# Guidelines for the Design of Steel Railroad Bridges for Constructability and Fabrication

AREMA
AMERICAN RAILWAY ENGINEERING AND MAINTENANCE-OF-WAY ASSOCIATION

NATIONAL STEEL BRIDGE ALLIANCE NSBA FOUNDED 1995

Smarter. Stronger. Steel.

# Guidelines for the Design of Steel Railroad Bridges for Constructability and Fabrication

National Steel Bridge Alliance

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## FOREWORD

This document has been prepared as a guide and thus much of the information is general in nature, representing the latest steel industry best practices. Recommendations should not be considered as strict rules to be followed by any individual or group. Also, this document should be used in conjunction with AREMA Manual for Railway Engineering, Chapter 15—Steel Structures, and other referenced documents for further clarification on specific issues. As acknowledged by AREMA, the AREMA Manual for Railway Engineering, Chapter 15-Steel Structures provides recommendations and not requirements, and therefore when this Guideline refers to the AREMA Manual, it refers to the Chapter 15 provisions as recommendations. This document is not intended to supersede any AREMA recommended practices. Owner requirements and preferences prevail over the details expressed throughout this document.

The AASHTO/NSBA Steel Bridge Collaboration has published a similar document, G12.1, *Guidelines to Design for Constructability and Fabrication,* for highway steel bridges. Some of the recommendations herein are based on that document, and reference is also made directly to it.

Except for Chapter 1, which contains general introductory material, the chapters in this document are presented in a two-column format with recommendations in the left column and commentary in the right column.

# PREFACE

This document presents guidelines developed by the AREMA/NSBA Steel Bridge Collaboration. It is desired that Owners adopt and support Collaboration guidelines in their entirety to facilitate the achievement of standardization. It is understood, however, that local statutes or preferences may prevent full adoption of the guidelines recommended herein. In such cases, Owners may adopt these guidelines with the exceptions they find necessary.

Published by the American Institute of Steel Construction (AISC) and The American Railway Engineering and Maintenance-of-Way Association (AREMA).

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## LIST OF ABBREVIATIONS

| | |
|---|---|
| **AAR** | Association of American Railroads |
| **AASHTO** | American Association of State Highway and Transportation Officials |
| **AISC** | American Institute of Steel Construction |
| **AREMA** | American Railway Engineering and Maintenance-of-Way Association |
| **ASD** | Allowable Stress Design |
| **AWS** | American Welding Society |
| **CFR** | Code of Federal Regulations |
| **CJP** | complete joint penetration |
| **CM/GC** | Construction Manager/General Contractor |
| **DB** | design-build |
| **DBB** | design-bid-build |
| **DF** | direct fixation |
| **DPG** | deck plate girder |
| **DTI** | direct tension indicator |
| **EAF** | electric arc furnace |
| **FCM** | fracture-critical member |
| **FRA** | Federal Railroad Administration |
| **GC** | general contractor |
| **HSR** | high-speed rail |
| **IOZ** | inorganic zinc |
| **LRFD** | Load and Resistance Factor Design |
| **NSBA** | National Steel Bridge Alliance |
| **OZ** | organic zinc |
| **QC** | quality control |
| **SBC** | Steel Bridge Collaboration |
| **SDLF** | steel dead load fit |
| **TDLF** | total dead load fit |
| **TPG** | through plate girder |
| **TSC** | thermal spray coating |

# SECTION 1. SPECIAL CONSIDERATIONS FOR RAILROAD BRIDGES

## 1.1. Introduction

Railway and highway bridges perform the same function of supporting vehicle loads on a working deck surface while providing a vertical separation for the vehicular corridor with the natural and built obstructions encountered in its path.

Both railway and highway bridges follow similar mathematical and physical principles in their execution, but the design and construction methods differ. This section provides an overview of special considerations for railroad bridges, including the key differences between railroad and highway bridges.

As used herein, the terms "rail", "railway", "railroad" are often used synonymously or interchangeably.

Information included within this publication is specific to operations and facilities within the United States of America.

## 1.2. Types of Railroads

Freight railroads include any railroad that moves raw materials and finished goods to markets domestically and globally. Freight rail consists of two major categories of railroad, Class I and Short Lines, which includes Class II and Class III. The designation of Class I, II and III railroad is based on annual revenues. The revenue levels are set by the Surface Transportation Board with Class I as the highest level. These railroads traditionally operate over exclusive railroad rights-of-way. Freight rail typically operates diesel-powered vehicles.

Intercity rail primarily covers passenger services that connect people from one city to another. Intercity rail operates on both exclusive rights-of-way and rights-of-way owned by other freight or passenger railroads. The most common intercity rail operator is Amtrak.

High speed rail (HSR) is a specific form of intercity railway passenger service. HSR covers railways whose maximum operating speeds are within 90 to 150 mph.

Transit railroads aid in the movement of people between locations. It includes system such as commuter rail, light rail, and streetcars.

Commuter rail is a form of passenger service that connects a suburban area with a city center in metropolitan regions. Traditionally, commuter trips encompass connecting people to work, retail, and entertainment venues. Commuter rail operates over exclusive right-of-way or existing freight or intercity railroad rights-of-way. Stations for commuter rail are farther apart when compared to their light rail or transit counterparts. Commuter rail operates both electric and diesel-powered vehicles.

Light rail is a form of urban passenger transportation. Light rail lines operate over both exclusive and shared rights-of-way. Stations for light rail lines are closer than those of commuter rail. Light rail operates vehicles powered from an overhead electric system called a catenary. Some operators use AASHTO bridge design specifications for the design of light rail structures.

Streetcar rail is a form of urban passenger transportation. Streetcar routes operate primarily over streets in mixed-traffic corridors. Streetcars make frequent stops compared to light rail. Similar to light rail, streetcars may use an overhead electric system. Streetcars are sometimes referred to as trollies.

Other rail-operated vehicles exist, such as funiculars or certain cable cars. These systems are not a consideration in this guideline.

### 1.3. Operations and Management

Similar to highway operations, most passenger rail systems are typically managed by governmental agencies. Rail lines operated by commuter railroads usually are either owned by the agency, a host railway, or a combination of the two.

Freight railways are mostly privately owned and operated, with the freight railway responsible for infrastructure management.

It is common for organizations to maintain trackage rights agreements which allow operation of one railroad over another's territory. In cases where trackage rights exist, both organization's design standards and requirements should be considered.

#### 1.3.1. Regulatory Environment

Both passenger and freight railways are regulated for safety by the Federal Railroad Administration (FRA). The FRA regulates some functions of infrastructure management in addition to specific areas of operations and maintenance. For bridges, the FRA requires the designated operator of a bridge to maintain a Bridge Management Program.

High-Speed Rail (HSR) systems operating in the United States are also under FRA auspices.

The Surface Transportation Board (STB) is a federal organization responsible for the economic regulation of various modes of surface transportation, primarily freight rail. The STB maintains jurisdiction over some passenger rail matters. The STB manages railroad rates or service issues and also reviews and facilitates restructuring transactions such as mergers or rail line sales, construction, or abandonments.

Bridges for transit and light rail are governed by the operating requirements of the agencies overseeing the operation.

#### 1.3.2. Operating Environment

Most railway operations are controlled by scheduling or signal systems governed by dispatching offices. Movements on these lines are tightly controlled.

Rail vehicle weights for standard railcars are governed by the individual railroads, but are generally based on recommendations by the Association of American Railroads (AAR). Vehicle weights outside of those requirements are allowed by agreement between rail companies under controlled conditions.

Operations for transit systems may include coordination with a host railroad (see section 1.2). Some transit bridges also share corridors with automobile and truck traffic, which requires a blending of design for both types of traffic.

#### 1.3.3. Maintenance Environment

The ability to simplify maintenance of bridges should be considered in design. Bridges are an investment that can provide long service given proper inspection and maintenance. Straightforward design methods that provide simple and effective construction provide ease of maintenance for all rail-based structures.

Maintenance, like operations, on passenger and freight railways is tightly controlled. This control stems from the need for safety in providing maintenance windows around operations. Planning for major maintenance activities in limited time windows frequently influences the design of repairs and may involve an extended period of planning and scheduling before execution.

Bridges for transit systems that are exclusive for those systems have similar concerns regarding the planning and scheduling for maintenance, which may have to occur during periodic off-line hours. Maintenance of bridges that share vehicular traffic should consider the needs of applicable transportation agencies.

### 1.3.4. Inspection Environment

Railway bridges are required to have an inspection on an annual basis. The regulations administered by the FRA for bridge inspection are contained in 49 CFR Part 237, "Bridge Safety Standards". These regulations include requirements for annual inspections, maximum time between inspections, review requirements, and inspection filing. Requirements for inspection facilities on the bridge are supplied by the owner or, in the case of transit and light rail, by the managing agency.

## 1.4. Design

### 1.4.1. Design Philosophy

A railroad bridge has a load path that is predetermined and live loads that that follow the centerline of the rail. By contrast, a highway bridge design needs to consider its live loads being placed anywhere on the deck of the structure with various different load paths to the supporting structure. Highway bridge loading is further refined by the likelihood and frequency that the bridge structure will be supporting the heaviest vehicles across the deck at the same time.

Transit and light rail design requirements have been developed in guide form by AASHTO but their use varies. Transit and light rail agencies can develop the requirements to meet their needs. These can include hybrid design requirements using aspects of both railway and highway bridge design.

Freight and some passenger structures do not have the same freedom of design for various reasons, but this is especially due to the magnitude and frequency of load cycles as well as limited construction windows for maintenance and replacement. These bridges are almost always simply supported. Use of readily available standard sections facilitate initial construction, repairs, and inspection. Details and connections must be thoughtfully developed, as fatigue considerations play a significant part in the design process.

Design of freight and some passenger steel structures are guided by the recommendations of Chapter 15 of the AREMA Manual for Railway Engineering (henceforth "AREMA Chapter 15"). These are published as recommendations and can be modified as desired by the owner.

Chapter 15 uses Allowable Stress Design (ASD) for its design method. This is unlike vehicular bridges, where Load and Resistance Factor Design (LRFD) is used. The probabilistic approach of LRFD offers advantages to highway bridge design due to factors such as the variety of load location placements among vehicular lanes and possibilities for extreme overloads in service. Such advantages are not as distinct for rail bridges.

HSR bridges have a critical need to control deflection compared to freight rail bridges and lower speed transit bridges. This is due to potential resonant response between the HSR train sets and the bridge at high speeds.

Because of the need for deflection control, overall section size for freight, some passenger, and HSR bridges are designed such that the use of high-strength steels (yield strength greater than 50 ksi) is typically not needed. Conformance with fracture toughness requirements is still necessary, but standard strength steels will usually provide sufficient strength.

### 1.4.2. Types of Bridges

Rail bridge types are chosen by the length of the bridge and other needs. See the recommendations for bridge type and lengths in section 2.2.

Shipping constraints (see section 1.5) may be a limiting factor and should be considered.

Box girders and other types of structures may be used in transit and HSR.

#### 1.4.3. General Design Characteristics

Bridges for freight and some passenger rail use design features that have long been standard for the industry. This includes simply supported structures and standard details that provide reliability when subjected to the highly cyclic railroad loads. Welded fabrication is suitable, but critical welds should be made in the shop. For fabrication in the field, bolting is usually preferable, when possible, considering the skills and equipment needed for field welding.

Decks can be either "open" or "ballast". Open decks rely upon bridge ties (usually timber) to carry the train load to either the girders or the floor system while the ballast deck provides a solid surface for application of ballast to support standard track. Ballasted decks can use either a steel plate, a concrete deck, or a timber deck to support the ballast above the floor system.

Passenger systems make frequent use of direct fixation of rail to either the deck or stringers. Direct fixation track fastens the track components directly to the load supporting elements, rather than distributing the loads through ties.

#### 1.4.4. Design Loads

Freight owners may require their own design live loads but generally follow current AREMA design criteria which recommend the Cooper E Load as the general design load with an Alternate Load which typically governs at shorter span lengths.

HSR bridge design loads can be either a specialized loading or some level of Cooper E Load. This choice is at the discretion of the owner.

Transit and HSR design loads are provided by the owner procuring the design. The transit design loadings are generally known loadings, typically reflecting the equipment in use on their system.

For all rail-based design loading systems, lateral and longitudinal loads create forces in bridges. These forces are significantly higher than those used in the design of highway bridges.

### 1.5. Considerations for Shipment

Shipping of the bridge parts to the bridge site should be considered in design of the bridge, particularly for larger structures. Shipping factors to consider include:

- Route
- Availability of equipment
- Support locations
- Permits
- Weather conditions
- Field splices
- Component orientation for shipping
- Shipping in pairs for stability
- Height, weight, length, and stability
- Site access

#### 1.5.1. Shipment by Truck

Trucking is suitable for most bridge components, including large girders. For long spans, it is prudent to consider limitations during type selection and design.

As needed, fabricators will use steerable dollies to facilitate shipping of long pieces. The length for which steerable dollies are needed varies among fabricators and depends upon constraints along the shipping route, but use of steerable dollies may start at lengths of 80 feet.

Maximum size consideration for truck shipping are as follows:

a. Maximum shipping length:
   - A reasonable maximum length is 140 feet long.
   - Beyond this length, shipments are more likely to have more constraints, such as requirements for police escorts, higher permitting costs, time of day restrictions, and limits to the number of loads.

b. Maximum vertical height of girders:
   - Shipping girders vertically (standing up) is preferred over shipping with girders horizontally (lying down) because this makes loading at the shop and unloading on the jobsite much easier and minimizes the introduction of shipping stresses in the member.
   - Often overhead clearances along the shipping route are the limiting constraint for what can be shipped vertically. Most bridges provide a minimum of 13'-6" vertical clearance. Depending on the available equipment and the shipping route, most fabricators can vertically ship girders having a depth of up to 11 feet.
   - Taller girders can be shipped horizontally, but the suitability of this approach also depends upon constraints along the route. Shipping girders 12' deep and greater is usually possible, but this should be verified in consideration of local constraints.
   - For girders that are supported by steerable dollies the vertical under-clearance must be considered so that the central portion, or overhang, does not bottom-out on humps along the route.

c. Maximum horizontal width (girders on side):
   - Maximum width should not exceed 15'-6".
   - Greater widths are possible but trigger additional constraints similar to those for very long girders.

d. Maximum shipment weight:
   - Weight of a single member should not exceed 120,000 lbs.
   - Greater weights are possible but require special equipment and often require permits and/or longer routes due to bridge weight limitations on the shipping route.

Constraints on project sites can limit shipping weight, height, and width restrictions beyond over-the-road limits. Designers can contact regional steel bridge fabricators, or the NSBA, to obtain more insight regarding possible shipping limitations for a given project site.

### 1.5.2. Shipment by Rail

Rail can be an effective means of transportation to remote railway bridge sites.

Shipments of girders and spans by rail typically require special handling and train operations. Shipments of girders and spans may also require special equipment (rail cars). The originating railroad company's clearance bureau or equivalent must determine if the shipment is a dimensional shipment. To pre-authorize a shipment, the following is typically required:

- Origin and destination (for route check).
- The following characteristics for determining equipment required: length, width, height, weight, and location of center of gravity of the shipment.

The shipment must be inspected by a railroad car inspector before it is billed and released after the railroad company's clearance bureau, or equivalent, issues a clearance notice.

Shipping by rail usually requires:

- Designed bolsters and tie-downs that must be approved by the railroad owners.
- Approval from the railroad for what is being shipped, including drawings and calculations.
- Trained personnel approved by the railroad for loading onto rail cars.
- Coordination with the railroad for an approved unloading location.

Maximum size consideration for shipping by rail are as follows:

a. Maximum shipping length:
   - Long girders may be shipped on bolsters on adjacent flatbed rail cars (flatcars).
   - Flatcars are typically 60 feet or 89 feet long.
   - Shipment length may be limited due to obstructions adjacent to curved track along the proposed route (i.e., through bridges, tunnels, signal structures and adjacent structures).

b. Maximum vertical height of girders:
   - Vertical clearances from top of the rail along railways are typically 17 to 19 feet depending on route.
   - Typical flatcars have a deck height from rail of about 4 feet. Therefore, depending on the route, girder heights of up to 15 feet can be shipped.

c. Maximum horizontal width (girders on side): Limited by flatcar deck width of about 9 feet.

d. Shipment weight: The shipping weight limit varies with the weight of the car, but four-axle cars can typically handle 100-ton loads. Heavy-duty flat-deck cars are available for extra-heavy shipments. Heavy-duty depressed-deck cars are available to provide additional clearance for oversize loads. These heavy-duty flatcars have capacities ranging from 100 tons to 370 tons.

### 1.5.3. Shipment by Barge

Shipping by barge is a useful option for some structures, particularly large structures that are located at or near navigable water. For shipping by barge, fabricators who are located on navigable water will load the barge at their facility. Other fabricators can ship components to a facility on navigable water and conduct assembly there, but this is not common due to the additional time and cost associated with this approach. Alternatively, other fabricators can partner with fabricators who are on navigable water.

Considerations for barge shipping include:

- The type of barge to be used (deck or hopper style)
- Engineered load plan for insurance and marine planning
- The required draft (depth of water) at loading point
- Height of berth at both low and high water
- Weight of payload
- Height of load on barge
- Required width and length of barge
- Loading and off-loading configurations and specialized equipment
- Port fees and marine insurance at load and offload points
- Time to load and offload and demurrage considerations
- Time on water and weather consideration

## 1.6. Sustainability

Sustainability is a growing consideration in bridge design. In terms of environmental impact, "sustainability" is a broad term capturing attributes that may include life cycle assessment, global warming impact, ozone depletion, embodied carbon, eutrophication, and acidification. The iron and steel industry worldwide accounts for around 21 percent of global industrial energy use and about 24 percent of industrial $CO_2$ emissions in the world (Hasanbeigi & Springer, 2019). Due to the share of steel produced using the less energy-intense electric arc furnace (EAF), as opposed to blast furnace-basic oxygen furnace, steel production energy use within the United States is ranked 4th lowest in the world for $CO_2$ admissions. Since the 1990s, the steel industry has reduced greenhouse gas emissions by 36 percent. This trend will continue to grow as steel producers transition toward EAFs and renewable energy sources to power them.

Typical bridge steels produced in the U.S. by electric arc furnace process (mini-mills) have some of the highest recycled content volumes in the world. For hot-rolled shapes, these steel mixes contain as much as 96% recycled content. For plate products, most bridge steel mixes contain between 80 and 90% recycled content. Further, nearly all of the structural steel is recaptured at the end of a structure's service life and it can be recycled endlessly.

The "four Rs" of environmental sustainability help summarize the impact of steel across the life cycle of the structure. The four Rs are recycled content, recyclability, recovery rate, and reuse (AISC, 2017). Sustainability is also affected by design practices. For example, constructable designs optimize material and fabrication labor, use of accelerated bridge construction reduces energy consumption while also making work zones safer, maximizing span lengths reduces substructure construction, and achieving a 100+ year service life reduces impact in the long term. The practices recommended in these guidelines align well with these sustainable practices.

# SECTION 2. DESIGN

## 2.1. Selecting Main Supporting Member Type

The choice of main supporting members for railroad bridges is based on structure type, length, and clearance needs.

## C2.1

The clearance envelope needed for the bridge will be specified by the owner. AREMA Chapter 15 Figure 15.1.1 recommends a minimum clearance envelope.

Through-girder and through-truss spans are characterized by a floor system located above the bottom flange of through-girders or the bottom chord of through-trusses. Trains then travel between (or through) the main exterior members. Through spans are used where vertical clearance under the bridge is limited.

Deck spans are characterized by a floor system that resides above the top flange of the main members or the top chord of the trusses. They are used where vertical clearances under the bridge are not limited.

Owners may have standard designs for most bridge types.

If needed for schedule and approved by the design engineer, plate girders can be substituted for rolled beams.

## 2.2. Span Length

Practical span lengths by superstructure type are:

- Rolled beam or welded deck girders for spans up to 70 feet.
- Deck plate girders for spans of 70 to 150 feet.
- Through plate girders for spans between 70 to 200 feet.
- Trusses over 200 feet. The maximum practical length of simply supported truss spans is 400 feet.

The above practical span lengths are applicable to open and ballasted deck bridges.

Rolled beams are usually more economical than welded plate girders for short spans.

Welded built-up plate girders are more economical than bolted construction.

## C2.2

Practical span lengths are based on AREMA Chapter 15 Article 1.2.3 and, in the case of rolled beams, currently available rolled sections.

While 400 feet is a practical limit for simply supported truss spans, some have been constructed with span lengths of 500 feet. Furthermore, even longer spans may be achieved through the use of continuous truss spans.

The additional dead load of ballasted decks may be a major consideration for longer spans. Timber, concrete, and steel have been used to construct ballast tubs to support the ballasted deck. The deck to be selected should balance the additional dead load with the long-term maintenance of the track and bridge.

Railroads may require bolted girder construction in consideration of environment, crack growth resistance, low vertical clearance to highway traffic below, or other considerations.

## 2.3. Span Girder and Beam Arrangement

### 2.3.1. Deck Span Arrangement

In general, the use of fewer girders in the bridge cross-section will result in a more economical design by reducing the overall weight of steel used, as well as minimizing the pieces to fabricate and erect.

Nevertheless, the decision on whether to choose a wider girder spacing should be made with consideration of other factors and not structural steel weight alone.

### C2.3.1

AREMA Chapter 15 Article 1.2.4 provides recommendations affecting the number and spacing of girders and beams and Article 1.3.4 addresses load distribution. Additionally, the following should be considered:

- AREMA Chapter 15 Article 1.11.4 provides guidance for open deck spans where all girders and beams are considered equally loaded when multiple girders or beams are sufficiently braced with diaphragms and are symmetrically spaced under tangent track.
- For ballasted (closed) deck spans with multiple girders or beams symmetrically spaced under tangent track, live load shall be considered equally distributed to all girders or beams with centroids within the lateral width defined by the length of the tie plus twice the minimum distance from the bottom of the tie to the top of the girders or beams. The lateral distribution of the loads should not exceed 14'-0" nor the width of the deck between ballast retainers.
- For multiple track structures the lateral distribution width for each track should not exceed the track centers.
- For other conditions, lateral distribution of load and girder or beam spacing should be determined by recognized methods of analysis.

Typically, the following benefits are derived from the use of wider girder spacing:

- Lower total structural steel weight
- Fewer girders to fabricate, inspect, handle, coat, transport, and erect
- Fewer cross-frames to fabricate, inspect, handle, coat, transport, and erect
- Fewer bolts and connections
- Reduced time of fabrication and erection
- Fewer bearings to purchase, install, and maintain
- Improved access for in-service inspections

The following issues need to be evaluated during the decision-making process when wider girder spacing is being considered:

- AREMA Chapter 15 recommendations affecting girder spacing
- Potential for a thicker concrete deck resulting in more weight, concrete, and reinforcing steel
- Methods for forming the deck
- Girder depth and infringement on vertical clearance
- For open deck spans, the girder spacing can affect the length and depth of bridge tie required. The cost and availability of larger bridge ties should be considered when using larger girder centers (over 7.5 feet)

### 2.3.2. Through Span Arrangement

Arrangement of through-girders and through-trusses on railway bridges is governed by recommendations in AREMA Chapter 15 and the preferences of the owner. Stability of the section from lateral forces is a major consideration in the AREMA recommendations with specific guidance given for lateral bracing.

The main members of through-girders and through-trusses require a floor system that supports the track. The location of all structural members must be outside of the minimum clearance envelope used by the owner.

#### 2.3.2.1. Arrangement Considerations for Through Girder Bridges

Through girder bridges have two or more main girders with a floor system that supports the track. The floor system is located above the bottom flange to minimize span depth under the track in low under-clearance situations.

The floor system can be either a series of transverse floorbeams or a stringer–floorbeam system. Depth and vertical clearance requirements may dictate which floor system is preferable. Knee braces are required on the main girders to prevent lateral-torsional buckling of the compression flanges.

The main girders must be outside the clearance envelope.

**C2.3.2.1**

Refer to AREMA Chapter 15 Article 1.2.4 for minimum spacing.

### 2.3.3. Arrangement Considerations for Truss Bridges

Some deck and all through truss spans require a floor system to support the track with the main trusses located on the outside of the floor system.

Through trusses require the main trusses to be placed outside of the minimum clearance envelope used by the owner.

Floor systems for trusses are usually stringer–floorbeam systems but some transverse floorbeam floor systems in trusses have been built. The stringer–floorbeam system is favored as it can be configured to match panel points of the truss.

### C2.3.3

Truss chord spacing encourages the use of a stringer/floor beam arrangement of the floor system. Truss spans with transverse floor beams are rarely found because of the bending induced in the chord members supporting the floor system and the difficulty with designing the connections.

Refer to AREMA Chapter 15 Article 1.2.4 for recommended spacing of through girders.

## 2.4. Fit and Differential Deflections

For typical single-track, tangent railroad bridge spans fabricated from steel deck girders in a single construction phase, the girders are arranged to transfer the track load to all girders as uniformly as possible. In these typical cases, differential deflections are minor and the cross-frames or diaphragms between the girders can be detailed to fit at erection, commonly referred to as steel dead load fit (SDLF).

Situations that may require additional consideration on specifying a fit condition for detailing cross-frames and diaphragms between girders include:

- Spans with significantly skewed supports
- Horizontally curved girders
- Phased construction

For recommendations on what fit condition is appropriate for a given bridge, and possible associated locked-in stresses on bridges with significant differential deflections, see "Skewed and Curved Steel I-Girder Bridge Fit," NSBA (2016).

## C2.4

"Differential deflection" refers to the difference in girder deflection at either end of each cross-frame or diaphragm. When differential deflections exist (as they do on skewed and curved bridges), cross-frames and diaphragms tend to deflect a different amount on either end. Since the cross-frames and diaphragms are very stiff, they cannot easily distort to accommodate these differential deflections, and so the result for most skewed and curved bridges is that the girders twist during construction.

In an I-girder bridge, "fit" refers to how the cross-frames or diaphragms are detailed and fabricated to fit to the girders. They may be:

- Detailed to fit when all dead loads are applied ("total dead load fit," TDLF, or "final fit"),
- Detailed to fit at erection ("steel dead load fit," SDLF, or "erected fit"),
- Detailed to fit in the no-load condition ("no-load fit," NLF, or "fully cambered fit"), or
- Detailed to fit at some other condition in between.

The girders, cross-frames, and diaphragms may actually fit at more than one or even all of these conditions. The distinction here is not whether the bridge components actually fit in these conditions; rather, it is how the bridge is detailed to fit.

The detailed fit condition can influence:

- The ability to construct the bridge. For example, choosing TDLF for a sharply curved bridge can make the bridge unconstructable.
- Internal loads associated with the fit condition.

For bridges with skewed supports and detailed to TDLF, there will be layover (twist) at erection, but usually the girders will come back to plumb under total dead load. Conversely, on a bridge with skewed supports and detailed to SDLF, girders will be plumb at the end of steel erection but will twist as the deck is cast and will have some final layover under total dead load. For this reason, the fit condition is sometimes referred to as the "plumb" condition. However, it is not recommended to refer to fit in this way because it confuses the issue—particularly when the "plumb" discussion is extended to curved girders in which layover and plumb do not work the same way.

The NSBA document referenced in the recommendation provides more explanation about these fit choices and phenomena. The NSBA document also provides guidance on how the severity of the skewed supports, bridge length, and bridge width can influence differential deflections. Typically, railroad girder bridges are detailed for SDLF.

When phased construction is required, the differential deflection between phases due to the application of dead loads at different times can be significant, particularly regarding the attachment of additional phased spans to existing spans that are already deflected under full dead load. There are many ways to address this in the design and detailing of cross-frames and diaphragms between the adjacent construction phases. Refer to the AASHTO/NSBA Steel Bridge Collaboration G12.1, *Guidelines to Design for Constructability and Fabrication* (henceforth SBC G12.1), for additional details and considerations for phased construction.

### 2.4.1. Oversize and Slotted Holes

Consider the use of oversize and slotted holes for lateral bracing connections.

### C2.4.1

Oversize and slotted holes can be useful for helping facilities erection of bolted assemblies, particularly when the assemblies are complex.

Oversize and slotted holes should not be used for main member connections.

However oversize or slotted holes should not be used in connections with many bolts, such as main member splices, and in situations when normal size holes may be needed to help hold geometry during erection, such as on diaphragm-to-girder connections on skewed bridges.

**2.5. Fracture Critical**

Only use the "fracture-critical" designation for tensile members (or tension portions of bending members) that meet the definition of "fracture-critical member" (FCM) in AREMA Chapter 15. Compression areas are not fracture-critical, nor, typically, are redundant or internally redundant members.

**C2.5**

On bridges with fracture-critical members (FCMs), not all materials and welds for fracture-critical members are fracture-critical. For example, for bending members, tension flanges and the parts of the web in tension would be fracture-critical, but compression flanges and the parts of the web in compression would not. Also, some attachments would be FCMs and others would not. Practices for indicating what is and is not a FCM vary and sometimes cause confusion. The best practices for designating FCMs are found in Section 7.2.4 of the FHWA Bridge Welding Reference Manual (FHWA, 2020).

There is a misconception in the bridge community that designating a member as "fracture critical" will result in a higher quality bridge. This is not the case, either for materials or fabrication practice.

Steel mills produce the same material for fracture-critical designation that they produce for non–-fracture-critical designation. The mills do not change their processes when producing fracture critical material; rather, for FCMs, toughness tests are performed at higher frequencies and have higher acceptance criteria. If a special condition warrants the higher minimum specified toughness of fracture-critical material but the member itself does not need to be designated as fracture-critical, do not use the "FCM" designation. Rather, note that the material needs to be ordered using the ASTM A709 "F" designator (e.g., A709 Gr. 50WF).

For FCMs, there are additional welder qualifications, inspector qualifications, nondestructive testing mandates, and welding process controls, but the acceptance criteria are the same.

**2.6. Bearings**

Horizontal and vertical reactions on bearings are significantly larger than those of roadway bridges.

**C2.6**

The unique demands of railroad loading, maintenance, and environmental exposure

In both fixed and expansion bearings, anchor bolts often pass through a rotating part of the bearing and are provided with a nut and washer to provide uplift resistance. In these situations, care shall be taken to not restrict the vertical component of the rotating part, which would otherwise impart unnecessary stress on the bearing and tension on the anchor bolt.

When a bolt through a slotted hole is used in expansion bearings, a plate washer large enough to cover the slot (at all expansion limits) is recommended to minimize debris that may fall into the slot and restrict expansion of the bearing.

When bearings or components require machining (such as sliding recesses in masonry plates or bronze rockers), machinists require controlling surfaces or reference lines to be clearly identified along with any critical tolerances and smoothness criteria.

typically lend themselves to more rigid bearing assemblies for typical highway girder structures.

Common bearing types and suitability are covered in AREMA Chapter 15, Section 5.1, and Table 15-5-1 “Bearing Suitability”.

Designers should keep in mind specific AREMA recommendations that are unique compared to roadway bridges, including the following:

- Bearings supporting spans under 50 feet need not have specific live load rotational details.
- Expansion bearings are required to accommodate construction tolerances.

Elastomeric bearings are not typically specified for railroad applications, though they are often found in short-span highway bridges. Elastomeric bearings may be most beneficial in wide structures where lateral thermal expansion may be more than longitudinal thermal expansion. The lateral thermal expansion can be calculated using traditional methods and need not incorporate the AREMA Chapter 15 Section 5.1 construction tolerances or expansion lengths.

Many railroads have commonly used details and preferences; early coordination of bearing details with the owner is important. In lieu of owner specific preferences and details, bearings with a history of long service life on rail structures are as follows (see AREMA Chapter 15 Article 5.1.1.3 for illustrations):

- Fixed
- Rocker plate
- Self-lubricating cylindrical and spherical bearings (cylindrical or spherical sliding surfaces on top only)
- Fixed pedestal
- Fixed elastomeric
- Expansion
- Self-lubricating cylindrical and spherical bearings (cylindrical or spherical surfaces on top of bronze and flat sliding surfaces on bottom of bronze)
- Rocker pedestal
- Elastomeric with or without a PTFE sliding top surface.

Allowing free rotation of a part with a bolt extending through it is often accomplished by specifying a small gap between the top of washer and bottom of nut, with the nut being held in place by use of double nuts or burring the threads of the bolt. A gap of 1/8 inch is common.

A plate washer used to cover a slot should have a hole diameter 1/32 inch larger than the anchor bolt diameter, further limiting debris intrusion.

High levels of restraint in thick weldments can result in residual stresses that can significantly reduce fatigue life or machinability of the resulting weldment. If thermal stress relief of bearing components is required, the contract drawings should clearly state the requirement since it is an integral part of the overall welding procedure.

## 2.7. Minimum Thickness for Stiffeners, Webs, and Flanges

Certain minimum requirements for material thicknesses are normally recommended to reduce deformation, reduce the potential for weld defects, and increase corrosion resistance/durability.

The recommended minimum thickness of any steel component, except for fillers, is 3/8 inch. Gusset plates connecting the chords and web members of a truss and connecting plates and stiffeners have a recommended minimum thickness of ½ inch. For additional minimum thickness recommendations, refer to AREMA Chapter 15.

It is preferable to detail stiffeners and connector plates, and smaller gusset plates, so that they can be fabricated from bar stock or cut from larger plate at the fabricator's discretion.

## C2.7

Preferred minimum thicknesses depend on the welding equipment used.

When sizing webs or stiffeners, it is common to start with ½ inch plates. For connection plates and stiffeners, ½ inch minimum thickness is recommended to facilitate the fabricator's use of submerged arc welding equipment that provides opposing arcs on either side of the stiffener. Keeping to a minimum ½ inch thickness helps keep the fillet welds on either side of the plate from bridging (i.e., penetrating enough that the molten weld puddles connect beneath the plate).

To allow the use of flat bar for stiffeners and connection plates, dimensions of these members should follow the guidance below:

- For design width of stiffeners up to 8 inches, increase the width stiffeners in ½-inch increments (e.g., if the calculated width of stiffener is 6¾ inches, increase the design width to 7 inches).
- For design widths of stiffeners between 8 and 12 inches, increase the stiffener width in 1-inch increments.

At girder field splices with web or flange plate thickness transitions across the splice of $^1/_{16}$ inch or less, fill plates are not required.

At girder field splices with web or flange plate thickness transitions across the splice greater than ⅛ inch, use fill plates on one side of the web only.

AREMA Chapter 15 addresses the use of cover plates, but these are not recommended for new construction. If a design using rolled beams necessitates the use of cover plates, a larger section or a built-up girder should be considered.

- Use standard plate thicknesses for stiffeners. (i.e., ½ inch, ⅝ inch, ¾ inch, ⅞ inch, 1 inch, etc.)

Fill plates with thicknesses of $^1/_{16}$ inch or less pose difficulties in fabrication and handling. If the fill plates are blast-cleaned, distortion from the blasting operations can become problematic. Handling of large $^1/_{16}$- inch-thick fill plates is also difficult due to the plate flexibility.

## 2.8. Material Size Availability

### 2.8.1. Plate Material Size Availability

When sizing girder flanges, maximum lengths available for the various plate widths and thicknesses should be considered.

For the design, select material that is readily available. SBC G12.1 includes tables that show dimensions of typically available plates. Designers should also contact a plate rolling mill or a fabricator for the latest plate availability information.

When considering maximum length, be mindful that fabricators prefer to order material in lengths less than 83 feet so that plates can be delivered by one rail car (if longer lengths are shipped by rail, this requires a triple-set railcar arrangement).

### C2.8.1

The availability of material sizes varies from mill to mill.

Designers should be aware of maximum plate length availability data. The layout of flanges and webs within a given plate to maximize plate usage is best left to steel detailers. Fabricators and steel detailers develop the exact shape and dimensions of each plate, understand the intricacies of flange and web nesting within a plate (including issues related to runout, camber, heat curving versus cut curving, cutting loss, etc.), and can best address the specific capacities and preferences of a given mill and fabricator.

Material sizes not typically produced by a mill may be produced in a special mill rolling. However, small quantities are not likely to be available and could be cost prohibitive to specify. Special rollings are usually scheduled on a specific date and involve large quantities of material.

It is good practice to allow the use of optional flange and web shop splices at the discretion of the fabricator and to include the preferred details for such splices in the contract drawings. When evaluating the economics of introducing flange thickness changes and their associated mandatory flange shop splices as discussed in section 2.9.3, it is useful to have some idea of maximum available plate lengths. If the length of a particular flange exceeds the limits recommended in SBC G12.1, there may be more reason to introduce a thickness change. In the case of very deep webs, fabricators may propose a shop splice to work with narrower plates that may be flatter than

In all circumstances, the guidance provided in SBC G12.1 regarding limiting the number of different plate thicknesses used in a design should be followed.

### 2.8.2. Structural Shape Availability

Structural shape sections of various sizes are produced domestically.

Refer to the American Institute of Steel Construction (AISC) website for specific section availability at <https://www.aisc.org/steelavailability/>.

Appendix A lists commonly available shapes.

**C2.8.2**

SBC G12.1 has a table giving examples of common rolled beam bridge sections. Longer lengths may be available, depending on the Producer. The AISC section availability website includes information on standard products for other typical structural shapes, finding suppliers, etc.

Appendix A is included because not all shapes found in published standards are commonly available in the market. Other published shapes may be available but not as readily, so if they are used in design, the fabricator may not be able to get them, or it may be difficult to get replacements.

### 2.8.3. Miscellaneous Material Availability

Miscellaneous materials for walkways, handrails, etc., are readily available domestically. These types of materials are usually supplied by warehouse steel suppliers.

Specialty materials for walkways may be manufacturer specific. The owner should be consulted for preferred materials specifications, manufacturers, and suppliers.

Contract drawings and specifications need to clearly list the desired materials for the miscellaneous components of the railway bridge.

**C2.8.3**

Warehouse steel suppliers usually stock standard lengths of materials (pipe, tubing, etc.) in 20-foot and 40-foot lengths. Design elements of the structure for walkways and handrails should not need special consideration for mill capacities. Plate, structural shapes, and other materials used for walkways are typically ancillary to the structure.

If the contract drawings or specifications do not clearly define the material specifications for the miscellaneous elements, this may lead to, among other things, inaccurate pricing, extra requests for information, and contract change proposals.

## 2.9. Plate Sizing

### 2.9.1. Flange Plate Width

Size flange material so that flanges can be economically cut from plate between 60 inches and 96 inches wide, even where girder flanges vary from girder to girder.

**C2.9.1**

For size availability, see section 2.8. Fabricators order plate with additional width and length to account for cutting, plate sweep tolerance, and waste.

When changing flange widths is unavoidable, avoid changing flange width at welded shop splices.

For straight plate girders, keep the width of flanges constant to make efficient use of material.

For straight plate girders composed of three flange plates, keep the center plate width, thickness, and length constant between girder lines and then splice the end flange as shown in Figure 2.9.1. It is preferable to keep the end plates the same width as the center plate but vary the center plate thickness across the width of the bridge and then splice the flanges as shown in the figure.

Designers should consider possible shipping and erection when selecting a flange width so that there is not a lateral torsional buckling issue during shipping and erection. Selecting a very narrow flange width can significantly reduce the lateral torsional buckling resistance of the girder and affect how the girder has to be shipped and erected.

For straight girder bridges, fabricators order girder flange material from wide plate and prefer to splice it as wide plate instead of as individual flanges after cutting to width. For constant-width flanges, the advantages to welding wide plate rather than stripping the plate and then splicing the flange includes having one set of run-on tabs and run-off tabs, fewer crane moves, and considerably fewer weld starts and stops. See Figure 2.9.1. Changes in thickness rather than width in a flange section saves labor required to join the flanges as individual pieces.

For bridges with non-parallel supports where the geometry of the flanges could vary from girder to girder, a designer should consider how the material should be ordered and spliced.

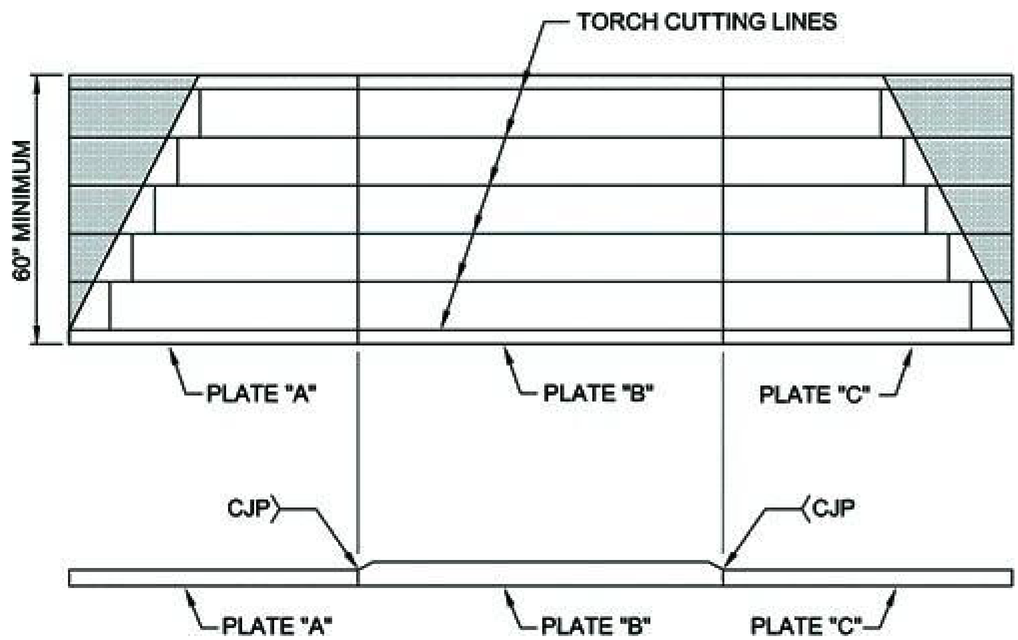


**Figure 2.9.1 Straight Girder Example of Welding Wide Plates, then Stripping out the Flanges**

### 2.9.2. Web Sizing

See section 2.7 for minimum web plate thickness.

The selection of web depth should consider geometric (such as minimum vertical clearances), fabrication and transportation issues and the overall economy of the design.

### C2.9.2

When establishing girder web depths, verify the minimum vertical clearance requirements of the shipping route. Other considerations include fabrication issues (maximum plate widths versus the need to introduce longitudinal shop splices in the web), transportation issues (maximum girder dimensions for transportation), and overall economy.

Typically, if webs are 12 feet deep or more, the designed web depth may not be achievable with a single plate. When multiple plates are needed to achieve the desired depth, a longitudinal shop welded splice may be used to make the web, or a bolted field splice can be used. See AREMA Chapter 15 for splice recommendations.

Economy is achieved using girders that can be shipped with web vertical by truck or railcar, which is limited by overhead clearances on the shipping route. Girders that are too deep to ship vertically can be shipped with the web horizontal, but supporting the full length with the girder's weak axis in bending tends to be much more challenging and costly. Additionally, the designer must account for stresses due to shipping in the horizontal position. Horizontal limits also depend upon constraints along the route. For shipping via truck, it may be necessary to obtain permits from affected state and local departments of transportation.

To assess overall economy, it may be valuable to perform a web depth study in which the web depth is incrementally increased, the girder is redesigned (targeting an unstiffened web design), and the resulting girder weight versus web depth is recorded. These data points (girder weight versus web depth) can then be plotted to determine the optimum girder weight and web depth.

Longitudinal web stiffeners should be avoided.

Typically, the use of longitudinal web stiffeners is only economical in long-span steel girders where using very slender webs can lead to significant reductions in girder weight.

### 2.9.3. Shop-Welded Splices

Specify a shop-welded splice when the savings in flange material and when plate length limitation or special circumstances dictate.

In the design or specifications, provide criteria the fabricator may follow to eliminate shop-welded flange splices by extending thicker plate.

### C2.9.3

Introduce a shop flange splice and a flange thickness transition when the weight savings will justify the work associated with the welded splice.

Efficiently locating thickness transitions in plate girder flanges is a matter of plate length availability and the economics of welding and inspecting a splice compared to the cost of extending a thicker plate. Design and specifications should consider allowing this practice, subject to the approval of the engineer. When evaluating the request, designers should, among other things, review the change in deflections and stresses.

Some owners may have guides or preferences for economical flange thickness transitions. Fabricators should also be consulted whenever possible.

For additional information related to estimated weight savings related to shop-welded splices, reference AASHTO/NSBA Steel Bridge Collaboration G12.1, *Guidelines to Design for Constructability and Fabrication*.

## 2.10. Erection Stresses and Constructability

Erecting plate girders in assemblies is the preferred method of construction, when possible. Erecting girders in pairs or assemblies helps to improve girder stability during erection and can also reduce track outage times. However, when bridge site conditions limit access to construction equipment and prohibit temporary support locations, such as erecting a steel bridge over a highway or railroad tracks, erecting a single girder at a time may be the only possible method of erection.

The designer needs to consider plate girder proportions such that the girders can resist the erection loads found in AREMA Chapter 15 Section 1.3, taking advantage of Table 15-1-10, and meeting the allowable stresses given in AREMA Chapter 15 Section 4.13. Consideration should be given to providing and calling out plate girder lifting points and jacking locations on the project plans. These recommendations need to be met regardless, whether erection consists of single DPGs or TPGs, in pairs, or in multiple DPGs. Another factor that needs consideration is the maximum shipping length; see section 1.5 for furth details. The designer could inquire with fabricators and contractors regarding shipping methods and maximum shipping lengths if lengths approach limitations noted in section 1.5. The designer may also wish to consult with the owner for maximum shipping lengths for railroad cars.

Single installed DPG or TPG also need to be designed and temporarily braced at the bearing locations to

## C2.10

The erection of deck plate girders (DPGs) should be in assemblies with the cross-frames and lateral bracing installed. Erecting DPGs in braced assemblies eliminates long unbraced compression flange lengths and provides girder stability during erection and after placement on the bridge seats.

Through plate girder (TPG) spans also should be erected in pairs. If possible, this should be done with the floor beams and knee braces installed, or with a minimum number of floor beams and knee braces installed, reducing the TPG compression flanges' unsupported length. A As with DPG spans, erecting TPG spans with floor beams and knee braces installed improves girder stability during erection and after placement onto the bridge seats.

Installing DPG and TPG spans in braced assemblies reduces the concerns of long unbraced compression flanges and lessens the concerns of wind loads and the potential of a lateral-torsional bending failure of the girders.

The use of self-propelled modular transporters (SPMTs) or lateral slide construction methods can be used effectively to reduce track outage time, if site conditions allow. For both methods, the entire steel superstructure can be built off to the side of the existing and then moved into place after the existing structure has been removed.

avoid a lateral-torsional bending failure of the girder. The temporary bracing needs to remain in place until the installation of all the design bracing.

# SECTION 3. GIRDERS

## 3.1. Stiffeners and Connection Plates

### 3.1.1. Orientation with Respect to Flanges

Stiffeners and connection plates for I-girder bridges should typically be oriented normal to the flanges.

In haunched girders, the stiffeners should be normal to the top flange.

Stiffeners should not project past the flange edges.

### C3.1.1

The difference between bearing stiffeners placed to be vertical after dead load or normal to the flanges is usually minimal, particularly for simple spans.

Keeping connection plates and stiffeners normal to the flanges avoids the need to bevel the ends of the stiffeners and connection plates.

### 3.1.2. Connection Details

#### 3.1.2.1. Bearing Stiffeners at Flanges

For the stiffener bearing end (almost always the lower end at the bottom flange) connected to the flange, the stiffener end should be finish-to-bear plus optional fillet welds.

For the connection to the top flange, finish-to-bear is typically unnecessary. Welding the stiffener to the top flange is necessary if there is a diaphragm or cross-frame connected to the bearing stiffener or where recommended by AREMA Chapter 15.

#### C3.1.2.1

Using finish-to-bear rather than a complete joint penetration (CJP) weld significantly reduces welding deformation of the bottom flange compared to a CJP weld and costs less. The fillet weld maintains the bearing contact during subsequent operations and is typically made at the minimum required size per the AWS D1.5 *Bridge Welding Code* (henceforth AWS D1.5).

#### 3.1.2.2. Intermediate Stiffener and Connection Plate Connections

Do not use bolted tab plates to connect the stiffener or connection plates to the flange.

The stiffeners should be clipped at the upper and lower ends to clear the weld connecting the flange plate to the web, or the rolled beam fillet, as applicable.

#### C3.1.2.2

Bolted tab plates are discouraged because the live load stress range at the surface of the flange is approximately equal to the live load stress range on the web at the weld's termination; therefore, replacement of a welded connection with a bolted connection will not improve the fatigue resistance of the girder. Bolted tab plates are more costly than welding a stiffener directly to the flange. Additionally, it is expensive to clean and paint the contact surface before installing the tab; also, the connection will be a source of dirt build-up and corrosion on the flange for weathering steel applications. See further discussion of bolted tab plates in SBC G12.1.

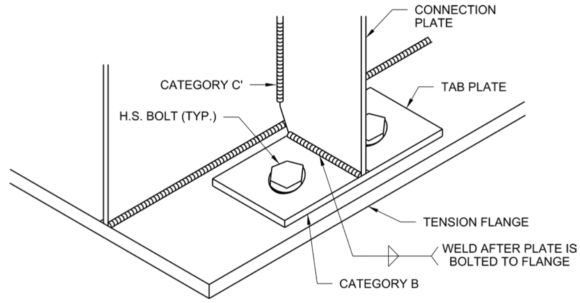


**Figure 3.1.2.2 – Bolted Tab Plate (NOT RECOMMENDED)**

### 3.1.2.3. Stiffeners at Webs

Stiffeners should be connected to the web by fillet welds.

### C3.1.2.3

Using bolted angles to connect stiffeners to the web carries many of the same concerns as bolted tab plates for flange connections (see section 3.1.2.2).

### 3.1.3. Minimum Spacing between Adjacent Stiffeners or Connection Plates

Provide 8-inch minimum spacing or 1½ times the stiffener or connection plate width for welding access. In the case of skewed stiffeners or connection plates, the spacing should be measured from the closest edge of the plate and not necessarily from the plate's intersection with the web; more space will be required than for stiffeners perpendicular to the web. See Figure 3.1.3.

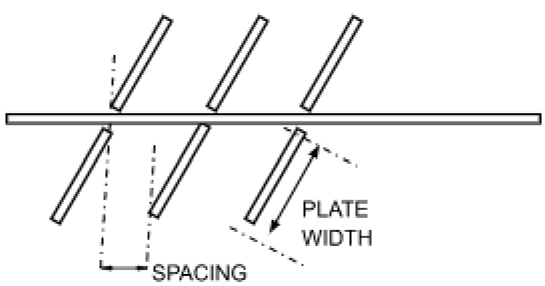


**Figure 3.1.3 – Skewed Stiffeners or Connection Plates**

### C3.1.3

This recommendation allows space for machine welding procedures to be used and applies to all transverse web stiffeners and connection plates.

Check with a fabricator for any other plates welded in very close proximity to each other to verify welding access.

Where multiple stiffeners are required over bearings, the bearing sole plate and masonry plate size and required connection bolts may affect where stiffeners can be located. There must be sufficient room to install bolts and replace the bearing in the future, as well as to inspect the connection during routine maintenance activities.

If jacking stiffeners are provided for future bearing replacement or adjustment, those may require close spacing to be positioned over the substructure, but access must be sufficient for welding the stiffener to the flange and web.

When multiple bearing stiffeners or jacking stiffeners are used along with cross-frames or diaphragms, the designer should evaluate whether there is space for the cross-frame or diaphragm to swing into place if multiple bearing or jacking stiffeners are present.

Additional stiffeners running transversely between main stiffeners (and parallel to the girder length) complicate welding access and create pockets prone to collect debris and corrosion, so they should be avoided.

## 3.2. General Details

### 3.2.1. Haunched Girders

On long-span continuous bridges, consider the use of haunched girders to optimize structural efficiency. If possible, keep the haunch within one shipping piece.

**C3.2.1**

On continuous bridges, haunches can be used at intermediate piers to optimize the handling of moment demand by the superstructure. Varying the depth of girders to make haunches adds cost, but on longer spans, this design optimization offsets this premium. Keeping the haunch within one shipping piece helps control girder fabrication cost and also simplifies field connections.

The simple-span structures that are common in freight rail structures do not need haunches unless it is a special condition of the nature of the project, such as when intermediate spans in steel tower viaducts are adjacent to shallower (shorter) tower spans.

#### 3.2.1.1. Haunch Shape

When using haunches, use a parabolic shape for aesthetics.

**C3.2.1.1**

Among haunch arrangements, parabolic haunches are considered the most aesthetically pleasing. In fabrication, producing the slight curve of the parabolic haunch is similar in cost to other shapes and possibly less than using a straight taper.

#### 3.2.1.2. Longitudinal Splice

For haunches greater than about 12 feet deep, use two stacked girders. To splice the girders, use a bolted longitudinal sub-flange splice (Figure 3.2.1.2.a) and avoid splices the girders with splice plates (Figure 3.2.1.2.b). Sub-flanges facilitate fabrication and shipping and also stiffen the girder in service.

**C3.2.1.2**

When haunched girders get very deep, fabrication cost goes up significantly, and shipping becomes a constraint. Longitudinal splices should be considered for haunches deeper than twelve feet due to material availability and shipping constraints. Sub-flange splices, versus sandwich-splicing the web plate, are easier to erect, stiffen the sections for shipping, and provide longitudinal stiffening in the final structure. The welded connection of the sub-flanges to the webs is a fatigue detail that must be considered in design.

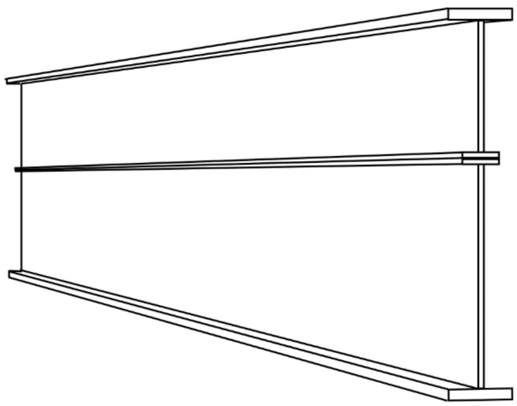

**Figure 3.2.1.2.a – Longitudinal Web Splice Using Sub-Flanges (preferred)**

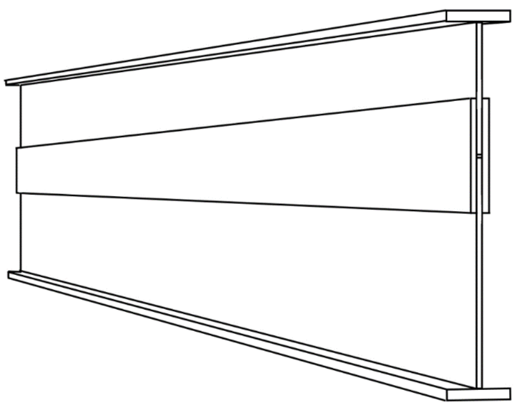

**Figure 3.2.1.2.b - Longitudinal Web Splice Using Splice Plates (not preferred)**

#### 3.2.1.3. Bearing Detail

At the bearing, keep the bearing area at least 12 inches long and use a 5-foot minimum radius for the parabola at the point of transition.

#### C3.2.1.3

The minimum 12-inch bearing length is intended to provide enough flat area to receive the sole plate.

Use of a radius where the bottom flange transitions from the flat bearing area to the curved haunch allows the transition to be made by either bending or a shop welded splice and thus allows the fabricator to choose the most cost-effective option.

### 3.2.2. Curved Girders

Permit either heat curving or cut curving of girders at the fabricator's option, with consideration of AASHTO *LRFD Bridge Design Specifications* heat-curving design limits.

#### C3.2.2

Curved girders are not used in freight railroad bridges but are sometimes used in transit bridges.

Fabricator preferences vary between cut-curving and heat-curving to produce curved I-girders.

Heat curving helps optimize material but takes more time and space in the shop.

There are practical limits to what can be effectively heat curved, and AASHTO also has design limits about what can heat curved. Practically, when flanges are very thick or radii are too small, girders cannot be effectively heat curved. AASHTO's limits are tighter than the practical limits and are based on concerns of introducing high residual compressive stresses into the girder webs.

### 3.2.3. Cross-Frames and Diaphragms

For cross-frames and diaphragm guidance, refer SBC G12.1 and G1.4, *Guidelines for Design Details,* as supplemented or excepted below.

### C3.2.3

Generally, the factors affecting constructability of railroad bridge cross-frames and diaphragms are the same as those for highway bridges, which are the primary focus of SBC G12.1 and G1.4.

AREMA Chapter 15 Article 1.11.4 limits the angle of cross-frame diagonal with the vertical to 60 degrees. See Figure C3.2.3. This is consistent with the SBC G12.1 recommendation to limit use of cross-frames to a 30-degree minimum angle between the horizontal and diagonal.

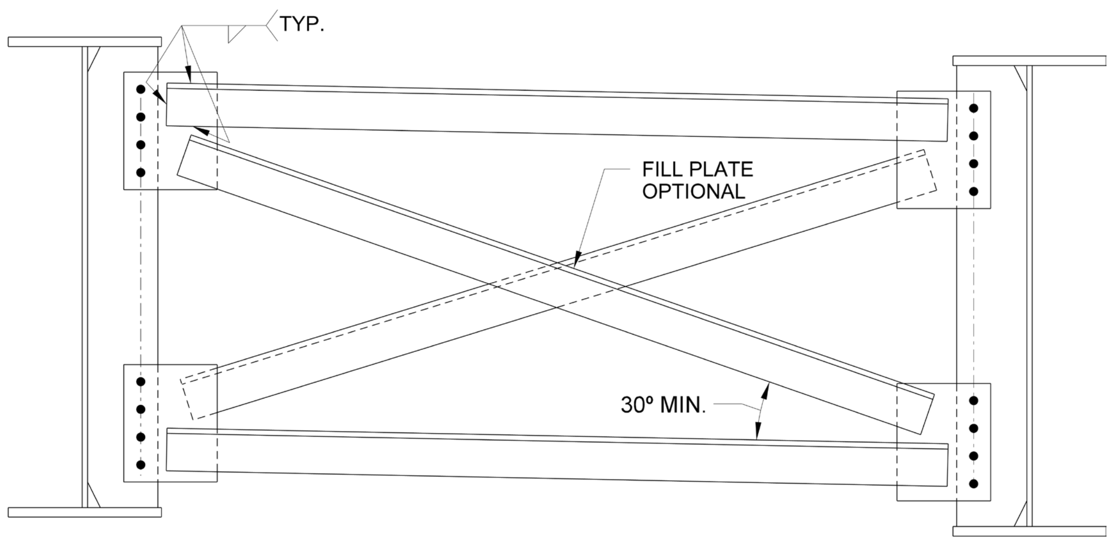


**Figure C3.2.3 – Intermediate Cross-Frame**

### 3.2.3.1. Uniformity

Keep cross-frames and diaphragms as identical as reasonably possible along the bridge.

### C3.2.3.1

Keeping cross-frame and diaphragm members identical gives the contractor more flexibility in the field, saving the time and effort needed to match unique members to unique locations.

### 3.2.3.2. Sealing

When cross-frames are to be galvanized, use seal welds at the fillet welded connections.

### C3.2.3.2

In bridge welding, it is customary to hold back fillet welds from plate edges; typically, the hold-back is about ¼ inch (owner requirements vary). However, when cross-frames are galvanized, the hold-back allows cleaning acid between the mating surfaces, and this acid often runs out of the joint over time, after galvanizing is complete, staining the galvanizing with rust. This stain is not deleterious but is undesirable. Sealing can trap air and therefore is not appropriate for small components. The American Galvanizers Association (AGA) provides limits online at <https://www.galvanizeit.com> for when sealing is acceptable. Usually, cross-frame sections are large enough to satisfy these limits, but these limits should be checked before sealing is prescribed.

## 3.3. Through Girder Details

### 3.3.1. Top Flange Terminations

Avoid rounded top flange corners.

### C3.3.1

Use of rounded ("bullnose") top flanges adds considerable time and effort to girder building, including:

- Processing the flange plate to introduce the curve. Usually this is a sublet fabrication step, adding time and shipping as well as cost.
- Additional butt splices to join the curved portion of the flange to the flat portion of the flange.
- Complicated bearing stiffener terminations, particularly on skewed bridges.

Figure C3.3.1 shows some of the operations needed to create a rounded top flange corner. The fabricator may need to use a combination of heat and force to make and fit the radius section of the flange.

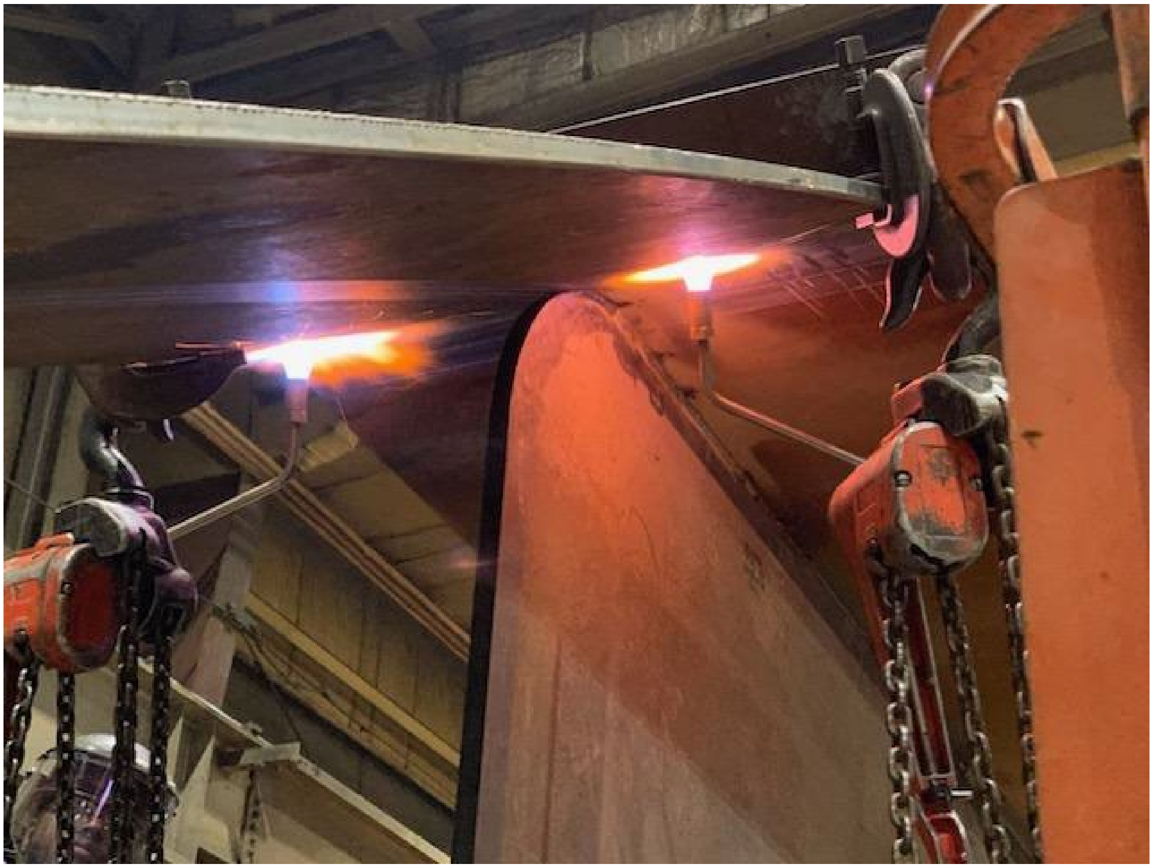

**Figure C3.3.1 - Fabrication of a Rounded Top Flange Corner** (High Steel LLC, 2023)

### 3.3.2. Flange-to-Web Welds

Use of fillet welds rather than complete joint penetration (CJP) groove welds is recommended where AREMA Chapter 15 provides a choice.

### C3.3.2

Compared to fillet welds, CJP groove welds add steps and significant fabrication steps to girder building, including the following:

- Beveling: The web must be beveled to produce the CJP weld, involving cutting and then (usually) grinding.
- Multiple passes: With CJP welds, there will be three to five times more passes (depending on the thickness of the web).
- Backgouging: Once the first side is welded, the fabricator must backgouge the other side to sound metal.
- Ultrasonic testing and associated repairs (if any): For a tee joint with the weld in shear, AWS D1.5 requires 25% of the weld to be tested ultrasonically.

### 3.3.3. Bracing of Top Flange of Through-Girders (Knee Braces)

Knee braces require special attention to detailing to prevent fatigue failure of various components.

Knee braces should not be welded to floorbeams or the transverse stiffeners.

### C3.3.3

Vertical connections are bolted to the transverse stiffener and horizontal connections are bolted to the floorbeam. Welding is not recommended at these locations because of fatigue issues associated with the connections.

Where the knee brace bottom flange plate meets the knee brace flange plate, the knee brace web plate should be coped to allow for proper termination of the welds.

Where a vertical deck plate (also known as vertical ballast plate) is used in conjunction with the horizontal deck plate, the attachment of the vertical deck plate to the knee brace flange plate can be either bolted or welded. If welded, the knee brace web plate should be held back from the connection between the knee brace diagonal flange plate to the knee brace bottom flange plate to minimize fatigue issues associated with that connection.

Suggested knee brace details are found in Figures C3.3.3(a) and (b).

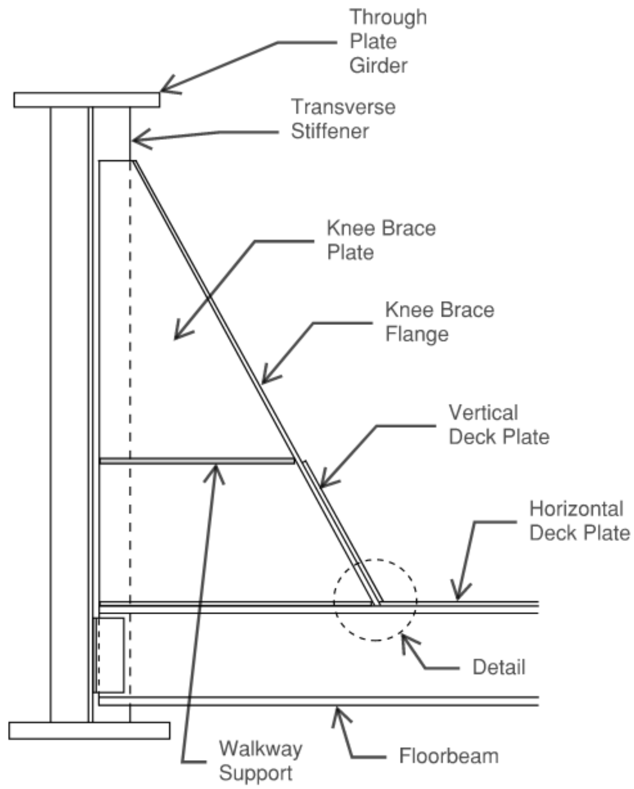


**Figure C3.3.3(a) – Example Knee Brace**

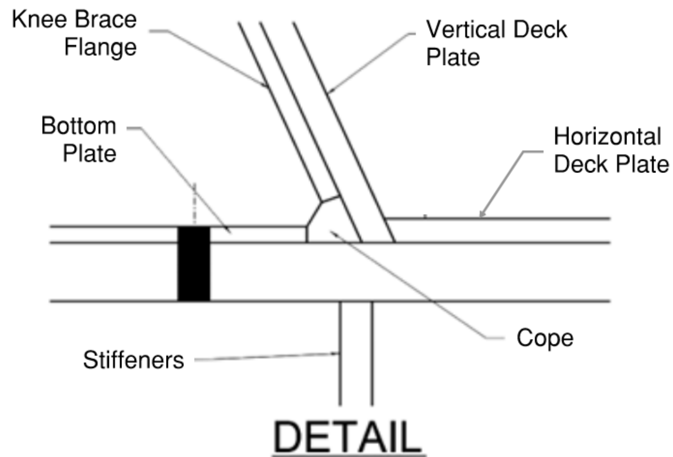


**Figure C3.3.3(b) – Knee Brace & Floorbeam Connection Detail**

### 3.4. Shop Assembly

For field connections, when numerically controlled equipment is used to produce bolt holes, AREMA Chapter 15 recommends a check assembly to be done if required by the Engineer. The Engineer should

### C3.4

The amount of shop assembly that may be needed depends on many factors, including the complexity of the structure, the relative flexibility of the members and their framing, the equipment consider a proposal from the fabricator to establish the number and type of shop check assemblies to verify fit of field connections.

used by the fabricator, and the type of bolt holes used (e.g., oversized or slotted). Fabricators are responsible for the fit of the steel in the field regardless of what the engineer requires, and check assemblies can be time consuming and costly. Therefore, it is prudent to consider the amount of check assemblies proposed by the fabricator.

# SECTION 4. BOXES

## 4.1. Tub Girder Bridges

Tub and box girders for railroad bridges are most commonly used in transit structures.

Follow the tub girder recommendation of AASHTO/NSBA Steel Bridge Collaboration G12.1, and the following:

- On heavily skewed supports, use one rather than two bearings per tub.
- In staged construction, consider external diaphragm fit-up between tubs that have all dead load applied and those that do not.
- Provide access for inspection.

## C4.1

Tub girders are a type of open-top box, usually trapezoidal, that is sometimes used in highway bridges. Such tubs are not typically used in freight railroad bridges but are sometimes used in transit railroad bridges. Generally, the detailing of highway tub bridges and transit tub bridges is similar.

On skewed bridges, the use of one bearing on each girder end is recommended because tubs rotate as they deflect as dead loads are applied. If there are two bearings at the end of the tub, the tub may rest effectively on the two bearings under steel dead load but then rotate as final dead loads are applied, lifting off one of the bearings.

For staged construction, exterior diaphragms that fit when both stages are deflected due to dead loads do not fit when the first phase is deflected due to dead loads and the second phase is not fully deflected due to dead loads. This deflection incompatibility during phased construction should be considered in design. If it is necessary to attach the second phase to the first phase before the second phase deflects, the exterior diaphragm should facilitate this movement.

## 4.2. Closed Box Sections

For closed box section details, follow the recommendations of AASHTO/NSBA Steel Bridge Collaboration G12.1.

## C4.2

Box sections are not used as often in railroad bridges as they are used in highway bridges.

On highway bridges, rectangular boxes are sometimes used in straddle bents. Such straddle bents are not typically used for freight railroad bridges but can be used for transit railroad bridges. A newer approach for straddle bents is to use a three-I-girder equivalent for straddle bents because they are redundant and avoid the need for in-service fracture-critical related inspections. These types of straddle bents are also more cost-effective and efficient to fabricate than closed box sections. See "Bent on Innovation" in the February 2021 issue of *Modern Steel Construction* for more details. The AASHTO/NSBA Steel Bridge Collaboration is currently working on a new publication, *Guidelines for Steel Bent Caps*.

# SECTION 5. TRUSSES

## 5.1. Truss Considerations

### 5.1.1. Type of Construction

A new truss may be constructed as a superstructure replacement placed on existing substructure or placed on completely new substructure.

### C5.1.1

When placing a new truss on existing substructure units, it is often undesirable to increase the load on the existing substructure. If the self-weight of the new truss or increased live loading is more than the existing substructure can tolerate, it may be necessary to strengthen the existing substructure and foundation through measures such as installation of deep foundations, underpinning, or ground improvement. Minimizing load on existing substructure may lend itself to an open deck structure with a lighter structure utilizing tension diagonals and a polygonal top chord.

A new truss on new foundations will require a significant footprint for the new foundations. Positioning of the new substructure units will greatly affect span lengths. New substructures in water may require temporary cofferdams for the drilling of caissons or placement of footings. This, combined with pier stability or scour mitigation applied to existing piers underwater, may require a significant separation between old and new substructure units. For new substructure, it is recommended that a bathymetric survey or similar be made to assess the potential for underwater obstructions as well as a subsurface evaluation to determine soil or rock bearing capacity.

### 5.1.2. Site Geometry Constraints

There may be obstructions above or below a truss that require the truss to be a specific height or use a shallow deck. These clearance constraints can be permanent in the truss's final position or temporary constraints that the assembled truss must accommodate during steel erection.

### C5.1.2

Examples of site geometry constraints that may impact the design of the truss include high voltage transmission lines, vehicular crossings below, or other bridges ahead or back station of the new structures.

### 5.1.3. Recommended Lengths for Various Truss Types

Span length is a significant factor in the selection of truss type. Fabrication is usually simplified when the truss top and bottom chords are parallel. However, as span length increases it becomes more favorable to

### C5.1.3

A Pratt truss is characterized by parallel top and bottom truss chords, with all diagonals sloping downward towards the center of the truss. See Figure C5.1.3(a). When a balanced live load is placed on the bridge, the diagonals are subjected increase the depth of the truss at midspan to be more efficient at carrying loads.

Pratt, Howe, and Warren trusses are typically used for span lengths that are 200 feet or less. Parker and camelback trusses are typically used for span lengths that are 200 feet or greater. Lattice trusses are not recommended for railroad use.

to tension forces; under partial live load, positive and negative panel shears occur, so diagonal members may be in tension or compression.

A Howe truss is characterized by parallel top and bottom truss chords, with all diagonals sloping upward towards the center of the truss. See Figure C5.1.3(b). When a balanced live load is placed on the bridge, the diagonals are subjected to compression forces; under partial live load, positive and negative panel shears occur, so diagonal members may be in tension or compression.

A Warren truss is characterized by parallel top and bottom truss chords, where the diagonals alternate sloping directions to create a sawtooth like pattern. In railroad bridges, Warren trusses typically have vertical member due to panel lengths for an efficient floor system. See Figure C5.1.3(c). When a balanced load is placed on the bridge, the diagonals are subjected to tension and compression forces; under partial live load, positive and negative panel shears occur, so diagonal members may be in tension or compression.

A Parker truss is characterized by a polygonal top chord that increases the depth of the truss at the center of the span. See Figure C5.1.3(d). Each segment of the top chord has a different slope moving towards the center of the span with the center panel having a horizontal top chord section. The diagonals slope down towards the center of the truss. When a balanced live load is placed on the bridge, the diagonals are subjected to tension forces; under partial live load, positive and negative panel shears occur, so diagonal members may be in tension or compression.

A camelback truss is characterized by a polygonal top chord that increases the depth of the truss at the center of the span. See Figure 5.1.3(e). There are exactly five different segments with sloped top chords. The depth of the truss increases towards the middle of the truss allowing for longer spans. The diagonals slope down towards the center of the truss. When a balanced live load is placed on the bridge, the diagonals are subjected to tension forces; under partial live load, positive and negative panel shears occur, so diagonal members may be in tension or compression.

A lattice truss is characterized by parallel top and bottom chords. The diagonals are closely spaced and alternate slope directions in a sawtooth fashion. When load is placed on the bridge, the diagonals are subjected to tension and compression forces. The diagonals are essentially composed of three trusses overlaid on top of each other, allowing for members that are typically smaller in cross-sectional area.

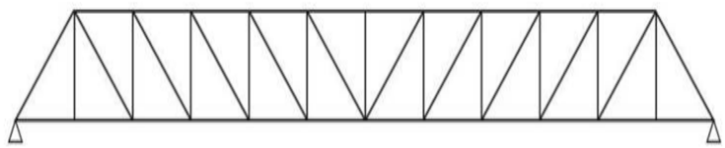

**Figure C5.1.3(a) – Pratt Truss Configuration**

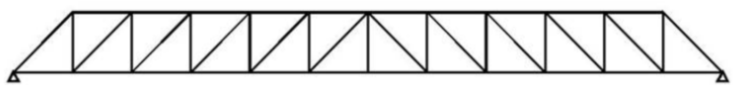

**Figure C5.1.3(b) – Howe Truss Configuration**

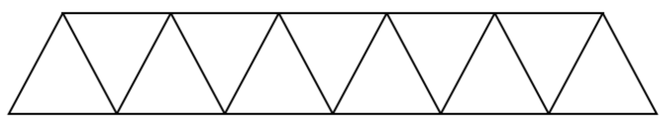

**Figure C5.1.3(c) - Warren Truss Configuration**

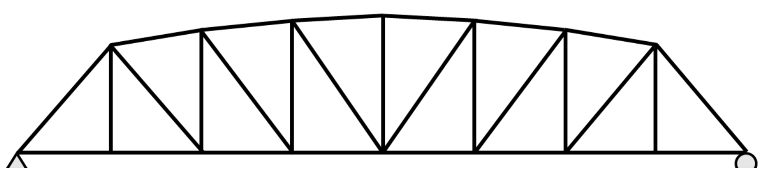

**Figure C5.1.3(d) – Parker Truss Configuration**

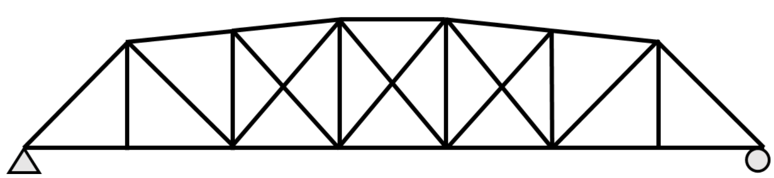

**Figure C5.1.3(e) – Camelback Truss Configuration**

### 5.1.4. Construction Sequencing

The construction and steel erection sequence of a truss can result in a critical load scenario that should be considered by the designer, and may affect the selection of truss type.

### C5.1.4

To erect a truss, it may be beneficial to support the truss at a single intermediate panel point or more. Intermediate support points may put diagonals in compression during erection when they are designed to function as tension members for the final in-service condition.

As an alternative, trusses are occasionally erected off-site and then moved into place.

Although it is beneficial for the designer to have some knowledge of erection of trusses, erection engineering of large trusses is complex and should be performed by an engineer with experience in this highly specialized field of bridge construction. See also sections 9.4 and 9.5.

#### 5.1.5. Diagonal Behavior

Depending on the type of truss that is selected, the diagonals can behave in compression, tension, or both. The consideration of accidental impact on the diagonals may lead to the usage of one truss type over another. A diagonal that is impacted resulting in an out-of-plane deformation will be subjected to secondary moments due to loading eccentricities, such as a P-delta effect. The risks associated with these secondary moments are considerably more detrimental for compression members than for tension members.

### 5.2. Truss Member

Truss members in steel railway truss spans are typically fabricated with welded plates forming built-up H-sections, built-up box-sections, or rolled sections. Rolled sections and H sections are much more economical than box-sections.

### C5.2

Built-up member sections typically use three welded plates to fabricate H-section (I-section rotated 90 degrees) members and four welded plates to create box-section members (top box flange, bottom box flange and two side or web plates). H-section and I-section members are usually more economical, but box-section members are often used for members carrying large live load forces and to increase the lateral-torsional buckling strength of compression members. Box sections normally have thicker side plates and thinner top and bottom flange plates. This is because the end connections are normally in the side plates only. This is certainly true for diagonals and verticals; however, end connections of chord members may engage all four plates. Top and bottom box flange plates in box-section members may be solid or have elliptical or ovaloid perforations to allow water egress and reduce weight. Box-section members with solid flange plates should have hand holes in the bottom flange plate at the end of members for access to bolted joint connections.

Built-up member plates should be adequately welded together.

Box-section corner fillet welds should be made at each side of the side (or web) plates at the bottom flange plates and at the outside at the side (or web) plate at the top flange plate. See SBC G12.1 for additional box-section corner details. The fillet welds used to connect the flange plates and web plates of H- and box-section members need to develop the member force and tensile force range over the length of the connection. Flange plate to web plate welds in box-section members and flange plate to web plate welds in H-section members may require large fillet welds or CJP welds to develop the weld within the connection.

When possible, members should be designed and detailed to avoid secondary bending stresses.

Bending stresses may be combined with axial stresses in truss members. Truss floorbeam hanger and sub-vertical members may be subjected to considerable in-plane and out-of-plane bending. The out-of-plane bending stresses are from floorbeam deformation (transverse frame behavior) and may be reduced by using narrow vertical members and deep floorbeams. In-plane secondary bending stresses in members may develop due to connection rigidity, member eccentricity, transverse force, and the deformation of chord members from the extension of vertical members.

Secondary stresses due to connection rigidity may be reduced with smaller, more compact gusset plates. Diagonal members may be trimmed to reduce gusset plate size but should be detailed such that most of the member width is perpendicular to the working line of the member for accurate bolt hole alignment. Secondary stresses due to truss distortion and connection rigidity are small for slender truss members and typically neglected for members with width, parallel to the plane of distortion, less than one-tenth of the member length. Secondary bending due to eccentricities are avoided by detailing joints such that the centroids of the member sections intersect at a common location and bolts are symmetrical about the neutral axis of members in the connection. Secondary bending may not need to be explicitly considered for usual truss design because localized yield stresses at the extreme fibers at some locations in a member will not precipitate failure.

Truss members should be fabricated to length to obtain the specified camber of the completed truss.

Truss members are fabricated short or long by defined dimensions to achieve the specified camber. AREMA Chapter 15 recommends that the camber of trusses to be equal to the deflection produced by the dead load plus a live load of 3,000 lb/ft of track.

Truss members within a single truss should be designed with the same or similar width.

Truss members should be designed with the same width (dimension normal to the plane of the truss) to avoid the use of fillers at gusset plate joints. This is achieved with built-up welded H- and box-section members by using appropriate plate sizes.

### 5.2.1. Truss Tension Members

Truss tension members are typically fracture-critical and nonredundant. Brittle rupture is made unlikely through appropriate material selection, design, detailing and fabrication quality control.

Tension members are designed against yielding on the gross section and fracture on the effective net section for strength.

Members that experience repeated tensile stress ranges or stress reversal are designed for allowable fatigue stress ranges on the effective net section for fatigue.

### C5.2.1

Fracture-critical members have specific material, fabrication, inspection, and testing requirements, which are given in the AREMA Chapter 15 recommendations.

Yielding typically occurs on the gross cross-sectional area of tension members prior to fracture at ultimate tensile stress on the effective net cross-sectional area. The effective net cross-sectional area for strength at a connection accounts for both the presence of bolt holes and the non-uniform distribution of stress across the tension member cross-section when the tensile force is not transmitted to all elements of the tension member at the connection (for example, a box-section with only the web plates, or an H-section with only the flange plates, bolted to the gusset plates). Longer connections are more efficient for strength against shear lag.

Cyclical tensile stresses and stress reversal are common in truss tension members due to moving train loads. The effective net cross-sectional area for fatigue of bearing-type bolted connections accounts for shear lag effects related to cyclical tensile stresses.

For bolted joints designed as slip-critical connections, the gross cross-sectional area of the connected member is used to calculate the fatigue stresses. Members with solid plates are designed for Fatigue Detail Category A or B. Perforated cover plate members are designed as Fatigue Detail Category C.

Hand holes in the bottom cover plate within gusset plate connections and tensile box-section members are typically designed as Fatigue Detail Category C and not a net section concern for strength and fatigue design of tension members.

However, where small open holes, perforations, slots, or other openings exist in a tension member, the allowable fatigue stress range may be severely reduced to Fatigue Detail Category D.

### 5.2.2. Truss Compression Members

Truss compression members are typically nonredundant built-up members susceptible to instability.

Stresses, determined from the prescribed load combinations, on the gross cross-sectional area should not exceed allowable compressive stresses.

Serviceability criteria should also be considered in truss compression member design.

#### C5.2.2

Box-section members with perforated cover plates are often used for built-up compression members. Deformations of the open section will reduce the overall stiffness and buckling strength.

The allowable elastic or inelastic compressive stress depends on the slenderness of the member. Nonlinear inelastic behavior occurs primarily from residual stresses but may also be a result of initial curvature and force eccentricity. Detail members without eccentricities that create instability. For combined axial and bending compression, interaction equations based on bending moment-curvature-axial compression relationships are used.

The slenderness ratio, L/r, should be limited to values that will preclude excessive secondary flexural compressive stress due to member curvature.

### 5.2.3. Portal Frames

The portal frame is designed to carry the transverse loads, such as wind, down to the bearings.

#### C5.2.3

The portal frame is a rigid frame or short truss member that is transverse between the upper chords of the two vertical truss planes. Two portal frames are typically used in a two-truss system and connect the first and last panel points of the truss. On a through-truss structure, the portal frame is a rigid frame composed of the two end posts and an upper horizontal member, either beam or truss, located at the top chords. The end posts may be either diagonal or vertical. A portal frame is located at each end of the span.

## 5.3. Truss Member Connections

Truss member connections are designed for the capacity of the member.

#### C5.3

Designing the connection for the capacity of the member ensures uniform strength in the member.

Truss member connections to gusset plates are designed for strength considering slip-critical allowable bolt shear stresses and allowable bearing stresses on the gusset plate. For more information on high-strength bolted connections, see section 7.

Truss member connections may be in tension or compression, or subject to stress reversal.

Truss member connections may be in tension, compression, or both due to moving live load.

Tension member connections are designed considering yielding of the gross cross-sectional area at the connection prior to fracture of the effective net cross-sectional area at the connection. The net cross-sectional area at the connection is determined at the potential tensile fracture line as the gross cross-sectional area at the connection with connection holes removed. The potential tensile fracture line depends on the bolt hole stagger (or pitch) and gage at the connection. A staggered bolt pattern may be used to minimize loss of section and maximize the net area. The effective net cross-sectional area accounts for the effects of stress concentrations and connection eccentricities (shear lag) at the connection.

At typical built-up box and H-section truss member connections there are elements in different planes and an effective cross-sectional area must be determined.

Truss tension member connections to gusset plates are designed to consider allowable yield, fracture, and block shear stresses at the connection. Member connections that experience repeated fluctuations in axial tension or are subject to stress reversal are also designed for allowable fatigue stresses.

Truss tension member connections to gusset plates designed for yield and fracture stresses over the Whitmore width promotes wider connections to the gusset plate. Truss tension member connections to gusset plates designed for block shear stresses often result longer connections to the gusset plate.

For truss members that are continuous through a joint, AREMA Chapter 15 indicates that in some circumstances shear lag may not be an issue of concern. Nevertheless, at joints with continuous chord members the shear lag factor is considered for the part of the tension force transferred from diagonal members to chord members. The allowable block shear strength is based on tensile fracture combined with shear fracture or shear yielding. When an axial member connection is subject to a net applied tensile stress range or stress reversal, the effective net cross-sectional area for fatigue is considered. For slip-critical bolted connections the effective net cross-sectional area for fatigue is the gross area of only the member elements that are directly connected. Bolts will generally not experience fatigue failure prior to the base metal and, therefore, AREMA Chapter 15 includes no recommendations concerning allowable shear stress ranges for bolts.

Truss compression member connections to gusset plates consider allowable compressive stresses at the connection.

Truss compression member connections to gusset plates designed for compressive stresses over the Whitmore width promotes wider connections to the gusset plate.

### 5.3.1. High-Strength Bolted Connections

The current preferred truss type connection is to use gusset plates connected to members with high-strength bolts rather than welds. The use of high-strength bolts usually makes steel erection and future repairs easier to address.

### 5.3.2. Gusset Plates

Gusset plates must be designed. Design gusset plates to be as compact as possible.

**C5.3.2**

Axial, shear, and block shear stresses transmitted to gusset plates from truss members are considered for typical gusset plate design. Gusset plate yielding and fracture may occur due to tensile axial stresses transferred to the gusset plate from tension members. Gusset plate yielding and fracture may also result from shear stresses transferred from truss members framing into the gusset plate. Gusset plate yielding and instability may ensue due to axial compressive stresses transferred to the gusset plate from compression members. Axial tension and compression stresses from truss members are transferred to gusset plates over an effective (or Whitmore) width. Block shear (tensile fracture combined with shear yield or shear fracture) failures in gusset plates, due to forces transferred from tension members, may occur. Gusset plate instability can also result from the formation of partial shear stress planes on the gross section of the gusset plate along vertical and horizontal planes adjacent to a compression diagonal member.

Compact gusset plates reduce plate material consumption and provide increased buckling strength with reduced plate slenderness ratios and free edge distances. Gusset plate geometry is often governed by diagonal member geometry (angle and the width of members at the connection).

Gusset plates that attach to fracture-critical truss members are also considered to be fracture-critical members.

Truss spans typically have floor systems supported by nonredundant trusses on each side of the track. Therefore, gusset plates connecting tension members in nonredundant trusses are fracture critical members.

The installation method of the bolts is to be considered in the design of the connection.

Closed truss members should have hand holes in the bottom flange (or bottom cover) plates at the connection to allow for the installation and tightening of high-strength steel bolts between the member and gusset plate.

Avoid eccentricities of member forces in the design of the connection.

Secondary bending stresses in truss members are avoided when all member forces meet at a common working point. If member forces all meet at a common working point, gusset plates at truss panel points connected to continuous chord members transfer the difference in the chord member forces across the panel point, which are the resultant forces of the vertical and diagonal members framing into the gusset plate. When continuous chord members through panel points are used, member splices in chord members should be made near the panel point in the least stressed member. Therefore, a common working point at each connection and a symmetrical arrangement of bolts in the continuous chord member about the centerline of the chord member are required.

Chord member splices should be incorporated into panel point gusset plate connections.

Gusset plates at truss panel points with spliced members may transfer a considerable proportion of the entire chord member force at the panel point through the gusset plate. Gusset plates in chord member splices at truss panel points are designed as components of the splice. Therefore, the chord member axial force eccentricity with respect to the centroid of the gusset plate requires consideration to determine stresses in the gusset plate.

### 5.4. Truss Shop Assembly

When numerically controlled equipment is used to produce bolt holes, AREMA Chapter 15 recommends a check assembly to be done if required by the Engineer. The Engineer should consider a proposal from the fabricator to establish the number and type of shop check assemblies to verify fit of field connections.

### C5.4

As with other types of bolted connections, fabricators produce holes either by drilling or reaming through the connection plies at the same time or by drilling individual components using computer numerically controlled (CNC) equipment. The fabricator may use one or a combination of these approaches depending upon the fabricator's equipment and the complexity of the bridge. See further discussion in section 3.4.

If the fabricator is using CNC equipment, then the fabricator may propose assembly of some or all of the truss parts in the shop to verify fit of the field connections. Prescribing shop assembly that is not needed adds time and cost to the project.

The fit condition of the truss, specified in the contract plans, affects which components can be assembled at one time. If the truss fit is, “cambered length, cambered angles” (a.k.a. no-load fit), then it is possible to include entire truss panels in assembly at one time. If the truss design fit is “cambered length, geometric angles” (a.k.a. “force fit” or “Chicago style fit”), then it is not possible to include both top and bottom truss chords with verticals and diagonals in assembly at the same time without force fitting. Rather, verticals and diagonals may be shop-assembled first to one chord and then, separately, to the other.

# SECTION 6. FLOOR SYSTEMS, DECKS, AND WALKWAYS

## 6.1. Decks

Steel bridges on freight railroads are almost always of open deck or ballast deck construction, and AREMA Chapter 15 design recommendations are limited in applicability to those two deck types. Open decks cost less than ballast decks up front, but as presented in the sections below, there are many factors to consider.

### 6.1.1. Open Deck Considerations

With open decks, railroad track loads are distributed from the rails to supporting steel members via transverse bridge ties supported directly by the steel members in question. Figure 6.1.1 shows an example of through plate girders with an open deck with the ties supported directly by the stringers.

With through girders, knee braces are commonly placed at every floorbeam or every other depending on the bracing spacing requirements. This style of deck requires a bottom lateral bracing system.

### C6.1.1

Open decks have been used since the earliest days of railroading and are still in common use on North American railroads. Physical characteristics of these decks are thus fairly uniform across the industry and are standardized on most railroads.

Bridge ties are typically pressure-treated timber, but synthetic materials may also be used. Bridge ties are deeper than traditional roadway ties for structural performance and so they can accommodate dapping around the girder flanges.

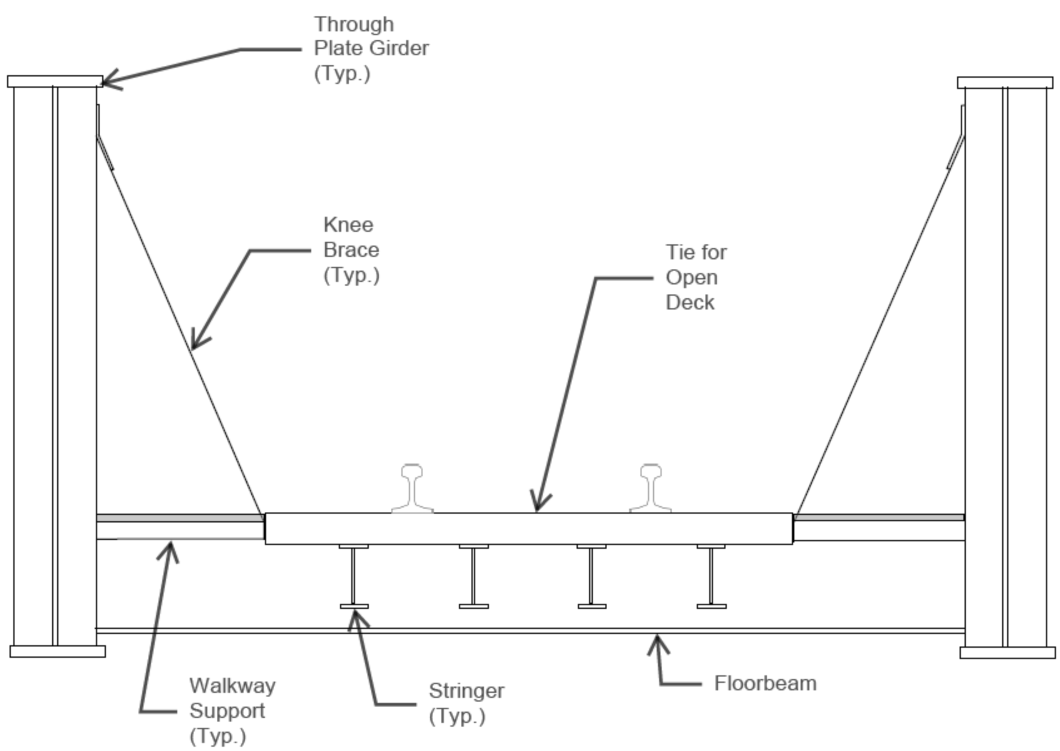


**Figure 6.1.1 – Open Deck Through-Girder Bridge**

Consideration should be given to the following issues related to design and construction of open deck steel spans:

- Impact loads as prescribed by AREMA Chapter 15 are higher for open decks.
- Open deck spans constructed on tangent alignment would, in theory, be centered about the centerline of track. In practice, design of such spans could be based on a nominal amount of track eccentricity to account for potential alignment shifts.
- AREMA Chapter 15 provides recommendations for flange-to-web connections affected by deck loads applied directly to the flange.
- Bridge ties are often notched, or "dapped" where they contact supporting steel members. Tie daps are typically about ½ inch deep, and in addition to preventing lateral movement or shifting of the deck they can account for minor cross-level or other irregularities between support lines. Depending on the thickness of the top flange and the depth of the dap, the bottom of the non-dapped portion of a bridge tie may conflict with bolts used for top lateral gusset connections.
- Recommendations for the special anchorage of ties to girders for moveable spans are given in AREMA.

### 6.1.2. Ballast Deck Considerations

With ballast decks, railroad track loads are distributed from the track surface (i.e., rails and transverse crossties) to supporting steel members through a layer of ballast spread on a deck supported directly on the steel members. Cross-ties are typically pressure treated timber, but may be concrete, steel, or other synthetic material. Decks must be pressure treated timber, engineered lumber, cast-in-place concrete, precast non-prestressed or prestressed concrete, or steel plate.

Steel deck plates can be attached to the supporting members with welds or bolts.

Ballast decks for through-girder bridges may use widely spaced floorbeams with two or more stringers to support the deck plate between floorbeams, with knee braces commonly placed at every other floorbeam (see Figure 6.1.2(a)), or closely spaced floorbeams (typically 18 to 36 inches) without

**C6.1.2**

Despite their higher cost to construct, ballast deck bridges are often preferable to open deck bridges because they offer certain advantages that can reduce long-term operational and maintenance costs. Preserving continuity of the approach embankment roadbed section on the bridge enhances uniformity of the track surface and stability of track line and grade. Furthermore, roadbed section continuity enables uninterrupted progression of track maintenance operations and equipment across the bridge.

stringers, with knee braces commonly placed every third or fourth floorbeam (see Figure 6.1.2(b)).

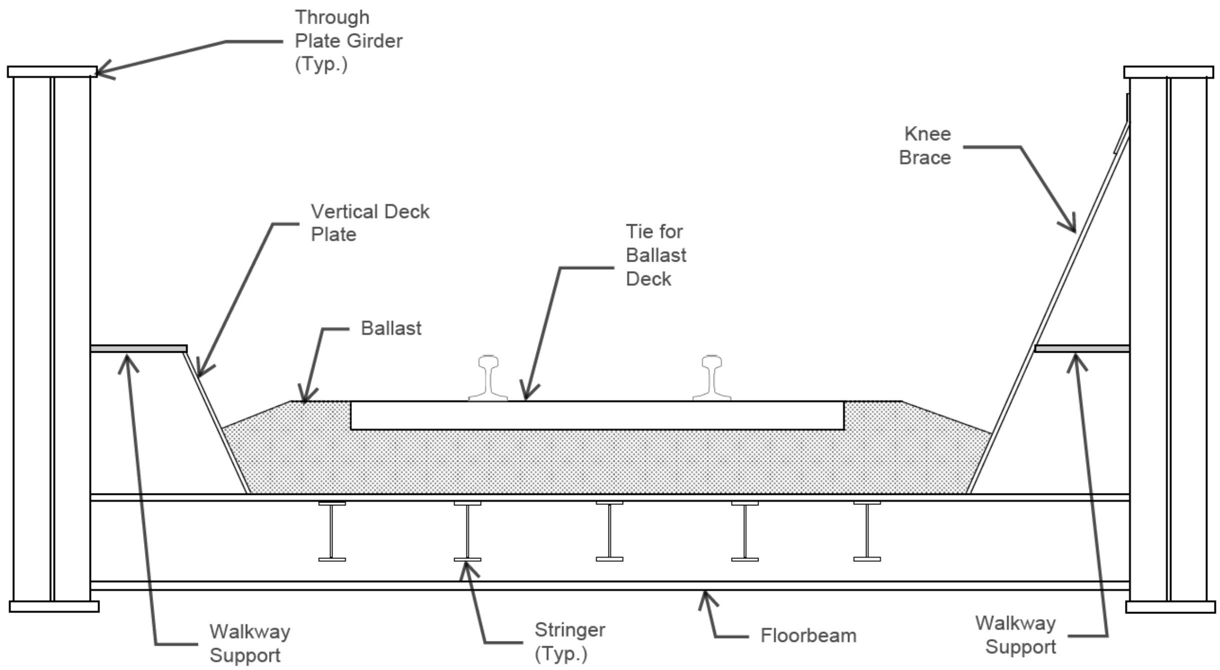


**Figure 6.1.2(a) – Through Girders with Ballast Deck and Stringers**

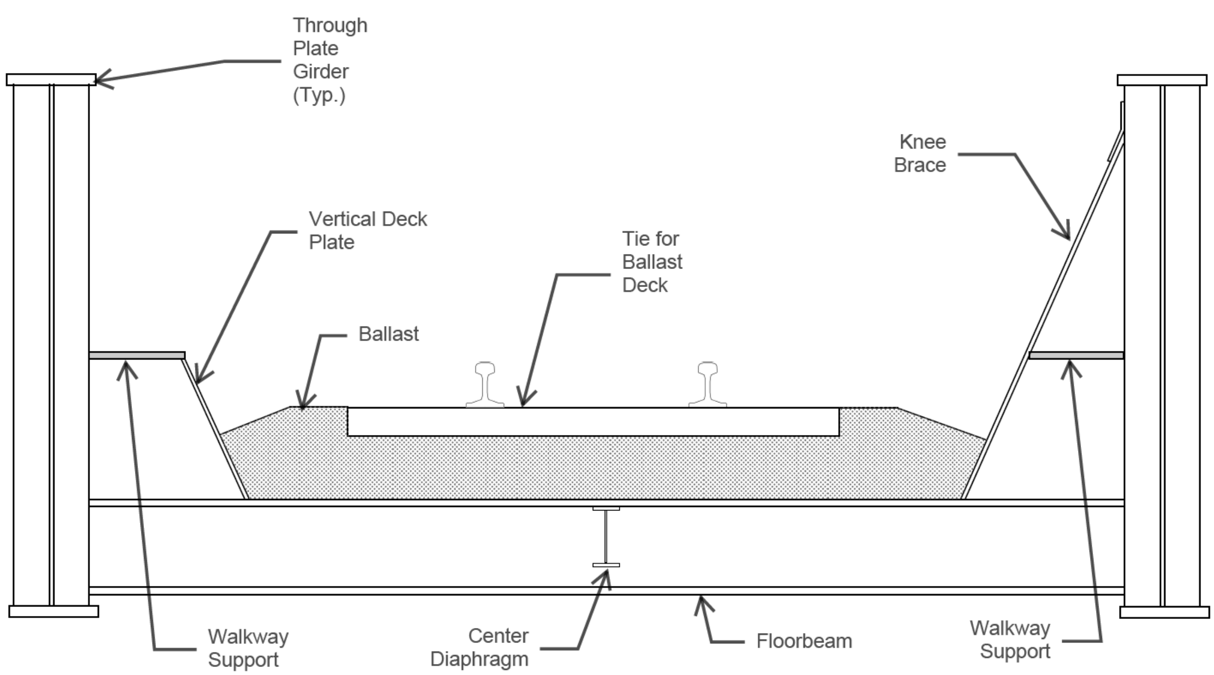


**Figure 6.1.2(b) – Through Girders with Ballast Deck and Without Stringers**

Widely spaced floorbeams and stringers are more economical, but closely spaced floorbeams without stringers can have a shallower structure depth.

Closely spaced floorbeams are only used with through-girder bridges.

For through-girder systems for single tracks and double tracks with track centers spaced closer than 20 feet, two girders are used. See Figure 6.1.2(c) for a two-girder example using the option of closely spaced floorbeams. For track centers that are wider than 20 feet, it becomes more feasible to use a shared center girder that accommodates the AREMA clearance window. This allows the floorbeam length to be reduced and thus structure depth reduced. See Figure 6.1.2(d) for a three-girder example using the stringer option.

When a shared center girder is used, sometimes one track will be fully assembled prior to the changeout for an accelerated span replacement. The remaining side can then be stick-built on a less demanding schedule.

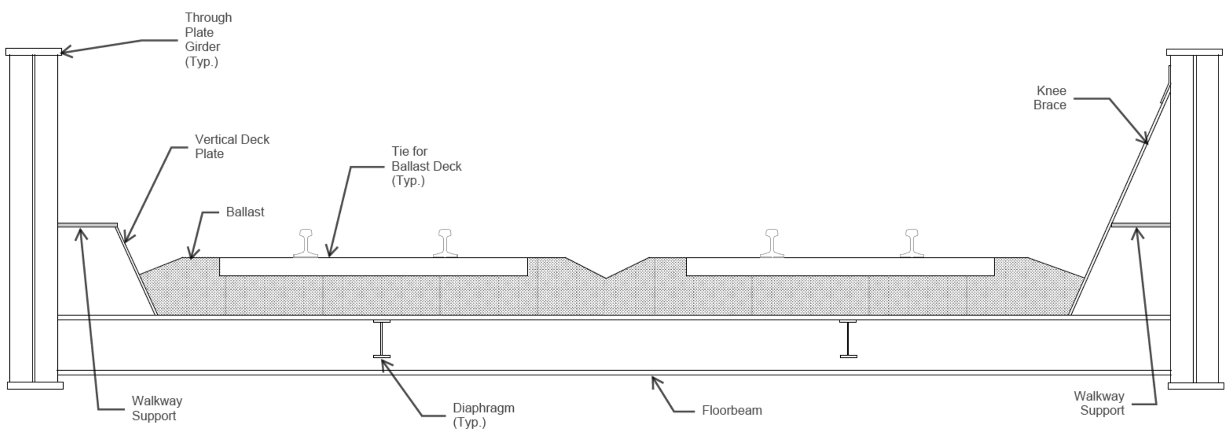


**Figure 6.1.2I(c) – Double Track Through-girder Bridge without Shared Center Girder**

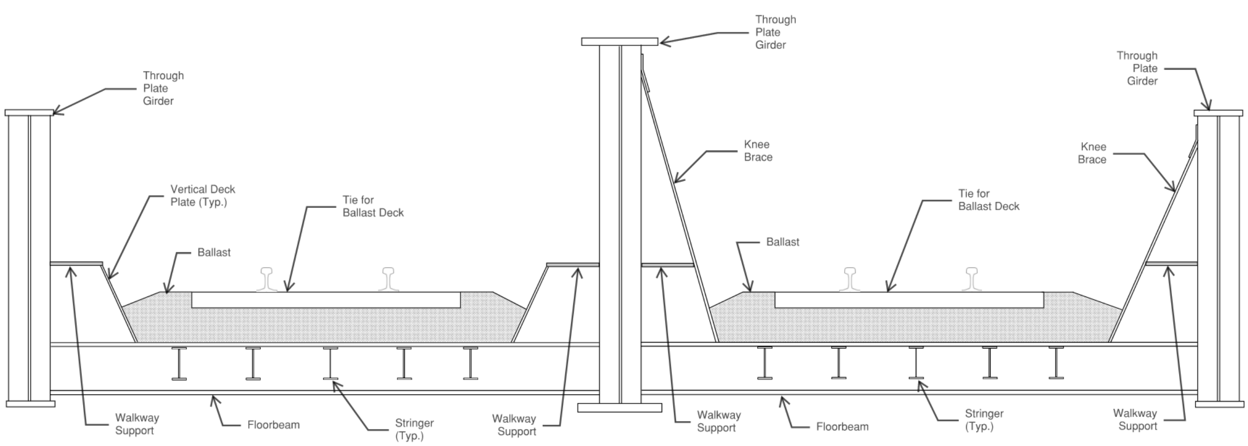


**Figure 6.1.2(d) – Double Track Through-girder Bridge with Shared Center Girder**

The left sides of Figures 6.1.2(a) and 6.1.2(b) show floorbeams with walkway support brackets. The right sides of the figures show floorbeams with knee braces. Knee braces and walkway supports are always matched for both ends of a given floorbeam. If designed for that purpose, the deck plate may be used as a large diaphragm to act as part of the lateral force resisting system. In such cases, bottom lateral bracing is not used.

Design considerations for decks include:

- Method of connecting deck plate to supporting steel members. If the deck is to function as a lateral force resisting system, connections should be designed and detailed accordingly.
- Provisions for waterproofing and deck drainage, including potential interplay with underlying steel framing.
- Span dead load and deck plate curb geometry may need to account for additional ballast.
- Allowance for eccentricity of live load due to potential track alignment shifts.
- Railroads may prescribe special derailment design load cases for decks.
- Detailing for panelized installation.
- Structural isolation of concrete deck pans that are located below the neutral axis of through plate girder spans.

**6.1.2.1. Steel Deck Plates**

Steel deck plates are the preferred option for ballast decks because they are the lightest and shallowest option. However, steel deck plates may not be practical with wide stringer spacing.

**6.1.2.2. Reinforced Concrete Decks**

Reinforced concrete deck can be used in lieu of deck plates.

Reinforced concrete decks are designed to span floorbeam to floorbeam and proportioned to carry both live and dead loads.

Reinforced decks can be affixed with shear studs along the centerline of the floorbeam or, less common, engage the top flange of the floorbeam to ensure that the concrete deck provides adequate transfer of the lateral loads. Note that the AREMA manual does not allow concrete decks below the neutral axis on through plate girder spans.

**C6.1.2.2**

Concrete decks are designed in accordance with the recommendations of Chapter 8 of the AREMA Manual.

The concrete deck cannot be used to provide strength to the floorbeam through composite action. Composite action can be used to reduce deflection. If studs are used to affix the concrete deck to the floorbeam, they should be designed for either composite action or to transfer lateral and longitudinal forces from the deck to the floorbeam. Note that some railroads do not allow design for composite action, even when shear studs are used.

Concrete decks are designed with protection boards and waterproofing membrane of sufficient thickness to prevent ballast from penetrating the protection boards and the membrane waterproofing.

#### 6.1.3. Direct Fixation Considerations

Direct fixation (DF) decks are those that have a concrete deck (typically composite) atop the steel members, with tracks directly affixed to the deck via a plinth. A plinth is used to set final rail elevations after casting the deck. The plinth dimensions and pattern may vary for utilities, drainage, and other needs, but are usually constructed to act monolithically with the deck but have intermittent breaks in their longitudinal direction, thus not contributing to the composite section properties of the superstructure. This type of deck is most common in transit structures but may be used for freight structures.

The following are some design considerations for direct fixation decks:

- Live load impact loads included in AREMA Chapter 15 do not cover direct fixation track.
- Localized impact can approach double those used by typical AREMA formulas; it is generally correlated to speed and equipment details. Industry publications are available that discuss this in more detail.
- The detailing of DF clip system should typically include a compressible layer to dissipate extreme impact loads to the plinth, deck, and steel structure.
- The thermal and strain interaction between the steel rails, concrete deck, steel superstructure, and substructure fixity/stiffness should be investigated on multi-span structures.

A clear understanding of the fastener stiffness is required for this analysis. This may include consideration of a broken rail and low temperatures, as well as forces imparted by the superstructure deflection interacting with the rail (including stresses in the rail from that interaction).

The designer should consult with the owner's preferences, AREMA Chapter 8, Part 27, and AREMA Chapter 12, Part 4, prior to design of a DF deck structure. The designer should be aware of long-

Concrete decks should be designed as a trough with curbs on either side parallel to the through girders to retain the ballast.

term maintenance considerations associated with direct fixation.

### 6.1.4. Deck Plates

Deck plates (also known as ballast plates or floor plates) should be designed to carry all track dead and live loads and are required to be at least ½ inch thick.

Deck plates should provide for positive drainage of the bridge deck.

For ballasted deck bridges, the deck plates should be protected with a waterproofing membrane.

It is desirable to extend the vertical deck plate to the through girder web to prevent ballast and debris from falling below.

### C6.1.4

Steel deck plates should end near the centerline of a floorbeam and are welded to the floorbeam with continuous fillet welds.

## 6.2. Floor Framing Systems

Floorbeams and stringers are the principal load-carrying members of the floor system of through girder and truss bridges. The longitudinal stringers frame into the transverse floorbeams and the floorbeams, in turn, transfer the load to the through girders or trusses. Two or more stringers are used to transfer loads from the track to floorbeams.

Floorbeam and stringer decks can either be open deck (ties resting directly on the stringer) or closed deck (usually a ballasted steel or concrete deck). Less common and less economical are transverse floorbeams which are at close spacings and have diaphragms between the floorbeams. These may be used with ballasted decks on through girders. See discussion in section 6.1.2.

## C6.2

Refer to AREMA Chapter 15 Section 1.8 for more specific direction regarding floorbeams and stringers.

For through girder bridge floor systems, a floorbeam-stringer system with as wide a floorbeam spacing as practical is recommended. For trusses, the floorbeam spacing is dictated by truss panel points.

### 6.2.1. End Floorbeams

End floorbeams are found at the end of a through-girder or truss bridge. They can be perpendicular to the through girder or truss, or skewed with the bridge.

Where end floorbeams are used for jacking the span, they should be proportioned to transfer the span dead load to the jacking locations through their connections to the girder. Jacking stiffeners that are attached to the end floor beam will need to transfer concentrated jacking loads at the jacking points.

### C6.2.1

Floorbeams can serve as a means of lifting the bridge (e.g., bearing replacement) when the through girder or truss itself cannot be used for this purpose. AREMA Chapter 15 Section 1.8 gives guidance for proportioning of end floorbeams when used in conjunction with jacking of through-girders and trusses.

### 6.2.2. Floorbeams and Floorbeam Hangers

With the exception of end floorbeams, the floorbeams should be perpendicular to the through-girder or truss. The length of floorbeams should be designed to provide sufficient gap between face of connecting member and edge of floorbeam to accommodate field erection. Floorbeams for trusses frame into the truss panel point. For floorbeam-to-stringer connections, use readily available angle and use the same size angle for all connections

### C6.2.2

Angles associated with stringer or floorbeam connections are subject to AREMA Chapter 15 Article 1.8.3.

Floorbeam hangers (sometimes referred to as suspenders or hip verticals) are vertical tension members in a truss that are directly loaded by floorbeams (no other "truss action" is present due to diagonal members of the truss). Therefore, axial tension forces in floorbeam hangers are equal to the joint load at the truss lower chord of a through truss.

The axial tension forces in floorbeam hangers are primary forces. Primary bending stresses in floorbeam hangers may also occur due to out-of-plane "frame" behavior. Secondary bending stresses may occur due to in-plane effects (typically small and related to joint rigidity of gusset plates and truss deflection).

The first interior floorbeam in a Pratt through truss is supported by a floorbeam hanger. The forces in the vertical members of a Warren through truss are either zero-force or hanger members.

### 6.2.3. Connections

The minimum connection angle (see Figure 6.2.3) size recommended in AREMA Chapter 15 is L4×4, to ensure the proper edge distance for bolted connections is maintained. Larger angle sizes will be needed for stringers and floorbeams spanning longer than 10'-0". AISC also provides guidance on combinations of angle thickness and gage and associated flexibility.

### C6.2.3

Larger angles are needed for longer members to allow space for more bolts.

At end connections in solid deck or open deck floor construction for stringers, AREMA Chapter 15 recommends the top third of the end connection to have a wider gage line to ensure the flexibility in the connection angle. Welding is not permitted for connection angles since they induce fatigue in the flexing leg.

AREMA Chapter 15 Article 1.8.3 provides further guidance for the design of connection angles for stringers and floorbeams.

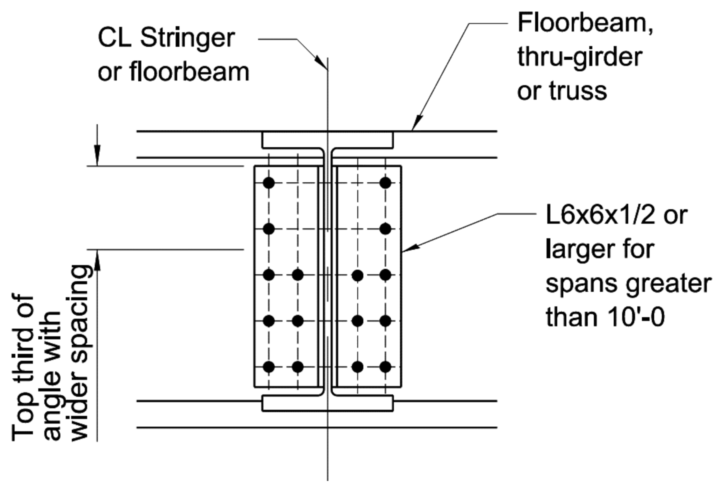


**Figure 6.2.3 – Connection Angle Detail**

### 6.2.4. Bottom Lateral Bracing

Use of an oversize hole in one ply of the lateral bracing connection facilitates fit-up. It is better to make the holes in the brace oversize instead of the holes in the gusset plate so that all the holes in the gusset plate are the same size (the other side of the gusset will be bolted to the girder). All connections using oversized holes are required to be slip-critical.

Bracing should be located at the top surface of the bottom flange.

**C6.2.4**

Bottom lateral bracing is found on through-girder and truss spans, and on deck girder spans, if needed.

Placing the bracing above the bottom flange, as opposed to under the bottom surface of the flange, increases clearance and facilitates erection. Typically, the connection is made using a gusset plate.

## 6.3. Walkways

The requirements to include walkways and handrails will be specified by the owner.

**C6.3**

Recommendations for walkway and handrail clearances and minimum dimensions are provided in AREMA Chapter 15 Section 8.5. Owners' requirements are typically based on these recommendations.

Some considerations for walkways are:

- Walkway type should lend to easy installation, inspection, and maintenance.
- Consider local environmental factors such as prevalence for ice/show accumulation.
- Walk surfaces need to provide a secure footing grip. Consideration for ballast and debris accumulation is appropriate.
- Fastening systems and corrosion protection should be adequate for the railroad environment.

If the intent of the walkway is to allow personnel to traverse past a train stopped on the bridge, then walkways and handrails on the outside of the through plate girder span may be required.

For walkways inside of through plate girder spans, consider whether the top flange height relative to the walking surface is adequate to meet fall protection requirements, or if supplemental handrail height is needed.

Further considerations for location and layout:

- Lateral clearances to handrails for train operations should be confirmed.
- If walkway is provided on one side of the bridge only, consider factors such as inspection, access, stream flow, and debris maintenance when setting the orientation.
- Locations of switches and other track features may require the use of additional walkways or larger work platform areas.
- Walkway layout on through structures should provide sufficient space to move around a train, if possible, or otherwise be provided outside the structure.

Further consideration for redundancy:

- When selecting walkway members, eliminate potential for failure of any one piece to cause a fall hazard as much as possible.
- Continuity over supports and using multiple narrower walk surface members are two methods of providing this added layer of safety.

# SECTION 7. BOLTS

## 7.1. High-Strength Bolts

ASTM F3125 is the specification for most high-strength structural steel bolts. Grade A325, A490, and F1852 bolts are options listed in AREMA Chapter 15 for structural steel joints. The use of Grade A325 bolts is currently preferred in the industry. ASTM A3148 is a newer specification for bolts that has been added to the 2023 edition of AREMA Chapter 15.

## C7.1

AREMA Chapter 15 recommends allowable stresses for bolts in slip-critical connections only. The designer must make an informed decision when choosing bolt size and grade to use. There are subtleties in the design and installation of each size, grade, and type of bolt, including the number of times the bolt can be reused and their pre-tensioned loads. Most railroad bridge designers use ASTM F3125 Grade A325 Type 1 galvanized or Type 3 bolts. The most common bolt diameter used is 7/8 inch, but longer span trusses may require the use of 1 inch diameter bolts to reduce connection dimensions.

The *Specification for Structural Joints Using High-Strength Bolts*, published by the Research Council on Structural Connections (RCSC), addresses the installation and inspection of the bolted assemblies and provides design guidance that may be useful. The RCSC Specification addresses ASTM F3125 Grade A325 and A490 bolts and their twist-off versions, Grade F1852 and F2280 bolts; and ASTM F3148 bolts. The use of F2280 twist-off bolts is not included in AREMA Chapter 15 due to concerns about decreased ductility and low ratio of ultimate to yield stress. Additional references are available and discuss industry preferred practices, such the AISC *Steel Construction Manual* Tables for Entering and Tightening Clearance. See AREMA Chapter 15 for further commentary on the use of A490 bolts.

ASTM F3125 and F3148 bolts are each available in two types, denoting chemical composition: Type 1 and Type 3. Type 3 is a weathering steel bolt intended for use in uncoated applications, and Type 1 is non-weathering. A490 bolts cannot be galvanized, as they become susceptible to stress corrosion cracking and hydrogen embrittlement, but alternative coatings have been developed and listed in ASTM F3125. Bolts other than A490 can be galvanized, though it is uncommon to galvanize Type 3 bolts.

Because the bolts look the same except for the markings on their heads, the designer should avoid

Switching bolt grade or size at a common field splice location for adjacent girders or truss members would require reverification of the tightening method and, if calibrated wrenches are using two bolt grades of the same dimensions (similar length and same diameter) within the structure.

used, either recalibration of the tightening equipment in the middle of the erection process, or the use of separate wrenches calibrated for each bolt grade and size.

The designer should also consider the possible tightening methods and ensure sufficient access is provided for workers, tools, and inspection.

AREMA Chapter 15 provides recommended practice concerning preparation, reaming, and drilling of holes for structural connections; testing (preinstallation verification and rotational capacity) and installation of high-strength fasteners, including provisions for reuse; and surface preparation of faying surfaces for slip-critical connections.

### 7.2. Mechanically Galvanized or Hot-Dipped Galvanized Bolts

Where galvanized fasteners are required, either hot-dipped or mechanically galvanized bolts can be used at the contractor's option when both options are available in the applicable ASTM specification.

### C7.2

Hot-dipped galvanized bolts may give better corrosion protection, but mechanically galvanized bolts are often considered to have more consistent coating thickness and fewer installation problems; hot-dipped galvanized bolts are more likely to fail rotational capacity testing.

Only mechanical galvanizing is permitted for ASTM F3148 bolts. Galvanizing of ASTM F3125 Grade A490 bolts is not allowed, although ASTM F3125 does list some alternative coatings available for these fasteners.

### 7.3. Uncoated Versus Coated Shop-Installed Fasteners

For areas that will have been primed prior to bolt installation, but also for areas that will later be blasted and primed, allow the option for either black (uncoated) bolts or galvanized bolts.

### C7.3

For shop-installed fasteners on coated structures, fabricators usually coat the connection before bolting, and later bolt up the connection with coated fasteners. However, there are situations where it is preferable to install uncoated ("black") fasteners into assemblies that are not pre-coated. For example, if a girder has a bolted tab plate on a painted bridge, the fabricator may prime the mating surfaces of the tab and girder, bolt the tab on with uncoated fasteners, and then later blast and paint the entire girder, including the tab plate and bolt.

Uncoated fasteners need to have oil removed before blasting. Also, bolts are often installed in situations where some parts of the bolt or nut may be shielded during blasting, resulting in an inadequate anchor profile (surface roughness). Blasting of coated bolts may not remove all of the coating, but the prime coat will adhere to any remaining coated surface. When galvanized nuts are used, consider the use of partially lubricated nuts to decrease cleaning time and use of solvents prior to coating. ASTM A563 Supplemental Requirement 3 gives an option for lubricant placement only on the bearing surface and internal threads. See further discussion on selection and coating of high-strength structural fasteners in AASHTO/NSBA Steel Bridge Collaboration document S8.1, *Guide Specification for Application of Coating Systems with Zinc-Rich Primers to Steel Bridges.*

Most owners have no specific requirement for bolts in coated structures, and assume that fabricators and erectors will use uncoated bolts. Some owners require galvanized or other coated bolts for painted structures, especially for field connections in new steel. Field blasting of uncoated bolts installed in shop-primed structures is likely to cause damage to adjacent primed areas. This damage may go undetected and unrepaired, shortening the life of the coating system. The use of coated fasteners on painted bridges eliminates this potential.

### 7.4. Bolted Faying Surfaces

AREMA Chapter 15 includes four classes of slip coefficient. Design plans should specify the class of slip-critical connections as Class A, B, C, or D in the contract documents. Ensure that any specified coating is compatible with the defined slip coefficients.

### C7.4

Pretensioned connections transfer load through friction between faying surfaces developed through the tension in the bolts clamping them together. In slip-critical connections, the capacity for this load transfer is calculated.

In design of slip-critical connections, a certain surface condition factor, or slip coefficient, is assumed based on faying surface classification. Faying surface classification with higher slip coefficients may result in more economical connection designs (i.e., fewer bolts).

The slip coefficient for a given surface condition or coating is determined by tests described in Appendix A of the RCSC *Specification for Structural Joints Using High-Strength Bolts*. The test regime includes a short-term test to attain the slip coefficient and a long-term creep test to ensure the surface will not compress and decrease the bolt tension. The coating type, surface preparation, and coating thickness are considered essential variables in the test.

Certain surface types, such as clean unpainted mill scale, unpainted blast-cleaned steel, certain unsealed thermal spray coatings, and hot-dip galvanizing, have well-established slip coefficients that are listed in AREMA Chapter 15. Paints are classified, typically by the manufacturer, on a per-product basis through certification testing in accordance with RCSC Appendix A.

For many years, the available slip classes were Class A and B for paint or bare steel and Class C for hot-dip galvanizing. Unblasted bare steel is Class A and blasted bare steel is Class B. The slip coefficients for Class A and B are 0.30 and 0.50, respectively, and Class B has often been recommended because of the larger number of bolts needed for a Class A connection. The slip coefficient for Class C is also 0.30. More recently, a Class D with slip coefficient of 0.45 was introduced. This allows the use of a much wider range of zinc-rich primers, without a considerable sacrifice in the required number of bolts. Testing for slip coefficients is done on a primer alone and rather than a full coating system, and the faying surfaces are then masked for subsequent coats. Testing establishes the maximum thickness that the coating may be applied on the faying surface.

During shop drawing review, the Designer verifies that the surface preparation and the coating type and thickness, if applicable, are appropriate. The submittal of the coating manufacturer's product data sheet during the shop drawing submittal may further demonstrate or confirm the coating has been tested and meets the minimum slip coefficient for the requested type of faying surface. The Designer should also verify that any required masking is shown on the drawings.

Because thermal spray coating (TSC), otherwise known as metalizing, is not a manufactured coating material but rather metal wires that may be produced for a variety of applications, such certificates of slip coefficient are not usually available for thermal spray. However, research has shown that unsealed TSC surfaces readily meet a Class B slip coefficient. On the other hand, these surfaces are often sealed with an additional coating to fill the pores of the metalizing and enhance corrosion resistance. It has been demonstrated in research that sealed TSC does not reliably meet Class B, and faying surfaces are typically masked if sealer is specified.

### 7.5. Installation Methods

AREMA Chapter 15 includes five installation methods. Any of these will produce sound connections if performed correctly, and no one method should be specified unless so required by the Owner. Similarly, where direct tension indicators (DTIs) are used, no specific DTI placement should be specified, but rather any of the four DTI configurations should be permitted.

### C7.5

The RCSC Specification shows four configurations with the DTI under either the nut or the bolt head, and with either the nut or the bolt head as the turned element, along with any hardened washers that may be required for each configuration.

# SECTION 8. CORROSION PROTECTION

## 8.1. Uncoated Weather Steel

For most applications, use uncoated weathering steel for corrosion protection. When desired for aesthetics, clean fascia surfaces to SSPC-SP 6 to facilitate a uniform appearance both initially and over time.

## C8.1

The use of uncoated steel has a long history of proven success in railroad bridges. This includes both bridges composed of uncoated weathering steel, and also many early bridges that were at first painted but have continued to perform well even when the original coatings have failed. Guidance related to proper use of weathering steel is provided in the NSBA Steel Bridge Design Handbook, Volume 19 *Corrosion Protection of Steel Bridges* (NSBA, 2022b), and the NSBA *Uncoated Weathering Steel Reference Guide* (NSBA, 2022c).

Use of uncoated weathering steel saves cost avoiding both the initial cost of cleaning and painting and in-service costs associated with coating maintenance.

If uncoated weathering steel is not blast-cleaned, mill scale will remain on the steel and, depending upon the environment, may remain in place for decades. An SSPC-SP 6 cleaning will remove mill scale.

## 8.2. Highly Corrosive Environments

For environments that are highly corrosive, one of the following can be used:

- Paint, using a zinc-primer based system, or
- A combination of galvanizing and, for members too large to galvanize, metalizing (also known as thermal spray coating)

## C8.2

In bridge service, "highly corrosive" is typically considered to be environments where the steel will be directly exposed to a high level of salt. These may include bridges over salt water and bridges subject to significant salt spray, such as bridges over highways that have both high traffic volumes and high deicing salt usage.

Bridge painting changed fundamentally in the 1970s with the advent of (1) better surface preparation, typically blast-cleaning to SSPC-SP 10, before the steel is painted, and (2) use of paint systems that includes zinc primers. The most common zinc primer-based system is a three-coat system including either an organic zinc (OZ) or inorganic zinc (IOZ) primer, an epoxy intermediate coat, and a urethane topcoat. Other systems include the use of IOZ by itself and some newer two-coat systems. Zinc primer systems applied to bridges cleaned to SP 10 have superior coating performance compared to predecessor systems. There are bridges that were painted with this system in the 1970s that are still performing well.

Primers are typically applied in the shop, but there is a mix of practice regarding application of midcoat and topcoat in the shop or in the field. Applying the subsequent coats in the shop allows for shorter painting time and less project intrusion in the field. However, the following concerns should be considered when applying the subsequent coats in the shop:

- Fabrication time is increased. If the additional coatings are applied in the shop, the use of OZ primer is preferred to facilitate throughput because the second coat can be applied as soon as the OZ dries, which is typically within a couple of hours. However, a second coat can only be applied over IOZ after the IOZ has cured, which can take a day or more depending upon temperature and humidity.
- For either primer, the primed faying surface is masked before the subsequent coats are applied in the shop; in the field, additional coats are applied after bolting, so masking is not needed.
- Damage during transport and construction to shop-applied topcoats, particularly acrylics, requires field touchup, which can be extensive if the steel is not handled carefully.
- Field-applying a subsequent coat over an IOZ primer allows the IOZ to cure longer. With shop-applied midcoats and topcoats, there is little performance difference between IOZ and OZ primers.

The processes for applying galvanizing and metalizing are very different from each other, but both coatings consist of a metallic layer. Metalizing lends itself to spraying large, flat areas, as are common for long deck girders. Galvanizing lends itself well to coating smaller but more complex items that can be a challenge to spray with metalizing. However, galvanizing is limited to what components can fit in the galvanizing kettle (see the American Galvanizers Association website at <galvanizeit.org> for hot dip limits). Therefore, a combination of metalizing for large, open girders and galvanizing for smaller, complex diaphragms is most effective when metallic coatings are desired

(SBC S8.2). For sealed metalizing systems, the faying surfaces are masked prior to application of sealer. (See discussion of faying surfaces in section 7.)

Maintenance costs of all coating systems can be minimized if an inspection is performed after the first year of service. Many coating problems will be more visible after a year of service than immediately after application, and catching and addressing them early can enhance overall performance of the coating system for the remainder of the bridge's service life.

### 8.3. Detailing

In the design plans, include a general note sheet or section for cleaning and painting requirements. In the design plans, clearly identify the following areas:

- blast-cleaned
- painted
- paint hold-back
- no paint

#### 8.3.1. Ponding

Avoid details that result in the ponding of water on the bridge. When such details cannot be avoided, ensure that weep holes are included for draining, and use large enough weep holes that they will not easily become clogged with debris.

**C8.3.1**

Ponding should be avoided regardless of the corrosion solution.

#### 8.3.2. Faying Surfaces

When the bridge is coated, require coating of the faying surface but ensure that the specified coating will provide the design coefficient of friction. If the bridge is not coated, the faying surfaces should be blast-cleaned to an SSPC-SP 6 finish or better.

**C8.3.2**

As found in AREMA Chapter 15, the coefficient of friction and associated friction design criteria vary among coatings. See section 7.4 for further discussion of slip-critical connection classes.

Special curing times may be needed for zinc primers to be applied to faying surfaces. If so, this information will be found in the zinc manufacturer's slip coefficient certification. These curing times will not affect work painted in the shop that is bolted in the field, but they can affect the fabrication cycle time when shop bolting.

#### 8.3.3. Stain Mitigation for Weathering Steel

When desired for aesthetic reasons, staining of concrete support structures can be mitigated using

**C8.3.3**

Weathering steel does rust to some extent, and therefore the rain that runs off weathering steel bridges stains the concrete substructure. Drip bars drip pans or drip bars, or by wrapping the concrete substructure during construction.

and drip pans can significantly reduce this staining. See SBC G1.4 for a recommended drip bar detail.

# SECTION 9. CONSTRUCTION

## 9.1. Construction Contracting Methods

This section presents the most common types of contract delivery methods and their applicability in the railroad industry. It is not an all-encompassing list, as there are many ways to administer construction contracts and new methods continue to be introduced. Each method has pros and cons associated with it, and there are many different variations of each type covered in this guide.

As applicable, the following may be appropriate to address in the contract;

- Procedures to be followed by the GC to minimize interference with the movement of trains where the structure is being erected under traffic.
- Requirements that may apply to interference with waterborne traffic including appropriate coordination with the governing waterway agency; fines for interference can be large and assessed on a daily basis.
- Condition of company equipment lent to or used by the GC.
- Provisions for the length of time work train or engine service is to be furnished to the GC, addressing excess time.
- GC responsibility for loss or damage to materials, for all damage to persons or property, etc., caused by the GC's operations during the progress of the work.
- Contractor responsibility to obtain permits for the location and construction of the structure.
- Requirements for the GC to comply with federal, state, and local laws, regulations, and ordinances.
- Requirements that the GC protect the railroad against claims on account of patented technologies used by the Contractor on the work.

Some railroads have their own fabrication shop or construction crews and handle some or all of the bridge construction using their own forces. If so, there may be a delineation between what the railroad forces will perform and a what a contractor will perform. Up-front communication and specifications to accommodate and coordinate this work among the railroad owners, engineers, fabricators, erectors, and general contractors is recommended. It is advisable

**C9.1**

The key metrics of cost, schedule, quality, safety, and owner tolerance for risk are affected by the chosen delivery method.

There are numerous studies on various contracting methods that examine the schedule, risks and costs associated with each type of project delivery method. Many of the studies have been published online.

that when steel is fabricated in house by the railroad, that these procedures still be followed where the railroad acts as its own GC.

### 9.1.1. Design-Bid-Build

Design-Bid-Build (DBB) is the most common and familiar method of project delivery used currently in the railroad industry. In this method, the owner issues a contract to a designer for design services (unless the design is done in-house, which is less common). Once the design package is complete, the owner advertises the package for Contractors to bid. The Owner selects a qualified bidder and issues a separate contract for construction.

### C9.1.1

DBB is well suited for a variety of project types and sizes. This method provides a well-defined scope of work at letting, and this well-defined scope provides contractual protection to both the owner and contractor.

There are two key items to consider with DBB. First, the DBB method typically takes longer to complete from project initiation through the end of construction and final acceptance of the work than the other, less traditional methods. Second, the owner assumes some risks that other methods may avoid, such as permitting and schedule delays, unforeseen site conditions, and changes in the scope of work associated with design errors or omissions discovered during construction.

### 9.1.2. Design- Build

The Design-Build (DB) method of project delivery is not as common as DBB but is occasionally employed by some railroad owners. With DB, the design and construction are done under the same contract, with the designer working for the contractor or as part of a joint venture with the contractor. The owner prepares a Request for Proposals with a scope of work requirements and specifications to be met and may include a conceptual design. The invited contractors and their designers then develop their proposals for the owner to evaluate. Use of this approach allows the owner to manage one contract with a single point of responsibility for both design and construction. It may also speed overall project delivery by having construction begin while some of the design is still underway, whereas with DBB, construction does not begin until after the design is complete. Usually, a certain amount of design work is required by the RFP to be part of the proposal; therefore, DB teams are often provided with a stipend as compensation for this design work. During design, DB teams are encouraged to solicit input from fabricators regrading design constructability.

### C9.1.2

The DB selection process typically includes evaluation of contractor qualifications, with experience in DB, cost, quality control, railroad experience, and safety being the major categories that are scored in the RFP. Scoring categories are weighted by the owner depending upon the owner's interests and values. Once the contractor and designer are selected, the owner's involvement consists of verification of progress, issuance of payments, approval of track outages, and other such considerations.

In DB, the contractor acting through the designer is responsible for design, but there are times when the owner may be responsible for a design change such as the discovery of undisclosed hazardous material.

Contractors assume much of the risk under DB. As such, the up-front costs may be higher with DB, but the projects using this method are usually completed more quickly than is typical with DBB delivery.

Section 6 of the NSBA Guide "Accelerated Steel: Achieving Speed in Steel Bridge Fabrication" provides best practice recommendations for steel bridge fabrication in DB projects.

### 9.1.3. Construction Manager/General Contractor (CM/GC)

The CM/GC delivery method is a hybrid version of the DBB and DB project delivery methods. Under this method, the owner remains in control of the process and independently hires the design team and the contractor. This process consists of two phases: pre-construction services and construction. In preconstruction, when the design is complete, the construction manager/general contractor prices the work on the project to provide a guaranteed maximum price, usually using an "open book" cost estimate. In the construction phase, the project team performs the work under agreed-to rates and a fixed fee for construction, giving the owner more price certainty.

### C9.1.3

CM/GC usually requires more involvement from the owner than with other methods to make decisions related to the project. The contract is written to give the owner the ability to replace team members and also the option to bid out the project at the end of the design phase if the chosen contractor is not performing to expectations. A distinct time advantage associated with this method relates to permitting because permit procurement can begin as soon as the design is sufficiently complete.

### 9.1.4. Price Plus Time and Multi Parameter

These types of contract delivery methods are a modified version of the DBB process that takes into account a contractor's qualifications, in addition to construction schedule and any other items deemed important to the owner for a particular project. The proposals are scored on the various parameters set forth by the owner. The scoring weight for contractor qualifications, schedule, etc., is determined by the owner. Typically, cost still accounts for a large portion of the overall score.

Price plus time is sometimes referred to as "A+B".

Multi-parameter contract models may include incentives/disincentives and quality.

### C9.1.4

This method is best suited to complex and time critical construction projects and allows the owner to evaluate contractors for a project best value which may not be the lowest cost. This type of contract can be particularly useful for rehabilitation work where the full scope of the needed work is not understood until the contractor is engaged and working.

Alternates are frequently added to the proposals. If the owner is uncertain that a portion of work can be done within the set budget, this portion of work can be separated from the main scope and evaluated as an alternate price proposal.

## 9.2. Procurement, Handling, Delivery, and Storage

This section covers general best practices for the procurement, handling, delivery, and storage of steel components that compose a superstructure. It is not intended to cover every situation that may arise but does recommend best practices from a railroad industry perspective.

### 9.2.1. Procurement Purchase Order

For each of the contract methods, fabricated steel is typically procured by the general contractor (GC). As part of the procurement process, the fabricator will submit shop drawings for approval by the Engineer of Record.

### C9.2.1

Procurement of fabricated steel is typically the responsibility of the GC because the GC is in control of the overall project schedule. As such, the GC and fabricator work together to optimize the overall project schedule. In some cases, usually for emergency situations, railroads procure fabricated steel themselves. Owner-procured fabricated steel usually results in faster delivery of fabricated steel. If bid times are constrained, as is often the case with this type of procurement, fewer fabricators may be able to respond to the bid request.

### 9.2.2. Handling of Material

Softeners should be used with rigging in handling to avoid damaging the material and, if present, paint.

If needed, lifting points should be designed by an engineer and adopted into a lifting plan.

### 9.2.3. Delivery and Storage

Girders should be stored on dunnage, supported to maintain the no-load condition. Girders must be anchored for safety in the yard and should be supported such that material is off the ground and protected from standing water, mud, and other yard debris. Fasteners must be kept in sealed containers to avoid contamination from rain. Water can cause corrosion and compromise lubricant on the fasteners and thereby preclude proper tightening of the fasteners.

The GC should provide timely sequence, and orientation information to the fabricator so delivery arrangements can be made and required permits can be acquired.

## 9.3. Quality Control (QC) Testing

Refer to AREMA Chapter 15 and specific project contract documents for QC testing recommendations.

## 9.4. Construction Means and Methods

Construction means and methods are traditionally determined by the Contractor. The contractor should develop detailed work plans outlining how work will be conducted, including calculations for any temporary structures needed to facilitate construction. Owner requirements will provide a starting point when developing means and methods. Additional requirements may apply if third party organizations are impacted by construction. A recommended sequence of work or construction staging drawings may be included as part of the design package.

Consideration should be given to the condition of the existing structure when modifying for construction of the new structure. Temporary works, such as shoring, temporary bents, strengthening of existing members,

**C9.4**

Almost all railroad projects are done under rail traffic, replacing existing structures. This poses challenges that are unique to the rail industry. A rapid replacement of the structure is the goal of owners to minimize impacts to rail traffic.

The replacement substructure is usually constructed around and through the existing structure, although other methods are used as well. Under a short track outage, the existing structure can be removed and the new structure is set in place on the new or existing foundation elements. Shorter spans can be constructed in a single track closure window, but longer span lengths may require a phased replacement approach. The overall length of time of track etc., should be used to allow safe passage of train traffic during construction. It is recommended that these systems be designed by an engineer familiar with the requirements of railroad loading.

### 9.4.1. Construction Engineering

Structural aids, such as falsework or erection towers (see section 9.5.4), used to facilitate construction are to be designed and sealed by a professional engineer.

All anticipated design loading that may be reasonably assumed to act on falsework needs to be considered and members designed in accordance with AREMA Chapter 15, AASHTO, or AISC. Unless specified by the owner, there should not be any decrease in the factors of safety provided by Chapter 15. Wind loads should be per Chapter 15 unless otherwise specified by the owner.

The design of steel members and connections of structural aids should be per AREMA Chapter 15, as applicable. Modifications of or connections to the existing or proposed structure used in the plans should be analyzed and detailed in the erection drawings. SBC S10.1, *Steel Bridge Erection Guide Specification,* has useful guidance regarding steel bridge erection engineering.

outage is dependent on many factors and the owner's needs.

It may also be possible to build the new structure adjacent to the existing structure and then line the track over to the new bridge. This has many advantages over an in-line replacement. This method can significantly reduce or eliminate impacts to rail traffic. It can often reduce project schedule. Because the new structure is located away from rail traffic, normal construction industry production rates on work can occur without interruption due to traffic. The main savings to an owner is in the form of uninterrupted or minimally affected revenue and service and a reduced construction schedule.

Alignment issues due to fabrication tolerances or errors have been decreasing as fabrication methods become more advanced. Thermal expansion and contraction of steel can still cause alignment issues, especially on larger spans. The mean temperature range during fabrication when compared to erection should be noted. Erection during temperatures close to the temperature range during fabrication can alleviate most alignment issues. Steel members can be cooled with water, but it is impractical to evenly heat large members.

## 9.5. Temporary Works

### 9.5.1. Erector Qualifications/Experience

The level of knowledge, skill and experience needed to erect the fabricated steel varies with the scope of the project. The erector or contractor who erects the steel bridge should be qualified to do so.

### C9.5.1

Owners may choose to specify qualification requirements in the contract or require AISC erector certification.

### 9.5.2. Erection Drawings

Erection drawings are provided by the fabricator, usually as part of the shop drawing package, to the contractor to provide instructions for erecting the steel for items that relate to fabrication.

### C9.5.2

Erection drawings may include:

- A framing plan showing where piecemarks are to be installed
- A bolt list indicating required bolt quantities, types, diameters, lengths, orientation, installation method, and, if applicable, coating
- Piece weights
- Field weld locations and requirements
- Shipping groups for various pieces
- Length from centerline of bearing to centerline of bearing
- Shear stud sizes and locations

### 9.5.3. Erection Plans

Erection plans prepared by the general contractor or contractor's engineer are detailed, engineered, plans and instructions describing the erection of the structures.

Erection plans should be prepared with sufficient time for familiarization by construction inspectors and facility owners. Erection plans should be prepared by a professional engineer or other responsible person-in-charge.

The contractor should designate a professional engineer or responsible person for all erection plans so that any required field changes can be reviewed and approved by a person knowledgeable of with the erection plans drawings and calculations.

For erection plans that rely upon commercially available equipment or parts, detail sheets of those systems should be provided with the submittal, with the applicable details/parts used highlighted for clarity.

Typical erection plan review times should be included in contract documents, so that a reasonable submission schedule can be planned. For most

### C9.5.3

Guidance about erection plans, their scope, and their use can be found in SBC S10.1.

Erection plans differ from contract plans in that erection plans convey the details of erection, temporary supports, formwork, or other items determined by the engineering, including materials, methods, and work sequencing.

Erection plans may include erection calculations, lifting points of girders, girder stability provisions, etc., that are sealed by a professional engineer. The need for sealing the erection plans and the level of detail required for erection depends on Owner preference, local jurisdiction requirements and the scope of erection.

railroad projects, the initial submittal should be made three months before the planned work.

Erection plans should include load charts and product details to allow reviewers to check their applicability and allow inspectors to quickly know what was specified.

### 9.5.4. Falsework

The contractor, typically working with a consultant, is usually responsible for designing and constructing safe and adequate falsework which provides the necessary rigidity, supports the loads imposed, and produces a finished structure to the dimensions shown on the project plans and established by the owner.

Falsework includes erection towers, cofferdams, etc.

### C9.5.4

This section is only intended to provide guidance on erection falsework, not concrete formwork. The AASHTO *Guide Design Specifications for Bridge Temporary Works* (AASHTO, 2017) are an excellent reference on this topic.

The term "falsework" means the temporary support of structures. The term "formwork" means the mold used to support wet concrete and immediate bracing thereof. The cost of falsework engineering and drawing production should typically be included in the contractor's cost for doing the work.

### 9.5.5. Other Aids and Tools

Developments in design and construction tools or changes to means and methods may result in increased safety and production.

### C9.5.5

Newer technologies for design and construction are always being developed. It is recommended that the owners, designers, and contractors explore the various options in the market to evaluate their benefits to future projects.

Advanced CAD programs allow the building of 3D models of structures at a low cost. Such models allow the erector the ability to plan their work sequence and find potential issues or interference problems prior to start of steel assembly. These models can also be used for pick plans which are especially helpful with complicated steel structures.

Equipment such as electric torque or angle control wrenches are also changing steel erection as these tools are adopted. This leads to increased quality, safety, and production during erection, and reduced cost.

Fabricators have used these and other tools as they modernize the fabrication process which has led to reduced fabrication time, resulted in higher quality control and a reduction in labor costs.

If the contractor wants the fabricator to provide connections for lifting devices, details for these connections should be provided early in the

Providing details for lifting devices to the fabricator early facilitates the best fabrication cost and schedule. For example, it is better to put fabrication process, while shop drawings are being prepared, so that the lifting device connections can be made during normal fabrication. Lifting device connections are subject to the approval of the engineer.

lifting device bolt holes in top flanges while the girder is being built than drilling the holes into the top flange after the girder is fabricated. The fabricator can drill holes while the girder is being built when the fabricator has early details.

#### 9.5.6. Construction

Construction of temporary works needs to conform with the erection drawings. Deviation from the approved erection drawings should only be made with written approval from the responsible person for the erection drawings.

### 9.6. Demolition of Existing Structures

The demolition of existing structures should be planned similar to erection drawings, as described in section 9.5.2, with regard to safe design and construction.

### C9.6

Demolition plans may list maximum equipment weights and location restrictions (if applicable). Anything other than simple span bridges should consider potential bearing uplift during removal sequencing and countermeasures (through loading or resistance) to uplift if required.

Special consideration may be given to the loading of existing structures, equipment placement during demolition operations, and material offloading locations and equipment.

If removing a deck with equipment atop or under the span, care should be taken so that a large change in top flange (compression flange) bracing will not make the girder or span unstable under the proposed loads.

When partial removal is specified, sufficient planning should be done to leave the remaining portion of the structure undamaged. Retrofits may be necessary to ensure that any existing infrastructure on the remaining portion is supported to remain active.

Care should be exercised when removing a deck acting to brace the top compression flange as this has resulted in collapse during demolition procedures.

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# APPENDIX A

| Commonly Available Shapes | | |
|---|---|---|
| **Channels** | **Angles** | |
| C3 × 3.5 | L2 × 2 × 1/4 | L4 × 3 × 3/8 |
| C3 × 4.1 | L2 × 2 × 1/8 | L4 × 3 × 5/16 |
| C3 × 5 | L2 × 2 × 3/16 | L4 × 4 × 1/2 |
| C4 × 5.4 | L2 × 2 × 3/8 | L4 × 4 × 1/4 |
| C4 × 7.25 | L2 × 2 × 5/16 | L4 × 4 × 3/4 |
| C5 × 6.7 | L2-1/2 × 2 × 1/4 | L4 × 4 × 3/8 |
| C5 × 9 | L2-1/2 × 2 × 3/16 | L4 × 4 × 5/16 |
| C6 × 10.5 | L2-1/2 × 2 × 3/8 | L4 × 4 × 5/8 |
| C6 × 13 | L2-1/2 × 2 × 5/16 | L5 × 3 × 1/2 |
| C6 × 8.2 | L2-1/2 × 2-1/2 × 1/2 | L5 × 3 × 1/4 |
| C7 × 9.8 | L2-1/2 × 2-1/2 × 1/4 | L5 × 3 × 3/8 |
| C8 × 11.5 | L2-1/2 × 2-1/2 × 3/16 | L5 × 3 × 5/16 |
| C8 × 13.75 | L2-1/2 × 2-1/2 × 3/8 | L5 × 3-1/2 × 1/2 |
| C8 × 18.75 | L2-1/2 × 2-1/2 × 5/16 | L5 × 3-1/2 × 1/4 |
| | L3 × 2 × 1/4 | L5 × 3-1/2 × 3/4 |
| **MC** | L3 × 2 × 3/16 | L5 × 3-1/2 × 3/8 |
| MC18 × 42.7 | L3 × 2-1/2 × 1/2 | L5 × 3-1/2 × 5/16 |
| MC8 × 18.7 | L3 × 2-1/2 × 1/4 | L5 × 5 × 1/2 |
| | L3 × 2-1/2 × 3/8 | L5 × 5 × 3/4 |
| | L3 × 2-1/2 × 5/16 | L5 × 5 × 3/8 |
| | L3 × 3 × 1/2 | L5 × 5 × 5/16 |
| | L3 × 3 × 1/4 | L6 × 3-1/2 × 1/2 |
| | L3 × 3 × 3/16 | L6 × 3-1/2 × 3/8 |
| | L3 × 3 × 3/8 | L6 × 3-1/2 × 5/16 |
| | L3 × 3 × 5/16 | L6 × 4 × 1/2 |
| | L3-1/2 × 2-1/2 × 1/2 | L6 × 4 × 3/4 |
| | L3-1/2 × 2-1/2 × 1/4 | L6 × 4 × 3/8 |
| | L3-1/2 × 2-1/2 × 3/8 | L6 × 4 × 5/16 |
| | L3-1/2 × 2-1/2 × 5/16 | L6 × 6 × 1 |
| | L3-1/2 × 3 × 1/2 | L6 × 6 × 1/2 |
| | L3-1/2 × 3 × 1/4 | L6 × 6 × 3/4 |
| | L3-1/2 × 3 × 3/8 | L6 × 6 × 3/8 |
| | L3-1/2 × 3 × 5/16 | L6 × 6 × 5/8 |
| | L3-1/2 × 3-1/2 × 1/2 | L8 × 4 × 1 |
| | L3-1/2 × 3-1/2 × 1/4 | L8 × 4 × 1/2 |
| | L3-1/2 × 3-1/2 × 3/8 | L8 × 4 × 3/4 |
| | L3-1/2 × 3-1/2 × 5/16 | L8 × 6 × 1 |
| | L4 × 3 × 1/2 | L8 × 6 × 1/2 |
| | L4 × 3 × 1/4 | L8 × 6 × 3/4 |

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