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Steel Bridge Design Handbook

Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge

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16. Abstract

The basic application of the AASHTO LRFD Bridge Design Specifications to the design of straight steel I-section flexural members is illustrated through this design example. The example illustrates the design of a typical three-span continuous straight steel I-girder bridge with spans of 140.0 feet – 175.0 feet – 140.0 feet. Specifically, the example illustrates the design of selected critical sections from an exterior girder at the strength, service and fatigue limit states. Constructibility checks, stiffener and shear connector designs are also presented.

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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

In 1993, the American Association of State Highway and Transportation Officials (AASHTO) adopted the Load and Resistance Factor Design (LRFD) specifications for bridge design. The First Edition of the design specifications was published by AASHTO in 1994. The publication of a Second Edition followed in 1998, along with the publication of the First Edition of a companion document – the AASHTO *LRFD Bridge Construction Specifications*. The design specifications are available in either customary U.S. units or in SI (metric) units, whereas the construction specifications are currently only available in SI units. The LRFD specifications were approved by AASHTO for use as alternative specifications to the AASHTO *Standard Specifications for Highway Bridges*.

The LRFD specifications evolved in response to a high level of interest amongst the AASHTO Subcommittee on Bridges and Structures in developing updated AASHTO bridge specifications together with accompanying commentary. The goal was to develop more comprehensive specifications that would eliminate any gaps and inconsistencies in the Standard Specifications, incorporate the latest in bridge research, and achieve more uniform margins of safety or reliability across a wide variety of structures. The decision was made to develop these new specifications in an LRFD-based format, which takes the variability of the behavior of structural elements into account through the application of statistical methods, but presents the results in a manner that is readily usable by bridge designers. A detailed discussion of the evolution of the LRFD design specifications and commentary is presented in NCHRP *Research Results Digest 198* (available from the Transportation Research Board) and elsewhere, and will not be repeated herein.

The design of steel structures is covered in Section 6 of the AASHTO Fifth Edition of the *LRFD Bridge Design Specification* [1], referred to herein as *AASHTO LRFD (5th Edition, 2010)*. The Fifth Edition of the design specifications contains a complete set of provisions for the design of straight steel I- and box-section flexural members within Articles 6.10 and 6.11, respectively. These provisions are structured to simplify their logic, organization and application, while also maintaining accuracy and generality. The provisions provide a unified design approach for both straight and horizontally curved girders within a single specification, which allows for overall efficiency of the design process for bridges that contain both straight and curved spans. The basic application of these provisions to the design of straight steel I-section flexural members is illustrated through the design example presented herein. The example illustrates the design of a typical three-span continuous straight steel I-girder bridge with spans of 140′-0″ – 175′-0″ – 140′-0″. Specifically, the example illustrates the design of selected critical sections from an exterior girder at the strength, service and fatigue limit states. Constructibility checks, stiffener and shear connector designs are also presented.

2.0 OVERVIEW OF LRFD ARTICLE 6.10

The design of I-section flexural members is covered within Article 6.10 of the *AASHTO LRFD (5th Edition, 2010)*. The provisions of Article 6.10 are organized to correspond to the general flow of the calculations necessary for the design of I-section flexural members. Each of the sub-articles are written such that they are largely self-contained, thus minimizing the need for reference to multiple sub-articles to address any of the essential design considerations. Many of the individual calculations and equations are streamlined and selected resistance equations are presented in a more general format as compared to earlier LRFD Specifications (prior to the 3rd Edition). The sub-articles within the Fifth Edition Article 6.10 are organized as follows:

6.10.1 General
6.10.2 Cross-section Proportion Limits
6.10.3 Constructibility
6.10.4 Service Limit State
6.10.5 Fatigue and Fracture Limit State
6.10.6 Strength Limit State
6.10.7 Flexural Resistance - Composite Sections in Positive Flexure
6.10.8 Flexural Resistance - Composite Sections in Negative Flexure and Noncomposite Sections
6.10.9 Shear Resistance
6.10.10 Shear Connectors
6.10.11 Stiffeners
6.10.12 Cover Plates

Section 6 also contains four appendices relevant to the design of flexural members as follows:

Appendix A - Flexural Resistance of Straight Composite I-Sections in Negative Flexure and Straight Noncomposite I-Sections with Compact or Noncompact Webs
Appendix B - Moment Redistribution from Interior-Pier I-Sections in Straight Continuous-Span Bridges
Appendix C - Basic Steps for Steel Bridge Superstructures
Appendix D - Fundamental Calculations for Flexural Members

For composite I-sections in negative flexure and noncomposite I-sections, the provisions of Article 6.10.8 limit the nominal flexural resistance to a maximum of the moment at first yield. As a result, the nominal flexural resistance for these sections is conveniently expressed in terms of the elastically computed flange stress. When these sections satisfy specific steel grade requirements and have webs that are classified as either compact or noncompact, the optional provisions of Appendix A may be applied instead to determine the flexural resistance, which may exceed the moment at first yield. Therefore, the flexural resistance is expressed in terms of moment in Appendix A. The provisions of Appendix A are a direct extension of and are fully consistent with the main provisions of Article 6.10.8.

The previous Specifications defined sections as either compact or noncompact and did not explicitly distinguish between a noncompact web and a slender web. The current provisions make explicit use of these definitions for composite I-sections in negative flexure and noncomposite I-sections because the noncompact web limit serves as a useful anchor point for a continuous representation of the maximum potential section resistance from the nominal yield moment up to the plastic moment resistance. Because sections with compact or nearly compact webs are less commonly used, the provisions for sections with compact or noncompact webs have been placed in an appendix in order to simplify and streamline the main provisions. The main provisions within the body of Article 6.10 may be used for these types of sections to obtain an accurate to somewhat conservative determination of the flexural resistance calculated using Appendix A. For girders that are proportioned with webs near the noncompact web slenderness limit, the provisions of Article 6.10 and Appendix A produce the same strength for all practical purposes, with the exception of cases with large unsupported lengths sometimes encountered during construction. In these cases, Appendix A gives a larger more accurate flexural resistance calculation. In the example to follow, a slender-web section is utilized for both the composite section in regions of negative flexure and for the noncomposite section in regions of positive flexure before the concrete deck has hardened. As a result, the main provisions of Article 6.10 must be applied for the strength limit state and constructibility checks for those sections and the optional Appendix A is not applicable.

Minor yielding at interior piers of continuous spans results in redistribution of the moments. For straight continuous-span flexural members that satisfy certain restrictions intended to ensure adequate ductility and robustness of the pier sections, the optional procedures of Appendix B may be used to calculate the redistribution moments at the service and/or strength limit states. These provisions replace the former ten-percent redistribution allowance as well as the former inelastic analysis procedures. They provide a simple calculated percentage redistribution from interior-pier sections. This approach utilizes elastic moment envelopes and does not require the direct use of any inelastic analysis. As such, the procedures are substantially simpler and more streamlined than the inelastic analysis procedures of the previous Specifications. Where appropriate, these provisions make it possible to use prismatic sections along the entire length of the bridge or between field splices, which can improve overall fatigue resistance and provide significant fabrication economies. Although the necessary steps could be taken to allow moment redistribution in the example presented herein, the provisions of Appendix B are not applied.

Flow charts for flexural design of I-sections, along with an outline giving the basic steps for steel-bridge superstructure design, are provided in Appendix C. Fundamental section property calculations for flexural members are provided in Appendix D.

The provisions of Article 6.10 and the optional Appendices A and B provide a unified approach for consideration of combined major-axis bending and flange lateral bending from any source in both straight and horizontally curved I-girders. As such, general design equations are provided that include the consideration of both major-axis bending and flange lateral bending. For straight girders, flange lateral bending is caused by wind and by torsion from various origins. Sources of significant flange lateral bending due to torsion include eccentric slab overhang loads acting on cantilever forming brackets placed along exterior members, staggered cross-frames, and significant support skew. When the above effects are judged to be insignificant or incidental, the flange lateral bending term, f_ℓ, is simply set equal to zero in the appropriate equations. The example to follow considers the effects of flange lateral bending caused by wind and by torsion due to the effects of eccentric slab overhang loads.

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

Structural Steel: AASHTO M 270 Grade HPS 70W (ASTM A 709 Grade HPS 70W) uncoated weathering steel with $F_y = 70$ ksi (for the flanges in regions of negative flexure)
AASHTO M 270, Grade 50W (ASTM A 709, Grade 50W) uncoated weathering steel with $F_y = 50$ ksi (for all other girder and cross-frame components)

The example design utilizes uncoated weathering steel. Where site conditions are adequate for successful application, uncoated weathering steel is the most cost-effective material choice in terms of savings in both initial and future repainting costs. In the years since its introduction into bridge construction by the Michigan DOT in the 1960's, uncoated weathering steel has become widely accepted as cost-effective, currently representing about 45 percent of the steel-bridge market. However, it has also frequently been misused because of inexperience or ignorance about the properties of the material. To counter this and increase the confidence in its performance, the FHWA issued a Technical Advisory (T5140.22) in 1989 entitled *Uncoated Weathering Steel in Structures.* The guidelines contained in this document, developed in cooperation with the steel industry, are a valuable source of information on the proper environments for the use of weathering steel. The guidelines also suggest good detailing practice to help ensure successful application of the material.

In regions of negative flexure, the example design utilizes a hybrid section consisting of ASTM A 709 Grade HPS 70W high-performance steel (HPS) flanges and an ASTM A 709 Grade 50W web. Grade HPS 70W was developed in the early 1990s under a successful cooperative research program between the Federal Highway Administration, the U.S. Navy, and the American Iron and Steel Institute. Grade HPS 70W possesses superior weldability and toughness compared to conventional steels of this strength range. Grade HPS 70W is currently produced by quenching and tempering (Q&T) or by thermo-mechanical-controlled-processing (TMCP). TMCP HPS is available in plate thicknesses up to 2 inches and in maximum plate lengths from approximately 600 to 1500 inches depending on weights. Q&T HPS is available in plate thicknesses up to 4 inches, but because of the furnaces that are used in the tempering process, is subject to a maximum plate-length limitation of 600 inches or less depending on weights. Therefore, when Q&T HPS is used, the maximum plate-length limitation should be considered when laying out flange and web transitions. Current information on maximum plate length availability can be obtained by contacting a steel producer. Guidelines for fabrication using Grade HPS 70W steel are available in the AASHTO *Guide Specifications for Highway Bridge Fabrication with HPS 70W Steel*. HPS is finding increasing application in highway bridges across the U.S., with hybrid designs utilizing Grade HPS 70W flanges in conjunction with a Grade HPS 50W web being the most popular application.

Concrete: $f'_c = 4.0\,\text{ksi}$

Slab Reinforcing Steel: AASHTO M 31, Grade 60 (ASTM A 615, Grade 60) with $F_y = 60$ ksi

Permanent steel deck forms are assumed between the girders; the forms are assumed to weigh 15.0 psf. The girders are assumed to be composite throughout.

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.

4.0 STEEL FRAMING

4.1. Span Arrangement

Proper layout of the steel framing is an important part of the design process. The example bridge has spans of 140′-0″ – 175′-0″ – 140′-0″, with the span lengths arranged to give similar positive dead load moments in the end and center spans. Such balanced span arrangements (i.e. end spans approximately 0.8 of the length of the center spans) in multiple continuous-span steel bridges result in the largest possible negative moments at the adjacent piers, along with smaller concomitant positive moments and girder deflections. As a result, the optimum depth of the girder in all spans will be nearly the same resulting in a much more efficient design.

Steel has the flexibility to be utilized for most any span arrangement. However, in some competitive situations, steel has been compelled to use a particular span arrangement that has been optimized for an alternate design. In a competitive situation, if the pier locations are flexible and if the spans have been optimized for the alternate design, the span arrangement for the steel design almost certainly will be different and must also be optimized. In situations where there are severe depth restrictions or where it is desirable to eliminate center piers (e.g. certain overpass-type structures), it may be desirable to provide short end spans. However, in cases where there are no such restrictions or needs, it will likely be more economical to extend the end spans to provide a balanced span ratio. This will avoid the costs associated with the possible need for tie-downs at the end bearings, inefficient girder depths and additional moment in some spans. In curved structures, extension of the end spans may also permit the use of radial supports where skewed supports might otherwise have been necessary.

It should be noted that the most efficient and cost-competitive steel bridge system can result only when the substructure for the steel design is evaluated and designed concurrently with the superstructure. Although the superstructure and substructure act in concert, each is often analyzed for separate loads and isolated from the other as much as possible both physically and analytically. Substructure costs represent a significant portion of the total bridge cost. The form chosen for the substructure, often based on past experience or the desire to be conservative, may unknowingly lead to an inefficient steel design. Substructure form also has a marked effect on the overall aesthetic appeal of the structure. When the site dictates difficult span arrangements and pier designs, steel is often the only material of choice. However, its efficiency often suffers when designed to conform to foundations developed for other materials.

For major projects, superstructure and substructure cost curves should be developed for a series of preliminary designs using different span arrangements. Since the concrete deck costs are constant and independent of span length, they need not be considered when developing these curves. The optimum span arrangement lies at the minimum of the sum of the superstructure and substructure costs. These curves should always be regenerated to incorporate changes in unit costs that may result from an improved knowledge of specific site conditions. While it is recognized that the locations of piers cannot be varied in many instances, for cases where pier locations are flexible, the use of poorly conceived span arrangements and/or substructure form can have the greatest total cost impact on a steel-bridge design.

4.2. Bridge Cross-Section

The example bridge cross-section consists of four (4) girders spaced at 12′-0″ centers with 3′-6″ deck overhangs and an out-to-out deck width of 43′-0″. The 40′-0″ roadway width can accommodate up to three 12-foot-wide design traffic lanes. The total thickness of the cast-in-place concrete deck is 9½″ including a ½″-thick integral wearing surface. The concrete deck haunch is 3½″ deep measured from the top of the web to the bottom of the deck. The width of the deck haunch is assumed to be 16.0 inches. Deck parapets are each assumed to weigh 520 pounds per linear foot. A future wearing surface of 25.0 psf is also assumed in the design. A typical cross-section is shown in Figure 1:

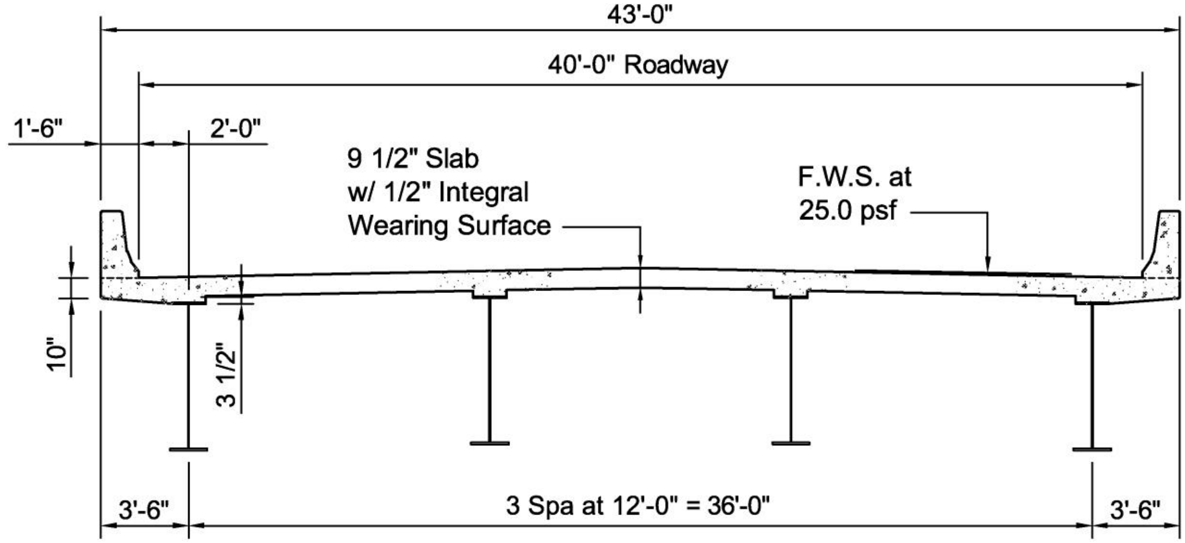

Figure 1: Typical Bridge Cross-Section

The deck overhangs are approximately 29 percent of the girder spacing. Reducing the girder spacing below 12′-0″ would lead to an increase in the size of the deck overhangs, which may lead to larger loading on the exterior girders. The effect of a wider girder spacing would have to be evaluated with respect to any potential increase in the cost of the concrete deck. Wide girder spacings offer the advantages of fewer girders and pieces to fabricate, inspect, ship and erect, and fewer bearings to purchase and set.

4.3. Cross-Frames

Cross-frames provide lateral stability to the top and bottom flanges of the girder, distribute vertical dead and live loads applied to the structure, transfer lateral wind loads from the bottom of the girder to the deck and from the deck to the bearings, reduce any flange lateral bending effects and transverse deck stresses and provide adequate distribution of load to ensure relatively equal girder deflection during construction. Cost-effective design of steel-bridge superstructures requires careful attention to details, including the design of diaphragms and cross-frames. Although these members account for only a small percentage of the total structure weight, they usually account for a significant percentage of the total erected steel cost.

Cross-frames in steel-girder bridges, along with the concrete deck, provide restoring forces that tend to make the steel girders deflect equally. During erection and prior to curing of the deck, the cross-frames are the only members available to provide the restoring forces that prevent the girders from deflecting independently. The restoring forces will be very small if the stiffnesses of the adjacent girders at the cross-frame connection points are approximately equal and the applied loads to each girder are approximately the same. For the more general case where the girders deflect by different amounts, the cross-frames and concrete deck will develop larger restoring forces, with the magnitude being dependent on the relative girder, cross-frame and deck stiffnesses.

With fewer cross-frame lines, the force in each cross-frame member will increase to some degree since the total restoring force between two adjacent girders is the same regardless of the number of cross- frames that are provided. Stresses in the concrete deck will also increase to a degree. For a tangent composite bridge with a regular framing plan, which is the case in this particular design example, the increases in these forces and stresses will typically be of less concern; particularly at the cross-frame spacings chosen for this example. However, the designer should be at least cognizant of these effects when fewer cross-frame lines are provided, especially for more irregular framing plans and when the bridge is non-composite.

When refined methods of analysis are used and the cross-frames are included in the structural model to determine force effects, the cross-frame members are to be designed for the calculated force effects. When approximate methods of analysis are used (e.g., lateral distribution factors), cross-frame force effects due to dead and live loads generally cannot be easily calculated. Thus, as a minimum, cross- frames are designed to transfer wind loads and to meet all applicable slenderness and minimum material thickness requirements. For the most part, such an approach has proven successful on tangent bridges without skewed supports or with small skews. For tangent bridges with moderate to highly skewed supports, where the effects of differential deflections between girders become more pronounced, and for all curved bridges, closer scrutiny of cross-frame force effects is warranted.

Since 1949, the AASHTO Standard Specifications for steel design have specified a limit of 25'-0" on the longitudinal diaphragm or cross-frame spacing for I-girder bridges. While this limit has ensured satisfactory performance of these structures over the years, it is essentially an arbitrary limit that was based on the experience and knowledge that existed at that time. This arbitrary requirement has been removed in the LRFD specifications. Instead, the need for cross-frames at all stages of construction and the final condition is to be established by rational analysis (Article 6.7.4.1). Article 6.7.4.1 further states that the investigation should include, but not be limited to, consideration of the transfer of lateral wind loads from the bottom of the girder to the deck and from the deck to the bearings, the stability of bottom flanges for all loads when subject to compression, the stability of top flanges in compression prior to curing of the deck and the distribution of vertical dead and live loads applied to the structure. Diaphragms or cross-frames required for conditions other than the final condition may be specified to be temporary bracing. Based on the preceding considerations, the cross-frame spacings shown on the framing plan in Figure 2 were chosen for this example.

Although the AASHTO design specifications are generally member based, the overall behavior of the entire bridge system must also be considered, particularly during the various stages of construction. As will be demonstrated later on in the design example, the noncomposite bridge structure acts as a system to resist wind loads during construction. The example calculations will illustrate how a couple of panels of top lateral bracing, as shown in the interior bays adjacent to the interior piers in Figure 2, can be added, if necessary, to provide a stiffer load path for wind loads acting on the noncomposite structure during construction. The lateral bracing helps to reduce the lateral deflections and lateral flange bending stresses due to the wind loads. A rational approach is presented to help the Engineer evaluate how many panels of lateral bracing might be necessary to reduce the lateral deflections and stresses to a level deemed acceptable for the situation under consideration. Such a system of lateral bracing adjacent to supports can also help provide additional rigidity to an I-girder bridge system to help prevent significant relative horizontal movements of the girders that may occur during construction, particularly in longer spans (e.g. spans exceeding approximately 200 feet). Unlike building columns, which are restrained against the ground by gravity and cannot translate with respect to each other, bare steel bridge girders are generally free to translate longitudinally with respect to adjacent girders. Lateral bracing provides a triangulation of the members to help prevent the rectangles formed by the girders and cross-frames from significantly changing shape and moving longitudinally with respect to each other. Bottom lateral bracing can serve similar functions to those described above, but unlike top bracing, would be subject to significant live-load forces in the finished structure that would have to be considered should the bracing be left in place.

4.4. Field Section Sizes

Field section lengths are generally dictated by shipping weight and length considerations. The Engineer should consult with fabricators regarding any specific restrictions that might influence the field-splice locations. For the example design, there is one field splice assumed in each end span and two field splices assumed in the center span resulting in five (5) field sections in each line of girders, or 20 field sections for the bridge (Figure 2).

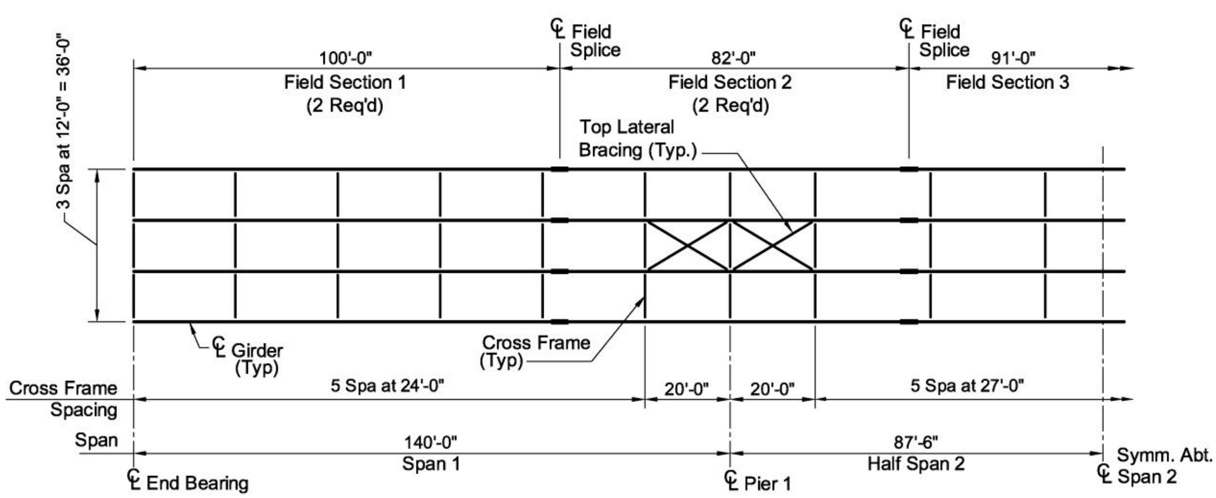

Figure 2: Framing Plan

5.0 PRELIMINARY GIRDER SIZES

5.1. Girder Depth

The proper girder depth is another extremely important consideration affecting the economy of steel-girder design. In the absence of any depth restrictions, Article 2.5.2.6.3 provides suggested minimum span-to-depth ratios. From Table 2.5.2.6.3-1, the suggested minimum depth of the steel section in a composite I-section in a continuous span is given as 0.027L, where L is the span length in feet. Using the longest span of 175′-0″, the suggested minimum depth of the steel section is:

$$0.027(175.0) = 4.725 \text{ ft} = 56.7 \text{ in}$$

Since there are no depth restrictions in this case, a deeper steel section is desired to provide greater stiffness to the girders in their noncomposite condition during construction (it should be noted that the optimum web depth is usually also greater than the suggested minimum web depth). Therefore, the suggested minimum overall depth of the composite I-section in a continuous span, equal to 0.032L, from Table 2.5.2.6.3-1 will be used here for the steel section:

$$0.032(175.0) = 5.60 \text{ ft} = 67.2 \text{ in.}$$

A web depth of 69 inches is used.

5.2. Cross-section Proportions

Cross-section proportion limits for webs of I-sections are specified in Article 6.10.2.1. In the span ranges given for this example, the need for longitudinal stiffeners on the web is not anticipated. For webs without longitudinal stiffeners, webs must be proportioned such that:

$$\frac{D}{t_w} \leq 150 \qquad \text{Eq. (6.10.2.1.1-1)}$$

Rearranging:

$$(t_w)_{min.} = \frac{D}{150} = \frac{69}{150} = 0.46 \text{ in.}$$

Because of concerns about the web bend-buckling resistance at the service limit state in regions of negative flexure and also the higher shears in these regions, try a web thickness of 0.5625 inches in regions of negative flexure and a web thickness of 0.5 inches in regions of positive flexure. Note that the AASHTO/NSBA Steel Bridge Collaboration *Guidelines for Design for Constructibility* (hereafter referred to as "the Guidelines") recommend a minimum web thickness of 0.4375 inches, with a minimum thickness of 0.5 inches preferred.

Cross-section proportion limits for flanges of I-sections are specified in Article 6.10.2.2. The minimum width of flanges is specified as:

$$b_f \geq D/6 \qquad \text{Eq. (6.10.2.2-2)}$$

Therefore:

$$(b_f)_{min.} = D/6 = 69/6 = 11.5 \text{ in.}$$

The minimum thickness of flanges is specified as:

$$t_f \geq 1.1 t_w \qquad \text{Eq. (6.10.2.2-3)}$$

Or:

$$(t_f)_{min} = 1.1 t_w = 1.1(0.5625) = 0.62 \text{ in.}$$

However, the Guidelines recommend a minimum flange thickness of 0.75 inches. Therefore, use $(t_f)_{min}$ = 0.75 inches.

For the top flange in regions of positive flexure in composite girders, Article C6.10.3.4 provides the following additional guideline for the minimum compression-flange width. This guideline is intended to provide more stable field pieces that are easier to handle during erection without the need for special stiffening trusses or falsework, and to help limit out-of-plane distortions of the compression flange and web during the deck-casting operation:

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

where L is the length of the girder shipping piece in feet. From Figure 3, the length of the longest field piece, which is assumed to also equal the length of the longest shipping piece in this case, is 100 feet. Therefore, for this particular shipping piece:

$$(b_{fc})_{min} = \frac{L}{85} = \frac{100}{85} = 1.176 \text{ ft} = 14.1 \text{ in.}$$

Based on the above minimum proportions, the trial girder shown in Figure 3 is assumed for the exterior girder, which is assumed to control.

Because the top flange of the exterior girders will be subject to flange lateral bending due to the effect of the eccentric deck overhang loads, and also due to wind loads during construction, top-flange sizes slightly larger than the minimum sizes are assumed in regions of positive flexure. The bottom flange plates in regions of positive flexure in this example are primarily sized based on the flange-stress limitation at the service limit state specified in Article 6.10.4.2.2. However, in the end spans, the size of the larger bottom-flange plate in this region is controlled by the stress-range limitation on a cross-frame connection plate weld to the tension flange at the fatigue and fracture limit state, as will be demonstrated later. The bottom-flange sizes in regions of negative flexure are assumed controlled by either the flange local buckling or lateral torsional buckling resistance at the strength limit state. Top-flange sizes in these regions are assumed controlled by tension-flange yielding at the strength limit state. At this stage, the initial trial plate sizes in regions of negative flexure are primarily educated guesses based on experience. Because the girder is assumed to be composite throughout, the minimum one-percent longitudinal reinforcement required in Article 6.10.1.7 will be included in the section properties in regions of negative flexure. As a result, a top flange with an area slightly smaller than the area of the bottom flange can be used in these regions. Recall that the flanges in regions of negative flexure are assumed to be Grade HPS 70W steel in this example.

Because the most economical plate to buy from a mill is between 72 and 96 inches wide, an attempt was made in the design to minimize the number of thicknesses of plate that must ordered for the flanges. As recommended in the Guidelines, flange thicknesses should be selected in not less than 1/8-inch increments up to 2½ inches in thickness and ¼-inch increments over 2½ inches in thickness. Note that individual flange widths are kept constant within each field piece, as recommended in the Guidelines. The Guidelines contain more detailed discussion on specific issues pertinent to the sizing of girder flanges as it relates to the ordering of plate and the fabrication of the flanges. Fabricators can also be consulted regarding these issues and all other fabrication-related issues discussed herein.

Flange transitions, or shop-welded splices, are located based on design considerations, plate length availability (as discussed earlier) and the economics of welding and inspecting a splice compared to the cost of extending a thicker plate. The design plans should consider allowing an option for the fabricator to eliminate a shop splice by extending a thicker flange plate subject to the approval of the Engineer. Usually, a savings in weight of between 800 to 1000 pounds should be realized in order to justify a flange butt splice. Again, the Guidelines contain more detailed discussion regarding this particular issue.

At flange splices, the cross-sectional area of the thinner plate should not be less than one-half the cross-sectional area of the thicker plate.

Article 6.10.2.2 contains two additional flange proportion limits as follows:

$$\frac{b_f}{2t_f} \leq 12.0 \qquad \text{Eq. (6.10.2.2-1)}$$

$$0.1 \leq \frac{I_{yc}}{I_{yt}} \leq 10 \qquad \text{Eq. (6.10.2.2-4)}$$

where: I_{yc} = moment of inertia of the compression flange of the steel section about the vertical axis in the plane of the web (in.^{4})

I_{yt} = moment of inertia of the tension flange of the steel section about the vertical axis in the plane of the web (in.^{4})

These criteria are each checked for the most critical case (refer to Figure 3):

$$\frac{b_f}{2t_f} = \frac{18}{2(0.875)} = 10.3 < 12.0 \quad \text{ok}$$

All other flanges have a ratio of $b_f/2t_f$ less than 10.3.

$$\frac{I_{yc}}{I_{yt}} = \frac{\frac{1(16)^3}{12}}{\frac{1.375(18)^3}{12}} = 0.51$$

$$0.1 < 0.51 < 10 \quad \text{ok}$$

At all other sections, the ratio of I_{yc}/I_{yt} is greater than 0.51 and less than 10.

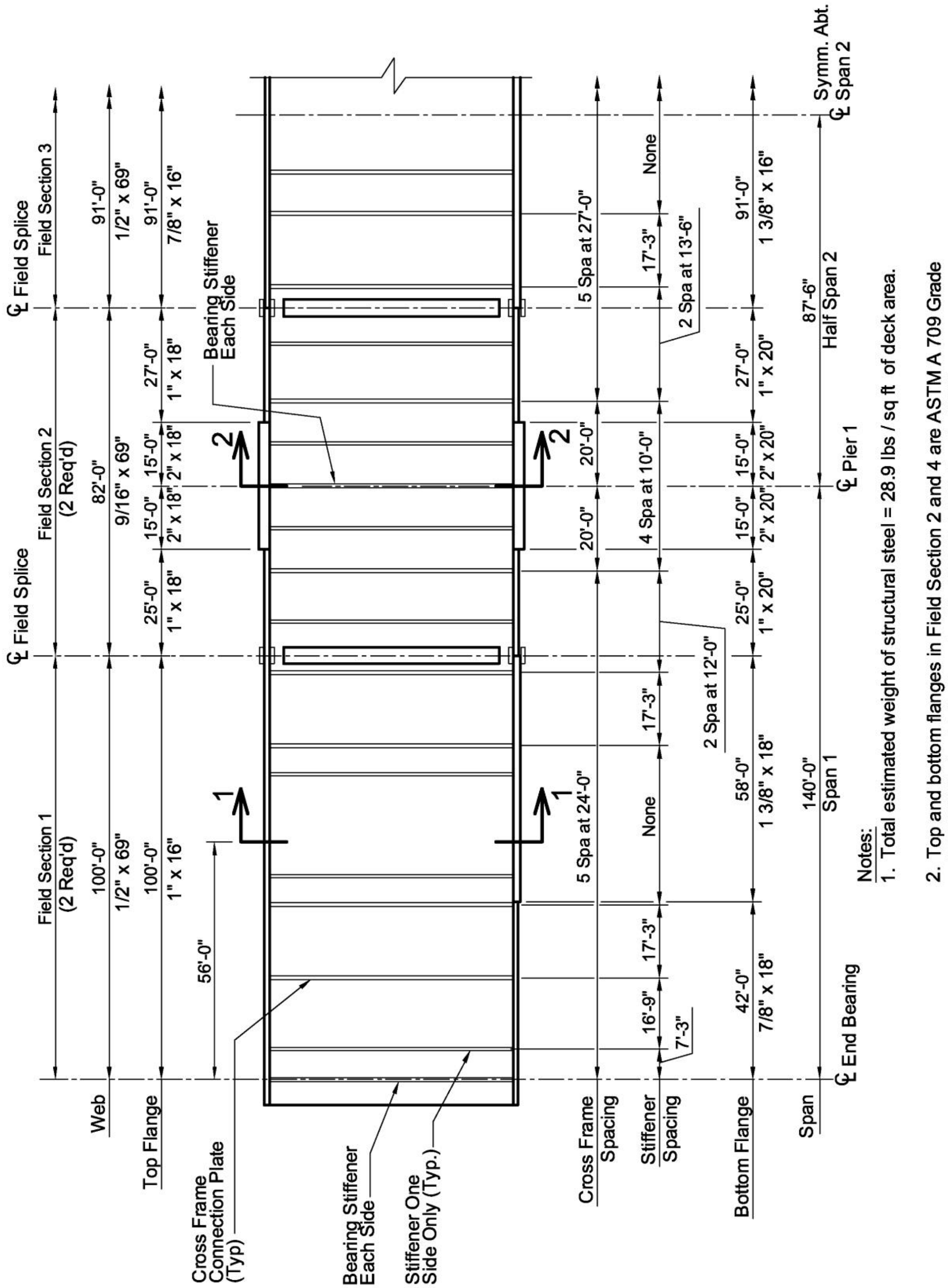

Figure 3: Elevation of Exterior Girder

6.0 LOADS

6.1. Dead Loads

As specified 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.

In the LRFD specification, the component dead load DC is assumed to consist of all the structure dead load except for any non-integral wearing surfaces and any specified utility loads. For composite steel-girder design, DC is assumed divided into two separate parts: 1) DC acting on the non-composite section (DC_1), and 2) DC acting on the composite section (DC_2). As specified in Article 6.10.1.1.1a, DC_1 represents permanent component load that is applied before the concrete deck has hardened or is made composite, and is assumed carried by the steel section alone. DC_2 represents permanent component load that is applied after the concrete deck has hardened or is made composite, and is assumed carried by the long-term composite section. For computing stresses from moments, the stiffness of the long-term composite section in regions of positive flexure is calculated by transforming the concrete deck using a modular ratio of $3n$ to account in an approximate way for the effect of concrete creep (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).

As discussed previously, cross-frames in steel-girder bridges, along with the concrete deck, provide restoring forces that tend to make the steel girders deflect equally. Under the component dead load, DC_1, applied prior to hardening of the deck or before the deck is made composite, the cross-frames are the only members available to provide the restoring forces that prevent the girders from deflecting independently. Therefore, aside from deflections resulting from elastic shortening of the cross-frames, which are generally negligible, it is reasonable to assume for typical deck overhangs and for bridges with approximately equal girder stiffnesses at points of connection of the cross-frames (e.g. straight bridges with approximately equal-size girders and bearing lines skewed not more than approximately 10° from normal) that all girders in the cross-section will resist the DC_1 loads equally. This assumption has been borne out analytically and in the field. Other assumptions may potentially lead to problems in the field, particularly when the DC_1 deflections are large. Therefore, in this example, the total DC_1 load will be assumed equally distributed to each girder in the cross-section. Note that Article 4.6.2.2.1 permits the permanent load of the deck to be distributed uniformly among the girders when certain specified conditions are met.

In the following, the unit weight of concrete is taken equal to 0.150 kcf (more conservative than Table 3.5.1-1), the concrete deck haunch width is taken equal to 16.0 inches, and the deck haunch thickness is conservatively taken equal to 2.75 inches (refer also to Figure 1):
Component dead load (DC_1):

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

Concrete deck overhang tapers $=2\left[\frac{1}{12}\left(\frac{13+10}{2}-9.5\right)\left(3.5-\frac{16/2}{12}\right)\right](0.150) = 0.142 \text{ kips/ft}$

Concrete deck haunches $=4\left[\frac{16(2.75)}{144}\right](0.150) = 0.183 \text{ kips/ft}$

Stay-in-place forms $=3\left[12.0-\frac{16}{12}\right](0.015) = 0.480 \text{ kips/ft}$

Cross-frames and details =0.120 kips/ft

DC_1 load per girder =6.031 kips/ft ÷ 4 girders = **1.508 kips/ft + girder self-weight**

DW in the *AASHTO LRFD (5th Edition, 2010)* consists of the dead load of any non-integral wearing surfaces and any utilities. DW is also assumed carried by the long-term composite section. DC_2 and DW are separated because different permanent-load load factors γ_p (Table 3.4.1-2) are applied to each load.

In this example, the wearing surface load, DW, is assumed applied over the 40′-0″ roadway width and equally distributed to each girder, which has been the customary practice for many years and is also permitted in Article 4.6.2.2.1 for bridges satisfying specified conditions. Over time, there has been a significant increase in the use of large concrete barriers that are often placed at the outer edges of the concrete deck. When refined methods of analysis are employed, these concrete barrier loads (the DC_2 loads in this case) should be applied at their actual locations at the outer edges of the deck, which results in the exterior girders carrying a larger percentage of these loads. Thus, in this example, the weight of each concrete barrier will be distributed equally to an exterior girder and the adjacent interior girder. The PennDOT DM-4 Design Manual follows such a practice (others have assigned 60 percent of the barrier weight to the exterior girder and 40 percent to the adjacent interior girder, while others continue to distribute the barrier weight equally to each girder). In this particular case, with only four girders in the cross-section, this is equivalent to equal distribution of the total barrier weight to all the girders, but this would not be the case when there are more girders in the cross-section. Therefore, the DW and DC_2 loads on a single exterior girder are computed as follows for this particular example:

Wearing surface load (DW) = [0.025 x 40.0]/4 girders = **0.250 kips/ft**

Component dead load -- Barrier load (DC_2) = 0.520/2 = **0.260 kips/ft**

6.2. Live Loads

In the *AASHTO LRFD (5th Edition, 2010)*, 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 of interest in this example are the basic design vehicular live load, a specified loading for optional live-load deflection evaluation, and a fatigue load, with the appropriate dynamic load allowance included.

Live loads are considered to be transient loads that are assumed applied to the short-term composite section. For computing stresses from moments, 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 at the fatigue and service limit states (see Articles 6.6.1.2.1 and 6.10.4.2.1) and when computing longitudinal flexural stresses in the concrete deck (see Article 6.10.1.1.1d).

6.2.1. Design Vehicular Live Load (Article 3.6.1.2)

The basic design vehicular live load in the LRFD specifications is designated as 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.

This represents a deviation from the traditional AASHTO approach in which the design truck or tandem is applied independently from the lane load. In the AASHTO Standard Specifications, the lane load is treated as a separate loading and one or two single concentrated loads are superimposed onto the lane loading to produce extreme force effects.

The design truck (Article 3.6.1.2.2) is equivalent to the AASHTO HS20 truck as specified previously in the AASHTO Standard Specifications with the spacing between the 32 kip rear-axle loads varied between 14 and 30 ft to produce extreme force effects (Figure 3.6.1.2.2-1). The 8 kip front axle is located at a constant distance of 14 ft from the closest rear axle. The transverse spacing of the wheels is 6 ft. The truck is assumed to occupy a design lane 12 ft in width with only one truck to be placed within each design lane (except as discussed below). The truck is to be positioned transversely within a lane to produce extreme force effects; however, the truck is to be positioned no closer than 2 ft from the edge of the design lane. For the design of the deck overhang, the truck is to be positioned no closer than 1 ft from the face of the curb or railing (Article 3.6.1.3. 1).

The design tandem (Article 3.6.1.2.3) consists of a pair of 25 kip axles spaced 4 ft apart with a transverse spacing of wheels equal to 6 ft.

The design lane load (Article 3.6.1.2.4) consists of a 0.64 kips/ft uniformly distributed load occupying a 10 ft lane width positioned to produce extreme force effects. The uniform load may be continuous or discontinuous as necessary to produce the maximum force effect.

For continuous spans, live-load moments in regions of positive flexure and in regions of negative flexure *outside* the points of permanent-load contraflexure are computed using only the HL-93 loading. For computing live-load moments in regions of negative flexure *between* the points of permanent-load contraflexure, a special negative-moment loading is also considered. For this special negative-moment loading, a second design truck is added in combination with the design lane load (Article 3.6.1.3.1). The *minimum* headway between the lead axle of the second truck and the rear axle of the first truck is specified to be 50 ft (a larger headway may be used to obtain the maximum effect). The distance between the two 32 kip rear axles of each of the design trucks is to be kept at a constant distance of 14 ft. In addition, all design loads (truck and lane) are to be reduced to 90 percent of their specified values. The live-load negative moments between points of permanent-load contraflexure are then taken as the *larger* of the moments caused by the HL-93 loading or this special negative-moment loading. The specification is currently silent regarding spans without points of permanent-load contraflexure. It is presumed that the special negative-moment loading should be considered over the entire span in such cases.

Live-load shears in regions of positive and negative flexure are to be computed using the HL-93 loading only. However, 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.

In all cases, axles that do not contribute to the extreme force effects under consideration are to be neglected.

For strength limit state and live-load deflection checks, a 33 percent dynamic load allowance (or impact factor) is applied only to the design truck or tandem portion of the HL-93 design live load or to the truck portion of the special negative-moment loading (Article 3.6.2). The dynamic load allowance is not to be applied to the lane portion of the loadings. As a result, the dynamic load allowance implicitly remains a function of the span length, although the span length is not explicitly used to compute the allowance.

The live-load models discussed above are not intended to represent a particular truck, but rather they are intended to simulate the moments and shears produced by groups of vehicles routinely permitted on highways of various states under "grandfather" exclusions to weight laws. The moment and shear effects from these notional live-load models were compared to selected weigh-in-motion data, the results of truck weight studies, the Ontario Highway Bridge Design Code live-load model, and statistical projections of 75-year vehicles, and were found to be representative when scaled by appropriate load factors. The HS20 and HS25 vehicles, as specified previously in the AASHTO Standard Specifications, by themselves were not considered to be accurate representations of the exclusion loads over a wide range of spans that were studied.

6.2.2. Loading for Optional Live-Load Deflection Evaluation (Article 3.6.1.3.2)

The vehicular live load for checking the optional live-load deflection criterion specified in Article 2.5.2.6.2 is taken as the larger of:

- the design truck alone.
- 25 percent of the design truck along with the design lane load.

These loadings are used to produce apparent live-load deflections similar to those produced by the previous 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). The appropriate multiple presence factors specified in Article

3.6.1.1.2 (discussed later) are to be applied. For composite design, Article 2.5.2.6.2 also permits the stiffness of the design cross-section used for the determination of the deflection to include the entire width of the roadway and the structurally continuous portions of any railings, sidewalks and barriers. The bending stiffness of an individual girder may be taken as the stiffness, determined as described above, divided by the number of 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.

6.2.3. Fatigue Load (Article 3.6.1.4)

The vehicular live load for checking fatigue in steel structures in the *AASHTO LRFD (5th Edition, 2010)* consists of a single design truck (without the lane load) with a constant rear-axle spacing of 30 ft (Article 3.6.1.4.1). The fatigue load is used to represent the variety of trucks of different types and weights in actual traffic. The constant rear-axle spacing approximates that for the 4- and 5-axle semi-trailers that do most of the fatigue damage to bridges.

The AASHTO fatigue-design procedures given in the Standard Specifications did not accurately reflect actual fatigue conditions in bridges; these procedures combined an artificially high fatigue stress range with an artificially low number of stress cycles to achieve a reasonable design. The specified fatigue load in the LRFD specifications produces a lower calculated stress range than produced by the loadings in the Standard Specifications. This reduction in calculated stress range is offset by an increase in the number of cycles of loading to be considered in the LRFD specifications. The lower stress range and the increased number of cycles are believed to be more reflective of actual conditions experienced by many bridges.

The number of cycles to be considered is the number of cycles due to the trucks actually anticipated to cross the bridge in the most heavily traveled lane in one direction averaged over its design life. This Average Daily Truck Traffic (ADTT) can be estimated as a reasonable fraction of the Average Daily Traffic (including all vehicles), which research has shown to be limited to about 20,000 vehicles per lane per day under normal conditions. In the absence of site-specific data, Table C3.6.1.4.2-1 in the Commentary to Article 3.6.1.4.2 may be used to estimate the fraction of trucks in the traffic. The frequency of the fatigue load is then taken as the *single lane* average daily truck traffic, $(ADTT)_{SL}$. In the absence of better information, $(ADTT)_{SL}$ can be computed by multiplying the ADTT by the fraction of truck traffic in a single lane p given in Table 3.6.1.4.2-1. It is believed adequate to assume that only one fatigue truck is on the bridge at a given time.

Two FATIGUE load combinations are given in Table 3.4.1-1 of the *AASHTO LRFD (5th Edition, 2010)*. The FATIGUE I load combination is to be used when designing a detail or component for an infinite fatigue life, and a load factor of 1.5 is applied to the fatigue stress range. The FATIGUE II load combination is to be used when designing a detail or component for a finite fatigue life, and a load factor of 0.75 is applied to the fatigue stress range.

The load factor of 0.75 for the FATIGUE II load combination, applied to the single design truck, reflects a load level found to be representative of the effective stress range of the truck population with respect to a small number of stress range cycles and to their cumulative effects in steel elements, components, and connections for finite fatigue life design. The load factor of 1.5 for the FATIGUE I load combination, applied to the single design truck, reflects the load levels found to be representative of the maximum stress range of the truck population for infinite fatigue life design. The load factor for FATIGUE I was chosen on the assumption that the maximum stress range in the random variable spectrum is twice the effective stress range caused by the FATIGUE II load combination.

Which fatigue load combination to use is dependent on the detail or component being designed and the projected 75-year single lane Average Daily Truck Traffic, $(ADTT)_{SL}$. Except for fracture critical members, as stated in Article 6.6.1.2.3, when the $(ADTT)_{SL}$ is greater than the value specified in Table 6.6.1.2.3-2 of the LRFD Specifications, the component or detail should be designed for infinite fatigue life using the Fatigue I load combination. Otherwise the component or detail shall be designed for finite fatigue life using the FATIGUE II load combination. For the FATIGUE I load combination, the factored fatigue stress range is checked against the constant amplitude fatigue threshold, and will typically be used for details on bridges subjected to high traffic volumes. For details on bridges with very low traffic volumes, or lower category details, the FATIGUE II combination is used, where the finite life resistance of the detail is computed from an equation defining the slope of the log S-log N curve for that detail. However, for non-fracture critical members, the designer can simply check that the stress range due to the FATIGUE I load combination is less than constant-amplitude fatigue threshold, thus ensuring an infinite fatigue life.

It is important to remember that fatigue is only to be considered if the maximum tensile stress due to twice the factored fatigue load at a particular detail is greater than or equal to the unfactored permanent load compressive stress at that detail, as specified in Article 6.6.1.2.1.

Where the bridge is analyzed using approximate analysis methods, the specified lateral live-load distribution factors for one traffic lane loaded are to be used in the fatigue check. Where the bridge is analyzed by any refined method, the single design truck is to be positioned transversely and longitudinally to maximize the stress range at the detail under consideration. A reduced dynamic load allowance of 15 percent is to be applied to the fatigue load (Article 3.6.2).

6.3. Wind Loads

The design horizontal wind pressure, P_D, used to compute the wind load on the structure, WS, is determined as specified in Article 3.8.1. It will be assumed that the example bridge superstructure is 35 feet above the low ground and that it is located in open country.

In the absence of more precise data, the design horizontal wind pressure is to be determined as follows:

$$P_D = P_B \left(\frac{V_{DZ}}{V_B} \right)^2 = P_B \frac{V_{DZ}^2}{10{,}000} \qquad \text{Eq. (3.8.1.2.1-1)}$$

where: P_B = base wind pressure 0.050 ksf for beams (Table 3.8.1.2.1-1)
V_{DZ} = design wind velocity at design elevation, Z (mph)

V_B = base wind velocity at 30 ft height 100 mph

For bridges or parts of bridges more than 30 feet above low ground, V_{DZ} is to be adjusted as follows:

$$V_{DZ} = 2.5V_o\left(\frac{V_{30}}{V_B}\right)\ln\left(\frac{Z}{Z_o}\right) \qquad \text{Eq. (3.8.1.1-1)}$$

where: V_o = friction velocity = 8.20 mph for open country (Table 3.8.1.1-1)
V_{30} = wind velocity at 30 feet above low ground = V_B = 100 mph in the absence of better information
Z = height of the structure measured from low ground (> 30 feet)
Z_o = friction length of upstream fetch = 0.23 feet for open country (Table 3.8.1.1-1)

Therefore,

$$V_{DZ} = 2.5(8.20)\left(\frac{100}{100}\right)\ln\left(\frac{35}{0.23}\right) = 103.0 \text{ mph}$$

$$P_D = 0.050\left[\frac{(103.0)^2}{10{,}000}\right] = 0.053 \text{ ksf}$$

P_D is to be assumed uniformly distributed on the area exposed to the wind. The exposed area is to be the sum of the area of all components as seen in elevation taken perpendicular to the assumed wind direction. The direction of the wind is to be varied to determine the extreme force effect in the structure or its components. For cases where the wind is not taken as normal to the structure, lateral and longitudinal components of the base wind pressure, P_B, for various angles of wind direction (assuming V_B = 100 mph) are given in Table 3.8.1.2.2-1. The angles are assumed measured from a perpendicular to the longitudinal axis. As specified in Article 3.8.1.2.1, the total wind load, WS, on girder spans is not to be taken less than 0.3 klf.

Assuming no superelevation for the example bridge and a barrier height of 42 inches above the concrete deck, the minimum exposed height of the composite superstructure is computed as:

$$h_{exp.} = (0.875 + 69.0 + 3.5 + 9.5 + 42.0)/12 = 10.41 \text{ ft}$$

The total wind load per unit length, w, for the case of wind applied normal to the structure is computed as:

$$w = P_D h_{exp.} = 0.053(10.41) = 0.55 \text{ kips/ft} > 0.3 \text{ kips/ft} \qquad \text{ok}$$

Wind pressure on live load, WL, is specified in Article 3.8.1.3. Wind pressure on live load is to be represented by a moving force of 0.1 klf acting normal to and 6 feet above the roadway, which results in an overturning force on the vehicle similar to the effect of centrifugal force on vehicles traversing horizontally curved bridges. The horizontal line load is to be applied to the same tributary area as the design lane load for the force effect under consideration. When wind on live load is not taken normal to the structure, the normal and parallel components of the force applied to the live load may be taken from Table 3.8.1.3-1.

Finally, for load cases where the direction of the wind is taken perpendicular to the bridge and there is no wind on live load considered, a vertical wind pressure of 0.020 ksf applied to the entire width of the deck is to be applied in combination with the horizontal wind loads to investigate potential overturning of the bridge (Article 3.8.2). This load case is not investigated in this example.

6.4. Load Combinations

Four limit states are defined in the LRFD specifications to satisfy the basic design objectives of LRFD; that is, to achieve safety, serviceability, and constructibility. Each of these limit states is discussed in more detail later on. 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.

As evident from the above equation, 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. The grouping of these three effects on the load side of the above equation through the use of the load modifier η_i represents an initial attempt at their codification. Improved quantification of these effects may be possible in the future. For loads for which a maximum value of γ_i is appropriate:

$$\eta_i = \frac{1}{\eta_D \eta_R \eta_I} \leq 1.0 \qquad \text{Eq. (1.3.2.1-2)}$$

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. Therefore, for the example design, η_i will be taken equal to 1.0 at all limit states.

The load combinations are presented in Table 3.4.1-1. STRENGTH I is the load combination to be used for checking the strength of a member or component under normal use in the absence of wind. The basic STRENGTH I load combination is 1.25 times the permanent load of member components (e.g. the concrete deck and parapets), plus 1.5 times the load due to any non-integral wearing surfaces and utilities, plus 1.75 times the design live load. When evaluating the strength of the structure during construction, the load factor for construction loads, for equipment and for dynamic effects (i.e. temporary dead and/or live loads that act on the structure during construction) is not to be taken less than 1.5 in the STRENGTH I load combination (Article 3.4.2). Also, the load factor for any non-integral wearing surface and utility loads may be reduced from 1.5 to 1.25 when evaluating the construction condition.

To check the strength of a member or component under special permit loadings in the absence of wind, the STRENGTH II load combination should be used. The STRENGTH II load combination is the same as the STRENGTH I load combination with the live-load load factor reduced to 1.35.

The STRENGTH III load combination is to be used for checking strength of a member or component assuming the bridge is exposed to a wind velocity exceeding 55 miles per hour in the absence of live load. The basic STRENGTH III load combination is 1.25 times the permanent load of member components, plus 1.5 times the load due to any non-integral wearing surfaces and utilities, plus 1.4 times the wind load on the structure. Note that the load factor for wind may be reduced to not less than 1.25 when checking the STRENGTH III load combination during construction (Article 3.4.2). Also, for evaluating the construction condition, the load factor for temporary dead loads that act on the structure during construction is not to be taken less than 1.25 and the load factor for any non-integral wearing surface and utility loads may be reduced from 1.5 to 1.25.

In the STRENGTH IV load combination, all permanent-load effects (for both the construction and final conditions) are factored by 1.5 and both live- and wind-load effects are not included. For the bridge in its final condition, the STRENGTH IV load combination basically relates to very high dead-to-live load force effect ratios. For longer-span bridges in their final condition, the ratio of dead-to-live load force effects is very high and could result in a set of resistance factors different from those determined to be suitable for the sample of smaller-span bridges (with spans not exceeding 200 ft) that were used in the calibration of the specification. Rather than using two sets of resistance factors with the STRENGTH I load combination, it was decided that it would be more practical to include this separate load case. It has also been found that this particular load combination can control during the investigation of various construction stages.

Finally, the STRENGTH V load combination is to be used to check the strength of a member or component assuming the bridge is exposed to a wind velocity equal to 55 miles per hour under normal use. The basic STRENGTH V load combination is 1.25 times the permanent load of member components, plus 1.5 times the load due to any non-integral wearing surfaces and utilities, plus 1.35 times the design live load (or any temporary live loads acting on the structure when evaluating the construction condition), plus 0.4 times the wind load on the structure, plus 1.0 times the wind on the live load. For evaluating the construction condition under the STRENGTH V load combination, the load factor for temporary dead loads that act on the structure during construction is not to be taken less than 1.25 and the load factor for any non-integral wearing surface and utility loads may be reduced from 1.5 to 1.25.

EXTREME EVENT I is the load combination including earthquake loading. EXTREME EVENT II is the load combination relating to vehicle and ship collisions and ice loads.

SERVICE I relates to normal operational use of the bridge and would be used primarily for crack control in reinforced concrete structures. However, the live-load portion of the SERVICE I load combination is used for checking live-load deflection in steel bridges. SERVICE II is used only for steel structures and corresponds to the Overload level in Standard Specifications. In the SERVICE II load combination, the permanent-load load factors are all reduced to 1.0 and the live-load load factor is reduced to 1.3. If the SERVICE II load combination is to be applied to a permit-load situation, consideration should be given to reducing the live-load load factor further. SERVICE III is used for crack control in prestressed concrete structures. Finally, there are the FATIGUE I and FATIGUE II load combinations, which have previously been discussed.

In strength load combinations where one force effect decreases another force effect, the specified minimum values of the load factors γ_p in Table 3.4.1-2 are to be applied instead to the permanent-load force effects. For example, when checking for uplift at end supports, the load factor applied to the permanent load of member components would be reduced from 1.25 to 0.90. The load factor applied to the non-integral wearing surface loads (if considered in this check) and utility loads would be reduced from 1.50 to 0.65.

In this particular example, the following load combinations will be evaluated. Only the maximum permanent-load load factors γ_p (from Table 3.4.1-2) are used in the following load combinations since uplift is not a concern for this particular bridge geometry.

STRENGTH I: $1.25DC + 1.5DW + 1.75(LL+IM)$
STRENGTH III: $1.25DC + 1.5DW + 1.4WS$
STRENGTH IV: $1.5(DC+DW)$
STRENGTH V: $1.25DC + 1.5DW + 1.35(LL+IM) + 0.4WS + 1.0WL$

Load factors are modified as specified as specified in Article 3.4.2 when checking the strength of a member or component during construction. No permit vehicle is specified in this example;

therefore, load combination STRENGTH II is not checked. The effect of the thermal gradient is not included. Extreme event limit state checks are also not demonstrated in this example.

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

In the above, LL is the HL-93 vehicular live load or the special negative-moment loading, WS is the wind load on the structure, and WL is the wind on the live load.

FATIGUE I: 1.50(LL+IM)
FATIGUE II: 0.75(LL+IM)

where LL is the fatigue load specified in Article 3.6.1.4.1.

SERVICE I and SERVICE III are not directly applicable to steel girder structures. However, the live-load deflection check will be performed as specified in Article 2.5.2.6.2 using the live-load portion of load combination SERVICE I, including the dynamic load allowance, as follows:

1.00(LL+IM)

where LL is the live loading for live-load deflection evaluation specified in Article 3.6.1.3.2.

7.0 STRUCTURAL ANALYSIS

Structural analysis is covered in Section 4 of the *AASHTO LRFD (5th Edition, 2010)*. Both approximate and refined methods of analysis are discussed in detail. 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. However, for this particular 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. Multiple Presence Factors (Article 3.6.1.1.2)

Multiple presence factors to account for the probability of coincident loadings are presented in Section 3 of the *AASHTO LRFD (5th Edition, 2010)* (Table 3.6.1.1.2-1). The factors are different than the factors given in the Standard Specifications. The extreme live-load force effect is to be determined by considering each possible combination of number of loaded lanes multiplied by the corresponding multiple presence factor. However, the specified multiple presence factors are only to be applied when the lever rule (discussed below), the special requirement for exterior girders assuming rigid rotation of the cross-section (also discussed below), or refined analysis methods are employed. The factors are *not* to be applied when the tabularized equations for live--load distribution factors given in the specification are used, as the multiple presence effect has already been factored into the derivation of the equations.

As specified in Article 3.6.1.1.2, multiple presence factors are also not to be applied to the fatigue limit state check for which one design truck is used. Therefore, when using the tabularized equation for the distribution factor for one-lane loaded *in the fatigue limit-state check,* the 1.2 multiple presence factor for one-lane loaded must be divided out of the calculated factor. Or, when using the lever rule or the special analysis to compute the factor for one-lane loaded for the exterior girder for the fatigue checks, the 1.2 multiple presence factor is not to be applied. The specified 1.2 multiple presence factor for one-lane loaded results from the fact that the statistical calibration of the LRFD specifications was based on pairs of vehicles rather than a single vehicle. The factor of 1.2 accounts for the fact that a single vehicle that is heavier than each one of a pair of vehicles (in two adjacent lanes) can still have the same probability of occurrence.

The proper use of the multiple presence factors is demonstrated below in the calculation of the live-load distribution factors for the example bridge.

7.2. Live-Load Distribution Factors (Article 4.6.2.2)

Equations for the lateral live-load distribution factors for I-girders, based on research done under NCHRP Project 12-26, are incorporated in the LRFD specifications. The factors vary according to the type of deck and girders, the number of design lanes loaded, and whether the girder is an interior or exterior girder. The factors are generally dependent on the span length, transverse girder spacing, and the stiffness of the member.

For example, the live-load distribution factor for the interior-girder bending moment for steel I-girder bridges with a concrete deck loaded by two or more design lanes is given as follows (Table 4.6.2.2.2b-1):

$$g = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$$

where: g = live-load distribution factor for bending moment (in units of lanes)
S = girder spacing (3.5 ft ≤ S ≤ 16 ft)
L = span length (20 ft ≤ L ≤ 240 ft) (see Table C4.6.2.2.1-1 for suggested values of L to use)
t_s = structural concrete deck thickness (4.5 in. ≤ t_s ≤ 12 in.)
K_g = $n(I+Ae_g^2)$
n = modular ratio
I = moment of inertia of the steel girder
A = cross-sectional area of the steel girder
e_g = distance from the centroid of the steel girder to the mid-point of the concrete deck

A different equation is given to compute the distribution factor for one-lane loaded. Note that the results from all the formulas are given in terms of *lanes* rather than *wheels*. Since the stiffness of the girders is usually not known in advance, the stiffness term *($K_g/12.OLt_s^3$)* may be taken as unity for preliminary design. The above equation is to be used when designing in Customary U.S. units.

The use of the approximate equations for I-girder bridges is limited to bridges where the deck is supported on four or more girders. The use of these equations is also subject to the limitations on girder spacing, span length, slab thickness, etc., as noted above. For cases outside these limits, engineering judgment should be employed in extending the application of the formulas beyond the limits, or else other approaches such as refined analysis methods may be used. When the upper limitation on girder spacing is exceeded, Article 4.6.2.2.1 requires that the lever rule (discussed below) be used to compute the lateral distribution of load to the individual girders, unless otherwise specified. The distribution factor for interior girders, as determined from the above equation, will generally result in lower live-load bending moments than when the moments are computed using a factor of S/5.5 as specified previously in the AASHTO Standard Specifications, except possibly for very short spans.

For exterior girders when two or more design lanes are loaded, a correction factor is applied to the computed distribution factor for the interior girders to compute the fraction of the wheel loads distributed to the exterior girders. The correction factor depends on the distance from the centerline of the exterior girder to the edge of the curb (Table 4.6.2.2.2d-1).

To compute the distribution factor for an exterior girder when one lane is loaded, the lever rule is applied. The lever rule involves the use of statics to determine the wheel-load reaction at the exterior girder by summing moments about the adjacent interior girder assuming the concrete deck is hinged at the interior girder.

For steel girders utilizing diaphragms or cross-frames, it is also specified that the distribution of live load to the exterior girders is not to be less than that computed from a special analysis assuming the entire bridge cross-section deflects and rotates as a rigid body. This latter clause was instituted into the specifications primarily because the distribution-factor formulas were developed without consideration of diaphragms or cross-frames and their effect on the distribution of load to the exterior girders of steel I-girder bridges. A formula to determine the reaction at an exterior girder under one or more lanes of loading based on the above assumption is given in the Commentary to Article 4.6.2.2.2d [Eq. (C4.6.2.2.2d-1)]; the procedure is equivalent to the conventional procedure used to approximate loads on pile groups.

When utilizing the lever rule and the special analysis, vehicles must be placed within their design lanes. As specified in Article 3.6.1.2.1, the HL-93 live loading is assumed to occupy a load lane width of 10 ft transversely within a 12-ft-wide design lane. Figure 3.6.1.2.2-1 shows that for the assumed transverse wheel spacing of 6 ft, a distance of 2 ft remains from the center of each wheel to each edge of the specified load lane width (note that the 6 ft transverse wheel spacing is also conservatively assumed to apply to the design lane load). The number of design traffic lanes to be placed on the bridge is determined by taking the integer part of w/12.0, where w is the roadway width measured between curbs. As specified in Article 3.6.1.1.1, roadway widths from 20 to 24 ft shall have two design lanes, each equal to one-half the roadway width. In the computation of the exterior-girder distribution factor according to the above procedures, the live loads occupying their individual load lane widths are to be placed within their design lanes. The design lanes are then to be placed within the roadway width to maximize the wheel-load reaction at the exterior girder. According to the provisions of Article 3.6.1.3.1, a wheel load can be no closer than 1 ft from the face of the curb or railing for the design of the deck overhang and 2 ft from the edge of the design lane for the design of all other components. These same rules for positioning of the live loads on the bridge would apply when performing refined analyses.

Also, as specified in Article 2.5.2.7.1, unless future widening of the bridge is virtually inconceivable, the total load carrying capacity of an exterior girder (considering dead plus live load) is not to be less than the total load carrying capacity of an interior girder. However, it should be noted that the use of the refined distribution factors given in the LRFD Specifications, along with the assumption of equal distribution of the DC_1 loads to each girder and the suggested increase in the percentage of the barrier weight assigned to the exterior girders (as discussed above), will typically result in larger total factored moments in the exterior girders than the interior girders, unless the deck overhangs are very small. For this reason, it is recommended that deck overhangs be limited to approximately 35 percent (or less) of the transverse girder spacing, if possible, to ensure a reasonable balance of the total moments in the interior and exterior girders.

Separate distribution factors are given for determining the bending moment and shear in individual I girders. The distribution factors for shear are specified in Tables 4.6.2.2.3a-1 and 4.6.2.2.3b-1 for interior and exterior girders, respectively. Correction factors, given in Tables 4.6.2.2.2e-1 and 4.6.2.2.3c-1, may be applied to the individual distribution factors for bending moment and shear to account, in a limited way, for the effects of skewed supports. Dead-load effects are currently not adjusted for the effects of skew.

The computation of the live-load distribution factors for an interior and exterior girder from the example bridge, utilizing the approximate methods discussed above is now illustrated.

7.2.1. Live-Load Lateral Distribution Factors - Positive Flexure

The following preliminary cross-section (Figure 4) is assumed to determine the longitudinal stiffness parameter K_g that is utilized in the approximate formulas to compute the live-load distribution factors for regions in positive flexure (refer also to Figure 3):

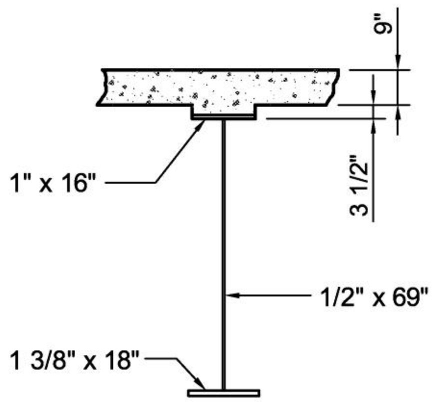

Figure 4: Preliminary Cross-section – Positive Flexure

Table 1 Preliminary Section Properties for Positive Flexure (Steel Only)

Component	A	d	Ad	Ad^2	I_o	I
Top Flange 1" x 16"	16.00	35.00	560.0	19,600	1.33	19,601
Web ½" x 69"	34.50				13,688	13,688
Bottom Flange 1⅜"x18"	24.75	−35.19	−871.0	30,649	3.90	30,653
	75.25		-311.0			63,942
					−4.13(311.0)	-1,284
					I_{NA} =	62,658 in.⁴

$$d_s = \frac{-311.0}{75.25} = -4.13 \text{ in.}$$

$$d_{TOP OF STEEL} = 35.50 + 4.13 = 39.63 \text{ in.} \qquad d_{BOT OF STEEL} = 35.88 - 4.13 = 31.75 \text{ in.}$$

$$S_{TOP OF STEEL} = \frac{62,658}{39.63} = 1,581 \text{ in.}^3 \qquad S_{BOT OF STEEL} = \frac{62,658}{31.75} = 1,973 \text{ in.}^3$$

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 0.145 kcf will be used for the concrete in the calculation of the modular ratio (since 0.005 kcf of the specified unit weight of 0.150 kcf is typically assumed to account for the weight of the reinforcement). The correction factor for source of aggregate, K_1, is taken as 1.0.

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

$$E_c = 33{,}000(1.0)(0.145)^{1.5}\sqrt{4.0} = 3{,}644 \text{ ksi}$$

$$n = \frac{29{,}000}{3{,}644} = 7.96$$

Note that for normal-density concrete, Article C6.10.1.1.1b permits *n* to be taken as 8 for 4.0-ksi concrete. Therefore, n = 8 will be used in all subsequent computations.

$$e_g = \frac{9.0}{2} + 3.5 + 39.63 - 1.0 = 46.63 \text{ in.}$$

$$K_g = n\left(I + Ae_g^2\right) = 8\left(62{,}658 + 75.25(46.63)^2\right) = 1.81 \text{ x } 10^6 \text{ in.}^4$$

For preliminary design, the entire term containing K_g in the approximate formulas may be taken as 1.0. Although the K_g term varies slightly along the span and between spans, the value at the maximum positive moment section in the end span is used in this example to compute the distribution factor to be used in all regions of positive flexure. Other options are to compute a separate K_g in each span based on the average or a weighted average of the properties along each span in the positive-flexure region, or to compute K_g based on the actual values of the section properties at each change of section resulting in a variable distribution factor along each span within the positive-flexure region. However, the distribution factor is typically not overly sensitive to the value of K_g that is assumed.

The girders satisfy the limitations defining the range of applicability of the approximate formulas; these limitations are specified in the individual tables containing the formulas. For example, the number of girders in the cross-section is greater than or equal to four, the transverse girder spacing is greater than or equal 3'-6" and less than or equal to 16'-0", and the span length is greater than or equal to 20'-0" and less than or equal to 240'-0". The limitations on K_g (specified for the shear distribution factor only) and on the slab thickness are also satisfied. The computation of the distribution factors (in units of lanes) is illustrated below.

7.2.1.1. Interior Girder - Strength Limit State

The live-load distribution factors for an interior girder for checking the strength limit state are determined using the approximate formulas given in the indicated tables. Multiple presence factors (Article 3.6.1.1.2) are not explicitly applied because these factors were included in the derivation of these formulas. Separate factors are given to compute the bending moment and shear. For regions in positive flexure, Table C4.6.2.2.1-1 suggests using the length of the span under consideration for *L*.

Bending Moment (Table 4.6.2.2.2b-1):

One lane loaded:

$$0.06+\left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_S^3}\right)^{0.1}$$

$$0.06+\left(\frac{12.0}{14}\right)^{0.4}\left(\frac{12.0}{140.0}\right)^{0.3}\left(\frac{1.81\,x\,10^6}{12.0(140.0)(9.0)^3}\right)^{0.1}=0.528\ \text{lanes}$$

Two or more lanes loaded:

$$0.075+\left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_S^3}\right)^{0.1}$$

$$0.075+\left(\frac{12.0}{9.5}\right)^{0.6}\left(\frac{12.0}{140.0}\right)^{0.2}\left(\frac{1.81\,x\,10^6}{12.0(140.0)(9.0)^3}\right)^{0.1}=0.807\ \text{lanes (governs)}$$

Shear (Table 4.6.2.2.3a-1):

One lane loaded:

$$0.36+\frac{S}{25.0}$$

$$0.36+\frac{12.0}{25.0}=0.840\ \text{lanes}$$

Two or more lanes loaded:

$$0.2+\frac{S}{12}-\left(\frac{S}{35}\right)^2$$

$$0.2+\frac{12.0}{12}-\left(\frac{12.0}{35}\right)^2=1.082\ \text{lanes (governs)}$$

7.2.1.2. Exterior Girder - Strength Limit State

The live-load distribution factors for an exterior girder for checking the strength limit state are determined as the governing factors calculated using a combination of the lever rule, approximate formulas, and a special analysis assuming that the entire cross-section deflects and rotates as a rigid body. Each method is illustrated below. As stated in Article 3.6.1.1.2, multiple presence factors are included at the strength limit state when the lever rule and the special analysis are used. Separate factors are again computed for bending moment and shear.

Bending Moment:

One lane loaded: Use the lever rule (Table 4.6.2.2.2d-1)

The lever rule involves the use of statics to determine the lateral distribution to the exterior girder by summing moments about the adjacent interior girder to find the wheel-load reaction at the exterior girder assuming the concrete deck is hinged at the interior girder (Figure 5). A wheel cannot be closer than 2'-0" to the base of the curb (Article 3.6.1.3.1). For the specified transverse wheel spacing of 6'-0", the wheel-load distribution to the exterior girder is computed as:

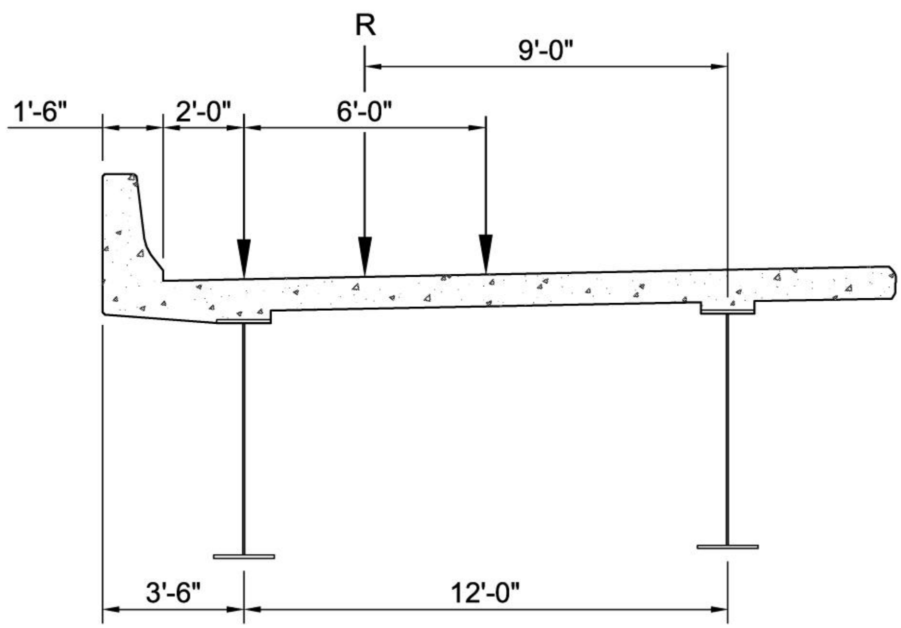

Figure 5: Exterior-Girder Distribution Factor - Lever Rule

$$\frac{9.0}{12.0} = 0.750$$

$$\text{Multiple presence factor } m = 1.2 \text{ (Table 3.6.1.1.2-1)}$$

$$1.2(0.750) = 0.900 \text{ lanes}$$

Two or more lanes loaded: Modify interior-girder factor by *e* (Table 4.6.2.2.2d-1)

$$e = 0.77 + \frac{d_e}{9.1}$$

$$e = 0.77 + \frac{2.0}{9.1} = 0.990$$

$$0.990(0.807) = 0.799 \text{ lanes}$$

The factor *e* is computed using the distance d_e, where d_e is the distance from the exterior girder to the edge of the curb or traffic barrier (must be less than or equal to 5.5 ft). d_e is negative if the girder web is outboard of the curb or traffic barrier (must be greater than or equal to -1.0 ft).

The multiple presence factor is not applied.

Special Analysis (C4.6.2.2.2d - Commentary):

Assuming the entire cross-section rotates as a rigid body about the longitudinal centerline of the bridge, distribution factors for the exterior girder are also computed for one, two and three lanes loaded using the following formula:

$$R = \frac{N_L}{N_b} + \frac{X_{ext} \sum^{N_L} e}{\sum^{N_b} x^2} \qquad \text{Eq. (C4.6.2.2.2d-1)}$$

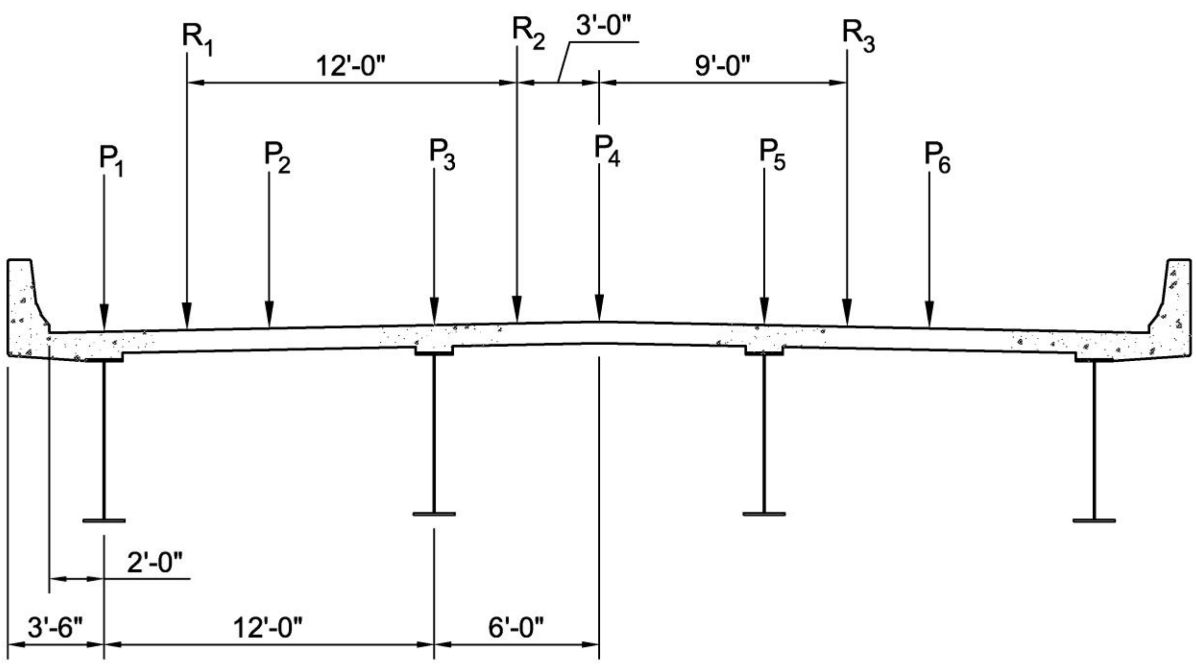

Figure 6: Exterior-Girder Distribution Factor – Special Analysis

where: R = reaction on exterior beam in terms of lanes
N_L = number of loaded lanes under consideration
e = eccentricity of a lane from the center of gravity of the pattern of girders (ft)
X = horizontal distance from the center of gravity of the pattern of girders to each girder (ft)
X_{ext} = horizontal distance from the center of gravity of the pattern of girders to the exterior girder (ft)
N_b = number of beams or girders

Multiple presence factors (Table 3.6.1.1.2-1):

1 lane:	$m_1 = 1.2$
2 lanes:	$m_2 = 1.0$
3 lanes:	$m_3 = 0.85$

Referring to Figure 6:

One lane loaded:

$$R = \frac{1}{4} + \frac{(12.0 + 6.0)(12.0 + 3.0)}{2(18.0^2 + 6.0^2)} = 0.625$$

$$m_1R = 1.2(0.625) = 0.750 \text{ lanes}$$

Two lanes loaded:

$$R = \frac{2}{4} + \frac{(12.0 + 6.0)(12.0 + 3.0 + 3.0)}{2(18.0^2 + 6.0^2)} = 0.950$$

$$m_2R = 1.0(0.950) = 0.950 \text{ lanes (governs)}$$

Three lanes loaded:

$$R = \frac{3}{4} + \frac{(12.0 + 6.0)(12.0 + 3.0 + 3.0 - 9.0)}{2(18.0^2 + 6.0^2)} = 0.975$$

$$m_3R = 0.85(0.975) = 0.829 \text{ lanes}$$

<u>Shear:</u>

One lane loaded: Use the lever rule (Table 4.6.2.2.3b-1)

0.900 lanes (See previous computation)

Two or more lanes loaded: Modify interior-girder factor by *e* (Table 4.6.2.2.3b-1)

$$e = 0.6 + \frac{d_e}{10}$$

$$e = 0.6 + \frac{2.0}{10} = 0.80$$

$$0.80(1.082) = 0.866 \text{ lanes}$$

Special Analysis (C4.6.2.2.2d - Commentary):

The factors computed for bending moment are also used for shear:

One lane loaded:	0.750 lanes
Two lanes loaded:	0.950 lanes (governs)
Three lanes loaded:	0.829 lanes

The resulting distribution factors used to check the strength limit state in regions of positive flexure are:

	Interior Girder	Exterior Girder
Bending Moment	0.807 lanes	0.950 lanes
Shear	1.082 lanes	0.950 lanes

7.2.1.3. Distribution Factors for Fatigue Limit State

When checking fatigue, the fatigue load is placed in a single lane. Therefore, the distribution factors for one-lane loaded are used when computing the stress and shear ranges due to the fatigue load, as specified in Article 3.6.1.4.3b. According to Article 3.6.1.1.2, multiple presence factors shall not be applied when checking the fatigue limit state. Therefore, the following values of the distribution factors for checking the fatigue limit state in regions of positive flexure reflect the preceding values for one-lane loaded divided by the specified multiple presence factor of 1.2 for one-lane loaded (Table 3.6.1.1.2-1):

	Interior Girder	Exterior Girder
Bending Moment	0.440 lanes	0.750 lanes
Shear	0.700 lanes	0.750 lanes

7.2.1.4. Distribution Factor for Live-Load Deflection

According to Article 2.5.2.6.2, when investigating the maximum absolute live-load deflection, all design lanes should be loaded, and all supporting components should be assumed to deflect equally. For multi-girder bridges, this is equivalent to saying that the distribution factor for computing live-load deflection is equal to the number of lanes divided by the number of girders. Also, the appropriate multiple presence factor from Article 3.6.1.1.2 shall apply as stated in Article 2.5.2.6.2.

$$DF = m_3\left(\frac{N_L}{N_b}\right)$$

$$= 0.85\left(\frac{3}{4}\right) = 0.638 \text{ lanes}$$

7.2.2. Live-Load Lateral Distribution Factors - Negative Flexure

The following preliminary cross-section (Figure 7) is assumed to determine the longitudinal stiffness parameter K_g that is utilized in the approximate formulas to compute the live-load distribution factors for regions in negative flexure (refer also to Figure 3):

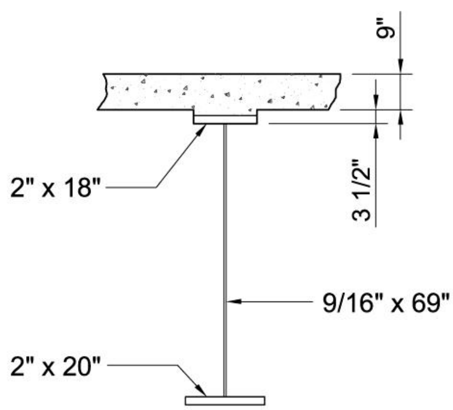

Figure 7: Preliminary Cross-Section - Negative Flexure

Table 2 Preliminary Section Properties for Negative Flexure (Steel Only)

Component	A	d	Ad	Ad^2	I_o	I
Top Flange 2" x 18"	36.00	35.50	1,278	45,369	12.00	45,381
Web 9⁄16" x 69"	38.81				15,399	15,399
Bottom Flange 2" x 20"	40.00	−35.50	−1,420	50,410	13.33	50,423
	114.8		−142.0			111,203
					-1.24(142.0) =	−176.1
					I_{NA} =	111,027 in.^{4}

$$d_s = \frac{-142.0}{114.8} = -1.24 \text{ in.}$$

$$d_{TOP\ OF\ STEEL} = 36.50 + 1.24 = 37.74 \text{ in.} \qquad d_{BOT\ OF\ STEEL} = 36.50 - 1.24 = 35.26 \text{ in.}$$

$$S_{TOP\ OF\ STEEL} = \frac{111{,}027}{37.74} = 2{,}942 \text{ in.}^3 \qquad S_{BOT\ OF\ STEEL} = \frac{111{,}027}{35.26} = 3{,}149 \text{ in.}^3$$

$$e_g = \frac{9.0}{2} + 3.5 + 37.74 - 2.0 = 43.74 \text{ in.}$$

n = 8

$$K_g = n(I + Ae_g{}^2) = 8(111{,}027 + 114.8(43.74)^2) = 2.65 \text{ x } 10^6 \text{ in.}^4$$

Again, for preliminary design, the entire term containing K_g in the approximate formulas may be taken as 1.0. In the preceding calculation, K_g is based on the section properties of the interior-pier section. K_g may instead be computed based on the section properties at each change of section resulting in a variable distribution factor along the span within the negative-flexure region, or K_g may be based on the average or weighted average of the properties along each span in the negative-flexure region.

7.2.2.1. Interior Girder - Strength Limit State

For regions in negative flexure between points of contraflexure, Table C4.6.2.2.1-1 suggests using the average length of the two adjacent spans for *L*.

Bending Moment (Table 4.6.2.2.2b-1):

One lane loaded: $$0.06 + \left(\frac{12.0}{14}\right)^{0.4}\left(\frac{12.0}{157.5}\right)^{0.3}\left(\frac{2.65 \text{ x } 10^6}{12.0(157.5)(9.0)^3}\right)^{0.1} = 0.524 \text{ lanes}$$

Two or more lanes loaded: $$0.075 + \left(\frac{12.0}{9.5}\right)^{0.6}\left(\frac{12.0}{157.5}\right)^{0.2}\left(\frac{2.65 \text{ x } 10^6}{12.0(157.5)(9.0)^3}\right)^{0.1} = 0.809 \text{ lanes (governs)}$$

All other distribution factors for regions in negative flexure for the interior girder and for the exterior girder are independent of the span length and the stiffness of the girder; therefore, they are identical to the values calculated earlier for regions in positive flexure.
The resulting distribution factors used to check strength limit state in regions of negative flexure are:

	Interior Girder	Exterior Girder
Bending Moment	0.809 lanes	0.950 lanes
Shear	1.082 lanes	0.950 lanes

7.2.2.2. Distribution Factors for Fatigue Limit State

The following values of the distribution factors for checking the fatigue limit state in regions of negative flexure reflect values computed previously for one-lane loaded divided by the specified multiple-presence factor of 1.2 for one-lane loaded (Table 3.6.1.1.2-1):

	Interior Girder	Exterior Girder
Bending Moment	0.437 lanes	0.750 lanes
Shear	0.700 lanes	0.750 lanes

7.3. Dynamic Load Allowance: IM (Article 3.6.2)

The dynamic load allowance 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)

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

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 discussed previously.
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.

8.0 ANALYSIS RESULTS

8.1. Moment and Shear Envelopes

The analysis results for the exterior girder (Figure 3) 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. The entire cross-sectional area of the deck associated with the exterior girder was assumed effective in the analysis for loads applied to the composite section. Note that for a continuous span with a nonprismatic member, changes to individual section stiffnesses 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 most always desirable to perform a new analysis.

In the first series of plots (Figures 8 and 9), 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 equal to 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. *Note that the live-load shears in Figure 9 are controlled by the interior girder in this example (the distribution factor for shear for the interior girder at the strength limit state is 1.082 lanes versus 0.950 lanes for the exterior girder).*

The second series of plots (Figures 10 and 11) 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.

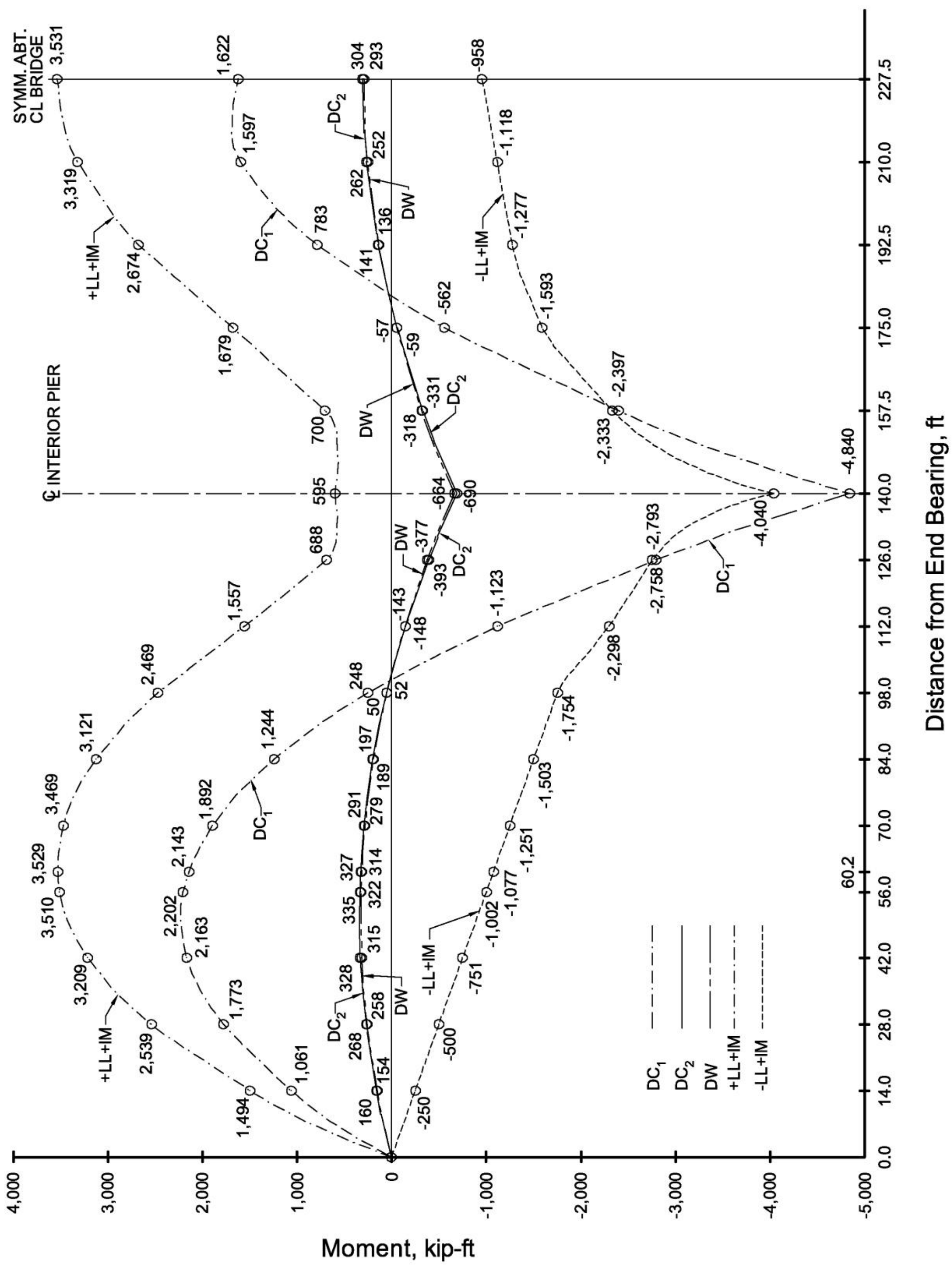

Figure 8: Dead- and Live-Load Moment Envelopes

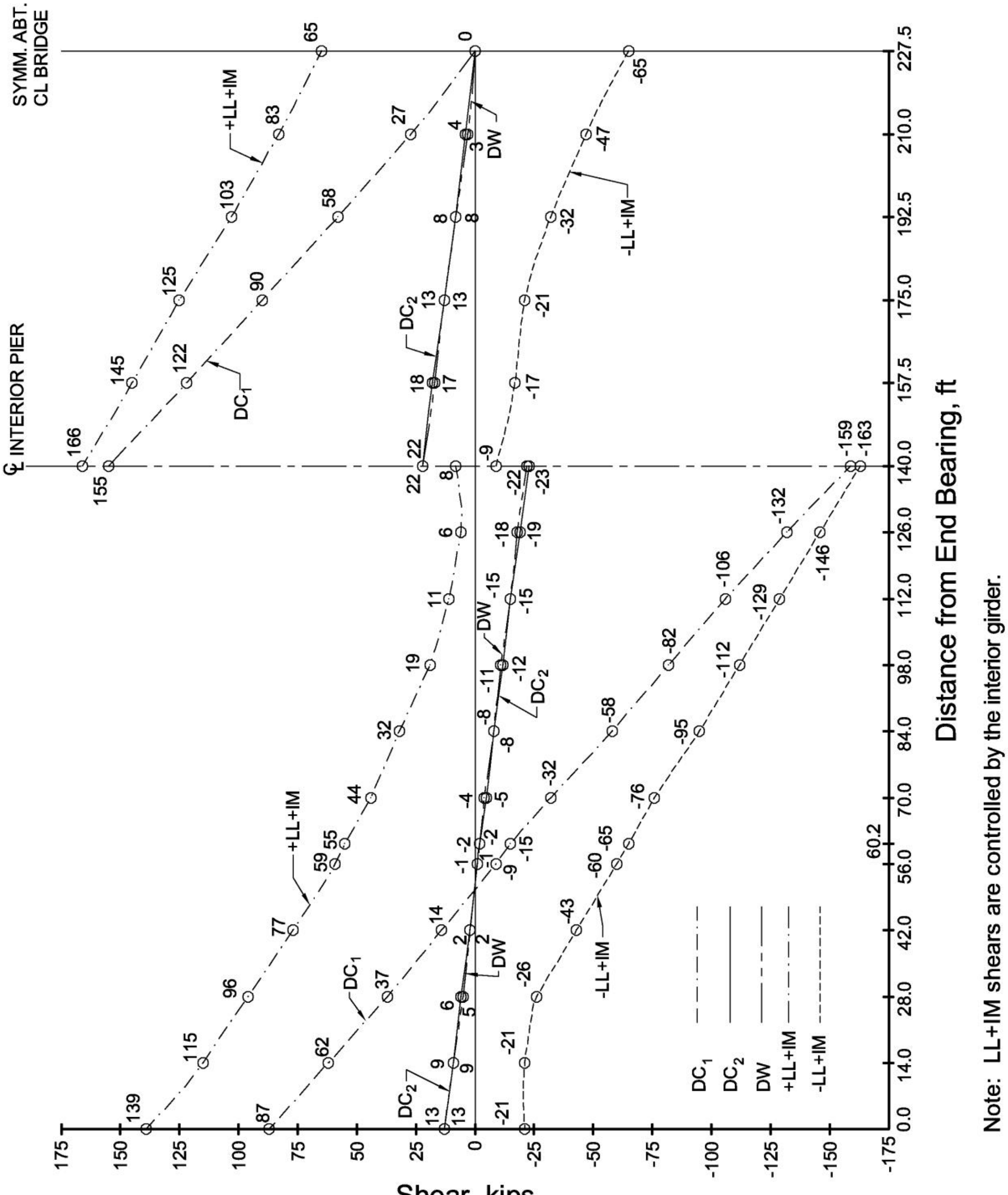

Figure 9: Dead- and Live-Load Shear Envelopes

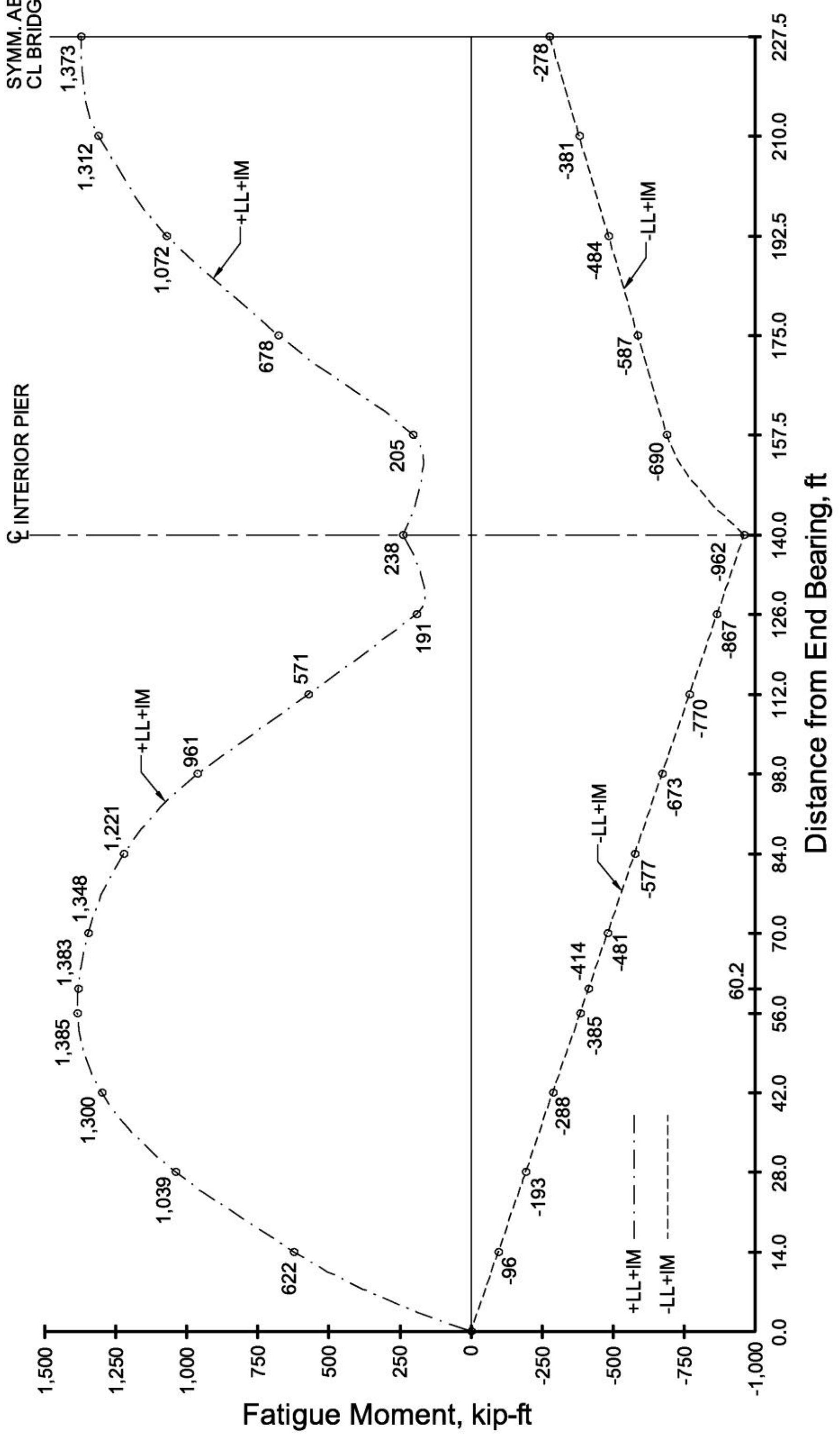

Figure 10: Fatigue-Load Moments

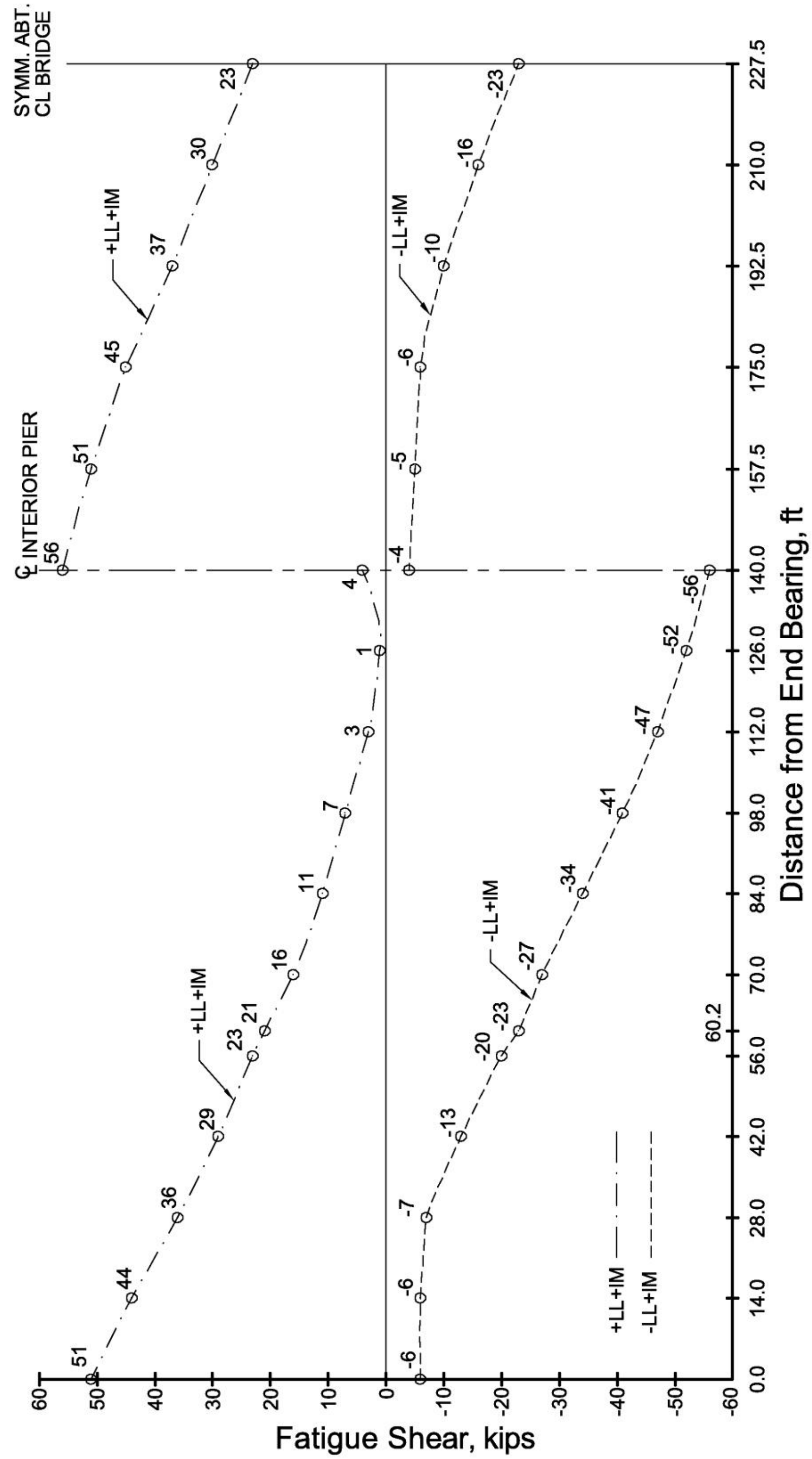

Figure 11: Fatigue-Load Shears

8.2. Live Load Deflection

As discussed previously, the optional live-load deflection check consists of evaluating two separate live-load conditions. Again, the two load conditions are (Article 3.6.1.3.2):

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

The dynamic load allowance of 33 percent is applied to the design truck in each case. A load factor of 1.0 is applied to the live load since the live-load portion of the SERVICE I load combination is to be used in the check. The lateral distribution factor for live-load deflection, computed earlier, is also used. The actual *n*-composite moments of inertia along the entire length of the girder are used in the analysis. Because live-load deflection is not anticipated to be a significant concern for the example bridge, the stiffness of the barriers is not included in the composite stiffness used in determining the live-load deflections. However, the full width of the concrete deck associated with the exterior girder (versus the effective flange width) is used in determining the composite stiffness, as recommended in Article 2.5.2.6.2 for the calculation of live-load deflections.

The maximum live-load deflections in the end span and center span due to the design truck plus the dynamic load allowance are:

$$(\Delta_{LL+IM})\text{ end span} = 0.91\text{ in. (governs)}$$
$$(\Delta_{LL+IM})\text{ center span} = 1.23\text{ in. (governs)}$$

The maximum live-load deflections in the end span and center span due to the design lane load plus 25 percent of the design truck plus the dynamic load allowance are:

$$(\Delta_{LL+IM})\text{ end span} = 0.60 + 0.25(0.91) = 0.83\text{ in.}$$
$$(\Delta_{LL+IM})\text{ center span} = 0.85 + 0.25(1.23) = 1.16\text{ in.}$$

9.0 LIMIT STATES

9.1. Service Limit State (Articles 1.3.2.2 and 6.5.2)

To satisfy the service limit state, restrictions on stress and deformation under regular service conditions are specified to ensure satisfactory performance of the bridge over its service life. As specified in Article 6.10.4.1, optional live load deflection criteria and span-to-depth ratios (Article 2.5.2.6) may be invoked to control deformations.

Steel structures must also satisfy the requirements of Article 6.10.4.2 under the SERVICE II load combination. The intent of the design checks specified in Article 6.10.4.2 is to prevent objectionable permanent deformations, caused by localized yielding and potential web bend-buckling under expected severe traffic loadings, which might impair rideability. The live-load portion of the SERVICE II load combination is intended to be the design live load specified in Article 3.6.1.1 (discussed previously). For a permit load situation, a reduction in the specified load factor for live load under the SERVICE II load combination should be considered for this limit-state check.

9.2. Fatigue and Fracture Limit State (Articles 1.3.2.3 and 6.5.3)

To satisfy the fatigue and fracture limit state, restrictions on stress range under regular service conditions are specified to control crack growth under repetitive loads and to prevent fracture during the design life of the bridge (Article 6.6.1). Material toughness requirements are also addressed (Article 6.6.2).

For checking fatigue in steel structures, the fatigue load and FATIGUE load combination (discussed previously) apply. Fatigue resistance of details is discussed in Article 6.6. A special fatigue requirement for webs (Article 6.10.5.3) is also specified to control out-of-plane flexing of the web that might potentially lead to fatigue cracking under repeated live loading.

9.3. Strength Limit State (Articles 1.3.2.4 and 6.5.4)

At the strength limit state, it must be ensured that adequate strength and stability is provided to resist the statistically significant load combinations the bridge is expected to experience over its design life. Extensive structural damage may occur, but overall structural integrity is maintained. The applicable STRENGTH load combinations (discussed previously) are used to check the strength limit state.

Although not specified as a separate limit state, constructibility is one of the basic design objectives of LRFD. The bridge must be safely erected and have adequate strength and stability during all phases of construction. Specific design provisions are given in Article 6.10.3 of the LRFD specifications to help ensure constructibility of steel I-girder bridges; in particular, when subject to the specified deck-casting sequence. The constructibility checks are typically made on the steel section only under the factored non-composite dead loads using the appropriate strength load combinations.

9.4. Extreme Event Limit State (Articles 1.3.2.5 and 6.5.5)

At the extreme event limit state, structural survival of the bridge must be ensured during a major earthquake or flood, or when struck by a vessel, vehicle, or ice flow. Extreme event limit states are not covered in this example.

10.0 SAMPLE CALCULATIONS

Sample calculations for two critical sections in an exterior girder from the example bridge follow. Section 1-1 (refer to Figure 3) represents the section of maximum positive flexure in the end spans, and Section 2-2 represents the section at each interior pier. The calculations are intended to illustrate the application of some of the more significant provisions contained in Article 6.10. The sample calculations illustrate calculations to be made at the service, fatigue and fracture and strength limit states. Detailed constructibility checks are also illustrated. Sample stiffener designs and the design of the stud shear connectors are included as well. The calculations make use of the moments and shears shown in Figures 8 through 11 and the section properties calculated below. In the calculation of the vertical bending stresses throughout the sample calculations, compressive stresses are always shown as negative values and tensile stresses are always shown as positive values. This convention is followed regardless of the expected sign of the calculation result, in which the sign of the major-axis bending moment is maintained.

Note that a direct comparison should not be made between the unit weight of the example design contained herein and the unit weight of the design given in Example 3 of the AISI/NSBA publication entitled "Four LRFD Design Examples of Steel Highway Bridges". Although the cross-section and span lengths are the same, the assumed component dead loads are significantly different in the two designs and a hybrid section is also used in regions of negative flexure in the design contained herein. This example design is NOT intended to provide a direct comparison between a girder designed using Article 6.10 provisions and Article 6.10 of *AASHTO LRFD (5th Edition, 2010)* provisions contained in preceding editions of the LRFD Specifications.

10.1. Section Properties

The calculation of the section properties for Sections 1-1 and 2-2 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. The modular ratio was computed earlier to be $n = 7.96 \therefore$ say $n = 8$.

10.1.1. Section 1-1

Section 1-1 is shown in Figure 12. For this section, the longitudinal reinforcement is conservatively neglected in computing the composite section properties.

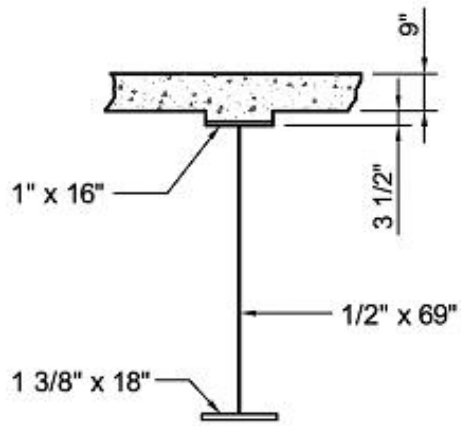

Figure 12: Section 1-1

10.1.1.1. Effective Flange Width (Article 4.6.2.6): Section 1-1

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. According to Article 4.6.2.6, for exterior girders, the effective flange width may be taken as one-half the distance to the adjacent interior girder plus the full overhang width.

Therefore, for an exterior girder, b_{eff} is equal to:

$$\frac{144.0}{2} + \text{width of the overhang} = 72.0 + 42.0\text{ in.} = 114.0\text{ in.}$$

10.1.1.2. Elastic Section Properties: Section 1-1

Table 3 Section 1-1: Steel Only Section Properties

Component	A	d	Ad	Ad^2	I_o	I
Top Flange 1" x 16"	16.00	35.00	560.0	19,600	1.33	19,601
Web ½" x 69"	34.50				13,688	13,688
Bottom Flange 1⅜" x 18"	24.75	−35.19	−871.0	30,649	3.90	30,653
	75.25		−311.0			63,942
					$-4.13(311.0) =$	−1,284
					$I_{NA} =$	62,658 in.^{4}

$$d_s = \frac{-311.0}{75.25} = -4.13\text{ in.}$$

$$d_{TOP\ OF\ STEEL} = 35.50 + 4.13 = 39.63\text{ in.}$$

$$d_{BOT\ OF\ STEEL} = 35.88 - 4.13 = 31.75\text{ in.}$$

$$S_{TOP\ OF\ STEEL} = \frac{62{,}658}{39.63} = 1{,}581\text{ in.}^3$$

$$S_{BOT\ OF\ STEEL} = \frac{62{,}658}{31.75} = 1{,}973\text{ in.}^3$$

Table 4 Section 1-1: Composite Section Properties; 3n = 24

Component	A	d	Ad	Ad^2	I_o	I
Steel Section	75.25		−311.0			63,942
Concrete Slab 9" x 114"/ 24	42.75	42.50	1,817	77,217	288.6	77,506
	118.0		1,506			141,448
					−12.76(1,506) =	−19,217
					I_{NA} =	122,231 in.^{4}

$$d_{3n} = \frac{1{,}506}{118.0} = 12.76\,\text{in.}$$

$$d_{TOPOFSTEEL} = 35.50 - 12.76 = 22.74\,\text{in.}$$

$$d_{BOTOFSTEEL} = 35.88 + 12.76 = 48.64\,\text{in.}$$

$$S_{TOP\,OF\,STEEL} = \frac{122{,}231}{22.74} = 5{,}375\,\text{in.}^3$$

$$S_{BOT\,OF\,STEEL} = \frac{122{,}231}{48.64} = 2{,}513\,\text{in.}^3$$

Table 5 Section 1-1: Composite Section Properties; n = 8

Component	A	d	Ad	Ad^2	I_o	I
Steel Section	75.25		−311.0			63,942
Concrete Slab 9" x 114"/ 8	128.25	42.50	5,451	231,652	865.7	232,518
	203.5		5,140			296,460
					−25.26(5,140) =	−129,836
					I_{NA} =	166,624 in.^{4}

$$d_n = \frac{5{,}140}{203.5} = 25.26\,\text{in.}$$

$$d_{TOPOFSTEEL} = 35.50 - 25.26 = 10.24\,\text{in.}$$

$$d_{BOTOFSTEEL} = 35.88 + 25.26 = 61.14\,\text{in.}$$

$$S_{TOP\,OF\,STEEL} = \frac{166{,}624}{10.24} = 16{,}272\,\text{in.}^3$$

$$S_{BOT\,OF\,STEEL} = \frac{166{,}624}{61.14} = 2{,}725\,\text{in.}^3$$

10.1.1.3. Plastic Moment: Section 1-1

Determine the plastic-moment M_p of the composite section using the equations provided in Appendix D to Section 6 of the specification (Article D6.1). The longitudinal deck reinforcement is conservatively neglected.

$$P_t + P_w + P_c = A_{steel}F_y = 75.25(50) = 3{,}763\,\text{kips}$$

$$P_s = 0.85 f_c' \, b_{eff} \, t_s = 0.85(4.0)(100.0)(9.0) = 3{,}060\,\text{kips}$$

$$3{,}060\,\text{kips} < 3{,}763\,\text{kips} \quad \therefore \text{PNA is in the top flange, use Case II in Table D6.1-1}$$

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

$$\bar{y} = \frac{1.0}{2}\left[\frac{50(69.0)(0.5) + 50(1.375)(18.0) - 3{,}060}{50(1.0)(16.0)} + 1\right]$$

$= 0.44 \text{ in. from the top of the top flange}$

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

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

$$d_s = \frac{9.0}{2} + 3.5 + 0.44 - 1.0 = 7.44 \text{ in.}$$

$$d_w = 1.0 + \frac{69.0}{2} - 0.44 = 35.06 \text{ in.}$$

$$d_t = 1.0 + 69.0 + \frac{1.375}{2} - 0.44 = 70.25 \text{ in.}$$

$$M_p = \left[\frac{50(1.0)(16.0)}{2(1.0)}\right]\left[(0.44)^2 + (1.0 - 0.44)^2\right] + \left[(3{,}060)(7.44) + 69.0(0.5)(50)(35.06) + 1.375(18.0)(50)(70.25)\right]$$

$$M_p = 170{,}382 \text{ kip-in.} = 14{,}199 \text{ kip-ft}$$

10.1.1.4. Yield Moment: Section 1-1

Calculate the yield moment M_y of the composite section using the equations provided in Appendix D to Section 6 of the specification (Article D6.2.2). Essentially, 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.

$$F_y = \frac{M_{D1}}{S_{NC}} + \frac{M_{D2}}{S_{LT}} + \frac{M_{AD}}{S_{ST}} \qquad \text{Eq. (D6.2-1)}$$

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):

$$50 = 1.0\left[\frac{1.25(2{,}202)(12)}{1{,}973} + \frac{1.25(335)(12) + 1.50(322)(12)}{2{,}483} + \frac{M_{AD}}{2{,}706}\right] \qquad \text{Eq. (D6.2-2)}$$

$$M_{AD} = 78{,}206 \text{ kip-in.} = 6{,}517 \text{ kip-ft}$$

$$M_y = M_{D1} + M_{D2} + M_{AD}$$

$$M_y = 1.0[1.25(2{,}202) + 1.25(335) + 1.50(322) + 6{,}517]$$

$$M_y = 10{,}171 \text{ kip-ft}$$

10.1.2. Section 2-2

Section 2-2 is shown in Figure 13.

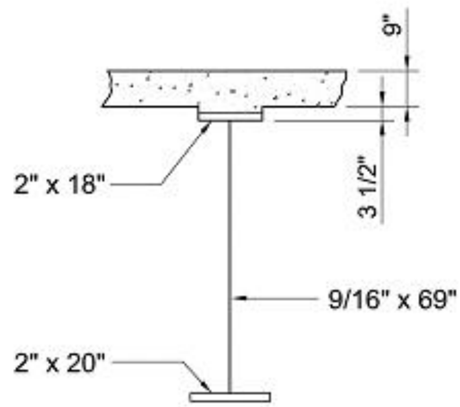

Figure 13: Section 2-2

10.1.2.1. Effective Flange Width (Article 4.6.2.6): Section 2-2

The effective flange width for Section 2-2 is equal to that of Section 1-1 calculated earlier:

b_{eff} = 114.0 in.

10.1.2.2. Minimum Negative Flexure Concrete Deck Reinforcement (Article 6.10.1.7)

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. This 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 in Table 3.4.1-1 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 the appropriate resistance factor for concrete in tension specified in Article 5.5.4.2.1. 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 in.

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:

$$A_{deck} = \frac{9.0}{12}(43.0) + 2\left[\frac{1}{12}\left(\frac{3.0}{2} + 0.5\right)\left(3.5 - \frac{18/2}{12}\right)\right] = 33.17 \text{ ft}^2 = 4{,}776 \text{ in.}^2$$

$$0.01(4{,}776) = 47.76 \text{ in.}^2$$

$$\frac{47.76}{43.0} = 1.11 \text{ in.}^2/\text{ft} = 0.0926 \text{ in.}^2/\text{in.}$$

$$0.0926(114.0) = 10.56 \text{ in.}^2$$

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 4.63 in. from the bottom of the concrete deck. Also in this example, the area of the longitudinal reinforcement is conservatively taken equal to the minimum required area of longitudinal reinforcement, although a larger area may be provided in the actual deck design.

Although not required by specification, for stress calculations involving the application of long-term loads to the composite section in regions of negative flexure in this example, the area of the longitudinal reinforcement is conservatively adjusted for the effects of concrete creep by dividing the area by 3 (i.e. 10.56/3 = 3.52 in.^{2}). The concrete is assumed to transfer the force from the longitudinal deck steel to the rest of the cross-section and concrete creep acts to reduce that force over time.

Finally, for members with shear connectors provided throughout their entire length that also satisfy the provisions of Article 6.10.1.7, Articles 6.6.1.2.1 and 6.10.4.2.1 permit the concrete deck to also be considered effective for negative flexure when computing stress ranges and flexural stresses acting on the composite section at the fatigue and service limit states, respectively. Therefore, section properties for the short-term and long-term composite section, including the concrete deck but neglecting the longitudinal reinforcement, are also determined for later use in the calculations for Section 2-2 at these limit states.

10.1.2.3. Elastic Section Properties: Section 2-2

Table 6 Section 2-2: Steel Only Section Properties

Component	A	d	Ad	Ad^2	I_o	I
Top Flange 2" x 18"	36.00	35.50	1,278	45,369	12.00	45,381
Web 9⁄16" x 69"	38.81				15,399	15,399
Bottom Flange 2" x 20"	40.00	−35.50	−1,420	50,410	13.33	50,423
	114.8		−142.0			111,203
					−1.24(142.0) =	−176.1
					I_{NA} =	111,027 in.⁴

$$d_s = \frac{-142.0}{114.8} = -1.24 \text{ in.}$$

$$d_{TOP\,OF\,STEEL} = 36.50 + 1.24 = 37.74 \text{ in.} \qquad d_{BOT\,OF\,STEEL} = 36.50 - 1.24 = 35.26 \text{ in.}$$

$$S_{TOP\,OF\,STEEL} = \frac{111{,}027}{37.74} = 2{,}942 \text{ in.}^3 \qquad S_{BOT\,OF\,STEEL} = \frac{111{,}027}{35.26} = 3{,}149 \text{ in.}^3$$

Table 7 Section 2-2: Steel Section + Long. Reinforcement/3

Component	A	d	Ad	Ad^2	I_o	I
Steel Section	114.8		−142.0			111,203
Long. Reinforcement/3	3.52	42.63	150.1	6,397		6,397
	118.3		8.10			117,600
					−0.07(8.10) =	−0.57
					I_{NA} =	117,599 in.⁴

$$d_{re\inf 3} = \frac{8.10}{118.3} = 0.07 \text{ in.}$$

$$d_{TOPOFSTEEL} = 36.50 - 0.07 = 36.43 \text{ in.} \qquad d_{BOTOFSTEEL} = 36.50 + 0.07 = 36.57 \text{ in.}$$

$$S_{TOP\,OF\,STEEL} = \frac{117{,}599}{36.43} = 3{,}228 \text{ in.}^3 \qquad S_{BOT\,OF\,STEEL} = \frac{117{,}599}{36.57} = 3{,}216 \text{ in.}^3$$

Table 8 Section 2-2: Steel Section + Long. Reinforcement

Component	A	d	Ad	Ad^2	I_o	I
Steel Section	114.8		−142.0			111,203
Long. Reinforcement	10.56	42.63	450.2	19,191		19,191
	125.4		308.2			130,394
					−2.46(308.2) =	−758.2
					I_{NA} =	129,636 in.⁴

$$d_{re\inf 3} = \frac{308.2}{125.4} = 2.46 \text{ in.}$$

$$d_{TOPOFSTEEL} = 36.50 - 2.46 = 34.04 \text{ in.} \qquad d_{BOTOFSTEEL} = 36.50 + 2.46 = 38.96 \text{ in.}$$

$$S_{TOP\,OF\,STEEL} = \frac{129{,}636}{34.04} = 3{,}808 \text{ in.}^3 \qquad S_{BOT\,OF\,STEEL} = \frac{129{,}636}{38.96} = 3{,}327 \text{ in.}^3$$

Table 9 Section 2-2: Composite Section Properties; 3n = 24

Component	A	d	Ad	Ad^2	I_o	I
Steel Section	114.8		−142.0			111,203
Concrete Slab 9" x 114"/ 24	42.75	42.50	1,817	77,217	288.6	77,506
	157.6		1,675			188,709
					−10.63 (1,675) =	−17,805
					I_{NA} =	170,904 in.^{4}

$$d_{3n} = \frac{1,675}{157.6} = 10.63 \text{ in.}$$

$$d_{TOPOFSTEEL} = 36.50 - 10.63 = 25.87 \text{ in.} \qquad d_{BOTOFSTEEL} = 36.50 + 10.63 = 47.13 \text{ in.}$$

$$S_{TOP\,OF\,STEEL} = \frac{170,904}{25.87} = 6,606 \text{ in.}^3 \qquad S_{BOT\,OF\,STEEL} = \frac{170,904}{47.13} = 3,626 \text{ in.}^3$$

Table 10 Section 2-2: Composite Section Properties; n = 8

Component	A	d	Ad	Ad^2	I_o	I
Steel Section	114.8		−142.0			111,203
Concrete Slab 9" x 114"/ 8	128.3	42.50	5,453	231,742	865.7	232,608
	243.1		5,311			343,811
					−21.85(5,311) =	−116,045
					I_{NA} =	227,766 in.^{4}

$$d_n = \frac{5,311}{243.1} = 21.85 \text{ in.}$$

$$d_{TOPOFSTEEL} = 36.50 - 21.85 = 14.65 \text{ in.} \qquad d_{BOTOFSTEEL} = 36.50 + 21.85 = 58.35 \text{ in.}$$

$$S_{TOP\,OF\,STEEL} = \frac{227,766}{14.65} = 15,547 \text{ in.}^3 \qquad S_{BOT\,OF\,STEEL} = \frac{227,766}{58.35} = 3,903 \text{ in.}^3$$

10.2. Exterior Girder Check: Section 1-1

10.2.1. Constructibility (Article 6.10.3)

Article 6.10.3.1 states that in addition to providing adequate strength, nominal yielding or reliance on post-buckling resistance is not to be permitted for main load-carrying members during critical stages of construction, except for yielding of the web in hybrid sections. This is accomplished by satisfying the requirements of Article 6.10.3.2 (Flexure) and 6.10.3.3 (Shear) under the applicable Strength load combinations specified in Table 3.4.1-1, with all loads factored as specified in Article 3.4.2. For the calculation of deflections during construction, all load factors are to be taken equal to 1.0.

As specified in Article 6.10.3.4, sections in positive flexure that are composite in the final condition, but noncomposite during construction, are to be investigated during the various stages of the deck placement. The effects of forces from deck overhang brackets acting on fascia girders are also to be considered. Wind-load effects on the noncomposite structure prior to casting of the deck are also an important consideration during construction, and are considered herein. Potential uplift at bearings should be investigated at each critical construction stage.

10.2.1.1. Deck Placement Analysis

During the deck placement, parts of the girders become composite in sequential stages. Temporary moments induced in the girders during the deck placement can be significantly higher than the final noncomposite dead load moments after the sequential placement is complete. A separate analysis was conducted using the BSDI, Ltd. Line Girder System (LGS) to determine the maximum moments in the exterior girders of the example bridge caused by the following assumed deck-placement sequence (Figure 14). Note the sequence assumes that the concrete is cast in the two end spans at approximately the same time. A check is not made for uplift should the cast in one end span be completed before the cast in the other end span has started.

Article 6.10.3.4 requires that changes in the stiffness during the various stages of the deck placement be considered. Therefore, in the analysis, all preceding deck casts are assumed composite for the casts that follow. Should the deck not be cast in separate stages, but instead be cast from one end of the bridge to the other, the end span must still be checked for the critical instantaneous unbalanced case where wet concrete exists over the entire end span, with no concrete cast yet on the remaining spans.

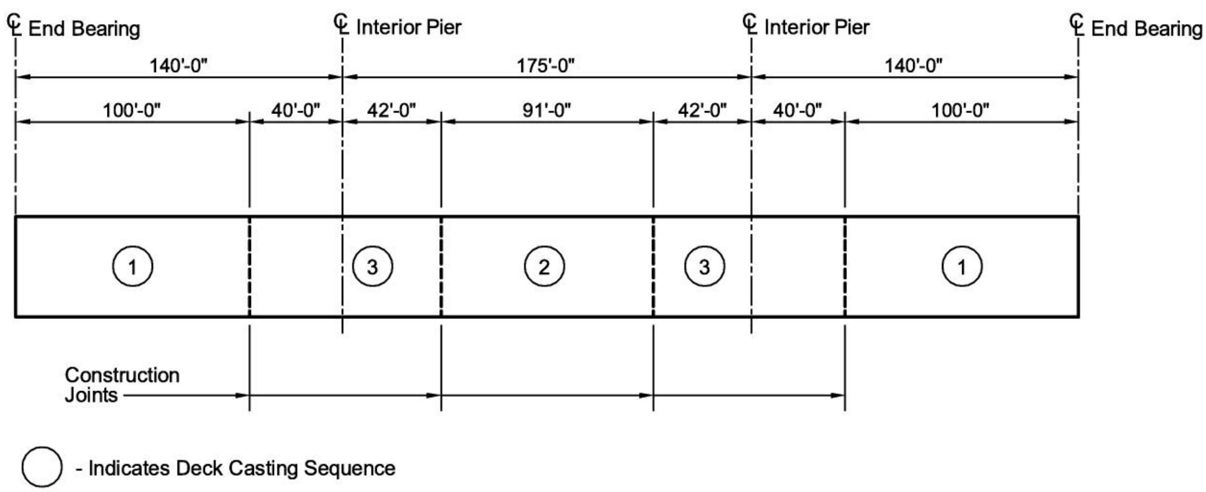

Figure 14: Deck-Placement Sequence

Unfactored dead-load moments in Span 1 from the abutment to the end of Cast 1, including the moments resulting from the preceding deck-placement sequence, are summarized in Table 11. In addition to the moments due to each of the individual casts, Table 11 gives the moments due to the steel weight, the moments due to the weight of the SIP forms, the sum of the moments due to the three casts plus the weight of the SIP forms, the maximum accumulated positive moments during the sequential deck casts (not including the steel weight), the sum of the moments due to the dead loads DC_2 and DW applied to the final composite structure, and the moments due to the weight of the concrete deck, haunches and SIP forms assuming that the concrete is placed all at once on the noncomposite girders. The assumed weight of the SIP forms includes the weight of the concrete in the form flutes. Although the forms are initially empty, the weight of the deck reinforcement is essentially equivalent to the weight of the concrete in the form flutes.

The slight differences in the moments on the last line of Table 11 and the sum of the moments due to the three casts plus the weight of the SIP forms are due to the changes in the girder stiffness with each cast. The principle of superposition does not apply directly in the deck-placement analyses since the girder stiffness changes at each step of the analysis. However, note the significant differences between the moments on the last line of Table 11 and the maximum accumulated positive moments during the sequential deck casts. In regions of positive flexure, the noncomposite girder should be checked for the effect of this larger maximum accumulated deck-placement moment. This moment at Section 1-1 is shown in bold in Table 11, along with the moment due to the steel weight. The sum of these moments is computed as:

$$M = 352 + 2{,}537 = 2{,}889 \text{ kip-ft}$$

Table 11 Moments from Deck-Placement Analysis

	Span -> 1	Unfactored Dead-Load Moments (kip-ft)								
Length (ft)	0.0	12.0	24.0	42.0	48.0	56.0	72.0	84.0	96.0	100.0
Steel Weight	0	143	250	341	353	**352**	296	206	74	21
SIP Forms (SIP)	0	63	110	147	151	150	124	84	27	4
Cast 1	0	870	1544	2189	2306	2387	2286	1983	1484	1275
2	0	-168	-336	-589	-673	-786	-1010	-1179	-1347	-1403
3	0	14	28	50	57	67	86	101	115	120
Sum of Casts + SIP	0	779	1346	1797	1841	1818	1486	989	279	-4
Max. +M	0	933	1654	2336	2457	**2537**	2410	2067	1511	1279
DC_2 + DW	0	275	477	643	661	657	551	386	148	52
Deck, haunches + SIP	0	786	1360	1822	1870	1850	1528	1038	335	53

The unfactored vertical dead-load deflections in Span 1 from the abutment to the end of Cast 1, including the deflections resulting from the preceding deck-placement sequence, are summarized in Table 12. Negative values are downward deflections and positive values are upward deflections.

Table 12 Vertical Deflections from Deck-Placement Analysis

Span ->1		Unfactored Vertical Dead-Load Deflections (in.)								
Length (ft)	0.0	12.0	24.0	42.0	48.0	56.0	72.0	84.0	96.0	100.0
Steel Weight	0	-.17	-.32	-.47	-.50	-.51	-.47	-.39	-.29	-.25
SIP Forms (SIP)	0	-.07	-.14	-.20	-.21	-.21	-.20	-.16	-.12	-.10
Cast 1	0	-1.32	-2.50	-3.78	-4.04	-4.27	-4.30	-3.95	-3.33	-3.08
2	0	.27	.52	.86	.96	1.08	1.25	1.32	1.32	1.31
3	0	-.01	-.03	-.04	-.04	-.05	-.05	-.05	-.04	-.03
Sum of Casts + SIP	0	-1.14	-2.14	-3.16	-3.34	-3.46	-3.30	-2.84	-2.17	-1.91
DC_2 + DW	0	-.17	-.32	-.46	-.48	-.49	-.45	-.38	-.28	-.24
Total	0	-1.48	-2.78	-4.09	-4.32	-4.46	-4.22	-3.61	-2.74	-2.40
Deck, haunches + SIP	0	-.92	-1.71	-2.47	-2.59	-2.64	-2.43	-2.02	-1.47	-1.27

Since the deck casts are relatively short-term loadings, the actual moments and deflections that occur during construction are more likely to correspond to those computed using a modular ratio of *n* for determining the stiffness of the sections that are assumed composite. Therefore, the *n*-composite stiffness is used for all preceding casts in computing the moments and deflections shown for Casts 2 and 3 in Table 11 and Table 12. The moments and deflections on the final composite structure due to the sum of the DC_2 and DW loads shown in Table 11 and Table 12 are computed using the 3*n*-composite stiffness to account for the long-term effects of concrete creep. The entire cross-sectional area of the deck associated with the exterior girder was assumed effective in the analysis in determining the stiffness of the composite sections.

Note the differences in the calculated deflections on the last line of Table 12 (assuming the deck is cast all at once on the noncomposite structure) and the sum of the accumulated deflections during the sequential deck casts. In many cases, the deflections shown on the last line can be used to estimate the girder cambers, as required in Article 6.10.3.5 to account for the dead-load deflections. When the differences in these deflections are not significant, the deflections due to the accumulated deck casts will eventually converge toward the deflections shown on the last line as concrete creep occurs. However, if the differences in the deflections are deemed significant, the Engineer may need to evaluate which set of deflections should be used, or else estimate deflections somewhere in-between to compute the girder cambers and avoid potential errors in the final girder elevations.

It is interesting to note that a refined 3D analysis of the example bridge yielded a maximum deflection in Span 1 (at Section 1-1) due to the weight of the concrete deck, haunches and SIP forms (assuming that the concrete is placed all at once on the noncomposite girders) of 2.61 inches in the exterior girders and 2.65 inches in the interior girders. From Table 11, the comparable maximum deflection from the line-girder analysis is 2.64 inches, which indicates the assumption of equal distribution of the DC_1 loads to all the girders is the proper assumption in this case.

The unfactored vertical dead-load reactions resulting from the deck-placement analysis are given in Table 13. Negative reactions represent upward reactions that resist the maximum downward force at the support under consideration. Conversely, positive reactions represent downward reactions that resist the maximum uplift force at the support.

Table 13 Unfactored Vertical Dead-Load Reactions from Deck-Placement Analysis (kips)

	Abut. 1	Pier 1	Pier 2	Abut. 2
Steel Weight	-13.	-53.	-53.	-13.
sum	-13.	-53.	-53.	-13.
SIP Forms (SIP)	-6.	-21.	-21.	-6.
sum	-19.	74.	-74.	-19.
Cast 1	-80.	-55.	-55.	-80.
sum	-99.	-129.	-129.	-99.
Cast 2	14.	-75.	-75.	14.
sum	-85.	-204.	-204.	-85.
Cast 3	-1.	-110.	-110.	-1.
sum	-86.	-314.	-314.	-86.
Sum of Casts + SIP	-73.	-261.	-261.	-73.
DC_2 + DW	-26.	-90.	-90.	-26.
Total	-112.	-404.	-404.	-112.
Deck, haunches + SIP	-74.	-261.	-261.	-74.

Shown in Table 13 (under 'sum') are the accumulated reactions for the steel weight plus the individual deck casts, which should be used to check for uplift under the deck placement. A net positive reaction indicates that the girder may lift-off at the support. Lift-off does not occur in this particular example; lift-off is most common when end spans of continuous units are skewed or relatively short. If the girder is permitted to lift-off its bearing seat, the staging analysis is incorrect unless a hold-down of the girder is provided at the location of a positive reaction.

Options to consider when uplift occurs include: 1) rearranging the concrete casts, 2) specifying a temporary load over that support, 3) specifying a tie-down bearing, or 4) performing another staging analysis with zero bearing stiffness at the support experiencing lift-off. Note that the sum of the reactions from the analysis of the staged deck casts may differ somewhat from the reactions assuming the deck is cast all at once on the noncomposite structure (as given on the last line of Table 13); however, in most cases, the reactions should not differ greatly.

Calculate the maximum flexural stresses in the flanges of the steel section due to the factored loads resulting from the deck-placement sequence. 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, cross-frames adjacent to Section 1-1 are located 48 ft and 72 ft from the left abutment. From inspection of Table 11, since the girder is prismatic between the two cross-frames, the largest stress within the unbraced length occurs right at Section 1-1. As discussed previously, the η factor is taken equal to 1.0 in this example. Therefore, For STRENGTH I:

$$\text{Top flange:} \quad f_{bu} = \frac{1.0(1.25)(2{,}889)(12)}{1{,}581} = -27.41 \text{ ksi}$$

$$\text{Bot. flange:} \quad f_{bu} = \frac{1.0(1.25)(2{,}889)(12)}{1{,}973} = 21.96 \text{ ksi}$$

For STRENGTH IV:

$$\text{Top flange:} \quad f_{bu} = \frac{1.0(1.5)(2{,}889)(12)}{1{,}581} = -32.89 \text{ ksi}$$

$$\text{Bot. flange:} \quad f_{bu} = \frac{1.0(1.5)(2{,}889)(12)}{1{,}973} = 26.36 \text{ ksi}$$

10.2.1.2. Deck Overhang Loads

Assume the deck overhang bracket configuration shown in Figure 15 with the brackets extending to the bottom flange, which is preferred. Alternatively, the brackets may bear on the girder web if means are provided to ensure that the web is not damaged and that the associated deformations permit proper placement of the concrete deck.

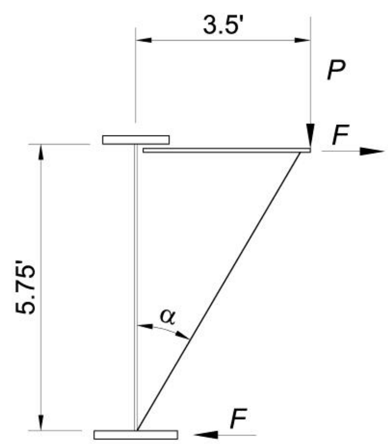

Figure 15: Deck Overhand Bracket

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 applied uniformly. Calculate the vertical loads acting on the overhang brackets. Because in this case the bracket is assumed to extend near the edge of the deck overhang, assume that half the deck overhang weight is placed on the exterior girder and half the weight is placed on the overhang brackets. Conservatively include one-half the deck haunch weight in the total overhang weight. Therefore:

Deck Overhang Weight:

$$P = 0.5 * 150\left[\frac{9.5}{12}(3.5) + \left[\frac{1}{12}\left(\frac{3.0}{2} + 0.5\right)\left(3.5 - \frac{16/2}{12}\right)\right] + \frac{2.75}{12}\left(\frac{16/2}{12}\right)\right] = 255 \text{ lbs / ft}$$

The other half of the overhang weight can be assumed to act at the edge of the top flange (at a distance of 8.0 inches from the shear center of the girder in this case). The effective deck weight acting on the other side of the girder can be assumed applied at the other edge of the top flange. The net torque can be resolved into flange lateral moments that generally act in the opposite direction to the lateral moments caused by the overhang loads. This effect is conservatively neglected in this example.

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 = 3,000 lbs

The finishing machine load is estimated as one-half of the total finishing machine truss weight, plus some additional load to account for the weight of the engine, drum and operator assumed to be located on one side of the truss. Note that the above loads are estimated loads used here for illustration purposes only. It is recommended that the Engineer consider talking to local Contractors to obtain more accurate values for these construction loads.

The lateral force on the top flange due to the vertical load on the overhang brackets is computed as:

$$F = P \tan \alpha$$

where: $\alpha = \tan^{-1}\left(\frac{3.5 \text{ ft}}{5.75 \text{ ft}}\right) = 31.3^\circ$

Note that the calculated lateral force and the design calculations that follow are dependent on the assumed angle of the deck overhang brackets. Thus, a sketch similar to Figure 15 with the assumed angle should be shown on the contract plans. Should the Contractor deviate significantly from this assumed angle, an additional investigation by the Contractor may be necessary.

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 flanges due to the lateral bracket forces. Assuming the flanges are 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:

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

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, containing Section 1-1 is equal to 24.0 feet (Figure 2).

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

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

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

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

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

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

For the steel section, the depth of the web in compression in the elastic range, D_c, at Section 1-1 is 38.63 inches. Therefore,

$$r_t = \frac{16}{\sqrt{12\left(1+\frac{1}{3}\frac{(38.63)(0.5)}{(16)(1)}\right)}} = 3.90\text{ in.}$$

$$L_p = \frac{1.0(3.90)}{12}\sqrt{\frac{29{,}000}{50}} = 7.83\text{ ft}$$

C_b is the moment gradient modifier specified in Article 6.10.8.2.3. Separate calculations show that $f_{mid}/f_2 > 1$ in the unbraced length under consideration. Therefore, C_b must 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 since web bend buckling is prevented during construction by a separate limit state check. 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} = -32.89 ksi, as computed earlier for the STRENGTH IV load combination (which controls in this particular computation). Therefore:

$$1.2(7.83)\sqrt{\frac{1.0(1.0)}{|-32.89|\ /50}} = 11.59\text{ ft} < L_b = 24.0\text{ ft}$$

Because the preceding equation 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:

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

$$F_{cr} = \frac{1.0(1.0)\pi^2(29{,}000)}{\left(\frac{24(12)}{3.90}\right)^2} = 52.49 \text{ ksi}$$

Note that the calculated value of F_{cr} for use in Eq. 6.10.1.6-4 is not limited to $R_bR_hF_{yc}$.

The amplification factor is then determined as follows:

For STRENGTH I:

$$AF = \frac{0.85}{\left(1 - \frac{|-27.41|}{52.49}\right)} = 1.78 > 1.0 \quad \text{ok}$$

For STRENGTH IV:

$$AF = \frac{0.85}{\left(1 - \frac{|-32.89|}{52.49}\right)} = 2.28 > 1.0 \quad \text{ok}$$

AF is taken equal to 1.0 for tension flanges. The above equation for the amplification factor conservatively assumes an elastic effective length factor for lateral torsional buckling equal to 1.0. Article C6.10.8.2.3 provides references to a relatively simple method that can be used in certain situations to potentially calculate a lower elastic effective length factor for the unbraced length under consideration. Appendix A (to this design example) illustrates the application of this method to this particular unbraced length. Should the unbraced length under consideration end up being the critical unbraced length for which K is less than 1.0, the lower value of K can then subsequently be used to appropriately modify F_{cr} in the amplification factor formula and also L_b when determining the lateral torsional buckling resistance.

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 the STRENGTH I load combination; a load factor of 1.5 is applied to all construction loads (Article 3.4.2).

For STRENGTH I:

Dead loads: $P = 1.0[1.25(255) + 1.5(40 + 85 + 25 + 125)] = 731.3 \text{ lbs/ft}$

$$F = F_\ell = P\tan\alpha = 731.3\tan(31.3^\circ) = 444.6 \text{ lbs/ft}$$

$$M_\ell = \frac{F_\ell L_b^2}{12} = \frac{0.4446(24)^2}{12} = 21.34 \text{ kip} - \text{ft}$$

Top flange: $f_\ell = \frac{M_\ell}{S_\ell} = \frac{21.34(12)}{1(16)^2/6} = 6.00 \text{ ksi}$

Bot. flange: $f_\ell = \frac{M_\ell}{S_\ell} = \frac{21.34(12)}{1.375(18)^2/6} = 3.45 \text{ ksi}$

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

$$F = P_\ell = P\tan\alpha = 4{,}500\tan(31.3^\circ) = 2{,}736 \text{ lbs}$$

$$M_\ell = \frac{P_\ell L_b}{8} = \frac{2.736(24)}{8} = 8.21 \text{ kip} - \text{ft}$$

Top flange: $f_\ell = \frac{M_\ell}{S_\ell} = \frac{8.21(12)}{1(16)^2/6} = 2.31 \text{ ksi}$

Bot. flange: $f_\ell = \frac{M_\ell}{S_\ell} = \frac{8.21(12)}{1.375(18)^2/6} = 1.33 \text{ ksi}$

Top flange: f_ℓ total $= 6.00 + 2.31 = 8.31$ ksi * AF = (8.31)(1.78) = 14.79 ksi < $0.6F_{yf}$ = 30 ksi ok

Bot. flange: f_ℓ total $= 3.45 + 1.33 = 4.78$ ksi * AF = (4.78)(1.0) = 4.78 ksi < $0.6F_{yf}$ = 30 ksi ok

For STRENGTH IV:

Dead loads: $P = 1.0[1.5(255 + 40 + 85 + 25 + 125)] = 795 \text{ lbs/ft}$

$$F = F_\ell = P\tan\alpha = 795\tan(31.3^\circ) = 483.4 \text{ lbs/ft}$$

$$M_\ell = \frac{F_\ell L_b^2}{12} = \frac{0.4834(24)^2}{12} = 23.20 \text{ kip} - \text{ft}$$

Top flange: $f_\ell = \frac{M_\ell}{S_\ell} = \frac{23.20(12)}{1(16)^2/6} = 6.52 \text{ ksi}$

Bot. flange: $f_\ell = \frac{M_\ell}{S_\ell} = \frac{23.20(12)}{1.375(18)^2/6} = 3.75 \text{ ksi}$

Finishing machine: Not considered

Top flange: f_ℓ total $= 6.52$ ksi * AF = 6.52(2.28) = 14.87 ksi < $0.6F_{yf}$ = 30 ksi ok

Bot. flange: f_ℓ total $= 3.75$ ksi * AF = 3.75(1.0) = 3.75 ksi < $0.6F_{yf}$ = 30 ksi ok

To account for potential uncertainties in the estimation of the preceding deck overhang loads, it may be desirable to ensure that the factored lateral forces determined above exceed, as a minimum, a percentage of the total factored weight of the structure. A reasonable percentage would be between three to five percent.

10.2.1.3. Wind Loads

Wind load acting on the noncomposite structure prior to casting of the concrete deck will be investigated. Conservatively using the smallest steel section, the total wind load per unit length, w, for the case of wind applied normal to the structure assuming no superelevation is computed as:

$$w = P_D h_{exp.} = 0.053[(0.875 + 69.0 + 1.0)/12] = 0.313 \text{ kips/ft} > 0.3 \text{ kips/ft} \quad \text{ok}$$

Note that the full design horizontal wind pressure, calculated earlier to be P_D = 0.053 ksf, is conservatively used here in this illustration. For the actual temporary construction condition however, consideration might be given to using a smaller design wind pressure depending on the specific situation and anticipated maximum wind velocity at the bridge site.

Determine the maximum flexural stress, f_{bu}, in the top and bottom flanges due to the factored steel weight within the unbraced length containing Section 1-1. The largest moment due to the steel weight within the unbraced length is equal to 352 kip-feet right at Section 1-1 (Table 11).

Therefore, since the member is prismatic in-between these two cross-frames, the largest stress in both flanges also occur at Section 1-1. The STRENGTH III load case applies to the case of dead plus wind load with no live load on the structure. η is taken equal to 1.0 at the strength limit state in this example. Therefore,

For STRENGTH III:

$$\text{Top flange:} \quad f_{bu} = \frac{1.0(1.25)(352)(12)}{1{,}581} = -3.34 \text{ ksi}$$

$$\text{Bot. flange:} \quad f_{bu} = \frac{1.0(1.25)(352)(12)}{1{,}973} = 2.68 \text{ ksi}$$

Since there is no deck to provide horizontal diaphragm action, assume the cross-frames act as struts in distributing the total wind force on the structure to the flanges on all girders in the cross-section. The force is then assumed transmitted through lateral bending of the flanges to the ends of the span or to the closest point(s) of lateral wind bracing. Determine the total factored wind force on the structure assuming the wind is applied to the deepest steel section and normal to the structure (with no superelevation). For the STRENGTH III load combination, the load factor for wind during construction is not to be taken less than 1.25.

$$W = \frac{1.0(1.25)(0.053)(2.0 + 69.0 + 2.0)}{12} = 0.403 \text{ kips/ft}$$

To illustrate the effect that a couple of panels of top lateral bracing can have in providing a stiffer load path for wind loads acting on the noncomposite structure during construction, assume the system of top lateral bracing shown in Figure 2; that is, top lateral bracing in the interior bays on each side of each interior-pier section. Assume that Span 1 of the structure (acting as a system) resists the lateral wind force as a propped cantilever, with an effective span length, L_e, of 120.0 feet. That is, the top lateral bracing is assumed to provide an effective line of fixity at the cross-frame 20.0 feet from the pier for resisting the lateral force. Calculate the moment on the propped cantilever at Section 1-1:

$$M_{1-1} = \frac{9}{128} W L_e^2 = \frac{9}{128}(0.403)(120.0)^2 = 408 \text{ kip - ft}$$

Calculate the moment on the propped cantilever at the assumed line of fixity (call it Section f-f -- 20.0 feet from the pier):

$$M_{f-f} = \frac{1}{8} W L_e^2 = \frac{1}{8}(0.403)(120.0)^2 = 725.4 \text{ kip - ft}$$

Note that a refined 3D analysis of the example noncomposite structure subjected to the factored wind load yielded a total lateral moment in the top and bottom flanges of all four girders of 405 kip-ft at Section 1-1 and 659 kip-ft at Section f-f.

Proportion the total lateral moment to the top and bottom flanges at Section 1-1 according to the relative lateral stiffness of each flange. Assume that the total flange lateral moment is then divided equally to each girder. The single bay of top bracing along with the line of cross frames adjacent to that bay (acting as an effective line of fixity) permits all the girders to work together as a system to resist the lateral wind force along the entire span.

Section 1-1: Top flange:

$$I_\ell = \frac{1(16)^3}{12} = 341.3 \text{ in.}^4$$

Bottom flange:

$$I_\ell = \frac{1.375(18)^3}{12} = 668.3 \text{ in.}^4$$

Top flange:

$$M_\ell = \frac{408.0(341.3)}{(341.3 + 668.3)4} = 34.48 \text{ kip} - \text{ft}$$

Bottom flange: $$M_\ell = \frac{408.0(668.3)}{(341.3+668.3)4} = 67.52\,\text{kip}-\text{ft}$$

A similar computation can be made at Section f-f (however, this section is not checked for this condition in this example).

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:

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

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} = -3.34 ksi. Earlier, it was determined that the moment gradient modifier, C_b, and the web load-shedding factor, R_b, are equal to 1.0. The limiting unbraced length, L_p, was also determined earlier to be 7.83 feet. Therefore,

$$1.2(7.83)\sqrt{\frac{1.0(1.0)}{|-3.34|/50}} = 36.35\,\text{ft} > L_b = 24.0\,\text{ft}$$

Therefore, lateral bending stresses determined from a first-order analysis may be used. 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.

Section 1-1: Top flange: $$f_\ell = \frac{34.48(12)}{1(16)^2/6} = 9.70\,\text{ksi} < 0.6F_{yf} = 30.0\,\text{ksi} \quad \text{ok}$$

Bottom flange: $$f_\ell = \frac{67.52(12)}{1.375(18)^2/6} = 10.91\,\text{ksi} < 0.6F_{yf} = 30.0\,\text{ksi} \quad \text{ok}$$

Calculate the shear in the propped cantilever at Section f-f:

$$V_{f-f} = \frac{5}{8}WL_e = \frac{5}{8}(0.403)(120.0) = 30.23\,\text{kips}$$

Resolve the shear into a compressive force in the diagonal of the top bracing:

$$P = 30.23\left(\frac{\sqrt{(20.0)^2+(12.0)^2}}{12.0}\right) = -58.76\,\text{kips}$$

In addition, the member carries a force due to the steel weight. Calculate the average stress in the top flange adjacent to the braced bay using the average moment due to the factored steel weight along the 20-foot unbraced length adjacent to the pier section (from Table 4) assumed applied to the larger section within this unbraced length (i.e. Section 2-2):

$$f_{tf_{avg.}} = \frac{1.0(1.25)(12)[-312+(-777)]/2}{2{,}942} = 2.78\,\text{ksi}$$

Resolve this stress into the diagonal:

$$f_{diag.} = 2.78\left(\frac{20.0}{\sqrt{(20.0)^2+(12.0)^2}}\right) = -2.38\ \text{kips}$$

Assuming an area of 8.0 in.^{2} for the diagonal yields a compressive force of –19.04 kips resulting in a total estimated compressive force of (-58.76) +(-19.04) = -77.80 kips. The diagonal must be designed to carry this force. Note that the refined 3D analysis, mentioned previously, yielded a total compressive force in the diagonal bracing member of approximately -67.0 kips.

Estimate the maximum lateral deflection of Span 1 of the structure (i.e. the propped cantilever) due to the factored wind load using the total of the lateral moments of inertia of the top and bottom flanges of all four girders at Section 1-1. For simplicity, this section is assumed to be an average section for the span (a weighted average section would likely yield greater accuracy):

$$\Delta_{\ell\,max.} = \frac{WL_e^4}{185EI} = \frac{0.403(120.0)^4(1{,}728)}{185(29{,}000)(341.3+668.3)4} = 6.7\ \text{in.}$$

Note that the refined 3D analysis yielded a maximum lateral deflection of approximately 7.0 inches in Span 1. If the top bracing were not present, L_e would increase to 140.0 feet and the estimated maximum lateral deflection calculated from the above equation would increase to 12.3 inches. Large lateral deflections may potentially result in damage to the bearings. Therefore such an approach may be helpful to determine how many panels of top lateral bracing, if any, might be necessary to reduce the lateral deflection to a level deemed acceptable for the particular situation under consideration.

To analyze the center span for this condition, a similar approach can be taken using the actions of an assumed fixed-fixed beam rather than a propped cantilever.

10.2.1.4. Flexure (Article 6.10.3.2)

For critical stages of construction, Article 6.10.3.2.1 requires that discretely braced flanges in compression satisfy the following requirements, except that: 1) for slender-web sections, Eq. 6.10.3.2.1-1 need not be checked when f_ℓ is equal to zero, and 2) for sections with compact or noncompact webs, Eq. 6.10.3.2.1-3 need not be checked.

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

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

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

Article 6.10.3.2.2 requires that discretely braced flanges in tension satisfy:

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

where: ϕ_f = resistance factor for flexure = 1.0 (Article 6.5.4.2)
F_{crw} = nominal bend-buckling resistance for webs determined as specified in Article 6.10.1.9
R_h = hybrid factor specified in Article 6.10.1.10.1 (= 1.0 at homogeneous Section 1-1)
F_{nc} = nominal flexural resistance of the compression flange determined as specified in Article 6.10.8.2 (i.e. local or lateral torsional buckling resistance, whichever controls). For sections with compact or noncompact webs, the provisions of Article A6.3.3 may optionally be used to determine the lateral torsional buckling resistance.

First, determine if the noncomposite Section 1-1 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):

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

$$\frac{2D_c}{t_w} = \frac{2(38.63)}{0.5} = 154.5$$

$$5.7\sqrt{\frac{E}{F_{yc}}} = 5.7\sqrt{\frac{29{,}000}{50}} = 137.3 < 154.5$$

Therefore, the noncomposite Section 1-1 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, Eq. 6.10.3.2.1-3 must also be checked, and the optional provisions of Appendix A (to Section 6 of *AASHTO LRFD (5th Edition, 2010)* – Article A6.3.3) cannot be used to determine the lateral torsional buckling resistance of the flange.

10.2.1.4.1. Top Flange

10.2.1.4.1.1. Local Buckling Resistance (Article 6.10.8.2.2)

Determine the slenderness ratio of the top flange:

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

$$\lambda_f = \frac{16}{2(1)} = 8.0$$

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

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

$$\lambda_{pf} = 0.38\sqrt{\frac{29{,}000}{50}} = 9.2$$

Since $\lambda_f < \lambda_{pf}$,

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

As specified in Article 6.10.3.2.1, in 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. Therefore,

$$F_{nc} = 1.0(1.0)(50) = 50.0 \text{ ksi}$$

10.2.1.4.1.2. Lateral Torsional Buckling Resistance (Article 6.10.8.2.3)

The limiting unbraced length, L_p, was computed earlier to be 7.83 feet. The effective radius of gyration for lateral torsional buckling, r_t, for the noncomposite Section 1-1 was also computed earlier to be 3.90 inches.

Determine the limiting unbraced length, L_r:

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

where: $F_{yr} = 0.7F_{yc} \le F_{yw}$

$$F_{yr} = 0.7(50) = 35.0 \text{ ksi} < 50 \text{ ksi} \quad \text{ok}$$

F_{yr} must also not be less than $0.5F_{yc} = 0.5(50) = 25.0$ ksi ok.

Therefore: $L_r = \dfrac{\pi(3.90)}{12}\sqrt{\dfrac{29{,}000}{35.0}} = 29.39 \text{ ft}$

Since $L_p = 7.83$ feet < $L_b = 24.0$ feet < $L_r = 29.39$ feet,

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

As discussed previously, since $f_{mid}/f_2 > 1$ in the unbraced length under consideration, the moment-gradient modifier, C_b, must be taken equal to 1.0. Therefore,

$$F_{nc} = 1.0\left[1-\left(1-\frac{35.0}{1.0(50)}\right)\left(\frac{24.0-7.83}{29.39-7.83}\right)\right](1.0)(1.0)(50) = 38.75 \text{ ksi} < 1.0(1.0)(50) = 50 \text{ ksi} \quad \text{ok}$$

F_{nc} is governed by the lateral torsional buckling resistance, which is less than the local buckling resistance of 50.0 ksi computed earlier. Therefore, $F_{nc} = 38.75$ ksi.

10.2.1.4.1.3. Web Bend-Buckling Resistance (Article 6.10.1.9)

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

$$F_{crw} = \frac{0.9Ek}{\left(\dfrac{D}{t_w}\right)^2} \qquad \text{Eq. (6.10.1.9.1-1)}$$

but not to exceed the smaller of R_hF_{yc} and $F_{yw}/0.7$,

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

$$k = \frac{9}{(38.63/69.0)^2} = 28.7$$

Therefore,

$$F_{crw} = \frac{0.9(29{,}000)(28.7)}{\left(\frac{69.0}{0.5}\right)^2} = 39.33\text{ ksi} < \min\left(R_h F_{yc}, F_{yw}/0.7\right) = R_h F_{yc} = 1.0(50) = 50\text{ ksi} \quad \text{ok}$$

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

For STRENGTH I:

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

$$f_{bu} + f_\ell = \left|-27.41\right|\text{ ksi} + 14.79\text{ ksi} = 42.20\text{ ksi}$$

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

$$42.20\text{ ksi} < 50.0\text{ ksi} \quad \text{ok} \qquad (\text{Ratio} = 0.844)$$

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

$$f_{bu} + \frac{1}{3} f_\ell = \left|-27.41\right|\text{ ksi} + \frac{14.79}{3}\text{ ksi} = 32.34\text{ ksi}$$

$$\phi_f F_{nc} = 1.0(38.75) = 38.75\text{ ksi}$$

$$32.34\text{ ksi} < 38.75\text{ ksi} \quad \text{ok} \qquad (\text{Ratio} = 0.835)$$

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

$$\phi_f F_{crw} = 1.0(39.33) = 39.33\text{ ksi}$$

$$\left|-27.41\right|\text{ ksi} < 39.33\text{ ksi} \quad \text{ok} \qquad (\text{Ratio} = 0.697)$$

For STRENGTH III:

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

$$f_{bu} + f_\ell = \left|-3.34\right|\text{ksi} + 9.70\text{ksi} = 13.04\text{ ksi}$$

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

$$13.04\,\text{ksi} < 50.0\,\text{ksi} \quad \text{ok} \qquad (\text{Ratio} = 0.261)$$

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

$$f_{bu} + \frac{1}{3} f_{\ell} = \left|-3.34\right| \text{ksi} + \frac{9.70}{3} \text{ksi} = 6.57\,\text{ksi}$$

$$\phi_f F_{nc} = 1.0(38.75) = 38.75\,\text{ksi}$$

$$6.57\text{ksi} < 38.75\text{ksi} \ \text{ok}(\text{Ratio} = 0.170)$$

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

$$\phi_f F_{crw} = 1.0(39.33) = 39.33\,\text{ksi}$$

$$\left|-3.34\right| \text{ksi} < 39.33\,\text{ksi} \quad \text{ok} \qquad (\text{Ratio} = 0.085)$$

For STRENGTH IV:

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

$$f_{bu} + f_{\ell} = \left|-32.89\right| \ \text{ksi} + 14.87\,\text{ksi} = 47.76\,\text{ksi}$$

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

$$47.76\,\text{ksi} < 50.0\,\text{ksi} \quad \text{ok} \qquad (\text{Ratio} = 0.955)$$

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

$$f_{bu} + \frac{1}{3} f_{\ell} = \left|-32.89\right| \ \text{ksi} + \frac{14.87}{3} \text{ksi} = 37.85\,\text{ksi}$$

$$\phi_f F_{nc} = 1.0(38.75) = 38.75\,\text{ksi}$$

$$37.85\,\text{ksi} < 38.75\,\text{ksi} \quad \text{ok} \qquad (\text{Ratio} = 0.977)$$

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

$$\phi_f F_{crw} = 1.0(39.33) = 39.33 \text{ ksi}$$

$$|-32.89| \text{ ksi} < 39.33 \text{ ksi} \quad \text{ok} \qquad (\text{Ratio} = 0.836)$$

Options to consider should the web bend-buckling resistance be exceeded include: 1) providing a larger compression flange or a smaller tension flange to decrease D_c, 2) adjusting the deck-placement sequence to reduce the compressive stress in the web, 3) providing a thicker web, and 4) adding a longitudinal web stiffener should the preceding options not prove to be practical or cost-effective.

10.2.1.4.2. Bottom Flange

For STRENGTH I:

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

$$f_{bu} + f_\ell = 21.96 \text{ ksi} + 4.78 \text{ ksi} = 26.74 \text{ ksi}$$

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

$$26.74 \text{ ksi} < 50.0 \text{ ksi} \quad \text{ok} \qquad (\text{Ratio} = 0.535)$$

STRENGTH III:

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

$$f_{bu} + f_\ell = 2.68 \text{ ksi} + 10.91 \text{ ksi} = 13.59 \text{ ksi}$$

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

$$13.59 \text{ ksi} < 50.0 \text{ ksi} \quad \text{ok} \qquad (\text{Ratio} = 0.272)$$

For STRENGTH IV:

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

$$f_{bu} + f_\ell = 26.36 \text{ ksi} + 3.75 \text{ ksi} = 30.11 \text{ ksi}$$

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

$$30.11 \text{ ksi} < 50.0 \text{ ksi} \quad \text{ok} \qquad (\text{Ratio} = 0.602)$$

Although the checks are illustrated here for completeness, the bottom flange will typically not control in this region.

10.2.1.5. Shear (Article 6.10.3.3)

For critical stages of construction, 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

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 critical panel in Field Section 1 will be checked. The critical panel for this check is the panel immediately to the left of the fourth intermediate cross-frame from the abutment, which is located 96.0 feet from the abutment. The transverse stiffener in this panel is assumed to be located at the maximum permitted spacing of d_o = 3D = 3(69.0) = 207.0 inches to the left of this cross-frame (see later shear calculations). Since shear is rarely increased significantly due to deck staging, the factored DC_1 shear at the cross-frame will be used in this check (the STRENGTH IV load combination governs by inspection):

$$(V_u)_{DC_1} = 1.0(1.5)(-79) = -119 \text{ kips} \text{ at } 96'\text{-}0'' \text{ from the abutment}$$

The shear buckling resistance of the 207-inch-long 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 from Eq. 6.10.9.3.2-4, 6.10.9.3.2-5 or 6.10.9.3.2-6, as applicable. First, compute the shear buckling coefficient, k

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

$$k = 5 + \frac{5}{\left(\frac{207.0}{69.0}\right)^2} = 5.56$$

Since, $1.40\sqrt{\frac{Ek}{F_{yw}}} = 1.40\sqrt{\frac{29{,}000(5.56)}{50}} = 79.5 < \frac{D}{t_w} = \frac{69.0}{0.5} = 138.0$

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

$$C = \frac{1.57}{(138.0)^2}\left(\frac{29{,}000(5.56)}{50}\right) = 0.266$$

V_p is the plastic shear force determined as follows:

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

$$V_p = 0.58(50)(69.0)(0.5) = 1{,}001 \text{ kips}$$

Therefore, $V_{cr} = 0.266(1{,}001) = 266 \text{ kips}$

$$\phi_v V_{cr} = 1.0(266) = 266 \text{ kips}$$

$$|-119| \text{ kips} < 266 \text{ kips} \qquad \text{ok} \qquad (\text{Ratio} = 0.447)$$

10.2.1.6. Concrete Deck (Article 6.10.3.2.4)

Article 6.10.2.3.4 requires that the longitudinal tensile stress in a composite concrete deck due to the factored loads not exceed ϕf_r during critical stages of construction, unless longitudinal reinforcement is provided according to the provisions of Article 6.10.1.7.

f_r is the modulus of rupture of the concrete determined as follows for normal weight concrete (Article 5.4.2.6):

$$f_r = 0.24\sqrt{f'_c} = 0.24\sqrt{4.0} = 0.480 \text{ ksi}$$

ϕ is the appropriate resistance factor for concrete in tension specified in Article 5.5.4.2.1. For reinforced concrete in tension, ϕ is equal to 0.90.

$$\phi f_r = 0.90(0.480) = 0.432 \text{ ksi}$$

Check the tensile stress in the concrete deck at the end of Cast 1 in Span 1 (100.0 feet from the abutment) caused by the negative moment due to Cast 2. From Table 11, the negative moment at the end of Cast 1 due to Cast 2 is –1,403 kip-feet. The longitudinal concrete deck stress is to be determined as specified in Article 6.10.1.1.1d; that is, using the short-term modular ratio n = 8. The STRENGTH IV load combination controls by inspection.

$$f_{deck} = \frac{1.0(1.5)(-1{,}403)(21.74)(12)}{166{,}624} = 0.329 \text{ ksi} < 0.432 \text{ ksi}$$

Therefore, the minimum one percent longitudinal reinforcement is not required at this section. Where it is required, the reinforcement should be No. 6 bars or smaller spaced at not more than 12 inches. Although not done in this example, a more accurate estimate of the concrete strength at the time Cast 2 is made, and the resulting modular ratio, can be used in this check.

Note that the total tensile force in the concrete deck at the end of Cast 1 is (0.329)(114.0)(9.0) = 338 kips. This force will be transferred from the deck through the shear connectors to the top flange. Sufficient shear connectors should be present at this location to resist this force and prevent potential crushing of the concrete around the studs or fracturing of the studs. To estimate the length over which this force must be transmitted, assume a 45-degree angle from the end of the cast to where the concrete deck is assumed effective. Therefore, the length in this particular case is estimated to be 57.0 inches. Later calculations show that the pitch of the studs is 12.0 inches in this region and that there are three studs per row. The nominal shear resistance of an individual stud is computed to be 36.0 kips (for f_c' equal to 4.0 ksi). The force resisted by the 12 studs within the 57-inch length is 12(36.0) = 432 kips > 338 kips. If necessary, the tensile force in the deck can be lowered by modifying the placement sequence.

10.2.2. Service Limit State (Article 6.10.4)

Article 6.10.4 contains provisions related to the control of elastic and permanent deformations at the service limit state.

10.2.2.1. Elastic Deformations (Article 6.10.4.1)

For control of elastic deformations, Article 6.10.4.1 refers back to Article 2.5.2.6, which contains optional live-load deflection criteria and criteria for span-to-depth ratios. The suggested span-to-depth ratios were utilized earlier to establish a reasonable minimum web depth for the example girder design.

The maximum computed live-load deflections at the service limit state for the example girder were reported earlier to be 0.91 inches in the end spans and 1.23 inches in the center span. The suggested live-load deflection limit for a vehicular load is Span/800 (Article 2.5.2.6.2). Therefore,

$$\text{End Spans:} \quad \Delta_{ALLOW} = \frac{140.0(12)}{800} = 2.10 \text{ in.} > 0.91 \text{ in.} \quad \text{ok} \quad (\text{Ratio} = 0.433)$$

$$\text{Center Span:} \quad \Delta_{ALLOW} = \frac{175.0(12)}{800} = 2.63 \text{ in.} > 1.23 \text{ in.} \quad \text{ok} \quad (\text{Ratio} = 0.468)$$

10.2.2.2. Permanent Deformations (Article 6.10.4.2)

Article 6.10.4.2 contains criteria intended to control objectionable permanent deformations due to expected severe traffic loadings 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 specified in Table 3.4.1-1.

According to Article 6.10.4.2.2, flanges must satisfy the following requirements:

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

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

where f_f is the flange stress at the section under consideration due to the SERVICE II loads calculated without consideration of flange lateral bending, and f_ℓ is the flange lateral bending stress due to the SERVICE II loads determined as specified in Article 6.10.1.6. Note that a resistance factor is not shown 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.

The sign of f_f and f_ℓ is always taken as positive in Eq. 6.10.4.2.2-2. f_ℓ is not included in Eq. 6.10.4.2.2-1 because the top flange of composite sections is continuously braced by the concrete deck at the service limit state; thus, flange lateral bending stresses are small and may be neglected. For straight-girder bridges, lateral bending in the bottom flange at the service limit state is only a consideration for bridges with staggered cross-frames in conjunction with skews exceeding 20°. Wind-load and deck overhang effects are not considered at the service limit state. Therefore, the f_ℓ term will be taken equal to zero in Eq. 6.10.4.2.2-2 in this example.

With the exception of composite sections in positive flexure in which the web satisfies the requirement of Article 6.10.2.1.1 (i.e. $D/t_w \leq 150$) such that longitudinal stiffeners are not required, 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)

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 bend-buckling resistance for webs determined as specified in Article 6.10.1.9. Because Section 1-1 is a composite section subject to positive flexure without longitudinal web stiffeners, 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.

Check the flange stresses due to the SERVICE II loads at Section 1-1. η is specified to always equal 1.0 at the service limit state (Article 1.3):

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

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

$$f_f = 1.0\left[\frac{1.0(2{,}202)}{1{,}581} + \frac{1.0(335+322)}{5{,}375} + \frac{1.3(3{,}510)}{16{,}272}\right]12 = -21.55\ \text{ksi}$$

$$|-21.55|\ \text{ksi} < 47.50\ \text{ksi} \quad \text{ok} \quad (\text{Ratio } 0.454)$$

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

$$f_f = 1.0\left[\frac{1.0(2{,}202)}{1{,}973} + \frac{1.0(335+322)}{2{,}513} + \frac{1.3(3{,}510)}{2{,}725}\right]12 = 36.62\ \text{ksi}$$

$$36.62\ \text{ksi} + 0 < 47.50\ \text{ksi} \quad \text{ok} \quad (\text{Ratio } 0.771)$$

Under the load combinations specified in Table 3.4.1-1 and in the absence of flange lateral bending, the above flange-stress criterion will often govern the size of the bottom flange for *compact* composite sections in positive flexure; that is, assuming fatigue limit state criteria do not control. In this particular example, fatigue limit state criteria control the size of the bottom flange at Section 1-1, as will be demonstrated in the next section. Regardless, it may be prudent and expedient in such cases to initially size the bottom flange to satisfy this stress criterion and then subsequently check the nominal flexural resistance at the fatigue and strength limit states.

Finally, it should be noted that for continuous span flexural members that satisfy the requirements of Article B6.2 to ensure adequate robustness and ductility of the pier sections, a calculated percentage of the negative moment due to the SERVICE II loads at the pier section under consideration may be redistributed prior to making the preceding checks. The moments may be redistributed using the optional procedures of Appendix B (to Section 6 of *AASHTO LRFD (5th Edition, 2010)* – specifically, Articles B6.3 or B6.6). When the redistribution moments are calculated according to these procedures, Eqs. 6.10.4.2.2-1 and 6.10.4.2.2-2 need not be checked within the regions extending from the pier section under consideration to the nearest flange transition or point of permanent-load contraflexure, whichever is closest, in each adjacent span. Eq. 6.10.4.2.2-4 must still be considered within these regions using the elastic moments prior to redistribution. At all locations outside of these regions, Eqs. 6.10.4.2.2-1, 6.10.4.2.2-2 and 6.10.4.2.2-4, as applicable, must be satisfied after redistribution.

10.2.2.3. Concrete Deck (Article 6.10.1.7)

As discussed previously, 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 (Table 3.4.1-1) exceeds ϕf_r. Earlier calculations showed that this minimum longitudinal reinforcement is not required within the limits of Cast 1 in Span 1 due to the factored construction loads.

Check the tensile stress in the concrete deck due to the SERVICE II load combination at the section 100.0 feet from the abutment in Span 1. The longitudinal concrete deck stress is to be determined as specified in Article 6.10.1.1.1d; that is, using the short-term modular ratio n = 8. Note that only DC_2, DW and LL+IM are assumed to cause stress in the concrete deck.

$$f_{deck} = \frac{1.0[1.0(25) + 1.0(27) + 1.3(-1{,}832)](21.74)(12)}{166{,}624} = 0.456 \text{ ksi} > 0.90 f_r = 0.432 \text{ ksi}$$

Therefore, check the tensile stress in the concrete deck due to the SERVICE II load combination at a section 98.0 feet from the abutment in Span 1.

$$f_{deck} = \frac{1.0[1.0(50) + 1.0(52) + 1.3(-1{,}754)](21.74)(12)}{166{,}624} = 0.426 \text{ ksi} < 0.432 \text{ ksi} \quad \text{ok}$$

Extend the minimum one-percent longitudinal reinforcement one foot further to a section 97.0 feet from the abutment in Span 1. The Engineer should further ensure that the reinforcement is adequately developed at this point.

10.2.3. Fatigue And Fracture Limit State (Article 6.10.5)

As specified in Article 6.10.5.1, details on I-section flexural members must be investigated for fatigue as specified in Article 6.6.1. For checking load-induced fatigue, the FATIGUE load combinations specified in Table 3.4.1-1 and the fatigue live load specified in Article 3.6.1.4 apply. As specified in Article 6.10.5.2, fracture toughness requirements in the contract documents must be in conformance with the provisions of Article 6.6.2. Finally, a special fatigue requirement for webs must be checked according to the provisions of Article 6.10.5.3.

10.2.3.1. Load Induced Fatigue (Article 6.6.1.2)

Fatigue of the base metal at the connection-plate welds to the flanges at the third intermediate cross-frame in Span 1, located 72.0 feet from the abutment, will be checked for the fatigue limit state. Separate calculations indicate that this is the critical connection-plate weld detail in Field Section 1. Fatigue of the base metal at the stud shear-connector weld to the top flange at the right end of Field Section 1 (located 100.0 feet from the abutment) will also be checked. 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 earlier, are also used.

From Article 3.6.1.4.2, the single-lane average daily truck traffic $(ADTT)_{SL}$ is:

$(ADTT)_{SL}$	=	p x ADTT	Eq. (3.6.1.4.2-1)
where: ADTT	=	number of trucks per day *in one direction* averaged over the design life (assumed to be 2,000 for this example)	
p	=	fraction of truck traffic in a single lane (Table 3.6.1.4.2-1)	
For a 3-lane bridge, p	=	0.80	
$\therefore$ $(ADTT)_{SL}$	=	0.80(2,000) = 1,600 trucks/day	

The provisions of Article 6.6.1.2 apply only to details subject to a net applied tensile stress. According to Article 6.6.1.2.1, in regions where the unfactored permanent loads produce compression, fatigue is to be considered only if this compressive stress is less than twice the maximum tensile stress resulting from the FATIGUE load combination. Note that the live-load stress due to the passage of the fatigue load is considered to be that of the heaviest truck expected to cross the bridge in 75 years. 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 provided 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 assumed effective in computing all stresses and stress ranges applied to the composite section in the subsequent fatigue calculations.

10.2.3.1.1. Top-Flange Connection-Plate Weld

Check fatigue of the base metal at the connection-plate welds to the flanges at the third intermediate cross-frame in Span 1, located 72.0 feet from the abutment.

Determine the fatigue detail category from Table 6.6.1.2.3-1. Under the condition of fillet-welded connections with welds normal to the direction of stress, the fatigue detail category for base metal at transverse stiffener-to-flange welds is Category C′. The total unfactored permanent-load compressive stress at the top-flange weld at this location (neglecting the future wearing surface) is computed as:

$$f_{DC_1} = \frac{1{,}824(12)(38.63)}{62{,}658} = -13.49 \text{ ksi}$$

$$f_{DC_2} = \frac{281(12)(21.74)}{122,231} = \underline{-0.600 \text{ ksi}}$$

$$-14.09 \text{ ksi}$$

According to Article 6.6.1.2.3, since the projected 75-year $(ADTT)_{SL}$ of 1,600 trucks per day exceeds the value of 754 trucks per day specified in Table 6.6.1.2.3-2, the detail should be designed for an infinite life using the FATIGUE I load combination. The maximum tensile stress at the top-flange weld at this location due to the negative moment caused by the factored fatigue load (factored by the 1.50 load factor specified for the FATIGUE I load combination) is:

$$f_{LL+IM} = \frac{(1.50)|-496|(12)(9.24)}{166,624} = 0.495 \text{ ksi}$$

$$|-14.09| \text{ ksi} > 0.495 \text{ ksi}$$

The fatigue live load negative bending does not overcome the positive bending due to permanent load. Therefore, fatigue of the base metal at the connection-plate weld to the top flange at this location need not be checked.

10.2.3.1.2. Bottom-Flange Connection-Plate Weld

By inspection, it is determined that the base metal at the connection-plate weld to the bottom flange at this location is subject to a net applied tensile stress. Thus, the stress range $\gamma(\Delta f)$ at the connection-plate weld due to the FATIGUE I load combination is computed using the properties of the short-term composite section as:

$$\gamma(\Delta f) = \frac{1.50(1,337)(12)(59.77)}{166,624} + \frac{1.50|-496|(12)(59.77)}{166,624} = 11.8 \text{ ksi}$$

According to Eq. 6.6.1.2.2-1, $\gamma(\Delta f)$ must not exceed the nominal fatigue resistance $(\Delta F)_n$. 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). From Eq. 6.6.1.2.5-1, the nominal fatigue resistance for the FATIGUE I load combination and infinite life is determined as:

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

For a Category C′ detail, $(\Delta F)_{TH}$ = 12.0 ksi (Table 6.6.1.2.5-3). Therefore:

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

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

11.8 ksi < 12.0 ksi ok (Ratio = 0.983)

The above fatigue limit-state check at the connection-plate weld to the bottom flange ends up governing the design of the bottom flange in this region (see the tabulation of performance ratios in the Design Example Summary at the end of the example). An alternative is to bolt the connection plates to the bottom flange, only in this region of high stress range, to raise the nominal fatigue resistance to that for a Category B detail. Bolting these particular connection plates to the tension flange will raise the nominal fatigue resistance to 16.00 ksi and may allow the designer to use a smaller bottom-flange plate in this region. However, the designer is cautioned that a Category C' detail still exists at the termination of the connection-plate weld to the web just above the bottom flange. Also, the bolted connections must be detailed properly to ensure a positive attachment to the flange that offers rotational fixity to prevent distortion-induced fatigue caused by out-of-plane deformations (Article 6.6.1.3). In most instances, bolting the connection plates to the flange is more expensive than welding the connection plates to the flange; thus, it is prudent for the Engineer to consult a fabricator to determine the most overall cost-effective solution.

The Engineer is also reminded that the nominal fatigue resistance of uncoated weathering steel base metal detailed in accordance with the Federal Highway Administration Technical Advisory (T5140.22) *Uncoated Weathering Steel in Structures* is determined for fatigue detail Category B (Table 6.6.1.2.3-1). However, it should be noted that fatigue considerations related to Category B details rarely control.

10.2.3.1.3. Stud Shear-Connector Weld

Check fatigue of the base metal at the stud shear-connector weld to the top flange at the right end of Field Section 1 (located 100.0 feet from the abutment). The total unfactored permanent-load compressive stress in the top flange at this location (neglecting the future wearing surface) is computed as:

$$f_{DC_1} = \frac{74(12)}{1{,}581} = -0.56\,\text{ksi}$$

$$f_{DC_2} = \frac{27(12)}{5{,}375} = \underline{-0.060\ \text{ksi}}$$

$$-0.620\ \text{ksi}$$

In order to compute the stress due to the factored fatigue load, first determine the fatigue detail category from Table 6.6.1.2.3-1.

Under the condition of longitudinally loaded fillet-welded attachments, the fatigue detail category for base metal adjacent to welded stud-type shear connectors is Category C.

From Table 6.6.1.2.3-2, the 75-year $(ADTT)_{SL}$ equivalent to infinite fatigue life for a Category C detail for *n* equal to 1.0 is 1,290 trucks per day. According to Article 6.6.1.2.3, since the projected 75-year $(ADTT)_{SL}$ of 1,600 trucks per day exceeds the value of 1,290 trucks per day specified in Table 6.6.1.2.3-2, the detail should be designed for an infinite life using the FATIGUE I load combination.

The maximum tensile stress at the top-flange weld at this location due to the negative moment caused by the FATIGUE I load combination is:

$$f_{LL+IM} = \frac{1.50\left|-688\right|(12)}{16{,}272} = 0.761 \text{ ksi}$$

$$\left|-0.620\right| \text{ksi} < 0.761 \text{ ksi}$$

Therefore, fatigue of the base metal at the stud shear-connector weld to the top flange at this location must be checked.

The stress range $\gamma(\Delta f)$ at the stud shear-connector weld due to the factored fatigue load (FATIGUE I load combination) is computed using the properties of the short-term composite section as:

$$\gamma(\Delta f) = \frac{1.50(912)(12)}{16{,}272} + \frac{1.50\left|-688\right|(12)}{16{,}272} = 1.77 \text{ ksi}$$

For a Category C detail, $(\Delta F)_{TH}$ = 10.0 ksi (Table 6.6.1.2.5-3). For the FATIGUE I load combination and infinite life, the nominal fatigue resistance is:

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

Therefore:

$$(\Delta F)_n = 10.0 \text{ ksi}$$

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

1.77 ksi < 10.0 ksi ok (Ratio = 0.177)

10.2.3.2. Distortion Induced Fatigue (Article 6.6.1.3)

To prevent distortion induced fatigue, all transverse connection-plate details will provide a positive connection to both the top and bottom flanges.

10.2.3.3. 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 Vnotch fracture toughness requirements for nonfracture-critical material (Table 6.6.2-2) specified for Temperature Zone 2 (Table 6.6.2-1).

10.2.3.4. Special Fatigue Requirement for Webs (Article 6.10.5.3)

Interior panels of stiffened webs must satisfy the following requirement:

$$V_u \leq V_{cr} \qquad \text{Eq. (6.10.5.3-1)}$$

where: V_u = shear in the web at the section under consideration due to the unfactored permanent loads plus the factored fatigue load
V_{cr} = shear buckling resistance determined from Eq. 6.10.9.3.3-1

In this check, the factored fatigue load is to be determined using the FATIGUE I load combination (Table 3.4.1-1), with the fatigue live load taken as specified in Article 3.6.1.4. Again, the fatigue live load is modified by the dynamic load allowance of 15 percent and the lateral distribution factors for the fatigue limit state are used. The live load stress for this check is intended to represent the heaviest truck expected to cross the bridge over a 75-year design life. Satisfaction of Eq. 6.10.5.3-1 is intended to control elastic flexing of the web so that the member is assumed able to sustain an infinite number of smaller loadings without fatigue cracking due to this effect.

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 critical panel in Field Section 1 will be checked. The critical panel for this check is the second panel from the abutment, which is located adjacent to the end panel. The transverse stiffener spacing in the end panel is d_o = 7.25 feet (see later shear calculations). The stiffener spacing in the second panel is d_o = 16.75 feet = 201.0 inches (up to the first intermediate cross-frame in Span 1). The shear 7.25 feet from the abutment to be used in this check is computed as follows:

$$V_u = 73.0 + 11 + 10 + 1.50(47)) = 165 \text{ kips at } 7'\text{-}3'' \text{ from the abutment}$$

The shear buckling resistance of the 201-inch-long panel is determined as:

$$V_n = V_{cr} = CV_p$$

C is the ratio of the shear buckling resistance to the shear yield strength determined from Eq. 6.10.9.3.2-4, 6.10.9.3.2-5 or 6.10.9.3.2-6, as applicable. First, compute the shear buckling coefficient, k

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

$$k = 5 + \frac{5}{\left(\frac{201.0}{69.0}\right)^2} = 5.59$$

Since, $$1.40\sqrt{\frac{Ek}{F_{yw}}} = 1.40\sqrt{\frac{29{,}000(5.59)}{50}} = 79.7 < \frac{D}{t_w} = \frac{69.0}{0.5} = 138.0$$

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

$$C = \frac{1.57}{(138.0)^2}\left(\frac{29{,}000(5.59)}{50}\right) = 0.267$$

V_p is the plastic shear force determined as follows:

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

$$V_p = 0.58(50)(69.0)(0.5) = 1{,}001 \text{ kips}$$

Therefore, $V_{cr} = 0.267(1{,}001) = 267 \text{ kips} > V_u = 165.0 \text{ kips}$ ok (Ratio = 0.618)

10.2.4. Strength Limit State (Article 6.10.6)

10.2.4.1. Flexure (Article 6.10.6.2)

For composite sections in positive flexure, Article 6.10.6.2.2 refers to the provisions of Article 6.10.7 to determine the nominal flexural resistance at the strength limit state.
Determine if Section 1-1 qualifies as a compact section. According to Article 6.10.6.2.2, composite sections in positive flexure qualify as compact when: 1) the specified minimum yield strengths of the flanges do not exceed 70 ksi, 2) the web satisfies the requirement of Article 6.10.2.1.1 such that longitudinal stiffeners are not required (i.e. $D/t_w \leq 150$), and 3) the section satisfies the following web-slenderness limit:

$$\frac{2D_{cp}}{t_w} \leq 3.76\sqrt{\frac{E}{F_{yc}}}$$ Eq. (6.10.6.2.2-1)

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 therefore, Eq. 6.10.6.2.2-1 is considered to be automatically satisfied. Section 1-1 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 1-1:

$$D_p = 9.0 + 3.5 - 1.0 + 0.44 = 11.94\text{ in.}$$

$$D_t = 1.375 + 69.0 + 3.5 + 9.0 = 82.88\text{ in.}$$

$$0.42D_t = 0.42(82.88) = 34.81\text{ in.} > 11.94\text{ in.} \quad \text{ok} \quad (\text{Ratio} = 0.343)$$

According to Article 6.10.7.1.1, at the strength limit state, compact composite sections in positive flexure must satisfy the following relationship:

$$M_u + \frac{1}{3} f_\ell S_{xt} \leq \phi_f M_n \qquad \text{Eq. (6.10.7.1.1-1)}$$

where: ϕ_f = resistance factor for flexure = 1.0 (Article 6.5.4.2)
f_ℓ = lateral bending stress in the tension flange determined as specified in Article 6.10.1.6
M_n = nominal flexural resistance of the section determined as specified in Article 6.10.7.1.2
M_u = bending moment about the major-axis of the cross-section determined as specified in Article 6.10.1.6
S_{xt} = elastic section modulus about the major-axis of the section to the tension flange taken as M_{yt}/F_{yt}
M_{yt} = yield moment with respect to the tension flange determined as specified in Article D6.2

As specified in Article 6.10.1.6, for design checks where the flexural resistance is based on yielding (which is the case here), M_u may be taken as the moment due to the factored loads at the section under consideration.

In this example, lateral bending in the bottom flange due to wind-load effects will be considered at the strength limit state. For composite sections in positive flexure, lateral bending does not need to be considered in the compression flange at the strength limit state because the flange is continuously supported by the concrete deck. In Eq. 6.10.7.1.1-1, f_ℓ is the flange lateral bending stress determined as specified in Article 6.10.1.6. According to Article 6.10.1.6, for design checks where the flexural resistance is based on yielding, f_ℓ may be determined as the stress at the section under consideration. For simplicity in this example, however, 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 I-girder bridges with composite concrete decks, wind load on the upper half of the exterior girder, the deck, the barriers and the vehicles may be assumed transmitted directly to the deck, which acts as a lateral diaphragm to carry the load to the supports. Wind load on the lower half of the exterior girder may be assumed applied laterally to the bottom flange, which transmits the load to the adjacent cross-frames or diaphragms by flexural action. The frame action of the cross-frames or diaphragms then transmits the forces to the deck, which in turn transmits them to the supports through diaphragm action.

Article C4.6.2.7.1 provides the following formula for the factored wind force per unit length applied to the bottom flange of composite or noncomposite exterior members with cast-in-place concrete or orthotropic steel decks:

$$W = \frac{\eta_i \gamma P_D d}{2} \qquad \text{Eq. (C4.6.2.7.1-1)}$$

where P_D is the design horizontal wind pressure specified in Article 3.8.1 and d is the depth of the girder. Earlier, P_D was computed to be 0.053 ksf.

For the wind-load path identified above, Article C4.6.2.7.1 also provides the following approximate equation for computing the maximum flange lateral bending moment due to the factored wind load within the unbraced length under consideration:

$$M_w = \frac{WL_b^2}{10} \qquad \text{Eq. (C4.6.2.7.1-2)}$$

Assemble the factored actions needed to check Eq. 6.10.7.1.1-1 at Section 1-1. The unbraced length, L_b, at Section 1-1 is 24.0 feet. In this example, η is taken equal to 1.0 at the strength limit state. The wind load acting on the live load (WL) is assumed transmitted directly to the deck and is therefore not considered in the STRENGTH V load combination in this example. For simplicity, the effect of the overturning force due to WL on the vehicle wheel loads is also not considered in this example. The amplification factor, AF, for f_ℓ (Article 6.10.1.6) is taken equal to 1.0 for flanges in tension.

Note again 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.

For STRENGTH I:

Dead and live loads: $M_u = 1.0[1.25(2{,}202 + 335) + 1.5(322) + 1.75(3{,}510)] = 9{,}797 \text{ kip} - \text{ft}$

Wind loads: Not considered $\therefore$ $f_\ell = 0$

For STRENGTH III:

Dead loads: $M_u = 1.0[1.25(2{,}202 + 335) + 1.5(322)] = 3{,}654 \text{ kip} - \text{ft}$

Wind loads: $$W = \frac{1.0(1.4)(0.053)(1.375 + 69.0 + 1.0)/12}{2} = 0.221 \text{ kips/ft}$$

$$M_w = \frac{0.221(24.0)^2}{10} = 12.73 \text{ kip} - \text{ft}$$

$$f_\ell = \frac{M_w}{S_\ell} = \frac{12.73(12)}{1.375(18)^2/6} = 2.06 \text{ ksi} * \text{AF} = 2.06(1.0) = 2.06 \text{ ksi} < 0.6F_{yf} = 30.0 \text{ ksi ok}$$

For STRENGTH IV:

Dead loads: $M_u = 1.0[1.5(2{,}202 + 335 + 322)] = 4{,}289 \text{ kip} - \text{ft}$

Wind loads: Not considered $\therefore$ $f_\ell = 0$

For STRENGTH V:

Dead and live loads: $M_u = 1.0[1.25(2{,}202 + 335) + 1.5(322) + 1.35(3{,}510)] = 8{,}393 \text{ kip} - \text{ft}$

Wind loads: $$W = \frac{1.0(0.4)(0.053)(1.375 + 69.0 + 1.0)/12}{2} = 0.063 \text{ kips/ft}$$

$$M_w = \frac{0.063(24.0)^2}{10} = 3.63 \text{ kip} - \text{ft}$$

$$f_\ell = \frac{M_w}{S_\ell} = \frac{3.63(12)}{1.375(18)^2/6} = 0.587 \text{ ksi} * \text{AF} = 0.587(1.0) = 0.587 \text{ ksi} < 0.6F_{yf} = 30.0 \text{ ksi ok}$$

From an examination of the above flange lateral bending stresses, it is apparent that for typical cross-frame spacings, the majority of the wind force on the lower half of a composite structure is transmitted directly to the deck through the cross-frames and only a small portion of the force is resisted through lateral bending of the bottom flange.

10.2.4.1.1. Nominal Flexural Resistance (Article 6.10.7.1.2)

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. However, in a continuous span, the nominal flexural resistance of the section is limited to the following:

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

where M_y is the yield moment of the composite section determined as specified in Article D6.2, unless the specific steps outlined in Article 6.10.7.1.2 are taken to ensure sufficient ductility and robustness of adjacent pier sections such that the redistribution of moments caused by partial yielding within the positive flexural regions is inconsequential. Specifically, Articles B6.2 and B6.6.2 in Appendix B (to Section 6 of *AASHTO LRFD (5th Edition, 2010)*) are referred to for obtaining the requirements that must be satisfied to avoid the limitation given by Eq. 6.10.7.1.2-3.

For Section 1-1, M_p and M_y were computed earlier to be 14,199 kip-ft and 10,171 kip-ft, respectively.

$$0.1D_t = 0.1(82.88) = 8.29 \text{ in.} < D_p = 11.94 \text{ in.}$$

Therefore, $$M_n = 14{,}199\left[1.07 - 0.7\left(\frac{11.94}{82.88}\right)\right] = 13{,}761 \text{ kip} - \text{ft}$$

Or, $$M_n = 1.3(1.0)(10{,}171) = 13{,}222 \text{ kip} - \text{ft} \qquad \text{(governs)}$$

Therefore, M_n = 13,222 kip-ft

Calculate S_{xt}. The yield moment, M_y, was calculated with respect to the tension flange; therefore, $M_{yt} = M_y$:

$$S_{xt} = \frac{M_{yt}}{F_{yt}} = \frac{10{,}171(12)}{50} = 2{,}441 \text{ in}^3$$

Now that all the required information has been assembled, check Eq. 6.10.7.1.1-1:

$$M_u + \frac{1}{3} f_\ell S_{xt} \le \phi_f M_n \qquad \text{Eq. (6.10.7.1.1-1)}$$

For STRENGTH I:

$$M_u + \frac{1}{3} f_\ell S_{xt} = 9{,}797 \text{ kip} - \text{ft} + 0 = 9{,}797 \text{ kip} - \text{ft}$$

$$\phi_f M_n = 1.0(13{,}222) = 13{,}222 \text{ kip} - \text{ft}$$

$$9{,}797 \text{ kip} - \text{ft} < 13{,}222 \text{ kip} - \text{ft} \quad \text{ok} \qquad (\text{Ratio} = 0.741)$$

For STRENGTH III:

$$M_u + \frac{1}{3} f_\ell S_{xt} = 3{,}654 \text{ kip} - \text{ft} + \frac{1}{3}\frac{(2.06)(2{,}441)}{12} = 3{,}794 \text{ kip} - \text{ft}$$

$$\phi_f M_n = 1.0(13{,}222) = 13{,}222 \text{ kip} - \text{ft}$$

$$3{,}794 \text{ kip} - \text{ft} < 13{,}222 \text{ kip} - \text{ft} \quad \text{ok} \qquad (\text{Ratio} = 0.287)$$

For STRENGTH IV:

$$M_u + \frac{1}{3} f_\ell S_{xt} = 4{,}289 \text{ kip} - \text{ft} + 0 = 4{,}289 \text{ kip} - \text{ft}$$

$$\phi_f M_n = 1.0(13{,}222) = 13{,}222 \text{ kip} - \text{ft}$$

$$4{,}289 \text{ kip} - \text{ft} < 13{,}222 \text{ kip} - \text{ft} \quad \text{ok} \qquad (\text{Ratio} = 0.324)$$

For STRENGTH V:

$$M_u + \frac{1}{3} f_\ell S_{xt} = 8{,}393 \text{ kip} - \text{ft} + \frac{1}{3}\frac{(0.587)(2{,}441)}{12} = 8{,}433 \text{ kip} - \text{ft}$$

$$\phi_f M_n = 1.0(13{,}222) = 13{,}222 \text{ kip} - \text{ft}$$

$$8{,}433 \text{ kip} - \text{ft} < 13{,}222 \text{ kip} - \text{ft} \quad \text{ok} \qquad (\text{Ratio} = 0.638)$$

10.2.4.2. Shear (6.10.6.3)

Article 6.10.6.3 refers to the provisions of Article 6.10.9 to determine the nominal flexural resistance at the strength limit state.

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 = shear in the 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. The total design shears due to the factored loads, V_u, at each tenth point along the *interior* girder for the STRENGTH I load combination are plotted in Figure 16. The STRENGTH I load combination controls for shear by inspection, and the total factored shears in the interior girder are larger under the STRENGTH I load combination. 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_u at the abutment is given below:

$$V_u = 1.0[1.25(87 + 13) + 1.5(13) + 1.75(139)] = 388 \text{ kips}$$

The required spacing of transverse stiffeners in Field Section 1 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 from Eq. 6.10.9.3.2-4, 6.10.9.3.2-5 or 6.10.9.3.2-6, as applicable, *with the shear buckling coefficient, k, taken equal to 5.0.*

Since, $$1.40\sqrt{\frac{Ek}{F_{yw}}} = 1.40\sqrt{\frac{29{,}000(5.00)}{50}} = 75.4 < \frac{D}{t_w} = \frac{69.0}{0.5} = 138.0$$

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

$$C = \frac{1.57}{(138.0)^2}\left(\frac{29{,}000(5.00)}{50}\right) = 0.239$$

V_p is the plastic shear force determined as follows:

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

$$V_p = 0.58(50)(69.0)(0.5) = 1{,}001 \text{ kips}$$

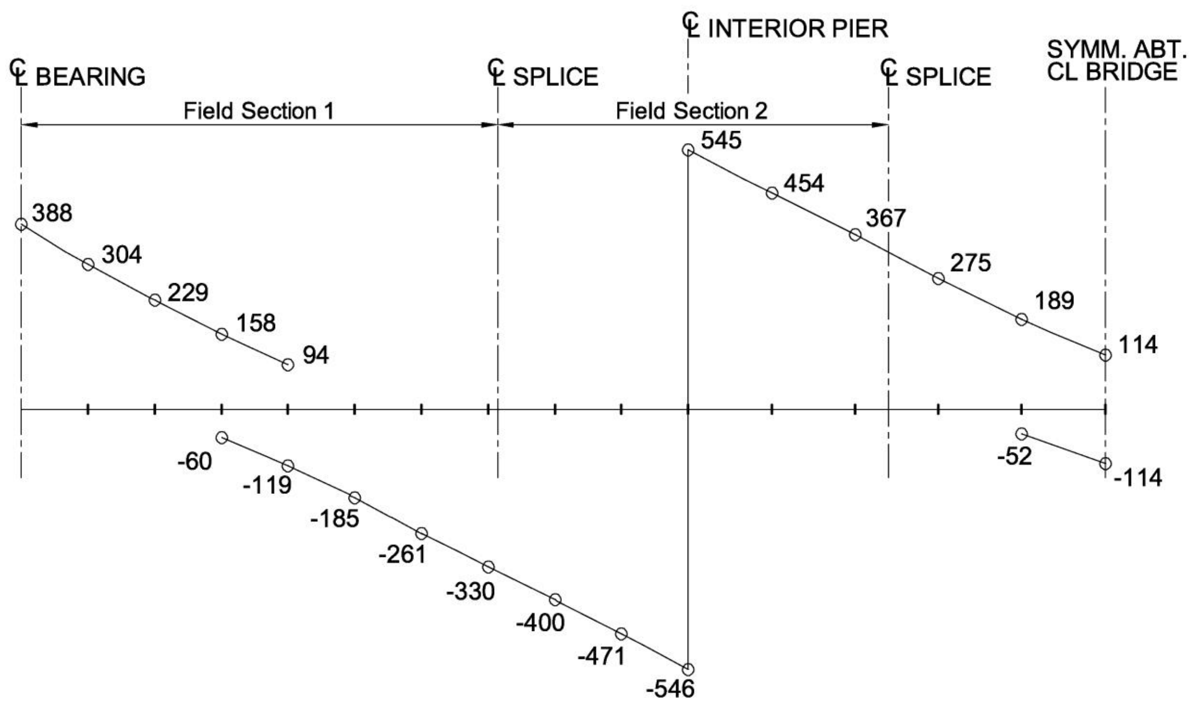

Figure 16: Design Shears Due to the Factored Loads - STRENGTH I

Shears shown are for the *interior* girder and are in kips

Therefore, $V_n = V_{cr} = 0.239(1{,}001) = 239 \text{ kips}$

$$\phi_v V_n = 1.0(239) = 239 \text{ kips}$$

The maximum value of V_u in Field Section 1 is 388 kips (Figure 16), which exceeds $\phi_v V_n = 239$ kips. Therefore, transverse stiffeners are required in Field Section 1 and the provisions of Article 6.10.9.3 apply.

10.2.4.2.1. End Panel (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 from Eq. 6.10.9.3.2-4, 6.10.9.3.2-5 or 6.10.9.3.2-6, as applicable. 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(69.0) = 103.5 inches. Assume the spacing from the abutment to the first transverse stiffener is d_o = 7.25 feet = 87.0 inches.

$$k = 5 + \frac{5}{\left(\frac{87.0}{69.0}\right)^2} = 8.15$$

Since, $$1.40\sqrt{\frac{Ek}{F_{yw}}} = 1.40\sqrt{\frac{29{,}000(8.15)}{50}} = 96.3 < \frac{D}{t_w} = \frac{69.0}{0.5} = 138.0$$

$$C = \frac{1.57}{(138.0)^2}\left(\frac{29{,}000(8.15)}{50}\right) = 0.390$$

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

$$V_p = 0.58(50)(69.0)(0.5) = 1{,}001 \text{ kips}$$

Therefore, $$V_n = V_{cr} = 0.390(1{,}001) = 390 \text{ kips}$$

$$\phi_v V_n = 1.0(390) = 390 \text{ kips} > V_u = 388 \text{ kips} \quad \text{ok} \quad (\text{Ratio} = 0.995)$$

10.2.4.2.2. Interior Panels (Article 6.10.9.3.2)

According to Article 6.10.9.1, the transverse stiffener spacing for interior panels without a longitudinal stiffener is not to exceed 3D = 3(69.0) = 207.0 inches. For the first interior panel to the right of the end panel, assume a transverse stiffener spacing of d_o = 16.75 feet = 201.0 inches, which is the distance from the first transverse stiffener to the first intermediate cross-frame in Span 1 (assume that the cross-frame connection plate serves as a transverse stiffener). At the first transverse stiffener located d_o = 7.25 feet from the abutment, V_u is equal to 345 kips.

For interior panels of both nonhybrid and hybrid members with the section along the entire panel proportioned such that:

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

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 resistance determined from Eq. 6.10.9.3.2-8. Previous specifications did not permit web panels of hybrid members to develop postbuckling resistance due to tension-field action. Also, 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:

$$\frac{2(69.0)(0.5)}{\left[16(1.0) + 18(0.875)\right]} = 2.17 < 2.5$$

Therefore:

$$k = 5 + \frac{5}{\left(\frac{201.0}{69.0}\right)^2} = 5.59$$

Since,

$$1.40\sqrt{\frac{Ek}{F_{yw}}} = 1.40\sqrt{\frac{29{,}000(5.59)}{50}} = 79.7 < \frac{D}{t_w} = \frac{69.0}{0.5} = 138.0$$

$$C = \frac{1.57}{(138.0)^2}\left(\frac{29{,}000(5.59)}{50}\right) = 0.267$$

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

$$V_p = 0.58(50)(69.0)(0.5) = 1{,}001\ \text{kips}$$

$$V_n = 1{,}001\left[0.267 + \frac{0.87(1-0.267)}{\sqrt{1+\left(\frac{201.0}{69.0}\right)^2}}\right] = 475 \text{ kips}$$

Therefore,

$$\phi_v V_n = 1.0(475) = 475 \text{ kips} > V_u = 345 \text{ kips} \quad \text{ok} \quad (\text{Ratio} = 0.726)$$

V_u at the first intermediate cross-frame in Span 1 located 24.0 feet from the abutment is equal to 250 kips, which is greater than $\phi_v V_n$ = 239 kips for an unstiffened web. Therefore, assume a transverse stiffener spacing of d_o = 3D = 17.25 feet = 207.0 inches from the cross frame to the next stiffener.

$$k = 5 + \frac{5}{\left(\frac{207.0}{69.0}\right)^2} = 5.56$$

Since,

$$1.40\sqrt{\frac{Ek}{F_{yw}}} = 1.40\sqrt{\frac{29{,}000(5.56)}{50}} = 79.5 < \frac{D}{t_w} = \frac{69.0}{0.5} = 138.0$$

$$C = \frac{1.57}{(138.0)^2}\left(\frac{29{,}000(5.56)}{50}\right) = 0.266$$

$$V_p = 1{,}001 \text{ kips}$$

Therefore,

$$V_n = 1{,}001\left[0.266 + \frac{0.87(1-0.266)}{\sqrt{1+\left(\frac{207.0}{69.0}\right)^2}}\right] = 468 \text{ kips}$$

$$\phi_v V_n = 1.0(468) = 468 \text{ kips} > V_u = 250 \text{ kips} \quad \text{ok} \quad (\text{Ratio} = 0.534)$$

V_u at this stiffener is equal to 162 kips, which is less than $\phi_v V_n$ = 239 kips for an unstiffened web. Therefore, no additional transverse stiffeners are required at the left end of Field Section 1. At the right end of Field Section 1, V_u at the fourth intermediate cross frame located 96.0 feet from the abutment is equal to 320 kips, which exceeds $\phi_v V_n$ = 239 kips for an unstiffened web. Assume a transverse stiffener spacing of d_o = 3D = 17.25 feet = 207.0 inches to the left of this cross frame. For this panel:

$$\frac{2(69.0)(0.5)}{\left[16(1.0)+18(1.375)\right]}=1.69<2.5$$

Therefore, the nominal shear resistance may be taken as the sum of the shear buckling resistance and the postbuckling resistance due to tension-field action. As determined above for this stiffener spacing,

$$\phi_v V_n = 1.0(468) = 468 \text{ kips} > V_u = 320 \text{ kips} \quad \text{ok} \quad (\text{Ratio} = 0.684)$$

V_u at this stiffener is equal to 233 kips, which is less than $\phi_v V_n$ = 239 kips for an unstiffened web. Therefore, no additional transverse stiffeners are required at the right end of Field Section 1.

10.3. Exterior Girder Check: Section 2-2

10.3.1. Strength Limit State (Article 6.10.6)

10.3.1.1. Flexure (Article 6.10.6.2)

For composite sections in negative flexure at the strength limit state, Article 6.10.6.2.3 first asks the Engineer to determine if the web of the section satisfies the following noncompact slenderness limit:

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

where D_c is the depth of the web in compression in the elastic range. For composite sections, D_c is to be determined as specified in Article D6.3.1. According to Article D6.3.1 (Appendix D to Section 6 of the *AASHTO LRFD (5th Edition, 2010)*), for composite sections in negative flexure at the strength limit state, D_c is to be computed for the section consisting of the steel girder plus the longitudinal reinforcement. Therefore, at Section 2-2, D_c is equal to 36.96 inches from the elastic section properties computed earlier. Recall that F_{yc} at Section 2-2 is 70 ksi. Therefore,

$$5.7\sqrt{\frac{29{,}000}{70}}=116.0$$

$$\frac{2(36.96)}{0.5625}=131.4>116.0$$

Thus, Section 2-2 is classified as slender-web section and the provisions of Article 6.10.8 must be used to compute the nominal flexural resistance. Since the specified minimum yield strengths of the flanges do not exceed 70 ksi, the optional provisions of Appendix A (to Section 6 of *AASHTO LRFD (5th Edition, 2010)*) could have been used to compute the nominal flexural resistance had Eq. 6.10.6.2.3-1 been satisfied. In Appendix A, which is applicable to either noncompact web or compact web sections, the nominal flexural resistance is permitted to exceed the moment at first yield. The provisions of Article 6.10.8 may be used instead for these types of sections, if desired, but at the expense of some economy; in particular, for compact web sections. The potential loss in economy increases with decreasing web slenderness.

According to Article 6.10.8.1, for composite sections in negative flexure, the following relationship must be satisfied for the discretely braced compression flange at the strength limit state:

$$f_{bu} + \frac{1}{3} f_\ell \leq \phi_f F_{nc} \qquad \text{Eq. (6.10.8.1.1-1)}$$

where: ϕ_f = resistance factor for flexure = 1.0 (Article 6.5.4.2)
F_{nc} = nominal flexural resistance of the compression flange determined as specified in Article 6.10.8.2 (i.e. local or lateral torsional buckling resistance, whichever controls)

The terms f_{bu} and f_ℓ are the same as defined earlier. At the strength limit state, the top (tension) flange is considered to be continuously braced by the composite concrete deck. According to Article 6.10.8.1.3, continuously braced flanges in tension must satisfy the following relationship at the strength limit state:

$$f_{bu} \leq \phi_f R_h F_{yf} \qquad \text{Eq. (6.10.8.1.3-1)}$$

As discussed in Article C6.10.1.6, any flange lateral bending stresses need not be considered once the flange is continuously braced.

Compute the maximum flange flexural stresses at Section 2-2 due to the factored loads under the STRENGTH I load combination, 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: $$f = 1.0\left[\frac{1.25(-4{,}840)}{2{,}942} + \frac{1.25(-690)}{3{,}228} + \frac{1.5(-664)}{3{,}228} + \frac{1.75(-4{,}040)}{3{,}808}\right]12 = 53.87 \text{ ksi}$$

Bot. flange: $$f = 1.0\left[\frac{1.25(-4{,}840)}{3{,}149} + \frac{1.25(-690)}{3{,}216} + \frac{1.5(-664)}{3{,}216} + \frac{1.75(-4{,}040)}{3{,}327}\right]12 = -55.49 \text{ ksi}$$

Calculate the nominal flexural resistance, F_{nc}, of the bottom (compression) flange taken as the smaller of the local buckling resistance and the lateral torsional buckling resistance according to Article 6.10.8.2.1.

10.3.1.1.1. Bottom Flange

10.3.1.1.1.1. Lateral Torsional Buckling Resistance (Article 6.10.8.2.3)

For illustration purposes, initially assume an unbraced length, L_b, on either side of the interior pier (Section 2-2) equal to 17.0 feet. In both unbraced lengths, there is a flange transition located 15.0 feet from the pier section (Figure 3). According to Article 6.10.8.2.3, for unbraced lengths containing a transition to a smaller section at a distance less than or equal to 20 percent of the unbraced length from the brace point with the smaller moment, the lateral torsional buckling resistance may be determined assuming the transition to the smaller section does not exist.
Based on this assumption, determine the limiting unbraced length, L_p:

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

where r_t is the effective radius of gyration for lateral torsional buckling determined as:

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

$$r_t = \frac{20}{\sqrt{12\left(1 + \frac{1}{3}\frac{(36.96)(0.5625)}{(20)(2)}\right)}} = 5.33 \text{ in.}$$

$$L_p = \frac{1.0(5.33)}{12}\sqrt{\frac{29{,}000}{70}} = 9.04 \text{ ft}$$

It should be emphasized here that the most economical solution is not usually achieved by limiting the unbraced length to L_p in order to reach the maximum lateral torsional buckling resistance (i.e. F_{max} in Figure C6.10.8.2.1-1). This is especially the case when the moment gradient modifier, C_b, (discussed below) is taken equal to 1.0.
Determine the limiting unbraced length, L_r:

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

where: $F_{yr} = 0.7F_{yc} \leq F_{yw}$

$$F_{yr} = 0.7(70) = 49.0 \text{ ksi} < 50 \text{ ksi} \quad \text{ok}$$

F_{yr} must also not be less than $0.5F_{yc} = 0.5(70) = 35.0$ ksi ok.

Therefore: $$L_r = \frac{\pi(5.33)}{12}\sqrt{\frac{29{,}000}{49.0}} = 33.95 \text{ ft}$$

For this unbraced length, since f_{mid}/f_2 is less than 1.0 and f_2 is not equal to zero, calculate the moment gradient modifier, C_b, according to Eq. 6.10.8.2.3-7 as follows:

$$C_b = 1.75 - 1.05\left(\frac{f_1}{f_2}\right) + 0.3\left(\frac{f_1}{f_2}\right)^2 \le 2.3 \qquad \text{Eq. (6.10.8.2.3-7)}$$

f_2 is generally taken as the largest compressive stress without consideration of lateral bending due to the factored loads at either end of the unbraced length of the flange under consideration, calculated from the critical moment envelope value. f_2 is always taken as positive. If the stress is zero or tensile in the flange under consideration at both ends of the unbraced length, f_2 is to be taken equal to zero (in this case, C_b = 1.0 and Eq. 6.10.8.2.3-7 does not apply). For the STRENGTH I load combination, which is assumed to control for this calculation in this example, f_2 is equal to the largest compressive stress in the bottom flange at Section 2-2 calculated previously to equal 55.49 ksi (f_2 is taken as positive for this calculation). The value of f_1 is given by Eq. 6.10.8.2.3-10 as:

$$f_1 = f_o \qquad \text{Eq. (6.10.8.2.3-10)}$$

where f_o is the stress without consideration of lateral bending due to the factored loads at the brace point opposite to the one corresponding to f_2. f_o is to be calculated from the moment envelope value that produces the largest compression at the point in the flange under consideration, or the smallest tension if that point is never in compression, and both are to be taken as positive in compression and negative in tension. Note that Article 6.10.8.2.3 states that for all cases where the variation in the moment along the entire length between the brace points is concave in shape, which is the case here, Eq. 6.10.8.2.3-10 is used to compute f_1.

The revisions to the definitions of f_1 and f_2 in Eq. 6.10.8.2.3-7 along with the introduction of f_{mid} (not used here) were done to remove ambiguities and to address a number of potentially important cases where the prior C_b calculations were significantly unconservative relative to more refined solutions. To illustrate, Appendix B (to this design example) shows the values of C_b calculated from Eq. 6.10.8.2.3-7 for a number of different potential cases.

For the unbraced length under consideration in this example, calculate $f_1 = f_o$ assuming the flange transition does not exist. Separate calculations show that the stress at the brace point on the left side of Section 2-2 controls for the STRENGTH I load combination. Therefore,

STRENGTH I:

$$\text{Bot. flange: } f_l = f_o = 1.0\left[\frac{1.25(-2{,}390)}{3{,}149} + \frac{1.25(-334)}{3{,}216} + \frac{1.5(-321)}{3{,}216} + \frac{1.75(-2{,}615)}{3{,}327}\right]12 = 31.24 \text{ ksi}$$

Note that f_o is taken as positive in compression.

$$C_b = 1.75 - 1.05\left(\frac{31.24}{55.49}\right) + 0.3\left(\frac{31.24}{55.49}\right) = 1.25 < 2.3 \quad \text{ok}$$

Determine the hybrid factor, R_h. According to the provisions of Article 6.10.1.10.1, the hybrid factor is to be taken as:

$$R_h = \frac{12 + \beta(3\rho - \rho^3)}{12 + 2\beta} \qquad \text{Eq. (6.10.1.10.1-1)}$$

where: $$\beta = \frac{2D_n t_w}{A_{fn}} \qquad \text{Eq. (6.10.1.10.1-2)}$$

and ρ equals the smaller of F_{yw}/f_n and 1.0. D_n is taken as the larger of the distances from the elastic neutral axis of the cross-section to the inside face of either flange. For sections where the neutral axis is at the mid-depth of the web, consult Article 6.10.1.10.1. At Section 2-2, D_n is equal to 36.96 inches (use D_n for the steel section plus the longitudinal reinforcement). A_{fn} is equal to the sum of the flange area and the area of any cover plates on the side of the neutral axis corresponding to D_n. For composite sections in negative flexure, the area of the longitudinal reinforcement may be included in calculating A_{fn} for the top flange (when applicable). At Section 2-2, A_{fn} is equal to the area of the bottom flange, or 20(2) = 40.0 in^2. Therefore,

$$\beta = \frac{2(36.96)(0.5625)}{40.0} = 1.040$$

For sections where yielding occurs first in the flange, a cover plate or the longitudinal reinforcement on the side of the neutral axis corresponding to D_n, f_n is taken as the largest of the specified minimum yield strengths of each component included in the calculation of A_{fn}. Otherwise, f_n is to be taken as the largest of the elastic stresses in the flange, cover plate or longitudinal reinforcement on the side of the neutral axis corresponding to D_n at first yield on the opposite side of the neutral axis. Separate calculations show that yielding occurs first in the bottom flange at Section 2-2. Therefore, f_n = 70.0 ksi.

$$\rho = \frac{F_{yw}}{f_n} = \frac{50.0}{70.0} = 0.714$$

$$R_h = \frac{12 + 1.040\left[3(0.714) - (0.714)^3\right]}{12 + 2(1.040)} = 0.984$$

Determine the web load-shedding factor, R_b. According to the provisions of Article 6.10.1.10.2, since:

$$\frac{2D_c}{t_w} = 131.4 > \lambda_{rw} = 5.7\sqrt{\frac{E}{F_{yc}}} = 116.0 \qquad \text{Eqs. (6.10.1.10.2-2), (6.10.1.10.2-4)}$$

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

where: $$a_{wc} = \frac{2D_c t_w}{b_{fc} t_{fc}} \qquad \text{Eq. (6.10.1.10.2-5)}$$

$$a_{wc} = \frac{2(36.96)(0.5625)}{20(2)} = 1.040$$

$$R_b = 1 - \left(\frac{1.040}{1{,}200 + 300(1.040)}\right)(131.4 - 116.0) = 0.989$$

Since L_p = 9.04 feet < L_b = 17.0 feet < L_r = 33.95 feet,

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

Eq. (6.10.8.2.3-2)

$$F_{nc} = 1.25\left[1 - \left(1 - \frac{49.0}{0.984(70.0)}\right)\left(\frac{17.0 - 9.04}{33.95 - 9.04}\right)\right](0.989)(0.984)(70) = 77.30\,\text{ksi} > 0.989(0.984)(70) = 68.12\,\text{ksi}$$

$\therefore F_{nc}$ = 68.12 ksi

For values of C_b greater than 1.0, Article D6.4.1 (Appendix D to Section 6 of *AASHTO LRFD (5th Edition, 2010)*) allows the maximum lateral torsional buckling resistance, $F_{nc} = F_{max} = R_b R_h F_{yc}$, to be reached at larger unbraced lengths. However, since F_{max} is already reached at L_b = 17.0 feet in this case, it is not necessary to utilize these provisions.

A lateral torsional buckling resistance of 68.12 ksi is not required for this particular unbraced length. Therefore, try a larger unbraced length of L_b = 20.0 feet on either side of Section 2-2. In this case, the flange transition is now located at a distance greater than 20 percent of the unbraced length from the brace point with the smaller moment. Therefore, according to Article 6.10.8.2.3, the lateral torsional buckling resistance is to be taken as the smallest resistance within the unbraced length under consideration. This resistance is to be compared to the largest value of the compressive stress due to the factored loads, f_{bu}, throughout the unbraced length calculated using the actual properties at each section. Note also that the moment gradient modifier, C_b, should be taken equal to 1.0, and L_b should not be modified by an elastic effective length factor when this approximate procedure is used.

Calculate the elastic section properties of the smaller section at the flange transition:

Table 14 Flange Transition: Steel Only Section Properties

Component	A	d	Ad	Ad^2	I_o	I
Top Flange 1" x 18"	18.00	35.00	630.0	22,050	1.50	22,052
Web 9/16" x 69"	38.81				15,399	15,399
Bottom Flange 1" x 20"	20.00	−35.00	−700.0	24,500	1.67	24,502
	76.81		−70.00			61,953
					$-0.91(70.00) =$	−63.70
					$I_{NA} =$	61,889 in.^{4}

$$d_s = \frac{-70.00}{76.81} = -0.91 \text{ in.}$$

$$d_{TOP\ OF\ STEEL} = 35.50 + 0.91 = 36.41 \text{ in.} \qquad d_{BOT\ OF\ STEEL} = 35.50 - 0.91 = 34.59 \text{ in.}$$

$$S_{TOP\ OF\ STEEL} = \frac{61{,}889}{36.41} = 1{,}700 \text{ in.}^3 \qquad S_{BOT\ OF\ STEEL} = \frac{61{,}889}{34.59} = 1{,}789 \text{ in.}^3$$

Table 15 Flange Transition: Steel Section + Long. Reinforcement/3

Component	A	d	Ad	Ad^2	I_o	I
Steel Section	76.81		−70.00			61,953
Long. Reinforcement/3	3.52	42.63	150.1	6,397		6,397
	80.33		80.10			68,350
					$-1.00(80.10) =$	−80.10
					$I_{NA} =$	68,270 in.^{4}

$$d_{reinf\ 3} = \frac{80.10}{80.33} = 1.00 \text{ in.}$$

$$d_{TOPOFSTEEL} = 35.50 - 1.00 = 34.50 \text{ in.} \qquad d_{BOTOFSTEEL} = 35.50 + 1.00 = 36.50 \text{ in.}$$

$$S_{TOP\ OF\ STEEL} = \frac{68{,}270}{34.50} = 1{,}979 \text{ in.}^3 \qquad S_{BOT\ OF\ STEEL} = \frac{68{,}270}{36.50} = 1{,}870 \text{ in.}^3$$

Table 16 Flange Transition: Steel Section + Long. Reinforcement

Component	A	d	Ad	Ad^2	I_o	I
Steel Section	76.81		−70.00			61,953
Long. Reinforcement	10.56	42.63	450.2	19,191		19,191
	87.37		380.2			81,144
				−4.35(380.2) =		−1,654
				I_{NA} =		79,490 in.^{4}

$$d_{reinf} = \frac{380.2}{87.37} = 4.35\,\text{in.}$$

$$d_{TOPOFSTEEL} = 35.50 - 4.35 = 31.15\,\text{in.} \qquad d_{BOTOFSTEEL} = 35.50 + 4.35 = 39.85\,\text{in.}$$

$$S_{TOP\,OF\,STEEL} = \frac{79{,}490}{31.15} = 2{,}552\,\text{in.}^3 \qquad S_{BOT\,OF\,STEEL} = \frac{79{,}490}{39.85} = 1{,}995\,\text{in.}^3$$

Table 17 Flange Transition: Composite Section Properties; 3n = 24

Component	A	d	Ad	Ad^2	I_o	I
Steel Section	76.81		−70.00			61,953
Concrete Slab 9" x 114"/ 24	42.75	42.50	1,817	77,217	288.6	77,506
	119.6		1,747			139,459
					−14.61(1,747) =	−25,524
					I_{NA} =	113,935 in.^{4}

$$d_{3n} = \frac{1{,}747}{119.6} = 14.61\,\text{in.}$$

$$d_{TOPOFSTEEL} = 35.50 - 14.61 = 20.89\,\text{in.} \qquad d_{BOTOFSTEEL} = 35.50 + 14.61 = 50.11\,\text{in.}$$

$$S_{TOP\,OF\,STEEL} = \frac{113{,}935}{20.89} = 5{,}454\,\text{in.}^3 \qquad S_{BOT\,OF\,STEEL} = \frac{113{,}935}{50.11} = 2{,}274\,\text{in.}^3$$

Table 18 Flange Transition: Composite Section Properties; n = 8

Component	A	d	Ad	Ad^2	I_o	I
Steel Section	76.81		−70.00			61,953
Concrete Slab 9" x 114"/ 8	128.3	42.50	5,433	231,742	865.7	232,608
	205.1		5,383			294,561
					−26.25(5,383) =	−144,304
					I_{NA} =	153,257 in.^{4}

$$d_n = \frac{5{,}383}{205.1} = 26.25\,\text{in.}$$

$$d_{TOPOFSTEEL} = 35.50 - 26.25 = 9.25\,\text{in.} \qquad d_{BOTOFSTEEL} = 35.50 + 26.25 = 61.75\,\text{in.}$$

$$S_{TOP\,OF\,STEEL} = \frac{153{,}257}{9.25} = 16{,}568\,\text{in.}^3 \qquad S_{BOT\,OF\,STEEL} = \frac{153{,}257}{61.75} = 2{,}482\,\text{in.}^3$$

Calculate F_{nc} using the smaller section at the transition:

$$r_t = \frac{20}{\sqrt{12\left(1+\frac{1}{3}\frac{(38.85)(0.5625)}{(20)(1)}\right)}} = 4.94\text{ in.}$$

$$L_p = \frac{1.0(4.94)}{12}\sqrt{\frac{29{,}000}{70}} = 8.38\text{ ft}$$

$$L_r = \frac{\pi(4.94)}{12}\sqrt{\frac{29{,}000}{49.0}} = 31.46\text{ ft}$$

Determine R_h:

$D_n = 38.85$ in.

$A_{fn} = 20(1) = 20.0$ in.^{2}

$f_n = 70.0$ ksi

$$\beta = \frac{2(38.85)(0.5625)}{20.0} = 2.185$$

$\rho = 50.0/70.0 = 0.714$

$$R_h = \frac{12 + 2.185\left[3(0.714) - (0.714)^3\right]}{12 + 2(2.185)} = 0.970$$

Determine R_b:

$$\frac{2D_c}{t_w} = \frac{2(38.85)}{0.5625} = 138.1 > \lambda_{rw} = 116.0$$

$$a_{wc} = \frac{2(38.85)(0.5625)}{20.0} = 2.185$$

$$R_b = 1 - \left(\frac{2.185}{1{,}200 + 300(2.185)}\right)(138.1 - 116.0) = 0.974$$

Since $L_p = 8.38$ feet $< L_b = 20.0$ feet $< L_r = 31.46$ feet,

$$F_{nc} = 1.0\left[1-\left(1-\frac{49.0}{0.970(70.0)}\right)\left(\frac{20.0-8.38}{31.46-8.38}\right)\right](0.974)(0.970)(70) = 56.87 \text{ ksi}$$

$$56.87 \text{ ksi} < 0.974(0.970)(70) = 66.13 \text{ ksi}$$

$\therefore$ F_{nc} = 56.87 ksi

Obviously, there is a significant discontinuity (reduction) in the predicted lateral torsional buckling resistance when a flange transition is moved beyond $0.2L_b$ from the brace point with the smaller moment, and the preceding approximate procedure is applied to determine the LTB resistance of the stepped flange. A more rigorous approximate solution for determining the LTB resistance for this unbraced length is presented for consideration in Appendix C (to this design example). However, the results from this procedure are not utilized in this example.

10.3.1.1.1.2. Local Buckling Resistance (Article 6.10.8.2.2)

Calculate the local buckling resistance of the bottom flange at Section 2-2. Determine the slenderness ratio of the flange:

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

$$\lambda_f = \frac{20}{2(2)} = 5.0$$

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

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

$$\lambda_{pf} = 0.38\sqrt{\frac{29{,}000}{70}} = 7.73$$

Since $\lambda_f < \lambda_{pf}$,

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

F_{nc} = (0.989)(0.984)(70.0) = 68.12 ksi

Calculate the local buckling resistance of the bottom flange in the smaller section at the flange transition. Determine the slenderness ratio of the flange:

$$\lambda_f = \frac{20}{2(1)} = 10.0$$

Since $\lambda_f > \lambda_{pf}$, determine the limiting slenderness ratio for a noncompact flange as follows:

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

$$F_{yr} = 0.7F_{yc} \le F_{yw}$$

$$F_{yr} = 0.7(70) = 49.0 \text{ ksi} < 50.0 \text{ ksi} \quad \text{ok}$$

F_{yr} must also not be less than $0.5F_{yc} = 0.5(70) = 35.0$ ksi ok

Therefore: $\lambda_{rf} = 0.56\sqrt{\frac{29{,}000}{49.0}} = 13.62$

And:

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

$$F_{nc} = \left[1 - \left(1 - \frac{49.0}{0.970(70.0)}\right)\left(\frac{10.0 - 7.73}{13.62 - 7.73}\right)\right](0.974)(0.970)(70.0) = 59.04 \text{ ksi}$$

At Section 2-2 and at the flange transition, F_{nc} is governed by the lateral torsional buckling resistance of 56.87 ksi, which is less than the local buckling resistance of 68.12 ksi at Section 2-2 and 59.04 ksi at the flange transition. Therefore, F_{nc} = 56.87 ksi at both locations.

10.3.1.1.2. Stress Check

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. Therefore, For STRENGTH I:

Section 2-2

Top flange: f = 53.87 ksi (computed earlier)
Bot. flange: f = -55.49 ksi (computed earlier)

Flange transition (Span 1)

$$\text{Top flange: } f = 1.0\left[\frac{1.25(-2{,}656)}{1{,}700} + \frac{1.25(-373)}{1{,}979} + \frac{1.5(-358)}{1{,}979} + \frac{1.75(-2{,}709)}{2{,}552}\right]12 = 51.81 \text{ ksi}$$

$$\text{Bot. flange: } f = 1.0\left[\frac{1.25(-2{,}656)}{1{,}789} + \frac{1.25(-373)}{1{,}870} + \frac{1.5(-358)}{1{,}870} + \frac{1.75(-2{,}709)}{1{,}995}\right]12 = -57.22 \text{ ksi}$$

$\therefore$ Bot. flange: f_{bu} = -57.22 ksi

For STRENGTH III:

Section 2-2

$$\text{Top flange: } f = 1.0\left[\frac{1.25(-4{,}840)}{2{,}942} + \frac{1.25(-690)}{3{,}228} + \frac{1.5(-664)}{3{,}228}\right]12 = 31.59 \text{ ksi}$$

$$\text{Bot. flange: } f = 1.0\left[\frac{1.25(-4{,}840)}{3{,}149} + \frac{1.25(-690)}{3{,}216} + \frac{1.5(-664)}{3{,}216}\right]12 = -29.99 \text{ ksi}$$

Flange transition (Span 2)

$$\text{Top flange: } f = 1.0\left[\frac{1.25(-2{,}718)}{1{,}700} + \frac{1.25(-378)}{1{,}979} + \frac{1.5(-364)}{1{,}979}\right]12 = 30.16 \text{ ksi}$$

$$\text{Bot. flange: } f = 1.0\left[\frac{1.25(-2{,}718)}{1{,}789} + \frac{1.25(-378)}{1{,}870} + \frac{1.5(-364)}{1{,}870}\right]12 = -29.33 \text{ ksi}$$

$\therefore$ Bot. flange: f_{bu} = -29.99 ksi

For STRENGTH IV:

Section 2-2

$$\text{Top flange: } f = 1.0\left[\frac{1.5(-4{,}840)}{2{,}942} + \frac{1.5(-690)}{3{,}228} + \frac{1.5(-664)}{3{,}228}\right]12 = 37.16 \text{ ksi}$$

$$\text{Bot. flange: } f = 1.0\left[\frac{1.5(-4{,}840)}{3{,}149} + \frac{1.5(-690)}{3{,}216} + \frac{1.5(-664)}{3{,}216}\right]12 = -35.24 \text{ ksi}$$

<u>Flange transition (Span 2)</u>

$$\text{Top flange: } f = 1.0\left[\frac{1.5(-2{,}718)}{1{,}700} + \frac{1.5(-378)}{1{,}979} + \frac{1.5(-364)}{1{,}979}\right]12 = 35.53 \text{ ksi}$$

$$\text{Bot. flange: } f = 1.0\left[\frac{1.5(-2{,}718)}{1{,}789} + \frac{1.5(-378)}{1{,}870} + \frac{1.5(-364)}{1{,}870}\right]12 = -34.49 \text{ ksi}$$

$\therefore$ Bot. flange: f_{bu} = -35.24 ksi

For STRENGTH V:

<u>Section 2-2</u>

$$\text{Top flange: } f = 1.0\left[\frac{1.25(-4{,}840)}{2{,}942} + \frac{1.25(-690)}{3{,}228} + \frac{1.5(-664)}{3{,}228} + \frac{1.35(-4{,}040)}{3{,}808}\right]12 = 48.77 \text{ ksi}$$

$$\text{Bot. flange: } f = 1.0\left[\frac{1.25(-4{,}840)}{3{,}149} + \frac{1.25(-690)}{3{,}216} + \frac{1.5(-664)}{3{,}216} + \frac{1.35(-4{,}040)}{3{,}327}\right]12 = -49.66 \text{ ksi}$$

<u>Flange transition (Span 1)</u>

$$\text{Top flange: } f = 1.0\left[\frac{1.25(-2{,}656)}{1{,}700} + \frac{1.25(-373)}{1{,}979} + \frac{1.5(-358)}{1{,}979} + \frac{1.35(-2{,}709)}{2{,}552}\right]12 = 46.72 \text{ ksi}$$

$$\text{Bot. flange: } f = 1.0\left[\frac{1.25(-2{,}656)}{1{,}789} + \frac{1.25(-373)}{1{,}870} + \frac{1.5(-358)}{1{,}870} + \frac{1.35(-2{,}709)}{1{,}995}\right]12 = -50.71 \text{ ksi}$$

$\therefore$ Bot. flange: f_{bu} = -50.71 ksi

In this example, lateral bending in the bottom flange due to wind-load effects is considered at the strength limit state. 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. Eqs. C4.6.2.7.1-1 and C4.6.2.7.1-2, presented earlier, are again used to compute the factored wind force per unit length, W, applied to the bottom flange, and the maximum flange lateral bending moment due to the factored wind load, M_w, within the unbraced length, respectively. Again, the wind load acting on the live load (WL) is assumed transmitted directly to the deck and is therefore not considered in the STRENGTH V load combination in this example. The overturning effect of WL on the wheel loads is also not considered.

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:

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

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, f_{bu} = -50.71 ksi, as computed earlier for the STRENGTH V load combination (which is the controlling load case with wind included for this particular computation). Therefore:

$$1.2(8.38)\sqrt{\frac{1.0(0.974)}{|-50.71|/70}} = 11.66\text{ ft} < L_b = 20.0\text{ ft}$$

Because the preceding equation 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 (assuming an elastic effective length factor for lateral torsional buckling equal to 1.0, which should not be modified since the flange is stepped within this unbraced length):

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

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 the smallest resistance within the unbraced length as:

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

$$F_{cr} = \frac{1.0(0.974)(\pi^2)(29{,}000)}{\left(\dfrac{20.0(12)}{4.94}\right)^2} = 118.1\ \text{ksi}$$

Note again that the calculated value of F_{cr} for use in Eq. 6.10.1.6-4 is not limited to $R_b R_h F_{yc}$.

The amplification factor is then determined as follows:

For STRENGTH III:

$$AF = \frac{0.85}{\left(1 - \frac{|-29.99|}{118.1}\right)} = 1.14 > 1.0 \quad \text{ok}$$

For STRENGTH V:

$$AF = \frac{0.85}{\left(1 - \frac{|-50.71|}{118.1}\right)} = 1.49 > 1.0 \quad \text{ok}$$

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. The largest section within the unbraced length will be conservatively used to compute W, and the smallest bottom flange will conservatively be used to compute f_{ℓ}. Therefore,

For STRENGTH I:

Wind loads: Not considered

For STRENGTH III:

Wind loads: $W = \frac{1.0(1.4)(0.053)(2.0 + 69.0 + 2.0)/12}{2} = 0.226 \text{ kips/ft}$

$$M_w = \frac{0.226(20.0)^2}{10} = 9.04 \text{ kip} - \text{ft}$$

$$f_{\ell} = \frac{M_w}{S_{\ell}} = \frac{9.04(12)}{1.0(20)^2/6} = 1.63 \text{ ksi} * AF = 1.63(1.14) = 1.86 \text{ ksi} < 0.6F_{yf} = 42.0 \text{ ksi} \quad \text{ok}$$

For STRENGTH IV:

Wind loads: Not considered

For STRENGTH V:

Wind loads: $W = \frac{1.0(0.4)(0.053)(2.0 + 69.0 + 2.0)/12}{2} = 0.064 \text{ kips/ft}$

$$M_w = \frac{0.064(20.0)^2}{10} = 2.56 \ \text{kip} - \text{ft}$$

$$f_\ell = \frac{M_w}{S_\ell} = \frac{2.56(12)}{1.0(20)^2/6} = 0.46 \ \text{ksi} * \text{AF} = 0.46(1.49) = 0.69 \ \text{ksi} < 0.6F_{yf} = 42.0 \ \text{ksi} \quad \text{ok}$$

Now that all the required information has been assembled, check Eqs. 6.10.8.1.1-1 and 6.10.8.1.3-1, as applicable:

10.3.1.1.2.1. Bottom Flange

$$f_{bu} + \frac{1}{3} f_\ell \le \phi_f F_{nc} \qquad \text{Eq. (6.10.8.1.1-1)}$$

For STRENGTH I:

$$f_{bu} + \frac{1}{3} f_\ell = |-57.22| \ \text{ksi} + 0 = 57.22 \ \text{ksi}$$

$\phi_f F_{nc}$ = 1.0(56.87) = 56.87 ksi

57.22 ksi > 56.87 ksi say ok (Ratio = 1.006)

For STRENGTH III:

$$f_{bu} + \frac{1}{3} f_\ell = |-29.99| \ \text{ksi} + \frac{1}{3}(1.86) = 30.61 \ \text{ksi}$$

$\phi_f F_{nc}$ = 1.0(56.87) = 56.87 ksi

30.61 ksi < 56.87 ksi ok (Ratio = 0.538)

For STRENGTH IV:

$$f_{bu} + \frac{1}{3} f_\ell = |-35.24| \ \text{ksi} + 0 = 35.24 \ \text{ksi}$$

$\phi_f F_{nc}$ = 1.0(56.87) = 56.87 ksi

35.24 ksi < 56.87 ksi ok (Ratio = 0.620)

For STRENGTH V:

$$f_{bu} + \frac{1}{3} f_{\ell} = \left| -50.71 \right| \text{ ksi} + \frac{1}{3}(0.69) = 50.94 \text{ ksi}$$

$\phi_f F_{nc} = 1.0(56.87) = 56.87$ ksi

50.94 ksi < 56.87 ksi ok (Ratio = 0.896)

10.3.1.1.2.2. Top Flange

$$f_{bu} \le \phi_f R_h F_{yf} \qquad \text{Eq. (6.10.8.1.3-1)}$$

For STRENGTH I:

Section 2-2: $f_{bu} = 53.87$ ksi

$\phi_f R_h F_{yf} = 1.0(0.984)(70.0) = 68.88$ ksi

53.87 ksi < 68.88 ksi ok (Ratio = 0.782)

Flange transition: $f_{bu} = 51.81$ ksi

$\phi_f R_h F_{yf} = 1.0(0.970)(70.0) = 67.90$ ksi

51.81 ksi < 67.90 ksi ok (Ratio = 0.763)

For STRENGTH III:

Section 2-2: $f_{bu} = 31.59$ ksi

$\phi_f R_h F_{yf} = 1.0(0.984)(70.0) = 68.88$ ksi

31.59 ksi < 68.88 ksi ok (Ratio = 0.459)

Flange transition: $f_{bu} = 30.16$ ksi

$\phi_f R_h F_{yf} = 1.0(0.970)(70.0) = 67.90$ ksi

30.16 ksi < 67.90 ksi ok (Ratio = 0.444)

For STRENGTH IV:

Section 2-2: $f_{bu} = 37.16$ ksi

$\phi_f R_h F_{yf} = 1.0(0.984)(70.0) = 68.88$ ksi

$37.16 \text{ ksi} < 68.88 \text{ ksi}$ ok (Ratio = 0.539)

Flange transition: $f_{bu} = 35.53$ ksi

$\phi_f R_h F_{yf} = 1.0(0.970)(70.0) = 67.90$ ksi

$35.53 \text{ ksi} < 67.90 \text{ ksi}$ ok (Ratio = 0.523)

For STRENGTH V:

Section 2-2: $f_{bu} = 48.77$ ksi

$\phi_f R_h F_{yf} = 1.0(0.984)(70.0) = 68.88$ ksi

$48.77 \text{ ksi} < 68.88 \text{ ksi}$ ok (Ratio = 0.708)

Flange transition: $f_{bu} = 46.72$ ksi

$\phi_f R_h F_{yf} = 1.0(0.970)(70.0) = 67.90$ ksi

$46.72 \text{ ksi} < 67.90 \text{ ksi}$ ok (Ratio = 0.688)

Finally, it should be noted that for continuous span flexural members that satisfy the requirements of Article B6.2 to ensure adequate robustness and ductility of the pier sections, a calculated percentage of the negative moment due to the factored loads at the pier section under consideration may be redistributed prior to making the preceding checks (Article 6.10.6.2.3). The moments may be redistributed using the optional procedures of Appendix B (to Section 6 of *AASHTO LRFD (5th Edition, 2010)* – specifically, Articles B6.4 or B6.6). When the redistribution moments are calculated according to these procedures, the flexural resistances at the strength limit state within the unbraced lengths immediately adjacent to interior-pier sections satisfying the requirements of Article B6.2 need not be checked. At all other locations, the provisions of Articles 6.10.7, 6.10.8.1 or A6.1, as applicable, must be satisfied after redistribution.

10.3.1.2. Shear (6.10.6.3)

Article 6.10.6.3 refers to the provisions of Article 6.10.9 to determine the nominal flexural resistance at the strength limit state.

Separate calculations similar to those shown previously for the interior panels in Field Section 1 are used to determine the spacing of the transverse stiffeners in the interior panels of Field Section 2, 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 for practical reasons in order to help simplify the detailing.

10.3.2. Service Limit State (Article 6.10.4)

10.3.2.1. Permanent Deformations (Article 6.10.4.2)

Article 6.10.4.2 contains criteria intended to control objectionable permanent deformations due to expected severe traffic loadings 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 specified in Table 3.4.1-1. These criteria were discussed previously under the service limit state checks for Section 1-1.

For members with shear connectors provided throughout their entire length that also satisfy the provisions of Article 6.10.1.7, Article 6.10.4.2.1 permits the concrete deck to also be considered effective for negative flexure when computing flexural stresses acting on the composite section at the service limit state. Earlier calculations were made to ensure that the minimum longitudinal reinforcement satisfied the provisions of Article 6.10.1.7 for both the factored construction loads and the SERVICE II loads. Therefore, the SERVICE II flexural stresses will be computed assuming the concrete deck to be effective for loads applied to the composite section.
Determine R_h:

Section 2-2: $D_n = 56.35$ in. (conservatively use the short-term composite section)

$A_{fn} = 20(2) = 40.0 \text{ in.}^2$

$f_n = 70.0$ ksi

$$\beta = \frac{2(56.35)(0.5625)}{20.0} = 1.585$$

$\rho = 50.0/70.0 = 0.714$

$$R_h = \frac{12 + 1.585\left[3(0.714) - (0.714)^3\right]}{12 + 2(1.585)} = 0.977$$

Flange transition: $D_n = 60.75$ in. (conservatively use the short-term composite section)

$A_{fn} = 20(1) = 20.0 \text{ in.}^2$

$f_n = 70.0$ ksi

$$\beta = \frac{2(60.75)(0.5625)}{20.0} = 3.417$$

$\rho = 50.0/70.0 = 0.714$

$$R_h = \frac{12 + 3.417\left[3(0.714) - (0.714)^3\right]}{12 + 2(3.417)} = 0.960$$

Check the flange stresses due to the SERVICE II loads at Section 2-2 and at the flange transition within the unbraced length in Span 1 adjacent to Section 2-2. η is specified to always equal 1.0 at the service limit state (Article 1.3). For the example bridge, f_ℓ is taken equal to zero at the service limit state:

For SERVICE II:

Section 2-2

Top flange: $$f_f = 1.0\left[\frac{1.0(-4{,}840)}{2{,}942} + \frac{1.0(-690 + -664)}{6{,}606} + \frac{1.3(-4{,}040)}{15{,}547}\right]12 = 26.26 \text{ ksi}$$

Bot. flange: $$f_f = 1.0\left[\frac{1.0(-4{,}840)}{3{,}149} + \frac{1.0(-690 + -664)}{3{,}626} + \frac{1.3(-4{,}040)}{3{,}903}\right]12 = -39.07 \text{ ksi}$$

Flange transition

Top flange: $$f_f = 1.0\left[\frac{1.0(-2{,}656)}{1{,}700} + \frac{1.0(-373 + -358)}{5{,}454} + \frac{1.3(-2{,}709)}{16{,}568}\right]12 = 22.91 \text{ ksi}$$

Bot. flange: $$f_f = 1.0\left[\frac{1.0(-2{,}656)}{1{,}700} + \frac{1.0(-373 + -358)}{2{,}274} + \frac{1.3(-2{,}709)}{2{,}482}\right]12 = -38.70 \text{ ksi}$$

Bottom Flange

$$f_f + \frac{f_\ell}{2} \le 0.95 R_h F_{yf} \qquad \text{Eq. (6.10.4.2.2-2)}$$

Section 2-2: $0.95R_hF_{yf}$ = 0.95(0.977)(70.0) = 64.97 ksi

$|-39.07|\text{ksi} + 0 < 64.97$ ksi ok (Ratio = 0.601)

Flange Transition: $0.95R_hF_{yf}$ = 0.95(0.960)(70.0) = 63.84 ksi

$|-38.70|\text{ksi} + 0 < 63.84$ ksi ok (Ratio = 0.606)

Top Flange

$$f_f \le 0.95 R_h F_{yf} \qquad \text{Eq. (6.10.4.2.2-1)}$$

Section 2-2: $0.95R_hF_{yf}$ = 0.95(0.977)(70.0) = 64.97 ksi

26.26 ksi < 64.97 ok (Ratio = 0.415)

Flange Transition: $0.95R_hF_{yf} = 0.95(0.960)(70.0) = 63.84$ ksi

$22.91 \text{ ksi} < 63.84 \text{ ksi}$ ok (Ratio = 0.359)

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 do not control and need not be checked for composite sections in negative flexure for which the nominal flexural resistance at the strength limit state is determined according to the provisions of Article 6.10.8 (see Article C6.10.4.2.2). Nevertheless, the checks are illustrated above for the sake of completeness.

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 according to Eq. 6.10.4.2.2-4 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 bend-buckling resistance for webs determined as specified in Article 6.10.1.9.

Determine the nominal web bend-buckling resistance at Section 2-2 and at the flange transition within the unbraced length in Span 1 adjacent to Section 2-2 according to the provisions of Article 6.10.1.9.1 as follows:

$$F_{crw} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} \qquad \text{Eq. (6.10.1.9.1-1)}$$

but not to exceed the smaller of R_hF_{yc} and $F_{yw}/0.7$,

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

According to Article D6.3.1 (Appendix D to Section 6 of *AASHTO LRFD (5th Edition, 2010)*), 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, D_c, is to be computed from Eq. D6.3.1-1 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 is the sum of the various tension-flange stresses caused by the factored loads, calculated without considering flange lateral bending, and d is the depth of the steel section. 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. Therefore,

Section 2-2: $$D_c = \left(\frac{-(-39.07)}{|-39.07| + 26.26} \right) 73.0 - 2.0 = 41.66 \text{ in.} > 0 \quad \text{ok}$$

$$k = \frac{9}{(41.66/69.0)^2} = 24.7$$

$$F_{crw} = \frac{0.9(29{,}000)(24.7)}{\left(\frac{69.0}{0.5625}\right)^2} = 42.85 \text{ ksi} < \min\left(R_h F_{yc},\ F_{yw}/0.7\right) = R_h F_{yc} = 0.977(70.0) = 68.39 \text{ ksi ok}$$

$$|-39.07| \text{ ksi} < 42.84 \text{ ksi} \quad \text{ok} \quad (\text{Ratio} = 0.912)$$

Flange transition:

$$D_c = \left(\frac{-(-38.70)}{|-38.70| + 22.91} \right) 71.0 - 1.0 = 43.60 \text{ in.} > 0 \quad \text{ok}$$

$$k = \frac{9}{(43.60/69.0)^2} = 22.5$$

$$F_{crw} = \frac{0.9(29{,}000)(22.5)}{\left(\frac{69.0}{0.5625}\right)^2} = 39.03 \text{ ksi} < \min\left(R_h F_{yc},\ F_{yw}/0.7\right) = R_h F_{yc} = 0.960(70.0) = 67.20 \text{ ksi ok}$$

$$|-38.70| \text{ ksi} < 39.03 \text{ ksi} \quad \text{ok} \quad (\text{Ratio} = 0.992)$$

10.3.3. Fatigue And Fracture Limit State (Article 6.10.5)

10.3.3.1. Load Induced Fatigue (Article 6.6.1.2)

Fatigue of the base metal at the connection-plate weld to the top (tension) flange at the intermediate cross-frame in Span 1, located 20.0 feet to the left of Section 2-2, will be checked for the fatigue limit state. 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 earlier, are also used.
From earlier computations, the $(ADTT)_{SL}$ was calculated to be 1,600 trucks/day. The provisions of Article 6.6.1.2 apply only to details subject to a net applied tensile stress, which by inspection is the case at this particular detail.

Determine the fatigue detail category from Table 6.6.1.2.3-1.

Under the condition of fillet-welded connections with welds normal to the direction of stress, the fatigue detail category for base metal at the toe of transverse stiffener-to-flange welds is Category C′.

According to Article 6.6.1.2.3, since the projected 75-year $(ADTT)_{SL}$ of 1,600 trucks per day exceeds the value of 754 trucks per day specified in Table 6.6.1.2.3-2, the detail should be designed for an infinite life using the FATIGUE I load combination.

As stated previously, the concrete deck is assumed effective in computing all stresses and stress ranges applied to the composite section in the fatigue calculations. Thus, the stress range $\gamma(\Delta f)$ at the connection-plate weld to the top flange due to the factored fatigue load (factored by the 1.50 load factor specified for the FATIGUE I load combination) at the cross-frame under consideration is computed using the properties of the short-term composite section as:

$$\gamma(\Delta f) = \frac{1.50(342)(8.25)(12)}{153{,}257} + \frac{1.50|-826|(825)(12)}{153{,}257} = 1.132\ \text{ksi}$$

According to Eq. 6.6.1.2.2-1, $\gamma(\Delta f)$ must not exceed the nominal fatigue resistance $(\Delta F)_n$. 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).

For a Category C′ detail, $(DF)_{TH}$ = 12.0 ksi (Table 6.6.1.2.5-3). For the FATIGUE I load combination and infinite life, the nominal fatigue resistance is:

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

Therefore:

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

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

1.132 ksi < 12.0 ksi ok (Ratio = 0.094)

10.3.3.2. Special Fatigue Requirement for Webs (Article 6.10.5.3)

As discussed previously, interior panels of stiffened webs must satisfy Eq. 6.10.5.3-1 in order to control elastic flexing of the web so that the member is assumed able to sustain an infinite number of smaller loadings without fatigue cracking due to this effect.

$$V_u \le V_{cr} \qquad \text{Eq. (6.10.5.3-1)}$$

where: V_u = shear in the web at the section under consideration due to the unfactored permanent loads plus the factored fatigue load

V_{cr} = shear buckling resistance determined from Eq. 6.10.9.3.3-1

In this check, the factored fatigue load is to be determined using the FATIGUE I load combination (Table 3.4.1-1), with the fatigue live load taken as specified in Article 3.6.1.4. Again, the fatigue live load is modified by the dynamic load allowance of 15 percent and the lateral distribution factors for the fatigue limit state are used.

In this example, the panel adjacent to Section 2-2 will be checked. The transverse stiffener spacing in this panel is d_o = 10.0 feet (Figure 3). The shear at Section 2-2 to be used in this check is computed as follows:

$$V_u = -159 + -23 + -22 + 1.50(-56) = -288 \text{ kips}$$

The shear buckling resistance of the 120-inch-long panel is 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 from Eq. 6.10.9.3.2-4, 6.10.9.3.2-5 or 6.10.9.3.2-6, as applicable. First, compute the shear buckling coefficient, k:

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

$$k = 5 + \frac{5}{\left(\frac{120.0}{69.0}\right)^2} = 6.65$$

Since, $$1.40\sqrt{\frac{Ek}{F_{yw}}} = 1.40\sqrt{\frac{29{,}000(6.65)}{50}} = 86.9 < \frac{D}{t_w} = \frac{69.0}{0.5625} = 122.7$$

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

$$C = \frac{1.57}{(122.7)^2}\left(\frac{29{,}000(6.65)}{50}\right) = 0.402$$

V_p is the plastic shear force determined as follows:

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

$$V_p = 0.58(50)(69.0)(0.5625) = 1{,}126 \text{ kips}$$

Therefore, $V_{cr} = 0.402(1{,}126) = 453 \text{ kips} > V_u = \left|-288\right| \text{ kips}$ ok (Ratio = 0.636)

10.3.4. Constructibility (Article 6.10.3)

10.3.4.1. Flexure (Article 6.10.3.2)

In regions of negative flexure, Eqs. 6.10.3.2.1-1, 6.10.3.2.1-2 and 6.10.3.2.2-1 specified in Article 6.10.3.2, to be checked for critical stages of construction, generally do not control because the sizes of the flanges in these regions are normally governed by the sum of the factored dead and live load stresses at the strength limit state. Also, the maximum accumulated negative moments from the deck-placement analysis in these regions, plus the negative moments due to the steel weight, typically do not differ significantly from the calculated DC_1 negative moments. The deck-overhang loads do introduce lateral bending stresses into the flanges in these regions, which can be calculated and used to check the above equations in a manner similar to that illustrated previously for Section 1-1. Wind load, when considered for the construction case, also introduces lateral bending into the flanges.

When applying Eqs. 6.10.3.2.1-1, 6.10.3.2.1-2 and 6.10.3.2.2-1 in these regions, the bottom flange would be the discretely braced compression flange and the top flange would be the discretely braced tension flange for all constructibility checks to be made before the concrete deck has hardened or is made composite. The nominal flexural resistance of the bottom flange, F_{nc}, would be calculated in a manner similar to that demonstrated above for Section 2-2 at the strength limit state. However, for loads applied before the deck has hardened or is made composite, F_{nc} would be computed ignoring any contribution from the longitudinal reinforcement. For the sake of brevity in this example, the application of Eqs. 6.10.3.2.1-1, 6.10.3.2.1-2 and 6.10.3.2.2-1 to the construction case for the unbraced lengths adjacent to Section 2-2 will not be shown.

10.3.4.1.1. Web Bend-Buckling

For critical stages of construction, web bend-buckling should always be checked in regions of negative flexure according to Eq. 6.10.3.2.1-3 as follows:

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

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

In this example, check Eq. 6.10.3.2.1-3 for the noncomposite section at Section 2-2 and at the flange transition within the unbraced length in Span 2 adjacent to Section 2-2. By inspection, the STRENGTH IV load combination governs this check. The sum of the accumulated unfactored negative moments during the deck casts plus the unfactored moment due to the steel weight is –4,918 kip-feet (versus the unfactored DC_1 moment of –4,840 kip-feet) at Section 2-2, and –2,796 kip-feet (versus the unfactored DC_1 moment of –2,718 kip-feet) at the flange transition (Table 4).

For STRENGTH IV:

<u>Section 2-2</u>

Bot. flange: $$f_{bu} = 1.0\left[\frac{1.5(-4{,}918)}{3{,}149}\right]12 = -28.11\ \text{ksi}$$

<u>Flange transition (Span 2)</u>

Bot. flange: $$f_{bu} = 1.0\left[\frac{1.5(-2{,}796)}{1{,}789}\right]12 = -28.13\ \text{ksi}$$

Table 19 Moments from Deck-Placement Analysis

Unfactored Dead-Load Moments (kip-ft)										
Span -> 1			Pier		Span -> 2					
Length (ft)	120.0	125.0	140.0	0.0	15.0	20.0	42.0	47.0	74.0	87.5
Steel Weight	-312	-414	-777	-777	**-428**	-331	9	67	264	288
SIP Forms (SIP)	-138	-182	-329	-329	-185	-143	5	31	118	129
Cast										
1	172	-103	-930	-930	-930	-930	-930	-930	-930	-930
2	-1684	-1754	-1965	-1965	-1038	-729	629	921	1911	2035
3	-127	-273	-917	-917	-215	-48	279	279	279	279
Sum of Casts + SIP	-1777	-2312	**-4141**	**-4141**	**-2368**	-1850	-17	301	1378	1513
Max. +M	34	0	0	0	0	0	5	301	1378	1513
DC_2 + DW	-549	-731	-1354	-1354	-742	-564	69	179	551	597
Deck, haunches + SIP	-1709	-2242	-4063	-4063	-2290	-1772	64	382	1459	1594

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

$$F_{crw} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} \qquad \text{Eq. (6.10.1.9.1-1)}$$

but not to exceed the smaller of R_hF_{yc} and $F_{yw}/0.7$,

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

At Section 2-2, D_c for the steel section is equal to 33.26 inches. At the flange transition, D_c for the steel section is equal to 33.59 inches. From separate calculations, R_h for the steel section is equal to 0.983 at Section 2-2 and 0.970 at the flange transition. Therefore,

Section 2-2: $$k = \frac{9}{(33.26/69.0)^2} = 38.7$$

$$F_{crw} = \frac{0.9(29{,}000)(38.7)}{\left(\frac{69.0}{0.5625}\right)^2} = 67.13\text{ ksi} < \min\left(R_hF_{yc}, F_{yw}/0.7\right) = R_hF_{yc}0.983(70) = 68.81\text{ ksi ok}$$

$$\phi_f F_{crw} = 1.0(67.13) = 67.13\text{ ksi}$$

$$|-28.11| < 67.13\text{ ksi} \quad \text{ok} \qquad (\text{Ratio} = 0.419)$$

Flange transition:

$$k = \frac{9}{(33.59/69.0)^2} = 38.0$$

$$F_{crw} = \frac{0.9(29{,}000)(38.0)}{\left(\frac{69.0}{0.5625}\right)^2} = 65.91\text{ ksi} < \min\left(R_hF_{yc}, F_{yw}/0.7\right) = R_hF_{yc} = 0.970(70) = 67.90\text{ ksi ok}$$

$$\phi_f F_{crw} = 1.0(65.91) = 65.91\text{ ksi}$$

$$|-28.13| < 65.91\text{ ksi} \quad \text{ok} \qquad (\text{Ratio} = 0.427)$$

10.3.4.2. Shear (Article 6.10.3.3)

For critical stages of construction, Article 6.10.3.3 requires that interior panels of stiffened webs satisfy the following requirement:

$$V_u \le \phi_v V_{cr}$$ 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 2-2 will be checked. The transverse stiffener spacing in this panel is d_o = 10.0 feet (Figure 3). Since shear is rarely increased significantly due to deck staging, the factored DC_1 shear at Section 2-2 will be used in this check (the STRENGTH IV load combination governs by inspection):

$$\left(V_u\right)_{DC_1} = 1.0(1.5)(-159) = -239 \text{ kips}$$

The shear buckling resistance of this 120-inch panel was previously determined to be V_{cr} = 453 kips. Therefore,

$$\phi_v V_{cr} = 1.0(453) = 453 \text{ kips}$$

$$\left|-239\right| \text{kips} < 453 \text{ kips} \quad \text{ok} \qquad (\text{Ratio} = 0.528)$$

10.4. Shear Connector Design (Article 6.10.10)

Shear connectors are designed according to the provisions of Article 6.10.10. According to Article 6.10.10.1, continuous composite bridges should normally be provided with shear connectors throughout the entire length of the bridge. In regions of negative flexure, shear connectors must be provided where the longitudinal reinforcement is considered to be a part of the composite section. Both stud and channel shear connectors are permitted in Article 6.10.10.1.1. Stud shear connectors will be utilized in this example.

10.4.1. Stud Proportions

Terminating the studs at approximately the mid-thickness of the concrete deck will place them well within the limits for cover and penetration specified in Article 6.10.10.1.4 and will also clear the reinforcing steel. Therefore,

$$\frac{9.0}{2} + (3.5 - 0.875) = 7.125 \text{ in.}$$

Use 7/8″ x 7″ studs. Check that the ratio of the height to the diameter is not less than 4.0, as required in Article 6.10.10.1.1.

$$\frac{h}{d} = \frac{7.0}{0.875} = 8.0 > 4.0 \qquad \text{ok}$$

10.4.2. Pitch (Article 6.10.10.1.2)

According to the provisions of Article 6.10.10.1.2, the pitch of the shear connectors along the longitudinal axis of the girder is to be initially determined to satisfy the fatigue limit state. The resulting number of shear connectors is then to be checked against the number required to satisfy the strength limit state. For the purpose of this design example, the pitch is determined at the interior pier section (Section 2-2). The pitch at other locations can be determined in a similar manner.

10.4.3. Fatigue Limit State

As specified in Article 6.10.10.1.2, the pitch, p, of the shear connectors must satisfy the following:

$$p \le \frac{nZ_r}{V_{sr}} \qquad \text{Eq. (6.10.10.1.2-1)}$$

where: n = number of shear connectors in a cross-section
Z_r = shear fatigue resistance of an individual shear connector determined as specified in Article 6.10.10.2
V_{sr} = horizontal fatigue shear range per unit length

V_{sr} is to be computed as follows:

$$V_{sr} = \sqrt{(V_{fat})^2 + (F_{fat})^2} \qquad \text{Eq. (6.10.10.1.2-2)}$$

where: V_{fat} = longitudinal fatigue shear range per unit length
F_{fat} = radial fatigue shear range per unit length

The longitudinal fatigue shear range is computed as follows:

$$V_{fat} = \frac{V_f Q}{I} \qquad \text{Eq. (6.10.10.1.2-3)}$$

where: V_f = vertical shear force range under the applicable fatigue load combination specified in Table 3.4.1-1 with the fatigue live load taken as specified in Article 3.6.1.4
Q = first moment of the transformed short-term area of the concrete deck about the neutral axis of the short-term composite section
I = moment of inertia of the short-term composite section

The parameters I and Q should be determined using the deck within the effective flange width. Article C6.10.10.1.2 does permit I and Q in regions of negative flexure to be determined using the longitudinal reinforcement within the effective flange width, unless the concrete deck is considered to be effective in tension for negative flexure in calculating the range of longitudinal stress, as permitted in Article 6.6.1.2.1. Since the minimum required one-percent longitudinal reinforcement is provided in the deck according to the provisions of Article 6.10.1.7, the concrete deck is considered to be effective in tension for negative flexure when computing longitudinal stress ranges in this example. Therefore, I and Q must be determined using the short-term area of the concrete deck along the entire girder.

From earlier calculations, the 75-year $(ADDTT)_{SL}$ was calculated to be 1,600 trucks per day. According to Article 6.10.10.2, where the projected 75-year $(ADDT)_{SL}$ is greater than or equal to 960 trucks per day, the FATIGUE I load combination shall be used and the fatigue shear resistance of an individual stud shear connector for infinite life shall be taken as:

$$Z_r = 5.5d^2 \qquad \text{Eq. (6.10.10.2-1)}$$

where: d = diameter of the stud

As stated earlier, the shear connectors are 7/8″ diameter x 7″. The number of shear connectors in a cross-section, n, will be assumed to equal three (3). Requirements for the transverse spacing of shear connectors across the top flange are given in Article 6.10.10.1.3. The fatigue resistance of one shear connector is computed as follows:

$$Z_r = 5.5(0.875)^2 = 4.211 \text{ kips}$$

The fatigue resistance for 3 shear connectors is:

$$nZ_r = 3(4.211) = 12.633 \text{ kips/row}$$

In order to compute the horizontal fatigue shear range, V_{sr}, the longitudinal and radial fatigue shear ranges must be determined. In order to compute the longitudinal fatigue shear range, V_{fat}, first compute the vertical shear force range, V_f, for the FATIGUE I load combination as follows:

$$V_f = 1.5[4 + |-56|] = 90 \text{ kips}$$

The terms I and Q are also needed to compute the longitudinal fatigue shear range, V_{fat}. As stated earlier, I and Q must be determined using the short-term area of the concrete deck. The structural deck thickness, t_s, is 9.0 inches; the modular ratio, n, equals 8.0; and the effective flange width is 114 inches (calculated previously).

Compute the transformed deck area as follows:

$$\text{Transformed deck area} = \frac{\text{Area}}{n} = \frac{(114)(9)}{8.0} = 128.3 \text{ in.}^2$$

Compute the first moment of the transformed short-term area of the concrete deck, Q, with respect to the neutral axis of the uncracked live load short-term composite section. Determine the distance from the center of the deck to the neutral axis. Section properties are taken from

Table 10. The neutral axis of the short-term composite section is 14.65 in. measured from the top of the top flange.

$$\text{Moment arm of the deck} = \text{Neutral axis} - t_{flg} + \text{haunch} + t_s/2$$

$$\text{Moment arm of the deck} = 14.65 - 2 + 3.5 + \frac{9}{2} = 20.65 \text{ in.}$$

$$Q = 128.3(20.65) = 2{,}649 \text{ in}^3$$

Compute the longitudinal fatigue shear range per unit length, V_{fat}:

$$V_{fat} = \frac{V_f Q}{I} = \frac{90(2{,}649)}{227{,}766} = 1.05 \text{ k/in. (factored)}$$

It is also necessary to compute F_{fat}, the radial fatigue shear range per unit length. Article 6.10.10.1.2 directs the designer to compute F_{fat} by taking the larger of two computed values from Eqs. 6.10.10.1.2-4 and 6.10.10.1.2-5. The first equation is an approximation based on the stress in the flange and the radius of curvature, which may be taken equal to zero for straight spans per Article 6.10.10.1.2. The second equation is a more exact calculation based on the actual cross frame force from the analysis. As permitted in Article 6.10.10.1.2, for straight or horizontally curved bridges with skew not exceeding 20 degrees, the radial fatigue shear range from Eq. 6.10.10.1.2-5 may be taken equal to zero. Therefore, in this case, $F_{fat} = F_{fat1} = F_{fat2} = 0$.

The positive and negative longitudinal shears due to major-axis bending are due to the fatigue vehicle located in Span 1 with the back axle on the left and then on the right of the point under consideration. This means that the truck actually has to turn around to produce the computed longitudinal shear range. This is not a realistic loading case but has been assumed to be practical and to be conservative. Combining the longitudinal and radial fatigue shear ranges vectorially, the total horizontal fatigue shear range per unit length is computed as follows:

$$V_{sr} = \sqrt{(V_{fat})^2 + (F_{fat})^2} \qquad \text{Eq. (6.10.10.1.2-2)}$$

$$V_{sr} = \sqrt{(1.05)^2 + (0)^2} = 1.95 \text{ kips/in.}$$

Compute the required shear connector pitch for fatigue for 3 studs per row.

$$p \le \frac{nZ_r}{V_{sr}} \qquad \text{Eq. (6.10.10.1.2-1)}$$

$$p \le \frac{12.633}{1.05} = 12.0 \text{ in./row}$$

As specified in Article 6.10.10.1.2, the pitch must not be less than six stud diameters = 6(0.875) = 5.25 inches nor more than 24.0 inches. The pitch computed above is satisfactory for fatigue at this location. The pitch at other locations can be determined in a similar manner.

10.4.4. Strength Limit State (Article 6.10.10.4)

The resulting number of shear connectors will now be checked against the number required to satisfy the strength limit state. According to Article 6.10.10.4.1, the factored shear resistance of a single shear connector, Q_r, at the strength limit state is to be taken as:

$$Q_r = \phi_{sc} Q_n \qquad \text{Eq. (6.10.10.4.1-1)}$$

where: ϕ_{sc} = resistance factor for shear connectors = 0.85 (Article 6.5.4.2)
Q_n = nominal shear resistance of a single shear connector determined as specified in Article 6.10.10.4.3

As specified in Article 6.10.10.4.3, the nominal shear resistance of one stud shear connector embedded in a concrete deck is to be taken as:

$$Q_n = 0.5 A_{sc} \sqrt{f'_c E_c} \le A_{sc} F_u \qquad \text{Eq. (6.10.10.4.3-1)}$$

where: A_{sc} = cross-sectional area of a stud shear connector
E_c = modulus of elasticity of the deck concrete determined as specified in Article 5.4.2.4 (= 3,644 ksi for this example as determined previously)
F_u = specified minimum tensile strength of a stud shear connector as specified in Article 6.4.4 = 60.0 ksi

$$A_{sc} = \frac{\pi}{4}(0.875)^2 = 0.60 \text{ in.}^2$$

$$A_{sc} F_u = (0.60)(60.0) = 36.00 \text{ kips}$$

$$Q_n = 0.5(0.60)\sqrt{4.0(3{,}644)} = 36.22 \text{ kips} > 36.00 \text{ kips}$$

$$\therefore \quad Q_n = 36.00 \text{ kips}$$

$$Q_r = 0.85(36.00) = 30.60 \text{ kips}$$

At the strength limit state, the minimum number of shear connectors, n, over the region under consideration is to be taken as:

$$n = \frac{P}{Q_r} \qquad \text{Eq. (6.10.10.4.1-2)}$$

where P is the total nominal shear force determined as specified in Article 6.10.10.4.2. According to Article 6.10.10.4.2, for continuous spans that are composite for negative flexure in the final condition, the total nominal shear force, P, between the point of maximum positive design live load plus impact moment and an adjacent end of the member is to be determined as:

$$P = \sqrt{P_p^2 + F_p^2} \qquad \text{Eq. (6.10.10.4.2-1)}$$

where P_p is the total longitudinal shear force in the concrete deck at the point of maximum positive live load plus impact moment taken as the lesser of:

$$P_{1p} = 0.85 f_c' b_s t_s \qquad \text{Eq. (6.10.10.4.2-2)}$$

where b_s and t_s are the effective width and thickness of the concrete deck, respectively.

or: $$P_{2p} = F_{yw} D t_w + F_{yt} b_{ft} t_{ft} + F_{yc} b_{fc} t_{fc} \qquad \text{Eq. (6.10.10.4.2-3)}$$

F_p is the total radial force in the concrete deck at the point of maximum positive live load plus impact moment and is taken equal to zero for straight spans per Article 6.10.10.4.2.

The point of maximum positive live load plus impact moment in Span 1 is located 60.2 feet from the abutment.

$$P_{1p} = 0.85(4.0)(114.0)(9.0) = 3{,}488 \text{ kips}$$

For the steel section yielding the smallest force in this region:

$$P_{2p} = (50.0)(69.0)(0.5) + (50.0)(18.0)(0.875) + (50.0)(16.0)(1.0) = 3{,}313 \text{ kips}$$

Taking into account that $F_p = 0$, P is computed as follows:

$$P = \sqrt{P_p^2 + 0} = P_p = P_{2p} = 3{,}313 \text{ kips}$$

$$n = \frac{P}{Q_r} = \frac{3{,}313}{30.60} = 108 \text{ studs}$$

Compute the required pitch, p, in this region at the strength limit state with 3 studs per row:

$$\text{No. of rows} = \frac{108}{3} = 36 \text{ rows}$$

$$p = \frac{60.2(12)}{(36-1)} = 20.6 \text{ in.}$$

The total nominal shear force, P, between the point of maximum positive design live load plus impact moment and the centerline of an adjacent interior support is to be determined as:

$$P = \sqrt{P_T^2 + F_T^2} \qquad \text{Eq. (6.10.10.4.2-5)}$$

where P_T is the total longitudinal force in the concrete deck between the point of maximum positive live load plus impact moment and the centerline of an adjacent interior support taken as:

$$P_T = P_p + P_n \qquad \text{Eq. (6.10.10.4.2-6)}$$

where: P_p = total longitudinal force in the concrete deck at the point of maximum positive live load plus impact moment (kips) taken as the lesser of either:

$$P_{1p} = 0.85f'_c\, b_s t_s \qquad \text{Eq. (6.10.10.4.2-2)}$$

or

$$P_{2p} = F_{yw}Dt_w + F_{yt}b_{ft}t_{ft} + F_{yc}b_{fc}t_{fc} \qquad \text{Eq. (6.10.10.4.2-3)}$$

P_n = total longitudinal force in the concrete deck over an interior support (kips) taken as the lesser of either:

$$P_{1n} = F_{yw}Dt_w + F_{yt}b_{ft}t_{ft} + F_{yc}b_{fc}t_{fc} \qquad \text{Eq. (6.10.10.4.2-7)}$$

or

$$P_{2n} = 0.45f'_c\, b_s t_s \qquad \text{Eq. (6.10.10.4.2-8)}$$

F_T = total radial force in the concrete deck between the point of maximum positive live load plus impact moment and the centerline of an adjacent interior support (kips) taken as zero for straight spans per Article 6.10.10.4.2

The following two terms were computed earlier and are applicable here as well:

P_p = 3,313 kips

b_s = 114 in.

Eq. 6.10.10.4.2-8 is a conservative approximation of the tension force in the concrete deck to account for the combined contribution of both the longitudinal reinforcement and also the concrete that remains effective in tension based on its modulus of rupture. A more precise value may be substituted, if desired.

The distance between the point of maximum positive live load plus impact moment in Span 1 and the adjacent interior support is (140.0 - 60.2) = 79.8 feet. For the steel section and effective concrete deck yielding the smallest forces in this region, P_n is determined as follows:

$$P_{1n} = 50(69)(0.5) + 50(18)(1.375) + 50(16.0)(1.0) = 3{,}763 \text{ kips}$$

$$P_{2n} = 0.45(4.0)(114.0)(9) = 1{,}847 \text{ kips}$$

The total longitudinal force in the deck over the interior support, $P_{n,}$ is the lesser of P_{1n} or P_{2n}; therefore, P_n is taken to be 1,847 kips.

Therefore, the total longitudinal force in the concrete deck in the region under consideration is:

$$P_T = 3{,}313 + 1{,}847 = 5{,}160 \text{ kips}$$

Taking into account that F_T = 0, the total nominal shear force in this portion of the span is computed as:

$$P = \sqrt{P_T^2 + 0^2} = P_T = 5{,}160 \text{ kips}$$

The minimum number of shear connectors, n, over the region under consideration is taken as:

$$n = \frac{P}{Q_r} \qquad \text{Eq. (6.10.10.4.1-2)}$$

$$n = \frac{5{,}160}{30.6} = 169$$

Compute the required pitch, p, in this region at the strength limit state with 3 studs per row.

$$\text{No. of rows} = \frac{169}{3} = 56.3 \text{ say 57 rows}$$

$$p = \frac{79.8(12)}{(57-1)} = 17.1 \text{ in.}$$

The distance between the point of maximum positive live load plus impact moment in Span 2 and each of the adjacent interior supports is 87.5 feet. Using calculations similar to the above:

P_p = 3,313 kips
P_n = 1,847 kips
$P = P_T$ = 5,160 kips
n = 169 studs
No. of rows = 57 rows

$p = 18.8$ in.

The final recommended pitches are governed by the fatigue limit state. The effective width of the concrete deck is larger for the interior girders, which in conjunction with different fatigue shear ranges, may result in slightly different recommended pitches. However, for practical purposes, unless the differences are deemed significant, it is recommended that the same pitches be used on all the girders.

10.5. Exterior Girder: Field Section 1

10.5.1. Transverse Intermediate Stiffener Design (Article 6.10.11.1)

Intermediate transverse stiffeners are designed according to the provisions of Article 6.10.11.1. In this example, each intermediate transverse stiffener consists of a plate welded to one side of the web. The distance between the end of the web-to-stiffener weld and the near edge of an adjacent web-to-flange or longitudinal stiffener-to-web weld must not be less than $4t_w$ or more than $6t_w$. Stiffeners not used as connection plates must be tight fit at the compression flange (and are generally fillet welded to the flange), but need not be in bearing with the tension flange. However, it should be noted that the aforementioned Guidelines recommend calling for a tight fit of these stiffeners to the tension flange. Stiffeners used as connecting plates for cross-frames or diaphragms must be connected by welding or bolting to both flanges. Welded connections are generally cheaper. Also, as noted earlier, a Category C' detail still exists at the termination of the connection-plate weld to the web just above (or below) the tension flange even when the stiffeners are bolted to that flange.

The design of the intermediate transverse stiffeners for Field Section 1 (not serving as cross-frame connection plates) will be illustrated in this example. The same size stiffeners will be used within the field section for practical purposes. Grade 50W steel will be used for the stiffeners (i.e. $F_{ys} = 50.0$ ksi).

10.5.1.1. Projecting Width (Article 6.10.11.1.2)

Size the stiffener width, b_t, to be greater than or equal to $b_f/4$ as required in Eq. 6.10.11.1.2-2. b_f is to be taken as the full width of the widest compression flange within the field section under consideration in order to ensure a minimum stiffener width that will help restrain local buckling of the widest compression flange.

$$b_t \geq \frac{16.0}{4} = 4.0 \text{ in.} \qquad \text{Eq. (6.10.11.1.2-2)}$$

Stiffeners are commonly made up of less expensive flat bar stock. Flat bars are generally produced in whole-inch width increments and 1/8-in. thickness increments in Customary U.S. units.

Use $b_t = 5.0$ in. > 4.0 in. ok

Check that:

$$b_t \geq 2.0 + \frac{D}{30} \qquad \text{Eq. (6.10.11.1.2-1)}$$

$$2.0 + \frac{69.0}{30} = 4.3 \text{ in.} < 5.0 \text{ in.} \quad \text{ok}$$

Try a stiffener thickness, t_p, of 0.5 inches. The Guidelines recommend a minimum thickness of 7/16″ for stiffeners and connection plates, with a minimum thickness of 1/2″ preferred.

Check that: $16t_p \geq b_t$ Eq. (6.10.11.1.2-2)

$$16(0.5) = 8.0 \text{ in.} > 5.0 \text{ in.} \quad \text{ok}$$

10.5.1.2. Moment of Inertia (Article 6.10.11.1.3)

The moment of inertia requirement ensures that the transverse stiffener has sufficient rigidity to maintain a vertical line of near zero lateral deflection along the line of the stiffener in order for interior web panels to adequately develop the shear buckling resistance. In this particular example, separate calculations show that the 207-inch panels govern the required moment of inertia for the intermediate transverse stiffeners in Field Section 1. The moment of inertia of the stiffener, I_t, must satisfy the smaller of the following limits:

$$I_t \geq I_{t1} \qquad \text{Eq. (6.10.11.1.3-1)}$$

$$I_t \geq I_{t2} \qquad \text{Eq. (6.10.11.1.3-2)}$$

in which:

$$I_{t1} = bt_w^3 J \qquad \text{Eq. (6.10.11.1.3-3)}$$

$$I_{t2} = \frac{D^4 \rho_t^{1.3}}{40}\left(\frac{F_{yw}}{E}\right)^{1.5} \qquad \text{Eq. (6.10.11.1.3-4)}$$

$$J = \frac{2.5}{(d_o / D)^2} - 2.0 \geq 0.5 \qquad \text{Eq. (6.10.11.1.3-5)}$$

$$F_{crs} = \frac{0.31E}{\left(\frac{b_t}{t_p}\right)^2} \leq F_{ys} \qquad \text{Eq. (6.10.11.1.3-6)}$$

where: ϕ_v = resistance factor for shear = 1.0 (Article 6.5.4.2)
I_t = moment of inertia of the transverse stiffener taken about the edge in contact with the web for single stiffeners
b = the smaller of d_o and D
d_o = the smaller of the adjacent web panel widths
J = stiffener bending rigidity parameter
ρ_t = the larger of F_{yw}/F_{crs} and 1.0
F_{crs} = local buckling stress for the stiffener
F_{ys} = specified minimum yield strength of the stiffener

Compute the stiffener bending rigidity parameter as follows:

$$J = \frac{2.5}{(207.0/69.0)^2} - 2.0 = -1.72 < 0.5$$

Therefore, J = 0.5 and I_{t1} is computed as follows:

$$I_{t1} = bt_w^3 J = (69.0)(0.5)(0.5) = 17.25 \text{ in.}^4$$

Compute the parameters necessary to determine I_{t2}.

$$F_{crs} = \frac{0.31(29{,}000)}{\left(\frac{5.0}{0.5}\right)^2} = 89.9 \text{ ksi} > F_{ys} = 50.0 \text{ ksi}$$

Therefore, F_{crs} = 50.0 ksi.

$$\frac{F_{yw}}{F_{crs}} = \frac{50.0}{50.0} = 1.0$$

Therefore, ρ_t = 1.0 and I_{t2} is computed as follows:

$$I_{t2} = \frac{D^4 \rho_t^{1.3}}{40}\left(\frac{F_{yw}}{E}\right)^{1.5} = \frac{(69.0)^4 (1.0)^{1.3}}{40}\left(\frac{50.0}{29{,}000}\right)^{1.5} = 40.57 \text{ in.}^4$$

Since I_{t1} is smaller than I_{t2}, Eq. 6.10.11.1.3-1 governs:

$$I_t \geq I_{t1} \qquad \text{Eq. (6.10.11.1.3-1)}$$

Compute the moment of inertia of the stiffener as follows:

$$I_t = \frac{1}{3}(0.5)(5.0)^3 = 20.83 \text{ in.}^4 > 17.25 \text{ in.}^4 \quad \text{ok}$$

The selected intermediate transverse stiffener is adequate.

10.6. Exterior Girder: Abutment 1

10.6.1. Bearing Stiffener Design (Article 6.10.11.2)

Bearing stiffeners are designed as columns to resist the reactions at bearing locations. According to Article 6.10.11.2.1, bearing stiffeners must be placed on the webs of built-up sections at all bearing locations. At bearing locations on rolled shapes and at other locations on built-up sections or rolled shapes subjected to concentrated loads, where the loads are not transmitted through a deck or deck system, either bearing stiffeners must be provided or else the web must be investigated for the limit states of web crippling or web local yielding according to the provisions of Article D6.5 (Appendix D to Section 6 of *AASHTO LRFD (5th Edition, 2010)*). It should be noted that the provisions of Article D6.5 should be checked whenever girders are incrementally launched over supports.

Bearing stiffeners must extend the full depth of the web and as closely as practical to the outer edges of the flanges. Each stiffener is to either be finished-to-bear (allowing the option of milling or grinding) against the flange through which it receives its load and attached with fillet welds, or else attached to that flange by a full penetration groove weld. The Guidelines recommend using finish-to-bear plus fillet welds to connect the bearing stiffeners to the appropriate flange, regardless of whether or not a cross-frame or diaphragm is connected to the stiffeners. Full penetration groove welds are costly and often result in welding deformation of the flange.

The design of the bearing stiffeners for Abutment 1 will be illustrated in this example. Grade 50W steel will be used for the stiffeners (i.e. F_{ys} = 50.0 ksi).

Assemble the bearing reactions due to the factored loads at Abutment 1. The STRENGTH I load combination controls.

$$R_u = 1.0[1.25(87+13)+1.5(13)+1.75(139)] = 388 \text{ kips}$$

10.6.1.1. Projecting Width (Article 6.10.11.2.2)

The width, b_t, of each projecting stiffener element must satisfy:

$$b_t \leq 0.48 t_p \sqrt{\frac{E}{F_{ys}}} \qquad \text{Eq. (6.10.11.2.2-1)}$$

Try two 7.0-inch-wide bars welded to each side of the web. Rearranging Eq. 6.10.11.2.2-1 gives:

$$\left(t_p\right)_{min.} = \frac{b_t}{0.48\sqrt{\frac{E}{F_{ys}}}}$$

$$\left(t_p\right)_{min.} = \frac{7.0}{0.48\sqrt{\frac{29{,}000}{50.0}}} = 0.61\text{ in.}$$

∴ Try $t_p = 5/8''$

10.6.1.2. Bearing Resistance (Article 6.10.11.2.3)

According to Article 6.10.11.2.3, the factored resistance for the fitted ends of bearing stiffeners is to be taken as:

$$\left(R_{sb}\right)_r = \phi_b\left(R_{sb}\right)_n \qquad \text{Eq. (6.10.11.2.3-1)}$$

where: ϕ_b = resistance factor for bearing = 1.0 (Article 6.5.4.2)
$(R_{sb})_n$ = nominal bearing resistance for the fitted end of bearing stiffeners

$$\left(R_{sb}\right)_n = 1.4A_{pn}F_{ys} \qquad \text{Eq. (6.10.11.2.3-2)}$$

A_{pn} = area of the projecting elements of the stiffener outside of the web-to-flange fillet welds but not beyond the edge of the flange

Assume for this example that the clip provided at the base of the stiffeners to clear the web-to-flange fillet welds is 1.5 inches in length. Therefore,

$$A_{pn} = 2(7.0 - 1.5)(0.625) = 6.88\text{ in.}^2$$

$$(R_{sb})_n = 1.4(6.88)(50.0) = 482\text{ kips}$$

$$\left(R_{sb}\right)_r = (1.0)(482) = 482\text{ kips} > R_u = 388\text{ kips} \quad \text{ok}$$

10.6.1.3. Axial Resistance (Article 6.10.11.2.4)

Determine the axial resistance of the bearing stiffener according to Article 6.10.11.2.4. This article directs the engineer to Article 6.9.2.1 for calculation of the factored axial resistance, P_r. The yield strength is F_{ys}, the radius of gyration is computed about the midthickness of the web, and the effective length is 0.75 times the web depth ($K\ell = 0.75D$).

$$P_r = \phi_c P_n \qquad \text{Eq. (6.9.2.1-1)}$$

where: P_n = nominal compressive resistance determined using the provisions of Article 6.9.4
ϕ_c = resistance factor for compression as specified in Article 6.5.4.2

As indicated in Article 6.9.4.1.1, P_n is the smallest value of the applicable modes of buckling, and in the case of bearing stiffeners, torsional buckling and flexural-torsional buckling are not applicable. Therefore, P_n is computed for flexural buckling only.

To compute P_n, first compute P_e and P_o. P_e is the elastic critical buckling resistance determined as specified in Article 6.9.4.1.2 for flexural buckling. P_o is the equivalent nominal yield resistance equal to QF_yA_g, where Q is the slender element reduction factor, taken equal to 1.0 for bearing stiffeners per Article 6.9.4.1.1

$$P_e = \frac{\pi^2 E}{\left(\frac{K\ell}{r_s}\right)^2} A_g \qquad \text{Eq. (6.9.4.1.2-1)}$$

Compute the effective length of the bearing stiffener according to Article 6.10.11.2.4.

$$K\ell = 0.75(69) = 51.8\ \text{in.}$$

Compute the radius of gyration about the midthickness of the web.

$$r_s = \sqrt{\frac{I_s}{A_s}}$$

According to the provisions of Article 6.10.11.2.4b, for stiffeners welded to the web, a portion of the web shall be included as part of the effective column section. For stiffeners consisting of two plates welded to the web, the effective column section shall consist of the two stiffener elements, plus a centrally located strip of web extending $9t_w$ on each side of the outer projecting elements of the group. The area of the web that is part of the effective section is computed as follows:

$$A_w = 2(9)(0.5)(0.5) = 4.50\ \text{in}^2$$

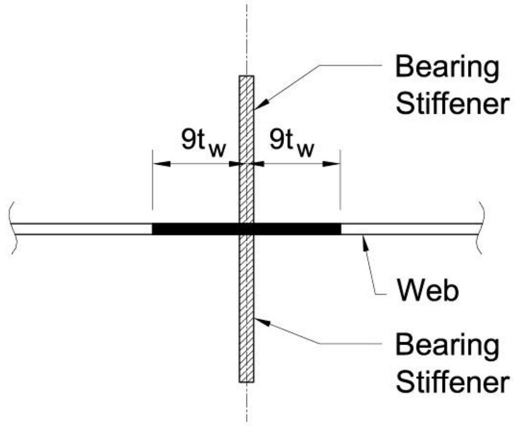

Figure 17: Effective Column Section for Bearing Stiffener Design

Conservatively, continue to use the area at the base of the stiffener to compute the axial resistance.

$$A_{pn} = 6.88\,\text{in}^2 \quad \text{(computed earlier)}$$

The total area of the effective section is therefore:

$$A_s = 4.50 + 6.88 = 11.4\,\text{in}^2$$

Next, compute the moment of inertia of the effective section:

$$I = \frac{0.625(7.0 + 0.5 + 7.0)^3}{12} = 159\,\text{in}^4$$

Compute the radius of gyration:

$$r_s = \sqrt{\frac{159}{11.4}} = 3.73 \text{ in.}$$

The elastic critical buckling resistance is computed as follows:

$$P_e = \frac{\pi^2(29{,}000)}{\left(\frac{51.8}{3.73}\right)^2}(11.4) = 16{,}918 \text{ kips}$$

The equivalent nominal yield resistance is computed as follows, with A_s used for A_g:

$$P_o = QF_yA_g = (1.0)(50)(11.4) = 570\,\text{kips}$$

Since $\frac{P_e}{P_o} = \frac{16{,}218}{570} = 28.5 > 0.44$,

the nominal axial compression resistance is computed as:

$$P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)} \right] P_o \qquad \text{Eq. (6.9.4.1.1-1)}$$

$$P_n = \left[0.658^{\left(\frac{1}{28.5}\right)} \right] (570) = 562 \text{ kips}$$

The factored resistance of the bearing stiffeners is computed as follows:

$$P_r = \phi_c P_n = 0.90(562) = 506 \text{ kips}$$

$$P_u = 388 \text{kips} < P_r = 506 \text{ kips} \text{ OK}$$

The bearing stiffeners selected for the exterior girder at Abutment 1 satisfy the requirements for design.

10.6.1.4. Bearing Stiffener-to-Web Welds

As specified in Article 6.13.3.2.4b, the resistance of fillet welds in shear which are made with matched or undermatched weld metal is to be taken as the product of the effective area of the weld and the factored resistance of the weld metal. For a fillet weld, the effective area is defined in Article 6.13.3.3 as the effective weld length multiplied by the effective throat. The effective throat is the shortest distance from the root of the joint to the face of the fillet weld (equal to 0.707 times the weld leg size for welds with equal leg sizes). As specified in Article 6.13.3.5, the effective length of a fillet weld is to be at least four times its nominal size, or 1½ inches, whichever is greater.

As specified in Article 6.13.3.1, matching weld metal (i.e. with the same or slightly higher minimum specified minimum yield and tensile strength compared to the minimum specified properties of the base metal) is generally to be used for fillet welds. Undermatched weld metal may be specified by the Engineer for fillet welds when the welding procedure and weld metal are selected to ensure sound welds, and is encouraged for fillet welds connecting steels with specified minimum yield strengths greater than 50.0 ksi. For ASTM A 709 Grade 50W steel, the specified minimum tensile strength is 70.0 ksi (Table 6.4.1-1). Thus, assume the classification strength of the weld metal is also 70.0 ksi. The classification strength of the weld metal is expressed as EXX, where the letters XX stand for the minimum strength level of the electrode in ksi.

According to Table 6.13.3.4-1, the minimum size fillet weld is ¼ inch when the base metal thickness (T) of the thicker part joined is less than ¾ inches. The factored shear resistance of the weld metal is taken as:

$$R_r = 0.6\phi_{e2}F_{exx} \qquad \text{Eq. (6.13.3.2.4b-1)}$$

where: ϕ_{e2} = resistance factor for shear in the throat of the weld metal = 0.8 (Article 6.5.4.2)
F_{exx} = classification strength of the weld metal = 70.0 ksi in this case

$$R_r = 0.6(0.80)(70.0) = 33.6 \text{ ksi}$$

The resistance of a ¼ inch fillet weld in shear in kips/inch is then computed as:

$$v = 33.6(0.707)(0.25) = 5.94 \text{ kips/in.}$$

The total length of weld, allowing 2.5 inches for the clips at the top and bottom of the stiffener, is:

$$L = 69.0 - 2(2.5) = 64.0 \text{ in.}$$

The total factored resistance of the four ¼-inch fillet welds connecting the stiffeners to the web is therefore:

$$4(64.0)(5.94) = 1{,}521 \text{ kips} > 388 \text{ kips} \quad \text{ok}$$

10.7. Exterior Girder: Design Example Summary

The results for this design example at each limit state are summarized below. 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.

10.7.1. Positive-Moment Region, Span 1 (Section 1-1)

10.7.1.1. Constructibility (Slender-web section)

Flexure (STRENGTH I)	
Eq. (6.10.3.2.1-1) – Top flange	0.844
Eq. (6.10.3.2.1-2) – Top flange	0.835
Eq. (6.10.3.2.1-3) – Web bend buckling	0.697
Eq. (6.10.3.2.2-1) – Bottom flange	0.535
Flexure (STRENGTH III – Wind load on noncomposite structure)	
Eq. (6.10.3.2.1-1) – Top flange	0.261
Eq. (6.10.3.2.1-2) – Top flange	0.170
Eq. (6.10.3.2.1-3) – Web bend buckling	0.085
Eq. (6.10.3.2.2-1) – Bottom flange	0.272
Flexure (STRENGTH IV)	
Eq. (6.10.3.2.1-1) – Top flange	0.955
Eq. (6.10.3.2.1-2) – Top flange	0.977
Eq. (6.10.3.2.1-3) – Web bend buckling	0.836
Eq. (6.10.3.2.2-1) – Bottom flange	0.602
Shear (96′-0″ from the abutment) (STRENGTH IV) Eq. (6.10.3.3-1)	0.447

10.7.1.2. Service Limit State

Live-load deflection	0.433
Permanent deformations (SERVICE II)	
Eq. (6.10.4.2.2-1) – Top flange	0.454
Eq. (6.10.4.2.2-2) – Bottom flange	0.771

10.7.1.3. Fatigue and Fracture Limit State

Base metal at connection plate weld to bottom flange (72′-0″ from the abutment) (FATIGUE I)	0.983
Stud shear connector weld to top flange (100′-0″ from the abutment) (FATIGUE I)	0.177

Special fatigue requirement for webs (shear - 7′-3″ from the abutment) (FATIGUE I)	0.618

10.7.1.4. Strength Limit State (Compact Section)

Ductility requirement – Eq. (6.10.7.3-1)	0.343
Flexure – Eq. (6.10.7.1.1-1) (STRENGTH I)	0.741
Flexure – Eq. (6.10.7.1.1-1) (STRENGTH III)	0.287
Flexure – Eq. (6.10.7.1.1-1) (STRENGTH IV)	0.324
Flexure – Eq. (6.10.7.1.1-1) (STRENGTH V)	0.638
Shear (End panel) (STRENGTH I) Eq. (6.10.9.1-1)	0.995

10.7.2. Interior-Pier Section (Section 2-2)

10.7.2.1. Strength Limit State (Slender-web section)

Flexure (STRENGTH I)	
Eq. (6.10.8.1.1-1) – Bottom flange	1.006
Eq. (6.10.8.1.3-1) – Top flange @ Section 2-2	0.782
Eq. (6.10.8.1.3-1) – Top flange @ Flange transition	0.763
Flexure (STRENGTH III)	
Eq. (6.10.8.1.1-1) – Bottom flange	0.538
Eq. (6.10.8.1.3-1) – Top flange @ Section 2-2	0.459
Eq. (6.10.8.1.3-1) – Top flange @ Flange transition	0.444
Flexure (STRENGTH IV)	
Eq. (6.10.8.1.1-1) – Bottom flange	0.620
Eq. (6.10.8.1.3-1) – Top flange @ Section 2-2	0.539
Eq. (6.10.8.1.3-1) – Top flange @ Flange transition	0.523
Flexure (STRENGTH V)	
Eq. (6.10.8.1.1-1) – Bottom flange	0.896
Eq. (6.10.8.1.3-1) – Top flange @ Section 2-2	0.708
Eq. (6.10.8.1.3-1) – Top flange @ Flange transition	0.688

10.7.2.2. Service Limit State

Permanent deformations (SERVICE II)	
Eq. (6.10.4.2.2-1) – Top flange @ Section 2-2	0.404
Eq. (6.10.4.2.2-1) – Top flange @ Flange transition	0.359
Eq. (6.10.4.2.2-2) – Bottom flange @ Section 2-2	0.601
Eq. (6.10.4.2.2-2) – Bottom flange @ Flange transition	0.606
Eq. (6.10.4.2.2-4) – Web bend buckling @ Section 2-2	0.912
Eq. (6.10.4.2.2-4) – Web bend buckling @ Flange transition	0.992

10.7.2.3. Fatigue and Fracture Limit State

Base metal at connection plate weld to top flange (20′-0″ to the left of the interior pier) (FATIGUE I)	0.094
Special fatigue requirement for webs (shear at interior pier) (FATIGUE I)	0.636

10.7.2.4. Constructibility (Slender-web section)

Flexure (STRENGTH IV)	
Eq. (6.10.3.2.1-3) – Web bend buckling @ Section 2-2	0.419
Eq. (6.10.3.2.1-3) – Web bend buckling @ Flange transition	0.427
Shear (at interior pier) (STRENGTH IV) Eq.(6.10.3.3-1)	0.528

11.0 REFERENCES

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

Appendix A:
Elastic Effective Length Factor for Lateral Torsional Buckling

By

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The equations for determining the nominal lateral torsional buckling (LTB) resistance of the compression flange in Articles 6.10.8.2.3 and A6.3.3 (Appendix A to LRFD Section 6) assume an elastic effective length factor of K = 1.0 for the critical unbraced length. When adjacent unbraced lengths are less critically loaded, substantial restraint can exist at the ends of a critical unbraced length such that K may be less than 1.0 for the critical length. Should the unbraced length under consideration end up being the critical unbraced length for which K is less than 1.0, the lower value of K can then subsequently be used to appropriately increase the elastic LTB resistance of the compression flange, F_{cr}. A lower value of F_{cr} will in turn result in a lower value of the amplification factor (specified in Article 6.10.1.6) that may be applied to calculated first-order compression-flange lateral bending stresses within the unbraced length, should they exist. The unbraced length, L_b, also can be modified by the effective length factor $K < 1$ to determine a larger nominal LTB resistance for the compression flange within the critical unbraced length.
Article C6.10.8.2.3 refers to Galambos (1988) and Nethercot and Trahair (1976) for a practical design procedure for determining elastic effective length factors associated with LTB, applicable for the case where a member is continuous with adjacent unbraced lengths. The procedure is based on the analogy between the buckling of a continuous beam and the buckling of an end-restrained column. As such, the alignment chart for nonsway columns given in the AISC LRFD Specifications (1999) can be used to determine the effective length factor for the critical unbraced length. The procedure is conservative because the moment-envelope values in adjacent unbraced lengths are assumed to be the concurrent loadings associated with LTB of the critical unbraced length.

The application of this procedure is demonstrated for the unbraced length in Span 1 of the example bridge containing Section 1-1. This unbraced length is in a region of positive flexure and spans between the cross-frames located 48.0 feet and 72.0 feet from the abutment. Therefore, L_b is equal to 24.0 feet. The LTB resistance of the top (compression) flange of the noncomposite section is computed for this unbraced length in the example in order to check the top flange for the construction condition. This unbraced length is identified herein as Segment M. The equal 24-foot-long unbraced lengths immediately to the left and to the right of Segment M (Figure 2) are identified as Segments L and R, respectively.

<u>STEP 1:</u> Determine the moment gradient modifier, C_b, for each segment.

Segment L: Segment L contains a bottom-flange transition 42.0 feet from the abutment (Figure 3). Since the transition is located at a distance greater than 20 percent of the unbraced length from the brace point with the smaller moment, C_b is taken equal to 1.0 (as recommended in Article C6.10.8.2.3)[1].

[1] The procedure outlined in Appendix C (to this design example) may be used to obtain a more precise estimate of the LTB resistance of unbraced lengths with stepped flanges.

Segment M: Since $f_{mid}/f_2 > 1$ within this segment, C_b must be taken equal to 1.0 according to the provisions of Article 6.10.8.2.3.

Segment R: Since the member is prismatic within Segment R and since f_{mid}/f_2 is less than 1.0 and f_2 is not equal to zero, calculate the moment gradient modifier, C_b, according to Eq. 6.10.8.2.3-7 as follows:

$$C_b = 1.75 - 1.05\left(\frac{f_1}{f_2}\right) + 0.3\left(\frac{f_1}{f_2}\right)^2 \le 2.3 \qquad \text{Eq. (6.10.8.2.3-7)}$$

f_2 is generally taken as the largest compressive stress without consideration of lateral bending due to the factored loads at either end of the unbraced length of the flange under consideration, calculated from the critical moment envelope value. f_2 is always taken as positive or zero. If the stress is zero or tensile in the flange under consideration at both ends of the unbraced length, f_2 is taken equal to zero (in this case, C_b = 1 and Eq. 6.10.8.2.3-7 does not apply). The value of f_1 is given by Eq. 6.10.8.2.3-10 as:

$$f_1 = 2f_{mid} - f_2 > f_o \qquad \text{Eq. (6.10.8.2.3-10)}$$

where f_{mid} is the stress without consideration of lateral bending due to the factored loads at the middle of the unbraced length. f_o is the stress without consideration of lateral bending due to the factored loads at the brace point opposite to the one corresponding to f_2. Both f_{mid} and f_o are to be calculated from the moment envelope value that produces the largest compression at the respective points, or the smallest tension if that point is never in compression, and both are to be taken as positive in compression and negative in tension.

In this particular example, the STRENGTH IV load combination governs the constructibility check. The stresses below are computed from the results of the deck-placement analysis (Table 11):

For STRENGTH IV:

Top flange:

$$f_2 = \frac{1.0(1.5)(2{,}706)(12)}{1{,}581} = 30.81 \text{ ksi}$$

$$f_{mid} = \frac{1.0(1.5)(2{,}273)(12)}{1{,}581} = 25.88 \text{ ksi}$$

$$f_o = \frac{1.0(1.5)(1{,}585)(12)}{1{,}581} = 18.05 \text{ ksi}$$

$$f_1 = 2(25.88) - (30.81) = 20.95 \text{ ksi} > f_o = 18.05 \text{ ksi} \quad \therefore f_1 = 20.95 \text{ ksi}$$

$$C_b = 1.75 - 1.05\left(\frac{20.95}{30.81}\right) + 0.3\left(\frac{20.95}{30.81}\right)^2 = 1.17 < 2.3 \quad \text{ok}$$

STEP 2: Identify the critical segment.

The critical segment is defined as the segment that buckles elastically at the smallest multiple of the design loadings based on the largest moment envelope value within each segment, and with F_{cr} calculated using the actual unbraced lengths L_b as the effective lengths. The multiple of the design loadings associated with the buckling of the critical segment is denoted as γ_m, and the multiples of the design loadings associated with the buckling of the adjacent segments (should they exist) are denoted as γ_{rL} and γ_{rR}, respectively. For all of these segments, the following equation applies:

$$\gamma = \frac{F_{cr}}{f_{bu}} \tag{A1}$$

where f_{bu} is the largest value of the compressive stress throughout the unbraced length in the flange under consideration and F_{cr} is the elastic LTB stress for the flange specified in Article 6.10.8.2.3 determined as:

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

For checking constructibility, the web load-shedding factor, R_b, is to be taken equal to 1.0 (Article 6.10.1.10.2) since web bend buckling is prevented during construction by a separate limit state check. The effective radius of gyration for LTB, r_t, is taken as the value within the unbraced length that produces the smallest buckling resistance. Therefore,

Segment L: Separate calculations show that f_{bu} is controlled by the section at the flange transition and F_{cr} is controlled by the larger section within the segment (r_t is smaller). Therefore,

$$f_{bu} = \frac{1.0(1.5)(2{,}677)(12)}{1{,}485} = -32.45 \text{ ksi}$$

$$F_{cr} = \frac{1.0(1.0)\pi^2(29{,}000)}{\left(\frac{24.0(12)}{3.90}\right)^2} = 52.49 \text{ ksi}$$

$$\gamma = \gamma_{rL} = \frac{52.49}{|-32.45|} = 1.62$$

Segment M: $f_{bu} = \dfrac{1.0(1.5)(2{,}889)(12)}{1{,}581} = -32.89 \text{ ksi}$

$$F_{cr} = 52.49 \text{ ksi}$$

$$\gamma = \gamma_m = \frac{52.49}{|-32.89|} = 1.60 \quad \text{(governs)}$$

Segment R: $$f_{bu} = \frac{1.0(1.5)(2{,}706)(12)}{1{,}581} = -30.81 \text{ ksi}$$

$$F_{cr} = \frac{1.17(1.0)\pi^2(29{,}000)}{\left(\frac{24.0(12)}{3.90}\right)^2} = 61.41 \text{ ksi}$$

$$\gamma = \gamma_{rR} = \frac{61.41}{|-30.81|} = 1.99$$

<u>STEP 3:</u> Calculate a stiffness ratio, α, for each of the segments.

The stiffness ratio, α_m, for the critical segment is determined as:

$$\alpha_m = \frac{2\left(b_{fc}t_{fc} + \frac{1}{6}D_c t_w\right)r_t^2}{L_{bcr}} \qquad \text{(A2)}$$

and for each adjacent segment is determined as:

$$\alpha_r = \frac{n\left(b_{fc}t_{fc} + \frac{1}{6}D_c t_w\right)r_t^2}{L_b}\left(1 - \frac{\gamma_m}{\gamma_r}\right) \qquad \text{(A3)}$$

where n = 2 if the far end of the adjacent segment is continuous, n = 3 if the far end of the adjacent segment is pinned, and n = 4 if the far end of the adjacent segment is fixed. These equations are a generalization of the procedures outlined by Nethercot and Trahair (1976) and Galambos (1998) to allow for consideration of the more general case of singly-symmetric I-sections, which are the most common type of section used in steel-bridge construction. If one end of the critical segment is a simply supported end, α_r = ¥ at that end. In this case, the far ends of the adjacent segments are both continuous; therefore, n = 2 for both segments. Also, for cases involving singly-symmetric I-sections and reverse curvature bending in any one of the above segments, the area ($b_{fc}t_{fc} + D_c t_w/6$) and r_t terms are the corresponding values that produce the smallest buckling resistance.

Segment L: $\alpha_r = \alpha_{rL} = \dfrac{2\left[16(1)+\dfrac{1}{6}(38.63)(0.5)\right](3.90)^2}{24.0(12)}\left(1-\dfrac{1.60}{1.62}\right) = 0.025$

Segment M: $\alpha_m = \dfrac{2\left[16(1)+\dfrac{1}{6}(38.63)(0.5)\right](3.90)^2}{24.0(12)} = 2.03$

Segment R: $\alpha_r = \alpha_{rR} = \dfrac{2\left[16(1)+\dfrac{1}{6}(38.63)(0.5)\right](3.90)^2}{24.0(12)}\left(1-\dfrac{1.60}{1.99}\right) = 0.398$

STEP 4: Determine the stiffness ratios, $G = \alpha_m/\alpha_r$, for each end of the critical segment.

Left end: $G = \dfrac{\alpha_m}{\alpha_{rL}} = \dfrac{2.03}{0.025} = 81.2 > 50.0 \quad \therefore$ use 50.0[2]

Right end: $G = \dfrac{\alpha_m}{\alpha_{rR}} = \dfrac{2.03}{0.398} = 5.10$

STEP 5: Obtain the effective length factor, K, from the nonsway restrained column nomograph. From Figure C-C2.2 of the AISC LRFD Specifications (1999), for the sidesway inhibited case:

$K = 0.96$

Therefore, for the critical unbraced length, the elastic lateral torsional buckling resistance may be computed as:

$$F^*_{cr} = F_{cr}\left(\frac{1}{K^2}\right) = 52.49\left(\frac{1}{(0.96)^2}\right) = 56.96 \text{ ksi} \qquad \text{an 8.5\% increase}$$

which will result in a slightly smaller amplification of the first-order lateral flange bending stresses in the compression flange within this unbraced length according to Eq. 6.10.1.6-4. Of course, the benefit is relatively small in this particular example, but it may be a significant benefit in some cases. A slightly smaller unbraced length of KL_b can also be used in this case, if desired, to determine the nominal LTB resistance of the compression flange within the critical unbraced length, F_{nc}.

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[2] The sidesway inhibited nomograph shown in AISC (1999) does not label values for G larger than 50. The top of the nomograph actually corresponds to $G = \infty$; however, effectively the same results are obtained using $G = \infty$ or $G = 50$.

Once the effective length factor for the critical segment has been determined, the effective length factor for the adjacent segments should be computed as:

$$K_r = \sqrt{\frac{\gamma_r}{\gamma_m^*}} \tag{A4}$$

where g_m^* is the multiple of the design loadings associated with the buckling of the critical segment based on the reduced K value. For this case,

$$\gamma_m^* = \frac{56.96}{|-32.89|} = 1.73$$

$$K_{rL} = \sqrt{\frac{1.62}{1.73}} = 0.97$$

$$K_{rR} = \sqrt{\frac{1.99}{1.73}} = 1.07$$

Note that the effective length factor for the adjacent segments may actually exceed 1.0, but these segments are always less critical segments. For all remaining unbraced lengths not adjacent to the critical segments, K should be taken equal to 1.0 for the condition under investigation. The procedure is focused on a local subassembly composed of the most critical segment and the unbraced lengths adjacent to this segment. The Engineer may assume that more remote unbraced lengths are not affected significantly by buckling interaction with the critical segment. Note that the same procedure may also be applied when the optional provisions of Appendix A (to LRFD Section 6 -- Article A6.3.3) are used to compute the nominal LTB resistance, and when Eq. 6.10.1.6-5 is used to compute the amplification factor for flange lateral bending.

References

AISC. 1999. “Load and Resistance Factor Design Specification for Structural Steel Buildings and Commentary.” American Institute of Steel Construction, Chicago, IL, Third Edition, December 27, 1999.

Galambos, T. V., ed. 1998. *Guide to Stability Design Criteria for Metal Structures*. 5th ed. Structural Stability Research Council. John Wiley and Sons, Inc., New York, NY.

Nethercot, D. A. and N. S. Trahair. 1976. “Lateral Buckling Approximations for Elastic Beams.” *The Structural Engineer*, Vol. 54, No. 6, pp. 197-204.

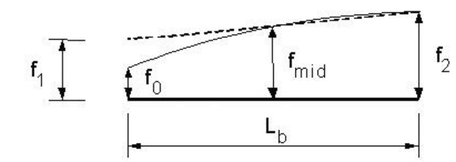
Appendix B:
Moment Gradient Modifier, C_b

Unbraced cantilevers and members where $f_{mid}/f_2 > 1$ or $f_2 = 0$: $C_b = 1$

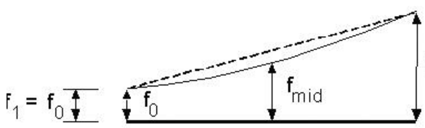
Otherwise: $C_b = 1.75 - 1.05\left(f_1/f_2\right) + 0.3\left(f_1/f_2\right)^2 \le 2.3$

$f_1 = 2f_{mid} - f_2 \ge f_0$

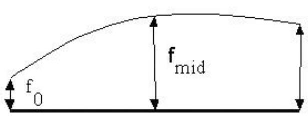
Examples:



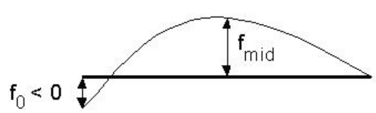
$f_{mid}/f_2 = 0.875$
$f_1/f_2 = 0.75$
$C_b = 1.13$



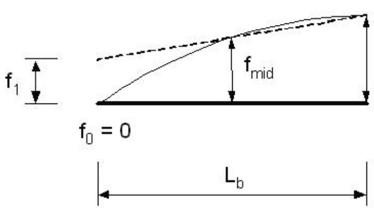
$f_1/f_2 = 0.375$
$C_b = 1.40$



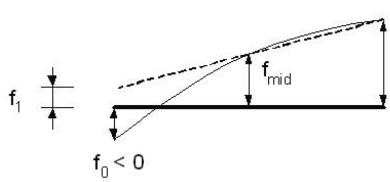
$f_{mid} > f_2$
$C_b = 1$



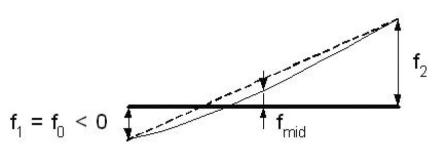
$f_2 = 0$
$C_b = 1$



$f_{mid}/f_2 = 0.75$
$f_1/f_2 = 0.5$
$C_b = 1.3$



$f_{mid}/f_2 = 0.625$
$f_1/f_2 = 0.25$
$C_b = 1.51$



$f_1/f_2 = -0.375$
$C_b = 2.19$

Note: The above examples assume that the member is prismatic within the unbraced length, or the transition to a smaller section is within $0.2L_b$ from the braced point with the lower moment. Otherwise, use $C_b = 1$.

Appendix C:
Lateral Torsional Buckling Resistance of Stepped Flanges

By

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As specified in Article 6.10.8.2.3, for unbraced lengths containing a transition to a smaller section at a distance less than or equal to 20 percent of the unbraced length from the brace point with the smaller moment, the lateral torsional buckling (LTB) resistance may be determined assuming the transition to the smaller section does not exist. For a case with more than one flange transition, any transition located within 20 percent of the unbraced length from the brace point with the smaller moment may be ignored and the LTB resistance of the remaining nonprismatic unbraced length may then be computed as the smallest resistance based on the remaining sections. When all flange transitions are located at a distance greater than 20 percent of the unbraced length from the brace point with the smaller moment, the LTB resistance is to be taken as the smallest resistance within the unbraced length under consideration. This resistance is to be compared to the largest value of the compressive stress due to the factored loads, f_{bu}, throughout the unbraced length calculated using the actual properties at each section. Note also that the moment gradient modifier, C_b, should be taken equal to 1.0, and L_b should not be modified by an elastic effective length factor when this approximate procedure is used.

As illustrated in the design example (i.e. in the design checks for Section 2-2), this approximate procedure typically results in a significant discontinuity (reduction) in the predicted LTB resistance when a flange transition is moved beyond $0.2L_b$ from the brace point with the smaller moment. In this particular example, an increase in the unbraced length, L_b, adjacent to the interior pier from 17.0 feet to 20.0 feet, with a single bottom-flange transition located 15.0 feet from the pier, resulted in a drop in the predicted lateral torsional buckling resistance from 68.19 ksi to 57.11 ksi (a 16 percent reduction).

To help determine if the predicted drop in the nominal flexural resistance is reasonable, a more rigorous approximate procedure is presented herein for predicting the LTB resistance of the compression flange within an unbraced length containing a single flange transition. The procedure is based on work by Carskaddan and Schilling (1974), which attempted to address the general case of lateral torsional buckling of singly-symmetric noncomposite or composite girders in negative flexure subjected to a moment gradient within any given unbraced length. The calculations in this report are based on the following ratio:

$$\chi = \frac{P_{cr}}{\pi^2 EI_2 / L_b^2} \tag{C1}$$

where P_{cr} is the elastic critical buckling load for a stepped column subjected to uniform axial compression, and p^2EI_2/L_b^2 is the corresponding elastic critical buckling load for a prismatic column having the larger of the two moments of inertia, I_2. This ratio is given by Figure C1, which is Figure 3 from Carskaddan and Schilling (1974). This figure is further adapted from Figure 2 of Dalal (1969), but with changes in notation, where:

$$\alpha_2 = \frac{L_2}{L_b} \tag{C2}$$

and $$\beta = \frac{I_2}{I_1} \tag{C3}$$

For an I-section in flexure, the above column analogy corresponds to lateral buckling of the compression flange, and therefore,

$$\beta = \frac{t_{fc2} b_{fc2}^3}{t_{fc1} b_{fc1}^3} \tag{C4}$$

Based on Eq. (C1), the compression-flange stress at the maximum moment location at elastic lateral torsional buckling of the stepped unbraced length, normalized with respect to the yield strength of the compression flange at the maximum moment location, may be expressed as:

$$\frac{F_{cr2}}{F_{yc2}} = \chi \frac{C_b R_{b2} \pi^2 E}{F_{yc2} \left(L_b / r_{t2}\right)^2} \tag{C5}$$

where the moment gradient modifier, C_b, is calculated according to the provisions of Article 6.10.8.2.3 (or Article A6.3.3 as applicable) assuming the unbraced length is prismatic and based on the larger section within the unbraced length. χ is determined from Figure C1 for the analogous equivalent stepped column. If available, other more rigorous estimations of F_{cr2} may be substituted for the value given by Eq. (C5). Carskaddan and Schilling (1974) show that for $\alpha_2 = 0.5$, Eq. (C5) is conservative relative to other more rigorous calculations of F_{cr2}. It is logical that χ would always be smaller for the case of uniform axial compression within an actual or equivalent column versus the case of the same column subjected to an axial compression that increases toward the end with the larger flexural rigidity. Thus, it is conservative to apply the χ value from Figure C1 as a factor that accounts for the reduction in the elastic critical stress level due to a single step in the geometry of a general member subject to moment gradient conditions. The elastic critical stress, F_{crs}, at the smaller section within the unbraced length when the elastic critical stress, F_{cr2}, is reached at the maximum moment point can be computed as follows (again normalized with respect to the yield strength of the compression flange at the smaller section):

$$\frac{F_{crs}}{F_{ycl}} = \frac{F_{cr2}}{F_{yc2}} \frac{F_{yc2}}{F_{ycl}} \frac{f_{bs}}{f_{b2}} \frac{R_{b1}}{R_{b2}} = \frac{F_{cr2}}{F_{yc2}} \frac{F_{yc2}}{F_{ycl}} \frac{M_s}{M_2} \frac{S_{xc2}}{S_{xcs}} \frac{R_{b1}}{R_{b2}} = \frac{F_{cr2}}{F_{yc2}} \frac{F_{yc2}}{F_{ycl}} \frac{S_{xc2}}{S_{xcs}} \frac{R_{b1}}{R_{b2}} \left[1 + \alpha_2 \left(\frac{M_1}{M_2} - 1\right)\right] \tag{C6}$$

where M_1 is a moment at the brace point with the lower moment, determined in general in the same manner that f_1 is calculated when determining C_b according to the specification provisions. The expression within the square brackets in the final right-hand side form of Eq. (C6) is based on the replacement of the moment envelope associated with the unbraced length under consideration with an equivalent linear variation between M_2 and M_1. The expression within the square brackets, multiplied by S_{xc2}/S_{xcs}, is f_{bs}/f_{b2} based on this equivalent linear variation of the moment along the unbraced length.

Once the ratios of the elastic critical buckling stresses to the corresponding yield strengths are determined at Location 2 and within the smaller section at the flange transition (denoted here as Location s), the corresponding F_n/F_{yc} values at each of the above locations may be calculated as follows:

$$\text{If } \frac{F_{cr}}{F_{yc}} \geq \pi^2 \qquad F_{nc} = R_b R_h F_{yc} \tag{C7}$$

$$\text{If } \frac{F_{yr}}{F_{yc}} \leq \frac{F_{cr}}{F_{yc}} < \pi^2 \qquad F_{nc} = \left[1 - \left(1 - \frac{F_{yr}}{R_h F_{yc}}\right)\left(\frac{\pi/\sqrt{F_{cr}/F_{yc}} - 1}{\pi/\sqrt{F_{yr}/F_{yc}} - 1}\right)\right] R_b R_h F_{yc} \tag{C8}$$

$$\text{If } \frac{F_{cr}}{F_{yc}} < \frac{F_{yr}}{F_{yc}} \qquad F_{nc} = F_{cr} \tag{C9}$$

Eqs. (C7) through (C9) are obtained by writing the LTB resistance expressions given by Eqs. 6.10.8.2.3-1 through 6.10.8.2.3-3 in terms of the ratio of F_{cr}/F_{yc} (computed assuming R_b is equal to 1.0), rather than in terms of the unbraced length L_b. Eqs. (C7) through (C9) give exactly the same result as Eqs. 6.10.8.2.3-1 through 6.10.8.2.3-3 for the case of a prismatic member subject to uniform bending moment. These equations give a conservative representation of the inelastic LTB resistance of unbraced lengths with a single step in the cross-section. The equations, configured in this manner, are based fundamentally on a uniform F_{cr}/F_{yc} within the compression flange of prismatic members. The compression flange in a stepped unbraced length is not stressed uniformly along its length, and thus the mapping from F_{cr}/F_{yc} to F_{nc} is conservative since the inelastic reduction in stiffness is less for this case than if the compression flange were stressed uniformly. Since for the stepped member, the smaller cross-section may experience significant yielding within the middle regions of the unbraced length, Eqs. (C7) through (C9) are employed both at Location 2 and at Location s to ensure that the result is still conservative for stepped members that experience significant yielding prior to reaching their maximum LTB resistance.

The application of this suggested procedure is illustrated to determine the LTB resistance of the stepped bottom (compression) flange within the 20-foot-long unbraced length adjacent to the pier section (Section 2-2) in the design example at the strength limit state (see Figure 3). For this unbraced length,

$$\alpha_2 = \frac{L_2}{L_b} = \frac{15.0}{20.0} = 0.75$$

$$\beta = \frac{t_{fc2} b_{fc2}^3}{t_{fc1} b_{fc1}^3} = \frac{2(20)^3}{1(20)^3} = 2.0$$

$$\chi = 0.9 \quad \text{from Figure C1}$$

Calculate the ratio of F_{cr2}/F_{yc2} at Location 2 from Equation (C5) and the ratio of F_{crs}/F_{yc1} at the section transition (Location s) from Equation (C6). The necessary data for these locations are obtained from the design example calculations for this particular unbraced length:

$$\frac{F_{cr2}}{F_{yc2}} = 0.9 \frac{1.25(0.990)\pi^2(29{,}000)}{70.0\left(20.0(12)/5.33\right)^2} = 2.50$$

$$\frac{F_{crs}}{F_{ycl}} = 2.50\left(\frac{70.0}{70.0}\right)\left(\frac{3{,}310}{1{,}975}\right)\left(\frac{0.977}{0.990}\right)\left[1 + 0.75\left(\frac{8{,}463}{14{,}979} - 1\right)\right] = 2.79$$

In both cases, $\frac{F_{yr}}{F_{yc}} = \frac{49.0}{70.0} = 0.7 < \frac{F_{cr}}{F_{yc}} < \pi^2 = 9.87$

Therefore, inelastic LTB governs at both locations and:

$$F_{nc2} = \left[1 - \left(1 - \frac{49.0}{(0.984)(70.0)}\right)\left(\frac{\pi/\sqrt{2.50} - 1}{\pi/\sqrt{0.7} - 1}\right)\right](0.990)(0.984)(70.0) = 61.14 \text{ ksi}$$

$$F_{ncs} = \left[1 - \left(1 - \frac{49.0}{(0.971)(70.0)}\right)\left(\frac{\pi/\sqrt{2.79} - 1}{\pi/\sqrt{0.7} - 1}\right)\right](0.977)(0.971)(70.0) = 60.48 \text{ ksi}$$

The flange stress f_{b2} at Location 2 due to the factored loads is compared to F_{nc2} and the flange stress at Location s, f_{bs}, due to the factored loads is compared to F_{ncs}, to determine that the unbraced length has adequate LTB resistance. Note, however, that the flange local buckling resistance of 59.26 ksi at Location s (as computed in the design example) would actually control in this case and would be taken as the nominal flexural resistance of the compression flange at Location s. The local buckling resistance of 68.19 ksi at Location 2 would not control.
In this particular case, the LTB resistance from the more rigorous approach at Location s is only 5.9 percent greater than the single value of 57.11 ksi predicted for this unbraced length using the less rigorous approximate approach given in the specifications. The value of 57.11 ksi is calculated assuming the unbraced length is prismatic based on the section at Location s, with C_b taken equal to 1.0. The increase in the LTB resistance may be more significant in other situations. The suggested method herein provides one possible approach for evaluating the calculated LTB resistance of a stepped flange (with a single step) in greater detail and for determining a larger resistance in situations where it may be desirable or necessary to do so.

Note that similar logic can be applied to develop a set of equations to be used in lieu of the LTB equations given in Article A6.3.3 (Appendix A to LRFD Section 6) for sections with compact or noncompact webs. The LTB equations given in Article A6.3.3 include the effect of the St. Venant torsional rigidity, GJ. However, the more basic equations provided herein, ignoring the influence of the torsional rigidity, may be conservatively used for these sections, if desired.

References

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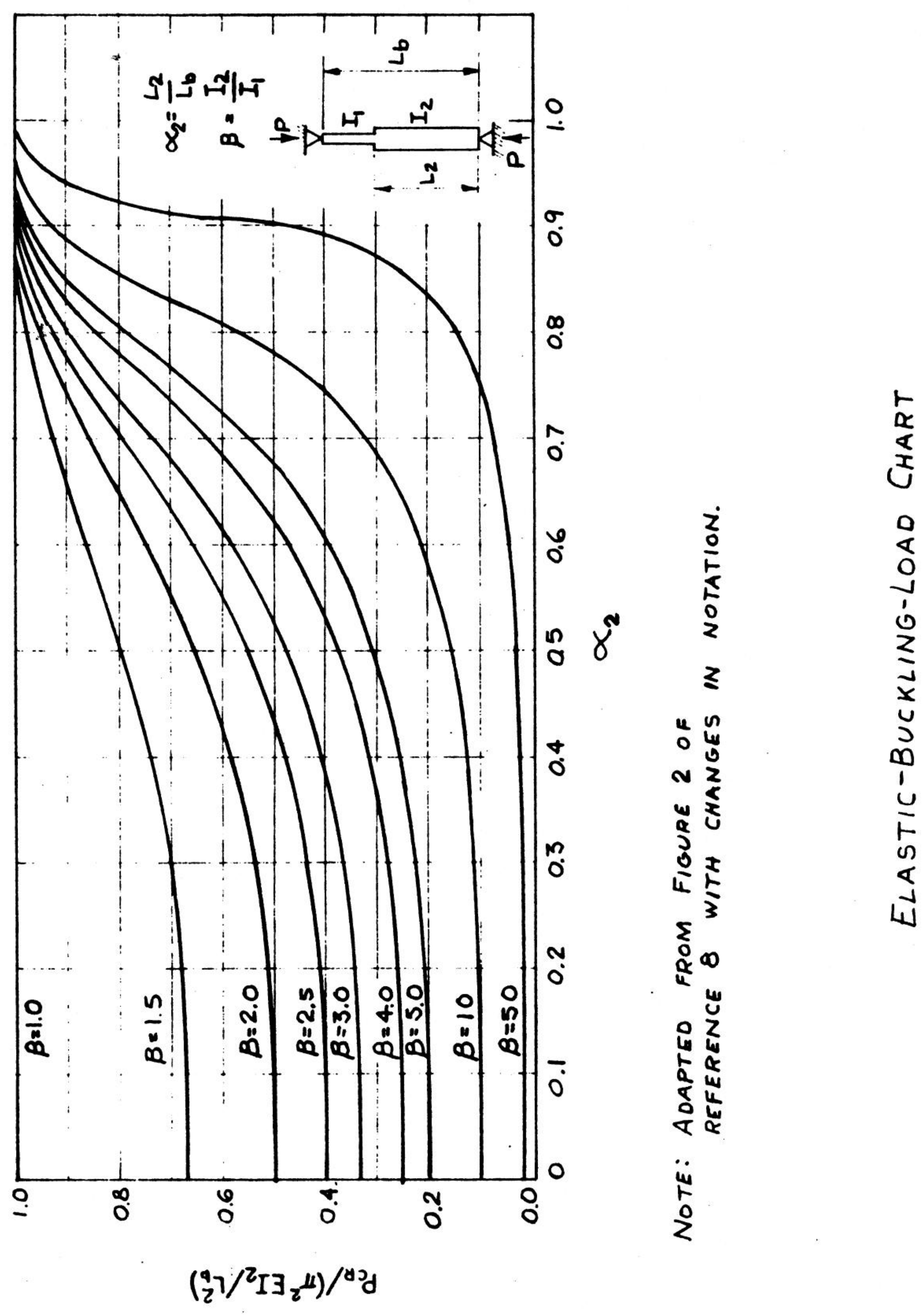

Figure C1: χ Ratio Chart