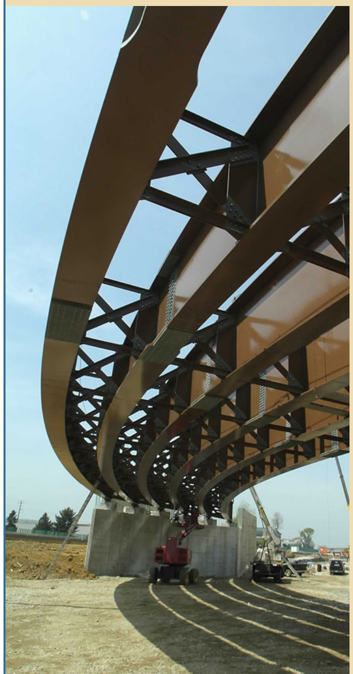

U.S. Department of Transportation
Federal Highway Administration

Steel Bridge Design Handbook

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

Publication No. FHWA-IF-12-052 - Vol. 24

November 2012

Notice

Quality Assurance Statement

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

Publication No. FHWA-IF-12-052 - Vol. 24

November 2012

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA-IF-12-052 - Vol. 24		

4. Title and Subtitle	5. Report Date
Steel Bridge Design Handbook Design Example 4: Three-Span Continuous Straight Composite Steel Tub Girder Bridge	November 2012
	6. Performing Organization Code
7. Author(s) Brandon Chavel, Ph.D., PE (HDR) and James Carnahan, PE (HDR)	8. Performing Organization Report No.
9. Performing Organization Name and Address HDR Engineering, Inc. 11 Stanwix Street Suite 800 Pittsburgh, PA 15222	10. Work Unit No. 11. Contract or Grant No.
12. Sponsoring Agency Name and Address Office of Bridge Technology Federal Highway Administration 1200 New Jersey Avenue, SE Washington, D.C. 20590	13. Type of Report and Period Covered Technical Report March 2011 – November 2012 14. Sponsoring Agency Code

15. Supplementary Notes

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

16. Abstract

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 girders 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, 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.

17. Key Words	18. Distribution Statement
Steel Tub Girder, Steel Box Girder, Lateral Bracing, Design Example, Steel Bridge	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161.

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

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

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

FOREWORD

It took an act of Congress to provide funding for the development of this comprehensive handbook in steel bridge design. This handbook covers a full range of topics and design examples to provide bridge engineers with the information needed to make knowledgeable decisions regarding the selection, design, fabrication, and construction of steel bridges. The handbook is based on the Fifth Edition, including the 2010 Interims, of the AASHTO LRFD Bridge Design Specifications. The hard work of the National Steel Bridge Alliance (NSBA) and prime consultant, HDR Engineering and their sub-consultants in producing this handbook is gratefully acknowledged. This is the culmination of seven years of effort beginning in 2005.

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

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

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

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

M. Myint Lwin, Director
Office of Bridge Technology

1.0 INTRODUCTION

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 girders clean appearance. Additionally, steel tub girder bridges offer 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 handling 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 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. For curved bridges, warping, or lateral flange bending, stresses are lower in tub girders, when compared to I-girders, since the torsional stiffness of a tub girder is much larger than the torsional stiffness of an I-girder. The larger torsional stiffness of tub girders is primarily due to the large Saint-Venant component that results from a closed cross section.

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, 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 easy 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. 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 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 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 limitations placed on the use of approximate live load distribution factors for straight tub girders currently given in the AASHTO LRFD Specifications. In order to apply the live load distribution factors, limitations are placed on the tub girder cross-section that may not make it quite as competitive as an I-girder cross-section. However, it can be interpreted that 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, future inspection and maintenance as well as fabrication.

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 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.

The bridge cross-section consists of two trapezoidal tub girders with top flanges spaced at 11.5 ft on centers, 12.0 ft between the centerline of adjacent top tub flanges, and 4.0 ft overhangs for a deck width of 43.0 ft out-to-out. For the sake of brevity, only the AASHTO-LRFD STRENGTH I and SERVICE II load combinations are demonstrated in this design example. The effects of wind loads are not considered. The reader may refer to Design Example 1 for information regarding additional load combination cases and wind load effects.

2.0 OVERVIEW OF LRFD ARTICLE 6.11

The design of tub girder flexural members is contained within Article 6.11 of the Fifth Edition of the *AASHTO LRFD Bridge Design Specifications* [1], referred to herein as *AASHTO LRFD (5th Edition, 2010)*. 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 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-girder sections 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 Constructibility
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 A and B because the applicability of these provisions to tub girders has not been demonstrated; however Appendices C and D are applicable. Flow charts for flexural design of steel girders according to the new provisions, along with a revised outline giving the basic steps for steel-bridge superstructure design, are provided in Appendix C. Appendix C may provide a useful reference for tub girder design. Fundamental calculations for flexural members have been relocated from the specification to a new Appendix D.

Example calculations demonstrating the provisions of Article 6.10, pertaining to I-girder design, is provided in the Steel Bridge Design Handbook Design Example 1. This section will highlight several of the provisions of the AASHTO LRFD as they relate to tub girder design.

One significant change that occurred in the AASHTO LRFD Third Edition from previous AASHTO LRFD Specifications is the inclusion of the lateral flange bending stress in the design checks. The provisions of Articles 6.10 and 6.11 have provided a unified approach for consideration of major axis bending and flange lateral bending, for both straight and 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, due to the width of the bottom flange, and can typically be ignored. Top flange lateral bending is caused by the outward thrust due to web inclination, wind load, temporary support brackets for deck overhangs, and from the lateral bracing system.

The constructibility provisions of Article 6.11.3 have been significantly changed from the constructibility provisions in the previous Specifications. Although the specified checks are similar in some regard, the arrangement of the provisions is much easier to follow and implement. In addition to providing adequate strength, the constructibility provisions ensure 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 specifies that for critical stages of construction, both compression and tension flanges must be investigated, and the effects of top flange lateral bending should be considered when deemed necessary by the Engineer. For noncomposite top flanges in compression, constructability design checks ensure that the maximum combined stress in the flange will not exceed the minimum yield strength, the member has sufficient strength to resist lateral torsional and flange local buckling, and that web-bend buckling will not occur. For noncomposite bottom flanges in compression, during critical stages of construction, local buckling of the flange is checked; in addition to web-bend buckling resistance. For noncomposite top and bottom flanges in tension, constructability design checks make certain that the maximum combined stress will not exceed the minimum yield strength of the flanges during construction.

3.0 DESIGN PARAMETERS

The following data apply to this example design:

Specifications: 2010 AASHTO LRFD Bridge Design Specifications, Customary U.S. Units, Fifth Edition including 2010 Interims [1]

Structural Steel: AASHTO M270, Grade 50W (ASTM A709, Grade 50W) uncoated weathering steel with F_y = 50 ksi

Concrete: f'_c = 4.0 ksi, γ = 150 pcf

Slab Reinforcing Steel: AASHTO M31, Grade 60 (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 interiors of tub girders should always be coated in a light color to aid visibility during girder inspection. Without owner direction towards a specific coating and preparation, girder interiors should receive a light brush blast and be painted with a white or light colored paint capable of telegraphing cracks in the steel section. Specified interior paint should be tolerant of minimal surface preparation. At the Engineer's discretion, an allowance may be made for the weight of the paint.

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 bottom flange's low side 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.steelbridge.org*, which is the AASHTO/NSBA Steel Bridge Collaboration's Website, with particular attention given to document G1.4, *Guidelines for Design Details* [3]. Three other detailing references offering guidance are the Texas Steel Quality Council's *Preferred Practices for Steel Bridge Design, Fabrication, and Erection* [4], the Mid-Atlantic States Structural Committee for Economic Fabrication (SCEF) Standards, and the AASHTO/NSBA Steel Bridge Collaboration *Guidelines for Design and Constructibility* [5].

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 for 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 ft - 275.0 ft - 187.5 ft. 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.

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 give consideration to providing 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 effects 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 very economical deck details standards that may not be suited, or even permitted, for wider decks spans. At the same time, wider deck spans are progressively more difficult to form and construct.

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 shall not 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 spaced at 11.5 ft on centers, 12.0 ft between the centerline of adjacent top flanges with 4.0 ft deck overhangs and an out-to-out deck width of 43.0 ft. The deck overhangs are 33 percent of the adjacent tub girder spacing. The 40.0 ft 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.

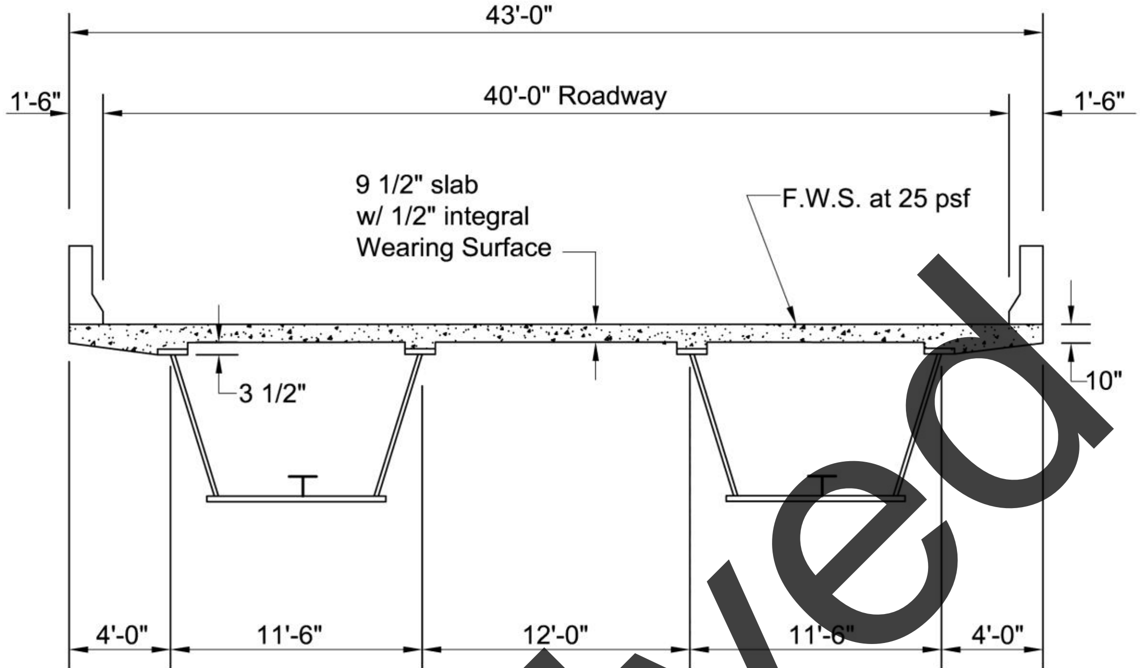

Figure 1 Typical Bridge Cross-Section

4.3 Intermediate Cross-frames

Internal intermediate cross-frames are provided in tub girders to control cross-sectional distortion. Cross-sectional distortion is caused by torsional loads that do not act on the tub girder in the same pattern as the St. Venant shear flow, which is uniformly distributed along the circumference of the tub girder cross-section. Cross-sectional distortion introduces additional stresses in the tub girder and, therefore, should be minimized. Distortional stresses can be neglected in design if a sufficient number of internal cross-frames with adequate stiffness are provided. At a minimum internal cross-frames shall be placed at points of maximum moment within a span and at points adjacent to field splices in straight bridges. Spacing of internal diaphragms, considered during development of the framing plan, should be influence by factors such as the angle and length of lateral bracing members.

Most cross-frames in modern tub girder bridges are K-frames, which allow 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 be investigated. 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 construction loads due to deck overhang brackets, in addition to satisfying slenderness requirements.

External intermediate cross-frames may be incorporated to control differential displacement and rotation of 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 would be large even with a small girder rotation.

External intermediate cross-frames typically utilize a K-frame configuration, with depth closely matching the girder depth for efficiency and simplification of supporting details. 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 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 field section, 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, in order to reduce the unbraced length of the top flange, so as to increase the lateral torsional buckling capacity for non-composite loading, a top strut is located in the center of each internal cross-frame bay.

4.4 Diaphragms at Supports

Internal diaphragms at points of support are typically full-depth plate girder 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. In order 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 at the interior supports must also be provided through the internal diaphragm. Typically, an access hole will be provided within the internal diaphragm; however care must be given in determining the location and size of the hole. The Engineer must investigate the flow of stress at the location of the hole in order to verify the sufficiency of the web near the access hole, or if reinforcing of the web may be required at the access hole.

Similar to internal diaphragms, external diaphragms are typically full-depth plate girder sections, but 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 moment 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 curved structures. Torsional moments in straight bridges are typically smaller, but should still be considered in design.

4.5 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 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 particular 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 particular 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 170,000 lbs, and the center span field section has a weight of approximately

95,000 lbs. Field sections in this length and weight range can generally be fabricated, shipped, and erected without significant issues.

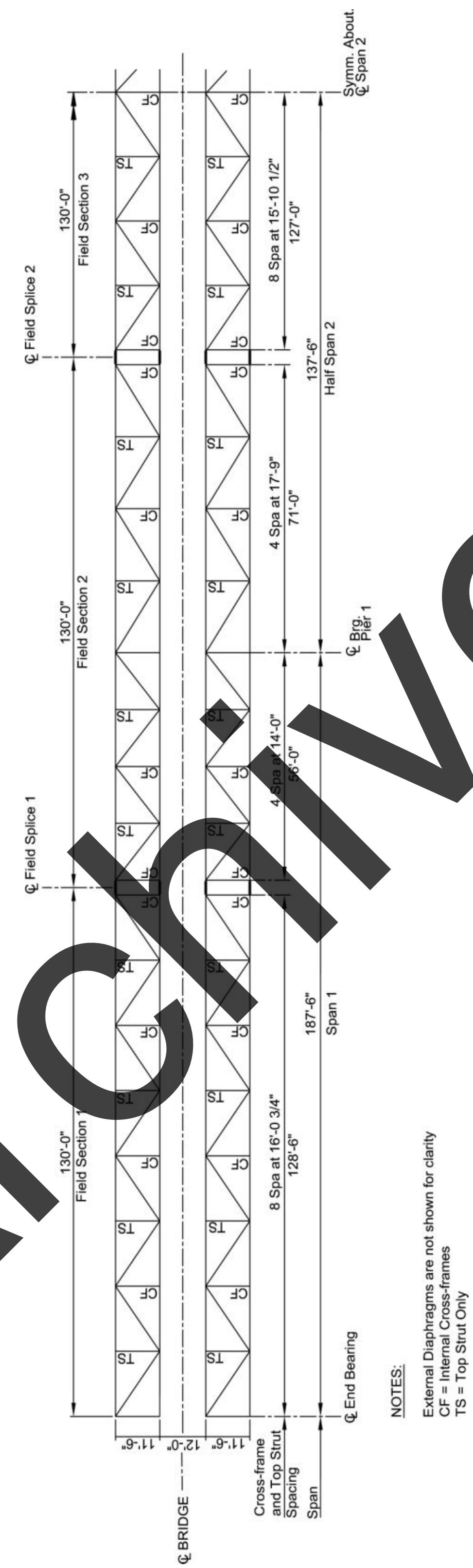

Figure 2 Sketch of the Framing Plan

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 criteria 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 in order to determine an appropriate optimal web depth based on minimum steel weight. This study considered various web depths and associated flange sizes that satisfied design requirements, in a preliminary sense. The optimal web depth was chosen from the preliminary design the 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 is used which results in a web plate size of 87.0 inches, using a 1:4 web inclination.

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 shall 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 bottom flange requirements to determine the optimal solution.

The maximum web inclination of 1:4 is used for this design example, so as to minimize the bottom flange width. Based on the previously mentioned web depth study, a vertical web depth of 84.4 inches is selected, resulting in a web plate size of 87.0 inches. This, in turn, provides a bottom flange width of 98.5 inches.

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. The web plate must be proportioned such that the web plate thickness (t_w) meets the requirement:

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

where D is the distance along the web. For inclined webs, Article 6.11.2.1.1 states that the distance along the web shall be used for all design checks. Eq. (6.11.2.1.2-1), revised from previous Specifications, simplifies proportioning of the web during preliminary design.

Rearranging:

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

Therefore, considering 1/16 inch increments for 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 flanges is specified as:

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

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

Constructibility provisions of Article 6.11.3.1 invoke the provisions of Article 6.10.3. For top flanges of composite girders in regions of positive flexure, Article C6.10.3.4 suggests the following additional guideline for the minimum compression-flange width (b_{fc}) be used in conjunction with flange proportion limits specified above. Although not implicitly intended for tub girders, this guideline is intended to provide more stable field pieces for handling during fabrication.

$$b_f \geq \frac{L}{85} \qquad \text{Eq. (C6.10.3.4-1)}$$

where L is the length of the girder shipping piece in feet. From Figure 3, the length of the longest field piece is 130 feet. Therefore, for this particular shipping piece:

$$\left(b_{fc}\right)_{min.} = \frac{L}{85} = \frac{130}{85} = 1.529\text{ ft} = 18.4\text{ in.}$$

A minimum top flange width of 18 inches will meet all required provisions and the intent of the L/85 guideline suggested above. Furthermore, it is advantageous to connect the top flange lateral bracing directly to the top flange. Therefore, to ensure that the flange is wide enough to accommodate the bolted connection, a minimum top flange width of 18 inches is proposed.

The minimum thickness of 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 *Guidelines for Design and Constructibility* [5] recommend 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 flange 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 top flange:

$$\frac{18}{2(0.75)} = 12.0 \leq 12.0$$

This example utilizes the provisions in *AASHTO LRFD (5th Edition, 2010)* to size the bottom flanges which imposes no limitations in regard to the b/t ratio of bottom flanges in tension. However, the design engineer should consider current industry practice regarding sizing the bottom flange of tub girders in positive moment regions. For positive moment regions, past and current literature has suggested a lower bound limit for the bottom flange thickness. These "rules of thumb" have suggested that a bottom flanges in tension have a maximum b/t ratio of 120, or an even more restrictive ratio of 80. These limits are intended to address several fabrication concerns, including waviness and warping effects during welding of the bottom flange to the webs. Additional discussion concerning this issue can be found in the NSBA publication *Practical Steel Tub Girder Design* [2].

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. Slenderness limits for the bottom tension flange have also been suggested to limit local vibrations, especially in very wide flanges that do not utilize any stiffening elements.

The engineer should consult with fabricators if it is determined that a bottom flange thickness that does not satisfy these previously discussed rules of thumb will be utilized in the final design of the structure. It should be verified that a tub girder with the selected bottom flange thickness can be fabricated without causing handling and distortion concerns. For this particular example, tension flange thicknesses that do not satisfy the b/t ratio of 120 are utilized, as they are allowed by the *AASHTO LRFD (5th Edition, 2010)*.

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

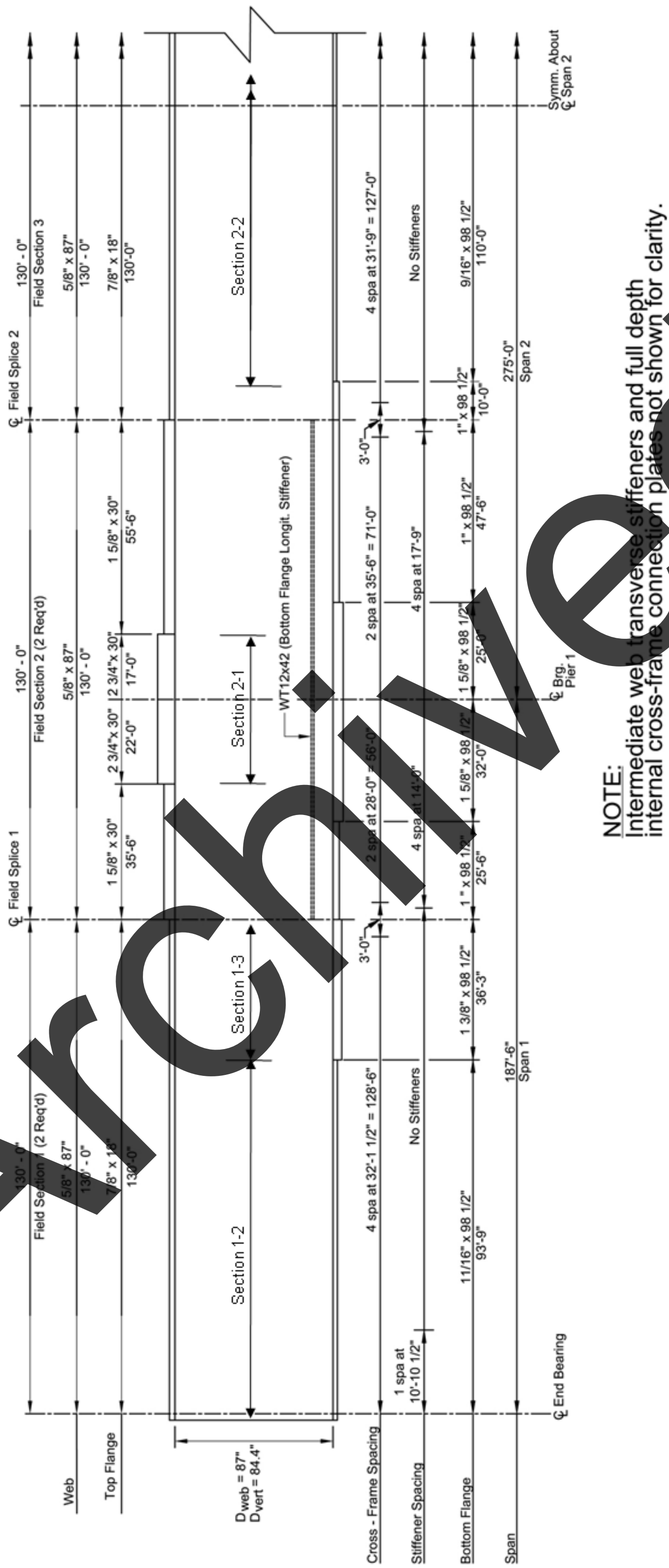

Figure 3 Sketch of the Girder Elevation

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 geometric restrictions specified in Article 6.11.2.3.

In particular:

- Bearing lines shall not be skewed.

- The distance center-to-center (a) of the top flanges of adjacent tubes, taken at mid-span, shall satisfy:

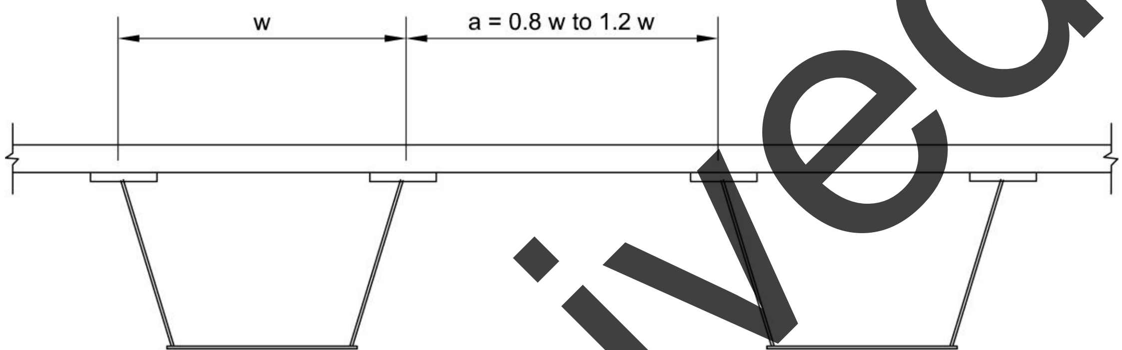

Figure 4 Center-to-Center Flange Distance

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 shall be the same.

- The inclination of the web shall 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.

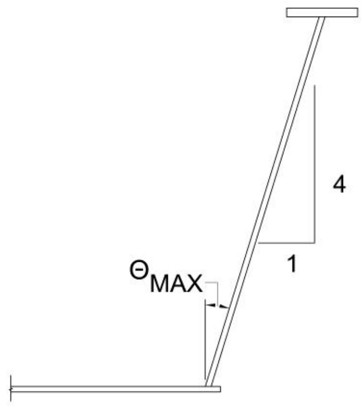

Figure 5 Maximum Web Inclination

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 ft $\leq$ a = 12 ft $\leq$ 1.2 (11.5) = 13.8 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:

- Non-composite dead load (DC_1) is the portion of loading resisted by the non-composite 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 $= \dfrac{9.5}{12}(43.0)(0.150) = 5.106\ kips/ft$ = 5.106 kips/ft

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

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

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

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

Steel girder self weight
(based on preliminary sizing and confirmed during subsequent analysis) = 1.876 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 generally 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.

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 = $[(0.025) \times 40] \div 2$ girders = 0.500 kips/ft per girder

For computing flexural stresses from 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 3*n* (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 considered to be 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 the HL-93 and consists of a combination of the following placed within each design lane:

- a design truck *or* design tandem.

- a design lane load.

The design vehicular live load is discussed in detail within Example 1.

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 detail within Example 1.

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 take into account all miscellaneous construction equipment that cannot be easily quantified at the time of design. Typically, load of 10 psf over the width of the bridge is used as the construction 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 constructibility. A detailed discussion of these limit states is provided within Example 1. 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 LRFD specifications, 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:

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

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 three η factors have default values of 1.0 specified at the strength limit state. At all other limit states, all three η factors must be taken equal to 1.0. For the purposes of this example, η_i will be taken equal to 1.0 at all limit states.

In this example, the STRENGTH I load combinations will be illustrated. Discussion regarding other load combinations is provided in Example 1. The SERVICE II load combination will be illustrated for permanent deflection checks.

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

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

And for Fatigue:

FATIGUE I: 1.50(LL+IM)

where LL is the fatigue load specified in Article 3.6.1.4.1.

It should be noted that when one force effect decreases another effect, minimum load factors shall be applied to the load reducing the total effect. Minimum load factors for permanent dead loads are specified in Table 3.4.1-2. For example, consider the Strength I 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, especially for structures that do not have an ideal span arrangement.

7.0 STRUCTURAL ANALAYSIS

Structural analysis is covered in Section 4 of the LRFD specifications. Both approximate and refined methods of analysis are discussed in the Specifications. Refined methods of analysis are given greater coverage in the LRFD specifications 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 will 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 tubes 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 is demonstrated in the Preliminary Girder Proportion section 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 live load distribution factors for straight tub girders utilized shear connectors throughout the negative flexure regions. Therefore Article 6.11.10 requires their use.

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

For the Strength Limit State, 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.

7.1.1 Live Load Distribution Factors (Article 4.6.2.2)

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 limit state and live-load deflection checks:

IM = 33% (Table 3.6.2.1-1)

Therefore, the factor applied to the static load shall 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.

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 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.

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. 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.

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.

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

7.2.1 Operational Live Load Deflection Evaluation (Article 3.6.1.3.2)

The LRFD Design Specifications permit, but do not mandate, the past practice for live load deflection control. However the specification does contain provisions for optional live load deflection criteria, to be invoked at the discretion of the Owner.

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 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 2.5.23.2 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. The bending stiffness of an individual girder may be taken as the stiffness, determined as described above, decided by the number of girders, in this case two girders. 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.

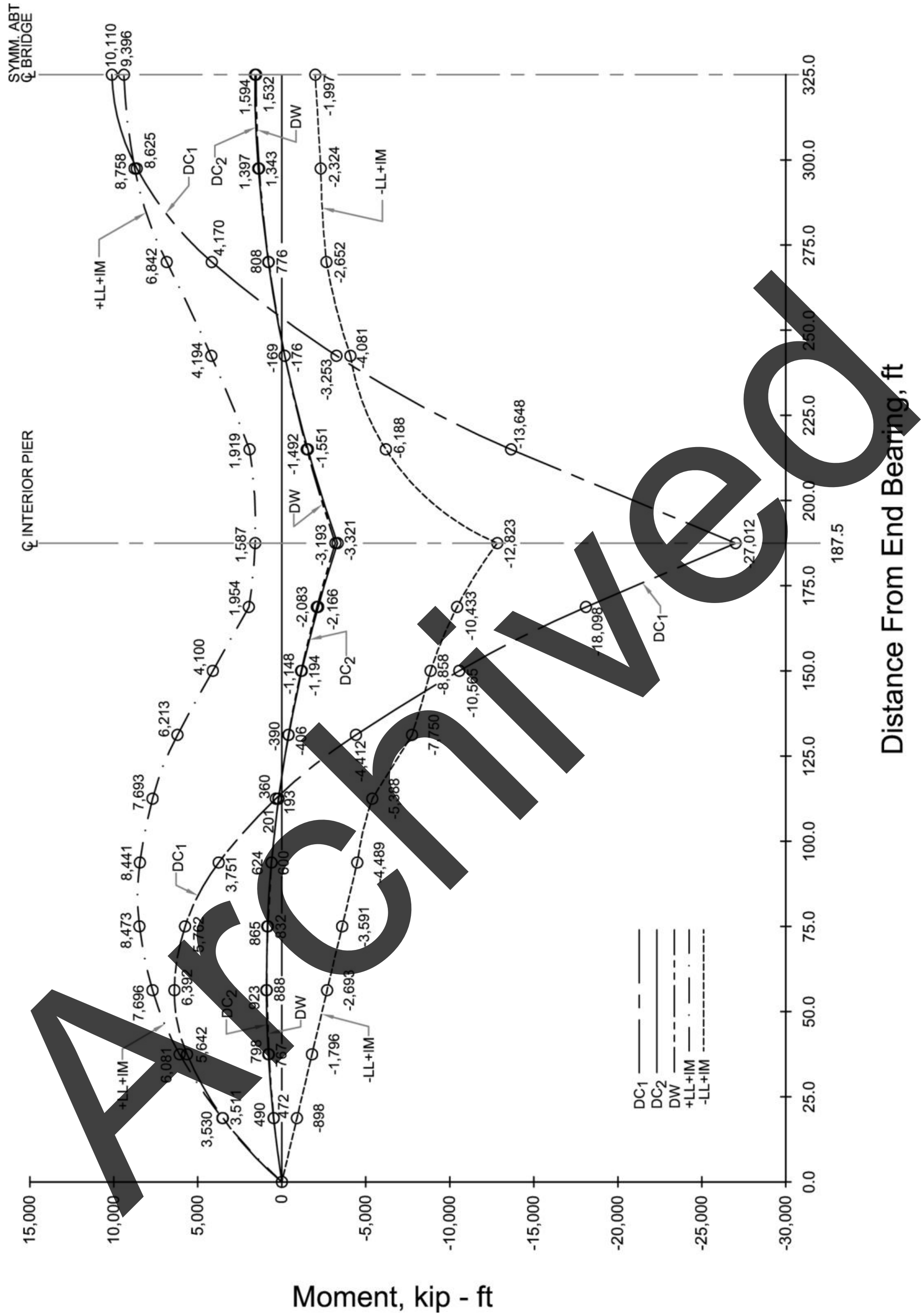

Figure 6 Graph showing the dead and live load moment envelopes

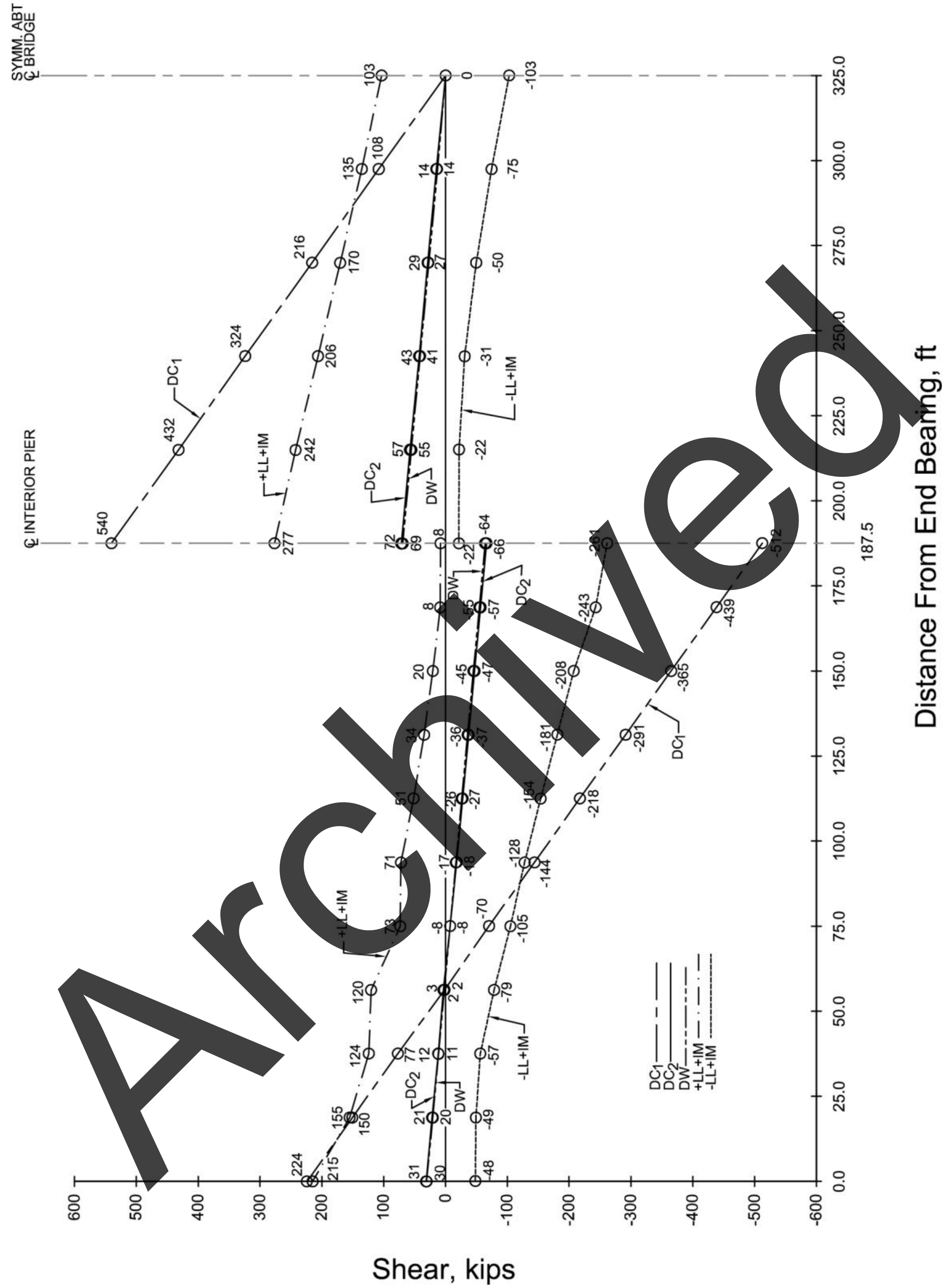

Figure 7 Graph showing the dead and live load shear envelopes

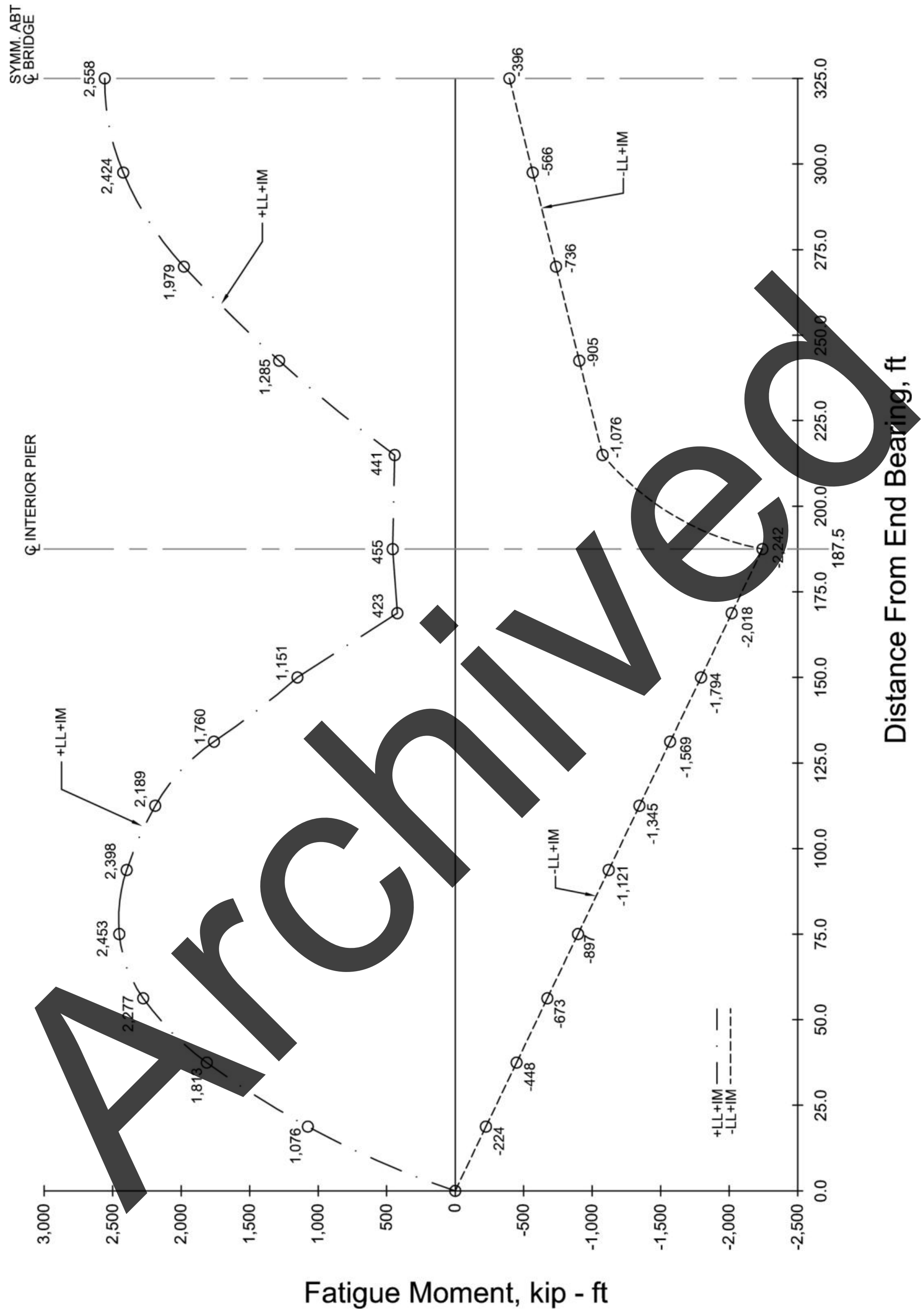

Figure 8 Graph showing the fatigue live load moment diagram

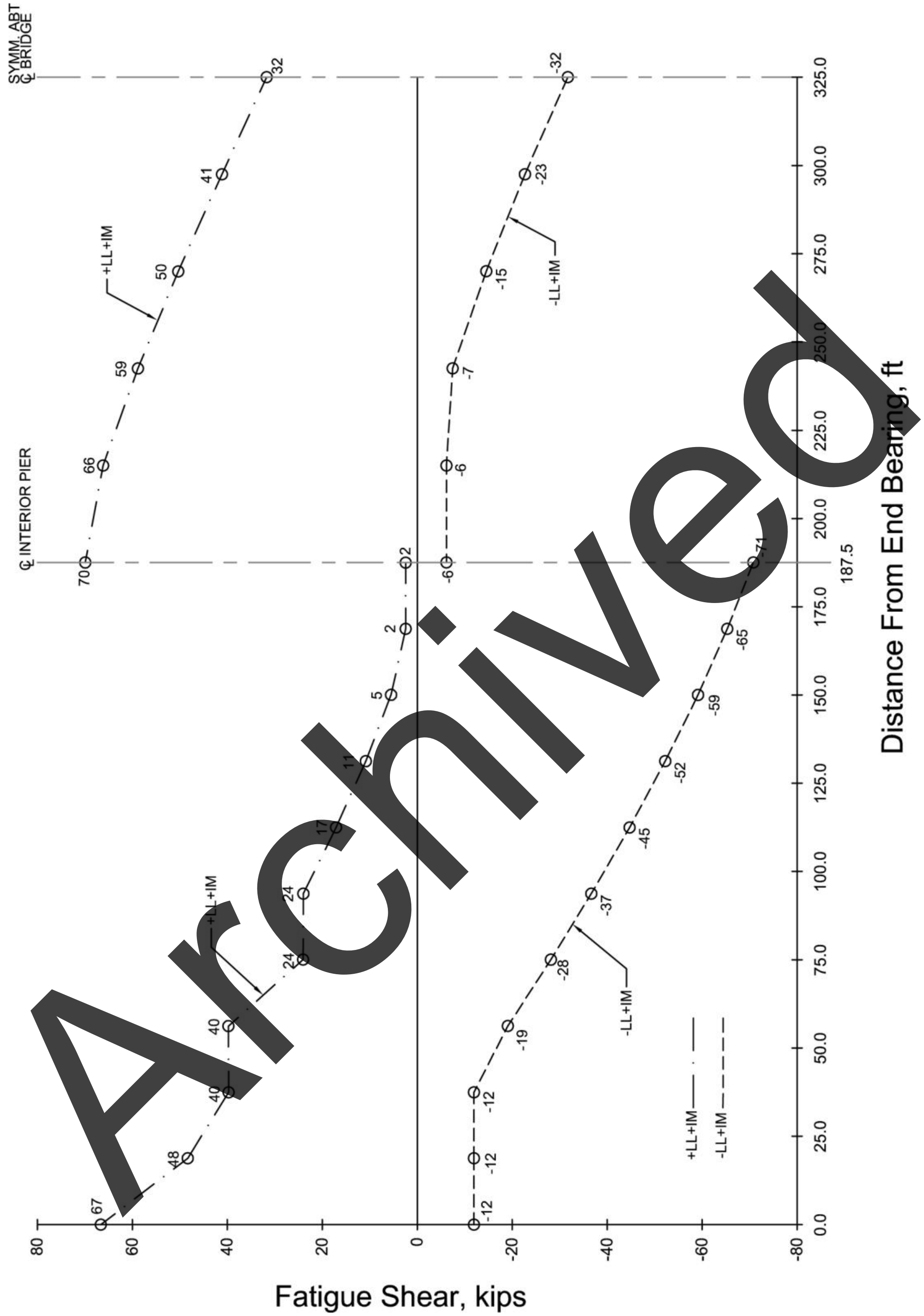

Figure 9 Graph showing the fatigue live load shear diagram

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 provisions contained in Article 6.11. The calculations include checks to be made at the Service and Strength Limit States. Detailed constructibility checks are also illustrated. Fatigue and Fracture Limit State checks, web-stiffener design, and the design of the stud shear connectors are not included in this example. Those provisions are illustrated in Example 1 and would be performed similarly for this particular example.

The calculations herein make use of the moment and shear envelopes shown in Figures 8 through 11 and the section properties calculated below. In the calculation of the vertical bending stress 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.

$$E_c = 33{,}000\, K_1\, w_c^{\,1.5} \sqrt{f'_c} \qquad \text{Eq. (5.4.2.4-1)}$$

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

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

Note that for normal-density concrete, Article 6.10.1.1.1b permits *n* to be taken as 8 for concrete with f'_c equal to 4.0 ksi. 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.

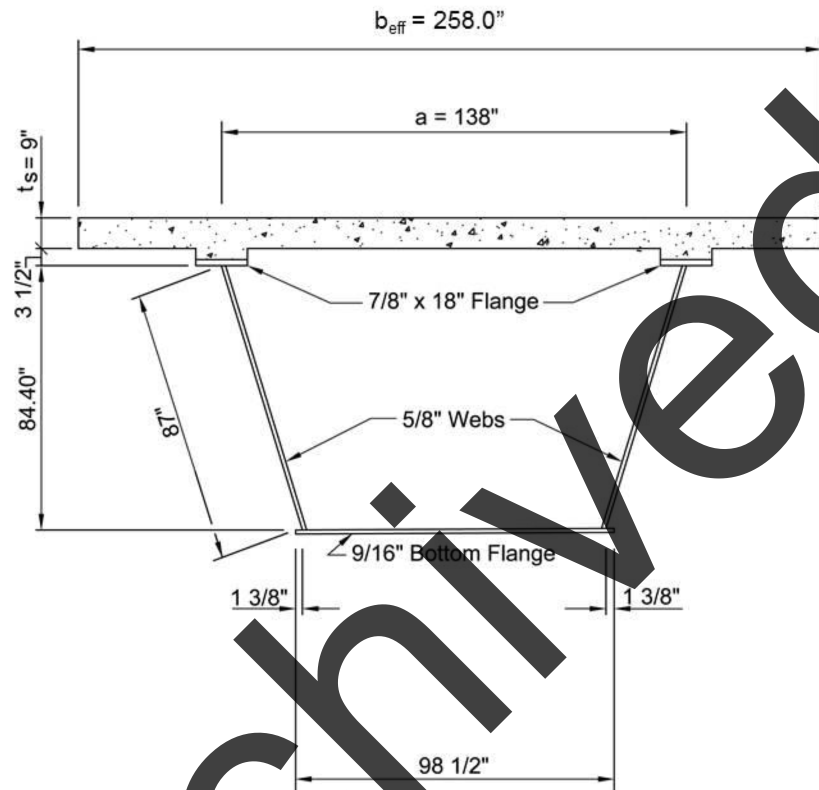

Figure 10 Sketch of Section 2-2

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 oof 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:

$$b_{eff} = 141.0 + 117.0 = 258.0 \text{ in.}$$

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

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

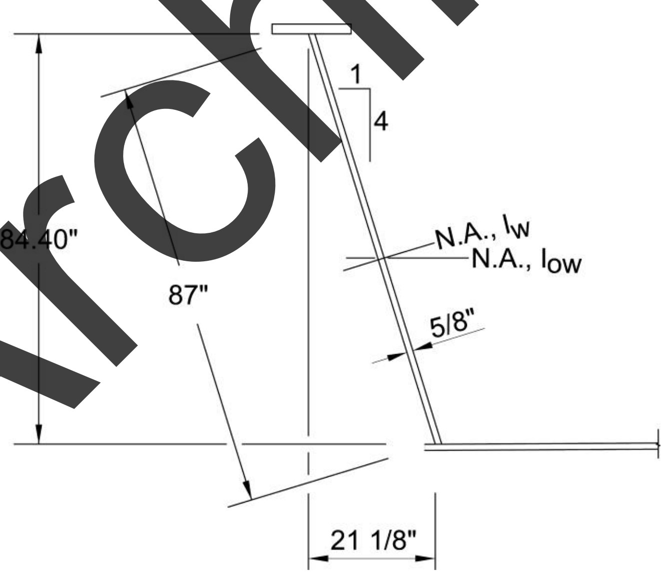

Figure 11 Moment of Inertia of an Inclined Web

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 7/8" x 18"	31.50	42.64	1,343	57,272	2.01	57,274
2 Webs 5/8" x 87"	108.75				64,560	64,560
Bottom Flange 9/16" x 98 ½"	55.41	-42.48	-2,354	99,990	1.46	99,991
	195.66		−1,011			221,825
					$-5.17(1{,}011) =$	−5,227
					$I_{NA} =$	216,598 in.^{4}

$$d_s = \frac{-1{,}011}{195.66} = -5.17 \text{ in.}$$

$$d_{TOP\,OF\,STEEL} = 43.08 + 5.17 = 48.25 \text{ in.} \qquad d_{BOT\,OF\,STEEL} = 42.76 - 5.17 = 37.59 \text{ in.}$$

$$S_{TOP\,OF\,STEEL} = \frac{216{,}598}{48.25} = 4{,}489 \text{ in.}^3 \qquad S_{BOT\,OF\,STEEL} = \frac{216{,}598}{37.59} = 5{,}762 \text{ in.}^3$$

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

Component	A	d	Ad	Ad^2	I_o	I
Steel Section	195.66		−1,011			221,825
Concrete Slab 9" x 258" /24	96.75	50.20	4,857	243,814	653.1	244,467
*Neglects Concrete Haunch	292.41		3,846			466,292
					$-13.15(3{,}846) =$	−50,575
					$I_{NA} =$	415,717 in.^{4}

$$d_{3n} = \frac{3{,}846}{292.41} = 13.15 \text{ in.}$$

$$d_{TOP\,OF\,STEEL} = 43.08 - 13.15 = 29.93 \text{ in.} \qquad d_{BOT\,OF\,STEEL} = 42.76 + 13.15 = 55.91 \text{ in.}$$

$$S_{TOP\,OF\,STEEL} = \frac{415{,}717}{29.93} = 13{,}889 \text{ in.}^3 \qquad S_{BOT\,OF\,STEEL} = \frac{415{,}717}{55.91} = 7{,}435 \text{ in.}^3$$

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

Component	A	d	Ad	Ad^2	I_O	I
Steel Section	195.66		−1,011			221,825
Concrete Slab 9" x 258" /8	290.25	50.20	14,571	731,442	1,959	733,401
*Neglects Concrete Haunch	485.91		13,560			955,226
				−27.91(13,560) =		−378,460
					I_{NA} =	576,766 in.^{4}

$$d_n = \frac{13,560}{485.91} = 27.91\text{ in.}$$

$$d_{TOPOFSTEEL} = 43.08 - 27.91 = 15.17\text{ in.} \qquad d_{BOTOFSTEEL} = 42.76 + 27.91 = 70.67\text{ in.}$$

$$S_{TOPOFSTEEL} = \frac{576,766}{15.17} = 38,020\text{ in.}^3 \qquad S_{BOTOFSTEEL} = \frac{576,766}{70.67} = 8,161\text{ in.}^3$$

*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 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 D of Section 6 in the Specification (Article D6.1). The longitudinal deck reinforcement is conservatively neglected. M_p is calculated for the tub girder as follows:

$P_t = F_{yt}\, b_t\, t_t$ = (50)(98.50)(0.5625) = 2,770 kips

$Pw = 2\, F_{yw}\, D\, t_w$ = (2)(50)(87.00)(0.625) = 5,438 kips

Pc = 2 Fyc bc tc = (2)(50)(18.0)(0.875) = 1,575 kips

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

$P_t + P_w < P_c + P_s$

8,208 kips < 9,470 kips; Therefore PNA is in the top flange, use Case II in Table D6-1.

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

$$\bar{y} = \frac{0.875}{2}\left[\frac{5{,}438 + 2{,}770 - 7{,}895}{1{,}575} + 1\right]$$
$$= 0.52 \text{ in. downward from the top of the top flange}$$

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

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

$$d_t = 0.875 + 84.4 + \frac{0.5625}{2} - 0.52 = 85.04 \text{ in.}$$
$$d_w = 0.875 + \frac{84.4}{2} - 0.52 = 42.56 \text{ in.}$$
$$d_s = \frac{9.0}{2} + 3.5 - 0.875 + 0.52 = 7.66 \text{ in.}$$

Calculate M_p:

$$M_p = \left[\frac{1{,}575}{2(0.875)}\right]\left[(0.52)^2 + (0.875 - 0.52)^2\right] + \left[(7{,}895)(7.66) + (5{,}438)(42.56) + (2{,}770)(85.04)\right]$$

M_p = 527,835 kip-in

M_p = 43,986 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 D of Section 6 in the Specification (Article D6.2.2). M_y is taken as the sum of the moments due to the factored loads 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):

$$50 = 1.0\left[\frac{1.25(10{,}110)(12)}{5{,}762} + \frac{1.25(1{,}594)(12) + 1.50(1{,}532)(12)}{7{,}435} + \frac{M_{AD}}{8{,}161}\right]$$

M_{AD} = 136,778 kip-in = 11,398 kip-ft

$M_y = M_{D1} + M_{D2} + M_{AD}$ Eq. (D6.2.2-1)

M_y = 1.0[1.25(10,110) + 1.25(1,594) + 1.50(1,532) + 11,398]

M_y = 28,326 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.

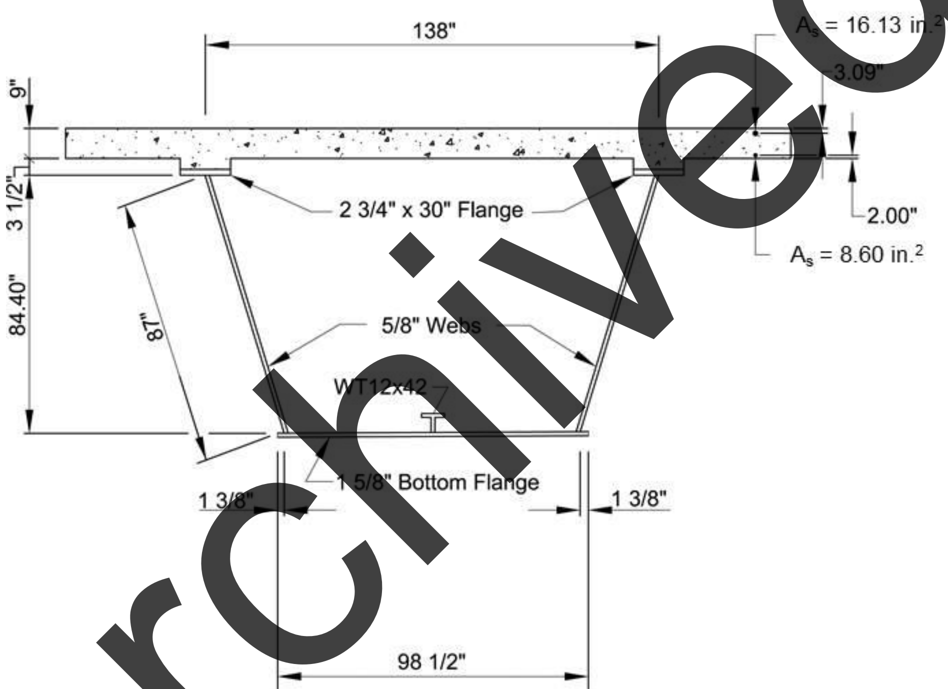

Figure 12 Sketch Showing Section 2-1

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:

$$b_{eff} = 141.0 + 117.0 = 258.0 \text{ in.}$$

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 5.4.2.6 and ϕ is taken as 0.90. It is further specified that the reinforcement is to have a specified minimum yield strength not less than 60 ksi and a size not exceeding 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 must 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: $\dfrac{0.31+0.44}{1.15}=0.65\approx\dfrac{2}{3}$

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, flexural stresses caused by the Fatigue and Service II loads, Article 6.6.1.2.1 and 6.10.4.2.1 respectively, applied to the composite section may be computed using the short-term or long-term composite section, as appropriate, assuming the concrete deck is fully effective in negative flexure regions. 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.

Table 5 Section 2-1: Steel Section Properties

Component	A	d	Ad	Ad^2	I$_o$	I
Top Flange 2 ¾" x 30"	165.00	43.58	7,191	313,371	104.0	313,475
Web 5/8" x 87"	108.75				64,560	64,560
Bottom Flange 1 5/8" x 98 ½"	160.06	−43.01	−6,884	296,089	35.22	296,124
Stiffener WT12x42	12.40	−33.07	−410.1	13,561	166.0	13,727
	446.2		−103.1			687,886
					−0.23(103.1) =	−23.7
					I_{NA} =	687,862 in.^{4}

$$d_s = \frac{-103.1}{446.2} = -0.23 \text{ in.}$$

$$d_{TOPOFSTEEL} = 44.95 + 0.23 = 45.18 \text{ in.} \qquad d_{BOTOFSTEEL} = 43.82 - 0.23 = 43.59 \text{ in.}$$

$$S_{TOPOFSTEEL} = \frac{687{,}862}{45.18} = 15{,}225 \text{ in.}^3 \qquad S_{BOTOFSTEEL} = \frac{687{,}862}{43.59} = 15{,}780 \text{ in.}^3$$

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

Component	A	d	Ad	Ad^2	I$_o$	I
Steel Section	446.2		−103.1			687,886
Long. Reinforcement	24.73	50.24	1,242	62,420		62,420
	470.3		1,138.9			750,306
					−2.42(1138.9) =	−2,756
					I_{NA} =	747,550 in.^{4}

$$d_{reinf} = \frac{1{,}138.9}{470.3} = 2.42 \text{ in.}$$

$$d_{TOPOFSTEEL} = 44.95 - 2.42 = 42.53 \text{ in.} \qquad d_{BOTOFSTEEL} = 43.82 + 2.42 = 46.24 \text{ in.}$$

$$S_{TOPOFSTEEL} = \frac{747{,}550}{42.53} = 17{,}342 \text{ in.}^3 \qquad S_{BOTOFSTEEL} = \frac{747{,}550}{46.24} = 16{,}167 \text{ in.}^3$$

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

Component	A	d	Ad	Ad²	I₀	I
Steel Section	446.2		−103.1			687,886
Concrete Slab 9" x 258"/24	96.75	50.2	4,857	243,814	653	244,467
	543.0		4,753.9			932,333
				$-8.75\,(4753.9) =$		−41,597
				$I_{NA} =$		890,736 in.⁴

$$d_{3n} = \frac{4,753.9}{543.0} = 8.75 \text{ in.}$$

$$d_{TOPOFSTEEL} = 44.95 - 8.75 = 36.20 \text{ in.} \qquad d_{BOTOFSTEEL} = 43.82 + 8.75 = 52.57 \text{ in.}$$

$$S_{TOPOFSTEEL} = \frac{890,736}{36.20} = 24,606 \text{ in.}^3 \qquad S_{BOTOFSTEEL} = \frac{890,736}{52.57} = 16,944 \text{ in.}^3$$

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

Component	A	d	Ad	Ad²	I₀	I
Steel Section	446.2		−103.1			687,886
Concrete Slab 9" x 258"/8	290.3	50.2	14,573	731,568	1,959	733,527
	736.5		14,470			1,421,413
				$-19.65(14,470) =$		−284,336
				$I_{NA} =$		1,137,077 in.⁴

$$d_n = \frac{14,470}{736.5} = 19.65 \text{ in.}$$

$$d_{TOPOFSTEEL} = 44.95 - 19.65 = 25.30 \text{ in.} \qquad d_{BOTOFSTEEL} = 43.82 + 19.65 = 63.47 \text{ in.}$$

$$S_{TOPOFSTEEL} = \frac{1,137,077}{25.30} = 44,944 \text{ in.}^3 \qquad S_{BOTOFSTEEL} = \frac{1,137,077}{63.47} = 17,915 \text{ in.}^3$$

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

Article 6.11.3 directs the engineer to Article 6.10.3 for the constructibility checks of tub girders. For critical stages of construction, the provisions of Articles 6.10.3.2.1 through 6.10.3.2.3 shall be applied to the top flanges of the tub girder. The noncomposite bottom tub flange in compression or tension shall satisfy requirements specified in Article 6.11.3.2. Web shear shall be checked in accordance with Article 6.10.3.3 with the shear shall be taken along the slope of the web in accordance with the provisions of Article 6.11.6.

For this example, a deck pour sequence is not investigated. The demonstration of a deck placement sequence, for the constructability checks, is shown in Example 1. In the absence of a deck pour sequence, the weight of the concrete deck is assumed to act in one stage. Furthermore, wind loads will not be considered for this example.

Calculate the maximum flexural stresses in the flanges of the steel section due to the factored loads resulting from the application of steel self-weight and the assumed full deck-placement (DC1). 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 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: $$(f_{bu})_{DC1} = \frac{1.0(1.25)(10{,}110)(12)}{4{,}489} = -33.78 \text{ ksi}$$

Bot. flange: $$(f_{bu})_{DC1} = \frac{1.0(1.25)(10{,}110)(12)}{5{,}762} = 26.32 \text{ ksi}$$

In addition to the applied steel, permanent metal deck forms, and concrete self-weight loads, it is pertinent to assume a construction live loading (CLL) on the structure during placement of the concrete deck, as discussed in the load calculations section. 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: $$(f_{bu})_{CLL} = \frac{1.0(1.5)(553)(12)}{4{,}489} = -2.22 \text{ ksi}$$

Bot. flange: $$(f_{bu})_{CLL} = \frac{1.0(1.5)(553)(12)}{5{,}762} = 1.73 \text{ ksi}$$

Top flange: f_{bu} = -33.78 + (-2.22) = -36.00 ksi

Bot. flange: f_{bu} = 26.32 +1.73 = 28.05 ksi

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 horizontal component of the web shear, as has been generally assumed in past practice

Fan and Helwig [6] and a greater portion of the lateral force is resisted in the top flanges. Fan and Helwig suggest that, with the exception of girder self-weight, the entire lateral forces 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:

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

The shear force used above is 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 C6.10.3.4-2, 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 0.875 inch x 18 inches top flange about a vertical axis through the web is:

$$S_f = \frac{(0.875)(18)^2}{6} = 47.25\ \text{in.}^3$$

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:

$$f_{LAT} = \frac{M_{LAT}}{S_f} = \frac{12(1.25)(10.29)}{47.25} = 3.27\ \text{ksi}$$

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:

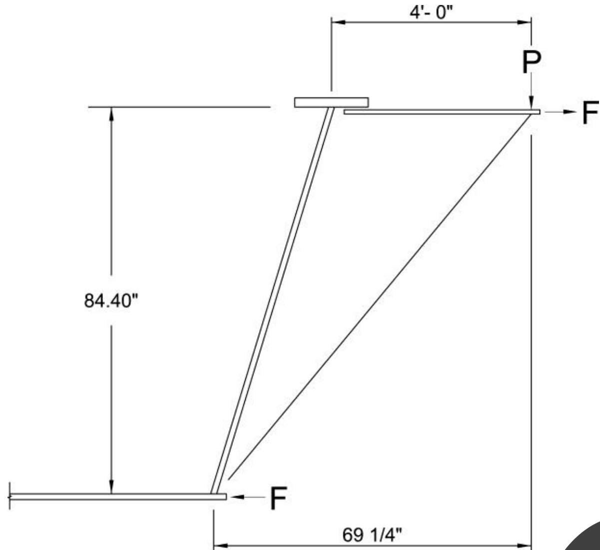

Figure 13 Sketch Showing Deck Overhang Bracket Loading

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:

$$P = 0.5*150\left[\left(4-\frac{18/12}{2}\right)\left(\frac{9.5}{12}\right)+\left(\frac{13-0.875}{12}\right)\left(\frac{18/12}{2}\right)+\frac{1}{2}\left(4-\frac{18/12}{2}\right)\left(\frac{(3.5+0.5)}{12}\right)\right] = 290 \text{ lbs/ft}$$

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 (referring to Figure 13) summation of the moments about the web-bottom flange junction:

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

$$F_{LAT} = (0.819)\,P$$

In the absence of a more refined analysis, the equations given in Article C6.10.3.4 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:

$$M_\ell = \frac{F_\ell L_b^2}{12} \qquad \text{Eq. (C6.10.3.4-2)}$$

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:

$$M_\ell = \frac{P_\ell L_b}{8} \qquad \text{Eq. (C6.10.3.4-3)}$$

In the STRENGTH I load combination; a load factor of 1.5 is applied to all construction loads (Article 3.4.2). The lateral bending stress in the bottom flange will be quite small as compared to the top flange, therefore bottom flange calculations are not shown for this particular example.

For STRENGTH I:

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

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

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

Top flange: $f_\ell = \dfrac{M_\ell}{S_\ell} = \dfrac{13.33(12)}{(0.875)(18)^2/6} = 3.39\ \text{ksi}$

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

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

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

$$\text{Top flange: } f_\ell = \frac{M_\ell}{S_\ell} = \frac{7.31(12)}{(0.875)(18)^2/6} = 1.86 \text{ ksi}$$

Deck Overhang Total:

$$f_\ell = 3.39 + 1.86 = 5.25 \text{ ksi}$$

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_ℓ, 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_ℓ may be determined as the stress at the section under consideration. For simplicity in this example, the largest value of f_ℓ within the unbraced length will conservatively be used in all design checks. f_ℓ is to be taken as positive in sign in all resistance equations. The unbraced length, L_b, 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:

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

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: $d_{\text{TOP OF STEEL}} = 48.25 \text{in.}$

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

$$D_c = (48.25 - 0.875)\sqrt{\frac{4^2 + 1}{4^2}}$$

D_c = 48.83 in.

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,

$$r_t = \frac{18}{\sqrt{12\left(1 + \frac{1}{3}\frac{48.83(0.625)}{18(0.875)}\right)}} = 4.05 \text{ in.}$$

$$L_p = \frac{1.0(4.05)}{12}\sqrt{\frac{29{,}000}{50}} = 8.13 \text{ ft}$$

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 constructibility. Finally, f_{bu} is the largest value of the compressive stress due to the factored loads throughout the unbraced length in the flange under consideration, calculated without consideration of flange lateral bending. In this case, use f_{bu} = 36.00 ksi, as computed earlier for the STRENGTH I load combination. Therefore:

$$1.2(8.13)\sqrt{\frac{1.0(1.0)}{36.00/50}} = 11.50 \text{ ft} < L_b = 15.875 \text{ ft}$$

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:

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

$$F_{cr} = \frac{1.0(1.0)\pi^2(29{,}000)}{\left(\frac{15.875(12)}{4.05}\right)^2} = 129.4 \text{ ksi}$$

The amplification factor is then determined as follows:

$$AF = \frac{0.85}{\left(1 - \frac{36.0}{129.4}\right)} = 1.18 > 1.0 \quad \text{ok}$$

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:

$$f_{lat} = (AF)[(f_{lat})_{\text{WEB SHEAR}} + (f_{\ell 1})_{\text{OVERHANG}}] = (1.18)[3.27 + 5.25] = 10.05 \text{ ksi}$$

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) in Article 6.10.1.6.

$$(0.6)F_{yf} = (0.6)(50) = 30 \text{ ksi} > 10.05 \text{ ksi} \qquad \text{OK}$$

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 top flange constructibility 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.

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

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

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

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

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

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 6.11.8.2 (i.e. local or lateral torsional buckling resistance, whichever controls). The provisions of Article A6.3.3 shall not 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 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:

$$\frac{2D_c}{t_w} \le 5.7\sqrt{\frac{E}{F_{yc}}} \qquad \text{Eq. (6.10.6.2.3-1)}$$

$$\frac{2D_c}{t_w} = \frac{2(48.83)}{0.625} = 156.3$$

$$5.7\sqrt{\frac{E}{F_{yc}}} = 5.7\sqrt{\frac{29{,}000}{50}} = 137.3 < 156.3$$

Therefore, the noncomposite Section 2-2 is a slender-web section. As a result, for the top flange, 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 flange:

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

$$\lambda_f = \frac{18}{2(0.875)} = 10.29$$

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

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

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

Determine the limiting slenderness ratio for a noncompact flange:

$$\lambda_{rf} = 0.56\sqrt{\frac{E}{F_{yc}}} \qquad \text{Eq. (6.10.8.2.2-5)}$$

$$\lambda_{rf} = 0.56\sqrt{\frac{29{,}000}{50}} = 13.49$$

Since $\lambda_f > \lambda_{pf}$, but $\lambda_f \leq \lambda_{rf}$

$$F_{nc} = \left[1-\left(1-\frac{F_{yr}}{R_h F_{yc}}\right)\left(\frac{\lambda_f-\lambda_{pf}}{\lambda_{rf}-\lambda_{pf}}\right)\right]R_b R_h F_{yc} \qquad \text{Eq. (6.10.8.2.2-2)}$$

Where F_{yr} is the compression flange stress at the onset on nominal yielding, and shall 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,

$$F_{yr} = 0.7(50) = 35\text{ ksi}$$

As specified in Article C6.10.3.2.1, when computing F_{nc} for constructibility, 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,

$$F_{nc} = \left[1-\left(1-\frac{35}{(1.0)(50)}\right)\left(\frac{10.29-9.15}{13.49-9.15}\right)\right](1.0)(1.0)(50) = 46.06\text{ ksi}$$

8.2.4.2 Top Flange - Lateral Torsional Buckling Resistance

The limiting unbraced length, L_p, was computed earlier to be 8.13 feet. The effective radius of gyration for lateral torsional buckling, r_t, for the noncomposite Section 2-2 was also computed earlier to be 4.05 inches. The computations for L_p an r_t are shown in a previous section discussing the top flange lateral bending amplification.

Determine the limiting unbraced length, L_r:

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

$$L_r = \frac{\pi(4.05)}{12}\sqrt{\frac{29{,}000}{35.0}} = 30.52 \text{ ft}$$

Since L_p = 8.13 feet < L_b = 15.875 feet < L_r = 30.52 feet,

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

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

$$F_{nc} = 1.0\left[1-\left(1-\frac{35.0}{1.0(50)}\right)\left(\frac{15.875-8.13}{30.52-8.13}\right)\right](1.0)(1.0)(50) = 44.81 \text{ ksi} < 1.0(1.0)(50) = 50 \text{ ksi} \quad \text{ok}$$

F_{nc} is governed by the lateral torsional buckling resistance, which is less than the local buckling resistance of 46.06 ksi computed earlier. Therefore, F_{nc} = 44.81 ksi.

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:

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

$$k = \frac{9}{(48.83/87.0)^2} = 28.58$$

Therefore,

$$F_{crw} = \frac{0.9(29{,}000)(28.58)}{\left(\frac{87.0}{0.625}\right)^2} = 38.50 \text{ ksi} < R_h\, F_{yc} = 50 \text{ ksi} \qquad \text{ok}$$

8.2.4.4 Top Flange Constructibility Checks

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

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

$$f_{bu} + f_{\ell} = 36.00 \text{ ksi} + 10.05 \text{ ksi} = 46.05 \text{ ksi}$$

$$\phi_f R_h F_{yc} = 1.0(1.0)(50) = 50.0 \text{ ksi}$$

$$46.05 \text{ ksi} < 50.0 \text{ ksi} \quad \text{ok} \qquad (\text{Ratio} = 0.921)$$

$$f_{bu} + \frac{1}{3} f_{\ell} \leq \phi_f F_{nc} \qquad \text{Eq. (6.10.3.2.1-2)}$$

$$f_{bu} + \frac{1}{3} f_{\ell} = 36.0 \text{ ksi} + \frac{10.05}{3} \text{ ksi} = 39.35 \text{ ksi}$$

$$\phi_f F_{nc} = 1.0(44.83) = 44.83 \text{ ksi}$$

$$39.35 \text{ ksi} < 44.81 \text{ ksi} \quad \text{ok} \qquad (\text{Ratio} = 0.878)$$

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

$$\phi_f F_{crw} = 1.0(38.50) = 38.50 \text{ ksi}$$

$$36.0 \text{ ksi} < 38.50 \text{ ksi} \quad \text{ok} \qquad (\text{Ratio} = 0.935)$$

8.2.4.5 Bottom Flange Constructibility Checks

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

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

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 St. Venant torsional shear stress in the flange due to factored loads 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 particular case ($f_v = 0$), and:

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

The longitudinal flange stress, calculated previously, is:

$$f_{bu} = 28.05 \text{ ksi}$$
$$\phi_f R_h F_{yf} \Delta = 1.0(1.0)(50)(1.0) = 50.0 \text{ ksi}$$
$$28.05 \text{ ksi} < 50.0 \text{ ksi} \quad \text{ok} \qquad (\text{Ratio} = 0.561)$$

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:

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

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)

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 are already limited to the shear buckling resistance at the strength limit state.

For this example, the web is unstiffened in the positive moment regions. Therefore, the constructibilty 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 sections 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) in the hardened deck. When cracking is predicted, longitudinal deck reinforcing as specified in Article 6.10.1.7 is required to control cracking. Otherwise, alternative deck casting sequences must be employed to minimize the anticipated stresses to acceptable levels.

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 must satisfy the following requirements:

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

Bottom flange of composite sections: $f_f + \dfrac{f_\ell}{2} \leq 0.95 R_h F_{yf}$ 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) shall be taken equal to zero, 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 and 6.10.2.1.1 ($D/t_w \leq 150$), 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 B shall not apply to tub girder sections. The applicability of the Appendix B 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: $f_f \le 0.95R_hF_{yf}$ Eq. (6.10.4.2.2-1)

$$f_f = 1.0\left[\frac{1.0(10{,}110)}{4{,}489} + \frac{1.0(1{,}594 + 1{,}532)}{13{,}889} + \frac{1.3(9{,}396)}{38{,}020}\right]12 = -33.58 \text{ ksi}$$

$$|-33.58| \text{ ksi} < 47.50 \text{ ksi} \quad \text{ok} \quad (\text{Ratio} = 0.734)$$

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

$$f_f = 1.0\left[\frac{1.0(10{,}110)}{5{,}762} + \frac{1.0(1{,}594 + 1{,}532)}{7{,}435} + \frac{1.3(9{,}396)}{8{,}161}\right]12 + \frac{0}{2} = 44.06 \text{ ksi}$$

$$44.06 \text{ ksi} < 47.50 \text{ ksi} \quad \text{ok} \quad (\text{Ratio} = 0.935)$$

8.4 Girder Fracture and Fatigue 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 and the fatigue live load specified in Article 3.6.1.4 shall be employed for checking load-induced fatigue in tub girder sections. The Fatigue I load combination is for infinite fatigue life and will typically be used in new designs. The Fatigue II load combination is for design checks assuming a finite fatigue life.

One additional requirement specified particularly for tub girders sections is in regard to longitudinal warping and transverse bending stresses. When tub girders are subjected to eccentric loads, 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. Therefore, longitudinal warping stresses and transverse bending stresses due to cross-section distortion shall be considered for:

- Single tub girder 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
- Any single or multiple tub girder with a tub flange that is not fully effective according to the provisions of Article 6.11.1.1.

When required, the stress range due to longitudinal warping shall 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. The transverse bending stress range shall be considered separately in evaluating 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. The transverse bending range shall consider a cycle of stress defined as 75 percent of the stress range determined by the passage of the factored fatigue live load in two different transverse positions. However, in no case shall this calculated stress range be less than the stress range due to a single passage of the factored fatigue load.

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, and 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 shall 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.

The fatigue details employed in this example, 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 Example 1.

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. For single tub girders, in accordance with Article 6.11.5, bottom flanges in tension shall be considered fracture critical, unless analysis shows that the section can support the full dead load and an appropriate portion of the live load after sustaining a hypothetical complete fracture of the bottom flange and webs at any point. Furthermore, for cross-sections comprised of two tub girders, the bottom flanges in positive moment regions should be designated as fracture critical, unless adequate strength and stability of a damaged structure can be verified by a refined analysis. Article C6.6.2 provides discussion in regard to the use of refined analyses to demonstrate that part of a structure is not fracture critical. If a cross-section contains more than two tub girders, none of the components need be considered fracture critical.

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 6.6.2-2) specified for Temperature Zone 2 (Table 6.6.2-1).

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 (Special Restrictions for use of live load distribution factors)

4) the tub flange (bottom flange) is fully effective as specified in Article 6.11.1.1 (i.e. bottom flange b_f less than one-fifth 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:

$$D_p = 9.0 + 3.5 - 0.875 + 0.52 = 12.15 \text{ in.}$$

$$D_t = 0.5625 + 84.4 + 3.5 + 9.0 = 97.46 \text{ in.}$$

$$0.42D_t = 0.42(97.46) = 40.93 \text{ in.} > 12.15 \text{ in.} \quad \text{ok} \quad (\text{Ratio} = 0.297)$$

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 be:

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

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 = 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 shall be taken as specified in Article 6.10.7.1.2, except that for continuous spans, the nominal flexural resistance shall always be subject to the limitation of Eq. (6.10.7.1.2-3). 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:

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

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 28,326 kip-ft and 43,986 kip-ft, respectively.

$$0.1D_t = 0.1(97.46) = 9.75 \text{ in.} < D_p = 12.15 \text{ in.}$$

Therefore, $$M_n = 43{,}986\left[1.07 - 0.7\left(\frac{12.15}{97.46}\right)\right] = 43{,}227 \text{ kip} - \text{ft}$$

Or, $$M_n = 1.3(1.0)(28{,}326) = 36{,}824 \text{ kip} - \text{ft} \qquad \text{(governs)}$$

Therefore: M_n = 36,824 kip-ft

For STRENGTH I:

$$M_u = 1.25(10{,}110 + 1{,}594) + 1.5(1{,}532) + 1.75(9{,}396) = 33{,}371 \text{ kip} - \text{ft}$$
$$\phi_f M_n = 1.0(36{,}824) = 36{,}824 \text{ kip} - \text{ft}$$
$$33{,}371 \text{ kip} - \text{ft} < 36{,}824 \text{ kip} - \text{ft} \quad \text{ok} \qquad (\text{Ratio} = 0.906)$$

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 shall 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:

$$V_u \leq \phi_v V_n \qquad \text{Eq. (6.10.9.1-1)}$$

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} = shear in a single web at the section under consideration due to the factored loads.

A flow chart for determining the shear resistance of I-sections is shown in Figure C6.10.9.1-1. Steel Bridge Design Handbook Design Example 1 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 Steel Bridge Design Handbook 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:

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

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

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

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)

According to Article 6.10.9.3.3, the nominal shear resistance of a web end panel is limited to the 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 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.

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

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 webs stiffeners are not required beyond the end panel, in the positive moment region. The STRENGTH I factored shear load in one web at 10.75 feet from the abutment is 327 kips (i.e. V_{ui} = 327 kips at 1.5D). Since the factored shear load, 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 regions.

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

8.6.1 Flexure (Article 6.11.3.2)

The bottom flange, in regions of negative flexure, shall 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. In regard to construction loads, 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 pour.

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

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

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

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

As stated previously, the deck pour sequence and the application of wind loads are not considered in this example. It is assumed, for this example, that the application of the concrete deck occurs all at once for the purpose of the constructibility checks.

Calculate the maximum flexural stresses in the flanges of the steel section due to the factored loads resulting from the application of steel self-weight and the assumed full deck-placement (DC1).

For STRENGTH I:

Top flange: $(f_{bu})_{DC1} = \dfrac{1.0(1.25)(27{,}012)(12)}{15{,}225} = 26.61 \text{ ksi}$

Bot. flange: $(f_{bu})_{DC1} = \dfrac{1.0(1.25)(27{,}012)(12)}{15{,}780} = -25.68 \text{ ksi}$

In addition to the applied steel and concrete self-weight loads, it is pertinent to assume a construction live loading (CLL) on the structure during placement of the concrete deck, as discussed in the load calculations section. 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:

Top flange: $(f_{bu})_{CLL} = \dfrac{1.0(1.5)(1{,}478)(12)}{15{,}225} = 1.75 \text{ ksi}$

Bot. flange: $(f_{bu})_{CLL} = \dfrac{1.0(1.5)(1,478)(12)}{15,780} = -1.69 \text{ ksi}$

Top flange: $f_{bu} = 26.61 + 1.75 = 28.36 \text{ ksi}$

Bot. flange: $f_{bu} = -25.68 + (-1.69) = -27.37 \text{ ksi}$

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 as demonstrated 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: $f_{lat} = 0.37$ ksi

f_{lat} due to cantilever deck overhang bracket: $f_{lat} = 0.71$ ksi

Total Top Flange $f_{lat} = 0.37 + 0.71 = 1.08$ ksi

8.6.1.2 Top Flange Constructibility Check

Checking compliance with Article 6.10.3.2.2:

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

For STRENGTH I:

$f_{bu} + f_{\ell} = 28.36 \text{ ksi} + 1.08 \text{ ksi} = 29.44 \text{ ksi}$

$\phi_f R_h F_{yc} = 1.0(1.0)(50) = 50.0 \text{ ksi}$

$29.44 \text{ ksi} < 50.0 \text{ ksi}$ ok $(\text{Ratio} = 0.588)$

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 constructibility, the web load-shedding factor, R_b, shall 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. Therefore, Article 6.11.8.2.3 applies.

Determine the slenderness ratio of the bottom flange:

$$\lambda_f = \frac{b_{fc}}{t_{fc}} \qquad \text{Eq. (6.11.8.2.2-4)}$$

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 particular case, since the longitudinal stiffener is at the center of the bottom flange, *w* is the distance from the longitudinal stiffener to the centerline of the web.

$$\lambda_f = \frac{(95.125)/2}{1.625} = 29.27$$

Calculate the first limiting slenderness ratio:

$$R_1 \sqrt{\frac{kE}{F_{yc}}}$$

where:

$$R_1 = \frac{0.57}{\sqrt{\frac{1}{2}\left[\Delta + \sqrt{\Delta^2 + 4\left(\frac{f_v}{F_{yc}}\right)^2 \left(\frac{k}{k_S}\right)^2}\right]}} \qquad \text{Eq. (6.11.8.2.2-8)}$$

and where:

$$\Delta = \sqrt{1 - 3\left(\frac{f_v}{F_{yf}}\right)^2} \qquad \text{Eq. (6.11.8.2.2-5)}$$

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

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

and,

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

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, $1.0 \leq k \leq 4.0$
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:

$$I_s = 166 + 12.4(9.08)^2 = 1188 \text{ in.}^4$$

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:

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

Furthermore,

$$k = \left(\frac{8\,(1188)}{(47.56)\,(1.625)^3}\right)^{\frac{1}{3}} = 3.60 \leq 4.0$$

$$k_S = \frac{5.34 + 2.84\left(\dfrac{1{,}188}{(47.56)\,(1.625)^3}\right)^{\frac{1}{3}}}{(1+1)^2} = 2.61 \leq 5.34$$

Therefore,

$$R_1 = \frac{0.57}{\sqrt{\dfrac{1}{2}\left[(1.0) + \sqrt{(1.0)^2 + 4\left(\dfrac{0}{50}\right)^2\left(\dfrac{3.60}{2.61}\right)^2}\right]}} = 0.57$$

$$R_1\sqrt{\frac{kE}{F_{yc}}} = 0.57\sqrt{\frac{(3.60)(29{,}000)}{50}} = 26.05$$

Since λ_f is > 26.05, it is necessary to compute the second limiting slenderness ratio:

$$R_2\sqrt{\frac{kE}{F_{yc}}}$$

where:

$$R_2 = \frac{1.23}{\sqrt{\frac{1}{1.2}\left[\frac{F_{yr}}{F_{yc}} + \sqrt{\left(\frac{F_{yr}}{F_{yc}}\right)^2 + 4\left(\frac{f_v}{F_{yc}}\right)^2\left(\frac{k}{k_S}\right)^2}\right]}} \qquad \text{Eq. (6.11.8.2.2-9)}$$

and where:

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

$$F_{yr} = (1.0 - 0.4)(50) = 30.0\text{ ksi} \le 50.0\text{ ksi}$$

Therefore,

$$R_2 = \frac{1.23}{\sqrt{\frac{1}{1.2}\left[\frac{30}{50} + \sqrt{\left(\frac{30}{50}\right)^2 + 4\left(\frac{0}{50}\right)^2\left(\frac{3.60}{2.61}\right)^2}\right]}} = 1.23$$

$$R_2\sqrt{\frac{kE}{F_{yc}}} = 1.23\sqrt{\frac{(3.60)(29{,}000)}{50}} = 56.20$$

Since $\lambda_f > R_1\sqrt{\frac{kE}{F_{yc}}}$, but $\lambda_f \le R_2\sqrt{\frac{kE}{F_{yc}}}$:

$$F_{nc} = R_b R_h F_{yc}\left[\Delta - \left(\Delta - \frac{F_{yr}}{R_h F_{yc}}\right)\left\{1 - \sin\left[\frac{\pi}{2}\left(\frac{R_2 - \frac{w}{t_{fc}}\sqrt{\frac{F_{yc}}{kE}}}{R_2 - R_1}\right)\right]\right\}\right] \qquad \text{Eq. (6.11.8.2.2-2)}$$

Note that since the bottom flange is stiffened, w is substituted for b_{fc} in Eq. (6.11.8.2.2-2).

$$F_{nc} = (1.0)(1.0)(50)\left[1.0 - \left(1.0 - \frac{30}{(1.0)(50)}\right)\left\{1 - \sin\left[\frac{\pi}{2}\left(\frac{1.23 - \frac{(95.125/2)}{1.625}\sqrt{\frac{50}{(3.60)(29{,}000)}}}{1.23 - 0.57}\right)\right]\right\}\right]$$

$$F_{nc} = 49.72 \text{ ksi}$$

For STRENGTH I:

$$f_{bu} = -27.37 \text{ ksi}$$
$$\phi_f F_{nc} = 1.0(49.72) = 49.72 \text{ ksi}$$
$$\left|-27.37\right| \text{ ksi} < 49.72 \text{ ksi} \quad \text{ok} \qquad (\text{Ratio} = 0.550)$$

8.6.1.4 Web Bend-Buckling

The web bend-buckling resistance shall 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:

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

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

$$k = \frac{9}{(43.26/87.0)^2} = 36.40$$

Therefore,

$$F_{crw} = \frac{0.9(29{,}000)(36.40)}{\left(\frac{87.0}{0.625}\right)^2} = 49.03 \text{ ksi} < R_h F_{yc} = 50 \text{ ksi} \quad \text{ok}$$

For STRENGTH I:

$$f_{bu} = -27.37 \text{ ksi}$$
$$\phi_f F_{crw} = 1.0(49.03) = 49.03 \text{ ksi}$$
$$\left|-27.37\right| \text{ ksi} < 49.03 \text{ ksi} \quad \text{ok} \qquad (\text{Ratio} = 0.558)$$

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:

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

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 load will include the contribution of noncomposite dead load (DC_1) and the construction live loading (CLL). Note that the shear loads used in the following calculation are 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:

$$V_p = 0.58F_{yw}Dt_w \qquad \text{Eq. (6.10.9.3.2-2)}$$

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

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

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

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

8.7 Girder Strngth 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 A shall not apply, nor is redistribution of negative moment per Appendix B.

At the strength limit state, tub flanges (bottom flanges) in compression shall satisfy:

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

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 continuously braced by the deck, shall satisfy:

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

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

Compute the maximum flange flexural stresses at Section 2-1 due to the factored loads under the STRENGTH I Limit State, 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:

$$f_{bu} = 1.0\left[\frac{1.25(-27{,}012)}{15{,}225} + \frac{1.25(-3{,}321)}{17{,}577} + \frac{1.5(-3{,}193)}{17{,}577} + \frac{1.75(-12{,}823)}{17{,}577}\right]12 = 48.04 \text{ ksi}$$

Bottom flange:

$$f_{bu} = 1.0\left[\frac{1.25(-27{,}012)}{15{,}780} + \frac{1.25(-3{,}321)}{16{,}167} + \frac{1.5(-3{,}193)}{16{,}167} + \frac{1.75(-12{,}823)}{16{,}167}\right]12 = -48.97 \text{ ksi}$$

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:

$$\lambda_f = \frac{b_{fc}}{t_{fc}} \qquad \text{Eq. (6.11.8.2.2-4)}$$

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 particular case, since the longitudinal stiffener is at the center of the bottom flange, *w* is the distance from the longitudinal stiffener to the centerline of the web.

$$\lambda_f = \frac{(95.125)/2}{1.625} = 29.27$$

Calculate the first limiting slenderness ratio:

$$R_1\sqrt{\frac{kE}{F_{yc}}}$$

where:

$$R_1 = \frac{0.57}{\sqrt{\frac{1}{2}\left[\Delta + \sqrt{\Delta^2 + 4\left(\frac{f_v}{F_{yc}}\right)^2\left(\frac{k}{k_S}\right)^2}\right]}} \qquad \text{Eq. (6.11.8.2.2-8)}$$

and where:

$$\Delta = \sqrt{1 - 3\left(\frac{f_v}{F_{yf}}\right)^2} \qquad \text{Eq. (6.11.8.2.2-5)}$$

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

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

and,

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

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 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 fv is zero:

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

Furthermore,

$$k = \left(\frac{8\,(1188)}{(47.56)\,(1.625)^3}\right)^{\frac{1}{3}} = 3.60 \le 4.0$$

$$k_S = \frac{5.34 + 2.84\left(\frac{1{,}188}{(47.56)\,(1.625)^3}\right)^{\frac{1}{3}}}{(1+1)^2} = 2.61 \le 5.34$$

Therefore,

$$R_1 = \frac{0.57}{\sqrt{\frac{1}{2}\left[(1.0) + \sqrt{(1.0)^2 + 4\left(\frac{0}{50}\right)^2\left(\frac{3.60}{2.61}\right)^2}\right]}} = 0.57$$

$$R_1\sqrt{\frac{kE}{F_{yc}}} = 0.57\sqrt{\frac{(3.60)(29{,}000)}{50}} = 26.05$$

Since λ_f is > 26.05, it is necessary to compute the second limiting slenderness ratio:

$$R_2\sqrt{\frac{kE}{F_{yc}}}$$

where:

$$R_2 = \frac{1.23}{\sqrt{\frac{1}{1.2}\left[\frac{F_{yr}}{F_{yc}} + \sqrt{\left(\frac{F_{yr}}{F_{yc}}\right)^2 + 4\left(\frac{f_v}{F_{yc}}\right)^2\left(\frac{k}{k_S}\right)^2}\right]}} \qquad \text{Eq. (6.11.8.2.2-9)}$$

and where:

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

$$F_{yr} = (1.0 - 0.4)(50) = 30.0\text{ ksi} \le 50.0\text{ ksi}$$

Therefore,

$$R_2 = \frac{1.23}{\sqrt{\frac{1}{1.2}\left[\frac{30}{50} + \sqrt{\left(\frac{30}{50}\right)^2 + 4\left(\frac{0}{50}\right)^2\left(\frac{3.60}{2.61}\right)^2}\right]}} = 1.23$$

$$R_2\sqrt{\frac{kE}{F_{yc}}} = 1.23\sqrt{\frac{(3.60)(29{,}000)}{50}} = 56.20$$

Since $\lambda_f > R_1\sqrt{\frac{kE}{F_{yc}}}$, but $\lambda_f \leq R_2\sqrt{\frac{kE}{F_{yc}}}$:

$$F_{nc} = R_b R_h F_{yc}\left[\Delta - \left(\Delta - \frac{F_{yr}}{R_h F_{yc}}\right)\left\{1 - \sin\left[\frac{\pi}{2}\left(\frac{R_2 - \frac{w}{t_{fc}}\sqrt{\frac{F_{yc}}{kE}}}{R_2 - R_1}\right)\right]\right\}\right] \quad \text{Eq. (6.11.8.2.2-2)}$$

Note that since the bottom flange is stiffened, w is substituted for b_{fc} in Eq. (6.11.8.2.2-2).

Determine the web load-shedding factor, R_b, in accordance with Article 6.10.1.10.2 First, compute the depth of the web in compression, D_c, in accordance with Article D6.3.1:

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

Compute D_c along the inclined web:

$$D_c = \left[\left(\frac{-48.97}{|-48.97| + 48.04}\right)88.775 - 1.625\right]\sqrt{\frac{4^2+1}{4^2}} = 44.52 \text{ in.}$$

According to the provisions of Article 6.10.1.10.2:

$$\frac{2D_c}{t_w} = \frac{2(44.52)}{0.625} = 142.5 \quad \text{Eq. (6.10.1.10.2-2)}$$

$$\lambda_{rw} = 5.7\sqrt{\frac{E}{F_{yc}}} = 5.7\sqrt{\frac{29000}{50}} = 137.3 \quad \text{Eq. (6.10.1.10.2-4)}$$

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

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

where,

$$a_{wc} = \frac{2D_c t_w}{b_{fc} t_{fc}} = \frac{2(44.52)(0.625)}{(47.56)(1.625)} = 0.72 \quad \text{Eq. (6.10.1.10.2-5)}$$

Therefore,

$$R_b = 1 - \left(\frac{0.72}{1200 + 300(0.72)}\right)\left(\frac{2(44.52)}{0.625} - 137.3\right) = 0.997 \leq 1.0$$

Computing the nominal flexural compressive resistance, F_{nc}:

$$F_{nc} = (0.997)(1.0)(50)\left[1.0 - \left(1.0 - \frac{30}{(1.0)(50)}\right)\left\{1 - \sin\left[\frac{\pi}{2}\left(\frac{1.23 - \frac{(95.125/2)}{1.625}\sqrt{\frac{50}{(3.60)(29{,}000)}}}{1.23 - 0.57}\right)\right]\right\}\right]$$

$$F_{nc} = 49.57 \text{ ksi}$$

For STRENGTH I:

$$f_{bu} = -48.97 \text{ ksi}$$
$$\phi_f F_{nc} = 1.0(49.57) = 49.57 \text{ ksi}$$
$$|-48.97| \text{ ksi} < 49.57 \text{ ksi} \quad \text{ok} \qquad (\text{Ratio} = 0.988)$$

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:

$$f_{bu} = 48.04 \text{ ksi}$$
$$\phi_f F_{nt} = 1.0(50.00) = 50.0 \text{ ksi}$$
$$48.04 \text{ ksi} < 50.0 \text{ ksi} \quad \text{ok} \qquad (\text{Ratio} = 0.961)$$

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, shall be taken as the depth of the web measured along the slope. The factored shear load in the inclined web shall 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:

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

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} = shear in a single web at the section under consideration due to the factored loads.

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:

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

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

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

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)

Article 6.10.9.1 stipulates that the transverse stiffener spacing for interior panels without a longitudinal stiffener shall not 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:

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

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 buckling resistance determined from Eq. (6.10.9.3.2-8). Note that previous provisions, related to the effects of moment-shear interaction, are no longer included in the specifications for reasons discussed in Article C6.10.9.3.2.

For the interior web panel under consideration:

$$\frac{2(87.0)(0.625)}{\left[(95.125/2)(1.625)+28(2.75)\right]} = 0.70 < 2.5$$

Therefore:

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

Since,

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

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

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

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

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 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 in order 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 would 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 B shall not apply 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:

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

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:

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

where F_{yc} is the specified minimum yield strength of the compression flange (Article C6.8.2.3),

and where:

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

According to Article D6.3.1 (Appendix D to LRFD Section 6), 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:

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

where: f_t = the sum of the tension-flange stresses caused by the SERVICE II factored 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 SERVICE II factored loads, in this case stresses in the bottom flange.

d = the 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.

Since shear connectors are provided throughout the entire length of the tub girder, and the minimum amount of negative flexure concrete deck reinforcement is provided in accordance with Article 6.10.1.7, 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,

Therefore, for SERVICE II:

Top flange (tension flange):

$$f_t = 1.0\left[\frac{1.0(-27{,}012)}{15{,}225} + \frac{1.0(-3{,}321)}{24{,}606} + \frac{1.0(-3{,}193)}{24{,}606} + \frac{1.30(-12{,}823)}{44{,}944}\right]12 = 28.92 \text{ ksi}$$

Bottom flange (compression flange):

$$f_c = 1.0\left[\frac{1.0(-27{,}012)}{15{,}780} + \frac{1.0(-3{,}321)}{16{,}944} + \frac{1.0(-3{,}193)}{16{,}944} + \frac{1.30(-12{,}823)}{17{,}915}\right]12 = -36.32 \text{ ksi}$$

Calculate the depth of the web that is in compression, as measured from the neutral axis down to the top of the bottom flange:

$$D_c = \left(\frac{-(-36.32)}{|-36.32| + 28.92} \right)(84.4 + 1.625 + 2.75) - 1.625 = 47.80 \text{ in.}$$

D_c along the web: $$D_c = \frac{47.80}{\cos\left(\arctan\left(\frac{1}{4}\right)\right)} = 49.27 \text{ in.}$$

$$k = \frac{9}{\left(\frac{D_C}{D}\right)^2} = \frac{9}{\left(\frac{49.27}{87.0}\right)^2} = 28.06$$

$$F_{crw} = \frac{0.9(29000)(28.06)}{\left(\frac{87.0}{0.625}\right)^2} = 37.80 \text{ ksi} < R_h F_{yc} = 50 \text{ ksi} < \frac{F_{yw}}{0.7} = 71.4 \text{ ksi}$$

$$|-36.32 \text{ ksi}| < 37.80 \text{ ksi} \qquad \text{ok} \qquad (\text{Ratio} = 0.961)$$

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 *and* 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 85.0 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.

$$f_{deck} = \frac{1.0[1.0(861) + 1.0(827) + 1.3(-1{,}787)](12)}{19{,}398} = 0.398 \text{ ksi} < 0.90 f_r = 0.432 \text{ ksi}$$

Extend the minimum reinforcement to a section 85.0 feet from the Pier 1 in Span 2.

Also, check the tensile stress in the concrete deck due to SERVICE II load combination at the section 60.0 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.

$$f_{deck} = \frac{1.0[1.0(911) + 1.0(877) + 1.3(-1{,}958)](12)}{22{,}479} = 0.404 \text{ ksi} < 0.90 f_r = 0.432 \text{ ksi}$$

From Pier 1, extend the minimum reinforcement to a section 60.0 feet from the abutment in Span 1.

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 shall be employed for checking load-induced fatigue in tub girder sections. Further discussion concerning load induced fatigue in tub girders is presented as part of the calculations for Section 2-2.

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 the Steel Bridge Design Handbook Design Example 1.

8.9.2 Fracture (Article 6.6.2)

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 6.6.2-2) specified for Temperature Zone 2 (Table 6.6.2-1). Further discussion concerning fracture in tub girders is presented as part of the calculations for Section 2-2.

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 of 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 buckling of the plate, represented in 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 maximum bottom flange flexural stress at Section 1-3 due to the factored loads 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 as follows:

M_{DC1} = -4,421 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 is carried through section 1-3, therefore the longitudinal reinforcement is included in composite section property calculations. Separate calculations similar to section property calculations at section 2-1, but not included herein, show that at section 1-3:

Steel Section only:	$S_{BOT\ OF\ STEEL}$ = 10,908 in.^{4}
Steel Section + Long. Reinforcement:	$S_{BOT\ OF\ STEEL}$ = 12,141 in.^{4}

Therefore, for STRENGTH I (at Section 1-3):

$$f_{bu} = 1.0\left[\frac{1.25(-4{,}421)}{10{,}908} + \frac{1.25(-406)}{12{,}141} + \frac{1.25(-390)}{12{,}141} + \frac{1.75(-7{,}750)}{12{,}141}\right]12 = -20.47\text{ ksi}$$

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 provided.

Since $\lambda_f = \frac{b_{fc}}{t_{fc}} = \frac{95.13}{1.375} = 69.2 > R_2\sqrt{\frac{kE}{F_{yc}}} = 1.23\sqrt{\frac{(4.0)(29{,}000)}{50}} = 59.24$, the nominal flexural resistance of the bottom flange in compression is computed per Eq. (6.11.8.2.2-3):

$$F_{nc} = \frac{0.9ER_bk}{\left(\frac{b_{fc}}{t_{fc}}\right)^2} - \frac{R_bf_v^{\,2}k}{0.9Ek_S^{\,2}}\left(\frac{b_{fc}}{t_{fc}}\right)^2 \qquad \text{Eq. (6.11.8.2.2-3)}$$

where,

R_b = 1.0 (calculated but not shown)
f_v = 0.0 (can be taken as zero, given satisfaction of Article 6.11.2.3 requirements)
k = 4.0 (taken as 4.0 since bottom flange is unstiffened)
k_S = 5.34 (taken as 5.34 since bottom flange is unstiffened)

$$F_{nc} = \frac{0.9(29{,}000)(1.0)(4.0)}{\left(\frac{95.13}{1.375}\right)^2} - \frac{(1.0)(0.0)^2(4.0)}{0.9(29{,}000)(5.34)^2}\left(\frac{95.13}{1.375}\right)^2 = 21.81\,\text{ksi}$$

Therefore, for Option A, STRENGTH I:

$$f_{bu} = -20.47\,\text{ksi}$$
$$\phi_f F_{nc} = (1.0)(21.81) = 21.81\,\text{ksi}$$
$$|-20.47\,\text{ksi}| < 21.81\,\text{ksi} \qquad \text{ok} \qquad (\text{Ratio} = 0.939)$$

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 14. This will require that the WT12 x 42 stiffener also be spliced at the field-splice, and will require careful attention so as to not create a fatigue prone detail at the termination of the flange stiffener in Span 1 (at Section 1-2).

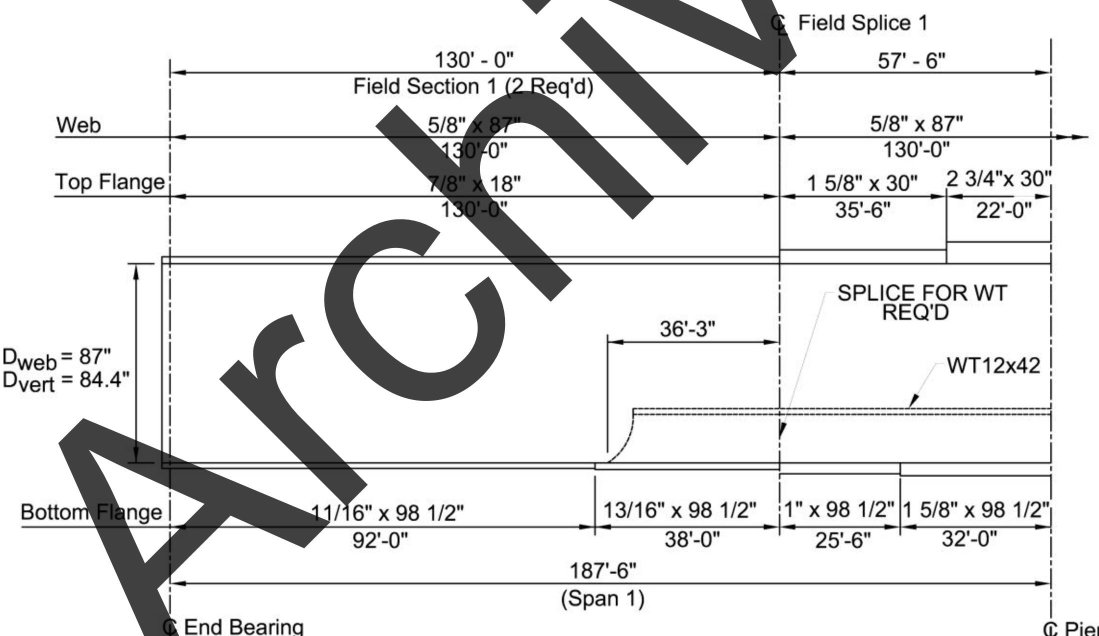

Figure 14 Sketch Showing Option B in Elevation, Stiffened Bottom Flange

For this particular option, a bottom flange plate 0.8125 inches thick, in combination with the WT12 x 42 bottom flange longitudinal stiffener may be used. The bottom flange plate and longitudinal flange stiffener are extended to Section 1-2 which is 93.75 feet from the abutment.

The negative flexure concrete deck reinforcement is carried through section 1-3, therefore the longitudinal reinforcement is included in composite section property calculations. Separate calculations similar to section property calculations at section 2-1, but not included herein, show that at section 1-3:

Steel Section only:	$S_{BOT\ OF\ STEEL} = 7{,}863\ \text{in.}^4$
Steel Section + Long. Reinforcement:	$S_{BOT\ OF\ STEEL} = 8{,}732\ \text{in.}^4$

Compute the maximum bottom flange flexural stress at Section 1-3 due to the factored loads 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

$$f_{bu} = 1.0\left[\frac{1.25(-4{,}421)}{7{,}863} + \frac{1.25(-406)}{8{,}732} + \frac{1.25(-390)}{8{,}732} + \frac{1.75(-7{,}750)}{8{,}732}\right]12 = -28.44\ \text{ksi}$$

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 provided.

Since $\lambda_f = \frac{b_{fc}}{t_{fc}} = \frac{95.13/2}{0.8125} = 58.54 < R_2\sqrt{\frac{kE}{F_{yc}}} = 1.23\sqrt{\frac{(4.0)(29{,}000)}{50}} = 59.24$, the nominal flexural resistance of the bottom flange in compression is computed per Eq. (6.11.8.2.2-2):

$$F_{nc} = R_b R_h F_{yc}\left[\Delta - \left(\Delta - \frac{F_{yr}}{R_h F_{yc}}\right)\left\{1 - \sin\left[\frac{\pi}{2}\left(\frac{R_2 - \frac{w}{t_{fc}}\sqrt{\frac{F_{yc}}{kE}}}{R_2 - R_1}\right)\right]\right\}\right] \qquad \text{Eq. (6.11.8.2.2-2)}$$

where,

$R_b = 1.0$ (calculated but not shown)
$\Delta = 0.0$ (calculated but not shown, $f_v = 0.0$ ksi)
$k = 4.0$ (calculated as 7.179, but the limit of 4.0 governs)
$k_S = 3.88$ (calculated but not shown)
$R_1 = 0.57$ (calculated but not shown)
$R_2 = 1.23$ (calculated but not shown)
$F_{yr} = 30.0$ ksi (calculated but not shown)

Computing the nominal flexural compressive resistance:

$$F_{nc} = (1.0)(1.0)(50)\left[1.0 - \left(1.0 - \frac{30}{(1.0)(50)}\right)\left\{1 - \sin\left[\frac{\pi}{2}\left(\frac{1.23 - \frac{(95.13/2)}{0.8125}\sqrt{\frac{50}{(4.0)(29{,}000)}}}{1.23 - 0.57}\right)\right]\right\}\right]$$

$$F_{nc} = 28.96\ \text{ksi}$$

Therefore, for Option B, STRENGTH I:

$$f_{bu} = -28.44\ \text{ksi}$$
$$\phi_f F_{nc} = (1.0)(28.96) = 28.96\ \text{ksi}$$
$$\left|-28.44\ \text{ksi}\right| < 28.96\ \text{ksi} \qquad \text{ok} \qquad (\text{Ratio} = 0.982)$$

At the termination of the flange stiffener, the bottom flange at Section 1-2 is subjected to both tensile and compressive stresses. Under the condition of longitudinally loaded fillet-welded attachments, for base metal at details with a length greater than 12 times the detail thickness attached by fillet welds with no special transition radius provided at the weld termination, the fatigue detail is either Category E or E’, depending on the detail thickness. Use of such details is not recommended and for many agencies, prohibited.

Since it is necessary to terminate the flange stiffener beyond the field splice in a region that can be subjected to a net tensile stress, while providing adequate fatigue performance, a transition radius at the stiffener termination with the end weld ground smooth is provided. A minimum-radius transition of 6 inches will provide a nominal fatigue resistance of a Category C detail. The complete penetration groove weld to the bottom flange, at the end of the stiffener, should be terminated at least 4 inches from the start of the transition, as shown in Figure 15. A continuous fillet weld is then placed over the grove weld on both sides of the stiffener, and the end of the transition is ground smooth.

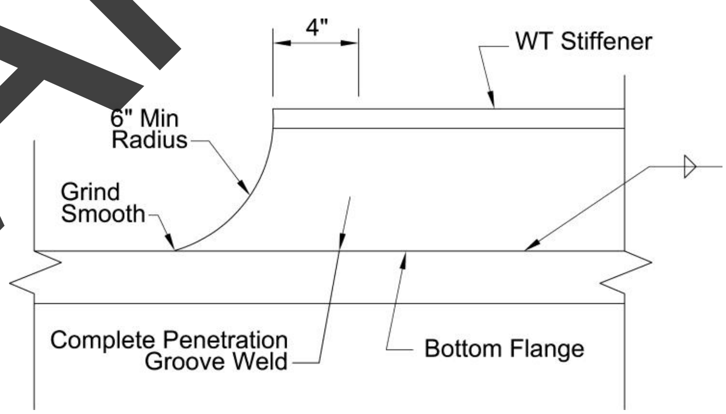

Figure 15 Sketch Showing Option B, Longitudinal Flange Stiffener Termination

Fatigue of the base metal at the longitudinal flange stiffener weld termination at Section 1-2, will be checked for the FATIGUE I 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 factors for the fatigue limit state, computed previously, are 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.

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 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. Earlier computations were made to ensure that the longitudinal concrete deck reinforcement satisfies the provisions of Article 6.10.1.7. Therefore, the concrete deck will be 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 1.50 load factor for the FATIGUE I load combination) is computed using the properties of the short-term composite section as follows. Note that, for simplicity, the stress range is conservatively calculated at Section 1-2, 1.75 feet from the termination of the flange stiffener, and has an unstiffened bottom flange thickness of 0.6875 inches.

At section 1-2 the unfactored fatigue bending moments are as follows:

Positive Flexure: $M_{LL+I} = 2{,}398$ kip-ft
Negative Flexure: $M_{LL+I} = -1{,}121$ kip-ft

Separate calculations similar to section property calculations at section 2-1, but not included herein, show that at section 1-2 the short-term composite section modulus for the bottom flange is:

Composite Section, n=8.0: $S_{BOT\ OF\ STEEL} = 9{,}224\ \text{in.}^4$

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 FATIGIUE I stress range is computed as:

$$\gamma(\Delta f) = \frac{1.50(2,398)(12)}{9,224} + \frac{1.50|-1,121|(12)}{9,224} = 6.87 \text{ ksi}$$

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 I load combination and infinite fatigue life, is determined as:

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

For a Category C detail, $(\Delta F)_{TH}$ = 10.0 ksi (Table 6.6.1.2.5-3). Therefore:

$$(\Delta F)_n = (\Delta F)_{TH} = 10.00 \text{ ksi}$$

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

6.87 ksi < 10.0 ksi ok (Ratio = 0.687)

8.10.1.3 Summary of Unstiffened Flange Versus Stiffened Flange

In order to provide the most economical solution for terminating the stiffener beyond a field splice, such as this, the Engineer, with consultation from 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 Engineer must consider prior to choosing either option, in regard to the amount of material, fabrication costs, installation costs, and the performance and long-term serviceability.

For this example, the material weight of each option for this particular bottom flange section is as follows:

$$\text{Option A} = (490)\left[(36.25)\frac{(98.5)(1.375)}{144}\right] = 16,706 \text{ lbs.}$$

$$\text{Option B} = (490)\left[(38.0)\frac{(98.5)(0.8125)}{144}\right] + (42)(36.25) = 11,870 \text{ lbs.}$$

While Option B saves approximately 4,800 lbs. of steel, it represents only a material savings, which will likely be overcome by the increase in labor costs associated with welding the stiffener, coping the end and making the CJP welds, and fabrication and installation of the WT splice.

8.11 Top Flange Lateral Bracing

Article 6.7.5.3 requires, for straight tub girders, that the need for a full length internal lateral bracing system be investigated to ensure that deformation of the section (local stability of the flanges and global stability of the girder) are controlled during erection and deck casting. Generally, lateral bracing will not be required between adjacent tub girders. The AASHTO LRFD Specifications further suggest that tub girders with spans less than 150 feet be braced, at a minimum on either side of a lifting point. For spans greater than 150 feet, full-length lateral bracing shall be installed.

Top flange lateral bracing increases the torsional stiffness of tub girder sections during erection, handling, and deck casting. 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 is torsionally more flexible and subject to rotation or twist. The top flange lateral bracing, then, 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 truss) or double diagonal arrangement (X-brace truss). The diagonal bracing members commonly frame into the workpoint of the girder top flange and internal diaphragm connection. Alternatively, the length between internal cross-frame 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 shall 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 of Eq. (C6.7.5.3-1).

Preliminary proportions of the diagonal members are determined as follows:

For bracing members in compression: $\frac{K\ell}{r} \leq 140$ (Article 6.9.3).

The maximum length workpoint to workpoint 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, WT sections will be used for the lateral bracing members so as to reduce connection eccentricities. If single angle sections were to be used, the effective length factor K may be taken as 1.0 to reflect the decreased stability of these sections.

$$\text{for } \frac{K\ell}{r} \leq 140\,; \quad r_{min} = \frac{(0.750)(21.15)(12)}{140} = 1.36\text{ in.}$$

Calculate the minimum required cross-sectional area, A_d:

$A_d \geq 0.03\,w$; where w is the center to center distance between top flanges (in.); Eq. (C6.7.5.3-1)

$$A_d \geq 0.03(138.0) = 4.14\text{ in.}^2$$

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 state, 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 these forces are resisted directly by the lateral struts of the bracing system and not by the diagonals. Therefore, the forces in diagonal members resulting form 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 [6 and 7] 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 equally 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.

The diagonal bracing should be examined for all applicable limit states. Bending moments resulting from connection eccentricity should also be included in the design.

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 $\Delta V_H = 0.49$ kip/ft. Therefore, the 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 interior supports).

Due to the inclination of the web, the struts are always in tension. Therefore, the member shall be 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 shall 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

$$P_r = \phi_u P_{nu} = \phi_u F_u A_n U \qquad \text{Eq. (6.8.2.1-2)}$$

In the preceding equation, 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, and two fasteners will be used in the direction of stress, U = 0.75, per Article 6.8.2.2. Values for U are presented in Article 6.8.2.2 for other common connection types.

Therefore,

$$P_r = \phi_y F_y A_g = (0.95)(50)(3.75) = 178\text{ kips}$$

$$P_r = \phi_u F_u A_n U = (0.80)(65)(3.25)(0.75) = 127\text{ kips} > 10.87\text{ kips} \qquad \text{(governs)}$$

Where A_n is based on the use of a 7/8 inch diameter bolt in a standard size (1″ 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 bracing members in tension: $\dfrac{\ell}{r} \leq 240$

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.

$$\frac{\ell}{r} = \frac{138}{0.776} = 177.8 \leq 240 \qquad \text{ok}$$

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 completely 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 shall be investigated to ensure that adequate connection material is provided to develop the factored resistance of the connection and prevent block shear rupture.

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 of 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 with regard to the use and design of bearing systems.

At the support, tub girder torsion can be directly resolved in to 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 diaphragm at support lines must be sufficient to resist torsional moments in the tub girders, as the diaphragm and adjacent girder form a structural system to counter the torsion at 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 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 examination and design guide for typical bearing types used in steel bridges can be found in the NSBA publication *Steel Bridge Bearing Selection and Design Guide* [8].

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)

Constructibility	
Flexure (STRENGTH I)	
Eq. 6.10.1.6-1 – Top Flange	0.335
Eq. 6.10.3.2.1-1 – Top Flange	0.921
Eq. 6.10.3.2.1-2 – Top Flange	0.878
Eq. 6.11.3.2-3 – Bottom Flange	0.561
Eq. 6.10.3.2.1-3. – Web Bend Buckling	0.935
Service Limit State	
Permanent Deformations (SERVICE II)	
Eq. 6.10.4.2.2-1 – Top Flange	0.707
Eq. 6.10.4.2.2-2 – Bottom Flange	0.928
Strength Limit State (Compact Section)	
Ductility Requirement (Eq. 6.10.7.3-1)	0.297
Flexure – Eq. 6.11.7.1.1-1 (STRENGTH I)	0.906
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)

Constructibility	
Flexure (STRENGTH I)	
Eq. 6.10.3.2.2-1 – Top Flange	0.588
Eq. 6.11.3.2-2 – Bottom Flange	0.550
Eq. 6.1.3.2-2 – Web Bend Buckling	0.558
Shear (STRENGTH I)	
Eq. 6.10.3.3-1	0.859
Service Limit State	
Web Bend Buckling - Eq. 6.10.4.2.2-4	0.961
Strength Limit State	
Flexure (STRENGTH I)	
Bottom Flange – Eq. 6.11.8.1.1-1	0.988
Top Flange – Eq. 6.11.8.1.2-1	0.961
Shear (at interior pier) – Eq. 6.10.9.1-1 (STRENGTH I)	0.863

9.0 REFERENCES

1. AASHTO (2010). *AASHTO LRFD Bridge Design Specifications*, 5th Edition with 2010 Interims, American Association of State Highway and Transportation Officials, Washington, DC.

2. Coletti, D.A., Fan, Z., Holt, J., Vogel, J., Gatti, W., (2005). *Practical Steel Tub Girder Design*, National Steel Bridge Alliance (NSBA).

3. AASHTO/NSBA, (2006). *G1.4-2006: Guidelines for Design Details*, AASHTO/NSBA Steel Bridge Collaboration.

4. Texas Department of Transportation (TxDOT), (2009). *Preferred Practices for Steel Bridge Design, Fabrication, and Erection*, Texas Steel Quality Council.

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

6. Fan, Z. and Helwig, T., (1999). "Behavior of Steel Box Girders with Top Flange Bracing," *Journal of Structural Engineering, ASCE,* Vol. 125, No. 8, pp. 829-837.

7. Helwig, T., Yura, J., Herman, R., Williamson, E., and Li, D., (2007). "Design Guidelines for Trapezoidal Box Girder Systems," Report No. FHWA/TX-07/0-4307-1, Center for Transportation Research, University of Texas at Austin.

8. AASHTO/NSBA, (2004). *G9.1-2004: Steel Bridge Bearing Design and Detailing Guidelines*, AASHTO/NSBA Steel Bridge Collaboration.