CONSTRUCTION REPORT

North Carolina Department of Transportation
Research Project No. 2009-17

Hybrid High Performance Steel Bridge on SR 1102 over I-77 in Iredell County

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This report documents the construction of a hybrid HPS 100W steel girder bridge on Langtree road (SR 1102) in Iredell County North Carolina. Each major phase of construction is described from the foundations to the concrete deck slab. There is also a section devoted to the fabrication of the girders that concentrates on documenting the major welds between the various grades of HPS and conventional steels on the bridge. The major welds on the bridge are divided into five main groups based on the material grade of the steel plates being joined. These groups are 1.) HPS 70W to HPS 70W welds, 2.) HPS 70W to HPS 100W welds, 3.) HPS 100W to HPS 100W welds, 4.) HPS 70W to AASHTO M270 Grade 50W welds, and 5.) HPS 100W to AASHTO M270 Grade 50W welds. Various properties for these welds including the consumable combination, welding process, diffusible hydrogen level, preheat, heat input, and interpass temperatures are listed in this report.
In addition to describing the fabrication of the bridge girders and the construction of the entire bridge this report also describes a load test that was carried out on the completed structure. For this load test, a new technique for measuring bridge deformations using a laser based image scanner was used. This laser based method can be used to determine deflections by comparing scans of the structure with and without live load present on the bridge. In addition, linear variable displacement transducers were used to record deflection measurements. This report documents the results of the load test performed on the hybrid HPS 100W bridge.</td></tr>
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EXECUTIVE SUMMARY

This report documents the construction of a hybrid HPS 100W steel girder bridge on Langtree Road (SR 1102) in Iredell County, North Carolina. Each major phase of construction is described from the foundations to the concrete deck slab. There is also a section devoted to the fabrication of the girders that concentrates on documenting the major welds between the various grades of HPS and conventional steels on the bridge. The major welds on the bridge are divided into five main groups based on the material grade of the steel plates being joined. These groups are 1.) HPS 70W to HPS 70W welds, 2.) HPS 70W to HPS 100W welds, 3.) HPS 100W to HPS 100W welds, 4.) HPS 70W to AASHTO M270 Grade 50W welds, and 5.) HPS 100W to AASHTO M270 Grade 50W welds. Various properties for these welds including the consumable combination, welding process, diffusible hydrogen level, preheat, heat input, and interpass temperatures are listed in this report.

In addition to describing the fabrication of the bridge girders and the construction of the entire bridge this report also describes a load test that was carried out on the completed structure. For this load test, a new technique for measuring bridge deformations using a laser based image scanner was used. This laser based method can be used to determine deflections by comparing scans of the structure with and without live load present on the bridge. In addition, linear variable displacement transducers were used to record deflection measurements. This report documents the results of the load test performed on the hybrid HPS 100W bridge.

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1.0 INTRODUCTION

1.1 General Information

In 2007 construction commenced on the replacement of an old four span steel girder bridge located on Langtree Road (SR 1102) over I-77 in Iredell County, North Carolina. The replacement bridge is the first in North Carolina to make use of the new HPS 100W steel grade. The new bridge is a skewed two span structure with continuous steel plate girders and a reinforced concrete deck. Each of the spans has the same clear span length of 146’ – 11.375” and the skew angle of the bridge is 47^{o} 37’ 30”. The girders in this bridge contain HPS 100W in a hybrid configuration with HPS 70W steel. The HPS 100W steel is located in the flanges over the intermediate pier whereas the rest of the girders are composed of HPS 70W steel.

This report describes the construction of the replacement bridge with a focus on the details of the welding and fabrication of the steel bridge girders. This report also describes the static load testing of the completed structure using a new laser-based displacement measurement technique. Figure 1 below shows an aerial photograph of the bridge in the final stages of construction. Construction of this bridge was completed during the summer of 2009.

Figure 1: Hybrid HPS 100W bridge on Langtree Road

1.2 Background Information on HPS steel

The development of high performance steel began in 1992 when the Federal Highway Administration (FHWA), the American Iron and Steel Institute (AISI), and the United States Navy (USN) created a program to develop high strength steels that had other enhanced properties in addition to yield strength. The enhanced properties included better weldability, improved toughness over other high strength steels, and better corrosion resistance. The HPS program has resulted in three grades of steel, HPS 50W, HPS 70W, and HPS 100W with yield strengths of 50 ksi, 70 ksi, and 100 ksi, respectively (Wilson, 2000; Wilson, Undated).

The extensive research on HPS 70W has led to a greater understanding of the behavior of the material in bridges. This has led to the removal of many of the restrictions in the AASHTO code for steels with a yield strength up to 70 ksi (Mertz, 2001). Currently HPS 70W steel has been used more than the other HPS grades and as a result has been tested and researched the most (Lwin, 2002).

High performance steel can be produced in two ways, the first is quenching and tempering (Q&T) method and the second is the thermo mechanical controlled processing (TMCP) method (Lwin, 2002; Wilson, 2000). The Q&T process can produce steel plates that are a maximum of 50' long and a maximum of 4" thick for both HPS 50W and HPS 70W (Teal, 2000). The TMCP process allows the production of HPS 70W plates up to 125' long and 2" thick although shorter lengths may be necessary for safe handling (Lwin, 2002; Wilson, 2000). HPS 100W steel can be produced up to 2.5" thick (Wilson, 2005).

Overall, the improved properties of HPS have led to an increased number of bridges that make use of high performance steel in the United States (US). Many states have HPS bridges in service, construction, or design. Several including New York, Pennsylvania, and Tennessee have documented their experience with HPS steel, primarily HPS 70W (FHWA and NYSTA, Undated; Macioce, Thompson, and Zielinski, 2000; Wasserman and Pate, 2000). The most current version of the HPS scoreboard lists 399 bridges in service, construction, or design across the US (Teal, 2007). Of the 399 bridges, most are composed of HPS 70W steel. Six HPS bridges are located in North Carolina (Teal, 2007).

The state of Tennessee had completed three bridges with HPS 70W by the year 2000 (Wasserman and Pate, 2000). The first bridge was originally designed with grade 50W steel but was switched to HPS 70W steel (Wasserman and Pate, 2000). The switch saved approximately 200,000 lbs of steel and lowered the cost by approximately $120,000 (Wasserman and Pate, 2000). Another bridge benefited from switching to HPS 70W from grade 50W. The switch resulted in a 30% reduction in the weight of the sections in the negative moment areas of the bridge (Wasserman and Pate, 2000).

2.0 CONSTRUCTION DOCUMENTATION

2.1 Staged Construction and Site Preparation

Given that the existing four span bridge was the only crossing over I-77 for several miles, the structure could not be removed before the new bridge was completed. The North Carolina Department of Transportation (NCDOT) used a staged construction technique for the new bridge to minimize disruption to ongoing traffic. Staged construction involves building only a portion of the new bridge next to the existing bridge and then removing the old structure to make room for the rest of the new bridge. Once stage I (roughly 40% of the width of the bridge) was completed, traffic was rerouted from the existing bridge to stage I bridge. The existing bridge was then demolished and stage II (the remaining 60% of the width of the bridge) was erected. Once stage II was completed, the two halves of the bridge were connected through a construction joint in the bridge deck. Figures 2 and 3 show typical cross sections of the bridge during each stage of construction (NCDOT, 2006).

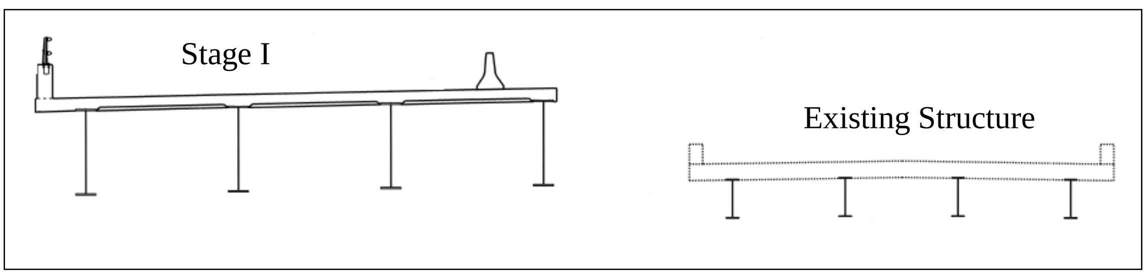


Figure 2: Stage I construction (NCDOT, 2006)

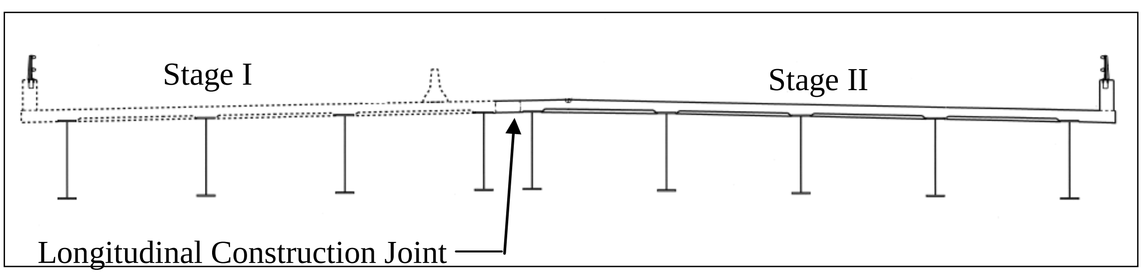


Figure 3: Stage II construction (NCDOT, 2006)

2.2 Substructure and Foundations

The substructure for the replacement bridge consisted of a stub abutment at each end bent and a column bent in between the two spans. Half of the intermediate column bent was constructed as part of stage I and the second half as part of stage II. The two halves were then spliced together at the cap during the construction of the second half of the column bent. The same construction procedure was used for each end bent.

For the intermediate pier, the bent cap is a minimum of 5’ deep by 4’ – 2” in wide. The cap increases in depth towards the middle of the bridge in order to create a superelevation in the bridge deck. The total length of the bent cap is 117’- 6”. The reinforcing steel in the cap consists of eight #10 bars on the top and seven #10 bars on the bottom. Four #5 bars were placed along each side of the cap and all of the rebar was enclosed by #5 ties at the ends of the cap and by #4 ties in the middle. There are eight circular columns with a diameter of 3’ – 6” supporting the bent cap. The columns are 16’ – 8” from the top of the footings to the bottom of the bent cap. Each column has fourteen #9 bars spaced at 8” on center around its circumference.

Piles were used for the foundations at each of the end bents. At each of the end bents, twenty-five steel HP 12x53 piles were used. Some of the piles were driven vertically whereas others were battered at 3:12 slope. Each of the piles in the end bents was driven to support a minimum bearing capacity of 60 tons. A pile footing support with dimensions of 7’ – 8” by 7’ – 8” by 3” deep, supports each column. Five HP 14x73 steel piles support each footing for minimum bearing capacity of 100 tons each.

2.3 Bearings

Both expansion bearings and fixed pot bearings were used to support the bridge superstructure. The expansion bearings were used at the supports at each end of the bridge girders. The fixed pot bearings were used at the supports on the intermediate pier. There are eighteen elastomeric expansion bearings and nine fixed pot bearings on this bridge. Eight of the elastomeric bearings and four of the pot bearings were placed during the stage I construction of and the rest during the stage II construction.

The elastomeric bearings are each 2'– 1" by 1' – 3" and 4" thick with elongated holes on each end to accommodate small thermal displacements around the anchor bolts. Each bearing has a beveled sole plate that is a minimum of 1.25" thick with plan dimensions of 2' – 2" by 1' – 5". The bearings have six layers of steel reinforcements. The girders were welded to the sole plates, which are anchored down by two 4" diameter steel bolts on each end. The welding of the girders to the sole plates did not take place immediately after the steel erection but rather upon completion of the bridge deck and parapet walls. The bridge superstructure was raised at each end using jacks so that the bearings could be properly aligned with the girders before the sole plates were welded to the bottom flange of each girder. Figure 4 shows one of the elastomeric expansion bearings used on the HPS 100W bridge.

Figure 4: Typical elastomeric bearing

The pot bearings used for this bridge were PF-594 fixed HLMR bearings manufactured by DS Brown Company. The piston is 14.71” in diameter and the neoprene layer in the bearing is 1” thick with a14.71” diameter. Four 1.5” diameter steel bolts were used to anchor down the bearings to the concrete bent cap.

2.4 Structural Steel Erection

The erection of the steel plate girders was carried out after the bearings were placed on the bridge seats. Each bridge girder consists of three sections that are connected through field splices. These sections were labeled A, AB, and B on the girder shop drawings (Structural Steel Products Corp., 2007). Sections A and B are composed entirely of HPS 70W steel and are located at opposite ends of the bridge in the positive moment region of each span. Section AB is located over the intermediate column bent and is composed of HPS 100W flanges and a HPS 70W web. Girder sections A and B are both 117’ - 1.375” long while girder section AB is 59’ – 7.75” long. The total length of the assembled girders is 295’ – 10.75”. Figure 5 shows a diagram of the different sections in a typical bridge girder. Figure 6 shows a photograph of some of the bridge girders after construction was completed.

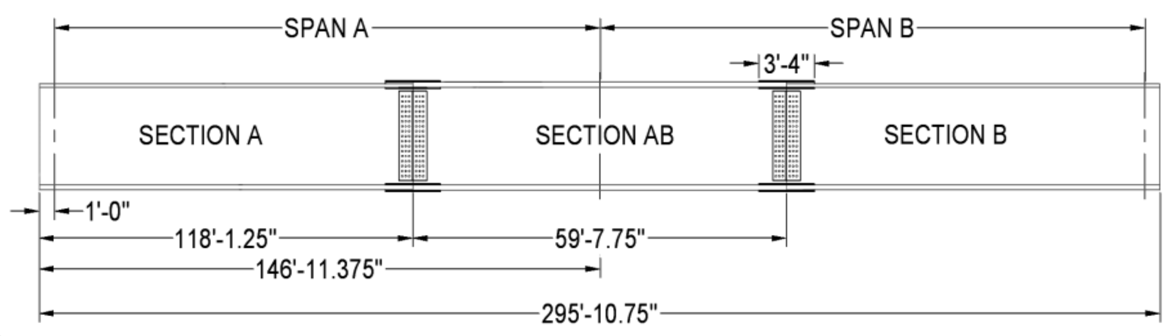


Figure 5: Typical bridge girder

Figure 6: Bridge girders

Sections A and B have the same cross sectional dimensions, whereas Section AB has thicker flanges: The top flange is 17” wide, the bottom flange is 19” wide, and the web plate is 64” deep for all three girder sections (Figure 7). The web plate is 0.5625” thick along the entire length of the bridge girders. For girder sections A and B the top flange is 1.25” thick and the bottom flange is 1.5” thick. For girder section AB, the top and bottom flanges are both 1.75” thick .

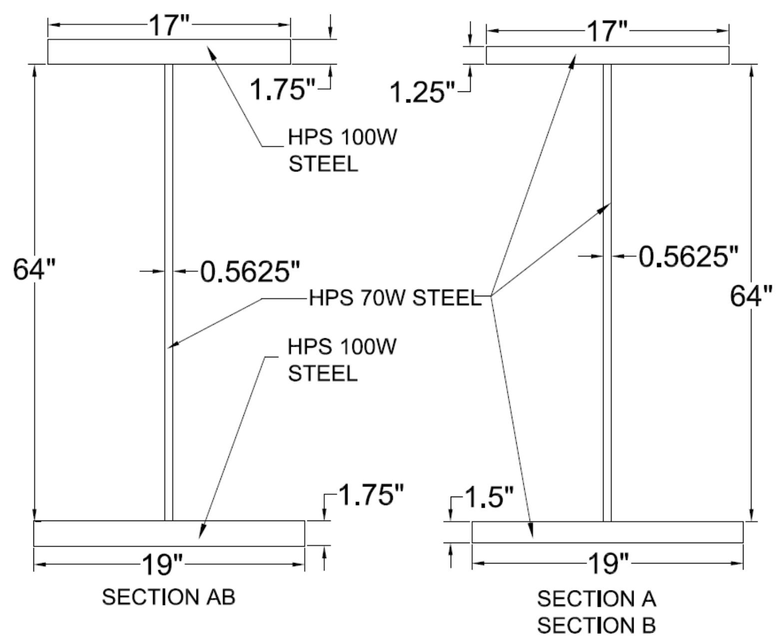


Figure 7: Cross sectional dimensions of girder sections

The intermediate cross frames consist of three L5x5x3/8 steel angles. These members are composed of AASHTO M270 Gr. 50W steel. The angles are connected directly to stiffeners that function as intermediate connection plates. The end bent diaphragms were fabricated to be of one piece. These diaphragms are composed of a MC18x42.7 channel and three WT5x13 steel members. The channels have shear studs at 1' – 0" on center so that the end bent diaphragms will act compositely with the concrete edge beams on the bridge deck. Figure 8 shows a typical intermediate cross frame and Figure 9 shows a typical end bent diaphragm.

Figure 8: Typical intermediate cross frame diaphragm

Figure 9: Typical end bent diaphragm

The erection procedure consisted of field splicing either section A or B to section AB on the ground and then lifting both sections together by crane to their final postion. The third section would then be lifted separately by another crane and set into position. The third section was spliced to section AB in the air over the interstate. For safety reasons interstate traffic was redirected along the exit and entrance ramps next to the bridge during the erection of the girders (Grey, 2009). Once the first two girders were in place, the intermediate cross frames and end bent diaphragms were connected to the intermediate and bearing stiffeners on each girder in the first bay. Once the diaphragms were in place the next girder was then erected. Figure 10 shows a diagram of the steel components of the completed bridge superstructure for both stages of construction. The bearing supports are labeled on the figure as A, B, and C.

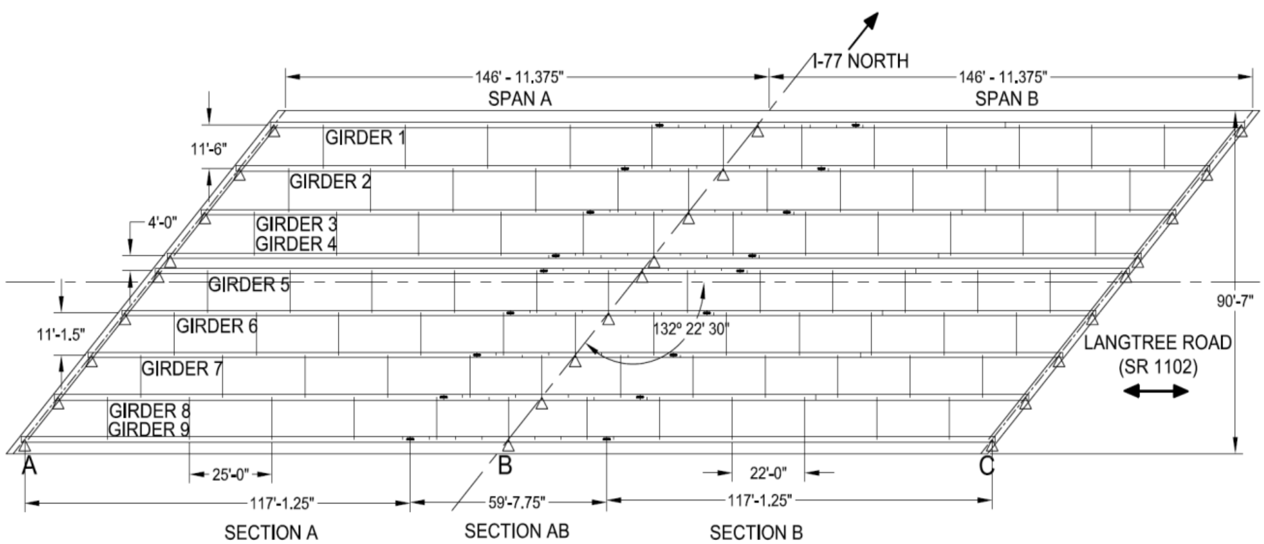


Figure 10: Layout of steel superstructure components

The steel erection was not flawless. During the stage II construction, one of the sections for girder nine became unstable while being lifted by the crane and the end of the girder twisted to one side. After detailed inspection, the girder was eventually lifted into place and construction resumed. Extra care must be taken when erecting HPS girders because they tend to be more slender than girders composed of lower strength steels (FHWA and NYSTA, Undated). The difficulty with the erection of girder nine during construction of the replacement bridge on Langtree Road reinforces the need for extra precautions when lifting HPS girders into place.

2.5 Reinforced Concrete Deck

Once the first four girders and all of the intermediate and end bent diaphragms were in place for stage I, work began on the concrete bridge deck construction. For this bridge, stay-in-place (SIP) corrugated metal formwork was used for the interior bays between the girders. Temporary formwork was used for the cantilevered section of the deck slab. Temporary formwork was also used at the longitudinal construction joints between stages I and II. To place the SIP formwork, steel angles were welded along top flanges of the girders to form the 2.5" concrete buildup. The SIP formwork was then attached to the steel angles using sheet metal screws.

After the placement of the formwork for the first stage was completed, the reinforcing steel for stage I was placed. The transverse reinforcing steel in the bridge deck consists of #5 bars on both the top and bottom of the slab, which were spaced at 5.5". In the longitudinal direction, #4 bars were placed along the top of the slab and #5 bars were placed along the bottom of the slab in the positive moment areas. The top bars are spaced at 1' – 3" on center and the bottom bars at 8" on center. Over the intermediate bent, #6 bars at 5" on center were placed in the top of the slab in the longitudinal direction. The bottom bars are of the same size and spacing as in the positive moment areas. Also at the location of the longitudinal construction joint between stage I and II, #6 dowels spaced at 1' – 0" on center were placed transversely along the length of the bridge for the eventual joining of the two stages. The bottom bars have a clear cover of 1.25” and the top bars have a clear cover of 2.5”.

After placement of the rebar for the first stage the concrete for the deck slab was poured. NCDOT class AA concrete, which has a compressive strength of 4,500 psi, was used in the deck slab (NCDOT, 2002). The concrete for the deck slab was placed in three pours for each stage of construction. For stage I, the first pour was 114.2 cubic yards, the second 137.4 cubic yards, and the third 137.9 cubic yards. For stage II, the first pour was 142.8 cubic yards, the second 172.7 cubic yards, and the third 172.5 cubic yards. The first and second pours were located at opposite ends of the bridge with the third and final pour in between the previous pours for each stage. Before the second pour could be completed, the contractor had to wait until the concrete from the first pour reached a minimum compressive strength of 3,000 psi. There was also a waiting period between the second and third pours. The width of the deck slab for stage I was 39’ – 3.5” and for stage II 51’ – 3.5” including the longitudinal construction joint. The deck slab is 9.5” thick. The overhangs at each side of bridge are 1’ – 0.25” thick and the longitudinal construction joint is 1’ – 0” thick. The total length of the slab is 298’ – 2.125”.

Once all of the concrete in the deck slab for the first stage had reached a minimum compressive strength of 3,000 psi, the concrete for the parapet wall for stage I was poured. The parapet wall dimensions are 2.5’ high by 1’ – 2” wide with a two bar aluminum rail on top. After completion of the slab and parapet walls small grooves were cut into the concrete deck in order to roughen the riding surface. In addition, the installation of other components was carried out including the guardrails, approach slabs, and armored expansion joint at each end of the bridge. The roadway was also widened to accommodate the increased width of the new bridge.

2.6 Removal of the Existing Structure and Joining of Stages I and II

Once the construction of stage I of the replacement bridge was completed traffic was redirected from the existing bridge to stage I. The existing bridge was then demolished to make room for the construction of the stage II portion of the new bridge. The contractor was required to salvage several of the bridge components during the removal of the existing structure. These components included the existing one bar metal rails and all of the component associated with the rails including the posts, nuts, washers, and screws. The remaining components of the bridge were disposed of.

After the removal of the existing structure the site was prepared for stage II construction, which was completed in the same manner as stage I. Once the concrete deck and parapet were finished for stage II, but before the installation of the rails and expansion joints, the two stages were joined together through a longitudinal construction joint. The width of the closure pour for joining the two stages was 2’ – 0”. After the closure pour was finished, other components of the bridge were installed and the roadway to bridge was completed.

3.0 WELDING DOCUMENTATION

3.1 Introduction

In general, there are few differences in the construction techniques used to build a bridge with high performance steel versus a bridge with conventional grades of steel. However the fabrication of bridge girders using high performance steel requires the use of non-traditional welding consumables and procedures (Miller, 2000). This is due to the improved weldability of HPS.

Improved weldabilty, defined as the reduction of the susceptibility of the steel to hydrogen induced cracking, allows for the use of lower preheat, heat input, and post weld treatments, which in turn leads to reduced fabrication costs (Lwin, 2002). Weldability is “the relative ease with which a metal can be welded using conventional practices” (HPS Welding Advisory Committee, 2003). Research conducted on high performance steel established that HPS is more resistant to hydrogen induced cracking and requires lower preheat and less post weld treatments (Adonyi, 2000). Although HPS does have more resistance to hydrogen cracking than traditional steels, special fabrication procedures are required (Miller, 2000).

Overall HPS steels have increased resistance to hydrogen cracking: In order to make use of the lower preheat and interpass temperature requirements, HPS steel requires special fabrication procedures that are more restrictive than for conventional steel grades (Miller, 2000). The use of proper fabrication procedures is necessary when working with HPS steel in order to ensure that welds will be adequate. If improper procedures are used, then the weld metal could become brittle due to diffusible hydrogen, which would make the weldments more susceptible to cracking. Damaged welds could lead to poor fatigue performance, lower load carrying capabilities, and structural failure.

The staff at the Structural Steel Products Corporation (SSP Corp.), the company that fabricated the girders for the new hybrid HPS 100W bridge, indicated that HPS 70W was more difficult to weld than conventional steels. The increased difficulty was due to the restrictions in the *AASHTO AWS D1.5 Bridge Welding Code* and the HPS 70W steel fabrication guide for preventing hydrogen induced cracking (AASTHO and AWS, 2002; HPS Welding Advisory Committee, 2003). They stated also that the HPS 100W was even more difficult to weld than the HPS 70W because of even greater restrictions for preventing cold cracking. In addition, the current bridge welding code (2002) at the time of fabrication did not adequately address HPS 100W.

A review of the welding properties and fabrication procedures was carried out for the hybrid HPS 100W bridge on Langtree Road with documentation of the significant properties and procedures that are critical to the welding of high performance steel. This information could be useful for the design, fabrication, and construction of future bridges containing high performance steel. The rest of this section of the construction report will be devoted to discussing the most significant weld properties for all 955 welds on the steel plate girders for the hybrid HPS bridge over I-77.

3.2 Overview of Weld Types and Locations

Based on steel material types, the welds on the bridge girders can be divided into five groups, which are: 1) HPS 70W to HPS 70W welds, 2) HPS 70W to HPS 100W welds, 3) HPS 100W to HPS 100W welds, 4) HPS 70W to AASHTO M270 Grade 50W welds, and 5) HPS 100W to AASHTO M270 Grade 50W welds. These main groups can be divided into subgroups based on the weld type. There are only two types of welds present on the bridge girders: full penetration groove welds and fillet welds.

On girder sections A and B, both flanges and the web are each composed of two HPS 70W plates that were spliced together with groove welds. The flanges were welded to the web using fillet welds. Figures 9 and 10 show the locations of the flange splice and web splice groove welds and flange to web fillet welds from the girder shop drawings (Structural Steel Products Corp., 2007). The ta14, tb14, ba14, bb14, wa14, and wb14 labels in figure 11 are the names for each top flange, bottom flange, and web plate used in the girder section (Structural Steel Products Corp., 2007). These labels, which vary for each section on all nine girders, are used by the fabricator to determine which steel plates make up each girder section. Figure 12 has similar labels (Structural Steel Products Corp., 2007).

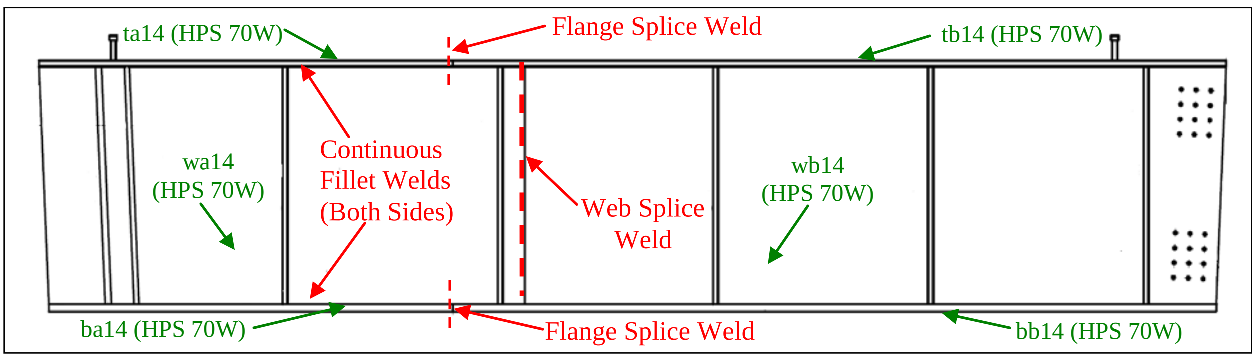


Figure 11: Girder 1 section A (Structural Steel Products Corp., 2007)

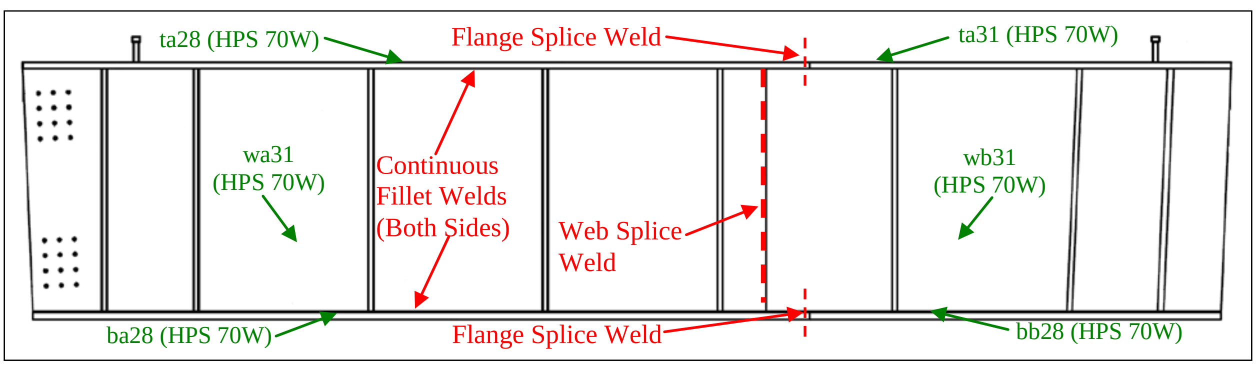


Figure 12: Girder 5 section B (Structural Steel Products Corp., 2007)

On girder section AB, the web is composed of one plate of HPS 70W steel and the flanges are composed of two HPS 100W steel plates each. The two plates for each flange were joined with groove welds and then attached to the web using fillet welds. Figure 13 shows the locations of typical flange groove welds for girder section AB from the shop drawings (Structural Steel Products Corp, 2007).

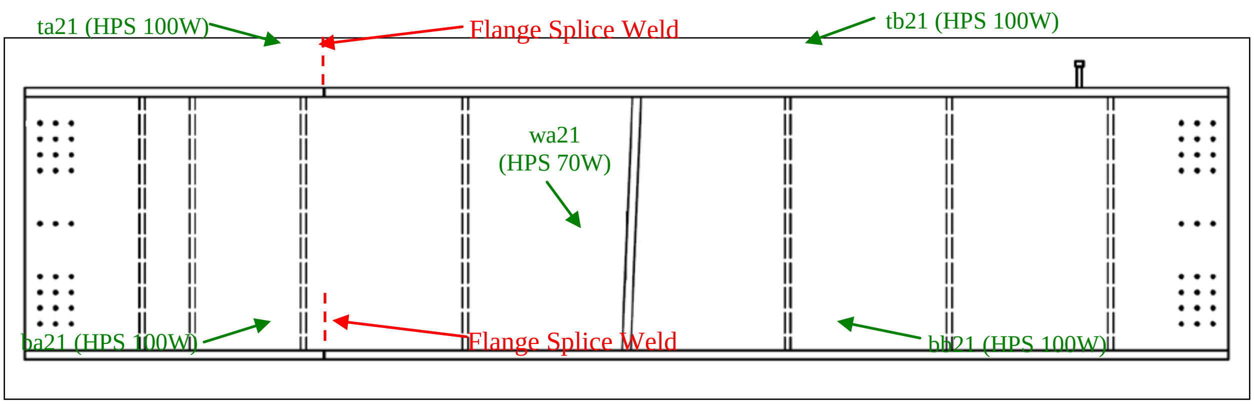


Figure 13: Girder 6 section AB (Structural Steel Products Corp., 2007)

3.3 Significant Weld Properties

Several properties can affect the quality of the welds on the bridge. These properties are summarized in table 1 below. Information relating to these properties was gathered for each weld type from the Procedure Qualification Record (PQR) and Welding Procedure Specification (WPS) documents provided by the Structural Steel Products Corporation. The fabricator did not keep records of the actual values of these properties for each individual weld during fabrication so the values specified by the PQR and WPS for each weld type are listed in this report instead. Each of the properties listed in table 1 will be discussed for all of the different welds on the hybrid HPS 100W bridge on Langtree Road in the following sections of this report.

Table 1: Documented welding properties

Welding process
Electrode
Flux
Filler metal classification
Maximum diffusible hydrogen level (mg/100L)
Preheat
Minimum and maximum heat input (kJ/in)
Maximum interpass temperature

3.4 HPS 70W to HPS 70W Weldments

This group refers to welds between HPS 70W plates only. There are three different types of welds in this group: the flange splice groove welds, the web splice groove welds, and the web to flange fillet welds. All of these welds are located on girder sections A and B.

3.4.1 Flange Splice Groove Welds

There are four flange splice groove welds between HPS 70W plates. These welds are located on the top and bottom of sections A and B only. There are thirty-six of these welds on the entire bridge. These welds are located at roughly 38' – 0" from each end of the bridge girders.

The shop drawings list these welds as B-L2c-S, which are a specific type of groove weld listed in the AASHTO AWS *D1.5 Bridge Welding Code* (Structural Steel Products Corp., 2007; AASTHO and AWS, 2002). The welding code indicates that this weld is a single "v" groove weld that required the use of backing welds instead of backing bars during fabrication (AASTHO and AWS, 2002; Hitt, 2009). This weld type can be used for joining plates between 0.5" to 2" thick (AASTHO and AWS, 2002). The top flange and bottom flanges on girder sections A and B are 1.25" and 1.5" thick and within the range specified by the code (AASTHO and AWS, 2002; Structural Steel Products Corp, 2007; NCDOT, 2006). The code states that the angle of the "v" opening should be 60^{o} ($+10^{o}$, -0^{o}) with a 0" root opening and a minimum root face of 0.375" (+0.25, -0) (AASTHO and AWS, 2002). Figure 14 shows the weld geometry for the top and bottom flange groove welds with typical dimensions (AASHTO and AWS, 2002). Table 2 summarizes the dimensions and tolerances of all thicknesses allowed for this type of weld (AASHTO and AWS, 2002).

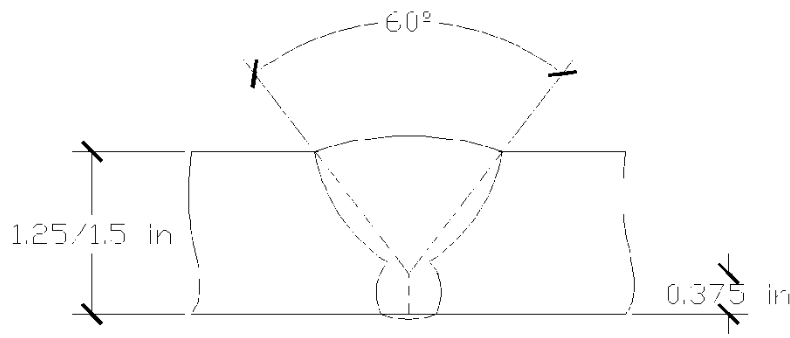


Figure 14: Typical HPS 70W flange splice weld geometry (AASTHO and AWS, 2002)

Table 2: Dimensions for B-L2c-S single "v" groove weld (AASHTO and AWS, 2002)

Welding Process	Joint Name	Base Metal Thickness	Root Opening	Root Face	Groove Angle	Allowed Positions
SAW	B-L2c-S	0.5" to 1"	0" ± 0"	1/4" + 1/4", -0"	60º + 10, -0º	F
		1" to 1.5"	0" ± 0"	3/8" + 1/4", -0"	60º + 10º, -0º	F
		1.5" to 2"	0" ± 0"	1/2" + 1/4", -0"	60º + 10º, -0º	F

In order to perform this weld, the welder would first cut the ends of the steel plates to the appropriate dimensions specified by the welding code. The side of the plates opposite of the "v" at the root of the weld would then be joined with a backing weld. Once the backing weld was completed, the groove weld would be performed on the side of the plates with the "v" cut out. Once the "v" side of the weld is completed, the backing weld would then be back gouged and the groove weld completed. In order to improve the quality of the flange splice welds at the ends of the plates, runoff tabs were used to the weld several inches past the edges of the steel plates during fabrication (Stratton, 2009). The excess weld material was removed after the weld was completed. Figure 15 shows how a typical flange splice weld was performed. Note that this figure does not show an actual flange splice for the bridge on Langtree Road but rather demonstrates the fabrication procedure.

Figure 15: Welding of a flange splice

The submerged arc welding (SAW) process was used to perform the HPS 70W flange groove welds as specified by bridge welding code figure 2.4 (AASHTO and AWS, 2002). This weld type was performed using a single arc with multiple passes. The consumable combination used for the flange splice welds consisted of the Lincolnweld LA85 electrode and the Lincolnweld MiL800-HPNi flux, both produced by the Lincoln Electric Company. The "High performance Steel Designer's Guide" states that the LA85 electrode and MiL800-HPNi flux have been demonstrated to produce quality weld metal for HPS 70W steel plates produced using both Q&T and TMCP methods (Lwin, 2002). The electrode size was 3/32" and the filler metal classification was F9A4-ENi5-G-H2, which indicates that the consumables should have been manufactured with a diffusible hydrogen level less than 2 mL/100g. For these welds, the maximum diffusible hydrogen was limited to 4 mL/100 grams or H4. For the HPS 70W flange groove welds on the I-77 bridge the maximum heat input was 47.6 kJ/in and the maximum interpass temperature was 450^{o}F. The required preheat for the top and bottom flange of the HPS 70W plates was 125^{o}F in order to limit diffusible hydrogen to a maximum of 4 ml/100g.

3.4.2 Web Splice Groove Welds

The groove welds for splicing together the HPS 70W web plates are located on girder sections A and B only. These welds are located approximately 43’ from the end of girder section A or 41’ from the end of girder section B. The web splice welds were offset approximately 5’ – 0” from the flange splice welds. There are two web splice welds per girder, and eighteen on the entire bridge.

These welds are listed as B-U7-S groove welds on the shop drawings (Structural Steel Products Corp. 2007). The B-U7-S designation refers to weld type in figure 2.4 of the *AASHTO AWS D1.5 Bridge Welding Code* (AASHTO and AWS, 2002), which is a double “u” groove. This weld has a “u” shape cut into each side of the web plates before the welding is performed. The welding code states that this weld can be used for steel plates of unlimited thickness (AASHTO and AWS, 2002). The bridge welding code states that this weld type has no root opening (±0”), a groove angle of 20^{o}, a maximum root face of 0.25” (+0, -0.25”), and a groove radius of 0.25” (AASHTO and AWS, 2002). Figure 16 shows the weld geometry with the actual dimensions on the bridge. This weld is fabricated by first performing a backing weld on one side of the steel plates to temporary hold them together while the first groove weld is performed on the other side. The backing weld is then back gouged and the second groove weld is completed.

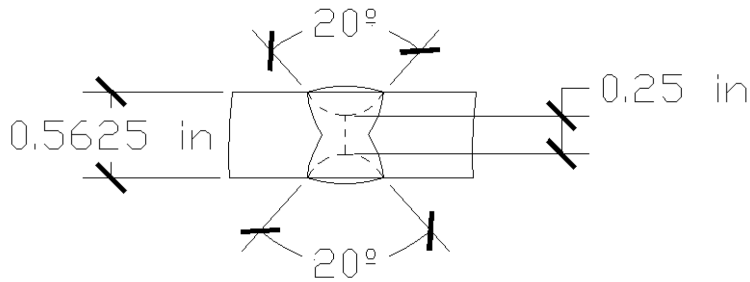


Figure 16: Typical HPS 70W web splice geometry (AASHTO and AWS, 2002)

The SAW welding process was also used to perform the web splice groove welds on girder sections A and B. A single arc with multiple passes was required in order to build the weld up to the same thickness as the steel web plates. The same 3/32" Lincolnweld LA85 electrode and Lincolnweld MiL800-HPNi flux were used in the web splice groove welds as in the flange splice groove welds discussed earlier. The web splice double "u" groove welds for the same filler metal classification of F9A4-ENi5-G-H2 as the flange splice welds for the HPS 70W flange plates on girder sections A and B. The web splice welds also have the same maximum diffusible hydrogen limit of H4 or 4 mL/100g. The HPS 70W web splice welds required the same maximum heat input of 47.6 kJ/in as the HPS 70W flange splice welds as well as the same maximum interpass temperature of 450^{o}F. The required preheat for the web splice welds was 50^{o}F. This is much lower than the 125^{o}F required for the flange splice welds. The lower preheat is due to the difference in thickness of the web plates versus the flange plates.

3.4.3 Web to Flange Fillet Welds

The flange to web fillet welds between HPS 70W steel plates are located on girder sections A and B only. Two 5/16" fillet welds join each flange to the web for a total of four welds per girder section, eight per girder, and thirty-six on the entire bridge. These welds are continuous for the entire length of sections A and B and are approximately 118' long (Structural Steel Products Corp, 2007). Figure 17 shows the layout and dimensions of the flange to web joint.

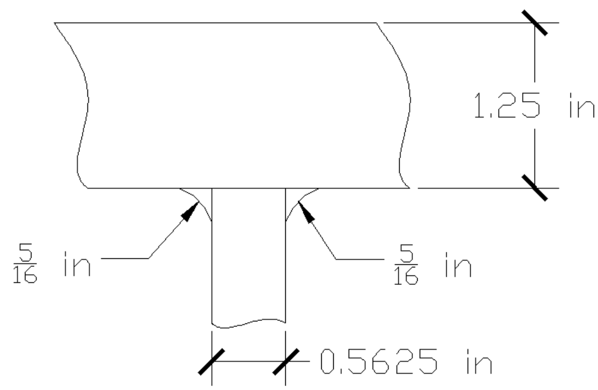


Figure 17: HPS 70W top flange to HPS 70W web 5/16" fillet welds for sections A and B

These welds were not performed manually but rather on a gantry by a welding machine. The machine performs the individual fillet welds in the t-joint for each flange simultaneously. The machine was able to perform the fillet welds by using two arcs, one on each side of the web plate (Stratton, 2009). Figure 18 shows a photograph of the gantry used to perform these welds at the Structural Steel Products Corporation.

The SAW process along with the Lincolnweld LA-85 electrode and Lincolnweld MiL800-HPNi flux was also used for these welds. Two arcs were used in a single pass by the welding machine. The first arc was a 5/32" direct current (DC) lead followed by the second arc, which was a 1/8" alternating current (AC) trail. These welds had the same filler metal classification of F9A4-ENi5-G-H2 and the same maximum diffusible hydrogen limit of 4 mL/100g as the flange and web splice welds.

Figure 18: Machine for performing web to flange fillet welds

The heat input, minimum preheats, and maximum interpass temperature for the flange to web joints were different than for the groove welds at the flange and web splice welds. The heat input ranged from a minimum value of 39.72 kJ/in to a maximum of 66.20 kJ/in. The maximum interpass temperature for this weld type was 400^{o}F. The preheat for these welds was 70^{o}F.

3.5 HPS 70W to HPS 100W Weldments

The second group of welds is those between HPS 70W plates and HPS 100W steel plates. There is only one type of weld in this category, which is the 5/16” fillet welds between the HPS 70W web plates and the HPS 100W flange plates of girder section AB. These welds are continuous along the length of the top and bottom flange joints of girder section AB, which makes them approximately 59’ – 7.75” long. There are two of these welds per girder and a total of eighteen on the entire bridge. Figure 19 shows the geometry of the HPS 70W web to HPS 100W flange fillet welds. These welds were also fabricated using the same equipment as the HPS 70W flange to web fillet welds.

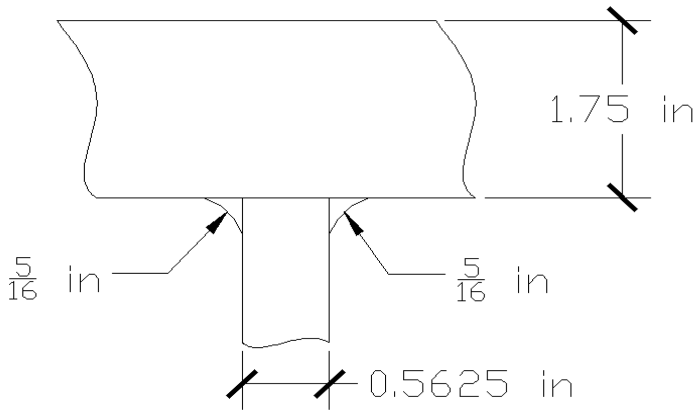


Figure 19: HPS 100W flange to HPS 70W web fillet welds for section AB

The SAW welding process, LA-85 electrode, and MiL800-HPNi flux were also used to weld the HPS 70W web plates to the HPS 100W flange plates for section AB on all nine girders. Two arcs were also used for this weld. The first arc was a 5/32" DC lead and the second arc was a 1/8" AC trail. The filler metal classification was F9A4-ENi5-G-H2, the same as all welds previously mentioned in this report. The diffusible hydrogen limit was also 4 mL/100g.

The heat input and maximum interpass temperature are the same as for the HPS 70W flange to HPS 70W fillet welds. The minimum heat input was 39.72 kJ/in and the maximum heat input was 66.20 kJ/in. The maximum interpass temperature was 400^{o}F. The minimum preheat for non-fracture critical applications for HPS 100W was 250^{o}F. Since HPS 100W steel was still in development there was no design or fabrication guide with recommended preheats at the time of fabrication. The bridge welding code (2002) did not address HPS 100W steel. The preheat for the HPS 100W came from the manufacturer's recommendations (Hitt, 2010).

3.6 HPS 100W to HPS 100W Weldments

There is only one type of weld between HPS 100W plates on the Langtree Road bridge over I-77, which is the top and bottom flange splice groove welds between the HPS 100W plates on section AB for all nine girders. These welds are located 14' – 10" from the end of section AB that attaches to section A. There are two of these welds per girder for a total of eighteen on the entire bridge.

The shop drawings specify that the flange splice welds on section AB are B-L2c-S single "v" groove welds, which is the same groove weld used for the flange splices on girder sections A and B. The specification for this weld type can be found in Figure 2.4 of the *AASHTO AWS D1.5 Bridge Welding Code* (AASHTO and AWS, 2002). Figure 20 shows the weld geometry with typical dimensions. This groove weld was performed by first cutting a "v" shape on one side of the two flange plates. A backing weld was used to temporarily connect the two flange plates while the welder built up the groove weld through multiple passes in the "v" cutout. The backing weld was then back gouged so that the groove weld could be completed.

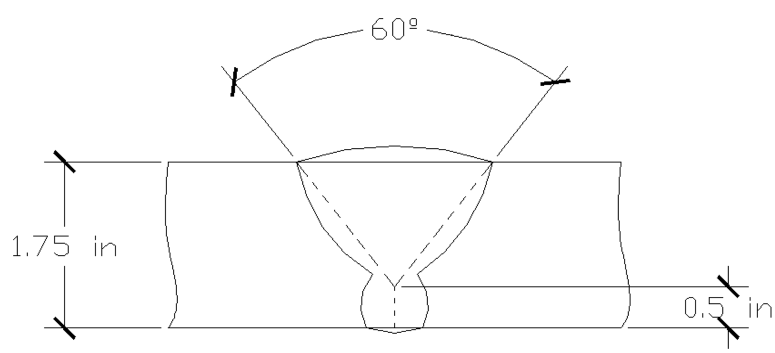


Figure 20: Typical HPS 100W flange splice weld (AASHTO and AWS, 2002)

The SAW welding process was used to fabricate the flange splice welds between the HPS 100W flange plates of section AB. A single arc was used to perform this weld; multiple passes were required to build up the weld to the same thickness as the flange plates. The Lincolnweld

LA-100 electrode along with the Lincolnweld MiL800-HPNi flux was used for these welds. The electrode size was 3/32". The filler metal classification for this consumable combination was F11A4-EM2-G-H2. The Lincoln Electric Company has conducted testing on this consumable combination, which established that it is suitable for creating matching strength welds for HPS 100W steel as long as the heat input is kept below 80kJ/in (Yost and James, 2005). The maximum diffusible hydrogen limit for the HPS 100W flange splice welds on section AB was 4 mL/100g, the same as for the HPS 70W flange splice, web splice, web to flange joint, and the HPS 100W flange to HPS 70W web joint.

The minimum and maximum heat input for the HPS 100W flange splice welds were 53.56 kJ/in and 55.67 kJ/in, respectively. The range for the heat input for these welds was much tighter than for many of the other welds on the bridge. The heat input for these welds was below the maximum heat input for the LA100 and MiL800-HPNi flux combination of 80kJ/in (Yost and James, 2005). The minimum preheat for this weld type was 250^{o}F. The maximum interpass temperature was 400^{o}F.

3.7 HPS 70W to AASHTO M270 Grade 50W

The bearing stiffeners, diaphragm connection plates and intermediate stiffeners are all composed of AASHTO M270 Gr. 50W steel. The stiffeners were welded to the flanges and web using fillet welds. There are two types of welds between HPS 70W steel and Grade 50W steel. The first type is the welds between the HPS 70W flanges and the Grade 50W stiffeners. The second type is between the HPS 70W web and the Grade 50W stiffeners. Even though both types of welds are fillet welds between the same grades of steel, they use different welding processes and electrodes.

3.7.1 HPS 70W flanges to AASHTO M270 Gr. 50W Stiffeners

The fillet welds between the HPS 70W flanges and the Gr. 50W stiffeners are located on sections A and B of all nine girders. The locations of these welds vary depending on the girder. There are several hundreds of these welds on the bridge with geometry varies depending on the stiffener type. A 5/16” fillet weld was used for the flange to stiffener connection on girder sections A and B. The weld lengths range from 4.5” - 6.5” (±0.25”) depending on the stiffener size.

These welds were performed using the SMAW process. A single arc was used and depending on the weld, single or multiple passes may have been used. An E8018 C3 H4R electrode was used for this weld, the manufacturer was not specified. Three different electrode sizes were specified as 3/32”, 1/8”, and 5/32”. The electrode was classified as hydrogen resistant and the maximum diffusible hydrogen for this weld type was 8 mL/100g.

The minimum and maximum heat inputs were not specified for this weld type on the WPS. In addition, there was no maximum interpass temperature listed. A 125^{o}F preheat was used for these welds (Hitt, 2010). The lack of information on the heat input and interpass temperature is because this type of weld is considered prequalified by the *AASHTO AWS D1.5 Bridge Welding Code* (Hitt, 2010).

3.7.2 HPS 70W web to AASHTO M270 Gr. 50W Stiffeners

The fillet welds between the HPS 70W web and the Gr. 50W stiffeners are located on sections A, B, and AB for all nine girders. There are a large number of these welds on the bridge. A 1/4” fillet weld was used to attach the stiffeners to the web. The weld lengths are all 57” ±0.25” because the web depth is constant throughout the entire girder.

These welds were performed using the SAW welding process. A single arc with a 1/8" electrode was used to perform these welds. The Lincolnweld L61 electrode and the Lincolnweld AXXX10 flux were used. This consumable combination has a filler metal classification of F7A4-Ni1-H8. The allowable diffusible hydrogen limit for these welds is 8 mL/100g. The minimum heat input was 40.58 kJ/in and the maximum heat input was 62.64 kJ/in. The minimum preheat was 50^{o}F and the maximum interpass temperature was 400^{o}F.

3.8 HPS 100W to AASHTO M270 Grade 50W

Fillet welds were used between the AASHTO M270 Grade 50W stiffeners and the HPS 100W flanges. These welds are only located on section AB for each girder. The exact location and number of welds vary between the different girder sections for this weld type. The fillet weld size for these connections is 5/16". The weld length varies from 4.5" (±0.25") to 6.5" (±0.25") depending on the stiffener.

The SMAW process was used to perform these welds. A single arc was used to perform these welds both a single pass or in multiple passes depending on the weld. Three different electrode sizes were listed for this weld type, 3/32", 1/8", and 5/16". The same E 8018 C3 H4R electrode classification was used for these welds as for the HPS 70W flange to Gr. 50W stiffener welds on girder sections A and B. The maximum diffusible hydrogen limit for this weld type was 4 mL/100g. There was no maximum or minimum heat inputs or maximum interpass temperature listed for these welds. A minimum preheat of 175^{o}F was used for this group of welds (Hitt, 2010).

4.0 STATIC LOAD TEST

4.1 Introduction

This section presents the procedure and results of static truck load testing that was conducted on the constructed hybrid HPS steel girder bridge. A new laser-based technique for measuring bridge deformations was implemented during the test. This technique involves the use of a LiDAR scanner, which can store three-dimensional geometrical information into images. These scan images, recorded before and after load placement, can be compared to quantify bridge displacements. This non-contact method is ideally suited to the testing of structures that are inaccessible for instrumentation. In case of the hybrid HPS bridge on Langtree Road, Interstate 77 highway underneath the bridge made instrumentation with traditional equipment such as strain gauges impossible. In order to verify the accuracy this laser based technique, several linear variable displacement transducers (LVDT) were placed in accessible locations on the bridge, which is near the ends of the girders over the shoulder of I-77 highway.

4.2 Bridge Condition and Layout during Field Testing

The bridge was nearly completed at the time of the static load testing. All major structural components on the bridge were finished. Minor components such as one of the parapet railings, the guardrails, and the concrete median at the center of the bridge were not yet completed. Since the bridge was partially closed to traffic at the time of the testing, a precast concrete jersey barrier was present on the bridge. This barrier ran the entire length of the bridge and weighed approximately 400 lbs/ft (Grey, 2009). Figure 21 shows a layout of the bridge deck during the static load testing.

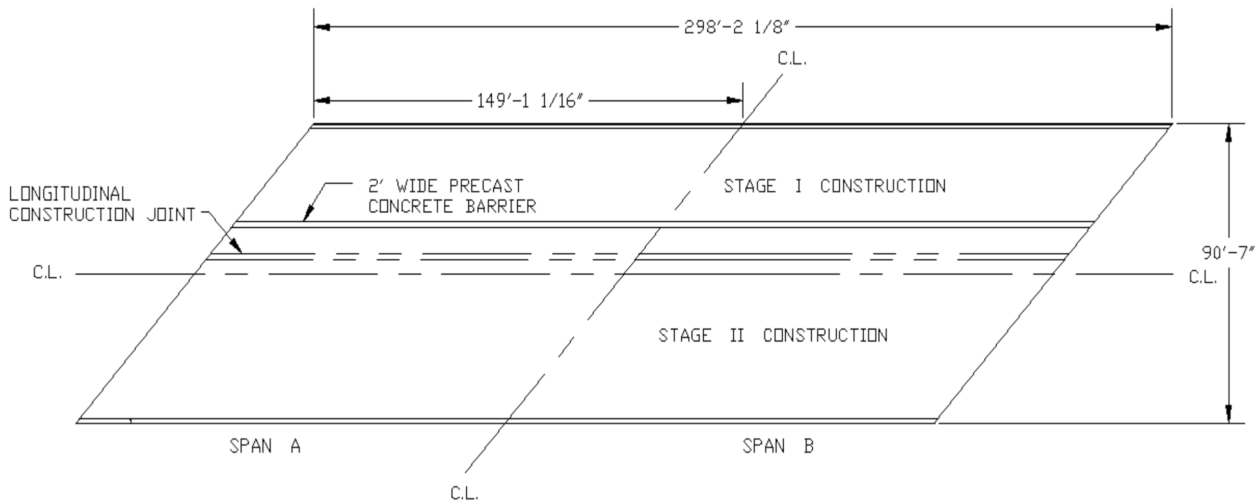


Figure 21: Layout of bridge deck during static load testing

4.3 Truck Information

The North Carolina Department of Transportation provided two tandem dump trucks for the static load test. Both trucks had approximately the same dimensions and weight. Figure 22 shows the wheel layout and axle dimensions for the dumps trucks used during the testing.

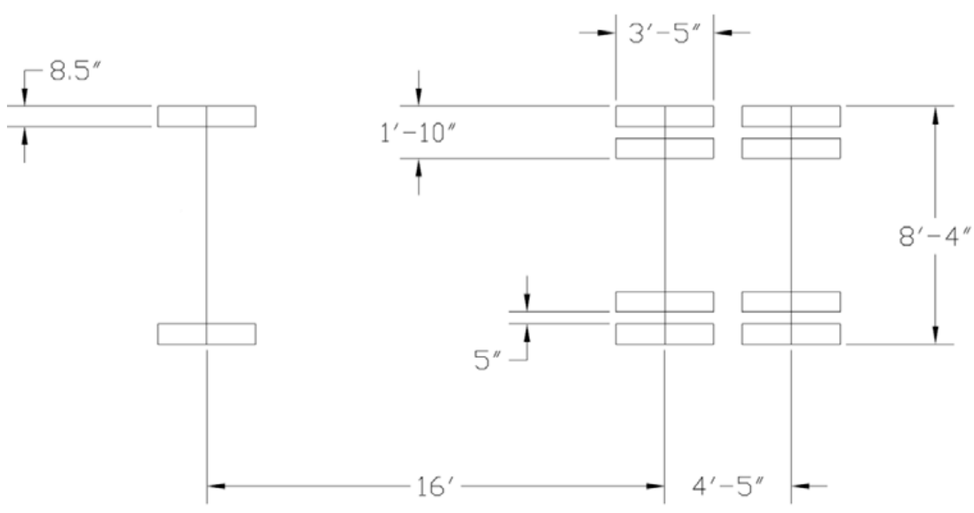


Figure 22: Truck dimensions and wheel layout

Each truck weighed approximately 25,000 lbs unloaded and approximately 55,000 lbs when fully loaded with stone from a local quarry. Each truck was weighed at the quarry in order to determine the total weight. The load distribution to each wheel was determined when the trucks arrived at the bridge by a highway patrol officer using portable scales. Tables 3 and 4 summarize the weight distribution for each of the trucks.

Table 3: Weight information for truck A (lbs)

Vehicle weight	24,660			
Stone weight	30,980			
Total (vehicle + stone) weight	55,640			
Front axle	Left tire	7,820	Right tire	7,720
First rear tandem axle	Left tire pair	9,420	Right tire pair	10,620
Second rear tandem axle	Left tire pair	9,640	Right tire pair	10,160

Table 4: Weight information for truck B (lbs)

Vehicle weight	24,920			
Stone weight	29,900			
Total (vehicle + stone) weight	54,820			
Front axle	Left tire	7,520	Right tire	7,780
First rear tandem axle	Left tire pair	9,400	Right tire pair	10,500
Second rear tandem axle	Left tire pair	8,840	Right tire pair	10,760

4.4 Truck Configuration and Placement

In order to measure the girder deflections using the laser image scanner, the two trucks had to remain stationary for the duration of each scan by the laser. Traffic control on the bridge was necessary to prevent other vehicles from disrupting the load testing until each scan was completed. A typical scan during this test lasted around ten minutes. In order to minimize the disruption to traffic only three load configurations were used for this load test.

All of the configurations were located on span B. Each truck position is located approximately 62' from the transverse centerline of the bridge. Figure 23 shows all three truck positions used for the load testing. A diagram of each truck position is shown in figures 24 through 29.

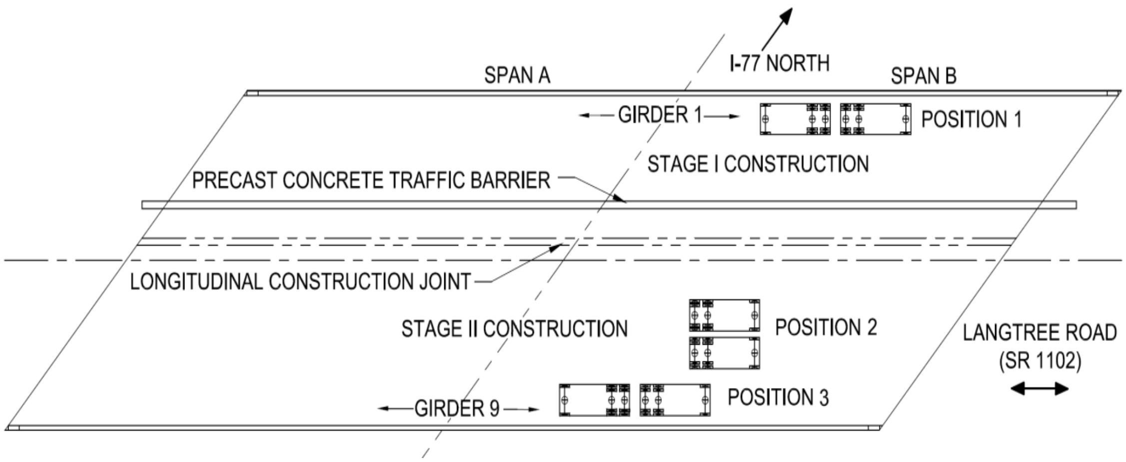


Figure 23: All truck configurations used during load testing

Figure 24: Truck position one

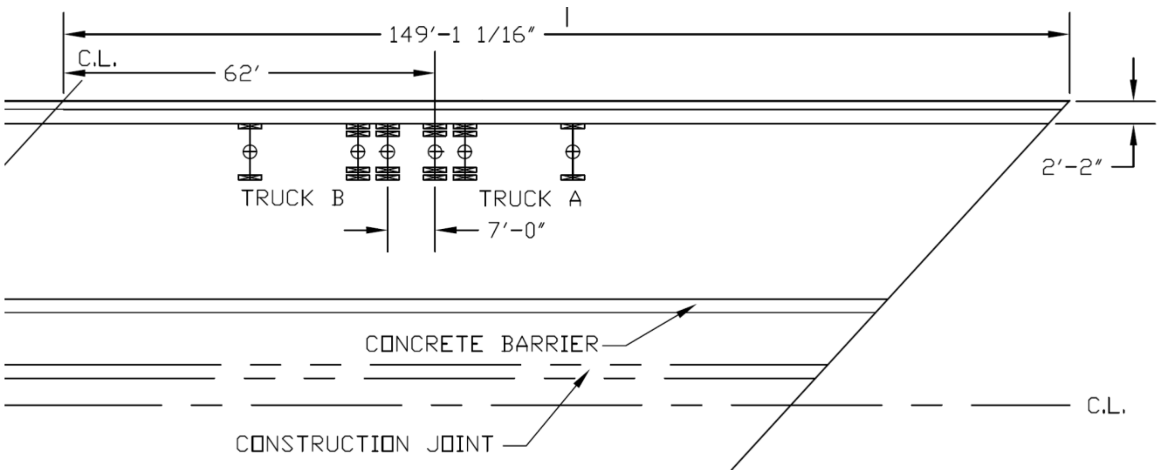


Figure 25: Placement of truck one

Figure 25: Truck position two

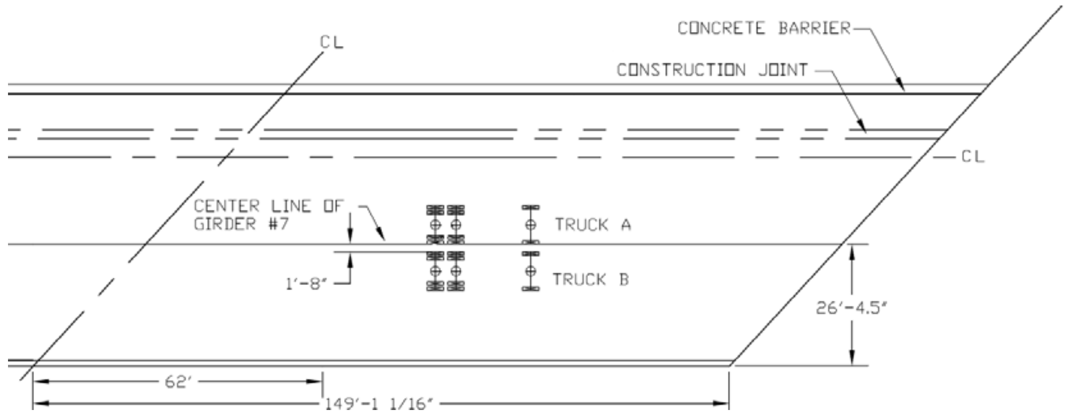


Figure 27: Placement of truck two

Figure 28: Truck position three

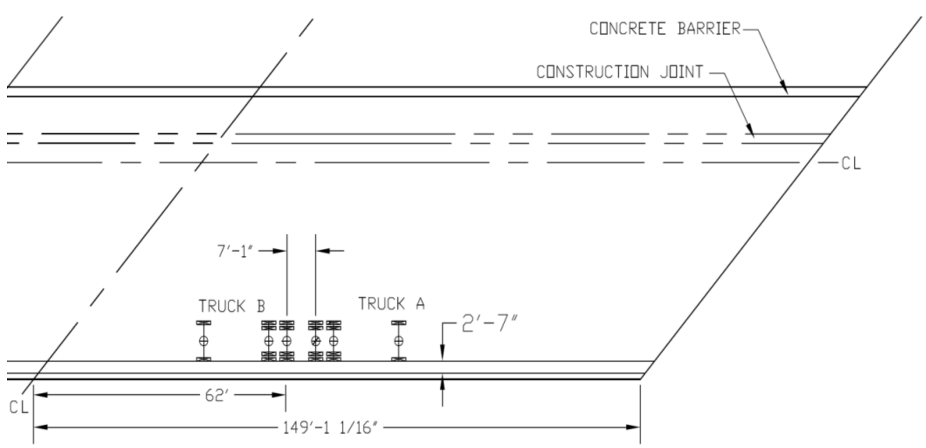


Figure 29: Placement of truck three

4.5 LiDAR Scanner

The LiDAR system used for the load test is a non-contact laser image scanner that collects three-dimensional measurements for thousands of points on an object. The system measures data points in both polar and Cartesian coordinates. The Faro LS 880HE laser system was used to measure the bridge deformations and geometry during this test (Faro Technology, 2007a). Laser scanning instruments have been used to measure bridge deformations during load testing in several other cases (Fuchs et al., 2004a; Fuchs et al., 2004b). The LiDAR system used for testing the hybrid HPS bridge over I-77 has also been successfully used for quantifying damages and measuring geometry and clearances in multiple bridge structures (Liu, 2010).

The LiDAR system has a 360^{o} field of view in the horizontal plane and a 320^{o} field of view in the vertical plane (Faro Technology, 2007a). The system measures distances by transmitting a laser pulse to an object and then detects the reflection of the beam in order to calculate the distance between the laser and the object. The laser system records the coordinates of the points into images that can be viewed with the Faro Scout LT software (Faro Technology, 2007b). For the load testing of the hybrid HPS 100W bridge, a gird of 9,000 by 4,000 points was used with an average scan time for each truck configuration of approximately ten minutes. The scan time can increase or decrease based on the number of points that are necessary for the desired accuracy level. The technical specifications for the laser system are listed in Table 5 (Faro Technology, 2007a). Figure 30 shows a photograph of the Faro LS 880HE laser scanner.

In order to use the LiDAR scanner to measure bridge deflections a simple method was developed that involves comparing the results of two laser scan images. First, a base line scan of the bridge is performed with no vehicles on the structure. Upon completion of the first scan, the bridge can then be loaded and rescanned with the laser. During this process, the laser system

Table 5: Specifications for Faro LS 880HE laser scanner (Faro Technology, 2007a)

Criteria	Specification
L x W x H	15.75 in x 6.3 in x 11 in
Weight	35 lbs
Range	1.96 ft to 249.3 ft
Resolution	0.024 in 17 bit range
Measurement Speed	120,000 points/second
Measurement error	0.12 in at 82 ft
Vertical field of view	320^o
Horizontal field of view	360^o
Vertical resolution	0.009^o (40.000 3D-pixel on 360^o)
Horizontal resolution	0.00076^o (470.000 3D-pixel on 360^o)
Angular resolution	$\pm 0.009^o$
Wavelength	785 nm
Beam divergence	0.014^o
Beam diameter	0.12 in, circular
Power supply and consumption	24 V DC (Battery pack or AC converter) Approx. 60W
Data storage	Internal PC 40GB hard drive, Windows 2000, Windows XP, Ethernet port for external storage
Ambient temperature	41^oF to 104^oF
Inclination Sensor	Accuracy 0.1^o, resolution 0.001^o, range $\pm 15^o$

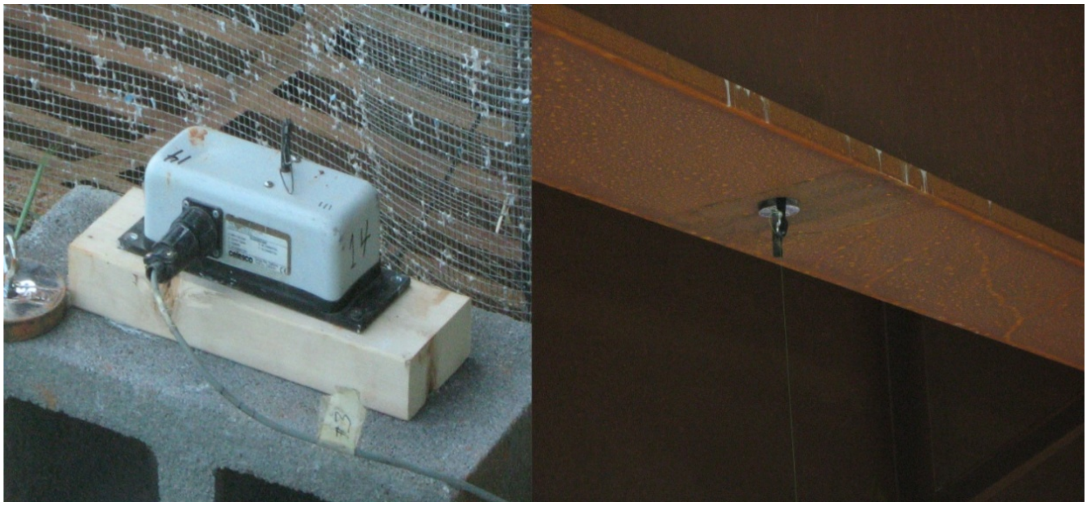
Figure 30: Faro LS 880HE laser scanner

cannot be moved and other vehicles must be temporarily stopped from crossing the bridge while a scan is in progress. Multiple load configurations and scans can be performed in a relatively short time using this technology. The three load cases for this bridge took approximately two hours to complete not counting the set up time. More time was required to position the trucks than to perform a scan.

The bridge deflections can then be determined using an automated method that makes used of a computer program that was developed by researchers at UNC Charlotte (Liu, 2010). The baseline scan must first be loaded into the Faro Scout LT software first (Faro Technology, 2007b). The program develops a horizontal reference plane of points underneath the bridge superstructure points. Points on the bridge structure and corresponding points on the reference plane are selected and the distance between them is determined for each scan (Liu, 2010). The difference in the heights of the two scans is the vertical deflection of the bridge superstructure (Liu, 2010). The use of the reference plane in the program helps to ensure that the points being compared are at the same location in each scan image, which improves the accuracy of the measurements. However, this program does require several hours to run depending on the point grid size used in the scan.

4.6 Linear Variable Transducers

Three linear variable displacement transducers were also placed on the bridge in addition to using the LiDAR system. Transducers were placed on span B near the ends of girder one, girder seven, and girder nine. This was the only location to which the transducers could be attached due to the interstate below the bridge. The transducers were not able to measure the maximum displacement of the girders in this position; however only the displacement at a location that could also be measured by the laser scanner was needed to compare the two methods. Figure 31 shows the transducers used for the load testing.



(a) LVDT equipment (b) LVDT attachment

Figure 31: Typical transducer set up

4.7 LiDAR Scan Results

The laser scan results consist of the raw geometrical data stored in the LiDAR images and the deflection results computed. Figure 32 shows a screenshot of a typical laser scan image. Figures 33, 34, and 35 show the live load deflection results for truck positions one, two, and three, respectively (Liu, 2010). In figures 33 through 35, the warmer colors represent the larger downward deflections whereas the cooler colors represent the smaller downward deflections and in some cases upward deflections. In these images, the points shown are from the bottom flanges of the girders and some of the cross frames. Only the points on the middle of span B where the trucks were placed are shown. In all three images showing the deflection results from the computer analysis, the shaded area between girders four and five is due to some plywood that was left over from construction.

Figure 32: Typical LiDAR scan image

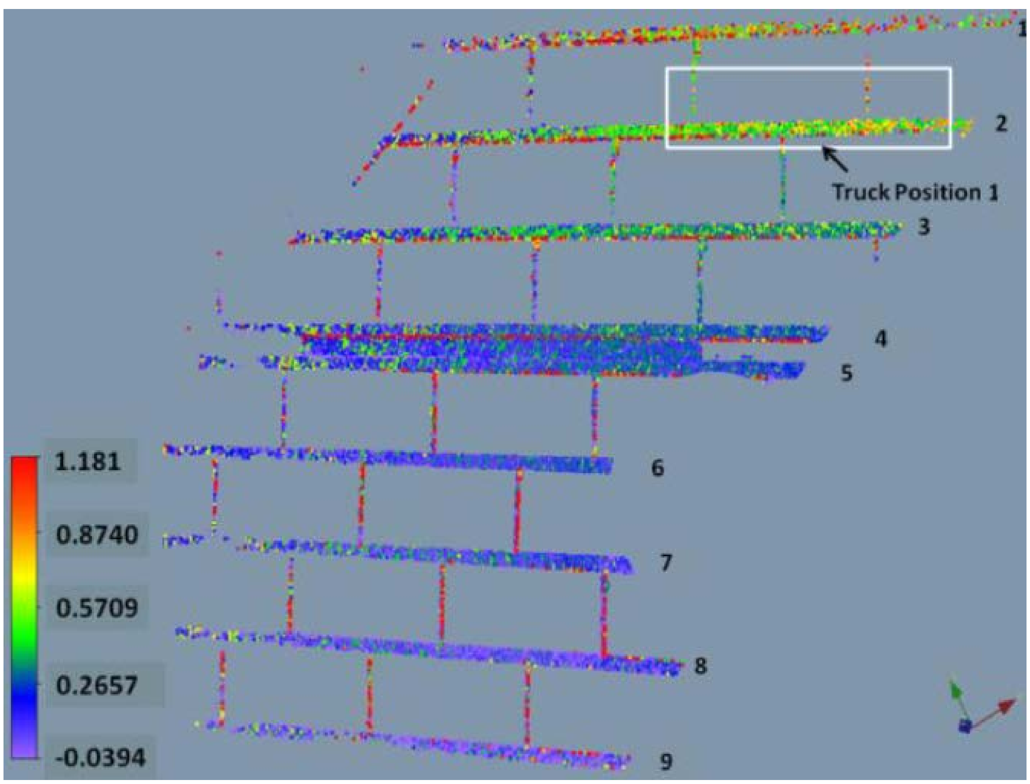


Figure 33: Deflection results for load case one (in)

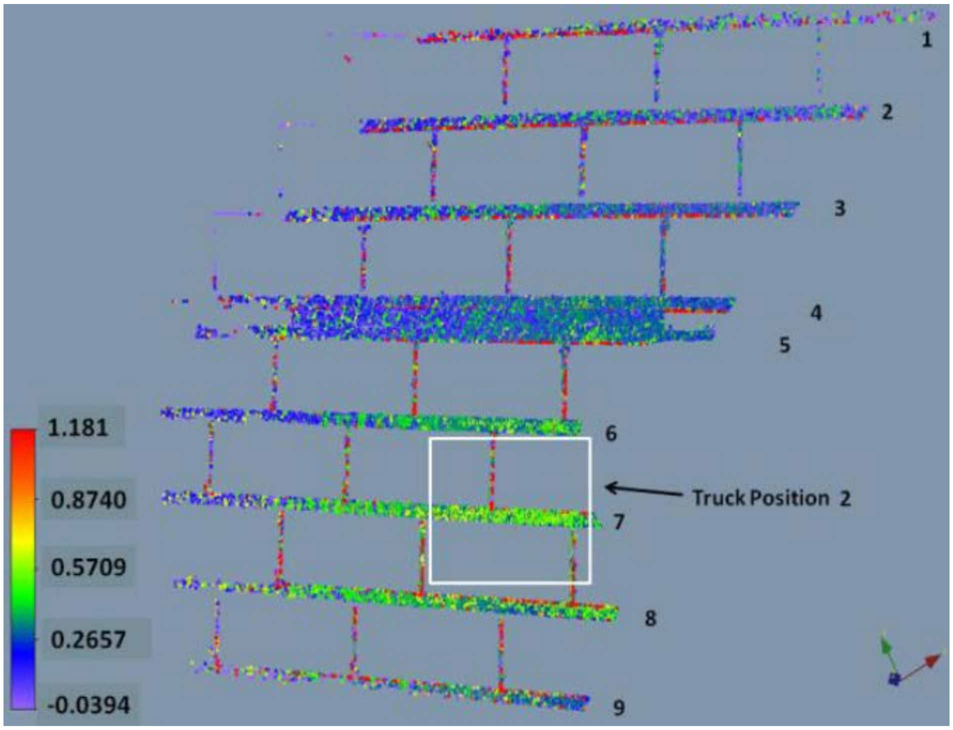


Figure 34: Deflection results for load case two (in)

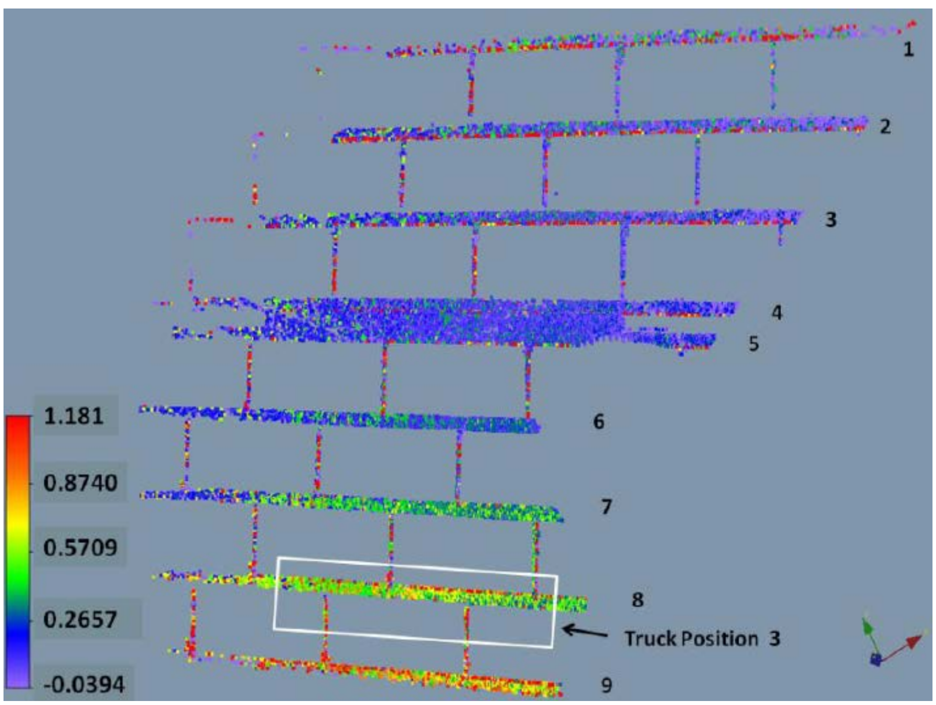


Figure 35: Deflection results for load case three (in)

The deflections from the LiDAR images at each of the transducer locations on girders one, seven, and nine are summarized in table 6. The highest deflections at each transducer location occur when the girder the transducer was attached to was loaded.

Table 6: LiDAR deflection results at transducer locations

Load case	Girder 1	Girder 7	Girder 9
Truck 1	0.59”	0.04”	0.06”
Truck 2	0.16”	0.36”	0.29”
Truck 3	-0.23”	-0.11”	0.63”

4.8 Linear Variable Transducer Results

The transducers measured the deflection at a single point on girder one, girder seven, and girder nine for each of the three truck load cases. Figure 36 shows the reading measured by each transducer throughout the duration of the load test. The deflection of each girder during load cases one, two, and three can be determined by examining the sudden drops in the time history graph. The transducers were set up an hour before the testing for span B was performed so the readings from the set up to the first load position are not shown on the graph.

For the first load position, only the transducer on girder one was close enough to the loading to measure any displacement. This transducer recorded a displacement of 0.54” due to the truck loading. The transducers on girders seven and nine measured no deformations. Given the distance from the truck loading to the transducers on girders seven and nine, it seems reasonable that only the transducer on girder one would measure any deformations.

For the second position, the transducer on girder seven recorded a displacement of 0.44” and the transducer on girder nine recorded a lower displacement of 0.23”. The transducer on girder one did not detect any deflections for this load case. Again it seems reasonable that for load case two where the trucks were positioned over girder seven that the transducer on girder one would not measure and downward deflections. It was also expected that girder seven would displace more than girder nine.

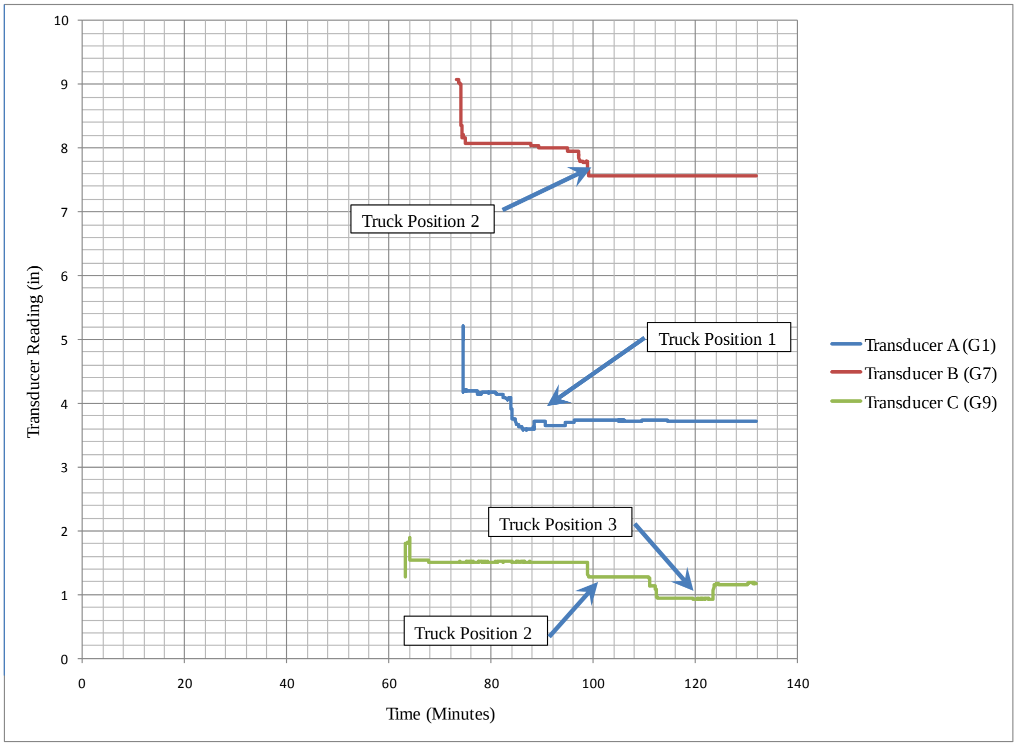


Figure 36: Transducer time history for span B load configurations

For position three only the transducer on girder nine, which was directly under the load, recorded any displacement. This transducer indicated that a downward deflection of 0.34" occurred on girder nine due to the truck loading. For position three the transducer on girder seven should have been close enough to measure some deflection, since no significant deformations were recorded the instrument may have been malfunctioning. The transducer on girder one did not measure any significant displacements due to the truck loading on girder nine as expected. The displacement recorded by each transducer for all load cases on span B is shown as table 7. In this table, positive values are for downward deflections and negative values are for upward deflections.

Table 7: Transducer displacement for each load case (in)

Girder	1	7	9
Truck position 1	0.54	0.00	0.01
Truck position 2	-0.08	0.44	0.23
Truck position 3	0.00	-0.01	0.34

4.9 Summary of Field Testing

This section presented the description and results of the static load testing carried out on the hybrid HPS 100W bridge over I-77. As part of the load testing, a new laser based technique was used to measure the bridge deflections. Linear variable displacement transducers were also used to measure the bridge deflections. The data from this load test was used to validate a finite element model of the hybrid HPS 100W bridge on Langtree road over I-77. This model will be used in future research to conduct more analyses on this bridge that could not be performed experimentally.

5.0 CONCLUSION

This report presented a description of the construction of the hybrid HPS 100W steel girder bridge over I-77 in Iredell County North Carolina. In addition, the fabrication procedures for the steel girders, specifically the weld details, were documented in this report. A description of a load test carried out on the bridge using a laser image scanner to measure deflections was presented in this report as well as some of the results of the test. The information documented in this construction report may be of use during the fabrication of other HPS steel girders and the construction of future bridges that contain HPS steel.

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