

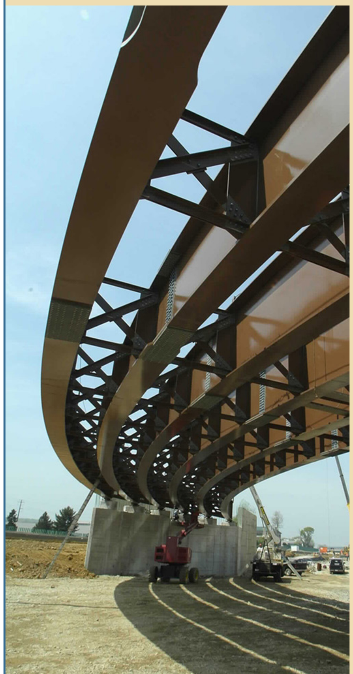


U.S. Department of Transportation

Federal Highway Administration

Steel Bridge Design Handbook

Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge

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16. Abstract

This design example presents an alternative design for that presented in the Steel Bridge Design Handbook Design Example 2A. Specifically, the design of a continuous steel wide-flange beam bridge is presented using a standard shape rolled I-beam, as an alternative to the plate girder design. The AASHTO LRFD Bridge Design Specifications are the governing specifications and all aspects of the provisions applicable to wide-flange beam bridge design (cross-section proportion limits, constructibility, serviceability, fatigue, and strength requirements) are considered. Furthermore, the optional moment redistribution specifications will be invoked. In addition to the beam design, the design of the concrete deck is also included. A basic wind analysis of the structure is also presented.

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Steel Bridge Design Handbook Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge

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FOREWORD

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

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

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

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

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

Myint Lwin

M. Myint Lwin, Director
Office of Bridge Technology

1.0 INTRODUCTION

This design example presents an alternative design for that presented in Design Example 2A. Specifically, the design of a continuous steel I-beam bridge is presented using a standard shape rolled I-beam, as an alternative to the preceding plate beam design. The Fifth Edition, with the 2010 Interims, of the *AASHTO LRFD Bridge Design Specifications* [1], referred to herein as *AASHTO LRFD (5th Edition, 2010)*, is the governing specifications and all aspects of the provisions applicable to I-beam design (cross-section proportion limits, constructibility, serviceability, fatigue, and strength requirements) are considered. Furthermore, the optional moment redistribution specifications will be invoked. In addition to the beam design, and the design of the concrete deck are also included. A basic wind analysis of the structure is also presented.

2.0 DESIGN PARAMETERS

The purpose of this example is to illustrate the design of a tangent two-span continuous bridge having equal spans of 90.0 feet with composite rolled beams using the moment redistribution method. The bridge cross-section (see Figure 1) has four rolled beams spaced at 10.0 feet with 3.5 foot overhangs providing for a 34.0 feet roadway width. The reinforced concrete deck is 8.5 inches thick, including a 0.5 inch integral wearing surface and a 2.0 inch haunch.

The framing plan for this design example (see Figure 2) has cross frames spaced at 30 feet near the abutments and 15 feet near the pier and are governed by constructability requirements in positive bending and moment redistribution requirements in negative bending.

ASTM A709, Grade 50W is used for all structural steel and the concrete is normal weight with a compressive strength of 4.0 ksi. The concrete slab is reinforced with reinforcing steel with 60 ksi yield strength.

The design specifications are the AASHTO LRFD Bridge Design Specifications, Fifth Edition, with the 2010 Interims. 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.

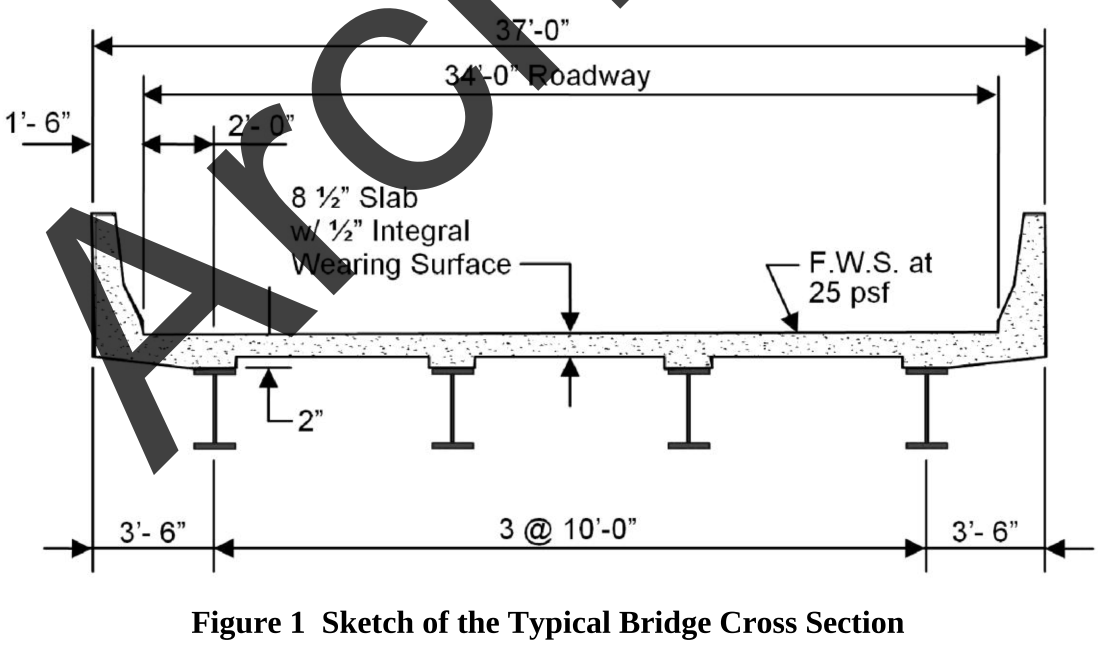

Figure 1 Sketch of the Typical Bridge Cross Section

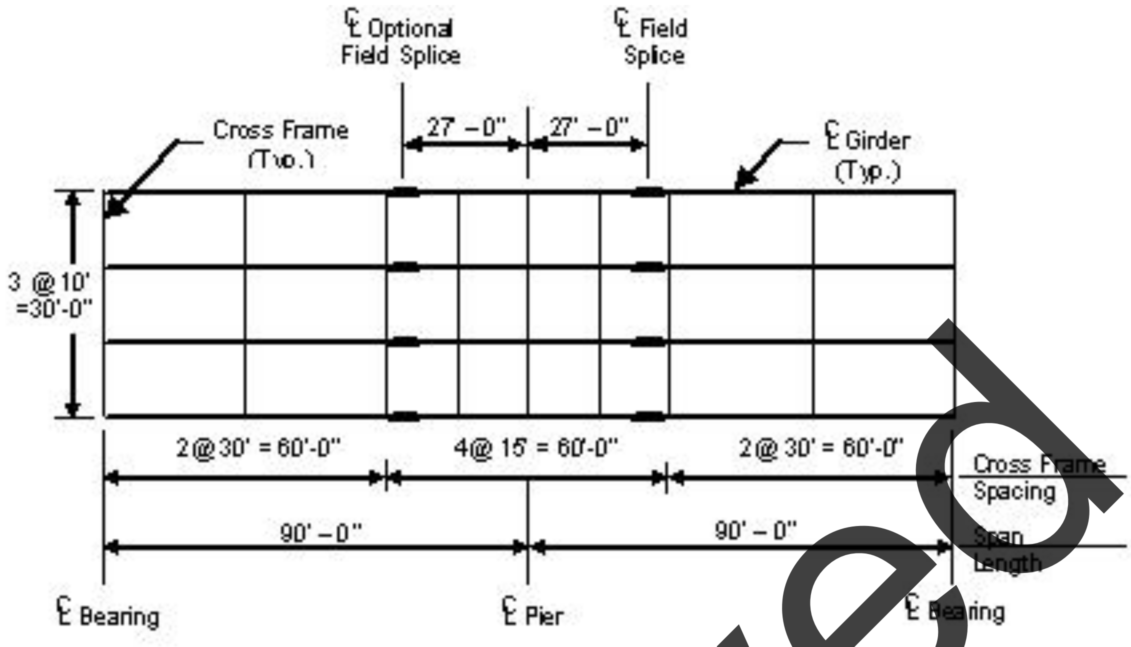

Figure 2 Sketch of the Superstructure Framing Plan

3.0 CROSS-SECTIONS

The beam elevation, shown in Figure 3, has section transitions at 30% of the span length from the interior pier. The design of the beam from the abutment to 63.0 feet in 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 I-beams from the standard shapes available, the selected cross-sections shown in Figure 3 were determined to be the most economical selections for this example.

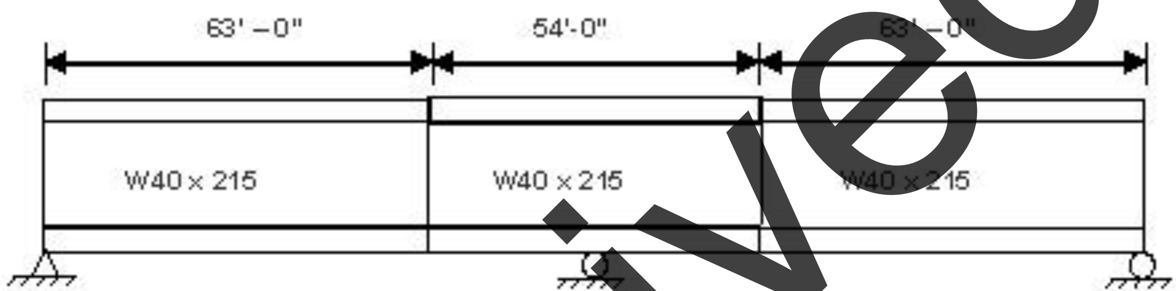

Figure 3 Sketch of the Beam Elevation

4.0 LOADS

This example considers all applicable loads acting on the super-structure 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 Steel Bridge Design Handbook Design Example 2A, the bridge dead loads are classified into three 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 And Attachment Dead Load (DC)

As discussed in the previous example, the component dead load is separated into two parts: dead loads acting on the non-composite section (DC1) and dead loads acting on the long term composite section (DC2). DC1 is assumed to be carried by the steel section alone. DC2 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.

DC1 includes the beam self weight, weight of concrete slab (including the haunch and overhang taper), 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 0.015 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-beam type of analysis implemented herein. Thus, the total DC1 loads used in this design are as computed below.

$$\text{Slab} = (8.5/12) \times (37) \times (0.150)/4 = 0.983 \text{ k/ft}$$

$$\text{Haunch} = (2-1.22)(15.8)/144 \times 0.150 = 0.013 \text{ k/ft}$$

$$\text{Overhang taper} = 2 \times (1/2) \times (3.5-7.9/12) \times (2/12) \times 0.150/4 = 0.018 \text{ k/ft}$$

$$\text{Beam} = 0.215 \text{ k/ft}$$

Cross-frames and misc. steel details = 0.015 k/ft

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

Total DC1 = 1.342 k/ft

DC2 is composed of the weight from the barriers, medians, and sidewalks. No sidewalks or medians are present in this example and thus the DC2 weight is equal to the barrier weight alone. The parapet 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 DC2 may be equally distributed to all beams or, alternatively, a larger proportion of the concrete barriers may be applied to the exterior beam. In this example, the barrier weight is equally distributed to all beams, resulting in the DC2 loads computed below.

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

DC2 = 0.260 k/ft

4.1.2 Wearing Surface Dead Load (DW)

Similar to the DC2 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

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 in 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 the magnitude of 90% the axle weights) in addition to the lane loading. The trucks must have a minimum headway of 50 feet between the two loads. The live load moments between the points of dead load contraflexure are to be taken as the larger of the HL-93 loading or the special negative loading.

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

The dynamic load allowance, which accounts for the dynamic effects of force amplification, is only applied to the truck portion of the live loading, and not 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 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.

4.3 Wind Loads

Article 3.8.1.2 discusses the design horizontal wind pressure, P_D, which is used to determine the wind load on the structure. The wind pressure is computed as follows:

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

where: P_B = base wind pressure of 0.050 ksf for beams (Table 3.8.1.2.1-1)

V_{DZ} = design wind velocity at design elevation, Z (mph)

In this example it is assumed the superstructure is less than 30 feet above the ground, at which the wind velocity is prescribed to equal 100 mph, which is designated as the base wind velocity, V_B. With V_{DZ} equal to the base wind velocity of 100 in Eq. 3.8.1.2.1-1 the horizontal wind pressure, P_D, is determined as follows.

$$P_D = 0.050 \frac{100^2}{10,000} = 0.050 \text{ ksf}$$

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. The subsequent sections discuss 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, wind load) are specified for each limit state.

$$\eta_D \eta_R \eta_I \Sigma \gamma_i Q_i \leq \varphi R_n R_r$$

where:

η_D = Ductility factor (Article 1.3.3)
η_R = Redundancy factor (Article 1.3.4)
η_I = Operational importance factor (Article 1.3.5)
γ_i = Load factor
Q_i = Force effect
φ = Resistance factor
R_n = Nominal resistance
R_r = Factored resistance

The factors relating to ductility and redundancy are related to the configuration of the structure, while the operational importance factor is related to the consequence of the bridge being out of service. The product of the three factors results in the load modifier, η, and is limited to the range between 0.95 and 1.00. In this example, the ductility, redundancy, and operational importance factors are each assigned a value equal to one. The load factors 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.

5.0 STRUCTURAL ANALYSIS

The *AASHTO LRFD (5th Edition, 2010)* allows the designer to use either approximate (e.g., line beam) 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, the line beam approach is employed to determine the beam moment and shear envelopes. Using the line beam 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 Article 4.6.2.2.1 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. Thus, for the present example, the multiple presence factors are only applicable when distribution factors are computed using the lever rule 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 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 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 of these requirements are satisfied, a refined analysis is not necessary and the computation of distribution factors using the approximate methods of Article 4.6.2.2 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 may be assumed to be equal to one for preliminary design. 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, the distribution factors are also uniform along the beam length. However, this is not always the case and separate calculations are typically required for the distribution factors 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 = 16,700 in.^{4} for the rolled beam
- A = area of the steel beam = 63.4 in.^{2} for the rolled beam
- e_g = distance between the centroid of the girder and centroid of the slab

The required section properties of the girder (in addition to other section properties that will be relevant for subsequent calculations) are determined as follows.

$$e_g = 19.50 + (2 - 1.22) + 4 = 24.28 \text{ in.}$$

$$K_g = n(I + Ae_g^2) = 8(16{,}700 + 63.4(24.28)^2) = 432{,}604 \text{ in.}^4$$

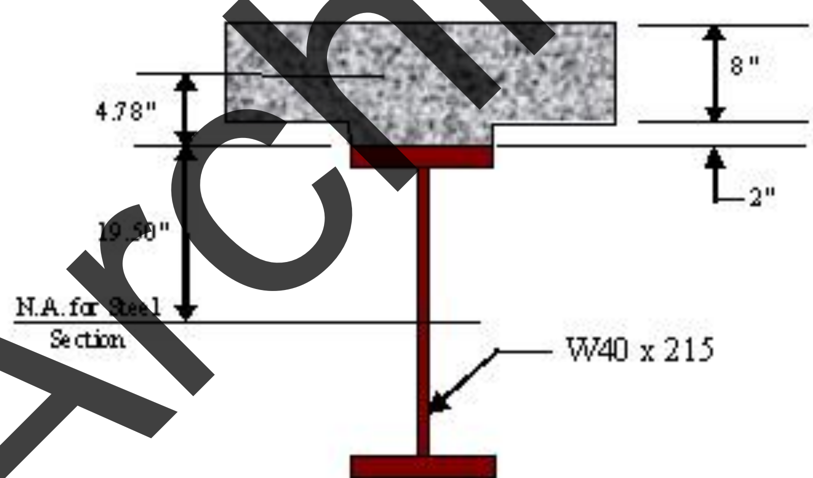

Figure 4 Rolled Beam Cross Section

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 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})

$$DF = 0.06 + \left(\frac{10}{14}\right)^{0.4}\left(\frac{10}{L}\right)^{0.3}\left(\frac{432,604}{12(90)(8.0)^3}\right)^{0.1} = 0.501 \text{ lanes}$$

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

$$DF = 0.075 + \left(\frac{10}{9.5}\right)^{0.6}\left(\frac{10}{L}\right)^{0.2}\left(\frac{432,604}{12(90)(8.0)^3}\right)^{0.1} = 0.723 \text{ lanes} \qquad \text{(governs)}$$

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.

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

$$DF = 0.36 + \frac{10}{25} = 0.760 \text{ lanes}$$

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

$$DF = 0.2 + \frac{10}{12} - \left(\frac{10}{35}\right)^2 = 0.952 \text{ lanes} \qquad \text{(governs)}$$

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) modification of the empirical equations for interior beams given above, (2) the lever rule, or (3) special analysis procedures.

5.2.2.1 Bending Moment

Lever Rule:
As specified in Table 4.6.2.2.2d-1, the lever rule is one method used to determine the distribution factor for the exterior beam. 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' 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 = 0.7 x 1.2 = 0.840 lanes

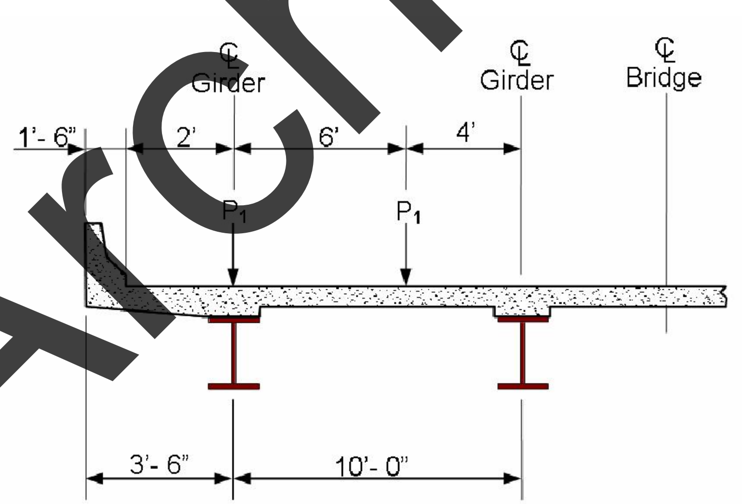

Figure 5 Sketch of the Truck Location for the Lever Rule

Modified of Interior Girder Distribution Factor:
Table 4.6.2.2.2d-1 gives modification factors that are to be multiplied by the interior beam distribution factors to determine the exterior beam distribution factors. These modification factors for moment are given by the following equation.

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

where d_e = the distance between the exterior beam and the interior of the barrier or curb (ft.)

d_e = 2

$$e = 0.77 + \left(\frac{2}{9.1}\right) = 0.990 < 1.0$$

Because e is less than one, it is obvious that the exterior beam distribution factor will not control when computed in this manner. However, multiplying the modification factor by the interior beam distribution factor for two lanes loaded (which controls compared to the distribution factor for one lane loaded) gives the following.

DF = 0.990(0.725) = 0.718 lanes

<u>Special Analysis:</u>
The special analysis assumes the entire bridge cross-section behaves as a rigid body 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, we precede with calculation of the distribution factors using the special analysis procedure, beginning with the calculations for one loaded lane.

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

$$DF = 1.2\left(\frac{1}{4} + \frac{(15)(12)}{2(15)^2 + 5^2}\right) = 0.732 \text{ lanes}$$

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

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

$$DF = 1.0\left(\frac{2}{4} + \frac{(15)(12+0)}{2((15)^2 + 5^2)}\right) = 0.860 \text{ lanes (governs)}$$

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, 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 0.723, 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.

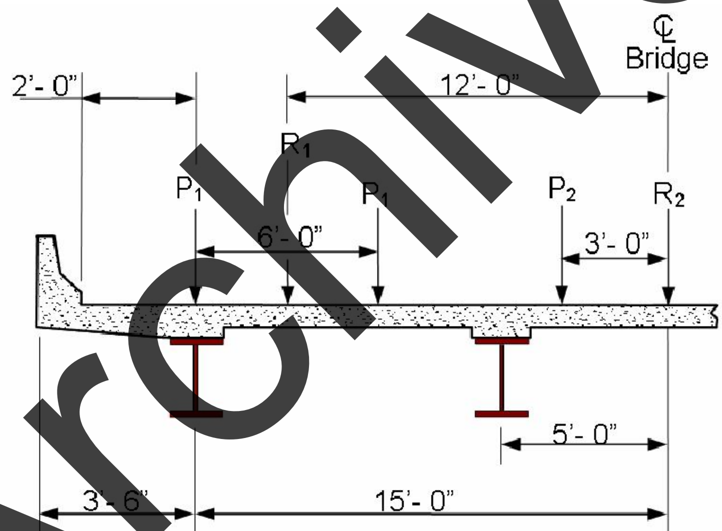

Figure 6 Sketch of the Truck Locations for Special Analysis

5.2.2.2 Shear

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

<u>Lever Rule:</u>
The above computations demonstrate that the distribution factor is equal to 0.840 lanes based on the lever rule.

DF = 0.840 lanes

Modified of Interior Girder Distribution Factor:
The shear modification factor is computed using the following formula.

$$e = 0.60 + \left(\frac{d_e}{9.1}\right) = 0.60 + \left(\frac{2}{9.1}\right) = 0.800$$

Applying this modification factor to the previously computed interior beam distribution factors for shear for one lane loaded and two or more lanes loaded, respectively, gives the following.

DF = 0.800(0.760) = 0.608 lanes

DF = 0.800(0.952) = 0.762 lanes

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

DF = 0.732 lanes

DF = 0.860 lanes (governs)

Thus, the controlling distribution factor for shear in the exterior beam is 0.860, which is less than that of the interior beam. Thus, the interior beam distribution factor of 0.952 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. Because the distribution factors calculated from empirical equations incorporate the multiple presence factors, the fatigue distribution factors are equal to the strength distribution factors divided by the multiple presence factor for one lane, as described subsequently.

5.2.3.1 Bending Moment

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

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

5.2.3.2 Shear

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

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

The following table summarizes the governing distribution factors, all of which, except for shear are controlled by the exterior beam.

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, an approximation of the live load deflection can be obtained by assuming that all beams deflect equally and applying the appropriate multiple presence factor. The controlling case occurs when two lanes are loaded, and the calculation of the corresponding distribution factor is shown below.

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

Table 1 Distribution Factors

	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 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 is based on the controlling distribution factors computed above. The envelopes shown are determined based on the section properties of the short-term composite section.

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 HL-93 loading and the special negative-moment loading, which is composed of 90 percent of both the truck-train moment and lane loading moment.

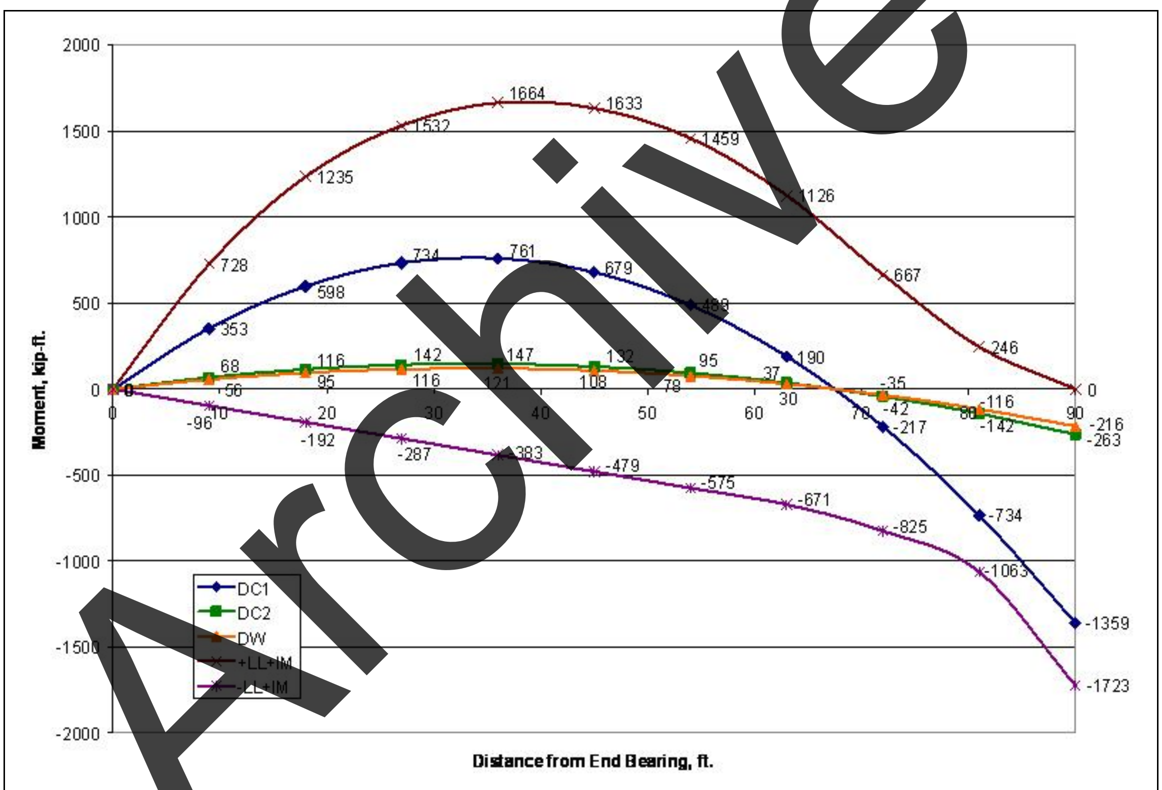

Figure 7 Dead and Live Load Moment Envelopes

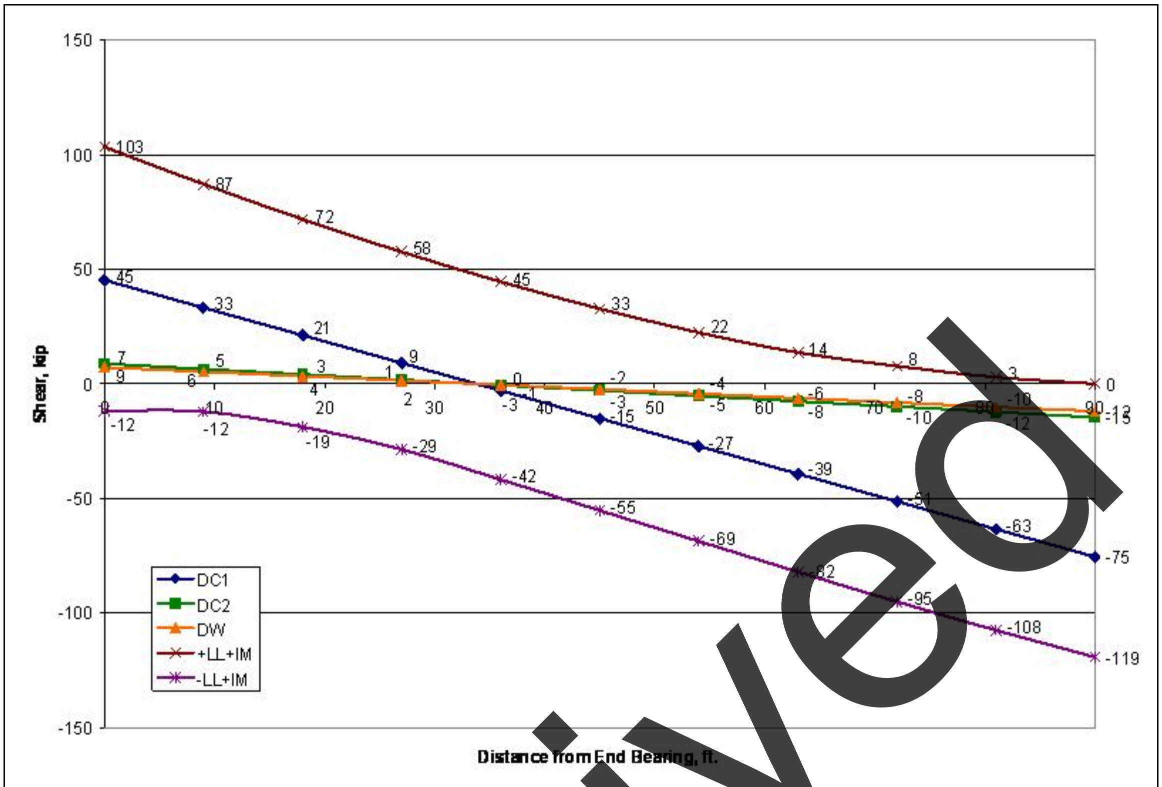

Figure 8 Dead and Live Load Shear Envelopes

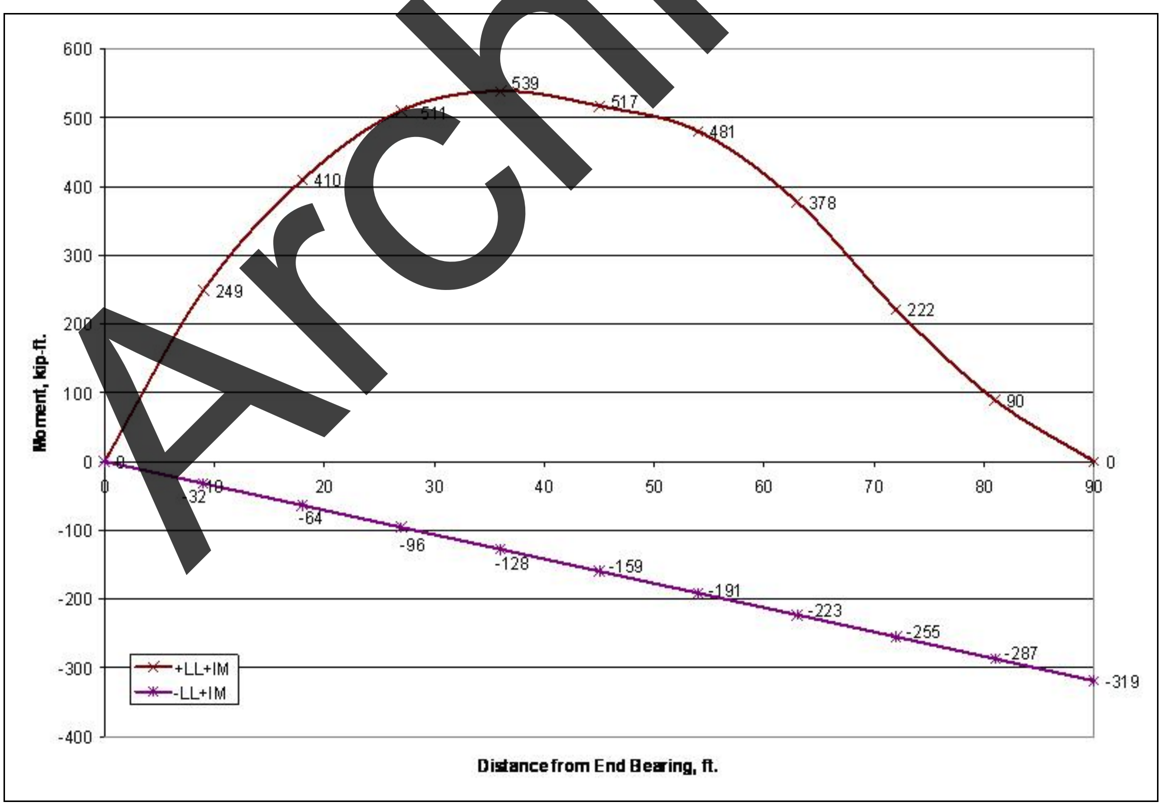

Figure 9 Fatigue Live Load Moments

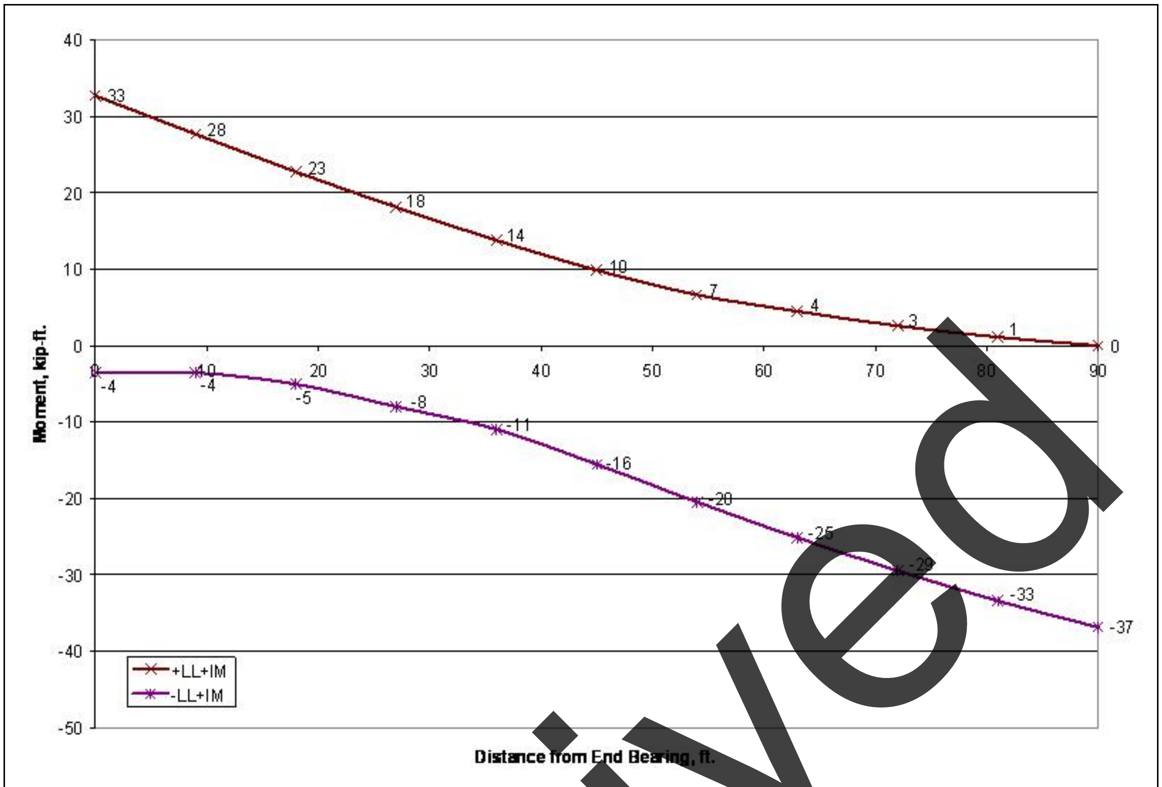

Figure 10 Fatigue Live Load Shears

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	353	68	56	486	-59	201	-32	381	-43	0	0	0	0
0.20	598	116	95	816	-119	350	-65	653	-86	0	0	0	0
0.30	734	142	116	1003	-178	447	-97	818	-130	0	0	0	0
0.40	761	147	121	1085	-238	492	-130	884	-173	0	0	0	0
0.50	679	132	108	1062	-297	486	-162	868	-216	0	0	0	0
0.60	489	95	78	954	-357	428	-194	781	-259	0	0	0	0
0.70	190	37	30	746	-416	318	-227	627	-302	0	0	0	0
0.80	-217	-42	-35	466	-475	156	-259	424	-346	0	-476	0	-607
0.90	-734	-142	-116	148	-535	30	-380	192	-389	0	-746	0	-682
1.00	-1359	-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 positive	Vehicle negative	Special negative	Standard negative	1.33 Vehicle + Lane positive	Distribution Factors	LL+I Positive	LL+I 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) positive	1.75 (LL+IM) negative	Strength I positive	Strength I negative
0.00	0	0	0	0	0	0	0
0.10	442	86	84	1274	-168	1885	444
0.20	747	145	142	2161	-335	3195	699
0.30	917	178	175	2681	-503	3951	766
0.40	951	184	181	2913	-671	4229	646
0.50	849	165	162	2857	-839	4033	337
0.60	611	118	116	2553	-1006	3399	-160
0.70	238	46	45	1971	-1174	2300	-845
0.80	-272	-53	-52	1167	-1444	790	-1820
0.90	-917	-178	-175	431	-1860	-839	-3129
1.00	-1698	-329	-323	0	-3016	-2351	-5367

Table 5 Service II Load Combination Moments (kip-ft)

Span 1	1.0 DC1	1.0 DC2	1.0 DW	1.3 (LL+IM) positive	1.3 (LL+IM) negative	Service II positive	Service II negative
0.00	0	0	0	0	0	0	0
0.10	353	68	56	947	-125	1424	353
0.20	598	116	95	1605	-249	2414	559
0.30	734	142	116	1992	-374	2984	619
0.40	761	147	121	2164	-498	3193	531
0.50	679	132	108	2123	-623	3041	296
0.60	489	95	78	1896	-747	2558	-86
0.70	190	37	30	1464	-872	1722	-615
0.80	-217	-42	-35	867	-1073	573	-1367
0.90	-734	-142	-116	320	-1382	-672	-2374
1.00	-1359	-263	-216	0	-2240	-1838	-4078

Table 6 Unfactored and Undistributed Shears (kip)

Span 1	Non-Com. Dead DC1	Com. Dead DC2	Wearing Surf. DW	Truck Load		Lane Load		Tandem	
				positive	negative	positive	negative	positive	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		1.33 V Vehicle + V Lane		Distribution Factors	V LL	
	positive	negative	positive	negative		positive	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)		Strength I	
				positive	negative	positive	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 provided in Article 3.6.1.3.2, control and 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 calculated earlier.

The maximum deflection due to the design truck is 1.114" Applying the impact and distribution factors gives the following deflection for the design truck load case.

$$\Delta_{LL+IM} = 0.500 \text{ x } 1.33 \text{ x } 1.114 = 0.741 \text{ in.} \qquad \text{(governs)}$$

The deflection due to the lane loading is 0.578 inches. Thus, the deflection due to 25% of the design truck plus the lane loading is equal to the following.

$$\Delta_{LL+IM} = 0.500 \text{ (} 1.33 \text{ x } 0.25 \text{ x } 1.114 + 0.578 \text{) } = 0.474 \text{ in.}$$

Thus the governing deflection, equal to 0.741 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)

The intent of the Service Limit State is to ensure the satisfactory performance and rideability of the bridge structure by preventing localized yielding. For steel members, these objectives are intended to be satisfied by limiting the maximum levels of stress that are permissible. The optional live-load deflection criterion is also included in the service limit state and is intended to ensure user comfort.

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

The intent of the Fatigue and Fracture Limit State is to control crack growth under cyclic loading. This is accomplished by limiting the stress range to which steel members are subjected. The allowable stress range varies for various design details and member types. The fatigue limit state also restricts the out-of-plane flexing of the web. Additionally, fracture toughness requirements are stated in Article 6.6.2 of the specifications and are dependent on the temperature zone.

7.3 Strength Limit State (Articles 1.3.2.4 and 6.5.4)

The strength limit state ensures 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 ensure 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 to ensure uniformity between predicted and actual stresses. In addition, lateral flange bending stresses resulting from forces applied to the overhang brackets during construction should also be considered during the constructability evaluation.

7.4 Extreme Event Limit State (Articles 1.3.2.5 and 6.5.5)

The extreme event limit state is to ensure 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 by 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 construction sequence is verified. The optional moment redistribution specifications are utilized. Other design components presented include the field splice design, shear stud design, bearing design, and concrete deck design. The moment and shear envelopes provided in Figures 7 through 10 are employed for the following calculations.

8.1 Section Properties

The section properties are first calculated as these properties will be routinely used in the subsequent evaluations of the various limit states. The structural slab thickness is taken as the slab thickness minus the integral wearing surface (8 inch) and the modular ratio is taken as 8 in these calculations. Because the section is prismatic, the effective flange width and section properties are constant along the length of the beam. However, separate calculations are necessary for the computation of the plastic moment and yield moment depending on if the section is in negative bending or positive bending.

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, where alternative calculations are specified for interior and exterior beams. The effective flange width for interior beams is one-half the distance to the adjacent girder on each side of the component.

For the interior beams in this example, b_{eff} is then computed as follows.

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

The effective flange width for exterior beams is determined as one-half the distance to the adjacent girder plus the full overhang width.

For the exterior beams, b_{eff} is then computed as.

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

Because the effective width is lesser while the moment distribution factor is greater for the exterior beam, the moment design is controlled by the exterior beam.

8.1.2 Elastic Section Properties

As discussed above, the section properties that are to be considered in the analysis of the beam vary based on the loading conditions. The section properties for the steel section (beam alone) are used in the constructability evaluation. 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 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 typical modular ratio, or 24. The section properties for the short-term and long-term composite sections are thus computed below (Tables 9 through 11). In negative bending, the applicable section consists of the steel beam in addition to the steel reinforcement of the concrete deck.

The section properties of the W40x215 beam are as follows.

$I_{NA} = 16{,}700\ \text{in}^4$

$d_{\text{TOP OF STEEL}} = 19.50\ \text{in.}$ $\qquad S_{\text{TOP OF STEEL}} = \dfrac{16{,}700}{19.50} = 856.4$

$d_{\text{BOT OF STEEL}} = 19.50\ \text{in.}$ $\qquad S_{\text{BOT OF STEEL}} = \dfrac{16{,}700}{19.50} = 856.4\ \text{in.}^3$

Table 9 Short Term Composite (n) Section Properties

Component	A	d	Ad	Ad^2	I_o	I
Steel Section	63.4					16,700
Concrete Slab (8"x 102"/8)	102.0	24.28	2,477	60,142	544	60,686
	165.4		2,477			77,386
				-14.98(2,477) =		-37,105
						40,371 in⁴

$d_s = \dfrac{2{,}477}{165.4} = 14.98\ \text{in.}$

$d_{\text{TOP OF STEEL}} = 19.50 - 14.98 = 4.52\ \text{in.}$ $\qquad S_{\text{TOP OF STEEL}} = \dfrac{40{,}371}{4.52} = 8{,}932\ \text{in.}^3$

$d_{\text{BOT OF STEEL}} = 19.50 + 14.98 = 34.48\ \text{in.}$ $\qquad S_{\text{BOT OF STEEL}} = \dfrac{40{,}371}{34.48} = 1{,}171\ \text{in.}^3$

Table 10 Long Term Composite (3n) Section Properties

Component	A	d	Ad	Ad^2	I_o	I
Steel Section	63.4					16,700
Concrete Slab (8"x 102"/24)	34.0	24.28	825.5	20,044	181.3	20,225
	97.4		825.5			36,925
				-8.48(825.5) =		-7,000 in^4
						29,925

$$d_s = \frac{825.5}{97.4} = 8.48 \text{ in.}$$

$d_{\text{TOP OF STEEL}} = 19.50 - 8.48 = 11.02$ in.

$d_{\text{BOT OF STEEL}} = 19.50 + 8.48 = 27.98$ in.

$$S_{\text{TOP OF STEEL}} = \frac{29{,}925}{11.02} = 2{,}716 \text{ in.}^3$$

$$S_{\text{BOT OF STEEL}} = \frac{29{,}925}{27.98} = 1{,}070 \text{ in.}^3$$

Table 11 Steel Section and Longitudinal Reinforcement Section Properties

Component	A	d	Ad	Ad^2	I_o	I
Steel Section	63.4					16,700
Top Long. Reinforcement	6.53	26.03	170.0	4,424		4,424
Bot. Long. Reinforcement	3.27	21.53	70.40	1,516		1,516
	73.2		240.4			22,640
				-3.28(240.4) =		-789
						21,851 in^4

$$d_s = \frac{240.4}{73.20} = 3.28 \text{ in.}$$

$d_{\text{TOP OF STEEL}} = 19.50 - 3.28 = 16.22$ in.

$d_{\text{BOT OF STEEL}} = 19.50 + 3.28 = 22.78$ in.

$$S_{\text{TOP OF STEEL}} = \frac{21{,}851}{16.22} = 1{,}347 \text{ in.}^3$$

$$S_{\text{BOT OF STEEL}} = \frac{21{,}851}{22.78} = 959 \text{ in.}^3$$

The specifications specify that the section modulus of the steel plus reinforcing section shall be taken about the first element to yield of either the top flange or the reinforcing steel. Using the distances from the neutral axis to each element it is determined that the reinforcing steel is the first to yield, as demonstrated below.

$$\frac{x}{22.75} = \frac{50}{16.22}$$

$$x = 70.13 \text{ ksi} > F_{yr} = 60 \text{ ksi}$$

Therefore, the reinforcing steel yields first.

$S_{REIN.} = \frac{21,851}{22.75} = 960.5 \text{ in.}^3$

$S_{BOT\ OF\ STEEL} = \frac{21,851}{22.78} = 959.2 \text{ in.}^3$

8.1.3 Plastic Moment

8.1.3.1 Positive Bending

The plastic moment M_p can be determined 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.

$P_s = 0.85f'_c b_s t_s = 0.85(4.0)(102.0)(8) \quad = 2,774 \text{ kips}$

$P_c = F_{yc} b_c t_c = (50)(15.8)(1.22) \quad = 964 \text{ kips}$

$P_w = F_{yw} D t_w = (50)(36.56)(0.65) \quad = 1,188 \text{ kips}$

$P_t = F_{yt} b_t t_t = (50)(15.8)(1.22) \quad = 964 \text{ kips}$

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.

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

$$964 + 1,188 \geq 964 + 2,774$$

$$2,152 \leq 3,738$$

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

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

$$964 + 1,188 + 964 \geq 2,774$$

$$3,116 \geq 2,774$$

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

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

$$\bar{y} = \left(\frac{1.22}{2}\right)\left[\frac{1{,}188 + 964 - 2{,}774}{964}\right] = 0.22\text{in.}$$

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.

$d_s = 0.22 + 8.0/2 + 2 – 1.22 = 5.00$ in.

$d_w = 1.22 – 0.22 + 36.56/2 = 19.28$ in.

$d_t = 1.22 – 0.22 + 36.56 + 1.22/2 = 38.17$ in.

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

$$M_p = \frac{964}{2(1.22)}\left[(0.35)^2 + \left(1.22 - 0.35\right)^2\right] + \left[(2{,}557)(5.13) + (1{,}188)(19.15) + (964)(38.04)\right]$$

$$M_p = \left(\frac{964}{2(1.22)}\right)\left[(0.22)^2 + (1.22 - 0.22)^2\right] + \left[(2{,}774)(5.00) + (1{,}188)(19.28) + (964)(38.17)\right]$$

$M_p = 73{,}985$ kip-in = 6,165 k-ft

8.1.3.2 Negative Bending

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

P_s	=	$0.85f'_c b_s t_s$	=	0.85(4.0)(102.0)(8)	=	2774 kips
P_c	=	$F_{yc} b_c t_c$	=	(50)(15.8)(1.22)	=	964 kips
P_w	=	$F_{yw} D t_w$	=	(50)(36.56)(0.65)	=	1188 kips
P_t	=	$F_{yt} b_t t_t$	=	(50)(15.8)(1.22)	=	964 kips
P_{rb}	=	$F_{yrb} A_{rb}$	=	(60)(3.27)	=	196 kips
P_{rt}	=	$F_{yrt} A_{rt}$	=	(60)(6.53)	=	392 kips

As before, the relative forces in each member are used to determine the location of the plastic neutral axis. Because the following equation is satisfied, it is determined that the PNA is in the web.

$$P_c + P_w \geq P_t + P_{rb} + P_{rt} = 964 + 1{,}188 \geq 964 + 196 + 392$$

2152 > 1552, therefore, the plastic neutral axis is in the web.

The plastic neutral axis location is then computed by the following equation.

$$\bar{y} = \left(\frac{D}{2}\right)\left[\frac{P_c - P_t - P_{rt} - P_{rb}}{P_w} + 1\right] = \left(\frac{36.56}{2}\right)\left[\frac{964 - 964 - 392 - 196}{1{,}188} + 1\right] = 9.23\text{ in.}$$

M_p is then computed as follows.

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

where, d_{rt} = 9.23 + 2 + 8 – 2.25 = 16.98 in.

d_{rb} = 9.23 + 2 + 1.25 = 12.48 in.

d_t = 9.23 + 1.22/2 = 9.84 in.

d_c = 36.56 – 9.23 + 1.22/2 = 27.94 in.

$$M_p = \frac{1{,}188}{2(36.56)}\left[(9.23)^2 + (36.56 - 9.23)^2\right] + \left[(392)(16.98) + (196)(12.48) + (964)(9.84) + (964)(27.94)\right]$$

M_p = 59,042 kip-in = 4920 k-ft

8.1.4 Yield Moment

8.1.4.1 Positive Bending

The yield moment, which is the moment which causes first yield in either flange (neglecting flange lateral bending) is detailed 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.

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

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

$$50 = \frac{(1.25)(761)(12)}{856.4} + \frac{(1.25)(147)(12) + (1.50)(121)(12)}{1{,}061} + \frac{M_{AD}}{1{,}163}$$

$$50 = 1.0\left[\frac{1.25(761)(12)}{856.4} + \frac{1.25(147)(12) + 1.50(120)(12)}{1{,}070} + \frac{\mathrm{M}_{\mathrm{AD}}}{1{,}171}\right]$$

M_{AD} = 38,145 k-in. = 3,179 k-ft.

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

M_y = 1.25(761) + 1.25(147) + 1.50(121) + 3179 Eq. (D6.2.2-2)

M_y = 4,496 k-ft

8.1.4.2 Negative Bending

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 reinforcing steel, the section modulus is the same for both the short-term and long-term composite sections.

As discussed above when computing the yield moment in positive bending, the yield moment is the minimum of the moment which causes yielding on the compression side and the moment which causes yielding on the tension side. Because, for negative bending, the section modulus values taken about the top and bottom of the beam are nearly equal to one another, it is not clear which yield moment value will control. Thus, the moments causing first yield in both compression and tension are computed below. The moment causing yielding in the compression flange is first computed based on Equation D6.2.2-1.

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

$$(50) = \frac{(1.25)(1{,}359)(12)}{856.4} + \frac{(1.25)(263)(12) + (1.50)(216)(12)}{959} + \frac{M_{AD}}{959}$$

M_{AD} = 17,290 k-in. = 1441 k-ft.

M_{yc} = (1.25)(1,359) + (1.25)(263) + (1.50)(216) + 1441

M_{yc} = 3793 k-ft. (governs)

Similarly, the moment which causes' yielding in tension (in the steel reinforcing) is computed as follows.

$$(50) = \frac{(1.25)(1,359)(12)}{856.4} + \frac{(1.25)(263)(12) + (1.50)(216)(12)}{960.5} + \frac{M_{AD}}{960.5}$$

M_{AD} = 17,329 k-in. = 1444 k-ft.

M_{yt} = (1.25)(1,359) + (1.25)(263) + (1.50)(216) + 1444

M_{yt} = 3796 k-ft.

Thus, the compression yield moment governs and the yield moment M_y = 3793 k-ft.

8.2 Exterior Beam Check: Negative Bending

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

8.2.1 Strength Limit State (Article 6.10.6)

8.2.1.1 Flexure

The strength requirements for negative flexure are given by Article 6.10.8, Appendix A of Section 6.10, or Appendix B of Article 6.10, at the designers option. Article 6.10.8 limits the maximum capacity to the yield moment of the section. Alternatively, Appendix A of Article 6.10 permits beam capacities up to M_p and may be used for beams having a yield strength less than or equal to 70 ksi and a compact or non-compact web, which is defined by Equation A6.1-1. Appendix B utilizes the moment capacities predicted from either 6.10.8 or Appendix A and allows up to 20% of the moment at the pier to be redistributed to positive bending sections. It is demonstrated below that Appendix A is applicable for this example. Therefore, the moment capacity of the section is first computed based on these strength prediction equations in presented below.

8.2.1.2 Flexural Resistance (Appendix A)

In order to evaluate the above flexural requirements, the flexural resistances based on buckling of the compression flange and yielding of the tension flange are evaluated in this section. The applicability of Appendix A for this design example is first evaluated below. The requirement that the nominal yield strength must be less than 70 ksi is easily evaluated.

$F_{yf} = 50\,ksi \le 70\,ksi$ (satisfied)

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

$$\frac{2D_c}{t_w} < 5.7\sqrt{\frac{E}{F_{yc}}} \qquad \text{Eq. (A6.1-1)}$$

As computed above the elastic neutral axis is located 22.78 in. from the bottom of the composite negative bending section. Subtracting the bottom flange thickness gives the web depth in compression in the elastic range (D_c) as computed below.

$D_c = 22.78 - 1.22 = 21.56$ in.

Substituting the applicable values into Equation A6.1-1 shows that the equation is satisfied.

$$\frac{2(21.56)}{(0.65)} < 5.7\sqrt{\frac{(29,000)}{(50)}} = 66.34 < 137..37 \qquad \text{(satisfied)}$$

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

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

$$1 \geq 0.3 \qquad \text{(satisfied)}$$

Thus, Appendix A is applicable. Use of Appendix A begins with the computation of the web plastification factors, as detailed in Article A6.2 and calculated below.

$$\frac{2D_{cp}}{t_w} < \lambda_{pw(D_{cp})}, \qquad \text{Eq. (A6.2.1-1)}$$

where: $$\lambda_{pw(D_{CP})} = \frac{\sqrt{\frac{E}{F_{yc}}}}{\left(0.54\frac{M_p}{R_h M_y} - 0.1\right)^2} \leq \lambda_{rw}\left(\frac{D_{cp}}{D_c}\right) \qquad \text{Eq. (A6.2.1-2)}$$

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

$$\lambda_{pw(D_{cp})} = \frac{\sqrt{\frac{(29,0000)}{(50)}}}{\left(0.54\frac{59,042}{(1.0)(3,793)(12)} - 0.1\right)^2} = 66.79$$

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.

$D_{cp} = 36.56 - 9.23 = 27.33$ in.

The web slenderness classification is then determined as follows.

$$\frac{2D_{cp}}{t_w} = \frac{2(27.33)}{0.65} = 84.09 > \lambda_{pw(D_{cp})} = 66.79 \qquad \text{(not satisfied)}$$

As shown, the section does not qualify as compact. However, it was previously demonstrated, when evaluating the Appendix A applicability, that the web does qualify as non-compact. Therefore, the applicable web plastification factors are specified by Eqs. A6.2.2-4 and A6.2.2-5 and are calculated as follows.

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

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

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

$$\lambda_{pw(D_c)} = (66.79)\left(\frac{21.56}{27.33}\right) = 52.69$$

$$R_{pc} = \left[1 - \left(1 - \frac{(1.0)(3,793)(12)}{59,042}\right)\left(\frac{66.34 - 52.69}{137.27 - 52.69}\right)\right]\frac{59,042}{(3,793)(12)} \le \frac{59,042}{(3,793)(12)}$$

$$R_{pc} = 1.249 \le 1.297$$

$$R_{pc} = 1.249$$

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

$$R_{pt} = \left[1 - \left(1 - \frac{(1.0)(3,793)(12)}{59,042}\right)\left(\frac{66.34 - 52.69}{137.27 - 52.69}\right)\right]\frac{59,042}{(3,796)(12)} \le \frac{59,042}{(3,796)(12)}$$

$$R_{pt} = 1.248 \le 1.296$$

$$R_{pt} = 1.248$$

The flexural resistance based on the compression flange is determined from Article A6.3 and is taken as the minimum of the local buckling resistance from Article A6.3.2 and the lateral torsional buckling resistance from Article A6.3.3. To evaluate the local buckling resistance, the flange slenderness classification is first determined, where the flange is considered compact if the following equation is satisfied.

$$\lambda_f \leq \lambda_{pt}$$

where: $\lambda_f = \dfrac{b_{fc}}{2t_{fc}} = \dfrac{15.8}{2(1.22)} = 6.48$ Eq. (A6.3.2-3)

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

$$\lambda_f = 6.48 \leq \lambda_{pf} = 9.15 \quad \text{(satisfied)}$$

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

$$M_{nc} = R_{pc}M_{yc} = (1.249)(3793) = 4737 \text{ k-ft.} \quad \text{Eq. (A6.3.2-1)}$$

Similarly, to evaluate the compressive flexural resistance based on lateral-torsional buckling, the lateral bracing distance must be first classified. Lateral bracing distances satisfying the following equation are classified as compact.

$$L_b \leq L_p \quad \text{Eq. (A6.3.3-4)}$$

where: $L_b = (15.0)(12)$

$$L_p = r_t\sqrt{\frac{E}{F_{yc}}}$$

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

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1 + \frac{1}{3}\frac{D_c}{b_{fc}}\frac{t_w}{t_{fc}}\right)}} \quad \text{Eq. (A6.3.3-11)}$$

$$r_t = \frac{15.8}{\sqrt{12\left(1 + \frac{1}{3}\frac{(21.56)(0.65)}{(15.8)(1.22)}\right)}} = 4.092 \text{ in.}$$

$$L_p = 4.092\sqrt{\frac{29{,}000}{50}} = 98.55$$

Therefore, $L_b > L_p$. (not compact)

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

$$L_p < L_b \le L_r$$

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

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

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

J = St. Venant torsional constant

h = depth between the centerline of the flanges

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

$$S_{xt} = \frac{(3,796)(12)}{50} = 911.0 \text{ in.}^3$$

$$S_{xc} = \frac{(3,793)(12)}{50} = 910.3 \text{ in.}^3$$

$$F_{yr} = \min\left(0.7(50), (1.0)(50)\frac{911.0}{910.3}, 50\right)$$

$$F_{yr} = \min(35, 50, 50)$$

$$F_{yr} = 35.0 \text{ ksi} > 0.5\,F_{yc} = 25 \text{ ksi} \qquad \text{(satisfied)}$$

$$J = \frac{1}{3}\left(\mathrm{D t_w}^3 + \mathrm{b_{fc} t_{fc}}^3\left(1 - 0.63\frac{\mathrm{b_{fc}}}{\mathrm{t_{fc}}}\right) + \mathrm{b_{ft} t_{ft}}^3\left(1 - .63\frac{\mathrm{b_{ft}}}{\mathrm{t_{ft}}}\right)\right) \qquad \text{Eq. (A6.3.3-9)}$$

$$J = \frac{1}{3}\left((36.56)(0.65)^3 + (15.8)(1.22)^3(0.948) + (15.8)(1.22)^3(0.948)\right) = 21.48 \text{in.}^3$$

$$h = \frac{1.22}{2} + 36.56 + \frac{1.22}{2} = 37.78 \text{ in.}$$

$$\mathrm{L_r} = 1.95(4.092)\frac{29{,}000}{35}\sqrt{\frac{21.48}{(910.3)(37.78)}}\sqrt{1+\sqrt{1+6.76\left(\frac{35}{29{,}000}\,\frac{(910.3)(37.78)}{21.48}\right)^2}} = 408.8$$

in.

$L_b = 180 \le L_r = 408.8$ (satisfied)

Therefore, the lateral bracing distance is classified as non-compact 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 = $2M_{mid} - M_2 \ge M_0$ Eq. (A6.3.3-12)

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

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

M_2 = largest 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 applicable moment values are as follows.

M_2 = 5367 k-ft. $M_1 = 2M_{mid} - M_2 \ge M_0$

M_0 = 2126 k-ft. $M_1 = 2(3502) - (5367) = 1637 \le 2126$

M_{mid} = 3502 k-ft. M_1 = 2126 k-ft.

$$C_b = 1.75-1.05\left(\frac{2126}{5367}\right)+0.3\left(\frac{2126}{5367}\right)^2 = 1.38 \le 2.3$$

$$M_{nc} = (1.38)\left[1-\left(1-\frac{(35.0)(910.3)}{(1.249)(3793)(12)}\right)\left(\frac{180-98.55}{410.7-98.55}\right)\right]\times(1.249)(3793) \le (1.249)(3793)$$

$$M_{nc} = 5788 \le 4737$$

$M_{nc} = 4737$ k-ft.

As previously stated, the flexural capacity 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.

$M_{nc} = 4737$ k-ft.

Multiplying the nominal moment capacity by the applicable resistance factor gives the following.

$\phi_f M_{nc} = (1.0)(4737)$

$\phi_f M_{nc} = 4737$ k-ft.

The moment capacity is also evaluated in terms of the tensile moment capacity. For a continuously braced tension flange at the strength limit state, the section must satisfy the requirements of Article A6.1.4.

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

Therefore, the factored moment resistance as governed by tension flange yielding is expressed by the following.

$\phi_f M_{nt} = \phi_f R_{pt} M_{yt} = (1.0)(1.248)(3796) = 4737$ k-ft.

8.2.1.3 Factored Moment

At the strength limit state, the design moment is equal to the sum of vertical bending moments applied from forces such as dead loads and vehicular loads. In addition, one-third of the lateral bending moment induced by loads such as wind loads must also be added for discretely braced flanges. These design moments must be less than the moment resistance of the section. For the present design example, these requirements are expressed by Eqs. A6.1.1-1 and A6.1.4-1.

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

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

Equation A6.1.1-1 requires that the vertical bending moment plus one-third of the lateral bending moment is less than the moment resistance based on buckling of the compression flange and is applicable for sections with discretely braced compression flanges. Equation A6.1.4-1 is intended to prevent yielding of the tension flanges and is applicable to continuously braced tension flanges, where lateral bending effects are not applicable.

Furthermore, at the Strength limit state there are five load combinations to consider. Considering only the loads applicable to the superstructure elements in this design example, the load combinations are as follows.

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

Strength II = 1.25DC + 1.5DW + 1.35(LL+I)

Strength III = 1.25DC + 1.5DW + 1.4WS

Strength IV = 1.5(DC + DW)

Strength V = 1.25DC + 1.5DW + 1.35(LL+I) + 0.4WS

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

DC = -1359 – 263 = -1622 ft.-kips

DW = -216 k-ft

From Table 3, the controlling LL+I moment is -1723 k-ft.

LL+I = -1723 k-ft

The horizontal pressure applied by the wind load loads was previously determined to be 0.050 ksf. 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. It is required that the wind force per unit length of the bridge must exceed 0.3 kips/ft. Multiplying the design pressure by the exposed height of the superstructure, assuming a 42 in. parapet height gives the following.

$F_D = (0.050)(39 - 1.22 + 2 + 8.5 + 42)/12$

$F_D = 0.376 \text{ k/ft} > 0.3$

Therefore, the design wind pressure exceeds the minimum required design pressure. 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 = 0.050 ksf

d = beam depth = 39.0 in = 3.25 ft

γ = varies depending on limit state

$$W = \frac{(1.0)(\gamma)(0.050)(3.25)}{2} = 0.8125\gamma \text{ k/ft}$$

The maximum flange lateral bending moment is then computed according to Eq. C4-9.

$$M_w = \frac{WL_b^2}{10} = \frac{(0.08125\gamma)(15)^2}{10} = 1.828\gamma \text{ k-ft} \qquad \text{Eq. (C4.6.2.7.1-2)}$$

Consideration should also be given to increasing the wind load moments to account for second-order force effects, as specified in Article 6.10.1.6 through application of the amplification factor. However, no increase is required for tension flanges, so the amplification factor is negligible in this case. Lateral bending forces due to the wind loading are then determined by dividing M_w by the section modulus of the bottom flange.

$$f_\ell \quad = \quad f_l = \frac{M_w}{S_\ell} = \frac{(1.828\gamma)12}{(15.8)^2(1.22)/6} = 0.432\gamma \text{ ksi}$$

It is required that the flange lateral bending stresses may not exceed 60% of the flange yield strength. Thus, for this example f_ℓ must be less than or equal to 30 ksi, which is easily satisfied for the above lateral stress of 0.432γ considering the maximum load factor is 1.4, e.g, the maximum lateral bending stress is 0.60 ksi.

The controlling strength limit state can now be determined based on the above information. For the Strength I load combination, the design moments are as follows.

$$M_u = 1.25(1622) + 1.5(216) + 1.75(1723) = 5367 \text{ k-ft} \qquad \text{(governs)}$$

$$M_u + \frac{1}{3} f_\ell S_{xc} = 5367 + 0 \qquad \text{(wind loads not considered)}$$

$$M_u + \frac{1}{3} f_\ell S_{xc} = 5367 \text{ k-ft} \qquad \text{(governs)}$$

It is obvious that the design moments for the Strength II load combination will be less than those at the Strength I load combination as all of the Strength II load factors are either equal to or less than those used for the Strength I combination. The design moments at Strength II are equal to the following.

$$M_u = 1.25(1622) + 1.5(216) + 1.35(1723) = 4678 \text{ k-ft}$$

$$M_u + \frac{1}{3} f_\ell S_{xc} = 4678 + 0 \qquad \text{(wind loads not considered)}$$

$$M_u + \frac{1}{3} f_\ell S_{xc} = 4678 \text{ k-ft}$$

For the Strength III load combination, wind load is incorporated and the design moments are equal to the following.

$$M_u = 1.25(1622) + 1.5(216) = 2352 \text{ k-ft}$$

$$M_u + \frac{1}{3} f_\ell S_{xc} = 2352 + (1/3)(1.828)(1.4) = 2353 \text{ ft.-kips}$$

The design moments for the Strength IV load combination are as follows.

$$M_u = 1.5(1622 + 216) = 2757 \text{ ft.-kips}$$

$$M_u + \frac{1}{3} f_\ell S_{xc} = 2757 + 0 = 2757 \text{ ft.-kips}$$

Lastly, the design moments computed using the Strength V load combination are equal to the following.

$$M_u = 1.25(1622) + 1.5(216) + 1.35(1723) = 4678 \text{ ft.-kips}$$

$$M_u + \frac{1}{3} f_\ell S_{xc} = 4678 + (1/3)(1.828)(0.4) = 4678 \text{ ft.-kips}$$

Reviewing the factored moments for each load combination computed above, it is determined that the Strength I moments govern for this example and that the design moment for both compression flange and tension flange resistances is equal to 5367 k-ft.

Comparing this design moment to the moment capacities computed above shows that moment redistribution will occur as the resistance of the section is less than the applied moment. Hence, the requirements of Appendix B are now used to evaluate the moment capacity of the negative bending section.

8.2.1.4 Moment Redistribution (Appendix B)

Article B6.2 defines the applicability of the Appendix B provisions. Specifically that the sections must be straight continuous span I-sections that are not skewed more than 10 degrees and do not have staggered cross-frames. The specified minimum yield strength of the section must not exceed 70 ksi. In addition, the section must satisfy web proportions (Article B6.2.1), compression flange proportions (Article B6.2.2), section transition (Article B6.2.3), compression flange bracing (Article B6.2.4), and shear (Article B6.2.5) requirements, which are discussed below. Equations B6.2.1-1, B6.2.1-2, and B6.2.1-3 specify the web proportion limits that must be satisfied.

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

$$\frac{D}{t_w} = \frac{36.56}{0.65} = 56.25 \leq 150 \qquad \text{(satisfied)}$$

$$\frac{2D_c}{t_w} 6 \leq 6.8\sqrt{\frac{E}{F_{yc}}} \qquad \text{Eq. (B6.2.1-2)}$$

$$\frac{2(21.56)}{0.65} = 66.34 \leq 6.8\sqrt{\frac{29{,}000}{50}} = 163.8 \qquad \text{(satisfied)}$$

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

$$D_{cp} = 27.33 \leq 0.75(36.56) = 27.42 \qquad \text{(satisfied)}$$

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

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

$$\frac{15.8}{2(1.22)} = 6.48 \leq 0.38\sqrt{\frac{29{,}000}{50}} = 9.15 \qquad \text{(satisfied)}$$

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

$$b_{fc} = 15.8 \geq \frac{36.56}{4.25} = 8.60 \qquad \text{(satisfied)}$$

The lateral bracing distance must satisfy:

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

$$L_b = 180.0 \leq \left[0.1 - 0.06\left(\frac{2{,}126}{5{,}367}\right)\right]\frac{(4.092)(29{,}000)}{50} = 180.9 \qquad \text{(satisfied)}$$

Additionally, the applied shear under the Strength I loading must be less than the shear buckling resistance of the beam as specified by.

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

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

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

V_p = plastic shear force (kip)

V_p = $0.58\ F_{yw}Dt_w$ 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

Alternative equations are providing 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.

$$\frac{D}{t_w} \le 1.12\sqrt{\frac{Ek}{F_{yw}}} = \frac{36.56}{0.65} = 56.25 < 1.12\sqrt{\frac{(29,000)(5)}{50}} = 60.31 \qquad \text{(satisfied)}$$

C = 1.00

The shear buckling resistance is then computed as follows.

$$V_{cr} = CV_p = (1.00)(0.58)(50)(36.56)(0.65)$$

$$\mathrm{V_{cr}} = 689\ \text{kips}$$

$$V = 339\ \text{kips} \le \phi_v V_{cr} = (1.0)(689) = 689\ \text{kips} \qquad \text{(satisfied)}$$

Therefore, all of the requirements for Appendix B applicability are satisfied.

Once it is determined that Appendix B is applicable, the effective plastic moment is then determined in order to evaluate if the section satisfies the design requirements. The effective plastic moment (M_{pe}) may be determined based on either the equations given in Article B6.5 or the refined procedure given in Article B6.6. In either case M_{pe}, is a function of the geometry and material properties of the section.

When using the M_{pe} equations in Article B6.5, alternative equations are provided for beams that satisfy the requirements for enhanced moment rotation characteristics, i.e., beams classified as ultracompact sections. To be classified as ultracompact, the beam must either (1) contain transverse stiffeners at a location less than or equal to one-half the web depth from the pier or (2) satisfy the web compactness limit given by Eq. B6.5.1-1.

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

$$\frac{2(27.33)}{0.65} = 84.1 > 2.3\sqrt{\frac{29000}{50}} = 55.4 \qquad \text{(not satisfied)}$$

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

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

$$M_{pe} = \left[2.63 - 2.3\frac{15.8}{1.22}\sqrt{\frac{50}{29000}} - 0.35\frac{36.56}{15.8} + 0.39\frac{15.8}{1.22}\sqrt{\frac{50}{29000}}\frac{36.56}{15.8} \right] 4737 \le 4737$$

$$M_{pe} = 5062 > 4737$$

$$M_{pe} = 4737 \text{ k-ft.}$$

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

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

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

where: M_e = vertical bending moment at the pier due to the factored loads

Since the lateral bending stresses are negligible for this example, the previous equations reduce to the following equation.

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

If this redistribution moment is less than 20 percent of the elastic moment, as specified by Eq. B6.4.2.1-3, the strength requirements at the pier are satisfied.

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

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

$$M_{rd} = |M_e| - M_{pe} = 5367 - 4737$$

$$M_{rd} = 630\, k - ft = 12\% M_e \le 20\% M_e$$

Therefore, the negative bending region of the beam satisfies strength requirements.

It is noted that moment redistribution may also be utilized at the service limit state. However, as demonstrated below, the stress requirements at the service limit state are satisfied based on the elastic stresses, and therefore, moment redistribution is not employed at the service limit state in this design example.

8.2.1.5 Shear (6.10.6.3)

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

$$V_n = V_{cr} = CV_p = 689 \text{ kips} \qquad \text{Eq. (6.10.9.2-1)}$$

The applied shear at the pier at the strength limit state was previously given in Table as 339 kips, thus the shear requirements are satisfied.

$$V = 339 \text{ kips} \le \phi_v V_{cr} = (1.0)(689) = 689 \text{ kips} \qquad \text{(satisfied)}$$

8.2.2 Constructibility (Article 6.10.3)

Article 2.5.3 requires that the engineer design bridge systems such that the construction is not difficult nor results 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. The applied loads to be considered are specified in Table 3.4.1-1 and the applicable load factors are provided in Article 3.4.2.

The beams are considered to be non-composite during the initial construction phase. The influence of various segments of the beam becoming composite at various stages of the deck casting sequence is then considered. The effects of forces from deck overhang brackets acting on the fascia beams are also included in the constructibility check.

8.2.2.1 Flexure

8.2.2.1.1 Deck Placement Analysis

Temporary moments the noncomposite girders experience during the casting of the deck can be significantly higher than those which may be calculated based on the final conditions of the system. An analysis of the moments during each casting sequence must be conducted to determine the maximum moments in the structure. The potential for uplift should also be investigated if the casting of the two end pours does not occur simultaneously.

Figure 15 depicts the casting sequence assumed in this design example. As required in Article 6.10.3.4, the loads are applied to the appropriate composite sections during each casting sequence. For example, it is assumed during Cast One that all sections of the girder are non-composite. Similarly, the dead load moments due to the steel components are also based on the non-composite section properties. However, to determine the distribution of moments due to Cast Two, the short-term composite section properties are used in the regions of the girders that were previously cast in Cast One, while the non-composite section properties are used in the region of the girder where concrete is cast in Cast Two. The moments used in the evaluation of the constructability requirements are then taken as the maximum moments that occur during any stage of construction, i.e., the sum of the moments due to the steel dead load and the first casting phase or the sum of the moments due to the steel dead load and both casting phases. Additionally, while not required, the dead load moment resulting from applying all dead load to the short-term composite section (DC1) is also considered.

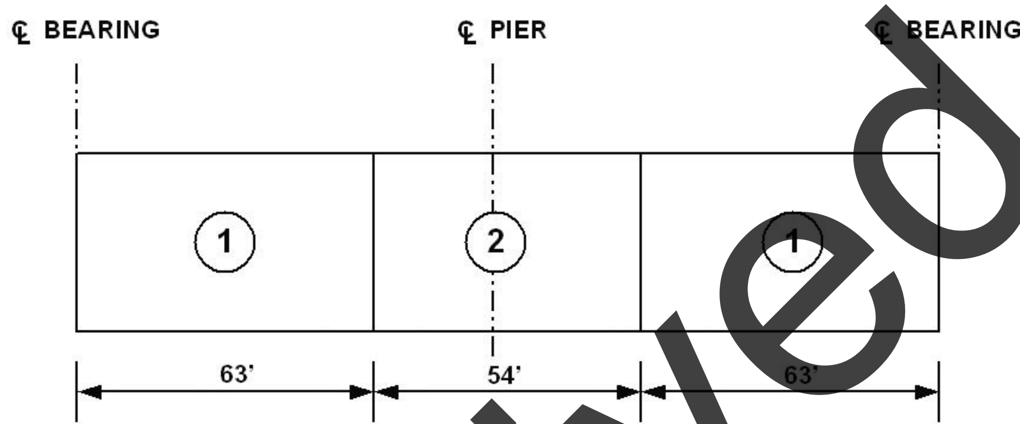

Figure 11 Deck Placement Sequence

The results of the deck placement analysis are shown in Table 12 where the maximum dead load moments in the positive and negative bending regions are indicated by bold text. Note that the maximum positive bending moment during construction occurs during Cast 2 and that the maximum negative bending moment occurs when it is assumed that the loads are simultaneously applied to the composite section.

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	62	106	131	137	124	93	44	-22	-104	-204
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	351	593	727	752	668	476	174	-173	-560	-964
Σ Cast 2	0	385	661	829	**888**	838	680	413	60	-408	-970
DC1	0	353	598	734	761	679	489	190	-217	-734	**-1359**

The controlling negative bending moment is at the pier. Dividing this moment by the section modulus gives the dead load vertical bending stresses, which are labeled f_{bu}. 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.

$$f_{bu} = \frac{1.0(1,359)(12)}{856.4} = 19.04 \text{ ksi}$$

8.2.2.1.2 Deck Overhang Loads

The loads applied to the deck overhang brackets induce lateral bending forces in the flanges of the section. This section illustrates the method to determine these lateral bending stresses using the approximate analysis method governed by Section 6.10.1.6 of the Specifications.

The deck overhang bracket configuration assumed in this example is shown in Figure 12. Typically the brackets are spaced between 3 and 4', inducing lateral bending forces at discrete locations, but the assumption is made here that the overhang dead loads induce a uniformly distributed load on the bottom flange of the beam. Due to the more discrete nature of the load due to the finishing machine, it is assumed that the finishing machine acts as a point load applied to the deck overhang. Half of the overhang weight is assumed to be carried by the exterior beam, and the remaining half is carried by the overhang brackets.

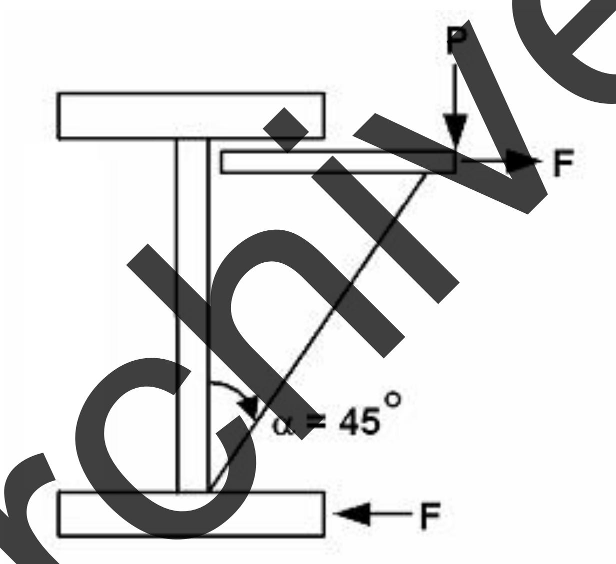

Figure 12 Deck Overhang Bracket Loads

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

$$P = 0.5(150)\left[\frac{8.5}{12}(3.5) + \left[\frac{1}{12}\left(\frac{2.0}{2}\right)\left(3.5 - \frac{14/2}{12}\right)\right] + \frac{1.25}{12}\left(\frac{14/2}{12}\right)\right] = 209 \text{ lbs/ft.}$$

The following is a list of typical construction loads assumed to act on the system before the concrete slab gains strength. The magnitudes of load listed represent only the portion of these loads that are assumed to be applied to the overhang brackets. Note that the finishing machine load 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

Thus, the total uniformly distributed load acting on the overhang form brackets due to the above construction loads is equal to the following.

w = 40 + 85 + 25 + 125

w = 275 lbs/ft

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

F = P tan 45

F =P

The equations provided in Article C6.10.3.4 to determine the lateral bending moment can be employed in the absence of a more refined method. From the article, the following equation determines the lateral bending moment for a uniformly distributed lateral bracket force:

$$M_l = \frac{F_l L_b^2}{12}$$

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 = The unbraced length of the section under consideration = 15 ft (at the location of maximum negative bending)

Thus, the lateral moment due to the component (overhang) dead load is equal to the following.

$$M_l = \frac{(.209)(15)^2}{12} = 3.92 \text{ ft-kips}$$

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.

$$f_l = \frac{M_\ell}{S_\ell} = \frac{3.92(12)}{1.22(15.8)^2/6} = 0.93\text{ ksi}$$

Similarly, the lateral moment and lateral bending stress due to the construction dead loads are computed as follows.

$$M_l = \frac{(.275)(15)^2}{12} = 5.16\text{ ft-kips}$$

$$f_l = \frac{M_\ell}{S_\ell} = \frac{5.16(12)}{1.22(15.8)^2/6} = 1.22\text{ ksi}$$

Note that the lateral moment contribution due to the construction loads and component loads is separated due to the alternative load factors applied to the different load types.

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

$$M_l = \frac{P_l L_b}{8}$$

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

The unfactored lateral bending moment and lateral bending stress due to the finishing machine are then equal to the following.

$$M_l = \frac{(3)(15)}{8} = 5.63\text{ ft-kips}$$

$$f_l = \frac{M_\ell}{S_\ell} = \frac{5.63(12)}{1.22(15.8)^2/6} = 1.33\text{ ksi}$$

For simplicity, the largest values of f_l within the unbraced length (computed above) 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.

It must then be determined if these first-order lateral bending stresses are applicable, or if these stresses must be increased to account for second-order effects. The first-order lateral bending stress may be used if the following limit is satisfied.

$$L_b \le 1.2 L_p \sqrt{\frac{C_b R_b}{f_{bm} / F_{yc}}}$$ 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_{bm} = largest factored compressive stress throughout the unbraced length under consideration, without consideration of lateral bending effects

F_{yc} = yield strength of the compression flange

$$L_p = r_t \sqrt{\frac{E}{F_{yc}}}$$

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\right)}} = \frac{15.8}{\sqrt{12\left(1+\frac{1}{3}\frac{(18.28)(0.65)}{(15.8)(1.22)}\right)}} = 4.154 \text{in.}$$

$$L_p = 4.154\sqrt{\frac{29000}{50}} = 100.0 \text{in.}$$

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

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

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

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

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

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

The following values correspond to the results of the deck placement analysis, i.e., are applicable to the

M_2	= 1359 k-ft.	M_1	$= 2M_{mid} - M_2 \geq M_0$
M_0	= 389 k-ft.	M_1	$= 2(838) - (1{,}359) = 317 \leq 389$
M_{mid}	= 838 k-ft.	M_1	= 389 k-ft.

C_b is then equal to the following.

$$C_b = 1.75 - 1.05\left(\frac{389}{1359}\right) + 0.3\left(\frac{389}{1359}\right)^2 = 1.47$$

According to Article 6.10.1.10.2, the web load-shedding factor, R_b, is 1.0 when checking constructibility. It was determined above in Section 8.2.2.1 that the largest compressive stress due to the vertical bending moment was equal to 19.04 ksi. Multiplying this stress by the governing (highest) load factor of 1.5 gives f_{bm} equal to 28.56 ksi.

$$f_{bm} = (1.5)(19.04)$$

$$f_{bm} = 28.56 \text{ ksi}$$

Thus, Equation 6.10.1.6-2 is evaluated as follows, where it is shown that the first-order elastic analysis is applicable.

$$L_b = 15\ ft. = 180\ in. \leq 1.2(100.0)\sqrt{\frac{(1.47)(1.0)}{28.56/50}} = 192.1 \qquad \text{(satisfied)}$$

8.2.2.1.3 Constructibility Evaluation

During construction, both the compression and tension flanges are discretely braced. Therefore, Article 6.10.3.2 requires the compression flange of 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 ensure 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. Additionally, Eq. 6.10.3.2.2-1 must be satisfied for discretely braced tension flanges, which limits the total flange stress to the yield stress of the tension flange.

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

$$f_{bu} + \frac{1}{3} f_l \leq \phi_f F_{nc} \qquad (6.10.3.2.2\text{-}2)$$

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

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

Lastly, the lateral bending stress in both flanges is limited to a maximum of 60% of the flange yield strength according to Article 6.10.1.6.

$$f_l \leq 0.60F_y$$

The value of each of these quantities at the Strength I load combination is first determined, where a load factor of 1.25 is applied to the component dead load. Furthermore, a load factor of 1.5 is applied to construction loads for all load combinations.

$$f_{bu} = 1.25(19.04)$$

$$f_{bu} = 23.80 \text{ ksi}$$

$$f_l = 1.25(0.92) + 1.5(1.22) + 1.5(1.33)$$

$$f_\ell = 4.98 \text{ ksi} \qquad \text{(governs)}$$

$$f_{bu} + f_l = 23.80 + 4.98$$

$$f_{bu} + f_l = 28.78 \text{ ksi}$$

$$f_{bu} + f_l/3 = 23.80 + (4.98)/3$$

$$f_{bu} + f_l/3 = 25.46 \text{ ksi}$$

The Strength II load combination will not be evaluated in detail as the load factors for this load combination are either equal to or less than those for the Strength I load combination.

The load factors for the Strength III load combination are as 1.25 for component dead load and 1.4 for wind load. Therefore, the vertical bending stress is the same under the Strength III and Strength I load combinations. The other stress values at the Strength III load combinations that are needed for evaluating the constructability requirements are as follows.

$$f_l = 1.25(0.92) + 1.5(1.22) + 1.4(0.43)$$

$$f_l = 3.58 \text{ ksi}$$

$$f_{bu} + f_l = 23.80 + 3.58$$

$$f_{bu} + f_l = 27.38 \text{ ksi}$$

$$f_{bu} + f_l/3 = 23.80 + (3.58)/3$$

$$f_{bu} + f_l/3 = 24.99 \text{ ksi}$$

Only dead loads are considered at the Strength IV load combination where the load factor for all dead loads under this load combination is equal to 1.5.

f_{bu} = 1.5(19.04)

f_{bu} = 28.56 ksi (governs)

f_l = 1.5(0.92) + 1.5(1.22)

f_l = 3.21 ksi

$f_{bu} + f_l$ = 28.56 + 3.21

$f_{bu} + f_l$ = 31.77 ksi (governs)

$f_{bu} + f_l/3$ = 28.56 + (3.21)/3

$f_{bu} + f_l/3$ = 29.63 ksi (governs)

Lastly, at the Strength V load combination the stresses are computed due to load factors of 1.25 for component dead load and 0.4 for wind load.

f_{bu} = 1.25(19.04)

f_{bu} = 23.80 ksi

f_l = 1.25(0.92) + 1.5(1.22) + 1.5(1.33) + 0.4(0.43)

f_l = 5.38 ksi

$f_b + f_l$ = 23.80 + 5.38

$f_b + f_l$ = 29.18 ksi

$f_{bu} + f_l/3$ = 23.80 + (5.38)/3

$f_{bu} + f_l/3$ = 25.59 ksi

Reviewing the computations above for each of the load combinations, the following are determined to be the controlling stesses.

f_{bu} = 28.56 ksi (Strength IV)

f_l = 5.38 ksi (Strength V)

$f_{bu} + f_l$ = 31.77 ksi (Strength IV)

$f_{bu} + f_l/3$ = 29.63 ksi (Strength IV)

Now that the controlling stresses are known the constructability requirements are evaluated, beginning with the check of Equation 6.10.3.2.1-1, which limits the stress in the compression flange to the nominal yield strength of the flange multiplied by the hybrid factor (Equation

6.10.3.2.1-1). For homogeneous material sections the hybrid factor is 1.0, as stated in Article 6.10.1.10.1.

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

$$31.77 \le (1.0)(1.0)(50) = 50\,\text{ksi} \qquad \text{(satisfied)}$$

The flexural resistance of the noncomposite section is required to be greater than the maximum bending moment as a result of the deck casting sequence plus one-third of the lateral bending stresses, as expressed by Equation 6.10.3.2.2-2.

$$f_{bu} + \frac{1}{3} f_l \le \phi_f F_{nc} \qquad (6.10.3.2.2\text{-}2)$$

According to Article 6.10.3.2.1, the flexural resistance, F_{nc}, is determined as specified in Article 6.10.8.2 or Article A6.3.3, if applicable. Two requirements provided in Article A6.1 must be satisfied for Article A6.3.3 to be applicable. These are that the flange yield strength may not exceed 70 ksi, which is satisfied in this case, and that the web slenderness must satisfy Eq. A.6.1-1.

$$F_{yf} = 50\,\text{ksi} \le 70\,\text{ksi} \qquad \text{(satisfied)}$$

$$\frac{2D_c}{t_w} < 5.7\sqrt{\frac{E}{F_{yc}}} \qquad (A6.1\text{-}1)$$

D_c of the non-composite section is simply half the web depth = 36.56 / 2 = 18.28 in.

Equation A6.1-1 may then be evaluated.

$$\frac{2(18.28)}{(0.65)} < 5.7\sqrt{\frac{(29{,}000)}{(50)}}$$

$$56.25 < 137.27 \qquad \text{(satisfied)}$$

Therefore, Appendix A is applicable.

The sections for which Appendix A is applicable have either compact or noncompact web sections where the web classification dictates the equations used to determine the moment capacity. The section qualifies as a compact web section if Eq. A6.2.1-1 is satisfied.

$$\frac{2D_{cp}}{t_w} < \lambda_{pw(D_{cp})} \qquad (A6.2.1\text{-}1)$$

where: D_{cp} = depth of web in compression at the plastic moment

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

$$\lambda_{pw(D_{cp})} = \frac{\sqrt{E/F_{yc}}}{\left(0.54\frac{M_p}{R_h M_y} - 0.1\right)^2} \qquad \text{Eq. (A6.2.1-2)}$$

R_h = hybrid factor

The location of the plastic neutral axis of the steel section is at the mid-depth of the web for this symmetric section.

$D_{cp} = 36.56 / 2 = 18.28$ in.

With this information known, the plastic moment capacity can be computed based on the equations in Appendix D.

$P_c = P_t = (15.8)(1.22)(50)$

$P_c = P_t = 963.8$ kips

$M_p = 2(963.8)(18.89) + 2(36.56/2)(0.65)(50)(36.56/4)$

$M_p = 47273$ k-in. = 3939 k-ft.

Therefore, Eq. A6.1-2 may now be evaluated.

$$\lambda_{pw(D_{cp})} = \frac{\sqrt{\frac{29000}{50}}}{\left(0.54\frac{47273}{(1.0)(50)(856)} - 0.1\right)^2} = 97.72$$

$$\frac{2D_{cp}}{t_w} = \frac{2(18.28)}{0.65} = 56.25 \qquad \text{(satisfied)}$$

Based on the calculations above, the web qualifies as compact. The web plastifications factors are then determined according to Eqs. A6.2.1-4 and A6.2.1-5, where M_{yc} and M_{yt} are the yield moments with respect to the compression and tension flanges, respectively.

$$R_{pc} = \frac{M_p}{M_{yc}} \qquad \text{(A6.2.1-4)}$$

$$R_{pc} = \frac{47273}{(50)(856.4)} = 1.104$$

Since the section is symmetric, the web plastification factor is the same with respect to the tension flange.

$$R_{pt} = 1.104$$

As previously discussed, the lateral torsional buckling resistance is provided in Article A6.3.3. If the following equation is satisfied the lateral brace spacing is classified as compact.

$$L_b \le L_p$$

where: L_b = unbraced length (in.) = 180 in.

L_p = limiting unbraced length to achieve the nominal flexural resistance of $R_{pc}M_{yc}$ under uniform bending (in.)

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

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

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{3}\frac{D_c t_w}{b_{fc}t_{fc}}\right)}} = \frac{15.8}{\sqrt{12\left(1+\frac{1}{3}\frac{(18.28)(0.65)}{(15.8)(1.22)}\right)}} = 4.154\text{in.} \qquad \text{Eq. (A6.3.3-10)}$$

$$L_b = 180\text{ in.} > L_p = 4.154\sqrt{\frac{29000}{50}} = 100.0\text{ in.}$$

Therefore, the unbraced spacing is not compact and the following inequality is evaluated to determine if the unbraced distance is classified as non-compact.

$$L_b \le L_r$$

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

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

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

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

J = St. Venant torsional constant

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

h = depth between the centerline of the flanges = 43.19 in.

Therefore,

$$F_{yr} = \min\left(0.7\mathrm{F_{yc}}, \mathrm{R_n F_y}\frac{\mathrm{S_{xt}}}{\mathrm{S_{xc}}}, \mathrm{F_{yw}}\right) = \min\left(0.7(50), (1.0)(50)\frac{856}{856}, 50\right)$$

$$F_{yr} = \min(35, 50.0, 50) = 35.0\,\text{ksi}$$

$$J = \frac{1}{3}\left(\mathrm{Dt_w}^3 + \mathrm{b_{fc}t_{fc}}^3\left(1 - 0.63\frac{\mathrm{b_{fc}}}{\mathrm{t_{fc}}}\right) + \mathrm{b_{ft}t_{ft}}^3\left(1 - 0.63\frac{\mathrm{b_{ft}}}{\mathrm{t_{ft}}}\right)\right) = 22.47\,\text{in.} \qquad \text{(A6.3.3-9)}$$

L_r is then computed as follows.

$$L_r = 1.95(4.154)\frac{29{,}000}{35}\sqrt{\frac{22.47}{(856.4)(39.0)}}\sqrt{1 + \sqrt{1 + 6.76\left(\frac{35}{29000}\frac{(856.4)(39)}{22.47}\right)^2}}$$

L_r = 418.1 in.

$$L_b = 180 \le L_r = 418.1$$

L_r is greater than the unbraced length, therefore the lateral torsional buckling resistance is controlled by Eq. A6.3.3-2 of the specifications.

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

$$R_{pc}M_{yc} = (1.104)(50)(856.4) = 47{,}273$$

$$M_{nc} = (1.30)\left[1 - \left(1 - \frac{(35)(856.4)}{(1.126)(50)(856.4)}\right)\left(\frac{180 - 100.0}{418.1 - 100.0}\right)\right](1.104)(50)(856.4) = 55{,}608$$

Therefore, M_{nc} = 47,273 k-in. = 3,939 k-ft.

Article 6.10.3.2.1 prescribes that the nominal flexural resistance, F_{nc}, can be taken as the M_{nc} determined from Article A6.3.3 divided by S_{xc}.

$$F_{nc} = \frac{3939(12)}{856.4} = 55.2 \text{ ksi}$$

Equation 6.10.3.2.1-2 may now be evaluated as follows.

$$f_{bu} + \frac{1}{3} f_l \le \phi_f F_{nc} = 28.56 + \frac{1}{3}(3.21) \le (1.0)(55.2) \qquad \text{Eq. (6.10.3.2.1-2)}$$

$$29.63 \text{ ksi} \le 55.2 \text{ ksi} \qquad \text{(satisfied)}$$

Thus, the moment capacity of the non-composite section is sufficient to resist the applicable construction loading.

Next, Eq. 6.10.3.2.1-3 is evaluated, which limits the flange stresses due to the construction loads to a maximum of the web bend-buckling stress.

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

The nominal elastic bend-buckling resistance for web, F_{crw}, is determined according to Article 6.10.1.9 of the Specifications.

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

where: k = bend-buckling coefficient

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

D_c is equal to half the web depth, as discussed above =1828 in.

Therefore, k and F_{crw} are equal to the following.

$$k = \frac{9}{\left(18.28/36.56\right)^2} = 36.0$$

$$F_{crw} = \frac{0.9(29{,}000)(36.0)}{\left(\frac{36.56}{0.65}\right)^2} = 297.0 \text{ ksi} \le R_n F_{yc} = 50 \text{ ksi}$$

The requirements of Eq. 6.10.1.9.1-2 are then satisfied.

$$f_{bu} = 28.56\,\text{ksi} \le \phi_f F_{crw} = (1.0)(50) = 50\,\text{ksi} \qquad \text{(satisfied)}$$

For a discretely braced tension flange, Eq. 6.10.3.2.2-1 requires the allowable stress in the tension flange due to the factored loading to be less than the nominal yield strength multiplied by the hybrid factor.

$$f_{bu} + f_l \le \phi_f R_h F_{yt} = 28.56 + 3.21 = 31.77 \le 1.0(1.0)(50) \qquad (6.10.3.2.2\text{-}1)$$

$$f_{bu}\, 28.56 + 3.21 = 31.77 \le 1.0(1.0)(50) = 50.0\,\text{ksi} \qquad \text{(satisfied)}$$

The last constructability requirement to be evaluated is to show that the lateral bending stresses due not exceed 60% of the flange yield strength, which is demonstrated below.

$$f_\ell \le 0.6F_{yf} = 5.38\ \text{ksi} \le 30\ \text{ksi} \qquad \text{(satisfied)}$$

8.2.2.2 Shear

The required shear capacity during construction is specified by Eq. 6.10.3.3-1. The unstiffened shear strength of the beam was previously demonstrated to be sufficient to resist the applied shear under the strength load combination. Therefore, the section will have sufficient strength for the constuctibility check.

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

8.2.3 Service Limit State (Article 6.10.4)

Permanent deformations are controlled under the service limit state, which is specified in Article 6.10.4.

The Service II limit state is intended to permanent deformations that may negatively impact the rideability of the structure by limiting the stresses in the section under expected severe traffic loadings. Specifically, under the Service II load combination, the top flange of composite sections must satisfy

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

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.

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

Under the service limit state, the lateral force effects due to wind-load and deck overhang are not considered. Therefore, for bridges with straight, non-skewed beams such as the present design example the lateral bending forces are zero and Eq. 6.10.4.2.2-2 reduces to Eq. 6.10.4.2.2-1.

Appendix B permits the redistribution of moment at the service load level before evaluating the above equations. Article B6.5.2 specifies the effective plastic moment for the Service Limit State is as follows.

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

$$M_{pe} = \left[2.90 - 2.3\frac{15.8}{1.22}\sqrt{\frac{50}{29000}} - 0.35\frac{36.56}{15.8} + 0.39\frac{15.8}{1.22}\sqrt{\frac{50}{29000}}\frac{36.56}{15.8} \right] 4737 \le 4737$$

$$M_{pe} = 6341 \le 4737$$

$$M_{pe} = 4737 \text{ k-ft}$$

Comparing M_{pe} to the factored moment at the service limit state shows that M_{pe} is greater than the applied moment. Therefore, no moment redistribution occurs at the service limit state.

M_{pe} = 4737 k-ft. > M_u = 4078 k-ft. Therefore, no redistribution at Service II.

The evaluation of the design then precedes with determining the flange stresses under Service II, which are computed using the following equation.

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

As permitted by Article 6.10.4.2.1, since shear connectors are provided throughout the span length, the stresses in the member as a result of the Service II load combination are computed assuming the concrete slab is fully effective in both the positive and negative bending region.

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

$$f_f = \frac{(1359)(12)}{856.4} + \frac{(263 + 216)(12)}{1070} + \frac{1.3(1723)(12)}{1171} = 47.37\text{ksi}$$

Then comparing this stress to the allowable stress shows that Equation 6.10.4.2.2-1 is satisfied within an acceptable tolerance; the applied stress and the stress limit differ by less than one percent.

f_f = 47.37 ksi ≤ $0.95R_hF_{yf}$ = 0.95(1.0)(50) = 47.5 ksi (satisfied)

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

$$f_f = \frac{(1359)(12)}{856.4} + \frac{(263 + 216)(12)}{2547} + \frac{1.3(1723)(12)}{7905} = 24.17\text{ksi}$$

Thus, it is also demonstated that Equation 6.10.4.2.2-1 is satisfied for the tension flange.

$$f_f = 24.17 \text{ ksi} \le 0.95R_hF_{yf} = 0.95(1.0)(50) = 47.5 \text{ ksi} \qquad \text{(satisfied)}$$

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

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

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

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

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

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

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

The depth of web in compression must be known in order to employ Equation 6.10.1.9.1-2. Thus, D_c is calculated using the method described in Article D6.3.1, which states Equation D6.3.1-1 is to be used when checking composite sections in negative flexure at the service limit state.

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

where: f_t = the sum of the tension-flange stresses caused by the various loads applied to the applicable cross-sections (ksi)

d = depth of steel section (in.)

$$D_c = \left[\frac{47.37}{47.37 + 24.17}\right]39.0 - 1.22 = 24.40$$

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

$$k = \frac{9}{(24.60/36.56)^2} = 19.88$$

$$F_{crw} = \frac{0.9(29{,}000)(19.88)}{\left(\frac{36.56}{0.65}\right)^2} = 164.0\ \text{ksi} < R_h\ F_{yc} = 50\ \text{ksi}$$

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

$$f_c = 47.37\ \text{ksi} < F_{crw} = 50.0\ \text{ksi}$$

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 fatigue load combination specified in Table 3.4.1-1 and the fatigue live load specified in Article 3.6.1.4. The fracture toughness requirements in Article 6.10.5.2 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 these checks is taken as twice the result of the fatigue load combination.

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 fatigue resistance. The fatigue resistance $(\Delta F)_n$ varies based on the fatigue category to which a particular member or detail belongs 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 = constant from Table 6.6.1.2.5-1

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

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

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

For this example infinite fatigue life is desired, and thus the Fatigue I Load combination and Eq. (6.6.1.2.5-1) are considered.

The fatigue resistance due to negative bending stresses is checked at the critical location, which is at the weld joining the lateral bracing connection plate to the flanges of the beam at the cross-frame 15 feet from the pier. From Table 6.6.1.2.3-1, it is determined that this detail is classified as fatigue category C'. The constant-amplitude fatigue threshold for a category C' detail is 12 ksi (see Table 6.6.1.2.5-3).

The permissible stress range is then computed according to Equation 6.6.1.2.5-1.

$$(\Delta F)_n = (\Delta F)_{TH} = 12.00\text{ksi}$$

The applied stress range is taken as the stress range resulting from the fatigue loading (shown in Figure 9), with a dynamic load allowance of 15 percent applied, and distributed laterally by the previously calculated distribution factor for fatigue. At the bottom of the top flange the applied stress range is computed as follows.

$$\gamma(\Delta f) = (1.50)\left[\frac{(178)(12)(4.52-1.22)}{40{,}371} + \frac{|-267|(12)(4.52-1.22)}{40{,}371}\right]$$

$$\gamma(\Delta f) = 0.65 \text{ ksi} \leq (\Delta F)_n = 12.00 \text{ ksi} \qquad \text{(satisfied)}$$

Similarly, the applied stress range at the top of the bottom flange is equal to the following.

$$\gamma(\Delta f) = (1.50)\left[\frac{(178)(12)(34.48-1.22)}{40{,}371} + \frac{|-267|(12)(34.48-1.22)}{40{,}371}\right]$$

$$\gamma(\Delta f) = 6.60 \text{ ksi} \leq (\Delta F)_n = 12.00 \text{ ksi} \qquad \text{(satisfied)}$$

Therefore, it is demonstrated that the applied stress range in the top and bottom flanges is acceptable.

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)

The appropriate Charpy V-notch fracture toughness, found in Table 6.6.2-2, must be specified for main load-carrying components subjected to tensile stress under Strength I load combination.

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 fatigue loading must be less than the shear-buckling resistance of 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: Positive Bending

8.3.1 Strength Limit State

8.3.1.1 Flexure (Article 6.10.6.2)

Equation 6.10.7.1.1-1 must be satisfied for positive bending sections at the strength limit state.

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

8.3.1.1.1 Flexural Resistance (6.10.7)

To calculate the flexural capacity, the classification of the section must be determined. The following requirements must be satisfied for a section to qualify as compact:

$F_y = 50$ ksi $\le$ 70 ksi (satisfied)

$$\frac{D}{t_w} = \frac{36.56}{0.65} = 56.25 \le 150 \qquad \text{(satisfied)}$$

$$\frac{2D_{cp}}{t_w} = \frac{2(0)}{0.4375} = 0 \le 3.76\sqrt{\frac{E}{F_{yc}}} = 3.76\sqrt{\frac{29{,}000}{50}} = 90.55 \qquad \text{Eq (10.6.2.2-1) (satisfied)}$$

Therefore, the section is compact, and the nominal flexural resistance is based on Article 6.10.7.1.2, where the moment capacity of beams satisfying $D_p \le 0.1D_t$ is given by Eq. 6.10.7.1.2-1 and Eq. 6.10.7.1.2-2. for those 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 location previously computed is utilized in these calculations.

$D_p = 8 + 2 - 1.22 + 0.22 = 9.00$ in.

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

$D_t = 8 + 2 + 36.56 + 1.22 = 47.78$ in.

Therefore, D_p is greater than 10% of D_t as computed below and the nominal flexural capacity is therefore determined using Equation 6.10.7.1.2-2.

$D_p = 9.00 > 0.1D_t = 0.1(47.78) = 4.78$ (not satisfied)

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

$$M_n = 6074\left(1.07 - 0.7\frac{9.00}{47.78}\right) = 5698\,\text{k-ft}$$

8.3.1.1.2 Factored Positive Bending Moment

In order to determine if the above determined moment resistance of 5,698 k-ft is adequate, the maximum value of $(M_u + f_l S_{xt}/3)$ must be determined, according to Eq. 6.10.7.1.1-2. Therefore the value of $(M_u + f_l S_{xt}/3)$ resulting from each of the five strength load combinations is now computed. As previously discussed during the evaluation of the negative bending capacity of the beam, the load factors applicable to this design example are as follows for each load combination.

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

Strength II = 1.25DC + 1.5DW + 1.35(LL+I)

Strength III = 1.25DC + 1.5DW + 1.4WS

Strength IV = 1.5(DC + DW)

Strength V = 1.25DC + 1.5DW + 1.35(LL+I) + 0.4WS

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 = 761 +147 = 908 k-ft

DW = 121 k-ft

From Table 3, the controlling LL+I moment is 1664 k-ft.

LL+I = 1664 k-ft

The unfactored moment ($f_\ell S_x$) due to wind load loads was previously determined to be 1.828 k-ft.

M_w = 1.828 k-ft

Consideration should also be given to increasing the wind load moments to account for second-order force effects, as specified in Article 6.10.1.6 through application of the amplification factor. However, no increase is required for tension flanges and the compression flange is continuously braced in positive bending, so the amplification factor is negligible in this case.

Lateral bending forces due to the wind loading are then determined by dividing M_w by the section modulus of the bottom flange.

$$f_l = \frac{M_w}{S_\ell} = \frac{(1.828)12}{(15.8)^2(1.22)/6} = 0.432\,\text{ksi}$$

It is required that the flange lateral bending stresses may not exceed 60% of the flange yield strength. Thus, for this example f_ℓ must be less than or equal to 30 ksi, which is easily satisfied for the above lateral stress. The maximum lateral stress is obtained by multiplying f_ℓ by the maximum wind load factor of 1.4, which is less than the allowable stress of 30 ksi.

$$(1.4)(0.432) = 0.6048 < 30 \qquad \text{(satisfied)}$$

The controlling strength limit state can now be determined based on the above information. For the Strength I load combination, the design moments are as follows:

$$M_u = 1.25(908) + 1.5(121) + 1.75(1664) = 4229 \text{ k-ft}$$

$$M_u + \frac{1}{3} f_\ell S_{xc} = 4229 + 0 \qquad \text{(wind loads not considered)}$$

$$M_u + \frac{1}{3} f_\ell S_{xc} = 4229 \text{ k-ft} \qquad \text{(governs)}$$

It is obvious that the design moments for the Strength II load combination will be less than those at the Strength I load combination as all of the Strength II load factors are either equal to or less than those used for the Strength I combination. The design moments at Strength II are equal to the following.

$$M_u = 1.25(908) + 1.5(121) + 1.35(1664) = 3563 \text{ k-ft}$$

$$M_u + \frac{1}{3} f_\ell S_{xc} = 3563 + 0 \qquad \text{(wind loads not considered)}$$

$$M_u + \frac{1}{3} f_\ell S_{xc} = 3563 \text{ k-ft}$$

For the Strength III load combination, wind load is incorporated and the design moments are equal to the following.

$$M_u = 1.25(908) + 1.5(121) = 1317 \text{ k-ft}$$

$$M_u + \frac{1}{3} f_\ell S_{xc} = 1317 + (1/3)(1.828)(1.4)$$

$$M_u + \frac{1}{3} f_\ell S_{xc} = 1317 \text{ k-ft}$$

The design moments for the Strength IV load combination are as follows.

$$M_u = 1.5(908 + 121) = 1544 \text{ k-ft}$$

$$M_u + \frac{1}{3} f_\ell S_{xc} = 1544 + 0 \qquad \text{(wind loads not considered)}$$

$$M_u + \frac{1}{3} f_\ell S_{xc} = 1544 \text{ k-ft}$$

Lastly, the design moments computed using the Strength V load combination are equal to the following.

$$M_u = 1.25(908) + 1.5(121) + 1.35(1664) = 3563 \text{ k-ft}$$

$$M_u + \frac{1}{3} f_\ell S_{xc} = 3563 + (1/3)(1.828)(0.4)$$

$$M_u + \frac{1}{3} f_\ell S_{xc} = 3563 \text{ k-ft}$$

Reviewing the factored moments for each load combination computed above, it is determined that the Strength I moments govern for this example and that the design moment is equal to 4229 k-ft.

8.3.1.1.3 Redistribution Moment

The redistribution moment must then be added to the above elastic moments. It was previously determined that the redistribution moment at the pier at the (governing) Strength I load combination is equal to 630 k-ft. Because the redistribution moment varies linearly from zero at the end-supports to a maximum at the interior pier, the redistribution moment at 36 ft from the abutment is simply computed as follows.

$$M_{rd} = \left(\frac{36}{90}\right)(630) = 0.4(630) = 252 \text{ k-ft.}$$

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

$$M_u = 4229 + 252 = 4481 \text{ k-ft.}$$

8.3.1.1.4 Flexural Capacity Check

The design moment of 4481 k-ft is then compared to the factored resistance of 5687 k-ft, which shows that the positive bending capacity of the beam is sufficient.

$M_u = 4481$ k-ft $\leq \phi_f M_n = (1.0)(5698) = 5698$ k-ft (satisfied)

8.3.1.1.5 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 crushing of the concrete slab.

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

Dp = 9.00 in. ≤ 0.42(47.78) = 20.07 in. (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 Constructibility (Article 6.10.3)

The constructibility of the system due to positive bending moments will be evaluated in a manner similar to the constructibility evaluation previously performed for the negative bending region of the beam. Because the shear requirements during construction are automatically satisfied for beams with unstiffened webs, only the evaluation of the flexural requirements is presented herein.

The design checks in this section will make use of the deck casting moments previously presented in Table 10 as well as the deck overhang loads presented in preceeding sections. Specifically, the vertical bending stress due to the deck casting loads is computed based on dividing the maximum positive bending moment from Table 10 by the section modulus of the section. 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.

$$f_{bu} = \frac{(888)(12)}{856.4} = 12.44\,\text{ksi}$$

The lateral moment due to the component (overhang) dead load is equal to the following.

$$M_l = \frac{(.207)(30)^2}{12} = 15.53\,\text{k-ft}$$

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.

$$f_l = \frac{M_\ell}{S_\ell} = \frac{15.53(12)}{1.22(15.8)^2/6} = 3.67\ ksi$$

Similarly, the lateral moment and lateral bending stress due to the construction dead loads are computed as follows.

$$M_l = \frac{(.275)(30)^2}{12} = 20.63\,\text{k-ft}$$

$$f_l = \frac{M_\ell}{S_\ell} = \frac{20.63(12)}{1.22(15.8)^2/6} = 4.88\,\text{ksi}$$

The unfactored lateral bending moment and lateral bending stress due to the finishing machine are equal to the following.

$$M_l = \frac{(3)(30)}{8} = 11.25\,\text{k-ft}$$

$$f_l = \frac{M_\ell}{S_\ell} = \frac{11.25(12)}{1.22(15.8)^2/6} = 2.66\,\text{ksi}$$

It must then be determined if these first-order lateral bending stresses are applicable, or if these stresses must be increased to account for second-order effects. As discussed previously, the first-order lateral bending stress may be used if the following limit is satisfied.

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

From previous computations in Section 8.2.2.1.2., L_p is equal to 100.0 inches.

L_p = 100.0 in.

As previously described, the moment gradient modifier is determined from the following equation.

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

The maximum positive bending stresses due to the deck casting occur at 36 ft from the pier. Thus, the critical lateral bracing segment is the lateral bracing panel that begins at 30 ft from the pier and ends at 60 ft from the pier. The applicable moment values for this lateral bracing segment are given below.

M_2	= 849 k-ft.	M_1	$= 2M_{mid} - M_2 \ge M_0$
M_0	= 502 k-ft.	M_1	$= 2(838) - (849) = 827 \ge 502$
M_{mid}	= 838 k-ft.	M_1	= 827 k-ft.

Thus, C_b is computed as follows.

$$C_b = 1.75 - 1.05\left(\frac{M_1}{M_2}\right) + 0.3\left(\frac{M_1}{M_2}\right)^2 \le 2.3$$

$$C_b = 1.75 - 1.05\left(\frac{827}{849}\right) + 0.3\left(\frac{827}{849}\right)^2 = 1.01$$

According to Article 6.10.1.10.2, the web load-shedding factor, R_b, is 1.0 for constructability evaluations.

It was determined that the largest compressive stress due to the vertical bending moment was equal to 12.44 ksi. Multiplying this stress by the two applicable load factors of 1.25 and 1.5 shows that f_{bm} is equal to 15.55 or 18.66 ksi.

$$f_{bm} = (1.25)(12.44) = 15.55 \text{ ksi}$$

$$f_{bm} = (1.50)(12.44) = 18.66 \text{ ksi}$$

Lastly, F_{yc} is equal to 50 ksi. The information required for evaluation of Eq. 6.10.1.6-2 is now known. It is shown below that regardless of the dead load factor used, the first-order elastic analysis is not applicable.

$$L_b = 30 \text{ ft.} = 360 \text{ in.} > 1.2(100.0)\sqrt{\frac{(1.01)(1.0)}{15.55/50}} = 216.2 \qquad \text{(not satisfied)}$$

$$L_b = 30 \text{ ft.} = 360 \text{ in.} > 1.2(100.0)\sqrt{\frac{(1.01)(1.0)}{18.66/50}} = 197.4 \qquad \text{(not satisfied)}$$

Therefore, the second-order compression flange lateral bending stresses are calculated using Eq. 6.10.1.6-4.

$$f_\ell = \left(\frac{0.85}{1 - \dfrac{f_{bm}}{F_{cr}}}\right) f_{\ell 1} \ge f_{\ell 1} \qquad (6.10.1.6\text{-}4)$$

To calculate the amplification factor (the term in bracket in Equation 6.10.1.6-4), the elastic lateral torsional buckling stress, F_{cr}, must be determined, which can be calculated from Appendix A or Section 6.10.8. As discussed above, Appendix A is applicable if the flange nominal yield strength is less than or equal to 70 ksi and the web is classified as either compact or non-compact, i.e., Equation A6.1.-1 is satisfied. The following calculations demonstrate that Appendix A is applicable.

$$F_{yf} = 50 \text{ ksi} \le 70 \text{ ksi} \qquad \text{(satisfied)}$$

$$\frac{2D_c}{t_w} < 5.7\sqrt{\frac{E}{F_{yc}}} = 5.7\sqrt{\frac{(29{,}000)}{(50)}} = 137.27 \qquad \text{(A6.1-1)}$$

D_c = 36.56/2 = 18.28 in.

$$\frac{2D_c}{t_w} = \frac{2(18.28)}{(0.65)} = 56.25 \qquad \text{(satisfied)}$$

Since Appendix A is applicable, the elastic lateral torsional buckling stress is determined from Eq. A6.3.3-8.

$$F_{ccr} = \frac{C_b \pi^2 E}{\left(L_b/r_t\right)^2}\sqrt{1 + 0.0779\frac{J}{S_{xc}h}\left(\frac{L_b}{r_t}\right)^2} \qquad \text{Eq. (A6.3.3-8)}$$

At this point, all of the parameters necessary for computing F_{cr} using Eq. A6.3.3-8 are known. Substituting these previously determined quantities into Eq. A6.3.3-8 gives that the elastic lateral torsional buckling stress is equal to 45.44 ksi.

$$F_{cr} = \frac{(1.01)\pi^2(29{,}000)}{\left(360/4.154\right)^2}\sqrt{1 + 0.0779\frac{22.47}{(856.4)(39.0)}(360/4.154)^2} = 45.44\,\text{ksi}$$

Therefore, when a dead load factor of 1.25 is used, the amplification factor is equal to the following.

$$\text{AF} = \left(\frac{0.85}{1 - 15.55/45.44}\right) = 1.292 > 1.0$$

Similarly the amplification factor, when a dead load factor of 1.50 is used, is as follows.

$$\text{AF} = \left(\frac{0.85}{1 - 18.66/45.44}\right) = 1.442 > 1.0$$

With the applied stresses and applicable amplification factors known, the controlling load combinations can now be determined. Recall that, during construction, both the compression and tension flanges are discretely braced. Therefore, Article 6.10.3.2 requires the compression flange of 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 ensure 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. Additionally, Eq. 6.10.3.2.2-1 must be satisfied for discretely braced tension flanges, which limits the total flange stress to the yield stress of the tension flange.

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

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

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

$$f_{bu} + f_l \leq \phi_f R_h F_{yt} \quad (6.10.3.2.2\text{-}1)$$

Lastly, the lateral bending stress in both flanges is limited to a maximum of 60% of the flange yield strength according to Article 6.10.1.6.

$$f_l \leq 0.6\mathrm{F_y}$$

Therefore, the next step in evaluating the constructability requirements in positive bending is to determine the load combination which produces the maximum values of f_{bu}, $f_{bu} + f_l/3$, $f_{bu} + f_l$, and f_l for the compression flange. The value of each of these quantities at the Strength I load combination is first determined, where a load factor of 1.25 is applied to the component dead load. Furthermore, a load factor of 1.5 is applied to construction loads for all load combinations.

$$f_{bu} = 1.25(12.44)$$

$$f_{bu} = 15.55 \text{ ksi}$$

$$f_l = 1.292*[1.25(3.67) + 1.5(4.88) + 1.5(2.66)]$$

$$f_\ell = 20.54 \text{ ksi}$$

$$f_{bu} + f_l = 15.55 + 20.54$$

$$f_{bu} + f_l = 36.09 \text{ ksi}$$

$$f_{bu} + f_l/3 = 15.55 + (20.54)/3$$

$$f_{bu} + f_l/3 = 22.40 \text{ ksi}$$

The Strength II load combination will not be evaluated in detail as the load factors for this load combination are either equal to or less than those for the Strength I load combination. Thus, it is obvious that the Strength II load combination will not govern.

The load factors for the Strength III load combination are 1.25 for component dead load and 1.4 for wind load. Live loads are not included in the Strength III load combination. The stress values at the Strength III load combination that are needed for evaluating the constructability requirements are as follows.

$$f_{bu} = 1.25(12.44)$$

$$f_{bu} = 15.55 \text{ ksi}$$

f_l = 1.292*[1.25(3.67) + 1.5(4.88) + 1.4(0.43)]

f_l = 16.16 ksi

$f_{bu} + f_l$ = 15.55 + 16.16

$f_{bu} + f_l$ = 31.71 ksi

$f_{bu} + f_l/3$ = 15.55 + (16.16)/3

$f_{bu} + f_l/3$ = 20.94 ksi

Only dead loads are considered at the Strength IV load combination where the load factor for all dead loads under this load combination is equal to 1.5.

f_{bu} = 1.5(12.44)

f_{bu} = 18.66 ksi (governs)

f_l = 1.442*[1.5(3.67) + 1.5(4.88)]

f_l = 18.49 ksi

$f_{bu} + f_l$ = 18.66 + 18.49

$f_{bu} + f_l$ = 37.15 ksi (governs)

$f_{bu} + f_l/3$ = 18.66 + (18.49)/3

$f_{bu} + f_l/3$ = 24.82 ksi (governs)

Lastly, at the Strength V load combination the stresses are computed due to load factors of 1.25 for component dead load and 0.4 for wind load.

f_{bu} = 1.25(12.44)

f_{bu} = 15.55 ksi

f_l = (1.292)[1.25(3.67) + 1.5(4.88) + 1.5(2.66) + 0.4(0.43)] (governs)
=20.76 ksi

$f_b + f_l$ = 15.55 + 20.76

$f_b + f_l$ = 36.31 ksi

$f_{bu} + f_l/3$ = 15.55 + (20.76)/3

$f_{bu} + f_l/3$ = 22.47 ksi

Reviewing the computations above for each of the load combinations, the following are determined to be the controlling stresses.

f_{bu} = 18.66 ksi (Strength IV)

f_l = 20.76 ksi (Strength V)

$f_{bu} + f_l$ = 37.15 ksi (Strength IV)

$f_{bu} + f_l/3$ = 24.82 ksi (Strength IV)

Now that the controlling stresses are known the constructability requirements are evaluated, beginning with the check of Equation 6.10.3.2.1-1, which limits the stress in the compression flange to the nominal yield strength of the flange multiplied by the hybrid factor (Equation 6.10.3.2.1-1). For homogeneous material sections the hybrid factor is 1.0, as stated in Article 6.10.1.10.1.

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

$37.15 \leq (1.0)(1.0)(50) = 50$ ksi (satisfied)

The flexural resistance of the noncomposite section is required to be larger than the maximum bending moment as a result of the deck casting sequence plus one-third of the lateral bending stresses, as expressed by Equation 6.10.3.2.1-2.

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

According to Article 6.10.3.2.1, the flexural resistance, F_{nc}, is determined as specified in Article 6.10.8.2 or Article A6.3.3, if applicable. It was demonstrated above that Appendix A is applicable for the non-composite, negative bending section. Due to symmetry of the non-composite section, this means that Appendix A is also applicable for the non-composite positive bending section. The applicable moment capacity prediction equation is then based on the lateral bracing classification. If the following equation is satisfied the lateral brace spacing is classified as compact.

$L_b \leq L_p$

L_p of the non-composite section is equal to 100.0 inches according to previous calculations. Therefore, the lateral bracing distance is not compact.

$360 > 100$ (not satisfied)

The following inequality is then evaluated to determine if the unbraced distance is classified as non-compact.

$L_b \leq L_r$

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

Again, because the non-composite section is symmetric, the L_r value is equal regardless of whether the section is in negative or positive bending.

L_r = 418.1 in.

$L_b = 360 \leq L_r = 418.1$ (satisfied)

Because the lateral bracing distance is non-compact, the lateral torsional buckling resistance is controlled by Eq. A6.3.3-2 of the specifications.

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

Substituting the applicable values into Eq. A6.3.3-2 gives the following.

$$M_{nc} = (1.01) \left[1 - \left(1 - \frac{(35)(856.4)}{(1.00)(50)(856.4)} \right) \left(\frac{360 - 100.0}{418.1 - 100.0} \right) \right] (1.104)(50)(856.4) \leq (1.104)(50)(856.4)$$

$M_{nc} = 36{,}042 \leq 47{,}273$

M_{nc} = 36,042 k-in. = 3,004 k-ft.

Article 6.10.3.2.1 prescribes that the nominal flexural resistance, F_{nc}, can be taken as the M_{nc} determined from Article A6.3.3 divided by S_{xc}.

$$F_{nc} = \frac{3{,}004(12)}{856.4} = 42.09 \text{ ksi}$$

Equation 6.10.3.2.1-2 may now be evaluated as follows.

$$f_{bu} + \frac{1}{3} f_l \leq \phi_f F_{nc} \quad \text{(6.10.3.2.1-2)}$$

24.82 ksi ≤ (1.0)(42.09) = 42.09 ksi (satisfied)

Thus, the moment capacity of the non-composite section is sufficient to resist the applicable construction loading.

Next, Eq. 6.10.3.2.1-3 is evaluated, which limits the flange stresses due to the construction loads to a maximum of the web bend-buckling stress.

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

The nominal elastic bend-buckling resistance for web, F_{crw}, was previously determined according to Article 6.10.1.9 of the Specifications.

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

F_{crw} = 297.0 ksi > 50.0 ksi, therefore F_{crw} = 50.0 ksi,

$$f_{bu} = 18.66\text{ ksi} \leq \phi_f F_{crw} = (1.0)(50) = 50\text{ ksi} \qquad \text{(satisfied)}$$

The last constructability requirement to be evaluated for the compression flange is to show that the lateral bending stresses due not exceed 60% of the flange yield strength, which is demonstrated below.

$$f_\ell \leq 0.6F_{yf}$$

$$20.59\text{ ksi} \leq 30\text{ ksi} \text{ (satisfied)}$$

For a discretely braced tension flange, Eq. 6.10.3.2.2-1 requires the stress in the tension flange due to the factored loading to be less than the nominal yield strength multiplied by the hybrid factor.

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

Because the amplification factor is equal to one for tension flanges, f_l for the tension flanges is found by dividing the previously determined values of f_l for the compression flanges by the amplification factors. The sum of f_{bu} and f_l at each of the strength load combinations is then as follows.

$(f_{bu} + f_l)_{\text{Strength I}}$ = 15.55 + 20.54/1.292 = 31.45 ksi

$(f_{bu} + f_l)_{\text{Strength II}}$ = NA

$(f_{bu} + f_l)_{\text{Strength III}}$ = 15.55 + 16.16/1.292 = 28.06 ksi

$(f_{bu} + f_l)_{\text{Strength IV}}$ = 18.66 + 18.49/1.442 = 31.48 ksi

$(f_{bu} + f_l)_{\text{Strength V}}$ = 15.55 + 20.76/1.292 = 31.62 ksi (governs)

Comparing $f_{bu} + f_l$ to the allowable stress shows that the resistance of the tension flange is adequate.

$$31.62 \leq (1.0)(1.0)(50) = 50\text{ ksi} \qquad \text{(satisfied)}$$

The lateral bending stresses in the tension flange must also be less than 60% of the flange yield strength. Reviewing the computations above it is determined by inspection that the controlling value of f_l will be obtained at the Strength V load combination.

$$f_\ell \leq 0.6F_{yf}$$

$$f_\ell = 20.76/1.292 = 16.07 \text{ ksi} \leq 30 \text{ ksi} \qquad \text{(satisfied)}$$

8.3.3 Service Limit State

Similar to the negative bending section, the positive bending section must be evaluated for permanent deformations, which are governed by Eq. 6.10.4.2.2-1.

$$F_f \leq 0.95R_hF_{yf} \qquad (6.10.4.2.2\text{-}1)$$

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

The stress in the compression flange at the critical positive bending location is then computed as follows based on the moment values given in Tables 2 and 3.

$$f_f = \frac{(761)(12)}{856.4} + \frac{(147+121)(12)}{2716} + \frac{1.3(1664)(12)}{8932} = 14.75\,\text{ksi}$$

Thus, the requirements of Eq. 6.10.4.2.2-1 are satisfied for the compression flange.

$$f_f = 14.75\,\text{ksi} \leq 0.95R_hF_{yf} = 0.95(1.0)(50) = 47.50\text{ksi} \qquad \text{(satisfied)}$$

Similarly, the stress in the tension flange is as follows.

$$f_f = \frac{(761)(12)}{856.4} + \frac{(147+121)(12)}{1070} + \frac{1.3(1664)(12)}{1171} = 35.84\,\text{ksi}$$

Thus, the service requirements for the tension flange are also satisfied.

$$f_f = 35.84\,\text{ksi} \leq 0.95R_hF_{yf} = 0.95(1.0)(50) = 47.50\text{ksi}$$

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 a welded lateral bracing connection plate for the cross frame located 60 feet from the abutment. It was previously determined that the allowable stress range for this detail was equal to 12.0 ksi, which was computed using Equation 6.6.1.2.5-1.

The permissible stress range is computed using Equation 6.6.1.2.5-1, for inifinite fatigue life is:

$$(\Delta F)_n = (\Delta F)_{TH} = 12.00\text{ksi} \qquad \text{Eq. (6.6.1.2.5-1)}$$

The stress range at the bottom of top flange is computed below for the Fatigue I load combination.

$$\gamma(\Delta f) = (1.50)\left[\frac{(492)(12)(4.52-1.22)}{40{,}371} + \frac{|-107|(12)(4.52-1.22)}{40{,}371}\right]$$

$$\gamma(\Delta f) = 0.88 \text{ ksi} \leq (\Delta F)_n = 12.00 \text{ ksi} \qquad \text{(satisfied)}$$

Similarly, the stress range at the top of the bottom flange is computed below.

$$\gamma(\Delta f) = (1.50)\left[\frac{(492)(12)(34.48-1.22)}{40{,}371} + \frac{|-107|(12)(34.48-1.22)}{40{,}371}\right]$$

$$\gamma(\Delta f) = 8.87 \text{ ksi} \leq (\Delta F)_n = 12.00 \text{ ksi} \qquad \text{(satisfied)}$$

8.3.4.2 Special Fatigue Requirement for Webs (Article 6.10.5.3)

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 is not explicitly evaluated. It has been demonstrated that the beam satisfies all design requirements.

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).

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

8.4.1 Effective Length (Article 9.7.2.3)

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

$$L_{eff} = (10.0)(12.0) - (12.0)\left(\frac{12 - 0.4375}{2}\right) = 112.78\text{in.} < 162.0\text{in.} \qquad \text{(satisfied)}$$

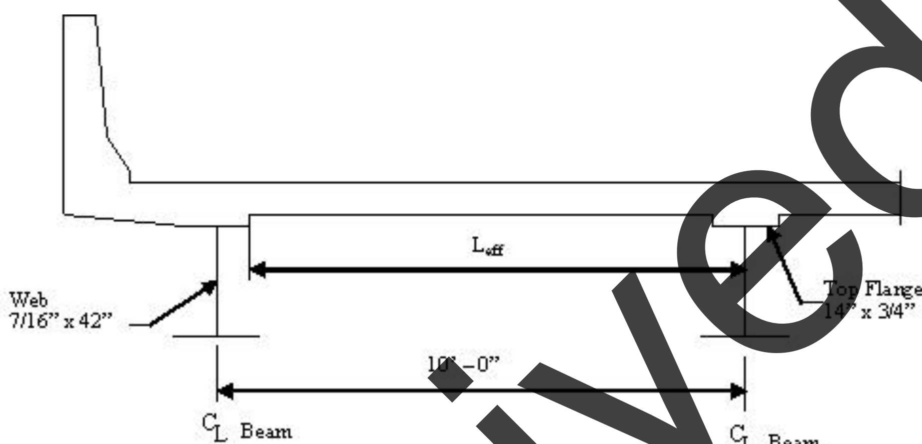

Figure 13 Effective Slab Length for Deck Design

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 be greater or equal to the specified 28 day compressive strength of 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 shall be provided in the negative moment region of continuous steel super structures. In addition the following specifications are also provided:

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

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

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

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

$$\frac{112.78}{8.0} = 14.10 < 18.0 \qquad \text{(satisfied)}$$

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 is not 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 in. > 4.0 in. (satisfied)

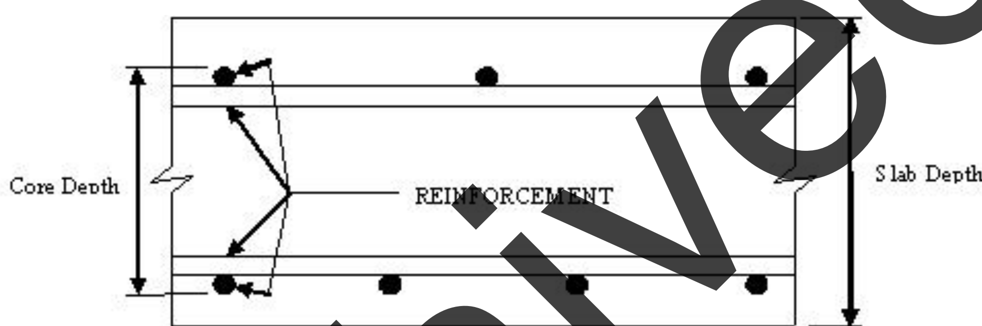

Figure 14 Core of the Concrete Slab

8.4.3 Positive Flexure Reinforcement Requirements

Article 9.7.2.5 specifies the use of 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 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:

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

Use 12 inch spacing to match that of the negative flexure region as shown later.

8.4.3.2 Bottom Layer (Longitudinal and Transverse)

Bottom layers of reinforcement are required to have a minimum reinforcement 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:

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

Therefore, use 12 inch spacing in both of the bottom layers to match that of the negative flexure region as shown later.

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 shall not 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 shall have a minimum yield strength no less than 60 ksi. and a size not exceeding No. 6 bars. The bars are to 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 is not to exceed 12.0 inch spacing center to center. Since shear connectors are provided the longitudinal reinforcement must be extended beyond the additional shear connectors as stated in Article 6.10.1.7.

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

$$A_{reinf} = \frac{0.44}{12} + \frac{0.31}{12} = 0.06\text{in.}^2/\text{in.} > 0.05\text{in.}^2/\text{in.} \qquad \text{(satisfied)}$$

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

$$A_{reinf} = \frac{0.31}{12} + \frac{0.20}{12} = 0.04\text{in.}^2/\text{in.} > 0.03\text{in.}^2/\text{in.} \qquad \text{(satisfied)}$$

8.4.4.3 Top and Bottom Layer (Transverse)

The transverse reinforcing steel in both the top and bottom layers will be No. 5 bar at 12 inch spacing, the same as the positive flexure regions, as shown in Figure 15.

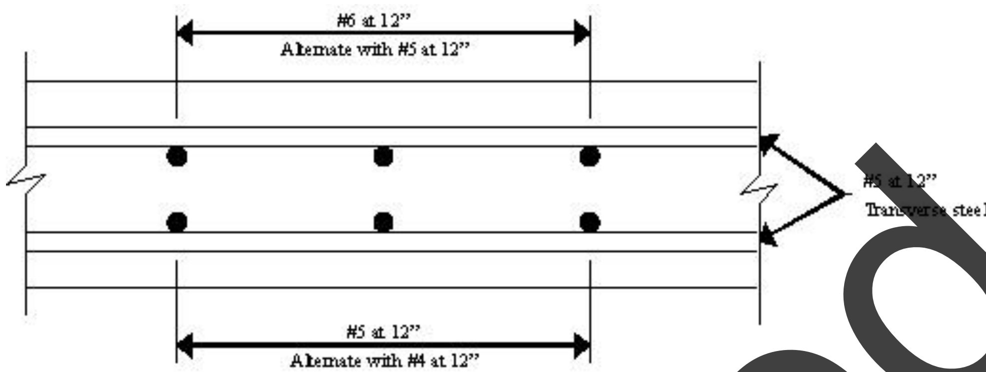

Figure 15 Deck Slab in Negative Flexure Region of the Beam

9.0 REFERENCES

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