Fabrication and Erection of Poplar Street Orthotropic-Plate Deck Girder Bridge


This paper describes design concepts concerning construction, and the fabrication and erection techniques employed on the first major orthotropic-plate deck girder bridge undertaken in the United States. The bridge was completed in November 1967.

Except for a small amount of structural shapes, the bridge is composed of plates of various types of steel. The article describes the jigs and special handling equipment used to fabricate and assemble large orthotropic-plate deck sections and large welded box girders. It illustrates the special mounting of welding equipment used to perform the miles of automatic submerged-arc welding, and it describes the shop-assembly procedures. Erection sequences, erection equipment, and suspended trolley platforms are described and illustrated.

It was learned that precise and accurate fabrication methods, coupled with good supporting equipment, permitted extensive use of automatic welding equipment for the miles of welding required on the bridge. Another conclusion was that large units of deck sections and girder sections could be economically fabricated and erected.

THE Poplar Street Bridge, crossing the Mississippi River at St. Louis, is the first major orthotropic-plate deck girder bridge in the United States. This bridge type (Fig. 1) was recommended in a preliminary engineering report completed in 1961 by Sverdrup & Parcel and Associates for the States of Illinois and Missouri as being aesthetically satisfying, as providing motorists a clear view of the St. Louis waterfront, and as being economically attractive. Final design was completed in 1964 and construction was finished in November 1967.

Dravo Corporation was the substructure contractor and, under a prime contract, Bethlehem Steel Corporation fabricated and erected the structural steel. A separate contract included the concrete curbs, parapets, and the wearing surface. Field painting was done under another contract. The total cost of all construction was $13,260,000.

The 2,165-ft bridge is composed of five spans; the 600-ft central span is the longest. Since the three main spans and parts of each of the end spans are over water during normal river stages, the design plans were based on water shipment to permit fabricating larger units in the shop, thus leaving fewer pieces to be erected and joined in the field.

The eight-lane roadway is divided into two separate four-lane structures supported by common substructure units. Each four-lane deck is supported on two box girders (Fig. 2). The deck is thus divided into a center deck section of about 27-ft width between the girders, and about 9-ft wide cantilever sections outside each girder. The deck units were field-bolted to the girders at the floorbeams, which are spaced 15 ft apart and are connected by continuous welds in the deck plate.

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Figure 1. General view of the completed bridge.

Figure 2. Spans and bridge cross section.
DECK UNIT FABRICATION

The main elements of the orthotropic plate are shown in Figure 3 along with the dimensions and spacing of the trapezoidal stiffening ribs. The stiffeners and deck plate are A-242 atmospheric-corrosion-resistant steel; the floorbeams are A-441 steel. Beginning with flat plates a little wider than required, the stiffeners were automatically

Figure 3. Selected orthotropic deck elements.

Figure 4. Gantry positioning ribs on deck plate.
Figure 5. Strongback in operation.

Figure 6. Cutting floor beams to fit the deck ribs.
flame-cut to the exact width and edge-bevel to suit the welding procedure and then cold-rolled into the trapezoidal-shaped ribs in a Yoder mill.

A gaging-gantry positioned four ribs on a 9-ft wide deck plate and also held the ribs in position while short lengths of submerged-arc welds were made (Fig. 4).

A strongback (Fig. 5) was developed to facilitate welding the ribs to the deck plate. This equipment has a template adjustment that was set to counteract welding distortion in the unsymmetrical section. One rib at a time was placed under the strongback, and pressure applied from the underside of the plate forced the assembly to the desired configuration. Submerged-arc, 80 percent penetration welds were deposited on both edges of the rib at the same time. The subassembly was shifted under the strongback as each rib was, in turn, welded to the deck plate.

Three deck plate and rib subassemblies were butt-welded along the bottom side of the deck plate to form a 27-ft wide centerdeck section. These units were later turned over for second-side welding after the floorbeams were fitted.

The floorbeams were made up as three-plate welded girders, and then cut to form two floorbeams. The formed beam was automatically flame-cut near the center of the web with a tape-controlled, multi-torch machine to obtain a good fit around the deck stiffeners (Fig. 6). The two halves were held together to control distortion.

The floorbeams were next fitted to the underside of the deck units, and manually welded to the deck plate and stiffeners (Fig. 7).

The fabrication procedures for the deck sections between the girders were also used for the 9-ft wide cantilever deck units and the 6-ft wide sections attached to the girders.

**BOX GIRDER FABRICATION**

The box girders are 5 ft 6 in. between webs and vary from 16 to 25 ft in depth. The bottom flange is 7 ft wide and varies from 3/4 to 3 3/4 in. in thickness. During the fabrication and erection periods a 6-ft width of orthotropic deck formed the top flange of the...
The girder webs and bottom flanges are A-36 and A-441 steel (Fig. 8). The top flange of the girder, comprised of the deck plate and stiffening ribs, is A-242 steel. In addition to providing greater protection against atmospheric corrosion, this metal type was specified to provide for the high stress resulting from girder moments and the stress caused by wheel loads. However, the girder is proportioned so the stress at the top of the web plate is within A-36 steel limitations.

The 2,165-ft girders were each divided into 35 sections, ranging between 56 and 80 ft in length. Items controlling the unit lengths included section weight, change of type of metal in the bottom flange, and change of bottom-flange plate thickness. The deck sections were made the same length as the corresponding girder sections. The heaviest girders were the 113-ton, 70-ft long haunched sections over the piers. The lightest girder sections were each 56 ft long and weighed 56 tons.

The 7-ft wide girder bottom-flange plate was butt-welded, where required, to make up the full length of a girder section. A table jig was used to permit welding on one side and inverting to weld the second side. The submerged arc-welds were ground smooth and fully X-rayed.

The same table was used for laying out the longitudinal plates making up the girder webs. The 1/8-in. web plates were butt-welded with automatic submerged-arc equipment. After welding on one side, a lifting rig was used to turn the assemblies for second-side welding. The welds on the exterior faces of the outside girders were then ground smooth. After the web plates were welded and inspected, the plates were flame-cut along the edges to the cambered dimensions and shape.

The diaphragms within the box girders were spaced at 15-ft centers, and an intermediate vertical T-stiffener was placed halfway between the diaphragms. The horizontal T-stiffeners were fitted between the vertical elements (Fig. 9). The number and spacing of the horizontal T's varied according to the stability requirements of the webs. The T's were tack-welded into position and, with a specially mounted submerged-arc welder, two fillet welds were simultaneously applied.

A boxing jig clamped the elements of a girder together for tack-and-finish-welding on the horizontal seams (Fig. 10). A sequence was established in which the bottom flange was pressed against one web plate and tack-welded; the diaphragms were then set in place, and the second web-plate was placed and tack-welded to the bottom flange and diaphragms; the top flange (deck section) was next positioned, pressed against the webs, and tack-welded. By turning the girders twice, nearly all longitudinal strength-welds were made in the horizontal position.
After the girders were boxed and welded, they were fitted with exterior stiffeners at floorbeam locations, bolt holes were subdrilled for end-connections, and miscellaneous items such as bracing connections and floorbeam supports were added.
SHOP ASSEMBLY

Three continuous girder sections were assembled to cambered shape in the shop (Fig. 11), and after reaming the end-connections to full size, two additional girder units were added to the last unit. This assembly procedure was followed for the full length of the girders.

Selected sections were check-assembled with the girders and all the deck units in place. These were then disassembled, cleaned, and spray-painted with three coats on the inside of the girders and one on the outside of all surfaces. The top surface of the deck was not painted.

BRIDGE SHOES

The bridge shoes are high-strength cast-steel units having a 48-in. radius spherical bearing surface which permits slight rotations of the superstructure. The expansion shoes are similar; however, they are supported on a nest of rollers that permit longitudinal motion.

ERECITION

Scheduled shipping operations began when sufficient bridge elements were fabricated, cleaned, and painted. The units were loaded into barges at Bethlehem’s Leetsdale Plant near Pittsburgh, and reached the bridge site about two weeks later.

A barge-mounted tower derrick, consisting of a 95-ft tower, an 85-ft boom, and a 20-ft jib was used to erect the steel. It had a 115-ton capacity at the boom hitch.

The steel erection began at Pier 3, which is the fixed pier in the five-span continuous unit. The first girder section (70 ft long, 113 tons) was set in September 1965. The girder was guyed to the pier by a triangular brace. All four girder strings were carried forward at the same time.

The sequence of erection is shown in Figure 12. The first erection phase centered about Pier 3 and required two erection bents. Following this, erection progressed

Figure 12. Erection sequence.
toward the Missouri shore only to allow more time for completion of concreting operations on Piers 4, 5, and 6. The third phase included erecting the remainder of the steel toward the Illinois shore.

Only six erection bents were used (Fig. 12). Erection bent locations and their removal had to be planned to keep a 300-ft navigation channel open at all times during construction. Jacking was necessary at the erection bents to obtain the desired configuration and to control stress. At each pier, the steel girders were landed directly on the shoes without a jacking assembly.

Figure 13 shows a typical scene during erection. The first step was to make the bolted splice in the girder web and the bottom flange, which was sufficient to support the cantilevered girder. The deck sections were installed and welded before setting the next girder section.

After a pair of girders were erected, K-bracing was placed near the free ends of the girders. This was the only bracing used between girders, except for the plate diaphragms over the interior piers which were designed to resist torsional forces and to transmit lateral forces to the substructure.

Figure 15. Traveling trusses.
Floorbeam and bracing connections to the girders were made with high-strength bolts (Fig. 14). The contract drawings permitted shims in both the floorbeam seat connections and the floorbeam web connections to the girder connection plate. The deck units were set to suit the welded connections, after which the holes were reamed-assembled and the joints bolted. The connection for the bracing was also field-reamed for the high-strength bolts.

Traveling trusses (Fig. 15) were positioned to relieve the stress in connections between the girder and deck plate connections while the welding was done.

The end of the central deck unit is shown in position in Figure 16. At this end of the deck plate, welding back-up strips were tacked in place on the underside of the deck plate in the shop. Back-up plates were also placed on the inner edges of the stiffener ribs to be placed with the next unit. The 3-in. projection of the stiffener ribs beyond the deck plate permitted easy setting of the next unit.

Both longitudinal and transverse welds in the deck plate were made with automatic submerged-arc machines. The ends of the girders were supported by the traveling trusses until the welding operations above and below the deck were completed. The deck plate back-up strip was tack-welded to the newly placed unit before the top side automatic welding was begun.

Manual welding was used in making the butt welds in the stiffeners. The shop welding of the ribs to the deck plate was stopped 12 in. from the end of the ribs to provide some flexibility in fitting-up the butt-welded joint. The rib was manually welded to the deck plate to complete all strength field-welding. The manual welds were inspected by magnetic-particle testing.

The edges of the deck plate joint were wedged into proper relationship prior to tack-welding. A continuous back-up strip of A-242 metal was tack-welded to both plates on the underside prior to beginning the deckside welding.

Figure 17 shows the procedure for making the automatic submerged-arc longitudinal welds on the deck plate. A guiderail was tacked to the deck to control the position of the welding machine, and a flux recovery unit was attached to the welder. Preheating was required for welding the low-alloy high-strength steel in the deck plate. The deck plate thickness varied from 9/16 to 3/4 in., requiring multiple passes to complete the weld. Similar procedures were used in making the transverse welds.

The transverse deck welds were radiographically inspected between ribs, and 10 percent of the longitudinal deck welds were radiographed.

By midwinter the superstructure erection from Pier 3 had progressed 270 ft toward midriver and 150 ft toward the Missouri shore. The steel was supported on Pier 3 and on one erection bent. An accident occurred at this point that seriously upset erection progress. A string of river barges traveling upstream went out of control and hit the erection bent, removing the pile support under one leg of the bent. Figure 18 shows that one-half of the superstructure was supported only at one girder. Fortunately, the meshing of the cantilever end of the units on the other side of the pier prevented the steel work from rotating further and falling into the river.

The first emergency move was to tie the two bridge roadways together to prevent further movement. To reestablish support and to get the steel back to its proper position, the contractor drove supporting piling beyond the edges of the structure and inserted a needle beam so the rotated bent could be moved back into proper position. This also was a temporary measure until further girder construction on the other side of the pier could relieve the load on this bent to facilitate placing a stable support directly under the bent column.

This accident clearly demonstrates the torsional rigidity of the orthotropic structure, and it was just this element which prevented a complete structural collapse. A thorough check of the structure after return to its proper position showed no structural damage in either the metal or the welds, except at areas of local contact.

The steel erection was completed in May 1967, just 21 months from its start, including delays caused by the barge accident, substructure completion, and labor disputes. Figure 19 is a view of the underside of the completed structure.

A separate contract was let for the metal railing, signing, the electrical work including lighting, the concrete curbs and parapets, and the wearing surfacing.
Figure 16. End of central unit—the wide-flange beams are the rails for the traveling scaffold (a traveling truss is in the background).

Figure 17. Welding a longitudinal deck joint.
A layered system of deck surfacing was used. The steel was blast-cleaned and coated with inorganic zinc. Two coats of coal-tar epoxy were applied, and two layers of asphaltic concrete with latex rubber additive were placed.

The wearing surface placement was completed, traffic markers were added, and the bridge was opened to traffic on November 9, 1967. Field painting was completed with the bridge under traffic.

In conclusion, this project again proved that accurate fabrication methods and good supporting equipment are required for successfully accomplishing the many miles of shop and field automatic welding required to build such a bridge. It was also apparent that the large orthotropic deck sections and large box girder sections could be readily handled both in the shop and in the field. The fabrication and erection techniques used by the contractor contributed to the low bid price of about $0.31 per pound for the 13,200 tons of metal.

ACKNOWLEDGMENTS

Most of the photographs used in this paper were furnished by the Bethlehem Steel Corporation; Kurt A. Keller, Project Engineer, Missouri State Highway Commission; and Carl Thunman, Jr., Engineer of Bridge and Traffic Structures, Division of Highways, Department of Public Works and Building.