The Bridge Delta Girder—Single-Webbed and Double-Webbed

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THE SPECIFICATIONS of the American Association of State Highway Officials (AASHO) govern the construction of the great majority of highway bridges in the United States. These specifications determine the dimensions and general relationship of parts in steel girders. Since compliance with these specifications is the *sine-qua-non* condition upon which allotment of Federal participating construction funds is based, deviations from these requirements are rare.

Yet it is a fact that the standard plate girder as produced by the AASHO specifications is not satisfactory in all respects. It is too flexible, too limber, too lacking in lateral stiffness. The 1959 AISC National Engineering Conference paper presented by Mr. E. L. Durkee of the Bethlehem Steel Company, dealing with bridge erection problems, is an earnest plea for providing lateral stiffness in steel members.[1] Of course, after installation of cross-bracing, bottom lateral bracing and a top roadway slab, the plate girder serves admirably, but it can not be said to do so in the early stages of bridge construction.

BACKGROUND AND DEVELOPMENT

It was in seeking to get increased lateral stiffness into a steel plate girder that the author conceived of the Delta section. Provisions of the specifications for plate girders provide that a flange plate may cantilever outward from its supporting web plate 12 times its thickness, and that in box girders the span between the two webs may be 40 times the flange thickness. Cantilevering outward on both sides of the web gives the 24 times thickness rule in the specification, which establishes the maximum width of flange permissible. Yet the resulting girder, standard and orthodox though it may be, lacks both lateral and torsional stiffness.

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1. *Durkee, E. L. Problems in Bridge Erection as Affected by Design Requirements, 1959 Proceedings, AISC National Engineering Conference.*

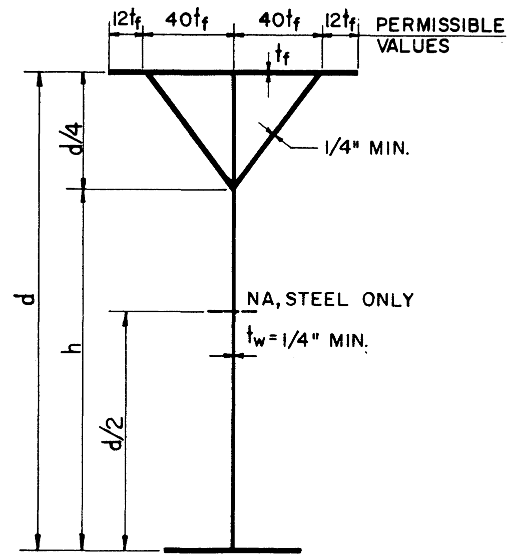

Fig. 1. Single Delta type girder used in Taylor Creek Bridge

Web stiffeners welded to the compression flange provide torsional strength at these points, but between the stiffeners the welded intersection of flange plate and web provides little if any torsional strength.

It was under the stimulation of such appraisals and corresponding discontent that the Delta section finally came to mind. It provides the same basic I-section girder, but one with a wide top flange plate, braced with a pair of inclined corner plates for the full length of the girder. Figure 1 shows the profile of a typical Delta girder. Such a design offers great lateral stiffness and considerable torsional strength. It offers compliance with the AASHO Specifications, not with a flange width limited to 24 times thickness, but, if needed, a maximum width of 24 plus 2 @ 40 = 104 times the plate thickness. All this is achieved with a pair of corner plates which also resist longitudinal flexure, rigidly brace the compressive zone of the web and substantially shorten the unsupported height of the web beneath their juncture point. Note, too that a large portion of this lower web zone is subjected to the longitudinal tensile stresses of flexure and is thus actually restrained from lateral bowing by these stresses. As for torsion, note that the corner plates and the web form a two-celled triangular box which cannot be distorted at any point without at the same time distorting long lengths of abutting girder on both sides of that point. Therefore, unlike the standard plate girder, the Delta girder offers a high resistance to torsional deformation.

FULL-SCALE TESTS

The interesting possibilities of such a girder led to the fabrication and testing of a specimen girder, 61 ft long and 3 ft deep, with a 24-in. wide top flange and 16-in. wide bottom flange. Two corner plates $11\frac{1}{2}$ x $\frac{1}{4}$ in. spanned from a point on the web 9 in. below the top flange to points on the flange 7 in. out from the web. After an initial full length test the girder was cut into three approximately equal pieces which were individually tested. The project was financed by the Pacific Northwest Steel Fabricators Association and by the American Institute of Steel Construction.

Details of the test were reported in *Civil Engineering.*[2] The substance of the test results was that the compressive flange itself and the entire compressive zone from the bottom of the Delta upwards were wholly free from distortion, except in the final case where, with a height-thickness ratio of $27.5/0.25 = 110$ and an aspect ratio of $18/2.25 = 8.0$, the girder supported a central concentrated load of 250,000 lbs (i.e., six and one-quarter H20 S16 trucks) without failure and with only minor local web buckling. In another test this girder carried a central 7-in. eccentrically located concentrated load of 200,000 lbs (i.e., five H20 S16 trucks)—this load being placed over the top edge of one of the flange corner plates—with only very minor deformations of that corner plate and no deformations of consequence in the main web.

In detailing this test girder the question of location of the corner plates first arose. Where should they contact the web and where the flange? In horizontally stiffened plate girder design, the stiffeners are located at the one-fifth depth point. At first thought this same one-fifth depth point seemed to be the logical place for corner plates to start. However, it was realized that the corner plate-web joint bore little resemblance to a joint formed by horizontal stiffeners and a web. The Delta design insured substantial freedom from rotation, angular fixity and, by a division of shear between the web and corner plates, a great reduction in unit shear in the main web above the junction point. Therefore, in the test girder the junction point was set at $d/4$. A value of $d/3.87$ was used in the Taylor Creek Bridge girders, and $d/3$ in the suspended span at the Parker Bridge.

2. *Hadley, Homer* Exploratory Tests on A Steel Delta Girder *Civil Engineering, May, 1961, p. 50.*

In composite construction—with the roadway slab integrated with the girder—the neutral axis shifts upward and may easily reach the junction point of web and corner plates. In so doing it increases the height of web in tension beneath the joint. Pending further tests, that which has proven to be successful—and may be said to possess pragmatic sanction—is the only basis for engineering opinion and judgement of actual structural behavior.

As for horizontal distances out from the web to intersections of the corner plates with the flange, those creating a 45° angle between the corner plate and web seem maximum. An angle whose tangent is approximately $\frac{3}{4}$ seems to be a better slope. These proportions have been followed, although not rigorously, in structures that have been built.

ACTUAL APPLICATIONS

The successful outcome of the test program led to the construction of two bridges, described below, of which the writer has personal knowledge and two others of which he has indirect knowledge. The first is the Taylor Creek Bridge, a small 75-ft single-lane, simple span structure in Seattle's Cedar River watershed. Because of occasional heavy loading, it employs a $\frac{5}{16}$ x 45-in. web. Corner plates are $\frac{1}{4}$ x 15 in. spanning from points 12 in. below the top of web to points on the flange 9 in. out from the web. The web is unstiffened except for vertical stiffeners at bearings and at third-point cross frames. Precast roadway slabs are integrated with the girders and with one another. Figures 2 and 3 show erection of the girders, their freedom from vertical stiffeners except at the cross-frames, and the precast slab attachment details.

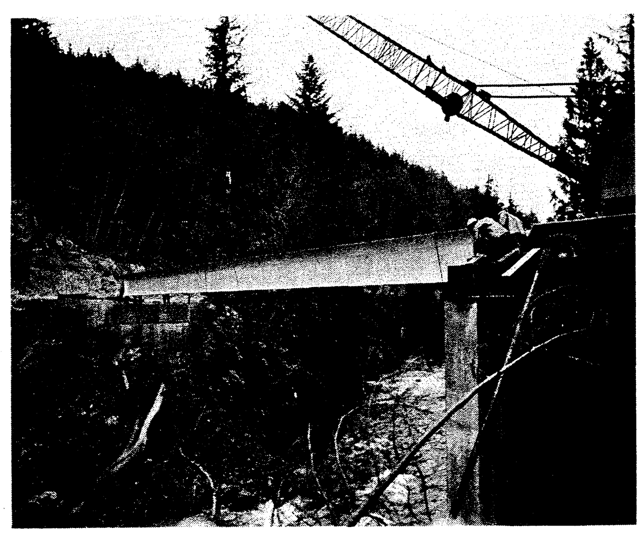

Fig. 2. Erection of Taylor Creek Bridge

The second bridge, known as the Parker Bridge,[3] located a few miles downriver from Yakima, Wash., is a much more extensive structure (Fig. 4). It is a two-lane bridge with a 24-ft roadway and 28-ft overall width, designed for H20 S16 loading. It has spans of 75, 240, 75 ft and, being continuous and on a vertical curve from end to end, is of varying depth. Girder webs vary in depth from 4 ft-7 in. at the abutments to 8 ft at the piers, 5 ft-8 in. at contraflexure points, and 5 ft-11 in. at midspan. The two girders are spaced 14 ft-0 in. apart. They were each fabricated in three sections: two of 135-ft length, extending from the abutments in which they were embedded and held down, to points 60 ft out into the main span where they joined the central 120-ft suspended section. At the time of its erection the 240-ft central span was the second longest girder span in the State of Washington, being exceeded only by a 300-ft standard plate girder span 16 ft-8 in. deep in Mount Rainier National Park.

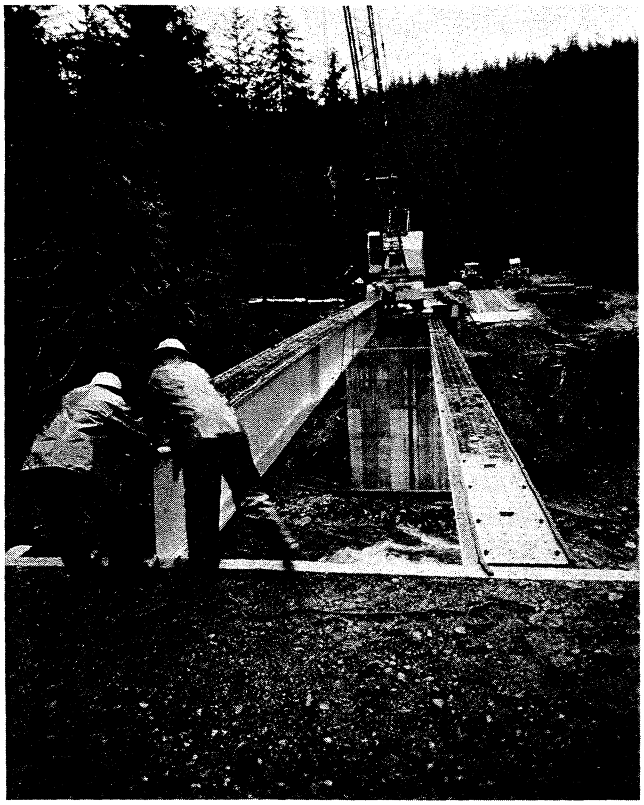

Fig. 3. Erection of Taylor Creek Bridge

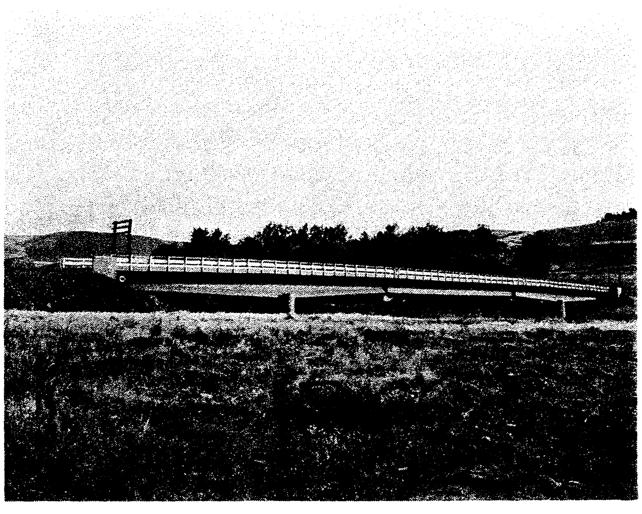

Fig. 4. Parker Bridge

The central 120-ft suspended span, with compression on the top and tension on the bottom, is adequately served with a single Delta section. It is a Delta girder of fairly large dimensions, with 5/16 x 28-in. corner plates starting 2 ft below the top of web and joining the top flange 15 in. out from the top of the web. Stresses reverse in the inshore sections, the compression flange being the bottom one. Therefore the Delta is required at the bottom. Although not required at the top in this region, the Delta was retained there unchanged to permit the same slab thickness and reinforcement, and to enhance the architectural appearance. This has been called the "double-Delta" section (Fig. 5).

The 36-in. wide top flange is 3/8 and 1/2 in. thick in the suspended span, and 3/4 to 2 1/4 in. thick in the inshore spans. Webs vary in thickness from 5/16 to 7/16 in. At the piers where maximum vertical shears and horizontal wind shears in the slabs are discharged, a 5 ft length of 5/8-in. web is used in conjunction with other special bracing at that point.

The massive abutment blocks, 7 x 38 ft in plan and averaging 19 ft high, are primarily counterweights and

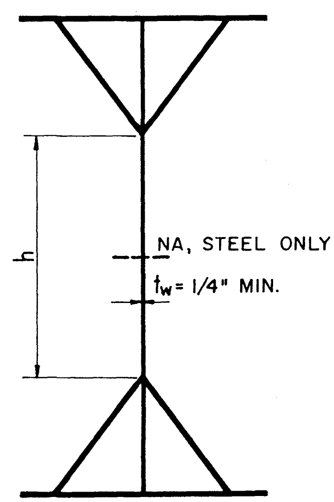

Fig. 5. Double Delta type girder used in Parker Bridge

3. *Hadley, Homer* Delta Girder Design Grows Up, Spans a Major Stream, *Engineering News-Record, May 17, 1962, p. 40.*

are made of 2,500-lb concrete. The roadway slab is of 3,600-lb concrete, except in the 240-ft central span, where a lightweight coarse aggregate was used in conjunction with natural sand. This produced a concrete weighing 108 lbs per cu ft.

Erection was indeed simple. The site was favorable, being everywhere of gently sloping, slightly weathered solid rock except at the blanketed abutments. A single 60 ton crane easily placed the four 135-ft inshore girder sections, and except for the deep channel of the river could have as readily placed the central 120-ft suspended section. Supplementary equipment in the form of certain steel cables, a 12-ft pipe gin-pole and two ordinary automobile wrecker trucks, provided all the additional assistance that was required to erect the suspended section. Not a single fear or apprehension as to lateral stiffness was felt by anyone.

Perhaps a word should be said regarding the general positioning of this bridge. It should have been and would have been set higher above the river except for the fact that about 100 ft from its east end it connects with a state highway. Under flood conditions water may well rise to the top of the piers.

To provide some protection for the bearings, the semi-circular ends of the pier` are carried up 2 ft higher as wall sections, 12 in. thick. These wall sections also screen the bearings from view (Fig. 6). While in this case the bearings would have been quite inconspicuous, in general deep, heavy girders seated upon piers by means of relatively tiny rockers or pins, seem so flagrantly out of scale that they ought to be screened. Unquestionably they are true and authentic steel, but are they aesthetic steel? Not in the eyes of the author.

Both the Taylor Creek and Parker Bridges followed the pattern of the University of Washington test girder in having their webs full height from flange to flange. However, this is not necessary. The web actually need extend only from corner plate junction to corner plate junction. So long as the connections at these points are sound and good, the two corner plates will afford shear-transfer ability equal to or greater than that of the web. The permissible flange width is reduced from 104 times the flange thickness to 64 times the flange thickness, but even with a plate as thin as 3/8 in., a 24-in. flange width complies with the AASHO rules.

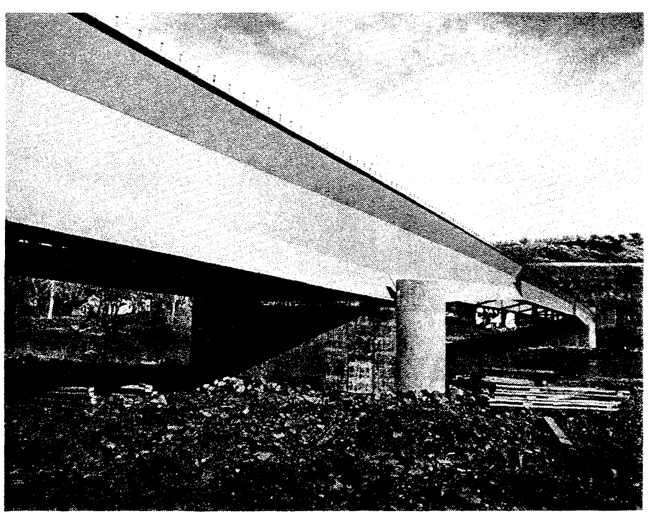

Fig. 6. Parker Bridge—piers designed to screen bearings from view

THE BOX-DELTA GIRDER

There are cases, of course, where the torsionally strong Delta girder is not sufficiently strong. Such a need for greater torsional strength led to the concept of a box-Delta section made of two webs. Because of its composite steel and concrete character, it has been named the U-V section to distinguish it from the all-steel Delta (see Fig. 7).

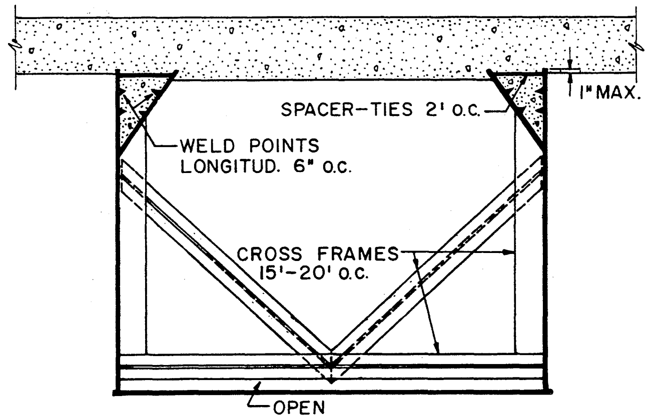

Fig. 7. Typical U-V box-girder section

This section is simply a wide, hollow rectangle with a steel tension flange, a pair of steel webs with interior steel cross-bracing, and inwardly inclined steel corner plates at the top of each web. These corner plates form continuous V-shaped troughs extending the full length of the girder, and in conjunction with their concrete filling create unsymmetrically-shaped Deltas at these compressive corners. Small, equally spaced cross-tie members connect the tops of the corner plate with the tops of the webs. The fabrication of this open-topped, U-shaped main steel work is simple and unrestricted. When it is completed, forms are installed between the corner plates, and the V-troughs are filled with accurately and precisely fitting concrete which becomes integral with the roadway slab.

The substitution of this slab for the normally steel compressive flange of a box section effects a great change. The reinforced concrete slab of 5 or 6-ft span is not subject to buckling problems or limitations imposed because of steel's tendency to bow and bend. Therefore, wide girders are easily obtainable with this combination of materials which might otherwise be troublesome and costly. The concrete slab brings the neutral axis close to the tip of the V-trough. Not to be overlooked is the fact that the inclined corner plates impart a definite lateral stiffening to the webs, roughly equal to that of horizontal flange plates whose width is equal to the corner plate's horizontal projection.

In the author's opinion, this combination of simple steel plates and a composite slab provides a rather surprising number of advantages. First, the steel girder, simple and easy to fabricate, with slabs added and V-troughs filled, possesses vertical strength, lateral strength and torsional strength. Second, the V-trough concrete braces the compressive zones of both steel webs for practically their full depth. Third, all vertical stiffeners are eliminated from the webs except at the interior cross-frames; great areas of contact surface are provided between steel and concrete; the need for steel-concrete shear connectors is eliminated, and no cross-bracing or lateral trussing is required between the separate, individual girders. Fourth, the clean, smooth exterior surfaces are ideally suited for maintenance painting.

Not only is this girder strong, but it looks strong; it looks massive; it looks clean and simple; it looks powerful. Architecturally it is beautiful. It gives steel a new look. It can compete architecturally with concrete, reinforced or prestressed, at any and all times, and since one of these wide U-V box sections replaces two or three prestressed concrete girders, it can successfully compete with them commercially.

When one considers the matter, it is really surprising what great changes result from the employment of corner plates. Without them there is nothing upon which to seat the roadway slab; there is no bracing of the compressive zone of the web; no torsional strength is developed at the junction of web and slab; stiffeners cannot be omitted With a concrete haunch on both sides, the top of the web might be punched for the passage of reinforcing bars which would bend up and be anchored into the roadway slab. Unquestionably bearing could be created in this manner, although torsional strength is another matter. However, the V-trough, with its cross-ties, its weld points, and its generous areas of contact surface, appears to the author to offer a simple and thoroughly reliable connection of concrete with steel, and for that reason is preferred.