BRIDGE DECK OVERHANG CONSTRUCTION

by

Sean P. Clifton and Oguzhan Bayrak

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Bridge Deck Overhang Construction: Summary

An experimental investigation was conducted at the Ferguson Structural Engineering Laboratory at the University of Texas at Austin to investigate the current bridge deck overhang construction practice as is relates to the use of the new Texas I-girders (Tx girders). Due to the wide and thin top flange of the Tx-girders, the Texas Department of Transportation funded a testing program to have the performance and behavior of the current systems for overhang forming investigated. The primary objectives of this investigation were to examine the performance of commercially available overhang forming products and provide recommendations for their use with Tx Girders. In order to achieve the goals set forth, four Tx-girders were fabricated in the laboratory and a total of thirteen load tests were performed on the overhang forming products and Tx girders. Recommendations for the use of overhang forming systems for the Tx girders were based on the results of the experiments and synthesis of those results.

At the completion of the testing of the currently available systems for overhang forming, a new concept was developed to use a precast overhang as an alternate solution to create the finished bridge deck overhang construction. The solution involved precasting a portion of the overhang after the fabrication of pretensioned girders and using the precast portion of the overhang as stay-in-place formwork.

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CHAPTER 1
Introduction

1.1 BACKGROUND

During the development of a new family of pretensioned concrete I-girders, the Texas Department of Transportation funded a test program at the Phil M. Ferguson Structural Engineering Laboratory to address a concern. The beam design in relation to the current bridge deck construction practice was investigated in this project; more specifically the construction of the overhang on exterior (or fascia) girders. As depicted in Figure 1-1, the new I-girders (shown on the left) have a wider and thinner top flange than that of current I-beams such as the Type C beam shown in the same figure. The top flange of the exterior girders is used to attach overhang brackets to support formwork and thus a more slender flange could perform poorly under construction loads. To investigate this potential problem, the University of Texas at Austin and the Texas Department of Transportation entered into an interagency testing contact.
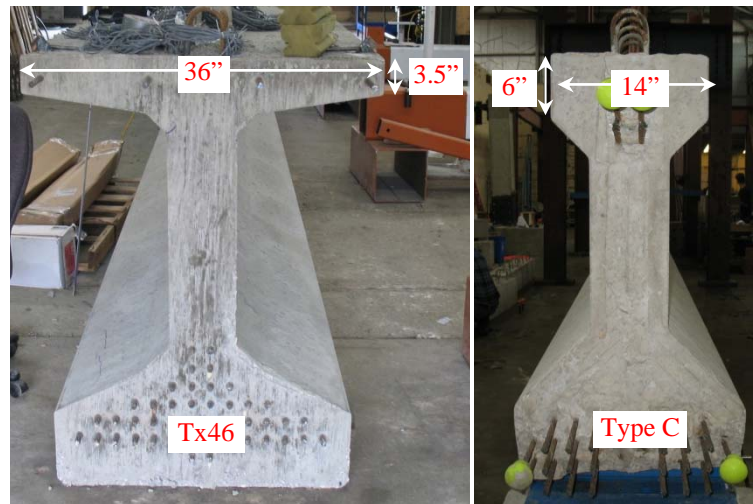
Figure 1-1: Comparison of new I-girder to Type C beam

1.2 OBJECTIVES

The primary goal of this project was to determine if a currently available commercial product could acceptably be used with the new pretensioned I-girder designs for use in the forming and construction of bridge deck overhangs in Texas. The Texas Department of Transportation was interested in an allowable

load recommendation for bracket systems used on Tx girders as well as any refinement to the Tx girder reinforcement pattern as it relates to overhang construction.

During the execution of the project, the investigators developed a new concept involving a precast stay-in-place overhang formwork solution for the construction of overhangs. With the development the new concept, a new objective was added to the project to determine the feasibility of the new construction technique for use in bridge deck construction.

1.3   SCOPE

In order to understand the behavior of current overhang bracket systems attached to the new TxDOT I-girders, thirteen tests were conducted. The experimental program consisted of the fabrication of four full scale test specimens in the laboratory followed by testing of the overhang systems. Multiple products from two overhang bracket and accessory manufacturers were tested as a representative sample of products being used in current construction practice.

Upon completion of the tests conducted on overhang bracket assembles, the new precast overhang construction concept developed by the investigating team was further evaluated. A full scale test specimen was fabricated at a precast beam manufacturing plant and tested to determine its functionality at the Phil M. Ferguson Laboratory.

1.4   PROGRAM OVERVIEW: CHAPTER OUTLINE

Chapter 2 includes a review of the current overhang construction practice as it relates to the project objectives. The various steps of bridge deck construction that relate to the overhang forming process are discussed in this chapter.

The testing program involving the overhang bracket systems and details of the different overhang forming systems evaluated, as well as the experimental methods used to evaluate the products are all discussed in Chapter 3. An explanation of the design of the test setup and instrumentation of the pretensioned I-girders is also included.

The results and observations from all 13 overhang bracket system tests are presented and discussed in Chapter 4. A comparative analysis of the results of the tests conducted on each bracket system is provided in this chapter. The failure mechanisms encountered in each test are illustrated with photographs throughout.

Final recommendations for the use of overhang bracket systems and their load rating on the new TxDOT girders are included at end of Chapter 4.

The newly developed precast overhang construction technique is explained in Chapter 5. A detailed description of the precast beam and the stay-in-place overhang fabrication and bridge deck construction is included. The experimental portion of the precast overhang study is also presented along with the test results and observations.

A summary of the experimental investigation and the resulting conclusions are provided in Chapter 6.

CHAPTER 2
Bridge Deck Overhang Construction

2.1 OVERVIEW

For many years concrete I-girder bridges in Texas and across the nation have been designed and built with a portion of the bridge deck spanning past the exterior beams. Figure 2-1 shows the bridge deck overhang on an I-beam bridge in Texas. This overhang is created to allow for the widest bridge using the fewest number of beams. The length of the overhang is typically 1.5 to 3 feet past the edge of the concrete beams but can reach over 5 feet in extreme situations when large horizontal curves are present along the span. In order to understand the objectives and the details of the experimental program, the various steps of bridge deck overhang construction are first explained. The current procedure for the construction of the bridge deck overhang is illustrated in this chapter.

Figure 2-1: Bridge Deck Overhang on Texas I-beam bridge

2.2 OVERHANG FORMWORK

In order to cast concrete, a system of formwork is necessary to shape the fluid concrete. The process of bridge deck forming has evolved significantly over the years. The process involves temporary plywood formwork, stay in place metal

deck formwork, and most recently stay in place precast prestressed concrete panels. Currently, typical bridge deck construction practices may use all of the aforementioned systems to form different parts of one bridge deck span.

One aspect of bridge deck forming that has not evolved much, is the method used for forming the bridge deck overhangs. While various commercial overhang formwork systems are available, the majority follow the same concept. The overhang formwork systems typically consist of plywood sheathing and timber joists supported on bridge overhang brackets. Figure 2-2 shows overhang brackets that are supporting plywood formwork at an I-beam bridge in Texas. The overhang brackets are attached to the concrete I-beam with a coil rod and hangers embedded in the top flange of the beam. Figure 2-3 shows two examples of commercially available embedded hangers. The overhang brackets are attached to the beam through the embedded hanger using a ½" coil rod that is threaded through the hanger and overhang bracket. A detailed view of that connection is shown in Figure 2-4.

Figure 2-2: Bridge overhang brackets supporting formwork
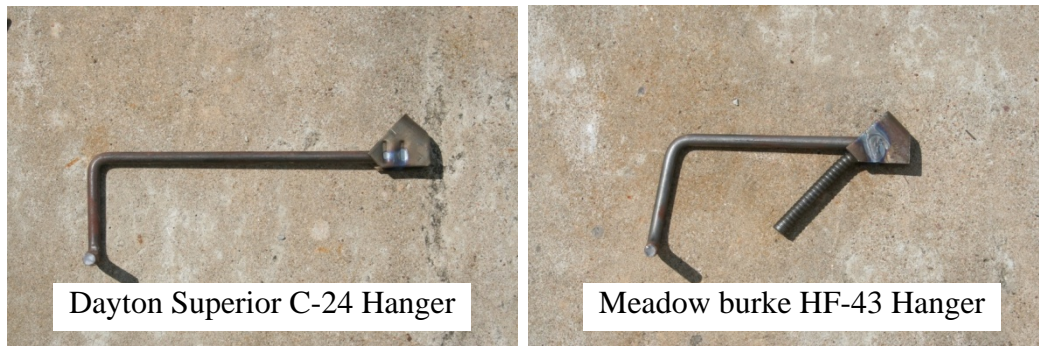
| Dayton Superior C-24 Hanger | Meadow burke HF-43 Hanger |

Figure 2-3: Embedded hangers used to support overhang brackets
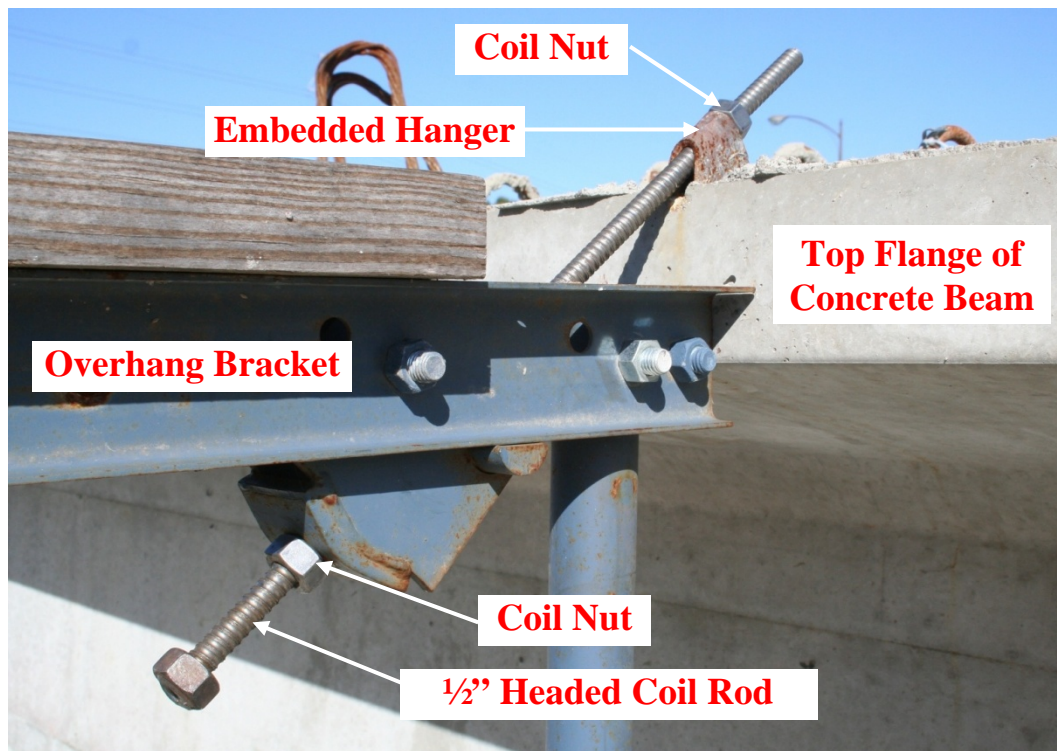
Figure 2-4: Connection of overhang bracket to concrete beam

After the overhang brackets are attached to the beam, the various pieces of timber that make up the formwork are placed on top of the brackets. The bottom side of the formwork is made up of a large piece of plywood sheathing supported on 2×4 or 4×4 joists running longitudinally across the overhang brackets. These pieces are set on a 2×6 that is already attached to the overhang brackets. The other integral piece of the formwork is the edge form that restrains the concrete from

flowing outward during the casting operation. The edge form typically consists of plywood supported by 2×4 studs and kickers. The cross section of the overhang formwork placed on top of the overhang bracket is illustrated in Figure 2-5. The underside of the formwork can be seen in Figure 2-6.
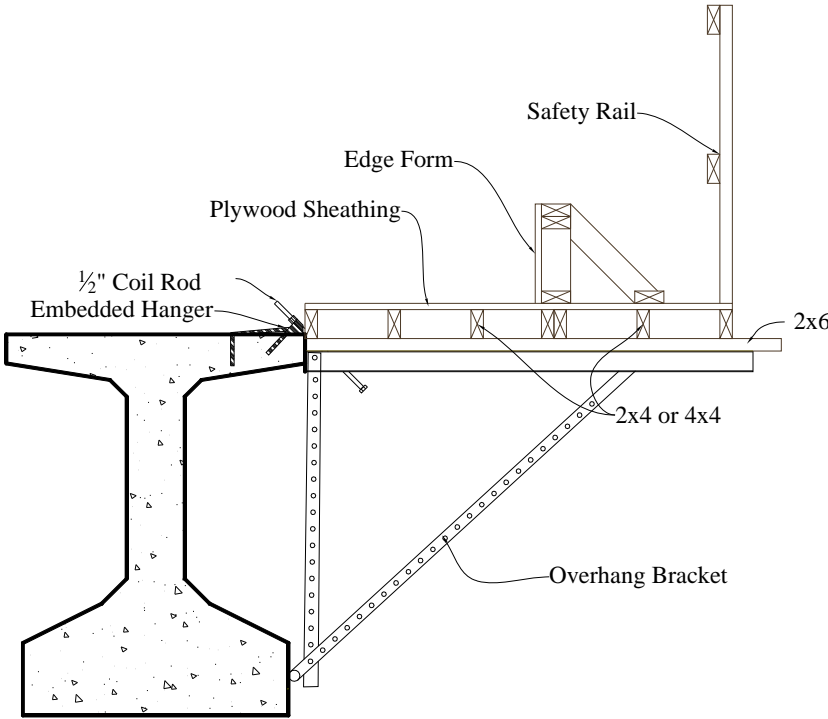
Figure 2-5: Cross section of overhang formwork on bracket
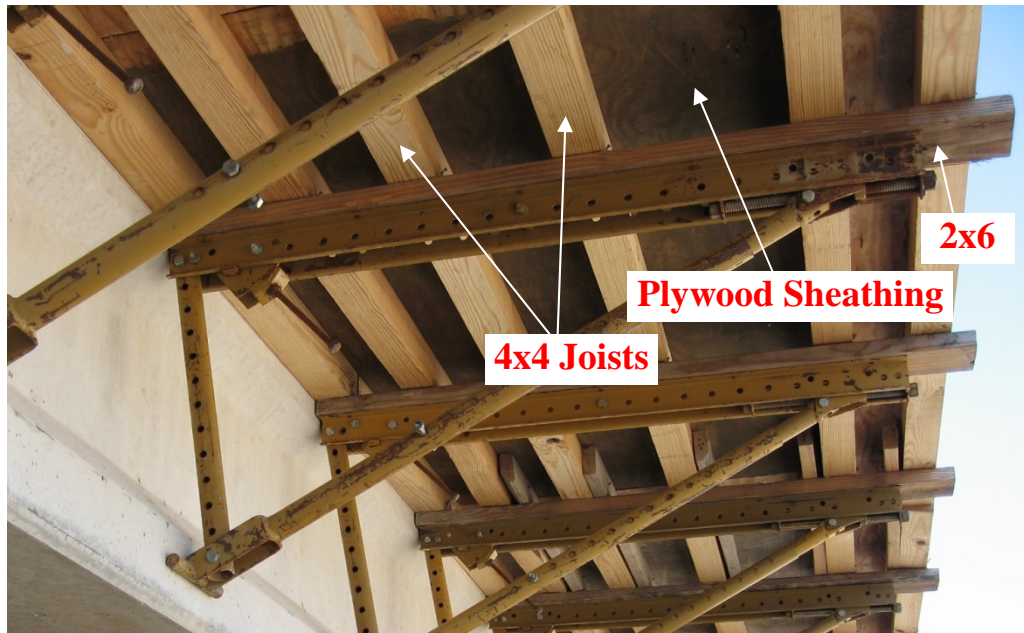
Figure 2-6: Underside of overhang formwork

Another function of the overhang brackets is to also support a work platform outside of the overhang. A platform is necessary to allow for workers to walk without interfering with the casting operation. Safety standards also require a fall protection barrier that is usually a wooden rail attached to the work platform. Figure 2-7 shows a work platform supported on overhang brackets just outside of the edge form. The safety rail is also shown in Figure 2-7.

Figure 2-7: Work platform and safety rail

Once the formwork is in place and assembled, sealant is applied to the joints. The mild steel reinforcing is then placed on the overhang and the rest of the bridge deck as prescribed in the standard detail as illustrated in Figure 2-8 and shown at a bridge construction site in Texas in Figure 2-9.

BAR TABLE		
Bar	Size	Spacing
A	#5	6" Max
B	#5	6" Max
D	#5	Shown
T	#4	9" Max
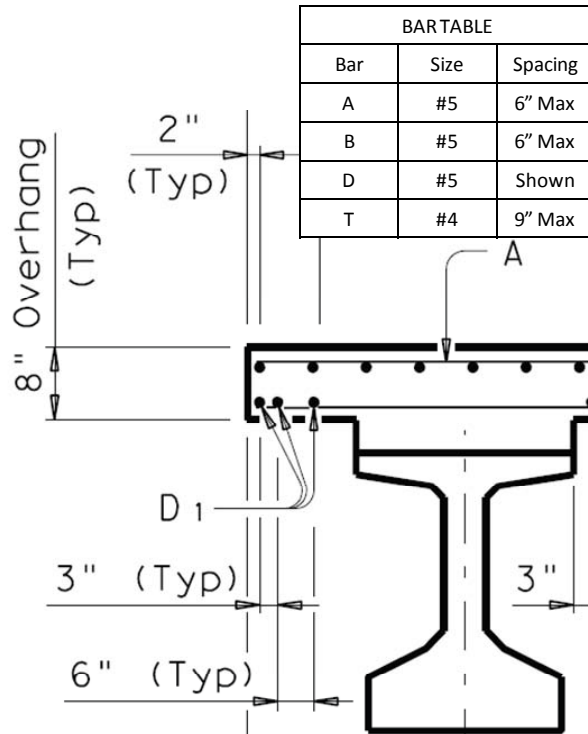
Figure 2-8: Typical TxDOT details for bridge deck overhang (TxDOT SIG Detail, 2007)

Figure 2-9: Mild steel reinforcing in bridge deck overhang

2.3 BRIDGE DECK FINISHING SCREED

A vital step in the casting phase of bridge deck construction is the screeding process. The screeding process involves spreading out the freshly poured concrete to a smooth finished surface. This is done through the use of a large mechanical finishing screed, also called a bridge paver. The finishing screed is a truss that spans the width of the bridge with a vibrating element that moves back and forth across the width of the bridge. A common finishing screed used in Texas is the Bid-Well 4800 and is shown in Figure 2-10. Important details of the Bid-Well 4800 are its standard operating weight (7,600 lbs) and the wheel base as shown in Figure 2-11 (Bid-Well, 2006).

Figure 2-10: Bid-Well 4800 Bridge Paver (Bid-Well, 2006)
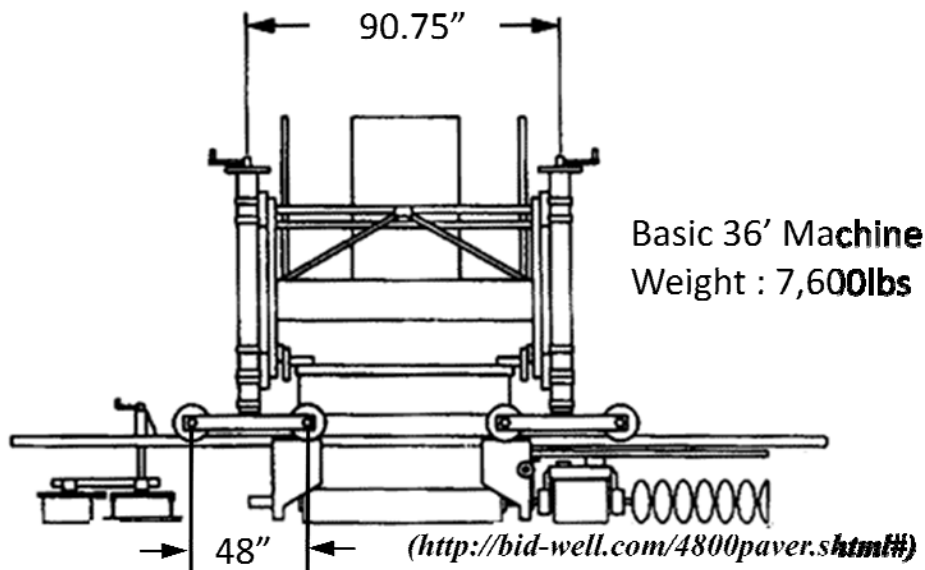
Figure 2-11: Bid-Well 4800 details (Bid-Well, 2006)

In order for the finishing screed to cover the largest amount of the bridge surface, the truss is supported at the extremities of the bridge width. The screed is thus supported just outside of the overhang. A small pipe known as the screed rail is used to support the screed and allow for movement along the bridge span. The screed rail is typically supported on a pipe holder that is placed on the edge form as shown in Figure 2-12. This method of supporting the screed allows for the height of the screed rail to be altered while the screed sits on the rail. The screed is of importance to the overhang forming system because the weight of the screed is often the largest component of the construction loads that are supported on the overhang brackets. The overhang brackets also support the fresh concrete, construction workers, formwork and other finishing equipment.
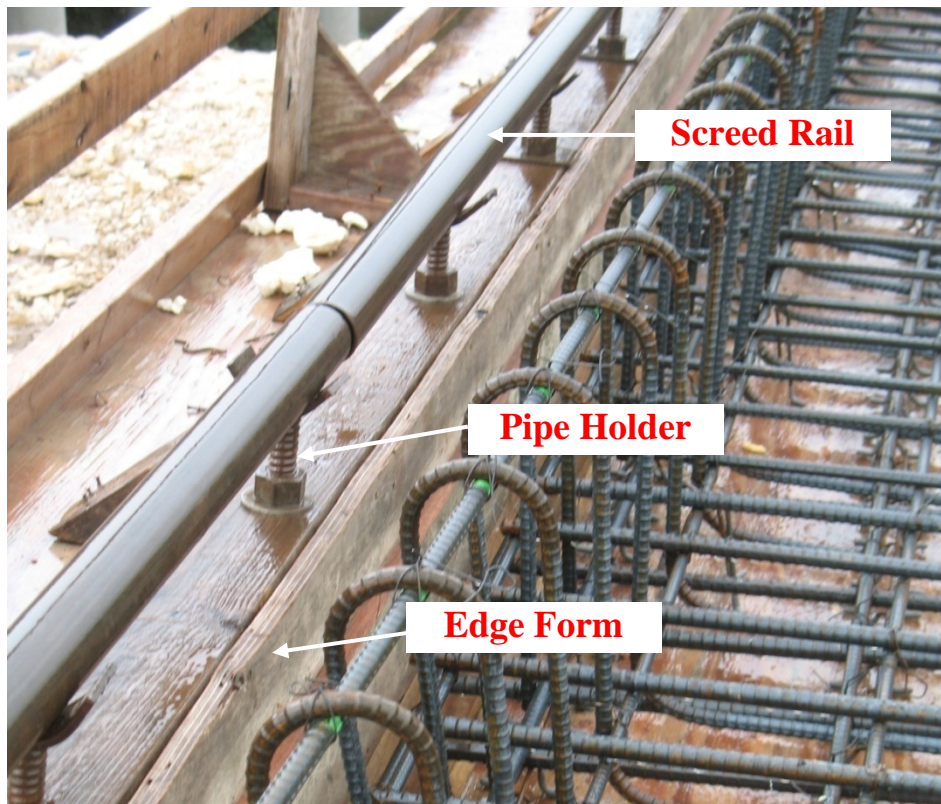
Figure 2-12: Screed supports

2.4 BRIDGE TRAFFIC RAIL

After the bridge deck and overhang are poured and finished using the screed, a second concrete pour is typically needed to create the traffic rail that is necessary. There exists many types of traffic rails and methods to create such rails. Among many others, a few common traffic rails in Texas include the Single Sloped Traffic Rail (SSTR), Concrete Parapet with Openings (T203), and the Heavy Truck Traffic Rail (HT) all shown in Figure 2-13. The bridge rails are formed with use of standard wood forming techniques or at times a slip forming technique is employed with the use of very low slump concrete. When wood forming is used, the overhang brackets are often left in place to support the rail formwork. However, when a slip forming technique is used, the overhang brackets are no longer needed and are removed before the rail is cast. Both wood forming the rail and using the slip forming technique are shown in Figure 2-14.

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Figure 2-13: Bridge Rails in Texas (Bridge Rail Guide 2005)

(http://www.mesalek.com/colo/steele/reconst.html)

Standard Wood Forms

(http://www.cyrusconcreteconstruction.com/walls.php

Slip Forms

Figure 2-14: Bridge rail forming techniques

2.5   OVERHANG BRACKET REMOVAL

When it is necessary, the overhang brackets must be removed from their position attached to the completed bridge deck. Access from below is often limited due to height and site geometry, therefore the removal process is usually completed through the use a cantilevered work buggy as shown in Figure 2-15. The coil rods shown in Figure 2-4 are typically lubricated prior to the casting of concrete so that they can easily be removed from the underside. The bracket removal process requires significant effort and can be very time consuming.

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Figure 2-15: Examples of bridge overhang buggies

2.6 SUMMARY

The creation of the bridge deck overhang is both a time consuming and important process. The critical steps include the overhang forming, placement of the screed rail, casting the bridge rail and removing the brackets. All the steps have importance to the experimental study presented in the subsequent chapters.

CHAPTER 3
Experimental Program

3.1 OVERVIEW

During the course of this project various overhang forming systems were tested. The overhang forming products tested and the development of the testing procedure are discussed in this chapter. All of the experimental work was completed at the Phil M. Ferguson Structural Engineering Laboratory of The University of Texas at Austin.

3.2 PRODUCTS

The commercial overhang forming products that were tested in the experimental potion of the research project are described in this section. In most cases the commercially available products were tested without any modification. In a few cases the commercial products were improved and/or modified. If the design of the commercial products was altered those products are noted as "*modified*".

3.2.1 Meadow Burke Products

Meadow Burke, based in Tampa, Florida, was one of the two product manufacturers involved in the project. Meadow Burke provided steel embedment hangers, coil rods, and overhang brackets. All nuts and washers were procured from other sources.

3.2.1.1 *HF-43 Precast Embed Hanger*

The HF-43 is the standard hanger that is placed during the casting operation of the precast beam. The hanger consists of a 0.44" diameter wire, a 0.5" formed coil rod, and a steel clip used to attach the connection coil rod.

Figure 3-1 shows a diagram from the Meadow Burke catalog of the HF-43 hanger as it is used with an I-beam. The catalog also includes the following disclaimer:

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Caution:

*Bulb Tee Special Notes:

...

- *Failure mode of HF-42/43-45° was punching shear of the flange at approx. 6000 lbs.*
- *Meadow Burke **does not recommend** use of HF-30,31,42, or 43 hanger on Bulb Tees with flange thickness less than 5 inches.*
- *Failure mode of HF-67**-45° was punching shear of the flange at approx. 12000 lbs.*
- ***Do not use** above hanger when Bulb Tee flange is less than 3 inches.*

(Meadow Burke Road & Bridge Catalog, 2006)

The catalog goes on to list the HF-43 to have a safe working load (SWL) of 7000 lbs for an I-beam and 3000 lbs for a Bulb Tee with 3" to 5" flange both with a safety factor of approximately 2:1 as shown in Figure 3-2. The SWL given is understood to be when load is applied directly along the attaching coil road at a 45° to the flange. Figure 3-3 shows the HF-43 embed hanger.

TOP VIEW

(HF-42) .375 DIA. DEFORMED WIRE
(HF-43) 0.440 DIA. MHC WIRE

2″

8″

Ø1/2 FORMED COIL ROD

4″

3″

45° OR 90°

HF-85 BRIDGE
OVERHANG BRACKET
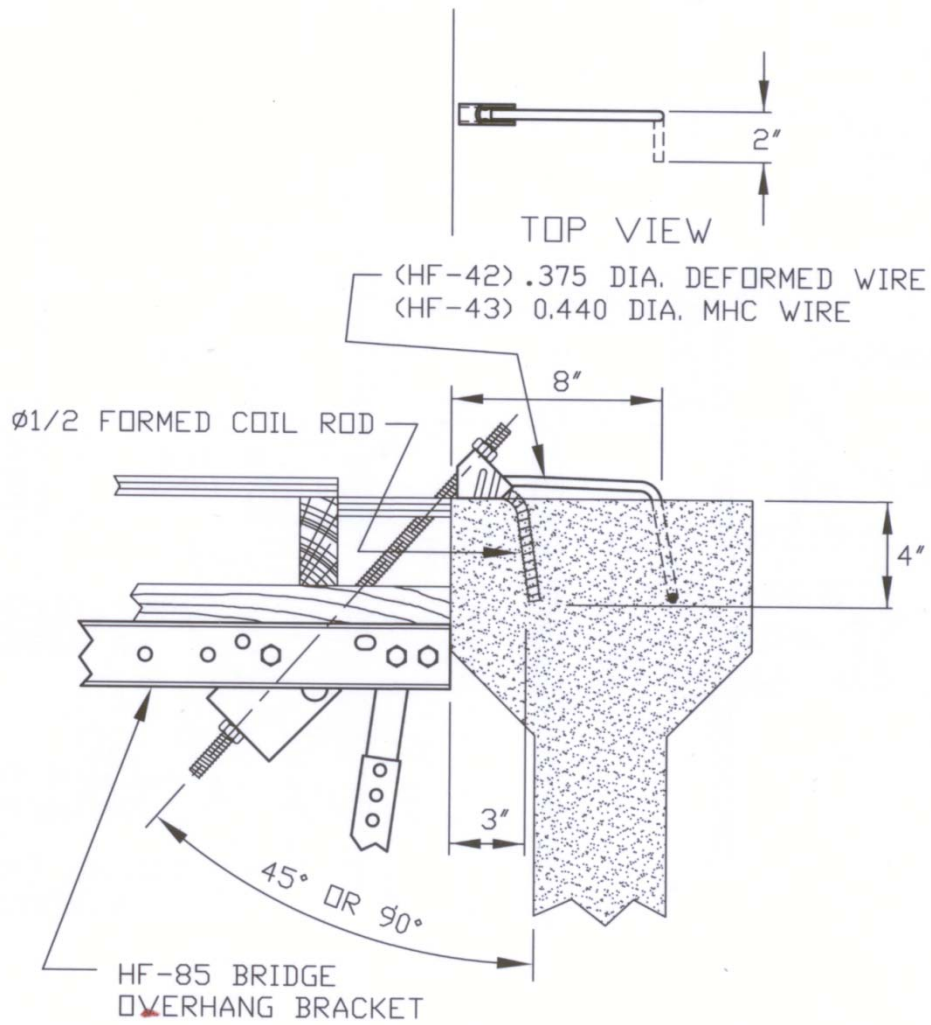
Figure 3-1: HF-43 Embed Hanger diagram (Meadow Burke Road & Bridge Catalog, 2006)

TYPE	ANGLE	NUMBER OF LEGS	I-BEAM (or similar) SWL (lbs.)	BULB TEE* 3" TO 5" Flg. SWL (lbs.)
HF-30	45°	1	3500	2500
HF-30	90°	1	3000	2000
HF-31	45°	1	4000	2750
HF-31	90°	1	3500	2000
HF-42	45°	2	5000	3000
HF-43	45°	2	7000	3000
HF-67**	45°	0	9000	6000

Safety Factor approximately 2:1
** HF-67 see page 43.

Figure 3-2: Safe Working Load Table(Meadow Burke Road & Bridge Catalog, 2006)

Figure 3-3: HF-43 Embed Hanger with angled coil rod

3.2.1.2   HF-67 Bulb Tee Bar Hanger

The HF-67 is a heavy duty hanger that is designed for use on beams with thin top flanges such as bulb tees. The primary feature that makes this hanger

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more effective with bulb tees is that the bearing plate at the tip of the hanger is moved slightly back from the edge of the flange in an attempt to engage a larger area of concrete. The other main feature of the HF-67 is that it is not installed in the casting operation. A ¾" diameter coil rod anchor is installed during the beam casting to attach the hanger to the beam; however, this anchor rod is also allowed to be post installed using epoxy grout giving the contractor flexibility at the construction site. Figure 3-4 shows a diagram of the HF-67 from the Meadow Burke product catalog. The catalog also notes a SWL of 6000 lbs again applied in the direction of the connecting coil rod as shown in Figure 3-2. Figure 3-5 is a photograph of the HF-67 Bulb Tee Bar Hanger.
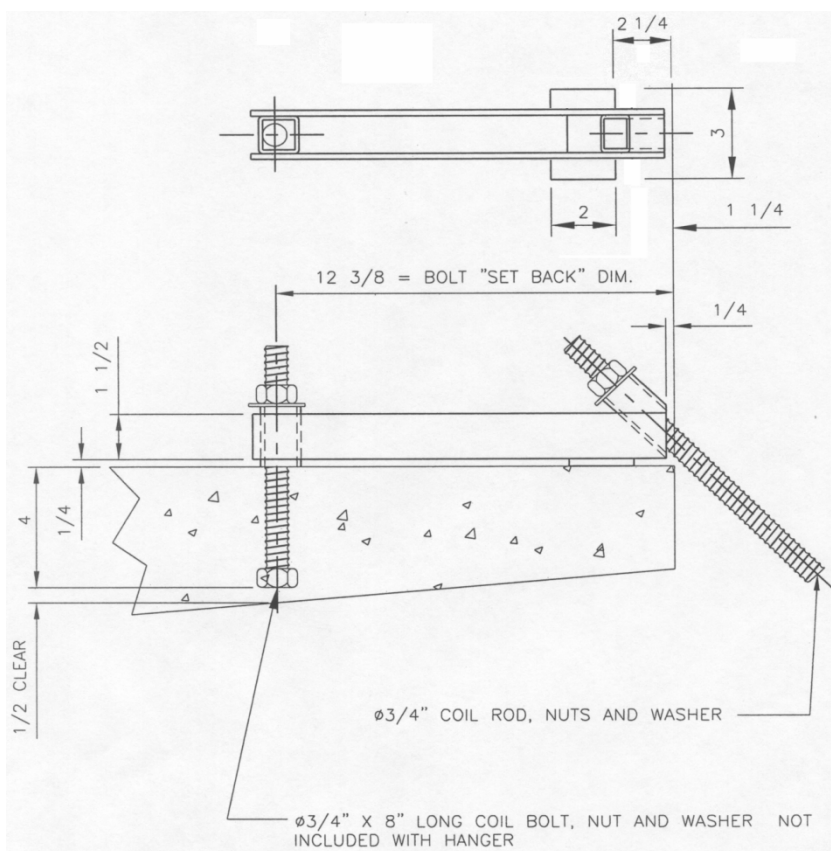
Figure 3-4: HF-67 Bulb Tee Bar Hanger (Meadow Burke Road & Bridge Catalog, 2006)

Figure 3-5: HF-67 Bulb Tee Bar Hanger

3.2.1.3   HF-67-M Bulb Tee Bar Hanger (Modified)

After a portion of the overhang tests were conducted at Ferguson Laboratory, Meadow Burke engineers revised their HF-67 Bulb Tee Bar Hanger design attempting to improve the performance when used on a thin flange. The hanger received numerous modifications including changing the shape of the hanger from two back to back plates to two back to back angles. The bearing plate was also increased and moved further back from the tip. Figure 3-6 shows the refined design of the HF-67 (Modified). Figure 3-7 is a photograph taken from the underside and shows the dimensions as compared with the original HF-67 design. It is important to note the change in location and size of the bearing plate on the bottom of the hanger.

Figure 3-6: HF-67 Modified Bulb Tee Hanger
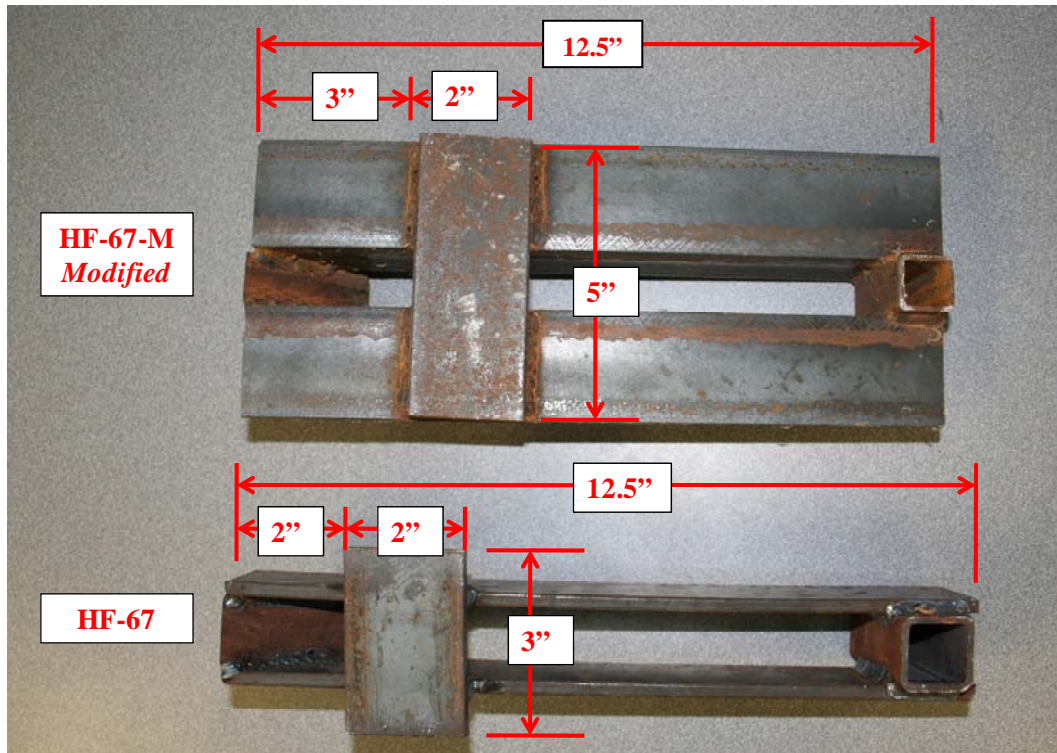
Figure 3-7: HF-67 Comparison of Bottom Side

3.2.1.4 HF-43-V Precast Embed Hanger (Modified)

The HF-43-V is a modification of the HF-43. The modification was limited to removing the ½" formed coil rod from the underside of the hanger and replacing it with two 0.44" diameter wires that stem out to the sides of the hanger. The thought process behind the modification was attempting to increase the area of concrete engaged by the hanger to increase the punching shear capacity. Figure 3-8 shows the HF-43-V hanger as it was modified by the manufacturer.

Figure 3-8: HF-43-V Modified Hanger

3.2.1.5 HF-43-I Precast Embed Hanger (Modified)

The HF-43-I received a similar modification to that of the HF-43-V in which the formed coil rod was removed and a wire was added in its place, this time in a hoop shape. In an attempt to engage a greater area of concrete in order to increase the punching shear capacity Meadow Burke engineers arrived at this modification. Figure 3-9 shows the additional wire added to the HF-43-I.

Figure 3-9: HF-43-I Modified Hanger

3.2.1.6 HF-86 Bridge Overhang Bracket

The other integral component of the forming system is the bridge overhang bracket. The primary Meadow Burke product tested was the HF-86 Bridge Overhang Bracket. The HF-86 consists of tubular and channel sections arranged to support the formwork and the additional loads of the casting operation. The HF-86 can be used on both concrete and steel beams. Figure 3-10 illustrates the overall set up of the HF-86 and the overhang formwork installed on a bulb tee concrete beam. Figure 3-11 includes a photograph of a HF-86 bracket that was tested in the project.
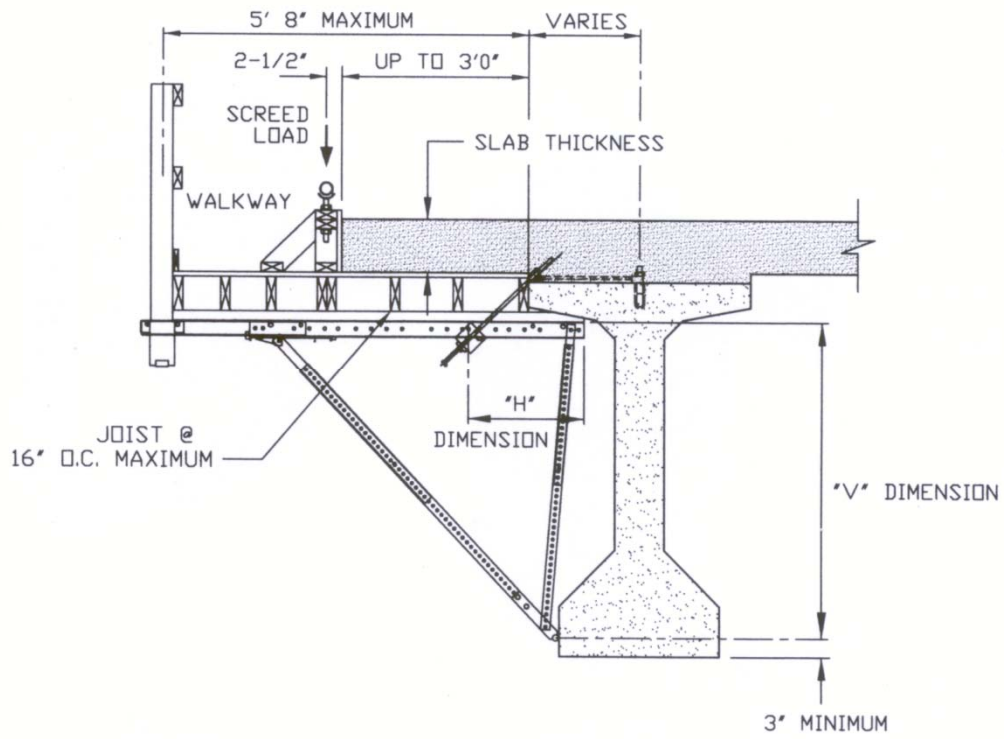
Figure 3-10: Diagram of HF-86 installed on bulb tee (Meadow Burke Road & Bridge Catalog, 2006)

Figure 3-11: HF-86 Bracket used in tests

3.2.1.7   HF-96 Aluminum Bridge Overhang Bracket

Meadow Burke also produces a heavy duty bracket for use with larger beams and heavier loads. The HF-96 Aluminum Bridge Overhang Bracket is a lightweight option that allows for increased spacing on longer overhangs. Figure 3-12 shows a HF-96 aluminum bracket that was used in the laboratory tests. The dimensions for the HF-96 are shown in Figure 3-13 taken from the product manual.

Figure 3-12: HF-96 Aluminum Bracket used in tests
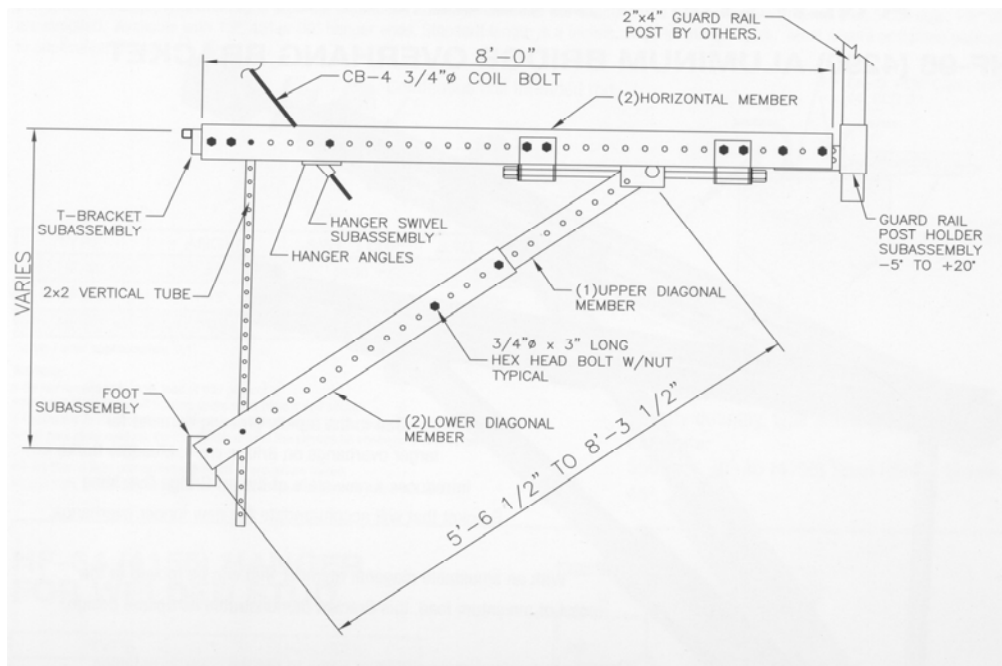
Figure 3-13: HF-96 Dimension drawing (Meadow Burke Road & Bridge Catalog, 2006)

3.2.2 Dayton Superior Products

In order to be comprehensive, overhang bracket systems produced by Dayton Superior were also used in the experimental program. Dayton Superior Concrete Accessories, a division of Dayton Superior, also provided overhang products to test with the Tx girders. Dayton Superior provided one type of hanger and one type of overhang bracket.

3.2.2.1 C-24 45° Pres-Steel Precast Half Hanger Type 4-APR

The C-24 Type 4-APR is the standard steel overhang hanger used in precast concrete applications. The hanger is placed in the top flange of the fascia girders during the prestressed concrete beam fabrication. The C-24 consists of a formed steel wire and end clip to hold the ½" coil rod. Figure 3-14 shows the C-24 Type 4-APR Hanger.

Figure 3-14: C-24 Type 4-APR Hanger

3.2.2.2 C-49 Bridge Overhang Bracket

Dayton's primary overhang bracket that is widely used in bridge construction is the C-49. The bracket is made of a light gauge steel pipe and channel sections. The C-49 can also be used on both steel and concrete beams and is attached with a coil rod passing through a hanger attached to the beam. Figure 3-16 shows one of the C-49 brackets used in the tests.
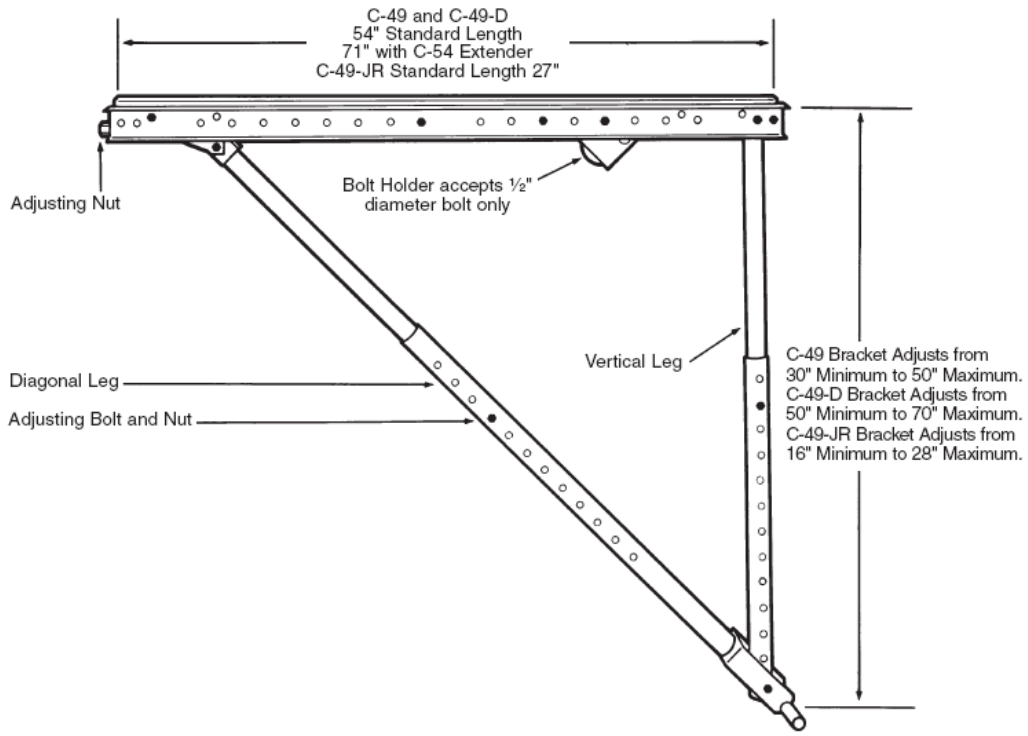
Figure 3-15: Diagram of C-49 Bridge Overhang Bracket(Dayton Superior Bridge Deck Product Handbook, 2007)

Figure 3-16: C-49 Bridge Overhang Bracket

3.3 LOADING CALCULATIONS ON OVERHANG SYSTEMS

In order to fully understand the construction loads supported by overhang bracket systems, it is valuable to review the loading cases and product manufacturer's guides on bracket spacing. The spacing of the overhang brackets along the length of the girder directly correlates to the load supported on each bracket. The design loads and their relation to the expected overhang bracket loads based on manufacturer guidelines are examined within this section of the report.

3.3.1 Meadow Burke Products

In the *Road and Bridge* catalog provided by Meadow Burke, there are numerous spacing tables for the various configurations of the overhang brackets. In order to determine the appropriate spacing of overhang brackets for a particular bridge site, a designer will enter the spacing tables with the overhang length, overhang concrete thickness, weight of the screed, and beam depth. Beneath each spacing table the design loads are summarized as follows in Table 3-1:

Table 3-1: Overhang Loads

Screed Load	3600	lb
Concrete Weight	150	lb/ft^3
Concrete Live Load	75	lb/ft^2
Platform Live Load	50	lb/ft^2
Platform Dead Load	10	lb/ft^2

These loads are an estimate of the actual construction loads. Using the loads from Table 3-1 and the bracket spacing tables provided in the product catalog, the gravity (vertical) load per bracket was calculated for all possible combinations of overhang length, thickness, screed weight, and beam depth shown in the catalog. The average gravity load on each bracket for all combinations shown in the spacing tables provided by Meadow Burke was approximately 4750 lbs with a maximum loading case of 5200 lbs.

In order to simulate field conditions, the design of the laboratory tests was based on typical parameters for bridge construction in Texas. The typical bridge deck overhang in Texas is 8 inches thick. While various overhang lengths exist for different beam depths and geometrical situations, the typical overhang is 3 ft, measured from the beam centerline. For the Tx girder, the resulting overhang supported on the overhang bracket is 18" and shown in Figure 3-17. For this case, the spacing tables give a vertical load of 4700 lbs.
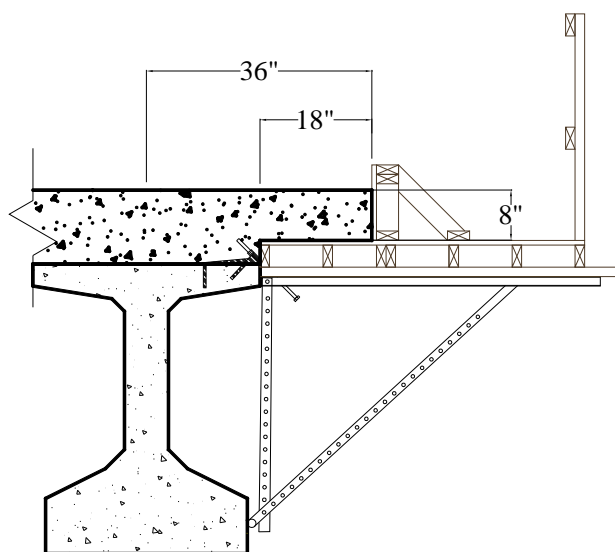
Figure 3-17: Typical overhang dimensions on Tx girder

It is also necessary to know the location of the centroid of the overhang loads in order to accurately design the experiments to simulate the construction loads. Calculations were performed using the same loads as Table 3-1 with the typical Texas bridge deck overhang characteristics. The computed average centroid was 18 inches from the edge of the top flange.

In summary, from the *Meadow Burke: Road and Bridge Product Catalog*, the overhang system of HF-86 Bridge Overhang Bracket used on a typical Texas bridge deck is rated to hold approximately 4700 pounds vertically with a centroid of load at 18" from edge of the beam top flange.

3.3.2   Dayton Superior Products

The *Bridge Deck Product Handbook* provided by Dayton Superior contains similar spacing tables for use with the C-49 Bridge Overhang Bracket.

Dayton Superior summarizes the loads with a screed load per bracket and a uniform design load in pounds per square foot. The uniform design load is noted to include a 50 psf live load on walk way area.

The spacing tables provided for the C-49 bracket are significantly more complicated than the tables for the Meadow Burke HF-86. Only two tables are given for precast concrete beams. The manual includes a warning on the bottom of every page recommending the reader to contact the technical service department if conditions vary from those shown.

The tables were interpreted for the typical conditions of an 8 inch thick 3 ft from centerline overhang. Using the C-24 Type 4-APR Hanger SWL of 4000 lbs, the average vertical load of 3500 lbs was calculated. The centroid of the load for the tests using the Dayton Superior products was assumed to be the same as the Meadow Burke products at 18" from the edge of the top flange. This assumption was made for two reasons: (i) construction loads including screed load, weight of the deck, and live load cannot be influenced by the bracket system (ii) to be able to compare test results in a consistent manner.

3.4   TEST SETUP

In order to test the new I-beams under simulated construction loads, a new test setup was designed and fabricated at Ferguson Structural Engineering Laboratory.

3.4.1   Design

The general design of the test setup was driven by a few constraints. First, the setup had to perform the intended function and allow vertical loading of the top flanges of the concrete girder through the overhang bracket systems. Second, in order to maximize the use of laboratory floor space, it was decided to build a self reacting frame due to location of the test setup. Third, the test setup needed to be simple and quick to assemble and disassemble in order to conduct the large number of tests desired.

Review of similar experimental testing conducted at the Constructed Facilities Laboratory at North Carolina State University, assisted in the development of a testing frame (Lackey, 2006). The loading frame used in the NC State tests involved a large reaction frame to resist the overturning load of the overhang brackets. For the testing at Ferguson Laboratory, a self reacting frame was of importance due to its simplicity. In order to create a stable self reacting

frame, both sides of the girder were loaded simultaneously to eliminate the overturning force that would be present if only one side were loaded. While the situation of simultaneously loading both sides with an overhang bracket system would certainly not exist in the field, the affect on the results was determined to be negligible because the area of concrete engaged by the bracket system typically does not extend past the center line of the girder. Figure 3-18 illustrates the final design of the test setup.

Figure 3-18: Rendering of cross section of test frame

In order to minimize the out of plane movement of the brackets and ensure stability, two brackets on each side of the beam were loaded simultaneously. This addition created a test region of four hanger/bracket systems for each test. Having a large number of systems for each test allowed for data collection from all four parts prior to the failure of one of the systems.

After further review, it was decided to conduct three tests on each 30 ft girder. With three tests per girder, the tests would not affect the neighboring test region yet are still large enough to be practical and resemble field conditions. The three separate test setups for one girder are shown in Figure 3-19.
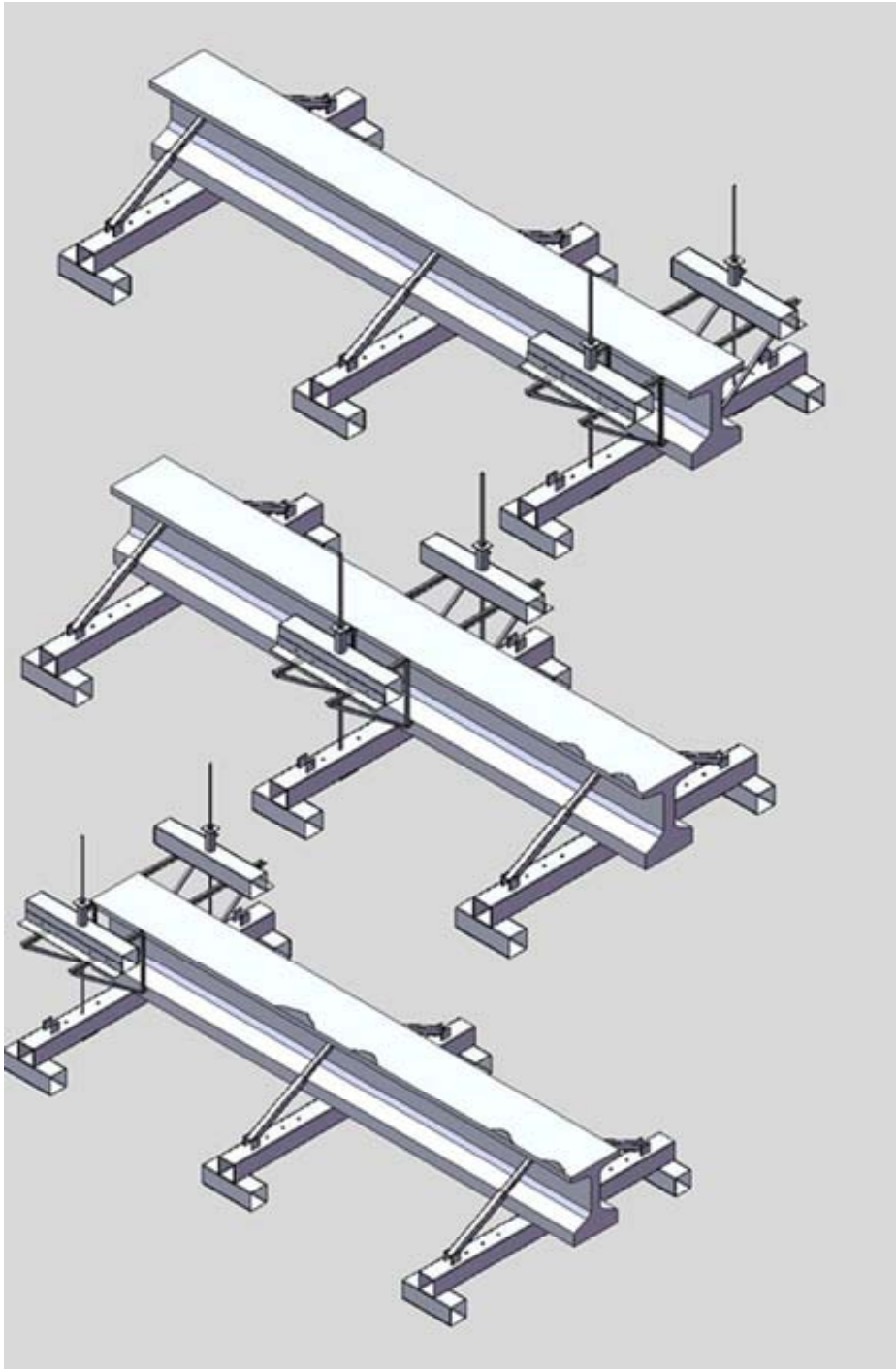
Figure 3-19: Three test setups for one beam

The Tx girders were supported on three 15' HSS 12x12x½" sections each placed at the center of one of the three test regions. The HSS sections were also supported off the ground by smaller 3-ft.-long HSS 12x12x¼" sections in order to elevate the beam and stabilize the frame. The other main steel sections used in the loading frame were two 6-ft.-long HSS 12x12x¼" sections that were placed on the overhang brackets parallel to the beam. These sections along with the large 15 foot sections supporting the beam were the primary loading members. The vertical load was applied to the brackets using 1½" threaded rods that were tensioned using hydraulic rams that reacted against the 6-ft.-long HSS section. As seen in Figure 3-18 the hydraulic rams put the loading rods into tension applying a downward force on the brackets and pulling up on the bottom support creating a self reacting frame. Additional tubular braces were added to the supports to brace the beam and prevent it from tipping in one direction when one side failed and significant torsional forces are applied to the beam. All the structural members were designed with a minimum factor of safety 3 to eliminate any possible damage to the test setup.

3.4.2   Construction

The construction process was straightforward due to the simple nature of the setup. The HSS sections were cut to length by the supplier and the only fabrication that was necessary was hole drilling, painting, and assembly. Approximately 116 holes were drilled in all the members and plates necessary for the setup. The frame was constructed so that the loading point could be changed for different systems. This flexibility allowed for the ability to test larger capacity systems where the centroid of the load is further from the beam edge. Figure 3-20 shows the entire test setup including a test specimen.

Figure 3-20: Completed Test Setup with Tx-28 in place

3.5 TxDOT New I-Girder sections

The Bridge Division at the Texas Department of Transportation developed the standards for the new family of I Girders tested in this research project. TxDOT saw a need for a new section type to improve upon the older beam cross sections that have been in use for many years across Texas and the country. The approach of the new "Tx" girder design was creating a more efficient shape. The Tx girder cross section has a very wide and thin top flange, slender web, and wider bottom flange to allow for use of more prestressing strands. TxDOT developed standards for seven beam cross sections ranging from 28" in depth to 70" in depth. For the tests at the laboratory, a range of specimens were fabricated consisting of two Tx28's, a Tx46, and a Tx70. Figure 3-21 clearly shows the cross-sectional dimensions and placement of mild reinforcing steel for the Tx girders that were fabricated. The transverse reinforcement in the overhang consists of No. 3 mild reinforcing bars spaced from 3" to 4" at the end regions and 12" in the center region. The specific reinforcing pattern for each test region is illustrated in the test results section.
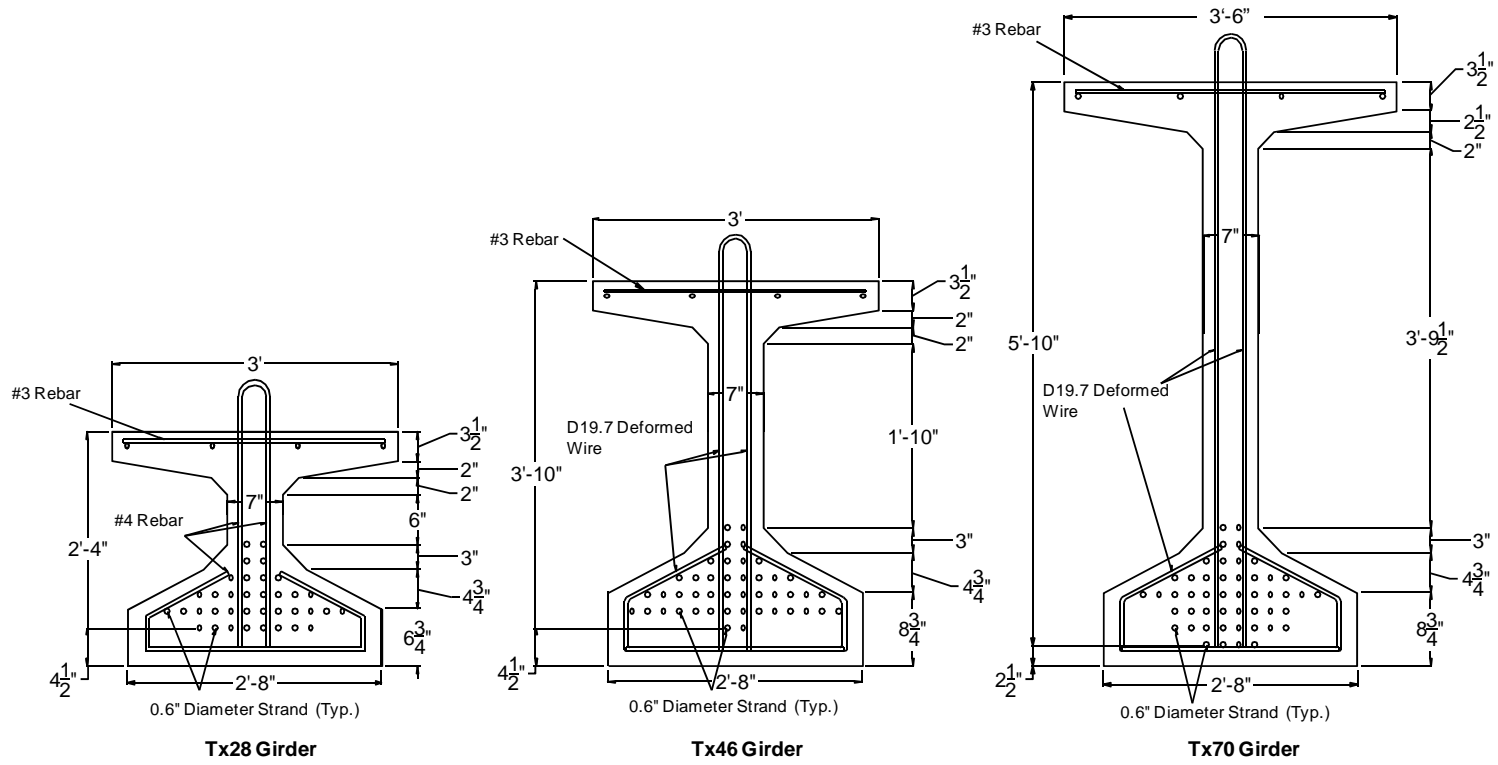
Figure 3-21: Tx Girder Cross Sections (O'Callaghan 2007)

3.6 INSTRUMENTATION

In order to accurately capture the behavior of the test specimens, a number of instruments were used in various locations and for different reasons. The main instruments used were strain gauges placed on mild reinforcing steel, load cells, and linear potentiometers.

3.6.1 Load cells

For the tests conducted on the overhang systems, an important measurement was the vertical load being applied to the system. In order to capture this load accurately, 100 kip load cells manufactured by Interface (Figure 3-22) was placed underneath the loading ram and reacting against the cross member that transferred the load to the overhang brackets as shown in Figure 3-23. Figure 3-22 shows one of the two 100 kip load cells that were used in the overhang tests.

Figure 3-22: 100k Load Cell from Interface

Figure 3-23: 100k Load Cell shown in place

Low profile load cells or "pressure washers" produced by Houston Scientific were also used in some of the tests. These low profile load cells were used on the 45° coil rods to record the axial load in the rod and applied load on the hanger. However, it is important to recognize that the load readings obtained from pressure washers are not as accurate as load cell readings and are likely only accurate to within approximately 4-7%. The pressure washers were used in six of the thirteen tests in an attempt to characterize the relationship between vertical load and hanger load. Figure 3-24 shows two of the four pressure washers that were used in the overhang tests. The pressure washers were positioned around the end of the coil rods that connect the hanger to the bracket and between two flat plate washers as shown in Figure 3-25.

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Figure 3-24: Pressure Washers used in tests

Figure 3-25: Position of pressure washer on hanger

3.6.2 Linear Potentiometers

Linear Potentiometers were also used to measure vertical displacements in many of the tests. These potentiometers were placed at various points for different tests. They were placed underneath the tip of the concrete flange to measure the tip deflection as load was applied as shown in Figure 3-26. The potentiometers were also placed at the tip of the bracket to monitor the bracket tip deflection with increasing loads as shown in Figure 3-27.

Figure 3-26: Linear Potentiometer under flange
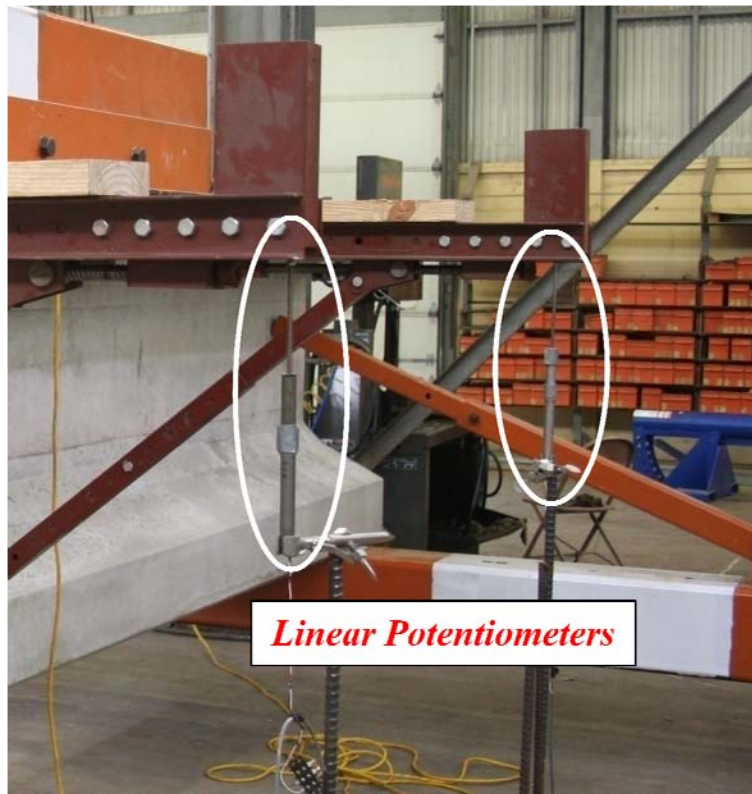
Figure 3-27: Linear Potentiometers under bracket tip

3.6.3 Strain Gauge Installation on Reinforcing Bars

Fabricating the beams in the laboratory allowed for extensive instrumentation of the mild steel reinforcing bars that were placed in the beam. For the overhang tests, the reinforcing bars of interest were the top flange transverse reinforcing bars. The strain gauges used were TML Strain Gauges from Tokyo Sokki Kenkyujo Co., Ltd., Type FLA-5-11-5LT, with a 5mm gauge length.

3.6.3.1 Strain Gauge Locations: Transverse Reinforcing Bars in Top Flange

The location of the strain gauges on the reinforcing bars was selected based on the hypothesized failure planes. Being the first beam cast and tested, the Tx-28-I was heavily instrumented. **Error! Not a valid bookmark self-reference.** shows the locations of the reinforcement, strain gauges, and hanger placement for

the HF-43 on the Tx-28-I. Diagrams for all 13 of the test locations were excluded from this chapter to avoid repetition yet are included in Appendix A for complete reference. Specifics of the test regions and girders are also included in the Appendix A.
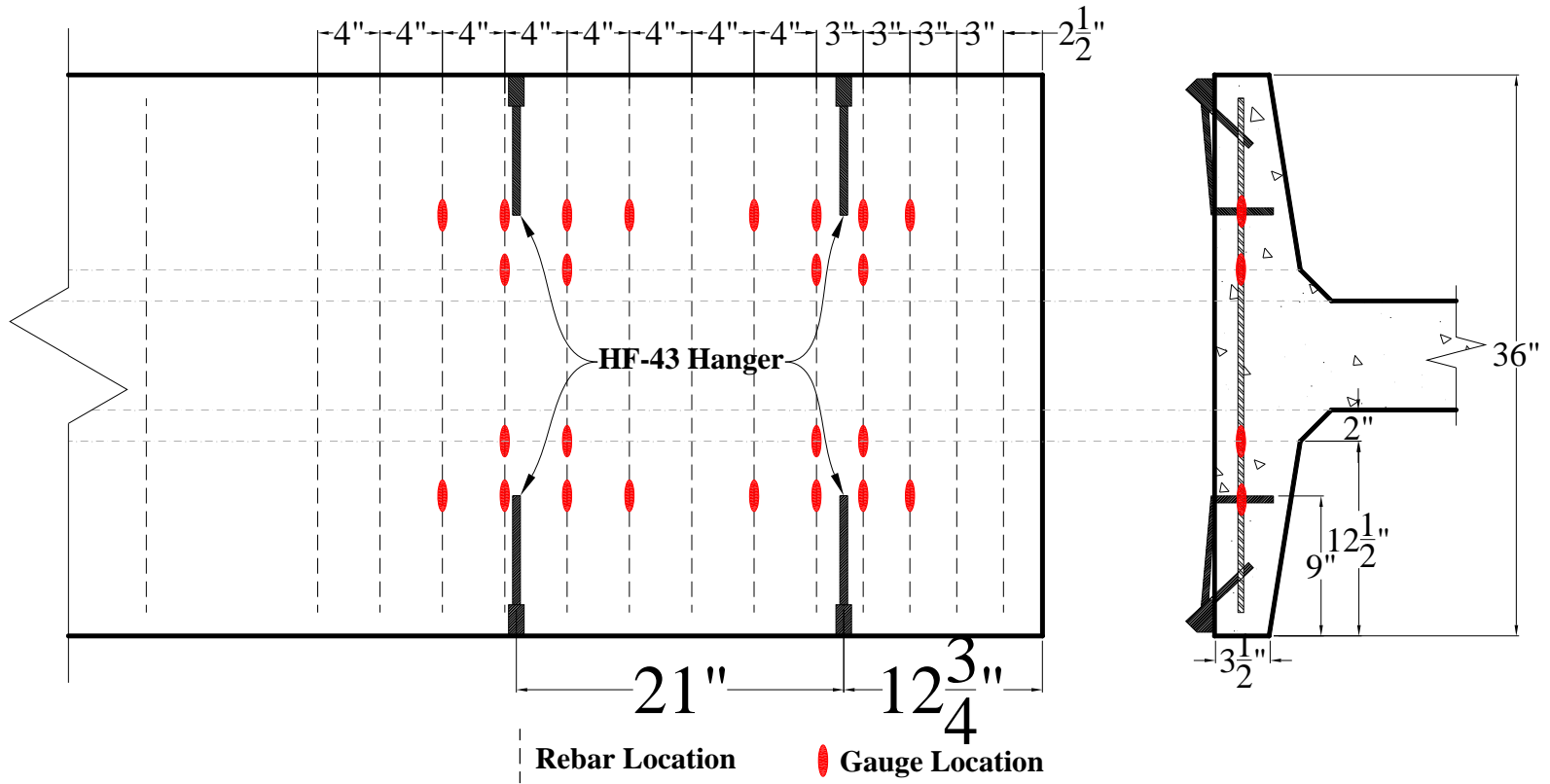
Figure 3-28: Locations of reinforcement, strain gauges and hangers for HF-43: Tx-28-I

CHAPTER 4
Test Results and Discussion

4.1 INTRODUCTION

Four girders ranging in depth from 28" to 70" were fabricated, instrumented, and tested at Phil M. Ferguson Structural Engineering Laboratory. A total of thirteen load tests were performed on the various overhang bracket systems outlined in Chapter 3. While the geometry of the overhang bracket systems varied from one test to another, the overall test procedure remained consistent throughout.

4.2 BRACKET-HANGER SYSTEM LOADS

In order to understand the effect of different geometries of the bracket configurations and girder sizes on the loads introduced into the top flange of the girder, the pertinent primary characteristics are discussed in this section. Figure 4-1 illustrates the locations of the main load components of the system. Each load component is defined as follows:

- Applied Vertical Load: The primary load applied into the system. In the field the load consists of the finishing screed, fresh concrete dead load, live load, and formwork dead load. In the laboratory the load is applied vertically with a hydraulic ram.
- Kicker Load: The load applied horizontally to the concrete girder at the support location of the bracket diagonal member.
- Hanger Load: The load in the 45° coil rod that is induced by the vertical load and kicker load and transferred to the top flange of the concrete girder.
- Reaction: With certain bracket geometries, the top chord of the bracket bears into the top flange of the concrete girder and applies a horizontal reaction load. When the top chord of the bracket pulls away from the top flange, this load component does not exist.
- Idealized Friction Load: Frictional resistance induced by the kicker load at the support location. Further discussion regarding the frictional resistance follows in the next section.

Additional sources of load not accounted for are the lateral concrete pressure and dynamic effects of the finishing screed. These load sources are assumed to be negligible in comparison to the aforementioned loads.
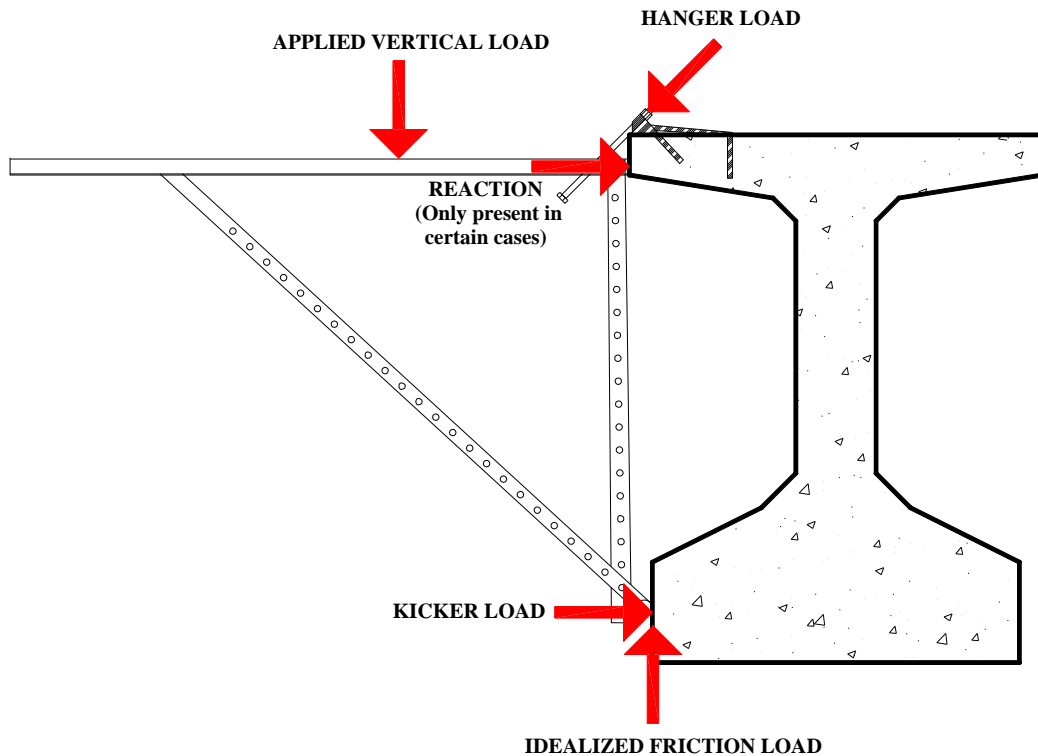
Figure 4-1: Loads in overhang bracket system

4.2.1 Frictional Resistance

The presence of friction in most structural applications is typically neglected; however, in the context of this experimental investigation, the potential impact of frictional resistance should be discussed further. First, the presence of this resistance during the experimental tests was clear due to the significant portion of the paint from the bracket support that was left on the concrete girder as shown in Figure 4-2. The magnitude of the resistance is a function of the horizontal kicker load and the coefficient of friction. To develop relationships between systems with and without friction, several models using differing geometries were created in the structural analysis software SAP 2000. Using a coefficient of friction of 0.5 (Baltay & Gjelsvik, 1990) and an iterative analysis procedure, the resulting hanger loads differed between 15% - 30%. The hanger

loads were decreased if friction was included in the model. The difference is useful in context of the test results; however, during bridge deck construction the friction between the bracket and girder should not be counted on due to the vibrating screed above and the unreliable nature of friction.

The load transferred to the girder top flange can be characterized by the relationship between applied vertical load and the load induced into the coil rod and transferred to the hanger or "hanger load". Using SAP 2000, the ratio of hanger load to applied vertical load was computed including friction and neglecting friction and the results are displayed in Table 4-1. Data from the use pressure washers on the Tx-28-I and Tx-28-II, show very similar results to the modeled load ratio from SAP 2000. In three of four tests using the same geometry, the load ratio ranged from 0.92-0.95, only a few percent different from 0.96 from the model. Based on the similarity between the actual data and modeled results for the Tx-28, the other two calculated load ratios for the Tx-46 and Tx-70 can be used with higher confidence.

Figure 4-2: Rub marks on bottom flange due to friction

Table 4-1: Load ratios from SAP 2000 for HF-86 Bracket Configurations

Girder	Hanger Load / Vertical Load Including Friction	Hanger Load / Vertical Load Neglecting Friction
Tx-28	0.96	1.41
Tx-46	1.19	1.41
Tx-70	1.17	1.41

4.2.2 Hanger Forces

The experimental study focuses on the connection detail between the bracket and the top flange of the prestressed concrete girder. Although the hangers tested all differ in some fashion, the main load transfer mechanism remains the same. All systems consist of a hanger load applied through the coil rod which results in vertical and horizontal components as depicted in Figure 4-3. The vertical component of the load creates a shear and bending moment at the critical section. The horizontal component applies a tension force on the top flange at the embedded portion of the hanger. A free body diagram of the top flange showing the forces at the critical plane is illustrated in Figure 4-4.
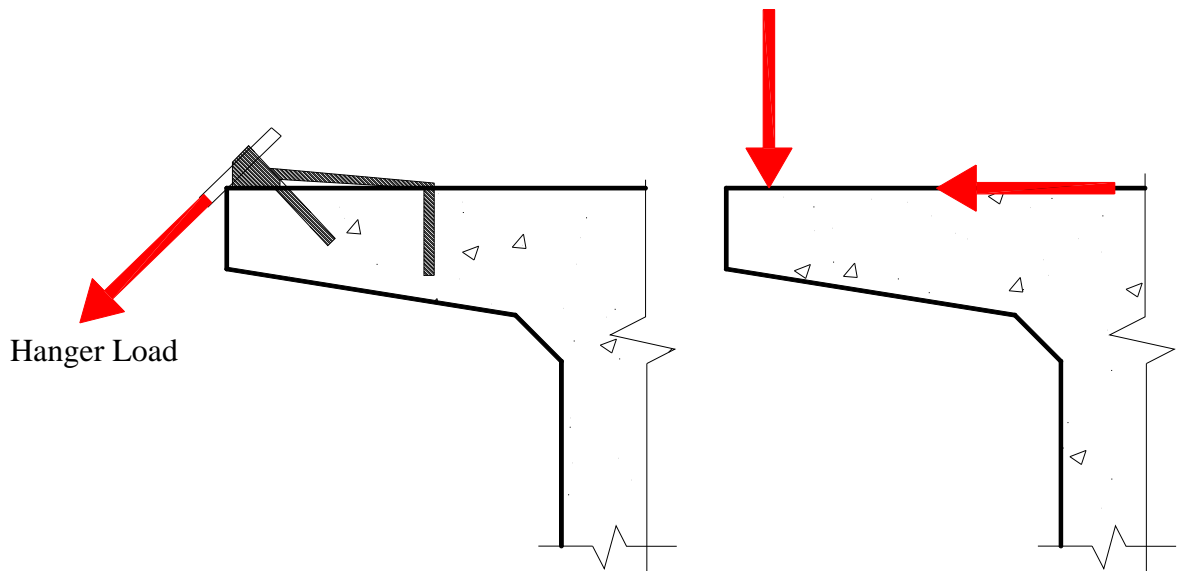
Hanger Load

Figure 4-3: Hanger load and components
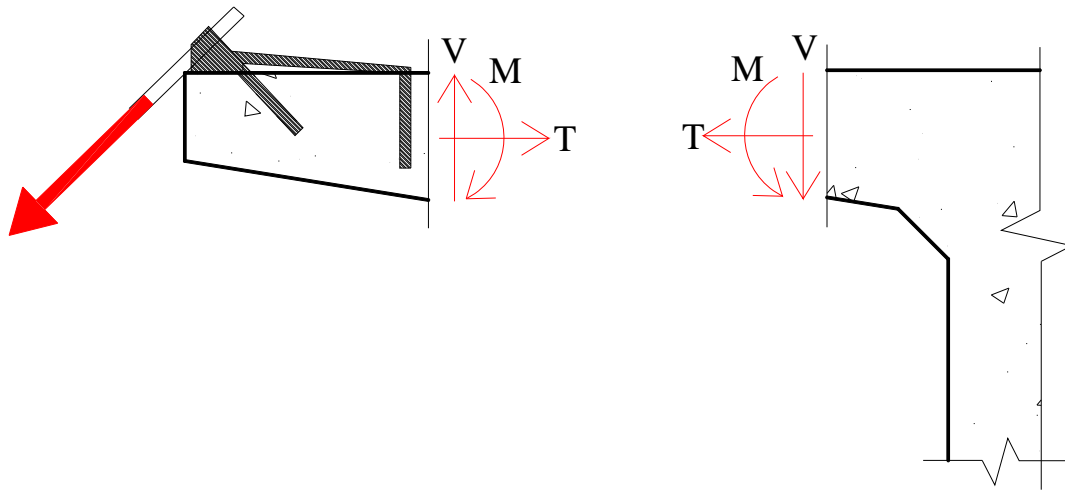
Figure 4-4: Free body diagram of top flange

4.2.3 Failure Mechanisms

Three different failure mechanisms were observed throughout the tests. The primary modes of failure were punching shear at hanger tip, combined punching shear/tensile failure at the embedded face, and excessive deformation of the bracket system. These failure mechanisms are defined as follows:

- Punching Shear at hanger tip: Failure mode when punching shear cone propagates from the tip of the hanger at the top flange edge due to vertical component of the hanger load.
- Combined punching shear/tensile failure at embedded face: Failure due to punching shear at face of embedded portion of hanger due to the loss of cross sectional area in compression caused by tensile cracking.
- Excessive deformation of bracket system: Failure defined by excessive damage, rotation, or buckling of bracket, coil rod, or hanger.

The failure of the overhang system is thus defined when the punching shear capacity at the hanger tip is exceeded, the tensile cracks propagate into the cross section causing punching shear at face of embedded portion of the hanger, or the bracket system undergoes excessive deformations. Each failure mode occurred more than once and in different systems. In depth explanations of each mechanism will follow in the individual test results.

While not a typical failure mode, it is important to recognize that significant crack formation greatly affects the behavior of the system. The test results in the following sections illustrate the change in behavior upon cracking and show to the reasons why the formation of cracks was used as a limit state.

4.3 TEST RESULTS: MEADOW BURKE HF-43

The HF-43 Precast Embed Hanger was tested on the Tx-28-I, Tx-28-II, and Tx-46. The two modified versions, the HF-43-V and HF-43-I, were both tested on the Tx-70. All test specimens where HF-43 hangers were used were loaded with the HF-86 Bridge Overhang Bracket. The loading geometry, reinforcement spacing, and hanger placement will be shown for each test. The placement of strain gauges for each test is shown in Appendix B.

4.3.1 Tx-28-I

The test region on the Tx-28-I was located at the girder end and consisted of four HF-43 hangers, with an edge distance of 13" and spacing of 21". The top flange transverse reinforcement consisted of No.3 bars spaced at 3" and 4". The hanger spacing, reinforcement spacing, and bracket geometry are shown in Figure 4-5. Figure 4-6 shows the test setup and the locations of hangers and load cells used in the test.
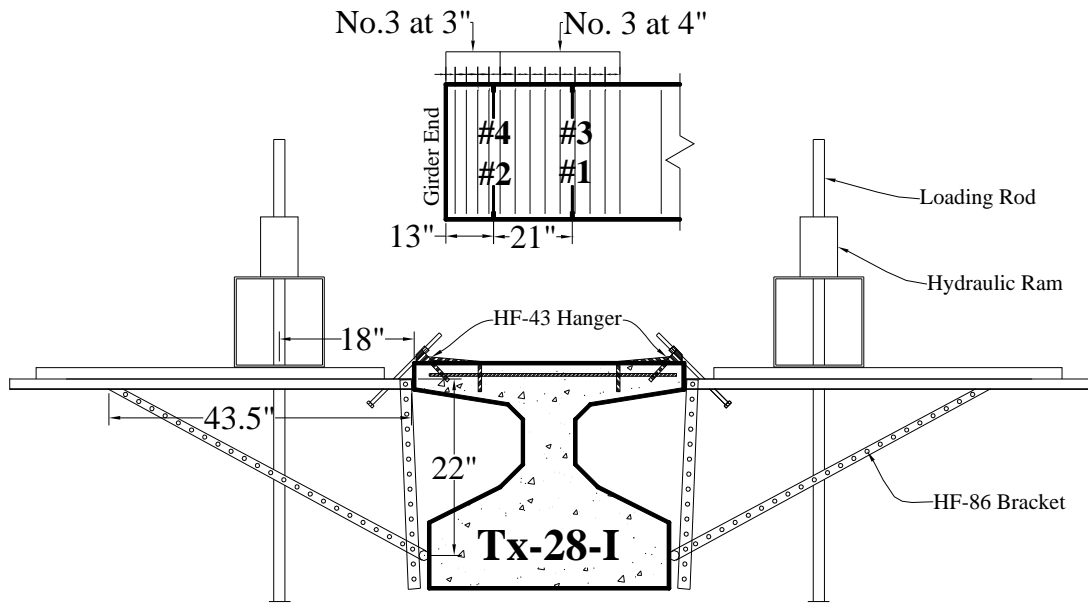
Figure 4-5: Reinforcement pattern and loading geometry for HF-43 on Tx-28-I
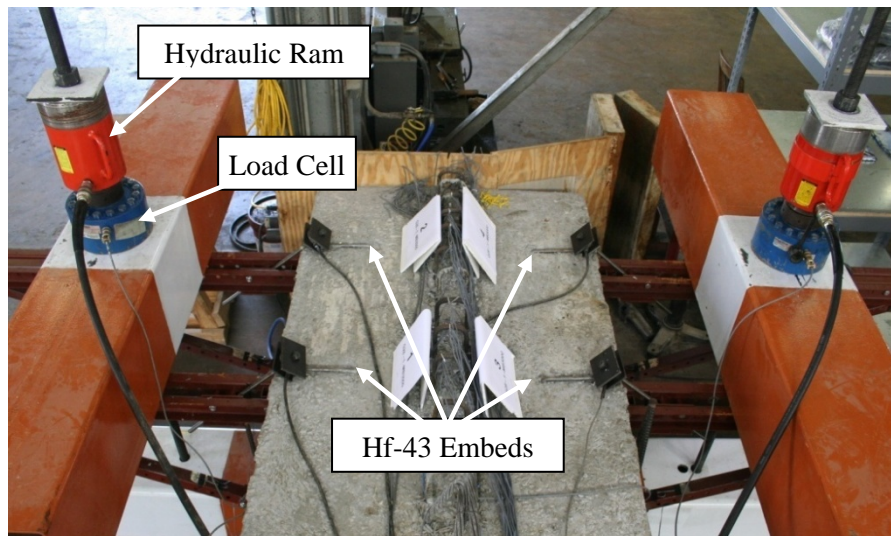
Figure 4-6: Test setup of HF-43 on Tx-28-I

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Cracks first appeared stemming from the embedded portion of the hanger at approximately 6700 lbs as illustrated in Figure 4-7. When the cracks appeared on the top surface, the strain values in the reinforcing steel crossing those cracks quickly increased as depicted in Figure 4-8. The sudden increase showed the loss of load carrying capacity in the concrete when it cracked. After the initial cracking occurred, the horizontal load is transferred to the steel reinforcement and the system was still capable of carrying more load until the punching shear capacity was reached at 8000 lbs. The punching shear failure plane propagated from the tip of hanger #2 where the flat portion of the hanger clip beared on the flange transferring the vertical component of the load. Figure 4-9 shows the punching shear failure cone after the bracket and coil rod had been removed at the primary location of failure in specimen Tx-28-I. After the test setup had been disassembled and the test locations were examined further, it was observed that top flange at the hanger #4 on the opposite side of the girder had also failed in punching shear.
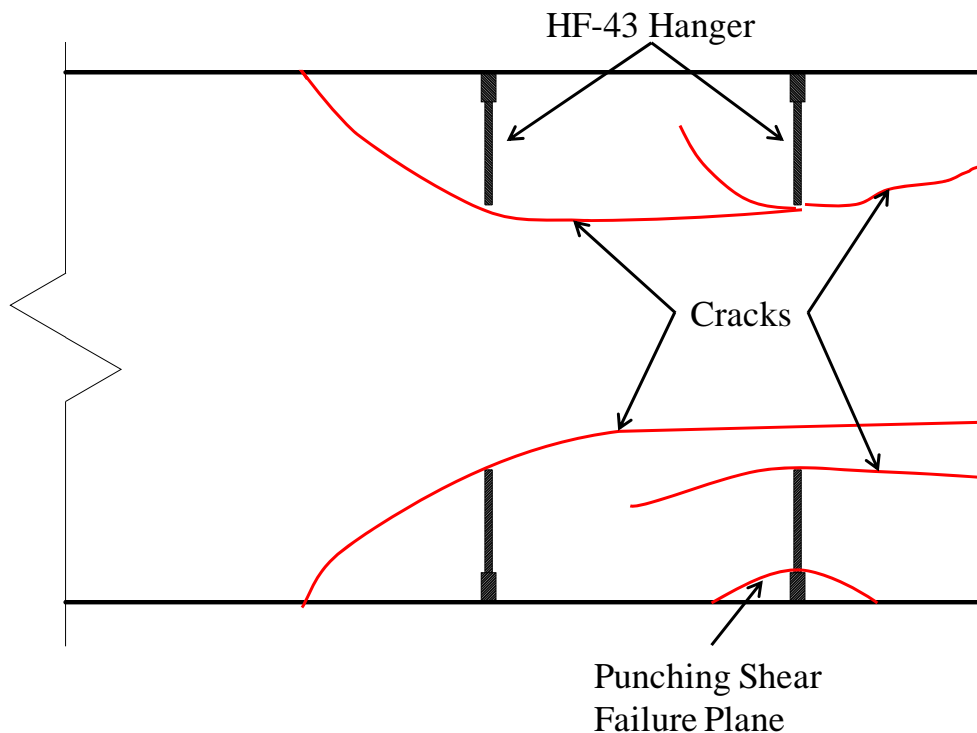
Figure 4-7: Crack pattern and locations of failure plane
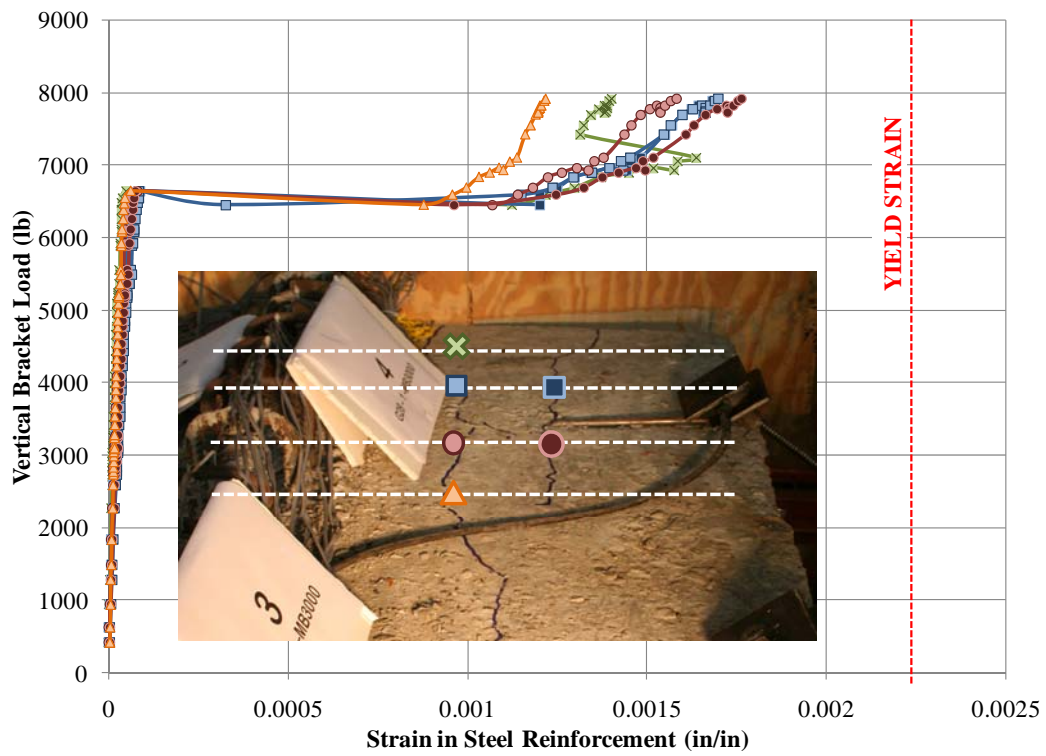
Figure 4-8: Tx-28-I HF-43 Reinforcement Strain at hanger #4

Figure 4-9: Punching shear failure at #2 HF-43 tip on Tx-28-I

4.3.2   Tx-28-II

The second HF-43 test was conducted on the Tx-28-II. The hanger placement in the Tx-28-II test region was at the center of the girder and the hangers were spaced at 5'-9" as to eliminate any possible group effect. The top flange transverse reinforcement consisted of No.3 bars at 12" as shown in Figure 4-10 along with the bracket geometry and hanger locations.
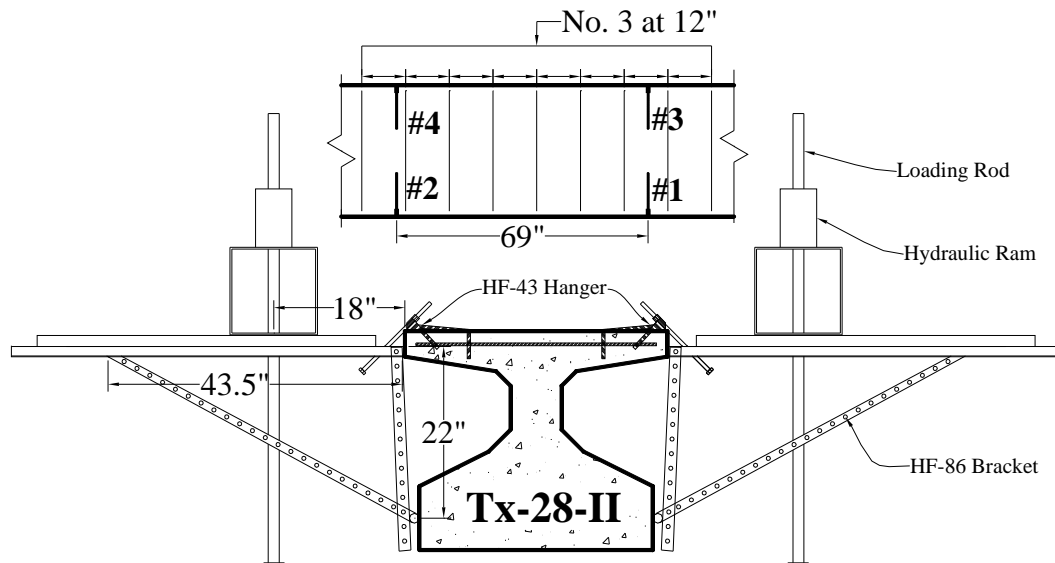
***Figure 4-10: Reinforcement pattern and loading geometry for
HF-43 on Tx-28-II***

Small surface cracks began to appear around 8000 lbs and strain values began to increase rapidly at 8500 lbs as illustrated in Figure 4-11. The failure in the system occurred at an applied vertical load of 9200 lbs with punching shear of the top flange at the #4 hanger tip as shown in Figure 4-12. The failure cone was essentially the same shape as that observed in Tx-28-I testing. The punching shear failure occurred very soon after cracking illustrating that the punching shear capacity was only slightly higher than the cracking load in this case. The maximum strain in the reinforcing steel was less than 10% of yield strain.
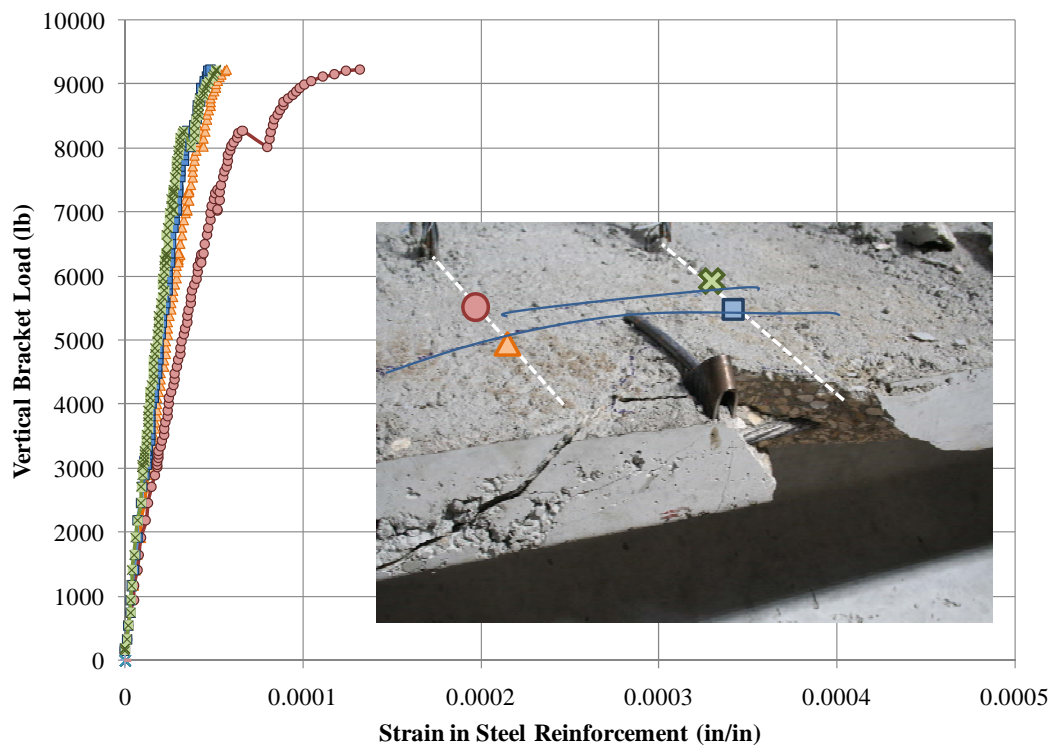
Figure 4-11: Tx-28-II HF-43 Reinforcement Strain at hanger #4

Figure 4-12: Punching shear failure at #4 HF-43 tip on Tx-28-II

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4.3.3   Tx-46

The third test with the HF-43 hanger was conducted on the Tx-46 girder. The same fabrication and instrumentation procedure was followed as the Tx-28 girders. The bridge overhang bracket geometry was altered to fit the deeper girder. The diagonal leg and vertical strut were adjusted to allow the bracket to react to the bottom flange of the girder. The four HF-43 hangers in the test region had an edge distance of 6" and spacing of 51". The hangers were placed very close to the edge to investigate the performance at extreme locations. The top flange transverse reinforcement was No. 3 bars spaced at 8" at the end and 12" towards the girder center as illustrated in Figure 4-13. The loading geometry and hanger placement is also shown in Figure 4-13.
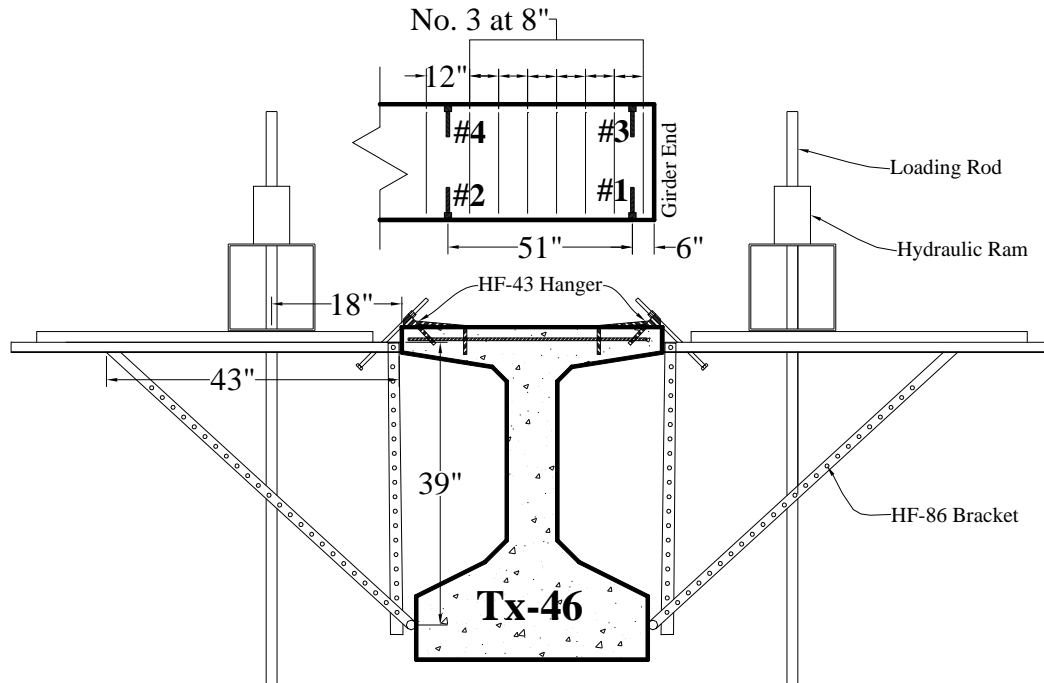
Figure 4-13: Reinforcement pattern and loading geometry for HF-43 on Tx-46

The top flange began to crack just prior to an applied load of 6000 lbs at the two hangers located closest to the girder end. A very clear cracking point can be observed in the strain gauge data in which the strain in the reinforcement rapidly increases until the initiation of strain hardening illustrated in Figure 4-14. The locations of the two hangers placed further from the edge showed no signs of cracking in the top flange or in the strain gauge data. The data collected from the

strain gauges closest to the girder edge contained strain values many times the reinforcement yield strain. The top flange at hanger #1 closest to the girder end failed at 7400 lbs in a combined punching shear/tension failure in which the entire corner of the flange broke free from the girder. The data and experimental observations illustrate a combined punching shear and tension failure in which the tensile crack propagated deep into the cross section resulting in a reduced cross sectional area and ultimately a punching shear failure. Figure 4-15 and Figure 4-16 show the corner of the top flange that broke-off due to punching shear and tensile stresses in specimen Tx-46.
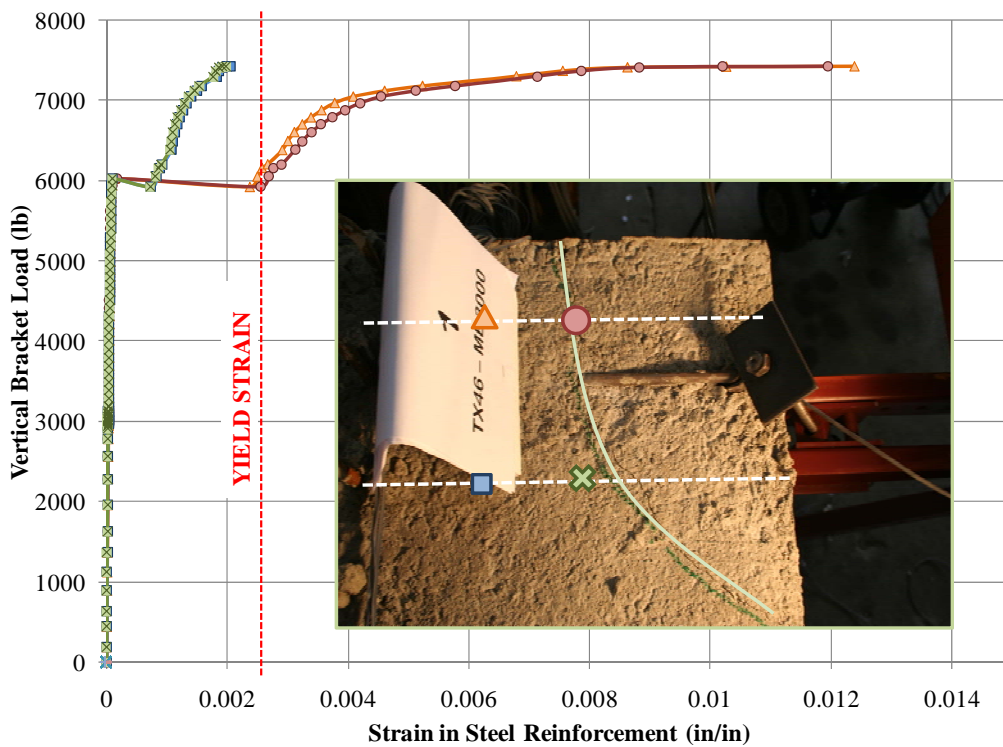
Figure 4-14: Tx-46 HF-43 Reinforcement Strain at hanger #1

Figure 4-15: Punching shear/tensile failure of #1 HF-43 on Tx-46

Figure 4-16: Side view of #1 HF-43 failure on Tx-46

4.3.4 Tx-70

After three tests were completed on the HF-43 hanger system, Meadow Burke engineers developed two alternatives to the HF-43. The primary reason for the modifications was to increase the capacity of the system when used with thin flanges such as those in the Tx family of girders. The two alternatives, named HF-43-V and HF-43-I due to their shape, were tested on specimen Tx-70. The geometry of the overhang bracket had to be adjusted due to the extreme depth of the girder. The bracket support was reacted against the lower portion of the girder web instead of the bottom flange as pictured in Figure 4-17.

Figure 4-17: HF-86 Brackets installed on Tx-70

4.3.4.1 HF-43-V

The modified HF-43-V hangers were placed towards the interior of the girder at a standard spacing of 36" as to model typical bridge deck construction practice. The top flange transverse reinforcement were No. 3 bars spaced at 12". The reinforcement pattern and loading geometry are in shown in Figure 4-18.
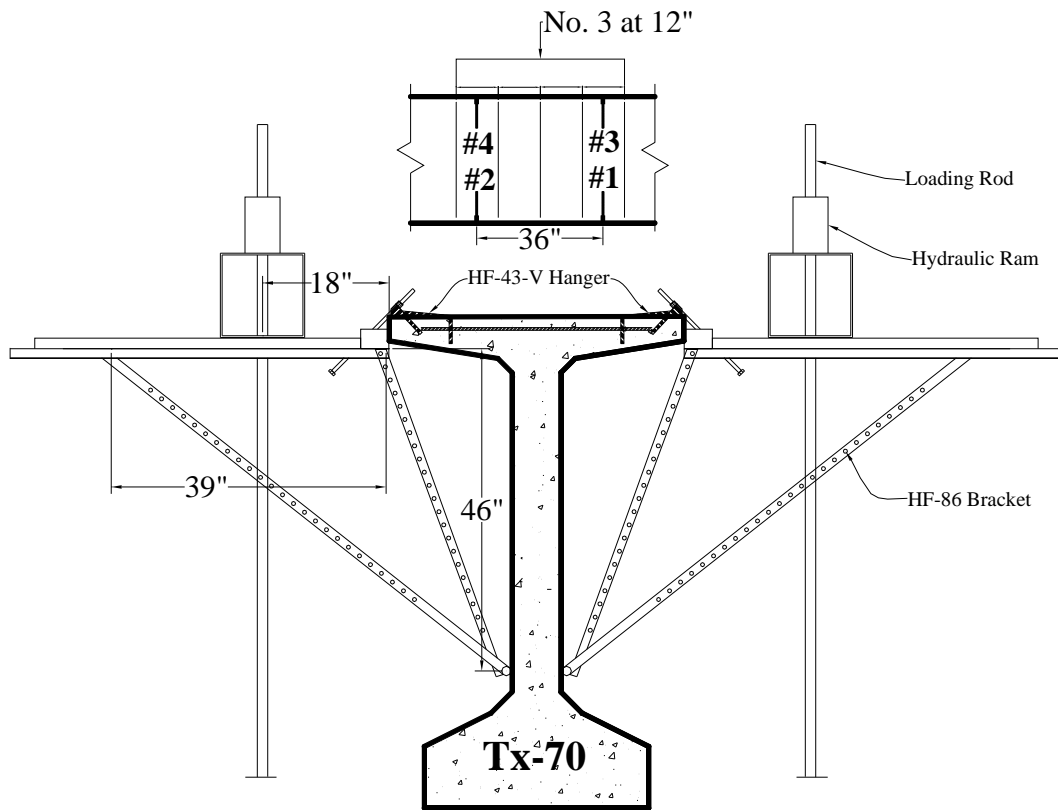
Figure 4-18: Reinforcement pattern and loading geometry for HF-43-V on Tx-70

First cracking was observed in the strain gauge data and through visual inspection at an applied load of approximately 6200 lbs. Only one of the four hanger locations had significant cracking and increased reinforcement strains as illustrated in Figure 4-19. The #4 HF-43-V hanger system failed at an applied load of 6700 lbs in a punching shear failure mode at the tip of the flange. The punching cone stemmed from the hanger tip and a volume of concrete including a large portion of the underside of the top flange punched out. Figure 4-20 shows the damage to the underside of the top flange at hanger #4 due to punching shear failure.
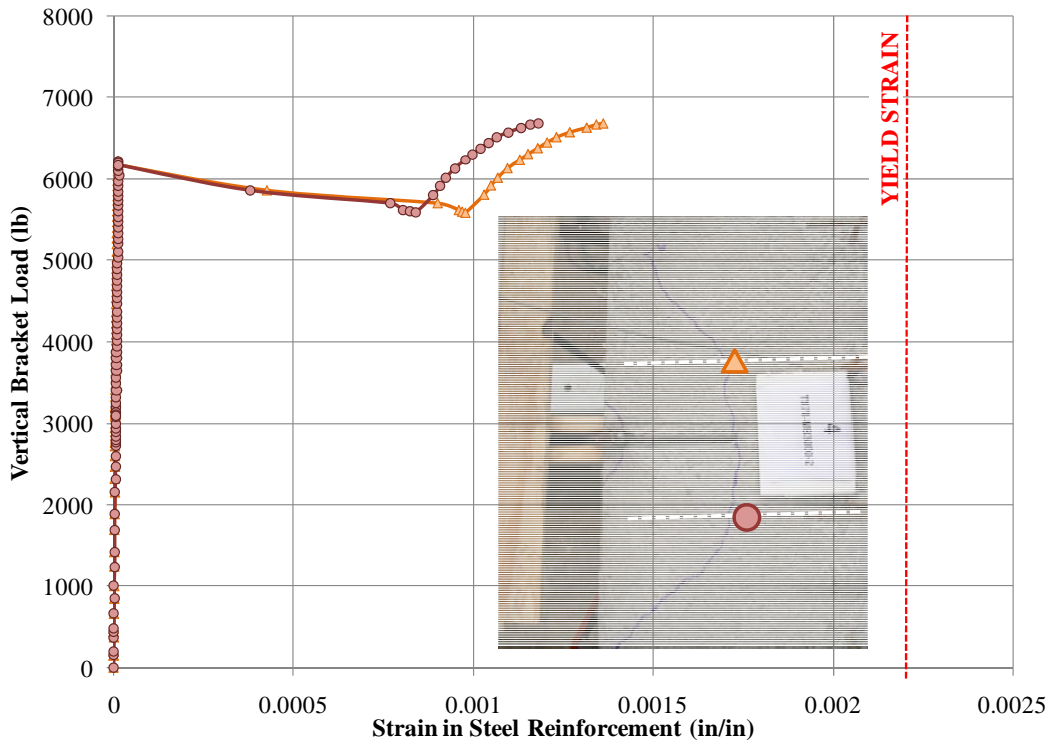
Figure 4-19: Tx-70 HF-43-V Reinforcement Strain at hanger #4

Figure 4-20: Underside of top flange of #4 HF-43-V on Tx-70 at failure zone

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4.3.4.2 HF-43-I

The modified HF-43-I hangers were placed 28" from the girder end at a spacing of 36". The top flange transverse reinforcement was No. 3 bars spaced at 8" at the girder end and 12" toward the mid-span as shown in Figure 4-21 along with bracket geometry and hanger locations.
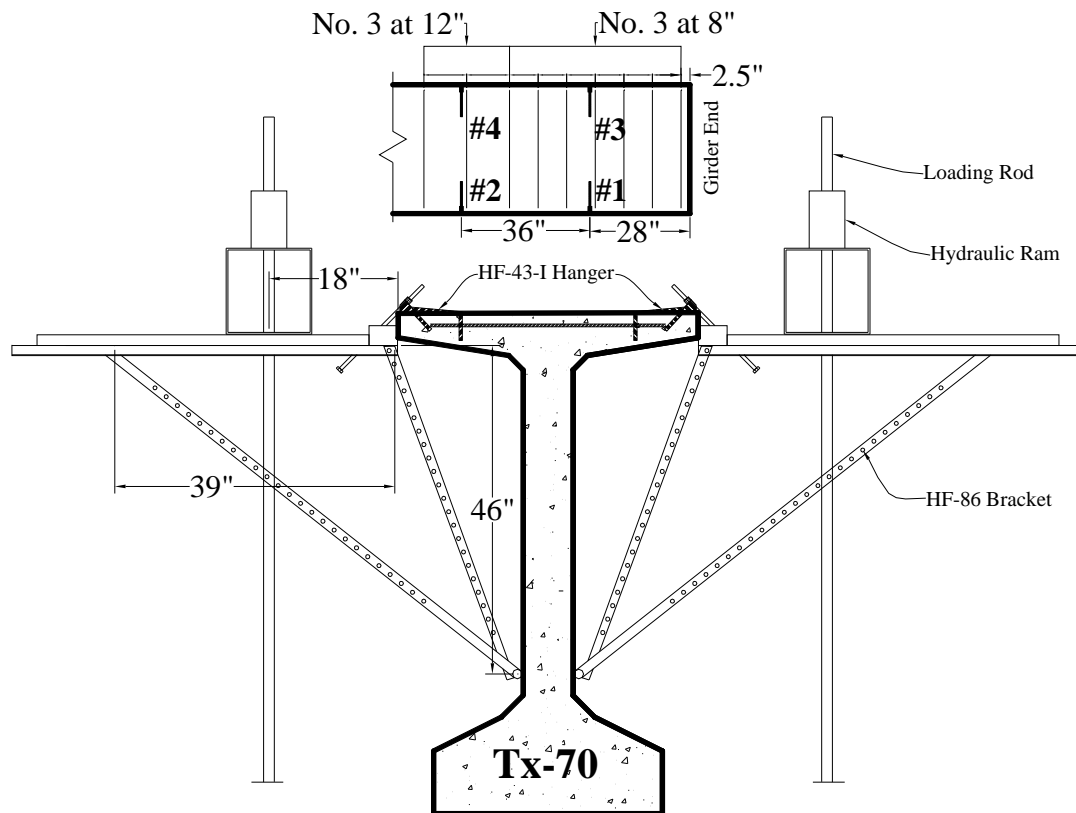
Figure 4-21:Reinforcement pattern and loading geometry for HF-43-I on Tx-70

The top flange began to crack under and an applied vertical load of approximately 6200 lbs. The strain gauge readings indicated significant cracking in the top flange around two of the four hangers with reinforcement strain approaching yield strain indicated in Figure 4-22. The failure of the #4 HF-43-I system occurred under an applied vertical load of 7500 lbs in punching shear of the top flange at the hanger tip. The punching shear cone was virtually the same as with the HF-43-V. The location of failure in the top flange is shown in Figure 4-23.
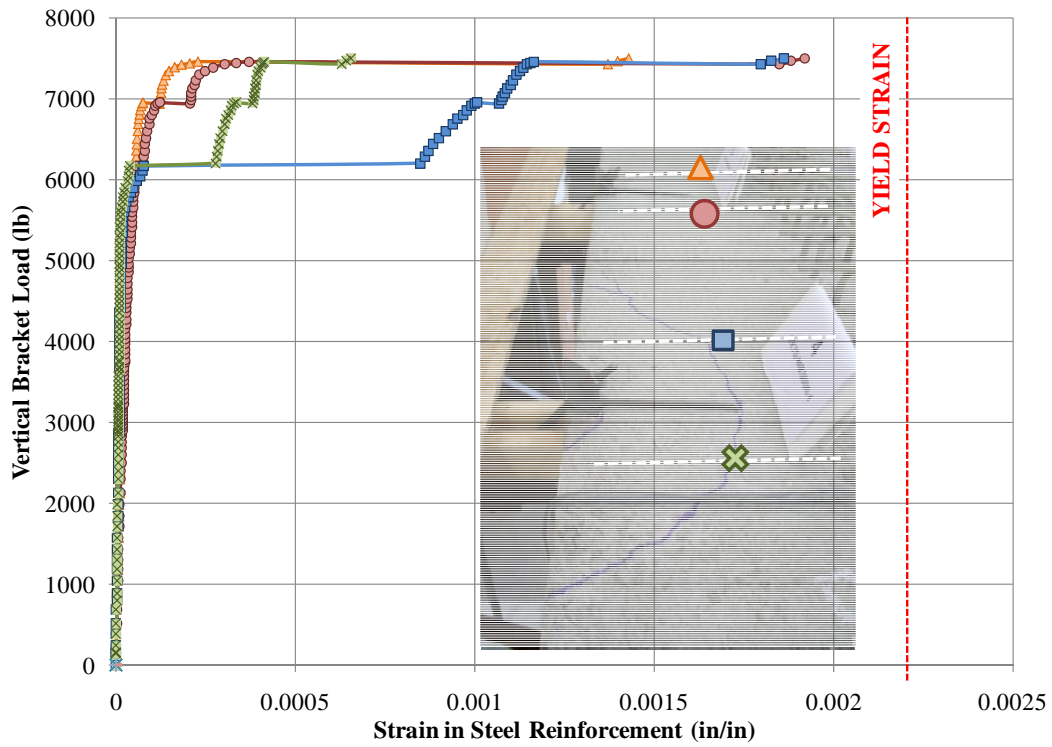
Figure 4-22: Tx-70 HF-43-I Reinforcement Strain and hangers #3 and #4

Figure 4-23: Underside of top flange of #4 HF-43-I on Tx-70 at failure zone

4.3.5 Summary of Results: HF-43

The results from the HF-43 and the modified HF-43 tests are summarized in Table 4-2. The applied vertical load at cracking and ultimate, the ratio of the cracking load to ultimate load, the failure mode, the location of the test region along the girder, the hanger spacing, the top flange reinforcement in the test region, the maximum strain in the steel reinforcing, and the concrete compressive strength are all tabulated.

Table 4-2: Summary of Test Results: HF-43

Hanger	Girder	Cracking Load (lb)	Ultimate Load (lb)	Ratio of Cracking to Ultimate	Failure Mode	Test Region and hanger spacing	Top Flange Reinf.	Max. Reinf. Strain	Girder f_c' (psi)	Photograph of failure zone
HF-43	Tx-28-I	6700	8000	0.81	Punching Shear at tip	13" from girder end 21"	#3 bars at 3"/4"	0.0024	13800	
	Tx-28-II	8000	9000	0.89	Punching Shear at tip	Mid-span 69"	#3 bars at 12"	0.0003	11400	
	Tx-46	6000	7400	0.81	Punching Shear/ Tensile	6" from girder end 51"	#3 bars at 8"/12"	0.0180	13200	
HF-43-V	Tx-70	6200	6700	0.93	Punching Shear at tip	Mid-span 36"	#3 bars at 12"	0.0014	11600	
HF-43-I	Tx-70	6200	7500	0.83	Punching Shear at tip	28" from girder end 36"	#3 bars at 8/12"	0.0020	11600	

While numerous variables exist among the five tests (bracket geometry, concrete strength, reinforcement pattern, hanger location and spacing, and hanger configuration), comparison of the test results presented in Table 4-2 indicate certain behavior of the HF-43 hanger used with the Tx girder.

(i) The group effect of closely spaced hangers can lower the load at which the top flange first cracks: Although Tx-28-I had a higher concrete strength than Tx-28-II, the cracking load was lower likely due to a hanger spacing of 21" rather than the 69" spacing on Tx-28-II.

(ii) Placing hangers near the girder end will result in a lower applied load to cause cracking of the top flange. The results from Tx-46 showed the lowest cracking load and highest reinforcing steel strain. The close proximity of the HF-43 hanger to the girder end caused the tensile crack to quickly increase in width resulting in punching shear failure due to tensile cracking.

(iii) The amount of transverse reinforcing steel does not appear to have a significant effect on the cracking and ultimate load when punching shear at the hanger tip is the failure mechanism: In all four tests where punching shear at the hanger tip was the ultimate failure mechanism, the reinforcement pattern does not show a correlation between the critical loads.

While other trends and correlations likely exist, further analysis of the variables present in the system would require more experimental testing and further isolating the main variables. Although every possible situation that may occur in construction was not tested, a wide variety of possibilities were examined with the testing of the HF-43 overhang bracket system on different girder depths, at various hanger spacings, and with many steel reinforcing patterns. Following the discussion of alternate overhang bracket systems, an allowable load recommendation will be presented at the conclusion of this chapter.

4.4   TEST RESULTS: MEADOW BURKE HF-67

The HF-67 Bulb Tee Hanger was tested on the Tx-28-I and Tx-28-II. The modified version, the HF-67-M, was tested on the Tx-46 and Tx-70. The hangers tested on the Tx-28-I, Tx-28-II, and Tx-46 were loaded with the HF-86 Bridge Overhang Bracket. The hangers tested on the Tx-70 were loaded with the HF-96 Aluminum Bridge Overhang Bracket.

4.4.1 Tx-28-I

The test region in specimen Tx-28-I was located at the girder end and consisted of four ¾" coil rods cast in the girder 12¼" from the flange tip and embedded 3½" into the girder as with all HF-67 tests shown in Figure 4-24. The coil rods for the Tx-28-I test specimen were embedded at an edge distance of 12" and a spacing of 28" as shown in Figure 4-25. The reinforcement located in the top flange consisted of No. 3 bars spaced at 4". The bracket geometry is also illustrated in Figure 4-25. The HF-67 Bulb Tee Bar Hanger was placed around the embedded coil rod and tightened into place with a coil nut.
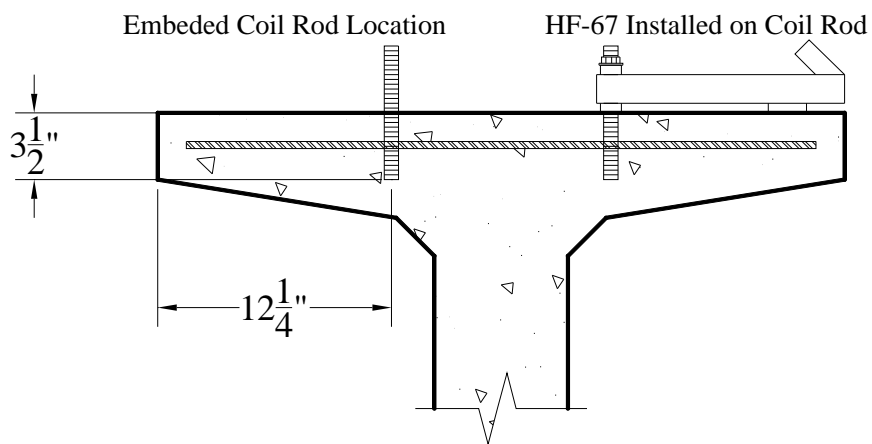
Figure 4-24: Diagram of HF-67 Coil Rod installation
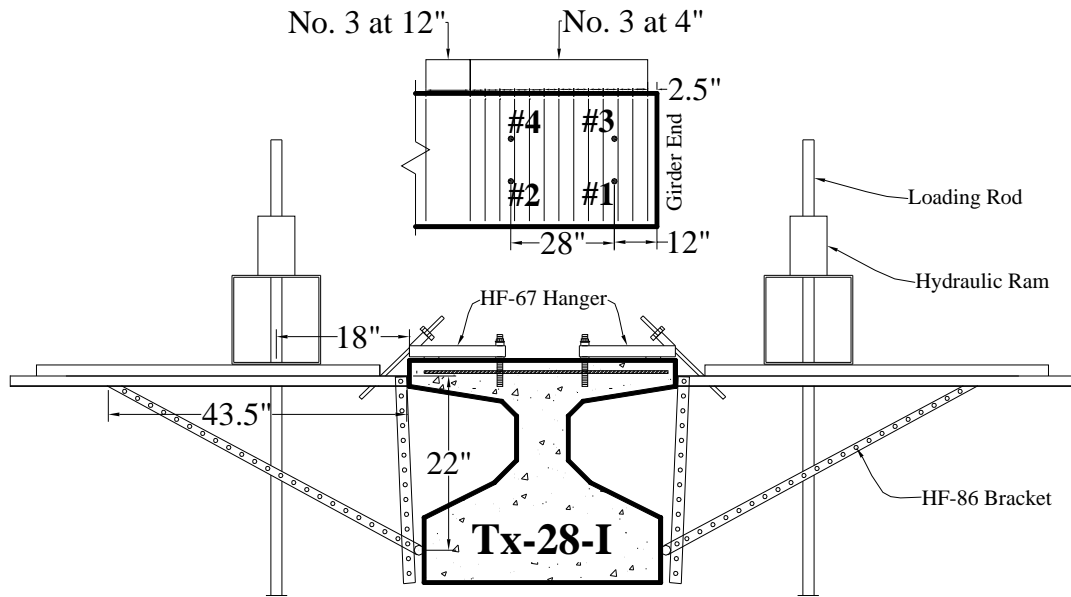
Figure 4-25: Reinforcement pattern and loading geometry for HF-67 on Tx-28-I

The top flange began to crack under a vertical bracket load of approximately 6900 lbs. The cracks propagated from the embedded coil rod toward the edge of the flange. The cracks grew in length and in width as the applied vertical load was increased to 8000 lbs as shown in Figure 4-26. After initial cracks formed in the top flange, the strain in the reinforcing bars rapidly increased towards yield strain as illustrated in Figure 4-27. The specimen was not taken to ultimate failure in order not to damage the HF-86 Overhang Brackets. The decision was made to suspend the load test at a vertical load of 8400 lbs per bracket because the brackets were needed to be undamaged for upcoming tests in which representatives from TxDOT and Meadow Burke were present. There was no damage to the HF-86 Overhang Bracket when the system was unloaded. As discussed earlier in this chapter, for purposes of establishing safe working loads for construction, cracking loads are of greater significance than the failure loads. While in this test the loading was suspended prior to failure, the load in which cracks first formed was obtained.

Figure 4-26: Progression of cracking with increasing vertical load at #2 hanger
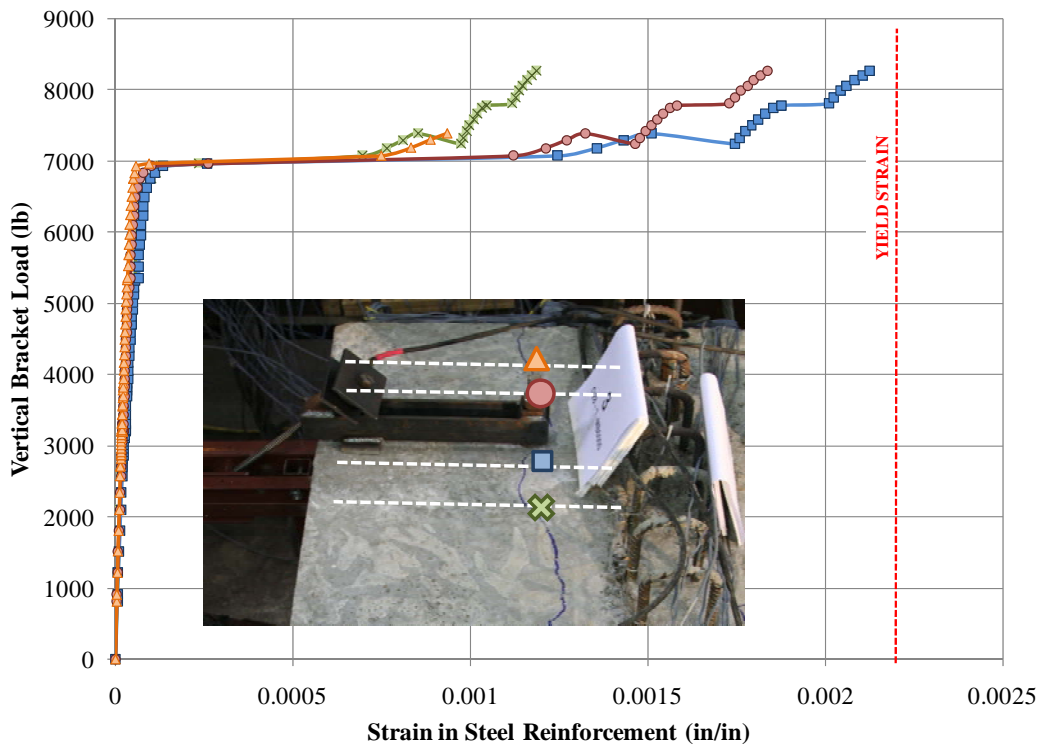
Figure 4-27: Tx-28-I HF-67 Reinforcement Strain at embed #3

4.4.2 Tx-28-II

The test region in specimen Tx-28-II was located at the girder end. The hanger edge distance was 19" with a spacing of 69" to eliminate the group effect as illustrated in Figure 4-28. The reinforcement located in the top flange was No. 3 bars spaced at 3" and 4" at the girder end and 12" toward the mid-span as depicted in Figure 4-28 in addition to the bracket geometry.
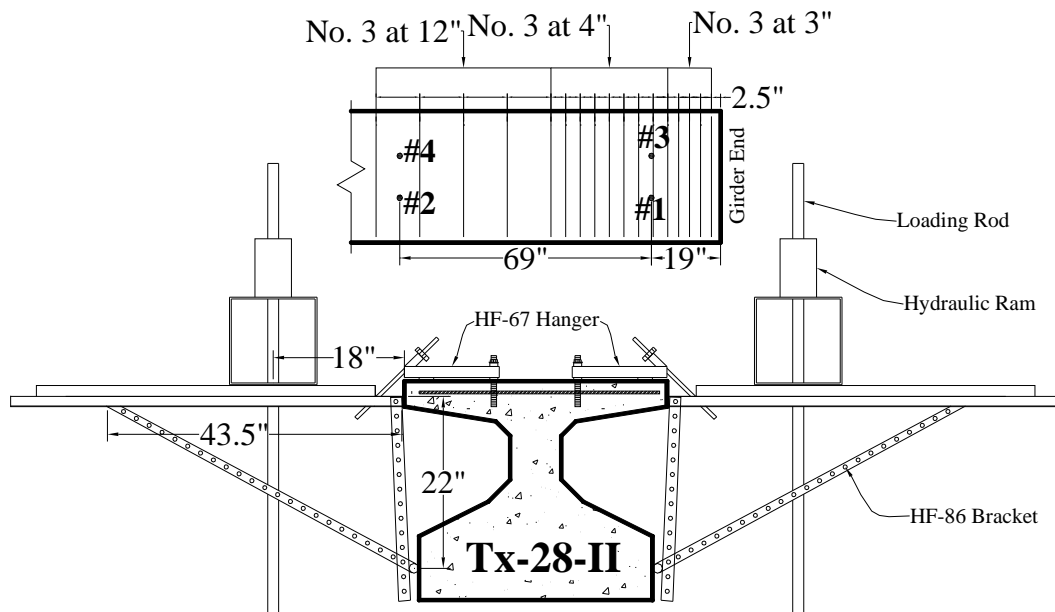
Figure 4-28: Reinforcement pattern and loading geometry for HF-67 on Tx-28-II

Cracks began to form stemming from the embedded coil rods at an applied vertical load of approximately 7300 lbs. Illustrated in Figure 4-28 the strain in the reinforcing bars did not increase as rapidly as previous tests results indicated. The top flange of the girder failed under an applied vertical load of 9000 lbs in punching shear at the hanger bearing plate as shown in Figure 4-30. The perimeter of the failure plane aligns with the back of the bearing plate on the underside of the HF-67.

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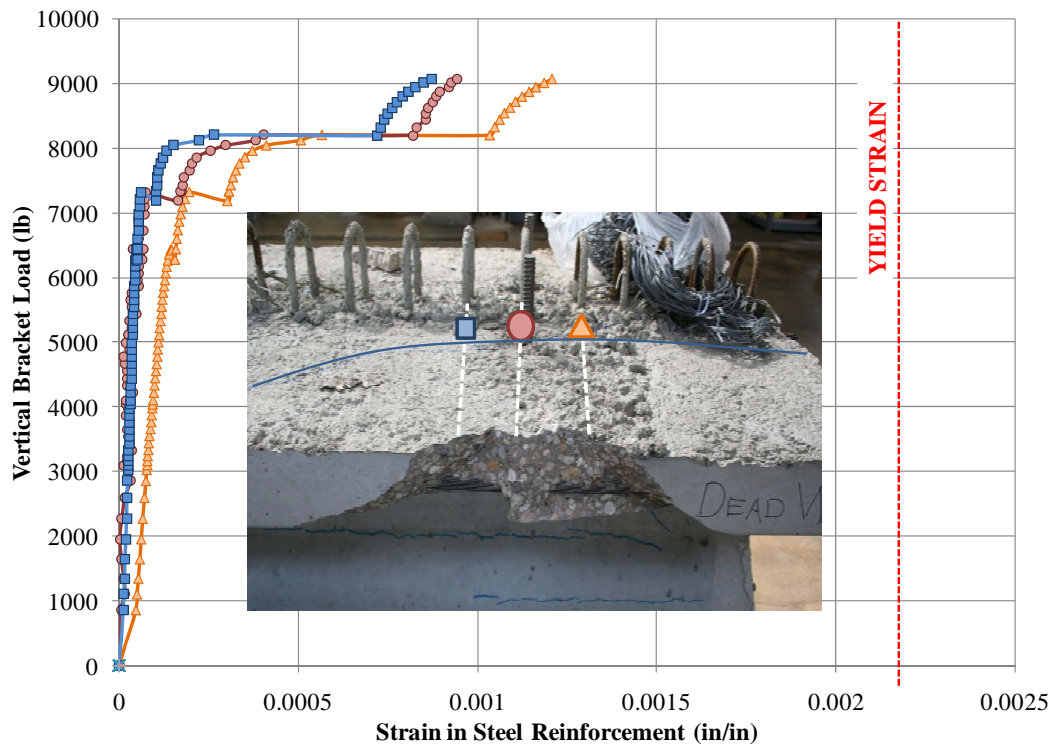
Figure 4-29: Tx-28-II HF-67 Reinforcement Strain at hanger #1

Figure 4-30: Punching shear failure at #1 HF-67 on Tx-28-II

4.4.3 Tx-46

After examining the test results in the first two tests conducted with the HF-67, Meadow Burke engineers revised their original design in an attempt to increase the punching shear capacity by increasing the area of concrete engaged by the bearing plate. Details of their refined design named HF-67-M are shown in Figure 3-7. The HF-67-M was tested on the Tx-46 in the middle region of the girder with the hangers spaced 52" apart. The top flange transverse reinforcement in the test region consisted of No.3 bars spaced at 12". The coil rod placement, reinforcing pattern and bracket geometry are shown in Figure 4-31.
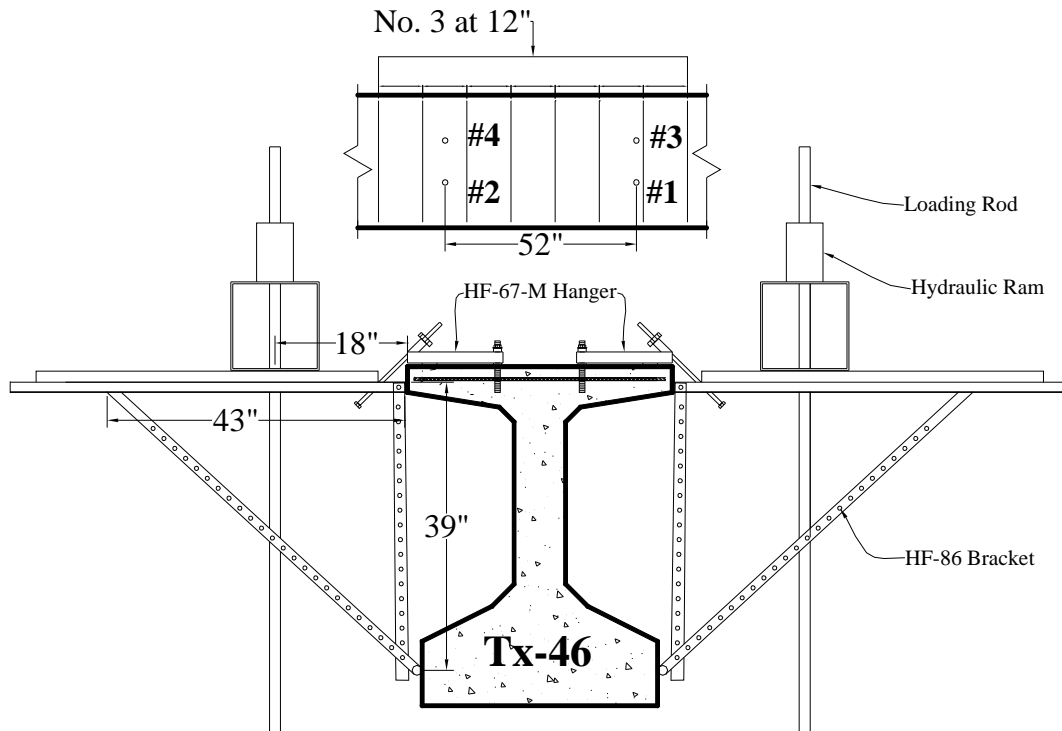
Figure 4-31: Reinforcement pattern and loading geometry for HF-67-M on Tx-46

Cracking of the top flange began under an applied vertical load of approximately 7000 lbs. The strain in the reinforcing steel crossing the cracks only slightly exceeded yield strain prior to failure shown in Figure 4-32. The HF-

67-M system failed at an applied vertical load of 12000 lbs in a punching shear failure mode at the top flange tip as seen in Figure 4-33. At the ultimate load, the punching shear failure plane clearly formed around the backside of the HF-67-M bearing plate. The modifications to the bearing plate on the HF-67, greatly increased the punching shear perimeter as shown in Figure 4-33. At higher loads, some amount of damage also occurred to the HF-67-M hanger, the attaching coil rod, and the HF-86 Overhang Bracket. Figure 4-34 shows the bending of the HF-67-M hanger as well as the bending of the attached coil rod. The rotation of the attaching coil rod was primarily due to the slippage of the top cord of the bracket from its original position bearing on the top flange. This slippage could have been prevented by providing a large wooden block between the top chord and the top flange to eliminate horizontal movement of the bracket causing the coil rod to deviate from its initial 45° inclined position. The resulting rotation may have reduced the failure load because when the coil rod changes angle, the horizontal and vertical load components are no longer equal. When the rod bends in the fashion it did in the Tx-46 test, the vertical component is larger than it would be in the 45° position reducing the failure load. However, the rotation of the coil rod occurred after initial cracking; thus the position of the coil rod was consistent with previous test results and the cracking load which is of primary importance, can be accurately evaluated.
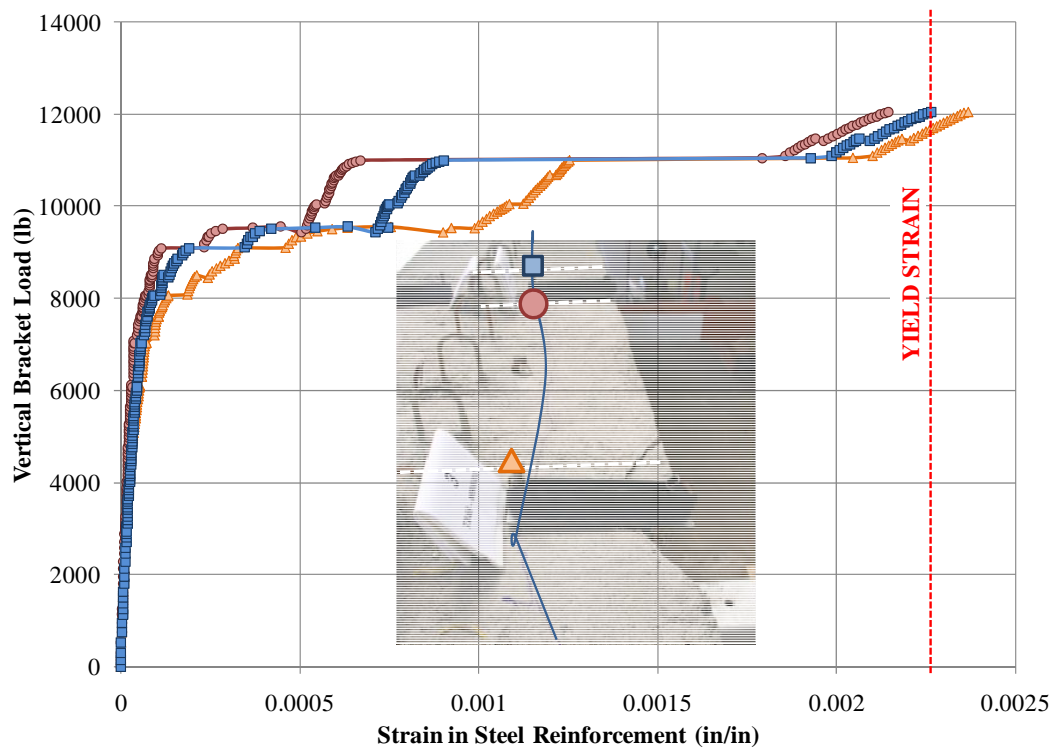
Figure 4-32: Tx-46 HF-67-M Reinforcement Strain at hanger #3 and #4

Figure 4-33: Punching shear at #3 HF-67-M hanger tip

Figure 4-34: Damage to #1 HF-67-M and coil rod

4.4.4 Tx-70

Two HF-67-M tests were conducted on specimen Tx-70. Both tests were performed using the heavy duty HF-96 Aluminum Bridge Overhang Bracket. The HF-96 was used in this case to decrease the possibility of having any damage to the overhang bracket or coil rod. The HF-96 bracket is used in bridge deck overhang construction when a larger than typical overhang is needed. In order to simulate this situation, the maximum allowable overhang was used in the design of the test. TxDOT engineers set the maximum overhang length for the Tx-70 at 5' from the girder center line. Using this criterion, calculations showed that the centroid of the overhang bracket construction loads was approximately 39" from the edge of the top flange. Therefore, the HF-96 brackets were loaded placing the loading rod 39" from the edge of the top flange. Figure 4-35 shows the HF-96 brackets installed on specimen Tx-70.
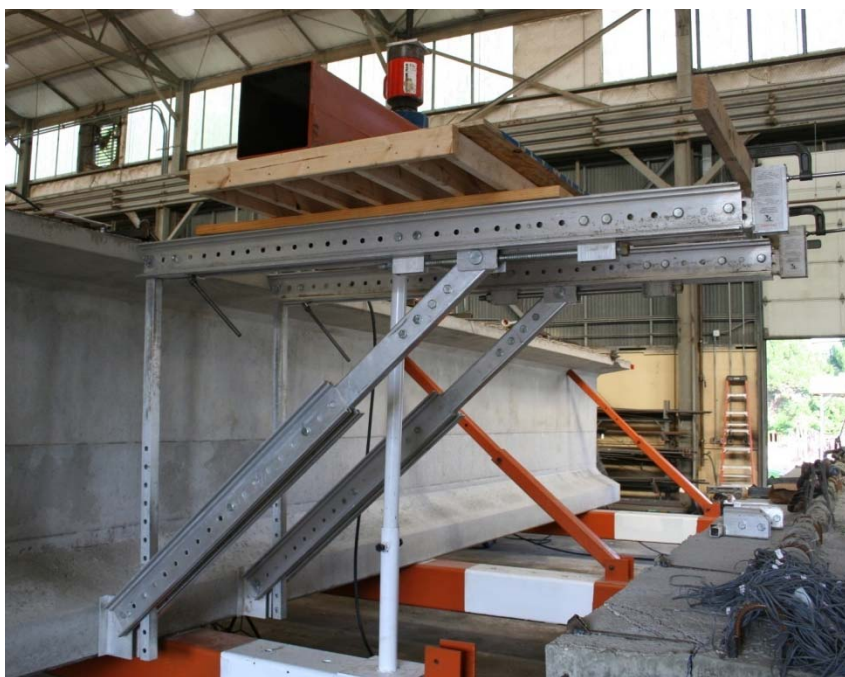
Figure 4-35: HF-96 Aluminum overhang brackets installed on Tx-70

4.4.4.1 Tx-70 Test #1

The first HF-67-M test on the Tx-70 girder took place at the girder end. The hangers had an edge distance of 28" and spacing of 36". The reinforcement in

the top flange was No. 3 bars spaced at 6" at the girder end and 12" toward mid-span as shown in Figure 4-36 along with the geometry of the HF-96.
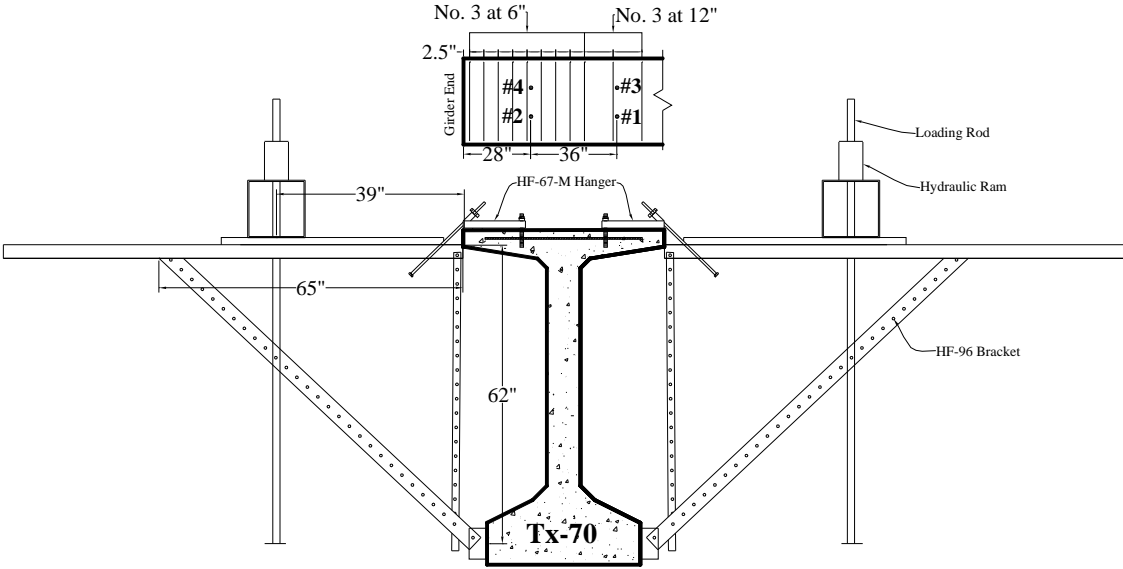
Figure 4-36: Reinforcement pattern and loading geometry for HF-67-M on Tx-70 Test 1

Initial cracks in the top flange began to form around 8000 lbs. As the applied vertical load was increased, the strain in the reinforcing steel increased to over 0.015 as shown in Figure 4-37. The top flange failed under an applied load of 11500 lbs in a combined punching shear/tensile failure mode. The failure occurred when the initial cracks that stemmed from the embedded coil rod propagated to the edge of the top flange and penetrated through the cross section until the punching shear capacity was exceeded. The failure mode is illustrated in Figure 4-38 at the #1 HF-67-M hanger.
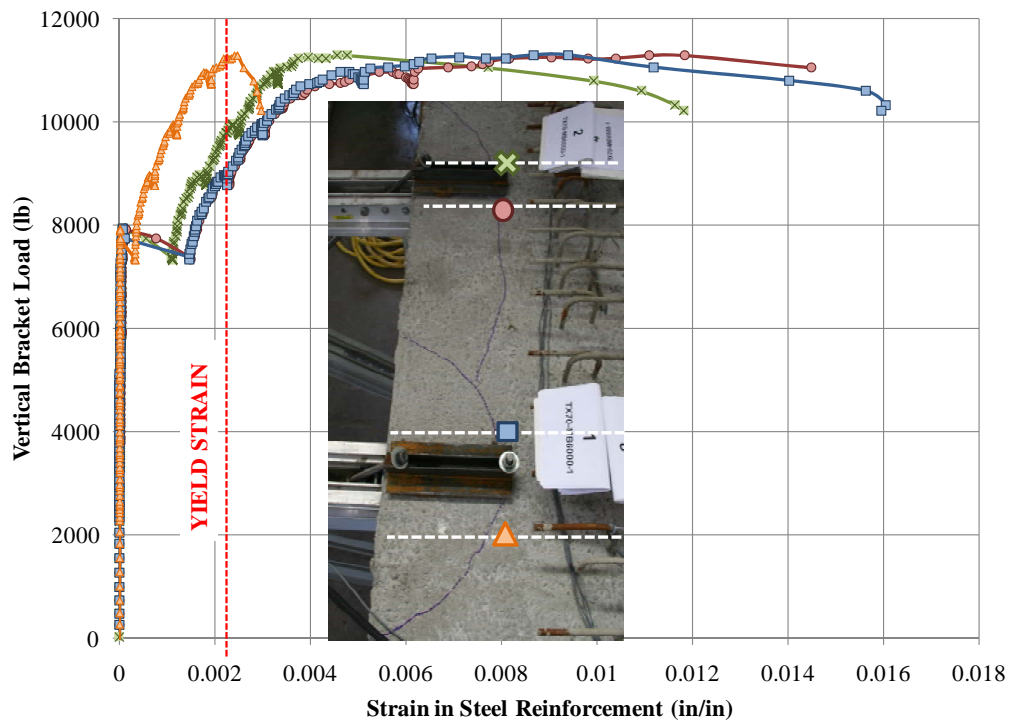
Figure 4-37: Tx-70 Test 1 HF-67-M Reinforcement Strain at hanger #1 and #2

Figure 4-38: Failure of top flange at #1 HF-67-M on Tx-70 Test #1

4.4.4.2 Tx-70 Test #2

The second test region of the HF-67-M on the specimen Tx-70 was in the interior of the girder span with hangers spaced at 36". The reinforcement pattern consisted of No. 3 bars spaced at 12" as illustrated in Figure 4-39.
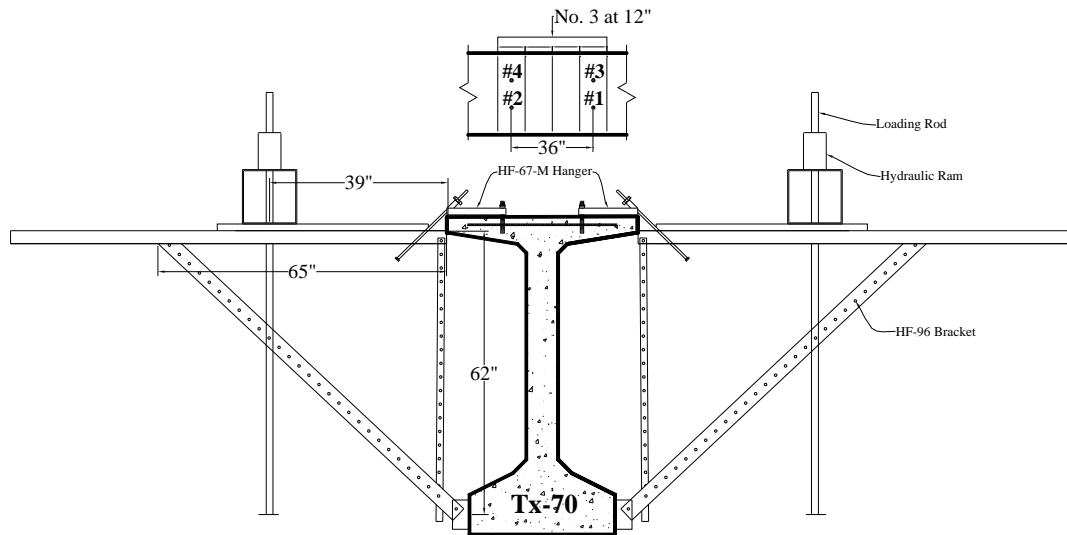
Figure 4-39: *Reinforcement pattern and loading geometry for HF-67-M on Tx-70 Test 2*

Cracking of the top flange first occurred under an applied vertical load of 6200 lbs around hangers #1 and #2. The other side of the top flange where hangers #3 and #4 were located did not crack until an applied load of 8000 lbs. The top flange failed around locations #3 and #4 under an applied vertical load of 9200 lbs in a punching shear/tensile failure with the entire top flange separating from the girder at the crack location was shown in Figure 4-40. Different than the HF-67-M Tx-70 Test #1, the crack propagated across to the neighboring hanger location and the punching shear occurred over a much longer distance. The strain in the reinforcing steel on the side of first cracking is shown in Figure 4-41.

Figure 4-40: Failure perimeter around HF-67-M hanger #3 and #4 on Tx-70 Test 2
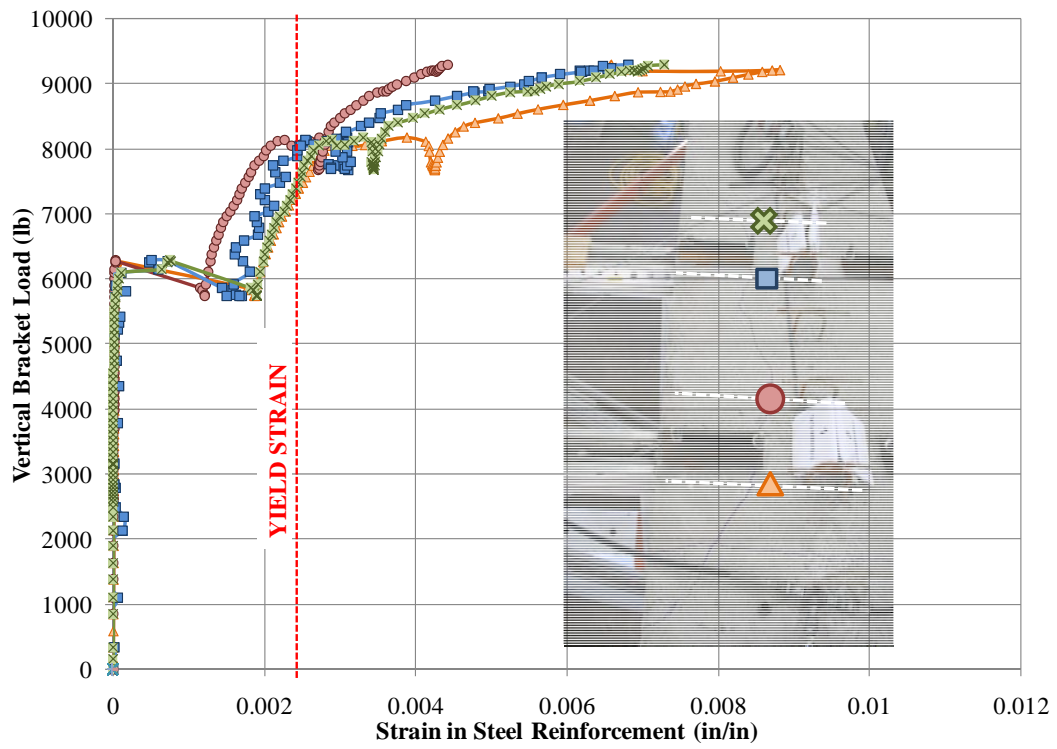
Figure 4-41: Tx-70 Test 2 HF-67-M Reinforcement Strain at hanger #1 and #2
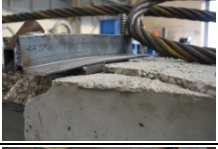
4.4.5　Summary of Results: HF-67

The results from the HF-67 and the modified HF-67-M tests are summarized in Table 4-3. The applied vertical load at cracking and ultimate, the ratio of the cracking to ultimate load, the failure mode, the location of the test region along the girder, the embed spacing, the top flange reinforcement in the test region, maximum reinforcement strain, and the concrete compressive strength are all tabulated.

Even though direct numerical comparisons between the tests are difficult due to the high number of variables between the tests, valuable correlations are observed in the test results and those observations are summarized as follows:

(i)　　The group effect of closely spaced hangers lowers the load in which cracking of the top flange first occurs. Similar to the HF-43 tests, the cracking load in Tx-28-I was lower than the cracking load on Tx-28-II despite having a higher concrete strength. This is likely due to the 28" spacing between hangers on the Tx-28-I in comparison to the 69" spacing on the Tx-28-II.

(ii)　　The higher failure load on the Tx-46 showed that the modification in hanger design increased the punching shear capacity by engaging a larger area under the bearing plate. However because the load transfer mechanism of the HF-67 and HF-67-M remained the same, the cracking load was similar for both hanger designs.

(iii)　　The difference in ultimate load results of the two tests on the Tx-70 can be explained by the amount of reinforcement in each test region. The first test with a cracking load of 8000 lbs and ultimate load of 11500 lbs was located close to the girder end where the spacing of reinforcement was 6". In the second test the reinforcement spacing in the test region was 12" resulting in a lower ultimate capacity of 9200 lbs. It is also interesting to note that the cracking load was also lower in this test (6200 lbs). The difference in amount of reinforcement had significant impact on controlling the penetration of the tensile crack at the embedded coil rod. Controlling the increase in depth and width of the crack directly influenced the effective cross sectional area that was available for punching shear resistance.

Table 4-3: Summary of HF-67 Results

Embed	Girder	Cracking Load (*lb*)	Ultimate Load (*lb*)	Ratio of Cracking to Ultimate	Failure Mode	Test Region and embed spacing	Top Flange Reinf.	Max. Reinf. Strain	Girder f_c' (*psi*)	Photograph of failure zone
HF-67	Tx-28-I	6900	N/A	N/A	No Failure	12" from girder end 28"	#3 bars at 4"	0.0022	13800	
	Tx-28-II	7300	9000	0.81	Punching Shear at bearing plate	19" from girder end 69"	#3 bars at 3"/4"/12"	0.0012	11400	
HF-67-M	Tx-46	7000	12000	0.58	Punching Shear at bearing plate	Mid-span 52"	#3 bars at 12"	0.0030	13200	
	Tx-70	8000	11500	0.70	Punching Shear/ Tensile	28" from girder end 36"	#3 bars at 6"/12"	0.0160	11600	
	Tx-70	6200	9200	0.65	Punching Shear/ Tensile	Mid-span 36"	#3 bars at 12"	0.0130	11600	

4.5 TEST RESULTS: DAYTON SUPERIOR C-24

The C-24 45° Pres-Steel Precast Half Hanger Type 4-APR was tested on the Tx-28-I, Tx-28-II, and Tx-46. All of the C-24 hangers were loaded with the C-49 Bridge Overhang Bracket.

4.5.1 Tx-28-I

The test region on Tx-28-I was located at the middle of the girder consisting of four C-24 hangers, two on each side spaced at 49" apart. The transverse reinforcing steel in the top flange was No. 3 bars at 12". The overhang bracket geometry, hanger spacing, and reinforcing pattern are shown in Figure 4-42.
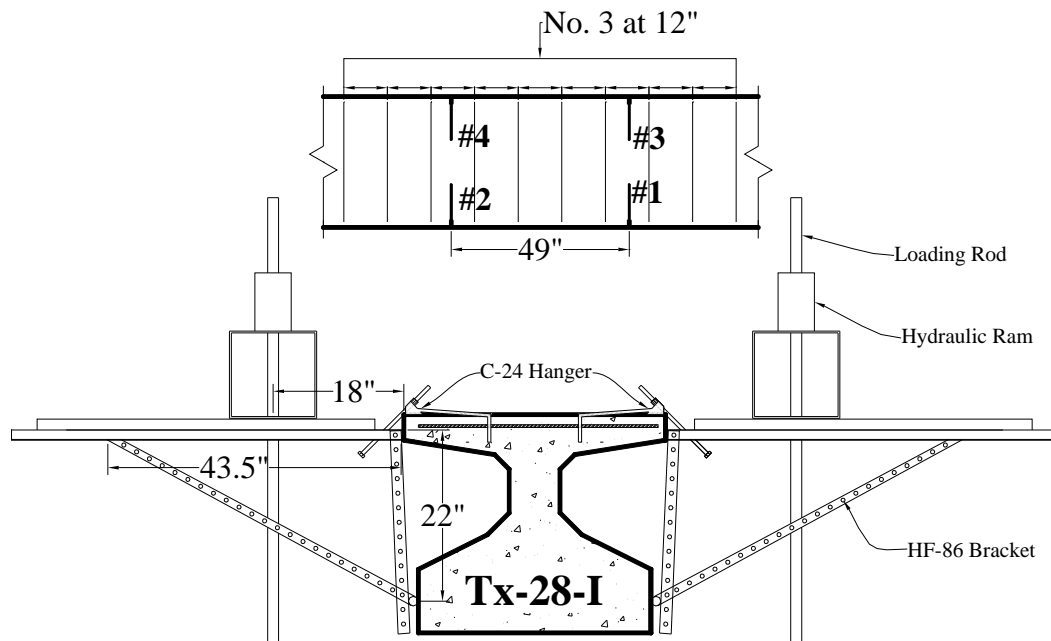
Figure 4-42: Reinforcement pattern and loading geometry for C-24 on Tx-28-I

Spalling of the concrete at the flange tip began to occur at approximately 4000 lbs due to rotation of the steel clip on the C-24 hangers. The top flange of the girder began to crack at 6500 lbs as indicated by the strain gauge data presented in Figure 4-43 . Failure of the system occurred under an applied vertical load of 7000 lbs with the top chord of the #4 C-49 overhang bracket buckling. Figure 4-44 shows a close up view of the buckled section of the overhang bracket.

When the system was unloaded and disassembled, inelastic damage was evident in all four brackets and some of the C-24 hangers also showed permanent rotations at the steel clip and significant spalling around hanger tip as shown in Figure 4-45.
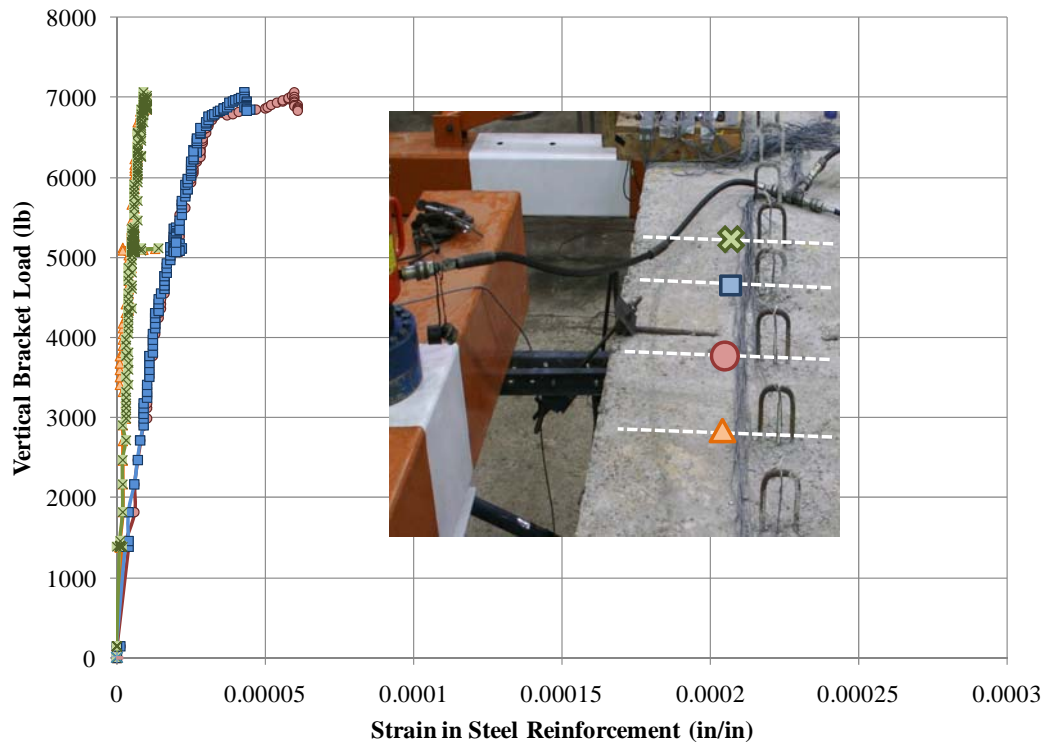
Figure 4-43: Tx-28-I C-24 Reinforcement Strain at hanger #3

Figure 4-44: Buckled section of #4 C-49 overhang bracket on Tx-28-I

Figure 4-45: Spalling of concrete around #2 C-24 hanger on Tx-28-I

4.5.2 Tx-28-II

The second C-24 test took place on the Tx-28-II. The test region was located towards the girder end with edge distance of 24" and spacing of 69". The transverse reinforcing steel in the top flange were No. 3 bars spaced at 4" at the girder end and 12" towards mid-span. The overhang bracket geometry, hanger spacing, and reinforcing pattern are shown in Figure 4-46.
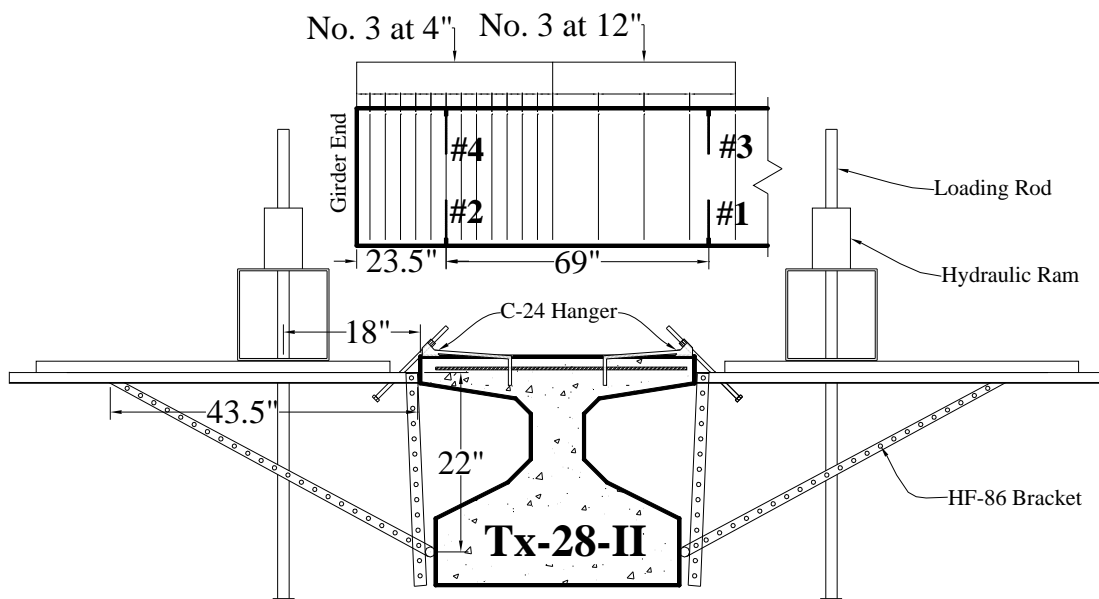
Figure 4-46: Reinforcement pattern and loading geometry for C-24 on Tx-28-II

Significant spalling of concrete around the hanger tip was observed at an applied load of 5000 lbs per bracket due to rotation of the hanger tip. The reinforcement strain gauge data indicated cracking in the top flange around 6800 lbs as illustrated in Figure 4-47. The rotation of the hanger tip and coil rod increased until it was determined too significant to continue loading. The failure mode on the Tx-28-II was characterized by excessive rotation of hanger tip and coil rod at an applied vertical load of approximately 8200 lbs. Figure 4-48 shows the most severe case of hanger tip rotation; however, significant rotation occurred at all four hanger locations. The C-49 overhang bracket also saw significant deformation under the ultimate vertical load as illustrated in Figure 4-49.

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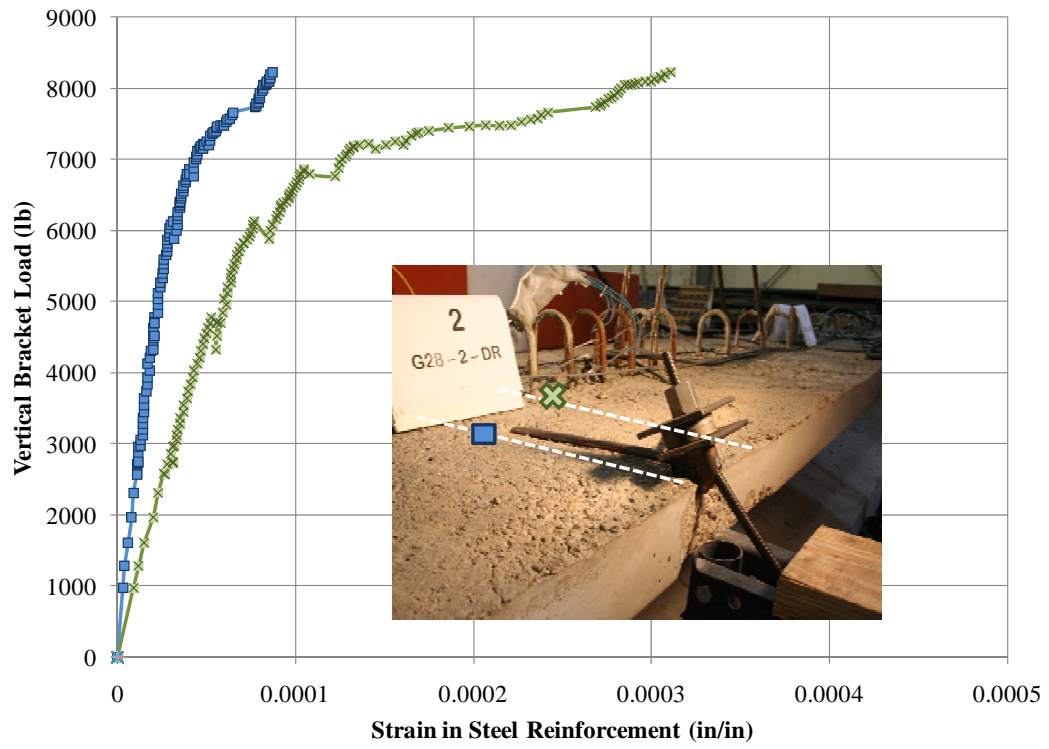
Figure 4-47: Tx-28-II C-24 Reinforcement Strain at hanger #2

Figure 4-48: Excessive #1 C-24 hanger tip and coil rod rotation at 8000 lbs on Tx-28-II

Figure 4-49: Deformation of #2 C-49 bracket at 8000 lbs on Tx-28-II

4.5.3 Tx-46

The final test on the Dayton Superior products took place on specimen Tx-46. In order to develop limits on edge distance for placement of the hangers, the C-24 hanger hangers were placed 6" from the girder edge and spaced at 36" apart. The transverse reinforcing steel in the top flange were No. 3 bars spaced at 6" and 8". The overhang bracket geometry, hanger spacing, and reinforcing pattern are shown in Figure 4-46.
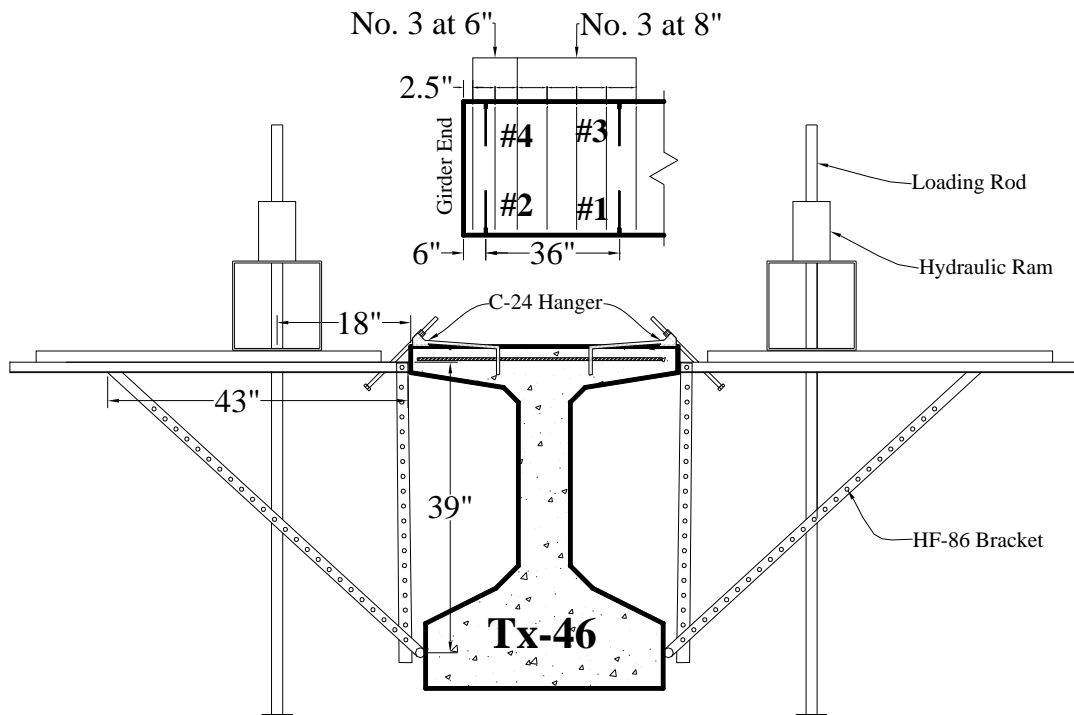
Figure 4-50: Reinforcement pattern and loading geometry for C-24 on Tx-46

The initial cracks in the top flange stemmed from the anchored portion of the hanger and propagated to the girder end as shown in Figure 4-51. The reinforcing steel strain gauges and visual observations indicated initial cracking at 4000 lbs. The maximum strain reading was almost 0.02 at the location nearest to the girder end at a ¼" maximum crack width illustrated in Figure 4-52. Failure occurred under an applied vertical load of 7000 lbs in a punching shear/tensile failure mode. The entire top flange corner separated from the girder at the #4 hanger location. Slight bending deformation was observed in the C-49 overhang brackets prior to the flange failure. Some evidence of spalling around the hanger

tip was observed after disassembly of the test setup and can be seen in Figure 4-51.

Figure 4-51: Punching shear/tensile failure at #4 C-24 Hanger on Tx-46
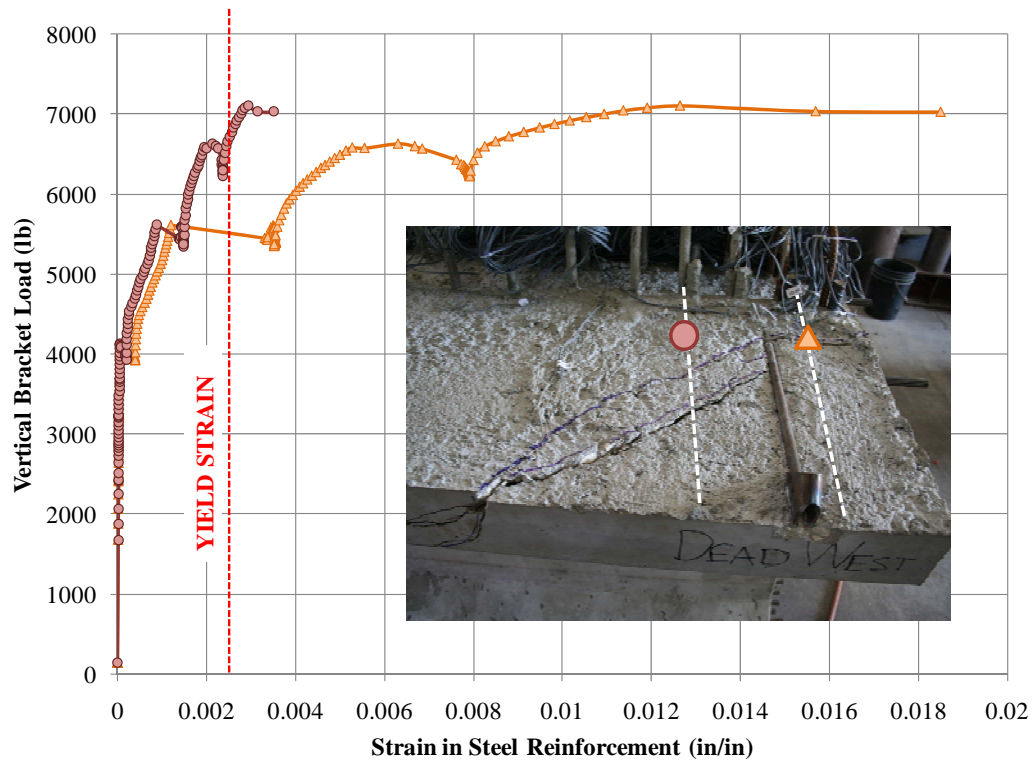
Figure 4-52: Tx-46 C-24 Reinforcement Strain at hanger #4

4.5.4   Summary of Results: C-24

The results from the C-24 tests are summarized in Table 4-4. The applied vertical load at cracking and ultimate, the ratio of the cracking load to the ultimate load, the failure mode, the location of the test region along the girder, the embed spacing, the top flange reinforcement in the test region, maximum reinforcement strain and the concrete compressive strength are all tabulated.

Table 4-4: Summary of C-24 Results

Embed	Girder	Cracking Load (lb)	Ultimate Load (lb)	Ratio of Cracking to Ultimate	Failure Mode	Test Region and hanger spacing	Top Flange Reinf.	Max Reinf. Strain	Girder f_c' (psi)	Photograph of failure zone
C-24	Tx-28-I	6500	7000	0.93	Bracket buckling	Mid-span - 49"	#3 bars at 12"	0.0001	13800	
	Tx-28-II	6800	8200	0.83	Excessive Coil Rod Deformation	24" from girder end 69"	#3 bars at 4"/12"	0.0006	11400	
	Tx-46	4000	7000	0.57	Punching Shear/ Tensile	6" from girder end 36"	#3 bars at 6"/8"	0.0190	13200	

Due to the different failure modes observed in the C-24 tests, direct comparison among the three tests is difficult; however, some pertinent observations follow:

(i)   The close proximity of a hanger to the girder end will reduce the applied load to cause tensile cracking. In the Tx-46 test, the lowest cracking load among all tests was observed due to the location of the hanger only 6" from the girder end.

(ii)   The failure modes of the first two tests were restricted to the brackets and hangers, not to the top flange of the girder. Thus, the thin flange of the Tx girder was less critical than the design of the bracket and hanger. Although the top flange did crack and the strain in the reinforcing steel was less than 15% of yield when the failure of the bracket system occurred.

4.6   RECOMMENDATIONS

The following recommendations are based on the test results presented in this chapter and the interpretation of those results.

It is recommended that any embed or hanger system used to support overhang brackets not be placed closer than 12" from the end of the girder. The HF-43 test on Tx-46 and the C-24 test on Tx-46 which both had an edge distance of 6" exhibited the lowest applied loads to cause cracking. If it is necessary to place a hanger within 12" from the end of the girder, a reduced allowable load for those hangers should be used in the design of the forming system.

It was decided to specify one maximum allowable load recommendation for all the tested products. It may have been possible to create slightly differing recommendations for each system investigated. However, one general load recommendation allows for a much simpler design procedure when the overhang bracket and embed system is not initially known at the design stage.

The maximum allowable load recommendation was based upon the factors outlined below:

- **Cracking of top flange is seen as limit state**
  In order to ensure a safe construction practice, the primary goal during construction is to prevent damage to primary structural components such as significant cracking of the concrete I-girders.

The experimental test results presented in this chapter show that cracks caused by the overhang bracket system are significant. Based on the data gathered through the use of strain gauges on the mild reinforcing steel placed transversely in the top flange of the girder, the initiation of tensile cracks stemming from the embedded portion of the hanger causes the stress in the reinforcement to rapidly increase often past yield stress. Therefore, the load supported on the overhang brackets should not cause cracking of the top flange. The two test results where the hangers were placed within 12" of the girder end were not included in the load recommendation because it is not recommended to place hangers near the end of the girder. For the eleven remaining results, the lower bound of the results of the applied vertical load at initial cracking was approximately 6200 lbs as shown in Table 4-5.

Table 4-5: Summary of cracking loads

Hanger Type	Applied Vertical Load to Cause Cracking (*lb*)
HF-43	6700
	8000
	~~6000~~
HF-43-V	6200
HF-43-I	6200
HF-67	6900
	7300
HF-67-M	7000
	8000
	6200
C-24	6500
	6800
	~~4000~~

- **Friction between overhang bracket and bottom flange cannot be relied upon during construction**
  As described in Section 4.2.1, the frictional resistance of the bracket kicker and concrete surface contributed to vertical load carrying capacity in the laboratory tests. Due to the inconsistent and unreliable nature of friction, it should not be accounted for when considering construction loads at a bridge site. Additionally, the screed or bridge paver that is riding along the overhang brackets has vibrating components which could effectively break any frictional resistance between the bracket support and the concrete girder. When friction exists in the system, the loads on the top flange are less than a case in which no frictional resistance exists. The exact amount of difference between the two situations (with or without friction) depends on the geometry of the system and the type of material of the bracket. Using engineering judgment, results from the test reported in this chapter, the load applied to the top flange is approximately 25% greater in a system where no frictional resistance occurs at the bottom flange reaction. Reducing the average cracking load appropriately, the applied vertical load to cause cracking would be approximately 5000 lbs in a system with no frictional resistance.
- **Construction loads should have a significant factor of safety**
  When discussing a system that is controlled by construction loads, some of which are unknown at the time of design, a considerable factor of safety need be applied. The 3000 lb allowable vertical load recommendation allows for a minimum factor of safety of roughly 1.65. This factor of safety is significant enough to account for the variability in the different systems and different geometries. A factory of safety of 1.65 is consistent with AASHTO LRFD Bridge Design Specifications (2007). AASHTO LRFD recommends a load factor for construction loads with any dynamic effects of 1.5. The resistance factor (ϕ factor) for flexure and tension is 0.9. The resulting factor of safety would be 1.67 consistent with the 1.65 factor of safety that is recommended.

The recommended allowable load of 3000 lbs was based on the tests conducted in this study. It is important to recognize that the applied vertical load to cause cracking is directly related to the tensile strength of concrete. The tensile strength of concrete is related to the compressive strength of concrete by the

square root of the compressive strength. The four beams tested in the study range in compressive strength from 11,400-13,800 psi. While the specified compressive strength of concrete for the Tx girders is most often much lower, typical prestressed concrete beam fabrication results in compressive strengths ranging from 11,000-14,000 psi due to the high specified release strength for the Tx girders. If a lower value of 28 day compressive strength is known, the 3000 lb applied vertical load recommendation can be adjusted accordingly following Table 4-6. However, the investigating team observed that nearly all prestressed concrete girders fabricated in Texas will have concrete compressive strengths near 12,500 psi and a load reduction typically not necessary. It is also important to appreciate the fact that the permissible load recommendation was based on the test results using a lower-bound approach. As can be seen in Table 4-5 a substantial percentage of the tests yielded a cracking load that is considerably higher than the 6200 lbs cracking load used to establish a permissible load of 3000 lbs. The use of lower bound approach results in inherently higher (higher than 1.65) factor of safety values for all cases except the lower bound. Although it is difficult to quantify statistically, it is believed that the occurrence of a low data point in conjunction with a low compressive strength in the field beams is considered to be unlikely –not impossible.

Table 4-6: Load Recommendation adjustment for girder concrete strength

Compressive Strength (psi)	Allowable Vertical Load on Bracket (lb)
11500	3000
10500	2900
9500	2700
8500	2600
7500	2400

A further recommendation for any Tx girder that will receive some type of hanger to support overhang brackets should have top flange reinforcement of No. 3 bars spaced at 6" maximum. The 6" value was derived based on the crack patterns observed in the experimental testing. Because most cracks initiated at the

embed portion of the hanger then propagated toward the edge of the top flange, a transverse reinforcement spacing of 6" is a conservative spacing value that will ensure steel is present should cracks form.

The implications of the 3000 lb vertical load recommendation mainly relate to the horizontal spacing of the bracket systems along the length of the fascia girders. Using the load estimates in Table 3-1 and a screed load of 1900 lbs per bracket, the bracket spacing was estimated (screed load based on Bid-Well 4800 Bridge Paver observed in use at Texas bridge construction site). A bracket spacing of 3 foot 6 inches would be sufficient for use with a typical 3 foot overhang (measured from the girder centerline). The 3'-6" spacing is only an estimate based on typical conditions and exact bracket spacing should be calculated for the particular site conditions including screed weight, overhang length and depth, formwork, and additional live load.

4.7   SUMMARY

Thirteen different load tests were performed on four different Tx girders. Three main types of overhang forming systems were examined in the study. The results of the tests have indicated that cracking of the top flange should be considered the limit state. For use of the overhang forming systems on the Tx girder type the following recommendations apply:

1. A maximum of 3000 lbs be applied vertically to any one overhang bracket and embedded hanger.
2. No hanger should be placed within 12" of the girder end.
3. The reinforcement pattern in the top flange should consist of No. 3 bars spaced at 6" maximum for Tx girders with attaching overhang hangers and brackets.

CHAPTER 5
Precast Overhang

5.1 INTRODUCTION

The testing of overhang forming systems currently being used in bridge deck construction was discussed in the previous chapters. A new procedure for overhang construction on precast concrete I-girder bridges is discussed in this chapter. The procedure outlined in the following sections can merely be considered as the first step towards further innovation and improvement. Additional refinement and investigation of the new overhang construction process discussed herein will lead to a more efficient construction practice.

The idea for a precast overhang was initiated by tests conducted at the Phil M. Ferguson Structural Engineering Laboratory on bridge overhang brackets and hangers with the new family of Texas I-Girders. Through experimental testing on the performance of the bracket/hanger systems, several disadvantages to the current practice of overhang construction were observed. Following previous innovation in bridge deck construction with the introduction of precast concrete deck panels as stay in place formwork, a precast solution was developed for construction of overhangs on prestressed concrete I-girder bridges.

5.1.1 Current Practice

In order to truly appreciate the potential benefits of a precast overhang system, the advantages and disadvantages of the precast solution and the overhang bracket system are outlined. The overhang bracket system referred to in this section is the system of bridge overhang brackets and embedded hangers as outlined in Chapter 2 and shown in Figure 5-1.

Figure 5-1: Currently used overhang forming system

5.1.1.1 Disadvantages

- **Time:** The installation and disassembly procedure for the overhang brackets is very time consuming. Time required to properly adjust the brackets and to form overhang at bridge site is significant.
- **Cost:** Process demands significant resources in the formwork, crane time, and form stripping buggy. Brackets and non-reusable embed hangers are costly. Labor cost of installation, forming, and disassembly is also significant.
- **Lack of Stiffness:** Overhang brackets and formwork undergo noticeable deflections during the casting operation that may lead to uneven finished surface in some cases.

5.1.1.2 Advantages

- **Contractor Flexibility:** Bracket system allows for last minute changes to overhang design at the bridge site.

- **Current Practice:** The system is in use today and has been for many years. All problems have been observed in practice and solutions have been found.

5.1.2   Proposed System

The proposed system involving a precast overhang solution is shown in Figure 5-2 and will be described in detail in the following sections. The advantages and disadvantages will become clearer as the process evolves and is implemented; however, potential advantages and disadvantages are outlined as follows as an initial comparison.

Figure 5-2: Precast overhang on Tx70 girder

5.1.2.1 Disadvantages

- **New concept:** As with any new procedure in construction, there exists a significant learning curve to refine and optimize the process.
- **Plan time:** Portion of overhang constructed at the fabrication yard will require that certain characteristics of the bridge geometry be determined with enough lead time to be applied in the fabrication yard.

5.1.2.2 *Advantages*

- **Cost:** While not clear at this time, the new construction technique should lead to a more efficient and ultimately cheaper construction process and result in a more economical bridge. A similar economy was achieved when the use of precast stay-in-place panels was implemented.
- **Stiffness:** The precast concrete overhang is a very stiff element and will not deflect significantly during the finishing process leading to a smooth riding surface.
- **Time:** Eliminating the need for assembly and disassembly of overhang brackets and wooden formwork, greatly reduces the time required to construct the bridge deck.

5.2 BACKGROUND

Prior to the in depth explanation of concept of creating a bridge overhang using a precast overhang, the general procedure is outlined as follows:

1. Tx fascia girders are fabricated in a precast plant.
2. After a girder is cast and released, it is moved to overhang formwork where a 4" thickness of overhang deck is cast onto fascia girder.
3. Girder is transported to a bridge site and lifted into place.
4. Work platform is hung from precast overhang and side form is placed on the platform
5. Topping slab is cast over interior deck panels and precast overhang.
6. Bridge traffic barrier is cast on overhang
7. Any finishing (e.g. rubbing etc.) is completed and work platform is removed

5.3 GIRDER FABRICATION

The initial step in any prestressed I-girder bridge construction is the actual girder fabrication at a prestressed concrete fabrication plant. The typical process for girder fabrication will be followed (string and stress strands, tie cage, place formwork, pour concrete, remove formwork, and release strands).

5.3.1 Tx Girder

The entire process of incorporating a precast overhang is centered around the use of the Texas Department of Transportation design for the new family of prestressed I-girders referred to as the "Tx" girder. The wide top flange of the new girders lends itself to the application of the precast overhang system described herein. The overhang is still measured from the girder centerline; as such, the wider top flange correlates to less of an overhang past the girder edge, i.e. the tip of the top flange. The calculations and recommended procedure presented is based on the use of Tx girder; however, the principles could be applied to Type IV, Type C, or other prestressed I-girders.

The only alteration to the girder during the initial fabrication is the addition of one bar type (Bar Type O) shown in Figure 5-3 for the overhang. The bar will be anchored deep into the web of the girder and provide a tie down for the precast section of the overhang. The bar will also serve as the flexural reinforcement for the overhang. The current TxDOT typical details were consulted when planning the reinforcement placement (TxDOT Detail SIG, 2007). The placement of the Bar Type O was slightly altered from the current details to increase the strength of the precast overhang by placing the bar on top of the longitudinal overhang reinforcement. Bar Type O is positioned 2.8" from the top surface of the girder. Bar Type O will replace the transverse reinforcement in the bottom of the overhang from the standard TxDOT bridge deck details and is recommended to remain at the same spacing at a 6" maximum (TxDOT Detail SIG, 2007). Figure 5-3 shows the placement of the additional bar on a Tx-70 with a typical 3'-6" overhang. Both legs of the Bar Type O are the same length so as to minimize error at the fabrication yard.
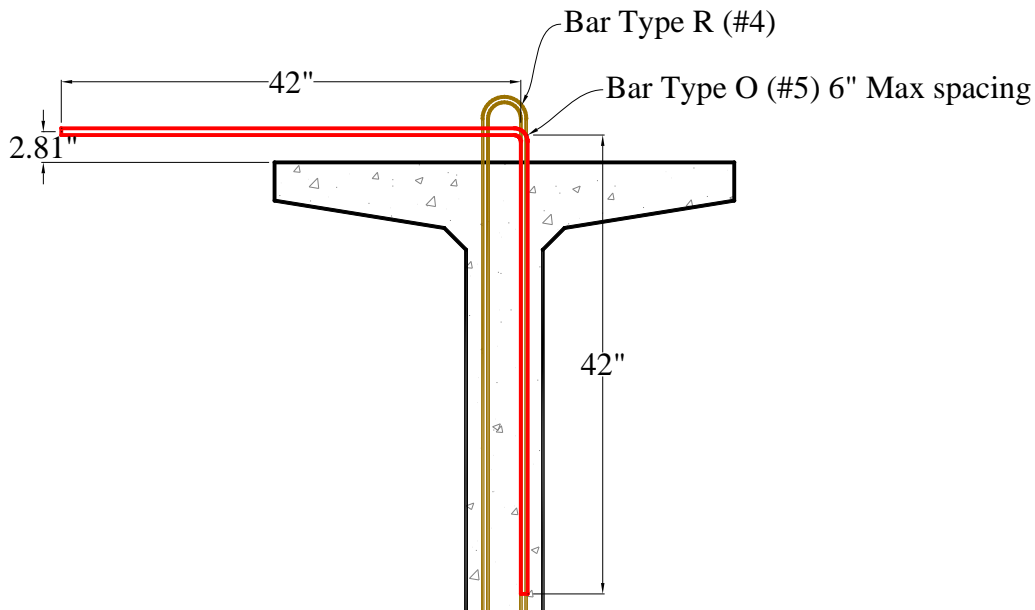
Figure 5-3: Placement of Bar Type O in Tx girder design

5.3.2 Overhang

After the special fascia girder is fabricated, the next step is to form and cast 4" of the overhang extending from just inside the girder centerline to the edge of the finished overhang. The system of formwork that is to be used should be designed by the fabricator and formwork manufacturer to best optimize the process with certain characteristic features.

5.3.2.1 Girder Profile

Due to the eccentrically applied prestressing force, a camber or initial upward deflection will exist in all prestressed I-girders. The amount of the camber is highly variable and impossible to predict accurately. It is recommended to match the girder camber with the overhang. With the overhang following the same profile of the girder, the overhang reinforcement (Bar Type O) will remain at the same depth along the length of the girder. Also, the aesthetic impact is minimized by matching the camber because all the lines will follow the same path.

5.3.2.2 Bridge Profile

Precast I-girder bridges are used in a wide variety of geometrical situations. There often exists horizontal curves, vertical curves, and cross sloping grades. The overhang forming process can become more difficult with a bridge of complex geometry. With a well planned out formwork system, the complex geometries can be accounted for in the overhang at the precast yard. It is recommended to use the precast overhang concept on bridges with simple geometry before this concept is applied more complex situations.

5.3.2.3 Forming

Significant effort is required in the design of a formwork system at the precast yard to make the process as efficient as possible resulting in a more economical product. The following are few conceptual ideas for a formwork system to meet the requirements.

5.3.2.3.1 Table Form

- The "Table" serves as the bottom form for the overhang
- Girder is placed next to formwork; formwork is stationary
- Majority of the table may consist of concrete blocks; steel form is placed on top of the blocks
- Steel form can be adjusted to fit camber and cross sloping grade
- Edge form flexible to match horizontal curves

Bexar Concrete Works showed the investigating team how similar formwork is in use to precast the overhang portions of the box-beam railroad bridges. The existence of such a system and the confirmation obtained from the beam fabrication plant regarding the practicality of the concept was found to be encouraging.

5.3.2.3.2 Cantilever Form

- System involves formwork that cantilevers from girder top flange
- Formwork automatically fits girder camber because form is support by girder
- Bolts through the top flange could support the formwork
- Long segments would allow for simple assembly
- System does not require new work area and can be used anywhere

5.3.2.4 Overhang Fabrication

Once overhang is formed according to the specific details of the bridge plans, the typical additional reinforcement is placed. The typical detail calls for three #5 bars (D_1) to be placed longitudinally in the overhang in addition to the transverse #5 bars that are already embedded in the girder (Bar Type O). Additional bars are necessary for the bridge rail that will be cast on the bridge. Bar Type U from the Single Sloped Traffic Rail (SSTR) is shown in Figure 5-4 as it would be added in the cast of the overhang. Four inch thickness of the overhang is then cast and the top surface is left roughened for improved shear friction of the topping concrete to be cast at the bridge site. Chamfer strips are added both at the bottom and top edges of the overhang to improve the appearance of the finished edge.
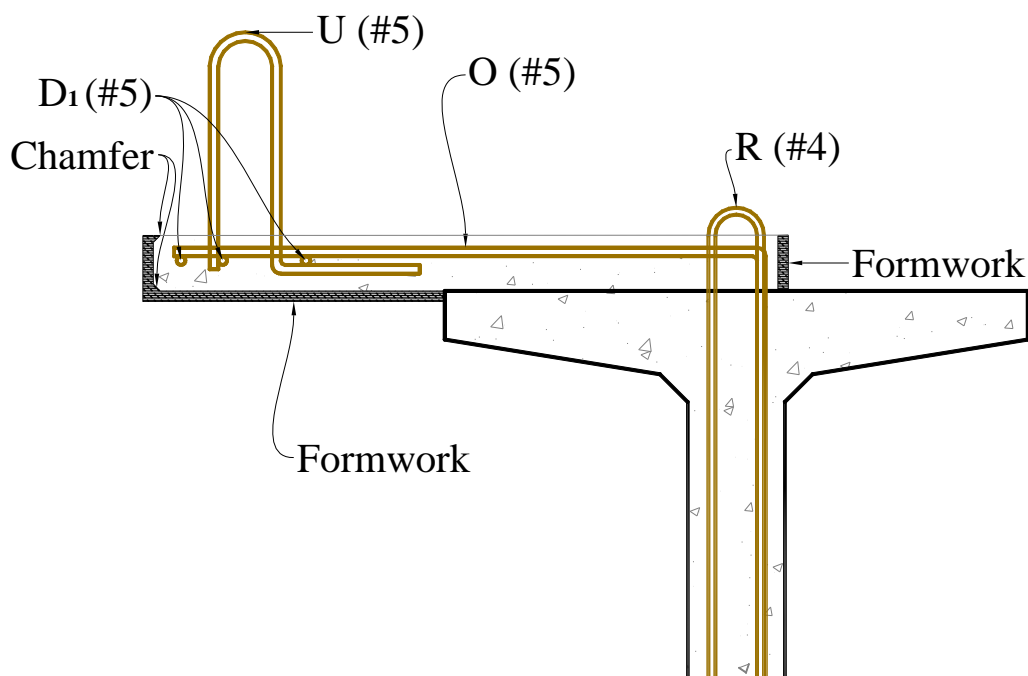
Figure 5-4: Reinforcement in Overhang

5.4 LIFTING AND TRANSPORTATION

The lifting and transportation need to be planned carefully due to the shift in the center of gravity after the 4" thick overhang is precast. The center of gravity

with the precast overhang was calculated for various girder sizes with typical overhang lengths as displayed in Table 5-1. The average horizontal location of the center of mass is 3.07" away from the girder centerline which remains within the 7" wide web. The center of gravity being within the web, allows for a lifting loop to be inserted roughly at the location of center of gravity. Doing so will greatly reduce the tipping effect during lifting. Figure 5-5 shows the lifting strands inserted just outside of the stirrups (Bar Type R). If the girder needs to be moved prior to the casting of the overhang, normal lifting strands should be inserted at the centerline of the girder.

Table 5-1: Horizontal Eccentricity due to Precast Overhang

	Girder Area	Overhang Length	Horizontal Eccentricity
	(in²)	*(ft)*	*(in)*
Tx28	585	3	**3.46**
Tx34	627	3	**3.27**
Tx40	669	3	**3.10**
Tx46	761	3	**2.79**
Tx54	817	3	**2.63**
Tx62	910	3.5	**3.21**
Tx70	966	3.5	**3.05**
		Average =	**3.07**
		Maximum =	**3.46**
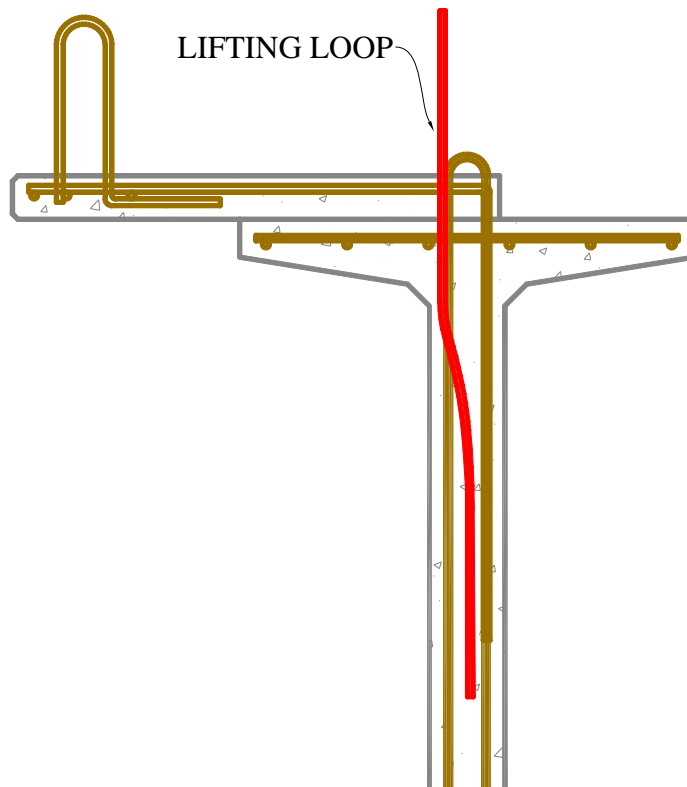
Figure 5-5: New position for additional lifting loops

5.4.1   Stability

The overall stability of the girder with the precast overhang was also considered with regards to placement on the ground and final placement on elastomeric bearing pads. Calculations show when the standard fascia girder is set on the ground, the factor of safety for overturning is well above 20 in all girder types. When the girder with typical precast overhang is supported on 21" wide elastomeric bearing pads (TxDOT standard pads), the factors of safety for overturning are reduced yet remain in the safe range as shown in Table 5-2 with a minimum factor of safety of 6. The factor of safety corresponds to ratio of the restoring moment due to the weight of the girder to the overturning moment caused by the precast overhang. Figure 5-6 illustrates the factor of safety when the beam is placed on bearing pads. Factors of safety shown in Table 5-2 can be reduced even more when wind forces and the non-linear behavior of bearing pads is taken into account. The determination of factors of safety against overturning due to the previously mentioned factors goes beyond the scope of this study.

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Table 5-2: Factor of safety against overturning
(girders placed on 21" wide bearing pads)

	Factor of Safety
Tx28	**6.8**
Tx34	**7.2**
Tx40	**7.7**
Tx46	**8.8**
Tx54	**9.4**
Tx62	**6.0**
Tx70	**6.4**
Average	**7.5**
Minimum	**6.0**

Overturning Moment ≈ 1 unit

Average Restoring Moment ≈7.5 unit

Tipping Point
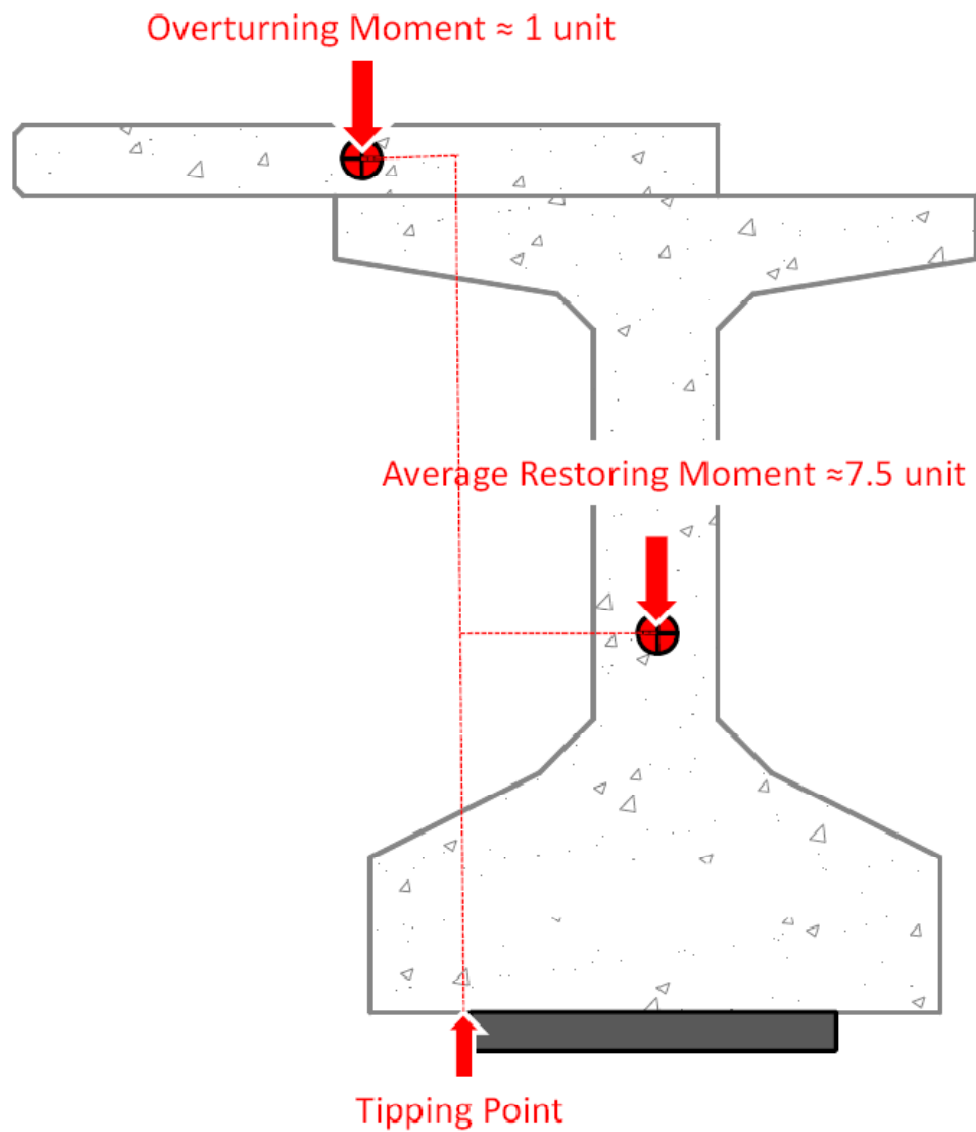
Figure 5-6: Overturning moment and restoring moment

5.4.2 Transportation

Due to the unusual shape of the girder with the precast overhang, the transportation of the fascia girders may require extra precaution. The best transportation method should be determined by the hauling company. A proposed possible solution would be to leave a small hole in the web during the casting of

the girder through which a connection device could be passed to better secure the girder to the truck. As it is the customary practice, the transportation of the girder needs to be examined further by those who will be responsible for transporting the girders.

5.5 BRIDGE DECK CONSTRUCTION

The next step in the process is the construction of the concrete deck. A typical bridge span consists of four to six Tx girders that are topped with 8 inches of bridge deck (4" of precast concrete plus 4" of cast in place concrete). The fascia girders are important in the construction of the deck as they typically support the finishing screed, work platform, and edge form as described in Chapter 2. All three requirements must be taken into account when planning the construction process using the precast overhang.

5.5.1 Screed Rail

In current practice, the finishing screed rides on a rail that is supported on overhang brackets hung from the fascia girders. Chapter 2 includes additional details on the current deck construction procedure. Remaining consistent with current practice, the screed is recommended to be supported on the precast concrete overhang at least 6" from the edge. The actual support piece to be used to support the screed rail on the precast overhang will need to be altered slightly from the current design, yet still meeting certain requirements:

- Support base is a reusable product to keep costs down
- It must allow for adjustment while screed in place
- The base must be stable to withstand movement of finishing screed

Using the requirements listed, a simple support system can be developed for applications on precast overhangs by concrete accessory manufactures.

5.5.2 Work Platform

During any bridge deck construction it is necessary to have an area along the bridge where workers can walk that does not interfere with the concrete casting operation. Safety standards also require a fall protection barrier that is usually a wooden rail attached to the work platform. In current practice the work platform and hand rail are supported on overhang brackets that also support the overhang formwork. The precast overhang eliminates the need for an additional heavy support for the cast in place concrete and finishing screed; however, the

need still exists to support a work platform. The necessary load to support consists only of the weight of the platform and construction workers. A recommended solution is shown in Section 5.6.2.2.

5.5.3   Edge Form

In order to create a smooth finished edge on the full depth overhang, an edge form needs to be in place to confine the fresh concrete to the bridge deck. Two alternatives are proposed as solutions, both with their own advantages and disadvantages.

5.5.3.1 Alternative #1

The first solution follows the current practice and involves a wooden edge form. The edge form would need to be supported along with the work platform. The major disadvantage of the wooden edge form is that it still involves labor in the forming process as well as the cost of the wood when it is replaced. In a meeting with the TxDOT project monitoring committee, the association of general contractors, and the investigating team, several contractors expressed the desire for a solution that did not require any wood forming.

5.5.3.2 Alternative #2

The second solution calls for a concrete edge form to be cast on the overhang during the fabrication of the precast overhang as shown in Figure 5-7. The second alternative solves certain problems; however, it also creates a potentially greater problem. The main problem that is created is to what depth to cast the edge piece. The problem is explained in great detail in the following section.
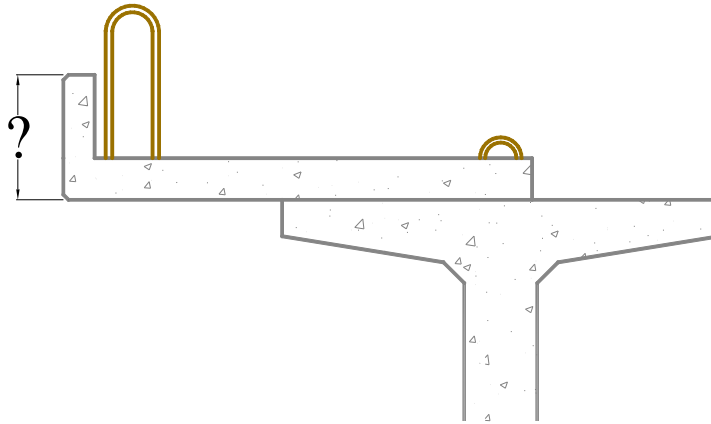
Figure 5-7: Concrete edge form on precast overhang

Significant surveying and planning of the geometry of bridge decks is necessary to create the finished bridge. One of the issues with bridge deck geometry is the variability of I-girder cambers. For a given span, each of the many girders may have a different camber; perhaps a difference of 1 to 3 inches in cambers may exist. TxDOT standard details call for a minimum deck thickness of 8 inches (TxDOT Detail SIG 2007). In order to minimize the use of excess concrete, the interior precast deck panels used as stay in place formwork are supported at different heights on bedding strips (TxDOT Detail PCP 2006). These strips even out the differential camber and create a fairly uniform 8" deck thickness the length of the span. This technique works very well with interior precast panels; however, the same procedure could not be applied to the exterior overhangs because the overhang is precast onto the girder.

In order to calculate the exact necessary height of the concrete edge form, the maximum camber of any girder in the span must be known. Figure 5-8 illustrates the correct edge form height on an example bridge where the maximum girder camber is 3".

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11.5"

8"

$$h_{endspan} = 8" + 0.5" + \Delta_{max}$$

$$h_{endspan} = 8" + 0.5" + 3" = 11.5"$$

AT BENT

10.5"

8"

$$h_{midspan} = 8" + 0.5" + \Delta_{max} - \Delta_{fascia}$$

$$h_{midspan} = 8" + 0.5" + 3" - 1" = 10.5"$$

MID-SPAN

1" CAMBER

3" CAMBER

1½" CAMBER
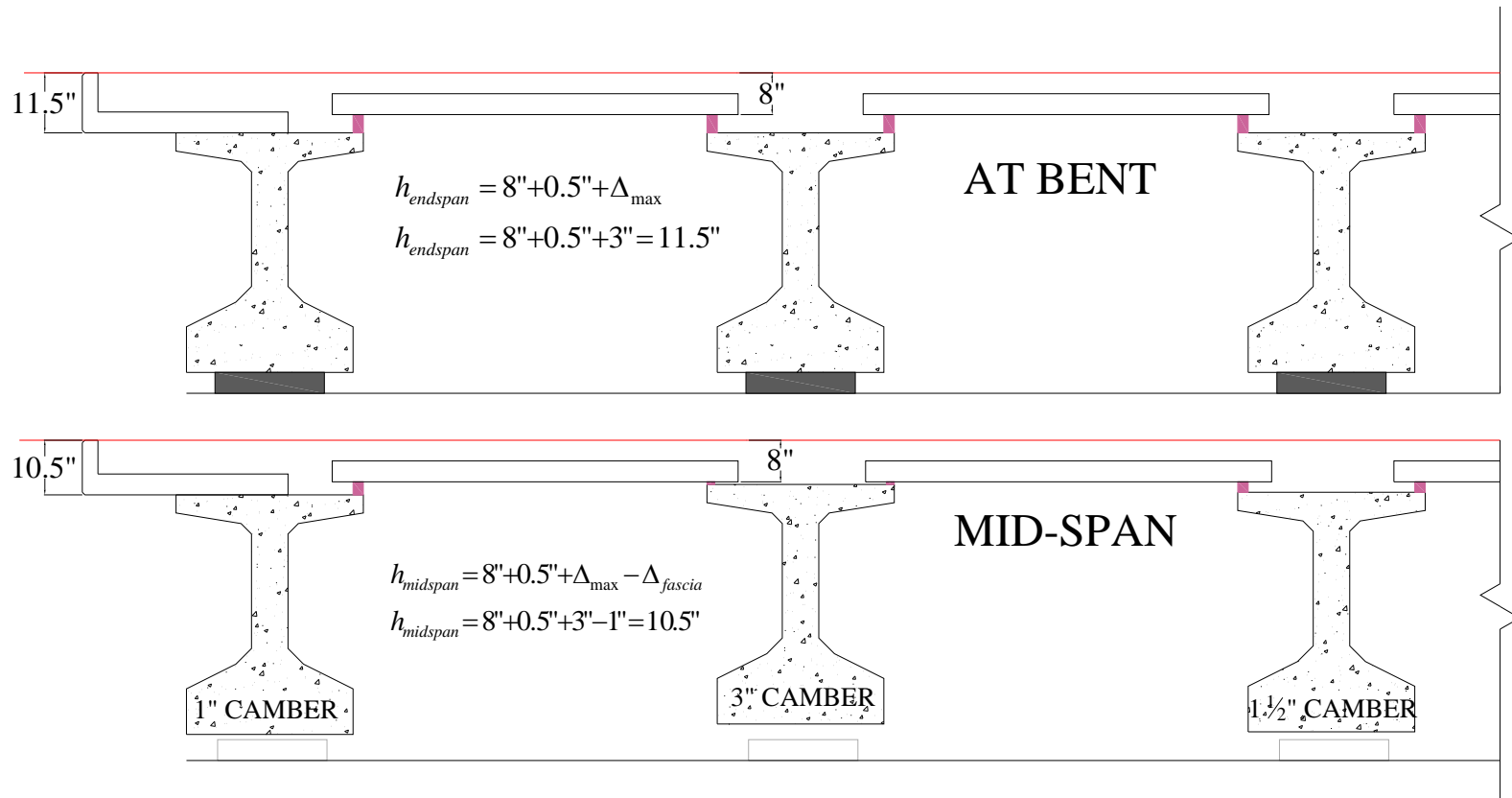
Figure 5-8: Example bridge span cross section with concrete edge form

Equation 5-1 and Equation 5-2 are given to calculate the required edge form height at the mid-span and at end of girder span at the bent cap. The two heights are not the same because the camber of the girders does not affect the top of girder elevation at the bent. The equations given are for a flat straight bridge span. If a cross slope or vertical curve exists in the span, the equations become significantly more complicated and thus the required height of the edge form is more difficult to estimate.

$$h_{midspan} = 8" + 0.5" + \Delta_{max} - \Delta_{fascia} \qquad \textbf{\textit{Equation 5-1}}$$

$$h_{endspan} = 8" + 0.5" + \Delta_{max} \qquad \textbf{\textit{Equation 5-2}}$$

where;

$h_{midspan}$ = *required height of edge form at mid-span of girder*

$h_{endspan}$ = *required height of edge form at end of girder at bent*

Δ_{max} = *maximum camber from any girder in span*

Δ_{fascia} = *camber of fascia girder*

The required height of the edge form could be estimated based on maximum expected girder camber; however, the estimates will inevitably be incorrect resulting in a lower than required height or a higher than required height. Having the edge form larger than necessary could be acceptable in some situations in which certain bridge rail barriers such as the SSTR or T501 are used where the remaining edge form could be covered as shown in Figure 5-9. However, some bridge rails require drainage slots or simply a flat surface at the overhang edge. For those situations, the larger than necessary edge form would not work as illustrated in Figure 5-9. A possible solution would be to attempt to under estimate the necessary edge form height by a couple of inches, and during the casting operation, simply taper the finished concrete surface down to match the edge form height using hand finishing. This solution would allow for drainage as shown in Figure 5-10. Though this solution may not be feasible, the concept may be useful in future research, design or implementation
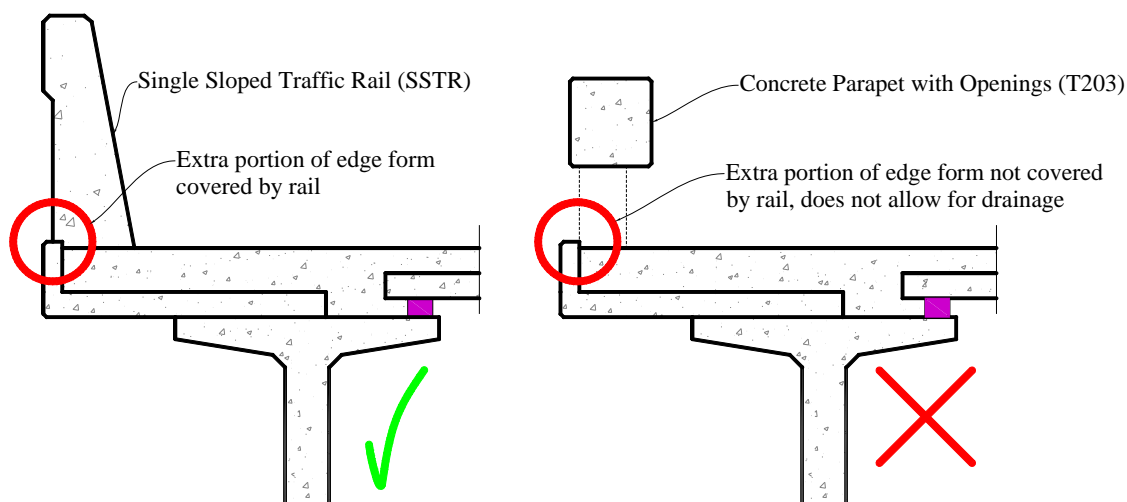
Figure 5-9: Examples of when the edge form can extend past the topping slab
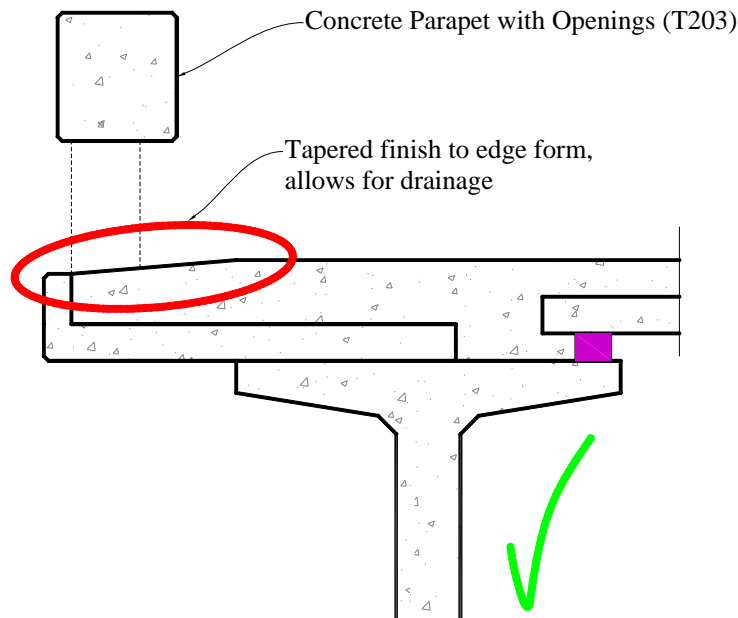
Figure 5-10: Concrete edge form with tapered finish

At this point, the recommendation will remain with alternative #1 with the standard wood edge form support on the work platform. Further investigation and design with alternative #2 is necessary before its feasibility for most, if not all existing cases can be proven. A field study of bridge deck construction in which measurements are taken of the necessary edge form thickness would be very valuable to the further planning of the alternative.

5.5.4 Construction

Once the work platform, screed rail support, and edge form are in place, the remainder of the work follows the same process as current bridge deck construction. Surveying and placement of precast panels, placement and tying of all mild reinforcing bars, and surveying of finishing screed height are all necessary steps prior to the concrete being poured on the bridge deck. After the concrete is cast and hardened, a second pour is typically necessary to add the rail barriers along the overhang. This process is outlined in more detail in Chapter 2.

5.6 EXPERIMENTAL PROGRAM

In order to examine the feasibility of the precast overhang concept, a fascia girder with a precast overhang was fabricated in a precast beam

manufacturing plant and tested at the Phil M. Ferguson Structural Engineering Laboratory. The primary goals of this portion of the project were to identify any unforeseen issues with the process in addition to the verification of strength and stiffness of the precast overhang through experiments.

5.6.1 Fabrication

A 30 foot long, Tx70 prestressed I-girder was fabricated at *Texas Concrete Co.* in Victoria, Texas. The overhang reinforcement, Bar Type O (#5), was added to the girder during fabrication. The bars were bundled with the transverse shear reinforcement at a maximum spacing of 8 inches. Details of the fabricated girder are shown in Appendix B.

Using available resources, a set of formwork was created in the laboratory in order to cast the overhang. Simulating the table forming process outlined in Section 5.3.2.3.1, the formwork was set on top of a large concrete slab as shown in Figure 5-11. No significant effort was made to match the girder camber with the formwork due to the very small amount of camber that existed in the girder because of the short length. Typical bridge spans will be much longer and thus have larger camber that need be met by the formwork system.

Additional reinforcement was added to the overhang following Figure 5-4. The actual test specimen reinforcement is shown in Figure 5-12. TML Strain Gauges from Tokyo Sokki Kenkyujo Co., Ltd., Type FLA-5-11-5LT, were applied on the transverse overhang bars. The critical section was determined through calculations to exist at the interface between the overhang and the edge of the top flange as such strain gauges were applied at that section (Figure 5-12).

Figure 5-11: Tx-70 fascia girder placed next to overhang formwork

Figure 5-12: Additional reinforcement added to test specimen

Once all the reinforcement and instrumentation was in place, 4" of concrete was cast in the overhang formwork. The specified compressive strength for ready-mix concrete was 6000 *psi*. When the overhang is cast at a fabrication yard, it is likely that fabricators will use Type III cement because most of the products at a precast plant are made with Type III cement. The resulting 28 day compressive strength will likely be between 10000-14000 *psi*. The cylinder compressive strength of the girder fabricated in the laboratory was 6600 psi simulating a slightly more critical case than if the overhang were cast with a concrete mixture design utilizing Type III cement.

During casting, the top surface of the overhang was intentionally roughed to improve horizontal shear transfer when the topping concrete is cast at the bridge site. The top surface of the test specimen is shown in Figure 5-13. The formwork was later removed and the completed precast overhang is shown in Figure 5-14.

Figure 5-13: Top surface of overhang

Figure 5-14: Finished precast overhang

5.6.2 Bridge Deck Construction Study

After completion of the fascia girder with precast overhang in the laboratory, the methods for bridge deck construction were examined further to test ideas for solutions and identify unforeseen issues.

5.6.2.1 Screed Rail Solution

An important function of the precast overhang is to support the screed rail on which the finishing screed rides. The superior strength and stiffness of the concrete overhang allows for the screed rail to be supported directly on the overhang surface. A screed base is necessary to support the rail. As stated in Section 5.5.1, the support base must be reusable to keep costs down, allow for adjustment when finishing screed is in place, and be stable when sitting on a concrete surface. Simple modifications can be made to the current system presented herein to meet these characteristics. Figure 5-15 shows an example of a possible solution that meets the requirements.

1. In order to meet the desired finishing height, the nut on the coil rod can be adjusted while the finishing screed sits on the rail.
2. The feet at the bottom provide a stable base.
3. The screed rail can be disassembled and base can be removed from the topping concrete before it sets.

While Figure 5-15 shows a base that would work, a concrete accessory manufacturer could best engineer a product for this application.

Figure 5-15: Screed rail base

5.6.2.2 Work Platform Solution

As outlined in Section 5.5.2, a work platform is necessary for the deck casting operation and because there exists no typical overhang brackets on which to place a work platform, a method to support the platform must be created for the precast overhang. A suggested solution involves using cantilevered support arms extending from the underside of the overhang on which a work platform and hand railing are placed. The solution would involve creating a hole in the overhang during the fabrication process through which a coil rod could be passed to support the cantilevered arms. The support arms used in the laboratory are shown on the test specimen in Figure 5-16. The members shown are 3" x 3" x ¼" tube sections and are used only for the conceptual study. The members should be designed to optimize the strength and use of material. Better solutions may involve other cross section such as back to back channels and possibly the use of a light weight material such as aluminum. The coil rod connection may also be refined as to better fit the application. Currently, the solution involves a ¾" headed coil rod that passes through a hole in the overhang. The coil rod supported on the top side with a large plate washer and nut as shown in Figure 5-17. The coil rod passes through the support arm and is tightened into a coil nut welded underneath the support arm allowing for tightening from the top side. It may be argued that this solution is the same as the currently used overhang brackets; however, this solution vastly improves upon the brackets because there is no adjustment necessary for different girder types and the spacing of support arms can be much

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larger. The only loads that the support arms need to support are construction workers and weight of the platform. The spacing of support arms can be optimized to be 8-10 feet whereas overhang brackets are typically spaced at 3 feet.

Figure 5-16: Support arms for work platform

Figure 5-17: Top side of support arm connection

The support arms can be attached to the overhang prior to final placement of the girder on the bent caps or potentially after the girder has been placed to allow for quicker erection times. Once in place, the work platform can be set and secured to the support arms. Figure 5-18 shows a work platform and safety rail added to the support arms in the laboratory test specimen. The edge form is attached to the work platform as recommended and is shown in Figure 5-18. After the casting is completed and the work platform is ready to be removed, the platform can be easily lifted off the supports onto the finished bridge surface. If lubricating grease is applied to the coil rods prior to casting, the rods can be easily removed from the top surface and the support arms can be lifted onto the bridge. It will not be necessary to use a work buggy that hangs off the overhang to remove the supports. The remaining hole in the overhang where the coil rod was removed can be filled in from the top side with grout or is automatically covered with some bridge rail types.
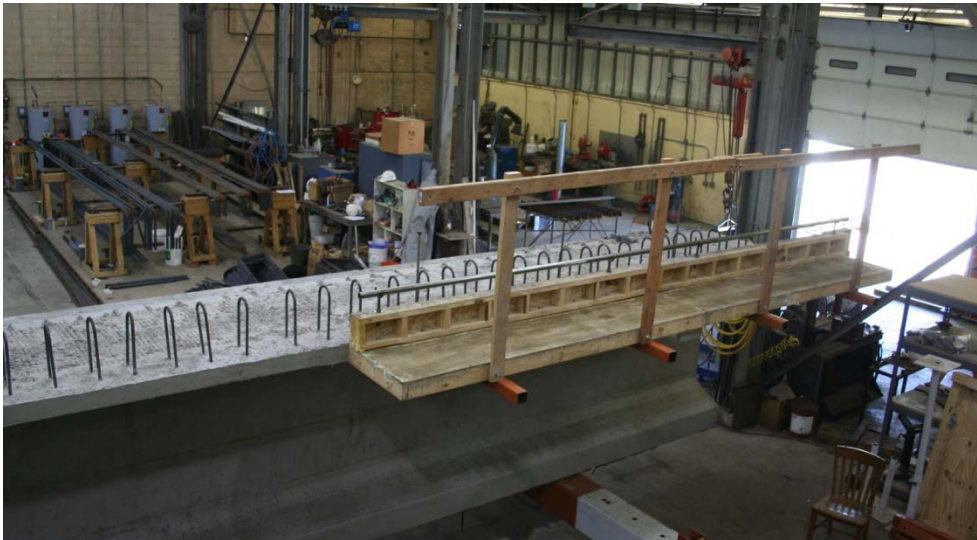
Figure 5-18: Work platform and hand rail on support arms

5.6.3 Load Testing

To verify the strength and stiffness of the precast overhang, two tests were performed on the test specimen fabricated in the laboratory. The same testing frame and procedure as the overhang bracket tests outlined in Chapter 3 were used in the testing of the precast overhang. In order to balance the load applied to the precast overhang, standard overhang brackets were attached to the inside of the fascia girder and loaded simultaneously with the precast overhang. An overview of the test setup is shown in Figure 5-19. On the precast overhang, two 3" x 3" x ¾" plates were used as loading points placed 6" from the overhang edge and 36" apart.
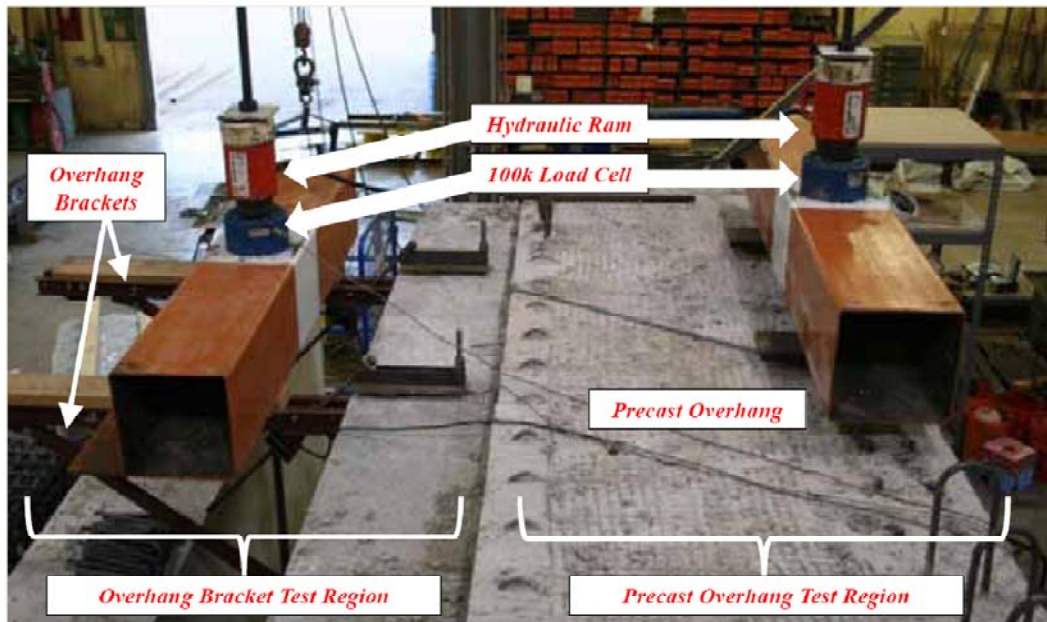
Figure 5-19: Overview of test setup

Calculations were completed to predict the applied vertical load that would cause cracking of the precast overhang. In order to determine the length of the overhang that was engaged by a point load applied near the edge, a 2:1 relationship for the dispersion of stress was assumed as shown in Figure 5-20. The vertical applied load per load point to cause cracking was calculated at 1625 lbs and is shown in

Table 5-3. This prediction turned out to be noticeably conservative with the experimental cracking load being approximately 6300 lbs. By observing the cracking pattern on the precast overhang and the strains in the overhang reinforcement, some factors were identified as causes for the underestimation of the cracking load as follows: (i) A much greater length of the precast overhang was engaged by each point load. A 1:1 relationship for the dispersion of stresses could be more representative of the experimental results; (ii) Given the proximity of the two loading points along the length of the beam (36 in), the two loading points acted as a group, spreading tensile stresses even more. Flexural cracks extended between the two point loads and further out as can be seen in Figure 5-21; (iii) The contribution of the overhang reinforcement was not taken into account. Accurate prediction of the cracking load for the precast overhang is

considered to be beyond the scope of this study and future investigations could determine more appropriate procedures for estimating the cracking load with a better degree of confidence. Nevertheless, the experimental cracking load carries a greater degree of significance than the predicted cracking load as will be demonstrated in the subsequent sections.
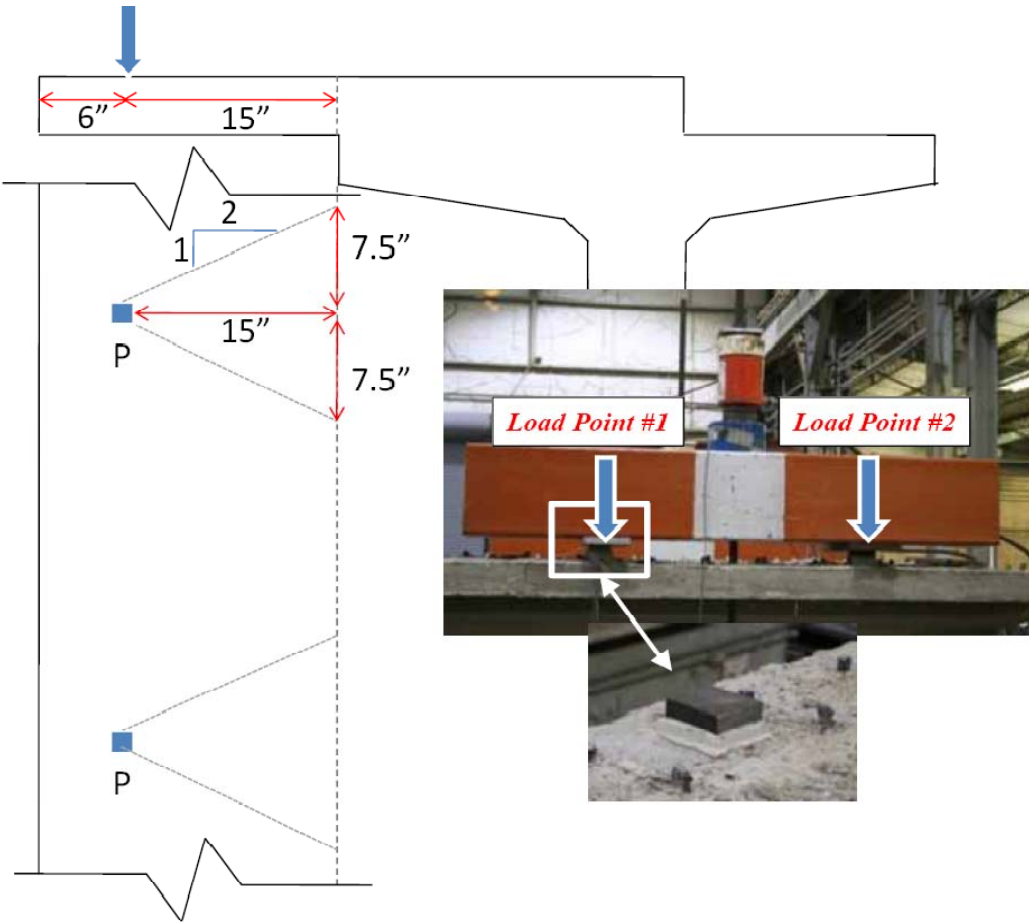

*Figure 5-20: Diagram of loading points and dimensions*

Table 5-3: Calculations of cracking load on precast overhang

$$f'_c = 6600 \, psi$$

$$\sigma_{cracking} = 7.5 \cdot \sqrt{f'_c} = 609 \, psi$$

$$I = \frac{b \cdot h^3}{12} = \frac{(7.5in + 7.5in) \cdot (4in)^3}{3} = 80in^4$$

$$\sigma = \frac{M \cdot c}{I} \Rightarrow M_{cracking} = \frac{\sigma_{cracking} \cdot I}{c}$$

$$M_{cracking} = \frac{609 \, psi \cdot 80in^4}{2in} = 24372 \, lbs \cdot in$$

$$P_{cracking} = \frac{M_{cracking}}{15in} = 1625lbs$$

In both tests, the precast overhang cracked under approximately the same vertical load of 6300 lbs per load point (total of 12600 lbs on precast overhang). The overhang cracked at the location where the 4" overhang meets the top flange of the girder which is 21" from the overhang edge as shown in Figure 5-21. The strain gauges were located at the crack location on the transverse overhang reinforcement. The relationship between strain in the reinforcement and the applied vertical load on each load point is illustrated in Figure 5-22. The plot shows a clearly defined cracking point around 6300 lbs. The applied load continued to increase to over 10000 lbs when the test was concluded. The overhang was not taken to failure. At the maximum applied load of 10000 lbs per load point, the strain in the transverse reinforcement was approximately half of yield strain.
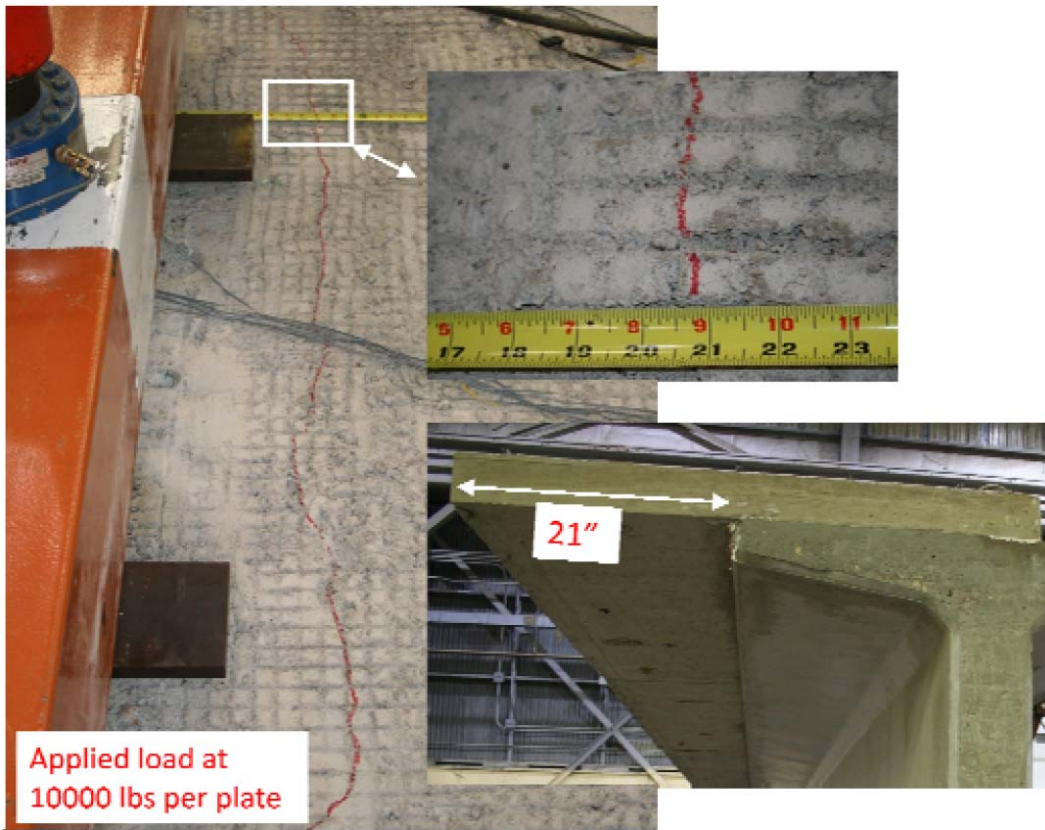
Figure 5-21: Crack at intersection of overhang and top flange
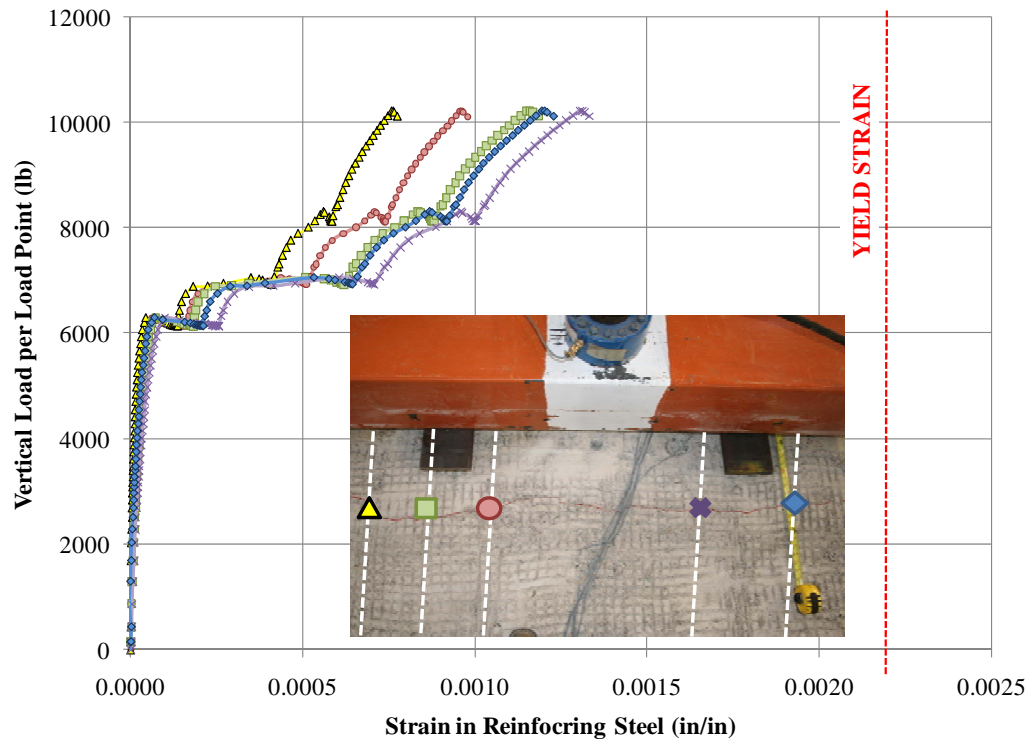
*Figure 5-22: Vertical Load versus Reinforcement Strain (Test #2)*

While the precast overhang is shown to be very strong, cracking in the overhang is undesirable during the construction and therefore the cracking load is seen as the limit state. Using the conservative load estimates for typical construction loads on the overhang as shown in Table 5-4, a factor of safety of approximately 2 was calculated.

Table 5-4: Construction load on precast overhang

Allowable				
Cracking Load	12600	lb		
Cracking Moment			15750	lb-ft
Applied				
Screed Load	3600	lb	4500	lb-ft
Concrete Weight	150	lb/ft^3	1378	lb-ft
Concrete Live Load	75	lb/ft^2	919	lb-ft
Platform Live Load	50	lb/ft^2	875	lb-ft
Platform Dead Load	10	lb/ft^2	175	lb-ft
Total Applied Moment			7847	lb-ft
Tributary Area	12	ft		
$\dfrac{\textbf{Cracking Moment}}{\textbf{Applied Moment}}$		=	**2.0**	

The stiffness of the precast concrete overhang was also evaluated through the use of string potentiometers that were located directly beneath the load points. Loading the girder both on the precast overhang and overhang brackets, allowed for a direct comparison of the two systems. The brackets used in these tests were Meadow Burke HF-86 as described in Section 3.2.1.6. Figure 5-23 illustrates the drastic difference in stiffness of the precast overhang to the overhang brackets. The deflection of the bracket at the load point is more than 5 times the deflection of the precast overhang. The HF-86 brackets tested are also significantly stiffer than other previously tested brackets. Hence it can be concluded that on average the precast overhang is 5-10 times stiffer than bracket systems.
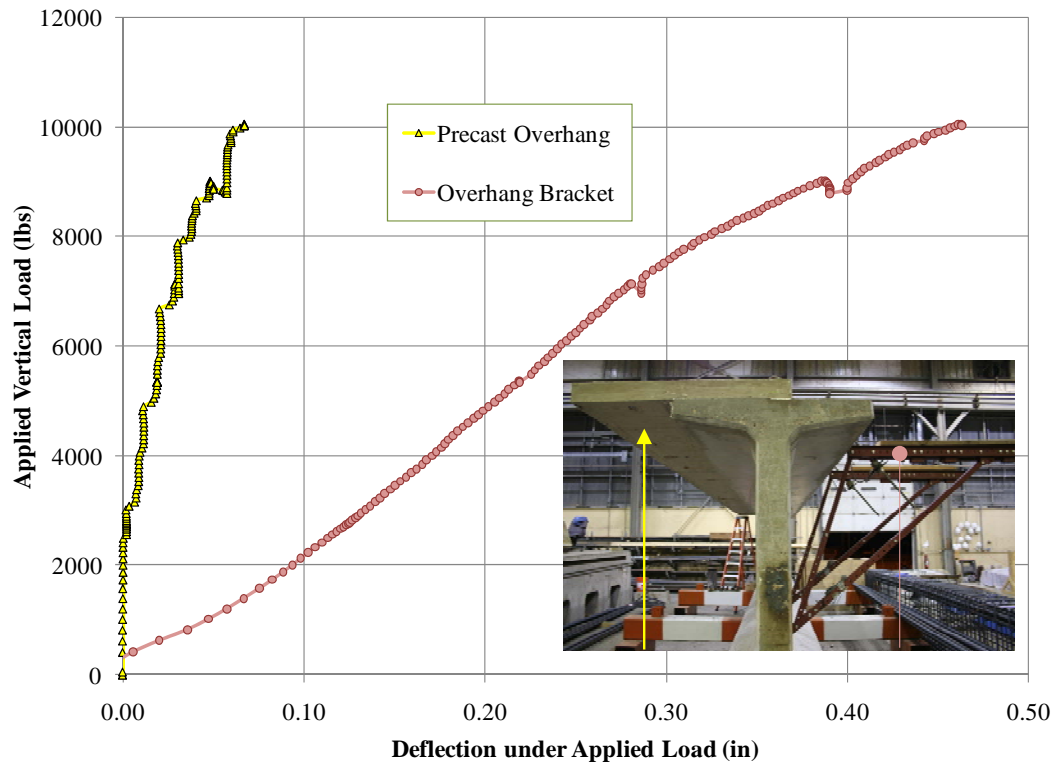
Figure 5-23: Load versus deflection (Test #1)

5.7   OVERVIEW OF PROPOSED PROCEDURE

The recommended procedure showing the steps from girder fabrication to the completed bridge is shown in Figure 5-24.

BEAM IS FABRICATED AS NORMAL
WITH ONE BAR TYPE ADDED

1

BEAM IS MOVED AND FORMED
TO CAST 4" OF OVERHANG

2

4" OVERHANG PLUS SIDE PIECE IS CAST AND
BEAM IS SHIPPED TO BRIDGE SITE

3

SUPPORT ARMS ARE INSTALLED
AND BEAM IS LIFTEN INTO PLACE

4
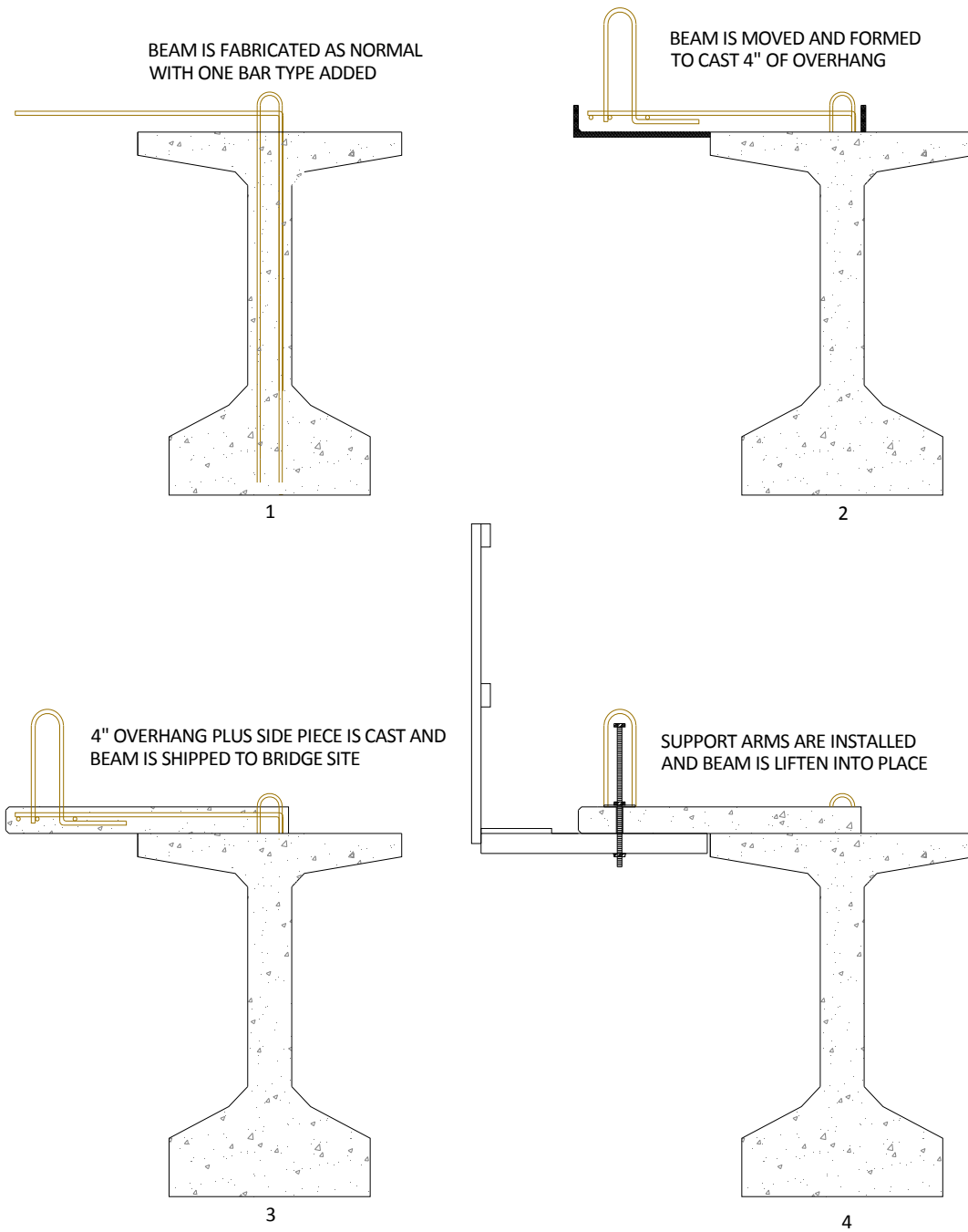
Figure 5-24: Recommended procedure: Steps 1-4

SCREED RAIL SUPPORTS ARE PLACED INSIDE RAIL BARS

5

SIDE FORM, PRECAST PANELS AND STEEL ARE PLACED

6

CONCRETE IS CAST TO TOP SURFACE DECK

7

PLATFORM IS REMOVED BY PULLING COIL RODS, BARRIER RAIL IS CAST ON OVERHANG

8
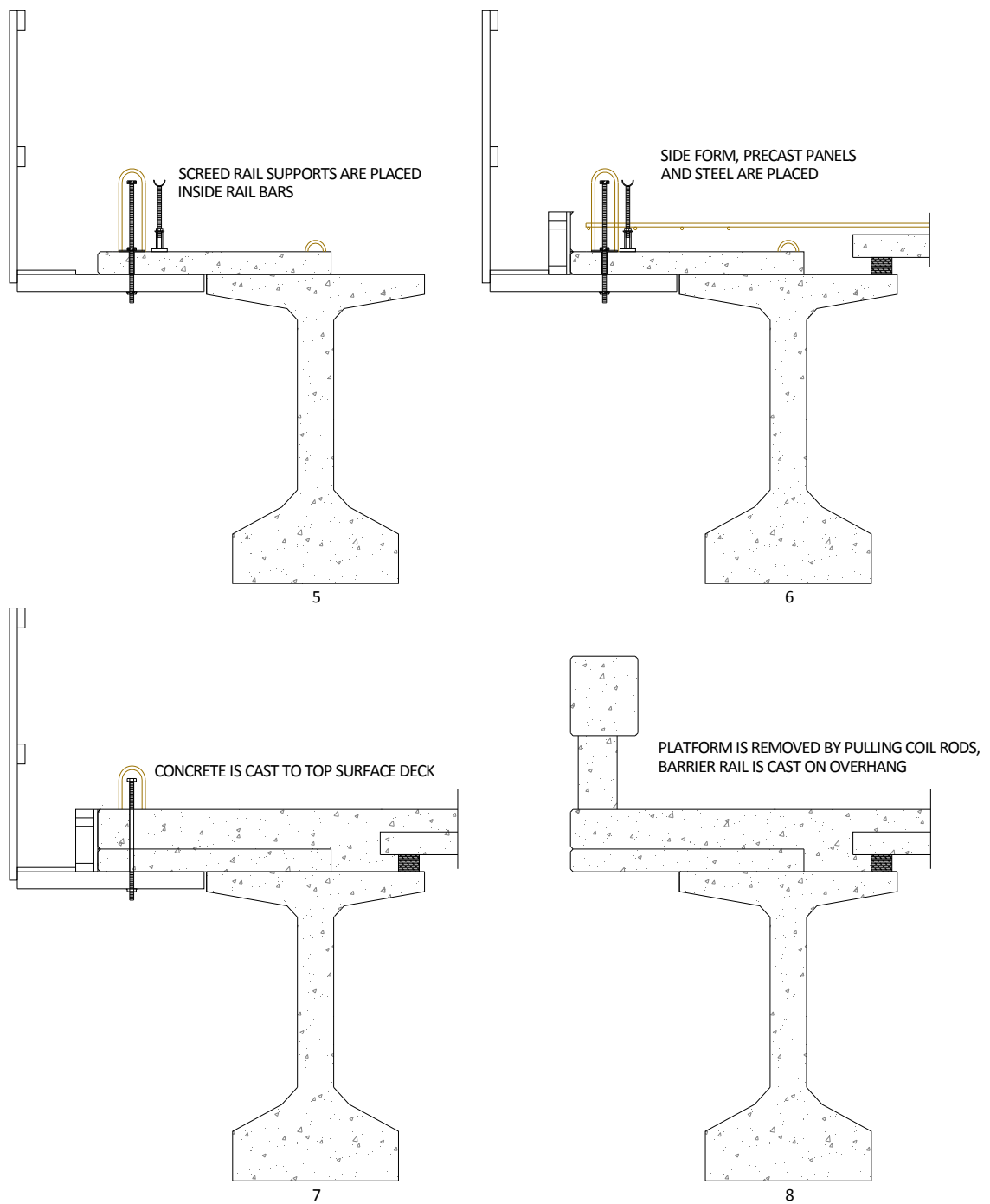
Figure 5-24 (cont.'d): Recommended procedure: Steps 5-8

5.8 Summary

A precast overhang system was developed during the course of this study. Its feasibility was investigated by building a full-scale mock-up in the laboratory and load testing the specimen. The system was presented to two members of Texas Prestressed Concrete Manufactures Associates (PCMA). These members showed interest in the concept and do not foresee significant problems with the fabrication. The system was further presented to the members of the Association of General Contractors (AGC). While some concerns and recommendations were expressed by AGC members, the majority received the concept of having a precast overhang alternative positively. In view of all these factors the implementation of this system is strongly recommended.

CHAPTER 6
Conclusions

6.1 SUMMARY

During the development of a new family of prestressed I-girders (Tx-girders), the Texas Department of Transportation expressed a concern with the new designs as they related to the construction of the bridge deck overhang. The concern expressed was related to the extreme slenderness of the top flange and the ability to support the formwork necessary for the overhang construction on the exterior girders. In order to further investigate this potential problem, an interagency testing contract was established between the University of Texas at Austin and the Texas Department of Transportation.

An experimental program was conducted at the Phil M. Ferguson Structural Engineering Laboratory to examine the strength of the thinner top flange of the new I-girder as it relates to overhang construction. Two Tx-28s, one Tx-46, and one Tx-70 were fabricated and instrumented for testing. A variety of commercial systems were load tested on the Tx girders. Two products from Meadow Burke and one product from Dayton Superior used in overhang forming systems were specifically looked at in this project in a total of thirteen load tests. The primary goal of this study was to determine if any of the products could be acceptably used with the new Tx girders and if any restrictions should apply to their use.

Through the testing conducted on overhang bracket systems, the investigating team observed the need for a more efficient system to form the bridge deck overhangs. Therefore a new procedure for the construction of the overhang was developed using a precast overhang solution. Following the development of the precast overhang design and construction, an experimental program was completed simulating certain aspects of the construction process as well as load testing of the precast overhang in order to prove the feasibility of the concept.

6.2 CONCLUSIONS AND RECOMMENDATIONS

With thirteen tests conducted on three different overhang bracket and hanger systems, scatter existed in the results. Through the analysis and interpretation of the test results certain characteristics of the behavior were clear. The following conclusions are based on the experimental observations and synthesis of the data gathered during the tests:

- The initiation of longitudinal tensile cracks in the top flange of the Tx girders should be considered a limit state. Due to the load transfer mechanism of the hanger system, both direct tension and bending moment are applied to the critical section. Thus, the initial cracks rapidly grow in size as the load is transferred to the reinforcing steel. Cracking of the top flange due to overhang forming systems does cause significant damage as observed in data acquired from the strain gauges attached to the reinforcing steel. The hanger systems are used to support construction loads, and the construction process should not result in damage to the structural components.
- The placement of overhang bracket systems near the end of the girders will produce lower cracking loads than cases in which bracket systems are attached to the interior parts of the fascia girders. Test results indicate that when a hanger is placed within 12" the end, the cracking load is reduced notably. As such, it is recommended that a hanger not be placed within 12" of the girder end. In cases where such a placement is inevitable a reduced working load on those hangers should be used.
- It is recommended that no more than 3000 lbs be applied vertically to any one overhang bracket hanger systems. The allowable load recommendation is based on observed cracking loads and a factor of safety of 1.65.
- The reinforcement in the top flange of girders that support overhang brackets should consist of No. 3 reinforcing bars spaced at a 6" maximum spacing.
- The developed solution for the precast overhang can greatly impact the bridge deck construction process. Although there exists certain aspects of the process that may require further design, the general procedure outlined in Chapter 5 is ready for implementation. Through the initial application of the process to actual bridge construction, the future possibilities and potential rewards will become apparent. It is recommended that the overhang construction alternative using a precast overhang be implemented on a typical bridge construction.

6.3 RECOMMENDATIONS FOR FUTURE WORK

Further product engineering considering the specific Tx girder geometry may yield a more efficient system by increasing the load at which initial cracking occurs. This could be accomplished by creating a hanger system that engages a larger area of concrete in the horizontal direction in which the tension force is being applied.

Further investigation into the precast overhang concept would greatly improve the overall procedure. Particular aspects that could use further investigation include a field study of necessary edge form depth, overhang profile on curved spans, and a cost analysis of current practice. Further innovation and investigation in the precast overhang concept will only lead to a more efficient construction practice.

APPENDIX A

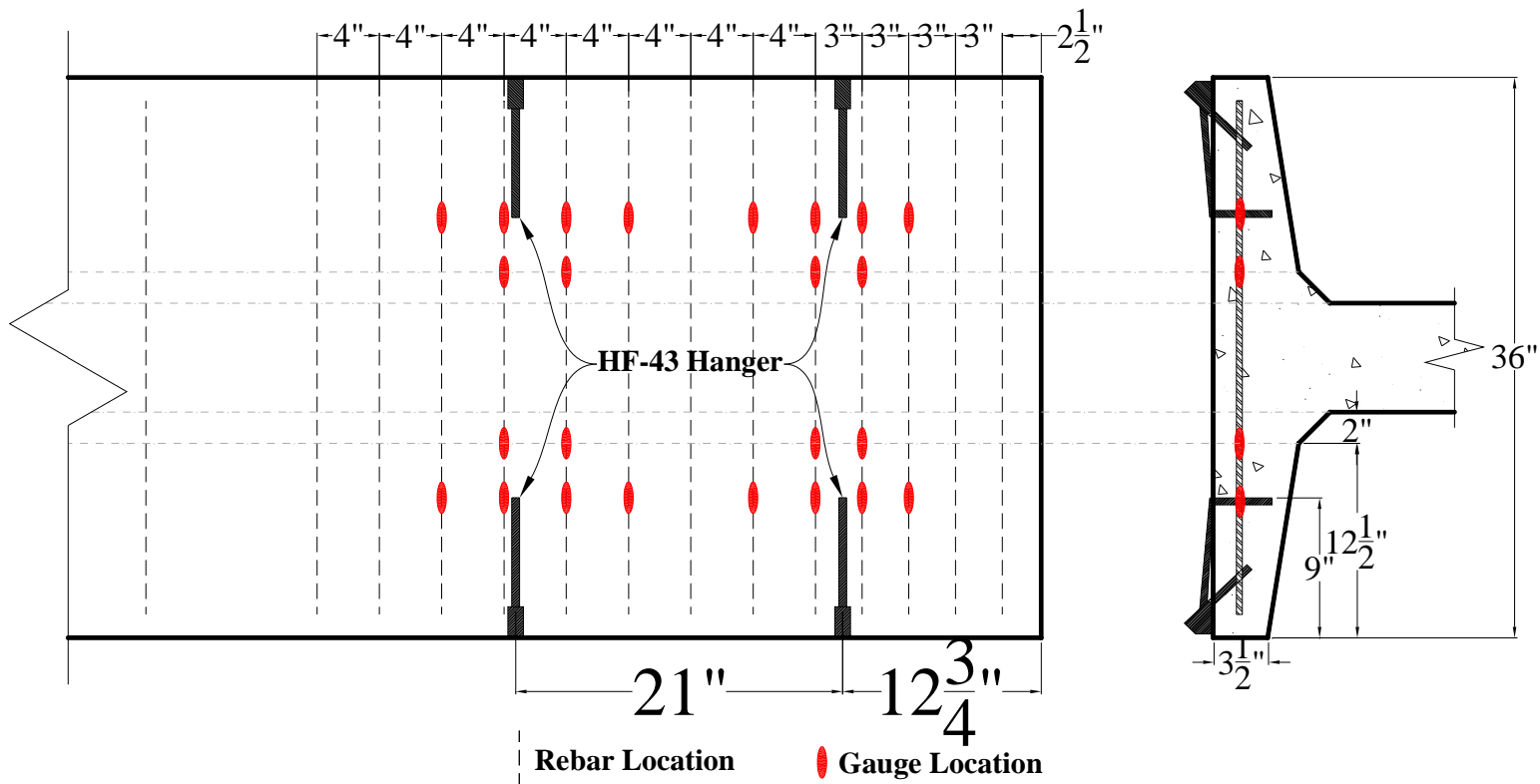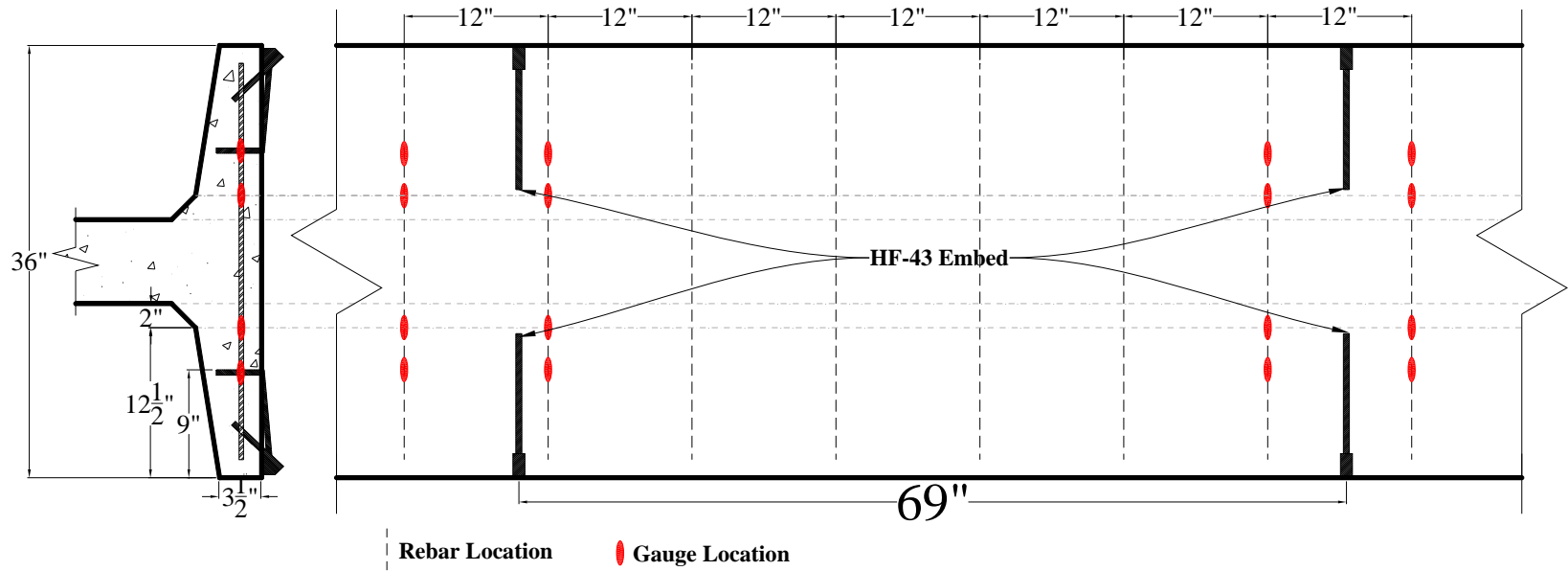## Top Flange Reinforcing Steel Strain Gauge Locations

Tx-28-I MB HF-43

Figure A-1: Tx-28-I HF-43 Strain Gauge Locations

Tx-28-II MB HF-43

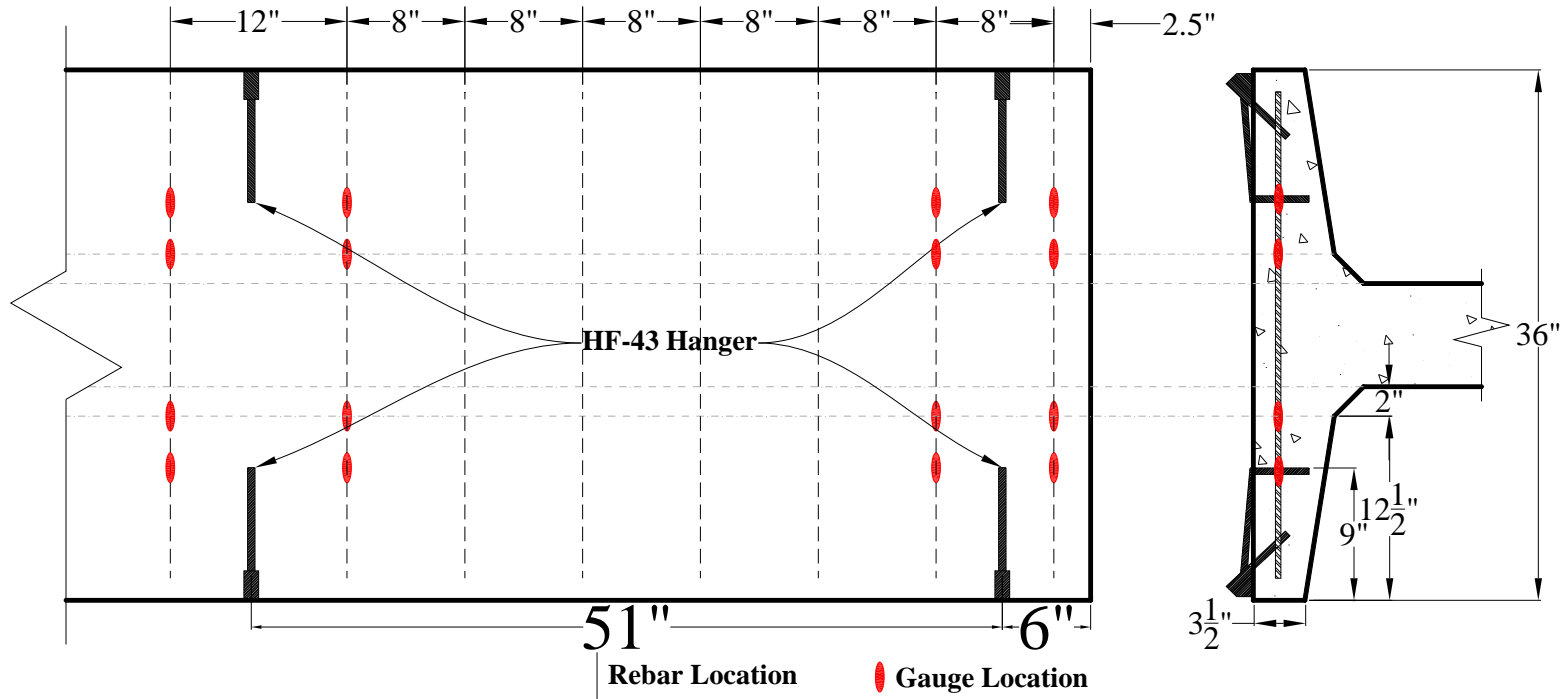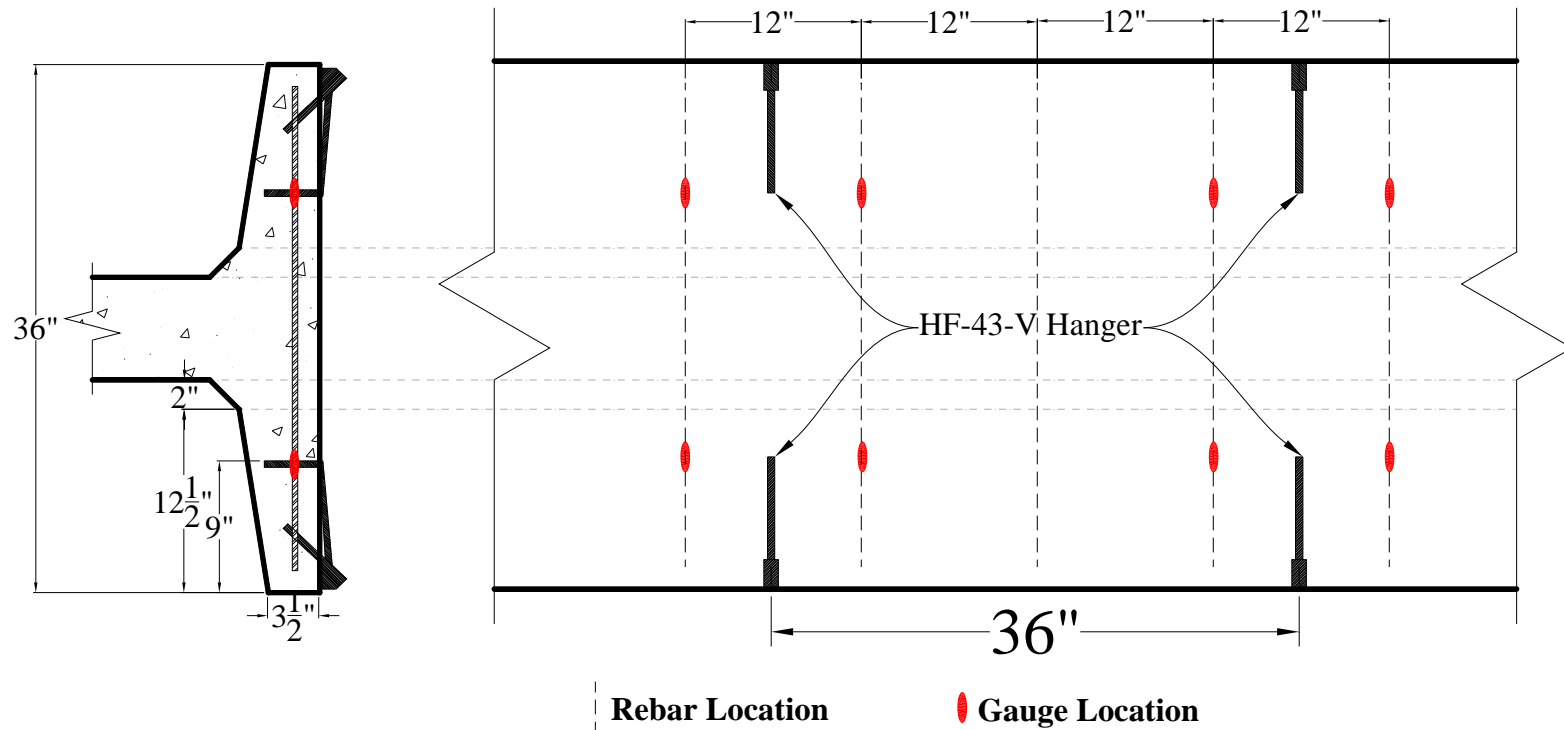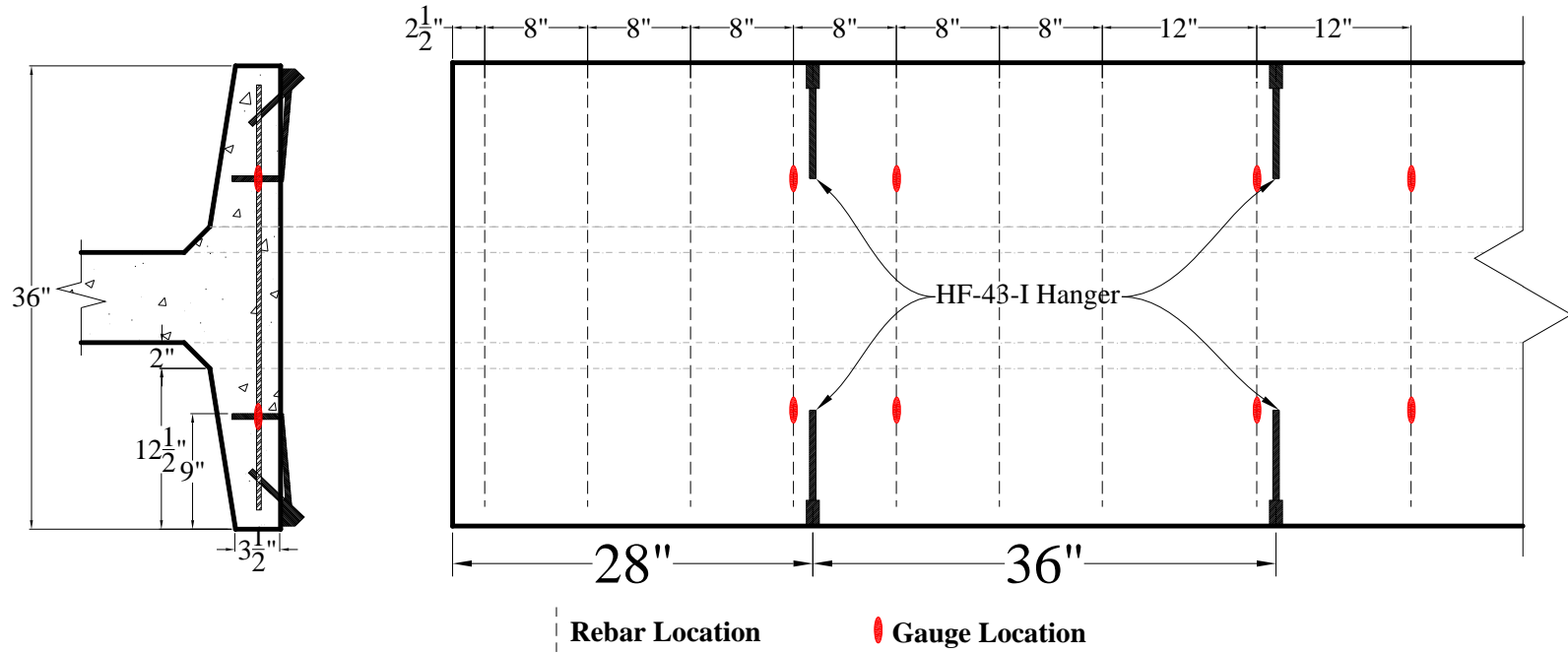**Figure A-2: Tx-28-II HF-43 Strain Gauge Locations**

Tx-46 MB HF-43

Figure A-3: Tx-46 HF-43 Strain Gauge Locations

Tx-70 MB HF-43-V

Figure A-4: Tx-70 HF-43-V Strain Gauge Locations

Tx-70 MB HF-43-I

Figure A-5: Tx-70 HF-43-I Strain Gauge Locations

Tx-28-I MB HF-67

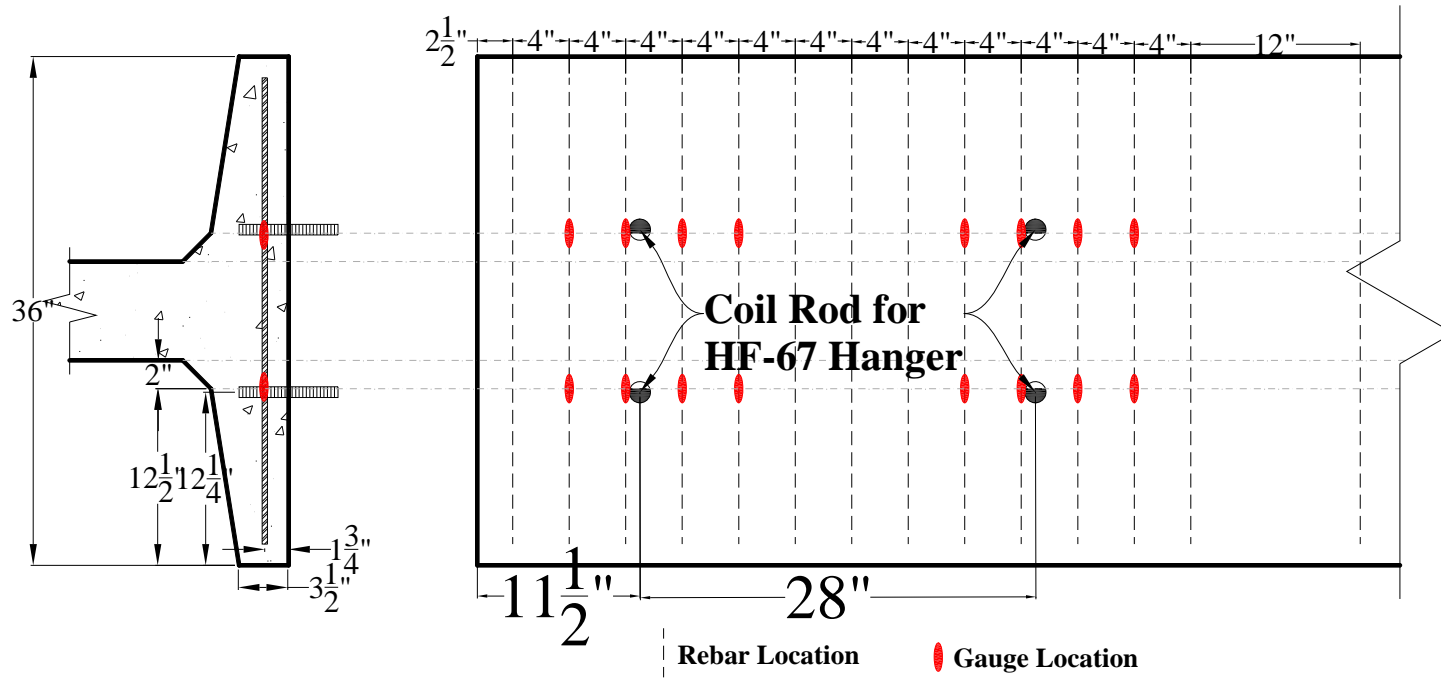**Figure A-6: Tx-28-I HF-67 Strain Gauge Locations**

Tx-28-II MB HF-67
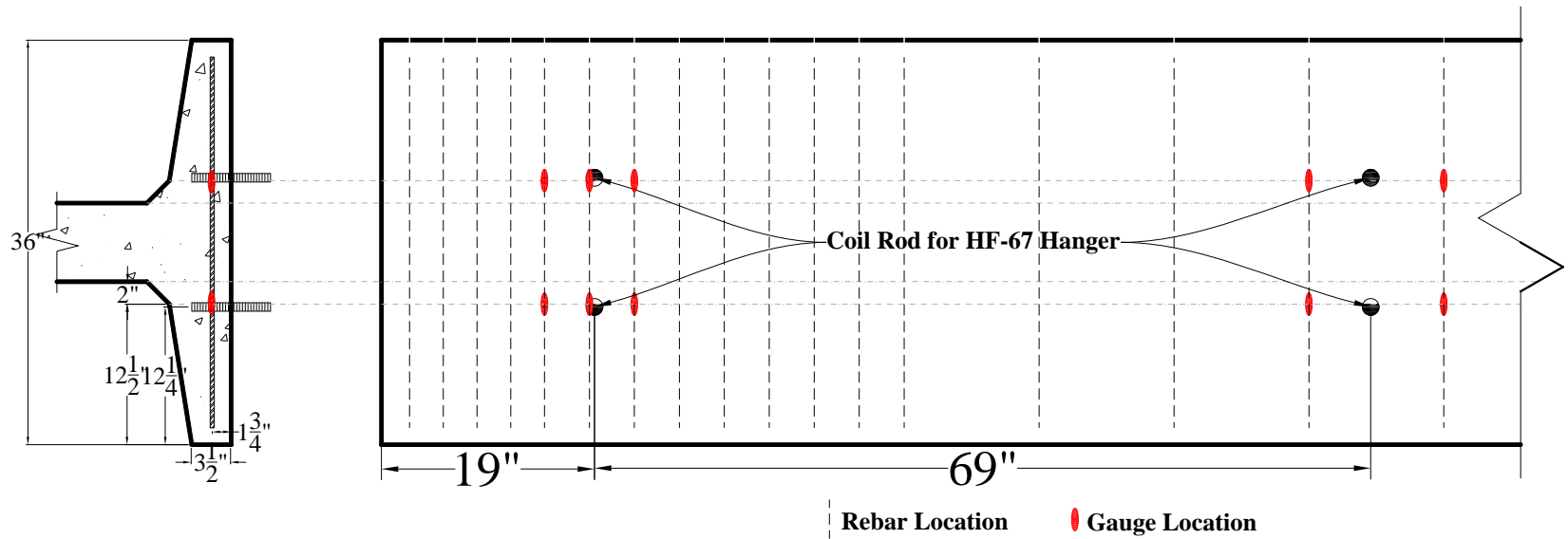

Figure A-7: Tx-28-II HF-67 Strain Gauge Locations

Tx-46 MB HF-67-M
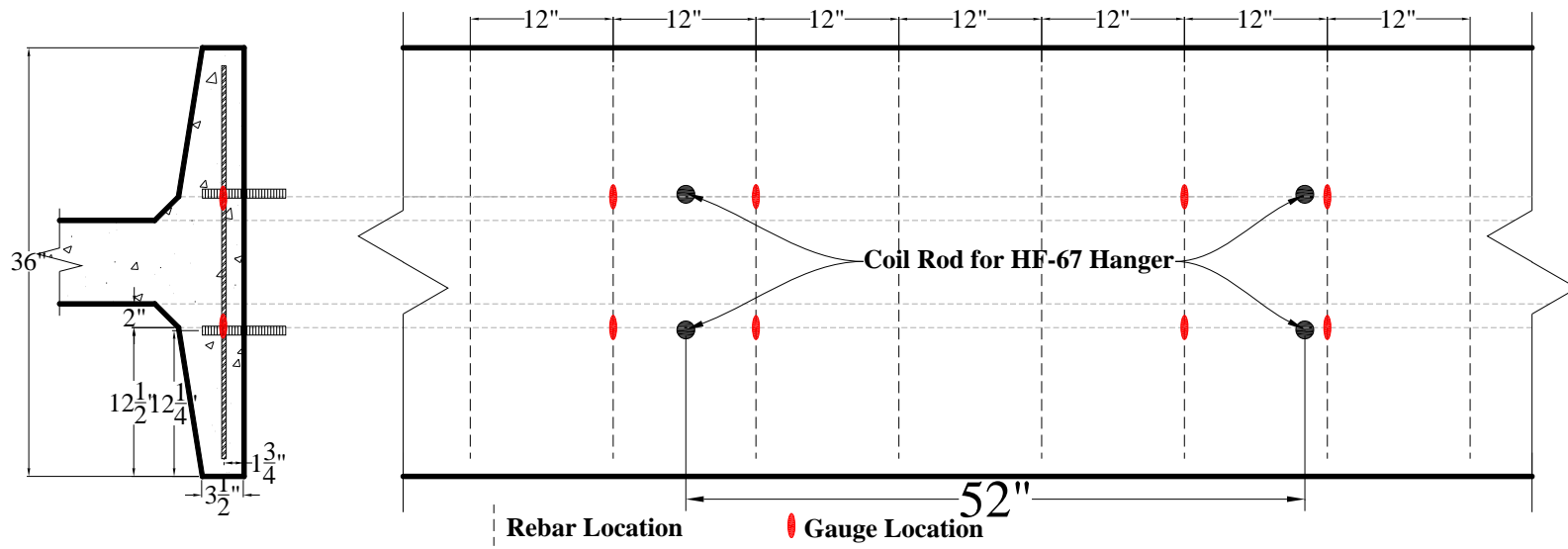
Figure A-8: Tx-46 HF-67-M Strain Gauge Locations

Tx-70 MB HF-67-M
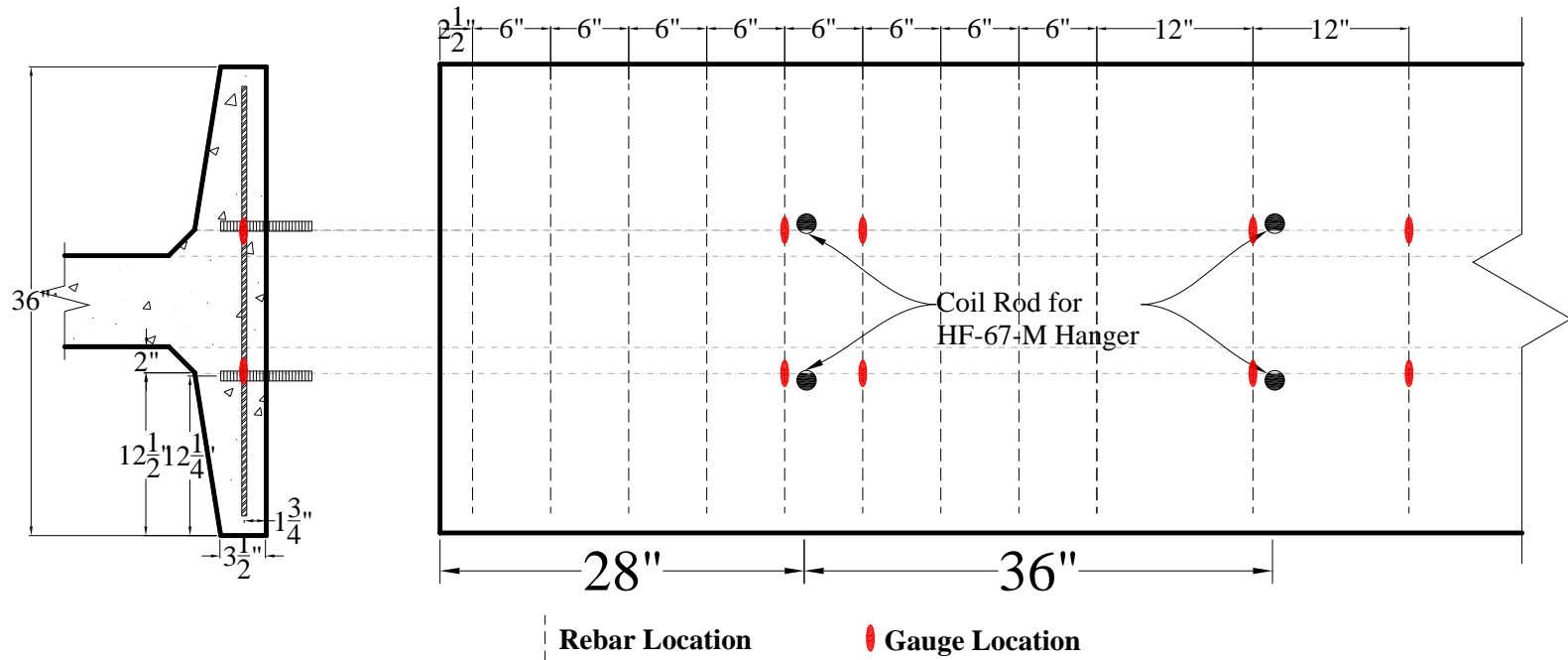
Figure A-9: Tx-70 HF-67-M Strain Gauge Locations

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Tx-70 MB HF-67-M

Rebar Location   Gauge Location

Figure A-10: Tx-70 HF-67-M Strain Gauge Locations

Tx-28-I DR C-24

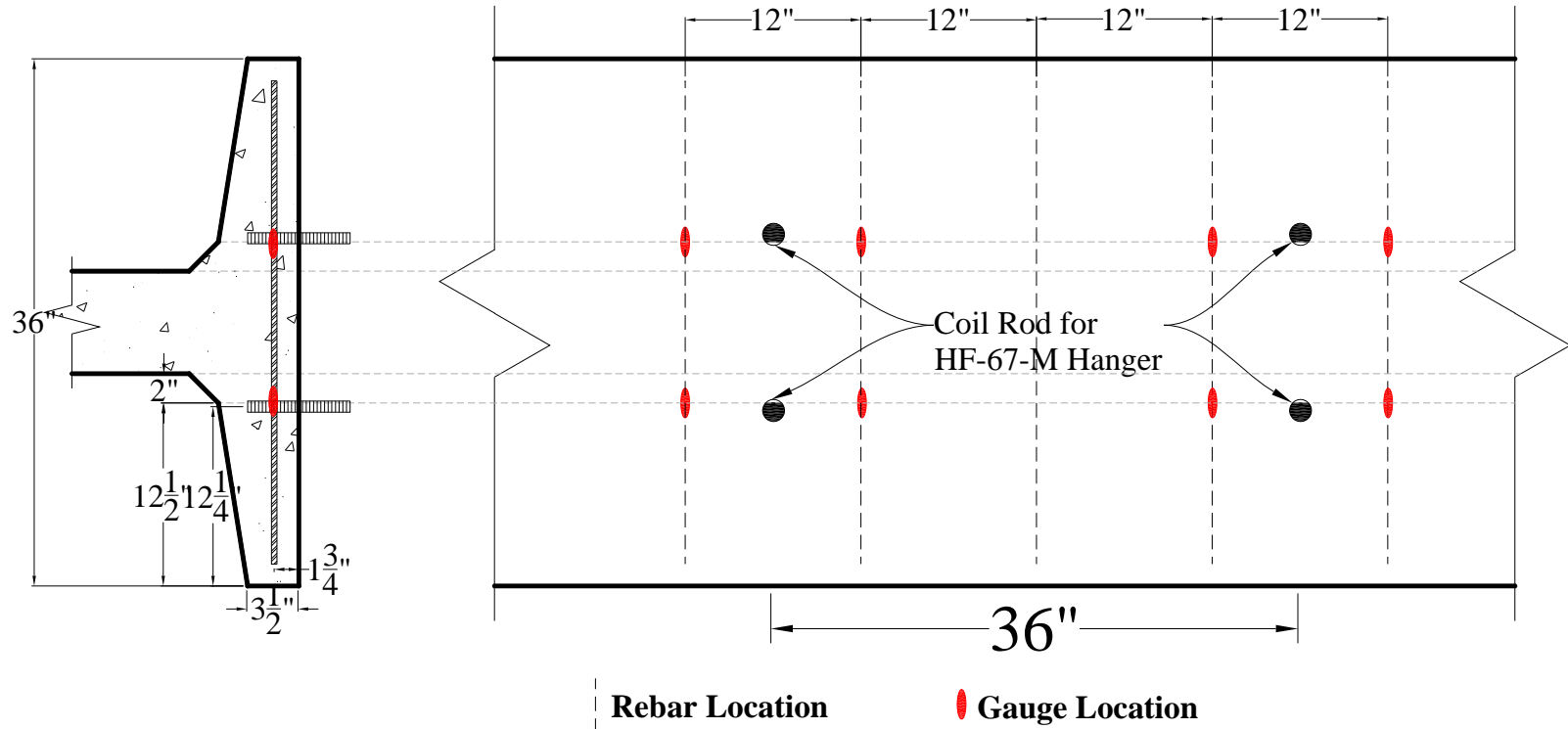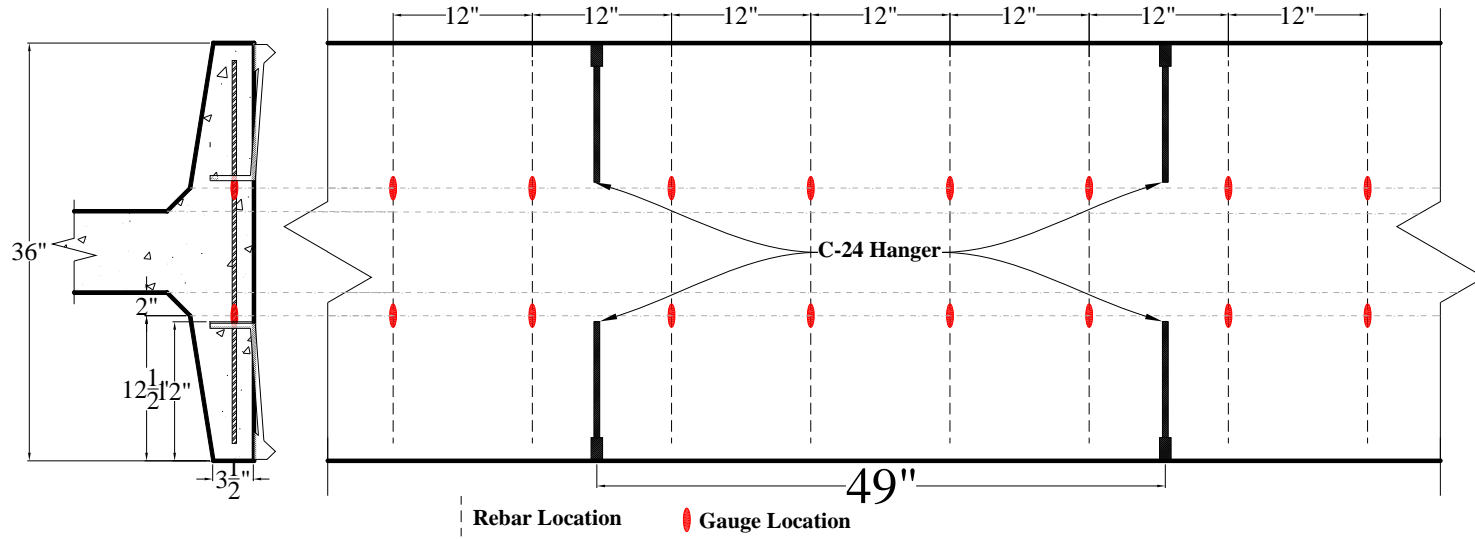*Figure A-11: Tx-28-I C-24 Strain Gauge Locations*

Tx-28-II DR C-24
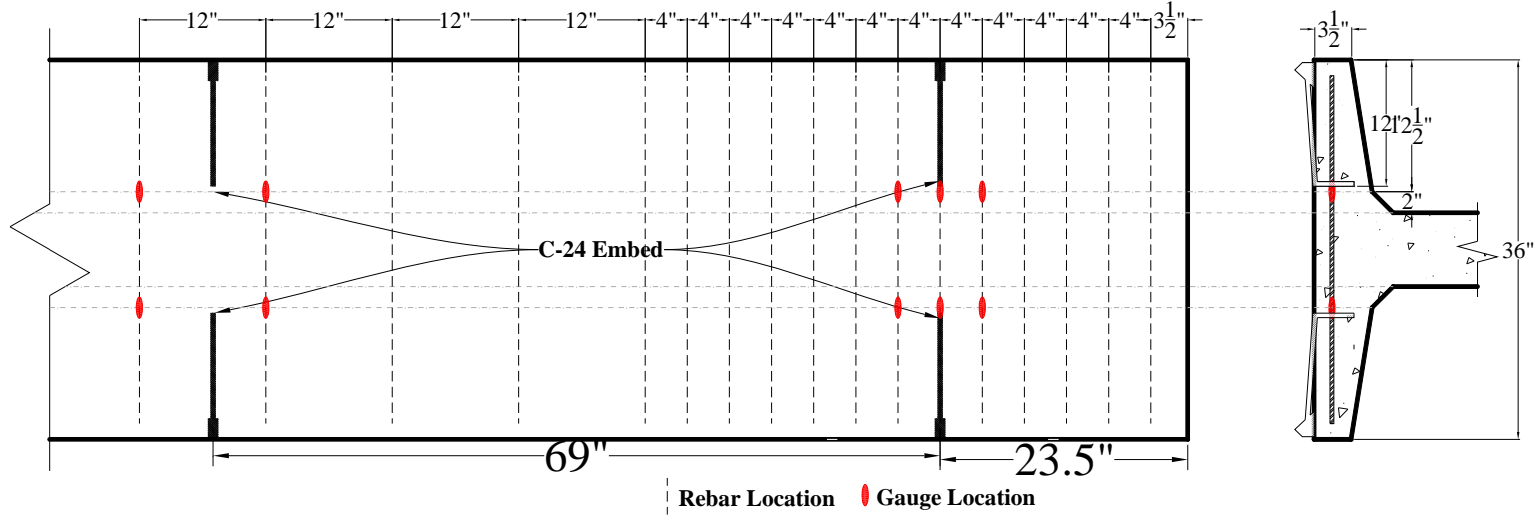
Figure A-12: Tx-28-II C-24 Strain Gauge Locations

Tx-46 DR C-24
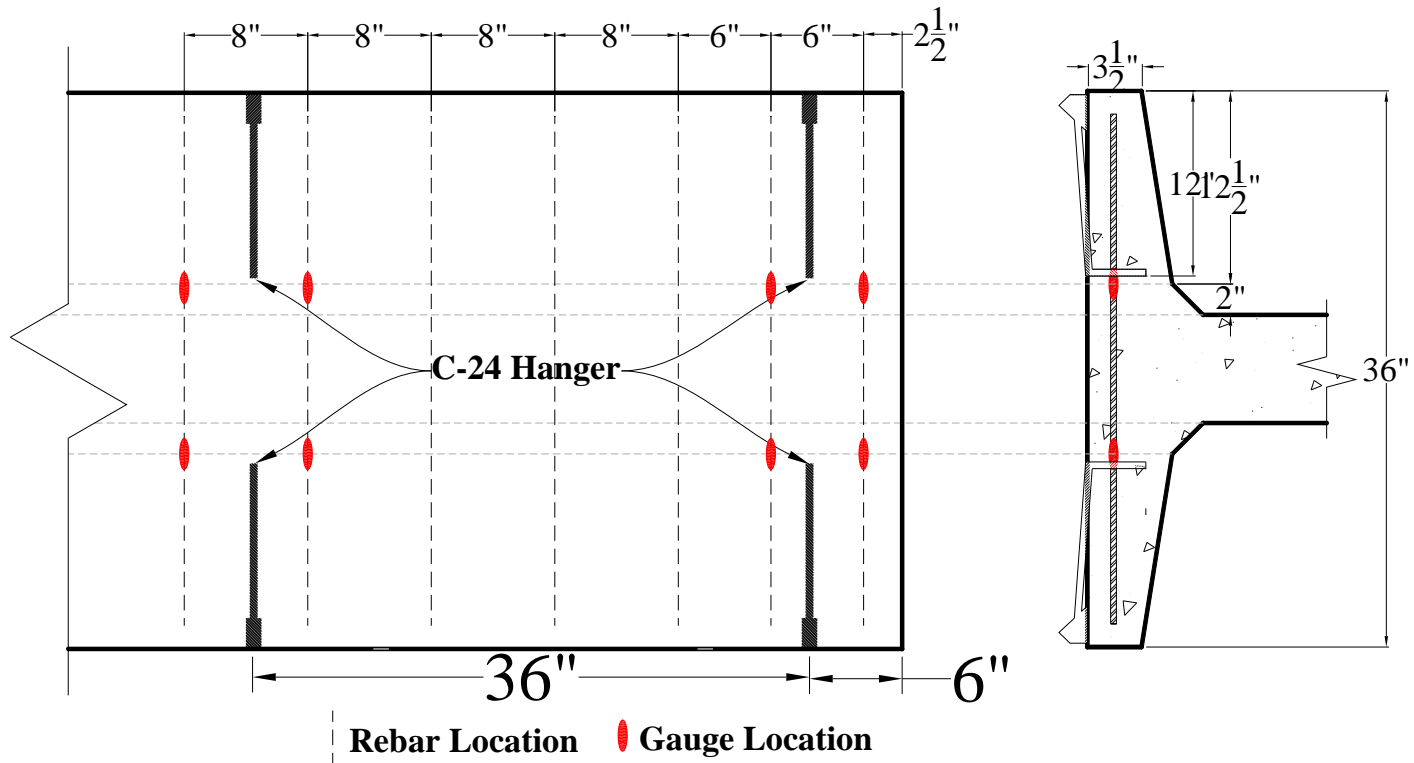
Figure A-13: Tx-46 C-24 Strain Gauge Locations

APPENDIX B

TxDOT Design Drawings for the Tx Family of Girders

Bars R Spa - 2½" | 4 Spa at 3" | 8 Spa at 4" | 4 Spa at 6" | 12" Max Spa | 4 Spa at 6" | 12 Spa at 4" | 2½"

T and F. Spa as shown

L

R

1½" Clr Cov

1½" Clr Cov

S (Bundle with R bars)

S (Bundle with R bars)

END A

V (Spa as shown)

END B

BEAM ELEVATION

Beam Length minus 3"

2'-9"

BARS U (#4)

BARS T (#3)

5¼" (Typ)

1'-4⅜" (Typ)

BARS F (#3)

3'-10"

1'-2"

6"

2

2'-3½"

BARS S (#6)

BARS V (#4)

3¼"

4'-1½"

4" (Typ)

BARS R (#4)

U (#4)

1" Clr Cov

T (#3)

1½" Clr Cov - Typ unless noted

F (#3)

R (#4)

V (#4) In pairs

TYPE G46 BEAM SECTION
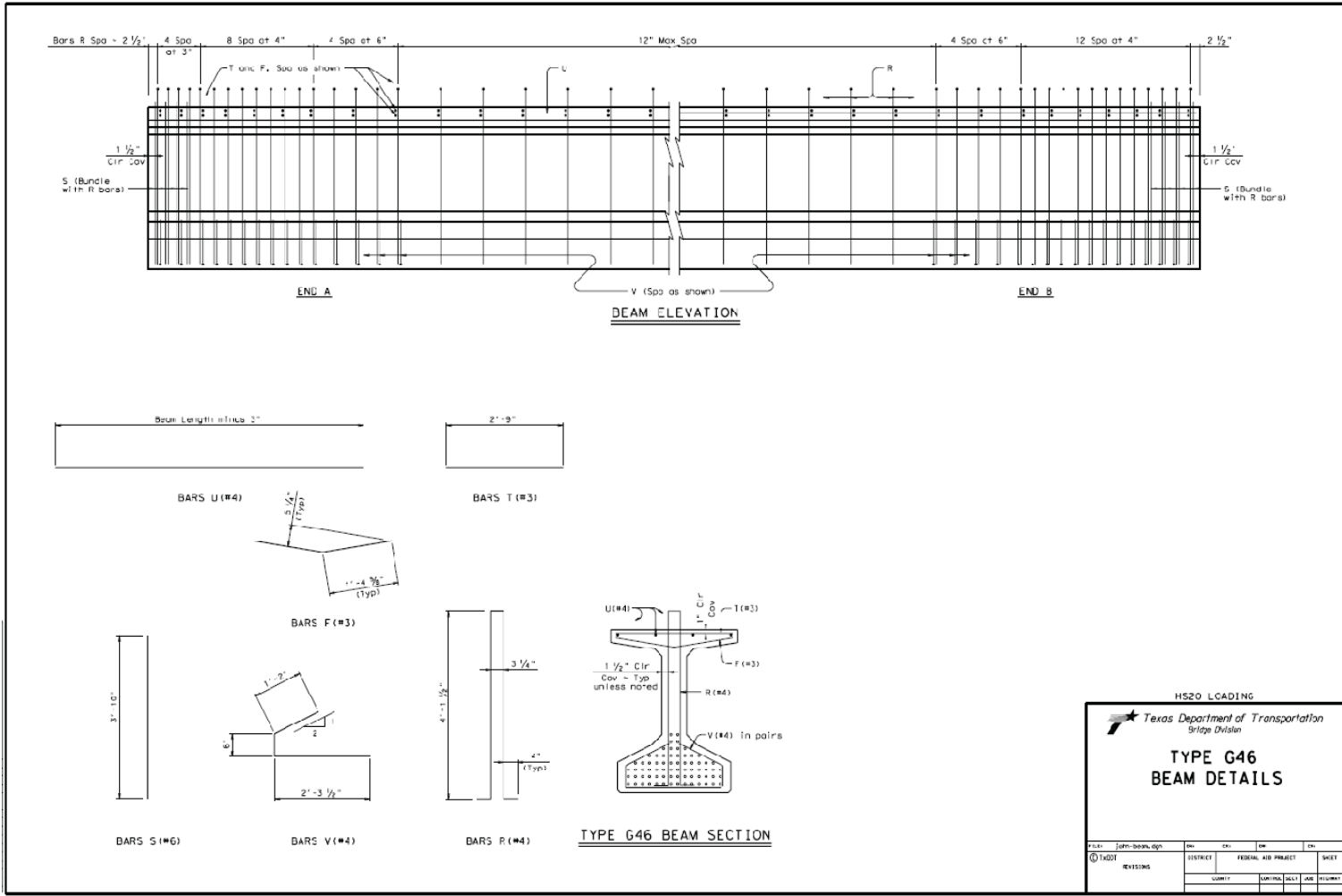
HS20 LOADING

Texas Department of Transportation
Bridge Division

TYPE G46
BEAM DETAILS

FILE: john-beam.dgn

© TxDOT

REVISIONS

DISTRICT | FEDERAL AID PROJECT | SHEET

COUNTY | CONTROL | SECT | JOB | HIGHWAY

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Bars R Spa ~ 2 ½"   4 Spa   8 Spa at 4"   10 Spa at 6"   12" Max Spa   10 Spa at 6"   12 Spa at 4"   2 ½"
                     at 3"

T and F, Spa as shown   U   ℄ 3" Dia P.T. Duct   R

1 ½"
Cir Cov

1 ½"
Cir Cov   S (Bundle
                                                                                                 with R bars)

S (Bundle
with R bars)

END A   V (Spa as shown)   END B

BEAM ELEVATION

Beam Length minus 3"   3'-3"

BARS U (#4)   BARS T (#3)

BARS F (#3)

5'-10"

U (#4)   1" Cir   T (#3)
          Cov

F (#3)

R (#4)   3" Dia P.T. Duct
1 ½" Cir
Cov ~ Typ
unless noted

1 ½"
Cir
(Typ)

V (#4) in pairs

HS20 LOADING

Texas Department of Transportation
Bridge Division

**TYPE G70
BEAM DETAILS**

BARS S (#6)   BARS V (#4)   BARS R (#4)   **TYPE G70 BEAM SECTION**

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GIRDER ELEVATION

GIRDER DIMENSIONS AND SECTION PROPERTIES

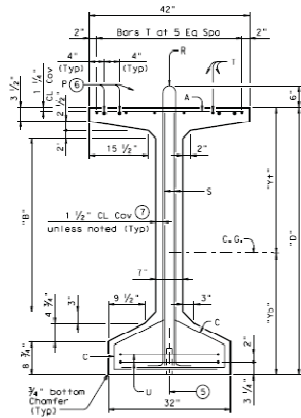| Girder Type	"D" (in.)	"A" (in.)	"Yt" (in.)	"Yb" (in.)	Area (in.²)	"Ix" (in.⁴)	"Iy" (in.⁴)	Weight (plf)
Tx28	28	8	15.02	12.98	585	52,772	40,939	610
Tx34	34	12	18.49	15.51	627	88,355	40,731	653
Tx40	40	18	21.90	18.10	669	134,990	40,902	697
Tx46	46	22	25.90	20.10	761	198,089	46,478	793
Tx54	54	30	30.49	23.51	817	299,740	46,707	851
Tx62	62	37 ½	33.72	28.28	910	463,072	57,351	948
Tx70	70	45 ½	38.09	31.91	966	628,747	57,579	1,006

Notes (Girder Elevation):
1. Bundle with Bars R.
2. Measured along ℄ Girder at Interior Bents; perpendicular to Abutment Bkwl or Inverted-T Stem.
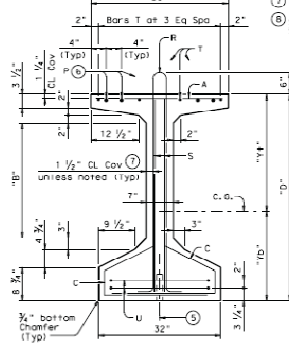3. The average of the top and bottom spacing of Bars R cannot exceed the required spacing.
4. L/20, but not less than 5'-0" (-0,+2").
5. ℄ 4" x 1 ½" Vertical Slotted Hole at doweled girder end (labeled (D) on Bridge Layout). Required for outside girder only or as shown on substructure details. Anchorage holes may be tapered (4 ¾" x 1 ⅝") at base. If holes are formed with sheet metal, forms may be left in place.
6. Bars P (#6 x 15'-0") are only required when "e" at girder ends exceeds 0.25 x "D". At the fabricator's option bars larger than #6 may be used. When L is less than 50 ft, Bars P are to be the same length as Bars T.
7. 1 ¾" Clear Cover to Bars S.
8. Space Bars A at 6" Max for girders requiring overhang bracket hangers. Space at 12" Max for all other girders. Tie to Bars R as necessary.

TYPE Tx62 & Tx70

TYPE Tx46 & Tx54

TYPE Tx28, Tx34 & Tx40

GENERAL NOTES:
Designed in accordance with AASHTO LRFD Specifications.
All concrete must be Class H.
All reinforcing bars must be Grade 60.
An equal area of deformed Welded Wire Reinforcement (WWR) (ASTM A497) may be substituted for Bars A, C, R or T unless otherwise noted.
It is permissible for bars or strands to come in contact with materials used in forming anchor holes.
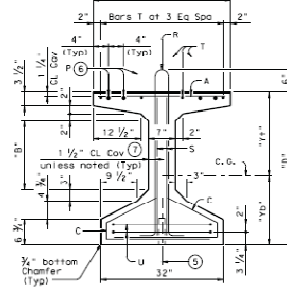
HL93 LOADING   SHEET 1 OF 2

Texas Department of Transportation
Bridge Division

PRESTRESSED CONCRETE
I-GIRDER DETAILS

IGD

PRELIMINARY
SUBJECT TO REVISION
DATE: May 15, 2007

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