ORTHOTROPIC STEEL DECKS
FOR HIGHWAY BRIDGES

Condensation of Paper
by
Robert F. Wellner
Sales Engineer
Bethlehem Steel Corporation
Bethlehem, Pennsylvania

The complete paper discusses the following major topics: characteristics and function of the components of orthotropic bridges including surfacing; basic concepts of design and analysis; and fabrication and erection procedures.

The presentation was illustrated by sketches, drawings, and photographs of a number of orthotropic steel deck bridges built or under construction in this country and abroad.

INTRODUCTION

Orthotropic bridge construction was first introduced in Europe following World War II during the replacement program for bridges destroyed in the war. One of the most significant advantages of orthotropic deck construction is a significant reduction of dead weight as compared to a conventional bridge system.

That aesthetics is not just idle talk, has been demonstrated on the two major orthotropic bridges now under construction in the United States.

The Poplar Street Bridge in St. Louis, (Fig. 1) was of orthotropic design because the specifications required a deck type structure that would not obstruct the view of the Gateway Arch, and would be in harmony with the redevelopment along the banks of the Mississippi River.

The San Mateo-Hayward Bridge in the San Francisco Bay area was originally conceived as a double deck cantilever structure. That design was challenged in the newspapers on aesthetic grounds and was finally replaced by the orthotropic box girder bridge illustrated in Fig. 2. This new design provides an aesthetically outstanding structure at an estimated additional cost of only 10%.

While aesthetics was not the primary deciding factor in the selection of
MAIN SUPPORTING ELEMENTS

TWO PLATE GIRDERS

MULTIPLE PLATE GIRDERS

SINGLE BOX GIRDERS

MULTIPLE BOX GIRDERS

FIGURE 5

STEEL DECK PLATE

FLOOR BEAM

FIGURE 6
the orthotropic deck arch bridge over the Frazier River in British Columbia, (Fig. 3), you can judge for yourself the aesthetic qualities of the slender floor system.

**DESIGN**

The steel floor in orthotropic design consists of a steel deck plate which is welded to orthogonally-arranged longitudinal stiffening ribs and transverse floor beams. This entire system is connected to the main supporting elements of the bridge system as illustrated in Fig. 4.

The principal characteristic of orthotropic steel decks is the complete structural integration of the deck plate with the stiffening ribs, the floor beams, and the main supporting elements.

There are two basic types of main supporting elements. The orthotropic deck may be supported either by two or more plate girders, or by single or multiple box girders. (Fig. 5)

In suspension and cable-stiffened bridges, which are economical for long spans, the orthotropic deck serves as the top flange of the stiffening girders; and in the case of tied-arch bridges, the orthotropic deck with the girders serves as the tie for the entire bridge system.

Floor beams, spanning between the main girders, normally consist of a web plate and a bottom flange plate.

A portion of the steel deck plate serves as the top flange of the floor beam. The floor beams are normally spaced at 6 to 15 feet on centers (Fig. 6) although the trend is to increase this spacing.

The steel deck plate also acts as the top flange of the stiffening ribs. (Fig. 7)

Two types of longitudinal stiffening ribs have been developed. Originally, they were of the open type. The open ribs are simple plates, shipbuilding profiles, angles, or inverted tees. The spacing between open ribs is normally about 12 inches.

A more recent development is the closed rib. (Fig. 8) The closed stiffening ribs are normally fabricated by bending of plates. Their spacing is normally about 24 inches on center, and the distance between longitudinal welds connecting the ribs of the floor plate is normally 12 inches.

One advantage of a closed rib system is increased floor beam spacing. While a spacing of 15 feet is common for closed ribs, the floor beam spacing seldom exceeds 10 feet for open ribs.
From the fabrication point of view, the reduction of welding in a closed rib system is of importance. First of all, the reduction in the number of floor beams results in a reduction of manual welding between the floor beams and the ribs, a particularly expensive operation. Furthermore, considering the longitudinal welds between the deck plate and the ribs, closed ribs require only half as many welds as the open ribs. (Fig. 9)

It is important to realize, however, that the open ribs are connected by fillet welds while the closed ribs require partial penetration butt welds.

Another advantage of closed ribs is maintenance reduction: the exposed surface of a closed rib system is about 40% less than that of an open rib system. Painting and maintenance of interior surfaces have been shown to be unnecessary by tests carried out on sealed closed sections. The absence of air and moisture prevents corrosion inside the boxes. Thus most of the advantages point toward the selection of the closed rib system.

STEEL DECK PLATES

The next element that we will discuss is the steel deck plate. You will recall that the deck plate serves as top flange of the stiffening ribs and also as top flange of the floor beams. Both the deck and the longitudinal ribs act as the top flange of the entire bridge system. In addition, the plate supports a wearing surface.

WEARING SURFACE

The success of orthotropic bridges will depend to a considerable extent on the successful development of a pavement to be placed on the steel deck. The function of the wearing surface is to protect the deck plate and to provide the road surface.

The requirements of a wearing surface are that it must be lightweight, skid resistant, stable, and durable. It also should be easy to maintain and must permit rapid repairs.

With the assistance of several illustrations, we will summarize the successful solutions for wearing surfaces. The most common of them is the so-called layered system. (Fig. 10)

The first step is to add a prime coat after the top surface of the steel plate has been blast-cleaned. Bituminous paint, lead base paint, zinc metallizing, and epoxy coatings have been used for this purpose. Little or no difficulty has been experienced with the various deck prime coats; the lead base paints and epoxies seem to do the job particularly well.

The isolation layer, placed on the top of the prime coat, has the dual functions of providing a water barrier and also of absorbing thermal move-
STABILIZED MASTICS

MASTIC AND STONE

PRIME COAT

STEEL PLATE AND LUGS

FIGURE 11

SYSTEM I

RIBS

DECK

GIRDER

FIGURE 12
ments between the steel plate and the top surfacing layers. Aluminum foil, copper foil, and asphalt mastics have been used for this purpose. The best of them turned out to be the mastics, which, incidentally, is the only material used nowadays in Europe for this purpose.

The next two layers, the leveling course and the surface course, are made of asphaltic materials. In Europe, they generally use the so-called "Gussasphalt", a hard, dense mixture of bitumen, filler, sand, and crushed stone. Gussasphalt is not used in the United States because of differences in construction techniques. However, an equivalent material can be doubtless designed for placement by highway equipment used in this country.

Excellent results have been obtained in Europe with a stabilized mastic system. (Fig. 11) The steel plate with transverse steel lugs welded to it is blast-cleaned and covered with a prime coat of the same type as that discussed with the layered system.

Then a 7/8" thick layer of pure mastic is applied over the prime coat, and crushed rock 3/4 to 1" in size is rolled into the mastic. The resultant pavement is about 1 - 1/2 to 2" thick.

In Germany, this paving system is considered the best, but has been used infrequently because of high cost.

Getting back to this country, the use of thin, single-layer paving systems such as filled epoxies and cement-latex mixtures, has been limited. Not enough service experience has been accumulated yet to provide reliable evidence about the performance and durability of single layer systems. However because these systems offer the promise not only of low first coat but also for ease of maintenance and repair, several studies are actively being pursued.

DESIGN AND ANALYSIS

Our next topic is the design and analysis of orthotropic bridges. Three separate functional systems have to be considered: the main bridge structure, the stiffened orthotropic plate deck, and the deck plate between the ribs.

System 1

The first system, the main bridge structure, considers the steel deck plate and the stiffening ribs as a part of the main carrying members of the bridge. The entire deck plate is fully active as the top flange of the girders. A reduction in the effectiveness of the deck plate need only be considered for spans less than three times the spacing between main girders.
**FIGURE 13**

**FIGURE 14**

$D_x \approx D_{\text{plate}}$
\[ D_y = \frac{EI_{rib}}{a} \]

**FIGURE 15**

**SYSTEM III**

**FIGURE 16**
Starting from this assumption concerning the effectiveness of the deck as the top flange, the design of System I (Fig. 12) follows conventional bridge design procedures.

System II

The second system, the orthotropic plate deck, consists of the steel deck plate, the longitudinal stiffening ribs, and the transverse floor beams (Fig. 13). The analysis of this rather complex system is concerned with the determination of moments in both the x and y directions. Several methods have been proposed for the solution, but we will concentrate only on the basic assumptions of the method currently favored in this country.

When the orthotropic deck is loaded with a concentrated load such as the wheel of a truck, the deck assumes a dished shape. In other words, the deck responds as a plate. The plate is stiffened in the two perpendicular or orthogonal directions. The stiffening is provided by the longitudinal ribs and transverse floor beams, and thus is different in one direction than in the other: the stiffened plate is orthogonally anisotropic.

Incidentally, the derivation of the term "orthotropic" is a consolidation of these two words, "orthogonal" and anisotropic".

Analysis is based on the theory of anisotropic elastic plates. The first step is to replace the actual deck system with an anisotropic plate which has the same stiffness characteristics in the X and Y directions as the actual structure.

When the deck is subjected to bending moments acting across the longitudinal stiffening ribs, the ribs add very little to the bending resistance of the plate; thus, the stiffness in the X direction is approximately the same as the stiffness of the steel plate. (Fig. 14)

In the Y direction, the ribs are effective in resisting the moments with the steel plate; thus, the stiffness $D_y$ is equal to the modulus of elasticity of steel times the moment of inertia of the rib with its contributory steel plate, and divided by the rib spacing. (Fig. 15)

In addition to the flexural stiffnesses $D_x$ and $D_y$, the deck system has a torsional stiffness $H$. The torsional stiffness of an orthotropic steel deck is a function of the shape, dimensions, and spacing of the ribs, of the flexibility of the deck plate, and of the spacing of the transverse floor beams. The torsional stiffness of a deck with open ribs is negligible, while an approximate formula is available for computing the torsional stiffness of decks with closed ribs.

Thus, for the purpose of analysis we replace the actual orthotropic steel deck with an anisotropic elastic plate having the flexural stiffness $D_x$ in the X
direction, the flexural stiffness \( D_y \) in the Y direction, and the torsional stiffness \( H \). Furthermore, this anisotropic plate is supported by the main girders assumed in the analysis of the deck system as rigid supports. The deck is also supported by the floor beams which are considered in this analysis as flexible.

The deflection of an anisotropic plate subjected to a transverse loading is given by this partial differential equation of the fourth order,

\[
D_x \frac{\partial^4 w}{\partial x^4} + 2H \frac{\partial^4 w}{\partial x^2 \partial y^2} + D \frac{\partial^4 w}{\partial y^4} = p(x, y)
\]

where \( w = \) deflection and \( p = \) load

Once the deflections are known, the moments in the X and Y directions may be computed for any point on the plate.

The differential equation of the plate may be simplified. For a deck with closed ribs and currently-used spacings of floor beams, the flexural stiffness \( D_x \) may be neglected so that the differential equation has only two terms on the left hand side.

For a plate with open stiffeners, both the flexural stiffness \( D_x \) and the torsional stiffness \( H \) may be taken as zero so that the differential equation has only one term on the left hand side.

However, even with these simplified equations the analysis is rather extensive since numerous loading conditions must be considered. This difficulty may be overcome by the use of the Design Manual for Orthotropic Steel Plate Deck Bridges, published by the American Institute of Steel Construction. The Manual includes several charts from which moments in the X and Y directions may be obtained for specific floor systems subjected to AASHO loadings. The stresses are then computed from the moments in the usual manner. And for unusual systems not covered by the Manual, the final analysis may have to be made on a computer.

System III

The third system to be considered in analysis is the deck plate between the stiffening ribs. (Fig. 16) This analysis deals with the stresses caused by the transfer of the load from the deck plate to the adjacent stiffening ribs. An elastic analysis of this system may be found in the AISC Manual. However the analysis of this system is usually disregarded in the design since experimental studies have shown that the strength of the flat steel plate is very much in excess of the effects indicated by this elastic analysis of first
FIGURE 19

POPLAR STREET BRIDGE

FIGURE 20

SAN MATEO-HAYWARD BRIDGE
order. Fatigue tests have also shown that it is safe to disregard the analysis of System III Stresses.

FABRICATION AND ERECTION

Now that we know what an orthotropic bridge is, what its advantages are, and how to design it, we will turn our attention to the problems of fabrication and erection. Examples for this part of the presentation were taken from United States practices developed for the Port Mann Bridge, the Poplar Street Bridge, and the San Mateo-Hayward Bridge.

Incidentally, the steel erection for the fourth major bridge of orthotropic design on this continent (Fig. 17) was completed in June 1964. It will link the site of the World's Fair on St. Helen's Island in the St. Lawrence River with the Montreal waterfront.

The design and fabrication details of this bridge, completed this past summer, are essentially the same as those of the Port Mann Bridge in British Columbia.

The Port Mann Bridge, (Fig. 18) is a tied arch structure. It spans the Frazier River near Vancouver, British Columbia. The deck and the two supporting box girders act as a tie for the arches. The deck was fabricated in sections 65 feet wide and 25 feet long.

The Poplar Street Bridge is located in St. Louis, (Fig. 19) and it spans the Mississippi River with five spans of cintinous box girders. The largest fabricated piece is the center portion of the deck spanning between the box beams 26-1/2 feet wide and 60 feet long. A similar procedure, that is fabrication of the center panel of the deck in one piece, is planned for the San Mateo-Hayward Bridge.

In addition to the channel spans shown in Fig. 20, the bridge will include twelve 292-foot long spans of orthotropic design similar to that shown, and a large number of shorter spans of different designs.

FABRICATION

The first step in fabrication of an orthotropic deck structure is the preparation of the deck plate for welding of the longitudinal ribs. Joints between the individual rolled plates are made by automatic butt welding. This is followed by automatic welding of the longitudinal stiffening ribs (Fig. 21) and by manual welding of the floor beams. Fabrication of the main girders is carried out according to standard methods. The ribs pass through openings in the floor beams, and are welded all the way around them.

Because of the large sizes of the shop-fabricated pieces, it is advantageous to transport them by water. The Poplar Street Bridge is fabricated at the Leetsdale fabricating works of the Bethlehem Steel Corporation.
shop is located on the banks of the Ohio River near Pittsburgh, so that the prefabricated units can be transported by barge down the river to the Mississippi and then up to St. Louis.

**ERECITION**

The erection of the Popular Street Bridge was started at Pier No. 3. (Fig. 22) The first pieces to be erected were the pier sections of the two box girders, parts A on the illustration, Fig. 23. The center portion of the deck, B, followed next, and the two cantilever portions of the deck, C, were erected last.

The deck sections are connected to the box girders with bolted connections in the bottom flanges and webs of the floor beams, (Fig. 24); the field splices in the deck plate were welded.

After completion of the entire bridge section over Pier 3, the box girders are extended out in the direction of Pier 4 with the aid of temporary supports. The bottom flange and the webs of the girders are spliced with high-strength bolts, and the deck is field welded.

After the completion of the sections just described, the erection continues toward Piers 2 and 4, (Fig. 22) utilizing both vertical temporary supports underneath, and overhead temporary cable supports. From Piers 2 and 4 the erection continues again in both directions until both banks of the river are reached.

Work on the substructure began in 1964 and continued through the winter. Erection of the superstructure started in September, 1965.

**CONCLUSION**

All the examples mentioned thus far were of major bridges. Orthotropic design of course, is most economical where long spans are required. However, it is probable that the advantages to be gained by automatic fabrication of large units may eventually lead to the adoption of orthotropic design for short span bridges. Thus, the advantages of simple and rapid maintenance of the riding surface may be extended to the bread-and-butter structures of our highway system.

For instance, a new concept in short span bridge construction has been demonstrated in the erection of an all-steel prefabricated structure over Humphrey's Creek in Bethlehem's Sparrows Point Plant in Maryland. The new bridge, a four-lane highway crossing, (Fig. 25) replaced a wooden structure built in 1920.
FIGURE 25

FIGURE 26

TRANSVERSE BAR

WELD
Designed by Bethlehem Steel Corporation's Research Department, the bridge is a prototype featuring "unitized" construction. Each prefabricated unit is 56 feet long, 8 feet wide, and 3 feet deep, and weighs 11 tons.

The deck is supported on a corrugated plate which performs the function of the longitudinal stiffening ribs. This corrugated plate is welded to the 3/16" thick deck plate. In turn, the deck is attached to unstiffened plate girder webs, 34-1/2" deep, placed at 4-foot centers. The corrugations and the deck plate form the compression flange of the unit.

The transverse floor beams are then inserted through slots in the webs of the girders and welded in place. (Fig. 26) The transverse beam is just a simple steel bar.

The unit is completed by adding cross bracing, and by blast-cleaning, painting, and coating the top surface of the plate with a shop coat of catalyzed epoxy resin mixed with fine sand. The sand provides a rough bonding surface for a subsequent field-applied asphalt topping.

At the bridge site, (Fig. 27) the units are simply placed on the substructure with a crane, and connected together, needing only railing and asphalt topping to ready them for traffic.

Preliminary tests have indicated that the bridge has a large reserve strength beyond the design requirements. Testing of these prototype units is being continued by Bethlehem's Homer Research Laboratories in order to arrive at the most economical design consistent with safety and durability.

Gentlemen, the introduction of orthotropic bridges to the North American continent represents the most important development in the bridge field in this decade. I am certain that the inherent advantages associated with this type of construction will make it one of the favorites of bridge builders in the years to come. We at Bethlehem are ready to help the profession in using this new tool.

Note: Bethlehem Steel Corporation gratefully acknowledges the assistance of Professor Jack G. Bouwkamp of the University of California in the preparation of this talk.