

Guide for Orthotropic Steel Deck Level 1 Design



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of Transportation

**Federal Highway
Administration**

Foreword

Over the course of many decades and across the globe, orthotropic steel deck (OSD) bridges used around the world have proven to be safe, redundant, efficient, and durable. For this reason, efforts have increased in recent years to inform engineers in the United States about successful OSD applications to encourage broader implementation of them. However, OSD applications in the United States have typically had higher relative costs associated with the complex analytical procedures and labor intensive fabrication design details usually employed.

In 2012, the Federal Highway Administration (FHWA) published the Manual for Design, Construction, and Maintenance of Orthotropic Steel Deck Bridges (Connor et al. 2012), which outlined three levels of design. In this Manual, each level is accompanied by a varying level of complexity, with Level 3 being the most complex and Level 1 being the least complex. Level 1 design is “by little or no structural analysis, but by selection of details that are verified to have adequate resistance by experimental testing (new or previous)” and is effectively proven through full-scale qualification testing or historical in-service performance. This Design Guide simplifies the level of complexity suggested to design, fabricate, and construct OSD bridges through Level 1 design.

This Design Guide provides general information and details for typical OSD bridges with either an open- or closed-rib system that satisfy American Association of State and Highway Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications (AASHTO 2017)¹. Also included is information on the details of the deck plate, wearing surface, and floorbeam/diaphragm of an OSD. The details presented are the result of an extensive review of in-service bridges that have a proven record of successful performance. Short summaries of the performance of several in-service bridges are provided to further emphasize key points and to provide additional information to engineers, designers, owners, and fabricators.



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1. AASHTO. 2017. *AASHTO LRFD-8. AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications*. 8th Ed. (2017). American Association of State Highway and Transportation Officials, Washington, DC. (Incorporated by Reference at 23 CFR 625.4(d)(1)(v).)

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16. Abstract This Guide provides bridge engineers and owners with general information and typical details to help standardize orthotropic steel deck (OSD) bridge design/fabrication to make it more competitive. This document does not intend to set a national standard but to help inform the effort through reduced parametric variations. OSD bridges can be either closed- or open-rib systems, and this Guide begins with background information regarding OSD bridge design. General considerations with respect to OSD bridges are discussed, followed by specific instructions for closed- and open-rib systems including rib geometry, size, and fabrication methods. Suggestions for deck plate selection are provided followed by a discussion of wearing surface types and selection considerations. Lastly, suggestions for floorbeam/diaphragm design are provided. Throughout the document, short summaries on the performance of several in-service OSD bridges are provided.			
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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH				
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
AREA				
in ²	square inches	645.2	square millimeters	mm ²
ft ²	square feet	0.093	square meters	m ²
yd ²	square yard	0.836	square meters	m ²
ac	acres	0.405	hectares	ha
mi ²	square miles	2.59	square kilometers	km ²
VOLUME				
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft ³	cubic feet	0.028	cubic meters	m ³
yd ³	cubic yards	0.765	cubic meters	m ³
NOTE: volumes greater than 1000 L shall be shown in m ³				
MASS				
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
TEMPERATURE (exact degrees)				
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
ILLUMINATION				
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m ²	cd/m ²
FORCE and PRESSURE or STRESS				
lbf	poundforce	4.45	newtons	N
lbf/in ²	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
AREA				
mm ²	square millimeters	0.0016	square inches	in ²
m ²	square meters	10.764	square feet	ft ²
m ²	square meters	1.195	square yards	yd ²
ha	hectares	2.47	acres	ac
km ²	square kilometers	0.386	square miles	mi ²
VOLUME				
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m ³	cubic meters	35.314	cubic feet	ft ³
m ³	cubic meters	1.307	cubic yards	yd ³
MASS				
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
TEMPERATURE (exact degrees)				
°C	Celsius	1.8C+32	Fahrenheit	°F
ILLUMINATION				
lx	lux	0.0929	foot-candles	fc
cd/m ²	candela/m ²	0.2919	foot-Lamberts	fl
FORCE and PRESSURE or STRESS				
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in ²

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

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1. Introduction

Orthotropic steel deck (OSD) bridges have been used successfully around the world since the 1940s (Connor et al. 2012). An OSD bridge deck system is a durable and redundant system that is lightweight compared to other deck systems. The system has been used in new design and rehabilitation scenarios alike. However, the wider application of OSDs for commonplace bridges was affected by the complexity of design, sophisticated analysis needs, high fabrication costs, and owner-mandated experimental fatigue testing (Connor et al. 2012).

The Federal Highway Administration (FHWA) published the Manual for Design, Construction, and Maintenance of Orthotropic Steel Deck Bridges (OSD Manual for short) in 2012 (Connor et al. 2012). Three levels of design were introduced in both the OSD Manual and the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications (LRFD Specifications) (AASHTO 2012) in 2012.

These include Level 1 design, which makes use of proven OSD solutions without the need for analysis,

Level 2 design, which makes use of simplified one-dimensional (1D) or two-dimensional (2D) analysis methods calibrated to experimental results, and Level 3 design, which makes use of refined three-dimensional (3D) analysis.

The 2012 FHWA OSD Manual provides resources to researchers and engineers. The intention of this new Design Guide is to further develop details of Level 1 design and encourage the implementation of OSD systems.

Over the past decade, the FHWA recognized the need for a more accessible process for designing and analyzing OSDs. As a result, the FHWA sponsored a research study on successful OSDs from which proven designs could be adopted and adapted for use on commonplace bridges, which aligns with the Level 1 design specified in the LRFD Specifications (AASHTO 2017)¹. At the same time, the AASHTO and National Steel Bridge Alliance (NSBA) have engaged in ongoing efforts to collaborate on addressing the manufacturability of OSDs, the complexity of design, and the evolution of complex detailing.

This Design Guide provides typical details similar to those for other

bridge deck types. The designer is given typical open-rib and closed-rib options based on experiences in real bridge applications.

Through the development of Level 1 design with typical details, the use of OSDs may become more common and fabrication costs may decrease as fabricators work with a small number of designs to establish economically viable fabrication processes.

Each chapter in this guide aims to provide information for a complete OSD system, whether it be open- or closed-rib. Key points are provided to highlight some of the benefits and drawbacks of one system over the other. Several short summaries are included to draw attention to the performance of in-service OSD bridges.

Options for typical closed- and open-rib OSD systems, including the deck plate, are provided for rib span length (i.e., the floorbeam or diaphragm spacing) and rib spacing. This guide's purpose is not to set a binding requirement or standard, but to encourage the efforts by designers and fabricators toward simpler modular design.

1. AASHTO. 2017. *AASHTO LRFD-8. AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications*. 8th Ed. (2017). American Association of State Highway and Transportation Officials, Washington, DC. (Incorporated by Reference at 23 CFR 625.4(d)(1)(v).)

Glossary of Terms

Blow-through: Excessive, undesirable penetration of the weld application leading to holes in the weld root and welded surfaces.

Crossbeam: Alternative name for floorbeam (see Floorbeam).

Deck plate: The top plate of an orthotropic deck that supports the wearing surface and directly supports the wheel loads.

Diaphragm: A diaphragm is a transverse component that is similar to a floorbeam but is typically characterized by not having a bottom flange or being seated atop a sub-floorbeam in the primary bridge framing (see Floorbeam). A diaphragm is generally smaller and does not necessarily connect to a main structural member.

Extended cut-out: The cut-out is a stress-relieving cut made in the floorbeam (diaphragm) web to alleviate the out-of-plane stresses induced by in-plane end rotations of the rib due to applied loads on the deck and/or to avoid welding to the bottom of the rib where longitudinal stresses are highly concentrated.

Floorbeam: A floorbeam is a transverse component that provides support to the ribs and transfers loads to the primary girders.

Girder: A main load-carrying member that runs longitudinally with the orthotropic deck ribs and the bridge. In orthotropic decks, girders are integrated with the deck plate and other components of the orthotropic system.

Melt-through: In orthotropic deck welding, an unintended but harmless condition where additional weld material penetrates, especially at the back side of the rib-to-deck weld, and forms additional reinforcing on the opposite side of the weld application.

Orthotropic: A word derived from two terms. The system of ribs and floorbeams are orthogonal and their elastic properties are different or anisotropic with respect to the deck: thus, orthogonal-anisotropic becomes orthotropic.

Orthotropic steel deck: A system where a steel deck plate is stiffened by longitudinal ribs and transverse floorbeams (or diaphragms) in which the ribs and floorbeams are orthogonal and their elastic properties are anisotropic with respect to the deck directly supporting live loads.

Redeck (Redecking): The rehabilitation of an existing bridge by removal and replacement of the existing deck with a new deck or deck system.

Rib: Longitudinal members that can be open (e.g., angle or plate rib) or closed (e.g., U-shaped or trapezoidal) and used to stiffen the steel deck plate.

Rib span: The span length of a longitudinal rib member between supporting floorbeams (or diaphragms).

Wearing surface: A top layer placed on the deck plate to provide a skid resistant surface with good ride quality to provide corrosion protection to the deck plate, accommodate deck plate irregularities, and potentially offer additional stiffness to the deck plate resulting in reduced stress levels.

Illustrative Glossary

Figure 1 and Figure 2 are a generic plan view and cross-sectional view of an OSD, respectively. Each figure is provided to give additional clarity to certain terms used throughout this guide. Note that all of the plan views and detail drawings in this guide were developed by the researchers for this project unless otherwise credited under the individual figures.

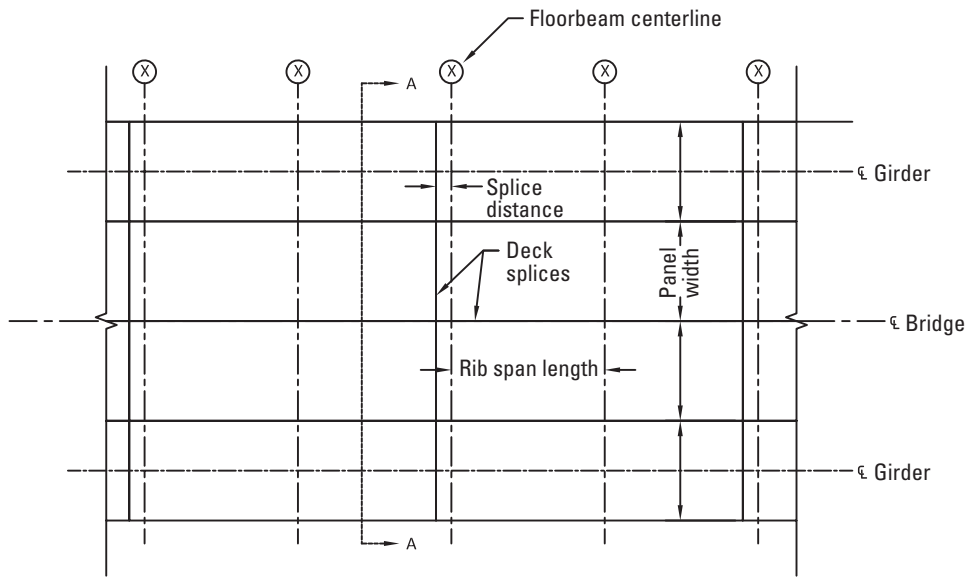


Figure 1. OSD plan view

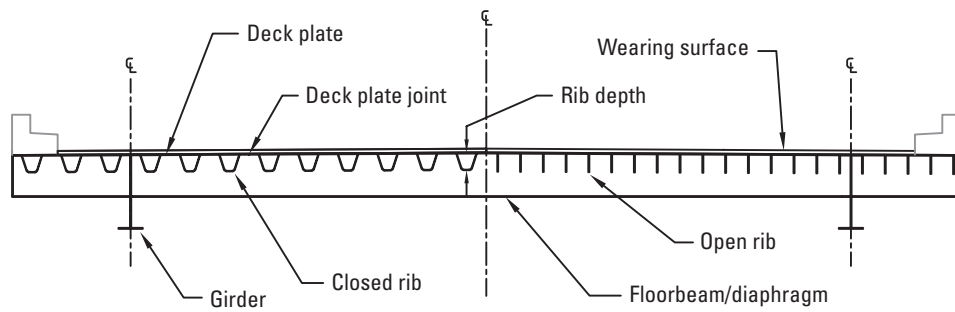


Figure 2. Detailed Section A-A view for Figure 1 OSD plan view with rib options

2. “Big Picture” Considerations

KEY POINTS

- Optimization of material use for OSDs in short- and medium-span bridges is a secondary matter in comparison to retrofit or long-span projects where weight minimization may be more critical
- OSDs are highly redundant, which alleviates safety concerns due to potential fatigue cracking or corrosion loss
- Maintenance of OSDs is similar to that for other steel bridges
- Automation is not a requirement for quality fabrication of OSDs

The typical rib designs and details provided in this guide are based on historically successful in-service performance of real bridges and balanced input from designers and manufacturers, in accordance with LRFD Specifications (AASHTO 2017)¹.

OSDs have been used in new construction, retrofit, and rehabilitation of bridges around the world—primarily for signature and long-span bridges. Those designs were often refined to minimize overall weight and depth to conform to geometrical restrictions. For OSDs of typical shorter span bridges, reducing weight and depth are only two of the considerations toward the goal of life-cycle cost optimization. The details suggested in this guide reflect an effort to design the panels for broader application, which may result in a lesser optimized design. However, the initial cost of additional material is nominal in comparison to the cost incurred by potential unexpected and/or ongoing serviceability and maintenance issues.

Some additional suggestions are offered with respect to the bridge geometry to further simplify the use of OSDs. Effort should be made to simplify deck plate geometry with the highway design. These are not requirements, but ways to remove unnecessary complexity:

- Maintain tangent geometry and ensure piers and abutments are orthogonal

- Maintain uniform cross slope or place the crown at a longitudinal weld location
- Set the ribs and floorbeams (diaphragms) normal to the cross slope and profile grade line

OSDs are highly redundant with respect to connections and load carrying members. Although it is expected that an OSD would deteriorate at the same rate as other steel bridge components, the inherent redundancies help alleviate potential concerns that fatigue cracking or section corrosion loss will become an issue. This is especially true when adopting design details of proven in-service bridges where the long-term performance is well known.

Maintenance of OSDs is the same as other routine maintenance. The OSD is inspected for fatigue and section loss due to corrosion. Maintenance of paint or other protective coatings is expected to be completed over the lifetime of the deck at intervals consistent with maintenance plans for a typical steel superstructure.

New technologies and methods are emerging to expedite rib fabrication, particularly for closed ribs. These technologies should be able to roll ribs of various shapes and sizes and help in the standardization of these shapes. The closed-rib shapes described in this guide are anticipated to be formed using a brake, although the guide is equally valid for ribs formed using the newer technologies.

1. AASHTO. 2017. *AASHTO LRFD-8. AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications*. 8th Ed. (2017). American Association of State Highway and Transportation Officials, Washington, DC. (Incorporated by Reference at 23 CFR 625.4(d)(1)(v).)

For those unfamiliar with them, OSDs might appear to require complex fabrication methods and even automated (robotic) fabrication, but high-quality fabrication can be readily achieved in OSD fabrication using traditional methods. With advancements in fabrication and investments in equipment and skills by fabricators, the costs of OSDs are continually being reduced.

Connection complexity has also been a deterrent for designers and a challenge for fabricators. For this reason, efforts have been made to adopt simple connection details in lieu of minimizing material use

alone. The designers should avoid over-prescribing the fabrication means and methods, leaving room for the fabricators to select viable cost-effective solutions.

To ensure field construction of OSD bridges does not encounter unnecessary difficulties, the fabricator should pay special attention to the alignment and geometry during shop trial assembly. The suggestion for panel flatness tolerance is $\frac{1}{8}$ in. over 10 ft, consistent with normal shop tolerance for flatness of other steel bridge members. This tolerance should also result in a suitable driving surface. However, in the

end, fabricators should produce panels such that they suitably fit-up in the field, and the check of this condition is the suitability fit of the deck joints.

While no special tolerance is suggested for panel length, width, and squareness, the fit of the panel joints controls these geometries. That is, for proper panel dimensions, the geometry and welding preparation of field-assembled deck joints should meet groove weld alignment tolerance per the AASHTO/American Welding Society (AWS) D1.5 Bridge Welding Code (AASHTO/AWS 2016)¹.

1. AASHTO/AWS. 2016. *AASHTO/AWS D1.5M/D1.5:2015_AMD1. Bridge Welding Code (BWC), 7th Ed., Amendment (Dec. 12, 2016)*. American Association of State Highway and Transportation Officials and American Welding Society, Washington, DC. (Incorporated by Reference at 23 CFR 625.4(d)(2)(iii).)

3. Closed-Rib System

KEY POINTS

- Closed-rib configurations have been shown to be an effective OSD solution
- Trapezoidal ribs are simpler to fabricate than U-shaped ribs
- A relaxation from some past practices in the minimum weld penetration for rib-to-deck plate partial joint penetration (PJP) welds has been established by AASHTO
- Fabricators should be given the flexibility to prepare the ribs as necessary to facilitate welding goals

Advantages

Closed-rib OSDs have proven to be an effective system in many in-service bridges (Connor et al. 2012). The inherent flexural and torsional rigidity of the closed-rib system provides some benefits over the open-rib system. Loads are more efficiently distributed transversely across the deck and, of the closed-rib shapes that are in existence, the trapezoidal-shaped ribs are simpler to fabricate and perform equally to other closed shapes. For this reason, the shape provided in the following sections is trapezoidal.

The connection between the rib and deck plate is completed using PJP groove welds. Past weld specifications have made this connection difficult to complete. Research has shown that the minimum weld penetration can be reduced and the allowance for melt-through can be accepted (Sim and Uang 2008) which can improve the constructability and cost of the connection.

Challenges

The connection between the rib and deck plate is only observable from outside the rib, which limits the ability to ensure penetration during fabrication and, later, if deemed necessary, during in-service inspections. It is important for fabricators to develop an effective procedure to ensure a sound weld with the correct minimum penetration.

Field splices between deck segments are not as easily completed when compared to an open-rib system. Tighter tolerances during fabrication and erection are needed to ensure proper fit. Rib splices are often completed using bolts that are accessed through handholes in the bottom of the rib, which adds to the overall fabrication challenges.

The connection of the rib to the floorbeam is also more complex when compared to an open-rib system. Larger cut-outs need to be accounted for in the floorbeam design, often resulting in greater structural web depth.

Weld Considerations

Given a minimum penetration for rib-to-deck welds, fabricators target a penetration such that, given the variation in penetration they expect, penetration is always above the minimum. Penetration can be as great as 100 percent provided there are not soundness concerns with the weld. Some mild melt-through is usual for welds that reach 100 percent, and melt-through is not deleterious. It is very unusual for blow-through to occur for properly designed welding procedures.

Ribs are tightly fit to deck plates to minimize melt-through and facilitate a sound weld. There is not a prescribed maximum fit-up gap as long as sound welds are achieved, but typically, rib-to-deck fit-up gap is no more than 0.020 in.

Gaps that are too large result in a concave profile and possible blow-through. The fabricator should be allowed to match the need by their own determination for weld joint design (e.g., angle of bevel, size of landing). The fabricator should be allowed to determine the number of tacks, tack size, tack frequency, and spacing needed to achieve the fit-up of the rib to deck, although an excessive amount of tack welding may undermine the quality of final welds. Typically, the approval of tack weld details comes from the Engineer of Record.

All other welds in this OSD design are fillet welds, subject to fillet weld non-destructive examination (NDE) requirements. The fatigue performance of these welds is not a major concern if preparation and welding are executed per AASHTO/AWS D1.5 (AASHTO/AWS 2016)¹.

Cut-Out at Floorbeam

Fully fitted connections between the closed rib and floorbeam are appealing for simplicity of fabrication. Even so, engineers have been hesitant to use a fully fitted connection, believing it would be too stiff and high stresses under

rib end rotation would result. To better understand this connection and investigate ways to reduce the stresses, extensive research has been completed over the past decades.

Most recently, it was observed in laboratory fatigue tests (Saunders et al. 2019) that, when the fitted connection was subjected to very high loads (25 percent greater than the AASHTO Fatigue I combination total factored tandem axle load for OSDs) and a large number of cycles (approximately 2 million), unexpected root cracks occurred. The same test completed under typical stresses under factored AASHTO fatigue loads would not result in the same weld cracks. Furthermore, lower stress levels are expected with deeper floor beams as are used in new and Level 1 typical detail OSDs. To date, no cases of similar root cracking occurring in an in-service fitted connection are known.

To reduce the potential for high stresses and weld fatigue cracks, several versions of details employing a cutout in the floorbeam/diaphragm near the bottom of the closed rib were developed and

tested (Saunders et al. 2019). Early forms of the detail needed intensive fabrication and subsequent forms were developed to facilitate fabrication. These connections were primarily directed at redecking applications where the structure depth was limited and the benefit of a deeper floorbeam/diaphragm could not be realized, therefore limiting the use of the fitted connection detail for redecking.

Closed-Rib Geometry

Typical rib dimensions for a closed-rib system are presented in Figure 3 and Table 1. Two rib size options are shown with maximum span lengths (and floorbeam spacing, as detailed in Figure 4) indicated for each option and a typical rib spacing of 2 ft 2 in. maintained.

The performance of in-service OSDs with closed ribs of this size extending beyond this length have not been investigated and are therefore not presented here. The rib geometry is held constant to promote OSD fabrication of Level 1 design. Figure 5 through Figure 8 show additional details.

1. AASHTO/AWS. 2016. *AASHTO/AWS D1.5M/D1.5:2015_AMD1. Bridge Welding Code (BWC), 7th Ed., Amendment (Dec. 12, 2016)*. American Association of State Highway and Transportation Officials and American Welding Society, Washington, DC. (Incorporated by Reference at 23 CFR 625.4(d)(2)(iii).)

Closed-Rib Details

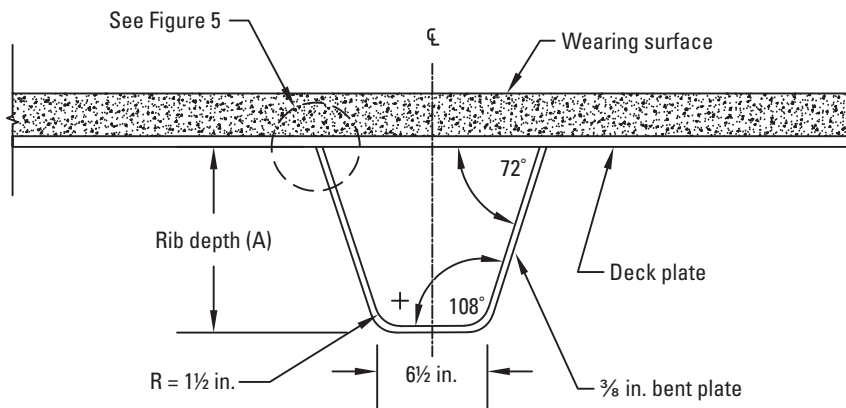


Figure 3. Typical closed-rib detail

Table 1. Typical closed-rib detail specifications

Option	Rib Depth (A)	Max Span Length*	Deck Plate Thickness
#1	10½ in.	15 ft	5⁄8 in.
#2	14 in.	18 ft	¾ in.

* See previous Figure 1

Figure 3 Commentary

The typical closed rib maintains a consistent bend angle and width at the bottom of the rib for standardization. The rib depth (A) is provided in Table 1. The width at the top of the rib is a function of the rib depth.

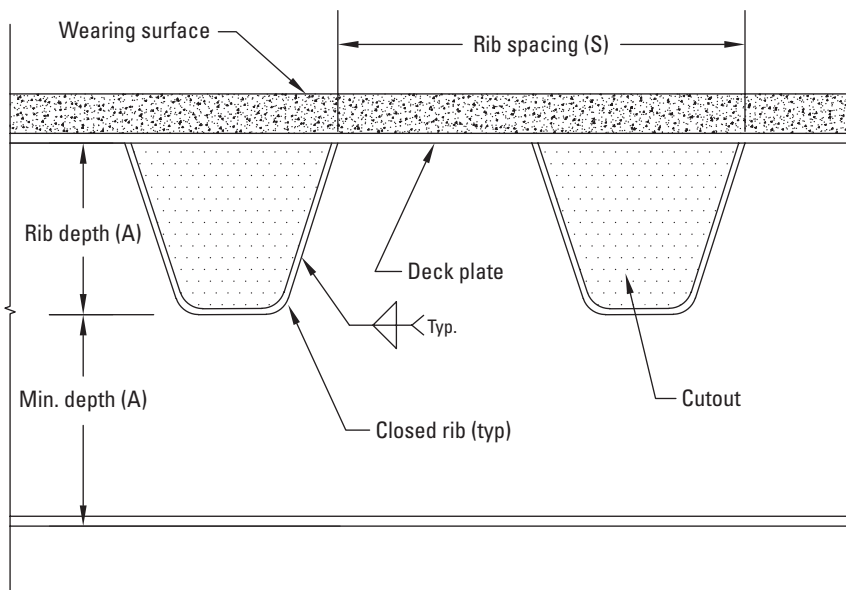


Figure 4. Closed rib to floorbeam detail

Figure 4 Commentary

The floorbeam is cut to match the rib contour and is welded as shown. The depth of the floorbeam (or diaphragm) below the ribs should be equal to or greater than the depth of the rib (A) to maintain proper flexibility.

The spacing of the rib (S) is 2 ft 2 in. but can be nominally reduced to accommodate overall bridge geometry.

Closed-Rib Details (continued)

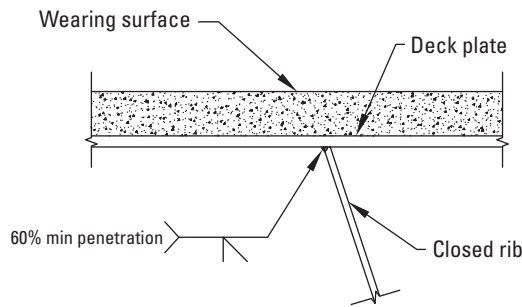


Figure 5. Closed rib to deck connection detail

Figure 5 Commentary

The closed-rib weld to the deck is a PJP weld as shown. Both laboratory and full fabrication panels have shown that the weld indicated produces consistent results eliminating weld blow-through and provides reasonable penetration tolerance. Joint preparation should be left to the fabricator.

Use the current AASHTO LRFD design, which specifies a minimum weld penetration of 60 percent (AASHTO 2020)¹.

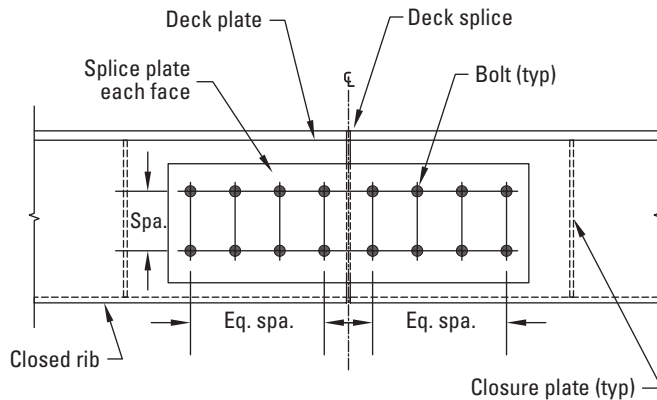


Figure 6. Closed-rib field splice detail

Figure 6 Commentary

The bolted splice represented should be designed to transfer the rib forces through the splice location. Typically, the splices are located at the inflection points of moment diagrams under dead and uniform live loads so that the design moments are very small. A maximum number of rows of bolts that can fit in the rib walls is typically used to minimize the offsets of the two bolt groups.

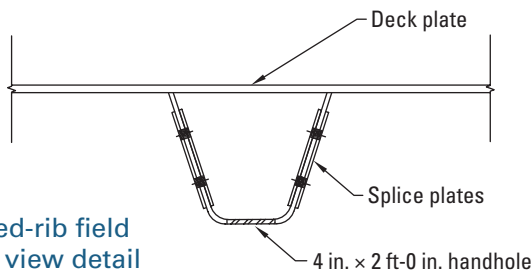


Figure 7. Closed-rib field splice section view detail

Figure 7 Commentary

The rib splice section view shows splice plates on each side of the trapezoidal rib plate and the bolt locations.

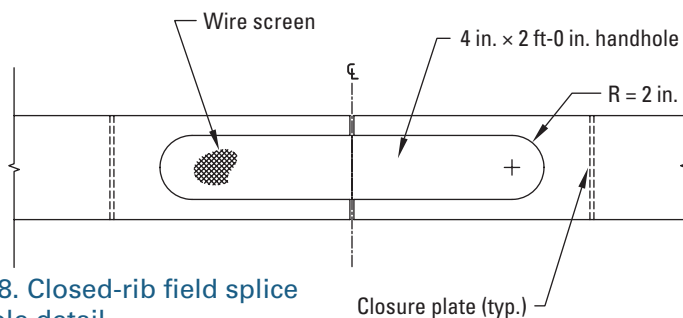


Figure 8. Closed-rib field splice handhole detail

Figure 8 Commentary

The handhole provides access to the inside of the rib plate where the bolted connection is completed. The removable wire screen prevents bird access and the potential for nesting.

1. AASHTO. 2020. AASHTO LRFD-9. AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications. 9th Ed. (2020). American Association of State Highway and Transportation Officials, Washington, DC. (Approved for use under the authority of 23 CFR 625.3(f)(2); see FHWA memorandum, [Approval of the use of structural design standards](#) (Apr. 11, 2022).)

Danziger Bridge – Louisiana



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Figure 9. The Danziger Bridge in New Orleans

The Danziger Bridge (Figure 9) spans the Industrial Canal in New Orleans, Louisiana. It is a vertical lift bridge on US 90, constructed in the mid-1980s using OSD panels (Figure 10). Upon completion, the bridge became the widest lift bridge in the world.

The Danziger Bridge OSD was functioning well according to the report resulting from the 2018 inspection, which was contracted by the Louisiana Department of Transportation and Development (DOTD), although notes included wearing surface cracking and patching, which may be indicative of an overly flexible rib/deck/overlay combined system. It should also be noted that the deck plate is 1/2 in. thick, which can reduce the overall rigidity of the bridge and is less than

the 5/8 in. thick plate observed with other OSD bridges.

The wearing surface cracking was likely caused by lower overall rigidity and might be partially related to the wearing surface material and application. The inspection record does not show that this is a symptom of possible rib-to-deck joint cracking. Any of the following may help in solving the wearing surface cracking problems: thicker deck plate, deeper ribs, or stiffer overlay.

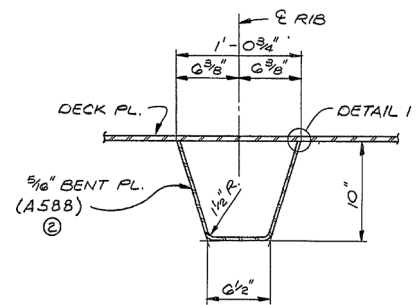
The ribs are closed and folded trapezoidal in shape (Figure 11) rather than U-shaped, which simplifies the fabrication between the two types.

The rib-to-deck connection was completed using a partial penetration weld with 80 percent minimum penetration. This connection

is commonly observed among many closed-rib OSD projects. No indications were provided in the inspection report suggesting this connection was not performing well.

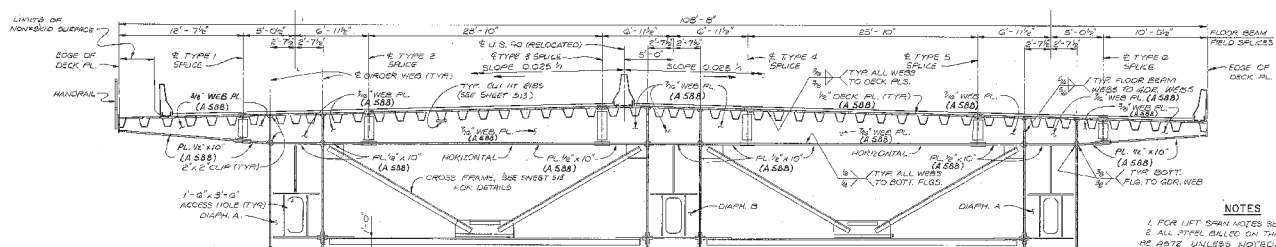
The rib splice locations incorporated a handhole cut-out at the bottom of the rib. This handhole is used to access the bolts to fasten the splice plates to the rib. Once the connection is completed, the handhole is covered with a screen to deter birds, primarily, from nesting in the ribs. The inspection report noted some screens were missing and, birds, in fact, have used the handholes as a nesting area.

Although the rib-to-web extended cut-out detail is different from the closed-rib design in this guide, past performance makes the Danziger Bridge an example for trapezoidal-shaped closed-rib design with its rib geometry and splice details.



Louisiana DOTD

Figure 11. Danziger Bridge typical trapezoidal rib detail



Louisiana DOTD

Figure 10. Danziger Bridge plan detail

4. Open-Rib System

KEY POINTS

- There are inherent simplicities to the fabrication of open-rib systems
- Fillet welds between the rib and the deck plate simplify fabrication compared to PJP groove welds
- Open-rib connections at the floorbeam are easier to accomplish than closed connections
- Field splicing between deck segments is performed with relative ease

Advantages

Open-rib deck systems are primarily fabricated using flat plates, although other shapes such as bulb-T and angles have been used in the past. For the purpose of simplifying fabrication, only flat plate shapes are shown in this guide. In comparison to closed-shaped ribs, several advantages exist with respect to fabrication.

Welds between the deck plate and the ribs are fillet welds, which reduce the need for weld preparation and simplify fabrication overall. In addition, access to these weld areas is not limited.

At the floorbeam/diaphragm, the connection of the continuous ribs is also performed with relative ease. This type of connection is straightforward given the fabricator can accurately cut floorbeam/diaphragm webs using computer numerical control (CNC), which can be used later as a template to help position the ribs and achieve fit.

Bolted rib splices are straightforward connections given each of the assembled parts is easily accessible.

Generally, the cost when considering fabrication, labor, and quality assurance/quality control (QA/QC) is less for open-rib systems than for an equivalent closed-rib system.

Challenges

Compared to closed-rib systems, open-rib systems have less torsional rigidity, which results in less efficient transverse load distribution. To overcome this disadvantage, a closer rib spacing is needed, which can double the number of welds. The depth of the ribs is typically greater than that for closed ribs of equal spans. Overall, the amount of steel required for typical open-rib deck systems in this guide is greater than that for an equivalent closed-rib system, although this loses significance when considering the overall cost of each system.

Open-Rib Geometry

Figure 12 through Figure 15 and Table 2 show the rib dimensions and details for an open-rib system. Two rib size options are shown with maximum span lengths (floorbeam spacing) indicated for each option and a typical rib spacing of 1 ft 3 in. The rib geometry is held constant.

The performance of in-service OSDs with open ribs of this size extending beyond this length have not been investigated and are therefore not presented here.

Open-Rib Details

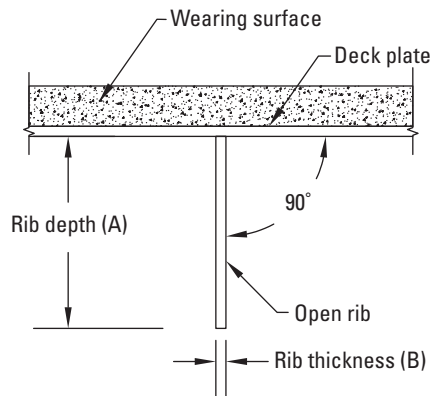


Figure 12. Typical open-rib detail

Table 2. Typical open-rib detail specifications

Option	Rib Depth (A)	Rib Thickness (B)	Max Span Length	Deck Plate Thickness
#1	10 in.	$\frac{5}{8}$ in.	10 ft	$\frac{5}{8}$ in.
#2	12 in.	$\frac{3}{4}$ in.	15 ft	$\frac{3}{4}$ in.

Figure 12 Commentary

The open rib uses a flat plate. The rib depth (A) and rib thickness (B) are provided in Table 2.

A $\frac{5}{8}$ in. minimum rib thickness for any open rib is suggested by AASHTO (2020)¹ and in this guide. In unique situations where a thinner rib is used, a minimum thickness of $\frac{1}{2}$ in. should be used to avoid bridging between the fillet welds on either side of the rib.

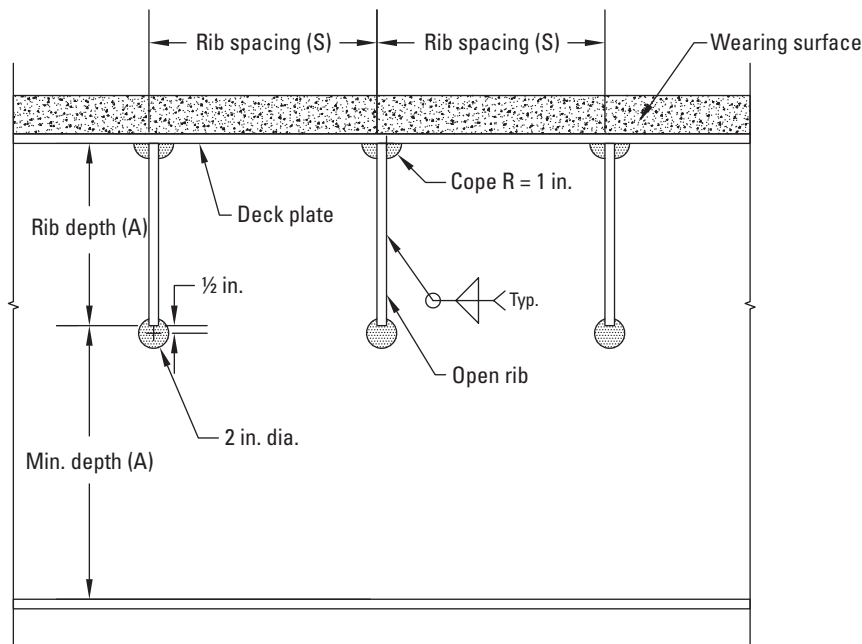


Figure 13. Open rib to floorbeam detail

Figure 13 Commentary

The floorbeam is cut to match the rib and is welded as shown. The depth of the floorbeam (or diaphragm) below the ribs needs to be equal to or greater than the depth of the rib (A) to maintain proper flexibility.

The spacing of the rib (S) is 1 ft 3 in. but can be nominally reduced to accommodate overall bridge geometry. Ribs spaced too closely may cause weld access problems.

Normal AASHTO/AWS (2016)² D1.5 tolerances should be used for fit-up of the rib to the floor beam and to the deck plate.

The designer should specify the fillet weld termination detail at the keyhole, either wrapping around or stopping short, with balanced consideration of fatigue resistance and fabrication access.

1. AASHTO. 2020. AASHTO LRFD-9. AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications. 9th Ed. (2020). American Association of State Highway and Transportation Officials, Washington, DC. (Approved for use under the authority of 23 CFR 625.3(f)(2); see FHWA memorandum, [Approval of the use of structural design standards](#) (Apr. 11, 2022).)

2. AASHTO/AWS. 2016. AASHTO/AWS D1.5M/D1.5:2015_AMD1. Bridge Welding Code (BWC), 7th Ed., Amendment (Dec. 12, 2016). American Association of State Highway and Transportation Officials and American Welding Society, Washington, DC. (Incorporated by Reference at 23 CFR 625.4(d)(2)(iii).)

Open-Rib Details (continued)

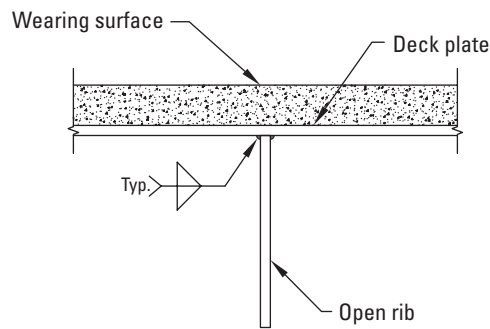


Figure 14. Open rib to deck weld detail

Figure 14 Commentary

Normal AASHTO/AWS (2016)¹ D1.5 tolerances should be used for fit-up of the rib to the deck plate. As with other bridge fillet welds, if fit-up gaps are larger than $\frac{1}{16}$ in., fillet weld sizes can be increased to make up for this gap as allowed by AASHTO/AWS D1.5.

Fillet welds are common on bridges and shops are accustomed to the practices needed to satisfy AASHTO/AWS D1.5 tolerances for fit-up to achieve suitable welds.

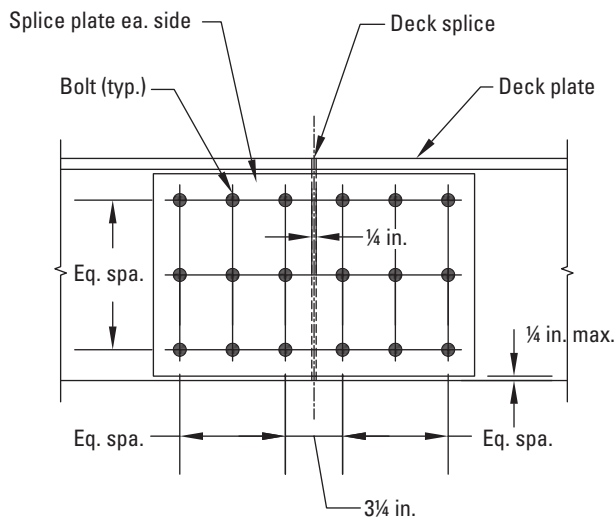


Figure 15. Open-rib field splice detail

Figure 15 Commentary

The bolted splice shown is designed to transfer the rib forces through the splice location. Typically, the splices are located at the inflection points of moment diagrams under dead and uniform live loads so that the design moments are very small. A maximum number of rows of bolts that can fit in the rib walls is typically used to minimize the offsets of the two bolt groups.

1. AASHTO/AWS. 2016. AASHTO/AWS D1.5M/D1.5:2015_AMD1. Bridge Welding Code (BWC), 7th Ed., Amendment (Dec. 12, 2016). American Association of State Highway and Transportation Officials and American Welding Society, Washington, DC. (Incorporated by Reference at 23 CFR 625.4(d)(2)(iii).)

San Mateo-Hayward Bridge – California



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Figure 16. San Mateo-Hayward Bridge

The San Mateo-Hayward Bridge (Figure 16) is a seven-mile-long bridge that carries California State Route (SR) 92 across the San Francisco Bay between Forster City and Hayward, California.

The bridge was built in the 1960s and was an early large-scale use of an OSD system. The western portion of the bridge includes 17 high-rise spans of all-steel construction, and the bridge consists of an OSD supported on two parallel box girders.

The OSD system on the bridge consists of 8 to 12 in. deep by $\frac{5}{8}$ to $\frac{3}{4}$ in. thick open ribs welded to a $\frac{5}{8}$ in. or $\frac{3}{4}$ in. thick deck plate. The deck plate and ribs are supported on I-shaped floorbeams, which are suspended between the box girders (or cantilevered from the box girders for the outer lanes) at approximately 10 ft-5 in. spacing (Figure 17).

The floorbeams, which work with the OSD system to provide the structure of the floor system, are about 2 ft deep. The ribs continuously pass through the floorbeams/diaphragms. For modern-day fabrication, this type of connection is straightforward given the fabricator can accurately cut diaphragm webs using CNC, including the rounded part of the cut-out (Figure 18).

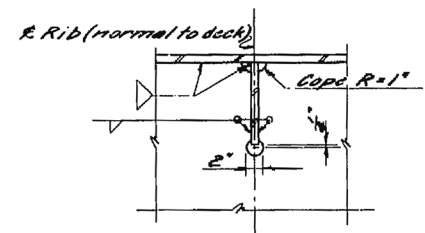
The diaphragms, then, coupled with a realistic tolerance to account for variation and impart straightness and flatness, can be used as a template to help position the ribs and achieve fit.

The connection of ribs to the deck plate can be achieved using a fillet weld, which is advantageous from a fabrication standpoint. The rib splices are simply constructed using splice plates and bolts (Figure 19) with the connection completed in the field.



Ric Maggenti, Caltrans, used with permission

Figure 17. Underside of San Mateo-Hayward Bridge deck



Caltrans, used with permission

Figure 18. Detail of rib to diaphragm connection



Ric Maggenti, Caltrans, used with permission

Figure 19. Splice plates and bolts

5. Deck Plate

KEY POINTS

- A minimum thickness of $\frac{5}{8}$ in. has been effectively demonstrated with in-service bridges
- Bolted splices are more easily erected in the field than welded splices
- Bolted splices need a thicker wearing surface
- Welded splices are a suitable option and have been used more often
- Wearing surface suitability should be discussed with product manufacturers

Varying deck plate thicknesses have been used for bridges in service. The in-service performance of deck plates and their associated wearing surfaces have proven to be problematic at times when the deck plate thickness is less than $\frac{5}{8}$ in. Deck plates less than this thickness do not provide the rigidity needed to limit the deformation and stresses in the wearing surface, which can lead to cracking and further degradation. Accordingly, the minimum thickness of the Level 1 typical OSD plate detail is $\frac{5}{8}$ in. (AASHTO 2020)¹.

Thicker deck plates ($\frac{3}{4}$ in.) have been used, which can help improve the fatigue performance of the OSD and also enhance the performance of the wearing surface. The design options provided in this guide use a minimum $\frac{5}{8}$ in. thick deck plate and a typical rib spacing for shorter span ribs and a $\frac{3}{4}$ in. thick deck plate and a typical rib spacing for longer span ribs.

The type of wearing surface selected may also contribute to the decision for a deck plate thicker than the minimum prescribed. A discussion with the wearing surface manufacturer should help identify the sensitivity to increased or decreased OSD flexibility. Furthermore, owners of bridges where a particular wearing surface has been used can be a valuable resource and could be consulted in the wearing surface decision.

The projects reviewed during the course of this guide development

have used both thin and thick overlays, and cases of successful performance were found for both options. In preparing this document, the research team learned that some owners having experience with both types tend to favor the thick type for its longevity, while others like the thinner types for their light weight and considerable advancement in recent years.

The engineer should be mindful that the wearing surface selection contributes to the overall deck self-weight. Furthermore, the climate conditions vary from location to location and should also be a consideration. A thicker wearing surface can contribute to the overall stiffness of the deck and reduce live-load induced stresses, although this is rarely considered during design.

Historically, welded deck splices have been used more commonly than bolted splices, and they have performed well. However, several advantages to using bolted deck splices, including field erection and fit-up, can lead to cost savings. Bolt heads at bolted splice locations need to be accommodated in the design by using countersunk bolts or by choosing a thicker wearing surface. Waterproofing of bolted deck splices can also be a key consideration. Typical bolted deck splice details are provided in Figure 20 and Figure 21 for closed-rib systems and Figure 24 and Figure 25 for open-rib systems.

1. AASHTO. 2020. *AASHTO LRFD-9. AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications*. 9th Ed. (2020). American Association of State Highway and Transportation Officials, Washington, DC. (Approved for use under the authority of 23 CFR 625.3(f)(2); see FHWA memorandum, *Approval of the use of structural design standards* (Apr. 11, 2022).)

Welded deck splices can be chosen with no inherent disadvantage to structural performance. The option to use either connection method (longitudinal bolted splices or transverse welded splices) or a combination of both is available to the engineer. Erection procedures and desired wearing surface should be considered when making the selection. Typical welded deck splice details are provided in Figure 22 and Figure 23 for closed-rib systems and Figure 26 and Figure 27 for open-rib systems.

A suggested tolerance for deck joint alignment is $\frac{1}{4}$ in. in an unclamped condition, so that, when clamps are used, the AASHTO/AWS D1.5 Bridge Welding Code alignment tolerance of $\frac{1}{8}$ in. is satisfied (AASHTO/AWS 2016)¹. In the field, clamps can be used on either side of the joint to bring the plates together. Despite this suggestion, the greater goal should not be disregarded for fabricators to produce panels such that they suitably fit-up in the field, whether that is less than or greater than $\frac{1}{4}$ in.

With respect to field joint backing of welded joints, the following suggestions are offered:

- Longitudinal backing should be removed
- Transverse backing can be left in place

- Non-steel backing should be allowed
- Mixed welding processes, such as flux cored arc welding (FCAW) in the root and submerged-arc welding (SAW) for fill passes, should be allowed

Deck joints need to be complete joint penetration (CJP) groove welds, and a common way to complete these welds is to use backing and complete the weld entirely from the top side. Steel backing is common because it facilitates clamping and is relatively easy to weld compared to other backing. Ceramic backing and copper backing for bridge structures are currently uncommon in the United States due to lack of successful application experiences.

Removing steel backing improves fatigue resistance. However, transverse backing can remain in place due to access limitations and the fact that transverse deck joint locations are typically chosen to be away from zones subjected to high tensile stress.

Removal of backing is a challenging operation and needs remedial work to provide the proper surface profile. To facilitate backing removal, copper backing or ceramic backing may be used. However, some remedial work to the joint is likely still needed.

Another option that avoids backing entirely is to put an initial weld pass on the underside of the joint in the overhead welding position, and then, on the top side of the joint, back gouge to the root, and then, complete the weld from the top side. This option may be suitable for longitudinal deck joints but may not be suitable for transverse joints if the presence of ribs limits access to the underside of the joint.

It is not unusual to mix welding processes in a welded joint. This is allowed by the AASHTO/AWS D1.5 Bridge Welding Code (AASHTO/AWS 2016)¹. Given the relatively tight confines of the groove weld root and the overall large size of the groove weld, some contractors prefer to make the root passes with FCAW or gas metal arc welding (GMAW) and use SAW for the fill passes and cap passes. Although AASHTO/AWS D1.5 allows the mixed welding process, a few states/owners limit the use of GMAW by specification.

1. AASHTO/AWS. 2016. AASHTO/AWS D1.5M/D1.5:2015_AMD1. Bridge Welding Code (BWC), 7th Ed., Amendment (Dec. 12, 2016). American Association of State Highway and Transportation Officials and American Welding Society, Washington, DC. (Incorporated by Reference at 23 CFR 625.4(d)(2)(iii).)

Deck Splice Details

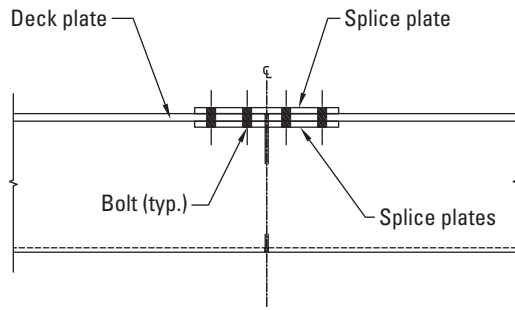


Figure 20. Closed-rib bolted transverse deck field splice detail

Figure 20 Commentary

Bolted deck splices need a thicker wearing surface to protect splice plates and bolt heads above the deck plate surface.

Web splice not shown (see Figure 6 and Figure 7 for details).

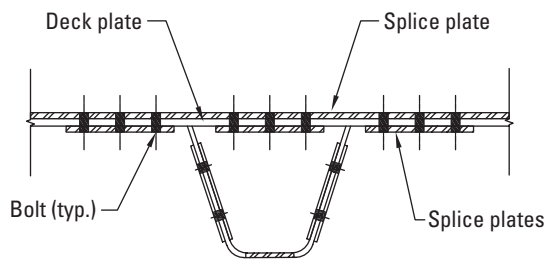


Figure 21. Closed-rib bolted transverse deck field splice section view detail

Figure 21 Commentary

Bolted deck splices need a thicker wearing surface to protect splice plates and bolt heads above the deck plate surface.

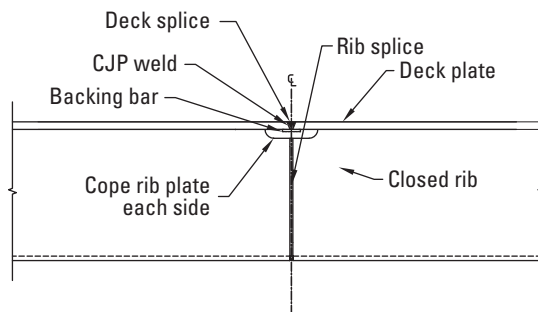


Figure 22. Closed-rib deck welded transverse deck field splice detail

Figure 22 Commentary

The deck splice consists of a CJP weld using a backing bar.

The transverse backing bar can be left in place.

Rib splice not shown (see previous Figure 6 and Figure 7 for details).

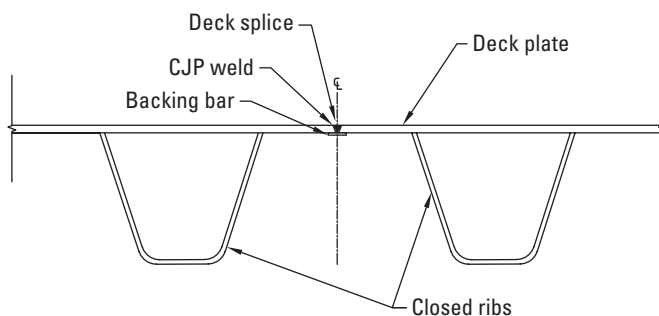


Figure 23. Closed-rib field-welded longitudinal deck splice section view detail

Figure 23 Commentary

The longitudinal backing bar should be removed for good fatigue performance and the splice positioned away from the primary wheel paths when possible.

Deck Splice Details (continued)

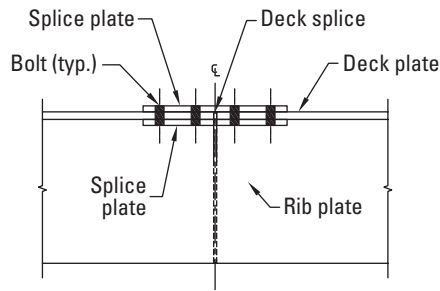


Figure 24. Open-rib bolted deck transverse field splice detail

Figure 24 Commentary

Bolted deck splices need a thicker wearing surface to protect splice plates and bolt heads above the deck plate surface.

Rib plate splice not shown (see previous Figure 15 for details).

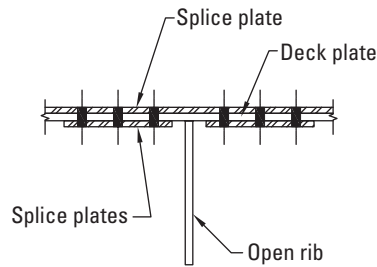


Figure 25. Open-rib bolted deck transverse field splice section view detail

Figure 25 Commentary

Bolted deck splices need a thicker wearing surface to protect splice plates and bolt heads above the deck plate surface.

Rib plate splice not shown (see previous Figure 15 for details).

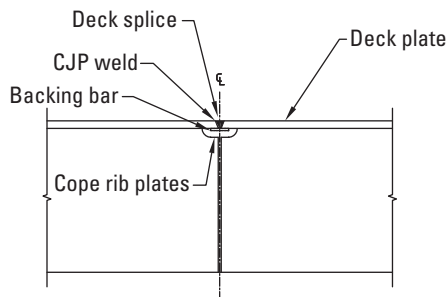


Figure 26. Open-rib welded deck transverse field splice detail

Figure 26 Commentary

The deck splice consists of a CJP weld using a backing bar.

The transverse backing bar can be left in place.

Rib plate field splice is not shown (see previous Figure 15 for details).

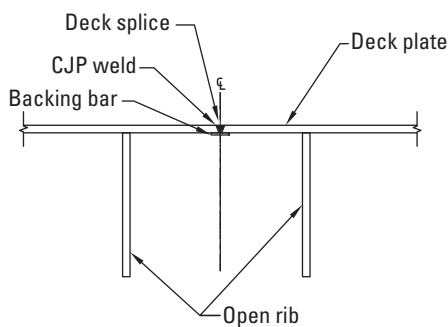


Figure 27. Open-rib welded deck longitudinal field splice section view detail

Figure 27 Commentary

The longitudinal backing bar should be removed for good fatigue performance and the splice positioned away from the primary wheel paths when possible.

Ben Franklin Bridge – New Jersey/Pennsylvania



Beyond My Ken, license at https://commons.wikimedia.org/wiki/File:2012_Ben_Franklin_Bridge_and_Race_Street_Pier.jpg

Figure 28. Ben Franklin Bridge over the Delaware River

The Ben Franklin Bridge (Figure 28) is a suspension bridge carrying I-676 over the Delaware River between Camden, New Jersey and Philadelphia, Pennsylvania.

The bridge consists of a 1,750 ft main span, two 716 ft side spans, and multiple truss and girder approach spans. The bridge was completed and opened to traffic in 1926 and is owned, operated, and maintained by the Delaware River Port Authority (DRPA). In the mid-1980s, the suspended, truss, and girder spans of the bridge were re-decked with an open-rib OSD system.

The OSD system on the suspension spans of the bridge consists of open bulb-shaped ribs with a maximum spacing of 1 ft. 3½ in. welded to a 5⁄8 in. thick deck plate (Figure 29 and Figure 30).

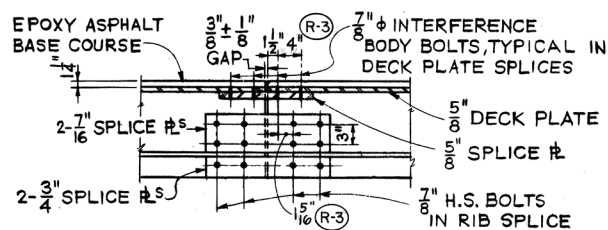
The ribs are approximately 13 in. deep and ½ in. thick and include a 1½ in. thick by 3½ in. deep bulb at the bottom. The rib-to-deck plate connection was completed using a double-sided ¼ in. fillet weld.

The 2020 inspection report completed for the DRPA by a private consulting firm and its subcontractors indicated the OSD is in overall good condition. The deck plate was spliced using bolted and welded connections and was considered to be in generally good condition. In some isolated areas, deteriorated/peeling paint, loose or missing nuts, missing/broken bolts,

and minor to moderate surface corrosion was observed.

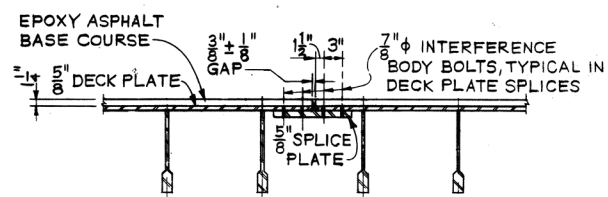
The bituminous concrete wearing surface on the bridge was replaced in 2004 and remained in good condition with some isolated areas in fair condition. Most recently (in 2018), portions of the bridge wearing surface were fully resurfaced. Other portions were planned for resurfacing in the future.

The wearing surface typically exhibited random fine to medium cracks, small spalls, and asphalt patches at scattered locations. Some rutting was observed in the wheel lines of heavily traveled lanes. A combination of grade, alignment, and heavy traffic braking had resulted in slippage cracks at isolated locations. Overall, the deck and wearing surface condition had not indicated undesirable flexibility in the deck plate, and the system was performing satisfactorily.



Delaware River Port Authority

Figure 29. Section 4 deck details



Delaware River Port Authority

Figure 30. Section 5 deck details

6. Wearing Surface

KEY POINTS

- Wearing surface options are most typically bituminous surfacing systems, polymer surfacing systems, or concrete surfacing systems
- Thick wearing surface options contribute to the overall deck stiffness and can reduce live-load induced stresses
- Each type of wearing surface option has its own prescribed installation procedure

Wearing surfaces on OSD bridges serve multiple functions including corrosion protection of the steel deck, improved ride quality, and increased rigidity and load distribution characteristics in some cases. Historical selection and performance of wearing surfaces has varied widely. The steel deck plate thickness, traffic volume, truck traffic, and climate, among other variables, are all contributors to the effectiveness of the wearing surface. The three most common surfacing systems used include bituminous, concrete, and polymer.

Bituminous surfacing systems are considered a thick wearing surface (2 in. or greater). Bituminous surfacing thickness can contribute to the reduction in live-load induced stresses in the deck plate, although its contribution is not considered in design. The wearing surface has been found to perform relatively well, especially on OSD systems with greater rigidity. Due to the nature of the materials used, the system can be sensitive to temperature effects, softening in high temperatures and hardening in low temperatures. The most common problems observed include rutting, shoving, and tensile cracking.

Similar to bituminous systems, concrete surfacing systems are considered thick (2 to 3 in.). In its basic form, concrete is placed with added reinforcement and topped with an epoxy/aggregate system. The concrete used can be of a specific mix design (e.g., high-performance concrete) and the reinforcement can take on several forms (welded wire reinforcement, steel fiber, carbon fiber, etc.). Furthermore, welded shear studs may be added to positively connect the wearing surface to the steel deck. An advantage of the concrete system is the ability to add deck stiffness, which can contribute to the reduction of live-load stresses in the deck plate.

Polymer surfacing systems are considered thin wearing surfaces ($\frac{1}{2}$ in.) and contribute minimally to the overall dead load, unlike bituminous and concrete systems. The final thickness of the system does not lend to additional deck stiffness. The most common problems observed include delamination from the steel deck and loss of surface aggregates, although recent advancements in polymer systems have helped reduce these occurrences.

Prior to placing any of the surfacing systems, the OSD is cleaned and shot blasted to eliminate oil, grease, dirt, dust, mill scale, rust, paint, oxides, corrosion products, and other foreign matter. Once this process is completed, a zinc-based primer is often used to protect the steel deck from corrosion.

Bituminous wearing surfaces are placed in multiple built-up layers of bond coats and epoxy asphalt concrete. Pneumatic tire and heavy steel rollers are used to achieve compaction. Polymer surface systems are often proprietary and have specific installation instructions. In a general sense, however, they tend to follow one of

two construction methods: multi-coat overlay or slurry.

The multi-coat method involves spreading a thin layer of polymer resin with rollers/squeegees followed by broadcasting coarse aggregate into the resin. Once dried, loose aggregate is removed, and the process is repeated until the desired thickness is achieved.

The slurry method consists of first placing a layer of polymer resin followed by a $\frac{3}{8}$ in. thick polymer concrete slurry. Coarse aggregate is broadcast onto the slurry layer. Once cured, the process is completed by sealing the surface with a polymeric resin.

Concrete surface systems are placed with slipform pavers following the placement of reinforcement. Shear transfer and bonding of the concrete wearing surface to the steel deck surface is done in a couple of different ways. The first way is using an epoxy embedded with granular aggregate on the steel deck, which is then overlaid by the concrete. The second way is to use shear studs welded to the steel deck and cast into the concrete.

Poplar Street Bridge – Missouri



Justin Dahlberg, Bridge Engineering Center at Iowa State University

Figure 31. Poplar Street Bridge over the Mississippi River

One of the primary aspects of OSD design in the past has been to optimize performance while minimizing weight. Accordingly, the deck plate thickness has often been minimized while still meeting the strength criteria. One downfall of a minimized thickness is the loss in stiffness. Over time, the flexibility becomes apparent in wearing surface degradation and/or fatigue cracking.

Review of numerous OSD bridges indicated that the well-performing decks have a minimum deck plate thickness of $\frac{5}{8}$ in.

A good example of the drawbacks of reduced deck plate stiffness is the Poplar Street Bridge (Figure 31), which spans the Mississippi River between St. Louis, Missouri and East St. Louis, Illinois.

The bridge was constructed in the mid-1960s and was the first long-span orthotropic bridge to be constructed in the United States. The bridge has known serviceability and deck performance issues tied to the wearing surface.

Until 2006, a 3 in. thick asphaltic-based overlay provided general protection and added stiffness to the OSD. In 2006, a $\frac{1}{2}$ in. thick epoxy concrete overlay was installed, which quickly de-bonded and led to failure of the overlay and eventual exposure of the steel deck to direct traffic. Analysis suggested this was directly related to both the thickness and the elasticity of the epoxy, lending to flexibility, particularly during the summer months when wearing surface temperatures could exceed 120 degrees Fahrenheit.

Overall, this reduction of wearing surface stiffness increased local flexibility and therefore stresses at the OSD weld details. Thus, the deck plate was more susceptible to cracking. Although deck cracking was discovered during inspections at several locations, this condition was not a safety concern and the resulting wearing surface cracking was largely only an inconvenience to the traveling public as a rough ride.

The bridge has three separate deck thicknesses of $\frac{1}{16}$ in., $\frac{5}{8}$ in., and $\frac{3}{4}$ in. Rib-to-deck plate weld cracking was the clear majority of all fatigue cracks with their concentration in areas with thinner deck plates and near the very stiff main girder webs. Cracks forming at the $\frac{1}{16}$ in. rib-to-deck plate weld grew through the weld throat, turned into the rib, or grew into the deck plate. Where cracks formed within the $\frac{5}{8}$ in. and $\frac{3}{4}$ in. deck plates, the cracks initiated in the rib-to-deck plate weld and either grew in the weld or turned into the rib; they did not grow into the deck plate as was observed in the $\frac{1}{16}$ in. plate deck.

Additional cracking was discovered at other locations, albeit in limited locations. Ultimately, to fix the deck plate flexibility issue, the deck plate was studded, and a 4 in. thick fiber-reinforced lightweight concrete wearing surface was placed. The serviceability of the OSD itself has not been in question since, and particularly where the thicker deck plates exist. Based on the inspection reports, the thin overlay and a thin deck plate were likely responsible for the Poplar Street Bridge fatigue cracking.

7. Floorbeam

KEY POINTS

- For new construction, floorbeam depth is not restricted as with retrofit scenarios
- It is beneficial to use a deeper floorbeam/diaphragm for added system stiffness and improved fatigue performance at rib-to-floorbeam connections
- Fit-up of ribs is readily achieved with appropriate tolerances

In the United States, OSDs have often been used in retrofit applications. In these applications, the structural depth restrictions have regularly limited the depth of the floorbeam/diaphragm. Accordingly, the floorbeam/diaphragm analysis and design have been more extensive, and the connection detailing has been more complex.

With new structures, the typical depth is not restricted in the same way. A deeper floorbeam/diaphragm can be used, which lends to added system stiffness and simplified connection to the ribs. The floorbeam/diaphragm depth below the ribs can be optimized for maximum structural performance rather than ensuring the total system depth remains within the bounds that an existing structure may impose.

It should be noted that this guide is intended for OSD systems where a minimum depth of the

floorbeam/diaphragm can be maintained. Where the depth of the floorbeam/diaphragm is less than the depth indicated in Table 1 and Table 2, the designer should pay additional attention to the possible need for cut-outs at the rib-to-floorbeam connection.

Normal shop tolerances are commonly used for rib to floorbeam/diaphragm fit-up and are readily achieved, especially for smaller bridges. A tighter tolerance is unnecessary and tighter tolerances have posed challenges that can be avoided. Fillet welds, which avoid the necessary effort for complete or partial penetration groove welds, may be used between the rib and floorbeam/diaphragm. As the Bridge Welding Code (AASHTO/AWS 2016)¹ allows, in places where a fit-up gap greater than $\frac{1}{16}$ in. occurs, this can be addressed by correspondingly increasing the size of the fillet weld (Figure 32).

Floorbeam/Diaphragm Details

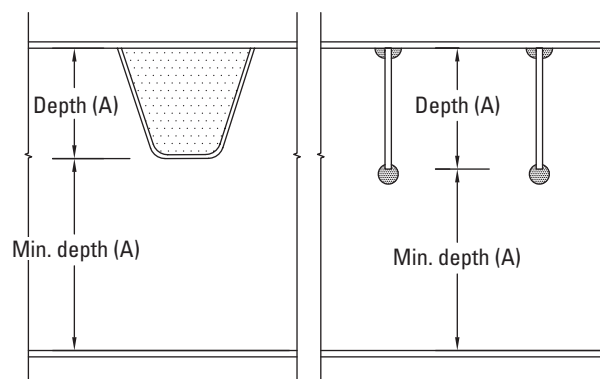


Figure 32. Floorbeam/diaphragm depth detail

Figure 32 Commentary

As presented for both open and closed ribs, the depth of the floorbeam/diaphragm below the ribs needs to be equal to or greater than the rib depth (A) to reduce potential for increased stress levels at the rib-to-floorbeam welds. Engineers still need to check the floorbeam for shear and bending moment forces.

1. AASHTO/AWS. 2016. AASHTO/AWS D1.5M/D1.5:2015_AMD1. Bridge Welding Code (BWC), 7th Ed., Amendment (Dec. 12, 2016). American Association of State Highway and Transportation Officials and American Welding Society, Washington, DC. (Incorporated by Reference at 23 CFR 625.4(d)(2)(iii).)

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