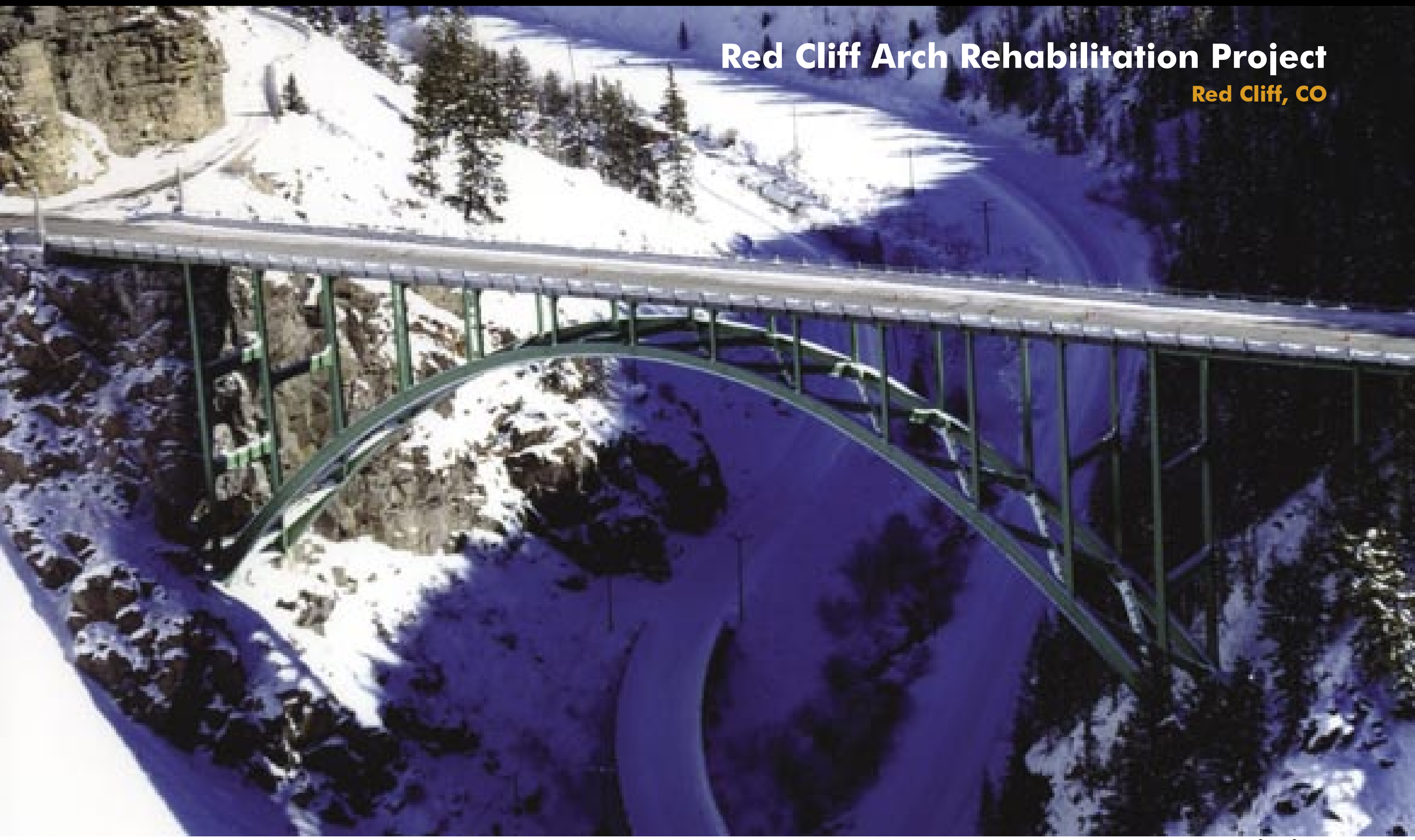
# Red Cliff Arch Rehabilitation Project

**Red Cliff, CO**



Gregg Gargan

Located near Red Cliff, CO, southwest of Vail on U.S. Highway 24, the Red Cliff Arch Bridge was originally completed in 1941. Sixty-three years later, the bridge was in dire need of help. Due to age, as well as loads and traffic that were not envisioned in the 1940s, the bridge needed an extensive rehabilitation.

The newly rehabilitated Red Cliff Arch Bridge was dedicated in November 2004. Preserving the structure's historical integrity, while updating it to current safety standards and strength requirements, was the driving force behind the project.

Safety was also a critical concern during construction. With the bridge 200' above a valley floor, traditional scaffolding was not feasible. The bridge crosses over an existing county road, a river, and a railroad. Any falling material would have been hazardous not only to the public, but also to the environment. A hard platform scaffolding system was required and provided several safety and schedule benefits.

**Owner**
Colorado Department of Transportation, Denver

**Engineer of Record**
Colorado Department of Transportation, Staff Bridge Branch, Denver

**Engineering Software**
SAP2000
Georgia Beam

**General Contractor**
Lawrence Construction Co., Littleton, CO

Since Highway 24 is the main access to this popular ski destination, closure of the highway was a concern. In order to achieve a superior concrete product for the deck, a complete closure for the bridge was deemed necessary. To minimize the impact on the town, the contract required the bridge to be completely closed from April until July. After the first of July, the bridge needed to be open to at least one-way traffic. This closure would provide the least impact to the ski traffic as well as for the summer tourist season. Once available, the second lane could be used for storage of construction equipment for continuing work on the bridge.

The bridge width was increased to 6' in order to increase safety for the increasing bicycle traffic along the route, as well as for regular traffic. The bridge was modeled using finite element modeling software to determine the allowable increase of dead loads.

A SAP computer model was used to determine allowable construction loads as well as removal and placement sequences. It was decided to keep the new dead loads on the bridge the same as or less than the original dead loads. This simplified the design by removing the necessity of checking all the existing connections. A bare concrete deck was used to minimize the dead loads. Because of the widening, the overhang became larger. To minimize the effects of the large overhang, a mandatory construction joint was placed beyond the exterior stringer. The core of the deck was placed first and allowed to gain strength. The curb and remaining deck was then placed with their loads distributed to the newly composite bridge core. To increase strength of the rehabilitated bridge, shear studs were added to stringers to create a composite deck.

Girder flanges ¾" and thicker of ASTM A7 steel were preheated in order to weld the shear studs. The composite strengthening allowed the load restrictions to be removed from the bridge.

Due to the action of the arch, both the removal and replacement sequence of the deck was rigorously controlled. The removal of the existing bridge deck was required to be symmetrical about the center of the arch. The contractor was allowed to be only two spans out of symmetry at any one time. The removal process was aided by the fact that the original deck had not been made composite with the stringers.

Because the stringers were embedded in the deck, the exact conditions of the top flanges were unknown. Ultrasonic testing conducted before construction indicated little section loss. A contingency plan was formulated to deal with any excess corrosion that may have been found on the flanges. Excessive corrosion was not discovered and remediation of the existing steel stringers was minimal. The new deck pour was also controlled to minimize deflections. With the help of the finite element model, acceptable deflections were determined and the pour sequence evaluated. Field measurements during the pours corroborated the computer model's estimates.

Although there had never been any accidents on the bridge, a new safety rail was added to the bridge to protect the public. The original silver painted ornamental rail was stripped, cleaned, and then galvanized. It was then placed on concrete corbels outside of the safety rail. These corbels were added to maintain the original appearance of the bridge.

The steel superstructure required some minor modifications during the rehabilitation. Due to rock fall near the northern abutment, several members had been bent and damaged. These members were heat-straightened prior to being repainted. In order to prevent future damage, the rock face was netted and anchored. The netting is almost invisible and does not affect the visual appearance of the bridge.

The original clip-angle stringer connections were showing signs of distress. Some had already cracked and had been previously repaired. Where space allowed, these connections were made redundant by adding new angle supports. The original support brackets used during construction were removed in order to install the new support angles. Where space or connection details precluded the use of the new support angles, the original A7 steel angles were replaced.

Gregg Gargan

Unfortunately, the connections of the columns to the arch also proved to be conducive to corrosion. Pockets of debris formed on the upper side of the connection which exacerbated the corrosion potential. In an effort to minimize the problem, larger holes were added to allow the debris to pass. This also allowed construction personnel to clean out accumulated debris due to the crumbling deck from the interior of the columns.

Due to water draining down the arches, corrosion on the concrete piers was also a continuing problem. The piers were rehabilitated during this project to replace the deteriorated concrete. Flow diverters were added at the base of the arches to minimize the future damage.

Forty percent of the project cost included removing the old paint system and repainting the bridge. Due to the age of the bridge, lead-based paint was assumed to be present. A containment system was required on the bridge for the complete capture, containment, and collection of all coating debris, spent abrasives, and dust. The new coating consisted of a three-ply paint system, an organic zinc primer, an epoxy intermediate coat, and an aliphatic polyurethane top coat. ★

# NATIONAL AWARD

LONG SPAN

## Appalachian Corridor H over Clifford Hollow

**Hardy County, WV**

Courtesy HDR Engineering.

**Owner**
West Virginia DOT

**Engineer of Record**
HDR Engineering, Inc., Pittsburgh

**Engineering Software**
BSDI 3D
GT STRUDL
STAAD.3

**Fabricator and Detailer**
Stupp Bridge Company, St. Louis, NSBA/AISC member

**Detailing Software**
AutoCAD

**General Contractor**
Dick Corporation, Pittsburgh

Tom Kessler

Clifford Hollow Bridge crosses Clifford Hollow, a deep valley adjacent to Route 55 on Appalachian Corridor H, a West Virginia Department of Transportation (WVDOT) highway development corridor. The six-span steel plate girder bridge crosses the hollow on a tangent alignment and carries two lanes of traffic in each direction. It is approximately 464 m (1,522') long from abutment to abutment and its deck is approximately 85 m (280') above the floor of the hollow.

### Superstructure Design

The bridge features six spans (one at 64 m [210'], four at 84 m [276'], and another at 64 m). It has a deck width of 75.3' and a permanent median barrier in the center. The deck is 220 mm (8.5") thick using 31 MPa (4500 psi) concrete with 420 MPa (60 ksi) epoxy-coated reinforcing steel, with a 40 mm (1.5") latex modified concrete overlay for the basic deck. A four-girder, three-substringer system with spacings of 6.8 m (22') between the main girders and deck overhangs of 1.275 m (4.2') was the most economical system of framing.

The main girders were designed composite with the deck and detailed with longitudinally stiffened webs 3,350 mm (11') deep. Most of the steel was Grade 345W (50W). However, HPS 485W (HPS 70W) steel was used in the field pieces over the interior piers, taking advantage of the higher strength steel in an area where the stress ranges are relatively low. The HPS 485W steel minimized the piece weights, making the design more efficient and erection easier. W760×147 (W30×99) rolled shape substringers were designed as non-composite and are supported by truss-type floor beams spanning between the main girders.

Laminated neoprene bearings were used at the expansion bearings. A PTFE sliding surface was used on the top of the laminated neoprene pads so that the movement capacity did not rely on deformation of the bearings. This design was made possible through the use of AASHTO Design Method B, which permits higher compressive stresses.

### Substructure Design

Hammerhead piers with hollow columns were used. The three center piers are fixed; therefore, they share externally applied longitudinal forces proportionally based on their relative stiffnesses. The pier caps were post-tensioned to provide the necessary strength. However, enough mild reinforcing was provided in the caps so the girders could be completely erected prior to post-tensioning. This gave the contractor more freedom in scheduling construction activities and reduced the amount of post-tensioning required.

The piers were founded on spread footings on rock. Due to stability concerns, rock anchors were provided in front of Abutment 1 and Pier 1. The anchors cross the assumed sliding surface between rock layers, providing a greater resistance to sliding.

## Design Analysis

The structure was designed using refined methods of analysis for the superstructure. These refined analyses provided load distributions in the superstructure based on the actual structural stiffness so that the structures could resist the applied loads efficiently. The refinement of the loads allowed a more efficient design: material was placed where it was actually required based on analysis, rather than on simplified assumptions. This also gave greater certainty that the structural demands would be met by the design.

A system design approach was applied to the project. The entire bridge was analyzed using the general analysis programs STAAD.3 and GT STRUDL. A deflection management system was incorporated into the bridge to minimize the global magnitude of deflections. Limiting the longitudinal deflections accomplished several important goals. The pier design loads were reduced as the longitudinal movement demands were reduced, resulting in an efficient pier design. Also, the global structure deflections could be accurately assessed, resulting in a more serviceable bridge. In this case, the bridge analysis identified that even though the deflection was limited, the expansion joints needed to have movement capacities much larger than would have been required by merely assessing thermal movement and superstructure rotation.

Courtesy HDR Engineering.

## Materials

The steel plate girders featured a significant amount of HPS-70W high performance steel. This recently developed steel has higher yield strength than commonly used bridge steels. In addition, the fracture toughness of the HPS-70W steel is superior to normal bridge steels, thus significantly reduces the likelihood of fatigue cracking in the future. High strength concrete was also used in the piers, allowing the use of reasonable pier component sizes and keeping the reinforcement to a manageable level.

To be consistent with all bridges on Corridor H, form liners on the substructures and outside faces of parapets were used to improve the appearance of the structures. In addition to form liners, both the pier shapes and the limits of the form liners used on the piers were carefully assessed to optimize the overall appearance of the bridge. Hammerhead piers were chosen due to the bridge's extreme height. The columns fit proportionally with this height. Also, the pier width in the longitudinal direction tapers from the top to the bottom of the piers. The base of the pier columns is wider and appears reasonably stout to support a structure that size. Finally, the HPS 70W weathering steel was not painted, creating a look that blends with the rustic environment and that will minimize future maintenance.

## Erection

Many of the bridge's design details were chosen to facilitate erection in the difficult terrain. Girder field section lengths were detailed to a maximum length of 36 m (120') to ensure that shipping would be feasible. Limited piece lengths eased erection of the structure because they were easier to handle and more stable during lifting. Given the extreme height of the structure and the long spans, lateral bracing was included in the structure's center bay to stiffen it against lateral wind loads prior to closure of the girders.

The contractor chose to launch the superstructure. Modifications to the girder designs were necessary to accommodate the launching, though the basic framing system was not changed. The steel weight was increased by about 600,000 lb to accommodate launching; however, web longitudinal stiffeners and many web transverse stiffeners were eliminated in the redesign, simplifying the fabrication. Lateral bracing was added at the top of the girder webs as well as the bottom in the leading span for the launch to stiffen the framing adequately and to accomplish the launch safely. A kingpost system with temporary stay cables was used to limit deflection of the nose. A light launching nose was also included to help guide the girders back onto the piers.

The girder framing was essentially performed on the ground, rather than high in the air. The overhang brackets for the deck forming were also installed prior to launching. The project demonstrated that a successful and safe launch of a steel girder is possible even in a sag vertical curve. ★

NATIONAL AWARD

SPECIAL AWARD WINNER

# Arthur J. Ravenel, Jr. Bridge over Cooper River

## Charleston, SC

By Michael J. Abrahams, P.E.

The dramatic Cooper River Bridge—the longest cable-stayed bridge in North America—opened in July 2005 a year ahead of schedule, saving the South Carolina Department of Transportation (SCDOT) an estimated $150 million. A week of festivities, including a fireworks display, an on-deck performance by the Charleston Symphony Orchestra, and a walk across the bridge by more than 100,000 people, celebrated this bridge dedicated to South Carolina State Senator Arthur Ravenel, Jr.

The approximately three-mile-long bridge, including the main span, high level approaches, ramps and interchanges, was designed and constructed in a four year period. The bridge's main span allows for both a widening of the navigation channel to 1,000' and a deepening of the dredged depth of 10' to accommodate larger shipping vessels.

The project, which is the largest single transportation infrastructure project in the state's history, evolved from studies that began in 1988 to address the need to replace the deficient 1929 John P. Grace Memorial and 1966 Silas N. Pearman bridges between Charleston and Mount Pleasant. Also, there was a more recent need to improve shipping clearances in the upper reaches of Charleston Harbor.

Photo by Anthony Winn.

**Owner**
South Carolina Department of Transportation

**Architect**
MacDonald Architects, San Francisco

**Engineer of Record**
Parsons Brinckerhoff Quade & Douglas, Inc., New York

**Engineering Software**
ADINA, STRUDL

**Detailers**
Tensor Engineering Company, Indian Harbour Beach, FL, NSBA/AISC member
Carolina Steel Corporation, Greensboro, NC, NSBA/AISC member

**Fabricators**
Carolina Steel Corporation, Greensboro, NC, NSBA/AISC member
High Steel Structures, Inc., Lancaster, PA, NSBA/AISC member

**General Contractor**
Palmetto Bridge Constructors, Charleston, SC

After completion of the final environmental impact statement in 1998, SCDOT selected a design team to prepare preliminary plans, anticipating that the project would follow the traditional design-bid-build process. But rising estimated construction costs and limited available funding prompted SCDOT to consider alternative means to accomplish the much-needed replacement.

Based on SCDOT's prior successes with other design-build projects, they decided to use the preliminary design that had already been completed to develop a design-build approach. Since SCDOT had already done much work in developing the project alignment, geotechnical information, and design criteria as part of the preliminary design, it had considerable information to provide the design-build teams. As a result, much of the preliminary design effort proved to be useful in advancing the project though the design-build process.

SCDOT used a two-phase approach in soliciting design-build proposals. Phase one was advertised in July 2000 and invited each design-build team to submit its qualifications and a non-binding proposal for a new eight-lane crossing. Phase two, which was advertised in February 2001, required each design-build team to submit fixed-price bids for a number of options including a four and an eight-lane crossing, as well as add-ons for a sidewalk, a transit lane, and additional access ramps. Three teams competed, and the low bid of $531 million for the selected option of an eight-lane crossing including a sidewalk and additional ramps was submitted by Palmetto Bridge Constructors.

The new cable-stayed span has a 1,546' main span, two 650' side spans, and two 225' anchor spans for a total suspended span length of 3,296'. The main span utilizes a composite concrete deck with I-shaped steel edge girders and floor beams. The high-level approaches also use composite steel construction with steel girders spaced 12' on center. Both high-level approaches are jointless over their full length: 4,351' on the Charleston side and 2,090' on the Mount Pleasant side. Beyond the high approach spans are the low-level approach and interchange structures. These structures use composite precast, post-tensioned Bulb Tee concrete girders for the straight portions and composite steel girders for all of the curved ramps.

The project design criteria called for a 100-year service life, and it was left to the design-build team to meet this requirement. By working closely with local concrete suppliers and recent advances in service life predictive techniques, the use of low permeability concrete combined with uncoated reinforcing steel was demonstrated to provide the required service life.

The foundation bearing stratum over the entire site is characterized by a thick layer of stiff silt and clay, known as Cooper Marl, at a depth of 40' to 75' below elevation 0.0 (approximate MWL). Above the marl, the river has soft alluvial deposits. The land portions of the project have soft surficial soils inter-layered with loose, sandy material that is potentially liquefiable during an earthquake. These soil conditions made the structural design of the foundations quite challenging. In effect, the structure that the engineers had to design extended down to the Cooper Marl: piers with clear column heights of up to 146' above ground effectively extended an additional 40' to 75' below ground.

The environmental conditions in Charleston are probably among the most challenging in the United States due to the occurrence of both hurricanes and earthquakes. The design Safety Evaluation Earthquake (SEE) is a 2,500-year return period event with seismic shaking intensity similar to that found in parts of California. In addition, the South Carolina coast is an area prone to hurricanes and the associated strong winds. Ship collision loads were also a major factor for design of the main span piers and the piers adjacent to a creek on the Charleston high-level approach.

The superstructure for the suspended main span and side spans, from Pier 2W to Pier 2E, consists of two 6'-6" deep steel I-shaped edge girders and steel floor beams at 15'-8" spacing composite with a 9½" concrete deck slab. The deck is comprised of 8,000 psi precast panels, with closure strips over the girders and floor beams, and a 2" latex modified concrete wearing surface. The 126'-wide bridge deck carries eight lanes of traffic, and a 12'-wide pedestrian walkway and bikeway is cantilevered outside of the south edge girder.

The edge girders were designed for an HS25 live load, following load factor design (Strength Design Method) provisions of the AASHTO Standard Specifications and the FHWA Proposed Design Specifications for Steel Box Girder Bridges. The girders were also designed for the stay cable replacement and stay cable loss cases, following the AASHTO LRFD specifications and the fourth edition of the PTI Recommendations for Stay-Cable Design, Testing and Installation.

Replacement and cable loss load combinations in the PTI manual dominated the edge girder design. The cable replacement load case is probably not relevant for newer stay cable bridges that have individually sheathed strands and that utilize monostrand (isotension) jacking systems. It also seems to be highly unlikely that there could be a sudden, instantaneous failure of an entire stay cable on a newer bridge like this, with individually sheathed strands and ungrouted cables. Even if this were to happen, an evaluation allowing plastic hinging of the edge girder would probably be appropriate.

Except for a limited amount of Grade 70 steel in the edge girders, the main span features Grade 50 steel throughout. The unit weight of the main span steel is 42.4 psf, not including the stay cables or tower anchorages.

The high-level approaches are composite Grade 50 structural steel with a unit weight of 46.4 psf. The girders are 8'-2" deep. Typical center to center girder spacing is 12'- 0". Typical spans are 250'-0". The approaches have an overall width of 128'-10" and carry eight lanes of traffic together with a 12'-0"-wide sidewalk.

The curved steel ramps are also composite Grade 50 structural steel with an average unit weight of 60.7 psf. The girders are 6' deep, and there are typically four girders carrying each ramp. The typical ramp is 36' wide and carries two lanes of traffic. One ramp, which carries one lane of traffic and a 12'-0" wide sidewalk, is 28' wide. The ramps are continuous and vary in overall length from 1,400' to 1,800', with typical spans ranging from 160' to 240'.


David Sailors

The contractor selected two fabricators for the structural steel work: AISC member High Steel Structures for the main span and high-level approaches; and AISC member Carolina Steel Corporation for the curved steel ramps. As this was a design-build process, the development of design details on the plans included input from the fabricators so that their preferences and requirements could be incorporated into the final plans. In a typical design-bid-build process, this interaction does not occur until after completion of the final plans and award of the contract, and under those conditions may be hindered by time constraints and contractual issues. The contractor self-performed all steel erection.

All plans were made available to the contractor and the fabricators on project FTP sites. Then, in turn, the fabricators' shop drawings were posted on their own FTP site so they could be downloaded and reviewed by the designers. While consideration was given to electronic transmittal of marked-up shop drawings, this method was not used on this project. It was judged that given the team's limited experience with this methodology, the large number of shop drawings, the large number of structures and reviewers, as well as the tight schedule, a paper trail of marked up drawings would provide far better means for ensuring that all shop drawings and comments could be tracked. All FTP postings and shop drawing transmittals were announced and confirmed by e-mail so that the status of all design and shop drawings was available at all times to the entire design-build team. ★

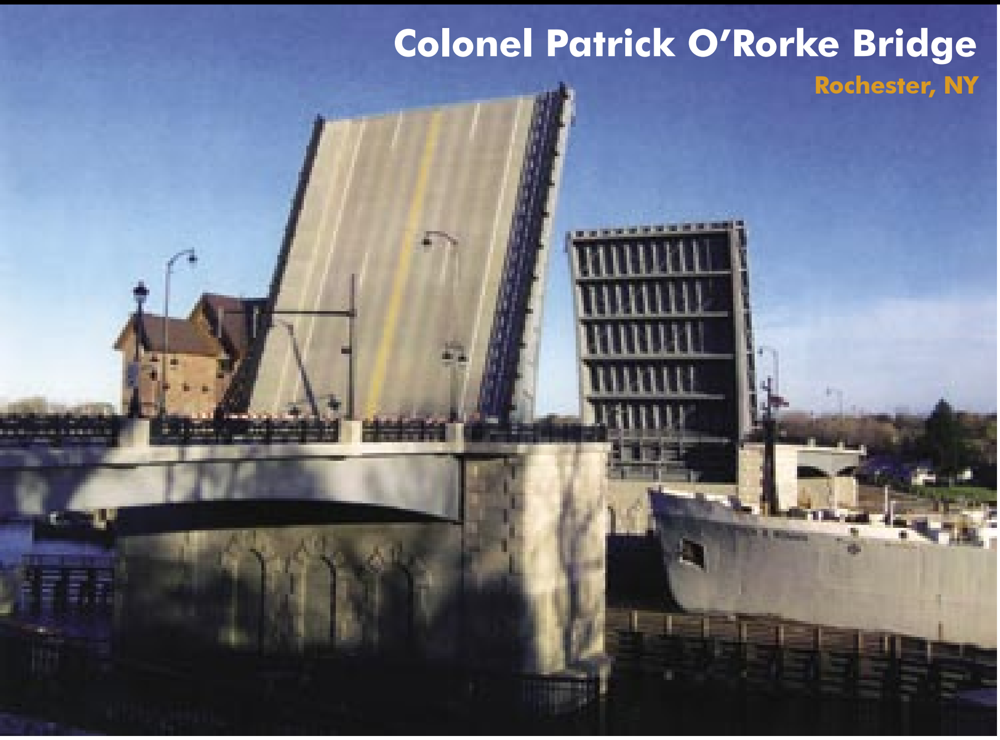
*Michael J. Abrahams is Senior Vice President and Technical Director of Major Bridges for Parsons Brinckerhoff Quade & Douglas, Inc. in New York.*

# NATIONAL AWARD

MOVABLE SPAN

## Colonel Patrick O'Rorke Bridge

**Rochester, NY**



Bergmann Associates

Bergmann Associates

**Owner**
New York State Department of Transportation, Rochester, NY

**Engineer of Record**
Bergmann Associates, Rochester, NY

**Engineering Software**
DESCUS I, STAAD, Merlin-DASH, FB-Pier, SEISAB

**Detailer**
Tensor Engineering, Indian Harbour Beach, FL, NSBA/AISC member, NISD member

**General Contractor**
Crane-Hogan Structural Systems, Inc., Spencerport, NY

After nearly a decade of planning and engineering and four years of construction, the Colonel Patrick O'Rorke Bridge, a replacement bridge for a 72-year-old bascule bridge, was opened to traffic on October 1, 2004. The $104 million bridge, which is located six miles north of Rochester, NY at the confluence of the Genesee River and Lake Ontario, was named in homage to a Rochester native and Civil War hero.

The new bridge consists of a 243'-long bascule span, which is flanked by a 148'-long single-span steel girder approach structure on the west side and a 530'-long, three-span steel girder approach structure on the east side. The bridge, constructed some 250' upstream of the previous bridge, is 22' higher than its predecessor and carries four 11'-wide vehicle lanes, two 5'-wide bicycle lanes, and oversized 7'-wide sidewalks.

Steel was chosen for the primary structural members due to its strength, the depth and length requirements of the main spans, and its ability to easily conform to the shapes needed for aesthetics. The project included more than 2,600 tons of structural steel, five miles of H and pipe piles, and 1.8 million pounds of reinforcement.

### Bascule Span

The centerpiece of the O'Rorke Bridge project is a double leaf, Scherzer rolling lift bascule span. Designers selected the rolling lift arrangement because it provided the necessary clear channel opening with approximately 10% less length in the bascule leaf when compared to a trunnion-type bascule bridge.

Steel framing for each leaf consists of two 13'-deep bascule girders with floor beams and stringers supporting a concrete filled steel grating deck. More than 850 tons of structural steel were used for the two bascule leaves alone. In order to minimize the power requirements needed to operate the bridge, the lift spans are counterweighted to produce a balanced condition. The bridge features a closed concrete deck system and requires very large concrete counterweights, each weighing over 590 tons. Heavy structural steel truss frames embedded inside the concrete counterweights transfer the counterweight load to the bascule girders.

Balancing the O'Rorke Bridge lift spans presented a unique challenge. Both the vertical and horizontal location of the center of gravity of the bascule span had to be accurately determined so that the span would remain balanced throughout the full range of movement. Throughout steel fabrication, erection of the bascule steel framing, and concrete counterweight placement, the engineers performed detailed balance computations to account for the exact weight and center of gravity of every component. Additionally, the engineers used strain gage instrumentation to aid in the balancing process. Even with these precise computations, the engineers provided for pockets cast inside the concrete counterweights in which small, concrete balance blocks could be added or subtracted to fine tune the balance.

## Approach Structures

Approach structures flank the bascule span on both the east and west sides. On the east side, geotechnical investigations revealed seams of organic materials adjacent to the river channel. Designers chose to span this area with a 530'-long, three-span approach structure founded on steel H-pile foundations to alleviate concerns regarding long-term settlement.

To simplify detailing, fabrication, and erection of the eight steel plate girders that make up this structure, the designers specified radial substructures with each girder being located on combined tangent and compound curve alignment. This eliminated the spiral on the girders but resulted in a variable deck overhang. The compound curves were non-concentric, resulting in a splayed configuration to accommodate the turning lane. The unique geometry required a combination of both X-type and K-type cross-frames (280 total).

A varying depth web on the fascia girders only (from 7' to 13') was chosen for aesthetics to mimic the haunches on the bascule spans and required special cross-frame and lateral bracing connection details, as the interior girders were a constant 7' in depth.

The fascia girders also serve as the live load uplift restraint mechanism for the bascule girders. In order to resist the large, 430 kip uplift forces generated by the bascule span, each fascia girder on the east approach structure is cantilevered 10' into the bascule pier and is detailed to mate with the tail of the bascule girders. The fascia girders are outfitted with specially designed uplift bearings with two 4"-diameter, high-strength steel anchor rods that are embedded over 30' into the wall of the bascule piers.

## Bascule Pier Foundations

There is a significant variation in site geology from one side of the river to the other, requiring different foundation systems at each pier. Due to the depth of overburden materials, pile-supported foundations were selected for the eastern bascule pier, which is founded on 114 HP14×117 steel piles, with an average installed length of approximately 100'. A spread footing was selected for the western bascule pier, which is founded on a series of 24 level rock benches, each approximately 12' by 20' with 3' steps between.

The installed foundations at the western bascule pier consist of a 3 by 8 grid of 53"-diameter shafts with permanent steel casings and 48"-diameter rock sockets. The drilled shaft lengths vary from 60' at the deepest part of the foundation footprint to 30' at the shallowest.

## Shop and Field Erection

The fabricator and designers worked together to develop and execute a successful numerical roll-through procedure using a system of precise computer controlled milling and measurements of the various components. A field roll-through of the heel sections of the bascule girders was also performed on site to verify the alignment of the track castings and rack girder support frames embedded in the bascule pier concrete.

Bergmann Associates

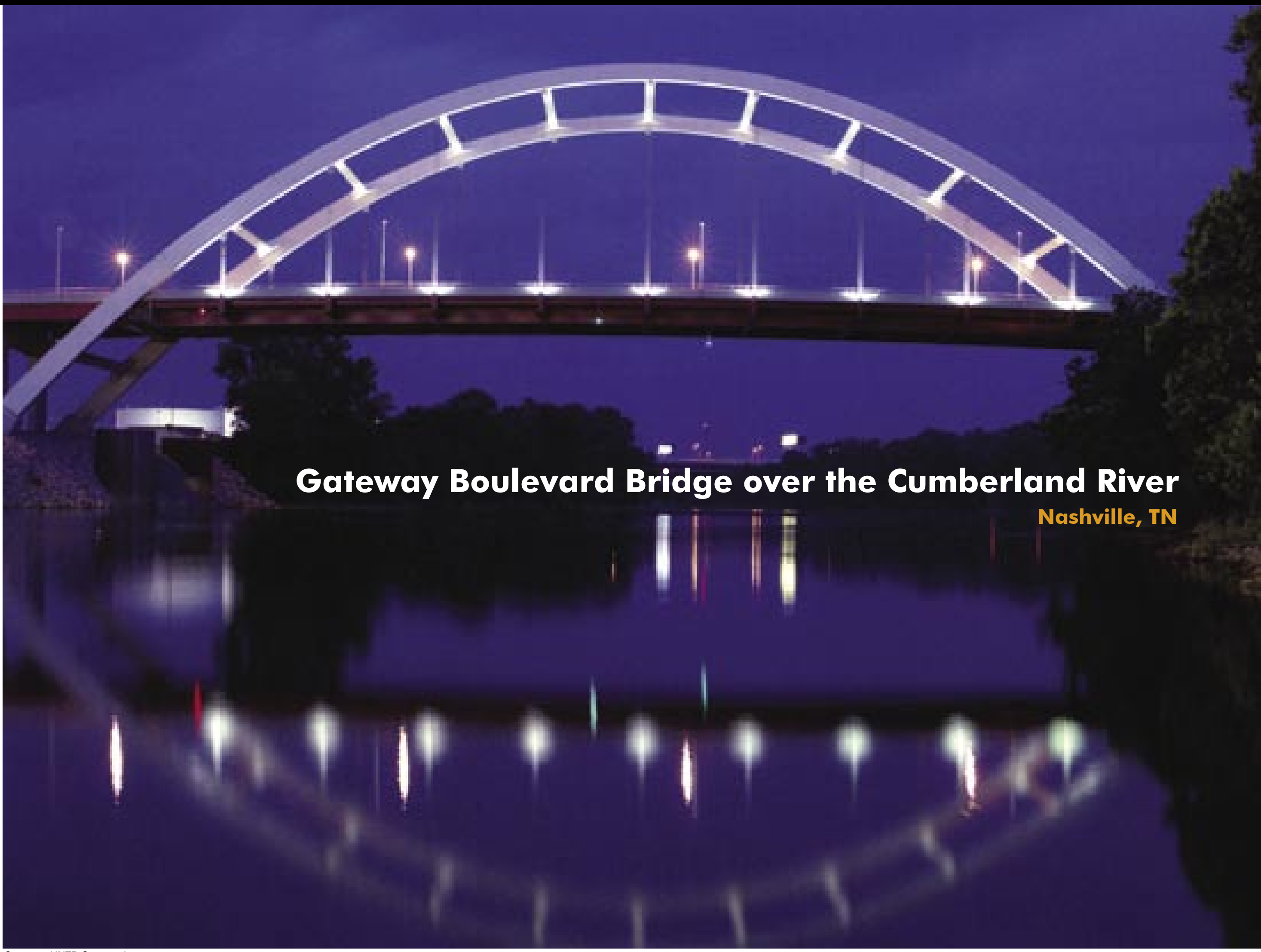
Contract provisions prohibited construction activities that blocked the navigation channel for extended periods of time. Rather than erecting the steel from a barge in the middle of the channel, the project team opted for a non-conventional erection plan involving placement of a 500 ton hydraulic crane on the 9"-thick concrete deck of the flanking approach structures when it came time to erect the structural steel for the toe end of the bascule leaves. The lift plan resulted in concentrated outrigger loads of more than 200 kips being transferred to the bridge deck, framing, and supporting piers. The designers and contractors worked together to develop a lift plan using a combination of temporary shoring and heavy steel grillages under the crane outriggers. Stresses on the supporting framing were minimized and no damage occurred to the bridge deck.

## Isolation Bearings

Because the flanking approach structure girders serve as the live load restrainers, the approach structure bearings needed to be fixed at the bascule pier. To minimize the seismic forces on the bascule pier, the designers specified innovative fused lead-core rubber isolation bearings at this location. The bearings feature an elastic restraint system (ERS) comprised of overlapping plates attached to the sole plate and masonry plates, which are then fastened together with fuse bolts. Under service loads, the ERS allows the bearing to function as a fixed bearing, preventing longitudinal translation of the structure. Higher forces during a seismic event will shear-off the fuse bolts allowing the eastern approach structure to "float" on the rubber bearings, with the lead core absorbing the earthquake energy. ★

# NATIONAL AWARD

MAJOR SPAN



Courtesy HNTB Corporation.

## Gateway Boulevard Bridge over the Cumberland River

**Nashville, TN**

**Owner**
Tennessee DOT, Nashville, TN

**Architect and Engineer of Record**
HNTB Corporation, Kansas City, MO

**Detailer**
Tensor Engineering, Indian Harbour Beach, FL, NSBA/AISC member, NISD member

**Erector**
American Bridge Co., Coraopolis, PA, NEA member

**General Contractor**
Ray Bell Construction Co., Inc., Brentwood, TN

The Gateway Boulevard Bridge in Nashville connects the eastern part of the city with its revitalized downtown business and entertainment district. This $31.5 million through-arch bridge totals more than a quarter mile in length—1,660' from end to end—with approximately 699' comprised of steel structure.

The main river span is a 545'-long steel through-arch bridge spanning the Cumberland River. The east end approach consists of 961' of 72" bulb tee girders arranged in nine spans. The continuous approach span is integral at the abutment. All thermal expansion is accomplished at a single expansion joint located at the interface with the transition span prior to the arch unit.

The west end approach consists of a 128'-long curved steel plate girder. Additionally, the main arch deck span is buffered by two transition spans made of 59'-long steel plate girders.

### Arch Ribs

The arch ribs are steel box girders, over 6' deep, 3' wide, and 638' long, with lateral support provided by steel box Vierendeel struts. The arch ribs are anchored to concrete abutments, resisting the total arch thrust, with 4"-thick base plates attached by 55 1 "-diameter post-tensioned rods. The post-tensioning process required tensioning each rod with a minimum of 142,000 lb and a minimum final elongation of ½". The base plate design required a 4"-thick steel plate shop welded to the arch rib with pre-drilled holes for the post-tensioning anchors.

Due to the through-arch design, the steel arch ribs not only have a large vertical load, but also tremendous horizontal thrust. The site is characterized as having a high limestone bluff on the west side of the river and relatively deep rock on the east side. The design had to determine the optimum configuration of the abutment stem and footing while providing adequate resistance from the fractured rock. On the east side, the contractor had to excavate well below the water table to achieve proper founding of the arch abutments. The top of the east arch abutment footings were built approximately 65' below existing grade and a minimum of 5' below the top of the rock. The bottom of the 16½' by 46'-long footings are 21½' below the top of the rock.

Courtesy HNTB Corporation.

The east arch abutment footings were constructed below the bottom of the sheet piles without a concrete seal. To provide the necessary foundation, the footings needed to be "socketed" into rock. It took tremendous skill to properly seat and seal the steel cofferdam sheets, thereby allowing the footings to be constructed in relatively dry conditions.

The ribs were erected working from both arch abutments, using steel towers and cable stays to hold the erected pieces in place. The members were erected using cranes on barges in the river. Only while the center three sections of arch rib were lifted into place was barge traffic stopped. These three sections totaled 115', weighed 95 tons, and were lifted into place more than 190' above the center of the river. To accomplish the proposed erection sequence, the contractor used a 4100 ringer crane with 280' of boom mounted across three barges.

The post-tensioned anchors at all arch abutments were grouted after tensioning to provide protection from corrosion and other environmental attacks. During the construction sequence, some of the grouting tubes for the rods were damaged. Due to concern about not being able to adequately blow out the tubes, it was decided to use a vacuum grouting procedure on each tube. This procedure provided a much more thorough cleaning and filling of the grout tubes, thus providing better protection for the post-tensioning rods. This is the one of the first instances where a vacuum grouting procedure has been used.

## Arch Deck

The arch deck has 49' of clearance above regulated high water and sits 79' above the normal pool stage for the river. The top of the arch ribs is 98' above the roadway. The arch deck is 453' long and is supported by 72 2"-diameter high-strength steel cables. The arch deck is a steel girder floor system made of nine transverse floor beams with two longitudinal stiffening girders and 11 stringers framed into them. The deck system is anchored at each end by steel box end struts where the transition span girders and floor system tie in. The connection to the west end strut is a pinned connection bolted to the web, while the connection to the east end strut has a pot bearing expansion device, capable of 8" of longitudinal and 1" of transverse movement. The floor system is topped with a concrete deck made composite using field welded shear studs.

The three additional steel spans include two transition spans and a curved span. The transition spans are simple spans, 59' in length, made of steel plate girders. They attach to the end struts with a pinned connection bolted to the web of the end strut. The west approach, or span one, is a 72"-deep steel plate girder with a 8˚-19'-00" degree of curvature and a minimum radius of 643'.

At 102' out-to-out, the extremely wide bridge deck geometry created challenges in placing the concrete deck itself. The contractor determined that it was not feasible to place concrete continuously in the transverse direction, instead recommending a longitudinal construction joint down the center of the arch deck. Because of the construction joint, deflections in the steel arch ribs, floor beams, and other floor system members had to be recalculated. The resulting stretch in the hanger cables subjected to unbalanced construction loading also had to be considered. ★

# NATIONAL AWARD

SHORT SPAN

## Germantown Avenue Bridge over Wissahickon Creek

**Philadelphia**

URS Corporation

The Germantown Avenue Bridge over Wissahickon Creek in Philadelphia is a three span curved steel girder bridge that was designed to safely replace a failing nine span straight bridge.

The location of the old bridge forced a hazardous curve, allowing a maximum speed of only 15 mph. In 1997, several undermined piers cracked and, after emergency repairs, vehicular traffic was restricted to one-way next to the downstream edge of the deck. Structurally, the bridge was rated "poor" even before the piers cracked. Collapse or removal of the sidewalk overhangs required the addition of traffic barriers that restricted roadway width and further reduced safe operating speed.

**Owner**
City of Philadelphia Department of Streets, Bridge Division

**Engineer of Record**
URS Corporation, Fort Washington, PA

**Engineering Software**
BSDI, ABUT5, BAR7, SEISAB

**Detailer**
Steelcon, Inc., Cordova, TN, AISC member, NISD member

**Detailing Software**
AutoCAD

**Fabricator and Erector**
High Steel Structures, Inc., NSBA/AISC member

**General Contractor**
Neshaminy Constructors, Inc., Feasterville, PA

A new, horizontally curved roadway alignment replaced the existing sharp bend to improve roadway safety. The resulting baseline radius of about 488' required the curved superstructure.

The 14,000 cfs flow of Wissahickon Creek was also severely constricted by this bridge's eight original stone piers, which created narrow hydraulic openings of approximately 18'. Upstream flooding occurred when one or more of the spans clogged with debris. Increased stream flow velocity in the remaining spans caused major scour beneath the foundations, which required periodic repairs.

The number of spans for the new bridge was limited to three to improve the stream flow underneath, and new abutments were placed beyond the existing abutments. The combination of the curved alignment and the positioning of the piers and abutments parallel to the stream caused skew angles as low as 50° between the bearing lines and baseline. A relatively shallow superstructure was proposed to keep the bridge above the 100-year storm water surface elevation.

A refined method of analysis was performed on the curved steel girders and radial positioned diaphragms to determine design forces. Initial results indicated very large diaphragm forces. Rather than proposing massive diaphragms, the engineers removed the diaphragms located closest to the piers and abutments. Re-analysis results indicated lower diaphragm forces while still providing adequate lateral bracing for the girder compression flanges.

### Aesthetic Considerations

Two separate superstructures were designed for sidewalks. The upstream sidewalk and roadway lanes were placed on a constant width curved deck. The downstream sidewalk was located on a variable width chorded deck with a curved upstream edge to match the downstream edge of the vehicular deck for two of the three spans. The engineers were able to successfully design each structure, given the significant curved beam requirements, through the use of fabricated structural steel to support each deck.

The owners wanted a pedestrian superstructure deck that was durable, aesthetically pleasing, and capable of being quickly replaced if damaged. Wood planking over steel grid flooring met these requirements. The steel grid was easily attached to the supporting steel beams. The wood deck was assembled in panels and bolted to the steel grid, which allowed rapid partial replacement of deck areas if needed.

The owners also wanted building materials that would look pleasing but require minimal maintenance. All visible structural steel was protected with a three-coat epoxy paint system pigmented to match the surrounding green foliage. Non-visible structural steel was galvanized. All cast in place concrete was textured or stone-lined and pigmented.

Excessive lateral movement was detected when an external horizontal movement was introduced after erection of the steel girders for the pedestrian bridge. It was believed that the installation of the steel grid and wood deck would stabilize the structure, but methods to reduce the movement were also considered as a precaution. The designers suggested that a "soft-tie," similar to a steering strut on an automobile, be placed between the outside vehicular bridge steel girder and the inside pedestrian bridge steel girder. The intent was to provide lateral restraint to the pedestrian bridge without transmitting live load deflections or vibrations from the vehicular bridge into the pedestrian bridge. A detail using steel rods and fabric pad washers and bushings was developed. The concept was installed before the steel grid and wood deck were attached to the girders. After completion, the pedestrian bridge exhibited no perceptible lateral movement due to external force and no perceptible vibration during car or truck passage on the vehicular bridge.

The engineers elected to replicate the original bridge's railing. Although both of the original railings had been lost during failure of the sidewalks, the city had detailed drawings of them. These drawings enabled a copy for use on the new structure. A shorter railing was used on top of the shaped safety barriers to protect bicycle riders from falling.

Realignment of the roadway necessitated the removal of one of two driveways to a nearby college. The college requested that the remaining entrance be widened and improved with a new traffic signal. The wider entrance would allow construction of a future guard house between the incoming and outgoing lanes. The existing entrance was flanked by a 15'-long by 5'-wide by 6'-tall pilaster on each side. The engineers proposed that one of the pilasters be carefully disassembled and reconstructed to provide the wider entrance. However, after excavation around the pilaster was completed, it was decided to roll the entire pilaster into its proposed location. This was performed successfully and the architectural integrity of the structure was maintained.

The new roadway was designed for a 35 mph travel speed. Lane widths were reduced to only 11' to encourage slower travel speeds. Superelevation was limited to 2%, and shoulder and bikeway rumble strips confine motorists within the cartway. The bridge provides hydraulic openings of at least 75'. A separate, adjacent, downstream structure provides a wider, wood-decked pedestrian crossing. Use of real and formed stone surfaces on parapets, traffic barriers, and substructures helps integrate the bridge into its surroundings. ★

# NATIONAL AWARD

SPECIAL PURPOSE


Rosales Gottemoeller & Associates, Inc.

The new Liberty Pedestrian Bridge over the Reedy River in Greenville, SC has become one of the defining landmarks of the city's downtown. The 340'-long bridge is unique in its use of a single suspension cable attached to only one side of a curved alignment plan. Its discrete cable pattern allows unhindered views of a park and waterfall.

Greenville's downtown is divided by a wooded valley that contains numerous trails and recreational areas. Until recently, a high-level vehicular bridge crossed over the river, obscuring the beauty of the waterfall to motorists and pedestrians. The city decided to demolish the existing bridge and replace it with this unique pedestrian bridge.

One of the main constraints on the site was the location of the falls in relation to the entrances into the park. The new bridge is curved in plan to establish the link between the two side entrances. The curved alignment relates better to the river, paths, and hills, which are also curvilinear in nature so that the bridge fits into the landscape. Also, the curved alignment made it possible to have an unusual and visually unique cable configuration.

**Owner**
City of Greenville, SC

**Architect and Engineers of Record**
Rosales Gottemoeller & Associates, Inc., Boston

**Engineering Software**
SOFiSTiK

**Detailer and Fabricator**
Dave Steel Company, Inc., Asheville, NC, AISC member

**General Contractor**
Taylor & Murphy Construction Co., Inc., Asheville, NC

## Aesthetic Considerations

The circular bridge is supported by a single suspension cable along the outer side of the curve, with two inclined towers. Locating the cable supports on the outer edge of the curvature created an amphitheater effect oriented toward the falls. The towers that support the main suspension cable are also located on the outer side and angled in profile to further emphasize the directionality of the views. The inner side of the curvature does not have visible means of support. The views along the bridge constantly change due to the curvature and the 3.5% slope of the bridge.

The clear span between the towers over the river and falls is approximately 200'. Thirty-two thin suspender cables attach to the outer ends of 32 radial light steel transverse frames, 4' in depth, which in turn support the 12'-wide, 7"-deep concrete deck.

The slender towers are approximately 100' high and are stabilized with single sloped backstay cables. In order to resolve the torsion forces created by the one-sided suspension of the deck frames, a series of ring cables connect all the steel frames below deck level.

## Structural Considerations

The bridge's alignment in plan allowed for the use of a structural system that required only a single-sided cable supported structure. The bridge has a ring girder supported at the outside edge, with compression occurring at the top of the section and tension at the bottom.

Cables easily support the tension at the bottom of the girder, while the deck slab supports the compression force at the top of the girder. Observing the ring forces in plan provided the best overview of the principle of the ring girder. The tension force at the underside of the girder produces a component force directed towards the center of the curvature, while the compression force produces an outwardly directed force component. The resulting pair of forces $u \times h$ with $u = C/R$ is in equilibrium with the pair of forces $P \times w/2$, with $P + p \times w$ and $p$ = loading of the bridge.

The deck needed a vertical support to be in equilibrium. This vertical support was formed by the hanger cables. Because of the geometry of the cable structure, there are also horizontal forces due to the inclination of the hangers. The horizontal forces cause a transverse bending moment that has no relation to the behavior previously explained. To provide the deck with additional stiffness for non-uniform distributed loads, a truss girder was formed in the plane between the hanger connections and the ring cables.

## Bridge Construction

In the initial construction phase, the main steel components were installed and the deck slab was constructed on site while being supported by temporary scaffolding. Prefabricated steel components were laid out in the required geometry and welded together. The bridge ribs consisted of steel plates at each cable axis supporting the diagonals of the truss


Rosales Gottemoeller & Associates, Inc.

and the corner profiles at the edges. After completion of the structural steel components, formwork was placed between the ribs, reinforcement was installed, and concrete was poured, forming the deck. At the same time, the two towers were constructed, installed, and stabilized using temporary stays.

After reaching the necessary strength in the concrete slab, cable installation began, which was broken down into two steps. In the first step, the ring cables were connected to the girder and stressed. The ring cables were supplied at a shorter length to take into account the deformation due to pre-stressing. The geometry of the ring cable prior to tensioning did not match the geometry of the structural steel work. A two-piece connection of the ring cable to the structural steel was conceived, which allowed radial displacements between the girder and cable until the final position. The ring cable was laid out in an arc of larger radius at the inside of the girder and then pushed outward and connected to the girder. This radial deformation caused the necessary pre-tensioning force in the cable. During the tensioning of the ring cable, the bridge deck was pulled inward. The deck developed a light upward tilt because the hanger cables were not yet installed. The horizontal components of the cables would later counteract this deformation.

In the second step, construction began of the main suspension cable, hanger cables, and the backstay cables. The hanger cables were installed without jacking by lengthening the backstays, causing the towers to tilt in the direction of the deck. Using this method, the relatively slack suspension cables could be installed easily. The backstay cables were then stressed to their necessary pre-stressing force, pulling the towers away from the bridge.

During pre-stressing of the cable structure, the hanger cables became the support for the bridge by lifting the deck from the scaffolding. The horizontal component of the hanger cable force counteracted the tilt of the bridge deck, bringing the bridge into the desired geometry and marking the end of construction.

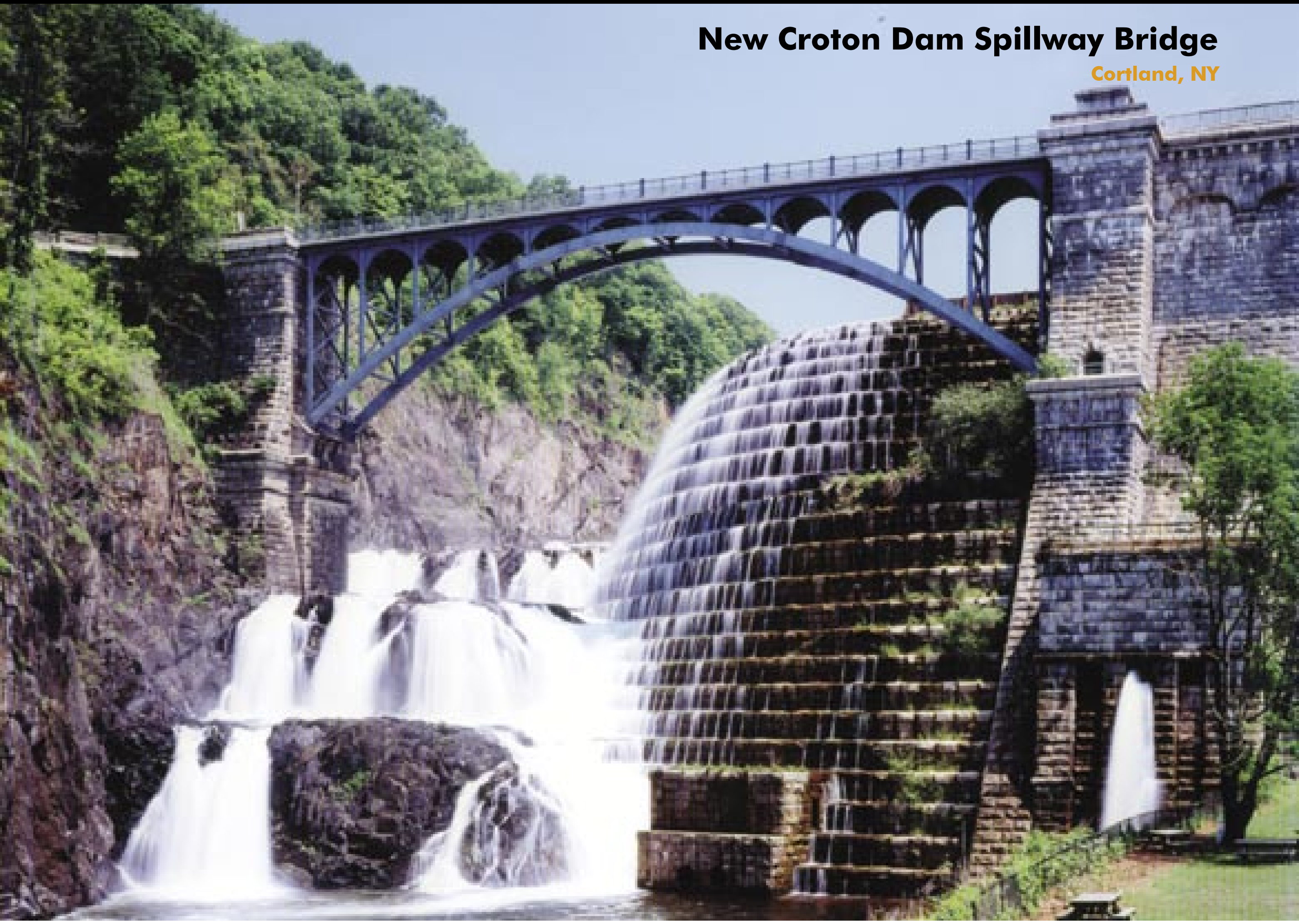
## Detailing

Detailing of the bridge's main elements was carefully designed to achieve a high level of visual appeal. Cast steel components were used in order to have visually clean, highly accurate connectors and anchors. Other important details included cable clamps connecting the hanger cables to the main cables. A cast steel element was chosen to allow an efficient member to carry the loads from the hanger cables to the main cables by the means of friction between the clamps and the cable.

The bridge railing is visually transparent and has a series of thin horizontal cables and slender vertical supports that are coordinated in plan and elevation with both the hanger cables and steel struts. ★

# New Croton Dam Spillway Bridge

**Cortland, NY**



Hardesty & Hanover

**Owner**
New York City Department of Environmental Protection, Valhalla, NY

**Engineer of Record**
Hardesty & Hanover, LLP, New York

**Engineering Software**
SAP2000, MicroStation

**Detailer**
Tensor Engineering, Indian Harbour Beach, FL, NSBA/AISC member, NISD member

**Fabricator**
High Steel Structures, Inc., Lancaster, PA, NSBA/AISC member

**Erector and General Contractor**
Kiewit Constructors, Inc., Park Ridge, NJ, NEA member

The historic New Croton Dam has been a key element of the New York City reservoir system since 1906. A new 200'-long steel arch bridge over the New Croton Dam spillway now provides a focal point for this historic stone masonry dam in time for its 100th anniversary.

Large arch base displacements and other problems necessitated emergency closure of the previous bridge to traffic. This bridge was constructed in 1975 to replace the original structure, which was constructed in 1905. The design team was challenged with a fast-track bridge replacement and overall aesthetic improvement, as well as maximizing service life and devising innovative procedures for erection over the spillway torrent.

Structural steel was chosen for this $4.6 million project because it could be crafted to an efficient form that was both light in appearance and timelessly durable. An innovative erection scheme was devised that supported both a work platform and erection shoring from the existing arches to facilitate rapid, economical erection.

## Replacement Design

The replacement bridge's design was completed and all permits and approvals were obtained in a three-month compressed schedule. Designers were faced with a dilemma as to whether to attempt a replication of the original 1905 structure or to create a visually distinct yet context-sensitive structure. A consensus was reached that the new bridge should be an improved and more durable version of the original.

Spandrel columns distribute deck loads over the length of the arch, as opposed to concentrating at the center. Spandrel columns not only allow the span to function efficiently as an arch, but also serve to restrain the arch bases to prevent displacement. The new arch bases were anchored at the skewbacks using rock anchors. Another basic design element was a lateral seismic restraint at deck level. By allowing the deck system to be partially supported by the abutments, multi-rotational bearings could serve double duty as seismic restraints.

It was decided to space the spandrel columns uniformly and further apart for compatibility with the dam detailing. The arches' fascia panels were designed as stiffened steel plates with a tube welded to their arched edge for stiffening and to add visual relief. The panels were set to the back of the spandrel columns to add shadowing and further relief to avoid a flat appearance.

The 1975 bridge had the deck girders nested into and supported on the abutments. The deck joints formed a "U" in plan with inherently problematic longitudinal joints at the gutter lines. In order to eliminate these longitudinal deck joints, the deck was stopped flush with the face of the masonry abutment and a concrete seat was cantilevered out to support the deck bearings. This seat was visually concealed in the shadows of the deck system. It was anchored into the dam masonry with rock bolts and into the rock face beyond the fill at the north abutment.

The new steel superstructure included 2' by 3' welded box sections for the ribs, welded steel box sections with integral connection plates at the spandrel columns and spandrel girders, and a rolled floor beam and stringer system. Bracing elements at the columns and arch ribs were sealed HSS. The new arch ribs were fabricated in three sections with bolted field splices for easy, cost-effective erection. The new ribs would bear on the existing granite skewbacks outboard of the 1975 bridge arches where the 1905 bridge was seated.

The new bridge seats and deck incorporate high performance concrete and solid stainless steel reinforcing steel for maximum service life.

All structural detailing was aimed at minimizing corrosion potential, so sealed welded boxes and HSS were used. Details that deflected and accumulated spray away from spandrel column bases were incorporated.

The visible finish coat of all steel is a highly durable, thermally sprayed metalizing coating. The metalizing coating was an 85% zinc, 15% aluminum material with a penetrating sealer, but no paint. The inaccessible interiors of the HSS were coated by hot dip galvanizing, and the exposed exteriors of the HSS were metalized for visual continuity. The interiors of the sealed welded boxes were painted to avoid potential warping associated with hot dip galvanizing.

## Construction and Erection

Construction started in summer 2003. Initial fieldwork proceeded while shop drawings were prepared and fabrication commenced. Steel was shop assembled to assure proper fit-up.

The spillway was a particularly difficult work site. Not only did heavy flow in the spillway channel preclude shoring from below, but the entire structure would also be difficult to access during the phases of demolition and new construction. While details of the erection scheme were left up to the contractor, it was felt that there would be an advantage to building the new arch ribs outboard of the existing ones. There was ample space for anchoring the arch to the skewbacks at the original location. With this arrangement, the new arch ribs could be installed before removal of the existing arches. The existing bridge could then be used for access and staging purposes.

The existing deck girders were first supported on temporary columns and partially removed so the construction of the new bridge seats could be expedited and erection of the new bridge could proceed uninterrupted. Due to a similar profile of the existing and proposed arches, it was decided to use the existing arches to support the new arch segments being erected. Later, the old ones were supported by the new ones while they were disassembled. Temporary beams below the existing arches cantilevered out to support the new ribs, as well as the work platform and protective shield. The deck was removed to lighten the load on the existing arch ribs and eliminate an obstruction to the rib erection. The new arch ribs were erected in segments with cranes from both ends of the bridge. The splices were bolted up and the bases grouted to prepare the ribs for carrying the load. The underslung beams were then connected to the new arch ribs and platform loads were transferred to them. As the existing arches were cut for removal, their loads were also transferred to the new ribs. The original work platform remained in service for the entire construction period. The remaining steel was erected using cranes at either end of the bridge. ★

# 17th Street Bridge

**Atlanta**

David Lawrence, Prime Images, Inc.

The 17th Street Bridge crosses the I-75 and I-85 interchange in Atlanta and provides a vital link between the city's Midtown neighborhood and the site of a future mixed-use development.

The bridge is a three-span structure with span lengths of 205'-336'-289', consisting of variable depth steel box girders. The structure depth varies from 14'-1" over the two intermediate piers to 7'-9" at the bridge ends and at midspan of the center span.

Considering the dimensions of the box girders, the field sections selected for the girders were relatively large to minimize the number of elements that needed to be handled over the busy highways. The longest field section erected was 190' long, while the heaviest field section was 100' in length and weighed 125 tons. Setting of the box girder field sections required only a few temporary traffic detours, and these occurred at night during weekends.

The main span length of 336' is the shortest span length that could be utilized, considering the limitations imposed by the current lanes of traffic and an I-85 bridge planned for construction under the main span. With a center span of 336', the end spans had to be relatively long as well, resulting in the final three span configurations.

A T-intersection located within the limits of the bridge at both ends presented sight-distance issues in regards to above-deck elements such as arch ribs or cable-stayed towers. The use of steel box girders addressed these concerns by eliminating all above-deck elements.

The south sidewalk, at 22'-0" wide, has an undulating shade canopy and continuous missile fencing, both made of a combination of stainless steel and galvanized carbon steel. The north sidewalk, at 30'-0" wide, also has continuous missile fencing, plus intermittently spaced trellises in planter boxes. Both the north and south sidewalks have perch rails at regular intervals along the length of the bridge, which allow pedestrians to rest while enjoying the views of the city. ★

**Owner**
Georgia Dept. of Transportation

**Engineer of Record**
URS Corporation, Tampa, FL

**Engineering Software**
MDX

**Detailer**
Tensor Engineering, Indian Harbour Beach, FL, NSBA/AISC member, NISD member

**Fabricator**
Tampa Steel Erecting Company, Tampa, FL, NSBA/AISC member

**Erector**
V & M Erectors, Pembroke Pines, FL, NSBA member

**General Contractor**
C.W. Matthews Contracting Company, Marietta, GA

# Iowa River Bridge

## near Steamboat Rock on U.S. 20 over Iowa River

Designers for the Iowa River Bridge on U.S. 20 near Steamboat Rock over the Iowa River recommended a launched steel I-girder design and erection technique for the 1,630' project. The design featured longer spans to reduce the number of piers needed and to minimize visual obstructions at river level. The launched-girder erection technique would eliminate the need for the temporary erection towers and piece-by-piece "in place" erection of structural steel required by conventional methods.

The incrementally launched erection process consisted of:

- Erection of all structural steel for the first 154 m of the eastbound bridge (including girders, diaphragms, and upper and lower lateral bracing) on temporary pile bents behind the east abutment in a launching pit.
- Attachment of a launching nose (leading end) and tail section (trailing end) to the girder train.
- Jacking of the girder train forward longitudinally 92 m from Pier 6 to Pier 5.
- Removal of the tail section and splicing of additional girder sections to the tail end of the girder train.
- Reinstallation of the tail section.
- Longitudinal jacking of the girder train to Pier 4

This sequence was repeated for a total of five spans. After steel erection was completed on each span in the launching pit, including all diaphragms and lateral bracing, the steel was launched downhill along a 0.64% grade, being pushed by hydraulic pistons towards the west abutment at a pace of approximately one foot per minute.

After adjustments were made to the steering mechanism to ensure the spans were guided in the proper alignment, the launching process moved forward. The temporary launching nose was attached to the front of the leading span to guide its placement and reduce deflection of a 302' cantilever. Temporary roller bearings placed on the bridge piers assisted with the process of rolling the sections across the valley.

Following the launch of the eastbound bridge, the contractors' equipment was moved to initiate an identical launching of the parallel westbound bridge. After the launch of the tenth and final span was completed, the launching skid was removed. The full length of the superstructure was jacked up to remove the rollers and then jacked down onto permanent bearings on the piers.

ASTM A709 Grade 345W weathering steel was selected for the girders for aesthetic reasons and to eliminate the need for costly painting. The project, which required 9.2 million lb of structural steel, was completed approximately seven months ahead of the original contract schedule. ★

**Owner**
Iowa Dept. of Transportation, Office of Bridges

**Engineer of Record**
HNTB Corporation, Kansas City, MO

**Engineering Software**
T187 (HNTB proprietary software)

**Fabricator**
Capitol Steel and Iron, Inc., Oklahoma City, NSBA/AISC member

**General Contractor**
Jensen Construction Company, Des Moines

**Erection Engineer**
Ashton Engineering, Davenport, IA

*To read more about the Iowa River Bridge, see "Landmark Launch" in the February 2004 issue of* MSC *at www.modernsteel.com.*

Photo courtesy of Iowa Department of Transportation.

# MERIT AWARDS

MAJOR SPAN

## Bill Emerson Memorial Bridge

**Cape Girardeau, MO**

The Bill Emerson Memorial Bridge, named after an eight-term Southeast Missouri congressman, is a 3,956'-long, 96'-wide structure linking Cape Girardeau, MO and East Cape Girardeau, IL. The bridge carries a four-lane roadway and includes an 1,870' eastern approach structure and a 2,086' cable-stayed unit with a 1,150' navigation span.

The bridge is located 50 miles from the New Madrid, MO fault zone and was designed for a magnitude 8.5 earthquake. In addition to seismic hazards, other design issues included the potential for liquefaction and lateral spreading at the Illinois river bank, geological issues, the probability of deep scour, and the potential of a barge collision with any of the bridge piers (the bridge was designed to resist the force of a 1,200'-long barge tow).

The bridge features a 1,150' cable-stayed navigation span with conventional steel and an approach structure comprised of eleven 170' conventional composite steel plate girder spans.

The cable-stayed structure is supported on rock-bearing footings and hydraulically dredged caissons. The approach spans are supported on deep, large-diameter drilled shafts socketed into rock.

Built by the balanced-cantilever method, the two halves of the main span were connected without any special jacking or counterweights needed to make the closure. A seamless connection was made at the middle of the river, and the bridge was within 1" of target.

The bridge also includes cable connections and tie-downs at the end of the cable-stayed spans. The cable connections are extensions of the girder webs designed to load the webs directly and to minimize the torsional stresses that could be induced with offset connections. The tie-downs are a combination of a sliding block and rotating pin, which allow the bridge to translate and rotate under both downward and uplift load conditions.

The bridge was fabricated with Grade 50 weathering steel. The cable connections above the deck were painted to match the two-tube bicycle rail atop the barrier curbs. ★

Photos courtesy of HNTB Corporation/Mark McCabe.

**Owner**
Missouri Department of Transportation

**Engineer of Record**
HNTB Corporation

**Detailer**
Tensor Engineering, Indian Harbour Beach, FL, NSBA/AISC member, NISD member

**Fabricator**
Vincennes Steel Corporation, Vincennes, IN, NSBA/AISC member

**Erector and General Contractor**
Traylor Brothers, Inc., Evansville, IN, AISC member

**General Contractor**
Massman Construction Co., Kansas City, MO

# Third Avenue Bridge over the Harlem River

**New York**

Photo by Raimondo di Egido.

**Owner**
New York City Department of Transportation

**Architect**
B. Thayer Associates, New York

**Engineer of Record**
Hardesty & Hanover, LLP, New York

**Engineering Software**
GT STRUDL

**General Contractor**
KiSKA Construction Corp., New York

The new Third Avenue Bridge over New York City's Harlem River consists of 17 steel girder approach spans and a 350'-long, 6 million lb steel through-truss swing span designed to maintain the historic aesthetics of its predecessor and of the surrounding region. An on-line, staged construction scheme employed innovative concepts, including float-in of the fully-assembled swing span and a pivot pier founded on 6'-diameter steel shafts arranged to complement construction staging and to minimize demolition efforts. Other critical design features include a 15'-deep steel box pivot girder and the highest load capacity spherical roller thrust bearing ever used for a swing bridge.

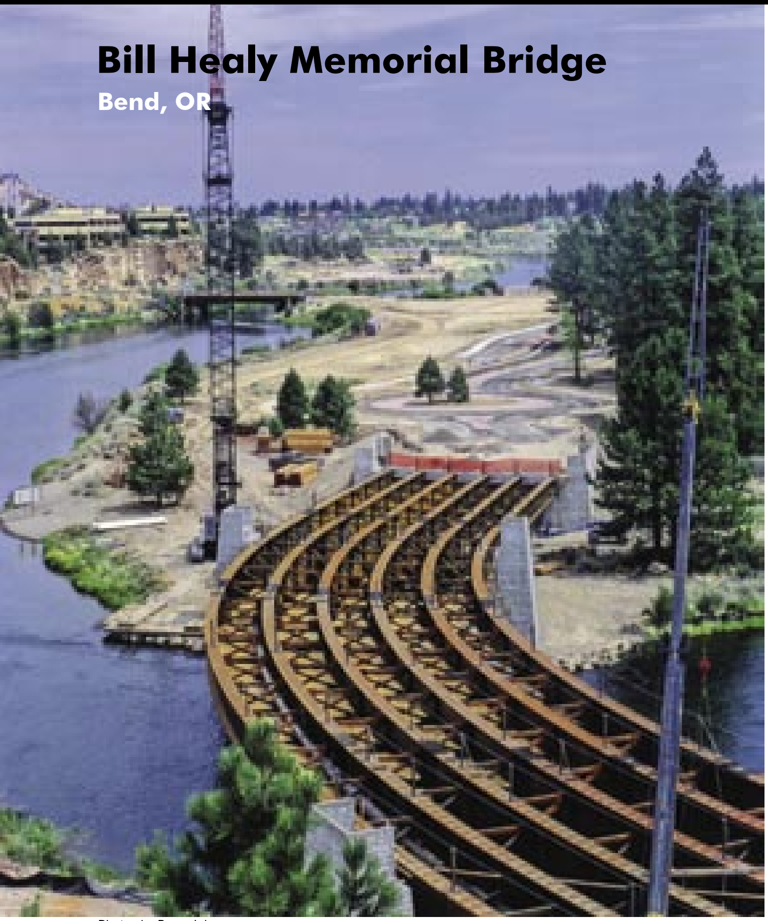
To prepare for the float-in of the new span, the 2000-ton existing swing span was cut in half with torches and saws and removed from the site in three major pieces. Next, the new swing span was transferred from a single barge to two barges to allow clearance with the fender and pivot pier during float-in. Over the next several months, the span remained atop these two barges. The barges were moored about 200 yards south of the bridge where installation of the bridge deck, barriers, and railings, completion of the control house, and installation of the electrical systems took place. During this same time period, the pivot pier was constructed, pier mounted machinery was installed, and the remaining work on the approach spans completed. With the bridge ready to receive its main span, the swing span was floated into position and permanently lowered onto its pivot assembly. ★

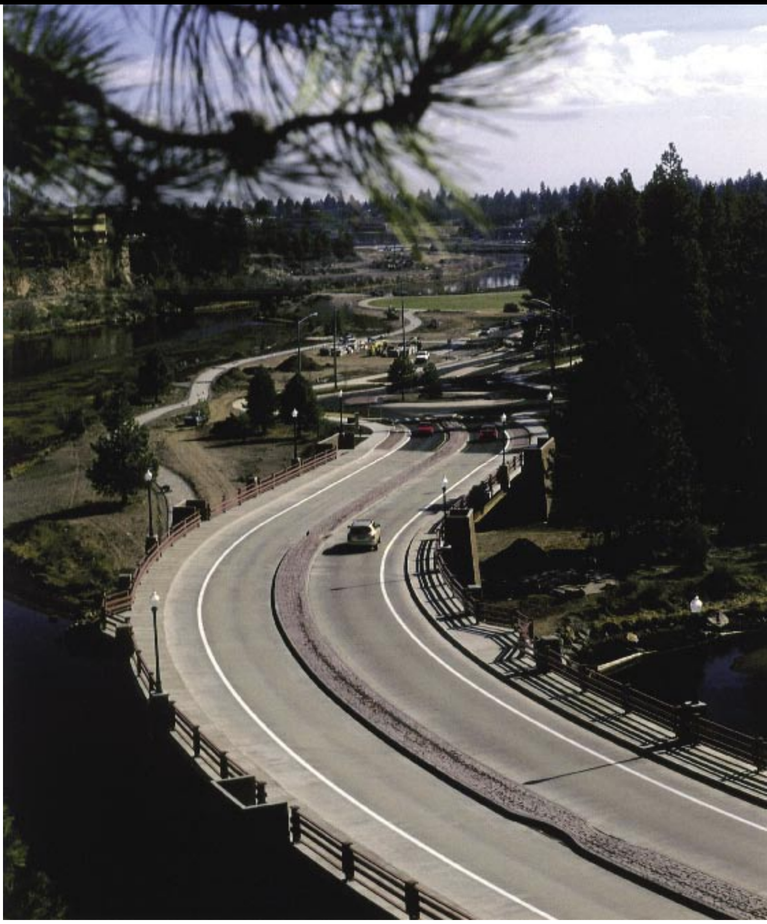
# MERIT AWARDS



## Bill Healy Memorial Bridge

**Bend, OR**





Photos by Barry Johnson.

**Owner**
City of Bend, OR

**Engineer of Record**
W&H Pacific, Bend, OR

**Engineering Software**
MDX

**Detailer**
Steelcon, Inc., Cordova, TN, AISC member, NISD member

**Detailing Software**
AutoCAD

**Fabricator**
Fought & Company, Inc., Tigard, OR, NSBA member, AISC member

**Erector and General Contractor**
Hamilton Construction Co., Springfield, OR, NSBA/AISC member

The Bill Healy Memorial Bridge is a 480'-long steel plate girder bridge crossing the Deschutes River and a one-mile extension of Reed Market Road in Bend, OR.

The bridge design incorporates curved, haunched steel plate girders combined with architecturally treated concrete piers that blend with the character of the canyon. The curvature allows the road and bridge to fit the existing constraints of the steep canyon walls and narrow ravines. The haunches create a streamlined, aesthetic appearance and an "open" feeling for people traveling beneath the structure.

The three-span steel girder structure minimizes structure depth, providing ample space for trail users and wildlife to cross under the approach spans, and eliminated the need for piers in the water. Multiple curves were required, which resulted in vertical curves at each end of the bridge with a constant 2% slope between. The horizontal alignment includes a tangent section with reversing curves at either end.

The end spans are typically 60% to 65% as long as the main center span to balance the loads and avoid uplift at the abutments. To keep a balance in the spans, for every foot increase in the length of the main span the bridge, total length would increase close to 2.5', or a special tie-down method at the abutments would be required. A balance between span lengths, individual girder lengths, roadway geometry, overall bridge length, and environmental impact dictated the location of the piers and abutments.

Because of the bridge's curvature, special bearing devices were selected to allow for multi-direction movement. In maintaining a balance between all of the geometric design constraints, the design team decided to allow minor uplift under maximum live load at the end bearings. To account for this, special uplift bearings were incorporated to eliminate any upward movement by the girder.

The faces of all walls slope to create a more visually balanced effect. Three large arched portals are in each pier to allow for more light under the bridge and better views of the river. A natural native rock wall appearance was created at the bridge piers and abutments with stained rock-like concrete shaped by form liners. Multiple arches within the pier structures were incorporated to increase visibility.

Another important feature is the bridge's "raised" bike lanes. By raising the bike lanes next to the travel lane, autos are restricted to a narrower lane that encourages lower speeds. At the same time, the roadway is wide enough to allow emergency vehicle access.

Girder erection was accomplished by erecting the end spans, pier sections, and the final drop in span in the center of the bridge. The contractor was able to use design coordinates for bearing seats and end sections of each girder piece to ensure the final placement of the girders. The girders were placed on temporary supports by cranes to splice the span lengths together. ★

## Nebraska Highway 2 over I-80

**Grand Island, NE**

Photos courtesy Nebraska Department of Roads.

**Owner**
Nebraska Department of Roads

**Engineer of Record**
Nebraska Department of Roads, Grand Island, NE

**Fabricator and Erector**
Capital Contractors, Inc., Lincoln, NE, NSBA/AISC member

The Highway 2 Bridge over I-80 in Grand Island, NE is one of the first bridges in the U.S. to use High Performance Steel (HPS)-100W. It is also among the first to use a new pier connection detail concept, "Simple for Dead Loads, Continuous for Live Loads and Superimposed Dead Loads," which eliminates bolted splices.

The bridge is a two-span steel box bridge, with each span at 139' long. Preliminary designs indicated that use of HPS and steel boxes in conjunction with the new system, which was developed to provide a cost-effective alternative to concrete bridges in short span ranges, would be economical. Use of HPS made it possible to increase the span length for each girder beyond the traditional 120', while keeping the total weight of each girder below 60,000 lb—the crane capacity of local fabricators.

Use of the new system significantly reduced the time required to place the girders over the supports. Elimination of bolted splices was accomplished by joining the girders over the pier using a concrete diaphragm. The detail over the pier allowed the girders to act as simple beams during casting of the concrete deck and to behave continuous after the concrete hardened.

Design of the box girders was based on the assumption that the girders would use a hybrid arrangement, with the bottom flanges of the box sections using 70 ksi HPS and the webs and top flanges using conventional 50 ksi steel. HPS plates permitted the use of thinner plates for bottom flange and reduction in web depth. ★

Photos courtesy HDR Engineering, Inc.

## Fort Pitt Bridge and Approaches

**Pittsburgh**

Built in the 1950s and used by 150,000 vehicles daily, Pittsburgh's Fort Pitt Bridge and Tunnel started showing their age with corrosion, cracking, and dangerous deterioration, the Pennsylvania Department of Transportation (PENNDOT) was faced with the task of rehabilitation. The $84 million Fort Pitt Bridge and Tunnel Rehabilitation was completed in September, 2003.

The project included the rehabilitation of dual 3,600'-long tunnels, a double-deck 750' tied arch river span, and a "spaghetti-bowl" of ramp structures. Innovative design solutions resulted in significant cost savings, enhancing aesthetics and improving the long-term serviceability of the structures, including:

➜ Simple steel approach spans spliced over the piers to eliminate joints.

➜ New traffic barrier designs that preserve views from the bridge.

The bridge rehabilitation featured complete deck replacements; strengthening of truss diagonals to meet construction-staging requirements and to provide increased capacity; and replacement of the existing lead based paint with a new three-coat paint system for all of the structural steel. Complete replacement of some steel spans was more economical than rehabilitation.

The original approach spans were primarily simple spans made up of rolled steel shapes and riveted plate girders. Open joints at the piers and years of exposure to de-icing salts caused the greatest deterioration. Where feasible, the existing simple span steel girders were spliced over the piers to eliminate the expansion joints. Fewer joints will enhance the long-term serviceability of the structures by preventing joint leakage and improving the ride quality of the deck. The splices have the added benefit of increasing the capacity of the girders by making them continuous for live load. ★

**Owner**
Pennsylvania Department of Transportation

**Engineer of Record**
HDR Engineering, Inc., Pittsburgh

**Engineering Software**
GT STRUDL, BAR7

**Detailers**
John Metcalfe Company, Monroeville, PA, AISC member, NISD member

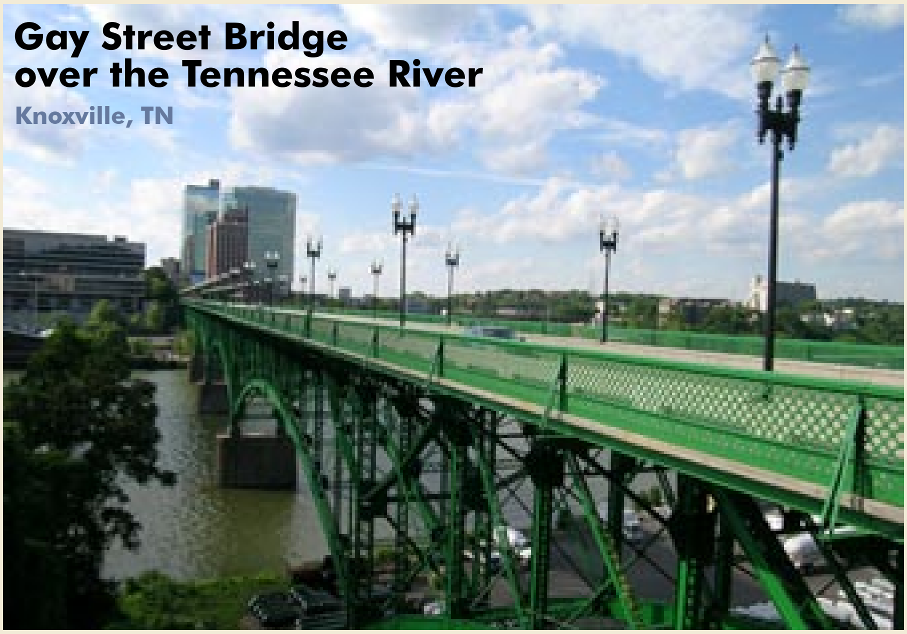
**Detailing Software**
SteelLogic

**Erector**
Multi-Phase Inc., Coraopolis, PA, AISC member

**General Contractor**
Trumbull Corporation, West Mifflin, PA

## Gay Street Bridge over the Tennessee River

**Knoxville, TN**



By George Hornal.

The Gay Street Bridge, constructed in 1897, crosses the Tennessee River in Knoxville. The bridge is composed of seven spans of pin-connected, arched cantilever trusses with a total length of 1,512'. The deck has a 30' roadway with two 6' sidewalks.

By the late 1990s the curbs and deck had deteriorated to the point that they were beyond repair. A study concluded that the Gay Street Bridge could be rehabilitated to modern standards while maintaining its historical character.

The entire floor system, except for the floor trusses, was replaced. A new lightweight concrete roadway and sidewalks, deck joints, drainage, street lighting, curb railings, and approaches were provided.

The centerpiece of the work was reconstruction of truss pin joints at 132 locations. Successful repair of the pin joints was vital to saving the bridge, otherwise total replacement would have been necessary. This entailed partial disassembly of the members meeting at a joint, with new components spliced into the end connections to replace the corroded steel. Pins remained in place and disassembly was only permitted on one side of the joint, either inboard or fascia, at a time. These structural repairs were complicated, as the member ends were stacked in layers on the pin. ★

**Owner**
City of Knoxville

**Engineer of Record**
Lichtenstein Consulting Engineers, Paramus, NJ

**Engineering Software**
GT STRUDL

**Detailer and Fabricator**
Beverly Steel, Inc., Knoxville, TN, NSBA/AISC member

**Detailing Software**
AutoCAD, AutoSD

**General Contractor**
Ray Bell Construction Co., Brentwood, TN

# SR 3008 (Hillsborough Street) over CSX and NS Railroads

**Raleigh, NC**

Hillsborough Street in Raleigh, NC crosses over a railroad corridor used by two Class A railroads and a future light rail facility. The Hillsborough Street Bridge (SR 3008) provides vehicular and pedestrian passage across this busy railroad corridor. And because the surrounding area is a historic district, construction of the bridge could not damage any of the surrounding buildings.

Urban arterial roadways east, west, and tangent to the new bridge set the transverse bridge configuration with a required typical section width at the west end of 22.664 meters (74.36') and a required east-end typical section width of 27.509 meters (90.25'), producing a width difference of 4.845 meters (15.90'). A rhombus-shaped footprint for the bridge deck was selected in lieu of a rectangular shape to minimize the required square footage of the deck. This configuration required the girders to be splayed instead of parallel and reduced the deck area by 92.33 square meters (994 sq ft).

The proposed bridge depth was governed by two site specific factors: an existing masonry building and the two railroads beneath Hillsborough Street. The result of upper and lower limitations produced a height envelop for the bridge superstructure of 688mm (2.25').

Steel was selected for the bridge due to its span capabilities, and a fixed haunched plate girder bridge type was found to meet the restrictive site requirements. The rigid frame consisted of steel haunched plate girders embedded into a concrete cap creating an integral beam/cap supported by reinforced concrete columns and drilled shafts.

By utilizing a fixed-fixed end condition, the induced dead and live moments were shifted from the center of the girder span to the girder ends. This permitted the girder depth to be minimized. To maintain the fixed-fixed condition necessary and to enable the superstructure to expand and contract freely, one end of rigid frame had to be reconfigured. The revised configuration included an additional short approach span adjacent to the long main span and utilized a two-span continuous girder, thus producing an equivalent single span rigid frame unit. ★

**Owner**
North Carolina Department of Transportation

**Engineer of Record**
TranSystems Corporation, Greenville, SC

**Engineering Software**
Merlin-DASH

**Detailer and Fabricator**
Carolina Steel Corporation, Greensboro, NC, NSBA/AISC member

# MERIT AWARDS

SPECIAL PURPOSE

## BP Pedestrian Bridge at Millennium Park

**Chicago**

Peter Barreras Photography.

The BP Pedestrian Bridge links Chicago's newly opened Millennium Park to the Lake Michigan across Columbus Drive. Overall, the bridge is some 920' long. Both bridge approaches are comprised entirely of reinforced concrete continuous bearing walls, curved in plan and elevation, with reinforced concrete slabs spanning between the bearing walls. The resulting length of the approaches allows for gentle grades (less than 5%), which avoids the requirement for longitudinal handrails and intermediate flat landings to meet accessibility standards.

The termination of the approach structures is formed by reinforced concrete abutments which in turn support a transition structure between the approaches and the Columbus Drive crossing proper. These transition structures are propped three-dimensional cantilever steel trusses spanning between the concrete abutments to single 6'-diameter reinforced concrete pylons, which are constructed atop existing garage structures and located along existing column lines. The trusses cantilever beyond the pylon supports approximately 8' on the west and 28' at the east; each locating longitudinal expansion joints in the structure.

Both cantilever trusses are formed by a four-chord spine composed of large diameter pipes with web members W14×43 throughout. The trusses to the east and west of the Columbus Drive span are significantly different geometrically, with the east truss doubly curved in plan in addition to the longer cantilever. All truss chords are curved in a single plane only and then inclined, with the west chords formed by two tangent arcs of radii varying from 72' to 130' and the east chords formed by three tangent arcs of radii varying from 52' to 130'. All truss pipe chords are 20"-diameter with wall thicknesses of 1.5" at the west and 2.0" at the east. While the top chords of the trusses are spaced to match the width of the walkway (15' apart), the bottom chords are considerably narrower (5') to bear on the pylon support. The central four-chord spine of the cantilevered trusses is further structured with transverse outrigger members, beyond the width of the walkway, which serve as auxiliary supports for the architecturally clad surfaces.

Structurally, the four-chord articulated cantilever trusses narrow as they approach the expansion joints, which then give way to a discrete rectangular steel box girder. The box girder is a two-span arrangement (approximately 80' each) between the ends of the cantilevered trusses and a reinforced concrete pylon. The box girder is 72" wide, 39" deep with 1.5" thick top and bottom flanges, and 0.5" web plates. The box girder is composed of six continuously curved tangent arc segments with radii varying between 92' and 140'. Longitudinal W12 purlins support the edge of the walkway deck. A series of transverse outrigger frames and longitudinal girts support the architectural cladding secondary frames. ★

**Owner**
City of Chicago

**Architect**
Gehry Partners LLP, Los Angeles

**Engineer of Record**
Skidmore, Owings & Merrill LLP, Chicago

**Detailer**
Mountain Enterprises, Inc., AISC member

**General Contractor**
Walsh Construction, Chicago

# Turtle Bay Sundial Bridge

**Redding, CA**

**Owner**
City of Redding, CA

**Architect**
Santiago Calatrava, SA, Zurich, Switzerland

**Engineer of Record**
Calatrava Firm, Zurich, Switzerland

**Detailer**
Tensor Engineering, Indian Harbour Beach, FL, NSBA/AISC member, NISD member

**Fabricator**
Universal Structural, Inc., Vancouver, WA, NSBA/AISC member

**General Contractor**
Kiewit Pacific Co., Vancouver, WA

Photo by David Greusel.

The 1,600-ton Turtle Bay Sundial Bridge opened July 2004 over the Sacramento River in Redding, CA. It consists of a 300'-long by 217'-wide single-inclined tapered pylon supporting a 722' truss span. The triangular pylon, inclined at 42 degrees, runs in a true north-south direction. At approximately 110' long by 75' wide, the open base of the pylon forms a plaza at a 36-degree angle between the east and west walls.

The west face, which supports the triangular deck span, inclines two degrees to the west, while the east face is inclined toward the west at 28 degrees. The north wall is a curved, warped surface connecting to the rear edge of the west and east walls. Each wall has both inner and outer faces which are 2'-8" apart at the base and taper to about 6" at the top.

The span is supported by 14 cables connected to transverse bulkheads in the deck truss and to plate brackets cantilevered from the west inclined face of the pylon. Horizontal and vertical stiffener systems, as well as skewed stiffeners, line up with the cable support brackets.

The entire structure is made up of plate material varying in thickness from 1" at the base and reducing to 5/8" at the top.

The 722' pedestrian bridge deck system is a triangular pipe truss with a 14'-diameter bottom chord and two 11'-diameter top chords. The 23' wire translucent deck is framed in glass panels with granite accents. The deck is not symmetrical, and so it had to be cambered vertically, longitudinally, and transversely.

*To learn more about the Turtle Bay Sundial Bridge, read "Sun Sculpture" in the October 2004 issue of* MSC*, available at www.modernsteel.com.* ★