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A CLOSER LOOK

Long-Span Bridges

This issue of BridgeLine focuses on long-span bridges. Designers and stakeholders face unique challenges in that the significance of appearance, maintenance, construction techniques and safety magnify as the physical size of the structure increases. Those issues and more are discussed here as we spotlight the Mountaineer Bridge. This new bridge crossing the Ohio River near the tip of the northern panhandle of West Virginia has a required navigational clearance of 800'. A number of alternates were considered conceptually, and in-depth preliminary engineering was performed for three alternates. Some critical aspects of the tied arch and cable-stayed alternates are highlighted inside.

INSIDE THIS ISSUE

Addressing the Challenges of Long-Span Bridges

By Kenneth J. Wright, P.E.

What makes the design of a long-span bridge different from that of a normal grade separation structure? While the basic stress checks are no different, there are other considerations for long-span structures that do not appear for shorter span lengths. For this discussion, span lengths over 500' will be considered long spans. Among the specific issues that enter into long-span bridges are:

AESTHETICS—Long-span bridges are, by definition, signature structures. They tend to dominate the local landscape, and the appearance should be carefully considered for harmony with the environment. The public usually has some input into the structure type, occasionally to the point that the structure type is determined by public opinion.

SERVICEABILITY—This becomes even more critical for long-span bridges. Given the investment made to construct long-span bridges, it is important to design a bridge that will not require constant maintenance to keep it in service. The owner must be comfortable that the bridge will last the desired service life.

CONSTRUCTABILITY—Given the large investment in materials and the equipment, temporary works and labor required to construct long-span bridges, it is imperative that constructability be considered during design.

SECURITY—Determining vulnerabilities of the bridge type, the value of the structure as a potential target and the ability of vandals or terrorists to get time on the structure to do their damage are important considerations when planning a long-span bridge. Designing to mitigate these risks may be required if the structure is considered an attractive target.

In the arena of long-span bridge design the West Virginia Department of Highways selected HDR to design a new Ohio River Bridge near the Mountaineer Racetrack and Gaming Resort, south of Chester, W.Va. The nearest Ohio River crossing is a severely load-restricted 100-year-old suspension bridge at Newell, approximately six miles north of the selected bridge location. Route 30 crosses the river approximately two miles farther upriver. The nearest crossing to the south is at Weirton, approximately 15 miles from the proposed bridge site.

The project was initiated to improve access to the Mountaineer Resort and to spur economic development in the northern panhandle of West Virginia. This region currently is served by WV Route 2, a two-lane road that passes through many small towns between Weirton and Chester. Ohio Route 7, a four-lane controlled-access highway, parallels the Ohio River through this same area. Locating a new bridge near the Mountaineer Gaming Resort would significantly improve access into the northern panhandle.

Above: I-470 Veterans Bridge, Wheeling, W.Va.
Below: Mountaineer Bridge, aerial view rendering.

The river is in curve and there is a lock and dam about three miles south of the site. The U.S. Coast Guard requires a navigational clearance of 800'. The bridge alignment crosses the Ohio River at a skew. The skew and piers required a main-span length of 885'. The client's primary interest was developing a serviceable bridge rather than a monument. They recognized that any bridge of this span length will be a signature structure, and chose to achieve an attractive structure through proper proportioning and good design.

Initially, eight bridge options were assessed, including various truss, tied arch and cable-stayed options. Three options were chosen for further study in the combined Type, Size and Location phase of the project: a three-span continuous variable-depth truss, a basket-handle tied arch and a three-span composite cable-stayed bridge. The articles in this edition of BridgeLine focus on issues encountered in the studies for the tied arch and cable-stayed alternates.

ABOUT THE AUTHORS

Matthew Bunner, P.E.

Structural Project Manager

Bunner is the project structural engineer for the Mountaineer Ohio River crossing. He coordinated and provided technical guidance for the design of alternates, including simple and continuous trusses, tied arches and steel and concrete cable-stayed bridges. Bunner's design experience includes straight and curved steel girder, concrete girder, steel truss and arch bridges. He has worked on a number of complex projects, including Hoover Dam Bypass and the Fort Pitt and Chelyan bridges, serving as the lead engineer for the three-span continuous truss at Chelyan.

As a National Science Foundation Engineering Research Scholar, Bunner conducted research on fatigue and fracture of steel bridges at the ATLSS Research Center from 1988 to 1990, where he worked with other experts to develop a knowledge-based computer system to aid bridge designers in the selection of fatigue resistant details during the design process. He recently co-authored the update of the Arch Bridge chapter in the fourth edition of the Structural Steel Designer's Handbook using the LRFD design method and is currently involved in the development of several chapters of the NSBA Steel Bridge Handbook.

Brian Kozy, Ph.D., P.E.

Structural Engineer

Dr. Kozy was the lead structural engineer for development of the cable-stayed alternate during the Mountaineer Bridge Type, Size and Location study. To ensure development of efficient and economical structural solutions, he visited the Cooper River Bridge construction site during the main span erection and met with designers, reviewers, construction engineers and lead contractor personnel. Dr. Kozy also worked closely with Walter Gatti of Tensor Engineering to establish the best possible cable-stayed bridge details. Dr. Kozy has managed structural designs, developed specifications, reviewed shop drawings and performed construction site inspections and engineering for new construction, rehabilitation and forensic investigations. He has taken a lead role on several major structures, including Hoover Dam Bypass, Cooper River Bridge review, Fort Pitt Bridge and Newark Airport Monorail.

Dr. Kozy's academic background is focused in the area of structural analysis; he has conducted research on the strength and stability of tubular steel structures using advanced nonlinear finite element analysis and experimental testing. This research led to new design specifications that he recently proposed to AASHTO T-12. In addition to his work at HDR, Dr. Kozy is an adjunct lecturer at the University of Pittsburgh.

Kenneth J. Wright, P.E.

Senior Vice President, Structures Section Manager and Project Manager

With more than 23 years experience at HDR's Pittsburgh office, Wright served as the bridge project manager for the Mountaineer Bridge, providing oversight and technical guidance for all alternates studied. He has worked on the design of various river bridges and has designed many tangent, curved and/or skewed steel bridges. Wright also has performed bridge erection and demolition engineering and construction inspection.

He managed the design of the Chelyan Bridge over the Kanawha River, which received an NSBA Award of Merit in the long span category in 1998, and the Appalachian Corridor H Bridge over Clifford Hollow in Hardy County, W.Va, which won the 2005 NSBA Prize Bridge award in the long span category. He recently co-authored the Arch Bridge chapter update in the fourth edition of the Structural Steel Designer's Handbook using the LRFD design method and has presented at numerous national conferences. As a recognized mentor, Wright is integral to identifying emerging talent within the company and assigning them to roles that both challenge them and contribute to their professional development.

Selecting the Shape of a Steel Arch

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

Determining the shape of the arch is one of the most critical choices when designing an arch bridge. Shape impacts both the performance and appearance of the structure. Selecting an appropriate shape results in an aesthetically pleasing structure that is both efficient and economical. Examples from recent preliminary design studies prepared for the Mountaineer Bridge project and other tied arch designs will be used to illustrate how individual aspects of arch design impact the final shape and vice versa.

Arch Rise

The rise of an arch directly impacts the magnitude of the axial compressive force (thrust) in the arch rib and, in the case of a tied arch, the corresponding tension in the tie girder. (General methods of providing internal redundancy for tie girders were discussed in a BridgeLine article authored by Art Hedgren in January 1993.) The maximum thrust can be estimated using a simple beam analogy once the approximate loads on the bridge have been determined. When load is expressed as a uniformly distributed load (w) over the length of the bridge, the maximum thrust is calculated as:

$$\frac{wl^2}{8h} \quad \text{where } l = \text{the span length and } h = \text{the arch rise}$$

Thrust is inversely proportional to arch rise. The member areas required for the rib and tie girder also follow this relationship. Other factors also impact the economy of the design. Although the member area is proportionately smaller for a larger rise, rib length and the amount of material in the top lateral bracing system increase. Thus the total material savings is not clear-cut. As the arch rise increases, so does the length of suspenders. In addition, erecting the arch requires larger falsework to support the partially assembled rib and, potentially, larger cranes to accommodate the increased height.

While efficient designs can be prepared for different span-to-depth ratios, the impact of rise on economy should be investigated on a case-by-case basis for each project. The aesthetic appeal of different span-to-rise ratios is subjective. Practically, the span-to-rise ratio for a tied arch can vary between 4 and 7. Often, a more pleasing appearance is realized in the 5.5 to 6 range. For the Mountaineer Bridge, an arch rise study was performed by analyzing span-to-rise ratios ranging from 5 to 7 in increments of 0.5. This study was based on a basket-handle configuration (ribs canted inward). Similar results could be expected for a tied arch with vertical ribs.

The arch ribs and tie girders used both 50 ksi and 70 ksi material. High performance steel, HPS 70W, was used within the end panels of the arch rib and throughout the tie girder. HPS 70W was used in the tie girder to increase fracture resistance of this fracture-critical bridge element. The out-to-out widths of the arch ribs and tie girders were held at 48" for ease of detailing. All options studied used one longitudinal stiffener in the web of the arch rib. The tie girders were designed to remain serviceable in the event of the complete fracture of one web plate or one flange plate. Quantities and costs for the five preliminary designs were based on a comparison of the arch ribs, tie girders, top lateral bracing and suspenders. Other components were not estimated because they were similar for all arch depths studied. The material quantities and normalized costs computed are summarized in Table 1.

The bridge pictured above has a span-to-rise ratio of 4, while the one below has a span-to-rise ratio of 6.

Practically, the span-to-rise ratio for a tied arch can vary between 4 and 7. Often, a more pleasing appearance is realized in the 5.5 to 6 range.

Appropriate material costs were used to develop cost estimates. A multiplier varying from 1.10 at a span-to-rise ratio of 5 to 1.00 at a ratio of 7 was applied to account for the difference in erection costs for the deeper arches. While this multiplier is subjective, it was important to reflect erection differences, and the 10 percent difference was considered realistic. The least cost was realized for a span-to-rise ratio of 5.5, but the difference between the least and most expensive ratio was only 2 percent. Live load deflections for all arch rises satisfied the L/800 criteria. The lateral displacement due to wind loading varied from 9.75" (span-to-rise ratio of 5) to 7.75" (span-to-rise ratio of

S:R Ratio	Rise (ft)	Suspenders (ft)	HPS 70 (tons)	Gr. 50 (tons)	Total (tons)	Erection Mult.	Normalized Cost
5	177	30,790	1550.4	1124.9	2675.3	1.10	1.019
5.5	161	28,569	1574.7	1149.1	2723.8	1.07	1.000
6	147.5	26,736	1699.4	1135.7	2835.1	1.04	1.004
6.5	136	25,205	1692.5	1252.4	2945.0	1.02	1.011
7	126.5	23,967	1720.3	1334.4	3054.6	1.00	1.020

Table 1. Material quantities and costs for various S:R ratios.

Rendering of basket-handle tied arch alternate.

7), which seemed reasonable. These results indicate that a functional, efficient structure can be achieved within the range of ratios studied. A span-to-rise ratio of 6 was recommended for final design, balancing both cost and appearance.

Arch Rib Geometry

It is now necessary to select the type of curve that will define the geometry of the arch rib. The goal is achieving a rib geometry that most closely follows the dead load thrust line of the rib. If the rib follows the thrust line precisely, there is no eccentricity of the compressive force and no resulting dead load moment in the rib, which is desirable. Because live loading is variable in position and magnitude, it will always induce some moment in the rib. However, on a long-span highway structure, the live loading effects are small in comparison to the dead load effects. The first task is accurately predicting the distribution of dead loads carried by the arch and the theoretical thrust line resulting from this distribution, and then setting a rib geometry that closely matches the thrust line.

If the dead load distribution on the arch was uniform from end to end, the thrust line would follow a parabolic shape. If there was no load on the arch other than self-weight, the thrust line would exhibit the shape of a transformed catenary, analogous to a rope hanging under its own weight.

The total dead load is actually a combination of the two conditions. The rib geometry of the HDR-designed I-470 Veterans Bridge, a 780’ tied arch in West Virginia, was examined for this article. Catenary, parabolic and circular curves were investigated, and the largest variance from the catenary shape was 15” for the parabolic curve and just under 4’ for the circular curve (falling between the catenary and parabolic curve). A higher-order curve was utilized in an attempt to reduce the variation from the actual dead load thrust line.

While defining the shape with a higher-order curve may seem unnecessary, the dead load rib thrust is extremely large. For the I-470 bridge, the dead load thrust was approximately 8,000 kips. A 6” eccentricity would produce a dead load moment of approximately 4,000 kip-feet, which is considerable. In general, the use of a parabolic shape for the arch rib is appropriate as a starting point, with adjustment to a higher-order curve as the design is refined.

Once the arch rib panel point elevations have been defined, it is necessary to choose whether to curve or chord the arch rib between panel points. While a chorded rib simplifies fabrication, higher moments develop due to deviation from the theoretical curve. Conversely, fabricating curved ribs is more costly than chorded ribs. This decision should be examined on a case-by-case basis, as both curved and chorded ribs have been used successfully. Curved ribs have been considered more contemporary and aesthetically pleasing.

Finally, it must be determined whether the depth of the rib itself should be varied. A tapered rib is aesthetically superior. On the Mountaineer Bridge project, the rib was tapered from a depth of 6’ at the knuckle to 3.75’ at the crown. The arch ribs for the I-470 bridge tapered from 7’ at the knuckle to 4’ at the crown. The use of HPS 70W in the arch rib near the knuckle led to the shallower rib depth for the Mountaineer Bridge.

Arch Rib Orientation

Two tied arch alternates were investigated for the Mountaineer Bridge: a conventional vertical rib tied arch and a basket-handle tied arch. For the basket-handle, the arches were canted inward at 12 degrees from vertical. The resulting arch spacing varied from 74’ at the tie girder to 11’ at the crown.

Both the conventional (vertical rib) and basket-handle style were designed to a level at which a meaningful economic comparison could be made. The studies showed significant differences in steel quantity for the arch rib, tie girder, tie bracing and rib bracing. Two variations of the conventional arch configuration were investigated. Design 1 used a limited amount of HPS 70W near the ends of the ribs and Grade 50 for the majority of the ribs. Design 2 used HPS 70W for the entire rib. The tie girder was HPS 70W for both designs. Cost estimates showed no significant benefit to using HPS 70W for the entire arch rib (Design 2), so that option was not pursued for the basket-handle option.

The steel unit prices assumed for the basket-handle were slightly higher to account for increased complexity of the details, fabrication and erection. Table 2 shows the quantities for items influencing the cost differential.

When the basket-handle alternate is compared to Design 1, the total quantity of steel in the arch ribs is nearly equal. The steel weight in the tie-girder is 14 percent greater for the conventional tied arch due primarily to the increased lateral bracing dead load. The largest difference in material is in the rib bracing, where the weight for the conventional tied arch is 336 percent greater than the basket-handle. There is approximately 2.3 million pounds of additional steel in the conventional tied arch when compared with the basket-handle.

When the basket-handle alternate is compared to Design 2, the amount of steel in the arch rib of the conventional arch is less than the basket-handle. However, due to the significant reduction in lateral bracing weight and the tie girders, the total basket-handle weight is approximately 1.6 million pounds lighter than Design 2.

Although the American Association of State Highway and Transportation Officials has no explicit requirements regarding the

SECTION	BASKET-HANDLE QUANTITY (LB)	CONVENTIONAL DESIGN #1 QUANTITY (LB)	DIFFERENCE (BH – Des. #1) (LB)	CONVENTIONAL DESIGN #2 QUANTITY (LB)	DIFFERENCE (BH – Des. #2) (LB)
RIB (GR. 50)	1,231,800	2,496,100	-1,264,300	0	+1,231,800
RIB (GR. 70)	1,343,200	229,100	+1,114,100	2,318,000	-974,800
Total Rib	2,575,000	2,725,200	-150,200	2,318,000	257,000
TIE-GIRDER	2,767,700	3,166,900	-399,200	3,128,000	-360,300
TIE BRACING	398,700	774,200	-375,500	774,200	-375,500
RIB BRACING	416,700	1,817,600	-1,400,900	1,578,000	-1,161,300

Table 2. Comparison of steel quantities for arch alternates.

depth of bracing members for an arch, AASHTO LRFD Bridge Design Specifications Section 6.7.5.4 regarding trusses states, "The members providing lateral bracing to compression chords should be as deep as practical and connected to both flanges." This requirement has typically been applied to arch rib bracing as well.

For the conventional tied arch, diamond bracing (X-bracing with no transverse struts) was used. This style of bracing is common in many tied arch structures and is generally analyzed as a truss system with the members subject to axial loads. For the basket-handle arch, a Vierendeel system of rib bracing was used. For this system, providing diamond bracing near the crown of the arch was geometrically impractical due to the aspect ratio of the panels. Secondarily, the increased lateral stiffness of the canted arches allows a Vierendeel system to be efficient.

When comparing lateral stiffness of the two arch types, the arch ribs for the conventional tied arch are less stiff laterally than the basket-handle. Horizontal loads applied to the arch rib of the conventional tied arch are transmitted to the base of the arch by shear only. In contrast, a horizontal load applied to the arch rib of the basket-handle is transmitted by both shear and axial load, somewhat analogous to the way in which battered piles resist both vertical and lateral loads. The relative lateral resistance from shear and axial load depends on the angle at which the arch rib is canted.

Different lateral bracing systems also differ in stiffness. The diamond bracing system used for the conventional tied arch is very stiff laterally, behaving as a wide truss between the arch ribs. The Vierendeel bracing used with the basket-handle arch system is much less stiff than the diamond bracing system. When looking at the overall bridge system, the diamond bracing in conjunction with the conventional arch results in a system that is relatively stiff laterally, with a majority of the lateral deflection occurring near the ends of the arch rib as the horizontal loads are transferred to the bearing via shear through the arch rib. The basket-handle arch with Vierendeel bracing results in an acceptably stiff system against lateral loads due to the degree of cant of the arch ribs. The resulting rib bracing is much lighter than would be possible with a diamond bracing system and fits the geometry of the arch ribs much better.

ARCH ALTERNATE	LATERAL RIB DEFLECTION (in)
BASKET-HANDLE	11.44
CONVENTIONAL (X-BRACING)	3.70
CONVENTIONAL (VIERENDEEL)	32.18

Table 3. Arch alternate lateral deflection.

The use of Vierendeel rib bracing was considered with the conventional arch, but was eliminated from further consideration due to the significant increase in lateral deflection that results. The vertical deflections caused by live load were examined and found to be within limits defined by AASHTO for all three alternates and therefore will be excluded from further discussion. The lateral deflections of each structure type when subjected to wind loads applied in accordance with AASHTO are shown in Table 3.

The basket-handle arch rib lateral deflections are larger than the conventional arch with X-bracing. The lateral deflections, however, are reasonable for a structure of this length. The lateral deflection of the arch rib is approximately equal to L/928, and the lateral deflection of the tie girders, which are near deck level, were generally significantly less than seen in the arch ribs. Although there are no limits on lateral deflection defined in AASHTO, the values obtained from the analysis are less than the limits on vertical live load deflection noted in the AASHTO Standard Specifications, which should lead to reasonable motorist comfort.

Rendering of basket-handle tied arch alternate.

The lateral deflections of the conventional arch with Vierendeel bracing are much higher than those for the basket-handle arch. The lateral deflection of the arch rib is approximately equal to L/330, which far exceeds the limits for vertical live load deflection as defined by AASHTO. This supports the assessment that the canted arch ribs in the basket-handle configuration are significantly stiffer laterally than vertical ribs. It also indicates that Vierendeel bracing is a more appropriate configuration when the struts can be shorter, increasing their stiffness relative to the stiffness of the arch rib.

The comparison of the models for conventional and basket-handle arches led us to conclude that arch rib orientation has very little impact on the arch forces generated by vertical loads. The bending moments and axial loads due to live load were very similar for the two arch orientations. Also, the vertical deflection was similar. Finally, basket-handle and conventional arch models with a complete fracture of one tie girder web or flange were compared. Neither arch orientation proved to be superior for load redistribution between the two arches, as both alternates could redistribute the loads without collapse.

Conclusion

The aesthetics of the two geometric configurations was the last variable to consider. It was believed that the basket-handle is a more aesthetically pleasing structure. The gentle curving of the arch ribs, minimal rib bracing and unique geometry yield a graceful and contemporary appearance that is superior to a more conventional vertical rib arch.

Another significant area of study for the tied arches on this project dealt with the configuration of the hangers for the tied arch. Most conventional tied arches have used vertical hangers between the arch rib and the tie girders. A networked hanger system, with the hangers sloping and crossing longitudinally, was compared with the vertical hangers and was found to significantly increase the stiffness of the structural system. This topic will be discussed in a future issue of BridgeLine.

Matthew Bunner, P.E., can be reached at HDR's Weirton, W.Va., office at (304) 748-8740 or e-mail mbunner@hdrinc.com. Contact Kenneth J. Wright, P.E., at HDR's Pittsburgh, Pa., office at (412) 497-6054 or e-mail kwright@hdrinc.com.

Development of a Composite Cable-Stayed Alternate

By Brian Kozy, P.h.D., P.E.

In recent years, as cable technologies have advanced, cable-stayed structures have proven to be economical for major river crossings with main spans in the range of Mountaineer Bridge. For this reason, the Type, Size and Location (TS&L) Study for this structure included preliminary design of a composite cable-stayed bridge alternate. Recently constructed bridges have shown composite cable-stayed design to be very economical. This article discusses two of the more important findings from the study.

Structure Configuration

The cable-stayed alternate is a composite design, with a precast concrete deck supported by steel framing. The span arrangement is two 375' back spans with an 885' center span. Two vertical planes of cables arranged in a fan, or radiating pattern connect to twin H-shaped towers rising 175' above the deck. This configuration yields a back span to main span ratio of 0.42, a ratio of tower height (above the deck) to main span length of approximately 0.2, and a minimum cable angle of 22 degrees from horizontal—all of which are desirable proportions for this type of structure. The nominal cable spacing along the structure deck was set at 46'6", which is within the typical range for a composite deck system.

Cable Pattern

The initial study phase identified two distinct cable patterns as viable possibilities: the fan pattern and the semi-harp pattern (see Figures 1 and 2). The fan pattern is characterized by all cables converging nearly to a single location at the top of the tower (the tower head) while the semi-harp pattern is characterized by spreading the cable anchorage work points vertically along the tower legs. For the fan pattern, a nominal spreading of the anchorage points is necessary to reasonably fit all of the anchorages into the tower head. In general, the fan pattern is found to be the lowest initial-cost solution, but the semi-harp pattern may offer long-term advantages with regard to serviceability, inspectability and, in some opinions, aesthetics.

Rendering of the cable-stayed alternate.

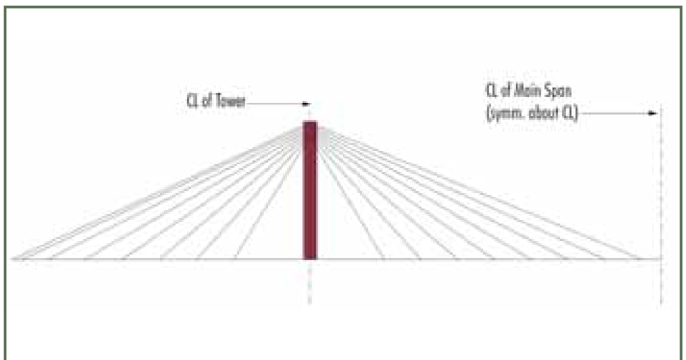

Figure 1. Fan Cable Pattern.

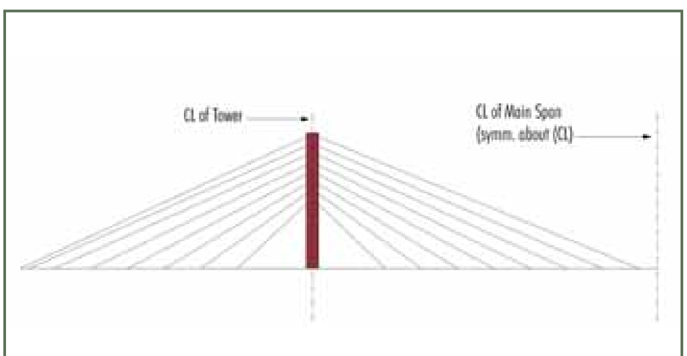

Figure 2. Semi-Harp Cable Pattern.

The fan cable pattern minimizes required cable sizes by maximizing vertical components of the stay forces. It also minimizes the axial compressive force in the deck (which benefits the edge girder design), and minimizes shear and bending in the upper portion of the tower legs (which benefits the tower design). Tower construction is simplified by concentrating anchorages into one location at the tower head as opposed to casting multiple individual anchorages along the height of the tower legs. Forming and casting tower legs is much easier and the difficult geometry of the cable anchorages can be addressed in steel shop fabrication of a single structural element where the tolerances are more easily controlled. The only stay anchorage geometry that must be controlled in the field is in the initial placement of the tower heads. Each tower head can be set atop the towers with relative ease, providing reliable geometric control as well as centralized access for the stressing operations.

Careful design and detailing is necessary to ensure that the tower head concept works effectively (see Figure 3, facing page). The tower head must be large enough to accommodate stressing operations and future inspections, yet small and light enough to be erected. Because stressing at the deck level typically adds undue expense to edge girder connection details, the tower anchorages are designed to be the stressing end, and edge girder anchorages are the "dead end." To make tower head erection more manageable, the tower head is divided into two individual cells that are fastened together during erection. A similar detail was used successfully on the recently constructed Leonard P. Zakim Bunker Hill Bridge in Boston, designed by another firm.

In contrast, the semi-harp pattern utilizes smaller, more manageable field pieces for the tower anchorage steel. However, each must be carefully positioned and cast into the tower leg concrete, which makes geometric control difficult and slows construction. Also, using separate anchorage frames is a less efficient use of the steel material since each individual frame uses moderately more steel to provide basic stability and continuity in the frames.

Preliminary designs and cost estimates were developed for both the fan and semi-harp cable pattern options. The estimated initial cost premium for the semi-harp over the fan pattern was estimated to be approximately $500,000 (1.7 percent of total structure cost).

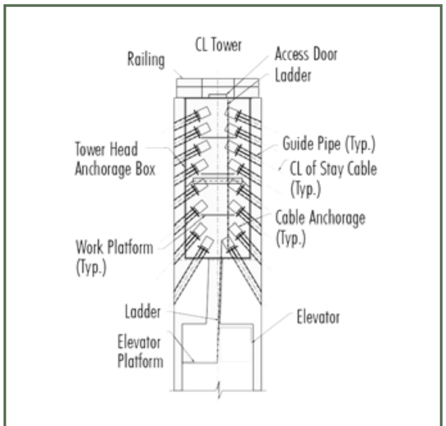

Figure 3. Tower head connection detail for fan cable pattern alternate.

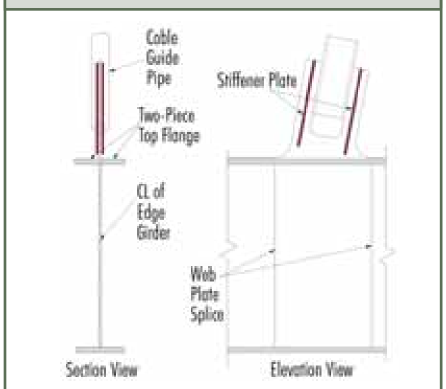

Figure 4. Integral web connection.

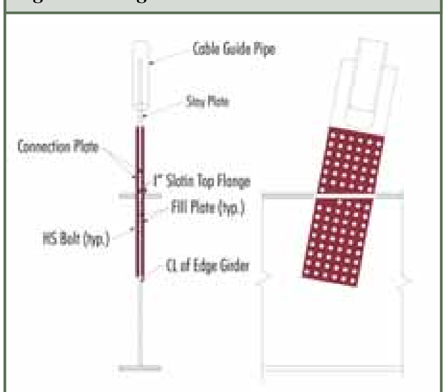

Figure 5. Slotted and bolted connection.

Edge Girder Cable Connection

One of the most important decisions to be made with regard to steel detailing in a composite cable-stayed bridge is at the edge girder-to-stay cable connection. The geometry is quite complicated and significant customized labor and/or materials are typically needed to build a mechanism for transferring the stay cable forces into the edge girder. Also, the potential for fatigue problems is high at this location due to the intersecting steel components and complex stress fields. Experience in the industry and recommendations of fabricators indicate the following options: integral web connection (Figure 4) or slotted and bolted connection (Figure 5).

The first option considered is the integral web connection, often referred to as the "shark fin" detail due to the shape of the web plate. The main advantage of the shark fin is that it does not require labor-intensive bolting. Bolting slows fabrication and erection, and makes painting more difficult. Another advantage is that there are no connection components on the web to interfere with the floorbeam connections. Rather than cutting a slot into the top flange to accommodate the shark fin, using two separate plates for the top flange that are continuously welded to the sides of the web plate are recommended.

This eliminates the fatigue-prone detail at the ends of a slotted flange. The shark fin detail has been used successfully on a number of recent cable-stayed bridges designed by others, including the US 82 Bridge over the Mississippi River (est. 2006) in Greenville, Miss., the Bill Emerson Memorial Bridge (Mississippi River) at Cape Girardeau, Mo., (2003), and the Arthur Ravenel Jr. (Cooper River) Bridge in Charleston, S.C., (2005). Two photos at right show examples of integral web connections used on the Ravenel Bridge.

The shark fin detail does have some disadvantages. First, there is a moderate amount of scrap steel generated in cutting the plate. It is desirable to have the rolling direction of the steel in the shark fin oriented along the direction of the attached stay cable, but this increases waste even more. While the amount of waste is smaller for locations where the stay cables are near vertical or horizontal, the waste increases where the stay cables are at intermediate angles. Also, this detail requires two additional longitudinal fillet welds for the two-piece top flange. However, these welds can be made using an automated process and should not add significant cost. The shark fin detail can be problematic in cases where the cable planes are inclined, such as with a diamond or inverted Y shaped tower configuration, but this is not a concern for the current design since the cable planes are vertical.

The slotted and bolted connection (Figure 5) uses two connection plates fitted through slots in the top flange of the edge girder and

Above is a girder with an integral web connection. Below is a shark fin after installation of the bridge deck and connection to the cable.

Photo courtesy of C. Frank Starmer

bolted to the web. Two slots are recommended so the longitudinal web-to-flange fillet welds on the top flange can be run continuously through the connection, thus avoiding a category E fatigue detail at the weld termination. The primary advantage of this detail is its efficient use of material. The web and flanges of the edge girder are standard plates requiring no special fabrication other than the top flange slots.

The total amount of steel needed is minimal, and the bolts are all loaded in double shear. Also, this detail is preferred in terms of fatigue considerations. There are few welded components and most could be repaired/replaced with relative ease should any future problems arise. There is concern with regard to fatigue in the edge girder top flange at the ends of the flange slots, which would be considered a Category D detail. However, the fatigue performance could be improved by installing high-strength bolts through the slots at the end.

The disadvantage of this detail is labor-intensive assembly. The bolt holes are typically cut into the web individually using a portable magnetic drill in the shop with the connection plate as the cutting template to ensure proper fit. Also, painting is more time-consuming due to the

(continued on page 8)

(continued from page 7)

sequences of assembly and faying surfaces. In addition, the connection plates may require the floorbeam connections to be moved farther away from the cable anchorage point to avoid interference between component connections.

Based on the current market for labor versus material cost, the integral web connection (shark fin) appears to be the more cost-effective detail. Labor savings due to eliminating bolted connections likely will more than offset the increase in material cost. It also is believed that use of HPS 70W steel for the shark fin would reduce fracture concerns.

Conclusion

This article has described two key issues that were addressed in the TS&L study of a cable-stayed alternate for the Mountaineer Bridge. In general, composite cable-stayed bridges produce a very durable deck system, a shallow superstructure which may allow for lowering of the profile grade, and a modern, aesthetically pleasing structure. Some disadvantages include specialized construction requirements, future maintenance of cable anchorages and vulnerability of critical structural elements. In the end, the TS&L study found the cable-stayed alternate to be the most economical of all alternates studied.

Brian Kozy, Ph.D., P.E., can be reached at HDR's Pittsburgh, Pa., office at (412) 497-6078 or e-mail bkozy@hdrinc.com.

Calendar of Bridge Conferences and Seminars

Event/Date	Contact
Wind Induced Vibration of Cable Stay Bridges Workshop April 25–27 St. Louis, Missouri	Carissa Hutson (573) 526-2119 carissa.hutson@modot.mo.gov www.modot.org/csb/
2006 Concrete Bridge Conference May 7–9 Reno, Nevada (Rapid Bridge Construction Conference May 10)	Shri Bhidé 847-972-9100 sbhide@cement.org www.nationalconcretebridge.org/cbc
2006 Structures Congress May 18–20 St. Louis, Missouri (17th Analysis & Computation Specialty Conference runs May 18–21)	Ms. Sheana Singletary conf@asce.org www.asce.org
2006 AASHTO Subcommittee on Bridges and Structures Annual Meeting May 21–26 Snowbird, Utah	Kelley Rehm krehm@aashto.org www.aashto.org/aashto/calendar.nsf
23rd Annual International Bridge Conference June 12–14 Pittsburgh, Pennsylvania	Ryan Bock (412) 261-0710 ext. 11 Fax (412) 261-1606 conf@eswp.com www.eswp.com
First International Conference on Fatigue and Fracture in the Infrastructure—Bridges and Structures of the 21st Century August 6–9 Philadelphia, Pennsylvania	Alyssa Clapp (610)758-3535 inffconf@lehigh.edu http://ffconf.atlss.lehigh.edu/
Seventh International Conference on Short and Medium Span Bridges 2006 August 23–25 Montreal, Canada	Lidia Issid (514) 393-1000 #7715 Fax: (514) 393-0156 bridgeconference2006@snclavalin.com www.bridgeconference2006.com
Fifth National Seismic Conference on Bridges and Highways September 18–20 San Francisco, California	Jerome O'Connor (716) 645-3391 ext 107 Fax (716) 645-3399 jso7@buffalo.edu http://mceer.buffalo.edu/meetings/5nsc/default.asp
American Segmental Bridge Institute 2006 National Convention November 5–7 San Diego, California	Clifford Freyermuth (602) 997-9964 asbi@earthlink.net www.asbi-assoc.org

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Staff

Editor..Ken Wall
Technical Editor................Edward H. Power, P.E.
Design..HDR Graphics

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Editor — *BridgeLine*
8404 Indian Hills Drive
Omaha, NE 68114-4049
(402) 399-4870
kwall@hdrinc.com

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