Project No. NCHRP 12-52 **COPY NO. _____**

AASHTO-LRFD DESIGN EXAMPLE
HORIZONTALLY CURVED STEEL BOX GIRDER BRIDGE

FINAL REPORT

Prepared for
National Cooperative Highway Research Program
Transportation Research Board
National Research Council

John M. Kulicki
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Christopher Smith
Kevin Johns
Modjeski and Masters, Inc.
Harrisburg, Pennsylvania

October 2005

ACKNOWLEDGMENT OF SPONSORSHIP

This work was sponsored by the American Association of State Highway and Transportation Officials, in cooperation with the Federal Highway Administration, and was conducted in the National Cooperative Highway Research Program which is administered by the Transportation Research Board of the National Research Council.

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

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PREFACE

AASHTO first published *Guide Specifications for Horizontally Curved Highway Bridges* in 1980. These Guide Specifications included Allowable Stress Design (ASD) provisions developed by the Consortium of University Research Teams (CURT) and approved by ballot of the AASHTO Highway Subcommittee on Bridges and Structures in November 1976. CURT consisted of Carnegie-Mellon University, the University of Pennsylvania, the University of Rhode Island and Syracuse University. The 1980 Guide Specifications also included Load Factor Design (LFD) provisions developed in American Iron and Steel Institute (AISI) Project 190 and approved by ballot of the AASHTO Highway Subcommittee on Bridges and Structures (HSCOBS) in October 1979. The Guide Specifications covered both I and box girders.

Changes to the 1980 Guide Specifications were included in the AASHTO *Interim Specifications - Bridges* for the years 1981, 1982, 1984, 1985, 1986, and 1990. A new version of the *Guide Specifications for Horizontally Curved Highway Bridges* was published in 1993. It included these interim changes, and additional changes, but did not reflect the extensive research on curved-girder bridges that has been conducted since 1980 or many important changes in related provisions of the straight-girder specifications.

As a result of the research work on curved bridges conducted by the FHWA and several research institutes, design provisions for both straight and curved bridges were developed. As part of the NCHRP 12-52 project, these design provisions were incorporated into the AASHTO-LRFD Bridge Design Specifications in two stages. The design provisions for straight bridges were approved by ballot of the HSCOBS in 2003 and were incorporated into the third edition of the AASHTO-LRFD Bridge Design Specifications, published in 2004. The design provisions for curved bridges were approved by ballot of the HSCOBS in 2004 and are to be published as part of the 2005 Interim Specifications to the AASHTO-LRFD Bridge Design Specifications.

This Horizontally Curved Steel Box Girder Bridge Design Example was originally developed in the NCHRP 12-38 project using the 1993 AASHTO Guide Specifications for Horizontally Curved Steel Girder Bridges. It was updated to illustrate the applicability of the revisions to the AASHTO-LRFD Bridge Design Specifications included in the 2005 Interim Specifications which were meant to incorporate curved bridges. As in the NCHRP 12-38 example, a composite bottom flange option is provided for the bottom flange in the negative moment regions. This Design Example was compiled as a part of the deliverables in National Cooperative Highway Research Program Project 12-52.

The following terms are used to identify particular specifications:

- ANSI/AASHTO/AWS refers to the 2002 edition of D1.5:2002 *Bridge Welding Code,* American Welding Society and 2003 Interim Specifications,

- LFD/ASD refers to the current year AASHTO *Standard Specifications for Highway Bridges*, 17th edition and Interim Specifications and

- LRFD refers to the 2003 AASHTO-*LRFD Bridge Design Specifications*, Third Edition, with the 2005 Interims. Article and equation numbers in this example refer to those of the AASHTO-LRFD Specifications.

OBJECTIVES

Using the 2004 AASHTO-LRFD Bridge Design Specifications with the 2005 Interim Specifications (hereafter referred to as the LRFD Specifications), design a three-span horizontally curved steel box girder bridge with two tub girders in the cross section.

DESIGN PARAMETERS

The bridge has spans of 160-210-160 feet measured along the centerline of the bridge. Span lengths are arranged to give relatively equal positive dead load moments in the end and center spans.

The radius of the bridge is 700 feet at the center of the roadway.

Out-to-out deck width is 40.5 feet. There are three 12-foot traffic lanes. Supports are radial with respect to the roadway. There are two tub girders in the cross section.

Structural steel having a specified minimum yield stress of 50 ksi is used throughout. The deck is conventional cast-in-place concrete with a specified minimum 28-day compressive strength of 4,000 psi. The structural deck thickness is 9.5 inches (no integral wearing surface is assumed). The deck haunch is 4.0 inches deep measured from the top of the web to the bottom of the deck. The width of the haunch is assumed to be 20.0 inches. A future wearing surface of 30 psf is specified. Parapets are each assumed to weigh 495 plf.

The roadway is superelevated 5 percent.

Live load used is the HL-93. Live load for fatigue is taken as defined in Article 3.6.1.4 of the LRFD Specifications. The bridge is designed for a 75-year fatigue life. The bridge site is assumed to be located in earthquake Zone A so earthquake loading need not be considered.

Sequential placement of the concrete deck is considered. Permanent steel deck forms are assumed to be used between the two girders and between the flanges of the individual tubs; the forms are assumed to weigh 15 psf.

STEEL FRAMING

The steel framing consists of two trapezoidal tub girders with the tops of the webs in each tub spaced 10 feet apart at the top of the tub and with a clear deck span of 12.5 feet between the top of the interior webs of the two tubs. The cross section is shown in Figure 1. Two bearings set one foot inside of each web are used under each box at each support, as permitted in Article 6.11.1.2.4.

Girder Depth

For I-beams Article 2.5.2.6.3 provides for a preferred minimum depth limit of 0.04 of the span of the girder, L, for simple spans and 0.032L for continuous spans. There is no explicit limit given for steel box beams. The longest effective span length (either end or interior span) controls. The length of the center span of the outside girder, G2, is 213.38 feet (measured along the longitudinal centerline of the box), which is the girder with the longest effective span in this example. Therefore, the recommended girder depth is computed as 0.032(213.38)(12) = 81.9 in. The actual vertical web depth is 78 inches, which is slightly less than the preferred minimum depth. However, box girders are generally stiffer than I-girders because an individual box nearly acts as two I-girders for vertical bending. For torsion, an individual box girder is significantly stiffer than two I-girders.

The slope of the webs is one-on-four, which is the limit given in Article 6.11.2.1.1. As a result, the width of the bottom flange of each tub is 81 inches between webs. The actual box flange width is 83 inches to provide a 1-inch lip outside of each web, which is needed for welding of the webs to the bottom flange.

Internal and External Bracing

The boxes are braced internally at intermediate locations with K-frames. The internal K-frames are spaced longitudinally at approximately 16 feet (measured along the centerline of the bridge). At locations where a longitudinal flange stiffener is not used, the transverse bracing members are attached to the bottom flange. At these locations, the bottom strut of the K-frame is assumed in this example to be welded to the bottom flange and bolted to the connection plates on the webs. At locations where a longitudinal flange stiffener is used, the bottom strut is assumed to be bolted to the top of the longitudinal stiffener and to the connection plates on the webs. The cross frames are assumed to be single-angle members bolted to connection plates. The working points are assumed to be located as close to the flange-web intersections as practical, except where the longitudinal flange stiffening causes the bracing to be offset from the flange.

Design of the internal cross bracing members is not shown in this example. It was determined from the analysis that the largest factored load in any of the internal cross frame members on the bridge is 80 kips in the diagonal members located at Nodes 11 and 12 in Span 1. Cross frame members were modeled as truss members in the analysis, with a cross-sectional area of 5.0 square inches. Article 6.7.4.3 specifies that the cross-sectional area and stiffness of the top and bottom transverse bracing members not be smaller than the area and stiffness of the diagonal members. In addition, at locations where a longitudinal flange stiffener is present, the moment of inertia of the transverse bracing member should equal or exceed the moment of inertia of the longitudinal stiffener taken about the base of the stiffener.

The largest range of stress due to fatigue loading in the internal cross frames was found to be approximately 15 ksi. This maximum stress range was determined by passing the factored fatigue truck defined in Article 3.6.1.4 over the left and right web of a tub, resulting in a reversal of stress in each member. The fatigue stress range, i.e. sum of the absolute values of the maximum tensile and compressive stresses, was approximately 15 ksi. According to Table 3.4.1-1, only 75 percent of the stress range so determined is used to check fatigue for transverse members. Thus, the design fatigue stress range is approximately 11 ksi. The fatigue category of these member end connections is Category E. The design stress range exceeds the nominal fatigue resistance of 2.25 ksi specified for a Category E detail according to Article 6.6.1.2. The value of 2.25 ksi is equal to one-half of the constant-amplitude fatigue threshold of 4.5 ksi specified for a Category E detail in Table 6.6.1.2.5-3 of AASHTO-LRFD. This value is used whenever the fatigue strength is governed by the constant-amplitude fatigue threshold, which is assumed to be the case in this example. Since the design fatigue stress range exceeds the nominal fatigue resistance for a fatigue Category E detail, fillet welds cannot be used for these member connections in this particular case.

As required in Article 6.7.4.3, there are full-depth internal and external diaphragms provided at support lines, but there are no other external braces provided between the boxes in this example. For the analysis, the diaphragm plates for both the internal and external diaphragms were assumed to be 0.5 inches thick. The external diaphragms were assumed to have top and bottom flanges with an area of 8.0 square inches for each flange.

Bracing of Tub Flanges

The top flanges of the individual tubs are braced with single members placed diagonally between the tub flanges. Figure 2 shows the arrangement of the top diagonal bracing in each girder. Figure 2 also gives the node numbers for part of Span 1 so that the locations can be related to subsequent sample design calculations given in Appendix D. The bracing is assumed to be directly connected to the flanges at each internal cross frame, i.e. in the plane of the flange, as required in Article 6.7.5.1. These top flange bracing members provide torsional continuity to the box before the deck cures, and therefore, must have adequate resistance to resist the torsional shear flow in the non-composite section at the constructibility limit state. One end of each internal cross frame does not have lateral bracing attached. The tub flanges tend to develop larger lateral flange bending stresses at the points where the lateral bracing is not connected because the top flange must provide the majority of the torsional resistance. Top flange bracing should be continuous along the length of the girder to ensure that the top flanges are not required to resist the entire torsion at any one location.

There are several causes of the lateral moments in the top flanges including curvature, inclination of the webs and overhang bracket loads. The effect of curvature can be conservatively estimated using Equation C4.6.1.2.4b-1. The inclination of the webs causes a radial force, which must be resisted by the flanges. On the exterior of the bridge, a portion of the deck weight is applied to overhang brackets, which results in a radial tensile force on the outside top flanges and an opposite force on the bottom flange.

The single top flange lateral bracing members used in the design example cause the lateral flange moments to vary depending on whether or not the brace is connected to an interior or exterior flange. To illustrate, both single-diagonal and double-diagonal (or X) top-flange bracing arrangements were analyzed using a 3D finite-element model assuming inclined webs. The lateral flange moments in the two top flanges, and in some cases, the forces in the top flange bracing members in part of Span 1 due to the entire deck weight and Cast #1 (with the effect of the overhang brackets considered in each case) are reported in Figures 3 through 5. Half of the overhang weight was assumed to be applied to the brackets in the analysis, as shown in Figure D-1 (Appendix D). In Figures 3 through 5, the lateral flange moments are shown above and below the top flanges of each girder, whereas the axial forces in the top chord of the internal K-braces and in the top lateral bracing are shown near the appropriate members. Note that the inverted K-bracing inside the boxes results in two top chord members across the tub in the finite-element model. Figure 3 shows the results for the case of the entire deck weight applied to the boxes and overhang brackets assuming double top flange lateral bracing and inclined webs. Figure 4 shows similar results for the case assumed in the design example (single-diagonal top flange lateral bracing and inclined webs) under the loading due to the entire deck weight. Finally, Figure 5 shows the results due to Cast #1 for the single-diagonal bracing case with inclined webs (again the case assumed in the design example). This loading case causes larger girder moments and bracing forces in Span 1 than does the entire deck load because the load in Span 2 tends to counter the load in Span 1.

From examination of the results shown in Figures 3 through 5, the single-diagonal bracing pattern chosen for the design example results in the largest lateral flange bending moments and bracing member forces. While these effects are reduced somewhat when double-diagonal bracing is utilized, additional bracing members and connections are required. A suggested solution is to utilize parallel single-diagonal bracing members in each bay, which would result in lower lateral flange bending moments in combination with fewer members and connections.

Longitudinal Flange Stiffener

A single longitudinal flange stiffener is used on the box flanges over the negative moment sections. The longitudinal stiffener is terminated at the bolted field splices in Spans 1, 2 and 3. By terminating the longitudinal flange stiffener at the bolted splices, there is no need to consider fatigue at the terminus of the stiffener. The bottom flange splice plates inside the box must be split to permit the stiffener to extend to the free edge of the flange where the longitudinal stress is zero, as shown in Figure D-6 (Appendix D). The weight and stiffness of the longitudinal flange stiffeners is considered in the analysis.

Field Sections

Final girder field sections for each girder are given in Appendix A. The longest field section, the center field section in Girder 2, is approximately 122 feet in length.

ANALYSES

Loading Combinations

Article 3.4 is used to determine load combinations for strength. Strength I loading is used for design of most members for the strength limit state. For temperature and wind loadings in combination with vertical loading, Strength III and V and Service I and II from Table 3.4.1-1 must also be checked. These load groups are defined as follows:

Strength I	η x [1.25(DC) + 1.5(DW) + 1.75((LL + IM) + CE + BR) + 0.5(TU)]
Strength III	η x [1.25(DC) + 1.5(DW) + 1.4(WS) + 0.5(TU)]
Strength V	η x [1.25(DC) + 1.5(DW) + 1.35((LL+IM)+CE+BR) + 0.4(WS)+WL+0.5(TU)]
Service I	η x [DC + DW + (LL+IM) + CE +0.3(WS) + WL + 1.0(TU)]
Service II	η x [DC + DW + 1.3((LL + IM) + CE) + 1.0(TU)]

where:

η	=	Load modifier specified in Article 1.3.2
DC	=	Dead load: components and attachments
DW	=	Dead load: wearing surface and utilities
LL	=	Vehicular live load
IM	=	Vehicular dynamic load allowance
CE	=	Vehicular centrifugal force
WS	=	Wind load on structure
WL	=	Wind on live load
TU	=	Uniform temperature
BR	=	Vehicular braking force

In addition to the above load combinations, the AASHTO-LRFD Specifications include a load combination for the constructibility limit state defined in Article 3.4.2 as follows:
Construction: η x [1.25(D) + 1.5(C) + 1.25(W_c)]

where:

D	=	Dead load
C	=	Construction loads
W_c	=	Wind load for construction conditions from an assumed critical direction. Magnitude of wind may be less than that used for final bridge design.

It has been assumed that there is no equipment on the bridge during construction and the wind load on the girders is negligible.

In this example, only the Strength I and Construction load combinations are checked. Other load cases may be critical, but for simplicity, these other load cases are not considered in this example. Selected analysis results for these two load combinations are given in Tables C-2 and C-4, Appendix C. Table C-2 gives the Strength I shears for Girder 2 at the tenth points of Span 1. Table C-4 gives the Strength I and Construction top flange bracing forces in Span 1 of Girder 2.

Three-Dimensional Finite Element Analyses

Article 4.4 requires that the analysis be performed using a rational method that accounts for the interaction of the entire superstructure. Small-deflection elastic theory is acceptable.

Analyses for this example are performed using a three-dimensional finite element program. The section depth is recognized. Girder webs and bottom flanges are modeled using plate elements. Top flanges are modeled with beam elements. Curvature is represented by straight elements with small kinks at node points rather than by curved elements.

The composite deck is modeled using a series of eight-node solid elements attached to the girders with beam elements, which represent the shear studs.

Bearings are represented by dimensionless elements called "foundation elements", which attach from a lower girder node to the "earth".

Cross frames are modeled as individual truss elements connected to the nodes at the top and bottom of the girders. Internal solid-plate diaphragms at supports are modeled with a single plate element and external solid-plate diaphragms at supports are modeled utilizing three full depth plate elements along the length for the web and three beam elements placed at the top and bottom of the web representing the top and bottom flanges of the diaphragm. Since the plate and beam elements are isoparametric, three elements are used to model the web and flanges of the external diaphragm to allow for the possibility of reverse curvature.

LOADS

Dead Load

The self weight of the steel girders and attachments, e.g. cross-frames and bracing, is applied to the erected steel structure. Steel weight is introduced into the 3D model by the use of body forces in the 3D finite elements. This analysis assumption requires that the steel be fit and erected in the no-load condition. The steel may be fit up by the fabricator prior to shipping. Erection without introduction of significant gravity induced stresses until the erection is completed is the responsibility of the steel erector. Falsework or multiple cranes may be required to support the girders until all the bolted connections are tightened.

The deck weight is also assumed to be placed at one time on the non-composite steel structure for the strength limit state checks. Deck weight includes the deck, concrete haunches and an assumed weight of 15 pounds per square foot for the permanent deck forms inside the boxes and between the boxes.

The superimposed dead load includes the parapets and an assumed future wearing surface of 30 pounds per square foot of roadway. The total superimposed dead load is assumed to be applied to the composite structure. The parapet weight is applied as line loads along the edges of the deck in the 3D analysis. Creep of the concrete deck is accounted for by using a modular ratio of "3n" in computing the transformed composite section properties, which produces larger stresses in the steel. The use of composite section properties computed using a modular ratio of "n" results in larger stresses in the concrete deck.

Dead load moments, shears and torques from the 3D analysis are given in Appendix B. Future wearing surface moments, shears and torques were calculated separately.

Live Load

Analysis for live load is accomplished by first applying a series of unit vertical loads, one at a time, to the deck surface in the 3D model. Numerous responses are determined for each unit load, including girder moments, shears, torques, deflections, reactions, cross frame forces, etc. The magnitude of the response for a particular unit load is the magnitude of the ordinate of the influence surface for that response at the point on the deck where that unit load is applied. Curve fitting is used to determine responses between points on the influence surfaces. The specified live loads are applied mathematically to each influence surface and a search is then made to determine the maximum and minimum value of each response for each live load position. The dynamic load allowance is applied according to Article 3.6.2. The multiple presence factors are considered. The effects of the centrifugal forces are not considered in this example. For additional information on the centrifugal force calculations, refer to Appendix D of the Horizontally Curved Steel I-Girder Design Example.

Unfactored live load plus the dynamic load allowance moments, shears and torques in each girder for LRFD HL-93 loading from the 3D analysis are also given in Appendix B.

LIMIT STATES

Strength

For the strength limit state, each component of the boxes is designed to ensure the component has adequate strength to resist the actions due to the factored loads. In reality, stresses or forces in the elements are factored so that the loads can be applied to the model or to the influence surfaces without factors in the analysis.

Constructibility

For the constructibility limit state, a check is made only with regard to placement of the concrete in this example. For this check, the deck is assumed to be placed in four separate casts. All casts are assumed to be made across the entire deck width. The first cast is in Span 1 from the beginning of the span through member 13 in Girder 1 (refer to Appendix A and Figure 2 for the location of the indicated members). The second cast is in Span 2 starting over member 23 through member 38. The third cast is in Span 3 starting over member 48 to the end of the bridge. The fourth cast is for the remaining sections over the piers. This sequence assures that uplift does not occur at any time and that the girder stresses and deflections are within the prescribed limits in Article 6.10.3.2. Shorter casts over the piers would have led to uplift and larger moments in Span 1. Larger top flange plates and perhaps a thicker web may have been required, as well as counter weights over some supports, to prevent uplift.

The unfactored moments from the deck staging analysis are presented in Table C-1, Appendix C. "Steel" identifies moments due to the steel weight based on the assumption that it was placed at one time; "Deck" identifies moments due to the deck weight assumed to be placed on the bridge at one time; "Cast" identifies the moments due to a particular deck cast; "SupImp" identifies the moments due to the superimposed dead load placed on the fully composite bridge; and “FWS” identifies the moments due to the future wearing surface placed on the fully composite bridge. Included in the "Deck" and "Cast" moments are the moments due to the deck haunch and the stay-in-place forms.

Reactions are accumulated sequentially in the staging analysis to check for uplift at each stage. Accumulated deflections by stage are also computed. In each analysis of the deck placement, prior casts are assumed to be composite. The modular ratio for the deck is assumed to be “3n” to account for creep. A somewhat smaller modular ratio may be desirable for the staging analysis since full creep usually takes approximately three years to occur. A modular ratio of “n” should be used to check deck stresses since a smaller modular ratio results in higher stresses in the deck. Moments and other actions determined from the deck-staging analysis are not considered for the strength limit state checks.

Fatigue

The fatigue limit state is checked by using the stress ranges due to the passage of one fatigue vehicle, defined in Article 3.6.1.4, traversing the length of the bridge in the critical transverse position on the deck for each response. The load factor is 0.75 for the fatigue truck, as specified in

Table 3.4.1-1. The dynamic load allowance is 15 percent for the fatigue truck (Table 3.6.2.1-1). Centrifugal force effects are included in this example. The transverse position of the truck may be different for each response and for positive and negative values of the same response. The fatigue truck is assumed to travel in either direction, or in opposite directions, to produce the maximum stress range. Marked traffic lanes are not considered. This assumption provides larger fatigue stresses than would be obtained if the fatigue truck were held to marked traffic lanes. The fatigue truck is permitted to travel within two feet of the curb line. As specified in Article 6.6.1.2.1, stress ranges are computed using the short-term composite section for both positive and negative bending given that the deck slab longitudinal reinforcement specified in 6.10.1.7 are satisfied.

For points where the dead load produces compressive stress, Article 6.6.1.2.1 specifies that twice the factored fatigue live load defined in Article 3.6.1.4, and factored according to the fatigue load combination of Table 3.4.1-1, is to be used to determine if a net tensile stress is produced at the point under consideration. The fatigue live load is placed in a single lane. If the dead load produces tensile stress or, where dead load produces compressive stress, a net tensile stress occurs under dead load combined with twice the factored fatigue load at a point, fatigue must be checked at that point using the stress range produced by the single factored fatigue truck, whether or not the factored fatigue truck by itself produces a net tensile stress.

Article 6.11.5 requires that longitudinal stress ranges due to warping and due to transverse moments be considered when determining the sum of the stress ranges used in fatigue analysis. In addition, the through-thickness bending stress range due to cross-sectional distortion at flange-to-web fillet welds and at the termination of fillet welds connecting transverse elements must be checked for fatigue. Computation of these through-thickness bending stresses is illustrated in the Sample Calculations given in Appendix D.

Live Load Deflection

Article 2.5.2.6.2 provides optional deflection criteria that may be checked if required by the bridge owner. Live load deflection is to be checked using the live load portion of Load Combination Service I (Table 3.4.1-1) including the dynamic load allowance. The limiting live load deflection is specified as the fraction of the span defined in Article 2.5.2.6.2. Different live load positions must be examined for each girder and span since the deflections of curved girders usually differ significantly at any one cross section. The uncracked composite section along the entire length of the bridge should be used in computing the deflections. Centrifugal force effects are to be considered. The multiple presence factors specified in Article 3.6.1.1.2 should be applied.

If a sidewalk were present, vehicular traffic would be constrained from a portion of the deck (unless vehicles were permitted to mount the sidewalk), which would cause the computed live load deflections to be reduced depending on which side of the bridge the sidewalk was placed. Sidewalk load is discussed further in Article 3.6.1.6.

DESIGN

Section Properties

Table C-5, Appendix C, gives selected section properties for Girder 2. Locations from the neutral axis to the top (T) and bottom (B) extreme fiber of the steel section are given. The section properties include the longitudinal component of the top-flange bracing area. Longitudinal flange stiffeners and the 1-inch bottom flange lips are also included in the section properties.

When the section is composite, the entire overhang, the concrete between the tub webs, and half of the concrete between girders is considered effective, as specified in Article 4.5.2.2. The haunch depth is considered in computing the section properties, but the area of the haunch is not included. Since the actual depth of the haunch may vary from its theoretical value to account for construction tolerances, many designers ignore the thickness of the haunch in all calculations. The longitudinal reinforcing steel area equal to 20.0 square inches per box is assumed placed at the neutral axis of the of the effective structural deck area. Considering that Article 6.10.1.7 requires that two thirds of the deck longitudinal steel be placed in the top layer and that the deck top concrete cover is thicker than the deck bottom concrete cover, the centroid of the deck reinforcement is usually close to the assumed location. The longitudinal reinforcing steel within the effective portion of the concrete deck is considered effective when the section is subjected to negative bending at the strength limit state. The deck area is divided by "3n" and the reinforcing steel area is divided by 3 (for positive and negative bending, respectively) for computing the transformed section properties to account for creep in the concrete for calculations involving the superimposed dead load. The reinforcing steel area is adjusted since the concrete is assumed to transfer the force from the deck steel to the rest of the cross section. This reduction in steel area is not applied by all designers and may be ignored if it is not consistent with the practices of the owner agency.

Table D-1 in the Sample Calculations (Appendix D) also gives section properties for Girder 2 for the case where the bottom flange is composed of composite steel and concrete, as an alternative to a conventional longitudinally stiffened bottom flange. The Sample Calculations in Appendix D discuss the computation of the section properties given in Table D-1 in more detail.

Shear Connectors

Shear connectors are 7/8-inch diameter by 6 inches long.

The sum of the torsional and vertical bending shears is used with half of the girder to design the shear connectors.

Flanges

The top flanges of the tubs must meet the criteria of Article 6.11.3.2 at the constructibility limit state.

Two types of bottom (box) flanges are used in this example. In positive moment regions, the bottom flange is an unstiffened plate. In the negative moment regions, a single longitudinal stiffener is used to increase the compressive strength of the bottom flange. The critical stress for box flanges is determined at the constructibility limit state.

Webs

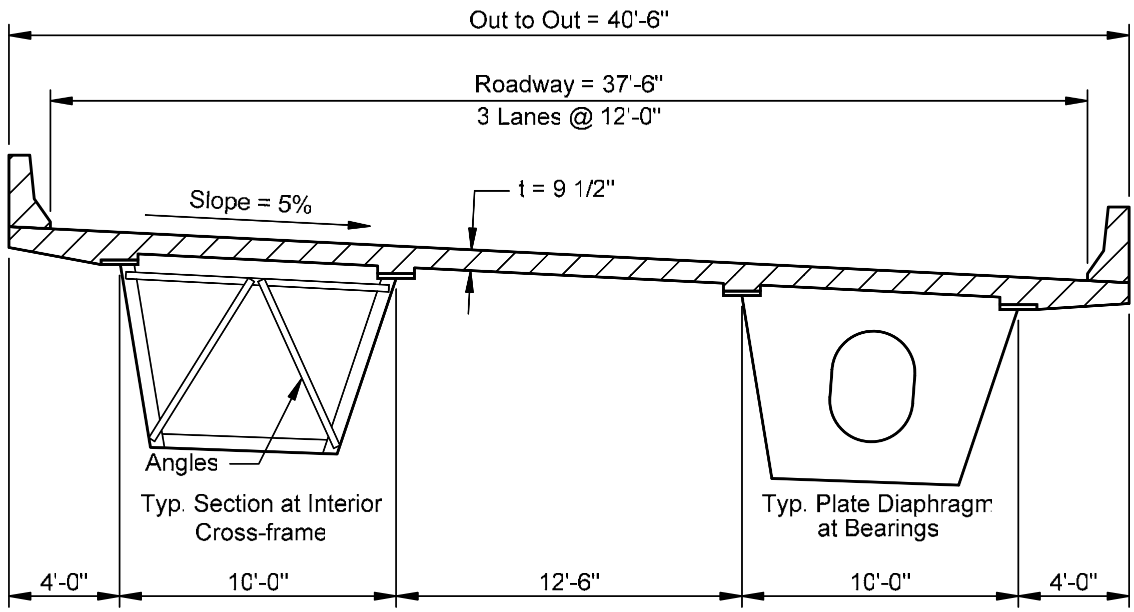
In this example, transversely stiffened webs are used throughout. Transverse stiffener designs are not shown, but are similar to the designs illustrated in the companion example of the I-girder curved bridge. Transverse stiffeners are required throughout most of the girder length. The spacing of the transverse stiffeners near the interior supports is 62 inches.

Diaphragms

Interior diaphragms at supports are solid plates with pairs of bearing stiffeners welded on each side of an access hole. External diaphragms at supports are also solid plates.

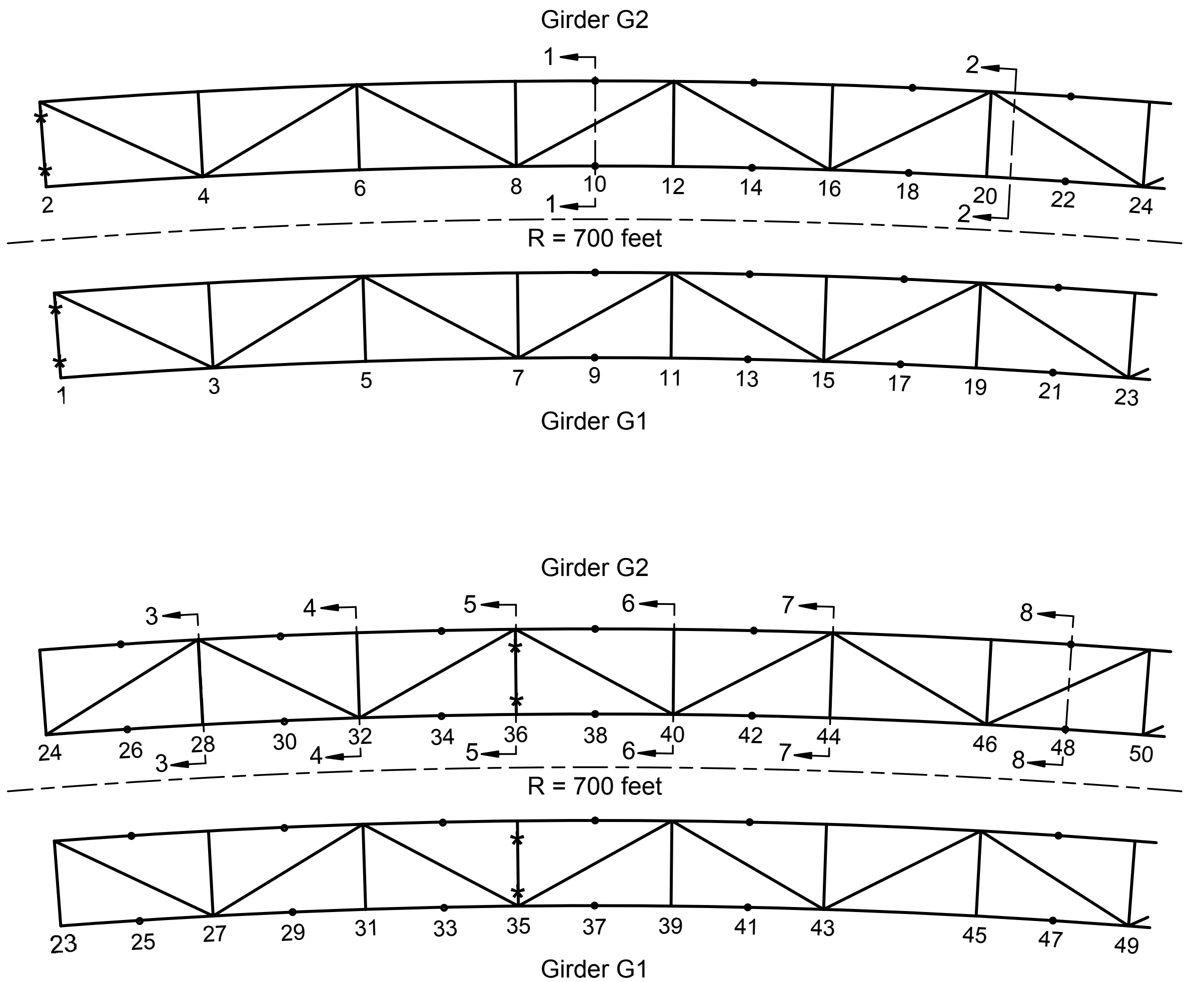
Sample Calculations

Sample calculations at selected critical locations of Girder 2 are presented in Appendix D. The calculations are intended to illustrate the application of some of the more significant provisions of the Specifications. As such, complete calculations are not shown at all sections for each design. The sample calculations illustrate calculations to be made at the Strength, Fatigue, Constructibility and Serviceability limit states. The calculations also include longitudinal flange stiffener and bearing stiffener designs, a top flange bracing member design, a diaphragm design, transverse bending stress computations and a bolted field splice design. The calculations make use of the moments, shears, torques, and top flange bracing forces contained in Tables C-1 through C-4 of Appendix C and the section properties contained in Table C-5.



Deck concrete – $f'_c = 4{,}000$ psi $E = 3.6 \times 10^6$ psi
Haunch – 20 in. wide, 4 in. deep measured from top of web
Permanent deck forms are present
Total deck thickness = 9.5 in.

Figure 1. Box Girder Bridge Cross Section



* Bearing Locations

Note: Sections 1-1, 2-2, etc. refer to the design sections in Appendix C tables.
Section 9-9 is at the midspan of Span 2.

Figure 2. Node Numbers

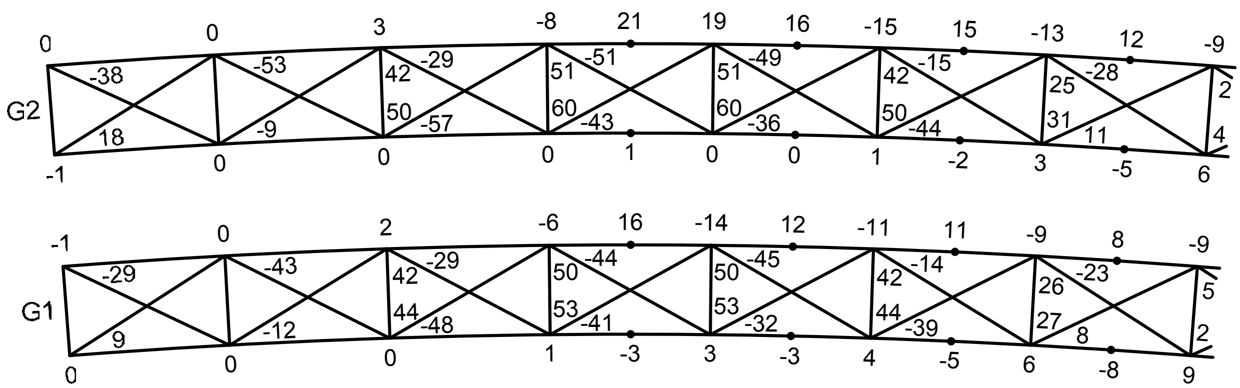


Figure 3. Case of Double-Diagonal Bracing: Lateral Flange Moments (k-ft) and Bracing Forces (kips) Due to Entire Deck Weight with Overhang Brackets, Inclined Webs

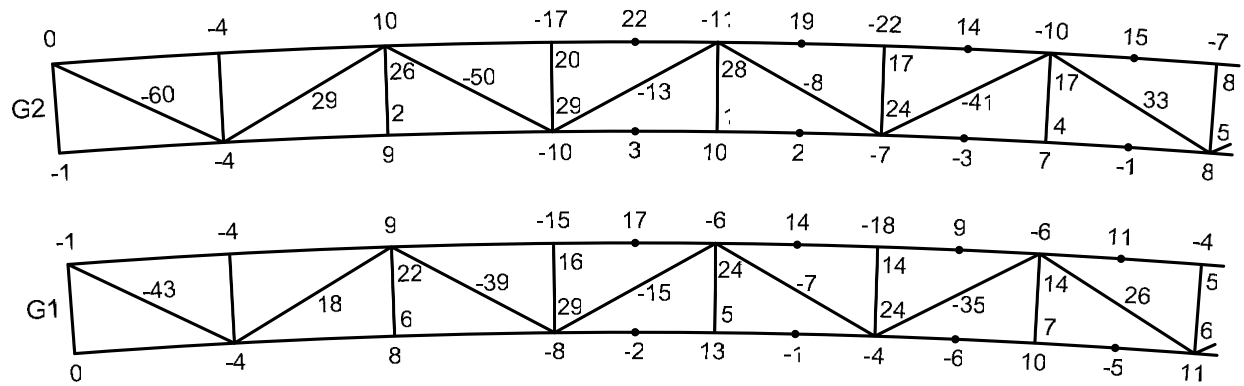


Figure 4. Case of Single-Diagonal Bracing: Lateral Flange Moments (k-ft) and Bracing Forces (kips) Due to Entire Deck Weight with Overhang Brackets, Inclined Webs

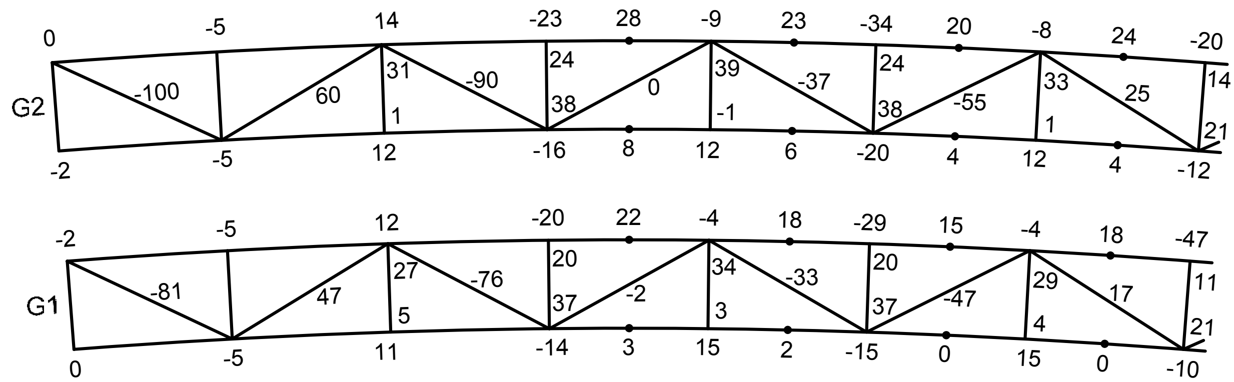


Figure 5. Case of Single-Diagonal Bracing: Lateral Flange Moments (k-ft) and Bracing Forces (kips) Due to Cast #1 with Overhang Brackets, Inclined Webs

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APPENDIX A

Girder Field Sections

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June 21, 1997 9:25 AM

Bridge Type --> Box Girder   Date Created -> 07/29/94
Project -----> Sample Box Design   Initials -----> DHH

Project ID ---> BOX1SAMPLE
Description --> 160-210-160 spans 2-boxes

Number of girders ---> 2
Number of spans ---> 3
Project units ---> English

BRIDGE-SYSTEMsm 3D Version -> 2.1

Box girder cross section

	---center line of box ---- --to the top of the web -- left side In	right side In.	width of bottom flng In.
Girder 1 -->	60.00	60.00	81.00
Girder 2 -->	60.00	60.00	81.00

Girder --> 1 Field Section --> 1

	Rght		-----Top Flange----			---Bottom Flange--			---- Web ------		
Mem.	Node	Length	Width	Thick.	Fy	Width	Thick.	Fy	Depth	Thick.	Fy
						Lip-> 1.00					
1	3	15.74	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
2	5	15.74	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
3	7	15.74	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
4	9	7.87	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
5	11	7.87	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
6	13	7.87	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
7	15	7.87	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
8	17	7.87	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
9	19	7.87	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.

	Top Flange	Bot Flange	Web	TOTAL	Length
Section Weight -->	10285.	16673.	29072.	56031.	Ft.-> 94.46

Girder --> 1 Field Section --> 2

	Rght		-----Top Flange----			---Bottom Flange--			---- Web ------		
Mem.	Node	Length	Width	Thick.	Fy	Width	Thick.	Fy	Depth	Thick.	Fy
						Lip-> 1.00					
10	21	7.87	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
11	23	7.87	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
12	25	7.87	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
13	27	7.87	18.00	1.5000	50.	81.00	1.0000	50.	78.00	.5625	50.
14	29	7.87	18.00	1.5000	50.	81.00	1.0000	50.	78.00	.5625	50.
15	31	7.87	18.00	1.5000	50.	81.00	1.0000	50.	78.00	.5625	50.
16	33	7.87	18.00	3.0000	50.	81.00	1.5000	50.	78.00	.5625	50.
17	35	7.87	18.00	3.0000	50.	81.00	1.5000	50.	78.00	.5625	50.
Sup --->		157.43									
18	37	7.38	18.00	3.0000	50.	81.00	1.5000	50.	78.00	.5625	50.
19	39	7.38	18.00	3.0000	50.	81.00	1.5000	50.	78.00	.5625	50.
20	41	7.38	18.00	1.5000	50.	81.00	1.0000	50.	78.00	.5625	50.
21	43	7.38	18.00	1.5000	50.	81.00	1.0000	50.	78.00	.5625	50.
22	45	14.76	18.00	1.5000	50.	81.00	1.0000	50.	78.00	.5625	50.

	Top Flange	Bot Flange	Web	TOTAL	Length
Section Weight -->	23544.	32096.	33009.	88649.	Ft.-> 107.25

Girder --> 1 Field Section --> 3

Mem.	Rght Node	Length	Top Flange Width	Top Flange Thick.	Top Flange Fy	Bottom Flange Width	Bottom Flange Thick.	Bottom Flange Fy	Web Depth	Web Thick.	Web Fy
						Lip-> 1.00					
23	47	7.38	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
24	49	7.38	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
25	51	7.38	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
26	53	7.38	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
27	55	7.38	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
28	57	7.38	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
29	59	7.38	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
30	61	7.38	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
31	63	7.38	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
32	65	7.38	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
33	67	7.38	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
34	69	7.38	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
35	71	7.38	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
36	73	7.38	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
37	75	7.38	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
38	77	7.38	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.

	Top Flange	Bot Flange	Web	TOTAL	Length
Section Weight -->	12857.	25010.	36340.	74207.	Ft.-> 118.07

Girder --> 1 Field Section --> 4

Mem.	Rght Node	Length	Top Flange Width	Top Flange Thick.	Top Flange Fy	Bottom Flange Width	Bottom Flange Thick.	Bottom Flange Fy	Web Depth	Web Thick.	Web Fy
						Lip-> 1.00					
39	79	14.76	18.00	1.5000	50.	81.00	1.0000	50.	78.00	.5625	50.
40	81	7.38	18.00	1.5000	50.	81.00	1.0000	50.	78.00	.5625	50.
41	83	7.38	18.00	1.5000	50.	81.00	1.0000	50.	78.00	.5625	50.
42	85	7.38	18.00	3.0000	50.	81.00	1.5000	50.	78.00	.5625	50.
43	87	7.38	18.00	3.0000	50.	81.00	1.5000	50.	78.00	.5625	50.
Sup	--->	206.63									
44	89	7.87	18.00	3.0000	50.	81.00	1.5000	50.	78.00	.5625	50.
45	91	7.87	18.00	3.0000	50.	81.00	1.5000	50.	78.00	.5625	50.
46	93	7.87	18.00	1.5000	50.	81.00	1.0000	50.	78.00	.5625	50.
47	95	7.87	18.00	1.5000	50.	81.00	1.0000	50.	78.00	.5625	50.
48	97	7.87	18.00	1.5000	50.	81.00	1.0000	50.	78.00	.5625	50.
49	99	7.87	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
50	101	7.87	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
51	103	7.87	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.

	Top Flange	Bot Flange	Web	TOTAL	Length
Section Weight -->	23544.	32097.	33009.	88650.	Ft.-> 107.25

Girder --> 1   Field Section --> 5

	Rght		-----Top Flange----			---Bottom Flange--			---- Web ------		
Mem.	Node	Length	Width	Thick.	Fy	Width	Thick.	Fy	Depth	Thick.	Fy
						Lip-> 1.00					
52	105	7.87	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
53	107	7.87	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
54	109	7.87	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
55	111	7.87	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
56	113	7.87	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
57	115	7.87	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
58	117	15.74	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
59	119	15.74	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
60	121	15.74	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
Sup --->		157.43									

	Top Flange	Bot Flange	Web	TOTAL		Length
Section Weight -->	10285.	16674.	29072.	56031.	Ft.->	94.46
Girder Weight -->	80515.	122550.	160504.	363569.	Ft.->	521.48

Girder --> 2   Field Section --> 1

	Rght		-----Top Flange----			---Bottom Flange--			---- Web ------		
Mem.	Node	Length	Width	Thick.	Fy	Width	Thick.	Fy	Depth	Thick.	Fy
						Lip-> 1.00					
61	4	16.26	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
62	6	16.26	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
63	8	16.26	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
64	10	8.13	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
65	12	8.13	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
66	14	8.13	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
67	16	8.13	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
68	18	8.13	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
69	20	8.13	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.

	Top Flange	Bot Flange	Web	TOTAL		Length
Section Weight -->	10621.	17218.	30022.	57862.	Ft.->	97.54

Girder --> 2 Field Section --> 2

Mem.	Rght Node	Length	-----Top Flange---- Width	Thick.	Fy	---Bottom Flange-- Width	Thick.	Fy	---- Web ------ Depth	Thick.	Fy
						Lip-> 1.00					
70	22	8.13	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
71	24	8.13	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
72	26	8.13	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
73	28	8.13	18.00	1.5000	50.	81.00	1.0000	50.	78.00	.5625	50.
74	30	8.13	18.00	1.5000	50.	81.00	1.0000	50.	78.00	.5625	50.
75	32	8.13	18.00	1.5000	50.	81.00	1.0000	50.	78.00	.5625	50.
76	34	8.13	18.00	3.0000	50.	81.00	1.5000	50.	78.00	.5625	50.
77	36	8.13	18.00	3.0000	50.	81.00	1.5000	50.	78.00	.5625	50.
Sup --->		162.57									
78	38	7.62	18.00	3.0000	50.	81.00	1.5000	50.	78.00	.5625	50.
79	40	7.62	18.00	3.0000	50.	81.00	1.5000	50.	78.00	.5625	50.
80	42	7.62	18.00	1.5000	50.	81.00	1.0000	50.	78.00	.5625	50.
81	44	7.62	18.00	1.5000	50.	81.00	1.0000	50.	78.00	.5625	50.
82	46	15.24	18.00	1.5000	50.	81.00	1.0000	50.	78.00	.5625	50.

	Top Flange	Bot Flange	Web	TOTAL	Length
Section Weight -->	24313.	33145.	34088.	91545.	Ft.-> 110.75

Girder --> 2 Field Section --> 3

Mem.	Rght Node	Length	-----Top Flange---- Width	Thick.	Fy	---Bottom Flange-- Width	Thick.	Fy	---- Web ------ Depth	Thick.	Fy
						Lip-> 1.00					
83	48	7.62	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
84	50	7.62	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
85	52	7.62	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
86	54	7.62	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
87	56	7.62	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
88	58	7.62	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
89	60	7.62	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
90	62	7.62	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
91	64	7.62	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
92	66	7.62	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
93	68	7.62	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
94	70	7.62	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
95	72	7.62	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
96	74	7.62	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
97	76	7.62	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.
98	78	7.62	16.00	1.0000	50.	81.00	.7500	50.	78.00	.5625	50.

	Top Flange	Bot Flange	Web	TOTAL	Length
Section Weight -->	13277.	25827.	37528.	76632.	Ft.-> 121.93

Girder --> 2 Field Section --> 4

Mem.	Rght Node	Length	-----Top Flange---- Width	Thick.	Fy	---Bottom Flange-- Width	Thick.	Fy	---- Web ------ Depth	Thick.	Fy
						Lip-> 1.00					
99	80	15.24	18.00	1.5000	50.	81.00	1.0000	50.	78.00	.5625	50.
100	82	7.62	18.00	1.5000	50.	81.00	1.0000	50.	78.00	.5625	50.
101	84	7.62	18.00	1.5000	50.	81.00	1.0000	50.	78.00	.5625	50.
102	86	7.62	18.00	3.0000	50.	81.00	1.5000	50.	78.00	.5625	50.
103	88	7.62	18.00	3.0000	50.	81.00	1.5000	50.	78.00	.5625	50.
Sup --->		213.38									
104	90	8.13	18.00	3.0000	50.	81.00	1.5000	50.	78.00	.5625	50.
105	92	8.13	18.00	3.0000	50.	81.00	1.5000	50.	78.00	.5625	50.
106	94	8.13	18.00	1.5000	50.	81.00	1.0000	50.	78.00	.5625	50.
107	96	8.13	18.00	1.5000	50.	81.00	1.0000	50.	78.00	.5625	50.
108	98	8.13	18.00	1.5000	50.	81.00	1.0000	50.	78.00	.5625	50.
109	100	8.13	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
110	102	8.13	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
111	104	8.13	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.

	Top Flange	Bot Flange	Web	TOTAL	Length
Section Weight -->	24313.	33145.	34088.	91546.	Ft.-> 110.75

Girder --> 2 Field Section --> 5

Mem.	Rght Node	Length	-----Top Flange---- Width	Thick.	Fy	---Bottom Flange-- Width	Thick.	Fy	---- Web ------ Depth	Thick.	Fy
						Lip-> 1.00					
112	106	8.13	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
113	108	8.13	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
114	110	8.13	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
115	112	8.13	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
116	114	8.13	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
117	116	8.13	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
118	118	16.26	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
119	120	16.26	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
120	122	16.26	16.00	1.0000	50.	81.00	.6250	50.	78.00	.5625	50.
Sup --->		162.57									

	Top Flange	Bot Flange	Web	TOTAL	Length
Section Weight -->	10621.	17218.	30022.	57862.	Ft.-> 97.54
Girder Weight -->	83145.	126554.	165747.	375446.	Ft.-> 538.52

------------ S T R U C T U R E ------------

	Top Flange	Bot Flange	Web	TOTAL
Weight -->	163660.	249104.	326251.	739015.

APPENDIX B

Girder Moments, Shears, and Torques at Tenth-Points

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STRENGTH -- HL-93 Plus Dynamic Load Allow., Multiple Presence, and Centrifugal Forces

April 5, 1997 10:51 AM

Revised November 16, 2001

```
Bridge Type --> Box Girder              Date Created -> 07/29/94
Project  -----> Sample Box Design       Initials -----> DHH

Project ID ---> BOX1SAMPLE
Description --> 160-210-160 spans 2-boxes

                 Number of girders --->  2
                 Number of spans   --->  3
                 Project units     --->  English


BRIDGE-SYSTEMsm   3D Version ->  2.1
```


Stage Definition

Stg1 = Load due to weight of structural steel including girders and internal cross bracing and top flange diagonal bracing

Stg6 = Load due to weight of concrete deck placed at one time

Stg7 = Load due to weight of parapets and wearing surface placed on composite bridge

Special = LRFD HL-93 live load vehicle responses including the dynamic load allowance

Girder -> 1 Span -> 1 Length -> 157.43

D E A D   L O A D S

Length	---- MOMENTS ---- Stg1	Stg6	Stg7	---- SHEARS ---- Stg1	Stg6	Stg7	----- TORQUES ---- Stg1	Stg6	Stg7
.00	0	0	0	27	114	58	42	286	-145
15.74	521	2191	790	19	80	27	82	398	-125
31.49	882	3666	1377	10	45	18	34	189	-93
47.23	1049	4321	1684	5	23	11	30	153	-92
62.97	1047	4320	1706	-6	-25	-7	-1	9	-54
78.71	851	3503	1441	-11	-44	-13	-29	-125	-30
94.46	493	2043	901	-16	-69	-19	-33	-158	0
110.20	-75	-315	83	-23	-98	-30	-54	-262	49
125.94	-837	-3461	-1010	-28	-116	-41	-25	-165	108
141.69	-1781	-7206	-2357	-34	-137	-56	-10	-135	193
157.43	-2969	-11629	-4097	-44	-171	-94	-22	-231	294

L I V E   L O A D S

	Moments							
	--- Lane ---		-- Truck --		-- Special--		1-Lane Truck	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	0	0	0	0	0	0
15.74	0	0	505	-66	2472	-469	505	-66
31.49	0	0	845	-132	4330	-938	845	-132
47.23	0	0	997	-198	5412	-1408	997	-198
62.97	0	0	1043	-260	5863	-1878	1043	-260
78.71	0	0	1014	-318	5777	-2338	1014	-318
94.46	0	0	923	-373	5189	-2795	923	-373
110.20	0	0	748	-450	4109	-3915	748	-450
125.94	0	0	482	-549	2602	-4547	482	-549
141.69	0	0	182	-669	1252	-5559	182	-669
157.43	0	0	156	-843	1061	-7784	156	-843

	Shears						-- Torque --	
	--- Lane ---		-- Truck --		-- Special--		--Maximums--	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	35	-3	139	-24	660	-398
15.74	0	0	28	-4	115	-29	775	-448
31.49	0	0	23	-6	94	-35	756	-482
47.23	0	0	19	-8	78	-41	597	-389
62.97	0	0	15	-11	53	-52	389	-307
78.71	0	0	11	-15	40	-63	309	-354
94.46	0	0	9	-18	31	-83	360	-479
110.20	0	0	7	-23	25	-101	462	-636
125.94	0	0	5	-27	21	-116	569	-766
141.69	0	0	5	-29	19	-127	668	-866
157.43	0	0	3	-36	14	-163	1049	-922

Girder -> 1   Span -> 2   Length -> 206.63

D E A D   L O A D S

Length	MOMENTS Stg1	MOMENTS Stg6	MOMENTS Stg7	SHEARS Stg1	SHEARS Stg6	SHEARS Stg7	TORQUES Stg1	TORQUES Stg6	TORQUES Stg7
.00	-2969	-11629	-4097	45	175	96	36	294	-335
20.66	-1422	-5845	-1864	31	128	54	4	105	-206
41.33	-326	-1516	-220	25	110	37	60	309	-120
61.99	493	1881	988	17	72	23	39	205	-52
82.65	977	3900	1705	11	47	11	61	261	-20
103.31	1118	4442	1944	0	0	0	0	0	0
123.98	976	3900	1705	-11	-47	-11	-64	-261	20
144.64	492	1880	986	-17	-72	-24	-39	-205	51
165.30	-327	-1519	-222	-25	-110	-37	-60	-309	120
185.96	-1422	-5848	-1867	-31	-127	-54	-4	-105	206
206.63	-2969	-11633	-4098	-45	-175	-96	-36	-294	334

L I V E   L O A D S

Length	Moments: Lane POS	Lane NEG	Truck POS	Truck NEG	Special POS	Special NEG	1-Lane Truck POS	1-Lane Truck NEG
.00	0	0	156	-843	1061	-7784	156	-843
20.66	0	0	245	-547	1310	-4411	245	-547
41.33	0	0	624	-417	2993	-3033	624	-417
61.99	0	0	894	-317	4784	-2275	894	-317
82.65	0	0	1050	-248	5926	-2008	1050	-248
103.31	0	0	1091	-188	6304	-1749	1091	-188
123.98	0	0	1050	-249	5928	-2013	1050	-249
144.64	0	0	895	-318	4775	-2279	895	-318
165.30	0	0	623	-416	3000	-3021	623	-416
185.96	0	0	257	-547	1315	-4421	257	-547
206.63	0	0	157	-830	1062	-7788	157	-830

Length	Shears: Lane POS	Lane NEG	Truck POS	Truck NEG	Special POS	Special NEG	Torque Maximums POS	Torque Maximums NEG
.00	0	0	39	-3	171	-15	1049	-922
20.66	0	0	30	-4	140	-23	995	-702
41.33	0	0	26	-5	124	-26	919	-598
61.99	0	0	21	-8	101	-37	716	-464
82.65	0	0	18	-10	78	-45	555	-383
103.31	0	0	15	-15	58	-57	446	-430
123.98	0	0	10	-18	43	-78	413	-540
144.64	0	0	8	-21	36	-101	500	-724
165.30	0	0	4	-26	26	-124	625	-906
185.96	0	0	4	-31	23	-140	713	-991
206.63	0	0	3	-37	14	-166	928	-1046

Girder -> 1 Span -> 3 Length -> 157.43

D E A D   L O A D S

Length	MOMENTS Stg1	MOMENTS Stg6	MOMENTS Stg7	SHEARS Stg1	SHEARS Stg6	SHEARS Stg7	TORQUES Stg1	TORQUES Stg6	TORQUES Stg7
.00	-2969	-11633	-4098	44	171	94	22	231	-296
15.74	-1780	-7203	-2359	34	137	56	10	134	-194
31.49	-837	-3459	-1013	28	116	41	25	166	-109
47.23	-74	-312	80	23	98	30	54	262	-50
62.97	493	2044	897	16	69	19	33	158	-1
78.71	851	3504	1437	11	44	13	30	125	29
94.46	1047	4320	1703	6	25	7	1	-10	53
110.20	1048	4321	1681	-5	-23	-11	-30	-153	90
125.94	882	3666	1375	-10	-45	-18	-34	-190	91
141.69	521	2189	788	-19	-80	-28	-82	-398	132
157.43	0	0	0	-27	-114	-59	-42	-285	174

L I V E   L O A D S

Length	Moments: Lane POS	Lane NEG	Truck POS	Truck NEG	Special POS	Special NEG	1-Lane Truck POS	1-Lane Truck NEG
.00	0	0	157	-830	1062	-7788	157	-830
15.74	0	0	182	-673	1248	-5556	182	-673
31.49	0	0	482	-547	2591	-4532	482	-547
47.23	0	0	747	-449	4099	-3900	747	-449
62.97	0	0	923	-372	5181	-2783	923	-372
78.71	0	0	1014	-312	5769	-2328	1014	-312
94.46	0	0	1042	-255	5855	-1868	1042	-255
110.20	0	0	997	-193	5405	-1402	997	-193
125.94	0	0	845	-129	4326	-993	845	-129
141.69	0	0	505	-65	2470	-466	505	-65
157.43	0	0	0	0	0	0	0	0

Length	Shears: Lane POS	Lane NEG	Truck POS	Truck NEG	Special POS	Special NEG	Torque Maximums POS	Torque Maximums NEG
.00	0	0	38	-3	167	-15	928	-1046
15.74	0	0	29	-5	128	-19	874	-657
31.49	0	0	27	-5	116	-21	770	-549
47.23	0	0	23	-7	101	-25	640	-434
62.97	0	0	18	-9	83	-31	482	-319
78.71	0	0	15	-11	64	-38	375	-281
94.46	0	0	11	-15	51	-52	346	-378
110.20	0	0	8	-19	41	-77	434	-591
125.94	0	0	6	-23	32	-92	512	-751
141.69	0	0	4	-28	27	-113	503	-772
157.43	0	0	3	-35	24	-139	399	-662

Girder -> 2 Span -> 1 Length -> 162.57

D E A D   L O A D S

Length	MOMENTS Stg1	Stg6	Stg7	SHEARS Stg1	Stg6	Stg7	TORQUES Stg1	Stg6	Stg7
.00	0	0	0	31	110	91	43	98	418
16.26	555	2268	816	19	74	39	87	276	323
32.51	938	3868	1418	11	44	26	35	92	241
48.77	1116	4632	1726	5	21	14	32	88	148
65.03	1115	4633	1733	-7	-26	-8	-2	-22	49
81.29	905	3780	1446	-11	-45	-14	-32	-129	-45
97.54	525	2207	867	-17	-69	-28	-36	-125	-134
113.80	-79	-256	-2	-24	-97	-40	-59	-203	-201
130.06	-892	-3579	-1166	-29	-117	-51	-28	-53	-247
146.31	-1896	-7599	-2610	-35	-137	-62	-10	63	-273
162.57	-3154	-12272	-4473	-46	-185	-96	-22	48	-346

L I V E   L O A D S

	Moments							
	--- Lane ---		-- Truck --		-- Special--		1-Lane Truck	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	0	0	0	0	0	0
16.26	0	0	537	-64	2606	-484	537	-64
32.51	0	0	898	-129	4559	-967	898	-129
48.77	0	0	1056	-195	5687	-1446	1056	-195
65.03	0	0	1100	-263	6152	-1931	1100	-263
81.29	0	0	1066	-336	6059	-2416	1066	-336
97.54	0	0	963	-416	5434	-2907	963	-416
113.80	0	0	775	-511	4308	-4097	775	-511
130.06	0	0	506	-619	2751	-4768	506	-619
146.31	0	0	194	-749	1305	-5836	194	-749
162.57	0	0	173	-934	1114	-8127	173	-934

	Shears						-- Torque --	
	--- Lane ---		-- Truck --		-- Special--		--Maximums--	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	41	-8	128	-26	621	-533
16.26	0	0	35	-8	110	-29	774	-503
32.51	0	0	30	-8	93	-35	785	-469
48.77	0	0	24	-8	75	-44	638	-427
65.03	0	0	17	-12	54	-52	412	-391
81.29	0	0	12	-18	40	-67	348	-439
97.54	0	0	9	-23	36	-85	333	-535
113.80	0	0	8	-29	33	-102	433	-676
130.06	0	0	5	-33	26	-114	552	-793
146.31	0	0	3	-36	16	-127	687	-848
162.57	0	0	3	-41	13	-155	980	-863

Girder -> 2 Span -> 2 Length -> 213.38

D E A D L O A D S

Length	MOMENTS Stg1	MOMENTS Stg6	MOMENTS Stg7	SHEARS Stg1	SHEARS Stg6	SHEARS Stg7	TORQUES Stg1	TORQUES Stg6	TORQUES Stg7
.00	-3154	-12272	-4473	47	185	102	36	-33	447
21.34	-1513	-6169	-2107	32	130	65	3	-101	372
42.68	-348	-1473	-371	26	105	50	64	183	333
64.01	525	2077	893	17	69	35	40	118	243
85.35	1040	4196	1638	12	46	17	68	237	126
106.69	1190	4826	1890	0	0	0	0	0	0
128.03	1039	4195	1638	-12	-46	-17	-68	-237	-127
149.36	525	2075	893	-17	-69	-35	-40	-118	-243
170.70	-348	-1476	-370	-26	-105	-50	-64	-183	-335
192.04	-1514	-6173	-2106	-32	-130	-65	-3	102	-373
213.38	-3155	-12275	-4469	-47	-185	-102	-36	33	-448

L I V E L O A D S

	Moments							
	--- Lane ---		-- Truck --		-- Special--		1-Lane Truck	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	173	-934	1114	-8127	173	-934
21.34	0	0	262	-609	1401	-4629	262	-609
42.68	0	0	630	-467	3176	-3197	630	-467
64.01	0	0	908	-356	5018	-2366	908	-356
85.35	0	0	1071	-265	6205	-2070	1071	-265
106.69	0	0	1117	-187	6598	-1786	1117	-187
128.03	0	0	1070	-264	6204	-2065	1070	-264
149.36	0	0	907	-354	5001	-2355	907	-354
170.70	0	0	629	-466	3166	-3165	629	-466
192.04	0	0	269	-608	1393	-4627	269	-608
213.38	0	0	172	-930	1114	-8128	172	-930

	Shears						-- Torque --	
	--- Lane ---		-- Truck --		-- Special--		--Maximums--	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	44	-3	160	-14	980	-863
21.34	0	0	37	-3	135	-22	978	-709
42.68	0	0	33	-6	120	-33	925	-569
64.01	0	0	28	-9	100	-42	754	-433
85.35	0	0	22	-11	78	-46	580	-425
106.69	0	0	16	-16	57	-57	477	-491
128.03	0	0	11	-22	46	-78	391	-596
149.36	0	0	9	-28	41	-99	456	-746
170.70	0	0	6	-34	33	-120	603	-915
192.04	0	0	3	-37	22	-135	725	-974
213.38	0	0	3	-43	14	-159	878	-976

Girder -> 2 Span -> 3 Length -> 162.57

D E A D L O A D S

Length	MOMENTS Stg1	Stg6	Stg7	SHEARS Stg1	Stg6	Stg7	TORQUES Stg1	Stg6	Stg7
.00	-3155	-12275	-4469	46	185	96	22	-48	346
16.26	-1895	-7595	-2606	35	137	62	10	-63	273
32.51	-891	-3577	-1162	29	117	51	28	53	247
48.77	-79	-253	3	24	97	40	59	203	201
65.03	525	2208	871	17	69	28	36	125	134
81.29	906	3781	1450	11	45	14	32	129	45
97.54	1115	4634	1737	7	26	8	1	22	-49
113.80	1116	4632	1729	-5	-21	-14	-32	-88	-148
130.06	938	3867	1421	-11	-44	-26	-35	-92	-241
146.31	555	2266	816	-19	-74	-39	-87	-276	-323
162.57	0	0	0	-31	-110	-91	-43	-98	-417

L I V E L O A D S

	Moments							
	--- Lane ---		-- Truck --		-- Special--		1-Lane Truck	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	172	-930	1114	-8128	172	-930
16.26	0	0	195	-751	1312	-5843	195	-751
32.51	0	0	507	-623	2762	-4778	507	-623
48.77	0	0	777	-513	4320	-4106	777	-513
65.03	0	0	965	-419	5445	-2917	965	-419
81.29	0	0	1067	-334	6068	-2424	1067	-334
97.54	0	0	1101	-261	6160	-1936	1101	-261
113.80	0	0	1056	-194	5689	-1451	1056	-194
130.06	0	0	898	-128	4560	-971	898	-128
146.31	0	0	538	-64	2607	-487	538	-64
162.57	0	0	0	0	0	0	0	0

	Shears						-- Torque --	
	--- Lane ---		-- Truck --		-- Special--		--Maximums--	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	43	-3	158	-14	878	-976
16.26	0	0	36	-3	128	-15	853	-674
32.51	0	0	33	-5	115	-26	799	-536
48.77	0	0	28	-8	102	-33	685	-430
65.03	0	0	22	-9	85	-36	542	-321
81.29	0	0	18	-12	67	-40	415	-360
97.54	0	0	12	-17	52	-54	385	-440
113.80	0	0	8	-24	44	-75	433	-626
130.06	0	0	8	-30	34	-93	502	-782
146.31	0	0	8	-35	28	-111	533	-783
162.57	0	0	8	-41	26	-129	533	-621

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APPENDIX C

Selected Design Forces and Girder 2 Section Properties

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Table C-1. Selected Unfactored Moments (k-ft) and Web Fatigue Shears (kips), Girder 2

Section Node	Steel	Deck	Cast(#)[1]	SupImp[2]	FWS[3]	LLmax[4]	Fat_{min}[5]	Fat_{max}[5]
1-1 10	1,144	4,747	2,979(1) 1,038(2)	765	1,006	5,920 -1,689	-239 V = -14k	1,258 V = 23k
2-2 20.3 Splice	462	1,941	2,749(1) -647(2)	326	428	5,264 -3,087	-451 V = -26k	1,108 V = 12k
3-3 28	-892	-3,579	752(1) -3,682(2)	-504	-663	2,751 -4,768	-649 V = -34	646 V = 8k
4-4 32	-1,896	-7,599	-317(1) -5,305(2)	-1,127	-1,483	1,305 -5,836	-784 V = -36	258 V = 2k
5-5 36	-3,154	-12,272	-1,378(1) -6,915(2)	-1,932	-2,541	1,114 -8,127	-961 V = -44k	173 V = 4k
6-6 40	-1,956	-7,866	-1,378(1) -4,654(2)	-1,163	-1,530	1,049 -5,445	-656 V = -5k	192 V = 37k
7-7 44	-979	-4,015	-1,378(1) -2,392(2)	-563	-740	2,132 -3,753	-542 V = -8k	554 V = 35k
8-8 48	184	634	-1,378(1) 922(2)	142	186	4,121 -2,525	-403 V = -11k	917 V = 32k
9-9 62	1,190	4,826	-1,378(1) 4,960(2)	816	1,074	6,598 -1,786	-182 V = -18k	1,271 V = 18K

[1](#) denotes Deck Cast number

Cast #1 begins at Section 1-1 and ends at Section 3-3 (similar for span 3)
Cast #2 begins at Section 8-8 and is symmetrical in the center span (includes Cast #1)
Steel, Deck and Cast moments are unfactored. Deck and Cast moments include the moments due to the deck haunch and stay-in-place forms.

[2]SupImp - Unfactored superimposed dead load
[3]FWS - Unfactored future wearing surface dead load
[4]LLmax - Unfactored live-load plus the dynamic load allowance moment due to multiple lanes of LRFD HL-93. Dynamic load allowance is included according to Article 3.6.2.
[5]Fat - Maximum and minimum fatigue moment due to one fatigue vehicle plus 15% dynamic load allowance times the load factor of 0.75 specified in Table 3.4.1-1. Vertical shears in the critical web (V) due to the factored fatigue vehicle are given in the "Fat" columns. Fatigue moments and shears are increased by 10 percent to allow for warping.

All live load moments and shears, including fatigue moments and shears, include centrifugal force effects.
Multiple presence factors (Table 3.6.1.1.2-1) were considered in determining LLmax.
The location of nodes and sections may be found by referring to Figure 2 and Appendix A.

Table C-2. Shear (kips), Girder 2 Span 1 at Tenth-Points

Tenth Point	Steel	Deck	SupImp	FWS	Total DL	LL + IM	Factored Shear
0	31	110	39	52	232	128	527
1	19	74	17	22	132	110	363
2	11	44	11	15	81	93	268
3	5	21	6	8	40	75	183
4	-7	-26	-3	-5	-41	-52	-143
5	-11	-45	-6	-8	-70	-67	-207
6	-17	-69	-12	-16	-114	-85	-295
7	-24	-97	-17	-23	-161	-102	-385
8	-29	-117	-22	-29	-197	-114	-453
9	-35	-137	-27	-35	-234	-127	-524
10	-46/47	-185/185	-41/44	-55/58	-327/334	-155/160	-694/712

Live load shear of the same sign as the dead load shear is reported. Reported shears are vertical shears and are for bending plus torsion in the critical web.

Table C-3. Selected Unfactored Torques (k-ft), Girder 2

Section Node	Steel	Deck	SupImp[1]	FWS[2]	LL_{max}[3]	Fat_{min}[4]	Fat_{max}[4]
1-1 10	59	205 464[5]	41	54	525 -409	-85	174
2-2 20.3 Splice	-36	-125 -188	-58	-76	348 -556	-165	96
3-3 28	-28	-53 352	-107	-140	552 -793	-238	108
4-4 32	-10	63 -420	-118	-155	687 -848	-241	132
5-5 36	-22 36	48 -33	-149 193	-197 254	980[6] -863	-232[6]	254[6]
6-6 40	23	-52 -335	163	215	979 -753	-171	260
7-7 44	28	13 -305	145	191	955 -649	-105	264
8-8 48	72	211 -298	125	164	839 -501	-90	244
9-9 62	0	0 0	0	0	477 -491	-100	116

[1]SupImp - Unfactored superimposed dead load
[2]FWS - Unfactored future wearing surface dead load
[3]LL_{max} - Unfactored live-load plus dynamic load allowance torque due to multiple lanes of LRFD HL-93. Dynamic load allowance is included according to Article 3.6.2.
[4]Fat - Maximum and minimum torques due to one fatigue vehicle plus 15% dynamic load allowance times the load factor of 0.75 specified in Table 3.4.1-1.
[5]Bottom value, where listed, is the torque due to Cast #1.
[6]Only the minimum and maximum live-load torques are reported at the pier section.

All live load torques, including fatigue torques, include centrifugal force effects. Multiple presence factors (Table 3.6.1.1.2-1) were considered in determining LL_{MAX}.
The location of nodes and sections are shown in Figure 2 and Appendix A.

Table C-4. Top Flange Bracing Forces (kip), Girder 2 Span 1

Element	Steel	Deck	SupImp	FWS	LL+I	Fact	*Cast #1	Cast #2	Const
1	-13	-40	-3	-4	-2	-79	-100	7	-141
2	6	12	-2	-2	-3	12	60	0	83
3	-11	-39	-5	-6	-4	-85	-90	13	-126
4	-4	-20	-4	-5	-5	-51	0	7	-5
5	-2	-7	-4	-5	-5	-33	-37	18	-49
6	-10	-38	-4	-5	-4	-80	-55	15	-81
7	7	25	-1	-2	-4	29	25	23	69
8	-6	-15	-2	-2	-3	-37	-76	31	-103
9	11	31	3	3	3	66	70	15	120
10	9	46	3	3	3	82	-51	64	-53
11	7	42	2	3	2	72	71	-31	98
12	12	33	3	4	3	71	-25	82	86
13	-8	-16	-2	-3	-3	-42	43	-76	-51

Notes:
1. Casts consider overhang bracing forces
2. Fact = 1.0[1.25(DL) + 1.75(LL+IM)]
3. Const = 1.25[Steel + Cast #1]
   or 1.25[Steel + Cast #1 + Cast #2]

*These values are taken from Figure 5

Table C-5. Selected Section Properties for Girder 2

Section Node	Section Size (in.)	Section Type	Moment of Inertia (in^4)	Neutral Axis B (in.)	Neutral Axis T (in.)
1-1 10 2-2 20.3	2 - 16 x 1 2 - 78 x 0.5625 83 x 0.625 A = 181 in^2	Noncomp	185,187	36.83	42.80
		Comp DL	354,925	55.35	24.27
		CompLL	479,646	68.84	10.78
3-3 28	2 - 18 x 1.5 2 - 78 x 0.5625 83 x 1 LS WT 8 x 28.5 A = 243 in^2	Noncomp	275,175	35.32	45.18
		CompDL	475,329	51.05	29.45
		CompDL Bars	292,858	36.72	43.78
		CompLL	650,889	64.77	15.73
		CompLL Bars	325,531	39.30	41.20
5-5 36	2 - 18 x 3 2 - 78 x 0.5625 83 x 1.5 LS WT 8 x 28.5 A = 338 in^2	Noncomp	438,966	38.81	43.69
		CompDL	633,467	50.44	32.06
		CompDL Bars	454,805	39.76	42.74
		CompLL	836,080	62.50	20.00
		CompLL Bars	484,714	41.55	40.95

Legend:

B	=	vertical distance from the neutral axis to the outermost edge of the bottom flange
T	=	vertical distance from the neutral axis to the outermost edge of the top flange
Noncomp	=	steel section only
Comp DL	=	steel section plus concrete deck transformed using modular ratio of 3n
Comp DL Bars	=	steel section plus longitudinal reinforcement area divided by 3
Comp LL	=	steel section plus concrete deck transformed using modular ratio of n
Comp LL Bars	=	steel section plus longitudinal reinforcement
LS	=	single longitudinal bottom flange stiffener
A	=	total steel area of box section

Composite section properties, including the concrete deck, are computed using the structural deck area including the overhang and half of the deck width between girders. The area of the deck haunch is not included. For composite section properties including only the longitudinal reinforcement, a haunch depth is considered when determine the position of the deck reinforcement relative to the steel girder. The longitudinal reinforcing steel area equal to 20.0 in^2 per box is assumed placed at the neutral axis of the effective structural deck area.

The modular ratio, n, for live load is 7.56 and 3n is used for superimposed dead load. The effective area of reinforcing steel used for superimposed dead load is adjusted for creep by a factor of 3. Thus, the reinforcing area used for the superimposed dead load is 6.67 in^2 (20.0 in^2/3).

The area and moment of inertia of the box section include the longitudinal component of the top flange bracing area, the longitudinal flange stiffener (where present) and the 1-inch bottom flange lips. A single top-flange bracing member of 8.0 in^2 placed at an angle of 30 degrees from tangent to the girder is assumed. The vertical web depth is shown in the above table. However, the total area of the inclined webs is used in computing all section properties.

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APPENDIX D

Sample Calculations

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Girder Stress Check Section 1-1 G2 Node 10
Girder Section Proportioning

The web and the flanges must be proportioned according to the provisions of Article 6.11.2.

Web proportions per Article 6.11.2.1:

For a web without longitudinal stiffeners, the web is proportioned such that:

$$\frac{D}{t_w} \le 150 \qquad \text{Eq (6.11.2.1.2-1)}$$

Determine the web depth along the incline. The web rise over run is 4:1.

$$D = 78\left(\frac{4.123}{4.0}\right) = 80.4 \quad \text{in.}$$

$$\frac{80.4}{0.5625} = 142.9 \quad < 150 \quad \textbf{OK}$$

Flange proportions (Article 6.11.2.2):

Top flanges of tub sections subjected to compression and tension are proportioned such that:

Top flanges: 16 in. x 1.0 in.

$$\frac{b_f}{2t_f} \le 12.0 \qquad \text{Eq (6.11.2.2-1)}$$

$$\frac{16}{2(1)} = 8 \quad < 12 \quad \textbf{OK}$$

$$b_f \ge \frac{D}{6} \qquad \text{Eq (6.11.2.2-2)}$$

$$\frac{80.4}{6} = 13.4 \quad \text{in.} < 16 \text{ in.} \quad \textbf{OK}$$

$$t_f \ge 1.1t_w \qquad \text{Eq (6.11.2.2-3)}$$

$$1.1(0.5625) = 0.62 \quad \text{in.} < 1.0 \text{ in.} \quad \textbf{OK}$$

Therefore, all section proportions for this location are satisfied. Section proportion checks for the other design locations will not be shown. All subsequent sections satisfy these limits.

Girder Stress Check Section 1-1 G2 Node 10
Constructibility - Web

In accordance with Article 6.11.1, the web bend-buckling provisions of Article 6.10.1.9 for the non-composite section must be checked for steel weight and for the Cast #1 of the concrete deck. The web bend-buckling check is not needed for the final condition. The unfactored moments are from Table C1 .

Load	Moment
Steel	1,144 k-ft
Cast #1	2,979 k-ft
Total Moment	4,123 k-ft

Constructibility Load Factor = 1.25 according to the provisions of Article 3.4.2. Neglect the effects of wind on the structure and the presence of construction equipment for this example.

Compute the bending stress at the top of the web due to the above moment using the section properties for the noncomposite section from Table C5.

$D = 80.4$ in.

$D_c =$ N.A. to top of top flange - top flange thickness

$= 42.80 - 1.0 = 41.8$ in. (vertical distance)

$= 41.8\left(\frac{4.123}{4.0}\right) = 43.09$ in. (inclined distance)

$$f_{cw} = f_{top\ web} = -\frac{4123(41.8)(12)}{185187}(1.25) = -13.96 \quad \text{ksi (C)}$$

Article C6.10.1.9.1 states that the compression flange stress may be used instead of the compression in the web since the difference is negligible. This approach will be used in all subsequent web checks in this example.

The nominal bend-buckling resistance in girder webs for constructibility is determined according to the provisions of Article 6.10.1.9.

Compute the nominal bend-buckling stress for the transversely stiffened web without longitudinal stiffeners.

$$F_{crw} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} \quad \text{but cannot exceed } R_hF_{yc} \text{ or } F_{yw}/0.7 \qquad \text{Eq (6.10.1.9.1-1)}$$

$$\text{where: } k = \frac{9}{\left(\frac{D_c}{D}\right)^2} = \frac{9}{\left(\frac{43.09}{80.4}\right)^2} = 31.3$$

Girder Stress Check Section 1-1 G2 Node 10
Constructibility - Web (continued)

$$\frac{0.9(29000)(31.3)}{\left(\frac{80.4}{0.5625}\right)^2} = 39.99 \text{ ksi} < R_hF_{yc} = 1.0(50) = 50 \text{ ksi, therefore, } F_{crw} = 39.99 \text{ ksi}$$

$|-13.96|$ ksi $<$ $\phi_f F_{crw} = 1.0(39.99) = 39.99$ ksi **OK**

Girder Stress Check Section 1-1 G2 Node 10
Constructibility - Top Flange in Compression

The flanges must be checked in flexure for steel weight and for Cast #1 of the concrete deck on the noncomposite section according to the provisions of Article 6.11.3.2. The factored steel stresses during the sequential placement of the concrete are not to exceed the nominal stresses. The effect of the overhang brackets on the flanges must also be considered since G2 is an outside girder. The provisions of Articles 6.10.3.2.1 through 6.10.3.2.3 are applied to the design of the top flange of tub box girders.

Overhang Bracket Load

Since G2 is an outside girder, half of the overhang weight is assumed placed on the girder and the other half is placed on the overhang brackets, as shown in Figure D-1.

The bracket loads are assumed to be applied uniformly although the brackets are actually spaced at approximately 3 feet along the girder.

The unbraced length of the top flange is approximately 16.3 feet in Span 1. Assume that the average deck thickness in the overhang is 10 inches. The weight of the deck finishing machine is not considered.

Compute the vertical load on the overhang brackets.

$$\text{Deck:}\quad \frac{1}{2} \times 4 \times \frac{10}{12} \times 150 = 250 \quad \text{lbs/ft}$$

Deck forms + Screed rail = 224 lbs/ft

Uniform load on brackets = 474 lbs/ft

Compute the lateral force on the flanges due to overhang brackets. See Figure D-1.

$$\alpha = \tan^{-1}\left(\frac{78}{67.5}\right) = 49.1 \text{ degrees}$$

$$F_l = \frac{474}{\tan\left[49.1\left(\frac{2\pi}{360}\right)\right](1000)} = 0.411 \quad \text{k/ft}$$

Since the example girder is a tub box girder, the provisions of Article 6.10.3.2 are used. Compute the lateral flange moment on the outermost tub flange due to the overhang forces. The lateral flange moment at the brace points due to the overhang forces is negative in the top flange of G2 on the outside of the curve because the stress due to the lateral moment is compressive on the convex side of the flange at the brace points. The opposite would be true on the convex side of the G1 top flange on the inside of the curve at the brace points, as illustrated in later calculations. The flange is treated as a continuous beam supported at brace points; therefore, the unfactored lateral moment is calculated as follows:

$$M_l = F_l L_b^2 / 12 = -\left[\frac{0.411\,(16.3)^2}{12}\right] = -9.1 \quad \text{k-ft} \qquad \text{Eq (C6.10.3.4-2)}$$

Girder Stress Check Section 1-1 G2 Node 10
Constructibility - Top Flange in Compression (continued)

In addition to the moment due to the overhang brackets, the inclined webs of the box cause a lateral force on the top flanges. This force is relatively small in this case and will be ignored. A third source of lateral bending is due to curvature, which can be conservatively estimated by the approximate V-load Equation (C4.6.1.2.4b-1) given in the LRFD Specifications, as illustrated below.

From Table C1, the moment due to the steel weight plus Cast #1 is 4,123 k-ft. The load factor for constructibility is 1.25 according to the provisions of Article 3.4.2. Using the section properties from Table C5, the bending stress, f_{bu}, in the top flange without consideration of longitudinal warping is computed as:

$$f_{top\ flg} = f_{bu} = -\left[\frac{4123(12)(42.80)}{185187}\right](1.25) = -14.29 \quad \text{ksi} \quad \text{(C)}$$

The top flange size is constant between brace points in this region. In positive moment regions, the largest value of f_{bu} may not necessarily be at either brace point. Generally though, f_{bu} will not be significantly larger than the value at adjacent brace points, which is the case in this example. Therefore, the computed value of f_{bu} at Section 1-1 will be conservatively used in the strength check. The approximate Eq (C4.6.1.2.4b-1) is used below to compute the lateral flange bending moment due to curvature. Eq (C4.6.1.2.4b-1) assumes the presence of a cross frame at the point under investigation and, as mentioned previously, M is constant over the distance between brace points. Although the use of Eq (C4.6.1.2.4b-1) is not theoretically pure for tub girders or at locations in-between brace points, it may conservatively be used. Note that the vertical web depth has been conservatively used in the following calculation. For a single flange, consider only half of the girder moment due to steel and Cast #1. M = 4,123/2 = 2062 k-ft.

$$M_{lat} = \frac{M l^2}{NRD} = -\left[\frac{2062\,(16.3)^2}{10(716.25)(6.5)}\right] = -11.77 \quad \text{k-ft} \qquad \text{Eq (C4.6.1.2.4b-1)}$$

The lateral flange moment at the brace points due to curvature is negative when the top flanges are subjected to compression because the stress due to the lateral moment is compressive on the convex side of the flange at the brace points. The opposite is true whenever the top flanges are subjected to tension.

$$M_{tot\_lat} = [-11.77 + (-9.1)](1.25) = -26.09 \quad \text{k-ft (factored)}$$

f_l is defined as the flange lateral bending stress determined using the provisions of Article 6.10.1.6 This value may be determined directly from first-order elastic analysis in discretely braced compression flanges if the following is satisfied.

Girder Stress Check Section 1-1 G2 Node 10
Constructibility - Top Flange in Compression (continued)

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

where:

$$L_p = \quad 1.0r_t\sqrt{\frac{E}{F_{yc}}} \quad = \quad \frac{1.0(3.77)\sqrt{\frac{29000}{50}}}{12} = 7.57 \quad \text{ft.} \qquad \text{Eq (6.10.8.2.3-4)}$$

where:

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1 + \frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\right)}} \qquad \text{Eq (6.10.8.2.3-9)}$$

$$= \frac{16}{\sqrt{12\left[1 + \frac{1}{3}\frac{43.08(0.5625)}{16(1)}\right]}} = 3.77 \quad \text{n.}$$

Since the stresses remain reasonably constant over the section, C_b is taken as 1.0.

Article 6.10.1.10.2 indicates that the web load-shedding factor, R_b, is taken as 1.0 for constructibility.

Check the relation given in Eq (6.10.1.6-2):

$$L_b = 16.3 \text{ ft.} < \quad 1.2(7.57)\sqrt{\frac{1.0(1.0)}{\frac{|-14.29|}{50}}} = 16.99 \quad .$$

Therefore, the flange lateral bending stress, f_l, may be determined from first-order elastic analysis.

$$S_{top\_flange} = \frac{1.0\,(16)^2}{6} = 42.7 \quad \text{in}^3$$

$$f_l = \frac{M_{tot\_lat}}{S_{top\_flange}} = \frac{-26.09(12)}{42.7} = -7.3 \quad \text{ksi} \quad \text{(factored)}$$

Girder Stress Check Section 1-1 G2 Node 10
Constructibility - Top Flange in Compression (continued)

Another significant source of lateral flange bending not considered in this calculation is the forces that develop in single-diagonal top flange bracing members (arranged in the pattern shown in Figure 2 of the introduction section) as a result of vertical bending of the box girder. This effect is recognized in lateral flange moments taken directly from a finite element analysis, but a closed-form solution is more elusive. As mentioned previously, this effect can probably be minimized most effectively by utilizing parallel single-diagonal bracing members in each bay.

For critical stages of construction, the resistance of the compression flange in noncomposite boxes with a single web (including tub flanges) at the constructibility limit state is to be computed according to the criteria given in Article 6.10.3.2. The resistance of a flange and the approximate V-load Eq (C4.6.1.2.4b-1) both assume that the lateral bending is equal at each end of a panel. As can be seen from an examination of Figures 3 through 5 in the introduction section, this is obviously not the case. Check the three relations given for discretely braced compression flanges in Article 6.10.3.2.1

First, check the relation given by Eq (6.10.3.2.1-1).

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

where:

$\phi_f = 1.0$ (Article 6.5.4.2)

$R_h = 1.0$ (Article 6.10.1.10.1)

Therefore, checking the relation given by Eq (6.10.3.2.1-1) we obtain:

$|-14.29| + |-7.3| = 21.59$ ksi $<$ $1.0(1.0)(50) = 50$ ksi **OK**

Second, check the relation given by Eq (6.10.3.2.1-2).

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

Article 6.10.3.2.1 directs the designer to the provisions of Article 6.10.8.2 for the determination of the nominal flexural resistance, F_{nc}, for the top flanges of tub box girders in compression. For constructibility, the web load-shedding factor, R_b, is taken as 1.0. The resistance is taken as the smaller of the local buckling resistance determined as specified in Article 6.10.8.2.2 and the lateral torsional buckling resistance determined using Article 6.10.8.2.3.

The local buckling resistance of the compression flange is computed as follows:

Check if the slenderness ratio, λ_f, of the compression flange is less than or equal to λ_{pf}

$$\lambda_f = \frac{b_{fc}}{2t_c} = \frac{16}{2(1)} = 8 \qquad \text{Eq (6.10.8.2.2-3)}$$

Girder Stress Check Section 1-1 G2 Node 10
Constructibility - Top Flange in Compression (continued)

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_{yc}}} = 0.38\sqrt{\frac{29000}{50}} = 9.15 \qquad \text{Eq (6.10.8.2.2-4)}$$

Since $\lambda_f < \lambda_{pf}$, the nominal flexural resistance is taken as $F_{nc} = R_b R_h F_{yc}$

Since R_b is taken as 1.0 for constructibility, F_{nc} is equal to 50 ksi for the local buckling resistance.

The lateral torsional buckling resistance of the compression flange is computed as follows:

Determine if the unbraced length, L_b, is greater than or less than L_p.

$$L_b = 16.3(12) = 195.6 \text{ in.}$$

$$L_p = 1.0 r_t\sqrt{\frac{E}{F_{yc}}} = 7.57 \text{ ft. (calculated previously)} \qquad \text{Eq (6.10.8.2.3-4)}$$

$$L_p = 7.57(12) = 91 \text{ in.} < L_b = 195.6 \text{ in.}$$

Determine if the unbraced length, L_b, is greater than or less than L_r.

$$L_r = \pi r_t\sqrt{\frac{E}{F_{yr}}} = \pi(3.77)\sqrt{\frac{29000}{0.7(50)}} = 341 \text{ in.} \qquad \text{Eq (6.10.8.2.3-5)}$$

$L_p < L_b < L_r$, therefore, use Eq (6.10.8.2.3-2) for the lateral torsional buckling resistance.

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

where:

C_b = for members where $f_{mid}/f_2 > 1$ or $f_2 = 0$, this value is taken as 1.0. Eq (6.10.8.2.3-6)

f_{mid} = 14.29 ksi (calculated above)

f_2 = largest compressive stress without consideration of lateral bending at either end of the unbraced length of the flange under consideration.

The largest compressive stress in the top flange occurs at Node 12 of the 3D model, approximately 65.04 ft. into span 1.

Girder Stress Check Section 1-1 G2 Node 10
Constructibility - Top Flange in Compression (continued)

Load	Moment	Source
Steel	1,115 k-ft	Appendix B
Cast #1	3,361 k-ft	From separate calculations
Total unfactored moment	4,476 k-ft	

$$f_2 = \frac{4476(12)(42.8)}{185187}(1.25) = 15.52 \quad \text{ksi}$$

$$f_{mid}/f_2 = \frac{14.29}{15.52} = 0.92 \quad < \quad 1.0$$ therefore, C_b is calculated using Eq (6.10.8.2.3-7)

$$C_b = 1.75 - 1.05\left(\frac{f_1}{f_2}\right) + 0.3\left(\frac{f_1}{f_2}\right)^2 \leq 2.3 \qquad \text{Eq (6.10.8.2.3-7)}$$

f_1 = stress without consideration of lateral bending at the brace point opposite to the one corresponding to f_2.

f_1 occurs at Node 8 of the 3D model, approximately 48.78 ft. into span 1.

Load	Moment	Source
Steel	1,116 k-ft	Appendix B
Cast #1	2,588 k-ft	From separate calculations
Total unfactored moment	3,704 k-ft	

$$f_1 = \frac{3704(12)(42.8)}{185187}(1.25) = 12.84 \quad \text{ksi}$$

$$C_b = 1.75 - 1.05\left(\frac{12.84}{15.52}\right) + 0.3\left(\frac{12.84}{15.52}\right)^2 = 1.09$$

Therefore, the lateral torsional buckling resistance is:

$$F_{nc} = 1.09\left[1 - \left[1 - \frac{0.7(50)}{1.0(50)}\right]\left(\frac{195.6 - 91}{341 - 91}\right)\right](1.0)(1.0)(50) = 47.7 \quad \text{ksi} \; < \; 1.0(1.0)(50) = 50 \text{ ksi}$$

Girder Stress Check Section 1-1 G2 Node 10
Constructibility - Top Flange in Compression (continued)

Therefore, the nominal flexural resistance, F_{nc}, of the compression flange is 47.7 ksi.

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

|-14.29| + 1/3(|-7.3|) = 16.72 ksi < 1.0(47.7) = 47.7 ksi   **OK**

Both flange requirements of Article 6.10.3.2.1 for discretely braced flanges in compression are satisfied.

The third requirement which involves the web bend-buckling resistance, Eq (6.10.3.2.1-3), was already satisfied on page D-5 for this section.

Girder Stress Check Section 1-1 G2 Node 10
Strength - Ductility Requirement

According to Article 6.11.6.2.2 for sections in positive flexure, the ductility requirements of Article 6.10.7.3 must be satisfied for compact and noncompact sections inorder to protect the deck from premature crushing. The total depth of the composite section, D_t, is calculated neglecting the haunch thickness and using the structural thickness of the deck. All other field sections are checked similarly (not shown).

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

where:

D_p = distance from the top of the concrete deck to the neutral axis of the composite section at the plastic moment, in.

This value can be calculated by solving for the depth of the web in compression at the plastic moment, D_{cp}, according to Table D6.1-1 of Appendix D, and adding the structural thickness of the deck plus the top flange thickness (Case I) or adding only the structural thickness (Case II).

First, determine if the plastic neutral axis is in the web, top flange, or deck. Neglect the affect of the deck steel.

P_s = force in the slab

$= 0.85 f_c' b_s t_s = \ 0.85(4)(109)(9.5) = 3521$ kips (the effective width, b_s, is taken from separate calculations per Article 4.6.2.6.1)

P_t = force in the compression flange

$= F_{yt} b_t t_t = \ 50(16)(1.0) = 800$ kips

P_c = force in the tension flange

$= F_{yc} b_c t_c = \ 50\left(\frac{83}{2}\right)(0.625) = 1297$ kips

P_w = force in the web

$= F_{yw} D t_w = \ 50(80.4)(0.5625) = 2261$ kips (D is meaured along the incline)

Check if Case I, plastic neutral axis is in the web:

$$P_t + P_w \ge P_c + P_s + P_{rb} + P_{rt}$$

$800 + 2261 = 3061 \ < \ 1297 + 3521 = 4818$ PNA not in the web

Check if Case II, plastic neutral axis is in the top flange:

$$P_t + P_w + P_c \ge P_s + P_{rb} + P_{rt}$$

$800 + 2261 + 1297 = 4358 \ > \ 3521$ PNA is in the top flange

Girder Stress Check Section 1-1 G2 Node 10
Strength - Ductility Requirement (continued)

Therefore, the following expression from Table D6.1-1 is used for determining the location of the plastic neutral axis measured from the top of the top flange.

$$Y_{bar} = \left(\frac{t_c}{2}\right)\left(\frac{P_w + P_t - P_s - P_{rt} - P_{rb}}{P_c} + 1\right)$$

$$= \left(\frac{1}{2}\right)\left(\frac{2261 + 800 - 3521 - 0 - 0}{1297} + 1\right) = 0.3 \quad \text{in.}$$

Therefore,

$$D_p = 0.3 + 9.5 = 9.8 \quad \text{in.} \quad < \quad 0.42D_t = \quad 0.42(0.625 + 80.4 + 1 + 9.5) = 38.4 \quad \text{in.} \quad \textbf{OK}$$

Girder Stress Check Section 5-5 G2 Node 36
Constructibility - Top Flange in Tension

For critical stages of construction, the following requirement must be met for tension in the top flange of a tub girder. It is assumed that the flange is not continuously braced for constructibility.

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

The noncomposite section must be checked for steel weight and for the maximum deck cast loading (Cast #2) of the concrete deck for the section in the negative bending region. The unfactored moments are from Table C1.

Load	Moment
Steel	-3,154 k-ft
Cast #2	-6,915 k-ft
Total unfactored moment	-10,069 k-ft

Determine the factored tensile flange stress, f_{bu}, calculated without consideration of flange lateral bending for the top flanges of tub box girders.

$$f_{bu} = -\left[\frac{-10069(12)(43.69)}{438966}\right](1.25) = 15.03 \quad \text{ksi} \quad \text{(T)}$$

Determine the tensile flange lateral bending stress, f_l, as specified in Article 6.10.1.6. A summary of this calculation is shown here, refer to Section 1-1 computations for the expanded version.

$$M_{lat} = \frac{Ml^2}{NRD} = -\left[\frac{-10069(16.3)^2}{10(716.25)(6.5)}\right] = 57.46 \quad \text{k-ft}$$

Assume the overhang bracket loading is applied to the top flange as well.

M_l = -9.1 k-ft

$$M_{tot\_lat} = [57.46 + (|-9.1|)](1.25) = 83.2 \quad \text{k-ft (factored)}$$

$$S_{top\_flange} = \frac{18^2(3)}{6} = 162 \quad \text{in}^3$$

$$f_l = \frac{M_{tot\_lat}}{S_{top\_flange}} = \frac{83.2(12)}{162} = 6.16 \quad \text{ksi (factored)}$$

Therefore,

15.03 + 6.16 = 21.19 ksi < 1.0(1.0)(50) = 50 ksi **OK**

Girder Stress Check Section 1-1 G1 Node 9
Constructibility - Top Flange in Compression

The load on the overhang bracket produces a lateral flange moment at the brace points on the convex side of the G1 inner top flange of the opposite sense from that on the convex side of the G2 outer top flange. Therefore, check the constructibility stress in the G1 top flange on the inside of the curve at this section. The basis of the following calculations is similiar to the girder G2 check.

Compute the bending moment in the box. Moment values used below are not tabulated.

Load	Moment	
Steel	1,075 k-ft	(Unfactored results are shown)
Cast #1	2,732 k-ft	
Total moment	3,807 k-ft	

Compute the lateral flange bending moment due to curvature using Eq (C4.6.1.2.4b-1). For a single flange, consider only half of the girder moment due to Steel plus Cast #1 according to Article C6.11.3.2.

M = 3,807/2 = 1,904 k-ft

Unbraced length of flange = 15.7 ft.

$$M_{lat} = \frac{Ml^2}{NRD} = -\left[\frac{1904\,(15.7)^2}{10(683.75)(6.5)}\right] = -10.56 \text{ k-ft (C)} \qquad \text{Eq (C4.6.1.2.4b-1)}$$

Compute the lateral flange moment due to the overhang bracket load.

$$M_l = \frac{0.411\,(15.7)^2}{12} = 8.44 \text{ k-ft}$$

Therefore, the total unfactored lateral moment is:

$$M_{lat\_tot} = -10.56 + 8.44 = -2.1 \text{ k-ft}$$

Compute the total factored lateral flange bending stress, f_l. From separate calculations, a first-order elastic analysis can be used to compute the flange lateral bending stress. The cross-section, and its associated properties, at this location for girder G1 are the same as previously calculated for girder G2.

$$f_l = \frac{M_{lat\_tot}}{S_{top\_flange}} = \frac{-2.1(12)}{42.7}(1.25) = -0.74 \text{ ksi}$$

Therefore, check Eq (6.10.3.2.1-1)

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

|-13.2| + |-0.74| = 13.94 ksi < 1.0(1.0)(50) = 50 ksi **OK**

Girder Stress Check Section 1-1 G1 Node 9
Constructibility - Top Flange in Compression (continued)

Check the second constructibility requirement according to Article 6.10.3.2.1.

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

Determine the nominal flexural resistance for compression flanges of tub box girders according to Article 6.10.3.2.1.

The local buckling resistance of the compression flange is computed as follows:

From previous calculations:

$\lambda_f = 8$

$\lambda_{pf} = 9.15$

$F_{nc} = R_b R_h F_{yc} = 1.0(1.0)(50) = 50$ ksi

The lateral torsional buckling resistance of the compression flange is computed as follows:

$L_b = 15.7(12) = 188$ in.

$L_p = 91$ in.

$L_r = 341$ in.

$L_p < L_b < L_r$, therefore, use Eq (6.10.8.2.3-2) for the lateral torsional buckling resistance.

Assume $C_b = 1.0$ for this girder for this example, a detailed calculation for C_b is given for girder G2

$$F_{nc} = 1.0\left[1 - \left[1 - \frac{0.7(50)}{1.0(50)}\right]\left(\frac{188 - 91}{341 - 91}\right)\right](1.0)(1.0)(50) = 44.2 \text{ ksi} < 1.0(1.0)(50) = 50 \text{ ksi}$$

Therefore, the nominal flexural resistance, F_{nc}, of the compression flange is 44.2 ksi.

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

$|-13.2| + 1/3(|-0.74|) = 13.45$ ksi $< 1.0(44.2) = 44.2$ ksi **OK**

Lastly, check the web bend-buckling given by Eq (6.10.3.2.1-3). The nominal web bend-buckling resistance, F_{crw}, was calculated previously for girder G2 and is the same for this location.

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

$|-13.2|$ ksi $< 1.0(39.99) = 39.99$ ksi **OK**

Girder Stress Check Section 1-1 G1 Node 9
Constructibility - Top Flange in Tension

The constructibility check for the top flange in tension may be conducted using the same procedure given for girder G2 at Section 5-5.

Girder Stress Check Section 1-1 G2 Node 10
Fatigue - Bottom Flange

Check the fatigue stress in the bottom flange at this section according to the provisions of Articles 3.6.1.4 and 6.11.5. The fatigue design life is 75 years.

Base metal at transverse stiffener weld terminations and at stiffener-connection plate welds at locations subject to a net tensile stress must be checked for Category C' (refer to Table 6.6.1.2.3-1). It is assumed that stiffener-connection plates are fillet welded to the bottom flange. Thus, the base metal at the top of the bottom flange adjacent to the weld must be checked for Category C'. It is further assumed that the 75-year ADTT in a single lane will exceed the value of 745 trucks/day for a Category C' detail above which the fatigue resistance is governed by the constant-amplitude fatigue threshold (refer to Table C6.6.1.2.5-1).

One factored fatigue vehicle is to be placed at critical locations on the deck per the AASHTO-LRFD fatigue provisions. According to Table 3.6.2.1-1, the dynamic load allowance is 0.15. Centrifugal force effects are not included in this example, but should be considered by the designer. One-half of the fatigue threshold is specified as the limiting stress range for this case since it is assumed that at some time in the life of the bridge, a truck loading of twice the magnitude of the factored fatigue truck will occur. By using half of the fatigue threshold, twice the factored truck is actually considered. According to the provisions of Article 6.6.1.2.1, uncracked concrete section properties are to be used for fatigue checks. As specified in Article 6.11.5, the stress range due to longitudinal warping is considered in checking the fatigue resistance of the base metal at all details on the box section (6.6.1). The transverse bending stress range is considered separately in evaluating the the fatigue resistance fo the base metal adjacent to flange-to-web fillet welds and adjacent to the termination of fillet welds connecting transverse elements to webs and box flanges. In this example, the fatigue moments have been increased by 10 percent to allow for warping.

	Moment		
M_{min}	-239 k-ft	Table C1	(factored)
M_{max}	1,258 k-ft	Table C1	(factored)
M_{range}	1,497 k-ft		

According to Article 6.6.1.2, the limiting stress range for Category C' = 6 ksi for the case where the fatigue resistance is governed by the constant-amplitude fatigue threshold. The value of 6 ksi is equal to one-half of the fatigue threshold of 12 ksi specified for a Category C' detail in Table 6.6.1.2.5-3.

Compute the range of vertical bending stress at the top of the bottom flange (section properties are taken from Table C5):

$$f_{range} = \frac{1497(68.84 - 0.625)}{479646}(12) = 2.55 \text{ ksi}$$

$f_{range} < \phi F_n$

2.55 ksi < 6.0 ksi **OK**

Girder Stress Check Section 1-1 G2 Node 10
Fatigue - Shear Connectors

The shear connectors are designed according to the provisions of Article 6.11.10 which refers to Article 6.10.10.

The longitudinal fatigue shear range per unit length, V_{fat}, for one top flange of a tub girder must be computed for the web subjected to additive flexural and torsionial shears.

Determine the required pitch, p, of the shear connectors for fatigue at this section according to the provisions of Article 6.10.10.1.2. The pitch, p, of the shear connectors must satisfy the following:

$$p \le \frac{n Z_r}{V_{sr}} \qquad \text{Eq (6.10.10.1.2-1)}$$

The fatigue threshold for one stud shear connector in kips, Z_r, is defined in Article 6.10.10.2 as follows:

$$Z_r = \alpha d^2 \ge \frac{5.5d^2}{2}$$

Since a value for $(ADTT)_{SL}$ is needed for the calculation of α, for purposes of this example this value has not been calculated. When traffic data is available, check αd^2.

Use: 3 - 6 in. x 7/8 in. dia. studs/row.

Fatigue threshold for one 7/8 in. dia. shear stud = $\frac{5.5}{2}(0.875)^2 = 2.105$ kips.

Fatigue threshold for 3 such connectors/row = nZ_r = $3(2.105) = 6.315$ kips/row.

From Table C1, the bending plus torsional shear range due to one factored fatigue truck = 23 + (-14) = 37 kips. The shear values in Table C1 are vertical shears and are for the critical web, which is subject to additive bending and torsional shears. The values have been increased by 10 percent to account for warping.

According to the provisions of Article 4.5.2.2, the entire deck cross sectional area is assumed to be effective. The structural deck thickness, t_s, is 9.5 in. The modular ratio, n, is 7.56. Calculate the effective width according to Article 4.6.2.6.1. For open boxes, the effective flange width of each web should be determined as though each web was an individual supporting element.

The effective width of the interior beam is the lesser of:

One-quarter of the effective span length: $0.25(112.4)(12) = 337$ in.

$12.0t_s$ + the greater of t_w or $1/2b_f$: $12(9.5) + 0.5625 = 115$ in.

or $12(9.5) + 0.5(16) = 122$ in. <- controls

The average spacing of adjacent beams: $11.25(12) = 135$ in.

Girder Stress Check Section 1-1 G2 Node 10
Fatigue - Shear Connectors (continued)

The effective width of the exterior beam is taken as on-half the effective width of the interior beam plus the lesser of:

One-eighth of the effective span length: $0.125(112.4)(12) = 169$ in.

$6.0t_s$ + the greater of t_w or $1/4b_f$: $6(9.5) + 0.5625 = 58$ in.

or $6(9.5) + 0.25(16) = 61$ in.

The width of the overhang: $4.0(12) = 48$ in. <- controls

$$\text{Transformed deck area} = \frac{\text{Area}}{n} = \frac{\left(\frac{122}{2} + 48\right)(9.5)}{7.56} = 137 \ \ \text{in}^2$$

Compute the first moment of the deck with respect to the neutral axis of the uncracked live load composite section.

Determine the distance from the center of the deck to the neutral axis.
Section properties are from Table C5.
Neutral axis of the section is 10.78 in. from the top of the steel.
Moment arm of the deck = Neutral axis - t_{flg} + haunch + $t_s/2$

$$\text{Moment arm} = 10.78 - 1.0 + 4.0 + \frac{9.5}{2} = 18.53 \ \ \text{in}^2$$

The horizontal fatigue shear range per unit length, V_{sr}, is determined as the vector sum of the longitudinal fatigue shear range, V_{fat}, given by Eq (6.10.10.1.2-3) and the torsional fatigue shear range in the concrete deck.

Compute the longitudinal fatigue shear range, V_{fat}, for one top flange of the tub girder. Use one-half of the moment of inertia.

$$Q = 137(18.53) = 2539 \ \ \text{in}^3$$

$$V_{fat} = \frac{VQ}{I} = \frac{37(2539)}{0.5(479646)} = 0.39 \ \ \text{k/in.} \qquad \text{Eq (6.10.10.1.2-3)}$$

The torsional fatigue shear range in the concrete deck is computed as:

$$T_{fat} = |-85| + 174 = 259 \ \ \text{k-ft} \qquad \text{(Table C3)}$$

Compute the enclosed area within the composite box section including the midheight of the deck.

$$A_o = \frac{[\,120 + [\,83 - 2(1)\,]\,]}{2}\left(\frac{0.625}{2} + 78 + 1 + \frac{9.5}{2}\right)\left(\frac{1}{144}\right) = 59 \ \ \text{ft}^2$$

Girder Stress Check Section 1-1 G2 Node 10
Fatigue - Shear Connectors (continued)

$$F_{fat} = \frac{T}{2A_o} = \frac{259}{2(59)(12)} = 0.18 \quad \text{k/in.}$$

Therefore,

$$V_{sr} = \sqrt{\left(V_{fat}\right)^2 + \left(F_{fat}\right)^2}$$

$$= \sqrt{(0.39)^2 + (0.18)^2} = 0.43$$

Compute the required shear connector pitch for fatigue for 3 studs per row.

$$\text{Shear stud pitch} = \frac{nZ_r}{V_{sr}} = \frac{6.315}{0.43} = 14.7 \quad \text{in./row}$$

Although not illustrated here, the number of shear connectors that is provided must also be checked at the strength limit state according to the provisions of Article 6.11.10 and subsequently Article 6.10.10.4.

Girder Stress Check Section 8-8 G2 Node 48
Shear Connectors - Maximum Transverse Spacing

Check the maximum transverse spacing, s_t, between shear connectors on composite box flanges using Eq 6.11.10-1. This limit insures that local buckling of the flange is prevented when it is subject to compression. In positive bending regions, the maximum torque occurs at Section 8-8, therefore, the maximum transverse shear connector spacing is determined at this location.

$$\frac{s_t}{t_f}\sqrt{\frac{F_{yf}}{k\,E}} \le R_1 \qquad \text{Eq (6.11.10-1)}$$

where:

k = 4.0 according to Article 6.11.8.2 for the plate-buckling coefficient for uniform normal stress

R_1 = limiting slenderness ratio for the box flange determined using Eq 6.11.8.2.2-8.

$$= \frac{0.57}{\sqrt{\frac{1}{2}\left[\Delta + \sqrt{\Delta^2 + 4\left(\frac{f_v}{F_{yc}}\right)^2\left(\frac{k}{k_s}\right)^2}\right]}} \qquad \text{Eq (6.11.8.2.2-8)}$$

where:

Load	Torque (k-ft)		
Steel	1.25(72)	= 90.0	(Table C3)
Deck	1.25(211)	= 263.75	
SupImp	1.25(125)	= 156.25	
FWS	1.5(164)	= 246.0	
LL + IM	1.75(839)	= 1468.25	
Total factored torque		= 2224.25 k-ft	

Section Properties at Section 8-8

Top flange:	16 in. x 1.0 in.	(compression flange)
Web:	78 in. x 0.5625 in.	
Bottom flange:	83 in. x 0.75 in.	

$$A_o = \frac{[\,120 + [\,83 - 2(1)\,]\,]}{2}\left(\frac{0.75}{2} + 78 + \frac{1}{2}\right)\left(\frac{1}{144}\right) = 55 \quad \text{ft}^2$$

$$f_v = \frac{T}{2A_o\,t_{fc}} = \frac{2224.25(12)}{2(55)\,144(1.0)} = 1.69 \quad \text{ksi}$$

Girder Stress Check Section 8-8 G2 Node 48
Shear Connectors - Maximum Transverse Spacing (continued)

$$\Delta = \sqrt{1 - 3\left(\frac{f_v}{F_{yc}}\right)^2} = \sqrt{1 - 3\left(\frac{1.69}{50}\right)^2} = 0.998 \qquad \text{Eq (6.11.8.2.2-5)}$$

Therefore,

$$R_1 = \frac{0.57}{\sqrt{\frac{1}{2}\left[0.998 + \sqrt{0.998^2 + 4\left(\frac{1.69}{50}\right)^2\left(\frac{4.0}{5.34}\right)^2}\right]}} = 0.57$$

Solve for the maximum transverse shear connector spacing, s_t.

$$\frac{s_t}{1.0}\sqrt{\frac{50}{4.0(29000)}} = 0.57$$

$$s_t = \frac{1.0(0.57)}{\sqrt{\frac{50}{4.0(29000)}}} = 27.45$$ in. > shear connector spacing provided = 14.7 in. from fatigue calculations

Girder Stress Check Section 5-5 G2 Node 36
Strength - Bottom Flange

Check the bottom (box) flange for strength at this section according to the provisions of Article 6.11.8.2 for compression flanges in negative flexure. The section will be checked for the Strength I limit state in the following computations. Assume one longitudinal compression flange stiffener.

Load	Moment	
Steel	-3,154 k-ft	All values are from Table C1
Deck	-12,272 k-ft	Unfactored results are shown
Total noncomposite	-15,426 k-ft	
Superimposed DL	-1,932 k-ft	
FWS	-2,541 k-ft	
LL + IM	-8,127 k-ft	

The dynamic load allowance has been applied to the live load according to Article 3.6.2. Multiple presence factors, specified in Table 3.6.1.1.2-1 were also considered in the live load analysis.

Compute the factored vertical bending stress in the bottom flange due to dead and live load. For loads applied to the composite section, assume a cracked section, as specified in Article 4.5.2.2. Section properties are from Table C5. Shear lag need not be considered since the box flange width does not exceed one-fifth of the span of the bridge (Article C6.11.1.1). The longitudinal vertical bending stress is, therefore, assumed to be uniform across the flange because shear lag need not be considered and because it is assumed that the spacing of the internal bracing is such that the longitudinal warping stress at the strength limit state is limited to 10 percent of the stresses due to major-axis bending (Article C6.7.4.3).

$$f_{bot\ flg} = f_{bu} = \left[\frac{\gamma_{DC} M_{DC} C_{nc}}{I_{nc}} + \frac{(\gamma_{DC2} M_{DC2} + \gamma_{DW} M_{DW})C_{3n}}{I_{3n}} + \frac{\gamma_{LL} M_{LL} C_n}{I_n}\right](12)\,\eta$$

$$= \left[\frac{1.25(-15426)(38.81)}{438966} + \frac{[\,1.25(-1932) + 1.5(-2541)\,]39.76}{454805} + \frac{1.75(-8127)(41.55)}{484714}\right](12)(1) = -41.6 \text{ ksi (C)}$$

Compute the factored St. Venant torsional shear stress, f_v, in the bottom flange due to the noncomposite loads. Torques are taken from Table C3.

Load	Torque	
Steel	1.25(-22)	= -28 k-ft
Deck	1.25(48)	= 60 k-ft
Total Noncomposite Torque		= 32 k-ft

Girder Stress Check Section 5-5 G2 Node 36
Strength - Bottom Flange (continued)

The nominal flexural resistance of the compression flange of a longitudinally stiffened flange is determined according to Article 6.11.8.2.3.

Compute the enclosed area within the noncomposite box section, A_o.

$$A_o = \frac{[120 + [83 - 2(1)]]}{2}\left(\frac{1.5}{2} + 78 + \frac{3}{2}\right)\left(\frac{1}{144}\right) = 56 \quad ft^2$$

$$f_v = \frac{T}{2A_o t_{fc}} = \frac{32(12)}{2(56)(144)(1.5)} = 0.016 \quad ksi \qquad \text{Eq (6.11.8.2.2-6)}$$

where: T = internal torque from factored loads (k-ft); t_f = bottom flange thickness (in.)

Compute the factored torsional shear stress in the bottom flange due to the composite loads. Torques are taken from Table C3.

Load	Torque (-)		Torque (+)	
SupImp DL	1.25(-149)	= -186 k-ft	1.25(193)	= 241 k-ft
FWS	1.50(-197)	= -296 k-ft	1.50(254)	= 381 k-ft
LL + IM	1.75(-863)	= -1,510 k-ft	1.75(980)	= 1,715 k-ft
Total Comp. Torque		= -1,992 k-ft		= 2,337 k-ft

Since $|-1992| < 2337$, use positive torque.

Compute the enclosed area of the composite box, A_o.

$$A_o = \frac{(120 + 81)}{2}(80.25 + 7.25)\left(\frac{1}{144}\right) = 61.1 \ ft^2$$

Therefore, the factored torsional shear stress is:

$$f_v = \frac{T}{2A_o t_{fc}} = \frac{2337(12)}{2(61.1)(144)(1.5)} = 1.06 \quad ksi$$

$$f_{v\,tot} = 0.016 + 1.06 = 1.08 \quad ksi$$

Check the applied torsional stress against the factored torsional shear resistance of the flange, F_{vr}.

$$F_{vr} = 0.75\phi_v \frac{F_{yf}}{\sqrt{3}} \qquad \text{Eq (6.11.1.1-1)}$$

$$= 0.75(1.0)\left(\frac{50}{\sqrt{3}}\right) = 21.65 \quad ksi \ > \ f_{v\,tot} = 1.08 \ ksi$$ **OK**

Girder Stress Check Section 5-5 G2 Node 36
Strength - Bottom Flange (continued)

Although the torques on the noncomposite and composite box act in opposite directions, the resulting shear flows are conservatively added together in determining the total factored torsional shear stress. Generally this is acceptable because of the small magnitude of the noncomposite torque.

Compute the slenderness ratio for the compression flange to determine which nominal flexural resistance equation to use. According to Article 6.11.8.2.3, "b_{fc}" is taken as "w" in the following design. The variable "w" is taken as the larger of the width of the flange between longitudinal flange stiffeners or the distance from a web to the nearest longitudinal flange stiffener.

$$\lambda_f = \frac{w}{t_{fc}} = \frac{\left(\frac{81}{2}\right)}{1.5} = 27$$

$$R_1 = \frac{0.57}{\sqrt{\frac{1}{2}\left[\Delta + \sqrt{\Delta^2 + 4\left(\frac{f_v}{F_{yc}}\right)^2\left(\frac{k}{k_s}\right)^2}\right]}} \qquad \text{Eq (6.11.8.2.2-8)}$$

where:

$$\Delta = \sqrt{1 - 3\left(\frac{f_v}{F_{yc}}\right)^2} = \sqrt{1 - 3\left(\frac{1.08}{50}\right)^2} = 0.999 \qquad \text{Eq (6.11.8.2.2-5)}$$

for flanges with one longitudinal stiffener (n = 1), the plate-buckling coefficient for uniform normal stress, k, is taken as:

$$k = \left(\frac{8I_s}{w t_{fc}^3}\right)^{\frac{1}{3}} \qquad \text{Eq (6.11.8.2.3-1)}$$

$$k_s = \frac{5.34 + 2.84\left(\frac{I_s}{w t_{fc}^3}\right)^{\frac{1}{3}}}{(n+1)^2} \le 5.34 \qquad \text{Eq (6.11.8.2.3-3)}$$

where: n = number of equally spaced longitudinal flange stiffeners
I_s = actual moment of inertia of one longiudinal flange stiffener about an axis parallel to the flange at the base of the stiffener (in^4)

t_{fc} = thickness of the flange plate (in.)
w = larger of the width of the flange between longitudinal flange stiffeners or the distance from a web to the nearest longitudinal flange stiffener (in.)

Girder Stress Check Section 5-5 G2 Node 36
Strength - Bottom Flange (continued)

From the AISC Manual of Steel Construction: I_l = 48.7 in^4; A = 8.38 in^2; N.A.= 6.275 in. from the tip of the stem (i.e. edge of the bottom flange)

Compute the moment of inertia about the base of the stiffener.

$$I_l = 48.7 + 8.38(6.275)^2 = 378.7 \quad in^4$$

$$k = \left[\frac{8(378.7)}{40.5(1.5)^3}\right]^{\frac{1}{3}} = 2.81$$

$$k_s = \frac{5.34 + 2.84\left[\dfrac{378.7}{40.5(1.5)^3}\right]^{\frac{1}{3}}}{(1+1)^2} = 2.33$$

$$R_1 = \frac{0.57}{\sqrt{\dfrac{1}{2}\left[0.999 + \sqrt{(0.999)^2 + 4\left(\dfrac{1.08}{50}\right)^2\left(\dfrac{2.81}{2.33}\right)^2}\right]}} = 0.57$$

$$R_1\sqrt{\frac{kE}{F_{yc}}} = 0.57\sqrt{\frac{2.81(29000)}{50}} = 23.01 < \lambda_f = 27.0$$

$$R_2 = \frac{1.23}{\sqrt{\dfrac{1}{1.2}\left[\dfrac{F_{yr}}{F_{yc}} + \sqrt{\left(\dfrac{F_{yr}}{F_{yc}}\right)^2 + 4\left(\dfrac{f_v}{F_{yc}}\right)^2\left(\dfrac{k}{k_s}\right)^2}\right]}} \qquad \text{Eq (6.11.8.2.2-9)}$$

where:

$$F_{yr} = (\Delta - 0.4)F_{yc} \le F_{yw} \qquad \text{Eq (6.11.8.2.2-7)}$$

$$= (0.999 - 0.4)(50) = 29.95 \quad ksi$$

Girder Stress Check Section 5-5 G2 Node 36
Strength - Bottom Flange (continued)

$$R_2 = \frac{1.23}{\sqrt{\frac{1}{1.2}\left[\frac{29.95}{50} + \sqrt{\left(\frac{29.95}{50}\right)^2 + 4\left(\frac{1.08}{50}\right)^2\left(\frac{2.81}{2.33}\right)^2}\right]}} = 1.23$$

$$R_2\sqrt{\frac{k\,E}{F_{yc}}} = 1.23\sqrt{\frac{2.81(29000)}{50}} = 49.7 \quad > \quad 27.0\text{, therefore, use Eq (6.11.8.2.2-2)}$$

The nominal flexural resistance of the compression flange, F_{nc}, is taken as:

$$F_{nc} = R_b\,R_h\,F_{yc}\left[\Delta - \left(\Delta - \frac{F_{yr}}{R_h\,F_{yc}}\right)\left[1 - \sin\left[\frac{\pi}{2}\left(\frac{R_2 - \frac{w}{t_{fc}}\sqrt{\frac{F_{yc}}{k\,E}}}{R_2 - R_1}\right)\right]\right]\right] \qquad \text{Eq (6.11.8.2.2-2)}$$

R_h is taken as 1.0 from Article 6.10.1.10.1 and R_b is calculated using the provisions of Article 6.10.1.10.2. For composite sections in negative flexure that satisfy Eq 2, R_b is also taken as 1.0 since the web slenderness, $2D_c/t_w$, is at or below the value at which the theoretcial elastic bend-buckling stress is equal to F_{yc} at the strength limit state.

$$\frac{2D_c}{t_w} \le \lambda_{rw} \qquad \text{Eq (6.10.1.10.2-2)}$$

D_c is calculated using the provisions of Article D6.3.1. These provisions state that for composite sections in negative flexure, D_c is computed for the section consisting of the steel girder plus the longitudinal deck reinforcement. For this example, the larger value between "Comp DL Bars" and "Comp LL Bars" from Table C5 was used.

$$D_c = (41.55 - 1.5)\left(\frac{4.123}{4.0}\right) = 41.28 \quad \text{in. (along the inclined web)}$$

$$\frac{2(41.28)}{0.5625} = 147 \quad > \quad \lambda_{rw} = 137$$

Since this relation is not satisfied, R_b must be calculated using Eq (6.10.1.10.2-3)

$$R_b = 1 - \left(\frac{a_{wc}}{1200 + 300a_{wc}}\right)\left(\frac{2D_c}{t_w} - \lambda_{rw}\right) \le 1.0 \qquad \text{Eq (6.10.1.10.2-3)}$$

where:

$$a_{wc} = \frac{2D_c t_w}{b_{fc} t_{fc}}$$ Eq (6.10.1.10.2-5)

$$= \frac{2(41.28)(0.5625)}{83(1.5)} = 0.373$$

$\lambda_{rw} = 137$ Eq (6.10.1.10.2-4)

$$R_b = 1 - \left[\frac{0.373}{1200 + 300(0.373)}\right]\left[\frac{2(41.28)}{0.5625} - 137\right] = 0.997$$

$$F_{nc} = 0.997(1.0)(50)\left[0.999 - \left[0.999 - \frac{29.95}{1.0(50)}\right]\left[1 - \sin\left[\frac{\pi}{2}\left[\frac{1.23 - \frac{\left(\frac{81}{2}\right)}{1.5}\sqrt{\frac{50}{2.81(29000)}}}{1.23 - 0.57}\right]\right]\right]\right] = 49.25 \text{ ksi}$$

$f_{bu} = |-41.6|$ ksi < $\phi_f F_{nc} = 49.25$ ksi **OK** Eq (6.11.7.2.1-1)

Article C6.11.8.1.1 states that in general, bottom box flanges at interior-pier sections are subjected to biaxial stresses due to major-axis bending of the box section and major-axis bending of the internal diaphragm over the bearing sole plate. The flange is also subject to shear stresses due to the internal diaphragm vertical shear, and in cases where it needs to be considered, the St. Venant torsional shear. For a box supported on two bearings (the case in this example), bottom-flange bending stresses due to bending of the diaphragm over the bearing sole plates are relatively small and will be neglected for simplicity in this example.

From previous calculations, the total factored St. Venant torsional shear stress in the bottom flange, f_v, is equal to 1.08 ksi.

To estimate the shear stress in the bottom flange due to the internal diaphragm shear, assume a 1 in. x 12 in. top flange for the diaphragm. As specified in Article C6.11.8.1.1, a flange width equal to 18 times its thickness (18 x 1.5 in. = 27 in.) may be considered effective with the internal diaphragm. The diaphragm is assumed to be 78 inches deep and 1 inch thick. From separate calculations, the moment of inertia of the effective section is 112,375 in^4 and the neutral axis is located 31.05 in. above the mid-thickness of the bottom flange. Subsequent calculations on page D-34 indicate that the total factored vertical component of the diaphragm shear is 1406 kips.

Girder Stress Check Section 5-5 G2 Node 36
Strength - Bottom Flange (continued)

The shear stress in the flange, f_d, caused by the internal diaphragm vertical shear due to factored loads is approximated as:

$$f_d = \frac{VQ}{I\left(t_{fc}\right)} = \frac{1406\left(\frac{27}{2}\right)(1.5)(31.05)}{112375(1.5)} = 5.24 \text{ ksi} \qquad \text{Eq (C6.11.8.1.1-2)}$$

$$f_{v\ tot} = 1.08 + 5.24 = 6.32 \text{ ksi}$$

The effect of bending in the plane of the diaphragm for boxes supported on two bearings is insignificant and was, therefore, ignored in the design of the example girder. The effect of these forces on a box supported on a single bearing is likely to be more significant and should be considered. The effective section specified in Article C6.11.8.1.1 may be used to compute the flange bending stress about the tangential z-axis due to bending of the internal diaphragm over the sole plate. In this case, the resulting minimum and maximum principal stresses in the flange should be input into the more general form of the Huber-von Mises-Hencky yield criterion given as follows:

$$\sqrt{f_{bu}^2 - f_{bu} f_{by} + f_{by}^2 + 3\left(f_d + f_v\right)^2} \le \phi_f R_b R_h F_{yc} \qquad \text{Eq (C6.11.8.1.1-1)}$$

The factored vertical bending stress in the bottom flange, f_{bu}, was computed earlier to be -41.6 ksi.

f_{by} = stress in the flange due to the factored loads caused by major-axis bending of the internal diaphragm over the bearing sole plate

= taken as 0.0 ksi for a box supported on two bearings

R_h = 1.0 (Article 6.10.1.10.1)

R_b = 0.997 (previously calculated)

$$\sqrt{(-41.6)^2 - (-41.6)(0) + (0)^2 + 3(5.24 + 1.08)^2} = 43.02 \text{ ksi} \quad < \quad 1.0(0.997)(1.0)\,50 = 49.85 \text{ ksi}$$

The combined principal stresses in the diaphragm due to the factored loads is checked using the general form of the Huber-von Mises-Hencky yield criterion.

$$\sqrt{\sigma_1^2 - \sigma_1\sigma_2 + \sigma_2^2} \le F_y$$

where σ_1, σ_2 are the maximum and minimum principle stresses in the diaphragm

Girder Stress Check Section 5-5 G2 Node 36
Strength - Bottom Flange (continued)

$$\sigma_1, \sigma_2 = \left(\frac{\sigma_y + \sigma_z}{2}\right) \pm \sqrt{\left(\frac{\sigma_y - \sigma_z}{2}\right)^2 + f_v^{\,2}}$$

σ_y = stress in the diaphragm due to vertical bending of the diaphragm over the bearing sole plate
σ_z = stress in the diaphragm due to bending of the diaphragm about its longitudinal axis
f_v = shear stress in the diaphragm
F_y = specified minimum yield stress of the diaphragm

Since the example box is supported on two bearings, the stress in the diaphragm due to vertical bending of the diaphragm over the bearing sole plate is typically relatively small and will be neglected for simplicity in this example. σ_z is also typically neglected. If no bending is assumed, the two principal stresses are simply equal to the tensile and compressive stresses with a magnitude equal to the shear stress.

$$\sigma_{1,2} = 0 \pm \sqrt{(0)^2 + 6.32^2} = \pm 6.32 \text{ ksi}$$

Check the combined principal stresses.

$$\sqrt{6.32^2 - (6.32)(-6.32) + (-6.32)^2} = 10.95 \text{ si} < 36 \text{ ksi} \quad \textbf{OK}$$

Girder Stress Check Section 5-5 G2 Node 36
Longitudinal Flange Stiffener

Try a WT 8x28.5 structural tee for the longitudinal stiffener with the stem welded to the bottom flange.

The projecting width, b_l, of the stiffener must satisfy the following requirement:

$$b_l \le 0.48 t_s \sqrt{\frac{E}{F_{yc}}} \quad = \quad 0.48(0.715)\sqrt{\frac{29000}{50}} = 8.27 \text{ in.} \qquad \text{Eq (6.11.11.2-1)}$$

where t_s is taken as the flange thickness of the structural tee since each half-flange buckles similarly to a single plate connected to a web.

For structural tees, b_l should be taken as one-half the width of the flange.

b_l = 7.12/2 = 3.56 in. < 8.27 in. **OK**

According to Article 6.7.4.3, transverse top and bottom bracing members (i.e. top and bottom struts of internal cross frames) are required to ensure that the cross section shape is retained. Whenever longitudinal flange stiffeners are present, the bottom transverse bracing members are to be attached to the longitudinal stiffener(s) to better control the transverse distortion of the box flange. At other locations, the bottom transverse member is to be attached directly to the box flange. The cross-sectional area and stiffness of the top and bottom transverse bracing members is not to be less than the area and stiffness of the diagonal members. At the pier section (the point of maximum compressive flexural stress in a box flange in most cases), the bottom transverse bracing member, when properly attached to the longitudinal flange stiffener, can be assumed to provide the required transverse stiffening of the box flange. Use a W10x68 (I = 394 in^4) for the bottom transverse bracing member.

The longitudinal flange stiffener should be attached to the internal diaphragm with a pair of clip angles as shown in Figure D-2 (page D-81).

Girder Stress Check Section 5-5 G2 Node 36
Design of the Internal Diaphragm

Article 6.11.1 directs the designer to the provisions of Article 6.7.4 for general design considerations for cross-frames and diaphragms.

Try a 1-inch thick A36 diaphragm plate.

Compute the maximum factored vertical shear in the diaphragm.

Load	Shear		Source
Steel	47 + |-46|	= 93 k	3D Finite Element Analysis
Deck	185 + |-185|	= 370 k	(in critical web from Table C2)
SupImp	44 + |-41|	= 85 k	Unfactored results are shown
FWS	58 + |-55|	= 113 k	
LL + IM	160 + |-155|	= 315 k	

$$V_u = 1.25(93 + 370 + 85) + 1.5(113) + 1.75(315) = 1406 \text{ kips}$$

The internal diaphragm is subject to vertical bending over the bearing sole plates in addition to shear. Therefore, Article 6.11.8.1.1 requires that the principal stresses in support diaphragms not exceed the factored compressive resistance given by Eq (C6.11.8.1.1-1), which is a yield criterion for combined stress. The example box is supported by two bearings, therefore, f_{by} in this equation is taken as 0.0 ksi since it is typically relatively small.

Compute the maximum factored shear stress in the diaphragm web. First, separate out the shears due to bending, V_b, and due to St. Venant torsion, V_T.

The sum of the total steel plus deck factored shears is equal to 1.25(93 + 370) = 579 kips. Referring to the calculations on page D-25, the shear flow in the noncomposite box is computed as:

$$f_v = \frac{T}{2A_o} = \frac{32}{2(56)(12)} = 0.024 \text{ k/in.} \qquad \text{Eq (C6.11.1.1-1)}$$

$$V_T = 0.024(80.4) = 1.93 \text{ kips}$$

The vertical component of V_T is computed as:

$$(V_T)_v = 1.93\left(\frac{78}{80.4}\right) = 1.87 \text{ kips}$$

$$V_b = 579 - 1.87 = 577.1 \text{ kips}$$

Girder Stress Check Section 5-5 G2 Node 36
Design of Internal Diaphragm (continued)

The sum of the total Superimposed Dead Load, including the FWS, plus Live Load factored shears is equal to 1.25(85) + 1.5(113) + 1.75(315) = 827 kips. Referring to the calculations on page D-26, the shear flow in the composite box is computed as:

$$f_v = \frac{T}{2A_o} = \frac{2337}{2(61.1)(12)} = 1.59 \quad \text{k/in.} \qquad \text{Eq (C6.11.1.1-1)}$$

$$V_T = 1.59(80.4) = 127.8 \quad \text{kips}$$

The vertical component of V_T is computed as:

$$(V_T)_v = 127.8\left(\frac{78}{80.4}\right) = 124 \quad \text{kips}$$

$$V_b = 827 - 124 = 703 \quad \text{kips}$$

The factored shear stress due to torsion is therefore equal to:

$$(f_v)_T = \frac{0.024}{1.0} + \frac{1.59}{1.0} = 1.61 \quad \text{ksi}$$

The average factored shear stress due to bending is equal to:

$$(f_v)_b = \frac{577.1 + 703}{78(1.0)} = 16.4 \quad \text{ksi}$$

As mentioned previously, for a box supported on two bearings, the bending stresses in the plane of the diaphragm due to vertical bending of the diaphragm over the bearing sole plates are relatively small and will be neglected in this example for simplicity. For a box supported on a single bearing, the effect of the bending stresses in the plane of the diaphragm are likely to be more significant and should be considered. As specified in Article C6.11.8.1.1, a width of the bottom (box) flange equal to 18 times its thickness may be considered effective with the diaphragm in resisting bending.

Therefore, for this case, since bending in the plane of the diaphragm is ignored, the maximum principal stress is simply equal to the total factored shear stress.

$$f_v = (f_v)_T + (f_v)_b = 1.61 + 16.4 = 18.01 \quad \text{ksi}$$

Girder Stress Check Section 5-5 G2 Node 36
Design of Internal Diaphragm (continued)

The combined principal stresses in the diaphragm due to the factored loads is checked using the general form of the Huber-von Mises-Hencky yield criterion (similarly calculations shown previously).

$$\sqrt{\sigma_1^2 - \sigma_1\sigma_2 + \sigma_2^2} \le F_y$$

where σ_1, σ_2 are the maximum and minimum principle stresses in the diaphragm

$$\sigma_1, \sigma_2 = \left(\frac{\sigma_y + \sigma_z}{2}\right) \pm \sqrt{\left(\frac{\sigma_y - \sigma_z}{2}\right)^2 + f_v^2}$$

where:

$\sigma_y = 0$
$\sigma_z = 0$

If no bending is assumed, the two principal stresses are simply equal to the tensile and compressive stresses with a magnitude equal to the shear stress.

$$\sigma_{1,2} = 0 \pm \sqrt{(0)^2 + 18.01^2} = \pm 18.01 \text{ ksi}$$

Check the combined principal stresses.

$$\sqrt{18.01^2 - (18.01)(-18.01) + (-18.01)^2} = 31.19 \text{ ksi} < F_y = 36 \text{ ksi} \quad \textbf{OK}$$

Compute the shear resistance according to Article 6.11.9 which specifies the use of the provisions of Article 6.10.9 for the horizontally curved I-girder design. Separate calculations indicate that C = 1.0.

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

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

$$V_p = 0.58F_yDt_w = 0.58(36)(78)(1.0) = 1629 \text{ kips} \qquad \text{Eq (6.10.9.2-2)}$$

$$V_n = 1.0(1629) = 1629 \text{ kips}$$

$$V_u = 1406 \text{ k} < \phi_v V_n = 1.0(1629) = 1629 \text{ k} \quad \textbf{OK}$$

Girder Stress Check Section 5-5 G2 Node 36
Design of Bearing Stiffeners

Compute the factored reactions.

	Reaction Location		
Load	Left	Right	Source
Steel	79 k	93 k	3D Finite Element Analysis
Deck	238 k	370 k	(Not tabulated)
SupImp	85 k	11 k	Unfactored results are shown
FWS	113 k	15 k	
Total DL	515 k	489 k	
LL + IM	294 k	287 k	
	-65 k	-16 k	uplift

$$R_{left} = 1.25(79 + 238 + 85) + 1.5(113) + 1.75(294) = 1187 \text{ kips (controls)}$$

$$R_{right} = 1.25(93 + 370 + 11) + 1.5(15) + 1.75(287) = 1117 \text{ kips}$$

Ignore uplift.

Assume that the bearings are fixed at the piers. Thus, there will be no expansion causing eccentric loading on the bearing stiffeners. Bearing stiffeners should be attached to diaphragms rather than inclined webs. According to Article 6.11.11.1, design the bearing stiffeners attached to the diaphragms using the provisions of Article 6.10.11.2.4b. The provisions are applied to the diaphragm rather than the web.

Use bars with F_y = 50 ksi. Compute the maximum permissible width-to-thickness ratio of the stiffener plates according to Eq (6.10.11.2.2-1).

Try 2-plates 11 in. x 1 in.; Bearing area = 2(11 - 1.0)(1.0) = 20.0 in^2 (Assume 1 in. for stiffener clip).

$$b_t \le 0.48\, t_p \sqrt{\frac{E}{F_{ys}}} = 0.48(1)\sqrt{\frac{29000}{50}} = 11.6 > 11 \text{ in. } \textbf{OK} \qquad \text{Eq (6.10.11.2.2-1)}$$

Compute the effective area of the diaphragm, A_d, to which the stiffeners are attached (t_w = 1.0 in.) according to the provisions of Article 6.10.11.2.4b.

$$A_d = 18 t_w^2 \left(\frac{F_{yw}}{F_{ys}}\right) = 18\,(1.0)^2\left(\frac{36}{50}\right) = 12.96 \text{ in}^2$$

Girder Stress Check Section 5-5 G2 Node 36
Design of Bearing Stiffeners (continued)

The factored bearing resistance for the fitted ends of bearing stiffeners is determined using the provisions of Article 6.10.11.2.3.

$$(R_{sb})_r = \phi_b(R_{sb})_n \qquad \text{Eq (6.10.11.2.3-1)}$$

The nominal bearing resistance for the fitted ends, $(R_{sb})_n$, is:

$$(R_{sb})_n = 1.4A_{pn}F_{ys} \qquad \text{Eq (6.10.11.2.3-2)}$$

where:

A_{pn} = area of the projecting elements of the stiffener outside the web-to-flange fillet welds but not beyond the edge of the flange (in^2)
= 20 in^2

F_{ys} = 50 ksi

$$(R_{sb})_n = 1.4(20)(50) = 1400 \quad k$$

The resistance factor, ϕ_b, is taken to be 1.0 from Article 6.5.4.2.

1187 k < 1.0(1400) = 1400 k **OK**

Assume the concentrated load is applied concentrically with respect to the centroidal axes of the bearing stiffener. The bearing stiffener assembly may then be designed as a centrally loaded compression member according to the provisions of Article 6.10.11.2.4 for the axial resistance of bearing stiffeners. These provisions state that the factored axial resistance, P_r, is determined using Article 6.9.2.1 where the radius of gyration is computed about the mid-thickness of the web and the effective length shall be taken as 0.75D.

$$P_r = \phi_c P_n \qquad \text{Eq (6.9.2.1-1)}$$

where: P_n = nominal compressive resistance as specified in Article 6.9.4
ϕ_c = resistance factor according to Article 6.5.4.2

Calculate P_n. Check if λ is greater than or less than 2.25.

$$\lambda = \left(\frac{Kl}{r_s\pi}\right)^2\left(\frac{F_y}{E}\right) \qquad \text{Eq (6.9.4.1-3)}$$

Girder Stress Check Section 5-5 G2 Node 36
Design of Bearing Stiffeners (continued)

Calculate r_s.

$$r_s = \sqrt{\frac{I}{A}}$$

$$A = 12.96 + (2)11 = 34.96 \quad in^2$$

$$I = 2\left[\frac{1(11)^3}{12} + 11(6)^2\right] = 1014 \quad in^4$$

$$r_s = \sqrt{\frac{1014}{34.96}} = 5.39$$

Therefore;

$$\lambda = \left[\frac{0.75(80.4)}{5.39(\pi)}\right]^2\left(\frac{50}{29000}\right) = 0.022 \quad < \ 2.25, \text{ use Eq (6.9.4.1-1) for } P_n$$

$$P_n = 0.66^{\lambda}F_yA_s = 0.66^{0.022}(50)(34.96) = 1732$$ ‹ Eq (6.9.4.1-1)

$$P_r = \phi_c P_n$$

$$P_r = (0.9)(1732) = 1559 \quad k$$

$P_u = 1187$ k $<$ $P_r = 1559$ k **OK**

Girder Stress Check G2 Span 1 Bay 1
Top Flange Bracing Member Design - Constructibility

Top flanges of tub girders subjected to torsional loads need braced so that the section acts as a pseudo-box for noncomposite loads applied before the concrete deck hardens or is made composite. Design the top (tub) flange single diagonal bracing member in Span 1 of Girder 2 in the first bay adjacent to the abutment (Element 1 in Table C4). Article 6.11.1 specifies that the top lateral bracing for tub sections must satisfy the provisions of Article 6.7.5. Tub flange bracing must be designed to satisfy the constructibility limit state as well as the final condition. Since lateral bracing is not required for continuously braced flanges, investigate the bracing for the construction staging only. The bracing is designed according to the provisions of Articles 6.8 and 6.9 for tension and compression, respectively. Wind lateral loading is neglected in this example.

Load	Force	Source
Steel	-13 k	Table C4 (from 3D finite element analysis)
Cast #1	-100 k	Unfactored results are shown
	-113 k	

Load Factor = 1.25 (Article 3.4.2); Design load = $1.25(-113) = -141$ kips (C)

Tub width at top = 120 in.; top flange width = 16 in.
Clear distance between top flanges = 120 - 16 = 104 in.
Distance between cross frames = 16.3 feet = 195.6 in., say 196 in.
Compute the bracing length, L_c.

$$L_c = \sqrt{104^2 + 196^2} = 222 \text{ in.}$$

Try a structural tee (WT) section with the stem down and its flange bolted to the bottom of the tub flanges, which is the preferable method of connection. Assume that a timber member will brace the structural tee at mid-length in the vertical plane during construction. Therefore, the unbraced length with respect to the x-axis equals 222/2 = 111 in. The unbraced length with respect to the y-axis = 222 in.

Try: WT 9x48.5
From AISC Manual: A = 14.3 in^2; y = 1.91 in.; S_x = 12.7 in^3; r_x = 2.56 in.; r_y = 2.65 in.

Check buckling about the y-axis. The limiting slenderness ratio for compression members is specified in Article 6.9.3. Use the effective length factor, as specified in Article 4.6.2.5, for bolted connections at both ends.

$$\frac{Kl}{r_y} = \frac{0.75(222)}{2.65} = 62.8 \quad < \quad 140 \quad \textbf{OK}$$

The provisions of Article 6.9.4.1 are used to determine the nominal compressive resistance of non-composite members when Eq (6.9.4.2-1) is satisfied. There is no eccentricity with respect to the y-axis.

$$\frac{b}{t} \le k\sqrt{\frac{E}{F_y}} \qquad \text{Eq (6.9.4.2-1)}$$

Girder Stress Check G2 Span 1 Bay 1
Top Flange Bracing Member Design - Constructibility (continued)

The plate buckling coefficient is taken as 0.75 from Table 6.9.4.2-1 for stems of rolled tees. The width, b, is taken as the full depth of the tee section and thickness, t, is for that of the stem.

$$\frac{9.295}{0.535} = 17.4 \quad < \quad 0.75\sqrt{\frac{29000}{50}} = 18.1 \quad \textbf{OK}$$

Therefore, the provisions of Article 6.9.4.1 apply. Determine if Eq (6.9.4.1-1 or -2) is to be used for the nominal compressive resistance, P_n.

$$\lambda = \left(\frac{Kl}{r_s\pi}\right)^2 \frac{F_y}{E} \quad = \quad \left[\frac{0.75(222)}{2.65\pi}\right]^2 \left(\frac{50}{29000}\right) = 0.69 \qquad \text{Eq (6.9.4.1-3)}$$

Since λ is less than 2.25, Eq (6.9.4.1-1) is used for the calculation of P_n.

$$P_n = 0.66^{\lambda} F_y A_s \qquad \text{Eq (6.9.4.1-1)}$$

$$= 0.66^{0.69}(50)(14.3) = 537 \quad \text{k}$$

$P_u = |-141|$ k $<$ $P_r = \phi_c P_n = 0.9(537) = 483.3$ k **OK**

Check buckling about the x-axis.

Consider the eccentricity of the connection.
Compute the moment due to the eccentricity of the force at the flange face.

$$M_{ux} = 141(1.91) = 269 \quad \text{k-in}$$

Verify that the limiting slenderness ratio of Article 6.9.3 is satisfied.

$$\frac{Kl}{r_x} = \frac{0.75(111)}{2.56} = 32.5 \quad < \quad 140 \quad \textbf{OK}$$

Use the provisions of Article 6.9.2.2 to check the resistance of the member under combined axial compression and flexure.

$$\frac{P_u}{P_r} = \frac{|-141|}{483.3} = 0.292 \quad > \quad 0.2\text{, therefore, use Eq (6.9.2.2-2)}$$

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0 \qquad \text{Eq (6.9.2.2-2)}$$

Girder Stress Check G2 Span 1 Bay 1
Top Flange Bracing Member Design - Constructibility (continued)

Determine the factored flexural resistance about the x-axis using the provisions of Article 6.12 for the miscellaneous flexural members.

$$M_{rx} = \phi_f M_n \qquad \text{Eq (6.12.1.2.1-1)}$$

Use the provisions of Article 6.12.2.2.4 for noncomposite structural tees. The nominal bending resistance is the lowest value as limited by yielding, lateral torsional buckling or local buckling of the elements. The Specifications direct the designer to the AISC Manual for Steel Construction for the determination of M_n. The nominal flexural resistance, M_n, is the lowest value according to the limit states of: yielding, lateral- torsional buckling, flange local buckling and web local buckling. For unbraced compact and noncompact tees, only the yielding and lateral-torsional buckling limit states are applicable.

Yielding:

$$M_n = M_p = F_y Z \le 1.5M_y \qquad \text{AISC Eq (F1-1)}$$

where:

$$M_y = F_y S = \frac{50(12.7)}{12} = 52.92 \quad \text{k-ft}$$

$1.5M_y = 79.38$ k-ft

$M_p = 50(22.6)/12 = 93.75$ k-ft $>$ 79.38 k-ft, therefore, $M_n = 79.38$ k-ft

Lateral-torsional buckling:

For tee sections, use Chapter F1.2c in the Specification section of the AISC Manual.

$$M_n = M_{cr} = \frac{\pi\sqrt{E I_y G J}}{L_b}\left(B + \sqrt{1 + B^2}\right) \qquad \text{AISC Eq (F1-15)}$$

where:

$M_n \le 1.5M_y$ for stems in tension, this is used for this example since the member is in positive bending. $1.5M_y$ was calculated previously as 79.38 k-ft.

$M_n \le 1.0M_y$ for stems in compression

$E = 29{,}000$ ksi

$I_y = 100\ \text{in}^4$

G = shear modulus of elasticity of steel, 11,200 ksi

Girder Stress Check G2 Span 1 Bay 1
Top Flange Bracing Member Design - Constructibility (continued)

$$J = \text{torsional constant} = \Sigma \frac{b\,t^3}{3}$$

$$= \frac{11.145\,(0.870)^3 + 8.425\,(0.535)^3}{3} = 2.88$$

$$B = 2.3\left(\frac{d}{L_b}\right)\sqrt{\frac{I_y}{J}} = 2.3\left(\frac{9.295}{222}\right)\sqrt{\frac{100}{2.88}} = 0.567$$ AISC Eq (F-16)

Therefore, the lateral-torsional buckling nominal flexural resistance is:

$$M_n = \frac{\pi\sqrt{29000(100)(11200)(2.88)}}{222}\left(0.567 + \sqrt{1 + 0.567^2}\right) = 7430 \cdot in$$

619.2 k-ft > $1.5M_y$

M_{rx} is taken as the value for yielding, 1.0(79.38) = 79.38 k-ft in Eq (6.9.2.2-2)

$$0.292 + \frac{8.0}{9.0}\left[\frac{269}{79.38(12)} + 0\right] = 0.54 \quad < 1.0 \ \textbf{OK}$$

Girder Stress Check Section 5-5 G2 Node 36
Transverse Bending Stress

Article 6.11.1.1 requires that the transverse bending stresses in webs and flanges be investigated and determined by rational structural analysis. These provisions limit the transverse bending stresses due to the factored loads at the strength limit state to 20 ksi. The transverse bending stress range due to cross-section distortion must be checked for fatigue as specified in Article 6.11.5 and at the strength limit state. Longitudinal warping stresses due to cross-section distortion are considered for fatigue as specified in Article 6.11.5, but may be ignored at the strength limit state.

The most critical condition is likely to be fatigue at the termination of fillet welds connecting transverse stiffeners to the web (Category E).

The "Design Guide to Box Girder Bridges," Bethlehem Steel Corporation, 1981, presents a method developed by Wright and Abdel-Samed (1968) to estimate transverse bending stresses using the Beam on Elastic Foundation Analogy (BEF). In this method, the deflection of the BEF is analogous to the transverse bending stress.

The fatigue loading produces a positive torque of 254 k-ft and a negative torque of -232 k-ft at the pier, Section 5-5 Node 36, as given in Table C3. The total range of factored torque is 486 k-ft (5,832 k-in).

I_{comp} = 836,080 in^4 (from Table C5)
Minimum transverse stiffener spacing = 62 in. (Calculations not shown)
Cross frame spacing = 16.3 ft. = 196 in.
t_c = web thickness = 0.5625 in.
t_b = bottom flange thickness = 1.50 in.
t_a = slab thickness = 9.5 in.
E_c = 3,834 ksi; E_s = 29,000 ksi

Transverse stiffener - try a plate 5.5 in. x 0.5 in.

Poisson's ratio for concrete, μ_c = 0.2 (Article 5.4.2.5); Poisson's ratio for steel, μ_s = 0.30

Compute the transverse flexural rigidities of the deck and bottom flange from Bethlehem Guide Equations (A3a) and (A3b), respectively.

D_a = flexural rigidity of deck; D_b = flexural rigidity of bottom flange

$$D_a = \frac{E_c t_a^3}{12\left(1 - \mu_c^2\right)} = \frac{3834\,(9.5)^3}{12\left(1 - 0.2^2\right)} = 285345 \text{ k-in}^2\text{/in.}$$

$$D_b = \frac{E_s t_b^3}{12\left(1 - \mu_s^2\right)} = \frac{29000\,(1.5)^3}{12\left(1 - 0.30^2\right)} = 8963 \text{ k-in}^2\text{/in.}$$

D_c = flexural rigidity of web

Compute D_c considering the transverse stiffeners according to Bethlehem Guide Equation (A3d) since Article 6.11.1.1 permits transverse stiffeners to be considered effective in resisting transverse bending.

Girder Stress Check Section 5-5 G2 Node 36
Transverse Bending Stress (continued)

Compute d_o in Figure D-4 using Equation (A4) from the Bethlehem Guide.

$$d_o = \frac{d \tanh\left(5.6\frac{d}{h}\right)}{5.6\frac{d}{h}\left(1-\mu_s^{\,2}\right)}$$ Bethlehem Guide Eq (A4)

d = 62 in. spacing of transverse stiffeners.

c = 80.4 in.

h = c

$$d_o = \frac{(62)\tanh\left[5.6\left(\frac{62}{80.4}\right)\right]}{5.6\left(\frac{62}{80.4}\right)\left(1-0.3^2\right)} = 15.8 \quad \text{in.}$$

Compute the location of the neutral axis of the effective section from the web face.

Area of stiffener = 5.5 x 0.5 = 2.75 in^2
Area of web = 15.8 x 0.5625 = 8.89 in^2
11.64 in^2

$$\text{N.A.} = \frac{2.75\left(0.5625+\frac{5.5}{2}\right)+8.89\left(\frac{0.5625}{2}\right)}{11.64} = 1.0 \quad \text{in.}$$

$$I_s = \left(\frac{1}{12}\right)(0.5)\,(5.5)^3 + 2.75\left(\frac{5.5}{2}+0.5625-1.0\right)^2 + \frac{1}{12}(0.5625)^3(15.8) + 8.89\left(\frac{0.5625}{2}-1\right)^2 = 26.5 \quad \text{n.}$$

$$D_c = \frac{E_s I_s}{d} = \frac{29000(26.5)}{62} = 12395 \quad \text{k-in}^2\text{/in.}$$ Bethlehem Guide Eq (A3d)

The stiffness of the transverse stiffener is assumed to be distributed evenly along the web.

Girder Stress Check Section 5-5 G2 Node 36
Transverse Bending Stress (continued)

Compute the compatibility shear at the center of the box (bottom) flange according to Bethlehem Guide Equation (A2).

$$v = \frac{\frac{1}{D_c}[(2a + b)abc] + \frac{1}{D_a}ba^3}{(a + b)\left(\frac{a^3}{D_a} + \frac{2c\left(a^2 + ab + b^2\right)}{D_c} + \frac{b^3}{D_b}\right)} \qquad \text{Bethlehem Guide Eq (A2)}$$

$$v = \frac{\frac{1}{12395}[[2(120) + 81](120)(81)(80.4)] + \frac{1}{285345}(81)\left(120^3\right)}{(120 + 81)\left[\frac{120^3}{285345} + \frac{2(80.4)\left[120^2 + 120(81) + 81^2\right]}{12395} + \frac{81^3}{8963}\right]} = 0.22$$

Compute δ_1, the box distortion per kip per inch of load without diaphragms, according to Equation (A1) from the Bethlehem Guide.

$$\delta_1 = \frac{ab}{24(a + b)}\left[\frac{c}{D_c}\left(\frac{2ab}{a + b} - v(2a + b)\right) + \frac{a^2}{D_a}\left(\frac{b}{a + b} - v\right)\right] \qquad \text{Bethlehem Guide Eq (A1)}$$

$$\delta_1 = \frac{120(81)}{24(120 + 81)}\left[\frac{80.4}{12395}\left[\frac{2(120)(81)}{120 + 81} - 0.22[2(120) + 81]\right] + \frac{120^2}{285345}\left(\frac{81}{120 + 81} - 0.22\right)\right] = 0.36 \ \mathrm{ı}^2/\mathrm{k}$$

Compute the BEF stiffness parameter, β, using Bethlehem Guide Equation (A5).

$$\beta = \sqrt[4]{\frac{1}{EI_c\delta_1}} \qquad \text{Bethlehem Guide Eq (A5)}$$

where: I = moment of inertia of the composite box

$$\beta = \sqrt[4]{\frac{1}{29000(836080)(0.36)}} = 0.00327 \ \ \mathrm{in}^{-1}$$

$$\beta \mathit{l} = 0.00327(196) = 0.64$$

where: l = the distance between cross frames (in.)

Girder Stress Check Section 5-5 G2 Node 36
Transverse Bending Stress (continued)

The transverse bending stress range at the top or bottom corners of the box section may be determined from Bethlehem Guide Equation (A8).

$$\sigma_t = C_t F_d \beta \frac{1}{2a} (m\mathit{l} \text{ or } T)$$ Bethlehem Guide Equation (A8)

where: C_t = BEF factor for determining the transverse distortional bending stress from Bethlehem Guide Figure A6.
m = uniform range of torque per unit length.
l = cross frame spacing
a = distance between webs at the top of the box
T = range of concentrated torque
F_d = (bv)/(2S) for bottom corner of the box
= a/(2S)[b/(a + b) - v] for top corner of box
S = section modulus of the transverse member (per inch)

Compute the section modulus, S, for stiffened portions of the web.

$$S = \frac{I}{c} = \frac{26.5}{(5.5 + 0.5625 - 1.0)} = 5.23 \text{ in}^3$$

Compute S per inch.

$$S = \frac{5.23}{62} = 0.084 \text{ in}^3\text{/in.}$$

$$F_d = \frac{bv}{2S} = \frac{81(0.22)}{2(0.084)} = 106 \text{ in}^{-1}$$ Bethlehem Guide Equation (A9a)

Compute S (per inch) for unstiffened portions of the web (more critical than the bottom flange).

$$S = \frac{(1)(0.5625)^2}{6} = 0.0527 \text{ in}^3\text{/in.}$$

For the bottom corner of the box, $F_d = \frac{bv}{2S}$

$$F_d = \frac{81(0.22)}{2(0.0527)} = 169 \text{ in}^{-1}$$

Girder Stress Check Section 5-5 G2 Node 36
Transverse Bending Stress (continued)

For the top corner of the box, $F_d = \dfrac{a}{2S}\left(\dfrac{b}{a+b} - v\right)$ Bethlehem Guide Equation (A9b)

$$F_d = \frac{120}{2(0.084)}\left(\frac{81}{120+81} - 0.22\right) = 131 \text{ in}^{-1}$$ (Stiffened)

$$F_d = \frac{120}{2(0.0527)}\left(\frac{81}{201} - 0.22\right) = 208 \text{ in}^{-1} \text{ (controls)}$$ (Unstiffened)

Compute f_t using Bethlehem Guide Equation (A8).

$$\sigma_t = C_t F_d \beta \frac{T}{2a}$$ Bethlehem Guide Equation (A8)

Read C_t from Bethlehem Guide Figure A6: $C_t = 0.03$

$$\sigma_t = 0.03(208)(0.00327)\left[\frac{5832}{2(120)}\right] = 0.5 \text{ ksi} < 20.0 \text{ ksi} \quad \textbf{OK}$$

The quantity, q, in Figure A6 represents the ratio of the diaphragm brace stiffness to the box stiffness per unit length. For the βl value in this example, the curves for q = 1,000 to infinity are clustered around a C_t value of 0.03. Therefore, $C_t = 0.03$ is used. For other cases, q may be calculated from Equation (A6) in the Bethlehem Guide (not shown). An additional example of the computation of transverse bending stresses is also given in the Bethlehem Guide.

The transverse bending stress range caused by the fatigue loading is negligible in this case.

Girder Stress Check Section 2-2 G2 Node 20.3
Stresses

Check the bottom flange bending stress at Section 2-2, which is located 100 feet from the abutment. Since this is the location of the bolted field splice in Span 1, it is desirable to terminate the longitudinal flange stiffener at this location where the longitudinal stress at the free edge of the flange is zero. By terminating the longitudinal flange stiffener at the free edge of the flange (at the bolted splice) and not extending it further into the end span, fatigue of the base metal at the terminus of the stiffener-to-flange weld need not be considered. The bottom flange splice plate inside the box must be split to permit the stiffener to extend to the free edge of the flange (Figure D-5). Also, the compressive resistance of the unstiffened bottom (box) flange on the side of the field splice directly across from the stiffener termination must be checked at the strength limit state to ensure that the stiffener can be terminated at this section. The section properties of the section without the flange stiffener are used below. The effect of the concrete reinforcement in the stress calculation is neglected in this example.

Compute the vertical bending stresses in the top extreme fiber of the steel at this section. Moments are from Table C1 and section properties are from Table C5. In this particular case, the girder sections immediately to the left and right of Section 2-2 are the same (except for the flange stiffener).

$$f_{top}\text{ (Steel)} = -\frac{1.25(462)(42.80)}{185187}(12) = -1.6 \quad \text{ksi (C)}$$

$$f_{top}\text{ (Deck)} = -\frac{1.25(1941)(42.80)}{185187}(12) = -6.73 \quad \text{ksi (C)}$$

$$f_{top}\text{ (Superimposed DL)} = -\frac{1.25(326)(24.27)}{354925}(12) = -0.33 \quad \text{ksi for 3n (C)}$$

$$f_{top}\text{ (Superimposed DL)} = -\frac{1.25(326)(42.80)}{185187}(12) = -1.13 \quad \text{ksi for cracked section w/o rebar (C)}$$

$$f_{top}\text{ (FWS)} = -\frac{1.5(428)(24.27)}{354925}(12) = -0.53 \quad \text{ksi for 3n (C)}$$

$$f_{top}\text{ (FWS)} = -\frac{1.5(428)(42.80)}{185187}(12) = -1.78 \quad \text{ksi for cracked section w/o rebar (C)}$$

$$f_{top}\text{ (LL + IM)} = -\frac{1.75(5264)(10.78)}{479646}(12) = -2.48 \quad \text{ksi for n (C)}$$

$$f_{top}\text{ (-LL + IM)} = \frac{1.75(|-3087|)(42.80)}{185187}(12) = 14.98 \quad \text{ksi for cracked section w/o rebar (T)}$$

Girder Stress Check Section 2-2 G2 Node 20.3
Stresses (continued)

Compute the factored vertical bending stress in the top flange at the strength limit state.

$$f_{top} = -1.6 + (-6.73) + (-0.33) + (-0.53) + (-2.48) = -11.67 \quad \text{ksi}$$

$$f_{top} = -1.6 + (-6.73) + (-1.13) + (-1.78) + 14.98 = 3.74 \quad \text{ksi} \quad \text{(for cracked section without rebar)}$$

By similar computations, Tables D-1 and D-2 are created.

Girder Stress Check Section 2-2 G2 Node 20.3
Strength - Bottom Flange

Check the compressive resistance of the unstiffened bottom flange directly across from the flange stiffener termination according to the provisions of Article 6.11.8.2.2.

Compute the St. Venant torsional shear stress in the bottom flange due to the noncomposite loads.

Load	Torque		
Steel	1.25(-36)	= -45 k-ft	All values are from Table C3
Deck	1.25(-125)	= -156 k-ft	
Total Factored NC DL Torque		= -201 k-ft	

Compute the bottom flange shear stress due to the noncomposite loads.

The enclosed area of the noncomposite box is computed to be A_o = 55.0 ft^2.

$$f_v = \frac{T}{2A_o t_{fc}} = \frac{|-201|}{2(55.0)(0.625)}\left(\frac{1}{12}\right) = 0.24 \text{ ksi} \qquad \text{Eq (6.11.8.2.2-6)}$$

where: T = torque; A_o = enclosed area of box; t_{fc} = flange thickness

Compute the St. Venant torsional shear stress in the bottom flange due to the composite loads.

Load	Torque		
SupImp DL	1.25(-58)	= -73 k-ft	All values are from Table C3
FWS	1.50(-76)	= -114 k-ft	
LL + IM	1.75(-556)	= -973 k-ft	
Total Factored Comp. DL Torque		= -1,160 k-ft	

Compute the bottom flange shear stress due to the composite loads.

The enclosed area of the composite box is computed to be A_o = 60.8 ft^2.

$$f_v = \frac{T}{2A_o t_{fc}} = \frac{|-1160|}{2(60.8)(0.625)}\left(\frac{1}{12}\right) = 1.27 \text{ ksi}$$

$$f_v = (0.24 + 1.27) = 1.51 \text{ ksi}$$

Compute the nominal flexural resistance for the bottom flange at the strength limit state according to Article 6.11.8 for sections in negative flexure.

First, determine λ_f, the slenderness ratio for the compression flange.

$$\lambda_f = \frac{b_{fc}}{t_{fc}} = \frac{81}{0.625} = 129.6 \qquad \text{Eq (6.11.8.2.2-4)}$$

where b_{fc} is the flange width between webs measured in inches

Girder Stress Check Section 2-2 G2 Node 20.3
Strength - Bottom Flange

Determine the equation used to compute the nominal flexural resistance.

$$R_1 \sqrt{\frac{k\,E}{F_{yc}}}$$

where:

$$R_1 = \frac{0.57}{\sqrt{\frac{1}{2}\left[\Delta + \sqrt{\Delta^2 + 4\left(\frac{f_v}{F_{yc}}\right)^2\left(\frac{k}{k_s}\right)^2}\right]}} \qquad \text{Eq (6.11.8.2.2-8)}$$

Compute Δ according to Eq (6.11.8.2.2-5).

$$\Delta = \sqrt{1 - 3\left(\frac{f_v}{F_y}\right)^2} \qquad \text{Eq (6.11.8.2.2-5)}$$

$$= \sqrt{1 - 3\left(\frac{1.51}{50}\right)^2} = 0.999$$

$$R_1 = \frac{0.57}{\sqrt{\frac{1}{2}\left[0.999 + \sqrt{0.999^2 + 4\left(\frac{1.51}{50}\right)^2\left(\frac{4.0}{5.34}\right)^2}\right]}} = 0.57$$

k = plate-buckling coefficient for uniform normal stress = 4.0
k_s = plate-buckling coefficient for shear stress = 5.34

$$0.57\sqrt{\frac{4.0(29000)}{50}} = 27.5 \quad < \quad \lambda_f = 129.6$$

Determine if λ_f is less than or greater than $R_2\sqrt{\frac{k\,E}{F_{yc}}}$

$$R_2 = \frac{1.23}{\sqrt{\frac{1}{1.2}\left[\frac{F_{yr}}{F_{yc}} + \sqrt{\left(\frac{F_{yr}}{F_{yc}}\right)^2 + 4\left(\frac{f_v}{F_{yc}}\right)^2\left(\frac{k}{k_s}\right)^2}\right]}} \qquad \text{Eq (6.11.8.2.2-9)}$$

Girder Stress Check Section 2-2 G2 Node 20.3
Strength - Bottom Flange

where:

$$F_{yr} = (\Delta - 0.4)F_{yc} \le F_{yw} \qquad \text{Eq (6.11.8.2.2-7)}$$

$$= (0.999 - 0.4)50 = 29.95 \text{ ksi}$$

$$R_2 = \frac{1.23}{\sqrt{\frac{1}{1.2}\left[\frac{29.95}{50} + \sqrt{\left(\frac{29.95}{50}\right)^2 + 4\left(\frac{1.51}{50}\right)^2\left(\frac{4.0}{5.34}\right)^2}\right]}} = 1.23$$

$1.23\sqrt{\frac{4.0(29000)}{50}} = 59.2$ < 129.6 therefore, use Eq (6.11.8.2.2-3) for the calculation of F_{nc}

$$F_{nc} = \frac{0.9E\,R_b\,k}{\left(\frac{b_{fc}}{t_{fc}}\right)^2} - \frac{R_b\,f_v^{\,2}\,k}{0.9E\,k_s^{\,2}}\left(\frac{b_{fc}}{t_{fc}}\right)^2 \qquad \text{Eq (6.11.8.2.2-3)}$$

where:

R_b is determined using the provisions of 6.10.1.10.2. Since this section is composite, is in positive flexure and satisfies the web proportioning limits of Article 6.10.2.1, R_b is taken as 1.0.

$$F_{nc} = \frac{0.9(29000)(1.0)(4.0)}{\left(\frac{81}{0.625}\right)^2} - \frac{(1.0)(1.51)^2(4.0)}{0.9(29000)\,(5.34)^2}\left(\frac{81}{0.625}\right)^2 = 6.01 \text{ ksi}$$

From Table D-1, the computed factored compressive stress in the bottom flange for strength = -3.22 ksi.

$$f_{bu} \le \phi_f F_{nc} \qquad \text{Eq (6.11.8.1.1-1)}$$

|-3.22| ksi < 1.0(6.01) = 6.01 ksi **OK**

Therefore, the longitudinal flange stiffener may be discontinued at the field splice.

Bolted Splice Design Section 2-2 G2 Node 20.3
Design Action Summary and Section Information

Design the bolted field splice at this section according to the provisions of Article 6.13.6.

Bolt capacities (Articles 6.13.2.3.1 and 6.4.3)

Use 7/8 in. diameter ASTM A 325 bolts. Table 6.13.2.4.2-1 provides a standard hole size of 15/16 in. for a 7/8 in. diameter bolt.

Use a Class B surface condition for unpainted blast-cleaned surfaces. Bolts are in double shear and threads are not permitted in the shear planes.

Service and Constructibility

Slip Resistance (Article 6.13.2.8)

For slip-critical connections, the factored resistance, R_r, of a bolt at the Service II Load Combination is taken as:

$R_r = R_n$ where: R_n = the nominal resistance as specified in Article 6.13.2.8

The nominal slip resistance of a bolt in a slip-critical connection is taken as:

$$R_n = K_h K_s N_s P_t \qquad \text{Eq (6.13.2.8-1)}$$

where:

N_s = number of slip planes per bolt
P_t = minimum required bolt tension specified in Table 6.13.2.8-1 (kips)
K_h = hole size factor specified in Table 6.13.2.8-2
K_s = surface condition factor specified in Table 6.13.2.8-3

$R_n = (1.0)(0.50)(2)(39) = 39$ k/bolt

Strength

The factored resistance, R_r of a bolted connection at the strength limit state shall be taken as:

$R_r = \phi R_n$ where ϕ is specified in Article 6.5.4.2 Eq (6.13.2.2-2)

Article 6.13.6.1.4a states that the factored flexural resistance of the section at the point of the splice at the strength limit state must satisfy the applicable provisions of Article 6.10.6.2.

Bolted Splice Design Section 2-2 G2 Node 20.3
Design Action Summary and Section Information (continued)

Shear Resistance (Article 6.13.2.7)

The nominal shear resistance, R_n, of a high-strength bolt at the strength limit state where the length between extreme fasteners measured parallel to the line of action of the force is less than 50.0 in. and the threads are excluded from the shear plane is as follows:

$R_n = 0.48A_bF_{ub}N_s$ Eq (6.13.2.7-1)

$R_n = 0.48(0.601)(120)(2) = 69.2$ k/bolt

$R_r = \phi_s R_n$ where ϕ_s is the shear resistance factor from Article 6.5.4.2

$R_r = 0.8(69.2) = 55.4$ k/bolt

Bearing Resistance (Article 6.13.2.9)

For standard holes, the nominal resistance of interior and end bolt holes at the strength limit state, R_n, is taken as:

With bolts spaced at a clear distance between holes not less than 2.0d and with a clear end distance not less than 2.0d:

$R_n = 2.4dtF_u$ Eq (6.13.2.9-1)

If either the clear distance between holes is less than 2.0d, or the clear end distance is less than 2.0d:

$R_n = 1.2L_ctF_u$ Eq (6.13.2.9-2)

where:

- d = nominal diameter of the bolt (in.)
- L_c = clear distance between holes or between the hole and the end of the member in the direction of the applied force (in.)
- t = thickness of the connected material (in.)
- F_u = tensile resistance of the connected material specified in Table 6.4.1-1 (ksi)

In this case, the end distance is 2.0 in. creating a clear end distance of 1.5 in. which less than 2.0d, therefore, Eq (6.13.2.9-2) applies. The nominal bolt resistance for the end row of bolts is:

$R_n = 1.2(1.5)(0.5625)(65) = 65.81$ k/bolt

$R_r = \phi_{bb}R_n$ where: ϕ_{bb} is from Article 6.5.4.2

$R_r = 0.8(65.81) = 52.65$ k/bolt

The nominal bolt resistance for the interior rows is computed as:

$R_n =$ $2.4(0.875)(0.5625)(65) = 76.78$ k/bolt Eq (6.13.2.9-1)

$R_r =$ $0.8(76.78) = 61.42$ k/bolt

Tensile Resistance (Article 6.13.2.10)

The nominal tensile resistance of a bolt, T_n, independent of any initial tightening force shall be taken as:

$T_n = 0.76A_bF_{ub}$ Eq (6.13.2.10.2-1)

$T_n =$ $0.76(0.601)(120) = 54.8$ k/bolt

The tensile bolt resistance is not used in this example.

Bolted Splice Design Section 2-2 G2 Node 20.3
Constructibility - Top Flange

Article 6.13.6.1.4a requires that high-strength bolted connections designated as slip critical be proportioned to prevent slip under Load Combination Service II. In addition, bearing, shear, and tensile resistance at the applicable strength limit state load combinations must be provided. Article 6.13.6.1.4a also requires that high-strength bolted connections be proportioned to prevent slip for constructibility.

Constructibility

Since Cast #1 causes a larger positive moment than the entire deck, Steel + Cast #1 controls.
Constructibility: Load factor = 1.25 (Article 3.4.2).

Article 6.13.6.1.4c requires that lateral bending effects in discretely braced top flanges of tub sections be considered in the design of bolted flange splices. Lateral flange bending must be considered for the top flanges of tub girders prior to hardening of the deck. To account for the effects of lateral flange bending, the flange splice bolts will be designed for the combined effects of shear and moment using the traditional elastic vector method. The shear on the bolts is caused by the flange force calculated from the average vertical bending stress in the flange and the moment on the bolts is caused by the lateral flange bending.

Compute the polar moment of inertia of the top flange bolt pattern shown in Figure D-5.

$$I_p = A_b\left[2(3)\left(3.0^2 + 6.0^2\right) + 2(4)\left(3.0^2\right)\right] = 342\ A_b\ in^4$$

where:

A_b = area of one bolt (in^2)

Moment = $462 + 2749 = 3211$ k-ft (unfactored from Table D-3)

Lateral flange moment = $-1 + (-15) = -16$ k-ft (unfactored from Table D-3)

The factored vertical bending stresses for steel and Cast #1 are taken from Table D-2.

$f_{top\ flg}$ = -11.13 ksi

$f_{top\ web}$ = -10.87 ksi

Compute the force in the top flange using the average vertical bending stress in the flange. The gross section of the flange is used to check for slip.

$$F_{top} = \left(\frac{-11.13 - 10.87}{2}\right)(16.0) = -176 \quad \text{kips}$$

Compute the force in each bolt resulting from the vertical bending stress.

$$F_{Long\ vert} = \frac{|-176|}{12} = 14.67 \quad \text{k/bolt}$$

Bolted Splice Design Section 2-2 G2 Node 20.3
Constructibility - Top Flange (continued)

Compute the longitudinal component of force in the critical bolt due to the lateral flange moment.

$$F_{Long\ lat} = \frac{|-16|(6.0)}{342}(12)(1.25) = 4.21 \quad \text{k/bolt}$$

Therefore,

$$F_{Long\ tot} = 14.67 + 4.21 = 18.88 \quad \text{k/bolt}$$

Compute the transverse component of force in the critical bolt.

$$F_{Trans} = \frac{|-16|(3.0)}{342}(12)(1.25) = 2.11 \quad \text{k/bolt}$$

Compute the resultant force on the critical bolt.

$$\Sigma_F = \sqrt{2.11^2 + 18.88^2} = 19 \quad \text{k/bolt}$$

Check $R_u \le R_r$

R_u = 19.0 k/bolt < R_r = 39 k/bolt **OK**

Bolted Splice Design Section 2-2 G2 Node 20.3
Constructibility - Bottom Flange

Since Cast #1 causes a larger positive moment than the entire deck, Steel + Cast #1 controls constructibility. Load factor = 1.25 (Article 3.4.2). The factored vertical bending stresses are taken from Table D-2.

$$f_{bot\ flg} = 9.58 \text{ ksi}$$

$$f_{bot\ web} = 9.42 \text{ ksi}$$

Compute the force in the bottom flange from the average constructibility vertical bending stress. The gross section of the flange is used to check for slip.

$$F_{bot} = \left(\frac{9.58 + 9.42}{2}\right)(51.88) = 493 \quad \text{kips}$$

To account for the effects of the St. Venant torsional shear in the bottom flange, the flange splice bolts will again be designed for the combined effects of shear and moment using the traditional elastic vector method, as illustrated below.

Compute the polar moment of inertia of the bottom flange bolt pattern shown in Figure D-5.

$$I_p = A_b[2(20)(2.25)^2 + 2(2)(2.5^2 + 6.25^2 + 10^2 + 13.75^2 + 17.5^2 + 21.25^2 + 25^2 + 28.75^2 + 32.5^2 + 36.25^2)] = 19{,}859A_b \text{ in}^4$$

Compute the factored St. Venant torsional shear in the bottom flange. From Table D-3 , the unfactored torque due to Steel plus Cast #1 = -36 + (-188) = -224 k-ft. The enclosed area of the noncomposite box, A_o, is computed to be 55.0 ft^2.

$$V = \frac{T}{2A_o}b_f = \frac{|-224|}{2(55.0)}\left(\frac{81}{12}\right)(1.25) = 17.2 \quad \text{kips}$$

Compute the factored moment in the bottom flange due to the torsional shear. Assume the shear is applied at the centerline of the splice (i.e. at the juncture of the two flange plates).

$$M = 17.2(2.25 + 2.25) = 77.4 \quad \text{k-in}$$

Compute the longitudinal component of force in the critical bolt due to the factored moment.

$$F_{Long\ M} = \frac{77.4(36.25)}{19859} = 0.14 \quad \text{k/bolt}$$

Bolted Splice Design Section 2-2 G2 Node 20.3
Constructibility - Bottom Flange (continued)

Compute the force in each bolt resulting from the vertical bending stress.

$$F_{Long\ vert} = \frac{493}{40} = 12.32 \quad \text{k/bolt}$$

Therefore,

$$F_{Long\ tot} = 0.14 + 12.32 = 12.46 \quad \text{k/bolt}$$

Compute the transverse component of force in the critical bolt.

$$F_{Trans} = \frac{77.4(2.25)}{19859} = 0.009 \quad \text{k/bolt}$$

Compute the force in each bolt resulting from the torsional shear.

$$F_v = \frac{17.2}{40} = 0.43 \quad \text{k/bolt}$$

$$F_{Trans\ tot} = 0.009 + 0.43 = 0.44 \quad \text{k/bolt}$$

Compute the resultant force in the critical bolt.

$$\Sigma_F = \sqrt{12.46^2 + 0.44^2} = 12.47 \quad \text{k/bolt}$$

Check $R_u \le R_r$

R_u = 12.47 k/bolt < R_r = 39 k/bolt **OK**

Bolted Splice Design Section 2-2 G2 Node 20.3
Strength - Top and Bottom Flange

The effective area of the top flange is computed from Article 6.13.6.1.4c as follows:

$$A_e = \left(\frac{\phi_u F_u}{\phi_y F_{yt}}\right) A_n \le A_g \qquad \text{Eq (6.13.6.1.4c-2)}$$

where:

F_u = minimum tensile resistance of the tension flange, ksi

F_{yt} = minimum yield resistance of the tension flange, ksi

A_n = net area of the flange calculated as specified in Article 6.8.3, in^2

$= [\, 16.0 - 4(0.875 + 0.125)\,](1.0) = 12 \ \ in^2$

A_g = gross area of the flange, in^2

$= (16.0)(1.0) = 16 \ \ in^2$

$$A_e = \left[\frac{0.8(65)}{0.95(50)}\right](12) = 13.14 \quad in^2 \ < \ 16 \ in^2$$

The effective width of the top flange is computed as:

$$(b_f)_{eff} = \frac{A_e}{t} = \frac{12}{1.0} = 12 \ \ in.$$

Section properties computed using the effective top flange width are used to calculate the vertical bending stresses in the flange at the splice for strength whenever the top flange is subjected to tension. The gross area is used for the bottom flange since it is in compression.

Similarly, the effective area of the bottom flange is computed as:

$$A_n = [\, 83.0 - 20(0.875 + 0.125)\,](0.625) = 39.4 \ \ in^2$$

$$A_g = (83.0)(0.625) = 51.9 \ \ in^2$$

$$A_e = \left[\frac{0.8(65)}{0.95(50)}\right](39.4) = 43.13 \quad in^2 \ < \ 51.9 \ in^2$$

Therefore,

$A_e = 43.13 \ in^2$

Bolted Splice Design Section 2-2 G2 Node 20.3
Strength - Top and Bottom Flange (continued)

For the bottom flange of the box, an effective flange thickness will be computed. The effective thickness of the bottom flange is computed as:

$$(t_f)_{eff} = \frac{A_e}{b_f} = \frac{43.13}{83.0} = 0.52 \quad \text{in.}$$

It is more advantageous to determine the effective thickness for the bottom flange and not the effective width in order to maintain the web slopes.

Section properties from Table C5 are computed using the effective bottom flange thickness are used to calculate the vertical bending stresses in the flange at the splice for strength whenever the bottom flange is subjected to tension. The gross area is used for the top flange in this case. For flanges and splice plates subjected to compression, net section fracture is not a concern and the effective area is taken equal to the gross area.

Calculate the factored vertical bending stress in the top and bottom flange mid-thicknesses at the strength limit state for both the positive and negative live load bending conditions. The longitudinal component of the top flange bracing area is again included in the effective section properties. The contribution of deck reinforcement is not included in the section properties at this section. The smaller section is to be used to design the splice, therefore, the longitudinal flange stiffener is not included. The provisions of Article 4.5.2.2 are followed to determine which composite section (cracked or uncracked) to use.

Using the effective section properties (from separate calculations), calculate the average factored bending stress in the top and bottom flange at the Strength limit state for both the positive and the negative live load bending conditions.

Negative live load bending case

$$F_{top\ flg} = -\left[\frac{1.25(2403)(41.29)}{179050} + \frac{[\,1.25(326) + 1.5(428)\,](43.02)}{179740} + \frac{1.75(-3087)(43.02)}{179740}\right]12 = 4.19 \quad \text{ksi} \quad \text{(T)}$$

$$F_{bot\ flg} = \left[\frac{1.25(2403)(37.52)}{179050} + \frac{[\,1.25(326) + 1.5(428)\,](35.79)}{179740} + \frac{1.75(-3087)(35.79)}{179740}\right]12 = -2.85 \quad \text{ksi} \quad \text{(C)}$$

Positive live load bending case

$$F_{top\ flg} = -\left[\frac{1.25(2403)(41.29)}{179050} + \frac{[\,1.25(326) + 1.5(428)\,](23.18)}{338310} + \frac{1.75(5264)(9.91)}{456064}\right]12 = -11.58 \quad \text{ksi} \quad \text{(C)}$$

$$F_{bot\ flg} = \left[\frac{1.25(2403)(37.52)}{179050} + \frac{[\,1.25(326) + 1.5(428)\,](55.63)}{338310} + \frac{1.75(5264)(68.90)}{456064}\right]12 = 26.32 \quad \text{ksi} \quad \text{(T)}$$

Bolted Splice Design Section 2-2 G2 Node 20.3
Strength - Top and Bottom Flange (continued)

An acceptable alternative to the preceding calculation is to calculate the average factored vertical bending stress in both flanges for both live load bending conditions using the appropriate gross section properties. Then, for the flange in tension, multiply the calculated average stress times the gross area, A_g, of the flange, and then divide the resulting force by the effective area, A_e, of the flange to determine an adjusted average tension-flange stress. Then, for the critical live load bending condition, use the adjusted average stress in the tension flange and the calculated average stress in the compression flange to determine which flange is the controllong flange, as defined below.

Separate calculations (similar to subsequent calculations) show that the positive live load bending case is critical. For this loading case, the bottom flange is the controlling flange since it has the largest ratio of the flexural stress to the corresponding critical flange stress. Article 6.13.6.1.4c defines the design stress, F_{cf}, for the controlling flange as follows:

$$F_{cf} = \frac{\left(\left|\frac{f_{cf}}{R_h}\right| + \alpha\phi_f F_{yf}\right)}{2} \geq 0.75\alpha\phi_f F_{yf} \qquad \text{Eq (6.13.6.1.4c-1)}$$

f_{cf} is the maximum flexural stress due to the factored loads at the mid-thickness of the controlling flange at the point of splice. The hybrid factor R_h is taken as 1.0 when F_{cf} does not exceed the specified minimum yield resistance of the web. α is taken as 1.0, except that a lower value equal to (F_n/F_{yf}) may be used for flanges where F_n is less than F_{yf}.

$$F_{cf} = \frac{\left[\left|\frac{26.32}{1.0}\right| + 1.0(1.0)(50)\right]}{2} = 38.16 \quad \text{ksi}$$

$0.75\alpha\phi_f F_{yf} = 0.75(1.0)(1.0)(50) = 37.5$ ksi; therefore, use 38.16 ksi

The minimum design force for the controlling (bottom) flange, P_{cf}, is taken equal to F_{cf} times the smaller effective flange area, A_e, on either side of the splice. The area of the smaller flange is used to ensure that the design force does not exceed the strength of the smaller flange. In this case, the effective flange areas are the same on both sides of the splice.

$P_{cf} = 38.16(43.13) = 1646$ kips (T)

The minimum design stress for the noncontrolling (top) flange for this case is specified in Article 6.13.6.1.4c as:

$$F_{ncf} = R_{cf}\left|\frac{f_{ncf}}{R_h}\right| \geq 0.75\alpha\phi_f F_y \qquad \text{Eq (6.13.6.1.4c-3)}$$

where α is again taken as 1.0. For a continuously braced top flange in tension, α should also be taken equal to 1.0.

Bolted Splice Design Section 2-2 G2 Node 20.3
Strength - Top and Bottom Flange (continued)

$$R_{cf} = \left|\frac{F_{cf}}{f_{cf}}\right| = \left|\frac{38.16}{26.32}\right| = 1.45$$

f_{ncf} is the factored vertical bending stress in the noncontrolling flange at the splice concurrent with f_{cf}.

$$R_{cf}\left|\frac{f_{ncf}}{R_h}\right| = 1.45\left|\frac{-11.58}{1.0}\right| = 16.79 \text{ ksi}$$

$$0.75\alpha\phi_f F_{yf} = 0.75(1.0)(1.0)(50) = 37.5 \text{ ksi (controls)}$$

The minimum design force for the noncontrolling flange, P_{ncf}, is computed as:

$$P_{ncf} = F_{ncf} A_e = (37.5)(16.0)(1.0) = 600 \text{ kips (C)}$$

where the effective flange area, A_e, is taken equal to the smaller gross flange area, A_g, on either side of the splice since the flange is subjected to compression. In this case, the gross flange areas are the same on both sides of the splice.

Top Flange

St. Venant torsional shears are not considered in the top flanges of tub girders. Lateral flange bending in the top flange is also not considered after the deck has hardened and the section is closed. Therefore:

$$\text{No. bolts required} = \frac{F_{ncf} A_e}{R_r} = \frac{600}{55.4} = 10.8 \text{ bolts, use 12 bolts}$$

$$\frac{600}{12} = 50 \text{ k/bolt} < 55.4 \text{ k/bolt } \textbf{OK}$$

Since a fill plate is not required for the top flange splice, no reduction in the bolt design shear resistance is required per the requirements of Article 6.13.6.1.5.

Bottom Flange

Compute the factored St. Venant torsional shear in the bottom flange at the strength limit state. Warping torsion is ignored since it is assumed in this example that the spacing of the internal bracing is sufficient to limit the warping stress to 10 percent of the vertical bending stress at the strength limit state (Article 6.7.5.3). Further, the specifications do not require warping to be considered in the design of bolted box flange splices at the strength limit state. From Table D-3, the torques are as follows:

Load	Torque		
Steel	1.25(-36)	=	-45 k-ft
Deck	1.25(-125)	=	-156 k-ft
Noncomposite torque			-201 k-ft

Bolted Splice Design Section 2-2 G2 Node 20.3
Strength - Top and Bottom Flange (continued)

SupImp DL	1.25(-58)	=	-73 k-ft
FWS	1.50(-76)	=	-114 k-ft
LL + IM	1.75(-517)	=	-905 k-ft
Composite torque			-1,092 k-ft

$$V = \frac{T}{2A_o} b_f$$

$$V_{noncomp} = \frac{|-201|}{2(55.0)}\left(\frac{81}{12}\right) = 12.3 \quad \text{kips}$$

$$V_{comp} = \frac{|-1092|}{2(60.8)}\left(\frac{81}{12}\right) = 60.6 \quad \text{kips}$$

$$V_{total} = 12.3 + 60.6 = 72.9 \quad \text{kips}$$

The total torsional shear is then factored up by R_{cf} = 1.45 (see earlier calculations) to be consistent with the computation of F_{cf} and P_{cf}.

$$V_{fact} = 72.9(1.45) = 105.7 \quad \text{kips}$$

Compute the factored moment in the bottom flange due to the torsional design shear. Assume the shear is applied at the centerline of the splice (i.e. at the juncture of the two flange plates).

$$M = 105.7(2.25 + 2.25) = 475.7 \quad \text{k-in}$$

Compute the longitudinal component of force in the critical bolt due to the factored moment.

$$F_{Long\ M} = \frac{475.7(36.25)}{19859} = 0.87 \quad \text{k/bolt}$$

Compute the force in each bolt due to the minimum design force, P_{cf}.

$$F_L = \frac{1646}{40} = 41.1 \quad \text{k/bolt}$$

Therefore,

$$F_{Long\ tot} = 0.87 + 41.1 = 42 \quad \text{k/bolt}$$

Compute the transverse component of force in the critical bolt.

$$F_{Trans} = \frac{475.7(2.25)}{19859} = 0.05 \quad \text{k/bolt}$$

Bolted Splice Design Section 2-2 G2 Node 20.3
Strength - Top and Bottom Flange (continued)

Compute the force in each bolt resulting from the factored torsional design shear.

$$F_v = \frac{105.7}{40} = 2.64 \quad \text{k/bolt}$$

$$F_{Trans\ tot} = 2.64 + 0.05 = 2.69 \quad \text{k/bolt}$$

Compute the resultant force on the critical bolt.

$$\Sigma_F = \sqrt{42^2 + 2.69^2} = 42.09 \quad \text{k/bolt}$$

Check $R_u \le R_r$

R_u = 42.09 k/bolt < R_r = 55.4 k/bolt **OK**

Note that a fill plate is also not required for the bottom flange splice. Therefore, no reduction in the bolt design shear resistance is necessary.

Bolted Splice Design Section 2-2 G2 Node 20.3
Constructibility - Web

A pattern of two rows of 7/8 in. bolts spaced vertically at 3.75 in. will be tried for the web splice. There are 40 bolts on each side of the web splice. The pattern is shown in Figure D-6. Although not illustrated here, the number of bolts in the web splice could be decreased by spacing a group of bolts closer to the mid-depth of the web (where the flexural stress is relatively low) at the maximum spacing specified for sealing (Article 6.13.2.6.2), and by spacing the remaining two groups of bolts near the top and bottom of the web at a closer spacing. Note that there is 4.625 in. between the inside of the flanges and the first bolt to provide sufficient assembly clearance. In this example, the web splice is designed under the conservative assumption that the maximum moment and shear at the splice will occur under the same loading condition.

Compute the polar moment of inertia of the web bolts about the centroid of the bolt group on one side of the connection.

$$I_p = \frac{nm}{12}\left[s^2\left(n^2-1\right)+g^2\left(m^2-1\right)\right] \qquad \text{Eq (C6.13.6.1.4b-3)}$$

where:

m = number of vertical rows of bolts
n = number of bolts in one vertical row
s = vertical pitch, in.
g = horizontal pitch, in.

For n = 20; m = 2; s = 3.75 in.; g = 3 in.,

$$I_p = \frac{20(2)}{12}\left[3.75^2\left(20^2-1\right)+3^2\left(2^2-1\right)\right] = 18793 \quad \text{in}^4$$

As stated previously, Article 6.13.6.1.4a requires that high-strength bolted connections be proportioned to prevent slip for constructibility. Article 6.13.6.1.4b requires that bolted web splices be designed to prevent slip under the most critical combination of the design actions at service load, Load Combination Service II.

Constructibility

From Table D-3, compute the factored vertical shear at the splice (bending plus torsional shear in the critical web) due to Steel plus Cast #1.

$$V = (-17-58)(1.25) = -93.75 \quad \text{kips}$$

Compute the moment, M_v, due to the eccentricity of the factored shear about the centroid of the connection (refer to the web bolt pattern in Figure D-6).

$$M_v = Ve = 93.75\left(\frac{3}{2}+\frac{4.5}{2}\right)\left(\frac{1}{12}\right) = 29.3 \quad \text{k-ft}$$

Bolted Splice Design Section 2-2 G2 Node 20.3
Constructibility - Web (continued)

Determine the portion of the design moment resisted by the web, M_{uw}, and the design horizontal force resultant in the web, H_{uw}, using equations similar to those provided in Article C6.13.6.1.4b for Load Combination Service II. M_{uw} and H_{uw} are assumed to be applied at the middepth of the web for designing the web splice plates and their connections. Using the results from earlier calculations (Table D-2), the average factored vertical bending stress in the top flange for Steel plus Cast #1 is computed as:

$$F_{cf} = \left(\frac{-11.13 - 10.87}{2}\right) = -11 \quad \text{ksi}$$

The average factored vertical bending stress in the bottom flange is (see Table D-2)

$$f_{ncf} = \left(\frac{9.58 + 9.42}{2}\right) = 9.5 \quad \text{ksi}$$

Using these stresses (set R_h and R_{cf} equal to 1.0)

$$M_{uw} = \frac{t_w D^2}{12}\left|R_h F_{cf} - R_{cf} f_{ncf}\right| = \frac{0.5625\,(78)^2}{12}\left|1.0(-11.0) - 1.0(9.5)\right|\left(\frac{1}{12}\right) = 487 \quad \text{k-ft}$$

Eq (C6.13.6.1.4b-1)

$$H_{uw} = \frac{t_w D}{2}\left(R_h F_{cf} + R_{cf} f_{ncf}\right) = \frac{0.5625(78)}{2}[\,1.0(-11.0) + 1.0(9.5)\,] = -32.9 \quad \text{kips}$$

Eq (C6.13.6.1.4b-2)

The total moment on the web splice is computed as:

$$M_{tot} = M_v + M_{uw} = 29.3 + 487 = 516 \quad \text{k-ft}$$

Compute the in-plane bolt force due to the factored vertical shear.

$$F_s = \frac{V}{N_b} = \frac{93.75}{40} = 2.34 \quad \text{k/bolt}; \qquad \frac{2.34}{\cos\left[14.0\left(\frac{2\pi}{360}\right)\right]} = 2.4 \quad \text{k/bolt}$$

Compute the in-plane bolt force due to the horizontal force resultant.

$$F_H = \frac{H_{uw}}{N_b} = \frac{\left|-32.9\right|}{40} = 0.82 \quad \text{k/bolt}$$

Bolted Splice Design Section 2-2 G2 Node 20.3
Constructibility - Web (continued)

Compute the in-plane horizontal and vertical components of the force on the extreme bolt due to the total moment on the splice.

$$F_{Mv} = \frac{M_{tot}\,x}{I_p} = \frac{487(12)\left(\frac{3}{2}\right)}{18793} = 0.47 \quad \text{k/bolt;} \quad \frac{0.47}{\cos\left[14.0\left(\frac{2\pi}{360}\right)\right]} = 0.48 \quad \text{k/bolt}$$

$$F_{Mh} = \frac{M_{tot}\,y}{I_p} = \frac{487(12)(35.625)}{18793} = 11.08 \quad \text{k/bolt}$$

Compute the resultant in-plane bolt force.

$$F_r = \sqrt{\left(F_s + F_{Mv}\right)^2 + \left(F_H + F_{Mh}\right)^2} = \sqrt{(2.4 + 0.48)^2 + (0.82 + 11.08)^2} = 12.24 \quad \text{k/bolt}$$

$F_r = 12.24$ k/bolt < $R_r = 39$ k/bolt **OK**

Bolted Splice Design Section 2-2 G2 Node 20.3
Strength - Web

Determine the vertical design shear, V_{uw}, for the web splice for strength according to the provisions of Article 6.13.6.1.4b.

From Table D-3, the factored vertical shear at the splice (bending plus torsional shear in the critical web at the strength limit state) is computed as:

$$V_u = 1.25(-17 - 69 - 12) + 1.5(-16) + 1.75(-85) = -295 \text{ kips}$$

Compute the nominal shear resistance of the 0.5625 in. thick web at the splice according to the provisions of Articles 6.10.9.2 and 6.10.9.3 for unstiffened and stiffened webs, respectively.

Separate calculations indicate that transverse stiffeners are required for this web thickness, therefore, use Article 6.10.9.3.

Try a stiffener spacing equal to the cross-frame spacing, d_o = 196 in.

Article 6.10.9.3.2 is used for the nominal shear resistance of an interior web panel. The section along the entire panel must be proportioned according to Eq (6.10.9.3.2-1).

$$\frac{2Dt_w}{\left(b_{fc}t_{fc} + b_{ft}t_{ft}\right)} \leq 2.5 \qquad \text{Eq (6.10.9.3.2-1)}$$

$$\frac{2(80.4)(0.5625)}{[\,81(0.625) + 16(1)\,]} = 1.4$$ < 2.5, the provisions of Article 6.10.9.3.2 may be used.

The nominal shear resistance, V_n, is taken as:

$$V_n = V_p\left[C + \frac{0.87(1 - C)}{\sqrt{1 + \left(\frac{d_o}{D}\right)^2}}\right] \qquad \text{Eq (6.10.9.3.2-2)}$$

where:

$$V_p = 0.58F_{yw}Dt_w \qquad \text{Eq (6.10.9.3.2-3)}$$

Determine which equation is to be used to compute the ratio of the shear-buckling resistance to the shear yield resistance, C.

$$\frac{D}{t_w} = \frac{80.4}{0.5625} = 143$$

$$k = 5 + \frac{5}{\left(\frac{d_o}{D}\right)^2} = 5 + \frac{5}{\left(\frac{196}{80.4}\right)^2} = 5.84 \qquad \text{Eq (6.10.9.3.2-7)}$$

Bolted Splice Design Section 2-2 G2 Node 20.3
Web - Strength (continued)

$$1.12\sqrt{\frac{Ek}{F_{yw}}} = 1.12\sqrt{\frac{29000(5.84)}{50}} = 65 \quad < \ 143$$

$$1.40\sqrt{\frac{Ek}{F_{yw}}} = 1.40\sqrt{\frac{29000(5.84)}{50}} = 81 \quad < \ 143$$

Therefore, use Eq (6.10.9.3.2-6).

$$C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2}\left(\frac{Ek}{F_{yw}}\right) = \frac{1.57}{\left(\frac{80.4}{0.5625}\right)^2}\left[\frac{29000(5.84)}{50}\right] = 0.26 \qquad \text{Eq (6.10.9.3.2-6)}$$

$$V_p = 0.58F_{yw}Dt_w = 0.58(50)(80.4)(0.5625) = 1312 \ \text{kips}$$

$$V_n = 1312\left[0.26 + \frac{0.87(1-0.26)}{\sqrt{1+\left(\frac{196}{80.4}\right)^2}}\right] = 662 \ \text{kips}$$

$$\phi_v V_n = 1.0(662) = 662 \text{ kips} \quad > \quad V_{ui} = \frac{295}{\cos\left[14.0\left(\frac{2\pi}{360}\right)\right]} = 304 \ \text{kips} \quad \textbf{OK} \qquad \text{Eq (6.10.9.1-1)}$$

Therefore, according to Article 6.13.6.1.4b, since $V_u < 0.5\phi_v V_n$:

$$V_{uw} = 1.5V_u = 1.5(304) = 456 \ \text{kips} \qquad \text{Eq (6.13.6.1.4b-1)}$$

The moment, M_{uv}, due to the eccentricity, e, of V_{uw} from the centerline of the splice to the centroid of the web splice bolt group is computed as follows (refer to web bolt pattern in Figure D-6):

$$M_{uv} = V_{uw}e$$

$$M_{uv} = 456\left(\frac{3}{2} + \frac{4.5}{2}\right)\left(\frac{1}{12}\right) = 143 \ \text{k-ft}$$

Bolted Splice Design Section 2-2 G2 Node 20.3
Web - Strength (continued)

Determine the portion of the design moment resisted by the web, M_{uw}, and the design horizontal force resultant in the web, H_{uw}, according to the provisions of Article C6.13.6.1.4b. M_{uw} and H_{uw} are applied at the middepth of the web. Separate calculations indicate that the positive live load bending condition controls.

As computed earlier (pages D-62 and D-63) for the positive live load bending case:

f_{cf} = 26.32 ksi

F_{cf} = 38.16 ksi

f_{ncf} = -11.58 ksi

R_{cf} = 1.45 ksi

From the equations in Article C6.13.6.1.4b:

$$M_{uw} = \frac{t_w D^2}{12}\left|R_h F_{cf} - R_{cf} f_{ncf}\right| = \frac{0.5625\,(78)^2}{12}\left|1.0(38.16) - 1.45(-11.58)\right|\left(\frac{1}{12}\right) = 1306 \text{ k-ft}$$

Eq (C6.13.6.1.4b-1)

$$H_{uw} = \frac{t_w D}{2}\left(R_h F_{cf} + R_{cf} f_{ncf}\right) = \frac{0.5625(78)}{2}[\,1.0(38.16) + 1.45(-11.58)\,] = 469 \text{ kips}$$

Eq (C6.13.6.1.4b-2)

The total moment on the web splice is computed as:

$$M_{tot} = M_{uv} + M_{uw} = 143 + 1306 = 1449 \text{ k-ft}$$

Compute the in-plane bolt force due to the vertical design shear.

$$F_s = \frac{V_{uw}}{N_b} = \frac{456}{40} = 11.4 \text{ k/bolt;} \quad \frac{11.4}{\cos\left[14.0\left(\frac{2\pi}{360}\right)\right]} = 11.75 \text{ k/bolt}$$

Compute the in-plane bolt force due to the horizontal design force resultant.

$$F_H = \frac{H_{uw}}{N_b} = \frac{443}{40} = 11.1 \text{ k/bolt}$$

Bolted Splice Design Section 2-2 G2 Node 20.3
Web - Strength (continued)

Compute the in-plane horizontal and vertical components of the force on the extreme bolt due to the total moment on the splice.

$$F_{Mv} = \frac{M_{tot}x}{I_p} = \frac{1449(12)\left(\frac{3}{2}\right)}{18793} = 1.39 \text{ k/bolt}; \quad \frac{1.39}{\cos\left[14.0\left(\frac{2\pi}{360}\right)\right]} = 1.43 \text{ k/bolt}$$

$$F_{Mh} = \frac{M_{tot}y}{I_p} = \frac{1449(12)(35.63)}{18793} = 32.97 \text{ k/bolt}$$

Compute the resultant in-plane bolt force.

$$F_{resultant} = \sqrt{\left(F_s + F_{Mv}\right)^2 + \left(F_H + F_{Mh}\right)^2} = \sqrt{(11.75 + 1.43)^2 + (11.1 + 32.97)^2} = 46 \text{ k/bolt}$$

F_{res} = 46.0 k/bolt < R_r = 55.4 k/bolt **OK**

Bolted Splice Design Section 2-2 G2 Node 20.3
Splice Plates

Web Splice Plate Design

Use nominal 0.375-in. thick splice plates. Fill plates are not required in this case.

The maximum permissible spacing of the bolts for sealing = $4 + 4t \leq 7.0$ = 4 + 4(0.375) = 5.5 in. < 7.0

Check bearing of the bolts on the connected material assuming the bolts have slipped and gone into bearing. Since the bearing resistance of the web controls, the bearing resistance of the outermost hole in the thinner web at the splice, calculated using the clear edge distance, will conservatively be checked against the maximum resultant force acting on the extreme bolt in the connection. This check is conservative since the resultant force acts in the direction of an inclined distance that is larger than the clear edge distance. Should the bearing resistance be exceeded, it is recommended that the edge distance be increased slightly in lieu of increasing the number of bolts or thickening the web. Another option would be to calculate the bearing resistance based on the inclined distance, or else resolve the resultant force in the direction parallel to the edge distance. In cases where the bearing resistance of the web splice plates controls, the smaller of the clear edge or end distance on the splice plates can be used to compute the bearing resistance of the outermost hole.

The maximum resultant in-plane force on the extreme bolt was computed earlier (page D-73) for strength to be:

$$F_{resultant} = 46.0 \text{ kips} < \phi_{bb}R_n = 0.8(65.81) = 52.65 \text{ kips} \quad \textbf{OK}$$

According to Article 6.13.6.1.4b, check for flexural yielding on the gross section of the web splice plates at the strength limit state. The flexural stress is limited to $\phi_f F_y$.

$$A_g = 2(0.375)(75.25) = 56.44 \text{ in}^2$$

$$S_{PL} = \frac{I_x \cos^2\theta}{c} = \frac{2\left[\frac{(0.375)75.25^3}{12}\right]\cos\left[14\left(\frac{2\pi}{360}\right)\right]^2}{\left(\frac{75.25}{2}\right)\cos\left[14\left(\frac{2\pi}{360}\right)\right]} = 686.8 \text{ in}^3$$

Bolted Splice Design Section 2-2 G2 Node 20.3
Splice Plates (continued)

$$f = \frac{M_{uv} + M_{uw}}{S_{PL}} + \frac{H_{uw}}{A_g}$$

$$= \frac{(143 + 1306)(12)}{686.8} + \frac{469}{56.44} = 33.63 \text{ ksi} < \phi_f F_y = 1.0(50) = 50 \text{ ksi} \quad \textbf{OK}$$

Since the thickness of the two splice plates exceeds t_w, say the shear resistance in the splice plates is adequate.

Flange Splice Plate Design

Top Flange

The width of the outside splice plate should be at least as wide as the width of the narrowest flange at the splice. In this case, however, the width of the top flange is the same on either side of the splice. Therefore;

Try: 16 x 0.5 in. outer plate — $A_g = 8.0 \text{ in}^2$

Try: 2 - 6 x 0.625 in. inner plates — $A_g = 7.50 \text{ in}^2$

As specified in Article 6.13.6.1.4c, the effective area, A_e, of each splice plate is to be sufficient to prevent yielding of each splice plate under its calculated portion of the minimum flange design force. For splice plates subjected to compression, the effective area is equal to the gross area.

The effective areas of the inner and outer splice plates are computed as:

$$A_e = \left(\frac{\phi_u F_u}{\phi_y F_{yt}}\right) A_n \le A_g \qquad \text{Eq (6.13.6.1.4c-2)}$$

Outer plate: $A_n = [\,16.0 - 4(0.875 + 0.125)\,](0.5) = 6 \text{ in}^2$

Inner plate: $A_n = 2[\,6.0 - 2(0.875 + 0.125)\,]\,0.625 = 5 \text{ in}^2$

Outer plate: $\left[\frac{0.8(65)}{0.95(50)}\right](6.0) = 6.57 < 8.0 \text{ in}^2$ **OK**

Inner plate: $\left[\frac{0.8(65)}{0.95(50)}\right](5.0) = 5.47 < 7.5 \text{ in}^2$ **OK**

Bolted Splice Design Section 2-2 G2 Node 20.3
Splice Plates (continued)

As specified in Article C6.13.6.1.4c, if the combined area of the inner splice plates is within 10 percent of the area of the outside splice plate, then both the inner and outer plates may be designed for one-half the flange design force (which is the case here). Double shear may then be assumed in designing the connections. If the areas differ by more than 10 percent, the design force in each splice plate and its connection at the strength limit state should be determined by multiplying the flange design force by the ratio of the area of the splice plate under consideration to the total area of the inner and outer splice plates. In this case, the shear resistance of the connection would be checked for the maximum calculated splice plate force actings on a single shear plane.

For the negative live load bending case, the controlling flange is the top flange. The flange is subjected to tension under this live load bending condition (see page D-62). Compute the minimum resistance, $F_{cf}A_e$, in the top flange for this load case. The factored tensile resistance, P_r, is taken as the lesser of the values given by Eqs (6.8.2.1-1 and 6.8.2.1-2). The factor α in Eq (6.13.6.1.4c-1) is generally taken equal to 1.0.

$$F_{cf} = \frac{\left|\frac{4.19}{1.0} + 1.0(1.0)(50)\right|}{2} = 27.09 \text{ ksi} \qquad \text{Eq (6.13.6.1.4c-1)}$$

$$0.75\alpha\phi_f F_{yf} = 0.75(1.0)(1.0)(50) = 37.5 \text{ ksi (controls)}$$

$$F_{cf}A_e = 37.5(13.14) = 493 \text{ kips}$$

As discussed previously, St. Venant torsional shear and lateral flange bending are not considered in the top flange at the strength limit state. Warping torsion is also ignored. According to Article 6.13.6.1.4c, the capacity of the splice plates to resist tension is computed using the provisions of Article 6.8.2. The factored tensile resistance, P_r, is taken as the lesser of:

$$P_r = \phi_y P_{ny} = \phi_y F_y A_g \qquad \text{Eq (6.8.2.1-1)}$$

$$P_r = 0.95(50)(8.0) = 380 \text{ kips} \quad \text{Outer plate}$$

$$P_r = 0.95(50)(7.50) = 356 \text{ kips} \quad \text{Inner plates}$$

or

$$P_r = \phi_u P_{nu} = \phi_u F_u A_n U \qquad \text{Eq (6.8.2.1-2)}$$

$$P_r = 0.80(65)(6.0)(1.0) = 312 \text{ kips} \quad \text{Outer plate}$$

$$P_r = 0.80(65)(5.0)(1.0) = 260 \text{ kips} \quad \text{Inner plates (controls)} > \frac{450}{2} = 225 \text{ kips } \textbf{OK}$$

Bolted Splice Design Section 2-2 G2 Node 20.3
Splice Plates (continued)

Under the positive live load bending case, the top flange is the noncontrolling flange and is subjected to compression. The minimum design force, $F_{ncf}A_e$, for the top flange for this load case was computed earlier (see page D-64) to be 600 kips. The factored compressive resistance, R_r, is taken as:

$R_r = \phi_c F_y A_s$ (Outer and Inner plates, respectively) Eq (6.13.6.1.4c-4)

$R_r = 0.9(50)(8.0) = 360$ kips

$= 0.9(50)(7.50) = 338$ kips $> \dfrac{600}{2} = 300$ kips **OK**

Bearing Resistance at Bolt Holes

Check bearing of the bolts on the connected material under the minimum design force, $F_{ncf}A_e$ = 600 kips, for the top flange. The design bearing resistance, R_n, is computed using the provisions of Article 6.13.2.9.

According to Article 6.13.2.9, the bearing resistance for the end and interior rows of bolts is computed using Eq (6.13.2.9-1) or Eq (6.13.2.9-2). Calculate the clear distance between holes and the clear end distance and compare to 2.0d to determine the equation to be used to solve for the bearing resistance.

The center-to-center distance between the bolts in the direction of the force is 3.0 in. Therefore:

Clear distance between holes = $3.0 - 1.0 = 2.0$ in.

For the four bolts adjacent to the end of the splice plate, the end distance is assumed to be 1.5 in. Therefore, the clear distance between the edge of the holes and the end of the splice plate is:

Clear end distance = $1.5 - \dfrac{1.0}{2} = 1.0$ in.

The value 2.0d is equal to 1.75 in. Since the clear end distance is less than 2.0d, use Eq (6.13.2.9-2).

$R_n = 1.2L_c t F_u = 1.2(1.0)(1.0)(65) = 78$ k/bolt Eq (6.13.2.9-2)

$\phi_{bb} = 0.8$

$F_{ncf}A_e = 600\text{ k} < \phi_{bb}R_n = 0.8(12)(78) = 748.8\text{ k}$ **OK**

Bottom Flange

Try: 75.5 x 0.375 in. outer plate
$A_g = 28.3\text{ in}^2$

Try: 2 - 36.75 x 0.375 in. inner plates
$A_g = 27.6\text{ in}^2$

Note: Since the inner splice plate must be partially split to accommodate the longitudinal flange stiffener (Figure D-5), it will conservatively be treated as two separate plates in the subsequent calculations although this is physically not the case.

Bolted Splice Design Section 2-2 G2 Node 20.3
Splice Plates (continued)

The minimum flange design force, $F_{cf}A_e$, was computed earlier to be 1,646 kips (tension) (page D-63). The factored-up moment for strength due to the St. Venant torsional shear was computed earlier (page D-65) to be 475.7 k-in. Warping torsion is ignored since it is assumed in this example that the spacing of the internal bracing is sufficient to limit the warping stress to 10 percent of the vertical bending stress at the strength limit state (Article 6.7.5.3). Further, the specifications do not require warping to be considered in the design of bolted box flange splices at the strength limit state.

The effective areas of the inner and outer splice plates are computed as:

$$A_e = \left(\frac{\phi_u F_u}{\phi_y F_{yt}}\right) A_n \le A_g \qquad \text{Eq (6.13.6.1.4c-2)}$$

Outer: $A_n = [75.5 - 20(0.875 + 0.125)](0.375) = 20.81 \ in^2$

$$\left[\frac{0.8(65)}{0.95(50)}\right](20.81) = 22.78 \ in^2 < 28.3 \ in^2 \quad \textbf{OK}$$

Inner: $A_n = 2[36.75 - 10(0.875 + 0.125)](0.375) = 20.06 \ \imath^2$

$$\left[\frac{0.8(65)}{0.95(50)}\right](20.06) = 21.96 \ in^2 < 27.6 \ in^2 \quad \textbf{OK}$$

Since the flange is subjected to a net tension, the holes will be considered in computing a net section modulus for the splice plates. The holes remove the following percentage of cross-sectional area from each splice plate:

Outer: $\dfrac{20(0.875 + 0.125)(0.375)}{75.5(0.375)} 100 = 26.5 \ \%$

Inner: $\dfrac{10(0.875 + 0.125)(0.375)}{36.75(0.375)} 100 = 27.2 \ \%$

According to Article 6.8.1, the application of the 85% maximum effeciency factor for splice plates should be included when using the net section. Therefore, the fraction of hole area that must be deducted in determining the net section modulus is:

Outer: $\dfrac{26.5 - 15.0}{26.5} = 0.43$

Bolted Splice Design Section 2-2 G2 Node 20.3
Splice Plates (continued)

Calculate ΣAd^2.

$$A = 2(0.43)(0.875 + 0.125)(0.375) = 0.323 \quad \text{in}^2$$

$$\Sigma Ad^2 = 0.323\left(2.5^2 + 6.25^2 + 10^2 + 13.75^2 + 17.5^2 + 21.25^2 + 25^2 + 28.75^2 + 32.5^2 + 36.25^2\right) = 1587 \quad \text{ı}^4$$

$$\text{Inner:} \quad \frac{27.2 - 15.0}{27.2} = 0.45$$

$$\Sigma Ad^2 = 2(0.45)(0.875 + 0.125)(0.375)\left(1.875^2 + 5.625^2 + 9.375^2 + 13.125^2 + 16.875^2\right) = 195.8 \quad \text{in}^4$$

The net section modulus of the inner and outer splice plates together is therefore equal to:

$$S_{net} = \frac{\left(\frac{1}{12}\right)(0.375)\,(75.5)^3 - 1587}{\left(\frac{75.5}{2}\right)} + 2\left[\frac{\left(\frac{1}{12}\right)(0.375)\,(36.75)^3 - 195.8 + 20.06\,(18.875)^2}{\left(\frac{75.5}{2}\right)}\right] = 765 \quad \text{n}^3$$

The combined stress in the bottom flange splice plates is equal to:

$$f = \frac{1646}{(20.81 + 20.06)} + \frac{475.7}{765} = 40.9 \quad \text{ksi}$$

$f = 40.9$ ksi $<$ $F_n = 50$ ksi **OK**

If the combined area of the equivalent inner splice plates had not been within 10 percent of the area of the outside splice plate, the minimum design force and factored-up moment would be proportioned to the inner and outer plates accordingly.

Separate calculations similar to those illustrated previously (page D-77) show that bearing of the bolts on the bottom flange is not critical.

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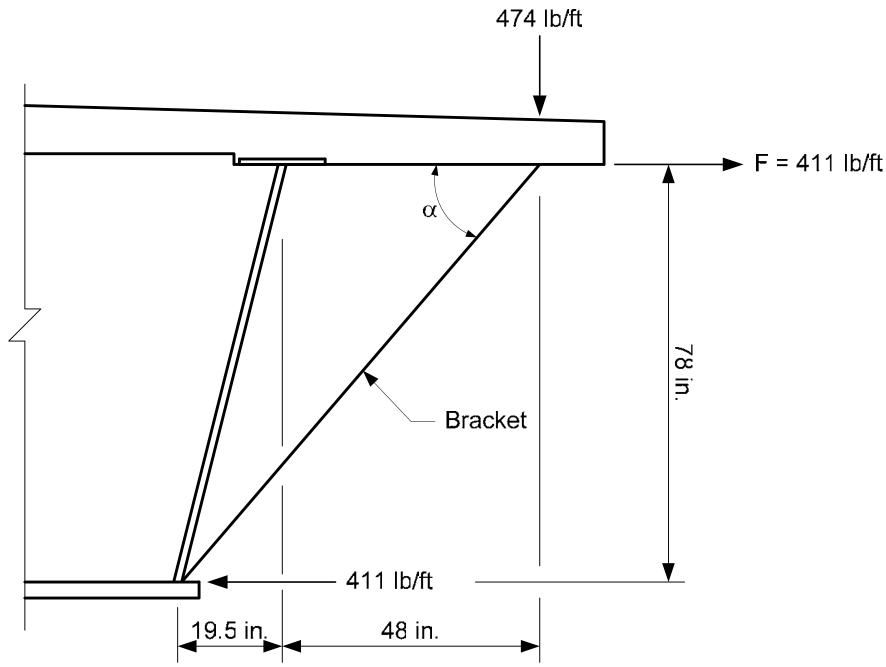


Figure D-1. Overhang Bracket Loading

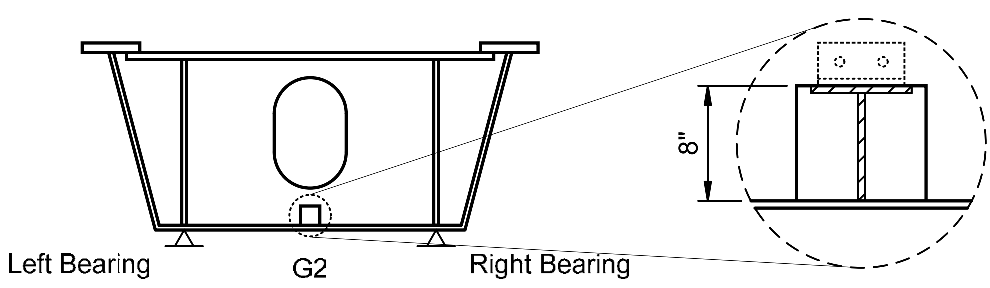


Figure D-2. Internal Diaphragm and Bearing Stiffeners at Pier of Girder 2 Looking Upstation

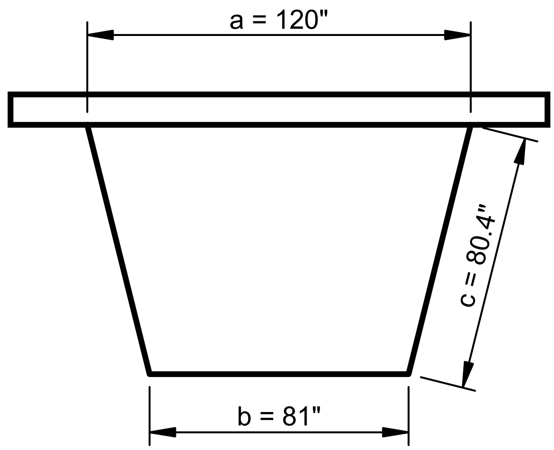


Figure D-3. Composite Box Cross Section, Girder 2

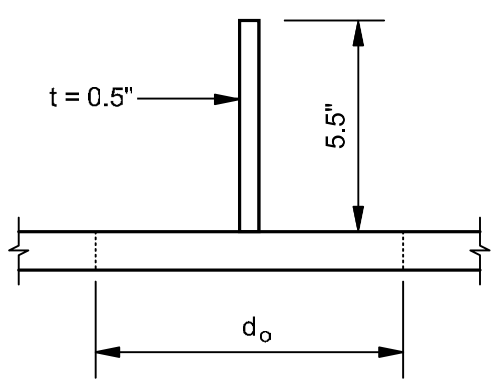
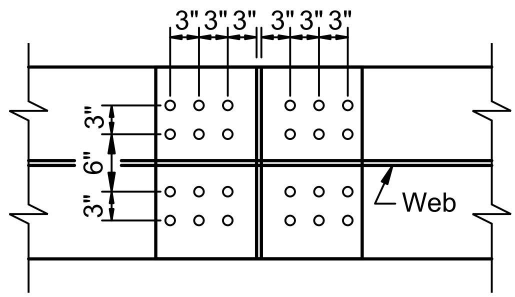


Figure D-4. Effective Width of Web Plate, d_o, Acting with Transverse Stiffener



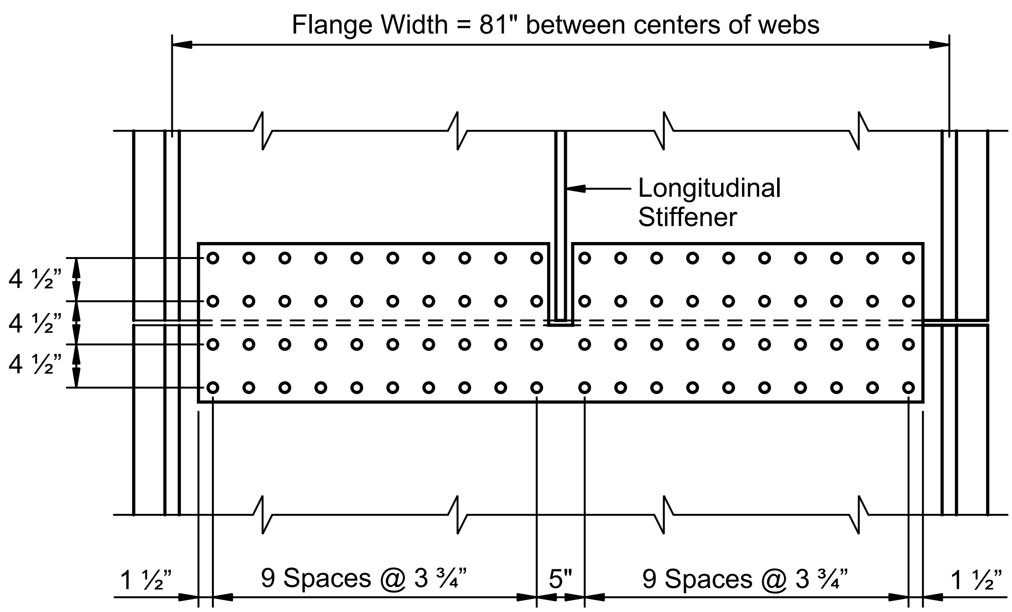
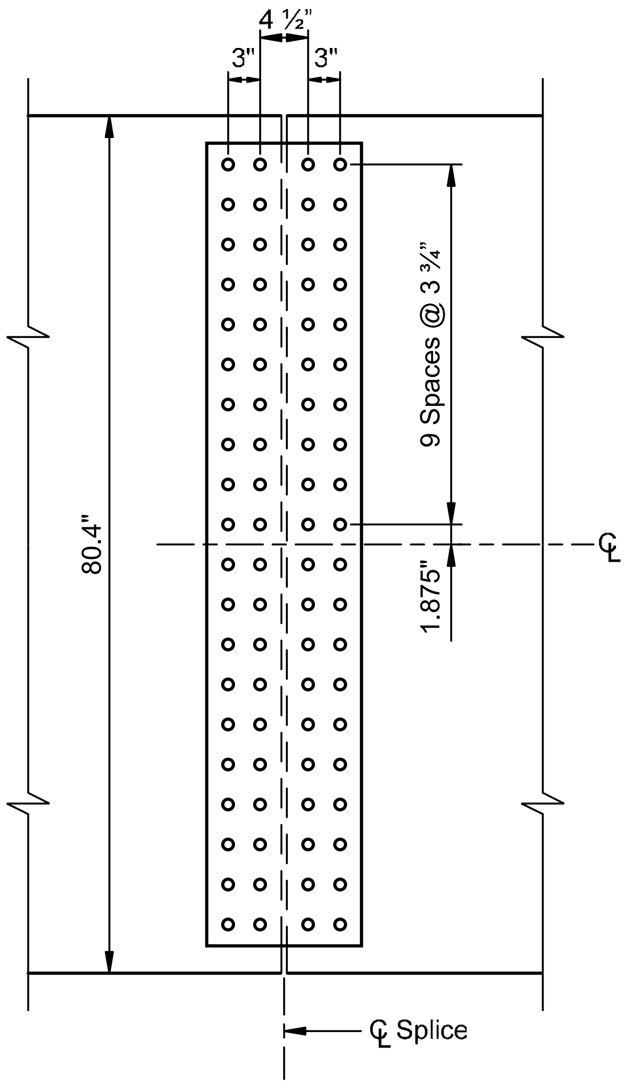


Figure D-5. Bolt Patterns for Top and Bottom Flange



Notes: (1) 1/2" gap assumed between the edges of the field pieces.
(2) The indicated distances are along the web slope.

Figure D-6. Bolt Pattern for Web

Table D-1. Strength Limit State at 100 feet from Left Abutment
Factored Loads Shown

Location	Steel	Deck	SupImp	FWS	LRFD (LL + IM)		Strength I Loading
	ksi	ksi	ksi	ksi		ksi	ksi
Top Flange	-1.6	-6.73	-0.33	-0.53	Positive	-2.48	-11.67
			-1.13	-1.78	Negative	14.98	3.74
Top Web	-1.56	-6.57	-0.32	-0.51	Positive	-2.25	-11.21
			-1.10	-1.74	Negative	14.63	3.66
Bottom Flange	1.38	5.79	0.79	1.20	Positive	15.87	25.03
			0.97	1.53	Negative	-12.89	-3.22
Bottom Web	1.35	5.69	0.75	1.19	Positive	15.72	24.70
			0.96	1.51	Negative	-12.67	-3.16

Table D-2. Constructability Limit State at 100 feet from Left Abutment
Service Loads Shown

Location	Steel	Cast #1	1.25 x Sum
	ksi	ksi	ksi
Top Flange	-1.28	-7.62	-11.13
Top Web	-1.25	-7.45	-10.87
Bottom Flange	1.10	6.56	9.58
Bottom Web	1.08	6.45	9.42

Table D-3. Unfactored Actions

Load	Moment (k-ft)	Torque (k-ft)	Top Flange Lateral Moment (k-ft)		Shear (kips)	
Steel	462	-36	-1		-17	
Deck	1,941	-125	-7		-69	
Cast #1	2,749	-188	-15		-61	
SupImp	326	-58	-1		-12	
FWS	428	-76	-2		-16	
Strength HL-93 with DLA	Moment (k-ft)		Torque (k-ft)		Shear (kips)	
	Pos	Neg	Pos	Neg	Pos	Neg
	5,221	-3,080	346	-517	36	-85

Note: Reported shears are vertical shears and are for bending plus torsion in the critical web.

Table D-4. Tub Cross Section

Component	Size (in.)	Area (in^2)	Yield (F_y)	Tensile (F_u)
Top Flanges	2 - 16 x 1	32.00	50	65
Web	2 - 78 x 0.5625	90.56	50	65
Bottom Flange	83 x 0.625	51.88	50	65

Note: Other section properties for the gross section may be found in Table C5. The cross section is the same on both sides of the splice except for the presence of a bottom flange longitudinal stiffener on one side.

APPENDIX E

Tabulation of Various Stress Checks

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INTRODUCTION

The following tables show various comparative stress checks between the 2003 AASHTO Guide Specifications and the 2004 AASHTO-LRFD including the 2005 Interim Specifications.

Table E-1. Constructability – Top Flange

Section/ Node	Guide Specifications (LFD)			2004 LRFD Specifications								
				Eq (6.10.3.2.1-1)			Eq (6.10.3.2.1-2)			Eq (6.10.3.2.1-3)		
	F_{cr}	f_b	ratio *	$\phi_f R_h F_{yc}$	$f_{bu} + f_l$	ratio	$\phi_f F_{nc}$	f_{bu} + 1/3f_l	ratio	$\phi_f F_{crw}$	f_{bu}	ratio
1-1 9 (G1)	-32.95	-29.32	0.89	-50.0	-30.69	0.61	-44.2	-27.68	0.63	-39.99	-26.18	0.65
1-1 10 (G2)	-36.56	-31.86	0.87	-50.0	-44.54	0.89	-43.7	-33.81	0.77	-39.99	-28.45	0.71

*Applied Stress divided by resistance.

Table E-2. Constructability – Web, Box Girder 2

Section/ Node	Guide Specifications (LFD)			2004 LRFD Specifications		
	F_{cr}	f_b	ratio	F_{crw}	f_{cw}	ratio
1-1 10	-39.89	-31.12	0.78	-39.99	-27.78	0.69

*Applied Stress divided by resistance.

Table E-3. Strength – Bottom Flange, Box Girder 2

Section/ Node	Guide Specifications (LFD)			2004 LRFD Specifications		
	F_{cr}	f_b	ratio	$\phi_f F_{nc}$	f_{bu}	ratio
5-5 36	-47.26	-46.47	0.98	-41.57	-41.6	1.00
At Splice (Unstiffened Flange)						
2-2 20.3	-6.04	-6.01	0.99	-6.01	-3.22	0.54

*Applied Stress divided by resistance.

Table E-4. Maximum Principal Stresses – Bottom Flange, Box Girder 2

Section/ Node	Guide Specifications (LFD)			2004 LRFD Specifications		
	F_{cr}	f_b	ratio	F_{nc}	f_{bu}	ratio
5-5 36	-49.5	-46.84	0.95	-41.57	-41.6	1.00

*Applied Stress divided by resistance.

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