Prefabricated Composite Highway Bridge Units With Inverted Steel T-Beams

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•TO MEET the current need of highway engineers for a low-cost prefabricated short-span bridge unit that can be rapidly erected, the U. S. Steel Applied Research Laboratory and the Indiana Steel Fabricators Association evolved a prefabricated bridge unit (Fig. 1) consisting of a concrete deck connected to two steel inverted T-beams by studs. It is intended that these units, including transverse bracing between the T-beams and including the deck, be prefabricated in steel fabricators' shops, transported to the bridge site by truck, placed side by side on the substructure, and field-connected by installing transverse tie rods through the deck and forcing nonshrinking grout into the longitudinal keyways between units. At this stage of construction, truck traffic could pass over the bridge. However, a bituminous wearing surface would probably be placed and railings installed before the bridge would be opened to traffic.

A main feature of these prefabricated composite bridges is that, unlike conventional composite beam bridges, the steel beams in these units have no top flanges. Although the top flange contributes little to the strength of a composite beam after the concrete deck has hardened, in conventional cast-in-place construction it does serve the important function of helping to support the dead weight of the concrete without excessive shoring or temporary supports. However, by prefabricating the units, the need for shoring is eliminated and the economy of steel T-beams can be fully realized. Another advantage of prefabrication is that most of the concrete shrinkage occurs before erection, and pre-erection shrinkage cracks parallel to the longitudinal axis of each unit would be unlikely. Thus, prefabrication greatly reduces the possibility of shrinkage cracks parallel to the longitudinal axis of the bridge in the finished structure.

This paper presents designs for units with inverted steel T-beams having several different unit widths (Fig. 2) and different spans. Although these designs are intended for prefabricated construction, they could also be used for cast-in-place construction if adequate shoring were employed. For cast-in-place construction, however, it would be more efficient to use an equal spacing of T-beams rather than the unequal spacing used in the prefabricated units.

GENERAL DESIGN REQUIREMENTS

The American Association of State Highway Officials (AASHO) Standard Specifications for Highway Bridges, eighth edition, 1961, was used as the design specification for this study, except for certain special live-load distribution formulas developed herein. HS20 live load was used in all beam designs, and a 16,000-lb wheel load (plus 30 percent for impact) was assumed in all deck designs. The weight of the bituminous wearing surface was assumed to be 30 psf.

All main material of the T-beams consists of high-strength structural steel conforming to ASTM designations A441 or A242. These steels, which have a yield strength of 50,000 psi for thicknesses up to $^3/_4$ in., inclusive, were chosen because of their superior strength-to-cost ratio; steels with higher yield strengths were not considered because of live-load deflection limitations. Greater economy could be achieved by the use of other commercially available high-strength low-alloy steels currently being considered for adoption by ASTM.

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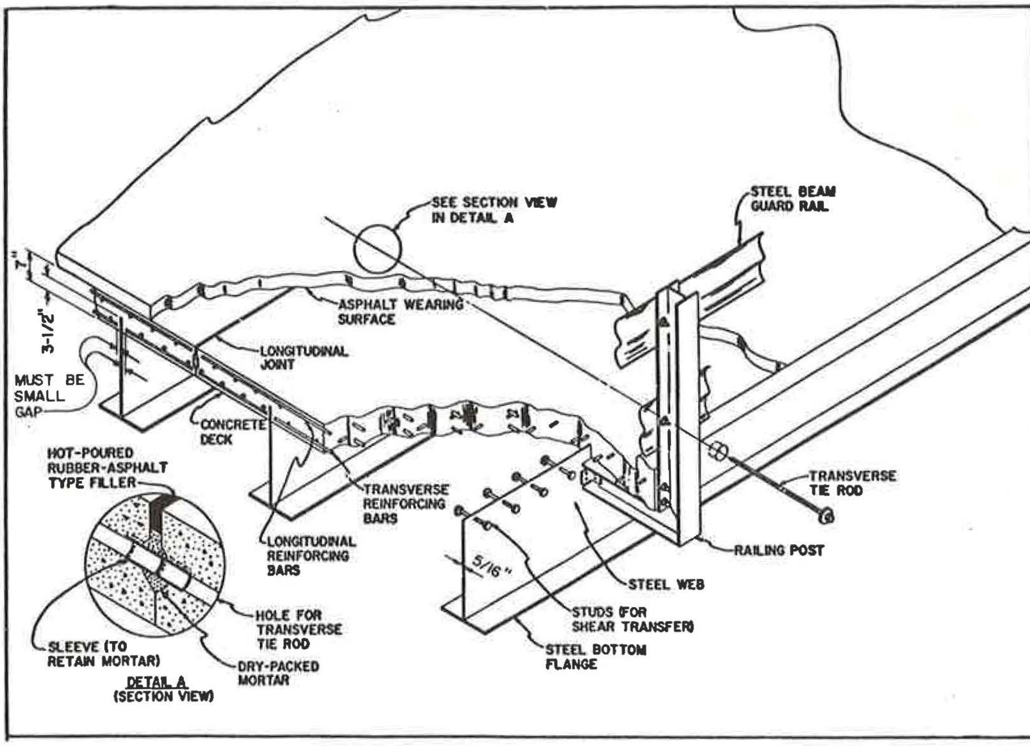

Figure 1. Prefabricated composite highway bridge unit with inverted steel T-beams.

Diaphragms, web stiffeners, railings, railing posts, and other detail material are made from A36 steel. Because the AASHO specification limits the minimum thicknesses of plates to $\frac{5}{16}$ in., all web plates were designed to be $\frac{5}{16}$ in. thick.

DESIGN OF DECK SLAB AS TRANSVERSE SPAN

The reinforced-concrete decks simultaneously perform two functions: (a) they act as transverse spans, distributing the wheel-load concentrations to the vertical web plates, and (b) they act as the top flanges of the longitudinal bridge beams. The first function, which governs reinforcing requirements and which generally governs the slab thickness, is discussed in this section; the second function is discussed in the following section.

For all designs, the concrete slabs are 7 in. thick because that appeared to be the minimum thickness practical to accommodate embedment of the beam webs and stud connectors, as well as positioning the holes for the transverse tie rods and placing the reinforcing steel. Concrete with 3,500-psi ultimate compression strength, which is within the 3,000-to-4,000 psi range most typical of bridge construction, was selected for all designs because many of the optimum beam designs, discussed next, result in longitudinal compression stresses in the deck close to the 1,400-psi allowable stress corresponding to 3,500-psi ultimate compression strength.

It was assumed that the 16,000-lb wheel load (plus 30 percent impact load) can be placed anywhere on the slab, but that the effective tire width in the transverse direction is 20 in., so that only part of the wheel load is exerted on a given slab unit if the center of the wheel is placed less than 10 in. from the longitudinal joint. In computation of transverse bending moments in the concrete, it was conservatively assumed that there was neither shear nor moment transfer at the longitudinal joints. This assumption was made to simplify the slab design and to make sure that the slab

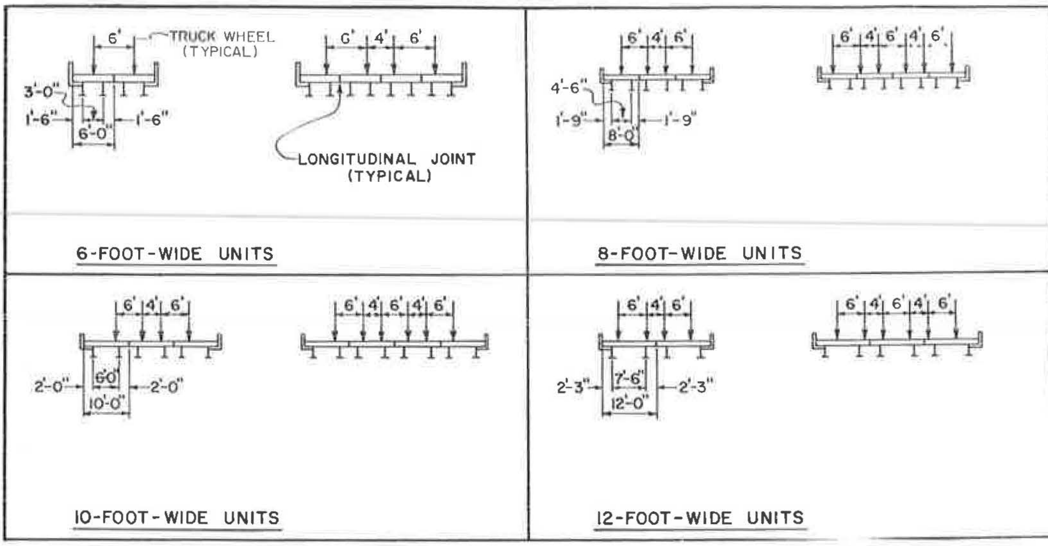

Figure 2. Typical bridge cross sections.

reinforcing steel would be adequate for any bridge assembly, even if the longitudinal joints were not keyed together. The vertical steel web plates, which support the deck slabs, were spaced so that the negative transverse bending moment (tension in top fibers of deck) in the slab at the steel webs would be equal to the maximum positive bending moment (tension in bottom fibers of deck) in the slab between webs and thereby allow the same top and bottom transverse reinforcement. Longitudinal reinforcement, which enables the slab to distribute wheel loads in the longitudinal direction, is specified by AASHO as a percentage of the transverse reinforcement.

On the basis of these considerations, the spacings of beam webs and reinforcing-bar requirements given in Table 1 were determined in accordance with the specifications. These beam-web spacings and reinforcing-bar requirements apply specifically to the prefabricated bridges. For shored cast-in-place bridges, it would be more logical to use equal spacing of tees, and the reinforcing-bar requirements would be less.

The possibility of longitudinal cracks occurring in the surface of the deck above the web of the T-beam due to the negative moment that occurs above the web was investigated experimentally in conjunction with T-beam punching shear tests (1) conducted at the U. S. Steel Applied Research Laboratory. In these tests, top surface cracking did not occur until a negative moment corresponding to a wheel load of about 40,000 lb—almost twice the wheel load for an HS20 truck plus impact—was applied. Thus, for static loading, top surface cracking probably would not be any more of a problem in the prefabricated units with inverted steel T-beams than in conventional composite bridges. To the author's knowledge, no fatigue tests have been conducted to determine whether problems of concrete cracking might occur in this type of construction under repeated loading.

Because about a 100-kip offset punching load was supported before failure, these T-beam punching shear tests indicated that, if the gaps between the bottom reinforcing bars and the steel webs are sufficiently small, the lap of the studs and the bottom reinforcing bars is sufficient to develop resistance to the small transverse positive bending moments that may occur in the slab directly over the web.

BEAM DESIGN

Live-Load Distribution Factors

The live-load bending moments in the longitudinal direction depend on the position of the truck wheels with respect to the beam in both the longitudinal and transverse

TABLE 1

BEAM-WEB SPACING AND SLAB REINFORCEMENT REQUIREMENTS

Width of Slab Unit (ft)	Spacing of Pair of Beam-Web Plates in Slab Unit (ft)	Distance from Beam-Web Plate to Longitudinal Joint (ft)	Transverse Reinforcing Steel[a]			Longitudinal Reinforcing Steel[a]		
			Size No.	Diameter (in.)	Top Bar Spacing and Bottom Bar Spacing (in.)	Size No.	Diameter (in.)	Top Bar Spacing and Bottom Bar Spacing (in.)
6. 00	3. 00	1. 50	5	5/8	8	4	1/2	10
8. 00	4. 50	1. 75	6	3/4	8	4	1/2	8
10. 00	6. 00	2. 00	6	3/4	7	4	1/2	7 1/2
12. 00	7. 50	2. 25	6	3/4	6	4	1/2	7 1/2

[a]Structural or intermediate grade.

directions. The longitudinal position resulting in maximum moment at a given location for any given span is easily calculated by conventional design procedures. However, once that is determined, it is necessary to determine what proportion of the truck wheel loads, and hence longitudinal moments, are supported by any given beam when the wheels are positioned transversely to cause maximum stress in that beam. (As used in this context, a beam consists of a single web, a steel flange, and the effective portion of the concrete slab acting with the single web.) The live-load distribution factor is the fraction of the moment of one longitudinal line of wheel loads (half of one truck, or half of one "lane" of loading equivalent to a single series of trucks on the span) that is carried by one beam when the bridge is loaded by trucks positioned to produce maximum moment in that beam.

An exact calculation of live-load distribution factors would be prohibitively complicated, since the load distribution depends on the stiffness of the steel T-beams, transverse diaphragms, and concrete slab acting as a composite unit. On the basis of in-service experience, AASHO specifies a live-load distribution factor equal to S/5. 5 for steel-beam bridges with concrete decks, where S is the average spacing of beam webs measured in feet. It could be argued that this AASHO factor might be unconservative for bridges with longitudinal joints in the deck because the presence of the longitudinal joints might reduce the transverse stiffness of the deck. However, the rigidity of the joint detail provided by the clamping action of the transverse tie rods would tend to keep any reduction in transverse deck stiffness small. Furthermore, any reduction in deck stiffness could be counteracted by using more steel diaphragms between T-beams. Therefore, it appears that reasonable designs can be obtained by using the AASHO live-load distribution factor. Nevertheless, two sets of bridge designs are included in the present study: one based on the AASHO live-load distribution factor, and the other based on what will be called the "hinged-joint live-load distribution factor."

This second factor, which is computed from the equations given in Figure 3, is based on the hypothetical assumption that the diaphragms between tees have no bending stiffness and that the longitudinal joints have zero bending rigidity; that is, they act like frictionless hinges. The further conservative assumption is made that the beams are infinitely stiff compared with the stiffness of the slab, so that a load applied to the slab directly over a beam web would affect only that beam. Thus, the lateral distribution of load to other beams provided by the slab is neglected. The conservatism of neglecting lateral distribution is indicated by the fact that when the spacing of wheels given by AASHO is used, theoretical distribution factors based on equally spaced, infinitely stiff supporting beams (1. 000, 1. 200, 1. 236, or 1. 336 corresponding to beam spacings of 3, 4, 5, or 6 ft, respectively) would be considerably greater for conventional bridges than the S/5. 5 specified by AASHO (0. 545, 0. 727, 0. 909, or 1. 091 corresponding to beam spacings of 3, 4, 5, or 6 ft, respectively).

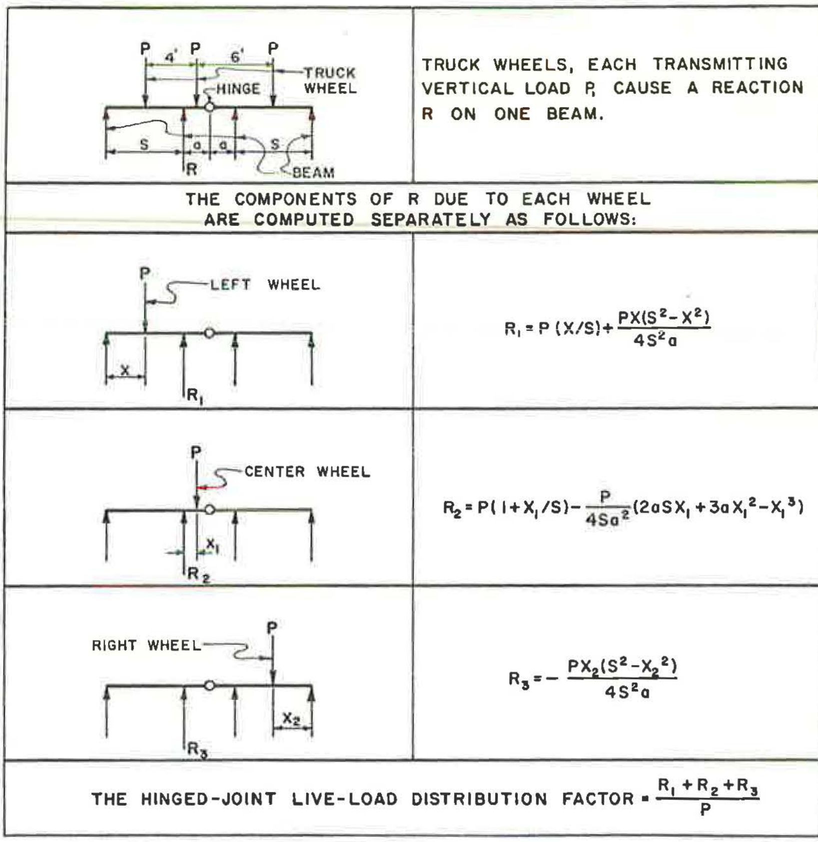

Figure 3. Loading diagrams and general formulas for hinged-joint live-load distribution factors.

For the different beam spacings of the present study, live-load distribution factors computed by the equations in Figure 3 were 1.000, 1.400, 1.390 (1.400 was used), and 1.532 for most critical position of loads on the 6-, 8-, 10-, and 12-ft wide units, respectively (Fig. 2). The rules about spacing of wheels given by AASHO were used to develop the hinged joint live-load distribution factors.

A different, less conservative assumption—that the stiffness of the transverse diaphragms causes all beams in a given bridge span to deflect the same amount—is allowed by AASHO for computing live-load deflections of bridges, and was also used for all deflection calculations of the present study. Most critical live-load distributions based on this assumption were 0.500, 0.750, 0.750, and 1.000 for the 6-, 8-, 10-, and 12-ft wide units, respectively (Fig. 2).

Calculations for Beam Main Material

Once the live-load distribution factors were determined, the beam requirements could be calculated by conventional bridge-design procedures. It was assumed that half of the concrete in each unit acted compositely with each steel T-beam. However, because of the many computations required for optimized solutions, a digital computer was used for calculations. Specifically, for a large number of different spans, several designs, differing only in the depth of the steel T-beams, were made for each of the eight cases: 6-, 8-, 10-, or 12-ft wide units with live-load bending moments determined by either the AASHO distribution factor or the hinged-joint distribution factor.

Although the designs with the hinged-joint distribution factors apply specifically to the prefabricated bridges, the designs with the AASHO factors apply both to the pre-

fabricated bridges, with beam spacing as shown in Figure 2, and to shored cast-in-place bridges with equal spacing of T-beams, with the 6-, 8-, 10-, and 12-ft wide designs applying to the cast-in-place bridges with 3-, 4-, 5-, and 6-ft tee spacings, respectively.

The data from these calculations, which should be useful to designers for estimating weight, selecting beam depths corresponding to minimum weight, and determining bottom-flange area required, are indicated as curves in Figures 4 through 11, one case being summarized in each figure. In these figures, the bottom flange areas and weights of the steel T-beams per square foot of deck area were plotted individually as functions of span length. The weights in these charts do not include the weights of the A36 steel diaphragms, about 2 to 3 psf, or the A36 steel beam stiffeners, about 2 to 7 psf. Any combination of flange thickness and width that does not violate the maximum width-to-thickness ratio of 12 stated in the specification for outstanding legs of flanges could be used to provide the required area, but if the thickness exceeds $^3/_4$ in., an increase in area is required, as discussed next. All designs are based on a steel allowable bending stress of 27,000 psi—the specification allowable stress for high strength A441 or A242 steel $^3/_4$ in. or less in thickness. If a plate thicker than $^3/_4$ in. is used, the required area of the plate must be increased by a factor slightly greater than 1.125, since the specification allowable stress for high-strength A441 or A242 steels in thicknesses exceeding $^3/_4$ in. but not $1^1/_2$ in. is 24,000 psi. This reduction in the allowable stress corresponds to the reduction of the specified minimum yield point of A441 or A242 steels from 50,000 to 46,000 psi when thicknesses exceed $^3/_4$ in. Alternatively, certain proprietary A441 (modified) steels that maintain a specified minimum yield point of 50,000 psi for considerably greater thicknesses could be used for flanges thicker than $^3/_4$ in. without increasing the area requirements.

Although bending moments are usually the most important considerations in determining the requirements for beam main material, shear forces sometimes set minimum thickness requirements for the webs. All webs in this study were stressed in shear well below the specification allowable value of 15,000 psi, and all meet the requirement for webs stiffened with transverse stiffeners, that the web thickness be not less than $^1/_{140}$ times the clear depth of the web, defined as the depth between the bottom of the deck slab and the top of the bottom flange plate. Where, in this study, the depth of the web exceeds about $15^1/_2$ in., i.e., 50 times the thickness for steel with a 50,000-psi yield point, the web must be stiffened with transverse intermediate stiffeners in accordance with Section 1.6.80 of the AASHO specification to prevent shear buckling of the webs.

Design of Studs Connecting Steel Webs to Concrete Decks

The spacing of the steel studs which connect the tops of the web plates to the concrete deck slabs should be determined in accordance with Section 1.9.5 of the AASHO specifications, with the modification that the calculated shearing forces on the studs should be the vector sum of: (a) the horizontal shear from composite action, computed by conventional procedures, and (b) the localized vertical punching shear that would be caused by a truck wheel offset horizontally a slight amount from the plane of the steel web. This vertical punching shear is not described in the AASHO specifications and is not important in conventional beams with top flanges because the stud connectors on such beams are generally vertical and therefore only have to resist the horizontal shear from composite action. To determine approximately what intensity of vertical punching shear should be considered in the stud designs of this study the Applied Research Laboratory performed punching tests on two composite specimens, each consisting of a 7-in. thick reinforced-concrete slab 10 ft wide by $5^1/_2$ ft long cast with two vertical steel webs 6 ft apart. (The punching shear carried by each stud cannot be determined analytically because the longitudinal distribution of the wheel loading and the percentage of the total punching shear that is carried directly by the web are not known.) Thus, the specimens represent a section of a composite bridge with 10-ft wide deck units (Fig. 2). As a result of these tests, it was suggested that the maximum vertical punching shear per stud, due to dead load and HS20 live load and impact, be 342S lb, where S is the spacing of studs on one side of the web in inches. Studs should

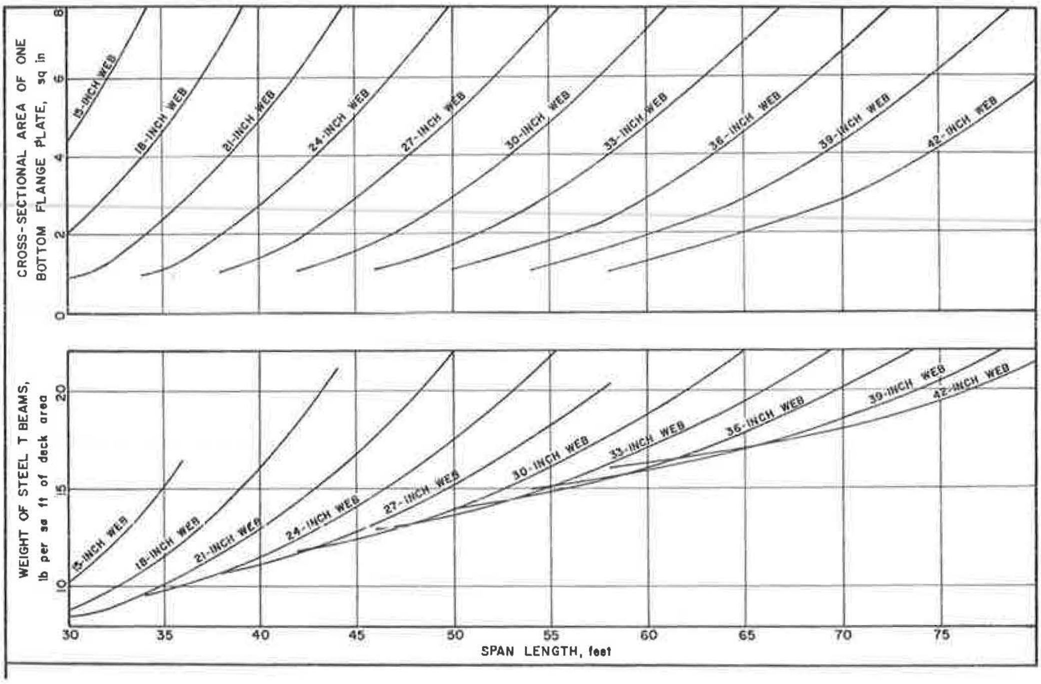

Figure 4. Six-ft wide composite bridge unit with two inverted T-beams, based on AASHO live-load distribution factor.

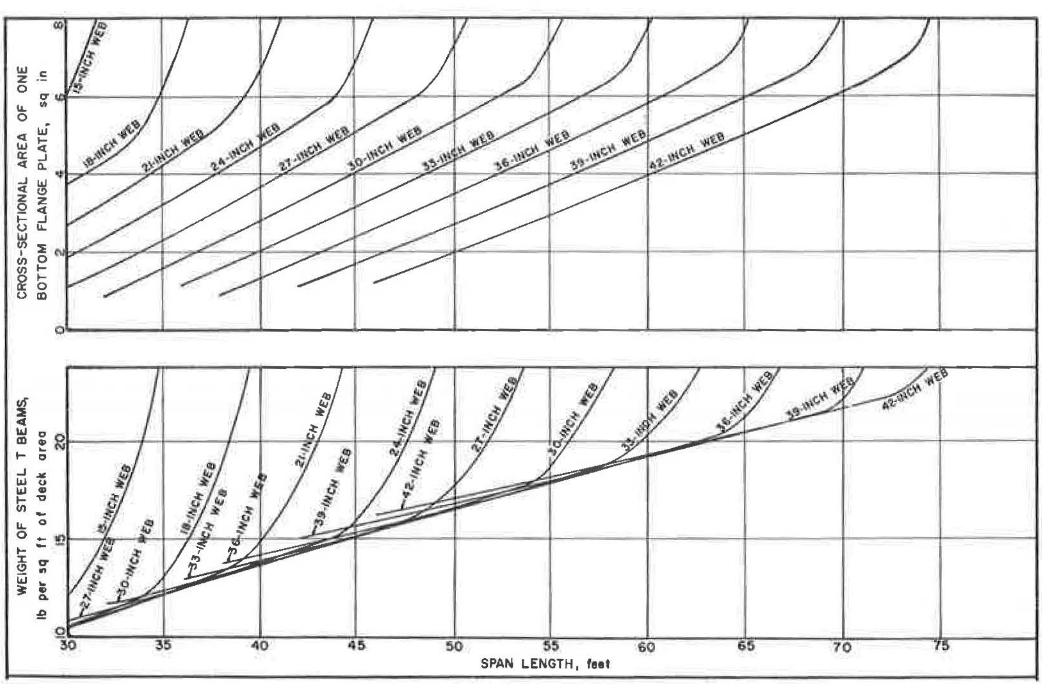

Figure 5. Six-ft wide composite bridge unit with two inverted T-beams, based on hinged-joint live-load distribution factor.

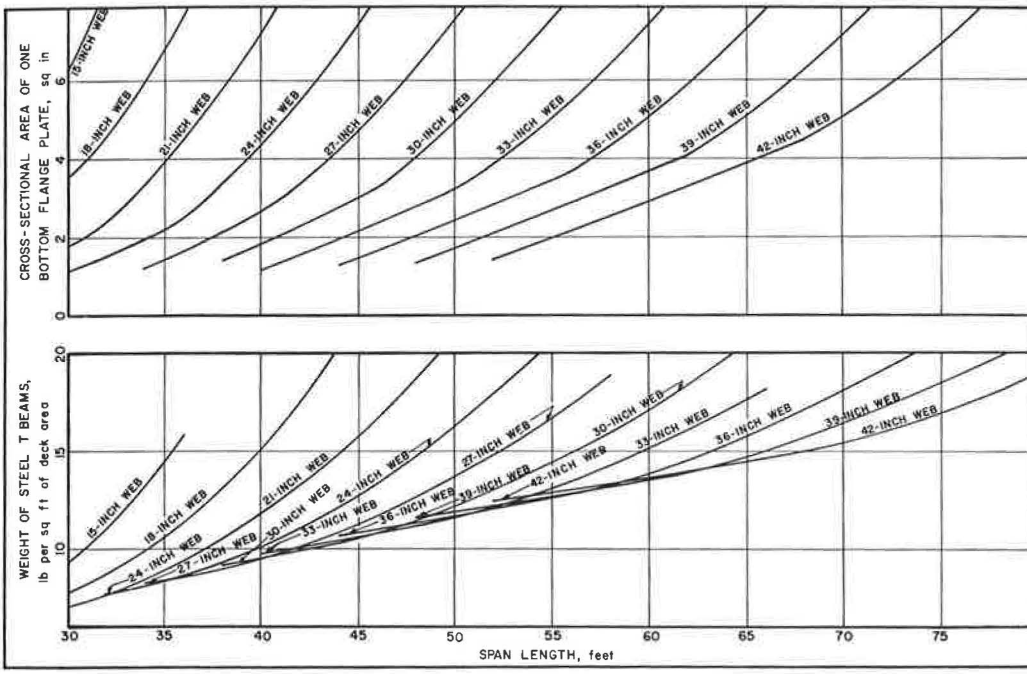

Figure 6. Eight-ft wide composite bridge unit with two inverted T-beams, based on AASHO live-load distribution factor.

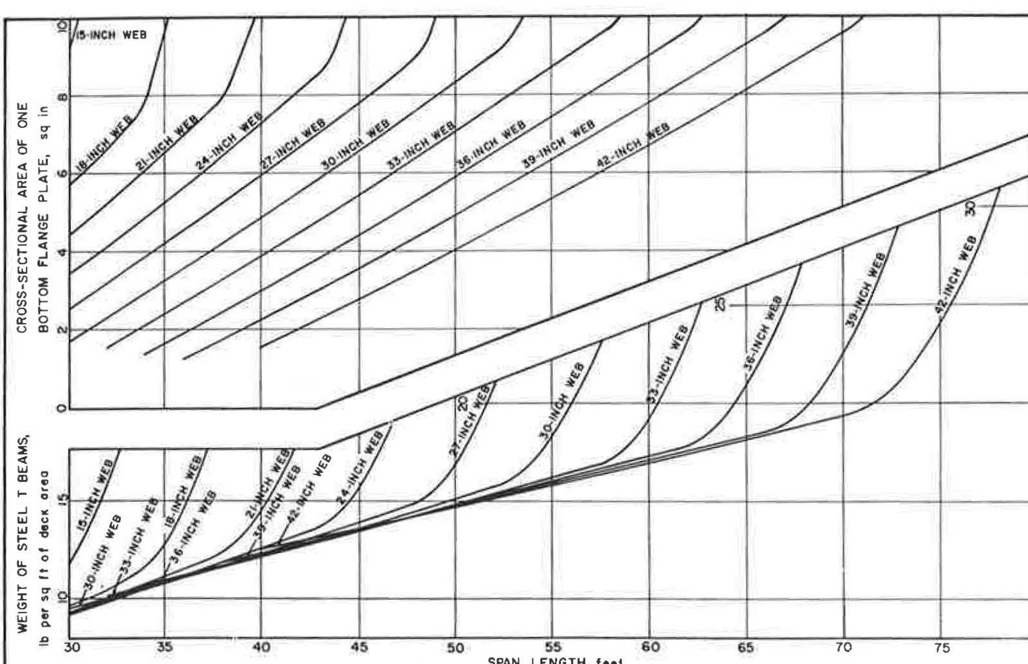

Figure 7. Eight-ft wide composite bridge unit with two inverted T-beams, based on hinged-joint live-load distribution factor.

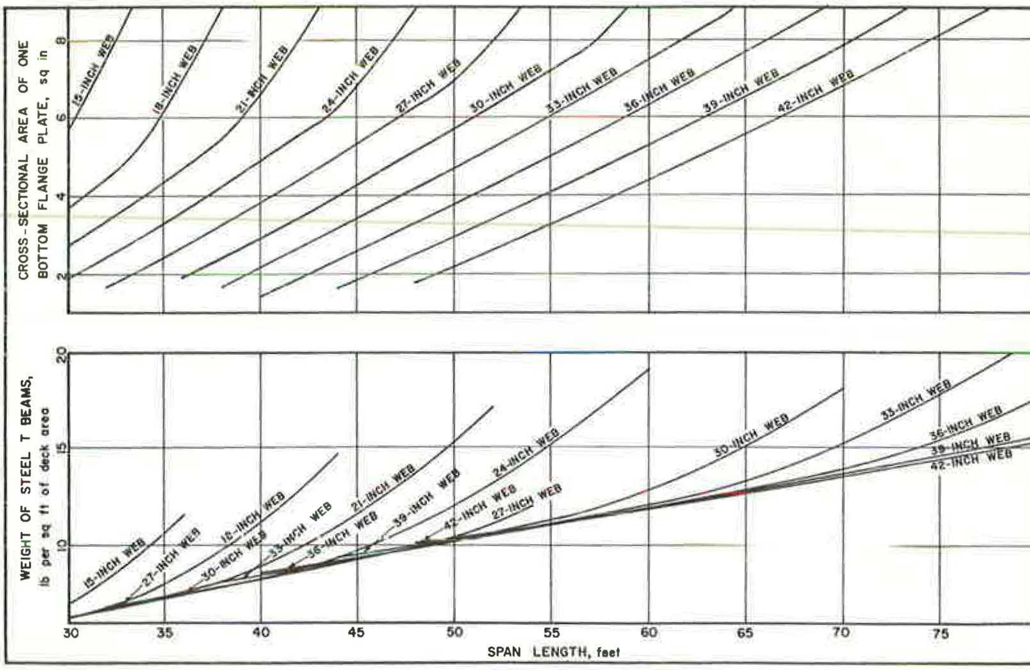

Figure 8. Ten-ft wide composite bridge unit with two inverted T-beams, based on AASHO live-load distribution factor.

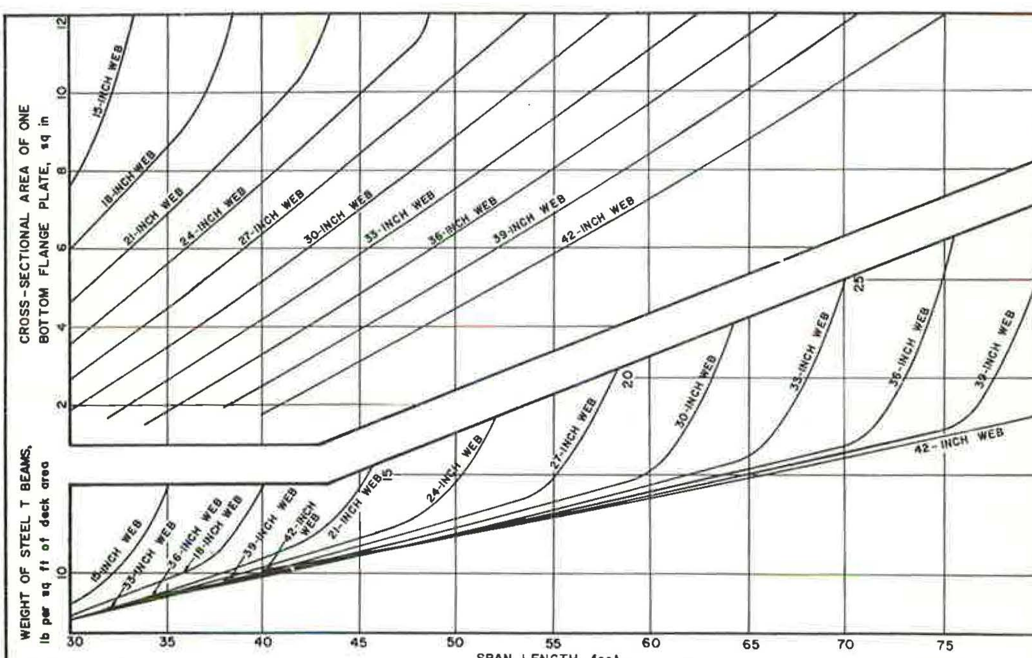

Figure 9. Ten-ft wide composite bridge unit with two inverted T-beams, based on hinged-joint live-load distribution factor.

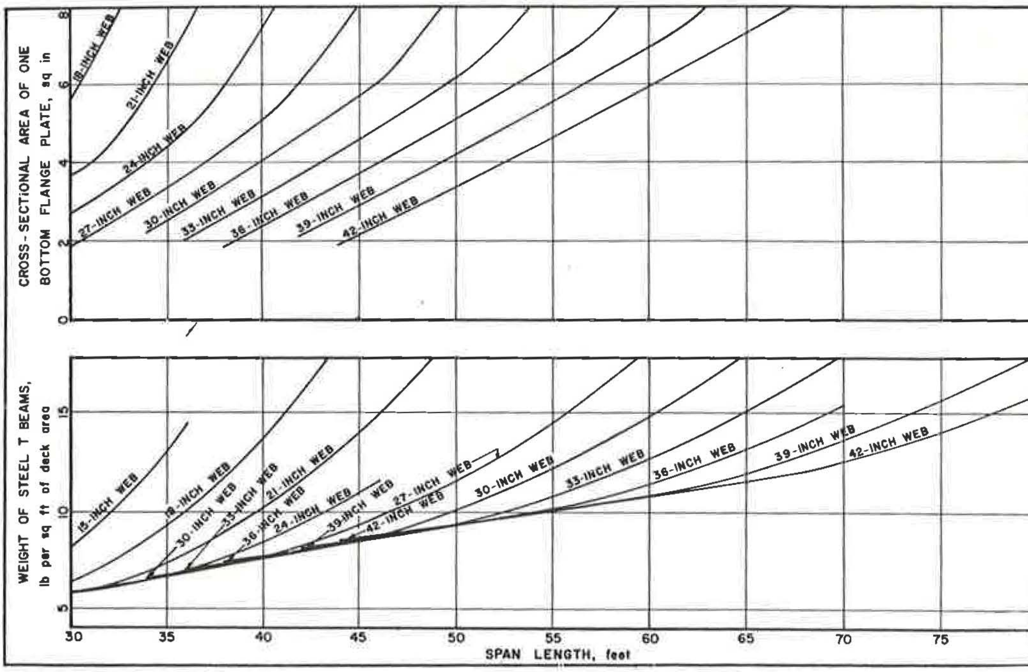

Figure 10. Twelve-ft wide composite bridge unit with two inverted T-beams, based on AASHO live-load distribution factor.

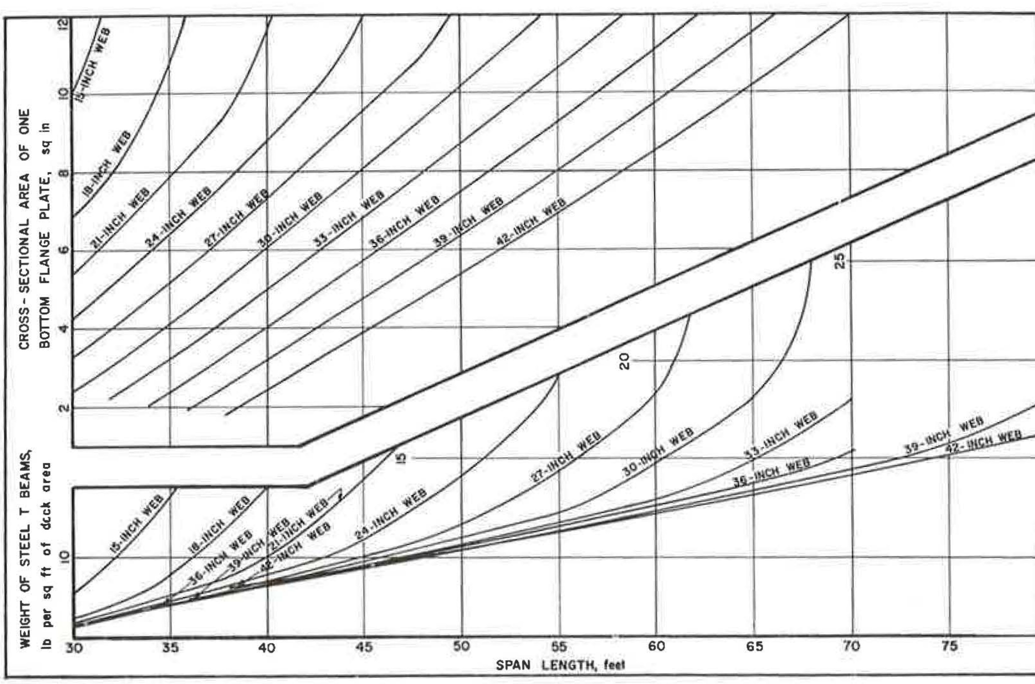

Figure 11. Twelve-ft wide composite bridge unit with two inverted T-beams, based on hinged-joint live-load distribution factor.

be equally spaced on both sides of the web, and to minimize the possibility of the web warping, they should be placed directly opposite each other. Because the punching-shear formula was derived from tests on a 10-ft wide configuration, it can also be applied conservatively to the 8-ft wide and 6-ft wide configurations. Also, since the total live-load punching shear for the 7½-ft web spacing of the 12-ft wide bridge units is theoretically only about 3 percent greater than the total live-load punching shear for the 6-ft web spacing of the tests, it also appears satisfactory to apply the formula to bridges with the 12-ft wide configuration.

To determine whether ¾-in. diameter studs could be used in the present designs without objectionable crowding, stud-spacing calculations were made for 16 typical bridge designs with spans ranging from 40 to 70 ft and with span-to-web depth ratios ranging from about 18 to 20. Specifically, the maximum allowable spacing at the end of span was calculated for ¾-in. diameter by 4-in. long steel studs that have an ultimate shearing strength of 11,000 lb per connector when embedded in concrete with an ultimate compressive strength of 3,500 psi. The calculated stud spacing on each side of the web ranged from about 5.2 in. for a 6-ft wide unit to about 3.2 in. for a 12-ft wide unit. Thus, it appears that the use of ¾-in. diameter studs would not generally result in objectionable crowding of studs, but that the use of smaller diameter studs, which have less strength, would probably result in an undesirably close spacing.

For a particular bridge, the most efficient stud spacing, which may vary by steps along the length of the bridge, can best be selected by the designer. Therefore, stud spacings are not given herein.

DESIGN OF BRACING BETWEEN BEAMS

To help distribute live loads and to resist racking from wind and other causes, the AASHO specifications require that in beam or girder bridges intermediate transverse bracing between adjacent beams or girders be placed at longitudinal intervals not exceeding 25 ft. However, the strength and stiffness requirements for the bracing are not specified and cannot readily be calculated. Therefore, the design of such bracing is based on engineering judgment or "rule-of-thumb" methods. In composite beam bridges of the span range considered here, a diaphragm consisting of a steel channel is usually used as the bracing.

To insure adequate live-load distribution so that the AASHO distribution factor may be confidently used for the present designs in spite of the existence of the slab longitudinal joints between adjacent units, it would be desirable to provide more transverse stiffness than is usually provided. This could be accomplished by using X bracing or stiff beam diaphragms and/or reducing the spacing of the bracing. As in conventional designs, however, the exact selection of the bracing must be left to the individual designer's judgment, because the bracing requirements are too complex to be readily calculated.

The end diaphragms, however, support the wheel loads positioned on the deck in the vicinity of the end bearings of the beams, and do not affect the apparent rigidity of the longitudinal joints. Therefore, the end diaphragms should consist of rolled beams, designed to support a 16,000-lb vertical wheel load, plus 30 percent for impact.

PREFABRICATED COMPOSITE BRIDGES WITH INVERTED STEEL T-BEAMS VS PRESTRESSED CONCRETE BOX-BEAM BRIDGES

In 1963, four members of the Indiana Steel Fabricators Association prepared detailed cost estimates for fabrication at plant, not delivered to site, of a 50-ft bridge consisting of one 10-ft wide and two 8-ft wide prefabricated composite units with inverted steel T-beams. On top, a 24-ft wide roadway is flanked by two 1-ft wide escape walks with post-and-guardrail railing attached. Each prefabricated unit has a pair of 33- by $^5/_{16}$-in. A441 steel webs, and the A441 steel bottom flanges are 9½ by ½ in. in the 10-ft wide unit and 6½ by ½ in. in the 8-ft wide units. A total of 156 ¾-in. diameter by 4-in. long steel studs is welded to each web of the 10-ft wide unit, and 128 studs are welded to each web of each 8-ft wide unit. Intermediate diaphragms between beam webs, which are spaced at 10-ft intervals in the longitudinal direction,

TABLE 2

ESTIMATED COSTS OF 50-FT LONG PREFABRICATED COMPOSITE BRIDGE MADE WITH INVERTED STEEL T-BEAMS

Fabricator Identifying Letter[a]	Cost of Units ($ per sq ft of bridge deck)		
	Structural Steel[b]	Reinforced Concrete	Total for Finished Unit[c]
A	3.62[d]	1.77	5.39
B	3.62	2.04	5.66
D	5.40	2.04	7.44
E	2.96	1.58	4.54
Avg.	3.90	1.86	5.76

[a] In Indiana Steel Fabricators Assoc. correspondence.
[b] Including beams, studs, stiffeners, bracing, railing, and tie rods.
[c] Ready to be shipped from fabrication plant.
[d] Fabricator A did not include the cost of guardrail. The other fabricators did include the cost of guardrail.

are X bracings consisting of A36 steel single angles 4 by 3 by 5/16 in., and the transverse diaphragms at the end of the span are A36 steel channels weighing about 20 lb per foot. The design loading was HS20, and the AASHO live-load distribution factors were used in the design. The fabricators' detailed estimates for this bridge are given in Table 2.

An August 1963 survey of three major Michigan producers of concrete products indicated that the cost of 27-in. deep prestressed concrete box beams adequate to span 50 ft under HS20 loading, delivered to a job site but not erected, was currently about $6.25/sq ft of bridge deck. However, it was not possible to determine the cost at the fabrication plant, because the producers usually absorb freight charges and sell directly to contractors at job sites. Nevertheless, one of the producers stated that throughout continental United States the cost at the fabrication plant of 27-in. deep prestressed concrete box beams capable of spanning 50 ft, figured as selling price minus freight cost, is probably between about $4.25 and $5.75/sq ft of bridge deck. On the basis of about $0.50/sq ft for railing, the cost at the fabrication plant would be between about $4.75 and $6.25/sq ft for the prestressed concrete box beams. These costs compare closely with the fabricators' estimates indicated in Table 2. Therefore, it appears that prefabricated composite bridge units with inverted steel T-beams would be competitive with prestressed concrete bridges.

CONCLUSION

It thus appears that prefabricated composite highway bridge units with inverted steel T-beams would be both structurally adequate and economical. However, to demonstrate the performance of these prefabricated bridges and to determine experimentally the appropriate live-load distribution factors, it would be desirable to build and test a prototype bridge.

ACKNOWLEDGMENT

Appreciation is extended to the Indiana Steel Fabricators Association for furnishing the cost estimates for the hypothetical bridge.

REFERENCE

1. McDermott, J. F. Tests Evaluating Punching Shear Resistance of Prefabricated Composite Bridge Units Made with Inverted Steel T-Beams. Highway Research Record 103, pp. 41-52, 1965.