Archived

U.S. Department of Transportation
Federal Highway Administration

Steel Bridge Design Handbook

Design for Constructibility

Publication No. FHWA-IF-12-052 - Vol. 11

November 2012

Archived

Notice

This document is disseminated under the sponsorship of the U.S. Department of Transportation in the interest of information exchange. The U.S. Government assumes no liability for use of the information contained in this document. This report does not constitute a standard, specification, or regulation.

Quality Assurance Statement

The Federal Highway Administration provides high-quality information to serve Government, industry, and the public in a manner that promotes public understanding. Standards and policies are used to ensure and maximize the quality, objectivity, utility, and integrity of its information. FHWA periodically reviews quality issues and adjusts its programs and processes to ensure continuous quality improvement.

Steel Bridge Design Handbook: Design for Constructibility

Publication No. FHWA-IF-12-052 - Vol. 11

November 2012

Archived

Archived

Technical Report Documentation Page

1. Report No. FHWA-IF-12-052 - Vol. 11	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Steel Bridge Design Handbook: Design for Constructibility		5. Report Date November 2012
		6. Performing Organization Code
7. Author(s) Roger A. Eaton, PE (HDR) and Calvin Boring (Trumbull Corporation)		8. Performing Organization Report No.
9. Performing Organization Name and Address HDR Engineering, Inc. 11 Stanwix Street Suite 800 Pittsburgh, PA 15222		10. Work Unit No.
		11. Contract or Grant No.
12. Sponsoring Agency Name and Address Office of Bridge Technology Federal Highway Administration 1200 New Jersey Avenue, SE Washington, D.C. 20590		13. Type of Report and Period Covered Technical Report March 2011 – November 2012
		14. Sponsoring Agency Code

15. Supplementary Notes

This module was edited in 2012 by HDR Engineering, Inc., to be current with the AASHTO LRFD Bridge Design Specifications, 5th Edition with 2010 Interims.

16. Abstract

In many cases, bridge designers concern themselves only with checking the stability and member stress levels in the structure for the in-service condition after erection of the entire superstructure is complete. But many times the critical stress in a particular component is encountered during the erection of the bridge where large unbraced lengths affect the stability of the partially completed structure. This module describes typical erection methods and procedures and highlights some of the aspects that should be considered by the designer. With this basic knowledge, the bridge designer can determine which, if any, of these aspects are a concern for his/her particular design situation.

17. Key Words Steel Bridge, Constructibility, Erection, Construction, , Deck Placement, Fit-up	18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161.

19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No of Pages	22. Price

Form DOT F 1700.7 (8-72) Reproduction of completed pages authorized

Archived

Archived

Steel Bridge Design Handbook: Design for Constructibility

Table of Contents

FOREWORD 1

1.0 INTRODUCTION 3

2.0 GENERAL 4

- 2.1 Equipment 4
- 2.2 Erection 7
- 2.3 Erection 9
- 2.4 Site Considerations 15

3.0 DESIGN CONSIDERATIONS 17

- 3.1 Construction Loads 17
- 3.2 Deck Placement 19
- 3.3 Stability 24
- 3.4 Fit-up 27
- 3.5 Erection Engineering and Drawings 29

4.0 REFERENCES 30

Archived

List of Figures

Figure 1 Photograph of a mobile hydraulic crane 4
Figure 2 Photograph of a mobile lattice boom crane 5
Figure 3 Photograph of a lattice boom crawler crane 5
Figure 4 Photograph of a lattice ringer crane 6
Figure 5 Photograph of a tower crane 6
Figure 6 Photograph of a timber crane mat 7
Figure 7 Photograph of a temporary bent 8
Figure 8 Photograph showing the use of pier brackets in an I-girder bridge 8
Figure 9 Photograph of showing the use of a spreader beam for I-girder erection 10
Figure 10 Photograph showing box girder erection using two crawler cranes 10
Figure 11 Photograph showing an I-girder bridge launching 11
Figure 12 Photograph showing an I-girder launch with cable tie-backs 11
Figure 13 Photograph showing cantilever erection of a continuous truss using barge mounted cranes 12
Figure 14 Photograph showing cantilever erection of a continuous truss using derricks 12
Figure 15 Photograph of an arch erection using stay towers and cables 13
Figure 16 Photograph of an arch bridge erection using the floor system as an erection tie to the canyon wall 13
Figure 17 Photograph showing cantilever cable stayed bridge erection using deck mounted derricks 14
Figure 18 Photograph of a suspension bridge erection using cable travelers 15
Figure 19 Photograph showing SIP forms attached to a top flange in a tension region 20
Figure 20 Photograph of a typical overhang bracket 21
Figure 21 Sketch showing a typical overhang bracket and screed application 22
Figure 22 Sketch showing a sample deck placement sequence 23
Figure 23 Sketches showing a half-width construction staging sequence 24
Figure 24 Photograph showing a temporary top flange stiffening truss 26

Archived

FOREWORD

It took an act of Congress to provide funding for the development of this comprehensive handbook in steel bridge design. This 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 is based on the Fifth Edition, including the 2010 Interims, of the AASHTO LRFD Bridge Design Specifications. The hard work of the National Steel Bridge Alliance (NSBA) and prime consultant, HDR Engineering and their sub-consultants in producing this handbook is gratefully acknowledged. This is the culmination of seven years of effort beginning in 2005.

The new *Steel Bridge Design Handbook* is divided into several topics and design examples as follows:

- Bridge Steels and Their Properties
- Bridge Fabrication
- Steel Bridge Shop Drawings
- Structural Behavior
- Selecting the Right Bridge Type
- Stringer Bridges
- Loads and Combinations
- Structural Analysis
- Redundancy
- Limit States
- Design for Constructibility
- Design for Fatigue
- Bracing System Design
- Splice Design
- Bearings
- Substructure Design
- Deck Design
- Load Rating
- Corrosion Protection of Bridges
- Design Example: Three-span Continuous Straight I-Girder Bridge
- Design Example: Two-span Continuous Straight I-Girder Bridge
- Design Example: Two-span Continuous Straight Wide-Flange Beam Bridge
- Design Example: Three-span Continuous Straight Tub-Girder Bridge
- Design Example: Three-span Continuous Curved I-Girder Beam Bridge
- Design Example: Three-span Continuous Curved Tub-Girder Bridge

These topics and design examples are published separately for ease of use, and available for free download at the NSBA and FHWA websites: http://www.steelbridges.org, and http://www.fhwa.dot.gov/bridge, respectively.

Archived

The contributions and constructive review comments during the preparation of the handbook from many engineering processionals are very much appreciated. The readers are encouraged to submit ideas and suggestions for enhancements of future edition of the handbook to Myint Lwin at the following address: Federal Highway Administration, 1200 New Jersey Avenue, S.E., Washington, DC 20590.

M. Myint Lwin, Director
Office of Bridge Technology

Archived

1.0 INTRODUCTION

As time has passed, technological advancements and heightened environmental awareness have led bridge designers to increase the maximum span lengths typically associated with a particular bridge type. For example, with the introduction of High Performance Steels over the last decade, steel plate girder bridges are becoming the structure of choice for spans up to 500 feet in length. The use of these higher strength steels in obtaining such long spans creates the need for designers to consider additional aspects associated with the construction of the bridge. All too often, bridge designers concern themselves only with checking the stability and member stress levels in the structure for the in-service condition after erection of the entire superstructure is complete. But many times the critical stress in a particular component is encountered during the erection of the bridge where large unbraced lengths affect the stability of the partially completed structure. This module describes typical erection methods and procedures and highlights some of the aspects that should be considered by the designer. With this basic knowledge, the bridge designer can determine which, if any, of these aspects are a concern for his/her particular design situation.

Archived

2.0 GENERAL

Bridge erection takes on many forms based on the site, the complexity of the structure, the availability of equipment, and the expertise of the erection contractor. In the following paragraphs, basic erection equipment is discussed with specific examples shown for different bridge types including considerations regarding the access to and topography of the construction site.

2.1 Equipment

Cranes come in various types and sizes. Each type of crane has specific advantages and disadvantages depending on numerous variables such as pick weight, pick height and radius, number of picks, site access, site location, constraints, etc. The following crane types are used in typical bridge erection:

Mobile hydraulic cranes are used for light- to medium-weight picks up to 650 tons. These cranes are used where the site is readily accessible via existing roadways, where pick heights are relatively low, and where crane area is limited. A typical application would be in the replacement of an existing grade separation bridge. These cranes come in a wide variety of sizes such that the appropriate crane can be used for the given pick weight and space availability. The set-up and tear down is quick through the use of telescoping hydraulic outriggers. In addition, the mobility and reach is versatile due to the telescoping boom and 360 degree rotational capability.

Figure 1 Photograph of a mobile hydraulic crane

Mobile lattice boom cranes are used for light-to medium-weight picks up to 300 tons. These cranes are used when the site is accessible via existing roadways and where pick heights are high. Through the use of telescoping hydraulic outriggers and self-assembly capabilities, the set-up and tear down is quick compared to other crane types, generally one to two days for assembly of multiple trailer loads. In addition, the reach is versatile with 360 degree rotational capability. It should be noted that mobile lattice boom cranes and mobile hydraulic cranes cannot move once the pick is lifted.

Archived

Figure 2 Photograph of a mobile lattice boom crane

Lattice boom crawler cranes are used for light-to medium-weight picks up to 300 tons. These cranes are used where the site is typically unfinished terrain and where pick heights are high (up to 400 feet). A typical application would be in the erection of a new bridge over a stream valley. These cranes come in a wide variety of sizes such that the appropriate crane can be used for the given pick weight and height requirements. The set-up and tear-down is long and labor intensive due to the number of components that must be site-assembled to meet hauling and site access restrictions. Once assembled, the mobility and reach is versatile with 360 degree rotational capability. In addition, crawler cranes are able to travel with the load lifted.

Figure 3 Photograph of a lattice boom crawler crane

Lattice ringer cranes are used for heavyweight picks up to 1,400 tons. These cranes are used where the site is typically unfinished open terrain and where pick heights are high (up to 400 feet). Typically, once assembled the crane is immobile due to the track work used to support the massive counterweights. A typical application would be the erection of a new bridge over a wide river or bay where the crane could be mounted onto a barge. By mounting the crane on a

Archived

barge, it becomes mobile increasing it versatility and ultimately making it more productive. The set-up and tear-down is long and labor intensive due to the number of components that must be site-assembled to meet hauling restrictions.

Figure 4 Photograph of a lattice ringer crane

Tower cranes are used for lightweight picks up to 20 tons. These cranes are used where no mobility is required and excessive vertical heights must be overcome. A typical application would be in the construction of a tower for a suspension or cable stayed bridge. These cranes come in a wide variety of sizes to meet the need for a particular height and reach. The setup and tear-down are extremely long due to the assembly process. Quite often, a separate foundation must be constructed to support the base of the tower leg. Once in place the crane can be highly productive in delivering materials to the elevation required. Some models come with self jacking tower legs that allow the crane to adjust its height as construction progresses.

Figure 5 Photograph of a tower crane

Archived

Along with these different crane types, various erection accessories are typically used to maximize the crane's capabilities and function. Some of these accessories are described in the following paragraphs.

2.2 Erection

The art of bridge erection has evolved over time to keep pace with technological advancements in machinery and accessories. Even with today's advanced computer controlled equipment, the most important aspect of bridge erection lies in the experience of the personnel performing the work. Since the objective of safely assembling the structure into its required configuration remains unchanged, the experience necessary to achieve this goal is paramount to success. The following paragraphs discuss different rigging schemes that are traditionally used in bridge erection along with various methods of erection available for I-girder, box girder, truss, arch and cable supported structures.

The rigging of a member in preparation for erection can take on many forms based on the size, weight, geometry and capacity of the individual member as well as the size, capacity and location of the erection crane. In its simplest form, rigging could consist of a single vertical sling hooked to the crane and attached to the member with a beam clamp or through the use of a wire rope sling around the member. In a more complex form, rigging could consist of a multi-point erection frame attached to the member with beam clamps or slings and lifted using multiple cranes attached at different locations along the frame. With both rigging options, tag lines attached to the ends of the member are used for hand guidance to ensure safe placement

Crane mats are series of timber or steel members assembled in sections beneath the crane to provide a level sound base to support the crane during lifting operations. Most often large timbers are used, since they are widely available, inexpensive and easily transported and assembled. If existing ground conditions are poor (such that proper support of the crane would be marginal), the use of a more complex system of steel framing or other custom designed site-cast crane pad could be implemented to span the poor soils.

Figure 6 Photograph of a timber crane mat

Temporary bents are often used in conjunction with cranes to aid in the erection process. Standard scaffolding is widely available for use in creating these temporary supports. Each section of scaffolding is assembled into a geometric configuration that can achieve the required elevation and load carrying capacity needed for each individual situation. Temporary bents are commonly used where it is necessary to provide support for a longer period of time during complex erection and when foundation conditions are favorable, due to shallow rock or good

Archived

soil. Other instances where temporary bents are used include long-span bridge erection, and/or situations where fit-up is to be at the "no load" condition and where site constraints limit the placement of cranes. It is important to note that the temporary bent must be able to be lowered once erection is complete to transfer the load to the structure and facilitate removal.

Figure 7 Photograph of a temporary bent

Temporary pier brackets are often used in the erection of river/water crossings to increase the number of brace and reaction points provided by the permanent pier. Pier brackets are typically comprised of a series of steel members assembled into a frame and temporarily attached to the face of the permanent pier. The configuration can vary greatly based on the geometry of the permanent pier and the intended function of the bracket. The brackets allow individual members to be erected onto the pier with the use of a single crane. The erected member is attached to the permanent bearing and to the pier bracket for stability during the erection of the remaining portion of the spans.

Figure 8 Photograph showing the use of pier brackets in an I-girder bridge

Hydraulic jacks are often used in combination with the temporary bents and pier brackets to allow for flexibility in vertical and horizontal location of the erected member. These jacks are

Archived

readily available in a wide variety of sizes and capacities to meet the requirements of each particular situation. Some other uses for hydraulic jacks include longitudinally aligning a continuous multi-span girder bridge with multiple fixed piers to ensure proper bearing/ pier alignment; aligning the ends of long span continuous trusses and simple span arches to ensure proper fit-up for the closure members; and launching completed bridges into their final location as described in the erection section that follows.

2.3 Erection

The art of bridge erection has evolved over time to keep pace with technological advancements in machinery and accessories. Even with today's advanced computer controlled equipment, the most important aspect of bridge erection lies in the experience of the personnel performing the work. Since the objective of safely assembling the structure into its required configuration remains unchanged, the experience necessary to achieve this goal is paramount to success. The following paragraphs discuss different rigging schemes that are traditionally used in bridge erection along with various methods of erection available for I-girder, box girder, truss, arch and cable supported structures.

The rigging of a member in preparation for erection can take on many forms based on the size, weight, geometry and capacity of the individual member as well as the size, capacity and location of the erection crane. In its simplest form, rigging could consist of a single vertical sling hooked to the crane and attached to the member with a beam clamp or through the use of a wire rope sling around the member. In a more complex form, rigging could consist of a multi-point erection frame attached to the member with beam clamps or slings and lifted using multiple cranes attached at different locations along the frame. With both rigging options, tag lines attached to the ends of the member are used for hand guidance to ensure safe placement.

I-girder erection is often accomplished using a single erection crane. Based on the size, weight and capacity of the girder, the erection crane could use either a single lift point located at the center of gravity of the member or two lift points centered about the center of gravity utilizing a spreader beam. Based on the overall size and complexity associated with the bridge erection, the individual girder erection could incorporate the use of temporary bents, temporary pier brackets or hold cranes to provide stability to the member until a sufficient number of girder lines are erected and connected via permanent cross frames or diaphragms to form a stable system.
Box girder erection is often accomplished using a pair of erection cranes working in tandem during the lift. The cranes are typically rigged with a spreader beam that contains two beam clamps each. The beam clamps are attached to the top flanges of the box girder near the ends of the member. Unlike I-girders, an individual box girder may be stable once it is erected onto its permanent supports, due to the internal bracing system.

Archived

Figure 9 Photograph showing the use of a spreader beam for I-girder erection

Box girder erection is often accomplished using a pair of erection cranes working in tandem during the lift. The cranes are typically rigged with a spreader beam that contains two beam clamps each. The beam clamps are attached to the top flanges of the box girder near the ends of the member. Unlike I-girders, an individual box girder may be stable once it is erected onto its permanent supports, due to the internal bracing system.

Figure 10 Photograph showing box girder erection using two crawler cranes

An alternate girder erection method that has been gaining popularity consists of launching the completely assembled bridge longitudinally across the permanent supports. This method of erection can be utilized for both I-girder and box girder bridges. To accomplish a girder launch, the superstructure is assembled on a roller system behind one of the abutments in segments of sufficient length to maintain stability during the launching. Once a segment is assembled, hydraulic jacks are used to move the structure across the abutment until it reaches the first pier support. After the girders are properly aligned on the first pier, additional segments are assembled on the roller system behind the abutment and the hydraulic jacks advance the structure until it reaches the next pier. This launching process is continued until the entire structure rests on its permanent supports. This erection procedure requires minimal equipment to perform, but the girders must be specifically designed both globally and locally to handle the large forces generated during the launch. In some cases the erection forces in the girders are minimized

Archived

through the use of an erection nose attached to the leading end of the girders, thus reducing the cantilever length, or through the use of a cable tie-back system attached to the top of the girders to counteract the cantilever moments. The cable tie-back system is also used to control deflections of the cantilever end to ensure proper position as each permanent support is reached. This erection method becomes feasible where the bridge height is excessive or when environmental and/or topographical constraints limit the access for erection cranes.

Figure 11 Photograph showing an I-girder bridge launching

Figure 12 Photograph showing an I-girder launch with cable tie-backs

Truss erection methods can vary greatly based on the number of spans, span continuity, site topography, truss type, etc. Methods that have been used successfully over the years are cantilever erection for continuous trusses and float-in erection of the completely assembled bridge for simple spans. The cantilever method of erection for continuous trusses starts with assembling the sections over an interior pier using temporary bents or pier brackets to stabilize the panels. Once the initial pier panels are erected, new panels are added to each end in an alternating fashion until mid-span or the abutment is reached. The same procedure is repeated again until the ends of the two cantilevers are ready for the installation of the closure pieces. Erecting a truss in this fashion can be performed using standard erection equipment or traveling derricks. In addition, erection can proceed simultaneously from each interior pier, and truss geometry and deflection can be controlled using hydraulic jacks at the temporary supports.

Archived

Figure 13 Photograph showing cantilever erection of a continuous truss using barge mounted cranes

Figure 14 Photograph showing cantilever erection of a continuous truss using derricks

Another commonly used method is to assemble truss section components off site, thus, providing erection of assemblies while filling in smaller members piece by piece with smaller equipment.

Simple span truss erection over navigable water can be performed using a scheme where the truss is fully erected on barges, usually on temporary bents, and then floated into position and lifted onto the permanent piers using hydraulics. This method of erection is advantageous where waterway traffic cannot be impeded, the erection contractor has the heavy lifting equipment on hand, or where temporary bents are not feasible due to poor foundation material or excessive water depth.

Arch bridge erection is typically performed using a temporary stay tower located at the spring line (end pier for tied arches) with stay cables connecting the partially completed arch to the tower on the leading end and the tower to counterweights or ground anchors on the trailing end. Erection begins at each spring line and progresses toward the crown. As arch pieces are erected, additional stay cables are added to provide temporary support until the closure piece is in place and the arches are self supporting. Arch pieces can be erected into place by cable and winch systems attached to the leading end of the previous segment or by cranes located beneath the bridge, if feasible. Using temporary stay towers is the most common erection method since arch bridges are predominantly used to span deep canyons (true arch) or wide bodies of water (tied arch).

Archived

Figure 15 Photograph of an arch erection using stay towers and cables

Another method of true arch erection that has been employed uses the canyon walls to support the arch as a cantilever. With this method the arch is erected from each spring line simultaneously towards the crown. As each arch member is erected it is "tied" back into the canyon wall using a temporary connection. The tie member can be part of the permanent structure (as shown below) or a temporary member/cable used solely for erection support. Once the arch is closed and erection complete, the temporary tie members/connections are removed and the arch is self supporting.

Figure 16 Photograph of an arch bridge erection using the floor system as an erection tie to the canyon wall

Cable stayed bridge erection is performed utilizing a permanent tower to support all loads during the assembly of the superstructure. After the majority of the tower construction is completed, the first superstructure segments are erected directly at the tower using temporary supports attached to the tower's cross strut. After the first segment is stabilized, additional members are cantilevered from each end in an alternating fashion until the location of the first permanent cable stay is passed. At this point, the first set of stay cables is installed. Alternating cantilever

Archived

erection of segments continues along with stay cable installation until the end pier or mid-span is reached. This procedure is repeated for each tower of the bridge. When two adjacent cantilever ends are completed, the closure pieces are installed and the erection is finished. This erection method is common, since all of the erection forces are resisted by permanent bridge components. Only simple beam and winch erection equipment attached to the cantilever ends or deck mounted derricks are necessary to lift the members into place.

Figure 17 Photograph showing cantilever cable stayed bridge erection using deck mounted derricks

Suspension bridge erection is performed by using the permanent bridge members for support of the partially completed superstructure. After the main towers are constructed and the main suspension cables are strung, the superstructure erection begins at the tower.

One method of erecting the deck members follows the cantilever scheme where members are erected directly at the tower and attached using temporary supports. After the first members are stabilized, additional pieces are cantilevered from each end in an alternating fashion until the location of the first permanent suspender is passed. At this point, the first set of suspenders is installed to connect the cantilevers to the main cable. Alternating cantilever erection of members continues along with suspender installation until the end pier or mid-span is reached. This procedure is repeated for each tower of the bridge. In addition to erecting the pieces as balanced cantilevers at each individual tower, the erection of the pieces at each tower along the length of the bridge must be considered relative to each other to maintain the shape of the main suspension cables. This will prevent large, unbalanced horizontal forces in the main suspension cables at the towers.

A second method of erecting the deck members is to load the main cables in a manner to try and mimic the final geometry of the cable. With this method individual pieces are supported from the main cables at specific locations prior to connecting any of the pieces to each other. After a sufficient number of pieces are suspended from the main cables, the pieces are connected to each

Archived

other and the remaining pieces erected. This method of erection is used in extremely long span suspension bridges to minimize the jacking forces required to erect the closure pieces.

Figure 18 Photograph of a suspension bridge erection using cable travelers

For both methods, only simple beam and winch erection equipment, typically attached to the main cables (cable travelers), are necessary to lift the members into place.

Since cable-supported structures are typically used for extremely long spans over rivers and open water, delivery of members for erection is accomplished using barges. The cable travelers are used to lift roadway sub assemblies into place directly off of barges.

2.4 Site Considerations

Often the most challenging obstacle in the design of a bridge is a result of the geographical location of the structure. Bridge construction often occurs in heavily populated urban areas where land use is high or scenic rural areas where environmental issues are of paramount importance. Since construction in these areas can dramatically affect the final cost of the bridge, careful consideration must be given to providing adequate access for the contractor and the equipment necessary to perform the work. In the following paragraphs the issues of contractor access and equipment areas are discussed.

Contractor access into and out of the construction site can have a dramatic effect on the overall cost of the project if not properly addressed in the design process. For instance, if a new low-level bridge is to be constructed over a non-navigable waterway 400 feet in width, the structure type most often recommended (based on material cost) would be a 3-span continuous girder bridge having two piers in the water. If environmental constraints due to the sensitive nature of the waterway allow the contractor access to the pier foundations for only one month out of the entire year, a more economical solution may be to completely span the waterway with a simple span truss, thus allowing the contractor unrestricted access to the construction site. Even though

Archived

the material costs would be greater, the construction time would be greatly reduced and the contractor's productivity increased, ultimately reducing his overall cost. For additional discussion on bridge layout and cost, see the Steel Bridge Design Handbook module titled Stringer Bridges.

Another scenario could be the replacement of an existing bridge that spans commuter rail lines in a congested urban area. Again, due to required rail clearances the recommended solution would be to construct a new multi-span girder bridge based on least material cost. If the commuter rail line imposes restrictions on the construction times with extreme penalties associated with violating track outage criteria, the contractor may need to include a large contingency to cover the potential fines. A more economical solution may be to design a single span structure to limit the construction work over and adjacent to the rail lines. The entire simple span structure could be completely erected on the adjacent roadway and lifted into position during a brief off-peak closure of the rail line. Again, the construction time would be greatly reduced, the contractor's risk would decrease, his productivity would increase, and consequently his overall cost would go down.

Another important consideration deals with equipment and storage areas. If the construction site has limited open areas within its boundaries, the contractor may need to mobilize the same equipment numerous times to and from the site as construction progresses. These multiple mobilization costs could make a longer span or more complex structure more economical than a structure that is the most economical based on material cost alone.

During the design process the bridge designer must be cognizant of the issues that cause contractors to increase their cost without adding value. If the bridge designer properly addresses these potential constraints and adjusts his design to minimize or eliminate their impact, the most cost effective bridge solution will be realized.

Archived

3.0 DESIGN CONSIDERATIONS

Various factors must be evaluated by the bridge designer to ensure the constructability of the bridge. In the following paragraphs, basic design topics dealing with construction loads, deck placement, stability, and member fit-up are discussed as they relate to construction of the bridge. In addition, a brief overview of erection engineering and erection drawings is provided. All assumptions made during the design relating to construction loads and construction methods should be documented on the drawings for the contractor to use in developing his/her detailed construction plans.

3.1 Construction Loads

The *AASHTO LRFD Bridge Design Specifications, 5th Edition*, (referred to herein as the AASHTO LRFD (5th Edition, 2010)) (**Error! Reference source not found.**) do not completely address the loadings that should be considered during construction of steel bridges. Requirements for deck placement sequences and overhang deck brackets are supplied, but other conditions are not. Some general statements are provided saying that investigations should be made for handling, transportation and erection, but no quantification is given. Some guidance is provided for the application of load factors for dead loads, dynamic effects (impact) and wind, but specific load combinations are not explicitly defined.

Construction loads that should be evaluated during the design include deck formwork, overhang formwork and brackets, screed rail loads, walkways, handrails, construction live loads, screed live loads, wind loads on the structure and equipment, and any other anticipated loads specific to the particular bridge being designed. In addition, permanent loads, such as the weight of the uncured deck concrete, need to be considered in combination with the construction loads.

Construction equipment loads can be estimated based on bridge construction methods typically used by contractors in the region. Construction equipment could consist of power screeds used for concrete deck placement, work bridges used to support personnel performing deck placement activities, bridge-mounted erection systems (used more often in large, specialized bridge construction), bridge-supported concrete delivery systems, etc. The anticipated equipment loads should be determined for each individual bridge based on bridge type, member size, site location, etc. For example, a typical I-girder bridge would usually be subjected to only screed and work bridge loads.

Wind loads during construction can be one of the most critical aspects to evaluate for conventional girder bridges, since the concrete deck is typically used to transmit these force effects back to the support locations. Until the deck is placed and cured, the individual girders must be capable of transmitting these loads back to the supports through lateral flange bending. If the girders are not capable of resisting the wind loads on their own, a permanent lateral bracing system or another temporary system designed to resist these loads must be provided. The magnitude of the wind loads must be evaluated based on site specific data. The specific data needed to determine appropriate wind loads include the friction velocity, friction length and wind velocity based on the bridge location and the height of the structure (see Article 3.8.1 of the AASHTO LRFD (5th Edition, 2010)). If this data cannot be obtained, the design wind velocity

Archived

could be estimated from bridge crossing information. For example, if a typical girder bridge is being designed over a stream, a design wind velocity of 70 mph may be appropriate for use. On the other hand, if the same bridge is being designed over an active freeway, a design wind velocity of 80 mph may be more appropriate.

Once the design wind velocity is determined, the design wind pressure is calculated using equation (3.8.1.2.1-1) of the AASHTO LRFD (5th Edition, 2010).

$$P_D = P_B \, V_{DZ}^2 / 10{,}000$$

Some owners give the minimum design wind pressure to use in the design. One state mandates the use of 25 psf as a standard and 30 psf for bridges over traffic. Since these values relate to maximum design wind velocities during construction (after erection), lower values should be used to evaluate member stress and stability during the erection of the bridge. Reasonable estimates must be made to correspond with the probable wind velocity during the short erection timeframe. Typical wind velocity ranges for setting initial girders are 25-50 mph for highway bridges. Additional information on wind loads can be found in ASCE/SEI 7-05 (2) and AASHTO's *Guide Design Specifications for Bridge Temporary Works* (3).

Temporary and permanent attachments to the bridge should be evaluated and included in the dead load effects. Examples of various types of temporary attachments include temporary scaffolding suspended from the bridge, temporary safety lines and supports, temporary shielding to protect traffic below the bridge, overhang brackets, temporary hand rails, screed rails, and form work. Examples of permanent attachments include inspection walkways and handrails, utilities, etc.

Site considerations that could be evaluated depending on geographic location, in addition to the wind loads previously described, relate to the effects of ice and snow during construction.

The operational importance of the bridge as it relates to construction should be considered based on location. For example, if the bridge site is located in a highly populated urban area where pedestrian and vehicular traffic on or under the bridge will be at risk due to construction activities, a higher level of safety may be necessary to minimize the potential for injury. The steps used to account for this in the design could include a blanket increase in the importance factor (η_I, see the Steel Bridge Design Handbook module *titled Loads and Combinations* for additional discussion) that will result in a higher level of safety through the use of greater loads, or localized provisions incorporated into the design of protection systems adjacent to the areas of concern. Good engineering judgment must to be used to determine the site-specific effects associated with each bridge and the appropriate level of design to be performed.

Construction live loads should also be considered in evaluating the adequacy of the bridge. This loading is intended to cover all miscellaneous equipment and personnel that cannot easily be quantified at the time of design. Often a blanket allowance of 10 psf is used to account for these loadings.

Load combinations must be evaluated to capture all critical conditions during construction of the bridge after a thorough determination of all anticipated construction loads is complete. In the

Archived

2005 interims, Article 3.4.2 of the AASHTO LRFD (5th Edition, 2010) calls for the use of Strength Load Combinations I, III and V during construction. In evaluating these combinations, engineering judgment must be exercised to ensure that the maximum feasible forces are being evaluated. Some sample load combinations are:

γ_p(DC+DW) + 1.5(CEL+CLL) for STR I

Where:

γ_p is the load factor for dead load 0.9 min and 1.25 max (see the Steel Bridge Design Handbook module titled Loads and Combinations for a detailed explanation).

DC includes all dead load associated with the bridge members and all form work, attachments, deck, etc.

DW includes all utilities.

CEL includes all construction equipment loads such as screeds, etc.

CLL is the construction live load.

γ_p(DC+DW) + 1.5(CEL) + 1.4(WS) for STR III

Where:

WS is the wind on the exposed height of the structure including all forming.

γp(DC+DW) + 1.5(CEL) + 1.35(CLL) + 0.4(WS) +1.0(WCEL) for STR V

Where:

WCEL is the wind on construction equipment (screeds, etc).

If there are varying stages of structural configuration associated with the design, additional construction load combinations should be evaluated to ensure that all pertinent conditions are checked. For example, if the girders behave as simply supported for self weight and continuous for all other dead and construction loads, then both conditions should be evaluated using the appropriate loadings and capacities.

3.2 Deck Placement

Typically, the most critical stage in the construction of girder bridges occurs during placement of the concrete deck, since the bridge is being loaded with the majority of its dead load while only discrete bracing stabilizes the most vulnerable top compression flange components. The following paragraphs describe typical interior deck forming, standard overhang bracket systems,

Archived

deck finishing machines (screeds), deck pouring sequences, and considerations for half-width construction.

Deck forming between interior girders typically consists of corrugated metal stay-in-place (SIP) members attached to the top flange of the girders. The SIP forms are supported by angles attached to the edges of the girders' top flange using a strapping detail spanning the flange (in tension and stress reversal areas) or by a direct weld to the flange (in compression areas). The SIP forms are usually attached to the angles using self tapping screws. Typical SIP forms can be used with girder spacings up to approximately 14 feet. SIP forms are available in a wide variety of types and sizes to meet the individual needs associated with each bridge. There are numerous suppliers of the products, making them widely available for use. For additional information about SIP forms, see the Steel Deck Institute website at www.sdi.org.

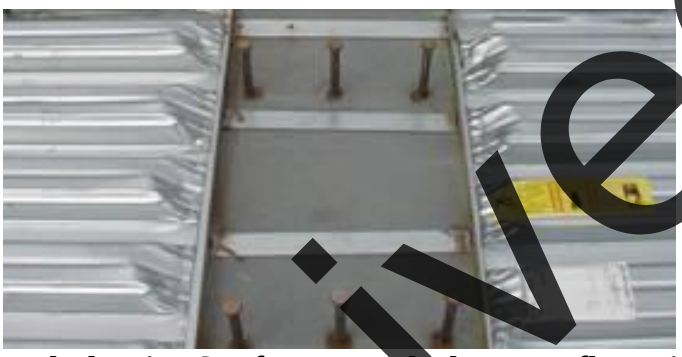

Figure 19 Photograph showing SIP forms attached to a top flange in a tension region

As an alternative to SIP forms, traditional removable formwork can be utilized between the flanges of the interior girders. Removable formwork usually consists of plywood attached to timber supports or prefabricated metal forms, either of which would be held in position by blocking down to the bottom flanges or hangers attached to the top flanges. Although this method of forming is more labor-intensive, some owners require inspection access to the underside of the concrete deck or are concerned about consolidation of the concrete in the SIP form valleys. To mitigate this concern some owners permit the placement of Styrofoam strips in the valleys. This has an added benefit since it lightens the deck dead load.

Overhang brackets are typically used to support deck forms in the area beyond the edges of the fascia girders. Standard overhang brackets consist of three steel members connected in a triangular shape that are suspended from the top flange and supported by the girder web. The brackets are normally spaced on three to four foot centers along the length of the bridge and support traditional wood forming. In addition to supporting the form work for the deck overhang, the brackets usually carry an access walkway and screed rail to allow for deck placement machinery and personnel. The overhang brackets can create significant lateral flange bending forces in the top and bottom flanges of the fascia girder due to the eccentricity of the loads and the hanger connection to the flange. Since these lateral flange bending loads can often control the design of the top flange, the designer must calculate the magnitude of these loads and verify the capacity of the girder. The AASHTO LRFD (5th Edition, 2010) provides a method for determining the lateral flange bending moments in the commentary of Article 6.10.3.4. If the web depth is such that the bottom of the overhang bracket would likely rest against the web a

Archived

significant distance above the flange to web intersection, constructability stiffeners are often shown on the plans for the fascia girders to resist the bracket loading. The contractor could choose to thicken the web to avoid the additional stiffeners or perform a refined analysis to verify the effects on the web, but discussion with the engineer is essential. In addition to the lateral flange bending effect, the designer must also include all vertical loads associated with the overhang bracket into the capacity checks for the exterior girders. Detailed information on the required capacity checks can be found in the AASHTO LRFD (5th Edition, 2010). For additional information on overhang brackets and bridge deck formwork, see the *AASHTO Construction Handbook for Bridge Temporary Works* (4).

Figure 20 Photograph of a typical overhang bracket

Archived

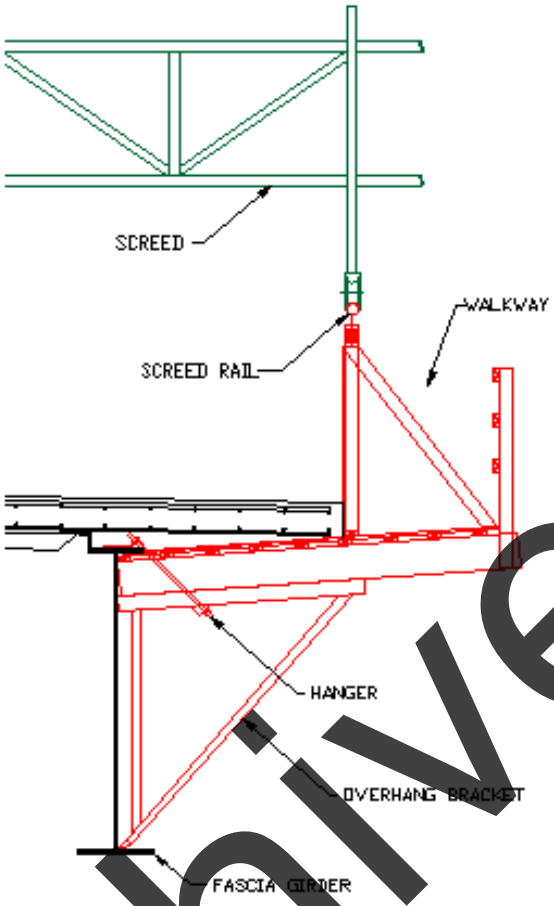

Figure 21 Sketch showing a typical overhang bracket and screed application

Deck finishing machine (screed) loads can also have a dramatic effect in the design of the fascia girders, since the loads are typically supported by the overhang brackets as previously described. Therefore, an estimate of the machine loads should be performed and included in the capacity evaluations. A good estimate of the machinery weight can be made by simply selecting an appropriately sized piece of equipment based on the width of the bridge and the placement sequence developed for the design. With this information, actual weights can be readily obtained from local equipment suppliers in the region of the bridge.

A deck placement sequence must be developed by the designer to ensure that the girders are capable of supporting the applied loads during each stage of placement. In Article 6.10.3.4 of the AASHTO LRFD (5th Edition, 2010) and its commentary explicit direction is given regarding the type of calculations that must developed. An example of a typical deck placement sequence for a two-span continuous girder bridge would be as follows: First, place the concrete in the positive moment portions of the spans leaving a block-out at each end for installation of the expansion devices (placement 1). Note that all of the concrete in placement 1 must remain plastic until the pour is complete. Allow the concrete to cure for a specified time or until a certain strength is obtained (this requirement is usually established by the owner). Second, place the concrete in the negative moment region over the pier up to the concrete previously placed during Stage 1 (placement 2). Finally, install the expansion devices at each abutment and place concrete in the

Archived

block-outs (placement 3), thus finishing the concrete deck placement. Construction of the protective concrete barriers finishes the deck.

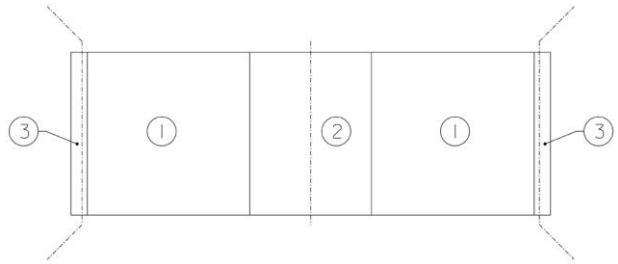

Figure 22 Sketch showing a sample deck placement sequence

Half-width staged construction is typical for bridge replacement or rehabilitation projects where traffic must be maintained on the bridge throughout all construction stages. A simple example would be the replacement of an existing bridge that carries two 12-foot wide travel lanes, two 12-foot wide outside shoulders, and a single turning lane. The ideal condition would be to maintain a single lane of traffic in each direction on half of the existing bridge while the other half is removed and replaced. Once this first stage of construction is complete, traffic would be transferred onto the new bridge while the remaining portion of the existing bridge is removed and replaced. The difficulty associated with this procedure lies in the connection of the first half of the new structure to the second half; particularly how to place the concrete deck in the connecting region. One method of performing this work is to build most of the second half of the bridge independently from the first by omitting the cross-frames connecting the two. In doing this, the second half of the bridge would be erected and the deck placed up to the centerline of the girder adjacent to the half of the bridge carrying traffic. At this point the cross-frames connecting the two independent structures into one would be installed, and the closure deck placement made. Many times the cross-frames connections are prepared only with shop drilled holes for connection to first half of the bridge. Once the cross-frames are erected and connected to the first half of the bridge, the bolt holes for the connection to the second half of the bridge are field drilled. Typically the holes in the cross frame members are used as a drill template to field drill the holes in the stiffener plate. Performing the work in this fashion allows for proper fit-up of the steel girders and reduces the amount of “locked-in” forces carried by the cross-frames connecting across construction stages.

Archived

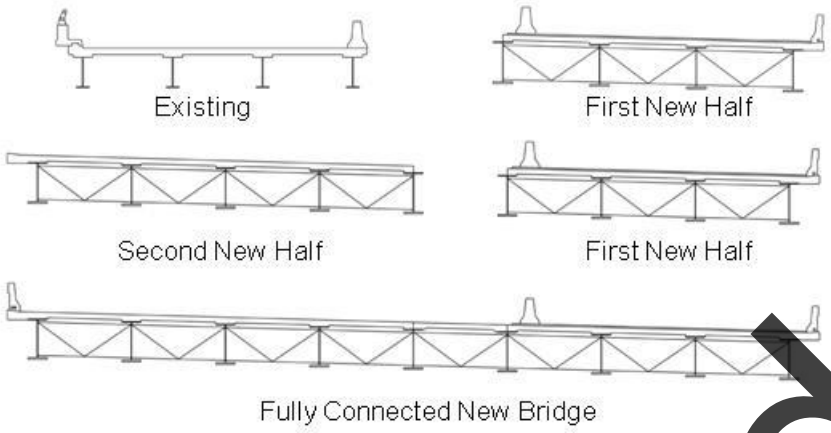

Figure 23 Sketches showing a half-width construction staging sequence

3.3 Stability

Stability of the girders during erection and subsequent construction stages is of primary importance to the designer since it is typically the driving factor in the selection of crossframe spacings, top flange width and lateral bracing requirements. The following paragraphs discuss stability of girders and how it relates to cross-frames, lateral bracing, temporary bracing, uplift at supports, bearing selection, and construction staging.

Cross-frames are typically considered secondary bracing members in girder bridges (with the exception of highly skewed and horizontally curved bridges) where the primary function is to transmit wind forces from the bottom flange into the concrete deck in the completed bridge and to provide bracing points for the girder flanges during construction (and in the final configuration for bottom flanges in the negative moment regions). The AASHTO LRFD (5th Edition, 2010) eliminated the limits on maximum spacing for cross-frames found in previous specifications. Instead, it is up to the designer to establish the spacing and properly design the girders for the forces accordingly. By removing the arbitrary 25 foot maximum spacing requirement from the code, it was not the intent of the code writers to have designers place cross-frames at large distances. Instead, the spacing requirement was removed to allow the designer flexibility in establishing reasonable equal spacing for the cross-frames (on the order of 20-30 feet) without the arbitrary maximum that often results in an additional line of cross-frames simply to comply with the code. Cross-frame spacing is established to avoid flange buckling and limit the amount of lateral flange bending based on the proportions of the girder and geometric layout of the bridge.

The configuration of the individual members that make up a cross-frame can take on various shapes depending on the ratio of the girder spacing to depth. Typical cross-frame configurations are K-frames, X-frames, solid bent plates, rolled sections, plate girders, inverted K-frames, and X-frames with horizontal struts. For a more detailed explanation of cross-frames and their practical uses, see the Steel Bridge Design Handbook module titled Stringer Bridges.

Once a cross-frame spacing and configuration are determined, the girders lateral stresses must be checked for construction loads and permanent wind loads to determine if the results are acceptable. If the lateral effects control the design of the flanges, the cross-frame spacing should

Archived

probably be reduced to allow for a balance between the vertical and lateral bending effects. One rule of thumb commonly used is to limit the lateral flange bending stresses to around $0.5F_y$. Lateral bracing is typically used in longer span structures where the lateral flange bending stresses and/or lateral deflection cannot be effectively controlled by flange width and cross-frame spacing. The need for lateral bracing may often be dictated by the client. One owner requires designs that do not use lateral bracing for spans under 200 feet, allows it to be investigated for spans between 200 and 300 feet, and requires it on spans over 300 feet. The bracing usually consists of WT sections and is normally connected directly to either girder flange. The lateral bracing creates a truss system between adjacent girders with the girder flanges acting as the chords, and the bracing members and cross-frames acting as the diagonals and "verticals", respectively. The optimum location (top or bottom flange) and configuration of the bracing has been debated for years and no explicit conclusions have been determined. The only definitive observation that can be made is that lateral bracing dramatically increases the lateral stiffness of the bridge regardless of location and configuration.

Often the largest drawback in the use of lateral bracing is the need to design the bracing connections for vertical girder bending effects. Frequently, the designer will utilize the bracing for final wind and construction effects and allow the connections to "slip" for vertical capacity determination. This is accomplished through the use of oversized holes in the slip critical connection design. For additional information on lateral bracing considerations, see the Steel Bridge Design Handbook module titled Stringer Bridges.

Temporary bracing is sometimes utilized to provide stability during the construction of the bridge. For example, many owners permit the use of temporary external cross-frames between boxes during the construction of steel tub girder bridges. The advantages of using temporary cross-frames are that: (**Error! Reference source not found.**) the aesthetics of the bridge, when viewed from below, are maximized with the completed bridge having clean, unobstructed lines; (**Error! Reference source not found.**) the details of the members are simplified, since they do not need to meet long-term fatigue and maintenance criteria; and (3) the flexibility given to the contractor regarding erection methods is also maximized. There are also some drawbacks to the use of temporary cross-frames: (**Error! Reference source not found.**) the members can be difficult to remove, since crane access is limited due to the concrete deck being in place; (**Error! Reference source not found.**) the contractor must remobilize equipment and personnel to remove the members long after their role on the project is complete; and (2) if the bridge is redecked in the future, the temporary cross-frames must be reinstalled. The advantages and disadvantages of using temporary bracing in a situation such as this must be thoroughly investigated before the decision is made to incorporate its use.

Another situation where temporary bracing could be used is in the erection of long span I-girder bridges. An example would be the use of a temporary stiffening truss attached to the top flange of a single I-girder to stabilize it until a second line of girders is erected and attached with cross-frames to form a stable system. In this instance, the use of the temporary truss allows the contractor flexibility in the use of the erection equipment to perform girder picks without the need for addition hold cranes. One drawback to this scenario is the cost associated with fabricating and assembling the truss. Again, the benefit of needing fewer cranes needs to be weighed against the cost of the additional material.

Archived

Figure 24 Photograph showing a temporary top flange stiffening truss

Uplift at support locations can lead to design issues during the construction stages as well as in the final configuration of the bridge. Typical girder bridges are proportioned to ensure that uplift does not occur at the support locations in the final configuration. Uplift is usually only a concern when dealing with bridges that contain sharply skewed supports, tight horizontal curvature, or incorporate complex bridge types. If a bridge configuration results in uplift, a traditional design solution is to attach the bridge to the substructure unit using a tension link. Although this approach has been used successfully for decades, it usually incorporates a fatigue-sensitive detail into the bridge. As a result, owners now typically require the designer to resist uplift effects through the use of additional permanent dead load. This can be successfully accomplished by incorporating full-depth concrete diaphragms over the support location or, if necessary, by placing permanent counterweights (usually a large volume of concrete) at the support in question. During the construction of the bridge, uplift can occur at various stages of the erection process depending on the methods being used. In these situations, the uplift can be resisted through the use of temporary counterweights positioned over the support or temporary cables anchored into the foundations. Temporary counterweights are commonly used when bridges are erected using the "cantilever method" as previously described.

Bearing selection is normally based on the final bridge configuration and the needs associated with intended long term performance of the bridge. Quite often the result is a single line of fixed bearings in the bridge with the remainder of the bearings being either free (unrestrained in all directions) or guided (normally restrained in the transverse direction). During erection of the bridge, such an arrangement can create complications since the permanent support locations are normally used to provide stability. Therefore, the permanent bearings are often augmented to provide the necessary fixity during erection. The type of modification depends on the specific permanent bearing type used, the type of substructure, the bridge type, the size of the bridge, etc. Many times a typical girder bridge is outfitted with blocking around the bearing to provide the necessary support or, if necessary, a system of temporary cables is used to anchor the girder to the substructure unit. To ensure proper fit-up, a survey of the horizontal and vertical alignment of the bearings should be made prior to erection.

Construction staging must be evaluated by the designer to ensure stability of the bridge during all phases of the initial deck placement sequence, the conditions anticipated for the proposed future redecking scheme, and every stage associated with half-width construction. For example, some

Archived

owners require five girder lines when four would suffice, specifically to accommodate future redecking schemes. As previously mentioned, Article 6.10.3 of the AASHTO LRFD (5th Edition, 2010) describes the explicit method of performing these design checks.

3.4 Fit-up

Fit-up of the steel bridge members is a critical component in the overall success of a project. The designer must be able to clearly convey to the fabricator and erector the intent of the design as it relates to bolting and pinning, camber, bolted field splices, and differential deflections. The use of oversize holes to facilitate fit-up is not recommended. In the following paragraphs some important aspects relating to these topics are discussed.

Bolting and pinning of connections requires that proper procedures be followed. NCHRP Synthesis 345, "Steel Bridge Erection Practices" recommends that the procedures might contain the following:

- Initial pinning and bolting should consist of filling the holes in the connections with 25% pins and 25% bolts and the bolts at least snug tightened before releasing the crane and having the adjacent girders erected.

- After the cranes are released, the balance of the holes in the connections should be filled with snug-tight bolts.

- Final tightening of the bolts to installation tension should not start until a continuous line or at least adjacent spans have been erected and the vertical and horizontal alignment has been verified.

- Pins should not be removed from the connection until after the previous step has been accomplished.

These recommendations are for typical bridges, for large or complex structures the number of bolts and pins required need to be determined by design for each connection.

Cambers are values prescribed by the bridge type, member size, and geometry for each particular structure. The cambers are derived by the designer to ensure that the bridge achieves its final design geometry after all dead loads are applied. A more detailed explanation of cambers for girder bridges is provided in the Steel Bridge Design Handbook module titled Stringer Bridges. For other larger bridge types the cambers can have a dramatic effect on the overall behavior of the bridge. For example, arch bridges must be properly cambered to ensure that the axis of the arch closely conforms to the dead load thrust line, since this is a standard assumption in the design of the arch members. If the cambers are incorrect, significant vertical bending will be generated in the arch rib due to the dead load, subjecting the bridge to loads that were not accounted for in the design. See Article 6.7.2 of the AASHTO LRFD (5th Edition, 2010) for camber requirements.

Bolted field splices on girder bridges are located to separate the individual girders into manageable pieces in terms of both size and weight. See the Steel Bridge Design Handbook module titled Stringer Bridges for a general discussion regarding splice locations and hauling

Archived

considerations. The designer should also be cognizant of member piece weights, since other concerns such as site access could limit the maximum member size. If member size is limited for concerns other than hauling, additional field splice locations may need to be designed and incorporated into the design drawings. It should be noted in the design drawings that field splice locations can be eliminated by the contractor. This will allow the contractor to tailor the field piece size and number of splice locations to his/her erection scheme, thus minimizing the cost of the bridge.

Differential defections between adjacent girder lines can result from severe skew, horizontal curvature, staged construction, or extreme load imbalance. Many of the issues associated with differential deflections result from lack of communication among the designer, fabricator and erector.

One way to describe the effect of differential deflection is through the example of a severely skewed bridge: A two-span multi-girder bridge has a skew angle of 70 degrees between the center line of supports and a line normal to the bridge centerline. With a skew angle this severe, the intermediate cross-frames would be detailed perpendicular to the girder webs. Therefore, one end of a particular cross-frame may be located at the 0.4L point of Girder 1 and the other end at the 0.5L point of Girder 2, where L is the span length. A comparison of the dead load deflections at these two locations would show that Girder 2 deflects much more than Girder 1. If the cross-frames are not properly detailed to account for the differential deflection between Girders 1 and 2, the erector may not be able to properly install the member.

If the girders' webs are detailed to be vertical under full dead load, then the girders must be erected with a predetermined twist (out-of-plumbness) such that when the erection is complete and all dead loads are applied (deck, barriers, etc) the differential deflection between cross-frame ends will "roll" the girder into its final vertical position. For this condition, the designer must clearly state on the drawings that the intent is to detail and fabricate the members such that the webs will be vertical in their full dead load position, and the erector needs to realize that the girders need to be erected with the predetermined twist.

For curved girders the key element in dealing with differential deflection is to provide consistent detailing between all components in the bridge. In the Second Edition of the AASHTO LRFD *Bridge Construction Specifications* (5) it states, "Unless otherwise specified in the Contractor's construction plan, fit-up shall be assumed to be performed under the no-load condition". Therefore, unless the design drawings explicitly define a different condition, the fabricator and erector should construct the bridge as though it is in the no-load condition. Good engineering judgment must be used in determining the appropriate condition to specify for each bridge.

Bridges that are both curved and skewed are beyond the scope of this document since this configuration is not currently covered in the AASHTO LRFD (5th Edition, 2010), and no recommendations are available. Additional research needs to be performed to support current theories.

For additional information on differential deflections see Section 1.6 of the *Guidelines for Design for Constructability* document G12.1-2003 by the AASHTO/NSBA Steel Bridge

Archived

Collaboration (6). See the Collaboration website at www.steelbridge.org/standards.htm for access to this document.

3.5 Erection Engineering and Drawings

After award of the erection contract, the erector must develop a clear detailed procedure for assembling the structure. Based on a recent survey of bridge owners, very few require the erector to develop a detailed erection procedure. But such a formal procedure is a critical step in the construction of the bridge and should be mandated. If the owner does not have standard requirements, the designer should develop a contract provision requiring the erector to develop a plan for each specific project. The procedure must include detailed drawings and written descriptions for each step necessary to complete the erection. In addition, complete calculations performed by a registered professional engineer must be developed for each stage of the process to ensure that the permanent bridge members are not overstressed and stability is maintained.

Provisions for erection procedures and erection drawings can be found in the Second Edition of the AASHTO LRFD *Bridge Construction Specifications* (5). In addition, the AASHTO/NSBA Steel Bridge Collaboration Task Group 10 has developed a document titled *Steel Bridge Erection Guide Specification* (7) that covers erection calculations and drawings in greater detail. The latest information can be found on the Collaboration website at www.steelbridge.org.

Archived

4.0 REFERENCES

1. AASHTO, (2010). AASHTO LRFD Bridge Design Specifications; 5th Edition, AASHTO, Washington D.C.

2. ASCE 7-05, (2006); American Society of Civil Engineers (ASCE 7-05), Minimum Design Loads for Buildings and Other Structures, ASCE Reston, Virginia.

3. AASHTO, (1995). AASHTO Guide Design Specification for Bridge Temporary Works, AASHTO, Washington D.C.

4. AASHTO, (1995). AASHTO Construction Handbook for Bridge Temporary Works, AASHTO, Washington D.C.

5. AASHTO, (2004). AASHTO LRFD Bridge Construction Specifications, Second Edition, AASHTO, Washington D.C.

6. AASHTO/NSBA, (2003). *G12.1-2003: Guidelines for Design for Constructibility*, AASHTO/NSBA Steel Bridge Collaboration.

7. AASHTO/NSBA, (2007). *S10.1-2007: Steel Bridge Erection Guide Specification*, AASHTO/NSBA Steel Bridge Collaboration.

Archived