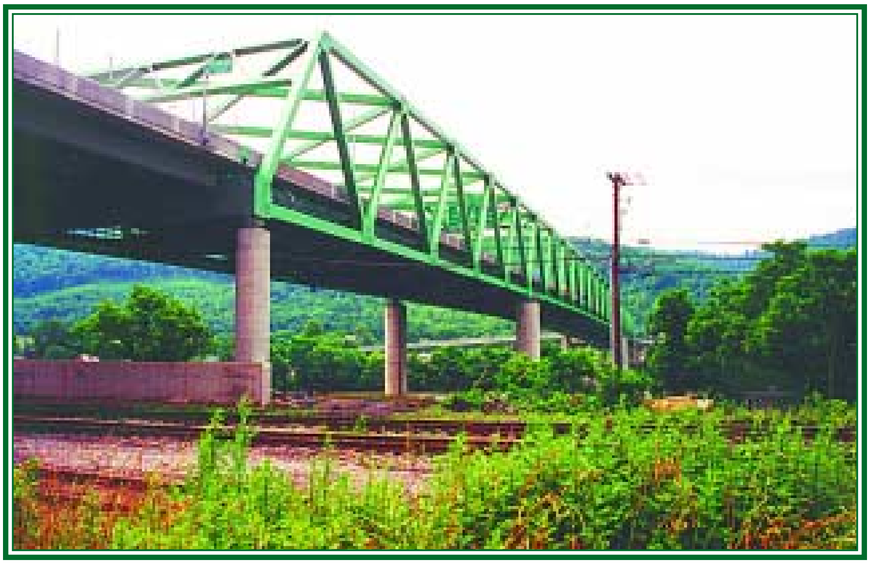
High Strength Steel Reduces Truss Costs

A new bridge across the Chelyan River in West Virginia utilized Gr.36, 50 and 70 steels



By Kenneth J. Wright, P.E. and Matthew A. Bunner, P.E.

The recently completed Admiral T. J. Lopez Bridge crosses the Kanawha River approximately 12 miles upstream of Charleston, WV, in the town of Chelyan. The river is navigable, with a large volume of commercial traffic. The old two lane Chelyan Bridge, which was replaced by the new structure, consisted of a series of short steel approach spans on the south side of the river, carrying traffic over a residential area and onto three truss spans over the Kanawha River, and connected W.V. 61 on the south side of the river with U.S. 60 on the north side. The bridge was built in 1929 and was posted at 12 tons, which severely limited its use by commercial traffic. The old bridge remained open during construction of the new bridge.

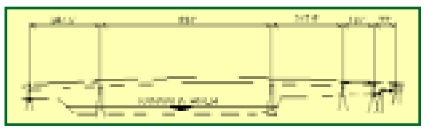
The four lane replacement structure was constructed approximately 1400' downstream of the old bridge and provides a direct connection between the West Virginia Turnpike (I-77) on the south end and U.S. 60 on the north end. A ramp providing access to W.V. 61 and the town of Chelyan intersects the mainline approaches at the south end of the river bridge, forming a T-intersection.

HDR Engineering, Inc., was engaged by the West Virginia Department of Transportation (WVDOT) in May of 1992 to design the steel alternate replacement structure and all associated approach roadways, including all walls required to support the approaches.

Preliminary Studies

Five different structure types were studied for the river bridge during the preliminary design phase. The span arrangements studied were controlled by geometric constraints and physical features in the area of the bridge. The U.S. Coast Guard (USCG) required a 550' clear navigation span with a vertical clearance over the channel of approximately 70'. Since the new bridge is skewed approximately 10 degrees relative to the navigation channel, a span length of over 590' is necessary to provide a 550' clear navigation channel. The location of the south end of the structure was dictated by the location of the connector ramp T-intersection, and the north end was dictated by the Conrail railroad tracks. The structure depth below grade was also critical, due to the vertical clearance over the navigation channel required by the USCG. For deep structures such as the plate girder option discussed below, an undesirable brokenback profile was required to achieve the required vertical clearance. The following structure types were studied for the river bridge:

1. Three-Span Continuous Plate Girder (247.5'-594'-247.5')



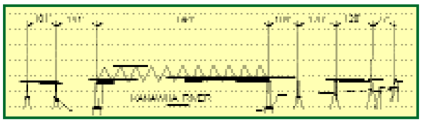
Girder studies compared a five girder system to a girder-substringer system with three girders and two rolled beam substringers supported on truss-type floorbeams. Both constant depth and variable depth girders were studied. The girder-substringer system with constant depth girders, utilizing both A709 Grade 50 and Grade 70W steels, proved to be the more economical solution. The main advantages to this option were that plate girder fabrication is common and well-defined and there would be significant duplication in fabrication.

However, the disadvantages for this option outweighed the advantages. The 17' web depth would require horizontal web splices due to limited plate availability. The poor span balance resulted in uplift at the end supports. Erection would also be difficult due to the large, heavy girder pieces, and would likely have required falsework in the river. Special permits would likely have been required to ship by truck. Finally the aesthetics of such deep girders were undesirable when compared to the other options studied.

2. Three Span Continuous Box Girders (247.5'-594'-247.5')

No preliminary designs were actually performed for this option. Presumptive comparisons indicated that box girders would be less economical than plate girders. Three box girders would have been required to avoid a fracture critical structure, resulting in additional web lines as compared to the plate girder option. In addition, shipping would likely have been a problem due to the size and weight of box sections required for this type of structure.

3. Simple Span Truss (594')



A simple span parallel-chord Warren truss with no verticals, consisting of eleven 54'-0" panels was studied. The trusses were on 68'-0" centers with a depth of 57'-0" center to center of chords. The 8" concrete deck was supported by eight stringers spaced at 8'-0", and carried a 56'-0" roadway, a 5'-0" sidewalk and parapets for a total out-to-out width of 63'-9". The entire deck was contained inside the trusses, eliminating the need for costly sidewalk brackets on the outside of the upstream truss.

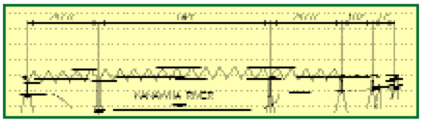
The chord members and compression diagonals were designed as box members and the tension diagonals as built-up I sections. ASTM A709 Grade 36, 50 and 70W steels were used for the truss members. The top lateral bracing system was a diamond pattern with no struts between panel points. The bottom lateral bracing was a K system which used fewer members and required fewer end connections than an X-bracing system. All bracing members were welded boxes.

The truss was designed with top chord sway bracing only at the end posts. Floorbeams were detailed with moment connections at the ends to provide rigidity. Elimination of sway frames reduced the number of members, improving the economy of the structure and creating a clean, contemporary appearance.

Tension members in a simple span truss are considered to be fracture critical. One method proposed to provide internal redundancy was to use bolted, built-up sections for the truss members. However, fabrication costs for built-up members were deemed prohibitive.

Constructability would be difficult since the truss is entirely over water. Two costly falsework bents in the river would likely be required for erection. Location of the falsework would be less than ideal due to USCG requirements of a 400'-0" temporary navigation channel during construction.

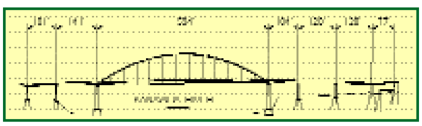
4. Three-Span Continuous Truss (247.5'-594'-247.5')



Panel lengths for the truss were 49'-6" throughout the length, yielding 5 panels in each end span and 12 panels in the center span. The truss is a parallel chord Warren truss with no verticals and a depth of 42'-0" center to center of chords. All deck, framing and member composition were as discussed for the simple span truss.

The redundancy issues for the continuous truss are similar to those discussed for the simple span truss. Erection of the continuous truss is feasible without expensive shoring in the river. Light falsework towers in the end spans will permit balanced cantilever construction of the truss.

5. Tied Arch (594.75')

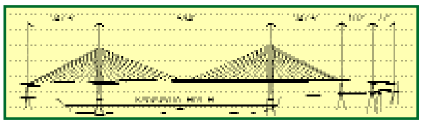


A solid rib tied arch structure consisting of thirteen 45'-9" panels was also studied. The arch ribs were 69'-0" apart, with a rise of 100' at the centerline of the span. The 9'-6" deep tie girder resists the thrust exerted by the arch and is primarily a tension member. It also provides most of the bending stiffness for the structure, carrying much of the live load bending from the floor system. The arch rib is primarily a compression member, carrying only a small amount of the live load bending.

The floor system, deck and bottom lateral bracing were similar to the truss alternates. The top bracing was designed as a Vierendeel system consisting of welded box members, resulting in an open, contemporary appearance.

Construction of the tied arch would have been more difficult than for the other options due to the need for back stays to erect the arch ribs. Redundancy of the tie girders was also an issue. A bolted, built-up box section was proposed, designed so that if any one of the four main plates fractured the remaining section could carry maximum service loads without yielding. Consideration was given to post-tensioning the tie girder to eliminate tension and thus the fracture critical concerns, but preliminary cost estimates indicated that this was not economically competitive.

6. Twin Tower Cable-Stayed (247.5'-594'-247.5')



Two symmetrical H-shaped towers were used with two planes of cables outside the structure. The deck was composed of 10" precast concrete panels acting compositely with the steel floorbeams and the longitudinal edge beams. The edge beams were designed to carry both bending and axial loads induced by the stay cables. A 2" concrete overlay was included over the panels to serve as a wearing surface and to protect the panels.

Preliminary Study Results

Estimates indicated that costs for the five options studied were similar. Total structure costs (including approach spans) were as follows:

Continuous Plate Girder $26,300,000
Simple Span Truss $25,100,000
Continuous Truss $25,300,000
Tied Arch $25,500,000
Twin Tower Cable-Stayed..................$25,900,000

Given the level of refinement of the studies, no significant difference in cost was identified. Therefore, HDR recommended the tied arch structure for its pleasing aesthetics

Pictured at top is the new bridge looking northeast. Note the T-intersection span in the foreground (painted white). Pictured above is the old Chelyan Bridge, which was built in 1929 and was posted at 29 tons.

and the fact that it would offer one of the lowest future maintenance costs of any of the options.

However, due to agency concerns over the redundancy of the arch tie girder, the three-span continuous truss alternate was chosen for final design.

FINAL DESIGN: APPROACH STRUCTURES

All continuous approach span units were designed by utilizing a three-dimensional finite element (3D) analysis from BSDI, Ltd. The girders were designed for HS25 loading, but “baseline” designs were performed using a line girder analysis with HS20 loading and S/5.5 live load distribution factors to assure that girder plate sizes were not reduced past a reasonable comfort level.

The simple span unit supporting the T-intersection at the south end of the truss was designed using a grid analysis in GTSTRUDL. This unit was isolated as a simple span, with complex framing to support the geometry for the turning radii required. Lateral bracing was included to provide torsional stability to the span.

The following design guidelines were followed in the final design of the girders:

- The Load Factor Design method was used.
- Composite design was used
- 7/8” x 14” minimum flange for continuous plate girders
- Minimum flange plate widths 1/85 times the field section length
- Partially stiffened girder webs were used
- Deck placement sequences were analyzed
- Maximum shipping lengths of 120’

Crossframes were designed as K-frames with the point of the K at the top of the diaphragm. Since WVDOT directed that removable deck forms be used for the approaches and the girder spacing varies from 10’-0” to 13’-9” on centers, this crossframe configuration provided a potential midspan support point for deck forms to allow economical construction.

Detail designs for items such as crossframes and field splices were grouped to provide maximum repetition during fabrication. Since crossframes, consisting of structural tees fastened through the flanges, were modeled as structural members in the 3D analysis, they were considered as main members for design.

FINAL DESIGN

Briefly recapping the truss configuration:

- 3-span continuous 247.5’-594.0’-247.5’
- 68’ center to center of trusses
- 42’ center to center of chords
- Stacked floor system with 8” concrete deck
- Box members for chords and compression diagonals
- I shapes for tension diagonals
- Diamond-bracing for top chord, K-bracing for bottom chord
- Portal frames at end posts and interior piers

TRUSS ANALYSIS

The truss was analyzed both as a two-dimensional (2-D) frame using STAAD III computer software for dead and live load effects, and as a three dimensional (3-D) frame using GTSTRUDL. The 2-D model was a single truss with pin-ended members in the plane of the truss, subjected to panel concentrations to determine dead load effects. For live load effects, in-house software for the generation of individual truss member influence lines was coupled with the 2-D analysis to determine the worst case live loading. Truss member designs were based on the 2-D model and were performed in accordance with the AASHTO Guide Specifications for Strength Design of Truss Bridges.

For the 3-D analysis the truss members were modeled as pin-ended in the plane of the truss and rigid out-of-plane, and the major components of the floor system and the lateral bracing and sway bracing systems were included in the model.

The 3-D analysis was used to verify the dead and live load results obtained in the 2-D analysis, and also to determine the effect of the moment-resisting floorbeam end connections and bracing systems on the distribution of both in-plane and transverse (wind) loads between trusses.

Results of the 3-D analysis indicated that while the floorbeam end connections were modeled as fully rigid, the flexibility of the trusses and top lateral system permitted the truss bottom joints to rotate, resulting in a partially fixed condition.

In addition, classical wind analyses assume that lateral wind forces in the plane of the top chord are carried back to the piers through the top lateral system, and down the portal or sway bracing frames to the bearings. The 3-D model, however, indicated that these wind loads are transferred down to the floor system at each panel point by the truss diagonals. The diagonals and floor system were designed to transfer these loads.

DESIGN CONSIDERATIONS

An economical truss depth of 42'-0" was set both to obtain an efficient panel aspect ratio and to achieve an approximate depth to span ratio of one-tenth the distance between dead load inflection points of the truss. Grade 50 steel was utilized to the greatest extent possible in the truss to provide the best combination of strength and economy. The poor span balance resulted in many lightly stressed truss members. Grade 36 steel, with its less restrictive b/t requirements for compression members, was used for these members, permitting the use of thinner plates, lighter sections, and higher performance ratios. Grade 70W steel was utilized for the most highly stressed members where the higher unit cost was offset by the lighter sections required and economy of fabrication using thinner plates.

Lateral bracing member end connections were designed for either 75 percent of the member capacity or the average of the capacity and the actual member loads. This resulted in larger connections than would be required under current AASHTO specifications, particularly for the top lateral system which according to AASHTO needed to be detailed as deep as the top chord and therefore generally had a much higher capacity than was required for strength considerations.

Truss members were also checked for aeroelastic instability. Past experience has indicated that the lighter longer truss members could be subject to wind excitation. Open truss diagonals near the center of the bridge proved to be the most critical; however, these members were shown not to be susceptible to excitation at normal sustained wind speeds.

SPECIAL DETAILS

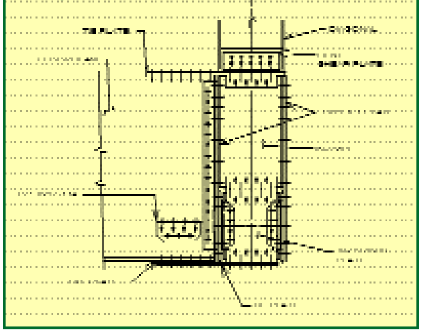
A number of unique design problems were addressed, resulting in structural details which are worthy of note:

No stress relief joints were provided in the deck of the truss. This mandated that the floorbeams be designed for transverse loads due to thermal movements and live load effects. Several fixity conditions were investigated for the stringer/floorbeam connections. It was determined that the optimum solution was to fix the deck/stringer system at the centerline of the bridge. All eight stringers are fixed at panel point L11. At typical floorbeams away from the centerline, only two stringers were fixed. These points of fixity were located approximately at the third points along the floorbeam. While these connections are the location of application of the out of plane forces on the floorbeam, they also provide points of lateral support for the compression flange of the floorbeam.

It should be noted that the fixed stringer bearings were detailed with slotted connections to allow relative movement between the deck and the floorbeams during deck placement prior to tightening the connections. Had this not been done, the transverse loads applied to the top flange of the floorbeam would have produced out of plane stresses in the floorbeam which would have required an increase in the section.

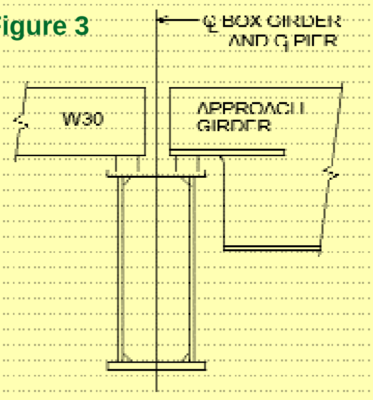
Truss joint connections were designed to transmit the wind loads into the individual diagonals at each panel point. Shear plates capable of carrying the transverse wind loads were provided at the pair of diagonals framing into each joint. These plates transmitted lateral shear forces between the trusses and the floor system. A path between the diagonals and the floorbeams capable of handling the moments induced at the bottom of the diagonals and in the floorbeams was provided. The joint, which was internally stiffened with diaphragms, was designed to accommodate the transfer of these moments back and forth between the members (Figure 2).

Figure 2



Uplift occurred under live load at end support locations as a result of the poor span balance. The approach girders were detailed to rest on the end floorbeam to overcome this uplift, thus avoiding costly tie-down details. Box floorbeams were provided at the ends of the truss to provide torsional stability due to eccentricity of reactions. An added benefit to this detail is that compatibility of deflections is achieved at the expansion joints, thus eliminating potential distortion of the joints (Figure 3).

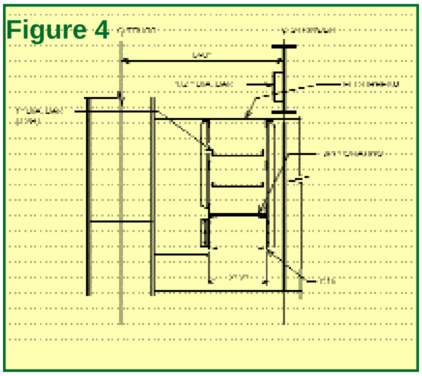
Figure 3



An inspection walkway was required adjacent to each truss. The typical detail of providing access manholes in the floorbeam webs at the inspection walkways would have resulted in a significant reduction in the shear capacity of the floorbeams where the shear is highest, requiring either a thicker web at the ends or costly web stiffening. Alternatively, the walkways were detailed to take advantage of the stacked floor system and clearance between the top of the floorbeam and the deck by providing a set of rungs on each side of the floorbeam webs to allow the inspector to climb over the floorbeam (Figure 4).

River piers for the truss were founded on spread footings on tremie concrete seals. Since the overburden is very small in the river (as little as 5'), the footings extend well up into the channel, with the top of footings located only 12' below normal pool. Tremie seals were recommended because hard sandstone was encountered in the river, making it unlikely that sheet piling could be toed into rock properly to seal the bottom of the cofferdam and permit dewatering without the concrete seal.

Figure 4



While it was necessary to provide access to all connections for erection, it was also desirable to prevent open spaces that could be used as nesting areas for birds, whose feces can cause considerable corrosion and deterioration of the steel members and connections. Bird screens were provided to seal off the manholes and therefore provide better performance over the life of the structure.

Fabrication and Erection Considerations

Throughout the design process, close contact was maintained with fabricators and erectors in the industry. Input on both general concepts and individual details was sought out to assure the economic and functional viability of the design. Examples of the comments received that were incorporated into the design included the use of a constant out-to-out dimension for the truss members, thereby eliminating the need for many fill plates, and details such as the use of rolled shapes rather than built-up sections for the bottom joint diaphragms.

A savings in fabrication cost for the truss was realized by PDM Bridge through the use of numerically controlled drilling of the joints. The detailer for the truss, Tensor Engineering of Indian Harbour Beach, FL, drew the connection details very carefully to scale, and PDM was then able to feed this information directly into their computers. This enabled the gusset plates and truss members to be fabricated without subpunching and reaming of the holes and eliminated costly shop fit-up of the joints.

Erection analysis was performed at the design stage for a balanced cantilever erection scheme assuming falsework towers would be used at joints L3 and L3' to check for overstresses during a number of different stages of the erection of the truss. End bearings were detailed with removable stools to allow the end of the truss to be dropped to facilitate closure of the center span. Certain key connections, particularly in the floor system, were designed with slotted holes and/or tightening of bolts was delayed until after deck placement to minimize participatory stresses.

Construction

The construction contract was let in May of 1995. Both steel and concrete alternate structures were advertised for bids in direct competition. The concrete alternate structure used prestressed concrete I-beams for the approach spans and a cable-stayed structure for the river bridge. Seven contractors bid the steel alternate and three bid the concrete alternate. The three low bids were on the steel alternate, with the low concrete bid approximately $990,000 above the low steel bid of $25,899,911. The second low steel bid was only $1571 above the low bid in very tight competition.

Construction of the approach spans began in the fall of 1995 and ran through the spring of 1997. The approaches consist of four continuous units and one simple span supporting the T intersection. Girder spacings varied between 10'-0" and 13'-9". The concrete decks were placed using removable deck forms as directed by the WVDOT.

Construction of the truss river bridge commenced early in 1996. Sheet pile cofferdams were required to construct the river piers. Dewatering was accomplished through the use of tremie concrete seals. Coring of the tremie seal for Pier 9 indicated some pockets near each end of the tremie seal where the aggregate had no grout, so repair work was considered. An attempt was made to drill and grout the tremie seal to fill the voids, but the grout did not communicate well between holes. Eventually the decision was made to drill several micropiles through the tremie seal into rock to support the outside edge of the tremie concrete in the event of future deterioration. Future deterioration is not highly probable, because the sheet piling for the cofferdam was to remain in place below

Typical top chord joint detail at interior pier sway bracing

the top of the tremie concrete seal. Since the voids are near the bottom of the tremie seal, it is unlikely that the sheet piling will deteriorate to the point that the bottom of the seal can be scoured.

Erection of the truss river bridge proceeded smoothly. The contractor chose to erect light falsework towers in each end span at panel points L3 and L3' as assumed by HDR during design. This permitted a balanced cantilever construction procedure working from both ends to close in the middle of the structure. The erection scheme submitted by the contractor provided for jacking of the superstructure at the falsework bents to align the two cantilevers to allow the closure pieces to be erected. However, vertical alignment of the cantilevers was only part of the issue. HDR's review of the erection analysis indicated that the ends of the cantilevers would need to be raised significantly higher than first proposed in order to obtain compatible rotations at the ends of the cantilevers so that the bolted joints could be assembled. This change proved to be very successful for the contractor and the closure was accomplished with little or no additional jacking required.

The contractor stopped construction of the end spans at panel points L1 and L1' until the closure was made at the center. The balanced cantilever construction was accomplished by stacking the stringers for the center span and concrete parapet sections at the end of the bridge to counteract the bridge weight in the center span. Once the closure was made, the counterweights were removed and the stringers were walked forward to their final positions. The truss was closed in 4½ months, and the remaining steel (floorbeams, stringers and lateral bracing) was erected over a period of approximately 2 months. The erection of the floor system was slowed somewhat by the fact that the stringers needed to be used as counterweight for the truss erection, and thus the pieces had to be moved multiple times to get them into position. In addition, since it was apparent that the deck could not be placed until the following spring, the erector assembled the floor system at a relaxed pace.

A total of 4,868 tons of fabricated structural steel was used in the superstructure for the project, of which 500 tons was A709 Grade 70W. In addition, the pile foundations for the piers and abutments incorporated another 994 tons of steel H-piles, for a total steel weight of 5,862 tons of steel on the bridge project. An additional 3400 tons of steel H-piles were used in the related approach roadway contract for construction of approximately 1650 feet of cantilever walls supported on H-piles.

Kenneth J. Wright, P.E. is a project manager and Matthew A. Bunner, P.E., is a project engineer with HDR Engineering in Pittsburgh.

Project Team

Owner: West Virginia DOT
Designer: HDR Engineering
GC: C. J. Mahan Const. Co.
Truss Steel Fab.: PDM Bridge
Approach Steel Fab.: Carolina Steel Fabrication, Inc.