Steel Bridge Design Handbook

APPENDIX

Design Example 4: Three-Span Continuous Straight Composite Steel Tub Girder Bridge

February 2022

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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.

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7. Supplementary Notes

The previous edition of this Handbook was published as FHWA-HIF-16-002 and was developed to be current with the 7th edition of the AASHTO LRFD Bridge Design Specifications. This edition of the Handbook was updated to be current with the 9th edition of the AASHTO LRFD Bridge Design Specifications, released in 2020.

8. Abstract

Tub girders are often selected over I-girders because of their pleasing appearance offering a smooth, uninterrupted, cross section. Bracing, web stiffeners, utilities, and other structural and nonstructural components are typically hidden from view within the steel tub girder resulting in the tub girder's clean appearance. Additionally, steel tub girder bridges offer advantages over other superstructure types in terms of span range, stiffness, durability, and future maintenance.

This design example demonstrates the design of a tangent three-span continuous composite tub girder bridge with a span arrangement of 187.5 ft - 275.0 ft - 187.5 ft. This example illustrates the flexural design of a section in positive flexure, the flexural design of a section in negative flexure, the shear design of the web, an evaluation of using a stiffened versus an unstiffened bottom flange in the negative flexure region, as well as discussions related to top flange lateral bracing design and bearing design.

9. Keywords Steel Tub Girder, Top-Flange Lateral Bracing, St. Venant Torsional Shear Flow, Diaphragms, Flange Longitudinal Stiffeners	**10. AISC Publication No.** B955-22

Steel Bridge Design Handbook
Design Example 4: Three-Span Continuous Straight Composite Steel Tub Girder Bridge

Table of Contents

List of Figures

List of Tables

1.0 INTRODUCTION

Steel boxes may either be tub sections or closed-box sections, with either inclined or vertical webs. Most composite box girders built in the U.S. are tub girders having a solid bottom flange, two solid webs, and an open top with two separate top flanges on each web connected with top lateral bracing to form a pseudo-box to resist the torsion prior to hardening of the concrete deck. Narrow noncomposite closed steel boxes are often employed as straddle beams to provide support and necessary underclearance.

Tub girders are sometimes selected over I-girders where aesthetic considerations are a significant factor because of their pleasing appearance offering a smooth, uninterrupted, cross-section. Bracing, web stiffeners, utilities, and other structural and nonstructural components are typically hidden from view within the steel tub girder, resulting in the tub girder’s clean appearance. Additionally, steel tub-girder bridges offer some distinct advantages over other superstructure types in terms of span range, stiffness, durability, and future maintenance.

Steel tub girders can potentially be more economical than steel plate I-girders in long-span applications due to the increased bending strength offered by their wide bottom flanges, and because they require less field work due to the handling of fewer pieces. Steel tub girders can also be suitable in short span ranges as well, especially when aesthetic preferences preclude the use of other structure types. However, tub girders should be no less than 5 feet deep to allow access for inspection, thus limiting the efficiency of conventional steel tub girders in short-span applications.

Tub girders provide a more efficient cross-section for resisting torsion than I-girders. The increased torsional resistance of a closed composite steel tub girder results in an improved lateral distribution of loads. Tub girders offer some distinct advantages over I-girders for horizontally curved bridges since the torsional stiffness of a tub girder is much larger than the torsional stiffness of an I-girder. The high torsional resistance of individual tub-girder sections permits the tub girder to carry more of the load applied to it rather than shifting the load to the adjacent tub girder with greater radius, as is the case for torsionally weaker I-girders. The tendency to share gravity loads more uniformly reduces the relatively large deflection of the girder on the outside of the curve. Also, less material needs to be added to tub girders to resist the torsional effects. Torsion in tub sections is resisted mainly by St. Venant torsional shear flow, rather than by warping torsion (which is the primary torsional response mechanism in I-shaped girders). Thus, warping shear and normal stresses due to warping torsion are typically quite small and *AASHTO LRFD BDS* Article C6.11.1.1 recommends that these stresses be neglected.. However, warping associated with distortion of the cross-section should be considered when evaluating the fatigue performance of tub girders in certain cases, as discussed further in Section 8.4.1 of this design example.

The exterior surfaces of tub girders are less susceptible to corrosion since there are fewer details for debris to accumulate, in comparison to an I-girder structure. For tub girders, stiffeners and most diaphragms are located within the tub girder and are protected from the environment. Additionally, the interior surface of the tub girder is protected from the environment, further reducing the likelihood of deterioration. Tub-girder bridges tend to be easier to inspect and maintain since much of the inspection can occur from inside the tub girder, with the tub serving as a protected walkway.

Erection costs for tub girders may be lower than that of I-girders because the erection of a single tub girder, in a single lift, is equivalent to the placement and connection of two I-girders. However, a single tub girder will typically require the use of a larger crane than an I-girder of the same length. Tub girders are also inherently more stable during erection, due to the presence of lateral bracing between the top flanges. Overall, the erection of a tub girder bridge may be completed in less time than that of an I-girder counterpart because there are fewer pieces to erect, a fewer number of external cross-frames or diaphragms to be placed in the field, and subsequently fewer field connections to be made. This is a significant factor to consider when available time for bridge erection is limited by schedule or site access.

In many instances, these advantages are not well reflected in engineering cost estimates based solely on quantity take-offs. Consequently, tub girder bridges have historically been considered more economical than I-girder bridges only if they have resulted in a reduction in the total number of webs in cross section, particularly for straight bridges. This is, in part, due to the cross-sectional restrictions placed on the use of approximate live load distribution factors for straight tub girders specified in Article 6.11.2.3 of the AASHTO *LRFD Bridge Design Specifications* [1], referred to hereafter as the *AASHTO LRFD BDS*, when the design is based on line girder analysis. For the AASHTO live load distribution factors to be applicable, the tub girder cross-sectional dimensions must satisfy limits that may make a tub girder cross-section less competitive than a comparable I-girder cross-section.

However, these cross-sectional restrictions do not apply when a refined analysis is employed, thus allowing the designer to explore additional, and perhaps, more economical design options. Also, if a particular fabricator has the experience and is equipped to produce tub girders efficiently, the competitiveness of tub girders in a particular application can be enhanced. Therefore, the comparative economies of I- and tub girder systems should be evaluated on a case-by-case basis, and the comparisons should reflect the appropriate costs of shipping, erection, and future inspection and maintenance, as well as fabrication. A more in-depth discussion on the relative advantages of steel tub girders and on steel tub girder design and construction may be found in the NSBA publication *Practical Steel Tub Girder Design* [2], which is available on the NSBA website (www.aisc.org/nsba).

This design example demonstrates the design of a tangent three-span continuous composite tub-girder bridge with a span arrangement of 187.5 feet — 275.0 feet — 187.5 feet. This example will illustrate the flexural design of a section in positive flexure, the flexural design of a section in negative flexure, the shear design of the web, the evaluation of using a stiffened versus an unstiffened bottom flange in the negative flexure region, as well as discussions related to top flange lateral bracing and bearing design. Since the cross-sectional restrictions on the use of approximate live-load distribution factors for straight tub girders are satisfied in this design example (Section 5.3), all flexural moments and shears in this example are determined using a line girder analysis. However, as mentioned above, more economical design options may be possible by going outside of these cross-sectional restrictions and utilizing a more refined analysis.

The bridge cross-section consists of two trapezoidal tub girders with top flanges spaced at 11.5 feet on centers, 12.0 feet between the centerline of adjacent top tub flanges, and 4.0-foot overhangs for a deck width of 43.0 feet out-to-out. For the sake of brevity, only the Strength I, Service II, and

Fatigue load combinations (Section 6.3) are examined for dead- and live-load force effects in this design example. The effects of wind loads, design permit loads, and other loads (braking forces, seismic forces, etc.) are not considered. It is recommended that the reader refer to NSBA's *Steel Bridge Design Handbook: Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge* [3] for information regarding additional load combination cases and design for wind-load force effects both during construction and in the final constructed condition.

2.0 OVERVIEW OF LRFD ARTICLE 6.11

The design of composite tub girder flexural members is contained within Article 6.11 of the 9th Edition of the *AASHTO LRFD BDS*. The provisions of Article 6.11 have been organized to correspond more closely to the general flow of the calculations necessary for the design of composite tub girder flexural members. Most of the provisions are written such that they are largely self-contained, however, to avoid repetition, some portions of Article 6.11 refer to provisions contained in Article 6.10 for the design of I-section flexural members when applicable. The provisions of Article 6.11 are organized as follows:

6.11.1 General
6.11.2 Cross-Section Proportion Limits
6.11.3 Constructability
6.11.4 Service Limit State
6.11.5 Fatigue and Fracture Limit State
6.11.6 Strength Limit State
6.11.7 Flexural Resistance - Sections in Positive Flexure
6.11.8 Flexural Resistance - Sections in Negative Flexure
6.11.9 Shear Resistance
6.11.10 Shear Connectors
6.11.11 Stiffeners

It should be noted that Article 6.11, and specifically Article 6.11.6.2, does not permit the use of Appendices A6 and B6 because the applicability of these provisions to tub girders has not been demonstrated; however, Appendices C6 and D6 are generally applicable. Flow charts for flexural design of steel I-girders, along with an outline giving the basic steps for steel-bridge superstructure design, are provided in Appendix C6. Appendix C6 may also prove to be a useful reference for tub girder design. Fundamental calculations for flexural members are contained in Appendix D6.

Example calculations demonstrating the provisions of Article 6.10, pertaining to straight and horizontally curved I-girder design and straight rolled-beam design, are provided in NSBA's *Steel Bridge Design Handbook* Design Examples 1, 2A, 2B, and 3 [3-6]. This design example will demonstrate the application of the provisions of Article 6.11 of the *AASHTO LRFD BDS* as they relate to straight tub girder design. NSBA's *Steel Bridge Design Handbook: Design Example 5: Three-Span Continuous Horizontally Curved Composite Steel Tub-Girder Bridge* [7] demonstrates the application of these provisions to a horizontally curved tub girder design.

The provisions of Articles 6.10 and 6.11 provide a unified approach for consideration of major-axis bending and flange lateral bending for both straight and horizontally curved bridges. Even for straight tub-girder bridges, the top flange can be subjected to significant lateral bending stresses during construction. Bottom flange lateral bending stresses tend to be quite small, since (as explained earlier) torsion in a tub girder is carried primarily by St. Venant torsional shear flow, rather than by warping torsion. Top flange lateral bending is caused by the outward thrust induced by the inclination of the webs, by wind loads, by eccentric loading of temporary support brackets for deck overhangs, and by forces introduced by the lateral bracing system.

In addition to checking that the design provides adequate strength, the constructability provisions of Article 6.11.3 verify that nominal yielding does not occur and that there is no reliance on post-buckling resistance for main load-carrying members during critical stages of construction. The *AASHTO LRFD BDS* specifies that for critical stages of construction, both compression and tension flanges must be investigated, and the effects of lateral bending in the top flanges should be considered. For noncomposite top flanges in compression, constructability design checks verify that the maximum combined stress in the flanges will not exceed the minimum specified yield strength, the flanges have sufficient strength to resist lateral torsional and flange local buckling, and that theoretical web-bend buckling and web shear buckling will not occur during construction. For noncomposite bottom flanges in compression during critical stages of construction, local buckling of the flange is checked in addition to the web-bend buckling and shear buckling resistance. For noncomposite top and bottom flanges in tension, constructability design checks verify that the maximum combined stress will not exceed the minimum specified yield strength of the flanges during construction. At the strength limit state, the top flanges are continuously braced by the hardened concrete deck and flange lateral bending stresses along with lateral torsional and flange local buckling of the flanges is not a concern. Also, due to the inherent torsional stiffness and strength of the closed section represented by the tub girder with the hardened composite concrete deck, global lateral torsional buckling of the composite tub girder is also not a concern.

3.0 DESIGN PARAMETERS

The following data apply to this example design:

Specifications:	2020 AASHTO *LRFD Bridge Design Specifications* [1], Customary U.S. Units, Ninth Edition
Structural Steel:	ASTM A709, Grade 50W uncoated weathering steel with F_y = 50 ksi, F_u = 70 ksi
Concrete:	f'_c = 4.0 ksi, γ = 150 pcf
Slab Reinforcing Steel:	ASTM A615, Grade 60 with F_y = 60 ksi

Permanent steel stay-in-place deck forms are used between the girders; the forms are assumed to weigh 15.0 psf, since it is assumed concrete will be in the flutes of the deck forms. In this example, the steel stay-in-place deck forms are used between the top flanges of individual tub girders, and between the top flanges of adjacent girders. The tub girders in this example are composite throughout the entire span, including regions of negative flexure.

An allowance for a future wearing surface of 25.0 psf is incorporated in the design. Also, an allowance for temporary construction loading of 10.0 psf is applied to the noncomposite structure during construction.

For the fatigue design, the Average Daily Truck Traffic (ADTT) in one direction, considering the expected growth in traffic volume over the 75-year fatigue design life, is assumed to be 2,000 trucks/day.

Composite tub girder bridges fabricated using uncoated weathering steel have performed successfully without any interior corrosion protection. However, the interior of tub girders should always be coated in a light color to aid visibility during girder inspection. Without Owner-agency direction towards a specific coating and preparation, the girder interior should receive a light brush blast and be painted with a white or light-colored coating capable of telegraphing cracks in the steel section. Specified interior coatings should be tolerant of minimal surface preparation. At the Engineer's discretion, for painted tub girders, an allowance may be made for the weight of the paint as discussed in Article C6.11.3.1.

Provisions for adequate draining and ventilation of the interior of the tub are essential. As suggested in the NSBA Publication *Practical Steel Tub Girder Design* [2], bottom flange drain holes should be 1 ½ inches in diameter and spaced along the low side of the bottom flange every 50 feet and be placed 4 inches away from the web plate. Access holes must be provided to allow for periodic structural inspection of the interior of the tub. The access holes should provide easy access for authorized inspectors. Solid doors can be used to close the access holes, however, they should be light in weight, and they should be hinged and locked, but not bolted. Alternatively, wire-mesh screens can be placed over access holes. Wire mesh should be 10-gage to withstand welding and blasting and have a weave of approximately ½ inch by ½ inch. Wire-mesh screens should always be used over the bottom flange drain holes to prevent entry of wildlife and insects.

Additional detailing guidelines can be found at www.aisc.org/nsba, which is the NSBA website, with particular attention given to the AASHTO/NSBA Steel Bridge Collaboration document G1.4, *Guidelines for Design Details* [8]. Three other detailing references offering guidance are the Texas Steel Quality Council's *Preferred Practices for Steel Bridge Design, Fabrication, and Erection* [9], the Mid-Atlantic States Structural Committee for Economic Fabrication (SCEF) Standards, and the AASHTO/NSBA Steel Bridge Collaboration document G12.1, *Guidelines to Design for Constructability and Fabrication* [10].

4.0 STEEL FRAMING

4.1 Span Arrangement

Careful consideration to the layout of the steel framing is an important part of the design process and involves evaluating alternative span arrangements based on the superstructure and substructure cost to arrive at the most economical solution. Often, site-specific features will influence the span arrangement required. However, in the absence of these issues, choosing a balanced span arrangement for continuous steel bridges (end spans approximately 80% of the length of the center spans) will provide an efficient design. The span arrangement for the example bridge has spans of 187.5 feet — 275.0 feet — 187.5 feet. It is evident that this is not an ideal balanced span arrangement; however, the span arrangement is chosen to illustrate some concepts generally not found in an ideal span arrangement. Refer to NSBA's *Steel Bridge Handbook Design: Example 1*: *Three-Span Continuous Straight Composite Steel I-Girder Bridge* [3] for further discussion on span arrangement considerations.

4.2 Bridge Cross-section

When developing the bridge cross-section, the designer will evaluate the number of girder lines required, relative to the overall cost. Specifically, the total cost of the superstructure is a function of steel quantity, details, and erection costs. Developing an efficient bridge cross-section should also consider the provision of an efficient deck design, which is generally influenced by girder spacing and overhang dimensions. Specifically, with the exception of an empirical deck design, girder spacing significantly affects the design moments in the deck slab. Larger deck overhangs result in a greater load on the exterior web of the tub girder. Larger overhangs will increase the bending moment in the deck, caused by the cantilever action of the overhang, resulting in additional deck slab reinforcing for the overhang region of the deck.

In addition, wider deck spans between top flanges can become problematic for several reasons. Some owners have economical deck detail standards that may not be suited, or even permitted, for wider deck spans. At the same time, wider deck spans are progressively more difficult to form and construct.

Special attention should be paid to the design of decks for steel tub girder bridges in the area near the girder top flanges between adjacent girders. The inherent torsional stiffness of tub girders can produce a situation where the deck is subjected to a racking effect when there is differential vertical displacement between adjacent girders. This phenomenon is illustrated in Figure C9.7.2.4-1 of the *AASHTO LRFD BDS* [1]. This effect is not directly addressed in the empirical deck design method (as noted in the *AASHTO LRFD BDS* Commentary C9.7.4.2). When the traditional deck design method is used, the effects of this phenomenon should be addressed either by approximate calculation methods (when a line girder analysis method is being used) or by evaluating deck stresses (when a refined analysis model is being used).

If empirical live load distribution factors are to be employed, the final cross-section must meet the requirements of Article 6.11.2.3, which states that the deck overhang should not exceed 60 percent of the distance between centers of the top flanges of adjacent tub girders, or 6.0 feet. Also, the distance center-to-center of adjacent tub girders is not to be greater than 120 percent nor less than 80 percent of the top flange center-to-center distance of a single tub girder.

The example bridge cross-section consists of two trapezoidal tub girders with top flanges spaced at 11.5 feet on centers, 12.0 feet between the centerline of adjacent top flanges with 4.0-foot deck overhangs and an out-to-out deck width of 43.0 feet. The deck overhangs are 33 percent of the adjacent tub girder spacing. The 40.0-foot roadway width can accommodate up to three 12-foot-wide design traffic lanes. The total thickness of the cast-in-place concrete deck is 9.5 inches, including a 0.5-inch thick integral wearing surface. The concrete deck haunch is 3.5 inches deep measured from the top of the web to the bottom of the deck. The width of the deck haunch is assumed to be 18.0 inches. Deck parapets are each assumed to weigh 520 pounds per linear foot. The typical cross-section is shown in Figure 1.

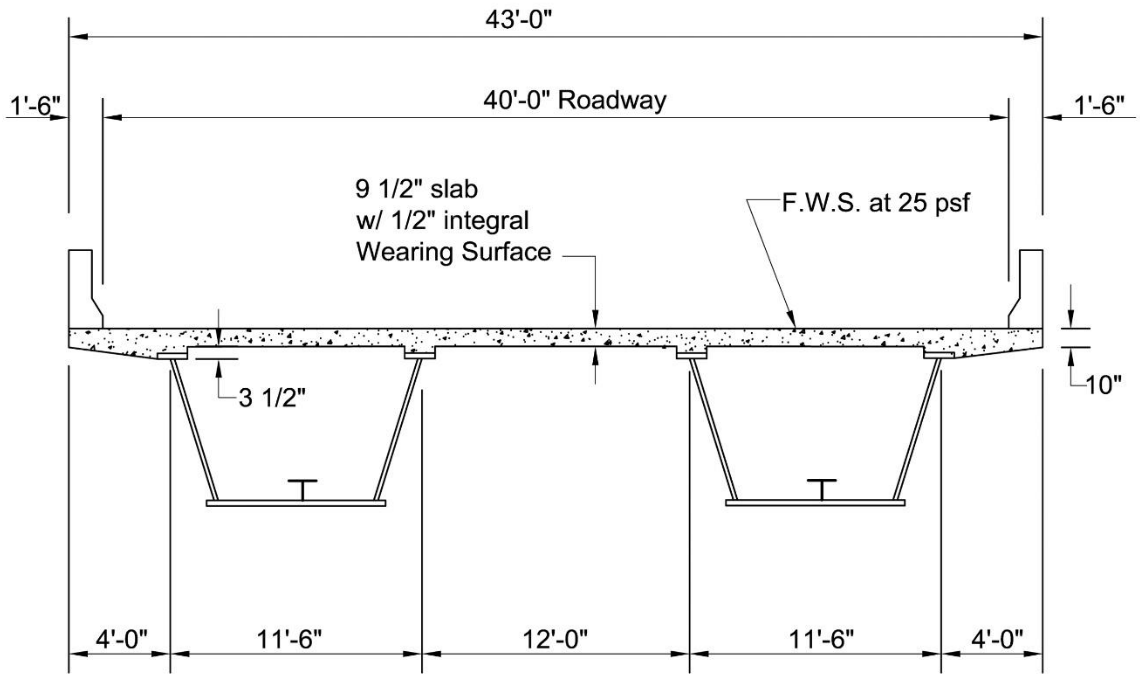

Figure 1 Typical Bridge Cross-Section

4.3 Intermediate Cross-frames

Internal intermediate cross-frames are provided in tub girders to control cross-sectional distortion. Cross-sectional distortion results due to the St. Venant torsion shear flow changing direction at the corners of the tub. Cross-sectional distortion introduces additional stresses in the tub girder and, therefore, should be minimized. The distortion stresses basically occur because the section is not perfectly round. The shear flow must change direction at the corners, which tends to distort the cross-section. Adequate internal cross-bracing usually controls the magnitude of these stresses in tub girders of typical proportion such that they are not critical to the ultimate resistance of the tub section at the strength limit state. As a minimum, internal cross-frames should be placed at points of maximum moment within a span and at points adjacent to field splices in straight bridges.

Spacing of internal cross-frames, considered during development of the framing plan, should be influenced by factors such as the angle and length of the lateral bracing members.

Most internal cross-frames in modern tub girder bridges are K-frames, often without a bottom strut, which allow for better access during construction and inspection. Slenderness requirements (KL/r) generally govern the design of cross-frame members, however handling and strength requirements should always considered. When refined analysis methods are used and the cross-frame members are included in the structural model to determine force effects, the cross-frame members are to be designed for the calculated force effects. Consideration should be given to the cross-frame member forces during construction. When simplified analysis methods are used, such cross-frame forces due to dead and live loads are typically difficult to calculate. Therefore, the cross-frame members should at least be designed to transfer wind loads and carry any construction loads due to deck overhang brackets, in addition to satisfying slenderness requirements.

External intermediate cross-frames may be incorporated to control the differential displacements and rotations between individual tub girders during deck placement. In a finished bridge, when the tub girders are fully closed and the concrete deck effectively attaches the girders together, transverse rotation is expected to be small and external cross-frames are not necessarily required. However, during construction the rotational rigidity of the tub girder is not nearly as large and, since the two top flanges of a single tub girder are spaced apart but rotate together, the resulting differential deflections may be large even with a small girder rotation. Helwig et al. [11] present an approximate method for estimating these differential deflections that can be very helpful in evaluating the possible need for external intermediate cross-frames early in the design process.

External intermediate cross-frames typically utilize a K-frame configuration, with the depth closely matching the girder depth for efficiency and simplification of supporting details. Solid web (plate girder) diaphragms have been successfully used as well. At locations of external intermediate cross-frames, there should be bracing inside the tub girder to receive the forces of the external bracing. In some cases, for aesthetic reasons, it may be desirable to remove the external intermediate cross-frames after the deck has hardened. However, extreme care should be taken in evaluating the effects that the removal of external intermediate cross-frames has on the structure. The NSBA Publication *Practical Steel Tub Girder Design* [2] offers further discussion on this topic.

Based on the preceding considerations, the cross-frame spacings shown on the framing plan in Figure 2 were chosen for this example. The internal cross-frames are uniformly spaced in the end-span and center-span field sections. However, this is not the case for the two field sections at the interior supports. Due to the lack of symmetry in the interior-support field sections, the internal cross-frame spacing in the end-span region differs from the internal cross-frame spacing in the center-span region. Internal cross-frame spacing in the center-span positive flexure region is 31′-9″; however, to reduce the unbraced length of the top flange so as to increase the lateral torsional buckling resistance for noncomposite loading, a top strut is located in the center of each internal cross-frame bay.

4.4 Top Lateral Bracing

Lateral bracing between common top flanges of a tub girder is required to provide proper shear flow in the individual tub girders. Without lateral bracing, the section acts as an open section and is much less stable under torsional loading. The bracing acts to enhance the global lateral torsional buckling stability of the section. Top lateral bracing raises the shear center to the inside of the tub section resulting in a pseudo-box section and significantly increasing the torsional stiffness. A single tub girder with a properly designed top flange lateral bracing system has substantially greater stability than an equivalent pair of I-shaped girders without lateral bracing, even if the pair of I-shaped girders has the same net major-axis bending section modulus as the single tub girder.

AASHTO LRFD BDS Article C6.7.5.3 recommends that a full-length lateral bracing system be provided within straight tub sections utilized on spans greater than about 150 feet; a full-length system is provided in this design example. For horizontally curved tub girders, a full-length lateral bracing system must always be provided according to Article 6.7.5.3. A full-length lateral bracing system is particularly important when the torques on the noncomposite section are large; e.g., in tub-section members on which the deck weight is applied unsymmetrically, or in members resting on skewed supports. A full-length lateral bracing system can also help limit distortions that may result from temperature changes occurring prior to deck placement, and resist the torsion and twist resulting from any eccentric loads that may act on the steel section during construction, including the effects of deck overhang brackets. If a full-length lateral bracing system is not provided for straight tub girders with spans less than about 150 feet, the designer is advised to perform a full investigation of girder stability during all stages of erection and deck placement, considering various possible lift points and lifting scenarios, various interim erection conditions, and various loading effects (including self-weight, wet concrete deck weight, construction dead and live loads, wind loads, etc.). Top lateral bracing should always be continuous across field-splice locations. Otherwise, large flange lateral bending stresses might occur in the top flanges of the tub where the bracing is discontinued.

Top lateral bracing is designed to resist the shear flow in the pseudo-box section resulting from any factored torsion acting on the steel section before the deck has hardened. Forces in the bracing due to flexure of the tub during construction should also be considered since these members act with the tub in flexure. In the absence of a refined analysis, design equations have been developed by Fan and Helwig [12] to evaluate the bracing member forces due to tub girder bending. Top lateral bracing members are also subject to wind-load forces acting on the noncomposite tub section during construction.

Top flange lateral bracing systems for steel tub girders typically take the form of a truss system in the horizontal plane between the tub girder top flanges, with transverse struts and diagonals. Various studies and guides have examined different options for the truss system. *AASHTO LRFD BDS* Article C6.7.5.3 notes that single-diagonal systems are preferred over X-type systems because they involve fewer members and fewer connections, facilitating fabrication and assembly. The Commentary also discusses the various merits of Warren-type vs. Pratt-type truss systems. This design example utilizes a Warren-type truss system. NSBA's *Steel Bridge Design Handbook: Bracing System Design* [13] discusses issues related to the selection of the lateral bracing configuration in greater detail.

Single angle or structural tee (WT) sections are most commonly used as top flange lateral bracing members; full rolled beam (W) sections have occasionally been used when forces are large. Whenever possible, direct attachment of the top flange lateral bracing members to the tub girder top flanges via bolted connections is preferred to avoid the extra fabrication and assembly costs associated with using gusset plates; in some cases, a slight increase in tub girder top-flange width to accommodate the bolted connections can be more economical than a "least weight" design that would require gusset plates to accommodate the connections to the flanges. Although not checked in this example, wherever the bracing members are bolted to a top flange subject to tension, *AASHTO LRFD BDS* Equation 6.10.1.8-1 must be satisfied at cross-sections of flexural members containing holes in the tension flange at the strength limit state and when checking constructability.

As noted previously, the internal intermediate cross-frames in tub girders are typically integrated with the top flange lateral bracing system by using some or all of the top flange lateral bracing transverse struts as the top chord members of the internal cross-frames. Section 8.11 illustrates the design of a diagonal and a top-strut lateral member in the top lateral bracing system from this example.

4.5 Support Diaphragms

Internal diaphragms at points of support are typically full-depth, solid web sections with a top flange. These diaphragms are subjected to bending moments which result from the shear forces in the inclined girder webs. If a single bearing is used at the support that does not approach the full width of the tub girder bottom flange, bending of the internal diaphragm over the bearing will result, causing tensile stresses in the top flange of the diaphragm and compressive stresses in the bottom flange of the tub girder. Additionally, a torsional moment reaction in the tub girder at the support will induce a shear flow along the circumference of the internal diaphragm. To provide the necessary force transfer between the tub girder and the internal diaphragms, the internal diaphragms should be connected to the web and top flanges of the tub girder.

Inspection access through the internal diaphragms at interior supports must be provided with access holes at least 18 inches wide and 24 inches high; however, if feasible, a larger hole at least 36.0 in. deep is preferable. In addition to restraining distortion of the box section, the internal diaphragms at supports also transfer load from the girder webs to the bearing(s). If a single centered bearing is used, the diaphragm must be stout enough to resist the reaction and transfer the load around any access hole. Bearing stiffeners are usually attached to the diaphragms. If a single centered bearing is employed, two stiffeners are generally used. A bearing stiffener on each side of the access hole generally removes the shear from the diaphragm before it is engaged by the hole. Torsion generally causes a different magnitude of shear in the webs of the box on the two sides of the diaphragm. Reinforcement around the hole may be required, particularly if the access hole requires a large portion of the diaphragm or if a single bearing is located under the diaphragm. Auxiliary stiffeners on the diaphragm or webs may be employed to spread out the reaction.

As discussed in *AASHTO LRFD BDS* Article C6.7.4.3, external plate diaphragms with aspect ratios, or ratios of length to depth, less than 4.0 and internal plate diaphragms act as deep beams and should be evaluated by considering principal stresses rather than by simple beam theory. Fatigue-sensitive details on these diaphragms and at the connection of the diaphragms to the flanges should be investigated by considering the principal tensile stresses.

Similar to internal diaphragms, external diaphragms are typically full-depth plate girder sections, with top and bottom flanges. As acknowledged in the NSBA publication *Practical Steel Tub Girder Design* [2], the behavior of an external diaphragm at a point of support is highly dependent on the bearing arrangement at that location. If dual bearings used at each girder sufficiently prevent transverse rotation, external diaphragms at the point of support should theoretically be stress free. The force couple behavior of a dual bearing system resists the torsion that would otherwise be resisted by the external diaphragm and, in turn, minimizes the bending moments applied to the external diaphragm.

If a single bearing under each tub girder is employed, torsional moments must be resisted by the external diaphragm through vertical bending. In a single bearing arrangement, the internal diaphragms of adjacent girders function with the external diaphragms to form a system (or beam) which resists the girder torsional moments. The total torque is resisted by differential reactions at the bearings of adjacent girders. The diaphragms then are subjected to bending and shear forces. Torsional moments resisted by the external diaphragm often require the use of a moment connection to the tub girder in which the flanges and webs of the external diaphragm are connected. The largest torsional moment will typically occur during the construction stage and can be quite large, particularly in horizontally curved structures. Torsional moments in straight bridges are typically smaller but should still be considered in the design.

4.6 Length of Field Sections

The lengths of field sections are generally dictated by shipping (weight and length) restrictions. Generally, the weight of a single shipping piece is restricted to 200,000 lbs, while the piece length is limited to a maximum of 140 feet, with an ideal piece length of 120 feet. However, shipping requirements are typically dictated by state or local authorities, in which additional restrictions may be placed on piece weight and length. Handling issues during erection and in the fabrication shop also need to be considered in the determination of field section lengths, as they may govern the length of field sections. Therefore, the Engineer should consult with contractors and fabricators regarding any specific restrictions that might influence the field-section lengths.

Field-section lengths should also be determined with consideration given to the number of field splices required, as well as the locations of the field splices. It is desirable to locate field splices as close as possible to dead-load inflection points so as to reduce the forces that must be carried by the field splice. Field splices located in higher moment regions can become quite large, with cost increasing proportionally to their size. The Engineer must determine what the most cost-competitive solution is for the given span arrangement. For complex and longer span bridges, the fabricator’s input can be helpful in reaching an economical solution.

Due to the span arrangement for this example, and the desire to limit field section lengths to 130.0 feet, field splices are not located ideally at dead-load inflection points. Five (5) field sections are used in each line of girders (Figure 2). For this layout, an end span field section weighs approximately 107,000 lbs, an interior support field section weights approximately 165,000 lbs, and the center span field section has a weight of approximately 101,000 lbs. Field sections in this length and weight range can generally be fabricated, shipped, and erected without significant issues.

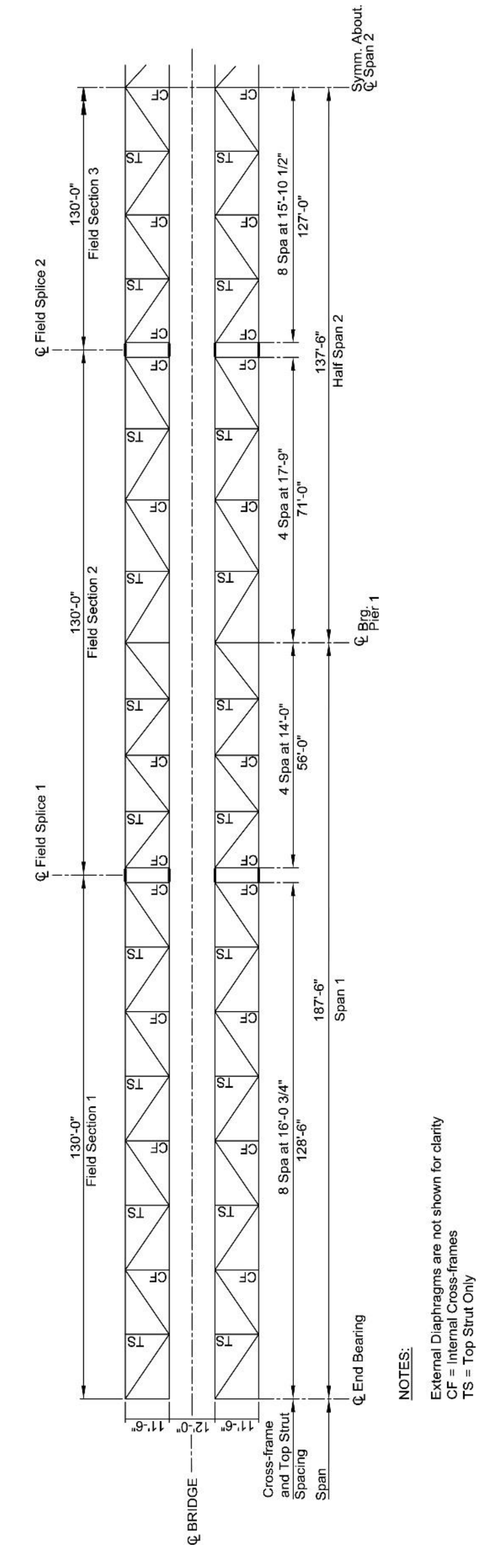

Figure 2 Sketch of the Framing Plan

5.0 PRELIMINARY GIRDER PROPORTIONS

5.1 Girder Depth

Proper proportioning of tub girders involves a study of various girder depths versus girder weight to arrive at the least weight solution that meets all performance and handling requirements. The overall weight of the tub girder can vary dramatically based on web depth. Therefore, selection of the proper girder depth is an extremely important consideration affecting the economy of steel-girder design. The NSBA Publication, *Practical Steel Tub Girder Design* [2] points out that a traditional rule of thumb for steel tub girder bridge depths is L/25, however designers should not be reluctant to exceed this ratio. Tangent steel tub girders have approached L/35 while meeting all code requirements for strength and deflection. Article 2.5.2.6.3 provides suggested minimum span-to-depth ratios for I-girders but does not specifically address tub girder sections. The suggested minimum depth of the steel section in a composite I-girder, in a continuous span, is given as 0.027L, where L is the span length in feet. This criterion may be applied to determine a starting depth of the tub girder for the depth studies. Using the longest span of 275.0 ft, the suggested minimum depth of the steel section is:

$$0.027(275.0) = 7.425 \text{ ft} = 89.1 \text{ in.}$$

Considering an approximate thickness for the top and bottom flange will lead to a vertical web depth of approximately 86.5 inches. A preliminary web depth study was performed to determine an appropriate optimal web depth based on minimum steel weight. This study considered various web depths and associated flange sizes that satisfied the design requirements in a preliminary sense. The optimal web depth was chosen from the preliminary design that resulted in the least amount of steel girder weight. The optimal vertical depth for this study was found to be 84.5 inches. Therefore, a vertical web depth of 84.4 inches was used which resulted in a web plate size of 87.0 inches, assuming a maximum web inclination of 1:4. This, in turn, resulted in a bottom flange width of 98.5 inches.

Tub girders typically employ inclined webs, as they are advantageous in reducing the width of the bottom flange. Article 6.11.2.1 specifies that the web inclination should not exceed 1:4 (horizontal:vertical). Because progressively deeper webs may result in a narrower and potentially thicker bottom flange plate (at location of maximum flexure), it is necessary for the Engineer to explore a wide range of web depths and web spacing options in conjunction with the bottom flange design requirements to determine the optimal solution.

5.2 Cross-section Proportions

Proportion limits for webs of tub girders are specified in Article 6.11.2.1. Provisions for webs with and without longitudinal stiffeners are presented. For this example, a longitudinally stiffened web is not anticipated. Therefore, the web plate must be proportioned such that the web plate thickness (t_w) meets the following requirement:

$$\frac{D}{t_w} \leq 150 \qquad \text{Eq. (6.11.2.1.2-1)}$$

For inclined webs, Article 6.11.2.1.1 states that the distance along the web is to be used for D in all design checks.

Rearranging:

$$\left(t_w\right)_{min.} = \frac{D}{150} = \frac{87}{150} = 0.58 \text{ in.}$$

Therefore, utilizing 1/16-inch increments for web plate thickness, select an initial web thickness of 0.625 inches.

Cross-section proportion limits for top flanges of tub girders are specified in Article 6.11.2.2. The minimum width of the top flanges is specified as:

$$b_f \geq \frac{D}{6} \qquad \text{Eq. (6.11.2.2-2)}$$

$$\left(b_f\right)_{min.} = D/6 = 87/6 = 14.5 \text{ in.}$$

Article C6.10.2.2 suggests the following additional guideline for the minimum top-flange width for each individual unspliced girder field section, b_{tfs}, to be used in conjunction with flange proportion limit specified above. This guideline is intended to provide more stable field pieces for lifting, erection, and shipping without the need for special stiffening trusses or falsework.

$$b_{tfs} \geq \frac{L_{fs}}{85} \qquad \text{Eq. (C6.10.2.2-1)}$$

L_{fs} is the length of the unspliced individual girder field section under consideration in feet.

Eq. C6.10.2.2-1 is intending primarily for application to unspliced I-girder field sections. However, as discussed in Article C6.11.3.2, in cases where a full-length lateral bracing system is not employed within a straight tub girder, which is not the standard practice and should only be considered if the spans are less than about 150 feet (Section 4.4), the minimum width of the top flanges within each individual unspliced field section should satisfy the preceding guideline. In this case, L_{fs} is to be taken as the larger of the distances along the field section between panels of lateral bracing or between a panel of lateral bracing and the end of the piece. For cases where a full-length lateral bracing system is employed, which is the typical case (and the case in this example), Eq. C6.10.2.2-1 need not be considered for top flanges of tub sections.

In this case, a minimum top flange width of 18 inches is advantageous to connect the top flange lateral bracing directly to the top flange. Therefore, a minimum top flange width of 18 inches will be provided so that the flange will be wide enough to accommodate the bolted lateral bracing connections.

The minimum thickness of the top flanges is specified as:

$$t_f \geq 1.1t_w \qquad \text{Eq. (6.11.2.2-3)}$$

or:

$$(t_f)_{min.} = 1.1t_w = 1.1(0.625) = 0.6875 \text{ in.}$$

However, the AASHTO/NSBA Steel Bridge Collaboration document G12.1 *Guidelines to Design for Constructability and Fabrication* [8] recommends a minimum flange thickness of 0.75 inches to enhance girder stability during handling and erection. Therefore, use $(t_f)_{min}$ = 0.75 inches.

Additionally, the top flanges must satisfy the following ratio:

$$\frac{b_f}{2t_f} \leq 12.0 \qquad \text{Eq. (6.11.2.2-1)}$$

Therefore, checking the minimum size top flanges:

$$\frac{18}{2(0.75)} = 12.0 \quad \text{ok}$$

Top-flange thicknesses of 1.0 inch will be used in regions of positive flexure so that the noncomposite section will have a greater likelihood of satisfying the constructability checks.

The *AASHTO LRFD BDS* currently imposes no limitation on the b/t ratio of bottom flanges of composite tub girders in tension. Past and current industry guidance has suggested “rules of thumb” for the maximum b/t ratio ranging from as slender as 120 to as stocky as 80. White et al. (2019) [14] developed guidance (described below) which has been adopted in the *AASHTO LRFD BDS* for noncomposite steel box girder members, and which White et al. suggested should also be considered for composite steel tub girder bottom flanges. These limits are intended to address several fabrication concerns, including waviness and warping effects during welding of the bottom flange to the webs. Furthermore, the Engineer should be aware that it is possible that the bottom flange in tension in the final condition may be in compression during lifting of the tub girder during erection, possibly causing buckling of the slender bottom flange.

Article 6.12.2.2.2b suggests a limit on the b/t ratio, based on the inside width of the flanges, of 90 for longitudinally unstiffened compression and tension flanges in noncomposite box-section members to address similar concerns. Compression flanges exceeding this value must include longitudinal stiffeners. Tension flanges in these members with a b/t ratio exceeding 130 must include longitudinal stiffeners to prevent noticeable out-of-plane deflections of the flange under self-weight or under self-weight with a small concentrated transverse load. Unless otherwise specified by the Owner, a minimum thickness of 0.5 inches is also specified for compression and tension flanges in these members to limit potential local deformation or distortion of box section flanges during fabrication, transportation, erection, and service conditions. Additional information on these limits may be found in White et al. (2019) [14]. Additional discussion concerning this issue can also be found in the NSBA publication *Practical Steel Tub Girder Design* [2].

If it is desired to exceed the suggested b/t limit of 90 for tension flanges, the Engineer should consult with fabricators to verify that a tub girder with the selected bottom flange thickness in regions of positive flexure can be fabricated without causing any significant handling and/or distortion concerns without providing any flange longitudinal stiffeners. For this example, tension flanges in regions of positive flexure with a thickness of 13/16 inches and a maximum b/t ratio (based on the inside width of the flanges) of approximately 117 are utilized. This represents a significant reduction in the b/t ratios from the original design for this example, which was completed before the preceding guidance was available. In an actual design, consideration should probably be given to using a somewhat lower b/t for these flanges.

Bottom flange extensions of 1-3/8 inches (measured from the centerline of the webs) were assumed in this design example for welding access. It should be noted that the AASHTO/NSBA Steel Bridge Collaboration document G1.4, *Guidelines for Design Details* [8] (Page 116) suggests preferred bottom flange extensions of 1-1/2 inches.

Based on the above minimum proportions, the trial girder shown in Figure 3 is suggested.

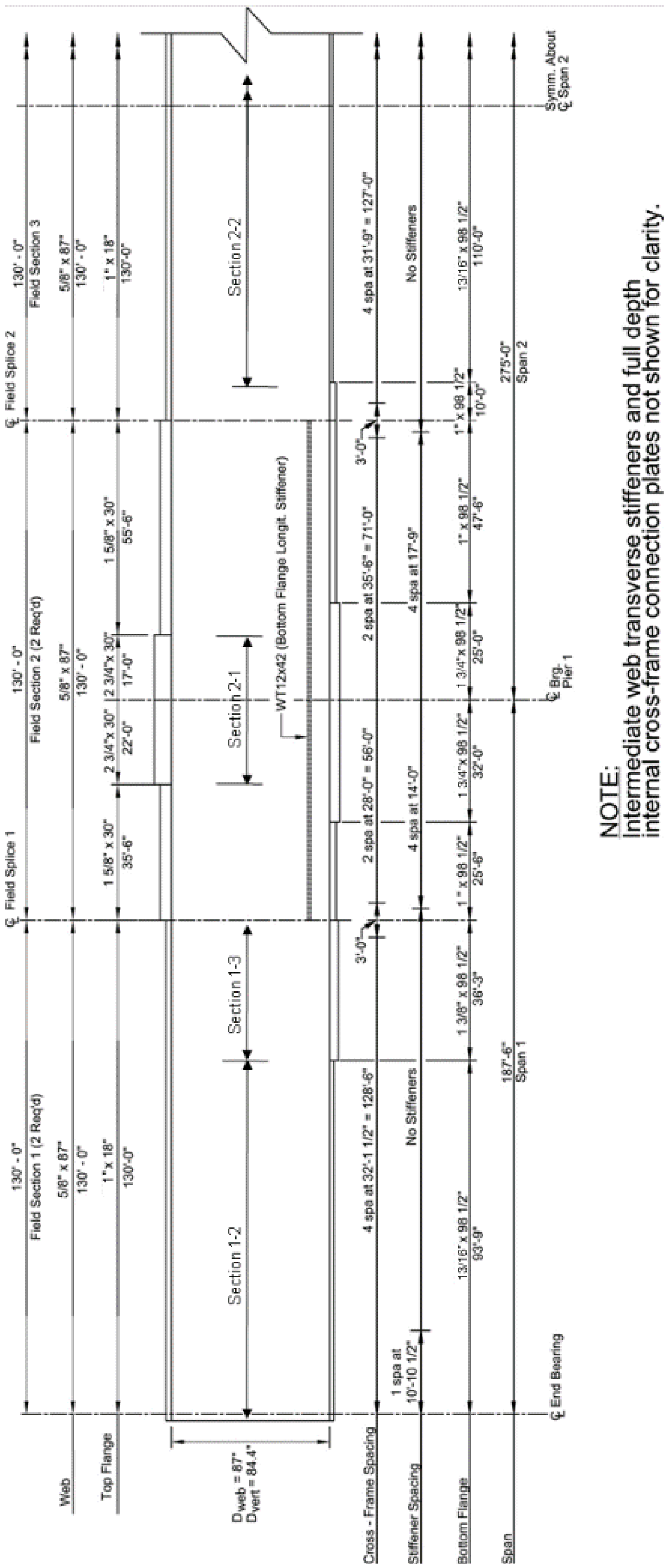

Figure 3 Sketch of the Girder Elevation

5.3 Special Restrictions for use of Live Load Distribution Factors

Special consideration must be given to preliminary proportions for straight tub girder bridges that will employ use of the live load distribution factors presented in Article 4.6.2.2.2b. Specifically, cross-sections of straight bridges consisting of two or more single-cell tub girders must satisfy the geometric restrictions specified in Article 6.11.2.3.

In particular:

- Bearing lines are not to be skewed.
- The distance center-to-center (a) of the top flanges of adjacent tubes, taken at mid-span, must satisfy:

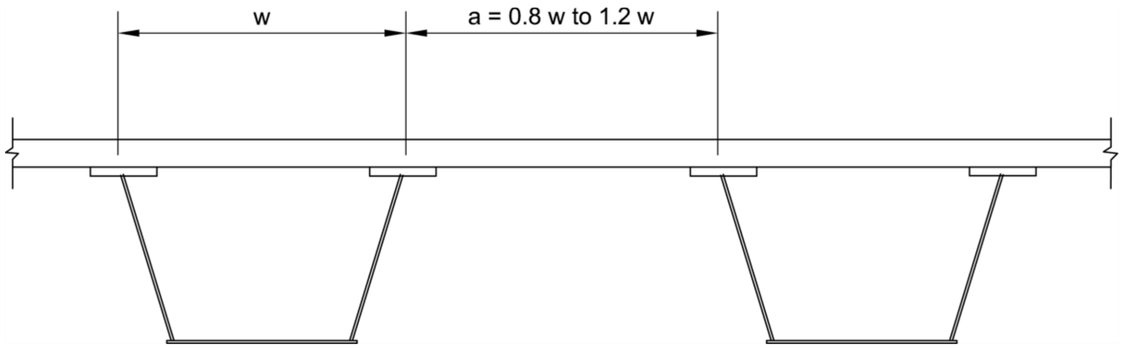

Figure 4 Center-to-Center Flange Distance

Note: For nonparallel tub girders, in addition to mid-span requirements, Article 6.11.2.3 imposes additional geometric restrictions at the supports.

- The distance center-to-center (w) of the top flanges of individual tub girders must be the same.
- The inclination of the web must not exceed 1 (horizontal) to 4 (vertical) to a plane normal to the bottom flange, as shown in Figure 5.
- The overhang of the concrete deck, including the curb and parapet cannot exceed 60 percent of the average distance between the centers of the top flanges of adjacent tub girders, a, or 6.0 feet.

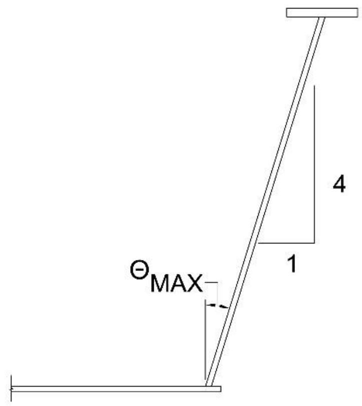

Figure 5 Maximum Web Inclination

For this example, there are no skewed supports and the distance center-to-center (w) of the top flanges of the individual tub girders is a constant 11.5 feet:

$0.8\ (11.5) = 9.2 \text{ ft} \leq a = 12 \text{ ft} \leq\ 1.2\ (11.5) = 13.8 \text{ ft}$

The inclination of the web is 1 (horizontal) to 4 (vertical) in this example, therefore satisfying the previously mentioned requirement.

The cantilever deck overhang used in this example is 4.0 feet, therefore less than 0.60(12.0) = 7.2 feet and 6.0 feet.

The requirements of Article 6.11.2.3 are satisfied for this example; therefore, live load flexural moments and shears for this example may be computed in accordance with Article 4.6.2.2.2b.

6.0 LOADS

6.1 Dead Loads

As defined in Article 3.5.1, dead loads are permanent loads that include the weight of all components of the structure, appurtenances and utilities attached to the structure, earth cover, wearing surfaces, future overlays and planned widenings.

The component dead load (DC) consists of all the structure dead load except for non-integral wearing surfaces, if anticipated, and any specified utility loads. For composite steel-girder design, DC is further divided into:

- Noncomposite dead load (DC_1) is the portion of loading resisted by the noncomposite section. DC_1 represents the permanent component load that is applied before the concrete deck has hardened or is made composite.
- Composite dead load (DC_2) is the portion of loading resisted by the long-term composite section. DC_2 represents the permanent component load that is applied after the concrete deck has hardened or is made composite.

For this example, the dead load component (DC_1) is calculated as follows:

Concrete deck $= \frac{9.5}{12}(43.0)(0.150)$ = 5.106 kips/ft

Concrete deck overhang tapers = $2\left[\frac{1}{12}\left(\frac{13.0+10}{2}-9.5\right)\left(4.0-\frac{18/2}{12}\right)\right](0.150)$ = 0.162 kips/ft

Concrete deck haunches $= 4\left[\frac{18(3.50\text{-}0.875)}{144}\right](0.150)$ = 0.197 kips/ft

(The minimum top flange thickness and associated width are used in the above computation.)

Stay-in-place forms $= \left[2(11.5)+12-3\left(\frac{18}{12}\right)\right](0.015) = 0.457$ kips/ft = 0.457 kips/ft

Steel girder self-weight
(based on preliminary sizing and confirmed during subsequent analysis)= 1.635 kips/ft

Cross-frames and details = 0.110 kips/ft

DC_1 load total (per 2 girders) = 7.908 kips/ft

Therefore, the distributed DC_1 load per a girder is:

DC_1 load per girder = 7.908 kips/ft ÷ 2 girders = 3.954 kips/ft per girder

Unless otherwise stipulated by the Owner, it is sometimes assumed, in accordance with Article 4.6.2.2.1, that composite dead loads are supported equally by all girders of straight, non-skewed bridges with typical deck overhangs and girders of similar stiffness. In most modern designs, large and heavy concrete barriers are often placed at the outer edges of the concrete deck. When refined methods of analysis are employed, the self-weight of the concrete barriers (the DC_2 loads in this case) should be applied at their actual locations at the outer edges of the deck. In I-girder bridges, this results in the exterior girders carrying a larger percentage of these loads. Some Owner-agencies prescribe the use of different, semi-arbitrary percentages for distribution of the barrier weight to the exterior girder and to the adjacent interior girders, while others continue to distribute the barrier weight equally among all girders. In this example, with a pair of torsionally stiff tub girders and a relatively narrow total cross-section width, the weight of each concrete barrier is assumed to be distributed equally to each girder in the cross-section; a uniform distribution assumption would be likely less valid for the design of a much wider bridge with more girders in the cross-section.

For this example, the composite section dead load (DC_2) will consist of the self-weight of the concrete barrier only. Therefore:

DC_2 load per girder = 0.520 kips/ft per girder

The component dead load (DW) consists of the dead load of any non-integral wearing surfaces and any utilities. DW is also assumed to be equally distributed to all girders. For this example, a future wearing surface is anticipated but no utilities are included. Therefore:

DW load per girder = [(0.025) x 40] ÷ 2 girders = 0.500 kips/ft per girder

For computing flexural stresses from the composite dead loads, DC_2 and DW, the stiffness of the long-term composite section in regions of positive flexure is calculated by transforming the concrete deck using a modular ratio of $3n$ (Article 6.10.1.1.1b). In regions of negative flexure, the long-term composite section is assumed to consist of the steel section plus the longitudinal reinforcement within the effective width of the concrete deck (Article 6.10.1.1.1c).

6.2 Live Loads

Live loads are assumed to consist of gravity loads (vehicular live loads, rail transit loads and pedestrian loads), the dynamic load allowance, centrifugal forces, braking forces and vehicular collision forces. Live loads illustrated in this example include the HL-93 vehicular live load and a fatigue load, with the appropriate dynamic load allowance included.

Live loads are treated as transient loads applied to the short-term composite section. For computing flexural stresses from transient loading, the short-term composite section in regions of positive flexure is calculated by transforming the concrete deck using a modular ratio of n (Article 6.10.1.1.1b). In regions of negative flexure, the short-term composite section is assumed to consist of the steel section plus the longitudinal reinforcement within the effective width of the concrete deck (Article 6.10.1.1.1c), except as permitted otherwise for the fatigue and service limit states (see Articles 6.6.1.2.1 and 6.10.4.2.1).

When computing longitudinal flexural stresses in the concrete deck (see Article 6.10.1.1.1d), due to permanent and transient loads, the short-term composite section should be used.

6.2.1 Design Vehicular Live Load (Article 3.6.1.2)

The design vehicular live load is designated as HL-93 and consists of a combination of the following placed within each design lane:

- a design truck *or* design tandem, and
- a design lane load.

The design vehicular live load is discussed in more detail in NSBA's *Steel Bridge Design Handbook: Design Example 1*: *Three-Span Continuous Straight Composite Steel I-Girder Bridge*[3].

6.2.2 Fatigue Live Load (Article 3.6.1.4)

The vehicular live load for checking fatigue consists of a single design truck (without the lane load) with a constant rear-axle spacing of 30 feet (Article 3.6.1.4.1).

The fatigue live load is discussed in more detail in NSBA's *Steel Bridge Design Handbook: Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge* [3].

6.2.3 Construction Live Load

A construction live load (CLL) should also be considered in evaluating the adequacy of the superstructure during construction. The construction live load is intended to account for all miscellaneous construction equipment that cannot be easily quantified at the time of design. Typically, a load of 10 psf over the width of the bridge is assumed for the construction live loading. A CLL of 10 psf is applied in this example, resulting in:

CLL load per girder = [(0.010) x 43] ÷ 2 girders = 0.215 kips/ft per girder

6.3 Load Combinations

Limit states are defined in the LRFD specifications to satisfy basic design objectives; that is, to achieve safety, serviceability, and constructability. A detailed discussion of these limit states is provided in NSBA's *Steel Bridge Design Handbook: Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge* [3]. For each limit state, the following basic equation (Article 1.3.2.1) must be satisfied:

$$\Sigma \eta_i \gamma_i Q_i \leq \phi R_n = R_r \qquad \text{Eq. (1.3.2.1-1)}$$

where: η_i = load modifier related to ductility, redundancy and operational importance

γ_i = load factor, a statistically based multiplier applied to force effects
ϕ = resistance factor, a statistically based multiplier applied to nominal resistance
Q_i = force effect
R_n = nominal resistance
R_r = factored resistance

The load factors are specified in Tables 3.4.1-1 and 3.4.1-2 of the specifications. For steel structures, the resistance factors are specified in Article 6.5.4.2.

In the *AASHTO LRFD BDS*, redundancy, ductility, and operational importance are considered more explicitly in the design. Ductility and redundancy relate directly to the strength of the bridge, while the operational importance relates directly to the consequences of the bridge being out of service. For loads for which a maximum value of γ_i is appropriate:

$$\eta_i = \eta_D \eta_R \eta_I \geq 0.95 \qquad \text{Eq. (1.3.2.1-2)}$$

where: η_D = ductility factor specified in Article 1.3.3
η_R = redundancy factor specified in Article 1.3.4
η_I = operational importance factor specified in Article 1.3.5

For loads for which a minimum value of γ_i is appropriate:

$$\eta_i = \frac{1}{\eta_D \eta_R \eta_I} \leq 1.0 \qquad \text{Eq. (1.3.2.1-3)}$$

Eq. (1.3.2.1-3) is only applicable for the calculation of the load modifier when dead- and live-load force effects are of opposite sign and the minimum load factor specified in Table 3.4.1-2 is applied to the dead-load force effects (e.g., when investigating for uplift at a support or when designing bolted field splices located near points of permanent load contraflexure); otherwise, Eq. (1.3.2.1-2) is to be used.

For typical bridges for which additional ductility-enhancing measures have not been provided beyond those required by the specifications, and/or for which exceptional levels of redundancy are not provided, the η_D and η_R factors have default values of 1.0 specified at the strength limit state. Note that some owner-agencies specify redundancy factors greater than 1.0 for certain types of steel tub girder bridges depending on the number of girders in the cross-section and the existence and number of external intermediate diaphragms. The value of the load modifier for operational importance η_I should be chosen with input from the Owner-agency. In the absence of such input, the load modifier for operational importance at the strength limit state should be taken as 1.0. At all other limit states, all three η factors must be taken equal to 1.0. For this example, η_i will be taken equal to 1.0 at all limit states.

For this example, it has been assumed that the Strength I load combination governs for the strength limit state, so only Strength I loads are checked in the sample calculations for the strength limit state included herein. In some design instances, other load cases may be critical, but for this example, these other load cases are assumed not to apply. The Service II and Fatigue load combinations will be investigated for permanent deflection checks at the service limit state and checks of selected welded details at the fatigue limit state, respectively. Refer to Design Example 1 of the NSBA Steel Bridge Design Handbook for further detail on all the load combinations specified in Table 3.4.1-1.

Strength I: 1.25DC + 1.5DW + 1.75(LL+IM)

Service II: 1.0DC + 1.0DW + 1.3(LL+IM)

And for the fatigue limit state:

Fatigue I: 1.75(LL+IM), or

Fatigue II: 0.80(LL+IM)

where LL is the fatigue load specified in Article 3.6.1.4.1.

When evaluating the strength of the structure for the maximum force effects during construction, the load factor for construction loads, for equipment and for dynamic effects (i.e., temporary dead and/or live loads that act on the structure only during construction) is not to be taken less than 1.5 in the Strength I load combination (Article 3.4.2.1). Also, the load factors for the weight of the structure and appurtenances, DC and DW, are not to be taken less than 1.25 when evaluating the construction condition.

Article 3.4.2.1 further states that unless otherwise specified by the Owner, primary steel superstructure components are to be investigated for maximum force effects during construction for an additional load combination consisting of the applicable DC loads and any construction loads that are applied to the fully erected steelwork. For this additional load combination, the load factor for DC and construction loads including dynamic effects (if applicable) is not to be taken less than 1.4. For steel superstructures, the use of higher-strength steels, composite construction, and limit-states design approaches in which smaller factors are applied to dead load force effects than in previous service-load design approaches, have generally resulted in lighter members overall. To provide adequate stability and strength of primary steel superstructure components during construction, an additional strength limit state load combination is specified for the investigation of loads applied to the fully erected steelwork (i.e., for investigation of the deck placement sequence and deck overhang effects).

Construction: Strength I: η x [1.25(D) + 1.5(C)]
Special Load Combination: η x [1.4(D + C)]

where:

D = Dead load
C = Construction loads

In this design example, for brevity, only the first of these load combinations is considered/illustrated in the constructability checks. Wind load effects during construction and in the final constructed condition are also not considered herein. Refer to NSBA's *Steel Bridge Design Handbook: Design Example 1*: *Three-Span Continuous Straight Composite Steel I-Girder Bridge* [3] for an illustration of these checks.

It should be noted that when one force effect decreases another effect, minimum load factors are to be applied to the load reducing the total effect at the strength limit state. Minimum load factors for permanent dead loads are specified in Table 3.4.1-2. For example, for the strength limit state when the permanent load vertical bending moment is positive, but the governing live load vertical bending moment is negative, the Strength I Load Combination would be: 0.90DC + 0.65DW + 1.75(LL+IM). It is important that these minimum load combinations are considered as appropriate, especially for structures that do not have an ideal balanced span arrangement.

7.0 STRUCTURAL ANALYSIS

Structural analysis is covered in Section 4 of the *AASHTO LRFD BDS*. Both approximate and refined methods of analysis are discussed in the Specifications. Refined methods of analysis are given greater coverage in the *AASHTO LRFD BDS* than they have been in the past recognizing the technological advancements that have been made to allow for easier and more efficient application of these methods. For this example, approximate methods of analysis (discussed below) are utilized to determine the lateral live load distribution to the individual girders, and the girder moments and shears are determined from a line-girder analysis.

7.1 Live Load Distribution Factors (Article 4.6.2.2)

Live loads are distributed to the individual girders according to the approximate methods specified in Article 4.6.2.2. For cross-sections with concrete decks on multiple steel tub girders, each tub may be assumed to carry the following number of lanes (Table 4.6.2.2.2b-1):

$$0.05 + 0.85\frac{N_L}{N_b} + \frac{0.425}{N_L}$$

where: N_L = number of design lanes
N_b = number of girders in the cross-section

and: $$0.5 \le \frac{N_L}{N_b} \le 1.5$$

For this example:

$$\frac{N_L}{N_b} = \frac{3}{2} = 1.5 \quad \text{ok}$$

As the ratio of N_L/N_b increases beyond the upper limit of 1.5 and fewer girders per lane are used, the effects of torsion increase and a more refined analysis is required. Where there are no depth or deflection limitations, the most efficient designs are those having the largest ratios of N_L/N_b, or the fewest practical number of tubs per design lane. Such designs will also require the least number of pieces to be fabricated, shipped, and erected.

As specified in Article 6.11.2.3, there are some restrictions to the use of the above equation for live load distribution. The satisfaction of the Article 6.11.2.3 requirements was demonstrated previously in Section 5.3 of this example.

Also, it should be noted that shear connectors must be provided in the negative flexure regions, in accordance with Article 6.11.10. Prototype bridges studied in the original development of the preceding live load distribution factor for straight tub girders utilized shear connectors throughout the negative flexure regions.

Distribution Factor for Three Lanes (for Strength and Service Limit State)

For the strength and service limit states, the lateral live load distribution factor for determining bending moment and shear in each tub girder in this example is computed as follows:

$$0.05+0.85\left(\frac{3}{2}\right)+\frac{0.425}{3}=1.467 \text{ lanes}$$

Distribution Factor for Single Lane (for Fatigue Limit State)

When checking the fatigue limit state, the fatigue vehicle is placed in a single lane. Therefore, the distribution factor for one design lane loaded is used when computing stress and shear ranges due to the fatigue load, as specified in Article 3.6.1.4.3b.

$$0.05+0.85\left(\frac{1}{2}\right)+\frac{0.425}{1}=0.900 \text{ lanes}$$

According to Article C4.6.2.2.2b, multiple presence factors, specified in Table 3.6.1.1.2-1, are not applicable to the preceding equation. Multiple presence factors have already been considered in the development of the current equation.

According to Article 3.6.1.1.2, multiple presence factors should not be applied when checking the fatigue limit state. However, the specified multiple presence factor of 1.2 for one-lane loaded (Table 3.6.1.1.2-1) is assumed not to apply to the distribution factor equation for tub girders. Thus, the preceding value of the distribution factor is not divided by 1.2.

7.1.1 Dynamic Load Allowance (Article 3.6.2.1)

The dynamic load allowance (IM) is an increment applied to the static wheel load to account for wheel-load impact from moving vehicles.

For the strength and service limit states and live-load deflection checks:

IM = 33% (Table 3.6.2.1-1)

Therefore, the factor applied to the static load is to be taken as:

$$\text{Factor } =1+\frac{\text{IM}}{100}=1+\frac{33}{100}=1.33$$

This factor is applied only to the design truck or tandem portion of the HL-93 design live load, or to the truck-train portion of the special negative-moment loading (Section 7.2).

For the fatigue limit state checks:

IM = 15% (Table 3.6.2.1-1)

$$\text{Factor} = 1 + \frac{15}{100} = 1.15$$

This factor is applied to the fatigue live load.

7.2 Analysis Results

The analysis results for a single girder are shown in the following figures. As specified in Article 6.10.1.5, the following stiffness properties were used in the analysis: 1) for loads applied to the noncomposite section, the stiffness properties of the steel section alone, 2) for permanent loads applied to the composite section, the stiffness properties of the long-term composite section assuming the concrete deck to be effective over the entire span length, and 3) for transient loads applied to the composite section, the stiffness properties of the short-term composite section assuming the concrete deck to be effective over the entire span length. Note that for a continuous span with a nonprismatic member, changes to the stiffness of individual sections can have a significant effect on the analysis results. Thus, for such a span, whenever plate sizes for a particular section are revised, it is always desirable to perform a new analysis.

NOTE: *The analysis results shown herein apply to an example girder designed using earlier versions of the AASHTO LRFD (i.e., prior to the 8th Edition). Revisions to some of the plate sizes in this example design were necessary to provide a more reasonable b/t ratio for the bottom (tension) flange in regions of positive flexure, and to reduce the performance ratio (i.e., demand-to-capacity ratio) for the stiffened bottom flange in compression over the interior piers below 1.0 at the strength limit state. While it is nearly always desirable to perform a new analysis whenever plate sizes are revised, the effect on the analysis results in this case was felt to be relatively minor and so new analyses were not performed. The primary intent of this example is to illustrate the proper application of the AASHTO LRFD provisions to the design of a straight continuous steel tub-girder bridge. However, this also illustrates that a designer should always be aware of specification changes and how they may affect a design and perhaps future load ratings.*

In the first series of plots (Figure 6 and Figure 7), moment and shear envelopes due to the *unfactored* dead and live loads are given. Live-load moments in regions of positive flexure and in regions of negative flexure *outside points of permanent-load contraflexure* are due to the HL-93 loading (design tandem or design truck with the variable axle spacing combined with the design lane load; whichever governs). Live-load moments in regions of negative flexure *between points of permanent-load contraflexure* are the larger of the moments caused by the HL-93 loading or a special negative-moment loading (90 percent of the effect of the truck-train specified in Article 3.6.1.3.1 combined with 90 percent of the effect of the design lane load). Live-load shears are due to the HL-93 loading only. However, it should be noted that interior-pier reactions are to be calculated based on the larger of the shears caused by the HL-93 loading or the special negative-moment loading. The indicated live-load moment and shear values include the appropriate lateral distribution factor and dynamic load allowance for the strength limit state, computed earlier

(Sections 7.1 and 7.1.1). DC_1 is the component dead load acting on the noncomposite section and DC_2 is the component dead load acting on the long-term composite section. DW is the wearing surface load also acting on the long-term composite section.

The second series of plots (Figure 8 and Figure 9) shows the moment and shear envelopes due to the *unfactored* fatigue load specified in Article 3.6.1.4.1. The appropriate lateral distribution factor and reduced dynamic load allowance for the fatigue limit state are included in the indicated values (Sections 7.1 and 7.1.1).

The unfactored moments and shears resulting from the application of the construction live load (CLL) are presented in Table 1.

7.2.1 Optional Live Load Deflection Evaluation (Article 3.6.1.3.2)

The *AASHTO LRFD BDS* contains provisions for optional live load deflection criteria, to be invoked at the discretion of the Owner-agency.

The vehicular live load for checking the optional live load deflection criterion specified in Article 3.6.1.3.2 is taken as the larger of:

- The design truck alone.
- The design lane load plus 25 percent of the design truck.

These loadings are used to produce apparent live load deflections similar to those produced by older traditional AASHTO HS20 design live loadings. It is assumed in the live load deflection check that all design lanes are loaded and that all supporting components are assumed to deflect equally (Article 2.5.2.6.2). For composite design, Article 2.5.2.6.2 also permits the stiffness of the design cross-section used for the determination of the deflection to include the entire width of the roadway and the structurally continuous portions of any railings, sidewalks, and barriers. Concrete barriers and sidewalks, and even railings, often contribute to the stiffness of composite superstructures at service load levels. However, inclusion of concrete items other than the deck complicates the calculation of the composite stiffness of the superstructure and is virtually never considered for routine bridges. Barriers are generally located at the edges of the deck, where they tend to stiffen and draw load to the exterior girders. Thus, any beneficial stiffening of the system tends to be counterbalanced by unequal distribution of the loading among the girders and the associated reduction in computed deflections resulting from consideration of the barriers tends to be negligible. Live load deflection is checked using the live load portion of the SERVICE I load combination (Table 3.4.1-1), including the appropriate dynamic load allowance.

Because live load deflection is not anticipated to be of significant concern for this example, the stiffness of the barriers is not included for simplicity. For this example, the maximum live load deflection was found to occur in the center span and is:

(Δ_{LL+IM}) center span = 3.32 in.

In the absence of specific criteria, the live load deflection limits of Article 2.5.2.6.2 may be used. Note that for steel tub girders, the provisions of Article 6.11.4 apply regarding control of permanent deflection at the service limit state.

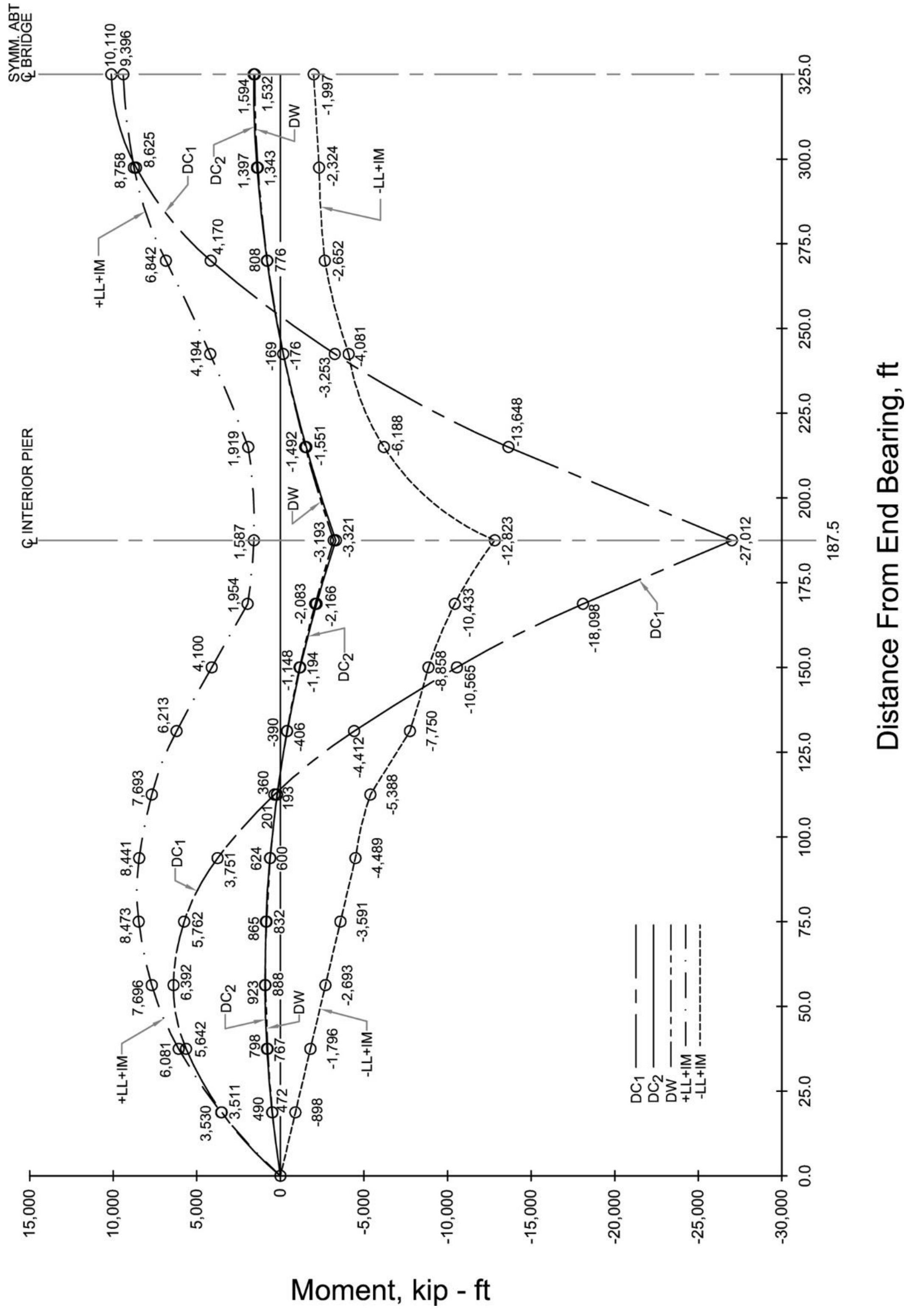

Figure 6 Dead- and Live-Load Moment Envelopes

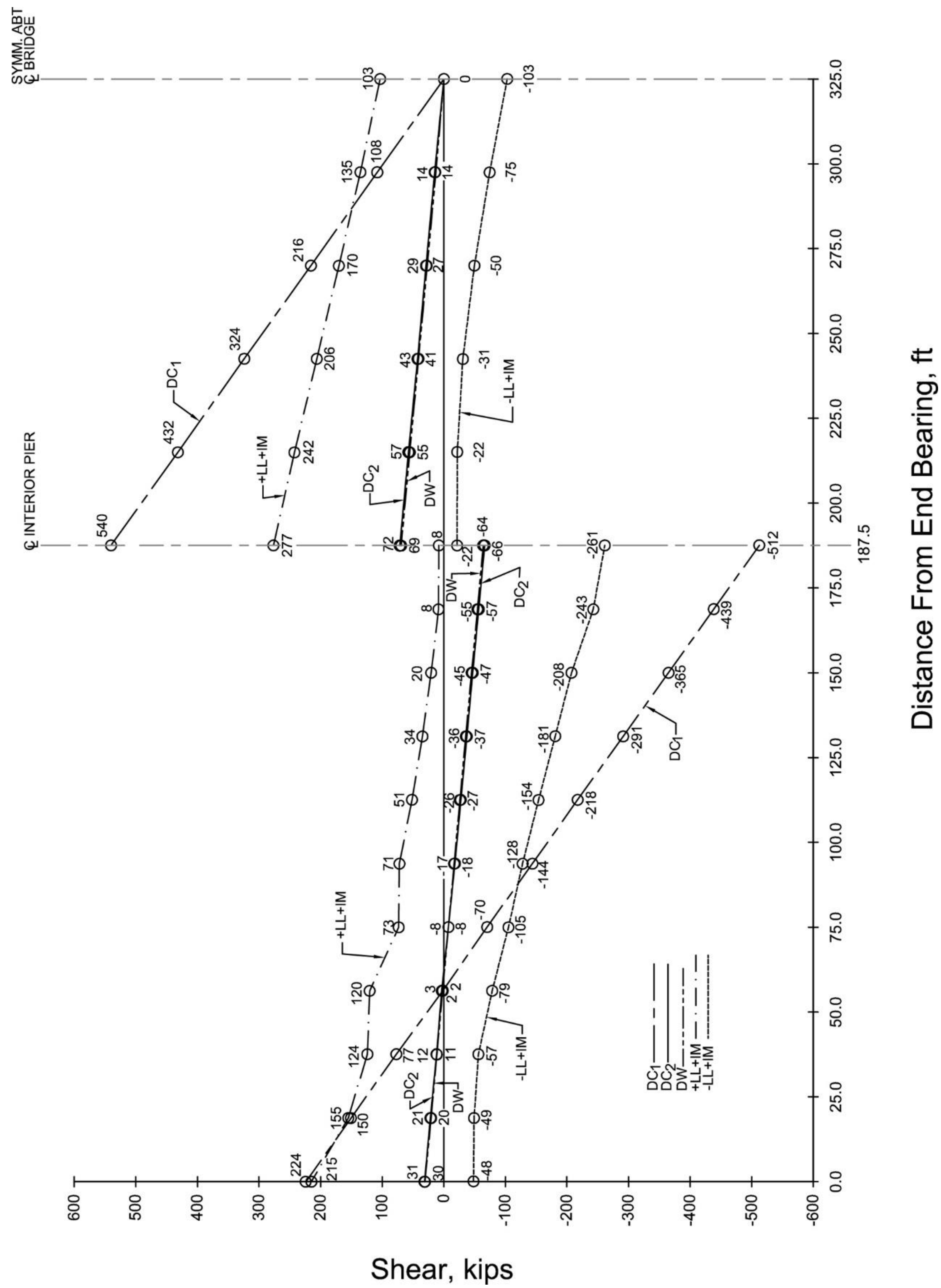

Figure 7 Dead- and Live-Load Shear Envelopes

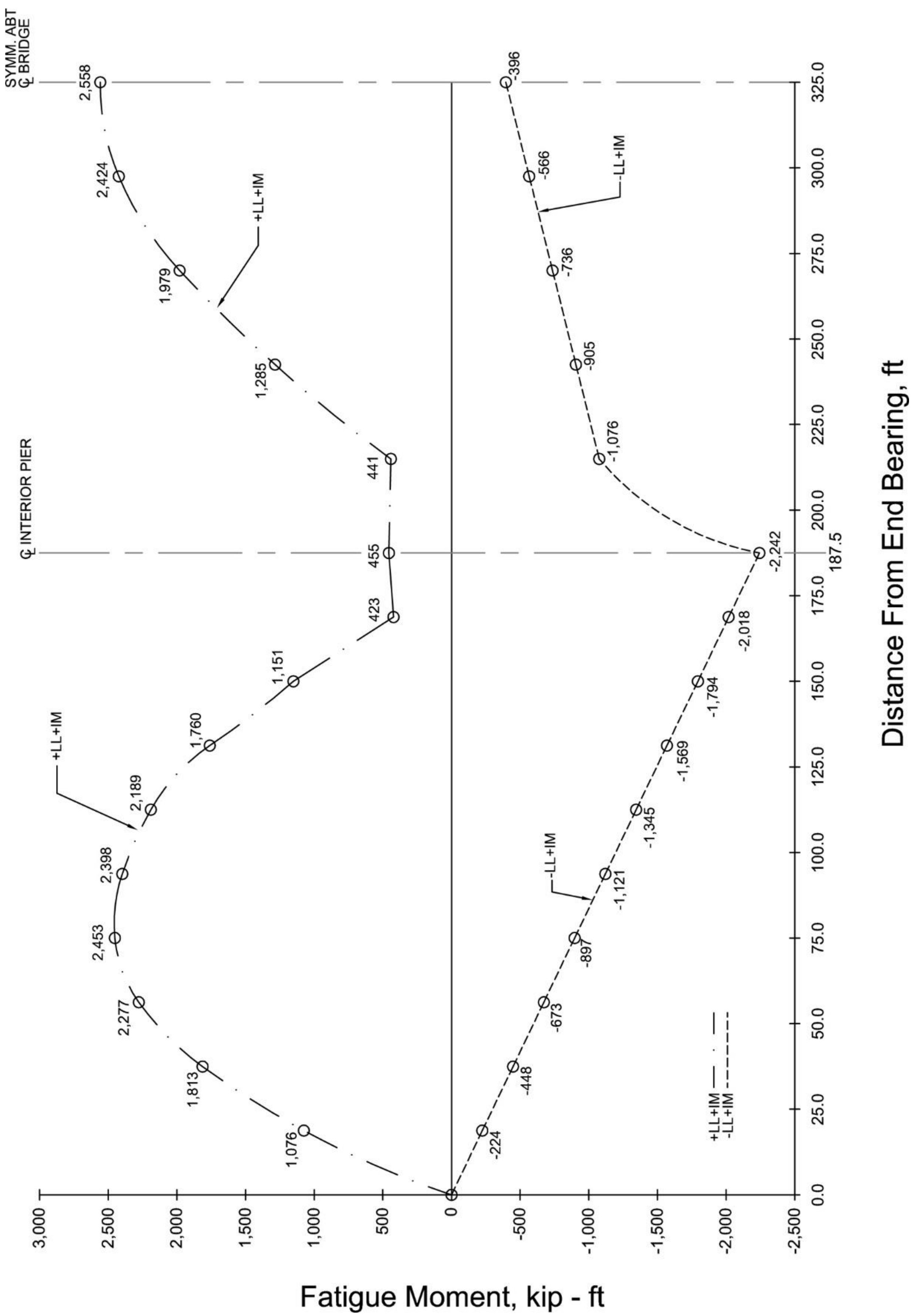

Figure 8 Fatigue Live Load Moments

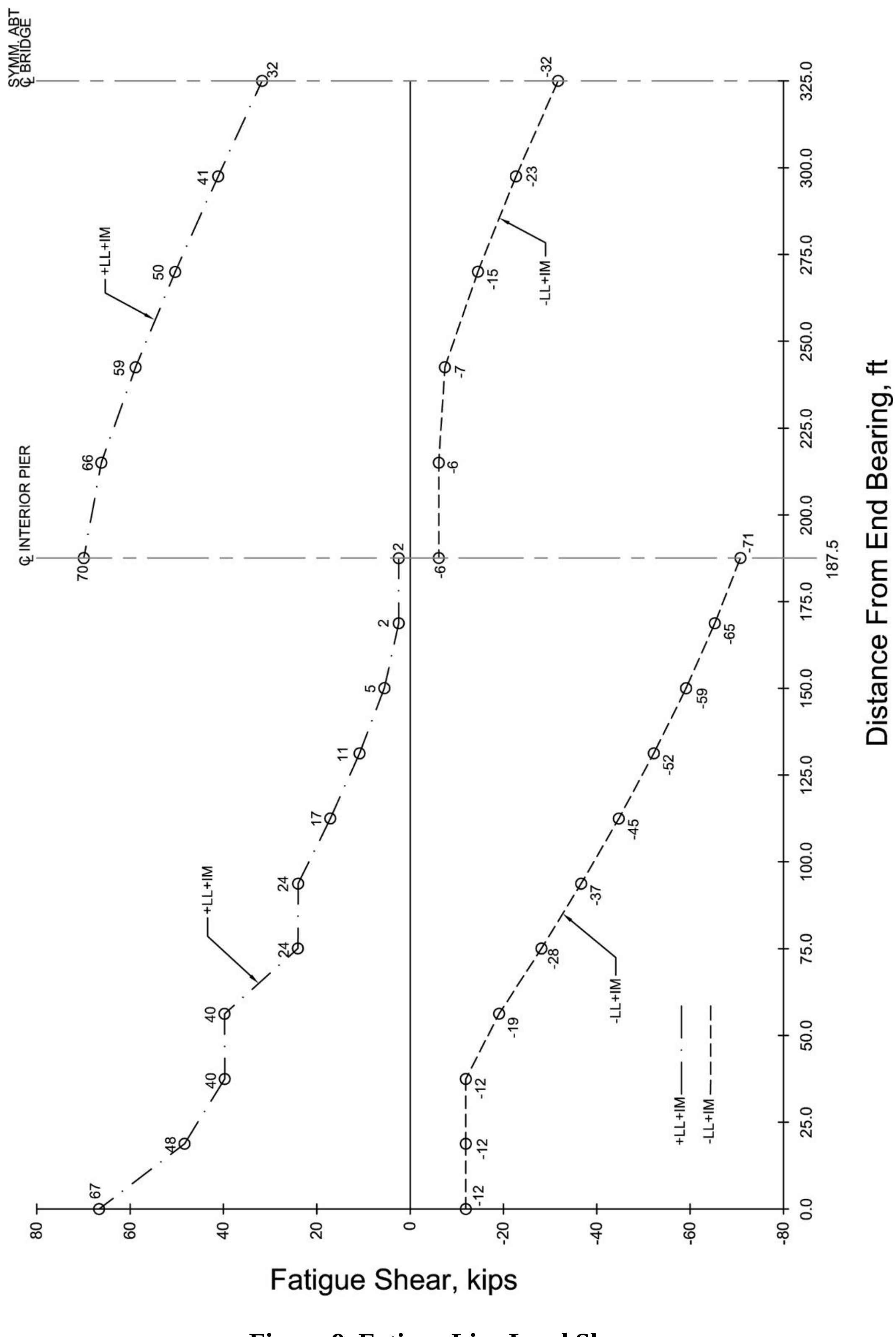

Figure 9 Fatigue Live Load Shears

Table 1 Construction Live Load (CLL) Moments and Shears

Span	Tenth Point	Moment (kip-ft)	Shear (kip)
1	0.0	0	12
1	0.1	192	8
1	0.2	309	4
1	0.3	350	0
1	0.4	315	-4
1	0.5	205	-8
1	0.6	20	-12
1	0.7	-241	-16
1	0.8	-578	-20
1	0.9	-990	-24
1	1.0	-1,478	-28
2	0.0	-1,478	30
2	0.1	-747	24
2	0.2	-178	18
2	0.3	228	12
2	0.4	472	6
2	0.5	553	0

8.0 SAMPLE CALCULATIONS

Sample calculations for two critical sections in the example bridge follow. Section 2-2 (refer to Figure 3) represents the section of maximum positive flexure in the center span (Span 2), and Section 2-1 represents the section at each interior pier. The calculations illustrate the application of some of the more significant design provisions contained in Article 6.11. The calculations include checks to be made at the service, fatigue, and strength limit states. Detailed constructability checks are also illustrated. Web-stiffener design and the design of the stud shear connectors are not illustrated in this example. The application of the provisions for the design of those elements is illustrated in Design Example 1 of the NSBA Steel Bridge Design Handbook and would be performed similarly for this example.

The calculations herein make use of the moment and shear envelopes shown in Figure 6 through Figure 9 and the section properties calculated below. In the calculation of the major-axis bending stresses throughout the sample calculations, compressive stresses are always shown as negative values and tensile stresses are always shown as positive values. This convention is followed regardless of the expected sign of the calculation result, in which the sign of the major-axis bending moment is maintained.

8.1 Section Properties

The calculation of the section properties for Sections 2-2 and 2-1 is illustrated below. In computing the composite section properties, the structural slab thickness, or total thickness minus the thickness of the integral wearing surface, is used.

Compute the modular ratio, *n* (Article 6.10.1.1.1b):

$$n = \frac{E}{E_c} \qquad \text{Eq. (6.10.1.1.1b-1)}$$

where E_c is the modulus of elasticity of the concrete determined as specified in Article 5.4.2.4. A unit weight of 0.150 kcf is used for the concrete in the calculation of the modular ratio. The correction factor for source of aggregate, K_1, is taken as 1.0. The traditional equation for E_c for normal-weight concrete given in Article C5.4.2.4 is used in this example.

$$E_c = 33{,}000\, K_1\, w_c^{1.5}\sqrt{f'_c} \qquad \text{Eq. (C5.4.2.4-2)}$$

$$E_c = 33{,}000\,(1.0)\,(0.150)^{1.5}\sqrt{4.0} = 3{,}834\text{ ksi}$$

$$n = \frac{29{,}000}{3{,}834} = 7.56,\ \text{use } 8.0$$

Therefore, n = 8 will be used in all subsequent computations.

8.1.1 Section 2-2: Maximum Positive Moment in Center Span

Section 2-2 located at the center of Span 2, as shown in Figure 10. For this section, the longitudinal reinforcement is conservatively neglected in computing the composite section properties as is typically assumed in design.

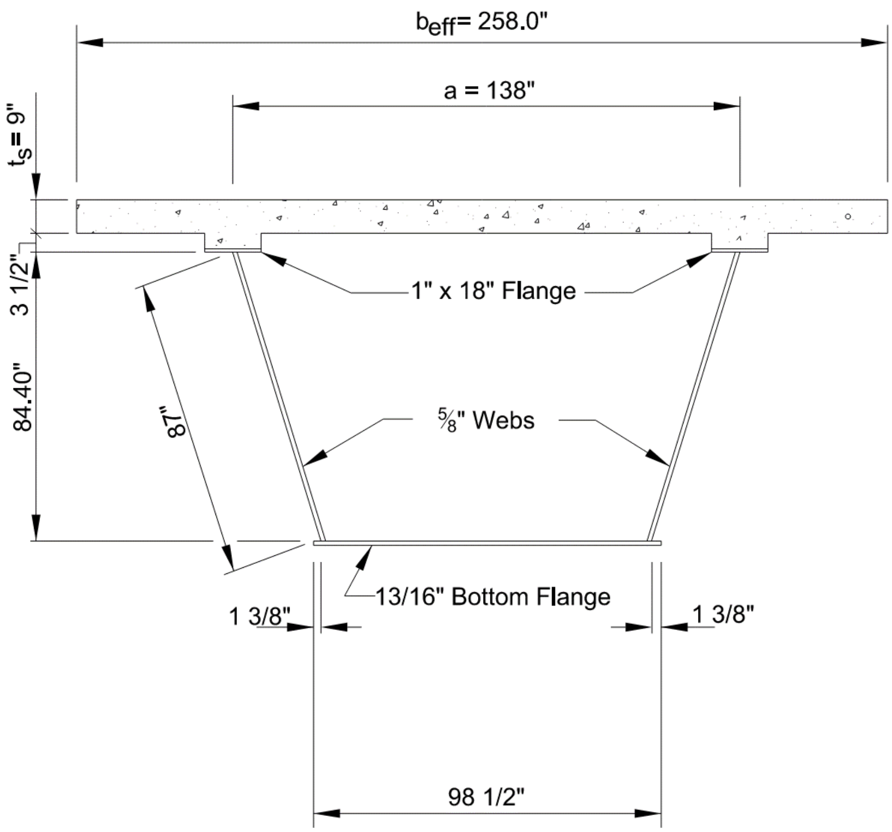

Figure 10 Sketch of Section 2-2

8.1.1.1 Effective Width of Concrete Deck (Article 6.10.1.1.1e)

As specified in Article 6.10.1.1.1e, the effective flange width is to be determined as specified in Article 4.6.2.6. The individual webs of the tub girder must be initially considered separately since one web is an exterior web and the other is an interior web. According to Article 4.6.2.6, for an exterior web, the effective flange width may be taken as one-half the effective width of the adjacent interior girder, plus the full width of the overhang.

For an interior web, the effective flange width may be taken as one-half the distance to the adjacent girder's nearest web plus one-half the distance to the adjacent web of the same girder.

For an interior web in regions of positive flexure, b_{eff} is the least of:

$$b_{eff\_int\_web} = \frac{144.0}{2} + \frac{138.0}{2} = 141.0 \text{ in.}$$

For an exterior web, b_{eff} is the least of:

$$b_{eff\_ext\_web} = \frac{138.0}{2} + 48.0 = 117.0 \text{ in.}$$

The total effective flange width for the tub girder is calculated as:

b_{eff} = 141.0 + 117.0 = 258.0 in.

8.1.1.2 Elastic Section Properties for Section 2-2

The moment of inertia of a single inclined web, I_{ow}, with respect to a horizontal axis at mid-depth of the web (Figure 11) is computed as:

$$I_{ow} = \frac{S^2}{S^2+1} I_w$$

where: S = web slope with respect to the horizontal = 4.00
I_w = moment of inertia with respect to an axis normal to the web

$$I_{ow} = \left(\frac{4.0^2}{4.0^2+1}\right) \frac{1}{12}(0.625)(87.0)^3 = 32{,}280 \text{ in.}^4$$

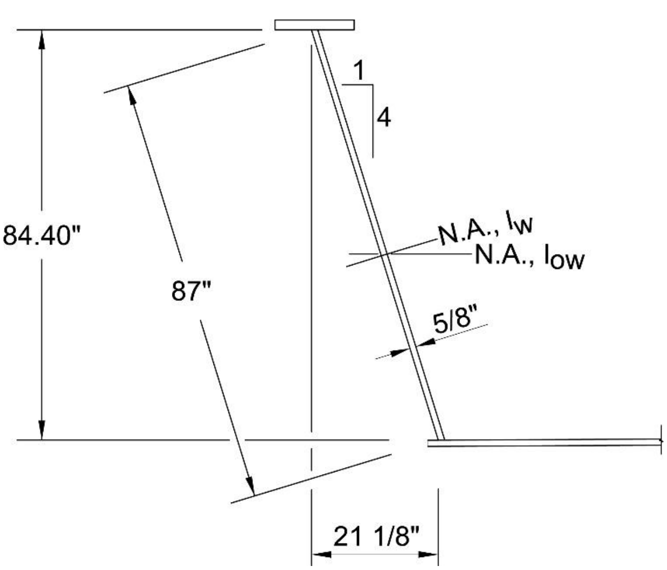

Figure 11 Moment of Inertia of an Inclined Web

In the calculation of the section properties, *d* is measured vertically from a horizontal axis through the mid-depth of the web to the centroid of each element of the tub girder.

Table 2 Section 2-2: Steel Section Properties

Component	A	d	Ad	Ad^2	I_o	I
2 Top Flanges 1" x 18"	36.00	42.70	1,537	65,638	3.00	65,641
2 Webs 5/8" x 87"	108.75				64,560	64,560
Bottom Flange 13/16" x 98½"	80.03	-42.61	-3,410	145,303	4.40	145,307
Σ	224.78		-1,873			275,508

-8.33(1,873) = -15,602

I_{NA} = 259,906 in.^{4}

$$d_s = \frac{-1,873}{224.78} = -8.33 \text{ in.}$$

$$d_{\text{Top of Steel}} = 43.20 + 8.33 = 51.53 \text{ in.}$$

$$d_{\text{Bot of Steel}} = 43.01 - 8.33 = 34.68 \text{ in.}$$

$$S_{\text{Top of Steel}} = \frac{259,906}{51.53} = 5,044 \text{ in.}^3$$

$$S_{\text{Bot of Steel}} = \frac{259,906}{34.68} = 7,494 \text{ in.}^3$$

Table 3 Section 2-2: Composite (3n) Section Properties

Component	A	d	Ad	Ad^2	I_o	I
Steel Section	224.78		-1,873			275,508
Concrete Slab 9″ x 258″/24	96.75	50.20	4,857	243,814	653.1	244,467
Σ	321.53		2,984			519,975

-9.28(2,984) = -27,692

I_{NA} = 492,283 in.^{4}

$$d_{3n} = \frac{2,984}{321.53} = 9.28 \text{ in.}$$

$$d_{\text{Top of Steel}} = 43.20 - 9.28 = 33.92 \text{ in.}$$

$$d_{\text{Bot of Steel}} = 43.01 + 9.28 = 52.29 \text{ in.}$$

$$S_{\text{Top of Steel}} = \frac{492,283}{33.92} = 14,513 \text{ in.}^3$$

$$S_{\text{Bot of Steel}} = \frac{492,283}{52.29} = 9,414 \text{ in.}^3$$

Table 4 **Section 2-2: Composite (n) Section Properties**

Component	A	d	Ad	Ad^2	I_o	I
Steel Section	224.78		-1,873			275,508
Concrete Slab 9″ x 258″/8	290.25	50.20	14,571	731,442	1,959	733,401
Σ	515.03		12,698			1,008,909

-24.65(12,698) = -313,006

I_{NA} = 695,903 in.^{4}

$$d_n = \frac{12,698}{515.03} = 24.65 \text{ in.}$$

$$d_{\text{Top of Steel}} = 43.20 - 24.65 = 18.55 \text{ in.}$$

$$d_{\text{Bot of Steel}} = 43.01 + 24.65 = 67.66 \text{ in.}$$

$$S_{\text{Top of Steel}} = \frac{695,903}{18.55} = 37,515 \text{ in.}^3$$

$$S_{\text{Bot of Steel}} = \frac{695,903}{67.66} = 10,285 \text{ in.}^3$$

*Note that the above computations for composite section properties consider the height of the concrete haunch but neglect the area of the concrete haunch. Including or excluding the concrete haunch area for section resistance is generally an Owner-agency preference. It has not been included in this example for simplicity.

8.1.1.3 Plastic Moment Capacity for Section 2-2

Determine the plastic-moment, M_p, of the composite section using the equations provided in Appendix D6 (Article D6.1). The longitudinal deck reinforcement is conservatively neglected. M_p is calculated for the tub girder as follows:

$P_t = F_{yt}b_tt_t$ = (50)(98.50)(0.8125) = 4,002 kips

$P_w = 2F_{yw}Dt_w$ = (2)(50)(87.00)(0.625) = 5,438 kips

$P_c = 2F_{yc}b_ct_c$ = (2)(50)(18.0)(1.0) = 1,800 kips

$P_s = 0.85f_c'b_{eff}t_s$ = (0.85)(4.0)(258.0)(9.0) = 7,895 kips

Is $P_t + P_w \geq P_c + P_s$?

$P_t + P_w$ = 9,440 kips < < $P_c + P_s$ = 9,695 kips; Therefore, PNA is in the top flange, use Case II in Table D6-1.

$$\bar{y} = \frac{t_c}{2}\left[\frac{P_w + P_t - P_s}{P_c} + 1\right]$$

$$\bar{y} = \frac{1.0}{2}\left[\frac{5,438 + 4,002 - 7,895}{1,800} + 1\right]$$

$= 0.93$ in. downward from the top of the top flange

$$M_p = \frac{P_c}{2t_c}\left[\bar{y}^2 + \left(t_c - \bar{y}\right)^2\right] + \left[P_s d_s + P_w d_w + P_t d_t\right]$$

Calculate the distances from the PNA to the centroid of each element:

$$d_t = 84.4 + 1.0 - 0.93 + 0.4063 = 84.88 \text{ in.}$$

$$d_w = \frac{84.4}{2} + 1.0 - 0.93 = 42.27 \text{ in.}$$

$$d_s = \frac{9.0}{2} + 3.5 - 1.0 + 0.93 = 7.93 \text{ in.}$$

Calculate M_p:

$$M_p = \left[\frac{1,800}{2(1.0)}\right]\left[(0.93)^2 + (1.0 - 0.93)^2\right] + \left[(7,895)(7.93) + (5,438)(42.27) + (4,002)(84.88)\right]$$

$M_p = 632,944$ kip-in

$M_p = 52,745$ kip-ft

8.1.1.4 Yield Moment for Section 2-2

Calculate the yield moment M_y of the composite section using the equations provided in Appendix D6 (Article D6.2.2). M_y is taken as the sum of the factored moments at the strength limit state applied separately to the steel, long-term, and short-term composite sections to cause first yield in either steel flange. Flange lateral bending is to be disregarded in the calculation.

$$F_y = \frac{M_{D1}}{S_{NC}} + \frac{M_{D2}}{S_{LT}} + \frac{M_{AD}}{S_{ST}} \qquad \text{Eq. (D6.2.2-1)}$$

where M_{D1}, M_{D2}, and M_{AD} are the moments applied to the steel, long-term and short-term composite sections, respectively, factored by η and the corresponding load factors.

Solve for M_{AD} (bottom flange governs by inspection):

$$50 = 1.0\left[\frac{1.25(10{,}110)(12)}{7{,}494} + \frac{1.25(1{,}594)(12) + 1.50(1{,}532)(12)}{9{,}414} + \frac{M_{AD}}{10{,}285}\right]$$

M_{AD} = 249,871kip-in = 20,823kip-ft

$M_y = M_{D1} + M_{D2} + M_{AD}$ Eq. (D6.2.2-2)

M_y = 1.0[1.25(10,110) + 1.25(1,594) + 1.50(1,532) + 20,823]

M_y = 37,751 kip-ft

8.1.2 Section 2-1: Maximum Negative Moment at Interior Support

Section 2-1 is at the interior support and is shown in Figure 12.

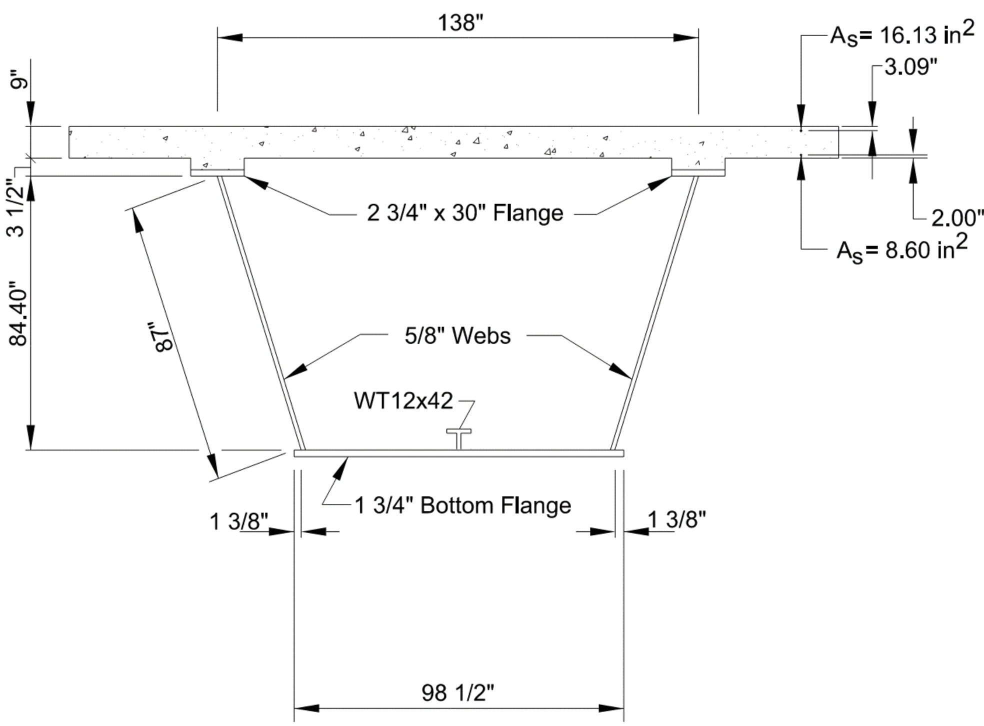

Figure 12 Sketch Showing Section 2-1

8.1.2.1 Effective Width of Concrete Deck (Article 6.10.1.1.1e)

The effective flange width for Section 2-1 is calculated using the procedures discussed previously for Section 2-2.

For an interior web in regions of negative flexure, b_{eff} is the least of:

$$b_{eff\_int\_web} = \frac{144.0}{2} + \frac{138.0}{2} = 141.0 \text{ in.}$$

For an exterior web, b_{eff} is the least of:

$$b_{eff\_ext\_web} = \frac{138.0}{2} + 48.0 = 117.0 \text{ in.}$$

The total effective flange width for the tub girder is calculated as:

b_{eff} = 141.0 + 117.0 = 258.0 in.

8.1.2.2 Minimum Negative Flexure Concrete Deck Reinforcement

To control concrete deck cracking in regions of negative flexure, Article 6.10.1.7 specifies that the total cross-sectional area of the longitudinal reinforcement must not be less than 1 percent of the total cross-sectional area of the deck. The minimum longitudinal reinforcement must be provided wherever the longitudinal tensile stress in the concrete deck due to either the factored construction loads or Load Combination Service II exceeds ϕf_r, where f_r is the modulus of rupture of the concrete determined as specified in Article 6.10.1.7 and ϕ is taken as 0.90. Article 6.10.1.7 further specifies that the reinforcement is to have a specified minimum yield strength not less than 60 ksi and the size should not exceed No. 6 bars. The reinforcement should be placed in two layers uniformly distributed across the deck width, and two-thirds should be placed in the top layer. The individual bars should be spaced at intervals not exceeding 12 inches.

Article 6.10.1.1.1c states that for calculating stresses in composite sections subjected to negative flexure at the strength limit state, the composite section for both short-term and long-term moments is to consist of the steel section and the longitudinal reinforcement within the effective width of the concrete deck. Referring to the cross-section shown in Figure 1:

A_{deck} = (entire width of 9-inch-thick deck) + (triangular portion of overhang)

$$A_{deck} = \frac{9.0}{12}(43.0) + 2\left[\frac{1}{12}\left(\frac{3.5+0.5}{2}\right)\left(4.0 - \frac{30/2}{12}\right)\right] = 33.17 \text{ ft}^2 = 4{,}777 \text{ in.}^2$$

$0.01(4{,}777) = 47.77 \text{ in.}^2$

$$\frac{47.77}{43.0} = 1.11 \text{ in.}^2/\text{ft} = 0.093 \text{ in.}^2/\text{in.}$$

$0.093(258.0) = 23.99 \text{ in.}^2$

For the top layer, alternate #5 bars @ 12 inches and #6 bars @ 12 inches, and in the bottom layer use #4 bars @ 6 inches. Therefore, the total area of steel in the given effective width of concrete deck is:

$$A_S = (0.31 + 0.44 + 0.40)\left(\frac{258.0}{12}\right) = 24.73 \text{ in.}^2 > 23.99 \text{ in.}^2$$

Also, two-thirds of the reinforcement is in the top layer: $\frac{0.31 + 0.44}{1.15} = 0.65 \approx \frac{2}{3}$

For the purposes of this example, the longitudinal reinforcement in the two layers is assumed to be combined into a single layer placed at the centroid of the two layers (with each layer also including the assumed transverse deck reinforcement). From separate calculations, the centroid of the two layers is computed to be 4.54 inches from the bottom of the concrete deck.

For members with shear connectors provided throughout their entire length that also satisfy the minimum reinforcement requirements of Article 6.10.1.7, dead load and live load stresses and live load stress ranges for the fatigue and service limit state design at all sections in the member due to loads applied to the composite section may be computed using the short-term or long-term composite section, as appropriate, assuming the concrete deck to be effective for both positive and negative flexure (Articles 6.6.1.2.1 and 6.10.4.2.1). Article 6.10.4.2.1 also requires that the maximum longitudinal tensile stress in the concrete deck at the service limit state (i.e., due to the Service II loads) not exceed two times the modulus of rupture of the concrete, f_r. Therefore, section properties for the short-term and long-term composite section, including the concrete deck but neglecting the longitudinal reinforcement, are also calculated.

8.1.2.3 Elastic Section Properties for Section 2-1

Calculations for the elastic section properties of Section 2-1 are shown in Table 5 through Table 8. The section properties include the contribution of the bottom flange longitudinal stiffener (WT 12 x 42).

Table 5 Section 2-1: Steel Section Properties

Component	A	d	Ad	Ad^2	I_o	I
2 Top Flanges 2¾" x 30"	165.00	43.58	7,191	313,371	104.0	313,475
2 Webs 5/8" x 87"	108.75				64,560	64,560
Bottom Flange 1¾" x 98½"	172.38	-43.08	-7,426	319,918	43.99	319,962
Stiffener WT12x42	12.40	-33.07	-410.1	13,561	166.0	13,727
Σ	458.53		-645.1			711,724

-1.41(645.1) = -910

I_{NA} = 710,814 in.^{4}

$$d_s = \frac{-645.1}{458.53} = -1.41 \text{ in.}$$

$$d_{\text{Top of Steel}} = 44.95 + 1.41 = 46.36 \text{ in.}$$

$$d_{\text{Bot of Steel}} = 43.95 - 1.41 = 42.54 \text{ in.}$$

$$S_{\text{Top of Steel}} = \frac{710,814}{46.36} = 15,332 \text{ in.}^3$$

$$S_{\text{Bot of Steel}} = \frac{710,814}{42.54} = 16,709 \text{ in.}^3$$

Table 6 **Section 2-1: Composite Section Properties with Longitudinal Steel Reinforcement**

Component	A	d	Ad	Ad^2	I_o	I
Steel Section	458.53		-645.1			711,724
Long. Reinforcement	24.73	50.24	1,242	62,420		62,420
Σ	483.26		596.9			774,144

-1.24(596.9) = <u>-740.2</u>

I_{NA} = 773,404 in.^{4}

$$d_{\text{reinf}} = \frac{596.9}{483.26} = 1.24 \text{ in.}$$

$$d_{\text{Top of Steel}} = 44.95 - 1.24 = 43.71 \text{ in.}$$

$$d_{\text{Bot of Steel}} = 43.95 + 1.24 = 45.19 \text{ in.}$$

$$S_{\text{Top of Steel}} = \frac{773,404}{43.71} = 17,694 \text{ in.}^3$$

$$S_{\text{Bot of Steel}} = \frac{773,404}{45.19} = 17,114 \text{ in.}^3$$

Table 7 **Section 2-1: Composite (3n) Section Properties**

Component	A	d	Ad	Ad^2	I_o	I
Steel Section	458.53		-645.1			711,724
Concrete Slab 9" x 258"/24	96.75	50.2	4,857	243,814	653	244,467
Σ	555.28		4,212			956,191

-7.59(4,212) = <u>-31,969</u>

I_{NA} = 924,222 in.^{4}

$$d_{3n} = \frac{4,212}{555.28} = 7.59 \text{ in.}$$

$$d_{\text{Top of Steel}} = 44.95 - 7.59 = 37.36 \text{ in.}$$

$$d_{\text{Bot of Steel}} = 43.95 + 7.59 = 51.54 \text{ in.}$$

$$S_{\text{Top of Steel}} = \frac{924,222}{37.36} = 24,738 \text{ in.}^3$$

$$S_{\text{Bot of Steel}} = \frac{924,222}{51.54} = 17,932 \text{ in.}^3$$

Table 8 **Section 2-1: Composite (n) Section Properties**

Component	A	d	Ad	Ad^2	I_o	I
Steel Section	458.53		-645.1			711,724
Concrete Slab 9" x 258"/8	290.3	50.2	14,573	731,568	1,959	733,527
Σ	748.83		13,928			1,445,251

-18.60(13,928) = -259,061

I_{NA} = 1,186,190 in.^{4}

$$d_n = \frac{13,928}{748.83} = 18.60 \text{ in.}$$

$$d_{\text{Top of Steel}} = 44.95 - 18.60 = 26.35 \text{ in.}$$

$$d_{\text{Bot of Steel}} = 43.95 + 18.60 = 62.55 \text{ in.}$$

$$S_{\text{Top of Steel}} = \frac{1,186,190}{26.35} = 45,017 \text{ in.}^3$$

$$S_{\text{Bot of Steel}} = \frac{1,186,190}{62.55} = 18,964 \text{ in.}^3$$

8.2 Girder Constructability Check: Section 2-2 (Positive Moment, Span 2)

Article 6.11.3 directs the Engineer to Article 6.10.3 for the constructability checks of tub girders. For critical stages of construction, the provisions of Articles 6.10.3.2.1 through 6.10.3.2.3 are to be applied to the top flanges of the tub girder. The noncomposite bottom tub flange in compression or tension is to satisfy the requirements specified in Article 6.11.3.2. Web shear is to be checked in accordance with Article 6.10.3.3 with the shear taken along the slope of the web in accordance with the provisions of Article 6.11.9.

For this example, a deck placement sequence is not investigated. The demonstration of the constructability checks for a deck placement sequence is provided in Design Example 1 of the NSBA Steel Bridge Design Handbook. In the absence of a deck placement sequence, the weight of the concrete deck is assumed placed in a single stage in this example. Although not illustrated in this example, Article 6.10.3.4.1 requires that sections in positive flexure that are composite in the final condition, but are noncomposite during construction, be investigated for flexure according to the provisions of Article 6.10.3.2 during the various stages of the deck placement. Furthermore, wind loads during construction are not considered in this example (refer again to Design Example 1 for an illustration of these checks).

Calculate the factored maximum flexural stresses in the flanges of the steel section resulting from the application of steel self-weight and the assumed single placement of the concrete deck (DC_1). As specified in Article 6.10.1.6, for design checks where the flexural resistance is based on lateral torsional buckling, f_{bu} is to be determined as the largest value of the compressive stress throughout the unbraced length in the flange under consideration, calculated without consideration of flange lateral bending. For design checks where the flexural resistance is based on yielding, flange local buckling or web bend buckling, f_{bu} may be determined as the stress at the section under consideration. From Figure 2, brace points adjacent to Section 2-2 are located at intervals of 15.875 feet, and the largest stress occurs within this unbraced length. As mentioned in Article C6.11.3.2, top lateral bracing attached to the flanges at points where only struts exist between the flanges may be considered as brace points at the discretion of the Engineer. In the case of this design example, which features a full-length top flange lateral bracing system, it is reasonable to consider both the struts with internal cross-frames and the alternating struts without internal cross-frames as brace points for the top flanges. As discussed previously, the η factor is taken equal to 1.0 in this example. Therefore,

For Strength I:

General: $$(f_{bu})_{DC1} = \frac{\eta\, \gamma\, M_{DC1}}{S_{nc}}$$

Top flange: $$(f_{bu})_{DC1} = \frac{1.0(1.25)(10{,}110)(12)}{5{,}044} = -30.07 \text{ ksi}$$

Bot. flange: $$(f_{bu})_{DC1} = \frac{1.0(1.25)(10{,}110)(12)}{7{,}494} = 20.24 \text{ ksi}$$

In addition to the steel, permanent metal deck forms, and concrete self-weight loads, it is prudent to assume a construction live loading (CLL) on the structure during placement of the concrete deck, as discussed in Section 6.2.3. In the Strength I load combination; a load factor of 1.5 is applied to all construction loads, in accordance with Article 3.4.2. Therefore,

Top flange: $$(f_{bu})_{CLL} = \frac{1.0(1.5)(553)(12)}{5{,}044} = -1.97 \text{ ksi}$$

Bot. flange: $$(f_{bu})_{CLL} = \frac{1.0(1.5)(553)(12)}{7{,}494} = 1.33 \text{ ksi}$$

Top flange: f_{bu} = -30.07 + (-1.97) = -32.04ksi

Bot. flange: f_{bu} = 20.24+1.33 = 21.57ksi

Although not included in this example in the interest of brevity, the special load combination specified in Article 3.4.2.1 must also be considered in the design checks for the deck placement sequence (see Section 6.3).

8.2.1 Top Flange Lateral Bending due to Horizontal Component of Web Shear

The change in the horizontal component of the web shear in the inclined web along the span acts as a lateral force in the flanges of the tub girder. Under initial noncomposite dead load DC_1, the lateral force due to shear is assumed to be distributed to the top flanges of the open tub girder. Recent research has suggested that the top and bottom flanges do not equally resist the lateral force due to the horizontal component of the web shear, as has been generally assumed in past practice Fan and Helwig [12] suggest that, with the exception of the girder self-weight, the entire lateral force should be assumed to act on the top flanges. To simplify the calculations for this example, it will conservatively be assumed that the entire DC_1 horizontal component of web shear is applied to the top flanges. The change in vertical shear force, equal to the lateral load on the top flanges, is constant and is equal to the change in DC_1 shear force in the girder measured at adjacent supports divided by the span length.

The change in DC_1 girder shear over the length of the Span 2 is:

$$\Delta V_V = \frac{\left[\left|-540\right| + 540\right]}{275} = 3.93 \text{ kip/ft}$$

The shear force used above is the total for the girder (2 webs). Therefore, the horizontal component of the web shear per top flange is:

$$\Delta V_H = \frac{1}{2}\ \Delta V_V\ \tan(\theta_{WEB})\ =\ \frac{1}{2}(3.93)(0.25) = 0.49 \text{ kip/ft}$$

Assuming the flange is continuous and that the adjacent unbraced lengths are approximately equal, the lateral bending moment due to a statically equivalent uniformly distributed lateral load may be estimated as follows, similar to Equation C6.10.3.4.1-1, where s is the brace spacing:

$$M_{LAT} = \frac{\Delta V_H s^2}{12} = \frac{(0.49)(15.875)^2}{12} = 10.29 \text{ kip-ft}$$

The section modulus of the 1.0 in. x 18 in. top flange about a vertical axis through the web is:

$$S_f = \frac{(1.0)(18)^2}{6} = 54.00 \text{ in.}^3$$

The Strength I lateral bending stress due to the horizontal component of web shear, including the dead load factor of 1.25, is then computed as:

$$f_{LAT} = \frac{M_{LAT}}{S_f} = \frac{12(1.25)(10.29)}{54.00} = 2.86 \text{ ksi}$$

8.2.2 Top Flange Lateral Bending due to Deck Overhang Loads

Assume the deck overhang bracket configuration shown in Figure 13 with the bracket extending to the bottom flange:

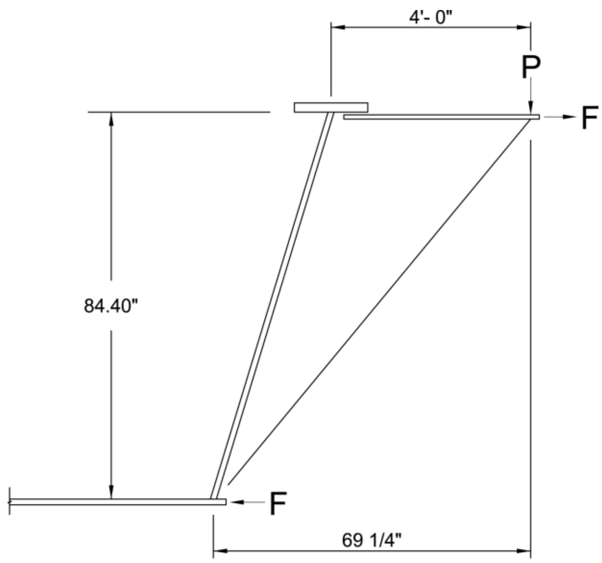

Figure 13 Deck Overhang Bracket Loading

Although the brackets are typically spaced at 3 to 4 feet along the exterior girder, all bracket loads except for the finishing machine load are assumed to be applied uniformly. For this example, the bracket is assumed to extend near the edge of the deck overhang. Therefore, it is assumed that half the deck overhang weight is placed on the exterior girder web and half the weight is placed on the overhang brackets. Conservatively, one-half the deck haunch weight will be included in the total overhang weight. Therefore:

Deck Overhang Weight:

$$P = 0.5*150\left[\left(4-\frac{18/12}{2}\right)\left(\frac{9.5}{12}\right)+\left(\frac{13-1.0}{12}\right)\left(\frac{18/12}{2}\right)+\frac{1}{2}\left(4-\frac{18/12}{2}\right)\left(\frac{(3.5+0.5)}{12}\right)\right] = 290 \text{ lbs/ft}$$

Construction loads, or dead loads and temporary loads that act on the overhang only during construction, are assumed as follows:

Overhang deck forms:	P = 40 lbs/ft
Screed rail:	P = 85 lbs/ft
Railing:	P = 25 lbs/ft
Walkway:	P = 125 lbs/ft
Finishing machine:	P = 3000 lbs

The force imposed by the weight of the finishing machine is estimated as one-half of the total finishing machine truss weight, plus additional load to account for the weight of the engine, drum and operator assumed to be located on one side of the truss.

The lateral force on the top flange, due to the vertical load on the overhang brackets, is computed by summation of the moments about the web-bottom flange junction (Figure 13):

F_{LAT} (84.40) - P(69.25) = 0

F_{LAT} = (0.819) P

In the absence of a more refined analysis, the equations given in Article C6.10.3.4.1 may be used to estimate the maximum flange lateral bending moments in the discretely braced compression flange due to the lateral bracket forces. Assuming the flange is continuous with the adjacent unbraced lengths and that the adjacent unbraced lengths are approximately equal, the lateral bending moment due to a statically equivalent uniformly distributed lateral bracket force may be estimated as:

$$M_\ell = \frac{F_\ell L_b^2}{12} \qquad \text{Eq. (C6.10.3.4.1-1)}$$

The lateral bending moment due to a statically equivalent concentrated lateral bracket force assumed placed at the middle of the unbraced length may be estimated as:

$$M_\ell = \frac{P_\ell L_b}{8} \qquad \text{Eq. (C6.10.3.4.1-2)}$$

In the Strength I load combination, a load factor of 1.5 is applied to all construction loads (Article 3.4.2.1). The lateral bending stress in the bottom flange will be small as compared to the top flange; therefore, bottom flange calculations are not shown for this particular example.

For Strength I:

Dead loads:

$$P = 1.0\left[1.25(290) + 1.5(40 + 85 + 25 + 125)\right] = 775.0 \text{ lbs/ft}$$

$$F = F_\ell = (0.819)P = (0.819)(775.0) = 635 \text{ lbs/ft}$$

$$M_\ell = \frac{F_\ell L_b^2}{12} = \frac{0.635(15.875)^2}{12} = 13.33 \text{ kip} - \text{ft}$$

$$\text{Top flange: } f_\ell = \frac{M_\ell}{S_\ell} = \frac{13.33(12)}{(1.0)(18)^2/6} = 2.96 \text{ ksi}$$

Finishing machine:

$$P = 1.0\left[1.5(3{,}000)\right] = 4{,}500 \text{ lbs}$$

$$F = P_\ell = (0.819)P = (0.819)(4{,}500) = 3{,}686 \text{ lbs}$$

$$M_\ell = \frac{P_\ell L_b}{8} = \frac{3.686(15.875)}{8} = 7.31\,\text{kip} - \text{ft}$$

$$\text{Top flange: } f_\ell = \frac{M_\ell}{S_\ell} = \frac{7.31(12)}{(1.0)(18)^2/6} = 1.62\,\text{ksi}$$

Deck Overhang Total:

$$f_\ell = 2.96 + 1.62 = 4.58 \text{ ksi}$$

8.2.3 Top Flange Lateral Bending Amplification

As specified in Article 6.10.1.6, for design checks where the flexural resistance is based on lateral torsional buckling, the stress, $f\ell$, is to be determined as the largest value of the stress due to lateral bending throughout the unbraced length in the flange under consideration. For design checks where the flexural resistance is based on yielding or flange local buckling, $f\ell$ may be determined as the stress at the section under consideration. For simplicity in this example, the largest value of $f\ell$ within the unbraced length will conservatively be used in all design checks. $f\ell$ is to be taken as positive in sign in all resistance equations. The unbraced length, Lb, for Section 2-2 is equal to 15.875 feet (Figure 2).

According to Article 6.10.1.6, lateral bending stresses determined from a first-order analysis may be used in discretely braced compression flanges for which:

$$L_b \le 1.2 L_p \sqrt{\frac{C_b R_b}{f_{bu}/F_{yc}}} \qquad \text{Eq. (6.10.1.6-2)}$$

L_p is the limiting unbraced length specified in Article 6.10.8.2.3 determined as:

$$L_p = 1.0 r_t \sqrt{\frac{E}{F_{yc}}} \qquad \text{Eq. (6.10.8.2.3-4)}$$

where r_t is the effective radius of gyration for lateral torsional buckling specified in Article 6.10.8.2.3 determined as:

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1 + \frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\right)}} \qquad \text{Eq. (6.10.8.2.3-9)}$$

For the steel section, the depth of the web in compression in the elastic range, D_c, at Section 2-2 is computed along the web as follows:

Note that for the steel section only: $d_{TOP\ OF\ STEEL} = 51.53$ in.

$$D_c = (d_{TOP\ OF\ STEEL} - t_f)\sqrt{\frac{S^2+1}{S^2}}$$

$$D_c = (51.53 - 1.0)\sqrt{\frac{4^2+1}{4^2}}$$

D_c = 52.09in.

It should be noted that values of D_c and D are taken as distances along the web, in accordance with Article 6.11.2.1.1. Therefore,

$$r_t = \frac{18}{\sqrt{12\left(1+\frac{1}{3}\frac{52.09(0.625)}{18(1.0)}\right)}} = 4.10\text{ in.}$$

$$L_p = \frac{1.0(4.10)}{12}\sqrt{\frac{29{,}000}{50}} = 8.23\text{ ft}$$

C_b is the moment gradient modifier specified in Article 6.10.8.2.3, and may conservatively be taken equal to 1.0. According to Article 6.10.1.10.2, the web load-shedding factor, R_b, is to be taken equal to 1.0 when checking constructability. Finally, f_{bu} is the largest value of the factored compressive stress throughout the unbraced length in the flange under consideration, calculated without consideration of flange lateral bending. In this case, use f_{bu} = -33.90 ksi, as computed earlier for the Strength I load combination. Therefore:

$$1.2(8.23)\sqrt{\frac{1.0(1.0)}{|-32.04|/50}} = 12.34\text{ ft} < L_b = 15.875\text{ ft}$$

Because the Equation 6.10.1.6-2 is not satisfied, Article 6.10.1.6 requires that second-order elastic compression-flange lateral bending stresses be determined. The second-order compression-flange lateral bending stresses may be determined by amplifying first-order values (i.e. $f_{\ell 1}$) as follows:

$$f_\ell = \left(\frac{0.85}{1-\frac{f_{bu}}{F_{cr}}}\right) f_{\ell 1} \geq f_{\ell 1} \qquad \text{Eq. (6.10.1.6-4)}$$

or:

$$f_\ell = (AF)f_{\ell 1} \geq f_{\ell 1}$$

where AF is the amplification factor and F_{cr} is the elastic lateral torsional buckling stress for the flange under consideration specified in Article 6.10.8.2.3 determined as:

$$F_{cr} = \frac{C_b R_b \pi^2 E}{\left(\frac{L_b}{r_t}\right)^2} \qquad \text{Eq. (6.10.8.2.3-8)}$$

$$F_{cr} = \frac{1.0(1.0)\pi^2(29{,}000)}{\left(\frac{15.875(12)}{4.10}\right)^2} = 132.6 \text{ ksi}$$

The amplification factor is then determined as follows:

$$AF = \frac{0.85}{\left(1 - \frac{|-32.04|}{132.6}\right)} = 1.12 > 1.0 \quad \text{ok}$$

The above equation for the amplification factor conservatively assumes an elastic effective length factor for lateral torsional buckling equal to 1.0.

Therefore, the total flange stress due to lateral bending, including the amplification factor is:

f_{lat} = (AF)[$(f_{lat})_{WEB\ SHEAR}$ + $(f_{\ell 1})_{OVERHANG}$] = (1.12)[2.86 + 4.58] = 8.33 ksi

Note that first or second-order flange lateral bending stresses, as applicable, are limited to a maximum value of $0.6F_{yf}$ according to Eq. (6.10.1.6-1).

$(0.6)F_{yf}$ = (0.6)(50) = 30 ksi > 8.33ksi ok (Ratio = 0.278)

8.2.4 Flexure (Article 6.11.3.2)

Article 6.11.3.2 directs the engineer to the provisions of Article 6.10.3.2 for the flange constructability checks. Article 6.10.3.2.1 requires that discretely braced flanges in compression satisfy the following requirements, except that for slender-web sections, Eq. (6.10.3.2.1-1) need not be checked when f_ℓ is equal to zero.

$$f_{bu} + f_\ell \leq \varphi_f R_h F_{yc} \qquad \text{Eq. (6.10.3.2.1-1)}$$

$$f_{bu} + \frac{1}{3} f_{\ell} \le \varphi_f F_{nc}$$ Eq. (6.10.3.2.1-2)

$$f_{bu} \le \varphi_f F_{crw}$$ Eq. (6.10.3.2.1-3)

Article 6.11.3.2 requires that the noncomposite tub flange (bottom flange) in tension satisfy:

$$f_{bu} \le \varphi_f R_h F_{yf} \ \Delta$$ Eq. (6.11.3.2-3)

where: ϕ_f = resistance factor for flexure = 1.0 (Article 6.5.4.2)
R_h = hybrid factor specified in Article 6.10.1.10.1 (= 1.0 at homogeneous Section 2-2)
F_{crw} = nominal elastic bend-buckling resistance for webs determined as specified in Article 6.10.1.9
F_{nc} = nominal flexural resistance of the compression flange determined as specified in Article 6.10.8.2 (i.e., local or lateral torsional buckling resistance, as applicable). The provisions of Article A6.3.3 are not to be used to determine the lateral torsional buckling resistance of top flanges of tub girders, per Article 6.11.3.2.
Δ = a factor dependent on the St. Venant torsional shear stress in the bottom flange. St. Venant torsional shear stress will be addressed later in this example.

First, determine if the noncomposite Section 2-2 is a compact or noncompact web section according to Eq. (6.10.6.2.3-1), or alternatively, see Table C6.10.1.10.2-2:

$$\frac{2D_c}{t_w} \le \lambda_{rw}$$ Eq. (6.10.6.2.3-1)

where:

$$4.6\sqrt{\frac{E}{F_{yc}}} \le \lambda_{rw} = \left(3.1 + \frac{5.0}{a_{wc}}\right)\sqrt{\frac{E}{F_{yc}}} \le 5.7\sqrt{\frac{E}{F_{yc}}}$$ Eq. (6.10.6.2.3-3)

$$a_{wc} = \frac{2D_c t_w}{b_{fc} t_{fc}}$$ Eq. (6.10.6.2.3-4)

$$\frac{2D_c}{t_w} = \frac{2(52.09)}{0.625} = 166.7$$

$$4.6\sqrt{\frac{E}{F_{yc}}} = 4.6\sqrt{\frac{29,000}{50}} = 111$$

$$5.7\sqrt{\frac{E}{F_{yc}}} = 5.7\sqrt{\frac{29,000}{50}} = 137$$

$$a_{wc} = \frac{2(52.09)(0.625)}{18(1.0)} = 3.62$$

$$111 > \lambda_{rw} = \left(3.1 + \frac{5.0}{3.62}\right)\sqrt{\frac{29,000}{50}} = 107.9 < 137$$

$$\therefore \lambda_{rw} = 111 < \frac{2D_c}{t_w} = 166.7$$

Therefore, the noncomposite Section 2-2 is a slender-web section. As a result, for the top flanges, Eq. (6.10.3.2.1-1) must be checked since f_ℓ is not zero.

8.2.4.1 Top Flange - Local Buckling Resistance (Article 6.10.8.2.2)

Determine the slenderness ratio of the top flanges:

$$\lambda_f = \frac{b_{fc}}{2t_{fc}} \qquad \text{Eq. (6.10.8.2.2-3)}$$

$$\lambda_f = \frac{18}{2(1.0)} = 9.00$$

Determine the limiting slenderness ratio for a compact flange (alternatively see Table C6.10.8.2.2-1):

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_{yc}}} \qquad \text{Eq. (6.10.8.2.2-4)}$$

$$\lambda_{pf} = 0.38\sqrt{\frac{29,000}{50}} = 9.15$$

Since $\lambda_f < \lambda_{pf}$, the top flanges are compact. Therefore:

$$F_{nc} = R_b R_h F_{yc} \qquad \text{Eq. (6.10.8.2.2-1)}$$

As specified in Article C6.10.3.2.1, when computing F_{nc} for constructability, the web load-shedding factor R_b is to be taken equal to 1.0 because the flange stress is always limited to the web bend-buckling stress according to Eq. (6.10.3.2.1-3) (see Article C6.10.3.2.1). Therefore,

$$F_{nc} = (1.0)(1.0)(50) = 50.00 \text{ ksi}$$

8.2.4.2 Top Flange - Lateral Torsional Buckling Resistance

The limiting unbraced length, L_p, was computed earlier to be 8.23 feet. The effective radius of gyration for lateral torsional buckling, r_t, for the noncomposite Section 2-2 was also computed earlier to be 4.10 inches (Section 8.2.3).

Determine the limiting unbraced length, L_r:

$$L_r = \pi\, r_t \sqrt{\frac{E}{F_{yr}}} \qquad \text{Eq. (6.10.8.2.3-5)}$$

where F_{yr} is the compression flange stress at the onset on nominal yielding, including residual stress effects, and is to be taken as the smaller of $0.7F_{yc}$ and F_{yw}, but not less than $0.5F_{yc}$. Since F_{yc} and F_{yw} are both equal to 50 ksi,

F_{yr} = 0.7(50) = 35 ksi

$$L_r = \frac{\pi\,(4.10)}{12}\sqrt{\frac{29{,}000}{35}} = 30.90 \text{ ft}$$

Since L_p = 8.23feet < L_b = 15.875 feet < L_r = 30.90feet,

$$F_{nc} = C_b\left[1-\left(1-\frac{F_{yr}}{R_h F_{yc}}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right]R_b R_h F_{yc} \le R_b R_h F_{yc} \qquad \text{Eq. (6.10.8.2.3-2)}$$

As discussed previously, the moment-gradient modifier, C_b, is taken equal to 1.0. Therefore,

$$F_{nc} = 1.0\left[1-\left(1-\frac{35.0}{1.0(50)}\right)\left(\frac{15.875-8.23}{30.90-8.23}\right)\right](1.0)(1.0)(50) = 44.94 \text{ ksi} < 1.0(1.0)(50) = 50 \text{ ksi} \quad \text{ok}$$

8.2.4.3 Web Bend-Buckling Resistance (Article 6.10.1.9)

Determine the nominal elastic web bend-buckling resistance at Section 2-2 according to the provisions of Article 6.10.1.9.1 as follows:

$$F_{crw} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} \le \min\left(R_h F_{yc},\ \frac{F_{yw}}{0.7}\right) \qquad \text{Eq. (6.10.1.9.1-1)}$$

where:

$$k = \frac{9}{\left(D_c / D\right)^2}$$ Eq. (6.10.1.9.1-2)

$$k = \frac{9}{\left(52.09/87.0\right)^2} = 25.11$$

Therefore,

$$F_{crw} = \frac{0.9(29{,}000)(25.11)}{\left(\frac{87.0}{0.625}\right)^2} = 33.82 \text{ ksi} < R_h F_{yc} = 50 \text{ ksi} \quad \text{ok}$$

8.2.4.4 Top Flange Constructability Checks

Now that all the required information has been assembled, check the requirements of Article 6.10.3.2.1:

For yielding:

$$f_{bu} + f_\ell \le \varphi_f R_h F_{yc}$$ Eq. (6.10.3.2.1-1)

$$f_{bu} + f_\ell = \left|-32.04\right| \text{ ksi} + 8.33 \text{ ksi} = 40.37 \text{ ksi}$$

$$\phi_f R_h F_{yc} = 1.0(1.0)(50.00) = 50.00 \text{ ksi}$$

$$40.37 \text{ ksi} < 50.00 \text{ ksi} \quad \text{ok} \qquad \left(\text{Ratio} = 0.807\right)$$

$$f_{bu} + \frac{1}{3} f_\ell \le \varphi_f F_{nc}$$ Eq. (6.10.3.2.1-2)

For local buckling:

$$f_{bu} + \frac{1}{3} f_\ell = \left|-32.04\right| \text{ ksi} + \frac{8.33}{3} \text{ ksi} = 34.82 \text{ ksi}$$

$$\phi_f F_{nc} = 1.0(50.00) = 50.00 \text{ ksi}$$

$$34.82 \text{ ksi} < 50.00 \text{ ksi} \quad \text{ok} \qquad (\text{Ratio} = 0.696)$$

For lateral torsional buckling:

$$f_{bu} + \frac{1}{3} f_{\ell} = \left|-32.04\right| \text{ ksi} + \frac{8.33}{3} \text{ ksi} = 34.82 \text{ ksi}$$

$$\phi_f F_{nc} = 1.0(44.94) = 44.94 \text{ ksi}$$

$$34.82 \text{ ksi} < 44.94 \text{ ksi} \quad \text{ok} \qquad (\text{Ratio} = 0.775)$$

For web bend buckling:

$$f_{bu} \le \varphi_f F_{crw} \qquad \text{Eq. (6.10.3.2.1-3)}$$

$$\phi_f F_{crw} = 1.0(33.82) = 33.82 \text{ ksi}$$

$$\left|-32.04\right| \text{ ksi} < 33.82 \text{ ksi} \quad \text{ok} \qquad (\text{Ratio} = 0.947)$$

8.2.4.5 Bottom Flange Constructability Checks

Noncomposite tub flanges (bottom flanges) in tension, must satisfy the following constructability requirement:

$$f_{bu} \le \phi_f R_h F_{yf} \Delta \qquad \text{Eq. (6.11.3.2-3)}$$

where:

$$\Delta = \sqrt{1 - 3\left(\frac{f_v}{F_{yf}}\right)^2} \qquad \text{Eq. (6.11.3.2-4)}$$

The term f_v is the factored St. Venant torsional shear stress in the flange at the section under consideration. However, in accordance with Article C6.11.2.3, if the provisions of Article 6.11.2.3 are satisfied, shear due to St. Venant torsion and secondary distortional bending stress effects may be neglected if the width of the tub flange does not exceed one-fifth the effective span defined in Article 6.11.1.1. For continuous spans, the effective span length is to be taken as the distance between points of permanent load contraflexure, or between a simple support and a point of permanent load contraflexure, as applicable. Therefore, Span 2 has an effective span length of 145 feet. One-fifth of the effective span length is equal to 29 feet, which is much greater than the bottom flange width of 8.208 feet. Therefore, the St. Venant torsional shear stresses can be neglected for this case ($f_v = 0$), and:

$$\Delta = \sqrt{1 - 3\left(\frac{0}{50}\right)^2} = 1.0$$

Consideration of St. Venant torsional shear stresses is illustrated in the horizontally curved tub girder design example NSBA's *Steel Bridge Design Handbook: Design Example 5: Three-Span Continuous Horizontally Curved Composite Steel Tub-Girder Bridge* [7] .

The longitudinal flange stress, calculated previously, is:

$$f_{bu} = 21.57 \text{ ksi}$$

$$\phi_f R_h F_{yf} \Delta = 1.0(1.0)(50)(1.0) = 50.00 \text{ ksi}$$

$$21.57 \text{ ksi} < 50.00 \text{ ksi} \quad \text{ok} \qquad (\text{Ratio} = 0.431)$$

Although the checks are illustrated here for completeness, the bottom flange will typically not control at the positive moment location.

8.2.5 Shear (Article 6.10.3.3)

Article 6.10.3.3 requires that interior panels of stiffened webs satisfy the following requirement:

$$V_u \le \varphi_v V_{cr} \qquad \text{Eq. (6.10.3.3-1)}$$

where: ϕ_v = resistance factor for shear = 1.0 (Article 6.5.4.2)
V_u = shear in the web at the section under consideration due to the factored permanent loads and factored construction loads applied to the noncomposite section
V_{cr} = shear-yield or shear-buckling resistance determined from Eq. (6.10.9.3.3-1)

Only the interior panels of stiffened webs are checked because the shear resistance of the end panel of stiffened webs and the shear resistance of unstiffened webs is already limited to the shear yield- or shear-buckling resistance at the strength limit state.

For this example, the web is unstiffened in the positive flexure regions. Therefore, the constructability check for shear is not required at this section. This check is demonstrated, however, for the stiffened web at Section 2-2.

8.2.6 Concrete Deck (Article 6.10.3.2.4)

Generally, the entire deck is not placed in a single pour. Typically, for continuous span bridges, the positive flexure regions are placed first. Thus, positive flexure regions may become composite prior to casting the other ections of the bridge. As the deck placement operation progresses, tensile stresses can develop in previously cast regions that will exceed the allowable rupture strength (ϕf_r) of the hardened concrete deck. When cracking is predicted, longitudinal deck reinforcing as specified in Article 6.10.1.7 is required to control the cracking. Otherwise, alternative deck casting sequences may be employed to minimize the anticipated stresses to acceptable levels. This check is illustrated in Design Example 1 of the NSBA Steel Bridge Design Handbook.

8.3 Girder Service Limit State Check: Section 2-2 (Positive Moment, Span 2)

Article 6.11.4 directs the Engineer to Article 6.10.4, which contains provisions related to the control of elastic and permanent deformations at the service limit state. For the sake of brevity, only the calculations pertaining to permanent deformations will be presented for this example.

8.3.1 Permanent Deformations (Article 6.10.4.2)

Article 6.10.4.2 contains criteria intended to control permanent deformations that would impair rideability. As specified in Article 6.10.4.2.1, these checks are to be made under the Service II load combination.

Article 6.10.4.2.2 requires that flanges of composite sections satisfy the following requirements:

Top flange of composite sections: $f_f \leq 0.95R_hF_{yf}$ Eq. (6.10.4.2.2-1)

Bottom flange of composite sections: $f_f + \frac{f_\ell}{2} \leq 0.95R_hF_{yf}$ Eq. (6.10.4.2.2-2)

The term f_f is the flange stress at the section under consideration due to the Service II loads calculated without consideration of flange lateral bending. The f_ℓ term, the flange lateral bending stress, in Eq. (6.10.4.2.2-2) is to be taken equal to zero for tub girders, in accordance with Article 6.11.4. A resistance factor is not included in these equations because Article 1.3.2.1 specifies that the resistance factor be taken equal to 1.0 at the service limit state.

With the exception of composite sections in positive flexure in which the web satisfies the requirement of Articles 6.11.2.1.2 (i.e., $D/t_w \leq 150$), web bend-buckling of all sections under the Service II load combination is to be checked as follows:

$f_c \leq F_{crw}$ Eq. (6.10.4.2.2-4)

The term f_c is the compression-flange stress at the section under consideration due to the Service II loads calculated without consideration of flange lateral bending, and F_{crw} is the nominal elastic bend-buckling resistance for webs determined as specified in Article 6.10.1.9. Because Section 2-2 is a composite section subject to positive flexure satisfying Article 6.11.2.1.2, Eq. (6.10.4.2.2-4) need not be checked. An explanation as to why these particular sections are exempt from the above web bend-buckling check is given in Article C6.10.1.9.1.

It should be noted that in accordance with Article 6.11.4, redistribution of negative moment due to the Service II loads at the interior-pier sections in continuous-span flexural members using the procedures specified in Appendix B6 is not to be applied to tub girder sections. The applicability of the Appendix B6 provisions to tub girder sections has not been demonstrated, hence the procedures are not permitted for the design of tub girder sections.

Check the flange stresses due to the Service II loads at Section 2-2. η is specified to always equal 1.0 at the service limit state (Article 1.3.2):

$$0.95R_hF_{yf} = 0.95(1.0)(50) = 47.50 \text{ ksi}$$

Top flange: $f_f \leq 0.95R_hF_{yf}$ Eq. (6.10.4.2.2-1)

$$f_f = 1.0\left[\frac{1.0(10{,}110)}{5{,}044} + \frac{1.0(1{,}594 + 1{,}532)}{14{,}513} + \frac{1.3(9{,}396)}{37{,}515}\right]12 = -30.54\ \text{ksi}$$

$$|-30.54|\ \text{ksi} < 47.50\ \text{ksi} \qquad \text{ok} \qquad (\text{Ratio} = 0.643)$$

Bottom flange: $f_f + \frac{f_\ell}{2} \leq 0.95 R_h F_{yf}$ Eq. (6.10.4.2.2-2)

$$f_f = 1.0\left[\frac{1.0(10{,}110)}{7{,}494} + \frac{1.0(1{,}594 + 1{,}532)}{9{,}414} + \frac{1.3(9{,}396)}{10{,}285}\right]12 + \frac{0}{2} = 34.43\ \text{ksi}$$

$$34.43\ \text{ksi} < 47.50\ \text{ksi} \qquad \text{ok} \qquad (\text{Ratio} = 0.725)$$

8.4 Girder Fatigue and Fracture Limit State Check: Section 2-2 (Span 2)

8.4.1 Fatigue (Article 6.10.5)

Article 6.11.5 directs the Engineer to Article 6.10.5, where details on tub girder section flexural members must be investigated for fatigue as specified in Article 6.6.1. Either the Fatigue I or Fatigue II load combination specified in Table 3.4.1-1, along with the fatigue live load specified in Article 3.6.1.4, are to be employed for checking load-induced fatigue in tub girder sections. The Fatigue I load combination is to be used in combination with design checks for infinite fatigue life. The Fatigue II load combination is to be used in combination with design checks for finite fatigue life.

One additional fatigue limit state requirement specified for certain tub-girder sections is related to longitudinal warping and transverse bending stresses due to cross-section distortion. When tub girders are subjected to eccentric loads (i.e., torsion), their cross-sections become distorted, resulting in secondary bending stresses. Loading the opposite side of the bridge will produce a stress reversal, and possible fatigue concerns for sections subjected to significant torsion. Therefore, according to Article 6.11.5, longitudinal warping stresses and transverse bending stresses due to cross-section distortion are to be considered for:

- Single tub girders in straight or horizontally curved bridges;
- Multiple tub girders in straight bridges that do not satisfy requirements of Article 6.11.2.3;
- Multiple tub girders in horizontally curved bridges; or
- Any single or multiple tub girder with a bottom flange that is not fully effective according to the provisions of Article 6.11.1.1.

In cases where these stresses are to be considered, the stress range due to longitudinal warping resulting from cross-section distortion is to be considered in checking the fatigue resistance of the base metal at all details in the tub girder according to the provisions of Article 6.6.1. Longitudinal warping stresses are considered additive to the longitudinal major-axis bending stresses.

The transverse bending stress range (i.e., the out-of-plane bending stress range due to box cross-section distortion) is to be considered separately in evaluating the fatigue resistance of the base metal adjacent to flange-to-web fillet welds and adjacent to the termination of fillet welds connecting transverse elements to the webs and tub flanges. Possible steps for reducing the transverse bending stress range, where necessary, are discussed further in Article C6.11.5. Where force effects in the cross-frames or diaphragms are computed from a refined analysis, stress ranges for checking load-induced fatigue and torque ranges for checking fatigue due to cross-section distortion in cross-frame and diaphragm members, the single fatigue truck should be positioned as specified in Article 3.6.1.4.3a, with the truck confined to a single transverse position during each passage of the truck along the bridge (per Article C6.6.1.2.1). Transverse bending and longitudinal warping stress ranges due to cross-section distortion can be determined using the Beam-on-Elastic Foundation (BEF) analogy, as discussed in Article C6.11.1.1. These calculations are illustrated in NSBA's *Steel Bridge Design Handbook: Design Example 5: Three-Span Continuous Horizontally Curved Composite Steel Tub-Girder Bridge* [7].

The tub girders in this design example do not fall under any of the categories listed above; hence, longitudinal warping and transverse bending stresses need not be considered.

In addition to checking fatigue of the base metal at the transverse element welded connections, there is a special fatigue requirement for the tub girder webs, with transverse stiffeners, that must be satisfied in accordance with Article 6.10.5.3. The satisfaction of Article 6.10.5.3 is intended to eliminate significant elastic flexing of the web due to shear, such that the member is assumed able to sustain an infinite number of smaller loadings without fatigue cracking due to this effect. For Article 6.10.5.3, the factored fatigue load is to be taken as the Fatigue I load combination specified in Table 3.4.1-1, with the fatigue live load taken as specified in Article 3.6.1.4. For sections with inclined webs, the factored shear is to be determined using Eq. (6.11.9-1). This check is not illustrated in this design example; refer to NSBA's *Steel Bridge Design Handbook: Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge* [3] for an illustration of this check.

8.4.1.1 Fatigue in Bottom Flange

At Section 2-2, it is necessary to check the base metal at the interior cross-frame connection plate welds to the bottom flange of the tub girder for fatigue. This detail corresponds to Condition 4.1 in Table 6.6.1.2.3-1 and is classified as a Category C′ fatigue detail. Only the bottom flange is checked herein, as a net tensile stress is not induced in the top flange by the fatigue loading at this location.

According to Eq. (6.6.1.2.2-1), the factored fatigue stress range, $\gamma(\Delta f)$, must not exceed the nominal fatigue resistance, $(\Delta F)_n$. In accordance with Article C6.6.1.2.2, the resistance factor, ϕ, and the load modifier, η, are taken as 1.0 for the fatigue limit state.

$$\gamma(\Delta f) \leq (\Delta F)_n \qquad \text{Eq. (6.6.1.2.2-1)}$$

From Table 6.6.1.2.3-2, the 75-year $(ADTT)_{SL}$ equivalent to infinite fatigue life for a Category C′ fatigue detail is 975 trucks per day. For the fatigue design in this design example, the Average Daily Truck Traffic (ADTT) in one direction, considering the expected growth in traffic volume over the 75-year fatigue design life, is assumed to be 2,000 trucks/day. Since the $(ADTT)_{SL}$ =

2,000 trucks per day x 0.8 = 1,600 trucks per day (refer to Article 3.6.1.4.2) exceeds the limit of 975 trucks per day equivalent to infinite life, the detail must be checked for infinite fatigue life using the Fatigue I load combination. Per Article 6.6.1.2.5, the nominal fatigue resistance for infinite fatigue life is equal to the constant-amplitude fatigue threshold:

$$(\Delta F)_n = (\Delta F)_{TH} \qquad \text{Eq. (6.6.1.2.5-1)}$$

where $(\Delta F)_{TH}$ is the constant-amplitude fatigue threshold and is taken from Table 6.6.1.2.5-3. For a Category C′ fatigue detail, $(\Delta F)_{TH}$ = 12.0 ksi, and therefore:

$$(\Delta F)_n = 12.0 \text{ ksi}$$

As shown in Figure 8, the unfactored negative and positive moments due to fatigue at Section 2-2 are -396 kip-ft and 2,558 kip-ft, respectively. As shown in Table 4, the short-term composite section properties (*n* = 8) used to compute the stress at the bottom of the web (top of the bottom flange, where the weld in question is located) are:

$I_{NA(n)}$ = 695,903 in.^{4}

$d_{BOT\ OF\ WEB} = d_{BOT\ OF\ STEEL} - t_{f\_BOT\ FLANGE}$ = 67.66 in. – 0.8125 in. = 66.85 in.

Therefore, the unfactored stress range at the bottom of the web due to vertical loads only is:

$$f_{range\_vert} = \left(\frac{\left(|-396| + 2,558\right)(12)(66.85)}{695,903} \right) = 3.41 \text{ ksi}$$

Per Table 3.4.1-1, the load factor, γ, for the Fatigue I load combination is 1.75. The total factored stress range at the edge of the connection is therefore:

$$\gamma(\Delta f) = (1.75)(3.41) = 5.97 \text{ ksi}$$

Checking Eq. (6.6.1.2.2-1),

$$\gamma(\Delta f) = 5.97 \text{ ksi} < (\Delta F) = 12.00 \text{ ksi} \quad \text{OK} \quad (\text{Ratio} = 0.498)$$

8.4.2 Fracture (Article 6.6.2)

As specified in Article 6.10.5.2, fracture toughness requirements in the contract drawings must be in conformance with the provisions of Article 6.6.2.1. Material for main load-carrying components subject to tensile stress under the Strength I load combination is assumed for this example to be ordered to meet the appropriate Charpy V-notch fracture toughness requirements (Table C6.6.2.1-1) specified for Temperature Zone 2 (Table 6.6.2.1-2).

Article 6.6.2.2 provides provisions for Fracture-Critical Members (FCMs). A FCM is defined as a steel primary member or portion thereof subject to tension whose failure would probably cause a portion of or the entire bridge to collapse. Article 6.6.2.2 specifies that the Engineer is to have the responsibility for identifying and designating on the contract plans which primary members or portions thereof are fracture-critical members (FCMs). The tension components of tub girders in single- and twin-tub girder systems have typically been designated as FCMs.

The designation of a particular member, or member component, as a FCM entails additional and more stringent fabrication requirements given in Clause 12 of the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code* (D1.5) [15], and hands-on inspections every two years. The additional fabrication requirements are an initial cost premium in the design of new bridges that has been proven to be effective in preventing fracture. However, the hands-on inspection requirements give rise to considerably larger expenses that take place throughout the service life of the bridge, which involve risks to the safety of the inspectors and bridge users.

Article 6.6.2.2 further indicates that a primary member or portion thereof subject to tension, for which the redundancy is not known by engineering judgment, but which is demonstrated to have redundancy in the presence of a simulated fracture in that member through the use of a refined analysis, is to be designated as a System Redundant Member (SRM) in the contract documents. SRMs are to be fabricated in accordance with Clause 12 of D1.5 and are to have routine inspections performed but need not be subject to the hands-on in-service inspection requirements.

One acceptable detailed finite element analysis and evaluation procedure for classification of SRMs [16] is provided in the AASHTO *Guide Specifications for Analysis and Identification of Fracture Critical Members and System Redundant Members* [17]. The Guide Specification is intended to provide Engineers and Owners with an analytical framework to evaluate the redundancy of typical steel bridges and designate primary steel members as FCMs or SRMs. This framework is composed of the finite element analysis procedure, techniques, and inputs needed to create a reliable model of the steel bridge; as well as the minimum required primary steel member failure scenarios, load combinations, and performance criteria used to evaluate the redundancy of a steel bridge. Connor et al. (2020) [18] also provides a suggested alternative simplified approach for classifying SRMs in continuous composite twin tub-girder bridges.

8.5 Girder Strength Limit State Check: Section 2-2 (Span 2)

8.5.1 Flexure (Article 6.11.6.2)

Determine if Section 2-2 qualifies as a compact section. According to Article 6.11.6.2.2, composite sections in positive flexure qualify as compact when:

1) the specified minimum yield strengths of the flanges and web do not exceed 70 ksi;

2) the web satisfies the requirement of Article 6.11.2.1.2 such that longitudinal stiffeners are not required (i.e., $D/t_w \leq 150$);

3) the section is part of a bridge that satisfies the requirements of Article 6.11.2.3 (i.e., the special restrictions for use of the live load distribution factors – Section 5.3);

4) the bottom flange is fully effective as specified in Article 6.11.1.1 (i.e., the bottom flange b_f is less than one-fifth of the effective span); and

5) the section satisfies the following web-slenderness limit:

$$\frac{2D_{cp}}{t_w} \leq 3.76\sqrt{\frac{E}{F_{yc}}} \qquad \text{Eq. (6.11.6.2.2-1)}$$

where D_{cp} is the depth of the web in compression at the plastic moment determined as specified in Article D6.3.2.

Earlier computations indicated that the plastic neutral axis of the composite section is located in the top flange. Therefore, according to Article D6.3.2, D_{cp} is taken equal to zero for this case and Eq. (6.11.6.2.2-1) is satisfied. Section 2-2 qualifies as a compact section.

Compact sections must satisfy the following ductility requirement specified in Article 6.10.7.3 to protect the concrete deck from premature crushing:

$$D_p \leq 0.42D_t \qquad \text{Eq. (6.10.7.3-1)}$$

where D_p is the distance from the top of the concrete deck to the neutral axis of the composite section at the plastic moment, and D_t is the total depth of the composite section. At Section 2-2:

$$D_p = 9.0 + 3.5 - 1.0 + 0.93 = 12.43 \text{ in.}$$

$$D_t = 0.8125 + 84.4 + 3.5 + 9.0 = 97.71 \text{ in.}$$

$$0.42D_t = 0.42(97.71) = 41.04 \text{ in.} > 12.43 \text{ in.} \quad \text{ok} \quad \text{(Ratio = 0.303)}$$

At the strength limit state, compact composite sections in positive flexure must satisfy the provisions of Article 6.11.7.1. Specifically, the nominal flexural resistance shall satisfy:

$$M_u \leq \varphi_f M_n \qquad \text{Eq. (6.11.7.1.1-1)}$$

where: ϕ_f = resistance factor for flexure = 1.0 (Article 6.5.4.2)
M_n = nominal flexural resistance of the section determined as specified in Article 6.11.7.1.2
M_u = factored bending moment about the major-axis of the cross-section

8.5.1.1 Nominal Flexural Resistance (Article 6.11.7.1.2)

The nominal flexural resistance of the section is to be taken as specified in Article 6.10.7.1.2, except that for continuous spans, the nominal flexural resistance is always to be subject to the limitation of Eq. (6.10.7.1.2-3) (see below). According to the provisions of Article 6.10.7.1.2, the nominal flexural resistance of compact composite sections in positive flexure is determined as follows:

If $D_p \leq 0.1D_t$, then: $$M_n = M_p \qquad \text{Eq. (6.10.7.1.2-1)}$$

Otherwise: $$M_n = M_p\left(1.07 - 0.7\frac{D_p}{D_t}\right) \qquad \text{Eq. (6.10.7.1.2-2)}$$

where M_p is the plastic moment of the composite section determined as specified in Article D6.1.

In continuous spans, the nominal flexural resistance of the section is also limited to the following:

$$M_n = 1.3R_hM_y \qquad \text{Eq. (6.10.7.1.2-3)}$$

where M_y is the yield moment of the composite section determined as specified in Article D6.2.

For Section 2-2, M_y and M_p were computed earlier to be 37,751 kip-ft and 52,745 kip-ft, respectively.

$$0.1D_t = 0.1(97.71) = 9.77\text{ in.} < D_p = 12.43\text{ in.}$$

Therefore,

$$M_n = 52{,}745\left[1.07 - 0.7\left(\frac{12.43}{97.71}\right)\right] = 51{,}740\text{ kip} - \text{ft}$$

Or,

$$M_n = 1.3(1.0)(37{,}751) = 49{,}076\text{ kip} - \text{ft} \qquad \text{(governs)}$$

Therefore: M_n = 49,076 kip-ft

For Strength I:

$$M_u = 1.25(10{,}110 + 1{,}594) + 1.5(1{,}532) + 1.75(9{,}396) = 33{,}371\text{ kip} - \text{ft}$$
$$\phi_f M_n = 1.0(49{,}076) = 49{,}076\text{ kip} - \text{ft}$$
$$33{,}371\text{ kip} - \text{ft} < 49{,}076\text{ kip} - \text{ft} \quad \text{ok} \qquad (\text{Ratio} = 0.680)$$

8.5.1.2 Shear (Article 6.11.6.3)

Article 6.11.6.3 invokes to the provisions of Article 6.11.9 to determine the shear at the Strength Limit State. Article 6.11.9 further directs the Engineer to the provisions of Article 6.10.9 for determining the factored shear resistance of a single web. For the case of inclined webs, D in Article 6.10.9 is taken as the depth of the web measured along the slope. Inclined webs are to be designed to resist a shear force taken as:

$$V_{ui} = \frac{V_u}{\cos(\theta)} \qquad \text{Eq. (6.11.9-1)}$$

where V_u is the shear due to factored loads on one inclined web, and θ is the angle of inclination of the web plate.

At the strength limit state, webs must satisfy the following:

$$V_u \le \varphi_v V_n \qquad \text{Eq. (6.10.9.1-1)}$$

where: ϕ_v = resistance factor for shear = 1.0 (Article 6.5.4.2)
V_n = nominal shear resistance determined as specified in Articles 6.10.9.2 and 6.10.9.3 for unstiffened and stiffened webs, respectively
V_u = V_{ui} = factored shear in a single web at the section under consideration

A flow chart for determining the shear resistance of I-sections is shown in Figure C6.10.9.1-1. Design Example 1 in the NSBA Steel Bridge Design Handbook presents a complete evaluation of shear requirements and design of an I-girder section. The shear design for tub girders, other than that previously presented, follows the same procedure as presented in the Design Example 1. Therefore, this example will limit discussion to checking on the Strength I Limit State at the girder end (abutment location). The η factor is again taken equal to 1.0 in this example at the strength limit state. The unfactored dead load and live load shears are as follows, where the live load shears are taken as the shear envelope values.

$$V_{DC1} = (224)\ /\ 2 = 112 \text{ kips / web}$$

$$V_{DC2} = (31)\ /\ 2 = 15.5 \text{ kips / web}$$

$$V_{DW} = (30)\ /\ 2 = 15 \text{ kips / web}$$

$$V_{LL+I} = (215)\ /\ 2 = 107.5 \text{ kips / web}$$

A sample calculation of V_{ui}, for a single web, at the abutment is given below:

$$V_{ui} = \frac{1.0\left[1.25(112+15.5)+1.5(15)+1.75(107.5)\right]}{\cos\left(\arctan\left(\frac{1}{4}\right)\right)} = 381 \text{ kips}$$

The need for and required spacing of transverse stiffeners at this location will now be determined. First, determine the nominal shear resistance of an unstiffened web according to the provisions of Article 6.10.9.2. According to Article 6.10.9.2, the nominal shear resistance of an unstiffened web is limited to the shear-yield or shear-buckling resistance, V_{cr}, determined as:

$$V_n = V_{cr} = CV_p \qquad \text{Eq. (6.10.9.2-1)}$$

C is the ratio of the shear-buckling resistance to the shear yield strength determined as specified in Article 6.10.9.3.2 with the shear-buckling coefficient, k, taken equal to 5.0 since the unstiffened web shear capacity is being calculated.

Since,

$$1.40\sqrt{\frac{Ek}{F_{yw}}} = 1.40\sqrt{\frac{29{,}000(5.00)}{50}} = 75.4 < \frac{D}{t_w} = \frac{87.0}{0.625} = 139.2$$

$$C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2}\left(\frac{Ek}{F_{yw}}\right) \qquad \text{Eq. (6.10.9.3.2-6)}$$

$$C = \frac{1.57}{(139.2)^2}\left(\frac{29{,}000(5.00)}{50}\right) = 0.235$$

V_p is the plastic shear force determined as follows:

$$V_p = 0.58F_{yw}Dt_w \qquad \text{Eq. (6.10.9.2-2)}$$

$$V_p = 0.58(50)(87.0)(0.625) = 1{,}577 \text{ kips}$$

Therefore,

$$V_n = V_{cr} = 0.235(1{,}577) = 370 \text{ kips}$$

$$\varphi_v V_n = 1.0(370) = 370 \text{ kips}$$

The value of V_{ui} at the end bearing is 381 kips which exceeds the nominal shear resistance of an unstiffened web, $\phi_v V_n$ = 370 kips. Therefore, transverse stiffeners are required and the provisions of Article 6.10.9.3 apply.

8.5.1.3 End Panel Shear (Article 6.10.9.3.3)

An end panel is defined as a web panel adjacent to the discontinuous end of a girder. According to Article 6.10.9.3.3, the nominal shear resistance of a web end panel is limited to the shear-yield or shear-buckling resistance, V_{cr}, determined as:

$$V_n = V_{cr} = CV_p \qquad \text{Eq. (6.10.9.3.3-1)}$$

C is the ratio of the shear-yield or shear-buckling buckling resistance to the shear yield strength determined as specified in Article 6.10.9.3.2. First, compute the shear buckling coefficient, k. According to Article 6.10.9.3.3, the transverse stiffener spacing for end panels is not to exceed 1.5D = 1.5(87.0) = 130.5 inches. Assume the spacing from the abutment to the first transverse stiffener is d_o = 10.75 feet = 129.0 inches.

$$k = 5 + \frac{5}{\left(\frac{129.0}{87.0}\right)^2} = 7.27 \qquad \text{Eq. (6.10.9.3.2-7)}$$

Since,

$$1.40\sqrt{\frac{Ek}{F_{yw}}} = 1.40\sqrt{\frac{29{,}000(7.27)}{50}} = 90.91 < \frac{D}{t_w} = \frac{87.0}{0.625} = 139.2$$

$$C = \frac{1.57}{(139.2)^2}\left(\frac{29{,}000(7.27)}{50}\right) = 0.342 \qquad \text{Eq. (6.10.9.3.2-6)}$$

$$V_p = 0.58F_{yw}Dt_w \qquad \text{Eq. (6.10.9.3.3-2)}$$

V_p is the plastic shear force, calculated as follows:

$$V_p = 0.58(50)(87.0)(0.625) = 1{,}577 \text{ kips}$$

Therefore,

$$V_n = V_{cr} = 0.342(1{,}577) = 539 \text{ kips} \qquad \text{Eq. (6.10.9.3.3-1)}$$

$$\phi_v V_n = 1.0(539) = 539 \text{ kips} > V_u = 381 \text{ kips} \quad \text{ok} \quad (\text{Ratio} = 0.707)$$

8.5.1.4 Interior Panel Shear (Article 6.10.9.3.2)

Additional web stiffeners are not required beyond the end panel in the positive moment region. The Strength I factored shear in one web at 10.75 feet from the abutment is 327 kips (i.e. V_{ui} = 327 kips). Since the factored shear, V_{ui}, is less than the unstiffened web shear capacity, $\phi_v V_n$ = 370 kips, no additional transverse stiffeners are required and Article 6.11.6.3 is satisfied through the remainder of the positive flexure region.

8.6 Girder Constructability Check: Section 2-1 (Interior Pier Location)

8.6.1 Flexure (Article 6.11.3.2)

The bottom flange in regions of negative flexure is to satisfy the requirements of Eqs. (6.11.3.2-1) and (6.11.3.2-2) for critical stages of construction. Generally, these provisions will not control because the size of the bottom flange in negative flexure regions is normally governed by the strength limit state. The maximum negative moment reached during the deck-placement analysis, plus the moment due to the self-weight, typically do not differ significantly from the calculated DC_1 negative moments assuming a single stage deck placement.

$$f_{bu} \le \varphi_f F_{nc} \qquad \text{Eq. (6.11.3.2-1)}$$

$$f_{bu} \le \varphi_f F_{crw} \qquad \text{Eq. (6.11.3.2-2)}$$

Additionally, the top flanges, which are discretely braced during construction, must satisfy the requirement specified in Article 6.10.3.2.2.

$$f_{bu} + f_{\ell} \le \varphi_f R_h F_{yt} \qquad \text{Eq. (6.10.3.2.2-1)}$$

As stated previously, the deck-placement sequence and the application of wind loads are not considered in this example. It is assumed, for this example that the placement of the concrete deck occurs in a single stage for the purpose of the constructability checks.

Calculate the factored maximum flexural stresses in the flanges of the steel section resulting from the application of steel self-weight and the assumed full deck-placement (DC_1).

For Strength I:

Top flange: $$(f_{bu})_{DC1} = \frac{1.0(1.25)(27{,}012)(12)}{15{,}332} = 26.43 \text{ ksi}$$

Bot. flange: $$(f_{bu})_{DC1} = \frac{1.0(1.25)(27{,}012)(12)}{16{,}709} = -24.25 \text{ ksi}$$

In addition to the applied steel and concrete self-weight loads, it is prudent to assume a construction live loading (CLL) on the structure during placement of the concrete deck, as discussed Section 6.2.3. In the Strength I load combination, a load factor of 1.5 is applied to all construction loads in accordance with Article 3.4.2. Therefore, For Strength I:

$$\text{Top flange:} \quad (f_{bu})_{CLL} = \frac{1.0(1.5)(1{,}478)(12)}{15{,}332} = 1.74 \text{ ksi}$$

$$\text{Bot. flange:} \quad (f_{bu})_{CLL} = \frac{1.0(1.5)(1{,}478)(12)}{16{,}709} = -1.59 \text{ ksi}$$

$$\text{Top flange:} \quad f_{bu} = 26.43 + 1.74 = 28.17 \text{ ksi}$$
$$\text{Bot. flange:} \quad f_{bu} = -24.25 + (-1.59) = -25.84 \text{ ksi}$$

Although not included in this example in the interest of brevity, the special load combination specified in Article 3.4.2.1 must also be considered in the design checks for the deck placement sequence (see Section 6.3).

8.6.1.1 Top Flange Stress due to Lateral Bending

The change in the horizontal component of the web shear in the inclined web along the span acts as a lateral force in the flanges of the tub girder, which in turn results in a top flange bending stress. In addition, the deck overhang bracket will impose lateral forces on the top flange, causing lateral top flange bending stress. Computation of the lateral bending stress is performed in the same manner as demonstrated previously for Section 2-2. For the sake of brevity, the calculations will not be shown, but instead will be summarized.

For Strength I:

f_{lat} due to horizontal component of web shear: f_{lat} = 0.47 ksi

f_{lat} due to cantilever deck overhang bracket: f_{lat} = 0.72 ksi

Total top flange f_{lat} = 0.47 + 0.72 = 1.19 ksi

8.6.1.2 Top Flange Constructability Check

Checking compliance with Article 6.10.3.2.2:

$$f_{bu} + f_{\ell} \le \varphi_f R_h F_{yt} \qquad \text{Eq. (6.10.3.2.2-1)}$$

For Strength I:

$$f_{bu} + f_{\ell} = 28.17 \text{ ksi} + 1.19 \text{ ksi} = 29.36 \text{ ksi}$$
$$\phi_f R_h F_{yc} = 1.0(1.0)(50) = 50.0 \text{ ksi}$$
$$29.36 \text{ ksi} < 50.0 \text{ ksi} \quad \text{ok} \qquad (\text{Ratio} = 0.587)$$

8.6.1.3 Bottom Flange - Flexural Resistance in Compression - Stiffened Flange (Article 6.11.8.2.3)

Calculate the nominal flexural resistance of the bottom flange in compression, F_{nc}, in accordance with Article 6.11.8.2. In computing F_{nc} for constructability, the web load-shedding factor, R_b, is to be taken as 1.0. The bottom flange is longitudinally stiffened at this location with a single WT12 x 42, placed at the center of the bottom flange, with the stem of the WT welded to the girder bottom flange. Therefore, Article 6.11.8.2.3 applies.

As specified in Article 6.11.11.2, longitudinal compression flange stiffeners on tub girder bottom flanges are to be equally spaced across the width of the flange. Furthermore, the yield strength of the longitudinal stiffeners must not be less than the yield strength of the flanges to which they are attached.

The projecting width, b_l, of the longitudinal flange stiffener must satisfy Eq. (6.11.11.2-1):

$$b_\ell \leq 0.48\, t_s\sqrt{\frac{E}{F_{yc}}} \qquad \text{Eq. (6.11.11.2-1)}$$

where:

t_s = thickness of the projecting longitudinal stiffener element (in.)

In the case of a structural tee, t_s is taken as the flange thickness of the structural tee since each half-flange would buckle similarly to a single plate connected to the web. Furthermore, the projecting width, b_l, of structural tees is to be taken as one-half the width of the tee flange. Therefore,

$$b_l \leq 0.48(0.770)\sqrt{\frac{29{,}000}{50}} = 8.90 \text{ in.}$$

$$b_l = \frac{9.02}{2} = 4.51 \text{ in.} < 8.90 \text{ in.}$$ WT 12x42 flange is OK

Determine the slenderness ratio of the bottom flange:

$$\lambda_f = \frac{b_{fc}}{t_{fc}} \qquad \text{Eq. (6.11.8.2.2-8)}$$

where:

b_{fc} = w = larger of the width of the flange between longitudinal flange stiffeners or the distance from a web to the nearest longitudinal flange stiffener.

In this case, since the longitudinal stiffener is at the center of the bottom flange, *w* is the distance from the longitudinal stiffener to the inside face of the web.

$$\lambda_f = \frac{(95.125)/2}{1.75} = 27.18$$

Since a single bottom flange stiffener is used, *n* = 1 and,

$$k = \left(\frac{8I_S}{wt_{fc}^{\,3}}\right)^{\frac{1}{3}} \qquad \text{Eq. (6.11.8.2.3-1)}$$

and,

$$k_S = \frac{5.34 + 2.84\left(\frac{I_S}{wt_{fc}^{\,3}}\right)^{\frac{1}{3}}}{\left(n+1\right)^2} \le 5.34 \qquad \text{Eq. (6.11.8.2.3-3)}$$

where: f_v = St. Venant torsional shear stress in the flange due to factored loads
n = number of equally spaced longitudinal flange stiffeners
k = plate buckling coefficient for uniform normal stress, $1.0 \le k \le 4.0$
k_S = plate buckling coefficient for shear stress
I_S = moment of inertia of a single longitudinal flange stiffener about an axis parallel to the flange and taken at the base of the stiffener

As specified in Article C6.11.11.2, the actual longitudinal flange stiffener moment of inertia, I_s, used in determining the plate-buckling coefficient for uniform normal stress, k, from either Eq. 6.11.8.2.3-1 or Eq. 6.11.8.2.3-2, as applicable, automatically satisfies Eq. 6.11.11.2-2. Alternatively, for preliminary sizing of the stiffener for example, a value of k can be assumed in lieu of using Eq. 6.11.8.2.3-1 or Eq. 6.11.8.2.3-2, as applicable, but a range of 2.0 to 4.0 should generally apply.

Structural tees are efficient shapes for longitudinal stiffeners because they provide a high ratio of stiffness to cross-sectional area. For the WT12x42 stiffener:

$$I_s = 166 + 12.4\left(9.13\right)^2 = 1{,}200 \text{ in.}^4$$

Therefore,

$$k = \left(\frac{8(1{,}200)}{(47.56)(1.75)^3}\right)^{\frac{1}{3}} = 3.35 < 4.0$$

$$k_S = \frac{5.34 + 2.84\left(\frac{1{,}200}{(47.56)(1.75)^3}\right)^{\frac{1}{3}}}{(1+1)^2} = 2.52 < 5.34$$

Calculate λ_p,

$$\lambda_p = 0.57\sqrt{\frac{Ek}{F_{yc}\Delta}} \qquad \text{Eq. (6.11.8.2.2-9)}$$

where:

$$\Delta = \sqrt{1 - 3\left(\frac{f_v}{F_{yf}}\right)^2} \qquad \text{Eq. (6.11.8.2.2-11)}$$

As stated previously, the St. Venant torsional shear stress, f_v, can be assumed to be zero because the bottom flange width does not exceed one-fifth of the effective span length, and all other requirements of Article 6.11.2.3 are satisfied (see Article C6.11.2.3).

Therefore, since f_v is zero:

$$\Delta = \sqrt{1 - 3\left(\frac{0}{50}\right)^2} = 1.0$$

$$\lambda_p = 0.57\sqrt{\frac{(29000)(3.35)}{(50)(1.0)}} = 25.13 < \lambda_f = 27.18$$

Since λ_f exceeds λ_p, it is necessary to calculate λ_r:

$$\lambda_r = 0.95\sqrt{\frac{Ek}{F_{yr}}} \qquad \text{Eq. (6.11.8.2.2-10)}$$

and where:

$$F_{yr} = (\Delta - 0.4)\, F_{yc} \le F_{yw} \qquad \text{Eq. (6.11.8.2.2-7)}$$

$$F_{yr} = (1.0 - 0.4)(50) = 30 \text{ ksi} < 50 \text{ ksi}$$

Therefore, λ_r is calculated as:

$$\lambda_r = 0.95\sqrt{\frac{(29000)(3.35)}{(30)}} = 54.06 > \lambda_f = 27.18$$

Since $\lambda_p < \lambda_f < \lambda_r$,

$$F_{cb} = R_b R_h F_{yc}\left[\Delta - \left(\Delta - \frac{\Delta\text{-}0.3}{R_h}\right)\left(\frac{\lambda_f - \lambda_p}{\lambda_r - \lambda_p}\right)\right] \qquad \text{Eq. (6.11.8.2.2-3)}$$

$$F_{cb} = (1.0)(1.0)(50)\left[1.0 - \left(1.0 - \frac{1.0 - 0.3}{1.0}\right)\left(\frac{27.18 - 25.13}{54.06 - 25.13}\right)\right]$$

F_{cb} = 48.94 ksi

The nominal flexural resistance of the compression flange, F_{nc}, is calculated as:

$$F_{nc} = F_{cb}\sqrt{1 - \left(\frac{f_v}{\varphi_v\ F_{cv}}\right)^2} \qquad \text{Eq. (6.11.8.2.2-1)}$$

Since f_v =0 ksi, $F_{nc} = F_{cb}$ = 48.94 ksi.

For Strength I:

f_{bu} = -25.84 ksi
$\phi_f\ F_{nc}$ = 1.0(48.94) = 48.94 ksi
|-25.84| ksi < 48.94 ksi ok (Ratio 0.528)

8.6.1.4 Web Bend-Buckling (Article 6.10.1.9)

The web bend-buckling resistance is to be compared with the maximum compressive stress in the bottom flange. Determine the nominal elastic web bend-buckling resistance at Section 2-2 according to the provisions of Article 6.10.1.9.1 as follows:

$$F_{crw} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} \le \min\left(R_h F_{yc},\ \frac{F_{yw}}{0.7}\right) \qquad \text{Eq. (6.10.1.9.1-1)}$$

where:

$$k = \frac{9}{\left(D_c / D\right)^2} \qquad \text{Eq. (6.10.1.9.1-2)}$$

$$D_c = 42.54 \text{ in.} - 1.75 \text{ in.} = 40.79 \text{ in.}$$

Compute D_c along the inclined web:

$$D_c = (40.79)\sqrt{\frac{4^2 + 1}{4^2}} = 42.05 \text{ in.}$$

$$k = \frac{9}{\left(42.05/87.0\right)^2} = 38.53$$

Therefore,

$$F_{crw} = \frac{0.9(29{,}000)(38.53)}{\left(\frac{87.0}{0.625}\right)^2} = 51.90 \text{ ksi} > R_h F_{yc} = 50 \text{ ksi}$$

F_{crw} = 50 ksi

For Strength I:

$$f_{bu} = -25.84 \text{ ksi}$$

$$\phi_f F_{crw} = 1.0(50) = 50 \text{ ksi}$$

$$\left|-25.84\right| \text{ ksi} < 50 \text{ ksi} \quad \text{ok} \qquad \left(\text{Ratio} = 0.517\right)$$

8.6.2 Shear (Article 6.11.3.3)

Article 6.10.3.3 requires that interior panels of stiffened webs satisfy the following requirement:

$$V_u \le \varphi_v V_{cr} \qquad \text{Eq. (6.10.3.3-1)}$$

where: ϕ_v = resistance factor for shear = 1.0 (Article 6.5.4.2)
V_u = shear in the web at the section under consideration due to the factored permanent loads and factored construction loads applied to the noncomposite section
V_{cr} = shear-buckling resistance determined from Eq. (6.10.9.3.3-1)

In this example, the panel adjacent to Section 2-1 will be checked. The transverse stiffener spacing in this panel is d_o = 17.75 feet (Figure 3). The total factored shear will include the contribution of noncomposite dead load (DC_1) and the construction live loading (CLL). Note that the shear used in the following calculation is based on a single web.

For Strength I:

$$V_u = 1.0(1.25)(-270) + 1.0(1.5)(-15) = -360 \text{ kips}$$

However, it is required that the shear be taken along the inclined web in accordance with Article 6.11.9:

$$V_{ui} = \frac{V_u}{\cos(\theta_{WEB})} \qquad \text{Eq. (6.11.9-1)}$$

$$V_{ui} = \frac{-360}{\cos(0.24 \text{ rad})} = -371 \text{ kip}$$

The shear-buckling resistance of the 213-inch panel is determined as:

$$V_n = V_{cr} = CV_p \qquad \text{Eq. (6.10.9.2-1)}$$

C is the ratio of the shear-buckling resistance to the shear yield strength determined as specified in Article 6.10.9.3.2. First, compute the shear buckling coefficient, k:

$$k = 5 + \frac{5}{\left(\frac{d_o}{D}\right)^2} \qquad \text{Eq. (6.10.9.3.2-7)}$$

$$k = 5 + \frac{5}{\left(\frac{213.0}{87.0}\right)^2} = 5.83$$

Since,

$$1.40\sqrt{\frac{Ek}{F_{yw}}} = 1.40\sqrt{\frac{29{,}000(5.83)}{50}} = 81.44 < \frac{D}{t_w} = \frac{87.0}{0.625} = 139.2$$

$$C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2}\left(\frac{Ek}{F_{yw}}\right) \qquad \text{Eq. (6.10.9.3.2-6)}$$

$$C = \frac{1.57}{(139.2)^2}\left(\frac{29{,}000(5.83)}{50}\right) = 0.274$$

V_p is the plastic shear force calculated as follows:

$$V_p = 0.58F_{yw}Dt_w \qquad \text{Eq. (6.10.9.3.2-3)}$$

$$V_p = 0.58(50)(87.0)(0.625) = 1{,}577 \text{ kips}$$

Therefore,

$$V_n = V_{cr} = 0.274(1{,}577) = 432 \text{ kips}$$

$$\varphi_v V_{cr} = 1.0(432) = 432 \text{ kips}$$

$$|-371| \text{ kips} < 432 \text{ kips} \qquad \text{ok} \qquad (\text{Ratio} = 0.859)$$

8.7 Girder Strength Limit State Check: Section 2-1 (Negative Moment at Interior Pier Location)

8.7.1 Flexure (Article 6.11.6.2)

For composite sections in negative flexure at the strength limit state, Article 6.11.6.2.3 directs the Engineer to Article 6.11.8. Furthermore, Article 6.11.6.2.3 states the provisions of Appendix A6 are not to be applied, nor is redistribution of negative moment per Appendix B6.

At the strength limit state, bottom flanges of tub girders in compression are to satisfy:

$$f_{bu} \le \varphi_f F_{nc} \qquad \text{Eq. (6.11.8.1.1-1)}$$

where F_{nc} is the nominal flexural resistance of the bottom flange determined as specified in Article 6.11.8.2.

At the strength limit state, the top flanges in tension are continuously braced by the deck, and are to satisfy:

$$f_{bu} \le \varphi_f F_{nt} \qquad \text{Eq. (6.11.8.1.2-1)}$$

where F_{nt} is the nominal flexural resistance of the top flanges determined as specified in Article 6.11.8.3.

Compute the factored Strength I maximum flange flexural stresses at Section 2-1, calculated without consideration of flange lateral bending. As discussed previously, the η factor is taken equal to 1.0 in this example. Therefore:

For Strength I:

Top flange:

$$f_{bu} = 1.0\left[\frac{1.25(-27{,}012)}{15{,}332} + \frac{1.25(-3{,}321)}{17{,}694} + \frac{1.5(-3{,}193)}{17{,}694} + \frac{1.75(-12{,}823)}{17{,}694}\right]12 = 47.71\ \text{ksi}$$

Bottom flange:

$$f_{bu} = 1.0\left[\frac{1.25(-27{,}012)}{16{,}709} + \frac{1.25(-3{,}321)}{17{,}114} + \frac{1.5(-3{,}193)}{17{,}114} + \frac{1.75(-12{,}823)}{17{,}114}\right]12 = -46.25\ \text{ksi}$$

8.7.1.1 Bottom Flange - Flexural Resistance in Compression - Stiffened Flange (Article 6.11.8.2.3)

Calculate the nominal flexural resistance of the bottom flange in compression, F_{nc}, in accordance with Article 6.11.8.2. The bottom flange is longitudinally stiffened at this location, with a single WT12 x 42, placed at the center of the bottom flange.

Determine the slenderness ratio of the bottom flange:

$$\lambda_f = \frac{b_{fc}}{t_{fc}} \qquad \text{Eq. (6.11.8.2.2-8)}$$

where:

b_{fc} = w = larger of the width of the flange between longitudinal flange stiffeners or the distance from a web to the nearest longitudinal flange stiffener.

In this case, since the longitudinal stiffener is at the center of the bottom flange, w is the distance from the longitudinal stiffener to the inside face of the web.

$$\lambda_f = \frac{(95.125)/2}{1.75} = 27.18$$

Since a single bottom flange stiffener is used, n = 1 and,

$$k = \left(\frac{8I_S}{wt_{fc}^{\ 3}}\right)^{\frac{1}{3}} \qquad \text{Eq. (6.11.8.2.3-1)}$$

and,

$$k_S = \frac{5.34 + 2.84\left(\frac{I_S}{wt_{fc}^{\;3}}\right)^{\frac{1}{3}}}{(n+1)^2} \leq 5.34 \qquad \text{Eq. (6.11.8.2.3-3)}$$

where: n = number of equally spaced longitudinal flange stiffeners

k = plate buckling coefficient for uniform normal stress, $1.0 \leq k \leq 4.0$

k_S = plate buckling coefficient for shear stress

I_S = moment of inertia of a single longitudinal flange stiffener about an axis parallel to the flange and taken at the base of the stiffener

Structural tees are efficient shapes for longitudinal stiffeners because they provide a high ratio of stiffness to cross-sectional area. For the WT12x42 stiffener:

$$I_s = 166 + 12.4(9.13)^2 = 1{,}200 \text{ in.}^4$$

Therefore,

$$k = \left(\frac{8(1{,}200)}{(47.56)(1.75)^3}\right)^{\frac{1}{3}} = 3.35 < 4.0$$

$$k_S = \frac{5.34 + 2.84\left(\frac{1{,}200}{(47.56)(1.75)^3}\right)^{\frac{1}{3}}}{(1+1)^2} = 2.52 < 5.34$$

Calculate λ_p,

$$\lambda_p = 0.57\sqrt{\frac{Ek}{F_{yc}\Delta}} \qquad \text{Eq. (6.11.8.2.2-9)}$$

where:

$$\Delta = \sqrt{1 - 3\left(\frac{f_v}{F_{yf}}\right)^2} \qquad \text{Eq. (6.11.8.2.2-11)}$$

As stated previously, the St. Venant torsional shear stress, f_v, can be assumed to be zero because the bottom flange width does not exceed one-fifth of the effective span length, and all other requirements of Article 6.11.2.3 are satisfied (see Article C6.11.2.3).

Therefore, since f_v is zero:

$$\Delta = \sqrt{1-3\left(\frac{0}{50}\right)^2} = 1.0$$

$$\lambda_p = 0.57\sqrt{\frac{(29000)(3.35)}{(50)(1.0)}} = 25.13 < \lambda_f = 27.18$$

Since λ_f exceeds λ_p, it is necessary to calculate λ_r:

$$\lambda_r = 0.95\sqrt{\frac{Ek}{F_{yr}}} \qquad \text{Eq. (6.11.8.2.2-10)}$$

where:

$$F_{yr} = (\Delta - 0.4)\, F_{yc} \leq F_{yw} \qquad \text{Eq. (6.11.8.2.2-7)}$$

$$F_{yr} = (1.0 - 0.4)(50) = 30 \text{ ksi} < 50 \text{ ksi}$$

Therefore, λ_r is calculated as:

$$\lambda_r = 0.95\sqrt{\frac{(29000)(3.35)}{(30)}} = 54.06 > \lambda_f = 27.18$$

Since $\lambda_p < \lambda_f < \lambda_r$,

$$F_{cb} = R_b R_h F_{yc}\left[\Delta - \left(\Delta - \frac{\Delta\text{-}0.3}{R_h}\right)\left(\frac{\lambda_f - \lambda_p}{\lambda_r - \lambda_p}\right)\right] \qquad \text{Eq. (6.11.8.2.2-3)}$$

Determine the web load-shedding factor, R_b, in accordance with Article 6.10.1.10.2. As discussed in Article C6.11.8.2.2, in calculating R_b for a tub section, use one-half of the effective box flange width in conjunction with one top flange and a single web, where the effective box flange width is defined in Article 6.11.1.1.

First, compute the depth of the web in compression, D_c, in accordance with Article D6.3.1. According to Article D6.3.1, at the strength limit state, D_c for composite sections in negative flexure is to be computed for the section consisting of the steel girder plus the longitudinal reinforcement as follows:

$$D_c = 45.19 \text{ in.} - 1.75 \text{ in.} = 43.44 \text{ in.}$$

Compute D_c along the inclined web:

$$D_c = (43.44)\sqrt{\frac{4^2+1}{4^2}} = 44.78 \text{ in.}$$

$$\frac{2D_c}{t_w} = \frac{2(44.78)}{0.625} = 143.3$$

According to the provisions of Article 6.10.1.10.2:

$$\text{If } \frac{2D_c}{t_w} \le \lambda_{rw}\text{, then } R_b = 1.0 \qquad \text{Eq. (6.10.1.10.2-1)}$$

where:

$$4.6\sqrt{\frac{E}{F_{yc}}} \le \lambda_{rw} = \left(3.1 + \frac{5.0}{a_{wc}}\right)\sqrt{\frac{E}{F_{yc}}} \le 5.7\sqrt{\frac{E}{F_{yc}}} \qquad \text{Eq. (6.10.1.10.2-5)}$$

$$a_{wc} = \frac{2D_c t_w}{b_{fc} t_{fc}} \qquad \text{Eq. (6.10.1.10.2-8)}$$

$$4.6\sqrt{\frac{E}{F_{yc}}} = 4.6\sqrt{\frac{29,000}{50}} = 111$$

$$5.7\sqrt{\frac{E}{F_{yc}}} = 5.7\sqrt{\frac{29,000}{50}} = 137$$

$$a_{wc} = \frac{2(44.78)(0.625)}{(95.125/2)(1.75)} = 0.673$$

$$111 < \lambda_{rw} = \left(3.1 + \frac{5.0}{0.673}\right)\sqrt{\frac{29,000}{50}} = 253.6 > 137$$

$$\therefore \lambda_{rw} = 137 < \frac{2D_c}{t_w} = 143.3$$

Since $\frac{2D_c}{t_w}$ > λ_{rw}, calculate R_b as follows:

$$R_b = 1 - \left(\frac{a_{wc}}{1200 + 300a_{wc}}\right)\left(\frac{2D_c}{t_w} - \lambda_{rw}\right) \leq 1.0 \qquad \text{Eq. (6.10.1.10.2-3)}$$

Therefore,

$$R_b = 1 - \left(\frac{0.673}{1200 + 300(0.673)}\right)\left(\frac{2(44.78)}{0.625} - 137\right) = 0.997 < 1.0$$

Calculate F_{cb},

$$F_{cb} = R_b R_h F_{yc}\left[\Delta - \left(\Delta - \frac{\Delta\text{-}0.3}{R_h}\right)\left(\frac{\lambda_f - \lambda_p}{\lambda_r - \lambda_p}\right)\right] \qquad \text{Eq. (6.11.8.2.2-3)}$$

$$F_{cb} = (0.997)(1.0)(50)\left[1.0 - \left(1.0 - \frac{1.0 - 0.3}{1.0}\right)\left(\frac{27.18 - 25.13}{54.06 - 25.13}\right)\right]$$

F_{cb} = 48.79 ksi

The nominal flexural resistance of the compression flange, F_{nc}, is calculated as:

$$F_{nc} = F_{cb}\sqrt{1 - \left(\frac{f_v}{\varphi_v F_{cv}}\right)^2} \qquad \text{Eq. (6.11.8.2.2-1)}$$

Since f_v =0 ksi, F_{nc} = F_{cb} = 48.79 ksi.

For Strength I:

f_{bu} = -46.25 ksi
$\phi_f F_{nc}$ = 1.0 (48.79) = 48.79 ksi
|-46.25| ksi < 48.79 ksi (Ratio = 0.948)

Longitudinal flange stiffeners are preferably discontinued at field splice locations at the free edge of the flange where the flange stress is zero, particularly when the span balance is such that the box flange on the other side of the field splice does not require stiffening, which is not the case in this design example (see Section 8.10.1). In such cases, the compressive resistance of the unstiffened box flange on the other side of the splice should always be checked to determine if the flange is satisfactory without a stiffener or is a slight increase in the flange thickness will suffice without providing a stiffener (see Section 8.10.1). Figure 14 illustrates a suggested box-flange bolted splice detail to accommodate a termination of the stiffener at the free edge of the flange. When the stiffener is terminated as such, fatigue of the base metal at the stiffener-to-flange weld termination need not be checked in regions subject to a net applied tensile stress because the flange stress is zero at the termination. Otherwise, the base metal at the stiffener termination would need to be checked as a fatigue Category E or E' detail depending on the stiffener thickness, unless a transition radius is provided at the termination (refer to Condition 4.3 in Table 6.6.1.2.3-1 and Section 8.10.1). For further discussion on tub girder bottom flange longitudinal stiffeners, refer to Section 3.7 of the AASHTO/NSBA Steel Bridge Collaboration document G12.1, *Guidelines to Design for Constructability and Fabrication* [10].

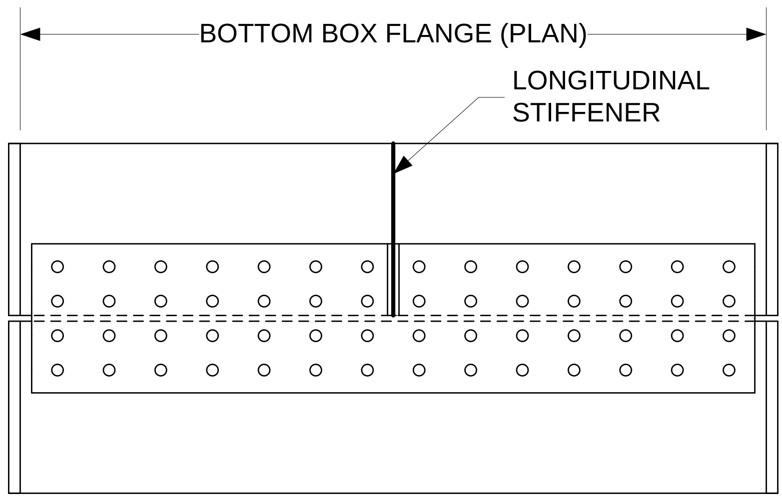

Figure 14 Suggested Bottom Flange Bolted Splice Detail at a Flange Stiffener Termination

8.7.1.2 Top Flange - Flexural Resistance in Tension (Article 6.11.8.3)

Calculate the nominal flexural resistance of the top flange in tension, F_{nt}, in accordance with Article 6.11.8.3.

$$F_{nt} = R_h F_{yt} \qquad \text{Eq. (6.11.8.3-1)}$$

For a homogeneous girder, R_h is equal to 1.0 (Article 6.10.1.10.1). Therefore,

$$F_{nt} = 1.0(50) = 50 \text{ ksi}$$

For Strength I:

$$f_{bu} = 47.71 \text{ ksi}$$

$$\phi_f F_{nt} = 1.0(50.00) = 50.0 \text{ ksi}$$

$$47.71 \text{ ksi} < 50.0 \text{ ksi} \quad \text{ok} \qquad (\text{Ratio} = 0.954)$$

8.7.2 Shear (Article 6.11.6.3)

Article 6.11.6.3 invokes to the provisions of Article 6.11.9 to determine the shear resistance at the strength limit state. Article 6.11.9 further directs the Engineer to the provisions of Article 6.10.9 for determining the factored shear resistance of a single web. For the case of inclined webs, D, is to be taken as the depth of the web measured along the slope. The factored shear in the inclined web is to be taken as:

$$V_{ui} = \frac{V_u}{\cos(\theta)} \qquad \text{Eq. (6.11.9-1)}$$

where V_u is the shear due to factored loads on one inclined web, and θ is the angle of inclination of the web plate.

At the strength limit state, webs must satisfy the following:

$$V_u \le \varphi_v V_n \qquad \text{Eq. (6.10.9.1-1)}$$

where: ϕ_v = resistance factor for shear = 1.0 (Article 6.5.4.2)
V_n = nominal shear resistance determined as specified in Articles 6.10.9.2 and 6.10.9.3 for unstiffened and stiffened webs, respectively
V_u = V_{ui} = factored shear in a single web at the section under consideration

The η factor is again taken equal to 1.0 in this example at the strength limit state. Live-load shears are taken as the shear envelope values. A sample calculation of V_{ui}, for a single web at the interior pier is given below for Strength I:

$$V_{ui} = \frac{1.0\left[1.25(270+36)+1.5(35)+1.75(139)\right]}{\cos\left(0.24 \text{ rad}\right)} = 698 \text{ kips}$$

It has been previously shown in this example (for the positive moment section) that the shear capacity of the unstiffened web is:

$$\varphi_v V_n = 1.0(370) = 370 \text{ kips}$$

The maximum value of V_u in Field Section 2 is 698 kips, which exceeds $\phi_v V_n$ = 370 kips. Therefore, transverse stiffeners are required in Field Section 2 and the provisions of Article 6.10.9.3 apply.

8.7.2.1 Interior Panel (Article 6.10.9.3.2)

An interior panel is a web panel not adjacent to the discontinuous end of a girder. Article 6.10.9.1 stipulates that the transverse stiffener spacing for interior panels without a longitudinal stiffener is not to exceed 3D = 3(87.0) = 261.0 inches. For the first panel to the right of the first interior support, assume a transverse spacing of d_o = 17.75 feet = 213.0 inches, which is the distance from the interior support to the first top lateral strut location in Span 2, and one-half of the internal cross-frame spacing.

For interior panels of girders with the section along the entire panel proportioned such that:

$$\frac{2Dt_w}{\left(b_{fc}t_{fc}+b_{ft}t_{ft}\right)}\leq 2.5 \qquad \text{Eq. (6.10.9.3.2-1)}$$

the nominal shear resistance is to be taken as the sum of the shear buckling resistance and the postbuckling resistance due to tension-field action, or:

$$V_n = V_p\left[C+\frac{0.87(1-C)}{\sqrt{1+\left(\frac{d_o}{D}\right)^2}}\right] \qquad \text{Eq. (6.10.9.3.2-2)}$$

Otherwise, the nominal shear resistance is to be taken as the shear resistance determined from Eq. (6.10.9.3.2-8).

Article 6.11.9 specifies that for bottom flanges of tub girders, b_{fc} or b_{ft}, as applicable, is be taken as one-half of the effective flange width between webs in checking Eq. 6.10.9.3.2-1, where the effective flange width is to be taken as specified in Article 6.11.1.1, but not to exceed $18t_f$ where t_f is the thickness of the flange.

$$b_{fc}=95.125/2=47.56\text{ in.}>18\left(1.75\right)=31.50\text{ in.}\quad\therefore\text{ Use }b_{fc}=31.50\text{ in.}$$

For the interior web panel under consideration:

$$\frac{2(87.0)(0.625)}{\left[(31.50)(1.75)+30(2.75)\right]}=0.79<2.5$$

Therefore:

$$k=5+\frac{5}{\left(\frac{213.0}{87.0}\right)^2}=5.83$$

Since,

$$1.40\sqrt{\frac{Ek}{F_{yw}}}=1.40\sqrt{\frac{29{,}000(5.83)}{50}}=81.44<\frac{D}{t_w}=\frac{87.0}{0.625}=139.2$$

$$C=\frac{1.57}{\left(139.2\right)^2}\left(\frac{29{,}000(5.83)}{50}\right)=0.274 \qquad \text{Eq. (6.10.9.3.2-6)}$$

$$V_p = 0.58F_{yw}Dt_w \qquad \text{Eq. (6.10.9.3.2-3)}$$

$$V_p = 0.58(50)(87.0)(0.625) = 1{,}577 \text{ kips}$$

Therefore,

$$V_n = 1{,}577\left[0.274 + \frac{0.87(1-0.274)}{\sqrt{1+\left(\frac{213.0}{87.0}\right)^2}}\right] = 809 \text{ kips}$$

$$\phi_v V_n = 1.0(809) = 809 \text{ kips} > V_u = 698 \text{ kips} \quad \text{ok} \quad (\text{Ratio} = 0.863)$$

Separate calculations, similar to these shown above, are used to determine the need for and the spacing of the transverse stiffeners in the remainder of the negative moment region and will not be repeated here. The resulting stiffener spacings are shown on the girder elevation in Figure 3. Note that although larger spacings could have been used in each panel in Field Section 2, the stiffeners in each panel were located midway between the cross-frame connection plates in each panel, and at locations of the top lateral struts, for practical reasons to help simplify the detailing.

8.8 Girder Service Limit State Check: Section 2-1 (Interior Pier)

Article 6.11.4 directs the Engineer to Article 6.10.4, which contains provisions related to the control of permanent deformations at the service limit state.

8.8.1 Permanent Deformations (Article 6.10.4.2)

Article 6.10.4.2 contains criteria intended to control permanent deformations that may impair rideability. As specified in Article 6.10.4.2.1, these checks are to be made under the Service II load combination.

Under the load combinations specified in Table 3.4.1-1, Eqs. (6.10.4.2.2-1) and (6.10.4.2.2-2) need not be checked for composite sections in negative flexure. For sections in negative flexure, these equations do not control and need not be checked (see Article C6.11.4).

It should be noted, in accordance with Article 6.11.4, that redistribution of negative moment due to the Service II loads at the interior-pier sections in continuous span flexural members using the procedures specified in Appendix B6 is not to be applied to tub-girder sections.

Web bend buckling must always be checked, however, at the service limit state under the Service II load combination for composite sections in negative flexure as follows:

$$f_c \le F_{crw} \qquad \text{Eq. (6.10.4.2.2-4)}$$

where f_c is the compression-flange stress at the section under consideration due to the Service II loads, calculated without consideration of flange lateral bending, and F_{crw} is the nominal elastic bend-buckling resistance for webs determined as specified in Article 6.10.1.9.

Determine the nominal elastic web bend-buckling resistance at Section 2-1 according to the provisions of Article 6.10.1.9.1 as follows:

$$F_{crw} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} \le \min\left(R_h F_{yc}, \frac{F_{yw}}{0.7}\right) \qquad \text{Eq. (6.10.1.9.1-1)}$$

where F_{yc} is the specified minimum yield strength of the compression flange and,

$$k = \frac{9}{\left(D_c / D\right)^2} \qquad \text{Eq. (6.10.1.9.1-2)}$$

According to Article D6.3.1 (Appendix D6), for composite sections in negative flexure at the service limit state where the concrete deck is considered effective in tension for computing flexural stresses on the composite section, the depth of the web in compression in the elastic range measured from the neutral axis down to the top of the bottom flange, D_c, is to be computed as follows:

$$D_c = \left(\frac{-f_c}{|f_c| + f_t}\right) d - t_{fc} \ge 0 \qquad \text{Eq. (D6.3.1-1)}$$

where: f_t = the sum of the tension-flange stresses caused by the factored Service II loads, in this case stresses in the top flange, calculated without considering flange lateral bending.
f_c = the sum of the compression-flange stresses caused by the factored Service II loads, in this case stresses in the bottom flange.
d = depth of the steel section.
t_{fc} = thickness of the compression flange, in this case the bottom flange.

Eq. (D6.3.1-1) recognizes the beneficial effect of the dead-load stress on the location of the neutral axis of the composite section (including the concrete deck) in regions of negative flexure. Otherwise, D_c is to be computed for the section consisting of the steel girder plus the longitudinal reinforcement.

In order to consider the deck to be considered effective in negative flexure, Article 6.10.4.2.1 requires that shear connectors be provided throughout the entire length of the tub girder, the minimum amount of negative flexure concrete deck reinforcement be provided in accordance with Article 6.10.1.7, and the maximum longitudinal tensile stress in the concrete deck at the section under consideration caused by the Service II loads be smaller than $2f_r$, where f_r is the modulus of rupture of the concrete specified in Article 6.10.1.7. If these conditions are satisfied, flexural stresses caused by Service II loads applied to the composite section may be computed using the short-term or long-term composite section, as appropriate, assuming the concrete to be effective in tension. Otherwise, the flexural stresses caused by Service II loads applied to the composite section must be computed using the section consisting of the steel section plus the longitudinal reinforcement.

The first two of the preceding conditions are satisfied (see Section 8.8.2Check the tensile stress in the concrete deck due to the Service II load combination at Section 2-1. The longitudinal concrete deck stress is determined as specified in Article 6.10.1.1.1d; that is, using the short-term modular ratio *n* = 8.00. Note that only DC_2, DW and LL+IM are assumed to cause stress in the concrete deck. The calculated stress on the transformed section must be divided by the modular ratio, n = 8, to determine the stress in the concrete deck.

$$f_{deck} = \frac{1.0\left[1.0(-3{,}321) + 1.0(-3{,}193) + 1.3(-12{,}823)\right](12)}{32{,}858(8)} = 1.058\ \text{ksi} > 2f_r = 0.960\ \text{ksi}$$

The concrete deck may not be considered effective in tension at the service limit state at Section 2-1.

Therefore, for Service II:

Top flange (tension flange):

$$f_t = 1.0\left[\frac{1.0(-27{,}012)}{15{,}332} + \frac{1.0(-3{,}321)}{17{,}694} + \frac{1.0(-3{,}193)}{17{,}694} + \frac{1.30(-12{,}823)}{17{,}694}\right]12 = 36.86\ \text{ksi}$$

Bottom flange (compression flange):

$$f_c = 1.0\left[\frac{1.0(-27{,}012)}{16{,}709} + \frac{1.0(-3{,}321)}{17{,}114} + \frac{1.0(-3{,}193)}{17{,}114} + \frac{1.30(-12{,}823)}{17{,}114}\right]12 = -35.66\ \text{ksi}$$

Calculate the depth of the web that is in compression, D_c. According to Article D6.3.1 (Appendix D6), since the deck is not permitted to be considered effective in tension at this section at the service limit state, D_c is to be computed for the section consisting of the steel girder plus the longitudinal reinforcement as follows:

$$D_c = 45.19\ \text{in.} - 1.75\ \text{in.} = 43.44\ \text{in.}$$

Calculate D_c along the web:

$$D_c = \frac{43.44}{\cos\left(\arctan\left(\frac{1}{4}\right)\right)} = 44.78 \text{ in.}$$

$$k = \frac{9}{\left(\frac{D_c}{D}\right)^2} = \frac{9}{\left(\frac{44.78}{87.0}\right)^2} = 33.97$$

$$F_{crw} = \frac{0.9(29{,}000)(33.97)}{\left(\frac{87.0}{0.625}\right)^2} = 45.76 \text{ ksi} < R_h F_{yc} = 50 \text{ ksi} < \frac{F_{yw}}{0.7} = 71.4 \text{ ksi}$$

$$\left|-35.66 \text{ ksi}\right| < 45.76 \text{ ksi} \qquad \text{ok} \qquad (\text{Ratio} = 0.779)$$

8.8.2 Concrete Deck (Article 6.10.1.7)

Article 6.10.1.7 requires the minimum one-percent longitudinal reinforcement in the concrete deck wherever the longitudinal tensile stress in the deck due to the factored construction loads or due to the Service II load combination exceeds ϕf_r.

Check the tensile stress in the concrete deck due to the Service II load combination at the section 45.0 feet from Pier 1 in Span 2. The longitudinal concrete deck stress is determined as specified in Article 6.10.1.1.1d; that is, using the short-term modular ratio n = 8.00. Note that only DC_2, DW and LL+IM are assumed to cause stress in the concrete deck.

$$f_{deck} = \frac{1.0\left[1.0(-676) + 1.0(-650) + 1.3(-4{,}847)\right](12)}{26{,}810(8)} = 0.427 \text{ ksi} < 0.90 f_r = 0.432 \text{ ksi}$$

Extend the minimum reinforcement to a section 45.0 feet from the Pier 1 in Span 2 for the Service II loads.

Also, check the tensile stress in the concrete deck due to Service II load combination at the section 115.0 feet from the abutment in Span 1. The longitudinal concrete deck stress is determined as specified in Article 6.10.1.1.1d; that is, using the short-term modular ratio n = 8.00, and only DC_2, DW and LL+IM are included.

$$f_{deck} = \frac{1.0\left[1.0(122) + 1.0(113) + 1.3(-5{,}703)\right](12)}{25{,}296(8)} = 0.426 \text{ ksi} < 0.90 f_r = 0.432 \text{ ksi}$$

From Pier 1, extend the minimum reinforcement to a section 115.0 feet from the abutment in Span 1 for the Service II loads. The above locations should also be similarly checked for the factored construction loads (i.e., the deck placement sequence) with adjustments made as necessary.

8.9 Girder Fatigue and Fracture Limit State Check: Section 2-1 (Negative Moment at Interior Pier Location)

8.9.1 Fatigue (Article 6.11.5)

Article 6.11.5 directs the Engineer to Article 6.10.5, where details on tub girder section flexural members must be investigated for fatigue as specified in Article 6.6.1. Either the Fatigue I or Fatigue II load combination specified in Table 3.4.1-1 and the fatigue live load specified in Article 3.6.1.4 is to be employed for checking load-induced fatigue in tub girder sections. Further discussion concerning load induced fatigue in tub girders is presented in Section 8.4.1.

The fatigue details employed in this example in the negative moment regions, such as the connection plate welds to the flanges, satisfy the limit state specified for load induced fatigue in Article 6.11.5. Furthermore, interior panels of webs with transverse stiffeners satisfy Article 6.10.5.3. The detailed checks are not illustrated in this example; however similar checks are illustrated in Section 8.4.1 and in Design Example 1 of the NSBA Steel Bridge Design Handbook.

8.9.2 Fracture (Article 6.6.2)

Discussion concerning the fracture limit state for tub girders was previously presented in Section 8.4.2.

8.10 Girder Check: Section 1-2 and 1-3

8.10.1 Comparison of Unstiffened and Stiffened Bottom Flange in End Spans

Because a field section length of 130 feet is required to minimize the number of field sections and field splices for the given span arrangement, girder Section 1-3 is not located at a point of dead load contraflexure. Due to the span balance, there is negative bending moment at Section 1-3 causing the bottom flange to be in compression. When proportioning the bottom flange at this location, two options exist:

- Option A – use a thicker, unstiffened, bottom flange
- Option B – use a longitudinally stiffened bottom flange which will allow a thinner bottom flange plate to be used.

For comparison, both these options are briefly presented in this section.

8.10.1.1 Option A - Unstiffened Flange

The resistance in compression of a tub girder bottom flange that is unstiffened is limited by the buckling resistance of the plate, which is a function of the flange slenderness (b/t) ratio. Therefore, a simple option that may be used to increase the resistance is to increase the thickness of the bottom flange plate. For this particular example, the bottom flange plate thickness is 1.375 inches at Section 1-3, and this plate is extended to Section 1-2 which is 93.75 feet from the abutment, as shown previously in Figure 3.

Compute the factored maximum bottom flange flexural stress in Section 1-3 at the field splice under the Strength I load combination. As discussed previously, the η factor is taken equal to 1.0 in this example. At this location, the unfactored bending moments are approximately as follows:

M_{DC1} = -4,412 kip-ft
M_{DC2} = -406 kip-ft
M_{DW} = -390 kip-ft
M_{LL+I} = -7,750 kip-ft

The negative flexure concrete deck reinforcement (Section 8.1.2.2) is assumed extended through Section 1-3. Therefore, the longitudinal reinforcement is included in the composite section property calculations. Separate calculations similar to the section property calculations at Section 2-1, but not included herein, show that at Section 1-3:

Steel Section only:	$S_{BOT\ OF\ STEEL}$ = 11,092 in.^{3}
Steel Section + Long. Reinforcement:	$S_{BOT\ OF\ STEEL}$ = 12,238in.^{3}

Therefore, for Strength I:

$$f_{bu} = 1.0\left[\frac{1.25(-4{,}412)}{11{,}092} + \frac{1.25(-406)}{12{,}238} + \frac{1.5(-390)}{12{,}238} + \frac{1.75(-7{,}750)}{12{,}238}\right]12 = -20.34 \text{ ksi}$$

Calculate the nominal flexural resistance of the bottom flange in compression, F_{nc}, in accordance with Article 6.11.8.2.2. This calculation is similar to the calculations shown to compute the bottom flange negative moment flexural resistance at Section 2-1, therefore calculations for Section 1-3 are briefly summarized below.

$$\lambda_f = \frac{b_{fc}}{t_{fc}} \qquad \text{Eq. (6.11.8.2.2-8)}$$

$$\lambda_f = \frac{(95.125)}{1.375} = 69.18$$

Compute λ_r,

$$\lambda_r = 0.95\sqrt{\frac{Ek}{F_{yr}}} \qquad \text{Eq. (6.11.8.2.2-10)}$$

where k = 4.0 (taken as 4.0 since bottom flange is unstiffened) and where:

$F_{yr} = (\Delta - 0.4)F_{yc} \leq F_{yw}$ Eq. (6.11.8.2.2-13)

$F_{yr} = (1.0 - 0.4)(50) = 30 \text{ ksi} < 50 \text{ ksi}$

Therefore, λ_r is calculated as:

$$\lambda_r = 0.95\sqrt{\frac{(29000)(4.0)}{(30)}} = 59.07$$

Since $\lambda_f = 69.18 > \lambda_r = 59.07$, then,

$$F_{cb} = \frac{0.9ER_bk}{(\lambda_f)^2} \quad \text{Eq. (6.11.8.2.2-4)}$$

where,

$R_b = 1.0$ (calculated but not shown)

$$F_{cb} = \frac{0.9(29{,}000)(1.0)(4.0)}{(69.18)^2} = 21.81 \text{ ksi}$$

The nominal flexural resistance of the compression flange, F_{nc}, is calculated as:

$$F_{nc} = F_{cb}\sqrt{1 - \left(\frac{f_v}{\varphi_v F_{cv}}\right)^2} \quad \text{Eq. (6.11.8.2.2-1)}$$

Given the satisfaction of Article 6.11.2.3 requirements, $f_v = 0$ ksi and, $F_{nc} = F_{cb} = 21.81$ ksi.

Therefore, for Option A, Strength I:

$f_{bu} = -20.34$ ksi
$\phi_f F_{nc} = 1.0\ (21.81) = 21.81$ ksi
$|-20.34|$ ksi < 21.81 ksi ok (Ratio = 0.933)

8.10.1.2 Option B - Stiffened Flange

As an alternative to using a thicker bottom flange plate (Option A), the WT12 x 42 bottom flange longitudinal stiffener can be extended further into the end span, up to 93.75 feet from the end support, as shown in Figure 15. This will require that the WT12 x 42 stiffener also be spliced at the field splice, and will require careful attention to the detail at the termination of the flange stiffener in Span 1 (at Section 1-2).

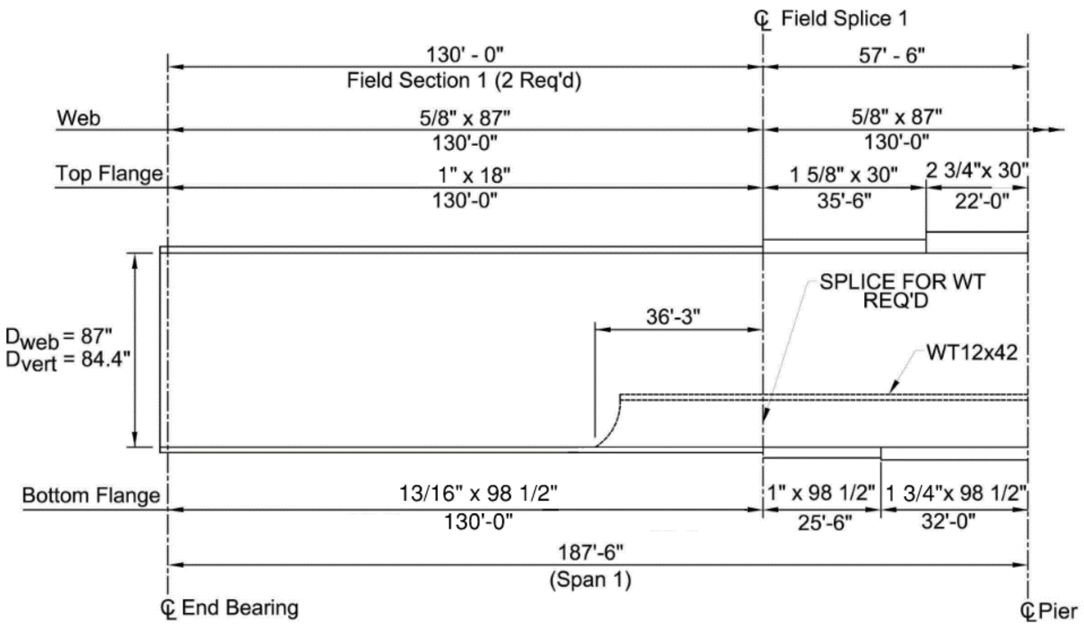

Figure 15 Option B in Elevation, Stiffened Bottom Flange

For this option, the existing 0.8125-inch-thick bottom flange plate, in combination with the WT12 x 42 bottom flange longitudinal stiffener, is used. The longitudinal flange stiffener is extended to Section 1-2, which is 93.75 feet from the abutment.

The negative flexure concrete deck reinforcement (Section 8.1.2.2) is assumed extended through Section 1-3. Therefore the longitudinal reinforcement is included in the composite section property calculations. Separate calculations similar to the section property calculations at Section 2-1, but not included herein, show that at Section 1-3:

Steel Section only: $S_{BOT\ OF\ STEEL} = 8{,}006\ in.^3$
Steel Section + Long. Reinforcement: $S_{BOT\ OF\ STEEL} = 8{,}902\ in.^3$

Compute the factored maximum bottom flange flexural stress in Section 1-3 at the field splice under the Strength I load combination. As discussed previously, the η factor is taken equal to 1.0 in this example. Therefore:

For Strength I:

$$f_{bu} = 1.0\left[\frac{1.25(-4{,}412)}{8{,}006} + \frac{1.25(-406)}{8{,}902} + \frac{1.5(-390)}{8{,}902} + \frac{1.75(-7{,}750)}{8{,}902}\right]12 = -28.02\text{ ksi}$$

Calculate the nominal flexural resistance of the bottom flange in compression, F_{nc}, in accordance with Article 6.11.8.2.2. This calculation is similar to the calculations shown to compute the bottom flange negative moment flexural resistance at Section 2-1, therefore calculations for Section 1-3 are briefly summarized below.

$$\lambda_f = \frac{b_{fc}}{t_{fc}} \qquad \text{Eq. (6.11.8.2.2-8)}$$

$$\lambda_f = \frac{(95.125 / 2)}{0.8125} = 58.54$$

Calculate λ_p,

$$\lambda_p = 0.57\sqrt{\frac{Ek}{F_{yc}\Delta}} \qquad \text{Eq. (6.11.8.2.2-9)}$$

where,

k = 4.0 (calculated as 7.22, but the limit of 4.0 governs)
Δ = 1.0 (calculated but not shown, f_v = 0.0 ksi)

$$\lambda_p = 0.57\sqrt{\frac{(29000)(4.0)}{(50.0)(1.0)}} = 27.45 < \lambda_f = 58.54$$

Compute λ_r,

$$\lambda_r = 0.95\sqrt{\frac{Ek}{F_{yr}}} \qquad \text{Eq. (6.11.8.2.2-10)}$$

where,

F_{yr} = 30.0 ksi (calculated but not shown)

$$\lambda_r = 0.95\sqrt{\frac{(29000)(4.0)}{(30.0)}} = 59.07$$

Since $\lambda_p < \lambda_f < \lambda_r$,

$$F_{cb} = R_b R_h F_{yc}\left[\Delta - \left(\Delta - \frac{\Delta\text{-}0.3}{R_h}\right)\left(\frac{\lambda_f - \lambda_p}{\lambda_r - \lambda_p}\right)\right] \qquad \text{Eq. (6.11.8.2.2-3)}$$

where:

R_b = 1.0 (calculated but not shown)

$$F_{cb} = (1.0)\,(1.0)\,(50)\left[1.0-\left(1.0-\frac{1.0\text{-}0.3}{1.0}\right)\left(\frac{58.54-27.45}{59.07-27.45}\right)\right]$$

F_{cb} = 35.25 ksi

The nominal flexural resistance of the compression flange, F_{nc}, is calculated as:

$$F_{nc} = F_{cb}\sqrt{1 - \left(\frac{f_v}{\varphi_v\ F_{cv}}\right)^2} \qquad \text{Eq. (6.11.8.2.2-1)}$$

Given the satisfaction of Article 6.11.2.3 requirements, f_v =0 ksi and, $F_{nc} = F_{cb}$ = 35.25 ksi.

Therefore, for Option B, Strength I:

f_{bu} = -28.02 ksi
$\phi_f\ F_{nc}$ = 1.0(35.25) = 35.25ksi
|-28.02| ksi < 35.25 ksi   ok   (Ratio = 0.795)

At the termination of the flange stiffener, the bottom flange is subjected to both tensile and compressive stresses under the fatigue live load. Since in this option it is necessary to terminate the flange stiffener beyond the field splice in a region that is subject to a net tensile stress, fatigue must be investigated at the stiffener termination. Under the condition of welded stiffener attachments, for base metal at the termination of longitudinal stiffener-to-box flange welds with no special transition radius provided at the weld termination, the fatigue detail is either Category E or E' depending on the stiffener thickness (refer to Condition 4.3 in Table 6.6.1.2.3-1). The thickness of the stiffener web is less than 1.0 inch and so the detail is a fatigue Category E detail. Separate computations similar to the computations given below indicate that a fatigue Category E detail is not sufficient at this location.

Therefore, a transition radius will be provided at the stiffener termination with the end weld ground smooth. A minimum-radius transition of 6 inches will provide the nominal fatigue resistance of a Category C detail (refer to Condition 4.3 in Table 6.6.1.2.3-1). A continuous fillet weld on both sides of the stiffener is used to attach the stiffener to the bottom flange. Consideration should be given to wrapping the weld around the end of the stiffener for sealing. The weld and stiffener material should then be ground to a smooth contour where the radiused stiffener end becomes tangent to the flange (Figure 16).

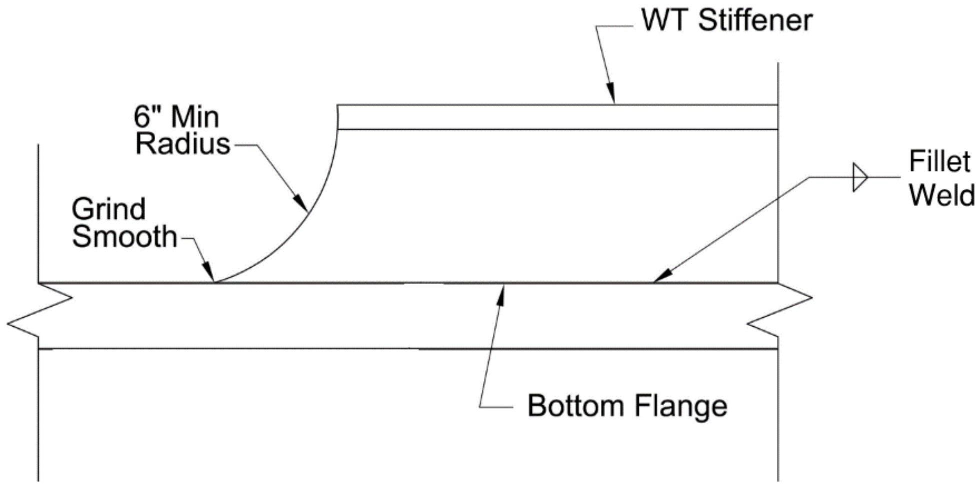

Figure 16 Option B, Longitudinal Flange Stiffener Termination

Since the $(ADTT)_{SL}$ = 2,000 trucks per day x 0.8 = 1,600 trucks per day (refer to Article 3.6.1.4.2) is less than the 75-year $(ADTT)_{SL}$ Equivalent to Infinite Life of 1,680 trucks per day specified in Table 6.6.1.3.2-2 for a Category C detail, fatigue of the base metal at the longitudinal flange stiffener weld termination will be checked for finite life using the Fatigue II load combination (Table 3.4.1-1). The stress range due to the fatigue live load modified by the corresponding dynamic load allowance of 15 percent will be used to make this check. The lateral distribution factor for the fatigue limit state, computed in Section 7.1, is also used.

The provisions of Article 6.6.1.2 apply only to details subject to a net applied tensile stress. In this example, the effect of the future wearing surface is conservatively ignored when determining if a detail is subject to a net applied tensile stress. Separate computations indicate that the stiffener termination at this location is subject to a net tensile stress under the unfactored permanent load plus the Fatigue I load combination (refer to Article 6.6.1.2.1).

According to Article 6.6.1.2.1, for flexural members with shear connectors provided throughout their entire length and with concrete deck reinforcement satisfying the provisions of Article 6.10.1.7, flexural stresses and stress ranges applied to the composite section at the fatigue limit state at all sections in the member may be computed assuming the concrete deck to be effective for both positive and negative flexure. Shear connectors are assumed along the entire length of the girder in this example, which is required for composite tub girders according to Article 6.11.10. Earlier computations were made to verify that the longitudinal concrete deck reinforcement satisfies the provisions of Article 6.10.1.7 (Section 8.8.2). Therefore, the concrete deck will be considered effective in computing all stresses and stress ranges applied to the composite section in the subsequent fatigue calculations.

The stress range, $\gamma(\Delta f)$, at the longitudinal flange stiffener weld termination due to the factored fatigue load (factored by the specified 0.80 load factor for the Fatigue II load combination) is computed using the properties of the short-term composite section as follows.

At the stiffener termination, the unfactored fatigue live load bending moments are as follows (Figure 8):

Positive Flexure: M_{LL+I} = 2,398 kip-ft
Negative Flexure: M_{LL+I} = -1,121 kip-ft

From Table 4, the short-term composite section modulus for the unstiffened bottom flange at the stiffener termination is:

Composite Section, n=8.0: $S_{BOT\ OF\ STEEL}$ = 10,285 in.^{3}

For load-induced fatigue, each detail must satisfy:

$$\gamma(\Delta f) \leq (\Delta F)_n \qquad \text{Eq. (6.6.1.2.2-1)}$$

where:

γ = load factor per Table 3.4.1-1 for the appropriate Fatigue Load Combination
(Δf) = live load stress range due to passage of fatigue truck
$(\Delta F)_n$ = nominal fatigue resistance per Article 6.6.1.2.5

Therefore, the Fatigue II stress range is computed as:

$$\gamma(\Delta f) = \frac{0.80(2{,}398)(12)}{10{,}285} + \frac{0.80|-1{,}121|(12)}{10{,}285} = 3.28 \text{ ksi}$$

Both the resistance factor ϕ and design factor η are specified to be 1.0 at the fatigue limit state (Article C6.6.1.2.2). The nominal fatigue resistance for the Fatigue II load combination and finite fatigue life is determined as:

$$(\Delta F)_n = \left(\frac{A}{N}\right)^{\frac{1}{3}} \qquad \text{Eq. (6.6.1.2.5-2)}$$

where:

$$N = (365)(75)n(ADTT)_{SL} \qquad \text{Eq. (6.6.1.2.5-3)}$$

The number of stress cycles per truck passage, n, is equal to 1.0 (Table 6.6.1.2.5-2). Therefore,

$$N = (365)(75)(1.0)(1{,}600) = 43.8 \text{ x } 10^6 \text{ cycles}$$

For a Category C detail, the detail category constant A = 44 x 10^8 ksi^3 (Table 6.6.1.2.5-1). Therefore:

$$(\Delta F)_n = \left(\frac{44 \text{ x } 10^8}{43.8 \text{ x } 10^6}\right)^{\frac{1}{3}} = 4.65 \text{ ksi}$$

$$\gamma(\Delta f) \leq (\Delta F)_n \quad \text{Eq. (6.6.1.2.2-1)}$$

3.28 ksi < 4.65 ksi ok (Ratio = 0.705)

8.10.1.3 Summary of Unstiffened Flange versus Stiffened Flange

To provide the most economical solution for terminating the stiffener beyond a field splice, the Engineer, preferably in consultation with a fabricator, should evaluate the relative cost to thicken the bottom flange adjacent to the field splice, terminate the stiffener in the span, or even run the stiffener the full length of the end span. There are several factors that the Engineer must consider prior to choosing an option, with regard to the amount of material, fabrication costs, installation costs, and the performance and long-term serviceability.

For this example, the additional material weight of each of the options that was investigated for this bottom flange section is as follows:

$$\text{Option A} = (490)\left[(36.25)\frac{(98.5)(1.375 - 0.8125)}{144}\right] = 6{,}834 \text{ lbs}$$

$$\text{Option B} = (42)(36.25) = 1{,}523 \text{ lbs}$$

While Option B saves approximately 5,300 pounds of steel per girder, it represents only a material savings, which will likely be overcome by the increase in labor costs associated with welding the stiffener, coping and grinding the stiffener termination, making the fillet welds, and fabrication and installation of the WT splice. Therefore, Option A was chosen for this design example.

8.11 Top Flange Lateral Bracing

As discussed previously in Section 4.4, top flange lateral bracing increases the torsional stiffness of tub girder sections during erection, handling, and deck placement. For composite tub girders closed by the deck slab, the cross-section of the tub is torsionally stiff. However, prior to placement of the deck slab, the open tub girder is torsionally more flexible and subject to rotation or twist. The top flange lateral bracing forms a quasi-closed section resisting shear flow from the noncomposite loading.

The lateral bracing is typically comprised of WT or angle sections and is often configured in a single diagonal (Warren or Pratt truss) arrangement. The diagonal bracing members commonly frame into the work point of the girder top flange and internal cross-frame or diaphragm connection. Alternatively, the length between internal cross-frames can be divided into multiple lateral bracing panels. Such framing arrangements usually include a single transverse strut at intermediate brace locations. The plane of the top flange lateral bracing system should be detailed to be as close as possible to the plane of the girder top flanges so as to increase the torsional stiffness of the section, while at the same time reducing connection eccentricities and excessive out-of-plane bending in the web.

8.11.1 Diagonal Bracing Members

Diagonal bracing is proportioned to resist tension or compression in combination with flexure as appropriate, based on connection geometry. Generally, design for compression will govern the member size. The member must also satisfy slenderness requirements specified in Article 6.9.3, the minimum thickness requirements of Article 6.7.3, and should satisfy the minimum area requirement given by Eq. (C6.7.5.3-1).

Preliminary proportions of the diagonal members are determined as follows:

For secondary bracing members in compression: $\frac{K\ell}{r} \leq 140$ (Table 6.6.2.1-1 and Article 6.9.3)

The maximum length work point to work point of a diagonal member is 21.15 feet in Span 2 near the interior supports. This length will be used for design since all diagonal bracing members will be the same size. For bolted or welded connections at both ends of the member, the effective length factor K may be taken as 0.750 (Article 4.6.2.5). In this example, a WT sections will be used for the lateral bracing members. If single-angle sections were to be used, the effective length factor K should be taken as 1.0.

$$\text{for } \frac{K\ell}{r} \leq 140; \quad r_{min} = \frac{(0.750)(21.15)(12)}{140} = 1.36 \text{ in.}$$

Calculate the minimum required cross-sectional area, A_d:

$$A_d \geq 0.03\ w \qquad \text{Eq. (C6.7.5.3-1)}$$

where:

w = center-to-center distance between the top flanges (in.)

$$A_d \geq 0.03\ (138.0) = 4.14 \text{ in.}^2$$

Therefore, select a WT5 x 15:

$$r_{min} = r_y = 1.37 \text{ in.} > 1.36 \text{ in.} \quad \text{ok}$$

$$A_d = 4.42 \text{ in.}^2 > 4.14 \text{ in.}^2 \quad \text{ok}$$

In the noncomposite condition, there are several loading conditions that will generate forces in the top flange bracing system. As discussed in the NSBA publication *Practical Steel Tub Girder Design* [2], torsional moments typically induced by dead loads and construction loads will result in lateral bracing member forces. These forces can be derived from the St. Venant shear flow at the girder cross-sections, assuming the horizontal truss acts as an equivalent plate. Where forces in bracing members are not readily available from a refined analysis, the shear flow across the equivalent plate can be computed from Eq. C6.11.1.1-1, and the resulting shear can then be resolved into diagonal bracing member forces.

The horizontal component of the web shear in the inclined web along the span also imposes a lateral force on the top flanges of the tub girder. In the noncomposite condition, the lateral force due to web shear is assumed to be distributed to the top flanges of the open tub girder. The majority of this force is resisted directly by the lateral struts of the bracing system and not by the diagonals. Therefore, the forces in diagonal members resulting from the web shear component are typically taken as zero.

The lateral bracing members, in conjunction with the tub girder top flanges, form a geometrically stable horizontal truss. In the noncomposite condition, the horizontal truss is connected to the girder top flanges in a region of high bending stress, considering that the neutral axis of the noncomposite section is typically near the mid-height of the steel section. Due to compatibility, the horizontal truss must experience the same axial strains as the tub girder top flanges that result from applied bending moments, therefore resulting in axial forces being carried by the bracing members. In the absence of a refined analysis, design equations have been developed by Fan and Helwig [12] to evaluate the bracing member forces due to tub girder bending.

Lateral bracing members are also subject to forces due to wind loads acting on the noncomposite girder prior to deck placement at any point during the construction sequence. The lateral load resulting from the wind pressure applied to the exposed tub girder area is typically distributed equally to the top and bottom flanges. In the noncomposite condition, the portion of lateral load applied to the top flange may then be resolved into bracing member axial forces.

Tee sections subject to axial compression can fail either by flexural buckling about the x-axis or by torsion combined with flexure about the y-axis, where the y-axis is defined as the axis of symmetry of the tee section (i.e., a failure mode known as flexural-torsional buckling). Since the flanges of the tee are typically connected to the girder flange or to a lateral connection plate (Section 8.11.3), the tee is also subject to a uniform bending moment about the major principal axis (i.e., the x-axis) due to the eccentricity of the connection at each end of the member. Second-order effects arise from the additional secondary moment caused by the axial compressive force acting through the member deflection. Article 4.5.3.2.2b specifies that the single-step adjustment or moment magnification method may be used to determine the second-order elastic moment in lieu of a more refined analysis. The flexural resistance of tee-section members is determined as specified in Article 6.12.2.2.4. The appropriate interaction curves provided in Article 6.9.2.2 can be conservatively used to check the tee-section member for the effects of combined axial compression and flexure.

8.11.2 Top Lateral Strut

Computations for a top lateral strut in Span 2 will be presented herein. It has been shown previously that for Span 2, the horizontal component of the unfactored noncomposite (DC_1) web shear per top flange is ΔV_H = 0.49 kip/ft. Therefore, the factored Strength I force resisted by the top lateral strut is:

$$F = \Delta V_H d_{STRUT} = 1.25(0.49)(17.75) = 10.87 \text{ kips}$$

where d_{STRUT} is the spacing of the lateral struts in Span 2 (near the interior supports). Note that the special load combination specified in Article 3.4.2.1 is not checked in this example.

Due to the inclination of the web, the struts are always in tension. Therefore, the member is designed in accordance with the provisions of Article 6.8.1. A L4 x 4 x ½ will be considered for the top lateral strut.

According to Article 6.8.2.1, the factored tensile resistance P_r is to be taken as the lesser of:

$$P_r = \phi_y P_{ny} = \phi_y F_y A_g \qquad \text{Eq. (6.8.2.1-1)}$$

or

$$P_r = \phi_u P_{nu} = \phi_u F_u A_n R_p U \qquad \text{Eq. (6.8.2.1-2)}$$

In the preceding equation, R_p is the reduction factor for holes taken equal to 0.90 for bolt holes punched full size and 1.0 for bolt holes drilled full size or subpunched and reamed to size. For the purposes of this example, it is assumed that holes are drilled full size; thus, R_p = 1.0.

Also, in Eq. 6.8.2.1-2, the reduction factor U (Article 6.8.2.2) accounts for the effect of shear lag in the connection. Assuming the top strut will utilize a bolted connection with two fasteners spaced at 3 inches on center in the direction of applied force U is calculated in accordance with Table 6.8.2.2-1 and Figure C6.8.2.2-1.

$$U = 1 - \frac{\bar{x}}{L} \qquad \text{Table (6.8.2.2-1)}$$

where:

$\bar{x}$ = connection eccentricity (in.); for L4 x 4 x ½, this value is 1.18 in.

L = length of connection (in.), and per Figure C6.8.2.2-1, the out-to-out distance of the bolt holes can be used. Thus, assuming 15/16-inch diameter bolt holes, L = 3 in. + 15/16 in. = 3.9375 in.

$$U = 1 - \frac{1.18}{3.9375} = 0.700$$

Values for U are presented in Table 6.8.2.2-1 for other common connection types.

Therefore,

$P_r = \phi_y F_y A_g = (0.95)(50)(3.75) = 178$ kips
$P_r = \phi_u F_u A_n R_p U = (0.80)(65)(3.28)(1.0)(0.700) = 119.4$ kips > 10.87 kips (governs)

where A_n is based on the use of a 7/8 inch diameter bolt in a standard size (15/16″ diameter) hole.

In addition to tensile resistance, the member must also satisfy the slenderness requirement specified in Article 6.8.4 for bracing members:

For secondary bracing members in tension: $\frac{\ell}{r} \leq 240$

The distance between the webs at the top of the tub girder is 138 inches. For an L4 x 4 x ½, $r_{min} = r_z = 0.776$ inches.

$$\frac{\ell}{r} = \frac{138}{0.776} = 177.8 < 240 \qquad \text{ok}$$

8.11.3 Detailing

Final detailing of lateral bracing and connections must consider long term service and performance of the structure as well as economy in fabrication and erection. The publication *Practical Steel Tub Girder Design* [2], available from NSBA, provides current guidance with regard to design philosophy and detailing practices for lateral bracing systems.

Whenever possible, the lateral bracing should be connected as close as possible to the horizontal plane of the tub girder top flanges. Providing bracing connections to the flanges is more economical than connections to the webs since they involve fewer connection components, and they are much simpler to fabricate and connect as compared to connections to the tub girder webs. The connection can be further simplified if gusset plates are eliminated and the bracing members are connected directly to the tub girder top flanges. Connecting the lateral bracing directly to the top flanges also provides a direct load path between the bracing member and the tub girder top flanges, further simplifying the design of the connection and eliminating concerns about out-of-plane bending of the web. Additionally, inspection of the lateral bracing connection is enhanced when the bracing is connected to the top flange, because there are fewer components in a top flange connection as compared to a web connection.

Furthermore, fatigue is an important consideration when selecting the type of connection detail to use. For example, welded connections to the top flanges, specifically in tension regions, are typically undesirable, and in some cases forbidden, due to fatigue concerns. Therefore, the use of gusset plates welded to top flanges is not recommended. A more suitable connection may be to bolt the gusset plate to the top flange, typically mitigating fatigue concerns. In some cases, where wide top flanges are used, the lateral bracing may be bolted directly to the top flange, eliminating the use of gusset plates, and providing a direct load path.

Additionally, the block shear rupture resistance of tension members at connections must be verified in accordance with Article 6.13.4. The lateral bracing members and the gusset plates were investigated to verify that adequate connection material is provided to develop the factored resistance of the connection and prevent block shear rupture (calculations not shown).

8.12 Bearings

Common tub girder designs may utilize one or two bearings at the supports. The number of bearings installed will have a significant effect on the design of the tub girder, as well as the design of the internal and external diaphragms at the support. Article 6.11.1.2 presents guidance on the use and design of bearing systems.

At the support, tub girder torsion can be directly resolved into a force couple with the use of two bearings under each tub girder. The use of two bearings also reduces the design reaction for the bearing, as compared to the use of a single bearing. Two-bearing arrangements work well for non-skewed or radial supports but are impractical for supports that are skewed more than a few degrees. In the case of a skewed support, the tub girder and external diaphragm tend to prevent uniform bearing contact during construction and deck placement.

If a single bearing is used under each tub girder at the support, contact between the tub girder and bearing is optimized. Single bearing systems tend to be more forgiving of construction tolerances, especially for skewed supports. When single bearing systems are used, the external diaphragms at support lines must be sufficient to resist the torsional moments in the tub girders, as the diaphragm and adjacent girder form a structural system to counter the torsion in the individual girders. Use of a single bearing will cause bending of the internal diaphragm, which can be significant in some cases. When the stresses in the bottom flange of the tub girder, caused by the bending of the internal diaphragm at interior pier locations, are deemed significant, the Commentary to Article 6.11.8.1.1 provides direction to check the combined stresses in the tub girder bottom flange at the strength limit state. For tub girders supported on two bearings, the flange stress due to major-axis bending of the internal diaphragm is typically small and can often be ignored.

Steel-reinforced neoprene pads and pot bearings are the most commonly used bearing types for tub girders, however, in some cases disc bearings have been successfully used as well. Steel-reinforced neoprene bearing pads are much more tolerant of construction movements. They also can be easily inspected, while generally being less expensive than pot bearings. Steel-reinforced neoprene bearing pads are not as suitable for higher reactions as compared to pot or disc bearings, and therefore may not be acceptable in some applications.

Girder movement can be accommodated by both steel-reinforced neoprene pads and pot bearings. Movement in steel-reinforced neoprene pads is accommodated by deformation within the elastomer. In cases where the magnitude of movement would require a thick and potentially unstable neoprene pad, a stainless steel/polytetrafluoroethylene (PTFE) sliding surface can be utilized. A stainless steel/PTFE sliding surface is always required for pot bearings when translation needs to be accommodated.

Regardless of the bearing type used, consideration should always be given to future jacking of the structure so that bearings can be repaired or replaced. A detailed design guide for typical bearing types used in steel bridges can be found in the AASHTO/NSBA Steel Bridge Collaboration publication *G9.1 Steel Bridge Bearing Design and Detailing Guidelines* [19].

8.13 Design Example Summary

The results for this design example at each limit state are summarized below for the maximum positive moment and maximum negative moment locations. The results for each limit state are expressed in terms of a performance ratio, defined as the ratio of a calculated value to the corresponding resistance.

8.13.1 Maximum Positive Moment Region, Span 2 (Section 2-2)

Constructability	
Flexure (Strength I)	
Eq. 6.10.1.6-1 – Top Flange	0.278
Eq. 6.10.3.2.1-1 – Top Flange, yielding	0.807
Eq. 6.10.3.2.1-2 – Top Flange, local buckling	0.696
Eq. 6.10.3.2.1-2 – Top Flange, lateral torsional buckling	0.775
Eq. 6.11.3.2-3 – Bottom Flange, yielding	0.431
Eq. 6.10.3.2.1-3 – Web Bend Buckling	0.947
Service Limit State	
Permanent Deformations (Service II)	
Eq. 6.10.4.2.2-1 – Top Flange	0.643
Eq. 6.10.4.2.2-2 – Bottom Flange	0.725
Fatigue Limit State	
Flexure (Fatigue I)	
Eq. (6.6.1.2.2-1) – Bottom Flange Connection-Plate Welds	0.498
Strength Limit State (Compact Section)	
Ductility Requirement (Eq. 6.10.7.3-1)	0.303
Flexure – Eq. 6.11.7.1.1-1 (Strength I)	0.680
Shear (at abutment) – Eq. 6.10.9.1-1 (Strength I)	0.707

8.13.2 Interior Pier Section, Maximum Negative Moment (Section 2-1)

Constructability	
Flexure (Strength I)	
Eq. 6.10.3.2.2-1 – Top Flange, yielding	0.587
Eq. 6.11.3.2-1 – Bottom Flange, local buckling	0.528
Eq. 6.11.3.2-2 – Web Bend Buckling	0.517
Shear (Strength I)	
Eq. 6.10.3.3-1	0.859
Service Limit State	
Web Bend Buckling (Service II) - Eq. 6.10.4.2.2-4	0.779

Fatigue Limit State

Flexure (Fatigue I)

Eq. (6.6.1.2.2-1) – Longitudinal Stiffener Termination 0.705

Strength Limit State

Flexure (Strength I)

Bottom Flange – Eq. 6.11.8.1.1-1 0.948

Top Flange – Eq. 6.11.8.1.2-1 0.954

Shear (at interior pier) – Eq. 6.10.9.1-1 (Strength I) 0.863

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