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CHAPTER 6

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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.

Steel Bridge Design Handbook: Stringer Bridges and Making the Right Choices

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1.0 INTRODUCTION

Once a bridge type is selected, the designer then advances to the detailed design of the bridge. Since the vast majority of steel bridges designed today are steel girders made composite with concrete bridge decks, this volume will cover many detail issues that are encountered when designing a composite deck girder system. This volume addresses the design of welded plate girders. However, many of the principles presented are also applicable to the design of rolled beam bridges.

2.0 SPAN ARRANGEMENT SELECTION

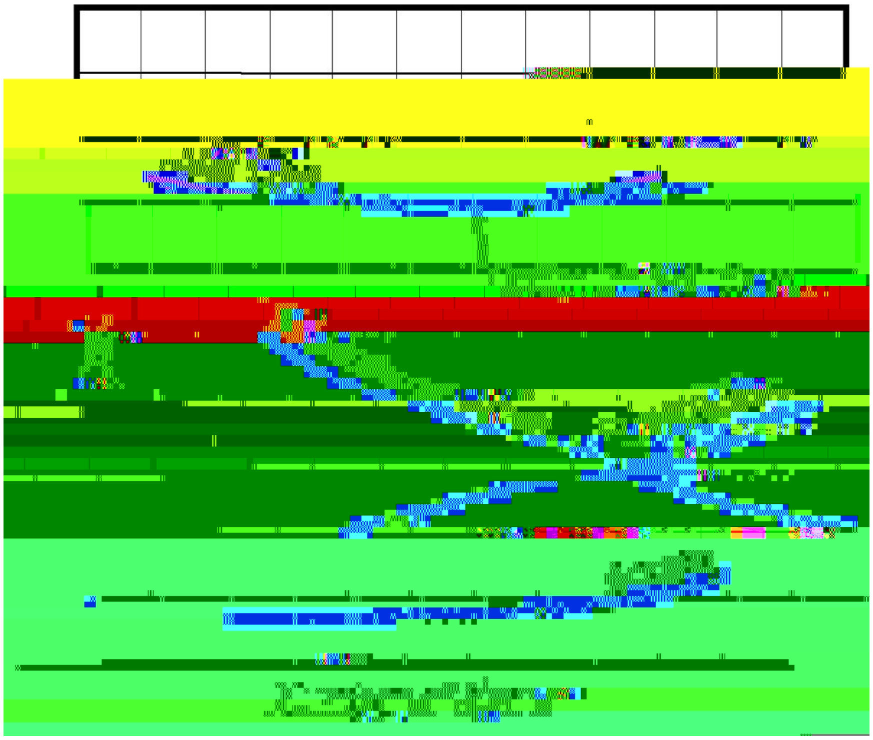

Figure 1

3.2 Redecking

In many cases, owners now require designers to develop framing options that will permit a phased partial-width deck replacement to occur safely while maintaining traffic on the structure. Depending upon the bridge width, designing to accommodate a staged redecking may require an additional girder beyond what would be optimal. However, the life-cycle cost savings provided by the staged redecking may outweigh the cost of the additional girder in the initial design.

4.0 CROSS-FRAME/DIAPHRAGM SELECTION

Historically, intermediate crossframes have been assumed to provide intermediate bracing for the girders during erection, particularly for the top flanges in the positive moment regions. The live load distribution factors contained in the AASHTO LRFD Bridge Design Specifications, 9th Edition (2020), (referred to herein as the AASHTO LRFD BDS) (1), were based on the assumption that live load distribution between the girders occurs through the deck stiffness rather than through frame action provided by the intermediate cross-frames. Crossframes have not been assumed to distribute live load except for curved girder bridges.

Top flanges of composite girders in positive moment regions are braced by the crossframes prior to hardening of the concrete decks. Intermediate crossframes for continuous composite girder bridges also provide bracing against lateral buckling of the compression flange in the negative moment regions both during erection and after the deck is placed. Additionally, intermediate cross-frames provide bracing for lateral wind loads on deep girders.

On skewed composite girder bridges, the cross-frames are assumed not to carry live load if the live load distribution is based on the factors found in the AASHTO LRFD BDS. If a grid or refined analysis is used that models the stiffness of the crossframes in the analysis, then the intermediate crossframes should be designed for the loads computed from the analysis results.

For curved girder bridges, the intermediate cross-frames play a significant role in the live load distribution and need to be designed and detailed as main load carrying members.
As with the intermediate crossframes, the end crossframes at abutments and those at the piers provide bracing during erection of composite steel girders. However, all support crossframes are required to distribute lateral loads from the superstructure to the substructure. These loads include wind, centrifugal, seismic and thermal forces for some curved girder bridges. In addition, end support crossframes generally are 405.19 Tm [(f)813ocorry ldirre 405.0 g 0 G [(s)-10wE62O W* 0

Figure 5 Detail sketch of a typical Knocked-Down cross-frame

5.0 GIRDER DESIGN

5.1 Selection of Appropriate Analysis Methods

Given the current level of advancement in computer software for girder analysis, as well as the lowest weight girder has historically provided the most-cost-effective solution. This is true only if the girder details are well-conceived and the designer is attentive to industry input on cost-effective details.

In some cases, the girder depth will be determined in order to optimize the appearance of the bridge. In most cases, more slender bridges are more attractive. Thus, sha 12 4 681.tiractive. Thus, sha constant within field sections of the girder. This will permit the fabricator to slab and rip the flanges, as illustrated in Figure 6 improve the lateral stability during fabrication and erection or to avoid flanges that are excessively thick.

Another rule of thumb is to limit flange transitions such that the smaller flange at a welded transition is no less than 50% of the area of the larger flange. This accomplishes two things. First, the bending stress gradient in the girder web due to the change in section properties does not become overly steep when this criterion is met. It has also been demonstrated in past designs that, if the flange transition results in greater than a 50% reduction in flange area, either the transition is not in the optimum location or an additional transition may prove to be economical.

One important design parameter in providing the appropriate number of welded flange transitions is to ensure that the fabrication cost associated with the butt weld does not exceed the material cost savings resulting from the flange transition. Each fabricator has their own parameters for determining the economy of welded flange transitions, which are considered proprietary information. However, there are two general approaches to determining the economy of welded transitions that have garnered some level of acceptance within the design community.

The first method (2) was developed in the 1970s and has served well over the years in avoiding excessive numbers of welded flange transitions, and uses equations based on flange areas and the yield strength of the steel. The equations are as follows:

For 36 ksi steel:

$$\text{Wt. Savings} \geq \text{Area of smaller flange (in.}^2\text{)}$$

For 50 ksi steel:

$$\text{Wt. Savings (Wt. Savings for 36 ksi)}$$

For 100 ksi steel:

$$\text{Wt. Savings} \geq \text{Wt. Savings for ksi}$$

This approach has typically yielded transitions that have been economical and not subject to redesign. However, these equations were developed in an era when material was a larger percentage of the fabrication cost than was the labor cost. In recent years, this trend has changed to the point that the labor costs during fabrication are a much larger percentage of the total cost, and thus developing a different method for determining the economy of butt-welded transitions was needed. As a result of these changes, the AASHTO/NSBA Steel Bridge Collaboration Guideline G12.1, Guidelines to Design for Constructability and Fabrication (3) has developed a method for determining the economy of butt-welded flange transitions that places a higher premium on the labor costs associated with fabrication than the earlier equations. Table 1 illustrates the suggested criteria for assessment of the economy of welded flange transitions.

It is prudent to consider both methods when assessing economy of welded transitions and leaning towards one or the other dependent upon the current market conditions. When

A partially stiffened design entails using a web 1/16 to 1/8 inch thicker than would be used for a fully stiffened design. This type of design will generally require transverse stiffeners in the first one or two bays between diaphragms at each end of each span.

An unstiffened design entails using a web thickness such that the shear buckling resistance of the web is equal to or greater than the factored shear demand. An unstiffened design would require only bearing stiffeners at the supports and diaphragm connection plates.

While the material costs do increase when unstiffened webs are used, there may be little change in the total fabrication cost of the fabricated girder. The amount of welding for the flange-to-web welds does not increase since minimum welds are generally adequate, thus limiting the increase in cost for the extra web material to the basic material cost of the steel. There may be a corresponding decrease in the size of the girder flanges when the thicker webs are used due to the increased web stiffness, and this decrease in flange material helps to offset the increased web material cost. Elimination of transverse stiffeners reduces labor costs associated with fabrication, fit-up and welding of the stiffener plates.

Other benefits associated with unstiffened webs are becoming increasingly important. If the girder is a painted design, minimizing the number of transverse stiffeners provides both a first cost benefit as well as a life cycle cost benefit by reducing the surface area requiring painting. The cost of bridge inspections may also be reduced since there are fewer details that require close inspection.

A fully stiffened design will provide the lightest possible web design, but will also have the highest unit fabrication cost of the three options. An unstiffened design will result in the heaviest design of the three options, but should have the lowest unit fabrication cost of the three. The partially stiffened option provides a trade-off between unit fabrication cost and material cost. Throughout the late 1980s and the 1990s, the predominant opinion throughout the fabrication industry was that partially stiffened girder webs provided the optimum solution. However, the percentage of total girder cost related to fabrication labor cost has increased relative to the percentage of cost associated with material. Consideration should be given to the use of unstiffened girder webs. However, partially stiffened webs, especially for spans that require only one or possibly two stiffeners per panel near the interior supports, should still prove to be cost effective.

When comparing the cost of additional stiffeners to the cost of the extra web material associated with an increase in thickness, the stiffener unit material cost should be assumed to be approximately 4 to 5 times the base material cost of the web to account for the additional fabrication required to weld the stiffeners to the girder.

Transverse stiffeners are important in minimizing the overall weight of the girders because they allow the web thickness to be minimized. However, there is a distinct cost associated with transverse stiffeners. There is a relatively large amount of welding associated with transverse stiffeners for the weight of steel involved, and the process is not as easily automated in the shop as are flange-to-web welds. Therefore, the increased stiffener cost must be balanced against the material savings associated with a reduction in web material.

The use of longitudinally stiffened girder webs becomes a consideration for web depths above 120 inches. For girder depths less than 120 inches, it has generally proven more economical to increase the web thickness rather than to include longitudinal web stiffeners. Longitudinal stiffeners are generally placed at approximately D/5 from the compression flange. This forces a buckling node in the web at the longitudinal stiffener location, allowing the compression depth of the web to be decreased accordingly when computing a required thickness. The web thickness can generally be reduced proportionally to this reduction, significantly reducing the amount of web material used. The AASHTO LRFD BDS now provides a method by which to compute the most common steels used in bridge girders are Grades 50 and 50W. Homogeneous designs in spans shorter than 200 feet have proven to be reasonably cost-effective over time.

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as close as practical to the edge of the girder flange. Bearing stiffeners are required on both sides of the beam or girder web.

There are two basic design criteria for bearing stiffeners. First, the bearing stress between the stiffener and the bottom flange must not exceed the bearing capacity of steel on steel. This ch is performed based on the area of the bearing stiffeners only, accounting for the width removed by the chamfer at the base of the stiffener. The girder web is not assumed to contribute to the bearing capacity of the stiffener.

The second check is an axial compression check of the

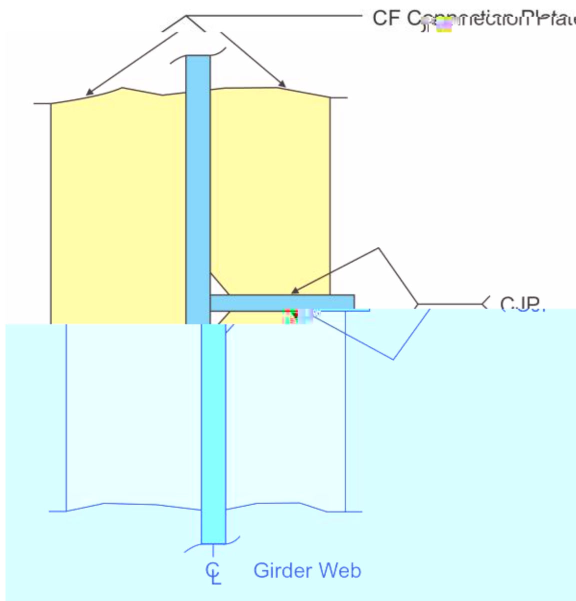

Figure 10 Sketch of a longitudinal and transverse stiffener intersection detail

6.6 Lateral Bracing

Lateral bracing can fulfill an important role in the design and erection of a plate girder bridge, but it also adds cost. The primary purpose of lateral bracing for plate girder bridges is to stiffen the bridge laterally in order to limit lateral deflections prior to the placement and hardening of the concrete deck. Lateral bracing should be avoided whenever possible, but there are certain situations where its use may be advantageous, such as providing stability for cantilever sections in erection of long spans. History has shown that a properly proportioned girder will rarely require lateral bracing in the final condition.

Lateral bracing may be consid00000912 W* n BT /F2 12 Tf 1 0 0 1 166.7 323.33 Tm 0 g 0 G [(. Th

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