DESIGN AND CONSTRUCTION OF AN ALL WELDED PLATE GIRDER HIGHWAY BRIDGE IN KANSAS

GEORGE W. LAMB, *District Engineer, American Institute of Steel Construction,* AND
E. S. ELCOCK, *Bridge Engineer, State Highway Commission of Kansas*

SYNOPSIS

This paper reports on the design and construction of an all welded deck plate girder highway bridge having spans of 84-108-108-84 feet. It was designed in competition with other designs using wide flange (WF) beams and riveted plate girders. Estimated weights showed a saving in metal of 18 percent under the riveted plate girder and 24 percent under the WF beams designed for stress alone Savings under a WF beam design which met stringent specifications limiting live load deflection to 1/800 of the span were 54 percent.

The design procedure is described, omitting the stress analysis.

Welding and erection procedures are described

The need for more complete design specifications for welded bridge structures is suggested.

The use of welding and continuous girder designs in Kansas bridges were developed concurrently. The first continuous bridge designed after formation of the Highway Commission in 1929 was built in 1932; it was of equal spans of 70 ft. using 36-in. wide flange (WF) beams and had no welding on it. The beam section was chosen to meet the maximum moment at the supports and the same beam section was used throughout. On the next designs steel was saved by selecting the beam section to accommodate the smaller moments at the center of the spans and reinforcing this beam section with welded cover plates in the high moment regions over the supports. These designs also permitted the stiffeners to be welded to the web as an alternate for riveted stiffeners.

The use of welding was increased without much information, other than visual inspection, being available on the results obtained in previous structures. In order to obtain additional information a full size all welded test girder was constructed and tested in 1934. This girder had a 27-ft. span and was 54 in. deep. It was constructed entirely of plates welded together. It was loaded beyond the yield point of the base metal and failed by lateral buckling with no indication of failure in the welds. This test was described in Engineering News Record, September 19, 1935. As a result of this test, an all welded viaduct was designed in 1935 incorporating some of the details used on the test girder. Approval of this all welded design however could not be obtained, and the structure was redesigned and constructed with the main girders riveted but with the floor beams, stringers, brackets, expansion joints and railing welded.

During the next 10 years welding was used extensively in rather important details but was not used in the make-up or splicing of main girders

Immediately after World War II another attempt was made to obtain approval of an all welded design and this effort was successful. The bridge selected was on US 77 about 9 miles south of Augusta, Kansas, over the Little Walnut River (Fig. 1).

DESIGN

The traffic, the location and the stream required that the bridge be of 26-ft. roadway designed for an AASHO Loading of H 20, with a length of from 350 to 400 ft. and having intermediate spans of about 100 ft. This span of 100 ft. or slightly more was also well in the economical range when considered in conjunction with the height of piers and the foundation conditions.

A study was made of several different designs First a continuous rolled beam layout was designed, having spans of 88-103-103-88 ft. Next a continuous riveted plate girder with spans of 84-108-108-84 ft. was studied and then the continuous welded plate girder with spans of 84-108-108-84 ft. was designed for comparison

The rolled beam layout using 36 in. WF beams had an estimated quantity of steel of 412,000 lb. and complied with the specifica-

tions for stress but would not meet the specifications limiting live load deflection to 1/800 of the span. When designed to meet this requirement for live load deflection the estimated weight of steel in the rolled beam layout was 680,000 lb. This large increase in weight required in order to comply with the deflection requirement indicated that a greater depth of girder was needed to reduce the deflections to the specification amount and have an economical use of material. A plate girder having a depth of 48 in. was selected as having sufficient depth to prevent the live load deflections from governing the design of the span, rather than unit stress.

The riveted plate girder had an estimated weight of 385,000 lb. and even though it involved a large increase in the fabrication cost, it was considered to be more economical than the rolled beam design, especially the design meeting the requirements of the specifications for live load deflection.

The 48-in. depth of the riveted plate girder proved to be sufficient to accommodate the specified deflection limitation and this depth was also used for the welded design. The welded plate girder had an estimated weight of 314,000 lb. which gave sufficient advantage in weight to make it appear very feasible for use and, with the knowledge and confidence in welding gained during the war, a welded design was considered to be well worth consideration again.

The riveted and welded plate girders studied had four lines of girders while the WF beam design had five lines of beams. Two girders with floor beams and stringers would have been somewhat more economical of metal in the riveted or welded girder designs but sufficient confidence in welding had not been gained to obtain approval of a two girder welded design.

The welded girder was designed using as many of the advantages of welding as possible; in other words it was not a modified riveted design. The girder make-up was as simple as possible and consisted of an upper and lower flange plate and a web plate. The thickness of the flange material was set at $1\frac{1}{8}$ in. to avoid preheating of the metal prior to welding. In order to simplify fabrication parallel flanges were used; that is, there were no haunches at the points of intermediate support (Fig. 1). As the flange plates were originally set at $1\frac{1}{8}$ in. maximum thickness the section of girder to accommodate the moments at the intermediate supports was obtained by making the flange plates 22 in. wide, while the smaller moments between supports could be accommodated by a 12-in. width of flange (Fig. 2). This is equivalent to haunching the girders

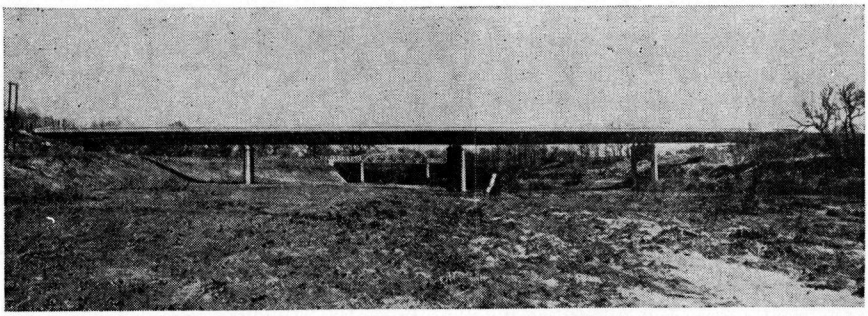

Figure 1. Welded Plate Girder Bridge

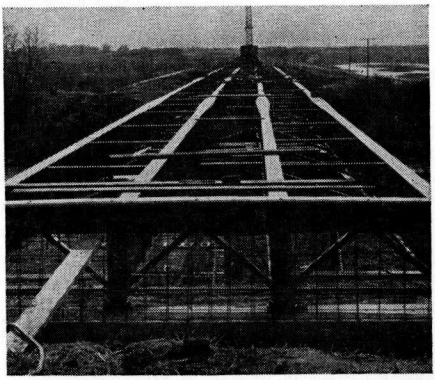

Figure 2. Widths of Flange

horizontally instead of vertically and it offered the simplest solution that could be thought of at the time Summarizing, the flanges as designed were of constant thickness, the top and bottom flanges were parallel and the width of the flange was wider at the piers than in the center of the spans.

The girders were not cambered for dead load deflection, a smooth floor surface being obtained by using a variable concrete fill over the girders to accommodate the calculated dead load deflections and any construction errors that might occur. An additional correction for vertical curvature had to be incorporated in the depth of this concrete fill over the girders as the girders themselves were built as straight members between splice points with a small angular correction at the splice point to fit the vertical curvature.

The splice points were located near the points of zero dead load moment because the flange stresses are zero during erection and are always well below the allowable under live load, also, when the structure is erected without falsework, the changes in slope of the adjacent girder sections on each side of the splice are numerically equal and the fit up of the splice is as good as if the girders were supported throughout their length

As a 22-in flange width was required over the piers, while in the center of the spans a 12-in flange was sufficient, the splice was made 12 in. wide and the 22-in flange was trimmed on a taper along both edges to this width at the splice (Fig 2) The distance that this trimmed edge should be carried back toward the piers was next considered, and it was decided that in order to conserve material and to provide a rather long transition from the 22-in width to the 12-in width that a trimmed edge 19 ft long would provide a desirable transition and use a minimum amount of material The material trimmed from the 22-in plates was considered to be of no value in this structure; whereas as scrap it could be remelted and used in another structure. Further, the long taper has a better appearance than a short taper (Fig. 2)

The web thickness required by the specifications was slightly under $\frac{3}{8}$ in as determined by the ($\frac{1}{70}$) (d^4) thickness clause of the specifications, so a 48- by $\frac{3}{8}$-in web was selected. A web thickness of $\frac{5}{16}$ in was used for the riveted girder because the depth is calculated as the distance between the toes of the flange angles During the design stage it was thought that the fabricator could obtain 48-in sheared plates that would be straight enough to be used without straightening For this reason no camber was specified

The fabricator however ordered plates 3 in wider than nominal and flame cut the plates along both edges to the nominal width This made the fit up of the girders very good but we do not know if the expense could be justified on other jobs

The design of the intermediate web stiffeners was one of the most troublesome points of the design. The specifications, written for riveted girders, specified pairs of angles Bars are the most efficient stiffeners for a welded design A ruling was made that the stiffeners should be in pairs and extend to within $\frac{1}{2}$ in. of the edge of the flanges As the flanges were 22 in wide near the supports and since the thickness of the stiffeners was required to be $\frac{1}{16}$ of their width the stiffeners near the piers were required to be pairs of 10- by $\frac{5}{8}$-in plates which were much larger than required for web stability and involved a considerable tonnage of steel A saving could have been made by using stiffeners on only one side of the web and proportioning them to meet the requirements for web stability.

The intermediate stiffeners were welded to the webs with intermittent welds $\frac{5}{16}$- by 2-in. at 6-in centers except at the ends where a length of continuous weld was used sufficient to develop the bracing. The reaction stiffeners were pairs of bars continously welded to the webs and welded to the bottom flanges

The web to flange welds were originally designed as $\frac{3}{16}$ in continuous fillet welds These welds were sufficient to develop the shearing stresses but the American Welding Society specifications require a minimum size of $\frac{5}{16}$-in fillets on material $1\frac{1}{8}$ in thick in order to avoid weld cracking. These welds were then redesigned as $\frac{5}{16}$-in. intermittent welds Finally, however, these welds were made $\frac{5}{16}$ in. continuous in order to provide a continuous flange to web connection and for water proofing purposes The increase in weld metal involved in these changes is about 200 percent We believe that a technique could be worked out that would permit the smaller size weld

The field butt welds in the flanges were

bevelled so that 15 of the 20 passes required to make this weld would be in the downhand position. Extension plates (Fig. 3) were used in making these butt welds, they were removed at the completion of welding the splice and the edge of the flange ground smooth (Fig. 4). In order to complete the butt weld of the flanges at the intersection of the web and flange, the web was coped out with a ½-in. radius semicircular cut. This was sufficient to allow the passage of the welding rod through the ⅜-in. web, this cope was later filled with weld metal (Fig. 4).

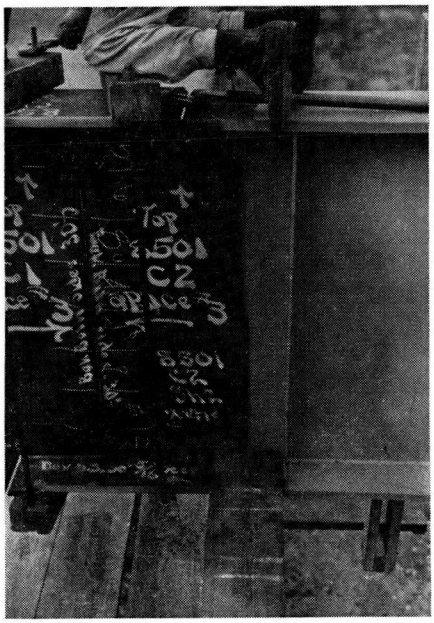

Figure 3

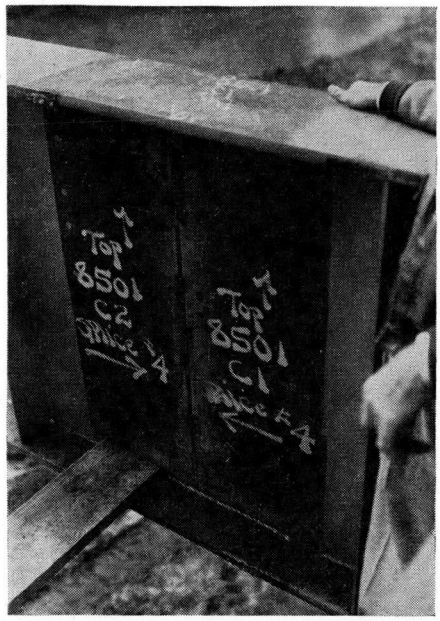

Figure 4

The bracing used between girders consisted of 4-by 4-by ⅜-in. angles welded to the girder stiffeners. The bottom angle of this bracing extended across the four girders just above the lower flange. Top angles were used only in the outside bays. A diagonal strut was placed in the outside bays sloping down toward the outside. The direction of the diagonal strut was selected so that it would be in tension under dead load, the dead load deflection being greater for the exterior girders. These cross frames were spaced at a maximum distance of 20 ft. apart.

The design plans did not specify the welding sequence to be used in shop fabrication or for the field welding. The choice of a suitable sequence being a responsibility of the contractor. The fabricator was required to place on the shop details the sequence that was to be used. This was checked for compliance with good practice and required as a matter of record. The sequence shown on the shop plans was successfully followed without any major changes. The sequence appearing on the shop details was as follows.

PROCEDURE FOR WELDING GIRDERS

After girder sections are completely assembled they will be placed with webs in horizontal position and the stiffeners on the top side welded to the web as follows.

The intermediate stiffeners will be welded to web by starting at top flange and proceeding toward the bottom flange, the intermittent welds being placed on each side of the stiffener at one location before proceeding to the next location.

The bearing stiffeners will be welded to the web by starting at center of web and proceeding toward the bottom flange on one side of stiffener. Then return to center and proceed toward top flange on opposite side of stiffener. Return to center and proceed toward bottom

flange on side of stiffener opposite weld first made. Complete welding of stiffeners to web by returning again to center and proceeding toward top flange on side of stiffener opposite second weld made.

The girder will then be turned over and stiffeners welded to web using same procedure.

The girder will then be placed on the bottom flange with web vertical and bottom flange welded to web by using an operator on each side of girder and making the fillet welds on each side simultaneously. On the girder section these welds will be made by starting at center of girder section and proceeding toward one end then return to center and proceed toward opposite end. After bottom flange is completely welded to web the bearing stiffeners will then be welded to bottom flange. The girder section will then be placed on the top flange with web vertical and the top flange welded to the web, using the same procedure as for bottom flange.

TABLE 1

Heat No.	Carbon (C)	Manganese (Mn)
	%	%
11613	0 30	0 44
5873	0 26	0 49
1234	0 25	0 43
10647	0 29	0 46
192859	0 27	0 47

The size and type of all welds shall be as shown on drawing and shall be made by the shielded arc process.

PROCEDURE FOR WELDING FIELD SPLICES

After a field joint has been completely assembled the web shall be butt welded in 6-in. increments using sequence shown on each side (Fig. 3). The bottom flange will next be butt welded using sequence shown. After bead number one has been made completely across the flange including extension bars, the underside of bead shall be chipped out before bead number two is placed. The top flange shall next be welded using the same procedure. Then weld bottom flange to the web by starting at the end of the shop weld on one side of the web joint and proceed to web joint Then start at end of shop weld on other side of joint and complete the weld. Then weld top flange to web using same procedure Each line of girders is to be assembled in shop to correct vertical curve and match-marked. Paint the match marking numbers on adjacent ends of girders, use yellow paint.

A chemical analysis of the flange material was made from borings for compliance with the specifications for carbon and manganese and were as shown in Table 1.

The material met ASTM A-7 Specifications.

The girders were fabricated in lengths of 48 ft. for the section over the piers and 60 ft. for the center portion of the span between piers. These lengths could be shipped and handled on the job satisfactorily and were within the maximum length of material that the mills would furnish at that time. It was unnecessary to make any shop splices in the main material.

No unusual difficulty was experienced in the shop fabrication and the girders were very satisfactory in regard to appearance, straightness and compliance with the plans and plan dimensions. No difficulty was experienced in the field splices except that when the web was welded to the flange for a distance of 12 in. from the butt splice in the web to the butt splice in the flange, the web buckled slightly in a horizontal direction. This buckling could be observed but was of very little consequence as far as the structural strength of the girders was concerned.

All welders on the shop fabrication and the field welding were qualified under the specifications of the Kansas Highway Commission. These qualifications require that the welder pass, in addition to the American Welding Society requirements, a deposited metal test. The qualifications are considered by the welders to be more difficult to meet than the AWS qualifications

There was no procedure for erection shown on the design plans, this being left to the contractor and the fabricator. They elected to place at the splice points four erection plates 6- by ⅜- by 6½-in , bolted through the web with four ¾-in. bolts. These plates were placed near the flanges The web was partially welded, the erection plates removed, then the web weld was completed. The four holes remaining in the web were filled with weld metal after completion of the splices No erection falsework was required (Fig. 3). Also used during erection were clamps placed at the splice points to draw the flanges tight against the edge of the web and also to line up the splices.

All of the field welding was done by two welders assisted by a helper, and was accomplished with one portable welding machine that complied with the specifications of the Kansas Highway Commission.

The fabricated steel was bought by the Highway Commission under a contract separate from the general contract. The low bid for this steel, fabricated and shop painted, delivered at Gordon, Kansas, about 4 miles from the site of the bridge, was nine cents per lb. The low bidders unit price for the erection of the steel which included two coats of field paint was 4.3 cents per lb. Thus the final cost of the welded structural steel for this project was 13 8 cents per lb. in place. The final pay quantity of steel was 312,841 lb. Included in the above weights were two all welded expansion joints. The bids on the fabricated steel were closed September 22, 1947, and the bids for erection were closed February 20, 1948. The contract price for the steel was reasonable considering the difficulty of obtaining steel at the time and the unfamiliarity of the contractors with welded fabrication and field erection. From the experience gained on this project by the contractor and the fabricator the price on future work should be decreased as costs were less than anticipated.

Recent all welded designs have been changed somewhat from this original design. The field splices are now detailed so that the flanges and webs are spliced at the same section instead of staggered 12 in. as on the first design. Longer spans require for economy more variation in flange to accommodate moment and in some spans the girders have been haunched vertically. Stiffeners have been lightened up considerably. In time and with a good experience record for the bridges already built it will be possible to take advantage of the economy of two girder designs over four girder designs for two lane bridges.

The design specifications of AASHO were written primarily for riveted construction and are difficult to apply to welded construction. Many provisions of the specifications that are the result of experience and which are expressed by empirical formulae do not apply to welded girders in many cases, the result is usually to adopt a very conservative attitude with resultant loss of economy.

The welded design used on this project was well justified economically and it is believed that future designs will show even greater savings, providing that the contractor is given the opportunity of selecting his procedure for fabrication and erection and providing also that a suitable design specification for welded designs can be drawn up. Such a specification should be written for welded designs only and the requirements placed in it should be based as far as possible on facts that have been developed by recent research.

This project was under the general supervision of R. C. Keeling, State Highway Engineer. Construction was under the supervision of H. O. Reed, Engineer of Construction and L. S Munn, Division Engineer. The design of this structure was initiated by Geo W. Lamb while he was Bridge Engineer for the Kansas Highway Commission and the bridge was constructed under the advisory supervision of E. S. Elcock, the present Bridge Engineer.