The Poplar Street Bridge

E. J. SHIELDS

THE POPLAR STREET BRIDGE will cross the Mississippi, between East St. Louis and the central business district of St. Louis. In addition to carrying Interstate Routes 55 and 70, it will be the terminal for Interstate 44 from the west and Interstate 64 from the east.

This will be the first long span orthotropic steel plate deck girder bridge to be constructed in the United States. A contract for the substructure has already been awarded, and the superstructure will be advertised for bids in mid-1964.

At the Missouri end of the bridge is the Jefferson National Expansion Memorial, a National Park commemorating the Louisiana Purchase, which will be completed, with its 630-ft stainless steel arch, in 1965. The view of the river from the Memorial will be framed on the north by the fine old Eads Bridge, a deck type arch truss design built in 1874, and on the south by the new bridge. Immediately west of the Memorial and adjacent to the central business district is an 80-acre area being redeveloped around a 50,000-seat sports stadium, which will be completed about the same time as the bridge.

All of these factors indicate a large volume of bridge traffic and high emphasis on aesthetic values. Compatibility with the Memorial development was considered essential.

To handle the 110,000 daily vehicular crossings estimated for 1974, an eight-lane bridge is required. Since about 50 percent of the east- and west-bound bridge traffic will move over a double-deck elevated expressway which is to be built on a restricted right-of-way on the Missouri side, double-level as well as single-level structures were considered for the river bridge.

PRELIMINARY DESIGN PHASE

In the preliminary design phase, in 1961, eight types of bridges (Figs. 1 and 2) as well as a tunnel were studied. Of the various bridge types considered feasible for the crossing, one double-level structure and two single-level structures were chosen for further study, to determine feasibility, costs and compatibility with the surroundings.

From the standpoint of appearance and economy, a three-span tied arch truss was selected as the most suitable of the double-level structures. However, all the double-level structures for this crossing had the disadvantage of steep approach grades on the Illinois side. An elevated railroad trestle approach to an adjacent bridge limited the grade change for the Poplar Street upper deck to such an extent that a 3.9 percent grade was necessary for a distance of 800 ft. Also, as in all double-level structures, with due respect to the success of some designers in handling details to improve the appearance of the lower deck, a "tunnel effect" is developed on long bridges, which has psychological disadvantages.

Of the single-level structures considered, two types were selected as having the best possibilities, and both of these were deck structures. A shallow depth deck truss layout was developed, but the resulting grades of 3.6 percent were considered undesirable compared with grades developed on the shallower depth deck girder bridge. A conventional girder bridge of unprecedented 600-ft span was developed, but the weight of steel was very high. Efforts to reduce the weight by use of lightweight roadways, welded construction and prestressed steel were effective, but cost was reduced only slightly from the conventional girder bridge.

In a further attempt to retain the clean lines and favorable grades of the girder bridge, consideration was given to orthotropic plate deck design. It was found that the weight of steel in a bridge of this type is much less than in a conventional deck girder or deck truss bridges.

Table I illustrates estimated costs of the various bridges studied.

Table I. Bridge Cost Comparison

Substructure and Superstructure for Eight-Lane Bridge of 2175 Linear Feet

Double-level bridge	
Arch truss	\$11,939,000
Single-level bridges	
Deck truss	\$13,378,000
Deck girder	\$14,770,000
Orthotropic deck plate girder	\$11,998,000

E. J. Shields is Project Engineer, Sverdrup & Parcel and Associates, Inc., St. Louis, Mo.

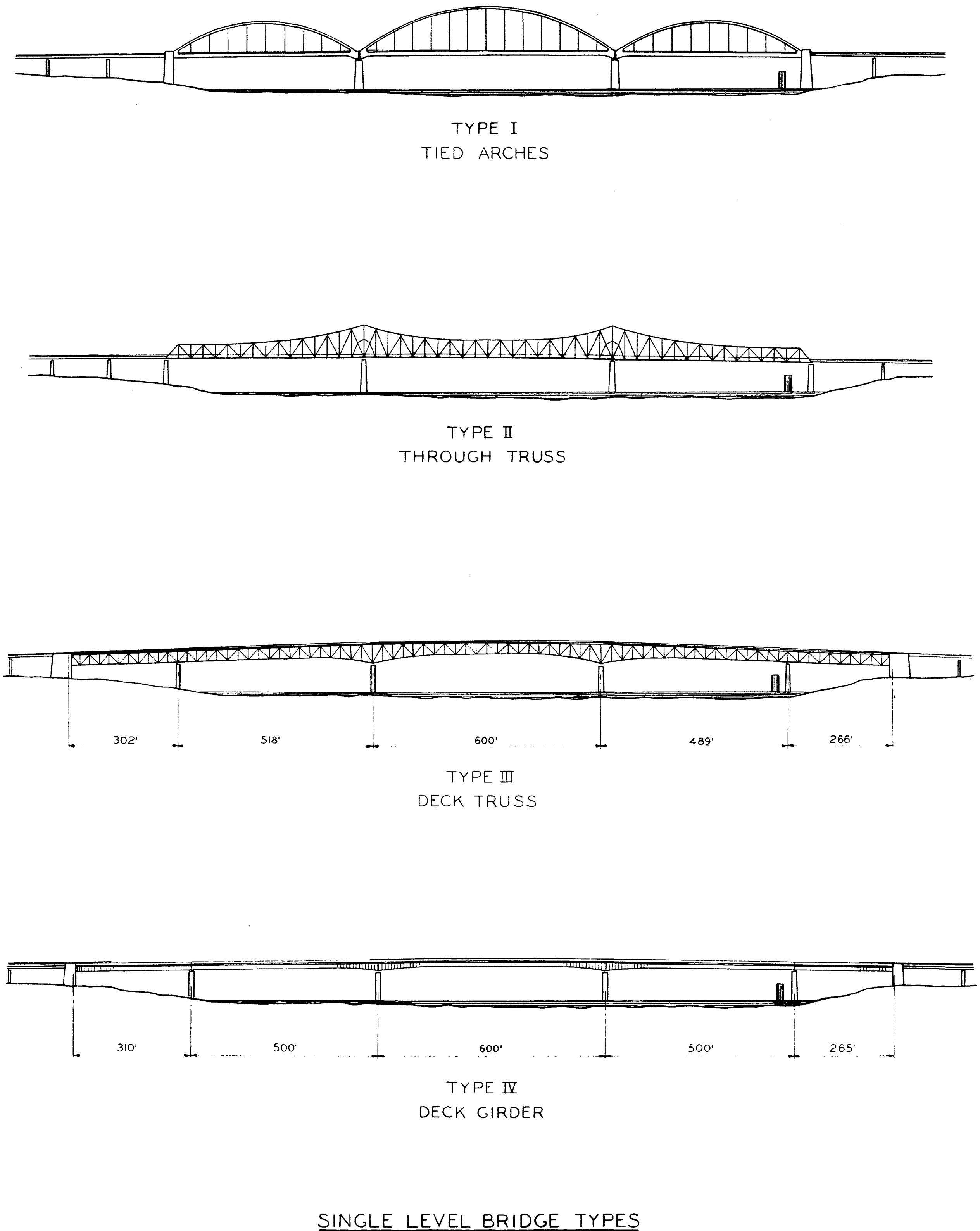

Fig. 1. Single-level bridge types considered

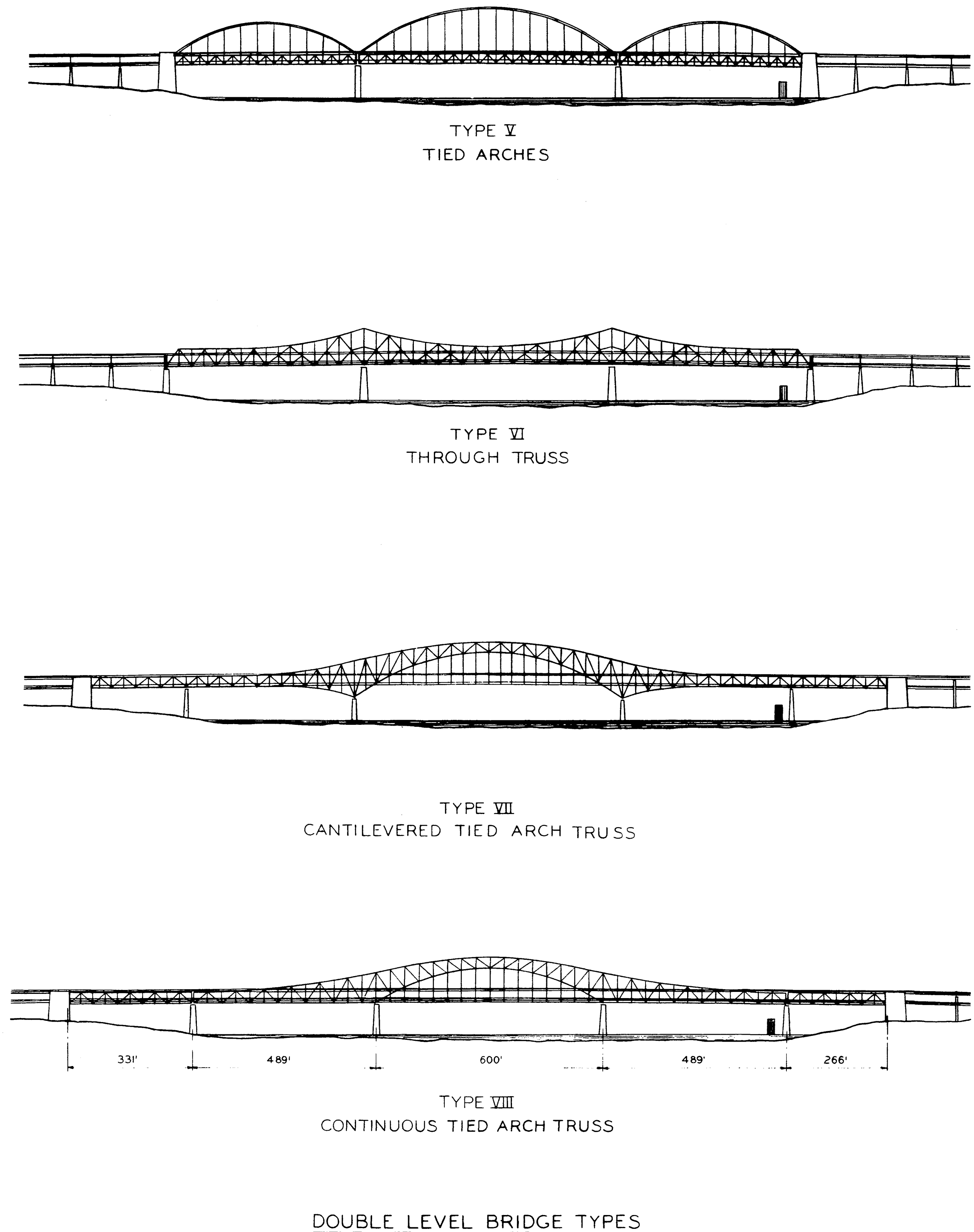

Fig. 2. Double-level bridge types considered

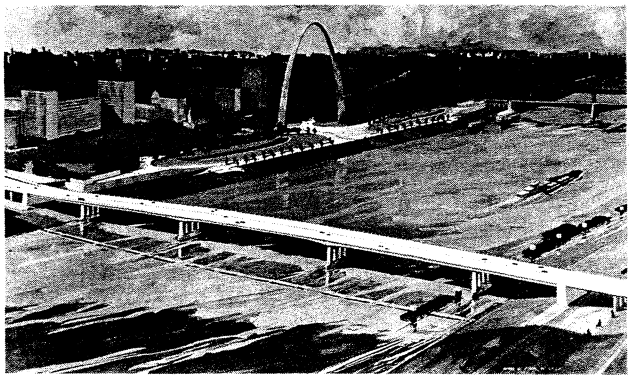

Fig. 3. Perspective of Poplar St. Bridge with arch in background

The orthotropic deck bridge had the lowest estimated cost of the single-level types and, within the degree of estimating accuracy based on preliminary design, could be considered as low in cost as the double-level arch truss type. Therefore, it was evident that the orthotropic deck plate girder bridge provided the easier grades of a single-level structure, and a cost as low as that of any other type considered. This, combined with the aesthetic advantages of the orthotropic bridge (Fig. 3), led Sverdrup & Parcel to recommend its adoption.

The States of Missouri and Illinois, and the Bureau of Public Roads, had previously authorized inclusion of the orthotropic design in the preliminary engineering report when its advantages were indicated. In view of the favorable costs and the desire of all parties to achieve a suitable and attractive design, the recommended bridge was adopted.

FINAL DESIGN

Spans of 300, 500, 600, 500 and 265 ft were used for the five-span, 2165-ft continuous structure (Fig. 4). The 600-ft span was required for navigation interests, fixed at the center of the river in alignment with adjacent bridge crossings. The 500-ft Illinois span was made a little larger than would be desirable for balance with the 600-ft central span in order to meet the requirements of an industrial interest operating on the river along the Illinois bank. To retain symmetry, the span on the Missouri side was also made 500 ft. As it was desirable to locate the face of the Missouri bank pier in line with the high walls of the National Park, the bank span was made 300 ft in length. The corresponding span on the Illinois side was restricted on the east by railroad tracks, making it 265 ft long.

Grades were set to provide 55-ft vertical clearance above the 2 percent line at the edges of a 500-ft channel. With a girder depth of 17 ft in the center span, grades of 1.54 percent were developed with a 600-ft vertical curve centered on the central span. 16-ft deep girders were sufficient in the flanking spans and were continued on to the end piers.

Rock is 35 ft below the top of ground on the Missouri bank and gradually slopes downward toward the Illinois shore where it is 135 ft below ground line (see Fig. 4). All the piers were founded on rock; Pier 2 rests directly on rock, and the others are supported on 6-ft diameter cylindrical caissons to rock.

A cross section of the bridge (Fig. 5) shows four-lane roadways for each direction of traffic, separated by a median barrier. The 113-ft width is divided into dual decks, each supported by two girders. Some orthotropic bridges in Europe with superstructures of this width (for six lanes of traffic, bicycle paths and walks) have utilized a single deck supported by two large box girders. With the dual-deck design selected for Poplar Street, girders can be fabricated in the shop ready for field erection, while with the single-deck design, top and bottom flanges and webs would probably be shipped knocked-down and field assembled during erection. It was considered an economic advantage to use four box girders for this bridge.

The relatively short-span floor beams framed into rigid box girders result in inherently stiff transverse members. Since this stiffness permits little distribution of loads in the longitudinal direction to adjacent floor beams, floor beams had to be designed for a high percentage of the wheel loads imposed on them. This was true for open and closed systems of stiffeners. The buckling characteristic of vertical stiffening elements used in an open system, such as plates, bulb flats, tees, etc., generally restrict floor beam spacing to 5 or 6 ft. A closed system of

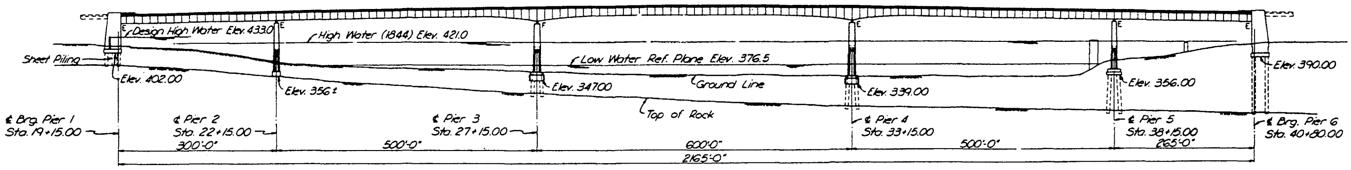

Fig. 4. General elevation

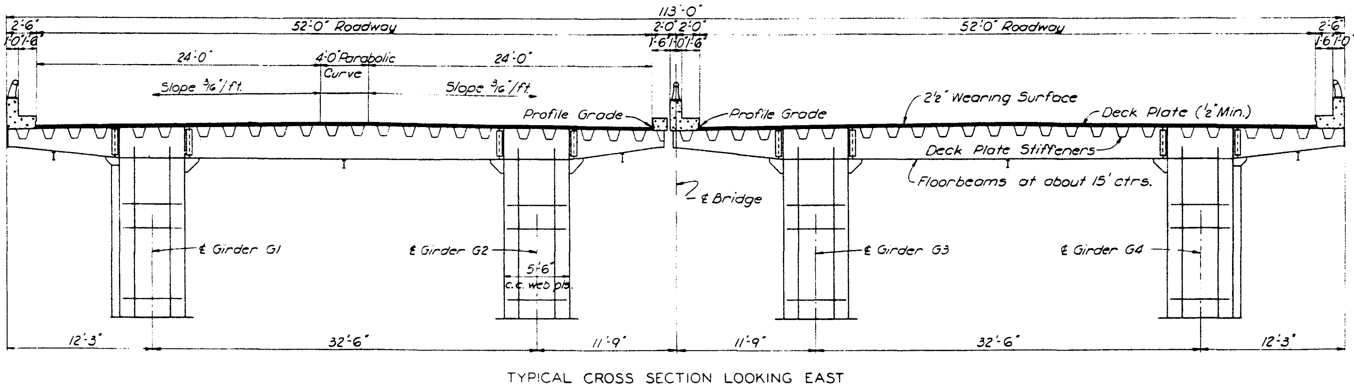

Fig. 5. Typical cross section

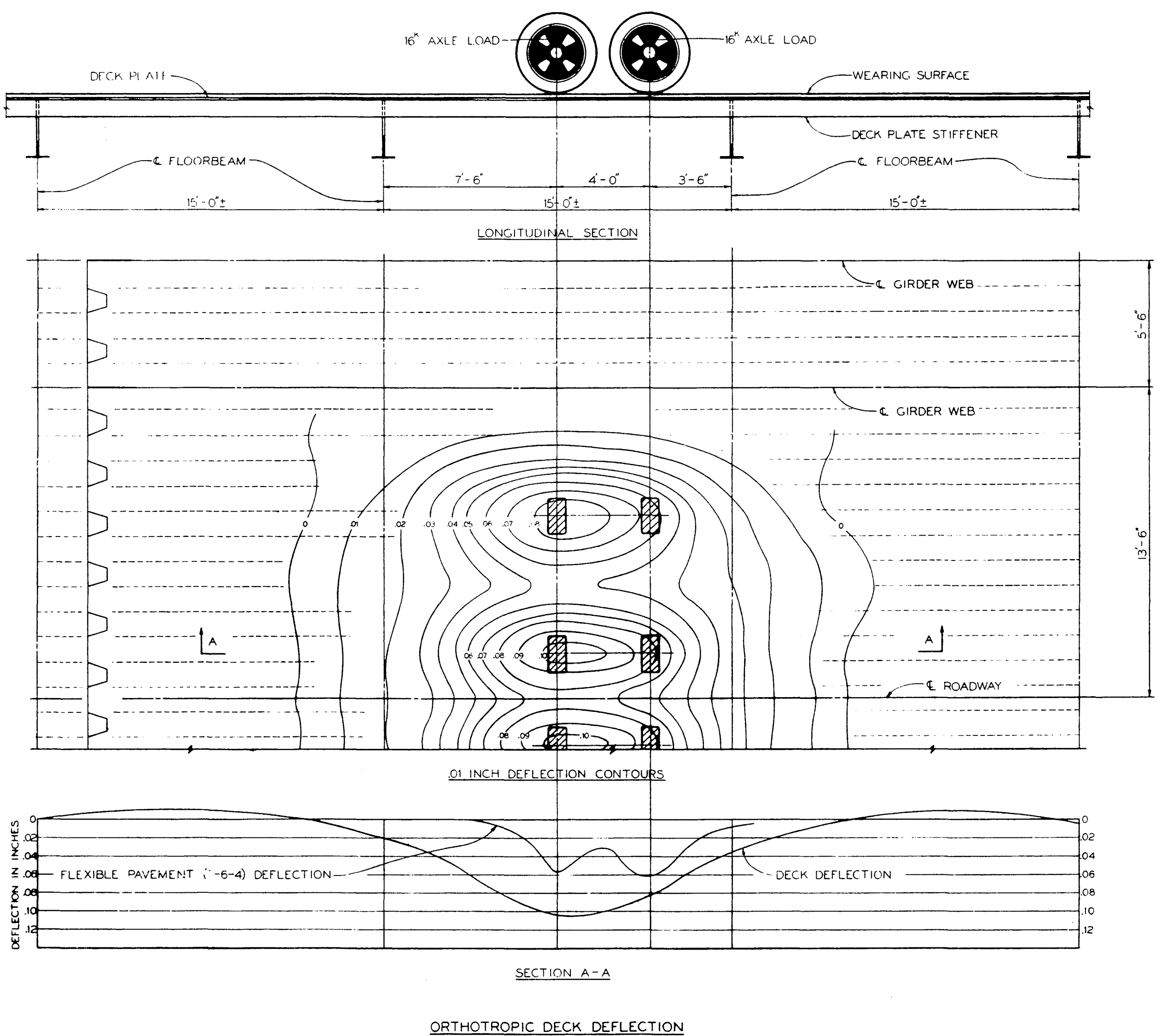

Fig. 6. Deck deflections—longitudinal

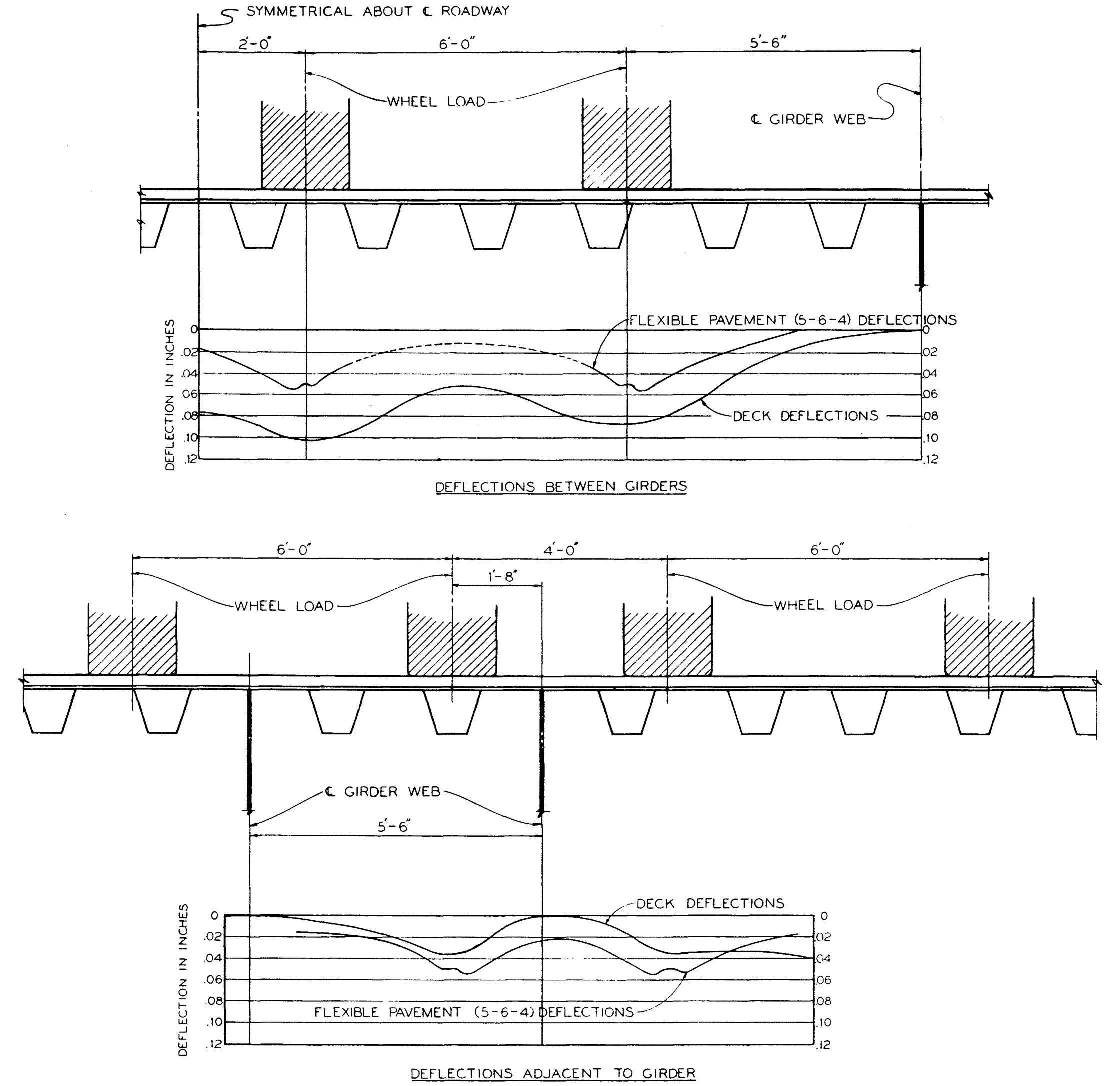

Fig. 7. Deck deflections—transverse

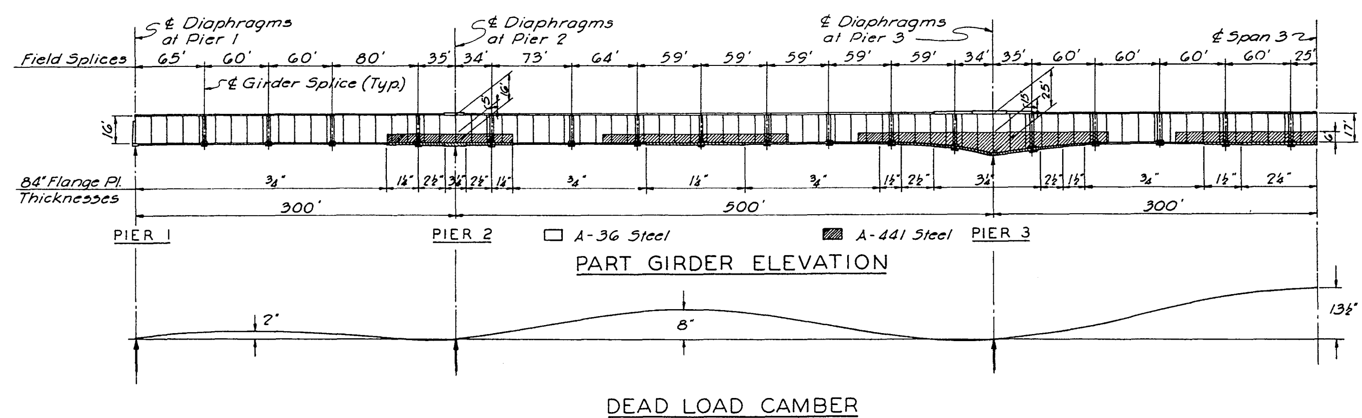

Fig. 8. Girder elevation

stiffeners such as vees, trapezoids or boxes with a circular lower surface combines better properties against buckling with high moment capacity and permits greater spacing of floor beams.

The torsional rigidity of a closed system also provides better transverse distribution of wheel loads, which is not only structurally desirable but also advantageous to the life of a wearing surface. Several designs with both types of systems were investigated, and for this bridge it was found that an appreciable saving in weight of steel could be realized with closed stiffeners of trapezoidal cross section, with floor beams spaced at 15-ft centers. The deck is composed of: a $\frac{1}{2}$-in. minimum thickness deck plate; $\frac{5}{16}$-in. stiffeners, 10 in. deep, $6\frac{1}{2}$ in. wide at the bottom and 13 in. wide at the top, spaced at 26-in. centers; and 30-in. deep floor beams which are welded to the underside of deck plate and stiffeners. Deck plate and stiffeners are weldable low-alloy, corrosion-resistant steel; and floor beams are of weldable low-alloy steel.

The deck plate and stiffeners function as the top flange of the girders and are stressed by dead and live loads as in any conventional girder. The deck plate, stiffeners and floor beams also act as the bridge floor, and local stresses produced by wheel loads combine with stresses caused by girder action. The permissible unit stress for this combined condition was arbitrarily set at 112 percent of normal unit stresses, amounting to an allowable stress of 30,400 psi for low-alloy steel. This increase in stress is considered justified because of the localized effect of the deck loads, and because of the improbability of deck and girder stresses attaining maximum values at the same location. At certain points in the Poplar Street Bridge, the maximum girder stress at the bottom of the deck stiffener is in the range of 18,500 psi. The bending stress at the bottom of the deck stiffener due to floor action under wheel loads is 11,700 psi. Due to the unsymmetrical deck section, stresses at the top surface of the deck plate from floor action are considerably under this figure. In the transverse direction, stresses in the deck plate are also low.

DECK DEFLECTIONS

Deck deflection under wheel loads is of interest, especially in connection with wearing surface design. Fig. 6 shows the position of wheel loads (H20-S16) on the deck. In the structural design, loads were placed in this position to compute stresses in the deck plate and stiffeners in the longitudinal and transverse directions. In conjunction with these computations, deflections were computed in the transverse and longitudinal directions and the values combined to produce the pattern of contours shown in Fig. 6. In computing deck stresses, the Interstate loading of two 24,000-lb axles plus 30 percent impact was used. The maximum deflection was, of course, under the wheel load mid-way between floor beams and amounted to 0.10 in. measured in a large bowl extending about 25 ft between points of zero deflection in the direction of traffic. Deflection is also compared in Fig. 6 with the deflection which takes place under the same loads on a 5-6-4 flexible pavement, which is considered a long-life pavement. The deflection bowl for this is illustrated in the AASHO Road Test Report 5, *Pavement Research*, and the deflection curve shown in Fig. 6 is taken from the contours given in that report. The degree of bending in this direction is much greater in the flexible pavement than on the orthotropic deck.

A transverse section (Fig. 7) taken through the deck under the same wheel loading used in Fig. 6, shows the deck deflecting in a general bowl shape; but due to the relatively low stiffness in this direction the effect of the individual wheels is apparent. The greatest deflection is again 0.10 in. near the center of roadway and is 0 and approximately a horizontal tangent at the girder web. The degree of bending in the deck is comparable with deflection in a 5-6-4 flexible pavement.

Another transverse section through the deck near the girder web is illustrated in Fig. 7 under the condition of live load shown, wherein truck wheels are located in a position to produce the greatest degree of bending over a girder web. As illustrated, the deflection is only about 0.035 in. under the wheels. Although the deflection is small, this degree of bending in a wearing surface is relatively high. It was only in these areas on the bridge deck that we found convex bending of such magnitude that conventional asphalt concrete pavement would not be expected to have a long life if used.

Flexible pavement deflections reported in these AASHO Tests were measured at moderate temperatures and would no doubt be less under winter conditions with frozen or more stable subgrade, while deflections on the bridge deck in cold weather would be modified only slightly by increased stability of the wearing surface. Therefore, for an asphalt concrete wearing surface to be used, it was considered necessary to provide greater flexibility than is afforded by conventional material.

A limited program of subjecting various wearing surfaces to traffic and weathering is under way and, therefore, decisions on the wearing surface for the Poplar Street Bridge will be made at a later date.

GIRDERS

A web thickness of $\frac{5}{8}$ in. was maintained in the box girders throughout the entire length of the bridge by providing stability through the use of multiple longitudinal and transverse stiffeners and girder diaphragms. Although low-alloy steel is used in the deck plate for strength and atmospheric corrosion properties, the stress in the deck plate derived from girder participation alone is within A36 steel stress capacity. As the stress is the same at the top of the web, A36 steel is used in the webs. Over the piers and near the center of the three long spans, A441 steel web plates are used near the bottom flange in lieu of A36 steel due to stress requirements (see Fig. 8).

The bottom flanges of the girders are composed of A36 and A441 steel plates, 84-in. constant width for the 66-in. web spacing, and varying in thickness from $\frac{3}{4}$ to $3\frac{1}{4}$ in. All shop splices will be butt welded, and high strength bolts will be used in the field splices.

In the preliminary stages of design, it was anticipated that considerable use could be made of heat-treated, high-yield strength steels in the bottom flanges and in the lower portions of the girder webs. The allowable stress range of this steel resulted in very desirable sections, and indicated economy in construction. However, deflections from live load plus impact would have been excessive by American standards, being in the neighborhood of 1/360 of the 600-ft span. It was recognized that the usual requirement of limiting deflections to 1/800 of the span, which has been established by AASHO for spans under 400 ft, would be too severe a restriction for this multiple-lane, long span structure, and the following criteria were agreed to by the States of Missouri and Illinois and the Bureau of Public Roads: (1) with the loading used for design (H20-S16 lane loading), deflections were not to exceed 1/650 of the span; (2) with an alternate loading of H15-S12 lane loading in the span under consideration and 25 percent thereof in all other spans, deflection was not to exceed 1/1000 of the span. These criteria were easily met by proportioning the girders for the allowable stress ranges of low-alloy and structural grade steels.

The dead load deflections amounted to $13\frac{1}{2}$ in. for the 600-ft span, about 8 in. for the flanking spans, and 2 in. for the end spans. In fabrication, the girders will be cambered to compensate for the dead load deflection values.

FABRICATION AND ASSEMBLY

The bridge is to be fabricated into as few erection pieces as practicable (see Fig. 9). In general, it will consist of girder units approximately 60 ft in length, deck units about 27 ft wide by 60 ft in length fitting between the girders, deck units about 9 ft wide and 60 ft in length cantilevered from the edge of the girders, and cross frames at about 60-ft centers. The girder sections in excess of 17 ft in depth are detailed with permissible field welded longitudinal web splices to accommodate rail shipment. Also, the 27-ft wide deck units have two longitudinal welded joints in the deck plate, which if field welded, would require full length floor beams field welded to the underside of the deck plate and stiffeners. If this welding is to be done in the field, it will necessitate a subassembly yard or dock located at the site or at a rail-to-barge transfer point. The contractor may make these welds in the shop, and no doubt will do so if there is access to water shipping from the fabricating plant.

Since there are 20 miles of trapezoidal pans comprising 1500 tons of steel, considerable savings can be realized if the pans are formed in one continuous operation for the full length of each deck unit (60 ft ±). However, the deck plate stiffeners may be formed to the trapezoidal shape by a conventional brake-press, which has a limited length of about 15 ft. If the brake-press is used, the short-length pans will have to be shop butt-welded prior to assembly with the deck plate. These welds will be permitted near the points of contraflexure in the 15-ft span between floor beams where the stress in the pan is low and the range of stress change will not cause a fatigue problem.

Where practicable, welds will be made by automatic submerged arc welding in accordance with the recent issue of the American Welding Society Specifications, which will be supplemented with Special Provisions where necessary. Joint preparation will not be shown on the plans, and joints not pre-qualified by the American Welding Society will be qualified by the contractor. All shop splices will be welded and all field connections will be made with high strength bolts except for the deck plate and stiffener field joints which will be welded.

Special attention will be given to the shop welding of the stiffeners to the underside of the deck plate. This welding, which amounts to 40 miles, is principally a shear weld, but flexural stresses are developed in the webs of the stiffener in resistant transverse bending in the deck plate under wheel loads. These flexural stresses can alternate in direction depending on position of the wheel, but the stress range is low. The stiffener pan to deck plate weld has to be made from one side, and backup plates within the pans are impracticable. The plans specify that 80 percent penetration is to be achieved and the contractor will determine joint preparation and procedures necessary to qualify the joint.

The girder diaphragms, which are located at the floor beams, are composed of flanged plates forming two large openings for access through the girders. The diaphragms provide stability to the girder webs and torsional rigidity to the girders, and (in conjunction with the floorbeams and the intermediate cross frames) distribute live loads fairly equally to the girders. ST 7 WF sections are used for longitudinal web stiffeners; one stiffener is used at some locations and as many as four in the haunched areas. The stiffeners are discontinuous at the girder diaphragms.

The girders are spliced at convenient locations making the girder units as near 60 ft in length as practicable. The average length for the 34 units is 64 ft, with the shortest length 50 ft and the longest 80 ft; changes in cover plate thickness and position of girder diaphragms

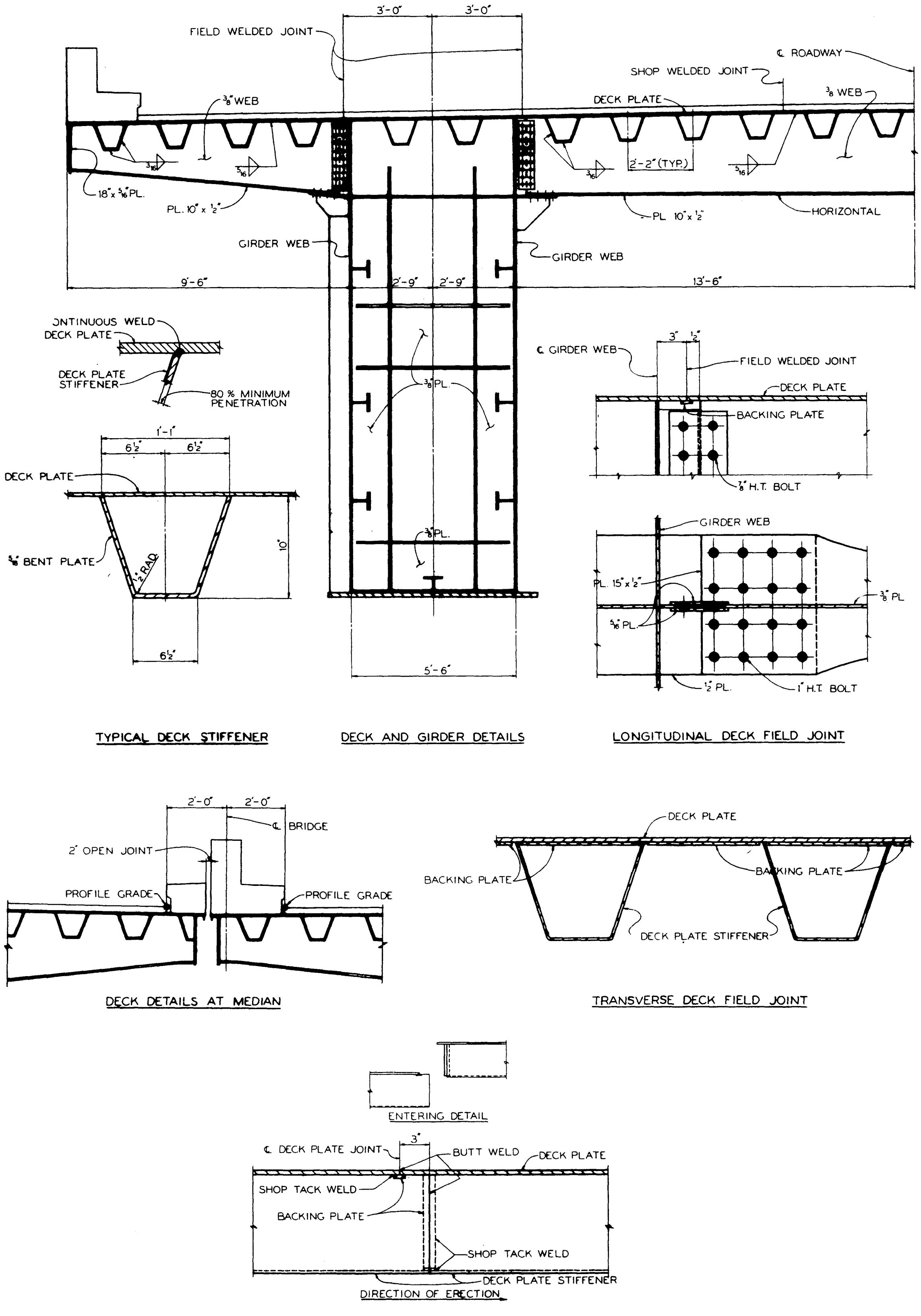

Fig. 9. Deck sections

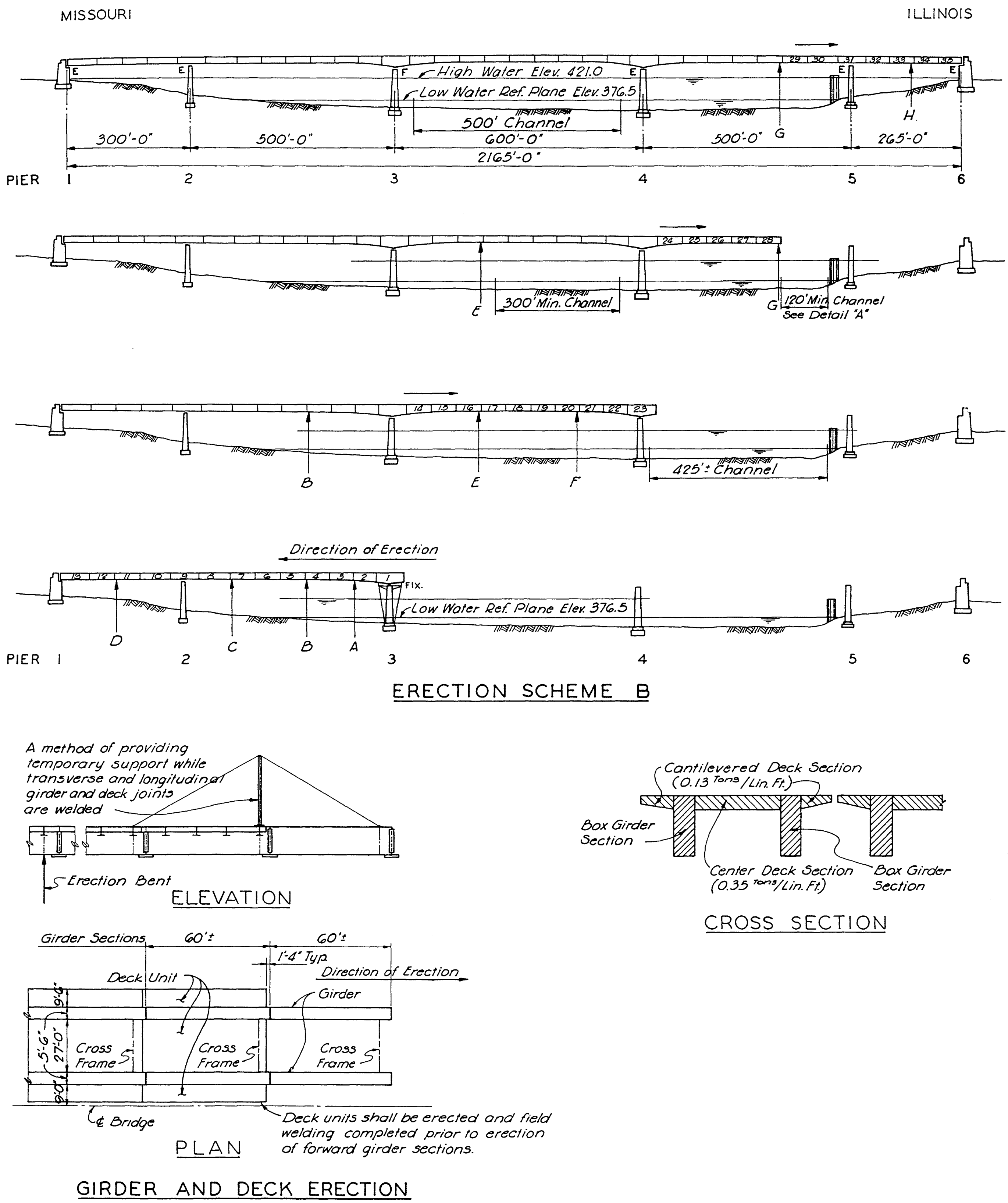

Fig. 10. Erection schemes

account for the variations. The girder splices are proportioned for direct transfer of stress across the members and are in accordance with the requirements of AASHO Specifications for capacity of section. This requires many lines of 1-in. high-tensile bolts in the web splices.

The girders are to be assembled in the shop to the cambered position, using a minimum of three sections at a time. With the splice plates in place, the subdrilled holes for the bolts in the web and bottom flange splices will be reamed. The deck units are to be end-matched with the mating section in assembling the deck stiffeners on the deck plate. Jigs and other means are to be used to obtain squareness, correct dimensions and position of the component parts. Since a number of floor beams are attached to one deck unit, and these have end connections to brackets on the girder, a permissible tolerance is established for the connection. It is recognized that this type of fabrication is not normally done in a structural shop. It is desirable that the deck units be shop assembled with the girders to check the fit of the floor beam connections and welded field joints. However, with sufficient provisions for field fitting of the field welded joints, the contractor may choose to erect the deck units without the benefit of shop assembly. In either case the floor beam connections are to be subdrilled and reamed assembled, and a tolerance of $\frac{1}{4}$ in. is permitted in the throat opening of the joints to be field welded.

The concrete curbs, median barrier, parapets and tube raillings will be placed as soon as steel erection permits, but under a separate contract. The concrete curbs will be anchored to the deck plate with welded studs; and at the face of the curb, a vertical plate welded to the deck plate will also anchor into the concrete. This plate will provide a positive seal by preventing water on the roadway surface from seeping under the curbs. For corrosion protection of the deck plate under the curbs, due to possible entrance of water through the curb and curb joints, the steel deck plate will be coated with epoxy coal tar. Single-tube railings will be provided on the parapets and the median barrier.

CONSTRUCTION

Most of the bridge can be erected by use of floating equipment, even during low stages of the river, and the extreme ends of the bridge may be erected by land-operated equipment. The girder sections over the channel piers are 70 ft in length and weigh 113 tons. Other girder sections weigh as little as 54 tons.

Since the full width of the deck plate is considered the top flange of the girders in this type of bridge, it is imperative to erect the deck plate as the girder erection proceeds. In order to obtain about the same stress in girder and deck sections after erection, it is required that the cantilever girder sections be supported in a substantially "no stress" position until the girder and deck sections are field welded.

One scheme of erection is shown in Fig. 10 to illustrate a method of providing for navigation and other interests during erection. The contractor is to keep a 300-ft minimum width navigation channel open at all times during construction. This channel can be in either the center span or the Illinois span, both of which have sufficient depth for navigation at the lowest river levels. As erection proceeds, the channel can be shifted from one span to the other when the need arises. An additional requirement is that a 120-ft channel be provided along the Illinois shore for use by a rail-to-barge coal loading facility which stores barges in the area under the proposed bridge in connection with its loading operation.

The plans include two schemes of erecting the bridge in order to convey to prospective bidders methods of erecting the bridge which require no additional steel incorporated into the structure. However, by this it is not intended to limit bidders' initiative in devising their own scheme. The contractor also has his choice of erecting one superstructure unit before the other, or of erecting both units of the dual-bridge at the same time.

In addition to the substructure contract which was awarded recently, the States intend to divide the remainder of work into four contracts, as follows: (1) fabrication and erection of structural steel, (2) field painting, (3) deck concrete, bridge rail and lighting, and (4) roadway wearing surface. Completion of the project is anticipated in the summer of 1966.

ACKNOWLEDGMENTS

Acknowledgment is made of the assistance given by Mr. D. B. Jenkins, Bridge Engineer of the Missouri State Highway Department; Mr. E. L. Sherertz, Engineer of Design and Mr. W. E. Baumann, Engineer of Bridge and Traffic Structures of the Illinois Division of Highways; and representatives of the Bureau of Public Roads.

Appreciation is extended to the American Institute of Steel Construction and the many steel producers, fabricators and erectors with whom we have consulted.