Steel Bridge Design Handbook

APPENDIX

Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge

February 2022

Smarter.
Stronger.
Steel.

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Foreword

The Steel Bridge Design Handbook covers a full range of topics and design examples to provide bridge engineers with the information needed to make knowledgeable decisions regarding the selection, design, fabrication, and construction of steel bridges. The Handbook has a long history, dating back to the 1970s in various forms and publications. The more recent editions of the Handbook were developed and maintained by the Federal Highway Administration (FHWA) Office of Bridges and Structures as FHWA Report No. FHWA-IF-12-052 published in November 2012, and FHWA Report No. FHWA-HIF-16-002 published in December 2015. The previous development and maintenance of the Handbook by the FHWA, their consultants, and their technical reviewers is gratefully appreciated and acknowledged.

This current edition of the Handbook is maintained by the National Steel Bridge Alliance (NSBA), a division of the American Institute of Steel Construction (AISC). This Handbook, published in 2021, has been updated and revised to be consistent with the 9th edition of the AASHTO LRFD Bridge Design Specifications which was released in 2020. The updates and revisions to various chapters and design examples have been performed, as noted, by HDR, M.A. Grubb & Associates, Don White, Ph.D., and NSBA. Furthermore, the updates and revisions have been reviewed independently by Francesco Russo, Ph.D., P.E., Brandon Chavel, Ph.D., P.E., and NSBA.

The Handbook consists of 19 chapters and 6 design examples. The chapters and design examples of the Handbook are published separately for ease of use, and available for free download at the NSBA website, www.aisc.org/nsba.

The users of the Steel Bridge Design Handbook are encouraged to submit ideas and suggestions for enhancements that can be implemented in future editions to the NSBA and AISC at solutions@aisc.org.

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7. Supplementary Notes The previous edition of this Handbook was published as FHWA-HIF-16-002 and was developed to be current with the 7th edition of the AASHTO LRFD Bridge Design Specifications. This edition of the Handbook was updated to be current with the 9th edition of the AASHTO LRFD Bridge Design Specifications, released in 2020.	
8. Abstract The purpose of this example is to illustrate the use of the AASHTO LRFD Bridge Design for the design of a two-span continuous steel I-girder bridge. The design process and corresponding calculations for the steel I-girders are the focus of this example, with particular emphasis placed on illustration of the optional moment redistribution procedures in Appendix B6 of the specifications. All aspects of the girder design are presented, including evaluation of the following: cross-section proportion limits, and constructability, service, fatigue, and strength limit state requirements. Also illustrated in this example are cross-frame and bearing stiffener designs along with the design of the flange-to-web welds.	
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Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge

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1.0 INTRODUCTION

The purpose of this example is to illustrate the use of the Ninth Edition of the *AASHTO LRFD Bridge Design Specifications* (2020) [1], referred to herein as *AASHTO LRFD BDS* for the design of a two-span continuous steel I-girder bridge. The design process and corresponding calculations for steel I-girders are the focus of this example, with particular emphasis placed on illustration of the optional moment redistribution procedures given in Appendix B6 of *AASHTO LRFD BDS*. It is important to note that the use of the optional provisions of Appendix B6 in the design should only be undertaken with the full knowledge and consent of the Owner. All aspects of the girder design are presented in this example, including evaluation of the following: cross-section proportion limits, and constructability, service, fatigue, and strength limit state requirements. Also illustrated in this example are cross-frame and bearing stiffener designs, along with the design of the flange-to-web welds.

The optional moment redistribution procedures in Appendix B6 allow for a limited degree of yielding at the interior supports of continuous-span girders. The subsequent redistribution of moment results in a decrease in the negative bending moments and a corresponding increase in positive bending moments. The current moment redistribution procedures utilize the same moment envelopes as used in a conventional elastic analysis and do not require the use of iterative procedures or simultaneous equations. The use of inelastic design procedures may offer cost savings by (1) requiring smaller girder sizes, (2) eliminating the need for cover plates (which have unfavorable fatigue characteristics) in rolled beams, and (3) reducing the number of flange transitions without increasing the amount of material required in plate girder designs, leading to both material and, more significantly, fabrication cost savings. As noted previously, the use of moment redistribution and the optional provisions of Appendix B6 in the design should only be undertaken with the full knowledge and consent of the Owner.

2.0 DESIGN PARAMETERS

The bridge cross-section for the tangent, two-span (90.0 feet – 90.0 feet) continuous bridge under consideration is given below in Figure 1. The example bridge has four plate girders spaced at 10.0 feet and 3.5-foot deck overhangs. The roadway width is 34.0 feet and is centered over the girders. The reinforced concrete deck is 8.5-inches thick, including a 0.5-inch integral wearing surface. The deck haunch thickness is 2.0 inches measured from the top of the girder web to the bottom of the concrete deck.

The framing plan for this design example is shown in Figure 2. As will be demonstrated subsequently, the cross-frame spacing is governed by constructability requirements in positive bending and by moment redistribution requirements in negative bending.

The structural steel is ASTM A709, Grade 50W, and the concrete is normal weight with a 28-day compressive strength, f'_c, of 4.0 ksi. The concrete slab is reinforced with nominal Grade 60 reinforcing steel.

The design specifications are the 9th Edition *AASHTO LRFD BDS* (2020). Unless stated otherwise, the specific articles, sections, and equations referenced throughout this example are contained in these specifications.

The girder design presented herein is based on the premise of providing the same girder design for both the interior and exterior girders. Thus, the design satisfies the requirements for both interior and exterior girders. Additionally, the girders are designed assuming composite action with the concrete slab in both the positive and negative bending regions.

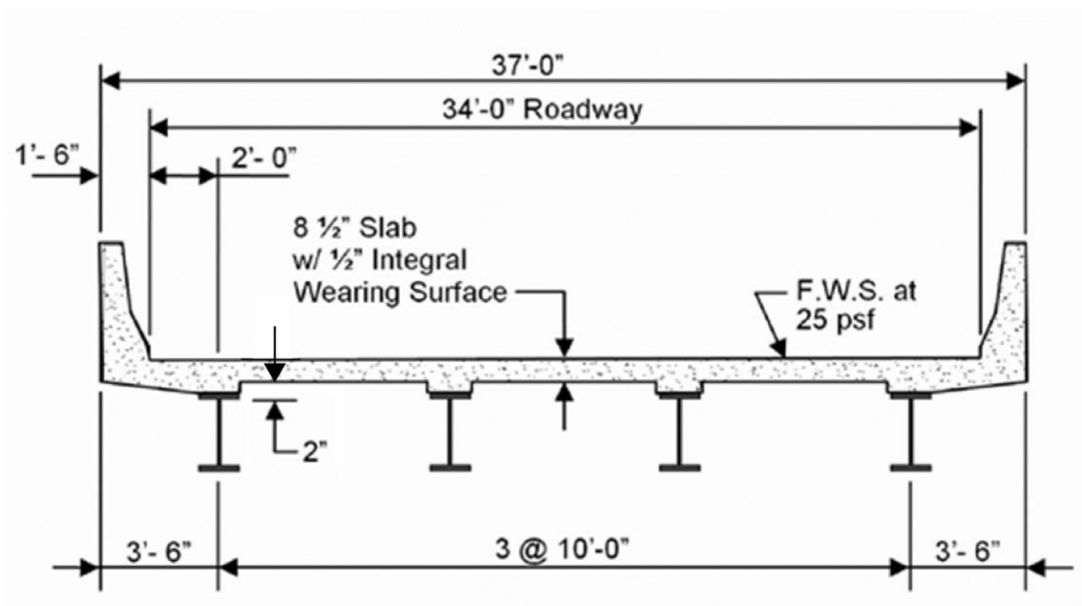

Figure 1 Sketch of the Typical Bridge Cross Section

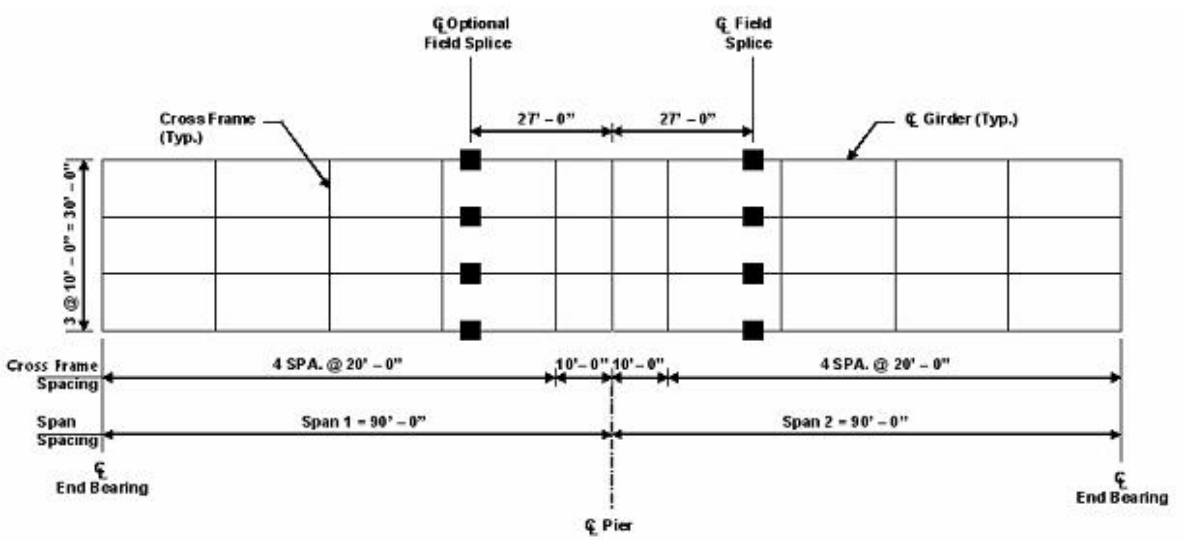

Figure 2 Sketch of the Superstructure Framing Plan

3.0 GIRDER GEOMETRY

The girder elevation is shown in Figure 3. As shown in Figure 3, section transitions are provided at 30% of the span length (27 feet) from the interior pier. The design of the girder from the abutment to 63 feet from the abutment is primarily based on positive bending moments; thus, this section of the girder is referred to as either the "positive bending region" or "Section 1" throughout this example. Alternatively, the girder geometry at the pier is controlled by negative bending moments; consequently, the region of the girder extending from 0 to 27 feet on each side of the pier will be referred to as the "negative bending region" or "Section 2". The rationale used to develop the cross-sectional geometry of these sections and a demonstration that this geometry satisfies the cross-section proportion limits specified in Article 6.10.2 is presented herein.

3.1 Web Depth

Selection of appropriate web depth has a significant influence on girder geometry. Deeper girders not only lead to a stiffer bridge but result in flanges that meet specified depth-to-width limits and girders that are easier to handle. The chosen depth also dictates the flange sizes. Clearance restrictions or poor span ratios in continuous-span structures can prevent the use of the desired depth. However, in the absence of such restrictions, it is usually desirable to use the near optimum depth for the largest span in the unit if feasible.

Thus, initial consideration should be given to the most appropriate web depth. In the absence of other criteria, the span-to-depth ratios given in Article 2.5.2.6.3 may be used as a starting point for selecting a web depth. Unless specified otherwise by the Owner-agency, these are only suggested and not required minimum depths; the Engineer is otherwise permitted to use a depth that is shallower than these suggested minimums, and in some cases, may be forced to do so by other constraints. However, when depths below these suggested minimums must be used, additional attention should be paid to the structure deformations and cross-frame forces. The most important thing to keep in mind is that the optimum depth will typically be larger than the suggested minimum depths.

As provided in Table 2.5.2.6.3-1, the suggested minimum depth of the steel I-beam portion of a continuous-span composite section is 0.027L, where L is the span length. Thus, the suggested minimum steel depth is computed as follows.

$$0.027(90\text{ ft})(12\text{ in./ft}) = 29.2\text{ inches}$$

For simplicity, it is recommended that the suggested minimum depth be applied to the web depth rather than to the total depth of the girder. Preliminary designs were evaluated for five different web depths satisfying the above requirement. These web depths varied between 36 inches and 46 inches and in all cases girder weight decreased as web depth increased. However, the decrease in girder weight became much less significant for web depths greater than 42 inches.

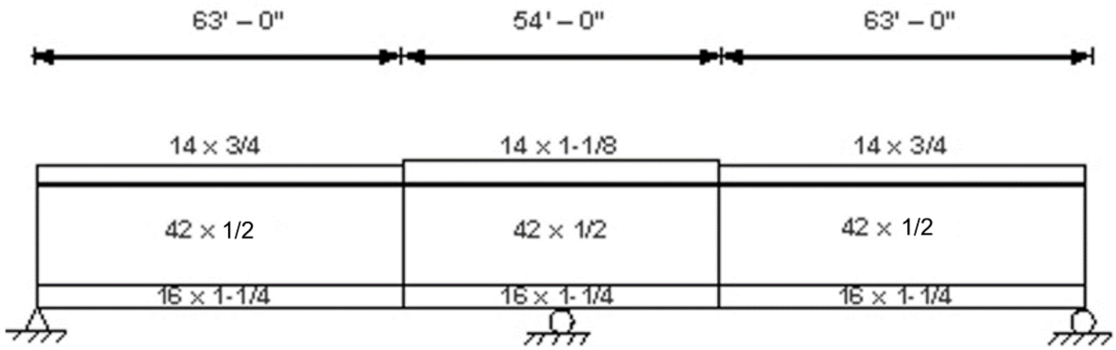

Figure 3 Sketch of the Girder Elevation

3.2 Web Thickness

In developing the preliminary cross-section it should also be verified that the selected dimensions satisfy the cross-section proportion limits specified in Article 6.10.2. The required web proportions are given in Article 6.10.2.1.1 where, for webs without longitudinal stiffeners, the web slenderness is limited to a maximum value of 150.

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

The AASHTO/NSBA Steel Bridge Collaboration G12.1 *Guidelines to Design for Constructability and Fabrication* (hereafter referred to as "the Guidelines") [2] recommend a minimum web thickness of 0.5 inches to reduce deformation and the potential for weld defects as well as to provide increased corrosion resistance. The following calculation demonstrates that Eq. 6.10.2.1.1-1 is satisfied for a web thickness of 0.5 inch.

$$\frac{D}{t_w} = \frac{42}{0.5} = 84 < 150 \qquad \text{(satisfied)}$$

3.3 Flange Geometries

The width of the compression flange and cross-frame spacing in the positive bending region is controlled by constructability requirements. Various cross-frame spacings were investigated and the corresponding flange width required to satisfy constructability requirements for each case was determined. Based on these investigations, it was determined that a minimum flange width of 14 inches was required to avoid the use of additional cross-frames. Thus, this minimum width was used for the top flanges. The Guidelines recommend a minimum flange thickness of 0.75 inches for the same reasons discussed previously for webs. Therefore, use $(t_f)_{min}$ = 0.75 inches.

All other plate sizes were iteratively selected to satisfy all applicable requirements while producing the most economical girder design possible. The resulting girder dimensions are shown in Figure 3.

Article 6.10.2.2 specifies four flange proportion limits that must be satisfied. The first of these is intended to prevent the flange from excessively distorting when welded to the web of the girder during fabrication.

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

Evaluation of Eq. 6.10.2.2-1 for each of the three flange sizes used in the example girder is demonstrated below.

$$\frac{b_f}{2t_f} = \frac{14}{2(0.75)} = 9.33 < 12.0 \qquad \text{(satisfied)}$$

$$\frac{b_f}{2t_f} = \frac{14}{2(1.125)} = 6.22 < 12.0 \qquad \text{(satisfied)}$$

$$\frac{b_f}{2t_f} = \frac{16}{2(1.25)} = 6.4 < 12.0 \qquad \text{(satisfied)}$$

The second flange proportion limit that must be satisfied corresponds to the relationship between the flange width and the web depth. The ratio of the web depth to the flange width significantly influences the flexural resistance of the member and is limited to a maximum value of 6, which is the maximum value for which the flexural resistance prediction equations for steel I-girders are proven to be valid.

$$b_f \geq \frac{D}{6} = \frac{42}{6} = 7.0 \qquad \text{Eq. (6.10.2.2-2)}$$

It is shown below that Eq. 6.10.2.2-2 is satisfied for both flange widths utilized in this design example.

b_f = 14.0 inches (satisfied)
b_f = 16.0 inches (satisfied)

Equation 3 of Article 6.10.2.2 limits the thickness of the flange to a minimum of 1.1 times the web thickness. This requirement is necessary to verify that some web shear buckling restraint is provided by the flanges, and that the boundary conditions at the web-flange junction assumed in the development of the web-bend buckling and flange local buckling resistance equations are sufficiently accurate.

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

Evaluation of Eq. 6.10.2.2-3 for the minimum flange thickness used in combination with the web thickness utilized in the example girder is demonstrated below.

$$t_f = t_{f\text{-min}} = 0.75\text{ in.} > 1.1(0.5\text{ in.}) = 0.55\text{ in.} \qquad \text{(satisfied)}$$

Equation 6.10.2.2-4 prevents the use of extremely monosymmetric sections ensuring more efficient flange proportions and a girder section that is suitable for handling during erection.

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

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})

Computing the ratio between the top and bottom flanges for the positive and negative bending regions, respectively, shows that this requirement is satisfied.

$$0.1 < \frac{(0.75)(14)^3/12}{(1.25)(16)^3/12} = \frac{171.5}{426.7} = 0.40 < 10 \qquad \text{(satisfied)}$$

$$0.1 < \frac{(1.125)(14)^3/12}{(1.25)(16)^3/12} = \frac{257.25}{426.7} = 0.60 < 10 \qquad \text{(satisfied)}$$

Article C6.10.2.2 provides the following additional guideline for the minimum top-flange width, b_{tfs}, within an individual unspliced girder field section. This guideline, which should be considered in conjunction with the flange proportioning limits specified in Article 6.10.2.2, is intended to provide more stable field pieces that are easier to handle during fabrication and erection without the need for special stiffening trusses or falsework:

$$\left(b_{tfs}\right)_{min} \ge \frac{L_{fs}}{85} \qquad \text{Eq. (C6.10.2.2-1)}$$

where L_{fs} is the length of the unspliced girder field section in feet. This equation is provided as a guideline and is not considered a mandatory requirement, but satisfying this proportional limit is strongly encouraged.

The guideline is applied to the top-flange width because the top flange of each girder field section is subject to compression over its entire length during lifting, erection, and shipping regardless of the final location of the field section in the bridge. The bottom flange is also typically either wider or of the same width as the top flange in most typical field sections. The guideline is also applied to unspliced girder field sections rather than to girder shipping pieces during the design. It is not intended in the application of this guideline that the Engineer attempt to anticipate how the individual field sections may eventually be assembled or spliced together and/or stabilized or supported for shipping or erection; such concerns should instead be considered the responsibility of the Contractor.

From Figure 3, the length of the longest unspliced girder field section is 63 feet. Therefore, applying the guideline for this field section gives:

$$\left(b_{tfs}\right)_{min} = \frac{L}{85} = \frac{63}{85} = 0.74\text{ ft} = 8.9\text{ in.} \qquad \text{(satisfied)}$$

The Guidelines contain 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. See also the discussion in NSBA's *Steel Bridge Design Handbook: Design Example 1* [3]. Fabricators can also be consulted regarding these issues and all other fabrication-related issues discussed herein.

The total estimated weight of structural steel based on the trial girder size (with all girders in the cross-section assumed to be the same size) is 24.4 lbs/ft^2 of deck area. The NSBA website (www.aisc.org/nsba) provides Steel Span to Weight Curves [4], which allow the designer to quickly estimate the weight per square foot of deck for straight, low skew plate-girder bridges for various span lengths and girder spacings. Referring to Figure 4, the curve gives a steel weight per square foot of deck area of approximately 23 lbs/ft^2 for a 90-ft span. Therefore, the trial girder appears to be a reasonable starting design.

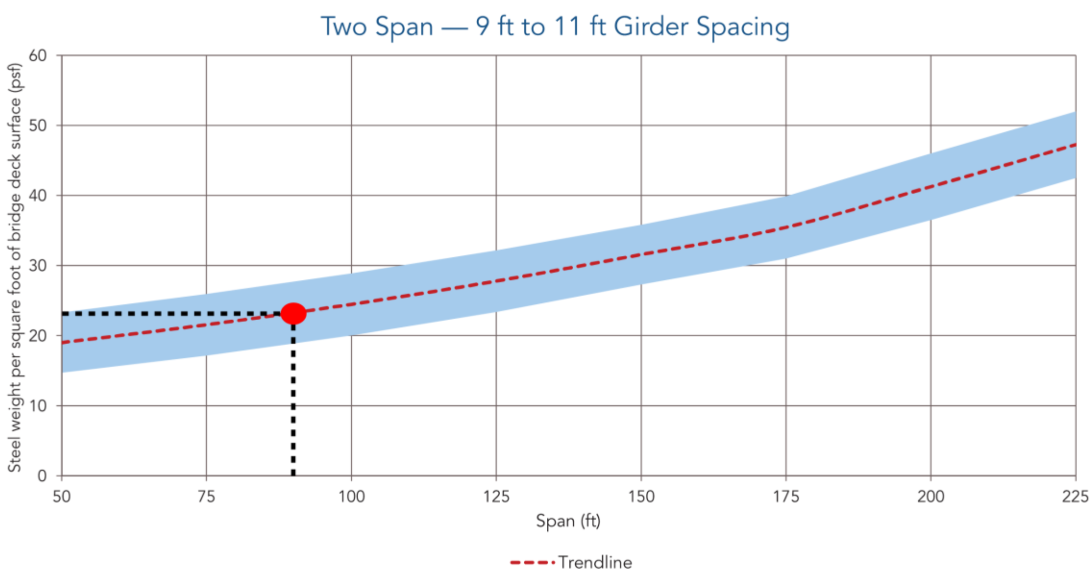

Figure 4 Steel Span to Weight Curve for Design Bridge

4.0 LOADS

This example considers all applicable loads acting on the superstructure including dead loads, live loads, and wind loads as discussed below. In determining the effects of each of these loads, the approximate methods of analysis specified in Article 4.6.2 are implemented.

4.1 Dead Loads

The dead load, according to Article 3.5.1, is to include the weight of all components of the structure, appurtenances and utilities, earth cover, wearing surface, future overlays, and planned widening. Dead loads are divided into two categories: dead load of structural components and non-structural attachments (DC) and the dead load of wearing surface and utilities (DW). Alternative load factors are specified for each of these categories of dead load depending on the load combination under consideration.

4.1.1 Component Dead Load (DC)

For composite girders consideration is given to the fact that not all dead loads are applied to the composite section and the DC dead load is separated into two parts: the dead load acting on the section before the concrete deck is hardened or made composite (DC_1), and the dead load acting on the composite section (DC_2). DC_1 is assumed to be carried by the steel section alone. DC_2 is assumed to be carried by the long-term composite section. In the positive bending region, the long-term composite section is comprised of the steel girder and an effective width of the concrete slab. Article 4.6.2.6.1 specifies the effective slab width over which a uniform stress distribution may be assumed, which in most cases may be taken as the tributary width of the slab perpendicular to the axis of the member. The effective width of the concrete slab is transformed into an equivalent area of steel by dividing the width by the ratio between the steel modulus and one-third the concrete modulus, or a modular ratio of 3n. The reduced concrete modulus is intended to account for the effects of concrete creep. In the negative bending region at the strength limit state, the composite section is comprised of the steel section and the longitudinal steel reinforcing within the effective width of the slab. At the fatigue and service limit states, the concrete deck may be considered effective in both negative and positive bending for loads applied to the composite section if certain specified conditions are met.

DC_1 includes the girder self-weight, weight of the concrete slab (including the haunch and deck overhang taper if present), deck forms, cross-frames, and stiffeners. The unit weight for steel (0.490 k/ft^3) used in this example is taken from Table 3.5.1-1, which provides approximate unit weights of various materials. Table 3.5.1-1 also lists the unit weight of normal weight concrete as 0.145 k/ft^3; the concrete unit weight is increased to 0.150 k/ft^3 in this example to account for the additional weight of the steel reinforcement within the concrete. The dead load of the stay-in-place forms is assumed to be 15 psf. To account for the dead load of the cross-frames, stiffeners and other miscellaneous steel details, a dead load of 0.015 k/ft is assumed. It is also assumed that these dead loads are equally distributed to all girders as permitted by Article 4.6.2.2.1 for the line-girder type of analysis implemented herein. Thus, the total DC_1 loads used in this design are as computed below.

Slab = (8.5/12) x (37.0) x (0.150)/4	= 0.983 k/ft
Haunch (average weight/length)	= 0.015 k/ft
Overhang taper = 2 x (1/2) x [3.5 - (7.0/12)] x (2.0/12) x 0.150/4	= 0.018 k/ft
Girder (average wt/length)	= 0.180 k/ft
Cross-frames and misc. steel	= 0.015 k/ft
Stay-in-place forms = 0.015 x (30 - 3 x (14.0/12))/4	= 0.100 k/ft
Total DC_1	= 1.311 k/ft

DC_2 is composed of the weight from the barriers, medians, and sidewalks. No sidewalks or medians are present in this example and thus the DC_2 weight is equal to the barrier weight alone. The barrier weight is assumed to be equal to 520 lb/ft. Article 4.6.2.2.1 specifies that when approximate methods of analysis are applied DC_2 may be equally distributed to all girders, or else different, semi-arbitrary proportions of the concrete barrier load may be applied to the exterior girder and to the adjacent interior girder which represents a more realistic distribution of these loads acting out on the deck overhangs (particularly in wider bridges with more girders in the cross-section). Since this example features a relatively narrow deck and only four girders in the cross-section, it is reasonable to assume that the barrier weight can be equally distributed to all girders, resulting in the DC_2 loads computed below.

Barriers = (0.520 x 2)/4 = 0.260 k/ft

DC_2 = 0.260 k/ft

4.1.2 Wearing Surface Dead Load (DW)

Similar to the DC_2 loads, the dead load of the future wearing surface is applied to the long-term composite section and is assumed to be equally distributed to each girder. A future wearing surface with a dead load of 25 psf is assumed. Multiplying this unit weight by the roadway width and dividing by the number of girders gives the following.

Wearing surface = (0.025) x (34)/4 = 0.213 k/ft

DW = 0.213 k/ft

4.2 Vehicular Live Loads

The *AASHTO LRFD BDS* considers live loads to consist of gravity loads, wheel load impact (dynamic load allowance), braking forces, centrifugal forces, and vehicular collision forces. Live loads are applied to the short-term composite section. In positive bending regions, the short-term composite section is comprised of the steel girder and the effective width of the concrete slab, which is converted into an equivalent area of steel by dividing the width by the modular ratio, or the ratio of the elastic moduli of the steel and the concrete. In other words, a modular ratio of n is used for short-term loads where creep effects are not relevant. In negative bending regions at the strength limit state, the short-term composite section consists of the steel girder and the longitudinal reinforcing steel. At the fatigue and service limit states, the concrete deck may be considered effective in both negative and positive bending if certain specified conditions are met.

4.2.1 General Vehicular Live Load (Article 3.6.1.2)

The *AASHTO LRFD BDS* vehicular live loading is designated as the HL-93 loading and is a combination of the design truck or tandem plus the design lane load. The design truck, specified in Article 3.6.1.2.2, is composed of an 8-kip lead axle spaced 14 feet from the closer of two 32-kip rear axles, which have a variable axle spacing of 14 feet to 30 feet. The transverse spacing of the wheels is 6 feet. The design truck occupies a 10 feet lane width and is positioned within the design lane to produce the maximum force effects but may be no closer than 2 feet from the edge of the design lane, except for the design of the deck overhang.

The design tandem, specified in Article 3.6.1.2.3, is composed of a pair of 25-kip axles spaced 4 feet apart. The transverse spacing of the wheels is 6 feet.

The design lane load is discussed in Article 3.6.1.2.4 and has a magnitude of 0.64 klf uniformly distributed in the longitudinal direction. In the transverse direction, the load occupies a 10-foot width. The lane load is positioned to produce extreme force effects, and therefore, need not be applied continuously.

For both negative moments between points of contraflexure and interior pier reactions, a special loading is used. The loading consists of two design trucks (as described above but with a magnitude of 90% of the axle weights) in addition to 90% of the lane loading. The trucks must have a minimum headway of 50 feet between the lead axle of the second truck and the rear axle of the first truck (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. The live load moments between the points of dead load contraflexure are to be taken as the larger of the moments caused by the HL-93 loading or the special loading.

Live load shears are to be calculated only from the HL-93 loading, except for interior pier reactions, which are to be taken as the larger of the reactions due to the HL-93 loading or the special loading.

The dynamic load allowance, which accounts for the amplification of the live loads due to dynamic effects, is only applied to the truck or tandem portion of the live loading, as applicable, and not to the lane load. For the strength and service limit states, the dynamic load allowance is taken as 33 percent, and for the fatigue limit state, the dynamic load allowance is taken as 15 percent.

4.2.2 Optional Live Load Deflection Load (Article 3.6.1.3.2)

The loading for the optional live load deflection criterion consists of the greater of the design truck, or 25 percent of the design truck plus the lane load. A dynamic load allowance of 33 percent applies to the truck portions (axle weights) of these load cases. During this check, all design lanes are to be loaded, and the assumption is made for straight-girder bridges with limited or no skew that all components deflect equally.

4.2.3 Fatigue Load (Article 3.6.1.4)

For checking the fatigue limit state, a single design truck with a constant rear axle spacing of 30 feet is applied. Note, again, that the dynamic load allowance is taken as 15 percent.

4.3 Wind Loads

Wind loading is to be considered when calculating force effects and deflections in the noncomposite steel girders prior to deck placement (i.e., wind loading acting on the fully erected steel frame), and during the deck placement before the top flanges are continuously braced by the concrete deck. In the final constructed condition after the deck is placed, wind loading is to be considered when determining flange lateral bending moments and stresses in the exterior girder bottom flange, as well as forces in the cross-frame members due to loading on the exterior girder web. Article C4.6.2.7.1 provides approximate methods for determining these wind-load force effects.

Article 3.8.1.2.1 discusses the static design horizontal wind pressure, P_Z, which is used to determine the wind load on the structure (WS). The design wind pressure is computed as follows:

$$P_z = 2.56 \text{ x } 10^{-6} V^2 K_z G C_D \qquad \text{Eq. (3.8.1.2.1-1)}$$

where:

- V = design 3-second gust wind speed specified in Table 3.8.1.1.2-1 (mph)
- K_z = pressure exposure and elevation coefficient taken equal to K_Z (B), K_Z (C), or K_Z (D) determined using Eqs. 3.8.1.2.1-2, 3.8.1.2.1-3, or 3.8.1.2.1-4, respectively, for the Strength III and Service IV load combination and to be taken as 1.0 for other load combinations
- G = gust effect factor determined using a structure-specific study or as specified in Table 3.8.1.2.1-1 for the Strength III load combination and 1.0 for other load combinations
- C_D = drag coefficient using a structure-specific study or as specified in Table 3.8.1.2.1-2

In this example, it is assumed that the average height of top of the superstructure is 30 feet above the surrounding ground and that it is located in western New York in a suburban area. As specified in Table 3.8.1.1.2-1, for the Strength III load combination (Table 3.4.1-1), the design 3-second gust wind speed, V, is to be determined from Figure 3.8.1.1.2-1; for western New York, V is taken as 115 mph. For the Strength V load combination (Table 3.4.1-1), V is taken as 80 mph (Table

3.8.1.1.2-1). An increase in the wind speed based on a site-specific wind study is assumed not to be warranted for this site.
For typical bridges, such as the bridge in this design example, the wind exposure category is to be determined perpendicular to the bridge (Article 3.8.1.1.3). Wind Exposure Category B is assumed (Article 3.8.1.1.5) since the Ground Surface Roughness Category B in this case is assumed to prevail in the upwind direction for a distance greater than 1,500 feet. Ground Surface Roughness Category B applies to bridges located in urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger (Article 3.8.1.1.4). For the Strength III load combination, the pressure exposure and elevation coefficient for Wind Exposure Category B, K_Z (B), is equal to 0.71 (Table C3.8.1.2.1-1). This value is computed from Eq. 3.8.1.2.1-2 using a structure height, Z, equal to 33.0 feet (note that a value of Z less than 33.0 feet is not to be used in computing K_Z). For the Strength V load combination, K_Z is to be taken equal to 1.0.

Since sound barriers are assumed not to be present and a structure-specific study is assumed not to be warranted for the example bridge, the gust effect factor, G, for the Strength III load combination is taken equal to 1.0 (Table 3.8.1.2.1-1). For the Strength V load combination, G is to be taken equal to 1.0. The drag coefficient, C_D, is taken equal to 1.3 for both the Strength III and Strength V load combinations (Table 3.8.1.2.1-2).

Therefore, P_Z is computed as follows:

Strength III: $P_z = 2.56 \text{ x } 10^{-6}(115)^2\left(0.71\right)(1.0)(1.3) = 0.031 \text{ ksf}$

Strength V: $P_z = 2.56 \text{ x } 10^{-6}(80)^2(1.0)(1.0)(1.3) = 0.021 \text{ ksf}$

P_Z 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 wind load is to be taken as the product of the design wind pressure and the exposed area. The direction of the wind is to be varied to determine the maximum force effect in the component under investigation. The wind loads are to be taken as the algebraic transverse and longitudinal components of the wind load assumed applied simultaneously (Article 3.8.1.2.2). For a routine I-girder bridge such as the one in this example, the wind effects in the girder flanges and cross-frames are controlled by wind acting perpendicular to the bridge; other wind skew angles do not need to be investigated. However, for the shorter-span bridge in this example, wind load effects on the girders are not expected to be significant during construction or at the strength limit state and are not evaluated herein; refer to NSBA's *Steel Bridge Design Handbook: Design Example 1* [3] for an illustration of these checks.

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 0.10 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. The applied wind on live load does not have a measurable influence on the design force effects in the girders or in the intermediate cross-frames. Wind on live load is primarily a design consideration for bearing and substructure design. However, the transmission of the load from the superstructure (resisted by diaphragm action of the concrete deck) to the bearings though the cross-frames or diaphragms at the supports must be considered in the design of those elements. Similar to wind load acting on the superstructure, wind on live load acting perpendicular to the bridge is generally the controlling direction for the design of cross-frames or diaphragms at the supports.

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 (i.e., the Strength III load combination only), a vertical wind pressure of 0.020 ksf times the entire width of the deck, including parapets and sidewalks, is to be applied as a vertical upward line load at the windward quarter-point of the deck width in combination with the horizontal wind loads to investigate potential overturning of the bridge (Article 3.8.2). The effect of this uplift wind load case on the superstructure design is negligible but must be considered in the design of the bearings and substructure; this load case is not investigated in this example.

4.4 Load Combinations

The specifications define four limit states: the service limit state, the fatigue and fracture limit state, the strength limit state, and the extreme event limit state. Section 7.0 discusses each limit state in more detail; however, for all limit states the following general equation from Article 1.3.2.1 must be satisfied, where different combinations of loads (i.e., dead load, live load, wind load) are specified for each limit state.

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

where:

η_i = Factor related to ductility, redundancy, and operational importance (Articles 1.3.3 through 1.3.5)
γ_i = Load factor
Q_i = Force effect
ϕ = Resistance factor
R_n = Nominal resistance
R_r = Factored resistance

The load factors for the load combinations to be considered at each limit state are given in Tables 3.4.1-1 and 3.4.1-2 of the specifications and the resistance factors for the design of steel members are given in Article 6.5.4.2. Refer to NSBA's *Steel Bridge Design Handbook: Design Example 1* [3] for detailed descriptions of each of the load combinations.

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

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

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

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

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

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

For typical bridges for which additional ductility-enhancing measures have not been provided beyond those required by the specifications, and/or for which exceptional levels of redundancy are not provided, the η_D and η_R factors have default values of 1.0 specified at the strength limit state. The value of the load modifier for operational importance η_I should be chosen with input from the Owner-agency. In the absence of such input, the load modifier for operational importance at the strength limit state should be taken as 1.0. At all other limit states, all three η factors must be taken equal to 1.0. For this example, η_i will be taken equal to 1.0 at all limit states.

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), unless otherwise specified by the Owner. Also, the load factor for the weight of the structure and appurtenances, DC and DW, is not to be taken less than 1.25 when evaluating the construction condition.

The load factor for wind during construction in the Strength III load combination is to be specified by the Owner. Any applicable construction loads are to be included with a load factor not less than 1.25. Again, the load factor for the weight of the structure and appurtenances, DC and DW, is not to be taken less than 1.25 when evaluating the construction condition. Refer to the AASHTO *Guide Specifications for Wind Loads on Bridges During Construction* [5] for further information on evaluating wind load effects during construction.

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

5.0 STRUCTURAL ANALYSIS

The *AASHTO LRFD BDS* allows the designer to use either approximate (e.g., line girder) or refined (e.g., grid or finite element) analysis methods to determine force effects; the acceptable methods of analysis are detailed in Section 4 of the specifications. In this design example, a line girder analysis is employed to determine the girder moment and shear envelopes. Using the line girder approach, vehicular live load force effects are determined by first computing the force effects due to a single truck or loaded lane and then by multiplying these forces by multiple presence factors, live-load distribution factors, and dynamic load allowance factors as detailed below.

5.1 Multiple Presence Factors (Article 3.6.1.1.2)

Multiple presence factors account for the probability of multiple lanes on the bridge being loaded simultaneously. These factors are specified for various numbers of loaded lanes in Table 3.6.1.1.2-1 of the specifications. There are two exceptions when multiple presence factors are not to be applied. These are when (1) distribution factors are calculated using the tabulated empirical equations given in Article 4.6.2.2 as these equations are already adjusted to account for multiple presence effects, and (2) when determining fatigue truck moments, since the fatigue analysis is only specified for a single truck. 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 (described further below), 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. Thus, for the present example, the multiple presence factors are only applicable when distribution factors are computed using the lever rule or the special analysis for the exterior girders at the strength and service limit states as demonstrated below.

5.2 Live-Load Distribution Factors (Article 4.6.2.2)

The distribution factors approximate the amount of live load (i.e., percentage of a truck or lane load) distributed to a given girder. These factors are computed based on a combination of empirical equations and simplified analysis procedures. Empirical equations are provided in Article 4.6.2.2.1 of the specifications and are specifically developed based on the location of the girder (i.e., interior or exterior), the force effect considered (i.e., moment or shear), and the bridge type. These equations are valid only if specific parameters of the bridge are within the ranges specified in the tables given in Article 4.6.2.2.1. If the limits are not satisfied, a more refined analysis must be performed. This design example satisfies all limits for use of the empirical distribution factors, in addition to the conditions listed in Article 4.6.2.2.1, and therefore, the analysis using the approximate equations and simplified analysis procedures of Article 4.6.2.2 may proceed as follows.

Distribution factors are a function of the girder spacing, slab thickness, span length, and the stiffness of the girder, which depends on the proportions of the section. Since the factor depends on girder proportions that are not initially known, the stiffness term $(K_g/12.0Lt_s^3)^{0.1}$ in the following equations may be taken as 1.02 (Table 4.6.2.2.1-3) for preliminary design when permitted by the Owner. In this section, calculation of the distribution factors is presented based on the girder proportions previously shown in Figure 3.

5.2.1 Live-Load Lateral Distribution Factors – Positive Flexure

In positive bending regions, the stiffness parameter required for the distribution factor equations, K_g, is determined based on the cross-section shown in Figure 5.

$$K_g = n(I + Ae_g^2) \qquad \text{Eq. (4.6.2.2.1-1)}$$

where:

- n = modular ratio (= 8)
- I = moment of inertia of the steel girder
- A = area of the steel girder
- e_g = distance between the centroid of the girder and centroid of the slab

Thus, K_g is determined as follows:

$$e_g = 8.0 / 2 + 2.0 + 25.79 - 0.75 = 31.04 \text{ in.}$$

$$n = 8$$

$$K_g = n(I + Ae_g^2) = 8(16{,}401 + 51.50(31.04)^2) = 528{,}162 \text{ in.}^4$$

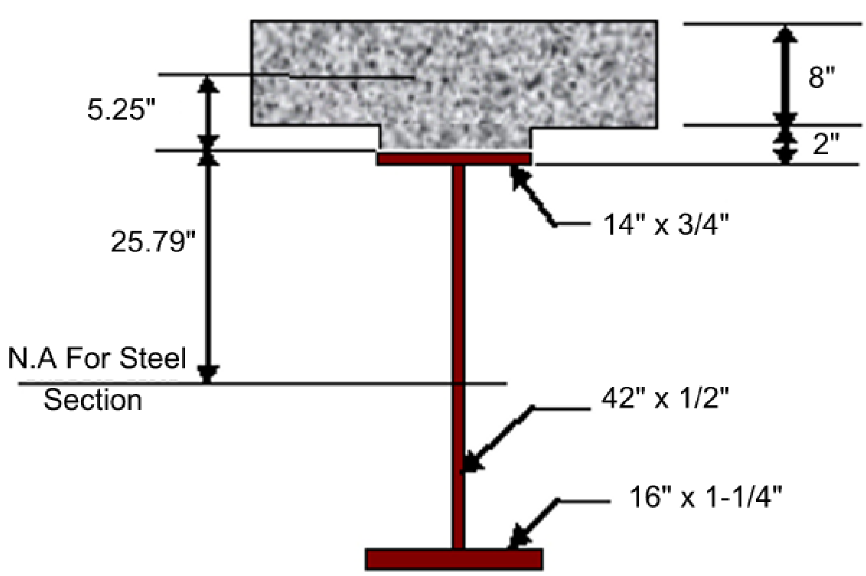

Figure 5 Sketch of Section 1, Positive Bending Region

Table 1 Section 1 Steel Only Section Properties

Component	A	d	Ad	Ad^2	I_o	I
Top Flange 14" x ¾"	10.50	21.38	224.5	4797	0.49	4798
Web 42″ x ½"	21.00				3087	3087
Bottom Flange 16" x 1-¼"	20.00	-21.63	-432.5	9353	2.60	9356
Σ	51.50		-208.0			17,241

-4.04(208) = -840

I_{NA} = 16,401 in.^{4}

$$d_s = \frac{-208.0}{51.50} = -4.04 \text{ in.}$$

$$d_{\text{Top of Steel}} = 21.75 + 4.04 = 25.79 \text{ in.}$$

$$d_{\text{Bot of Steel}} = 22.25 - 4.04 = 18.21 \text{ in.}$$

$$S_{\text{Top of Steel}} = \frac{16,401}{25.79} = 635.9 \text{ in.}^3$$

$$S_{\text{Bot of Steel}} = \frac{16,401}{18.21} = 900.7 \text{ in.}^3$$

5.2.1.1 Interior Girder – Strength and Service Limit States

For interior girders, computation of the distribution factors for the strength and service limit states is based on the empirical equations given in Article 4.6.2.2.2 as described below.

5.2.1.1.1 Bending Moment

The empirical equations for distribution of live load moment at the strength and service limit states are given in Table 4.6.2.2.2b-1. Alternative expressions are given for one loaded lane and multiple loaded lanes, where the maximum of the two equations governs as shown below. It is noted that the maximum number of design lanes possible for the 34-foot roadway width considered in this example is two lanes.

$$DF = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1} \text{ for one-lane loaded}$$

where:

- S = girder spacing
- L = span length
- t_s = slab thickness
- K_g = stiffness term

$$DF = 0.06 + \left(\frac{10.0}{14}\right)^{0.4}\left(\frac{10.0}{90}\right)^{0.3}\left(\frac{528162}{12.0(90)(8.0)^3}\right)^{0.1} = 0.510 \text{ lanes}$$

$$DF = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt^3}\right)^{0.1} \text{ for two or more lanes loaded}$$

$$DF = 0.075 + \left(\frac{10.0}{9.5}\right)^{0.6}\left(\frac{10.0}{90}\right)^{0.2}\left(\frac{528162}{12.0(90)(8.0)^3}\right)^{0.1} = 0.736 \text{ lanes}$$

(governs)

Thus, the controlling distribution factor for moment of an interior girder in the positive moment region at the strength or service limit state is 0.736 lanes.

5.2.1.1.2 Shear

The empirical equations for distribution of live load shear in an interior girder at the strength and service limit states are given in Table 4.6.2.2.3a-1. Similar to the equations for moment given above, alternative expressions are given based on the number of loaded lanes.

$$DF = 0.36 + \frac{S}{25.0} \text{ for one lane loaded}$$

$$DF = 0.36 + \frac{10.0}{25.0} = 0.760 \text{ lanes}$$

$$DF = 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^2 \text{ for two or more lanes loaded}$$

$$DF = 0.2 + \frac{10.0}{12} - \left(\frac{10.0}{35}\right)^2 = 0.952 \text{ lanes}$$ (governs)

5.2.1.2 Exterior Girder – Strength and Service Limit States

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 the entire cross section deflects and rotates as a rigid cross-section, which is required for steel-bridge cross-sections with diaphragms or cross-frames. Each method is illustrated below.

5.2.1.2.1 Bending Moment

Lever Rule:

As specified in Table 4.6.2.2.2d-1, the lever rule is the method used to determine the distribution factor for the exterior girder for the case of one-lane loaded. The lever rule assumes the deck is hinged at the interior girder, and statics is employed to determine the percentage of the truck weight resisted by the exterior girder, i.e., the distribution factor. It is specified that the truck is to be placed such that the closest wheel is two feet from the barrier or curb, which results in the truck position shown in Figure 6 for the present example. The calculated reaction of the exterior girder is multiplied by the multiple presence factor for one lane loaded, m_1, to determine the distribution factor.

$$DF = \left(0.5 + 0.5\left(\frac{10-6}{10}\right)\right)m_1$$

m_1 = 1.20 (from Table 3.6.1.1.2-1)

DF = 0.700 x 1.2 = 0.840 lanes

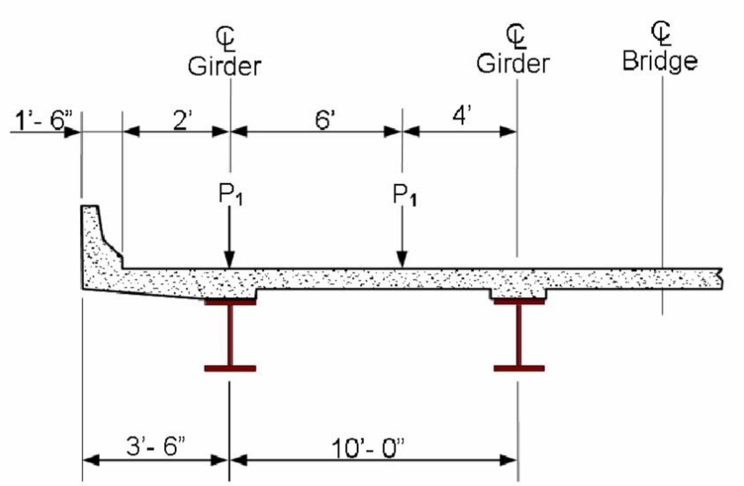

Figure 6 Sketch of the Truck Location for the Lever Rule

<u>Modified Interior Girder Distribution Factor:</u>

For the case of two or more lanes loaded, the modification factor, e, to be applied to the distribution factor for the interior girder is found in Table 4.6.2.2.2d-1 and is given below.

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

In the above equation, d_e is the horizontal distance between the centerline of the exterior girder at deck level and the interior face of the traffic barrier or curb in feet. Thus, for the present example d_e is equal to 2.0 feet.

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

Modifying the interior girder distribution factor for moment for the case of two or more lanes loaded by the factor, e, gives the following:

$DF = 0.990(0.736) = 0.729$ lanes

Special Analysis:

The special analysis assumes the entire bridge cross-section behaves as a rigid cross-section rotating about the transverse centerline of the structure and is discussed in the commentary of Article 4.6.2.2.2d. The reaction on the exterior beam is calculated from the following equation:

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

where:

- N_L = number of lanes loaded
- N_b = number of beams or girders
- X_{ext} = horizontal distance from center of gravity of the pattern of girders to the exterior girder (ft.)
- e = eccentricity of a design truck or a design lane load 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.)

Figure 7 shows the truck locations for the special analysis. Here it is shown that the maximum number of trucks that may be placed on half of the cross-section is two. Thus, the calculation of the distribution factors using the special analysis procedure for one loaded lane and two loaded lanes proceeds as follows (the appropriate multiple presence factors, MPF, that are applied in each case are shown):

$$DF = 1.2\left(\frac{1}{4} + \frac{(15.0)(12)}{2\left((15.0)^2 + (5.0)^2\right)}\right) = 0.732 \text{ lanes for one lane loaded (Note, MPF = 1.2)}$$

$$DF = 1.0\left(\frac{2}{4} + \frac{(15.0)(12.0+0)}{2\left((15.0)^2 + (5.0)^2\right)}\right) = 0.860 \text{ lanes for two lanes loaded (Note, MPF = 1.0)}$$

$DF = 0.860$ lanes (governs)

Based on the computations for the exterior girder distribution factors for moment in the positive bending region shown above, it is determined that the controlling factor for this case is equal to 0.860 lanes, which is based on the special analysis with two lanes loaded. Compared to the interior girder distribution factor for moment in the positive bending region, which was computed to be 0.736 lanes, it is shown that the exterior girder distribution factor is larger, and therefore controls the bending strength design at the strength and service limit states in the positive bending region.

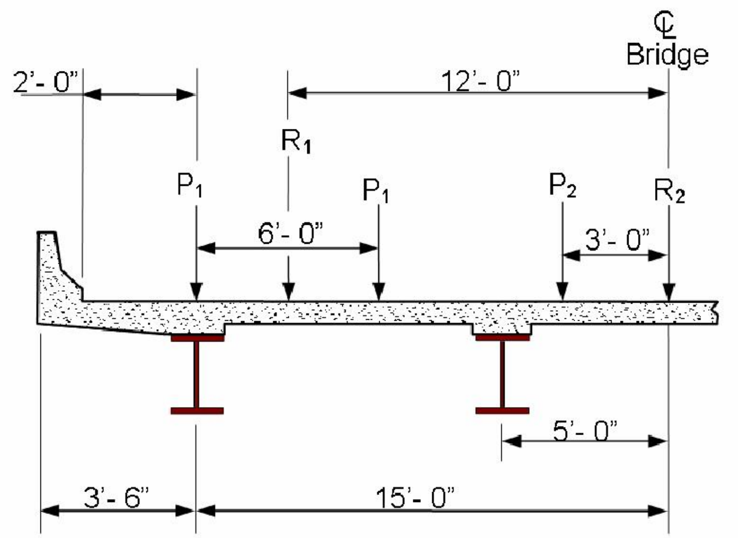

Figure 7 Sketch of the Truck Locations for the Special Analysis

5.2.1.2.2 Shear

The distribution factors computed above using the lever rule, approximate formulas, and special analysis are also applicable to the distribution of shear.

Lever Rule:

The above computations demonstrate that for the case of one-lane loaded the distribution factor is equal to 0.840 lanes based on the lever rule.

DF = 0.840 lanes

Modified Interior Girder Distribution Factor:

For the case of two or more lanes loaded, the shear modification factor is computed using the following formula, Table 4.6.2.2.3b-1:

$$e = 0.60 + \frac{d_e}{10.0}$$

$$e = 0.60 + \frac{2.0}{10.0} = 0.800$$

Applying this modification factor to the previously computed interior girder distribution factor for shear for two or more lanes loaded gives the following:

DF = 0.800(0.952) = 0.762 lanes

Special Analysis:
It was demonstrated above that the special analysis yields the following distribution factors for one lane and two or more lanes loaded, respectively:

DF = 0.732 lanes

DF = 0.860 lanes (governs)

Thus, the controlling distribution factor for shear in the positive bending region of the exterior girder is 0.860 lanes, which is less than that of the interior girder. Thus, the interior girder distribution factor of 0.952 lanes controls the shear design in the positive bending region.

5.2.1.3 Fatigue Limit State

As stated in Article 3.6.1.1.2, the fatigue distribution factor is based on one-lane loaded, and does not include the multiple presence factor, since the fatigue loading is specified as a single truck load.

5.2.1.3.1 Bending Moment

Upon reviewing the moment distribution factors for one-lane loaded computed above, it is determined that the maximum distribution factor results from the lever rule calculations. Dividing this distribution factor of 0.840 lanes by the multiple presence factor for one-lane loaded results in the following distribution factor for fatigue moment in the positive bending region.

Exterior Girder, DF = 0.840/1.20 = 0.700 lanes

Interior Girder, DF = 0.510/1.20 = 0.425 lanes

5.2.1.3.2 Shear

Similarly, based on the above distribution factors for shear due to one-lane loaded, the controlling distribution factor is calculated by again dividing the lever rule distribution factor by the multiple presence factor.

Exterior Girder, DF = 0.840/1.20 = 0.700 lanes

Interior Girder, DF = 0.760/1.20 = 0.633 lanes

5.2.1.4 Distribution Factor for Live-Load Deflection

Article 2.5.2.6.2 states that all design lanes must be loaded when determining the live load deflection of the structure. In the absence of a refined analysis, for straight-girder bridges with limited or no skew, an approximation of the live load deflection can be obtained by using a distribution factor computed assuming that all girders deflect equally with the appropriate multiple presence factor applied. The controlling case occurs when two lanes are loaded, and the calculation of the corresponding distribution factor is shown below.

$$\text{DF} = m\left(\frac{N_L}{N_b}\right) = 1.0\left(\frac{2}{4}\right) = 0.500 \text{ lanes}$$

Table 2 summarizes the governing distribution factors for the positive bending region.

Table 2 Positive Bending Region Distribution Factors (lanes)

	Interior Girder	Exterior Girder
Bending Moment	0.736	0.860
Shear	0.952	0.860
Fatigue (Bending Moment)	0.425	0.700
Fatigue (Shear)	0.633	0.700
Deflection	0.500	0.500

5.2.2 Live-Load Lateral Distribution Factors – Negative Flexure

Many of the distribution factors are the same in both the positive and negative bending regions. This section demonstrates the computation of the distribution factors that are unique to the negative bending region. Specifically, the distribution factor for bending moment in the interior girder at the strength and service limit states is directly influenced by to the girder proportions. As in the above calculations for the positive moment region, this process begins with the determination of the stiffness parameter, K_g, of the section. The cross-section for the negative bending region is shown in Figure 8. The section properties of the girder are determined as follows:

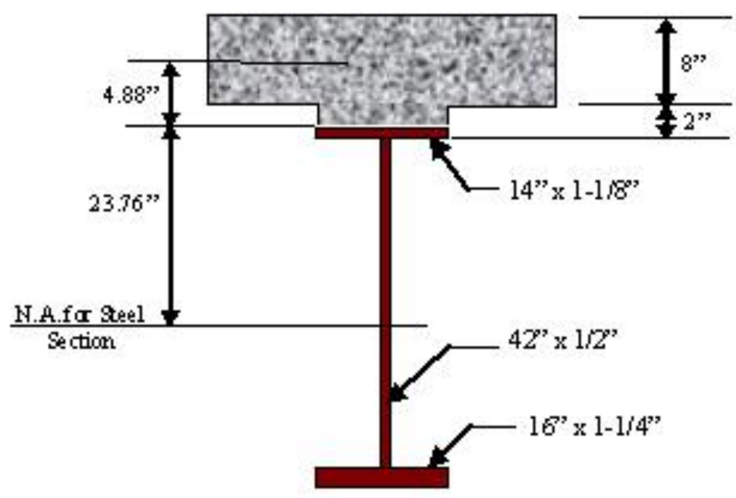

Figure 8 Sketch of Section 2, Negative Bending Region

Table 3 Section 2 Steel Only Section Properties

Component	A	D	Ad	Ad²	I_o	I
Top Flange 14" x 1-1/8"	15.75	21.56	340	7323	1.66	7325
Web 42" x 1/2"	21.00				3087	3087
Bottom Flange 16" x 1-1/4"	20.00	-21.63	-432.5	9352	2.60	9355
	56.75		-92.5			19,767
					-1.63(92.5) =	-151
					I_{NA} =	19,616 in⁴

$$d_s = \frac{-92.5}{56.75} = -1.63 \text{ in.}$$

$$d_{\text{TOP OF STEEL}} = 22.125 + 1.63 = 23.76 \text{ in.} \qquad S_{\text{TOP OF STEEL}} = \frac{19{,}616}{23.76} = 825.6 \text{ in.}^3$$

$$d_{\text{BOT OF STEEL}} = 22.25 - 1.63 = 20.62 \text{ in.} \qquad S_{\text{BOT OF STEEL}} = \frac{19{,}616}{20.62} = 951.3 \text{ in.}^3$$

e_g = 8.0 / 2 +2.0 + 23.76 – 1.125 = 28.64 in.

n = 8

$K_g = n(I + Ae_g^2) = 8(19{,}616 + 56.75(28.64)^2) = 529{,}321 \text{ in.}^4$

As discussed above, the distribution factors for bending moment in interior girders at the strength and service limit states are computed based on the tabulated empirical equations given in Article 4.6.2.2.2.

The applicable equations for moment distribution factors from Table 4.6.2.2.2b-1 are shown below.

$$DF = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1} \quad \text{for one lane loaded}$$

$$DF = 0.06 + \left(\frac{10.0}{14}\right)^{0.4}\left(\frac{10.0}{90.0}\right)^{0.3}\left(\frac{529{,}321}{12.0(90.0)(8.0)^3}\right)^{0.1} = 0.510 \text{ lanes}$$

$$DF = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_S^3}\right)^{0.1} \quad \text{for two or more lanes loaded}$$

$$DF = 0.075 + \left(\frac{10.0}{9.5}\right)^{0.6}\left(\frac{10.0}{90.0}\right)^{0.2}\left(\frac{529{,}321}{12.0(90.0)(8.0)^3}\right)^{0.1} = 0.737 \text{ lanes}$$

Table 4 summarizes the distribution factors for the negative bending region, where it is shown that the exterior girder controls all aspects of the design except for shear at the strength and service limit states.

Table 4 Negative Bending Region Distribution Factors

	Interior Girder	Exterior Girder
Bending Moment	0.737	0.860
Shear	0.952	0.860
Fatigue (Bending Moment)	0.425	0.700
Fatigue (Shear)	0.633	0.700
Deflection	0.500	0.500

5.2.3 Dynamic Load Allowance

The dynamic effects of the truck loading are taken into consideration by the dynamic load allowance, IM. The dynamic load allowance, which is discussed in Article 3.6.2 of the specifications, accounts for the hammering effect of the wheel assembly and the dynamic response of the bridge. IM is only applied to the design truck or tandem, not to the lane loading. Table 3.6.2.1-1 specifies IM equal to 1.33 for the strength, service, and live load deflection evaluations, while IM of 1.15 is specified for the fatigue limit state.

6.0 ANALYSIS RESULTS

6.1 Moment and Shear Envelopes

Figures 9 through 12 show the moment and shear envelopes for this design example, which are based on the data presented in Tables 5 through 11. These figures show distributed moments for the exterior girder and distributed shears for an interior girder, which are the controlling girders for each force effect, based on the distribution factors computed above. For loads applied to the composite section, the envelopes shown are determined based on the composite section properties assuming the concrete deck to be effective over the entire span length.

As previously mentioned, the live load in the positive bending region between the points of dead load contraflexure is the result of the HL-93 loading. In the negative bending region between the points of dead load contraflexure, the moments are the larger of the moments due to the HL-93 loading and the special negative-moment loading, which is composed of 90 percent of both the truck-train moment and lane loading moment.

NOTE: *The analysis results shown herein, including the results of the deck-placement analysis shown later in Section 8.3.1.1, apply to an earlier design of the example bridge. The web thickness shown herein for the example girder has been increased from 7/16 inch in the previous design to ½ inch to follow the latest industry guidelines. While it is nearly always desirable to perform a new analysis whenever plate sizes are revised, the effect on the analysis results in this case was felt to be relatively minor and so new analyses were not performed. The primary intent of this example is to illustrate the proper application of the AASHTO LRFD BDS provisions to the design of a straight continuous steel plate-girder bridge with no skew. However, this also illustrates that a designer should always be aware of specification changes and how they may affect a design and perhaps future load ratings.*

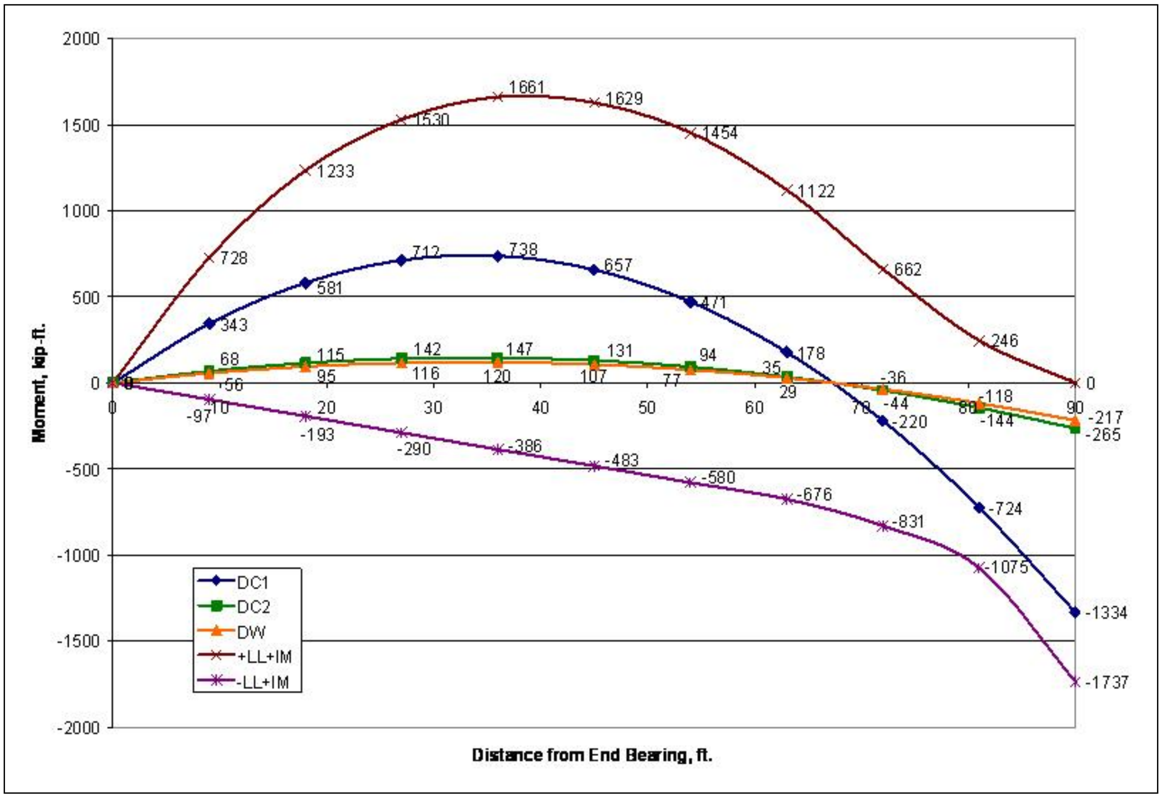

Figure 9 Dead and Live Load Moment Envelopes

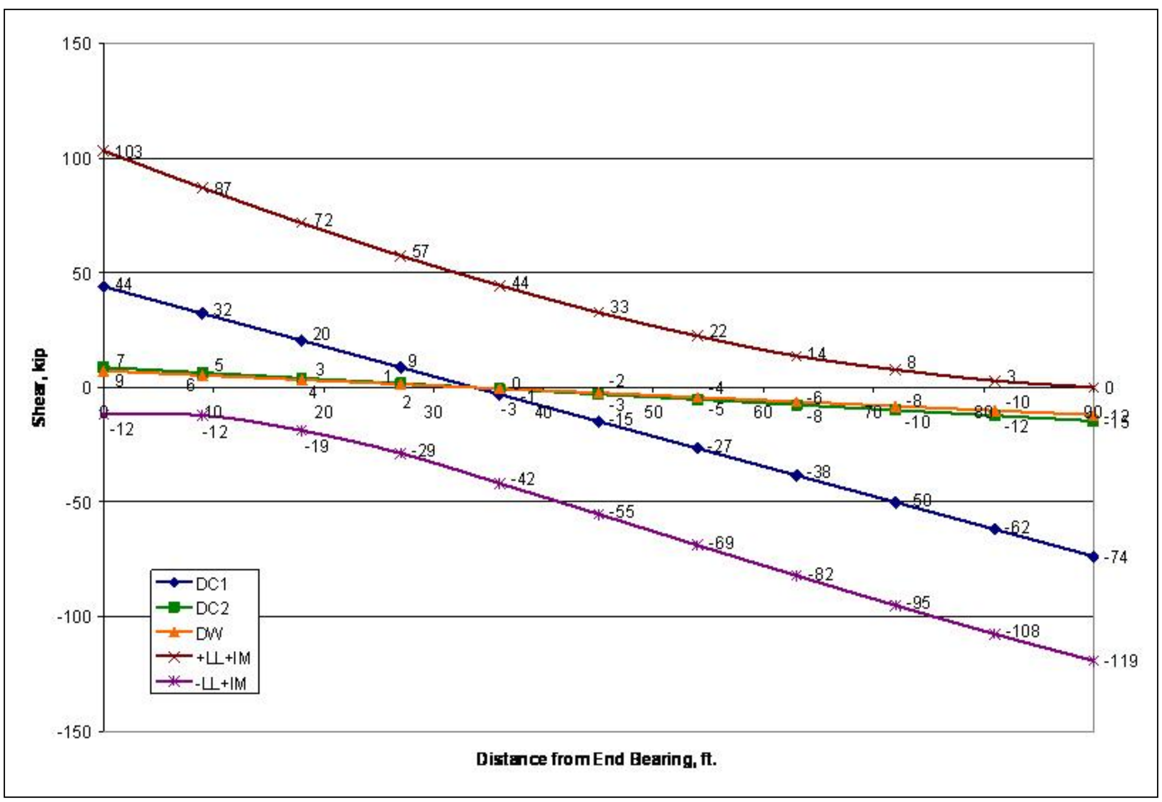

Figure 10 Dead and Live Load Shear Envelopes

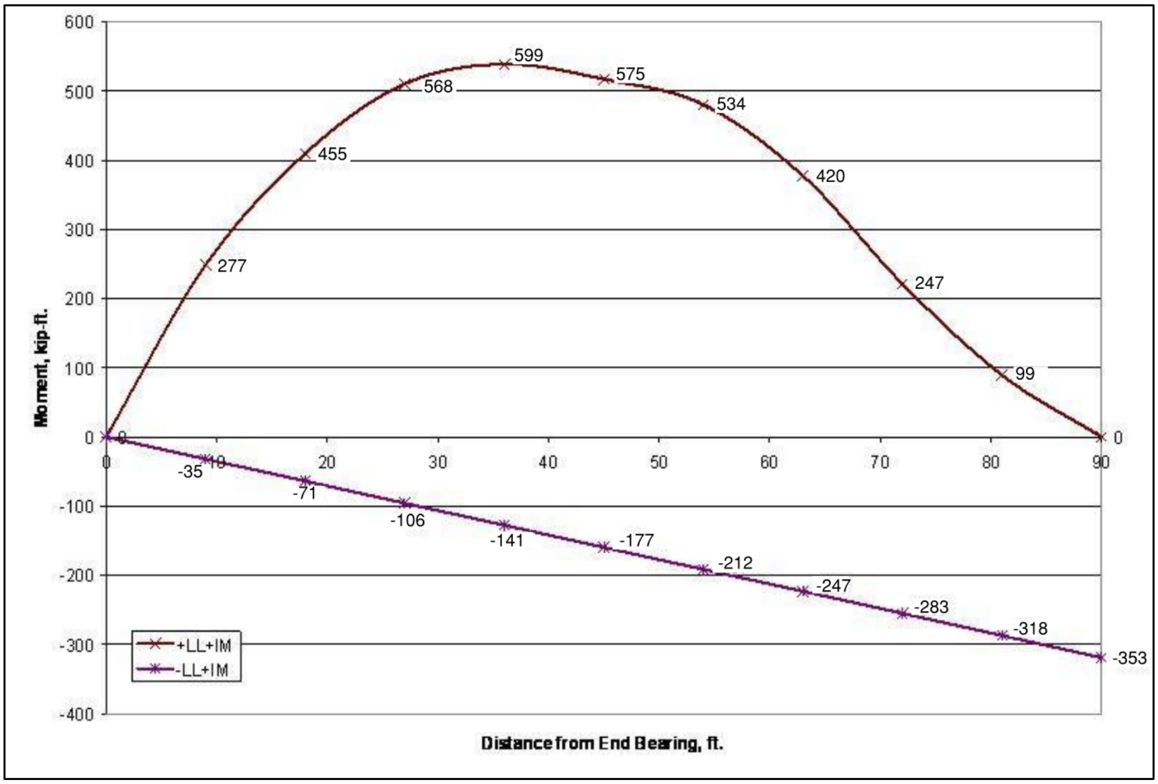

Figure 11 Fatigue Live Load Moments

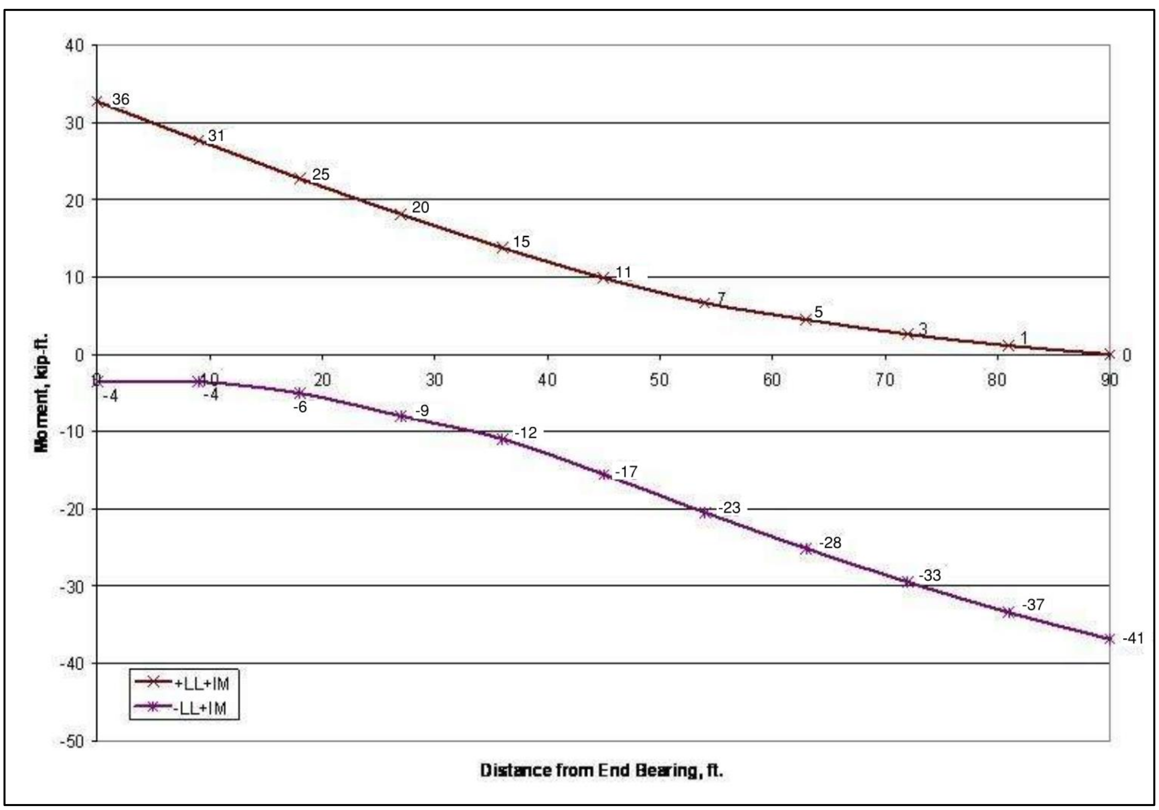

Figure 12 Fatigue Live Load Shears

Table 5 Unfactored and Undistributed Moments (kip-ft)

Span 1	Non-Com. Dead	Com. Dead	Wearing Surf.	Truck Load		Lane Load		Tandem		Double Truck		Double Tandem	
	DC1	DC2	DW	pos.	neg.	pos.	neg.	pos.	neg.	pos.	neg.	pos.	neg.
0.00	0	0	0	0	0	0	0	0	0	0	0	0	0
0.10	343	68	56	485	-60	201	-33	381	-44	0	0	0	0
0.20	581	115	95	816	-120	349	-65	652	-87	0	0	0	0
0.30	712	142	116	1002	-180	446	-98	817	-131	0	0	0	0
0.40	738	147	120	1083	-240	491	-131	883	-174	0	0	0	0
0.50	657	131	107	1059	-300	485	-163	866	-218	0	0	0	0
0.60	471	94	77	951	-359	426	-196	779	-261	0	0	0	0
0.70	178	35	29	743	-419	316	-229	625	-305	0	0	0	0
0.80	-220	-44	-36	463	-479	154	-261	423	-348	0	-479	0	-611
0.90	-724	-144	-118	148	-539	30	-385	192	-392	0	-755	0	-687
1.00	-1334	-265	-217	0	-599	0	-653	0	-436	0	-1196	0	-764

Table 6 Unfactored and Undistributed Live Load Moments (kip-ft)

Span 1	vehicle		special	standard	1.33 Vehicle + Lane positive	Distribution Factors	Positive	Negative
	positive	negative	negative	negative				
0.00	0	0	0	0	0	0.86	0	0
0.10	485	-60	-29	-112	846	0.86	728	-97
0.20	816	-120	-59	-225	1434	0.86	1233	-193
0.30	1002	-180	-88	-337	1779	0.86	1530	-290
0.40	1083	-240	-118	-449	1932	0.86	1661	-386
0.50	1059	-300	-147	-562	1894	0.86	1629	-483
0.60	951	-359	-176	-674	1691	0.86	1454	-580
0.70	743	-419	-206	-786	1304	0.86	1122	-676
0.80	463	-479	-967	-899	770	0.86	662	-831
0.90	192	-539	-1250	-1102	285	0.86	246	-1075
1.00	0	-599	-2020	-1450	0	0.86	0	-1737

Table 7 Strength I Load Combination Moments (kip-ft)

Span 1	1.25 DC1	1.25 DC2	1.5 DW	1.75 (LL+IM)		Strength I	
				positive	negative	positive	negative
0.00	0	0	0	0	0	0	0
0.10	429	85	84	1273	-169	1871	429
0.20	726	144	142	2158	-338	3170	674
0.30	890	177	174	2677	-507	3918	734
0.40	922	183	180	2907	-676	4192	609
0.50	821	163	161	2850	-845	3995	300
0.60	588	117	115	2545	-1014	3365	-194
0.70	223	44	44	1963	-1183	2274	-873
0.80	-275	-55	-54	1159	-1455	775	-1838
0.90	-905	-180	-177	430	-1882	-833	-3144
1.00	-1668	-332	-326	0	-3039	-2326	-5365

Table 8 Service II Load Combination Moments (kip-ft)

Span 1	1.0 DC1	1.0 DC2	1.0 DW	1.3 (LL+IM)		Service II	
				positive	negative	positive	negative
0.00	0	0	0	0	0	0	0
0.10	343	68	56	946	-126	1413	342
0.20	581	115	95	1603	-251	2394	540
0.30	712	142	116	1989	-377	2959	593
0.40	738	147	120	2160	-502	3164	502
0.50	657	131	107	2117	-628	3012	267
0.60	471	94	77	1890	-753	2531	-113
0.70	178	35	29	1458	-879	1701	-636
0.80	-220	-44	-36	861	-1081	561	-1380
0.90	-724	-144	-118	319	-1398	-667	-2384
1.00	-1334	-265	-217	0	-2258	-1816	-4075

Table 9 Unfactored and Undistributed Shears (kip)

Span 1	Non-Com. Dead	Com. Dead	Wearing Surf.	Truck Load		Lane Load		Tandem	
	DC1	DC2	DW	positive	negative	positive	negative	positive	negative
0.00	44	9	7	63	-7	25	-4	49	-5
0.10	32	6	5	54	-7	20	-4	42	-5
0.20	20	4	3	45	-10	15	-5	36	-11
0.30	9	2	1	37	-18	11	-7	30	-17
0.40	-3	-1	0	29	-26	8	-9	25	-23
0.50	-15	-3	-2	22	-34	5	-12	19	-29
0.60	-27	-5	-4	15	-42	3	-16	14	-34
0.70	-38	-8	-6	10	-50	2	-20	10	-38
0.80	-50	-10	-8	5	-56	1	-25	6	-43
0.90	-62	-12	-10	2	-62	0	-30	2	-46
1.00	-74	-15	-12	0	-67	0	-36	0	-49

Table 10 Unfactored and Undistributed Live Load Shears (kip)

Span 1	vehicle		(1.33 V Vehicle + V Lane)		Distribution	Live Load	
	positive	negative	positive	negative		Positive	Negative
0.00	63	-7	108	-12	0.952	103	-12
0.10	54	-7	91	-13	0.952	87	-12
0.20	45	-11	75	-20	0.952	72	-19
0.30	37	-18	60	-30	0.952	57	-29
0.40	29	-26	47	-44	0.952	44	-42
0.50	22	-34	34	-58	0.952	33	-55
0.60	15	-42	24	-72	0.952	22	-69
0.70	10	-50	14	-86	0.952	14	-82
0.80	6	-56	8	-100	0.952	8	-95
0.90	2	-62	3	-113	0.952	3	-108
1.00	0	-67	12	-108	0.952	12	-103

Table 11 Strength I Load Combination Shear (kip)

Span 1	1.25 DC1	1.25 DC2	1.5 DW	1.75 (LL+IM)		Strength I	
				positive	negative	positive	negative
0.00	55	11	11	181	-21	257	56
0.10	40	8	8	152	-21	209	35
0.20	26	5	5	126	-33	161	3
0.30	11	2	2	101	-50	116	-35
0.40	-4	-1	-1	78	-73	72	-79
0.50	-19	-4	-4	57	-97	31	-123
0.60	-33	-7	-6	39	-120	-7	-167
0.70	-48	-10	-9	24	-144	-43	-211
0.80	-63	-12	-12	13	-167	-74	-254
0.90	-77	-15	-15	5	-188	-103	-296
1.00	-92	-18	-18	0	-209	-128	-337

6.2 Live Load Deflection

As indicated in Article 3.6.1.3.2, control of live-load deflection is optional. Evaluation of this criterion is based on the flexural rigidity of the short-term composite section and consists of two load cases: deflection due to the design truck, and deflection due to the design lane plus 25 percent of the design truck. The dynamic load allowance of 33 percent is applied to the design truck load only for both loading conditions. For this example, the live load is distributed using a distribution factor of 0.500 lanes calculated earlier.

The maximum deflection due to the design truck is 0.880 inches. Applying the impact and distribution factors gives the following.

$$\Delta_{LL+IM} = 0.500 \text{ x } 1.33 \text{ x } 0.880 = 0.585 \text{ in.} \qquad \text{(governs)}$$

The maximum deflection due to lane load only is 0.456 inches. Therefore, the deflection due to 25% of the design truck plus the lane loading is equal to the following:

$$\Delta_{LL+IM} = 0.500\ (1.33 \text{ x } 0.25 \text{ x } 0.880 + 0.456) = 0.374 \text{ in.}$$

Thus, the governing deflection equal to 0.585 inch will be used to assess the girder design based on the live-load deflection criterion.

7.0 LIMIT STATES

As discussed previously, there are four limit states applicable to the design of steel I-girders. Each of these limit states is described below.

7.1 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 provide 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 HL-93 design live load specified in Article 3.6.1.1 (discussed previously in Section 4.2). For evaluation of the Service II load combination under Owner-specified special design vehicles and/or evaluation permit vehicles, a reduction in the specified load factor for live load should be considered for this limit-state check.

7.2 Fatigue and Fracture Limit State (Article 1.3.2.3 and 6.5.3)

To satisfy the fatigue limit state, restrictions on stress range under regular service conditions are specified to control crack growth under repetitive loads (Article 6.6.1). Material toughness requirements, which are dependent on the temperature zone in which the structure is located, are specified to satisfy the fracture limit state (Article 6.6.2).

For checking fatigue in steel structures, the fatigue load and Fatigue load combinations 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.

7.3 Strength Limit State (Articles 1.3.2.4 and 6.5.4)

The strength limit state verifies the design is stable and has adequate strength when subjected to the highest load combinations considered. The bridge structure may experience structural damage (e.g., permanent deformations) at the strength limit state, but the integrity of the structure is preserved.

The suitability of the design must also be investigated to verifty adequate strength and stability during each construction phase. The deck casting sequence can have a significant effect on the distribution of stresses within the structure. Therefore, the deck casting sequence should be considered in the design and specified on the plans. In addition, flange lateral bending stresses resulting from forces applied to the overhang brackets during construction should also be considered during the constructability evaluation. Specific design provisions are given in Article 6.10.3 to help verify constructability of steel I-girder bridges; in particular, when subject to the specified deck-casting sequence and deck overhang force effects. The constructability checks are typically made on the steel section only under the factored noncomposite dead loads using the appropriate strength load combinations.

7.4 Extreme Event Limit State (Articles 1.3.2.5 and 6.5.5)

The extreme event limit state is to verify the structure can survive a collision, earthquake, or flood. The collisions investigated under this limit state include the bridge being struck by a vehicle, vessel, or ice flow. This limit state is not addressed in this design example.

8.0 SAMPLE CALCULATIONS

This example presents sample calculations for the design of positive and negative bending sections of the girders for the strength, fatigue and fracture, and service limit states. In addition, calculations evaluating the constructability of the bridge system for the deck-casting sequence are included and the optional provisions for moment redistribution are presented. Also presented are cross-frame and bearing stiffener designs and the design of the flange-to-web welds. The moment and shear envelopes provided in Figs. 8 through 11 are referenced in the following calculations.

8.1 Section Properties

The section properties for Section 1 and Section 2 are first calculated and will be routinely used in the subsequent evaluations of the various code checks at each limit state. The structural slab thickness is taken as the slab thickness minus the thickness of the integral wearing surface (8.0 inches) and the modular ratio (n) is taken to be 8 in these calculations.

8.1.1 Section 1 – Positive Bending Region

Section 1 represents the positive bending region and was previously shown in Figure 5. The longitudinal reinforcement is neglected in the computation of these section properties.

8.1.1.1 Effective Flange Width (Article 4.6.2.6)

Article 4.6.2.6 of the *AASHTO LRFD BDS* governs the determination of the effective flange width of the concrete slab when designing composite sections.

For the interior girders of this example, b_{eff} in the positive bending region is determined as one-half the distance to the adjacent girder on each side of the girder under consideration.

$$b_{eff} = \frac{120}{2} + \frac{120}{2} = 120.0 \text{ in.}$$

For the exterior girders of this example, b_{eff} in the positive bending region is determined as one-half the distance to the adjacent girder plus the full overhang width.

$$b_{eff} = \frac{120}{2} + 42 = 102.0 \text{ in.}$$

The exterior girder has both a smaller effective width and a larger live load distribution factor than the interior girder; therefore, the moment design of the positive bending region is controlled by the exterior girder.

8.1.1.2 Elastic Section Properties: Section 1

As discussed above, the section properties considered in the analysis of the girder vary based on the loading conditions. Specifically, live loads are applied to the short-term composite section, where the modular ratio of 8 is used in the computations. Alternatively, dead loads are applied to the long-term composite section. The long-term composite section accounts for the reduction in strength that may occur in the deck over time due to creep effects. This is reflected in the section property calculations through use of a modular ratio equal to 3 times the typical modular ratio (3n), or in this example, 24. The effective width of the deck is divided by the appropriate modular ratio for each case in the determination of the composite section properties. The section properties for the short-term and long-term composite sections are computed below, in Tables 12 and 13. Recall that the section properties for the steel section (girder alone) were previously computed for the purpose of determining live load distribution factors.

Table 12 Section 1 Short Term Composite (n) Section Properties (Exterior Girder)

Component	A	d	Ad	Ad^2	I_o	I
Steel Section	51.50		-208.0			17,241
Concrete Slab (8″ x 102″/8)	102.0	27.00	2,754	74,358	544.0	74,902
Σ	153.5		2,546			92,143

-16.59(2,546) = -42,238

I_{NA} = 49,905 in.^{4}

$$d_n = \frac{2,546}{153.5} = 16.59 \text{ in.}$$

$$d_{\text{Top of Steel}} = 21.75 - 16.59 = 5.16 \text{ in.}$$

$$d_{\text{Bot of Steel}} = 22.25 + 16.59 = 38.84 \text{ in.}$$

$$S_{\text{Top of Steel}} = \frac{49,905}{5.16} = 9,672 \text{ in.}^3$$

$$S_{\text{Bot of Steel}} = \frac{49,905}{38.84} = 1,285 \text{ in.}^3$$

Table 13 Section 1 Long Term Composite (3n) Section Properties (Exterior Girder)

Component	A	d	Ad	Ad^2	I_o	I
Steel Section	51.50		-208.0			17,241
Concrete Slab (8″ x 102″/24)	34.00	27.00	918.0	24,786	181.3	24,967
Σ	85.50		710.0			42,208

-8.30(710.0) = -5,893

I_{NA} = 36,315 in.^{4}

$$d_{3n} = \frac{710.0}{85.50} = 8.30 \text{ in.}$$

$$d_{\text{Top of Steel}} = 21.75 - 8.30 = 13.45 \text{ in.}$$

$$d_{\text{Bot of Steel}} = 22.25 + 8.30 = 30.55 \text{ in.}$$

$$S_{\text{Top of Steel}} = \frac{36,315}{13.45} = 2,700 \text{ in.}^3 \qquad S_{\text{Bot of Steel}} = \frac{36,315}{30.55} = 1,189 \text{ in.}^3$$

8.1.1.3 Plastic Moment: Section 1

The plastic moment, M_p, is the resisting moment of an assumed fully-yielded cross-section and may be determined for sections in positive flexure using the procedure outlined in Table D6.1-1 as demonstrated below. The longitudinal deck reinforcement is conservatively neglected in these computations. The plastic forces acting in the slab (P_s), compression flange (P_c), web (P_w), and tension flange (P_t) are first computed.

$P_s = 0.85f'_c b_s t_s \quad = 0.85(4.0)(102)(8) \quad = 2,774$ kips

$P_c = F_{yc} b_c t_c \quad = (50)(14)(0.75) \quad = 525$ kips

$P_w = F_{yw} D t_w \quad = (50)(42)(0.5) \quad = 1,050$ kips

$P_t = F_{yt} b_t t_t \quad = (50)(16)(1.25) \quad = 1,000$ kips

The plastic forces for each element of the girder are then compared to determine the location of the plastic neutral axis (PNA). The position of the PNA is determined by equilibrium; i.e., no net axial force when considering the summation of plastic forces. Table D.6.1-1 provides seven cases, with accompanying conditions for use, to determine the location of the PNA and subsequently calculate the plastic moment.

Following the conditions set forth in Table D6.1-1, the PNA is generally located as follows:

CASE I:

$P_t + P_w \geq P_c + P_s$

$1,000 + 1,050 \geq 525 + 2,774$?

$2,050$ kips $< 3,299$ kips   Therefore, PNA is not in the web

CASE II:

$P_t + P_w + P_c \geq P_s$

$1,000 + 1,050 + 525 \geq 2,774$?

$2,575$ kips $< 2,774$ kips   Therefore, PNA is not in the top flange

Therefore, the plastic neutral axis is in the concrete deck and $\bar{y}$ is computed using the following equation derived from that provided in Table D6.1-1 when deck reinforcement is ignored:

$$\bar{y} = (t_s)\left[\frac{P_c + P_w + P_t}{P_s}\right]$$

$$\bar{y} = (8.0)\left[\frac{525 + 1,050 + 1,000}{2,774}\right] = 7.43 \text{ inches from the top of the concrete slab}$$

The plastic moment M_p is then calculated using the following equation derived from that provided in Table D6.1-1 when deck reinforcement is ignored.

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

The distance from the PNA to the centroid of the compression flange, web, and tension flange (respectively) is as follows:

d_c = 8.0 + 2.0 – 0.5(0.75) - 7.43 = 2.195 in.

d_w = 8.0 + 2.0 + 0.5(42.0) – 7.43 = 23.57 in.

d_t = 8.0 + 2.0 + 42.0 + 0.5(1.25) – 7.43 = 45.20 in.

Substitution of these distances and the above computed plastic forces, into the preceding equation, gives the following:

$$M_p = \left(\frac{(7.43)^2(2,774)}{2(8.0)}\right) + \left[(525)(2.195) + (1,050)(23.57) + (1,000)(45.20)\right]$$

M_p = 80,672 kip-in. = 6,723 kip-ft

8.1.1.4 Yield Moment: Section 1

The yield moment, which is the moment causing first yield in either flange (neglecting flange lateral bending), is determined according to the provisions specified in Article D6.2.2 of the specifications. This computation method for the yield moment recognizes that different stages of loading (e.g., composite dead load, noncomposite dead load, and live load) act on the girder when different cross-sectional properties are applicable. The yield moment is determined by solving for M_{AD} using Equation D6.2.2-1 (given below) and then summing M_{D1}, M_{D2}, and M_{AD}, where, M_{D1}, M_{D2}, and M_{AD} are the factored moments applied to the noncomposite, long-term composite, and short-term composite section, respectively.

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

Due to the significantly higher section modulus of the short-term composite section about the top flange, compared to the short-term composite section modulus taken about the bottom flange, the minimum yield moment results when using the bottom flange section moduli.

Computation of the yield moment for the bottom flange is thus demonstrated below. First the known quantities are substituted into Equation D6.2.2-1 to solve for M_{AD}.

$$50 = 1.0\left[\frac{1.25(738)(12)}{900.7} + \frac{1.25(147)(12) + 1.50(120)(12)}{1{,}189} + \frac{M_{AD}}{1{,}285}\right]$$

M_{AD} = 43,740 kip-in. = 3,645 kip-ft

M_y is then determined by applying the applicable load factors and summing the dead loads and M_{AD}.

M_y = 1.25(738) + 1.25(147) + 1.50(120) + 3,645 = 4,931 kip-ft Eq. (D6.2.2-2)

8.1.2 Section 2 – Negative Bending Region

This section details the calculations to determine the section properties of the composite girder in the negative bending region, which was previously shown in Figure 8.

8.1.2.1 Effective Flange Width (Article 4.6.2.6)

As discussed previously, the effective flange width for interior girders is computed as one-half the distance to the adjacent girder one each side of the girder being analyzed.

$$b_{eff} = \frac{120}{2} + \frac{120}{2} = 120.0 \text{ in.}$$

For an exterior girder, b_{eff} is determined as one-half the distance to the adjacent girder plus the full overhang width.

$$b_{eff} = \frac{120}{2} + 42 = 102.0 \text{ in.}$$

8.1.2.2 Minimum Negative Flexure Concrete Deck Reinforcement (Article 6.10.1.7)

The total area of the longitudinal reinforcement, provided in negative bending regions, is not to be less than one percent of the total cross-sectional area of the concrete deck. This provision is intended to control cracking of the concrete deck in regions where the tensile stress due to the factored construction loads or the Service II load exceeds ϕf_r. (f_r is the modulus of rupture of the concrete and is to be taken equal to $0.24(f_r)^{0.5}$ for normal weight concrete, with ϕ taken equal to 0.90).

The total area of the concrete deck in this example is computed as follows.

$$A_{deck} = \frac{8.0}{12}(37.0) + 2\left[\left(\frac{1}{2}\right)\left(\frac{1}{12}\right)(2.0 - 0.75)\left(3.5 - \frac{14.0/2}{12}\right)\right] = 24.97 \text{ ft}^2 = 3{,}596 \text{ in.}^2$$

The minimum area of reinforcing steel required is taken as:

$$0.01(3{,}596) = 35.96 \text{ in.}^2$$

Reinforcement is to be distributed uniformly across the deck width. The area of reinforcement required within the effective width (102 inches) of an exterior girder is determined as shown below.

$$\frac{35.96 \text{ in.}^2}{37.0 \text{ ft}} = 0.97 \text{ in.}^2/\text{ft}$$

$$0.97(102/12) = 8.25 \text{ in.}^2$$

8.25 in.^{2} of longitudinal reinforcement must be provided at a minimum. This reinforcement should be placed in two layers with two-thirds of the reinforcement in the top layer and the remaining one-third placed in the bottom layer. Therefore, the minimum area of the top and bottom reinforcement is 5.50 in.^{2} and 2.75 in.^{2}, respectively. A bar layout of #6 bars (A = 0.44 in.^{2}) spaced at 6 inches in the top layer, and #5 bars (A = 0.31 in.^{2}) spaced at 8 inches in the bottom layer is selected. Additionally, transverse reinforcement of #5 bars spaced at 12 inches is provided above the top layer and below the bottom layer, with 2 inches and 1 inch of cover, respectively. The resulting total longitudinal reinforcement area is calculated as follows:

Top Layer: $$\frac{0.44 \text{ in.}^2}{6.0 \text{ in.}} = 0.073 \text{ in.}^2/\text{in. x } 102.0 \text{ in.} = 7.45 \text{ in.}^2$$

Bottom Layer: $$\frac{0.31 \text{ in.}^2}{8.0 \text{ in.}} = 0.039 \text{ in.}^2/\text{in. x } 102.0 \text{ in.} = 3.98 \text{ in.}^2$$

It should be noted that the reinforcement should not use bar sizes exceeding No. 6 bars or bar spacings exceeding 12.0 inches. The reinforcement must have a specified minimum yield strength not less than 60 ksi.

8.1.2.3 Elastic Section Properties: Section 2

Similar to the computation of section properties presented above for Section 1, section properties for the short-term and long-term composite sections in Section 2 are presented below. Section properties are computed assuming the concrete is effective in tension (for potential use at the fatigue and service limit states), and also for the section consisting of the girder and reinforcing steel only assuming that the concrete is not effective in tension (for use at the strength limit state).

Table 14 Section 2 Short Term Composite (n) Section Properties

Component	A	d	Ad	Ad²	I_o	I
Steel Section	56.75		-92.5			19,767
Concrete Slab (8"x 102"/8)	102.0	27.00	2,754	74,358	544	74,902
	158.75		2,662			94,669
				-16.77(2,662) =		<u>-44,642</u>
						50,027 in⁴

$$d_s = \frac{2{,}662}{158.75} = 16.77 \text{ in.}$$

$$d_{\text{TOP OF STEEL}} = 22.125 - 16.77 = 5.36 \text{ in.} \qquad S_{\text{TOP OF STEEL}} = \frac{50{,}027}{5.36} = 9{,}333 \text{ in.}^3$$

$$d_{\text{BOT OF STEEL}} = 22.25 + 16.77 = 39.02 \text{ in.} \qquad S_{\text{BOT OF STEEL}} = \frac{50{,}027}{39.02} = 1{,}282 \text{ in.}^3$$

Table 15 Section 2 Long Term Composite (3n) Section Properties

Component	A	d	Ad	Ad²	I_o	I
Steel Section	56.75		-92.5			19,767
Concrete Slab (8"x 102"/24)	34.0	27.00	918	24,786	181.3	24,967
	90.75		825.5			44,734
				-9.10(825.5) =		<u>-7,512</u>
						37,222 in⁴

$$d_s = \frac{825.5}{90.75} = 9.10 \text{ in.}$$

$$d_{\text{TOP OF STEEL}} = 22.125 - 9.10 = 13.03 \text{ in.} \qquad S_{\text{TOP OF STEEL}} = \frac{37{,}222}{13.03} = 2{,}857 \text{ in.}^3$$

$$d_{\text{BOT OF STEEL}} = 22.25 + 9.10 = 31.35 \text{ in.} \qquad S_{\text{BOT OF STEEL}} = \frac{37{,}222}{31.35} = 1{,}187 \text{ in.}^3$$

Table 16 Section 2 Steel Section and Longitudinal Reinforcement Section Properties

Component	A	d	Ad	Ad²	I_o	I
Steel Section	56.75		-92.5			19,767
Top Reinforcement Layer	7.45	28.00	208.6	4,398		5,841
Bottom Reinforcement Layer	3.98	24.94	98.5	1,785		2,476
Σ	68.18		214.6			28,084

-3.15(214.6) = <u>-676</u>

$I_{NA} = 27{,}408 \text{ in.}^4$

$$d_{s+\text{reinf}} = \frac{214.6}{68.18} = 3.15 \text{ in.}$$

$$d_{\text{Top of Steel}} = 22.125 - 3.15 = 18.975 \text{ in.} \qquad d_{\text{Bot of Steel}} = 22.25 + 3.15 = 25.40 \text{ in.}$$

$$S_{\text{Top of Steel}} = \frac{27,408}{18.975} = 1,444 \text{ in.}^3$$

$$S_{\text{Bot of Steel}} = \frac{27,408}{25.40} = 1,079 \text{ in.}^3$$

$$d_{\text{Top of Rebar}} = 28.375 - 3.15 = 25.225 \text{ in.}$$

$$S_{\text{Top of Rebar}} = \frac{27,408}{25.225} = 1,087 \text{ in.}^3$$

8.1.2.4 Plastic Moment: Section 2

Similar to the calculation of the plastic moment for Section 1, Table D6.1-2 is used to determine the plastic moment (M_p) for the negative bending section as demonstrated below. The concrete slab in tension is neglected in the computation of M_p. The plastic force acting in each element of the girder is first computed.

$P_c = F_{yc}b_ct_c$ = (50)(16)(1.25) = 1,000 kips

$P_w = F_{yw}Dt_w$ = (50)(42)(0.50) = 1,050 kips

$P_t = F_{yt}b_tt_t$ = (50)(14)(1.125) = 788 kips

$P_{rb} = F_{yrb}A_{rb}$ = (60)(3.98) = 239 kips

$P_{rt} = F_{yrt}A_{rt}$ = (60)(7.45) = 447 kips

The plastic forces in each element are used to determine the general location of the plastic neutral axis as follows:

CASE I

$P_c + P_w \geq P_t + P_{rb} + P_{rt}$

$1,000 + 1,050 \geq 788 + 239 + 447$?

$2,050 \geq 1,474$ Therefore, the plastic neutral axis is in the web.

The location of plastic neutral axis ($\bar{y}$) is determined by the following equation:

$$\bar{y} = \left(\frac{D}{2}\right)\left[\frac{P_c - P_t - P_{rt} - P_{rb}}{P_w} + 1\right]$$

$$\bar{y} = \left(\frac{42}{2}\right)\left[\frac{1,000 - 788 - 447 - 239}{1,050} + 1\right] = 11.52 \text{ in.}$$

The plastic moment (M_p) is then computed as follows:

$$M_p = \frac{P_w}{2D}\left[\bar{y}^2 + \left(D - \bar{y}^2\right)\right] + \left[P_{rt}d_{rt} + P_{rb}d_{rb} + P_t d_t + P_c d_c\right]$$

where,

d_{rt} = 11.52 + 2.0 + 8.0 – 2.0 – 5/8 – (6/8)/2 = 18.52 in.

d_{rb} = 11.52 + 2.0 + 1.0 + 5/8 + (5/8)/2 = 15.46in.

d_t = 11.52 + 1.125/2 = 12.08 in.

d_c = 42.0 – 11.52 + 1.25/2 = 31.11 in.

$$M_p = \left[\frac{1{,}050}{2\left(42.0\right)}\right]\left[\left(11.52\right)^2 + \left(42.0 - 11.52\right)^2\right]$$
$$+\left[\left(447\right)\left(18.52\right) + \left(239\right)\left(15.46\right) + \left(788\right)\left(12.08\right) + \left(1{,}000\right)\left(31.11\right)\right]$$

M_p = 65,874 kip-in. = 5,490 kip-ft

8.1.2.5 Yield Moment: Section 2

The process for determining the yield moment of the negative bending section is similar to the process for the positive bending section. The one difference, though, is that since the composite short-term and the composite long-term bending sections are both composed of the steel section and the reinforcing steel only at the strength limit state, the section modulus is the same for both the short-term and long-term composite sections.

The yield moment is the lesser of the moment which causes first yielding of the section, either yielding in the bottom flange or yielding in the tension flange or steel reinforcing. Because, for the negative bending region it is not clear which yield moment value will control, the moments causing first yield in both compression and tension are computed.

The moment causing yielding in compression flange is first computed based on Equation D6.2.2-1.

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

$$50 = 1.0\left[\frac{1.25\left|-1{,}334\right|\left(12\right)}{951.3} + \frac{1.25\left|-265\right|\left(12\right) + 1.50\left|-217\right|\left(12\right)}{1{,}079} + \frac{M_{AD}}{1{,}079}\right]$$

M_{AD} = 23,373 kip-in. = 1,948 kip-ft

M_{yc} = (1.25)(1,334) + (1.25)(265) + (1.50)(217) + 1,948 = 4,272 kip-ft

The specifications indicate that for regions in negative flexure, M_{yt} is to be taken with respect to either the tension flange or the longitudinal steel reinforcement, whichever yields first. Therefore, compute M_{yt} for both and use the smaller value.

The moment which causes yielding in the tension flange is computed as follows:

$$50 = 1.0\left[\frac{1.25\left|-1,334\right|(12)}{825.6} + \frac{1.25\left|-265\right|(12) + 1.50\left|-217\right|(12)}{1,444} + \frac{M_{AD}}{1,444}\right]$$

M_{AD} = 29,321 kip-in. = 2,443 kip-ft

M_{yt} = (1.25)(1,334) + (1.25)(265) + (1.50)(217) + 2,443 = 4,767 kip-ft

The moment which causes yielding in the longitudinal steel reinforcement is computed as follows. It is necessary to recognize that there is no noncomposite moment acting on the longitudinal steel reinforcement, and that F_{yf} should be taken as 60 ksi.

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

$F_{yf} = F_y$ = 60 ksi $\qquad$ M_{D1} = 0 kip-ft

$$60 = 1.0\left[0 + \frac{1.25\left|-265\right|(12) + 1.50\left|-217\right|(12)}{1,087} + \frac{M_{AD}}{1,087}\right]$$

M_{AD} = 57,339 kip-in. = 4,778 kip-ft

M_{yt} = (1.25)(265) + (1.50)(217) + 4,778 = 5,435 kip-ft

Therefore, the top flange yields before the longitudinal reinforcement, and M_{yt} = 4,767 kip-ft.

For the whole section, the compression flange governs, thus $M_y = M_{yc}$ = 4,272 kip-ft

8.2 Exterior Girder Check: Section 2

This design example illustrates the use of the optional moment redistribution procedures given in Appendix B6, where moment is redistributed from the negative bending region to the positive bending region; therefore the negative bending region will be checked first in order to determine the amount of moment that must be redistributed to the positive bending region.

8.2.1 Strength Limit State (Article 6.10.6)

8.2.1.1 Flexure (Appendix A6)

For sections in negative flexure, the flexural resistance of the member can be determined for general steel I-girders using Article 6.10.8, which limits the maximum resistance to the yield moment of the section. Alternatively, Appendix A6 permits girder flexural resistances up to M_p

and may be used for girders: having a yield strength less than or equal to 70 ksi, with a compact or non-compact web (which is defined by Eq. A6.1-1), and satisfying Eq. A6.1-2 (given below). The applicability of Appendix A6 for this design example is evaluated below.

The first requirement for use of Appendix A6 is that the specified minimum yield strength of the flanges and web must be less than or equal to 70 ksi.

F_y= 50 ksi < 70 ksi (satisfied)

The web slenderness requirement is evaluated using Eq. A6.1-1.

$$\frac{2D_c}{t_W} \leq \lambda_{rw} \qquad \text{Eq. (A6.1-1)}$$

where:

$$4.6\sqrt{\frac{E}{F_{yc}}} \leq \lambda_{rw} = \left(3.1+\frac{5.0}{a_{wc}}\right)\sqrt{\frac{E}{F_{yc}}} \leq 5.7\sqrt{\frac{E}{F_{yc}}} \qquad \text{Eq. (A6.1-3)}$$

$$a_{wc} = \frac{2D_c t_w}{b_{fc} t_{fc}} \qquad \text{Eq. (A6.1-4)}$$

As computed above for the section consisting of the steel beam plus the longitudinal reinforcement (Table 16), the elastic neutral axis is located 25.40 inches from the bottom of the composite negative bending section. Subtracting the bottom flange thickness gives the web depth in compression in the elastic range (D_c) as computed below.

D_c = 25.40 – 1.25 = 24.15 in.

$$\frac{2(24.15)}{0.5} = 96.60$$

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

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

$$a_{wc} = \frac{2(24.15)(0.5)}{16.0(1.25)} = 1.21$$

$$111 < \lambda_{rw} = \left(3.1+\frac{5.0}{1.21}\right)\sqrt{\frac{29,000}{50}} = 174.2 > 137$$

$$\therefore \lambda_{rw} = 137 > \frac{2D_c}{t_w} = 96.60 \qquad \text{(satisfied)}$$

Equation A6.1-2 prevents the use of extremely monosymmetric girders, which analytical studies indicate have significantly reduced torsional rigidity.

$$\frac{I_{yc}}{I_{yt}} \geq 0.3 \qquad \text{Eq. (A6.1-2)}$$

$$\frac{(1/12)(1.25)(16.0)^3}{(1/12)(1.125)(14.0)^3} = 1.7 > 0.3 \qquad \text{(satisfied)}$$

Thus, Appendix A6 is applicable.

The strength requirements specified by Appendix A6 are given in Section A6.1.1. Since the compression flange is discretely braced at Section 2, the flexural resistance of the compression flange must exceed the maximum negative moment plus one-third of the flange lateral bending stress due to the factored Strength I loading multiplied by the section modulus for the compression flange, see Eq. A6.1.1-1.

$$M_u + \frac{1}{3} f_\ell S_{xc} \leq \phi_f M_{nc} \qquad \text{Eq. (A6.1.1-1)}$$

However, because the flange lateral bending stresses are zero at the strength limit state for the straight girders considered in this example, the left side of the equation reduces to only the maximum factored moment. The tension flange at Section 2 is continuously braced by the concrete deck at the strength limit state, and must therefore satisfy the following, see Eq. A6.1.4-1.

$$M_u \leq \phi_f R_{pt} M_{yt} \qquad \text{Eq. (A6.1.4-1)}$$

Use of Appendix A6 begins with the computation of the web plastification factors, as detailed in Article A6.2 and calculated below. If the section has a web which satisfies the compact web slenderness limit of Eq. A6.2.1-1, the section can reach M_p provided the flange slenderness and unbraced length requirements are satisfied.

$$\frac{2D_{cp}}{t_W} < \lambda_{PW(D_{CP})} \qquad \text{Eq. (A6.2.1-1)}$$

where: $$\lambda_{pw(D_{cp})} = \frac{\sqrt{\frac{E}{F_{yc}}}}{\left(0.54\frac{M_p}{R_h M_y} - 0.09\right)^2} \le \lambda_{rw}\left(\frac{D_{cp}}{D_c}\right)$$ Eq. (A6.2.1-2)

The web depth in compression at M_p is computed by subtracting the previously determined distance between the top of the web and the plastic neutral axis from the total web depth.

D_{cp} = 42.0 – 11.52 = 30.48 in.

The hybrid factor, R_h, is determined from Article 6.10.1.10.1, and is 1.0 for this example since the design has a homogeneous material configuration. Therefore, λ_{pw} is computed as follows.

$$\lambda_{rw} = 137$$

$$\lambda_{pw(D_{cp})} = \frac{\sqrt{\frac{29,000}{50}}}{\left(0.54\frac{65,874}{(1.0)(4,272)(12)} - 0.09\right)^2} = 66.0 < 137\left(\frac{30.48}{24.15}\right) = 172.9$$

The web slenderness classification is then determined as follows.

$$\frac{2D_{cp}}{t_w} = \frac{2(30.48)}{0.5} = 121.9 > \lambda_{pw(D_c)} = 66.0$$ (noncompact)

As shown, the web does not qualify as compact. However, it was previously demonstrated when evaluating the Appendix A6 applicability that the web does qualify as noncompact. Therefore, the applicable web plastification factors for noncompact web sections are used and are determined as specified by Eqs. A6.2.2-4 and A6.2.2-5:

$$R_{pc} = \left[1 - \left(1 - \frac{R_h M_{yc}}{M_p}\right)\left(\frac{\lambda_w - \lambda_{pw(D_C)}}{\lambda_{rw} - \lambda_{pw(D_C)}}\right)\right]\frac{M_p}{M_{yc}} \le \frac{M_p}{M_{yc}}$$ Eq. (A6.2.2-4)

where: $\lambda_w = \frac{2D_c}{t_w}$ Eq. (A6.2.2-2)

$\lambda_{pw(D_c)}$ = limiting slenderness ratio for a compact web corresponding to $2D_c/t_w$

$$\lambda_{pw(D_c)} = \lambda_{pw(D_{cp})}\left(\frac{D_c}{D_{cp}}\right) = (66.0)\left(\frac{24.15}{30.48}\right) = 52.3 < \lambda_{rw} = 137$$ Eq. (A6.2.2-6

$$R_{pc} = \left[1 - \left(1 - \frac{(1.0)(4{,}272)(12)}{65{,}874}\right)\left(\frac{96.60 - 52.3}{137 - 52.3}\right)\right]\frac{65{,}874}{(4{,}272)(12)} \le \frac{65{,}874}{(4{,}272)(12)}$$

$R_{pc} = 1.136 \le 1.285 = 1.136$

$$R_{pt} = \left[1 - \left(1 - \frac{R_h M_{yt}}{M_p}\right)\left(\frac{\lambda_w - \lambda_{pw(D_c)}}{\lambda_{rw} - \lambda_{pw(D_c)}}\right)\right]\frac{M_p}{M_{yt}} \le \frac{M_p}{M_{yt}} \qquad \text{Eq. (A6.2.2-5)}$$

$$R_{pt} = \left[1 - \left(1 - \frac{(1.0)(4{,}767)(12)}{65{,}874}\right)\left(\frac{96.60 - 52.3}{137 - 52.3}\right)\right]\frac{65{,}874}{(4{,}767)(12)} \le \frac{65{,}874}{(4{,}767)(12)}$$

$R_{pt} = 1.072 \le 1.152 = 1.072$

The flexural resistance based on the compression flange is determined from Article A6.3 and is taken as the minimum of the local buckling resistance from Article A6.3.2 and the lateral torsional buckling resistance from Article A6.3.3.

To evaluate the local buckling resistance, the flange slenderness classification is first determined, where the flange is considered compact if the following equation is satisfied:

$$\lambda_f \le \lambda_{pf}$$

where: $$\lambda_f = \frac{b_{fc}}{2t_{fc}} \qquad \text{Eq. (A6.3.2-3)}$$

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{f_{yc}}} \qquad \text{Eq. (A6.3.2-4)}$$

$$\lambda_f = \frac{b_{fc}}{2t_{fc}} \le \lambda_{pf} = 0.38\sqrt{\frac{E}{F_{yc}}}$$

$$\lambda_f = \frac{16.0}{2(1.25)} \le \lambda_{pf} = 0.38\sqrt{\frac{29{,}000}{50}}$$

$$\lambda_f = 6.40 < \lambda_{pf} = 9.15 \qquad \text{(satisfied)}$$

Therefore, the compression flange is considered compact, and the flexural resistance based on local buckling of the compression flange is governed by Eq. A6.3.2-1.

$M_{nc} = R_{pc}\, M_{yc} = (1.136)(4{,}272) = 4{,}853$ kip-ft Eq. (A6.3.2-1)

Similarly, to evaluate the compressive flexural resistance based on lateral-torsional buckling, the unbraced length must be first classified. Unbraced lengths satisfying the following equation are classified as compact.

$$L_b \le L_p$$

where: L_b= (10.0)(12) = 120 in.

$$L_p = r_t \sqrt{\frac{E}{F_{yc}}} \qquad \text{Eq. (A6.3.3-4)}$$

where: r_t = effective radius of gyration for lateral torsional buckling (in.)

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{3}\frac{D_c\ t_w}{b_{fc}\ t_{fc}}\right)}} = \frac{16.0}{\sqrt{12\left(1+\frac{1}{3}\frac{(24.15)(0.5)}{(16.0)(1.25)}\right)}} \qquad \text{Eq. (A6.3.3-10)}$$

r_t = 4.21 in.

$$\therefore L_b > L_p = 4.21\sqrt{\frac{29{,}000}{50}} = 101.4 \qquad \text{(noncompact)}$$

Because the lateral bracing distance does not satisfy the compact limit, the non-compact limit is next evaluated.

$$L_p < L_b \le L_r$$

where: L_r = limiting unbraced length to achieve the nominal onset of yielding in either flange under uniform bending with consideration of compression flange residual stress effects (in.)

$$L_r = 1.95 r_t \frac{E}{F_{yr}} \sqrt{\frac{J}{S_{xc} h}} \sqrt{1+\sqrt{1+6.76\left(\frac{F_{yr} S_{xc} h}{EJ}\right)^2}} \qquad \text{Eq. (A6.3.3-5)}$$

F_{yr} = smaller of the compression flange stress at the nominal onset of yielding of either flange, with consideration of compression-flange residual stress effects but without consideration of flange lateral bending, or the specified minimum yield strength of the web (ksi)

$$F_{yr} = \min\left(0.7F_{yc}, R_h F_{yt}\frac{S_{xt}}{S_{xc}}, F_{yw}\right) \qquad \text{Article A6.3.3}$$

S_{xt} = (4,767)(12)/50 = 1,144 in.^{3}

S_{xc} = (4,272)(12)/50 = 1,025 in.^{3}

$$F_{yr} = \min\left(0.7(50),(1.0)(50)\frac{1{,}144}{1{,}025},50\right)$$

F_{yr} = min(35, 55.8, 50)

F_{yr} = 35.0 ksi > $0.5F_{yc}$ = 25.0 ksi (satisfied)

J = St. Venant torsional constant

$$J = \frac{1}{3}\left(Dt_w{}^3 + b_{fc}t_{fc}{}^3\left(1-0.63\frac{t_{fc}}{b_{fc}}\right)+b_{ft}t_{ft}{}^3\left(1-0.63\frac{t_{ft}}{b_{ft}}\right)\right) \qquad \text{Eq. (A6.3.3-9)}$$

$$J = \frac{1}{3}\left((42)(0.5)^3+(16)(1.25)^3(0.95)+(14)(1.125)^3(0.95)\right)$$

J = 17.96 in.^{3}

h = depth between the centerline of the flanges

h = 1.125/2 + 42 + 1.25/2 = 43.19 in.

$$L_r = 1.95(4.21)\frac{29{,}000}{35.0}\sqrt{\frac{17.96}{(1{,}025)(43.19)}}\sqrt{1+\sqrt{1+6.76\left(\frac{35.0(1{,}025)(43.19)}{(29{,}000)(17.96)}\right)^2}}$$

L_r = 406.4 in.

L_b = 120 in. < L_r = 406.4 in. (satisfied)

Therefore, the unbraced length is classified as noncompact and the lateral torsional buckling resistance is controlled by Eq. A6.3.3-2 of the Specifications.

$$M_{nc} = C_b\left[1-\left(1-\frac{F_{yr}S_{xc}}{R_{pc}M_{yc}}\right)\left(\frac{L_b-L_p}{L_r-L_p}\right)\right]R_{pc}M_{yc} \le R_{pc}M_{yc} \qquad \text{Eq. (A6.3.3-2)}$$

where: C_b = moment gradient modifier

The moment gradient modifier is discussed in Article A6.3.3 and is calculated in the following manner.

$$C_b = 1.75-1.05\left(\frac{M_1}{M_2}\right)+0.3\left(\frac{M_1}{M_2}\right)^2 \le 2.3 \qquad \text{Eq. (A6.3.3-7)}$$

where: $M_1 = M_o$ when the variation in moment between brace points is concave

Otherwise:

$M_1 = 2M_{mid} - M_2 \geq M_0$

M_{mid} = factored major-axis bending moment at the middle of the unbraced length

M_0 = factored moment at the brace point opposite to the one corresponding to M_2

M_2 = largest factored major-axis bending moment at either end of the unbraced length causing compression in the flange under consideration

For the critical moment location at the interior pier, the variation in moment is concave throughout the unbraced length and the applicable moment values are as follows:

$M_2 = 5{,}365$ kip-ft

$M_0 = 2{,}999$ kip-ft

$M_1 = M_0 = 2{,}999$ k-ft   Eq. (A6.3.3-11)

$$C_b = 1.75 - 1.05\left(\frac{2{,}999}{5{,}365}\right) + 0.3\left(\frac{2{,}999}{5{,}365}\right)^2 = 1.26 < 2.3$$

$C_b = 1.26$

Therefore, M_{nc} is equal to the following:

$$M_{nc} = (1.26)\left[1 - \left(1 - \frac{(35.0)(1{,}025)}{(1.136)(4{,}272)(12)}\right)\left(\frac{120 - 101.4}{406.4 - 101.4}\right)\right](1.136)(4{,}272) \leq 1.136(4{,}272)$$

$M_{nc} = 5{,}972$ kip-ft $\leq 4{,}853$ kip-ft ?

$M_{nc} = 4{,}853$ kip-ft

If the computed M_{nc} had been less than $R_{pc}M_{yc}$ in this case, then the equations of Article D6.4.2 could have alternatively been used to potentially obtain a larger resistance. As previously stated, the flexural resistance based on the compression flange is the minimum of the local buckling resistance and the lateral torsional buckling resistance, which in this design example are equal.

$M_{nc} = 4{,}853$ kip-ft

Multiplying the nominal flexural resistance by the applicable resistance factor gives the following:

$\phi_f M_{nc} = (1.0)(4{,}853 \text{ kip-ft})$

$$\phi_f M_{nc} = 4{,}853 \text{ kip-ft}$$

The flexural resistance is also evaluated in terms of the resistance based on tension flange yielding. For a continuously braced tension flange at the strength limit state, the section must satisfy the requirements of Article A6.1.4.

$$M_u \leq \phi_f R_{pt} M_{yt} \qquad \text{Eq. (A6.1.4-1)}$$

$$\phi_f M_{nt} = \phi_f R_{pt} M_{yt}$$

$$\phi_f M_{nt} = (1.0)(1.072)(4{,}767)$$

$$\phi_f M_{nt} = 5{,}110 \text{ kip-ft}$$

This flexural resistance is less than the applied Strength I factored moment of 5,365 k-ft but is not less than the flexural resistance based on the compression flange. Thus, the flexural resistance based on the compression flange will govern the flexural resistance for the negative bending region of the girder.

$$\phi_f M_{nt} = 5{,}110 \text{ kip-ft} < M_u = 5{,}365 \text{ kip-ft}$$

$$\phi_f M_{nt} = 5{,}110 \text{ kip-ft} > \phi_f M_{nc} = 4{,}853 \text{ kip-ft}$$

$$\phi_f M_n = 4{,}853 \text{ kip-ft}$$

Comparing this flexural resistance to the Strength I factored moment at the pier shows that the factored moment is greater than the flexural resistance. Thus, moment redistribution may be considered.

$$M_u = 5{,}365 \text{ kip-ft} > \phi_f M_{nc} = 4{,}853 \text{ kip-ft}$$

8.2.1.2 Moment Redistribution (Appendix B6, Articles B6.1 – B6.5)

Article B6.2 defines the applicability of the optional Appendix B6 provisions. Specifically, the provisions may only be applied to straight continuous span I-section members whose support lines are not skewed more than 10 degrees from normal and along which there are no staggered (or discontinuous) cross-frames. The specified minimum yield strength of the section must not exceed 70 ksi. Holes are not to be placed within the tension flange over a distance of two times the web depth on either side of the interior-pier sections from which moments ae redistributed. In addition, cross-sections throughout the unbraced lengths immediately adjacent to interior-pier sections from which moments are redistributed must satisfy the web proportion (Article B6.2.1), compression-flange proportion (Article B6.2.2), section transition (Article B6.2.3), compression-flange bracing (Article B6.2.4), and shear (Article B6.2.5) requirements discussed below.

8.2.1.2.1 Web Proportions

Equations B6.2.1-1, B6.2.1-2, and B6.2.1-3 specify the web proportion limits that must be satisfied.

$$\frac{D}{t_w} \leq 150$$ Eq. (B6.2.1-1)

$$\frac{D}{t_w} = \frac{42.0}{0.5} = 84.0 < 150$$ (satisfied)

$$\frac{2D_c}{t_w} \leq 6.8\sqrt{\frac{E}{F_{yc}}}$$ Eq. (B6.2.1-2)

$$\frac{2\left(24.15\right)}{0.50} = 96.60 < 6.8\sqrt{\frac{29,000}{50}} = 163.8$$ (satisfied)

$D_{cp} \leq 0.75D$ Eq. (B6.2.1-3)

D_{cp} = 30.48 in. < 0.75(42.0) = 31.50 in. (satisfied)

8.2.1.2.2 Compression Flange Proportions

Section B6.2.2 requires that the following two compression flange proportion limits be satisfied.

$$\frac{b_{fc}}{2t_{fc}} \leq 0.38\sqrt{\frac{E}{F_{yc}}}$$ Eq. (B6.2.2-1)

$$\frac{16.0}{2(1.25)} = 6.40 < 0.38\sqrt{\frac{29,000}{50}} = 9.15$$ (satisfied)

$$b_{fc} \geq \frac{D}{4.25}$$ Eq. (B6.2.2-2)

$$b_{fc} = 16.0 \text{ in.} > \frac{42}{4.25} = 9.88 \text{ in.}$$ (satisfied)

8.2.1.2.3 Compression Flange Bracing Distance

The compression flange bracing distance must satisfy the following:

$$L_b \leq \left[0.1 - 0.06\left(\frac{M_1}{M_2}\right)\right]\frac{r_t E}{F_{yc}}$$ Eq. (B6.2.4-1)

$$L_b = 120.0 \text{ in.} < \left[0.1 - 0.06\left(\frac{2,999}{5,365}\right)\right]\frac{(4.21)(29,000)}{50} = 162.3 \text{ in.}$$ (satisfied)

8.2.1.2.4 Shear

Additionally, the factored shear under the Strength I loading must be less than or equal to the shear buckling resistance of the girder as follows:

$$V_u \leq \phi_v V_{cr} \qquad \text{Eq. (B6.2.5-1)}$$

where: V_{cr} = shear buckling resistance (kip)

V_{cr} = CV_p (for unstiffened webs) Eq. (6.10.9.2-1)

V_p = plastic shear force (kip)

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

C = ratio of the shear buckling resistance to the shear yield strength determined as specified in Article 6.10.9.3.2, with the shear buckling coefficient, k, taken equal to 5.0

Equations are provided for computing the value of C based on the web slenderness of the girder. First the web slenderness is evaluated using the following equation:

$$\frac{D}{t_w} \leq 1.12\sqrt{\frac{Ek}{F_{yw}}}$$

$$\frac{42.0}{0.50} = 84.0 > 1.12\sqrt{\frac{(29{,}000)(5)}{50}} = 60.31 \qquad \text{(not satisfied)}$$

The web slenderness is next evaluated using the following equation.

$$1.12\sqrt{\frac{Ek}{F_{yw}}} \leq \frac{D}{t_w} \leq 1.40\sqrt{\frac{Ek}{F_{yw}}}$$

$$1.12\sqrt{\frac{Ek}{F_{yw}}} = 60.31 < \frac{D}{t_w} = 84.0 > 1.40\sqrt{\frac{Ek}{F_{yw}}} = 75.4 \qquad \text{(not satisfied)}$$

Thus, the governing equation for computing the ratio C is given by Eq. 6.10.9.3.2-6, which is applicable when:

$$\frac{D}{t_w} = 84.0 > 1.40\sqrt{\frac{Ek}{F_{yw}}} = 75.4 \qquad \text{(satisfied)}$$

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

$$C = \frac{1.57}{(84.0)^2}(2{,}900) = 0.645$$

The shear buckling resistance is then computed as follows.

$$V_{cr} = CV_p = (0.645)(0.58)(50)(42)(0.5) = 392.8 \text{ kips}$$

The shear requirement for Appendix B6 can then be evaluated.

$$V = |-337 \text{ kips}| < \phi_v V_{cr} = (1.0)(392.8) = 392.8 \text{ kips}$$ (satisfied)

In addition, as specified in Article B6.2.6, bearing stiffeners must also be provided at all interior-pier sections from which moments are redistributed (see Section 8.5.1). The provisions of Article B6.2.1 through B6.2.6 are satisfied for this section. Therefore, moments may be redistributed in accordance with Appendix B6.

The effective plastic moment, determined from Article B6.5, is a function of the geometry and material properties of the section. Furthermore, alternative equations are provided for girders that satisfy the requirements for enhanced moment rotation characteristics, i.e., classification as ultracompact sections. To be classified as ultracompact, the girder must either: (1) contain transverse stiffeners at a location less than or equal to one-half the web depth from the pier, or (2) satisfy the web compactness limit given by Eq. B6.5.1-1.

$$\frac{2D_{cp}}{t_w} \le 2.3\sqrt{\frac{E}{F_{yc}}}$$ Eq. (B6.5.1-1)

$$\frac{2(30.48)}{0.50} = 121.9 > 2.3\sqrt{\frac{29{,}000}{50}} = 55.4$$ (not satisfied)

Therefore, the section does not satisfy the web compactness limit and because the section uses an unstiffened web, the girder does not satisfy the transverse stiffener requirement. Thus, the girder is not considered to be ultracompact and the applicable M_{pe} equation at the strength limit state is given by Eq. B6.5.2-2 as follows:

$$M_{pe} = \left[2.63 - 2.3\frac{b_{fc}}{t_{fc}}\sqrt{\frac{F_{yc}}{E}} - 0.35\frac{D}{b_{fc}} + 0.39\frac{b_{fc}}{t_{fc}}\sqrt{\frac{F_{yc}}{E}}\frac{D}{b_{fc}}\right]M_n \le M_n$$ Eq. (B6.5.2-2)

$$M_{pe} = \left[2.63 - 2.3\left(\frac{16}{1.25}\right)\sqrt{\frac{50}{29,000}} - 0.35\left(\frac{42.0}{16.0}\right) + 0.39\left(\frac{16.0}{1.25}\right)\sqrt{\frac{50}{29,000}}\left(\frac{42.0}{16.0}\right) \right] 4,853$$

$$\leq 4,853$$

M_{pe} = 5,013 kip-ft > 4,853 kip-ft = 4,853 kip-ft

The redistribution moment, M_{rd}, for the strength limit state at the interior pier is taken as the larger of the values calculated from Eqs. B6.4.2.1-1 and B6.4.2.1-2.

$$M_{rd} = |M_e| + \frac{1}{3} f_\ell S_{xc} - \phi_f M_{pe} \qquad \text{Eq. (B6.4.2.1-1)}$$

$$M_{rd} = |M_e| + \frac{1}{3} f_\ell S_{xt} - \phi_f M_{pe} \qquad \text{Eq. (B6.4.2.1-2)}$$

where: M_e = critical elastic moment envelope value at the interior-pier section due to the factored loads

Since the lateral bending stresses are negligible for this example at the strength limit state, the previous equations reduce to the following equation:

$$M_{rd} = |M_e| - \phi_f M_{pe}$$

In addition, the redistribution moment is limited to 20 percent of the elastic moment by Eq. B6.4.2.1-3.

$$0 \leq M_{rd} \leq 0.2|M_e| \qquad \text{Eq. (B6.4.2.1-3)}$$

Therefore, the redistribution moment is computed as follows, which is shown to satisfy the 20% limit.

$M_{rd} = |M_e| - \phi_f M_{pe}$ = 5,365 - (1.0)(4,853)

M_{rd} = 512 kip-ft = 9.5% M_e < 20% M_e

As specified in Article B6.4.1.1, the flexural resistance of sections within the unbraced lengths immediately adjacent to interior-pier sections from which moments are redistributed need not be checked. Sections at all other locations must satisfy the provisions of Articles 6.10.7, 6.10.8.1, or A6.1, as applicable, after moment redistribution (Article B6.4.1.2)..

8.2.1.3 Moment Redistribution - Refined Method (Appendix B6, Article B6.6)

Article B6.6 of Appendix B6 contains specifications for computing redistribution moments at the strength and service limit states using a direct method of analysis. Using this analysis procedure, the effective plastic moments are computed based on the rotation at which the continuity curve intersects the moment-rotation curve, as opposed to assuming that this intersection occurs at a plastic rotation of 30 mrads, as was assumed in the development of the effective plastic moment equations utilized above. If the refined method is used for calculation of the redistribution moments, all interior-pier sections are not required to satisfy the requirements of Article B6.2; however, moments are not to be redistributed from sections that do not satisfy these requirements. Such sections instead must satisfy the provisions of Articles 6.10.4.2, 6.10.8.1 or Article A6.1, as applicable, after redistribution.

In cases such as this example, where the effective plastic moment is equal to the nominal flexural resistance of the negative bending section, there is no real advantage to be gained by using the refined method. This is because the peak value of the moment-rotation curve is equal to M_n, the maximum value of M_{pe} possible, irrespective of using the effective plastic moment equations from Article B6.5 or the refined method of Article B6.6. However, in other cases the use of the refined method may lead to higher values of M_{pe}, further increasing the economic benefits of using the moment redistribution procedures. For this reason, the use of the refined method for the present design at the strength limit state is demonstrated below (the application of the method at the service limit state is similar – see Articles B6.6 and B6.3).

The first step in using the refined method for moment redistribution is to determine the moment-rotation curve for the negative bending section. This is done using Figure B6.6.2-1 from the *AASHTO LRFD BDS* , which is reproduced in Figure 13. At the strength limit state, the ordinates of the curve are to be multiplied by the resistance factor for flexure, ϕ_f. From Figure 13 it is observed that the moment-rotation relationship is a function of the single parameter, θ_{RL}, which is the rotation at which the moment begins to decrease below the nominal flexural resistance. Similar to the equations for M_{pe} given for the simplified method introduced above, alternative equations for θ_{RL} are given based on whether the negative bending section satisfies the criteria for enhanced moment rotation characteristics given by Section B6.5. It has been shown above that the negative bending section does not satisfy either of the requirements for sections with enhanced moment-rotation performance. Thus, θ_{RL} is given in radians by Eq. B6.6.2-2.

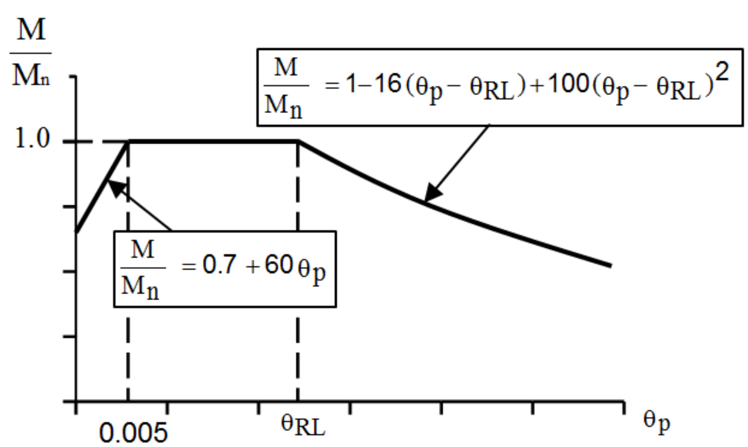

Figure 13 AASHTO LRFD BDS Moment-Rotation Model

$$\theta_{RL} = 0.128 - 0.143\frac{b_{fc}}{t_{fc}}\sqrt{\frac{F_{yc}}{E}} - 0.0216\frac{D}{b_{fc}} + 0.0241\frac{b_{fc}}{t_{fc}}\frac{D}{b_{fc}}\sqrt{\frac{F_{yc}}{E}} \quad \text{Eq. (B6.6.2-2)}$$

Substituting the applicable values into Eq. B6.6.2-2 gives the following.

$$\theta_{RL} = 0.128 - 0.143\left(\frac{16.0}{1.25}\right)\sqrt{\frac{50}{29,000}} - 0.0216\left(\frac{42}{16}\right) + 0.0241\left(\frac{16.0(42.0)}{1.25(16.0)}\right)\sqrt{\frac{50}{29,000}}$$
$$= 0.029$$

Thus, θ_{RL} is equal to 0.029 radians or 29 mrads. Recalling that the nominal flexural resistance of the negative moment section of this girder, M_n, is 4,853 kip-ft, the predicted moment-rotation relationship of the example girder is as illustrated in Figure 14.

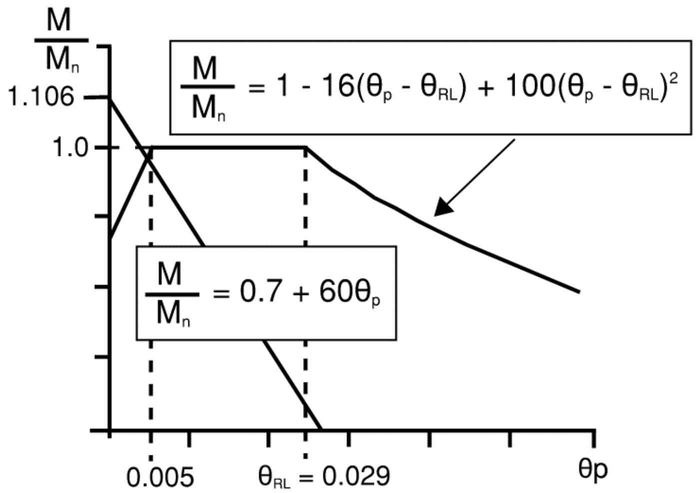

Figure 14 Determination of M_{pe} Using Refined Method

In addition to the moment-rotation relationship, the continuity relationship must also be determined. The continuity relationship is a linear relationship between the elastic moment at the pier (where no plastic rotation occurs) and the rotation assuming no continuity at the pier. The factored elastic Strength I moment at the pier has previously been determined to equal 5365 kip-ft; thus, the y-intercept for the continuity relationship is $M/\phi_f M_n$ = 5,365/(1.0)(4,853) = 1.106. To determine the x-intercept of the continuity relationship, the beam is analyzed assuming that a hinge exists at each pier, and rotations due to applied moments equal to the elastic moment are computed as shown in Figure 15. In this analysis, the *AASHTO LRFD BDS* stipulates that the section properties of the short-term composite section are to be used. Thus, the applicable moment of inertia of the positive bending section is 49,905 in.^{4} and the moment of inertia value used for the negative bending section is 50,027 in^4. From basic structural analysis, or the use of structural analysis software, the rotation at the pier for the situation depicted in Figure 15 is computed to be 0.032 rads = 32 mrads, which is the x-intercept for the continuity relationship. Based on the x- and y- intercepts of the continuity relationship, the continuity equation is thus expressed as

$$M/\phi_f M_n = 1.106 - 34.5625 * \theta_p$$

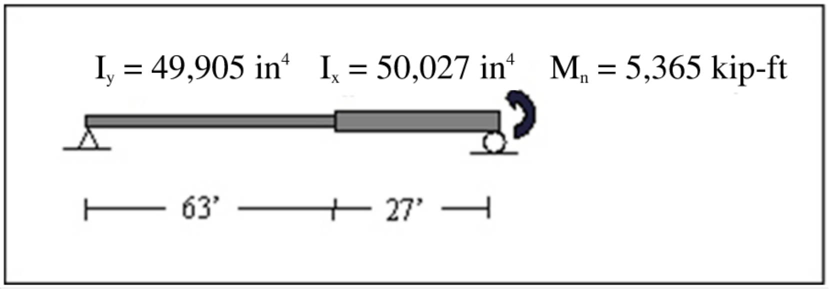

Figure 15 Determination of Rotation at Pier Assuming No Continuity

The moment at the intersection of the continuity relationship and the moment-rotation relationship is the effective plastic moment. Since the continuity relationship intersects the moment-rotation relationship at a value of θ_p less than 5 mrads, the moment-rotation relationship is (Figure 14):

$$M/\phi_f M_n = 0.7 + 60 * \theta_p$$

The effective plastic moment is determined mathematically by iteratively selecting θ_p values to be substituted into both the moment-rotation and continuity relationships until the moment converges. After several iterations, θ_p is determined to be 0.00429 rads = 4.29 mrads and $M/\phi_f M_n$ = 0.957. Therefore, M_{pe} is equal to $0.957(\phi_f M_n) = 0.957(1.0)(4{,}853) = 4{,}644$ kip-ft.

Once M_{pe} is determined, the moment redistribution analysis proceeds in the same manner used in the simplified method outlined above, where the redistribution moments are computed as the difference between the elastic and the effective plastic moments as specified in Article B6.4.2 and the flexural resistance of the unbraced lengths immediately adjacent to interior-pier sections from which moments are redistributed need not be checked if the redistribution moment is less than 20% of the elastic moment.

8.2.1.4 Shear (6.10.6.3)

As computed above the shear resistance of the negative bending region is governed by Article 6.10.9.2 because the girder is comprised of an unstiffened web, i.e., no transverse stiffeners are provided. The nominal shear resistance of the section was previously calculated to be:

$$V_n = CV_p = 392.8 \text{ kips} \qquad \text{Eq. (6.10.9.2-1)}$$

The factored shear at the pier at the strength limit state is -337 kips. Thus, the shear requirements are satisfied.

$$V = |-337| \text{kips} < \phi_V V_n = (1.0)(392.8) = 392.8 \text{ kips}$$

(satisfied)

8.2.2 Constructability (Article 6.10.3)

Article 2.5.3 requires the Engineer to design bridge systems such that the construction does not result in unacceptable locked-in forces. In addition, Article 6.10.3 states the main load-carrying members are not permitted to experience nominal yielding or reliance on post-buckling resistance during the construction phases. The sections must satisfy the requirements of Article 6.10.3 at each construction stage 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.

The girders are considered to be noncomposite during the initial construction phase. The influence of various segments of the girder becoming composite at various stages of the deck casting sequence is to be considered. The effects of forces from deck overhang brackets acting on the fascia girders are also to be considered in the constructability checks.

8.2.2.1 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, which are 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 design checks 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 (or may be smaller than) the calculated DC_1 negative moments ignoring the effects of the sequential deck placement. 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 later on in this example for Section 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 considered to be a discretely braced compression flange and the top flange would be considered to be a discretely braced tension flange for all constructability checks to be made before the concrete deck has hardened or is made composite. The nominal flexural resistance of the bottom flange, F_{nc}, for checking Eq. 6.10.3.2.1-2 would be calculated in a manner similar to that demonstrated below for Section 1. For the sake of brevity, 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 will not be shown in this example.
Note that for sections with slender webs, web bend-buckling should always be checked in regions of negative flexure according to Eq. 6.10.3.2.1-3 for critical stages of construction. In this example, however, Section 2 is not a slender-web section.

8.2.2.2 Shear (Article 6.10.3.3)

The required shear resistance during construction is specified by Eq. 6.10.3.3-1. Later in this design example, the unstiffened shear resistance of the girder is demonstrated to be sufficient to resist the factored shear at the strength limit state. Therefore, the section will have sufficient shear resistance for the constructability check.

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

8.2.3 Service Limit State (Article 6.10.4)

Permanent deformations are controlled under the service limit state. Service limit state design checks for steel I-girder bridges are specified in Article 6.10.4.

8.2.3.1 Permanent Deformations (Article 6.10.4.2)

Permanent deformations that may negatively impact the rideability of the structure are controlled by limiting the stresses in the section under expected severe traffic loadings. Specifically, under the Service II load combination, the top flange of composite sections must satisfy:

$$f_f \leq 0.95 R_h F_{yf} \qquad \text{Eq. (6.10.4.2.2-1)}$$

Because the bottom flange is discretely braced (as opposed to the top flange), Eq. 6.10.4.2.2-2 must be satisfied for the bottom flange of composite sections as follows:

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

At the service limit state, the lateral force effects due to wind loads and deck overhang loads are not considered. Therefore, for bridges with straight, non-skewed girders such as the case in the present design example, the lateral bending stresses are taken equal to zero and Eq. 6.10.4.2.2-2 reduces to Eq. 6.10.4.2.2-1.

For sections satisfying the requirements of Article B6.2, Appendix B6 permits the redistribution of moment at the service limit state before evaluating the above equations. As demonstrated previously, Section 2 satisfies the requirements of Article B6.2. Article B6.5.2 specifies the effective plastic moment to be used at the service limit state as follows:

$$M_{pe} = \left[2.90 - 2.3\frac{b_{fc}}{t_{fc}}\sqrt{\frac{F_{yc}}{E}} - 0.35\frac{D}{b_{fc}} + 0.39\frac{b_{fc}}{t_{fc}}\sqrt{\frac{F_{yc}}{E}}\frac{D}{b_{fc}}\right]M_n \leq M_n \qquad \text{Eq. (B6.5.2-1)}$$

$$M_{pe} = \left[2.90 - 2.3\left(\frac{16.0}{1.25}\right)\sqrt{\frac{50}{29,000}} - 0.35\left(\frac{42.0}{16.0}\right) + 0.39\left(\frac{16.0}{1.25}\right)\sqrt{\frac{50}{29,000}}\left(\frac{42.0}{16.0}\right)\right]4,853$$
$$\leq 4,853$$

M_{pe} = 6,323 kip-ft > 4,853 kip-ft

M_{pe} = 4,853 kip-ft > M_u = 4,075 kip-ft

Because the effective plastic moment is greater than the maximum factored moment for the Service II load combination, it is assumed that there is no moment redistribution at this limit state. The elastic stresses under the Service II load combination are therefore computed using the following equation assuming no moment redistribution:

$$f_f = \frac{M_{DC1}}{S_{nc}} + \frac{M_{DC2} + M_{DW}}{S_{lt}} + \frac{1.3M_{LL+IM}}{S_{st}}$$

For members with shear connectors provided throughout their entire length that also satisfy the provisions of Article 6.10.1.7, and where the maximum longitudinal tensile stresses in the concrete deck at the section under consideration caused by the Service II loads are smaller than $2f_r$, 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. f_r is the modulus of rupture of the concrete specified in Article 6.10.1.7.

Separate calculations (not shown) were made to verify that the minimum longitudinal reinforcement (determined previously) satisfied the provisions of Article 6.10.1.7 for both the factored construction loads and the Service II loads. Check the maximum longitudinal tensile stresses in the concrete deck under the Service II loads at Section 2. 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.

$$f_r = 0.24\sqrt{f_c^{'}} = 0.24\sqrt{4.0} = 0.48 \text{ ksi}$$

$$f_{deck} = \frac{1.0\left[1.0\left|-265\right| + 1.0\left|-217\right| + 1.3\left|-1,737\right|\right](14.235)(12)}{50,027(8)} = 1.17 \text{ ksi} > 2f_r$$

$$= 2(0.48) = 0.96 \text{ ksi}$$

Therefore, since the concrete deck may not be considered effective in tension at Section 2, the Service II flexural stresses will be computed using the section consisting of the steel girder plus the longitudinal reinforcement only for loads applied to the composite section.

The stress in the compression flange is thus computed as follows.

$$f_f = \frac{(1.0)(\text{-}1,334)(12)}{951.3} + \frac{(1.0)(\text{-}265 \text{ - } 217)(12)}{1,079} + \frac{1.3(\text{-}1,737)(12)}{1,079} = -47.30 \text{ ksi}$$

Comparing this stress to the allowable stress shows that Eq. 6.10.4.2.2-1 is satisfied within an acceptable tolerance; the applied stress and the stress limit differ by approximately one percent.

$$f_f = \left|\text{-}47.30 \text{ ksi}\right| > 0.95R_hF_{yf} = 0.95(1.0)(50) = 47.50 \text{ ksi} \qquad \text{(satisfied)}$$

Similarly, the computation of the stress in the tension flange is computed as follows.

$$f_f = \frac{(1.0)(-1{,}334)(12)}{825.6} + \frac{(1.0)(-265 - 217)(12)}{1{,}444} + \frac{1.3(-1{,}737)(12)}{1{,}444} = 42.16 \text{ ksi}$$

Thus, it is also demonstrated that Eq. 6.10.4.2.2-2 is satisfied for the tension flange.

$$f_f = 42.16 \text{ ksi} \leq 0.95R_hF_{yf} = 0.95(1.0)(50) = 47.50 \text{ ksi} \qquad \text{(satisfied)}$$

The compression flange stress at service loads is also limited to the elastic bend-buckling resistance of the web by Eq. 6.10.4.2.2-4.

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

where: f_c = compression flange stress at the section under consideration due to the Service II loads calculated without consideration of flange lateral bending

F_{crw} = nominal elastic bend buckling resistance for webs with or without longitudinal stiffeners, as applicable, determined as specified in Article 6.10.1.9

From Article 6.10.1.9, the bend-buckling resistance for the web is determined using the following equation.

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

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

As specified in Article D6.3.1, the depth of web in compression for composite sections in negative flexure where the concrete deck is not considered to be effective in tension at the service limit state is to be calculated for the section consisting of the steel girder plus the longitudinal reinforcement.

$$D_c = 25.40 – 1.25 = 24.15 \text{ in.}$$

Therefore, k and F_{crw} are computed as follows.

$$k = \frac{9}{\left(\frac{24.15}{42.0}\right)^2} = 27.2$$

$$F_{crw} = \frac{0.9(29{,}000)(27.2)}{\left(\frac{42}{0.50}\right)^2} = 100.6 \text{ ksi} > R_h F_{yc} = 50 \text{ ksi}$$

$\therefore\ F_{crw} = 50$ ksi

It can then be demonstrated that Eq. 6.10.4.2.2-4 is satisfied as shown below.

$$f_c = |\text{-}47.30 \text{ ksi}| \leq F_{crw} = 50 \text{ ksi} \qquad \text{(satisfied)}$$

8.2.4 Fatigue and Fracture Limit State (Article 6.10.5)

The fatigue and fracture limit state incorporates three distinctive checks: fatigue resistance of details (Article 6.10.5.1), fracture toughness (Article 6.10.5.2), and a special fatigue requirement for webs (Article 6.10.5.3). The first requirement involves the assessment of the fatigue resistance of details as specified in Article 6.6.1 using the appropriate Fatigue load combination specified in Table 3.4.1-1 and the fatigue live load specified in Article 3.6.1.4. The fracture toughness requirements in Article 6.10.5.2 specify that the fracture toughness must satisfy the requirements of Article 6.6.2.1. The special fatigue requirement for the web controls the elastic flexing of the web to prevent fatigue cracking. The factored fatigue load for this check is to be taken as the Fatigue I load combination specified in Table 3.4.1-1.

8.2.4.1 Load Induced Fatigue (Article 6.6.1.2)

Article 6.10.5.1 requires that fatigue be investigated in accordance with Article 6.6.1. Article 6.6.1 requires that the live load stress range be less than the nominal fatigue resistance. The nominal fatigue resistance, $(\Delta F)_n$, varies based on the fatigue detail category and is computed using Eq. 6.6.1.2.5-1 for the Fatigue I load combination and infinite fatigue life; or Eq. 6.6.1.2.5-2 for Fatigue II load combination and finite fatigue life.

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

$$(\Delta F)_n = \left(\frac{A}{N}\right)^{\frac{1}{3}} \qquad \text{Eq. (6.6.1.2.5-2)}$$

where: N = $(365)(75)n(ADTT)_{SL}$ Eq. (6.6.1.2.5-3)

A = detail category constant from Table 6.6.1.2.5-1

n = number of stress range cycles per truck passage taken from Table 6.6.1.2.5-2

$(ADTT)_{SL}$ = single-lane ADTT as specified in Article 3.6.1.4

$(\Delta F)_{TH}$ = constant-amplitude fatigue threshold taken from Table 6.6.1.2.5-3

The fatigue resistance of the base metal at the weld joining the cross-frame connection plate located 10 feet from the pier to the flanges is evaluated below. From Table 6.6.1.2.3-1, it is determined that this detail is classified as a fatigue Detail Category C'.

For this example, a projected $(ADTT)_{SL}$ of 950 trucks per day is assumed. Since this $(ADTT)_{SL}$ is less than the value of the 75-year $(ADTT)_{SL}$ Equivalent to Infinite Life for n equal to 1.0 of 975 trucks per day specified in Table 6.6.1.2.3-2 for a Category C′ detail, the nominal fatigue resistance for this particular detail is to be determined for the Fatigue II load combination and finite fatigue life using Eq. 6.6.1.2.5-2. Therefore:

$$(\Delta F)_n = \left(\frac{A}{N}\right)^{\frac{1}{3}} \qquad \text{Eq. (6.6.1.2.5-2)}$$

For a Detail Category C', the detail category constant, A, is 44 x 10^8 ksi^3 (Table 6.6.1.2.5-1).

$$N = (365)(75)n(ADTT)_{SL} \qquad \text{Eq. (6.6.1.2.5-3)}$$

$$N = (365)(75)(1.0)(950) = 26.0 \text{ x } 10^6 \text{ cycles}$$

Therefore:

$$(\Delta F)_n = \left(\frac{44 \text{ x } 10^8}{26.0 \text{ x } 10^6}\right)^{\frac{1}{3}} = 5.5 \text{ ksi}$$

The applied stress range is taken as the result of the fatigue loading with a dynamic load allowance of 15 percent applied and distributed laterally by the previously calculated distribution factor for fatigue.

According to Article 6.6.1.2.1, for flexural members with shear connectors provided throughout their entire length and with concrete deck reinforcement satisfying the provisions of Article 6.10.1.7, flexural stresses and stress ranges applied to the composite section at the fatigue limit state at all sections in the member may be computed assuming the concrete deck to be effective for both positive and negative flexure. Shear connectors are assumed provided along the entire length of the girder in this example. Separate computations (not shown) were made to verify 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 dead load and live load stresses and live load stress ranges applied to the composite section in the subsequent fatigue calculations.

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 the maximum tensile stress resulting from the Fatigue I load combination specified in Table 3.4.1-1. 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. At this location, the unfactored permanent loads produce tension at the top of the girder and compression at the bottom of the girder. 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.

Bottom of Top Flange:

$$\gamma(\Delta f) = (0.80)\left[\frac{(115)(12)(4.24)}{50{,}027} + \frac{|-314|(12)(4.24)}{50{,}027}\right]$$

$\gamma(\Delta f) = 0.35 \text{ ksi} < (\Delta F)_n = 5.5 \text{ ksi}$ (satisfied)

Top of Bottom Flange:

$$f_{DC1} = \frac{(-668)(12)(19.37)}{19{,}616} = -7.92 \text{ ksi}$$

$$f_{DC2} = \frac{(-133)(12)(30.10)}{37{,}222} = -1.29 \text{ ksi}$$

Σ = -7.92 + -1.29 = -9.21 ksi

$$f_{LL+IM} = \frac{1.75(115)(12)(37.77)}{50{,}027} = 1.82 \text{ ksi}$$

$|-9.21 \text{ ksi}| > 1.82 \text{ ksi}$ $\therefore$ fatigue does not need to be checked

8.2.4.2 Distortion Induced Fatigue (Article 6.6.1.3)

A positive connection is to be provided for all transverse connection-plate details to both the top and bottom flanges to prevent distortion induced fatigue.

8.2.4.3 Fracture (Article 6.6.2)

Material for primary load-carrying components subject to tensile stress under the Strength I load combination is assumed for this example to be ordered to meet the appropriate Charpy V-notch fracture toughness requirements for nonfracture-critical material (Table C6.6.2.1-1) specified for Temperature Zone 2 (Table 6.6.2.1-2).

8.2.4.4 Special Fatigue Requirement for Webs (Article 6.10.5.3)

Article 6.10.5.3 requires that the shear force applied due to the unfactored permanent loads plus the factored fatigue loading (i.e., the Fatigue I load combination) must be less than the shear-buckling resistance in interior panels of stiffened webs.

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

However, designs utilizing unstiffened webs at the strength limit state, as is the case here, automatically satisfy this criterion. Thus, Eq. 6.10.5.3-1 is not explicitly evaluated herein.

8.3 Exterior Girder Check: Section 1

8.3.1 Constructability (Article 6.10.3)

8.3.1.1 Deck Placement Analysis

In regions of positive flexure, temporary moments that the noncomposite girders experience during the sequential placement of the deck can sometimes be significantly higher than the final noncomposite dead load moments after the sequential placement is complete. An analysis of the moments during each sequential placement must be conducted to determine the maximum moments in the structure acting on the noncomposite girders in those regions. The potential for uplift during the deck placement should also be investigated.

Figure 16 depicts the deck placement sequence assumed in this design example. Note that for simplicity in this illustration, the sequence assumes that the concrete is cast in the two end spans at approximately the same time. Typically, it is more common to cast the two placements in the end spans in sequence. 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. Because this example assumes both sections labeled "1" are cast at the same time, it underpredicts the maximum positive moment during deck casting that would occur if only the region labeled "1" were cast in a single span.

As required in Article 6.10.3.4.1, the loads must be applied to the appropriate section during each sequential placement. For example, it is assumed during the first placement that all sections of the girder are noncomposite. Similarly, the dead load moments due to the steel components are also based on the noncomposite section properties. However, to determine the distribution of moments due to the second placement, the short-term composite section properties are used in the regions of the girders that were previously cast in the first placement (since the deck placements are relatively short-term loadings), while the noncomposite section properties are used in the remaining regions of the girder for the second placement. The moments used in the evaluation of the constructability requirements are then taken as the maximum moments *that occur on the noncomposite section* during any stage of construction, i.e., the sum of the moments due to the steel dead load and the first placement or the sum of the moments due to the steel dead load and both placements, as applicable. Additionally, while not required, the dead load moment assuming all the dead load is applied at once (i.e., without consideration of the sequential placement) to the noncomposite section (DC_1) is also considered. Refer to NSBA's *Steel Bridge Design Handbook: Design Example 1* [3] for further discussion on the deck placement analysis.

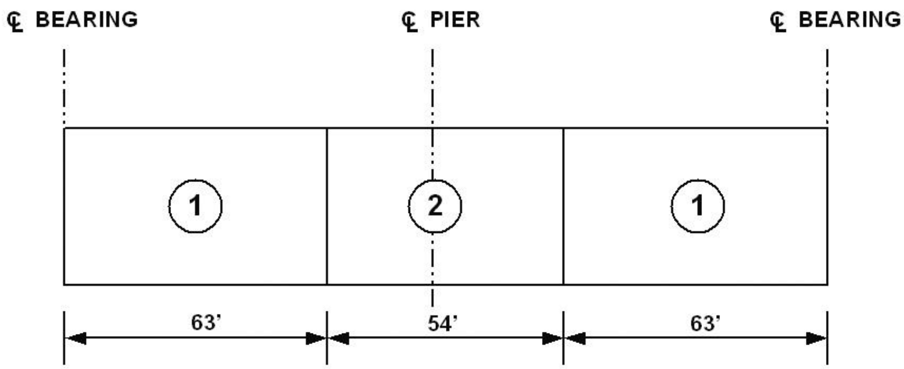

Figure 16 Deck Placement Sequence

The results of the deck placement analysis are shown in Table 17 where the maximum dead load moments in the positive bending region acting on the noncomposite section at Section 1 are indicated by bold text. Note that because of the deck placement sequence chosen for this example and the relatively short spans, the maximum positive bending moment acting on the noncomposite section is not caused by the sequential deck placement (i.e., Cast 1). Therefore, the DC_1 moment of 738 kip-ft at Section 1, ignoring the effect of the sequential deck placement, will be used in the subsequent constructability design checks for Section 1.

Table 17 Moments from Deck Placement Analysis (kip-ft)

x/L	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
Dist. (ft.)	0	9	18	27	36	45	54	63	72	81	90
Steel Wt.	0	49	82	101	106	96	71	31	-22	-90	-173
SIP Forms	0	27	46	56	58	53	39	16	-13	-50	-94
Cast 1	0	260	437	532	544	474	321	86	-181	-447	-714
Cast 2	0	301	518	654	707	677	565	370	105	-238	-656
Σ Cast 1	0	335	565	689	**708**	622	430	133	-216	-587	-981
Σ Cast 2	0	376	646	811	871	825	674	417	70	-377	-923
DC1	0	343	581	712	**738**	657	471	178	-220	-724	-1334

Article 6.10.1.6 states that when checking the flexural resistance based on lateral torsional buckling, f_{bu} is to be taken as the largest compressive stress in the flange under consideration, without consideration of flange lateral bending, throughout the unbraced length. When checking the flexural resistance based on yielding, flange local buckling or web bend buckling, f_{bu} is to be taken as the stress at the section under consideration. The maximum factored flexural stresses within the unbraced length containing Section 1 occur right at Section 1; the resulting DC_1 stresses are calculated below. The load modifier, η, is taken equal to 1.0.

8.3.1.1.1 Strength I

Top Flange

$$f_{bu} = \frac{1.0(1.25)(738)(12)}{635.9} = -17.41\,\text{ksi}$$

Bottom Flange

$$f_{bu} = \frac{1.0(1.25)(738)(12)}{900.7} = 12.29\,\text{ksi}$$

8.3.1.1.2 Special Load Combination (Article 3.4.2.1)

Top Flange

$$f_{bu} = \frac{1.0(1.4)(738)(12)}{635.9} = -19.50\,\text{ksi}$$

Bottom Flange

$$f_{bu} = \frac{1.0(1.4)(738)(12)}{900.7} = 13.77\,\text{ksi}$$

8.3.1.2 Deck Overhang Loads

The loads applied to the deck overhang brackets induce torsion on the fascia girders, which introduces flange lateral bending stresses. This section illustrates the recommended approach to estimate these lateral bending stresses.

The deck overhang bracket configuration assumed in this example is shown in Figure 17. Typically, the brackets are spaced between 3 and 4 feet, but the assumption is made here that the loads are uniformly distributed, except for the finishing machine. Half of the overhang weight is assumed to be carried by the exterior girder, and the remaining half is assumed carried by the overhang brackets.

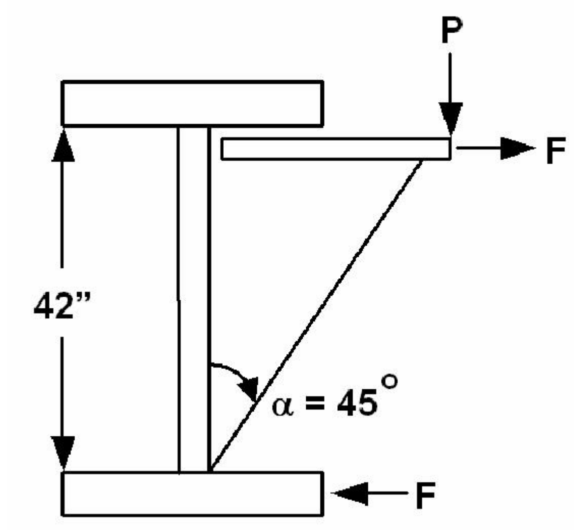

Figure 17 Deck Overhang Bracket Loads

The following calculation determines the weight of the deck overhang acting on the overhang brackets.

$$P = 0.5(150)\left[\frac{8.5}{12}(3.5) + \left[\frac{1}{12}\left(\frac{2.0}{2}\right)\left(3.5 - \frac{14.0/2}{12}\right)\right] + \frac{1.25}{12}\left(\frac{14.0/2}{12}\right)\right] = 209 \text{ lbs/ft}$$

The following is a list of typical construction loads assumed to act on the system before the concrete slab gains strength. The magnitudes of load listed are those that are applied to only the overhang brackets. Note that the finishing machine load shown represents one-half of the finishing machine truss weight.

Overhang Deck Forms:	P = 40 lb/ft
Screed Rail:	P = 85 lb/ft
Railing:	P = 25 lb/ft
Walkway:	P = 125 lb/ft
Finishing Machine:	P = 3,000 lb

The lateral force acting on the girder section due to the vertical loading is computed as follows.

$$F = P \tan \alpha$$

where: $\alpha = \tan^{-1}\left(\frac{42 \text{ in.}}{42 \text{ in.}}\right) = 45^\circ$

The equations provided in Article C6.10.3.4.1 to determine the lateral bending moment can be employed in the absence of a more refined method. From the article, the following equation determines the lateral bending moment for a uniformly distributed lateral bracket force:

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

where: M_ℓ = lateral bending moment in the top flange due to the eccentric loadings from the form brackets

F_ℓ = statically equivalent uniformly distributed lateral force due to the factored loads

The equation which estimates the lateral bending moment due to a concentrated lateral force at the middle of the unbraced length is as follows:

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

where: P_ℓ = statically equivalent concentrated force placed at the middle of the unbraced length

For simplicity, the largest value of f_ℓ within the unbraced length is conservatively used in the design checks, i.e., the maximum value of f_ℓ within the unbraced length is the assumed stress level throughout the unbraced length. The unbraced length for the section under consideration is 20 feet.

Article 6.10.1.6 specifies the process for determining the lateral bending stress. The first-order lateral bending stress may be used if the following limit is satisfied.

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

where: L_p = limiting unbraced length from Article 6.10.8.2.3 of the Specifications

C_b = moment gradient modifier

R_b = web load-shedding factor

F_{yc} = yield strength of the compression flange

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 as 1.0 when checking constructability.

Calculate L_p:

D_c = 25.79 – 0.75 = 25.04 in.

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\right)}} = \frac{14}{\sqrt{12\left(1+\frac{1}{3}\frac{(25.04)(0.5)}{14.0(0.75)}\right)}} = 3.42 \text{ in.}$$

$$L_p = 1.0 r_t \sqrt{\frac{E}{F_{yc}}} = 1.0(3.42)\sqrt{\frac{29{,}000}{50}} = 82.4 \text{ in.} \qquad \text{Eq. (6.10.8.2.3-4)}$$

Thus, Eq. 6.10.1.6-2 is evaluated as follows.

$$L_b = 240 \text{ in.} > 1.2(82.4)\sqrt{\frac{(1.0)(1.0)}{|-17.41|/50}} = 167.6 \text{ in.}$$

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

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

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

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

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

$$F_{cr} = \frac{1.0(1.0)\pi^2(29{,}000)}{\left(\frac{20(12)}{3.42}\right)^2} = 58.12 \text{ ksi}$$

Note that the calculated value of F_{cr} for use in Eq. 6.10.1.6-4 is not limited to $R_b R_h F_{yc}$ (Article C6.10.1.6).

The amplification factor is then determined as follows:

For Strength I:

$$AF = \frac{0.85}{\left(1 - \frac{|-17.41|}{58.12}\right)} = 1.21 > 1.0 \quad \text{ok}$$

For the Special Load Combination specified in Article 3.4.2.1:

$$AF = \frac{0.85}{\left(1 - \frac{|-19.50|}{58.12}\right)} = 1.28 > 1.0 \quad \text{ok}$$

AF is taken equal to 1.0 for tension flanges.

8.3.1.2.1 Strength I

The flange lateral bending stresses for the Strength I load combination are computed as follows. As specified in Article 3.4.2.1, the load factor for construction loads and any associated dynamic effects is not to be taken less than 1.5 for the Strength I load combination.

Dead loads:

P = [1.25(209) + 1.5(40 + 85 + 25 + 125)] = 673.8 lbs/ft

$F = F_\ell = P \tan \alpha$ = 673.8 tan (45°) = 673.8 lbs/ft

$$M_\ell = \frac{F_\ell L_b^2}{12} = \frac{(0.6738)(20.0)^2}{12} = 22.46 \text{ kip} - \text{ft}$$

$$\text{Top Flange: } f_\ell = \frac{M_\ell}{S_\ell} = \frac{22.46(12)}{0.75(14.0)^2/6} = 11.00 \text{ ksi}$$

$$\text{Bottom Flange: } f_\ell = \frac{M_\ell}{S_\ell} = \frac{22.46(12)}{1.25(16.0)^2/6} = 5.05 \text{ ksi}$$

Finishing machine load:

P = [1.5(3,000)] = 4,500 lbs

$F = P_\ell = P \tan \alpha$ = 4,500 tan (45°) = 4,500 lbs

$$M_\ell = \frac{P_\ell L_b}{8} = \frac{(4.5)(20.0)}{8} = 11.25 \text{ kip} - \text{ft}$$

$$\text{Top Flange: } f_\ell = \frac{M_\ell}{S_\ell} = \frac{11.25(12)}{0.75(14.0)^2/6} = 5.51 \text{ ksi}$$

$$\text{Bottom Flange: } f_\ell = \frac{M_\ell}{S_\ell} = \frac{11.25(12)}{1.25(16.0)^2/6} = 2.53 \text{ ksi}$$

Total:

Top flange: f_ℓ = (11.00 + 5.51)AF = (11.00 + 5.51)(1.21) = 19.98 ksi

Bot. flange: f_ℓ = (5.05 + 2.53)AF = (5.05 + 2.53)(1.0) = 7.58 ksi

8.3.1.2.2 Special Load Combination (Article 3.4.2.1)

The computation of the flange lateral bending stresses for the special load combination specified in Article 3.4.2.1 is demonstrated below.

Dead loads:

$$P = \left[1.4\left(209 + 40 + 85 + 25 + 125\right)\right] = 677.6 \text{ lbs / ft}$$

$$F = F_\ell = P \tan \alpha = 677.6 \tan\left(45^\circ\right) = 677.6 \text{ lbs / ft}$$

$$M_\ell = \frac{F_\ell L_b^2}{12} = \frac{(0.6776)(20.0)^2}{12} = 22.59 \text{ kip} - \text{ft}$$

$$\text{Top Flange: } f_\ell = \frac{M_\ell}{S_\ell} = \frac{22.59(12)}{0.75(14.0)^2/6} = 11.06 \text{ ksi}$$

$$\text{Bottom Flange: } f_\ell = \frac{M_\ell}{S_\ell} = \frac{22.59(12)}{1.25(16.0)^2/6} = 5.08 \text{ ksi}$$

Finishing machine load:

P = [1.4(3,000)] = 4,200 lbs

F = P_ℓ = P tan α = 4,200 tan (45°) = 4,200 lbs

$$M_\ell = \frac{P_\ell L_b}{8} = \frac{(4.2)(20.0)}{8} = 10.50 \text{ kip} - \text{ft}$$

Top Flange: $f_\ell = \dfrac{M_\ell}{S_\ell} = \dfrac{10.50(12)}{0.75(14.0)^2/6} = 5.14 \text{ ksi}$

Bottom Flange: $f_\ell = \dfrac{M_\ell}{S_\ell} = \dfrac{10.50(12)}{1.25(16.0)^2/6} = 2.36 \text{ ksi}$

Total:

Top flange: f_ℓ = (11.06 + 5.14)(AF) = (11.06 + 5.14)(1.28) = 20.74 ksi

Bot. flange: f_ℓ = (5.08 + 2.36)(AF) = (5.08 + 2.36)(1.0) = 7.44 ksi

According to Article 6.10.1.6, the flange lateral bending stresses (after amplification) must be less than 60 percent of the yield stress of the flange under consideration. It is shown above that the lateral bending stresses are highest in the top flange under the Special Load Combination, and highest in the bottom flange under the Strength I load combination. Thus:

$$f_\ell \leq 0.6F_y \qquad \text{Eq. (6.10.1.6-1)}$$

Top flange: f_ℓ = 20.74 ksi < $0.6F_{yf}$ = 30 ksi (satisfied)

Bot. flange: f_ℓ = 7.58 ksi < $0.6F_{yf}$ = 30 ksi (satisfied)

8.3.1.3 Flexure (Article 6.10.3.2)

During construction, both the compression and tension flanges are discretely braced. Therefore, Article 6.10.3.2 requires the noncomposite section to satisfy Eqs. 6.10.3.2.1-1, 6.10.3.2.1-2, and 6.10.3.2.1-3, which verifies the flange stress is limited to the yield stress, the section has sufficient strength under the lateral torsional and flange local buckling limit states, and web bend buckling does not occur during construction, respectively.

First, determine if the noncomposite section satisfies the noncompact slenderness limit as follows:

$$\frac{2D_c}{t_W} \leq \lambda_{rw} \qquad \text{Eq. (6.10.6.2.3-1)}$$

where:

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

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

$$\frac{2D_c}{t_w} = \frac{2(25.04)}{0.5} = 100.2$$

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

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

$$a_{wc} = \frac{2(25.04)(0.5)}{14.0(0.75)} = 2.38$$

$$111 < \lambda_{rw} = \left(3.1 + \frac{5.0}{2.38}\right)\sqrt{\frac{29,000}{50}} = 125.3 < 137$$

$$\therefore \lambda_{rw} = 125.3 > \frac{2D_c}{t_w} = 100.2 \qquad \text{(satisfied)}$$

The section is nonslender (i.e., the section has a compact or noncompact web). Therefore, Eq. 6.10.3.2.1-3 (web bend-buckling) need not be checked.

8.3.1.3.1 Compression Flange:

Flange tip yielding:

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

Since the section under consideration is homogeneous, the hybrid factor, R_h, is 1.0, as stated in Article 6.10.1.10.1. Thus, Eq. 6.10.3.2.1-1 is evaluated as follows:

For Strength I:

$$17.41 + 19.98 \leq (1.0)(1.0)(50)$$

$$37.39\ \text{ksi} < 50\ \text{ksi} \qquad \text{(satisfied)}$$

For the Special Load Combination (Article 3.4.2.1):

$$19.50 + 20.74 \leq (1.0)(1.0)(50)$$

$$40.24\ \text{ksi} < 50\ \text{ksi} \qquad \text{(satisfied)}$$

Flexural Resistance:

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

As specified in Article 6.10.3.2.1, the nominal flexural resistance of the compression flange, F_{nc}, is to be determined as specified in Article 6.10.8.2. For sections in straight I-girder bridges with compact or noncompact webs, the lateral torsional buckling resistance may be taken as M_{nc} determined as specified in Article A6.3.3 (Appendix A6) divided by the elastic section modulus about the major axis of the section to the compression flange, S_{xc}. As mentioned in Article C6.10.3.2.1, this may be useful for sections in bridges with compact or noncompact webs having larger unbraced lengths, if additional lateral torsional buckling resistance is required beyond that calculated based on the provisions of Article 6.10.8.2. However, for this example, the increased lateral torsional buckling resistance obtained by using the provisions of Article A6.3.3 is not deemed to be necessary. Thus, the provisions of Article 6.10.8.2.3 will be used to compute the lateral torsional buckling resistance for this check.

First, calculate the local buckling resistance of the top (compression) flange. Determine the slenderness ratio of the top flange:

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

$$\lambda_f = \frac{14.0}{2(0.75)} = 9.3$$

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 $\lambda_f > \lambda_{pf}$,

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

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

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

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

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

$$\lambda_{rf} = 0.56\sqrt{\frac{29{,}000}{35.0}} = 16.1$$

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

$$\left(F_{nc}\right)_{FLB} = \left[1-\left(1-\frac{35.0}{(1.0)(50)}\right)\left(\frac{9.3-9.2}{16.1-9.2}\right)\right](1.0)(1.0)(50) = 49.78\ \text{ksi}$$

For Strength I:

$$f_{bu} + \frac{1}{3}f_{\ell} \le \varphi_f\left(F_{nc}\right)_{FLB}$$

$$f_{bu} + \frac{1}{3}f_{\ell} = \left|-17.41\right|\ \text{ksi} + \frac{19.98}{3}\ \text{ksi} = 24.07\ \text{ksi}$$

$$\phi_f\left(F_{nc}\right)_{FLB} = 1.0(49.78) = 49.78\ \text{ksi}$$

$$24.07\ \text{ksi} < 49.78\ \text{ksi} \quad \text{(satisfied)}$$

For the Special Load Combination specified in Article 3.4.2.1:

$$f_{bu} + \frac{1}{3}f_{\ell} \le \varphi_f\left(F_{nc}\right)_{FLB}$$

$$f_{bu} + \frac{1}{3}f_{\ell} = \left|-19.50\right|\ \text{ksi} + \frac{20.74}{3}\ \text{ksi} = 26.41\ \text{ksi}$$

$$\phi_f\left(F_{nc}\right)_{FLB} = 1.0(49.78) = 49.78\ \text{ksi}$$

$$26.41\ \text{ksi} < 49.78\ \text{ksi} \quad \text{(satisfied)}$$

Next, determine the lateral torsional buckling resistance of the top (compression) flange within the unbraced length under consideration. The limiting unbraced length, L_p, was computed earlier to be 82.4 in. or 6.87 ft. The effective radius of gyration for lateral torsional buckling, r_t, for the non-composite section was also computed earlier to be 3.42 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: $F_{yr} = 0.7F_{yc} \leq F_{yw}$

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

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

$$\text{Therefore: } L_r = \frac{\pi(3.42)}{12}\sqrt{\frac{29{,}000}{35.0}} = 25.77 \text{ ft}$$

Since L_p = 6.87 feet < L_b = 20.0 feet < L_r = 25.77 feet,

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

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,

$$F_{nc} = 1.0\left[1-\left(1-\frac{35.0}{1.0(50)}\right)\left(\frac{20.0-6.87}{25.77-6.87}\right)\right](1.0)(1.0)(50) = 39.58 \text{ ksi} < 1.0(1.0)(50) = 50 \text{ ksi}$$

ok

For Strength I:

$$f_{bu} + \frac{1}{3}f_\ell \leq \varphi_f \left(F_{nc}\right)_{LTB}$$

$$f_{bu} + \frac{1}{3}f_\ell = \left|-17.41\right| \text{ ksi} + \frac{19.98}{3} \text{ ksi} = 24.07 \text{ ksi}$$

$$\phi_f \left(F_{nc}\right)_{LTB} = 1.0(39.58) = 39.58 \text{ ksi}$$

$$24.07 \text{ ksi} < 39.58 \text{ ksi} \quad \text{(satisfied)}$$

For the Special Load Combination specified in Article 3.4.2.1:

$$f_{bu} + \frac{1}{3}f_\ell \leq \varphi_f \left(F_{nc}\right)_{LTB}$$

$$f_{bu} + \frac{1}{3} f_{\ell} = \left|-19.50\right| \text{ ksi} + \frac{20.74}{3} \text{ ksi} = 26.41 \text{ ksi}$$

$$\phi_f \left(F_{nc}\right)_{LTB} = 1.0(39.58) = 39.58 \text{ ksi}$$

$$26.41 \text{ ksi} < 39.58 \text{ ksi} \quad \text{(satisfied)}$$

8.3.1.3.2 Tension Flange:

Flange Tip Yielding:

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

For Strength I:

$$12.29 + 7.58 \leq (1.0)(1.0)(50)$$

$$19.87 \text{ ksi} < 50 \text{ ksi} \qquad \text{(satisfied)}$$

For the Special Load Combination (Article 3.4.2.1):

$$13.77 + 7.44 \leq (1.0)(1.0)(50)$$

$$21.21 \text{ ksi} < 50 \text{ ksi} \qquad \text{(satisfied)}$$

8.3.1.4 Shear (Article 6.10.3.3)

As previously stated, since the design does not require any transverse stiffeners, the shear check under the construction loading is automatically satisfied.

8.3.2 Service Limit State (Article 6.10.4)

Service limit state requirements for steel I-girder bridges are specified in Article 6.10.4. The evaluation of the positive bending region based on these requirements follows.

8.3.2.1 Elastic Deformations (Article 6.10.4.1)

Since the bridge is not designed to permit pedestrian traffic, the live load deflection will be limited to L/800. It is shown below that the maximum deflection along the span length using the service loads and a line girder approach is less than the L/800 limit. It is noted, however, that the application of this requirement is optional.

δ = 0.585 in. < L/800 = (90.0 x 12)/800 = 1.35 in.

8.3.2.2 Permanent Deformations (Article 6.10.4.2)

To control permanent deformations at the service limit state, factored top-flange flexural stresses in composite sections under the Service II load combination are limited according to Eq. 6.10.4.2.2-1 as follows:

$$f_f \le 0.95 R_h F_{yf} \qquad \text{Eq. (6.10.4.2.2-1)}$$

It is noted that the moment values in the above equation represent the moments resulting from elastic analysis since it has previously been determined that moment redistribution is not applicable at the service limit state.

The factored Service II stress in the top (compression) flange at Section 1 is computed as follows based on the moment values given in Tables 5 and 6:

$$f_f = \frac{(1.0)(738)(12)}{635.9} + \frac{(1.0)(147+120)(12)}{2{,}700} + \frac{1.3(1{,}661)(12)}{9{,}672} = -17.79 \text{ ksi}$$

$$f_f = \left|-17.79\right| \text{ ksi} < 0.95 R_h F_{yf} = 0.95(1.0)(50) = 47.50 \text{ ksi} \qquad \text{(satisfied)}$$

Because the bottom flange is discretely braced, lateral bending stresses are included in the design requirements for the bottom flange, which are given by Eq. 6.10.4.2.2-2 as follows:

$$f_f + \frac{f_\ell}{2} \le 0.95 R_h F_{yf} \qquad \text{Eq. (6.10.4.2.2-2)}$$

At the service limit state, the lateral force effects due to wind loads and deck overhang loads are not considered. Therefore, for bridges with straight, non-skewed beams such as the case in the present design example, the flange lateral bending stresses are taken equal to zero.

Similarly, the factored Service II stress in the bottom (tension) flange is computed as:

$$f_f = \frac{(1.0)(738)(12)}{900.7} + \frac{(1.0)(147+120)(12)}{1{,}189} + \frac{1.3(1{,}661)(12)}{1{,}285} = 32.69 \text{ ksi}$$

$$f_f = 32.69 \text{ ksi} < 0.95 R_h F_{yf} = 0.95(1.0)(50) = 47.5 \text{ ksi} \qquad \text{(satisfied)}$$

For composite sections in positive flexure, since the web satisfies the requirement of Article 6.10.2.1.1 (i.e. $D/t_w \le 150$) such that longitudinal stiffeners are not required, web bend-buckling under the Service II load combination need not be checked at Section 1. Thus, all service limit state requirements are satisfied.

8.3.3 Fatigue and Fracture Limit State (Article 6.10.5)

8.3.3.1 Load Induced Fatigue (Article 6.6.1.2)

The fatigue calculation procedures in the positive bending region are similar to those previously presented for the negative bending region. In this section, the fatigue requirements are evaluated for the flange welds of a cross-frame connection plate located 40 feet from the abutment.

From Table 6.6.1.2.3-1, it is determined that this detail is classified as a fatigue Detail Category C'.

For this example, a projected $(ADTT)_{SL}$ of 950 trucks per day is assumed. Since this $(ADTT)_{SL}$ is less than the value of the $(ADTT)_{SL}$ Equivalent to Infinite Life for n equal to 1.0 of 975 trucks per day specified in Table 6.6.1.2.3-2 for a Category C′ detail, the nominal fatigue resistance for this particular detail is to be determined for the Fatigue II load combination and finite fatigue life using Eq. 6.6.1.2.5-2. Therefore:

$$(\Delta F)_n = \left(\frac{A}{N}\right)^{\frac{1}{3}} \qquad \text{Eq. (6.6.1.2.5-2)}$$

For a Detail Category C', the detail category constant, A, is 44 x 10^8 ksi^3 (Table 6.6.1.2.5-1).

$$N = (365)(75)n(ADTT)_{SL} \qquad \text{Eq. (6.6.1.2.5-3)}$$

$$N = (365)(75)(1.0)(950) = 26.0 \text{ x } 10^6 \text{ cycles}$$

Therefore:

$$(\Delta F)_n = \left(\frac{44 \text{ x } 10^8}{26.0 \text{ x } 10^6}\right)^{\frac{1}{3}} = 5.5 \text{ ksi}$$

Again, as discussed previously, the concrete deck will be assumed effective in computing all dead load and live load stresses and live load stress ranges applied to the composite section in the subsequent fatigue calculations.

At this location, the unfactored permanent loads produce compression at the top of the girder and tension at the bottom of the girder. 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.

Bottom of Top Flange:

$$f_{DC1} = \frac{(702)(12)(25.04)}{16,401} = -12.86 \text{ ksi}$$

$$f_{DC2} = \frac{(140)(12)(12.70)}{36,315} = -0.59 \text{ ksi}$$

Σ = -12.86 + -0.59 = -13.45 ksi

$$f_{LL+IM} = \frac{1.75\left|-157\right|(12)(4.41)}{49,905} = 0.29 \text{ ksi}$$

$\left|-13.45 \text{ ksi}\right| > 0.29 \text{ ksi}$ $\therefore$ fatigue does not need to be checked

Top of Bottom Flange:

$$\gamma(\Delta f) = (0.80)\left[\frac{(588)(12)(37.59)}{49{,}905} + \frac{\left|-157\right|(12)(37.59)}{49{,}905}\right]$$

$\gamma(\Delta f) = 5.39 \text{ ksi} < (\Delta F)_n = 5.5 \text{ ksi}$ (satisfied)

8.3.3.2 Special Fatigue Requirement for Webs (Article 6.10.5.3)

As discussed previously, the following shear requirement must be satisfied at the fatigue limit state:

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

However, this is an unstiffened web. Therefore, this limit does not control and is not explicitly evaluated.

8.3.4 Strength Limit State (Article 6.10.6)

8.3.4.1 Flexure (Article 6.10.6.2)

For compact sections in positive bending, Equation 6.10.7.1.1-1 must be satisfied at the strength limit state.

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

The flange lateral bending stresses are negligible at the strength limit state for the straight, composite girder considered herein (i.e., bottom-flange lateral bending stresses due to wind load at the strength limit state are not expected to be significant for this shorter-span bridge and are not examined in this example).

The following requirements must be satisfied for a composite section in positive bending to qualify as compact:

F_y = 50 ksi < 70 ksi (satisfied)

$$\frac{D}{t_w} = \frac{42.0}{0.5} = 84.0 < 150 \quad \text{(satisfied)}$$

$$\frac{2D_{cp}}{t_w} = \frac{2(0)}{0.4375} = 0 < 3.76\sqrt{\frac{E}{F_{yc}}} \quad \text{(satisfied)}$$

Therefore, the section is compact, and the nominal flexural resistance is based on Article 6.10.7.1.2. The following requirement must be evaluated.

$D_p \leq 0.1D_t$

The plastic neutral axis was determined previously to be located 7.43 in. from the top of the concrete deck. Therefore, the depth of the composite section in compression at the plastic moment, D_p, is

D_p = 7.43 in.

D_t = total depth of the composite section

D_t = 8.0 + 2.0 + 42.0 + 1.25 = 53.25 in.

D_p = 7.43 > 0.1D_t = 0.1(53.25) = 5.33 (not satisfied)

Therefore, the nominal flexural resistance is determined as:

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

$$M_n = 6{,}723\left(1.07 - 0.7\frac{7.43}{53.25}\right) = 6{,}537 \text{ k-ft}$$

Since the span under consideration and all adjacent interior-pier sections satisfy the requirements of Article B6.2 (as determined previously), and θ_{RL} at all adjacent interior-pier sections determined previously (Section 8.2.1.3) exceeds 0.009 radians (9 mrads), M_n is not limited to 1.3R_hM_y according to Eq. 6.10.7.1.2-3 in this case.

From elastic analysis procedures, the maximum positive moment under the Strength I load combination is 4,192 kip-ft (Table 7), which is at a distance of 36 feet from the left support. The redistribution moment must then be added to this moment to determine the total factored moment. The redistribution moment varies linearly from zero at the end supports to a maximum at the interior pier of 512 kip-ft (Section 8.2.1.2). Thus, the redistribution moment at 36 feet from the abutment is computed as follows (Article B6.4.2.2):

$$M_{rd} = 36/90 * (512) = 0.4(512) = 205 \text{ kip-ft}$$

The total design moment is then the sum of the redistribution moment and the elastic moment.

$$M_u = 4{,}192 + 205 = 4{,}397 \text{ kip-ft}$$

The bending strength of the positive bending region is then shown to be sufficient.

$$M_u \leq \phi_f M_n$$

$$4{,}397 \text{ kip-ft} < (1.0)(6{,}537) = 6{,}537 \text{ k-ft} \qquad \text{(satisfied)}$$

8.3.4.2 Ductility Requirement (6.10.7.3)

Sections in positive bending are also required to satisfy Eq. 6.10.7.3-1, which is a ductility requirement intended to prevent premature crushing of the concrete slab.

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

$$D_p = 7.43 \text{ in.} < 0.42(53.25) = 22.37 \text{ in.} \qquad \text{(satisfied)}$$

8.3.4.3 Shear (6.10.6.3)

The shear requirements at the strength limit state are expressed by:

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

where: V_n = V_{cr}

V_{cr} = shear buckling resistance (kip)

V_{cr} = CV_p (for unstiffened webs) Eq. (6.10.9.2-1)

V_p = plastic shear force (kip)

V_p = $0.58\, F_{yw} D t_w$ Eq. (6.10.9.2-2)

C = ratio of the shear buckling resistance to the shear yield strength determined as specified in Article 6.10.9.3.2, with the shear buckling coefficient, k, taken equal to 5.0

The computation of C is based on the web slenderness. Thus, the web slenderness is first evaluated in terms of the following equation:

$$\frac{D}{t_w} \leq 1.12\sqrt{\frac{Ek}{F_{yw}}}$$

$$\frac{D}{t_w} = \frac{42.0}{0.5} = 84.0 > 1.12\sqrt{\frac{Ek}{F_{yw}}} = 1.12\sqrt{\frac{29,000(5)}{50}} = 60.31 \qquad \text{(not satisfied)}$$

The web slenderness is next evaluated in terms of the following equation:

$$1.12\sqrt{\frac{Ek}{F_{yw}}} < \frac{D}{t_w} = 84.0 \leq 1.40\sqrt{\frac{Ek}{F_{yw}}}$$

$$1.12\sqrt{\frac{Ek}{F_{yw}}} = 60.31 < \frac{D}{t_w} = 84.0 > 1.40\sqrt{\frac{Ek}{F_{yw}}} = 75.4 \qquad \text{(not satisfied)}$$

Lastly, the web slenderness is evaluated as follows:

$$\frac{D}{t_w} = 84.0 > 1.40\sqrt{\frac{Ek}{F_{yw}}} = 75.4 \qquad \text{(satisfied)}$$

Thus, C is calculated according to Eq. 6.10.9.3.2-6.

$$C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2}\left(\frac{Ek}{F_{yw}}\right) = \frac{1.57}{(84.0)^2}(2,900) = 0.645 \qquad \text{Eq. (6.10.9.3.2-6)}$$

Therefore, the nominal shear resistance is equal to:

$$V_{cr} = CV_p = (0.645)(0.58)(50)(42)(0.\ 5) = 392.8 \text{ kips}$$

$$V = 257 \text{ kips} \leq \phi_v V_{cr} = (1.0)(392.8) = 392.8 \text{ kips} \qquad \text{(satisfied)}$$

Thus, the shear requirements at the strength limit state (and consequently all other limit states as previously discussed) are satisfied.

8.4 Cross-frame Design

The cross-frames alone provide restoring forces during construction to enable the girders to deflect equally. Once the system acts compositely, the concrete slab also contributes to providing restoring forces and continuously braces the top flanges at the girder. Although several configurations of cross-frames may be used (refer to the Guidelines for a more complete discussion on cross-frame configurations), a typical K-type cross-frame (as shown in Figure 18) is used for this example. The design of the intermediate and end cross-frames is demonstrated in the sections that follow. In this example, the cross-frame members are designed as a minimum to satisfy slenderness requirements and to transfer wind loads at the strength limit state (Article 6.7.4.1).

Although not currently required by AASHTO, it is recommended that cross-frames for routine I-girder bridges (such as the one in this example) also be designed to satisfy the stability bracing strength and stiffness requirements specified in AISC Specification Appendix 6 (Article 6.3.2a). Consult NSBA's *Steel Bridge Design Handbook: Bracing System Design* [6] and *National Cooperative Highway Research Project Report 962: Proposed Modification to AASHTO Cross-Frame Analysis and Design* [7] for further information on these requirements. 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.

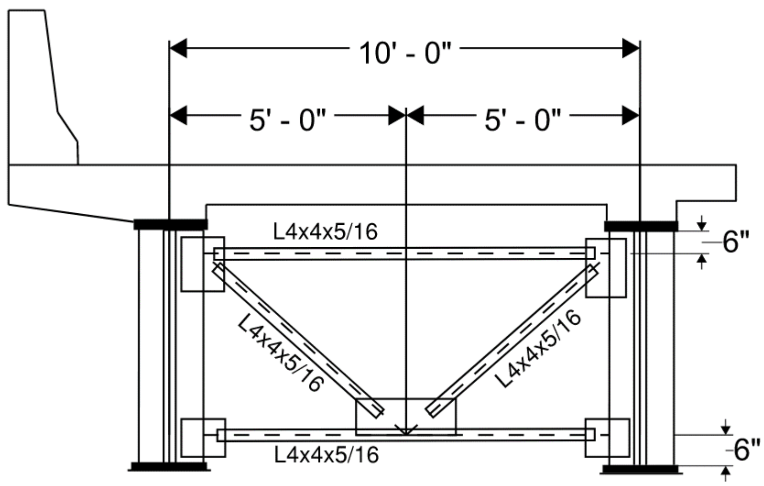

Figure 18 Intermediate Cross-Frame

8.4.1 Intermediate Cross-frame Design

This section describes the design process for an intermediate cross-frame.

For structures with composite concrete decks, the wind load on the upper half of the outside girder, the deck, barriers, and appurtenances is assumed transmitted directly to the concrete deck acting as lateral diaphragm carrying the load to the supports. The remaining half of the load is assumed applied laterally to the bottom flange. The bottom flange is assumed to carry the wind load to adjacent cross-frames by flexural action. The frame action of the cross-frames in turn transmits the wind-load forces into the deck, which then transmits the forces to the supports through diaphragm action. The factored wind force per unit length on the bottom flange is given by Eq. C4.6.2.7.1-1 as follows:

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

where: η_i = load modifier specified in Article 1.3.2.1

γ = load factor for WS specified in Table 3.4.1-1 (1.0 for Strength III and Strength V)

P_D = design horizontal wind pressure specified in Article 3.8.1 (ksf) = P_Z (Section 4.3)

d = depth of the member (ft)

W will be conservatively computed using the deepest steel section. W need not be applied to the top flange at the strength limit state.

For Strength III: $$W = \frac{(1.0)(1.0)(0.031)\left[(1.25+42.0+1.125)/12\right]}{2} = 0.057\text{ klf}$$

For Strength V: $$W = \frac{(1.0)(1.0)(0.021)\left[(1.25+42.0+1.125)/12\right]}{2} = 0.039\text{ klf}$$

8.4.1.1 Bottom Strut

The bottom strut is in compression under the wind loading; therefore, the limiting slenderness ratio for bracing members in compression must be satisfied as specified in Article 6.9.3.

A L4x4x5/16 single angle is selected for the bottom strut. The angle satisfies the minimum material thickness requirements specified in Article 6.7.3. Section properties are calculated below and depicted in Figure 19. In these computations it is assumed that the connection plate is ½-inch thick.

$A = 2.40\text{ in.}^2$

$r_z = 0.781\text{ in.}$

$I_z = Ar_z^2 = (2.40)(0.781)^2 = 1.46\text{ in.}^4$

$I_w = I_x + I_y - I_z = 3.67 + 3.67 - 1.46 = 5.88\text{ in.}^4$

$r_w = \sqrt{\frac{I_w}{A}} = \sqrt{\frac{5.88}{2.40}} = 1.57\text{ in.}$

$r_x = r_y = 1.24\text{ in.}$

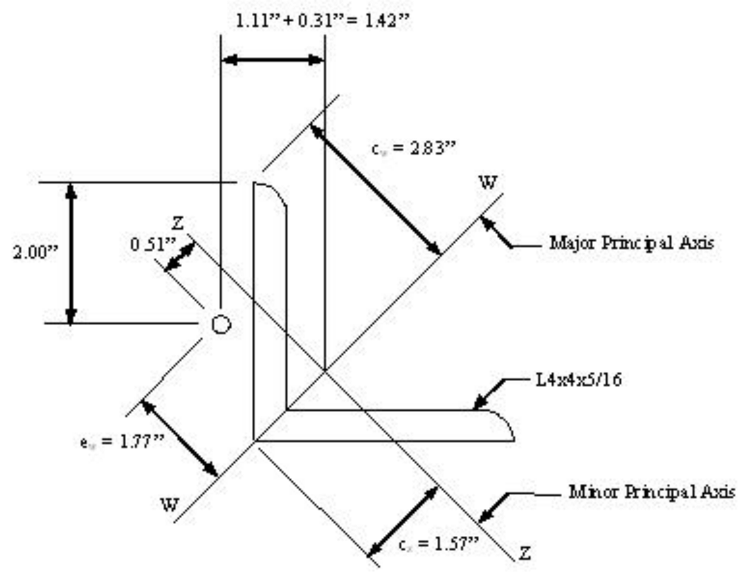

Figure 19 Single Angle for Intermediate Cross-Frame

The horizontal wind force applied to the brace point can be calculated as follows:

$$P_W = WL_b \qquad \text{Eq. (C4.6.2.7.1-4)}$$

where L_b is the maximum cross-frame spacing. The Strength III load combination controls. Therefore:

$$P_W = (0.057)(20.0) = 1.14 \text{ kips}$$

The bottom struts in the exterior bays of the system must carry the entire wind force P_W; therefore, all of the bottom struts will be conservatively designed to satisfy the requirements of the exterior bay struts.

Connected through one leg only, the strut is eccentrically loaded. Thus, the member experiences both flexure and axial compression.

To determine if the effects of local buckling of the outstanding angle legs on the nominal compressive resistance of the member need to be considered, check the width-to-thickness ratio provision of Article 6.9.4.2.1 for the cross-frame bottom strut member:

$$\frac{b}{t} \leq \lambda_r \qquad \text{Eq. (6.9.4.2.1-1)}$$

where:

λ_r = width-to-thickness ratio limit specified in Table 6.9.4.2.1-1
b = the full width of the outstanding leg for a single angle (in.)
t = element thickness (in.)

$$\frac{b}{t} = \frac{4}{0.5625} = 7.1 < 0.45\sqrt{\frac{29{,}000}{50}} = 10.8 \qquad \text{(Angles legs are nonslender)}$$

Check the limiting slenderness ratio of Article 6.9.3. As a secondary compression member (see Table 6.6.2.1-1), the angle must satisfy the following:

$$\frac{K\ell}{r} \le 140$$

where: K = effective length factor specified in Article 4.6.2.5 taken as 1.0 for single angles regardless of end connection (in.)

ℓ = unbraced length (in.)

r = minimum radius of gyration (in.)

For checking the slenderness of the member about the minor principal axis (i.e., the z-axis), ℓ will be assumed equal to 4'-9″. For checking the slenderness of the member about the vertical geometric axis (i.e., the y-axis), ℓ will be assumed equal to 9'-6″.

$$\frac{K\ell}{r_z} = \frac{1.0(4.75)(12)}{0.781} = 73 < 140 \qquad \text{(satisfied)}$$

$$\frac{K\ell}{r_y} = \frac{1.0(9.5)(12)}{1.24} = 92 < 140 \qquad \text{(satisfied)}$$

8.4.1.1.1 Combined Axial Compression and Bending

Having satisfied the basic slenderness provisions, the angle is then checked for combined axial compression and bending at the strength limit state in accordance with Article 6.9.4.4.

Single angles are commonly used as members in cross-frames of steel girder bridges. Since the angle is typically connected through one leg only, the member is subjected to combined axial load and flexure. In other words, the eccentricity of the applied axial load induces moments about both principal axes of the angle. As a result, it is difficult to predict the nominal compressive resistance of these members. The provisions of Article 6.9.4.4 provide a simplified approach by permitting the effect of the eccentricities to be neglected when the single angles are evaluated as axially loaded compression members for flexural buckling only using an appropriate specified effective slenderness ratio, $(K\ell/r)_{eff}$, in place of $(K\ell/r_s)$ in Eq. 6.9.4.1.2-1. By following this approach, the single angles may be designed as axially loaded compression members for flexural buckling only according to the provisions of Articles 6.9.2.1, 6.9.4.1.1, and 6.9.4.1.2. It should be noted that according to Article 6.9.4.4, the actual maximum slenderness ratio of the angle, not the effective slenderness ratio, is not to exceed the limiting slenderness ratio specified in Article 6.9.3, as checked above. Also, per Article 6.9.4.4, single angles designed using $(K\ell/r)_{eff}$ need not be checked for flexural-torsional buckling.

Compute the effective slenderness ratio per Article 6.9.4.4 based on the criteria for equal-leg angles. First, check the ℓ/r_x limit of 80. ℓ in the effective slenderness ratio equations is taken as the distance between the work points of the joints measured along the length of the angle:

$$\frac{\ell}{r_x} = \frac{(9.5)(12)}{1.24} = 91.9 > 80$$

where: r_x = radius of gyration about the geometric axis of the angle parallel to the connected leg (Although not relevant for equal-leg angles, the term r_x should be taken as the smaller value of the radius of gyration about the angle geometric axes, which is r_y when unequal-leg angles are used and are connected through the longer leg.)

Therefore, compute the effective slenderness ratio as follows:

$$\left(\frac{K\ell}{r}\right)_{eff} = 32 + 1.25\frac{\ell}{r_x} \qquad \text{Eq. (6.9.4.4-2)}$$

$$\left(\frac{K\ell}{r}\right)_{eff} = 32 + 1.25\frac{(9.5)(12)}{1.24} = 147$$

In accordance with the provisions for single-angle members in Article 6.9.4.4 and using the effective slenderness ratio, $(K\ell/r)_{eff}$, the factored compressive resistance of the angle is taken as:

$$P_r = \phi_c P_n \qquad \text{Eq. (6.9.2.1-1)}$$

where: P_n = nominal compressive resistance determined using the provisions of Article 6.9.4.1.1

ϕ_c = resistance factor for axial compression = 0.95 (Article 6.5.4.2)

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, which is the applicable buckling mode for single angles. P_o is the nominal yield resistance equal to F_yA_g.

$$P_e = \frac{\pi^2 E}{\left(\frac{K\ell}{r_s}\right)^2} A_g \qquad \text{Eq. (6.9.4.1.2-1)}$$

where $(K\ell/r)_{eff}$ is used in place of $(K\ell/r_s)$ in the denominator.

$$P_e = \frac{\pi^2 E}{\left(\frac{K\ell}{r}\right)_{eff}^2} A_g = \frac{\pi^2(29000)}{(147)^2}(2.40) = 31.8 \text{ kips}$$

$P_o = F_y A_g = (50)(2.40) = 120$ kips

Since

$$\frac{P_o}{P_e} = \frac{120}{31.8} = 3.77 > 2.25,$$

the nominal axial resistance in compression for a member composed only of nonslender longitudinally unstiffened elements satisfying the width-to-thickness ratio limits specified in Article 6.9.4.2.1 (checked above) is computed as:

$$P_n = 0.877 P_e \qquad \text{Eq. (6.9.4.1.1-2)}$$

$P_n = 0.877(31.8) = 27.9$ kips

Compute the factored compressive resistance of the angle as follows:

$P_r = \phi_c P_n = 0.95(27.9) = 26.5$ kips

$P_u = |-1.14 \text{ kips}| < P_r = 26.5$ kips (satisfied)

8.4.1.2 Diagonals

The diagonals carry a compressive force that is the result of wind loads and reactions from the loads carried in the top strut. It is assumed that each bay carries a portion of P_w, and the two diagonals carry equal loads. From statics, the following equation can be derived to determine the factored axial wind-load force in the diagonals (for Strength III):

$$\left(P_w\right)_{diag.} = \sqrt{a^2 + b^2}\left(\frac{P_w}{2na}\right)$$

where:

a = horizontal distance between working points for the top strut

b = vertical distance between working points for the diagonals

P_w = total applied wind-load force

n = number of bays

$$\left(P_w\right)_{diag.} = \sqrt{\left(\frac{(9.5)(12)}{2}\right)^2 + (30)^2}\left(\frac{1.14}{2(3)\left(9.5(12)/2\right)}\right) = 0.21\,\text{kips}$$

The unbraced length of the diagonal in compression, taken as the distance between the working points, is calculated below:

$$l = \sqrt{\left(\frac{9.5(12)}{2}\right)^2 + (30.0)^2} = 64.41\,\text{in.}$$

A similar analysis was conducted for the diagonals as was conducted for the bottom strut, and the L4x4x5/16 member was determined to be adequate for the design wind loading.

8.4.2 End Cross-frame Design

The lateral wind forces are transmitted from the deck to the substructure by the end cross-frames. The following section describes the design of end cross-frames (see Figure 20).

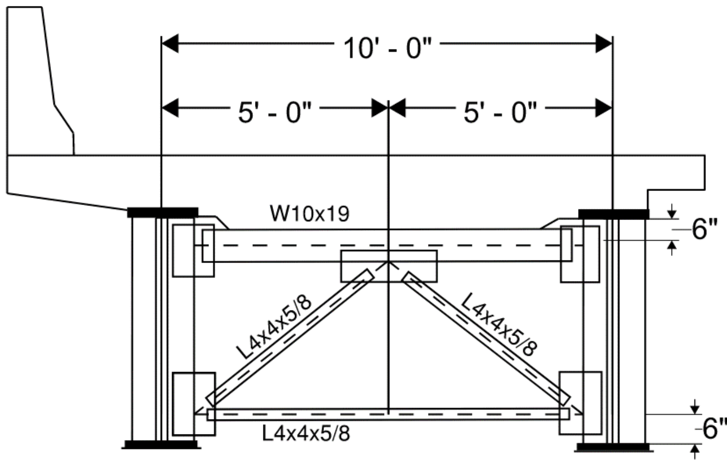

Figure 20 End Cross-Frame

8.4.2.1 Top Strut

The top strut of the end cross-frames carries the compressive forces that are a result of the wind load on the structure and vehicles, dead load of the slab, including the haunch, and the wheel loads, including the dynamic load allowance. The total design horizontal wind pressure P_D (= P_Z), calculated previously (Section 4.3), is 0.031 ksf for Strength III and 0.021 ksf for Strength V. The total height of the structure is as follows:

Barrier	=	42.00 in.
Deck	=	8.50 in.
Haunch	=	2.00 in.
Girder below haunch	=	43.25 in.
	=	93.75 in. = 7.98 ft

The wind load per unit length on the structure is computed as follows:

Strength III: WS = (7.98)(0.031) = 0.25 kips/ft

Strength V: WS = (7.98)(0.021) = 0.17 kips/ft

From Article 3.8.1.3, the wind load per unit length acting normal to the vehicles at a distance of 6.0 feet above the roadway is:

WL = 0.10 kips/ft

The wind load on the end cross-frames is assumed to be half of the total wind load and is computed below.

Strength III: $P_{WS} = 0.25\left(\frac{90.0}{2}\right) = 11.25 \text{ kips}$

$$\text{Strength V:} \quad P_{WS} = 0.17\left(\frac{90.0}{2}\right) = 7.65 \text{ kips}$$

$$P_{WL} = 0.10\left(\frac{90.0}{2}\right) = 4.50 \text{ kips}$$

Each bay is assumed to carry an equal portion of the wind load; therefore, the axial force in the top strut is calculated as follows:

Strength III: $(P_{WS})_{top\ strut} = 11.25/3 = 3.75$ kips

Strength V: $(P_{WS})_{top\ strut} = 7.65/3 = 2.55$ kips

$(P_{WL})_{top\ strut} = 4.50/3 = 1.50$ kips

Consider the section through the top strut shown in Figure 21:

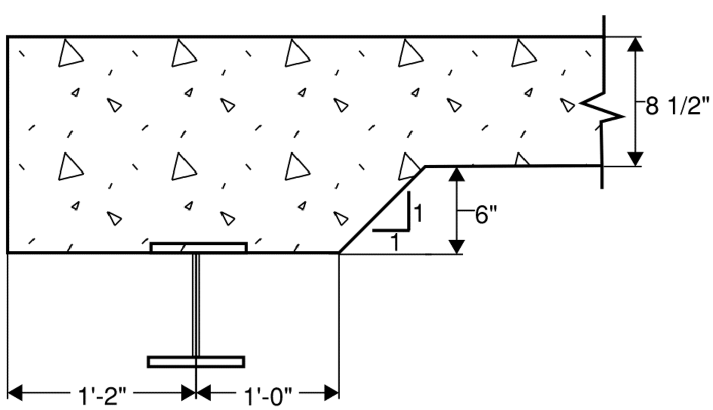

Figure 21 Section Cut at Top Strut

The dead loads acting on the top strut are computed as follows:

Slab	=	8.50(14.00 + 12.00 + 6.00)(1/144)(0.150) = 0.28
Concrete Haunch	=	6.00(14.00 + 12.00 + 6.00/2)(1/144)(0.150) = 0.18
Steel Beam	=	<u>0.019</u>
	=	0.48 kip/ft

As specified in Article 3.6.1.2.4, the design lane load is a 0.64 kips/ft load distributed over a 10.0-foot width.

$$w_{LL} = \frac{0.64}{10.0(12)}(14.0 + 12.0 + 6.00) = 0.17 \text{ kips/ft}$$

Although including the design lane load in the design of the end cross-frame is conservative and perhaps debatable, it is considered herein for completeness.

The design truck wheel load plus the dynamic load allowance is discussed in Article 3.6.1.2.2 and is as follows.

$$P_{LL} = \frac{32.0}{2}(1.33) = 21.28 \text{ kips}$$

Figure 22 illustrates the position of the above computed live loads that produce the maximum moment and shear in the strut. Under vertical loads, the point of intersection of the top strut and the diagonals is assumed to act as a vertical support. Thus, from a separate analysis assuming a prismatic two-span continuous beam loaded by the computed dead and live loads (conservatively assuming 5-ft-long spans), the maximum moments and reactions in the top strut due to the unfactored dead and live loads (at the intersection with the diagonals) are computed as:

M_{DC} = 1.50 kip-ft

M_{LL+IM} = 17.88 kip-ft

R_{DC} = 3.00 kips

R_{LL+IM} = 24.17 kips

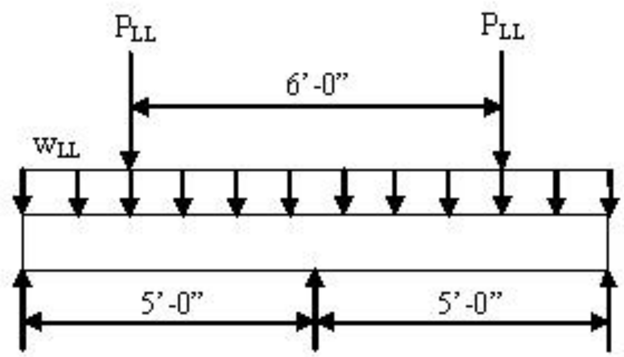

Figure 22 Live Load on Top Strut

Since live load is the primary loading, a beam will be selected and checked for flexure only under the Strength I load combination, and then the beam will be checked for combined axial compression and flexure under the Strength III and Strength V load combinations (which include the wind loads).

8.4.2.1.1 Strength I:

M_u = 1.00[1.25(1.50) + 1.75(17.88)] =33.17 k-ft

$$V_u = 1.0\left[1.25\left(\frac{3.0}{2}\right) + 1.75\left(\frac{24.17}{2}\right)\right] = 23.02 \text{ kips}$$

A W10 x 19 is selected as a trial member. To avoid having to cope the top flange at the end connections, a channel could be used in lieu of a wide-flange shape for the top strut if the top-flange width is sufficient to accommodate any necessary expansion joint details. The top flange of the channel must be encased in the concrete haunch, or shear connectors must be provided (as is common policy for many Owner-agencies) in lieu of encasing the top flange, to help control twist of the member since the loads are not applied through the shear center of the channel.

To determine the flexural resistance of the W10x19 section, the applicability of Appendix A6 is first evaluated.

$F_y = 50$ ksi < 70 ksi

$$\frac{2D_c}{t_W} \le \lambda_{rw} \qquad \text{Eq. (A6.1-1)}$$

where:

$$4.6\sqrt{\frac{E}{F_{yc}}} \le \lambda_{rw} = \left(3.1 + \frac{5.0}{a_{wc}}\right)\sqrt{\frac{E}{F_{yc}}} \le 5.7\sqrt{\frac{E}{F_{yc}}} \qquad \text{Eq. (A6.1-3)}$$

$$a_{wc} = \frac{2D_c t_w}{b_{fc} t_{fc}} \qquad \text{Eq. (A6.1-4)}$$

$$\frac{2(9.41/2)}{0.25} = 37.64$$

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

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

$$a_{wc} = \frac{2(37.64)(0.25)}{4.02(0.395)} = 11.85$$

$$111 > \lambda_{rw} = \left(3.1 + \frac{5.0}{11.85}\right)\sqrt{\frac{29,000}{50}} = 84.8 < 137$$

$$\therefore \lambda_{rw} = 111 > \frac{2D_c}{t_w} = 37.64 \qquad \text{(satisfied)}$$

Since the section is doubly symmetric, the ratio of I_{yc}/I_{yt} = 1.0 and Eq. A6.1-2 is satisfied.

Therefore, Appendix A6 is applicable. The web slenderness is then evaluated based on Eq. A6.2.1-1.

$$\frac{2D_{cp}}{t_w} \le \lambda_{pw(D_{CP})} \qquad \text{Eq. (A6.2.1-1)}$$

For the W10 x 19, the plastic modulus, Z_x, is 21.6 in.^{3} and the elastic section modulus, S_x, is 18.8 in.^{3}.

$$\lambda_{pw(D_{cp})} = \frac{\sqrt{\frac{E}{F_{yc}}}}{\left(0.54\frac{M_p}{R_h M_y} - 0.09\right)^2} \le \lambda_{rw}\left(\frac{D_{cp}}{D_c}\right) \qquad \text{Eq. (A6.2.1-2)}$$

$$\lambda_{pw(D_{cp})} = \frac{\sqrt{\frac{29000}{50}}}{\left(0.54\frac{(21.6)(50)}{(1.0)(18.8)(50)} - 0.09\right)^2} = 88.92 \le 111\left(\frac{9.41/2}{9.41/2}\right) = 111$$

$$\frac{2D_{cp}}{t_W} = \frac{2(9.41/2)}{0.25} = 37.64 < 88.92 \qquad \text{(satisfied)}$$

Therefore, the web is compact and the web plastification factors are thus computed as follows:

$$R_{pc} = \frac{M_p}{M_{yc}} = \frac{(21.6)(50)}{(18.8)(50)} = 1.149 \qquad \text{Eq. (A6.2.1-5)}$$

$$R_{pt} = \frac{M_p}{M_{yt}} = \frac{(21.6)(50)}{(18.8)(50)} = 1.149 \qquad \text{Eq. (A6.2.1-6)}$$

Calculate the nominal flexural resistance of the section based on local buckling. The following calculations show that the compression flange is compact.

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_{yc}}} = 9.15 \qquad \text{Eq. (A6.3.2-4)}$$

$$\lambda_f = \frac{b_{fc}}{2t_{fc}} = \frac{4.02}{2(0.395)} = 5.09 < 9.15 \qquad \text{(satisfied)}$$

Therefore, the nominal flexural resistance of the section based on local buckling is equal to the product of the web plastification factor and the yield moment, as specified in Eq. A6.3.2-1.

$$M_{nc(FLB)} = R_{pc}M_{yc} = 1.149(50)(18.8)/12 = 90.0\,\text{k-ft}$$

The point of intersection of the top strut with the diagonals is not considered to act as a brace point for lateral-torsional buckling. Therefore, the strut is assumed to act as a simple span for lateral-torsional buckling. However, the top (compression) flange of the beam is encased in concrete and is considered to be continuously braced (as mentioned previously, another option in lieu of encasing the top chord in the concrete haunch would be to provide shear connectors on the top flange of the strut, in which case the flange would still be considered to be continuously braced – see Article C6.10.1.6). Thus, the nominal flexural resistance of the section based on lateral-torsional buckling need not be checked.

The factored flexural resistance based on the compression flange is computed as:

$$M_{rc} = \phi_f M_{nc} = (1.0)(90.0) = 90.0\text{ k-ft} > M_u = 33.17\text{ k-ft} \qquad \text{(satisfied)}$$

The nominal flexural resistance based on tension flange yielding is computed as:

$$M_{nt} = R_{pt}M_{yt} = 1.149(50)(18.8)/12 = 90.0\,\text{k-ft}$$

The factored flexural resistance based on the tension flange yielding is computed as:

$$M_{rt} = \phi_f M_{nt} = (1.0)(90.0) = 90.0\text{ k-ft} > M_u = 33.17\text{ k-ft} \qquad \text{(satisfied)}$$

In addition to the flexural resistance, the shear resistance must be evaluated to verify the member is acceptable. The nominal shear resistance of the member is computed below:

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

where: $V_p = 0.58F_{yw}Dt_w = 0.58(50)(9.41)(0.25) = 68.22\,\text{kips}$ Eq. (6.10.9.2-2)

The formula used to compute C varies depending on the web slenderness as shown below.

$$1.12\sqrt{\frac{Ek}{F_{yw}}} = 1.12\sqrt{\frac{(29{,}000)(5.0)}{50}} = 60.31$$

$$\frac{D}{t_w} = \frac{9.41}{0.25} = 37.64 < 60.31$$

Therefore, C = 1.0

$$\phi_v V_n = (1.0)(68.22) = 68.22\,\text{kips} > V_u = 23.02\,\text{kips} \qquad \text{(satisfied)}$$

8.4.2.1.2 Strength III:

P_u = 1.00[1.25(0.00) + 1.00(3.75)] = 3.75 kips

M_{ux} = 1.00[1.25(1.50) + 1.00(0.00)] = 1.88 k-ft

Article 6.9.2.1 specifies the factored compressive resistance as follows:

$$P_r = \phi_c P_n \qquad \text{Eq. (6.9.2.1-1)}$$

where: $\phi_c = 0.95$

To determine if the effects of local buckling of the web and the projecting width of the flanges on the nominal compressive resistance of the member need to be considered, check the width-to-thickness ratio provision of Article 6.9.4.2.1 for the cross-frame top strut member:

$$\frac{b}{t} \leq \lambda_r \qquad \text{Eq. (6.9.4.2.1-1)}$$

where:

λ_r = width-to-thickness ratio limit specified in Table 6.9.4.2.1-1
b = half-flange width for the flanges and clear distance between the flanges minus the fillets at each flange for the web (in.)
t = element thickness (in.)

$$\frac{b}{t} = \frac{4.02/2}{0.395} = 5.09 < 0.56\sqrt{\frac{29000}{50}} = 13.5 \qquad \text{(Flanges are nonslender)}$$

$$\frac{b}{t} = \frac{(10.2 - 2(0.695))}{0.250} = 35.24 < 1.49\sqrt{\frac{29000}{50}} = 35.88 \qquad \text{(Web is nonslender)}$$

The elastic critical buckling resistance, P_e, based on flexural buckling controls in this case (Table 6.9.4.1.1-1). The effective length factor, K, is taken as 0.750 (Article 4.6.2.5):

$$P_e = \frac{\pi^2 E}{\left(\frac{K\ell}{r_s}\right)^2} A_g = \frac{\pi^2 (29000)}{\left(\frac{0.750(9.5)(12)}{0.874}\right)^2}(5.62) = 168 \text{ kips} \qquad \text{Eq. (6.9.4.1.2-1)}$$

$P_o = F_y A_g$ = (50)(5.62) = 281 kips

Since

$$\frac{P_o}{P_e} = \frac{281}{168} = 1.67 < 2.25,$$

the nominal compressive resistance for a member composed only of nonslender longitudinally unstiffened elements satisfying the width-to-thickness ratio limits specified in Article 6.9.4.2.1 (checked above) is computed as:

$$P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] P_o = \left[0.658^{\frac{281}{168}}\right](281) = 139.5 \text{ kips} \qquad \text{Eq. (6.9.4.1.1-1)}$$

Compute the factored compressive resistance of the top strut as follows:

$$P_r = \phi_c P_n = 0.95(139.5) = 132.5 \text{ kips}$$

The factored flexural resistance, M_{rx}, was computed previously to be:

$$M_{rx} = 90.0 \text{ k-ft}$$

Since,

$$\frac{P_u}{P_r} = \frac{3.75}{132.5} = 0.03 < 0.2$$

and the cross-section elements of the member are compact for flexure according to the provisions of Articles A6.2.1 and A6.3.2 (as checked previously), the following interaction equation applies (Article 6.9.2.2.1). Magnification of M_{ux} to account for the magnification of the moment caused by the factored axial load as the member deflects is neglected since the axial force is relatively small in this case.

$$\frac{P_u}{2P_r} + \frac{M_{ux}}{M_{rx}} \leq 1.0 \qquad \text{Eq. (6.9.2.2.1-1)}$$

$$\frac{3.75}{2(132.5)} + \frac{1.88}{90.0} = 0.04 < 1.0 \qquad \text{(satisfied)}$$

Since the influence of the axial load is typically quite small, consideration could be given to neglecting consideration of the axial load in the design of the top strut, especially if the top strut is anchored to the concrete deck with shear connectors.

8.4.2.1.3 Strength V:

Similarly, the applied axial force and moment for the Strength V load combination are computed below.

$$P_u = 1.00[1.25(0.00) + 1.35(0.00) + 1.00(2.55) + 1.00(1.50)] = 4.05 \text{ kips}$$

$$M_{ux} = 1.00[1.25(1.50) + 1.35(17.88) + 1.00(0.00) + 1.00(0.00)] = 26.01 \text{ k-ft.}$$

The axial load and moment interaction Eq. 6.9.2.2.1-1 is also shown to be satisfied for this load combination below.

$$\frac{4.05}{2(132.5)} + \frac{26.01}{90.0} = 0.30 < 1.0 \qquad \text{(satisfied)}$$

8.4.2.2 Diagonals

The diagonals carry a compressive force that is the result of wind loads and reactions from the loads carried in the top strut. The geometry of the end cross-frames was previously illustrated in Figure 20. As previously discussed, the design of the cross-frame is based on the assumption that each bay carries an equal portion of the total wind forces. The axial force is computed below using the same process used earlier in this cross-frame design example.

For Strength III: $P_{WS} = 11.25 \text{ kips}$

For Strength V: $P_{WS} = 7.65 \text{ kips}$

$P_{WL} = 4.50 \text{ kips}$

$$\left(P_W\right)_{diag.} = \sqrt{a^2 + b^2}\left(\frac{P_W}{2na}\right)$$

For Strength III: $\left(P_{WS}\right)_{diag.} = \sqrt{\left(\frac{9.5(12)}{2}\right)^2 + 30^2}\left(\frac{11.25}{2(3)\left(9.5(12)/2\right)}\right) = 2.12 \text{ kips}$

For Strength V: $\left(P_{WS}\right)_{diag.} = \sqrt{\left(\frac{9.5(12)}{2}\right)^2 + 30^2}\left(\frac{7.65}{2(3)\left(9.5(12)/2\right)}\right) = 1.44 \text{ kips}$

$$\left(P_{WL}\right)_{diag.} = \sqrt{\left(\frac{9.5(12)}{2}\right)^2 + 30^2}\left(\frac{4.50}{2(3)\left(9.5(12)/2\right)}\right) = 0.85 \text{ kips}$$

The axial force in the diagonal as a result of the dead-load reaction, R_{DC}, on the top strut is computed below.

$$\left(P_{DC}\right)_{diag.} = \sqrt{\left(\frac{9.5(12)}{2}\right)^2 + 30^2}\left(\frac{3.0}{2(30)}\right) = 3.22 \text{ kips}$$

The axial force in the diagonal as a result of the live-load reaction, R_{LL+IM}, on the top strut is computed as follows:

$$\left(P_{LL+IM}\right)_{diag.} = \sqrt{\left(\frac{9.5(12)}{2}\right)^2 + 30^2}\left(\frac{24.17}{2(30)}\right) = 25.95\,\text{kips}$$

The following calculations determine the controlling load combination.

8.4.2.2.1 Strength I:

P_u = 1.00[1.25(3.22) + 1.75(25.95)] = 49.4 kips (governs)

8.4.2.2.2 Strength III:

P_u = 1.00[1.25(3.22) + 1.00(2.12)] = 6.15 kips

8.4.2.2.3 Strength V:

P_u = 1.00[1.25(3.22) + 1.35(25.95) + 1.00(1.44) + 1.00(0.85)] = 41.3 kips

The initial member selection will be based on the compressive strength slenderness requirements of the member and minimum material thickness requirements. The distance between the working points will be taken as the unbraced length, ℓ.

$$\frac{K\ell}{r} < 140$$

where:
- K = effective length factor specified in Article 4.6.2.5 as 1.0 for single angles regardless of end connection (in.)
- ℓ = unbraced length (in.)
- r = minimum radius of gyration (in.)

$$l = \sqrt{\left(\frac{9.5(12)}{2}\right)^2 + 30^2} = 64.41\text{in.}$$

$$r_{min} = \frac{1.0(64.41)}{140} = 0.460\text{ in.}$$

Thus, an L4x4x5/8 is selected as the trial member, assuming a ½-inch connection plate. Similar to the bottom strut of the intermediate cross-frames, the member must be evaluated for the combined influence of flexure and axial compression as detailed below. The necessary cross-sectional properties for the L4x4x5/8 are listed below:

$r_x = r_y$ = 1.20 in.

r_z = 0.774 in.

A_s = 4.61 in.^{2}

To determine if the effects of local buckling of the outstanding angle legs on the nominal compressive resistance of the member need to be considered, check the width-to-thickness ratio provision of Article 6.9.4.2.1 for the cross-frame diagonals:

$$\frac{b}{t} \le \lambda_r \qquad \text{Eq. (6.9.4.2.1-1)}$$

where:

λ_r = width-to-thickness ratio limit specified in Table 6.9.4.2.1-1
b = the full width of the outstanding leg for a single angle (in.)
t = element thickness (in.)

$$\frac{b}{t} = \frac{4}{0.625} = 6.4 < 0.45\sqrt{\frac{29000}{50}} = 10.8 \qquad \text{(Angle legs are nonslender)}$$

8.4.2.2.4 Combined Axial Compression and Flexure

Compute the effective slenderness ratio per Article 6.9.4.4 based on the criteria for equal-leg angles. First, check the ℓ/r_x limit of 80:

$$\frac{\ell}{r_x} = \frac{(64.41)}{1.20} = 53.7 < 80$$

where: r_x = radius of gyration about the geometric axis of the angle parallel to the connected leg (Although not relevant for equal-leg angles, the term r_x should be taken as the smaller value of the radius of gyration about the angle geometric axes, which is r_y when unequal-leg angles are used and are connected through the longer leg.)

Therefore, compute the effective slenderness ratio as follows:

$$\left(\frac{K\ell}{r}\right)_{eff} = 72 + 0.75\frac{\ell}{r_x} \qquad \text{Eq. (6.9.4.4-1)}$$

$$\left(\frac{K\ell}{r}\right)_{eff} = 72 + 0.75\frac{(64.41)}{1.20} = 112.3$$

In accordance with the provisions for single-angle members in Article 6.9.4.4 and using the effective slenderness ratio, $(k\ell/r)_{eff}$, the factored compressive resistance of the angle is taken as:

$$P_r = \phi_c P_n \qquad \text{Eq. (6.9.2.1-1)}$$

where: P_n = nominal compressive resistance determined using the provisions of Article 6.9.4.1.1

ϕ_c = resistance factor for axial compression = 0.95 (Article 6.5.4.2)

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, which is the applicable buckling mode for single angles. P_o is the nominal yield resistance equal to F_yA_g.

$$P_e = \frac{\pi^2 E}{\left(\frac{K\ell}{r_s}\right)^2} A_g \qquad \text{Eq. (6.9.4.1.2-1)}$$

where $(K\ell/r)_{eff}$ is used in place of $(K\ell/r_s)$ in the denominator.

$$P_e = \frac{\pi^2 E}{\left(\frac{K\ell}{r}\right)^2_{eff}} A_g = \frac{\pi^2 (29000)}{(112.3)^2}(4.61) = 104.6 \text{ kips}$$

$P_o = F_yA_g = (50)(4.61) = 230.5$ kips

Since

$$\frac{P_o}{P_e} = \frac{230.5}{104.6} = 2.20 < 2.25,$$

the nominal axial resistance in compression for a member composed only of nonslender longitudinally unstiffened elements satisfying the width-to-thickness ratio limits specified in Article 6.9.4.2.1 (checked above) is computed as:

$$P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] P_o \qquad \text{Eq. (6.9.4.1.1-1)}$$

$$P_n = \left[0.658^{\left(\frac{230.5}{104.6}\right)}\right](230.5) = 91.6 \text{ kips}$$

Compute the factored compressive resistance of the angle as follows:

$P_r = \phi_c P_n = 0.95(91.6) = 87.0$ kips

$P_u = |-49.4 \text{ kips}| < P_r = 87.0$ kips (satisfied)

8.5 Stiffener Design

8.5.1 Bearing Stiffener Design

Bearing stiffeners must be provided on the webs of built-up sections at all bearing locations (Article 6.10.11.2.1). In addition, as specified in Article B6.2.6, bearing stiffeners must also be provided at all interior-pier sections from which moments are redistributed. The bearing stiffeners are typically plates welded to both sides of the web that extend the full depth of the web, and as close as practical to the outer edges of the flanges. Each stiffener should be finished to bear against the flange through which it receives its load. Bearing stiffeners also serving as cross-frame connection plates are to be attached to both flanges of the cross-section, typically by fillet welds. The use of full penetration groove welds to attach each stiffener to the flange though which it receives its load is permitted but is not recommended in order to significantly reduce the welding deformation of the flange (refer to the Guidelines for further information on the detailing of bearing stiffeners). This example illustrates the design of the bearing stiffeners at Abutment 1.

8.5.1.1 Minimum Thickness (Article 6.10.11.2.2)

The thickness, t_p, of each projecting stiffener element must satisfy:

$$t_p \geq \frac{b_t}{0.48\sqrt{\frac{E}{F_{ys}}}} \qquad \text{Eq. (6.10.11.2.2-1)}$$

It will be assumed that 6-inch-wide plates are welded to each side of the web. Therefore:

$$\left(t_p\right)_{min.} = \frac{b_t}{0.48\sqrt{\frac{E}{F_{ys}}}} = \frac{6.0}{0.48\sqrt{\frac{29,000}{50}}} = 0.52\,\text{in.}$$

Thus, 6 inch by 5/8 inch plates will be used to evaluate the bearing stiffener requirements.

8.5.1.2 Bearing Resistance (Article 6.10.11.2.3)

The factored resistance for the 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 bearing stiffeners

= $1.4A_{pn}F_{ys}$ Eq. (6.10.11.2.3-2)

A_{pn} = area of the projecting elements for the stiffener outside of the web-to-flange fillet welds but not beyond the edge of the flange

In this design example, it is assumed the clip provided at the base of the stiffener to clear the web-to-flange weld is 1.5 inches in length.

$$A_{pn} = 2(6.0 - 1.5)(0.625) = 5.63 \text{ in.}^2$$

$$(R_{sb})_n = 1.4(5.63)(50) = 394 \text{ kips}$$

From Table 11, the factored bearing reaction at the abutment, R_u, is equal to 257 kips.

$$(R_{sb})_r = (1.00)(394) = 394 \text{ kips} > R_u = 257 \text{ kips} \qquad \text{(satisfied)}$$

8.5.1.3 Axial Resistance of Bearing Stiffeners (Article 6.10.11.2.4)

The factored axial resistance is calculated from Article 6.9.2.1 of the specifications, where the radius of gyration is computed about the mid-thickness of the web, and the effective length is taken as 0.75D. For stiffeners welded to the web, part of the web is considered in the effective column section. The strip of web included in the effective column is not more than $9t_w$ on each side of the stiffeners. Therefore, the area of the effective column section is computed below:

$$A_s = 2[(6.0)(0.625) + 9(0.5)(0.5)] = 12.00 \text{ in.}^2$$

The moment of inertia of the effective column section is computed as follows:

$$I_s = \frac{0.625(6.0 + 0.5 + 6.0)^3}{12} = 101.7 \text{ in.}^4$$

The radius of gyration computed about the mid-thickness of the web is computed as:

$$r_s = \sqrt{\frac{I_s}{A_s}} = \sqrt{\frac{101.7}{12.00}} = 2.91 \text{ in.}$$

The effective length is computed as follows:

$$K\ell = 0.75D = 0.75(42.0) = 31.50 \text{ in.}$$

The bearing stiffeners must satisfy the limiting slenderness ratio, stated in Article 6.9.3, which is 140 for secondary members (Table 6.6.2.1-1) in compression.

$$\frac{K\ell}{r_s} = \frac{31.50}{2.91} = 10.82 < 140$$

(satisfied)

As previously mentioned, the factored axial resistance of the effective column section is calculated from Article 6.9.2.1 using the specified minimum yield strength of the stiffener.

$P_r = \phi_c P_n$ Eq. (6.9.2.1-1)

where:

ϕ_c = resistance factor for axial compression = 0.95 (Article 6.5.4.2)

P_n = nominal compressive resistance from Article 6.9.4.1

Determine P_n using Article 6.9.4.1.1.For bearing stiffeners, only the limit state of flexural buckling is applicable (Table 6.9.4.1.1-1). In addition, given the width-to-thickness ratio limits for bearing stiffener cross-section elements specified by Article 6.10.11.2.2 and 6.10.11.2.4b, bearing stiffeners are effectively composed only of nonslender elements. First, determine the elastic critical buckling load for flexural buckling, P_e, per Article 6.9.4.1.2 as follows:

$$P_e = \frac{\pi^2 E}{\left(\frac{K\ell}{r_s}\right)^2} A_g \qquad \text{Eq. (6.9.4.1.2-1)}$$

$$P_e = \frac{\pi^2 (29000)}{(10.82)^2}(12.00) = 29{,}338 \text{ kips}$$

$P_o = F_y A_g$

where,

P_o = nominal yield resistance

$P_o = F_y A_g$ = (50)(12.00) = 600 kips

P_o/P_e = 600/29,338= 0.02 < 2.25

Therefore, Eq. 6.9.4.1.1-1 applies.

$$P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] P_o \qquad \text{Eq. (6.9.4.1.1-1)}$$

$$P_e = \left[0.658^{\frac{600}{29{,}338}}\right](600) = 595 \text{ kips}$$

P_r = 0.95(595) = 565 kips > R_u = 257 kips (satisfied)

8.5.1.4 Bearing Stiffener-to-Web Welds

Adequate shear resistance of the fillet welds joining the bearing stiffener to the web must also be verified. As specified in Article 6.13.3.2.4, the resistance of fillet welds is to be taken as the smaller of the factored shear rupture resistance of the connected material adjacent to the weld leg (Article 6.13.5.3) and 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.

First the factored resistance of the weld metal is determined, which is taken as 60 percent of the classification strength of the weld metal, F_{exx}, times the resistance factor for shear parallel to the axis of the weld, ϕ_{e2}. The classification strength of the weld metal is the minimum specified tensile strength of the weld metal in ksi, which is reflected in the classification designation of the electrode. Matching weld metal (i.e., with the same or slightly higher minimum specified tensile strength compared to the minimum specified properties of the base metal) is used.

$$R_r = 0.6\phi_{e2}F_{exx} \qquad \text{Eq. (6.13.3.2.4-1)}$$

where:

ϕ_{e2} = resistance factor for shear parallel to the axis of the weld = 0.80 (Article 6.5.4.2)

F_{exx}= classification strength of the weld metal = 70 ksi for this example

$$R_r = 0.6(0.80)(70) = 33.6 \text{ ksi}$$

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. Using this weld size the factored shear resistance per unit length of weld is computed as follows:

$$v = 33.6(0.707)(0.25) = 5.94 \text{ k/in.}$$

The factored shear rupture resistance of the connected material adjacent to the weld leg is computed as follows (Article 6.13.5.3) substituting the thickness of the connected material, t, for A_{vn} in the equation to express the factored resistance in units of kips/in.:

$$R_r = \varphi_{vu}0.58R_pF_ut \qquad \text{Eq. (6.13.5.3-2)}$$

where: ϕ_{vu} = resistance factor for shear rupture of connection elements = 0.8 (Article 6.5.4.2)

F_u = tensile strength of the connected element specified in Table 6.4.1-1 (ksi)

R_p = reduction factor for punched holes taken equal to 1.0 for a welded connection

$$R_r = 0.80(0.58)(1.0)(70)(0.5) = 16.24 \text{ kips / in.}$$

The factored shear rupture resistance of the connected material does not control.

The length of the weld, allowing 2.5 inches for clips at both the top and bottom of the stiffener, is:

$$L = 42.0 - 2(2.5) = 37.0 \text{ in.}$$

The total factored resistance of the weld connecting the stiffener to the web of the section is then 879 kips, which is greater than the factored shear of 257 kips.

$$4(37.0)(5.94) = 879 \text{ kips} > 257 \text{ kips} \qquad \text{(satisfied)}$$

8.6 Flange-to-Web Weld Design

This section outlines the fillet weld design for the flange-to-web welds. The weld design resistance is checked against the factored horizontal shear flow associated with the design loads. The horizontal shear flow at the end bearing is computed from the following equation:

$$s = \frac{VQ}{I}$$

where:

V = shear force (kips)

Q = statical moment of the area about the neutral axis (in.^{3})

I = moment of inertia (in.^{4})

Similar to previous calculations, the shear flow will be computed by considering the cross-sectional properties applicable to various applied forces. Thus, the statical moment of the area about the neutral axis, calculated as the area of the component times its centroidal distance to the neutral axis of the section, is computed for each applicable section.

8.6.1 Steel Section:

Top Flange: $Q = (10.50)(25.42) = 266.9 \text{ in.}^3$

Bottom Flange: $Q = (20.00)(17.59) = 351.8 \text{ in.}^3$

8.6.2 Long-term Section:

Top Flange:	Q = (10.50)(13.08) =	137.3 in.^{3}
Slab:	Q = (34.0)(18.70) =	635.8 in.^{3}
		773.1 in.^{3}
Bottom Flange:	Q = (20.00)(29.93) =	598.6 in.^{3}

8.6.3 Short-term Section:

Top Flange:	Q = (10.50)(4.79) =	50.3 in.^{3}
Slab:	Q = (102.0)(10.41) =	1,061.8 in.^{3}
		1,112.1 in.^{3}
Bottom Flange:	Q = (20.00)(38.22) =	764.4 in.^{3}

The factored shear flow under each loading is thus computed as follows, where it is determined that the top flange experiences the highest level of shear flow.

Top Flange:

DC_1:	s = (1.25)(44)(266.9)/16,401	=	0.90
DC_2:	s = (1.25)(9)(773.1)/36,315	=	0.24
DW:	s = (1.5)(7)(773.1)/36,315	=	0.22
LL+IM	s = (1.75)(103)(1,112.1)/49,905	=	4.02
		=	5.38 kip/in.

Bottom Flange:

DC_1:	s = (1.25)(44)(351.8)/16,401	=	1.18
DC_2:	s = (1.25)(9)(598.6)/36,315	=	0.19
DW:	s = (1.5)(7)(598.6)/36,315	=	0.17
LL+IM	s = (1.75)(103)(764.4)/49,905	=	2.76
		=	4.30 kip/in.

The factored shear flow of 5.38 k/in. must be evaluated in comparison to the smaller of the factored shear resistance of the weld metal and the factored shear rupture resistance of the connected material. The specifications limit the minimum size of a fillet weld in which the base metal is thicker than 0.75 in. to 5/16 in. (Table 6.13.3.4-1). Therefore, a 5/16-in. fillet weld is assumed on each side of the plate. The factored shear resistance of the weld metal is determined as follows:

$$R_r = 0.6\phi_{e2}F_{exx} \qquad \text{Eq. (6.13.3.2.4b-1)}$$

where:

ϕ_{e2} = resistance factor for shear parallel to the axis of the weld = 0.80 (Article 6.5.4.2)

F_{exx} = classification strength of the weld metal = 70 ksi

R_r = 0.6(0.80)(70) = 33.6 ksi

The factored shear resistance per unit length of weld for the 5/16-inch welds is:

$$v = 33.6(0.707)(0.3125)(2) = 14.85 \text{ kips/in.}$$

From Article 6.13.5.3, the factored shear rupture resistance of the connected material is computed as follows:

$$R_r = \phi_{vu}0.58R_pF_uA_{vn} \qquad \text{Eq. (6.13.5.3-2)}$$

where:

A_{vn} = net area of the connection element subject to shear (equal to the gross area for welded connections)

F_u = tensile strength of the connection element specified in Table 6.4.1-1

ϕ_{vu} = resistance factor for shear rupture of connection elements = 0.80 (Article 6.5.4.2)

R_p = reduction factor for holes taken equal to 0.90 for bolt holes punched full size and 1.0 for bolt holes drilled full size or subpunched and reamed to size (equal to 1.0 for welded connections)

The factored shear rupture resistance of the connected material is therefore:

$$v = (0.80)(0.58)(1.0)(70)(0.5) = 16.24 \text{ kips/in.}$$

The factored shear rupture resistance of the connected material does not control. Since v = 14.85 kips/in. is greater than the factored shear flow of 5.38 kips/in., the 5/16" fillet weld is adequate for the flange-to-web welds.

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