APPLICATION OF PREFABRICATED HOLLOW STEEL DECKS TO BRIDGE CONSTRUCTIONS

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At the 10th Congress of International Association for Bridge and Structural Engineering in Tokyo, in 1976, the authors introduced a prefabricated hollow steel deck as a floor system for long-span suspension bridges. In this paper, they are proposing the application of this new deck to the main structure of smaller bridges. This deck has been used in bridges and floor systems in Japan. Here, the authors discuss the structural features and fabrication of this deck and present examples of its application to bridge construction. They also discuss the problems of design and construction of bridges using this deck, the methods of solving these problems, and the experimental studies conducted on the deck. This hollow steel deck is a prefabricated slab structure made up of two face plates with evenly-spaced core plates arranged diagonally between them. Tests have shown that this deck improves a bridge's lateral and torsional rigidity as well as its shearing resistance and fatigue strength. In that this deck is a prefabricated structure, its application facilitates bridge construction.
Furthermore, the total thickness of the bridge structure can be reduced, thus improving its aesthetic appearance.

The steel deck, a light-weight, prefabricated structure, has been used for about twenty years for bridge and floor-system construction not only because of its high strength, but also because it helped to increase economy, safety, and the speed of completion. This conventional orthotropic steel deck, however, having an upper face plate stiffened with longitudinal and lateral ribs, has a low level of lateral and torsional rigidity.
Recently, a new type of steel deck, having two face plates, both a top and a bottom, connected by longitudinal ribs only, was developed by the authors. This prefabricated structure was intended primarily for decks and floor systems, but it is also applicable to bridges with a span of up to about 30 meters, increasing both aesthetic appearance and degree of prefabrication. This hollow steel deck is not applicable to briges with a long span in that they require box girders.
A bridge using this hollow steel deck as its main structure can be referred to as a "slab bridge".

Details concerning the fabrication of this hollow steel deck will be discussed later. It may be helpful at this point, however, to refer breifly to Figure 10. Please note that the bottom plate is actually a series of bands welded longitudinally to the core plates. Thus welding is performed in only one direction.

Figure 1. Completed hollow steel deck showing cross section.
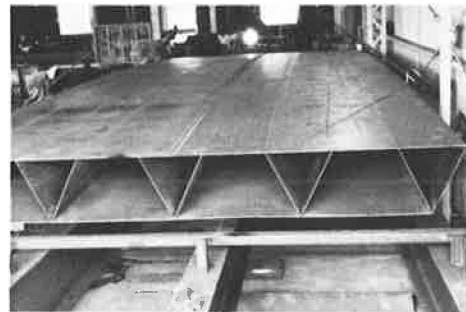
Features and Problems

A bridge using the hollow steel deck has the following advantages:

1. Greater load-carrying capacity, since both top and bottom face plates act effectively against an applied load.
2. Higher accuracy fabrication due to less distortion from welding (less lateral welding).
3. Easier and faster construction.
4. A more slender and aesthetic appearance due to reduced thickness.

There are the following problems in applying the hollow steel deck to bridges:

1. Limitation of application to short-span bridges.
2. Due to the closed nature of the deck:
   a. Splicing units together is difficult.
   b. Inspecting interior welding is difficult.
   c. Interior corrosion may be a problem.

Experimental Studies

Various experimental studies were made to obtain basic data on the behavior and characteristics of the hollow steel deck. Three of these will be discussed here.

Test One—Bending and Torsional Rigidity

In this study the authors intended to determine whether the conventional, practical method of analysis could be applied to the analysis of the hollow steel deck subjected to a concentrated load. That is, the tests were planned to obtain data on the longitudinal bending rigidity (Dx), lateral bending rigidity (Dy), and effective torsional rigidity (H) of the deck. These data required for practical analysis of a slab bridge by the Guyon-Massonnet method.

First, from a test it was concluded that Dx can be calculated by using the moment inertia of the section. Since Dy can not be determined so simply, however, the authors performed an additional test to determine its value.

The test to determine lateral bending rigidity (Dy) of decks with different cross-sectional shapes was carried out using six model specimens. Each was 20 cm wide. Other dimensions are shown in Table 1.

Table 1. Dimensions of model specimens for test of lateral bending rigidity.

(Unit: mm)

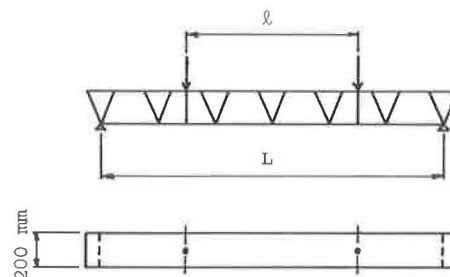| Dimension	Test Specimens					
	A	B	C	D	E	F
a	0	100	150	150	225	300
b	135	150	150	150	150	150
h	108	138	141	140	138	138
S	127	158	159.	157	157	157
t_1	6	6	4.5	6	6	6
t_2	6	6	4.5	4.5	6	6
t_3	6	6	4.5	4.5	6	6
L	1350	1500	1800	1800	1875	2250
ℓ	675	750	600	600	750	900

Remarks: 1)
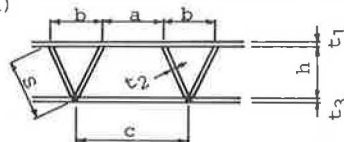
b) L and ℓ are shown in Fig. 2.

Note: 1 mm = 0.03937 in.

Each specimen was loaded as shown in Figure 2. Deflection was measured at several load levels. From these measurements, Dy was calculated for each specimen.
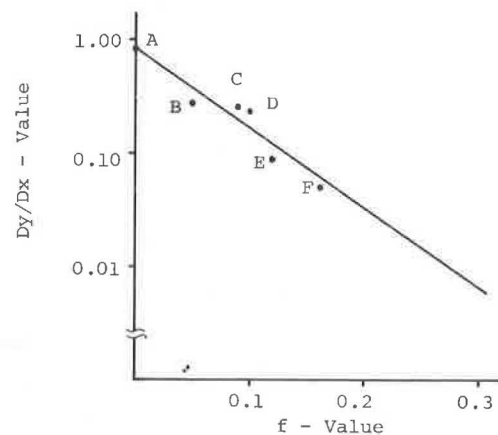
The relationship between rigidity and cross-sectional shape was determined by constructing the graph in Figure 3.

Figure 2. Loading of model specimen for test of lateral bending rigidity.

Note: 1 mm = 0.03937 in.

Figure 3. Relationship between rigidity and cross-sectional shape

In this graph f is a coefficient calculated from the dimensions of the specimen (Table 1) according to the following formula:
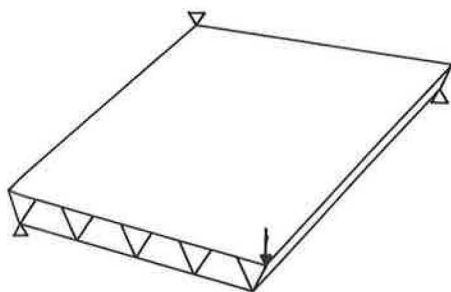
$$f = \frac{a}{h\left(t_1 + \dfrac{2t_2 S}{3(a+b)} + t_3\right)}$$

The f—Dx/Dy curve can be expressed by the following formula:

$$\log \frac{Dy}{Dx} = -0.077 - 6.97f$$

The test to determine torsional rigidity (II) was performed as shown in Figure 4. From the results, the value of H was found to be roughly equal to Dy.
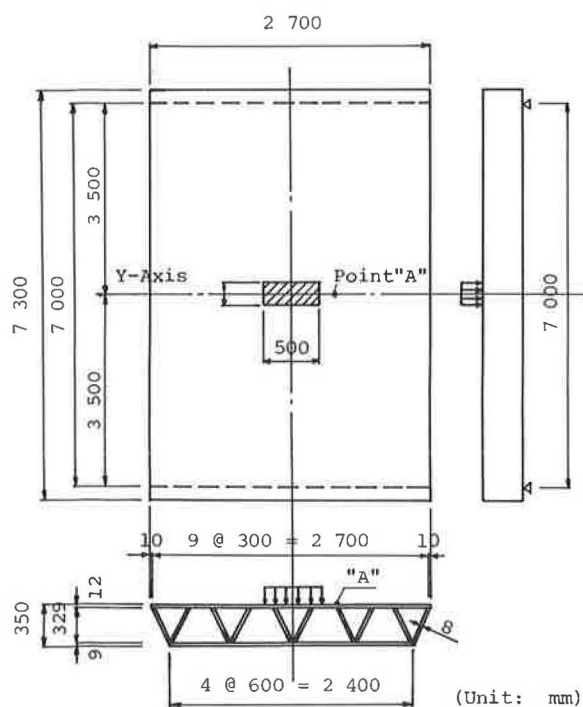
Figure 4. Test for torsional rigidity

Test Two—Deflection and Stress

'The test to determine deflection and stress was performed using the sample deck shown in Figure 5. The position of this load is also shown in Figure 5.

Figure 5. Dimensions and loading of sample deck used in Tests Two and Three.


(Unit: mm)

Note: 1 mm = 0.0397 in.

Figures 6 and 7 show the measured values for deflection of top face plate along the X and Y axes, respectively; under a 98 kN (22 kip.) load. Using the data obtained in Test One, deflection for this sample was also calculated according to both the Guyon-Massonnet method and the finite strip method. These values are also shown in Figures 5 and 6.
The values calculated according to the Guyon-Massonnet method are smaller than the measured values presumably, because the calculation in Test One did not take into account the effect of shearing force.
Figure 8 shows both the measured values for strain of the top face plate on the Y-axis under a load of 98 kN (22 kip.) and those calculated according to the finite strip method. One can see from the graph that the top face plate under the load acts on the Y-axis as a continuous plate.

Figure 6. Deflection along X-axis under a load of 98 KN.
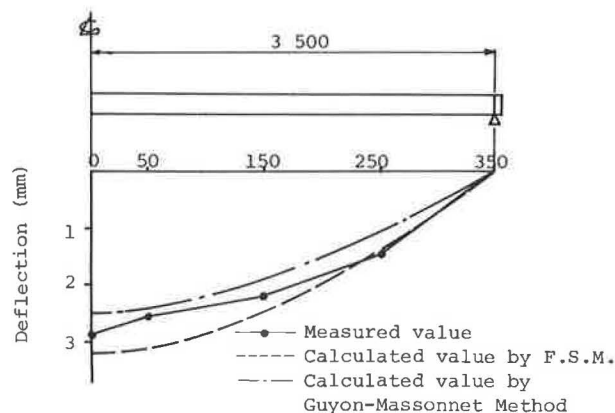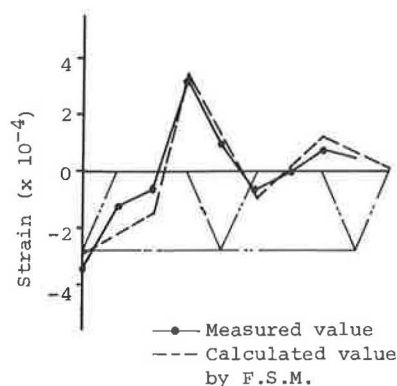

Note: 1 mm = 0.03937 in.

Figure 7. Deflection along Y-axis under a load of 98 kN.
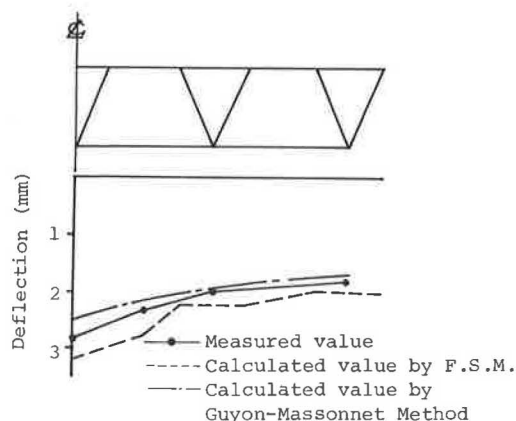
Note: 1 mm = 0.03937 in.

Figure 8. Strain on top plate along Y-axis under a load of 98 kN.

These results suggest that the values for deflection and strain calculated according to the finite strip method are reasonable approximations of the actual values. The authors intend to continue their investigation concerning the application of the Guyon-Massonnet method.

Test Three—Fatigue Strength

To test fatigue strength, the sample deck used in Test Two was subjected to cyclic loads at the same point of loading. First, the deck was subjected to a load of 9.8 kN (2.2 kip.) to 225 kN (50.6 kip.) for 2,000,000 cycles. The maximum load was increased to 343 kN (77.2 kip.) for an additional 1,000,000 cycles. Then, the maximum load was increased to 465 kN (104.7 kip.) for an additional 1,000,000 cycles. The calculated stress from measured strain of the top face plate at paint A under a load of 465 kN was then 164.4 MPa (23,870 lbf/in.^{2}). Finally, the deck was subjected to a more concentrated load with lower maximum for another 5,000,000 cycles at a position on the other side of point A. The stress at point A then reached 226.38 MPa (32,825 lbf/in.^{2}), but no fatigue crack occurred. This test confirms that this hollow steel deck has great fatigue strength.

Design of Slab Bridge

Structural Analysis

The construction and design of a bridge must take into account deflection and stress imposed on each structural member by a live load. When a hollow steel deck is used this analysis becomes complicated by the complicity of elements.

Beam Theory Method. If the bridge will be subjected to a uniform load or if lateral bending moment is not a consideration, stress and deflection can be calculated by using the common beam theory. In this case, the entire assembled deck can be considered as a single beam. Pedstrian bridges are designed by using this method.

Orthotropic Plate Deck Theory Method. If the bridge is to be subjected to a concentrated or non-formly distributed (one-side) load, the beam theory is not applicable. Rather, the hollow deck must be considered as an orthotropic plate. Stress and deflection can be calculated by using the Guyon-Massonnet method. As mentioned earlier, however, values calculated according to this method are less than actual measurements. This problem has not yet been completely solved.

Finite Strip Method. Stress and deflection can be more exactly calculated by using the finite strip method, but the calculations become much more complicated. In this case, each element of a given deck is considered as a finite strip. Stress and deflection are calculated for each strip. This analysis makes use of a computer program designed by the authors specifically for this purpose.

It should be pointed out that the cross-sectional shape and dimensions of the hollow steel deck are determined by allowable deflection, rather than by allowable stress as is the usual case for other bridge structures. In Japan the allowable limitation of deflection due to the live load is $\delta/\ell=1/600$ (δ: deflection due to the live load, ℓ: span length).

Unit Weight and Slab Thickness

Using the Guyon-Massonnet method approximation for designing slab bridges have been calculated. Figure 9 shows the relationship between unit weight and length of span for both roadway and pedestrian bridges. The unit weight here is exclusive of curbs, h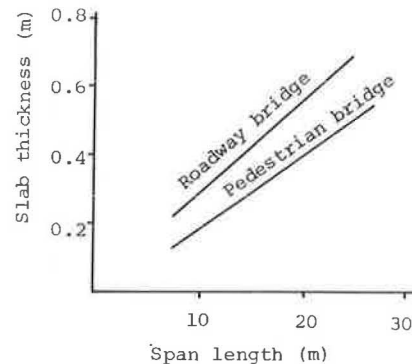andrails, shoes, and pavement. Figure 10 shows the relationship between slab thickness and length of span for both types of bridges.

Figure 9. Relationship between unit weight and length of span.
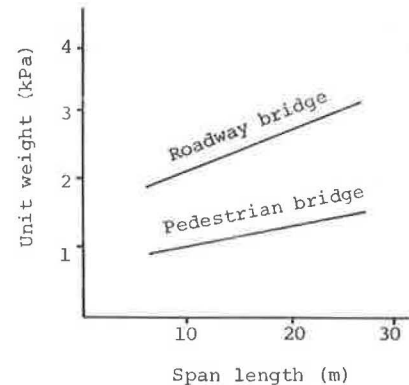
Note: 1 m = 3.28 ft. 1 kPa = 0.145 lbf/in.^{2}

Figure 10. Relationship between slab thickness and length of span.

Note: 1 m = 3.28ft.

Structural Details

Steel Plate Thickness. The relationships governing the thickness of each steel plate of hollow steel decks used in roadway and pedestrian bridges are shown in Table 2. Each figure represents the minimum thickness. t_1, t_2, and t_3 are explained in the sketch attached to Table 1.

Table 2. Minimum steel plate thickness of slab bridges.

(Unit: mm)

	Roadway Bridge	Pedestrian Bridge
t_1	0.035a or 12	0.02a or 6
t_2	8	4.5
t_3	0.0125c or 9	0.0125c or 6

Stiffener. Both ends of the slab bridge are closed by stiffeners (Figure 11). Additional stiffeners are placed at the point of each intermediate support of a bridge. These stiffeners, made of steel plate more than 12 mm (15/32 in.) thick, are welded securely to the top and bottom face plate as well as to the core plates. They provide for proper distribution of the reaction force, as well as securing perfect closure to inhibit interior rusting. They also serve as cross beams.

Figure 11. A hollow steel deck under fabrication showing stiffeners and spacing struts.

Pavement

For roadway bridges having steel decks it is a common practice to place a 7-8 cm (2 3/4 — 3 1/8 in.) layer of asphalt on the steel plate. Mastic asphalt is generally preferred. The Honshu-Shikoku Bridge Authority is now studying on the method of coating the steel plate surface with an adhesive, before applying the 3-4 cm (1 3/16 — 1 9/16 in.) layer of mastec asphalt, and then finishing with a 3-4 cm top layer of special asphalt fortified with a selected type of rubber or synthetic resin. Asphalt pavement is known to substantially improve the surface slip resistance.
Pedestrian bridges, on the other hand, are usually paved with a thin layer of asphalt belended with epoxy resin or tiled.

Corrosion

Admittedly, interior corrosion may be problem with the hollow steel deck, but this can be greatly inhibited by the perfect closure at both ends of a bridge with stiffeners, as mentioned earlier. Furthermore, the parts near the abutments at both ends of the bridge, which are not easily accessible for maintenance, can also be coated with a tar epoxy point known for its durable corrosion-inhibiting performance.
The use of weather-resistant steel plates will, of course, be another effective approach to overcome the corrosion problems.

Fabrication and Erection

The fabrication sequence for the hollow steel deck is shown in Figure 12.
Core plates forming evenly-spaced triangles are welded to a steel plate, actually the top face plate of the deck. Next the bottom face plate is welded strip by strip to the core plates.
First each set of two core plates is welded together at the prescribed angle as shown in Figure 13.

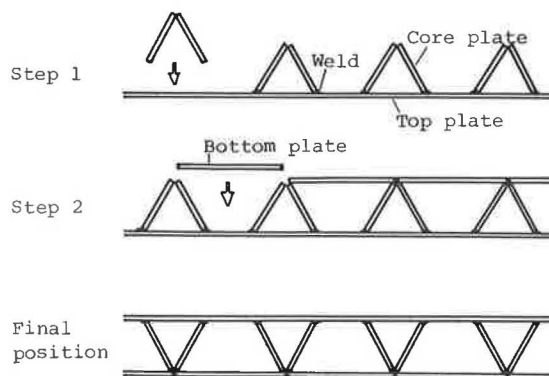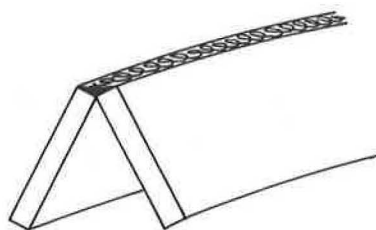Figure 12. Fabrication sequence of hollow steel deck.


Figure 13. Welding together of core plates.

The core plates are then welded to the top face plate either by the full penetration method using backing as shown in Figure 14 or by fillet welding as shown in Figure 15. In this step deformation from welding is relatively small, about the same as in the fabrication of conventional steel decks. If the bridge is to be cambered, the core plates can be cut and curved accordingly.

Figure 14. Full penetration method of welding   Figure 15. Fillet welding
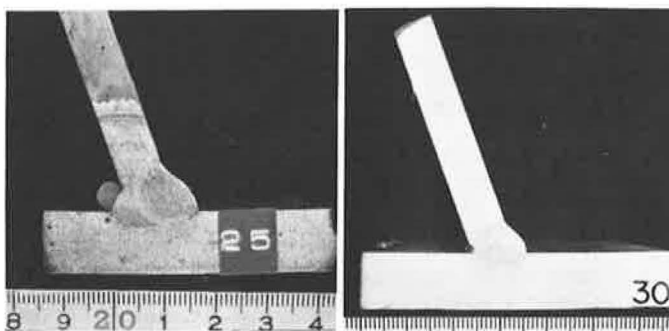
Next, stiffeners are welded where necessary (Figure 11). In order to strengthen the deck against deformation from welding in the next step, spacing struts, also shown in Figure 11, are welded into place. A plate is then welded between the tips of each set of core plates, as shown in Figure 16. These strips from the bottom face plate. The root opening between the strips is roughly equal to the plate thickness.

Figure 16. Welding of bottom face plates into position.
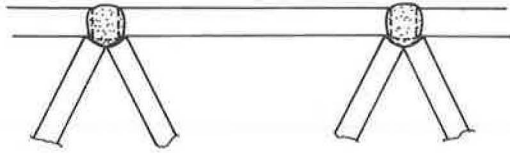
Automatic Welding Machine

Since welding involved in the fabrication of the hollow steel deck can all be performed in one direction, it is relatively easy to automate the welding operation. The authors have developed and tested an automatic welding machine shown in Figure 17.

Figure 17. Automatic welding machine
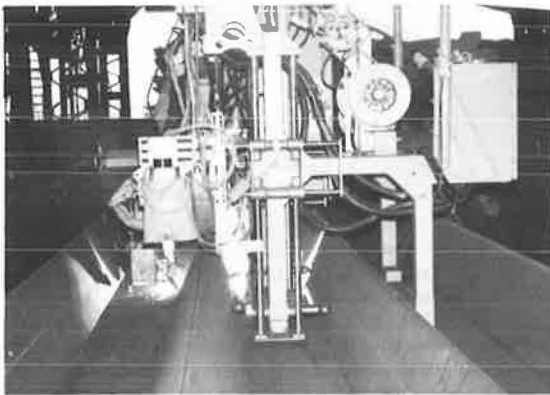
This proto-type can weld only one set of core plates at a time, and is, therefore, not practical for actual use. If further improvements enable automatic, simultaneous welding of a multitude of core plates, however, it will greatly facilitate fabrication of hollow steel decks.

Erection and Spacing

The proper size of the prefabricated units of a slab bridge depends on the mode and conditions of transportation from the shop to the site. In the case of land transportation by truck in Japan, usually, the dimensions of the load are limited to 25 m (33 ft.) in length, 3 m (10 ft.) in width, and 3 m in height. Hence the weight per unit deck does not exceed 200 kN (44 kip.). The most difficult part of the erection work is splicing together the prefabricated units. As shown in Figure 18, there are two modes of splicing involved in the erection of a slab bridge, namely longitudinal or side-on-side splicing and transverse or end-on-end splicing.

Longitudinal Splicing. Units which are to be longitudinally spliced have a slightly different design from that previously described. Edge core plates are added, resulting in a trapezoidal cross-sectional shape, wide at the bottom rather than at the top, as can be seen in Figure 11. These units are erected side by side with their bottom face plates adjoining, as shown in Figure 19a. First, the bottom face plates are joined by butt welding. Next, a top

face plate strip is welded into place, as shown in Figure 19b.

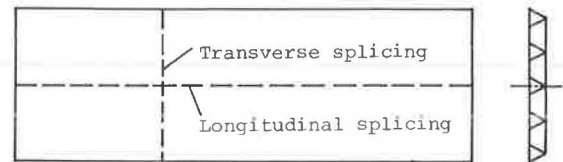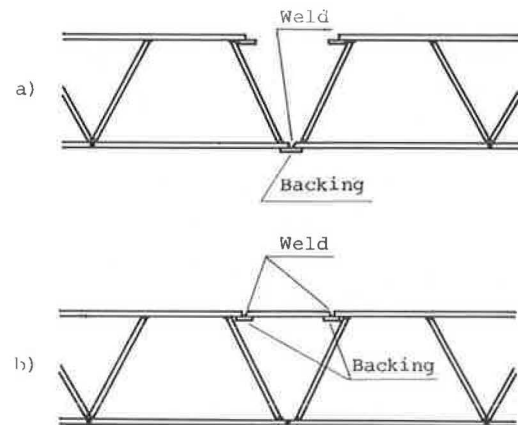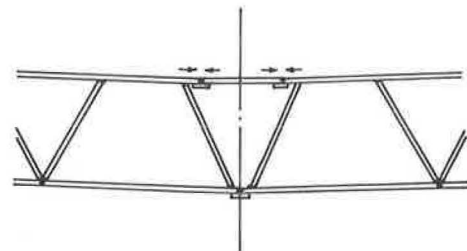Figure 18. Diagram of splicing.

Figure 19. Longitudinal splicing

Although deformation caused by welding is small during fabrication, it would be more marked in the process described above, as shown in Figure 20.

Figure 20. Possible distortion due to longitudinal splicing.

One countermeasure is the addition of splicing ribs, properly spaced, and welded to the core plates during fabrication (Figure 21). Before welding on the top face plates, these ribs can be bolted together. They also help to position the units properly, facilitating the welding.

Transverse Splicing. Transverse splicing of hollow steel decks is very complicated and difficult. It has, however, been successful in all three cases where it has been tried—one pedestrian bridge and' two roadway bridges.

Again, the decks must be specially designed. In this case, the top of the end section of each deck to be spliced is left open. After the bottom face plate and the core plates are joined, the top

Figure 21. Splicing rib
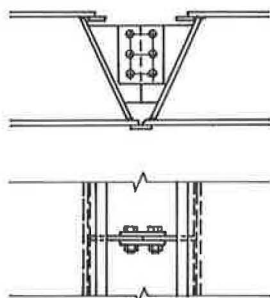
face plates are welded on to the core plates in strips, the same as the bottom face plates in the fabrication sequence. In the case of the roadway bridges, both the bottom face plates and the core plates were bolted. The bottom face plates of the pedestrian bridge, however, were welded, as shown in Figure 22.

Figure 22. Transverse splicing for pedestrian bridge.
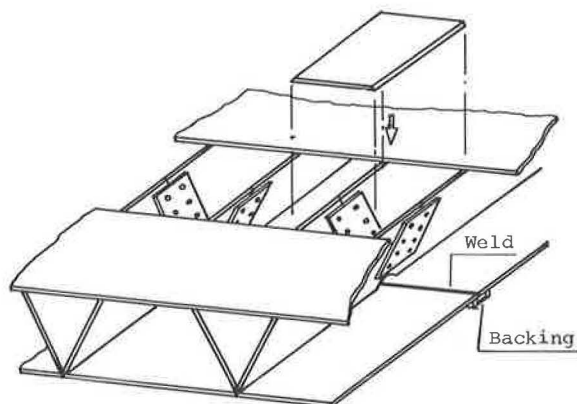
Examples of Application

Pedestrian Bridge

Prefabricated hollow steel decks have been used in the construction of many pedestrian bridges. Figure 23 shows one example.

Figure 23. View of pedestrian bridge.

The dimensions of this bridge are given in Figure 24. This bridge was designed for a uniformly distributed live load of 4.9 kPa (102 PSF).
With a slab thickness of only 1/50 the length of span, the bridge has a very slender and lightweight appearance. The two prefabricated units 19 m (62 ft.) and 12.5 m (41 ft.) long, and each 3 m (9.8 ft.) wide, were transported and erected by truck crane. The decks were joined by two pin-type hinges at the point A in Figure 24, hence no field welding of the decks was required. A circular pipe welded securely into the deck during fabrication, was welded at point B to the same size column pier at the site.

Roadway Bridge

Figure 25 shows the dimensions of a roadway bridge applying the hollow steel deck. This bridge was designed for a 137 kN (30.8 kip.) truck loaded as specified in the Japanese Highway Bridge Specifications. Prefabricated units in sets of two were delivered to the site and joined by longitudinal splicing along the center line of the bridge as described above. Each prefabricated unit was complete with curb and shoes, the latter designed with a sandwiches core of sheet rubber.

Temporary Movable Bridge

Figure 26 shows a temporary movable bridge constructed with a hollow steel deck. This bridge was erected below the high water level across a river for transportation of soil for the construction of an expressway.
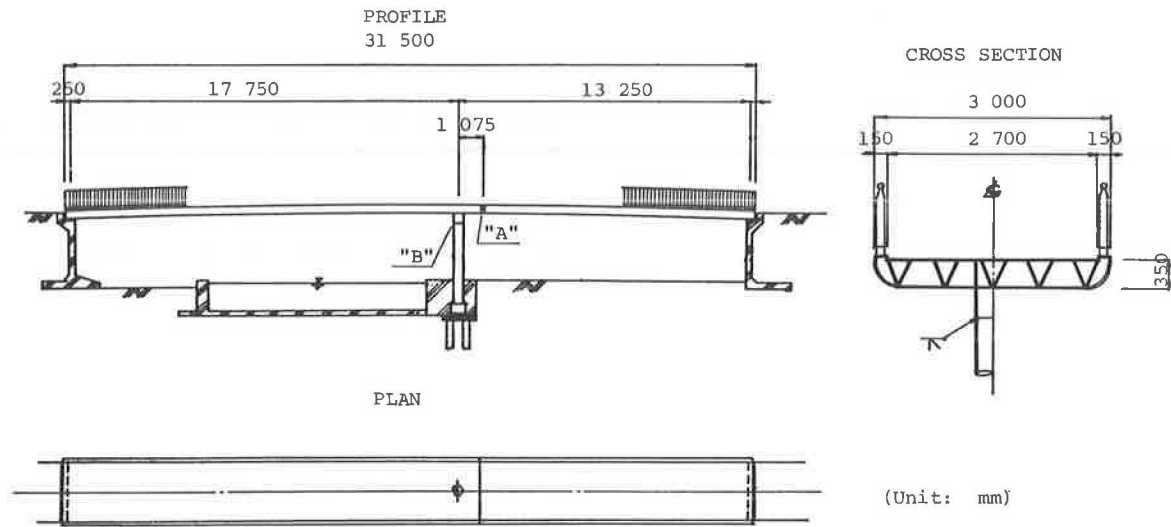
Figure 26. View of temporary movable bridge

Special wheels were installed in the decks, and rollers to the top of the pier and the abutment, as shown in Figure 27. Thus, when the river floods, the superstructures can be moved to a higher elevation along rails on the approach road. The use of the hollow steel deck for this kind of bridge allows for a lower and lighter weight structure with greater rigidity.
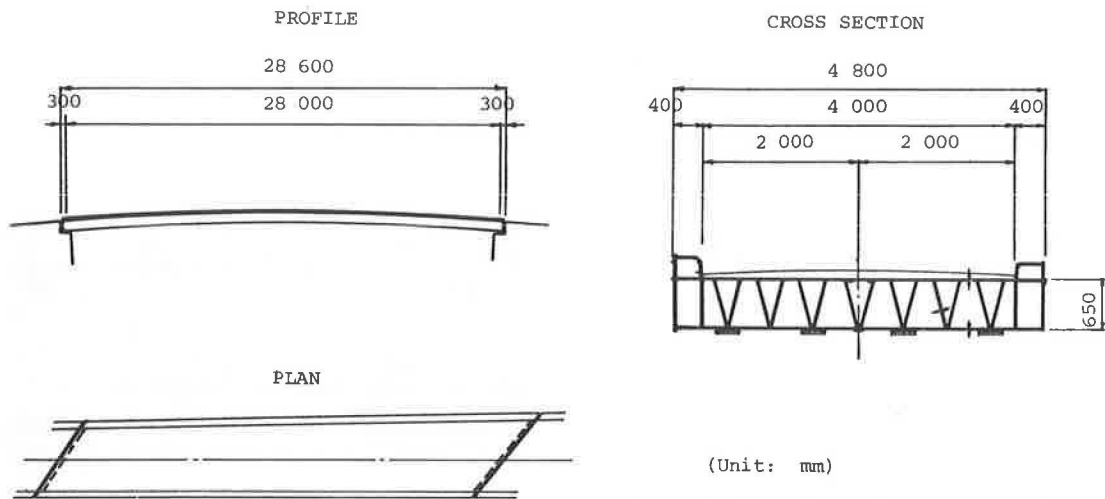
These three examples of bridges successfully contructed and in use in Japan show the practical application of this hollow steel deck to bridge design. Within the accepted limitations this new type of prefabricated unit has several advantages over those currently in use. The authors are confident that the difficulties involved in designing and contructing bridges with this deck can be effectively overcome. Through continued research and development the authors hope that this prefabricated hollow steel deck will have even wider applications.
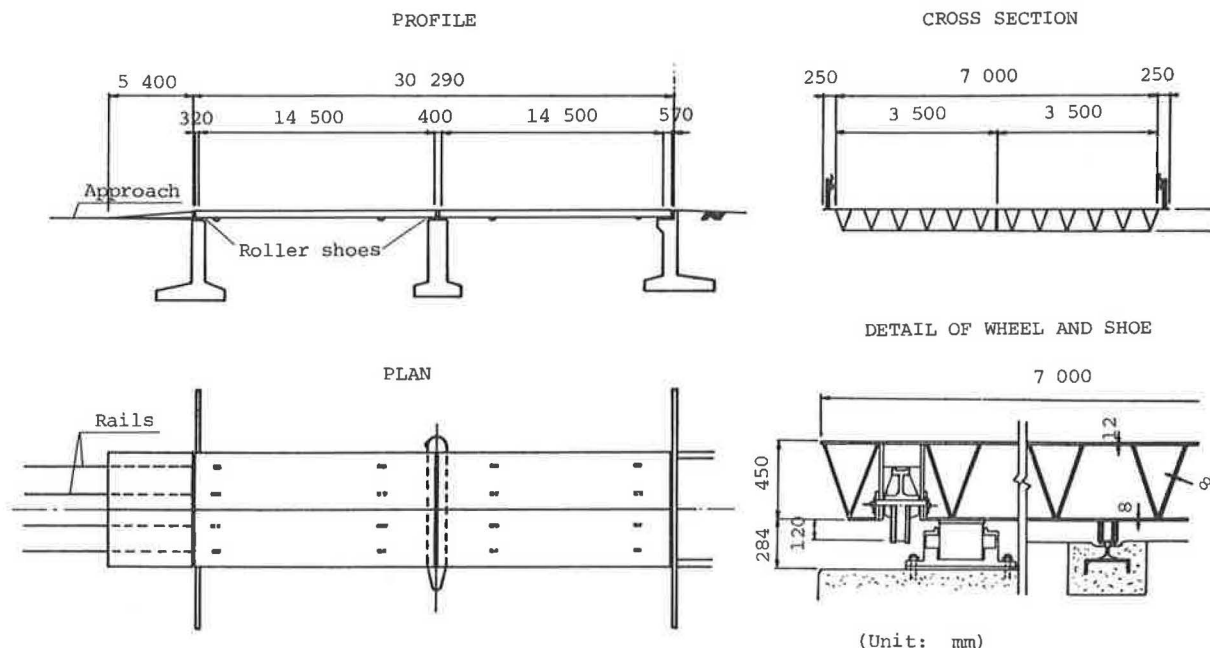
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Figure 24. General view of pedestrian bridge.

PROFILE
31 500

CROSS SECTION

250   17 750   13 250

1 075

3 000

150   2 700   150

"B"   "A"

350

PLAN

(Unit: mm)

Note: 1 mm = 0.03937 in. 1 m = 3.28 ft.

Figure 25. General view of roadway bridge.

PROFILE

CROSS SECTION

28 600

4 800

300   28 000   300

400   4 000   400

2 000   2 000

650

PLAN

(Unit: mm)

Note: 1 mm = 0.03937 in 1 m = 3.28 ft.

Figure 27. General view of temporary movable bridge.

Note: 1 mm = 0.03937 in. 1 m = 3.28 ft.

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