Product Application of a Hybrid-Composite Beam System

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IDEA PROGRAM

Funding and technical support for this project was provided by the High-Speed Rail IDEA Program and the NCHRP Highway IDEA Program. The mission of the IDEA Programs is to foster innovation in transportation by providing start-up R&D funding and support for promising but unproven concepts.

The High-Speed Rail-IDEA Program is funded by the Federal Railroad administration and managed by the Transportation Research Board (TRB) of the National Research Council. NCHRP Highway IDEA, which focuses on advances in the design, construction, safety, and maintenance of highway systems, is part of the National Cooperative Highway Research Program. The High-Speed Rail-IDEA Program and the NCHRP Highway IDEA Program are two of four IDEA programs managed by TRB. The other two are Transit IDEA, and Safety IDEA.

- Transit IDEA, which seeks advances in efficiency, safety, and maintenance of transit systems, is part of the Transit Cooperative Research Program and funded by the Federal Transit Administration.
- Safety IDEA focuses on innovative approaches to improving railroad, intercity bus, and truck safety. The program is supported by the Federal Motor Carrier Safety Administration and the FRA.

IDEA programs support new concepts often too risky for traditional R&D funding. The focus is on concepts with potential for technology breakthroughs rather than incremental improvements to conventional technology.

Proposals to the IDEA program are welcome from any business, organization, institution, or individual. All proposals are evaluated by committees established by TRB. Technical guidance for projects is provided by expert panels with members that possess relevant technical expertise and that represent the user community for the specific IDEA product under development.

Additional information can be obtained on the IDEA website at: www.trb.org/idea

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- Ashland Specialty Chemical
- Owens-Corning
- Hardwire, LLC
- Coastal Precast Systems

The most rewarding conclusion of this project is that a mechanism exists within the transportation industry that allows for the opportunity to conceive an innovative idea and bring the concept to fruition in a manner that perpetuates the advancement of knowledge in the engineering profession. That being said, I would like to thank the HSR-IDEA Program Committee and the NCHRP Highway IDEA Committee for taking a chance on a concept that might otherwise have been forgotten. Last, but not least, I would like to thank HSR-IDEA Project Manager, Chuck Taylor, for his invaluable guidance, patience and encouragement.

Abstract and Key Words

This report presents the results from manufacturing and testing of a prototype Hybrid-Composite Beam (HCB) bridge for railroad and highway applications. The hybrid-composite beam presented in this report is comprised of three main sub-components that are a shell, compression reinforcement and tension reinforcement. The compression reinforcement consists of SCC concrete which is pumped into a profiled conduit within the beam shell. The tension reinforcement consists of Hardwire® steel reinforcing fabrics which run along the bottom flanges of the beams.

Whereas FRP materials are generally too expensive and too flexible when arranged in a homogeneous form, the strength and stiffness of the HCB are provided by a more efficient use of materials that are well suited to purely axial tension or compression. The classical arch shape of the compression reinforcing dramatically reduces the shear carried by the FRP webs. Due to the low density of the FRP materials and the ability to place the compression reinforcing in-situ, what results is an economical structural member that can be used in the framing system of a bridge structure in the same manner as a steel or prestressed concrete beam, but that is much lighter and well suited to accelerated bridge construction and also provides for a potentially longer service life.

This report discusses the evolution of the HCB as it has lead up to the world's first railroad bridge utilizing advanced composite materials. The paper will also discuss test results and performance from the full scale test.

Key Words: Hybrid-Composite Beam, Composite Bridge, Railroad Bridge, Highway Bridge, Accelerated Bridge Construction, Lightweight Structures, Corrosion Resistant

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EXECUTIVE SUMMARY

This High-Speed Rail IDEA project investigated the product application of a Hybrid-Composite Beam (HCB) as a structural member for use in railroad and highway bridges. The HCB is comprised of three main sub-components that are a shell, compression reinforcement and tension reinforcement. The shell is comprised of a fiber reinforced plastic (FRP) box beam. The compression reinforcement consists of portland cement concrete which is pumped into a profiled conduit within the beam shell. The tension reinforcement consists of steel fibers anchored at the ends of the compression reinforcement. The orientations of these sub-components are graphically displayed in Figure ES1. This report addresses steps that were involved in the design, fabrication and load testing of the first prototype Hybrid-Composite Beam Bridge.

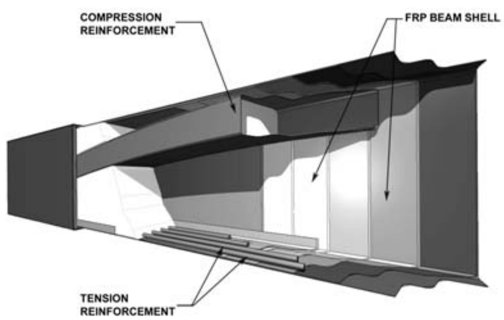

FIGURE ES1. Fragmentary Perspective of Hybrid-Composite Beam

The project findings documented that the HCB can be manufactured with minimum tooling costs and that the girders can be predictably designed to comply with the recommended practices for bridge strength and serviceability of the American Railway Engineering and Maintenance of Way Association (AREMA). Cost metrics indicate that the HCB does appear to offer a cost-effective alternate to concrete or steel beams in a bridge. What distinguishes the HCB from beams of conventional materials is that the FRP materials offer greater corrosion resistance and potentially longer life. Further, because of their reduced weight, shipping and erection costs for the HCB offer a distinct advantage. With these and other inherent benefits, the HCB offers an attractive alternative to consider in the construction of new railroad bridges as well as the reconstruction of the existing bridge inventory.

The first step in the investigation was to investigate cost-effective manufacturing processes and to fabricate and test one prototype HCB representative of the beams to be used in the product application. After extensive experimentation, tooling and processes were developed to ensure that the HCB could be manufactured in a repeatable process at a very reasonable cost. This prototype was then subjected to 2,000,000 cycles of fatigue loading and tested to failure, validating that the structural response was compliant with the AREMA recommended practices.

Subsequent to a successful prototype test, the next Stage of the investigation involved manufacturing eight (8) identical beams to be used together as the product application bridge. Many refinements to the manufacturing process were made along the way to reduce the time required for beam fabrication and to continually improve quality control of the final product.

Stage 3 of the project was devoted to addressing concerns related to the performance of the materials in the beam and to address more specific performance requirements raised by panel members and members of the railroad industry. These issues included degradation from ultraviolet (UV) radiation, fire ratings and lateral impacts to the bridge. This Stage also involved a discussion of non-destructive evaluation (NDE) techniques that can be used to monitor the service performance of an HCB Bridge over time.

The final stage in the investigation involved the construction and testing of the prototype HCB Bridge on the Facility for Accelerated System Testing (FAST) at the Transportation Technology Center, Inc., (TTCI) in Pueblo, Colorado. In this Stage, the 30-foot prototype bridge was erected on a live track (?) and subjected to the design live loads from a heavy axle freight train comprised of two diesel locomotives and a consist of twenty-six fully loaded coal cars. Live load tests included static positioning of the axle loads as well as dynamic testing at speeds ranging from 5 mph to 45 mph. Stresses and deflection in the prototype HCB Bridge were monitored and the bridge was found to perform as predicted. The success of this product application has helped garner the support of the Class 1 Railroad community to pursue further testing and revenue service deployment in the not too distant future.

1.0 BACKGROUND AND OBJECTIVES

1.1 TYPE 2 IDEA PROJECT – OBJECTIVES

The ultimate objective of this Product Application (Type 2) High-Speed Rail IDEA Project (HSR-43) investigation was to perform a product demonstration that included fabrication, erection and monitoring of a full size 30-foot prototype railroad bridge constructed using Hybrid-Composite Beam (HCB) technology. In the initial proposal for this project, the first preference was to load test the prototype bridge at the Transportation Technology Center, Inc. (TTCI) in Pueblo, CO. Due to uncertainties in the funding and concerns regarding performance, the final location of the in-situ testing of a complete HCB bridge was not concluded until late summer of 2007. The final decision to test the bridge on the Heavy Axle Load (HAL) FAST loop at TTCI was finally cemented when a consortium of Class 1 Railroads stepped up to provide the funds for the installation and train operations for the testing during the summer (?) of 2007. This consortium included BNSF, Canadian National, Canadian Pacific, Norfolk Southern and Union Pacific. With the funding finally in place, the final objective of this investigation could be realized.

1.2 RECAP OF STAGES

(John: This will be the only published report on HSR-43.) Before the HSR-43 investigation was undertaken a successful Concept Exploration (Type 1) Investigation (HSR-23: "Investigation of a Hybrid-Composite Beam Syste) was conducted that resulted in initial quantification of cost metrics, development of limit states equations for analysis and proof of concept through load testing of the first prototype Hybrid-Composite Beam fabricated at the University of Delaware. (1) The results of the four stages of the HSR-43 investigation are as follows:

Stage 1: Stage 1 was dedicated to the fabrication of one 30-foot prototype girder representative of the beams to be used in the final product demonstration. Before getting to the fabrication of the 30-foot prototype, extensive manufacturing experiments were conducted on smaller scale beams of 8-foot length to determine tooling, lay-up and infusion techniques that could be scaled up for fabrication of larger beams. Fabrication and testing of the first 30-foot beam was conducted. Due to a glitch in saving the load test data from the first test, a second 30-foot beam was also fabricated and tested in the laboratory in September of 2005. Both successful laboratory tests yielded similar results and demonstrated that the prototype beams met all performance requirements in accordance with AREMA recommended practices.

Stage 2: Following completion of Stage 1, the next stage of the investigation was dedicated to the fabrication of eight (8) 30-foot beams to be used in the prototype bridge installation as part of Stage 4. The Stage 2 efforts proved more difficult than anticipated as the manufacturing techniques continued to evolve over the course of the project until an infusion process was arrived at that was simplistic and consistent in terms of the quality of the beams. In addition to the fabrication studies, standards for design, and shop and erection drawings related to the construction of HCB bridges were developed and documented.

Stage 3: Stage 3 was interjected into the project to address other technical aspects of this composite technology as they relate to questions that have come up from representatives of the bridge industry. This stage of the investigation addressed many of the aspects related to the preparation of the beams for deployment in the prototype installation as well as issues related to maintenance, durability, fire resistance and other constructability and performance-related issues. Guidelines for material specifications as well as references for documents related to long-term monitoring of FRP transportation structures were also developed.

Stage 4: The last and final stage of the HSR-43 investigation culminated with field testing (?)of the first HCB Bridge at the Transportation Technology Center on November 7, 2007. The following sections of this report document the results recorded during this final objective. A more detailed discussion of the actual prototype deployment is contained in Chapter 4.0 of this report.

2.0 IDEA PRODUCT

2.1 THE HYBRID-COMPOSITE BEAM (HCB)

The product resulting from this IDEA investigation is a composite beam system that provides lighter weight for transportation and erection with enhanced corrosion resistance. The Hybrid-Composite Beam (HCB) which is protected by U.S. Patent No. 6,145,270 consists of three main sub-components. The first of these is the fiber reinforced plastic (FRP) beam shell, which encapsulates the other two sub-components. The second major sub-component is the compression reinforcement which consists of portland cement grout or concrete which is pumped or pressure injected into a continuous conduit fabricated into the beam shell. The third and final major sub-component of the beam is the tension reinforcement, which is used to equilibrate the internal forces in the compression reinforcing. This tension reinforcing could consist of unidirectional carbon or glass fibers or it could consist of steel fibers, e.g., standard mild reinforcing steel or prestressing strand infused in the same matrix during fabrication of the glass beam shell.

The beam shell is constructed of a vinyl ester resin reinforced by glass fibers optimally oriented to resist the anticipated forces in the beam. The beam shell includes a top flange, bottom flange and a continuous conduit. The conduit is fabricated into the shell and runs longitudinally and continuously between the ends of the beam along a profile that is designed to conform to the internal load path resisting the external forces applied to the beam. The beam shell also includes two vertical webs, which serve to transfer the applied loads to the composite beam as well as to transfer the shear forces between the compression reinforcement and tension reinforcement. All of the components of the beam shell are fabricated monolithically using a vacuum assisted resin transfer method (VARTM).

The compression reinforcement, which consists of portland cement concrete, is pumped into the conduit within the beam shell through the injection port located at the ends of the beams. The profile of the compression reinforcement follows a parabolic profile which starts at the bottom corners of the beam ends and reaches an apex at the center of the beam such that the conduit is tangent to the top flange. The profile of the compression reinforcement is designed to resist the compression and shear forces resulting from vertical loads applied to the beam in much the same manner as an arch structure.

The thrust resisted by the compression reinforcement resulting from externally applied loads is equilibrated by the tension reinforcement. The tension reinforcement consists of layers of unidirectional steel reinforcing fibers with a high tensile strength and high elastic modulus. The fibers, which are located just above the glass reinforcing of the bottom flange, are oriented along the longitudinal axis of the hybrid-composite beam. The tension reinforcement is fabricated monolithically into the composite beam at the same time the beam shell is constructed.

A bridge can be built quickly and easily using the HCB. The beams are erected prior to injection of the compression reinforcement by placing them with a crane in the same manner as a steel or prestressed concrete beam. The composite beams are easily self-supporting prior to and during the installation of the compression reinforcement. In the case of bridge replacement or rehabilitation it may be possible to reuse existing abutments and/or intermediate piers. The compression reinforcement is then introduced into the composite beam by injecting portland cement concrete into the profiled conduit in the glass beam shell. No temporary falsework is required for the erection of the composite beams or during the injection of the compression reinforcing. For some applications, such as railroad bridges, it may be desirable to inject the compression reinforcing prior to shipping the beams to the project site. This was the case in the prototype bridge test as it was more desirable to begin testing of the bridge immediately after erection, rather than exploiting the lighter erection weight that would have resulted from placing the compression reinforcing and composite concrete deck after erection.

For most typical applications, the weight of the HCB during transportation and erection is approximately one fifth of the weight of the conventional steel beam required for the same span and approximately one tenth of the weight required for a precast prestressed concrete beam for the same span. This light weight coupled with the corrosion resistant nature of the FRP materials make this technology well suited to "Accelerated Bridge Construction" and for providing bridges with service lives that could exceed one hundred years.

2.2 POTENTIAL IMPACT

Based on the current rate of spending for bridge replacement and rehabilitation, the number of deficient bridges is only gradually being reduced. Given that the structural framing members comprise a significant portion of the overall cost of a bridge; this provides all the more reason to develop a product that can increase the service life of the infrastructure. The HCB investigated provides a new alternative for consideration in replacement of our deteriorating bridges. Some of the inherent benefits of the beam are as follows:

- Faster erection, resulting in less out of service time for highways and railways.
- Lightweight, allowing reuse of existing substructures and potentially reducing span replacement costs.
- Design governed by deflection criteria results in reserve capacity for strength. This can benefit short-line railroads as well as other bridges that require upgrading for additional capacity or heavier axle loads.
- Resilient materials can improve seismic performance and fatigue resistance.
- Corrosion resistant materials result in better life cycle costs.

Because the beam presented uses basically the same erection methods as steel or precast concrete beams, contractors are subjected to virtually no learning curve despite the use of advanced composite materials. The beam can accommodate any alignment, skew or curve as a conventional beam but with the added feature of greater corrosion resistance. It is also well suited for bridges requiring future widening. Due to simple tooling and readily available raw materials, these beams are also well suited for emergency replacement of girders or entire spans on existing structures. Further, the embodiment of this beam lends itself well to implementation in current design codes such as AASHTO or AREMA, in limit states that can easily be followed by structural engineers with little or no background in FRP.

It is also worth noting that based on the information obtained during this investigation, current indications are that the Class 1 Railroads are constructing and replacing approximately 70,000 track feet of bridge per year. Average costs for railroad bridges based on cost estimates prepared under this investigation as well as input from representatives of the Class 1 railroads would be on the order of $4,000/track foot. Based on the rate of replacement and these average rates, this translates to a $280,000,000 a year industry. It is also worth noting that this amounts to less than 1% of the entire Class 1 bridge inventory and would further imply that the rate of replacement would have to be increased even to satisfy a 100 year design life for these structures.

The highway market is potentially an order of magnitude greater than for the railroad market. Ascertaining the potential of the highway bridge market is in some ways easier than with the railroads. This is due to a comprehensive database of information maintained by the Federal Highway Administration (FHWA) for the National Highway System (NHS). This database is commonly referred to as the National Bridge Inventory (NBI). Extensive information is available through this database including the number of bridges in each state classified by material type, span lengths as well as the number of deficient bridges eligible for replacement. The statistics available are too numerous to include herein, however a few important facts are worth noting.

- According to current information published by the National Bridge Inventory Database "Bridge deterioration rate studies suggest that concrete bridges (with a typical design life of 50 years) deteriorate slowly over the early part of their lives, followed by medium to rapid decline during the last two decades. Of the bridges on file for the year 2000, nearly 80,000 concrete structures (excluding culverts) were built between 1952 and 1977. There are more bridges of this material type and age than any other category, and include the bridges built during the "Interstate Boom" of the 1960's." The NBI concludes that the estimated bridge improvement cost for these structures is $11.2 billion.
- In general "Of the bridges in the Year 2000 NBI Database, 134,000 are recommended for replacement due to substandard load carrying capacity or substandard bridge roadway geometry. These substandard bridges constitute 1 of every 5 structures on public roads." The NBI concludes that "The estimated cost of these bridges is $66.5 billion".[1] (John: Need to change numbering of references and delete the footnotes at the bottom of this page.)
- A memo was issued by the Director of Bridge Technology for the FHWA in July of 1999 citing the specific goal of "(1) improving the condition of NHS bridges so that by 2006 less than 20 percent are classified as deficient, and (2) improving the condition of all bridges so that by 2006 less than 25 percent are classified as deficient."
- For fiscal years 2003-2009, the FHWA has apportioned an average of $3.72 billion annually for Highway Bridge Replacement and Rehabilitation Projects (HBRRP). This does not include other federal programs under the $33.5 billion Annual Surface Transportation Program (STP) that also help fund bridge projects nationwide. [2]

These are but a few documented facts that substantiate the future market for bridge construction and rehabilitation. These statistics also lend credence to the objective of providing new types of construction materials that can potentially

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[1] www.nationalbridgeinventory.com , NBI Report 2003
[2] www.fhwa.dot.gov/bridge/transfer.htm, HBRRP Fund Transfer by State and Fiscal Year

offer extended service lives compared to the conventional materials of the past. It should also be noted that no less than 80% of all of the bridges in the NBI database have span lengths less than 50 feet, which is ideally suited to economical span lengths for bridges using HCB technology.

3.0 CONCEPT AND INNOVATION

3.1 CURRENT TRENDS IN FRP INFRASTRUCTURE APPLICATIONS

Considerable work has been done in recent years with respect to FRP in the transportation infrastructure. In order to emphasize the unique characteristics of the HCB, it is important to point out that most of the research in advanced composite materials for transportation applications has been limited to structural shapes comprised of homogeneous FRP materials. Other research includes the application of FRP materials to conventional structural members to enhance strength. The following is a list of the various FRP research categories with respect to the bridge industry:

* Bridge framing systems using glass FRP pultruded beams or trusses manufactured from pultruded shapes.
* Glass FRP cable stayed pedestrian bridges fabricated from pultruded shapes with FRP cable stays.
* FRP reinforcing and post-tensioning strand for reinforcing and prestressing conventional concrete beams.
* Bonding FRP sheets to existing concrete and steel structures as a means of repairing, strengthening and upgrading these structures.
* FRP column wraps to provide confinement for enhanced seismic performance of concrete columns.
* Concrete filled, circular FRP tubes as an alternate to reinforced concrete columns.
* FRP bridge decks manufactured as pultruded sections, or VARTM sandwich panels.
* Hybrid pultruded beams using carbon reinforced flanges and glass reinforced webs.

In most cases, where bridges are constructed of structural members fabricated entirely of FRP, these bridges are subject to several constraints that have precluded widespread acceptance of the technology. First and foremost has been cost. The increased costs can be directly traced to raw material costs and constitutive material properties. In general, both glass and carbon have strength capacity that meets or greatly exceeds that of steel. However, they are much more flexible materials and require additional material to satisfy the serviceability requirements for deflections. The amount of material required and the higher material costs make it virtually impossible for a purely FRP beam to be cost competitive with concrete or steel beams at the present time.

Other limitations in application and span length in FRP beams are a result of lower shear strength capacity and low elastic buckling capacity in compression. Again, combined with the flexible nature of these materials, applications of purely FRP bridges have generally been confined to pedestrian bridges and short span county bridges. In any case, these bridges are still more expensive than those using conventional materials. Although the increased service life can improve the life cycle costs, most clients are still partial to construction materials that offer lower first costs.

3.2 UNIQUENESS FOR APPLICATION

What distinguishes the HCB is that it uses conventional materials, i.e. concrete and steel, in conjunction with FRP components to create a structural member that exploits the inherent benefits of each material in such a manner as to optimize the overall performance of the beam. Whereas FRP materials are generally too expensive and too flexible when arranged in a homogeneous form, the strength and stiffness are provided by a more efficient use of materials that are well suited to purely axial tension or compression. The classical arch shape of the compression reinforcing also helps reduce the shear carried by the FRP webs and ensures an optimal use of the compression and tension reinforcing.

Despite the unique configuration and diverse nature of the building materials used in the HCB, the end product does not result in any unique characteristics that would preclude the beam from being used in the same manner as more conventional framing systems. The cross-section is very conducive to standardization, similar to prestressed beams or rolled steel sections. Yet at the same time the tooling required for the VARTM manufacturing process is simplistic and inexpensive and allows for considerable variation with respect to overall dimensions, shape and internal lay-up. The embodiment of the HCB results in a beam that has several inherently unique benefits, while remaining cost competitive with conventional materials.

4.0 INVESTIGATION

4.1 30-FOOT PROTOTYPE BRIDGE CONSTRUCTION

The last and final stage of the HSR-43 investigation culminated with the prototype load test of the first HCB Bridge on November 7, 2007. This Chapter of the final report documents the results recorded during this final objective.

4.1.1 Assembly of Beams

Although the fabrication of the eight beams for the prototype bridge was completed in 2006, there was still some ongoing work necessary to prepare the bridge elements for deployment. These efforts were conducted at Coastal Precasting Systems in Chesapeake, VA over the spring and summer of 2007. The following work was performed in Virginia prior to shipping and installation of the beams on the test bridge in Colorado:

- Drill top of shell and install shear connectors
- Drill sides of beams and install tie-rods
- Paint exterior of the beams for UV protection
- Cast concrete in arch conduits for compression reinforcement
- Cast overlay
- Cast ballast curbs

The first task in preparing the bridge for shipment was to assemble the eight girders into two groups of four beams each. The four beam units were then bolted together with tie-rods at the two ends and at the center of the span. The tie-rods consisted of ¾ inch diameter all-thread bars installed in a 7/8-inch inside diameter, pvc electrical conduit. The holes for the tie-rods were drilled into the laminate and poly-iso foam about six inches from the bottom of the beam. Once the tie-rods were aligned with the holes in the adjacent beam, the beams were snugged together simply using a 4-foot lever arm to slide them along the hardwood dunnage supporting all of the beams. Figure 4.1 shows the girders being assembled into one of the two groups of four beams each. Although tying the beams together was a simple process, modifications in the fabrication process will be discussed later that mitigate the need to tie the beams together.

FIGURE 4.1 Tying Together HCB Units

FIGURE 4.2 Drilling Holes for Shear Connectors

4.1.2 Casting of Arch-Compression Reinforcement

The next stage of assembling the HCB Bridge involved casting of the concrete into the arch ribs. Prior to casting the concrete arches, shear connectors were inserted into holes predrilled in the top flange of the HCB that extended through the foam core on a 45 degree angle. The shear connectors were comprised of a ½-inch diameter coil rod with a hex nut screwed onto the end embedded in the concrete arch and an 8-inch 90-degree bend that anchors in the composite slab on tops of the beams. Prior to installation, three of the shear connectors on one of the beams were fitted with strain gages glued on at mid-length of the connectors to measure the forces during shear transfer. Prior to casting the arch for this same beam, a #3 size reinforcing bar was also installed in the arch cavity with five strain gages located at both spring lines, both quarter points and at mid-span.

FIGURE 4.3 Four HCB Assembly without Concrete Reinforcing

FIGURE 4.4 HCB Assembly with Compression

The concrete arch for the first beam was cast on an extremely hot and sunny day, with temperatures approaching 100-degrees Fahrenheit. An 8-inch slump, 5000 psi precast concrete mix design was used. Placement began by pouring the concrete into a whole cut in the top flange of the HCB and attempting to vibrate it down to the ends of the beam. Due to the high temperatures the concrete rapidly became very stiff and the remainder of the arch concrete (concrete arch?)was completed by filling at the ends and vibrating it up the arch. Despite the high temperatures, concrete placement on this beam was completed.

For subsequent beams, a self-consolidating concrete (SCC) mix design was utilized. This concrete was placed by pouring into the chimneys at the ends of the beams and vibrating up the arches until nearly filled and then filling in the remaining concrete at mid-span on the beams. The SCC flowed significantly better than the other high-slump concrete, but it was evident that for future projects, the SCC needs to be pumped into the beams from injection ports installed on the top ends of the beams to ensure timely placement of the concrete. It should be noted that the shear connectors also served as a good method of vibrating the concrete in the arch ribs during placement.

4.1.3 Casting of Composite Deck and Ballast Curbs

Once the concrete was placed in all of the beams, forms were placed around the perimeter to cast the composite deck on the two, four-beam assemblies. A four-inch composite slab was cast using a 6,000 psi bridge deck concrete. The deck was reinforced with a 6x6 W10xW10 welded wire fabric. Lessons learned in casting of the deck included developing methods of integrating an FRP form into the fabrication of the tops of the beams to avoid having to expend the labor and materials necessary to form the deck.

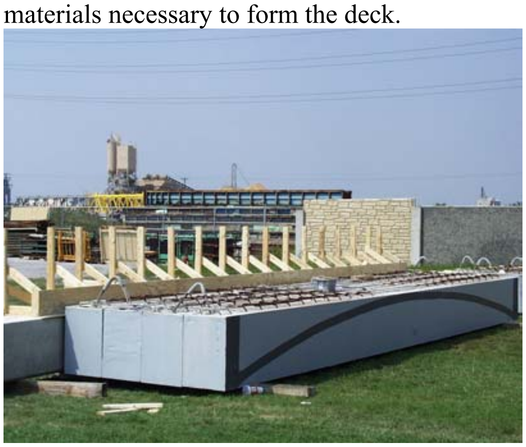

FIGURE 4.5 Forming Ballast Curbs and Deck

Following casting of the deck, it was also necessary to form and cast ballast curbs for the four-beam units. A standard Union Pacific Railroad (UP) ballast curb was used for this test. A cross-section showing the beams, deck and ballast curbs can be seen in Fig 4.6. Similar to the deck, a significant amount of labor and materials were required to construct these ballast curbs. In a production environment, the cost for the ballast curbs could be reduced by using standard forms. There is also some potential synergy that can be realized by using a precast concrete supplier to cast the concrete arches, deck and ballast curbs subsequent to the beam fabrication and prior to installation. A better solution yet is to integrate an FRP ballast curb into the fabrication of the HCB assembly itself. This would not only eliminate a significant number of steps in the bridge assembly, but it would also result in a significantly lighter weight ballast curb with the same corrosion resistant characteristics as the HCB.

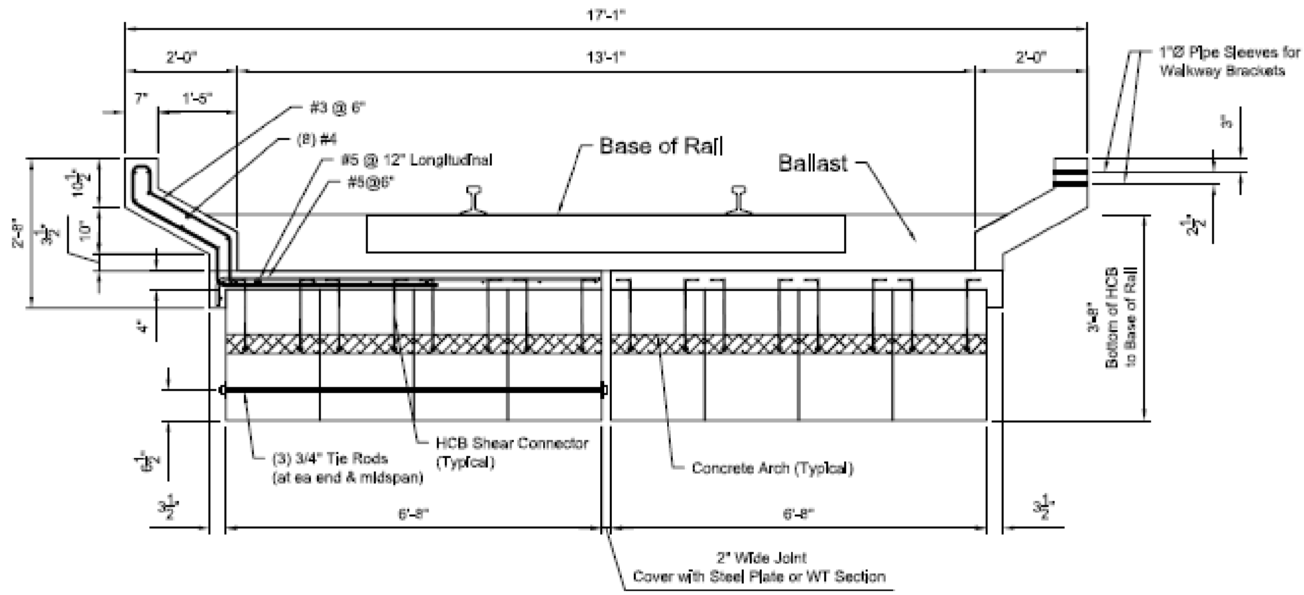

FIGURE 4.6 Cross-Section of Prototype Bridge as Tested

4.1.4 Shipping and Erection

Shipping and erection of the HCB's will vary depending on the application. For example, in a highway bridge it will likely be desirable to cast the arch concrete (concrete arch?)and concrete deck in place. Construction time is generally less critical for highway bridges, resulting in ample time to cast concrete on the site. As a result, the shipping and erection weight of the HCB beam shells will be very light. This results in a significant cost savings as multiple beams can be shipped on a single truck and the beams can be erected with smaller cranes than would be necessary for a purely precast concrete bridge.

For a railroad bridge however, it will generally be more desirable to erect the bridge in place with concrete arches, deck and ballast curb already cast. Although this scenario does not provide for the maximum benefit of the lightweight nature of the HCB, it does help expedite erection. This was the methodology used in the construction of the test bridge at TTCI in Pueblo. Subsequent to casting the arches, deck and ballast curbs, the two completed bridge units were loaded on a flat car in Chesapeake, VA and shipped via rail to the TTCI facilities. Once arriving at TTCI, the units were lifted off of the flat car with a 50-ton rubber-tired crane, trucked to the bridge location and set in place on neoprene bearing pads placed the full width of the girder sections. Each of the two units weighed approximately 23.5 kips (12 tons), including the 4-inch deck and ballast curbs. Figures 4.7 through 4.10 depict the final erection sequences that took place to get the bridge in position.

Overall the HCB erection went very well and both units were set in a few hours. One issue that had to be addressed was that a few of the individual beams had a concave surface on the bottom of the beam at the bearings that resulted in some difficulties in providing a uniform bearing surface. As a result, steel shims were placed between the bearing seats and the neoprene pads at some of the beams to try and provide for more uniform bearing. These anomalies in a select few of the beams did impact the uniformity of the structural behavior and this will be addressed in a subsequent section of this report.

FIGURE 4.7 Unloading HCB Unit from Flatcar

FIGURE 4.8 Setting HCB Unit on Bridge Substructure

As for the cause of the concave bearing surfaces, upon further investigation it became evident that this was an artifact of difficulties experienced in refining the manufacturing process. During a couple of the beam infusions, a loss of vacuum pressure became an issue. As a result, pooling of resin occurred at the ends of the arches, directly at the locations of the bearings. Since curing of the vinyl ester resins is an exothermic reaction, resin concentrations can result in extremely high temperatures, possibly as high as 300 to 400 degrees Fahrenheit. Given the amount of resin evident in the bottom of the mold after infusion it is entirely likely that the high temperatures during curing caused expansion of the steel mold. Because the tooling surface of the mold is restrained, the steel buckled up and caused a permanent concave surface at the ends of the beams. In retrospect, this condition was only evident on a few of the beams that experienced difficulties during infusion. Modifications to the manufacturing process now make it possible to inspect these portions of the beam during infusion, mitigating the likelihood of this from occurring on future beams.

FIGURE 4.9 End View of HCB Unit During Erection

FIGURE 4.10 Complete HCB Span

Once the beams were in position on the substructure, steel T-sections cut out of H-piles were set in the gap between the beams so that the ballast and track work could be placed on tops of the HCB units. The FAST bridge is on a horizontal alignment with a 5 degree curve resulting in superelevation of the track with the outside rail being approximately 4-inches higher than the inside of the curve. Once the track was repositioned and spliced it became evident that the track was not centered, but rather shifted approximately 9.5 to 10 inches towards the inside of the curve. Although this would result in unequal distributions to the two HCB units, it was decided not to center the track and instead to leave it as is to simulate real world conditions that might arise in future installations. The impact to the wheel distribution loads resulting from the off-center track is discussed later. Similarly, the variation in inside and outside wheel loads as a result of centrifugal forces caused by the 5 degree horizontal curve are discussed in the section on the live-load test results.

4.2 INSTRUMENTATION LAYOUT

Appendix A contains two sketches depicting the instrumentation plan for the HCB Bridge test at TTCI. The following is a summary of the instrumentation followed by a brief explanation of each component (the numbers correspond to the numbers circled in the sketches):

1. Linear strain gages (1/4") bonded to the bottom flange of each girder at center span (8 each)
2. Linear strain gages (1/4") bonded to the top flange of select girders at center span (6 each)
3. Linear (embedded) strain gage placed in concrete arch rib at center span (6 each)
4. Linear (embedded) strain gages placed in bottom of concrete arch rib at spring line (2 each)
5. Linear (embedded) strain gages placed in bottom of concrete arch rib at quarter points (2 each)
6. LVDT's at ends of beams to measure relative displacement between overlay and beam (4 each)
7. Linear strain gages (2") bonded to top of overlay at center span (4 each)
8. Linear strain gages (2") bonded to top of overlay at quarter span (8 each)
9. String Pots under each beam at center span (8 each)
10. Linear strain gages (1/4") bonded to the shear connectors (3 each)

Gages 1, 2, 3 and 7 measure bending strains at center span and quantify maximum stresses in the top and bottom laminate as well as the tension reinforcing, compression arch and overlay. Gages types 1 were placed at each girder to try and quantify any differential distribution of forces in the assembly of girders. For gages type 2, only six girders were instrumented and for gages 3, 4 and 5 only one girder was instrumented. Strain gages type 7 were only positioned on the center and sides of the deck (approximately 10 inches from edges outside beams) to document any differential compression stresses in the overlay.

Gages 3, 4 and 5 were all placed in or on the concrete arch ribs with gage 3 being at center span and gages 4 being placed closer to the center line of the bearings (approximately 2-feet in from the ends of the beam) and gages 5 being placed half way between at the quarter points. Gages 7 and 8 are all bonded to the top of the overlay with gages 7 being at mid-span and gages 8 at quarter span. The gages are spaced transversely as noted above. The intent of gages 7 and 8 are to quantify the composite behavior of the overlay slab relative to the HCB's.

The string pots measured the deflections under each beam at mid-span. The linear variable displacement transducers (LVDT's) (6) placed at the ends of the beam were installed to measure any relative slip between the concrete deck and the HCB and validate whether or not there is full composite action between the deck and the beams.

Lastly, we attempted to apply strain gages 10 to the shear connectors at select points along the length of a girder. The gages were attached directly to the steel shear connectors. The intent of these gages was to validate the shear transfer and magnitude of forces in the connectors assuming full composite action between the overlay and the HCB girders under service loads.

In all, the instrumentation plan presented resulted in 40 channels of data. 350 ohm gages were used at all locations. It should be noted that due to environmental conditions between installing the gages and damage resulting during casting of the overlay and ballast curbs, all of the gages 3, 4, 5 and 10 were non-functioning during the test. Several of the gages 2 on the top flanges of the beams were also damaged and yielded no valuable data. Regardless, the remaining gages, string pots and LVDT's performed well and the results are discussed later.

4.2.1 Static Load Test

The static load tests were conducted using heavily-loaded coal cars. The overall car lengths were 53.0 feet with truck center spacing of 40.5 feet. Each truck is comprised of two axles spaced at 6.0 feet. The truck positions for the static tests were derived to position the axles at locations that cause the maximum shears and moments. To accurately compare the response of the bridge to the predicted loads, the three rail cars at the very back end of the consist were weighed on October 29, 2007 just prior to the tests. The cars and their corresponding axle weights are listed in the Table 4.1. The magnitudes of the resulting axle loads of 80 kips spaced at 72-inches, 78-inches and 72-inches simulate a design load equivalent to roughly Cooper E-60. The actual design load of the HCB Bridge was the equivalent Cooper E-80 load comprised of four 100 kip axle loads spaced the same as those used in the test.

TABLE 4.1 Static Load Car Weights

Car Number	Truck A (lbs)	Truck B (lbs)	Total Car Weight(lbs)
309	163,450	163,575	327,025
362	159,975	160,175	320,150
476	158,500	161,300	319,800

The first three static tests included only one truck on the bridge. Based on the orientation of the train, these load cases resulted in Truck A of car number 362 being positioned on the bridge. Load position 1 (See Page 25) results in a high shear load at one end of the bridge only. Load position 2 provided a symmetrical load for a large moment at mid-span. Load position 3 was selected to produce the maximum moment under a single truck load.

The next set of static tests provided additional axle loads. Load position 4 provided a high shear load with essentially 3 axles located on the span. Load position 5 placed two trucks on the bridge for four axle loads located to provide maximum shear at one side of the span. Load position 6 provided a symmetric positioning of two trucks (four axles) located for a large moment. Load position 7 shifted the four axles approximately 1.75 feet to create the maximum moment on the span. The initial tests for Load positions 4 through 6 were conducted with Truck B of car 362 adjacent to Truck A of car 476. The load tests for positions 5 through 6 were then repeated using Truck B of car 476 located adjacent to Truck A of car 309. A diagram showing the load positions and axle loads for the various static tests can be found in Appendix A.

4.2.2 Dynamic Load Test

With the static tests complete, dynamic tests were initiated to observe the behavior of the HCB Bridge under live train operations starting with the train operating at 2 mph. The train speed was then increased to 5 mph and then an additional 5 mph per every pass was added up until the train was going 45 mph. At each velocity the train was run in both directions. Up to 20 mph the train was stopped and reversed for the opposite direction. From 20 to 45 mph the train continued on the continuous loop with the 5 mph increase in velocity as it traversed the loop. Once 45 mph was achieved in one direction, the locomotives were repositioned for the same dynamic tests in the opposite direction.

On November 8, 2008, additional dynamic tests were run with two locomotives and a consist of 26 heavy axle cars. This train assembly was run around the FAST loop more than forty-six times. All in all, the bridge was subjected to approximately 0.25 Million Gross Tons (MGT) of live loading. The effects of train velocity and change in the structural behavior with extended loading will be discussed in the subsequent section.

4.2.3 Evaluation of Test Results

Throughout the two days of testing, data was recorded for nearly every pass of the train. This resulted in a vast amount of output to be considered for evaluation. After careful consideration, the data files for detailed evaluation were narrowed down to four runs defined as follows:

- Conc9_6 – (07-Nov-07) – 5mph Counter-Clockwise Run, Data Set Number 6
- Conc9_17 – (07-Nov-07) – 45mph Counter-Clockwise Run, Data Set Number 17
- Conc9_35 – (08-Nov-07) – 45 mph run, 4th Pass, Data Set Number 35
- Conc9_79 – (08-Nov-07) – 45 mph run, 46th Pass, Data Set Number 79

Using the train-in-motion tests proved easier data with which to isolate the maximum effects from the axle loads. The 5mph run was slow enough to essentially provide the same structural behavior as a static load case. Data sets 6 and 17 allow for isolating the combined effect of live load impact along with the shifting of the load resulting from centrifugal forces caused by the 5 degree horizontal curve on the bridge alignment. It should be noted that in general, there did not appear to be much evidence of change in the behavior of the bridge due to impact forces. However the graph in Figure 4.11 indicates the anticipated changes in the loads to the inside and outside rail due to the 5 degree curvature coupled with the four inch difference in the rail heights due to superelevation of the track.

Along with the superelevation, as mentioned previously, the track was installed with an eccentricity of between 9.5 to 10 inches towards the inside of the curve. This resulted in additional load being directed towards the four-beam unit on the inside of the curve. Although this same phenomenon reduced the total live load going into the four-beam unit on the outside of the curve, it also resulted in a situation where the effect of loads on the outside rail were now concentrated on the two interior beams of the outside unit. Conversely, on the inside unit the eccentricity of the rails made for a more uniform distribution of the live loads to the four-beam unit. The results of this track eccentricity are evident in the data when looking at the distribution of deflections across each four beam unit.

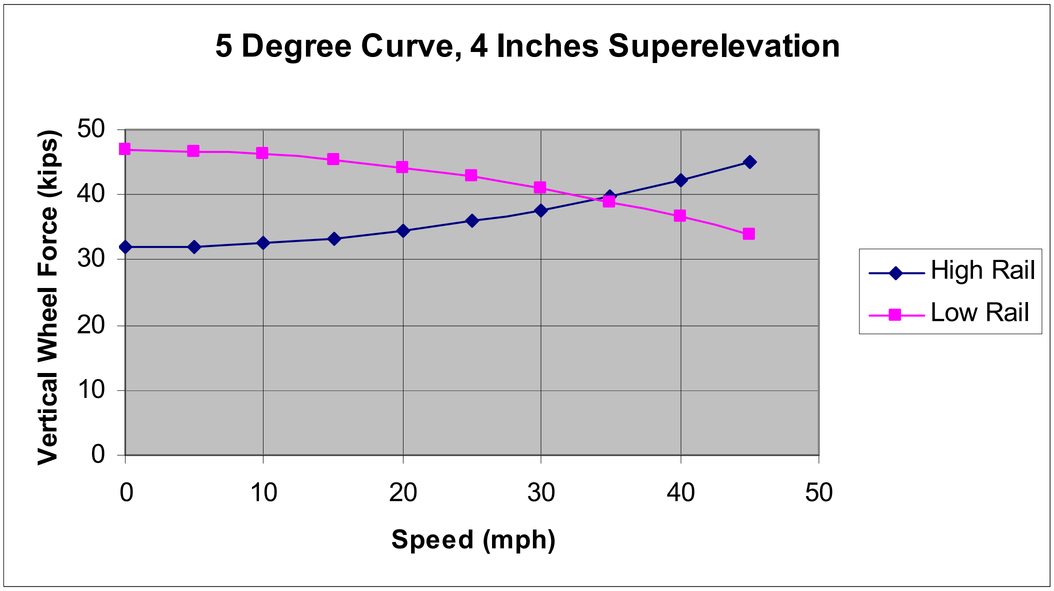

FIGURE 4.11 – Wheel Load Distribution for Horizontal Curvature in Track

As noted above, when evaluating the measured strains and displacements as compared to the predicted values, it was necessary to adjust the predicted values to account for the effects of track superelevation and eccentricities as well as train velocity. The modification factors for these effects are listed below. The measured values for both strains and deflections were taken by subtracting the maximum and minimum values for each run. For the deflection measurements, 1/16-inch was subtracted to account for the elastic deformations of the bearings. The 1/16-inch value was determined from measurements taken at the abutment and interior pier during the static tests.

- 10-inch Track Eccentricity – Low Side – 1.196 x Predicted Value
- 10-inch Track Eccentricity – High Side – 0.804 x Predicted Value
- 5mph Adjustment for Speed – Low Side – 1.185 x Predicted Value
- 5mph Adjustment for Speed – High Side – 0.815 x Predicted Value
- 45mph Adjustment for Speed – Low Side – 0.857 x Predicted Value
- 45mph Adjustment for Speed – High Side – 1.143 x Predicted Value

The data was synthesized to look at the individual readings at each of the eight beams as well as the average values for each four beam unit. The following tables show the distribution of deflections along the transverse cross-section as well as the average deflection compared to the predicted deflection. It should be noted that string pots D24 and D25 are located on the insides of the four beam units. It is evident from the data that the distribution is higher towards the interior girders and lesser towards the outside girders. The distributions across the widths of the four beam units are fairly linear. The variability is likely due to the rail being located closer to the inside girder. The smaller deflections on the outside girders might also be in part due to the additional stiffness of the ballast curbs, even though they have expansion joints. In all cases the predicted values as compared to the average measured values are within a few hundredths of an inch.

TABLE 4.2 Live Load Deflections

Deflection Measurements - Under Each Beam at Center Span - High Side						
Run File	D21 in	D22 in	D23 in	D24 in	Average in	Predicted in
Conc9_6	0.077	0.151	0.234	0.327	0.197	0.186
Conc9_17	0.153	0.238	0.305	0.389	0.271	0.261
Conc9_35	0.174	0.257	0.332	0.421	0.296	0.261
Conc9_79	0.165	0.255	0.341	0.413	0.294	0.261

TABLE 4.2 Live Load Deflection (Cont'd)

Deflection Measurements - Under Each Beam at Center Span - Low Side						
Run File	D25 in	D26 in	D27 in	D28 in	Average in	Predicted in
Conc9_6	0.342	0.340	0.290	0.273	0.311	0.403
Conc9_17	0.359	0.332	0.273	0.203	0.291	0.291
Conc9_35	0.385	0.346	0.273	0.202	0.302	0.291
Conc9_79	0.379	0.352	0.279	0.219	0.307	0.291

Similar behavior was evident in the strains measured on the gages attached to the bottom of the laminate at center span for each of the eight girders. Typically the strains at the interior girders were higher than the strains in the outside girders. Further, like the deflection behavior, the strains on the outside of the horizontal curve increase with speed whereas the strains on the inside of the curve decrease with speed. The only anomaly in the data appears in the strain gage on S21, which consistently reads higher than any of the other gages by nearly a factor of two. This is inconsistent with the deflection data which always reads lower than the interior girders. One plausible explanation for this is that gage S21 was located on the girder on the outside of the curve with a southern exposure. The increased strain on this gage may be in part due to thermal strains as the sun was out during both days of testing.

The highest laminate strain of 614 μE translates to a stress in the glass laminate of approximately 1.9 ksi. The ultimate tensile capacity of the glass laminate is in excess of 50 ksi. Similarly, the stress in the Hardwire® tension reinforcement at 614 μE is 7.32 ksi, which is well below the 360 ksi ultimate tensile strength of the steel.

TABLE 4.3 Live Load Strains in Bottom of HCB

Strain Measurements - Bottom of Beams at Center Span High Side						
Run File	S21 μE	S22 μE	S23 μE	S24 μE	Average μE	Predicted μE
Conc9_6	629	237	421	404	423	320
Conc9_17	936	332	520	491	570	448
Conc9_35	1017	359	547	527	612	448
Conc9_79	1026	354	548	529	614	448

Strain Measurements - Bottom of Beams at Center Span Low Side						
Run File	S25 μE	S26 μE	S27 μE	S28 μE	Average μE	Predicted μE
Conc9_6	507	341	422	309	394	692
Conc9_17	567	345	462	327	425	500
Conc9_35	592	362	450	318	430	500
Conc9_79	596	364	423	288	418	500

In addition to the strain gages on the bottom of the beams, eight strain gages were attached to the top of the four-inch composite concrete deck overlay. Unlike the gages and string pots on the bottom of the span, the deck gages were not placed in line with each girder. Instead some of the gages were placed at center span and others were placed at the quarter points of the span. Although the data from the concrete gages did not correlate as closely as the deflections or the gages on the bottoms of the beams, the measured strains were still reasonably close to the predicted values. The highest measured values are still within the same range as the predicted values and correlate to stress levels in the concrete of approximately 900 psi as compared to the ultimate strength of 6,000 psi.

TABLE 4.4 Live Load Strains in Top of Concrete Overlay

Strain Readings - Top of Deck at Center Span						
Run File	Predicted μE	SD2 μE	SD3 μE	SD4 μE	SD7 μE	Predicted μE
Conc9_6	134	39	76	165	191	291
Conc9_17	188	50	90	231	191	210
Conc9_35	188	61	100	242	182	210
Conc9_79	188	74	84	240	181	210

Strain Readings - Top of Deck at Quarter Span						
Run File	Predicted μE	SD1 μE	SD5 μE	SD6 μE	SD8 μE	Predicted μE
Conc9_6	122	194	58	54	69	265
Conc9_17	172	248	109	58	71	192
Conc9_35	172	253	106	53	73	192
Conc9_79	172	251	139	58	67	192

Finally, the LVDT's placed at the four corners of the bridge read displacements that were on the order of a few thousandths of an inch. Between these low readings and the close correlation between the measured and predicted values of strains and deflections, it is reasonable to assume that full composite action was obtained between the concrete overlay and the HCB through the shear connectors tying the two elements together.

All in all the HCB prototype bridge performed as expected and validated the analytical computations. It should be noted that although the distribution of loads across the bridge resulted in a range of deflections across the width of the bridge, the average deflections were still below the allowable deflection value of span/640 which is equal to 0.54 inches for this span. Even after compensating for the axle loads less than Cooper E-80, an acceptable deflection would have been 0.41 inches which is consistent with the maximum deflections seen under the interior girders. For the most part the shifting in load from the inside bridge unit to the outside unit based on train velocity was consistent with calculations. In general there did not appear to be any major influence on the strains and deflections from impact caused by the train velocity. Finally, there were very minor increases in strains and deflections between the first and second day of testing, but it did not appear that there was any change in behavior between the 4th pass and the 46th pass on November 8, 2007.

4.2.4 Post Test Inspection

On November 9, 2007, following the successful test, the track work and ballast were removed from the bridge so that a post-test inspection could be made of the entire bridge. In general there was no visible change to the HCB's to indicate any degradation of the structural members. The only damage evident was some cracking of the composite concrete overlay on the east end of the outside unit as evident in the photograph in Figure 4.12. The cracking appeared to be shear cracking of the concrete slab at the ends of the inside girders of the outside unit. Cracking at this location would be more likely as the two interior girders of this unit were the more highly loaded girders as a result of the track eccentricity. It should also be noted that at the particular location where the cracks occurred, the wire fabric reinforcing the deck had been cut to fit around the lifting loops in the beams. As a result, it is entirely possible that there was essentially no shear reinforcing in the slab at the location of the cracking. It's difficult to predict at what time during the testing the cracks actually occurred or if they would have continued to worsen with time. As noted above, there was no change in the structural performance evident in the data on the second day of extended testing.

As a side note, there was cracking evident in the concrete overlay prior to testing. Most of these cracks resulted from shrinkage cracks caused by a delay in initiation of moist cure. All of the existing cracks were carefully mapped prior to placing of the ballast and track work so that they could be distinguished from any new cracks formed during the testing.

FIGURE 4.12 Shear Cracks in Overlay – Post Test

Regardless of when the shear cracks occurred a collective decision was made by the Consortium members of the Class 1 Railroads supporting the HCB test to remove the HCB Bridge from the FAST facility and reinstall a precast concrete bridge also undergoing extensive service life testing. The shear cracks in the overlay are most likely the result of insufficient reinforcing steel in the concrete deck to accommodate these stresses. It was also suggested by the Class 1 Consortium that it may be desirable to increase the thickness of the deck from four inches to at least five inches. Either way, the consortium members appeared to be in agreement that the main objective of validating the structural performance of the HCB was successfully demonstrated through the tests at TTCI.

5.0 PROJECT PANEL

The Project Panel for HSR-IDEA 43 was comprised of four members that include:

Duane Otter, P.E., Ph.D. – Dr. Otter is a Principal Engineer with the Association of American Railroads (AAR) and oversees all structural testing conducted at TTCI, in Pueblo, CO.

Ian Friedland, P.E, Ph.D. – Dr. Friedland is Technical Director of Bridge & Structure, Research & Development for the Federal Highway Administration (FHWA) in McLean, VA.

Mike Franke, P.E., - Mr. Franke is Assistant Vice President for Midwest Regional Rail Initiative for Amtrak in Chicago, IL. Formerly of BNSF, Mr. Franke has remained throughout his career a strong advocate for research and development in the railroad industry.

Brian Hornbeck, P.E., - Mr. Hornbeck currently serves as Associate Director of Bridging for TACOM – US Army TARDEC.

Throughout the course of this project it has been difficult to arrange for regular meetings of the advisory panel. Regardless, the panel members were copied on each of the previous three Stage Reports. Further an effort was made to contact each of the members on a fairly regular basis to solicit input regarding the development of the HCB.

Equally important to the Project Panel, two other oversight committees that provided significant input on the development of the HCB were the Bridge Technical Advisory Group (Bridge TAG) and the Engineering Research Committee (ERC), both of AAR. One or the other of these committees was addressed on an annual basis to update them as to the status of the HCB research and solicit input. In particular, the Bridge TAG issued a letter on November 18, 2005 identifying several points that should be addressed in development. This list included:

- Effects from lateral loads
- Inspection (in particular internal components of the hybrid beams)
- Transverse ties to keep beams together and develop integral behavior
- Overlay (composite deck)
- Fire resistance and failure mode in fire
- Ductility of the beams and brittle failure mode under load

Similar feedback was offered by Dr. Friedland of FHWA. An effort has been made to address many of the concerns noted above. (John: Can we include a summary of the issues and your responses?). Some of the issues still require further testing, including the performance of the beams subjected to lateral loads and the fire resistance as it relates to any specific hourly rating.

Throughout this project, one of the more pertinent questions consistently raised is in regards to the inspection and maintenance of the HCB. Although it is not possible to adequately address this concern within the confines of this report, this issue is discussed in NCHRP Report 564, *Field Inspection of In-Service FRP Bridge Decks.* Although the FRP decks and bridge elements addressed in this report are more consistent with homogenous FRP structures, many of the NDE techniques discussed and explanations of characteristics of FRP performance are applicable to the HCB.

Continued feedback from this panel and other agencies involved in future deployments will be solicited as development progresses.

6.0 FINDINGS AND CONCLUSIONS

6.1 GENERAL PERFORMANCE

This development of the HCB has been a long journey, not without its set backs and delays. Regardless, the project has concluded in the successful deployment of a prototype installation that validated the predictable structural behavior of the HCB for its intended purpose in a bridge structure. Throughout the course of the project, refinements were made to improve both the structural performance and constructability of this bridge system. A list of undesirable manufacturing techniques would be too numerous to mention, however the subsequent section will elaborate on some of the more successful and proposed improvements to the manufacturing process.

In regards to the structural performance of the HCB, the stresses, strains and deflections proved to be very predictable and within the code specified limits. A couple of improvements could be realized in evaluating the distribution of stresses and deflections across the cross-section of the bridge. Ideally, it would be nice to get a more even distribution within the cross-section, rather than seeing more load towards the interior girders and less on the outside. This may be accomplished through some of the proposed manufacturing improvements.

Another performance issue warranting consideration was the shear cracking of the concrete overlay at the ends of the bridge. This particular issue can easily be addressed by revising the reinforcing details to provide better distribution of the shear forces at the ends of the beams and will be accounted for in future deployments.

6.2 PROPOSED REFINEMENTS

Although the HCB has many inherent benefits over bridges manufactured from conventional materials, it is essential that this technology be cost competitive on a first cost basis with concrete and steel. This concern has not so much been expressed as implied. Although when considering life cycle costs, the fact that the FRP materials are resistant to corrosion will yield some additional economy over the life of the structure, it is common knowledge that bridge owners and bridge designers are seldom ever given the latitude to consider these benefits in making the initial selection of material type. This has been a driving consideration throughout the development of the HCB. As a result numerous modifications have been made to the tooling, materials and lay-up over the course of developing a viable manufacturing process. Even still, there will be inevitable improvements as economies of scale are introduced and others in the composite industry take an interest in developing this product for commercialization.

One modification under consideration is to fabricate multi-celled beams with multiple webs for railroad applications. Upon finalizing the fabrication process it became evident that there is no reason why a multi-celled beam could not be built that would significantly reduce the time required for fabrication and erection. For example, instead of fabricating four, 20-inch wide girders, the same materials could be used to fabricate a single girder, 80-inches wide. This would require a slight increase in the lay-up time; however the infusion time would be exactly the same by providing additional infusion ports for each cell of the beam. The entire process could still be performed in one day. A reduction in erection time should also be obvious in that the contractor is handling one girder instead of four and the need to tie the girders together with high-strength bars would be completely eliminated. In terms of fabrication and erection, this configuration would be even more analogous to the typical precast bridge using two double-celled boxes to comprise the cross-section, except that the girders would weigh only 10% of the weight of concrete for fabrication and shipping.

Additional improvements to the manufacturing process have only recently been introduced as fabrication of the first highway bridge begins. One of these modifications is that the beam is now fabricated in two pieces instead of a single shot. This is facilitated by fabricating everything except the top flange in an open bathtub mold that accommodates the bottom flange, two webs and beam ends. The top flange is fabricated separately with the upper section of foam forming the compression arch infused onto the top flange laminate. Once both parts are infused, they are joined together with high strength adhesives and bolted together prior to casting of the concrete arch. This process provides much better access for inspection during the infusion process.

Another modification to the HCB is experimentation with galvanized pre-stressing strand as the tension reinforcement in place of Hardwire®. The strand still provides for the deflection limit states, and provides more than adequate strength at a yield strength of 250 ksi, but at a fraction of the cost of previously tested materials.

Further streamlining of the manufacturing process will also evolve with automation of off-line lay-up of the various performs that are installed into the tooling. Acceleration of the curing process can be facilitated through heating of the mold subsequent to the infusion. It is even possible that methods other than VARTM could be adapted for fabrication of the beam shells. The final cycle time for fabrication is ultimately controlled by the amount of time consumed with elements of the beam in the mold before, during and after infusion. Similar size FRP elements fabricated in the composites industry indicate that it is not unlikely that several beams could actually be fabricated in the same mold during the course of an eight hour day.

6.3 STANDARDIZATION OF DESIGN, FABRICATION &INSTALLATION

One of the primary goals from the inception of the HCB was to strive for standardization of every aspect of the system including design, fabrication and installation. Part of this standardization involves conforming to details and processes that are consistent with those of bridges using conventional materials. This objective has also been validated through the success of this investigation. This is evident in the strain-compatibility and force equilibrium equations used to quantify the structural capacity of the HCB in the same manner as would be done for a concrete beam. It is also evident in the standardization of beam sizes and depths that are fabricated to the same span to depth aspect ratios as conventional concrete and steel beams. Finally it is evident in the fact that the prototype HCB was easily interchanged and erected in exactly the same manner as the prestressed concrete bridge that was temporarily displaced during the test.

As further development of the HCB progresses, this objective will likely serve as the most important factor in being able to seamlessly substitute this technology in place of conventional bridges. Conforming to the design and construction characteristics of conventional bridges makes it much easier for the transportation industry to accept the evolution of this new technology. It also helps that the primary load carrying materials, i.e. concrete and pre-stressing steel are those that the industry is already familiar with and comfortable with.

The final conclusion is that through the evolution of the HCB technology, the refinements of the manufacturing technology and the adaptation of the ideal materials, the HCB has evolved into a structural member that not only satisfies all of the strength and serviceability requirements of the codes, but also provides the benefits of lighter weight for shipping and erection and extended service life through corrosion resistant materials. The HCB can provide all of these benefits and at the same time do it at a first cost that is competitive with conventional concrete and steel beams.

FIGURE 6.1 Conclusion of HSR-IDEA

7.0 PLANS FOR IMPLEMENTATION

7.1 STATUS OF OTHER PENDING PILOT PROJECTS

Throughout the development and deployment of the first railroad bridge test, extensive efforts were put forth in trying to secure additional prototype applications. Currently there are two additional projects that have been pursued under the Federal Highway Administration's, Innovative Bridge Research and Deployment (IBRD) Program. Both of these bridges will go to construction in 2008.

One bridge is the New Jersey Route 23 over Peckman Brook in Essex County, NJ. This bridge will be approximately a 31-foot clear span with an overall width of 66'-5". The cross-section will be comprised of 1'-9" deep HCB deck beams placed side by side with a composite, variable depth concrete overlay. The construction of the bridge will be in two stages to maintain traffic.

The other bridge currently under fabrication is the High Road Bridge over Long Run Creek in Lockport Township, IL. This will also be a highway bridge with a clear span of 57'-0" having an overall width of 43'-2". In this bridge, the cross-section will be comprised of six (6), 42-inch deep HCB's placed at 7'-4" spacings with a conventional 8-inch reinforced concrete deck made composite with the beams.

Both of these highway bridges represent examples of how the HCB can be directly substituted for a conventional concrete or steel bridge system. Although these two examples represent highway bridge applications, the Class 1 Railroad Bridge community has also expressed a keen interest in the deployment of HCB railroad bridges. There has been enough interest demonstrated by the railroads that the Bridge Technical Advisory Group (TAG) of AAR requested and secured $140,000 specifically for further testing of the HCB on the TTCI research facility in Pueblo, CO.

As part of both the highway projects and the continued testing at TTCI, HC Bridge Maine, LLC has been established to continue research and development for commercialization of the HCB. Currently HC Bridge has licensed Harbor Technologies of Brunswick, ME to serve as the fabricator for the HCB. HC Bridge has also cultivated a relationship with the Advanced Engineering Wood Composite (AEWC) Center at the University of Maine. The consortium of HC Bridge Maine, Harbor Tech and AEWC center currently have a $450,000 Development Grant pending to further the commercialization of the HCB nationwide with emphasis on growing the composite manufacturing sector of the State of Maine.

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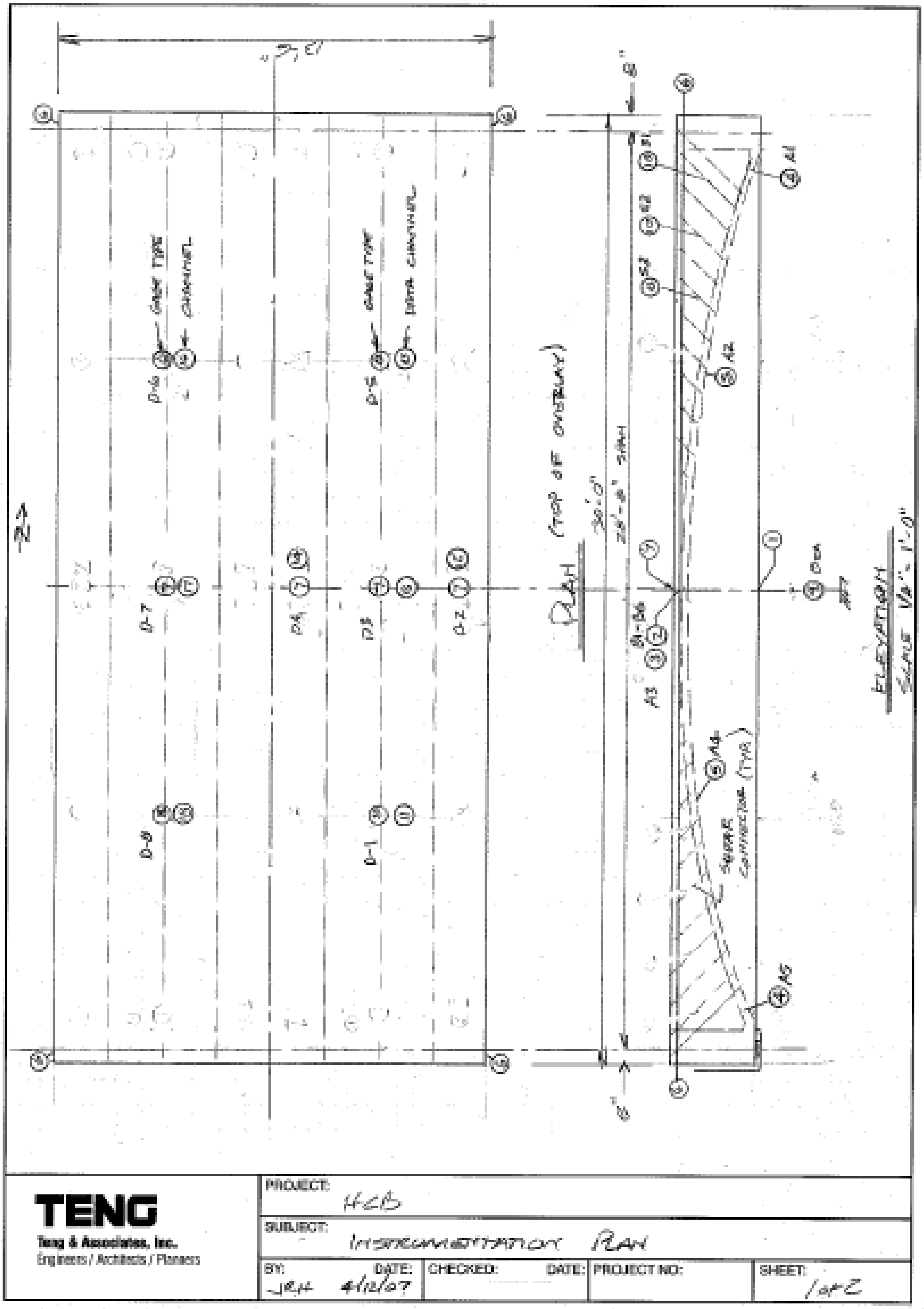
John: Need to add as reference number 1:

J.R. Hillman. *Investigation of a Hybrid-Composite Beam System.* High-Speed Rail IDEA Report HSR-23, Transportation Research Board, 2002

APPENDIX A – Test Plan from Prototype Implementation at FAST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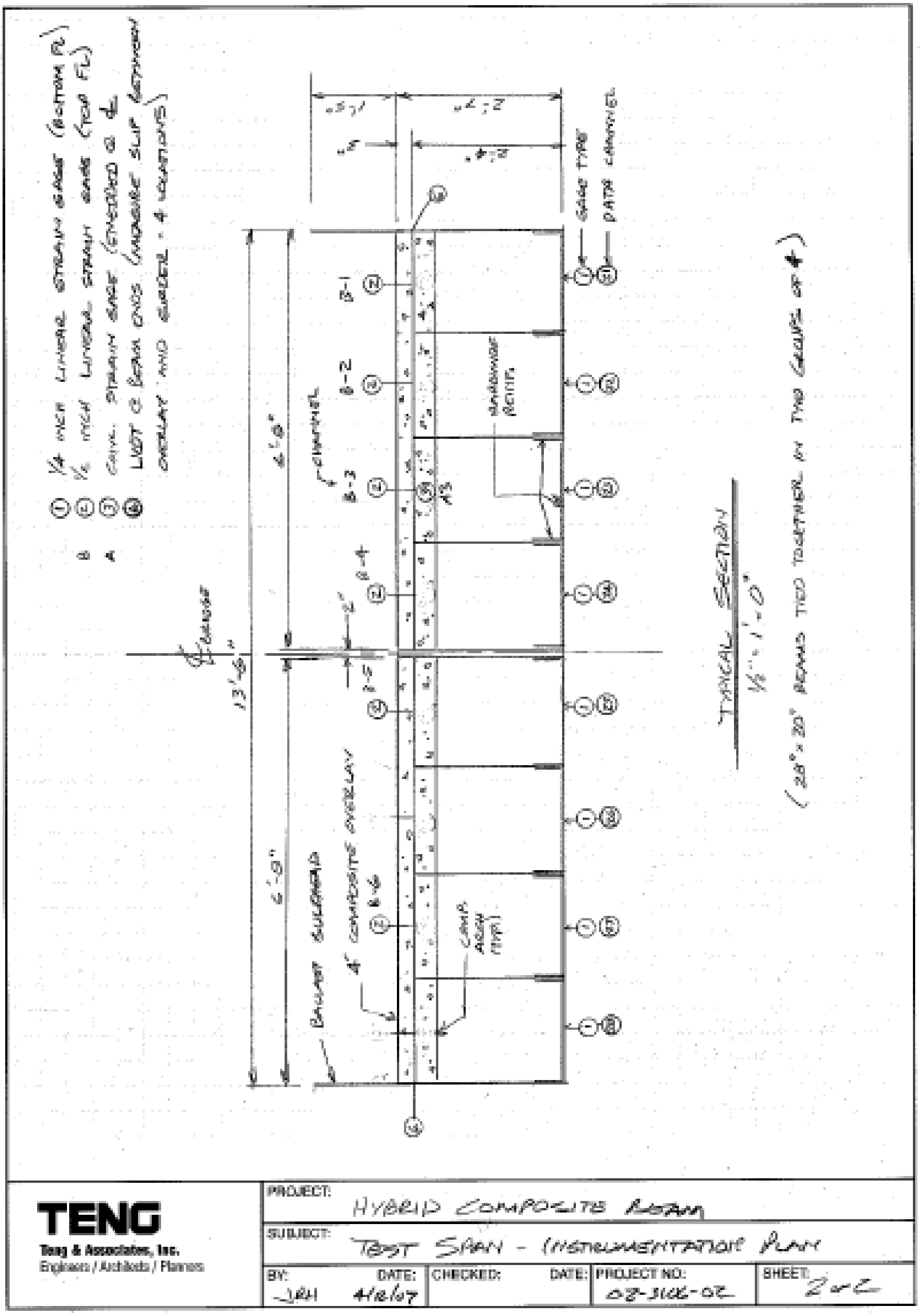
TENG
Teng & Associates, Inc.
Engineers / Architects / Planners
PROJECT: HYBRID COMPOSITE BEAM
SUBJECT: TEST SPAN - INSTRUMENTATION PLAN
BY: JRH
DATE: 4/12/07
CHECKED:
DATE:
PROJECT NO: 03-3106-02
SHEET: 2 of 2

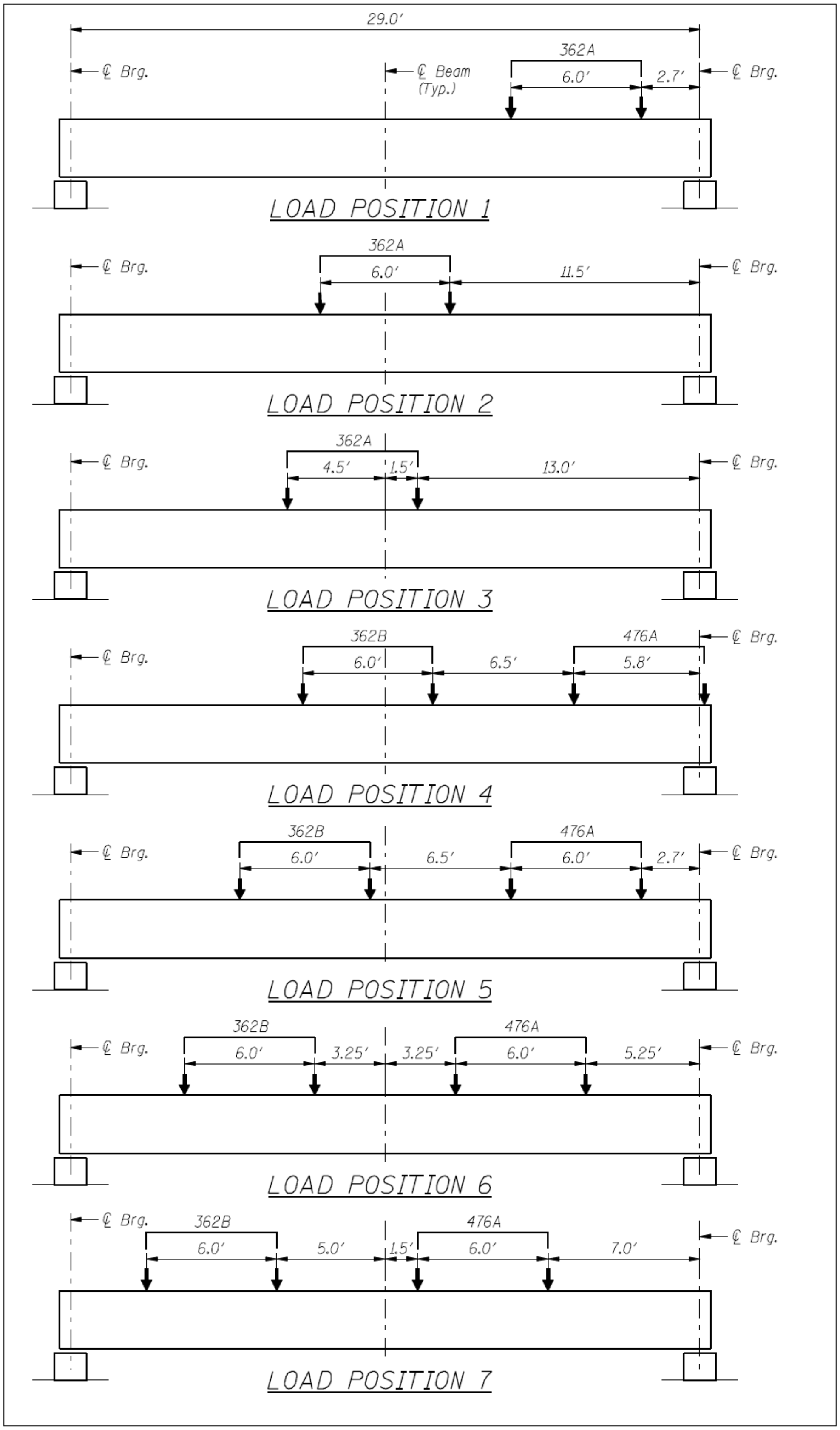

FIGURE A.3 Load Positions for Static Load Tests