Prefabricated Press-Formed Steel T-Box Girder Bridge System

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The conventional bridge construction methods[17,24,25] are time-consuming and costly because of the high costs of material and labor. In addition, bridge maintenance costs are increasing rapidly. Hence, the need for more economical techniques of rehabilitating existing bridges and building new ones is well recognized.[6,13,15] To this end, several innovative modular bridge systems are being developed by the authors and have been reported in the literature.[19,25,32] This paper presents the details of an all-steel superstructure system that is economical and suitable for span ranges of 40 to 100 ft under HS20-44 loading. Herein, it is referred to as the press-formed prefabricated steel box girder-bridge system.

DISTINCTIVE FEATURES

The proposed system (Fig. 1) consists of a trapezoidal trough section which is press-formed from 3/8-in. thick A36 steel plate and shop-welded to a 3/8-in. thick steel top flange with 6-ft or 8-ft widths. The top flange of the proposed girder is stiffened by a system of longitudinal ribs (WT 5×7.5) and partial-depth intermediate diaphragms (WT 8×13 @ 6.5 ft c/c). The webs and the bottom flanges are not stiffened internally or externally. The ends of the girders are closed by a 3/8-in. thick plate diaphragm welded all around the flanges and webs of the girder. In addition, bearing stiffeners have also been provided. Typical girder sections are shown in Figs. 2 and 4.

Different girder sections (Figs. 2 and 3) are suggested for different span lengths. Essentially, the girders are produced in two top-flange widths, 6 ft and 8 ft, each having three variations in depth: 2.5 ft, 3.0 ft and 3.5 ft. A suitable combination of these sections results in the desired deck widths at 2-ft intervals for spans up to 65 ft. Longer spans are possible with deeper girders made of thicker plates.

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Lateral load distribution between the adjacent girder units is achieved through continuous welds provided at the junction of the flanges. An anti-skid wearing surface is provided for adequate traction and to protect the steel deck against the direct effects of atmospheric conditions and traffic.

WHY AN ALL-STEEL STRUCTURE?

Two functional considerations for economy in bridge design are: (a) efficient utilization of the material by optimizing the dead weight of the structure and (b) minimizing the shop and field operations required to fabricate and erect the structure. These conditions are fulfilled by utilizing steel for the entire bridge superstructure, since steel is ideally suited for these purposes, and also for future expansion or increased load-carrying capacities.

Innovative application of high-strength steels and welding techniques have considerably enhanced the efficiency (strength-to-weight ratio) of steel structural members in recent years. A comparison made for three European bridges that were rebuilt after World War II shows that savings in steel weight of 25 to 55% and in total dead weight of 52 to 60% were achieved using all-steel orthotropic construction.[3,16]

Generally, the dead weight savings in superstructure are much more significant and important for long-span bridges. However, in comparison with the present practice for short-span bridge construction, there are several other advantages of the steel-deck construction of the proposed system:

1. Due to higher torsional stiffness of a closed section, the lateral distribution of live load is more favorable (smaller distribution factor) in this kind of construction, as compared to that in a conventional slab-stringer system.[20] Consequently, the design bending moment is theoretically smaller for this type of bridge system than for an I-beam bridge system.
2. Since 95% of this type of a bridge system is prefabricated under controlled conditions, better quality

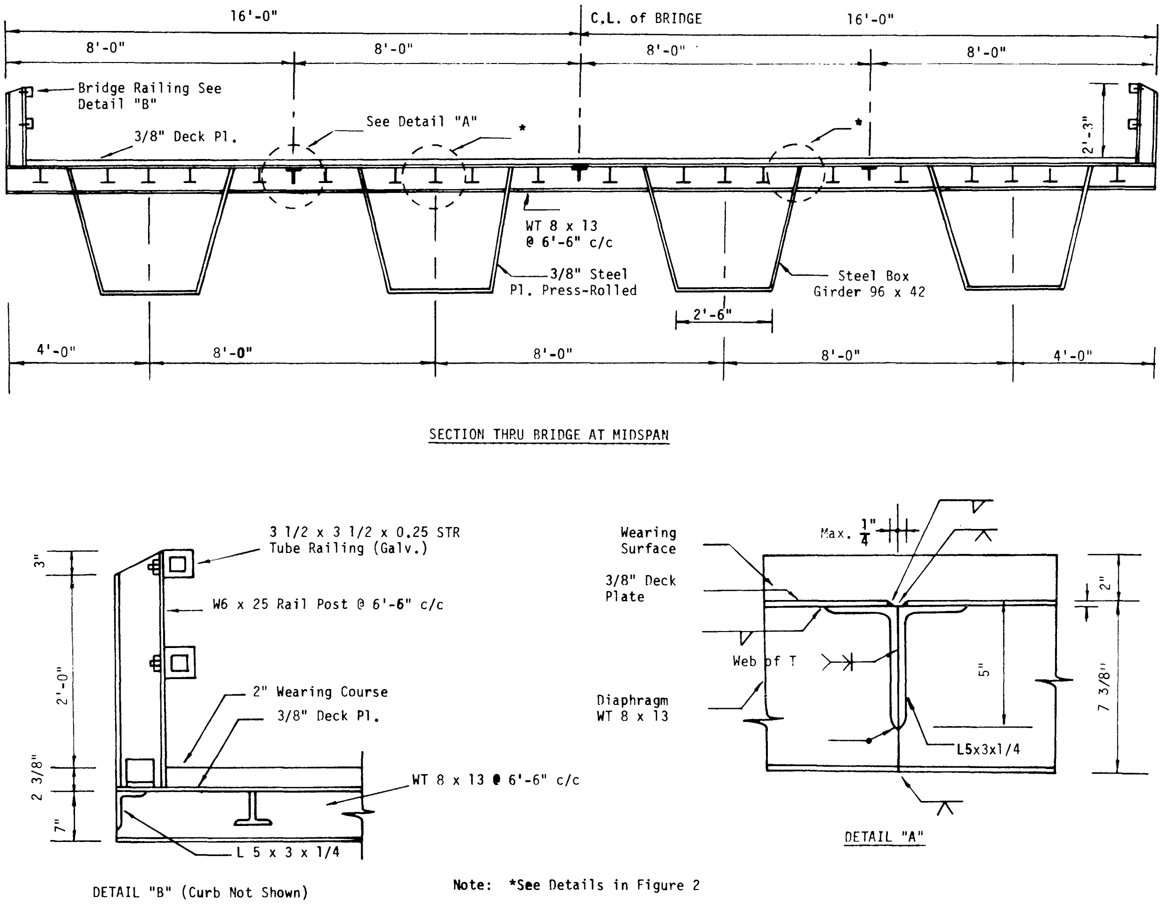

Fig. 1. Typical bridge section

products can be obtained. This also leads to faster erection of the bridge, which results in avoiding extended detours and traffic tie-ups.[14]

3. Economy in fabrication and erection is achieved by press-forming the trough-section (instead of welding web plates to the bottom flange) and eliminating the need for any interior or exterior stiffeners or diaphragms.
4. The weight of a typical unit (girder) is about 330 plf. Thus, a 65-ft long girder of 42-in. depth (adequate for a 65-ft span) weighs about 11 tons only. This would enable the use of low capacity equipment for handling, transportation, and erection, resulting in reduced construction cost.
5. Large top flange widths provide safe working space during erection and minimize the number of girders required for a bridge of given width.
6. The shallow box girders are esthetically appealing, due to their slenderness and clean underside appearance.
7. It is possible to stiffen and strengthen the bridge should it become necessary to raise the load limit. This can be done by welding steel plates to the bottom flanges of the girder. Similarly, additional girders can be easily welded to the existing ones for future widening of a bridge.

DESIGN

The girder unit as a whole has been designed as a simple beam,[1,2] whereas the ribs and the supporting diaphragms have been designed according to orthotropic theory.[3,26,27] The selection of the ⅜-in. thick top flange is based upon the minimum thickness requirements from practical considerations.[16] Based on the authors' design calculations, inverted WT5×7.5 ribs @ 12 in. c/c with inverted WT8×13 diaphragms @ 6 ft-6 in. c/c resulted in an optimum weight of the superstructure and are suggested in the proposed system.

The AASHTO specifications[1] do not provide any criteria for design with press-formed steel members. Consequently, the design of the webs and the bottom flange of the proposed

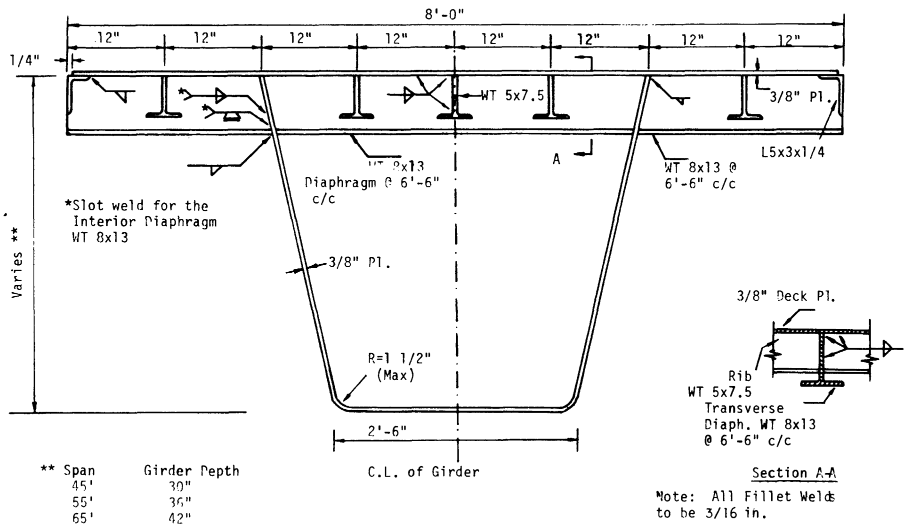

Fig. 2. Girder section showing fabrication details (8 ft wide) for 65-ft span

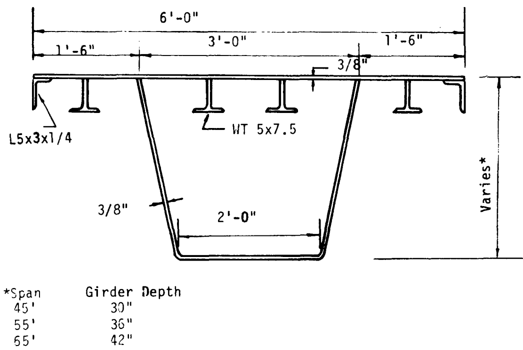

Fig. 3. Girder section (6 ft wide) for 45-ft span

girder has been checked according to the AISI specification.[4]

The crippling resistance of the unstiffened webs over the bearings is rather low, due to bends at the junction of bottom flange and webs. For this reason, a 5 × ½ interior stiffener, continuous along the bottom flange, has been provided, as shown in Fig. 4. In addition, a ⅜-in. thick steel plate as a full-depth diaphragm has been provided between the flanges and the webs, giving additional stiffness. Typical section properties and stresses for the proposed system are given in Table 1. It can be seen that the design is governed by the live load deflection, rather than strength. A superimposed dead load of 25 psf has been considered in design, conservatively, although thin lightweight wearing surfaces weighing only 3 to 5 psf, with a thickness of about ¾-in., are available.[3,9]

FABRICATION

To cut down fabrication costs, a minimum number of welded connections has been suggested. The girder webs do not have any interior or exterior stiffeners. The trough section can be cold-formed on hydraulic press-brakes, thus eliminating the flange-web welding.

The ribs and the transverse diaphragms are welded to the deck plate before welding to it the press-formed trough section. All the required welding can be done by an automatic process.[3,10,14]

Slots are precut in the inclined girder webs to align with the interior transverse diaphragms (WT8×13). These slots are plug-welded and machine-finished. The exterior diaphragms, which must align with the interior ones, are then welded to webs. These details are shown in Fig. 2. The ends of the girder are closed by welding a ⅜-in. plate diaphragm across the opening to the webs and the flanges, as shown in Fig. 4.

The welding indicated in Figs. 1 through 4 is schematic only. References 3 and 10 are good guides to select proper welding methods and sequence, to minimize the residual stresses and fabrication costs. All welding has been designed for resistance to fatigue.[1,34]

FEASIBILITY STUDY

An extensive literature search[24,25] revealed that the potential of press-formed box-type bridge systems has not been explored before. A bridge system with a steel plate deck supported on I-beams, which is apparently less efficient and less versatile than the proposed system, is reported in the literature.[9,12,29,31] Furthermore, in bridge construction, the use of press-formed steel members has been, to date, limited only to stay-in-place deck forms and guard rails.[30]

To ascertain the practicality of the proposed system, an inquiry with several fabricators/producers, including U.S. Steel Corp. and Bethlehem Steel Corp., indicated that:

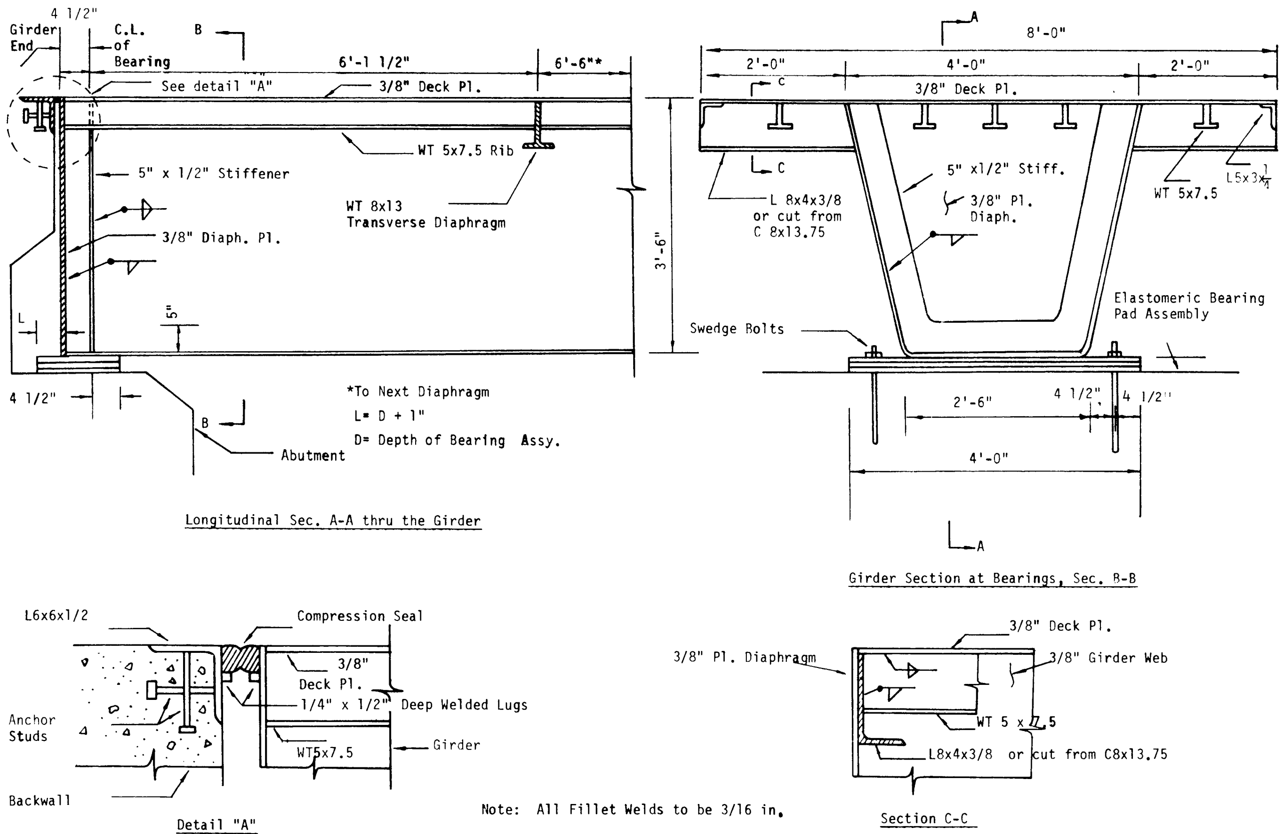

Fig. 4. Bearing end details

Table 1. Summary of Section Properties and Design Stresses for Press-Formed Steel Box Girder

Parameters	Girder Size					
	6 ft wide			8 ft wide		
	72″ × 30″	72″ × 36″	72″ × 42″	96″ × 30″	96″ × 36″	96″ × 42″
Top flange width, ft	6	6	6	8	8	8
Bottom flange width, ft	2	2	2	2.5	2.5	2.5
Overhang, ft	1.5	1.5	1.5	2	2	2
Total depth, ft	2.5	3.0	3.5	2.5	3.0	3.5
Deck plate thickness, in.	3/8	3/8	3/8	3/8	3/8	3/8
Bottom flange thickness, in.	3/8	3/8	3/8	3/8	3/8	3/8
Web thickness, in.	3/8	3/8	3/8	3/8	3/8	3/8
N.A. from bottom fibers, in.	20.74	24.61	28.38	21.32	24.35	29.27
Moment of inertia, in.^{4}	8,734	13,626	19,775	10,623	16,263	22,806
Sect. modulus, top fibers, S_t, in.^{3}	943.2	1,196.3	1,451.9	1,223.8	1,396.0	1,791.5
Sect. modulus bottom fibers, S_b, in.^{3}	421.1	553.7	696.8	498.3	667.9	779.2
Top fiber stress, ksi	7.506	8.045	8.43	6.897	8.189	8.172
Bottom fiber stress, ksi	16.813	17.381	17.566	16.940	17.117	18.789
Max. span capacity, ft	45	55	65	45	55	65
Dead load deflection, in.*	0.14	0.21	0.29	0.15	0.23	0.32
Live load deflection, in.**	0.65	0.825	0.973	0.64	0.82	1.0
Allowable live load deflection, in.†	0.675	0.825	0.975	0.675	0.825	0.975
Weight, plf (no wearing surface)	220	235	250	296	310	330
Weight, psf (no wearing surface)	37	39	42	37	39	41

* Dead load includes 25 psf of wearing surface.
** For maximum span.
† $L/800$ for maximum span.

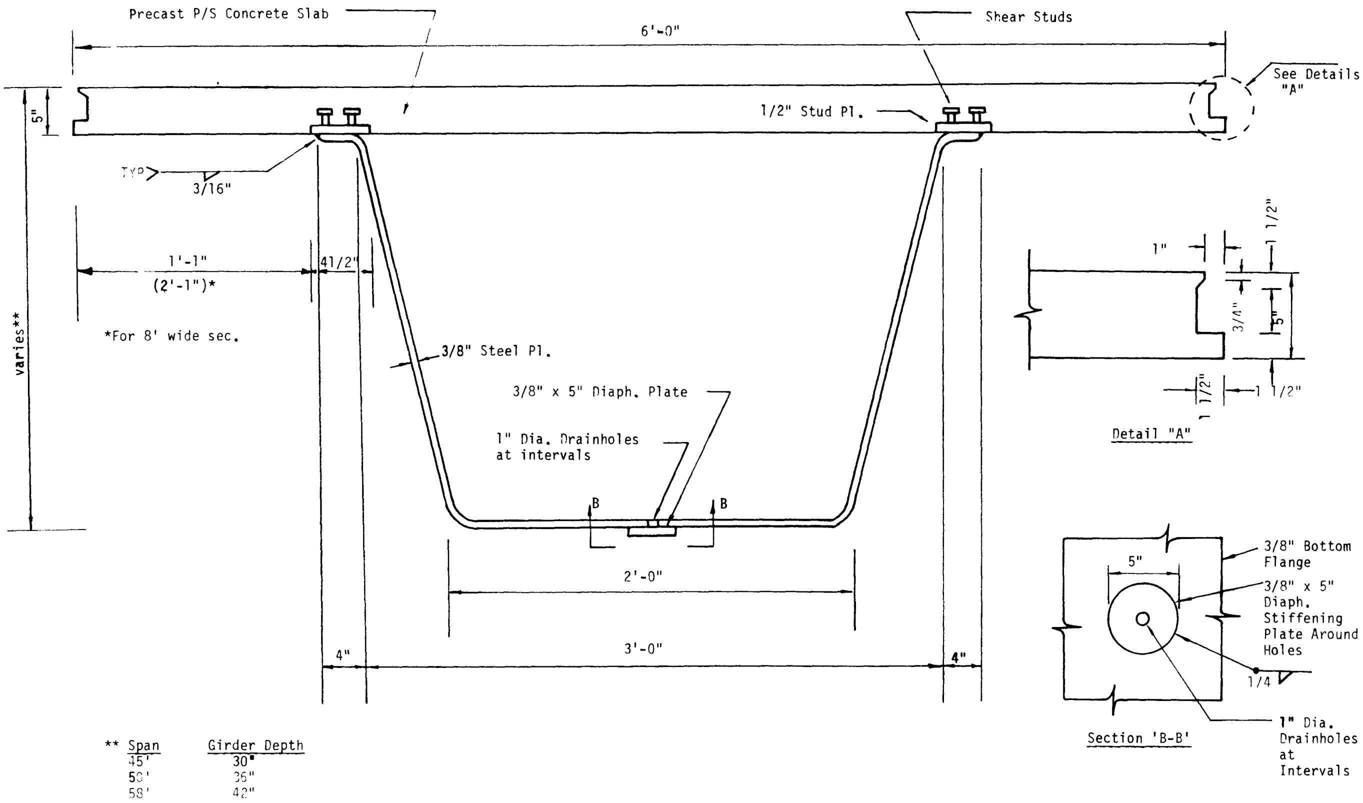

Fig. 5. Composite box girder for 45-ft span

1. Plates up to 1½-in. thickness and 56-ft length can be brake-formed by hydraulic press brakes currently in use. The widths between the adjacent bends of a section, limited to 38¼-in. by the throat capacity (36¼-in. horizontal), can be increased to more than 50 in. by minor alterations in the operation.
2. For ASTM A36 steel, the minimum radius that can be obtained for the corners of the trough section is 1.25 times the plate thickness. A smaller radius could develop cracking at the corners.
3. The press-formed section was estimated to be about 35% cheaper than a section welded between the flanges and the webs.

Thus, it was concluded that the proposed section is economically feasible from the industry's viewpoint.

ERECTION

Due to the low weight of the girders, only one, or at the most two, 20-ton cranes are required for their installation. During the erection, longitudinal welding between the edges of the top flanges of the adjacent girders along their entire length, to prevent leakage through the longitudinal joint and for transverse load distribution, is the only major on-site work. To overcome unequal deflection of two adjacent girders, temporary fitting aids can be field-welded at the required points which will be used to level the girders.

BEARINGS

Typical details at the bearings are shown in Fig. 4. Elastomeric bearing pads, such as Fabreeka or equal, are provided at each bearing.[9,29]

Since the bottom flange terminates flush with the end plate diaphragm, the sole plate (welded to its underside) is extended on the sides, as shown in Fig. 4, to facilitate joint connection at the abutment.

END JOINTS

Transverse roadway joints between the abutment and the girders are sealed with armored preformed compression seals, as shown in Fig. 4. Transverse lugs (¼-in. x ½-in.) are welded on the outside faces of the end diaphragm plates to provide seats for the compression seals.

CURB, PARAPET AND RAILING

A typical arrangement with 3½ × 3½ × 0.25-in. structural tube railing and rail posts (both galvanized) is shown in Fig. 1. In combination with this, a rub rail section or a press-formed L-section can be used to provide curbs. Additional information on curbs and railings can be found in Ref. 26.

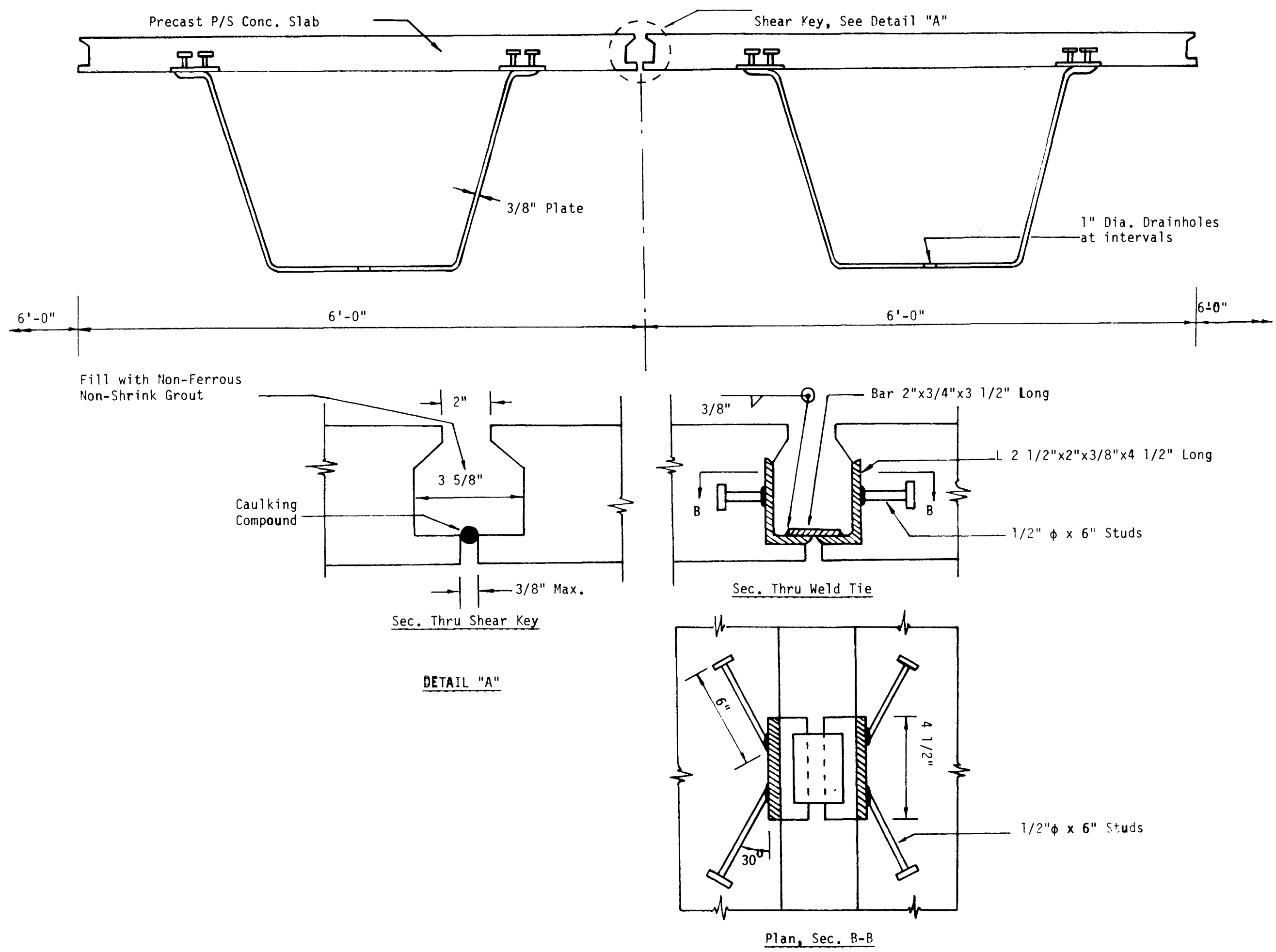

Fig. 6. Shear key details

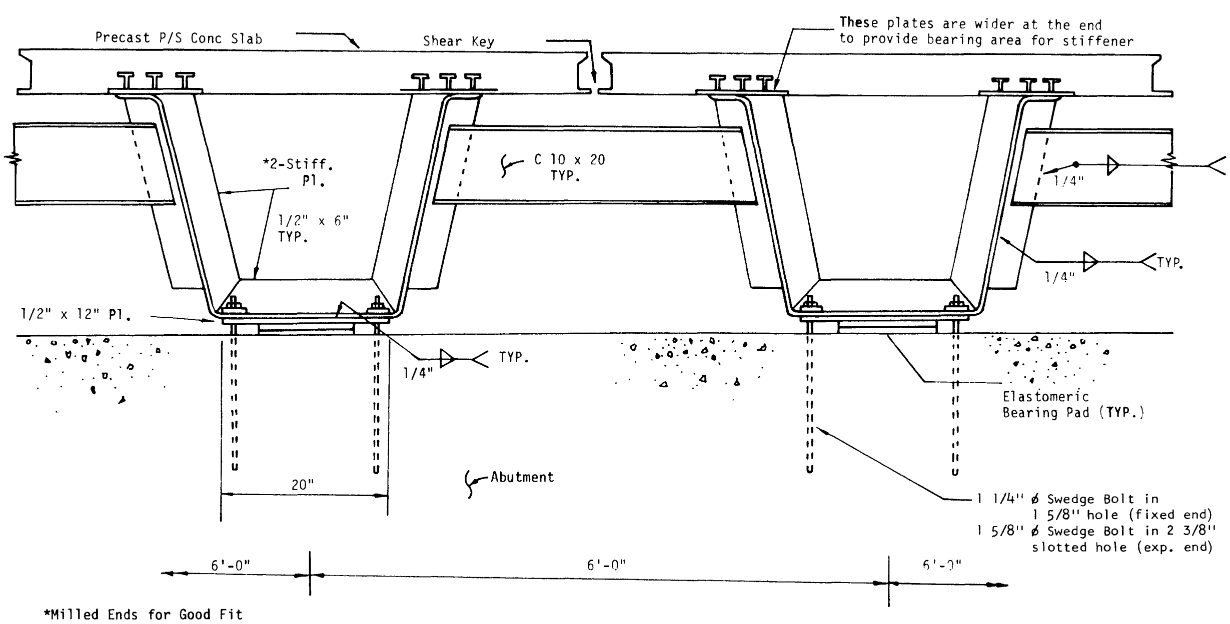

Fig. 7. Section through deck near abutment, showing end stiffeners and diaphragms

Table 2. Summary of Section Properties and Design Stresses for Composite Box Girder

Parameters	Girder Size 6 ft wide		
	72″ × 30″	72″ × 36″	72″ × 43″
Top flange width, ft	6	6	6
Bottom flange width, ft	2	2	2
Overhang, ft	1.5	1.5	1.5
Total depth, in.	30	36	42
Plank thickness, in.	5	5	5
Bottom flange thickness, in.	3/8	3/8	3/8
Web thickness, in.	3/8	3/8	3/8
N. A. from bottom fibers, in.	19.15	24.46	28.38
Moment of inertia*, in.^{4}	8,304	12,810	18,730
Sect. modulus, top, in.^{3}*	765	1,110	1,375
Sect. modulus, bottom, in.^{3}*	434	523.7	660
Stress, top fibers (conc.), psi	1,197	974	984
Stress, bottom fibers, psi	18,310	18,610	18,510
Maximum span, ft	45	50	58
Deflection (D. L.) in.**	0.44	0.28	0.47
Deflection (L. L.) in.	0.69	0.64	0.71
Allowable deflection, in.†	0.68	0.75	0.87
Weight, plf	500	516	531
Weight, psf	83	86	89

* Composite section, $n = 8, f'_c = 4{,}000$ psi.
** 30 psf superimposed dead load incl. for wearing surface.
† For maximum span, $L/800$.

WEARING COURSE

Experience with several existing bridges having unprotected checkered or patterned plate steel decks shows that their resistance to corrosion and wear is good. However, their skid-resistance is not satisfactory, particularly when the deck is subject to icing.

The epoxy asphalt (resinous wearing surface) manufactured by Adhesive Engineering Company of San Carlos, Calif., is judged to be promising[21] in terms of self-weight, skid-resistance, durability, bonding qualities, and maintenance aspects. Hence, it is suggested for the proposed system. The installed cost of this type of wearing surface has been estimated to be about $4.50 per sq ft.[21] Additional information on wearing surfaces can be obtained from Refs. 8, 9, 21, and 28.

MAINTENANCE ASPECTS

Special care has been taken in detailing the system units to avoid sharp corners, stiffeners, bolt heads, etc., in order to minimize the potential for the collection of debris and consequent corrosion, and stress raisers. Accordingly, the corners at the bottom flange of the girders have been rounded off and edges of plates at the girder ends have been eliminated.

Investigations on several bridges with hollow members that have been in service for over 60 years have shown no signs of moisture or corrosion on the inside surfaces.[3,5,32] Accordingly, no corrosion-protective treatment is suggested for the interior of the proposed girders. Recent developments in the paint industry claim the service life of some paints to be more than twenty years without any maintenance.[7,11] Therefore, it is preferable to paint the exterior faces of the girders, unless weathering steel, such as Corten or Mayari-R, is used for fabrication.[22]

ALTERNATIVE DESIGN

As an alternative to the steel plate deck of the proposed system, it is entirely feasible to have a deck made from precast, prestressed planks, as shown in Fig. 5. The design of this system is very similar to the all-steel design, except that the deck plate is replaced by 5-in. thick precast prestressed concrete panels. A few simple modifications are required in the all-steel design when concrete planks are used:

1. At the junction of the two adjacent beams, shear keys with weld-ties are provided to distribute live load laterally, as shown in Fig. 6.
2. The girders are sealed at the ends. To drain out any moisture that might creep in through the concrete deck, drainage holes are provided in the bottom flange, as shown in Fig. 5. This arrangement is considered adequate to prevent the corrosion of the interior of the girder. The inside faces of steel plates need not be painted.[5]
3. The concrete plank is precast first, with the stud-plate embedded in it. The whole assembly is then shop-welded to the steel trough-section.

Typical bearing details are shown in Fig. 7. The precast concrete planks are nominally prestressed to minimize shrinkage cracks.

A summary of section properties and design stresses is given in Table 2 for 6-ft wide sections. As can be expected, the superstructure weight of this system exceeds the weight of the all-steel system by about 35 psf. This is reflected in the corresponding lower span capacity.

CONCLUSIONS

An all-steel, all-welded box-girder system for short-span highway bridges designed for HS20-44 loading has been presented. For spans up to 65 ft, the proposed superstructure is light and weighs only about 40–45 psf, as compared with over 100 psf for a conventional slab-stringer bridge system. The system is developed from the viewpoints of segmental assemblage, low initial and maintenance costs, and ease of future expansion.

As an alternative to the steel plate deck, a design with precast, prestressed concrete deck that acts compositely with the steel section is also presented. This system weighs about 85 psf. It is lighter than the conventional slab-stringer system, and lends itself better to modular construction.

Six different girder sections have been presented with a maximum span capacity of 65 ft. The girders have widths of 6 and 8 ft and depths of 30, 36, and 42 in. A suitable combination of these sections can be used to achieve the required bridge width. By increasing the depth of the girders, the system can be used for longer spans.

The proposed system has been designed under the constraints of 6 and 8-ft wide girder units, from the viewpoint of modular systems. However, further optimization can be achieved by increasing the flange widths to 10 or 12 ft. This would decrease the unit superstructure weight to about 35 psf. With deeper girders and thicker webs and flanges, the system is adaptable to longer spans.

ACKNOWLEDGEMENTS

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