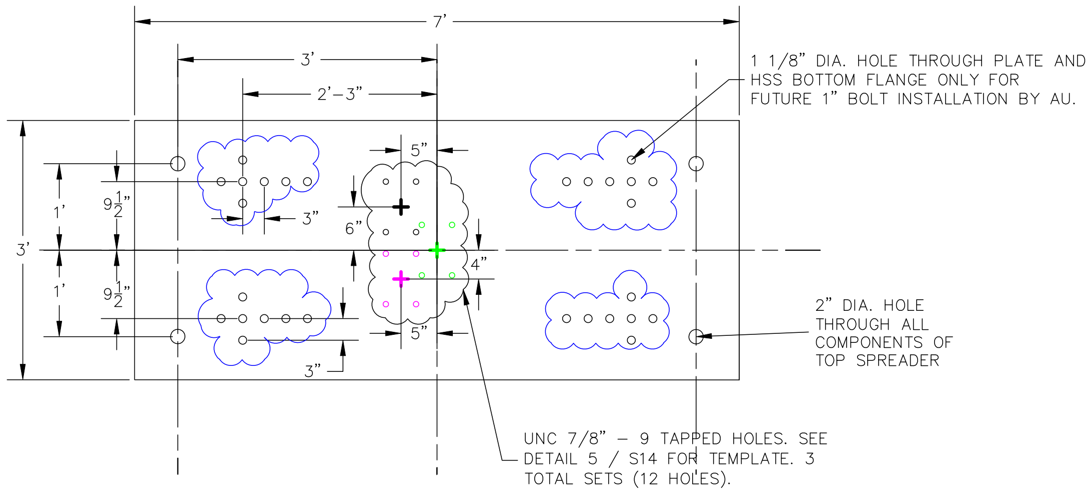
1 S14 BUILT-UP TOP SPREADER PLAN
SCALE: 3/4" = 1'-0"



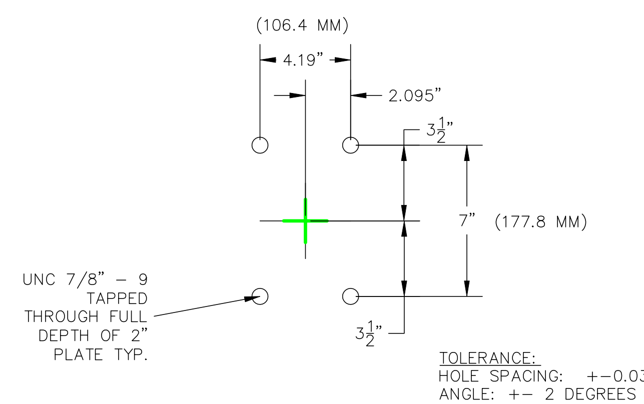
2 S14 BUILT-UP TOP SPREADER ELEVATION
SCALE: 1 1/2" = 1'-0"



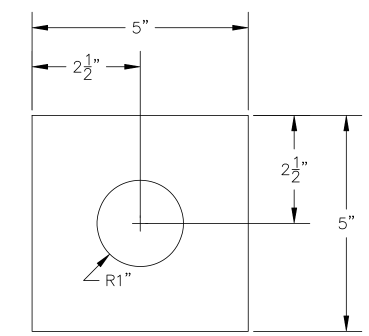
3 S14 BUILT-UP TOP SPREADER ELEVATION
SCALE: 1" = 1'-0"



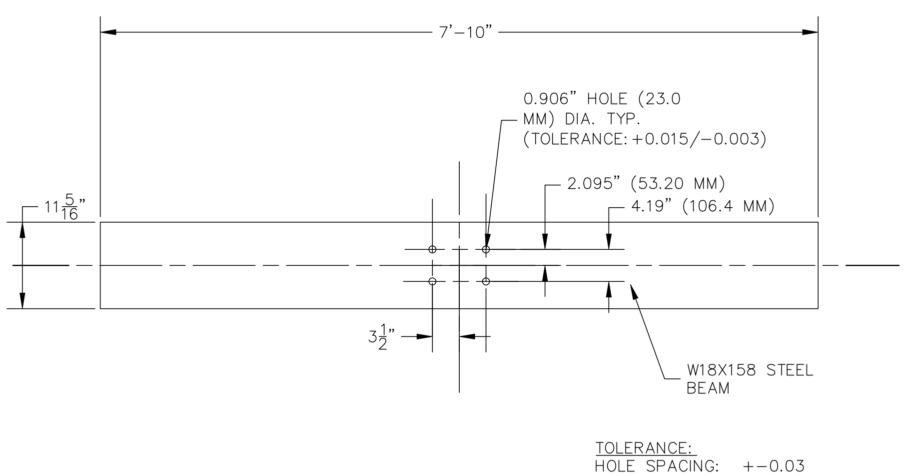
4 S14 BUILT-UP TOP SPREADER PLAN FROM BELOW
SCALE: 1" = 1'-0"



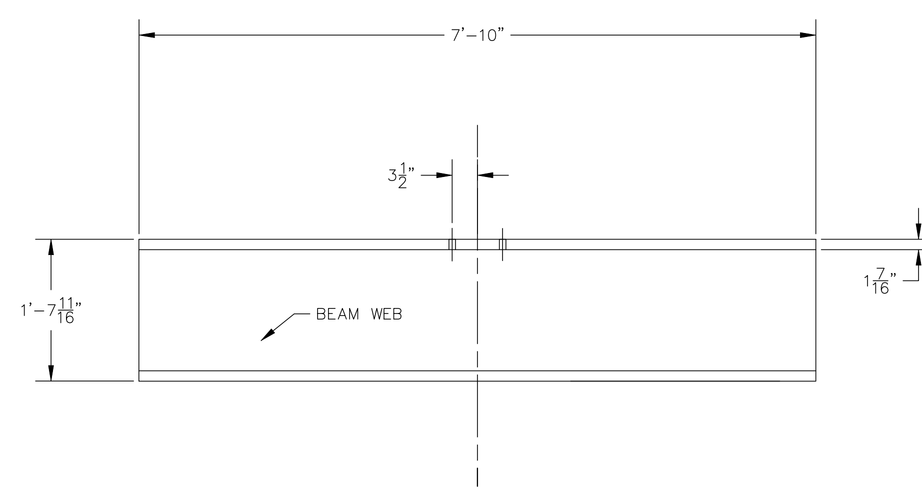
5 S14 2" PLATE TAPPING TEMPLATE
SCALE: 3" = 1'-0"



6 S14 1" THICK PLATE WASHER PLAN (10 TOTAL)
SCALE: 6" = 1'-0"



7 S14 BOTTOM SPREADER W18X158 BEAM PLAN
SCALE: 1" = 1'-0"



8 S14 BOTTOM SPREADER W18X158 ELEVATION
SCALE: 1" = 1'-0"

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Project Name and Address
LOAD FRAME STEEL
FABRICATION DETAILS

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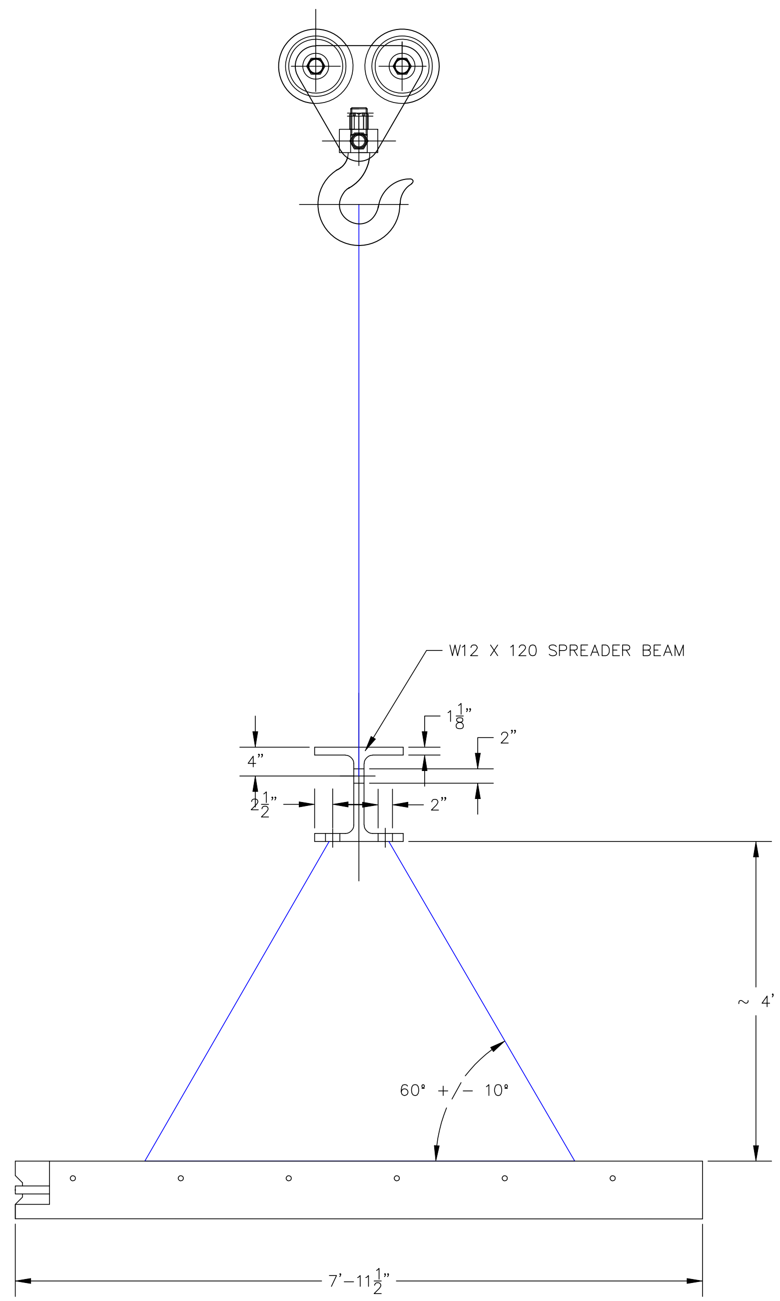
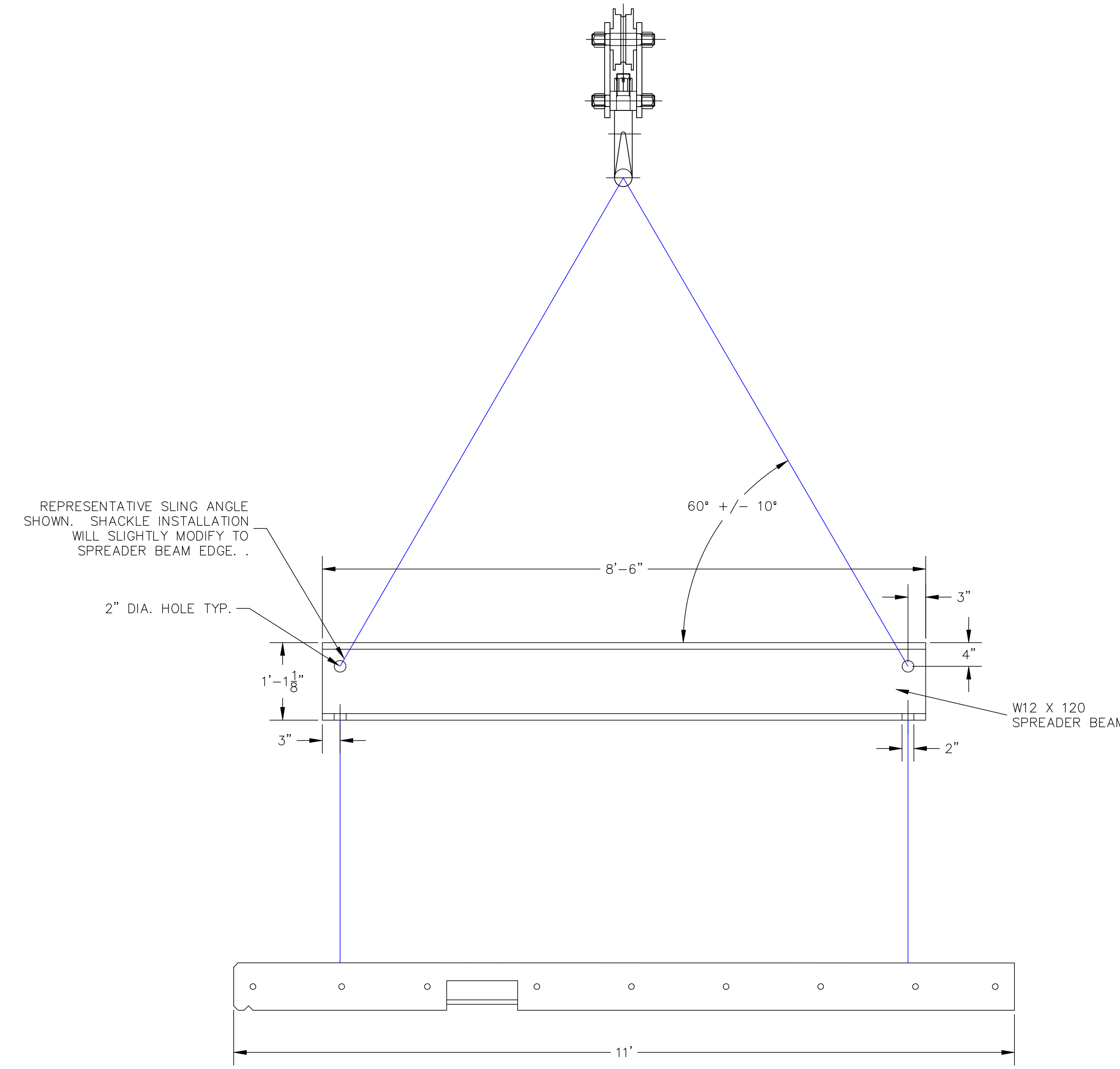


1 RIG RIGGING SPREADER BEAM
SCALE: 1" = 1'-0"



2 RIG RIGGING SPREADER BEAM
SCALE: 1" = 1'-0"

RIGGING NOTES:

1. THE ORIENTATION OF THE BEAM/SLAB AND SLING ANGLES AS SHOWN ABOVE ARE CRITICAL.
2. TOTAL SPREADER BEAM WEIGHT = 1020 #

   TOTAL ANTICIPATED PANEL WEIGHT (INCL. DYNAMIC ALLOWANCE) = 11,800 #
3. SUGGESTED LIFTING PLAN:

   RIGGING BEAM TOP HOLES INTENDED FOR SLING TO 3/4" SHACKLE OR GREATER WITH WORKING LOAD 8000# OR GREATER.

   RIGGING BEAM BOTTOM HOLES INTENDED FOR SLING TO 5/8" SHACKLE OR GREATER WITH WORKING LOAD 6000# OR GREATER OR AS HOLE GEOMETRY REQUIRES FOR CLEARANCE.

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RIGGING BEAM DETAILS

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Date 8/5/2011

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Sheet RIG