Testing of a Prototype Hybrid-Composite Beam Span under Railroad Loadings

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ABSTRACT

This paper presents the results from testing of a prototype hybrid-composite beam span under railroad loadings at the Facility for Accelerated Service Testing (FAST). The hybrid-composite beam presented in this paper is comprised of three main sub-components that are a shell, compression reinforcement and tension reinforcement. The compression reinforcement consists of self-consolidating concrete (SCC) 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 fiber reinforced plastic (FRP) materials are generally too expensive and too flexible when arranged in a homogeneous form, the strength and stiffness of the Hybrid-Composite Beam (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 paper will discuss 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.
1. INTRODUCTION

Throughout the evolution of structural engineering, the most significant advances have been founded on simple concepts. For example, by placing blocks of stone sequentially along a circular curve, the first arch structures were created. Centuries later, iron and steel made it possible to span greater distances with much lighter structures. In 1879, Francois Hennebique developed concrete structures with steel tension reinforcement. In 1907, Eugene Freyssinet designed the first concrete bridges to employ prestressed steel reinforcement. Each of these advances in structural engineering sought to employ a more efficient use of building materials, generally through a combination of building materials. Further, these innovations have always been predicated on simple fundamental principals of structural Behavior and willingness by the designer to deviate from traditional form in an effort to push the envelope of structural efficiency.

These same principals led to the development of the Hybrid-Composite Beam (HCB) as a structural member for use in highway and railroad 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 1.0.
In order to provide a cursory account of the actual deployment of the first HCB railroad bridge, a brief summary of the initial stages of development have been discussed as follows:

Stage 1 of this investigation 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.

The HCB’s are manufactured using a closed-mold, Vacuum Assisted Resin Transfer Method, or VARTM process. In this process, the preforms that include; quad-weave glass fabrics, Hardwire® steel reinforcement and a low density (polyisocyanurate) foam core are all placed inside of a simple, inexpensive, reusable mold. Once the preforms are in place, the mold is sealed and vacuum pressure applied to the evacuate all of the air. While maintaining vacuum pressure at 1 atmosphere, a vinyl ester resin is pulled into the mold to wet-out the preforms and consolidate the glass, foam and resin into a monolithic unit. The entire process for lay-up and infusion and demolding of a beam can be performed in a one day cycle.

Following completion of Stage 1, eight (8) 30-foot beams were fabricated for use in the prototype bridge installation. Stage 3 investigated 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.

The final stage in the investigation involved the construction and testing of a prototype HCB Bridge on the Facility for Accelerated System Testing (FAST) at the Transportation Technology Center, Inc., (TTCI) in Pueblo, Colorado. The 30-foot prototype bridge was erected on a live track alignment 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 on November 7, 2007. Additional details regarding the live track test are discussed herein.

2. PROTOTYPE BRIDGE FABRICATION AND CONSTRUCTION

2.1 Assembly of Beams

Although the fabrication of the eight beams for the prototype bridge was completed in 2006, there was still some effort necessary to prepare the bridge elements for deployment on the test track. 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 center span. The tie-rods consisted of ¾ inch diameter all-thread bars installed in a 7/8-inch inside diameter, pvc electrical conduit. Figure 2.1 shows the girders being assembled into 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.
2.2 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.

A self-consolidating concrete (SCC) mix design was utilized as the compression reinforcement in the beams. 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. In more recent applications of the HCB, the SCC was simply pumped into the beam shells through attachments installed in the top flanges at the ends of the beams. This process can be completed in a matter of minutes.

2.3 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 a composite concrete 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.

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 2.3. 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. In the future the FRP ballast curb will be integrated 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.

Fig.2.3 Cross-Section of Prototype Bridge as Tested
3. 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 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 would be approximately 1/3 that of a steel beam or 1/10 the weight of a similar concrete beam. 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 railroad bridges however, it may 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. TTCI installed the 30-foot HCB span at the State-of-the-Art concrete test bridge located in Section 3 of HTL using a 50-ton crane. Each of the two units weighed approximately 23.5 kips (12 tons), including the 4-inch deck and ballast curbs. The HCB span pieces were supported at each end on elastomeric bearing pads of the type typically used to support precast concrete spans. The bearing pads were carefully placed during installation. Steel shims were added as necessary to minimize gaps along the ends of the spans. Figures 3.1 thru 3.4 depict the final erection sequence 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.
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 deck of the HCB span was 2 inches higher than the deck on the adjacent span. This height mismatch was expected ahead of time. The HCB beams themselves are 28 inches deep, with a 4 inch concrete overlay, making a total depth of 32 inches. The original concrete span at this same location had a total depth of 30 inches.

Track was constructed using granite ballast, UP standard concrete ties on 24-inch centers, and 136 RE continuous welded rail. The track is on a 5-degree curve with 4 inches of
superelevation. Depth of ballast was 10 inches beneath the bottom of the ties on the low rail, 14 inches on the high rail. The lifting loops for the span were left in place.

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 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. INSTRUMENTATION LAYOUT

The original instrumentation plan for the test installation included approximately 40 channels of information to be monitored during the tests. Due to damaged wires during concrete casting, several gages were lost and the final instrumentation remaining for monitoring included:

Instrumentation Installed during fabrication:

- Linear strain gages (1/4”) bonded to the top flange of the HCB at center span (under deck)
- Strain gages on diagonal reinforcing bars connecting the deck overlay to the concrete arch (damaged and compromised prior to testing)
- Strain gages bonded to the reinforcing bars cast into the concrete arches (also lost prior to test)

Additional channels installed after the HCB span was in place at FAST:

- Linear strain gages (1/4”) bonded to the bottom flange of each girder at center span
- Horizontal linear variable displacement transducers (LVDT’s) at ends of beams to measure relative displacement between overlay and beam (at four corners of span)
- Linear strain gages (2”) bonded to top of overlay at center span (4 each)
- Linear strain gages (2”) bonded to top of overlay at quarter span (4 each)
- Vertical displacement transducers (String Pots) under each beam at center span

The surviving gages made it possible to measure bending strains at center span and quantify maximum stresses in the top and bottom laminate as well as compression stresses in the overlay. Positioning gages and string pots on the bottom of each beam made it possible to quantify any differential distribution of forces in the assembly of girders. The linear variable displacement transducers (LVDT’s) 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.

In all, the instrumentation plan available during testing comprised 28 channels of data. 350 ohm gages were used at all locations. All transducers were connected to a dynamic data collection system base including a Megadac linked to a laptop computer, running a TTCI analysis program for real time review of selected channels and test parameters during the load tests.

5. LOAD TEST

The initial live load testing began with axle loads carefully positioned for maximum moments and shears using approximately seven different positions of the cars in a static condition. Once this was completed, dynamic tests were initiated to observe the behavior of the HCB Bridge under live train operations starting with the train operating at 2 mph.

5.1 Static Tests

Static testing was conducted first. Three of the heaviest FAST gondolas were selected for the static test. Nominal car weight is 315,000 lbs, or 157,500 lbs per truck. Each truck was
weighed to provide known loads for the static tests. The cars and their truck weights were as follows:

<table>
<thead>
<tr>
<th>Car</th>
<th>A truck:</th>
<th>B truck:</th>
</tr>
</thead>
<tbody>
<tr>
<td>309</td>
<td>163,450 lb</td>
<td>163,575 lb</td>
</tr>
<tr>
<td>362</td>
<td>159,975 lb</td>
<td>160,175 lb</td>
</tr>
<tr>
<td>476</td>
<td>158,500 lb</td>
<td>161,300 lb</td>
</tr>
</tbody>
</table>

The cars were positioned at one end of the train as follows:

- Car 362: A end leading
- Car 476: A end leading
- Car 309: A end leading

Total weight of axles in 3 load groups on the HCB span for the load sequence was as follows:

<table>
<thead>
<tr>
<th>Car</th>
<th>Position</th>
<th>Total weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>362</td>
<td>A end</td>
<td>159,975 lb</td>
</tr>
<tr>
<td>362</td>
<td>B end &amp; Car 476</td>
<td>318,675 lb</td>
</tr>
<tr>
<td>476</td>
<td>B end &amp; Car 309</td>
<td>324,750 lb</td>
</tr>
</tbody>
</table>

Car dimensions are as follows:

- Overall length: 53'0"
- Truck centers: 40'6"
- Axle spacing: 6'0"
- Distance between adjacent axles of coupled cars: 6'6"
Figures 5.1 and 5.2 show static load testing. The train was positioned to have the load groups at locations to produce maximum shear force, maximum bending moment, and centered on the span.

### 5.2 Dynamic Tests

After successful completion of the static tests, a series of dynamic tests were performed. A train with 16 cars, all nominally 315,000 lbs, and two 4-axle locomotives (about 270,000 lbs each) crossed the bridge at speeds 2 mph and 5 mph to 45 mph in 5 mph increments. Dynamic test runs were performed in both clockwise and counterclockwise directions. 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.

### 5.3 Service Simulation Tests

On November 8, 2008, after successful completion of the dynamic tests, additional dynamic tests were run with two locomotives and a consist of 26 heavy axle cars, each car nominally 315,000 lbs. These tests were intended to provide a short-term endurance test representative of revenue service conditions. A total of 47 passes were made over the bridge at 40 mph in the clockwise direction. At the end of these tests, a total tonnage of about 261,000 gross tons (0.26 MGT) had crossed over the HCB span. The dynamic tests contributed about 56,000 gross tons; the service simulation tests about 205,000 gross tons. Further extended endurance testing is still planned on the FAST facility in the near future. The effects of train
velocity and change in the structural Behavior with extended loading will be discussed in the subsequent section.

5.4 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) – 5 mph Counter-Clockwise Run, Data Set Number 6
- Conc9_17 – (07-Nov-07) – 45 mph Counter-Clockwise Run, Data Set Number 17
- Conc9_35 – (08-Nov-07) – 45 mph run, 4\textsuperscript{th} Pass, Data Set Number 35
- Conc9_79 – (08-Nov-07) – 45 mph run, 46\textsuperscript{th} Pass, Data Set Number 79

Using the train-in-motion tests provided 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 it was necessary to account for 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. The curves used to evaluate the modifications to the predicted responses due to superelevation and train velocity can be seen in the graphs shown in Figure 5.4.
Along with the superelevation, 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.

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 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.

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 5.1 Live Load Deflections

<table>
<thead>
<tr>
<th>Run File</th>
<th>D21 in</th>
<th>D22 in</th>
<th>D23 in</th>
<th>D24 in</th>
<th>Average in</th>
<th>Predicted in</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conc9_6</td>
<td>0.077</td>
<td>0.151</td>
<td>0.234</td>
<td>0.327</td>
<td>0.197</td>
<td>0.186</td>
</tr>
<tr>
<td>Conc9_17</td>
<td>0.153</td>
<td>0.238</td>
<td>0.305</td>
<td>0.389</td>
<td>0.271</td>
<td>0.261</td>
</tr>
<tr>
<td>Conc9_35</td>
<td>0.174</td>
<td>0.257</td>
<td>0.332</td>
<td>0.421</td>
<td>0.296</td>
<td>0.261</td>
</tr>
<tr>
<td>Conc9_79</td>
<td>0.165</td>
<td>0.255</td>
<td>0.341</td>
<td>0.413</td>
<td>0.294</td>
<td>0.261</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Run File</th>
<th>D25 in</th>
<th>D26 in</th>
<th>D27 in</th>
<th>D28 in</th>
<th>Average in</th>
<th>Predicted in</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conc9_6</td>
<td>0.342</td>
<td>0.340</td>
<td>0.290</td>
<td>0.273</td>
<td>0.311</td>
<td>0.403</td>
</tr>
<tr>
<td>Conc9_17</td>
<td>0.359</td>
<td>0.332</td>
<td>0.273</td>
<td>0.203</td>
<td>0.291</td>
<td>0.291</td>
</tr>
<tr>
<td>Conc9_35</td>
<td>0.385</td>
<td>0.346</td>
<td>0.273</td>
<td>0.202</td>
<td>0.302</td>
<td>0.291</td>
</tr>
<tr>
<td>Conc9_79</td>
<td>0.379</td>
<td>0.352</td>
<td>0.279</td>
<td>0.219</td>
<td>0.307</td>
<td>0.291</td>
</tr>
</tbody>
</table>
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 highest laminate strain of $614 \ \mu e$ 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 $614 \ \mu e$ is 7.32 ksi, which is well below the 360 ksi ultimate tensile strength of the steel. The predicted strains were approximately between 450\(\mu e\) and 500 \(\mu e\).

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. Some of these 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 were 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.

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 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.

5.5 Post-Test Inspection

After the completion of the endurance testing, the track was removed from the HCB span in order to inspect the concrete deck. Cracks developed at each end of the span, and in the ballast curb, as shown in Figure 5.4. The shear cracks in the overlay are most likely the result of insufficient reinforcing steel in the concrete deck to accommodate these stresses at the ends of the span.
Although there was no indication of distress to the HCB’s, due to the cracking in the deck, and the potential for the deck to become a maintenance issue, the HCB span was removed and the original concrete span was reinstalled for continued FAST train operations. In discussion with railroad bridge engineers, a minimum concrete thickness of 6 inches was suggested to mitigate the possibility for cracking. For comparison, the deck thickness on the original prestressed concrete span in that location is 6 inches.

6. DISCUSSION, CONCLUSIONS, ACKNOWLEDGEMENTS

As the technology for the HCB continues to evolve and approach commercialization, it is worth reflecting on the experiences of trying to introduce a new technology in an industry that is careful not to adapt change too quickly. The challenges encountered in quantifying the structural behavior and understanding the manufacturing processes provide no small amount of intellectual stimulation that at many times can be perceived as purely academic. The more fascinating challenges and certainly the ones that require the most persistence involve
interfacing with people in every facet of the transportation infrastructure and trying to ascertain what compels people to adapt to change. The success of this project can be directly attributed to the overwhelming support received from the organizations listed below that recognized the need to advance the technology of bridge construction.

The ultimate objective of this adventure has always been to develop a new type of structural element that provides some inherent improvements over conventional bridge framing members. In summary, some of the benefits of the HCB include: lighter weight for reduced erection time and cost, optimization of every material utilized, excellent corrosion resistance lending to better life-cycle costs and simplicity in design, fabrication and erection. Despite the intrinsic benefits of any new technology, the key to commercial success can almost inevitably be traced to lower initial first costs. Through years of refinements and validation, the HCB does offer a unique solution to bridge technology that is cost competitive with conventional methods of construction.

7. ACKNOWLEDGEMENTS

TRB and the HSR/HCHR-P-IDEA programs

University of Delaware – Center for Composite Materials

Transportation Technology Center, Inc.

BNSF

Canadian Pacific

Canadian National

Norfolk Southern

Union Pacific
Hardwire LLC
Ashland
Owens-Corning

8. REFERENCES

FIGURES AND TABLES

Fig. 1.0 Fragmentary Perspective of Hybrid-Composite Beam

Fig. 2.1 Tying to Together HCB Units

Fig. 2.2 HCB Unit with Compression Reinforcing

Fig. 2.3 Cross-Section of Prototype Bridge as Tested

Fig. 3.1 Unloading HCB Unit from Flatcar

Fig. 3.2 Setting HCB Unit on Bridge Substructure

Fig. 3.3 Installing first HCB Span Half Section

Fig. 3.4 First HCB Span Half Section in Place

Fig. 4.1 Strain Gage Installation on Deck

Fig. 4.2 Track Work Completed on HCB Span

Fig. 5.1 Static Load Test-Maximum Moment

Fig. 5.2 Dynamic Load Testing - Counterclockwise

Figure 5.3: Superelevation Effects for Test Bridge with 315,000 lb Cars.

Fig. 5.4 Cracking of Concrete Deck Overlay After T

Table 5.1 Live Load Deflections