EVALUATION OF SMDI SHORT SPAN STEEL BRIDGE DESIGN STANDARDS

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ABSTRACT

The Federal Highway Administration's National Bridge Inventory consists of over 600,000 bridges. Of these bridges, over 25% are considered either structurally deficient or functionally obsolete. While several state bridge departments have standard designs for bridge components in order to speed up the design process in replacing these bridges, few have standard designs for the bridge superstructure.

Standard short-span steel bridge designs were developed to create a design aid for bridge engineers. In this package, designs with spans ranging from 40 feet to 140 feet in 5 foot increments were developed for rolled wide flange beams, homogeneous steel plate girder sections and hybrid steel plate girder sections. The rolled sections were designed using two design approaches: the lightest weight possible and the lightest weight possible with a limited section depth. This limited suite provides the opportunity for stock piling common rolled beam sections and common steel plate sizes. In utilizing this design aid, a more efficient transition from design to construction can be achieved.

Contained in this report is a design evaluation of two sample bridges. Both of these bridges were 80-foot simple span bridges with cross-sections capable of carrying two traffic lanes. One of the bridges evaluated utilized homogeneous plate girders for the steel superstructure whereas the other utilized rolled beams. Girders for these bridges were selected from the standard designs and evaluated according to current AASHTO LRFD Specifications. Both design options are shown to perform adequately under AASHTO Specifications.

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CHAPTER 1: INTRODUCTION

1.1 OVERVIEW OF DESIGN STANDARDS

The goal of this work is to develop a set of standardized designs that increase the efficiency of short span steel bridge design. This set of designs, the Steel Market Development Institute (SMDI) Short Span Design Standards, was developed based on optimized girder designs. Efforts have been made in this design package to include technical feedback from all aspects of the steel construction market, from accounting for availability of structural steel to fabrication and erection issues.

There are four major sets of bridge designs in this work: homogeneous steel plate girder sections, hybrid steel plate girder sections, "Lightest Weight" rolled beam sections, and "Limited Depth" rolled beam sections. The girder designs were designed according to current AASHTO LRFD Specifications (2014).

It should be noted that in this design suite, once the girders were optimized for traditional plate size selections, a design review, which is presented herein, was conducted by evaluating limit state checks by reducing the respective flange width's by ¼ inch. This was performed to aid in potential fabrication practices that could optimize nesting of plates even though it is understood this ¼ inch reduction is not typical of conventional design practice.

1.2 DESIGN METHODS

Girders in this design package were designed for a variety of span configurations and bridge cross-sections. Specifically, girders were designed to accommodate span lengths from 40 feet up to 140 feet, in 5-foot increments. For each of these span lengths, girders were designed for girder spacings of 6.0 feet, 7.5 feet, 9.0 feet, and 10.5 feet, respectively. These designs, along with typical details (diaphragms, bearings, shear stud layouts, etc.) have been included in the SMDI Detail design package.

Four different steel girder options are presented in this design package. These options are as follows:

- Homogeneous Plate Girders
    - Plate girders incorporating A709-50W steel (F_y = 50 ksi) were designed in order to achieve the lightest weight possible and are provided for a span range of 60’-140’. In many cases, flange transitions are included in order to achieve optimum economy (see Figure 3.1). All dimensions have been included in the SMDI Details. It should be noted that all of the dimensions shown in the Details are nominal. For design calculations, the widths of all plates shown have been reduced by ¼” to account for burn tolerances and optimal nesting during fabrication.
- Hybrid Plate Girders
    - Hybrid plate girders were designed in order to achieve the lightest weight possible and are provided for a span range of 80’-140’. Hybrid girders were designed incorporating A709-50W steel (F_y = 50 ksi) and A709-70W steel (F_y = 70 ksi); 50-ksi steel was employed in the top flanges and webs whereas 70-ksi steel was employed in the bottom flanges. In many cases, as with homogeneous plate girders, flange transitions are included in order to achieve optimum economy. All dimensions have been included in the SMDI Details. It should be noted that, as with homogeneous plate girders, all of the dimensions shown in the Details are nominal.
- “Lightest Weight” Rolled Beams
    - Standard rolled shapes were designed incorporating A709-50W steel (F_y = 50 ksi) in order to achieve the lightest weight possible and are provided for a span range of 40’-100’.
- “Limited Depth” Rolled Beams
    - Standard rolled shapes were designed incorporating A709-50W steel (F_y = 50 ksi) in order to achieve a target span-to-depth ratio of 25 and are provided for a span range of 40’-100’.

1.3 OVERVIEW OF DESIGN EVALUATION

Contained in this report is a design evaluation of two sample bridges. Both of these bridges were 80-foot simple span bridges with cross-sections capable of carrying two traffic lanes. One of the bridges evaluated utilized homogeneous plate girders for the steel superstructure whereas the other utilized rolled beams. The chapters of this report are organized as follows:

- Chapter 2 contains the overall bridge layout used for this design evaluation (bridge cross-section, span length, etc.) as well as the details regarding the design procedure utilized for this document. In addition, parameters and calculations common to both design evaluations are included in this chapter.
- Chapter 3 contains the design evaluation of the homogeneous plate girder design option. This girder was selected from Sheet 107 of the SMDI Short Span Design Details.
- Chapter 4 contains the design evaluation of the "Lightest Weight" rolled beam design option. This girder was selected from Sheet 201 of the SMDI Short Span Design Details.
- Chapter 5 contains a summary of the two design evaluations.

CHAPTER 2: COMMON DESIGN PARAMETERS

2.1 INTRODUCTION

Contained in this chapter is an overview of the layout of the sample bridge assessed in this design evaluation. In addition, a comprehensive overview of loads, load combinations and limit states employed for both design options is included are included. Finally, a discussion of parameters and calculations common to both girder solutions is presented.

2.2 BRIDGE LAYOUT

As shown in the figure below, the bridges in this design evaluation are designed for two 12 foot travel lanes and two 5 foot shoulders. The bridges have two Jersey barriers that are 15.25 inches wide. To accommodate the lanes and shoulders, both of the bridges in this design evaluation consist of 4 girders spaced at 10.5 feet with 2.52-foot-wide overhangs. An 8-inch-thick concrete deck is employed, which includes a ¼ inch sacrificial wearing surface (also referred to as an integral wearing surface, or IWS) and 2-inch haunch (measured from the bottom of the top flange to the bottom of the deck). In addition, both of these bridges are designed for a simple span of 80 feet with diaphragms spaced at 20 feet. No skew is present in this girder layout.

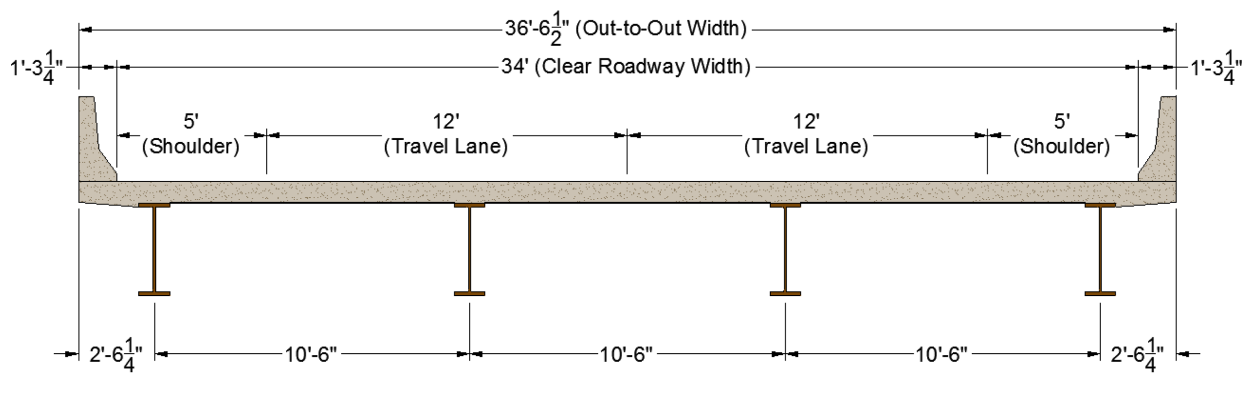

Figure 2.1 Typical Bridge Cross Section

2.3 Design Parameters

All bridges in this design package have been designed according to the seventh edition of the AASHTO LRFD Bridge Design Specifications (2014). All Articles referred to hereafter will refer directly to the AASHTO Specifications. Contained in this section is a description of the loads and load combinations employed, the limits states assessed in this design evaluation, and the loads used throughout this design process.

2.3.1 Loads & Load Combinations

For this set of design evaluations, the following permanent and transient loads are evaluated:

- DC = dead load of structural components and nonstructural attachments
    - Divided into two components: DC1 (applied to the noncomposite section) and DC2 (applied to the composite section)
- DW = dead load of wearing surface and utilities
- IM = vehicular dynamic load allowance
    - Serves to amplify the vehicular components of the HL-93 live load (i.e. the truck and tandem)
    - For the fatigue limit state, IM = 15% (Table 3.6.2.1-1)
    - For all other limit states, IM = 33% (Table 3.6.2.1-1)
- LL = vehicular live load
    - The HL-93 vehicular live load as defined in Article 3.6.1.2.
        - Combination of either design truck + design lane or the design tandem + design lane (whichever yields the largest force effect).
    - Note that for the fatigue limit state, the fatigue load consists of only one design truck with a fixed rear axle spacing of 30 feet (Article 3.6.1.4.1)

Using these specified loads, the following load combinations are assessed (values for load factors were derived from Tables 3.4.1-1 and 3.4.1-2 unless otherwise specified). For this set of design calculations, η_D (ductility factor), η_R (redundancy factor), and η_I (operational importance factor) are all taken to be 1.00.

- Strength I: basic load combination relating to the normal vehicular use of the bridge without wind
  - 1.25 DC + 1.50 DW + 1.75 (LL + IM)
  - In addition, for evaluating the constructability requirements of Article 6.10.3, according to Article 3.4.2, all load factors associated with construction loads were taken to be 1.50.
- Strength IV: load combination relating to very high dead to live load force effect ratios
  - 1.50 DC + 1.50 DW
- Service I: load combination associated with evaluation of live load deflections (Article 3.4.2.2)
  - 1.00 (LL + IM)
- Service II: load combination intended to control yielding of steel structures
  - 1.00 DC + 1.00 DW + 1.30 (LL + IM)
- Fatigue I: fatigue load combination related to infinite load-induced fatigue life (see 2.4.3 for evaluation)
  - 1.50 (LL + IM)

The following loads were taken for all of the calculations in this design evaluation:

- Unit weight of concrete = .150 kcf
- Compressive strength of concrete = 4.0 ksi
  - These values correspond to normal weight concrete. For normal weight concrete, according to the provisions of Article C6.10.1.1.1b, this yields a modular ratio, *n*, of 8.
- Unit weight of steel = .490 kcf
- Steel stay-in-place formwork (SIP) unit weight = .015 ksf

- Future wearing surface = .025 ksf
- Weight of concrete Jersey barriers = .304 kip/ft
- To account for miscellaneous steel details, such as diaphragms and connection stiffeners, the weight of the steel girders was increased by 5%.
- Construction loads:
  - Overhang deck forms = .040 kip/ft
  - Screed rail = .085 kip/ft
  - Railing = .025 kip/ft
  - Walkway = .125 kip/ft
  - Finishing machine = 3.0 kip

2.3.2 Limit States Evaluated

The limit states that pertain to the performance of the girders are discussed in this section. It should be noted that, for all limit states, according to Article 6.5.4.2, the resistance factor for flexure, ϕ_f, and for shear, ϕ_v, are both taken to be 1.00. In addition, since both girders are fully comprised of 50-ksi steel, the hybrid factor, R_h, is taken as 1.0.

2.3.2.1 Cross-Section Proportion Limits (Article 6.10.2)

The girders in this design evaluation were evaluated to meet the cross-section proportion limits of Article 6.10.2. These limits are divided into two main categories: flange proportions and web proportions.

For webs without longitudinal stiffeners, the following limit is employed from Article 6.10.2.1.1.

$$\frac{D}{t_w} \leq 150$$

Eq. 6.10.2.1.1-1

The following limits are employed for flange proportions. In addition to the limits set forth in Article 6.10.2.2, Article C6.10.3.4 specifies an additional limit to prevent out-of-plane distortions of the girder compression flanges and web during construction, which is also employed throughout this design evaluation.

$$\frac{b_f}{2t_f} \leq 12.0$$

Eq. 6.10.2.2-1

$$b_f \geq \frac{D}{6}$$

Eq. 6.10.2.2-2

$$t_f \geq 1.1 t_w$$

Eq. 6.10.2.2-3

$$0.1 \leq \frac{I_{yc}}{I_{yt}} \leq 10$$

Eq. 6.10.2.2-4

$$b_{fc} \geq \frac{L}{85}$$

Eq. C6.10.3.4-1

2.3.2.2 Constructability (Article 6.10.3)

Article 2.5.3 requires that bridges should be designed in a manner such that fabrication/erection can be performed without undue difficulty or distress and that locked-in construction force effects are within tolerable limits. To meet this requirement, the provisions of Article 6.10.3 are employed. Article 6.10.3 outlines several provisions for limiting stress in discretely-braced compression and tension flanges related to yielding of the flanges, flexural resistance of the compression flange, and web bend-buckling resistance, and are as follows. Details regarding the computation of the flexural resistance of the compression flange, F_{nc}, and the web bend-buckling resistance, F_{crw}, are reserved for Chapters 3 and 4.

$$f_{bu} + f_l \leq \phi_f R_h F_{yc}$$ Eq. 6.10.3.2.1-1

$$f_{bu} + \frac{1}{3} f_l \leq \phi_f F_{nc}$$ Eq. 6.10.3.2.1-2

$$f_{bu} \leq \phi_f F_{crw}$$ Eq. 6.10.3.2.1-3

$$f_{bu} + f_l \leq \phi_f R_h F_{yt}$$ Eq. 6.10.3.2.2-1

To determine the stresses resulting from lateral loads during construction, an approximation for lateral moments is specified Article C6.10.3.4, which idealizes the girder as a fixed beam between lateral bracing elements. Lateral bending moments are approximated as shown for statically equivalent uniform loads, F_l, and concentrated loads, P_l. For both girders, constructability requirements are evaluated at the middle unbraced segment, which has an unbraced length, L_b, of 20 feet.

$$M_l = \frac{F_l \, L_b^{\,2}}{12}$$ Eq. C6.10.3.4-2

$$M_l = \frac{P_l \, L_b}{8}$$ Eq. C6.10.3.4-3

In addition to this approximation, Article 6.10.1.6 specifies that a second-order analysis must be performed for lateral flange bending stresses in the compression flange if the unbraced length violates the limit set forth in Eq. 6.10.1.6-3. If this limit is not satisfied, an approximation is provided which amplifies first-order lateral flange bending stresses, f_{l1}, as a function of the major-axis bending stress and the elastic lateral torsional buckling stress, F_{cr}.

$$L_b \leq 1.2 L_p \sqrt{\frac{C_b R_b}{M_u / M_{yc}}}$$ Eq. 6.10.1.6-3

$$f_l \leq \left(\frac{0.85}{1 - \dfrac{M_u}{F_{cr} \, S_{xc}}} \right) f_{l1} \geq f_{l1}$$ Eq. 6.10.1.6-5

In lieu of performing a deck casting sequence analysis, since this bridge layout is a simple span, the deck is conservatively assumed to be cast in one pour. Therefore, the major-axis bending stress, f_{bu}, is that from the total noncomposite dead load, or DC1. Also, when checking constructibility, the web load-shedding factor, R_b, is taken as 1.0, according to Article 6.10.1.10.2.

It should be noted that Article 6.10.3 also specifies that the webs shall satisfy a capacity requirement during construction. However, as the construction shear loads in this design evaluation are lower than the shear loads the girder must withstand at the strength limit state, this requirement is not explicitly evaluated here; instead, this is evaluated at the strength limit state (see 2.3.2.5).

2.3.2.3 Service Limit State (Article 6.10.4)

The intent of the service limit state is to limit stresses and deformations under regular operating conditions. This is accomplished by limiting the levels of stress that the member experiences in order to prevent localized yielding. This is shown in the equations below. Note that for the girders in the design evaluation, no lateral stresses are considered at service conditions.

FOR THE TOP STEEL FLANGE OF COMPOSITE SECTIONS

$$f_f \leq 0.95 R_h F_{yf}$$

Eq. 6.10.4.2.2-1

FOR THE BOTTOM STEEL FLANGE OF COMPOSITE SECTIONS

$$f_f + \frac{f_l}{2} \leq 0.95 R_h F_{yf}$$

Eq. 6.10.4.2.2-2

In addition to the limit set forth for permanent deformations, many state DOTs and owner agencies choose to invoke optional live load deflection criteria which are meant to ensure user comfort. This optional limit is also evaluated. Article 2.5.2.6.2 specifies deflection criteria that may be used; for bridges subjected to vehicular loads only, a limit of L/800 is specified. Therefore, for a span length of 80 ft, this equates to a live load deflection limit of 1.2 inches.

2.3.2.4 Fatigue Limit State (Article 6.10.5)

The intent of the fatigue limit state is to control crack growth under cyclic loading conditions by limiting the range of live load stress, Δf, that steel members are subjected to. Specifically, load induced fatigue categories must satisfy the limit below. For the limit state, the load factor, γ, and the nominal fatigue resistance, $(\Delta F)_n$, associated with the fatigue limit state are a function of the number of stress cycles the girder is subjected to. This is discussed explicitly in 2.4.3.

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

Article 6.10.5 also specifies a special fatigue requirement for webs with interior transverse shear stiffeners. For the girders chosen for this design evaluation, the webs are unstiffened by transverse shear stiffeners. Therefore, the special web fatigue requirement specified in Article 6.10.5.3 does not need to be evaluated for either design.

2.3.2.5 Strength Limit State (Article 6.10.6)

The intent of the strength limit state is to ensure that the structure has adequate strength and stability when subjected to maximum factored loads. For composite sections in positive flexure, sections must meet flexural resistance requirements as well as a ductility requirement as specified in Article 6.10.7.3. In addition, the section must also have adequate shear capacity under maximum factored loads. The computation of the girders' flexural resistance, shear resistance, and ductility are discussed in each design's respective chapter, along with the factored loads and force effects that these sections must withstand.

2.4 COMMON PARAMETERS & CALCULATIONS

Contained herein is a brief description of parameters and values that are common to both the homogeneous plate girder solution and the rolled beam solution chosen for this design evaluation.

2.4.1 Section Properties

As stated in Article 6.10.1.1.1, stresses in a composite section due to applied loads shall be the sum of stresses applied separately to the noncomposite (or steel) section, the short-term composite section, and the long-term composite section. For calculating flexural stresses, the concrete deck is transformed to an equivalent area of steel through the use of the modular ratio, n. As stated in 2.3.1, for these bridges, $n = 8$. For loads applied to the short-term composite section (i.e. LL + IM), the concrete is transformed by dividing the concrete's effective flange width by n; for loads applied to the long-term composite section (i.e. DC2 and DW), the concrete is transformed by dividing the concrete's effective flange width by $3n$.

To compute the effective flange width, Article 4.6.2.6.1 states that the effective flange width of a concrete deck shall be taken as the tributary width. As barrier rails are often not structurally continuous, the added deck width allowed by Equation 4.6.2.6.1-1 is not included. Therefore, for the bridge layout in this evaluation, for interior and exterior girders, the effective flange width is 126 inches and 93.25 inches, respectively.

2.4.2 Multiple Presence Factors & Live Load Distribution Factors

Multiple presence factors account for the probability of coincident live loadings, and are listed in Article 3.6.1.1.2. These factors have already been included in the empirical equations listed in Article 4.6.2.2. However, when employing the lever rule or special analysis, the engineer must apply these factors. For the reader's convenience, these factors are listed in Table 2.1. It should be noted that multiple presence factors are not applied when evaluating the fatigue limit state.

Table 2.1 Multiple Presence Factors

Number of Lanes Loaded	m
One Lane Loaded	1.20
Two Lanes Loaded	1.00
Three Lanes Loaded	0.85
More Than Three Lanes Loaded	0.65

In lieu of a complex three-dimensional analysis, live load distribution factors were employed to determine live loads on individual girders. As stated in Article 4.6.2.2, these factors are only applicable if the bridge falls within a certain range of parameters.

Parameters for this set of bridges as well as their specified limits in Article 4.6.2.2 are listed. As shown, all parameters are within the specified limits. Note that the limit for K_g is not explicitly evaluated here as this value will change with the two bridge girders evaluated in this document (discussed later).

- $3.5 \leq S \leq 16.0$
  - S = girder spacing (ft) = 10.5
- $4.5 \leq t_s \leq 16$
  - t_s = structural slab thickness (in) = 7.75
- $20 \leq L \leq 240$
  - L = span length (ft) = 80
- $N_b \geq 4$
  - N_b = number of bridge girders = 4
- $-1.0 \leq d_e \leq 5.5$
  - d_e = distance from the centerline of the exterior girder's web to the edge of the barrier (ft) = 1.25
- $10{,}000 \leq K_g \leq 7{,}000{,}000$

Any of the distribution factors in Article 4.6.2.2 are a function of a longitudinal stiffness parameter, K_g, which is found as follows.

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

Once the longitudinal stiffness parameter is found, the distribution factors used in these analyses are found as follows:

BENDING MOMENT FOR AN INTERIOR GIRDER, ONE LANE LOADED

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

Tab. 4.6.2.2.2b-1

BENDING MOMENT FOR AN INTERIOR GIRDER, MULTIPLE LANES LOADED

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

Tab. 4.6.2.2.2b-1

SHEAR FOR AN INTERIOR GIRDER, ONE LANE LOADED

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

Tab. 4.6.2.2.3a-1

SHEAR FOR AN INTERIOR GIRDER, MULTIPLE LANES LOADED

$$g = 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^{2.0}$$

Tab. 4.6.2.2.3a-1

BENDING MOMENT FOR AN EXTERIOR GIRDER, ONE LANE LOADED

Use of the Lever Rule is employed (Table 4.6.2.2.2d-1)

BENDING MOMENT FOR AN EXTERIOR GIRDER, MULTIPLE LANES LOADED

$$g = \left(0.77 + \frac{d_e}{9.1}\right) g_{\text{interior}}$$

Tab. 4.6.2.2.2d-1

SHEAR FOR AN EXTERIOR GIRDER, ONE LANE LOADED

Use of the Lever Rule is employed (Table 4.6.2.2.3b-1)

SHEAR FOR AN EXTERIOR GIRDER, MULTIPLE LANES LOADED

$$g = \left(0.6 + \frac{d_e}{10}\right) g_{\text{interior}}$$

Tab. 4.6.2.2.3b-1

According to Article 4.6.2.2.2d, an additional investigation is required for steel slab-on-beam bridges, which assumes the entire cross-section rotates as a rigid body about the longitudinal centerline of the bridge. Additional distribution factors for bending moment and shear for exterior girders are computed according to the following formula.

$$R = \frac{N_L}{N_b} + \frac{X_{ext} \sum_{N_L} e}{\sum_{N_b} x^2}$$

Eq. C4.6.2.2.2d-1

To determine the distribution of live load deflections, according to Article 2.5.2.6.2, all design lanes should be loaded, and all supporting components should be assumed to deflect equally. In addition, it is stated that the appropriate multiple presence factor shall be applied. This is described mathematically in the formula below.

$$g = m \frac{N_L}{N_b}$$

Art. 2.5.2.6.2

2.4.2.1 Lever Rule Analysis

To determine the live load distribution of moment and shear in exterior beams for one lane loaded scenarios, the Specifications state that the lever rule shall be employed. A diagram showing the placement of the truck for the Lever Rule is shown in the Figure 2.2. According to Article 3.6.1.3.1, for the design of all bridge components other than the deck overhang, the design vehicle is to be positioned transversely such that the center of any wheel load is not closer than 2.0 feet from the edge of the design lane. Therefore, to produce the extreme force effect in the exterior girder, the truck is placed as close to the edge of the bridge as possible, i.e. 2 feet from the barrier or curb. To determine the distribution factor, moments are summed at the assumed hinge at the adjacent interior girder to determine the percentage of load resisted by the exterior girder.

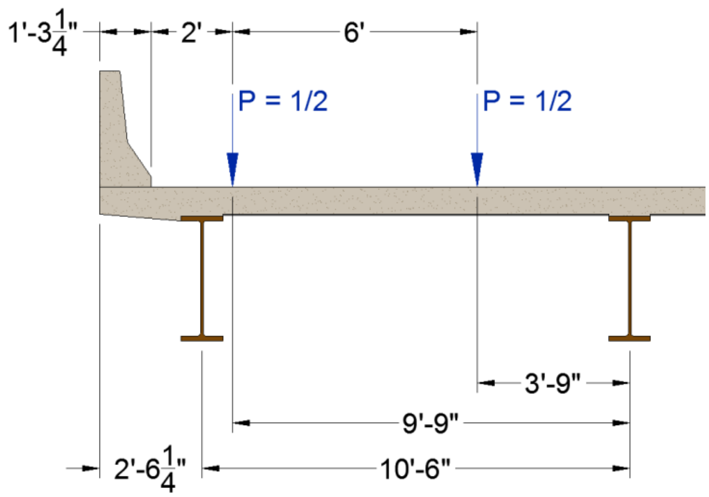

Figure 2.2 Lever Rule Truck Placement

Therefore, the lever rule analysis is as follows:

$$\text{Lever Rule Analysis} = \frac{0.5\,(9.75) + 0.5\,(3.75)}{10.5} = 0.643$$

To obtain the resulting distribution factor, this value is simply multiplied by the appropriate multiple presence factor for one-lane-loaded scenarios, or 1.20.

$$g = 1.20\,(0.643) = 0.771$$

<u>2.4.2.2 Special Analysis (Article 4.6.2.2.2d)</u>

As stated, an additional investigation is required which assumes the entire cross-section rotates as a rigid body about the longitudinal centerline of the bridge. When applying Special Analysis, the process is iterated for as many design vehicles that can fit onto the bridge cross-section. Also, it is the responsibility of the designer or analyst to apply the appropriate multiple presence factors to the derived reactions.

The first step is determining the eccentricities of the girders from the center-of-gravity of the girder group (x values) and the squares of those values. These values are listed in the table below.

Table 2.2 Girder Eccentricities

Girder	x (ft)	x^2 (ft^2)
1	-15.75	248.06
2	-5.25	27.56
3	5.25	27.56
4	15.75	248.06
	Σ =	551.25

Therefore, $\sum_{N_b} x^2 = 551.25 \text{ ft}^2$.

The next step is to determine the placement of trucks and the eccentricity of these trucks from the center-of-gravity of the girder group (e values). This step is shown graphically in the figure below. For these bridges, since the placement of Truck 2 lies directly on the girders' center of gravity, the eccentricity for this truck is zero.

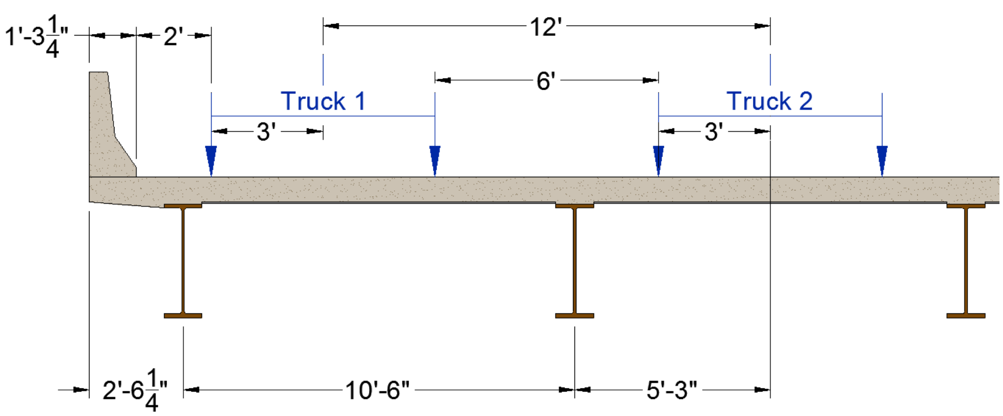

Figure 2.3 Special Analysis Truck Placement

Therefore, for this truck placement scheme, the eccentricities and their sums are as follows:

$e_1 = 12 \text{ ft}, \qquad \sum_{N_L} e = 12 \text{ ft}$

$e_2 = 0 \text{ ft}, \qquad \sum_{N_L} e = 12 \text{ ft} + 0 \text{ ft} = 12 \text{ ft}$

Employing these values and the appropriate multiple presence factors (Article 3.6.1.1.2), special analysis distribution factors can then be calculated. For these calculations, X_{ext} is simply the distance from the center-of-gravity of the girder group to the exterior girder, or 15.75 feet.

$$R_1 = 1.20\left[\frac{1}{4} + \frac{(15.75 \text{ ft})(12 \text{ ft})}{551.25 \text{ ft}^2}\right] = 0.711$$

$$R_2 = 1.00\left[\frac{2}{4} + \frac{(15.75 \text{ ft})(12 \text{ ft})}{551.25 \text{ ft}^2}\right] = 0.843$$

<u>2.4.2.3 Distribution Factor for Live Load Deflection (Article 2.5.2.6.2)</u>

To determine the distribution factor for live load deflections, all girders are assumed to deflect equally as previously stated, and the appropriate multiple presence factor shall be applied. For this bridge, with a clear roadway width of 34 feet, this equates to two design lanes (Article 3.6.1.1.1). Therefore, with a multiple presence factor of 1.00 for two loaded lanes (Article 3.6.1.1.2), the distribution factor is as follows:

$$g = 1.00\left(\frac{2}{4}\right) = 0.500$$

2.4.3 Nominal Fatigue Resistance

Article 6.10.5.1 requires that fatigue be investigated in accordance with Article 6.6.1, which states 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. The nominal fatigue resistance is taken as follows:

For the Fatigue I load combination (infinite life):

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

For the Fatigue II load combination (finite life):

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

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

For both of these design evaluations, the detail chosen for evaluation is the base metal at the weld joining the connection plates at interior diaphragms. According to Table 6.6.1.2.3-1, this detail is listed with a fatigue category C'. For a C' fatigue category, a constant amplitude fatigue threshold, $(\Delta F)_{TH}$ = 12 ksi (Table 6.6.1.2.5-3) is obtained.

Values for *n*, or the cycles per truck passage, are listed in Table 6.6.1.2.5-2. For a simple-span girder with a span length larger than 40 feet, *n* is taken as 1.0.

To determine the single-lane average daily truck traffic, $(ADTT)_{SL}$, a value of the average daily truck traffic, *ADTT*, must be assumed. For this example, an *ADTT* of 4000 trucks per day was assumed. Table 3.6.1.4.2-1 list *p* values, which are fractions of *ADTT* that can be expected in a single lane. For a two-lane bridge, p = 0.85. Therefore, according to Equation 3.6.1.4.2-1, $(ADTT)_{SL}$ can be easily evaluated.

$$(ADTT)_{SL} = p\,(ADTT) = 0.85\,(4000 \text{ trucks/day}) = 3400 \text{ trucks/day}$$

Table 6.6.1.2.3-2 lists average daily truck traffic values which are equivalent to infinite life. Specifically, Article 6.6.1.2.3 states that when the actual $(ADTT)_{SL}$ value is larger than that listed in the Table, the detail in question shall be designed for the Fatigue I load combination for infinite life. For a fatigue category C', a value of 745 trucks/day is listed. Therefore, the details chosen for these design evaluations are evaluated for the Fatigue I load combination for infinite life.

2.5 SUMMARY

This chapter contained an overview of the layout of the sample bridge assessed in this design evaluation. In addition, a comprehensive overview of loads, load combinations and limit states employed for both design options was included. Finally, a discussion of parameters and calculations common to both girder solutions is presented. These common parameters will be used to evaluate the two girder solutions in the following chapters.

CHAPTER 3: DESIGN ASSESSMENT – HOMOGENEOUS PLATE GIRDER

3.1 INTRODUCTION

Contained in this chapter is a design assessment according to current AASHTO LRFD Specifications of a homogeneous plate girder selected from the proposed SMDI Short Span Design Standards. In this design assessment, an evaluation of the girder at the strength, service, and fatigue limit states is conducted. Additionally, an analysis is conducted to determine whether the girder meets constructibility requirements under typical construction loads as specified in Article 6.10.3.

3.2 GIRDER GEOMETRY

As shown in Figure 3.1, homogeneous plate girders are selected for this design and were obtained from Sheet 107 of the SMDI Details. They are comprised of A507-50W steel (F_y = 50 ksi). A constant top flange (16”×1”) and web (32”×1/2”) are used throughout. The bottom flange is 16 inches wide and consists of a 1.5-inch-thick segment in the middle 60% and 1-inch-thick segments at the ends (see Figure 3.1 for details).

Note that, as stated in Chapter 1, these dimensions are nominal and do not take into account plate width tolerances due to cutting of the steel plate. Therefore, for all structural calculations, all of the nominal widths of the plates shown in the figure below are reduced by ¼” to account for burn tolerances.

Throughout this design evaluation, the end segments will be referred to as Section 1 and the middle segment will be referred to as Section 2. These sections are also shown in Figure 3.1.

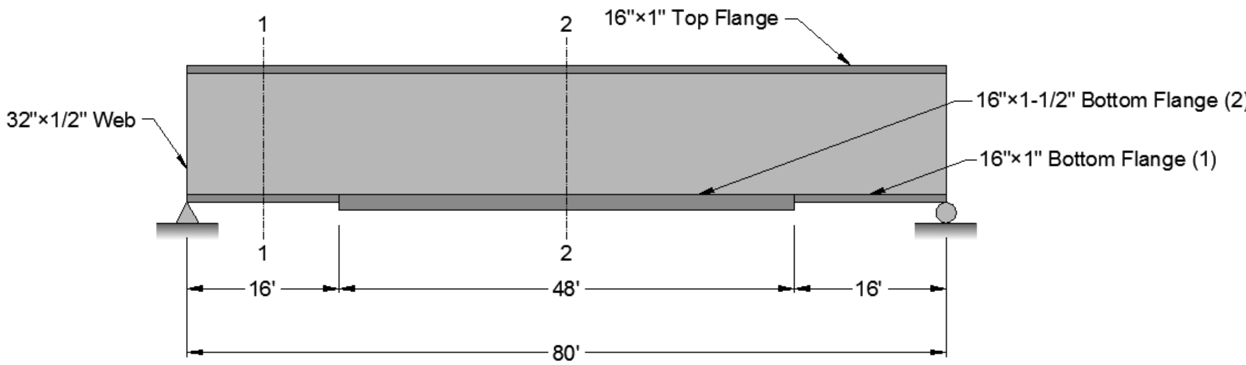

Figure 3.1 Elevation of Homogeneous Plate Girder

3.2.1 Section Properties

Section properties for the girder are listed on the following pages. For these calculations, all "*y*" distances are taken from the bottom of the bottom flange. Section properties are calculated for short-term composite sections (dividing the effective flange width by *n*) and the long-term composite sections (dividing the effective flange width by 3*n*). As stated in Chapter 2, the modular ratio for these bridges is taken as 8, and the effective flange widths of these bridges are as follows.

- For interior girders, 126 inches
- For exterior girders, 93.25 inches

Noncomposite Section - Section 1						
Shape	A (in^2)	y (in)	Ay (in^3)	I_0 (in^4)	d (in)	I (in^4)
TF	15.8	33.25	523.7	1.3	16.38	4224.5
W	15.9	16.88	267.9	1333.6	0.00	1333.6
BF	15.8	0.50	7.9	1.3	-16.38	4224.5
Σ	47.4		799.5			9782.6

Short Term Composite Section (Interior Girder) - Section 1						
Shape	A (in^2)	y (in)	Ay (in^3)	I_0 (in^4)	d (in)	I (in^4)
Girder	47.4	16.88	799.5	9782.6	-15.67	21413.5
Slab	122.1	38.63	4714.7	610.9	6.08	5125.1
Σ	169.4		5514.1			26538.7

Short Term Composite Section (Exterior Girder) - Section 1						
Shape	A (in^2)	y (in)	Ay (in^3)	I_0 (in^4)	d (in)	I (in^4)
Girder	47.4	16.88	799.5	9782.6	-14.27	19426.5
Slab	90.3	38.63	3489.2	452.2	7.48	5509.7
Σ	137.7		4288.7			24936.2

Long Term Composite Section (Interior Girder) - Section 1						
Shape	A (in^2)	y (in)	Ay (in^3)	I_0 (in^4)	d (in)	I (in^4)
Girder	47.4	16.88	799.5	9782.6	-10.05	14566.8
Slab	40.7	38.63	1571.6	203.6	11.70	5774.2
Σ	88.1		2371.0			20341.0

Long Term Composite Section (Exterior Girder) - Section 1						
Shape	A (in^2)	y (in)	Ay (in^3)	I_0 (in^4)	d (in)	I (in^4)
Girder	47.4	16.88	799.5	9782.6	-8.45	13167.1
Slab	30.1	38.63	1163.1	150.7	13.30	5475.5
Σ	77.5		1962.5			18642.6

Noncomposite Section - Section 2						
Shape	A (in^2)	y (in)	Ay (in^3)	I_0 (in^4)	d (in)	I (in^4)
TF	15.8	33.75	531.6	1.3	18.82	5577.4
W	15.9	17.38	275.8	1333.6	2.44	1428.2
BF	23.6	0.75	17.7	4.4	-14.18	4757.5
Σ	55.3		825.1			11763.1

Short-Term Composite Section (Interior Girder) - Section 2						
Shape	A (in^2)	y (in)	Ay (in^3)	I_0 (in^4)	d (in)	I (in^4)
Girder	55.3	14.93	825.1	11763.1	-16.65	27085.3
Slab	122.1	39.13	4775.7	610.9	7.54	7546.4
Σ	177.3		5600.8			34631.7

Short-Term Composite Section (Exterior Girder) - Section 2						
Shape	A (in^2)	y (in)	Ay (in^3)	I_0 (in^4)	d (in)	I (in^4)
Girder	55.3	14.93	825.1	11763.1	-15.01	24211.6
Slab	90.3	39.13	3534.4	452.2	9.18	8065.8
Σ	145.6		4359.5			32277.4

Long-Term Composite Section (Interior Girder) - Section 2						
Shape	A (in^2)	y (in)	Ay (in^3)	I_0 (in^4)	d (in)	I (in^4)
Girder	55.3	14.93	825.1	11763.1	-10.26	17578.5
Slab	40.7	39.13	1591.9	203.6	13.93	8100.5
Σ	95.9		2417.0			25679.0

Long-Term Composite Section (Exterior Girder) - Section 2						
Shape	A (in^2)	y (in)	Ay (in^3)	I_0 (in^4)	d (in)	I (in^4)
Girder	55.3	14.93	825.1	11763.1	-8.53	15786.4
Slab	30.1	39.13	1178.1	150.7	15.66	7532.8
Σ	85.4		2003.2			23319.2

3.2.2 Cross-Section Proportion Limits

The girders in this design evaluation were evaluated to meet the cross-section proportion limits of Article 6.10.2. For webs without longitudinal stiffeners, the following limit is employed from Article 6.10.2.1.1.

$$\frac{D}{t_w} \leq 150$$

$$\frac{31.75}{0.5} \leq 150$$

$$63.5 \leq 150 \therefore OK$$

As previously stated, the following limits are employed for flange proportions. In addition to the limits set forth in Article 6.10.2.2, Article C6.10.3.4 specifies an additional limit for the compression flange, and is presented below. For this evaluation, the results are tabulated for Sections 1 and 2, demonstrating that the girders meet all applicable cross-section proportion limit.

SECTION 1

Ratio/Dimension	Value of Ratio/Dimension	Limit
$\frac{b_{fc}}{2t_{fc}}$	7.88	≤ 12
$\frac{b_{ft}}{2t_{ft}}$	7.88	≤ 12
b_{fc}	15.75	$\geq \frac{D}{6} = 5.29$
b_{ft}	15.75	$\geq \frac{D}{6} = 5.29$
t_{fc}	1.0	$\geq 1.1\, t_w = 0.55$
t_{ft}	1.0	$\geq 1.1\, t_w = 0.55$
$\frac{I_{yc}}{I_{yt}}$	$\frac{325.6}{325.6} = 1.0$	Between 0.1 and 10
b_{fc}	15.75	$\geq \frac{L}{85} = 11.29$

SECTION 2

Ratio/Dimension	Value of Ratio/Dimension	Limit
$\frac{b_{fc}}{2t_{fc}}$	7.88	≤ 12
$\frac{b_{ft}}{2t_{ft}}$	5.25	≤ 12
b_{fc}	15.75	$\geq \frac{D}{6} = 5.29$
b_{ft}	15.75	$\geq \frac{D}{6} = 5.29$
t_{fc}	1.0	$\geq 1.1\, t_w = 0.55$
t_{ft}	1.5	$\geq 1.1\, t_w = 0.55$
$\frac{I_{yc}}{I_{yt}}$	$\frac{325.6}{488.4} = 0.67$	Between 0.1 and 10
b_{fc}	15.75	$\geq \frac{L}{85} = 11.29$

3.3 DEAD LOADS

The dead loads computed for this girder consist of the component and attachment dead load (DC) and the wearing surface dead load (DW) and are described herein.

3.3.1 Component and Attachment Dead Load (DC)

The dead load of structural components and nonstructural attachments are computed as follows. As previously stated, the DC load is divided into two components, the load applied to the noncomposite section (DC1) and the load applied to the long-term composite section (DC2). Loads such as the slab, overhang tapers, the Jersey barriers, and the SIP formwork are assumed to be equally distributed to all of the girders. In addition, a weighted approach is employed to calculate an effective uniformly distributed weight of the homogeneous plate girder.

NONCOMPOSITE DEAD LOAD (DC1):

$$\text{Slab} = \frac{0.150}{4}\left[\left(\frac{8.0}{12}\right)\left(36+\frac{6.5}{12}\right)\right] \qquad 0.914 \text{ kip/ft}$$

$$\text{Haunch} = 0.150\left[\left(\frac{16.0}{12}\right)\left(\frac{2.0-1.0}{12}\right)\right] \qquad 0.017 \text{ kip/ft}$$

$$\text{Taper} = 0.150\left(\frac{2}{4}\right)\left[\frac{1}{2}\left(\frac{2.0}{12}\right)\left(\frac{30.25-16.0/2}{12}\right)\right] \qquad 0.012 \text{ kip/ft}$$

$$\text{SIP} = \frac{0.015}{4}3\left[10.5-\frac{16.0}{12}\right] \qquad 0.103 \text{ kip/ft}$$

$$\text{Girder} = \frac{0.490}{144}\left[40\%(48.0)+60\%(56.0)\right] \qquad 0.180 \text{ kip/ft}$$

$$\text{Misc.Details} = 5\%(0.177) \qquad 0.009 \text{ kip/ft}$$

1.235 kip/ft

COMPOSITE DEAD LOAD (DC2):

$$\text{Barrier} = \frac{2}{4}(0.304) \qquad 0.152 \text{ kip/ft}$$

0.152 kip/ft

3.3.2 Wearing Surface Dead Load (DW)

The dead load of the future wearing surface is applied across the clear roadway width of 34 feet. Like DC1 and DC2, loads are assumed to be equally distributed to all of the girders.

WEARING SURFACE DEAD LOAD (DW):

$$\text{Wearing Surface} = \frac{0.025}{4}(34)$$

0.213 kip/ft

0.213 kip/ft

3.4 STRUCTURAL ANALYSIS

For this design evaluation, an approximate analysis is conducted which employs a line-girder analysis model. Dead loads, as stated earlier, are assumed to be evenly distributed to all girders. For live loads, live load distribution factors are used to distribute the vehicular live load to the line-girder model.

3.4.1 Live Load Distribution Factors (Article 4.6.2.2)

As previously stated, many of the bending moment distribution factors specified in Article 4.6.2.2 are a function of K_g, a longitudinal stiffness parameter. K_g is computed according to Eq. 4.6.2.2.1-1, and is shown below for an interior girder at midspan. Note that K_g does not need to be calculated for exterior girders since the lever rule, special analysis, and modified interior distribution factors serve as the exterior girder distribution factors. In addition, as previously stated, K_g must lie between 10,000 in^4 and 7,000,000 in^4 for the application of these distribution factors to be valid; as shown, this limit is clearly met.

$K_g = n\left(I + Ae_g{}^2\right)$

SECTION 1:

$$K_g = 8\left[9782.6 + (47.4)\left(1.0 + 31.75 + 2.0 + \frac{7.75}{2} - \frac{799.5}{47.4}\right)^2\right] = 257{,}552 \text{ in}^4$$

SECTION 2:

$$K_g = 8\left[11763.1 + (55.3)\left(1.5 + 31.75 + 2.0 + \frac{7.75}{2} - \frac{825.1}{55.3}\right)^2\right] = 352{,}763 \text{ in}^4$$

3.4.1.1 General Live Load Distribution Factors

Using the formulas and methods discussed in 2.4.2, moment and shear distribution factors for the strength and service limit states are calculated and listed below. Note that many of the values are repeated as the lever rule and special analysis apply to both moment and shear distribution.

STRENGTH AND SERVICE LIMIT STATE

Section 1		Section 2	
Bending Moment - Interior Girder		*Bending Moment - Interior Girder*	
One Lane Loaded	0.519	One Lane Loaded	0.533
Multiple Lanes Loaded	0.745	Multiple Lanes Loaded	0.766
Shear - Interior Girder		*Shear - Interior Girder*	
One Lane Loaded	0.780	One Lane Loaded	0.780
Multiple Lanes Loaded	0.985	Multiple Lanes Loaded	0.985
Bending Moment - Exterior Girder		*Bending Moment - Exterior Girder*	
One Lane Loaded	0.771	One Lane Loaded	0.771
Multiple Lanes Loaded	0.676	Multiple Lanes Loaded	0.695
Special Analysis (1 Lane)	0.711	Special Analysis (1 Lane)	0.711
Special Analysis (2 Lanes)	0.843	Special Analysis (2 Lanes)	0.843
Shear - Exterior Girder		*Shear - Exterior Girder*	
One Lane Loaded	0.771	One Lane Loaded	0.771
Multiple Lanes Loaded	0.714	Multiple Lanes Loaded	0.714
Special Analysis (1 Lane)	0.711	Special Analysis (1 Lane)	0.711
Special Analysis (2 Lanes)	0.843	Special Analysis (2 Lanes)	0.843

3.4.1.2 Fatigue Live Load Distribution Factors

Using the formulas and methods discussed in 2.4.2, live load distribution factors for the fatigue limit state are calculated and listed below. To obtain these values, the previously computed distribution factors for one-lane-loaded scenarios (chosen since the fatigue loading consists of only one design truck) are divided by 1.20, the multiple presence factor for one lane loaded (as previously stated, multiple presence factors are not applied at the fatigue limit state).

FATIGUE LIMIT STATE

Section 1		Section 2	
Bending Moment - Interior Girder		*Bending Moment - Interior Girder*	
One Lane Loaded	0.432	One Lane Loaded	0.444
Bending Moment - Exterior Girder		*Bending Moment - Exterior Girder*	
One Lane Loaded	0.643	One Lane Loaded	0.643
Special Analysis (1 Lane)	0.593	Special Analysis (1 Lane)	0.593

3.4.1.3 Live Load Distribution Factor Summary

Governing distribution factors are listed below for interior and exterior girders. As shown, distribution factors for exterior girders, on average, exceed those for interior girders. Also, the distribution factor for deflection (computed earlier) is also presented.

SECTION 1 SUMMARY	Interior	Exterior	SECTION 2 SUMMARY	Interior	Exterior
Moment	0.745	0.843	Moment	0.766	0.843
Shear	0.985	0.843	Shear	0.985	0.843
Fatigue Moment	0.432	0.643	Fatigue Moment	0.444	0.643
Deflection	0.500	0.500	Deflection	0.500	0.500

3.5 ANALYSIS RESULTS

The tables in this section contain the moments, shears, and deflections resulting from structural analysis of the girder. Analyses were generated using the commercial software package LEAP CONSYS (2008), which idealizes the structure as a simple-span line-girder. For these analyses, properties from the exterior girder were utilized for the stiffness of the line-girder model. This was due to the reduced section properties (due to a smaller effective flange width) and the increased live load distribution factors. An exception to this, however, is the set of distributed shears, which are distributed according to the interior girder (chosen for its high live load distribution factor).

Unfactored/Undistributed Moments (ft-kip)											
x/L	*DC1*	*DC2*	*DW*	*Truck*		*Lane*		*Tandem*		*Fatigue Truck*	
				(+)	*(−)*	*(+)*	*(−)*	*(+)*	*(−)*	*(+)*	*(−)*
0	0	0	0	0	0	0	0	0	0	0	0
0.1	354.6	43.9	61.2	451.2	0	184.3	0	350.0	0	387.2	0
0.2	630.3	78.1	108.8	787.2	0	327.7	0	620.0	0	659.2	0
0.3	827.3	102.5	142.8	1008.0	0	430.1	0	810.0	0	843.2	0
0.4	945.5	117.1	163.2	1136.0	0	491.5	0	920.0	0	931.2	0
0.5	984.9	122.0	170.0	1160.0	0	512.0	0	950.0	0	904.0	0
0.6	945.5	117.1	163.2	1136.0	0	491.5	0	920.0	0	931.2	0
0.7	827.3	102.5	142.8	1008.0	0	430.1	0	810.0	0	843.2	0
0.8	630.3	78.1	108.8	787.2	0	327.7	0	620.0	0	659.2	0
0.9	354.6	43.9	61.2	451.2	0	184.3	0	350.0	0	387.2	0
1	0	0	0	0	0	0	0	0	0	0	0

Unfactored/Undistributed Shears (kip)									
x/L	*DC1*	*DC2*	*DW*	*Truck*		*Lane*		*Tandem*	
				(+)	*(−)*	*(+)*	*(−)*	*(+)*	*(−)*
0	49.2	6.1	8.5	63.5	0	25.6	0	48.7	0
0.1	39.4	4.9	6.8	56.4	-3.2	20.7	-0.3	43.8	-3.8
0.2	29.5	3.7	5.1	49.2	-7.2	16.4	-1.0	38.8	-8.8
0.3	19.7	2.4	3.4	42.0	-13.6	12.5	-2.3	33.8	-13.8
0.4	9.8	1.2	1.7	34.8	-20.4	9.2	-4.1	28.8	-18.8
0.5	0	0	0	27.6	-27.6	6.4	-6.4	23.8	-23.8
0.6	-9.8	-1.2	-1.7	20.4	-34.8	4.1	-9.2	18.8	-28.8
0.7	-19.7	-2.4	-3.4	13.6	-42.0	2.3	-12.5	13.8	-33.8
0.8	-29.5	-3.7	-5.1	7.2	-49.2	1.0	-16.4	8.8	-38.8
0.9	-39.4	-4.9	-6.8	3.2	-56.4	0.3	-20.7	3.8	-43.8
1	-49.2	-6.1	-8.5	0	-63.5	0	-25.6	0	-48.7

Unfactored/Undistributed Deflections (in)				
x/L	*Truck*		*Lane*	
	(+)	*(−)*	*(+)*	*(−)*
0	0	0	0	0
0.1	0.42	0	0.21	0
0.2	0.80	0	0.39	0
0.3	1.09	0	0.53	0
0.4	1.28	0	0.62	0
0.5	1.34	0	0.65	0
0.6	1.28	0	0.62	0
0.7	1.09	0	0.53	0
0.8	0.80	0	0.39	0
0.9	0.42	0	0.21	0
1	0	0	0	0

Unfactored/Distributed Moments (ft-kip)							
x/L	*1.33 Truck + Lane*		*1.33 Tandem + Lane*		*DF*	*LL + IM*	
	(+)	*(−)*	*(+)*	*(−)*		*(+)*	*(−)*
0	0	0	0	0	0.843	0	0
0.1	784.4	0	649.8	0	0.843	661.1	0
0.2	1374.7	0	1152.3	0	0.843	1158.7	0
0.3	1770.7	0	1507.4	0	0.843	1492.5	0
0.4	2002.4	0	1715.1	0	0.843	1687.7	0
0.5	2054.8	0	1775.5	0	0.843	1731.9	0
0.6	2002.4	0	1715.1	0	0.843	1687.7	0
0.7	1770.7	0	1507.4	0	0.843	1492.5	0
0.8	1374.7	0	1152.3	0	0.843	1158.7	0
0.9	784.4	0	649.8	0	0.843	661.1	0
1	0	0	0	0	0.843	0	0

Unfactored/Distributed Shears (kip)							
x/L	*1.33 Truck + Lane*		*1.33 Tandem + Lane*		*DF*	*LL + IM*	
	(+)	*(−)*	*(+)*	*(−)*		*(+)*	*(−)*
0	110.1	0	90.4	0	0.985	108.4	0
0.1	95.7	-4.6	79.0	-5.4	0.985	94.3	-5.3
0.2	81.8	-10.6	68.0	-12.7	0.985	80.6	-12.5
0.3	68.4	-20.4	57.5	-20.7	0.985	67.3	-20.3
0.4	55.5	-31.2	47.5	-29.1	0.985	54.7	-30.8
0.5	43.1	-43.1	38.1	-38.1	0.985	42.5	-42.5
0.6	31.2	-55.5	29.1	-47.5	0.985	30.8	-54.7
0.7	20.4	-68.4	20.7	-57.5	0.985	20.3	-67.3
0.8	10.6	-81.8	12.7	-68.0	0.985	12.5	-80.6
0.9	4.6	-95.7	5.4	-79.0	0.985	5.3	-94.3
1	0	-110.1	0	-90.4	0.985	0	-108.4

Strength I Moments (ft-kip)							
x/L	*1.25 DC1*	*1.25 DC2*	*1.50 DW*	*1.75 LL + IM*		*Strength I*	
				(+)	*(−)*	*(+)*	*(−)*
0	0	0	0	0	0	0	0
0.1	443.2	54.9	91.8	1157.0	0	1746.9	589.9
0.2	787.9	97.6	163.2	2027.6	0	3076.4	1048.7
0.3	1034.1	128.1	214.2	2611.8	0	3988.3	1376.4
0.4	1181.9	146.4	244.8	2953.5	0	4526.6	1573.1
0.5	1231.1	152.5	255.0	3030.8	0	4669.4	1638.6
0.6	1181.9	146.4	244.8	2953.5	0	4526.6	1573.1
0.7	1034.1	128.1	214.2	2611.8	0	3988.3	1376.4
0.8	787.9	97.6	163.2	2027.6	0	3076.4	1048.7
0.9	443.2	54.9	91.8	1157.0	0	1746.9	589.9
1	0	0	0	0	0	0	0

Strength I Shears (kip)							
x/L	*1.25 DC1*	*1.25 DC2*	*1.50 DW*	*1.75 LL + IM*		*Strength I*	
				(+)	*(−)*	*(+)*	*(−)*
0	61.6	7.6	12.8	189.7	0	271.6	81.9
0.1	49.2	6.1	10.2	165.0	-9.2	230.5	56.3
0.2	36.9	4.6	7.7	141.1	-21.9	190.2	27.3
0.3	24.6	3.1	5.1	117.8	-35.6	150.6	-2.8
0.4	12.3	1.5	2.6	95.6	-53.8	112.0	-37.5
0.5	0	0	0	74.3	-74.3	74.3	-74.3
0.6	-12.3	-1.5	-2.6	53.8	-95.6	37.5	-112.0
0.7	-24.6	-3.1	-5.1	35.6	-117.8	2.8	-150.6
0.8	-36.9	-4.6	-7.7	21.9	-141.1	-27.3	-190.2
0.9	-49.2	-6.1	-10.2	9.2	-165.0	-56.3	-230.5
1	-61.6	-7.6	-12.8	0	-189.7	-81.9	-271.6

Service II Moments (ft-kip)							
x/L	*1.00 DC1*	*1.00 DC2*	*1.00 DW*	*1.30 LL + IM*		*Service II*	
				(+)	*(−)*	*(+)*	*(−)*
0	0	0	0	0	0	0	0
0.1	354.6	43.9	61.2	859.5	0	1319.2	459.7
0.2	630.3	78.1	108.8	1506.3	0	2323.5	817.2
0.3	827.3	102.5	142.8	1940.2	0	3012.8	1072.6
0.4	945.5	117.1	163.2	2194.0	0	3419.8	1225.8
0.5	984.9	122.0	170.0	2251.5	0	3528.4	1276.9
0.6	945.5	117.1	163.2	2194.0	0	3419.8	1225.8
0.7	827.3	102.5	142.8	1940.2	0	3012.8	1072.6
0.8	630.3	78.1	108.8	1506.3	0	2323.5	817.2
0.9	354.6	43.9	61.2	859.5	0	1319.2	459.7
1	0	0	0	0	0	0	0

Service I Deflections (in)							
x/L	*Truck*		*0.25 Truck + Lane*		*DF*	*Service I*	
	(+)	*(−)*	*(+)*	*(−)*		*(+)*	*(−)*
0	0	0	0	0	0.500	0	0
0.1	0.56	0	0.35	0	0.500	0.28	0
0.2	1.06	0	0.66	0	0.500	0.53	0
0.3	1.45	0	0.89	0	0.500	0.73	0
0.4	1.70	0	1.05	0	0.500	0.85	0
0.5	1.78	0	1.10	0	0.500	0.89	0
0.6	1.70	0	1.05	0	0.500	0.85	0
0.7	1.45	0	0.89	0	0.500	0.73	0
0.8	1.06	0	0.66	0	0.500	0.53	0
0.9	0.56	0	0.35	0	0.500	0.28	0
1	0	0	0	0	0.500	0	0

Fatigue Moments (ft-kip)					
x/L	*LL + IM*		*DF*	*1.50 (LL + IM)*	
	(+)	*(−)*		*(+)*	*(−)*
0	0	0	0.643	0	0
0.1	445.3	0	0.643	429.4	0
0.2	758.1	0	0.643	731.0	0
0.3	969.7	0	0.643	935.0	0
0.4	1070.9	0	0.643	1032.6	0
0.5	1039.6	0	0.643	1002.5	0
0.6	1070.9	0	0.643	1032.6	0
0.7	969.7	0	0.643	935.0	0
0.8	758.1	0	0.643	731.0	0
0.9	445.3	0	0.643	429.4	0
1	0	0	0.643	0	0

3.6 Limit State Evaluations

Presented in this section is an evaluation of a typical exterior girder of the chosen bridge layout. The exterior girder was chosen due to the reduced section properties (due to a smaller effective flange width) and the increased live load distribution factors. Specifically, properties, loads, and resistances will correspond to Section 2 of the girder. In this evaluation, all of the aforementioned limit states, including strength, service, and fatigue, are assessed. In addition, a constructibility evaluation is also performed.

3.6.1 Constructibility

The provisions of Article 6.10.3 are employed to ensure adequate performance related to yielding of the flanges, flexural resistance of the compression flange, and web bend-buckling resistance during stages of construction. During construction, the noncomposite girder must have sufficient capacity to resist construction force effects. Therefore, the capacity of the noncomposite girder must be evaluated.

3.6.1.1 Compression Flange Resistance

The first step is determining which Article is applicable in determining the flexural capacity of the noncomposite girder. Article 6.10.6.2.3 states that Appendix A6 may be employed if the girder meets certain limits. This is preferable, as Appendix A6 allows the girder's noncomposite capacity to exceed the yield moment. For Appendix A6 to be applicable, the flanges' yield strengths must not exceed 70.0 ksi (this limit is met since F_y = 50 ksi), the skew must not exceed 20° (no skew is present) and two additional limits must be met.

$$\frac{2D_c}{t_w} < 5.7\sqrt{\frac{E}{F_{yc}}}$$

Eq. 6.10.6.2.3-1

$$\frac{I_{yc}}{I_{yt}} \geq 0.3$$

Eq. 6.10.6.2.3-2

The depth of the web in compression of the noncomposite girder in the elastic range, D_c, is the distance from the top of the web to the neutral axis of the girder. In addition, I_{yc} and I_{yt} have already been determined for this girder (see 3.2.2). Therefore, the evaluation of these limits is as follows.

$$D_c = 1.5 + 23.75 - \frac{825.1}{55.3} = 18.32 \text{ in}$$

$$\frac{2D_c}{t_w} < 5.7\sqrt{\frac{E}{F_{yc}}}$$

$$\frac{2(18.32)}{0.5} < 5.7\sqrt{\frac{29000}{50}}$$

$$73.26 < 137.27 \therefore OK$$

$$\frac{I_{yc}}{I_{yt}} \geq 0.3$$

$$\frac{325.6}{488.4} \geq 0.3$$

$$0.67 > 0.3 \therefore OK$$

Therefore, Appendix A6 may be employed. To employ Appendix A6, the yield moment, M_y, and the plastic moment, M_p, of the noncomposite girder must be computed. The yield moment of the girder is the smaller of the yield moment with respect to the tension flange and compression flange, respectively. The plastic moment is computed by summing moments of the girder's plastic forces about the plastic neutral axis, or where the total plastic compressive forces equal the total plastic tensile forces.

YIELD MOMENT

$$S_{xc} = \frac{11763.1}{1.5 + 31.75 + 1.0 - \left(\frac{825.1}{55.3}\right)} = 608.99 \text{ in}^3$$

$$S_{xt} = \frac{11763.1}{\left(\frac{825.1}{55.3}\right)} = 787.67 \text{ in}^3$$

Therefore:

$$M_y = M_{yc} = F_y S_{xc} = \frac{(50)(608.99)}{12} = 2537.4 \text{ ft-kip}$$

PLASTIC MOMENT

$$P_c = (50)(15.75)(1.0) = 787.5 \text{ kip}$$

$$P_w = (50)(31.75)(0.5) = 793.8 \text{ kip}$$

$$P_t = (50)(15.75)(1.5) = 1181.3 \text{ kip}$$

$$P_c + P_w \geq P_t$$

$$1581.3 \text{ kip} \leq 1181.3 \text{ kip}$$

$$\therefore \text{PNA is in the web}$$

$$787.5 + (50)(\overline{Y})(0.5) = (50)(31.75 - \overline{Y})(0.5) + 1181.3$$

$$\therefore \overline{Y} = 23.75 \text{ in (from the top of the web)}$$

$$M_p = \frac{787.5\left(23.75 + \frac{1.0}{2}\right) + 793.8\left(23.75 - \frac{31.75}{2}\right) + 1181.3\left(\frac{1.5}{2} + 31.75 - 23.75\right)}{12}$$

$$M_p = 2973.6 \text{ ft-kip}$$

The first step in employing Appendix A6 is to determine whether the section is a compact web section or a noncompact web section. Compact web sections are those that meet the following requirements.

$$\frac{2D_{cp}}{t_w} \leq \lambda_{pw(D_{cp})}$$ Eq. A6.2.1-1

D_{cp} is the depth of the web in compression at the plastic moment. Since the plastic neutral axis of the noncomposite girder was found to be in the web, this value is simply equal to the depth of the PNA, or 23.75 inches. $\lambda_{pw(D_{cp})}$ is then computed as follows.

$$\lambda_{rw} = 5.7\sqrt{\frac{E}{F_{yc}}}$$

Eq. A6.2.1-3

$$\lambda_{rw} = 5.7\sqrt{\frac{29000}{50}}$$

$$\lambda_{rw} = 137.27$$

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

Eq. A6.2.1-2

$$\lambda_{pw(D_{cp})} = \frac{\sqrt{\frac{29000}{50}}}{\left[0.54\frac{2973.6}{(1.0)(2537.6)} - 0.09\right]^2} \le 137.27\left(\frac{23.75}{18.32}\right)$$

$$\lambda_{pw(D_{cp})} = 81.73 < 178.00$$

$$\lambda_{pw(D_{cp})} = 81.73$$

Therefore, as shown below, the girder cannot qualify as a compact web section.

$$\frac{2D_{cp}}{t_w} \le \lambda_{pw(D_{cp})}$$

$$\frac{2(23.75)}{0.5} \le 81.75$$

$95 > 81.75 \therefore$ Web is not compact

For the girder to be classified as a noncompact web section (as opposed to a slender web section), according to Equation A6.2.2-1, λ_w must be less than λ_{rw}, where λ_w and λ_{rw} are the slenderness ratios defined below. As can be seen, this section qualifies as a noncompact web section.

$$\lambda_w = \frac{2D_c}{t_w} = \frac{2(18.32)}{0.5} = 73.27$$

Eq. A6.2.2-2

$$\lambda_{rw} = 5.7\sqrt{\frac{E}{F_{yc}}} = 137.27$$

Eq. A6.2.2-3

To determine the flexural capacity of the compression flange for a noncompact web section, a web plastification factor for the compression flange, R_{pc}, must be determined. This essentially determines how much the girder's flexural capacity can exceed M_y. In addition, they can account for the influence of web slenderness on the maximum potential flexural resistance. The web plastification factor is computed as follows.

$$\lambda_{pw(D_c)} = \lambda_{pw(D_{cp})}\left(\frac{D_c}{D_{cp}}\right) \leq \lambda_{rw}$$

Eq. A6.2.2-6

$$\lambda_{pw(D_c)} = 81.73\left(\frac{18.32}{23.75}\right) \leq 137.27$$

$$\lambda_{pw(D_c)} = 63.03 \leq 137.27$$

$$\lambda_{pw(D_c)} = 63.03$$

$$R_{pc} = \left[1 - \left(1 - \frac{R_h M_{yc}}{M_p}\right)\left(\frac{\lambda_w - \lambda_{pw(D_c)}}{\lambda_{rw} - \lambda_{pw(D_c)}}\right)\right]\frac{M_p}{M_{yc}} \le \frac{M_p}{M_{yc}}$$

Eq. A6.2.2-4

$$R_{pc} = \left[1 - \left(1 - \frac{1.0 \cdot 2537.4}{2973.6}\right)\left(\frac{73.27 - 63.03}{137.27 - 63.03}\right)\right]\frac{2973.6}{2537.4} \le \frac{2973.6}{2537.4}$$

$$R_{pc} = 1.15 < 1.17$$

$$R_{pc} = 1.17$$

The flexural capacity of the compression flange is a function of the slenderness ratio of the flange and whether or not the flange is classified as compact. The web plastification factor computed earlier is then used to compute the section's flexural capacity. For flanges to be classified as compact, the slenderness ratio for the flange, λ_f, must be less than a limiting value, λ_{pf}. As shown, the flange meets the requirements for compactness.

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

Eq. A6.3.2-3

$$\lambda_f = \frac{15.75}{2(1.0)}$$

$$\lambda_f = 7.88$$

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_y}}$$

Eq. A6.3.2-4

$$\lambda_{pf} = 0.38\sqrt{\frac{29000}{50}}$$

$$\lambda_{pf} = 9.15$$

$\lambda_f < \lambda_{pf}$ $\therefore$ Flange is compact

Therefore, the flexural capacity of the compression flange is computed as follows. Equation A6.3.2-1 yields the flexural capacity in terms of the girder's overall capacity, not the flange's capacity. To obtain the capacity of the flange, in accordance with Article 6.10.3.2.1, the flange's capacity can be computed by dividing the girder's capacity by S_{xc}.

$$M_{nc} = R_{pc} M_{yc} \qquad \text{Eq. A6.3.2-1}$$

$$M_{nc} = (1.15)(2537.4)$$

$$M_{nc} = 2913.5 \text{ ft-kip}$$

$$F_{nc} = \frac{M_{nc}}{S_{xc}}$$

$$F_{nc} = \frac{2913.5(12)}{608.99}$$

$$F_{nc} = 57.41 \text{ ksi}$$

3.6.1.2 Major Axis and Lateral Flange Bending Stresses

The next step in performing this constructability analysis is to determine the major axis and lateral flange bending stresses that the girder will be subjected to during construction. First, major-axis bending stresses will be computed. As previously stated, the deck is assumed to be cast in one pour; therefore, major axis bending stresses will be computed according to DC1. From analysis results, the unfactored DC1 moment was found to be 984.9 ft-kip. Therefore, major axis bending stresses are as follows. For this computation, the Strength IV load combination is employed in addition to Strength I. This is because, during construction, the bridge is subjected to very high dead to live load force effect ratios.

STRENGTH I:

Top flange: $f_{bu} = \frac{1.25(984.9)(12)}{608.99} = 24.26 \text{ ksi}$

Bottom flange: $f_{bu} = \frac{1.25(984.9)(12)}{787.67} = 18.76 \text{ ksi}$

STRENGTH IV:

Top flange:

$$f_{bu} = \frac{1.50(984.8)(12)}{608.99} = 29.11\,\text{ksi}$$

Bottom flange:

$$f_{bu} = \frac{1.50(984.9)(12)}{787.67} = 22.51\,\text{ksi}$$

Next, stresses due to lateral flange bending forces from construction loads must be computed. Before calculating lateral flange bending stresses, a determination must be made regarding whether or not a second-order analysis must be carried out for compressive stresses. To make this determination, a number of variables must be computed, including the effective radius of gyration for lateral torsional buckling, r_t, and the limiting unbraced length to achieve the maximum flexural resistance, L_p.

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1 + \frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\right)}}$$

Eq. 6.10.8.2.3-9

$$r_t = \frac{15.75}{\sqrt{12\left[1 + \frac{1}{3}\frac{(18.32)(0.5)}{(15.75)(1.0)}\right]}}$$

$$r_t = 4.16\,\text{in}$$

$$L_p = 1.0 r_t \sqrt{\frac{E}{F_{yc}}}$$

Eq. 6.10.8.2.3-4

$$L_p = 1.0(4.16)\sqrt{\frac{29000}{50}}$$

$$L_p = 100.22\,\text{in}$$

A moment gradient modifier, C_b, must then be computed in order to determine whether or not a second-order analysis must be carried out. C_b is a coefficient which accounts for different moment gradients on lateral torsional buckling.

It was previously determined that Appendix A6 was applicable for this noncomposite girder. Therefore, to compute C_b, moments must be found at various lengths along the unbraced segment of interest. Therefore, for this evaluation, L_b, is simply the distance between diaphragms, or 20 feet.

From analysis results (interpolating between tenth points), the following factored moments were obtained for the unbraced segment at midspan. It should be noted that since deck casting moments will result solely from DC1, this calculation for C_b will be valid for both Strength I and Strength IV load combinations.

M_{mid} = major-axis bending moment at the middle of the unbraced length = 915.9 ft-kip
M_0 = major-axis bending moment at one end of the unbraced segment = 728.8 ft-kip
M_2 = major-axis bending moment at the other end of the unbraced segment = 984.9 ft-kip

C_b is then calculated as follows (since M_{mid}/M_2 is less than 1).

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

$$M_1 = 2(915.9) - 984.9 \geq 728.8$$

$$M_1 = 847.0 > 728.8$$

$$M_1 = 847.0 \text{ ft-kip}$$

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

$$C_b = 1.75 - 1.05\left(\frac{847.0}{984.9}\right) + 0.3\left(\frac{847.0}{984.9}\right)^2 \leq 2.3$$

$$C_b = 1.069 < 2.3$$

$$C_b = 1.069$$

The limit for first-order elastic analyses can now be computed as follows.

$$L_b \le 1.2L_p\sqrt{\frac{C_bR_b}{M_u / M_{yc}}}$$ Eq. 6.10.1.6-3

STRENGTH I:

$$240 \le 1.2(100.22)\sqrt{\frac{(1.069)(1.0)}{1.25(984.9)/2537.4}}$$

$$240 > 178.5 \therefore \text{Not Satisfied}$$

STRENGTH IV:

$$240 \le 1.2(100.22)\sqrt{\frac{(1.069)(1.0)}{1.50(984.9)/12537.4}}$$

$$240 \le 162.9 \therefore \text{Not Satisfied}$$

Therefore, a second-order analysis must be performed for the Strength I and Strength IV load combinations. Article 6.10.1.6 provides an approximate method for computing second-order compression-flange lateral bending stresses by multiplying first-order values by an amplification factor (this calculation is not required for tensile stresses). This amplification factor is a function of the compression flange's elastic lateral torsional buckling stress, F_{cr}. To compute F_{cr}, the height between the centerline of the flanges, h, and the St. Venant torsional constant, J, must be calculated. F_{cr} is then computed as follows according to the provisions for Appendix A6. It should be noted that, according to Article C6.10.1.6, F_{cr} is not limited to $R_bR_hF_{yc}$.

$$h = \frac{1.0}{2} + 31.75 + \frac{1.5}{2} = 33.0 \text{ in}$$

$$J = \frac{Dt_w^{\ 3}}{3} + \frac{b_{fc}t_{fc}^{\ 3}}{3}\left(1 - 0.63\frac{t_{fc}}{b_{fc}}\right) + \frac{b_{ft}t_{ft}^{\ 3}}{3}\left(1 - 0.63\frac{t_{ft}}{b_{ft}}\right)$$

Eq. A6.3.3-9

$$J = \frac{(31.75)(0.5)^3}{3} + \frac{(15.75)(1.0)^3}{3}\left[1 - 0.63\left(\frac{1.0}{15.75}\right)\right] + \frac{(15.75)(1.5)^3}{3}\left[1 - 0.63\left(\frac{1.5}{15.75}\right)\right]$$

$$J = 23.02 \text{ in}^4$$

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

Eq. A6.3.3-8

$$F_{cr} = \frac{(1.069)(\pi^2)(29000)}{\left(\frac{240}{4.16}\right)^2}\sqrt{1 + 0.078\frac{23.02}{(608.99)(33.0)}\left(\frac{240}{4.16}\right)^2}$$

$$F_{cr} = 101.07 \text{ ksi}$$

For Strength IV, the amplification factor for first-order lateral flange bending stresses is as follows.

$$AF = \frac{0.85}{1 - \frac{M_u}{F_{cr}S_{xc}}}$$

Eq. 6.10.1.6-5

STRENGTH I:

$$AF = \frac{0.85}{1 - \frac{1.25(984.9)(12)}{(101.07)(608.99)}} = 1.118$$

STRENGTH IV:

$$AF = \frac{0.85}{1 - \frac{1.50(984.9)(12)}{(101.07)(608.99)}} = 1.194$$

To compute deck overhang loads, lateral forces are computed by determining the force statically equivalent to the couple resulting from the eccentric vertical loads. This computation involves the angle, α, between the overhang bracket and the web of the girder. The bracket is assumed to extend from the end of the overhang to the web-bottom flange junction. The angle between the web of the girder and the bracket, along with the lateral force relation, are as follows.

$$F_l = F\tan(\alpha)$$
$$\alpha = \tan^{-1}\left(\frac{30.25}{31.75}\right) = 43.61$$

In addition, half of the overhang load is assumed to act on the overhang bracket, and is computed as follows.

$$\frac{150}{2}\left(\frac{1}{144}\right)\left[(8.0)(30.25)+\frac{1}{2}\left(30.25-\frac{15.75}{2}\right)(2.0)+(2.0-1.0)\left(\frac{15.75}{2}\right)\right]=141.80\,\frac{\text{lb}}{\text{ft}}$$

The lateral forces, bending moments, and lateral stresses are summarized as follows. Lateral bending moments are computed according to the approximations discussed in 2.3.2.2. To compute lateral stresses from lateral bending moments, moments are divided by the major-axis section modulus of the flange, or $(t_f)(b_f)^2/6$.

Lateral Flange Bending Moments & Stresses

Components	F / P	tan(α)	F_l / P_l	L_b (ft)	M_l ("k)	S_{lc} (in³)	S_{lt} (in³)	f_{lc} (ksi)	f_{lt} (ksi)
Deck Weight (lb/ft)	141.80	0.953	135.10	20	54.04	41.34	62.02	1.307	0.871
Overhang Deck Forms (lb/ft)	40	0.953	38.11	20	15.24	41.34	62.02	0.369	0.246
Screed Rail (lb/ft)	85	0.953	80.98	20	32.39	41.34	62.02	0.784	0.522
Railing (lb/ft)	25	0.953	23.82	20	9.53	41.34	62.02	0.230	0.154
Walkway (lb/ft)	125	0.953	119.09	20	47.64	41.34	62.02	1.152	0.768
Finishing Machine (lb)	3000	0.953	2858.27	20	85.75	41.34	62.02	2.074	1.383

Factored lateral flange bending stresses are computed below. Note that, for the Strength IV load combination, no live loads are considered; therefore the finishing machine load is neglected. Also, the limit specified in Equation 6.10.1.6-1, which limits lateral flange bending stresses to 60% of F_y, is also met.

Factored First-Order Lateral Flange Bending Stresses						
Components	Strength I			Strength IV		
	γ_i	f_{lc} (ksi)	f_{lt} (ksi)	γ_i	f_{lc} (ksi)	f_{lt} (ksi)
Deck Weight	1.25	1.63	1.09	1.50	1.96	1.31
Overhang Deck Forms	1.50	0.55	0.37	1.50	0.55	0.37
Screed Rail	1.50	1.18	0.78	1.50	1.18	0.78
Railing	1.50	0.35	0.23	1.50	0.35	0.23
Walkway	1.50	1.73	1.15	1.50	1.73	1.15
Finishing Machine	1.50	3.11	2.07	-	-	-
Σ		8.55	5.70		5.76	3.84

3.6.1.3 Limit State Evaluation

The nominal bend-buckling resistance, F_{crw}, shall be calculated as follows. Note that F_{crw} shall not exceed the smaller of R_hF_{yc} (50 ksi) or F_{yw}/0.7 (71.4 ksi).

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

$$k = \frac{9}{(18.32 / 31.75)^2}$$

$$k = 27.03$$

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

$$F_{crw} = \frac{0.9(29000)(27.03)}{(31.75 / 0.5)^2}$$

$$F_{crw} = 174.97 \text{ ksi} > 50 \text{ ksi}$$

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

The limit states are evaluated as follows. As shown, the girder performs satisfactorily under all applicable constructibility limit states. Note that the second order amplification factor is not applied to tensile stresses.

COMPRESSION FLANGE YIELDING

$$f_{bu} + f_l \leq \phi_f R_h F_{yc}$$

Strength I: $24.26 + 1.118(8.55) \leq (1.00)(1.0)(50) \Rightarrow 33.82\text{ ksi} \leq 50\text{ ksi} \therefore OK\ (\text{Ratio} = 0.676)$

Strength IV: $29.11 + 1.194(5.76) \leq (1.00)(1.0)(50) \Rightarrow 36\text{ ksi} \leq 50\text{ ksi} \therefore OK\ (\text{Ratio} = 0.720)$

COMPRESSION FLANGE FLEXURAL RESISTANCE

$$f_{bu} + \frac{1}{3} f_l \leq \phi_f F_{nc}$$

Strength I: $24.26 + \frac{1.118(8.55)}{3} \leq (1.00)(57.41) \Rightarrow 27.45\text{ ksi} \leq 57.41\text{ ksi} \therefore OK\ (\text{Ratio} = 0.478)$

Strength IV: $29.11 + \frac{1.194(5.76)}{3} \leq (1.00)(57.41) \Rightarrow 31.4\text{ ksi} \leq 57.41\text{ ksi} \therefore OK\ (\text{Ratio} = 0.547)$

WEB BEND-BUCKLING RESISTANCE

$$f_{bu} \leq \phi_f F_{crw}$$

Strength I: $24.26 \leq (1.00)(50) \Rightarrow 24.26\text{ ksi} \leq 50\text{ ksi} \therefore OK\ (\text{Ratio} = 0.485)$

Strength IV: $29.11 \leq (1.00)(50) \Rightarrow 29.11\text{ ksi} \leq 50\text{ ksi} \therefore OK\ (\text{Ratio} = 0.582)$

TENSION FLANGE YIELDING

$$f_{bu} + f_l \leq \phi_f R_h F_{yt}$$

Strength I: $18.76 + 5.70 \leq (1.00)(1.0)(50) \Rightarrow 24.46\text{ ksi} \leq 50\text{ ksi} \therefore OK\ (\text{Ratio} = 0.489)$

Strength IV: $22.51 + 3.84 \leq (1.00)(1.0)(50) \Rightarrow 26.35\text{ ksi} \leq 50\text{ ksi} \therefore OK\ (\text{Ratio} = 0.527)$

3.6.2 Service Limit State

The service limit state is evaluated according to the provisions of Articles 6.10.4.1 (governing elastic deformations) and 6.10.4.2 (governing permanent deformations).

3.6.2.1 Elastic Deformations

The elastic deformation limit state, as previously stated, is evaluated against a maximum deformation of *L*/800, or 1.2 inches. From the analysis results, a maximum live load deflection of 0.89 inches was determined. Therefore, this meets elastic deformation requirements (Ratio = 0.742).

3.6.2.2 Permanent Deformations

The first step in evaluating the girder's performance under permanent deformation limits is to determine the girder's service level stresses. This will be derived solely from gravity and vehicular loadings, as lateral loads are not being considered at the service limit state in this design evaluation.

From the analysis results, the following Service II moments were found.

$1.00\ M_{DC1} = 984.9$ ft-kip

$1.00\ M_{DC2} = 122.0$ ft-kip

$1.00\ M_{DW} = 170.0$ ft-kip

$1.30\ M_{LL+IM} = 2251.5$ ft-kip

Using these moments, Service II stresses for the top and bottom flange are found as follows. Therefore, according to Equations 6.10.4.2.2-1 and 6.10.4.2.2-2, respectively, the flanges are shown to meet the requirements for permanent deformations at the service limit state.

TOP FLANGE:

$$f_f = \frac{(984.9)(12)(19.32)}{11763.1} + \frac{(122.0+170.0)(12)(10.78)}{23319.2} + \frac{(2251.5)(12)(4.31)}{32277.4} = 24.63\text{ ksi}$$

$$f_f \le 0.95 R_h F_{yf}$$

$$24.63 < 0.95(1.0)(50) \Rightarrow 24.63\text{ ksi} < 47.5\text{ ksi} \therefore OK(\text{Ratio} = 0.519)$$

BOTTOM FLANGE:

$$f_f = \frac{(984.9)(12)(14.93)}{11763.1} + \frac{(122.0+170.0)(12)(23.47)}{23319.2} + \frac{(2251.5)(12)(29.95)}{32277.4} = 43.60\text{ ksi}$$

$$f_f + \frac{f_l}{2} \le 0.95 R_h F_{yf}$$

$$43.60 + \frac{0}{2} < 0.95(1.0)(50) \Rightarrow 43.6\text{ ksi} < 47.5\text{ ksi} \therefore OK(\text{Ratio} = 0.918)$$

3.6.3 Fatigue Limit State

As previously discussed, the detail chosen for these design evaluations is the base metal at the weld joining the lateral bracing connection plates at interior diaphragms. These details are evaluated for the Fatigue I load combination for infinite life, with a nominal fatigue resistance of 12.0 ksi, previously determined as the constant amplitude fatigue threshold.

From the previously determined factored fatigue moments, a maximum fatigue moment of 1002.5 ft-kip was determined (see 3.5) at the diaphragm location. Since this is a simple-span bridge, a minimum fatigue moment of zero was found. Therefore, a fatigue stress range can be found for both the top flange and bottom flange by determining the stress resulting from the calculated moment. As shown, this detail performs satisfactorily.

TOP FLANGE

$$\gamma(\Delta f) = \frac{1002.5(12)(3.31)}{32277.4} = 1.23\text{ ksi}$$

$$1.23\text{ ksi} < 12.0\text{ ksi} \therefore OK(\text{Ratio} = 0.103)$$

BOTTOM FLANGE

$$\gamma(\Delta f) = \frac{1002.5(12)(28.44)}{32277.4} = 10.60\text{ ksi}$$

$$10.60\text{ ksi} < 12.0\text{ ksi} \therefore OK(\text{Ratio} = 0.883)$$

3.6.4 Strength Limit State

At the strength limit state, as specified in Article 6.10.6, the girder must meet requirements for flexure and shear as well as a ductility requirement. Each of these criteria will be evaluated.

<u>3.6.4.1 Flexure</u>

For flexure, in order to determine a section's capacity, a determination must be made whether the section is classified as compact or noncompact. For this determination, the section's plastic moment capacity must be calculated. The plastic moment capacity of the section is evaluated according to the provisions of Article D6.1. For this evaluation, the reinforcement in the concrete slab is conservatively neglected.

The first step in determining the section's plastic moment capacity is to determine the plastic forces in each of the section's components.

$$P_s = 0.85 f_c' b_s t_s = 0.85(4)(93.25)(7.75) = 2547.1\text{ kip}$$

$$P_c = F_{yc} b_c t_c = (50)(15.75)(1) = 787.5\text{ kip}$$

$$P_w = F_{yw} D t_w = (50)(31.75)(0.5) = 793.8\text{ kip}$$

$$P_t = F_{yt} b_t t_t = (50)(15.75)(1.5) = 1181.3\text{ kip}$$

Next, the location of the plastic neutral axis (PNA) must be determined. Table D6.1 gives a straightforward procedure in determining the location of the PNA and is adopted here.

Case I (PNA is in the web)

$P_t + P_w \geq P_c + P_s$

$1975.0 \text{ kip} < 3244.6 \text{ kip} \therefore \text{PNA is not in the web}$

Case II (PNA is in the top flange)

$P_t + P_w + P_c \geq P_s$

$2762.5 \text{ kip} < 2547.1 \text{ kip} \therefore \text{PNA is in the top flange}$

Therefore, the location of the PNA is as follows (measured from the top of the top flange).

$$\bar{Y} = \left(\frac{t_c}{2}\right)\left[\frac{P_w + P_t - P_s}{P_c} + 1\right] = \left(\frac{1.0}{2}\right)\left[\frac{793.8 + 1181.3 - 2547.1}{787.5} + 1\right] = 0.194 \text{ in}$$

Next, the distances of the individual components from the location of PNA are computed. Note that, as d_c (the distance from the compression flange plastic force to the PNA) is not necessary to compute the plastic moment according to the methods of Table D6.1, it is not explicitly evaluated.

$$d_s = 0.194 + (2.0 - 1.0) + \frac{7.75}{2} = 5.069 \text{ in}$$

$$d_w = (1.0 - 0.194) + \frac{31.75}{2} = 16.681 \text{ in}$$

$$d_t = (1.0 - 0.194) + 31.75 + \frac{1.5}{2} = 33.306 \text{ in}$$

The plastic moment of the composite section, M_p, can now be evaluated.

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

$$M_p = \frac{787.5}{2(1.0)}\left[(0.194)^2 + (1.0 - 0.194)^2\right] + \left[(2547.1)(5.069) + (793.8)(16.681) + (1181.3)(33.306)\right]$$

$$M_p = 65309.1 \text{ in-kip} = 5442.4 \text{ ft-kip}$$

For a composite section in positive flexure to be considered compact, according to Article 6.10.6.2.2, the section must meet three requirements. The first states that the minimum yield strengths of the flanges must not exceed 70.0 ksi, which is met since 50 ksi steel is used throughout. The second is that the web satisfies the requirement of Article 6.10.2.1.1, which was evaluated earlier (see 3.2.2). The third is that the section satisfies the following web slenderness limit, where D_{cp} is the depth of the web in compression at the plastic moment.

$$\frac{2D_{cp}}{t_w} \leq 3.76\sqrt{\frac{E}{F_{yc}}}$$

Eq. 6.10.6.2.2-1

It was previously determined that the plastic neutral axis was in the top flange. Therefore, D_{cp} = 0, and this third requirement is met. Since all of the aforementioned requirements have been met, this section is classified as compact.

For compact composite sections in positive flexure, Article 6.10.7.1.2 states that the nominal flexural resistance, M_n, is computed as follows.

If $D_p \leq 0.1\ D_t$, then:

$$M_n = M_p$$

Eq. 6.10.7.1.2-1

Otherwise:

$$M_n = M_p\left(1.07 - 0.7\frac{D_p}{D_t}\right)$$

Eq. 6.10.7.1.2-2

D_p, the distance from the top of the concrete deck to the PNA, and D_t, the total depth of the composite section, are as follows:

$$D_p = 7.75 + (2.0 - 1.0) + 0.194 = 8.944 \text{ in}$$

$$D_t = 7.75 + 2.0 + 31.75 + 1.5 = 43.0 \text{ in}$$

$$0.1D_t = 4.30 \text{ in}$$

$$D_p > 0.1D_t$$

Therefore:

$$M_n = 5442.4\left(1.07 - 0.7\frac{8.944}{43.0}\right) = 5031.0 \text{ ft-kip}$$

To satisfy strength limit state requirements, the section must satisfy the following relation.

$$M_u + \frac{1}{3} f_l S_{xt} \le \phi_f M_n$$

Eq. 6.10.7.1.1-1

For this relation, f_l = 0 as wind forces and other lateral loads are being neglected at the finished state. From the moments generated for this girder, a maximum Strength I bending moment of 4669.4 ft-kip was found (see 3.5), indicating that this girder meets strength limit state requirements for flexure.

$$M_u \le \phi_f M_n$$

$$4669.4 \text{ ft-kip} < 1.00(5031.0 \text{ ft-kip}) \therefore OK(\text{Ratio} = 0.928)$$

3.6.4.2 Shear

The provisions of Article 6.10.9 are applied to determine whether sections meet strength limit state requirements for shear. As previously stated, the distributed shear forces were based on the interior girder distribution factor. Therefore, the shear capacity of an interior girder is computed. However, since the interior and exterior girders have the same dimensions, their shear capacities will be identical.

The first step is to determine the plastic shear capacity of the web, which is found as follows.

$$V_p = 0.58 F_{yw} D t_w \qquad \text{Eq. 6.10.9.2-2}$$

$$V_p = 0.58(50)(31.75)(0.5) = 460.4 \text{ kip}$$

The plastic shear capacity of the web is then modified by a value, *C*, to obtain the nominal shear resistance. *C* is simply the ratio of the shear-buckling resistance to the shear yield strength and is a function of the slenderness of the web. For this computation, a shear buckling coefficient, *k*, is introduced. However, as this web is unstiffened, the value of *k* is taken as a constant value of 5.0. Therefore, *C* is determined as follows.

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

$$1.12\sqrt{\frac{(29000)(5.0)}{(50)}} < \frac{31.75}{0.5} \le 1.40\sqrt{\frac{(29000)(5.0)}{(50)}}$$

$$60.3 < 63.5 < 75.4$$

Therefore:

$$C = \frac{1.12}{D/t_w}\sqrt{\frac{Ek}{F_{yw}}} = \frac{1.12}{63.5}\sqrt{\frac{(29000)(5.0)}{(50)}} = 0.950 \qquad \text{Eq. 6.10.9.3.2-5}$$

The nominal shear capacity of the web can now be determined.

$$V_n = V_{cr} = CV_p$$ Eq. 6.10.9.2-1

$$V_n = (0.950)(344.4) = 437.3 \text{ kip}$$

From the shears generated for this girder, a maximum Strength I shear of 271.6 kip was found (see 3.5), indicating that this girder meets strength limit state requirements for shear.

$$V_u \le \phi_v V_n$$ Eq. 6.10.9.1-1

$$271.6 \text{ kip} < (1.0)(437.3 \text{ kip}) \therefore OK(\text{Ratio} = 0.621)$$

3.6.4.3 Ductility

An additional ductility requirement is placed on composite sections in positive flexure. Specifically, sections shall meet the requirements in the relation below. For this requirement, as shown, the section performs satisfactorily.

$$D_p \le 0.42\ D_t$$ Eq. 6.10.7.3-1

$$8.944 \le (0.42)(43.0)$$

$$8.944 \text{ in} < 18.06 \text{ in} \therefore OK(\text{Ratio} = 0.495)$$

3.7 PERFORMANCE SUMMARY

A tabulated summary of all of the girder's performance ratios is presented below. As shown, the girder performs satisfactorily under all evaluated design checks, with the flexural capacity at the strength limit state governing (Ratio = 0.928).

<u>CONSTRUCTIBILITY</u>	
Compression Flange Yielding	
Strength I	0.676
Strength IV	0.720
Compression Flange Flexural Resistance	
Strength I	0.478
Strength IV	0.547
Web Bend Buckling	
Strength I	0.485
Strength IV	0.582
Tension Flange Yielding	
Strength I	0.489
Strength IV	0.527
<u>SERVICE LIMIT STATE</u>	
Elastic Deformations	0.742
Permanent Deformations	
Top Flange	0.519
Bottom Flange	0.918
<u>FATIGUE LIMIT STATE</u>	
Base Metal at Connection Plate Weld	
Top Flange	0.103
Bottom Flange	0.883
<u>STRENGTH LIMIT STATE</u>	
Moment	0.928
Shear	0.621
Ductility	0.495

CHAPTER 4: DESIGN ASSESSMENT – LIGHTEST WEIGHT ROLLED BEAM

4.1 INTRODUCTION

Contained in this chapter is a design assessment according to current AASHTO LRFD Specifications of a "Lightest Weight" rolled beam selected from the proposed SMDI Short Span Design. In this design assessment, an evaluation of the girder at the strength, service, and fatigue limit states is conducted. Additionally, an analysis is conducted to determine whether the girder meets constructibility requirements under typical construction loads as specified in Article 6.10.3.

4.2 GIRDER GEOMETRY

Rolled beams are selected from Sheet 201 of the SMDI Details. They are comprised of A507-50W steel (F_y = 50 ksi). For this evaluation, a "Selected Section" was chosen from the "Lightest Weight" design option. The properties of this selection, a W36×210, were obtained from the current edition of the AISC Steel Construction Manual, and are listed below:

$A_g = 61.9\ \text{in}^2$ $\qquad d = 36.7\ \text{in}$

$t_w = 0.830\ \text{in}$ $\qquad b_f = 12.2\ \text{in}$

$t_f = 1.36\ \text{in}$ $\qquad \dfrac{b_f}{2t_f} = 4.48$

$I_x = 13200\ \text{in}^4$ $\qquad S_x = 719\ \text{in}^3$

$Z_x = 833\ \text{in}^3$ $\qquad r_{ts} = 3.18\ \text{in}$

$h_o = 35.3\ \text{in}$ $\qquad J = 28.0\ \text{in}^4$

4.2.1 Section Properties

Section properties for the girder are listed on the following pages. For these calculations, all "*y*" distances are taken from the bottom of the bottom flange. Section properties are calculated for short-term composite sections (dividing the effective flange width by *n*) and long-term composite sections (dividing the effective flange width by 3*n*). As stated in Chapter 2, the modular ratio, *n*, for these bridges is taken as 8, and the effective flange widths of these bridges are as follows.

- For interior girders, 126 inches
- For exterior girders, 93.25 inches

Short Term Composite Section (Interior Girder)						
Shape	A (in²)	y (in)	Ay (in³)	I_0 (in⁴)	d (in)	I (in⁴)
Girder	61.9	18.35	1135.9	13200.0	15.17	27447.5
Slab	122.1	41.22	5030.8	610.9	-7.69	7836.1
Σ	184.0		6166.7			35283.6

Short Term Composite Section (Exterior Girder)						
Shape	A (in²)	y (in)	Ay (in³)	I_0 (in⁴)	d (in)	I (in⁴)
Girder	61.9	18.35	1135.9	13200.0	13.57	24595.1
Slab	90.3	41.22	3723.2	452.2	-9.30	8260.3
Σ	152.2		4859.1			32855.5

Long Term Composite Section (Interior Girder)						
Shape	A (in²)	y (in)	Ay (in³)	I_0 (in⁴)	d (in)	I (in⁴)
Girder	61.9	18.35	1135.9	13200.0	9.07	18290.6
Slab	40.7	41.22	1676.9	203.6	-13.80	7948.2
Σ	102.6		2812.8			26238.8

Long Term Composite Section (Exterior Girder)						
Shape	A (in²)	y (in)	Ay (in³)	I_0 (in⁴)	d (in)	I (in⁴)
Girder	61.9	18.35	1135.9	13200.0	7.48	16666.0
Slab	30.1	41.22	1241.1	150.7	-15.38	7275.5
Σ	92.0		2376.9			23941.5

4.2.2 Cross-Section Proportion Limits

The girders in this design evaluation were evaluated to meet the cross-section proportion limits of Article 6.10.2. For webs without longitudinal stiffeners, the following limit is employed from Article 6.10.2.1.1.

$$\frac{D}{t_w} \leq 150$$

$$\frac{36.7 - 2(1.36)}{0.830} \leq 150$$

$$40.9 \leq 150 \therefore OK$$

As previously stated, the following limits are employed for flange proportions. In addition to the limits set forth in Article 6.10.2.2, Article C6.10.3.4 specifies an additional limit for the compression flange, and is presented below. For this evaluation, the results show that the girder meets all applicable cross-section proportion limits.

$$\frac{b_f}{2t_f} \leq 12.0$$

$$4.48 \leq 12.0 \therefore OK$$

$$b_f \geq \frac{D}{6}$$

$$12.2 \geq \frac{36.7 - 2(1.36)}{6}$$

$$12.2 \geq 5.66 \therefore OK$$

$$t_f \geq 1.1\, t_w$$

$$1.36 \geq 1.1\,(0.830)$$

$$1.36 \geq 0.92 \therefore OK$$

$$0.1 \le \frac{I_{yc}}{I_{yt}} \le 10$$

$$0.1 \le \frac{(1.36)(12.2^3)/12}{(1.36)(12.2^3)/12} \le 10$$

$$0.1 \le 1.0 \le 10 \therefore OK$$

$$b_{fc} \ge \frac{L}{85}$$

$$12.2 \ge \frac{80(12)}{85}$$

$$12.2 \ge 11.3 \therefore OK$$

4.3 Dead Loads

The dead loads computed for this girder consist of the component and attachment dead load (DC) and the wearing surface dead load (DW) and are described herein.

4.3.1 Component and Attachment Dead Load (DC)

The dead load of structural components and nonstructural attachments are computed as follows. As previously stated, the DC load is divided into two components, the load applied to the noncomposite section (DC1) and the load applied to the long-term composite section (DC2). Loads such as the slab, overhang tapers, the Jersey barriers, and the SIP formwork are assumed to be equally distributed to all of the girders.

NONCOMPOSITE DEAD LOAD (DC1):

$$\text{Slab} = \frac{0.150}{4}\left[\left(\frac{8.0}{12}\right)\left(36+\frac{6.5}{12}\right)\right]$$ 0.914 kip/ft

$$\text{Haunch} = 0.150\left[\left(\frac{12.2}{12}\right)\left(\frac{2.0-1.36}{12}\right)\right]$$ 0.008 kip/ft

$$\text{Taper} = 0.150\left(\frac{2}{4}\right)\left[\frac{1}{2}\left(\frac{2.0}{12}\right)\left(\frac{30.25-12.2/2}{12}\right)\right]$$ 0.013 kip/ft

$$\text{SIP} = \frac{0.015}{4}3\left[10.5-\frac{12.2}{12}\right]$$ 0.107 kip/ft

$$\text{Girder} = \text{W}36\times 210$$ 0.210 kip/ft

$$\text{Misc.Details} = 5\%(0.210)$$ 0.011 kip/ft

1.263 kip/ft

COMPOSITE DEAD LOAD (DC2):

$$\text{Barrier} = \frac{2}{4}(0.304)$$ 0.152 kip/ft

0.152 kip/ft

4.3.2 Wearing Surface Dead Load (DW)

The dead load of the future wearing surface is applied across the clear roadway width of 34 feet. Like DC1 and DC2, loads are assumed to be equally distributed to all of the girders.

WEARING SURFACE DEAD LOAD (DW):

$$\text{Wearing Surface} = \frac{0.025}{4}(34)$$ 0.213 kip/ft

0.213 kip/ft

4.4 STRUCTURAL ANALYSIS

For this design evaluation, an approximate analysis is conducted which employs a line-girder analysis model. Dead loads, as stated earlier, are assumed to be evenly distributed to all girders. For live loads, live load distribution factors are used to distribute the vehicular live load to the line-girder model.

4.4.1 Live Load Distribution Factors (Article 4.6.2.2)

As previously stated, many of the bending moment distribution factors specified in Article 4.6.2.2 are a function of K_g, a longitudinal stiffness parameter. K_g is computed according to Eq. 4.6.2.2.1-1, and is shown below for an interior girder. Note that K_g does not need to be calculated for exterior girders since the lever rule, special analysis, and modified interior distribution factors serve as the exterior girder moment distribution factors. In addition, as previously stated, K_g must lie between 10,000 in^4 and 7,000,000 in^4 for the application of these distribution factors to be valid; as shown, this limit is clearly met.

$$K_g = n\left(I + Ae_g^{\ 2}\right)$$

$$K_g = 8\left[13200 + (61.9)\left(\frac{36.7}{2} + (2.0 - 1.36) + \frac{7.75}{2}\right)^2\right]$$

$$K_g = 364{,}495\ \text{in}^4$$

<u>4.4.1.1 General Live Load Distribution Factors</u>

Using the formulas and methods discussed in 2.4.2, moment and shear distribution factors for the strength and service limit states are calculated and listed as follows. Note that many of the values are repeated as the lever rule and special analysis apply to both moment and shear distribution.

STRENGTH AND SERVICE LIMIT STATE

Bending Moment - Interior Girder	
One Lane Loaded	0.535
Multiple Lanes Loaded	0.768
Shear - Interior Girder	
One Lane Loaded	0.780
Multiple Lanes Loaded	0.985
Bending Moment - Exterior Girder	
One Lane Loaded	0.771
Multiple Lanes Loaded	0.697
Special Analysis (1 Lane)	0.711
Special Analysis (2 Lanes)	0.843
Shear - Exterior Girder	
One Lane Loaded	0.771
Multiple Lanes Loaded	0.714
Special Analysis (1 Lane)	0.711
Special Analysis (2 Lanes)	0.843

4.4.1.2 Fatigue Live Load Distribution Factors

Using the formulas and methods discussed in 2.4.2, live load distribution factors for the fatigue limit state are calculated and listed below. To obtain these values, the previously computed distribution factors for one-lane-loaded scenarios (chosen since the fatigue loading consists of only one design truck) are divided by 1.20, the multiple presence factor for one lane loaded (as previously stated, multiple presence factors are not applied at the fatigue limit state).

FATIGUE LIMIT STATE

Bending Moment - Interior Girder	
One Lane Loaded	0.446
Bending Moment - Exterior Girder	
One Lane Loaded	0.643
Special Analysis (1 Lane)	0.593

4.4.1.3 Live Load Distribution Factor Summary

Governing distribution factors are listed below for interior and exterior girders. As shown, distribution factors for exterior girders, on average, exceed those for interior girders. Also, the distribution factor for deflection (computed earlier) is also presented.

SUMMARY	Interior	Exterior
Moment	0.768	0.843
Shear	0.985	0.843
Fatigue Moment	0.446	0.643
Deflection	0.500	0.500

4.5 ANALYSIS RESULTS

The tables in this section contain the moments, shears, and deflections resulting from structural analysis of the girder. Analyses were generated using the commercial software package LEAP CONSYS (2008), which idealizes the structure as a continuous line-girder. For these analyses, properties from the exterior girder were utilized for the stiffness of the line-girder model. This was due to the reduced section properties (due to a smaller effective flange width) and the increased live load distribution factors. An exception to this, however, is the set of distributed shears, which are distributed according to the interior girder (chosen for its high live load distribution factor).

Unfactored/Undistributed Moments (ft-kip)											
x/L	*DC1*	*DC2*	*DW*	*Truck*		*Lane*		*Tandem*		*Fatigue Truck*	
				(+)	*(−)*	*(+)*	*(−)*	*(+)*	*(−)*	*(+)*	*(−)*
0	0	0	0	0	0	0	0	0	0	0	0
0.1	363.3	43.9	61.2	451.2	0	184.3	0	350.0	0	387.2	0
0.2	645.9	78.1	108.8	787.2	0	327.7	0	620.0	0	659.2	0
0.3	847.7	102.5	142.8	1008.0	0	430.1	0	810.0	0	843.2	0
0.4	968.8	117.1	163.2	1136.0	0	491.5	0	920.0	0	931.2	0
0.5	1009.2	122.0	170.0	1160.0	0	512.0	0	950.0	0	904.0	0
0.6	968.8	117.1	163.2	1136.0	0	491.5	0	920.0	0	931.2	0
0.7	847.7	102.5	142.8	1008.0	0	430.1	0	810.0	0	843.2	0
0.8	645.9	78.1	108.8	787.2	0	327.7	0	620.0	0	659.2	0
0.9	363.3	43.9	61.2	451.2	0	184.3	0	350.0	0	387.2	0
1	0	0	0	0	0	0	0	0	0	0	0

Unfactored/Undistributed Shears (kip)										
x/L	*DC1*	*DC2*	*DW*	*Truck*		*Lane*		*Tandem*		
				(+)	*(−)*	*(+)*	*(−)*	*(+)*	*(−)*	
0	50.5	6.1	8.5	63.5	0	25.6	0	48.7	0	
0.1	40.4	4.9	6.8	56.4	-3.2	20.7	-0.3	43.8	-3.8	
0.2	30.3	3.7	5.1	49.2	-7.2	16.4	-1.0	38.8	-8.8	
0.3	20.2	2.4	3.4	42.0	-13.6	12.5	-2.3	33.8	-13.8	
0.4	10.1	1.2	1.7	34.8	-20.4	9.2	-4.1	28.8	-18.8	
0.5	0	0	0	27.6	-27.6	6.4	-6.4	23.8	-23.8	
0.6	-10.1	-1.2	-1.7	20.4	-34.8	4.1	-9.2	18.8	-28.8	
0.7	-20.2	-2.4	-3.4	13.6	-42.0	2.3	-12.5	13.8	-33.8	
0.8	-30.3	-3.7	-5.1	7.2	-49.2	1.0	-16.4	8.8	-38.8	
0.9	-40.4	-4.9	-6.8	3.2	-56.4	0.3	-20.7	3.8	-43.8	
1	-50.5	-6.1	-8.5	0	-63.5	0	-25.6	0	-48.7	

Unfactored/Undistributed Deflections (in)				
x/L	*Truck*		*Lane*	
	(+)	*(−)*	*(+)*	*(−)*
0	0	0	0	0
0.1	0.40	0	0.19	0
0.2	0.76	0	0.37	0
0.3	1.05	0	0.50	0
0.4	1.23	0	0.59	0
0.5	1.29	0	0.62	0
0.6	1.23	0	0.59	0
0.7	1.05	0	0.50	0
0.8	0.76	0	0.37	0
0.9	0.40	0	0.19	0
1	0	0	0	0

Unfactored/Distributed Moments (ft-kip)							
x/L	*1.33 Truck + Lane*		*1.33 Tandem + Lane*		*DF*	*LL + IM*	
	(+)	*(−)*	*(+)*	*(−)*		*(+)*	*(−)*
0	0	0	0	0	0.843	0	0
0.1	784.4	0	649.8	0	0.843	661.2	0
0.2	1374.7	0	1152.3	0	0.843	1158.6	0
0.3	1770.7	0	1507.4	0	0.843	1492.5	0
0.4	2002.4	0	1715.1	0	0.843	1687.7	0
0.5	2054.8	0	1775.5	0	0.843	1731.9	0
0.6	2002.4	0	1715.1	0	0.843	1687.7	0
0.7	1770.7	0	1507.4	0	0.843	1492.5	0
0.8	1374.7	0	1152.3	0	0.843	1158.6	0
0.9	784.4	0	649.8	0	0.843	661.2	0
1	0	0	0	0	0.843	0	0

x/L	*1.33 Truck + Lane* (+)	*1.33 Truck + Lane* (−)	*1.33 Tandem + Lane* (+)	*1.33 Tandem + Lane* (−)	*DF*	*LL + IM* (+)	*LL + IM* (−)
0	110.1	0	90.4	0	0.985	108.4	0
0.1	95.7	-4.5	78.9	-5.2	0.985	94.3	-5.2
0.2	81.8	-10.6	67.9	-12.7	0.985	80.6	-12.5
0.3	68.4	-20.4	57.4	-20.6	0.985	67.4	-20.3
0.4	55.5	-31.2	47.5	-29.0	0.985	54.7	-30.8
0.5	43.1	-43.1	38.0	-38.0	0.985	42.5	-42.5
0.6	31.2	-55.5	29.0	-47.5	0.985	30.8	-54.7
0.7	20.4	-68.4	20.6	-57.4	0.985	20.3	-67.4
0.8	10.6	-81.8	12.7	-67.9	0.985	12.5	-80.6
0.9	4.5	-95.7	5.2	-78.9	0.985	5.2	-94.3
1	0	-110.1	0	-90.4	0.985	0	-108.4

Unfactored/Distributed Shears (kip)

x/L	*1.25 DC1*	*1.25 DC2*	*1.50 DW*	*1.75 LL + IM* (+)	*1.75 LL + IM* (−)	*Strength I* (+)	*Strength I* (−)
0	0	0	0	0	0	0	0
0.1	454.1	54.9	91.8	1157.0	0	1757.8	600.8
0.2	807.3	97.6	163.2	2027.6	0	3095.7	1068.1
0.3	1059.6	128.1	214.2	2611.8	0	4013.7	1401.9
0.4	1211.0	146.4	244.8	2953.5	0	4555.7	1602.2
0.5	1261.4	152.5	255.0	3030.8	0	4699.8	1668.9
0.6	1211.0	146.4	244.8	2953.5	0	4555.7	1602.2
0.7	1059.6	128.1	214.2	2611.8	0	4013.7	1401.9
0.8	807.3	97.6	163.2	2027.6	0	3095.7	1068.1
0.9	454.1	54.9	91.8	1157.0	0	1757.8	600.8
1	0	0	0	0	0	0	0

Strength I Moments (ft-kip)

x/L	*1.25 DC1*	*1.25 DC2*	*1.50 DW*	*1.75 LL + IM* (+)	*1.75 LL + IM* (−)	*Strength I* (+)	*Strength I* (−)
0	63.1	7.6	12.8	189.8	0	273.2	83.4
0.1	50.5	6.1	10.2	165.0	-9.0	231.8	57.7
0.2	37.8	4.6	7.7	141.0	-21.8	191.1	28.2
0.3	25.2	3.1	5.1	117.9	-35.5	151.3	-2.1
0.4	12.6	1.5	2.6	95.7	-53.8	112.4	-37.1
0.5	0	0	0	74.3	-74.3	74.3	-74.3
0.6	-12.6	-1.5	-2.6	53.8	-95.7	37.1	-112.4
0.7	-25.2	-3.1	-5.1	35.5	-117.9	2.1	-151.3
0.8	-37.8	-4.6	-7.7	21.8	-141.0	-28.2	-191.1
0.9	-50.5	-6.1	-10.2	9.0	-165.0	-57.7	-231.8
1	-63.1	-7.6	-12.8	0	-189.8	-83.4	-273.2

Strength I Shears (kip)

Service II Moments (ft-kip)							
x/L	*1.00 DC1*	*1.00 DC2*	*1.00 DW*	*1.30 LL + IM*		*Service II*	
				(+)	*(−)*	*(+)*	*(−)*
0	0	0	0	0	0	0	0
0.1	363.3	43.9	61.2	859.5	0	1327.9	468.4
0.2	645.9	78.1	108.8	1506.2	0	2339.0	832.7
0.3	847.7	102.5	142.8	1940.2	0	3033.2	1093.0
0.4	968.8	117.1	163.2	2194.1	0	3443.2	1249.1
0.5	1009.2	122.0	170.0	2251.5	0	3552.6	1301.2
0.6	968.8	117.1	163.2	2194.1	0	3443.2	1249.1
0.7	847.7	102.5	142.8	1940.2	0	3033.2	1093.0
0.8	645.9	78.1	108.8	1506.2	0	2339.0	832.7
0.9	363.3	43.9	61.2	859.5	0	1327.9	468.4
1	0	0	0	0	0	0	0

Service I Deflections (in)							
x/L	*Truck*		*0.25 Truck + Lane*		*DF*	*Service I*	
	(+)	*(−)*	*(+)*	*(−)*		*(+)*	*(−)*
0	0	0	0	0	0.500	0	0
0.1	0.53	0	0.32	0	0.500	0.27	0
0.2	1.01	0	0.62	0	0.500	0.51	0
0.3	1.40	0	0.85	0	0.500	0.70	0
0.4	1.64	0	1.00	0	0.500	0.82	0
0.5	1.72	0	1.05	0	0.500	0.86	0
0.6	1.64	0	1.00	0	0.500	0.82	0
0.7	1.40	0	0.85	0	0.500	0.70	0
0.8	1.01	0	0.62	0	0.500	0.51	0
0.9	0.53	0	0.32	0	0.500	0.27	0
1	0	0	0	0	0.500	0	0

Fatigue Moments (ft-kip)					
x/L	*LL + IM*		*DF*	*1.50 (LL + IM)*	
	(+)	*(−)*		*(+)*	*(−)*
0	0	0	0.643	0	0
0.1	445.3	0	0.643	429.4	0
0.2	758.1	0	0.643	731.0	0
0.3	969.7	0	0.643	935.0	0
0.4	1070.9	0	0.643	1032.6	0
0.5	1039.6	0	0.643	1002.5	0
0.6	1070.9	0	0.643	1032.6	0
0.7	969.7	0	0.643	935.0	0
0.8	758.1	0	0.643	731.0	0
0.9	445.3	0	0.643	429.4	0
1	0	0	0.643	0	0

4.6 LIMIT STATE EVALUATIONS

Presented in this section is an evaluation of a typical exterior girder of the chosen bridge layout. The exterior girder was chosen due to the reduced section properties (due to a smaller effective flange width) and the increased live load distribution factors. In this evaluation, all of the aforementioned limit states, including strength, service, and fatigue, are assessed. In addition, a constructibility evaluation is also performed.

4.6.1 Constructibility

The provisions of Article 6.10.3 are employed to ensure adequate performance related to yielding of the flanges, flexural resistance of the compression flange, and web bend-buckling resistance during stages of construction. During construction, the noncomposite girder must have sufficient capacity to resist construction force effects. Therefore, the capacity of the noncomposite girder must be evaluated.

4.6.1.1 Compression Flange Resistance

The first step is determining which Article is applicable in determining the flexural capacity of the noncomposite girder. Article 6.10.6.2.3 states that Appendix A6 may be employed if the girder meets certain limits. This is preferable, as Appendix A6 allows the girder's noncomposite capacity to exceed the yield moment. For Appendix A6 to be applicable, the flanges' yield strengths must not exceed 70.0 ksi (this limit is met since F_y = 50 ksi), the skew must not exceed 20° (no skew is present) and two additional limits must be met.

$$\frac{2D_c}{t_w} < 5.7\sqrt{\frac{E}{F_{yc}}}$$

Eq. 6.10.6.2.3-1

$$\frac{I_{yc}}{I_{yt}} \geq 0.3$$

Eq. 6.10.6.2.3-2

The depth of the web in compression of the noncomposite girder in the elastic range, D_c, is the distance from the top of the web to the neutral axis of the girder. In addition, I_{yc} and I_{yt} have already been determined for this girder (see 4.2.2). Therefore, the evaluation of these limits is as follows.

$$D_c = \frac{36.7 - 2(1.36)}{2} = 16.99 \text{ in}$$

$$\frac{2D_c}{t_w} < 5.7\sqrt{\frac{E}{F_{yc}}}$$

$$\frac{2(16.99)}{0.830} < 5.7\sqrt{\frac{29000}{50}}$$

$$40.94 < 137.27 \therefore OK$$

$$\frac{I_{yc}}{I_{yt}} \geq 0.3$$

$$1.0 > 0.3 \therefore OK$$

Therefore, Appendix A6 may be employed. To employ Appendix A6, the yield moment, M_y, and the plastic moment, M_p, of the noncomposite girder must be computed. The yield moment of the girder is simply the yield stress, F_y, multiplied by the section modulus, S_x. The plastic moment of the girder is simply the yield stress, F_y, multiplied by the plastic section modulus, Z_x.

$$M_y = F_y S_x$$

$$M_y = \frac{(50)(719)}{12}$$

$$M_y = 2995.8 \text{ ft-kip}$$

$$M_p = F_y Z_x$$

$$M_p = \frac{(50)(833)}{12}$$

$$M_p = 3470.8 \text{ ft-kip}$$

The first step in employing Appendix A6 is to determine whether the section is a compact web section or a noncompact web section. Compact web sections are those that meet the following requirements.

$$\frac{2D_{cp}}{t_w} \leq \lambda_{pw(D_{cp})}$$

Eq. A6.2.1-1

D_{cp} is the depth of the web in compression at the plastic moment. Since the plastic neutral axis of a rolled beam is at the same location as the elastic neutral axis, this value is the same as D_c, or 16.99 inches. $\lambda_{pw(D_{cp})}$ is then computed as follows.

$$\lambda_{rw} = 5.7\sqrt{\frac{E}{F_{yc}}}$$

Eq. A6.2.1-3

$$\lambda_{rw} = 5.7\sqrt{\frac{29000}{50}}$$

$$\lambda_{rw} = 137.27$$

$$\lambda_{pw(D_{cp})} = \frac{\sqrt{\frac{E}{F_{yc}}}}{\left(0.54\frac{M_p}{R_h M_y} - 0.09\right)^2} \leq \lambda_{rw}\left(\frac{D_{cp}}{D_c}\right)$$

Eq. A6.2.1-2

$$\lambda_{pw(D_{cp})} = \frac{\sqrt{\frac{29000}{50}}}{\left[0.54\frac{3470.8}{(1.0)(2995.8)} - 0.09\right]^2} \leq 137.27\left(\frac{16.99}{16.99}\right)$$

$$\lambda_{pw(D_{cp})} = 83.95 < 137.24$$

$$\lambda_{pw(D_{cp})} = 83.95$$

Therefore, as shown below, the girder qualifies as a compact web section.

$$\frac{2D_{cp}}{t_w} \leq \lambda_{pw(D_{cp})}$$

$$\frac{2(16.99)}{0.830} \leq 83.95$$

$$40.94 < 83.95 \therefore \text{Web is compact}$$

To determine the flexural capacity of the compression flange for a compact web section, a web plastification factor for the compression flange, R_{pc}, must be determined. This essentially determines how much the girder's flexural capacity can exceed M_y. In addition, they can account for the influence of web slenderness on the maximum potential flexural resistance. The web plastification factor is computed as follows.

$$R_{pc} = \frac{M_p}{M_{yc}}$$

Eq. A6.2.1-4

$$R_{pc} = \frac{3470.8}{2995.8}$$

$$R_{pc} = 1.159$$

The flexural capacity of the compression flange is a function of the slenderness ratio of the flange and whether or not the flange is classified as compact. The web plastification factor computed earlier is then used to compute the section's flexural capacity. For flanges to be classified as compact, the slenderness ratio for the flange, λ_f, must be less than a limiting value, λ_{pf}. As shown, the flange meets the requirements for compactness.

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

Eq. A6.3.2-3

$$\lambda_f = 4.48$$

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_y}}$$ Eq. A6.3.2-4

$$\lambda_{pf} = 0.38\sqrt{\frac{29000}{50}}$$

$$\lambda_{pf} = 9.15$$

$\lambda_f < \lambda_{pf}$ $\therefore$ Flange is compact

Therefore, the flexural capacity of the compression flange is computed as follows. Equation A6.3.2-1 yields the flexural capacity in terms of the girder's overall capacity, not the flange's capacity. To obtain the capacity of the flange, in accordance with Article 6.10.3.2.1, the flange's capacity can be computed by dividing the girder's capacity by S_{xc}.

$$M_{nc} = R_{pc}M_{yc}$$ Eq. A6.3.2-1

$$M_{nc} = (1.159)(2995.8)$$

$$M_{nc} = 3470.8 \text{ ft-kip}$$

$$F_{nc} = \frac{M_{nc}}{S_{xc}}$$

$$F_{nc} = \frac{3470.8(12)}{719}$$

$$F_{nc} = 57.93 \text{ ksi}$$

4.6.1.2 Major Axis and Lateral Flange Bending Stresses

The next step in performing this constructability analysis is to determine the major axis and lateral flange bending stresses that the girder will be subjected to during construction. First, major-axis bending stresses will be computed. As previously stated, the deck is assumed to be cast in one pour; therefore, major axis bending stresses will be computed according to DC1. From analysis results, the unfactored DC1 moment was found to be 1009.2 ft-kip. Therefore, major axis bending stresses are as follows. For this computation, the Strength IV load combination is employed in addition to Strength I. This is because, during construction, the bridge is subjected to very high dead to live load force effect ratios. In addition, since this section is a symmetric rolled beam, the top flange stresses during construction will be equal (in magnitude) to the bottom flange stresses.

STRENGTH I:

$$f_{bu} = \frac{1.25(1009.2)(12)}{719} = 21.05 \text{ ksi}$$

STRENGTH IV:

$$f_{bu} = \frac{1.50(1009.2)(12)}{719} = 25.26 \text{ ksi}$$

Next, stresses due to lateral flange bending forces from construction loads must be computed. Before calculating lateral flange bending stresses, a determination must be made regarding whether or not a second-order analysis must be carried out for compressive stresses. To make this determination, a number of variables must be computed, including the effective radius of gyration for lateral torsional buckling, r_t, and the limiting unbraced length to achieve the maximum flexural resistance, L_p. For rolled beams, the AISC Steel Construction Manual provides a value for r_t (or r_{ts} as it is listed); for a W36×210, the value is 3.18 inches.

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

$$L_p = 1.0(3.18)\sqrt{\frac{29000}{50}}$$

$$L_p = 76.58 \text{ in}$$

A moment gradient modifier, C_b, must then be computed in order to determine whether or not a second-order analysis must be carried out. C_b is a coefficient which accounts for different moment gradients on lateral torsional buckling.

It was previously determined that Appendix A6 was applicable for this noncomposite girder. Therefore, to compute C_b, moments must be found at various lengths along the unbraced segment of interest. For this structure, the unbraced length, L_b, is simply the spacing of diaphragms, or 20 feet.

From analysis results (interpolating between tenth points), the following unfactored moments were obtained for the unbraced segment at midspan. It should be noted that since deck casting moments will result solely from DC1, this calculation for C_b will be valid for both Strength I and Strength IV load combinations.

M_{mid} = major-axis bending moment at the middle of the unbraced length = 938.5 ft-kip
M_0 = major-axis bending moment at one end of the unbraced segment = 746.8 ft-kip
M_2 = major-axis bending moment at the other end of the unbraced segment = 1009.2 ft-kip

C_b is then calculated as follows (since M_{mid}/M_2 is less than 1.0).

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

$$M_1 = 2(938.5) - 1009.2 \geq 746.8$$

$$M_1 = 867.9 > 746.8$$

$$M_1 = 867.9 \text{ ft-kip}$$

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

$$C_b = 1.75 - 1.05\left(\frac{867.9}{1009.2}\right) + 0.3\left(\frac{867.9}{1009.2}\right)^2 \leq 2.3$$

$$C_b = 1.069 < 2.3$$

$$C_b = 1.069$$

The limit for first-order elastic analyses can now be computed as follows.

$$L_b \le 1.2 L_p \sqrt{\frac{C_b R_b}{M_u / M_{yc}}}$$ Eq. 6.10.1.6-3

STRENGTH I:

$$240 \le 1.2(76.58)\sqrt{\frac{(1.069)(1.0)}{1.25(1009.2)/2995.8}}$$

$$240 > 146.4 \therefore \text{Not Satisfied}$$

STRENGTH IV:

$$240 \le 1.2(76.58)\sqrt{\frac{(1.069)(1.0)}{1.50(1009.2)/2995.8}}$$

$$150 >> 133.7 \therefore \text{Not Satisfied}$$

Therefore, a second-order analysis must be performed for the Strength I and Strength IV load combinations. Article 6.10.1.6 provides an approximate method for computing second-order compression-flange lateral bending stresses by multiplying first-order values by an amplification factor (this calculation is not required for tensile stresses). This amplification factor is a function of the compression flange's elastic lateral torsional buckling stress, F_{cr}. To compute F_{cr}, the height between the centerline of the flanges, h, and the St. Venant torsional constant, J, must be calculated. The AISC Steel Construction Manual provides these values for rolled shapes. For a W36×210:

- $h = h_o = 35.3$ in.
- $J = 28.0$ in^4

F_{cr} is then computed as follows according to the provisions for Appendix A6. It should be noted that, according to Article C6.10.1.6, F_{cr} is not limited to $R_b R_h F_{yc}$.

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

Eq. A6.3.3-8

$$F_{cr} = \frac{(1.069)(\pi^2)(29000)}{\left(\frac{240}{3.18}\right)^2} \sqrt{1 + 0.078 \frac{28.0}{(719)(35.3)} \left(\frac{240}{3.18}\right)^2}$$

$$F_{cr} = 65.56 \text{ ksi}$$

The amplification factor for first-order lateral flange bending stresses is as follows.

$$AF = \frac{0.85}{1 - \frac{M_u}{F_{cr} S_{xc}}}$$

Eq. 6.10.1.6-5

STRENGTH I:

$$AF = \frac{0.85}{1 - \frac{1.25(1009.2)(12)}{(65.56)(719)}} = 1.252$$

STRENGTH IV:

$$AF = \frac{0.85}{1 - \frac{1.50(1009.2)(12)}{(65.56)(719)}} = 1.383$$

To compute deck overhang loads, lateral forces are computed by determining the force statically equivalent to the couple resulting from the eccentric vertical loads. This computation involves the angle, α, between the overhang bracket and the web of the girder. The bracket is assumed to extend from the end of the overhang to the web-bottom flange junction. The angle between the web of the girder and the bracket, along with the lateral force relation, are as follows.

$$F_l = F \tan(\alpha)$$

$$\alpha = \tan^{-1}\left[\frac{30.25}{36.7 - 2(1.36)}\right] = 41.68°$$

In addition, half of the wet concrete overhang load is assumed to act on the overhang bracket, and is computed as follows.

$$\frac{150}{2}\left(\frac{1}{144}\right)\left[(8.0)(30.25)+\frac{1}{2}\left(30.25-\frac{12.2}{2}\right)(2.0)+(2.0-1.36)\left(\frac{12.2}{2}\right)\right]=140.7\frac{\text{lb}}{\text{ft}}$$

The lateral forces, bending moments, and lateral stresses are summarized as follows. Lateral bending moments are computed according to the approximations discussed in 2.3.2.2. To compute lateral stresses from lateral bending moments, moments are divided by the major-axis section modulus of the flange, or $(t_f)(b_f)^2/6$.

Lateral Flange Bending Moments & First-Order Stresses							
Components	F / P	tan(α)	F_l / P_l	L_b (ft)	M_l ("k)	S_l (in³)	f_l (ksi)
Deck Weight (lb/ft)	140.7	0.890	125.2	20	50.09	33.74	1.48
Overhang Deck Forms (lb/ft)	40	0.890	35.6	20	14.24	33.74	0.42
Screed Rail (lb/ft)	85	0.890	75.7	20	30.27	33.74	0.90
Railing (lb/ft)	25	0.890	22.3	20	8.90	33.74	0.26
Walkway (lb.ft)	125	0.890	111.3	20	44.51	33.74	1.32
Finishing Machine (lb)	3000	0.890	2670.7	20	80.12	33.74	2.37

Factored lateral flange bending stresses are computed below. Note that, for the Strength IV load combination, no live loads are considered; therefore the finishing machine load is neglected. Also, the limit specified in Equation 6.10.1.6-1, which limits lateral flange bending stresses to 60% of F_y, is also met.

Factored First-Order Lateral Flange Bending Stresses				
Components	Strength I		Strength IV	
	γ_i	f_l (ksi)	γ_i	f_l (ksi)
Deck Weight (lb/ft)	1.25	1.86	1.50	2.23
Overhang Deck Forms (lb/ft)	1.50	0.63	1.50	0.63
Screed Rail (lb/ft)	1.50	1.35	1.50	1.35
Railing (lb/ft)	1.50	0.40	1.50	0.40
Walkway (lb.ft)	1.50	1.98	1.50	1.98
Finishing Machine (lb)	1.50	3.56	-	-
		9.77		6.58

4.6.1.3 Limit State Evaluation

The nominal bend-buckling resistance, F_{crw}, shall be calculated as follows. Note that F_{crw} shall not exceed the smaller of R_hF_{yc} (50 ksi) or F_{yw}/0.7 (71.4 ksi).

$$k = \frac{9}{(D_c / D)^2}$$

Eq. 6.10.1.9.1-2

$$k = \frac{9}{(16.99/33.98)^2}$$

$$k = 36.0$$

$$F_{crw} = \frac{0.9Ek}{(D/t_w)^2}$$

Eq. 6.10.1.9.1-1

$$F_{crw} = \frac{0.9(29000)(36.0)}{(33.68/0.830)^2}$$

$$F_{crw} = 560.6 \text{ ksi} > 50 \text{ ksi}$$

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

The limit states are evaluated as follows. As shown, the girder performs satisfactorily under all applicable constructibility limit states. Note that the second order amplification factor is not applied to tensile stresses.

COMPRESSION FLANGE YIELDING

$$f_{bu} + f_l \le \phi_f R_h F_{yc}$$

Strength I: $21.05 + 1.252(9.77) < (1.00)(1.0)(50) \Rightarrow 33.28 \text{ ksi} < 50 \text{ ksi} \therefore OK \ (\text{Ratio} = 0.666)$

Strength IV: $25.26 + 1.383(6.58) < (1.00)(1.0)(50) \Rightarrow 34.36 \text{ ksi} < 50 \text{ ksi} \therefore OK \ (\text{Ratio} = 0.687)$

COMPRESSION FLANGE FLEXURAL RESISTANCE

$$f_{bu} + \frac{1}{3} f_l \leq \phi_f F_{nc}$$

Strength I: $21.05 + \frac{1.252(9.77)}{3} < (1.00)(57.93) \Rightarrow 25.13 \text{ ksi} < 57.93 \text{ ksi} \therefore OK \; (\text{Ratio} = 0.434)$

Strength IV: $25.26 + \frac{1.383(6.58)}{3} < (1.00)(57.93) \Rightarrow 28.3 \text{ ksi} < 57.93 \text{ ksi} \therefore OK \; (\text{Ratio} = 0.488)$

WEB BEND-BUCKLING RESISTANCE

$$f_{bu} \leq \phi_f F_{crw}$$

Strength I: $21.05 < (1.00)(50) \Rightarrow 21.05 \text{ ksi} < 50 \text{ ksi} \therefore OK \; (\text{Ratio} = 0.421)$

Strength IV: $25.26 < (1.00)(50) \Rightarrow 25.26 \text{ ksi} < 50 \text{ ksi} \therefore OK \; (\text{Ratio} = 0.505)$

TENSION FLANGE YIELDING

$$f_{bu} + f_l \leq \phi_f R_h F_{yt}$$

Strength I: $21.05 + 9.77 < (1.00)(1.0)(50) \Rightarrow 30.82 \text{ ksi} < 50 \text{ ksi} \therefore OK \; (\text{Ratio} = 0.616)$

Strength IV: $25.26 + 6.58 < (1.00)(1.0)(50) \Rightarrow 31.84 \text{ ksi} < 50 \text{ ksi} \therefore OK \; (\text{Ratio} = 0.637)$

4.6.2 Service Limit State

The service limit state is evaluated according to the provisions of Articles 6.10.4.1 (governing elastic deformations) and 6.10.4.2 (governing permanent deformations).

4.6.2.1 Elastic Deformations

The elastic deformation limit state, as previously stated, is evaluated against a maximum deformation of $L/800$, or 1.2 inches. From the analysis results, a maximum live load deflection of 0.858 inches was determined. Therefore, this meets elastic deformation requirements (Ratio = 0.715).

4.6.2.2 Permanent Deformations

The first step in evaluating the girder's performance under permanent deformation limits is to determine the girder's service level stresses. This will be derived solely from gravity and vehicular loadings, as lateral loads are not being considered at the service limit state in this design evaluation.

From the analysis results, the following Service II moments were found.

$1.00\, M_{DC1} = 1009.2 \text{ ft-kip}$

$1.00\, M_{DC2} = 122.0 \text{ ft-kip}$

$1.00\, M_{DW} = 170.0 \text{ ft-kip}$

$1.30\, M_{LL+IM} = 2251.5 \text{ ft-kip}$

Using these moments, Service II stresses for the top and bottom flange are found as follows. Therefore, according to Equations 6.10.4.2.2-1 and 6.10.4.2.2-2, respectively, the flanges are shown to meet the requirements for permanent deformations at the service limit state.

TOP FLANGE:

$$f_f = \frac{(1009.2)(12)}{719} + \frac{(122.0 + 170.0)(12)}{2203.1} + \frac{(2251.5)(12)}{6870.6} = 22.37 \text{ ksi}$$

$$f_f \le 0.95 R_h F_{yf}$$

$$22.37 < 0.95(1.0)(50) \Rightarrow 22.37 \text{ ksi} < 47.5 \text{ ksi} \therefore OK(\text{Ratio} = 0.471)$$

BOTTOM FLANGE:

$$f_f = \frac{(1009.2)(12)}{719} + \frac{(122.0 + 170.0)(12)}{926.8} + \frac{(2251.5)(12)}{1029.4} = 46.87 \text{ ksi}$$

$$f_f + \frac{f_l}{2} \le 0.95 R_h F_{yf}$$

$$46.87 + \frac{0}{2} < 0.95(1.0)(50) \Rightarrow 46.87 \text{ ksi} < 47.5 \text{ ksi} \therefore OK(\text{Ratio} = 0.987)$$

4.6.3 Fatigue Limit State

As previously discussed, the detail chosen for these design evaluations is the base metal at the weld joining the lateral bracing connection plates at interior diaphragms. These details are evaluated for the Fatigue I load combination for infinite life, with a nominal fatigue resistance of 12.0 ksi, previously determined as the constant amplitude fatigue threshold.

From the previously determined factored fatigue moments, a fatigue moment of 1002.5 ft-kip was determined (see 4.5) at the diaphragm location at midspan. Since this is a simple-span bridge, a minimum fatigue moment of zero was found. Therefore, a fatigue stress range can be found for both the top flange and bottom flange by determining the stress resulting from the calculated moment. As shown, this detail performs satisfactorily.

TOP FLANGE

$$\gamma(\Delta f) = \frac{1002.5(12)(3.42)}{32855.5} = 1.29 \text{ ksi}$$

$$1.29 \text{ ksi} < 12.0 \text{ ksi} \therefore OK(\text{Ratio} = 0.108)$$

BOTTOM FLANGE

$$\gamma(\Delta f) = \frac{1002.5(12)(30.56)}{32855.5} = 11.53 \text{ ksi}$$

$$11.52 \text{ ksi} < 12.0 \text{ ksi} \therefore OK(\text{Ratio} = 0.960)$$

4.6.4 Strength Limit State

At the strength limit state, as specified in Article 6.10.6, the girder must meet requirements for flexure and shear as well as a ductility requirement. Each of these criteria will be evaluated.

4.6.4.1 Flexure

For flexure, in order to determine a section's capacity, a determination must be made regarding whether the section is classified as compact or noncompact. For this determination, the section's plastic moment capacity must be calculated. For this evaluation, the reinforcement in the concrete slab is conservatively neglected.

The first step in determining the section's plastic moment capacity is to determine the plastic forces in each of the section's components.

$$P_s = 0.85 f_c' b_s t_s = 0.85(4)(93.25)(7.75) = 2547.1 \text{ kip}$$

$$P_{tf} = F_y b_f t_f = (50)(12.2)(1.36) = 829.6 \text{ kip}$$

$$P_b = F_y A_g - P_{tf} = (50)(61.9) - 829.6 = 2265.4 \text{ kip}$$

$$P_s + P_t > P_b \therefore \text{PNA is in the top flange}$$

Next, the location of the plastic neutral axis (PNA) must be determined (measured from the top of the top flange).

$$P_s + F_y b_f \bar{Y} = P_s + F_y b_f \left(t_f - \bar{Y}\right) + P_b$$

$$\therefore \bar{Y} = \frac{P_b - P_s + F_y b_f t_f}{2F_y b_f} = \frac{2265.4 - 2547.1 + (50)(12.2)(1.36)}{2(50)(12.2)} = 0.523 \text{ in}$$

Next, the distances of the individual components from the location of PNA are computed.

$$d_s = (2.0 - 1.36) + 0.523 + \frac{7.75}{2} = 5.038 \text{ in}$$

$$d_{tf} = \frac{0.523}{2} = 0.261 \text{ in}$$

$$d_b = \frac{36.7}{2} - 0.523 = 17.827 \text{ in}$$

The plastic moment of the composite section, M_p, can now be evaluated.

$$M_p = P_s d_s + F_y A_g d_b + 2\left(F_y b_f \bar{Y}\right)\left(\frac{\bar{Y}}{2}\right)$$

$$M_p = \frac{(2457.1)(5.038) + (50)(61.9)(17.827) + 2(50)(12.2)(0.523)\left(\frac{0.523}{2}\right)}{12}$$

$$M_p = 5643.4 \text{ ft-kip}$$

For a composite section in positive flexure to be considered compact, according to Article 6.10.6.2.2, the section must meet three requirements. The first states that the minimum yield strengths of the flanges must not exceed 70.0 ksi, which is met since 50 ksi steel is used. The second is that the web satisfies the requirement of Article 6.10.2.1.1, which was evaluated earlier (see 4.2.2). The third is that the section satisfies the following web slenderness limit, where D_{cp} is the depth of the web in compression at the plastic moment.

$$\frac{2D_{cp}}{t_w} \le 3.76\sqrt{\frac{E}{F_{yc}}}$$

Eq. 6.10.6.2.2-1

It was previously determined that the plastic neutral axis was in the top flange. Therefore, $D_{cp} = 0$, and this third requirement is met. Since all of the aforementioned requirements have been met, this section is classified as compact.

For compact composite sections in positive flexure, Article 6.10.7.1.2 states that the nominal flexural resistance, M_n, is computed as follows.

If $D_p \le 0.1\ D_t$, then:

$$M_n = M_p$$

Eq. 6.10.7.1.2-1

Otherwise:

$$M_n = M_p\left(1.07 - 0.7\frac{D_p}{D_t}\right)$$

Eq. 6.10.7.1.2-2

D_p, the distance from the top of the concrete deck to the plastic neutral axis, and D_t, the total depth of the composite section, are as follows:

$$D_p = 7.75 + (2.0 - 1.36) + 0.523 = 8.913\text{ in}$$

$$D_t = 7.75 + (2.0 - 1.36) + 36.7 = 45.09\text{ in}$$

$$0.1D_t = 4.51\text{ in}$$

$$D_p > 0.1D_t$$

Therefore:

$$M_n = 5643.4\left(1.07 - 0.7\frac{8.913}{45.09}\right) = 5257.6\text{ ft-kip}$$

To satisfy strength limit state requirements, the section must satisfy the following relation.

$$M_u + \frac{1}{3} f_l S_{xt} \le \phi_f M_n$$

Eq. 6.10.7.1.1-1

For this relation, f_l = 0 as wind forces and other lateral loads are being neglected at the finished state. From the moments generated for this girder, a maximum Strength I bending moment of 4699.8 ft-kip was found (see 4.5), indicating that this girder meets strength limit state requirements for flexure.

$$M_u \le \phi_f M_n$$

$$4699.8\text{ ft-kip} < 1.00(5257.6\text{ ft-kip}) \therefore OK(\text{Ratio} = 0.894)$$

4.6.4.2 Shear

The provisions of Article 6.10.9 are applied to determine whether sections meet strength limit state requirements for shear. As previously stated, the distributed shear forces were based on the interior girder distribution factor. Therefore, the shear capacity of an interior girder is computed. However, since the interior and exterior girders are the same, their shear capacities will be identical.

The first step is to determine the plastic shear capacity of the web, which is found as follows.

$$V_p = 0.58 F_{yw} D t_w \qquad \text{Eq. 6.10.9.2-2}$$

$$V_p = 0.58(50)(33.98)(0.830) = 817.9 \text{ kip}$$

The plastic shear capacity of the web is then modified by a value, *C*, to obtain the nominal shear resistance. *C* is simply the ratio of the shear-buckling resistance to the shear yield strength and is a function of the slenderness of the web. For this computation, a shear buckling coefficient, *k*, is introduced. However, as this web is unstiffened, the value of *k* is taken as a constant value of 5.0. Therefore, *C* is determined as follows.

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

$$\frac{33.98}{0.830} \le 1.12\sqrt{\frac{(29000)(5.0)}{(50)}}$$

$$40.9 < 60.3$$

Therefore:

$$C = 1.0 \qquad \text{Eq. 6.10.9.3.2-4}$$

The nominal shear capacity of the web can now be determined.

$$V_n = V_{cr} = CV_p$$ Eq. 6.10.9.2-1

$$V_n = (1.0)(817.9) = 817.9 \text{ kip}$$

From the shears generated for this girder, a maximum Strength I shear of 273.2 kip was found (see 4.5), indicating that this girder meets strength limit state requirements for shear.

$$V_u \leq \phi_v V_n$$ Eq. 6.10.9.1-1

$$273.2 \text{ kip} < (1.0)(817.9 \text{ kip}) \therefore OK(\text{Ratio} = 0.334)$$

4.6.4.3 Ductility

An additional ductility requirement is placed on composite sections in positive flexure. Specifically, sections shall meet the requirements in the relation below. For this requirement, as shown, the section performs satisfactorily.

$$D_p \leq 0.42\ D_t$$ Eq. 6.10.7.3-1

$$8.913 \leq (0.42)(45.09)$$

$$8.913 \text{ in} < 18.938 \text{ in} \therefore OK(\text{Ratio} = 0.471)$$

4.7 Performance Summary

A tabulated summary of all of the girder's performance ratios is presented below. As shown, the girder performs satisfactorily under all evaluated design checks, with bottom flange permanent deformations at the service limit state governing (Ratio = 0.987).

CONSTRUCTIBILITY	
Compression Flange Yielding	
Strength I	0.666
Strength IV	0.687
Compression Flange Flexural Resistance	
Strength I	0.434
Strength IV	0.488
Web Bend Buckling	
Strength I	0.421
Strength IV	0.505
Tension Flange Yielding	
Strength I	0.617
Strength IV	0.637
SERVICE LIMIT STATE	
Elastic Deformations	0.715
Permanent Deformations	
Top Flange	0.471
Bottom Flange	0.987
FATIGUE LIMIT STATE	
Base Metal at Connection Plate Weld	
Top Flange	0.108
Bottom Flange	0.960
STRENGTH LIMIT STATE	
Moment	0.894
Shear	0.334
Ductility	0.471

Chapter 5: Summary and Concluding Remarks

5.1 Summary of Evaluation Procedures

The focus of this report was to summarize the SMDI Short Span Design Standards performance. The performance ratios for this design evaluation are summarized below. For this summary, "PG" relates to the homogeneous plate girder evaluation in Chapter 3, and "RB" relates to the rolled beam evaluation in Chapter 4.

	PG	RB
CONSTRUCTIBILITY		
Compression Flange Yielding		
Strength I	0.676	0.666
Strength IV	0.720	0.687
Compression Flange Flexural Resistance		
Strength I	0.478	0.434
Strength IV	0.547	0.488
Web Bend Buckling		
Strength I	0.485	0.421
Strength IV	0.582	0.505
Tension Flange Yielding		
Strength I	0.489	0.617
Strength IV	0.527	0.637
SERVICE LIMIT STATE		
Elastic Deformations	0.558	0.715
Permanent Deformations		
Top Flange	0.519	0.471
Bottom Flange	0.918	0.987
FATIGUE LIMIT STATE		
Base Metal at Connection Plate Weld		
Top Flange	0.103	0.108
Bottom Flange	0.883	0.960
STRENGTH LIMIT STATE		
Moment	0.928	0.894
Shear	0.621	0.334
Ductility	0.495	0.471

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