



Steel Bridge Design Handbook

.....

CHAPTER 5

Selecting the Right Bridge Type

February 2022



.....
**Smarter.
Stronger.
Steel.**

© AISC 2022

by

American Institute of Steel Construction

*All rights reserved. This book or any part thereof must not be reproduced in any form without the written permission of the publisher.
The AISC and NSBA logos are registered trademarks of AISC.*

The information presented in this publication has been prepared following recognized principles of design and construction. While it is believed to be accurate, this information should not be used or relied upon for any specific application without competent professional examination and verification of its accuracy, suitability and applicability by a licensed engineer or architect. The publication of this information is not a representation or warranty on the part of the American Institute of Steel Construction, its officers, agents, employees or committee members, or of any other person named herein, that this information is suitable for any general or particular use, or of freedom from infringement of any patent or patents. All representations or warranties, express or implied, other than as stated above, are specifically disclaimed. Anyone making use of the information presented in this publication assumes all liability arising from such use.

Caution must be exercised when relying upon standards and guidelines developed by other bodies and incorporated by reference herein since such material may be modified or amended from time to time subsequent to the printing of this edition. The American Institute of Steel Construction bears no responsibility for such material other than to refer to it and incorporate it by reference at the time of the initial publication of this edition.

Printed in the United States of America

Foreword

The Steel Bridge Design Handbook covers a full range of topics and design examples to provide bridge engineers with the information needed to make knowledgeable decisions regarding the selection, design, fabrication, and construction of steel bridges. The Handbook has a long history, dating back to the 1970s in various forms and publications. The more recent editions of the Handbook were developed and maintained by the Federal Highway Administration (FHWA) Office of Bridges and Structures as FHWA Report No. FHWA-IF-12-052 published in November 2012, and FHWA Report No. FHWA-HIF-16-002 published in December 2015. The previous development and maintenance of the Handbook by the FHWA, their consultants, and their technical reviewers is gratefully appreciated and acknowledged.

This current edition of the Handbook is maintained by the National Steel Bridge Alliance (NSBA), a division of the American Institute of Steel Construction (AISC). This Handbook, published in 2021, has been updated and revised to be consistent with the 9th edition of the AASHTO LRFD Bridge Design Specifications which was released in 2020. The updates and revisions to various chapters and design examples have been performed, as noted, by HDR, M.A. Grubb & Associates, Don White, Ph.D., and NSBA. Furthermore, the updates and revisions have been reviewed independently by Francesco Russo, Ph.D., P.E., Brandon Chavel, Ph.D., P.E., and NSBA.

The Handbook consists of 19 chapters and 6 design examples. The chapters and design examples of the Handbook are published separately for ease of use, and available for free download at the NSBA website, www.aisc.org/nsba.

The users of the Steel Bridge Design Handbook are encouraged to submit ideas and suggestions for enhancements that can be implemented in future editions to the NSBA and AISC at solutions@aisc.org.

TECHNICAL REPORT DOCUMENTATION PAGE

1. Title and Subtitle Steel Bridge Design Handbook Chapter 5: Selecting the Right Bridge Type	2. Report Date February 2022
3. Original Author(s) Ken Wright, PE (HDR)	4. Revision Author(s) Ryan Jenkins, PhD, PE (HDR)
5. Sponsoring Agency Name and Address National Steel Bridge Alliance, a division of the American Institute of Steel Construction 130 E. Randolph, Suite 2000 Chicago, IL 60601	6. Revision Performing Organization Name and Address HDR, Inc. 301 Grant Street, Suite 1700 Pittsburgh, PA 15216
7. Supplementary Notes The previous edition of this Handbook was published as FHWA-HIF-16-002 and was developed to be current with the 7 th edition of the AASHTO LRFD Bridge Design Specifications. This edition of the Handbook was updated to be current with the 9 th edition of the AASHTO LRFD Bridge Design Specifications, released in 2020.	
8. Abstract One of the initial tasks when designing a bridge is selecting the most appropriate bridge type for the site. This choice may or may not be straightforward, but selecting the right structure type is one of the most important aspects of designing a cost-effective bridge. This volume of the Handbook provides bridge designers with the tools and guidance for selecting the right bridge type for the given site. Bridge types discussed included rolled steel beam, steel plate girder, trusses, arches, cable-stayed, and suspension bridges.	
9. Keywords Steel Bridge, Bridge Type, Span Length, Economics, Constructability	10. AISC Publication No. B905-22

Steel Bridge Design Handbook: Selecting the Right Bridge Type

Table of Contents

1.0	INTRODUCTION	1
2.0	BRIDGE TYPES	2
2.1	Rolled Beam Bridges	2
2.2	Plate Girder Bridges.....	2
2.3	Steel Tub Girder Bridges	5
2.4	Truss Bridges	6
2.5	Arch Bridges	11
2.6	Cable-Stayed Bridges.....	14
2.7	Suspension Bridges.....	17
3.0	BRIDGE TYPE AND SPAN LENGTH CONSIDERATIONS	19
3.1	Owner Preferences	19
3.2	Stakeholder Commitments.....	19
3.3	Existing Features.....	19
3.4	Other Factors.....	20
4.0	ECONOMICS	21
5.0	CONSTRUCTABILITY	22
5.1	Rolled Beams and Plate Girders	22
5.2	Trusses	22
5.3	Arches	23
5.4	Cable-Stayed Bridges.....	25
5.5	Suspension Bridges.....	25
6.0	REFERENCES	27

List of Figures

Figure 1 Photo of a typical multi-girder system with x-type intermediate cross-frames.....	3
Figure 2 Photo of a typical girder substringer system showing a stringer sitting on top of cross-frames.....	4
Figure 3 Photo of a typical tub girder bridge under construction (Credit: RS&H)	5
Figure 4 Typical Truss Components [10]	7
Figure 5 Photo of steel deck truss bridge (Foresthill Bridge over American River's North Fork, Auburn, California).....	8
Figure 6 Photo of a steel through truss bridge (Chelyan Bridge over Kanawha River, Kanawha County, West Virginia).....	8
Figure 7 Photo of a steel half-through truss bridge (Glenwood Bridge over Monongahela River, Pittsburgh, Pennsylvania)	9
Figure 8 Photo showing a typical stacked floor system.....	10
Figure 9 Photo of a typical framed floor system.....	10
Figure 10 Photo a steel deck arch bridge (New River Gorge Bridge near Fayetteville, West Virginia).....	12
Figure 11 Photo of a steel tied arch bridge (Kentucky Lake Bridge in western Kentucky)	12
Figure 12 Photo of a cable-stayed bridge with H-Towers (Sidney Lanier Bridge over Brunswick River, Brunswick, Georgia)	14
Figure 13 Photo of a cable-stayed bridge with A-Towers (William H. Natcher Bridge over Ohio River near Owensboro, Kentucky)	15
Figure 14 Photo of a cable-stayed bridge with inverted Y-Towers (Normandy Bridge over Seine River near Le Havre, France).....	16
Figure 15 Photo of a cable-stayed bridge with single column towers (Sunshine Skyway Bridge, Tampa Bay, Florida).....	16
Figure 16 Photo of a steel suspension bridge (Golden Gate Bridge, San Francisco, California)	18
Figure 17 Photo showing the cantilever erection of a truss (Point Marion Bridge over the Monongahela River in southwestern Pennsylvania) (Credit: HDR).....	23
Figure 18 Photo showing an arch being floated into place (Broadway Bridge over the Arkansas River near Little Rock Arkansas) (Credit: HNTB Corporation)	24

Figure 19 Photo showing the erection of a deck arch (Greenfield Bridge over Interstate 376 near Pittsburgh, Pennsylvania) (Credit: HDR) 25

1.0 INTRODUCTION

One of the initial tasks when designing a bridge is selecting the most appropriate bridge type for the site. This choice may or may not be straightforward but selecting the right structure type is one of the most important aspects of designing a cost-effective bridge. Specifically, this volume of the Handbook is focused on the selection of an appropriate type of superstructure. The choice of superstructure type is often interrelated with the choice of substructure type, and designers should keep substructure and foundation types in mind when choosing an appropriate superstructure type, but the discussion in this Handbook will focus primarily on superstructure type selection.

There are several types of commonly used steel superstructures, each with its own strengths, advantages, and best-suited applications. The most common steel superstructure types are discussed in Section 2.0. When determining the most appropriate type of superstructure for a bridge, the designer should consider several different factors, including span length considerations, economics, and constructability; each of these topics is discussed in Sections 3.0, 4.0, and 5.0 respectively.

2.0 BRIDGE TYPES

There are many bridge types that are typical for current construction. The various types are ideally suited to different span lengths. However, there is generally significant overlap in the applicable ranges for the most common span ranges, so multiple bridge types are generally viable at most span ranges.

2.1 Rolled Beam Bridges

Rolled beam bridges using W-shapes are efficient in certain situations, mainly for simple spans up to approximately 100 feet or, sometimes, for continuous spans up to approximately 120 feet. They are generally made composite with the bridge deck. Using rolled beams instead of plate girders generally results in designs that feature higher unit weights of steel (in pounds per foot) and greater total weight of steel. However, the unit cost of a rolled beam is significantly lower than a comparably sized plate girder due to the elimination of the fabrication efforts associated with cutting flanges and webs from steel plate and making flange-to-web welds. Furthermore, rolled beam webs typically do not require transverse stiffeners, and rolled beams usually use simpler, more economical rolled shape diaphragms such as rolled channels which involve less fabrication effort and simpler connection details. As a result, rolled beam bridges can often be more economical than plate girder bridges in short span ranges.

When choosing rolled beams, it's important to consider market conditions, girder depth restrictions, and product availability. Additionally, rolled beam bridges can lose their efficiency when the girders require large camber, are horizontally curved, require cover plates, have span lengths over 60 feet, or have a large lead time on material procurement. As a result, it may be prudent to allow the Contractor the option to substitute a plate girder design in place of the specified rolled beam design; in that case, consider including an alternate welded plate girder design in the contract documents. For more details and information on product availability, see NSBA's *Guidelines to Design for Constructability and Fabrication* [1]. Additionally, refer to the Short Span Steel Bridge Alliance's website (www.shortspansteelbridges.org) for eSPAN140, an online tool to quickly size short-span rolled-beam steel bridges.

2.2 Plate Girder Bridges

Due in large part to their simplicity and efficiency, plate girder bridges are the most common type of steel bridge. Over time, different types of plate girder bridges have been used, but in modern times, composite, multi-girder bridges have been, by far, the most common type of plate girder bridge configuration. Deck girder bridges also offer great flexibility to accommodate variable width roadways and horizontally curved geometry. Additionally, shop layout is generally less complex than would be the case for through girders, trusses or arches.

In general, wider girder spacings are typically more efficient, specifically spacings between 11 feet and 13 feet. This is particularly true for bridges with average spans greater than 175 feet. According to NSBA's *Guidelines to Design for Constructability and Fabrication* [1], for bridges with average spans less than 175 feet, the total weight of steel is relatively insensitive to various girder spacings. It should be noted, however, that other considerations, such as deck thickness,

number of cross-frames, and superstructure depth play a larger role in the selection of the girder spacings at these lengths. While the total weight of the steel may be similar, cross-frames and bracing have much higher fabrication costs than girders. As a result, the fabricated cost of bridges with smaller girder spacings tend to be higher than similar bridges with large girder spacings, even if the total steel weight is similar. Also, smaller girder spacings result in a greater number of total girders required, which thus drives erection costs. Based on historical practices and preferences, some agencies still limit girder spacing to the 8 to 9 foot range. For more information, NSBA provides *Steel Span to Weight Curves* [2] which provides a quick way to compare costs of various span arrangements and girder spacings.

Cross-frames or diaphragms have been provided at a maximum spacing of 25 feet for many years in accordance with the provisions of the *AASHTO Standard Specifications for Highway Bridges (2002)* [3]. The *AASHTO LRFD Bridge Design Specifications, 9th Edition* (AASHTO LRFD BDS) (2020) [4] does not include the historical 25 foot maximum spacing requirement of earlier AASHTO specifications and leaves the spacing up to the designer. The intent was solely to remove what was an arbitrary limit that sometimes led to inefficient framing plans; 25' to 30' is still a good target for maximum cross-frame spacing. Using cross-frame spacings larger than this range typically leads to uneconomical girder designs and can produce undesirable second-order amplification of flange lateral bending effects. Note that in some cases, such as in severely skewed bridges, omitting select cross-frames or using lean-on bracing details may provide both better performance and improved economy, while still providing adequate bracing for the girders. For a more complete discussion of cross-frame spacing and configurations, refer to the NSBA's *Steel Bridge Design Handbook: Stringer Bridges – Making the Right Choices* [5].



Figure 1 Photo of a typical multi-girder system with x-type intermediate cross-frames

One variation of the multi-girder system is the girder substringer system. This arrangement is generally used on wider bridges with relatively long spans. As discussed earlier, fewer girders in the cross section will result in greater overall economy in the superstructure design. When the span lengths exceed approximately 275 feet, it may be economical to use a girder substringer arrangement. This system uses several heavy girders with wider girder spacing. Truss cross-frames, which in many cases look like large K-frames, are used between the main girders with the rolled beam stringers supported midway between the main girders.



Figure 2 Photo of a typical girder substringer system showing a stringer sitting on top of cross-frames

As recently as the 1970s, many bridges were designed using two deck girders, transverse floorbeams at regular intervals, and longitudinal stringers either continuous over the floorbeams or framed into the floorbeam webs. The use of two-girder bridges was phased out in response to their fracture-critical nature, difficulty in widening, and a greater use of multi-girder bridges.. At the same time, composite construction became nearly universal for girder bridges. Designing for composite action allows the designer to account for the strength of the bridge deck in the section properties of the girders. In addition, the top flanges in the positive moment regions are fully braced in the final condition, allowing the use of higher allowable compressive stresses than are possible for flanges braced at discrete points. Both aspects led to the recent predominance of composite, multi-girder bridges.

Through girders provide another plate girder option. A through girder bridge generally has two girders near the edges of the deck with shallow floorbeams connecting the bottom flanges of the girders and often will have longitudinal stringers framed into the floorbeams. The girder top flanges and much of the web extend above the top of the bridge deck. Knee braces are generally provided at each floorbeam location in order to brace the girder top flange. There are several reasons that through girder bridges are not commonly used for highway structures. The system results in two-girder structures, which are generally considered fracture critical. The main girders cannot be made composite with the bridge deck, meaning that the deck offers no strength benefit to the girders. Finally, the top flanges of the girders in their compression regions are braced only at the floorbeam spacing, rather than full length, as would be the case for a composite deck girder bridge. This will require additional steel in the top flanges of the girders because of the strength reductions resulting from the unbraced length. They are also prone to being struck by errant vehicles. Through girder bridges are still somewhat common for railroad bridges where deep girders, located under the deck, pose a clearance problem.

Difficulty in quantitatively assessing the structural redundancy of two girder deck bridges limits their use today as they are generally classified as fracture critical. However, ongoing research, as well as the increased use of advanced analysis software and modeling techniques, has led to a better understanding of the structural behavior of two girder deck bridges and may lead to a

return of their use. Specifically, AASHTO's *Guide Specifications for Analysis and Identification of Fracture Critical Members and System Redundant Members* (2018) [6] provides methods to designate certain members as System Redundant Members (SRMs). Members designated as SRMs may be exempted from the in-depth inspection requirements of Fracture Critical Members (FCMs). Traditionally designated as FCMs, examples of possible SRMs on two-girder bridges are floorbeams and, if proportioned adequately, the girders themselves.

In addition, AASHTO's *Guide Specifications for Internal Redundancy of Mechanically-Fastened Built-Up Steel Members* (2018) [7] considers internal redundancy of built-up members that include mechanical separation of the elements that comprise a given structural member. If proportioned correctly, members that might otherwise be deemed fracture critical could be reclassified as Internally Redundant Members (IRMs) through demonstration of internal redundancy. Older, riveted, built-up bridge members can often be shown to satisfy these requirements, but new bolted, built-up member designs can also qualify. The material cost savings resulting from a design optimized by using a reduced number of girders may outweigh the additional costs associated with fabrication of bolted, built-up members required to demonstrate that the bridge is not fracture critical.

Note that these guide specifications are still in a trial phase, and Owner's adoptions may vary. At the time of this writing, FHWA approval is required to classify members as SRMs, and while FHWA recognizes the benefits of IRMs, it does not currently exempt IRM members from in-depth fracture critical inspections for members that would otherwise classify as nonredundant. For more information, see NSBA's *Steel Bridge Design Handbook: Redundancy* [8].

2.3 Steel Tub Girder Bridges

Steel tub girders are another option for a girder-type bridge. These types of bridges can handle more span ranges and complex curvatures than typical multi-girder bridges and are much simpler than complex structures such as trusses, arches, and cable bridges. These bridges provide a sort of middle ground between simple and complex bridges. In addition, many stakeholders appreciate their clean aesthetics.



Figure 3 Photo of a typical tub girder bridge under construction (Credit: RS&H)

For curved structures, steel tub girder bridges provide advantages over typical plate girder bridges. Specifically, the torsional stiffness of a composite steel tub girder bridges is substantially greater than a typical plate girder bridge. Since they are much more efficient at carrying torsional loads, they can be built to extremely tight radii and have been used this way since their introduction. Steel tub girder bridges can also have improved durability since much of the girder is enclosed. To allow for internal inspection, a practical minimum web depth of about 5 foot is preferred by owners.

Like other girder bridges, the rule-of-thumb of $L/25$ for girder depth provides a good starting point, but shallower tub girder bridges have been successfully designed and constructed (where L is the span length). A minimum practical width for the bottom flange is about 4 feet to provide access for inspection, and while there is no maximum, extremely wide bottom flanges require longitudinal and/or transverse stiffeners, which can increase fabrication costs even further.

In general, steel tub girder bridges are more costly and complex to fabricate than typical I-shaped plate girder bridges due to their geometric complexity and requirement for higher skilled workers. If carefully detailed and designed, however, these types of bridges can be competitive for a wide range of applications. For more information, see NSBA's *Practical Steel Tub Girder Design* [9].

2.4 Truss Bridges

Trusses are used for some longer spans and are comprised of discrete members that are subjected primarily to axial loads. The members are generally arranged to form a series of triangles that act together to form the structural system. The chords are the top and bottom members that behave as the flanges of a girder. Diagonals and verticals function in a manner similar to the web in a plate girder. Diagonals generally provide the necessary shear capacity. Verticals carry shear and provide additional panel points through which deck and vehicle loads can be applied to the truss. Tension verticals are commonly called hangers, and compression verticals are often called posts. They also serve to limit the dead load bending stresses in the chord members by reducing the unsupported member length. Joints are the locations where truss members intersect and are referred to as panel points. See Figure 4 below from the *Bridge Inspector's Reference Manual* [10], which shows the typical components of a truss.

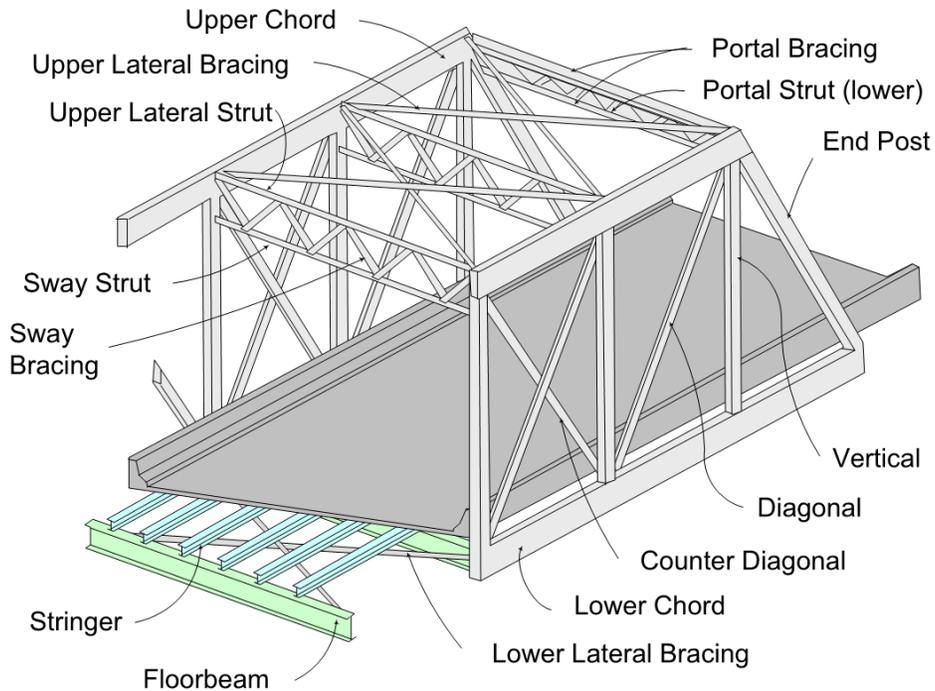


Figure 4 Typical Truss Components [10]

The deck is the structural element that directly supports applied traffic loads. Stringers are longitudinal beams, generally placed parallel to traffic, that carry deck loads to the floorbeams. Floorbeams are usually set normal to the direction of traffic and are designed to transmit loads from the bridge deck to the trusses. Some trusses in the past have been designed without stringers, relying on the deck to transmit the loads directly to the floorbeams. This requires the joint spacing along the chords to be relatively small, and as a result is not economical in the current market.

Lateral bracing is normally provided in the plane of both the top and bottom chords of the trusses. Its purpose is to stiffen the trusses laterally and to carry wind loads and other applied lateral loads back to the support locations. The configuration of the lateral bracing systems is not required to be the same between the top and bottom bracing systems.

Sway bracing is provided between the trusses in the plane of either verticals or diagonals, and its purpose is to minimize the relative vertical deflections between the truss lines by retaining the cross-sectional shape as well as providing an efficient load path for lateral loads such as wind loads. Portal bracing, most commonly used in through trusses, is sway bracing placed in the plane of the end posts to allow for traffic to pass through. For deck trusses, the end posts are generally vertical members, and for through trusses the end posts are generally the diagonal members extending up from the end joints.

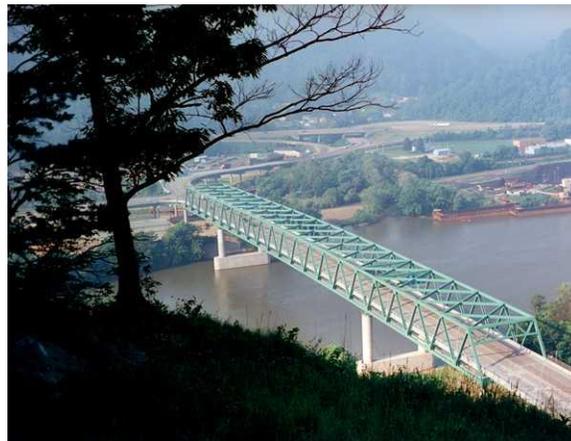
There are three basic truss types. For deck trusses, the entire truss is below the bridge deck. The floorbeams may either run between the top chords of the trusses or rest on top of the trusses. Deck trusses can be particularly cost-effective when the floorbeams rest on top of the truss rather than framing in, because the floorbeams can take advantage of the continuity effect due to the cantilevered overhangs. Also, the trusses can be closer together, which reduces the length, and

therefore the cost, of the lateral bracing and sway frames. Deck trusses are generally more feasible in cases where vertical clearance below the bridge is not restricted. Deck trusses result in more economical substructures because they can be significantly shorter since the bridge is below the deck. Deck trusses are also easier to widen in the future since the deck is above the trusses. Future deck widening is limited only by the increased structural capacity obtained by modifying or replacing the floorbeams and strengthening the truss members.



Figure 5 Photo of steel deck truss bridge (Foresthill Bridge over American River's North Fork, Auburn, California)

Through trusses are detailed so that the bridge deck is located as close to the bottom chord as possible. The bottoms of the floorbeams are normally located to line up with the bottom of the bottom chord. Through trusses are generally used when there is a restricted vertical clearance under the bridge. The depth of the structure under the deck is controlled by the depth of the floor system and can therefore be minimized so that the profile can be kept as low as possible. For shorter spans, the minimum depth of the truss may be controlled by the clearance envelope required to pass traffic under the sway bracing (i.e., through the portal frames). The trusses must be spaced to pass the entire deck section inside the truss lines. It is generally not feasible to widen a through truss structure without adding a parallel truss to the original two truss lines.



**Figure 6 Photo of a steel through truss bridge (Chelyan Bridge over Kanawha River, Kanawha County, West Virginia)
(Note the top chord lateral bracing and the portal bracing)**

If a sidewalk is required on a through truss bridge, it can either be located inside the trusses with the roadway portion of the deck or supported outside the trusses by brackets that bolt into the outside of the truss joints. Locating the sidewalks inside the trusses adds some cost to the lateral bracing and sway bracing since the truss spacing is increased. However, the loading is better balanced between the two trusses, and all the construction can be accomplished inside the truss lines, making it easier to move materials to the desired locations. Supporting sidewalks outside the trusses allows the truss spacing to be minimized, saving cost in the bracing systems. However, the cost of the overhang framing and the additional difficulty of constructing sidewalks outside the trusses, as well as the increase in the number of components that need to be fabricated and erected, may outweigh the savings in bracing weight. In some cases, there have been serviceability problems in the overhang brackets supporting the sidewalks, which increases the maintenance cost of the bridge. The loading between the trusses is much more eccentric than is the case when the sidewalk is kept inside the truss lines.

Half-through trusses carry the deck high enough that sway bracing cannot be used above the deck. Much of the previous discussion regarding through trusses also applies to half-through trusses. It is very difficult to design a half through truss if the chosen truss type does not have verticals. Without verticals, the deck must be supported from diagonal members away from the joints. This condition results in significant bending stresses in the truss members, which are generally not efficient in carrying bending, thus resulting in inefficient member designs for the diagonals. Many of the recent trusses designed in the United States have been designed without verticals to achieve a cleaner and more contemporary appearance, thus minimizing the use of half-through trusses.

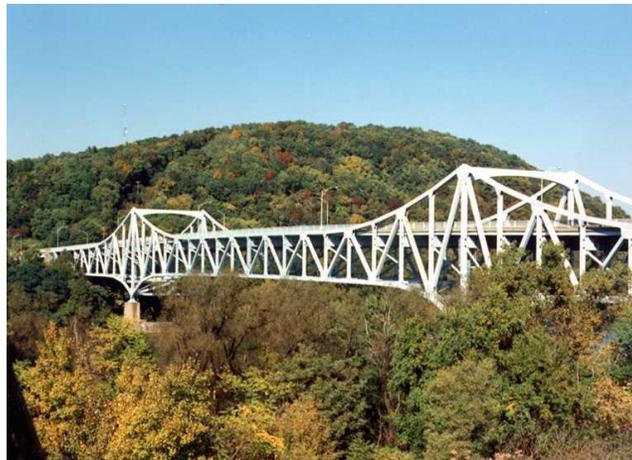


Figure 7 Photo of a steel half-through truss bridge (Glenwood Bridge over Monongahela River, Pittsburgh, Pennsylvania)

There are several geometric guidelines that are helpful when determining truss configurations. AASHTO LRFD BDS (2020) recommends minimum truss depths of one-tenth the span length for simple spans. For continuous trusses, the distance between inflection points can be used as the equivalent simple span length to determine the minimum truss depth.

It is generally desirable to proportion the truss panel lengths so that the diagonals are oriented between 40 degrees and 60 degrees from horizontal. This keeps the members steep enough to be

efficient in carrying shear between the chords. This angular range also allows the designer to maintain a joint geometry that is relatively compact and efficient.

There are two floor system configurations that are used for trusses. Stacked floor systems are the most commonly used detail in recent trusses. In a stacked floor system, the longitudinal stringers rest on top of the floorbeam top flanges, supported by bearings. The stringers are generally made continuous across several floorbeams. This allows for more efficient stringer designs with fewer pieces to fabricate and erect. It also minimizes deck expansion joints which may leak, reducing the probability of de-icing salts washing over the members below. Rolled W-shapes have most commonly been used for the stringers since they can efficiently span the normal panel lengths used for trusses.



Figure 8 Photo showing a typical stacked floor system

Framed floor systems use a series of simple span stringers that frame into the floorbeam webs. When faced with depth restrictions, framed systems serve to reduce the overall depth of the floor system by the depth of the stringers.



Figure 9 Photo of a typical framed floor system

Truss analysis is typically idealized assuming that the members are pinned at the joints (free to rotate independent of other members at the joint) so that secondary stresses are ordinarily not needed to be considered in the design. Joints are typically detailed so that the working lines for the diagonals, verticals and chords intersect at a single point. However, bending stresses resulting from the self-weight of the members should be considered in the design.

Trusses traditionally have members that are considered to be fracture critical members. The simplified approach during design has been to designate all truss tension members and members subjected to stress reversals as fracture critical members. However, similar to plate girder bridges (discussed in Section 2.2), recent guidance on SRMs and particularly IRMs also applies to trusses. Fracture critical studies can be performed based on analyses that model the entire framing system, including the bracing systems, to determine whether certain lightly loaded tension or reversal members are truly fracture critical. In many cases the number of fracture critical members can be reduced through this process, which reduces inspection costs. Additionally, certain members can be designed for internal redundancy [8].

2.5 Arch Bridges

Arches carry loads in a combination of beam action and axial forces, or thrusts. Arches take several basic forms. True arches carry the horizontal component of each reaction directly into a buttress, which also resists the vertical reaction. The arch ribs carry both thrust and moment. Tied arches increase the applicability of the arch form by adding a tie, which is a tension member between the ends of the span. In a tied arch, the thrust is carried by the tie, but the moment is divided between the arch and the tie, somewhat in proportion to the stiffness of the two members.

Arches can have either trussed or solid ribs. Solid ribbed arches are generally used for shorter spans. Trussed arches tend to become economically feasible as the span lengths increase past 1000 feet.

Arch bridges are also constructed using various degrees of articulation. A fixed arch prevents rotation at the ends of the span and is statically indeterminate to the third degree. A two-hinged arch permits rotation at the ends of the rib and is also one degree statically indeterminate. Occasionally a hinge is also provided at the crown of the arch rib, making the arch statically determinate. This detail, however, has become less prevalent with the increased availability and power of computer analysis programs.

Arches are classified as deck arches when the entire arch is located below the deck. Most true arches are deck arches. Tied arches are normally constructed as through arches with the arch entirely above the deck and the tie member at the deck level. Both true and tied arches can be constructed with the deck at some intermediate level that can be classified as half-through.

Deck arches are usually used when crossing deep valleys with steep walls. Assuming that rock is relatively close to the surface, the arch foundations can then remain relatively short, effectively carrying both vertical loads and horizontal thrusts directly into the rock. The foundation costs increase significantly when deep foundations are required. Occasionally, there are site constraints that dictate placing the foundations closer to the deck profile, such as a desire to keep the bearings for the arch above the high-water elevation at the site. In such cases, half-through arches may better satisfy the design constraints.



**Figure 10 Photo a steel deck arch bridge
(New River Gorge Bridge near Fayetteville, West Virginia)**

Tied arches are generally more effective in cases where deep foundations are required or where high piers may be required to achieve a desired clearance under the bridge, such as a long span crossing a river where a navigation clearance envelope must be provided.



**Figure 11 Photo of a steel tied arch bridge
(Kentucky Lake Bridge in western Kentucky)**

The floor systems for arches are similar to those for trusses. Transverse floorbeams are located at spandrel columns for deck arches, and longitudinal girders transmit the deck loads to the floorbeams, which in turn transmit the forces to the spandrel columns and into the arch ribs. However, for deck arches the spandrel columns are generally spaced much farther apart than the panel points on a truss. Therefore, it is common to use welded plate girders or box girders as the longitudinal beams instead of the rolled beams that are common for trusses. As with trusses, it is more common to work with a stacked floor system in order to take advantage of continuity in the longitudinal beams and to minimize the number of pieces that must be erected and bolted.

The columns supporting the floor system generally are bolted to the top of the arch ribs. Sway bracing is provided over the full height of these columns to provide stability when subjected to lateral loads and to reduce the unbraced lengths of the columns when assessing their axial and bending capacity.

For tied arches, the ribs and tie girders are linked together with a system of hangers. Traditionally, the hangers have been installed vertically between panel points of the ribs and ties. The hangers force both components to participate in carrying moments induced in the system. However, recent designs have begun to consider networked hanger systems; that is, hangers that connect non-concurrent panel points in the rib and tie. This results in a “trussed” appearance of the ties. The networked hangers provide additional stiffness to the structural system which may lead to a more economical design in certain cases.

Hangers for tied arches in the United States have historically been made using groups of 2 to 4 bridge strand or bridge rope. These consist of wound wires, so there is significant rigidity in the cables. As a result, there has not been a history of undue vibration in these cables. The ropes are pre-stretched during fabrication so that, when they are installed, the correct geometric positioning of the bridge is provided. However, as cable-stayed bridge technology has improved, some designers have begun to consider the use of parallel strand hangers for tied arches as well.

Aesthetic considerations may impact the fabrication and construction costs of arches. For solid ribbed arches, the rib can be curved or chorded between the panel points. Curving the rib adds fabrication and material cost but provides the optimum appearance. Chording the rib between panel points can provide an acceptable appearance as well, with a lower material and fabrication cost. If the choice is made to chord the ribs, it may be desirable to decrease the panel point spacing to reduce the angular change between sections. Another detail issue that relates to the appearance is whether to maintain a constant depth rib or whether to vary the depth so that the rib is shallow at the crown and increases in size toward the spring line.

Selection of the rib lateral bracing system can also significantly impact the appearance of the bridge. K-bracing configurations have been used, but they tend to result in a more cluttered appearance since struts between the arch ribs are required at the point of each K-brace. X-bracing systems are also common and have been detailed with and without transverse struts. X-bracing without transverse struts provides a cleaner and more modern appearance. Additionally, some lateral bracing designs also only utilize a single transverse strut without diagonal bracing (Vierendeel Truss) and per AASHTO LRFD BDS (2020) shall be treated as unbraced frames when determining effective length factors. Similar to trusses, tied arches traditionally have members considered to be fracture critical members, since the failure of a tie girder would be expected to lead to collapse of the bridge. For a period of time through the 1980s and early 1990s, there was great reluctance within the design community to use tied arches because of these concerns. However, with improved detailing practices and the development of high-performance steel (HPS), with its superior fracture toughness, tied arches have recently gained more support. In addition, many agencies accept tie girder designs that are internally redundant. This can be accomplished by either post-tensioning the tie girders to eliminate tension in the girder plates or by detailing bolted, built-up sections with discrete components that are not connected by welds. Thus, if one component fractures, new cracks would have to initiate in the

other components before complete fracture could occur. Due to recent guidance on SRMs and particularly IRMs, it may be possible to eliminate the fracture critical classification. Regardless, this detailing provides some security that a sudden, catastrophic failure is highly unlikely [8].

2.6 Cable-Stayed Bridges

Cable-Stayed bridges rely on high strength steel cables as major structural elements. The stay cables are inclined from the supporting towers to edge girders at or below the deck elevation. These bridges are generally signature structures with excellent aesthetics characterized by very slender superstructures and tall towers.

Generally, the towers are very tall with the height determined as a function of the span length. The slope of the longest stay cables dictates the minimum tower height. The flattest cable angle should not be less than about 22 degrees with the horizontal. Below 22 degrees, the cables become inefficient in carrying the vertical load component and they exert very high compressive forces on the edge girders.

There is a significant variety in tower configurations that have been used on cable-stayed bridges in the U.S. and around the world. H-Towers have two legs that rise vertically above the edge of the bridge and the cables are outside of the roadway. There is generally an intermediate strut part of the way up the towers but below the level of the stay cable anchorages. The appearance of H-Towers is more utilitarian than other shapes. One of the main advantages of the H-Tower is that the stay cables can be installed in a vertical plane directly above the edge girder. The vertical planes of stay cables simplify the detailing of the deck-level anchorages because the connections to the edge girders are parallel to the webs of the edge girders. If ice forms on the cables during inclement winter weather, it will not fall directly onto traffic.



**Figure 12 Photo of a cable-stayed bridge with H-Towers
(Sidney Lanier Bridge over Brunswick River, Brunswick, Georgia)**

A-Towers have inclined legs that meet at the top of the tower. The stay cables are usually in a plane that parallels the slope of the tower legs. From a detailing standpoint, the edge girder anchorages are more complicated since they intersect the edge girder at an angle. Moreover,

because every stay cable intersects the edge girders at a slightly different angle, exact repetition of anchorage geometry is not possible. There have been instances where steel edge girders have been fabricated with sloped webs in an effort to simplify the anchorage details, but there is some increase in fabrication cost for the sloped edge girders. The inwardly sloped cable planes exert compression into the superstructure in both the longitudinal and transverse directions. The sloped cables provide some additional torsional stiffness, which is particularly important on narrow and slender superstructures. In many cases, the center-to-center spacing of the edge girders needs to be increased so that the sloped cables do not infringe upon the vertical clearance envelope required to pass highway traffic.



**Figure 13 Photo of a cable-stayed bridge with A-Towers
(William H. Natcher Bridge over Ohio River near Owensboro, Kentucky)**

Inverted Y-Towers are a variation of the A-Tower. In this case, the sloped tower legs meet well above the roadway and a vertical column extends upward from the intersection of the sloped legs. The stay cables are then anchored in the vertical column. Every stay cable intersects the edge girders at a slightly different angle so that exact repetition of anchorage geometry is not possible. The advantages and disadvantages discussed for the cables are similar to those previously discussed regarding A-Towers. Achieving the desired clearance envelope for traffic may be more critical for the inverted Y-Towers, possibly requiring even further widening of the superstructure since the inward cable slopes are more significant.



**Figure 14 Photo of a cable-stayed bridge with inverted Y-Towers
(Normandy Bridge over Seine River near Le Havre, France)**

Single column towers have been used on a few cable-stayed bridges. These towers consist of a single column in the center of the bridge with a single plane of stay cables down the center of the bridge. The single plane of cables offers some economy in cable installation and maintenance. However, the transverse beams supporting the superstructure at each anchorage location are designed as cantilevers in both directions and will be somewhat heavier than beams supported by two cable planes at the outside edges of the bridge.



**Figure 15 Photo of a cable-stayed bridge with single column towers
(Sunshine Skyway Bridge, Tampa Bay, Florida)**

Cable-stayed superstructures have taken many forms around the world. Concrete cable stays have used concrete edge girders with concrete decks (either precast or cast-in-place) as well as post-tensioned box girder sections. Steel and/or composite cable-stayed bridges have used either I-shaped or box-shaped edge girders. The edge girders carry bending between stay cable anchorages plus axial compression resulting from the horizontal component of the stay cable forces. The compressive forces are additive from the end of the longest cables toward the towers. In most cases the edge girder design is not controlled by bending but by the compressive forces imparted by the stay cables, requiring a heavier section in order to avoid buckling.

Various deck systems have been used for cable-stayed bridges, including cast-in-place concrete decks and precast concrete deck panels with specialized overlays. When concrete decks are used, it is common to provide longitudinal post-tensioning of the deck to prohibit cracking of the deck. The deck post-tensioning is generally heaviest far away from the towers. Compression from the anchorage forces spreads through the concrete near the towers and the need for post-tensioning diminishes. It is common to use specialized concrete overlays even for cast-in-place concrete decks

Orthotropic decks have also been used. However, in the U.S. the overall cost for orthotropic decks has limited their use to extremely long spans. For more information on orthotropic decks, see NSBA's *Steel Bridge Design Handbook: Bridge Deck Design* [11].

Modern stay cables consist of a cluster of parallel prestressing strands sized to carry the design load of the particular stay cable. The cables are typically housed in PVC pipes to protect them from the effects of weather. Early cable-stayed technology indicated that the cables should be grouted inside the pipe as an additional level of corrosion protection. Based on the performance of some of the early cable systems, however, the state of the art has moved away from grouting as a corrosion protection system. Instead, the current preferred protection system consists of individually HDPE or HDPP sheathed strands filled with flexible filler corrosion inhibitor (grease or wax), contained in UV-resistant HDPE cable pipes.

2.7 Suspension Bridges

Suspension bridges are also cable-suspended structures but use a different system. As with cable-stayed structures, suspension bridges rely on high-strength cables as major structural elements. Suspension bridges, whose towers are significantly shorter than those required for cable-stayed bridges, are economical for very long spans.

The most common form of suspension bridge uses an externally anchored suspender cable system. The main cables typically are located at the outside edges of the bridge and are draped over the towers where they rest in saddles. The ends of the main suspension cables are anchored either into large counterweights or directly into bedrock at the ends of the structure. Both the size of the cables and the magnitude of the tension in these main suspender cables determine the vertical stiffness of the bridge.



**Figure 16 Photo of a steel suspension bridge
(Golden Gate Bridge, San Francisco, California)**

Suspension bridge superstructures are generally very light relative to their span length, which leads to structures that may be very flexible. The vertical stiffness is a complex interaction of the size and tension of the main suspender cables as well as the type and geometry of the floor system, such as stiffening trusses. Accordingly, it is common to provide a lateral stiffening truss under the deck to avoid excessive excitation of the superstructure under wind or seismic loadings. The first Tacoma Narrows Suspension Bridge illustrated the pitfalls of providing the lightest superstructure possible based on strength design while not considering the possible harmonic effects induced by lateral wind loads. The structure collapsed due to the harmonic frequency set up during a period of only moderately high winds.

The superstructure is generally supported at relatively short intervals by vertical cables that connect at the ends of transverse floorbeams. The vertical cables are relatively light and are attached to the main suspension cables by saddle clips. While vertical cables are most common there have been a few suspension bridges that used inclined suspenders, such as the Bosphorus Bridge in Turkey and the Severn Bridge in England.

Due to the extreme span lengths, lightweight deck options, including lightweight concrete, are often used to reduce the dead load demand. Similar to cable stay bridges, steel orthotropic deck systems have often been used both to limit the deck dead load and to simplify the floor system. Asphaltic overlays are typically used over the top plates of orthotropic decks and can be replaced efficiently without a total deck replacement.

3.0 BRIDGE TYPE AND SPAN LENGTH CONSIDERATIONS

Various types of bridge superstructures provide efficient solutions for different span arrangements. There are many possible reasons for choosing particular span lengths for bridges, some of which are discussed below. In addition to the partial list of factors discussed below, Engineers use their historical experience and technical expertise to determine the most appropriate solutions and make recommendations. One of the roles of the NSBA's *Steel Bridge Design Handbook* is to help Engineers make these informed recommendations when planning a project.

3.1 Owner Preferences

In many cases, the owner's desires will drive the selection of the bridge type. Some owners tend to push their bridges toward the shortest spans possible, with an eye toward allowing a choice of materials or to prefer a specific material type.

Owners will occasionally choose a bridge because they desire to construct a specific bridge type at a location. These desires may come directly from the owner, but sometimes public opinion influences the selection of the bridge type, particularly for long span bridges.

3.2 Stakeholder Commitments

One critical requirement that often controls the main span lengths for water crossings is navigational clearance. The U. S. Coast Guard is generally the controlling agency regarding required span lengths for navigable inland waterways.

Environmental commitments can also have a non-negotiable effect on bridge span lengths. Given the increased sensitivity to minimizing adverse environmental impacts, there have been many cases where the span arrangements have been set to meet environmental commitments.

3.3 Existing Features

Existing constraints often control the locations of substructures and foundations and, thus, includes the span arrangement for new structures. For new, high-level structures, the locations of existing features that are being crossed by the new structures may control certain span lengths or span arrangements. This situation may occur where expanded interchanges are being constructed on or near the site of existing interchanges where the original structures will be retained or used for staging during construction. The location of existing surface roadways often controls span arrangements for new bridges. Occasionally it is more cost-effective to move the surface roadways to accommodate the new bridge, but in congested urban areas this may not be possible. Similarly, existing overhead or subterranean utilities may cause interference during construction of substructure elements, thus requiring a cost-risk comparison analysis of relocating the utility or modifying the span arrangement.

Another constraint encountered by designers and owners is railroad rights-of-way. There are significant costs associated with moving tracks or interrupting rail service to accommodate

construction of new bridges. It is often beneficial to increase span lengths to minimize impacts to the railroads. Since the railroads are for-profit enterprises, they tend to be very protective of their facilities in order to maintain profitability.

3.4 Other Factors

Site access may affect the location and constructability of substructures and foundations. In some cases, this may suggest the use of longer spans, while in other cases this may suggest the use of shorter spans and top-down construction methods. Site access may also affect the ability to mobilize large enough cranes or other equipment that may be required for construction of certain superstructure types. This may preclude the use of certain structure types that might allow the use of longer spans.

Occasionally, the desired construction schedule may impact the structure type that is chosen. Certain types of structures lend themselves to short construction durations, which may be important to the owner. Additionally, Accelerated Bridge Construction (ABC) techniques have been increasingly used by States to reduce construction schedules even more aggressively than the most efficient traditional construction methods, though, oftentimes at a moderate to significant increase in construction costs. ABC techniques may not directly affect the structure type, but certain structure types are more suitable for ABC techniques. For additional information, see the FHWA's *Accelerated Bridge Construction - Experience in Design, Fabrication and Erection of Prefabricated Bridge Elements and Systems* [12].

Local contractor expertise may also affect the selection of the structure type. Certain types of structures are not commonly used in certain regions. For example, while segmental structures are relatively common in the southeast, they are rare in many parts of the northeast, where many owners will not use segmental structures because of the difficulties associated with deck replacement. As a result, few contractors in the northeast region have experience in segmental construction, which will likely result in either high construction costs or out-of-state contractors winning the bid. However, eliminating one or more structure types may limit the ability to achieve certain span lengths.

4.0 ECONOMICS

Girder bridges are currently the most common type of steel bridge. Applicable in span lengths from approximately 90 feet to over 500 feet, they provide a versatile choice for the designer. As the span lengths increase, girders do not necessarily provide the lightest steel unit weight, but they often provide the most cost-effective design because the fabrication and erection costs are much lower than for trusses and arches. For more information, see NSBA's *Guidelines to Design for Constructability and Fabrication* [1]. In addition, NSBA provides *Steel Span to Weight Curves* and *Continuous Span Standards*, which provide quick ways to compare costs of various span arrangements and girder spacings [2 and 13].

Trusses are generally not cost-effective for span lengths under 450 feet. Below 450 feet, the labor required to fabricate and erect a truss will generally exceed any additional material cost required for a deck girder design. The practical upper limit for simple span trusses is approximately 750 feet, while continuous trusses begin to be cost-effective when the span length exceeds 550 feet. Continuous or cantilever trusses can span considerably longer distances than simple span trusses. The cost of truss spans increases rapidly as the span length exceeds 900 feet. In recent years, as cable-stayed bridges have become more common and the technology refined, trusses longer than 900 feet have not been constructed in the United States.

Worldwide, arches have been used for span lengths from approximately 200 feet up to 1800 feet. Arches used for the shorter span lengths, either as tied arches or true arches, are generally used more for aesthetics than cost-effectiveness. Tied arches generally can be considered cost-effective for span lengths from approximately 450 to 900 feet. Once the span approaches the 900 foot range, erection costs begin to climb significantly. True arches have been used for span lengths up to 1800 feet, such as the Lupu Bridge in China, constructed in 2003. Solid rib arches are economical and attractive for shorter span lengths; however, once the span lengths approach approximately 1000 feet, trussed arches often prove to be more cost-effective.

Cable-stayed bridges have been constructed with main span lengths less than 200 feet and extending up to nearly 3600 feet. In the U.S., cable-stayed bridges become very cost competitive for main span lengths over 750 feet. Almost all bridges with main span lengths in excess of 1000 feet built in the U.S. since 1980 have been cable-stayed structures. They are more versatile than suspension bridges in that they offer more possible configurations and towers can be constructed in less than optimal foundation conditions because having good rock near the surface is not a requirement.

Prior to 1970, it was generally considered that suspension bridges provided the most economical solution for span lengths in excess of about 1200 feet. With the development of cable-stayed bridge technology, suspension bridges do not become economical until the main span length approaches 3000 feet. Following a 30 year hiatus from suspension bridge construction in the U.S., the new Carquinez Bridge in Vallejo, California was opened in 2003, and the new Tacoma Narrows Bridge near Tacoma, Washington was opened in 2007. Suspension bridges require large expensive steel castings for the cable saddles on the main towers, at rocker bents, at the ends of the side spans and at splay bents and anchorages.

5.0 CONSTRUCTABILITY

Below is a brief overview of constructability of various bridge types. For more information, see NSBA's *Steel Bridge Design Handbook: Design for Constructability* [14].

5.1 Rolled Beams and Plate Girders

Rolled beam and plate girder bridges are generally easily constructed, particularly for span lengths less than 200 feet, which encompasses a large majority of the steel bridges constructed in the U.S. For span lengths under 200 feet, the girders can generally be erected with little or no falsework, but in certain cases, lateral bracing may be required to facilitate construction of the bridge. Pier brackets can be used to provide stability over interior supports until the adjacent spans can be erected. Although falsework towers are sometimes used for shorter spans, only when the spans exceed about 200 feet do towers become necessary. Erection of plate girders generally proceeds quickly since there are a limited number of field sections and bolted connections that the erector needs to deal with in the field.

Stability of the girders during erection also becomes increasingly critical as the span lengths exceed 200 feet. Lateral bracing or additional falsework towers may be necessary to verify adequate lateral stiffness or to provide proper positioning of the girders for long span and/or curved girders. While this case should be rare when the girders are properly proportioned, lateral bracing may be particularly useful for spans over 300 feet or for very tall structures where winds encountered may be significantly higher than would be expected on structures close to the ground surface.

To open up the bidding on projects to the largest number of fabricators possible, it is generally advisable to limit the length of field sections to a maximum length of 120 feet, although shipping pieces up to 175 feet have been used. Hauling permits become more expensive for longer sections, especially for lengths greater than 140 feet, and in some cases the piece weight may become prohibitive. The piece weight also affects erection because heavier cranes may be required.

5.2 Trusses

Erection of trusses is more complex and labor intensive than that for welded plate girder structures. There are many more discrete members to erect including individual truss members, lateral bracing, sway bracing and floor system members. Truss bridges also have more bolted field connections than are generally required for plate girder bridges. These require additional labor to erect which generally makes the erection time for trusses longer than for girder bridges.

For simple span trusses, at least two falsework towers are generally required to facilitate erection. They are generally placed two or three panel points away from the supports. The panels can then be erected as cantilevers out past the falsework towers. Depending upon the span length of the truss, counterweights may be required near the end supports to avoid uplift at these locations. It is also necessary to have the ability to provide elevation adjustment at the support locations in order to close the truss at the center. For trusses over navigable waterways, agency

requirements for maintaining temporary navigation channels may dictate the location of falsework towers, and towers constructed in water are usually more costly than towers constructed on land.

Alternatively, the trusses can be built off-site and floated in on barges. After the barges move into place, the trusses are typically strand jacked onto the piers. In certain situations, this can be more economical, but this depends on navigation restrictions, structure size, and site-availability for constructing the truss elsewhere. For additional information, see the FHWA's *Accelerated Bridge Construction – Experience in Design, Fabrication and Erection of Prefabricated Bridge Elements and Systems* [12].



Figure 17 Photo showing the cantilever erection of a truss (Point Marion Bridge over the Monongahela River in southwestern Pennsylvania) (Credit: HDR)

Continuous trusses can usually be erected by using relatively light falsework towers in the back spans to facilitate a balanced cantilever style of erection. As with the simple span truss, the ability to make elevation adjustments must be provided at the support locations to facilitate rotating the trusses into position so that the span can be closed. This can be particularly important when crossing navigable waterways where it is necessary to maintain a navigation channel at all times. While the falsework towers for a simple span truss by necessity are placed adjacent to the navigation channel, the towers for continuous trusses can usually be placed in spans adjacent to the navigation span. This often allows the towers to be lighter while minimizing the risk of the towers being struck by commercial traffic during truss erection.

The discrete piece weights for the truss members are relatively small when compared with some of the other structure types. Thus, the erector can accomplish the erection with smaller cranes than would be necessary for girder or arch bridges.

5.3 Arches

Arches are more complex to erect than trusses or plate girders. The arch rib must have temporary support during erection, either by falsework towers under the rib or through a system of backstays that support the ribs until they can be closed at the crown. Similar to trusses, arches

can be floated into place if arch is over a waterway. Also, arches can be rolled into place with self-propelled modular transports (SPMTs). For additional information, see the FHWA's *Accelerated Bridge Construction – Experience in Design, Fabrication and Erection of Prefabricated Bridge Elements and Systems* [12].



Figure 18 Photo showing an arch being floated into place (Broadway Bridge over the Arkansas River near Little Rock Arkansas) (Credit: HNTB Corporation)

In many cases, the piece weights that are used for arches are heavier than the pieces used for truss members and, in some cases, heavier than those used for plate girders as well. The erector also needs to exercise greater care during erection to provide the proper structure geometry, even more than is generally required for girders or trusses.

Cantilever erection is one of the more common methods of arch erection. This can be accomplished by providing temporary towers to support the partially erected arch. For tied arches, towers will likely be necessary between the rib and the tie to control the distance between the two members. A variation of the cantilever erection method is to provide falsework towers near the end of the ribs with a system of back stays to support the arch during erection. This approach necessitates that the towers can be anchored economically to balance the lateral forces transmitted through the back stays.

Another concern may be the height of the arch. In order to have a span-to-rise ratio that provides an economical design, arches are generally higher (from the spring line to the crown) than a truss with a comparable span. The additional height, in conjunction with the heavier member weights, can require significantly larger cranes than would be required to erect a truss.

When over water, some tied arches have been erected on barge-supported falsework, floated into position and then lowered onto the piers.



Figure 19 Photo showing the erection of a deck arch (Greenfield Bridge over Interstate 376 near Pittsburgh, Pennsylvania) (Credit: HDR)

5.4 Cable-Stayed Bridges

Cable-stayed bridges are becoming more common worldwide. One significant reason for this is that the erection methods required for cable-stayed bridges have become better understood over the last few decades.

The towers for cable-stayed bridges are very tall relative to the span length and in most cases consist of hollow concrete sections. The towers carry very high loads because of the long spans and must also resist significant bending. The foundations are often relatively large, which may lead to constructability issues when crossing rivers or other bodies of water.

Once the towers are completed, the cable-stayed superstructures can be constructed without falsework towers, which can provide significant benefits over navigable waterways. The construction uses a balanced cantilever method proceeding away from the towers until the spans can be closed. The stay cable tensions may require adjustment through the construction process to provide the correct final forces and bridge geometry.

5.5 Suspension Bridges

Erection of suspension bridges is relatively specialized. The installation of the main suspender cables requires expertise that very few contractors possess. Maintaining the geometry of the main cables is critical. The cable saddles on top of the towers may need to be adjusted horizontally during erection to keep the resultant loads on the towers relatively vertical and to avoid horizontal loads at the top of the towers. The tower design may need to be heavier to provide the capacity to resist these loads unless such adjustment is provided.

The time to construct a suspension bridge is often relatively long as well. The foundations for the towers and anchorages are elaborate and time-consuming.

There are also numerous pieces to erect due to the need to provide a stiffening truss to mitigate lateral movement of the bridge. In many cases, complete panels of the stiffening truss/floorbeams are erected together. The longitudinal stringers and deck, or the orthotropic

deck system, are generally installed later. As with the cable-stayed bridges, falsework is usually not necessary to facilitate superstructure erection.

6.0 REFERENCES

1. AASHTO/NSBA Steel Bridge Collaboration. *Guidelines to Design for Constructability and Fabrication, G12.1-2020*. NSBAGDC-4. American Association of State Highway and Transportation Officials, Washington, DC, 2020.
2. NSBA. *Steel Span Weight Curves*. National Steel Bridge Alliance, Chicago, IL, 2019.
3. AASHTO. *Standard Specifications for Highway Bridges, 17th Edition*. American Association of State Highway and Transportation Officials, Washington, DC, 2002.
4. AASHTO. *LRFD Bridge Design Specifications, 9th Edition*. LRFDBDS-9: American Association of State Highway and Transportation Officials, Washington, DC, 2020.
5. NSBA. *Steel Bridge Design Handbook: Stringer Bridges – Making the Right Choice*. National Steel Bridge Alliance, 2021.
6. AASHTO. *Guide Specifications for Analysis and Identification of Fracture Critical Members and System Redundant Members, 1st Edition*. GSFCM-1-UL. American Association of State Highway and Transportation Officials, Washington, DC, 2018.
7. AASHTO. *Guide Specifications for Internal Redundancy of Mechanically-Fastened Built-Up Steel Members, 1st Edition*. GSBSM-1-UL. American Association of State Highway and Transportation Officials, Washington, DC, 2018.
8. NSBA. *Steel Bridge Design Handbook: Redundancy*. National Steel Bridge Alliance, 2021.
9. NSBA. *Practical Steel Tub Girder Design*. National Steel Bridge Alliance, Chicago, IL, 2005.
10. Ryan, T. W., J. E. Mann, Z. M. Chill, B. T. Ott. *Bridge Inspector's Reference Manual (BIRM)*. Federal Highway Administration, National Highway Institute, Washington, DC, 2012.
11. NSBA. *Steel Bridge Design Handbook: Bridge Deck Design*. National Steel Bridge Alliance, 2021.
12. FHWA. *Accelerated Bridge Construction - Experience in Design, Fabrication and Erection of Prefabricated Bridge Elements and Systems*. FHWA-HIF-12-013. Federal Highway Administration, McLean, VA, 2011.
13. NSBA. *Continuous Span Standards*. National Steel Bridge Alliance, Chicago, IL, 2015.
14. NSBA. *Steel Bridge Design Handbook: Design for Constructability*. National Steel Bridge Alliance, 2021.



Smarter. Stronger. Steel.

National Steel Bridge Alliance
312.670.2400 | aisc.org/nsba