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.

Courtesy HDR Engineering.

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.

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. ★