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14

Orthotropic Deck Bridges

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14.1 Introduction

This chapter will discuss the major design issues of orthotropic steel-deck systems. Emphasis will be given to the design of the closed-rib system, which is practicably the only system selected for orthotropic steel deck by the engineers around the world. Examples of short spans to some of the world's long-span bridges utilizing trapezoidal ribs will be presented. The subject of fabrication detailing and fatigue resistant details necessary to prepare a set of contract bridge plans for construction is beyond the scope of this chapter. However, the basic issues of fatigue and detailing are presented. For more detailed discussion, the best references are four comprehensive books on orthotropic steel deck systems by Wolchuk [1], Troitsky [2], and the British Institution of Civil Engineers [3,4].

14.2 Conceptual Decisions

14.2.1 Typical Sections

Modern orthotropic welded steel-deck bridge rib systems were developed by German engineers in the 1950s [1,2]. They created the word *orthotropic* which is from *orthogonal* for *ortho* and *anisotropic* for *tropic*. Therefore, an orthotropic deck has anisotropic structural properties at 90°. Structural steel is used by most engineers although other metals such as aluminum can be used, as well as advanced composite (fiberglass) materials.

The open (torsionally soft) and closed (torsionally stiff) rib-framing system for orthotropic deck bridges developed by the Germans is shown in Figure 14.1. The open-rib and closed-rib systems are the two basic types of ribs that are parallel to the main span of the bridge. These ribs are also used to stiffen other plate components of the bridge. Flat plates, angles, split Ts, or half beams are types of open ribs that are always welded to the deck plate at only one location. A bent or rolled piece of steel plate is welded to the deck plate to form a closed space. The common steel angle can either be used as an open or closed rib depending on how it is welded to the steel deck plate. If the angle is welded at only one leg, then it is an open rib. However, if the angle is rotated to 45° and both legs are welded to the deck plate forming a triangular space or rib, it is a closed rib. Engineers have experimented with a variety of concepts to shape, roll, or bend a flat plate of steel into the optimum closed rib. The trapezoidal rib has been found to be the most practicable by engineers and the worldwide steel industry. Recently, the Japanese built the record span suspension bridge plus the record span cable-stayed bridge with trapezoidal rib construction (see Chapter 65).

The ribs are normally connected by welding to transverse floor beams, which can be a steel hot-rolled shape, small plate girder, box girder, or full-depth diaphragm plate. In Figure 14.1 small welded plate girders are used as the transverse floor beams. The deck plate is welded to the web(s) of the transverse floor beam. When full-depth diaphragms are used, access openings are needed for bridge maintenance purposes. The holes also reduce dead weight and provide a passageway for mechanical or electrical utilities. Since the deck plate is welded to every component, the deck plate is the top flange for the ribs, the transverse floor beam, and the longitudinal plate girders or box girders. All these various choices for the ribs, floor beam, and main girders can be interchanged, resulting in a great variety of orthotropic deck bridge superstructures.

14.2.2 Open Ribs vs. Closed Ribs

A closed rib is torsionally stiff and is essentially a miniature box girder [6]. The closed-rib deck is more effective for lateral distribution of the individual wheel load than the open-rib system. An open rib has essentially no torsional capacity. The open-rib types were initially very popular in the precomputer period because of simpler analysis and details. Once the engineer, fabricator, and contractor became familiar with the flat plate rib system shown in Figure 14.1 and Table 14.1, the switch to closed ribs occurred to reduce the dead weight of the superstructure, plus 50% less rib surface area to protect from corrosion. Engineers discovered these advantages as more orthotropic decks were built. The shortage and expense of steel after the World War II forced the adoption of closed ribs in Europe. The structural detailing of bolted splices for closed ribs requires handholds located in the bottom flange of the trapezoidal ribs to allow workers access to install the nut to the bolt. For a more-detailed discussion on handhold geometry and case histories for solutions to field-bolted splicing, refer to the four comprehensive books [1-4].

Compression stress occurs over support piers when the rib is used as a longitudinal interior stiffener for the bottom flanges of continuous box girders and can be graphically explained [6]. Ribs are usually placed only on the inside face of the box to achieve superior aesthetics and to minimize exterior corrosion surface area that must also be painted or protected. Compression also occurs when the rib is used as a longitudinal interior stiffener for columns, tower struts, and other components. The trapezoidal rib system quite often is field-welded completely around the superstructure cross section to achieve full structural continuity, rather than field bolted.

Table 14.2 [5] shows the greater bending efficiency in load-carrying capacity and stiffness achieved by the trapezoidal (closed) rib. It is readily apparent that a series of miniature box girders placed side by side is much more efficient that a series of miniature T-girders placed side by side. In the tension zones, the shape of the rib can be open or closed depending on the designers' preferences.

A trapezoidal rib can be quickly bent from a piece of steel as shown in Figure 14.2. A brake press is used to bend the shape in a jig in a few minutes. Rollers can also be used to form these trapezoidal ribs.

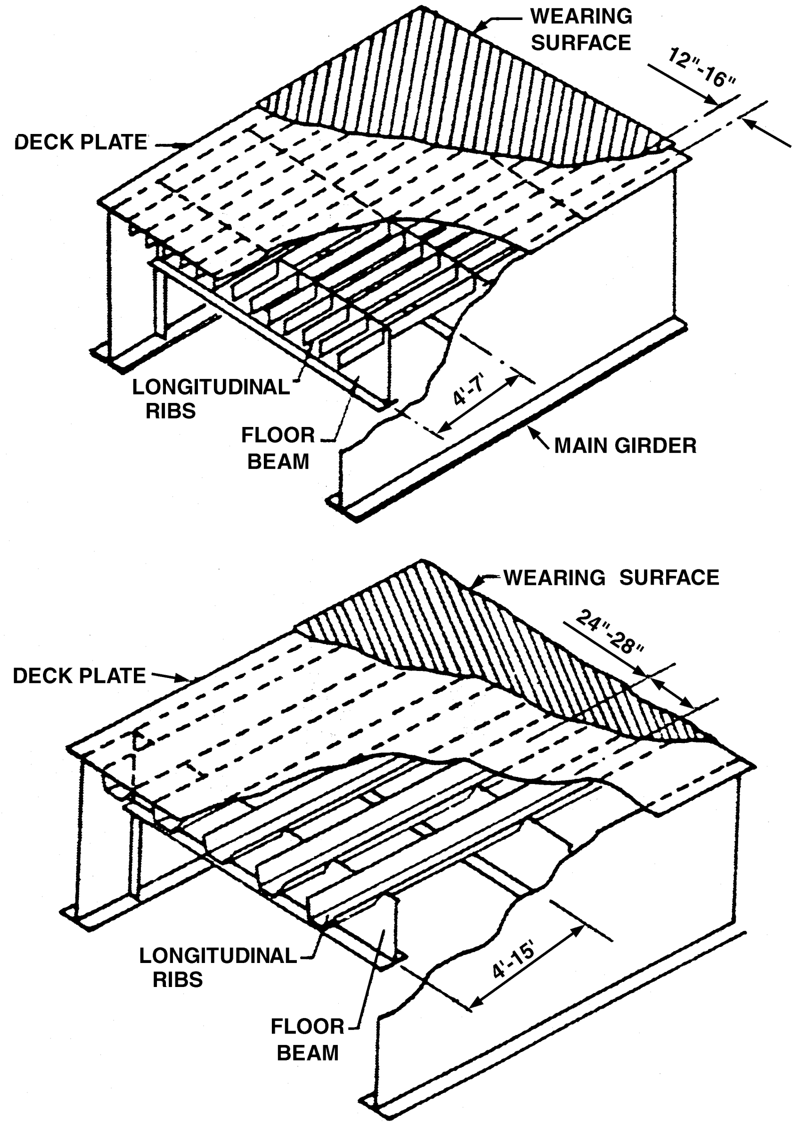
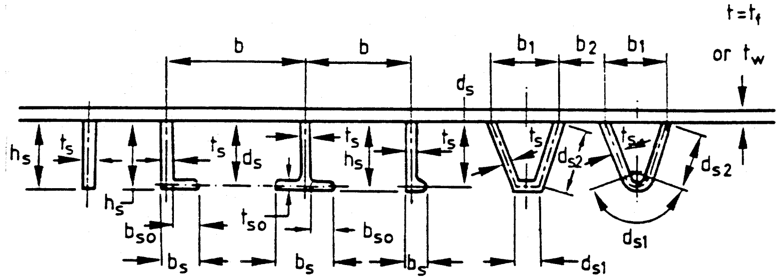


FIGURE 14.1 Typical components of orthotropic deck bridges. (From Troitsky, M. S., *Orthotropic Steel Deck Bridges*, 2nd ed., JFL Arch Welding Foundation, Cleveland, OH, 1987. Courtesy of The James F. Lincoln Arc Welding Foundation.)

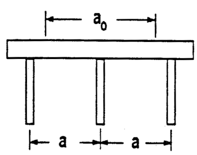
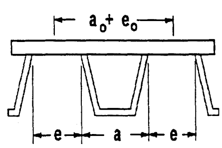
One American steel company developed Table 14.3 to encourage the utilization of orthotropic deck construction. This design aid was developed using main-frame computers in 1970, but due to lack of interest in orthotropic deck by bridge engineers this design aid eventually went out of print; nor was it updated to reflect changes in the AASHTO Bridge Code [5]. Tables 14.4 and 14.5 are excerpts from this booklet intended to assist an engineer quickly to design an orthotropic deck system and comply with minimum deck plate thickness; maximum rib span; and rib-spacing requirements of AASHTO [5]. AASHTO standardization of ribs has yet to occur, but many bridges built in the United States using ribs from Table 14.3 are identified throughout this chapter. The German and Japanese steel companies have developed standard ribs (see Table 14.3)

TABLE 14.1 Limiting Slenderness for Various Types of Ribs

Flanges and web Stiffeners	 d, h = stiffener depth b_s = width of angle t_o, t', t_s = stiffener thickness t = plate thickness w, b = spacing of stiffeners l_s = span of stiffener between supporting members r_y = radius of gyration of stiffener (without plate) about axis normal to plate F_y = yield stress of plate, N/mm^2 F_{ys} = yield stress of stiffener, N/mm^2 F_{max} = maximum factored compression stress, N/mm^2
Draft U.S. rules	Effective slenderness coefficient C_s shall meet requirement $$C_s = \begin{cases} \dfrac{d}{15t_o}+\dfrac{w}{12t} & \text{for flats} \\ \dfrac{d}{1.35t_o+0.56r_y}+\dfrac{w}{12t} & \text{for Ts or angles} \end{cases} \le \begin{cases} \dfrac{0.4}{\sqrt{F_y/E}} & \text{for } f_{max} \ge 0.5F_y \\ \dfrac{0.65}{\sqrt{F_y/E}} & \text{for } f_{max} \le 0.5F_y \end{cases}$$ For any outstand of a stiffener $\dfrac{b'}{t'} \le \dfrac{0.48}{\sqrt{F_{ys}/E}}$
British Standard 5400	For flats: $\dfrac{h_s}{t_s}\sqrt{\dfrac{F_{ys}}{355}} \le 10$ For angles: $b_s \le h_s$; $\dfrac{b_s}{t_s}\sqrt{\dfrac{F_{ys}}{355}} \le 11$; $\dfrac{h_s}{t_s}\sqrt{\dfrac{F_{ys}}{355}} \le 7$

Source: Galambos, T. V., Ed., *Guide to Stability Design Criteria for Metal Structures,* 4th ed., John Wiley & Sons, New York, 1988. With permission.

TABLE 14.2 AASHTO Effective Width of Deck Plate Acting with Rib

Calculation of	 	
Rib section properties for calculation of deck rigidity and flexural effects due to dead loads	$a_o = a$	$a_o + e_o = a + e$
Rib section properties for calculation of flexural effects due to wheel loads	$a_o = 1.1a$	$a_o + e_o = 1.3(a + e)$

Source: American Association of State Highway and Transportation Officials, *LRFD Bridge Design Specifications,* Washington, D.C., 1994. With permission.

FIGURE 14.2 Press brake forming rib stiffener sections. (Photo by Lawrence Lowe and courtesy of Universal Structural, Inc.)

14.2.3 Economics

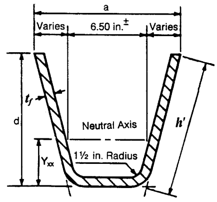
Orthotropic deck bridges become an economical alternative when the following issues are important: lower mass, ductility, thinner or shallower sections, rapid bridge installation, and cold-weather construction.

Lower superstructure mass is the primary reason for the use of orthotropic decks in long-span bridges. Table 14.6 shows the mass achieved by abandoning the existing reinforced concrete deck and switching to a replacement orthotropic deck system relationship. The mass was reduced from 18 to 25% for long-span bridges, such as suspension bridges. This is extremely important since dead load causes 60 to 70% of the stresses in the cables and towers [7,8]. The mass is also important for bridge responses during an earthquake. The greater the mass, the greater the seismic forces. The Golden Gate Bridge, San Francisco, California, was retrofitted from a reinforced concrete deck built in 1937 to an orthotropic deck built in 1985 (see Figure 14.3). This retrofit reduced seismic forces in the suspension bridge towers and other bridge components. The engineering statistics of redecking are shown in Table 14.6. The Lions Gate Bridge of Vancouver, Canada was retrofitted in 1975 from a reinforced concrete deck to an orthotropic deck, which increased its seismic durability. Economics or cost of materials can be multiplied against the material saved to calculate money saved by reducing the weight.

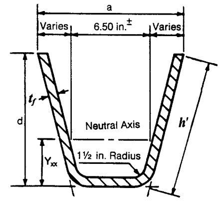
A very thin deck structure can be built using this structural system, as shown by the Creitz Road Grade Separation in Figure 14.4 or German Railroad Bridge in Figure 14.5. An orthotropic deck may be the most expensive deck system per square meter in a short-span bridge. So why would the most expensive deck be a standard for the German railroads? The key component in obtaining the thinnest superstructure is the deck thickness. An orthotropic deck is thin because the ribs nest between the floor beams. Concrete decks are poured on top of steel beams. Thin superstructures can be very important for a grade-crossing situation because of the savings to a total project. The two components are bridge costs plus roadway or site costs. High-speed trains require minimal grade changes. Therefore, the money spent on highway or railway approach backfill can far exceed the cost of a small-span bridge. A more expensive superstructure will greatly reduce the backfill work and cost. In urban situations, approach fills may not be possible. The local street may need to be excavated below the railway bridge; therefore a more expensive thin orthotropic deck–floor system may result in the lowest total cost for the entire project.

TABLE 14.3 Properties of Trapezoidal Ribs

American Rib (English Units)



Japanese Rib (Metric)



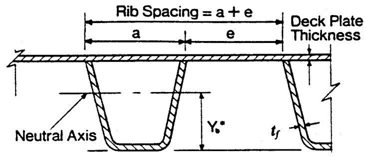
Depth of Rib *d* (in.)	Width at Top, *a* (in.)	Rib Wall Thickness, t_f (in.)	Weight per Foot, *w* (lb)	Moment of Inertia, I_{xx} (in^4)	Neutral Axis Location, Y_{xx} (in.)	Sloping Face Length, *h′* (in.)
8.0	11.50	5/16	23.43	46.3	3.09	8.382
		3/8	27.95	54.6	3.12	
		7/16	32.40	62.7	3.14	
9.0	12.12	5/16	25.64	63.8	3.56	9.428
		3/8	30.60	75.5	3.59	
		7/16	35.53	86.8	3.61	
10.0	12.75	5/16	27.88	85.1	4.04	10.477
		3/8	33.29	100.8	4.06	
		7/16	38.66	116.1	4.09	
11.0	13.38	5/16	30.09	110.4	4.52	11.525
		3/8	35.94	131.0	4.54	
		7/16	41.57	151.0	4.57	
12.0	14.00	5/16	32.33	140.2	5.00	12.572
		3/8	38.62	166.4	5.02	
		7/16	44.88	192.1	5.05	
13.0	14.63	5/16	34.53	174.7	5.48	13.621
		3/8	41.31	207.6	5.51	
		7/16	48.01	239.7	5.53	
14.0	15.25	5/16	36.75	214.4	5.97	14.668
		3/8	43.96	254.8	5.99	
		7/16	51.10	294.4	6.02	
Depth of Rib *d* (mm)	**Width at Top, *a* (mm)**	**Rib Wall Thickness, t_f (mm)**	**Weight per Foot, *w* (Kg/m)**	**Moment of Inertia, I_{xx} (cm^4)**	**Neutral Axis Location, Y_{xx} (mm)**	**Sloping Face Length, *h′* (mm)**
240	320	6	31.6	2460	88.6	246
260	320	6	33.1	3011	99.1	266
242	324	8	42.3	3315	89.9	248
262	324	8	44.3	4055	100.3	268

14.3 Applications

Some of the most notable world bridges were built using an orthotropic steel deck with trapezoidal rib construction. There are about only 50 bridges in North America using orthotropic decks, and eight are built and two more being designed in California. However, there is a vast array of bridge types utilizing the orthotropic deck from very small to some of the longest clear-span bridges of the world. Some orthotropic deck bridges have unique framing systems. Bridges featured and discussed in the following sections were selected to demonstrate the breath of reasons for selecting orthotropic deck superstructure [11-15]. All of these bridges utilize trapezoidal ribs in the deck area or compression zone of the superstructure. These types are: simple span with two plate or box girders, multiple plate girder, single-cell box girder, multicell box girder, wide bridges that have cantilever floor beams supported by struts, a monoarch bridge, a dual-arch bridge, a through-truss

TABLE 14.4 Orthotropic Deck Design Properties — Rigid Floor Beams

H = effective torsional rigidity of orthotropic plate (kip-in.^{2}/in.)
D_y = flexural rigidity of orthotropic plate in y direction (kip-in.^{2}/in.)
H/D_y = rigidity ratio (unitless)
I_r = moment of inertia (in.^{4})
Y_b = centroid (in.)
t_p = deck plate thickness (in.)



Deck Plate t_p (in.)	$a + e$ (in.)	Rib Wall (in.)	Value	Span (ft)	Rib Depth 8 in.	9 in.	10 in.	Span (ft)	Rib Depth 11 in.	12 in.	13 in.	14 in.
9/16	22	5/16	H/D_y	7	0.039	0.034	0.030	10	0.048	0.045	0.042	0.040
			I_r		165	217	278		351	431	520	620
			Y_b		6.45	7.14	7.81		8.54	9.20	9.85	10.49
9/16	26	3/8	H/D_y	11	0.057	0.049	0.043	14	0.056	0.051	0.047	0.044
			I_r		197	259	331		417	512	620	740
			Y_b		6.48	7.18	7.86		8.56	9.23	9.88	10.53
9/16	30	7/16	H/D_y	15	0.066	0.056	0.049	18	0.057	0.051	0.047	0.043
			I_r		226	298	382		480	591	716	855
			Y_b		6.50	7.19	7.88		8.57	9.24	9.89	10.54
5/8	22	5/16	H/D_y	7	0.044	0.038	0.033	10	0.053	0.049	0.046	0.043
			I_r		171	225	288		364	446	539	643
			Y_b		6.59	7.30	7.99		8.73	9.41	10.07	10.72
5/8	30	7/16	H/D_y	15	0.079	0.067	0.058	18	0.067	0.061	0.055	0.051
			I_r		234	309	396		498	612	742	886
			Y_b		6.64	7.35	8.05		8.76	9.44	10.11	10.77
11/16	22	5/16	H/D_y	7	0.048	0.041	0.036	10	0.056	0.052	0.048	0.045
			I_r		177	232	297		375	460	557	664
			Y_b		6.72	7.44	8.14		8.90	9.59	10.27	10.93
11/16	30	7/16	H/D_y	15	0.090	0.078	0.068	18	0.077	0.070	0.064	0.059
			I_r		242	318	408		513	632	765	915
			Y_b		6.76	7.49	8.21		8.93	9.62	10.31	10.98
3/4	22	5/16	H/D_y	7	0.052	0.044	0.038	10	0.059	0.054	0.050	0.047
			I_r		182	239	305		386	474	573	683
			Y_b		6.84	7.57	8.29		9.06	9.76	10.45	11.13
3/4	26	3/8	H/D_y	11	0.084	0.073	0.064	14	0.079	0.072	0.067	0.063
			I_r		216	284	364		458	563	682	815
			Y_b		6.87	7.61	8.34		9.08	9.78	10.48	11.16
3/4	30	7/16	H/D_y	15	101	0.087	0.077	18	0.086	0.078	0.072	0.067
			I_r		248	327	420		528	650	787	941
			Y_b		6.88	7.62	8.35		9.08	9.79	10.49	11.17

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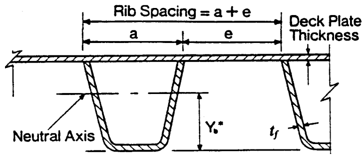
bridge, a deck-truss bridge, a monoplane cable-stayed bridge, a dual-plane cable-stayed bridge, a monocable suspension bridge, and a dual-cable suspension bridge.

14.3.1 Plate-Girder Bridges

In the 1960s small orthotropic steel-deck bridges were built in California, Michigan, and for the Poplar Street Bridge as prototypes to examine steel construction systems as well as various wearing surface materials. Each bridge used trapezoidal ribs with a split-beam section as floor beam and two plate girders as the main girders. The California Department of Transportation (Caltrans) built the I-680 over U.S. 580 bridge as their test structure [9,13] in 1968. This bridge has two totally different rib/deck systems including two different wearing surfaces. The two-lane cross section of the bridge and a very similar one were built by the Michigan Department of Transportation. The Creitz Road Bridge is a typical grade crossing built over I-496 near Lansing, Michigan (see

TABLE 14.5 Orthotropic Deck Design Properties — Flexible Floor Beams

Wt. = weight (PSF)
I_r = moment of inertia (in^4)
Y_b = centroid (in.)
t_p = deck plate thickness (in.)



Deck Plate t_p (in.)	$a + e$ (in.)	Rib Wall (in.)	Value	Rib Depth 8 in.	9 in.	10 in.	11 in.	12 in.	13 in.	14 in.
9/16	22	5/16	Wt.	35.7	36.9	38.1	39.4	40.6	41.8	43.0
			I_r	169	222	284	355	437	528	630
			Y_b	6.54	7.24	7.94	8.62	9.29	9.95	10.60
9/16	26	3/8	Wt.	35.8	37.1	38.3	39.5	40.8	42.0	43.2
			I_r	199	262	335	420	517	625	747
			Y_b	6.54	7.24	7.93	8.61	9.28	9.94	10.59
9/16	30	7/16	Wt.	35.9	37.2	38.4	39.7	40.9	42.1	43.4
			I_r	228	301	386	484	595	721	861
			Y_b	6.54	7.24	7.93	8.61	9.28	9.93	10.58
5/8	22	5/16	Wt.	38.3	39.5	0.033	41.9	43.1	44.3	45.6
			I_r	175	230	288	368	452	547	653
			Y_b	6.68	7.40	8.11	8.80	9.49	10.17	10.83
5/8	30	7/16	Wt.	38.5	39.7	41.0	42.2	43.5	44.7	45.9
			I_r	236	311	399	501	616	745	892
			Y_b	6.68	7.40	8.10	8.80	9.48	10.15	10.82
11/16	22	5/16	Wt.	40.8	42.0	43.2	44.5	45.7	46.9	48.1
			I_r	180	237	303	379	466	564	674
			Y_b	6.81	7.54	8.26	8.97	9.67	10.36	11.04
11/16	30	7/16	Wt.	41.0	42.3	43.5	44.8	46.0	47.2	48.5
			I_r	244	321	412	516	636	770	920
			Y_b	6.81	7.54	8.26	8.96	9.66	10.35	11.03
3/4	22	5/16	Wt.	43.4	44.6	45.8	47.0	48.2	49.4	50.7
			I_r	185	243	311	390	479	580	693
			Y_b	6.92	7.67	8.40	9.13	9.84	10.54	11.24
3/4	26	3/8	Wt.	43.5	44.7	46.0	47.2	48.4	49.7	50.9
			I_r	218	287	368	461	567	687	821
			Y_b	6.92	7.66	8.40	9.12	9.83	10.53	11.22
3/4	30	7/16	Wt.	43.6	44.8	46.1	47.3	48.6	49.8	51.0
			I_r	250	330	423	531	653	792	947
			Y_b	6.92	7.67	8.40	9.12	9.83	10.53	11.22

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Figure 14.4). It is a typical two-lane bridge that carries local traffic over an interstate freeway, with two symmetrical spans of 29 m. The bridge uses the 5/16-in.-thick, 9-in.-deep, 25.64-plf rib as shown in Table 14.3. The rigid steel bent comprises three welded steel box members aesthetically shaped [14,15]. Caltrans built a weigh station as an orthotropic deck prototype for the Hayward–San Mateo Bridge [17]. All of these short-span orthotropic deck bridges are still in use after 30 years of service, but the wearing surface has been replaced on many of these bridges.

Single-track railroad bridges through steel plate are the most common type for short spans. A two-girder bridge shown in Figure 14.5 is AASHTO fracture critical because, if one girder fractures, the bridge will collapse [18]. Both versions of the current AASHTO codes require the designer to label fracture-critical components, which have more stringent fabrication requirements. Orthotropic bridges can be erected quickly when the entire superstructure is fabricated as a full-width component. Many railroads prefer weathering steel since maintenance painting is not required. The German Federal Railroads have a standard, classic two plate girder with orthotropic deck system

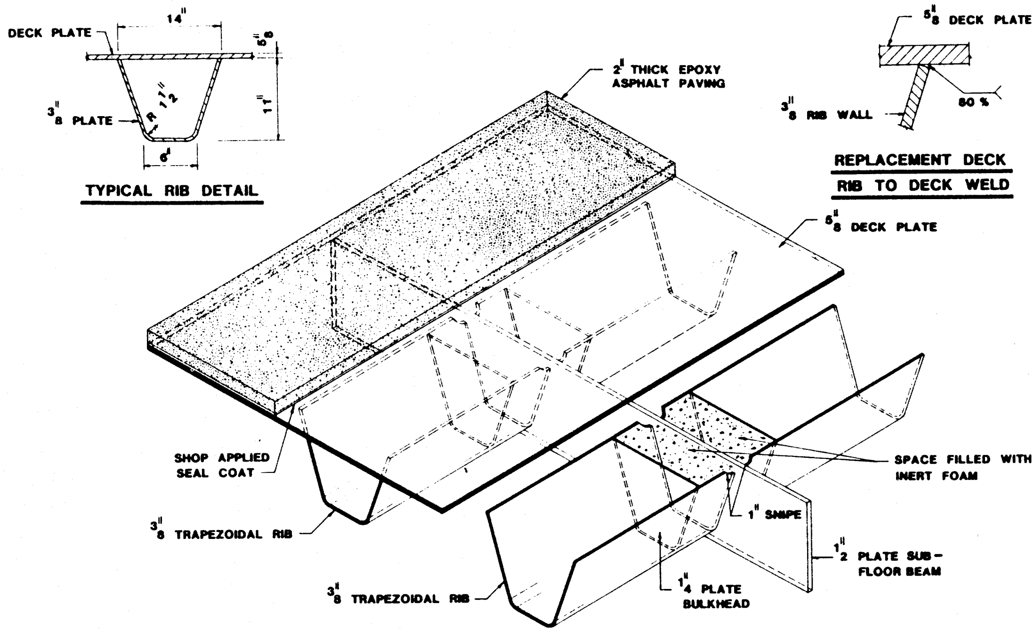


FIGURE 14.3 Golden Gate orthotropic deck details. (From Troitsky, M. S., *Orthotropic Steel Deck Bridges*, 2nd ed., JFL Arc Welding Foundation, Cleveland, OH, 1987. Courtesy of the James F. Lincoln Arc Welding Foundation.)

for their common short-span railroad bridges. Edge plates are used to keep the gravel ballast in place on top of the superstructure (Figure 14.5).

14.3.2 Box-Girder Bridges

Box-girder bridges can be subdivided into three basic categories: the single-cell box, the multicell box, and the box with struts supporting a cantilevered deck. In Figure 14.6, the typical cross section of the Valdez Floating City Dock Transfer Bridge is shown. The entire bridge, only about 3 to 6 m above the waterline, was completed in 1981 using the orthotropic deck with trapezoidal ribs. The two identical bridges were built at each end of the floating dock. Each bridge has only two box girders and has a simple span of 61 m. The transfer bridge provides traffic access to and from the floating dock and serves as the primary mooring tie for dock forces perpendicular to the shoreline. Trapezoidal rubber marine fenders absorb kinetic energy as the floating dock moves with the waves. These bumpers are at each end of the bridge. Box girders are more efficient in transmitting compression forces than plate girders [19]. ASTM A-36 steel was used to meet charpy impact requirements of 15 foot-pounds at –15°F. Automatic flux cored welding was used, and all full-penetration welds were either radiographically or ultrasonically inspected. The bridge uses the ⅜-in.-thick, 12-in.-deep, 38.62-plf rib as shown in Table 14.3. The floor beams are 2 ft deep by 1 ft wide ⅜-in.-thick plate bent in a U shape pattern. The ribs pass through the floor beam.

The typical cross sections of the Yukon River or "E. L. Patton" Bridge are shown in Figure 14.7. The 671-m-long bridge, with spans of 128 m, crosses over the Yukon River and was completed in 1976. The haul road is a gravel road originally built to transport supplies for the pipeline and oil field facilities at Prudhoe Bay, Alaska. The bridge was field-bolted together in cold weather, since the Alaskan winter lasts 6 months. It was important to keep the construction on schedule since the bridge was built to carry the 1.46-m-diameter trans-Alaskan crude oil pipeline. The bridge is the first built in Alaska across the Yukon River [2,20]. It is still the only bridge in Alaska across the

TABLE 14.6 Orthotropic Redecking Statistics Table (weight, deck area, etc.) Metric

Bridge	Lions Gate, Vancouver, BC, Canada	George Washington, New York, NY, USA	Golden Gate, San Francisco, CA, USA	Throngs Neck Viaduct, New York, NY, USA	Ben Franklin, Philadelphia, PA, USA	Champlain, Montreal, PQ, Canada
Bridge type	Girder	Suspension	Suspension + Approaches	Girder	Suspension + Approaches	Trusses
Redecking	Finished 1975	Finished 1978	Finished 1985	Finished 1986	Finished 1987	Finished 1992
Main Spans	13	186	343	42	219	118
	to	1,067	1,280	to	534	215
	38	186	343	58	219	118
Redecked area (m^2)	8600	40,320	52,680	45,800	55,740	18,620
Rib type	Closed	Open (Ts)	Closed	Closed	Open (bulb section)	Closed
Rib spans (m)	4.12	1.60	7.62	6.10 to 8.50	5.80 to 6.70	6.40 to 9.80
Wearing surface	40 mm epoxy asphalt	40 mm bitum. asphalt	50 mm epoxy asphalt	40 mm bitum. asphalt	32 mm epoxy asphalt + 32 mm bitum. asphalt	50 mm epoxy asphalt
Original concrete deck weight	N/A	517 (kg/m^2)	508 (kg/m^2)	522 (kg/m^2)	601 (kg/m_2)	N/A
Total weight of new deck[a]	300 (kg/m^2)	293 (kg/m^2)	386 (kg/m^2)	406 (kg/m^2)	435 (kg/m^2)	402 (kg/m_2)
Weight savings	N/A	224 (kg/m^2)	122 (kg/m^2)	116 (kg/m^2)	166 (kg/m_2)	N/A
New deck + main members	Yes, integral	No, integral	No, integral	No, integral	Yes, integral	Yes, integral
Cost/m^2[b]	U.S. $500	U.S. $460	U.S. $1070	U.S. $770	U.S. $1010	U.S. $402
Redeck	Nighttime	Nighttime	Nighttime	Nighttime	Daytime	Nighttime

[a] Including surfacing, parapets, shear connectors;
[b] Total bid price/deck area. Note that bid prices reflect such variable factors as specific project characteristics, contractors profit margins, etc.
Source: Wolchuk, R., *Structural Engineering International*, IABSE, Zurich, 2(2), 125, 1992. With permission.

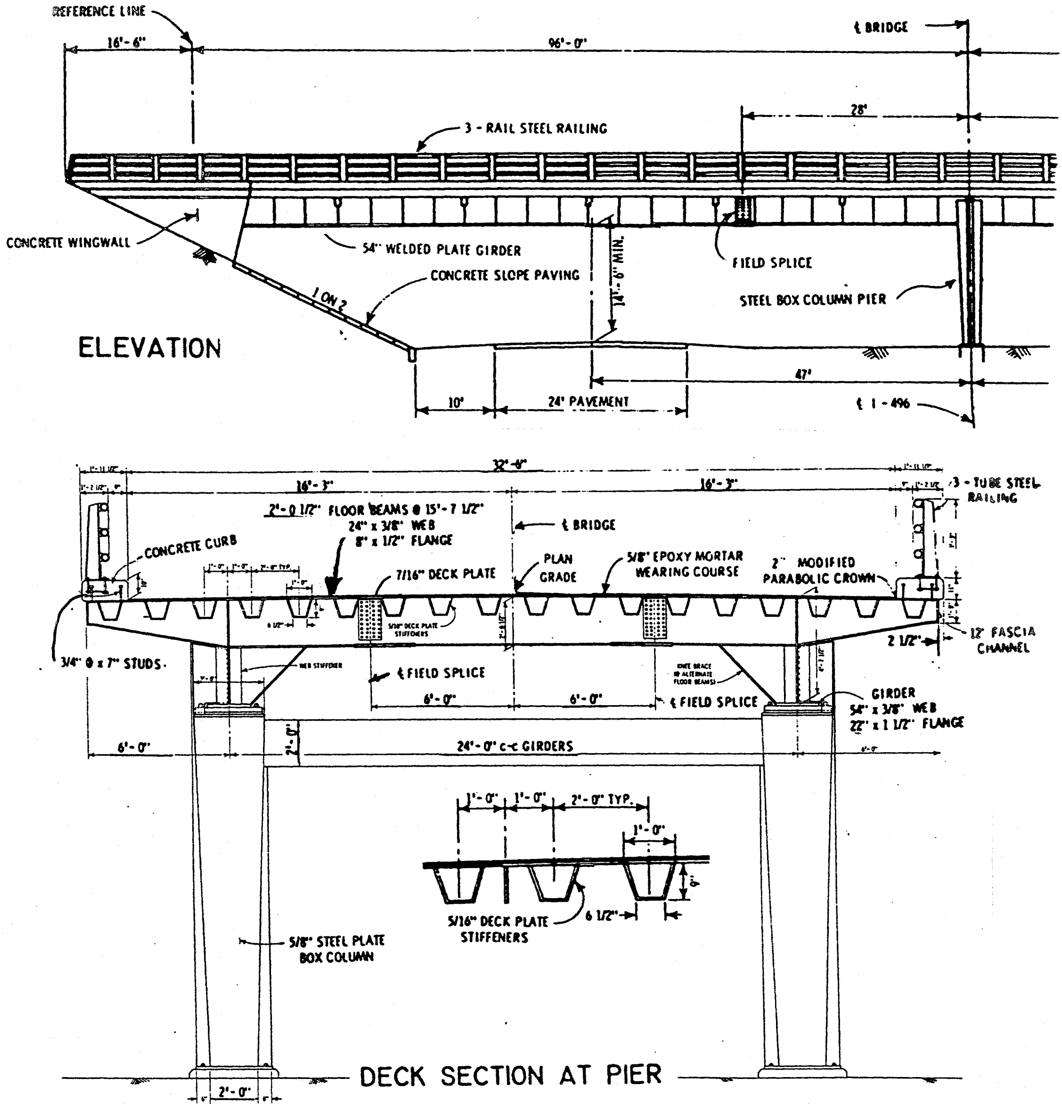


FIGURE 14.4 Typical grade separation — Creitz Road, Lansing, Michigan. (From *Modern Welded Steel Structure,* III, JFL Arc Welding Foundation, Cleveland, OH, 1970, B-10. Courtesy of the James F. Lincoln Arc Welding Foundation.)

Yukon, which has river ice 2 m thick. The superstructure consists of constant-depth twin rectangular box girders which have unique cantilevering brackets that support the trans-Alaskan crude oil pipeline on one side. The future trans-Alaska natural gas line, yet to be built, can be supported on the opposite side of the bridge with these specially designed cantilever support brackets. The bridge, which was fabricated in Japan, uses ribs as shown in Table 14.3. The ⅜-in.-thick, 11-in.-deep, 35.94-plf rib for the deck and the 5⁄16-in.-thick, 8-in.-deep, 23.43-plf rib in other locations are shown in Figure 14.7. Concrete deck construction requires curing temperature of a minimum of 40°F, otherwise, the water freezes during hydration. This was another factor in the selection of the orthotropic deck system, which was erected in temperatures as cold as –60°F [20]. The main bridge components, such as tower columns, tower cross frames, box girders and orthotropic steel deck were stiffened by trapezoidal ribs. The goal of maximizing the number of locations of trapezoidal rib was to reduce

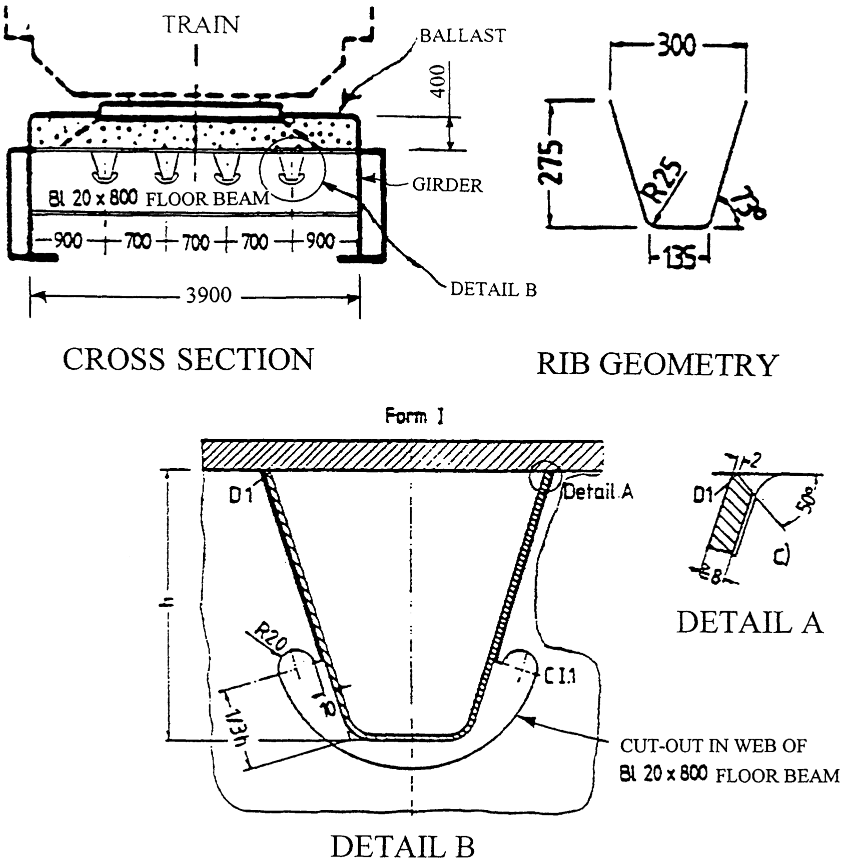


FIGURE 14.5 German railroad bridge. (From Haibach E. and Plasil, I., Der Stahlbau, 269, Ernst & Sohn, Berlin, Germany, 1983 [in German]. With permission.) (Metric units).

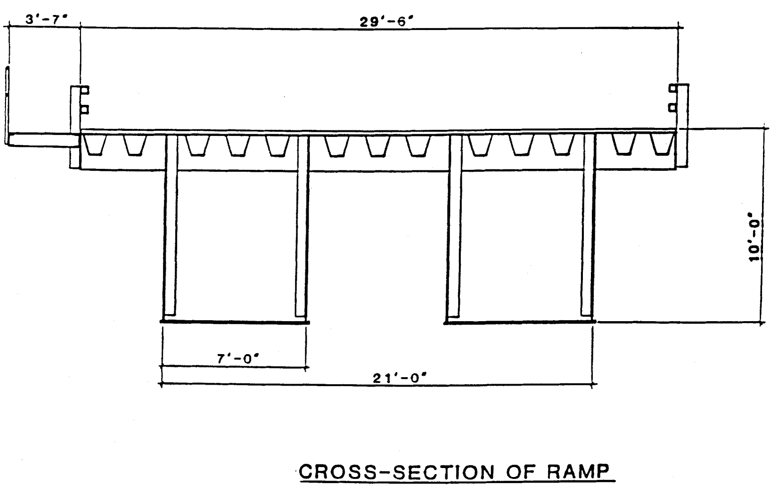


FIGURE 14.6 Valdez Floating City Dock Transfer Bridge. (Courtesy of Berger/ABAM Engineers, Federal Way, WA.) (English units)

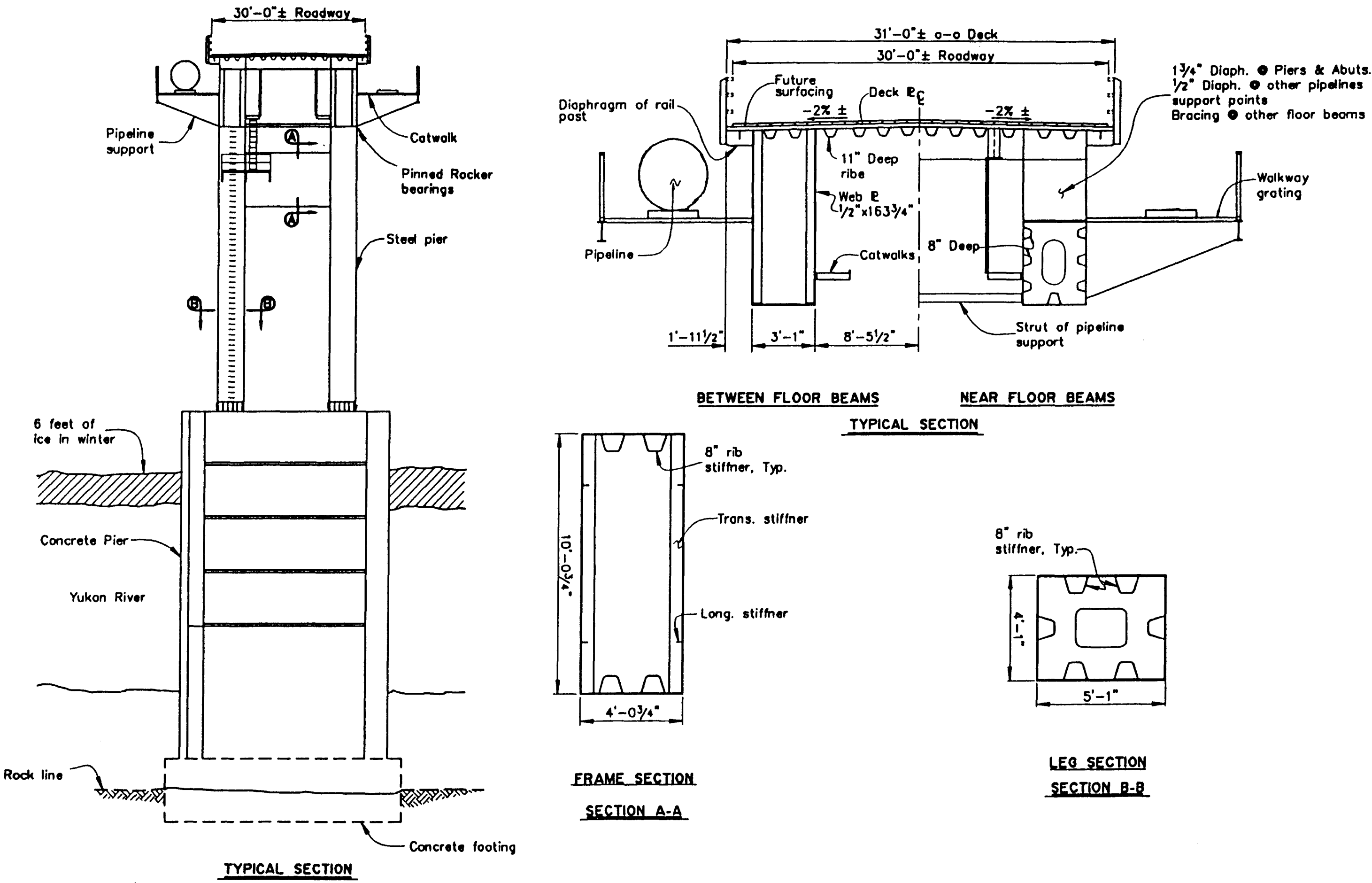


FIGURE 14.7 Yukon River Bridge — orthotropic deck and columns. (Courtesy of Alaska Department of Transportation and Public Facilities.)

the fabricator's setup costs to make a rib. The bridge tower columns and cross frames were built of shop-welded steel and field-bolted splices. The wearing surface of treated timber boards was bolted to the steel bridge because this could be built during cold weather, plus the haul road remains unpaved. The bridge utilized cold-weather steel (ASTM A537 and A514) with high charpy impact test characteristics for subzero temperatures [2, 20].

Shown in Figure 14.8a is the San Diego–Coronado Bridge, which was completed in June 1969. This California toll bridge sweeps around the harbor area of San Diego [3]. The Caltrans engineers selected single-cell box-girder orthotropic steel deck (continuous length of orthotropic portion = 573 m) because a constant-depth box could be used for the 201, -201, 171 m main spans over the shipping channel. Steel plate girders with concrete deck were used on the remaining length of 1690 m. The bridge was erected in these large pieces with a barge crane. The sections were field-bolted together. The bridge is painted on the inside and outside to resist corrosion and carries six lanes of traffic [21–23].

The Queens Way Bridges, identical three span twin bridges, were completed in June 1970 and are near the tourist attraction of the decommissioned Queen Mary ocean liner [25]. Each orthotropic bridge has a main drop-in span of 88 m suspended with steel hanger bars from cantilever side spans of 32 m. Thus, the center to center of the concrete piers or clear span is 152 m, with two side spans of 107 m. Each superstructure cross section is a single-cell box with deck overhangs with components similar to the San Diego–Coronado. The bridge was fabricated in 14 pieces, and the superstructure was erected in 11 days. The 88-m suspended or drop-in span was fabricated in one piece weighing 618 U.S. tons in Richmond, California, floated 700 miles south to Long Beach, and lifted up 15.2 m. by the same barge crane [26].

Shown in Figure 14.8b is the Maritime Off-Ramp a curved "horseshoe"-shaped bridge crossing over I-80 in Oakland, California, which was completed in 1997 as part of the I-880 Replacement Project. This superstructure has a very high radius of 76 m and a very shallow web depth of only 2.13 m for 58 m spans. The bridge has two lanes of traffic that create large centrifugal forces. The box-girder superstructure is divided into three separate cells to resist the very high torsional forces. To reduce the fabrication costs, the trapezoidal ribs were used in the top and bottom box-girder flanges since this was a continuous structure. The bridge sections were erected over busy I-80 on two Saturday nights creating an instant superstructure. The bridge was fabricated in 13 segments weighing as much as 350 U.S. tons and erected with two special hydraulic jacks supported by special multiwheeled trailers [27-29]. The orthotropic superstructure has a wider top deck plate with a 16-mm thickness and narrower flange plate of 19-mm thickness. In addition, each of three cells has four ribs for the top deck and two in the bottom flange. There are two exterior inclined webs and two interior vertical webs.

When a bridge gets very wide in relationship to the depth of the box girder superstructure, the German solutions shown in Figure 14.9 become the most economical. Shown in Figure 14.9a is the Jagst Viaduct, Widdern in the German Interstate system or Heilbronn–Wurzburg Autobaun carrying eight lanes of traffic over a deep valley. The superstructure is 30 m wide and 5.25 m deep. The most economical solution was to brace the ends of the cantilevering floor beams with struts attached to the bottom flange of the box girder. The box system remains constant depth so the strut remains a constant length. This keeps fabrication costs lower since the struts are all identical components [30].

Shown in Figure 14.9b is the Moselle Viaduct, Winningen, which is near Koblenz, Germany. The engineers decided to utilize a bottom soffit or flange following a parabolic curve. The superstructure is 30.5 m wide and 6 m deep at midspan and 8.5 m deep at the concrete piers. Therefore, to keep the struts a constant length, additional interior framing or cross-bracing members was devised. The struts are bolted to the side of the superstructure at a constant depth. At the inside face of the box girder, web cross-bracing members were attached and aligned with the exterior struts. This also produces a more pleasant architectural appearance [30]. The top deck utilizes "martini-glass"-shaped ribs, which consist of two standard shapes welded together. First a V-shaped rib is welded to the deck, second, a split-T is welded to the bottom of a V-shaped rib. It is a hybrid rib because

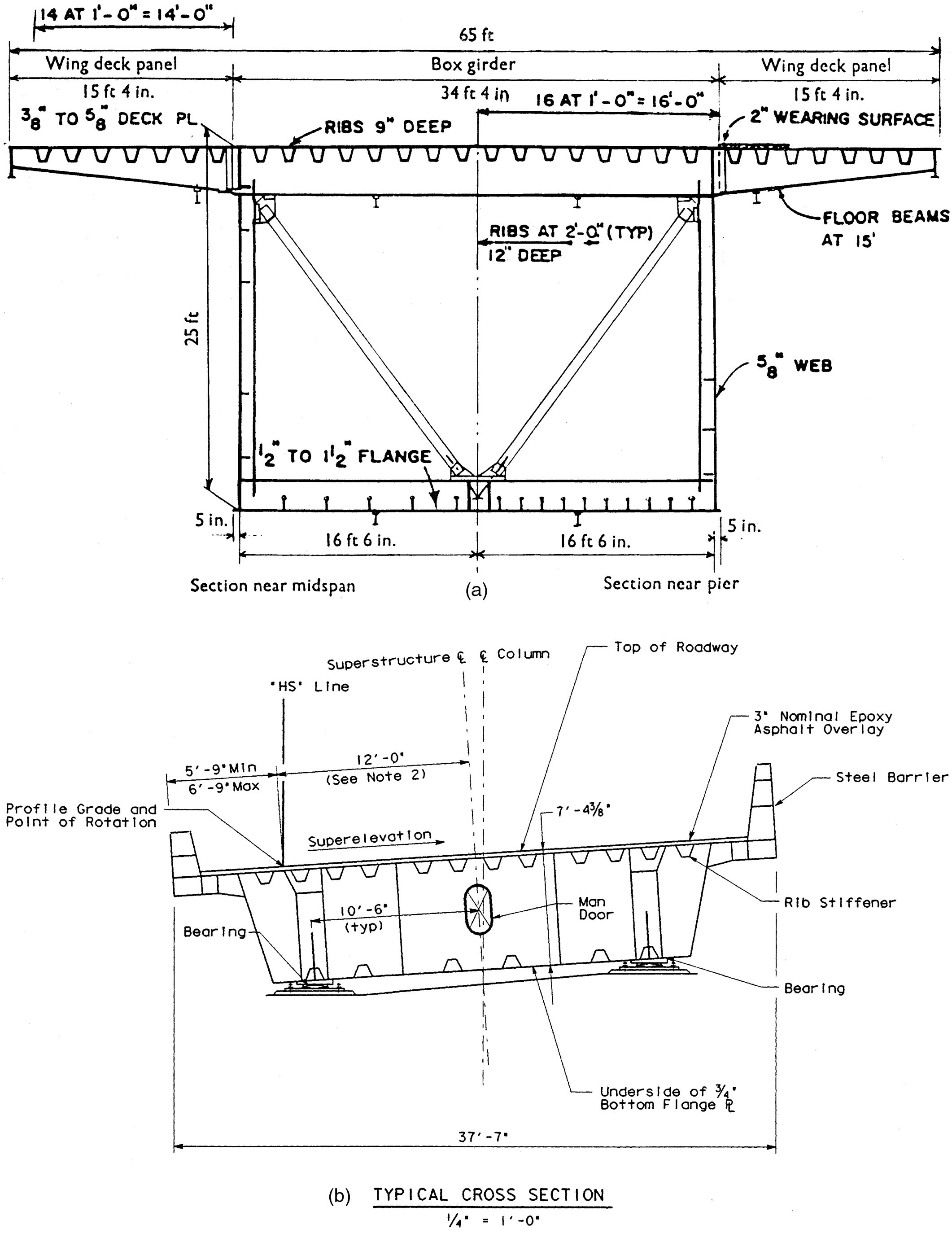
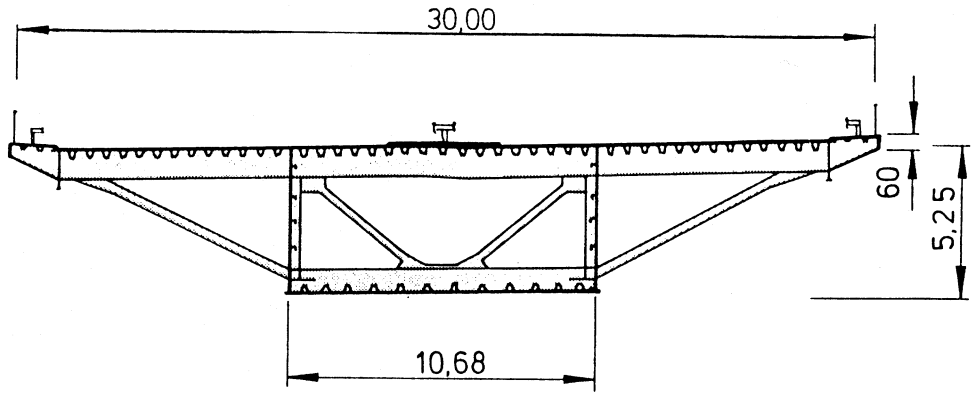
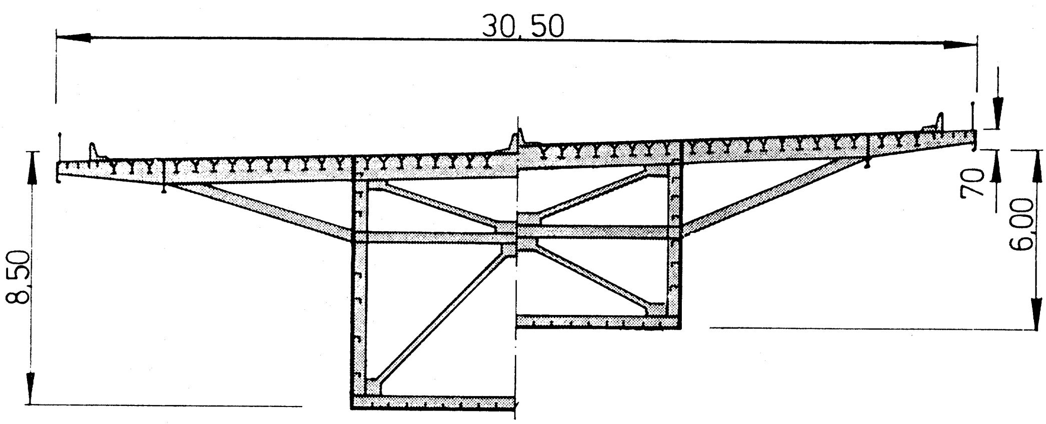


FIGURE 14.8 Steel box-girder bridges (a) San Diego–Coronado Bridge, California. (From Institute of Civil Engineers, Steel box-girder bridges, in *Proceedings of the International Conference*, Thomas Telford Publishing, London, 1973. With permission. (b) Maritime Off-Ramp Bridge, California. (Courtesy of the ICF KAISER Engineers.)

it has characteristics of both the open and closed rib. Some references have categorized it with closed ribs. The top portion provides good torsional stability to the deck, but the lower split-T portion has the buckling and corrosion disadvantages of an open rib. The split-T provides much greater bending strength, but no torsional stiffness to the deck. An open rib has one weld, a closed rib has



(a) Wurzburg Viaduct



(b) Moseltal Bridge Winninzen (Autobahn A61 Koblenz)

FIGURE 14.9 Steel box girder with strutted deck bridges. (From Leonhardt, F., *Bridge Aesthetics and Design,* MIT Press, Cambridge, MA, 1984. Deutsche Verlags-Anstalt, Stuttgart, Germany. With permission.)

two welds, and a hybrid rib has three welds. This third weld is another possible source for fatigue cracking (see Section 14.4.5). The martini-glass-shaped ribs have only been used by the Germans for about 30 bridges, and ignored by engineers in the United States (See Table 14.1).

14.3.3 Arch Bridges

Arch superstructures also utilize orthotropic deck systems. The arch can either support the deck in one line or two lines of support. The Barqueta Bridge (Figure 14.10a) is a unique signature bridge with high aesthetic appearance built for the Expo 90 Fair held in Seville, Spain [31,32]. This bridge utilizes an aerodynamic system with torsional rigidity because it is supported by a single line of suspender cables from a single arch located at the centerline of the bridge. The center portion of the superstructure cross section is reinforced for the high stress concentrations from the suspender cables. Trapezoidal ribs are also used for the bottom flange or soffit. The bridge was erected on one side of the river and floated in a rotating pattern into a permanent orientation.

The Fremont Bridge of Portland Oregon has an orthotropic deck with trapezoidal ribs for the upper deck with a conventional reinforced concrete slab on steel girders for the lower deck

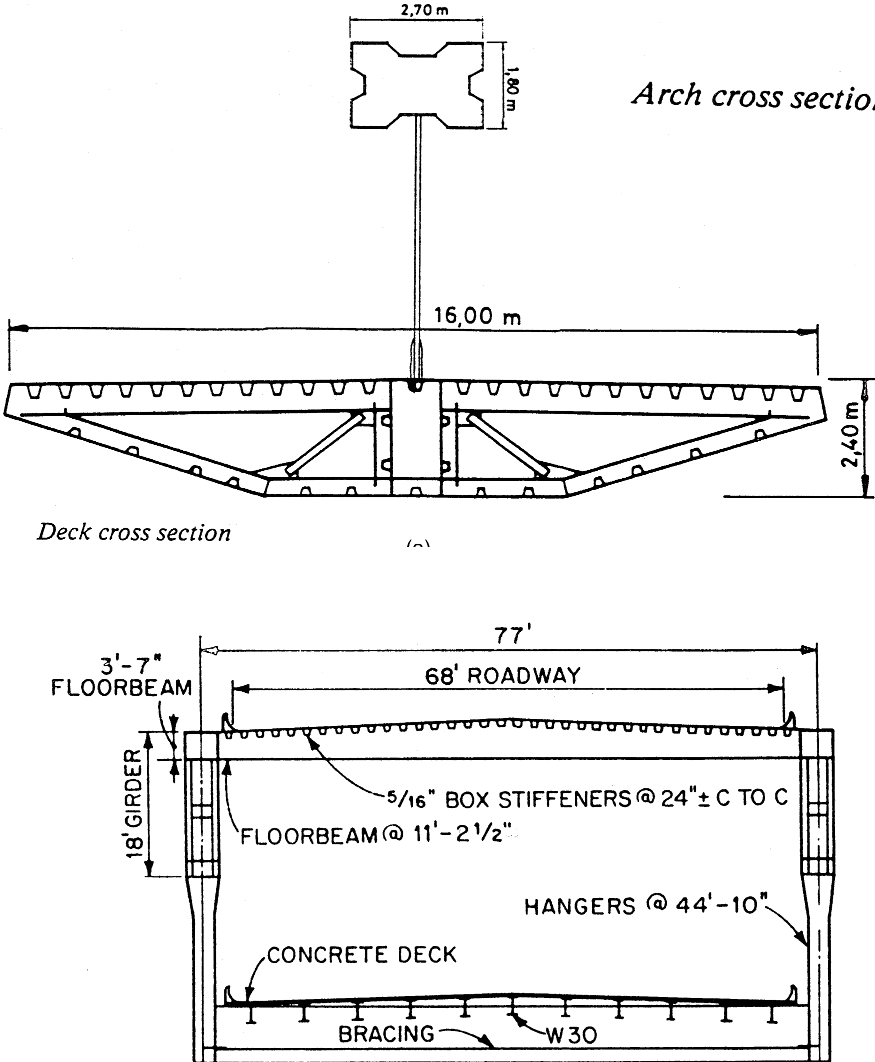
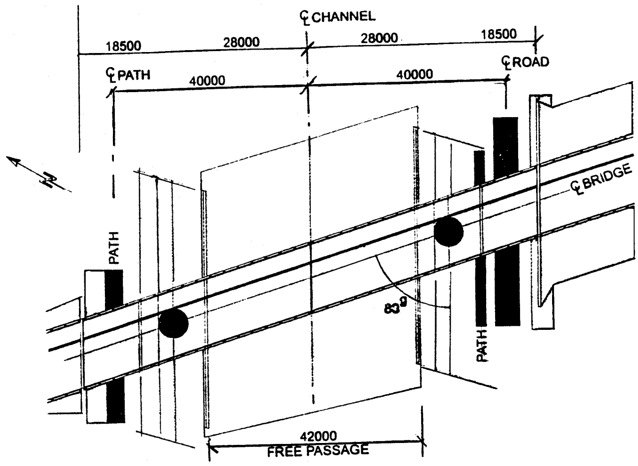


FIGURE 14.10 Arch bridges. (a) Barqueta Bridge, Seville, Spain. (From Arenas, J. J., and Pantaleon, M. J., *Structural Engineering International*, IABSE, 2(4), 251, 1992. With permission.) (b) Fremont Bridge, Portland, Oregon. (From Hedelfine, A. and Merritt, F. S., Ed., *Structural Steel Engineering Handbook*, McGraw Hill, New York, NY., 1972. With permission.)

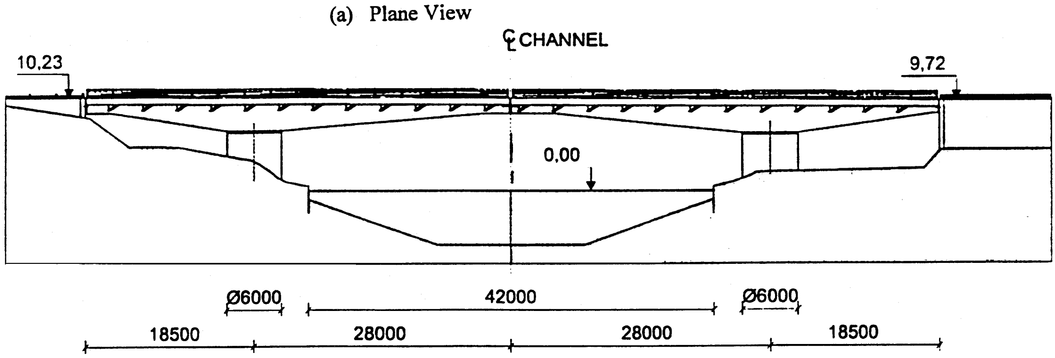
[Figure 14.10b]. This bridge is a tied arch with a 383-m main span and was the fourth longest arch in the world upon completion in 1973. The deck provides lateral stability to the truss. The truss has two levels of traffic and is painted. The bridge was erected in large pieces with an oceangoing barge crane from the river dividing downtown Portland [33].

14.3.4 Movable Bridges

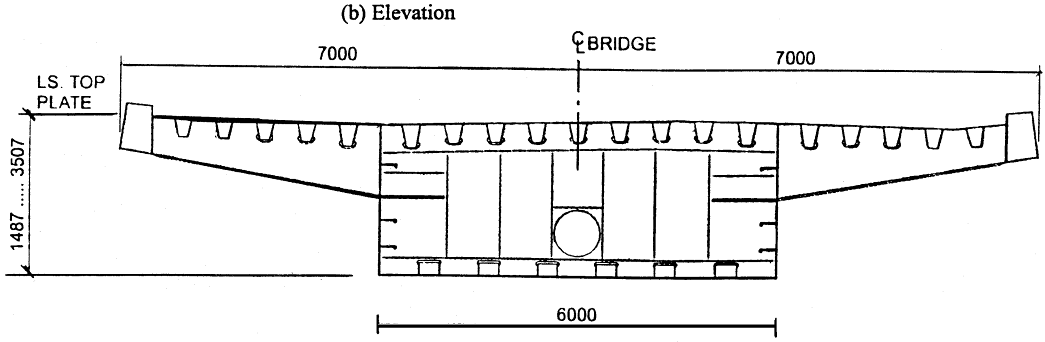
The orthotropic deck system has the lowest weight for movable bridges, so it is surprising how few examples there are of this excellent system. The swing-bridge design across a 42-m-wide navigation channel near Naestvad, Denmark was selected over 14 proposals from five competing consulting firms (Figure 14.11a). The superstructure is divided into two symmetrical components [34]. The



(a) Plane View



(b) Elevation



(c) Cross Section

FIGURE 14.11 Swing bridge near Naestved, Denmark. (a) Plane view; (b) elevation; (c) cross section. (Thomsen, K. and Pedersen, K. E, *Structural Engineering International,* IABSE, 8(3), 201, 1998. With permission.)

main component is a 6-m-wide variable-depth box girder with tapered cantilevered floor beams (Figure 14.11b). The top deck plate is stiffened by a cold-rolled trapezoidal rib (Figure 14.11c). The bottom flange plate of the box girder is stiffened by a cold-rolled rectangular-shaped rib. The two exterior web plates of the box girder are stiffened by a bulb-shaped rib. The two sections were fabricated in a shop and barged to the bridge site utilizing the navigation channel. The exterior is painted, and the interior uses dehumidification equipment (see Section 14.4.6) in each of the two sections. The bridge opened to traffic in 1997.

There is an orthotropic swing bridge at the southern mouth of a small slough east of the main channel of the St. Clair River on the Walpole Island Indian Reservation, Ontario, Canada built in

1970. This movable bridge allows small pleasure craft traveling between Lake Huron and Lake St. Clair to pass through. Vehicles can travel to and from the island across this bridge, which was the first movable bridge built with an orthotropic deck in North America. The advantages of this solution are discussed in detail in Reference [35].

The Danziger Vertical Lift Bridge, completed in 1988, is the world's widest vertical-lift bridge and carries seven lanes of traffic on U.S. 90 through downtown New Orleans across the Industrial Canal. The orthotropic deck that is lifted is a 33 m wide × 97 m span supported by three steel 4.26 m deep × 1.82 m wide box beams. The spacing of the box beams is 11.5 m with split-T shaped tapered floor beams at 4.42 m on center. The cantilever on the floor beams is 3.31 m from the face of the box girder. The rectangular boxes are fabricated of ASTM A572 and A588 steel for the main plate and A36 steel for secondary members including all steel median barriers. The ASTM A572 ribs are 5⁄16-in.-thick, 10-in.-deep, 27.88-plf rib as shown in Figure 14.7.

The world's largest double-leaf bascule bridge was opened to traffic in 1969 in Cadiz, Spain [2]. The main girders cantilever 48.3 m, providing a channel between Puerto de Santa Maria and Cadiz of 96.7 m. The orthotropic deck spans 2 m from between split-T-shaped transverse floor beams, which cantilever 2.6 m. At each side of the 12-m deck plate are sidewalks. The two main plate girders are tapered, with maximum depth of 5 m, and are 6 m on center. Sway struts are between floor beam and midspan of plate girders. The signature bridge Erasmus of Rotterdam, the Netherlands utilizes trapezoidal orthotropic deck on both the cable-stayed portion and bascule span. This 33-m-wide by 50-m-long bascule span is skewed at 22°, with an opening of 56 m. The bridge has a very thin, 8-mm wearing surface and was opened in 1996. The Miller Sweeny bridge of Alameda Island California is the only orthotropic bascule bridge in North America.

A unique concept is to have an entire 11.6-m-wide by 33-m-long midsection removed by two cranes to allow ship traffic to pass through once every 2 to 3 years [36]. A conventional concrete box-girder bridge supports a drop-in orthotropic box-girder component that has a much smaller mass than concrete. This allows two smaller cranes to move the drop-in unit. The ribs were fabricated from 610-mm-wide plate, a standard plate dimension in the United States. Four of these plates would be cut without waste from warehoused stock plate received directly from the factory. Apparently, there have been no plate optimization studies performed by the steel industry. Also 254-mm-thick urethane foam insulation was sprayed on the bottom face of the steel deck to reduce the tendency of the steel deck to change temperatures more quickly.

A much more dramatic system is planned to move an entire 410-m superstructure in Japan. An all-steel superstructure is planned for the Yumeshima–Maishima Bridge in the "Tech Port Osaka" to be completed in the year 2000. Each end of the all-steel bridge is supported by a 58 × 58 m steel pontoon and will be moved by tugboats. This would be the world's largest movable bridge, with a deck area of 12,000 m^2 [37]. A scale model has been built and the estimated cost of the completed bridge is U.S. $400 million.

A unique civil engineering structure is the curved tidal surge gates of Rotterdam, The Netherlands. The two floating gates, each about the size of the Eiffel Tower, are made of orthotropic deck with trapezoidal ribs. Each gate has 20,500 tons of steel, with 14 mm deck plate. A seawater ballast system is used to adjust this structure to various tidal surge freeboard heights.

14.3.5 Truss Bridges

The German Federal Railways uses its standard orthotropic deck system, shown in Figure 14.5, for one-track bridges using a steel truss superstructure. The lateral bracing for the through truss is provided by the stiffness of the orthotropic deck. The standardization of their steel bridge deck plus floor beams keeps fabrication cost to a minimum [18].

A steel truss is used in both the transverse and longitudinal directions for the double-wall steel pontoon for the port of Iquitos on the Amazon River, in Peru (Figure 14.12a). The orthotropic steel deck with trapezoidal rib is used for the top deck that supports vehicular traffic. The side walls that

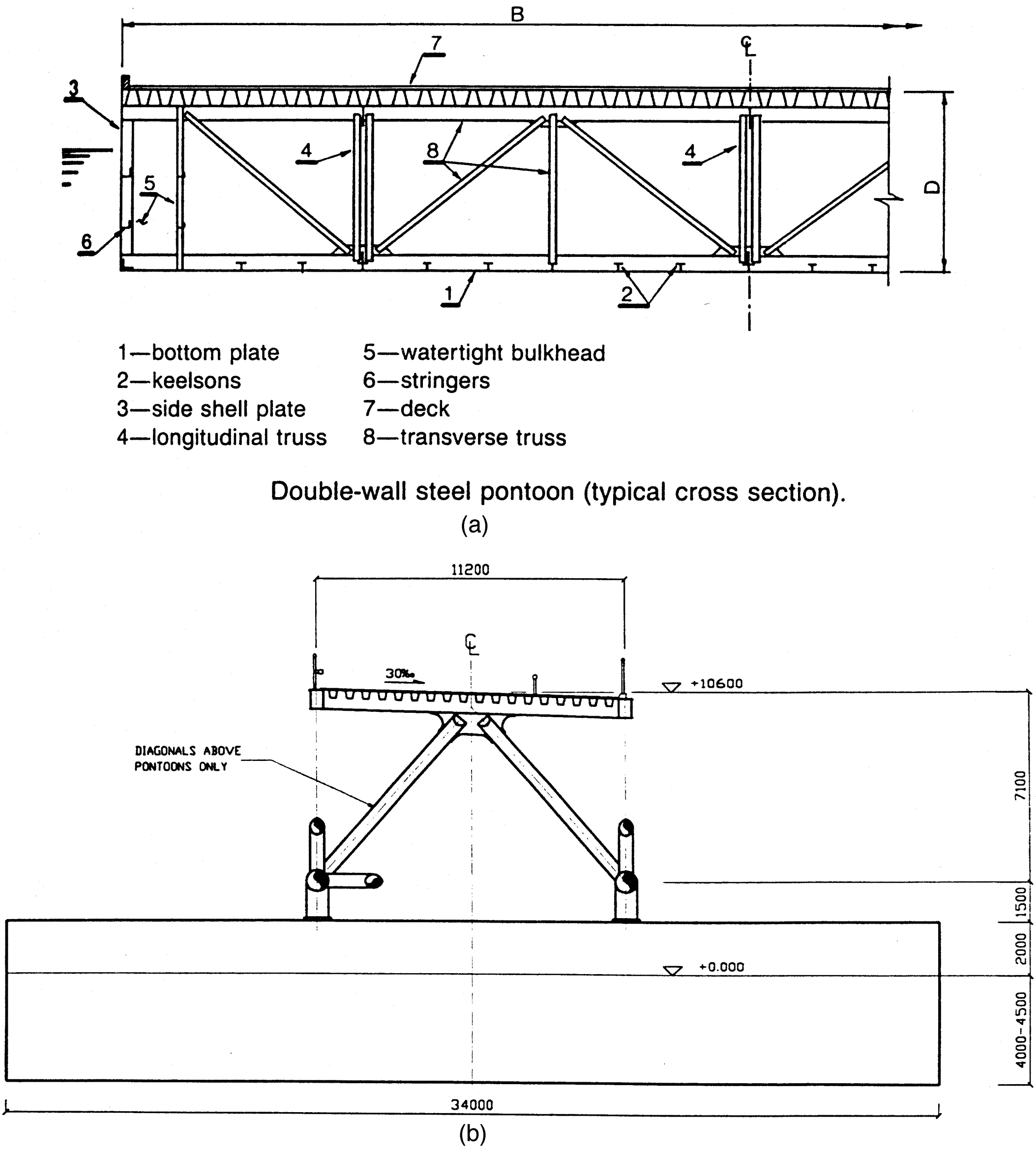


FIGURE 14.12 Truss bridges. (a) Floating dock on Amazon River. (From Tsinker, G. P., *Floating Ports Design and Construction Practices,* Gulf Publishing Company, Houston, TX, 1986. With permission.) (b) Bergsoysund Floating Bridge. (From Solland, G., Stein, H., and Gustavsen, J. H., *Structural Engineering International,* IABSE, 3(3), 142, 1993. With permission.)

have lower water pressure utilize small angles for the ribs. The bottom plate ribs, or keelsons (nautical terminology), are larger split-Ts because of the higher water pressure. This watertight floating pontoon is completely built of welded steel [38]. This framing system is also known as a "space truss" commonly used for building roofs.

The Bergsoysund Floating Bridge comprises floating concrete pontoons with a painted steel truss superstructure (Figure 14.12b). Floating orthotropic bridges become very economical for Norwegian fjords, which are actually deeper than the adjacent Atlantic Ocean floor. Lateral stability of the entire bridge is provided by an arch shape (in plan view) rather than cables with anchors in the 300-m-deep

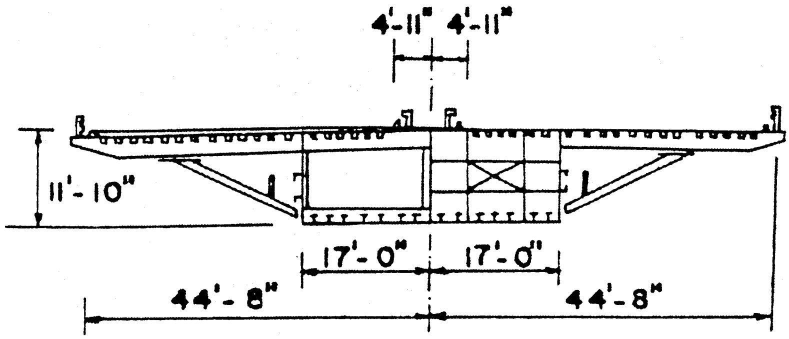


FIGURE 14.13 Papineau–Leblanc cable-stayed bridges. (From Troitsky, M. S., *Orthotropic Steel Deck Bridges*, 2nd ed., JFL Arc Welding Foundation, Cleveland, OH, 1987. Courtesy of the James F. Lincoln Arc Welding Foundation.)

fjord. The lateral stability of the top chords of the trusses is assisted by transverse stiffness of the orthotropic deck. The three-dimensional space truss is built of hollow steel pipe tubular joints, which have the minimum exposed area to resist corrosion. Detailing and design of these joints were based on experience developed for tubular offshore structures built in the North Sea. The closed trapezoidal rib was used, since the bridge is totally exposed to corrosive saltwater spray. Also, it was very important to minimize total bridge weight to reduce the size of the concrete pontoons. To avoid future painting in the ocean water, concrete was selected over orthotropic pontoons similar to Figure 14.12a. This bridge is a state-of-the-art solution utilizing offshore oil platform technology combined with floating bridge design technology [39].

14.3.6 Cable-Stayed Bridges

In cable-stayed bridges, the superstructures can be supported by one or two planes or lines of cables. Additional compression stresses occur in a cable-stayed bridge superstructure where the orthotropic deck is the compression component since the cables are the tension component. The Papineau–Leblanc Bridge, completed in 1969, linking the city of Montreal to Laval Islands, Canada, is a strutted deck box girder supported by a single line of cable stays at the centerline of the superstructure as shown in Figure 14.13. The bridge has spans of 90, 241, and 90 m with a superstructure width of 27.44 m [2,40]. Extra diaphragm plates were located at the cable support locations to transfer the loads from the deck into the cable stay as shown in right side of the split cross section of Figure 14.13. Closed U-shaped ribs were used with the top 7⁄16-in.-thick top deck plate and open ribs for the lower flange plate. The two bridge piers are cone-shaped. The pier face is at a 23° slope to bend the river ice, thus breaking the ice into pieces.

The Bratislava Bridge, completed in 1972 with a main span of 303 m crosses the Danube River, a major transportation river for barges, in Czechoslovakia. The orthotropic superstructure is a double-deck cellular box girder supported by a single line of cable stays at the centerline of the superstructure. The bridge has an anchor span of 75 m, and a superstructure width of 21 m. The feature that makes this bridge a unique signature span is the circular coffeehouse on top of the 85-m-high A-frame tower. Tourists can ride up an elevator in one tower leg to reach the sight-seeing windows. An emergency staircase is located in the other steel tower leg. Another nice feature is the protected pedestrian walkways on each side of the lower orthotropic deck. This feature gives wind and rain protection. The interior of the cross section contains utilities that cross the Danube [40, 41]. The coffeehouse is “saucer shaped,” probably inspired by the Seattle Space Needle. The framing consists of steel bowstring trusses for the roof and floor.

The Luling or Luling–Destrehan or Hale Boggs Bridge near New Orleans is a weathering steel bridge that spans the Mississippi River and was completed in 1983. Its superstructure has twin trapezoidal box girders. The floor beams and deck have four bolted splice points in the longitudinal direction and are supported by two planes of cable stays. Trapezoidal ribs are used for the deck system [42]. The main span is 383 m and has an aerodynamic shape to withstand hurricanes. The bridge uses the 5⁄16-in.-thick, 9-in.-deep, 25.64-plf rib as shown in Table 14.3. The center barrier and exterior barriers are welded steel plate bolted to the deck with welded studs. This bridge was fabricated in Japan and shipped to the United States. The world's longest clear-span cable-stayed bridge is a steel orthotropic deck bridge in Japan, and the second longest is the Normandie Bridge in France. Both bridges have two planes of cable stays.

Shown in Figure 14.14 is half of the proposed orthotropic superstructure option B for the cable-stayed replacement bridge for the East Span of the San Francisco–Oakland Bay Bridge. A family of solutions with single and dual towers has reached the 30% design development. The concept illustrated has a divided or separated superstructure connected by steel stiffening trusses to be built of steel tubes. Each half is planned to carry five lanes of traffic and be supported by a single tower. The separated superstructure allows the wind to flow around each side, as well as through the center, reducing wind forces [44].

14.3.7 Suspension Bridges

Suspension bridge superstructures may be supported by one or two planes of cables. Shown in Figure 14.15 is the orthotropic aerodynamic superstructure option for the suspension replacement bridge for the East Span of the San Francisco–Oakland Bay Bridge. A family of solutions with single and dual towers has reached the 30% design development. The concept illustrated has a separated superstructure, and each bridge is planned to carry five lanes of traffic in one direction. Each half is actually an independent bridge, and this superstructure solution is based on the British Severn Bridge completed in 1966. Note how each rib has a different cutout hole to eliminate fatigue cracks depending on the rib shape [44]. The San Francisco–Oakland Bay Bridge East replacement Spans design is currently evolving, and final approved plans have not yet been completed.

The Konohana Bridge in Japan has a main span of 300 m with side spans of 120 m [45]. This is a monocable self-anchoring suspension bridge. Its superstructure is supported only at the centerline of the bridge (Figure 14.16a). This signature span is supported by suspender cables attached to the hanger connection plate. An isometric of the main stiffening plates and diaphragms of the superstructure is shown in Figure 14.16a. Note the concentration of plates needed to distribute cable forces throughout the superstructure. The cantilevering sides of the superstructure mandate an aerodynamic box-girder superstructure. The Japanese rib 242 mm × 324 mm × 8 mm was used (Table 14.3).

Shown in Figure 14.16b is the suspension bridge with two planes of cables and a 762 m main span currently under final design for the Third Carquinez Strait Bridge between San Francisco and Sacramento, California. It is an aerodynamic superstructure with trapezoidal ribs for the deck and bottom soffit [46]. This bridge design has been heavily influenced by the successful bridges built in Europe and Japan.

Shown in Figure 14.17 is a drawing of the world's longest concept suspension bridge, proposed to span the straits of Messina [47, 48]. This clear span between Sicily and Italy has been a historical challenge to engineers. This concept has been given technical approval for the final design by the various Italian authorities and political approval is in progress. European engineers have already performed wind tunnel studies in order to develop contract plans and specifications. The superstructure is supported by two pairs of cables and a 3000-m main span is composed of the middle of welded steel framing system that is 52 m wide. A separated superstructure concept allows wind to flow through the superstructure. The Messina and the option B for the cable-stayed replacement bridge for the East Span of the San Francisco–Oakland Bay Bridge are similar in principle. The

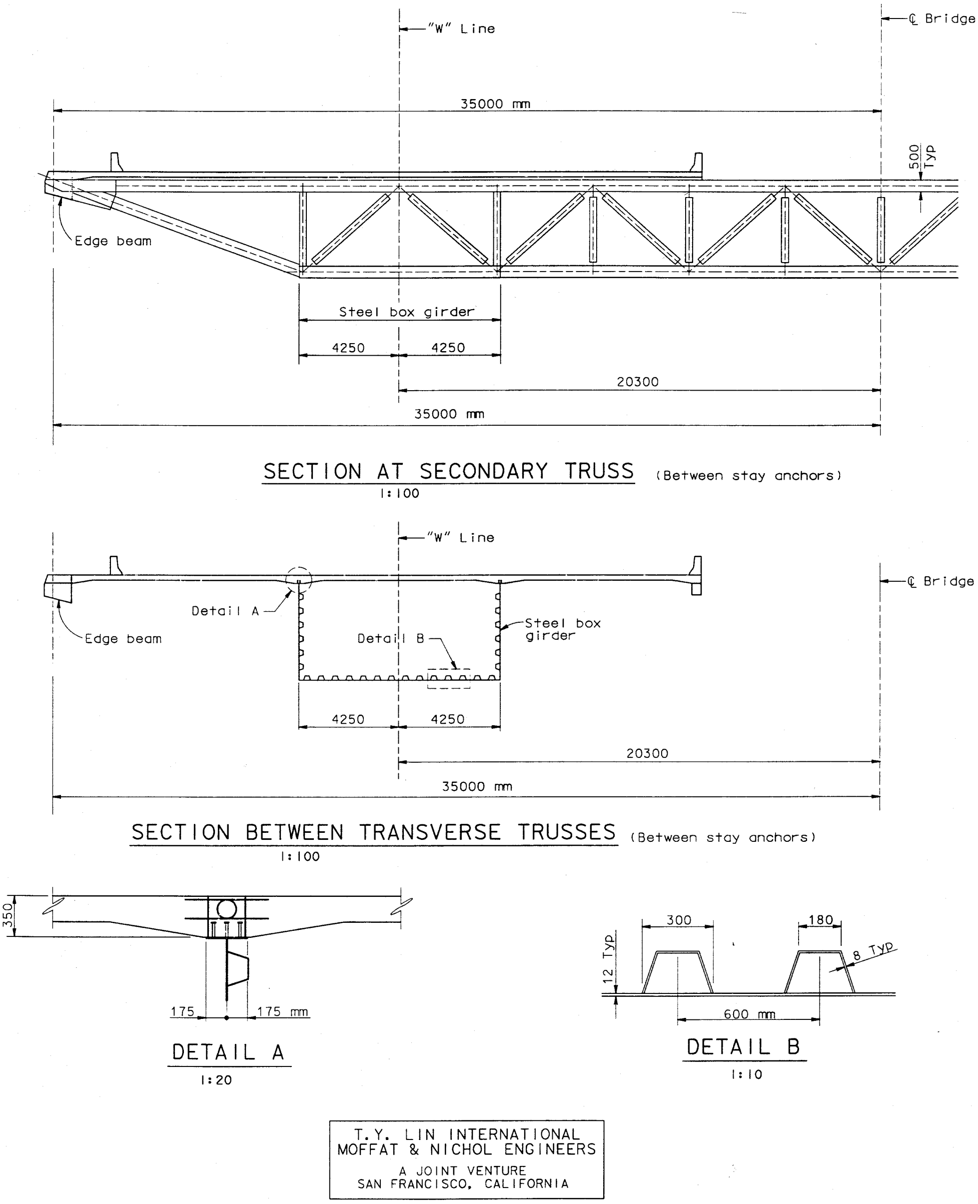


FIGURE 14.14 San Francisco–Oakland Bay Bridge East Signature Bridge Span — proposed cable-stayed bridge option. (Courtesy of T. Y. Lin International and Moffat & Nichols.)

open spaces, while strongly increasing the aeraelastic stability (e.g. flutter stability), reduce wind loading and have a grillage or grating that allow lanes for emergency stop and maintenance vehicle traffic. Three longitudinal independent wing-shaped box girders are linked transversely with very large welded steel box-shaped cross girders. The orthotropic deck is stiffened by trapezoidal ribs for the top deck and open ribs for the bottom soffit. All the barriers have aerodynamic shape and grating to reduce wind forces. New suspension bridges with double-decker superstructures with an upper orthotropic deck for vehicles and a lower orthotropic deck for commuter trains are in use in Asia. Examples are the Tsing Ma Bridge of Hong Kong, China and Yong Jong Grand self-anchoring suspension bridge of Seoul, Korea.

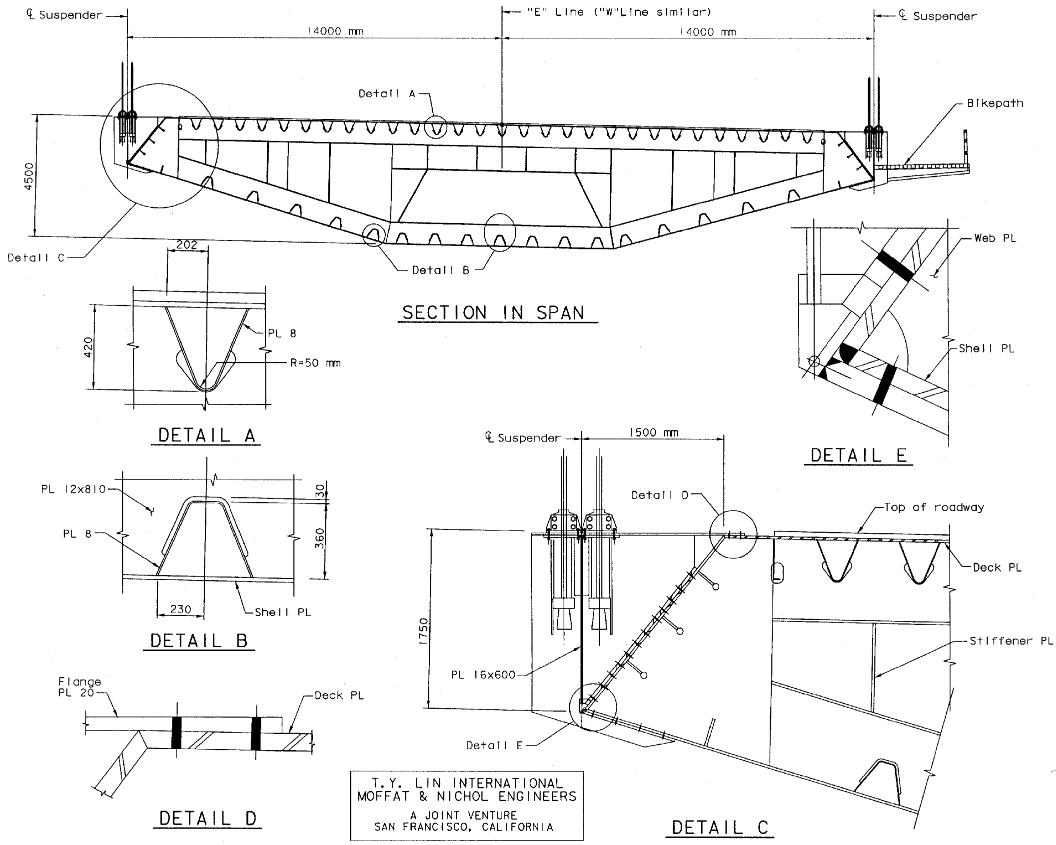


FIGURE 14.15 San Francisco–Oakland Bay Bridge East Signature Bridge Span — selected suspension bridge option. (Courtesy of T. Y. Lin International and Moffat & Nichols.)

Some older suspension bridges have been retrofitted with installation of small orthotropic panels to accommodate higher traffic loading and to extend the useful or fatigue life of bridges. Currently, there are more retrofitted North American suspension bridges than new bridges with orthotropic decks. Small deck panels have been trucked onto various bridges to replace the portion of reinforced concrete deck that was removed (see Table 14.6 and Figure 14.3). The Golden Gate Bridge uses essentially the same 3/8-in.-thick, 11-in.-deep, 35.94-plf rib as shown in Table 14.3. The Wakato Bridge in Kitakusyu, Japan was redecked, very wide pedestrian sidewalks with only two traffic lanes were eliminated, and the bridge was converted to four lanes of vehicular traffic without pedestrian access. The historical Williamsburg Suspension Bridge, built in 1903, is the most recent redecking project and, essentially, uses this rib shape again [49]. Extensive testing of a full-size mockup of the designer's concept was performed to verify its durability or fatigue life. An extra internal plate or miniature diaphragm or rib bulkhead aligns with the floor beam web. Welding detail options and cutout holes or scallops were tested and showed that the selected system should have a long fatigue life.

14.4 Design Considerations

14.4.1 General

In contrast to the conventionally designed bridge, where the individual structural elements (stringers, floor beams, and main girders) are assumed to perform separately, an orthotropic steel deck bridge is a complex structural system in which the component members are closely interrelated.

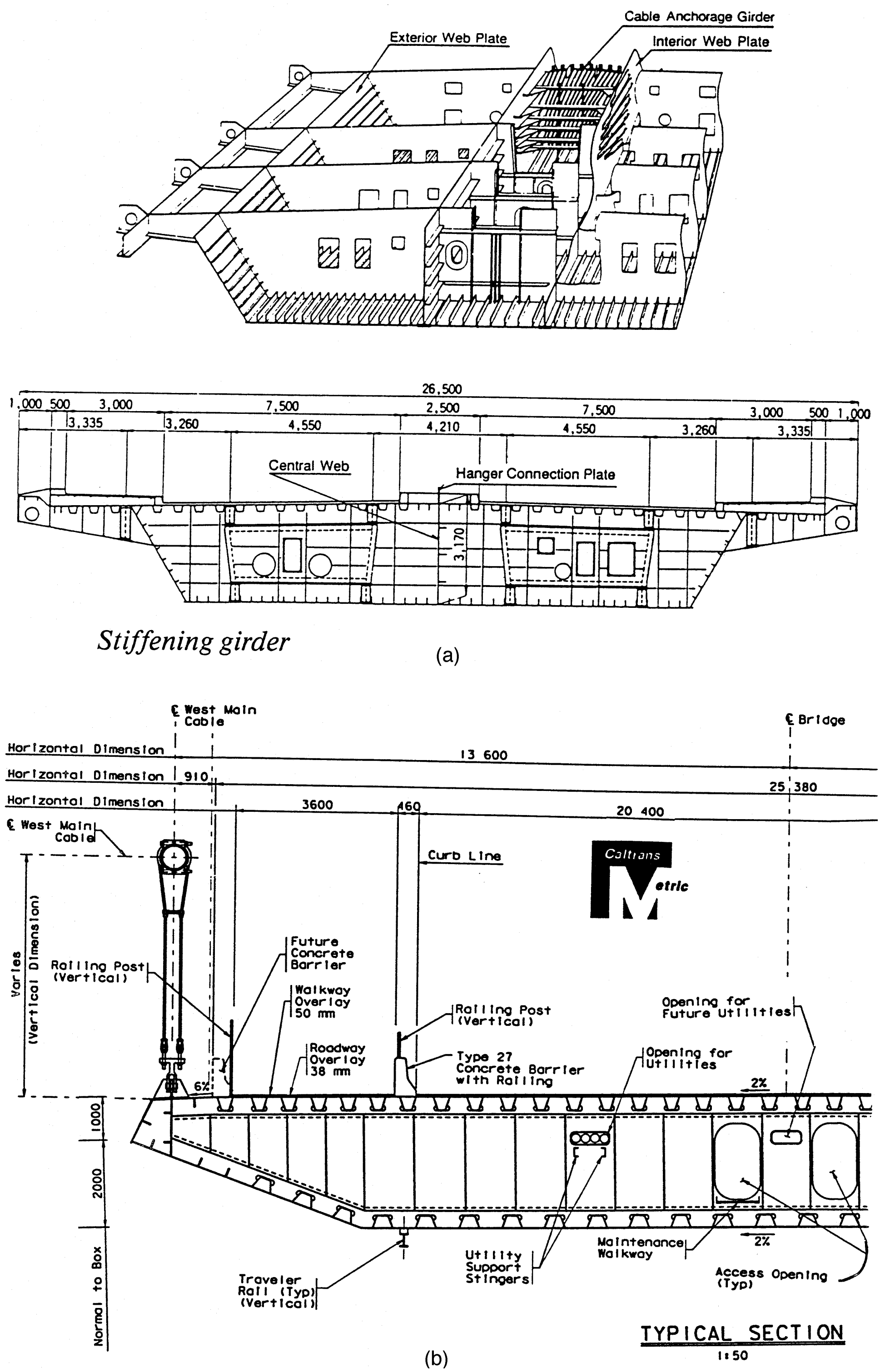


FIGURE 14.16 Suspension bridges (a) Konohana Bridge superstructure. (From Kamei, M., Maruyama, T. and Tanaka H., *Structural Engineering International*, IABSE, 2(1), 4, 1992. With permission.) (b) Third Carquinez Bridge stiffening girder. (Courtesy of De Leuw-OPAC-Steinman.)

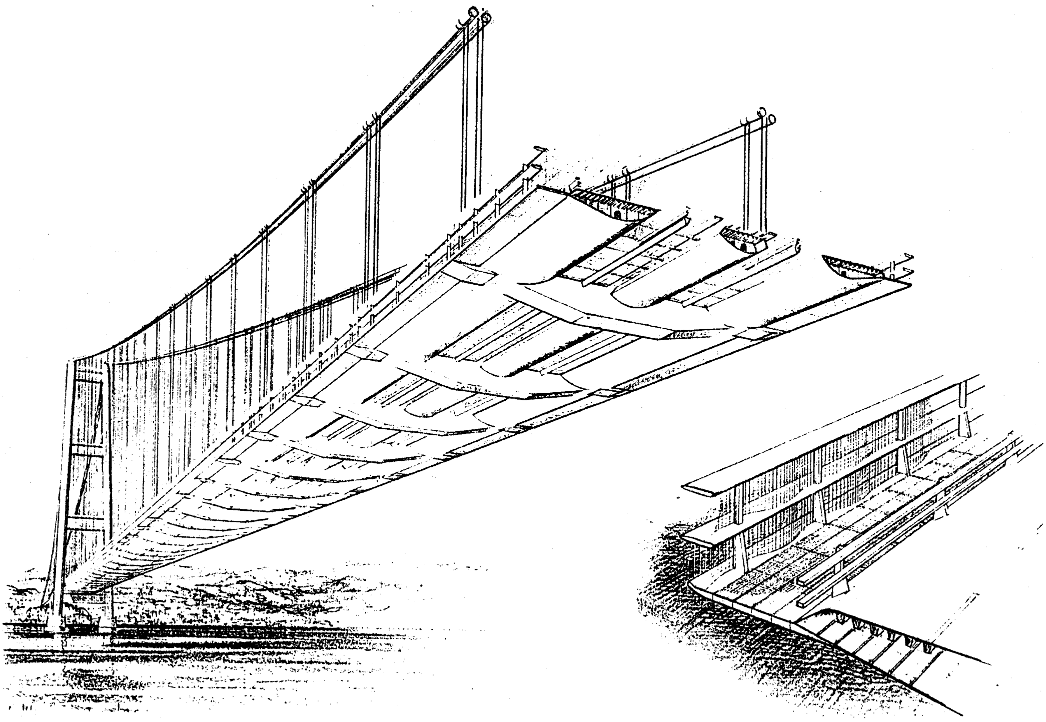


FIGURE 14.17 World's biggest single-span bridge. The deck arrangement of the Messina Bridge can be clearly seen from this drawing; grillages will help with the aerodynamics. (Courtesy of Brown, W. at Brown Beech & Associates.)

The stress in the deck plate is the combination of the effects of the various functions performed by the deck. For structural analysis under dead and live loads, it is necessary for design convenience to treat the following structural members separately.

In a typical orthotropic deck system as shown in Figure 14.18, Member I is defined as the deck, a flat plate supported by welded ribs as shown in Figure 14.1 and Table 14.1. The deck plate acts locally as a continuous member directly supporting the concentrated wheel loads placed between the ribs and transmitting the reactions to the ribs. The design of the deck plate is discussed in Section 14.4.2.

Member II is defined as a rib spanning from a floor beam to a floor beam (normally a continuous element of at least two spans) as shown in Figure 14.18. The stiffened steel plate deck (acting as a bridge floor between the floor beams) consists of the ribs plus the deck plate that is the common upper flange. A detailed discussion of rib design is given in Section 14.4.3.

Member III is the floor beam that spans between the main girders. Member IV is defined as a girder spanning from a column (or cable) to column (or cable) as shown in Figure 14.18 and is normally a continuous element of at least two spans to be economically viable. The deck also acts as part of this member. In the computation of stresses, the effective cross-sectional area of the deck plus the inclusion of all longitudinal ribs is considered as the flange. The determination of the effective width of the deck and design stresses will be discussed in Section 14.4.4. The orthotropic deck plate receives stresses under multiple loading combinations as shown in Figure 14.19. This is because the deck plate is the top flange of the Member II, Member III, and Member IV. An orthotropic steel deck should be considered an integral part of the bridge superstructure. The deck plate acts as a common flange of the ribs, the floor beams, and the main longitudinal components of the bridge. Any structural arrangement in which the deck plate is made to act independently from the main components is undesirable.

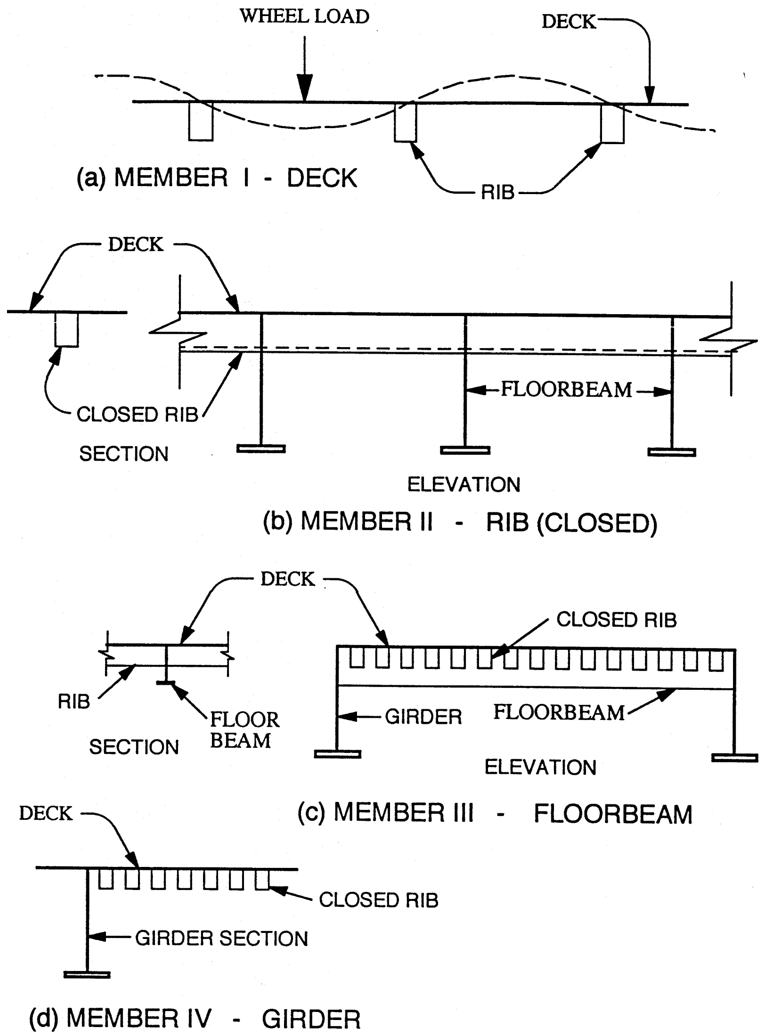


FIGURE 14.18 Four members to be analyzed in an orthotropic deck. (a) Member I — deck; (b) Member II — rib (closed); Member III — floorbeam; (d) Member IV — girder.

When redecking the bridge, if the orthotropic deck is supported by existing floor beams, the connection between the deck and the floor beam should be designed for full composite action, even if the effect of composite action is neglected in the design of floor beams. Where practical, connection suitable to develop composite action between the deck and the main longitudinal components should be provided.

The effects due to global tension and compression should be considered and combined with local effects. When decks are in global tension, the factored resistance of decks subject to global tension, *Pu*, due to the factored loads with simultaneous global shear combined with local flexural must satisfy [5]:

$$\frac{P_u}{P_r} + \frac{M_{lr}}{M_{rr}} \leq 1.33 \tag{14.1}$$

$$P_u = A_{d,\text{eff}}\sqrt{f_g^2 + 3f_{vg}^2} \tag{14.2}$$

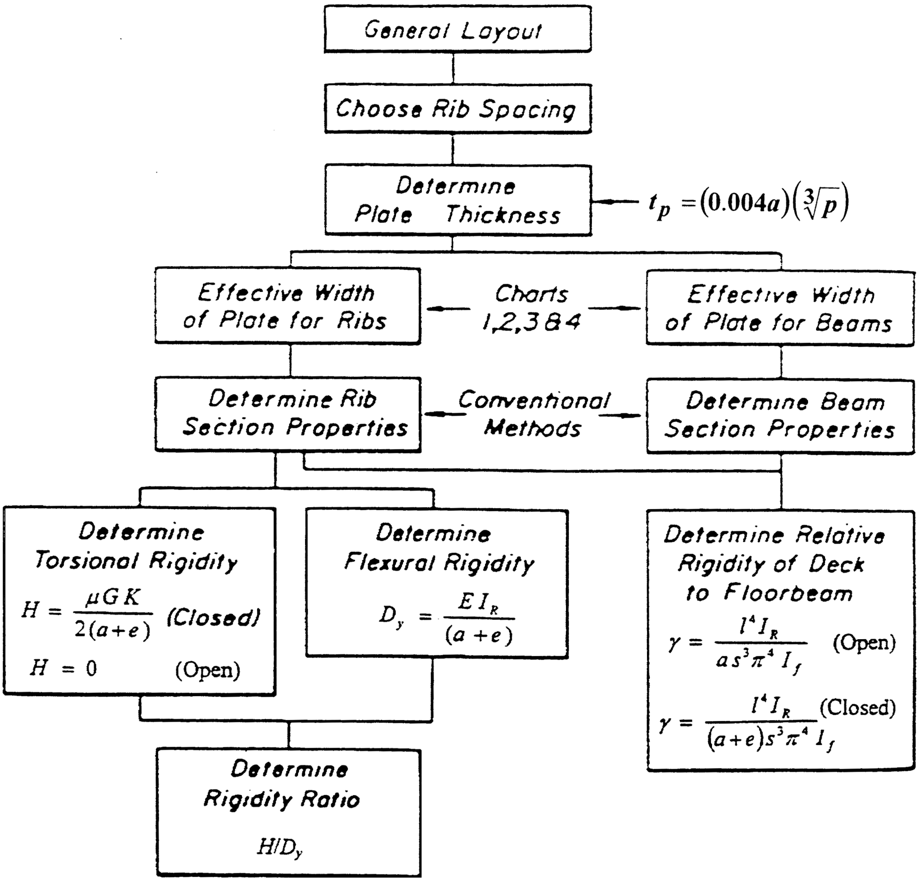


FIGURE 14.19 Determination of required section properties flow chart (metric formula). (From Milek, W. A., Jr., *Eng. J. AISC*, 40, 1974. With permission.)

where

f_g = axial global stress in deck (MPa)
f_{vg} = simultaneous global shear in the deck (MPa)
$A_{d,\,\text{eff}}$ = effective cross section area of the deck, including longitudinal ribs (mm^2)
P_r = nominal tensile resistance of the deck with consideration of effective deck width (N)
M_{lr} = local flexural moment of longitudinal rib due to the factored loads (N-mm)
M_{rr} = Flexural resistance of longitudinal rib, governed by yielding in extreme fiber (N-mm)

The effect of simultaneous shear is usually not significant in orthotropic decks of girder or truss bridges, but may be important in decks used as tension ties in arches or in compression for cable-stayed bridges.

When decks are under global compression, longitudinal ribs, including effective width of deck plate, should be designed as individual columns assumed to be simply supported at transverse beams. Buckling formulas for steel decks can be found in the AISC *Design Manual for Orthotropic Steel Plate Deck Bridges* [1].

Diaphragms or cross frames should be provided at each support and should have sufficient stiffness and strength to transmit lateral forces to the bearings and to resist transverse rotation, displacement, and distortion. Intermediate diaphragms or cross frames should be provided at locations consistent with the analysis of the girders and should have sufficient stiffness and strength to resist transverse distortions.

14.4.2 Deck Design

The primary function of the steel deck (Member I) is to directly support the traffic loads and to transmit the reactions to the longitudinal ribs. An important characteristic of the steel deck is its capacity for carrying concentrated loads. When loads approach the ultimate load the deck plate practically acts as a membrane and can carry on the order of 15 to 20 times the ultimate load computed in accordance with the ordinary flexural theory. Thus, the bridge deck plate possesses an ample local overload-carrying capacity.

The minimum thickness of the deck plate may be determined by allowable deflection of the deck plate under a wheel load, which should not exceed 1/300 of the spacing of the deck supports. Based on this criteria, the plate thickness, t_p, may be determined by Kloeppel's formula:

$$t_p \geq (0.004\,a)\left(\sqrt[3]{p}\right) \tag{14.3}$$

where:

a = spacing of the open ribs, or the maximum spacing of the walls of the closed ribs, in mm

p = wheel load unit pressure, under the AASHTO LRFD design tandem wheel load 55 kN, including 33% dynamic load allowance, in kPa. For 50 mm wearing surface, p is 449 kPa.

The distribution of the wheel load is assumed in a 45° footprint from the top of the wearing surface to the top of the deck plate. The AASHTO Specifications, 16th edition [50] tabulates wheel loads and contact area:

Wheel Load (kN)	Width Perpendicular to Traffic (mm)	Length Direction of Traffic (mm)
36	508 + 2t	203 + 2t
54	508 + 2t	203 + 2t
72	610 + 2t	203 + 2t

t = the thickness of the wearing surface in mm.

Using the AASHTO-LRFD [5], the wheel loads and contact area can be tabulated:

Wheel Load (kN)	Width Perpendicular to Traffic (mm)	Length Direction of Traffic (mm)
17.5 (truck)	510 + 2t	53 + 2t
55 (tandem)	510 + 2t	167 + 2t
72.5 (truck)	510 + 2t	220 + 2t

t = the thickness of the wearing surface in mm.

The current AASHTO-LRFD [5] requires that the minimum deck plate thickness, t_p, shall not be less than either 14 mm or 4% of the largest rib spacing. Experience from the durability of previously built bridges shows that this requirement is advisable for both constructibility and long-term bridge life.

For a rib spacing of 300 mm, a 14-mm plate is required per the AASHTO-LRFD, while a 9-mm plate can be derived using Eq. 14.3. For a rib spacing of 380 mm, a 15-mm plate is required per the AASHTO-LRFD, while a 12-mm can be derived using Eq. 14.3.

14.4.3 Rib Design

Table 14.2 gives the effective width of the deck plate acting with a rib. Since most of the ribs used in the current practice are closed type, this section will only discuss closed ribs. The ribs span between and are continuous at floor beams (Figure 14.18). Spacing of ribs depends on the deck plate thickness and usually for closed ribs ($a + e$) varies between 610 and 760 mm. Rib spans of approximately 4500 mm have been common in North American practice. But, spans up to 8500 mm, required by spacing of existing floor beams, have been used in bridge redeckings. Long rib spans are feasible and may be economical.

The minimum thickness of closed ribs should not be less than 4.75 mm per AASHTO-LRFD. Fatigue tests concluded that local out-of-plane flexural stress in the rib web at the junction with the deck plate should be minimized. It is necessary to limit the stress in the rib web caused by the rotation of the rib–deck plate junction by making the rib webs relatively slender compared with the deck plate. To achieve this, AASHTO-LRFD [5] specifies that the cross-sectional dimensions of an orthotropic steel deck shall satisfy:

$$\frac{t_r a^3}{t_{d,\mathrm{eff}}^3 h'} \leq 400 \tag{14.4}$$

where

t_f = thickness of rib web (mm)

$t_{d,\mathrm{eff}}$ = effective thickness of the deck plate, with consideration of the stiffening effect of the wearing surfacing (mm)

a = largest spacing between the rib webs (mm)

h' = length of the inclined portion of the rib web (mm)

To prevent overall buckling of the deck under compression induced by the bending of the girder, the slenderness, L/r, of a longitudinal rib shall not exceed the value given by the following equation in the AASHTO Standard Specification [50]:

$$\left(\frac{L}{r}\right)_{\max} = 83\sqrt{\frac{1500}{F_y} - \frac{2700F}{F_y^2}} \tag{14.5}$$

where

L = distance between transverse floor beams

r = radius of gyration about the horizontal centroidal axis of the rib including an effective width of the deck plate

F = maximum compressive stress in MPa in the deck plate as a result of the deck acting as top flange of the girder; this stress shall be taken as positive

F_y = yield strength of rib material in MPa

Orthotropic analysis furnishes distribution of loads to ribs and stresses in the member. Despite many simplifying assumptions, orthotropic plate theories that are available and reasonably in accordance with testing results and behaviors of existing structures require long, tedious computations. Computer modeling and analysis may be used to speed up the design. The AISC manual [1] was used when only expensive main-frame computers were available. The flowchart for this process is shown in Figure 14.19. Tables 14.3 to 14.5 were later created to assist those without computers. Today, engineers can write their own software or create the appropriate spreadsheet using a personal computer to expedite the iterative computations. Published tables provide useful databases to check against "bugs" in software. The following method, known as the Pelikan–Esslinger method, has been used in design of orthotropic plate bridges. In this method, the closed-rib decks are analyzed in two stages.

First, the deck with closed ribs is assumed on rigid supports. Only the longitudinal flexural rigidity and the torsional rigidity of the ribs are considered. The transverse flexural rigidity can be negligible. A good approximation of the deflection w may be presented in the form of Huber's differential equation:

$$D_y \frac{\partial^4 w}{\partial y^4} + 2H \frac{\partial^4 w}{\partial x^2 \partial y^2} = p(x,y) \tag{14.6}$$

where

D_y = flexural rigidity of the substitute orthotropic plate in the direction of the rib
H = equivalent torsional rigidity of the substitute orthotropic plate
$p(x,y)$ = load expressed as function of coordinates x and y

In the computation of D_y and H, the contribution of the plate to these parameters must be included. The flexural rigidity in the longitudinal direction usually is calculated as the rigidity of one rib with effective deck width divided by the rib spacing:

$$D_y = \frac{EI_y}{a+e} \tag{14.7}$$

where I_r = moment of inertia, including a rib and effective plate width.

Because of the flexibility of the orthotropic plate in the transverse direction, the full cross section is not completely effective in resisting torsion. Therefore, the formula for computing H includes a reduction factor u.

$$H = \frac{\mu GK}{2(a+e)} \tag{14.8}$$

where

G = shearing modulus of elasticity of steel
K = torsional factor, a function of the cross section

In general, for hollow closed ribs, the torsional factor may be obtained from

$$K = \frac{4A_v^2}{\frac{p_e}{t_r} + \frac{a}{t_p}} \tag{14.9}$$

where

A_r = mean of area enclosed by inner and outer boundaries of ribs
p_r = perimeter of rib, exclusive of top flange
t_r = rib thickness
t_p = plate thickness

The reduction factor u for a trapezoidal rib may be closely approximated by

$$\frac{1}{u} = 1 + \frac{GK}{EI_p} \frac{a^3}{12(a+e)^2} \left(\frac{\pi}{S_e}\right)^2 \left[\left(\frac{e}{a}\right)^3 + \left(\frac{e-b}{a+b} + \frac{b}{a}\right)^2\right] \tag{14.10}$$

where

I_p = moment of inertia = $Et_p^3 \big/ 12\left(1-u^2\right)$
s_2 = effective rib span for torsion = 0.81s
s = rib span

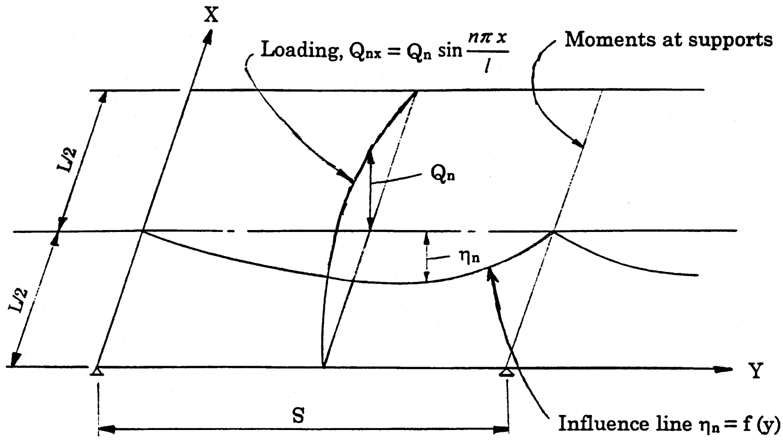


FIGURE 14.20 Computation of bending moments in orthotropic plate.

The values of D_y and H are combined in the relative rigidity coefficient H/D_y, which is a parameter characterizing the load-distributing capacity of the deck in the direction perpendicular to the ribs. For any given rib size, spacing, and deck plate thickness, H does not remain constant but increases with the rib span. Therefore, the parameter H/D_y is also a function of span, and the transverse load distribution of the deck structure improves as the span of the ribs is increased.

The general solution of Eq. (14.4) can only be given as an infinite series:

$$w_n = \left(C_{1n} \sinh a_n y + C_{2n} \cosh a_n y + C_{3n} a_n y + C_{4n}\right) \sin \frac{n\pi x}{l} \tag{14.11}$$

where

n = integer ranging from 1 to ∞ (odd numbers for symmetrical loads)
l = floor beam span
C_{in} = integration constant, determined by boundary conditions

$$a_n = \frac{n\pi}{l}\sqrt{\frac{2H}{D_y}} \tag{14.12}$$

The plate parameter H/D_y can be obtained from Table 14.4 or calculated.

Bending moments in the substitute orthotropic plate due to the given loading can be computed by formulas derived from the above solution. Since the solution can only be given as an infinite series, values of the influence ordinates, bending moments, etc. must be expressed as sums of the component values for each term n of the series of component loads Q_{nx}. The values needed for the design of the ribs are the bending moments in the orthotropic plate over the *floor beams* and at the midspan of the ribs. These moments are obtained by multiplying the values of the component loads Q_{nx} by corresponding influence-ordinate components η_n:

$$M = \sum Q_{nx}\eta_n \tag{14.13}$$

This is shown in Figure 14.20. Formulas and design charts for Q_{nx} and η_n for the various AASHTO loading cases are given in Ref. [1].

The bending moments M_y in the direction of the ribs are obtained in the substitute orthotropic plate system of unit width of the deck. Usually only the maximum moment ordinate $M_{y\max}$ at the center of the loaded rib is computed and the moment acting on one rib is then obtained, conservatively, as

$$M_R = M_{y\max}(a+e) \tag{14.14}$$

In the second stage of the design, the effect of the floor beam flexibility is considered. This effect will result in an increase of bending moment in the middle span and reduction of bending moment at the floor beam. The magnitude of the effect is determined based on the relative rigidity between ribs and floor beam. The effective width of the plate used for the first stage calculations generally can be used for the second stage with small error. Modifications of bending moments and shears in the ribs due the floor beam flexibility may be computed in the formulas and design charts in Ref. [1].

14.4.4 Floor Beam and Girder Design

Dead-load bending moments and shears in a *floor beam* (Member III) are calculated based on its own weight and the weights of tributary area of the deck. Live-load bending moments and shears in a floor beam are computed in two stages. In the first, the floor beams are assumed to act as rigid supports for the continuous ribs. The bending moments and shears are determined using a conventional method.

In the second stage, the effect of the floor beam flexibility is calculated, which tends to distribute the load on a directly loaded floor beam to the adjacent floor beams. The magnitude of this effect is a function of the relative rigidities between ribs and floor beams. The floor beam flexibility will reduce the bending moments and shears calculated in the first stage. The bending moment and shear corrections in the floor beams may be computed by the formulas and charts in Ref. [1].

The floor beam design per AASHTO-LRFD [5] for transverse flexure is

$$\frac{M_{fb}}{M_{rb}} + \frac{M_{ft}}{M_{rt}} \le 1.0 \tag{14.15}$$

where

M_{fb} = applied moment due to the factored loads in transverse beam (N-mm)

M_{rb} = factored moment resistance of transverse beam (N-mm)

M_{ft} = applied transverse moment in the deck plate due to the factored loads as a result of the plate-carrying wheel loads to adjacent longitudinal ribs (N-mm)

M_{rt} = factored moment resistance of deck plate in carrying wheel loads to adjacent ribs (N-mm)

For deck configurations with the spacing of the transverse beams larger than at least three times the spacing of longitudinal rib webs, the second term in Eq. (14.15) may be ignored. It should be noted that the applied moment M_{fb} should be obtained and based on the superstructure configuration.

The methods of analysis and design of the girders (Member IV) are described in other chapters of this book. The effective width of an orthotropic deck acting as the top flange of a girder or a floor beam is a function of the ratio of the span length to the girder or floor beam spacing, the cross-sectional area of the stress-carrying stiffeners, and the type and position of loading. Values of effective widths for the case of uniform loading based on a study by Moffatt and Dowling[58] are given in Figure 14.21 [5]. This figure was originally developed to determine the effective width of deck to be considered active with each web of a box girder, but is also applicable to other types of girders. The cumulative effect of the stresses on a specific orthotropic deck needs to be carefully reviewed by the design engineer and is generalized by Figure 14.22.

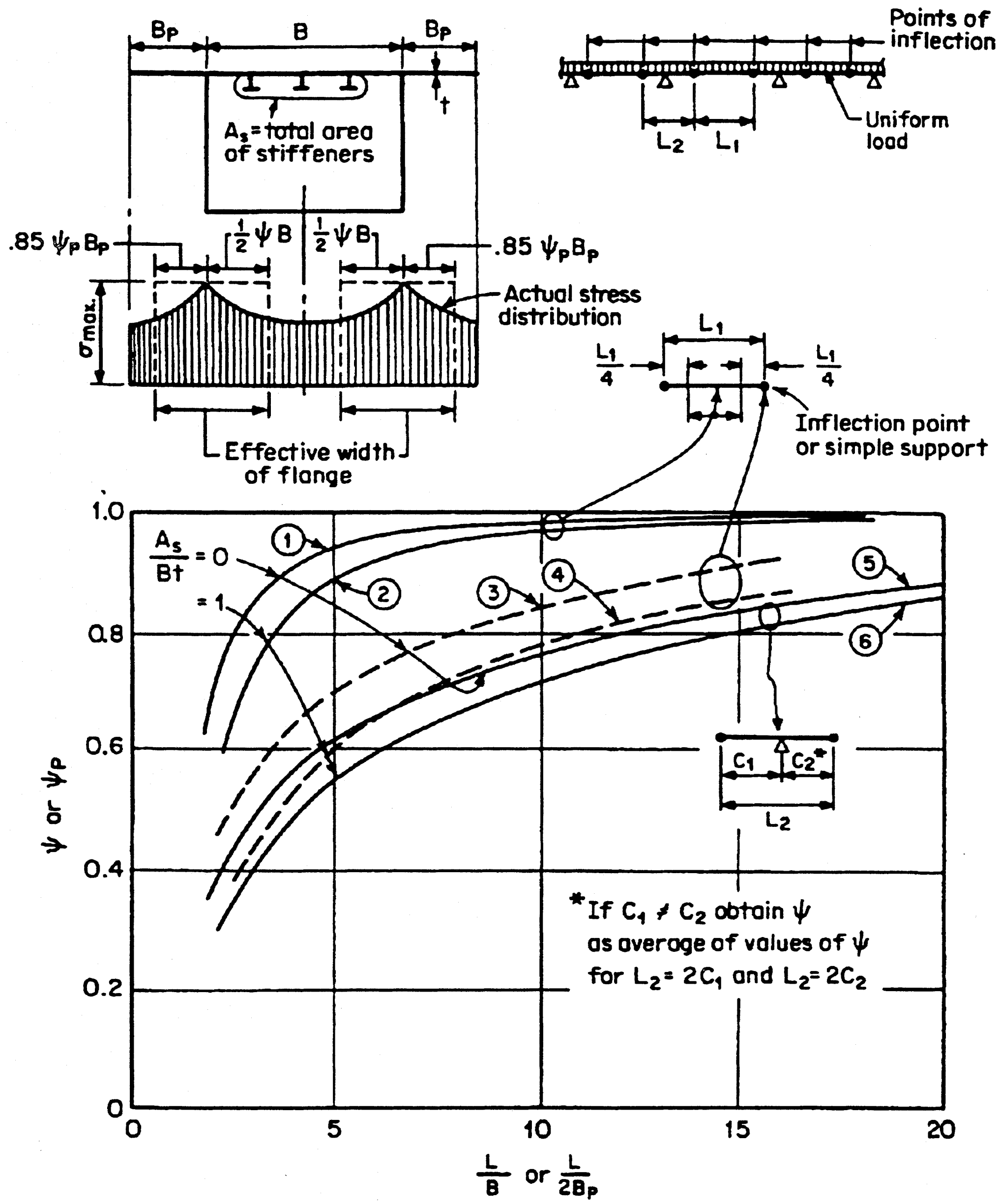


FIGURE 14.21 AASHTO effective deck width. (From American Association of State Highway and Transportation Officials, *LRFD Bridge Design Specifications*, Washington, D.C., 1994.)

14.4.5 Fatigue Considerations

Detailing of an orthotropic bridge is more involved because of all the numerous plates (see Figure 14.16 isometric). In-fill plates and complex geometric detailing make many orthotropic deck bridges one of a kind with unique details. The fatigue strength of the deck plate is very high, but thin deck plates may cause fatigue failures in other components. Fatigue of low-alloy deck plate of usual proportions subject to the AASHTO wheel loads is not considered a critical design factor. Reference [51] shows sketches of several European bridges with steel cracking caused by fatigue stress, and Ref. [52] shows how one bridge with wine-glass ribs was repaired. The original design

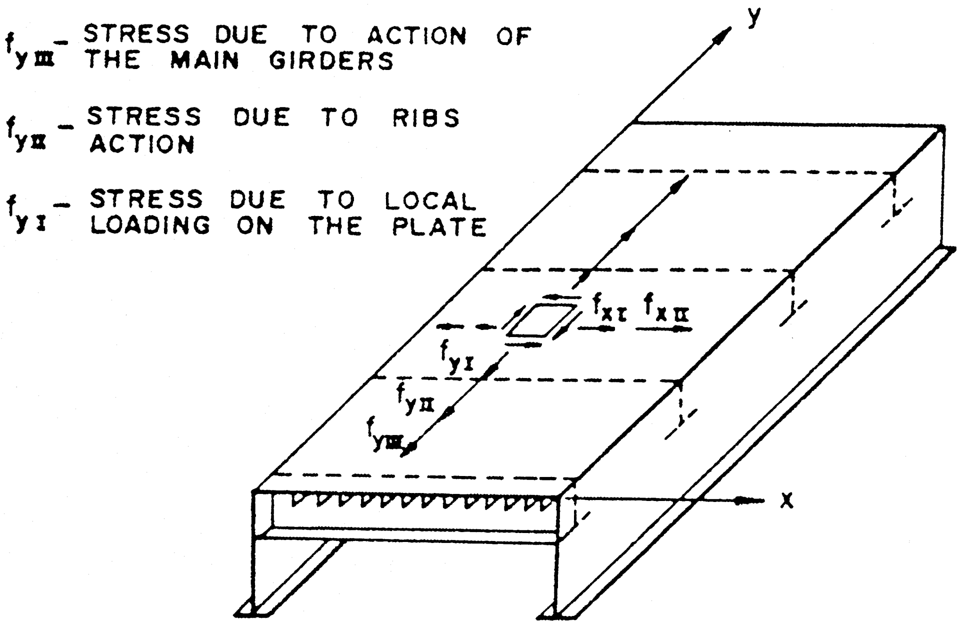


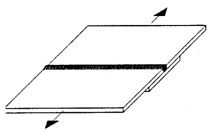
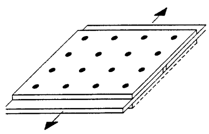
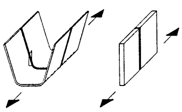
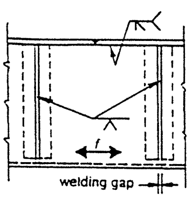
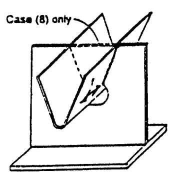
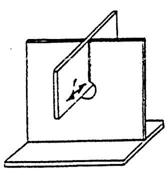
FIGURE 14.22 Resulting stresses in an orthotropic deck. (From Trotsky, M. S., *Orthotropic Steel Deck Bridges*, 2nd ed., JFL Arch Welding Foundation, Cleveland, OH, 1987. Courtesy of the James F. Lincoln Arc Welding Foundation.)

engineers did not fully understand long-term fatigue stress and CAFL (constant-amplitude fatigue life) issues. The AASHTO LRFD commentary [5] states: "Fatigue stress tests indicate that local out-of-plane flexural stress in the rib web at the junction with the deck plate should be minimized. One way to achieve this is to limit the stress in the rib web caused by the rotation of the rib–deck plate junction by making the rib welds relatively slender compared to the deck plate." Fatigue issues add to the complexity of orthotropic box-girder bridge design.

Excessive welding causes shrinkage of the weld metal which may cause additional locked-in stresses. In addition there are more components that are cut to fit between other components. If a component is cut too short this may also cause problems during welding. If additional filler welding is installed to span a larger incorrectly fabricated gap or root opening, then more weld width can increase the probability of shrinkage stresses. Detailing, imperfections in the alignment of a components, and quality control in fabrication have a direct result on the end product. Some designers prefer to let the fabricator select the welding processes (submerged arc welding, gas welding, etc.). Therefore, the welders and management of the fabrication plant are only required to achieve the strength and performance. Both AASHTO specifications require the use of 80% partial penetration welds of the ribs to deck plate (see Figure 14.3).

The AASHTO LRFD [5] maximum allowable stress limit for various details is shown in Table 14.7. The floor beam web cutouts around the rib were developed by the Germans. When ribs are designed to be continuous, it has been found to be best to make the rib go through the floor beam or diaphragm plate. Coping near the bottom of the rib and below greatly reduces the chances of fatigue cracking. There is still research going on about the actual shape of the holes. The cutout pattern per AASHTO is shown in Figure 14.23. However, it should be pointed out that in Figure 14.5 the German Railways has experimented with a different shape. AASHTO does not provide guidance on V-shaped ribs as shown in Figure 14.15. When components align on opposite sides of a plate, AASHTO Detail A must be followed for stress limits. Misaligned plates normally contribute to long-term fatigue failures. Full-scale fatigue testing of the Williamsburg Suspension Bridge has shown that code-acceptable details have a shorter fatigue life than what experts are able to develop with

TABLE 14.7 AASHTO Fatigue Detailing

Illustrative Example	Detail	Description of Condition	Detail Category
	Transverse or longitudinal deck plate splice or rib splice	1. Ceramic backing bar. Weld ground flush parallel to stress. 1. Ceramic backing bar. 3. Permanent backing bar. Backing bar fillet welds shall be continuous if outside of groove or may be intermittent if inside of groove.	B C D
	Bolted deck plate or rib splice	4. In unsymmetrical splices, effects of eccentricity shall be considered in calculating stress.	B
	Deck plate or rib splice Double groove welds	5. Plates of similar cross-section with welds ground flush. Weld run-off tabs shall be used and subsequently removed, plate edges to be ground flush in direction of stress. 6. The height of weld convexity shall not exceed 20% or weld width. Run-off tabs as for 5.	B
	Welded rib "window" field splice Single groove butt weld	7. Permanent backing bar—deck plate weld made with ceramic backing bar only. Welding gap > rib wall thickness f = axial stress range in bottom of rib	D
	Rib wall at rib/floorbeam intersection Fillet welds between rib and floorbeam web	8. Closed rib with internal diaphragm inside the rib or open rib. Welding gap > rib wall thickness f = axial stress range in rib wall	C
		9. Closed rib, no internal diaphragm inside of rib $f = f_1 = f_2$ f2 = local bending stress range in rib wall due to out-of-plane bending caused by rib–floorbeam interaction, obtained from a rational analysis	C

Source: American Association of State Highway and Transportation Officials, *AASHTO LRFD Bridge Design Specifications*,2nd ed. Washington, D.C., 1998. With permission.

proper funding, as shown in Figure 14.24. This solution was selected as the optimal detail [49]. Termination of a weld is a very common place for a fatigue crack to begin. A runoff tab plate allows the welder to terminate the weld in this piece of steel. The tab plate is cut off and tossed in the recycle bin. The source of a potential flaw is now no longer part of the final structure. The weld tab plate system was introduced to the orthotropic deck detail. This tab plate is cut off with the ½-in. radius as shown in Figure 14.24 after welding. This extra step was used on one test panel to compare against test panel without tab plates. These researchers believe that deck plates of 8 to 12 mm are too thin for more than 2 million cycles. For more-detailed discussion of fatigue, see Chapter 53.

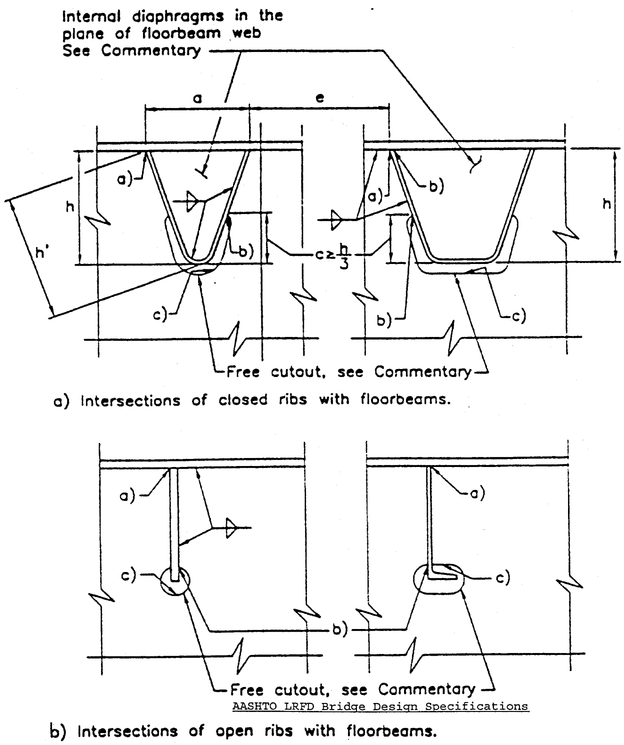


FIGURE 14.23 Detailing requirements for orthotropic deck. (From American Association of State Highway and Transportation Officials, *LRFD Bridge Design Specifications,* 2nd ed., Washington, D.C., 1998.)

14.4.6 Bridge Failures

Unfortunately, from 1969 to 1971 there were four steel box-girder collapses in four different countries, which caused the bridge engineering industry to reevaluate its different design code formulas and methods of erections. The Rhine River Bridge at Koblenz, Germany; the fourth Danube Bridge in Vienna, Austria; the Milford Haven Bridge in Wales, Great Britain; and the Yarra River Bridge in Australia were the four bridges that collapsed, making it a global bridge design issue [53-55]. It is a sobering thought to realize that 35 construction workers died at the Yarra River Bridge collapse [56,57]. This bridge was redesigned with an orthotropic steel deck to reduce its original dead weight. In Great Britain, a Board of Inquiry under the chairmanship of A. W. Merrison produced a list of new design rules and code details for fatigue issues. Extensive testing, research, and symposiums were held in Great Britain [3,4]. The British Institution of Civil Engineers [1979] held a symposium in London to have engineers from around the world share their experiences and ideas. It is very important to remember that codes are imperfect and the long-term fatigue details are still evolving and research continuing [58-64]. When the West Virginia bridge collapsed due to fracture critical failure of a single steel member, 50 individuals died. The FHWA responded by initiating a mandatory bridge maintenance program.

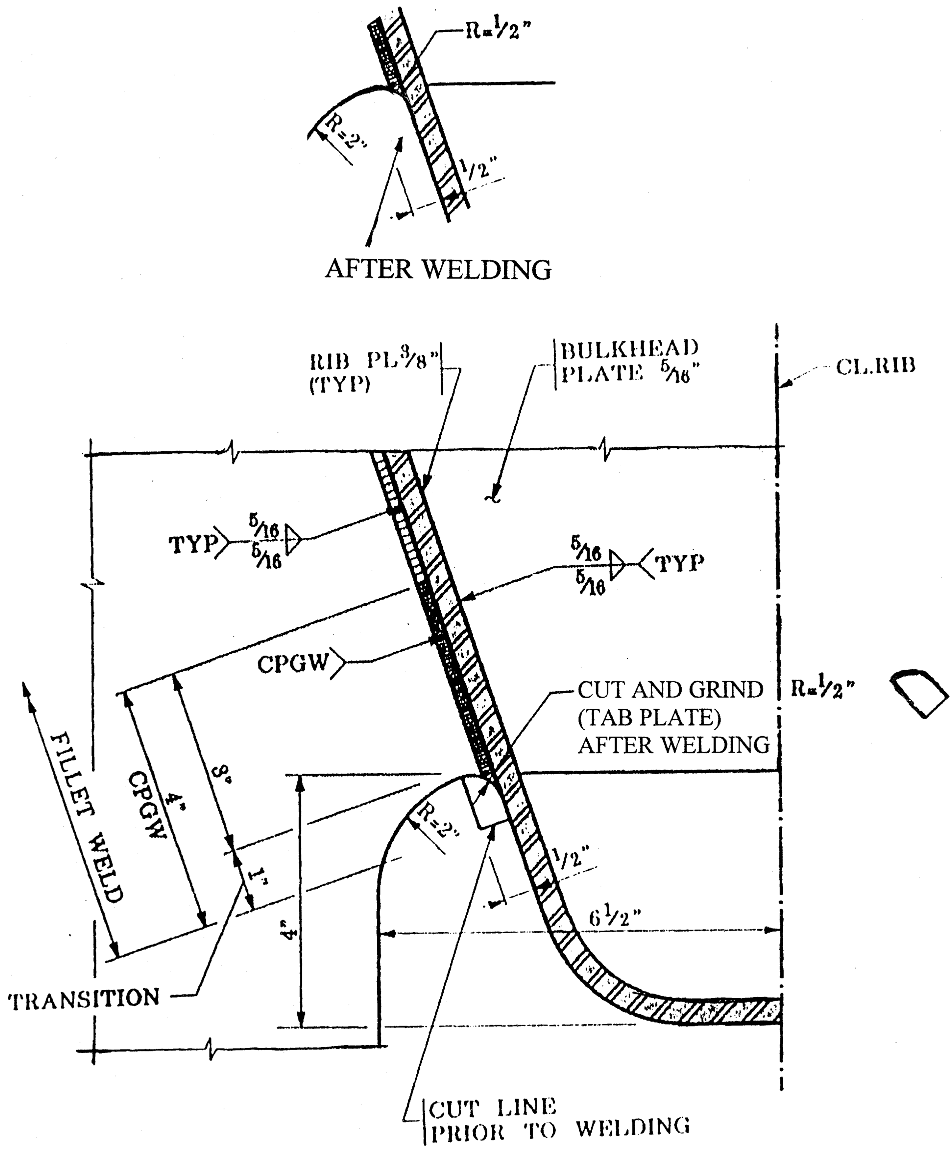


FIGURE 14.24 Fatigue-resistant detail Williamsburg Suspension Bridge. (From Khazen, D., *Civil Eng. ASCE*, June, 1998. With permission.)

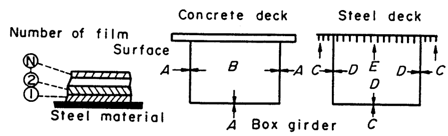
14.4.7 Corrosion Protection

The AASHTO LRFD commentary [5] states: " The interior of the closed ribs cannot be inspected and/or repaired. It is essential to hermetically seal them against the ingress of moisture and air." (see Figure 14.3 for a solution). Atmospheric corrosion of steel requires water and a continuous supply of fresh air. Abrasion will speed up the process.

The three different methods that can be used to protect corrosion for new bridges with orthotropic decks are painting, weathering steel, or dehumidification. Painting is the most common method (see Table 14.8 for one Japanese Standard) reference [64]. Weathering steel was invented by the steel industry to eliminate painting. Corrosion can continue if the rusted layer is abraded away exposing bare steel, thus allowing the continuation of corrosion or rusting. Therefore, some designers normally provide an extra thickness of steel with weathering steel in case abrasion occurs. The steel

TABLE 14.8 Japanese Painting Specifications for Steel Bridges

Surface	*N*	Painting in Workshop	Painting in Field	Remarks
A	6	1 Etching primer 2,3 Lead anticorrosive paint[a] 4 Phenolic MIO[b] paint	5,6 Long oil phethalic resin coating 5,6 Chlorinated rubber paint	General location Near sea and over sea
B	3	1 Etching primer 2,3 Epoxy coal-tar paint		
C	5	1 Inorganic zinc-rich primer 2 Organic zinc-rich paint 3 Epoxy MIO paint	4,5 Urethane resin paint 4,5 Chlorinated rubber paint	Painting before surfacing with asphalt Painting after surfacing
D	3	1 Inorganic zinc-rich primer 2,3 Epoxy coal-tar paint	No painting	Inner surface of box girder with steel deck
E	2	1 Inorganic zinc-rich primer 2 Organic zinc-rich paint		



[a] For example, lead suboxide anticorrosive paint (red lead).
[b] MIO = micaceous iron oxide
N = number of painting film
Source: Nakai, H. and Yoo, C.H., *Analysis and Design of Curved Steel Bridges*, MacGraw-Hill, New York, 1968. With permission.

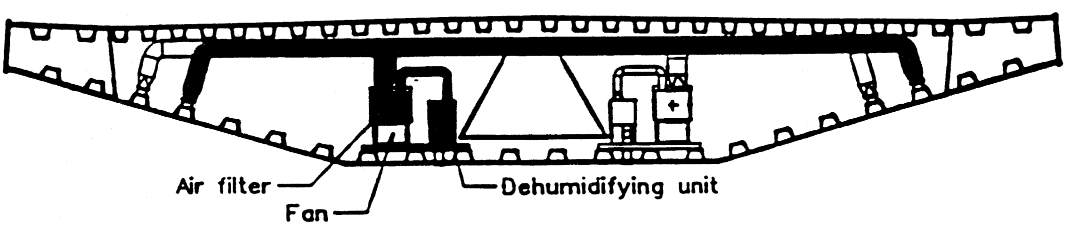


FIGURE 14.25 Dehumidification plant. (Courtesy of Monberg and Thorsen.)

towers for the Luling Bridge are also fabricated of weathering steel; for a color photo of its aesthetic appearance, see Ref. [42]. Another type of steel has been developed for use for contact with salt water or mariner steel.

Inspecting bridges with small access holes is facilitated with an electric-driven inspection cart; the individual rides inside long shallow orthotropic superstructures looking for fatigue cracks or corrosion [65]. The Normandie Bridge in France utilizes a mechanical dehumidification system (see Figure 14.25). The segments were shop-fabricated and then were full butt field-welded at the bridge site. This produced full structural continuity plus completely sealed the bridge superstructure. Exterior access doors for maintenance personnel have gasket seals to eliminate infiltration of air into the superstructure. Dehumidification is popular in European bridges (see Figure 14.11). An air supply system or aspirators are needed for maintenance personnel during bridge inspections. Some engineers are skeptical that dehumidification can actually remove all the moisture. Proponents of dehumidification point out that an air supply system or aspirators are needed for maintenance personnel during future maintenance painting inside the confined space of the box girder. State-of-the-art European technology went into the detailing of this bridge. Rather than paint the interior of the bridge, a mechanical dehumidification process is utilized to prevent corrosion of the interior of the superstructure. A central bridge maintenance walkway gallery was created in a triangular opening. The sections were prefabricated in an assembly-line process, then full butt-welded together at the bridge site.

Hot-dip galvanizing may be utilized for corrosion protection of smaller field-bolted orthotropic deck panels, which are used on deck retrofits and temporary bridges. The fabricated steel component is dipped into a molten zone in a tub. The maximum size of a component is limited by the tub size. A disadvantage is that warping of fabricated components can occur from the hot zinc. Hot-dip galvanizing can be cost-effective for limited-size components, especially near highly corrosive salt water. Drain holes would be needed in any closed ribs to allow molten zinc to drip out. Field welding of hot-dipped galvanizing creates toxic fumes.

14.3.8 Wearing Surface

An orthotropic steel plate deck must be paved with a wearing surface to provide a durable and skid-resistant surface for vehicular traffic. AASHTO [5] states: "The wearing surface should be regarded as an integral part of the total orthotropic deck system and shall be specified to be bonded to the top of the deck plate. For the purpose of designing the wearing surface itself, and its adhesion to the deck plate, the wearing surface shall be assumed to be composite with the deck plate, regardless of whether or not the deck plate is designed on that basis." AREA (American Railroad Engineers Association) specifications should be used for railroad bridges as shown in Figure 14.5. Some materials used in wearing surfaces are proprietary or patented. Aesthetic issues may control for pedestrian bridges or sidewalk areas as shown in Figure 14.16. The surfacing for vehicular traffic performs several functions:

- Exhibits high skid resistance throughout the life of the surfacing;
- Provides a smooth riding surface for the comfort of the drivers using the bridge;
- Helps waterproof the steel deck;
- Resists cracking, delaminating, and displacing for a long service life with low maintenance cost and disruption to traffic for repairs;
- A stiff and well bonded surfacing can also provide some reduction in fatigue stresses in the steel deck, ribs, and welds by dynamic composite action with the deck plate. In addition, surfacing with thickness of about 50 mm provides some distribution of the wheel loads and damping of the steel plate.

The number of wheel load applications during the life of the surfacing can be enormous as each passage of a truck wheel stresses the surfacing and the steel deck. For example, assume a bridge carries 10,000 vehicles a day with 5% being three-axle trucks. If half of these are loaded trucks, the annual full-load applications are about a quarter of a million or 4.5 million for a 20-year service life. A busy bridge carrying a high percentage of trucks can exceed this figure. The wheel load applications for the design of a surfacing should be calculated for the specific bridge site. Vehicle tires also wear and polish the aggregates on the surface of the surfacing causing a reduction in skid resistance. Hard, durable, and polish-resistant aggregates should be selected, preferably with small asperities projecting from the surface. The asperities help to increase the dry skid resistance and also provide a rough surface for improved bonding with the aggregate within the matrix of the surfacing. A long life is required for the surfacing of a heavily traveled bridge as replacement of the surfacing, whether for fatigue cracking, debonding, or for lack of skid resistance, is costly. The surfacing is costly to remove and relay and is also costly to the user as it disrupts and delays traffic.

Treated timber plank wearing surfaces have been used for a few bridges, as shown in Figure 14.7 and described in Section (14.3). Timber planks are not watertight so the steel deck must be painted or other corrosion protection provided. Welded threaded studs or economy head bolts can hold the planks in place. There are three basic classifications of surfacing for orthotropic decks:

1. Thin surfacings from about 4 to 8 mm thick;
2. Surfacings composed of mastics with binders of asphalt or polymer resins usually laid from about 12 to 75 mm thick;
3. Surfacings composed of concertos with binders of asphalt or polymer resins usually laid from about 18 to 60 mm thick.

All three types require a bond coat on the steel deck to hold the surfacing in place against the forces of braking truck wheels and to provide intimate contact with the steel deck for dynamic composite action for each passage of a wheel. The strength of the bond must last for the life of the surfacing.

Thin surfacings are usually laid by flooding the deck with a thermosetting polymer resin and broadcasting a hard aggregate that is locked into the binder to provide skid resistance. Thin surfacings are not appropriate for decks carrying high truck traffic. The repetitive wheel loads will wear away the aggregate exposing the slick resin surface producing a very low skid resistance. The repetitive wheel loads can also wear away the resin and expose the steel deck. However, a thin surfacing can be used as a temporary wearing course for bridge deck replacements. The new orthotropic deck panels can be paved in the shop and installed a panel at a time during replacement of an existing concrete deck. Traffic can run on the temporary surfacing up to 2 years, as was done for the deck of the Golden Gate Bridge. After all the panels are installed, the permanent pavement can be laid as a continuous operation for a smooth riding surface.

Mastics are usually a mixture of aggregates and binder of asphalt, polymer-modified asphalt, or polymer resins. The binder is proportioned in excess of that required to fill the air voids. The strength of mastic surfacing is dependent on the strength of the binder material rather than on the interlock of the aggregates. During placement, the mix is flowed onto the deck and leveled, usually by a vibrating screed, before the binder sets. Hard, durable stones can be broadcast over the surface to improve skid resistance. The high-binder-content polymer resins may cause high shrinkage strains and high bond stress on the deck plate. Gussasphalt (German word for poured asphalt) is a mastic using a low penetration asphalt as a binder heated to a high temperature of about 200°C and applied usually by poring and leveling to a thickness of about 75 mm or more by hand labor. It has been used apparently successfully in Europe and Japan.

Concrete is usually a mixture of polymer resins or polymer-extended asphalt with aggregates with some air voids up to 4% remaining unfilled. Concrete surfacing for steel decks should not be confused with portland cement concrete, which is not suitable for steel-deck surfacing. The strength of concrete surfacings is dependent on both the strength of the interlock of the aggregates and the strength of the binder material. They require compaction usually by a steel roller or vibrating screed. They have the advantage of being mixed, placed, and compacted using conventional paving equipment. Ordinary or modified asphalt concrete surfacings have low first cost, but have not given long, trouble-free service life. If the binder is a thermoset-resin-extended asphalt, such as epoxy asphalt, the cost is increased somewhat but the added strength imparted by the thermoset resin greatly improves the performance and life of the surfacing.

Failure of the wearing surface is common, but the Hayward-San Mateo Bridge still has its original wearing surface after 30 years [24]. Prior to the construction of this bridge a small test panel (truck weight scale) was used to test the durability of the wearing surface under actual truck traffic. The San Diego–Coronado wearing surface was replaced after 25 years. The Caltrans test bridge used two types of wearing surfaces (thin and thick) [13]. The failure of a wearing surface can be caused by the deck plate thickness (stiffness); poor construction practices; installation quality control; bridge deck splice details (bolt heads and splice plates); and/or the temperature range (freeze–thaw action) plus humidity conditions expected at the bridge site. Due to all these factors wearing surfaces can fail very quickly, and few last over 20 years. Flexible orthotropic decks can cause a stiffer wearing surface to pop off. This is one reason AASHTO wants the designer to think of the wearing surface as a composite material that can "structurally" fail, when not deflecting in synchronization with the deck. For case history details on the wearing surfaces of many bridges, the reader can refer to References [1, 2]. The Miller Sweeney bascule bridge's wearing surface failed by creep while the movable span was in the vertical position.

A wearing surface of a sacrificial material is placed on top of the deck since vehicles' wheels cause abrasion. Asphalt and epoxy concrete are the most commonly used materials for wearing surfaces. Timber wearing surfaces have been used as described in Section 14.3. For details on the many possible types of wearing surfaces, the reader may refer to Ref. [1–4]. The proposed test system has been developed for Caltrans [44].

14.3.9 Future Developments

The second generation of orthotropic deck bridges will be better based on lessons learned from the first group of bridges built. Many orthotropic bridges were never built and have remained only a dream. The most interesting is the Ruck-A-Chucky cable-stayed bridge designed by Prof. T. Y. Lin to be built across the flooded American River Canyon after the completion of the Auburn Dam near Sacramento, California [66]. This horseshoe-shaped superstructure was planned to be supported by cable stays anchored into the sloping canyon rock walls. Therefore, no towers would be necessary. Scale models were built for wind tunnel and earthquake shake-table testing. The 1977 orthotropic design featured trapezoidal ribs for a five-cell, 14.61-m-wide superstructure. The bridge plans used a 396-m clear span on a 457-m radius for two lanes of traffic and an equestrian trail.

A promising concept, patent pending to a Redding, California firm, is to have the bridge deck comprise only nested trapezoidal ribs, which are welded together. The three-sided "rectangular" ribs are placed at 90° to the driving surface, so that the sides of the rib become the top and bottom flanges. The result is to achieve a 12-m span that is about 275 mm deep or wide rib is required. This system would compete economically against concrete slab bridges and has been marketed at bridge conventions. Another unique rib system for floating steel bridges or pontoons is the "biserrated-rib" developed by Dr. Arsham Amirikian, consultant to Naval Facilities Engineering Command, Department of the Navy. Portions of the sides of the trapezoidal ribs are removed in a repetitive scallop pattern to reduce the weight of the rib. The structural strength is almost identical to a full rib. The disadvantage would be increased (double) surface area exposed to corrosion.

A panelized orthotropic deck system was developed and built for trapezoidal ribs for a temporary detour bridge in New York [67]. A cross section of this bridge built in 1991 is shown in Figure 14.26a. This system is the engineering evolution of a previous concept for open ribs. A system was developed and engineered for a steel grating company utilizing open ribs for a bridge system [68]. A cross section of the bridge proposed in 1961 is shown in Figure 14.26b. This system would use similar materials stored in the steel grating company warehouse, and allow them to market another product.

The future of orthotropic deck cost reduction lies in the standardization of ribs and details by AASHTO or the steel industry. Such standardization has led to the popularization of precast prestressed concrete girders.

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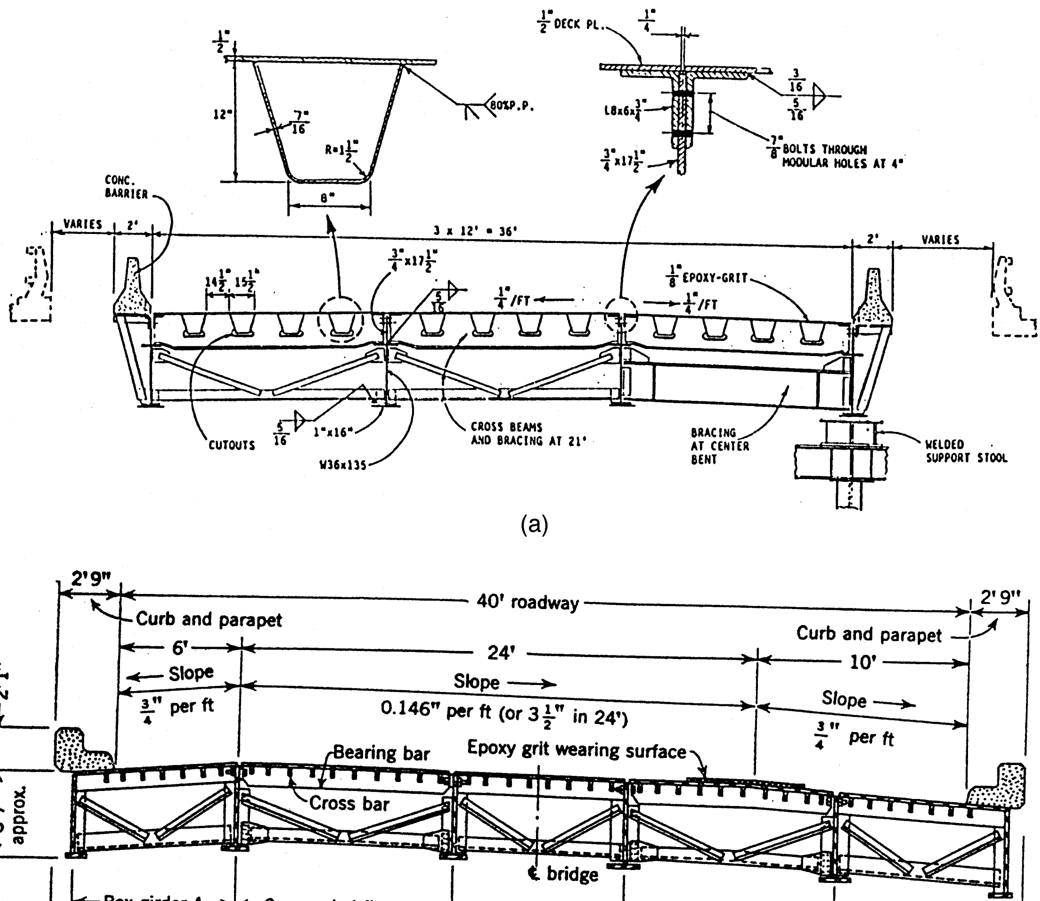

FIGURE 14.26 Prefabricated orthotropic deck panel bridge. (a) A temporary bridge. (From Wolchuk, R., *Welding Innovation Q.*, IX(2), 19, 1992. Courtesy of the James F. Lincoln Arc Welding Foundation.) (b) A short-span bridge. (From Chang, J.C.L., *Civil Eng. ASCE*, Dec. 1961. With permission.)

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