Steel Bridge Design Handbook

APPENDIX

Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam 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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8. Abstract This design example presents an alternative design to that presented in the Steel Bridge Design Handbook Design Example 2A. Specifically, the design of a straight two-span continuous steel bridge is presented using rolled wide-flange beams, as an alternative to welded plate-girder sections. The AASHTO LRFD Bridge Design Specifications are the governing specifications and all aspects of the provisions applicable to I-section bridge design (i.e., evaluation of cross-section proportion limits, and constructability, service, fatigue, and strength limit state requirements) are considered. In addition to the beam design, the design of the concrete deck using the Empirical Design Method is illustrated. A basic wind analysis of the structure at the strength limit state is also presented.	
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Steel Bridge Design Handbook
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1.0 INTRODUCTION

This design example presents an alternative design to that presented in NSBA's *Steel Bridge Design Handbook Design: Example 2A* [1]. Specifically, the design of a straight two-span continuous steel bridge is presented herein using rolled wide-flange beam sections, as an alternative to the welded plate-girder sections used in Design Example 2A. The Ninth Edition of the *AASHTO LRFD Bridge Design Specifications* [2], referred to herein as the *AASHTO LRFD BDS*, is the governing specification and all aspects of the provisions applicable to I-section design (i.e., evaluation of cross-section proportion limits, and constructability, service, fatigue, and strength limit state requirements) are considered. The optional moment redistribution specifications given in Appendix B6 of the *AASHTO LRFD BDS* are not invoked in this example. In addition to the beam design, the design of the concrete deck is using the Empirical Design Method is illustrated (note that the use of the Empirical Design Method in the design of the concrete deck should only be undertaken with the full knowledge and consent of the Owner). A basic wind analysis of the structure at the strength limit state is also presented. Although not illustrated herein, the need for bearing stiffeners at all bearing locations on rolled shapes is to be evaluated according to the provisions of Article D6.5 in the *AASHTO LRFD BDS*.

2.0 DESIGN PARAMETERS

The purpose of this example is to illustrate the design of a straight two-span continuous composite bridge having equal spans of 90.0 feet using rolled wide-flange beams. The bridge cross-section (see Figure 1) has four rolled wide-flange beams spaced at 10.0 feet with 3.5-foot deck overhangs providing for a 34.0-foot roadway width. The reinforced concrete deck is 8.5 inches thick, including a 0.5-inch integral wearing surface and a 2.0-inch concrete haunch measured from the bottom of the top flange to the bottom of the concrete deck. Considerations in choosing between a rolled beam versus a welded plate girder are discussed further in the AASHTO/NSBA Steel Bridge Collaboration G12.1 *Guidelines to Design for Constructability and Fabrication* [3].

The framing plan for this design example is shown in Figure 2. As will be demonstrated subsequently, the spacing of the cross-frames is governed by constructability requirements in regions of positive bending and lateral-torsional buckling requirements in regions of negative bending at the strength limit state.

ASTM A709, Grade 50W is used for all structural steel 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*. Unless stated otherwise, the specific articles, sections, and equations referenced throughout this example are contained in these specifications.

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

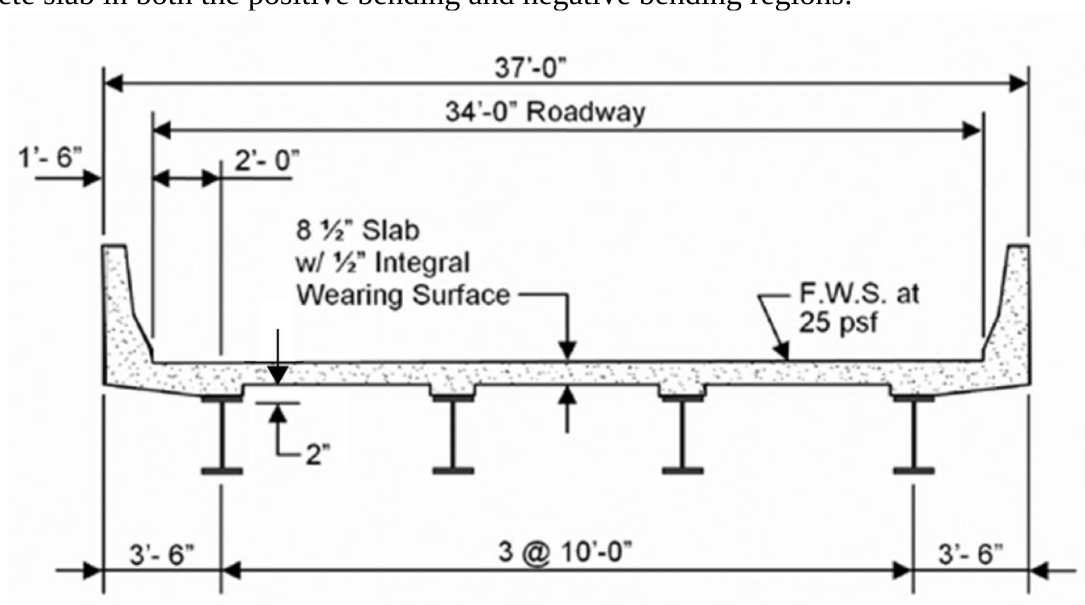

Figure 1 Sketch of the Typical Bridge Cross Section

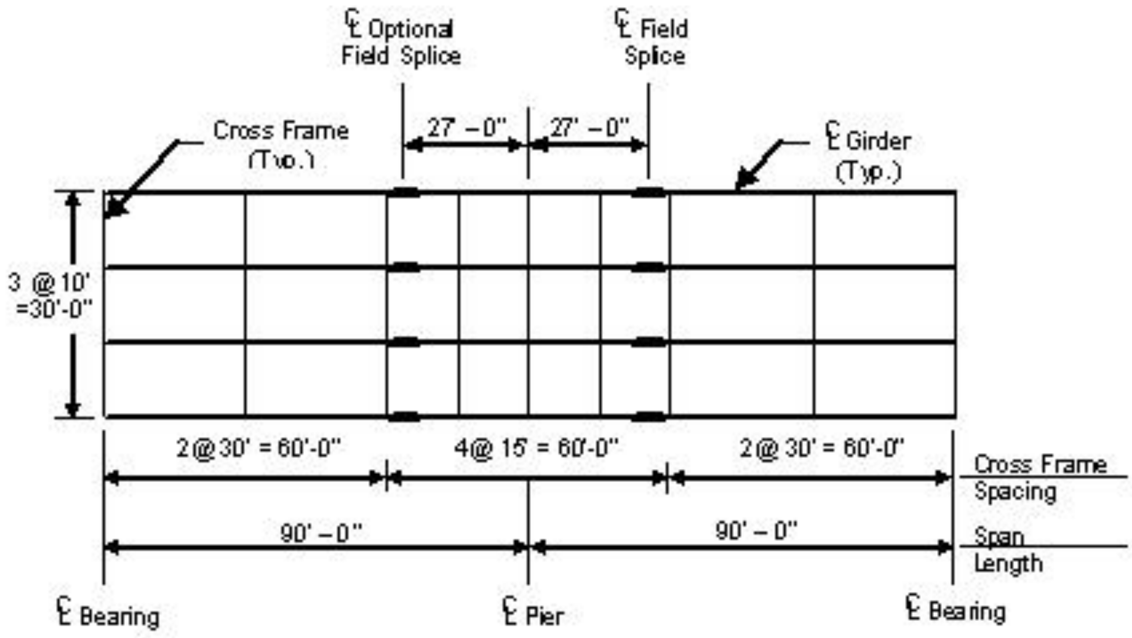

Figure 2 Sketch of the Superstructure Framing Plan

3.0 BEAM ELEVATION

The beam elevation, shown in Figure 3, assumes a bolted field splice is provided at 30% of the span length (27 feet) from the interior pier. The design of the beam from the abutment to a location 63.0 feet into each span is primarily based on positive bending moments; thus, these sections of the beam are referred to as either the "positive bending region" or "Section 1" throughout this example. Alternatively, the beam geometry at the pier is controlled by negative bending moments; consequently, the region of the beam extending 27.0 feet on either side of the pier will be referred to as the "negative bending region" or "Section 2".

By iteratively selecting various rolled wide-flange beams from the available shapes, the rolled wide-flange section shown in Figure 3 was selected for this example. The beam satisfies all the cross-section proportion limits specified in Articles 6.10.2.1 and 6.10.2.2. Refer to in NSBA's *Steel Bridge Design Handbook Design: Example 2A* [1] for an illustration of these checks.

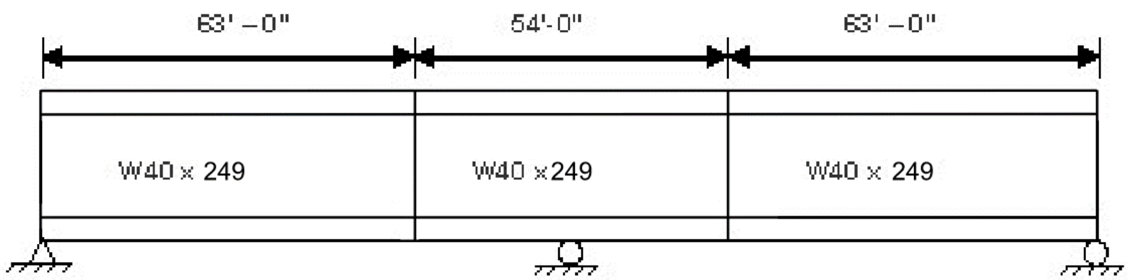

Figure 3 Sketch of the Beam Elevation

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

As discussed in the in NSBA's *Steel Bridge Design Handbook Design: Example 2A* [1], the bridge dead loads are classified into two categories: dead load of structural components and non-structural attachments (DC), and dead load of wearing surface and utilities (DW).

Load factors of 1.25 and 1.00 are used for DC at the Strength and Service Limit States, respectively. For DW, a load factor of 1.50 is used at the Strength Limit State and a load factor of 1.00 is used at the Service Limit State.

4.1.1 Component Dead Load (DC)

As discussed in Design Example 2A, the component dead load is separated into two parts: dead loads acting on the non-composite section (DC_1) and dead loads acting on the long-term composite section (DC_2). DC_1 is assumed to be carried by the steel section alone. DC_2 is assumed to be resisted by the long-term composite section, which consists of the steel beam plus an effective width of the concrete slab when the beam is in positive bending, and the beam plus the longitudinal steel reinforcing within the effective width of the slab when the beam is in negative bending at the strength limit state. 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. 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 conditions are met.

DC_1 includes the beam 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 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 0.018 k/ft is assumed. It is also assumed that these dead loads are equally distributed to all beams 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 = (2.0 - 1.42)(15.8)/144 x 0.150 = 0.010 k/ft

Overhang taper = 2 x (1/2) x (3.5 - 7.90/12) x (2.0/12) x 0.150/4 = 0.018 k/ft

Beam = 0.249 k/ft

Cross-frames and misc. steel details = 0.018 k/ft

Stay-in-place forms = 0.015 x (30 - 3 x (15.8/12))/4 = 0.098 k/ft

Total DC_1 = 1.376 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 beams, 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 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. Only a single lane of live load is considered on the bridge.

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. Wind load effects on the girders during construction are not evaluated herein; refer to in NSBA's *Steel Bridge Design Handbook Design: Example 1* [4] for an illustration of these checks.

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 28 feet above the surrounding ground and that the bridge is located in central Ohio 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 central Ohio, 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.

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.

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

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* [4] for detailed descriptions of each of the load combinations.

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

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

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:

$$\eta_i = \frac{1}{\eta_D \eta_R \eta_I} \leq 1.0$$

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, unless otherwise specified by the Owner (Article 3.4.2). 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.

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 verify 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 beam 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 multiplying these forces by multiple presence factors, live-load distribution factors, and dynamic load 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., fraction of a truck or lane load) distributed to a given beam. 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 based on the location of the beam (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. For a slab-on-stringer bridge, as considered in the present example, the following criteria must be satisfied: the beam spacing must be between 3.5 and 16.0 feet, the slab must be at least 4.5 inches thick and less than or equal to 12.0 inches thick, the span length must be between 20 and 240 feet, and the cross-section must contain at least 4 beams. Because all these requirements are satisfied in this example, in addition to the conditions listed in Article 4.6.2.2.1, the computation of distribution factors using the approximate methods and simplified analysis procedures of Article 4.6.2.2 may proceed as follows.

Distribution factors are a function of the beam spacing, slab thickness, span length, and the stiffness of the beam. Since the stiffness parameter depends on the beam geometry that is 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 beam geometry previously shown in Figure 3. It is noted that due to the uniform cross-section of the beam in this example, the distribution factors are also uniform along the beam length. In cases where the cross-section varies along the length of the beam, distribution factors should be calculated for each unique cross-section.

5.2.1 Interior Beam - Strength and Service Limit State

For interior beams, the distribution factor at the strength and service limit states is determined based on the empirical equations given in Article 4.6.2.2.2. The stiffness parameter, K_g, required for the distribution factor equations is computed as follows.

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

where:

n = modular ratio (= 8)
I = moment of inertia of the steel beam = 19,600 in.^{4} for the rolled beam
A = area of the steel beam = 73.5 in.^{2} for the rolled beam
e_g = distance between the centroid of the girder and centroid of the slab

Thus, K_g is determined as follows (refer to Figure 4):

$$e_g = 19.70 + (2.0 - 1.42) + 4.0 = 24.28 \text{ in.}$$

$$K_g = n(I + Ae_g^2) = 8(19{,}600 + 73.5(24.28)^2) = 503{,}437 \text{in.}^4$$

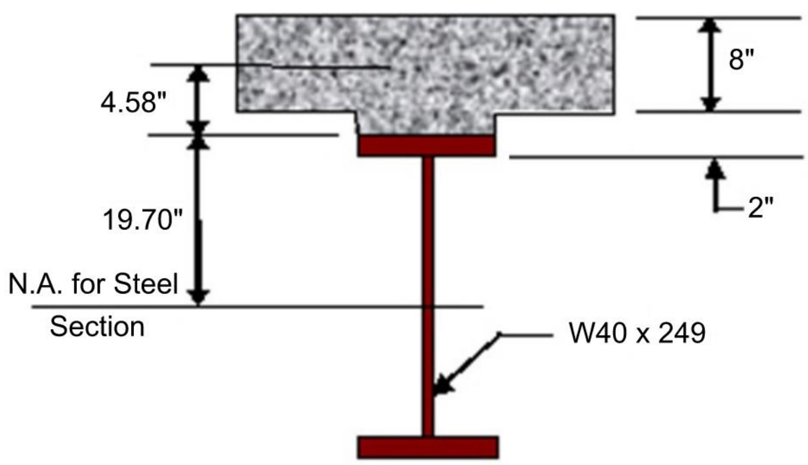

Figure 4 Rolled Beam Cross Section

5.2.1.1 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.

$$DF = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1} \text{ (for one lane loaded)}$$

where: S = beam spacing (ft)

L = span length (ft)

t_s = slab thickness (in.)

K_g = stiffness term (in.^{4})

$$DF = 0.06 + \left(\frac{10.0}{14}\right)^{0.4}\left(\frac{10.0}{90.0}\right)^{0.3}\left(\frac{503,437}{12(90.0)(8.0)^3}\right)^{0.1} = 0.508 \text{ lanes}$$

$$DF = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1} \text{ (for two or more lanes loaded)}$$

$$DF = 0.075 + \left(\frac{10.0}{9.5}\right)^{0.6}\left(\frac{10.0}{90.0}\right)^{0.2}\left(\frac{503,437}{12.0(90.0)(8.0)^3}\right)^{0.1} = 0.733 \text{ lanes (governs)}$$

5.2.1.2 Shear

The empirical equations for distribution of live load shear in an interior beam 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.

$$DF = 0.36 + \frac{S}{25} \text{ (for one lane loaded)}$$

$$DF = 0.36 + \frac{10.0}{25} = 0.760 \text{ lanes}$$

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

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

5.2.2 Exterior Girder – Strength and Service Limit States

Distribution factors for the exterior beam at the strength and service limit states are based on the maximum of: (1) a modification of the empirical equations for interior beams given above, (2) the lever rule, or (3) 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.

5.2.2.1 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 beam for the case of one-lane loaded. The lever rule assumes the deck is hinged at the interior beam, and statics is then employed to determine the percentage of the truck weight resisted by the exterior beam, i.e., the distribution factor, for one loaded lane. 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 5 for the present example. The calculated reaction of the exterior beam 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

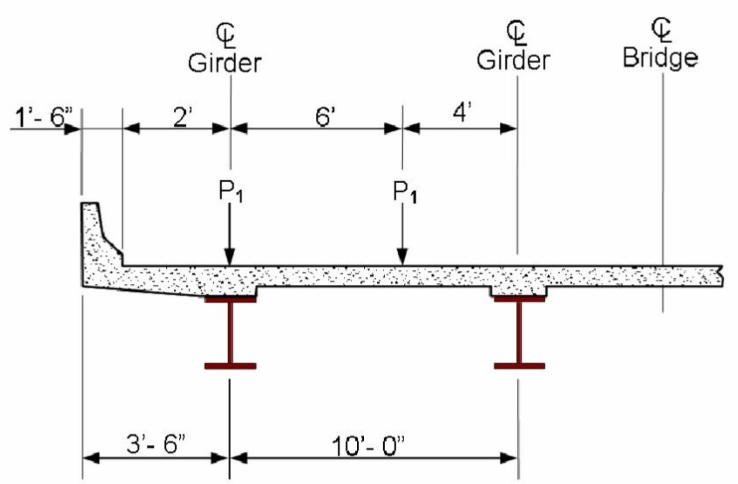

Figure 5 Sketch of the Truck Location for the Lever Rule

Modified Interior Girder Distribution Factor:

For the case of two or more lanes loaded, Table 4.6.2.2.2d-1 gives a modification factor that is to be multiplied by the interior beam distribution factor to determine the exterior beam distribution factor. The modification factor for moment is given by the following equation:

$$e = 0.77 + \left(\frac{d_e}{9.1}\right)$$

where:

d_e = the horizontal distance between the centerline of the exterior beam at deck level and the interior face of the traffic barrier or curb (ft)

d_e = 2.0 ft

$$e = 0.77 + \left(\frac{2.0}{9.1}\right) = 0.990 < 1.0$$

Multiplying the modification factor by the interior beam distribution factor for two or more lanes loaded gives the following:

DF = 0.990(0.733) = 0.726 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 6 shows the truck locations for the special analysis. 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 proceeds as follows beginning with the calculations for one loaded lane (the appropriate multiple presence factors, MPF, that are applied in each case are shown):

DF = $m_1(R_1)$ (one lane loaded)

$$DF = 1.2\left(\frac{1}{4} + \frac{(15.0)(12.0)}{2\left[(15.0)^2 + (5.0)^2\right]}\right) = 0.732 \text{ lanes (Note, MPF = 1.2)}$$

Similarly, for two loaded lanes the distribution factor is computed as follows:

DF = $m_2(R_2)$ (two lanes loaded)

$$DF = 1.0\left(\frac{2}{4} + \frac{(15.0)(12.0 + 0.0)}{2\left[(15.0)^2 + (5.0)^2\right]}\right) = 0.860 \text{ lanes (governs) (Note, MPF = 1.0)}$$

Comparing the four distribution factors computed above for moment in the exterior beam, it is determined that the controlling distribution factor is equal to 0.860 lanes, which is determined based on the special analysis procedure considering two lanes loaded. Compared to the interior beam distribution factor for moment, which was computed to be 0.733 lanes, it is shown that the exterior beam distribution factor is larger, and thus, the exterior beam distribution factor controls the bending strength design at the strength and service limit state.

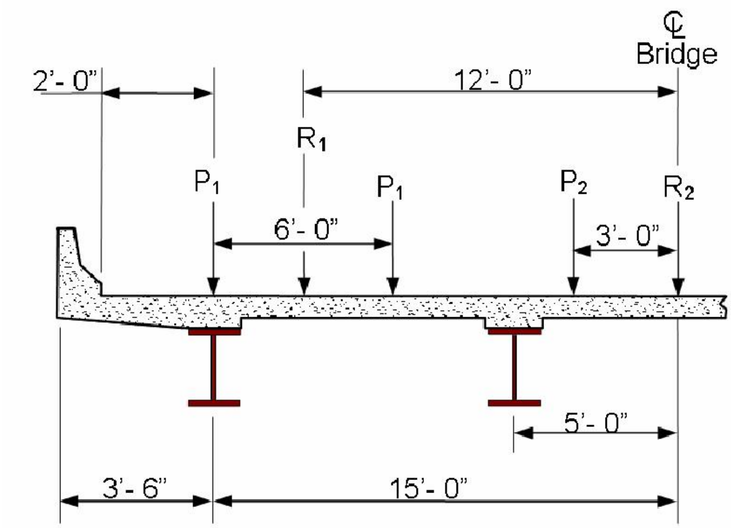

Figure 6 Sketch of the Truck Locations for the Special Analysis

5.2.2.2 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 the distribution factor for shear for one-lane loaded 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:

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

$$e = 0.60 + \left(\frac{2.0}{10.0}\right) = 0.800$$

Applying this modification factor to the previously computed interior beam 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 for one lane loaded

DF = 0.860 lanes for two lanes loaded (governs)

Thus, the controlling distribution factor for shear in the exterior beam is 0.860 lanes, which is less than that of the interior beam. Additionally, the interior beam distribution factor of 0.952 lanes controls the shear design.

5.2.3 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.

5.2.3.1 Bending Moment

It was determined above that the governing distribution factor for moment at the strength and service limit states for one loaded lane was equal to 0.840 lanes, which was based on the lever rule. Dividing this value by the multiple presence factor for one-lane loaded gives the following distribution factor for fatigue moment:

$$\text{DF} = \frac{0.840}{1.20} = 0.700 \text{ lanes (exterior girder)}$$

5.2.3.2 Shear

From review of the shear distribution factors computed above for the strength and service limit states, it was determined that the maximum distribution factor for one-lane loaded was equal to 0.840 lanes, which was based on the lever rule. Thus, the distribution factor for fatigue shear is equal to 0.840 lanes divided by the multiple presence factor for one-lane loaded of 1.2.

$$\text{DF} = \frac{0.840}{1.20} = 0.700 \text{ lanes}$$

5.2.4 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 beams 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.

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

The governing live load distribution factors are summarized below in Table 1.

Table 1 Governing Live Load Distribution Factors (Lanes)

	Distribution Factor
Strength/Service Bending Moment	0.860
Strength/Service Shear	0.952
Fatigue Bending Moment	0.700
Fatigue Shear	0.700
Deflection	0.500

5.3 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 7 through 10 show the moment and shear envelopes for this design example, which are based on the data presented in Tables 2 through 8. The live load moments and shears shown in these figures are based on the controlling 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 load moment.

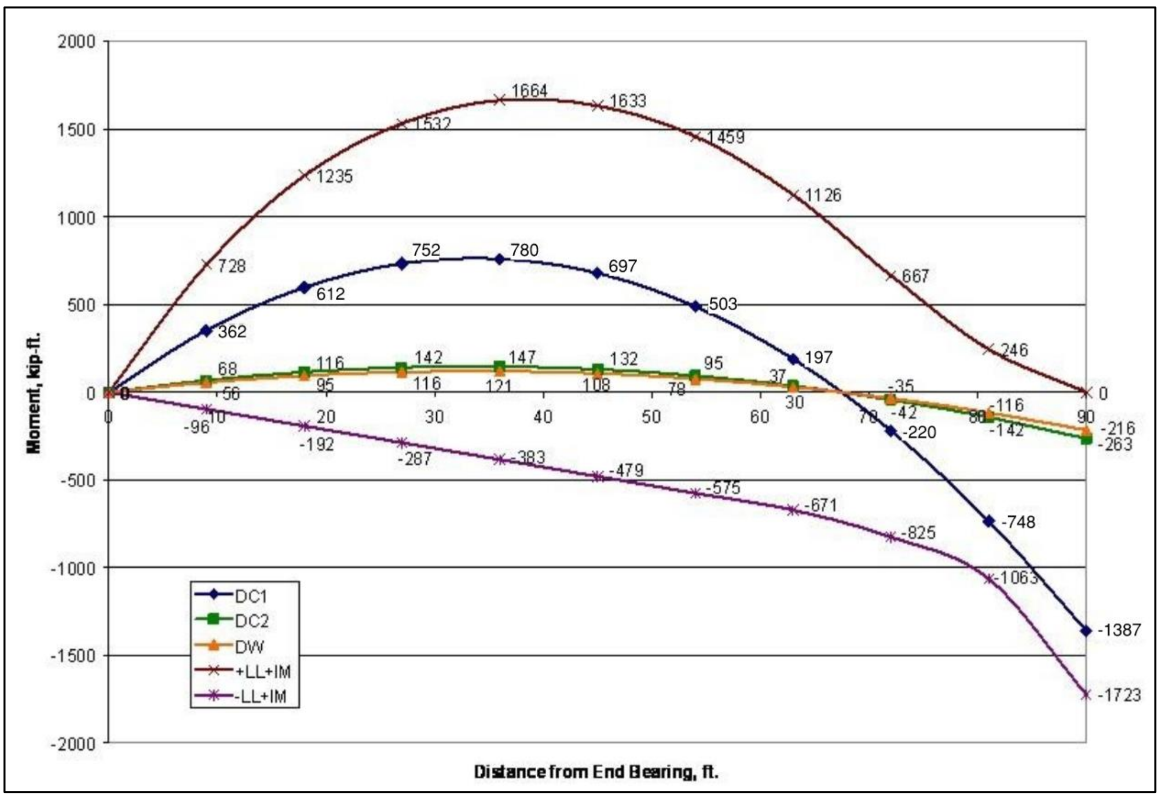

Figure 7 Dead and Live Load Moment Envelopes

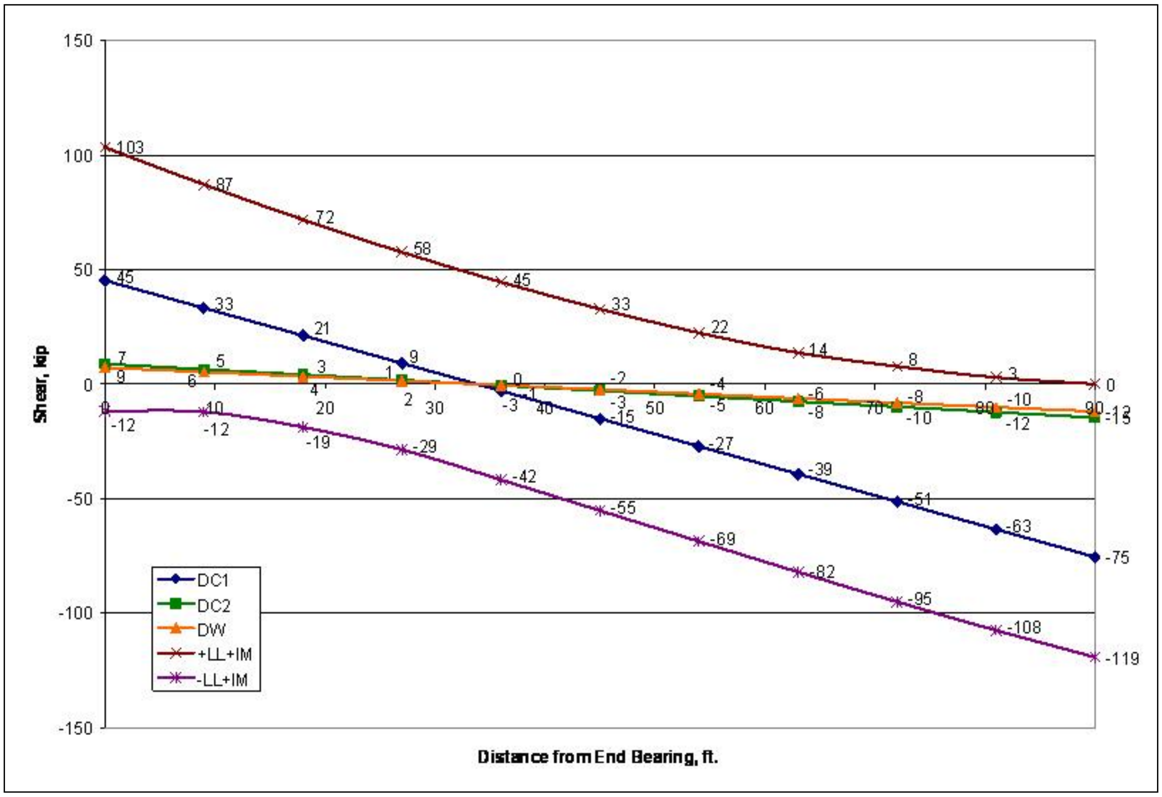

Figure 8 Dead and Live Load Shear Envelopes

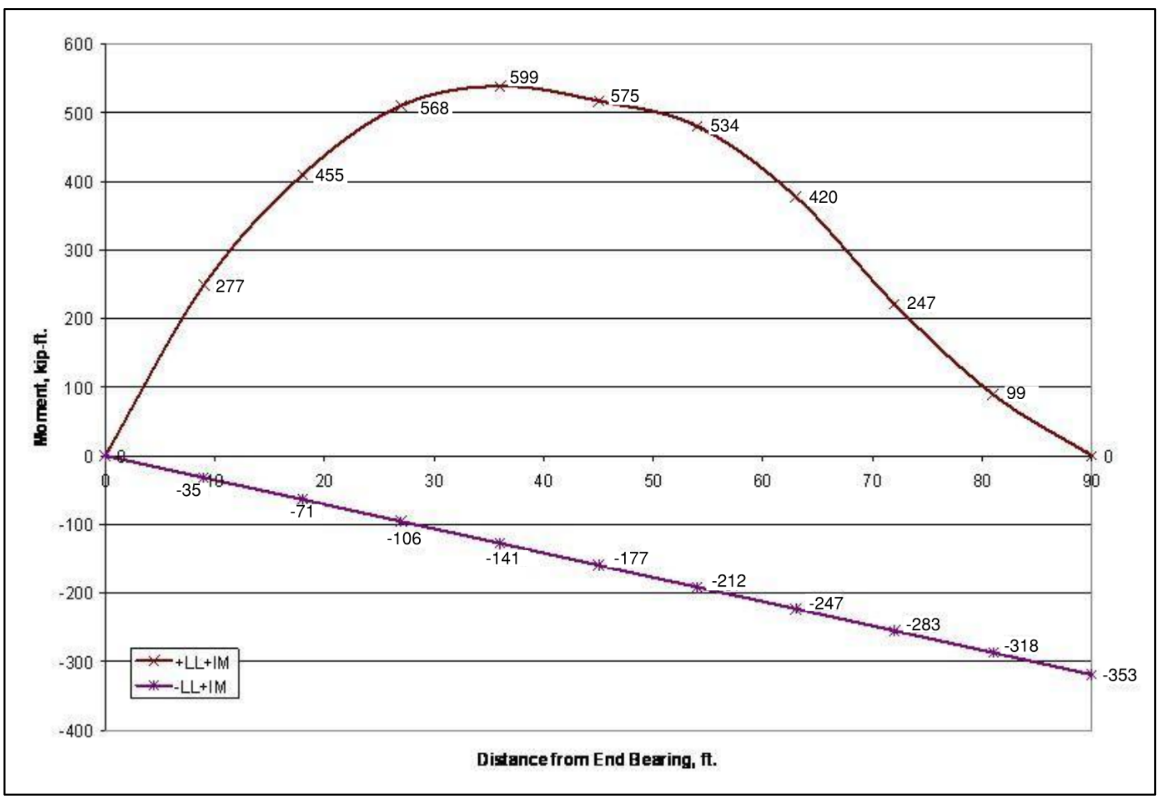

Figure 9 Fatigue Live Load Moments

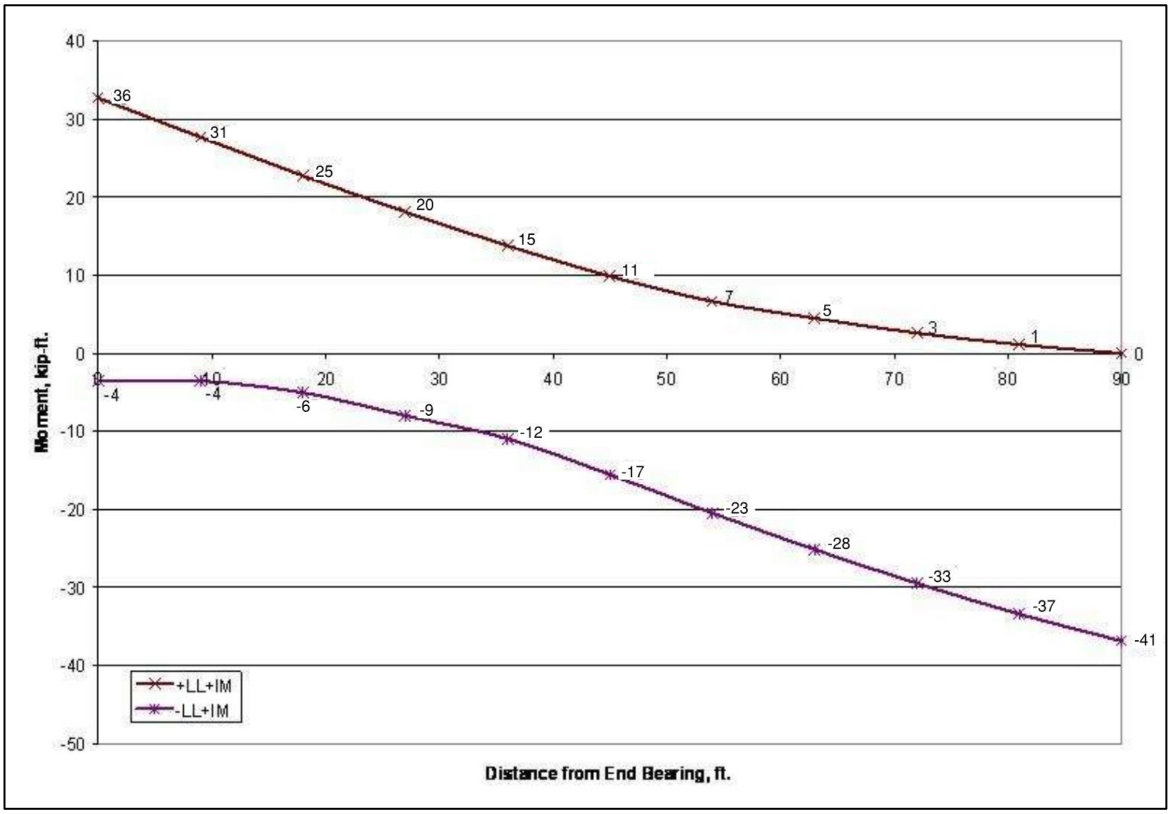

Figure 10 Fatigue Live Load Shears

Table 2 Unfactored and Undistributed Moments (kip-ft)

	Non-Com. Dead	Com. Dead	Wearing Surface	Truck Load		Lane Load		Tandem		Double Truck		Double Tandem	
Span 1	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	362	68	56	486	-59	201	-32	381	-43	0	0	0	0
0.20	612	116	95	816	-119	350	-65	653	-86	0	0	0	0
0.30	752	142	116	1003	-178	447	-97	818	-130	0	0	0	0
0.40	780	147	121	1085	-238	492	-130	884	-173	0	0	0	0
0.50	697	132	108	1062	-297	486	-162	868	-216	0	0	0	0
0.60	503	95	78	954	-357	428	-194	781	-259	0	0	0	0
0.70	197	37	30	746	-416	318	-227	627	-302	0	0	0	0
0.80	-220	-42	-35	466	-475	156	-259	424	-346	0	-476	0	-607
0.90	-748	-142	-116	148	-535	30	-380	192	-389	0	-746	0	-682
1.00	-1387	-263	-216	0	-594	0	-648	0	-432	0	-1187	0	-758

Table 3 Unfactored and Undistributed Live Load Moments (kip-ft)

Span 1	Vehicle		Special	Standard	1.33 Vehicle + Lane	Distribution	LL+I	
	positive	negative	negative	negative	positive	Factors	Positive	Negative
0.00	0	0	0	0	0	0.86	0	0
0.10	486	-59	-29	-111	847	0.86	728	-96
0.20	816	-119	-58	-223	1436	0.86	1235	-192
0.30	1003	-178	-87	-334	1782	0.86	1532	-287
0.40	1085	-238	-117	-446	1935	0.86	1664	-383
0.50	1062	-297	-146	-557	1899	0.86	1633	-479
0.60	954	-357	-175	-669	1696	0.86	1459	-575
0.70	746	-416	-204	-780	1310	0.86	1126	-671
0.80	466	-475	-959	-891	775	0.86	667	-825
0.90	192	-535	-1236	-1092	286	0.86	246	-1063
1.00	0	-594	-2004	-1438	0	0.86	0	-1723

Table 4 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	453	86	84	1274	-168	1897	455
0.20	765	145	142	2161	-335	3213	717
0.30	940	178	175	2681	-503	3974	790
0.40	975	184	181	2913	-671	4253	669
0.50	871	165	162	2857	-839	4055	359
0.60	629	118	116	2553	-1006	3416	-143
0.70	246	46	45	1971	-1174	2308	-837
0.80	-275	-53	-52	1167	-1444	787	-1824
0.90	-935	-178	-175	431	-1860	-857	-3148
1.00	-1734	-329	-323	0	-3016	-2386	-5402

Table 5 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	362	68	56	947	-125	1433	361
0.20	612	116	95	1605	-249	2428	574
0.30	752	142	116	1992	-374	3002	636
0.40	780	147	121	2164	-498	3212	550
0.50	697	132	108	2123	-623	3060	314
0.60	503	95	78	1896	-747	2572	-71
0.70	197	37	30	1464	-872	1728	-608
0.80	-220	-42	-35	867	-1073	570	-1370
0.90	-748	-142	-116	320	-1382	-686	-2388
1.00	-1387	-263	-216	0	-2240	-1866	-4106

Table 6 Unfactored and Undistributed Shears (kip)

Span 1	Non-Com. Dead DC1	Com. Dead DC2	Wearing Surf. DW	Truck Load positive	Truck Load negative	Lane Load positive	Lane Load negative	Tandem positive	Tandem negative
0.00	45	9	7	63	-7	25	-4	49	-5
0.10	33	6	5	54	-7	20	-4	42	-5
0.20	21	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	-28
0.60	-27	-5	-4	15	-42	3	-16	14	-34
0.70	-39	-8	-6	10	-50	2	-20	10	-38
0.80	-51	-10	-8	5	-56	1	-25	6	-43
0.90	-63	-12	-10	2	-62	0	-30	2	-46
1.00	-75	-15	-12	0	-67	0	-36	0	-49

Table 7 Unfactored and Undistributed Live Load Shears (kip)

Span 1	vehicle positive	vehicle negative	1.33 V Vehicle + V Lane positive	1.33 V Vehicle + V Lane negative	Distribution Factors	V LL positive	V LL negative
0.00	63	-7	109	-12	0.952	103	-12
0.10	54	-7	92	-13	0.952	87	-12
0.20	45	-11	75	-20	0.952	72	-19
0.30	37	-18	60	-30	0.952	58	-29
0.40	29	-26	47	-44	0.952	45	-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	0	-125	0.952	0	-119

Table 8 Strength I Load Combination Shear (kip)

Span 1	1.25 DC1	1.25 DC2	1.5 DW	1.75 (LL+IM) positive	1.75 (LL+IM) negative	Strength I positive	Strength I negative
0.00	57	11	11	181	-21	259	58
0.10	42	8	8	153	-21	210	36
0.20	26	5	5	126	-33	162	4
0.30	11	2	2	101	-50	116	-35
0.40	-4	-1	-1	78	-73	73	-78
0.50	-19	-4	-4	57	-97	31	-123
0.60	-34	-7	-6	39	-120	-8	-167
0.70	-49	-10	-9	24	-144	-44	-212
0.80	-64	-12	-12	13	-166	-75	-255
0.90	-79	-15	-15	5	-188	-105	-298
1.00	-94	-18	-18	0	-209	-131	-339

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. The load is distributed using the distribution factor of 0.500 lanes calculated earlier.

The maximum deflection due to the design truck is 0.982 inches. Applying the impact and distribution factors gives the following deflection for the design truck load case:

$$\Delta_{LL+IM} = 0.500 \text{ x } 1.33 \text{ x } 0.982 = 0.653 \text{ in.} \qquad \text{(governs)}$$

The maximum deflection due to the lane loading only is 0.510 inches. Thus, the deflection due to 25% of the design truck plus the lane loading is equal to the following:

$$\Delta_{LL+IM} = 0.500 \text{ (1.33 x 0.25 x 0.982 + 0.510)} = 0.418 \text{ in.}$$

Thus, the governing deflection, equal to 0.653 inches, will subsequently be used to assess the beam 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 I and Fatigue II 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 provide adequate strength and stability during each construction phase. The deck casting sequence has a significant influence 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 section presents the calculations necessary to evaluate the preliminary beam design for adequate resistance at the strength, service, and fatigue limit states. Adequate strength of the bridge in its final condition and at all stages of the deck-casting sequence is verified. Also presented is the concrete deck design utilizing the Empirical Design Method. The moment and shear envelopes provided in Figures 7 through 10 are employed for the following calculations.

8.1 Section Properties

The section properties for the beam are first calculated as these properties 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 as 8 in these calculations.

8.1.1 Effective Flange Width (Article 4.6.2.6)

Article 4.6.2.6 of the specifications governs the determination of the effective flange width of the concrete slab, where alternative calculations are specified for interior and exterior beams.

The effective flange width, b_{eff}, for interior beams is determined as one-half the distance to the adjacent girder on each side of the component as follows:

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

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

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

8.1.2 Elastic Section Properties

As discussed previously in Section 4.0, the elastic section properties vary based on the loading conditions. The section properties for the steel section (beam alone) are used for the dead loads applied to the noncomposite section. In positive bending, live loads are applied to the full composite section, termed the short-term composite section, where the modular ratio of 8 is used in the computations. Alternatively, dead loads on the composite section are applied to what is termed the long-term composite section. The long-term composite section is considered to be comprised of the full steel beam and one-third of the concrete deck to account for the reduction in strength that may occur in the deck over time due to creep effects. This is accounted for in the section property calculations through use of a modular ratio equal to 3 times the base modular ratio, or 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 (Tables 9 and 10). Section properties are also computed for the section consisting of the beam and longitudinal reinforcing steel only assuming that the concrete is not effective in tension for use at the strength limit state (Table 11). Refer to Section 8.4 for the design of the deck reinforcing steel. Typically, the area of the concrete deck haunch is only considered in the computation of the DC_1 load and is not considered in the computation of the composite section properties; the haunch depth is considered in the computation of the composite section properties in this design example, however. Note that some Owner-agencies do not allow the consideration of the haunch depth in the computation of these section properties.

The section properties of the W40x249 beam are as follows:

I_{NA} = 19,600 in.^{4}

$d_{TOP\ OF\ STEEL}$ = 19.70 in. $S_{TOP\ OF\ STEEL}$ = 993 in.^{3}

$d_{BOT\ OF\ STEEL}$ = 19.70 in. $S_{BOT\ OF\ STEEL}$ = 993 in.^{3}

Table 9 Short Term Composite (n) Section Properties

Component	A	d	Ad	Ad^2	I_o	I
Steel Section	73.5					19,600
Concrete Slab (8″ x 102″/8)	102.0	24.28	2,477	60,131	544.0	60,675
Σ	175.5		2,477			80,275

-14.11(2,477) = -34,950

I_{NA} = 45,325 in.^{4}

$$d_n = \frac{2,477}{175.5} = 14.11 \text{ in.}$$

$$d_{\text{Top of Steel}} = 19.70 - 14.11 = 5.59 \text{ in.} \qquad d_{\text{Bot of Steel}} = 19.70 + 14.11 = 33.81 \text{ in.}$$

$$S_{\text{Top of Steel}} = \frac{45,325}{5.59} = 8,108 \text{ in.}^3 \qquad S_{\text{Bot of Steel}} = \frac{45,325}{33.81} = 1,341 \text{ in.}^3$$

Table 10 Long Term Composite (3n) Section Properties

Component	A	d	Ad	Ad^2	I_o	I
Steel Section	73.5					19,600
Concrete Slab (8″ x 102″/24)	34.0	24.28	825.5	20,044	181.3	20,225
Σ	107.5		825.5			39,825

-7.68(825.5) = -6,340

I_{NA} = 33,485 in.^{4}

$$d_{3n} = \frac{825.5}{107.5} = 7.68 \text{ in.}$$

$$d_{\text{Top of Steel}} = 19.70 - 7.68 = 11.02 \text{ in.} \qquad d_{\text{Bot of Steel}} = 19.70 + 7.68 = 27.38 \text{ in.}$$

$$S_{\text{Top of Steel}} = \frac{33,485}{11.02} = 3,039 \text{ in.}^3$$

$$S_{\text{Bot of Steel}} = \frac{33,485}{27.38} = 1,223 \text{ in.}^3$$

Table 11 Steel Section and Longitudinal Reinforcement Section Properties

Component	A	d	Ad	Ad^2	I_o	I
Steel Section	73.5					19,600
Top Long. Reinforcement	6.375	25.30	161.3	4,081		4,081
Bot. Long. Reinforcement	4.335	22.19	96.2	2,135		2,135
Σ	84.21		257.5			25,816

-3.06(257.5) = -788

I_{NA} = 25,028 in.^{4}

$$d_{\text{s+reinf}} = \frac{257.5}{84.21} = 3.06 \text{ in.}$$

$$d_{\text{Top of Steel}} = 19.70 - 3.06 = 16.64 \text{ in.}$$

$$d_{\text{Bot of Steel}} = 19.70 + 3.06 = 22.76 \text{ in.}$$

$$S_{\text{Top of Steel}} = \frac{25,028}{16.64} = 1,504 \text{ in.}^3$$

$$S_{\text{Bot of Steel}} = \frac{25,028}{22.76} = 1,100 \text{ in.}^3$$

$$d_{\text{Top of Rebar}} = 25.66 - 3.06 = 22.60 \text{ in.}$$

$$S_{\text{Top of Rebar}} = \frac{25,028}{22.60} = 1,107 \text{ in.}^3$$

8.1.3 Plastic Moment

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

$P_s = 0.85f'_c b_s t_s = 0.85(4.0)(102.0)(8)$ = 2,774 kips

$P_c = F_{yc} b_c t_c = (50)(15.8)(1.42)$ = 1,122 kips

$P_w = F_{yw} D t_w = (50)(36.56)(0.75)$ = 1,371 kips

$P_t = F_{yt} b_t t_t = (50)(15.8)(1.42)$ = 1,122 kips

The forces within each element of the beam are then compared to determine the location of the plastic neutral axis (PNA). If the following equation is satisfied, then the PNA is in the web.

CASE I:

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

1,122 kips + 1,371 kips $\geq$ 1,122 kips + 2,774 kips ?

2,493 kips < 3,896 kips

Therefore, the PNA is not in the web and the following equation is evaluated to determine if the PNA is in the top flange:

CASE II:

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

1,122 kips + 1,371 kips + 1,122 kips $\geq$ 2,774 kips ?

3,615 kips > 2,774 kips

Therefore, the plastic neutral axis is in the top flange and $\bar{y}$ is computed using the following equation:

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

$$\bar{y} = \left(\frac{1.42}{2}\right)\left[\frac{1{,}371 + 1{,}122 - 2{,}774}{1{,}122} + 1\right] = 0.53 \text{ in. from the top of the top flange}$$

The plastic moment is then calculated using the following equation:

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

The distances from the PNA to the centroid of the compression flange, web, and tension flange (respectively) are as follows.

d_s = 0.53 + 8.0/2 + 2.0 – 1.42 = 5.11 in.

d_w = 1.42 – 0.53 + 36.56/2 = 19.17 in.

d_t = 1.42 – 0.53 + 36.56 + 1.42/2 = 38.16 in.

Substitution of these distances and the above computed element forces into the M_p equation gives the following:

$$M_p = \left(\frac{1,122}{2(1.42)}\right)\left[(0.53)^2 + (1.42 - 0.53)^2\right] + \left[(2,774)(5.11) + (1,371)(19.17) + (1,122)(38.16)\right]$$

M_p = 83,697 kip-in. = 6,975 kip-ft

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

P_c	=	$F_{yc}b_ct_c$	=	(50)(15.8)(1.42)	=	1,122 kips
P_w	=	$F_{yw}D_{tw}$	=	(50)(36.56)(0.75)	=	1,371 kips
P_t	=	$F_ytb_tt_t$	=	(50)(15.8)(1.42)	=	1,122 kips
P_{rb}	=	$F_{yrb}A_{rb}$	=	(60)(4.335)	=	260 kips
P_{rt}	=	$F_{yrt}A_{rt}$	=	(60)(6.375)	=	383 kips

The plastic forces in each element are used to determine the general location of the plastic neutral axis. Because the following equation is satisfied, it is determined that the PNA is in the web:

CASE I:

$P_c + P_w \geq P_t + P_{rb} + P_{rt}$ = 1,122 kips + 1,371 kips ≥ 1,122 kips + 260 kips + 383 kips ?

2,493 kips > 1,765 kips, therefore, the plastic neutral axis is in the web.

The plastic neutral axis location measured from the top of the web is then computed from the following equation:

$$\bar{y} = \left(\frac{D}{2}\right)\left[\frac{P_c - P_t - P_{rt} - P_{rb}}{P_w} + 1\right] = \left(\frac{36.56}{2}\right)\left[\frac{1,122 - 1,122 - 383 - 260}{1,371} + 1\right] = 9.71\,\text{in.}$$

M_p is then computed as follows.

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

where:

d_{rt} = 9.71 + 2.0 + 8.0 – 2.98 = 16.73 in.

d_{rb} = 9.71 + 2.0 + 1.91= 13.62 in.

d_t = 9.71 + 1.42/2 = 10.42 in.

d_c = 36.56 – 9.71 + 1.42/2 = 27.56 in.

$$M_p = \frac{1,371}{2(36.56)}\left[(9.71)^2 + \left(36.56 - 9.71\right)^2\right]$$
$$+\left[(383)(16.73) + (260)(13.62) + (1,122)(10.42) + (1,122)(27.56)\right]$$

M_p = 67,847 kip-in. = 5,654 kip-ft

8.1.4 Yield Moment

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 Section D6.2.2 of the specifications. This computation method for the yield moment recognizes that different stages of loading (e.g., composite dead load, non-composite dead load, and live load) act on the beam 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.

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

In regions of positive bending, 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 demonstrated below. First the known quantities are substituted into Equation D6.2.2-1 to solve for M_{AD}.

$$50 = 1.0\left[\frac{1.25(780)(12)}{993} + \frac{1.25(147)(12) + 1.50(121)(12)}{1,223} + \frac{M_{AD}}{1,341}\right]$$

M_{AD} = 46,444 kip-in. = 3,870 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(780) + 1.25(147) + 1.50(121) + 3,870 Eq. (D6.2.2-2)

M_y = 5,210 kip-ft

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 is that, since the composite short-term and the composite long-term bending sections are both composed of the steel section and the longitudinal reinforcing steel, 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 the compression flange is first computed based on Equation D6.2.2-1.

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

$$(50) = \frac{(1.25)\left|-1{,}387\right|(12)}{993} + \frac{(1.25)\left|-263\right|(12) + (1.50)\left|-216\right|(12)}{1{,}100} + \frac{M_{AD}}{1{,}100}$$

M_{AD} = 24,120 kip-in. = 2,010 kip-ft

M_{yc} = (1.25)(1,387) + (1.25)(263) + (1.50)(216) + 2,010

M_{yc} = 4,397 kip-ft (governs)

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:

$$50 = 1.0\left[\frac{1.25\left|-1{,}387\right|(12)}{993} + \frac{1.25\left|-263\right|(12) + 1.50\left|-216\right|(12)}{1{,}504} + \frac{M_{AD}}{1{,}504}\right]$$

M_{AD} = 35,856 kip-in. = 2,988 kip-ft

M_{yt} = (1.25)(1,387) + (1.25)(263) + (1.50)(216) + 2,988 = 5,375 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

$$60 = 1.0\left[0 + \frac{1.25\left|-263\right|(12) + 1.50\left|-216\right|(12)}{1,107} + \frac{M_{AD}}{1,107}\right]$$

$M_{AD} = 58,587$ kip-in. = 4,882 kip-ft

$M_{yt} = (1.25)(263) + (1.50)(216) + 4,882 = 5,535$ kip-ft

Therefore, the top flange yields before the longitudinal reinforcement, and $M_{yt} = 5,375$ kip-ft

For the whole section, the compression flange governs, thus $M_y = M_{yc} = 4,397$ kip-ft

8.2 Exterior Beam Check: Section 2

8.2.1 Strength Limit State (Article 6.10.6)

8.2.1.1 Flexural Resistance (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 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 use of Appendix A6 is strongly recommended for bridges utilizing rolled-beam sections. 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 (i.e, the beam in this case) must be less than or equal to 70 ksi.

$F_y = 50 \text{ ksi} < 70 \text{ ksi}$ (satisfied)

Rolled-beam sections are not currently available in yield strengths exceeding 50 ksi.

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

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

where:

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

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

As computed above for the section consisting of the steel beam plus the longitudinal reinforcement, the elastic neutral axis is located 22.76 in. from the bottom of the beam (Table 11). Subtracting the bottom flange thickness gives the web depth in compression in the elastic range (D_c) as:

D_c = 22.76 – 1.42 = 21.34 in.

$$\frac{2(21.34)}{0.75} = 56.91$$

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

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

$$a_{wc} = \frac{2(21.34)(0.75)}{15.8(1.42)} = 1.43$$

$$111 < \lambda_{rw} = \left(3.1 + \frac{5.0}{1.43}\right)\sqrt{\frac{29,000}{50}} = 158.9 > 137$$

$$\therefore \lambda_{rw} = 137 > \frac{2D_c}{t_w} = 56.91$$

(satisfied)

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

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

$$I_{yc} = I_{yt}$$

$1.0 > 0.3$ (satisfied)

Thus, Appendix A6 is applicable.

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

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} = 36.56 – 9.71 = 26.85 in.

The hybrid factor, R_h, is determined from Article 6.10.1.10.1, and is 1.0 for this example since the since the section is a homogeneous section. Therefore, $\lambda_{pw(Dcp)}$ is computed as follows:

$$\lambda_{pw(D_{cp})} = \frac{\sqrt{\dfrac{29000}{50}}}{\left(0.54\dfrac{67{,}847}{(1.0)(4{,}397)(12)} - 0.09\right)^2} = 65.94 < 137\left(\frac{26.85}{21.34}\right) = 172.4$$

The web slenderness classification is then determined as follows.

$$\frac{2D_{cp}}{t_w} = \frac{2(26.85)}{0.75} = 71.60 > \lambda_{pw(D_c)} = 65.94 \qquad \text{(noncompact)}$$

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:

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

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 $2D_c/t_w$

$$\lambda_{pw(D_c)} = \lambda_{pw(D_{cp})}\left(\frac{D_c}{D_{cp}}\right) \leq \lambda_{rw}$$ Eq. (A6.2.2-6)

$$\lambda_{pw(D_c)} = (65.94)\left(\frac{21.34}{26.85}\right) = 52.41 < \lambda_{rw} = 137$$

$$R_{pc} = \left[1 - \left(1 - \frac{(1.0)(4{,}397)(12)}{67{,}847}\right)\left(\frac{56.91 - 52.41}{137 - 52.41}\right)\right]\frac{67{,}847}{(4{,}397)(12)} \leq \frac{67{,}847}{(4{,}397)(12)}$$

$R_{pc} = 1.271 < 1.286$

$R_{pc} = 1.271$

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

$$R_{pt} = \left[1 - \left(1 - \frac{(1.0)(5{,}375)(12)}{67{,}847}\right)\left(\frac{56.91 - 52.41}{137 - 52.41}\right)\right]\frac{67{,}847}{(5{,}375)(12)} \leq \frac{67{,}847}{(5{,}375)(12)}$$

$R_{pt} = 1.049 < 1.052$

$R_{pt} = 1.049$

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 determined from Article A6.3.2 and the lateral torsional buckling resistance determined 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:

$$\lambda_f \leq \lambda_{pf}$$

where: $\lambda_f = \frac{b_{fc}}{2t_{fc}} = \frac{15.8}{2(1.42)} = 5.56$ Eq. (A6.3.2-3)

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_{yc}}} = 0.38\sqrt{\frac{29,000}{50}} = 9.15$$ Eq. (A6.3.2-4)

$$\lambda_f = 5.56 < \lambda_{pf} = 9.15$$ (satisfied)

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

$M_{nc} = R_{pc}M_{yc}$ = (1.271)(4,397) = 5,589 k-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.

$$L_b \leq L_p$$

where: L_b = (15.0)(12.0) = 180 in.

$$L_p = 1.0 r_t \sqrt{\frac{E}{F_{yc}}}$$ Eq. (A6.3.3-4)

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

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

$$r_t = \frac{15.8}{\sqrt{12\left(1 + \frac{1}{3}\frac{(21.34)(0.75)}{(15.8)(1.42)}\right)}} = 4.10 \text{ in.}$$

$$L_p = 1.0\left(4.10\right)\sqrt{\frac{29,000}{50}} = 98.74 \text{ in.}$$

Therefore, $L_b > L_p$. (noncompact)

Because the unbraced length does not satisfy the compact limit, the noncompact limit is next evaluated.

$$L_p < L_b \leq L_r$$

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.)

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

where: 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

$$F_{yr} = \min\left(0.7F_{yc}, R_h F_{yt}\frac{S_{xt}}{S_{xc}}, F_{yw}\right) > 0.5F_{yc}$$

$$S_{xt} = \frac{(5{,}375)(12)}{50} = 1{,}290 \text{ in.}^3$$

$$S_{xc} = \frac{(4{,}397)(12)}{50} = 1{,}055 \text{ in.}^3$$

$$F_{yr} = \min\left(0.7(50), (1.0)(50)\frac{1{,}290}{1{,}055}, 50\right) > 0.5F_{yc}$$

F_{yr} = min (35,61.1, 50) > 0.5(50)

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

J = St. Venant torsional constant

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

$J = (1/3)[(36.56)(0.75)^3 + (15.8)(1.42)^3(0.943) + (15.8)(1.42)^3 (0.943)] = 33.58 \text{ in.}^4$

h = depth between the centerline of the flanges

$$h = \frac{1.42}{2} + 36.56 + \frac{1.42}{2} = 37.98 \text{ in.}$$

$$L_r = 1.95(4.10)\frac{29{,}000}{35.0}\sqrt{\frac{33.58}{(1{,}055)(37.98)}}\sqrt{1+\sqrt{1+6.76\left(\frac{35.0(1{,}055)(37.98)}{(29{,}000)(33.58)}\right)^2}}$$

$$= 423.4 \text{ in.}$$

L_b = 180 in. < L_r = 423.4 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 (discussed in Article A6.3.3)

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

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

Otherwise:

M_1 = $2M_{mid} - M_2 \geq M_0$ Eq. (A6.3.3-12)

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 = 5,402 kip-ft

M_0 = 2,265 kip-ft

M_1 = M_0 = 2,265 kip-ft

$$C_b = 1.75-1.05\left(\frac{2,265}{5,402}\right)+0.3\left(\frac{2,265}{5,402}\right)^2 = 1.36 < 2.3$$

$$M_{nc} = (1.36)\left[1-\left(1-\frac{(35.0)(1,055)}{(1.271)(4,397)(12)}\right)\left(\frac{180-98.74}{423.4-98.74}\right)\right](1.271)(4,397)$$
$$\leq (1.271)(4,397)$$

M_{nc} = 6,746 kip-ft > 5,589 kip-ft

M_{nc} = 5,589 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} = 5,589 kip-ft

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

$\phi_f M_{nc}$ = (1.0)(5,589)$\phi_f M_{nc}$ = 5,589 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.

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

Therefore, the factored flexural resistance as governed by tension flange yielding is calculated as follows:

$\phi_f M_{nt} = \phi_f R_{pt} M_{yt}$ = (1.0)(1.049)(5,375) = 5,638 kip-ft

8.2.1.2 Factored Moment

The strength requirements specified by Appendix A6 are given in Section A6.1.1. Since the compression flange (i.e, the bottom 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 lateral bending stress due to the factored Strength I loads multiplied by the section modulus for the compression flange, see Eq. A6.1.1-1.

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

The tension flange at Section 2 (i.e., the top flange) is continuously braced by the concrete deck at the strength limit state, and must therefore satisfy the following, see Eq. A6.1.4-1. Since the flange is continuously braced, the flange lateral bending stresses are not considered.

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

At the Strength limit state, there are five load combinations to consider. The four load combinations applicable to the superstructure elements (i.e., the beams) in this design example are as follows (Strength II is not applicable):

Strength I = 1.25DC + 1.5DW + 1.75(LL+I)

Strength III = 1.25DC + 1.5DW + 1.0WS

Strength IV = 1.5(DC + DW)

Strength V = 1.25DC + 1.5DW + 1.35(LL+I) + 1.0WS

At the location of peak negative moment (e.g., at the pier), the unfactored DC and DW moments are given in Table 2.

DC = -1387 kip-ft + -263 = -1,650 kip-ft

DW = -216 kip-ft

From Table 3, the controlling LL+I moment is -1,723 kip-ft.

LL+I = -1,723 kip-ft

Calculate the factored moment, M_u, for each limit state load combination:

Strength I: M_u =1.25(-1,650) + 1.5(-216) + 1.75(-1,723) = -5,402 kip-ft

Strength III: M_u = 1.25(-1,650) + 1.5(-216) = -2,387 kip-ft

Strength IV: M_u = 1.5(-1,650 + 216) = -2,799 kip-ft

Strength V: M_u = 1.25(-1,650) + 1.5(-216) + 1.35(-1,723) = -4,713 kip-ft

In this example, lateral bending in the bottom flange due to wind-load effects is considered at the strength limit state. For simplicity in this example, the largest value of f_ℓ within the unbraced length will conservatively be used in all design checks. f_ℓ is to be taken as positive in sign. Eqs. C4.6.2.7.1-1 and C4.6.2.7.1-2 are used to compute the factored wind force per unit length, W, applied to the bottom flange, and the maximum flange lateral bending moment due to the factored wind load, M_w, within the unbraced length, respectively, as demonstrated below. The wind load acting on the live load (WL) is assumed transmitted directly to the deck and is therefore not considered in the Strength V load combination in this example. The overturning effect of WL on the wheel loads is also not considered.

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

$$L_b \leq 1.2L_p\sqrt{\frac{C_bR_b}{M_u / M_{yc}}}$$

Eq. (6.10.1.6-3)

M_u is the largest major-axis bending moment throughout the unbraced length causing compression in the flange under consideration. In this case, M_u = -2,387 kip-ft, as computed earlier for the Strength III load combination (which is the controlling load case with wind included for this computation). Since the web is noncompact, the web load-shedding factor, R_b, is equal to 1.0. Therefore:

$$1.2(4.10)\sqrt{\frac{1.36(1.0)}{|-2,387|/4,397}} = 7.79 \text{ ft} < L_b = 15.0 \text{ ft}$$

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

$$f_\ell = \left(\frac{0.85}{1-\frac{M_u}{F_{cr}S_{xc}}}\right)f_{\ell 1} \geq f_{\ell 1}$$

Eq. (6.10.1.6-5)

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 determined from Eq. A6.3.3-8 since this is a straight-girder bridge and the web is noncompact.

$$F_{cr} = \frac{C_b\pi^2E}{(L_b/r_t)^2}\sqrt{1+0.078\frac{J}{S_{xc}h}(L_b/r_t)^2}$$

Eq. (A6.3.3-8)

$$F_{cr} = \frac{1.36\pi^2(29,000)}{(180/4.10)^2}\sqrt{1+0.078\left(\frac{(33.58)}{(1,055)(37.98)}\right)(180/4.10)^2} = 214.3 \text{ ksi}$$

Note that the calculated value of F_{cr} for use in Eq. 6.10.1.6-5 is not limited to $R_bR_hF_{yc}$ (Article C6.10.1.6).

The amplification factor is then determined as follows:

For Strength III:

$$AF = \frac{0.85}{\left(1 - \frac{|-2,387(12)|}{214.3(1,055)}\right)} = 0.97 < 1.0 \therefore AF = 1.0$$

For Strength V:

$$AF = \frac{0.85}{\left(1 - \frac{|-4,713(12)|}{214.3(1,055)}\right)} = 1.13 > 1.0$$

The horizontal pressure applied by the wind load loads, P_Z, was previously determined to be 0.031 ksf for the Strength III load combination and 0.021 ksf for the Strength V load combination (Section 4.3). It is assumed in this example that this pressure acts normal to the structure. The procedure given in Article C4.6.2.7.1 is then used to determine the force effects caused by the wind loading.

At the strength limit state, it may be assumed that the wind pressure acting on the parapets, deck, and top half of the beam is resisted by diaphragm action of the deck for members with cast-in-place concrete or orthotropic steel decks. The beam must then only resist the wind pressure on the bottom half of the beam. This force is expressed by Eq. C4.6.2.7.1-1.

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

where: η = 1.0

P_D = design horizontal wind pressure specified in Article 3.8.1 = P_Z

d = beam depth = 39.4 in. = 3.28 ft

γ = load factor (= 1.0 for WS)

For Strength III:

$$W = \frac{(1.0)(1.0)(0.031)(3.28)}{2} = 0.051 \text{ kips/ft}$$

For Strength V:

$$W = \frac{(1.0)(1.0)(0.021)(3.28)}{2} = 0.034 \text{ kips/ft}$$ The maximum flange lateral bending moment is then computed as follows:

For Strength III:

$$M_w = \frac{WL_b^2}{10} = \frac{(0.051)(15.0)^2}{10} = 1.15 \text{ kip} - \text{ft} \qquad \text{Eq. (C4.6.2.7.1-2)}$$

For Strength V:

$$M_w = \frac{WL_b^2}{10} = \frac{(0.034)(15.0)^2}{10} = 0.765 \text{ kip} - \text{ft} \qquad \text{Eq. (C4.6.2.7.1-2)}$$

Lateral bending stresses due to the wind loading are then determined by dividing M_w by the section modulus of the bottom flange as follows:

For Strength III:

$$f_\ell = \frac{M_w}{S_\ell} = \frac{(1.15)12}{(15.8)^2(1.42)/6} = 0.23 \text{ ksi} * AF = 0.23(1.0) = 0.23 \text{ ksi}$$

For Strength V:

$$f_\ell = \frac{M_w}{S_\ell} = \frac{(0.765)12}{(15.8)^2(1.42)/6} = 0.15 \text{ ksi} * AF = 0.15(1.13) = 0.17 \text{ ksi}$$

As specified in Article 6.10.1.6, the flange lateral bending stresses must not exceed 60 percent of the flange yield strength (after amplification). Thus, for this example f_ℓ must be less than or equal to 30 ksi, which is satisfied for both the Strength III and Strength V load combinations.

The controlling strength limit state can now be determined based on the above information.

Strength I (wind loads not considered):

$M_u + \frac{1}{3} f_\ell S_{xc}$ = 5,402 + 0 = 5,402 kip-ft < $\phi_f M_{nc}$ = 5,589 kip-ft (satisfied - governs)

M_u = 5,402 kip-ft < $\phi_f M_{nt}$ = 5,638 kip-ft (satisfied)

Strength III (wind loads considered):

$$M_u + \frac{1}{3} f_\ell S_{xc} = 2{,}387 + (1/3)(0.23)(1{,}055)(1/12) = 2{,}394 \text{ kip-ft} < \phi_f M_{nc} = 5{,}589 \text{ kip-ft}$$

(satisfied)

M_u = 2,387 kip-ft < $\phi_f M_{nt}$ = 5,638 kip-ft (satisfied)

Strength IV (wind loads not considered):

$$M_u + \frac{1}{3} f_\ell S_{xc} = 2{,}799 + 0 = 2{,}799 \text{ kip-ft} < \phi_f M_{nc} = 5{,}589 \text{ kip-ft}$$ (satisfied)

M_u = 2,799 kip-ft < $\phi_f M_{nt}$ = 5,638 kip-ft (satisfied)

Strength V (wind loads considered):

$$M_u + \frac{1}{3} f_\ell S_{xc} = 4{,}713 + (1/3)(0.17)(1{,}055)(1/12) = 4{,}718 \text{ kip-ft} < \phi_f M_{nc} = 5{,}589 \text{ kip-ft}$$

(satisfied)

M_u = 4,713 kip-ft < $\phi_f M_{nt}$ = 5,638 kip-ft (satisfied)

Although wind loads are considered in the strength limit state checks in this design example, the Strength III and Strength V load combinations including wind load effects rarely control for the bridge in its final constructed condition. Wind load effects are of greater concern during construction.

8.2.1.3 Shear (6.10.6.3)

The shear resistance of the negative bending region is governed by Article 6.10.9.2 because the beam is comprised of an unstiffened web, i.e., no transverse stiffeners are provided. The shear resistance of the section is calculated as follows:

$$V_u \le \phi_v V_{cr}$$

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 beam. If the web slenderness satisfies the following equation, C is equal to 1.0.

$$\frac{D}{t_w} \leq 1.12\sqrt{\frac{Ek}{F_{yw}}} = \frac{36.56}{0.75} = 48.75 < 1.12\sqrt{\frac{(29,000)(5)}{50}} = 60.31 \qquad \text{(satisfied)}$$

$$C = 1.00$$

The shear buckling resistance is then computed as follows.

$$V_{cr} = CV_p = (1.00)(0.58)(50)(36.56)(0.75) \qquad \text{Eq. (6.10.9.2-1)}$$

$$V_{cr} = 795 \text{ kips}$$

The factored shear at the pier at the strength limit state is given in Table 8 as -339 kips. Thus:

$$V_u = |-339| \text{ kips} < \phi_v V_{cr} = (1.0)(795) = 795 \text{ kips} \qquad \text{(satisfied)}$$

8.2.2 Constructability (Article 6.10.3)

Article 2.5.3 requires the Engineer 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 rely 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 beams are considered to be noncomposite during the initial construction phase. The influence of various segments of the beam 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 beams 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 DC1 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. The unstiffened shear resistance of the beam was previously demonstrated to be sufficient to resist the factored shear at the strength limit state. Therefore, the section will have sufficient shear resistance for the constuctibility check.

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

8.2.3 Service Limit State (Article 6.10.4)

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

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 the following:

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

Because the bottom flange is discretely braced, 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:

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

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 beams such as the case in the present design example, the flange lateral bending stresses are taken equal to zero.

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 DC2, DW and LL+IM are assumed to cause stress in the concrete deck.

$$f_r = 0.24\sqrt{f'_c} = 0.24\sqrt{4.0} = 0.48 \text{ ksi}$$

$$f_{deck} = \frac{1.0\left[1.0\left|-263\right| + 1.0\left|-216\right| + 1.3\left|-1,723\right|\right](14.17)(12)}{45,325(8)} = 1.27 \text{ ksi} > 2f_r$$

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

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 Service II stress in the bottom (compression) flange is computed as:

$$f_f = \frac{(1.0)(-1,387)(12)}{993} + \frac{(1.0)(-263 + -216)(12)}{1,100} + \frac{1.3(-1,723)(12)}{1,100} = -46.42 \text{ ksi}$$

Comparing the calculated stress to the permissible stress given by Equation 6.10.4.2.2-2 (note that the Service II flange lateral bending stress, f_ℓ, is equal to zero in this case):

$f_f = |-46.42| \text{ ksi} < 0.95R_hF_{yf} = 0.95(1.0)(50) = 47.50 \text{ ksi}$ (satisfied)

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

$$f_f = \frac{(1.0)\left|-1,387\right|(12)}{993} + \frac{(1.0)\left|-263 + -216\right|(12)}{1,504} + \frac{1.3\left|-1,723\right|(12)}{1,504} = 38.45 \text{ ksi}$$

Comparing the calculated stress to the permissible stress given by Eq. 6.10.4.2.2-1:

$f_f = 38.45 \text{ ksi} < 0.95R_hF_{yf} = 0.95(1.0)(50) = 47.5 \text{ ksi}$ (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.

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

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.

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

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.

$D_c = 22.76 - 1.42 = 21.34$ in.

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

$$k = \frac{9}{(21.34/36.56)^2} = 26.42$$

$$F_{crw} = \frac{0.9(29{,}000)(26.42)}{\left(\frac{36.56}{0.75}\right)^2} = 290.2 \text{ ksi} > R_h F_{yc} = (1.0)(50.0) = 50.0 \text{ ksi}$$

$\therefore F_{crw} = 50$ ksi

Eq. 6.10.4.2.2-4 is satisfied as shown below:

$f_c = |-46.42|$ ksi $< F_{crw} = 50.0$ ksi (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), which includes provisions for load-induced fatigue and distortion-induced fatigue, 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 6.6.2.1 are essentially material requirements. 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 taken 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 to the flanges of the beam at the cross-frame located 15 feet from the pier 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 stress range resulting from 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 nearly 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.

At the bottom of the top flange the factored stress range is computed as follows:

$$\gamma(\Delta f) = (0.80)\left[\frac{(198)(12)(5.59 - 1.42)}{45,325} + \frac{|-295|(12)(5.59 - 1.42)}{45,325} \right]$$

$\gamma(\Delta f) = 0.44 \text{ ksi} \leq (\Delta F)_n = 5.5 \text{ ksi}$ (satisfied)

At the top of the bottom flange:

$$f_{DC1} = \frac{(-396)(12)(19.70-1.42)}{19,600} = -4.43\ \text{ksi}$$

$$f_{DC2} = \frac{(-75)(12)(27.38-1.42)}{33,485} = -0.70\ \text{ksi}$$

Σ = -4.43 + -0.70 = -5.13 ksi

$$f_{LL+IM} = \frac{1.75(198)(12)(33.81-1.42)}{45,325} = 2.97\ \text{ksi}$$

$$\left|-5.13\ \text{ksi}\right| > 2.97\ \text{ksi} \quad \therefore \text{ fatigue does not need to be checked}$$

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.

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

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 Beam Check: Section 1

8.3.1 Strength Limit State

8.3.1.1 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.

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

8.3.1.1.1 Flexural Resistance (6.10.7.1)

To calculate the flexural resistance at the strength limit state, the classification of the section must first be determined. The following requirements must be satisfied for a section in positive bending to qualify as compact:

F_y = 50 ksi < 70 ksi (satisfied)

$$\frac{D}{t_w} = \frac{36.56}{0.75} = 48.75 < 150 \qquad \text{(satisfied)}$$

$$\frac{2D_{cp}}{t_w} = \frac{2(0)}{0.75} = 0 \le 3.76\sqrt{\frac{E}{F_{yc}}} = 3.76\sqrt{\frac{29,000}{50}} = 90.55 \qquad \text{Eq (6.10.6.2.2-1) (satisfied)}$$

Therefore, the section is compact, and the nominal flexural resistance is based on Article 6.10.7.1.2, where the flexural resistance of beams satisfying $D_p \le 0.1D_t$ is given by Eq. 6.10.7.1.2-1 and by Eq. 6.10.7.1.2-2 for those beams violating this limit.

$$M_n = M_p \qquad \text{Eq. (6.10.7.1.2-1)}$$

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

D_p is the distance from the top of the concrete deck to the neutral axis of the composite section at the plastic moment and is computed as follows. The plastic neutral axis was determined previously to be located 0.53 in. from the top of the top flange. Therefore:

D_p = 8.0 + 2.0 – 1.42 + 0.53 = 9.11 in.

The total depth of the composite beam, D_t, is equal to the following:

D_t = 8.0 + 2.0 + 36.56 + 1.42 = 47.98 in.

D_p = 9.11 > 0.1D_t = 0.1(47.98) = 4.98 (not satisfied)

Therefore, the nominal flexural resistance is determined using Equation 6.10.7.1.2-2 as follows:

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

$$M_n = 6,975\left(1.07 - 0.7\frac{9.11}{47.98}\right) = 6,536 \text{ kip-ft}$$

From separate calculations, the unbraced length adjacent to the interior pier does not satisfy the compression-flange bracing requirement given by Eq. B6.2.4-1, therefore, M_n is limited to $1.3R_hM_y$ according to Eq. 6.10.7.1.2-3 in this case as follows, where My was computed previously in Section 8.1.4:

$$M_n \leq 1.3R_hM_y = 1.3(1.0)(5,210) = 6,773\ \text{kip}-\text{ft} > M_n = 6,536\ \text{kip}-\text{ft} \qquad \text{Eq. (6.10.7.1.2-3)}$$

$$\therefore\ M_n = 6,536\ \text{kip}-\text{ft}$$

The factored flexural resistance is determined as:

$$\phi_f M_n = (1.0)(6,536) = 6,536\ \text{kip}-\text{ft}$$

8.3.1.1.2 Factored Positive Bending Moment

In order to determine if the factored flexural resistance of 6,536 kip-ft is adequate, the maximum value of ($M_u + f_\ell S_{xt}/3$) must be determined according to Eq. 6.10.7.1.1-2. Therefore, the value of ($M_u + f_\ell S_{xt}/3$) resulting from each of the four strength load combinations applicable to this design example is computed (Strength II is not applicable). As previously discussed during the evaluation of the negative bending region of the beam, the load factors for each of the applicable load combinations are as follows:

Strength I = 1.25DC + 1.5DW + 1.75(LL+I)

Strength III = 1.25DC + 1.5DW + 1.0WS

Strength IV = 1.5(DC + DW)

Strength V = 1.25DC + 1.5DW + 1.35(LL+I) + 1.0WS

The location of the maximum positive moment is at 36 ft from the abutments. The DC and DW moments at this location are given in Table 2 and are equal to the following:

DC = 780 +147 = 927 kip-ft

DW = 121 kip-ft

From Table 3, the controlling LL+I moment is 1,664 kip-ft.

LL+I = 1,664 k-ft

Calculate the factored moment, M_u, for each limit state load combination:

Strength I: M_u =1.25(927) + 1.5(121) + 1.75(1,664) = 4,252 kip-ft

Strength III: M_u = 1.25(927) + 1.5(121) = 1,340 kip-ft

Strength IV: M_u = 1.5(927 + 121) = 1,572 kip-ft

Strength V: M_u = 1.25(927) + 1.5(121) + 1.35(1,664) = 3,587 kip-ft

The section modulus, Sxt, is determined as:

$$S_{xt} = \frac{M_{yt}}{F_{yt}} = \frac{5,210(12)}{50} = 1,250 \text{ in.}^3$$

From calculations similar to those illustrated previously for the negative bending region, the maximum factored flange lateral bending moments due to wind loading at the strength limit state are computed as follows:

For Strength III:

$$M_w = \frac{WL_b^2}{10} = \frac{(0.051)(30.0)^2}{10} = 4.59 \text{ kip} - \text{ft} \qquad \text{Eq. (C4.6.2.7.1-2)}$$

For Strength V:

$$M_w = \frac{WL_b^2}{10} = \frac{(0.034)(30.0)^2}{10} = 3.06 \text{ kip} - \text{ft} \qquad \text{Eq. (C4.6.2.7.1-2)}$$

The factored flange lateral bending stresses due to the wind loading are then determined by dividing M_W by the section modulus of the bottom flange as follows (flange lateral bending stresses in the top flange are not considered since the top flange is continuously braced by the concrete deck):

For Strength III:

$$f_\ell = \frac{M_w}{S_\ell} = \frac{(4.59)12}{(15.8)^2(1.42)/6} = 0.93 \text{ ksi} * AF = 0.93(1.0) = 0.93 \text{ ksi}$$

For Strength V:

$$f_\ell = \frac{M_w}{S_\ell} = \frac{(3.06)12}{(15.8)^2(1.42)/6} = 0.62 \text{ ksi} * AF = 0.62(1.0) = 0.62 \text{ ksi}$$

Consideration should also be given to increasing the first-order flange lateral bending stresses to account for second-order force effects, as specified in Article 6.10.1.6, through application of an amplification factor. However, no amplification is required for tension flanges (i.e., AF = 1.0).

As specified in Article 6.10.1.6, the flange lateral bending stresses must not exceed 60 percent of the flange yield strength (after amplification). Thus, for this example f_ℓ must be less than or equal to 30 ksi, which is satisfied for both the Strength III and Strength V load combinations.

The controlling strength limit state can now be determined based on the above information.

Strength I (wind loads not considered):

$$M_u + \frac{1}{3} f_\ell S_{xt} = 4{,}252 + 0 = 4{,}252 \text{ kip-ft} < \phi_f M_n = 6{,}536 \text{ kip-ft} \qquad \text{(satisfied - governs)}$$

Strength III (wind loads considered):

$$M_u + \frac{1}{3} f_\ell S_{xt} = 1{,}340 + (1/3)(0.93)(1{,}250)(1/12) = 1{,}372 \text{ kip-ft} < \phi_f M_n = 6{,}536 \text{ kip-ft}$$

(satisfied)

Strength IV (wind loads not considered):

$$M_u + \frac{1}{3} f_\ell S_{xt} = 1{,}572 + 0 = 1{,}572 \text{ kip-ft} < \phi_f M_n = 6{,}536 \text{ kip-ft} \qquad \text{(satisfied)}$$

Strength V (wind loads considered):

$$M_u + \frac{1}{3} f_\ell S_{xt} = 3{,}587 + (1/3)(0.62)(1{,}250)(1/12) = 3{,}609 \text{ kip-ft} < \phi_f M_n = 6{,}536 \text{ kip-ft}$$

(satisfied)

8.3.1.1.3 Ductility Requirement

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.

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

$$D_p = 9.11 \text{ in.} < 0.42(47.98) = 20.15 \text{ in.} \qquad \text{(satisfied)}$$

8.3.1.2 Shear (Article 6.10.3.3)

The shear requirements at the strength limit state were previously shown to be satisfied.

8.3.2 Constructability (Article 6.10.3)

8.3.2.1 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. Wind load during the deck casting is not investigated in this design example (refer to NSBA's *Steel Bridge Design Handbook Design: Example 1* [4], for an illustration of wind-load checks during the deck casting).

Figure 11 depicts the deck placement sequence assumed in this design example. Note that for simplicity in this illustration, the sequence assumes that the concrete is placed in the two end spans at approximately the same time. Typically, it is more desirable to cast the two placements in the end spans in sequence. A check is not made for uplift should the placement in one end span be completed before the placement in the other end span has started. This situation could occur if the contractor elected to place the entire bridge deck, end-to-end, in one continuous placement.

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* [4] for further discussion on the deck placement analysis.

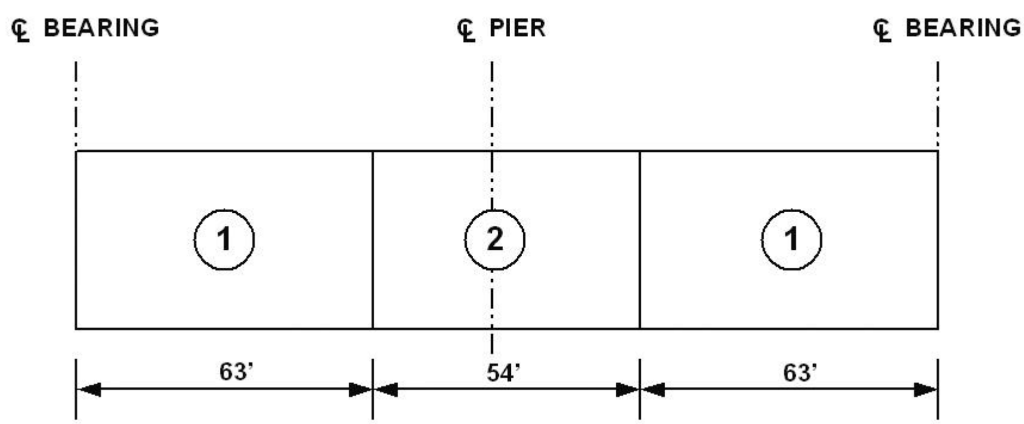

Figure 11 Deck Placement Sequence

The results of the deck placement analysis are shown in Table 12 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 780 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 12 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	65	110	135	140	125	90	35	-39	-134	-248
SIP Forms	0	27	45	56	58	53	40	19	-2	-45	-87
Cast 1	0	262	443	541	557	491	342	112	-150	-411	-672
Cast 2	0	297	511	643	693	661	547	350	84	-259	-678
Σ Cast 1	0	354	598	732	**755**	669	472	166	-191	-590	-1007
Σ Cast 2	0	389	666	834	891	839	677	404	43	-438	-1013
DC1	0	362	612	752	**780**	697	503	197	-220	-748	-1387

Because the shear requirements during construction are automatically satisfied for beams with unstiffened webs, only the evaluation of the flexural requirements is presented herein.

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.

Because the section modulus with respect to the top flange is the same as the section modulus with respect to the bottom flange at this phase of construction, f_{bu} is the same for both flanges and is equal to the following:

For Strength I:

$$f_{bu} = \frac{1.0(1.25)(780)(12)}{993} = \pm 11.78 \text{ ksi}$$

For the Special Load Combination (Article 3.4.1.2):

$$f_{bu} = \frac{1.0(1.4)(780)(12)}{993} = \pm 13.20 \text{ ksi}$$

8.3.2.1.1 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 12. Typically, the brackets are spaced between 3 and 4 feet, and given their relatively close proximity a typical assumption is 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.

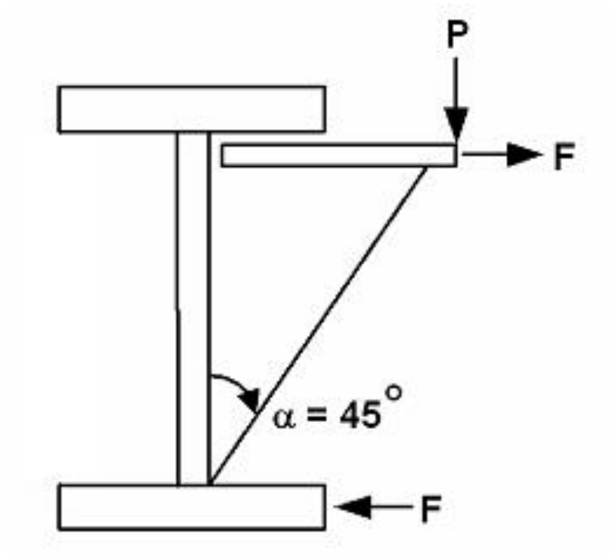

Figure 12 Deck Overhang Bracket Loads

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

$$P = 0.5(150)\left[\frac{8.5}{12}(3.5)+\left[\frac{1}{12}\left(\frac{2.0}{2}\right)\left(3.5-\frac{15.8/2}{12}\right)\right]+\frac{(2.0-1.42)}{12}\left(\frac{15.8/2}{12}\right)\right] = 206 \text{ lbs/ft}$$

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 represent only the portion of these loads that are assumed to be applied to 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 beam section due to the overhang loading is computed as follows:

$$F = P \tan\alpha$$

where: $\alpha = 45$ degrees (for this beam depth and OH length, varies for other situations)

$F = P \tan 45$

$F = P$

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:

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

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

L_b = unbraced length of the section under consideration = 30.0 ft (at the location of maximum positive bending)

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

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

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

For simplicity, the largest values of f_ℓ within the unbraced length will be used in the design checks, i.e., the maximum value of f_ℓ within the unbraced length is conservatively assumed to be the stress level throughout the unbraced length.

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.

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

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:

$$D_c = 19.70 - 1.42 = 18.28 \text{ in.}$$

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\right)}} = \frac{15.8}{\sqrt{12\left(1+\frac{1}{3}\frac{(18.28)(0.75)}{15.8(1.42)}\right)}} = 4.16 \text{ in.}$$

$$L_p = 1.0 r_t \sqrt{\frac{E}{F_{yc}}} = 1.0(4.16)\sqrt{\frac{29,000}{50}} = 100.2 \text{ in.} \qquad \text{Eq. (6.10.8.2.3-4)}$$

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

$$L_b = 360 \text{ in.} > 1.2(100.2)\sqrt{\frac{(1.0)(1.0)}{|-11.78|/50}} = 247.7 \text{ in.}$$

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:

$$F_{cr} = \frac{C_b R_b \pi^2 E}{\left(\frac{L_b}{r_t}\right)^2}$$

Eq. (6.10.8.2.3-8)

$$F_{cr} = \frac{1.0(1.0)\pi^2(29,000)}{\left(\frac{30.0(12)}{4.16}\right)^2} = 38.22 \text{ ksi}$$

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

The amplification factor is then determined as follows:

For Strength I:

$$AF = \frac{0.85}{\left(1 - \frac{|-11.78|}{38.22}\right)} = 1.23 > 1.0 \quad \text{ok}$$

For the Special Load Combination specified in Article 3.4.2.1:

$$AF = \frac{0.85}{\left(1 - \frac{|-13.20|}{38.22}\right)} = 1.30 > 1.0 \quad \text{ok}$$

AF is taken equal to 1.0 for tension flanges.

8.3.2.1.2 Strength I

The 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(206) + 1.5(40 + 85 + 25 + 125)] = 670.0 lbs/ft

$F = F_\ell = P = 670.0$ lbs/ft

$$M_\ell = \frac{F_\ell L_b^2}{12} = \frac{(0.670)(30.0)^2}{12} = 50.25 \text{ kip} - \text{ft}$$

The flange lateral bending stresses due to the component dead load are then determined by dividing the lateral bending moment by the section moduli of the flanges, which in this case are equal for the top and bottom flanges.

$$f_\ell = \frac{M_\ell}{S_\ell} = \frac{50.25(12)}{1.42(15.8)^2/6} = 10.21\,\text{ksi}$$

Finishing machine load:

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

F = P_ℓ = P = 4,500 lbs

$$M_\ell = \frac{P_\ell L_b}{8} = \frac{(4.5)(30.0)}{8} = 16.88\,\text{kip} - \text{ft}$$

$$f_\ell = \frac{M_\ell}{S_\ell} = \frac{16.88(12)}{1.42(15.8)^2/6} = 3.43\,\text{ksi}$$

Total:

Top flange: f_ℓ = (10.21 + 3.43)(AF) = (10.21 + 3.43)(1.23) = 16.78 ksi

Bot. flange: f_ℓ = (10.21 + 3.43)(AF) = (10.21 + 3.43)(1.0) = 13.64 ksi

8.3.2.1.3 Special Load Combination (Article 3.4.2.1)

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

Dead loads:

$$P = \left[1.4\left(206 + 40 + 85 + 25 + 125\right)\right] = 673.4\,\text{lbs/ft}$$

$$F = F_\ell = P = 673.4\,\text{lbs/ft}$$

$$M_\ell = \frac{F_\ell L_b^2}{12} = \frac{(0.6734)(30.0)^2}{12} = 50.51\,\text{kip} - \text{ft}$$

$$f_\ell = \frac{M_\ell}{S_\ell} = \frac{50.51(12)}{1.42(15.8)^2/6} = 10.26\,\text{ksi}$$

Finishing machine load:

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

F = P_ℓ = P = 4,200 lbs.

$$M_\ell = \frac{P_\ell L_b}{8} = \frac{(4.2)(30.0)}{8} = 15.75 \text{ kip} - \text{ft}$$

$$f_\ell = \frac{M_\ell}{S_\ell} = \frac{15.75(12)}{1.42(15.8)^2/6} = 3.20 \text{ ksi}$$

Total:

Top flange: f_ℓ = (10.26 + 3.20)(AF) = (10.26 + 3.20)(1.30) = 17.50 ksi

Bot. flange: f_ℓ = (10.26 + 3.20)(AF) = (10.26 + 3.20)(1.0) = 13.46 ksi

According to Article 6.10.1.6, the 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, evaluation of Eq. 6.10.1.6-1 for the Strength I load combination is shown below.

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

Top flange: f_ℓ = 17.50 ksi < $0.6F_{yf}$ = 30 ksi (satisfied)

Bottom flange: f_ℓ = 13.64 ksi < $0.6F_{yf}$ = 30 ksi (satisfied)

8.3.2.2 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 web slenderness limit as follows:

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

where:

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

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

$$\frac{2D_c}{t_w} = \frac{2(18.28)}{0.75} = 48.75$$

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

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

$$a_{wc} = \frac{2(18.28)(0.75)}{15.8(1.42)} = 1.22$$

$$111 < \lambda_{rw} = \left(3.1 + \frac{5.0}{1.22}\right)\sqrt{\frac{29,000}{50}} = 173.4 > 137$$

$$\therefore \lambda_{rw} = 137 > \frac{2D_c}{t_w} = 48.75$$

(satisfied)

The web 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.

8.3.2.2.1 Compression Flange:

Flange tip yielding:

$f_{bu} + f_{\ell} \leq \phi_f R_h F_{yc}$ 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 the compression flange:

For Strength I:

$$|-11.78| + 16.78 \leq (1.0)(1.0)(50)$$

$$28.56 \text{ ksi} < 50 \text{ ksi}$$ (satisfied)

For the Special Load Combination (Article 3.4.2.1):

$$13.20 + 17.50 \leq (1.0)(1.0)(50)$$

$$30.70 \text{ ksi} < 50 \text{ ksi} \qquad \text{(satisfied)}$$

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:

$$\lambda_f = \frac{b_{fc}}{2t_{fc}}$$

Eq. (6.10.8.2.2-3)

$$\lambda_f = \frac{15.8}{2(1.42)} = 5.6$$

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

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

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

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

$$F_{nc} = R_b R_h F_{yc}$$

Eq. (6.10.8.2.2-1)

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:

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

For Strength I:

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

$$f_{bu} + \frac{1}{3} f_{\ell} = |-11.78| \text{ ksi} + \frac{16.78}{3} \text{ ksi} = 17.37 \text{ ksi}$$

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

$$17.37 \text{ ksi} < 50.00 \text{ ksi} \quad \text{(satisfied)}$$

For the Special Load Combination specified in Article 3.4.2.1:

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

$$f_{bu} + \frac{1}{3} f_{\ell} = |-13.20| \text{ ksi} + \frac{17.50}{3} \text{ ksi} = 19.03 \text{ ksi}$$

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

$$19.03 \text{ ksi} < 50.00 \text{ ksi} \quad \text{(satisfied)}$$

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 100.2 in. or 8.35 ft. The effective radius of gyration for lateral torsional buckling, r_t, for the noncomposite section was also computed earlier to be 4.16 inches.

Determine the limiting unbraced length, L_r:

$$L_r = \pi r_t \sqrt{\frac{E}{F_{yr}}}$$

Eq. (6.10.8.2.3-5)

where:

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

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

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

Therefore: $$L_r = \frac{\pi(4.16)}{12}\sqrt{\frac{29,000}{35.0}} = 31.35\text{ ft}$$

Since L_p = 8.35 feet < L_b = 30.0 feet < L_r = 31.35 feet,

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

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,

$$F_{nc} = 1.0\left[1-\left(1-\frac{35.0}{1.0(50)}\right)\left(\frac{30.0-8.35}{31.35-8.35}\right)\right](1.0)(1.0)(50) = 35.88\text{ ksi} < 1.0(1.0)(50) = 50\text{ ksi}$$

For Strength I:

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

$$f_{bu} + \frac{1}{3}f_\ell = \left|-11.78\right|\text{ ksi} + \frac{16.78}{3}\text{ ksi} = 17.37\text{ ksi}$$

$$\phi_f\left(F_{nc}\right)_{LTB} = 1.0(35.88) = 35.88\text{ ksi}$$

$17.37\text{ ksi} < 35.88\text{ ksi}$ (satisfied)

For the Special Load Combination specified in Article 3.4.2.1:

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

$$f_{bu} + \frac{1}{3} f_{\ell} = \left|-13.20\right| \text{ksi} + \frac{17.50}{3} \text{ ksi} = 19.03 \text{ ksi}$$

$$\phi_f \left(F_{nc}\right)_{LTB} = 1.0(35.88) = 35.88 \text{ ksi}$$

$$19.03 \text{ ksi} < 35.88 \text{ ksi} \quad \text{(satisfied)}$$

8.3.2.2.2 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:

$$11.78 + 13.64 \leq (1.0)(1.0)(50)$$

$$25.42 \text{ ksi} < 50 \text{ ksi} \qquad \text{(satisfied)}$$

For the Special Load Combination (Article 3.4.2.1):

$$13.20 + 13.46 \leq (1.0)(1.0)(50)$$

$$26.66 \text{ ksi} < 50 \text{ ksi} \qquad \text{(satisfied)}$$

8.3.3 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.

8.3.3.1 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 = 0.653 in. < L/800 = (90 x 12)/800 = 1.35 in. (satisfied)

8.3.3.2 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:

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

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 2 and 3.

$$f_f = \frac{1.0(780)(12)}{993} + \frac{(1.0)(147+121)(12)}{3{,}039} + \frac{1.3(1{,}664)(12)}{8{,}108} = -13.69 \text{ ksi}$$

$$f_f = \left|-13.69\right| \text{ ksi} < 0.95 R_h F_{yf} = 0.95(1.0)(50) = 47.50 \text{ ksi} \qquad \text{(satisfied)}$$

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:

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

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 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:

$$f_f = \frac{(1.0)(780)(12)}{993} + \frac{(1.0)(147+121)(12)}{1{,}223} + \frac{1.3(1{,}664)(12)}{1{,}341} = 31.41 \text{ ksi}$$

$$f_f = 31.41 \text{ ksi} < 0.95 R_h F_{yf} = 0.95(1.0)(50) = 47.50 \text{ ksi} \qquad \text{(satisfied)}$$

For composite sections in positive flexure, since the web satisfies the requirement of Article 6.10.2.1.1 (i.e., $D/t_w \leq 150$) 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.

8.3.4 Fatigue and Fracture Limit State (Article 6.10.5)

8.3.4.1 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 30 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:

$$f_{DC1} = \frac{(761)(12)(19.70 - 1.42)}{19{,}600} = -8.52 \text{ ksi}$$

$$f_{DC2} = \frac{(144)(12)(11.02 - 1.42)}{33{,}485} = -0.50 \text{ ksi}$$

Σ = -8.52 + -0.50 = -9.02 ksi

$$f_{LL+IM} = \frac{1.75\left|-118\right|(12)(5.59-1.42)}{45,325} = 0.23\text{ ksi}$$

$$\left|-9.02\text{ ksi}\right| > 0.23\text{ ksi} \quad \therefore \text{ fatigue does not need to be checked}$$

Top of Bottom Flange:

$$\gamma(\Delta f) = (0.80)\left[\frac{578(12)(33.81-1.42)}{45,325} + \frac{\left|-118\right|(12)(33.81-1.42)}{45,325}\right]$$

$\gamma(\Delta f) = 4.77\text{ ksi} \leq (\Delta F)_n = 5.5\text{ ksi}$ (satisfied)

8.3.4.2 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 design utilizes an unstiffened web. Therefore, this limit does not control and is not explicitly evaluated.

8.4 Deck Design

The following section will illustrate the design of the deck by the Empirical Deck Design Method specified in Article 9.7.2. This design process recognizes the strength gained by complex in-plane membrane forces forming an internal arching effect (see Commentary to Article 9.7.2.1). Caution should be exercised in the application of the Empirical Design Method to the design of the concrete deck for more complex bridges subject to larger differential deflections between the girders (refer to Article C9.7.2.4). The use of the Empirical Design Method should only be undertaken with the full knowledge and consent of the Owner.

To be able to use the Empirical Deck Design Method, certain design conditions must first be met, as specified in Article 9.7.2.4. It is also specified that four layers of minimum isotropic reinforcement are to be provided as specified in Article 9.7.2.5.

The Empirical Deck Design Method does not apply for the design of the deck overhang (see Article 9.7.2.2), which must be designed by traditional design methods. The design of the deck overhang is not illustrated in this example.

8.4.1 Effective Length (Article 9.7.2.3)

For the Empirical Design Method, the effective length is taken equal to the distance between the flange tips, plus the flange overhang, taken as the distance from the extreme flange tip to the face of the web disregarding any fillets. The effective slab length must not exceed 13.5 feet. Figure 13 illustrates the effective slab length.

$$L_{eff} = (10.0)(12.0) - 15.8 + \left(\frac{15.8 - 0.75}{2}\right) = 111.73 \text{ in.} < 162.0 \text{ in.}$$

(satisfied)

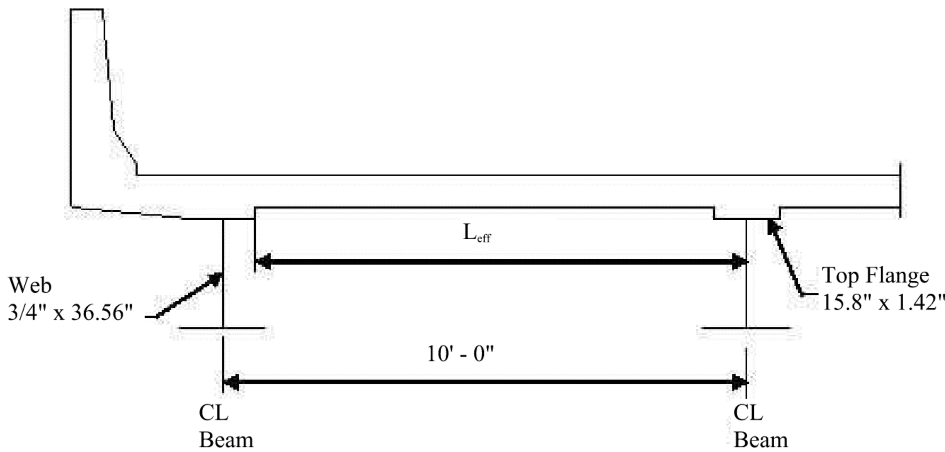

Figure 13 Effective Slab Length for Deck Design

8.4.2 Design Conditions (Article 9.7.2.4)

Specific design conditions must be met in order to use the Empirical Deck Design Method. The deck must be fully cast-in-place and water cured. The deck must also maintain a uniform cross section over the entire span, except in the locations of the haunches located at the beam flanges. Concrete used for the deck must have a specified 28-day compressive strength greater than or equal to 4.0 ksi. The supporting beams must be made of either steel or concrete, and the deck must be made composite with the beams. A minimum of two shear connectors at 24.0-inch centers must be provided in the negative moment regions of continuous steel superstructures. In addition, the following requirement must be satisfied:

$$6.0 \le \frac{L_{eff}}{t_s} \le 18.0$$

where: L_{eff} = effective slab length (Article 9.7.2.3)

t_s = structural slab thickness, which is the total thickness minus integral wearing surface (Article 9.7.2.6), and must be greater than 7 inches

$t_s = 8.0 \text{ in.} > 7.0 \text{ in.}$ (satisfied)

$$6.0 < \frac{111.73}{8.0} = 13.97 < 18.0$$ (satisfied)

The deck overhang beyond the centerline of the outside beam must be at least 5.0 times the depth of the slab.

$$(5.0)(8.0) = 40.0 \text{ in.} < 42.0 \text{ in.} \qquad \text{(satisfied)}$$

The core depth of the slab must not be less than 4.0 inches. An illustration of the core depth is shown in Figure 14.

Assuming a 2-inch cover on the top and a 1-inch cover on the bottom of the slab:

$$5.0 \text{ in.} > 4.0 \text{ in.} \qquad \text{(satisfied)}$$

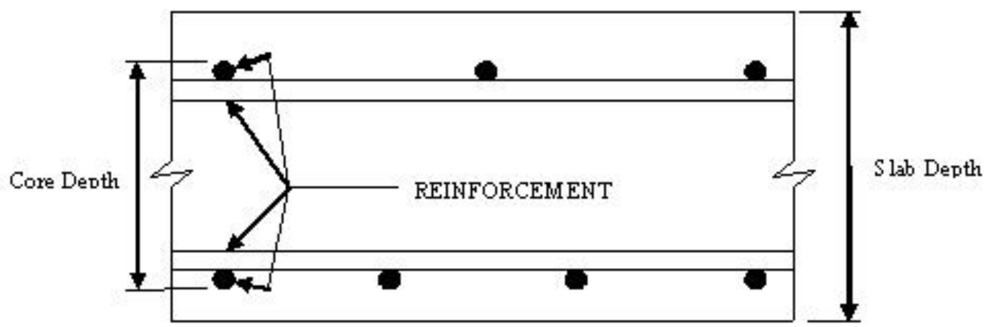

Figure 14 Core Depth of the Concrete Slab

8.4.3 Positive Flexure Reinforcement Requirements

Article 9.7.2.5 specifies that four layers of isotropic reinforcement be provided. The reinforcement is to be provided in each face of the slab, with the outermost layers placed in the direction of the effective length.

8.4.3.1 Top Layer (Longitudinal and Transverse)

The top layers are required to have a minimum reinforcement area of 0.18 in.^{2}/ft, with the maximum spacing permitted to be 18 inches.

Using No. 5 bars with a cross-sectional area of 0.31 in.^{2}, the required spacing is:

$$s = \frac{(0.31)(12)}{(0.18)} = 20.67 \text{ in.} > 18.0 \text{ in.} \ (\text{max.})$$

Use a 12-inch spacing to match that of the negative flexure region as determined below.

8.4.3.2 Bottom Layer (Longitudinal and Transverse)

Bottom layers of reinforcement are required to have a minimum reinforcement area of 0.27 in.^{2}/ft, with the maximum spacing permitted to be 18 inches.

Using No. 5 bars with a cross-sectional area of 0.31 in.^{2}, the required spacing is:

$$s = \frac{(0.31)(12)}{(0.27)} = 13.78\text{ in.} > 18.0\text{ in. } (\text{max.})$$

Therefore, use a 12-inch spacing in both the bottom layers to match that of the negative flexure region as determined below.

8.4.4 Negative Flexure Reinforcement Requirements

Article 6.10.1.7 states that in regions of negative flexure, the total cross-sectional area of the longitudinal reinforcement is not to be less than 1 percent of the total cross-sectional area of the concrete deck. The slab thickness is taken to be 8.0 inches; therefore, the minimum area of longitudinal reinforcement is:

Min. area of longitudinal reinforcement = (8.0)(0.01) = 0.08 in.^{2}/in.

The reinforcement used to satisfy this requirement is to have a specified minimum yield strength not less than 60 ksi and should have a size not exceeding No. 6 bars. The bars should be placed in two layers that are uniformly distributed across the deck width, with two thirds in the top layer and the remaining one third in the bottom layer. Bar spacing should not exceed 12.0 inches center-to-center.

8.4.4.1 Top Layer (Longitudinal)

$$\text{Minimum } A_{reinf} = \left(\frac{2}{3}\right)(0.08) = 0.05\text{ in.}^2/\text{in.}$$

Use No. 6 bars (A=0.44 in.^{2}) at 12-inch spacing with No. 5 bars (A = 0.31 in.^{2}) at 12-inch spacing:

$$A_{reinf} = \frac{0.44}{12} + \frac{0.31}{12} = 0.0625\text{ in.}^2/\text{in.} > 0.05\text{ in.}^2/\text{in.}$$

(satisfied)

8.4.4.2 Bottom Layer (Longitudinal)

$$\text{Minimum } A_{reinf} = \left(\frac{1}{3}\right)(0.08) = 0.03\text{ in.}^2/\text{in.}$$

Use No. 5 bars (A=0.31 in.^{2}) at 12-inch spacing with No. 4 bars (A = 0.20 in.^{2}) at 12-inch spacing:

$$A_{reinf} = \frac{0.31}{12} + \frac{0.20}{12} = 0.0425\text{ in.}^2/\text{in.} > 0.03\text{ in.}^2/\text{in.}$$

(satisfied)

8.4.4.3 Top and Bottom Layer (Transverse)

The transverse reinforcing steel in both the top and bottom layers will be No. 5 bars at 12- inch spacing (Figure 15); the same as in the positive flexure regions.

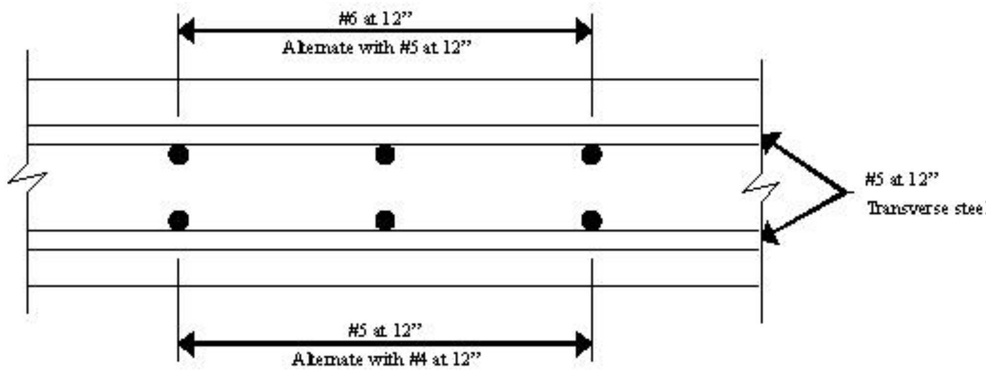

Figure 15 Deck Slab in Negative Flexure Region of the Beam

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