Study of Two-Span Continuous Tubular Flange Girder Demonstration Bridge

Final Report to
Commonwealth of Pennsylvania
Department of Transportation

(PennDOT Open End Agreement E00511, Work Order No. 003)

by

Bong-Gyun Kim and Richard Sause

ATLSS Report No. 07-01

February 2008

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ACKNOWLEDGEMENTS

The work summarized in this report was conducted at the ATLSS Center at Lehigh University, funded by the Pennsylvania Department of Transportation (PENNDOT) with additional funding from the Pennsylvania Infrastructure Technology Alliance (PITA) through a grant from the Pennsylvania Department of Community and Economic Development. Personnel from HDR, Inc. and PENNDOT provided technical input to the work.

The authors are grateful for this funding and technical input. The authors are also grateful for assistance provided by Peter Bryan and Bob Alpago of the ATLSS Center. The findings, opinions, and conclusions expressed in this report are the authors' and do not necessarily reflect the views of those acknowledged here.

ABSTRACT

This report presents a study of a demonstration bridge designed with concrete-filled tubular flange girders (CFTFGs), conducted for the Pennsylvania Department of Transportation (PENNDOT). A CFTFG consists of a conventional web plate and bottom flange plate, with the top flange fabricated with a rectangular tube that is then filled with concrete. The main advantage of the CFTFG is an increased torsional stability that enables the number of diaphragms (or cross-frames) needed to brace the girders under construction loading conditions to be reduced. As a result, the time and cost of fabricating and erecting the bridge girder system can be reduced.

The CFTFGs of the demonstration bridge are designed to be constructed as simple spans for dead loads, and are then made continuous for superimposed dead loads and live loads by adding continuity at the pier. This construction sequence reduces the design moments and shears for the interior-pier section of the girder and for the field splice at the pier. The bridge is also designed to be constructed with precast deck panels to promote accelerated construction.

Design criteria for CFTFGs were developed in a format compatible with the 2000 PENNDOT Design Manual Part 4 (PENNDOT 2000) and the 2004 AASHTO LRFD Bridge Design Specifications (AASHTO 2004). A preliminary design of the CFTFGs for the two-span demonstration bridge was developed. In addition, preliminary designs of the field splice over the pier and of the precast concrete deck were developed. Finally, finite element analyses of the stability of the CFTFGs under critical construction loading conditions were conducted.

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CHAPTER 1 INTRODUCTION

1.1 OVERVIEW

The tubular flange girder system is one of several innovative steel bridge girder systems proposed by Wassef et al. (1997) and Sause and Fisher (1996) over the past several years. Research funded by the Federal Highway Administration (Wimer and Sause 2004, and Kim and Sause 2005) has taken tubular flange girders from concept to laboratory prototype (Figure 1.1). This research has established fundamental information on the behavior of these girders under simulated bridge loading conditions. The concrete-filled tubular flange girders (CFTFGs) shown in Figure 1.1 have several advantages compared to conventional I-girders (Kim and Sause 2005). Two main advantages are: (1) the concrete-filled tubular flange provides more strength, stiffness, and lateral torsional stability than a flat plate flange with the same amount of steel, and (2) the vertical dimension of the tube reduces the web depth, thereby reducing the web slenderness. In particular, the increased torsional stability of the girders will reduced the number of diaphragms (or cross-frames) needed to brace the girders, thus reducing the time and cost of fabricating and erecting the bridge girder system.

This report presents a design study of a tubular flange girder demonstration bridge, conducted for the Pennsylvania Department of Transportation (PENNDOT). The bridge girders are CFTFGs comprised of a conventional web plate and bottom flange plate, and a top flange fabricated from a rectangular tube that is then filled with concrete.

The CFTFGs are designed to be constructed as simple spans for dead loads, and are then made continuous for superimposed dead loads and live loads by adding continuity at the pier. This construction sequence reduces the loads carried by the continuous girders so that the design moments and shears for the interior-pier sections of the girders and for the field splices at the pier are reduced. Also, to promote accelerated construction, the bridge is designed to be constructed with precast deck panels.

1.2 COMPLETED TASKS

The study included the following completed tasks:

(1) Develop Design Criteria

Based on the results of previous research on CFTFGs, CFTFG design criteria were developed in a format (i.e., LRFD format) compatible with the 2000 PENNDOT Design Manual Part 4 (PENNDOT 2000) and the 2004 AASHTO LRFD Bridge Design Specifications (AASHTO 2004). This task considered the main loading conditions considered in bridge design (maximum load, overload, fatigue, etc.) and particularly emphasized construction conditions, where CFTFGs provide their greatest benefits.

(2) Preliminary Design of CFTFGs for Demonstration Bridge

A preliminary design of the CFTFGs for the demonstration bridge was developed. The bridge is a two-span bridge, designed to be constructed as simple spans for dead load, which are made continuous for superimposed dead loads and live loads by adding continuity at the pier. The preliminary design was developed for spans of 100 ft. Preliminary dimensions of the CFTFGs were developed. The process of selecting these dimensions illustrates the application of the design criteria. The resulting girder dimensions were used in the remaining tasks, and will provide a starting point for engineers responsible for the design of the demonstration bridge.

(3) Preliminary Design of Field Splice

The demonstration bridge is a two-span bridge, which requires a field splice that is located at the pier. A preliminary design of the splice was developed. This design provides a starting point for engineers responsible for the design of the demonstration bridge.

(4) Preliminary Design of Precast Concrete Deck

A preliminary design for a precast concrete deck for the demonstration bridge was developed.

(5) Finite Element Analyses

Based on CFTFG stability analyses conducted by previous research, finite element analyses of the stability of the demonstration bridge girders under critical construction loading conditions were conducted. These analyses validated the design criteria, and provide information for the engineers responsible for the design of the demonstration bridge.

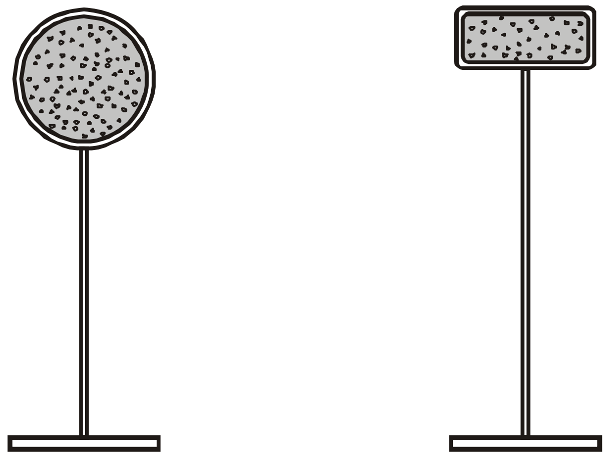


Figure 1.1 Tubular flange girders

CHAPTER 2 DESIGN CRITERIA FOR TUBULAR FLANGE BRIDGE GIRDERS

2.1 INTRODUCTION

Design criteria for concrete-filled tubular flange girders (CFTFGs) recommended herein were developed from the results of an analytical and experimental investigation conducted by Kim and Sause (2005). This investigation studied CFTFGs with steel yield strengths of 70 ksi and 100 ksi. The design criteria are considered applicable for CFTFGs with yield strength ranging from 50 ksi to 100 ksi.

2.2 GENERAL

Design criteria presented here apply to flexure of straight CFTFGs that are symmetrical about a vertical axis in the plane of the web. These criteria cover the following types of CFTFGs.

- CFTFGs that are composite with a concrete deck in positive flexure, where the concrete-filled tubular flange is the top (compression) flange.
- CFTFGs that are non-composite with a concrete deck in positive or negative flexure, where the concrete-filled tubular flange is the compression flange.

When the CFTFG is loaded in positive or negative flexure so that the concrete-filled tubular flange is the tension flange, then the concrete in the steel tube is neglected, and the CFTFGs can be designed based on the 2004 AASHTO LRFD Bridge Design Specifications (AASHTO 2004).

The design criteria presented here are compatible with the 2004 AASHTO LRFD specifications (AASHTO 2004). Criteria are given for the design of CFTFGs for the following requirements. Other requirements may need to be considered.

- The Strength I limit state requirements.
- The Constructibility requirements.
- The Service II limit state requirements.
- The Fatigue limit state requirements.

Strength I limit state requirements ensure that strength and stability, both local and global, are provided to resist the set of loading conditions that represents the maximum loading under normal use of the bridge. Constructibility requirements ensure that adequate strength is provided to resist the set of loading conditions that develop during critical stages of construction, but under which nominal yielding or reliance on post-buckling resistance is not permitted. Service II limit state requirements restrict yielding and permanent deformation of the steel structure under the set of loading conditions that represent normal service conditions. Fatigue limit state requirements restrict the stress range due to the passage of the fatigue design truck.

2.3 CFTFGS COMPOSITE WITH CONCRETE DECK

Sections consisting of a CFTFG section connected with sufficient shear connectors to a concrete deck to provide composite action and lateral support are considered composite sections.

2.3.1 Strength I Limit State

Flexural Strength

Composite sections are designed as compact sections by satisfying the following conditions:

- Compact section web slenderness limit:

$$2\frac{D_{cp}}{T_{web}} \le 3.76\sqrt{\frac{E}{F_{yc}}} \tag{2.1}$$

- Tube local buckling requirement:

$$\frac{B_{tube}}{T_{tube}} \le \quad 1.7\sqrt{\frac{E}{F_{yc}}} \tag{2.2}$$

where, D_{cp} is the depth of the web in compression at the composite compact section moment, M_{cc}^{sc},which is given below, T_{web} is the web thickness, E is the elastic modulus of the steel, F_{yc} is the yield stress of the compression flange (tube steel), B_{tube} is the tube width, and T_{tube} is the tube thickness. Equation (2.2) is adopted from ***Article 6.9.4*** of the 2004 AASHTO LRFD specifications (AASHTO 2004). It allows the tubular flange to yield before buckling locally in compression, and is conservative for a concrete-filled tube. Equations (2.1) and (2.2) replace Equation 6.10.6.2.2-1 from ***Article 6.10.6.2.2*** and Equations 6.10.2.2-1 and 6.10.2.2-3 from ***Article 6.10.2.2*** of the 2004 AASHTO LRFD specifications (AASHTO 2004).

The design criterion for flexure of composite CFTFGs for the Strength I limit state is as follows:

$$M_u \le \phi_f M_n \tag{2.3}$$

where, M_u is the largest value of the major-axis bending moment in the girder due to the factored loads as specified in ***Chapter 3*** of the 2004 AASHTO LRFD specifications (AASHTO 2004), ϕ_f is the resistance factor for flexure, taken as 1.0 in the 2004 AASHTO LRFD specifications (AASHTO 2004), and M_n is the nominal flexural strength. Equation (2.3) replaces Equation 6.10.7.1.1-1 from ***Article 6.10.7.1.1*** of the 2004 AASHTO LRFD specifications (AASHTO 2004).

The nominal flexural strength is taken as:

$$M_n = M_{cc}^{sc} \tag{2.4}$$

M_{cc}^{sc} is determined using an equivalent rectangular stress block for the concrete and an elastic perfectly plastic stress-strain curve for the steel. The maximum usable strain at the extreme concrete compression fiber, which is at the top of the deck, is taken as 0.003. Note that for the calculation of M_{cc}^{sc}, the concrete in the haunch is ignored. Figures 2.1 and 2.2 compare stress distributions based on the actual response, simple plastic theory, and strain compatibility for composite compact-section CFTFGs at the positive flexural strength limit, when the plastic neutral axis (PNA) is located in the deck and girder, respectively. β_1 shown in these figures is based on the compressive strength (f_c') of the concrete deck. If f_c' is less than or equal to 4 ksi, then β_1 is 0.85, and β_1 is reduced continuously by 0.05 for each 1 ksi of strength in excess of 4 ksi. These figures indicate that the strain compatibility approach reasonably approximates the actual stress distribution regardless of the PNA location and steel grade, and thus the method should accurately estimate the flexural strength. Equation (2.4) generally replaces the nominal flexural resistance calculations of ***Article 6.10.7.1.2*** of the 2004 AASHTO LRFD specifications (AASHTO 2004), although the limit on M_n given by Equation 6.10.7.1.2-3 from ***Article 6.10.7.1.2*** should be applied.

Shear Strength

The design criterion for shear of composite CFTFGs for the Strength I limit state is as follows:

$$V_u \leq \phi_v V_n \tag{2.5}$$

where, V_u is the shear in the web at the section under consideration due to the factored loads as specified in the 2004 AASHTO LRFD specifications (AASHTO 2004), ϕ_v is the resistance factor for shear, taken as 1.0 in the 2004 AASHTO LRFD specifications (AASHTO 2004), and V_n is the nominal shear strength determined as specified in ***Article 6.10.9.2*** of the 2004 AASHTO LRFD specifications (AASHTO 2004) without modification. Note that Equation (2.5) simply restates Equation 6.10.9.1-1 from ***Article 6.10.9.1*** of the 2004 AASHTO LRFD specifications (AASHTO 2004). All of the vertical shear force is assumed to be carried by the web.

2.3.2 Constructibility

The design criteria presented here pertain to conditions before the CFTFG is made composite with the concrete deck. These criteria apply only when the following conditions are satisfied:

- Web slenderness limit for “stocky web” under flexure:

$$\frac{2D_c}{T_{web}} \leq \lambda_b \sqrt{\frac{E}{F_{yc}}} \tag{2.6}$$

- Web slenderness limit to minimize web distortion:

$$\frac{D_{web}}{T_{web}} \leq 11\left(\frac{E}{F_{yct}}\right)^{\frac{1}{3}} \tag{2.7}$$

- The tube local buckling requirement given by Equation (2.2) is satisfied.
- Transverse stiffeners are provided at three (or more) equally-spaced locations along the span (i.e., quarter-span, mid-span, and three quarter-span) plus the bearing locations (more details are presented below).

In Equations (2.6) and (2.7), D_c is the depth of the web in compression at the yield moment (M_y) for the CFTFG when it is non-composite with the concrete deck, λ_b is a coefficient related to the boundary conditions provided to the web by the flanges, D_{web} is the web depth, and F_{yct} is the smaller of the yield stress for the compression flange and the yield stress for the tension flange. Equation (2.6) replaces Equation 6.10.3.2.1-3 from ***Article 6.10.3.2.1*** of the 2004 AASHTO LRFD specifications (AASHTO 2004).

If the area of the compression flange (the area of the steel tube plus the transformed area of the concrete infill) is less than that of tension flange, the value of λ_b is 4.64, otherwise, the value of λ_b is 5.76 as given in ***Article 6.10.4.3.2*** of the 1998 AASHTO LRFD specifications (AASHTO 1998).

The web slenderness requirement given by Equation (2.7) is based on finite element analysis results for CFTFGs with a stiffener arrangement having three intermediate stiffeners equally spaced along the span and stiffeners at each bearing. The details behind this equation are discussed in Kim and Sause (2005).

The arrangement of three intermediate transverse stiffeners along the span, suggested here, minimizes the effect of section distortion on the LTB strength without requiring too many stiffeners. The stiffeners should be placed in pairs, one on each side of the web, and the stiffeners should be spaced equally along the span. The following suggestions are made:

- The bearing and intermediate transverse stiffeners are made identical to simplify fabrication.
- The total width of each pair of stiffeners, including the web thickness, is 95% of the smaller of the tube width and the bottom flange width.
- The yield stress of the stiffeners is equal to yield stress of the steel elements of the girder cross-section.

The design criterion for flexure of composite CFTFGs for Constructibility is $M_u \leq \phi_f M_n$, which is identical in form to Equation (2.3). Again, M_u is the largest value of the major-axis bending moment in the girder due to factored loads specified in ***Chapter 3*** of the 2004 AASHTO LRFD specifications (AASHTO 2004). Here, Equation (2.3) is used in place of Equations 6.10.3.2.1-1 and 6.10.3.2.1-2 from ***Article 6.10.3.2.1*** and Equation 6.10.3.2.2-1 from ***Article 6.10.3.2.2***, and the calculation of M_n (given below) replaces the calculation of F_{yc}, F_{nc}, and F_{yt} for a noncomposite section from ***Article 6.10.3.2.1*** and ***Article 6.10.3.2.2***, which refer to ***Article 6.10.8*** of the 2004 AASHTO LRFD specifications (AASHTO 2004).

The nominal flexural strength, M_n, is taken as:

$$M_n = M_d^{br} \quad \leq \quad (M_s \quad \text{and} \quad M_d) \qquad (2.8)$$

where, M_d^{br} is the design flexural strength for torsionally braced CFTFGs, M_s is the cross-section flexural capacity which can be taken as the yield moment, M_y, when the steel tube yield stress is 70 ksi or less, and M_d is an ideal design flexural strength that corresponds to buckling between the brace points (assuming each diaphragm provides perfect lateral and torsional bracing at the brace point). Note that if the tube yield stress is large (e.g., 100 ksi) and the compressive strength of the concrete infill is small (e.g., 4 ksi), then the non-composite compact section moment capacity, M_{ncc}^{sc}, may be less than M_y. In this case, M_{ncc}^{sc} should be calculated and used for M_s (Kim and Sause 2005).

M_y for a CFTFG non-composite with the concrete deck is taken as the smaller of the yield moment based on analysis of a linear elastic transformed section, M_y^{tr}, and the yield moment based on strain compatibility, M_y^{sc}, which uses an equivalent stress block for concrete in the tube. M_y is also the smaller of the yield moment with respect to the compression flange, M_{yc}, and the yield moment with respect to the tension flange, M_{yt}. In calculating M_y^{tr}, the concrete in the steel tube is transformed to an equivalent area of steel using the modular ratio as shown in Figure 2.3 ($n = \frac{E}{E_c}$, where, E_c is the elastic modulus of concrete). M_y^{sc} is calculated based on an equivalent rectangular stress block for the concrete in the steel tube and a linear elastic stress-strain curve for the steel with the yield strain, ε_y, reached at either the top or bottom fiber. Note that for the calculation of M_y^{sc}, the strain in the concrete in the steel tube is not calculated, because the strain is limited to the yield strain of the tube. Figure 2.4 shows M_y^{sc} when either the top (compression) or the bottom (tension) flange yields first. A suggestion, that must be used with care, is that when the ratio of the yield stress of the tube steel, F_{ytube}, to the compressive strength of the concrete infill, f_c', is smaller than 8.5, M_y is taken as M_y^{tr}. Otherwise, M_y is taken as M_y^{sc}.

M_{ncc}^{sc} is the flexural strength based on strain compatibility, and is determined using an equivalent rectangular stress block for the concrete and an elastic perfectly plastic stress-strain curve for the steel as shown in Figure 2.5. The maximum usable strain is assumed to be 0.003 at the top of the concrete in the steel tube. The stress distributions based on the actual response, simple plastic theory, and strain compatibility for non-composite compact-section CFTFGs at the positive flexural limit state are shown in Figure 2.5.

The ideal design flexural strength is given by

$$M_d = C_b \alpha_s M_s \quad \leq \quad M_s \tag{2.9}$$

where, C_b is the moment gradient correction factor and α_s is the strength reduction factor. The moment gradient correction factor is given by either

$$C_b = \frac{12.5\, M_{max}}{2.5\, M_{max} + 3\, M_A + 4\, M_B + 3\, M_C} \tag{2.10a}$$

or

$$C_b = 1.75 - 1.05\left(\frac{M_1}{M_2}\right) + 0.3\left(\frac{M_1}{M_2}\right)^2 \leq 2.3 \tag{2.10b}$$

where in Equation (2.10a), M_{max} is the absolute value of the maximum moment in the unbraced segment and M_A, M_B, and M_C are the absolute values of the moment at the quarter, center, and three-quarter points in the unbraced segment, respectively. In Equation (2.10b), M_1 is the moment at the bracing point opposite to the one corresponding to M_2, and is taken as positive when it causes compression and negative when it causes tension in the flange under consideration. M_2 is the largest major-axis bending moment at either end of the unbraced length causing compression in the flange under consideration, and is taken as positive. Equation (2.10a) provides more accurate results for cases with non-linear moment diagrams, and has been used in calculations made for the preliminary design of the CFTFGs for the demonstration bridge discussed in Chapter 3. Equation (2.10a) was given in the commentary of past editions of the AASHTO LRFD specifications, but is not in the 2004 AASHTO LRFD specifications (AASHTO 2004), which provide specific guidance on definition of M_1 and M_2 for non-linear moment diagrams to make the results from Equation (2.10b) conservative.

The strength reduction factor is given by

$$\alpha_s = 0.8\left(\sqrt{\left(\frac{M_s}{M_{cr}}\right)^2 + 2.2} - \frac{M_s}{M_{cr}}\right) \leq 1.0 \tag{2.11}$$

where, M_{cr} is the elastic LTB moment, given by

$$M_{cr} = \frac{\pi E}{L_b/r_y}\sqrt{0.385 K_T A_{tr} + 2.467\frac{d^2 A_{tr}^{\;2}}{\left(L_b/r_y\right)^2}} \tag{2.12}$$

where, E is the elastic modulus of steel, L_b is the unbraced length, r_y is the radius of gyration, K_T is the St. Venant torsional inertia of the transformed section (using the short-term modular ratio), A_{tr} is the transformed section area (using the short-term modular ratio), and d is the section depth. The radius of gyration is given by

$$r_y = \sqrt{\frac{I_{tf} + I_{bf}}{A_{tr}}} \tag{2.13}$$

where, I_{tf} and I_{bf} are the moments of inertia of the top and bottom flanges about the vertical axis, respectively. Note that I_{tf} is based on a transformed section for the concrete-filled steel tube using the short-term modular ratio to account for the concrete in the tube.

For Equation (2.8), the design flexural strength for torsionally braced CFTFGs, M_d^{br}**,** is considered because research (Kim and Sause 2005) shows that the bracing provided to a CFTFG by a typical system of interior diaphragms may not be sufficiently stiff to brace the CFTFGs so that lateral buckling occurs only between the brace points. The approach taken here is given by Kim and Sause (2005) and is based on the approach described by Yura et al. (1992). M_d^{br} is given by

$$M_d^{br} = C_{bu} \alpha_s^{br} M_s \tag{2.14}$$

where, C_{bu} is the moment gradient correction factor corresponding to the girder when it is braced only at the ends of the span (without interior bracing within the span), obtained by applying Equation (2.10) to the entire girder span and α_s^{br} is a strength reduction factor for the torsionally braced girder. The strength reduction factor for the torsionally braced girder is given by

$$\alpha_s^{br} = 0.8\left(\sqrt{\left(\frac{M_s}{M_{cr}^{br}}\right)^2 + 2.2} - \frac{M_s}{M_{cr}^{br}}\right) \leq 1.0 \tag{2.15}$$

where, M_{cr}^{br} is the elastic LTB moment including the torsional brace stiffness, which is based on the approach described by Yura et al. (1992), and given by

$$M_{cr}^{br} = \sqrt{M_{cr}^{ubr\,2} + \frac{C_{bb}^{\ 2}}{C_{bu}^{\ 2}} M_{br}^{\ 2}} \tag{2.16}$$

where, M_{cr}^{ubr} is the elastic LTB moment for the girder without interior bracing within the span, C_{bb} is the moment gradient correction factor corresponding to the unbraced segment under investigation, assuming the adjacent brace points provide perfect bracing, obtained by applying Equation (2.10) to the unbraced segment, and M_{br} is the moment including the torsional bracing effect, given later. M_{cr}^{ubr} is given by

$$M_{cr}^{ubr} = \frac{\pi E}{L/r_y}\sqrt{0.385 K_T A_{tr} + 2.467 \frac{d^2 A_{tr}^{\ 2}}{\left(L/r_y\right)^2}} \tag{2.17}$$

Note that Equation (2.17) is Equation (2.12) with L_b replaced by the span length L. The moment including the torsional bracing effect, M_{br}, which is derived by Yura et al. (1992), is given by

$$M_{br} = \sqrt{\frac{\beta_T\ E\ I_{eff}\ n}{1.2\ L}} \tag{2.18}$$

where, β_T is the effective brace stiffness, I_{eff} is the effective vertical axis moment of inertia of the girder to account for singly-symmetric sections, and n is the number of interior braces within the span. The effective brace stiffness is given by (Yura et al. 1992)

$$\frac{1}{\beta_T} = \frac{1}{\beta_b} + \frac{1}{\beta_{sec}} + \frac{1}{\beta_g} \tag{2.19}$$

where, β_b is the discrete brace stiffness, β_{sec} is the stiffness of the web and stiffeners, and β_g is the stiffness of the girder system. β_b, β_g, and β_{sec} have dimensions of force-length. For multi-girder systems connected with diaphragms, they can be calculated from the following equations (Yura et al. 1992).

$$\beta_b = \frac{6\,E\,I_b}{S} \tag{2.20}$$

$$\beta_g = \frac{24\,(n_g - 1)^2}{n_g} \frac{S^2\,E\,I_x}{L^3} \tag{2.21}$$

$$\beta_{sec} = 3.3\,\frac{E}{h}\left(\frac{(N + 1.5\,h)\,t_w{}^3}{12} + \frac{t_s\,b_s{}^3}{12}\right) \tag{2.22}$$

In Equations (2.20) to (2.22), S is the spacing of girders, I_b is the moment of inertia of the bracing member about the strong axis, I_x is the horizontal axis moment of inertia of the girder, n_g is the number of girders, h is the distance between flange centroids, N is the contact length of the torsional brace, t_w is the web thickness, t_s is the stiffener thickness, and b_s is the stiffener width. N can be taken as the thickness of the diaphragm connection plate. The effective vertical axis moment of inertia of the girder is given by

$$I_{eff} = I_{yc} + \frac{t}{c} I_{yt} \tag{2.23}$$

where, I_{yc} and I_{yt} are the vertical axis moment of inertia of the compression and tension flanges respectively, and c and t are the distances from the neutral axis to the centroid of the compression and tension flanges respectively.

2.3.3 Service II Limit State

The design criterion for composite CFTFGs for the Service II limit state is as follows:

$$f_f \leq 0.95\, R_h\, F_{yf} \qquad (2.24)$$

where, f_f is the flexural stress in the flanges caused by the factored loads as specified in ***Chapter 3*** of the 2004 AASHTO LRFD specifications (AASHTO 2004), R_h is the hybrid factor, and F_{yf} is the yield stress of the flange. Note that R_h accounts for the nonlinear variation of stresses caused by yielding of the lower strength steel in the web of a hybrid girder (a coefficient $\leq$ 1.0) as specified in ***Article 6.10.1.10.1*** of the 2004 AASHTO LRFD specifications (AASHTO 2004).

Equation (2.24) replaces Equations 6.10.4.2.2-1 and 6.10.4.2.2-2 from ***Article 6.10.4.2.2*** of the 2004 AASHTO LRFD specifications (AASHTO 2004). Equations (2.1), (2.6), and (2.7) are intended to prohibit the use of slender webs in CFTFGs. For CFTFGS that are composite with the concrete deck and under positive flexure, no further check on web slenderness is needed, and Equation 6.10.4.2.2-4 from ***Article 6.10.4.2.2*** of the 2004 AASHTO LRFD specifications (AASHTO 2004) is not considered. However, for CFTFGs that are composite with a concrete deck under *negative flexure* with the concrete-filled tubular flange as the compression (bottom) flange (a condition that is *not* covered by the design criteria presented in this chapter), Equation 6.10.4.2.2-4 from ***Article 6.10.4.2.2*** of the 2004 AASHTO LRFD specifications (AASHTO 2004) should be considered.

Two different approaches are used to include the concrete in the steel tube in the calculation of the flexural stress. The first approach uses a transformed section to include the concrete in the tube, and the second approach uses an equivalent rectangular stress block for the concrete.

When $M_y^{tr} \leq M_y^{sc}$, then the transformed section approach is used for the concrete in the steel tube, and the flexural stresses are calculated as the sum of the stresses due to following individual loading conditions (Figure 2.6):

- The factored DC moment acting on the non-composite section, where the long-term modular ratio is used to account for the concrete in the steel tube (which makes a significant contribution to the stiffness and strength of the non-composite section).
- The factored DW moment acting on the long-term composite section, including the concrete deck but neglecting the concrete in the steel tube (which makes a negligible contribution to the stiffness and strength of the composite section).
- The factored LL moment acting on the short-term composite section, including the concrete deck but neglecting the concrete in the steel tube.

When $M_y^{tr} > M_y^{sc}$, then the equivalent rectangular stress block approach is used for the concrete in the steel tube, and the flexural stresses are calculated as the sum of the stresses due to following individual loading conditions (Figure 2.7):

- The factored DC moment acting on the non-composite section, where the equivalent rectangular stress block is used to account for the concrete in the steel tube.
- The factored DW moment acting on the long-term composite section, including the

concrete deck but neglecting the concrete in the steel tube.

- The factored LL moment acting on the short-term composite section, including the concrete deck but neglecting the concrete in the steel tube.

The long-term composite section is a transformed section based on an increased modular ratio (i.e., the long-term modular ratio equal to 3n) to account for the creep of the concrete that will occur over time. The short-term composite section is a transformed section based on the usual modular ratio (i.e., the short-term modular ratio equal to n).

2.3.4 Fatigue Limit State

The design criterion for composite CFTFGs for the Fatigue limit state is as follows:

$$\gamma\left(\Delta f\right) \leq \left(\Delta F\right)_n \tag{2.25}$$

where, γ is the load factor and Δf is the stress range due to the fatigue load as specified in ***Chapter 3*** of the 2004 AASHTO LRFD specifications (AASHTO 2004). $\left(\Delta F\right)_n$ is the nominal fatigue resistance as specified in ***Article 6.6.1.2.5*** of the 2004 AASHTO LRFD specifications (AASHTO 2004). Equation (2.25) is a restatement of Equation 6.6.1.2.2-1 from ***Article 6.6.1.2.2*** of the 2004 AASHTO LRFD specifications (AASHTO 2004).

Δf is calculated using the transformed section approach. The concrete in the steel tube and concrete deck are transformed to an equivalent area of steel using the short-term composite section (Figure 2.8). The provisions of ***Article 6.10.5.3.1*** of the 2004 AASHTO LRFD specifications (AASHTO 2004) should be considered, but are unlikely to control for CFTFGs with stocky webs.

2.4 CFTFGS NON-COMPOSITE WITH CONCRETE DECK

Sections consisting of a CFTFG that is not connected to the concrete deck by shear connectors are considered non-composite sections.

2.4.1 Strength I Limit State

Flexural Strength

Non-composite sections are designed to be either compact sections or non-compact sections by satisfying the following conditions:

- Compact sections satisfy the compact section web slenderness limit given by Equation (2.1):
- Non-compact sections satisfy the non-compact section web slenderness limit given by:

$$2\frac{D_c}{T_{web}} < 5.7\sqrt{\frac{E}{F_{yc}}} \tag{2.26}$$

- Compact sections and non-compact sections satisfy the tube local buckling requirement given by Equation (2.2):

The design criterion for flexure of non-composite CFTFGs for the Strength I limit state is expressed in the same form as Equation (2.3). In Equation (2.3), M_u is, again, the largest value of the major-axis bending moment within an unbraced length due to the factored loads as specified in ***Chapter 3*** of the 2004 AASHTO LRFD specifications (AASHTO 2004). The nominal flexural strength, M_n, is determined from Equation (2.8) with small modifications. If the girders are laterally braced by the deck, it is assumed that the attachments to the deck provide perfectly lateral and torsional bracing. Therefore, for calculating M_d^{br} for Equation (2.8), the unbraced length (L_b) between attachments to the deck is used instead of the span length (L) in Equations (2.17) and (2.18). If the deck does not brace the girders, the span length (L) is used to calculate M_d^{br} for Equation (2.8). In both cases, the cross-section flexural capacity, M_s, is taken as

$$M_s = R_{pc} M_{yc} \tag{2.27}$$

where, R_{pc} is the web plastification factor for the compression flange as specified in ***Article A6.2*** of the 2004 AASHTO LRFD specifications (AASHTO 2004), and M_{yc} is the yield moment with respect to the compression flange, described earlier in Section 2.3.2.

Shear Strength

The design recommendations for shear of non-composite CFTFGs for the Strength I limit state are the same as those for composite CFTFGs given in Section 2.3.1.

2.4.2 Constructibility

Design recommendations for non-composite CFTFGs for Constructibility are the same as those for composite CFTFGs given in Section 3.2.

2.4.3 Service II Limit State

The design criterion for non-composite CFTFGs for the Service II limit state is as follows:

$$f_f \leq 0.80\, R_h\, F_{yf} \tag{2.28}$$

where, f_f is the flexural stress in the flanges caused by the factored loads as specified in ***Chapter 3*** of the 2004 AASHTO LRFD specifications (AASHTO 2004), R_h is the hybrid factor, and F_{yf} is the yield stress of the flange. Equation (2.28) replaces Equations 6.10.4.2.2-3 from ***Article 6.10.4.2.2*** of the 2004 AASHTO LRFD specifications (AASHTO 2004). Equation 6.10.4.2.2-4 from ***Article 6.10.4.2.2*** of the 2004 AASHTO LRFD specifications (AASHTO 2004) should be considered.

Similar to composite CFTFGs, two different approaches (i.e., the transformed section approach and the equivalent rectangular stress block approach) are used to include the concrete in the steel tube in the calculating the flexural stress.

When $M_y^{tr} \le M_y^{sc}$, then the transformed section approach is used for the concrete in the steel tube, and the flexural stresses are calculated as the sum of the stresses due to following individual loading conditions (Figure 2.9):

- The factored DC moment and DW moment acting on the non-composite section, where the long-term composite section is used to account for the concrete in the steel tube.
- The factored LL moment acting on the non-composite section, where the short-term composite section is used to account for the concrete in the steel tube.

When $M_y^{tr} > M_y^{sc}$, then the equivalent rectangular stress block approach is used for the concrete in the steel tube, and the flexural stresses are calculated as the sum of the stresses due to following individual loading conditions (Figure 2.10):

- The factored DC, DW, and LL moments acting on the non-composite section, where the equivalent rectangular stress block is used to account for the concrete in the steel tube.

2.4.4 Fatigue Limit State

Design recommendations for non-composite CFTFGs for the Fatigue limit state are the same as those for composite CFTFGs given in Section 3.4, except for the calculation of Δf. The calculation of Δf is based on the short-term composite section, including only the steel girder and the concrete in the steel tube (Figure 2.11).

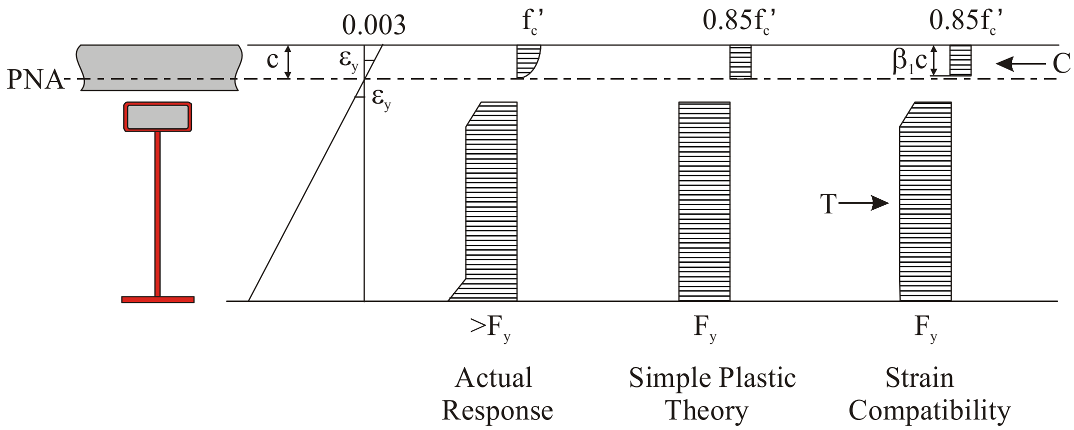


Figure 2.1 Comparison of stress distribution based on actual response, simple plastic theory, and strain compatibility for composite compact-section flexural strength when PNA is in deck

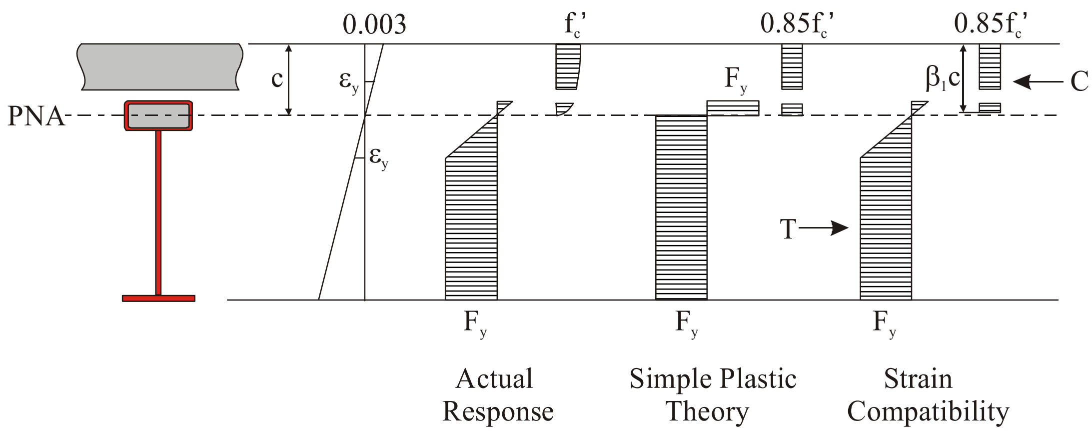


Figure 2.2 Comparison of stress distribution based on actual response, simple plastic theory, and strain compatibility for composite compact-section flexural strength when PNA is in girder

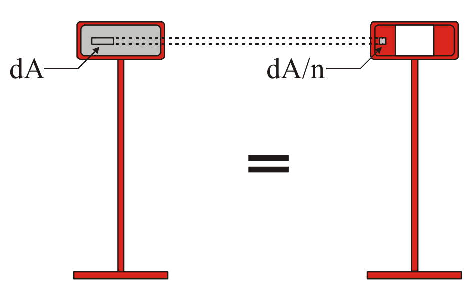
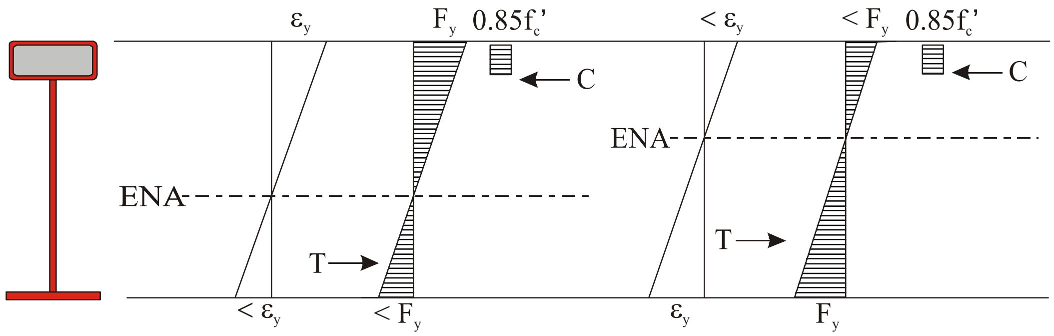


Figure 2.3 Transformed section for CFTFG



(a) When top (compression) flange yields first

(b) When bottom (tension) flange yields first

Figure 2.4 Yield moment based on strain compatibility

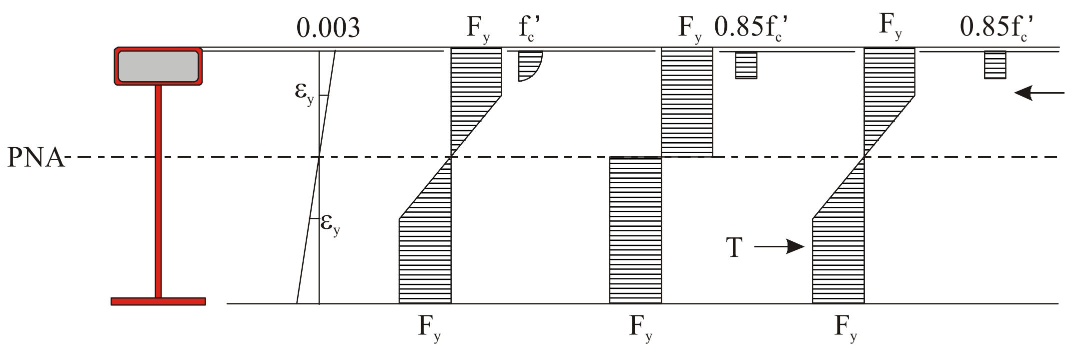
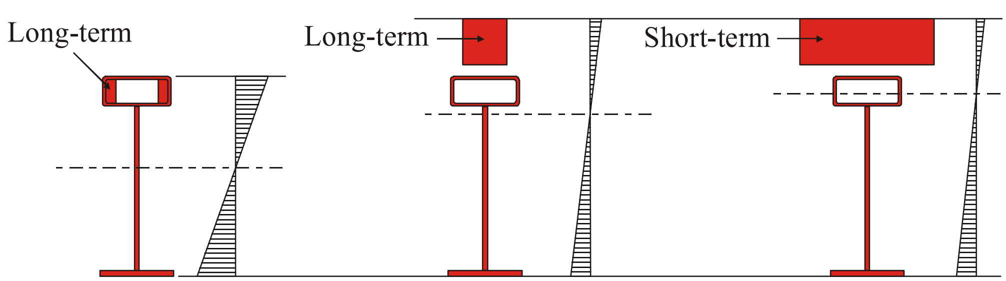


Figure 2.5 Comparison of stress distribution based on actual response, simple plastic theory, and strain compatibility for non-composite compact-section flexural strength



(a) Due to DC

(b) Due to DW

(c) Due to LL

Figure 2.6 Flexural stress for composite CFTFG under Service II loading conditions (transformed section approach)

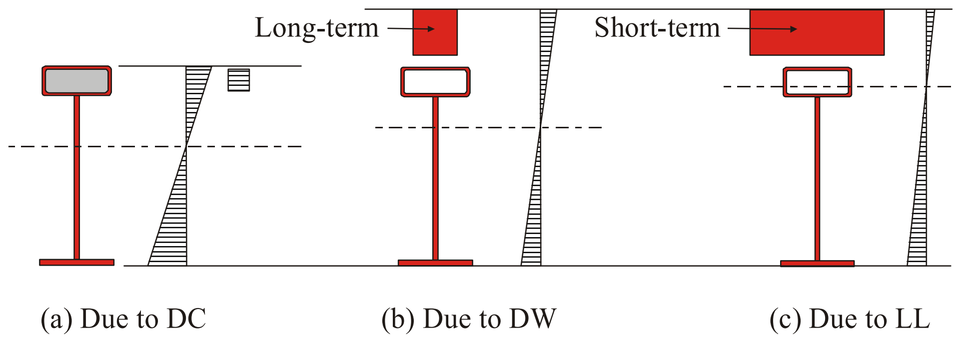


Figure 2.7 Flexural stress for composite CFTFG under Service II loading conditions (equivalent rectangular stress block approach)

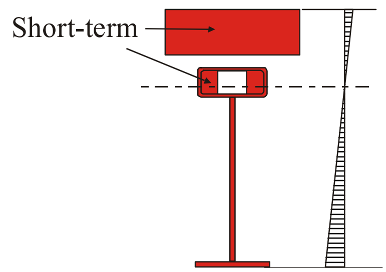


Figure 2.8 Flexural stress for composite CFTFG under Fatigue loading conditions

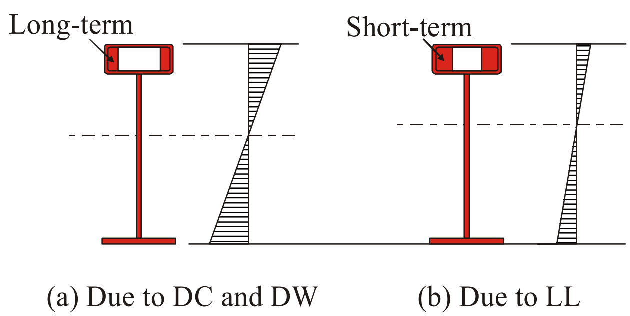
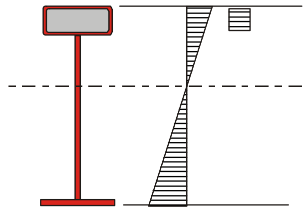


Figure 2.9 Flexural stress for non-composite CFTFG under Service II loading conditions (transformed section approach)



Due to DC, DW, and LL

Figure 2.10 Flexural stress for non-composite CFTFG under Service II loading conditions (equivalent rectangular stress block approach)

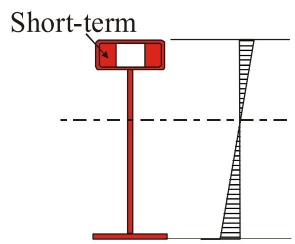


Figure 2.11 Flexural stress for non-composite CFTFG under Fatigue loading conditions

CHAPTER 3 PRELIMINARY DESIGN OF CFTFGS FOR DEMONSTRATION BRIDGE

3.1 INTRODUCTION

A design study of a two-span continuous composite CFTFG demonstration bridge with spans of 100 ft-100 ft is summarized here. This study developed a preliminary flexural design of the critical positive moment section and the interior-pier section of the CFTFGs for the demonstration bridge. The study also developed a preliminary design of the field splice at the pier. The interior-pier section design and the field splice design were actually completed after precast concrete deck design, presented in the next chapter, was completed, but the design results are included in this chapter.

3.2 BRIDGE CROSS-SECTION

The demonstration bridge cross-section was provided by PENNDOT, and consists of four girders spaced at 8 ft-5.5 in centers with 3 ft overhangs (Figure 3.1). The concrete deck is 8 in. thick. ASTM A 709 Grade 50 steel and concrete with compressive strength of 4 ksi were used. This design study considers the 2004 AASHTO LRFD Bridge Design Specifications (AASHTO 2004) and the PENNDOT Design Manual Part 4 (PENNDOT 2000) as well as the design criteria given in Chapter 2. The design study results are based on several assumptions: (1) end diaphragms, but no interior diaphragms within the spans under construction conditions (during erection and deck placement) and one interior diaphragm at mid-span under service conditions, (2) diaphragms are W21X57 steel sections, (3) bearing stiffeners and three equally-spaced intermediate transverse stiffeners (per span) with Category C′ fatigue details, (4) similar cross-sections for the positive moment section and the pier section, and (5) a field splice located at the pier section.

3.3 GIRDER DESIGN

Two construction sequence options, shown in Figures 3.2 and 3.3, were considered for the bridge. For Construction Option 1 (Figure 3.2), precast concrete deck panels are placed on top of the girders except for the pier section where the field splice is located. The field splice is then made and the final deck panel is placed. For Construction Option 2, the precast concrete deck panels are placed on top of the girders after the field splice is made. Consequently, Construction Option 1 has less dead load applied to the continuous span, which affects the design of the interior-pier section and the design of the field splice.

3.3.1 Design Loads

The girders were designed for various dead and live load conditions. Lateral loads such as wind loads and earthquake loads were not considered in this study, however they could be treated as they are in a conventional steel I-girder bridge.

The dead loads considered include the weight of all components of the structure, the wearing surface, and the attached appurtenances. The dead load is divided into two categories: (1) the weight of the bridge components and girders (D_c) and (2) the weight of the future wearing surfaces (D_w). D_c includes the weight of the girders, the weight of the deck, the weight of the haunch, the weight of the secondary steel (diaphragms, etc), and the weight of the barriers. D_w includes the weight of the non-integral wearing surface. D_c is also divided into two categories according to the time of field splice. D_{c1} is D_c applied to the simple spans and D_{c2} is D_c applied to continuous spans. The dead loads were computed as a weight per linear foot of bridge girder. The numerical values of these loads are summarized in Table 3.1.

The live loads (LL) consist of either a design truck or design tandem acting coincident with a uniformly distributed design lane load. The 2004 AASHTO LRFD specifications (AASHTO 2004) specify the values and positions of these loads. The design lane load is a 0.64 k/ft force distributed across a 10 ft design lane and over the bridge such to cause the greatest load effect. In general, the live load analysis treats one design truck or one design tandem on the bridge at a time, and this load is placed on the bridge to cause the greatest load effect. Multiple presence factors account for loading in more than one lane. Note that for the negative moment section at pier, as specified in the 2004 AASHTO LRFD specifications (AASHTO 2004), 90% of the effect of two design trucks spaced a minimum of 50 ft between the lead axle of one truck and the rear axle of the other truck was considered (along with 90% of the design lane load).

The design truck is an HS-20 truck, based on the 2004 AASHTO LRFD specifications (AASHTO 2004) and the 2000 PENNDOT Design Manual Part 4 (PENNDOT 2000). The HS-20 truck includes three axle loads, the first is 8 kips, and the second and the third are 32 kips. There is 14 ft between the first and second axle and 14 to 30 ft between the second and the third axle. The distance between the second and third axle is varied to cause the greatest load effect on each girder.

The tandem load is a military loading which consists of a pair of 31.25 kip axles spaced 4 ft apart (PENNDOT 2000). These loads are 125% of the AASHTO LRFD design tandem (AASHTO 2004).

The fatigue load is based on an HS-20 truck with the axle spacing fixed at 14 ft between the first and second axle and 30 ft between the second and the third axle. The fatigue load consists of one such truck placed where it causes the greatest load effect. The design lane load is not included in the fatigue load.

The live loads are increased by a dynamic load allowance to account for the dynamic response. For most load cases, the effects of the design truck or tandem are increased by 33% (AASHTO 2004). The dynamic load allowance is 15% for the fatigue load effects. The lane load is not increased by the dynamic load allowance.

The live loads are given as lane loads and are not directly applied to each girder. The loads are transmitted though the deck to the girders, and then to the supporting substructure. ***Article 4.6.2.2*** of the 2004 AASHTO LRFD specifications (AASHTO 2004) has live load distribution provisions to distribute the lane loads to the girders. Distribution factors are applied to the live loads to determine the load applied to a girder, and these distributed loads are used in calculating the girder moment and shear demands. The distribution factors are calculated by using formulas in the specifications or by the lever rule. The distribution factor formulas depend on the type of deck and the spacing between the girders. In the lever rule, the fraction of live load distributed to each girder is calculated by placing the loads on the bridge and summing moments about the adjacent girder line. In addition, ***Article 4.6.2.2.2d*** of the 2004 AASHTO LRFD specifications (AASHTO 2004) requires an additional distribution factor calculation which distributes loads to an exterior girder by an analysis that treats the bridge cross-section as a rigid cross-section that deflects and rotates as a rigid body under live loads (called the “rigid body rule” distribution factor).

For interior girders, the specification formulas for a steel girder bridge with concrete deck were used to calculate the distribution factors for shear and moment for the girders of the demonstration bridge. For exterior girders, the lever rule was used with one design lane loaded, the specification formulas were used with two or more design lanes loaded to calculate the distribution factors for shear and moment. For the exterior girders, the rigid body rule was also applied to both the one lane-loaded and the two or more lane-loaded cases to calculate distribution factors for moment, and these distribution factors controlled.

Tables 3.2 and 3.3 show the live load distribution factors for the non-fatigue limit states and the Fatigue limit state, respectively. The interior and exterior girders of the demonstration bridge were designed for same shear and moment, using the largest distribution factors from those given in Tables 3.2 and 3.3. These distribution factors were applied for both the positive and negative bending regions of the girders.

Figures 3.4, 3.5, 3.6, and 3.7 summarize the unfactored dead and live load girder moment envelopes and shear envelopes for Construction Option 1 and Construction Option 2. As shown in these figures, the girder dead and live load analyses generated results at 10 ft intervals along the girder length. The figures show that the envelopes for live load (LL) plus dynamic load allowance (IM) and for dead load due to the wearing surface (D_w) are the same for Construction Option 1 and Construction Option 2. The envelopes for dead load due to D_{c1} and D_{c2} vary for the different options. More D_{c1} is applied for Construction Option 1 than for Construction Option 2, but less D_{c2} is applied for Construction Option 1 than for Construction Option 2. As shown in Figures 3.4 and 3.6, Construction Option 1 has smaller negative moment at interior-pier section and field splice location than Construction Option 2. Therefore, the design study was conducted based on Construction Option 1.

With Construction Option 1 selected and the construction sequence more clear, the dead load (D_c) can be further refined as follows. Dead load is applied to girders that may be either simple-span or continuous and either non-composite with the deck or composite with the deck. Dead load D_{c1}, as defined earlier, is applied to simple-span non-composite girders, and includes the weight of the girders, the weight of the deck, and the weight of the secondary steel (diaphragms, etc). Dead load D_{c2}, as defined earlier, is applied to either non-composite or composite girders. Specifically, the weight of the haunch (defined as D_{c2a}) is applied to girders that are continuous, but non-composite with the deck, and the weight of the barriers (defined as D_{c2b}) is applied to girders that are continuous and composite with the deck. D_w is also applied to girders that are continuous and composite with the deck.

To simplify the preliminary design of the CFTFGs for the demonstration bridge these various dead loads were treated as follows:

- To design the positive moment section, D_{c1} and D_{c2a} are treated as D_c dead loads applied to non-composite girders. When D_{c1} is applied to the simple-span girders, the maximum positive moment is at midspan. When the remaining loads are applied to the continuous girders, the maximum positive moment is 40 ft from the abutment end of the girders. For simplicity, these maximum positive moments were treated as if they acted at the same cross section. More accurate design calculations would treat these two cross sections independently.
- To design the negative moment region and splice at the pier, D_{c1} which is applied to

the simple-span girders is omitted. D_{c2a} and D_{c2b} are treated as D_c dead loads applied to continuous girders that are composite with the deck, even though the haunch (D_{c2a}) is actually placed when the girders are non-composite. Since the haunch weight is small, this simplification should have little effect on the design results.

3.3.2 Limit States

Similar to the 2004 AASHTO LRFD specifications (AASHTO 2004), the proposed design criteria presented in Chapter 2 consider the following limit state categories: (1) strength limit states, (2) service limit states, and (3) fatigue and fracture limit states. Extreme event limit states are treated by the 2004 AASHTO LRFD specifications (AASHTO 2004), but were not considered in this preliminary design study. Each limit state has a corresponding load combination with different load factors. The load combinations considered in this study correspond to the Strength I, Service II, and Fatigue limit states. With consideration of the Strength I load combination load factors, a construction load combination ("Constructibility") was developed. To simplify the preliminary design process, the load factor on the D_c dead load acting during deck placement (D_{c1} and D_{c2a}) was increased from 1.25 to 1.50, and construction live load was neglected (which is equivalent to assuming that the factored construction live load was 25% of the D_{c1} dead load, approximately 0.32 kip/ft per girder). The load combinations and corresponding load factors considered in the study are shown in Table 3.4.

The effective width of the deck for conditions when the girders are composite with the deck was calculated for both the interior and exterior girders. The effective width was smaller for the exterior girders, and the exterior girder effective width was used for the calculations of flexural stresses and flexural resistance of the composite girders.

For the design of the positive moment section, each limit state was considered as follows:

- For the Strength I limit state, the flexural strength was calculated using Equation 2.4 based on the section shown in Figure 2.2, and the shear strength was determined as specified in ***Article 6.10.9*** of the 2004 AASHTO LRFD specifications (AASHTO 2004).
- For Constructability, the flexural strength was calculated using Equation 2.8.
- For the Service II limit state, the flexural stress in the flanges was calculated based on the section shown in Figure 2.6.
- For the Fatigue limit state, the stress range due to the fatigue load was calculated based on the section shown in Figure 2.8.

For the design of the negative moment section (pier section), each limit state was considered as follows:

- For the Strength I limit state, the flexural strength was determined as specified in ***Appendix A*** (***Article A6.3.3***) of the 2004 AASHTO LRFD specifications (AASHTO 2004). The unbraced length of the girder of the demonstration bridge, which is 50 ft, is in the inelastic range. However, the inelastic lateral-torsional buckling strength of the girder is larger than the section capacity due to the large St. Venant torsional constant (K_T) and large moment gradient correction factor (C_b). As a result, lateral-torsional buckling is not a controlling limit state. The flexural strength was calculated by considering the steel girder with the cut out in the steel tube (needed to make the pier splice), and the post-tensioned strands (neglecting the concrete deck and concrete in the steel tube) as shown in Figure 3.8 (a). As discussed in Chapter 4, 120 post-tensioned strands are used in the longitudinal direction of the bridge deck, and 30 of these strands were assigned to each girder for calculating the negative moment section flexural capacity. Note that the C_b factor for the unbraced length adjacent to the pier was calculated based on Figure 3.9, which is the factored moment envelope for the Strength I loading (based on Construction Option 1). For calculating K_T, the steel girder section with the complete top flange tube (neglecting the presence of the cut out) and neglecting the concrete in the steel tube was used.

- The post-tensioned strands have a significant contribution to the negative moment section flexural capacity used for the Strength I limit state check. Because of the post-tensioning, the strands have substantial tensile stress at the time when the deck decompresses, much larger than would be calculated from a simple section analysis of a combined cross section of steel girder and strands (without concrete) under the Strength I moment demand. Therefore, calculations are needed to account for the stresses in the post-tensioned strands and the steel girder when the deck decompresses. These stresses are then added to the additional stresses that develop on the combined cross section of steel girder and strands (without concrete) under the Strength I maximum load condition.
- For the Strength I limit state, the shear strength was determined as specified in ***Article 6.10.9*** of the 2004 AASHTO LRFD specifications (AASHTO 2004).
- During the application of the D_{c1} loads, the CFTFGs are not continuous, and therefore the flexural demand at the pier section is zero for D_{c1}.
- For the Service II limit state, the flexural stress in the flanges and concrete deck was

calculated for a transformed section including the steel girder with the cut out in the tube and the short-term modular ratio for the concrete deck (but neglecting the concrete in the steel tube) as shown in Figure 3.8 (b).

- For the Fatigue limit state, the stress range due to the fatigue load was calculated based on the section shown in Figure 3.8 (b). The Fatigue limit state was checked for the bearing stiffener/diaphragm connection plate (as a Category C' detail) and for shear studs attached to the tube (as a Category C fatigue detail) to make the girders composite with the deck. Other Fatigue limit state checks were made for the field splice at the pier, as discussed later.

3.3.3 Design Results

Figure 3.10 shows the girders (CFTFGS) for the demonstration bridge that resulted from the design calculations. The calculations are given in Appendix A, and the performance ratios (factored load effect over factored resistance) for selected critical limit states are listed in Table 3.5. The girder cross-section satisfied the maximum girder depth of 36 in imposed on the girders for the demonstration bridge. Note that as mentioned previously, transverse stiffeners are needed at three intermediate locations along the span (i.e., quarter-span, mid-span, and three quarter-span) and at the bearings.

3.4 FIELD SPLICE DESIGN

The bolted field splice was located at the pier to simplify the erection of the bridge. The alternative of putting the splice at the location of dead load contraflexure would either increase the number of field pieces and number of splices (from two to three and one to two, respectively) for each girder, or increase the length of the longer of the two field pieces, if the same number of pieces were used. Consequently, the girders are designed as simple spans for dead load and continuous for superimposed dead load and live loads.

3.4.1 Design Procedures

The bolted field splice design is based on AASHTO LRFD specifications (AASHTO 2004). Similar to the girder design, Strength I, Service II, and Fatigue limit states were considered. The field splice was designed to be a slip-critical connection for Service II loading, and a bearing-type connection, with threads excluded from the shear planes, for Strength I loading. The splice (Figure 3.11) uses 7/8 in. diameter A325 bolts in standard holes. The splice plates are A709 Grade 50 steel. The sections shown in Figures 3.8 (a) and (b) were considered to design the bearing-type connection for Strength I loading, and the slip-critical connection for Service II loading, respectively.

For the design of bottom flange splice, the design force demand for the bottom flange was calculated from the girder moment at the splice location. The number of bolts was determined based on the following: (1) to develop the Strength I design force in the flange with the bolts in bearing and (2) to develop the Service II design force in the flange with the bolts designed as slip-critical. A single splice plate was used for the bottom flange. Yielding and fracture of the splice plate and of the flange plate were checked based on the Strength I design force. Also, the Fatigue limit state was checked for the splice plate and the flange plate using stresses based on the section in Figure 3.8 (b), and treating the bolt hole as a Category B fatigue detail. Based on these design considerations, the dimensions of the splice plate were determined.

For the design of top flange (tube) splice, the approach was similar to that used for the bottom flange splice. However, instead of using single splice plate, double splice plates were used on both the top and bottom walls of the tube. The following load-induced fatigue conditions were checked: (1) the tube walls and the splice plates with bolt holes using stresses based on the section shown in Figure 3.8 (b), treating the bolt hole as a Category B fatigue detail; and (2) the tube wall at the end of the cut out shown in Detail A of Figure 3.11, considering the stress concentration from the cut out where the nominal stress in the tube wall (based on the section in Figure 3.8 (b)) is factored by 2 and treating the base metal in the tube as a Category A fatigue detail.

For the design of the web splice, the portion of the moment resisted by the web, and the horizontal force carried by the web, due to the difference in design forces carried by the top and bottom flanges, were considered. Double splice plates were used on the web.

3.4.2 Design Results

From the field splice design results, it was found that more bolts are required for the bearing-type connection under Strength I loading than for the slip-critical connection under Service II loading. Based on these findings, the field splice was designed as a bearing-type connection based on Strength I loading. Slip does not occur under Service II loading. The design calculations are given in Appendix B. Figure 3.11 shows the final results of the field splice design.

Table 3.1 Dead loads for demonstration bridge with four I-girders

Type	Component	Calculation	Load/Length
D_{c1}	Slab	$0.15\frac{k}{ft^3}*\frac{8in}{12\frac{in}{ft}}*8.5\,ft$	$0.85\frac{k}{ft}$
D_{c1}	Steel Girder (assume 60 in^2 steel area)	$0.49\frac{k}{ft^3}*\frac{60in^2}{144\frac{in^2}{ft^2}}$	$0.20\frac{k}{ft}$
D_{c1}	Concrete Infill (assume 126 in^2 concrete area)	$0.15\frac{k}{ft^3}*\frac{126in^2}{144\frac{in^2}{ft^2}}$	$0.13\frac{k}{ft}$
D_{c1}	Secondary Steel	0.10*steel girder wt.	$0.04\frac{k}{ft}$
D_{c2}	Concrete Haunch	$0.15\frac{k}{ft^3}*\frac{16in}{144\frac{in^2}{ft^2}}*3in$	$0.05\frac{k}{ft}$
D_{c2}	Miscellaneous (Parapet, railing, lights, etc.)	(assumed)	$0.275\frac{k}{ft}$
D_W	Future Wearing Surface	$0.03\frac{k}{ft^2}*\frac{28\,ft}{4\,girders}$	$0.21\frac{k}{ft}$

Table 3.2 Live load distribution factor for non-fatigue limit states

	Interior Girder		Exterior Girder	
	One Design Lane Loaded	Two Design Lanes Loaded	One Design Lane Loaded	Two Design Lanes Loaded
Bending Moment	0.598	0.706	0.685	0.714
Shear	0.698	0.847	0.677	0.619

Table 3.3 Live load distribution factor for Fatigue limit state

	Interior Girder		Exterior Girder	
	One Design Lane Loaded	Two Design Lanes Loaded	One Design Lane Loaded	Two Design Lanes Loaded
Bending Moment	0.498	-	0.571	-
Shear	0.582	-	0.564	-

Table 3.4 Load factors and load combinations

Limit state	DC	DW	LL+IM
Strength I	1.25	1.50	1.75
Constructability	1.50	-	-
Service II	1.00	1.00	1.30
Fatigue	-	-	0.75

Table 3.5 Performance ratios for positive moment section

Limit State	Performance Ratio (Load Effect/Resistance)	Controlling Design Check
Strength I	0.82	Flexure
Constructability	0.79	Lateral-Torsional Buckling
Service II	0.88	Flexure (Bottom Flange)
Fatigue	0.73	Transverse Stiffeners

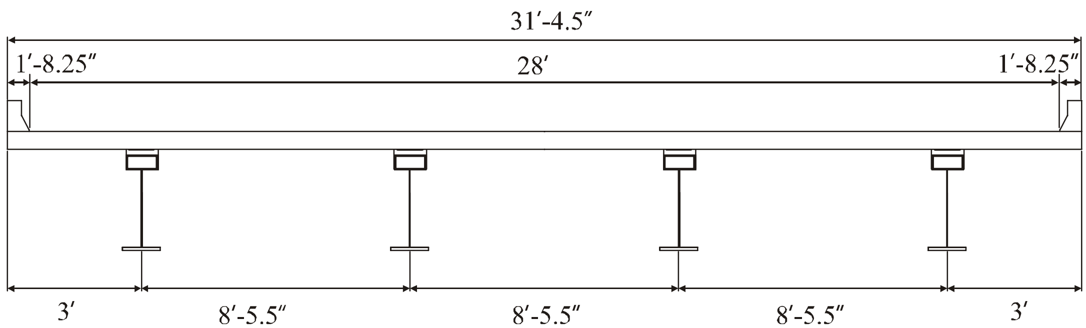


Figure 3.1 Demonstration bridge cross-section

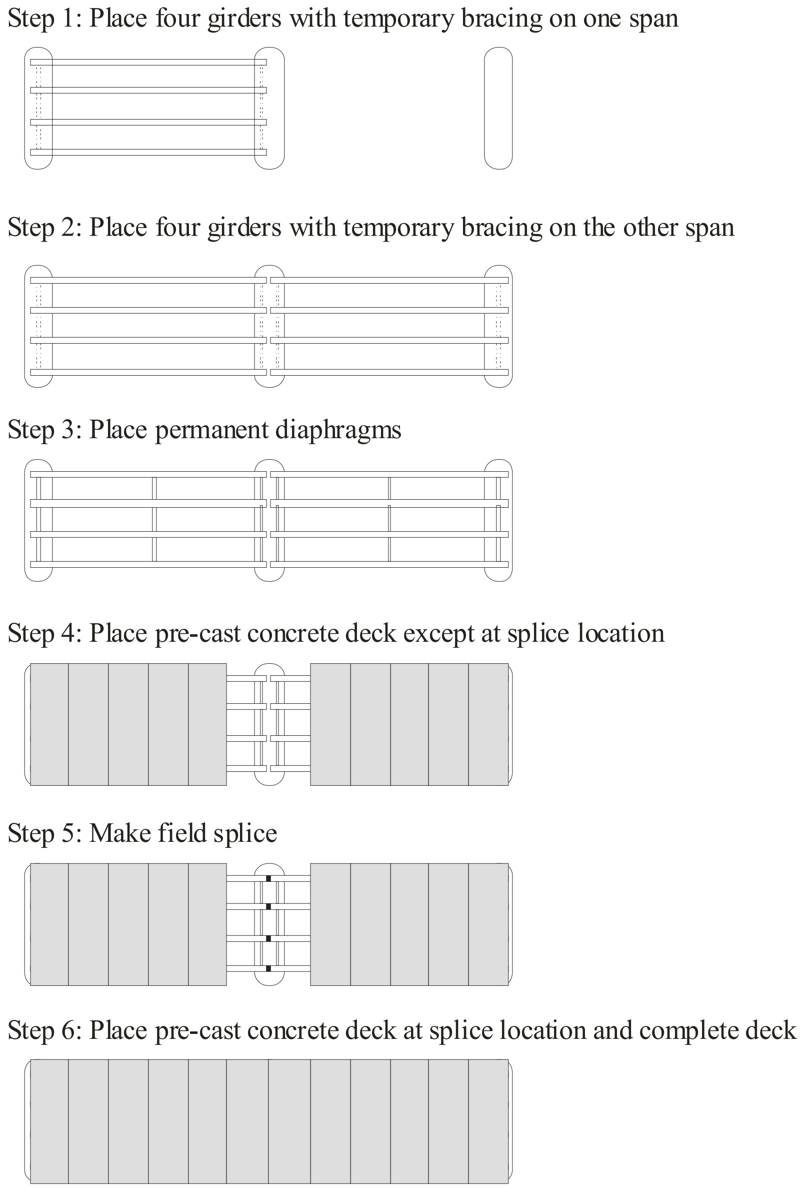


Figure 3.2 Option 1 for construction sequence of bridge (Construction Option 1)

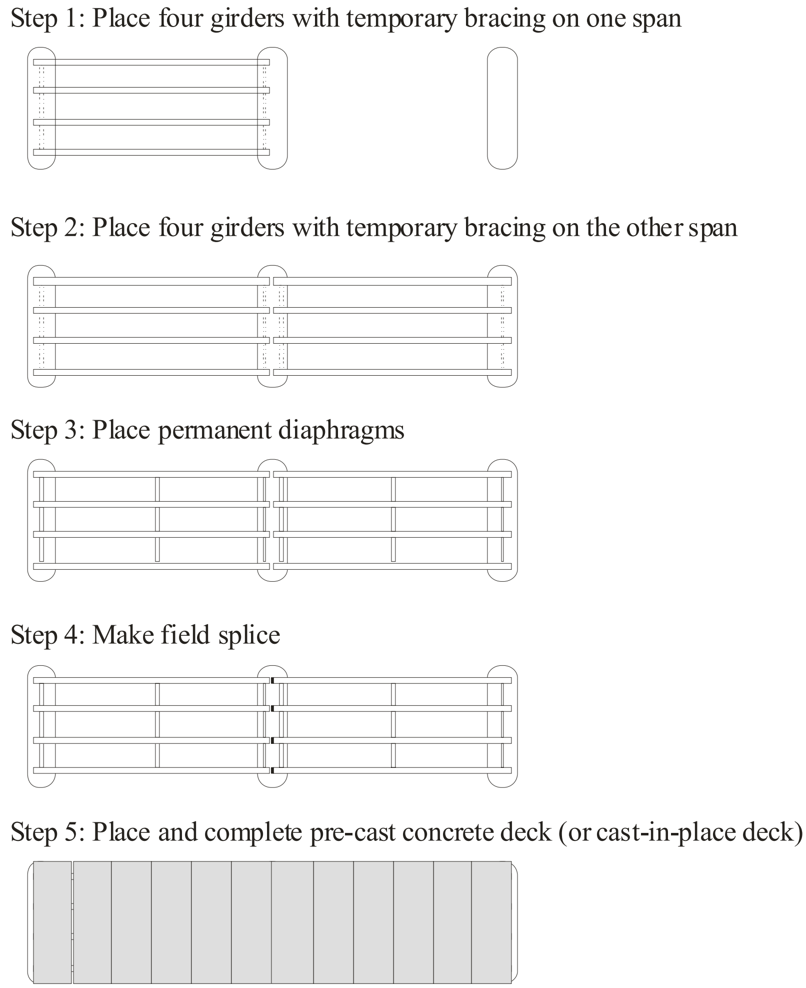


Figure 3.3 Option 2 for construction sequence of bridge (Construction Option 2)

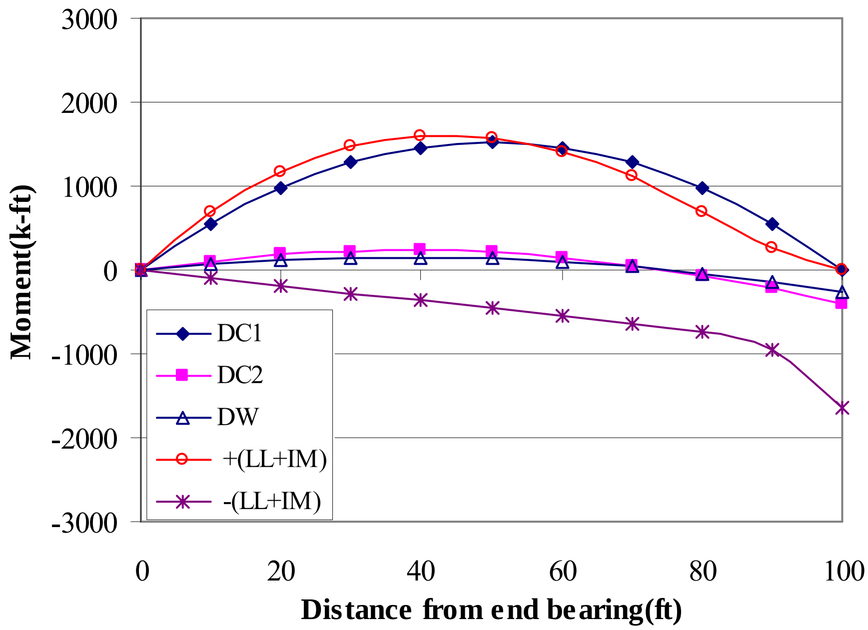


Figure 3.4 Unfactored dead and live load moment envelopes for Construction Option 1

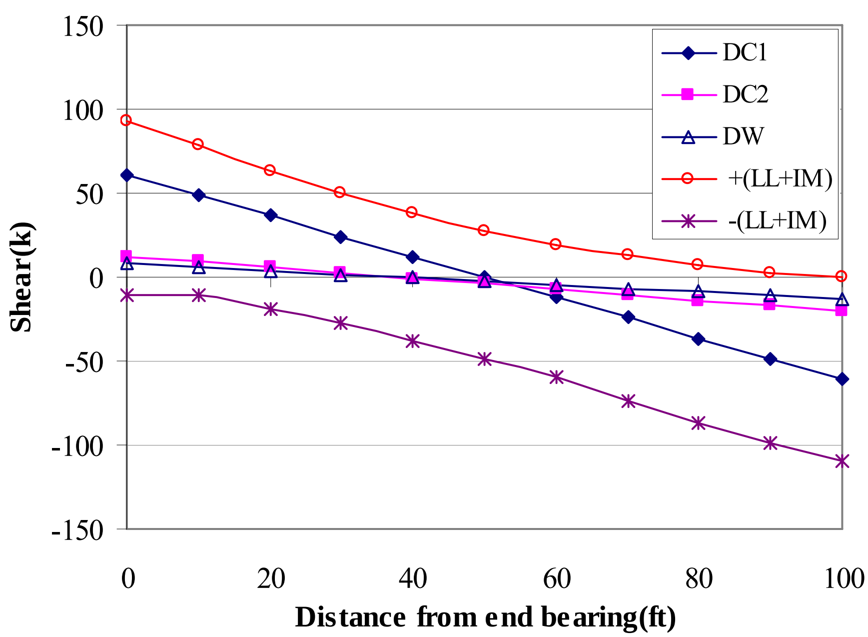


Figure 3.5 Unfactored dead and live load shear envelopes for Construction Option 1

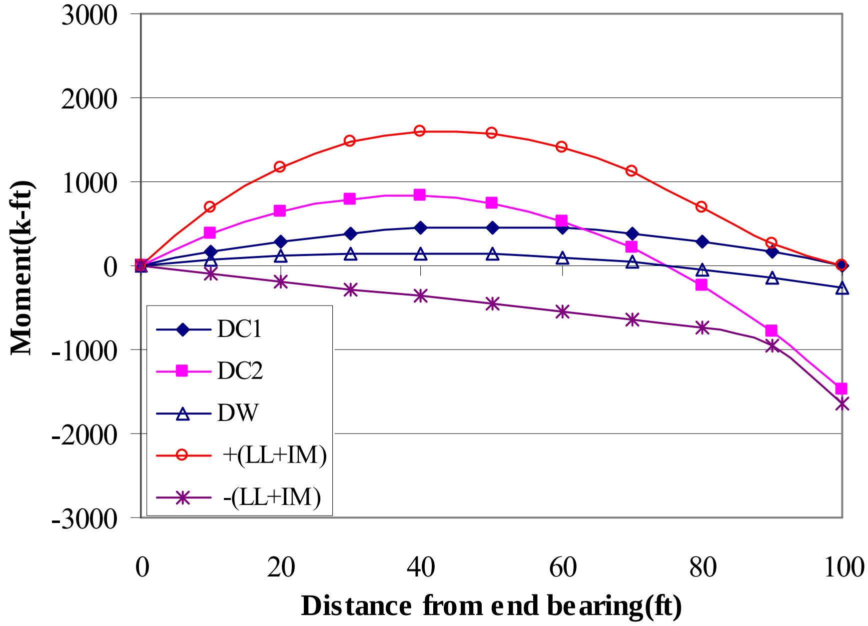


Figure 3.6 Unfactored dead and live load moment envelopes for Construction Option 2

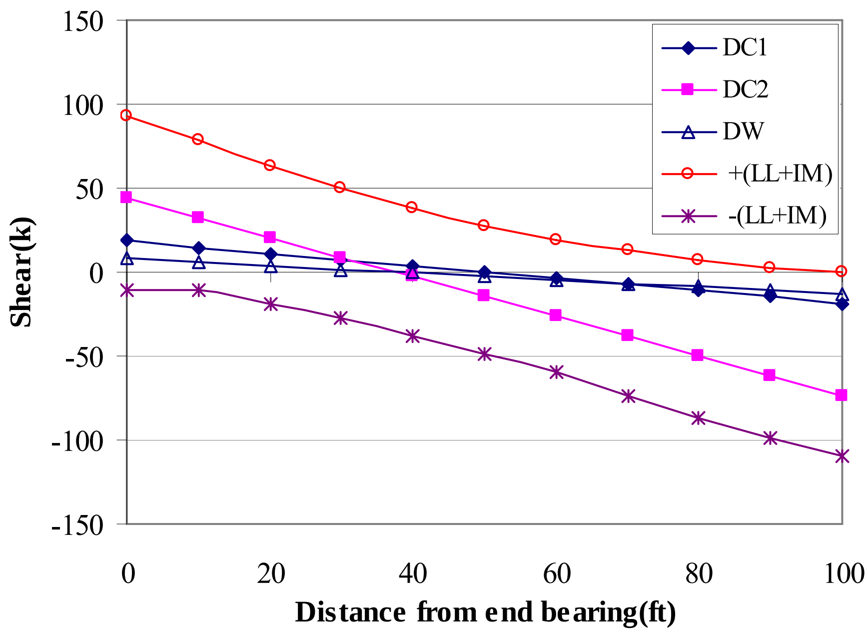


Figure 3.7 Unfactored dead and live load shear envelopes for Construction Option 2

(a) Section used for section ultimate flexural strength analysis of negative bending at pier splice

(b) Section used for transformed section analysis of negative bending stresses at pier splice

Figure 3.8 Sections considered for pier section design

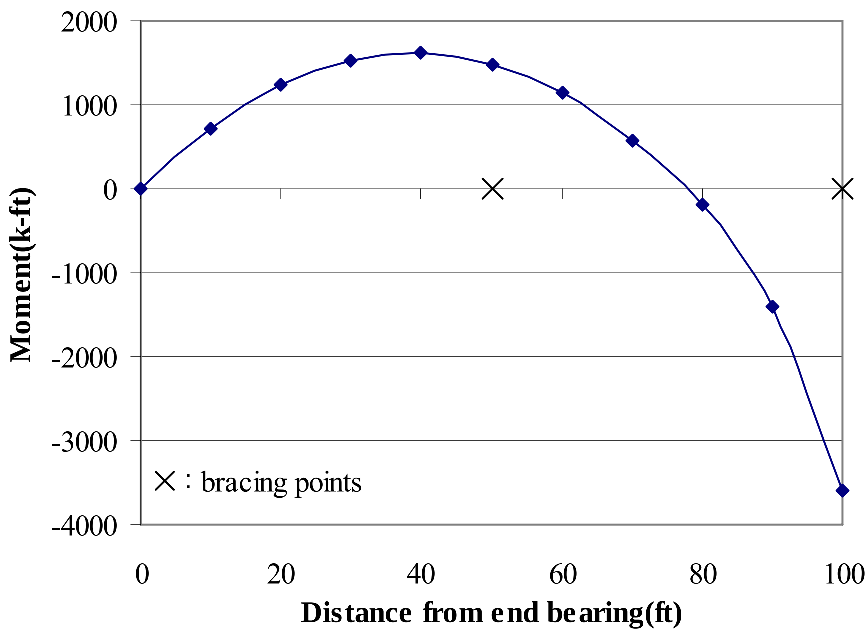
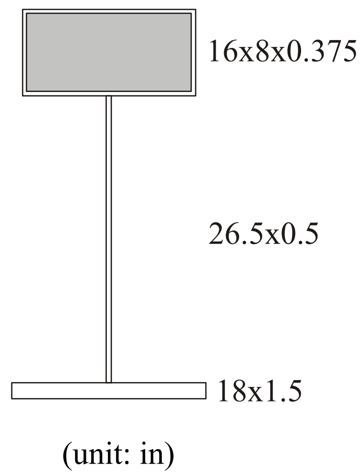
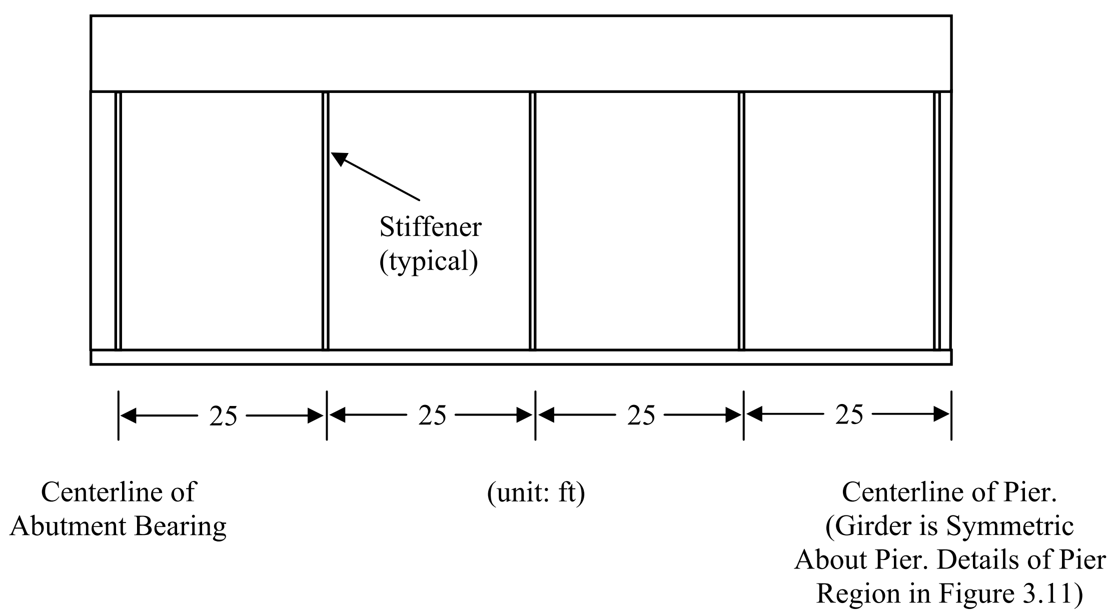


Figure 3.9 Factored moment envelopes due to Strength I loading for Construction Option 1



(a) Cross Section



(b) Elevation

Figure 3.10 Girders of demonstration bridge

Notes:

1. Use 7/8in diameter A325 bolt
2. No. of bolts in bottom flange is 48 on each side (No slip under service II loading)
3. No. of bolts in top flange is 16 on each side (No slip under service II loading)
4. No. of bolts in web is 24 on each side (No slip under service II loading)
5. Class B surface conditions
6. Unit: inch

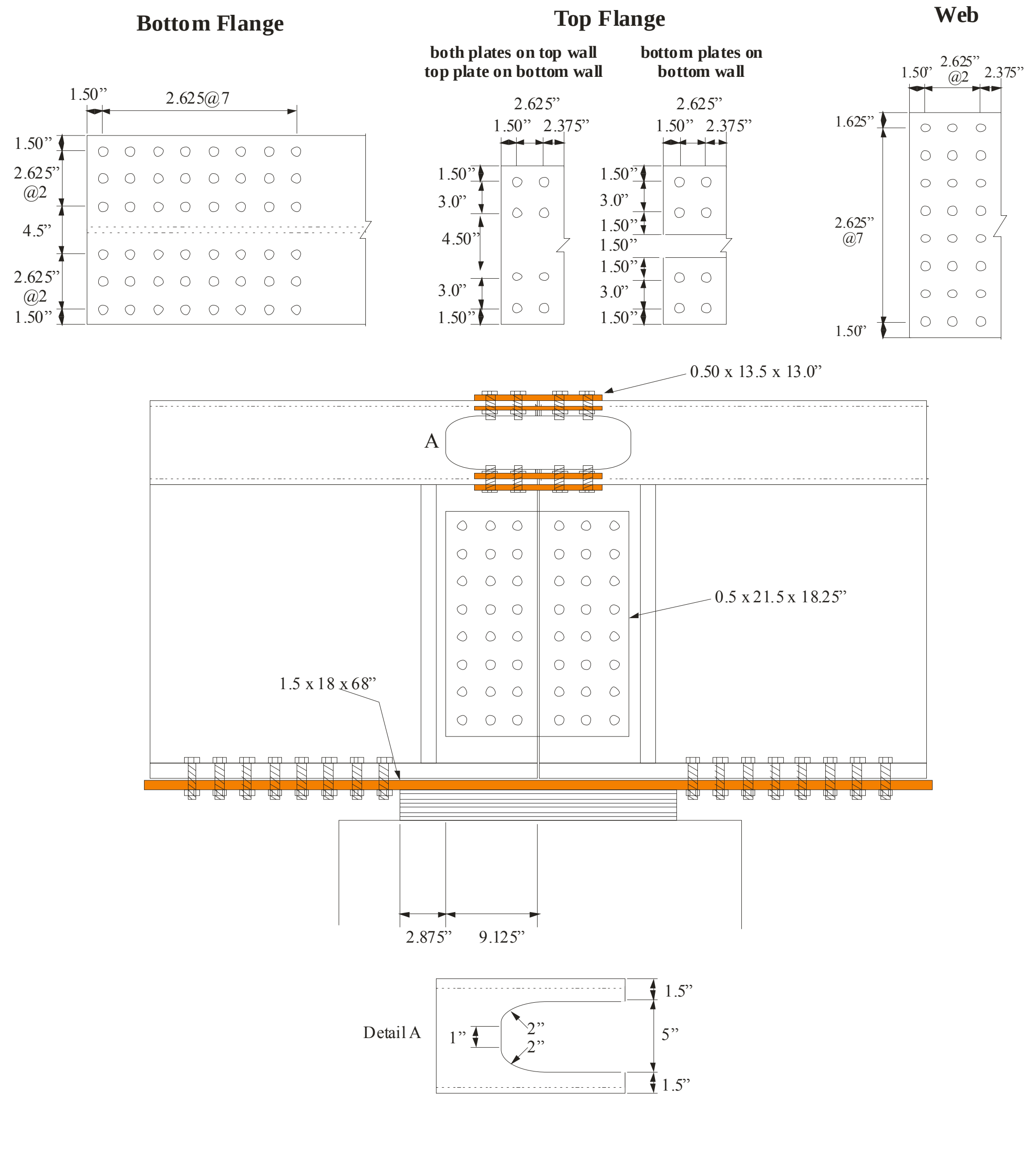


Figure 3.11 Field splice design

CHAPTER 4 PRELIMINARY DESIGN OF PRECAST CONCRETE DECK

4.1 INTRODUCTION

To promote accelerated construction, the demonstration bridge was designed to be built with precast deck panels. This design reduces the effort needed to cast a concrete deck in the field, including installation of forms and steel reinforcement. This deck design also eliminates the time (weeks) required for the concrete to cure. The preliminary design of the precast concrete deck uses concepts and procedures outlined by Shelala (2006) and some details in a test specimen deck described by Kim and Sause (2005).

4.2 DESIGN PROCEDURES

The precast concrete deck consists of twenty five precast panels, which are the full width of the bridge deck. The size of the panels was selected based on shipping and handling considerations. Each precast concrete panel is 8 in thick, 31.375 ft wide, and 8 ft long. A general view of the precast concrete deck system is shown in Figure 4.1. The panels were designed with a compressive strength of 4 ksi, but higher strength could be easily achieved in a precast concrete plant. Each panel is designed with mild steel reinforcing bars, pre-tensioned strands, and post-tensioned strands.

The mild steel reinforcing bars were designed as if the concrete deck system was a cast-in-place (CIP) deck, and the equivalent strip method was used (Shelala 2006). For the equivalent strip method analysis, the girders act as supports, and the deck acts as a continuous beam spanning from support to support. In the transverse direction, which is perpendicular to the girders, the interior spans and the overhangs were designed for live load moments determined using Table A4-1 in **Appendix A4** of **Chapter 4** of the AASHTO LRFD Bridge Design Specifications (AASHTO 2004). The table provides positive and negative live load moments, based on girder spacing and distance from the center of the girder to the design section for negative moment. The flexural strength of the deck was checked for the Strength I limit state load combination, and flexural cracking was checked with allowable stress in the tensile reinforcement for the Service II limit state load combination.

In the longitudinal direction of the girders, the mild steel reinforcing bars located in the lower layer were designed to satisfy the distribution requirement. This reinforcement is considered secondary reinforcement, and has an effect on distributing the wheel loads in the longitudinal direction of the girders to the primary reinforcement in the transverse direction. The mild steel reinforcing bars located in the upper layer were designed to satisfy temperature and shrinkage requirements.

Pre-tensioned strands were included in the transverse direction of the deck. These strands were designed to prevent cracking of the deck panels in the transverse direction (i.e., longitudinally oriented cracks) caused by the shipping and handling of the panels.

To create continuity between the panels (i.e., to prevent the transverse joints from opening and closing), the deck is post-tensioned in the longitudinal direction after all panels are in place on the girders. The post-tensioned strands were designed to keep the critical transverse joint, that is, the joint between the middle panel over the pier and the adjacent panel, closed. This critical joint is under maximum tension from the negative bending moment, and no mild steel reinforcing bars cross this joint (or the other joints). Opening of this joint was checked for the Service II limit state loading. This check compared the compressive stress in the deck due to the post-tensioned strands with the tensile stress in the top fiber of the concrete deck, calculated using the short-term composite section for the deck and girder, under the Service II loading.

The calculation of the tensile stress in the top fiber of the deck was performed using Service II moment at the pier centerline for the cross section with the cut out in the steel tube (needed to make the pier splice) and neglecting the concrete in the steel tube (see Figure 3.8(b)). The calculation of the compressive stress considered the time dependent losses in post-tensioning stresses. For the interior girders of the bridge, the compressive stress equaled the tensile stress, indicating that the joint would not open. For the exterior girders of the bridge, the tensile stress exceeded the compressive stress by about 10%, suggesting that the joint might open. The main difference between the interior and exterior girders is the effective slab width used to calculate the short-term composite section for the deck and girder. However, since the critical deck joint is actually located one half of the panel width from the pier centerline, where the Service II moment is smaller, further calculations may show that the joint stays closed. Alternatively, if needed, the deck properties could be slightly modified to ensure the critical joint stays closed under Service II load effects.

4.3 DESIGN RESULTS

Figure 4.2 shows schematic drawings of the final design of the precast concrete deck panels. All the panels had an identical configuration.

The mild steel reinforcing bars in the transverse direction are as follows:

- No. 4 at 5.5 in spacing in the upper layer and lower layer (i.e., 17 No. 4 equally spaced in the upper layer and lower layer per panel).

The mild steel reinforcing bars in the longitudinal direction are as follows:

- No. 4 at 8 in spacing in the upper layer and lower layer (i.e., 47 No. 4 equally spaced in the upper layer and lower layer per panel)

For the pre-tensioned strands in the transverse direction, 0.5 in diameter 7-wire low-relaxation prestressing strands with 270 ksi tensile strength were used. The initial prestress in the strands was assumed to be 70% of the tensile strength after anchorage seating and elastic shortening at transfer. The time dependent losses were assumed to be 15% of the initial prestress. The transverse pre-tensioned strands are as follows:

- 5 strands in the upper layer and 5 strands in the lower layer with equal spacing per panel

For the post-tensioned strands in the longitudinal direction, 0.6 in diameter 7-wire low-relaxation prestressing strands with 270 ksi tensile strength were used. The initial post-tensioning stress in the strands was assumed to be 70% of the tensile strength after friction losses, anchorage seating, and any initial elastic shortening during the post-tensioning operation. The time dependent losses were assumed to be 15% of the initial post-tensioning stress. The longitudinal post-tensioned strands are as follows:

- 120 strands at the center of the panel thickness with equal spacing using 5 strands per bundle with 24 bundles and ducts

4.4 ADDITIONAL CONSIDERATIONS

This section includes other issues, which were not considered in the preliminary design presented in this report, but should be considered for the final design.

The panels should be cast with voids or pockets at girder location. These pockets house the shear studs, providing the composite connection to the supporting girders. The pocket dimensions are dependent on the width of the top flange and the number of rows of shear studs placed in each pocket. Figure 4.3 shows the schematic drawings of the precast concrete deck panels with pockets. As shown in this figure, each panel is assumed to have 8 pockets (2 pockets per girder), and some of the mild steel reinforcing bars in the transverse direction and the longitudinal direction can be excluded because of the contribution of the pre-tensioned strands and post-tensioned strands, respectively. The mild steel reinforcing bars and pre-tensioned strands are relocated to provide the pocket space.

The panels should be leveled to eliminate eccentricity when the panels are post-tensioned longitudinally. One possible option is to use leveling bolts cast into the panels. A minimum of two leveling bolts per girder is suggested to be used to allow the dead load of the precast panels to be distributed to each support girder. Grout may be needed for the joints between panels, and the joints should be detailed for this grout.

In the preliminary precast deck design, the longitudinal post-tensioned strands run the full length of the deck to permit post-tensioning from the ends. Therefore, the concrete deck has a significant longitudinal prestress (approximately 1.4 ksi) in the positive moment region (before dead loads D_{c2b} and D_w and live loads are on the bridge). This concrete prestress was not considered in the positive moment region girder design. Further design calculations should be made to consider the effect of the deck prestress on the flexural strength of the positive moment cross section and to consider the effect of creep due to prestress on the positive moment region section behavior. If the longitudinal prestress needs to be reduced, some of the post-tensioned strands can be debonded in the positive moment region.

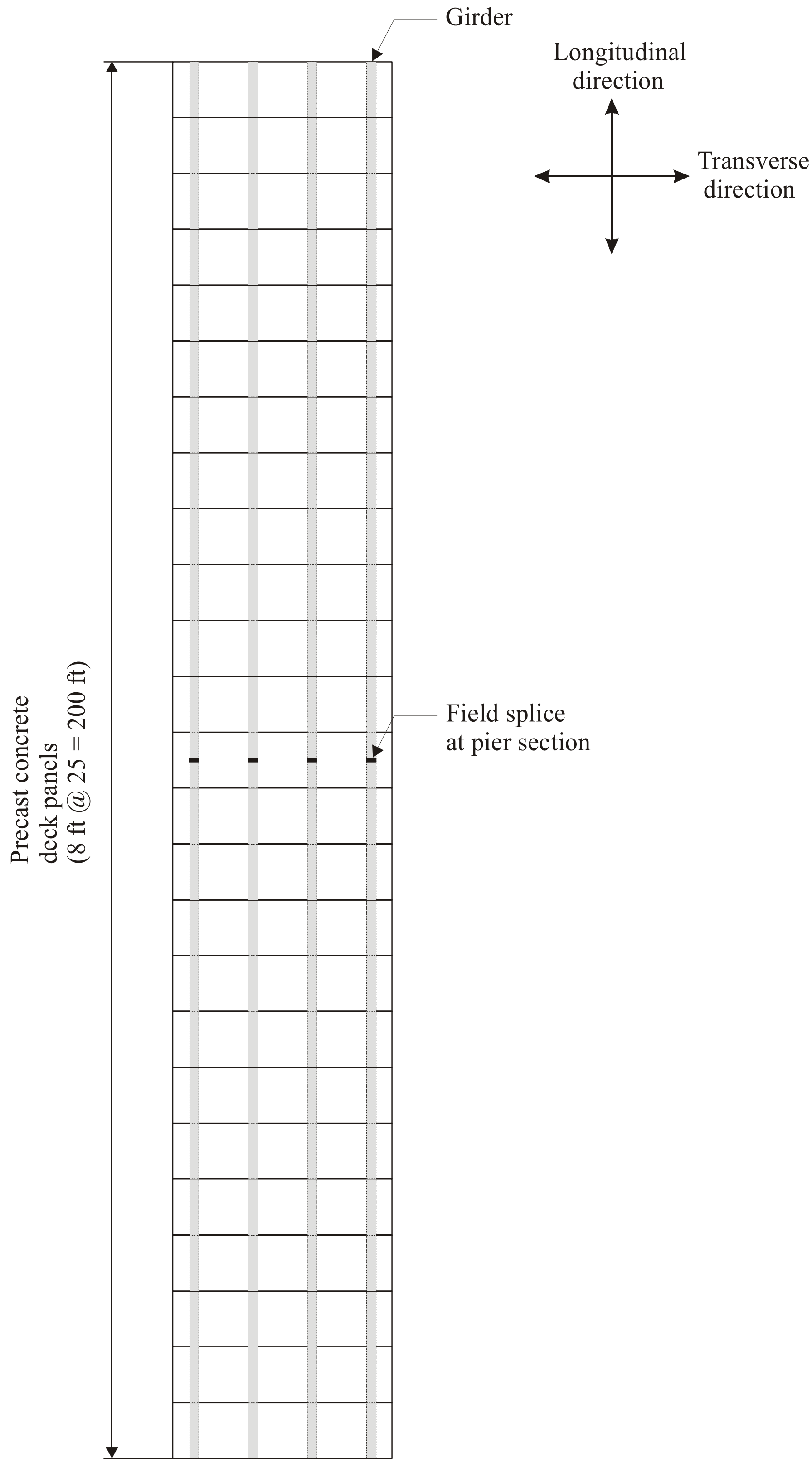
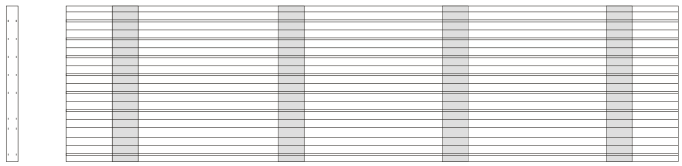


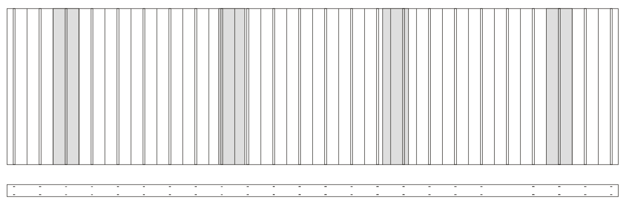
Figure 4.1 General overview of precast concrete deck system

Mild steel reinforcing bars in transverse direction



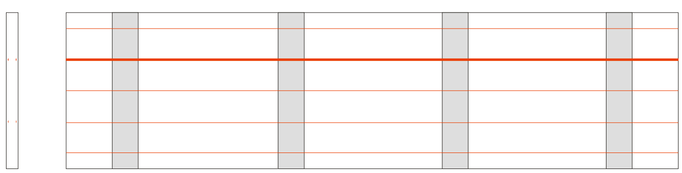
- 17 No. 4 at upper layer and lower layer per panel

Mild steel reinforcing bars in longitudinal direction



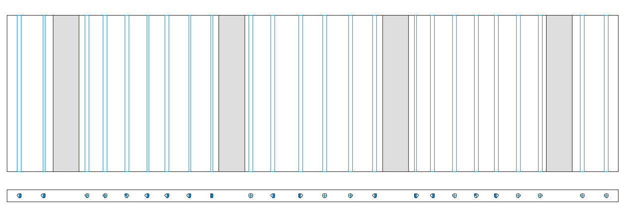
- 47 No. 4 at upper layer and lower layer per panel

Pre-tensioned strands in transverse direction



- 5 ½ in dia. 270 ksi low-relaxation strand at upper layer and lower layer per panel

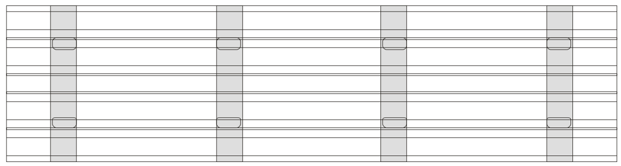
Post-tensioned strands in longitudinal direction



- 120 0.6 in dia. 270 ksi low-relaxation strand at the center of panel thickness
- Use 5 strands per one bundle

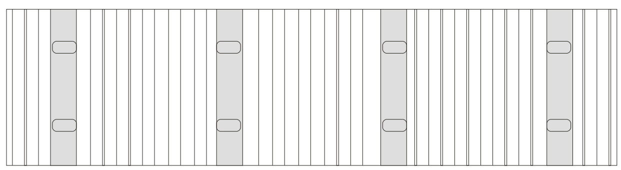
Figure 4.2 Schematic drawings of precast concrete deck panels

Mild steel reinforcing bars in transverse direction



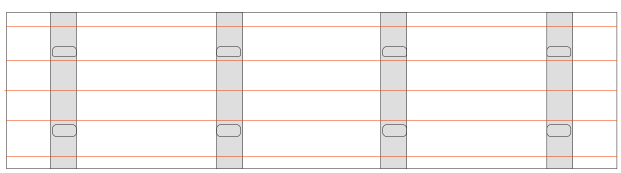
- 12 No. 4 at upper layer and lower layer per panel

Mild steel reinforcing bars in longitudinal direction



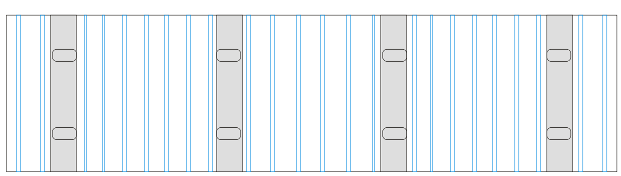
- 35 No. 4 at upper layer and lower layer per panel

Pre-tensioned strands in transverse direction



- 5 ½ in dia. 270 ksi low-relaxation strand at upper layer and lower layer per panel

Post-tensioned strands in longitudinal direction



- 120 0.6 in dia. 270 ksi low-relaxation strand at the center of panel thickness
- Use 5 strands per one bundle

Figure 4.3 Schematic drawings of precast concrete deck panels including pockets

CHAPTER 5 FINITE ELEMENT SIMULATION OF CFTFGS DURING DEMONSTRATION BRIDGE CONSTRUCTION

5.1 INTRODUCTION

An analytical study of finite element (FE) models of the concrete-filled tubular flange girders (CFTFGs) of the demonstration bridge was conducted. The study simulated construction loading conditions before the field splice is made and before the girders are connected with shear connectors to a concrete deck. Therefore, the FE models were simple span girders. The results of the study validate the design criteria for the lateral-torsion buckling strength of the arrangement of girders and diaphragms in the demonstration bridge.

5.2 FE ANALYSIS MODELS

The FE models were developed using ABAQUS (ABAQUS 2002). To understand the possible buckling modes, elastic buckling analyses of the FE models were conducted. To investigate the flexural strength, nonlinear load-displacement analyses of the FE models, including both material and geometric nonlinearity, were conducted.

As mentioned previously, simple span girders were analyzed. However, to investigate the influence of adjacent girders, single girder and multiple girder models (i.e., two girders, three girders, and four girders) were developed and analyzed. Two different diaphragm arrangements were also studied. Scheme 9 (S9) has three diaphragms, including two end diaphragms and one interior diaphragm. Scheme 10 (S10) has only the two end diaphragms. The diaphragms are W21X57 steel sections. The girder cross-section used in the model is shown in Figure 3.10.

A simply supported boundary condition was applied both in plane and out of plane at the locations of the bearing stiffeners at each end of each girder. This boundary condition simulates the combined effect of the end diaphragms, bearing stiffeners, and bearings to provide stiff lateral and torsional bracing at the bearings. A uniformly distributed load was applied on top of the tube along the entire length to simulate the weight of the girders, the weight of the diaphragms, and the weight of the deck.

The tube, web, and bottom flange were modeled using four node three-dimensional shell (S4R) elements. The steel material was modeled using a linear elastic isotropic model in the elastic range and the ABAQUS metal plasticity model in the inelastic range. The plasticity model uses the Von Mises yield criterion, associated plastic flow theory, and isotropic hardening. A simplified bilinear stress-strain curve with no strain hardening was used. Residual stresses in the steel were not included.

The concrete infill was modeled using eight node three-dimensional solid (C3D8R) elements. The concrete material was modeled using a linear elastic isotropic model in the elastic range and a multiaxial plasticity model in the inelastic range. For the multiaxial plasticity model, a linear Drucker-Prager model with a non-associated flow rule, and isotopic hardening and softening behavior was used. The linear Drucker-Prager model is defined by a stress-strain curve under uniaxial compression, and three parameters, namely, the ratio of the yield stress in triaxial tension to the yield stress in triaxial compression (K), the friction angle (β), and the dilation angle (ψ). An empirical stress-strain model for unconfined concrete developed by Oh (2002) was used as the stress-strain curve under uniaxial compression for the concrete infill. The value of K was assumed to be 1. Values of β and ψ were determined by calibrating the compressive strength of confined concrete to the empirical expression from Richart et al. (1928) and calibrating the ratio of transverse strain to axial strain at the peak stress to the value of 0.4 proposed by Oh (2002). The resulting values of β and ψ were 56.7 and 15.0 degrees, respectively.

The interface between the steel tube and concrete infill was modeled as a rigid connection based on the results of the previous study by Kim and Sause (2005).

Initial geometric imperfections were introduced into the model for nonlinear load-displacement analyses. The imperfection shapes were defined by scaling the buckling mode shapes obtained from an elastic buckling analysis (see Table 5.1). Several imperfection shapes were considered, and the shapes were scaled so the flange out-of-straightness (sweep) had a magnitude of either L/1000 or L/2000. If the buckling mode is a single half sine wave, only the magnitude of L/1000 was considered. However, if the buckling mode is a double half sine wave, two cases, with magnitudes of L/1000 and L/2000, respectively, were considered.

5.3 FE ANALYSIS RESULTS

Table 5.1 shows the elastic buckling analysis results for different FE models with different numbers of girders and the two bracing arrangements, S9, with the interior diaphragm, and S10, without the interior diaphragm. The buckling mode shape and the elastic buckling strength (given as the maximum moment reached at mid-span) are included in this table.

From the elastic buckling analysis results, the following observations were made:

- The buckling mode shape with a single half sine wave shape results in a smaller buckling strength than the buckling mode shape with a double or triple half sine wave shape.
- As the number of girders increases, the buckling strength increases due to contributions of the diaphragms.
- The girders with the S9 bracing arrangement have larger buckling strengths than the girders with the S10 bracing arrangement.
- For the multi girder systems, regardless of whether the girders buckle in a single or double half sine wave, a symmetric buckling mode shape along the longitudinal axis of the girder, in which the diaphragm is deformed in single curvature, results in a smaller buckling strength than asymmetric buckling mode shape along the longitudinal axis of the girder, in which the diaphragm is deformed in double curvature.
- The smallest buckling strength, which is 49720 k-in is larger than the yield moment of the girder (35401 k-in), so the elastic buckling results shown in the table do not control the strength, which is controlled by inelastic buckling. The inelastic buckling capacity is obtained from nonlinear load-displacement analysis.

Tables 5.2 through 5.5 show the nonlinear load-displacement analysis results. For these analyses, different numbers of girders, different bracing arrangements, and different imperfection modes and magnitudes were considered. The maximum moment (M_{max}) obtained from the analyses were compared with the yield moment (M_y = 35401 k-in) and the plastic moment (M_p= 44709 k-in) obtained from a cross-section analysis; the mid-span moment produced by the factored design loads for the construction loading condition ($1.5M_{DC}$ = 26982 k-in, where M_{DC} is the mid-span moment due to D_{c1} and D_{c2a}); and the design flexural strength (M_d^{br} which varies as the number of girders is varied) obtained from the design flexural strength formula (Equation 2.8). Note that M_p is calculated from simple plastic theory using an equivalent rectangular stress block for the concrete in the steel tube and the yield stress for the steel.

Figures 5.1 compares the design flexural strength for single and multiple girders (i.e., two, three, and four girders) and the mid-span moment produced by the factored design loads under the construction loading condition.

From the nonlinear load-displacement analysis results, the following observations were made:

- The maximum moments obtained from the FE analyses are at least 42% larger than the mid-span moment produced by the factored design loads.
- The maximum moments obtained from the FE analyses are at least 14% larger than the design flexural strength for construction loading conditions.

Note that the design flexural strength equation for construction loading (Equation 2.8) was not developed to represent the maximum moment during lateral-torsional buckling, but represents the minimum of: (1) the bending moment at the onset of lateral-torsional instability, (which is the moment when an incremental strain reversal occurs at any location on the cross-section due to lateral bending), and (2) the bending moment at first yielding on the cross-section (Kim and Sause 2005). The moment at the onset of instability or first yielding is always less than the maximum moment. This difference produces the 14% difference between the maximum moment from the finite element analyses and the design flexural strength from Equation (2.8)

From the above observation, it is concluded that the girders with either S10 or S9 bracing arrangement are safe under the construction loading conditions considered in the study, and the actual strength under construction conditions is conservatively estimated by the proposed design flexural strength equation.

Table 5.1 Elastic buckling analysis results

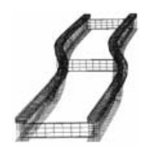
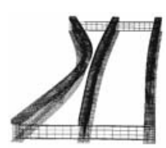
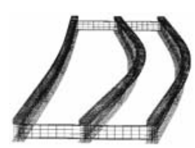
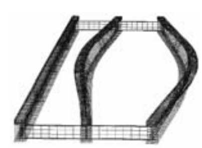
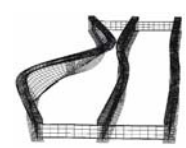
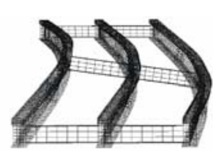
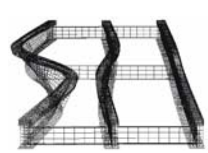
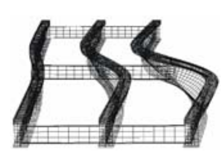
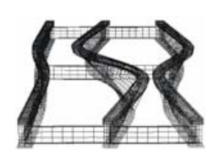
No of Girders	Bracing Type	Buckling Mode & Buckling Strength (k-in)			
1	-				-
		49720	98180	173600	-
2	S10				
		51890	53790	98680	99120
2	S9				-
		74450	99640	101000	-
3	S10				
		52720	56250	72780	98870
3	S9				
		93510	100180	102450	103267

Table 5.1 Elastic buckling analysis results (continued)

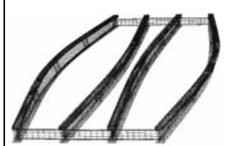
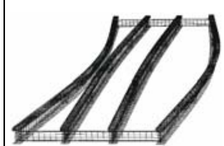
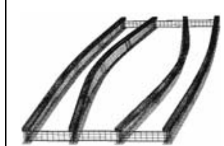
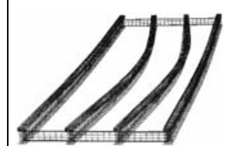
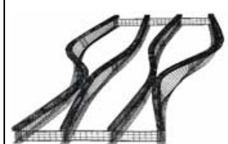
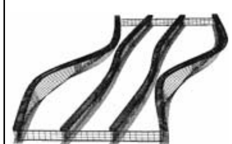
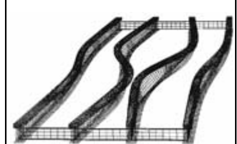
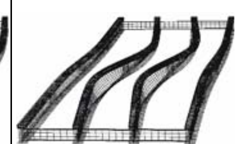
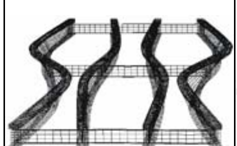
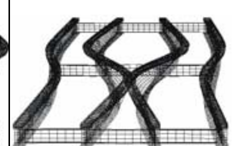
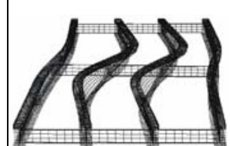
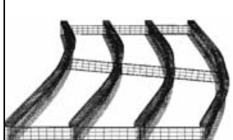
No of Girders	Bracing Type	Buckling Mode & Buckling Strength (k-in)			
4	S10				
		52460	52650	54880	56190
					
		98810	98860	99400	99730
4	S9				
		100100	100200	101900	103000
			-	-	-
		105800	-	-	-

Table 5.2 Nonlinear load-displacement analysis results for single girder

No. of Girders	Bracing Arrangement	Imperfection Mode	Imperfection Mag.	M_{max}/M_y	M_{max}/M_p	$M_{max}/1.5M_{DC}$	M_{max}/M_d^{br}
1	S10	1	L/1000	1.107	0.876	1.452	1.317
		2	L/1000	1.102	0.873	1.446	1.311
			L/2000	1.169	0.926	1.534	1.391

Table 5.3 Nonlinear load-displacement analysis results for two girders

No. of Girders	Bracing Arrangement	Imperfection		M_{max}/M_y	M_{max}/M_p	$M_{max}/1.5M_{DC}$	M_{max}/M_d^{br}
		Mode	Mag.				
2	S10	1	L/1000	1.084	0.859	1.423	1.290
		2	L/1000	1.090	0.863	1.429	1.296
		3	L/1000	1.105	0.875	1.450	1.315
			L/2000	1.159	0.918	1.521	1.379
		4	L/1000	1.105	0.875	1.450	1.315
			L/2000	1.158	0.917	1.520	1.378
	S9	1	L/1000	1.174	0.930	1.540	1.282
		2	L/1000	1.109	0.878	1.454	1.211
			L/2000	1.174	0.929	1.540	1.282
		3	L/1000	1.117	0.884	1.465	1.219
			L/2000	1.177	0.932	1.544	1.286

Table 5.4 Nonlinear load-displacement analysis results for three girders

No. of Girders	Bracing Arrangement	Imperfection		M_{max}/M_y	M_{max}/M_p	$M_{max}/1.5M_{DC}$	M_{max}/M_d^{br}
		Mode	Mag.				
3	S10	1	L/1000	1.080	0.855	1.417	1.285
		2	L/1000	1.124	0.890	1.475	1.337
		3	L/1000	1.170	0.927	1.536	1.392
		4	L/1000	1.104	0.874	1.449	1.314
			L/2000	1.159	0.918	1.521	1.379
		5	L/1000	1.106	0.876	1.451	1.316
			L/2000	1.169	0.925	1.533	1.390
		6	L/1000	1.106	0.876	1.451	1.315
			L/2000	1.168	0.925	1.532	1.389
	S9	1	L/1000	1.209	0.958	1.587	1.230
		2	L/1000	1.128	0.893	1.480	1.148
			L/2000	1.183	0.937	1.553	1.204
		3	L/1000	1.126	0.891	1.477	1.145
			L/2000	1.183	0.937	1.553	1.204
		4	L/1000	1.142	0.904	1.499	1.162
			L/2000	1.195	0.946	1.568	1.216

Table 5.5 Nonlinear load-displacement analysis results for four girders

No. of Girders	Bracing Arrangement	Imperfection Mode	Imperfection Mag.	M_{max}/M_y	M_{max}/M_p	$M_{max}/1.5M_{DC}$	M_{max}/M_d^{br}
4	S10	1	L/1000	1.093	0.866	1.434	1.300
		2	L/1000	1.086	0.860	1.425	1.292
		3	L/1000	1.101	0.872	1.444	1.309
		4	L/1000	1.112	0.881	1.459	1.323
		5	L/1000	1.104	0.874	1.449	1.314
			L/1000	1.145	0.906	1.502	1.362
		6	L/1000	1.104	0.874	1.449	1.313
			L/2000	1.165	0.922	1.528	1.386
		7	L/1000	1.105	0.875	1.450	1.314
			L/2000	1.165	0.923	1.529	1.386
		8	L/1000	1.106	0.876	1.451	1.316
			L/2000	1.168	0.925	1.532	1.389
	S9	1	L/1000	1.140	0.903	1.495	1.140
			L/2000	1.192	0.944	1.564	1.192
		2	L/1000	1.146	0.907	1.503	1.146
			L/2000	1.198	0.949	1.572	1.198
		3	L/1000	1.142	0.905	1.499	1.142
			L/2000	1.198	0.949	1.572	1.198
		4	L/1000	1.163	0.921	1.526	1.163
			L/2000	1.210	0.958	1.588	1.210
		5	L/1000	1.223	0.968	1.604	1.223

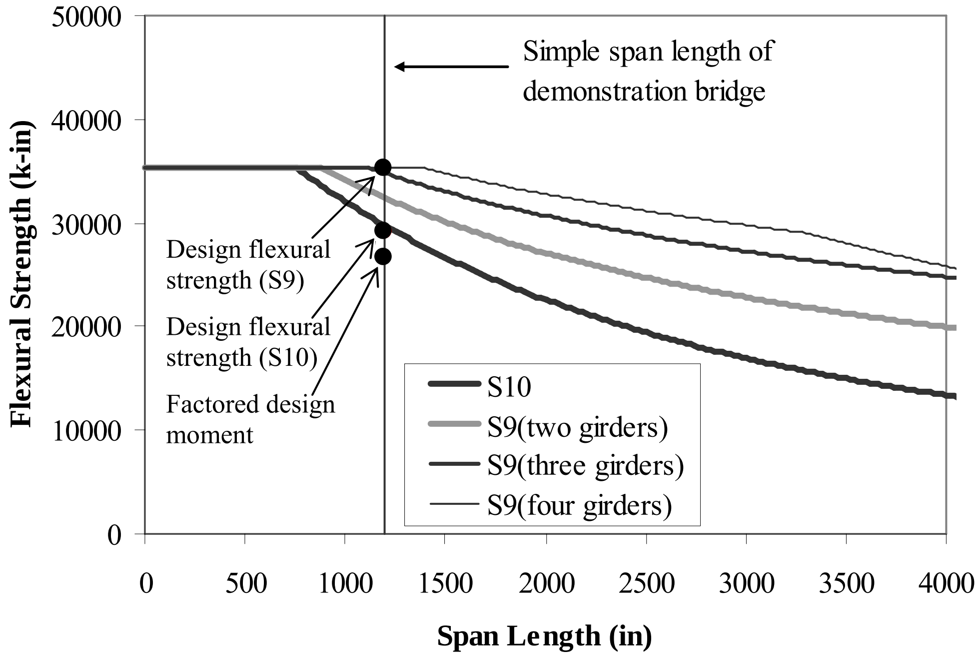


Figure 5.1 Design flexural strength for construction loading conditions

CHAPTER 6 SUMMARY AND CONCLUSIONS

6.1 SUMMARY

This report presents a design study of a demonstration bridge with concrete-filled tubular flange girders (CFTFGs), conducted for the Pennsylvania Department of Transportation (PENNDOT). The bridge girders consist of a conventional web plate and bottom flange plate, with the top flange fabricated from a rectangular tube that is then filled with concrete.

From previous research on CFTFGs at Lehigh University, funded by the Federal Highway Administration (Wimer and Sause 2004, and Kim and Sause 2005), it was founded that CFTFGs have several advantages. Two main advantages are: (1) the concrete-filled tubular flange provides more strength, stiffness, and lateral torsional stability than a flat plate flange with the same amount of steel, and (2) the vertical dimension of the tube reduces the web depth, thereby reducing the web slenderness. In particular, the increased torsional stability of the girders will reduce the number of diaphragms (or cross-frames) needed to brace the girders, thus reducing the time and cost of fabricating and erecting the bridge girder system.

For this project, CFTFGs are designed to be constructed as simple spans for dead loads, and are then made continuous for superimposed dead loads and live loads by splicing the CFTFGs at the pier. This construction sequence reduces the design moments and shears for the interior-pier sections of the CFTFGs and for the field splices at the pier. The bridge is also designed to be constructed with precast deck panels to promote accelerated construction.

To accomplish the project, the following tasks were conducted: (1) develop design criteria, (2) preliminary design of CFTFGs for the demonstration bridge, (3) preliminary design of the field splice, (4) preliminary design of the precast concrete deck, and (5) finite element analyses.

(1) Develop Design Criteria

Design criteria for CFTFGs were developed based on the results of previous analytical and experimental investigations (Wimer and Sause 2004, Kim and Sause, 2005). The design criteria are generally compatible with the 2000 PENNDOT Design Manual Part 4 (PENNDOT 2000) and the 2004 AASHTO LRFD Bridge Design Specifications (AASHTO 2004).

(2) Preliminary Design of CFTFGs for Demonstration Bridge

A preliminary design of a two-span continuous composite tubular flange girder bridge with spans of 100 ft-100 ft was developed. This study developed a preliminary design of the critical positive moment section, the interior-pier section, and the field splice. The CFTFGs were designed as simple spans for dead loads and continuous spans for superimposed dead loads and live loads.

The demonstration bridge cross-section was provided by PENNDOT and consists of four girders spaced at 8 ft-5.5 in centers with 3 ft overhangs. The concrete deck was 8 in. thick. Grade 50 steel and concrete with a compressive strength of 4 ksi were used. The design study considers the 2004 AASHTO LRFD Bridge Design Specifications (AASHTO 2004) and the PENNDOT Design Manual Part 4 (PENNDOT 2000) as well as the design criteria developed by the project. The design study results are based on several assumptions: (1) end diaphragms, but no interior diaphragms within the spans under construction conditions (during erection and deck placement) and one interior diaphragm at mid-span under service conditions, (2) bearing stiffeners and three equally-spaced intermediate transverse stiffeners (per span) with Category C′ fatigue details, (3) similar cross-sections for the positive moment section and the pier section, and (4) a field splice located at the pier section.

(3) Preliminary Design of Field Splice

The demonstration bridge is a two-span bridge with a field splice located at the pier. A preliminary design of the bolted field splice located at the pier was provided.

(4) Preliminary Design of Precast Concrete Deck

To promote accelerated construction, the demonstration bridge deck was designed to be built with precast concrete deck panels. The size of the panels was selected based on shipping and handling considerations. The panels were designed with a concrete compressive strength of 4 ksi, but higher strength could be easily achieved in a precast concrete plant. Each panel was designed with mild steel reinforcing bars, pre-tensioned strands, and post-tensioned strands.

(5) Finite Element Analyses

An analytical study of finite element (FE) models of the concrete-filled tubular flange girders (CFTFGs) of the demonstration bridge was conducted under simulated construction loading conditions using ABAQUS (ABAQUS 2002). Conditions before the field splice is made and before the girders are connected with shear connectors to the precast concrete deck were simulated. Therefore, the FE models were simple span girders non-composite with the deck.

To understand the possible buckling modes, elastic buckling analyses of the FE models were conducted. To investigate the lateral-torsional buckling strength, nonlinear load-displacement analyses of the FE models, including both material and geometric nonlinearity, were conducted. Single girder and multiple girder models (i.e., two girders, three girders, and four girders) were developed and analyzed to investigate the influence of adjacent girders. Two different diaphragm arrangements were studied. Scheme 9 (S9) has three diaphragms, including two end diaphragms and one interior diaphragm. Scheme 10 (S10) has only the two end diaphragms.

The FE analyses validated the design criteria and validated the preliminary design of the demonstration bridge for the construction conditions that were considered.

6.2 CONCLUSIONS

Based on the results of the accomplished tasks, the following conclusions are drawn:

- The CFTFGs designed for the demonstration bridge have enough lateral torsional stability under the construction loading conditions that were considered, even with no interior bracing within the span, so that fabrication and erection effort can be reduced by eliminating diaphragms.
- The field splice at the pier can simplify fabrication and erection, and reduce the dead load effects at the pier section. With this splice, the CFTFGs are constructed as simple spans for the weight of the CFTFGs and the bridge deck, but are made continuous for superimposed dead loads and live loads. As a result, the design moments and shears for the interior-pier section and for the field splice at the pier are reduced.
- The precast concrete deck can reduce the time needed for construction, compared to

a cast-in-place concrete deck, by reducing the time needed to place forms and reinforcing steel, and eliminating the time needed for the concrete to cure.

- The CFTFGs designed for the demonstration bridge with either the S9 (one interior diaphragm and two end diaphragms) or the S10 (no interior diaphragm and two end diaphragms) bracing arrangement are adequate for the construction loading conditions that were considered in the study.

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Appendix A. Positive Moment Region Preliminary Design

BRIDGE PARAMETERS (yellow highlight indicates input data)

Description: Two span continuous (for superimposed dead load and live load) composite CFTFG with each span of 100 ft and width of 31 ft - 4.5 in. The bridge cross section consists of 4 girders spaced at 8 ft - 5.5 in with 3 ft overhangs.

Bridge Width (in)	Span Length (in)	Girder Spacing (in)	Number of girders	Overhang (from girder centerline) (in)
$W := 376.5$	$L := 1200$	$s := 101.5$	$ng := 4$	$se := 36$

Yield Strength (ksi): $Fy := 50$ $\quad Rh := 1$

Young's Modulus (ksi): $E := 29000$

Concrete Strength (ksi): $fcprime := 4$

Slab Thickness (in): $Tslab := 8$

Haunch Thickness (in): $Thaunch := 3$

Concrete Modulus (ksi): $Ec := \dfrac{57000 \cdot \sqrt{fcprime \cdot 1000}}{1000}$

Modular ratio (input): $n := 8$ $\quad n_s := n$

Modular ratio (actual): $\dfrac{E}{Ec} = 8.044$

Long-term modular ratio: $n_l := 3 \cdot n$

Resistance Factor: $\Phi := 1.0$

MAXIMUM UNFACTORED MOMENTS DUE TO UNFACTORED LOADS

DC1 is weight of girders, slab, and bracing. DC2 is weight of haunch and barrier. DW is weight of wearing surface. Note that DC1 acts on simple spans and the max positive moment is at midspan. Remaining dead load and live load act on continuous spans and the max positive moment is at 40 ft. from abutment bearings (analysis of sections spaced at 10 ft). For simplicity, these moments are treated as if they act at the same section. More accurate calculations would consider each section separately.

$M_{dc1\_pos} := 18300$ kip*in $\quad M_{dc2\_pos} := 2730$ kip*in $\quad M_{dw\_pos} := 1764$ kip*in $\quad M_{ll\_pos} := 19232$ kip*in

$$MDC := \left[M_{dc1\_pos} + \left(\frac{0.050}{0.275 + 0.050}\right) \cdot M_{dc2\_pos}\right]$$

moment applied to non-composite section including haunch $\quad MDC = 18720$ kip*in

MAXIMUM FATIGUE + IMPACT MOMENT

$M_{fat\_pos} := 8325$ kip*in

MAXIMUM UNFACTORED SHEAR FORCES DUE TO UNFACTORED LOADS

$V_{dc1\_pos} := 61$ kips $\quad V_{dc2\_pos} := 12.19$ kips $\quad V_{dw\_pos} := 7.88$ kips $\quad V_{ll\_pos} := 93.44$ kips

$$VDC := \left[V_{dc1\_pos} + \left(\frac{0.050}{0.275 + 0.050}\right) \cdot V_{dc2\_pos}\right]$$

shear applied to non-composite section including haunch $\quad VDC = 62.9$ kips

MAXIMUM FATIGUE + IMPACT SHEAR FORCES

$V_{fat\_pos} := 38.2$ kips $\quad V_{fat\_neg} := -42.6$ kips

FACTORED LOAD COMBINATIONS

$Mconst\_pos := 1.5 \cdot MDC$	$Mconst\_pos = 28080$	kip*in
$MstI\_pos := (1.25 \cdot M_{dc1\_pos} + 1.25 \cdot M_{dc2\_pos} + 1.5 \cdot M_{dw\_pos} + 1.75 \cdot M_{ll\_pos}) \cdot 0.95$	$MstI\_pos = 59460$	kip*in
$MsvII\_pos := 1.0 \cdot M_{dc1\_pos} + 1.0 \cdot M_{dc2\_pos} + 1.0 \cdot M_{dw\_pos} + 1.3 \cdot M_{ll\_pos}$	$MsvII\_pos = 47795.6$	kip*in
$Mfat\_pos := 0.75 \cdot M_{fat\_pos}$	$Mfat\_pos = 6243.8$	kip*in
$Vconst\_pos := 1.5 VDC$	$Vconst\_pos = 94.3$	kips
$VstI\_pos := (1.25 \cdot V_{dc1\_pos} + 1.25 \cdot V_{dc2\_pos} + 1.5 \cdot V_{dw\_pos} + 1.75 \cdot V_{ll\_pos}) \cdot 0.95$	$VstI\_pos = 253.5$	kips
$Vfat\_pos := V_{dc1\_pos} + V_{dc2\_pos} + V_{dw\_pos} + 2(0.75 \cdot V_{fat\_pos})$	$Vfat\_pos = 138.4$	kips

GIRDER DIMENSIONS

Tube horizontal plate thickness: $Tt1 := \dfrac{3}{8}$ inches

Tube vertical plate thickness: $Tt2 := \dfrac{3}{8}$ inches

Bottom flange thickness: $Tbf := 1.5$ inches

Web thickness: $Tweb := \dfrac{8}{16}$ inches

Tube horizontal plate width: $Bt1 := 16$ inches

Tube vertical plate width: $Bt2 := 7.25$ inches

Bottom flange width: $Bbf := 18$ inches

Web depth	$Dweb := 36 - 2 \cdot Tt1 - Bt2 - Tbf$	$Dweb = 26.5$	inches
Total girder depth	$Dgirder := Tbf + Dweb + 2 \cdot Tt1 + Bt2$	$Dgirder = 36$	inches
Depth including deck	$Dtotal := Dgirder + Thaunch + Tslab$	$Dtotal = 47$	inches

GIRDER AREAS

$Abf := Bbf \cdot Tbf$	$Abf = 27$	in^2	Area of bottom flange
$Atube := 2 \cdot Tt1 \cdot Bt1 + 2 \cdot Tt2 \cdot Bt2$	$Atube = 17.44$	in^2	Area of tube
$Aw := Dweb \cdot Tweb$	$Aw = 13.25$	in^2	Area of web
$Asteel := Aw + Atube + Abf$	$Asteel = 57.69$	in^2	Total steel area
$Acon := \frac{Bt2 \cdot (Bt1 - 2 \cdot Tt2)}{n_s}$	$Acon = 13.82$	in^2	Equivalent area of concrete in tube (short term)
$Aclong := \frac{Bt2 \cdot (Bt1 - 2 \cdot Tt2)}{n_l}$	$Aclong = 4.607$	in^2	Equivalent area of concrete in tube (long term)

EFFECTIVE WIDTH OF SLAB (INTERIOR GIRDER)

$beff1 := \frac{L}{4}$ $\quad beff1 = 300$ $\quad beff2 := s$ $\quad beff2 = 101.5$ $\quad beff3 := 12 \cdot Tslab + \frac{Bt1}{2}$ $\quad beff3 = 104$

The smallest beff governs

$$Beffi := \begin{cases} beff1 & \text{if } beff1 \le beff2 \wedge beff1 \le beff3 \\ beff2 & \text{if } beff2 \le beff1 \wedge beff2 \le beff3 \\ beff3 & \text{otherwise} \end{cases}$$

$Beffi = 101.5$ in

EFFECTIVE WIDTH OF SLAB (EXTERIOR GIRDER)

$beff4 := \left(\frac{s}{2}\right) + se$ $\quad beff4 = 86.75$

The smallest beff governs

$$Beffe := \begin{cases} beff1 & \text{if } beff1 \le beff2 \wedge beff1 \le beff3 \\ beff4 & \text{if } beff4 \le beff1 \wedge beff4 \le beff3 \\ beff3 & \text{otherwise} \end{cases}$$

$Beffe = 86.75$ in

SELECT EFFECTIVE WIDTH OF SLAB (use Beffi for interior girder, Beffe for exterior girder, or minimum)

$Beff := \min(Beffe, Beffi)$ $\quad Beff = 86.75$ $\quad$ Note: here the minimum is used, which is for an exterior girder.

TUBE THICKNESS REQUIREMENT

$C_{t1} := \frac{Bt1 - 2 \cdot Tt2}{Tt1}$ $\quad C_{t1} = 40.67$ $\quad C_{t2} := \frac{Bt2}{Tt2}$ $\quad C_{t2} = 19.33$ $\quad D_t := 1.7 \cdot \sqrt{\frac{E}{Fy}}$ $\quad D_t = 40.94$

$Ratio_{tubethickness1} := \frac{C_{t1}}{D_t}$ $\quad Ratio_{tubethickness1} = 0.993$ $\quad Ratio_{tubethickness2} := \frac{C_{t2}}{D_t}$ $\quad Ratio_{tubethickness2} = 0.472$

$$Ratio_{tubethickness} := \begin{cases} Ratio_{tubethickness1} & \text{if } Ratio_{tubethickness1} > Ratio_{tubethickness2} \\ Ratio_{tubethickness2} & \text{otherwise} \end{cases}$$

$Ratio_{tubethickness} = 0.993$ $\quad$ < 1 therefore wall thickness is okay

MINIMUM DEPTH OF GIRDER

$Dgirder_{min} := 0.027 \cdot L$ $Dgirder_{min} = 32.4$ inches $Dgirder = 36$ > $Dgirder_{min} = 32.4$ o.k.

CONSTANT FOR CALCULATING KT

$\beta := .257$ β is a constant dependent upon (Bt1-2*Tt2)/Bt2

SECTION PROPERTIES

Calculate the elastic neutral axis for
1. **Noncomposite steel section with short term concrete (n) and with long term concrete (3n) in tube**
2. **Short term composite (with deck) section (n) with short term concrete (n) in tube**
3. **Long term composite (with deck) section (3*n) with long term concrete (3*n) in tube**

1. Steel section elastic neutral axis (reference line taken at the top of the top flange)

Assuming short term concrete in tube

$A_{girder} := Acon + Atube + Aw + Abf$

$$ENA_{girder} := \frac{Acon \cdot \left(Tt1 + \frac{Bt2}{2}\right) + Atube \cdot \left(Tt1 + \frac{Bt2}{2}\right) + Aw \cdot \left(2 \cdot Tt1 + Bt2 + \frac{Dweb}{2}\right) + Abf \cdot \left(2 \cdot Tt1 + Bt2 + Dweb + \frac{Tbf}{2}\right)}{A_{girder}}$$

$ENA_{girder} = 19.00$ in from the top of the girder **<== short term concrete in tube**

Assuming long term concrete in tube

$A_{girderlong} := Aclong + Atube + Aw + Abf$

$$ENA_{girderlong} := \frac{Aclong \cdot \left(Tt1 + \frac{Bt2}{2}\right) + Atube \cdot \left(Tt1 + \frac{Bt2}{2}\right) + Aw \cdot \left(2 \cdot Tt1 + Bt2 + \frac{Dweb}{2}\right) + Abf \cdot \left(2 \cdot Tt1 + Bt2 + Dweb + \frac{Tbf}{2}\right)}{A_{girderlong}}$$

$ENA_{girderlong} = 21.21$ in from the top of the girder **<== long term concrete in tube**

Assuming no concrete in tube

$A_{girdernoconc} := Atube + Aw + Abf$

$$ENA_{girdernoconc} := \frac{Atube \cdot \left(Tt1 + \frac{Bt2}{2}\right) + Aw \cdot \left(2 \cdot Tt1 + Bt2 + \frac{Dweb}{2}\right) + Abf \cdot \left(2 \cdot Tt1 + Bt2 + Dweb + \frac{Tbf}{2}\right)}{A_{girdernoconc}}$$

$ENA_{girdernoconc} = 22.59$ in from the top of the girder **<== no concrete in tube**

2. Short term elastic neutral axis (reference line taken at the top of the concrete deck)

ENA(short) = the elastic neutral axis of the short term composite (with deck) section with short term concrete in tube

Btr(hshort) = transformed width of the haunch for short term composite section

Btr(short) = transformed width of the slab for the short term composite section
(here taken as zero to neglect haunch area, otherwise =Bt1/ n_s)

$Btr_{short} := \frac{Beff}{n_s}$ $Btr_{hshort} := 0$

Q 1..5 = first moment of area

$$Q1 := Abf \cdot \left[Tslab + Thaunch + (2 \cdot Tt1 + Bt2) + Dweb + \frac{Tbf}{2} \right]$$

$$Q2 := Aw \cdot \left[Tslab + Thaunch + (2 \cdot Tt1 + Bt2) + \frac{Dweb}{2} \right]$$

$$Q3 := Atube \cdot \left[Tslab + Thaunch + \left(Tt1 + \frac{Bt2}{2} \right) \right]$$

$$Q4 := Acon \cdot \left[Tslab + Thaunch + \left(Tt1 + \frac{Bt2}{2} \right) \right]$$

$$Q5 := Thaunch \cdot Btr_{hshort} \cdot \left(Tslab + \frac{Thaunch}{2} \right) + Tslab \cdot Btr_{short} \cdot \frac{Tslab}{2}$$

$$A_{short} := A_{girder} + Thaunch \cdot Btr_{hshort} + Tslab \cdot Btr_{short}$$

$ENA_{short} := \frac{Q1 + Q2 + Q3 + Q4 + Q5}{A_{short}}$ $ENA_{short} = 15.75$ in from the top of the slab

Assuming no concrete in tube

$$A_{shortnoconc} := A_{girdernoconc} + Thaunch \cdot Btr_{hshort} + Tslab \cdot Btr_{short}$$

$ENA_{shortnoconc} := \frac{Q1 + Q2 + Q3 + Q5}{A_{shortnoconc}}$ $ENA_{shortnoconc} = 15.82$ in from the top of the slab

3. Long term elastic neutral axis

ENA(long) = the elastic neutral axis of the long term composite (with deck) section with long term concrete in tube

Btr(hlong) = transformed width of the haunch for long term composite section

Btr(long) = transformed width of the slab for the long term composite section
(here taken as zero to neglect haunch area, otherwise =Bt1/ n_l)

$Btr_{long} := \frac{Beff}{n_l}$ $Btr_{hlong} := 0$

$$Q1 := Abf \cdot \left[Tslab + Thaunch + (2 \cdot Tt1 + Bt2) + Dweb + \frac{Tbf}{2} \right]$$

$$Q2 := Aw \cdot \left[Tslab + Thaunch + (2 \cdot Tt1 + Bt2) + \frac{Dweb}{2} \right]$$

$$Q3 := Atube \cdot \left[Tslab + Thaunch + \left(Tt1 + \frac{Bt2}{2} \right) \right]$$

$$Q4 := Aclong \cdot \left[Tslab + Thaunch + \left(Tt1 + \frac{Bt2}{2} \right) \right]$$

$$Q5 := Thaunch \cdot Btr_{hlong} \cdot \left(Tslab + \frac{Thaunch}{2} \right) + Tslab \cdot Btr_{long} \cdot \frac{Tslab}{2}$$

$A_{long} := A_{girderlong} + Thaunch \cdot Btr_{hlong} + Tslab \cdot Btr_{long}$

$ENA_{long} := \dfrac{Q1 + Q2 + Q3 + Q4 + Q5}{A_{long}}$ $\quad ENA_{long} = 23.27$ in from the top of the concrete

Assuming no concrete in tube

$A_{longnoconc} := Atube + Aw + Abf + Thaunch \cdot Btr_{hlong} + Tslab \cdot Btr_{long}$

$ENA_{longnoconc} := \dfrac{Q1 + Q2 + Q3 + Q5}{A_{longnoconc}}$ $\quad ENA_{longnoconc} = 23.709$ in from the top of the concrete

Calculate the moment of inertia for

1. Steel section with short term concrete (n) in tube and with long term concrete(3n) in tube

2. Short term composite (with deck) section (n) with short term concrete(n) in tube

3. Long term composite (with deck) section (3*n) with long term concrete(3*n) in tube

1. Steel section moment of inertia

Ix(girder) = moment of inertia of the steel section with short term concrete in tube

$$Ix1 := \frac{1}{12} \cdot Bbf \cdot Tbf^3 + Abf \cdot \left[(2 \cdot Tt1 + Bt2) + Dweb + \frac{Tbf}{2} - ENA_{girder}\right]^2$$

$$Ix2 := \frac{1}{12} \cdot Tweb \cdot Dweb^3 + Aw \cdot \left[ENA_{girder} - (2 \cdot Tt1 + Bt2) - \frac{Dweb}{2}\right]^2$$

$$Ix3 := \left[\frac{1}{12} \cdot Bt1 \cdot (Bt2 + 2 \cdot Tt1)^3 - \frac{1}{12} \cdot (Bt1 - 2 \cdot Tt2) \cdot Bt2^3\right] + Atube \cdot \left[ENA_{girder} - \left(Tt1 + \frac{Bt2}{2}\right)\right]^2$$

$$Ix4 := \frac{\frac{1}{12} \cdot (Bt1 - 2 \cdot Tt2) \cdot Bt2^3}{n_s} + \frac{(Bt1 - 2 \cdot Tt2) \cdot Bt2}{n_s} \cdot \left[ENA_{girder} - \left(Tt1 + \frac{Bt2}{2}\right)\right]^2$$

$Ix_{girder} := Ix1 + Ix2 + Ix3 + Ix4$ $\quad Ix_{girder} = 15269$ in^4 **<== short term concrete in tube**

Ix(girderlong) = moment of inertia of the steel section with long term concrete in tube

$$Ix1lo := \frac{1}{12} \cdot Bbf \cdot Tbf^3 + Abf \cdot \left[(2 \cdot Tt1 + Bt2) + Dweb + \frac{Tbf}{2} - ENA_{girderlong}\right]^2$$

$$Ix2lo := \frac{1}{12} \cdot Tweb \cdot Dweb^3 + Aw \cdot \left[ENA_{girderlong} - (2 \cdot Tt1 + Bt2) - \frac{Dweb}{2}\right]^2$$

$$Ix3lo := \left[\frac{1}{12}\cdot Bt1\cdot(Bt2 + 2\cdot Tt1)^3 - \frac{1}{12}\cdot(Bt1 - 2\cdot Tt2)\cdot Bt2^3\right] + Atube\cdot\left[ENA_{girderlong} - \left(Tt1 + \frac{Bt2}{2}\right)\right]^2$$

$$Ix4lo := \frac{\frac{1}{12}\cdot(Bt1 - 2\cdot Tt2)\cdot Bt2^3}{n_l} + \frac{(Bt1 - 2\cdot Tt2)\cdot Bt2}{n_l}\cdot\left[ENA_{girderlong} - \left(Tt1 + \frac{Bt2}{2}\right)\right]^2$$

$Ix_{girderlong} := Ix1lo + Ix2lo + Ix3lo + Ix4lo$ $\quad Ix_{girderlong} = 12850$ $\quad in^4$ $\quad$ **<== long term concrete in tube**

Ix(girdernoconc) = moment of inertia of steel section assuming no concrete in tube

$$Ix1nc := \frac{1}{12}\cdot Bbf\cdot Tbf^3 + Abf\cdot\left[(2\cdot Tt1 + Bt2) + Dweb + \frac{Tbf}{2} - ENA_{girdernoconc}\right]^2$$

$$Ix2nc := \frac{1}{12}\cdot Tweb\cdot Dweb^3 + Aw\cdot\left[ENA_{girdernoconc} - (2\cdot Tt1 + Bt2) - \frac{Dweb}{2}\right]^2$$

$$Ix3nc := \left[\frac{1}{12}\cdot Bt1\cdot(Bt2 + 2\cdot Tt1)^3 - \frac{1}{12}\cdot(Bt1 - 2\cdot Tt2)\cdot Bt2^3\right] + Atube\cdot\left[ENA_{girdernoconc} - \left(Tt1 + \frac{Bt2}{2}\right)\right]^2$$

$Ix_{girdernoconc} := Ix1nc + Ix2nc + Ix3nc$ $\quad Ix_{girdernoconc} = 11356$ $\quad in^4$ $\quad$ **<== no concrete in tube**

$Ix_{girdermidnoconc} := Ix_{girdernoconc}$

2. Short term moment of interia

Ix(short) = the moment of intertia for the short term composite (with deck) section with short term concrete in tube

$$Ix1 := \frac{1}{12}\cdot Bbf\cdot Tbf^3 + Abf\cdot\left[Tslab + Thaunch + (2\cdot Tt1 + Bt2) + Dweb + \frac{Tbf}{2} - ENA_{short}\right]^2$$

$$Ix2 := \frac{1}{12}\cdot Tweb\cdot Dweb^3 + Aw\cdot\left[Tslab + Thaunch + (2\cdot Tt1 + Bt2) + \frac{Dweb}{2} - ENA_{short}\right]^2$$

$$Ix3 := \left[\frac{1}{12}\cdot Bt1\cdot(Bt2 + 2\cdot Tt1)^3 - \frac{1}{12}\cdot(Bt1 - 2\cdot Tt2)\cdot Bt2^3\right] + Atube\cdot\left[ENA_{short} - Tslab - Thaunch - \left(Tt1 + \frac{Bt2}{2}\right)\right]^2$$

$$Ix4 := \frac{\frac{1}{12}\cdot(Bt1 - 2\cdot Tt2)\cdot Bt2^3}{n_s} + \frac{(Bt1 - 2\cdot Tt2)\cdot Bt2}{n_s}\cdot\left[ENA_{short} - Tslab - Thaunch - \left(Tt1 + \frac{Bt2}{2}\right)\right]^2$$

$$Ix5 := \frac{1}{12}\cdot Btr_{short}\cdot Tslab^3 + Btr_{short}\cdot(Tslab)\cdot\left(ENA_{short} - \frac{Tslab}{2}\right)^2$$

$$Ix6 := \frac{1}{12} \cdot Btr_{hshort} \cdot Thaunch^3 + Btr_{hshort} \cdot Thaunch \cdot \left(ENA_{short} - Tslab - \frac{Thaunch}{2}\right)^2$$

$Ix_{short} := Ix1 + Ix2 + Ix3 + Ix4 + Ix5 + Ix6$ $\qquad Ix_{short} = 42221$ $\qquad in^4$

Ix(shortnoconc) = the moment of inertia for the short term composite (with deck) section assuming no concrete in tube

$$Ix1nc := \frac{1}{12} \cdot Bbf \cdot Tbf^3 + Abf \cdot \left[Tslab + Thaunch + (2 \cdot Tt1 + Bt2) + Dweb + \frac{Tbf}{2} - ENA_{shortnoconc}\right]^2$$

$$Ix2nc := \frac{1}{12} \cdot Tweb \cdot Dweb^3 + Aw \cdot \left[Tslab + Thaunch + (2 \cdot Tt1 + Bt2) + \frac{Dweb}{2} - ENA_{shortnoconc}\right]^2$$

$$Ix3nc := \left[\frac{1}{12} \cdot Bt1 \cdot (Bt2 + 2 \cdot Tt1)^3 - \frac{1}{12} \cdot (Bt1 - 2 \cdot Tt2) \cdot Bt2^3\right] + Atube \cdot \left[ENA_{shortnoconc} - Tslab - Thaunch - \left(Tt1 + \frac{Bt2}{2}\right)\right]^2$$

$$Ix4nc := 0$$

$$Ix5nc := \frac{1}{12} \cdot Btr_{short} \cdot Tslab^3 + Btr_{short} \cdot (Tslab) \cdot \left(ENA_{shortnoconc} - \frac{Tslab}{2}\right)^2$$

$$Ix6nc := \frac{1}{12} \cdot Btr_{hshort} \cdot Thaunch^3 + Btr_{hshort} \cdot Thaunch \cdot \left(ENA_{shortnoconc} - Tslab - \frac{Thaunch}{2}\right)^2$$

$Ix_{shortnoconc} := Ix1nc + Ix2nc + Ix3nc + Ix5nc + Ix6nc$ $\qquad Ix_{shortnoconc} = 42152$ $\qquad in^4$

3. Long term moment of inertia

Ix(long) = the moment of intertia for the long term composite (with deck) section with long term concrete in tube

$$Ix1 := \frac{1}{12} \cdot Bbf \cdot Tbf^3 + Abf \cdot \left[Tslab + Thaunch + (2 \cdot Tt1 + Bt2) + Dweb + \frac{Tbf}{2} - ENA_{long}\right]^2$$

$$Ix2 := \frac{1}{12} \cdot Tweb \cdot Dweb^3 + Aw \cdot \left[Tslab + Thaunch + (2 \cdot Tt1 + Bt2) + \frac{Dweb}{2} - ENA_{long}\right]^2$$

$$Ix3 := \left[\frac{1}{12} \cdot Bt1 \cdot (Bt2 + 2 \cdot Tt1)^3 - \frac{1}{12} \cdot (Bt1 - 2 \cdot Tt2) \cdot Bt2^3\right] + Atube \cdot \left[ENA_{long} - Tslab - Thaunch - \left(Tt1 + \frac{Bt2}{2}\right)\right]^2$$

$$Ix4 := \frac{\frac{1}{12} \cdot (Bt1 - 2 \cdot Tt2) \cdot Bt2^3}{n_l} + \frac{(Bt1 - 2 \cdot Tt2) \cdot Bt2}{n_l} \cdot \left[ENA_{long} - Tslab - Thaunch - \left(Tt1 + \frac{Bt2}{2}\right)\right]^2$$

$$Ix5 := \frac{1}{12} \cdot Btr_{long} \cdot Tslab^3 + Btr_{long} \cdot (Tslab) \cdot \left(ENA_{long} - \frac{Tslab}{2}\right)^2$$

$$Ix6 := \frac{1}{12} \cdot Btr_{hlong} \cdot Thaunch^3 + Btr_{hlong} \cdot Thaunch \cdot \left(ENA_{long} - Tslab - \frac{Thaunch}{2}\right)^2$$

$$Ix_{long} := Ix1 + Ix2 + Ix3 + Ix4 + Ix5 + Ix6 \qquad Ix_{long} = 28725 \quad in^4$$

Ix(longnoconc) = the moment of inertia for the long term composite (with deck) section assuming no concrete in tube

$$Ix1nc := \frac{1}{12} \cdot Bbf \cdot Tbf^3 + Abf \cdot \left[Tslab + Thaunch + (2 \cdot Tt1 + Bt2) + Dweb + \frac{Tbf}{2} - ENA_{longnoconc}\right]^2$$

$$Ix2nc := \frac{1}{12} \cdot Tweb \cdot Dweb^3 + Aw \cdot \left[Tslab + Thaunch + (2 \cdot Tt1 + Bt2) + \frac{Dweb}{2} - ENA_{longnoconc}\right]^2$$

$$Ix3nc := \left[\frac{1}{12} \cdot Bt1 \cdot (Bt2 + 2 \cdot Tt1)^3 - \frac{1}{12} \cdot (Bt1 - 2 \cdot Tt2) \cdot Bt2^3\right] + Atube \cdot \left[ENA_{longnoconc} - Tslab - Thaunch - \left(Tt1 + \frac{Bt2}{2}\right)\right]^2$$

$$Ix4nc := 0$$

$$Ix5nc := \frac{1}{12} \cdot Btr_{long} \cdot Tslab^3 + Btr_{long} \cdot (Tslab) \cdot \left(ENA_{longnoconc} - \frac{Tslab}{2}\right)^2$$

$$Ix6nc := \frac{1}{12} \cdot Btr_{hlong} \cdot Thaunch^3 + Btr_{hlong} \cdot Thaunch \cdot \left(ENA_{longnoconc} - Tslab - \frac{Thaunch}{2}\right)^2$$

$$Ix_{longnoconc} := Ix1nc + Ix2nc + Ix3nc + Ix4nc + Ix5nc + Ix6nc \qquad Ix_{longnoconc} = 28373 \quad in^4$$

Calculate the section modulus for (Sx)

1. **Steel section with short term filled concrete(n) and with long term filled concrete(3n)**
2. **Short term composite section (n) with short term filled concrete(n)**
3. **Long term composite section (3*n) with long term filled concrete(3*n)**

1. Steel section modulus

Sx(girder1) = section modulus about the elastic neutral axis of the steel section only with respect to the **compression** steel tube

$$Sx_{girder1} := \frac{Ix_{girder}}{ENA_{girder}} \qquad Sx_{girder1} = 803.8 \quad in^3 \qquad Sx_{girder1long} := \frac{Ix_{girderlong}}{ENA_{girderlong}} \qquad Sx_{girder1long} = 605.8 \quad in^3$$

Sx(girder1noconc) = section modulus about the elastic neutral axis of the steel section assuming no concrete in tube with respect to the compression steel tube

$$Sx_{girder1noconc} := \frac{Ix_{girdernoconc}}{ENA_{girdernoconc}} \qquad Sx_{girder1noconc} = 502.8 \quad in^3$$

Sx(girder2) = section modulus about the elastic neutral axis of steel section with respect to the **tension** flange

$$Sx_{girder2} := \frac{Ix_{girder}}{Dgirder - ENA_{girder}} \qquad Sx_{girder2} = 898.0 \quad in^3$$

$$Sx_{girder2long} := \frac{Ix_{girderlong}}{Dgirder - ENA_{girderlong}} \qquad Sx_{girder2long} = 869.1 \quad in^3$$

Sx(girder2noconc) = section modulus about the elastic neutral axis of the steel section assuming no concrete in tube with respect to the tension flange

$$Sx_{girder2noconc} := \frac{Ix_{girdernoconc}}{Dgirder - ENA_{girdernoconc}} \qquad Sx_{girder2noconc} = 846.743 \quad in^3$$

2. Short term section modulus

Sx(short1) = the section modulus about the elastic neutral axis for the **compression steel tube** of the short term composite section

$$Sx_{short1} := \frac{Ix_{short}}{ENA_{short} - Thaunch - Tslab} \qquad Sx_{short1} = 8896 \quad in^3$$

Sx(short1noconc) = the section modulus about the elastic neutral axis for the compression steel tube of the short term composite section assuming there is no concrete in the tube

$$Sx_{short1noconc} := \frac{Ix_{shortnoconc}}{ENA_{shortnoconc} - Thaunch - Tslab} \qquad Sx_{short1noconc} = 8749.9 \quad in^3$$

Sx(short2) = the section modulus about the elastic neutral axis for the **tension flange** of the short term composite section

$$Sx_{short2} := \frac{Ix_{short}}{Dtotal - ENA_{short}} \qquad Sx_{short2} = 1350.9$$

Sx(short2noconc) = the section modulus about the elastic neutral axis for the tension flange of the short term composite section assuming there is no concrete in the tube

$$Sx_{short2noconc} := \frac{Ix_{shortnoconc}}{Dtotal - ENA_{shortnoconc}} \qquad Sx_{short2noconc} = 1351.8 \quad in^3$$

3. Long term section modulus

Sx(long1) = the section modulus about the elastic neutral axis for the **compression steel tube** of the long term composite section

$$Sx_{long1} := \frac{Ix_{long}}{ENA_{long} - Thaunch - Tslab} \qquad Sx_{long1} = 2341.3 \quad in^3$$

Sx(long1noconc) = the section modulus about the elastic neutral axis for the compression steel tube of the long term composite section assuming there is no concrete in the tube

$$Sx_{long1noconc} := \frac{Ix_{longnoconc}}{ENA_{longnoconc} - Thaunch - Tslab} \qquad Sx_{long1noconc} = 2232.6 \quad in^3$$

Sx(long2) = the section modulus about the elastic neutral axis for the **tension flange** of the long term composite section

$$Sx_{long2} := \frac{Ix_{long}}{Dtotal - ENA_{long}} \qquad Sx_{long2} = 1210.5 \quad in^3$$

Sx(long2noconc) = the section modulus about the elastic neutral axis for the tension flange of the long term composite section assuming there is no concrete in the tube

$Sx_{long2noconc} := \dfrac{Ix_{longnoconc}}{Dtotal - ENA_{longnoconc}}$ $Sx_{long2noconc} = 1218.2$ in^3

STRESS IN COMPRESSION FLANGE (TUBE) (fc) AND TENSION FLANGE (ft) DUE TO CONSTRUCTION LOADING BASED ON TRANSFORMED SECTION

$fc := \dfrac{Mconst\_pos}{Sx_{girder1}}$ $fc = 34.933$ ksi $ft := \dfrac{Mconst\_pos}{Sx_{girder2}}$ $ft = 31.27$ ksi

$f_{incon} := \dfrac{ENA_{girder} - Tt1}{ENA_{girder}} \cdot \dfrac{fc}{n_s}$ $f_{incon} = 4.28$ ksi **stress in concrete within tube under construction loading based on transformed section analysis**

YIELD MOMENT OF STEEL SECTION WITH SHORT TERM CONCRETE IN TUBE FROM TRANSFORMED SECTION

My = yield moment of the girder, taken as Fy times the section modulus.

$My_{girder1} := Fy \cdot Sx_{girder1}$ $My_{girder1} = 40190.991$ k-in

$My_{girder2} := Fy \cdot Sx_{girder2}$ $My_{girder2} = 44898.016$ k-in

$$My_{girder\_ts} := \begin{cases} My_{girder1} & \text{if } My_{girder1} < My_{girder2} \\ My_{girder2} & \text{if } My_{girder1} \geq My_{girder2} \end{cases}$$ $My_{girder\_ts} = 40190.991$ k-in

SECTION PROPERTIES FROM STRAIN COMPATIBILITY CALCULATIONS USING STRESS BLOCK (This is from other calculation sheets)

$Mp_{com\_pos\_sb} := 80985$ kip*in **Capacity for section composite with deck using strain compatibility and stress block for deck (from Appendix C)**

$Dcp_{com\_pos\_sb} := 0$ in **Depth of web in compression when composite section capacity is reached (from Appendix C)**

$My_{girder\_pos\_sb} := 35401$ kip*in **Yield moment for section non-composite with deck using strain compatibility and stress block for concrete inside tube (from Appendix D)**

$ENA_{girder\_pos\_sb} := 19.984$ in **Elastic neutral axis depth from top of flange for section non-composite with deck using stress block for concrete inside of tube (from Appendix D)**

$fstopMDC := 16.820$ ksi **Stress in top fiber of steel tube for M_{DC} (MDC on page A-1) for section non-composite with deck based on strain compatibility and stress block for concrete inside tube (from Appendix D, used for Service II check)**

$fsbottomMDC := 20.372$ ksi **Corresponding stress in bottom flange**

CONSTRUCTION LOADING CHECK FOR FLEXURE

Yield moment of the steel section

$My_{girder\_ts} = 40191$ transformed section My

$My_{girder\_pos\_sb} = 35401$ strain compatibility (stress block) My

$$My_{girder\_pos} := \begin{cases} My_{girder\_ts} & \text{if } My_{girder\_ts} < My_{girder\_pos\_sb} \\ My_{girder\_pos\_sb} & \text{if } My_{girder\_ts} \geq My_{girder\_pos\_sb} \end{cases}$$

$My_{girder\_pos} = 35401$ k-in

Calculate depth of web in compression for construction, Dc

$Dc\_ts := ENA_{girder} - (2Tt1 + Bt2)$ $\quad Dc\_ts = 11.00$ in

$Dc\_pos\_sb := ENA_{girder\_pos\_sb} - (2Tt1 + Bt2)$ $\quad Dc\_pos\_sb = 11.98$ in

$$Dc\_pos := \begin{cases} Dc\_ts & \text{if } My_{girder\_ts} < My_{girder\_pos\_sb} \\ Dc\_pos\_sb & \text{if } My_{girder\_ts} \geq My_{girder\_pos\_sb} \end{cases}$$

$Dc\_pos = 11.98$ in

Web Slenderness Limit for "stocky web"

$$Atf := \frac{Bt2 \cdot (Bt1 - 2 \cdot Tt2)}{n_s} + Atube$$

$Atf = 31.258$ in² $\quad Abf = 27$ in²

$$\lambda b := \begin{cases} 5.76 & \text{if } Atf \geq Abf \\ 4.64 & \text{if } Atf < Abf \end{cases}$$

$\lambda b = 5.76$

$$A_w := 2 \cdot \frac{Dc\_pos}{Tweb} \qquad F_w := \lambda b \cdot \sqrt{\frac{E}{Fy}} \qquad Ratio_{web\_stocky} := \frac{A_w}{F_w}$$

if $Ratio_{web\_stocky} = 0.346$ < 1 then this section has **stocky web**

Web Slenderness Limit to minimize web distortion

$$A_{w\_dist} := \frac{Dweb}{Tweb} \qquad F_{w\_dist} := 11 \cdot \left(\frac{E}{Fy}\right)^{\frac{1}{3}} \qquad Ratio_{web\_dist} := \frac{A_{w\_dist}}{F_{w\_dist}}$$

if $Ratio_{web\_dist} = 0.578$ < 1 then this section is ok

General Cross Section Properties

$$Iyc := \frac{1}{12} \cdot (Bt2 + 2 \cdot Tt1) \cdot Bt1^3 - \frac{1}{12} \cdot Bt2 \cdot (Bt1 - 2 \cdot Tt2)^3 + \frac{\frac{1}{12} \cdot Bt2 \cdot (Bt1 - 2 \cdot Tt2)^3}{n_s}$$

$Iyc = 855.8$ in⁴

$$Iyt := \frac{1}{12} \cdot Tbf \cdot Bbf^3$$

$Iyt = 729.0$ in⁴

$$Iy := \frac{1}{12} \cdot Dweb \cdot Tweb^3 + Iyt + Iyc$$

$Iy = 1585.1$ in⁴

$$ry := \sqrt{\frac{Iyc + Iyt}{A_{girder}}} \qquad ry = 4.71 \quad in$$

Torsional Properties

$$K_T := \frac{Dweb \cdot Tweb^3}{3} + \frac{Bbf \cdot Tbf^3}{3} + \frac{\beta \cdot (Bt1 - 2 \cdot Tt2) \cdot Bt2^3}{n_s} + \frac{2 \cdot Tt2 \cdot Tt1 \cdot (Bt1 - Tt2)^2 \cdot (Bt2 + Tt1)^2}{(Bt1 - Tt2) \cdot Tt2 + (Bt2 + Tt1) \cdot Tt1} \qquad K_T = 665.9 \quad in^4$$

Calculate the nominal moment capacity for lateral torsional buckling (LTB)

For <u>demonstration</u> purposes, calculate the nominal LTB capacity twice:
(1) without the midspan cross frame and (2) including the midspan cross frame.
<u>Normally the calculation would be done only once for the appropriate bracing condition</u>

1. Here the calculation is performed for no interior bracing within the span. The nominal LTB flexural strength is calculated assuming the entire span is the unbraced length with fixed brace points at the ends of the span (known as the ideal flexural strength).

For the parabolic moment diagram with M=0 at the ends, Cb = 1.0 by 2004 AASHTO. Here use more accurate Cb=1.136

$$Lb0 := L \qquad Lb0 = 1200 \; in$$

$$Cb0 := 1.136$$

Using equations for ideal design flexural strength

$$Lb := Lb0 \qquad Cb := Cb0$$

Critical elastic LTB moment

$$Mcr := \frac{\pi \cdot E}{\frac{Lb}{ry}} \cdot \sqrt{0.385 \cdot K_T \cdot A_{girder} + \frac{2.467 \cdot Dgirder^2 \cdot A_{girder}{}^2}{\left(\frac{Lb}{ry}\right)^2}} \qquad Mcr = 48725.6 \quad k\text{-}in$$

Cross section moment capacity

$$Ms := My_{girder\_pos} \qquad Ms = 35401.0 \quad k\text{-}in$$

Strength reduction factor to account for LTB

$$\alpha s\_var := 0.8 \cdot \left[\sqrt{\left(\frac{Ms}{Mcr}\right)^2 + 2.2} - \frac{Ms}{Mcr}\right] \qquad \alpha s := \begin{cases} \alpha s\_var & \text{if } \alpha s\_var \le 1.0 \\ 1.0 & \text{otherwise} \end{cases} \qquad \alpha s = 0.74$$

Design flexural strength accounting for LTB

$$Md\_var := Cb \cdot \alpha s \cdot Ms \qquad Md0 := \begin{cases} Md\_var & \text{if } Md\_var \le Ms \\ Ms & \text{otherwise} \end{cases} \qquad Md0 = 29762.2 \quad k\text{-}in$$

2. Here, the calculation is performed for one interior brace at the midspan of the span. So the torsionally braced nominal flexural strength is calculated assuming the unbraced length is one half the span.

First, the ideal design flexural strength that corresponds to LTB between the brace points is calculated (i.e., using the unbraced length equal to 1/2 of the span and the appropriate Cb). This is upper bound on the nominal LTB flexural strength.

For the parabolic moment diagram with M=0 at one end and Mmax at the other end, Cb is calculated according to 2004 AASHTO as follows (using moment instead of stress since section is constant).

$Lb1 := \frac{L}{2}$ $\quad Lb1 = 600$ in

Determine M1 and M2 (maximum moment moment at midspan brace). Here M0 = 0 (the moment at the end of the span, and Mmid = the moment at the quarter point, which is 3/4 of the midspan moment

$M0 := 0 \quad Mmid := 0.75 \cdot Mconst\_pos$ kip-in $\quad M2 := Mconst\_pos \quad M2 = 28080$ kip-in

$M1 := 2 \cdot Mmid - M2 \quad M1 = 14040$ > $M0 = 0$ OK

Determine Cb from M1 and M2

$$Cb1 := 1.75 - 1.05 \cdot \left(\frac{M1}{M2}\right) + 0.3 \cdot \left(\frac{M1}{M2}\right)^2 \qquad Cb1 = 1.3$$

Ideal design flexural strength for LTB between braces calculated as above with the following Lb and Cb

$Lb := Lb1 \qquad Cb := Cb1$

$$Mcr := \frac{\pi \cdot E}{\frac{Lb}{ry}} \cdot \sqrt{0.385 \cdot K_T \cdot A_{girder} + \frac{2.467 \cdot Dgirder^2 \cdot A_{girder}^{\ 2}}{\left(\frac{Lb}{ry}\right)^2}} \qquad Mcr = 99410.5 \text{ k-in}$$

$Ms := My_{girder\_pos} \qquad Ms = 35401.0$ k-in

$$\alpha s\_var := 0.8 \cdot \left[\sqrt{\left(\frac{Ms}{Mcr}\right)^2 + 2.2} - \frac{Ms}{Mcr}\right] \qquad \alpha s := \begin{cases} \alpha s\_var & \text{if } \alpha s\_var \le 1.0 \\ 1.0 & \text{otherwise} \end{cases} \qquad \alpha s = 0.935$$

$$Md\_var := Cb \cdot \alpha s \cdot Ms \qquad Md1 := \begin{cases} Md\_var & \text{if } Md\_var \le Ms \\ Ms & \text{otherwise} \end{cases} \qquad Md1 = 35401 \text{ k-in}$$

Then, the design flexural strength for the torsionally braced CFTFGs is calculated.

Consider the entire span length and corresponding moment diagram. For the parabolic moment diagram with M=0 at the ends, Cb = 1.0 according to 2004 AASHTO. Here use more accurate Cb=1.136

$Cbu := 1.136$

Critical elastic LTB moment for girder without interior bracing in span (note Lb=L in formula)

$$Mcr\_ubr := \frac{\pi \cdot E}{\frac{L}{ry}} \cdot \sqrt{0.385 \cdot K_T \cdot A_{girder} + \frac{2.467 \cdot Dgirder^2 \cdot A_{girder}^{\ 2}}{\left(\frac{L}{ry}\right)^2}} \qquad Mcr\_ubr = 48725.6 \text{ k-in}$$

Critical moment including the bracing effect - required input regarding bracing properties

nb := 1		Number of interior bracing per span
Ib := 1170	in^4	Moment of inertia of bracing member about strong axis
N := 1	in	Contact length of torsional brace
ts := 1.5	in	stiffener thickness
bs := 16	in	stiffener width

Critical moment including the bracing effect - calculate Ieff, β, etc.

$c := Dc\_pos + Tt1 + \frac{Bt2}{2}$ $c = 15.98$ in $t := Dgirder - Dc\_pos - 2 \cdot Tt1 - Bt2 - \frac{Tbf}{2}$ $t = 15.27$ in

$Ieff := Iyc + \frac{t}{c} \cdot Iyt$ $Ieff = 1552.0$ Effective vertical axis moment of inertia

$h := \frac{Bt2}{2} + Tt1 + Dweb + \frac{Tbf}{2}$ $h = 31.25$ Distance between flange centroids

$\beta b := \frac{6 \cdot E \cdot Ib}{s}$ $\beta b = 2.006 \times 10^6$

$\beta g := \frac{24 \cdot (ng - 1)^2}{ng} \cdot \frac{s^2 \cdot E \cdot Ix_{girder}}{L^3}$ $\beta g = 1.426 \times 10^5$

$\beta sec := 3.3 \cdot \frac{E}{h} \cdot \left[\frac{(N + 1.5 \cdot h) \cdot Tweb^3}{12} + \frac{ts \cdot bs^3}{12}\right]$ $\beta sec = 1.569 \times 10^6$

$\beta T := 1$ Determine βT

$eq(\beta T) := \frac{1}{\beta b} + \frac{1}{\beta sec} + \frac{1}{\beta g} - \frac{1}{\beta T}$ $\beta T := root(eq(\beta T), \beta T)$ $\beta T = 1.227 \times 10^5$

Critical moment including the bracing effect.

$Mbr := \sqrt{\frac{\beta T \cdot E \cdot Ieff \cdot nb}{1.2 \cdot L}}$ $Mbr = 61927$ k-in

Critical elastic LTB moment including torsional brace effect

$Cbb := Cb1$ $Cbb = 1.3$

$Mcr\_br := \sqrt{Mcr\_ubr^2 + \frac{Cbb^2}{Cbu^2} \cdot Mbr^2}$ $Mcr\_br = 86002$ k-in

Strength reduction factor for torsionally braced girder

$\alpha s\_br\_var := 0.8 \cdot \left[\sqrt{\left(\frac{Ms}{Mcr\_br}\right)^2 + 2.2} - \frac{Ms}{Mcr\_br}\right]$

$\alpha s\_br := \begin{cases} \alpha s\_br\_var & \text{if } \alpha s\_br\_var \le 1.0 \\ 1.0 & \text{otherwise} \end{cases}$

$\alpha s = 0.935$

Design flexural strength for torsionally braced CFTFG

$Md\_br\_var := Cbu \cdot \alpha s\_br \cdot Ms$ $\quad Md\_br\_var = 36279.8$ k-in

Check that ideal flexural strength is not exceeded

$$Md\_br1 := \begin{cases} Md\_br\_var & \text{if } Md\_br\_var \le Md1 \\ Md1 & \text{otherwise} \end{cases}$$

$Md\_br1 = 35401$ k-in

Check the nominal moment capacity for lateral torsional buckling (LTB) against the demand from construction load.
For <u>demonstration</u> purposes, check the nominal LTB capacity twice:
(1) without the midspan cross frame and (2) including the midspan cross frame.
<u>Normally the calculation would be done only once for the appropriate bracing condition</u>

1. With no interior bracing within the span.

$$Ratio_{ltbresistance0} := \frac{Mconst\_pos}{\Phi \cdot Md0}$$

$Ratio_{ltbresistance0} = 0.943$ $\quad$ < 1 therefore ok for construction

2. With one interior brace at the midspan of the span.

$$Ratio_{ltbresistance1} := \frac{Mconst\_pos}{\Phi \cdot Md\_br1}$$

$Ratio_{ltbresistance1} = 0.793$ $\quad$ < 1 therefore ok for construction

CONSTRUCTION LOADING CHECK FOR SHEAR

The web is quite stocky and the stiffeners are widely spaced, so the web was designed for the Strength I limit state as unstiffened. Calculations given below for the Strength I limit state show that the web shear capacity (Vn = Vcr) equals Vp (i.e., C = 1.0) when the web is treated as unstiffened (AASHTO LRFD Article 6.10.9.2). Tension field action is not included (or needed). Also, note that the web thickness and depth are constant, so the calculations apply to all regions of the web. As shown later, the shear capacity exceeds the shear demand for the Strength I load combination (VstI_pos) and therefore the requirement of AASHTO LRFD Article 6.10.3.3 (Vu = Vconst_pos < Vcr) is also satisfied. (Strictly speaking, since the web is treated as unstiffened, AASHTO LRFD Article 6.10.3.3 does not apply). If Vn=Vcr were less than Vp and tension field action was included in calculating Vn = Vcr for the Strength I limit state, then a separate calculation of Vcr according to Article 6.10.9.3.3 would be needed and this Vcr would be checked against Vconst_pos here.

SERVICE II LIMIT STATE CHECK FOR FLEXURE

f(DC1+DC2a) = flexural stress due to the dead load acting on steel girder (with concrete in tube)
f(DC2b) = flexural stress due to the dead load acting on (long term) section composite with deck
f(DW) = flexural stress due to dead load acting on (long term) section composite with deck
f(LL) = flexural stress of due to live load acting on (short term) section composite with deck

Calculate f(DC1+DC2a)

Using transformed section

$$f_{DC\_ts} := \frac{M_{dc1\_pos} + \left(\frac{0.050}{0.275 + 0.050}\right) \cdot M_{dc2\_pos}}{Sx_{girder2long}}$$ account for haunch $\quad f_{DC\_ts} = 21.54$ ksi

$$f_{DCT\_ts} := \frac{M_{dc1\_pos} + \left(\frac{0.050}{0.275 + 0.050}\right) \cdot M_{dc2\_pos}}{Sx_{girder1long}}$$ $\quad f_{DCT\_ts} = 30.9$ ksi

Using stress block

$f_{DC\_sb} := fsbottomMDC \quad f_{DC\_sb} = 20.37$ ksi $\quad f_{DCT\_sb} := fstopMDC \quad f_{DCT\_sb} = 16.82$ ksi

Find correct result

$$f_{DC} := \begin{cases} f_{DC\_ts} & \text{if } My_{girder\_ts} < My_{girder\_pos\_sb} \\ f_{DC\_sb} & \text{otherwise} \end{cases}$$ $\quad f_{DC} = 20.37$ ksi

$$f_{DCT} := \begin{cases} f_{DCT\_ts} & \text{if } My_{girder\_ts} < My_{girder\_pos\_sb} \\ f_{DCT\_sb} & \text{otherwise} \end{cases}$$ $\quad f_{DCT} = 16.82$ ksi

Calculate f(DC2b)

$$f_{DC2} := \frac{\left(\frac{0.275}{0.275 + 0.050}\right) \cdot M_{dc2\_pos}}{Sx_{long2noconc}}$$ account for barrier $\quad f_{DC2} = 1.896$ ksi

$$f_{DC2T} := \frac{\left(\frac{0.275}{0.275 + 0.050}\right) \cdot M_{dc2\_pos}}{Sx_{long1noconc}}$$ $\quad f_{DC2T} = 1.035$ ksi

Calculate f(DW)

$$f_{DW} := \frac{M_{dw\_pos}}{Sx_{long2noconc}}$$ $\quad f_{DW} = 1.448$ ksi

$$f_{DWT} := \frac{M_{dw\_pos}}{Sx_{long1noconc}}$$ $\quad f_{DWT} = 0.79$ ksi

Calculate f(LL)

$f_{LL} := \frac{M_{ll\_pos}}{Sx_{short2noconc}}$ $f_{LL} = 14.227$ ksi $f_{LLT} := \frac{M_{ll\_pos}}{Sx_{short1noconc}}$ $f_{LLT} = 2.198$ ksi

Check Service II limit state for bottom (tension) flange

$f_F := f_{DC} + f_{DC2} + f_{DW} + (1.3 \cdot f_{LL})$ $f_F = 42.21$ ksi

$f_{allowable} := 0.95 \cdot Fy$ $f_{allowable} = 47.5$ ksi

$Ratio_{serviceII2} := \frac{f_F}{f_{allowable}}$ $Ratio_{serviceII2} = 0.89$ < 1 therefore section is okay for service II

Check Service II limit state for top (compression) flange

$f_{FT} := f_{DCT} + f_{DC2T} + f_{DWT} + (1.3 \cdot f_{LLT})$ $f_{FT} = 21.5$

$Ratio_{serviceII1} := \frac{f_{FT}}{f_{allowable}}$ $Ratio_{serviceII1} = 0.453$ < 1 therefore section is okay for service II

$$Ratio_{serviceII} := \begin{cases} Ratio_{serviceII1} & \text{if } Ratio_{serviceII1} > Ratio_{serviceII2} \\ Ratio_{serviceII2} & \text{otherwise} \end{cases}$$

$Ratio_{serviceII} = 0.89$ < 1 therefore section is okay for service II

CHECK STRENGTH I LIMIT STATE FOR FLEXURE

Since $Dcp_{com\_pos\_sb} = 0$ The depth of the web in compression at the plastic moment is zero.

The web-slenderness requirement is satisfied. Therefore, the section qualifies as a compact section. The ductility check is not required because the plastic moment is determined through strain compatibility.

Check the section capacity against the upper limit on section capacity for continuous spans

Calculate My according to AASHTO 2004

Bottom (tension) flange

$M_{AD} := Sx_{short2noconc} \cdot \left[Fy - \left[1.25 \cdot \left(f_{DC} + f_{DC2}\right) + 1.5 \cdot f_{DW}\right]\right]$ $M_{AD} = 27025.4$ k-in

$My\_bf\_st := M_{AD} + 1.25\left(M_{dc1\_pos} + M_{dc2\_pos}\right) + 1.5 \cdot M_{dw\_pos}$ $My\_bf\_st = 55958.9$ k-in

Top (compression) flange

$M_{ADT} := Sx_{short1noconc} \cdot \left[Fy - \left[1.25 \cdot \left(f_{DCT} + f_{DC2T}\right) + 1.5 \cdot f_{DWT}\right]\right]$ $M_{ADT} = 231840.8$ k-in

$My\_tf\_st := M_{ADT} + 1.25\left(M_{dc1\_pos} + M_{dc2\_pos}\right) + 1.5 \cdot M_{dw\_pos}$ $My\_tf\_st = 260774.3$ k-in

Upper limit on section capacity according to AASHTO 2004

$Mn\_pos\_max := 1.3 \cdot Rh \cdot \min(My\_bf\_st, My\_tf\_st)$ $Mn\_pos\_max = 72746.6$ k-in

Section capacity.

$Mn := \min\left(Mn\_pos\_max, Mp_{com\_pos\_sb}\right)$ $Mn = 72746.6$ k-in

Check Strength I limit state for flexure

$RatioI_{flexure} := \frac{MstI\_pos}{\Phi \cdot Mn}$ $RatioI_{flexure} = 0.8174$ < 1 therefore ok

CHECK STRENGTH I LIMIT STATE FOR SHEAR

Nominal shear resistance (transverse stiffener spacing is 1/4 of span so essentially unstiffened web (k=5 and no tension field))

$A_v := \frac{Dweb}{Tweb}$ $\quad B_v := 1.12 \cdot \sqrt{\frac{E \cdot 5}{Fy}}$ $\quad C_v := 1.40 \cdot \sqrt{\frac{E \cdot 5}{Fy}}$

$A_v = 53$ $\quad B_v = 60.314$ $\quad C_v = 75.392$

$$C := \begin{cases} 1.0 \text{ if } A_v < B_v & \text{: plastic} \\ \frac{1.12}{A_v} \cdot \sqrt{\frac{E \cdot 5}{Fy}} \text{ if } B_v \le A_v \le C_v & \text{: inelastic} \\ \frac{1.57}{A_v^2} \cdot \frac{E \cdot 5}{Fy} \text{ if } A_v > C_v & \text{: elastic} \end{cases}$$

$V_p := 0.58 \cdot Fy \cdot Dweb \cdot Tweb$ $\quad V_p = 384.25 kip$ $\quad$ plastic shear resistance of web

$C = 1$ $\quad V_n := C \cdot V_p$ $\quad V_n = 384.25 kip$ $\quad$ nominal shear resistance of an unstiffened web

Check shear resistance

$Ratio_{shear} := \frac{VstI\_pos}{\Phi \cdot V_n}$ $\quad Ratio_{shear} = 0.66$ $\quad < 1$ therefore ok

CHECK FATIGUE AND FRACTURE LIMIT STATE FOR FLEXURE

Load Induced Fatigue

ADTT = number of trucks per day in one direction averaged over the design life
a = fraction of trucks in traffic for a rural class of highway designation
p = fraction of truck traffic in a single lane
ADT = average daily traffic including all vehicles
ADTT(singlelane) = the number of trucks per day in a single-lane averaged over the design life

$ADT := 20000$ vehicles per lane per day $a := 0.15$ fraction of trucks in traffic $p := 1$ for 1 lane available to trucks

$ADTT := ADT \cdot a$ $ADTT = 3000$ trucks/day

$ADTT_{singlelane} := ADTT \cdot p$ $ADTT_{singlelane} = 3000$ trucks/day

Check the base metal at stiffener/connection plate weld. Assume transverse stiffener is located at the maximum moment section and is welded directly to the tension flange

Δf = the force effect, live load stress range due to the passage of the fatigue load
ΔF = the nominal fatigue resistance

$$\Delta f := \frac{Mfat\_pos}{Ix_{short}} \cdot \left(Dtotal - ENA_{short} - Tbf\right) \qquad \Delta f = 4.4 \text{ ksi}$$

Category C '

Condition 1:

$$n := \begin{cases} 1.0 & \text{if } L > 480 \\ 2.0 & \text{if } L \le 480 \end{cases}$$ n = number of stress range cycles per truck passage

$N := 365 \cdot 75 \cdot n \cdot ADTT_{singlelane}$ $N = 8.213 \times 10^{7}$

$A := 44 \cdot 10^{8}$ **For Fatigue Category C' :**

$$\Delta Fn\_1 := \left(\frac{A}{N}\right)^{\frac{1}{3}} \qquad \Delta Fn\_1 = 3.77$$

Condition 2:

$\Delta F\_TH := 12$

$$\Delta Fn\_2 := \frac{1}{2} \cdot \Delta F\_TH \qquad \Delta Fn\_2 = 6$$

$$\Delta Fn := \begin{cases} \Delta Fn\_1 & \text{if } \Delta Fn\_1 \ge \Delta Fn\_2 \\ \Delta Fn\_2 & \text{otherwise} \end{cases} \qquad \Delta Fn = 6$$

$\Delta f = 4.4$ $<$ $\Delta Fn = 6$ **O.K.**

$$Ratio_{fatigue\_stiffener} := \frac{\Delta f}{\Delta Fn} \qquad Ratio_{fatigue\_stiffener} = 0.733 < 1$$ therefore ok

CHECK FATIGUE AND FRACTURE LIMIT STATE FOR WEBS (SHEAR)

The web is quite stocky and the stiffeners are widely spaced, so the web was designed for the Strength I limit state as unstiffened. Calculations given below for the Strength I limit state show that the web shear capacity (Vn = Vcr) equals Vp (i.e., C = 1.0) when the web is treated as unstiffened (AASHTO LRFD Article 6.10.9.2). Tension field action is not included (or needed). Also, note that the web thickness and depth are constant, so the calculations apply to all regions of the web. As shown later, the shear capacity exceeds the shear demand for the Strength I load combination (VstI_pos) and therefore the requirement of AASHTO LRFD Article 6.10.5.3 (Vu = Vfat_pos < Vcr) is also satisfied. (Strictly speaking, since the web is treated as unstiffened, AASHTO LRFD Article 6.10.3.3 does not apply). If Vn=Vcr were less than Vp and tension field action was included in calculating Vn = Vcr for the Strength I limit state, then a separate calculation of Vcr according to Article 6.10.9.3.3 would be needed and this Vcr would be checked against Vfat_pos here.

SUMMARYOF LIMIT STATE CHECKS

Strength I

$RatioI_{flexure} = 0.8174$

$Ratio_{shear} = 0.6597$

Construction

$Ratio_{web\_stocky} = 0.3456$

$Ratio_{web\_dist} = 0.5778$

$Ratio_{ltbresistance1} = 0.7932$

Service II

$Ratio_{serviceII} = 0.8887$

Fatigue

$Ratio_{fatigue\_stiffener} = 0.7334$

Tube Requirement

$Ratio_{tubethickness} = 0.9933$

Appendix B. Pier Section and Negative Moment Region Preliminary Design

Part 1. Negative Moment Region Preliminary Design

I. Cross Section Information (yellow highlight indicates input data)

Yield strength: $Fy := 50$ ksi

Tensile strength: $Fu := 65$ ksi

Tube horizontal plate thickness: $Tt1 := \frac{3}{8}$ in

Tube vertical plate thickness: $Tt2 := \frac{3}{8}$ in

Tube horizontal plate width: $Bt1 := 16$ in

Tube vertical plate width: $Bt2 := 7.25$ in

Tube depth: $Dtube := Bt2 + 2 \cdot Tt2$ $Dtube = 8.00$ in

Bottom flange thickness: $Tbf := 1.5$ in

Bottom flange width: $Bbf := 18$ in

Web thickness: $Tweb := 0.5$ in

Web depth: $Dweb := 26.5$ in

Girder depth: $Dgird := Dtube + Dweb + Tbf$ $Dgird = 36.00$ in

Deck Thickness: $Tslab := 8$ in

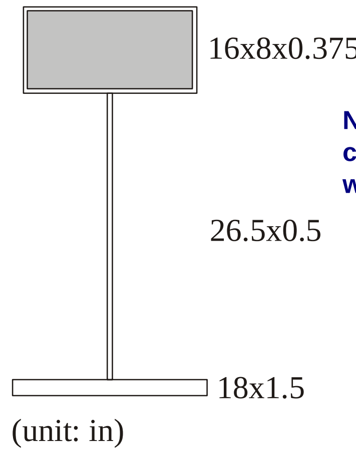
Haunch Thickness: $Thaunch := 3$ in

Deck Width: $W := 376.5$ in

Span Length: $L := 1200$ in

Girder Spacing: $s := 101.5$ in

Overhang (from girder centerline): $se := 36$ in



Note that tube has 5 in. cut out on each side wall for splice access.

Post-tensioning in Deck (per girder)

Number of Strands: $Nstr := 30$

Area of Strands: $Astr := Nstr \cdot 0.217$ in^2

$Astr = 6.51$ in^2

Strand Strength:

$Fu\_str := 270$ ksi

$Fy\_str := 0.9 \cdot Fu\_str$

$Fy\_str = 243$ ksi

Deck Concrete Strength: $fcprime := 4$ ksi

Short-term Modular Ratio: $n_s := 8$

Number of Girders: $ng := 4$

Girder Areas

$Abf := Bbf \cdot Tbf$ $Abf = 27$ in^2

$Atube := 2 \cdot Tt1 \cdot Bt1 + 2 \cdot Tt2 \cdot Bt2 - 2 \cdot 0.375 \cdot 5$ $Atube = 13.69$ in^2 **Includes cut-out**

$Aw := Dweb \cdot Tweb$ $Aw = 13.25$ in^2

$Agird := Aw + Atube + Abf$ $Agird = 53.94$ in^2

EFFECTIVE WIDTH OF SLAB (INTERIOR GIRDER)

$beff1 := \frac{L}{4}$ $beff1 = 300.00$ $beff2 := s$ $beff2 = 101.50$ $beff3 := 12 \cdot Tslab + \frac{Bt1}{2}$ $beff3 = 104.00$

The smallest beff governs

$$Beffi := \begin{cases} beff1 & \text{if } beff1 \le beff2 \wedge beff1 \le beff3 \\ beff2 & \text{if } beff2 \le beff1 \wedge beff2 \le beff3 \\ beff3 & \text{otherwise} \end{cases}$$

$Beffi = 101.50$ in

EFFECTIVE WIDTH OF SLAB (EXTERIOR GIRDER)

$beff4 := \left(\frac{s}{2}\right) + se$ $beff4 = 86.75$

The smallest beff governs

$$Beffe := \begin{cases} beff1 & \text{if } beff1 \le beff2 \wedge beff1 \le beff3 \\ beff4 & \text{if } beff4 \le beff1 \wedge beff4 \le beff3 \\ beff3 & \text{otherwise} \end{cases}$$

$Beffe = 86.75$ in

SELECT EFFECTIVE WIDTH OF SLAB (use Beffi for interior girder, Beffe for exterior girder, or minimum)

$Beff := \min(Beffe, Beffi)$ $Beff = 86.75$ Note: here the minimum is used, which is for an exterior girder.

Deck Transformed Cross Section Area:

$Ad\_tr := \frac{Beff \cdot Tslab - Astr}{n_s} + Astr$ $Ad\_tr = 92.45$ in^2

SECTION PROPERTIES

Calculate the elastic neutral axis for steel girder section with post-tensioning steel in the deck. The concrete in the tube is neglected since it is not present at the pier centerline section where the splice is made. The tube cross section includes the cut out for the splice. The reference line is taken at the bottom of the bottom flange.

$A_{gird\_pt} := Agird + Astr$ $A_{gird\_pt} = 60.45$ in^2

$$ENA_{gird\_pt} := \frac{Astr \cdot \left(Dgird + Thaunch + \frac{Tslab}{2}\right) + Atube \cdot \left(Tbf + Dweb + \frac{Dtube}{2}\right) + Aw \cdot \left(Tbf + \frac{Dweb}{2}\right) + Abf \cdot \left(\frac{Tbf}{2}\right)}{A_{gird\_pt}}$$

$ENA_{gird\_pt} = 15.45$ in from the bottom of the girder

Calculate the corresponding moment of inertia for steel girder section with post-tensioning steel.

$$Ix1 := \frac{1}{12} \cdot Bbf \cdot Tbf^3 + Abf \cdot \left(\frac{Tbf}{2} - ENA_{gird\_pt}\right)^2$$

$$Ix2 := \frac{1}{12} \cdot Tweb \cdot Dweb^3 + Aw \cdot \left(Tbf + \frac{Dweb}{2} - ENA_{gird\_pt}\right)^2$$

$$Ix3 := \left[\frac{1}{12}\cdot Bt1\cdot(Bt2 + 2\cdot Tt1)^3 - \frac{1}{12}\cdot Bt1\cdot Bt2^3\right] + Atube\cdot\left(Tbf + Dweb + \frac{Dtube}{2} - ENA_{gird\_pt}\right)^2$$

$$Ix4 := Astr\cdot\left(Dgird + Thaunch + \frac{Tslab}{2} - ENA_{gird\_pt}\right)^2$$

$Ix_{gird\_pt} := Ix1 + Ix2 + Ix3 + Ix4$ $\qquad Ix_{gird\_pt} = 15486$ in^4

Calculate the corresponding section moduli for steel girder section with post-tensioning steel.

Sx to bottom of bottom flange: $Sx_{gird\_pt\_bf\_bot} := \dfrac{-Ix_{gird\_pt}}{-ENA_{gird\_pt}}$ $\qquad Sx_{gird\_pt\_bf\_bot} = 1003$ in^3

Sx to middle of bottom flange: $Sx_{gird\_pt\_bf\_mid} := \dfrac{-Ix_{gird\_pt}}{\frac{Tbf}{2} - ENA_{gird\_pt}}$ $\qquad Sx_{gird\_pt\_bf\_mid} = 1054$ in^3

Sx to middle of top flange: $Sx_{gird\_pt\_tf\_mid} := \dfrac{-Ix_{gird\_pt}}{Tbf + Dweb + \frac{Dtube}{2} - ENA_{gird\_pt}}$ $\qquad Sx_{gird\_pt\_tf\_mid} = -935$ in^3

Sx to top of top flange: $Sx_{gird\_pt\_tf\_top} := \dfrac{-Ix_{gird\_pt}}{Dgird - ENA_{gird\_pt}}$ $\qquad Sx_{gird\_pt\_tf\_top} = -753$ in^3

Sx to post-tensioning steel: $Sx_{gird\_pt\_pt} := \dfrac{-Ix_{gird\_pt}}{Dgird + Thaunch + \frac{Tslab}{2} - ENA_{gird\_pt}}$ $\qquad Sx_{gird\_pt\_pt} = -562$ in^3

Sx to bottom of deck: $Sx_{gird\_pt\_deck\_bot} := \dfrac{-Ix_{gird\_pt}}{Dgird + Thaunch - ENA_{gird\_pt}}$ $\qquad Sx_{gird\_pt\_deck\_bot} = -657$ in^3

Calculate the elastic neutral axis for steel girder with the composite deck using the short-term loading modular ratio. The concrete in the tube is neglected since it is not present at the pier centerline section where the splice is made. The tube cross section includes the cut out for the splice. The reference line is taken at the bottom of the bottom flange.

$A_{short} := Agird + Ad\_tr$ $\qquad A_{short} = 146.38$ in^2

$$ENA_{short} := \frac{Ad\_tr\cdot\left(Dgird + Thaunch + \frac{Tslab}{2}\right) + Atube\cdot\left(Tbf + Dweb + \frac{Dtube}{2}\right) + Aw\cdot\left(Tbf + \frac{Dweb}{2}\right) + Abf\cdot\left(\frac{Tbf}{2}\right)}{A_{short}}$$

$ENA_{short} = 31.62$ in $\qquad$ from the bottom of the girder

Calculate the corresponding moment of inertia for steel girder with short-term composite deck.

$$Ix1 := \frac{1}{12} \cdot Bbf \cdot Tbf^3 + Abf \cdot \left(\frac{Tbf}{2} - ENA_{short}\right)^2$$

$$Ix2 := \frac{1}{12} \cdot Tweb \cdot Dweb^3 + Aw \cdot \left(Tbf + \frac{Dweb}{2} - ENA_{short}\right)^2$$

$$Ix3 := \left[\frac{1}{12} \cdot Bt1 \cdot (Bt2 + 2 \cdot Tt1)^3 - \frac{1}{12} \cdot Bt1 \cdot Bt2^3\right] + Atube \cdot \left(Tbf + Dweb + \frac{Dtube}{2} - ENA_{short}\right)^2$$

$$Ix4 := \frac{1}{12}\left(\frac{Beff}{n_s}\right) \cdot Tslab^3 + Ad\_tr \cdot \left(Dgird + Thaunch + \frac{Tslab}{2} - ENA_{short}\right)^2$$

$Ix_{short} := Ix1 + Ix2 + Ix3 + Ix4$ $\qquad Ix_{short} = 42893$ in^4

Calculate the corresponding section moduli for steel girder with short-term composite deck.

Sx to bottom of bottom flange: $Sx_{short\_bf\_bot} := \frac{-Ix_{short}}{-ENA_{short}}$ $\qquad Sx_{short\_bf\_bot} = 1356$ in^3

Sx to middle of bottom flange: $Sx_{short\_bf\_mid} := \frac{-Ix_{short}}{\frac{Tbf}{2} - ENA_{short}}$ $\qquad Sx_{short\_bf\_mid} = 1389$ in^3

Sx to middle of top flange: $Sx_{short\_tf\_mid} := \frac{-Ix_{short}}{Tbf + Dweb + \frac{Dtube}{2} - ENA_{short}}$

$Sx_{short\_tf\_mid} = -113328$ in^3

Sx to top of top flange: $Sx_{short\_tf\_top} := \frac{-Ix_{short}}{Tbf + Dweb + Dtube - ENA_{short}}$

$Sx_{short\_tf\_top} = -9796$ in^3

Sx to top of deck: $Sx_{short\_deck\_top} := \frac{-Ix_{short}}{Tbf + Dweb + Dtube + Thaunch + Tslab - ENA_{short}}$

$Sx_{short\_deck\_top} = -2789$ in^3

II. Pier Section Design Loads (yellow highlight indicates input data)

Girder Moment at Pier Section:

DC1: $M_{dc1} := 0$ kip-in

DC2: $M_{dc2} := -4875$ kip-in

DW: $M_{dw} := -3150$ kip-in

LL(positive): $M_{pll} := 0$ kip-in

LL(negative): $M_{nll} := -19715$ kip-in

Fatigue(positive): $M_{pfll} := 0$ kip-in

Fatigue(negative): $M_{nfll} := -4843$ kip-in

Girder Shear in Negative Moment Region:

DC1: $V_{dc1} := 61$ kip

DC2: $V_{dc2} := 20.3$ kip

DW: $V_{dw} := 13.1$ kip

LL: $V_{ll} := 109.7$ kip

Fatigue: $V_{fll} := 42.6$ kip

Compute Flange Stresses at Top of Top Flange, Bottom of Bottom Flange, and Top of Deck

No positive moment at pier centerline section so consider only Dead Load + Negative Live Load

DC2:

$$f_{DC2\_short\_tf\_top} := \frac{M_{dc2}}{Sx_{short\_tf\_top}} \qquad f_{DC2\_short\_tf\_top} = 0.50 \text{ ksi}$$ **Tension**

$$f_{DC2\_short\_bf\_bot} := \frac{M_{dc2}}{Sx_{short\_bf\_bot}} \qquad f_{DC2\_short\_bf\_bot} = -3.59 \text{ ksi}$$ **Compression**

$$f_{DC2\_short\_deck\_top} := \frac{M_{dc2}}{Sx_{short\_deck\_top} \cdot n_s} \qquad f_{DC2\_short\_deck\_top} = 0.22 \text{ ksi}$$ **Tension**

DW:

$$f_{DW\_short\_tf\_top} := \frac{M_{dw}}{Sx_{short\_tf\_top}} \qquad f_{DW\_short\_tf\_top} = 0.32 \text{ ksi}$$ **Tension**

$$f_{DW\_short\_bf\_bot} := \frac{M_{dw}}{Sx_{short\_bf\_bot}} \qquad f_{DW\_short\_bf\_bot} = -2.32 \text{ ksi}$$ **Compression**

$$f_{DW\_short\_deck\_top} := \frac{M_{dw}}{Sx_{short\_deck\_top} \cdot n_s} \qquad f_{DW\_short\_deck\_top} = 0.14 \text{ ksi}$$ **Tension**

LL:

$$f_{NLL\_short\_tf\_top} := \frac{M_{nll}}{Sx_{short\_tf\_top}} \qquad f_{NLL\_short\_tf\_top} = 2.01 \text{ ksi}$$ **Tension**

$$f_{NLL\_short\_bf\_bot} := \frac{M_{nll}}{Sx_{short\_bf\_bot}} \qquad f_{NLL\_short\_bf\_bot} = -14.53 \text{ ksi}$$ **Compression**

$$f_{NLL\_short\_deck\_top} := \frac{M_{nll}}{Sx_{short\_deck\_top} \cdot n_s} \qquad f_{NLL\_short\_deck\_top} = 0.88 \text{ ksi}$$ **Tension**

Fatigue

$$f_{NFLL\_short\_tf\_top} := \frac{M_{nfll}}{Sx_{short\_tf\_top}}$$ $f_{NFLL\_short\_tf\_top} = 0.49$ ksi **Tension**

$$f_{NFLL\_short\_bf\_bot} := \frac{M_{nfll}}{Sx_{short\_bf\_bot}}$$ $f_{NFLL\_short\_bf\_bot} = -3.57$ ksi **Compression**

Compute Force Effects for Strength I, Service II. and Fatigue Limit State Load Combinations

Strength I Limit State: Dead Load + Negative Live Load Effect

$M_{NLL\_st} := (1.25 \cdot M_{dc2} + 1.5 \cdot M_{dw} + 1.75 \cdot M_{nll})$ $M_{NLL\_st} = -45320$ kip-in **Negative**

$V_{st} := [1.25 \cdot (V_{dc1} + V_{dc2}) + 1.5 \cdot V_{dw} + 1.75 \cdot V_{ll}]$ $V_{st} = 313.25$ kip

Service II Limit State: Dead Load + Negative Live Load

$M_{NLL\_sv} := (1.0 \cdot M_{dc2} + 1.0 \cdot M_{dw} + 1.3 \cdot M_{nll})$ $M_{NLL\_sv} = -33655$ kip-in **Negative**

$f_{tf\_top\_sv} := (1.0 \cdot f_{DC2\_short\_tf\_top} + 1.0 \cdot f_{DW\_short\_tf\_top} + 1.3 \cdot f_{NLL\_short\_tf\_top})$

$f_{tf\_top\_sv} = 3.44$ ksi **Tension**

$f_{bf\_bot\_sv} := (1.0 \cdot f_{DC2\_short\_bf\_bot} + 1.0 \cdot f_{DW\_short\_bf\_bot} + 1.3 \cdot f_{NLL\_short\_bf\_bot})$

$f_{bf\_bot\_sv} = -24.81$ ksi **Compression**

$f_{deck\_top\_sv} := (1.0 \cdot f_{DC2\_short\_deck\_top} + 1.0 \cdot f_{DW\_short\_deck\_top} + 1.3 \cdot f_{NLL\_short\_deck\_top})$

$f_{deck\_top\_sv} = 1.51$ ksi **Tension**

Fatigue Limit State: Negative Live Load

$f_{tf\_top\_fat} := (0.75 \cdot f_{NFLL\_short\_tf\_top})$ $f_{tf\_top\_fat} = 0.37$ ksi **Tension**

$f_{bf\_bot\_fat} := (0.75 \cdot f_{NFLL\_short\_bf\_bot})$ $f_{bf\_bot\_fat} = -2.68$ ksi **Compression**

$V_{fat} := V_{dc1} + V_{dc2} + V_{dw} + 2(0.75 \cdot V_{fll})$ $V_{fat} = 158.30$ kip

III. Pier Section Design Checks

The concrete in the tube is neglected since it is not present at the pier centerline section where the splice is made. The tube cross section includes the cut out for the splice.

Strength I Limit State: Negative Flexure (First determine the sequence of events under flexure)

Plastic moment capacity of section without concrete assuming the plastic neutral axis is in web. The reference axis is the bottom of the bottom flange.

$Comp(x) := -Abf \cdot Fy - x \cdot Tweb \cdot Fy$

$Tens(x) := Astr \cdot Fy\_str + Atube \cdot Fy + (Dweb - x) \cdot Tweb \cdot Fy$

$Func(x) := Tens(x) + Comp(x)$

$x := 1$ $\quad Dcp := root(Func(x), x)$

$PNA := Dcp + Tbf$

$Dtp := Dweb - Dcp$

$Dcp = 31.58$ in

$PNA = 33.08$ in

$Dtp = -5.08$ in

PNA is not in web

$M1 := Fy\_str \cdot Astr \cdot \left(Dgird + Thaunch + \frac{Tslab}{2}\right)$	strands	$M1 = 68023$	kip-in
$M2 := Fy \cdot Atube \cdot \left[Tbf + Dweb + \left(\frac{Dtube}{2}\right)\right]$	tube	$M2 = 21900$	kip-in
$M3 := Fy \cdot \left[[Tweb \cdot (Dtp)] \cdot \left(\frac{Dtp}{2} + PNA\right)\right]$	web in tension	$M3 = -3875$	kip-in
$M4 := -Fy \cdot [Tweb \cdot (Dcp)] \cdot \left(\frac{Dcp}{2} + Tbf\right)$	web in compression	$M4 = -13647$	kip-in
$M5 := -Fy \cdot Abf \cdot \left(\frac{Tbf}{2}\right)$	bottom flange	$M5 = -1013$	kip-in
$Mp\_web := M1 + M2 + M3 + M4 + M5$		$Mp\_web = 71388$	kip-in

Plastic moment capacity of section without concrete assuming the plastic neutral axis is in middle of tube bottom wall. Because of cut-out, treat area of tube as concentrated at top wall and bottom wall. The reference axis is the bottom of the bottom flange.

$Comp2 := -Abf \cdot Fy - Dweb \cdot Tweb \cdot Fy - \frac{Atube}{4} \cdot Fy$ $\quad Comp2 = -2184$ kip

$Tens2 := Astr \cdot Fy\_str + \left(\frac{3Atube}{4}\right) \cdot Fy$ $\quad Tens2 = 2095$ kip

OK

$Dcp := Dweb$

$PNA := Dcp + Tbf$

$Dtp := Dweb - Dcp$

$Dcp = 26.50$ in

$PNA = 28.00$ in

$Dtp = 0.00$ in

$M1 := Fy\_str \cdot Astr \cdot \left(Dgird + Thaunch + \frac{Tslab}{2}\right)$	strands	$M1 = 68023$	kip-in
$M2 := Fy \cdot \left(\frac{Atube}{2}\right) \cdot (Tbf + Dweb + Dtube)$	top wall of tube	$M2 = 12319$	kip-in

$M3 := 0$ bottom wall of tube $M3 = 0$ kip-in

$M4 := -Fy \cdot [Tweb \cdot (Dweb)] \cdot \left(\frac{Dweb}{2} + Tbf\right)$ web $M4 = -9772$ kip-in

$M5 := -Fy \cdot Abf \cdot \left(\frac{Tbf}{2}\right)$ bottom flange $M5 = -1013$ kip-in

$Mp\_tube := -(M1 + M2 + M3 + M4 + M5)$ $Mp\_tube = -69557$ kip-in

Determine conditions when deck decompresses at top surface. Use transformed section based on short-term loading concrete in deck. Prestress in deck is based on an estimate of 15% time dependent prestress losses and an initial prestress of 70% of Fu of strands.

$0.7 \cdot Fu\_str = 189$ ksi

Moment when deck decompresses at top surface.

$f\_str\_losses := 0.7 \cdot Fu\_str \cdot (1 - 0.15)$ $f\_str\_losses = 161$ ksi

$bridge\_deck\_width := W$ $bridge\_deck\_width = 376.50$ in

$f_{deck\_prestress} := \frac{4 \cdot Astr \cdot f\_str\_losses}{Tslab \cdot (bridge\_deck\_width)}$ $f_{deck\_prestress} = 1.389$ ksi

$M_{deck\_top\_decomp} := n_s \cdot Sx_{short\_deck\_top} \cdot f_{deck\_prestress}$ $M_{deck\_top\_decomp} = -30990$ kip-in

Stresses when deck decompresses at top surface.

$f_{deck\_top\_decomp\_bf\_bot} := \left(\frac{M_{deck\_top\_decomp}}{Sx_{short\_bf\_bot}}\right)$ $f_{deck\_top\_decomp\_bf\_bot} = -22.85$

$f_{deck\_top\_decomp\_tf\_top} := \left(\frac{M_{deck\_top\_decomp}}{Sx_{short\_tf\_top}}\right)$ $f_{deck\_top\_decomp\_tf\_top} = 3.16$

Determine conditions when deck fully decompresses. These conditions control the stresses that develop on the section without the deck (the steel girder and post-tensioning strands), under moments that exceed the moment causing deck decompression.

Moment when deck fully decompresses.

$M1_{full\_decomp} := -f\_str\_losses \cdot Astr \cdot \left(Dgird + Thaunch + \frac{Tslab}{2} - ENA_{gird\_pt}\right)$ $M1_{full\_decomp} = -28818$ kip-in

$M2_{full\_decomp} := f_{deck\_prestress} \cdot n_s \cdot Sx_{gird\_pt\_deck\_bot}$ $M2_{full\_decomp} = -7305$ kip-in

$M_{full\_decomp} := M1_{full\_decomp} + M2_{full\_decomp}$ $M_{full\_decomp} = -36123$ kip-in

Stresses when deck fully decompresses.

$$f_{full\_decomp\_bf\_bot} := \frac{-f\_str\_losses \cdot Astr}{Agird} + \left(\frac{M2_{full\_decomp}}{Sx_{gird\_pt\_bf\_bot}}\right) \qquad f_{full\_decomp\_bf\_bot} = -26.68 \text{ ksi}$$

$$f_{full\_decomp\_bf\_mid} := \frac{-f\_str\_losses \cdot Astr}{Agird} + \left(\frac{M2_{full\_decomp}}{Sx_{gird\_pt\_bf\_mid}}\right) \qquad f_{full\_decomp\_bf\_mid} = -26.32 \text{ ksi}$$

$$f_{full\_decomp\_tf\_top} := \frac{-f\_str\_losses \cdot Astr}{Agird} + \left(\frac{M2_{full\_decomp}}{Sx_{gird\_pt\_tf\_top}}\right) \qquad f_{full\_decomp\_tf\_top} = -9.69 \text{ ksi}$$

$$f_{full\_decomp\_tf\_mid} := \frac{-f\_str\_losses \cdot Astr}{Agird} + \left(\frac{M2_{full\_decomp}}{Sx_{gird\_pt\_tf\_mid}}\right) \qquad f_{full\_decomp\_tf\_mid} = -11.58 \text{ ksi}$$

$$f_{full\_decomp\_pt} := f\_str\_losses + \left(\frac{M2_{full\_decomp}}{Sx_{gird\_pt\_pt}}\right) \qquad f_{full\_decomp\_pt} = 173.65 \text{ ksi}$$

Determine moments which cause yielding. Determine the additional moment needed to cause yield (using section without concrete) and add to moment at time deck fully decompressed

$$M_{yield\_bf} := Sx_{gird\_pt\_bf\_bot} \cdot \left(-Fy - f_{full\_decomp\_bf\_bot}\right) + M_{full\_decomp} \qquad M_{yield\_bf} = -59509 \text{ kip-in}$$

$$M_{yield\_tf} := Sx_{gird\_pt\_tf\_top} \cdot \left(Fy - f_{full\_decomp\_tf\_top}\right) + M_{full\_decomp} \qquad M_{yield\_tf} = -81096 \text{ kip-in}$$

$$M_{yield\_pt} := Sx_{gird\_pt\_pt} \cdot \left(Fy\_str - f_{full\_decomp\_pt}\right) + M_{full\_decomp} \qquad M_{yield\_pt} = -75099 \text{ kip-in}$$

Strength I Limit State: Negative Flexure (sequence of events)

- Deck decompresses at top surface (joint would open if at pier centerline) — $M_{deck\_top\_decomp} = -30990$ kip-in
- Bottom flange yields — $M_{yield\_bf} = -59509$ kip-in
- Section reaches plastic moment — $Mp\_tube = -69557$ kip-in
- Post-tensioning steel yields based on elastic section (inaccurate) — $M_{yield\_pt} = -75099$ kip-in
- Top flange yields based on elastic section (inaccurate) — $M_{yield\_tf} = -81096$ kip-in
- Factored design Strength I design moment — $M_{NLL\_st} = -45320$ kip-in

Strength I Limit State: Negative Flexure (section capacity using Appendix A of AASHTO)

$Dc := ENA_{gird\_pt} - Tbf$ $Dc = 13.95$ in $Dcp = 26.50$ in $\lambda w := 2\frac{Dc}{Tweb}$ $\lambda rw := 5.7\sqrt{\frac{29000}{Fy}}$

$\lambda w = 55.78$ < $\lambda rw = 137.27$ OK, Appendix A can be used

Consider the web slenderness and calculate the web plastification factors

$Mp := |Mp\_tube|$ $Mp = 69557$ kip-in

$Myt := |M_{yield\_tf}|$ $Myt = 81096$ kip-in

$Myc := |M_{yield\_bf}|$ $Myc = 59509$ kip-in $My := \min(Myt, Myc)$ $My = 59509$ kip-in

$\lambda pw1 := \frac{\sqrt{\frac{29000}{Fy}}}{0.54\frac{Mp}{My} - 0.09}$ $\lambda pw1 = 44.50$ $\lambda pw2 := \lambda rw\cdot\left(\frac{Dc}{Dcp}\right)$ $\lambda pw2 = 72.24$

$\lambda pw := \min(\lambda pw1, \lambda pw2)$

$\frac{2Dcp}{Tweb} = 106.00$ > $\lambda pw = 44.50$ Not compact section

$\lambda w = 55.78$ < $\lambda rw = 137.27$ Noncompact web section

$\lambda pw3 := \lambda pw\cdot\left(\frac{Dc}{Dcp}\right)$ $\lambda pw3 = 23.42$

$Rpc := \left[1 - \left(1 - \frac{Myc}{Mp}\right)\frac{\lambda w - \lambda pw3}{\lambda rw - \lambda pw3}\right]\frac{Mp}{Myc}$ $Rpc = 1.12$ < $\frac{Mp}{Myc} = 1.17$ OK

$Rpt := \left[1 - \left(1 - \frac{Myt}{Mp}\right)\frac{\lambda w - \lambda pw3}{\lambda rw - \lambda pw3}\right]\frac{Mp}{Myt}$ $Rpt = 0.90$ < $\frac{Mp}{Myt} = 0.86$ NG

$Rpt := \frac{Mp}{Myt}$

Nominal resistance for tension flange yielding

$Mnt := Rpt\cdot Myt$ $Mnt = 69557$ kip-in

Consider the compression flange slenderness

$\lambda f := \frac{Bbf}{2\cdot Tbf}$ $\lambda pf := 0.38\cdot\sqrt{\frac{29000}{Fy}}$

$\lambda f = 6.00$ < $\lambda pf = 9.15$ Compact flange

$Mnc\_cf := Rpc\cdot Myc$ $Mnc\_cf = 66701$ kip-in

Consider lateral-torsional buckling

$$r_t := \frac{Bbf}{\sqrt{12\left[1 + \left(\frac{1}{3}\right)\cdot\frac{Dc\cdot Tweb}{Bbf\cdot Tbf}\right]}} \qquad r_t = 4.99$$

$$K_T := \frac{Dweb\cdot Tweb^3}{3} + \frac{Bbf\cdot Tbf^3}{3} + \frac{2\cdot Tt2\cdot Tt1\cdot(Bt1 - Tt2)^2\cdot(Bt2 + Tt1)^2}{(Bt1 - Tt2)\cdot Tt2 + (Bt2 + Tt1)\cdot Tt1}$$

$J := K_T$ $\qquad J = 479$ in^4

$Sxc := Sx_{gird\_pt\_bf\_bot}$ $\qquad Sxc = 1003$ in^3

$Sxt := \left|Sx_{gird\_pt\_tf\_top}\right|$ $\qquad Sxt = 753$ in^3

$h := Dweb + \frac{Dtube}{2} + \frac{Tbf}{2}$ $\qquad h = 31.25$ in

$Fyr1 := 0.7Fy \qquad Fyr2 := Fy\cdot\left(\frac{Sxt}{Sxc}\right)$

$Fyr := \min(Fyr1, Fyr2)$ $\qquad Fyr = 35.00$ ksi

$Lp := r_t\cdot\sqrt{\frac{29000}{Fy}}$ $\qquad Lp = 120$ in

$$Lr := 1.95\cdot r_t\cdot\left(\frac{29000}{Fyr}\right)\sqrt{\frac{J}{Sxc\cdot h}}\cdot\sqrt{1 + \sqrt{1 + 6.76\cdot\left(\frac{Fyr}{29000}\cdot\frac{Sxc\cdot h}{J}\right)^2}} \qquad Lr = 1416 \text{ in}$$

$Lb := \frac{100\cdot 12}{2}$ Span is 100 ft. Single cross-frame at midspan $\qquad Lb = 600$ in

Cb calculation:

M0 will be midspan moment

$M_{dc1\_pos\_midspan} := 17500 \quad M_{dc2\_pos\_midspan} := 2700 \quad M_{dw\_pos\_midspan} := 1700 \quad M_{ll\_neg\_midspan} := -7200$

$$M_{midspan\_min} := \left[1.25\cdot\left(M_{dc1\_pos\_midspan} + M_{dc2\_pos\_midspan}\right) + 1.5\cdot M_{dw\_pos\_midspan} + 1.75\cdot M_{ll\_neg\_midspan}\right]$$

$M_{midspan\_min} = 15200$ kip-in

$M0 := -M_{midspan\_min} \qquad M0 = -15200$

Mmid will be moment halfway between pier and midspan

$M_{dc1\_pos\_q} := 13000 \quad M_{dc2\_pos\_q} := 0 \quad M_{dw\_pos\_q} := 0 \quad M_{ll\_neg\_q} := -11000$

$$M_{q\_min} := \left[1.25\cdot\left(M_{dc1\_pos\_q} + M_{dc2\_pos\_q}\right) + 1.5\cdot M_{dw\_pos\_q} + 1.75\cdot M_{ll\_neg\_q}\right]$$

$M_{q\_min} = -3000$ kip-in

$Mmid := -M_{q\_min} \qquad Mmid = 3000$

Determine M1 and M2 (moment at pier)

$M1 := 2 \cdot Mmid - M2$ $M1 = -6319$ > $M0 = -15200$ OK

$M2 := -M_{NLL\_st}$ $M2 = 45320$ kip-in

Determine Cb from M1 and M2

$$Cb := 1.75 - 1.05 \cdot \left(\frac{M1}{M2}\right) + 0.3 \cdot \left(\frac{M1}{M2}\right)^2 \qquad Cb = 1.90$$

Since Lp<Lb<Lr

$$Mnc1 := Cb \cdot \left[1 - \left(1 - \frac{Fyr \cdot Sxc}{Rpc \cdot Myc}\right)\left(\frac{Lb - Lp}{Lr - Lp}\right)\right] \cdot Rpc \cdot Myc \qquad Mnc1 = 104620 \text{ kip-in}$$

$Mnc2 := Rpc \cdot Myc$ $Mnc2 = 66701$ kip-in

$Mnc\_ltb := \min(Mnc1, Mnc2)$ $Mnc\_ltb = 66701$ kip-in

Nominal resistance for compression flange

$Mnc := \min(Mnc\_cf, Mnc\_ltb)$ $Mnc = 66701$ kip-in

Strength I Limit State: Negative Flexure (section capacity check)

Note: The pier section design is controlled by splice. These calculations only show adequacy of cross section away from splice.

$Mu := \left|M_{NLL\_st}\right|$ $\phi_f := 1.0$

$Mu = 45320$ kip-in

$Mu = 45320$ < $\phi_f \cdot Mnc = 66701$ **O.K.**

< $\phi_f \cdot Mnt = 69557$ **O.K.**

Strength I Limit State: Shear

Nominal shear resistance. Transverse stiffener spacing is 1/4 of span. Design as unstiffened web with k=5):

$V_p := 0.58 \cdot Fy \cdot Dweb \cdot Tweb$

$A_{v1} := \frac{Dweb}{Tweb}$ $B_{v1} := 1.12 \cdot \sqrt{\frac{29000 \cdot 5}{Fy}}$ $C_{v1} := 1.40 \cdot \sqrt{\frac{29000 \cdot 5}{Fy}}$

$A_{v1} = 53.00$ $B_{v1} = 60.31$ $C_{v1} = 75.39$

$$C := \begin{cases} 1.0 \text{ if } A_{v1} < B_{v1} & \text{: plastic} \\ \frac{1.10}{A_{v1}} \cdot \sqrt{\frac{29000 \cdot 5}{Fy}} \text{ if } B_{v1} \le A_{v1} \le C_{v1} & \text{: inelastic} \\ \frac{1.52}{A_{v1}^2} \cdot \frac{29000 \cdot 5}{Fy} \text{ if } A_{v1} > C_{v1} & \text{: elastic} \end{cases}$$

$C = 1.00$

$V_n := C \cdot V_p$ $V_n = 384.25$ kip $Vr := 1 \cdot V_n$ $Vr = 384.25$ kip

$V_{st} = 313.25$ < $Vr = 384.25$ **O.K.**

Service II Limit State: Negative Flexure (composite section with Rh=1)

$f_{tf\_top\_sv} = 3.44$ ksi **Top flange (Tension)**

$f_{bf\_bot\_sv} = -24.81$ ksi **Bottom Flange (Compression)**

$f_{allowable} := 0.95 \cdot Fy$ $f_{allowable} = 47.50$ ksi

$f_{tf\_top\_sv} = 3.44$ < $f_{allowable} = 47.50$ **O.K.**

$\left|f_{bf\_bot\_sv}\right| = 24.81$ < $f_{allowable} = 47.50$ **O.K.**

Service II Limit State: Check Compressive Stress Against Web Bend Buckling

$Dc := ENA_{short} - Tbf$ $Dc = 30.12$ $kw := \dfrac{9}{\left(\dfrac{Dc}{Dweb}\right)^2}$ $kw = 6.97$

$Fcrw := (0.9) \cdot 29000 \cdot \dfrac{kw}{\left(\dfrac{Dweb}{Tweb}\right)^2}$ $Fcrw = 64.72$ ksi

$\left|f_{bf\_bot\_sv}\right| = 24.81$ < $Fcrw = 64.72$ **O.K.**

Service II Limit State: Check Moment at Post-Tensioned Deck Joint Opening

As determined above:

- Moment at which deck decompresses at top surface (joint would open if at pier centerline). $M_{deck\_top\_decomp} = -30990$ kip-in
- Service II Limit State Moment at pier section $M_{NLL\_sv} = -33655$ kip-in
- Service II Limit State Moment at pier section exceeds the moment at which the deck decompresses at the top surface by 10%. This is only a problem if the deck joint is located directly at the pier section. If the center deck panel is centered on the pier section, the tensile stress of the concrete can be utilized, and the nearest joints may not open.
- Note that these calculations are based on the exterior girder Beff. A similar check was performed for the interior girder, and the deck does not decompress at the top surface under the Service II Limit State Moment at the pier, because the wider Beff of the interior girder increases the section modulus.

Fatigue Limit State: Negative Flexure

$f_{tf\_top\_fat} = 0.37$ ksi **Tension**

$f_{bf\_bot\_fat} = -2.68$ ksi **Compression - Do Not Consider**

Nominal fatigue resistance at bearing stiffener near pier section:

Condition 1:

$n := 1.5$ $ADTT\_SL := 3000$

$N := 365 \cdot 75 \cdot n \cdot ADTT\_SL$ $N = 1.23 \times 10^{8}$

$A\_bs := 44 \cdot 10^{8}$ **For Fatigue Category C' :**

$$\Delta Fn\_bs1 := \left(\frac{A\_bs}{N}\right)^{\frac{1}{3}} \qquad \Delta Fn\_bs1 = 3.29$$

Condition 2:

$\Delta F\_TH\_bs := 12$

$$\Delta Fn\_bs2 := \frac{1}{2} \cdot \Delta F\_TH\_bs \qquad \Delta Fn\_bs2 = 6.00$$

$$\Delta Fn\_bs := \begin{cases} \Delta Fn\_bs1 & \text{if } \Delta Fn\_bs1 \ge \Delta Fn\_bs2 \\ \Delta Fn\_bs2 & \text{otherwise} \end{cases} \qquad \Delta Fn\_bs = 6.00$$

$f_{tf\_top\_fat} = 0.37$ < $\Delta Fn\_bs = 6.00$ **O.K.**

Nominal fatigue resistance at shear stud near pier section:

Condition 1:

$A\_s := 44 \cdot 10^{8}$ **For Fatigue Category C :**

$$\Delta Fn\_s1 := \left(\frac{A\_s}{N}\right)^{\frac{1}{3}} \qquad \Delta Fn\_s1 = 3.29$$

Condition 2:

$\Delta F\_TH\_s := 10$

$$\Delta Fn\_s2 := \frac{1}{2} \cdot \Delta F\_TH\_s \qquad \Delta Fn\_s2 = 5.00$$

$$\Delta Fn\_s := \begin{cases} \Delta Fn\_s1 & \text{if } \Delta Fn\_s1 \ge \Delta Fn\_s2 \\ \Delta Fn\_s2 & \text{otherwise} \end{cases} \qquad \Delta Fn\_s = 5.00$$

$f_{tf\_top\_fat} = 0.37$ < $\Delta Fn\_s = 5.00$ **O.K.**

Fatigue Limit State: Shear

$V_{fat} = 158.30$ kip

Note: The web was designed for the Strength I limit state as unstiffened. Calculations for the Strength I limit state show that the web shear capacity (Vn = Vcr) equals Vp (i.e., C = 1.0) even though the web is treated as unstiffened (AASHTO LRFD Article 6.10.9.2). As shown, the shear capacity exceeds the shear demand for the Strength I load combination (Vst) and therefore the requirement of AASHTO LRFD Article 6.10.5.3 (Vu = Vfat < Vcr) is also satisfied. (Strictly speaking, since the web is treated as unstiffened, and also because this is an end panel AASHTO LRFD Article 6.10.3.3 does not apply).

Part 2. Bolted Field Splice Preliminary Design at Pier Section

I. Cross Section Information (yellow highlight indicates input data)

Yield strength: Fy = 50 ksi

Tensile strength: Fu := 65 ksi

Tube horizontal plate thickness: Tt1 = 0.38 in

Tube vertical plate thickness: Tt2 = 0.38 in

Tube horizontal plate width: Bt1 = 16.00 in

Tube vertical plate width: Bt2 = 7.25 in

Bottom flange thickness: Tbf = 1.50 in

Bottom flange width: Bbf = 18.00 in

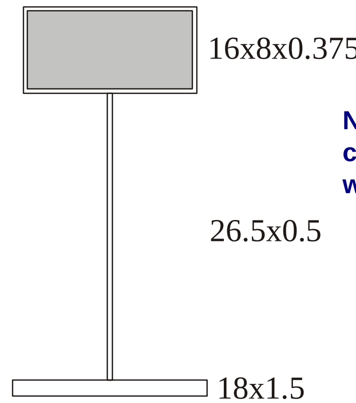
Web thickness: Tweb = 0.50 in

Web depth: Bweb := Dweb   Dweb = 26.50 in

Bolt diameter: dbolt := 0.875 in

Bolt std. hole width: dhole := 1.0 in

Bolt tensile strength: Fubolt := 120 ksi



(unit: in)

Note that tube has 5 in. cut out on each side wall for splice access.

Post-tensioning in Deck (per girder)

Number of Strands: Nstr = 30

Area of Strands: Astr = 6.51 in^2

Short-term Modular Ratio: $n_s = 8$

Area of bottom flange Abf = 27 in^2

Area of tube at cut-out at pier section Atube = 13.6875 in^2

Area of one bolt $Abolt := \frac{\pi}{4} \cdot dbolt^2$ Abolt = 0.6013 in^2

II. Flange Splice Design Loads

Girder Moment at Splice Locations:

DC1: $M_{dc1} = 0$ kip-in

DC2: $M_{dc2} = -4875$ kip-in

DW: $M_{dw} = -3150$ kip-in

LL(positive): $M_{pll} = 0.00$ kip-in

LL(negative): $M_{nll} = -19715$ kip-in

Fatigue(positive): $M_{pfll} = 0$ kip-in

Fatigue(negative): $M_{nfll} = -4843$ kip-in

Splices are designed for Strength I, Service II, and Fatigue Limit States.

Section Modulus - at midthickness of the top flange (tube) and bottom flange

For steel girder plus post-tensioning steel

$Stop_{pt} := Sx_{gird\_pt\_tf\_mid}$ $\quad Stop_{pt} = -935$ in^3

$Sbot_{pt} := Sx_{gird\_pt\_bf\_mid}$ $\quad Sbot_{pt} = 1054$ in^3

For steel girder with short term concrete deck

$Stop_{short} := Sx_{short\_tf\_mid}$ $\quad Stop_{short} = -113328$ in^3

$Sbot_{short} := Sx_{short\_bf\_mid}$ $\quad Sbot_{short} = 1389$ in^3

$Sdeck_{short} := Sx_{short\_deck\_top}$ $\quad Sdeck_{short} = -2789$ in^3

Flange Stress Computation:

Case 1: Dead Load + Positive Live Load (there is no positive moment at pier centerline section)
Case 2: Dead Load + Negative Live Load
==> Case 2 controls, therefore only check Case 2

DC2:

$f_{deckDC2\_short} := \frac{M_{dc2}}{Sdeck_{short} \cdot n_s}$ $\quad f_{deckDC2\_short} = 0.22$ ksi **Tension**

$f_{topDC2\_short} := \frac{M_{dc2}}{Stop_{short}}$ $\quad f_{topDC2\_short} = 0.04$ ksi **Tension**

$f_{botDC2\_short} := \frac{M_{dc2}}{Sbot_{short}}$ $\quad f_{botDC2\_short} = -3.51$ ksi **Compression**

DW:

$f_{deckDW\_short} := \frac{M_{dw}}{Sdeck_{short} \cdot n_s}$ $\quad f_{deckDW\_short} = 0.14$ ksi **Tension**

$f_{topDW\_short} := \frac{M_{dw}}{Stop_{short}}$ $\quad f_{topDW\_short} = 0.03$ ksi **Tension**

$f_{botDW\_short} := \frac{M_{dw}}{Sbot_{short}}$ $\quad f_{botDW\_short} = -2.27$ ksi **Compression**

-LL:

$f_{deckNLL\_short} := \frac{M_{nll}}{Sdeck_{short} \cdot n_s}$ $\quad f_{deckNLL\_short} = 0.88$ ksi **Tension**

$f_{topNLL\_short} := \frac{M_{nll}}{Stop_{short}}$ $\quad f_{topNLL\_short} = 0.17$ ksi **Tension**

$f_{botNLL\_short} := \frac{M_{nll}}{Sbot_{short}}$ $\quad f_{botNLL\_short} = -14.19$ ksi **Compression**

Strength I Limit State: Dead Load + Negative Live Load

$M_{NLL\_st} := (1.25 \cdot M_{dc2} + 1.5 \cdot M_{dw} + 1.75 \cdot M_{nll})$ $M_{NLL\_st} = -45320$ kip-in **Negative**

Flange stresses are stresses when deck fully decompresses plus additional stresses for remaining moment up the Strength Limit State moment demand.

$$f_{topNLL\_st} := f_{full\_decomp\_tf\_mid} + \frac{M_{NLL\_st} - M_{full\_decomp}}{Stop_{pt}}$$ $f_{topNLL\_st} = -1.75$

$$f_{botNLL\_st} := f_{full\_decomp\_bf\_mid} + \frac{M_{NLL\_st} - M_{full\_decomp}}{Sbot_{pt}}$$ $f_{botNLL\_st} = -35.05$

Service II Limit State: Dead Load + Negative Live Load

$f_{topNLL\_sv} := (1.0 \cdot f_{topDC2\_short} + 1.0 \cdot f_{topDW\_short} + 1.3 \cdot f_{topNLL\_short})$ $f_{topNLL\_sv} = 0.30$ ksi **Tension**

$f_{botNLL\_sv} := (1.0 \cdot f_{botDC2\_short} + 1.0 \cdot f_{botDW\_short} + 1.3 \cdot f_{botNLL\_short})$ $f_{botNLL\_sv} = -24.22$ ksi **Compression**

$M_{NLL\_sv} := (1.0 \cdot M_{dc2} + 1.0 \cdot M_{dw} + 1.3 \cdot M_{nll})$ $M_{NLL\_sv} = -3.37 \times 10^{4}$ kip-in **Negative**

$f_{deckNLL\_sv} := (1.0 \cdot f_{deckDC2\_short} + 1.0 \cdot f_{deckDW\_short} + 1.3 \cdot f_{deckNLL\_short})$ $f_{deckNLL\_sv} = 1.508$ ksi **Compression**

Fatigue Limit State: Negative Live Load

$$f_{topNFLL\_short} := \frac{M_{nfll}}{Stop_{short}}$$ $f_{topNFLL\_short} = 0.04$ ksi **Tension**

$$f_{botNFLL\_short} := \frac{M_{nfll}}{Sbot_{short}}$$ $f_{botNFLL\_short} = -3.49$ ksi **Compression**

$f_{topNLL\_fa} := 0.75 \cdot f_{topNFLL\_short}$ $f_{topNLL\_fa} = 0.03$ ksi **Tension**

$f_{botNLL\_fa} := 0.75 \cdot f_{botNFLL\_short}$ $f_{botNLL\_fa} = -2.61$ ksi **Compression**

$M_{NLL\_fa} := 0.75 \cdot M_{nfll}$ $M_{NLL\_fa} = -3.63 \times 10^{3}$ kip-in **Negative**

Strength I Minimum Design Force - Controlling Flange:

From above results, the bottom flange is the controlling flange for the Strength I Limit State.

Minimum design stress for the controlling (bottom) flange:

$Rh := 1.0 \qquad \alpha := 1.0 \qquad \phi f := 1.0 \qquad Fcf2 := 0.75 \cdot \alpha \cdot \phi f \cdot Fy \qquad Fcf2 = 37.50$

$$Fcf1\_NLL := \frac{\left|\frac{f_{botNLL\_st}}{Rh}\right| + \alpha \cdot \phi f \cdot Fy}{2} \qquad Fcf1\_NLL = 42.52$$

$Fcf\_NLL := \max(Fcf1\_NLL, Fcf2) \qquad Fcf\_NLL = 42.52$ ksi

$Pcu\_NLL := Fcf\_NLL \cdot Abf \qquad Pcu\_NLL = 1.15 \times 10^{3}$ kip

Strength I Minimum Design Force - Noncontrolling Flange:

From above results, the top flange is the noncontrolling flange for the Strength I Limit State.

Minimum design stress for the noncontrolling (top) flange:

$f_{topNLL\_st} = -1.75 \qquad Fcf\_NLL = 42.52 \qquad f_{botNLL\_st} = -35.05$

$$Rcf\_NLL := \left|\frac{Fcf\_NLL}{f_{botNLL\_st}}\right| \qquad Rcf\_NLL = 1.21$$

$$Fncf1\_NLL := Rcf\_NLL \cdot \left|\frac{f_{topNLL\_st}}{Rh}\right| \qquad Fncf1\_NLL = 2.12$$

$Fncf\_NLL := \max(Fncf1\_NLL, Fcf2) \qquad Fncf\_NLL = 37.50$ ksi

$Pncu\_NLL := Fncf\_NLL \cdot Atube \qquad Pncu\_NLL = 513$ kip

Service II Limit State Flange Force:

$Ps\_bf := f_{botNLL\_sv} \cdot Abf \qquad Ps\_bf = -654$ kip

$Ps\_tf := f_{topNLL\_sv} \cdot Atube \qquad Ps\_tf = 4.06$ kip

Fatigue Limit State Stresses:

$\Delta f\_bf := \left|f_{botNLL\_fa}\right| \qquad \Delta f\_bf = 2.61$ ksi

$\Delta f\_tf := \left|f_{topNLL\_fa}\right| \qquad \Delta f\_tf = 0.03$ ksi

III. Design Bottom Flange Splice (yellow highlight indicates input)

Splice Plate Dimensions:

Try 1.5 x 18" outside splice plate (no inside plate)

Thickness of the outside splice palte: $Tout := 1.5$ in

Width of the outside splice plate: $Bout := 18$ in

$Aout := Tout \cdot Bout \qquad Aout = 27.00$ in^2

Yielding and Fracture of Splice Plates:

Compression.

$Pcu\_NLL = 1148$

Check compression yielding of outside splice plate:

$Prout\_yield\_comp := 0.90 \cdot Fy \cdot Tout \cdot Bout \qquad Prout\_yield\_comp = 1215$ kip

$Pcu\_NLL = 1148 \quad < \quad Prout\_yield\_comp = 1215$ **O.K.**

Compression stress on actual net section of outside splice plate. This stress check is not required, but shows that the net section is somewhat small when 6 bolts per row are used:

$nb := 6$

$$\frac{Pcu\_NLL}{Aout - \left[nb \cdot \left(dbolt + \frac{1}{16}\right) \cdot Tout\right]} = 61.85 \text{ ksi}$$

Yielding of Flange and Fracture of Flange at Holes:

Compression. Note that flange stress and the flange splice design stress are both less than yield stress.

$f_{botNLL\_st} = -35.05$ ksi **O.K.**

Stress on flange and splice plate are similar. They have the same width and thickness.

$Fcf\_NLL = 42.52$ ksi **O.K.**

Bolts - Shear:

Determine the number of bolts for the bottom flange splice plates that are required to develop the Strength I design force in the flange in shear assuming the bolts in the connection have slipped and gone into bearing.

$Pcu\_NLL = 1148$

Assume that the threads are excluded from the shear planes and the design force acts on one shear plane.

$Ns1 := 1$

$Rn\_bf := 0.48 \cdot Abolt \cdot Fubolt \cdot Ns1 \qquad Rn\_bf = 34.64$ kip

$Ru\_bf := 0.80 \cdot Rn\_bf \qquad Ru\_bf = 27.71$ kip

$$Nbf\_eachside := \frac{Pcu\_NLL}{Ru\_bf} \qquad Nbf\_eachside = 41.44$$

The minimum number of bolts required on each side of the splice to resist the Strength I flange design force in shear is 42. The number of bolts used is 48, 8 rows of 6 bolts.

Bolts - Slip Resistance:

Bolted flange splice designed as slip-critical connections for the Service II flange design force.

$Ps\_bf = -654$

Determine the factored resistance per bolt assuming a Class B surface condition.

Minimum required bolt tension: $Pt := 39$ kip

Hole size factor: $Kh := 1.0$

Surface condition factor for Class B surface conditions: $Ks := 0.5$

$Rn\_slip\_bf := Kh \cdot Ks \cdot Ns1 \cdot Pt$

$Rr\_slip\_bf := Rn\_slip\_bf$ $Rr\_slip\_bf = 19.50$ kip

$Nbf\_eachside\_slip\_bf := \left| \frac{Ps\_bf}{Rr\_slip\_bf} \right|$ $Nbf\_eachside\_slip\_bf = 33.54$

The minimum number of bolts required on each side of the splice to resist the Service II flange design force against slip is 34. The number of bolts used is 48, 8 rows of 6 bolts.

Bolts - Minimum Spacing:

$dbolt = 0.875$ $s\_min := 3 \cdot dbolt$ $s\_min = 2.625$ in

Bolts - Edge Distance and Spacing for Splice Plate:

The edge distance is 1.5in. and the bolt spacing is 3*dbolt= 2.625 in.

Bolts - Bearing at Bolt Holes on Splice Plate:

$Pcu\_NLL = 1148$

The clear end distance between the edge of the hole and the end of the splice plate:

$Lc1\_bf := 1.5 - \frac{dhole}{2}$ $Lc1\_bf = 1.00$ in

The clear distance between edges of adjacent holes in the direction of the force is computed as:

$Lc2\_bf := 3 \cdot dbolt - dhole$ $Lc2\_bf = 1.63$ in

For the outside splice plate:

n1: Number of bolts in the end row $n1 := 6$

n2: Number of remaining bolts $n2 := 48 - n1$

$Rn1 := n1 \cdot (1.2 \cdot Lc1\_bf \cdot Tout \cdot Fu)$ $Rn2 := n2 \cdot (1.2 \cdot Lc2\_bf \cdot Tout \cdot Fu)$

$Rn\_bf\_bearing := Rn1 + Rn2$ $Rn\_bf\_bearing = 8.69 \times 10^{3}$ kips

$Rr\_bf\_bearing := 0.80 \cdot Rn\_bf\_bearing$ $Rr\_bf\_bearing = 6.95 \times 10^{3}$ kips

$Pcu\_NLL = 1148$ < $Rr\_bf\_bearing = 6950$ **O.K.**

Bolts - Bearing at Bolt Holes on Flange:

Note flange is same thickness as splice plate and edge distance is greater - no check required.

Fatigue of Flange at Bolt Holes:

Load-induced fatigue:

$\Delta f\_bf = 2.61$ ksi

Nominal fatigue resistance:

Condition 1:

$n := 1.5$ $ADTT\_SL := 3000$

$N := 365 \cdot 75 \cdot n \cdot ADTT\_SL$ $N = 1.23 \times 10^{8}$

$A := 120 \cdot 10^{8}$ **For Fatigue Category B:**

$$\Delta Fn1 := \left(\frac{A}{N}\right)^{\frac{1}{3}} \qquad \Delta Fn1 = 4.60$$

Condition 2:

$\Delta F\_TH := 16$

$$\Delta Fn2 := \frac{1}{2} \cdot \Delta F\_TH \qquad \Delta Fn2 = 8.00$$

$$\Delta Fn := \begin{cases} \Delta Fn1 & \text{if } \Delta Fn1 \geq \Delta Fn2 \\ \Delta Fn2 & \text{otherwise} \end{cases} \qquad \Delta Fn = 8.00$$

$\Delta f\_bf = 2.61$ < $\Delta Fn = 8.00$ **O.K.**

Fatigue of Splice Plate at Bolt Holes:

Load-induced fatigue:

$\Delta f\_out := \Delta f\_bf \cdot \frac{Abf}{Aout}$ $\Delta f\_out = 2.61$ ksi < $\Delta Fn = 8.00$ **O.K.**

IV. Design Top Flange Splice (yellow highlight indicates input)

- **Splice plates are on top wall and bottom wall of tube.**
- **The top wall top splice plate, the top wall bottom splice plate, and the bottom wall top splice plate are identical. Call this plate the *outside plate*.**
- **The bottom wall bottom splice plate pair (adjacent to web) differ. Call these plates the *inside plate*.**

Splice Plate Dimensions:

Try 0.5 x 13.5" plate for outside splice plate

Thickness of the outside splice plate: $Tout\_tf := 0.5$ in

Width of the outside splice plate: $Bout\_tf := 13.5$ in

Try (2) 0.5 x 6.0" plates for inside splice plate

Thickness of the inside top wall splice plate: $Tin\_tf := 0.5$ in

Width of the inside top wall splice plate: $Bin\_tf := 2 \cdot 6$ in

$Aout\_tf := Tout\_tf \cdot Bout\_tf$ $\quad Aout\_tf = 6.75$ in^2

$Ain\_tf := Tin\_tf \cdot Bin\_tf$ $\quad Ain\_tf = 6.00$ in^2

n1_tf: Number of bolts across the width of single splice plate $n1\_tf := 4$

Check $\left(1 - \dfrac{Ain\_tf}{Aout\_tf}\right) \cdot 100 = 11.11$ **The areas are essentially within ten percent ==> O.K.**

Yielding and Fracture of Splice Plates:

Total Tension (apply 1/2 to set of plates on each tube wall and apply 1/2 of that to each plate):

$Pncu\_NLL = 513$

For yielding on the outside splice plate :

$Prout\_yield\_ten := 0.95 \cdot Fy \cdot Tout\_tf \cdot Bout\_tf$ $\quad Prout\_yield\_ten = 320.63$ kip

$\dfrac{Pncu\_NLL}{2 \cdot 2} = 128$ $<$ $Prout\_yield\_ten = 321$ **O.K.**

For yielding on the inside splice plates :

$Prin\_yielding\_ten := 0.95 \cdot Fy \cdot Tin\_tf \cdot Bin\_tf$ $\quad Prin\_yielding\_ten = 285.00$ kip

$\dfrac{Pncu\_NLL}{2 \cdot 2} = 128$ $<$ $Prin\_yielding\_ten = 285$ **O.K.**

For fracture of the outside splice plate:

$Bn\_out\_tf := Bout\_tf - n1\_tf \cdot dhole$ $\quad Bn\_out\_tf = 9.50$ in

$An\_out\_tf := Bn\_out\_tf \cdot Tout\_tf$ $\quad An\_out\_tf = 4.75$ in^2

$Prout\_fra\_tf := 0.8 \cdot Fu \cdot An\_out\_tf$ $\quad Prout\_fra\_tf = 247.00$ kip

$\dfrac{Pncu\_NLL}{2 \cdot 2} = 128$ $<$ $Prout\_fra\_tf = 247$ **O.K.**

For fracture of the inside splice plates:

$Bn\_in\_tf := Bin\_tf - n1\_tf \cdot dhole$ $Bn\_in\_tf = 8.00$ in

$An\_in\_tf := Bn\_out\_tf \cdot Tin\_tf$ $An\_in\_tf = 4.75$ in^2

$Prin\_fra\_tf := 0.80 \cdot Fu \cdot An\_in\_tf$ $Prin\_fra\_tf = 247.00$ kip

$\frac{Pncu\_NLL}{2 \cdot 2} = 128$ < $Prin\_fra\_tf = 247$ **O.K.**

Yielding of Tube Flange and Fracture of Tube Flange at Holes:

Note that flange stress and the flange splice design stress are both less than yield stress.

$f_{topNLL\_st} = -1.75$ ksi < $Fy = 50$ ksi **O.K.**

$Fncf\_NLL = 37.50$ ksi < $Fy = 50$ ksi **O.K.**

For checking net section fracture of the top flange at cut-out (AASHTO 6.10.1.8).
Note that the number of holes on the net section is 2 times number of holes in one bolt row on splice plates.

$n\_holes\_tube := 2 \cdot n1\_tf$

$$Ft\_max := 0.84 \cdot \frac{Atube - n\_holes\_tube \cdot dhole \cdot Tt1}{Atube} \cdot Fu$$

$f_{topNLL\_st} = -1.75$ ksi < $Ft\_max = 42.63$ ksi **O.K.**

$Fncf\_NLL = 37.50$ ksi < $Ft\_max = 42.63$ ksi **O.K.**

Alternate check of net section fracture of the top flange at cut-out.

$Ptube\_cutout\_fracture\_ten := 0.80 \cdot Fu \cdot (Atube - n\_holes\_tube \cdot dhole \cdot Tt1)$ $Ptube\_cutout\_fracture\_ten = 556$

$Pncu\_NLL = 513$ < $Ptube\_cutout\_fracture\_ten = 556$ **O.K.**

Bolts - Shear:

Number of bolts for each wall of the tube required to develop the Strength I design force assuming bolts have slipped and gone into bearing. Design for 1/2 of the following top flange force:

$Pncu\_NLL = 513.28$

Assume that the threads are excluded from the shear planes and the design force acts on two shear planes (double shear).

$Ns2 := 2$

$Rn\_tf := 0.48 \cdot Abolt \cdot Fubolt \cdot Ns2$ $Rn\_tf = 69.27$ kip

$Ru\_tf := 0.80 \cdot Rn\_tf$ $Ru\_tf = 55.42$ kip

$$Ntf\_eachside := \frac{\frac{Pncu\_NLL}{2}}{Ru\_tf}$$ $Ntf\_eachside = 4.63$

The minimum number of bolts required on each side of the splice to resist the 1/2 of the Strength I flange design force in shear is 5. The number of bolts used is 8, 2 rows of 4 bolts.

Bolts - Slip Resistance:

Number of bolts for the top flange top wall splice required for slip-critical connection. Design for 1/2 the following Service II flange design force.

$Ps\_tf = 4.06$

Determine the factored resistance per bolt assuming a Class B surface condition.

Minimum required bolt tension: $Pt = 39.00$ kip

Hole size factor: $Kh = 1.00$

Surface condition factor for Class B surface conditions: $Ks = 0.50$

$Rn\_slip\_tf := Kh \cdot Ks \cdot Ns2 \cdot Pt$ $Rr\_slip\_tf := Rn\_slip\_tf$ $Rr\_slip\_tf = 39.00$ kip

$$Ntf\_eachside\_slip := \left| \frac{\frac{Ps\_tf}{2}}{Rr\_slip\_tf} \right|$$ $Ntf\_eachside\_slip = 0.05$

The minimum number of bolts required on each side of the splice to resist the 1/2 of the Service II flange design force in shear is 1. The number of bolts used is 8, 2 rows of 4 bolts.

Bolts - Minimum Spacing:

$s\_min = 2.63$ in

Bolts - Edge Distance and Spacing for Splice Plates:

The edge distance is 1.5in. and the bolt spacing is 3*dbolt= 2.625 in.

Bolts - Bearing at Bolt Holes on Splice Plate:

Check bolt bearing strength for the Strength I design force assuming bolts have slipped and gone into bearing. For each wall of tube, design for 1/2 of the following top flange force and apply 1/2 to each plate :

$Pncu\_NLL = 513$

The clear end distance between the edge of the hole and the end of the splice plate:

$Lc1\_tfsp := 1.5 - \frac{dhole}{2}$ $Lc1\_tfsp = 1.00$ in

The clear distance between edges of adjacent holes in the direction of the force is computed as:

$Lc2\_tfsp := 3 \cdot dbolt - dhole$ $Lc2\_tfsp = 1.63$ in

Both the outside and inside splice plates have the same thickness so the calculation is the same:

n1: Number of bolts holes in the end row $n1\_tf = 4$

n2: Number of remaining bolts holes $n2\_tf := 8 - n1\_tf$

$Rn1\_tfsp := n1\_tf \cdot (1.2 \cdot Lc1\_tfsp \cdot Tout\_tf \cdot Fu)$ $Rn2\_tfsp := n2\_tf \cdot (1.2 \cdot Lc2\_tfsp \cdot Tout\_tf \cdot Fu)$

$Rn\_tfsp\_bearing := Rn1\_tfsp + Rn2\_tfsp$ $Rn\_tfsp\_bearing = 410$ kips

$Rr\_tfsp\_bearing := 0.80 \cdot Rn\_tfsp\_bearing$ $Rr\_tfsp\_bearing = 328$ kips

$\frac{Pncu\_NLL}{2 \cdot 2} = 128$ **<** $Rr\_tfsp\_bearing = 328$ **O.K.**

Bolts - Edge Distance and Spacing for Tube Flange:

The edge distance is 2.125 in., leaving 1/2 in between girder field pieces at pier.
The bolt spacing is 3*dbolt= 2.625 in.

Bolts - Bearing at Bolt Holes on Tube Flange:

Check bolt bearing strength for the Strength I design force assuming bolts have slipped and gone into bearing. For each wall of tube, design for 1/2 of the following top flange force:

$Pncu\_NLL = 513$

The clear end distance between the edge of the hole and the end of the splice plate:

$Lc1\_tf := 2.125 - \frac{dhole}{2}$ $Lc1\_tf = 1.63$ in

The clear distance between edges of adjacent holes in the direction of the force is computed as:

$Lc2\_tf := 3 \cdot dbolt - dhole$ $Lc2\_tf = 1.63$ in

$Rn1\_tf := n1\_tf \cdot (1.2 \cdot Lc1\_tf \cdot Tt1 \cdot Fu)$ $Rn2\_tf := n2\_tf \cdot (1.2 \cdot Lc2\_tf \cdot Tt1 \cdot Fu)$

$Rn\_bf\_bearing\_tf := Rn1\_tf + Rn2\_tf$ $Rn\_bf\_bearing\_tf = 380$ kips

$Rr\_bf\_bearing\_tf := 0.80 \cdot Rn\_bf\_bearing\_tf$ $Rr\_bf\_bearing\_tf = 304$ kips

$\frac{Pncu\_NLL}{2} = 257$ **=** $Rr\_bf\_bearing\_tf = 304$ **O.K.**

Fatigue of Flange at Bolt Holes:

Load-induced fatigue:

$\Delta f\_tf = 0.032$ ksi

Nominal fatigue resistance:

Condition 1:

$N = 1.23 \times 10^{8}$

Condition 2:

$\Delta F\_TH = 16.00$

$A = 1.20 \times 10^{10}$ **For Fatigue Category B:**

$\Delta Fn1 = 4.60$

$\Delta Fn2 = 8.00$

$$\Delta Fn := \begin{cases} \Delta Fn1 & \text{if } \Delta Fn1 \geq \Delta Fn2 \\ \Delta Fn2 & \text{otherwise} \end{cases}$$

$\Delta Fn = 8.00$

$\Delta f\_tf = 0.03$ < $\Delta Fn = 8.00$ **O.K.**

Fatigue of Splice Plates at Bolt Holes:

Inside splice plates have smallest area so calculate for the inside plate.

Load-induced fatigue:

$$\Delta f\_tf\_in := \Delta f\_tf \cdot \frac{Atube}{4Ain\_tf}$$

$\Delta f\_tf\_in = 0.018$ ksi < $\Delta Fn = 8.00$ **O.K.**

Fatigue at Cutout Location:

$$Stop3_{short} := \frac{44253}{14.522 - 8 - 3}$$

$Stop3_{short} = 1.26 \times 10^4$

$$f_{topNFLL\_short3} := \frac{M_{nfll}}{Stop3_{short}}$$

$f_{topNFLL\_short3} = -0.39$

$f_{topNLL\_fa3} := 0.75 \cdot f_{topNFLL\_short3}$

$f_{topNLL\_fa3} = -0.29$

$\Delta f\_tf3 := \left| f_{topNLL\_fa3} \right| \cdot 2$

$\Delta f\_tf3 = 0.58$

Nominal fatigue resistance:

Condition 1:

$N = 1.23 \times 10^8$

$A3 := 250 \cdot 10^8$ **For Fatigue Category A:**

$$\Delta Fn1\_3 := \left(\frac{A3}{N}\right)^{\frac{1}{3}}$$

$\Delta Fn1\_3 = 5.88$

Condition 2:

$\Delta F\_TH3 := 24$

$$\Delta Fn2\_3 := \frac{1}{2} \cdot \Delta F\_TH3$$

$\Delta Fn2\_3 = 12.00$

$$\Delta Fn3 := \begin{cases} \Delta Fn1\_3 & \text{if } \Delta Fn1\_3 \geq \Delta Fn2\_3 \\ \Delta Fn2\_3 & \text{otherwise} \end{cases}$$

$\Delta Fn3 = 12.00$

$\Delta f\_tf3 = 0.58$ < $\Delta Fn3 = 12.00$ **O.K.**

V. Compute Web Splice Design Loads (yellow highlight indicates input)

Girder Shear Forces at Splice Locations:

DC1:	$V_{dc1} := 0$	kip
DC2:	$V_{dc2} := 0$	kip
DW:	$V_{dw} := 0$	kip
LL(positive):	$V_{pll} := 0$	kip
LL(negative):	$V_{nll} := 0$	kip
Fatigue(positive):	$V_{pfll} := 0$	kip
Fatigue(negative):	$V_{nfll} := 0$	kip

Web Moments and Horizontal Force Resultant:

Muw : Portion of the flexural moment assumed to be resisted by the web
Huw : Horizontal design force resultant
Vuw : Design shear force
Muv : Moment due to the eccentricity of the design shear (Muv = Vuw x e)
e : Distance from the centerline of the splice to the centroid of the connection on the side of the joint under consideration
Mtotal = Muw + Muv

$e := 2.375 + 2.625$ $\quad e = 5.00$ in **Based on three verical rows of bolts in each side**

Strength I Limit State:

Design Shear:

The nominal shear resistance:

$$A_v := \frac{Bweb}{Tweb} \qquad B_v := 1.10 \cdot \sqrt{\frac{29000 \cdot 5}{Fy}} \qquad C_v := 1.38 \cdot \sqrt{\frac{29000 \cdot 5}{Fy}}$$

$A_v = 53.00$ $\quad B_v = 59.24$ $\quad C_v = 74.32$

$$V_p := 0.58 \cdot Fy \cdot Bweb \cdot Tweb$$

$$C := \begin{cases} 1.0 \text{ if } A_v < B_v & \text{: plastic} \\ \frac{1.10}{A_v} \cdot \sqrt{\frac{29000 \cdot 5}{Fy}} \text{ if } B_v \le A_v \le C_v & \text{: inelastic} \\ \frac{1.52}{A_v^{\,2}} \cdot \frac{29000 \cdot 5}{Fy} \text{ if } A_v > C_v & \text{: elastic} \end{cases}$$

$C = 1.00$

$V_n := C \cdot V_p$ $\quad V_n = 384.25$ kip $\quad Vr := 1 \cdot V_n$ $\quad Vr = 384.25$ kip

The factored shear for the negative live load:

$$Vu\_NLL := \left(1.25 \cdot V_{dc1} + 1.25 \cdot V_{dc2} + 1.5 \cdot V_{dw} + 1.75 \cdot V_{nll}\right) \cdot 0.95$$

$Vu\_NLL = 0.00$ kip

Therefore, with $Vu := |Vu\_NLL|$ $Vu = 0.00$ $Vr = 384.25$

AASHTO requires the following web splice design shear:

$$Vuw := \begin{cases} 0.75 \cdot Vr & \text{if } Vu < 0.5 \cdot Vr \\ \dfrac{Vu + Vr}{2} & \text{otherwise} \end{cases}$$ $Vuw = 288.19$ kip

However, since web splice is over the bearing, set the design shear to zero:

$Vuw := 0$ kip

Web Moments and Horizontal Force Resultants:

Dead Load + Negative Live Load:

$f_{botNLL\_st} = -35.05$ ksi — **Maximum elastic flexural stress due to the factored loads at the midthickness of the controlling flange**

$Fcf\_NLL := -Fcf\_NLL$ — **Design stress for the controlling flange**

$Fcf\_NLL = -42.52$ ksi

$f_{topNLL\_st} = -1.75$ ksi — **Maximum elastic flexural stress due to the factored loads at the midthickness of the noncontrolling flange**

$Rcf\_NLL = 1.21$

Portion of the flexural moment to be resisted by the web:

$$Mw\_st\_neg := \frac{Tweb \cdot Bweb^2}{12} \cdot \left| Rh \cdot Fcf\_NLL - Rcf\_NLL \cdot f_{topNLL\_st} \right|$$ $Mw\_st\_neg = 1.18 \times 10^3$ kip-in

Total web moment:

$$Mtot\_st\_neg := Mw\_st\_neg + Vuw \cdot e$$ $Mtot\_st\_neg = 1.18 \times 10^3$ kip-in

Horizontal force resultant:

$$Hw\_st\_neg := \frac{Tweb \cdot Bweb}{2} \cdot \left(Rh \cdot Fcf\_NLL + Rcf\_NLL \cdot f_{topNLL\_st} \right)$$ $Hw\_st\_neg = -295.78$ kip

Service II Limit State:

Design Shear:

The factored shear for the negative live load:

$$Vser\_NLL := 1.0 \cdot V_{dc1} + 1.0 \cdot V_{dc2} + 1.0 \cdot V_{dw} + 1.3 \cdot V_{nll}$$ $Vser\_NLL = 0.00$ kip

Therefore

$$Vw\_ser := |Vser\_NLL|$$ $Vw\_ser = 0.00$ kip

Web Moments and Horizontal Force Resultants:

Dead Load + Negative Live Load:

$f_{botNLL\_sv} = -24.22$ ksi — **Maximum Service II midthickness flange stress**

$f_{topNLL\_sv} = 0.30$ ksi — **Service II midthickness flange stress in other flange**

Portion of the flexural moment to be resisted by the web:

$$Mw\_ser\_neg := \frac{Tweb \cdot Bweb^2}{12} \cdot \left| f_{botNLL\_sv} - f_{topNLL\_sv} \right| \qquad Mw\_ser\_neg = 717.45 \quad \text{kip-in}$$

Total web moment:

$$Mtot\_ser\_neg := Mw\_ser\_neg + Vw\_ser \cdot e \qquad Mtot\_ser\_neg = 717.45 \quad \text{kip-in}$$

Horizontal force resultant:

$$Hw\_ser\_neg := \frac{Tweb \cdot Bweb}{2} \cdot \left(f_{botNLL\_sv} + f_{topNLL\_sv} \right) \qquad Hw\_ser\_neg = -158.51 \quad \text{kip}$$

Fatigue Limit State:

Design Shear:

The factored shear for the negative live load:

$$Vfat\_NLL := 0.75 \cdot V_{nfll} \qquad Vfat\_NLL = 0.00 \quad \text{kip}$$

Web Moments and Horizontal Force Resultants:

Negative Live Load:

$f_{botNLL\_fa} = -2.61$ ksi

$f_{topNLL\_fa} = 0.03$ ksi

Portion of the flexural moment to be resisted by the web:

$$Mw\_fat\_neg := \frac{Tweb \cdot Bweb^2}{12} \cdot \left(f_{botNLL\_fa} - f_{topNLL\_fa} \right) \qquad Mw\_fat\_neg = -77.43 \quad \text{kip-in}$$

Total web moment:

$$Mtot\_fat\_neg := Mw\_fat\_neg + Vfat\_NLL \cdot e \qquad Mtot\_fat\_neg = -77.43 \quad \text{kip-in}$$

Horizontal force resultant:

$$Hw\_fat\_neg := \frac{Tweb \cdot Bweb}{2} \cdot \left(f_{botNLL\_fa} + f_{topNLL\_fa} \right) \qquad Hw\_fat\_neg = -17.11 \quad \text{kip}$$

VI. Design Web Splice (yellow highlight indicates input)

Web Splice Configuration:

1. Three vertical rows of bolts with eight bolts per row.
2. 1/2" x 21.5" splice plates on each side of the web.

$twp := \frac{1}{2}$ $dwp := 21.5$

Bolts - Minimum Spacing:

$dbolt = 0.875$ $s\_min := 3 \cdot dbolt$ $s\_min = 2.625$ in

Bolts - Edge Distance:

The smallest edge distance is 1.5in. and the bolt spacing is 3*dbolt= 2.625 in.

Bolts - Shear:

$m := 3$ **: Number of vertical rows of bolts**

$n := 8$ **: Number of bolts in one verical row**

$s := 2.625$ in   **: Vertical pitch**

$g := 2.625$ in   **: Horizontal pitch**

$Ip := \frac{n \cdot m}{12} \cdot \left[s^2 \cdot \left(n^2 - 1\right) + g^2 \cdot \left(m^2 - 1\right) \right]$ $Ip = 978.47$ in^2 **: Polar moment of inertia**

$Nbw := n \cdot m$ $Nbw = 24.00$ **: Total number of web bolts on each side of the splice**

Strength I Limit State:

Assume that the threads are excluded from the shear planes

$Ru\_web := Ru\_tf$ $Ru\_web = 55.42$ kip

Dead Load + Negative Live Load:

$Vuw = 0.00$ kip $Mtot\_st\_neg = 1.18 \times 10^3$ kip-in $Hw\_st\_neg = -295.78$ kip

Vertical shear force in the bolts due to applied shear force:

$Pv\_st := \frac{Vuw}{Nbw}$ $Pv\_st = 0.00$ kip

Horizontal shear force in the bolts due to horizontal force resultant:

$Ph\_st\_neg := \frac{|Hw\_st\_neg|}{Nbw}$ $Ph\_st\_neg = 12.32$ kip

Horizontal and vertical components of the bolt shear force on the extreme bolt due to the total moment in the web:

$x := \frac{g}{2}$ $x = 1.31$ in $y := \frac{15 \cdot s}{2}$ $y = 19.69$ in

$Pmv\_st\_neg := \frac{Mtot\_st\_neg \cdot x}{Ip}$ $Pmv\_st\_neg = 1.59$ kip

$Pmh\_st\_neg := \frac{Mtot\_st\_neg \cdot y}{Ip}$ $Pmh\_st\_neg = 23.79$ kip

$Pr\_st\_neg := \sqrt{(Pv\_st + Pmv\_st\_neg)^2 + (Ph\_st\_neg + Pmh\_st\_neg)^2}$ $Pr\_st\_neg = 36.15$

$Pr\_st := Pr\_st\_neg$

$Pr\_st = 36.15$ < $Ru\_web = 55.42$ **O.K.**

Service II Limit State:

Determine the factored resistance per bolt assuming a Class B surface condition.

$Rr\_slip\_web := Rr\_slip\_tf$ $Rr\_slip\_web = 39.00$ kip

Dead Load + Negative Live Load:

$Vw\_ser = 0.00$ kip $Mtot\_ser\_neg = 717.45$ kip-in $Hw\_ser\_neg = -158.51$ kip

Vertical shear force in the bolts due to applied shear force:

$Ps\_ser := \frac{Vw\_ser}{Nbw}$ $Ps\_ser = 0.00$ kip

Horizontal shear force in the bolts due to horizontal force resultant:

$Ph\_ser\_neg := \frac{|Hw\_ser\_neg|}{Nbw}$ $Ph\_ser\_neg = 6.60$ kip

Horizontal and vertical components of the bolt shear force on the extreme bolt due to the total moment in the web:

$Pmv\_ser\_neg := \frac{Mtot\_ser\_neg \cdot x}{Ip}$ $Pmv\_ser\_neg = 0.96$ kip

$Pmh\_ser\_neg := \frac{Mtot\_ser\_neg \cdot y}{Ip}$ $Pmh\_ser\_neg = 14.44$ kip

$Pr\_ser\_neg := \sqrt{(Ps\_ser + Pmv\_ser\_neg)^2 + (Ph\_ser\_neg + Pmh\_ser\_neg)^2}$ $Pr\_ser\_neg = 21.06$ kip

$Pr\_ser := Pr\_ser\_neg$

$Pr\_ser = 21.06$ < $Rr\_slip\_web = 39.00$ **O.K.**

Shear Yielding of Splice Plates:

$Vuw = 0.00$ kip

$Nwp := 2$: Number of splice plates

$twp = 0.50$ in : Thickness of splice plate

$dwp = 21.50$ in : Depth of splice plate

$Agross\_wp := Nwp \cdot twp \cdot dwp$ $Agross\_wp = 21.50$ in^2

$Rr\_wp := 0.58 \cdot Fy \cdot Agross\_wp$ $Rr\_wp = 623.50$ kip

$Vuw = 0.00$ < $Rr\_wp = 623.50$ **O.K.**

Fracture of Splice Plates:

$Nfn := n$ $Nfn = 8.00$: Number of bolts along one plane

$Avn := Nwp \cdot (dwp - Nfn \cdot dhole) \cdot twp$ $Avn = 13.50$ in^2

$A85 := 0.85 \cdot Agross\_wp$ $A85 = 18.27$ in^2

$Avn = 13.50$ < $A85 = 18.27$ **O.K.**

$Rr\_web\_fra := 0.80 \cdot (0.58 \cdot Fu \cdot Avn)$

$Vuw = 0.00$ < $Rr\_web\_fra = 407.16$ **O.K.**

Flexural Yielding of Splice Plates:

$Sp1 := \frac{1}{6} \cdot Agross\_wp \cdot dwp$ $Sp1 = 77.04$ in^3

$Mtot\_st\_neg = 1.18 \times 10^3$ kip-in $Hw\_st\_neg = -295.78$ kip

$fst\_neg := \frac{Mtot\_st\_neg}{Sp1} + \frac{|Hw\_st\_neg|}{Agross\_wp}$ $fst\_neg = 29.10$ ksi

$fst\_neg = 29.10$ < $Fy = 50.00$ **O.K.**

Bolts - Bearing at Bolt Holes in Splice Plate:

The clear distance between the edge of the hole and the edge of the splice plate:

$Lc1\_web\_sp := 1.5 - \frac{dhole}{2}$ $\quad Lc1\_web\_sp = 1.00$ in

The clear distance between holes:

$Lc2\_web\_sp := 3.0 \cdot dbolt - dhole$ $\quad Lc2\_web\_sp = 1.63$ in

The clear distance to edge of plate controls.

$Rn\_web\_sp\_bearing := 1.2 \cdot Lc1\_web\_sp \cdot twp \cdot Fu$ $\quad Rn\_web\_sp\_bearing = 39.00$ kips

$Rr\_web\_sp\_bearing := 0.80 \cdot Rn\_web\_sp\_bearing$ $\quad Rr\_web\_sp\_bearing = 31.20$ kips

$\frac{Pr\_st}{2} = 18.07$ $\quad < \quad$ $Rr\_web\_sp\_bearing = 31.20$ **O.K.**

Bolts - Bearing at Bolt Holes in Web:

The edge distance is 2.125 in., leaving 1/2 in between girder field pieces at pier.
The bolt spacing is 3*dbolt= 2.625 in.

The clear distance between the edge of the hole and the edge of the girder:

$Lc1\_web := 2.125 - \frac{dhole}{2}$ $\quad Lc1\_web = 1.63$ in

The clear distance between holes:

$Lc2\_web := 3.0 \cdot dbolt - dhole$ $\quad Lc2\_web = 1.63$ in

The two clear distances are the same.

$Rn\_web\_bearing := 1.2 \cdot Lc1\_web \cdot Tweb \cdot Fu$ $\quad Rn\_web\_bearing = 63.38$ kips

$Rr\_web\_bearing := 0.80 \cdot Rn\_web\_bearing$ $\quad Rr\_web\_bearing = 50.70$ kips

$Pr\_st = 36.15$ $\quad = \quad$ $Rr\_web\_bearing = 50.70$ **O.K.**

Fatigue of Splice Plates:

Nominal stresses at the bottom edge of the splice plates due to the total positive and negative fatigue-load web moments and the coresponding horizontal force resultants:

Case 2 - Negative Live Load:

$Mtot\_fat\_neg = -77.43$ kip-in $\quad Hw\_fat\_neg = -17.11$ kip

$ffat\_neg := \frac{Mtot\_fat\_neg}{Sp1} + \frac{Hw\_fat\_neg}{Agross\_wp}$ $\quad ffat\_neg = -1.80$ ksi $\quad \Delta f\_w\_sp := |ffat\_neg|$ $\quad \Delta f\_w\_sp = 1.80$

$\Delta f\_w\_sp = 1.80$ $\quad < \quad$ $\Delta Fn = 8.00$ **O.K.**

Appendix C. Plastic Moment for Composite Section

1. Procedure

- Calculate the axial force and the moment of the rectangular CFT flange part including the slab in terms of the plastic neutral axis (PNA).
- Combine those results with compression or tension forces of the web and flat tension flange in terms of the PNA.
- Determine the location of the PNA, referenced from the top of the concrete slab, by the equilibrium condition.
- Calculate the plastic moment.

2. Properties and Dimensions (yellow highlight indicates input data)

BRIDGE PARAMETERS

Description: Two span continuous (for superimposed dead load and live load) composite CFTFG with each span of 100 ft and width of 31 ft - 4.5 in. The bridge has 4 girders spaced at 8 ft - 5.5 in with 3 ft overhangs.

Bridge Width (in): $W := 376.5$

Bridge Span Length (in): $L := 1200$

Slab Thickness (in): $Tslab := 8$

Haunch Thickness (in): $Thaunch := 3$

Girder Spacing (in): $s := 101.5$

Number of Girders: $ng := 4$

Overhang (from girder centerline) (in): $se := 36$

Combined Slab and Haunch Thickness (in): $Tconc := Tslab + Thaunch$ $\quad Tconc = 11$

MATERIAL PROPERTIES

Yield Strength (ksi): $Fy := 50$

Young's Modulus (ksi): $E := 29000$

Concrete Strength (ksi): $fc := 4$

Modular ratio (input): $n := 8$

Yield Strain (ksi): $\varepsilon y := \frac{Fy}{E}$

Concrete Modulus (ksi): $Ec := \frac{57000 \cdot \sqrt{fc \cdot 1000}}{1000}$

Concrete Stress Block Parameter: $\beta 1 := 0.85$

Max. Concrete Strain: $\varepsilon u_{slab} := 0.003$

Modular ratio (actual): $\frac{E}{Ec} = 8.04$

STEEL GIRDER DIMENSIONS

Tube horizontal plate thickness: $Tt1 := \frac{3}{8}$ inches

Tube vertical plate thickness: $Tt2 := \frac{3}{8}$ inches

Bottom flange thickness: $Tbf := 1.5$ inches

Web thickness: $Tweb := \frac{8}{16}$ inches

Tube horizontal plate width: $Bt1 := 16$ inches

Tube vertical plate width: $Bt2 := 7.25$ inches

Bottom flange width: $Bbf := 18$ inches

Web depth	$Dweb := 36 - 2 \cdot Tt1 - Bt2 - Tbf$	$Dweb = 26.5$	inches
Total girder depth	$Dgirder := Tbf + Dweb + 2 \cdot Tt1 + Bt2$	$Dgirder = 36$	inches
Depth including deck	$Dtotal := Dgirder + Thaunch + Tslab$	$Dtotal = 47$	inches

COMPOSITE GIRDER PROPERTIES

$Abf := Bbf \cdot Tbf$	$Abf = 27$	in^2	Area of bottom flange
$Atube := 2 \cdot Tt1 \cdot Bt1 + 2 \cdot Tt2 \cdot Bt2$	$Atube = 17.44$	in^2	Area of tube
$Aw := Dweb \cdot Tweb$	$Aw = 13.25$	in^2	Area of web
$Asteel := Aw + Atube + Abf$	$Asteel = 57.69$	in^2	Total steel area

$$Acon := \frac{Bt2 \cdot (Bt1 - 2 \cdot Tt2)}{n} \qquad Acon = 13.82 \quad in^2$$

Equivalent area of concrete in tube (short term)

EFFECTIVE WIDTH OF SLAB (INTERIOR GIRDER)

$$beff1 := \frac{L}{4} \quad beff1 = 300 \qquad beff2 := s \quad beff2 = 101.5 \qquad beff3 := 12 \cdot Tslab + \frac{Bt1}{2} \quad beff3 = 104$$

The smallest beff governs

$$Beffi := \begin{cases} beff1 & \text{if } beff1 \le beff2 \wedge beff1 \le beff3 \\ beff2 & \text{if } beff2 \le beff1 \wedge beff2 \le beff3 \\ beff3 & \text{otherwise} \end{cases} \qquad Beffi = 101.5 \quad in$$

EFFECTIVE WIDTH OF SLAB (EXTERIOR GIRDER)

$$beff4 := \left(\frac{s}{2}\right) + se \qquad beff4 = 86.75$$

The smallest beff governs

$$Beffe := \begin{cases} beff1 & \text{if } beff1 \le beff2 \wedge beff1 \le beff3 \\ beff4 & \text{if } beff4 \le beff1 \wedge beff4 \le beff3 \\ beff3 & \text{otherwise} \end{cases} \qquad Beffe = 86.75 \quad in$$

SELECT EFFECTIVE WIDTH OF SLAB (use Beffi for interior girder, Beffe for exterior girder, or minimum)

$$Beff := \min(Beffe, Beffi) \qquad Beff = 86.75$$

Note: here the minimum is used, which is for an exterior girder.

ELASTIC NEUTRAL AXIS (of transformed section from top of slab)

$$A := \frac{(Bt1 - 2 \cdot Tt2) \cdot Bt2}{n} + Asteel + \left(\frac{1}{n} \cdot Tslab \cdot Beff + \frac{1}{n} \cdot Thaunch \cdot Bt1\right) \qquad A = 164.3 \quad in^2$$

transformed section area of composite girder

$$Num1 := \left[\frac{(Bt1 - 2 \cdot Tt2) \cdot Bt2}{n} \cdot \left(Tconc + Tt1 + \frac{Bt2}{2}\right) + (2 \cdot Bt1 \cdot Tt1 + 2 \cdot Bt2 \cdot Tt2) \cdot \left(Tconc + Tt1 + \frac{Bt2}{2}\right)\right]$$

$$Num2 := \left[Dweb \cdot Tweb \cdot \left(Tconc + 2 \cdot Tt1 + Bt2 + \frac{Dweb}{2}\right) + Bbf \cdot Tbf \cdot \left(Tconc + 2 \cdot Tt1 + Bt2 + Dweb + \frac{Tbf}{2}\right)\right]$$

$$Num3 := \left[\frac{1}{n} \cdot Tslab \cdot Beff \cdot \frac{Tslab}{2} + \frac{1}{n} \cdot Thaunch \cdot Bt1 \cdot \left(Tslab + \frac{Thaunch}{2}\right)\right]$$

$$yo := \frac{Num1 + Num2 + Num3}{A} \qquad yo = 15.52$$

COORDINATES OF CROSS-SECTION ELEMENTS:

Coordinates denoted with "a" are taken from center of CFT compression flange. Upward is positive

$a1 := Tslab + Thaunch + Tt1 + \frac{Bt2}{2}$ **: top face of slab**

$a2 := Thaunch + Tt1 + \frac{Bt2}{2}$ **: bottom face of slab**

$a3 := Tt1 + \frac{Bt2}{2}$ **: outer face of top plate of steel tube**

$a4 := \frac{Bt2}{2}$ **: inner face of top plate of steel tube**

$a5 := -a4$ **: inner face of bottom plate of steel tube**

$a6 := -a3$ **: outer face of bottom plate of steel tube**

$a7(c) := a1 - c$ **: plastic neutral axis (PNA) relative to center of tube (positive indicates PNA is above center of tube)**

$a8(c) := a1 - \beta 1 \cdot c$ **: bottom edge of concrete stress block**

$a9(c) := a1 - c - \frac{\varepsilon y}{\varepsilon u_{slab}} \cdot c$ **: location where yield strain is reached in tension zone of steel (used for tube)**

$a10(c) := a1 - c + \frac{\varepsilon y}{\varepsilon u_{slab}} \cdot c$ **: location where yield strain is reached in compression zone of steel (used for tube)**

Coordinates denoted with "g" are taken from elastic neutral axis (ENA) of section. Upward is positive.

$g1 := yo - Tconc - (2 \cdot Tt1 + Bt2)$ **: top edge of web**

$g2 := yo - Tconc - (2 \cdot Tt1 + Bt2) - Dweb$ **: bottom edge of web**

$g3 := yo - Tconc - (2 \cdot Tt1 + Bt2) - Dweb - Tbf$ **: bottom face of tension flange**

$g4(c) := yo - c - \frac{\varepsilon y}{\varepsilon u_{slab}} \cdot c$ **: location where yield strain is reached in tension zone of steel (used for web)**

$g5(c) := yo - c + \frac{\varepsilon y}{\varepsilon u_{slab}} \cdot c$ **: location where yield strain is reached in compression zone of steel (used for web)**

$g6(c) := yo - c$ **: plastic neutral axis (PNA) relative to ENA (positive indicates PNA is above ENA)**

Functions denoted with "z" give the variation of stress with position on cross section.

$z1(c,y) := \frac{y - a7(c)}{a10(c) - a7(c)} \cdot Fy$ **: stress variations about center of CFT flange (compression is positive, used for tube)**

$z2(c,y) := \frac{c - yo + y}{c - yo + g5(c)} \cdot Fy$ **: stress variations about ENA (compression is positive, used for web)**

3. Assume PNA is in Slab or Haunch

3-1. Rectangular CFT Compression Flange and Deck

Deck

$$\mathrm{Pcon}(c) := \begin{cases} \int_{a8(c)}^{a1} 0.85 \cdot fc \cdot Beff \, dy & \text{if } a8(c) > a2 \\ \int_{a2}^{a1} 0.85 \cdot fc \cdot Beff \, dy & \text{if } a8(c) \le a2 \end{cases}$$

$$\mathrm{Mcon}(c) := \begin{cases} \int_{a8(c)}^{a1} 0.85 \cdot fc \cdot Beff \cdot y \, dy & \text{if } a8(c) > a2 \\ \int_{a2}^{a1} 0.85 \cdot fc \cdot Beff \cdot y \, dy & \text{if } a8(c) \le a2 \end{cases}$$

Top plate of tube

$$\mathrm{Puptube}(c) := \begin{cases} \int_{a4}^{a3} -Bt1 \cdot Fy \, dy & \text{if } a9(c) > a3 \\ \int_{a9(c)}^{a3} Bt1 \cdot z1(c,y) \, dy + \int_{a4}^{a9(c)} -Bt1 \cdot Fy \, dy & \text{if } a4 < a9(c) \le a3 \\ \int_{a4}^{a3} Bt1 \cdot z1(c,y) \, dy & \text{if } a9(c) \le a4 \end{cases}$$

$$\mathrm{Muptube}(c) := \begin{cases} \int_{a4}^{a3} -Bt1 \cdot Fy \cdot y \, dy & \text{if } a9(c) > a3 \\ \int_{a9(c)}^{a3} Bt1 \cdot z1(c,y) \cdot y \, dy + \int_{a4}^{a9(c)} -Bt1 \cdot Fy \cdot y \, dy & \text{if } a4 < a9(c) \le a3 \\ \int_{a4}^{a3} Bt1 \cdot z1(c,y) \cdot y \, dy & \text{if } a9(c) \le a4 \end{cases}$$

Side plates of tube

$$\mathrm{Pmidtube}(c) := \begin{cases} \int_{a5}^{a4} -2 \cdot Tt2 \cdot Fy \, dy & \text{if } a9(c) > a4 \\ \int_{a9(c)}^{a4} 2 \cdot Tt2 \cdot z1(c,y) \, dy + \int_{a5}^{a9(c)} -2 \cdot Tt2 \cdot Fy \, dy & \text{if } a5 < a9(c) \le a4 \\ \int_{a4}^{a3} 2 \cdot Tt2 \cdot z1(c,y) \, dy & \text{if } a9(c) \le a5 \end{cases}$$

$$\mathrm{Mmidtube}(c) := \begin{cases} \int_{a5}^{a4} -2\cdot Tt2\cdot Fy\cdot y\,dy & \text{if } a9(c) > a4 \\ \int_{a9(c)}^{a4} 2\cdot Tt2\cdot z1(c,y)\cdot y\,dy + \int_{a5}^{a9(c)} -2\cdot Tt2\cdot Fy\cdot y\,dy & \text{if } a5 < a9(c) \le a4 \\ \int_{a4}^{a3} 2\cdot Tt2\cdot z1(c,y)\cdot y\,dy & \text{if } a9(c) \le a5 \end{cases}$$

Bottom plate of tube

$$\mathrm{Pdowntube}(c) := \begin{cases} \int_{a6}^{a5} -Bt1\cdot Fy\,dy & \text{if } a9(c) > a5 \\ \int_{a9(c)}^{a5} Bt1\cdot z1(c,y)\,dy + \int_{a6}^{a9(c)} -Bt1\cdot Fy\,dy & \text{if } a9(c) \le a5 \end{cases}$$

$$\mathrm{Mdowntube}(c) := \begin{cases} \int_{a6}^{a5} -Bt1\cdot Fy\cdot y\,dy & \text{if } a9(c) > a5 \\ \int_{a9(c)}^{a5} Bt1\cdot z1(c,y)\cdot y\,dy + \int_{a6}^{a9(c)} -Bt1\cdot Fy\cdot y\,dy & \text{if } a9(c) \le a5 \end{cases}$$

$$\mathrm{Ptopflange}(c) := \mathrm{Pcon}(c) + \mathrm{Puptube}(c) + \mathrm{Pmidtube}(c) + \mathrm{Pdowntube}(c)$$

$$\mathrm{Mtopflange}(c) := \mathrm{Mcon}(c) + \mathrm{Muptube}(c) + \mathrm{Mmidtube}(c) + \mathrm{Mdowntube}(c)$$

3-2. Web and Bottom Flange (assume web and bottom flange are fully yielded in tension)

Web

$$Pw := \int_{g2}^{g1} -Tweb\cdot Fy\,dy$$

$$Mw := \int_{g2}^{g1} -Tweb\cdot Fy\cdot y\,dy$$

Bottom flange

$$Pbf := \int_{g3}^{g2} -Bbf\cdot Fy\,dy$$

$$Mbf := \int_{g3}^{g2} -Bbf\cdot Fy\cdot y\,dy$$

3-3. Combine

$Pp(c) := Ptopflange(c) + Pw + Pbf$

$$Mp(c) := Mtopflange(c) + Ptopflange(c)\cdot\left[yo - Tconc - \left(Tt1 + \frac{Bt2}{2}\right)\right] + Mw + Mbf$$

Assume : $c := Tconc$

$c := root(Pp(c), c)$

Note: this calculation does not converge because the PNA is in the middle region of the tube for an exterior girder. For an interior girder the calculation converges and the PNA is in the haunch.

$Pp(c) = ▮$

$Mp(c) = ▮$ kip-in

Check : If $a9(c) = ▮$ > $a6 = -4$ **then, O.K.** **<== web and bottom flange yield Note that this is a comparison of locations, and therefore the algebraic sign is relevant.**

If 0 < $c = ▮$ < $Tconc = 11$ **then, O.K.** **<== PNA is in the slab or haunch. Otherwise ignore the above calculations.**

$Dweb\_comp := c - Tconc - (2\cdot Tt1 + Bt2)$

$$Dweb\_cp := \begin{cases} Dweb\_comp & \text{if } Dweb\_comp > 0 \\ 0 & \text{otherwise} \end{cases}$$

$Dweb\_cp = ▮$ in **Depth of web in compression**

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4. Assume PNA is in Top Plate of Steel Tube

4-1. Rectangular CFT Compression Flange and Deck

Deck

$$Pcon(c) := \begin{cases} \int_{a8(c)}^{a1} 0.85\cdot fc\cdot Beff\, dy & \text{if } a8(c) > a2 \\ \int_{a2}^{a1} 0.85\cdot fc\cdot Beff\, dy & \text{if } a8(c) \le a2 \end{cases}$$

$$Mcon(c) := \begin{cases} \int_{a8(c)}^{a1} 0.85\cdot fc\cdot Beff\cdot y\, dy & \text{if } a8(c) > a2 \\ \int_{a2}^{a1} 0.85\cdot fc\cdot Beff\cdot y\, dy & \text{if } a8(c) \le a2 \end{cases}$$

Top plate of tube

$$Puptubeten(c) := \begin{cases} \int_{a9(c)}^{a7(c)} Bt1\cdot z1(c,y)\, dy + \int_{a4}^{a9(c)} -Bt1\cdot Fy\, dy & \text{if } a9(c) > a4 \\ \int_{a4}^{a7(c)} Bt1\cdot z1(c,y)\, dy & \text{if } a9(c) \le a4 \end{cases}$$

$$\text{Muptubeten}(c) := \begin{cases} \int_{a9(c)}^{a7(c)} Bt1 \cdot z1(c,y) \cdot y \, dy + \int_{a4}^{a9(c)} -Bt1 \cdot Fy \cdot y \, dy & \text{if } a9(c) > a4 \\ \int_{a4}^{a7(c)} Bt1 \cdot z1(c,y) \cdot y \, dy & \text{if } a9(c) \le a4 \end{cases}$$

$$\text{Puptubecom}(c) := \int_{a7(c)}^{a3} Bt1 \cdot z1(c,y) \, dy$$

$$\text{Muptubecom}(c) := \int_{a7(c)}^{a3} Bt1 \cdot z1(c,y) \cdot y \, dy$$

$$\text{Puptube}(c) := \text{Puptubeten}(c) + \text{Puptubecom}(c)$$

$$\text{Muptube}(c) := \text{Muptubeten}(c) + \text{Muptubecom}(c)$$

Side plates of tube

$$\text{Pmidtube}(c) := \begin{cases} \int_{a5}^{a4} -2 \cdot Tt2 \cdot Fy \, dy & \text{if } a9(c) > a4 \\ \int_{a9(c)}^{a4} 2 \cdot Tt2 \cdot z1(c,y) \, dy + \int_{a5}^{a9(c)} -2 \cdot Tt2 \cdot Fy \, dy & \text{if } a5 < a9(c) \le a4 \\ \int_{a5}^{a4} 2 \cdot Tt2 \cdot z1(c,y) \, dy & \text{if } a9(c) \le a5 \end{cases}$$

$$\text{Mmidtube}(c) := \begin{cases} \int_{a5}^{a4} -2 \cdot Tt2 \cdot Fy \cdot y \, dy & \text{if } a9(c) > a4 \\ \int_{a9(c)}^{a4} 2 \cdot Tt2 \cdot z1(c,y) \cdot y \, dy + \int_{a5}^{a9(c)} -2 \cdot Tt2 \cdot Fy \cdot y \, dy & \text{if } a5 < a9(c) \le a4 \\ \int_{a5}^{a4} 2 \cdot Tt2 \cdot z1(c,y) \cdot y \, dy & \text{if } a9(c) \le a5 \end{cases}$$

Bottom plate of tube

$$\text{Pdowntube}(c) := \begin{cases} \int_{a6}^{a5} -Bt1 \cdot Fy \, dy & \text{if } a9(c) > a5 \\ \int_{a9(c)}^{a5} Bt1 \cdot z1(c,y) \, dy + \int_{a6}^{a9(c)} -Bt1 \cdot Fy \, dy & \text{if } a6 < a9(c) \le a5 \\ \int_{a6}^{a5} Bt1 \cdot z1(c,y) \, dy & \text{if } a9(c) < a6 \end{cases}$$

$$\text{Mdowntube}(c) := \begin{cases} \int_{a6}^{a5} -Bt1 \cdot Fy \cdot y \, dy & \text{if } a9(c) > a5 \\ \int_{a9(c)}^{a5} Bt1 \cdot z1(c,y) \cdot y \, dy + \int_{a6}^{a9(c)} -Bt1 \cdot Fy \cdot y \, dy & \text{if } a6 < a9(c) \le a5 \\ \int_{a6}^{a5} Bt1 \cdot z1(c,y) \cdot y \, dy & \text{if } a9(c) < a6 \end{cases}$$

$$\text{Ptopflange}(c) := \text{Pcon}(c) + \text{Puptube}(c) + \text{Pmidtube}(c) + \text{Pdowntube}(c)$$

$$\text{Mtopflange}(c) := \text{Mcon}(c) + \text{Muptube}(c) + \text{Mmidtube}(c) + \text{Mdowntube}(c)$$

4-2. Web and Bottom Flange (assume web and bottom flange are fully yielded in tension)

Web

$$Pw := \int_{g2}^{g1} -Tweb \cdot Fy \, dy$$

$$Mw := \int_{g2}^{g1} -Tweb \cdot Fy \cdot y \, dy$$

Bottom flange

$$Pbf := \int_{g3}^{g2} -Bbf \cdot Fy \, dy$$

$$Mbf := \int_{g3}^{g2} -Bbf \cdot Fy \cdot y \, dy$$

4-3. Combine

$$Pp(c) := \text{Ptopflange}(c) + Pw + Pbf$$

$$Mp(c) := \text{Mtopflange}(c) + \text{Ptopflange}(c) \cdot \left[yo - Tconc - \left(Tt1 + \frac{Bt2}{2} \right) \right] + Mw + Mbf$$

Assume : $c := Tconc$

$c := \text{root}(Pp(c), c)$

$c = 12.21$

$Pp(c) = -2.27 \times 10^{-13}$

$Mp(c) = 8.1 \times 10^{4}$ kip-in

Note: this calculation does not control because the PNA is in the middle tube region for an exterior girder. For an interior girder the PNA is in the haunch.

Check : **If** $a9(c) = -4.22$ > $a6 = -4$ **then, O.K.** **<== web and bottom flange yield**

If $Tconc = 11$ < $c = 12.21$ < $Tconc + Tt1 = 11.38$ **then, O.K.**

<== PNA is in the top of the steel tube. Otherwise ignore the above calculations.

$$Dweb\_comp := c - Tconc - (2 \cdot Tt1 + Bt2)$$

$$Dweb\_cp := \begin{cases} Dweb\_comp & \text{if } Dweb\_comp > 0 \\ 0 & \text{otherwise} \end{cases}$$

$Dweb\_cp = 0$ in **Depth of web in compression**

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5. Assume PNA is in Middle Region of Steel Tube

5-1. Rectangular CFT Compression Flange and Deck

Deck and concrete in tube

$$\text{Pcon}(c) := \begin{cases} \int_{a8(c)}^{a1} 0.85 \cdot fc \cdot Beff\, dy & \text{if } a8(c) > a2 \\ \int_{a2}^{a1} 0.85 \cdot fc \cdot Beff\, dy & \text{if } a4 < a8(c) \le a2 \\ \int_{a2}^{a1} 0.85 \cdot fc \cdot Beff\, dy + \int_{a8(c)}^{a4} 0.85 \cdot fc \cdot (Bt1 - 2 \cdot Tt2)\, dy & \text{if } a8(c) \le a4 \end{cases}$$

$$\text{Mcon}(c) := \begin{cases} \int_{a8(c)}^{a1} 0.85 \cdot fc \cdot Beff \cdot y\, dy & \text{if } a8(c) > a2 \\ \int_{a2}^{a1} 0.85 \cdot fc \cdot Beff \cdot y\, dy & \text{if } a4 < a8(c) \le a2 \\ \int_{a2}^{a1} 0.85 \cdot fc \cdot Beff \cdot y\, dy + \int_{a8(c)}^{a4} 0.85 \cdot fc \cdot (Bt1 - 2 \cdot Tt2) \cdot y\, dy & \text{if } a8(c) \le a4 \end{cases}$$

Top plate of tube

$$\text{Puptube}(c) := \begin{cases} \int_{a4}^{a3} Bt1 \cdot z1(c, y)\, dy & \text{if } a10(c) > a3 \\ \int_{a10(c)}^{a3} Bt1 \cdot Fy\, dy + \int_{a4}^{a10(c)} Bt1 \cdot z1(c, y)\, dy & \text{if } a4 < a10(c) \le a3 \\ \int_{a4}^{a3} Bt1 \cdot Fy\, dy & \text{if } a10(c) \le a4 \end{cases}$$

$$\text{Muptube}(c) := \begin{cases} \int_{a4}^{a3} Bt1 \cdot z1(c, y) \cdot y\, dy & \text{if } a10(c) > a3 \\ \int_{a10(c)}^{a3} Bt1 \cdot Fy \cdot y\, dy + \int_{a4}^{a10(c)} Bt1 \cdot z1(c, y) \cdot y\, dy & \text{if } a4 < a10(c) \le a3 \\ \int_{a4}^{a3} Bt1 \cdot Fy \cdot y\, dy & \text{if } a10(c) \le a4 \end{cases}$$

Side plates of tube

$$\mathrm{Pmidtubeten}(c) := \begin{cases} \int_{a9(c)}^{a7(c)} 2\cdot Tt2\cdot z1(c,y)\,dy + \int_{a5}^{a9(c)} -2\cdot Tt2\cdot Fy\,dy & \text{if } a9(c) \geq a5 \\ \int_{a5}^{a7(c)} 2\cdot Tt2\cdot z1(c,y)\,dy & \text{if } a9(c) < a5 \end{cases}$$

$$\mathrm{Mmidtubeten}(c) := \begin{cases} \int_{a9(c)}^{a7(c)} 2\cdot Tt2\cdot z1(c,y)\cdot y\,dy + \int_{a5}^{a9(c)} -2\cdot Tt2\cdot Fy\cdot y\,dy & \text{if } a9(c) \geq a5 \\ \int_{a5}^{a7(c)} 2\cdot Tt2\cdot z1(c,y)\cdot y\,dy & \text{if } a9(c) < a5 \end{cases}$$

$$\mathrm{Pmidtubecom}(c) := \begin{cases} \int_{a7(c)}^{a4} 2\cdot Tt2\cdot z1(c,y)\,dy & \text{if } a10(c) > a4 \\ \int_{a10(c)}^{a4} 2\cdot Tt2\cdot Fy\,dy + \int_{a7(c)}^{a10(c)} 2\cdot Tt2\cdot z1(c,y)\,dy & \text{if } a10(c) \leq a4 \end{cases}$$

$$\mathrm{Mmidtubecom}(c) := \begin{cases} \int_{a7(c)}^{a4} 2\cdot Tt2\cdot z1(c,y)\cdot y\,dy & \text{if } a10(c) > a4 \\ \int_{a10(c)}^{a4} 2\cdot Tt2\cdot Fy\cdot y\,dy + \int_{a7(c)}^{a10(c)} 2\cdot Tt2\cdot z1(c,y)\cdot y\,dy & \text{if } a10(c) \leq a4 \end{cases}$$

$$\mathrm{Pmidtube}(c) := \mathrm{Pmidtubeten}(c) + \mathrm{Pmidtubecom}(c)$$

$$\mathrm{Mmidtube}(c) := \mathrm{Mmidtubeten}(c) + \mathrm{Mmidtubecom}(c)$$

Bottom plate of tube

$$\mathrm{Pdowntube}(c) := \begin{cases} \int_{a6}^{a5} -Bt1\cdot Fy\,dy & \text{if } a9(c) > a5 \\ \int_{a9(c)}^{a5} Bt1\cdot z1(c,y)\,dy + \int_{a6}^{a9(c)} -Bt1\cdot Fy\,dy & \text{if } a6 < a9(c) \leq a5 \\ \int_{a6}^{a5} Bt1\cdot z1(c,y)\,dy & \text{if } a9(c) < a6 \end{cases}$$

$$\mathrm{Mdowntube}(c) := \begin{cases} \int_{a6}^{a5} -Bt1\cdot Fy\cdot y\,dy & \text{if } a9(c) > a5 \\ \int_{a9(c)}^{a5} Bt1\cdot z1(c,y)\cdot y\,dy + \int_{a6}^{a9(c)} -Bt1\cdot Fy\cdot y\,dy & \text{if } a6 < a9(c) \leq a5 \\ \int_{a6}^{a5} Bt1\cdot z1(c,y)\cdot y\,dy & \text{if } a9(c) < a6 \end{cases}$$

$Ptopflange(c) := Pcon(c) + Puptube(c) + Pmidtube(c) + Pdowntube(c)$

$Mtopflange(c) := Mcon(c) + Muptube(c) + Mmidtube(c) + Mdowntube(c)$

5-2. Web and Bottom Flange (assume bottom flange is fully yielded in tension, but check web)

Web

$$Pw(c) := \begin{cases} \int_{g2}^{g1} -Tweb \cdot Fy\, dy & \text{if } g4(c) \ge g1 \\ \int_{g4(c)}^{g1} Tweb \cdot z2(c,y)\, dy + \int_{g2}^{g4(c)} -Tweb \cdot Fy\, dy & \text{if } g4(c) < g1 \end{cases}$$

$$Mw(c) := \begin{cases} \int_{g2}^{g1} -Tweb \cdot Fy \cdot y\, dy & \text{if } g4(c) \ge g1 \\ \int_{g4(c)}^{g1} Tweb \cdot z2(c,y) \cdot y\, dy + \int_{g2}^{g4(c)} -Tweb \cdot Fy \cdot y\, dy & \text{if } g4(c) < g1 \end{cases}$$

Bottom flange

$$Pbf := \int_{g3}^{g2} -Bbf \cdot Fy\, dy$$

$$Mbf := \int_{g3}^{g2} -Bbf \cdot Fy \cdot y\, dy$$

5-3. Combine

$Pp(c) := Ptopflange(c) + Pw(c) + Pbf$

$$Mp(c) := Mtopflange(c) + Ptopflange(c) \cdot \left[yo - Tconc - \left(Tt1 + \frac{Bt2}{2}\right)\right] + Mw(c) + Mbf$$

Assume : $c := Tconc + Tt1$

$c := root(Pp(c), c)$ $c = 12.21$

$Pp(c) = 2.27 \times 10^{-13}$

$Mp(c) = 8.0985 \times 10^{4}$ kip-in

Note: this calculation contols for an exterior girder. For an interior girder the PNA is in the haunch.

Check : **If** $g4(c) = -3.7$ > $g2 = -29.98$ **then, O.K.** **<== bottom flange is fully yielded**

If $Tconc + Tt1 = 11.38$ < $c = 12.21$ < $Tconc + Tt1 + Bt2 = 18.63$ **then, O.K.**

<== PNA is in the middle region of the steel tube. Otherwise ignore the above calculations.

$Dweb\_comp := c - Tconc - (2 \cdot Tt1 + Bt2)$

$$Dweb\_cp := \begin{cases} Dweb\_comp & \text{if } Dweb\_comp > 0 \\ 0 & \text{otherwise} \end{cases}$$

$Dweb\_cp = 0$ in **Depth of web in compression**

6. Assume PNA is in Bottom of Steel Tube

6-1. Rectangular CFT Compression Flange Part

Deck and concrete in tube (assumes β*c>a2, check later)

$$\mathrm{Pcon}(c) := \begin{cases} \int_{a2}^{a1} 0.85\cdot fc\cdot Beff\, dy + \int_{a8(c)}^{a4} 0.85\cdot fc\cdot (Bt1 - 2\cdot Tt2)\, dy & \text{if } a8(c) \ge a5 \\ \int_{a2}^{a1} 0.85\cdot fc\cdot Beff\, dy + \int_{a5}^{a4} 0.85\cdot fc\cdot (Bt1 - 2\cdot Tt2)\, dy & \text{if } a8(c) < a5 \end{cases}$$

$$\mathrm{Mcon}(c) := \begin{cases} \int_{a2}^{a1} 0.85\cdot fc\cdot Beff\cdot y\, dy + \int_{a8(c)}^{a4} 0.85\cdot fc\cdot (Bt1 - 2\cdot Tt2)\cdot y\, dy & \text{if } a8(c) \ge a5 \\ \int_{a2}^{a1} 0.85\cdot fc\cdot Beff\cdot y\, dy + \int_{a5}^{a4} 0.85\cdot fc\cdot (Bt1 - 2\cdot Tt2)\cdot y\, dy & \text{if } a8(c) < a5 \end{cases}$$

Top plate of tube

$$\mathrm{Puptube}(c) := \begin{cases} \int_{a4}^{a3} Bt1\cdot z1(c,y)\, dy & \text{if } a10(c) > a3 \\ \int_{a10(c)}^{a3} Bt1\cdot Fy\, dy + \int_{a4}^{a10(c)} Bt1\cdot z1(c,y)\, dy & \text{if } a4 < a10(c) \le a3 \\ \int_{a4}^{a3} Bt1\cdot Fy\, dy & \text{if } a10(c) \le a4 \end{cases}$$

$$\mathrm{Muptube}(c) := \begin{cases} \int_{a4}^{a3} Bt1\cdot z1(c,y)\cdot y\, dy & \text{if } a10(c) > a3 \\ \int_{a10(c)}^{a3} Bt1\cdot Fy\cdot y\, dy + \int_{a4}^{a10(c)} Bt1\cdot z1(c,y)\cdot y\, dy & \text{if } a4 < a10(c) \le a3 \\ \int_{a4}^{a3} Bt1\cdot Fy\cdot y\, dy & \text{if } a10(c) \le a4 \end{cases}$$

$$Pmidtube(c) := \begin{cases} \int_{a5}^{a4} 2\cdot Tt2\cdot z1(c,y)\,dy & \text{if } a10(c) > a4 \\ \int_{a10(c)}^{a4} 2\cdot Tt2\cdot Fy\,dy + \int_{a5}^{a10(c)} 2\cdot Tt2\cdot z1(c,y)\,dy & \text{if } a5 < a10(c) \le a4 \\ \int_{a5}^{a4} 2\cdot Tt2\cdot Fy\,dy & \text{if } a10(c) \le a5 \end{cases}$$

Side plates of tube

$$Mmidtube(c) := \begin{cases} \int_{a5}^{a4} 2\cdot Tt2\cdot z1(c,y)\cdot y\,dy & \text{if } a10(c) > a4 \\ \int_{a10(c)}^{a4} 2\cdot Tt2\cdot Fy\cdot y\,dy + \int_{a5}^{a10(c)} 2\cdot Tt2\cdot z1(c,y)\cdot y\,dy & \text{if } a5 < a10(c) \le a4 \\ \int_{a5}^{a4} 2\cdot Tt2\cdot Fy\cdot y\,dy & \text{if } a10(c) \le a5 \end{cases}$$

$$Pdowntubeten(c) := \begin{cases} \int_{a9(c)}^{a7(c)} Bt1\cdot z1(c,y)\,dy + \int_{a6}^{a9(c)} -Bt1\cdot Fy\,dy & \text{if } a9(c) \ge a6 \\ \int_{a6}^{a7(c)} Bt1\cdot z1(c,y)\,dy & \text{if } a9(c) < a6 \end{cases}$$

Bottom plate of tube

$$Mdowntubeten(c) := \begin{cases} \int_{a9(c)}^{a7(c)} Bt1\cdot z1(c,y)\cdot y\,dy + \int_{a6}^{a9(c)} -Bt1\cdot Fy\cdot y\,dy & \text{if } a9(c) \ge a6 \\ \int_{a6}^{a7(c)} Bt1\cdot z1(c,y)\cdot y\,dy & \text{if } a9(c) < a6 \end{cases}$$

$$Pdowntubecom(c) := \begin{cases} \int_{a7(c)}^{a5} Bt1\cdot z1(c,y)\,dy & \text{if } a10(c) \ge a5 \\ \int_{a10(c)}^{a5} Bt1\cdot Fy\,dy + \int_{a7(c)}^{a10(c)} Bt1\cdot z1(c,y)\,dy & \text{if } a10(c) < a5 \end{cases}$$

$$Mdowntubecom(c) := \begin{cases} \int_{a7(c)}^{a5} Bt1\cdot z1(c,y)\cdot y\,dy & \text{if } a10(c) \ge a5 \\ \int_{a10(c)}^{a5} Bt1\cdot Fy\cdot y\,dy + \int_{a7(c)}^{a10(c)} Bt1\cdot z1(c,y)\cdot y\,dy & \text{if } a10(c) < a5 \end{cases}$$

$Pdowntube(c) := Pdowntubeten(c) + Pdowntubecom(c)$

$Mdowntube(c) := Mdowntubeten(c) + Mdowntubecom(c)$

$Ptopflange(c) := Pcon(c) + Puptube(c) + Pmidtube(c) + Pdowntube(c)$

$Mtopflange(c) := Mcon(c) + Muptube(c) + Mmidtube(c) + Mdowntube(c)$

6-2. Web and Bottom Flange (assume bottom flange is fully yielded, but check web)

Web

$$Pw(c) := \begin{cases} \int_{g2}^{g1} -Tweb \cdot Fy\, dy & \text{if } g4(c) \ge g1 \\ \int_{g4(c)}^{g1} Tweb \cdot z2(c,y)\, dy + \int_{g2}^{g4(c)} -Tweb \cdot Fy\, dy & \text{if } g4(c) < g1 \end{cases}$$

$$Mw(c) := \begin{cases} \int_{g2}^{g1} -Tweb \cdot Fy \cdot y\, dy & \text{if } g4(c) \ge g1 \\ \int_{g4(c)}^{g1} Tweb \cdot z2(c,y) \cdot y\, dy + \int_{g2}^{g4(c)} -Tweb \cdot Fy \cdot y\, dy & \text{if } g4(c) < g1 \end{cases}$$

Bottom flange

$$Pbf := \int_{g3}^{g2} -Bbf \cdot Fy\, dy$$

$$Mbf := \int_{g3}^{g2} -Bbf \cdot Fy \cdot y\, dy$$

6-3. Combine

$Pp(c) := Ptopflange(c) + Pw(c) + Pbf$

$$Mp(c) := Mtopflange(c) + Ptopflange(c) \cdot \left[yo - Tconc - \left(Tt1 + \frac{Bt2}{2}\right)\right] + Mw(c) + Mbf$$

Assume : $c := Tconc + Tt1 + Bt2$

$c := root(Pp(c), c)$ $\quad c = 12.47$

$Pp(c) = -4.55 \times 10^{-13}$

$Mp(c) = 8.08 \times 10^{4}$ kip-in

Note: this calculation does not control because the PNA is in the middle tube region for an exterior girder. For an interior girder the PNA is in the haunch.

Check : **If** $g4(c) = -4.12$ > $g2 = -29.98$ **then, O.K.** **<== bottom flange is totally yielded**

If $a8(c) = 4.4$ > $a2 = 7$ **then, O.K.** **<== β*c>a2 (depth of deck)**

If $Tconc + Tt1 + Bt2 = 18.63$ < $c = 12.47$ < $Tconc + 2 \cdot Tt1 + Bt2 = 19$ **then, O.K.**

<== PNA is in the bottom of the steel tube. Otherwise ignore the above calculations.

$$Dweb\_comp := c - Tconc - (2 \cdot Tt1 + Bt2)$$

$$Dweb\_cp := \begin{cases} Dweb\_comp & \text{if } Dweb\_comp > 0 \\ 0 & \text{otherwise} \end{cases}$$

$Dweb\_cp = 0$ in **Depth of web in compression**

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7. Assume PNA is in Web

7-1. Rectangular CFT Compression Flange and Deck

Deck and concrete in tube (assumes β*c>a2, check later)

$$Pcon(c) := \begin{cases} \int_{a2}^{a1} 0.85 \cdot fc \cdot Beff\, dy + \int_{a8(c)}^{a4} 0.85 \cdot fc \cdot (Bt1 - 2 \cdot Tt2)\, dy & \text{if } a8(c) \ge a5 \\ \int_{a2}^{a1} 0.85 \cdot fc \cdot Beff\, dy + \int_{a5}^{a4} 0.85 \cdot fc \cdot (Bt1 - 2 \cdot Tt2)\, dy & \text{if } a8(c) < a5 \end{cases}$$

$$Mcon(c) := \begin{cases} \int_{a2}^{a1} 0.85 \cdot fc \cdot Beff \cdot y\, dy + \int_{a8(c)}^{a4} 0.85 \cdot fc \cdot (Bt1 - 2 \cdot Tt2) \cdot y\, dy & \text{if } a8(c) \ge a5 \\ \int_{a2}^{a1} 0.85 \cdot fc \cdot Beff \cdot y\, dy + \int_{a5}^{a4} 0.85 \cdot fc \cdot (Bt1 - 2 \cdot Tt2) \cdot y\, dy & \text{if } a8(c) < a5 \end{cases}$$

Top plate of tube

$$Puptube(c) := \begin{cases} \int_{a4}^{a3} Bt1 \cdot z1(c,y)\, dy & \text{if } a10(c) > a3 \\ \int_{a10(c)}^{a3} Bt1 \cdot Fy\, dy + \int_{a4}^{a10(c)} Bt1 \cdot z1(c,y)\, dy & \text{if } a4 < a10(c) \le a3 \\ \int_{a4}^{a3} Bt1 \cdot Fy\, dy & \text{if } a10(c) \le a4 \end{cases}$$

$$Muptube(c) := \begin{cases} \int_{a4}^{a3} Bt1 \cdot z1(c,y) \cdot y\, dy & \text{if } a10(c) > a3 \\ \int_{a10(c)}^{a3} Bt1 \cdot Fy \cdot y\, dy + \int_{a4}^{a10(c)} Bt1 \cdot z1(c,y) \cdot y\, dy & \text{if } a4 < a10(c) \le a3 \\ \int_{a4}^{a3} Bt1 \cdot Fy \cdot y\, dy & \text{if } a10(c) \le a4 \end{cases}$$

Side plates of tube

$$\mathrm{Pmidtube}(c) := \begin{cases} \displaystyle\int_{a5}^{a4} 2\cdot Tt2\cdot z1(c,y)\,dy & \text{if } a10(c) > a4 \\ \displaystyle\int_{a10(c)}^{a4} 2\cdot Tt2\cdot Fy\,dy + \int_{a5}^{a10(c)} 2\cdot Tt2\cdot z1(c,y)\,dy & \text{if } a5 < a10(c) \le a4 \\ \displaystyle\int_{a5}^{a4} 2\cdot Tt2\cdot Fy\,dy & \text{if } a10(c) \le a5 \end{cases}$$

$$\mathrm{Mmidtube}(c) := \begin{cases} \displaystyle\int_{a5}^{a4} 2\cdot Tt2\cdot z1(c,y)\cdot y\,dy & \text{if } a10(c) > a4 \\ \displaystyle\int_{a10(c)}^{a4} 2\cdot Tt2\cdot Fy\cdot y\,dy + \int_{a5}^{a10(c)} 2\cdot Tt2\cdot z1(c,y)\cdot y\,dy & \text{if } a5 < a10(c) \le a4 \\ \displaystyle\int_{a5}^{a4} 2\cdot Tt2\cdot Fy\cdot y\,dy & \text{if } a10(c) \le a5 \end{cases}$$

Bottom plate of tube

$$\mathrm{Pdowntube}(c) := \begin{cases} \displaystyle\int_{a6}^{a5} Bt1\cdot z1(c,y)\,dy & \text{if } a10(c) \ge a5 \\ \displaystyle\int_{a10(c)}^{a5} Bt1\cdot Fy\,dy + \int_{a6}^{a10(c)} Bt1\cdot z1(c,y)\,dy & \text{if } a6 \le a10(c) < a5 \\ \displaystyle\int_{a6}^{a5} Bt1\cdot Fy\,dy & \text{if } a10(c) < a6 \end{cases}$$

$$\mathrm{Mdowntube}(c) := \begin{cases} \displaystyle\int_{a6}^{a5} Bt1\cdot z1(c,y)\cdot y\,dy & \text{if } a10(c) \ge a5 \\ \displaystyle\int_{a10(c)}^{a5} Bt1\cdot Fy\cdot y\,dy + \int_{a6}^{a10(c)} Bt1\cdot z1(c,y)\cdot y\,dy & \text{if } a6 \le a10(c) < a5 \\ \displaystyle\int_{a6}^{a5} Bt1\cdot Fy\cdot y\,dy & \text{if } a10(c) < a6 \end{cases}$$

$$\mathrm{Ptopflange}(c) := \mathrm{Pcon}(c) + \mathrm{Puptube}(c) + \mathrm{Pmidtube}(c) + \mathrm{Pdowntube}(c)$$

$$\mathrm{Mtopflange}(c) := \mathrm{Mcon}(c) + \mathrm{Muptube}(c) + \mathrm{Mmidtube}(c) + \mathrm{Mdowntube}(c)$$

7-2. Web and Bottom Flange Parts (assume bottom flange and web may be partially yielded)

Web

$$Pwten(c) := \begin{cases} \int_{g4(c)}^{g6(c)} Tweb \cdot z2(c,y)\,dy + \int_{g2}^{g4(c)} -Tweb \cdot Fy\,dy & \text{if } g4(c) > g2 \\ \int_{g2}^{g6(c)} Tweb \cdot z2(c,y)\,dy & \text{if } g4(c) \le g2 \end{cases}$$

$$Mwten(c) := \begin{cases} \int_{g4(c)}^{g6(c)} Tweb \cdot z2(c,y) \cdot y\,dy + \int_{g2}^{g4(c)} -Tweb \cdot Fy \cdot y\,dy & \text{if } g4(c) > g2 \\ \int_{g2}^{g6(c)} Tweb \cdot z2(c,y) \cdot y\,dy & \text{if } g4(c) \le g2 \end{cases}$$

$$Pwcom(c) := \begin{cases} \int_{g6(c)}^{g1} Tweb \cdot z2(c,y)\,dy & \text{if } g5(c) > g1 \\ \int_{g5(c)}^{g1} Tweb \cdot Fy\,dy + \int_{g6(c)}^{g5(c)} Tweb \cdot z2(c,y)\,dy & \text{if } g5(c) \le g1 \end{cases}$$

$$Mwcom(c) := \begin{cases} \int_{g6(c)}^{g1} Tweb \cdot z2(c,y) \cdot y\,dy & \text{if } g5(c) > g1 \\ \int_{g5(c)}^{g1} Tweb \cdot Fy \cdot y\,dy + \int_{g6(c)}^{g5(c)} Tweb \cdot z2(c,y) \cdot y\,dy & \text{if } g5(c) \le g1 \end{cases}$$

$$Pw(c) := Pwten(c) + Pwcom(c) \qquad Mw(c) := Mwten(c) + Mwcom(c)$$

Bottom flange

$$Pbf(c) := \begin{cases} \int_{g3}^{g2} -Bbf \cdot Fy\,dy & \text{if } g4(c) > g2 \\ \int_{g4(c)}^{g2} Bbf \cdot z2(c,y)\,dy + \int_{g3}^{g4(c)} -Bbf \cdot Fy\,dy & \text{if } g3 < g4(c) \le g2 \\ \int_{g3}^{g2} Bbf \cdot z2(c,y)\,dy & \text{if } g4(c) \le g3 \end{cases}$$

$$Mbf(c) := \begin{cases} \int_{g3}^{g2} -Bbf \cdot Fy \cdot y \, dy & \text{if } g4(c) > g2 \\ \int_{g4(c)}^{g2} Bbf \cdot z2(c,y) \cdot y \, dy + \int_{g3}^{g4(c)} -Bbf \cdot Fy \cdot y \, dy & \text{if } g3 < g4(c) \le g2 \\ \int_{g3}^{g2} Bbf \cdot z2(c,y) \cdot y \, dy & \text{if } g4(c) \le g3 \end{cases}$$

7-3. Combine

$$Pp(c) := Ptopflange(c) + Pw(c) + Pbf(c)$$

$$Mp(c) := Mtopflange(c) + Ptopflange(c) \cdot \left[yo - Tconc - \left(Tt1 + \frac{Bt2}{2}\right)\right] + Mw(c) + Mbf(c)$$

Assume : $c := Tconc + 2 \cdot Tt1 + Bt2$

$c := root(Pp(c), c)$ $c = 12.47$

$Pp(c) = 1.75 \times 10^{-11}$

$Mp(c) = 8.08 \times 10^{4}$ kip-in

Note: this calculation does not control because the PNA is in the middle tube region for an exterior girder. For an interior girder the PNA is in the haunch.

Check :

If $a8(c) = 4.4$ > $a2 = 7$ **then, O.K.** **<== β*c>a2 (depth of deck)**

If $Tconc + 2 \cdot Tt1 + Bt2 = 19$ < $c = 12.47$ $Tconc + 2 \cdot Tt1 + Bt2 + Dweb = 45.5$ **then, O.K.**

<== PNA is in the web. Otherwise ignore the above calculations

$$Dweb\_comp := c - Tconc - (2 \cdot Tt1 + Bt2)$$

$$Dweb\_cp := \begin{cases} Dweb\_comp & \text{if } Dweb\_comp > 0 \\ 0 & \text{otherwise} \end{cases}$$

$Dweb\_cp = 0$ in **Depth of web in compression**

Appendix D. Yield Moment for Non-composite Section Using Stress Block

1. Conditions and Assumptions

- **The steel is elastic and the concrete in the tube is represented by the appropriate concrete stress block.**
- **c is the location of the elastic neutral axis referenced from the top of the steel tube.**
- **If c is less than Dgirder/2, then the bottom flange will yield first.**
- **If c is greater than Dgirder/2, then the steel tube will yield first.**
- **Compression is positive.**

2. Properties and Dimensions (yellow highlight indicates input data)

MATERIAL PROPERTIES

Yield Strength (ksi): $Fy := 50$

Young's Modulus (ksi): $E := 29000$

Concrete Strength (ksi): $fc := 4$

Modular ratio (input): $n := 8$

Yield Strain (ksi): $\varepsilon y := \frac{Fy}{E}$

Concrete Modulus (ksi): $Ec := \frac{57000 \cdot \sqrt{fc \cdot 1000}}{1000}$

Concrete Stress Block Parameter: $\beta 1 := 0.85$

Max. Concrete Strain: $\varepsilon u_{con} := 0.003$

Modular ratio (actual): $\frac{E}{Ec} = 8.0444$

STEEL GIRDER DIMENSIONS

Tube horizontal plate thickness: $Tt1 := \frac{3}{8}$ inches

Tube vertical plate thickness: $Tt2 := \frac{3}{8}$ inches

Bottom flange thickness: $Tbf := 1.5$ inches

Web thickness: $Tweb := \frac{8}{16}$ inches

Tube horizontal plate width: $Bt1 := 16$ inches

Tube vertical plate width: $Bt2 := 7.25$ inches

Bottom flange width: $Bbf := 18$ inches

Web depth: $Dweb := 36 - 2 \cdot Tt1 - Bt2 - Tbf$ $Dweb = 26.5$ inches

Total girder depth: $Dgirder := Tbf + Dweb + 2 \cdot Tt1 + Bt2$ $Dgirder = 36$ inches

NON-COMPOSITE GIRDER PROPERTIES

$Abf := Bbf \cdot Tbf$ $Abf = 27$ in^2 Area of bottom flange

$Atube := 2 \cdot Tt1 \cdot Bt1 + 2 \cdot Tt2 \cdot Bt2$ $Atube = 17.44$ in^2 Area of tube

$Aw := Dweb \cdot Tweb$ $Aw = 13.25$ in^2 Area of web

$Asteel := Aw + Atube + Abf$ $Asteel = 57.69$ in^2 Total steel area

$Acon := \frac{Bt2 \cdot (Bt1 - 2 \cdot Tt2)}{n}$ $Acon = 13.8203$ in^2 Equivalent area of concrete in tube (short term)

$A := Acon + Asteel$ $A = 71.5$ in^2 Transformed area of non-composite girder including concrete in tube

ELASTIC NEUTRAL AXIS (of non-composite section including concrete in tube from top of steel tube)

$$Num1 := \left[\frac{(Bt1 - 2\cdot Tt2)\cdot Bt2}{n}\cdot\left(Tt1 + \frac{Bt2}{2}\right) + (2\cdot Bt1\cdot Tt1 + 2\cdot Bt2\cdot Tt2)\cdot\left(Tt1 + \frac{Bt2}{2}\right)\right]$$

$$Num2 := \left[Dweb\cdot Tweb\cdot\left(2\cdot Tt1 + Bt2 + \frac{Dweb}{2}\right) + Bbf\cdot Tbf\cdot\left(2\cdot Tt1 + Bt2 + Dweb + \frac{Tbf}{2}\right)\right]$$

$$yo := \frac{Num1 + Num2}{A} \qquad yo = 18.9957$$

COORDINATES OF CROSS-SECTION ELEMENTS:

Coordinates denoted with "a" are taken from center of CFT compression flange. Upward is positive

$a1 := Tt1 + \frac{Bt2}{2}$ **: outer face of top plate of steel tube**

$a2 := \frac{Bt2}{2}$ **: inner face of top plate of steel tube**

$a3 := -a2$ **: inner face of bottom plate of steel tube**

$a4 := -a1$ **: outer face of bottom plate of steel tube**

$a5 := -\left(Tt1 + \frac{Bt2}{2}\right) - Dweb$ **: bottom edge of web**

$a6 := -\left(Tt1 + \frac{Bt2}{2}\right) - Dweb - Tbf$ **: bottom face of tension flange**

$a7(c) := a2 - \beta 1\cdot(c - Tt1)$ **: bottom edge of concrete stress block**

$a8(c) := a1 - c$ **: neutral axis (c) referenced from the center of the concrete filled steel tube**

Functions denoted with "z" give the variation of stress with position on cross section (compression is positive).

$z1(c,y) := \frac{y - a8(c)}{Dgirder - c}\cdot Fy$ **==> Case 1 : yield of the bottom flange first.**

$z2(c,y) := \frac{y - a8(c)}{c}\cdot Fy$ **==> Case 2 : yield of the steel tube first.**

$$z(c,y) := \begin{cases} z1(c,y) & \text{if } c \le \frac{Dgirder}{2} \\ z2(c,y) & \text{otherwise} \end{cases}$$

3. Calculation of Yield Moment

Stress block for concrete in tube

$$Pc(c) := \begin{cases} \int_{a3}^{a2} 0.85\cdot fc\cdot(Bt1 - 2\cdot Tt2)\,dy & \text{if } a7(c) < a3 \\ \int_{a7(c)}^{a2} 0.85\cdot fc\cdot(Bt1 - 2\cdot Tt2)\,dy & \text{otherwise} \end{cases}$$

$$Mc(c) := \begin{cases} \int_{a3}^{a2} 0.85\cdot fc\cdot(Bt1 - 2\cdot Tt2)\cdot y\,dy & \text{if } a7(c) < a3 \\ \int_{a7(c)}^{a2} 0.85\cdot fc\cdot(Bt1 - 2\cdot Tt2)\cdot y\,dy & \text{otherwise} \end{cases}$$

Top plate of steel tube

$$Put(c) := \int_{a2}^{a1} Bt1\cdot z(c,y)\,dy \qquad Mut(c) := \int_{a2}^{a1} Bt1\cdot z(c,y)\cdot y\,dy$$

Side plates of steel tube

$$Pmt(c) := \int_{a3}^{a2} 2\cdot Tt2\cdot z(c,y)\,dy \qquad Mmt(c) := \int_{a3}^{a2} 2\cdot Tt2\cdot z(c,y)\cdot y\,dy$$

Bottom plate of steel tube

$$Pdt(c) := \int_{a4}^{a3} Bt1\cdot z(c,y)\,dy \qquad Mdt(c) := \int_{a4}^{a3} Bt1\cdot z(c,y)\cdot y\,dy$$

Web

$$Pw(c) := \int_{a5}^{a4} Tweb\cdot z(c,y)\,dy \qquad Mw(c) := \int_{a5}^{a4} Tweb\cdot z(c,y)\cdot y\,dy$$

Bottom flange

$$Pbf(c) := \int_{a6}^{a5} Bbf\cdot z(c,y)\,dy \qquad Mbf(c) := \int_{a6}^{a5} Bbf\cdot z(c,y)\cdot y\,dy$$

$$Py(c) := Pc(c) + Put(c) + Pmt(c) + Pdt(c) + Pw(c) + Pbf(c)$$

$$My(c) := Mc(c) + Mut(c) + Mmt(c) + Mdt(c) + Mw(c) + Mbf(c)$$

Assume : $c := yo$

$c := root(Py(c), c)$ $c = 19.9839$ in

$Py(c) = 0$ (should be zero)

$My(c) = 3.5401 \times 10^4$ kip-in

$My := My(c)$

$My = 35401$ kip-in

4. Check Results and Calculate Stresses

If $c = 19.9839$ **<** $\frac{Dgirder}{2} = 18$ **then, bottom flange yields first.**

If $c = 19.9839$ **>** $\frac{Dgirder}{2} = 18$ **then, steel tube yields first.**

$fstopatyield := z(c, a1)$ $fsbottomatyield := z(c, a6)$

$fstopatyield = 50.00$ **==> stress of top fiber of steel tube**

$fsbottomatyield = -40.07$ **==> stress of bottom fiber of bottom flange**

5. Calculate Stresses due to M_{DC} (unfactored) for Non-Composite Section Using Stress Block

$MDC := 18720$ kip-in (from Appendix A) $My = 3.5401 \times 10^4$ kip-in

"z" gives the variation of stress with position on cross section as function of stress on bottom flange.

$$z(c, y, fs) := \frac{y - a8(c)}{Dgirder - c} \cdot fs$$

==> Based on absolute value of stress of bottom flange (fs)

Express corresponding forces to determine neutral axis location and bottom flange stress.

Stress block for concrete in tube

$$Pc(c, fs) := \begin{cases} \int_{a3}^{a2} 0.85 \cdot fc \cdot (Bt1 - 2 \cdot Tt2)\, dy & \text{if } a7(c) < a3 \\ \int_{a7(c)}^{a2} 0.85 \cdot fc \cdot (Bt1 - 2 \cdot Tt2)\, dy & \text{otherwise} \end{cases}$$

$$Mc(c, fs) := \begin{cases} \int_{a3}^{a2} 0.85 \cdot fc \cdot (Bt1 - 2 \cdot Tt2) \cdot y\, dy & \text{if } a7(c) < a3 \\ \int_{a7(c)}^{a2} 0.85 \cdot fc \cdot (Bt1 - 2 \cdot Tt2) \cdot y\, dy & \text{otherwise} \end{cases}$$

Top plate of steel tube

$$Put(c, fs) := \int_{a2}^{a1} Bt1 \cdot z(c, y, fs)\, dy \qquad Mut(c, fs) := \int_{a2}^{a1} Bt1 \cdot z(c, y, fs) \cdot y\, dy$$

Side plates of steel tube

$$Pmt(c, fs) := \int_{a3}^{a2} 2 \cdot Tt2 \cdot z(c, y, fs)\, dy \qquad Mmt(c, fs) := \int_{a3}^{a2} 2 \cdot Tt2 \cdot z(c, y, fs) \cdot y\, dy$$

Bottom plate of steel tube

$$Pdt(c, fs) := \int_{a4}^{a3} Bt1 \cdot z(c, y, fs)\, dy \qquad Mdt(c, fs) := \int_{a4}^{a3} Bt1 \cdot z(c, y, fs) \cdot y\, dy$$

Web

$$Pw(c, fs) := \int_{a5}^{a4} Tweb \cdot z(c, y, fs)\, dy \qquad Mw(c, fs) := \int_{a5}^{a4} Tweb \cdot z(c, y, fs) \cdot y\, dy$$

Bottom flange

$$Pbf(c, fs) := \int_{a6}^{a5} Bbf \cdot z(c, y, fs)\, dy \qquad Mbf(c, fs) := \int_{a6}^{a5} Bbf \cdot z(c, y, fs) \cdot y\, dy$$

Assume neutral axis location and bottom flange stress.

$c := yo$ $\qquad$ $fs := Fy$

Solve for neutral axis location and bottom flange stress.

Given

$Pc(c,fs) + Put(c,fs) + Pmt(c,fs) + Pdt(c,fs) + Pw(c,fs) + Pbf(c,fs) = 0$ **<== total axial force = 0**

$Mc(c,fs) + Mut(c,fs) + Mmt(c,fs) + Mdt(c,fs) + Mw(c,fs) + Mbf(c,fs) = MDC$ **<== given moment due to DC**

$vec := Find(c,fs) \qquad vec = \begin{pmatrix} 16.2809 \\ 20.3723 \end{pmatrix}$

$c := vec_0$ $\qquad$ $c = 16.2809$ in **: Neutral axis using stress block from top fiber of tube**

$fs := vec_1$ $\qquad$ $fs = 20.3723$ ksi **: Stress of bottom flange (absolute value)**

$fstopMDC := z(c,a1,fs)$ $\qquad$ $fstopMDC = 16.8201$ ksi **: stress in top fiber of steel tube**

$fsbottomMDC := z(c,a6,fs)$ $\qquad$ $fsbottomMDC = -20.3723$ ksi **: stress in bottom fiber of bottom flange**

If these values are greater than Fy, then the above procedure is incorrect and the design must be modified because the section is yielding under MDC.