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

AASHTO-LRFD DESIGN EXAMPLE
HORIZONTALLY CURVED STEEL I-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

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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 example represents an updated version of the Horizontally Curved Steel I-Girder Bridge design example developed for the NCHRP 12-38 project using the same provisions that were published as the 1993 AASHTO Guide Specifications for Horizontally Curved Steel I-Girder Bridges. It was updated to illustrate the applicability of the revisions planned to appear in the 2005 Interims to the AASHTO-LRFD Specifications to incorporate curved bridges. The NCHRP 12-38 example included three alternative designs: unstiffened web, transversely stiffened web, and a transversely and longitudinally stiffened web. For webs to be considered stiffened, the Specification on which the previous example was based did not allow intermediate stiffener spacing greater than the web depth. The Specification on which the example herein is based allows the webs to be considered stiffened when the transverse stiffeners are spaced at less than three times the web depth. The cross-frame spacing used in this example is less than three times the web depth, therefore, since there are transverse stiffeners at the cross-frame locations, the web is always considered stiffened. Therefore, there are only two alternative designs presented herein, transversely stiffened and transversely and longitudinally stiffened

The following terms are used to identify particular specifications:

- ANSI/AASHTO/AWS refers to the 2002 edition of the *Bridge Welding Code D1.5:2002,* American Welding Society and 2003 Interim Specifications,
- LRFD refers to the 2004 AASHTO-*LRFD Bridge Design Specifications*, Third Edition, and the revisions to incorporate curved bridge design provisions which will appear in the 2005 Interim. Article and equation numbers referenced in this example refer to those of the Specifications.

OBJECTIVES

1. Using the Third Edition of the AASHTO-LRFD Design Specifications including the 2005 Interim (hereafter referred to as the Specifications), design a three-span horizontally curved steel I-girder bridge with four girders in the cross section.

2. Compare the stresses in the design using the AASHTO Guide Specifications 2003 (NCHRP 12-38) to the stresses in the design using the AASHTO-LRFD Specifications, Third Edition, with the 2005 Interim Specifications.

DESIGN PARAMETERS

The bridge has spans of 160-210-160 feet measured along the centerline of the bridge. Span lengths are arranged to give similar 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 four I-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 total deck thickness is 9.5 in. (with one-half inch integral wearing surface assumed). The deck haunch thickness is conservatively taken as 4.0 in. and is measured from the top of the web to the bottom of the deck. The width of the haunch is assumed to be 20 inches. A future wearing surface of 30 psf is specified. Deck parapets are each assumed to weigh 495 plf.

Bridge underclearance is limited such that the total bridge depth may not exceed 120 in. at the low point on the cross section. The roadway is superelevated 5 percent.

Live load is LRFD HL-93 for the strength limit state. Live load for fatigue is taken as defined in Article 3.6.1.4. The bridge is subjected to a temperature range from -40 degrees to 120 degrees Fahrenheit. The bridge is designed for a 75-year fatigue life.

Wind loading is 50 pounds per square foot. Earthquake loading is not explicitly considered.

Steel erection is not explicitly examined in this example. Sequential placement of the concrete deck is considered. Permanent steel deck forms are assumed to be used between girders; the forms are assumed to weigh 15 psf.

STEEL FRAMING

Proper layout of the steel framing is an important part of the design process. Five different framing plans considering different girder depths, cross-frame spacings and with and without lateral flange bracing were examined. Only the final layout is shown is this example.

Girder Spacing

The four I-girders are spaced at 11 feet with 3.75-foot deck overhangs. Reducing the girder spacing below 11 feet would lead to an increase in the size of the deck overhangs which would, in turn, lead to larger loading on the exterior girders, particularly the girder on the outside of the curve. A wider girder spacing would increase the deck thickness with a concomitant increase in dead load. The bridge cross section is shown in Figure 1.

Girder Depth

Article 2.5.2.6.3 sets the maximum span-to-depth ratio, L_{as}/D, to 25 where the specified minimum yield stress is not greater than 50 ksi.

In checking this requirement, the arc girder length, L_{as}, for spans continuous on both ends is defined as eighty percent of the longest girder in the span (girder length is taken as the arc length between bearings). The arc girder length of spans continuous on only one end is defined as ninety percent of the longest girder in the span. The longest arc span length (either end or interior span) controls. The maximum arc length occurs at the center span of the outside girder, G4, and measures 214.95 feet. Therefore, the recommended girder depth is computed as follows:

$$0.80 \times 214.95 \times 12 / 25 = 82.5 \text{ in.}$$

A web depth of 84 inches is used.

Minimum Plate Sizes

A minimum thickness of one inch for the flange plates is arbitrarily chosen to minimize distortion due to welding. Article 6.10.2.2 provides several limits for flange proportioning. The provisions recommend a minimum flange thickness of 1.1 times the web thickness. Based on earlier, preliminary designs, a web thickness of 0.625 in. was found to be sufficient for a transversely stiffened web. Therefore, Article 6.10.2.2 recommends a minimum flange thickness of 1.1 x 0.625" = 0.6875 inches. The assumed flange thickness of one inch satisfies this requirement. According to the provisions of Article 6.10.6.2.2, composite sections in horizontally curved steel girder bridges are to be considered noncompact. The provisions of Article 6.10.7.2 for noncompact sections must be satisfied. Article 6.10.2.2 two other flange proportioning limits. The limits are the same for both the compression and tension flanges. The minimum flange width can be determined from equations 1 and 2 knowing the flange thickness and the web depth [$b_f \leq 2t_f(12) \leq 2(1)(12) \leq 24$ in. and $b_f \geq D/6 \geq 84/6 \geq 14$ in.]. In addition, to minimize out-of-plane distortion during construction, Article C6.10.3.4 recommends that the compression flange width, b_{fc}, should satisfy $b_{fc} \geq L/85$ where L is the girder segment shipping length. The minimum flange width is set at 15 in.

Two options are investigated for the web design; a transversely stiffened web, and a longitudinally and transversely stiffened web.

Article 6.10.2.1.2 limits the thickness of longitudinally stiffened webs to D/300. A 7/16-inch web is used throughout the girder for this option (84"/0.4375" = 192 < 300). Although a thinner web could have been used, it would have been difficult to fabricate and to maintain ANSI/AASHTO/AWS flatness requirements without costly straightening. If a thinner web had been used, more than one longitudinal stiffener would have been required in many locations.

Article 6.10.2.1.1 limits the slenderness of webs without longitudinal stiffeners to 150. A 0.5625 in. thick web is used in positive-moment regions of the transversely stiffened web design (84 / 0.5625 = 149 < 150). The web thickness is increased to 0.625 in. in the field sections over the interior piers.

Cross-Frames

The recommended cross-frame spacing of 21 feet is within the maximum spacing allowed by Eq (6.7.4.2-1). Reduction of the cross-frame spacing reduces cross-frame forces since the load transferred between girders is a function of the curvature, and therefore is nearly constant. Reduction of cross-frame spacing also reduces lateral flange bending moments and transverse deck stresses. By reducing lateral flange bending, flange sizes can be reduced, but at the expense of requiring more cross-frames. For the preliminary design, a constant cross-frame spacing of approximately 16 feet was investigated. The final design uses a spacing of approximately 20 feet measured along the centerline of the bridge.

In the analytical model used to analyze the bridge, cross-frames are composed of single angles with an area of 5.0 square inches. Cross-frames with an "X" configuration with top and bottom chords are used for intermediate cross-frames and at interior supports because they generally require the least labor to fabricate. If the girder spacing and or depth is large, a "K" configuration may be used to reduce forces in the diagonals. A “K” configuration is assumed at the simple supports with the “K” pointing up and connected to a beam used to support the edge of the deck (Figure 1).

Field Section Sizes

There is one field splice in each end span and two field splices in the center span resulting in five (5) field sections in each line of girders or 20 field sections for the bridge. An additional girder-line would increase the number of field sections to 25, which would increase fabrication by approximately 25 percent.

FRAMING PLAN FOR FINAL DESIGN

General

A 3D finite element analyses of the aforementioned framing plan was performed. The specified live load(s) were applied to influence surfaces built from the results of analyses for a series of unit vertical loads applied to the deck. All bridge bearings but one girder line were assumed to be free to translate laterally and all bearings were assumed to be fully restrained in the vertical direction for dead and live load analyses.

Noncomposite dead load was applied to the steel section. Separate analyses were made for the self-weight of the steel and for the deck.

Superimposed dead load was composed of the parapets and the future wearing surface and was applied to the fully composite section. The parapet weight was applied at the edges of the deck overhangs.

Cross-Frames

The cross-frame spacing is made nearly uniform over each span in the final design. The preliminary studies were made with 10 panels in the end spans and 14 panels in the center span creating a spacing of approximately 16 feet. A transverse stiffener exists at each cross-frame location which results in a transverse stiffener to web depth ratio of 2.29. This value is less than the ratio of 3 required to consider the web to be an unstiffened web. Therefore, the web was not checked as unstiffened. For this cross-frame spacing, the nominal flange stress was often found to exceed the nominal stress in the web. The cross-frame spacing can be increased causing a reduction in the nominal flange resistance, thereby bringing it closer to the nominal web resistance, which is not affected by the cross-frame spacing. This balancing of the nominal web and flange stresses results in fewer cross-frames without any increase in girder size. In the final design, the cross-frame spacing was increased to approximately 20 ft. resulting in 8 panels in the end spans and 11 panels in the center span. The transverse stiffener to web depth ratio for this cross-frame arrangement is still below 3, therefore, the web is treated as a stiffened web even though there are no intermediate transverse stiffeners between cross-frame locations. Since the number of panels per girder is reduced to 27 from 34, the number of intermediate transverse stiffeners per girder is reduced by 14 [(34 - 27) x 2 = 14] or by 56 for the bridge. The number of cross- frames is reduced by 21. The flange sizes are not increased since the nominal web stress usually limits the design.

Figure 2 shows the framing plan with cross-frame spacing at approximately 20 feet measured along the centerline of the bridge. The node numbering for the three-dimensional finite element model is also shown in this figure. These node numbers will be referred to frequently in the following narratives, tables and sample calculations.

Field Sections

The girder field sections for the transversely stiffened girder design are given in Appendix A for all the girders. The longest field section, the center field section of G4, is approximately 137 feet in length. Field section profiles for the transversely stiffened girder design are given in Appendix F.

FINAL DESIGN

Loads

Noncomposite Dead Load

The steel weight is applied as body forces to the fully erected noncomposite structure in the analysis.

The deck concrete is assumed to be placed and screeded at one time for the Strength limit state.

Constructibility

Staging of the steel erection is considered in addition to the sequential placement of the deck. The deck is considered to be placed in the following sequence for the Constructibility limit state. The concrete is first cast from the left abutment to the dead load inflection point in Span 1. The concrete between dead load inflection points in Span 2 is cast second. The concrete beyond the dead load inflection point to the abutment in Span 3 is cast third. Finally, the concrete between the points of dead load contraflexure near the two piers is cast. In the analysis, earlier concrete casts are made composite for each subsequent cast.

The noncomposite section is checked for these moments when they are larger than the moments computed assuming the entire deck is cast at one time.

The deck load is assumed to be applied through the shear center of the interior girders in the analysis. However, the weight of the fresh concrete on the overhang brackets produces significant lateral force on the flanges of the exterior girders. This eccentric loading further reduces the nominal resistance of these girders.

Superimposed Dead Load

The parapet loads are applied along the edges of the deck in the analysis. These superimposed dead loads are applied to the fully composite structure in the analysis.

Future Wearing Surface

The future wearing surface is applied uniformly over the deck area and is applied to the fully composite structure.

Live Load

A series of vertical unit loads are applied, one at a time, in a pattern covering the deck surface. Live load responses including girder moments, shears, deflections, reactions, cross-frame forces, etc. are determined for each unit load. The magnitude of each response for a particular unit load is the ordinate of the influence surface for that response at the point on the deck where the unit load is applied. Curve fitting is used to create a continuum between these points to develop the influence surface for each response. A computer program then applies the specified live loads to the surfaces according to the AASHTO-LRFD Specifications for live load placement. Multiple presence reduction factors are considered.

Sample calculations for centrifugal force, computed for a design speed of 35 miles per hour, are given at the end of Appendix D. The centrifugal force creates an overturning moment on the truck, which causes an increase in the wheel load on the outside of the curve and a concomitant decrease in the inside wheel load. This overturning effect is also considered when loading the influence surfaces.

Analyses

Load Combinations

Table 3.4.1-1 is used to determine load combinations for strength according to Article 3.4. Strength I loading is used for design of most members for the Strength limit state. However, Load Combinations Strength III and V and Service I and II from Table 3.4.1-1 are also checked for temperature and wind loadings in combination with vertical loading.

The following load combinations and load factors are checked in this design example. In some design instances, other load cases may be critical, but for this example, these other load cases are assumed not to apply.

From Table 3.4.1-1 (minimum load factors of Table 3.4.1-2 are not considered here):

Strength I	η x [1.25(DC) + 1.5(DW) + 1.75((LL + IM) + CE + BR) + 1.2(TU)]
Strength III	η x [1.25(DC) + 1.5(DW) + 1.4(WS) + 1.2(TU)]
Strength V	η x [1.25(DC) + 1.5(DW) + 1.35((LL + IM) + CE + BR) + 0.4(WS) + 1.0(WL) + 1.2(TU)]
Service I	η x [DC + DW + (LL + IM) + CE + BR + 0.3(WS) + WL + 1.2(TU)]
Service II	η x [DC + DW + 1.3((LL + IM) + CE + BR) + 1.2(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, a load combination is included 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.

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 were performed using a three-dimensional finite element program. The section depth is recognized. Girder webs are modeled with shell elements. 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 represented as a series of eight-node solid elements attached to the girders by 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." For the thermal analyses and certain other analyses, proper lateral bearing restraints are specified for the foundation elements.

Cross-frames are modeled as individual truss elements connected to the nodes at the top and bottom of the girders.

Limit States

Strength

Live load responses for HL-93 plus the dynamic load allowance are generated for the Strength limit state. One, two and three traffic lanes are considered. Multiple presence reduction factors specified in Table 3.6.1.1.2-1 are applied. Centrifugal force effects are included. The dynamic load allowance values specified in Table 3.6.2.1-1 for I-girders are used for the truck only. The deck is considered as placed at once on the noncomposite steel.

Constructibility

The erection sequence is investigated to check both deflections and stress according to Article 6.10.3. Sequential deck placement is investigated to check deflections, stress, and concrete crack control. The effects of forces from deck overhang brackets acting on the fascia girders is to be considered according to Article C6.10.3.4.

Fatigue

The range of stress for fatigue is determined by computing the maximum and minimum stress due to one fatigue truck, 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 transverse position of the truck may be different for each response and for positive and negative values of the same 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. 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 when positioning the truck. This assumption provides larger fatigue stresses than would be obtained if the fatigue truck is held to marked traffic lanes. The fatigue truck is permitted to travel within two feet of the curb line. Article 4.5.2.2 specifies that the uncracked composite section is to be used to compute fatigue stresses.

Article 6.6.1.2 specifies that twice the factored fatigue live load defined in Article 3.6.1.4 is to be used to determine if a net tensile stress is created at the point under consideration. The fatigue live load is placed in a single lane. If a net tensile stress occurs under the effect of dead load plus 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 6.6.1.2 requires that lateral bending stresses also be included when computing the stress ranges in the flanges. Lateral bending does not contribute to the stress range at the web-to-flange weld. However, if the connection plates receiving cross- frames are welded to a tension flange, lateral bending contributes to the longitudinal flange stress range at the end of that weld and should be considered.

Cross-frame members are fillet welded to gusset plates, which are bolted to the connection plates in this example. The base metal adjacent to the fillet welds at the end of the cross-frame members must be checked for fatigue Category E. The stress range in these members is computed according to Article 3.6.1.4, which requires that the stress range be determined using a single truck with a fixed rear axle spacing of 30 ft. The dynamic load allowance of 15% is applied according to Table 3.6.2.1-1 and the load factor of 0.75 is applied according to Table 3.4.1-1.

Live Load Deflection

Article 2.5.2.6.2 requires that live load deflection be checked using the live load portion of the Service I Load Combination plus 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 greatly at any one cross section.

Table 1 gives the preferred maximum live load deflections for the center span of each girder according to Article 2.5.2.6.2.

Computed maximum girder deflections in the center span due to the Service I live load plus the dynamic load allowance are given in Table 2 and are based on the use of the uncracked composite section along the entire length of the bridge in the analysis. Centrifugal force effects are included. When multiple lanes are loaded to produce a deflection value given in Table 2, the multiple presence factors specified in Table 3.6.1.1.2-1 are applied.

If a sidewalk existed, vehicular traffic would be constrained from a portion of the deck, which would cause the computed live load deflections to be reduced for either G1 or G4, 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-4, Appendix C, gives selected section properties for G4 for both web designs. Locations from the neutral axis to the top (T) and bottom (B) extreme fiber of the steel section are given, as well as the depth of web in compression, D_c. These values are used in the selected sample calculations that follow in Appendix D.

Composite properties are computed using the provisions of Article 4.6.2.6.1 to calculate the effective flange width. A constant haunch height of 4 inches from the top of the web to the bottom of the deck is assumed. However, the concrete in the 20 inch wide haunch is ignored in the computation of the section properties.

Concrete creep under dead load is accounted for by dividing the effective width by three times the normal modular ratio. For calculations involving the superimposed dead load, the reinforcing steel is also adjusted for creep of the concrete by dividing its area by 3 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. In the negative moment regions, an area of 8.0 square inches per girder is assumed for the longitudinal reinforcement. The neutral axis of the reinforcing is assumed to be 4 inches above the bottom of the deck. The cracked section is assumed for loads applied to the composite section at the Strength limit state. Longitudinal deck reinforcing is considered to be effective for negative moment only.

Flanges

The size of curved I-girder flanges is a function of girder depth, girder radius, cross-frame spacing, and minimum specified yield stress of the flange. Article 6.10.8.2.2 defines a compact flange width-to-thickness ratio limit such that the tip stress in a discretely braced noncomposite compression flange may reach the yield stress prior to the onset of local buckling.

Article 6.10.8.2.2 also defines a noncompact flange width-to-thickness ratio limit which determines if the type of buckling will be elastic or inelastic. At the Strength limit state, the bending stress in discretely braced noncompact compression and tension flanges needs to satisfy the expressions given by Eqns. (6.10.7.2.1-1) and (6.10.7.2.1-2) respectively.

For a compact section at the Strength limit state, the section must satisfy the provisions of Article 6.10.7.1.1.

The Specifications do not require that a check of the flange strength be made at locations where the plate widths change between brace points. The smaller flange plate must be used to compute the strength of a partially braced flange between brace points when the flange size changes within a panel. The largest vertical bending stress at either brace point should be used in conjunction with the lateral flange bending stress at the more critical brace point and the smallest flange size within the panel to compute the nominal flange stress.

For the Constructibility limit state, Article 6.10.3 requires that noncomposite top flanges in compression be designed as discretely braced flanges prior to hardening of the concrete to ensure that no yielding occurs, which tends to lead to the use of wider flanges. Lateral bending in top flanges is not considered after the deck has hardened for any limit state.

Tables 3 and 4 show top and bottom lateral flange bending moments computed by the 3D finite element method and by the approximate Eq (C4.6.1.2.4b-1) near the point of maximum positive moment in Span 1 (Node 40) and at the pier (Node 100) for G4. Lateral moments computed by Eq (C4.6.1.2.4b-1) are generally larger than the comparable values from the 3D analysis in this case.

Webs

According to the Specifications, webs are investigated for elastic bend-buckling at all limit states without consideration of post-buckling shear or bending strength. Bend-buckling must be considered for both the noncomposite and composite cases since the effective slenderness changes when the neutral axis shifts.

Transversely stiffened webs without longitudinal stiffeners may have a slenderness, D/t_w, up to 150 (Article 6.10.2.1.1).

In this example, the maximum allowable spacing of transverse stiffeners equals three times the web depth (Article 6.10.9.1). By limiting the maximum cross-frame spacing to approximately 20 feet, no intermediate transverse stiffeners, other than the connection stiffeners for the cross-frames, are required to consider the web to be stiffened.

Although the final field section profiles given in Appendix F are for the transversely stiffened web design only, Appendix D provides selected calculations for the longitudinally and transversely stiffened web design to show application of the Specifications to this web type.

Shear Connectors

Shear connectors are to be provided throughout the entire length of the bridge in cases of curved continuous structures according to Article 6.10.10.1. The required pitch of the shear connectors is determined for fatigue and checked for strength. Three 7/8" x 6" shear studs per row are assumed in the design. The fatigue strength specified in Article 6.10.10.2 is used for the design of the shear connectors.

The design longitudinal shear range in each stud is computed for a single passage of the factored fatigue truck. The analysis is made assuming that the heavy wheel of the truck is applied to both the positive and negative shear sides of the influence surfaces. This computation tacitly assumes that the truck direction is reversed. In addition to vertical bending shear, Article 6.10.10.1.2 requires that the radial shear due to curvature or radial shear due to causes other than curvature (whichever is larger) be added vectorially to the bending shear for the fatigue check. The deck in the regions between points of dead load contraflexure is considered fully effective in computing the first moment for determining the required pitch for fatigue. This assumption requires tighter shear connector spacing in these regions than if only the longitudinal reinforcing is assumed effective, as is often done. There are several reasons the concrete is assumed effective. First, known field measurements indicate that it is effective at service loads. Second, the horizontal shear force in the deck is considered effective in the analysis and the deck must be sufficiently connected to the steel girders to be consistent with this assumption. Third, maximum shear range occurs when the truck is placed on each side of the point under consideration. Most often this produces positive bending so that the deck is in compression, even when the location is between the point of dead load contraflexure and the pier. The point of dead load contraflexure is obviously a poor indicator of positive or negative bending when moving loads are considered.

The strength check for shear connectors requires that a radial shear force due to curvature be considered. The deck concrete nominal resistance in the negative-moment region is given as $0.45f_c'$ in Article 6.10.10.4.2. This value is a conservative approximation to account for the combined contribution of both the longitudinal reinforcing steel and the concrete that remains effective in tension based on its modulus of rupture.

For both fatigue and strength checks, the parameters used in the equations are determined using the deck within the effective flange width.

Bearing Orientation

Although it is well known that the vertical stiffness of supports affects the analysis of indeterminate beams, the importance of lateral restraint of bearings is less well known. The orientation and lateral restraint of bearings affects the behavior of most girder bridges for most load conditions. Although this is true for most all bridges, it is particularly true for curved and skewed girder bridges.

In this example, the bearings at the piers are assumed fixed against translation in both the radial and tangential directions. The bearings at the abutments are assumed fixed against radial movement but free in the tangential direction. The pier stiffness in the tangential directions is considered and is simulated in the analysis by using a spring with a spring constant smaller than infinity. In the radial directions, the piers and abutments are assumed perfectly rigid. However, for the wind and temperature analyses, only girder G2 is restrained in the radial direction. This is done in the temperature analysis to ensure that thermal expansion and shrinkage between bearings at each pier and abutment does not create very large radial forces, which would not exist in reality because of "slop" in the bearings. The same assumption is made for the wind analyses. This is a very conservative assumption and it permits a very conservative design of the cross-frame with regard to wind. The bearings may be designed by dividing the full wind force by a number between one and the number of girders; four in this example.

The lateral restraints resist the elastic lengthening of the girders due to bending. The result is large lateral bearing forces, which in turn cause an arching effect on the girders that reduces the apparent bending moments due to gravity loads. If the reduced moments were used in the girder design, the bearings would have to function as assumed for the life of the bridge to prevent possible overstress in the girders. To avoid this situation, the lateral bearing restraints are assumed free for the gravity load analyses used to design the girders. However, the proper bearing restraints are assumed in the analyses to determine cross-frame forces and lateral bearing forces for the design of these elements.

Details

In this example, there are no intermediate transverse web stiffeners between the ones at cross-frame locations. If intermediate stiffeners exist, they are typically fillet welded to one side of the web and to the compression flange. Article 6.10.11.1.1 states that when single transverse stiffeners are used, they should be attached to both flanges. Where present, the intermediate stiffeners are fillet welded to the tension flange. The termination of the stiffener-to-web weld adjacent to the tension flange is typically stopped a distance of $4t_w$ from the flange-to-web weld. The base metal adjacent to the stiffener weld to the tension flange is checked for fatigue Category C' (refer to Table 6.6.1.2.3-1). Where the stiffener is fillet welded to the compression flange and the flange undergoes a net tension, the flange must also be checked for the Category C'. When the girder is curved, the lateral flange bending creates an additional stress at the tip of the stiffener-to-flange weld away from the web. Thus, the total stress range is computed from the sum of the lateral and vertical bending stress ranges.

Transverse web stiffeners used as connection plates at cross-frames are fillet welded to the top and bottom flange. When flanges are subjected to a net tensile stress, fatigue must be checked at these points for Category C'.

Base metal at the shear stud connector welds to the top flange must be checked for fatigue Category C whenever the flange is subjected to a net tensile stress.

Cross-frame angles are fillet welded to gusset plates. Therefore, the cross-frame members must be checked for Category E fatigue. The welds are balanced on the two sides of the angles to eliminate eccentricity in one plane.

Erection

Erection is one of the most significant issues pertaining to curved girder bridges. Curved I-girder bridges often require more temporary supports than a straight I-girder bridge of the same span in order to provide stability and deflection control.

Erection of girders in this case is assumed to be performed by assembling and lifting pairs of girders with the cross-frames between the girders bolted into place.

The first lift is composed of two pairs of girders, G1 and G2 and G3 and G4, in Span 1. The positive moment sections of each pair are spliced to the corresponding pier sections before lifting. Each pair of girders is fit up with cross-frames prior to erection and the bolts are tightened. These assemblies are assumed to be accomplished while the girders are fully supported so that strain due to self-weight is negligible in order to simulate the no-load condition in the shop. Each girder pair is then erected. Cross-frames between the two girders, G2 and G3, are then erected and their bolts are tightened. This procedure is repeated in Span 3. The sections in Span 2 are similarly fit up in pairs and erected. Finally, the bolts in the splices in Span 2 are installed and tightened and the cross-frames between the two girders, G2 and G3, in Span 2 are installed. The need for temporary supports in the end spans is investigated in Appendix F.

Wind

Loading. Article 3.8 provides the wind loading to be used for analysis. Article 3.8.1 requires that wind application be multi-directional rather than just perpendicular to the bridge in order to determine the extreme force effects. The wind force on a curved bridge, therefore, equals the wind intensity times the projected area of the bridge. Thus, the total force on the curved bridge is less than that computed if the wind is assumed to be applied along the arc length. According to the Specifications, the wind force must also be applied in various directions to determine the maximum force in the various elements of the structure. In the design example, the wind load is applied with respect to global axes. This requires that the force be separated into X and Y components, which are applied at nodes. Since there are nodes at the top and bottom of the girder, it is possible to divide the wind force between the top and bottom flange. The tributary area for the top of the windward girder equals half of the girder depth plus the height of the exposed deck and parapet concrete times the average spacing to each adjacent node. The tributary area for the bottom of the girder is simply half of the girder depth times the average spacing to each adjacent node.

Since the bridge is superelevated, the girders on the inside of the curve extend below the outside girder G4. Each girder extends downward approximately 6 inches. This exposed area is also recognized in the loading if the wind is applied from the G4 side of the bridge. If wind is applied from the G1 side of the bridge, an additional upward projection due to superelevation is manifest in the parapet on the opposite side near G4 and is recognized in computing the wind loading.

When the girders are being erected, wind load may be applied across the ends of the girders, which are temporarily exposed.

Analysis. The completed bridge has an exposed height of approximately 10.5 feet. The design wind intensity of 50 psf results in a total wind force of 525 pounds per foot applied to the projected length of bridge. Load Combination Service I includes wind on the live load in conjunction with wind on the structure. The wind on the live load is specified as 100 pounds per linear foot. For this load combination, the wind on the structure is factored by 0.3 (0.3 x 525 = 157.5 pounds per linear foot). Thus, the net wind load is 257.5 pounds per foot.

If the wind load is applied such that all girders are exposed, such as across the end of the first phase of the erection process, wind load is applied to each girder. Since this bridge is superelevated 5 percent, the exposed height of bridge must be calculated considering the effect of superelevation. If wind is coming from the outside of the curve, additional wind load must be applied to the bottom of the girders away from the windward side. If the wind is applied from the inside of the curve, additional wind load is applied to the opposite parapet. The girder spacing of 11 feet causes the elevation of adjacent girders to differ by 0.55 feet. Additional wind load is applied to the bottoms of each interior girder when the wind is applied from the Girder 4 side (outside of the curve). Girder 4 is the highest girder. The width of the bridge is approximately 40 feet, so the parapet on the outside of the curve is approximately 2 feet higher than on the inside of the curve. Thus, the parapet receives additional wind load on its projected area when the wind is applied from the inside of the curve.

A three-dimensional finite element analysis was made assuming that the wind load was applied from the G4 side at 248 degrees clockwise from north. The first abutment is oriented north and south. This angle is orthogonal to a chord drawn between the abutments. The wind was applied to the outside of the curve at an angle which caused the largest possible total wind force. The majority of the wind force was applied to G4. Another analysis applied the wind in the opposite direction (248 -180 = 68 degrees). Superelevation exposes the upper portion of the bridge so an additional wind force (2 feet times 50 pounds per square foot = 100 pounds per linear foot) was applied to the parapet on the outside of the curve in the analysis. Results from this analysis produced the largest cross bracing forces for the assumed bearing arrangement.

Construction. In addition to the AASHTO-LRFD load combinations, Article 2.5.3 requires that each critical phase of construction be examined. A load factor of 1.25 is used for this limit state as specified in Article 3.4.2.

Two stages of steel erection were considered. Since the deck is not in-place, the girders are capable of taking almost no lateral load without top flange bracing. Therefore, bracing was added between the top flanges to resist wind during erection. The top flanges act as chords to a horizontal truss formed by the two girders.

The wind analysis for the construction condition was made assuming the wind to be acting perpendicular to the bridge at the first abutment. An additional wind load of 50 psf x 0.55 ft was applied to the top flange of Girder G2 to account for the superelevation. An additional wind load was also applied to the top and bottom flanges at the end of Girder G2 to account for the projection of Girder G2 three feet beyond Girder G1 in the X- direction. The results from this analysis are discussed further in Appendix F.

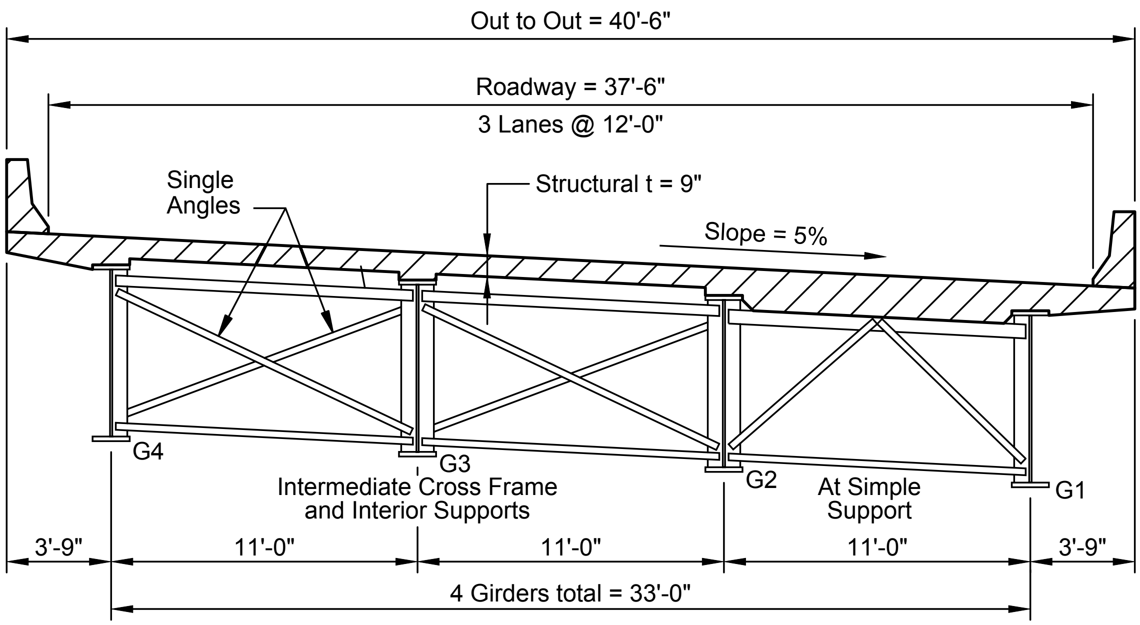
Deck Staging

The deck is assumed to be placed in four casts. The first cast is in Span 1 commencing at the abutment and ending at the point of dead load contraflexure. The second cast is in Span 2 between points of dead load contraflexure. The third cast is in Span 3 from the point of dead load contraflexure to the abutment. The fourth cast is over both piers.

The unfactored moments from the deck staging analysis for the transversely stiffened girder design 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" and "FWS" identify the moments due to the superimposed dead load and future wearing surface, respectively, 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 analysis so that uplift can be checked 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 analyses since full creep usually takes approximately three years to occur. A modular ratio of "n" should be used to check the deck stresses.

Sample Calculations

Sample calculations at selected critical sections of the exterior girder, G4, are presented in Appendix D. Calculations are illustrated for all three web designs. The calculations are intended to illustrate the application of some of the more significant curved girder design provisions contained in the 2005 Interim to the AASHTO-LRFD Bridge Design 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 illustrate stiffener designs, a bolted field splice design, a cross-frame diagonal design and centrifugal force calculations. The calculations make use of the moments and shears contained in Tables C-1 through C-3 of Appendix C and the section properties contained in Table C-4. The same moments and shears are used for both designs in the sample calculations for simplicity and since the cross-sectional stiffnesses do not vary significantly in the two designs (transversely stiffened and longitudinally stiffened).



Deck concrete – f'_c = 4,000 psi   E = 3.6×10^6 psi
Haunch – 20 in. wide, 4 in. deep measured from top of web
Permanent deck forms are present
Total deck thickness = 9.5 in., structural thickness = 9.0 in.

Figure 1. I-Girder Bridge Cross Section

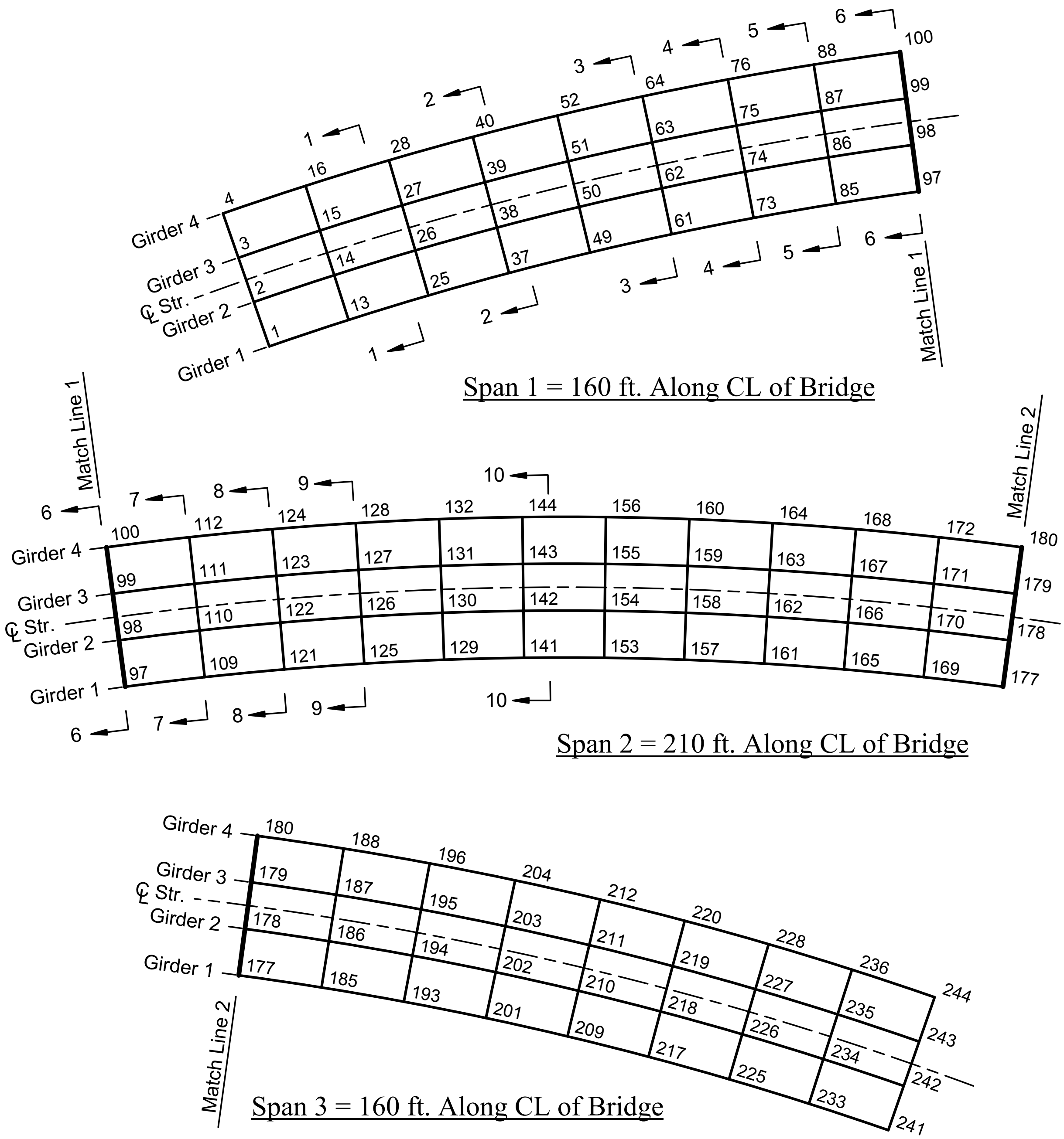


Figure 2. Framing Plan and Nodal Numbering

Table 1. Preferred Maximum Live Load Deflections in Center-Span (in.)

Girder	L (ft)	L/800	L/1,000
G1	205.0	3.08	2.46
G2	208.4	3.13	2.50
G3	211.6	3.18	2.54
G4	215.0	3.23	2.58

Table 2. Computed Maximum Live Load Deflections in Center-Span (in.)

Girder	HL-93 Loading
G1	1.38
G2	1.33
G3	1.70
G4	2.32

Table 3. Comparison of Lateral Flange Moments from 3D Analysis and Eq (4.6.1.2.4b-1)

Loading	Lat. Mom. (k-ft) Top Flange		Lat. Mom. (k-ft) Bottom Flange	
	3D	Eq. (4.6.1.2.4b-1)	3D	Eq. (4.6.1.2.4b-1)
Steel	-4	-6	4	6
Deck	-16	-23	16	23
SupImp DL	--	--	2	4
FWS	--	--	4	6
Total DL	-20	-29	26	39
Single truck	--	--	16	10
HL-93 Load	--	--	21	42
Total	-20	-29	47	81

Node 40 Near mid-span 1 Girder 4

Table 4. Comparison of Lateral Flange Moments from 3D Analysis and Eq (4.6.1.2.4b-1)

Loading	Lat. Mom. (k-ft) Top Flange		Lat. Mom. (k-ft) Bottom Flange	
	3D	Eq. (4.6.1.2.4b-1)	3D	Eq. (4.6.1.2.4b-1)
Steel	8	16	-7	-16
Deck	35	60	-30	-60
SupImp DL	--	--	-4	-13
FWS	--	--	-3	-12
Total DL	43	76	-44	-101
Single truck	--	--	-11	-8
HL-93 Load	--	--	-16	-56
Total	43	76	-60	-157

Node 100 Interior support Girder 4

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APPENDIX A

Girder Field Sections

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Bridge Type --> I - Girder                Date Created -> 09/04/95
Project  -----> I Girder Example          Initials -----> DHH

Project ID ---> DESIGN IG2
Description --> 3-span 4-girder 700-foot radius

                 Number of girders --->  4
                 Number of spans   --->  3
                 Project units     --->  English

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

Girder --> 1 Field Section --> 1

	Rght		-----Top Flange----			---Bottom Flange--			---- Web ------		
Mem.	Node	Length	Width	Thick.	Fy	Width	Thick.	Fy	Depth	Thick.	Fy
1	5	6.51	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
2	9	6.51	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
3	13	6.51	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
4	17	6.51	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
5	21	6.51	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
6	25	6.51	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
7	29	6.51	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
8	33	6.51	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
9	37	6.51	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
10	41	6.51	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
11	45	6.51	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
12	49	6.51	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
13	53	6.51	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
14	57	6.51	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
15	61	6.51	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
16	65	6.51	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
17	69	6.51	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
18	73	6.51	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.

	Top Flange	Bot Flange	Web	TOTAL	Length
Section Weight -->	5984.	6383.	18848.	31215.	Ft.-> 117.23

Girder --> 1 Field Section --> 2

	Rght		-----Top Flange----			---Bottom Flange--			---- Web ------		
Mem.	Node	Length	Width	Thick.	Fy	Width	Thick.	Fy	Depth	Thick.	Fy
19	77	6.51	21.00	1.2500	50.	21.00	1.5000	50.	84.00	.6250	50.
20	81	6.51	21.00	1.2500	50.	21.00	1.5000	50.	84.00	.6250	50.
21	85	6.51	21.00	1.2500	50.	21.00	1.5000	50.	84.00	.6250	50.
22	89	6.50	21.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
23	93	6.50	21.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
24	97	6.45	21.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
Sup	--->	156.23									
25	101	6.21	21.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
26	105	6.21	21.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
27	109	6.21	21.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
28	113	6.21	21.00	1.2500	50.	21.00	1.5000	50.	84.00	.6250	50.
29	117	6.21	21.00	1.2500	50.	21.00	1.5000	50.	84.00	.6250	50.
30	121	6.21	21.00	1.2500	50.	21.00	1.5000	50.	84.00	.6250	50.

	Top Flange	Bot Flange	Web	TOTAL	Length
Section Weight -->	10214.	12257.	13623.	36094.	Ft.-> 76.26

Girder --> 1 Field Section --> 3

	Rght		-----Top Flange----			---Bottom Flange--			---- Web ------		
Mem.	Node	Length	Width	Thick.	Fy	Width	Thick.	Fy	Depth	Thick.	Fy
31	125	18.64	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
32	129	18.64	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
33	133	6.21	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
34	137	6.21	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
35	141	6.21	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
36	145	6.21	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
37	149	6.21	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
38	153	6.21	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
39	157	18.64	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
40	161	18.64	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
41	165	18.64	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.

	Top Flange	Bot Flange	Web	TOTAL	Length
Section Weight -->	6659.	7991.	20976.	35625.	Ft.-> 130.46

Girder --> 1 Field Section --> 4

	Rght		-----Top Flange----			---Bottom Flange--			---- Web ------		
Mem.	Node	Length	Width	Thick.	Fy	Width	Thick.	Fy	Depth	Thick.	Fy
42	169	18.64	21.00	1.2500	50.	21.00	1.5000	50.	84.00	.6250	50.
43	173	9.34	21.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
44	177	9.34	21.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
Sup --->		205.05									
45	181	9.76	21.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
46	185	9.76	21.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
47	189	9.76	21.00	1.2500	50.	21.00	1.5000	50.	84.00	.6250	50.
48	193	9.76	21.00	1.2500	50.	21.00	1.5000	50.	84.00	.6250	50.

	Top Flange	Bot Flange	Web	TOTAL	Length
Section Weight -->	10237.	12284.	13646.	36167.	Ft.-> 76.39

Girder --> 1 Field Section --> 5

	Rght		-----Top Flange----			---Bottom Flange--			---- Web ------		
Mem.	Node	Length	Width	Thick.	Fy	Width	Thick.	Fy	Depth	Thick.	Fy
49	197	9.76	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
50	201	9.76	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
51	205	9.76	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
52	209	9.76	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
53	213	9.76	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
54	217	9.76	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
55	221	9.76	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
56	225	9.76	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
57	229	9.76	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
58	233	9.76	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
59	237	9.76	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
60	241	9.76	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.

Sup ---> 156.23

	Top Flange	Bot Flange	Web	TOTAL	Length
Section Weight -->	5981.	6379.	18839.	31199.	Ft.-> 117.17
Girder Weight -->	39074.	45293.	85932.	170300.	Ft.-> 517.51

Girder --> 2 Field Section --> 1

	Rght		-----Top Flange----			---Bottom Flange--			---- Web ------		
Mem.	Node	Length	Width	Thick.	Fy	Width	Thick.	Fy	Depth	Thick.	Fy
61	6	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
62	10	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
63	14	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
64	18	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
65	22	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
66	26	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
67	30	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
68	34	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
69	38	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
70	42	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
71	46	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
72	50	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
73	54	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
74	58	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
75	62	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
76	66	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
77	70	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
78	74	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.

	Top Flange	Bot Flange	Web	TOTAL	Length
Section Weight -->	6080.	6485.	19152.	31717.	Ft.-> 119.12

Girder --> 2   Field Section --> 2

	Rght		-----Top Flange----			---Bottom Flange--			---- Web ------		
Mem.	Node	Length	Width	Thick.	Fy	Width	Thick.	Fy	Depth	Thick.	Fy
79	78	6.62	18.00	1.2500	50.	19.00	1.5000	50.	84.00	.6250	50.
80	82	6.62	18.00	1.2500	50.	19.00	1.5000	50.	84.00	.6250	50.
81	86	6.62	18.00	1.2500	50.	19.00	1.5000	50.	84.00	.6250	50.
82	90	6.61	18.00	2.5000	50.	19.00	3.0000	50.	84.00	.6250	50.
83	94	6.61	18.00	2.5000	50.	19.00	3.0000	50.	84.00	.6250	50.
84	98	6.56	18.00	2.5000	50.	19.00	3.0000	50.	84.00	.6250	50.
Sup	--->	158.74									
85	102	6.31	18.00	2.5000	50.	19.00	3.0000	50.	84.00	.6250	50.
86	106	6.31	18.00	2.5000	50.	19.00	3.0000	50.	84.00	.6250	50.
87	110	6.31	18.00	2.5000	50.	19.00	3.0000	50.	84.00	.6250	50.
88	114	6.31	18.00	1.2500	50.	19.00	1.5000	50.	84.00	.6250	50.
89	118	6.31	18.00	1.2500	50.	19.00	1.5000	50.	84.00	.6250	50.
90	122	6.31	18.00	1.2500	50.	19.00	1.5000	50.	84.00	.6250	50.

	Top Flange	Bot Flange	Web	TOTAL		Length
Section Weight -->	8896.	11268.	13843.	34006.	Ft.->	77.49

Girder --> 2   Field Section --> 3

	Rght		-----Top Flange----			---Bottom Flange--			---- Web ------		
Mem.	Node	Length	Width	Thick.	Fy	Width	Thick.	Fy	Depth	Thick.	Fy
91	126	18.94	15.00	1.0000	50.	17.00	1.0000	50.	84.00	.5625	50.
92	130	18.94	15.00	1.0000	50.	17.00	1.0000	50.	84.00	.5625	50.
93	134	6.31	15.00	1.0000	50.	17.00	1.0000	50.	84.00	.5625	50.
94	138	6.31	15.00	1.0000	50.	17.00	1.0000	50.	84.00	.5625	50.
95	142	6.31	15.00	1.0000	50.	17.00	1.0000	50.	84.00	.5625	50.
96	146	6.31	15.00	1.0000	50.	17.00	1.0000	50.	84.00	.5625	50.
97	150	6.31	15.00	1.0000	50.	17.00	1.0000	50.	84.00	.5625	50.
98	154	6.31	15.00	1.0000	50.	17.00	1.0000	50.	84.00	.5625	50.
99	158	18.94	15.00	1.0000	50.	17.00	1.0000	50.	84.00	.5625	50.
100	162	18.94	15.00	1.0000	50.	17.00	1.0000	50.	84.00	.5625	50.
101	166	18.94	15.00	1.0000	50.	17.00	1.0000	50.	84.00	.5625	50.

	Top Flange	Bot Flange	Web	TOTAL		Length
Section Weight -->	6766.	7668.	21313.	35748.	Ft.->	132.56

Girder --> 2 Field Section --> 4

	Rght		-----Top Flange----			---Bottom Flange--			---- Web ------		
Mem.	Node	Length	Width	Thick.	Fy	Width	Thick.	Fy	Depth	Thick.	Fy
102	170	18.94	18.00	1.2500	50.	19.00	1.5000	50.	84.00	.6250	50.
103	174	9.49	18.00	2.5000	50.	19.00	3.0000	50.	84.00	.6250	50.
104	178	9.49	18.00	2.5000	50.	19.00	3.0000	50.	84.00	.6250	50.
Sup ---> 208.35											
105	182	9.92	18.00	2.5000	50.	19.00	3.0000	50.	84.00	.6250	50.
106	186	9.92	18.00	2.5000	50.	19.00	3.0000	50.	84.00	.6250	50.
107	190	9.92	18.00	1.2500	50.	19.00	1.5000	50.	84.00	.6250	50.
108	194	9.92	18.00	1.2500	50.	19.00	1.5000	50.	84.00	.6250	50.

	Top Flange	Bot Flange	Web	TOTAL	Length
Section Weight -->	8916.	11293.	13866.	34074.	Ft.-> 77.62

Girder --> 2 Field Section --> 5

	Rght		-----Top Flange----			---Bottom Flange--			---- Web ------		
Mem.	Node	Length	Width	Thick.	Fy	Width	Thick.	Fy	Depth	Thick.	Fy
109	198	9.92	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
110	202	9.92	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
111	206	9.92	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
112	210	9.92	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
113	214	9.92	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
114	218	9.92	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
115	222	9.92	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
116	226	9.92	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
117	230	9.92	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
118	234	9.92	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
119	238	9.92	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
120	242	9.92	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
Sup ---> 158.74											

	Top Flange	Bot Flange	Web	TOTAL	Length
Section Weight -->	6077.	6482.	19142.	31701.	Ft.-> 119.06
Girder Weight -->	36734.	43197.	87315.	167246.	Ft.-> 525.84

Girder --> 3   Field Section --> 1

	Rght		-----Top Flange----			---Bottom Flange--			---- Web ------		
Mem.	Node	Length	Width	Thick.	Fy	Width	Thick.	Fy	Depth	Thick.	Fy
121	7	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
122	11	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
123	15	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
124	19	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
125	23	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
126	27	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
127	31	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
128	35	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
129	39	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
130	43	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
131	47	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
132	51	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
133	55	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
134	59	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
135	63	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
136	67	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
137	71	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
138	75	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.

	Top Flange	Bot Flange	Web	TOTAL	Length
Section Weight -->	6588.	7411.	19455.	33454.	Ft.-> 121.00

Girder --> 3   Field Section --> 2

	Rght		-----Top Flange----			---Bottom Flange--			---- Web ------		
Mem.	Node	Length	Width	Thick.	Fy	Width	Thick.	Fy	Depth	Thick.	Fy
139	79	6.72	20.00	1.2500	50.	21.00	1.5000	50.	84.00	.6250	50.
140	83	6.72	20.00	1.2500	50.	21.00	1.5000	50.	84.00	.6250	50.
141	87	6.72	20.00	1.2500	50.	21.00	1.5000	50.	84.00	.6250	50.
142	91	6.71	20.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
143	95	6.71	20.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
144	99	6.66	20.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
Sup ---> 161.26											
145	103	6.41	20.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
146	107	6.41	20.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
147	111	6.41	20.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
148	115	6.41	20.00	1.2500	50.	21.00	1.5000	50.	84.00	.6250	50.
149	119	6.41	20.00	1.2500	50.	21.00	1.5000	50.	84.00	.6250	50.
150	123	6.41	20.00	1.2500	50.	21.00	1.5000	50.	84.00	.6250	50.

	Top Flange	Bot Flange	Web	TOTAL	Length
Section Weight -->	10041.	12651.	14062.	36754.	Ft.-> 78.71

Girder --> 3 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
151	127	19.24	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
152	131	19.24	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
153	135	6.41	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
154	139	6.41	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
155	143	6.41	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
156	147	6.41	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
157	151	6.41	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
158	155	6.41	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
159	159	19.24	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
160	163	19.24	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
161	167	19.24	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.

	Top Flange	Bot Flange	Web	TOTAL	Length
Section Weight -->	6873.	8248.	21651.	36772.	Ft.-> 134.66

Girder --> 3 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
162	171	19.24	20.00	1.2500	50.	21.00	1.5000	50.	84.00	.6250	50.
163	175	9.65	20.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
164	179	9.65	20.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
Sup --->		211.65									
165	183	10.08	20.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
166	187	10.08	20.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
167	191	10.08	20.00	1.2500	50.	21.00	1.5000	50.	84.00	.6250	50.
168	195	10.08	20.00	1.2500	50.	21.00	1.5000	50.	84.00	.6250	50.

	Top Flange	Bot Flange	Web	TOTAL	Length
Section Weight -->	10063.	12679.	14085.	36828.	Ft.-> 78.84

Girder --> 3 Field Section --> 5

	Rght		-----Top Flange----			---Bottom Flange--			---- Web ------		
Mem.	Node	Length	Width	Thick.	Fy	Width	Thick.	Fy	Depth	Thick.	Fy
169	199	10.08	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
170	203	10.08	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
171	207	10.08	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
172	211	10.08	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
173	215	10.08	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
174	219	10.08	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
175	223	10.08	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
176	227	10.08	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
177	231	10.08	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
178	235	10.08	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
179	239	10.08	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
180	243	10.08	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.

Sup ---> 161.26

	Top Flange	Bot Flange	Web	TOTAL	Length
Section Weight -->	6585.	7408.	19445.	33438.	Ft.-> 120.94
Girder Weight -->	40150.	48398.	88698.	177246.	Ft.-> 534.16

Girder --> 4 Field Section --> 1

	Rght		-----Top Flange----			---Bottom Flange--			---- Web ------		
Mem.	Node	Length	Width	Thick.	Fy	Width	Thick.	Fy	Depth	Thick.	Fy
181	8	6.83	20.00	1.0000	50.	21.00	1.0000	50.	84.00	.5625	50.
182	12	6.83	20.00	1.0000	50.	21.00	1.0000	50.	84.00	.5625	50.
183	16	6.83	20.00	1.0000	50.	21.00	1.0000	50.	84.00	.5625	50.
184	20	6.83	20.00	1.0000	50.	21.00	1.0000	50.	84.00	.5625	50.
185	24	6.83	20.00	1.0000	50.	21.00	1.0000	50.	84.00	.5625	50.
186	28	6.83	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
187	32	6.83	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
188	36	6.83	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
189	40	6.83	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
190	44	6.83	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
191	48	6.83	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
192	52	6.83	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
193	56	6.83	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
194	60	6.83	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
195	64	6.83	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
196	68	6.83	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
197	72	6.83	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
198	76	6.83	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.

	Top Flange	Bot Flange	Web	TOTAL	Length
Section Weight -->	8363.	11953.	19758.	40074.	Ft.-> 122.89

Girder --> 4 Field Section --> 2

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
199	80	6.83	28.00	1.2500	50.	27.00	1.5000	50.	84.00	.6250	50.
200	84	6.83	28.00	1.2500	50.	27.00	1.5000	50.	84.00	.6250	50.
201	88	6.83	28.00	1.2500	50.	27.00	1.5000	50.	84.00	.6250	50.
202	92	6.82	28.00	2.5000	50.	27.00	3.0000	50.	84.00	.6250	50.
203	96	6.82	28.00	2.5000	50.	27.00	3.0000	50.	84.00	.6250	50.
204	100	6.77	28.00	2.5000	50.	27.00	3.0000	50.	84.00	.6250	50.
Sup ---> 163.77											
205	104	6.51	28.00	2.5000	50.	27.00	3.0000	50.	84.00	.6250	50.
206	108	6.51	28.00	2.5000	50.	27.00	3.0000	50.	84.00	.6250	50.
207	112	6.51	28.00	2.5000	50.	27.00	3.0000	50.	84.00	.6250	50.
208	116	6.51	28.00	1.2500	50.	27.00	1.5000	50.	84.00	.6250	50.
209	120	6.51	28.00	1.2500	50.	27.00	1.5000	50.	84.00	.6250	50.
210	124	6.51	28.00	1.2500	50.	27.00	1.5000	50.	84.00	.6250	50.

	Top Flange	Bot Flange	Web	TOTAL	Length
Section Weight -->	14276.	16520.	14281.	45077.	Ft.-> 79.94

Girder --> 4 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
211	128	19.54	17.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
212	132	19.54	17.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
213	136	6.51	17.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
214	140	6.51	17.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
215	144	6.51	17.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
216	148	6.51	17.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
217	152	6.51	17.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
218	156	6.51	17.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
219	160	19.54	17.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
220	164	19.54	17.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
221	168	19.54	17.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.

	Top Flange	Bot Flange	Web	TOTAL	Length
Section Weight -->	7911.	14659.	21988.	44558.	Ft.-> 136.76

Girder --> 4   Field Section --> 4

	Rght		-----Top Flange----			---Bottom Flange--			---- Web ------		
Mem.	Node	Length	Width	Thick.	Fy	Width	Thick.	Fy	Depth	Thick.	Fy
222	172	19.54	28.00	1.2500	50.	27.00	1.5000	50.	84.00	.6250	50.
223	176	9.80	28.00	2.5000	50.	27.00	3.0000	50.	84.00	.6250	50.
224	180	9.80	28.00	2.5000	50.	27.00	3.0000	50.	84.00	.6250	50.
Sup ---> 214.95											
225	184	10.24	28.00	2.5000	50.	27.00	3.0000	50.	84.00	.6250	50.
226	188	10.24	28.00	2.5000	50.	27.00	3.0000	50.	84.00	.6250	50.
227	192	10.24	28.00	1.2500	50.	27.00	1.5000	50.	84.00	.6250	50.
228	196	10.24	28.00	1.2500	50.	27.00	1.5000	50.	84.00	.6250	50.

	Top Flange	Bot Flange	Web	TOTAL	Length
Section Weight -->	14308.	16556.	14305.	45169.	Ft.-> 80.07

Girder --> 4   Field Section --> 5

	Rght		-----Top Flange----			---Bottom Flange--			---- Web ------		
Mem.	Node	Length	Width	Thick.	Fy	Width	Thick.	Fy	Depth	Thick.	Fy
229	200	10.24	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
230	204	10.24	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
231	208	10.24	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
232	212	10.24	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
233	216	10.24	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
234	220	10.24	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
235	224	10.24	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
236	228	10.24	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
237	232	10.24	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
238	236	10.24	20.00	1.0000	50.	21.00	1.0000	50.	84.00	.5625	50.
239	240	10.24	20.00	1.0000	50.	21.00	1.0000	50.	84.00	.5625	50.
240	244	10.24	20.00	1.0000	50.	21.00	1.0000	50.	84.00	.5625	50.
Sup ---> 163.77											

	Top Flange	Bot Flange	Web	TOTAL	Length
Section Weight -->	8359.	12069.	19749.	40176.	Ft.-> 122.83
Girder Weight -->	53218.	71756.	90081.	215055.	Ft.-> 542.49

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APPENDIX B

Girder Moments and Shears at Tenth Points

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September 23, 1995 8:31 PM

```
Bridge Type --> I - Girder              Date Created -> 09/04/95
Project  -----> I Girder Example        Initials -----> DHH

Project ID ---> DESIGN IG2
Description --> 3-span 4-girder 700-foot radius

                 Number of girders --->  4
                 Number of spans   --->  3
                 Project units     --->  English
```

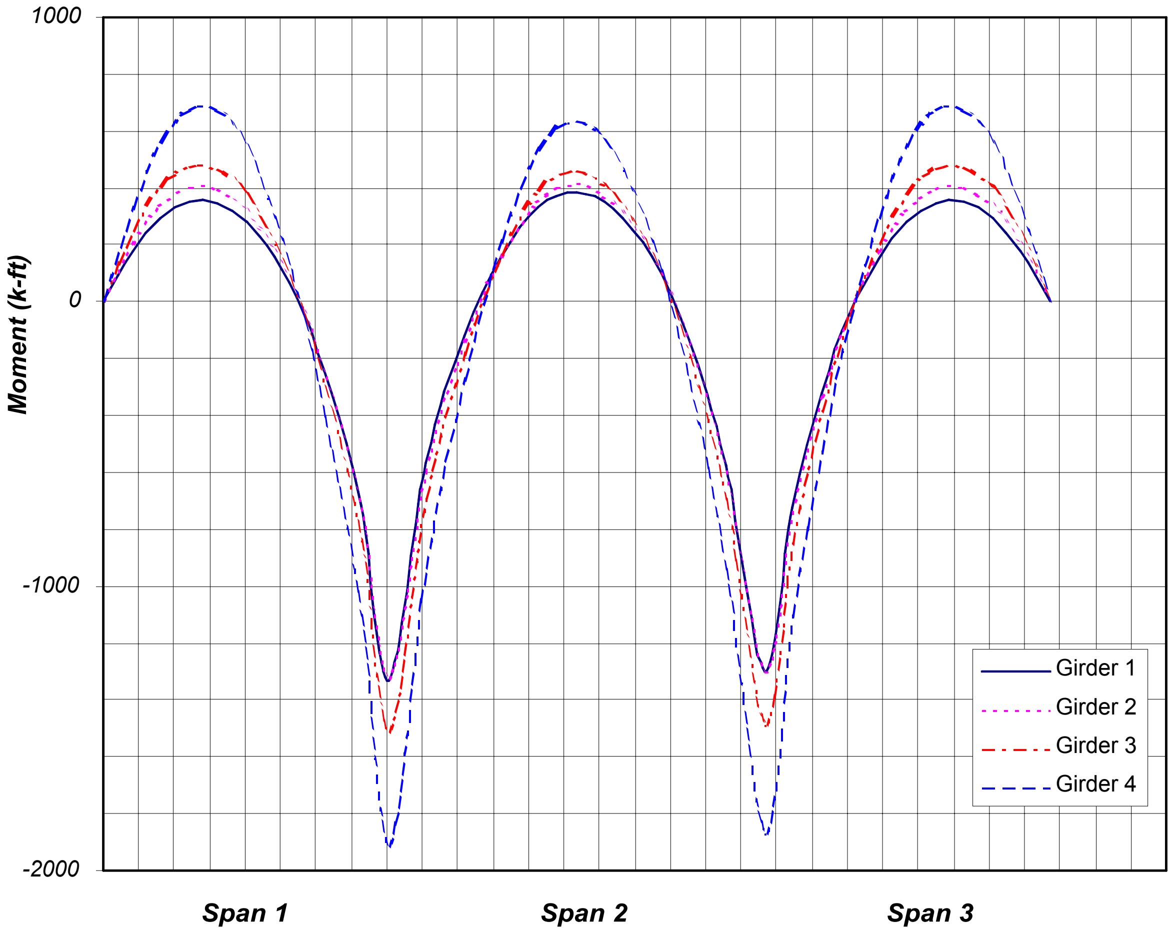
BRIDGE-SYSTEMsm 3D Version -> 2.1

Stage Definition

Stg1 = Load due to weight of structural steel including girders and cross-frames

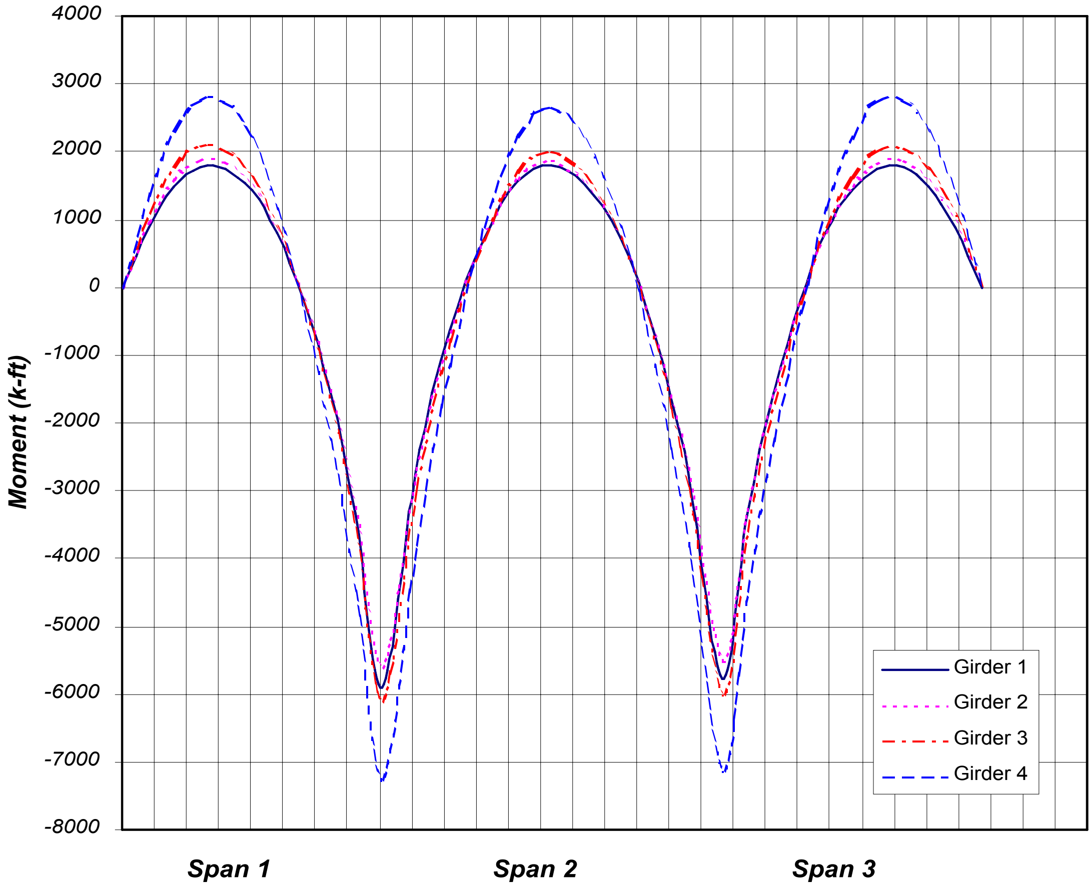
Stg6 = Load due to weight of concrete deck placed at one time

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



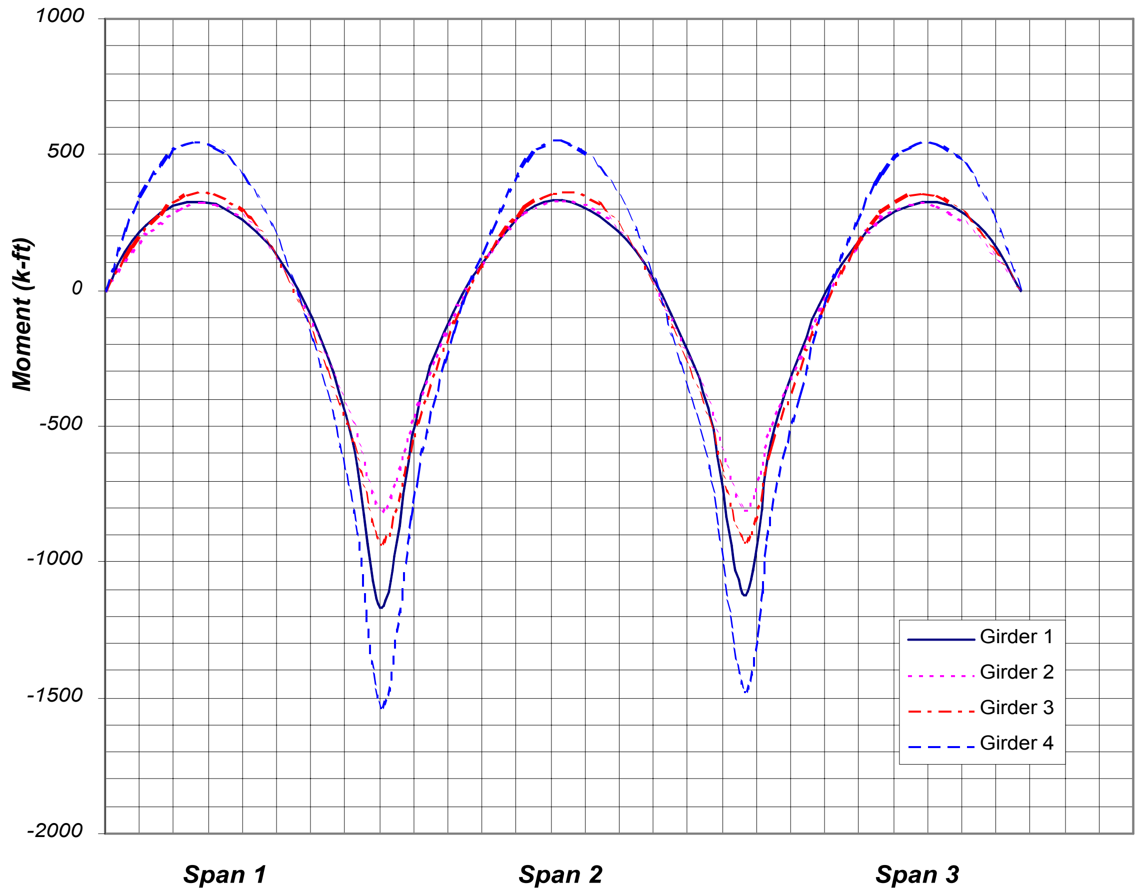
STAGE 1 DEAD LOAD MOMENT (STRUCTURAL STEEL)
(GIRDERS)

Figure B-1. Dead Load (Structural Steel) Moment



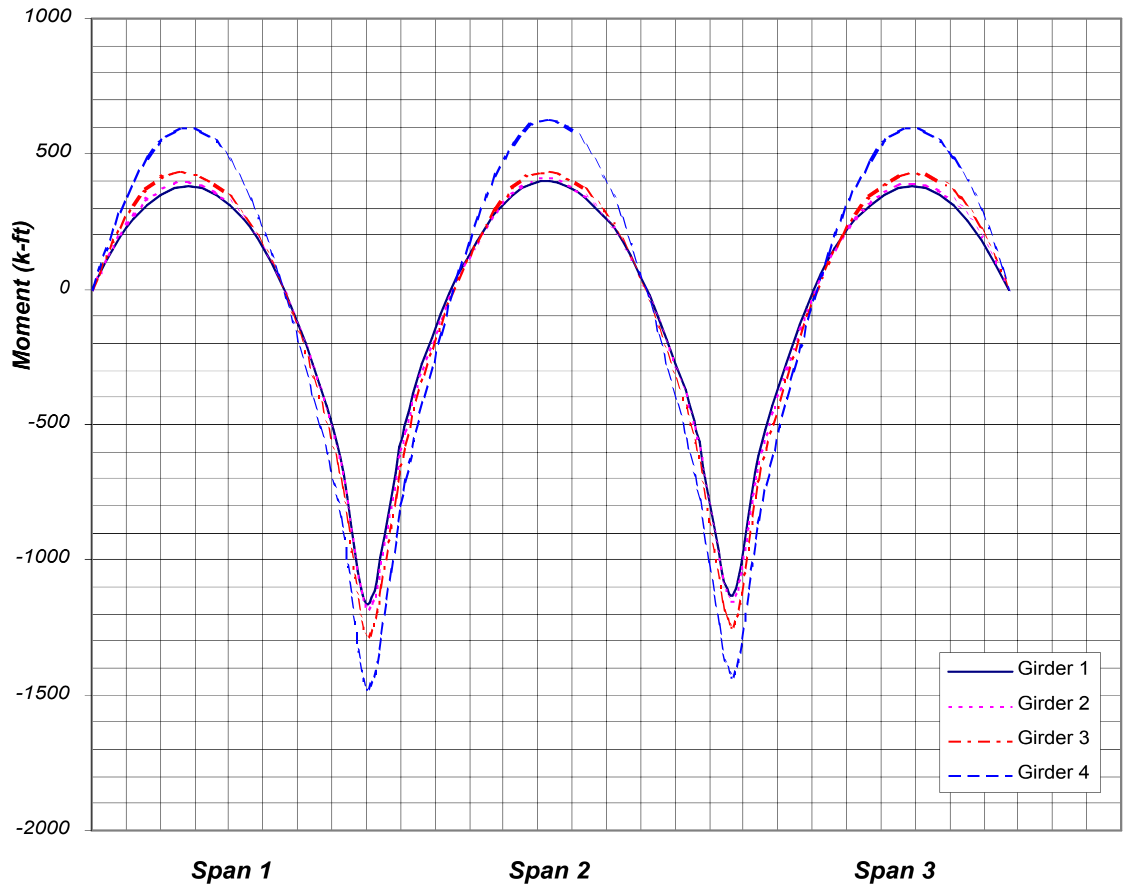
STAGE 6 DEAD LOAD MOMENT (CONCRETE DECK)
(GIRDERS)

Figure B-2. Dead Load (Concrete Deck) Moment



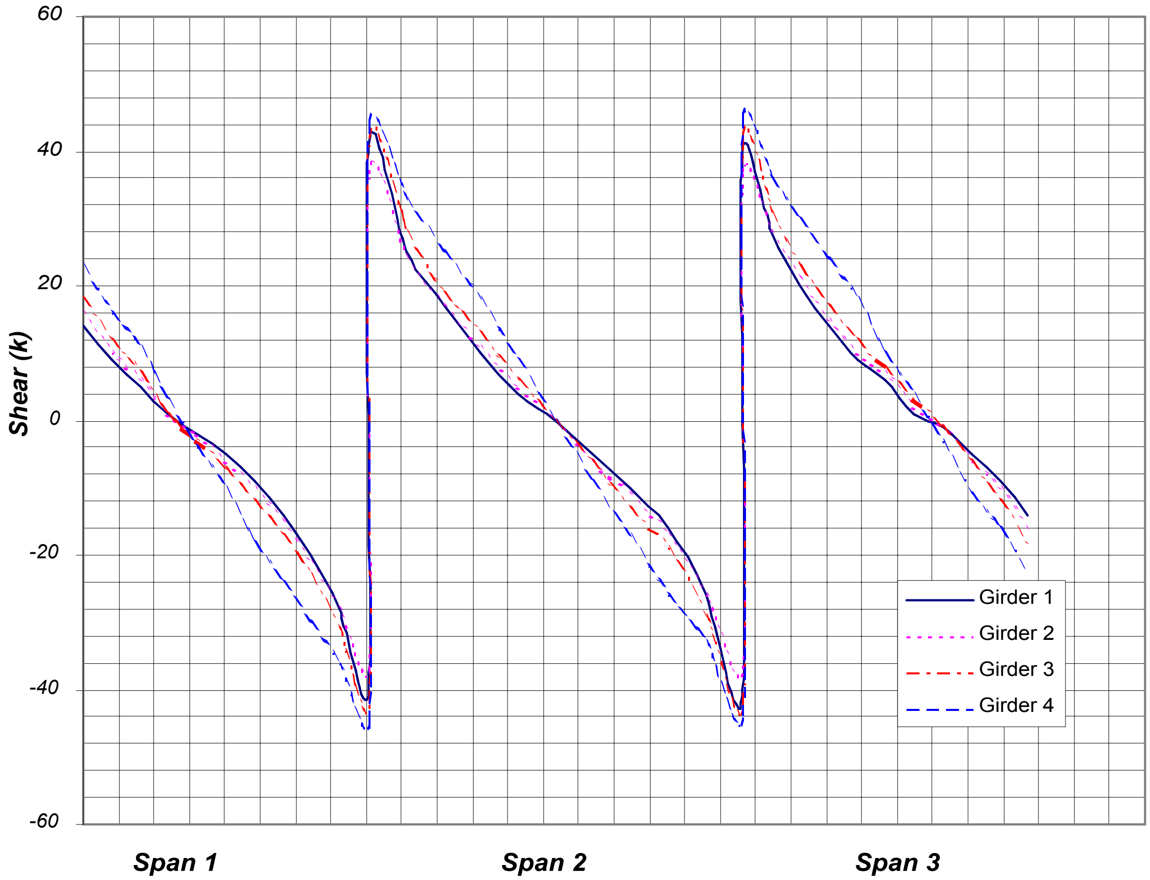
STAGE 7 DEAD LOAD MOMENT (SUPERIMPOSED DEAD LOAD)
(GIRDERS)

Figure B-3. Dead Load (Superimposed Dead Load) Moment



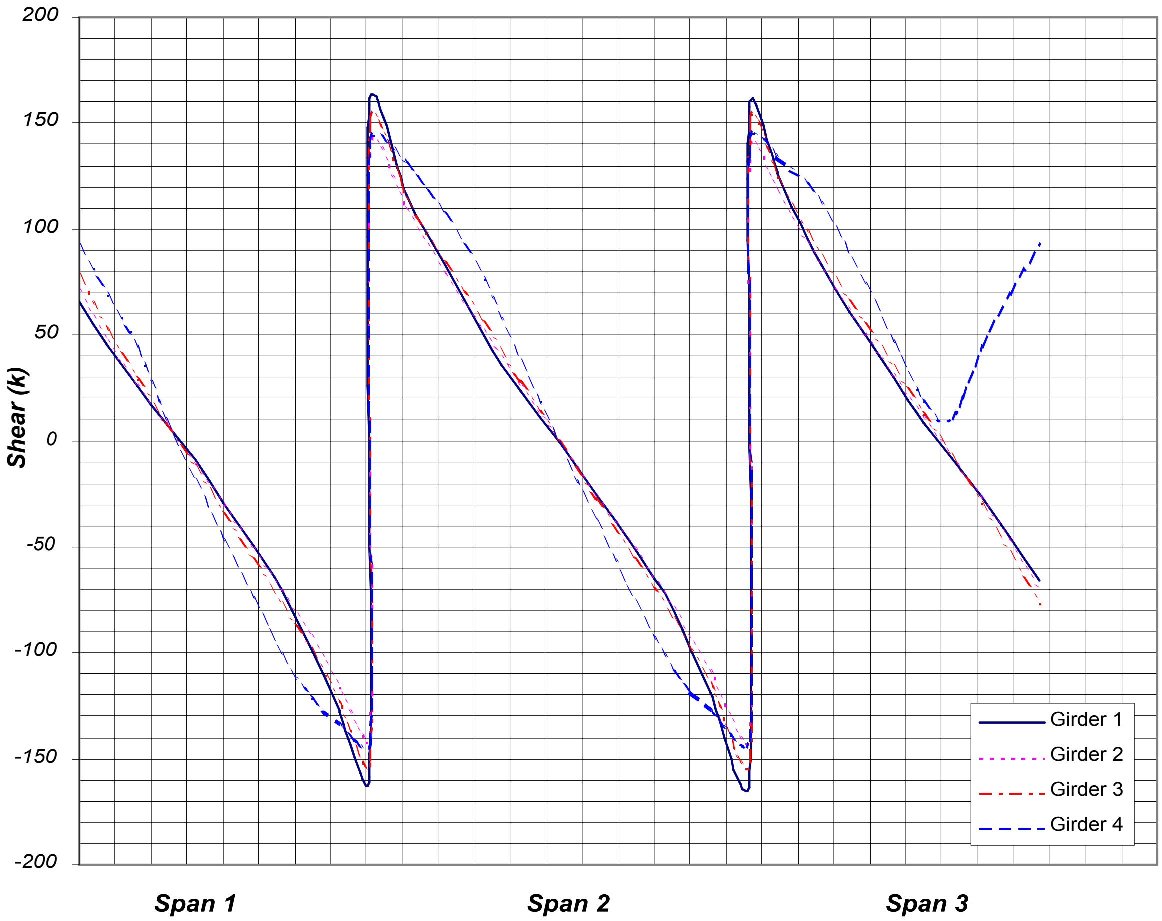
FUTURE WEARING SURFACE DEAD LOAD MOMENT
(GIRDERS)

Figure B-4. Dead Load (Future Wearing Surface) Moment



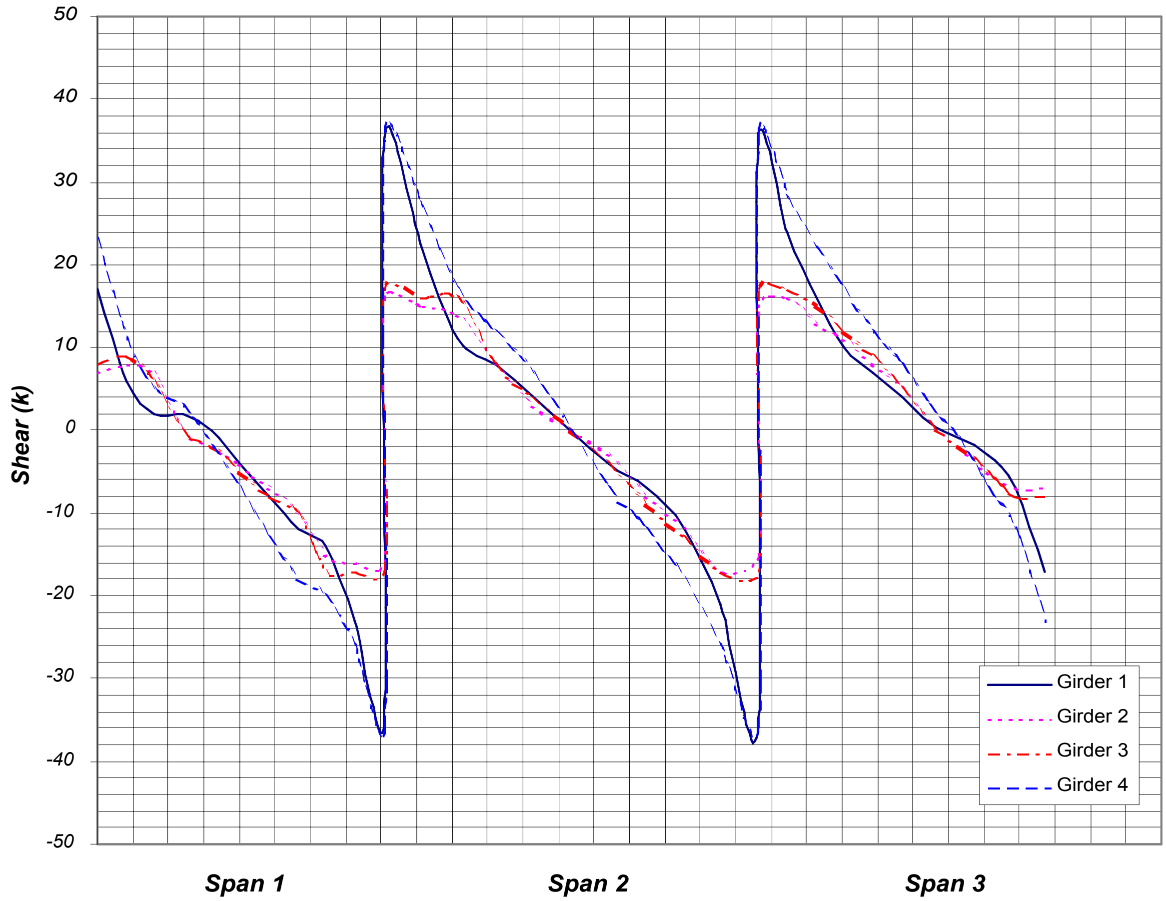
STAGE 1 DEAD LOAD SHEAR (STRUCTURAL STEEL)
(GIRDERS)

Figure B-5. Dead Load (Structural Steel) Shear



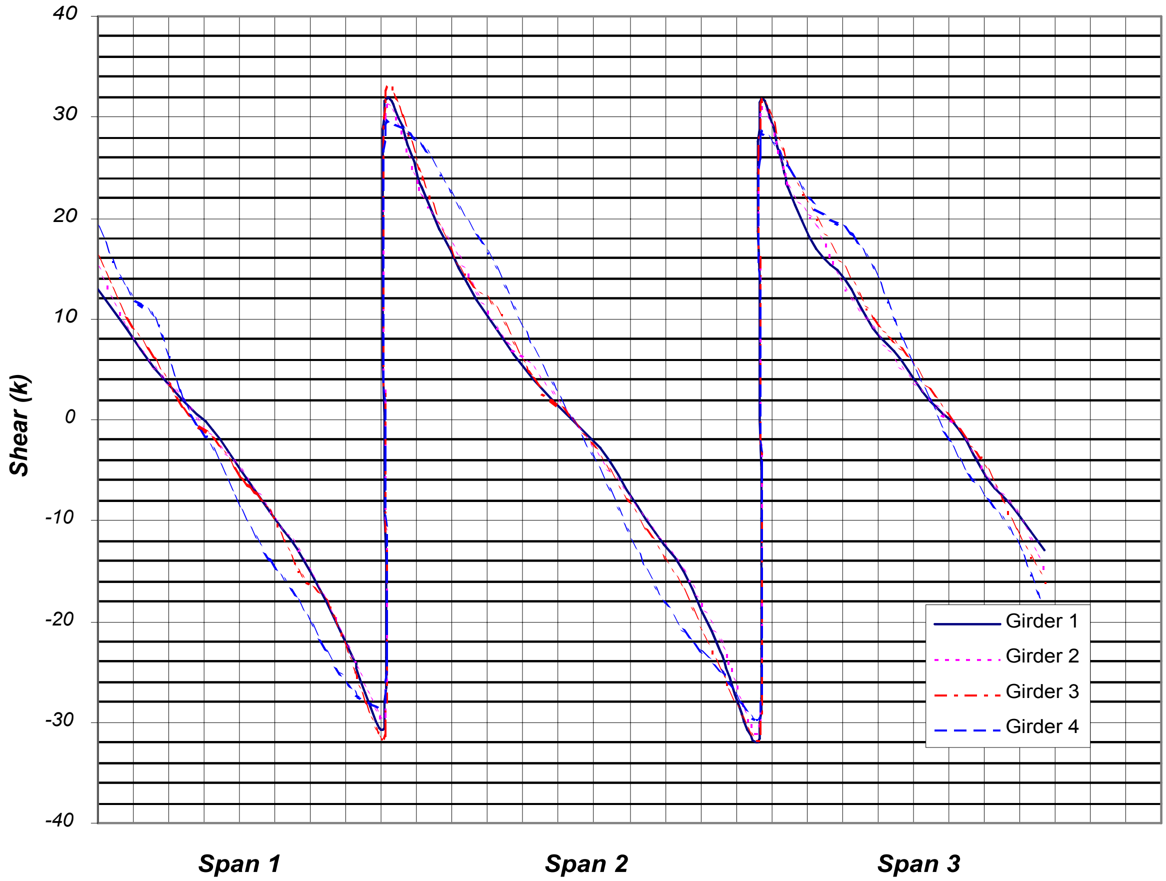
STAGE 6 DEAD LOAD SHEAR (CONCRETE DECK)
(GIRDERS)

Figure B-6. Dead Load (Concrete Deck) Shear



STAGE 7 DEAD LOAD SHEAR (SUPERIMPOSED DEAD LOAD)
(GIRDERS)

Figure B-7. Dead Load (Superimposed Dead Load) Shear



FUTURE WEARING SURFACE DEAD LOAD SHEAR
(GIRDERS)

Figure B-8. Dead Load (Future Wearing Surface) Shear

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Girder -> 1 Span -> 1 Length -> 156.23

D E A D L O A D S

Length	----	MOMENTS	----	----	SHEARS	----
	Stg1	Stg6	Stg7	Stg1	Stg6	Stg7
.00	0	0	0	14	66	30
15.62	178	889	372	9	45	15
31.25	295	1478	599	5	26	7
46.87	351	1767	702	1	9	4
62.49	348	1754	689	-2	-9	-1
78.11	284	1438	573	-5	-29	-9
93.74	156	804	350	-9	-49	-17
109.36	-42	-184	0	-14	-70	-25
124.98	-322	-1553	-503	-20	-98	-32
140.61	-716	-3348	-1183	-28	-127	-47
156.23	-1333	-5897	-2336	-40	-159	-65

Girder -> 1 Span -> 2 Length -> 205.05

D E A D L O A D S

Length	----	MOMENTS	----	----	SHEARS	----
	Stg1	Stg6	Stg7	Stg1	Stg6	Stg7
.00	-1333	-5897	-2336	41	159	66
20.50	-569	-2719	-952	25	116	45
41.01	-123	-648	-172	17	83	26
61.51	157	709	317	10	50	17
82.02	331	1554	640	4	24	8
102.52	384	1812	735	0	0	0
123.03	323	1513	610	-5	-25	-8
143.53	159	717	332	-10	-51	-17
164.04	-131	-688	-190	-16	-80	-27
184.54	-575	-2733	-922	-26	-119	-44
205.05	-1302	-5781	-2254	-41	-160	-67

Girder -> 1 Span -> 3 Length -> 156.23

D E A D L O A D S

Length	MOMENTS Stg1	MOMENTS Stg6	MOMENTS Stg7	SHEARS Stg1	SHEARS Stg6	SHEARS Stg7
.00	-1302	-5781	-2254	40	158	66
15.62	-726	-3371	-1177	28	126	47
31.25	-323	-1555	-514	20	96	33
46.87	-42	-187	-5	14	72	24
62.49	154	797	347	9	50	16
78.11	283	1433	575	6	30	10
93.74	347	1750	688	1	9	3
109.36	350	1761	695	-1	-8	-2
124.98	294	1473	591	-5	-26	-9
140.61	177	881	367	-9	-45	-16
156.23	0	0	0	-14	-66	-30

Girder -> 2 Span -> 1 Length -> 158.74

D E A D L O A D S

Length	MOMENTS Stg1	MOMENTS Stg6	MOMENTS Stg7	SHEARS Stg1	SHEARS Stg6	SHEARS Stg7
.00	0	0	0	16	71	22
15.87	206	962	340	10	47	17
31.75	340	1585	577	6	26	12
47.62	404	1875	704	1	9	2
63.50	397	1840	711	-2	-11	-4
79.37	322	1488	592	-6	-30	-9
95.25	177	820	338	-10	-51	-16
111.12	-38	-182	-40	-15	-71	-23
126.99	-334	-1533	-538	-21	-92	-33
142.87	-733	-3262	-1138	-28	-116	-40
158.74	-1324	-5605	-2003	-37	-139	-45

Girder -> 2 Span -> 2 Length -> 208.35

D E A D L O A D S

	----	MOMENTS	----	----	SHEARS	----
Length	Stg1	Stg6	Stg7	Stg1	Stg6	Stg7
.00	-1324	-5605	-2003	37	139	46
20.83	-597	-2681	-945	24	109	37
41.67	-143	-676	-204	17	78	30
62.50	159	700	318	11	52	17
83.34	355	1600	639	5	26	8
104.17	416	1879	743	0	0	0
125.01	347	1550	637	-6	-26	-8
145.84	162	714	317	-11	-51	-18
166.68	-150	-708	-226	-17	-79	-27
187.51	-602	-2690	-925	-26	-109	-39
208.35	-1297	-5504	-1962	-37	-139	-45

Girder -> 2 Span -> 3 Length -> 158.74

D E A D L O A D S

	----	MOMENTS	----	----	SHEARS	----
Length	Stg1	Stg6	Stg7	Stg1	Stg6	Stg7
.00	-1297	-5504	-1962	37	139	45
15.87	-742	-3274	-1135	28	117	39
31.75	-336	-1539	-543	21	93	32
47.62	-39	-185	-39	15	71	24
63.50	176	816	335	10	50	17
79.37	321	1485	584	7	31	10
95.25	395	1833	706	2	11	3
111.12	403	1865	703	-1	-7	-3
126.99	338	1575	576	-6	-27	-11
142.87	203	950	331	-10	-48	-16
158.74	0	0	0	-16	-71	-22

Girder -> 3 Span -> 1 Length -> 161.26

D E A D L O A D S

Length	MOMENTS Stg1	MOMENTS Stg6	MOMENTS Stg7	SHEARS Stg1	SHEARS Stg6	SHEARS Stg7
.00	0	0	0	18	78	24
16.13	248	1090	389	12	53	19
32.25	406	1775	647	7	29	12
48.38	478	2080	778	1	8	1
64.50	468	2024	777	-3	-12	-4
80.63	379	1622	639	-7	-34	-11
96.75	206	873	352	-12	-56	-17
112.88	-48	-237	-64	-17	-77	-25
129.01	-388	-1708	-618	-23	-98	-35
145.13	-842	-3570	-1270	-31	-123	-42
161.26	-1517	-6112	-2214	-42	-151	-48

Girder -> 3 Span -> 2 Length -> 211.65

D E A D L O A D S

Length	MOMENTS Stg1	MOMENTS Stg6	MOMENTS Stg7	SHEARS Stg1	SHEARS Stg6	SHEARS Stg7
.00	-1517	-6112	-2214	42	150	49
21.16	-694	-2960	-1063	28	114	40
42.33	-183	-803	-251	19	84	31
63.49	164	708	328	13	56	19
84.66	390	1696	684	6	28	8
105.82	461	2006	801	0	0	0
126.99	380	1646	684	-6	-28	-9
148.15	167	721	328	-13	-56	-20
169.32	-191	-832	-274	-19	-84	-30
190.48	-700	-2965	-1038	-29	-115	-42
211.65	-1486	-5999	-2167	-42	-150	-48

Girder -> 3   Span -> 3   Length -> 161.26

D E A D   L O A D S

	---- MOMENTS	----		---- SHEARS	----
Length	Stg1	Stg6	Stg7	Stg1	Stg6	Stg7
.00	-1486	-5999	-2167	42	151	48
16.13	-852	-3586	-1267	31	124	42
32.25	-389	-1714	-623	23	99	35
48.38	-47	-240	-67	17	77	27
64.50	206	870	350	12	55	19
80.63	378	1619	631	8	35	12
96.75	468	2017	772	3	13	4
112.88	476	2065	776	-1	-7	-3
129.01	403	1759	643	-6	-29	-10
145.13	244	1071	376	-12	-53	-19
161.26	0	0	0	-18	-77	-24

Girder -> 4   Span -> 1   Length -> 163.77

D E A D   L O A D S

	---- MOMENTS	----		---- SHEARS	----
Length	Stg1	Stg6	Stg7	Stg1	Stg6	Stg7
.00	0	0	0	23	92	42
16.38	328	1364	575	16	69	24
32.75	558	2305	946	11	44	15
49.13	678	2775	1128	3	10	5
65.51	675	2744	1113	-4	-19	-5
81.89	546	2192	904	-10	-47	-16
98.26	293	1136	510	-18	-74	-27
114.64	-69	-374	-56	-24	-101	-36
131.02	-532	-2263	-786	-30	-121	-43
147.39	-1108	-4482	-1660	-36	-134	-53
163.77	-1917	-7272	-3015	-45	-144	-64

Girder -> 4 Span -> 2 Length -> 214.95

D E A D   L O A D S

	---- MOMENTS	----		---- SHEARS	----	
Length	Stg1	Stg6	Stg7	Stg1	Stg6	Stg7
.00	-1917	-7272	-3015	44	142	65
21.49	-940	-3811	-1388	33	131	54
42.99	-277	-1151	-320	25	107	38
64.48	208	881	471	18	77	27
85.98	531	2224	1011	9	38	14
107.47	635	2658	1183	0	-1	0
128.97	518	2173	978	-9	-38	-14
150.46	210	888	485	-17	-76	-27
171.96	-284	-1174	-340	-26	-109	-39
193.45	-945	-3805	-1337	-33	-127	-51
214.95	-1871	-7126	-2906	-44	-141	-65

Girder -> 4 Span -> 3 Length -> 163.77

This span is missing from the example

LRFD Output

March 8, 2001 1:46 PM

```
Bridge Type --> I - Girder                Date Created -> 09/04/95
Project  -----> I Girder Example          Initials -----> DHH

Project ID ---> DESIGN IG2
          Description --> 3-span 4-girder 700-foot radius

                Number of girders --->  4
                Number of spans   --->  3
                Project units     --->  English
```

Output Clarification

Stg7 = Dead Load due to the weight of the future wearing surface only

Live Load Moments and Shears due to the HL-93 LRFD live load vehicle

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Girder -> 1 Span -> 1 Length -> 156.23

D E A D L O A D S

Length	MOMENTS Stg1	MOMENTS Stg6	MOMENTS Stg7	SHEARS Stg1	SHEARS Stg6	SHEARS Stg7
.00	0	0	0	0	0	17
15.62	0	0	184	0	0	6
31.25	0	0	288	0	0	2
46.87	0	0	327	0	0	2
62.49	0	0	316	0	0	0
78.11	0	0	260	0	0	-4
93.74	0	0	161	0	0	-8
109.36	0	0	6	0	0	-12
124.98	0	0	-229	0	0	-14
140.61	0	0	-564	0	0	-23
156.23	0	0	-1169	0	0	-35

	LIVE LOAD MOMENTS				LIVE LOAD SHEARS			
	Lane		Truck		Lane		Truck	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	0	0	0	0	34	-8
15.62	0	0	397	-87	0	0	25	-4
31.25	0	0	655	-150	0	0	20	-6
46.87	0	0	787	-189	0	0	17	-9
62.49	0	0	827	-218	0	0	14	-12
78.11	0	0	800	-245	0	0	10	-15
93.74	0	0	716	-309	0	0	7	-20
109.36	0	0	556	-384	0	0	6	-25
124.98	0	0	347	-466	0	0	6	-28
140.61	0	0	136	-573	0	0	5	-31
156.23	0	0	139	-743	0	0	3	-36

	LIVE LOAD MOMENTS				LIVE LOAD SHEARS			
	Special		Maximum		Special		Maximum	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	0	0	109	-31	109	-31
15.62	1415	-381	1415	-381	87	-21	87	-21
31.25	2409	-718	2409	-718	69	-27	69	-27
46.87	3003	-1006	3003	-1006	55	-36	55	-36
62.49	3249	-1245	3249	-1245	43	-46	43	-46
78.11	3192	-1448	3192	-1448	34	-58	34	-58
93.74	2875	-1605	2875	-1605	27	-73	27	-73
109.36	2201	-2003	2201	-2003	25	-89	25	-89
124.98	1465	-2569	1465	-2569	22	-106	22	-106
140.61	770	-3305	770	-3305	20	-125	20	-125
156.23	883	-5274	883	-5274	12	-146	12	-146

Girder -> 1 Span -> 2 Length -> 205.05

D E A D   L O A D S

Length	MOMENTS Stg1	MOMENTS Stg6	MOMENTS Stg7	SHEARS Stg1	SHEARS Stg6	SHEARS Stg7
.00	0	0	-1169	0	0	35
20.50	0	0	-447	0	0	22
41.01	0	0	-78	0	0	11
61.51	0	0	141	0	0	8
82.02	0	0	293	0	0	4
102.52	0	0	335	0	0	0
123.03	0	0	272	0	0	-4
143.53	0	0	150	0	0	-7
164.04	0	0	-87	0	0	-12
184.54	0	0	-433	0	0	-21
205.05	0	0	-1124	0	0	-36

	LIVE LOAD MOMENTS				LIVE LOAD SHEARS			
	Lane		Truck		Lane		Truck	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	139	-743	0	0	37	-3
20.50	0	0	174	-468	0	0	29	-5
41.01	0	0	441	-363	0	0	27	-7
61.51	0	0	688	-277	0	0	22	-7
82.02	0	0	814	-247	0	0	18	-9
102.52	0	0	858	-270	0	0	14	-14
123.03	0	0	817	-245	0	0	11	-18
143.53	0	0	693	-278	0	0	8	-22
164.04	0	0	448	-373	0	0	7	-27
184.54	0	0	196	-465	0	0	5	-30
205.05	0	0	135	-717	0	0	3	-38

	LIVE LOAD MOMENTS				LIVE LOAD SHEARS			
	Special		Maximum		Special		Maximum	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	883	-5274	883	-5274	148	-12	148	-12
20.50	842	-2755	842	-2755	124	-24	124	-24
41.01	1694	-1796	1694	-1796	104	-31	104	-31
61.51	2655	-1485	2655	-1485	83	-33	83	-33
82.02	3273	-1481	3273	-1481	66	-37	66	-37
102.52	3498	-1462	3498	-1462	51	-52	51	-52
123.03	3297	-1488	3297	-1488	41	-66	41	-66
143.53	2678	-1528	2678	-1528	33	-81	33	-81
164.04	1705	-1871	1705	-1871	29	-102	29	-102
184.54	906	-2700	906	-2700	25	-121	25	-121
205.05	885	-5113	885	-5113	12	-152	12	-152

Girder -> 1 Span -> 3 Length -> 156.23

D E A D L O A D S

Length	MOMENTS Stg1	MOMENTS Stg6	MOMENTS Stg7	SHEARS Stg1	SHEARS Stg6	SHEARS Stg7
.00	0	0	-1124	0	0	35
15.62	0	0	-560	0	0	24
31.25	0	0	-237	0	0	16
46.87	0	0	0	0	0	10
62.49	0	0	160	0	0	7
78.11	0	0	262	0	0	4
93.74	0	0	315	0	0	1
109.36	0	0	323	0	0	-1
124.98	0	0	282	0	0	-3
140.61	0	0	183	0	0	-7
156.23	0	0	0	0	0	-17

	LIVE LOAD MOMENTS				LIVE LOAD SHEARS			
	Lane		Truck		Lane		Truck	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	135	-717	0	0	39	-3
15.62	0	0	143	-558	0	0	32	-4
31.25	0	0	351	-459	0	0	29	-4
46.87	0	0	558	-379	0	0	25	-6
62.49	0	0	717	-304	0	0	21	-8
78.11	0	0	801	-242	0	0	18	-11
93.74	0	0	830	-213	0	0	13	-14
109.36	0	0	789	-188	0	0	10	-17
124.98	0	0	660	-153	0	0	7	-20
140.61	0	0	407	-84	0	0	6	-25
156.23	0	0	0	0	0	0	7	-34

	LIVE LOAD MOMENTS				LIVE LOAD SHEARS			
	Special		Maximum		Special		Maximum	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	885	-5113	885	-5113	154	-11	154	-11
15.62	776	-3236	776	-3236	121	-18	121	-18
31.25	1464	-2544	1464	-2544	107	-21	107	-21
46.87	2196	-1980	2196	-1980	91	-25	91	-25
62.49	2866	-1567	2866	-1567	75	-30	75	-30
78.11	3186	-1420	3186	-1420	62	-34	62	-34
93.74	3247	-1222	3247	-1222	48	-44	48	-44
109.36	3003	-988	3003	-988	38	-55	38	-55
124.98	2420	-706	2420	-706	31	-69	31	-69
140.61	1436	-376	1436	-376	24	-86	24	-86
156.23	0	0	0	0	29	-108	29	-108

Girder -> 2 Span -> 1 Length -> 158.74

D E A D L O A D S

Length	MOMENTS Stg1	MOMENTS Stg6	MOMENTS Stg7	SHEARS Stg1	SHEARS Stg6	SHEARS Stg7
.00	0	0	0	0	0	7
15.87	0	0	139	0	0	8
31.75	0	0	247	0	0	7
47.62	0	0	312	0	0	0
63.50	0	0	322	0	0	-2
79.37	0	0	271	0	0	-4
95.25	0	0	149	0	0	-7
111.12	0	0	-23	0	0	-10
126.99	0	0	-247	0	0	-15
142.87	0	0	-494	0	0	-16
158.74	0	0	-817	0	0	-16

	LIVE LOAD MOMENTS				LIVE LOAD SHEARS			
	Lane		Truck		Lane		Truck	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	0	0	0	0	31	-2
15.87	0	0	280	-32	0	0	17	-2
31.75	0	0	436	-65	0	0	14	-5
47.62	0	0	511	-99	0	0	11	-7
63.50	0	0	536	-134	0	0	9	-9
79.37	0	0	521	-171	0	0	9	-11
95.25	0	0	473	-210	0	0	7	-14
111.12	0	0	380	-251	0	0	4	-16
126.99	0	0	248	-293	0	0	1	-19
142.87	0	0	111	-341	0	0	0	-23
158.74	0	0	84	-420	0	0	1	-33

	LIVE LOAD MOMENTS				LIVE LOAD SHEARS			
	Special		Maximum		Special		Maximum	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	0	0	109	-12	109	-12
15.87	1210	-185	1210	-185	73	-13	73	-13
31.75	1996	-376	1996	-376	59	-24	59	-24
47.62	2444	-570	2444	-570	49	-33	49	-33
63.50	2632	-772	2632	-772	39	-42	39	-42
79.37	2582	-986	2582	-986	32	-52	32	-52
95.25	2325	-1196	2325	-1196	25	-63	25	-63
111.12	1813	-1635	1813	-1635	17	-75	17	-75
126.99	1203	-2146	1203	-2146	8	-89	8	-89
142.87	605	-2683	605	-2683	1	-108	1	-108
158.74	556	-4053	556	-4053	4	-138	4	-138

Girder -> 2 Span -> 2 Length -> 208.35

D E A D L O A D S

	---- MOMENTS ----			---- SHEARS ----		
Length	Stg1	Stg6	Stg7	Stg1	Stg6	Stg7
.00	0	0	-817	0	0	16
20.83	0	0	-419	0	0	15
41.67	0	0	-95	0	0	14
62.50	0	0	145	0	0	8
83.34	0	0	284	0	0	3
104.17	0	0	333	0	0	0
125.01	0	0	293	0	0	-3
145.84	0	0	139	0	0	-8
166.68	0	0	-106	0	0	-12
187.51	0	0	-412	0	0	-17
208.35	0	0	-811	0	0	-15

	---- LIVE LOAD MOMENTS ----				---- LIVE LOAD SHEARS ----			
	--- Lane ---		-- Truck --		--- Lane ---		-- Truck --	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	84	-420	0	0	33	-1
20.83	0	0	125	-277	0	0	21	-2
41.67	0	0	300	-226	0	0	15	-4
62.50	0	0	443	-181	0	0	15	-6
83.34	0	0	527	-138	0	0	12	-7
104.17	0	0	554	-107	0	0	9	-10
125.01	0	0	527	-139	0	0	8	-12
145.84	0	0	439	-182	0	0	6	-15
166.68	0	0	297	-231	0	0	4	-17
187.51	0	0	134	-273	0	0	3	-19
208.35	0	0	82	-412	0	0	1	-35

	---- LIVE LOAD MOMENTS ----				---- LIVE LOAD SHEARS ----			
	-- Special--		-- Maximum -		-- Special -		-- Maximum -	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	556	-4053	556	-4053	138	-4	138	-4
20.83	652	-2177	652	-2177	101	-9	101	-9
41.67	1351	-1347	1351	-1347	84	-22	84	-22
62.50	2070	-931	2070	-931	70	-27	70	-27
83.34	2505	-760	2505	-760	58	-33	58	-33
104.17	2668	-664	2668	-664	45	-46	45	-46
125.01	2521	-764	2521	-764	34	-56	34	-56
145.84	2060	-927	2060	-927	28	-68	28	-68
166.68	1355	-1375	1355	-1375	19	-84	19	-84
187.51	688	-2142	688	-2142	12	-97	12	-97
208.35	552	-3942	552	-3942	4	-148	4	-148

Girder -> 2 Span -> 3 Length -> 158.74

D E A D L O A D S

	---- MOMENTS ----			---- SHEARS ----		
Length	Stg1	Stg6	Stg7	Stg1	Stg6	Stg7
.00	0	0	-811	0	0	15
15.87	0	0	-495	0	0	16
31.75	0	0	-248	0	0	13
47.62	0	0	-25	0	0	11
63.50	0	0	148	0	0	8
79.37	0	0	264	0	0	5
95.25	0	0	318	0	0	1
111.12	0	0	314	0	0	-2
126.99	0	0	248	0	0	-5
142.87	0	0	135	0	0	-7
158.74	0	0	0	0	0	-7

	---- LIVE LOAD MOMENTS ----				---- LIVE LOAD SHEARS ----			
	--- Lane ---		-- Truck --		--- Lane ---		-- Truck --	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	82	-412	0	0	35	-1
15.87	0	0	123	-335	0	0	23	-1
31.75	0	0	254	-290	0	0	20	-3
47.62	0	0	382	-249	0	0	17	-5
63.50	0	0	475	-209	0	0	15	-7
79.37	0	0	524	-171	0	0	13	-9
95.25	0	0	539	-133	0	0	10	-10
111.12	0	0	514	-98	0	0	8	-12
126.99	0	0	441	-65	0	0	6	-15
142.87	0	0	287	-32	0	0	4	-18
158.74	0	0	0	0	0	0	2	-32

	---- LIVE LOAD MOMENTS ----				---- LIVE LOAD SHEARS ----			
	-- Special--		-- Maximum -		-- Special -		-- Maximum -	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	552	-3942	552	-3942	148	-4	148	-4
15.87	649	-2644	649	-2644	101	-7	101	-7
31.75	1236	-2139	1236	-2139	89	-14	89	-14
47.62	1835	-1640	1835	-1640	77	-21	77	-21
63.50	2344	-1214	2344	-1214	66	-27	66	-27
79.37	2600	-992	2600	-992	56	-34	56	-34
95.25	2650	-775	2650	-775	47	-42	47	-42
111.12	2458	-572	2458	-572	38	-51	38	-51
126.99	2017	-379	2017	-379	29	-60	29	-60
142.87	1240	-189	1240	-189	20	-76	20	-76
158.74	0	0	0	0	12	-111	12	-111

Girder -> 3 Span -> 1 Length -> 161.26

D E A D L O A D S

	---- MOMENTS ----			---- SHEARS ----		
Length	Stg1	Stg6	Stg7	Stg1	Stg6	Stg7
.00	0	0	0	0	0	8
16.13	0	0	163	0	0	9
32.25	0	0	281	0	0	6
48.38	0	0	349	0	0	0
64.50	0	0	355	0	0	-2
80.63	0	0	294	0	0	-5
96.75	0	0	156	0	0	-8
112.88	0	0	-44	0	0	-10
129.01	0	0	-292	0	0	-17
145.13	0	0	-568	0	0	-17
161.26	0	0	-931	0	0	-17

	---- LIVE LOAD MOMENTS ----				---- LIVE LOAD SHEARS ----			
	--- Lane ---		-- Truck --		--- Lane ---		-- Truck --	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	0	0	0	0	30	-3
16.13	0	0	292	-53	0	0	17	-2
32.25	0	0	450	-100	0	0	14	-5
48.38	0	0	525	-146	0	0	11	-7
64.50	0	0	550	-192	0	0	9	-9
80.63	0	0	531	-237	0	0	8	-11
96.75	0	0	479	-286	0	0	6	-14
112.88	0	0	381	-339	0	0	4	-16
129.01	0	0	254	-394	0	0	2	-19
145.13	0	0	127	-456	0	0	0	-23
161.26	0	0	157	-549	0	0	1	-33

	---- LIVE LOAD MOMENTS ----				---- LIVE LOAD SHEARS ----			
	-- Special--		-- Maximum -		-- Special -		-- Maximum -	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	0	0	113	-17	113	-17
16.13	1388	-301	1388	-301	84	-18	84	-18
32.25	2296	-581	2296	-581	64	-28	64	-28
48.38	2814	-845	2814	-845	51	-37	51	-37
64.50	3038	-1105	3038	-1105	41	-45	41	-45
80.63	2993	-1365	2993	-1365	32	-54	32	-54
96.75	2703	-1628	2703	-1628	26	-67	26	-67
112.88	2143	-2126	2143	-2126	19	-81	19	-81
129.01	1435	-2711	1435	-2711	11	-95	11	-95
145.13	727	-3254	727	-3254	3	-114	3	-114
161.26	750	-4594	750	-4594	6	-143	6	-143

Girder -> 3 Span -> 2 Length -> 211.65

D E A D L O A D S

	---- MOMENTS ----			---- SHEARS ----		
Length	Stg1	Stg6	Stg7	Stg1	Stg6	Stg7
.00	0	0	-931	0	0	17
21.16	0	0	-485	0	0	16
42.33	0	0	-122	0	0	16
63.49	0	0	149	0	0	8
84.66	0	0	307	0	0	4
105.82	0	0	362	0	0	0
126.99	0	0	317	0	0	-4
148.15	0	0	145	0	0	-9
169.32	0	0	-134	0	0	-13
190.48	0	0	-476	0	0	-17
211.65	0	0	-923	0	0	-17

	---- LIVE LOAD MOMENTS ----				---- LIVE LOAD SHEARS ----			
	--- Lane ---		-- Truck --		--- Lane ---		-- Truck --	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	157	-549	0	0	33	-1
21.16	0	0	130	-316	0	0	21	-3
42.33	0	0	278	-258	0	0	15	-5
63.49	0	0	406	-204	0	0	15	-5
84.66	0	0	494	-155	0	0	12	-7
105.82	0	0	522	-120	0	0	9	-10
126.99	0	0	493	-157	0	0	8	-12
148.15	0	0	401	-207	0	0	6	-14
169.32	0	0	276	-264	0	0	4	-17
190.48	0	0	138	-314	0	0	3	-20
211.65	0	0	152	-533	0	0	1	-35

	---- LIVE LOAD MOMENTS ----				---- LIVE LOAD SHEARS ----			
	-- Special--		-- Maximum -		-- Special -		-- Maximum -	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	750	-4594	750	-4594	143	-6	143	-6
21.16	699	-2517	699	-2517	109	-14	109	-14
42.33	1454	-1560	1454	-1560	90	-24	90	-24
63.49	2255	-1160	2255	-1160	75	-27	75	-27
84.66	2837	-1015	2837	-1015	60	-34	60	-34
105.82	3026	-914	3026	-914	46	-47	46	-47
126.99	2851	-1020	2851	-1020	36	-60	36	-60
148.15	2259	-1165	2259	-1165	28	-73	28	-73
169.32	1459	-1591	1459	-1591	20	-90	20	-90
190.48	727	-2461	727	-2461	16	-103	16	-103
211.65	733	-4458	733	-4458	6	-153	6	-153

Girder -> 3 Span -> 3 Length -> 161.26

D E A D   L O A D S

	---- MOMENTS ----			---- SHEARS ----		
Length	Stg1	Stg6	Stg7	Stg1	Stg6	Stg7
.00	0	0	-923	0	0	17
16.13	0	0	-569	0	0	17
32.25	0	0	-293	0	0	15
48.38	0	0	-40	0	0	12
64.50	0	0	155	0	0	9
80.63	0	0	287	0	0	5
96.75	0	0	352	0	0	1
112.88	0	0	350	0	0	-2
129.01	0	0	281	0	0	-5
145.13	0	0	156	0	0	-8
161.26	0	0	0	0	0	-8

	---- LIVE LOAD MOMENTS ----				---- LIVE LOAD SHEARS ----			
	--- Lane ---		-- Truck --		--- Lane ---		-- Truck --	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	152	-533	0	0	35	-1
16.13	0	0	137	-446	0	0	23	-2
32.25	0	0	259	-389	0	0	20	-4
48.38	0	0	383	-336	0	0	17	-5
64.50	0	0	481	-283	0	0	15	-7
80.63	0	0	533	-235	0	0	13	-9
96.75	0	0	551	-190	0	0	10	-10
112.88	0	0	523	-144	0	0	8	-12
129.01	0	0	449	-99	0	0	6	-14
145.13	0	0	296	-51	0	0	4	-18
161.26	0	0	0	0	0	0	3	-31

	---- LIVE LOAD MOMENTS ----				---- LIVE LOAD SHEARS ----			
	-- Special--		-- Maximum -		-- Special -		-- Maximum -	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	733	-4458	733	-4458	153	-6	153	-6
16.13	747	-3200	747	-3200	108	-6	108	-6
32.25	1450	-2685	1450	-2685	95	-15	95	-15
48.38	2153	-2120	2153	-2120	83	-22	83	-22
64.50	2711	-1623	2711	-1623	69	-28	69	-28
80.63	3002	-1360	3002	-1360	57	-35	57	-35
96.75	3044	-1100	3044	-1100	48	-42	48	-42
112.88	2811	-837	2811	-837	39	-52	39	-52
129.01	2299	-572	2299	-572	30	-65	30	-65
145.13	1408	-298	1408	-298	23	-84	23	-84
161.26	0	0	0	0	17	-112	17	-112

Girder -> 4 Span -> 1 Length -> 163.77

D E A D L O A D S

	---- MOMENTS ----			---- SHEARS ----		
Length	Stg1	Stg6	Stg7	Stg1	Stg6	Stg7
.00	0	0	0	0	0	23
16.38	0	0	287	0	0	11
32.75	0	0	463	0	0	5
49.13	0	0	542	0	0	3
65.51	0	0	527	0	0	-2
81.89	0	0	425	0	0	-7
98.26	0	0	241	0	0	-13
114.64	0	0	-24	0	0	-18
131.02	0	0	-375	0	0	-20
147.39	0	0	-814	0	0	-26
163.77	0	0	-1537	0	0	-36

	---- LIVE LOAD MOMENTS ----				---- LIVE LOAD SHEARS ----			
	--- Lane ---		-- Truck --		--- Lane ---		-- Truck --	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	0	0	0	0	40	-8
16.38	0	0	521	-107	0	0	31	-7
32.75	0	0	894	-217	0	0	27	-6
49.13	0	0	1123	-327	0	0	22	-8
65.51	0	0	1208	-435	0	0	16	-14
81.89	0	0	1170	-536	0	0	12	-19
98.26	0	0	1047	-632	0	0	7	-25
114.64	0	0	804	-718	0	0	3	-30
131.02	0	0	493	-795	0	0	2	-34
147.39	0	0	187	-871	0	0	2	-37
163.77	0	0	263	-986	0	0	2	-41

	---- LIVE LOAD MOMENTS ----				---- LIVE LOAD SHEARS ----			
	-- Special--		-- Maximum -		-- Special -		-- Maximum -	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	0	0	143	-37	143	-37
16.38	2009	-529	2009	-529	119	-33	119	-33
32.75	3570	-1059	3570	-1059	99	-33	99	-33
49.13	4636	-1582	4636	-1582	79	-42	79	-42
65.51	5134	-2076	5134	-2076	58	-58	58	-58
81.89	5084	-2546	5084	-2546	40	-77	40	-77
98.26	4575	-2966	4575	-2966	25	-96	25	-96
114.64	3498	-3745	3498	-3745	17	-114	17	-114
131.02	2286	-4502	2286	-4502	14	-132	14	-132
147.39	1135	-5092	1135	-5092	13	-148	13	-148
163.77	1368	-6726	1368	-6726	9	-159	9	-159

Girder -> 4 Span -> 2 Length -> 214.95

D E A D   L O A D S

Length	MOMENTS Stg1	MOMENTS Stg6	MOMENTS Stg7	SHEARS Stg1	SHEARS Stg6	SHEARS Stg7
.00	0	0	-1537	0	0	36
21.49	0	0	-675	0	0	27
42.99	0	0	-155	0	0	17
64.48	0	0	214	0	0	12
85.98	0	0	474	0	0	7
107.47	0	0	554	0	0	0
128.97	0	0	452	0	0	-7
150.46	0	0	225	0	0	-12
171.96	0	0	-163	0	0	-18
193.45	0	0	-648	0	0	-26
214.95	0	0	-1474	0	0	-36

	LIVE LOAD MOMENTS				LIVE LOAD SHEARS			
	Lane		Truck		Lane		Truck	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	263	-986	0	0	41	-2
21.49	0	0	210	-639	0	0	35	-4
42.99	0	0	562	-553	0	0	34	-5
64.48	0	0	905	-465	0	0	28	-6
85.98	0	0	1113	-371	0	0	23	-10
107.47	0	0	1184	-296	0	0	17	-17
128.97	0	0	1115	-375	0	0	11	-23
150.46	0	0	919	-473	0	0	7	-27
171.96	0	0	575	-569	0	0	5	-33
193.45	0	0	238	-633	0	0	4	-36
214.95	0	0	252	-944	0	0	2	-42

	LIVE LOAD MOMENTS				LIVE LOAD SHEARS			
	Special		Maximum		Special		Maximum	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	1368	-6726	1368	-6726	159	-9	159	-9
21.49	1078	-3926	1078	-3926	150	-24	150	-24
42.99	2307	-2610	2307	-2610	137	-26	137	-26
64.48	3687	-2110	3687	-2110	114	-30	114	-30
85.98	4842	-1924	4842	-1924	90	-41	90	-41
107.47	5192	-1768	5192	-1768	65	-65	65	-65
128.97	4832	-1940	4832	-1940	45	-88	45	-88
150.46	3765	-2147	3765	-2147	35	-110	35	-110
171.96	2337	-2677	2337	-2677	27	-132	27	-132
193.45	1130	-3812	1130	-3812	24	-146	24	-146
214.95	1309	-6519	1309	-6519	7	-159	7	-159

Girder -> 4 Span -> 3 Length -> 163.77

D E A D L O A D S

Length	MOMENTS Stg1	MOMENTS Stg6	MOMENTS Stg7	SHEARS Stg1	SHEARS Stg6	SHEARS Stg7
.00	0	0	-1474	0	0	36
16.38	0	0	-806	0	0	28
32.75	0	0	-381	0	0	22
49.13	0	0	-24	0	0	17
65.51	0	0	243	0	0	12
81.89	0	0	429	0	0	8
98.26	0	0	529	0	0	3
114.64	0	0	540	0	0	-1
131.02	0	0	460	0	0	-7
147.39	0	0	286	0	0	-12
163.77	0	0	0	0	0	-23

	LIVE LOAD MOMENTS				LIVE LOAD SHEARS			
	Lane		Truck		Lane		Truck	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	252	-944	0	0	45	-2
16.38	0	0	203	-843	0	0	37	-2
32.75	0	0	499	-774	0	0	35	-2
49.13	0	0	802	-703	0	0	31	-4
65.51	0	0	1045	-620	0	0	26	-7
81.89	0	0	1170	-527	0	0	22	-12
98.26	0	0	1209	-427	0	0	16	-16
114.64	0	0	1127	-321	0	0	10	-22
131.02	0	0	907	-214	0	0	6	-27
147.39	0	0	537	-108	0	0	6	-32
163.77	0	0	0	0	0	0	7	-40

	LIVE LOAD MOMENTS				LIVE LOAD SHEARS			
	Special		Maximum		Special		Maximum	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	1309	-6519	1309	-6519	169	-7	169	-7
16.38	1140	-4897	1140	-4897	140	-15	140	-15
32.75	2272	-4379	2272	-4379	130	-15	130	-15
49.13	3470	-3676	3470	-3676	116	-17	116	-17
65.51	4553	-2915	4553	-2915	98	-26	98	-26
81.89	5070	-2505	5070	-2505	81	-40	81	-40
98.26	5127	-2044	5127	-2044	59	-57	59	-57
114.64	4643	-1557	4643	-1557	45	-78	45	-78
131.02	3607	-1051	3607	-1051	36	-98	36	-98
147.39	2054	-531	2054	-531	30	-117	30	-117
163.77	0	0	0	0	36	-142	36	-142

APPENDIX C

Selected Design Forces and Girder 4 Section Properties

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Table C-1. Girder 4 Selected Moments (k-ft)

Section Node	Steel	Deck	Cast(#)[1]	SupImp[2]	FWS[3]	LLmax[4]	Fat_{min}[5]	Fat_{max}[5]
1-1 24	574	2,367	3,018(1)	470	498	3,661	-226	914
2-2 44	661	2,682	3,932(1) -3,035(2)	510	583	5,125	-452 V = -15k	1,202 V = 15k
3-3 64	213	802	2,554(1) -3,113(2)	175	206	4,338 -3,080	-654 V = -27k	986 V = 6k
4-4 76	-290	-1,282	1,023(1) -3,469(2)	-200	-212	2,906 -4,243	-757	649
5-5 88	-958	-3,921	-549(1) -3,685(2)	-704	-753	1,341 -4,855	-852	263
6-6 100	-1,917	-7,272	---	-1,537	-1,478	-6,726	-986 V = -41k	263 V = 2k
7-7 112	-1,010	-4,082	---	-753	-748	987 -4,103	-670	215
8-8 124 Splice	-382	-1,585	-1,910(1) -169(2)	-250	-237	2,054 -2,772	-569	498
10-10 148	634	2,652	-1,045(1) 4,089(2)	542	637	5,138	-307	1,173

[1](#) denotes Deck Cast

Cast #1 begins at Section 1-1 and ends at Section 3-3

Cast #2 begins at Section 8-8 and is symmetrical in the center span

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 dynamic load allowance moment due to multiple lanes of HL-93. Dynamic load allowance (DLA) is according to Article 3.6.2.1.

[5]Fat - Maximum and minimum fatigue moments due to one fatigue vehicle plus 15% (DLA) times the load factor of 0.75 specified in Article 3.6.1.4.

All live load moments, including fatigue moments, include centrifugal force effects (refers to Centrifugal Force Calculations in Appendix D - page D-100. Multiple presence reduction factors (Table 3.6.1.2-1) 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 4 Span 1

Tenth Point	Steel	Deck	SupImp	FWS	Tot DL	LL+IM	Factored Shear
0	23	92	19	23	157	143	452
1	16	69	13	11	109	119	347
2	11	44	10	5	70	99	262
3	3	10	2	3	18	79	162
4	-4	-19	-3	-2	-28	-58	-137
5	-10	-47	-9	-7	-73	-77	-228
6	-18	-74	-14	-13	-119	-96	-320
7	-24	-101	-18	-18	-161	-114	-405
8	-30	-121	-23	-20	-194	-132	-479
9	-36	-134	-27	-26	-223	-148	-544
10	-45	-144	-28	-36	-253	-159	-604

Appropriate dynamic load allowance (1.33) is included in live load shears.
#1 and #2 under Deck denote casts (see Table C-1).
Factored Shear = 1.25 x (Steel + Deck + SupImp) + 1.5 x (FWS) + 1.75 x (LL+IM)

Table C-3. Shear (kips), Girder 4 Span 2

Tenth Point	Steel	Deck	SupImp	FWS	Tot DL	LL+IM	Factored Shear
0	44	142	36	29	251	159	599
1	33	131	27	27	218	150	542
Sect 8-8 Node 124 (Splice)	27	112 7(#1) 92(#2)	19	22	180	139	472
2	25	107	17	21	170	137	458
3	18	77	12	15	122	114	356
4	9	38	7	7	61	90	236
5	0	-1	0	0	-1	-65	-115
6	-9	-38	-7	-7	-61	-88	-232

Appropriate dynamic load allowance (1.33) is included in live load shears.
#1 and #2 under Deck denote casts (see Table C-1).
Factored Shear = 1.25 x (Steel + Deck + SupImp) + 1.5 x (FWS) + 1.75 x (LL+IM)

Table C-4. Selected Girder 4 Section Properties--Transversely Stiffened Web

Point Node	Section Size	Section Type	Moment of Inertia	Neutral Axis B	Neutral Axis T	D_c
1 24	20 x 1.0 84 x 0.5625 21 x 1.0 A = 88.25 in^2	Noncomp	101,818	42.52	43.48	42.48
		Comp DL	181,371	60.28	25.72	24.72
		Comp DL Bars				
		Comp LL	242,459	73.79	12.21	11.21
		Comp LL Bars				
2 44 3 64 4 76	20 x 1.0 84 x 0.5625 21 x 1.5 A = 98.75 in^2	Noncomp	118,978	38.47	48.03	47.03
		Comp DL	217,079	56.44	30.06	29.06
		Comp DL Bars	126,842	39.92	46.58	38.42
		Comp LL	297,525	71.08	15.42	14.42
		Comp LL Bars	141,391	42.60	43.90	41.10
5 88	28 x 1.25 84 x 0.625 27 x 1.5 A = 128.0 in^2	Noncomp	168,029	41.63	45.12	40.13
		Comp DL				
		Comp DL Bars	175,058	42.69	44.06	41.19
		Comp LL				
		Comp LL Bars	188,290	44.68	42.07	43.18

Legend: B = Vertical distance from the N. A. to the outermost edge of the bottom flange
T = Vertical distance from the N. A. to the outermost edge of the top flange
D_c = depth of the web in compression; where two values are shown, the top value is for positive moment and the bottom value is for negative moment.
A = total steel area
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
Lstiff = distance of the longitudinal stiffener from the bottom of the top flange

Composite section properties, including the concrete deck, are computed using the structural deck area including the overhang and half the deck width between girders. The area of the deck haunch is not included. The haunch depth is considered. For composite section properties including only the longitudinal reinforcing steel area equal to 8.0 square inches is assumed placed 4.0 inches from the bottom of the deck.

The modular ration, 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 superimposed dead load is 2.67 in^2 (8.0/3).

Table C-4. Girder 4 Section Properties (continued)--Transversely Stiffened Web

Point Node	Section Size	Section Type	Moment of Inertia	Neutral Axis B	Neutral Axis T	D_c
6 100	28 x 2.5 84 x 0.625 27 x 3.0 A = 203.5 in^2	Noncomp	313,872	42.56	46.94	39.56
		Comp DL	420,273	52.49	37.01	49.49
		Comp DL Bars	321,111	43.24	46.26	40.24
		Comp LL	545,757	64.17	25.33	61.17
		Comp LL Bars	335,040	44.55	44.95	41.55
7 112	28 x 1.25 84 x 0.625 27 x 1.5 A = 128.0 in^2	Noncomp	168,029	41.63	45.12	40.13
		Comp DL				
		Comp DL Bars	175,058	42.69	44.06	41.19
		Comp LL				
		Comp LL Bars	188,290	44.68	42.07	43.18
8 124	17 x 1.0 84 x 0.5625 21 x 1.5 A = 95.75 in^2	Noncomp	111,989	36.98	49.52	48.52 35.48
		Comp DL	213,901	55.73	30.77	29.77 54.23
		Comp DL Bars	120,277	38.51	47.99	37.01
		Comp LL	296,306	70.80	15.70	14.70 69.30
		Comp LL Bars	135,575	41.34	45.16	39.84

Table C-4. Girder 4 Section Properties (continued)--Longitudinally Stiffened Web

Point Node	Section Size	Section Type	Moment of Inertia	Neutral Axis B	Neutral Axis T	D_c
2 44	21 x 1.0 84 x 0.4375 22 x 1.5 A = 90.75 in^2 d_s = 18”	Noncomp	116,890	37.79	48.71	47.71
		Comp DL	214,623	57.03	29.47	28.47
		Comp DL Bars				
		Comp LL	290,835	71.93	14.57	13.57
		Comp LL Bars				
3 64	21 x 1.0 84 x 0.4375 22 x 1.5 A = 90.75 in^2 d_s = 42”	Noncomp	116,890	37.79	48.71	47.71 36.29
		Comp DL	214,623	57.03	29.47	28.47
		Comp DL Bars	124,931	39.37	47.12	37.88
		Comp LL	290,835	71.93	14.57	13.57 70.43
		Comp LL Bars	139,710	42.30	44.20	40.80
4 76	21 x 1.0 84 x 0.4375 22 x 1.5 A = 90.75 in^2 d_s = 56”	Noncomp	116,890	37.79	48.71	47.71 36.29
		Comp DL	214,623	57.03	29.47	55.53
		Comp DL Bars	124,931	39.37	47.12	37.88
		Comp LL	290,835	71.93	14.57	13.57 70.43
		Comp LL Bars	139,710	42.30	44.20	40.80
5 88	29 x 1.25 84 x 0.4375 28 x 1.5 A = 115.0 in^2 d_s = 66”	Noncomp	163,684	41.32	45.43	39.82
		Comp DL				
		Comp DL Bars	170,780	42.50	44.24	41.00
		Comp LL				
		Comp LL Bars	184,047	44.72	42.03	43.22
6 100	29 x 2.5 84 x 0.4375 28 x 3.0 A =193.25 in^2 d_s = 66”	Noncomp	314,783	42.32	47.18	39.32
		Comp DL	421,112	52.72	36.78	49.72
		Comp DL Bars	322,084	43.03	46.47	40.03
		Comp LL	543,850	64.69	24.81	61.69
		Comp LL Bars	336,106	44.41	45.09	41.41

APPENDIX D

Sample Calculations

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Girder Stress Check Section 1-1 G4 Node 44
Transversely Stiffened Web - Section Proportioning

The web and the flanges must be proportioned according to the provisions of Article 6.10.2.

Web proportions per Article 6.10.2.1:

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

$$\frac{D}{t_w} \le 150 \qquad \text{Eq (6.10.2.1.1-1)}$$

$$\frac{84}{0.5625} = 149.3 \quad < \ 150 \quad \textbf{OK}$$

Flange proportions per Article 6.10.2.2:

Compression and tension flanges are proportioned such that:

Top flange (compression flange): 20 in. x 1.0 in.
Bottom flange (tension flange): 21 in. x 1.5 in.

$$\frac{b_f}{2t_f} \le 12.0 \qquad \text{Eq (6.10.2.2-1)}$$

Top fl.: $\frac{20}{2(1)} = 10$ Bottom fl.: $\frac{21}{2(1.5)} = 7$ Both flanges **OK**

$$b_f \ge \frac{D}{6} \qquad \text{Eq (6.10.2.2-2)}$$

$\frac{84}{6} = 14$ in. Both flanges **OK**

$$t_f \ge 1.1t_w \qquad \text{Eq (6.10.2.2-3)}$$

$1.1(0.5625) = 0.619$ Both flanges **OK**

$$0.1 \le \frac{I_{yc}}{I_{yt}} \le 10 \qquad \text{Eq (6.10.2.2-4)}$$

$I_{yc} = \frac{1(20)^3}{12} = 667$ $I_{yt} = \frac{1.5(21)^3}{12} = 1158$

$\frac{667}{1158} = 0.576$ **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 2-2 G4 Node 44
Transversely Stiffened Web - Constructibility - Top Flange

The girder must be checked for steel weight and for Cast #1 of the concrete deck on the noncomposite section according to the provisions of Article 6.10.3. The factored steel stresses during the sequential placement of the concrete are not to exceed the nominal resistances specified in Article 6.10.3.2.1 for compression and Article 6.10.3.2.2 for tension flanges. The effect of the overhang brackets on the flanges must also be considered according to Article C6.10.3.4 since G4 is an exterior girder.

Overhang Bracket Load

Since G4 is an exterior 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 about 3 feet along the girder.

The unbraced length of the top flange is 20 feet. 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.

$$\text{Deck} = \frac{1}{2} \times 3.75 \times 10 \times \frac{1}{12} 150 = 234 \ \text{lbs/ft}$$

Deck forms + Screed rail = 240 lbs/ft
Uniform load on brackets = 474 lbs/ft

Compute the lateral force on the flange due to the overhang brackets.

α = arctan(3.75 ft/7.00 ft) = 28 degrees

$$F_l = \frac{474 \tan\left[28\left(\frac{2\pi}{360}\right)\right]}{1000} = 0.252 \quad \text{k/ft}$$

Compute the lateral flange moment due to the overhang forces in accordance with Article C6.10.3.4. The lateral flange moment at the brace points due to the overhang forces is negative in the top flange of G4 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 at the brace points. The flange is treated as a continuous beam supported at brace points.

$$M_l = F_l L_b^2 / 12 \qquad \text{Eq (C6.10.3.4-3)}$$

$$= -\left[\frac{0.252\,(20)^2}{12}\right] = -8.4 \quad \text{k-ft (unfactored)}$$

Girder Stress Check Section 2-2 G4 Node 44
Transversely Stiffened Web - Constructibility - Top Flange (continued)

From Table C-1, the moment due to the steel weight plus Cast #1 is 661 + 3,932 = 4,593 k-ft. The load factor for constructibility checks is 1.25 according to the provisions of Article 3.4.2. It has been assumed that there is no equipment on the bridge during construction and the wind load on the girders is negligable. Using the section properties for the transversely stiffened web design from Table C-4, the flange stress calculated without consideration of the lateral bending, f_{bu}, in the top flange is computed as:

$$f_{top\ flg} = \quad f_{bu} = -\frac{4593(12)(48.03)}{118978}(1.25) = -27.81 \ \text{ksi} \ \ (C)$$

Eq (C4.6.1.2.4b-1) assumes the presence of a cross-frame at the point under investigation and that the major-axis moment, M, is constant over the distance between brace points. Although the use of Eq (C4.6.1.2.4b-1) is not theoretically pure at locations between brace points, it can conservatively be used. Note that throughout this example, the web depth is conservatively used for the term "D" in this equation.

$$M_{lat} = \frac{M l^2}{NRD} \qquad \text{Eq (C4.6.1.2.4b-1)}$$

$$= \frac{-4593\,(20)^2}{10(717)(7)} = -36.6 \quad \text{k-ft}$$

Although the lateral flange bending stresses are always additive to the major-axis bending stresses, it is helpful to understand the correct lateral flange moment sign when checking analysis results. The lateral flange moment at the brace points due to curvature is negative in the top flange of all four girders whenever the top flange is 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 flange is subjected to tension.

$$M_{tot\_lat} = [-36.6 + (-8.4)] \times 1.25 = -56.3 \quad \text{k-ft (factored)}$$

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.

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

where:

$$L_p = \quad 1.0 r_t \sqrt{\frac{E}{F_{yc}}} \quad = \quad \frac{1.0(4.81)\sqrt{\frac{29000}{50}}}{12} = 9.65 \quad \text{ft.} \qquad \text{Eq (6.10.8.2.3-4)}$$

where:

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1 + \frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\right)}} = \frac{20}{\sqrt{12\left[1 + \frac{1}{3}\frac{47.03(0.5625)}{20(1)}\right]}} = 4.81 \ \text{in.} \qquad \text{Eq (6.10.8.2.3-9)}$$

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):

$$L_b = 20 \text{ ft.} > \quad 1.2(9.65)\sqrt{\frac{1.0(1.0)}{\frac{27.81}{50}}} = 15.5 \quad \text{ft.}$$

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

$$S_{top\_flange} = \frac{1.0\,(20)^2}{6} = 66.7 \quad in^3$$

$$f_{l1} = \frac{M_{tot\_lat}}{S_{top\_flange}} = \frac{-56.3(12)}{66.7} = -10.13 \quad \text{ksi (factored)}$$

For critical stages of construction, each of the following requirements will be satisfied for discretely braced compression flanges. If the section has a slender web, Eq -1 below is not checked when f_l is zero. For sections with compact or noncompact webs, Eq -3 is not checked.

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

where:

$$f_l = \left(\frac{0.85}{1 - \frac{f_{bu}}{F_{cr}}}\right) f_{l1} \ge f_{l1} \qquad \text{(second-order analysis)} \qquad \text{Eq (6.10.1.6-4)}$$

where:

F_{cr} = elastic lateral torsional buckling stress for the flange under consideration determined using Eq (6.10.8.2.3-8)

$$= \frac{C_b R_b \pi^2 E}{\left(\frac{L_b}{r_t}\right)^2} = \frac{1.0(1.0)\left(\pi^2\right)(29000)}{\left[\frac{20(12)}{4.81}\right]^2} = 115 \quad \text{ksi} \qquad \text{Eq (6.10.8.2.3-8)}$$

Girder Stress Check Section 2-2 G4 Node 44
Transversely Stiffened Web - Constructibility - Top Flange (continued)

$$= \left(\frac{0.85}{1 - \frac{27.81}{115}} \right) (10.13) = 11.36 \quad \text{ksi}$$

ϕ_f = 1.0 (Article 6.5.4.2)

R_h = 1.0 (Article 6.10.1.10.1)

Therefore,

27.81 + 11.36 = 39.17 ksi < 1.0(1.0)(50) = 50 ksi **OK**

Secondly, check the following relation:

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

The nominal flexural resistance of the compression flange, F_{nc}, is taken as the smaller of the local buckling resistance (Article 6.10.8.2.2) and the lateral torsional buckling resistance (Article 6.10.8.2.3).

Determine the local buckling resistance of the compression flange.

Check flange slenderness.

$$\lambda_f = \frac{b_{fc}}{2t_{fc}} = \frac{20}{2(1)} = 10 \qquad \text{Eq (6.10.8.2.2-3)}$$

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

$$\lambda_{rf} = 0.56 \sqrt{\frac{E}{F_{yr}}} = 0.56 \sqrt{\frac{29000}{0.7(50)}} = 16.12 \qquad \text{(noncompact flange)} \qquad \text{Eq (6.10.8.2.2-5)}$$

Since $\lambda_{pf} < \lambda_f < \lambda_{rf}$, the nominal flexural resistance is determined using Eq (6.10.8.2.2-2).

Girder Stress Check Section 2-2 G4 Node 44
Transversely Stiffened Web - Constructibility - Top Flange (continued)

R_b is taken as 1.0 for constructibility checks per Article 6.10.3.2.1 and R_h is taken as 1.0 per Article 6.10.1.10.1. Therefore, F_{nc} for the local buckling resistance is calculated as:

$$F_{nc} = \left[1 - \left(1 - \frac{F_{yr}}{R_h F_{yc}}\right)\left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] R_b R_h F_{yc} \qquad \text{Eq (6.10.8.2.2-2)}$$

$$= \left[1 - \left[1 - \frac{0.7(50)}{1.0(50)}\right]\left(\frac{10 - 9.15}{16.12 - 9.15}\right)\right](1.0)(1.0)(50) = 48.17 \text{ ksi}$$

Determine the lateral torsional buckling resistance of the compression flange.

L_p = 9.65 ft. (calculated previously)

$$L_r = \pi r_t \sqrt{\frac{E}{F_{yr}}} = \frac{\pi(4.81)\sqrt{\frac{29000}{0.7(50)}}}{12} = 36.2 \text{ ft.} \qquad \text{Eq (6.10.8.2.3-5)}$$

Since $L_p < L_b < L_r$, use Eq (6.10.8.2.3-2) to calculate the lateral torsional buckling resistance.

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

$$= 1.0\left[1 - \left[1 - \frac{0.7(50)}{1.0(50)}\right]\left(\frac{20 - 9.65}{36.2 - 9.65}\right)\right](1.0)(1.0)(50) = 44.15 \text{ ksi} \qquad \text{<- controls}$$

Therefore,

27.81 + (1/3)(11.36) = 31.60 ksi   <   1.0(44.15) = 44.15 ksi   **OK**

Determine if the web is slender according to Article 6.10.6.2.3 for noncomposite sections.

$$\frac{2D_c}{t_w} \le 5.7\sqrt{\frac{E}{F_{yc}}} \qquad \text{Eq (6.10.6.2.3-1)}$$

$$\frac{2(47.03)}{0.5625} = 167.2 \quad > \quad 5.7\sqrt{\frac{29000}{50}} = 137.3$$ slender web, noncompact section

Girder Stress Check Section 2-2 G4 Node 44
Transversely Stiffened Web - Constructibility - Top Flange (continued)

Since, the web is slender, Eq (6.10.3.2.1-3) is checked to prevent web bend-buckling from occurring during construction.

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

where the nominal web bend buckling, F_{crw}, is taken as:

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

but cannot exceed R_hF_{yc} and $F_{yw}/0.7$ per Article 6.10.1.9.1 for webs without longitudinal stiffeners Eq (6.10.1.9.1-1)

where:

$$k = \frac{9}{\left(\frac{D_c}{D}\right)^2} = \frac{9}{\left(\frac{47.03}{84}\right)^2} = 28.7$$

(D_c taken from Table C-4) Eq (6.10.1.9.1-2)

$$F_{crw} = \frac{0.9(29000)(28.7)}{\left(\frac{84}{0.5625}\right)^2} = 33.6 \text{ ksi} < 1.0(50.0) = 50.0 \text{ ksi} < \frac{50}{0.7} = 71.4 \text{ ksi}$$

Therefore, use F_{crw} = 33.6 ksi

27.81 ksi < 1.0(33.6) = 33.6 ksi   **OK**

The compression flange proportions satisfy the criteria given in Article 6.10.3.2.1.

Since economical composite girders normally have smaller top flanges than bottom flanges, the relation given by Eq (6.10.3.4-1) should be satisfied to eliminate potential problems during construction such as out-of-plane distortions of the compression flanges and web. L is taken as the length of the shipping piece, say 100 ft.

$$b_{fc} \ge \frac{L}{85} \qquad 20 \text{ in.} > \frac{100(12)}{85} = 14.1 \text{ in.} \quad \textbf{OK} \qquad \text{Eq (C6.10.3.4-1)}$$

Girder Stress Check Section 2-2 G4 Node 44
Transversely Stiffened Web - Constructibility - Web

The girder must be checked for the steel weight and for Cast #1 of the concrete deck acting on the noncomposite section for web bend-buckling.

Load	Moment	
Steel	661 k-ft	Table C-1
Cast #1	3,932 k-ft	Table C-1
Total Moment	4,593 k-ft	

Constructibility Load Factor = 1.25 according to the provisions of Article 3.4.2.

Using the section properties for the transversely stiffened web design from Table C-4, compute the vertical bending stress at the top of the web for constructibility.

$$f_{cw} = -\frac{4593(47.03)}{118978}(12)(1.25) = -27.23 \quad \text{ksi} \quad \text{(factored)}$$

Compute the nominal web bend-buckling stress, F_{crw}, according to Article 6.10.1.9.1 since the web is transversely stiffened without longitudinal stiffeners.

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

where: k = 28.71 (calculated on previous page)

$$F_{crw} = \frac{0.9(29000)(28.71)}{\left(\frac{84}{0.5625}\right)^2} = 33.6 \quad \text{ksi} \quad < \quad 1.0(50) = 50 \text{ ksi}$$

$$f_{cw} = |-27.23| \quad \text{ksi} \quad < \quad F_{crw} = 1.0(33.6) = 33.6 \text{ ksi} \quad \textbf{OK}$$

Girder Stress Check Section 2-2 G4 Node 44
Transversely Stiffened Web - Constructibility - Bottom Flange

For critical stages of construction, the following requirement will be satisfied for discretely braced tension flanges according to Article 6.10.3.2.2.

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

Using the section properties for the transversely stiffened web design from Table C-4, the tensile flange stress calculated without consideration of the lateral bending, f_{bu}, in the bottom flange is computed as:

$$f_{bot\ flg} = \quad f_{bu} = \frac{4593(12)(38.47)}{118978}(1.25) = 22.28 \quad \text{ksi (T)} \quad \text{(factored)}$$

$$f_l = \frac{M_{tot\_lat}}{S_{bot\_fl}} = \frac{56.3(12)}{110.3} = 6.13 \quad \text{ksi}$$

Bottom flange: 21 in. x 1.5 in., $S_{bot\_fl} = \dfrac{(1.5)(21)^2}{6} = 110.3 \quad in^3$

Therefore,

22.28 + 6.13 = 28.41 ksi < 1.0(1.0)(50) = 50 ksi **OK**

Girder Stress Check Section 1-1 G4 Node 4
Transversely Stiffened Web - Constructibility Shear Strength - Web

The end panels of stiffened webs are not investigated for shear during construction staging according to Article C6.10.3.3. Interior panels of webs with transverse stiffeners are investigated for constructibility, with or without longitudinal stiffeners, and must satisfy the requirement provided in Article 6.10.3.3 during critical stages of construction. This calculation is similar to the shear strength and is, therefore, not shown in this example.

Girder Stress Check Section 2-2 G4 Node 44
Transversely Stiffened Web - Constructibility - Deck

The concrete deck is checked for constructibility according to Article 6.10.3.2.4. Check the deck tensile stress at this section in Span 1 due to Cast #2. The longitudinal tensile stress in a composite concrete deck due to the factored loads must not exceed ϕf_r during critical stages of construction, unless longitudinal reinforcement is provided according to Article 6.10.1.7. Moment due to Cast #2 = -3,035 k-ft from Table C-1. This section is checked since it lies within the Cast #1 composite section, which is 100 feet long and assumed to be hardened for Cast #2.

According to Article 6.10.1.1.1d, the short-term modular ratio, n, is used to calculate longitudinal flexural stresses in the concrete deck due to all permanent and transient loads.

Assume no creep: n = 7.56.

$$f_{deck} = -\frac{(-3035)(27.42)}{297525}\left(\frac{12}{7.56}\right)(1.25) = 0.55 \quad \text{ksi} \quad \text{(factored)}$$

Article 6.10.1.7 states that whenever the tensile stress in the deck exceeds ϕf_r, defined in Article 5.4.2.6, longitudinal reinforcement equal to at least one percent to the total cross-sectional area of the deck must be placed in the deck. Assume the compressive strength of the hardened concrete from Cast #1 is 3,000 psi at the time Cast #2 is made. The modulus of rupture is:

$$f_r = 0.24\sqrt{f'_c} = 0.24\sqrt{3} = 0.42 \quad \text{ksi}$$

Therefore,

$$\phi f_r = 0.9(0.42) = 0.38 \quad \text{ksi} < 0.55 \text{ ksi}$$

where ϕ = 0.9 from Article 5.5.4.2.1

Therefore, one percent longitudinal reinforcement is required at this section. The reinforcement is to be 60.0 ksi or higher strength, a No. 6 bar or smaller and spaced at not more than 12 inches according to Article 6.10.1.7. The required reinforcement should be placed in two layers uniformly distributed across the deck width and two-thirds should be placed in the top layer.

The concrete stress could be lowered by modifying the placement sequence.

Girder Stress Check Section 6-6 G4 Node 100
Transversely Stiffened Web - Fatigue - Top Flange

Fatigue of the base metal at the bottom of the top flange adjacent to the bearing stiffener/connection plate weld to the flange at this section will be checked for Category C'. Stresses are computed using the factored fatigue vehicle defined in Article 3.6.1.4. The vehicle is placed in an adjacent span to create a negative moment and in the third span to create a positive moment at this section.

M_{min}	-986 k-ft	Table C-1
M_{max}	263 k-ft	Table C-1
M_{range}	1,249 k-ft	

According to the provisions of Article 6.10.1.6, the lateral flange bending stress in the top flange is assumed to be zero since the top flange is considered continuously braced after the deck has hardened. According to the provisions of Article 4.5.2.2, uncracked concrete section properties are to be used for the fatigue checks.

$$\Delta_f = \frac{1249(25.33 - 2.5)}{545757}(12) = 0.63 \quad \text{ksi}$$

One-half of the fatigue threshold for Category C' is $(\Delta F)_n$ = 6 ksi (discussed previously).

Δ_f = 0.63 ksi < $(\Delta F)_n$ = 6 ksi   **OK**

Girder Stress Check Section 2-2 G4 Node 44
Transversely Stiffened Web - Fatigue - Web

Check the special fatigue requirements for webs per Article 6.10.5.3.

Interior panels of webs with transverse stiffeners must satisfy the following:

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

where:

V_u = shear in the web at the section under consideration due to the unfactored permanent load plus the factored fatigue load

The following values are obtained from the analysis output information:

Stage	Shear	
Steel	-5.0 k	Table C-2
Deck	-23.8 k	Table C-2
Superimposed	-4.0 k	Table C-2
FWS	-2.9 k	Table C-2
Total permanent load	-35.7 k	
Fatigue LL	-15 k	Table C-1

$$V_u = -35.7 + 0.75(-15) = -47 \quad \text{k}$$

V_{cr} = 559 k (previously calculated)

Therefore,

$V_u = |-47|$ k < V_{cr} = 559 k   **OK**

<u>Girder Stress Check Section 6-6 G4 Node 100</u>
<u>Transversely Stiffened Web - Fatigue - Web</u>

Check the special fatigue requirements for webs per Article 6.10.5.3. t_w = 0.625 in.

Interior panels of webs with transverse stiffeners must satisfy the following:

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

where:

$$V_{cr} = CV_p \qquad \text{Eq (6.10.9.3.3-1)}$$

$$V_p = 0.58F_{yw}Dt_w \qquad \text{Eq (6.10.9.3.3-2)}$$

$$= 0.58(50)(84)(0.625) = 1522 \text{ k}$$

The ratio, C, is determined as follows:

Since $\dfrac{D}{t_w} = \dfrac{84}{0.625} = 134.4 > 1.40\sqrt{\dfrac{Ek}{F_{yw}}} = 1.40\sqrt{\dfrac{29000(5.61)}{50}} = 80$

where:

$$k = 5 + \frac{5}{\left(\dfrac{d_o}{D}\right)^2} = 5 + \frac{5}{\left[\dfrac{20(12)}{84}\right]^2} = 5.61 \qquad \text{Eq (6.10.9.3.2-7)}$$

Therefore,

$$C = \frac{1.57}{\left(\dfrac{D}{t_w}\right)^2}\left(\frac{Ek}{F_{yw}}\right) = \frac{1.57}{\left(\dfrac{84}{0.625}\right)^2}\left[\frac{29000(5.61)}{50}\right] = 0.283$$

Therefore, V_{cr} = 0.283(1522) = 431 k

V_u = shear in the web at the section under consideration due to the unfactored permanent load plus the factored fatigue load

The following values are obtained from the analysis output information:

Stage	Shear	
Steel	-45.0 k	All values from Table C-2
Deck	-144.0 k	
Superimposed	-28.0 k	
FWS	-36.0 k	
Total permanent load	-253.0 k	
Fatigue LL	-41 k	Table C-1

Girder Stress Check Section 6-6 G4 Node 100
Transversely Stiffened Web - Fatigue - Web (continued)

$$V_u = -253.0 + 0.75(-41) = -283.8 \quad k$$

$V_u = |-283.8|$ k $<$ $V_{cr} = 431$ k **OK**

Girder Stress Check Section 2-2 G4 Node 44
Transversely Stiffened Web - Fatigue - Bottom Flange

Check the fatigue stress in the bottom flange at this section according to the provisions of Article 6.5.3. The fatigue design life is 75 years.

Base metal at transverse stiffeners 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 strength 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 the provisions of Article 3.6.2.1-1, the dynamic load allowance for fatigue is 0.15. Centrifugal force effects are included. One-half of the fatigue threshold is specified as the limiting stress range 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. According to the provisions of Article 4.5.2.2 and Article 6.6, uncracked concrete section properties are to be used for fatigue checks.

M_{min}	-452 k-ft	Table C-1
M_{max}	1,202 k-ft	Table C-1
M_{range}	1,654 k-ft	

According to Article 6.6.1.2, the limiting stress range for Category C' is $(\Delta F)_n$ = 6 ksi for the case where the fatigue strength 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:

$$f_{range} = \frac{1654(71.08 - 1.5)}{297525}(12) = 4.64 \quad \text{ksi}$$

The lateral flange bending stress in the flange at the connection plate must be also considered according to Article C6.10.5.1. Assume that the connection plates are 6 inches wide.

Compute the lateral flange bending stress range at the top of the bottom flange due to curvature. Compute the lateral flange moment of inertia.

$$I_{flg} = \frac{1.5(21)^3}{12} = 1158 \quad \text{in}^4$$

Compute the range of lateral flange moment at the connection plate.

$$M_{lat} = \frac{Ml^2}{NRD} = \frac{1654(20)^2}{10(717)(7)} = 13.18 \quad \text{k-ft} \qquad \text{Eq (C4.6.1.2.4b-1)}$$

Girder Stress Check Section 2-2 G4 Node 44
Transversely Stiffened Web - Fatigue - Bottom Flange (continued)

Compute the stress range due to lateral flange bending at the edge of the connection plate.

$$c = 6 + \frac{0.5625}{2} = 6.3 \text{ in.}$$

$$f_{lat} = \frac{13.18(6.3)}{1158}(12) = 0.86 \text{ ksi}$$

Total live load stress range due to the passage of the fatigue load, $\Delta_f = 4.64 + 0.86 = 5.5$ ksi

$\Delta_f = 5.5$ ksi $<$ $(\Delta F)_n = 6.0$ ksi **OK**

Girder Stress Check Section 2-2 G4 Node 44
Transversely Stiffened Web - Fatigue - Shear Connectors

Shear connectors are to be provided throughout the entire length of the bridge according to the provisions of Article 6.10.10.1.

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

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

The fatigue resistance for an individual stud shear connector, Z_r, is defined in Article 6.10.10.2 as follows:

$$Z_r = \alpha d^2 \ge \frac{5.5d^2}{2} \qquad \text{Eq (6.10.10.2-1)}$$

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

Use: 3 - 6 in. x 7/8 in. dia. studs/row.
Fatigue resistance for one 7/8 in. shear stud = $\frac{5.5}{2}(0.875)^2 = 2.105$ kips

Fatigue resistance for 3 such shear connectors/row = nZ_r = $3(2.105) = 6.315$ kips/row

From Table C-1, the shear force range, V_f, due to one factored fatigue truck = 15 + |-15| = 30 kips.

According to the provisions of Article 6.6.1.2.1 , the live load stress range may be calculated using the short- term composite section assuming the concrete deck to be effective for both positive and negative flexure. The structural deck thickness, t_s, is 9.0 in. The modular ratio, n, equals 7.56. Calculate the effective flange width according to Article 4.6.2.6.1.

First, determine the effective flange width of the interior girder.

One-quarter of the effective span length: $0.25(113.16)(12) = 339$ in.

$12.0t_s$ + the greater of t_w or $1/2b_f$: $12(9) + 0.5625 = 109$ in.
or $12(9) + 0.5(20) = 118$ in. <- controls

The average spacing of adjacent beams: $11.0(12) = 132$ in.

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

One-eighth of the effective span length: $0.125(113.16)(12) = 170$ in.

$6.0t_s$ + the greater of t_w or $1/4b_f$: $6.0(9) + 0.5625 = 55$ in.
or $6.0(9) + 0.25(20) = 59$ in.

The width of the overhang: $3.75(12) = 45$ in. <- controls

Girder Stress Check Section 2-2 G4 Node 44
Transversely Stiffened Web - Fatigue - Shear Connectors (continued)

$$\text{Transformed deck area} = \frac{\text{Area}}{n} = \frac{\left(\frac{118}{2} + 45\right)(9)}{7.56} = 123.8 \quad \text{in}^2$$

Compute the first moment of the transformed short-term area of the concrete deck, Q, with respect to the neutral axis of the uncracked live load short-term composite section.

Determine the distance from the center of the deck to the neutral axis.
Section properties are from Table C-4.
Neutral axis of the short-term composite section is 15.42 in. measured from the top of the top flange.
Moment arm of the deck = Neutral axis - t_{flg} + haunch + $t_s/2$

$$\text{Moment arm} = 15.42 - 1 + 4 + \frac{9}{2} = 22.92 \quad \text{in.}$$

Compute the longitudinal fatigue shear range per unit length, V_{fat}.

$$Q = 123.8(22.92) = 2837 \quad \text{in}^3$$

$$V_{fat} = \frac{V_f Q}{I} = \frac{30(2837)}{297525} = 0.29 \quad \text{k/in.}$$

Compute the radial fatigue shear range per unit length, F_{fat}, due to the factored fatigue vehicle as the larger of F_{fat1} and F_{fat2} specified in Article 6.10.10.1.2.

Compute the radial component of the fatigue shear range due to curvature, F_{fat1}, according to Eq (6.10.10.1.2-4).

Use the range of fatigue moment 1,654 k-ft (computed earlier, page D-18) to compute the bottom flange stress range at the mid-thickness of the flange.

$$F_{fat1} = \frac{A_{bot}\sigma_{flg}\ell}{wR} \qquad \text{Eq (6.10.10.1.2-4)}$$

where:

$$\sigma_{flg} = \frac{1654(71.08 - 0.75)}{297525}(12) = 4.69 \quad \text{ksi}$$

ℓ = 20 ft.; A_{bot} = 31.5 in^2; w = 48 in.; R = 717 ft.

$$F_{fat1} = \frac{31.5(4.69)(20)}{48(717)} = 0.09 \quad \text{k/in.}$$

Girder Stress Check Section 2-2 G4 Node 44
Transversely Stiffened Web - Fatigue - Shear Connectors (continued)

Compute the radial component of the fatigue shear range, F_{fat2}, according to Eq (6.10.10.1.2-5).

Use the cross-frame forces from the 3D analysis to compute F_{fat2}:

Cross-frame force range due to the factored fatigue vehicle (from separate calculations):

Force range in cross-frame diagonal going from bottom of G3 to top of G4

$F = 5.6 + |-1.2| = 6.8$ k

Diagonal forms a 33-degree angle from the horizontal.

Horizontal component of force = $\cos\left[33\left(\frac{2\pi}{360}\right)\right](6.8) = 5.7$ k

Force range in top chord = $0.9 + |-0.2| = 1.1$ k

Top chord force acts in opposite direction to the horizontal component in the diagonal. Therefore, the net range of force transferred from the cross-frame to the top flange is F_{rc}.

$F_{rc} = 5.7 - 1.1 = 4.6$ k

Compute the radial force according to Eq (6.10.10.1.2-5).

$$F_{fat2} = \frac{F_{rc}}{w} \qquad \text{Eq (6.10.10.1.2-5)}$$

$$F_{fat2} = \frac{4.6}{48} = 0.1 \quad \text{k/in.}$$

The fact that F_{fat1} and F_{fat2} are similar in magnitude can be interpreted to indicate that all of the torsion is due to curvature. If other sources of torsion were present, such as skew, the radial shear range computed from the net range of cross-frame force, F_{fat2}, would be significantly greater than the radial shear range due to curvature, F_{fat1}.

Using the larger radial component of shear range, F_{fat2}, compute the net range of shear for fatigue.

The positive and negative longitudinal shears due to vertical bending are due to the factored fatigue vehicle located in Span 1 with the back axle on the left and then on the right of the point under consideration. This means that the truck actually has to turn around to produce the computed longitudinal shear range. The positive and negative radial shear ranges are produced by loading first in Span 1 and then in Span 2. Again, this is not a realistic loading case to combine with the longitudinal shear case, but has been done to be practical and to be conservative.

$$V_{sr} = \sqrt{V_{fat}^2 + F_{fat2}^2} \qquad \text{Eq (6.10.10.1.2-2)}$$

$$V_{sr} = \sqrt{0.29^2 + 0.10^2} = 0.31 \quad \text{k/in.}$$

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

Shear stud pitch, $p \le \frac{nZ_r}{V_{sr}} = \frac{6.315}{0.31} = 20.4$ in./row

Although not illustrated here, the number of shear connectors that is provided must also be checked for strength limit state according to the provisions of Article 6.10.10.4. The strength limit state check for shear connectors is illustrated later in this example.

Girder Stress Check Section 3-3 G4 Node 64
Transversely Stiffened Web - Fatigue - Shear Connectors

Determine the required pitch of the shear connectors at this section (Section 3-3) according to the fatigue provisions of Article 6.10.10.1.2.

The calculation of nZ_r is the same as Section 2-2.
The moment arm from Section 2-2 is the same for this section.

$$\text{Moment arm} = 15.42 - 1 + 4 + \frac{9}{2} = 22.92 \quad \text{in.}$$

Bending shear range due to the factored fatigue vehicle = $6 + |-27| = 33$ kips (Table C-1)

Compute the longitudinal fatigue shear range, V_{fat}.

$$Q = 123.8\,(22.92) = 2837 \quad \text{in}^3$$

$$V_{fat} = \frac{V_f Q}{I} = \frac{33(2837)}{297525} = 0.31 \quad \text{k/in.}$$

Compute the radial fatigue shear range per unit length, F_{fat}, due to the factored fatigue vehicle as the larger of F_{fat1} and F_{fat2} specified in Article 6.10.10.1.2.

Compute the radial component of the fatigue shear range due to curvature, F_{fat1}.

Use the range of fatigue moment |-654| + 986 = 1,640 k-ft (from Table C-1) to compute the bottom flange stress range at the mid-thickness of the flange.

$$F_{fat1} = \frac{A_{bot}\sigma_{flg}\ell}{wR} \qquad \text{Eq (6.10.10.1.2-4)}$$

where:

$$\sigma_{flg} = \frac{1640(71.08 - 0.75)}{297525}(12) = 4.65 \quad \text{ksi}$$

ℓ = 20 ft.; A_{bot} = 31.5 in^2; w = 48 in.; R = 717 ft.

$$F_{fat1} = \frac{31.5(4.65)(20)}{48(717)} = 0.09 \quad \text{k/in.}$$

Use the cross-frame forces from the 3D analysis to compute F_{fat2}:

Cross-frame force range due to the factored fatigue vehicle (from separate calculations):

Force range in cross-frame diagonal going from bottom of G3 to top of G4

$$F = 1.2 + |-4.3| = 5.5 \quad \text{k}$$

Diagonal forms a 33-degree angle from the horizontal.

Horizontal component of force = $\cos\left[33\left(\frac{2\pi}{360}\right)\right](5.5) = 4.6$ k

Force range in top chord = $0.2 + |-0.9| = 1.1$ k

Top chord force acts in opposite direction to the horizontal component in the diagonal. Therefore, the net range of force transferred from the cross-frame to the top flange is F_{rc}.

$F_{rc} = 4.6 - 1.1 = 3.5$ k

Compute the radial force according to Eq (6.10.10.1.2-5).

$$F_{fat2} = \frac{F_{rc}}{w} \qquad \text{Eq (6.10.10.1.2-5)}$$

$$F_{fat2} = \frac{3.5}{48} = 0.07 \text{ k/in.}$$

Using the radial component of shear range F_{fat1}, compute the net range of shear for fatigue.

$$V_{sr} = \sqrt{V_{fat}^2 + F_{fat}^2} \qquad \text{Eq (6.10.10.1.2-2)}$$

$$V_{sr} = \sqrt{0.31^2 + 0.09^2} = 0.32 \text{ k/in.}$$

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

Shear stud pitch, $p \le \frac{nZ_r}{V_{sr}} = \frac{6.315}{0.32} = 19.7$ in./row

Although not illustrated here, the number of shear connectors that is provided must also be checked for strength limit state according to the provisions of Article 6.10.10.4. The strength limit state check for shear connectors is illustrated later in this example.

Girder Stress Check Section 6-6 G4 Node 100
Transversely Stiffened Web - Fatigue - Shear Connectors

Shear connectors are to be provided throughout the entire length of the bridge according to to the provisions of Article 6.10.10.1.

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

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

The calculation of nZ_r is the same as performed for Section 2-2.
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.
N.A. is 25.33 in. from the top of the steel (Table C-4).
Moment arm of the deck = Neutral axis - t_{flg} + haunch + $t_s/2$

$$\text{Moment arm} = 25.33 - 2.5 + 4 + \frac{9}{2} = 31.33 \ \text{in.}$$

Bending shear range due to the factored fatigue vehicle = $2 + |-41| = 43$ kips (from Table C-1)

Compute the longitudinal fatigue shear range per unit length, V_{fat}.

Transformed deck area = 123.8 in^2 (calculations not shown)

$$Q = 123.8(31.33) = 3879 \ \text{in}^3$$

$$V_{fat} = \frac{V_f Q}{I} = \frac{43(3879)}{545757} = 0.31 \ \text{k/in.} \qquad \text{Eq (6.10.10.1.2-3)}$$

Compute the controlling radial shear range, F_{fat}, due to the factored fatigue vehicle as the larger of F_{fat1} and F_{fat2} specified in Article 6.10.10.1.2..

Compute the radial component of the fatigue shear range due to curvature, F_{fat1}, according to Eq (6.10.10.1.2-4).

Use the range of fatigue moment |-986| + 263 = 1,249 k-ft (from Table C-1) to compute the bottom flange stress range at the mid-thickness of the flange.

$$F_{fat1} = \frac{A_{bot}\sigma_{flg} l}{wR} \qquad \text{Eq (6.10.10.1.2-4)}$$

where:

$$\sigma_{flg} = \frac{1249(64.17 - 1.5)}{545757}(12) = 1.72 \ \text{ksi}$$

l = 20 ft.; A_{bot} = 81 in^2; w = 48 in.; R = 717 ft.

Girder Stress Check Section 6-6 G4 Node 100
Transversely Stiffened Web - Fatigue - Shear Connectors (continued)

$$F_{fat1} = \frac{81(1.72)(20)}{48(717)} = 0.08 \text{ k/in.}$$

Compute the radial component of the fatigue shear range, F_{fat2}, according to Eq (6.10.10.1.2-5).

Use the cross-frame forces from the 3D analysis to compute F_{fat2}:

Cross-frame force range due to the factored fatigue vehicle (from separate calculations):

Force range in cross-frame diagonal going from bottom of G3 to top of G4

$F = 0.8 + |-5.3| = 6.1 \text{ k}$

Diagonal forms a 33-degree angle from the horizontal.

Horizontal component of force = $\cos\left[33\left(\frac{2\pi}{360}\right)\right]6.1 = 5.1 \text{ k}$

Force range in top chord = $0.2 + |-0.6| = 0.8 \text{ k}$

Top chord force acts in opposite direction to the horizontal component in the diagonal.

Therefore, the net range of force transferred from the cross-frame to the top flange is F_{rc}.

$F_{rc} = 5.1 - 0.8 = 4.3 \text{ k}$

Compute the radial force according to Eq (6.10.10.1.2-5)

$$F_{fat2} = \frac{F_{rc}}{w} \qquad \text{Eq (6.10.10.1.2-5)}$$

$$F_{fat2} = \frac{4.3}{48} = 0.09 \text{ k/in.}$$

The fact that F_{fat1} and F_{fat2} are similar in magnitude can be interpreted to indicate that all of the torsion is due to curvature. If other sources of torsion were present, such as skew, the radial shear range computed from the net range of cross-frame force, F_{fat2}, would be significantly greater than the radial shear range due to curvature, F_{fat1}.

Using the radial component of shear range F_{fat2}, compute the net range of shear for fatigue.

$$V_{sr} = \sqrt{V_{fat}^2 + F_{fat}^2} \qquad \text{Eq (6.10.10.1.2-2)}$$

$$V_{sr} = \sqrt{0.31^2 + 0.09^2} = 0.32 \text{ k/in.}$$

Girder Stress Check Section 6-6 G4 Node 100
Transversely Stiffened Web - Fatigue - Shear Connectors (continued)

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

Shear stud pitch, $$p \le \frac{nZ_r}{V_{sr}} = \frac{6.315}{0.32} = 19.7 \text{ in./row}$$

Girder Stress Check Section 2-2 G4 Node 44
Transversely Stiffened Web - Strength - Top Flange

The section will be checked for strength for the Strength I load combination in the following computations.

Check the top flange at this section for strength according to the provisions of Article 6.10.6.2.2. Cross sectional properties are given in Table C-4.

Load	Moment	
Steel	661 k-ft	Table C-1
Deck	2,682 k-ft	Table C-1
Total Non-composite	3,343 k-ft	
Superimposed DL	510 k-ft	Table C-1
FWS	583 k-ft	Table C-1
HL-93 LL+IM	5,125 k-ft	Table C-1

Live load is due to three lanes of LRFD HL-93 plus the appropriate centrifugal force effects specified in Article 3.6.3. The dynamic load allowance has been applied to the live load according to Article 3.6.2. The overturning effect of the centrifugal force has been considered by increasing the exterior wheel load and decreasing the interior wheel load in each lane, as computed on pages D-102 and D-103. Although centrifugal force needs to be considered for truck loads only (Article C3.6.3), the vertical effect also has been conservatively included for lane load in this example. The live loads were also multiplied by 0.85 in the analysis to account for the probability of multiple presence, as specified in Table 3.6.1.1.2-1.

Compute the factored vertical bending stress in the top flange due to dead and live load, f_{bu}.

$$f_{bu,\,C} = \left[\frac{\left(\gamma_{DC1}\ M_{DC1}\, C_{nc}\right)}{I_{nc}} + \frac{\left[\left(\gamma_{DC2}\ M_{DC2} + \gamma_{DW}\ M_{DW}\right) C_{3n}\right]}{I_{3n}} + \frac{\left(\gamma_{LL}\ M_{LL}\, C_n\right)}{I_n} \right] (12)\ \eta$$

$$= -\left[\frac{1.25(3343)(48.03)}{118978} + \frac{[\,1.25(510) + 1.5(583)\,]30.06}{217079} + \frac{1.75(5125)(15.42)}{297525} \right] (12)(1) = -28.3 \text{ ksi (C)}$$

Composite sections in horizontally curved steel girder bridges are considered as noncompact sections and must satisfy the requirements of Article 6.10.7.2.

The compression flange must satisfy the following relation:

$$f_{bu} \le \phi_f F_{nc} \qquad \text{Eq (6.10.7.2.1-1)}$$

where:

ϕ_f = 1.0 (Article 6.5.4.2)

$$F_{nc} = R_b R_h F_{yc} \qquad \text{Eq (6.10.7.2.2-1)}$$

R_b = 1.0 from Article 6.10.1.10.2 for D/t_w = 84 / 0.5625 = 149 < 150

Girder Stress Check Section 2-2 G4 Node 44
Transversely Stiffened Web - Strength - Top Flange (continued)

R_h = 1.0 from Article 6.10.1.10.1

Therefore,

$|-28.3|$ ksi < 1.0(1.0)(1.0)(50) = 50 ksi **OK**

According to the provisions of Article C6.10.7.2.1, lateral flange bending need not be considered after the deck has hardened.

Check the ductility requirements of Article 6.10.7.3 to protect the concrete 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).

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

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.

First, determine if the plastic neutral axis is in the web, top flange, or deck. This example neglects the affect of the deck reinforcing steel.

P_s = force in the slab

$= 0.85f_c'b_st_s = 0.85(4)(104)(9) = 3182$ kips

P_t = force in the tension flange

$= F_{yt}b_tt_t = 50(21)(1.5) = 1575$ kips

P_c = force in the compression flange

$= F_{yc}b_ct_c = 50(20)(1.0) = 1000$ kips

P_w = force in the web

$= F_{yw}Dt_w = 50(84)(0.5625) = 2363$ kips

Girder Stress Check Section 2-2 G4 Node 44
Transversely Stiffened Web - Strength - Top Flange (continued)

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

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

$1575 + 2363 = 3938 \quad < \quad 1000 + 3182 = 4182$ PNA not in the web

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

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

$1575 + 2363 + 1000 = 4938 \quad > \quad 3182$ PNA is in the top flange

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.

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

$$= \left(\frac{1}{2}\right)\left(\frac{2363 + 1575 - 3182 - 0 - 0}{1000} + 1\right) = 0.9 \text{ in.}$$

Therefore,

$D_p = 0.9 + 9 = 9.9$ in. $<$ $0.42D_t = 0.42(1.5 + 84 + 1 + 9) = 40.11$ in. **OK**

Girder Stress Check Section 2-2 G4 Node 44
Transversely Stiffened Web - Strength - Web

Article C6.10.1.9.1 states that composite sections subjected to positive flexure need not be checked for web bend-buckling in its final composite condition when the web does not require longitudinal stiffeners.

Girder Stress Check Section 2-2 G4 Node 44
Transversely Stiffened Web - Strength - Bottom Flange

The tension flange must satisfy the following relation:

$$f_{bu} + \frac{1}{3}f_l \le \phi_f F_{nt} \qquad \text{Eq (6.10.7.2.1-2)}$$

Compute the factored bottom flange vertical bending stress due to dead and live load, $f_{bu,\,T}$.

$$f_{bu,\,T} = \left[\frac{(\gamma_{DC1}\ M_{DC1} C_{nc})}{I_{nc}} + \frac{[(\gamma_{DC2}\ M_{DC2} + \gamma_{DW}\ M_{DW})C_{3n}]}{I_{3n}} + \frac{(\gamma_{LL}\ M_{LL} C_n)}{I_n}\right](12)\ \eta$$

$$= \left[\frac{1.25(3343)(38.47)}{118978} + \frac{[\,1.25(510) + 1.5(583)\,]56.44}{217079} + \frac{1.75(5125)(71.08)}{297525}\right](12)(1) = 46.6 \text{ ksi (T)}$$

Compute the lateral bending stress at the cross-frame due to curvature, f_l, using Eq (C4.6.1.2.4b-1).

$$M_{lat\_NC} = \frac{3343\,(20)^2}{10(717)(7)} = 26.64 \text{ k-ft;} \qquad f_{l\_NC} = \frac{M_{lat\_NC}}{S_{bot\_fl}} = \frac{26.64(12)}{110.3} = 2.90 \text{ ksi}$$

$$M_{lat\_C1} = \frac{510\,(20)^2}{10(717)(7)} = 4.06 \text{ k-ft;} \qquad f_{l\_C1} = \frac{M_{lat\_C1}}{S_{bot\_fl}} = \frac{4.06(12)}{110.3} = 0.44 \text{ ksi}$$

$$M_{lat\_C2} = \frac{583\,(20)^2}{10(717)(7)} = 4.65 \text{ k-ft;} \qquad f_{l\_C2} = \frac{M_{lat\_C2}}{S_{bot\_fl}} = \frac{4.65(12)}{110.3} = 0.51 \text{ ksi}$$

$$M_{lat\_LL} = \frac{5125\,(20)^2}{10(717)(7)} = 40.84 \text{ k-ft;} \qquad f_{l\_LL} = \frac{M_{lat\_LL}}{S_{bot\_fl}} = \frac{40.84(12)}{110.3} = 4.44 \text{ ksi}$$

The total factored lateral bending stress is:

$$f_l = 1.25(2.90 + 0.44) + 1.5(0.51) + 1.75(4.44) = 12.71 \text{ ksi}$$

Therefore,

$46.6 + 1/3(12.71) = 50.84$ ksi $\approx$ $1.0(50) = 50$ ksi **Say OK** since the unfactored loads have arbitrarily been rounded up.

Girder Stress Check Section 6-6 G4 Node 100
Transversely Stiffened Web - Strength - Top and Bottom Flange

Check the top and bottom flange for strength at this section according to the provisions of Article 6.10.6. The section will be checked for the Strength I load combination in the following computations. This section is in negative flexure.

Load	Moment	
Steel	-1,917 k-ft	Table C-1
Deck	-7,272 k-ft	Table C-1
Total noncomposite	-9,189 k-ft	
Superimposed DL	-1,537 k-ft	Table C-1
FWS	-1,478 k-ft	Table C-1
HL-93 LL+IM	-6,726 k-ft	Table C-1

Section proportioning limits for this section have been independently checked and have satisfied all criteria of Article 6.10.2.

Top Flange

Check the top flange according to the provision of Article 6.10.6.2.3. Composite sections of curved girders in negative flexure are to be designed as noncompact sections as specified in Article 6.10.8. Compute the vertical bending stress in the top flange. Lateral bending need not be checked for the top flange for composite sections in negative flexure since it is continuously braced by the hardened concrete deck. For loads applied to the composite section, assume a cracked section, as specified in Article 4.5.2.2. Section properties are from Table C-4.

$$f_{bu} = \left[\frac{\left(\gamma_{DC1}\, M_{DC1}\, C_{nc}\right)}{I_{nc}} + \frac{\left[\left(\gamma_{DC2}\, M_{DC2} + \gamma_{DW}\, M_{DW}\right)C_{3n}\right]}{I_{3n}} + \frac{\left(\gamma_{LL}\, M_{LL}\, C_n\right)}{I_n}\right](12)\ \eta$$

$$= -\left[\frac{1.25(-9189)(46.94)}{313872} + \frac{[\,1.25(-1537) + 1.5(-1478)\,]46.26}{321111} + \frac{1.75(-6726)(44.95)}{335040}\right](12)(1) = 46.7\ \text{ksi (T)}$$

The top flange is in tension at this location, therefore, use the provisions of Article 6.10.8.1.3 for continuously braced tension flanges.

$$f_{bu} \le \phi_f R_h F_{yt} \qquad \text{Eq (6.10.8.1.3-1)}$$

46.7 ksi < $\phi_f R_h F_{yf}$ = 1.0(1.0)(50) = 50 ksi **OK**

Girder Stress Check Section 6-6 G4 Node 100
Transversely Stiffened Web - Strength - Top and Bottom Flange (continued)

Bottom Flange

The flange size is constant between brace points. The largest vertical bending stress, f_{bu}, between brace points is at this section. The factored vertical bending stress is calculated according to Article 6.10.1.6.

$$f_{bu} = \left[\frac{1.25(-9189)(42.56)}{313872} + \frac{[1.25(-1537) + 1.5(-1478)]43.24}{321111} + \frac{1.75(-6726)(44.55)}{335040}\right](12) = -44.2 \text{ ksi (C)}$$

The bottom flange is in compression at this location. The compression flange must satisfy the provisions of Article 6.10.8.1.1:

$$f_{bu} + \frac{1}{3}f_l \le \phi_f F_{nc} \qquad \text{Eq (6.10.8.1.1-1)}$$

Compute the lateral bending stress at the cross-frame due to curvature, f_l, using Eq (C4.6.1.2.4b-1).

$$M_{lat\_NC} = \frac{-9189(20)^2}{10(717)(7)} = -73.23 \text{ k-ft;} \qquad f_{l\_NC} = \frac{M_{lat\_NC}}{S_{bot\_fl}} = \frac{-73.23(12)}{364.5} = -2.41 \text{ ksi}$$

$$M_{lat\_C1} = \frac{-1537(20)^2}{10(717)(7)} = -12.25 \text{ k-ft;} \qquad f_{l\_C1} = \frac{M_{lat\_C1}}{S_{bot\_fl}} = \frac{-12.25(12)}{364.5} = -0.4 \text{ ksi}$$

$$M_{lat\_C2} = \frac{-1478(20)^2}{10(717)(7)} = -11.78 \text{ k-ft;} \qquad f_{l\_C2} = \frac{M_{lat\_C2}}{S_{bot\_fl}} = \frac{-11.78(12)}{364.5} = -0.39 \text{ ksi}$$

$$M_{lat\_LL} = \frac{-6726(20)^2}{10(717)(7)} = -53.6 \text{ k-ft;} \qquad f_{l\_LL} = \frac{M_{lat\_LL}}{S_{bot\_fl}} = \frac{-53.6(12)}{364.5} = -1.76 \text{ ksi}$$

Note: During construction, the overhang brackets may introduce lateral bending in the top and bottom flanges depending on the location of the bracket and its configuration. The resulting force, and subsequently the stress, on the flanges will vary. The stress in the bottom flange from this affect will disappear when the brackets are removed after the deck hardens. This is why the affect of these forces is included for construction, but not in the strength case.

The flange lateral bending stress due to first-order elastic analysis effects is calculated as:

$$f_{l1} = 1.25(-2.41 + -0.4) + 1.5(-0.39) + 1.75(-1.76) = -7.2 \text{ ksi}$$

Girder Stress Check Section 6-6 G4 Node 100
Transversely Stiffened Web - Strength - Top and Bottom Flange (continued)

Use the provisions of Article 6.10.1.6 to determine if second-order elastic analysis may be included in the flange lateral bending stress.

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

where:

$$L_p = 1.0 r_t \sqrt{\frac{E}{F_{yc}}} = \frac{1.0(7.42)\sqrt{\frac{29000}{50}}}{12} = 14.89 \text{ ft.} \qquad \text{Eq (6.10.8.2.3-4)}$$

Note: D_c in the effective radius of gyration calculation is determined using Article D6.3.1.

$$L_b = 20 \text{ ft.} > 1.2(14.89)\sqrt{\frac{1.0(1.0)}{\frac{|-44.2|}{50}}} = 19 \text{ ft.}$$ therefore, second-order effects may be included.

The second-order compression flange lateral bending stresses may be approximated using Eq (6.10.1.6-4).

$$f_l = \left(\frac{0.85}{1 - \frac{f_{bu}}{F_{cr}}}\right) f_{l1} \ge f_{l1} \qquad \text{Eq (6.10.1.6-4)}$$

$$= \left(\frac{0.85}{1 - \frac{|-44.2|}{273.6}}\right)(-7.2) = -7.3 \text{ ksi}$$

The nominal flexural resistance of the compression flange, F_{nc}, is taken as the smaller of the local buckling resistance determined using the provisions of Article 6.10.8.2.2 and the lateral torsional buckling resistance determined in Article 6.10.8.2.3.

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

$$\lambda_f = \frac{b_{fc}}{2t_{fc}} = \frac{27}{2(3)} = 4.5 \qquad \text{Eq (6.10.8.2.2-3)}$$

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_{yc}}} = 0.38\sqrt{\frac{29000}{50}} = 9.2$$

Girder Stress Check Section 6-6 G4 Node 100
Transversely Stiffened Web - Strength - Top and Bottom Flange (continued)

Since $\lambda_f < \lambda_{pf}$, $F_{nc} = R_b R_h F_{yc}$ (Eq 6.10.8.2.2-1)

$R_h = 1.0$

$R_b = 1.0$ if the following relation is satisfied:

$$\frac{2D_c}{t_w} \le \lambda_{rw}$$ (note: D_c is computed according to Article C6.10.1.10.2) Eq (6.10.1.10.2-2)

$$\frac{2(41.55)}{0.625} = 133 \quad < \quad 5.7\sqrt{\frac{E}{F_{yc}}} = 5.7\sqrt{\frac{29000}{50}} = 137.3$$

Therefore, the local buckling resistance is:

$F_{nc} = 1.0(1.0)(50) = 50$ ksi

The lateral torsional buckling resistance is determined using Article 6.10.8.2.3.

$L_b = 20$ ft. > $L_p = 14.87$ ft.

$$L_r = \pi r_t \sqrt{\frac{E}{F_{yr}}} = \frac{\pi(7.42)\sqrt{\frac{29000}{0.7(50)}}}{12} = 55.9$$ ft. > L_b, therefore, use Eq (6.10.8.2.3-2)

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

where:

C_b = moment gradient modifier

For members where $f_{mid}/f_2 > 1$ or $f_2 = 0$, C_b is taken as 1.0, otherwise, use Eq (6.10.8.2.3-7).

From previous calculations, the largest compressive stress is taken at Node 100 as:

$f_2 = -44.2$ ksi

From separate calculations,

$f_1 = -26.7$ ksi (at Node 88)

$f_{mid} = -35.5$ ksi (halfway between Nodes 92 and 96)

Girder Stress Check Section 6-6 G4 Node 100
Transversely Stiffened Web - Strength - Top and Bottom Flange (continued)

The ratio f_{mid}/f_2 is less than 1.0, therefore, Eq (6.10.8.2.3-7) is used to calculate C_b.

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

$$= 1.75 - 1.05\left(\frac{26.7}{44.2}\right) + 0.3\left(\frac{26.7}{44.2}\right)^2 = 1.23$$

$$F_{nc} = 1.23\left[1 - \left[1 - \frac{0.7(50)}{1.0(50)}\right]\left(\frac{20 - 14.87}{55.9 - 14.87}\right)\right](1.0)(1.0)(50) = 59.19 \quad \text{ksi} \quad > \quad 50.0 \text{ ksi}$$

Therefore,

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

$|-44.2 + 1/3(-7.3)| = 46.6$ ksi $<$ $\phi_f F_{nc} = 1.0(50) = 50$ ksi **OK**

Bend-buckling of the web must also be checked at the strength limit state according to the provisions of Article 6.10.1.9, as illustrated for this section in a subsequent calculation. It should be noted that the critical compressive flange stress, determined above, should not exceed the critical compressive web stress (adjusted for the thickness of the flange).

Girder Stress Check Section 6-6 G4 Node 100
Transversely Stiffened Web - Bending Strength - Web

Article C6.10.1.9 states that composite sections subjected to negative flexure need to be checked for web bend-buckling in its final composite condition when the web does not require longitudinal stiffeners. D_c will be computed utilizing the steel section and the deck reinforcing steel.

Check the web strength at this section according to the provisions of Article 6.10.1.9. Use the moments from Table C-1 and the section properties from Table C-4. The composite section is assumed cracked for this condition according to the provisions of Article 4.5.2.2. Compute the factored vertical bending stress at the top and bottom of the web due to dead and live load. t_w = 0.625 in.

$$f_{cw\ top} = -\left[\frac{1.25(-9189)(44.44)}{313872} + \frac{[1.25(-1537) + 1.5(-1478)]43.76}{321111} + \frac{1.75(-6726)(42.45)}{335040}\right](12) = 44.2 \text{ ksi (T)}$$

$$f_{cw\ bot} = \left[\frac{1.25(-9189)(39.56)}{313872} + \frac{[1.25(-1537) + 1.5(-1478)]40.24}{321111} + \frac{1.75(-6726)(41.55)}{335040}\right](12) = -41.1 \text{ ksi (C)}$$

The effective D_c for this section is calculated in accordance with Article D6.3.1 of the Specification.

$$D_c = \left(\frac{-f_c}{|f_c| + f_t}\right)d - t_{fc} \geq 0 \quad = \quad \left[\frac{-(-41.1)}{|-41.1| + 44.2}\right](2.5 + 84 + 3) - 3 = 40.12 \quad \text{in.} \quad \text{Eq (D6.3.1-1)}$$

Compute the critical stress according to Article 6.10.1.9.1.

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

$$k = \frac{9.0}{\left(\frac{D_c}{D}\right)^2} = \frac{9.0}{\left(\frac{40.12}{84}\right)^2} = 39.5 \qquad \text{Eq (6.10.1.9.1-2)}$$

$$F_{crw} = \frac{0.9(29000)(39.5)}{\left(\frac{84}{0.625}\right)^2} = 57.07 \quad \text{ksi} > 1.0(50) = 50 \text{ ksi, therefore, } F_{crw} = 50 \text{ ksi}$$

$|-41.1|$ ksi < 50 ksi **OK**

Article C6.10.1.9.1 states that the difference between the compressive stress in the top or bottom of the web when compared to the compression flange stress is minimal, therefore, for all other sections in this example, the compressive flange stress will be used for the web check (only applies to sections where the flange compressive stress is calculated).

Girder Stress Check Section 1-1 G4 Node 4
Transversely Stiffened Web - Shear Strength - Web

Determine the shear strength of the end panel in girder G4 at Node 4.

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

where:

V_u = shear in the web at the section under consideration due to the factored loads (k)

V_n = nominal shear resistance as specified in Articles 6.10.9.2 and 6.10.9.3 (k)

ϕ_v = resistance factor for shear specified in Article 6.5.4.2 taken as 1.0

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

$$V_p = 0.58F_yDt_w \qquad \text{Eq (6.10.9.3.3-2)}$$

$$V_p = 0.58(50)(84)(0.5625) = 1370 \quad \text{k}$$

C, the ratio of the shear-buckling resistance to the shear yield strength, is calculated using the provisions of Article 6.10.9.3.1.

$$\frac{D}{t_w} > 1.40\sqrt{\frac{Ek}{F_{yw}}}$$

$$149.3 \quad > \quad 1.40\sqrt{\frac{29000(10)}{50}} = 106.6$$

where:

d_o = 84 in.

k = shear-buckling coefficient

$$= \quad 5 + \frac{5}{\left(\frac{d_o}{D}\right)^2} \quad = \quad 5 + \frac{5}{\left(\frac{84}{84}\right)^2} = 10$$

Therefore, C is calculated using Eq (6.10.9.3.2-6).

$$C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2}\left(\frac{Ek}{F_{yw}}\right) \qquad \text{Eq (6.10.9.3.2-6)}$$

Girder Stress Check Section 1-1 G4 Node 4
Transversely Stiffened Web - Shear Strength - Web (continued)

$$C = \frac{1.57}{\left(\frac{84}{0.5625}\right)^2}\left[\frac{29000(10)}{50}\right] = 0.408$$

Therefore, the nominal shear strength is calculated as:

$$V_n = CV_p = 0.408(1370) = 559 \text{ k}$$

From Table C-2, the factored shear at the end support (Section 1-1) is V_u = 452 kips.

V_u = 452 k < 1.0(559) = 559 k **OK**

Girder Stress Check Section 6-6 G4 Node 100
Transversely Stiffened Web - Shear Strength - Web

Determine the required transverse stiffener spacing in Field Section 2 (defined in Appendix F of this example) of G4 according to the provisions of Article 6.10.9.1. t_w = 0.625 in.

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

Determine which nominal resistance equation for an interior panel is to be used per Article 6.10.9.3.2.

$$\text{if} \quad \frac{2Dt_w}{\left(b_{fc}t_{fc} + b_{ft}t_{ft}\right)} \le 2.5 \quad \text{then use Eq (6.10.9.3.2-2)}$$

$$\frac{2(84)(0.625)}{[\,27(3) + 28(2.5)\,]} = 0.695 \quad < \quad 2.5$$

$$V_n = V_p\left[C + \frac{0.87(1-C)}{\sqrt{1+\left(\frac{d_o}{D}\right)^2}}\right] \qquad \text{Eq (6.10.9.3.2-2)}$$

$$V_p = 0.58F_yDt_w \qquad \text{Eq (6.10.9.3.2-3)}$$

$$V_p = 0.58(50)(84)(0.625) = 1522 \quad k$$

Try a stiffener spacing d_o = 20(12) = 240 in. (assume connection plates are the only transverse stiffeners present). From page D-16, the following values were calculated.

$$k = 5.61 \qquad \text{Eq (6.10.9.3.2-7)}$$

$$C = 0.283$$

From Table C-2, the maximum factored shear within this field section is at the pier (Section 6-6), V_u = 604 kips.

$$V_n = 1522\left[0.283 + \frac{0.87(1-0.283)}{\sqrt{1+\left(\frac{240}{84}\right)^2}}\right] = 744 \quad k$$

$$V_u \le \phi_v V_n$$

604 k < 1.0(744) = 744 k **OK**

Girder Stress Check G4 Span 1
Transversely Stiffened Web - Strength - Shear Connectors

Compute the number of shear connectors required for the strength limit state in Span 1 according to the provisions of Article 6.10.10.4.

The factored shear resistance of a single shear connector, Q_r, at the strength limit state is taken as:

$$Q_r = \phi_{sc} Q_n \qquad \text{Eq (6.10.10.4.1-1)}$$

Shear connectors are 6 in. long by 7/8 in. diameter.

Compute the nominal resistance of one shear connector embedded in a concrete deck using Article 6.10.10.4.3.

$$Q_n = 0.5 A_{sc} \sqrt{f'_c E_c} \le A_{sc} F_u \qquad \text{Eq (6.10.10.4.3-1)}$$

where:

$$A_{sc} = \frac{\pi (0.875)^2}{4} = 0.6 \quad in^2$$

$$Q_n = 0.5(0.6)\sqrt{4(3834)} = 37.2 \quad kips$$

$$A_{sc} F_u = 0.6(60) = 36 \quad kips$$

Since Q_n is limited to $A_{sc}F_u$, Q_n = 36 kips

Compute the nominal shear force, P, according to the provisions of Article 6.10.10.4.2 for a composite section in the final condition. This nominal shear force is taken between the point of maximum positive design live load plus the dynamic load allowance moment and each end of the span.

Simply supported end to the point of maximum positive live load plus dynamic load allowance

$$P = \sqrt{P_p^2 + F_p^2} \qquad \text{Eq (6.10.10.4.2-1)}$$

The total longitudinal shear force, P_p, in the concrete deck is taken as the lesser of Eqs (6.10.10.4.2-2) and (6.10.10.4.2-3).

$$P_{1p} = 0.85 f_c' \, b_s t_s \qquad \text{Eq (6.10.10.4.2-2)}$$

$$P_{2p} = F_{yw} D t_w + F_{yt} b_{ft} t_{ft} + F_{yc} b_{fc} t_{fc} \qquad \text{Eq (6.10.10.4.2-3)}$$

Girder Stress Check G4 Span 1
Transversely Stiffened Web - Strength - Shear Connectors (continued)

The effective width, b_s, is calculated according to Article 4.6.2.6.1 for an exterior girder (calculated previously). Although G4 is an exterior girder with an overhang less than half of the girder spacing, the width of the deck is assumed to be equal to the interior girder effective width so that all girders will have the same stud spacing.

$$P_{1p} = 0.85(4)(118)(9) = 3611 \quad \text{k}$$

$$P_{2p} = 50(84)(0.5625) + 50(20)(1) + 50(21)(1.5) = 4938 \quad \text{k}$$

Therefore, P_p is taken to be 3611 k.

The total radial shear force in the concrete deck, F_p, at the point of maximum positive live load plus the dynamic load allowance moment is taken as:

$$F_p = P_p \frac{L_p}{R} \qquad \text{Eq (6.10.10.4.2-4)}$$

The arc length, L_p, between the point of maximum positive live load plus the dynamic load allowance moment and the end of the girder is 73 feet.

$$F_p = 3611\left(\frac{73}{717}\right) = 367.6 \quad \text{k}$$

$$P = \sqrt{3611^2 + 367.6^2} = 3630 \quad \text{k}$$

The minimum number of shear connectors, n, over the region under consideration is taken as:

$$n = \frac{P}{\phi_{sc}\, Q_n} = \frac{3630}{0.85(36)} = 119 \qquad \text{Eq (6.10.10.4.1-2)}$$

where ϕ_{sc} is obtained from Article 6.5.4.2

Compute the required pitch, p, with 3 studs per row.

$$\text{No. of rows} = \frac{119}{3} = 39.7 \qquad \text{say 40 rows}$$

$$p = \frac{73(12)}{(40-1)} = 22.5 \quad \text{in.}$$

The shear connector pitch for strength is less critical than for fatigue in this region.

Girder Stress Check G4 Span 1
Transversely Stiffened Web - Strength - Shear Connectors (continued)

Between the maximum positve live load plus the dynamic load allowance moment and the adjacent interior support

Compute the required pitch at this location.

Compute the total nominal shear force, P, in the concrete deck at this location according to the provisions of Article 6.10.10.4.2.

$$P = \sqrt{P_T^2 + F_T^2} \qquad \text{Eq (6.10.10.4.2-5)}$$

where:

P_T = total longitudinal shear force in the concrete deck between the point of maximum positive live load plus the dynamic load allowance moment and the centerline of an adjacent interior support

$$= P_p + P_n \qquad \text{Eq (6.10.10.4.2-6)}$$

The total longitudinal shear force, P_p, in the concrete deck at the point of maximum positive live load plus the dynamic load allowance moment is taken as the lesser of:

$$P_{1p} = 0.85 f_c' b_s t_s \qquad \text{Eq (6.10.10.4.2-2)}$$

$$= 0.85(4)(118)(9) = 3611 \quad \text{k}$$

or

$$P_{2p} = F_{yw} D t_w + F_{yt} b_{ft} t_{ft} + F_{yc} b_{fc} t_{fc} \qquad \text{Eq (6.10.10.4.2-3)}$$

$$= 50(84)(0.5625) + 50(21)(1.5) + 50(20)(1.0) = 4938 \quad \text{k}$$

Therefore, P_p = 3611 k

The total longitudinal shear force, P_n, in the deck over an interior support is taken as the lesser of:

$$P_{1n} = F_{yw} D t_w + F_{yt} b_{ft} t_{ft} + F_{yc} b_{fc} t_{fc} \qquad \text{Eq (6.10.10.4.2-7)}$$

$$= 50(84)(0.625) + 50(28)(2.5) + 50(27)(3) = 10175 \quad \text{k}$$

or

$$P_{2n} = 0.45 f_c' b_s t_s = 0.45(4)(118)(9) = 1912 \quad \text{k} \qquad \text{Eq (6.10.10.4.2-8)}$$

Therefore, P_n = 1912 k

$$P_T = 3611 + 1912 = 5523 \quad \text{k}$$

Girder Stress Check G4 Span 1
Transversely Stiffened Web - Strength - Shear Connectors (continued)

L_n = arc length between the point of maximum positive live load plus the dynamic load allowance moment and the centerline of an adjacent interior support

$= 163.77 - 73 = 90.8$ ft.

$$F_T = P_T \frac{L_n}{R} = 5523\left(\frac{90.8}{717}\right) = 699 \text{ k}$$ Eq (6.10.10.4.2-9)

$$P = \sqrt{P_T^2 + F_T^2} = \sqrt{5523^2 + 699^2} = 5567 \text{ k}$$

$$n = \frac{P}{\phi_{sc}\, Q_n} = \frac{5567}{0.85(36)} = 182$$ Eq (6.10.10.4.1-2)

Compute the required pitch p with 3 studs per row.

No. of rows $= \dfrac{182}{3} = 60.7$ say 61 rows

$$p = \frac{90.8(12)}{(61 - 1)} = 18.2 \text{ n.}$$

The shear connector pitch for strength is more critical than for fatigue in this region.

Girder Stress Check G4 Field Section 2 (Span 1)
Transversely Stiffened Web - Transverse Stiffener Design

Design the transverse stiffeners in the Span 1 portion of Field Section 2 of the transversely stiffened web design according to the provisions of Article 6.10.11.1.

The entire length of all 4 girders, except the end panels, may use a 240 in. stiffener spacing, as determined previously.

In general, stiffeners not used as connection plates will be tight fit at both the compression and tension flanges. This helps restrain the flanges and aides in retaining the cross-sectional configuration of the girder when subjected to torsion.

Transverse stiffeners used as connecting plates for diaphragms or cross-frames are to be connected by welding or bolting to both flanges. The projecting width for the stiffener is the only provision to be satisfied if the following relation is true.

$$\frac{D}{t_w} \leq 2.5\sqrt{\frac{E}{F_{yw}}} \qquad \text{Eq (6.10.11.1.1-1)}$$

$$\frac{84}{0.625} = 134.4 \quad > \quad 2.5\sqrt{\frac{29000}{50}} = 60.2$$

Therefore, the connecting plates will be checked with the same provisions of the transverse stiffeners.

Determine the minimum required stiffener projecting width according to the provisions of Article 6.10.11.1.2 The minimum required width, b_t, must satisfy the following two relations.

Try one plate 6.75 in. x 0.625 in.

$$b_t \geq 2 + \frac{d}{30} \qquad \text{Eq (6.10.11.1.2-1)}$$

$$= 2 + \frac{84 + 2.5 + 3.0}{30} = 4.983 \text{ in.} < 6.75 \text{ in.}$$

$$16t_p \geq b_t \geq \frac{b_f}{4} \qquad \text{Eq (6.10.11.1.2-2)}$$

Assume a transverse stiffener thickness of 0.625 in.

$$16(0.625) = 10 \text{ in.} > 6.75 \text{ in.} \geq \frac{27}{4} = 6.75 \text{ in.}$$

Compute the required moment of inertia of the stiffener according to Eq (6.10.11.1.3-1).

$$I_t \geq d_o t_w^3 J \qquad \text{Eq (6.10.11.1.3-1)}$$

where:

$$J = 2.5\left(\frac{D}{d_o}\right)^2 - 2 \geq 0.5 \qquad \text{Eq (6.10.11.1.3-2)}$$

$$= 2.5\left(\frac{84}{240}\right)^2 - 2 = -1.694$$ since this value cannot be less than 0.5, use J = 0.5

$$d_o t_w^3 J = 240\,(0.625)^3(0.5) = 29.3 \quad in^4$$

For a single stiffener, the moment of inertia is taken about the edge in contact with the web:

t_p = 0.625 in., b_s = 6.75 in.

$$I_t = \frac{t_p\, b_s^3}{3} = \frac{0.625\,(6.75)^3}{3} = 64.07 \quad in^4 > 29.30 \;\; in^4$$ **OK**

Transverse stiffeners need sufficient area to resist the vertical component of the tension field. Check the stiffener area according to Article 6.10.11.1.4 using the data from Section 6-6 (previously calculated).

$$A_s \geq \left[0.15B\,\frac{D}{t_w}(1 - C)\left(\frac{V_u}{\phi_v V_n}\right) - 18\right]\frac{F_{yw}}{F_{crs}}\,t_w^2 \qquad \text{Eq (6.10.11.1.4-1)}$$

where:

$$F_{crs} = \frac{0.31E}{\left(\frac{b_t}{t_p}\right)^2} \leq F_{ys} \qquad \text{Eq (6.10.11.1.4-2)}$$

$$= \frac{0.31(29000)}{\left(\frac{6.75}{0.625}\right)^2} = 77.1 > F_{ys} = 50.0 \text{ ksi, therefore, use } F_{crs} = 50.0 \text{ ksi}$$

$$6.75(0.625) = 4.22 \;\; in^2 > \left[0.15(1.8)\,\frac{84}{0.625}(1 - 0.283)\left[\frac{604}{1.0(744)}\right] - 18\right]\frac{50.0}{50.0}(0.625)^2 = 1.22 \;\; in^2$$

OK

Girder Stress Check Section 1-1 G4 Node 4
Transversely Stiffened Web - Bearing Stiffener Design

This location has the largest total reaction at a simple end support (Table C-2).

Load	Reaction (kips)	
Steel	$23 \times 1.25 \times \eta$	= 28.8
Deck	$92 \times 1.25 \times \eta$	= 115.0
Superimposed DL	$19 \times 1.25 \times \eta$	= 23.75
FWS	$23 \times 1.50 \times \eta$	= 34.5
HL-93 LL + IM	$143 \times 1.75 \times \eta$	= 250.3
Total	300	452.4 (say 452 k)

Design the bearing stiffeners according to Article 6.10.11.2. Use stiffeners with F_{ys} = 50 ksi.

The projecting width, b_t, of each stiffener element must satisfy the following criteria (assume the thickness of the bearing stiffener is 0.75 in.):

$$b_t \le 0.48 t_p \sqrt{\frac{E}{F_{ys}}} = 0.48\,(0.75)\sqrt{\frac{29000}{50}} = 8.7 \qquad \text{Eq (6.10.11.2.2-1)}$$

According to Article 6.10.11.2.3, the factored bearing resistance for the fitted ends of bearing stiffeners is taken as:

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

where:

$(R_{sb})_n$ = nominal bearing resistance for the fitted ends of the bearing stiffeners

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

Try 2-plates 7 in. x 0.75 in. $A_{pn} = 2(7-1)(0.75) = 9$ in^2 (Assume 1 in. for the stiffener clip).

$$(R_{sb})_n = 1.4(9)(50) = 630 \text{ k}$$

$(R_{sb})_r = 1.0(630) = 630$ k > R_u = 452 k **OK**

where ϕ_b is obtained from Article 6.5.4.2

Determine the axial resistance of the bearing stiffener according to Article 6.10.11.2.4. This article directs the designer to Article 6.9.2.1 for calculation of the factored axial resistance, P_r. The yield stress is F_{ys}, the radius of gyration is computed about the mid-thickness of the web and the effective length is 0.75 times the web depth.

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

Girder Stress Check Section 1-1 G4 Node 4
Transversely Stiffened Web - Bearing Stiffener Design (continued)

The nominal compressive resistance, P_n, for a noncomposite axially loaded member is computed using Article 6.9.4.

Check if λ is greater than or less than 2.25.

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

Compute the effective area of the web contributing to the column (t_w = 0.5625 in.) per Article 6.10.11.2.4b. This example assumes $9t_w$ is available behind the bearing stiffener at the simply supported end.

$$A_w = 18t_w^2 = 18\,(0.5625)^2 = 5.7 \quad in^2$$

Calculate the radius of gyration, r_s.

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

where:

$$A = 5.70 + 9 = 14.7 \quad in^2$$

(Note: conservatively continue using the area at the base of the stiffener to compute the axial resistance.)

$$I = 2\left[\frac{0.75\,(7)^3}{12} + 7(0.75)\,(3.781)^2\right] = 193 \quad in^4$$

$$r_s = \sqrt{\frac{193}{14.7}} = 3.62 \quad in.$$

Therefore,

$$\lambda = \left[\frac{0.75(84)}{3.62(\pi)}\right]^2 \left(\frac{50}{29000}\right) = 0.053 \quad < \quad 2.25, \text{ use Eq (6.9.4.1-1) for } P_n$$

$$P_n = 0.66^{\lambda} F_y A_s = 0.66^{0.053}(50)(14.7) = 719 \quad k \qquad \text{Eq (6.9.4.1-1)}$$

Girder Stress Check Section 1-1 G4 Node 4
Transversely Stiffened Web - Bearing Stiffener Design (continued)

$P_r = \phi_c P_n$

$P_r = (0.9)(719) = 647$ k $>$ $R_u = 452$ k **OK**

where ϕ_c is taken from Article 6.5.4.2 as 0.9 for axial compression, steel only.

Girder Stress Check Section 6-6 G1 Node 97
Transversely Stiffened Web - Bearing Stiffener Design

This location has the largest total reaction at an interior support (Example Appendix B - Computer Output).

Load	Reaction (kips)	
Steel	$81 \times 1.25 \times \eta$	= 101
Deck	$318 \times 1.25 \times \eta$	= 398
SupImp	$61 \times 1.25 \times \eta$	= 76
FWS	$70 \times 1.50 \times \eta$	= 105
HL-93 LL + IM	$294 \times 1.75 \times \eta$	= 515
Total Reaction	824	1195

Design the bearing stiffeners according to Article 6.10.11.2. Use stiffeners with F_y = 50 ksi.

Try 2-plates 11 in. x 1 in. $A_{pn} = 2(11 - 1)(1) = 20$ n^2 (Assume 1 in. for the stiffener clip).

Check the factored bearing resistance provisions of Article 6.10.11.2.3 and compare it to the applied loading.

$(R_{sb})_r = 1.0(1.4)(20)(50) = 1400$ k > R_u = 1195 k **OK** Eq (6.10.11.2.3-1)

Determine the factored axial resistance of the bearing stiffener similarly to the method used for Section 1-1.

$$P_r = \phi_c P_n$$ Eq (6.9.2.1-1)

The nominal compressive resistance, P_n, for a noncomposite axially loaded member is computed using Article 6.9.4. Check if λ is greater than or less than 2.25.

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

Calculate the radius of gyration, r_s.

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

Using t_w = 0.625 in. and allowing 18tw of the girder web to be considered effective with the stiffener:

$$A_w = 18(0.625)^2 = 7.03 \text{ in}^2$$

Therefore,

$$A = 20 + 7.03 = 27.03 \text{ in}^2$$

Girder Stress Check Section 6-6 G1 Node 97
Transversely Stiffened Web - Bearing Stiffener Design (continued)

$$I = 2\left[\frac{1(11)^3}{12} + 1(11)(5.8125)^2\right] = 965.1 \quad \text{in}^4$$

$$r_s = \sqrt{\frac{965.1}{27.03}} = 5.98 \quad \text{in.}$$

Therefore,

$$\lambda = \left[\frac{0.75(84)}{5.98(\pi)}\right]^2\left(\frac{50}{29000}\right) = 0.019 \quad < \ 2.25, \ \text{use Eq (6.9.4.1-1) for } P_n$$

$$P_n = 0.66^{\lambda} F_y A_s = 0.66^{0.019}(50)(27.03) = 1341 \ ‹ \qquad \text{Eq (6.9.4.1-1)}$$

$$P_r = \phi_c P_n$$

$$P_r = (0.9)(1340) = 1206 \quad \text{k} \ > \ R_u = 1195 \text{ k} \ \ \textbf{OK}$$

Girder Stress Check Section 2-2 G4 Node 44
Longitudinally Stiffened Web - Constructibility - Web

Check the longitudinally stiffened web at this section for the noncomposite load due to the steel weight and Cast #1. Use the moments for the transversely stiffened web design from Table C-1.

Compute the vertical bending stress at the top of the web with a load factor = 1.25, as specified in Article 3.4.2. Use the section properties for the longitudinally stiffened web design from Table C-4. The section proportioning of Article 6.10.2 has been independently checked for the top and bottom flange.

Check section proportion limits for a longitudinally stiffened web.

$$\frac{D}{t_w} = \frac{84}{0.4375} = 192 < 300 \quad \textbf{OK} \qquad \text{Eq (6.10.2.1.2-1)}$$

The longitudinal stiffener is located 18 in. from the top flange; d_s = 18 in. Article C6.10.11.3.1 states that the optimum location of the longitudinal stiffener for a noncomposite section is $2D_c/5$ for bending and D/2 for shear. In addition to the longitudinal stiffener, an intermediate transverse stiffener is also present at a constant spacing of 10 ft. (approximately one-half of the cross-frame spacing along the entire length of the bridge) to satisfy the provisions of Article 6.10.9.1 for a stiffened web.

Check the web in positive bending according to the provisions of Article 6.10.1.9.2. The compression-flange strength is used (Article C6.10.1.9.1).

$$f_{cw} = -\frac{4593(48.71)}{116890}(12)(1.25) = -28.71 \quad \text{ksi}$$

Compute the critical web bend-buckling stress.

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

Calculate k using either Eq (6.10.1.9.2-1) or Eq (6.10.1.9.2-2) as applicable.

$$\frac{d_s}{D_c} = \frac{18}{47.71} = 0.38 < 0.40, \text{ therefore, use Eq (6.10.1.9.2-2).}$$

$$k = \frac{11.64}{\left(\frac{D_c - d_s}{D}\right)^2} = \frac{11.64}{\left(\frac{47.71 - 18}{84}\right)^2} = 93.05 \qquad \text{Eq (6.10.1.9.2-2)}$$

$$F_{crw} = \frac{0.9(29000)(93.05)}{\left(\frac{84}{0.4375}\right)^2} = 65.9 \quad \text{ksi} > 1.0(50) = 50 \text{ ksi, therefore, } F_{crw} = 50 \text{ ksi}$$

$$|-28.71| \text{ ksi} < 50 \text{ ksi} \quad \textbf{OK}$$

Girder Stress Check Section 3-3 G4 Node 64
Longitudinally Stiffened Web - Constructibility - Web

Check the longitudinally stiffened web for Cast #1, which ends at this point, Section 3-3. The section is noncomposite for this condition.

From Table C-1, the noncomposite moment = $213 + 2554 = 2767$ k-ft

The section properties are from Table C-4. The neutral axis is 47.71 in. from the top of the web. Compute the factored vertical bending stress at the top of the web.

$$f_{cw} = -\frac{2767(47.71)}{116890}(12)(1.25) = -16.94 \text{ ksi}$$

Locate the longitudinal stiffener 42 in. from the top (compression) flange; d_s = 42 in.

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

$$\frac{d_s}{D_c} = \frac{42}{47.71} = 0.88 > 0.40, \text{ therefore, use Eq (6.10.1.9.2-1).}$$

$$k = \frac{5.17}{\left(\frac{42}{84}\right)^2} = 20.7 < \frac{9}{\left(\frac{47.71}{84}\right)^2} = 27.9 \quad \text{therefore, use } k = 27.9 \qquad \text{Eq (6.10.1.9.2-1)}$$

$$F_{crw} = \frac{0.9(29000)(27.9)}{\left(\frac{84}{0.4375}\right)^2} = 19.75 \text{ ksi} < R_hF_{yc} = 1.0(50) = 50 \text{ ksi}$$

$|-16.94|$ ksi < 19.75 ksi **OK**

<u>Girder Stress Check Section 4-4 G4 Node 76</u>
<u>Longitudinally Stiffened Web - Constructibility - Web</u>

The longitudinal stiffener is located 56 in. from the top flange at this section; d_s = 56 in.

Compute the factored bending stress in the compression flange due to Cast #1. The constructibility load factor equals 1.25 according to the provisions of Article 3.4.2.

Moments are from Table C-1. $M_{constr} = -290 + 1023 = 733$ k-ft; the top flange is in compression.

Section properties are from Table C-4.

$$f_{top\ fl} = -\frac{733(48.71)}{116890}(12)(1.25) = -4.58 \quad \text{ksi (C)}$$

Compute the ratio of d_s to D_c to determine the method of computing k. D_c is taken from Table C-4.

$$\frac{d_s}{D_c} = \frac{56}{47.71} = 1.17 > 0.40$$

Therefore, compute k as follows from Eq (6.10.1.9.2-1)

$$k = \frac{5.17}{\left(\frac{56}{84}\right)^2} = 11.63 < \frac{9}{\left(\frac{47.71}{84}\right)^2} = 27.9 \qquad \text{use } k = 27.9$$

$$F_{crw} = \frac{0.9(29000)(27.9)}{\left(\frac{84}{0.4375}\right)^2} = 19.8 \quad \text{ksi} < R_h F_{yw} = 1.0(50) = 50 \text{ ksi} \qquad \text{Eq (6.10.1.9.1-1)}$$

$|-4.58|$ ksi < 19.8 ksi **OK**

Girder Stress Check Section 3-3 G4 Node 64
Longitudinally Stiffened Web - Constructibility - Deck

Cast #2

Check the deck stress due to Cast #2, which ends at Section 3-3. Assume the section is composite for this condition.

From Table C-1, M = -3,113 k-ft. Compute the stress in the top of the deck. Assume no creep. Use n = 7.56, which corresponds to the strength of the concrete in Cast #1 at the time Cast #2 is being poured. The distance from the neutral axis to the ext $14.57 + 3 + 9 = 26.57$ in.

$$f_{deck} = -\frac{(-3113)(26.57)}{290835}(12)\left(\frac{1.25}{7.56}\right) = 0.56 \quad \text{ksi}$$

The stress in the deck is most likely not this high at the free end of the deck placed in Cast #1. It is likely that the composite section does not remain plane at this location and some slip occurs between the deck and the top flange. Some overstress likely occurs in the shear connectors.

Nevertheless, since the tensile stress in the deck due to the factored construction loads exceeds 0.9 times the modulus of rupture, Article 6.10.1.7 requires that longitudinal reinforcement equal to at least one percent of the total cross sectional area of the deck be placed in the deck at this location. The reinforcement is to be No. 6 bars or smaller, spaced at not more than 12 inches.

Girder Stress Check Section 3-3 G4 Node 64
Longitudinally Stiffened Web - Fatigue - Top Flange

Since stress reversal occurs at this section, the top and bottom of the girder must be examined. To determine if a single longitudinal stiffener is sufficient at this section, constructibility, fatigue, and strength must be checked at this point.

The uncracked composite section is to be used in the computation of fatigue stresses according to the provisions of Article 4.5.2.2. Use the section properties from Table C-4.

Load	Moment	
Steel	213 k-ft	Table C-1
Deck	802 k-ft	Table C-1
Total Noncomposite DL	1,015 k-ft	
Superimposed DL	175 k-ft	Table C-1
FWS	206 k-ft	Table C-1
HL-93 LL + IM	4,338 k-ft	Table C-1
M_{min}	-654 k-ft	
M_{max}	986 k-ft	
M_{range}	1,640 k-ft	

Compute the maximum unfactored dead load vertical bending stresses in the top and bottom of the web of the steel girder.

$$f_{top\ web} = -\left[\frac{1015(47.71)}{116890} + \frac{381(28.47)}{214623}\right](12) = -5.6 \quad \text{ksi (C)}$$

$$f_{bot\ web} = \left[\frac{1015(37.79 - 1.5)}{116890} + \frac{381(57.03 - 1.50)}{214623}\right](12) = 4.96 \quad \text{ksi (T)}$$

Compute the vertical bending stress at the bottom of the web due to the positive fatigue moment.

$$f_{bot\ web} = \frac{986(71.93 - 1.5)}{290835}(12) = 2.87 \quad \text{ksi}$$

Compute the vertical bending stress at the top of the web due to the negative fatigue moment.

$$f_{top\ web} = -\frac{(-654)(13.57)}{290835}(12) = 0.37 \quad \text{ksi}$$

Check if two times the tensile fatigue stress overcomes the dead load stress in the top of the web.

Two times the tensile fatigue stress = $2 \times 0.37 = 0.74$ ksi

Girder Stress Check Section 3-3 G4 Node 64
Longitudinally Stiffened Web - Fatigue - Bottom Flange

|-5.6| ksi > 0.74 ksi, therefore fatigue need not be checked at the bottom of the top flange. If fatigue needed to be checked, the lateral flange bending stress may be assumed equal to zero since the top flange is considered continuously braced after the deck has hardened (Article 6.10.1.6).

Check the base metal at the top of the bottom flange at this section adjacent to the transverse stiffener fillet weld to the flange for fatigue (Category C').

Compute the live load fatigue stress range due to vertical bending at the bottom of the web.

$$f_{bot\ web} = \frac{1640(71.93 - 1.5)}{290835}(12) = 4.77 \quad \text{ksi}$$

The lateral flange bending stress at the transverse stiffener must be considered since the stiffeners are welded to the tension flange. Assume that the transverse stiffeners are 6 in. wide. Compute the fatigue stress range at 6 in. from the center of the web due to lateral flange bending.

Compute the moment of inertia of the flange about a vertical axis in the plane of the web.

$$I_{flg} = \frac{1.5(22)^3}{12} = 1331 \quad \text{in}^4$$

Compute the range of lateral flange moment due to curvature according to Article 4.6.1.2.4.

$$M_{lat} = \frac{Ml^2}{NRD} = \frac{1640(20)^2}{10(717)(7)} = 13.07 \quad \text{k-ft} \qquad \text{Eq (4.6.1.2.4b-1)}$$

Compute the stress range due to lateral flange bending.

$$f_{fat} = \frac{13.07(6)}{1331}(12) = 0.71 \quad \text{ksi}$$

Total stress range = $4.77 + 0.71 = 5.48$ ksi

5.48 ksi < 6.0 ksi

Where 6 ksi is one-half of the Constant Amplitude Fatigue Threshold as given in Table 6.6.1.2.5-3 for Category C'.

Girder Stress Check Section 2-2 G4 Node 44
Longitudinally Stiffened Web - Bending Strength - Web

Check the web bend-buckling strength of the longitudinally stiffened web according to the provisions of Article C6.10.1.9.1. The longitudinal stiffener is located 18 inches from the top flange; d_s = 18 in.

Use the moments from Table C-1 and section properties from Table C-4. Compute the factored bending stress at the top and bottom of the web due to dead and live load plus the dynamic load allowance.

$$f_{top\ fl} = -\left[\frac{1.25(3343)(47.71)}{116890} + \frac{[1.25(510) + 1.5(583)]29.47}{214623} + \frac{1.75(5125)(14.57)}{290835}\right](12)(1) = -28.4 \text{ ksi (C)}$$

$$f_{bot\ fl} = \left[\frac{1.25(3343)(37.79)}{116890} + \frac{[1.25(510) + 1.5(583)]57.03}{214623} + \frac{1.75(5125)(71.93)}{290835}\right](12)(1) = 47.7 \text{ ksi (T)}$$

Calculate the effective D_c from the factored stresses in the top and bottom flanges. Assume the web is transversely stiffened as well.

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

$$= \left(\frac{|-28.4|}{|-28.4| + 47.7}\right)(1.0 + 84 + 1.5) - 1.0 = 31.28 \text{ in.}$$

Compute the nominal bend-buckling resistance according to Article 6.10.1.9.

$$\frac{d_s}{D_c} = \frac{18}{31.28} = 0.58 > 0.40, \text{ therefore, use Eq (6.10.1.9.2-1).}$$

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

$$k = \frac{5.17}{\left(\frac{18}{84}\right)^2} = 112.6 > \frac{9}{\left(\frac{31.28}{84}\right)^2} = 64.9 \qquad \text{therefore, k is taken as 112.6}$$

Eq (6.10.1.9.2-1)

$$F_{crw} = \frac{0.9(29000)(112.6)}{\left(\frac{84}{0.4375}\right)^2} = 79.7 \text{ ksi} > 1.0(50) = 50 \text{ ksi, therefore, } F_{crw} = 50 \text{ ksi}$$

$|-28.4|$ ksi < 50 ksi **OK**

Girder Stress Check Section 3-3 G4 Node 64
Longitudinally Stiffened Web - Bending Strength - Web

Try the longitudinal stiffener at 42 in. from the top flange; d_s = 42 in. Since longitudinal stiffeners are used, check the strength of the web at this section according to the provisions of Article 6.10.1.9.2.

Positive Live Load Bending Case

Use the moments from Table C-1 and the section properties from Table C-4. The composite section is assumed uncracked for this condition according to the provisions of Article 4.5.2.2.

Compute the factored bending stress in the top and bottom flanges due to dead and live load (Article C6.10.1.9.1).

$$f_{top\ fl} = -\left[\frac{1.25(1015)(48.71)}{116890} + \frac{[1.25(175) + 1.5(206)]29.47}{214623} + \frac{1.75(4338)(14.57)}{290835}\right](12)(1) = -11.8 \text{ ksi (C)}$$

$$f_{bot\ fl} = \left[\frac{1.25(1015)(37.79)}{116890} + \frac{[1.25(175) + 1.5(206)]57.03}{214623} + \frac{1.75(4338)(71.93)}{290835}\right](12)(1) = 29.1 \text{ ksi (T)}$$

Compute the effective D_c.

$$D_c = \left[\frac{-(-11.8)}{|-11.8| + 29.1}\right](1.0 + 84 + 1.5) - 1.0 = 23.96 \text{ in.} \qquad \text{Eq (D6.3.1-1)}$$

Compute the ratio of the stiffener distance from the compression flange, d_s, to the depth of the web in compression, D_c, to determine the method of computing k.

$$\frac{d_s}{D_c} = \frac{42}{23.96} = 1.75 > 0.40,$$ therefore, use Eq (6.10.1.9.2-1) to determine the bend-buckling coefficient, k.

$$k = \frac{5.17}{\left(\frac{d_s}{D}\right)^2} = \frac{5.17}{\left(\frac{42}{84}\right)^2} = 20.7 < \frac{9}{\left(\frac{D_c}{D}\right)^2} = \frac{9}{\left(\frac{23.96}{84}\right)^2} = 110.6 \qquad \text{Eq (6.10.1.9.2-1)}$$

The nominal bend-buckling resistance is taken as:

$$F_{crw} = \frac{0.9(29000)(110.6)}{\left(\frac{84}{0.4375}\right)^2} = 78.3 \text{ ksi} > 50 \text{ ksi, therefore, } F_{crw} = 50 \text{ ksi} \qquad \text{Eq (6.10.1.9.1-1)}$$

$|-11.3|$ ksi < 50 ksi **OK**

Girder Stress Check Section 3-3 G4 Node 64
Longitudinally Stiffened Web - Bending Strength - Web (continued)

Negative live load bending case

Longitudinal stiffener is located 42 in. from the top flange; d_s = 42 in. Check the strength of the web at this section for the negative live load bending case according to the provisions of Article 6.10.1.9.

Use the moments from Table C-1 and the section properties from Table C-4. The composite section is assumed cracked for this condition according to the provisions of Article 4.5.2.2.

The compressive deck stress due to the factored superimposed dead load is overcome by the tensile deck stress due to the factored live load plus the dynamic load allowance. Compute the factored bending stress in the top and bottom flanges due to dead and live load plus DLA (Article C6.10.1.9.1).

$$f_{top\ fl} = -\left[\frac{1.25(1015)(48.71)}{116890} + \frac{[1.25(175) + 1.5(206)]47.12}{124931} + \frac{1.75(-3080)(44.20)}{139710}\right](12)(1) = 11.7 \text{ ksi (T)}$$

$$f_{bot\ fl} = \left[\frac{1.25(1015)(37.79)}{116890} + \frac{[1.25(175) + 1.5(206)]39.37}{124931} + \frac{1.75(-3080)(42.30)}{139710}\right](12)(1) = -12.7 \text{ ksi (C)}$$

Compute the effective D_c.

$$D_c = \left[\frac{-(-12.7)}{|-12.7| + 11.7}\right](1.0 + 84 + 1.5) - 1.5 = 43.52 \quad \text{in.}$$

$$\frac{d_s}{D_c} = \frac{42}{43.52} = 0.97 > 0.40, \text{ therefore, use Eq (6.10.1.9.2-1).}$$

Compute k according to the provisions of Article 6.10.1.9.2.

$$k = \frac{5.17}{\left(\frac{42}{84}\right)^2} = 20.7 < \frac{9}{\left(\frac{43.52}{84}\right)^2} = 33.53 \quad \text{therefore, k = 33.58} \qquad \text{Eq (6.10.1.9.2-1)}$$

$$F_{crw} = \frac{0.9(29000)(33.53)}{\left(\frac{84}{0.4375}\right)^2} = 23.7 \text{ ksi} < R_h F_{yc} = 1.0(50) = 50 \text{ ksi}$$

$|-12.7|$ ksi < 23.7 ksi **OK**

Girder Stress Check Section 4-4 G4 Node 76
Longitudinally Stiffened Web - Bending Strength - Web

Positive live load bending case

Since this section is potentially subjected to stress reversal, check both positive and negative live load bending conditions. First, check the strength of the longitudinally stiffened web at this section for the positive live load bending case according to the provisions of Article 6.10.1.9.2.

Use the moments from Table C-1 and the section properties from Table C-4. The composite section is assumed uncracked for the positive live load moment according to the provisions of Article 4.5.2.2. Compute the factored bending stress in the top and bottom flanges due to dead and live load.

$$f_{top\ fl} = -\left[\frac{1.25(-1572)(48.71)}{116890} + \frac{[1.25(-200) + 1.5(-212)]29.47}{214623} + \frac{1.75(2906)(14.57)}{290835}\right](12) = 7.7 \quad \text{ksi (T)}$$

$$f_{bot\ fl} = \left[\frac{1.25(-1572)(37.79)}{116890} + \frac{[1.25(-200) + 1.5(-212)]57.03}{214623} + \frac{1.75(2906)(71.93)}{290835}\right](12) = 5.7 \quad \text{ksi (T)}$$

The section is entirely in tension. Therefore, web bend-buckling need not be checked for this case. A second longitudinal stiffener is not required.

Negative live load bending case

The longitudinal stiffener is located 56 inches from the top flange; d_s = 28 in. Check the strength of the web at this section for the negative live load bending case according to the provisions of Article 6.10.1.9.

$$f_{top\ fl} = -\left[\frac{1.25(-1572)(48.71)}{116890} + \frac{[1.25(-200) + 1.5(-212)]47.12}{124931} + \frac{1.75(-4243)(44.20)}{139710}\right](12) = 40.6 \quad \text{ksi (T)}$$

$$f_{bot\ fl} = \left[\frac{1.25(-1572)(37.79)}{116890} + \frac{[1.25(-200) + 1.5(-212)]39.37}{124931} + \frac{1.75(-4243)(42.30)}{139710}\right](12) = -36.7 \quad \text{ksi (C)}$$

Compute the effective D_c using the factored stresses in the top and bottom flanges.

$$D_c = \left[\frac{-(-36.7)}{|-36.7| + 40.6}\right](1.0 + 84 + 1.5) - 1.5 = 39.57 \quad \text{in.}$$

Compute the ratio of d_s to D_c to determine the method of computing k.

$$\frac{d_s}{D_c} = \frac{28}{39.57} = 0.71 > 0.40, \text{ therefore, use Eq (6.10.1.9.2-1).}$$

Girder Stress Check Section 4-4 G4 Node 76
Longitudinally Stiffened Web - Bending Strength - Web (continued)

$$k = \frac{5.17}{\left(\frac{28}{84}\right)^2} = 46.5 \quad > \quad \frac{9}{\left(\frac{39.57}{84}\right)^2} = 40.56 \qquad \text{Eq (6.10.1.9.2-1)}$$

$$F_{crw} = \frac{0.9(29000)(46.5)}{\left(\frac{84}{0.4375}\right)^2} = 32.92 \text{ ksi}$$

$$|-36.7| \text{ ksi} > 32.92 \text{ ksi} \ \textbf{NG}$$

To resolve this deficiency, begin the Section 5-5 beam section properties (see Appendix A) 6.83 ft. closer to Abutment 1 than is shown in Figure 2. Make this revision for all four girders.

Girder Stress Check Section 5-5 G4 Node 88
Longitudinally Stiffened Web - Bending Strength - Web

The longitudinal stiffener is located 66 inches from the top flange at this section; d_s = 18 in. Check the strength of the web at this section for the negative live load bending case according to the provisions of Article 6.10.1.9.2.

The composite section is assumed cracked for this condition according to the provisions of Article 4.5.2.2. Moments are from Table C-1 and section properties are from Table C-4.

$$f_{top\ fl} = -\left[\frac{1.25(-4879)(45.43)}{163684} + \frac{[1.25(-704) + 1.5(-753)]44.24}{170780} + \frac{1.75(-4855)(42.03)}{184047}\right](12) = 49.8 \text{ ksi (T)}$$

$$f_{bot\ fl} = \left[\frac{1.25(-4879)(41.32)}{163684} + \frac{[1.25(-704) + 1.5(-753)]42.50}{170780} + \frac{1.75(-4855)(44.72)}{184047}\right](12) = -49.2 \text{ ksi (C)}$$

Compute the effective D_c.

$$D_c = \left[\frac{-(-49.2)}{|-49.2| + 49.8}\right](1.25 + 84 + 1.5) - 1.5 = 41.61 \text{ in.} \qquad \text{Eq (D.6.3.1-1)}$$

Compute the ratio of d_s to D_c to determine the method of computing k.

$$\frac{d_s}{D_c} = \frac{18}{41.61} = 0.43 > 0.40, \text{ therefore, use Eq (6.10.1.9.2-1).}$$

$$k = \frac{5.17}{\left(\frac{18}{84}\right)^2} = 112.6 \quad > \quad \frac{9}{\left(\frac{41.61}{84}\right)^2} = 36.68 \qquad \text{Eq (6.10.1.9.2-1)}$$

$$F_{crw} = \frac{0.9(29000)(112.6)}{\left(\frac{84}{0.4375}\right)^2} = 79.7 \text{ ksi} > 50 \text{ ksi, therefore } F_{crw} = 50 \text{ ksi}$$

$|-49.2|$ ksi < 50 ksi **OK**

Girder Stress Check Section 6-6 G4 Node 100
Longitudinally Stiffened Web - Bending Strength - Web

The longitudinal stiffener is located 66 inches from the top flange at this section; d_s = 18 in. Check the strength of the web at this section for negative bending according to the provisions of Article 6.10.1.9.2.

The composite section is assumed cracked according to the provisions of Article 4.5.2.2. Moments are from Table C-1 and section properties are from Table C-4.

$$f_{top\ fl} = -\left[\frac{1.25(-9189)(47.18)}{314783} + \frac{[1.25(-1537) + 1.5(-1478)]46.47}{322084} + \frac{1.75(-6726)(45.09)}{336106}\right](12) = 46.8 \text{ ksi (T)}$$

$$f_{bot\ fl} = \left[\frac{1.25(-9189)(42.32)}{314783} + \frac{[1.25(-1537) + 1.5(-1478)]43.03}{322084} + \frac{1.75(-6726)(44.41)}{336106}\right](12) = -43.8 \text{ ksi (C)}$$

Compute D_c.

$$D_c = \left[\frac{-(-43.8)}{|-43.8| + 46.8}\right](2.5 + 84 + 3.0) - 3.0 = 40.27 \text{ in.} \qquad \text{Eq (D6.3.1-1)}$$

Compute the ratio of d_s to D_c to determine the method of computing k.

$$\frac{d_s}{D_c} = \frac{18}{40.27} = 0.45 \quad > 0.40, \text{ therefore, use Eq (6.10.1.9.2-1).}$$

$$k = \frac{5.17}{\left(\frac{18}{84}\right)^2} = 112.6 \quad > \quad \frac{9}{\left(\frac{40.27}{84}\right)^2} = 39.16 \qquad \text{Eq (6.10.1.9.2-1)}$$

$$F_{crw} = \frac{0.9(29000)(112.6)}{\left(\frac{84}{0.4375}\right)^2} = 79.7 \text{ ksi} > 50 \text{ ksi, therefore } F_{crw} = 50 \text{ ksi}$$

$|-43.8|$ ksi < 50 ksi **OK**

Girder Stress Check Section 6-6 G4 Node 100
Longitudinally Stiffened Web - Shear Strength - Web

Since the section has one longitudinal stiffener with a transverse stiffener spacing of 120 in. (approximately 1.5D), compute the shear strength at this section according to the provisions of Article 6.10.9.3. t_w = 0.4375 in.

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

Check the proportions of the interior panel with Eq (6.10.9.3.2-1).

$$\frac{2D\,t_w}{\left(b_{fc}\,t_{fc} + b_{ft}\,t_{ft}\right)} \le 2.5 \qquad \text{Eq (6.10.9.3.2-1)}$$

$$\frac{2(84)(0.4375)}{[\,28(3.0) + 29(2.5)\,]} = 0.47 \quad < \quad 2.5 \quad \textbf{OK}$$

Therefore, the nominal shear resistance 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:

$$V_p = 0.58F_{yw}Dt_w \qquad \text{Eq (6.10.9.3.2-3)}$$

$$= 0.58(50)(84)(0.4375) = 1066 \quad \text{k}$$

The intermediate transverse stiffener spacing, d_o = 120 in. Calculate the shear-buckling coefficient, k.

$$k = 5 + \frac{5}{\left(\frac{d_o}{D}\right)^2} = 5 + \frac{5}{\left(\frac{120}{84}\right)^2} = 7.45 \qquad \text{Eq (6.10.9.3.2-7)}$$

Determine which equation is to be used to compute C.

$$\frac{D}{t_w} = \frac{84}{0.4375} = 192$$

$$1.40\sqrt{\frac{E\ k}{F_{yw}}} = 1.40\sqrt{\frac{29000(7.45)}{50}} = 92 \quad < \quad 192$$

Girder Stress Check Section 6-6 G4 Node 100
Longitudinally Stiffened Web - Shear Strength - Web (continued)

Therefore, use Eq (6.10.9.3.2-6).

$$C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2}\left(\frac{E\,k}{F_{yw}}\right) = \frac{1.57}{\left(\frac{84}{0.4375}\right)^2}\left[\frac{29000(7.45)}{50}\right] = 0.184 \qquad \text{Eq (6.10.9.3.2-6)}$$

From Table C-2, the maximum factored shear within this field section is at this location, V_u = 604 k.

$$V_n = 1066\left[0.184 + \frac{0.87(1-0.184)}{\sqrt{1+\left(\frac{120}{84}\right)^2}}\right] = 630 \quad \text{k}$$

604 k < 1.0(630) = 630 k **OK**

Therefore, the intermediate transverse stiffener spacing of 120 in. is adequate for this section.

Girder Stress Check G4
Longitudinally Stiffened Web - Longitudinal Stiffener Design

Design the longitudinal stiffener according to the provisions of Article 6.10.11.3. Size the longitudinal stiffener assuming the actual transverse stiffener spacing, d_o, is equal to the cross-frame spacing. The longitudinal stiffener is located 18 in. from the compression flange.

The flexural stress in the longitudinal stiffener, f_s, due to the factored loads at the strength limit state and when checking constructibility must satisfy the following relation:

$$f_s \le \phi_f R_h F_{ys} \qquad \text{Eq (6.10.11.3.1-1)}$$

$$f_s = \frac{Mc}{I} \qquad \text{where: } c = \bar{y} - t_{fc} - d_s$$

Since the longitudinal stiffener yield strength is the same as the yield strength of the girder, this check is unnecessary. The girder elements satisfy the resistance equations, therefore, the longitudinal stiffener stress will be less than the limit given in Eq (6.10.11.3.1-1).

The projecting width, b_l, of the stiffener must satisfy Eq (6.10.11.3.2-1). Assume trial t_s = 0.75".

$$b_l \le 0.48 t_s \sqrt{\frac{E}{F_{ys}}} = 0.48(0.75)\sqrt{\frac{29000}{50}} = 8.67 \text{ in.} \qquad \text{Eq (6.10.11.3.2-1)}$$

The longitudinal stiffeners must satisfy Eqs (6.10.11.3.3-1 and -2):

$$I_l \ge D t_w^3 \left[2.4\left(\frac{d_o}{D}\right)^2 - 0.13\right]\beta \qquad \text{Eq (6.10.11.3.3-1)}$$

$$r \ge \frac{0.16 d_o \sqrt{\frac{F_{ys}}{E}}}{\sqrt{1 - 0.6\frac{F_{yc}}{R_h F_{ys}}}} \qquad \text{Eq (6.10.11.3.3-2)}$$

Since the stiffener is on the side of the web away from the center of curvature, use Eq (6.10.11.3.3-3) for the curvature correction factor for longitudinal stiffener rigidity, β.

$$\beta = \frac{Z}{6} + 1 \qquad \text{Eq (6.10.11.3.3-3)}$$

Where the curvature parameter, Z, is taken as:

$$Z = \frac{0.95 d_o^2}{R t_w} \le 10 \qquad \text{Eq (6.10.11.3.3-5)}$$

Girder Stress Check G4
Longitudinally Stiffened Web - Longitudinal Stiffener Design (continued)

$$Z = \frac{0.95(120)^2}{717(12)(0.4375)} = 3.63 \quad < \quad 10 \;\; \textbf{OK}$$

Therefore,

$$\beta = \frac{3.63}{6} + 1 = 1.61$$

The required moment of inertia of the longitudinal stiffeners, I_l, must be greater than or equal to:

$$84(0.4375)^3\left[2.4\left(\frac{120}{84}\right)^2 - 0.13\right](1.6) = 53.66 \quad in^4$$

According to the provisions of Article 6.10.11.3.2, consider a plate section acting with a web width equal to $18t_w$.

Try: one plate 8 in. x 0.75 in.

Compute the centroidal moment of inertia of the longitudinal stiffener.

First, determine the contributing area of the longitudinal stiffener and web.

$$A_{web} = 18(0.4375)^2 = 3.45 \quad in^2; \quad A_{ls} = 8(0.75) = 6 \quad in^2$$

$$A_{tot} = 3.45 + 6 = 9.45 \quad in^2$$

Next, determine the location of the neutral axis from the outside tip of the longitudinal stiffener.

$$\bar{y} = \frac{6(4.0) + 3.45(8.219)}{9.45} = 5.54 \quad in.$$

Compute the centroidal moment of inertia.

$$I_l = \frac{0.75(8)^3}{12} + 4.375(5.54 - 4.0)^2 + 3.45(8.219 - 5.54)^2 = 67.14 \quad in^4$$

$$67.14 \; in^4 \; > \; 53.66 \; in^4 \quad \textbf{OK}$$

Calculate the radius of gyration and check against Eq (6.10.11.3.3-2).

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

Girder Stress Check G4
Longitudinally Stiffened Web - Longitudinal Stiffener Design (continued)

$$r = \sqrt{\frac{67.14}{9.45}} = 2.67 \text{ in.}$$

$$2.67 \text{ in.} > \frac{0.16(120)\sqrt{\frac{50}{29000}}}{\sqrt{1 - 0.6\frac{50}{1.0(50)}}} = 1.261 \text{ in.} \quad \textbf{OK}$$

Therefore, the trial longitudinal stiffener of 8" x 0.75" satisfies the provisions given in the specifications.

Girder Stress Check G4 Spans 1 & 2
Longitudinally Stiffened Web - Transverse Stiffener Spacing

Calculations similar to those shown on page D-67 reveal that the following transverse stiffener spacings are satisfactory in Spans 1 and 2 for the longitudinally stiffened web design. The factored shears and stiffener spacings are graphically shown in Figure D-2 for Span 1 and in Figure D-3 for Span 2.

Span 1

Panels	Spacing (in.)	No. Transverse Stiffeners
1 - 8	120	8

Span 2

Panels	Spacing (in.)	No. Transverse Stiffeners
1 - 11	120	11

Total for girder G4
8(2) + 11 = 27

Transverse stiffeners used in web panels with longitudinal stiffeners must satisfy:

$$I_t \geq \left(\frac{b_t}{b_l}\right)\left(\frac{D}{3.0d_o}\right)I_l \qquad \text{Eq (6.10.11.1.3-3)}$$

The dimensions of the intermediate transverse stiffeners are 6.75 in. x 0.625 in. and the longitudinal stiffeners are 8 in. x 0.75 in. Both stiffeners are single plates. The moment of inertia of the transverse stiffeners, I_t, was calculated previously.

$$64.07 \text{ in}^4 \quad > \quad \frac{6.75}{8}\left[\frac{84}{3.0(120)}\right]67.14 = 13.2 \quad \text{in}^4 \quad \textbf{OK}$$

Bolted Splice Design Section 8-8 G4 Node 124
Design Action Summary and Section Information

Design the bolted field splice for the transversely stiffened girder at this section according to the provisions of Article 6.13.

Bolt capacities

Use 7/8 in. diameter A325 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 Critical Bolt Resistance (Article 6.13.2.1.1)

The factored resistance, R_r, of a bolt at the Service II Load Combination (Table 3.4.1-1) 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 shall be 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
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

$R_r = R_n = (1.0)(0.50)(2)(39) = 39$ 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 flanges at the point of the splice at the strength limit state must satisfy the applicable provisions of Article 6.10.6.2.

Shear Resistance (Article 6.13.2.7)

The nominal shear resistance, R_n, of a high-strength bolt at the strength limit state where 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. Eq (6.13.2.2-2)

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

Bearing Resistance (Article 6.13.2.9)

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 strength of the connected material specified in Table 6.4.1-1 (ksi)

In this case, the end distance is 1.5 in. creating a clear end distance of 1.0 in. which is less than 2.0d, therefore, Eq (6.13.2.9-2) applies. The nominal bolt resistance for the end row of bolts is:

$R_n = 1.2(1.0)(0.5625)(65) = 43.87$ k/bolt

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

$R_r = 0.8(43.87) = 35.1$ k/bolt

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

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

$= 2.4(0.875)(0.5625)(65) = 76.78$ k/bolt

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

Tensile Resistance (Article 6.13.2.10)

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

$T_n = 0.76A_bF_{ub}$

$T_n = 0.76(0.601)(120) = 54.8$ k/bolt Eq (6.13.10.2-1)

The tensile bolt strength is not used in this example.

Section 8-8 G4 Node 124 Bolted Splice
Constructibility - Top Flange

Constructibility

According to Article 6.13.6.1.4a the connections must be proportioned to prevent slip during the erection of the steel and during the casting of the concrete deck.

Since Cast #1 causes a larger negative 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 be considered in the design of curved girder splices. Since the flange is discretely braced for this case, lateral flange bending must be considered. 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 E-4, in terms of the bolt area.

$$I_p = A_b\left[2\,(4)\left(3.5^2 + 6.5^2\right) + 2(4)\left(1.5^2 + 4.5^2\right)\right] = 616A_b \text{ in}^4$$

where:

A_b = area of the bolt

Moment = $-382 + (-1910) = -2292$ k-ft (Table D-1)

Lateral flange moment = $2.9 + 17.8 = 20.7$ k-ft (Table D-1)

The factored vertical bending stresses are as follows

$$f_{top\ flg} = -\left[\frac{(-2292)(49.52)}{111989}\right](12)(1.25) = 15.2 \text{ ksi}$$

$$f_{top\ web} = -\left[\frac{(-2292)(48.52)}{111989}\right](12)(1.25) = 14.9 \text{ ksi}$$

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.

$$F_{top} = \left(\frac{15.2 + 14.9}{2}\right)(17.0) = 256 \text{ kips}$$

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

$$F_{Long\ vert} = \frac{256}{16} = 16 \text{ k/bolt}$$

Section 8-8 G4 Node 124 Bolted Splice
Constructibility - Top Flange (continued)

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

$$F_{Long\ lat} = \frac{20.7(6.5)}{616}(12)(1.25) = 3.28 \quad \text{k/bolt}$$

Therefore,

$$F_{Long\ tot} = 16 + 3.28 = 19.3 \quad \text{k/bolt}$$

Compute the transverse component of force in the critical bolt.

$$F_{Trans} = \frac{20.7(4.5)}{616}(12)(1.25) = 2.27 \quad \text{k/bolt}$$

Compute the resultant force on the critical bolt.

$$\Sigma_F = \sqrt{19.3^2 + 2.27^2} = 19.43 \quad \text{k/bolt}$$

Check $R_u \le R_r$

R_u = 19.43 k/bolt < R_r = 39 k/bolt **OK**

Section 8-8 G4 Node 124 Bolted Splice
Constructibility - Web

Article 6.13.6.1.4a directs the designer to check the bolted splice to prevent slip in the bolts during the constructibility stage. A pattern of two rows of 7/8 in. diameter bolts spaced vertically at 3.5 in. will be tried for the web splice. There are 46 bolts on each side of the web splice. The pattern is shown in Figure D-5. 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 flexural stress is relatively low) at the maximum specified spacing 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 3.5 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 the bolt group on one side of the connection in terms of the bolt area.

$$I_p = A_b[2(2)(3.5^2 + 7.0^2 + 10.5^2 + 14.0^2 + 17.5^2 + 21.0^2 + 24.5^2 + 28.0^2 + 31.5^2 + 35.0^2 + 38.5^2) + [46(1.5)^2] = 24{,}898A_b \text{ in}^4$$

An alternate equation to compute I_p is provided in Article C6.13.6.1.4b.

Constructibility

Compute the factored shear at the splice due to Steel plus Cast #1 and #2. The shears are taken from Table C-3 and Cast #2 is conservatively used in the calculation.

$$V = (27 + 7 + 92)(1.25) = 158 \text{ kips}$$

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-5).

$$M_v = V_e = 158\left(\frac{3}{2} + 2.125\right)\left(\frac{1}{12}\right) = 47.7 \quad \text{k-ft}$$

Determine the portion of the vertical bending moment resisted by the web, M_{uw}, and the horizontal force resultant in the web, H_{uw}, using the equations provided in Article C6.13.6.1.4b. M_{uw} and H_{uw} are assumed to be applied at the middepth of the web. Using the results from earlier calculations (page D-76), the average factored bending stress in the top flange for Steel plus Cast #1 is computed as:

$$F_{cf} = \left(\frac{15.2 + 14.9}{2}\right) = 15.05 \quad \text{ksi (T)}$$

The average factored vertical bending stress in the bottom flange is (see page D-80)

$$f_{ncf} = \left[\frac{-11.35 + (-10.89)}{2}\right] = -11.12 \quad \text{ksi (C)}$$

Section 8-8 G4 Node 124 Bolted Splice
Constructibility - Web (continued)

Using these stresses

$$M_{uw} = \frac{t_w D^2}{12}\left|R_h F_{cf} - R_{cf} f_{ncf}\right| = -\frac{0.5625(84)^2}{12}\left|1.0(15.05) - 1.0(-11.12)\right|\left(\frac{1}{12}\right) = -721 \quad \text{k-ft}$$

Eq (C6.13.6.1.4b-1)

$$H_{uw} = \frac{t_w D}{2}\left(R_h F_{cf} + R_{cf} f_{ncf}\right) = \frac{0.5625(84)}{2}[1.0(15.05) + 1.0(-11.12)] = 92.8 \quad \text{kips}$$

Eq (C6.13.6.1.4b-2)

The total moment on the web splice is computed as:

$$M_{tot} = M_v + \left|M_{uw}\right| = 47.7 + \left|-721\right| = 769 \quad \text{k-ft}$$

Compute the vertical bolt force due to the factored shear.

$$F_s = \frac{V}{N_b} = \frac{158}{46} = 3.43 \quad \text{k/bolt}$$

Compute the bolt force due to the horizontal force resultant.

$$F_H = \frac{H_{uw}}{N_b} = \frac{40.3}{46} = 0.88 \quad \text{k/bolt}$$

Compute the horizontal and vertical components of the force on the extreme bolt due to the total moment on the splice.

$$F_{Mv} = \frac{M_{tot}\, x}{I_p} = \frac{769(12)\left(\frac{3}{2}\right)}{24898} = 0.56 \quad \text{k/bolt}$$

$$F_{Mh} = \frac{M_{tot}\, y}{I_p} = \frac{769(12)(38.5)}{24898} = 14.27 \quad \text{k/bolt}$$

Compute the resultant bolt force.

$$F_r = \sqrt{\left(F_s + F_{Mv}\right)^2 + \left(F_H + F_{Mh}\right)^2} = \sqrt{(3.43 + 0.56)^2 + (0.88 + 14.27)^2} = 15.67 \quad \text{k/bolt}$$

$F_r = 15.67$ k/bolt $<$ $R_r = 39$ k/bolt **OK**

The preceding check is obviously conservative since the maximum factored moment after Cast #1 is assumed to be concurrent with the maximum factored shear after Cast #2.

Section 8-8 G4 Node 124 Bolted Splice
Constructibility - Bottom Flange

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

$$f_{bot\ flg} = \frac{(-2292)(36.98)}{111989}(12)(1.25) = -11.35 \quad \text{ksi}$$

$$f_{bot\ web} = \frac{(-2292)(35.48)}{111989}(12)(1.25) = -10.89 \quad \text{ksi}$$

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.

$$F_{bot} = \frac{-11.35 + (-10.89)}{2}(31.5) = -350 \quad \text{kips}$$

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

$$F_{Long\ vert} = \frac{350}{24} = 14.58 \quad \text{k/bolt}$$

Compute the polar moment of inertia of the bottom flange bolt pattern shown in Figure D-4a, in terms of the bolt area.

$$I_p = A_b\left[2(6)\left(1.5^2 + 4.5^2\right) + 2(4)\left(2.5^2 + 5.5^2 + 8.5^2\right)\right] = 1140A_b \quad \text{in}^4$$

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

$$F_{Long\ lat} = \frac{20.7(8.5)}{1140}(12)(1.25) = 2.32 \quad \text{k/bolt}$$

Therefore,

$$F_{Long\ tot} = 14.58 + 2.32 = 16.9 \quad \text{k/bolt}$$

Compute the transverse component of force in the critical bolt.

$$F_{Trans} = \frac{20.7(4.5)}{1140}(12)(1.25) = 1.23 \quad \text{k/bolt}$$

Compute the resultant force in the critical bolt.

$$\Sigma_F = \sqrt{16.9^2 + 1.23^2} = 16.94 \quad \text{k/bolt}$$

Check $R_u < R_r$.

R_u = 16.94 k/bolt < R_r = 39 k/bolt **OK**

Section 8-8 G4 Node 124 Bolted Splice
Service - Top and Bottom Flange

Bolted connections for flange splices are designed as slip-critical connections for the flange design force. As a minimum, for checking slip of the flange splice bolts, the design force for the flange under consideration must be taken as the Service II design stress, F_s, times the smaller gross flange area on either side of the splice. F_s is calculated using Eq (6.13.6.1.4c-5).

$$F_s = \frac{f_s}{R_h} \qquad \text{Eq (6.13.6.1.4c-5)}$$

where:

f_s = maximum flexural stress due to Load Combination Service II at the midthickness of the flange under consideration for the smaller section at the point of the splice (ksi)

R_h = 1.0 for homogeneous girders

Since the girder is homogeneous, $F_s = f_s$.

Factor the loads using the Service II Load Combination from Table 3.4.1-1

Negative live load bending case

$$f_{s,\ \text{top flg}} = -\left[\frac{1.0(-1967)(49.02)}{111989} + \frac{[\,1.0[-250 + (-237)]\,]47.49}{120277} + \frac{1.30(-2772)(44.66)}{135575}\right](12)(1) = 26.9 \quad \text{ksi (T)}$$

$$f_{s,\ \text{bot flg}} = \left[\frac{1.0(-1967)(36.23)}{111989} + \frac{[\,1.0[-250 + (-237)]\,]37.76}{120277} + \frac{1.30(-2772)(40.59)}{135575}\right](12)(1) = -22.4 \quad \text{ksi (C)}$$

Positive live load bending case

$$f_{s,\ \text{top flg}} = -\left[\frac{1.0(-1967)(49.02)}{111989} + \frac{[\,1.0[-250 + (-237)]\,]30.27}{213901} + \frac{1.30(2054)(15.20)}{296306}\right](12)(1) = 9.5 \quad \text{ksi (C)}$$

$$f_{s,\ \text{bot flg}} = \left[\frac{1.0(-1967)(36.23)}{111989} + \frac{[\,1.0[-250 + (-237)]\,]54.98}{213901} + \frac{1.30(2054)(70.05)}{296306}\right](12)(1) = -1.6 \quad \text{ksi (T)}$$

Include the force resultant in the bolt group due to the flange lateral bending stress. Apply only the noncomposite dead load lateral moment to the top flange since this moment is locked-in when the deck hardens. No other loads deflect the top flange in the transverse direction after the deck hardens since it acts as a diaphragm between girders. The lateral moment due to all loadings is applied to the bottom flange.

Section 8-8 G4 Node 124 Bolted Splice
Service - Top and Bottom Flange (continued)

Determine the lateral moments using Eq. (C4.6.1.2.4b-1).

$M_{lat\_NC} = \dfrac{-1967\,(20)^2}{10(717)(7)} = -15.68$ k-ft

$M_{lat\_LL-} = \dfrac{-2772\,(20)^2}{10(717)(7)} = -22.09$ k-ft

$M_{lat\_SupImp} = \dfrac{-250\,(20)^2}{10(717)(7)} = -1.99$ k-ft

$M_{lat\_LL+} = \dfrac{2054\,(20)^2}{10(717)(7)} = 16.37$ k-ft

$M_{lat\_FWS} = \dfrac{-237\,(20)^2}{10(717)(7)} = -1.89$ k-ft

Top flange

Determine the force on the critical fastener which is taken as the bolt farthest from the centroid of the bolt group. See Figure D-4b for location of the critical fastener in the top flange bolt group.

The torsional shear force for the critical fastener is calculated as follows:

$$\tau_{H,V} = \frac{Mr}{J}$$

where:

M = $1.0(-15.68)\,12 = -188.2$ k-in (the sign is not needed in these calculations)

r = the straight-line distance measured from the centroid of the bolt group to the fastener.
Using Figure D-4b, these distances are computed as:
r_1 = 7.91 in., r_2 = 6.67 in., r_3 = 5.70 in., r_4 = 3.81 in.
Using symmetry, the r values for the remaining 12 bolts are identical to these.

J = polar moment of inertia (disregard the individual torsional resistances of the bolts)

$= \Sigma r^2$

$= 4\left(7.91^2 + 6.67^2 + 5.70^2 + 3.81^2\right) = 616$ in^4

Therefore, the horizontal and vertical components of the torsional shear force are taken as:

$$\tau_H = \frac{188.2(6.50)}{616} = 1.99 \text{ ksi}$$

$$\tau_V = \frac{188.2(4.50)}{616} = 1.37 \text{ ksi}$$

Section 8-8 G4 Node 124 Bolted Splice
Service - Top and Bottom Flange (continued)

The total force on the critical bolt in the top flange splice including the controlling flange force can, therefore, be calculated.

$$\frac{F_s A_{top\_fl}}{N_b} = \frac{26.9(17)(1)}{16} = 28.58 \quad \text{k/bolt}$$

$$F_{crit.} = \sqrt{(28.58 + 1.99)^2 + (1.37)^2} = 30.6 \quad \text{k}$$

Bottom flange

Determine the force on the critical fastener which is taken as the bolt farthest from the centroid of the bolt group. See Figure D-4c for location of the critical fastener in the bottom flange bolt group. The calculations for the bottom flange are similar to the previous calculations for the top flange.

The torsional shear force for the critical fastener is calculated as follows:

$$\tau_{H,V} = \frac{Mr}{J}$$

where:

$$M = [\,1.0[-15.68 + (-1.99) + (-1.89)] + 1.30(-22.09)\,]12 = -579.3 \quad \text{k-in}$$

(the sign is not needed in these calculations)

r = Use Figure D-4c

r_1 = 9.62 in., r_2 = 8.63 in., r_3 = 7.11 in., r_4 = 5.70 in., r_5 = 5.15 in., r_6 = 2.92 in.

Using symmetry, the r values for the remaining 18 bolts are identical to these.

$$J = 4\left(9.62^2 + 8.63^2 + 7.11^2 + 5.70^2 + 5.15^2 + 2.92^2\right) = 1140 \quad \text{in}^4$$

Therefore, the horizontal and vertical components of the torsional shear force are taken as:

$$\tau_H = \frac{579.3(8.50)}{1140} = 4.32 \quad \text{ksi}$$

$$\tau_V = \frac{579.3(4.50)}{1140} = 2.29 \quad \text{ksi}$$

Section 8-8 G4 Node 124 Bolted Splice
Service - Top and Bottom Flange (continued)

The total force on the critical bolt in the bottom flange splice including the controlling flange force can, therefore, be calculated.

$$\frac{F_s A_{bot\_fl}}{N_b} = \frac{-22.4(21)(1.5)}{24} = -29.4 \quad \text{k/bolt}$$

$$F_{crit.} = \sqrt{(29.4 + 4.32)^2 + (2.29)^2} = 33.8 \quad \text{k}$$

Therefore, the bottom flange controls.

For slip-critical connections, the factored resistance, R_n, was calculated previously as 39 k/bolt.

$F_{crit.} = 33.8$ k < $R_n = 39$ k/bolt **OK**

Therefore, by including the effects of the flange lateral bending stress, the resultant force in the top and bottom flange bolts increase from 28.58 k/bolt to 30.6 k (7.07%) and 29.4 k/bolt to 33.8 k (14.97%), respectively.

Section 8-8 G4 Node 124 Bolted Splice
Strength - Top and Bottom Flange

Bolted splices are designed at the strength limit state to satisfy the requirements specified in Article 6.13.1. Where the section changes at the splice, the smaller of the two connected sections will be used.

The effective area of the top flange when it is in tension is computed using Eq (6.13.6.1.4c-2). A_n is calculated using the provisions of Article 6.8.3.

$$A_e = \left(\frac{\phi_u F_u}{\phi_y F_{yt}}\right) A_n \le A_g \qquad \text{Eq (6.13.6.1.4c-2)}$$

where:

4 bolts per row

$$A_n = [\,17.0 - 4\,(0.875 + 0.125)\,](1.0) = 13 \quad in^2$$

$$A_e = \left[\frac{0.8(65)}{0.95(50)}\right] 13 = 14.2 \quad in^2$$

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:

$$A_n = [\,21.0 - 6(0.875 + 0.125)\,]1.5 = 22.5 \quad in^2$$

Section properties computed using the effective bottom flange width are used to calculate the 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.

Section 8-8 G4 Node 124 Bolted Splice
Strength - Top and Bottom Flange (continued)

Using the effective section properties (from separate calculations), calculate the average factored bending stress in the top and bottom flange mid-thicknesses for both the positive and negative live load bending conditions. The provisions of Article 4.5.2.2 are followed to determine which composite section (cracked or uncracked) to use.

Negative live load bending case

$$F_{top\ flg} = -\left[\frac{1.25(-1967)(49.75)}{108575} + \frac{[1.25(-250) + 1.5(-237)]48.17}{117075} + \frac{1.75(-2772)(45.27)}{132743}\right](12) = 36.7 \text{ ksi (T)}$$

$$F_{bot\ flg} = \left[\frac{1.25(-1967)(35.50)}{108575} + \frac{[1.25(-250) + 1.5(-237)]37.08}{117075} + \frac{1.75(-2772)(39.98)}{132743}\right](12) = -29.7 \text{ ksi (C)}$$

Positive live load bending case

$$F_{top\ flg} = -\left[\frac{1.25(-1967)(49.75)}{108575} + \frac{[1.25(-250) + 1.5(-237)]48.17}{117075} + \frac{1.75(2054)(14.61)}{272106}\right](12) = 14.5 \text{ ksi (C)}$$

$$F_{bot\ flg} = \left[\frac{1.25(-1967)(49.75)}{108575} + \frac{[1.25(-250) + 1.5(-237)]37.08}{117075} + \frac{1.75(2054)(70.64)}{272106}\right](12) = -4.9 \text{ ksi (T)}$$

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 by 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 controlling flange, as defined below.

Separate calculations similar to subsequent calculations show that the negative live load bending case is critical. For this loading case, the top flange is the controlling flange since it has the largest ratio of the flexural stress to the corresponding flange stress. Splice plates and their connections on the controlling flange need to be proportioned to provide a minimum resistance taken as the design stress times the smaller effective flange area, A_e, on either side of the splice. Article 6.13.6.1.4c defines the design stress, F_{cf}, for the controlling flange as:

$$F_{cf} = \frac{\left|\frac{f_{cf}}{R_h}\right| + \alpha\phi_f F_{yf}}{2} \geq 0.75\alpha\phi_f F_{yf} \qquad \text{Eq (6.13.6.1.4c-1)}$$

f_{cf} is the maximum flexural stress due to the factored loads at the mid-thickness of the controlling flange at the splice. The hybrid factor R_h is taken as 1.0 since all plates have the same yield strength and α is taken as 1.0.

Section 8-8 G4 Node 124 Bolted Splice
Strength - Top and Bottom Flange (continued)

$$F_{cf} = \frac{\left|\frac{36.7}{1.0}\right| + 1.0(1.0)(50)}{2} = 43.35 \quad \text{ksi} \quad \text{(controls)}$$

$$0.75\alpha\phi_f F_{yf} = 0.75(1.0)(1.0)(50) = 37.5 \quad \text{ksi}$$

The area of the smaller flange is used to ensure that the design force does not exceed the strength of the smaller flange.

$$F_{cf}A_e = 43.35(14.2) = 616 \quad \text{kips} \quad \text{(tension)}$$

Splice plates and their connections on the noncontrolling flange at the strength limit state must be proportioned to provide a minimum resistance taken as the design stress, F_{cnf}, times the smaller of the effective flange area, A_e, on either side of the splice. F_{ncf} is calculated using Eq (6.13.6.1.4c-3).

$$F_{ncf} = R_{cf}\left|\frac{f_{ncf}}{R_h}\right| \geq 0.75\alpha\phi_f F_y \qquad \text{Eq (6.13.6.1.4c-3)}$$

where:

$$R_{cf} = \frac{F_{cf}}{f_{cf}} = \frac{43.35}{36.7} = 1.181$$

f_{ncf} = factored bending stress in the noncontrolling flange at the splice concurrent with f_{cf}.

R_h = 1.0

$$F_{ncf} = 1.181\left|\frac{-29.7}{1.0}\right| = 35.08 \quad \text{ksi}$$

or

$$= 0.75(1.0)(1.0)(50) = 37.5 \quad \text{ksi} \quad \text{(this value is used as the minimum)}$$

The minimum design force for the noncontrolling flange, $F_{ncf}A_e$, is computed as:

$$F_{ncf}A_e = 37.5(21.0)(1.5) = 1181 \quad \text{kips} \quad \text{(compression)}$$

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.

Section 8-8 G4 Node 124 Bolted Splice
Strength - Top and Bottom Flange (continued)

Top Flange

Try a 17" x 0.5" outside plate and 2 - 7" x 0.625" inner plates.

Since the required fill plate for the top flange splice is 1/4 in. thick, in lieu of extending or developing the filler plates, a reduction in the bolt design shear strength is applied to the factored resistance of the bolts in shear at the strength limit state per Article 6.13.6.1.5.

$$R = \frac{(1+\gamma)}{(1+2\gamma)} \qquad \text{Eq (6.13.6.1.5-1)}$$

where:

A_f = sum of the area of the fillers on top and bottom of the connected plates (in^2)

$= 17(0.25) = 4.25 \quad in^2$

A_p = smaller of either the connected plate area or the sum of the splice plate areas on the top and bottom of the connected plate (in^2)

Top flange splice plates (17" x 0.5", 2 - 7" x 0.625"), Total area = 17.25 in^2

Connected plates (17" x 1.0"), Total area = 17.0 in^2 (minimum)

$$\gamma = \frac{A_f}{A_p} = \frac{4.25}{17.0} = 0.25$$

$$R = \frac{(1+0.25)}{[1+2(0.25)]} = 0.83$$

Therefore, reduce the bolt design shear strength by 0.83 for the strength check only.

Lateral flange bending is not considered in the top flange after the deck has hardened and the flange is continuously braced. Therefore:

$$\text{No. bolts req'd} = \frac{F_{cf} A_e}{R(R_r)} = \frac{616}{0.83(55.4)} = 13.4 \quad \text{bolts, use 16 bolts}$$

$$\frac{616}{16} = 38.5 \quad \text{k/bolt} \quad < \quad R(R_r) = \quad 0.83(55.4) = 45.98 \quad \text{k/bolt} \quad \textbf{OK}$$

Section 8-8 G4 Node 124 Bolted Splice
Strength - Top and Bottom Flange (continued)

Bottom Flange

For the bottom flange, lateral flange bending must be considered since the flange is discretely braced. For the following calculation, the dead and live load values have been taken directly from the analysis.

$$M_{lat} = 1.25[-2.9 + (-12.1) + (-1.89)] + 1.5(-1.81) + 1.75(-16.8) = -53.2 \quad \text{k-ft}$$

The lateral moment is then factored up by R_{cf} to be consistent with the computation of F_{cf} and $F_{ncf}A_e$.

$$M_{lat} = -53.2(1.181) = -62.8 \quad \text{k-ft}$$

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

$$F_{Long\ lat} = \frac{62.8(8.5)}{1140}(12) = 5.62 \quad \text{k/bolt}$$

Compute the force in each bolt due to the minimum design force, $F_{ncf}A_e$.

$$F_{Long} = \frac{1181}{24} = 49.21 \quad \text{k/bolt}$$

Therefore,

$$F_{Long\ tot} = 5.62 + 49.21 = 54.83 \quad \text{k/bolt}$$

Compute the transverse component of force in the critical bolt.

$$F_{Trans} = \frac{62.8(4.5)}{1140}(12) = 2.97 \quad \text{k/bolt}$$

Compute the resultant force on the critical bolt.

$$\Sigma_F = \sqrt{54.83^2 + 2.97^2} = 54.91 \quad \text{k/bolt;}$$

R_u = 54.91 k/bolt < R_r = 55.4 k/bolt **OK**

Note that a fill plate is not required for the bottom flange splice. Therefore, no reduction in the bolt design shear strength is necessary.

Section 8-8 G4 Node 124 Bolted Splice
Strength - Web

Determine the design shear, V_{uw}, for the web splice according to the provisions of Article 6.13.6.1.4b.

The factored shear at the splice is computed as:

$$V_u = 1.25(27 + 112 + 22) + 1.5(19) + 1.75(139) = 473 \text{ kips}$$

The shear resistance of the 0.5625 in. thick web at the splice (the smaller web) was determined in separate calculations similar to those on page D-42 according to the provisions of Article 6.10.9.1 as:

$$V_u \le \phi_v V_n = 617 \text{ kips} \qquad \text{Eq (6.10.9.1-1)}$$

$$0.5\phi_v V_n = 0.5(617) = 309 \text{ kips} < 473 \text{ kips}$$

Therefore, according to Article 6.13.6.1.4b, since $V_u > 0.5\phi_v V_n$:

$$V_{uw} = \frac{(V_u + \phi_v V_n)}{2} = \frac{(473 + 617)}{2} = 545 \text{ kips} \qquad \text{Eq (6.13.6.1.4b-2)}$$

Check that the design shear does not exceed the lesser of the factored shear resistance of the web splice plates specified in Article 6.13.4 or the factored shear resistance of the web splice plates specified in Article 6.13.5.3.

According to Article 6.13.4 the factored resistance of the combination of parallel and perpendicular planes is taken as:

If $A_{tn} \ge 0.58A_{vn}$ then:

$$R_r = \phi_{bs}(0.58F_yA_{vg} + F_uA_{tn}) \qquad \text{Eq (6.13.4-1)}$$

Otherwise:

$$R_r = \phi_{bs}(0.58F_uA_{vn} + F_yA_{tg}) \qquad \text{Eq (6.13.4-2)}$$

$$A_{tn} = 2[12.125 - 3.5(0.875 + 0.125)](0.375) = 6.47 \text{ in}^2$$

$$A_{vn} = 2[79 - 22.5(0.875 + 0.125)](0.375) = 42.38 \text{ in}^2$$

A_{tn} is less than $0.58A_{vn}$, therefore, use Eq (6.13.4-2) for R_r

$$A_{tg} = 2(12.125)(0.375) = 9.1 \text{ in}^2$$

$$R_r = \phi_{bs}(0.58F_uA_{vn} + F_yA_{tg})$$

$$R_r = 0.8[0.58(65)(42.38) + 50(9.1)] = 1642 \text{ k}$$

Section 8-8 G4 Node 124 Bolted Splice
Strength - Web (continued)

For connection elements in shear, the factored resistance according to Article 6.13.5.3 is taken as:

$$R_r = \phi_v R_n \qquad \text{Eq (6.13.5.3-1)}$$

$$R_n = 0.58 A_g F_y \qquad \text{Eq (6.13.5.3-2)}$$

$$A_g = 2(80)(0.375) = 60 \quad in^2$$

$$R_n = 0.58(60)(50) = 1740 \quad k$$

$$R_r = (1.0)(1740) = 1740 \quad k$$

Therefore, the lesser of the factored shear resistances is R_r = 1642 k > V_{uw} = 545 k **OK**

The moment, M_{uv}, due to the eccentricity of V_{uw} from the centerline of the splice to the centroid of the web splice bolt group is computed from Article 6.13.6.1.4b as follows (refer to web bolt pattern in Figure D-5):

$$M_{uv} = V_{uw} e$$

$$M_{uv} = 545\left[\left(\frac{3}{2}\right) + 2.125\right]\left(\frac{1}{12}\right) = 165 \quad \text{k-ft}$$

Determine the portion of the vertical bending moment resisted by the web, M_{uw}, and the horizontal design force resultant in the web, H_{uw}, according to the provisions of Article C6.13.6.1.4b. M_{uw} and H_{uw} are assumed to act at the middepth of the web. Separate calculations indicate that the negative live load bending condition controls.

As computed earlier (page D-86) for the negative live load bending case:

f_{cf} = 36.7 ksi

F_{cf} = 43.35 ksi

f_{ncf} = -29.7 ksi

R_{cf} = 1.181

From the equations in Article C6.13.6.1.4b:

$$M_{uw} = \frac{t_w D^2}{12}\left|R_h F_{cf} - R_{cf} f_{ncf}\right| \qquad \text{Eq (C6.13.6.1.4b-1)}$$

$$= -\frac{0.5625(84)^2}{12}\left|1.0(43.35) - 1.181(-29.7)\right|\left(\frac{1}{12}\right) = -2162 \quad \text{k-ft}$$

Section 8-8 G4 Node 124 Bolted Splice
Strength - Web (continued)

$$H_{uw} = \frac{t_w D}{2}\left(R_h F_{cf} + R_{cf} f_{ncf}\right) = \frac{0.5625(84)}{2}[\,1.0(43.35) + 1.181(-29.7)\,] = 195 \quad \text{kips}$$

Eq (C6.13.6.1.4b-2)

The total moment on the web splice is computed as:

$$M_{tot} = M_{uv} + M_{uw} = 165 + \left|-2162\right| = 2327 \quad \text{k-ft}$$

Compute the vertical bolt force due to the design shear.

$$F_s = \frac{V_{uw}}{N_b} = \frac{545}{46} = 11.85 \quad \text{k/bolt}$$

Compute the bolt force due to the horizontal design force resultant.

$$F_H = \frac{H_{uw}}{N_b} = \frac{195}{46} = 4.24 \quad \text{k/bolt}$$

Compute the horizontal and vertical components of the force on the extreme bolt due to the total moment on the splice.

$$F_{Mv} = \frac{M_{tot}\,x}{I_p} = \frac{2327(12)\left(\frac{3}{2}\right)}{24898} = 1.68 \quad \text{k/bolt}$$

$$F_{Mh} = \frac{M_{tot}\,y}{I_p} = \frac{2327(12)(38.5)}{24898} = 43.18 \quad \text{k/bolt}$$

Compute the resultant bolt force.

$$F_r = \sqrt{\left(F_S + F_{Mv}\right)^2 + \left(F_H + F_{Mh}\right)^2} = \sqrt{(11.85 + 1.68)^2 + (4.24 + 43.18)^2} = 49.31 \quad \text{k/bolt}$$

$R_r = \phi_s R_n$ Eq (6.13.2.2-2)

$F_r = 49.31$ k/bolt $<$ $R_r = 55.4$ k/bolt **OK**

Section 8-8 G4 Node 124 Bolted Splice
Splice Plates

Web Splice Plate Design

Use nominal 0.375 in. thick splice plates. As permitted in Article 6.13.6.1.5, a fill plate is not included since the difference in thickness of the web plates on either side of the splice is only 1/16 in.

According to Article 6.13.2.6.2, the maximum permissible spacing of the bolts for sealing:

$$s \leq 4 + 4t \leq 7.0 \quad = \quad 4 + 4(0.375) = 5.5 \text{ in. } \textbf{OK} \qquad \text{Eq (6.13.2.6.2-1)}$$

Check bearing of the bolts on the connected material assuming the bolts have slipped and gone into bearing. Since the bearing strength of the web controls, the bearing strength 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 strength 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 strength based on the inclined distance, or else resolve the resultant force in the direction parallel to the edge distance. In cases where the bearing strength of the web splice plate controls, the smaller of the clear edge or end distance on the splice plates can be used to compute the bearing strength of the outermost hole.

The clear distance between the edge of the hole and the edge of the field piece is computed as:

$$L_c = 2.0 - \frac{1.0}{2} = 1.5 \text{ in.} \quad < \quad 2.0d = 2.0(0.875) = 1.75 \text{ in.}$$

According to Article 6.13.2.9, the bearing strength at the bolt holes is computed as:

$$R_n = 1.2L_ctF_u = 1.2(1.5)(0.5625)(65) = 65.81 \text{ k/bolt} \qquad \text{Eq (6.13.2.9-2)}$$

see Table 6.4.1-1 for F_u values

$$R_r = \phi_{bb}R_n$$

$$R_r = (0.8)(65.81) = 52.6 \text{ k/bolt}$$

The maximum force on the extreme bolt was computed earlier (page D-92) for strength as:

$$F_r = 49.31 \text{ k/bolt} \quad < \quad R_r = 52.6 \text{ k/bolt} \quad \textbf{OK}$$

Check for flexural yielding on the gross section of the web splice plates at the strength limit state. The flexural stress is limited to $\phi_f F_y$.

$$\text{Web splice plate length} = 22(3.5) + 2(1.5) = 80 \text{ in.}$$

$$S_{PL} = \frac{2(0.375)(80)^2}{6} = 800 \text{ in}^3$$

Section 8-8 G4 Node 124 Bolted Splice
Splice Plates (continued)

$$f_g = \frac{M_{uv} + M_{uw}}{S_{PL}} + \frac{H_{uw}}{A_g} = \frac{(165 + |-2162|)(12)}{800} + \frac{195}{60} = 38.16 \text{ ksi} < \phi_f F_y = 1.0(50) = 50 \text{ ksi} \quad \textbf{OK}$$

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

The width of the outside splice plate should be at least as wide as the width of the narrowest flange at the splice.

Top Flange

17 in. x 0.5 in. outer plate
$A_g = 8.50 \text{ in}^2$

2 - 7 in. x 0.625 in. inner plates
$A_g = 8.75 \text{ in}^2$

The effective area for the tension flange, A_e, of each splice plate as specified in Article 6.13.6.1.4c is to be sufficient to prevent yielding of each splice plate under its calculated portion of the minimum flange design force.

The effective areas of the outer and inner splice plates are computed as:

$$A_e = \left(\frac{\phi_u F_u}{\phi_y F_{yt}}\right) A_n \leq A_g \qquad \text{Eq (6.13.6.1.4c-2)}$$

$$\text{Outer} = \left[\frac{0.8(65)}{0.95(50)}\right](6.5) = 7.12 \text{ in}^2 \qquad A_n = [17 - 4(0.875 + 0.125)]0.5 = 6.5 \text{ in}^2$$

$$\text{Inner} = \left[\frac{0.8(65)}{0.95(50)}\right](6.25) = 6.84 \text{ in}^2 \qquad A_n = [2(7) - 4(0.875 + 0.125)]0.625 = 6.25 \text{ in}^2$$

As specified in Article C6.13.6.1.4c, if the areas of the inner and outer splice plates are within 10 percent, then the flange design at the strength limit state may be divided equally to the inner and outer plates and their connections. Double shear may then be assumed in designing the bolts. If the areas differ by more than 10 percent, the flange design force is to be proportioned to the inner and outer plates by the ratio of the area(s) of the splice plate under consideration to the total area of the splice plates. In this case, the shear strength of the bolts would be checked assuming the maximum calculated splice plate force acts on a single shear plane.

The top flange is subjected to tension. The minimum design force, $F_{cf}A_e$, for the top flange was computed earlier (page D-87) to be 616 kips. Lateral flange bending need not be considered in the top flange after the deck has hardened. The capacity of the splice plates to resist tension is computed according to Article 6.8.2.1.

Section 8-8 G4 Node 124 Bolted Splice
Splice Plates (continued)

The factored tensile resistance, P_r, is taken as the lesser of the values given by Eqs (6.8.2.1-1 and -2).

$P_r = \phi_y P_{ny} = \phi_y F_y A_g$ (yielding on the gross section)) Eq (6.8.2.1-1)

$P_r = 0.95(50)(8.50 + 8.75) = 819$ k

or

$P_r = \phi_u P_{nu} = \phi_u F_u A_n U$ (fracture on the net section) Eq (6.8.2.1-2)

where:

$A_n = [\,17 - 4(0.875 + 0.125)\,](0.5) + [\,2(7) - 4(0.875 + 0.125)\,](0.625) = 12.75$ in

$P_r = 0.80(65)(12.75)(1.0) = 663$ k > 616 k Controlling Flange is **OK**

Verify that block shear rupture does not control over the fracture on the net section according to Article 6.13.4. The factored resistance of the combination of parallel and perpendicular planes is taken as:

If $A_{tn} \geq 0.58A_{vn}$ then:

$R_r = \phi_{bs}(0.58F_y A_{vg} + F_u A_{tn})$ Eq (6.13.4-1)

Otherwise,

$R_r = \phi_{bs}(0.58F_u A_{vg} + F_y A_{tg})$ Eq (6.13.4-2)

Calculate the variables for the above equations.

$A_{tn} = [\,13 - 3(1.0)\,]0.5 + 2(3.0 - 1.0)\,0.625 = 7.5$ in^2

$A_{vn} = [\,11 - 3.5(1.0)\,] \times [\,2(0.5) + 4(0.625)\,] = 26.25$ in^2

$A_{tg} = 13(0.5) + 2(3)(0.625) = 10.25$ in^2

Since,

7.5 in^2 < $0.58(26.25) = 15.22$ in^2

Use Eq (6.13.4-1) for calculation of the factored resistance.

$R_r = 0.8[\,0.58(65)(26.25) + 50(11.25)\,] = 1242$ k > $R_u = 616$ k **OK** Eq (6.13.4-1)

Section 8-8 G4 Node 124 Bolted Splice
Splice Plates (continued)

Check the noncontrolling flange; in this case it is the bottom flange. $F_{ncf}A_e$ = 1,181 kips (compression) from page D-87. According to Article 6.13.6.1.4c, flange splice plates subjected to compression at the strength limit state are to be checked only for yielding on the gross section of the plates according to Eq (6.13.6.1.4c-4).

Bottom Flange

Try: 21 in. x 0.75 in. outer plate
$A_g = 15.75 \text{ in}^2$

Try: 2 - 9.5 in. x 0.875 in. inner plates
$A_g = 16.63 \text{ in}^2$

$R_r = \phi_c F_y A_s$ Eq (6.13.6.1.4c-4)

$A_s = 21(0.75) + 2(9.5)(0.875) = 32.375 \text{ in}^2$

$R_r = 0.9(50)(32.375) = 1457$ k > 1,181 k Noncontrolling Flange is **OK**

Bearing Resistance at Bolt Holes

Check bearing of the bolts on the connected material under the minimum design force. The design bearing strength, R_n, is computed using the provisions of Article 6.13.2.9. By inspection, the bottom flange governs the bearing strength of the connection.

According to specifications, the bearing strength 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 strength (where "d" is the diameter of the bolt).

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 edge of the splice plate, the edge 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).

$R_n = 1.2L_c t F_u = 1.2(1.0)(1.5)(65) = 117$ k/bolt Eq (6.13.2.9-2)

$F_{ncf}A_e$ = 1181 k < 16 bolts(117) = 1872 k **OK**

Section 8-8 G4 Node 124 Bolted Splice
Splice Plates (continued)

Since the splice plates are on a partially braced flange and subjected to compression, check for yielding on the gross section of the splice plates under their portion of the minimum design force, $F_{ncf}A_e$, plus the factored lateral flange bending moment.

The minimum flange design force, $F_{ncf}A_e$, was computed earlier (page D-87) to be 1,181 kips (compression). The factored lateral flange moment for strength was computed earlier (page D-89) to be -62.8 k-ft.

The lateral section modulus of the inner and outer splice plates together is:

$$S = \frac{(0.75 + 0.875)(21)^2}{6} - \frac{(0.875)(2)^2}{6} = 118.9 \ \text{in}^3$$

Check for flexural yielding on the gross section of the flange splice plates at the strength limit state. The flexural stress is limited to $\phi_f F_y$.

$$f_g = \frac{1181}{(15.75 + 16.63)} + \frac{62.8(12)}{118.9} = 42.81 \quad \text{ksi}$$

$$42.81 \text{ ksi} < \phi_f F_y = 1.0(50) = 50 \text{ ksi} \quad \textbf{OK}$$

If the difference in area of the inner splice plates had not been within 10 percent of the area of the outside splice plate, the minimum design force factored moment would then be proportioned to the inner and outer splice plates accordingly (Article C6.13.6.1.4c).

Separate calculations similar to those illustrated previously show that bearing of the bolts on the top flange is not critical.

Girder Stress Check G4 Node 99-100
Strength - Cross-Frame Diagonal

Evaluation of the cross-frame results shows that the diagonal member between G4 and G3 at Node 100 has the largest force. The largest factored load of the Load Combinations examined is -88 kips (compression). Compression members are design according to Article 6.9. According to Article 6.7.4.1, cross-frames in horizontally curved bridges are considered primary members.

Determine the effective length of the diagonal.

$$l = \sqrt{11^2 + 7^2} = 13 \text{ ft.}$$

Try: 1 - L8x6x9/16; F_y = 50 ksi; r_{xx} = 2.55 in.; r_{yy} = 1.78 in.; r_{zz} =1.30 in.; A_s = 7.56 in^2 (from the AISC LRFD Manual of Steel Construction)

Check the b/t provision of Article 6.9.4.2 for the cross-frame diagonal member.

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

where:

k = plate buckling coefficient, 0.45 for other projecting elements from Table 6.9.4.2-1.

b = the full width of the outstanding leg for a single angle (in.)

t = plate thickness (in.)

$$\frac{6}{0.625} = 9.6 \quad < \quad 0.45\sqrt{\frac{29000}{50}} = 10.8 \qquad \textbf{OK}$$

Check the limiting slenderness ratio of Article 6.9.3.

$$\frac{Kl}{r} \le 120$$

where: K = effective length factor specified in Article 4.6.2.5 as 1.0 for single angles regardless of end connection (in.)

l = unbraced length (in.)
r = minimum radius of gyration (in.)

$$\frac{1.0(13)(12)}{1.30} = 120 \quad = \ 120$$

In an actual design, an additional iteration would be necessary since the cross-frame member area in the model was 5.0 in^2 and the design area is 7.56 in^2. Since the cross-frames are truss members in the 3D analysis, the area of the cross-frame elements affects the structure rigidity which in-turn alters the girder moments and shears.

Girder Stress Check G4 Node 99-100
Strength - Cross-Frame Diagonal (continued)

Since Eq (6.9.4.2-1) is satisfied, the factored resistance of components in compression, P_r, can be taken as:

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

where:

P_n = nominal compressive resistance determined using the provisions of Article 6.9.4.1.

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

$$= \left[\frac{1.0(13)(12)}{1.30\pi}\right]^2 \frac{50}{29000} = 2.52$$ $>$ 2.25 therefore, use Eq (6.9.4.1-2) for P_n

$$P_n = \frac{0.88 F_y A_s}{\lambda} = \frac{0.88(50)(7.56)}{2.52} = 132 \text{ k} \qquad \text{Eq (6.9.4.1-2)}$$

ϕ_c = 0.9 from Article 6.5.4.2

$P_r = 0.9(132) = 119$ k $>$ 88 k **OK**

Girder Stress Check G4 Node 99-100
Diagonal - Strength and Connection

A more rigorous design of an eccentrically loaded single-angle member may be performed utilizing the AISC Specification for Load and Resistance Factor Design of Single-Angle Members contained in the Second Edition of the AISC LRFD Manual of Steel Construction.

Check fatigue of the member assuming the diagonal is connected to a gusset plate with fillet welds. The range of force in this member due to the factored fatigue loading is 7 kips. However, there are other members with a range of force of 17 kips. The larger range will be used for the fatigue design.

Table 3.4.1-1 requires that a factor of 0.75 be applied to the range for checking fatigue.

$17(0.75) = 12.75$ kips

Fatigue stress range = $\frac{12.75}{7.56} = 1.69$ ksi

Material adjacent to fillet discontinuous welds is Category E.

1.69 ksi < 2.25 ksi permitted 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. This value is used whenever the fatigue strength is governed by the constant-amplitude fatigue threshold, which is assumed to be the case here.

According to Article 6.13.3.2.4, the factored resistance for fillet-welded connections subjected to tension or compression parallel to the axis of the weld is taken as the factored resistance of the base metal. Assume a 5/16" fillet weld.

Weld resistance = $0.45(0.707)(0.3125)(70) = 7$ k/in.

Required length of weld = $\frac{88}{7} = 12.57$ in.

Check fatigue due to the shear stress on the throat of the weld metal, which is also checked for Category E according to Table 6.6.1.2.3-1.

12.75 kips/12.57 in. = 1.014 k/in.

1.014 k/in. / [0.707(0.3125 in.)] = 4.59 ksi > 2.25 ksi

Recalculate the necessary weld length.

$2.25(0.707)(0.3125) = 0.5$ kips/in.

Length of weld = $\frac{12.75}{0.5} = 25.5$ in.

Girder Stress Check G4 Node 99-100
Diagonal - Strength and Connection (continued)

In order to minimize eccentricity, the welds should be proportioned on each side of the angle leg such that the resultant force is along the neutral axis of the angle. The neutral axis is 2.50 in. from the heel. The angle will be connected along the 8 in. leg. Compute the required length of weld along the heel, l_1. l_2 is the required length of weld along the connected leg.

$$l_1(2.50) = l_2(8 - 2.50) \qquad \text{Eq (1)}$$

$$l_2 = 25.5 - l_1 \qquad \text{Eq (2)}$$

Substituting l_2 into Eq (1) and solving for l_1,

$$l_1 = (25.5 - l_1)5.5/2.50$$

$$l_1 = 17.53 \text{ in.}; \quad l_2 = 25.5 - 17.53 = 7.97 \text{ in.}$$

The gusset plate should be of at least the same thickness as the angle and have at least the same equivalent net area. The gusset plate is bolted to the connection plate, which is welded to the girder web and flanges. The angle diagonal is attached near the bottom of G4. The bottom chord carries 40 kips out of the connection so the resultant force is approximately 48 kips (88-40) that is transferred into the girder through bolts. Since the minimum number of bolts in a connection is two, there is no need to examine the bolt design. Welds between the connection plate and bottom flange must be able to transfer 48 kips of shear. Since this location is at a bearing, the welds are deemed to be adequate.

Centrifugal Force Calculations

The centrifugal force is to be determined according to Article 3.6.3. The radial component of the centrifugal force is assumed to be transmitted from the deck through the end cross-frames or diaphragms and the bearings to the substructure.

Centrifugal force also causes an overturning effect. The center of gravity of the design truck is assumed to be 6 ft. above the roadway (Article 3.6.3). The spacing of the wheels is 6 ft. (Figure 3.6.1.2.2-1). Figure D-7 shows the relationship between the centrifugal force and superelevation effect graphically. An overturning effect occurs because the radial component of centrifugal force is applied 6 ft. above the top of the deck.

The overturning component causes the exterior (with respect to curvature) wheel line to be more than half the weight of the truck and the interior wheel line to be less than half the weight of the truck by the same amount. Thus, the outside of the bridge is more heavily loaded. The overturning force must be adjusted for the roadway superelevation.

Article 3.6.3 states that the centrifugal force shall be taken as the product of the axle weights of the design truck or tandem and the factor C, taken as:

$$C = f\frac{v^2}{gR} \qquad \text{Eq (3.6.3-1)}$$

where:

f = 4/3 for load combinations other than fatigue, 1.0 for fatigue

v = highway design speed (ft/s)

g = gravitational acceleration: 32.2 ft/s^2

R = radius of curvature of traffic lane (ft.)

Use an average bridge radius in this case, R = 700 ft.

$$C\ (\%) = \frac{4}{3}\left(\frac{v^2}{32.2R}\right) = 0.0414\left(\frac{v^2}{R}\right)$$

Design speed = 35 mph (51.3 ft/s)

$$C\ (\%) = 0.0414\left(\frac{51.3^2}{700}\right) \times 100 = 15.6$$

Apply the factor C to the axle weights.

Centrifugal Force Calculations (continued)

Compute the radial force.

Truck in one lane = $1.2(0.156)(72) = 13.48$ kips

Truck in two lanes = $1.0(0.156)(72)(2) = 22.464$ kips

Truck in three lanes = $0.85(0.156)(72)(3) = 28.642$ kips

All three cases have been factored according to the multiple presence factors give in Table 3.6.1.1.2-1.

The centrifugal force due to trucks in two lanes is used since the two lanes loaded case controls for the vertical bending. The force will be applied to the deck in the radial direction. The force is resisted by the shear in the deck and is transferred to the bearings through the cross-frames at the bearings. Other cross-frames are not affected by the radial force.

Compute the overturning force.

Sum the moments about the inside wheel.

The 5 percent cross slope of the deck must be taken into account.
Vehicle gravity acts 3 ft. - (6 ft. x 0.05) = 2.70 ft. from the inside wheel.

Sum moments about inside wheel equal to zero.

$W \times [2.70 \text{ ft.} + 0.156(6 \text{ ft.})] - F_o(6 \text{ ft.}) = 0$

where:

F_o = Reaction of the outside wheel.

$F_o = 0.61W$

F_i = Force on inside wheel

$F_i = W(1.0 - 0.61) = 0.39\ W$

The wheel loads in each lane that are applied to the influence surfaces are adjusted by two times these factors, or 1.22 applied to the outside wheels and 0.78 applied to the inside wheels. The result is that the outermost girder will receive slightly higher load and the innermost girder will receive slightly lower load. Thus, it is also necessary to compute the condition with no centrifugal force, i.e., a stationary vehicle, and select the worst case. The inside of the bridge will be more heavily loaded for the stationary vehicle case. Although not typically done, if the superelevation is significant, the Engineer may wish to consider its effect for this case, since the superelevation will cause an increase in the vertical wheel loads toward the inside of the bridge and an unloading of the vertical wheel loads toward the outside of the bridge.

Although centrifugal force effects were conservatively considered for lane loads in the example, Article 3.6.3 specifies that lane load is neglected in computing the centrifugal force, as the spacing of vehicles at high speeds is assumed to be large, resulting in a low density of vehicles following and/or preceding the design truck.

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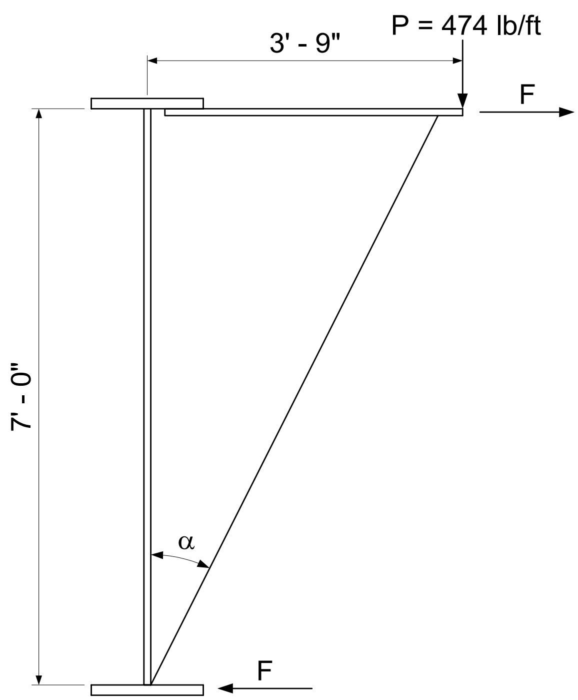


Figure D-1. Overhang Bracket Loading

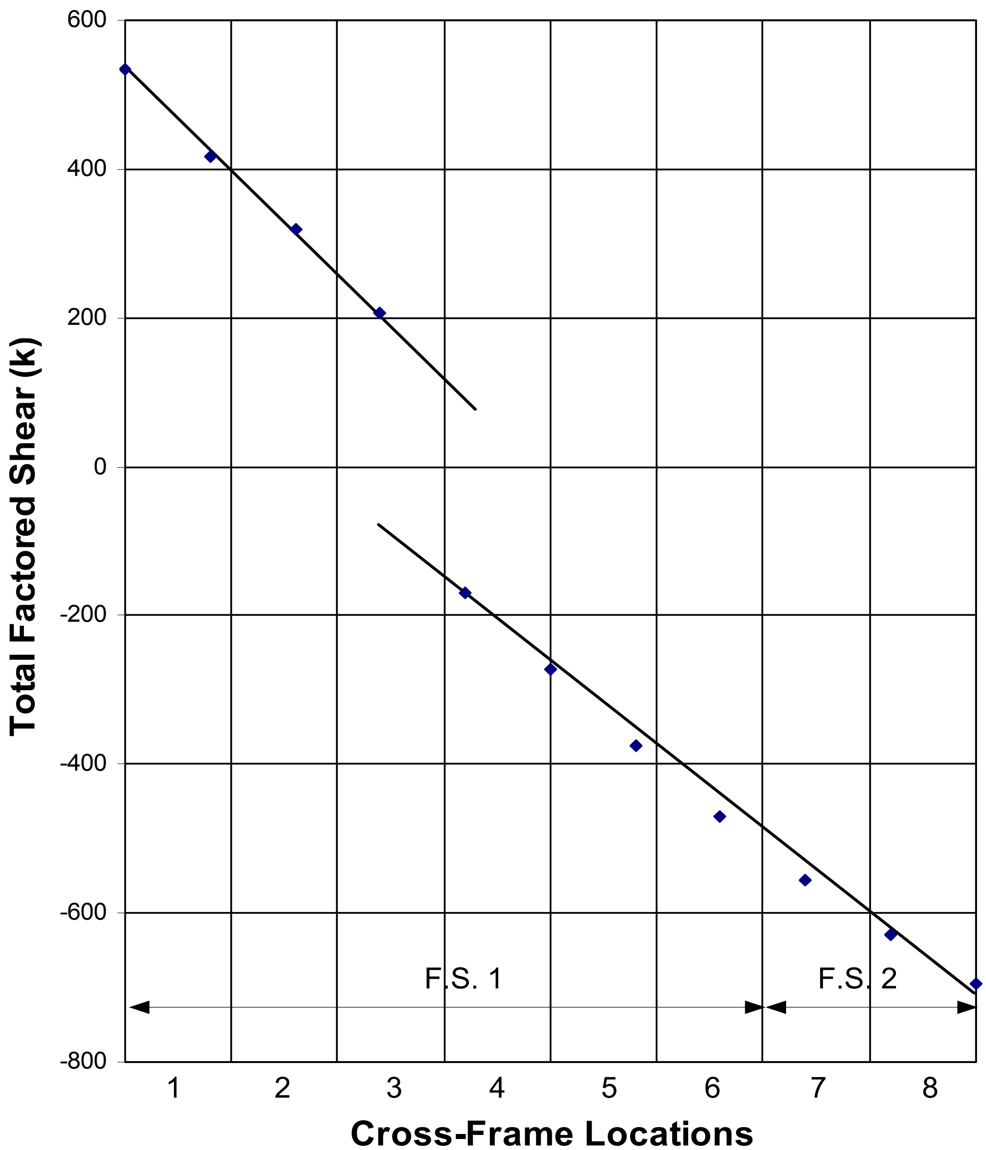


Figure D-2. Factored Shear and Transverse Stiffener Spacing - Span 1

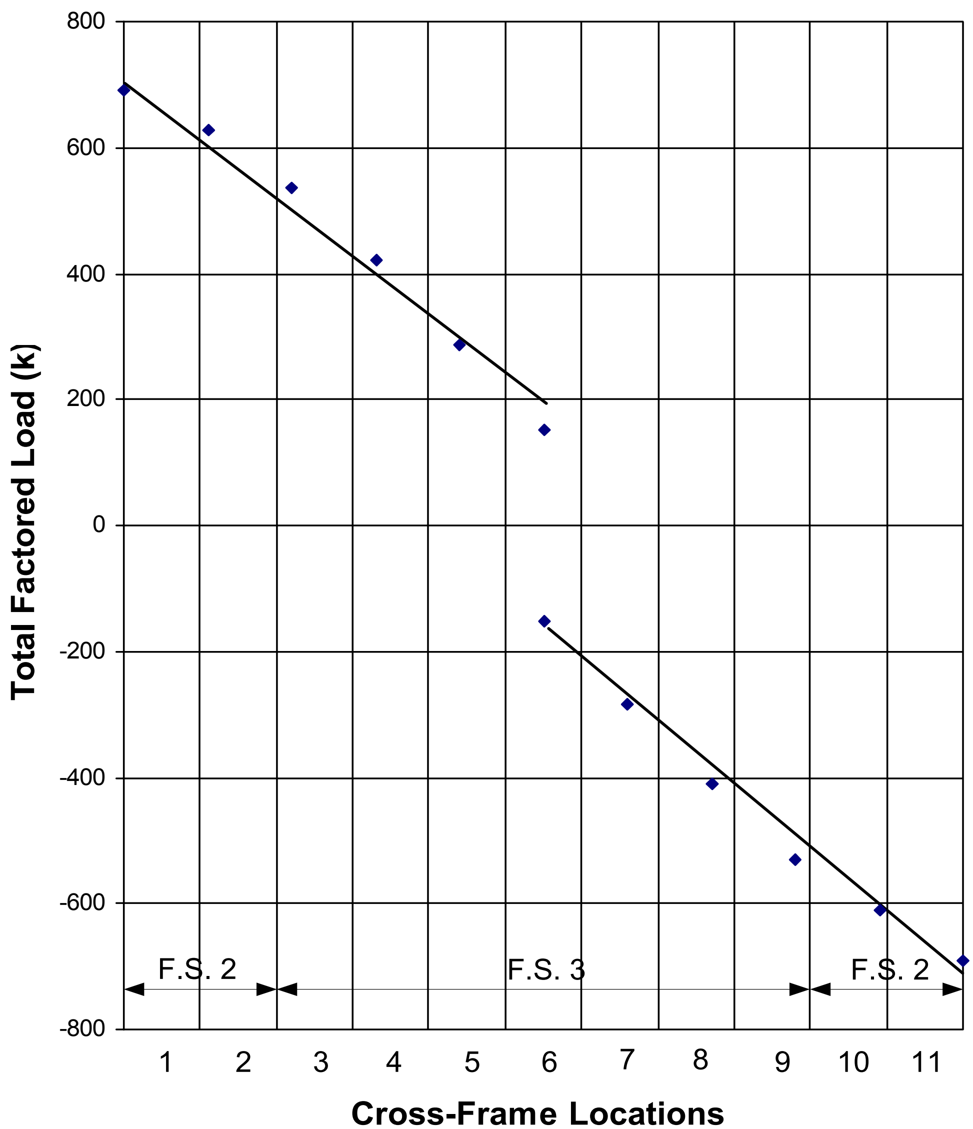
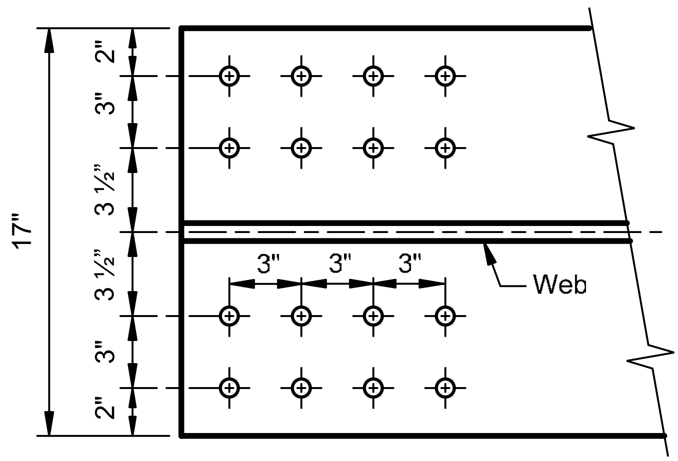
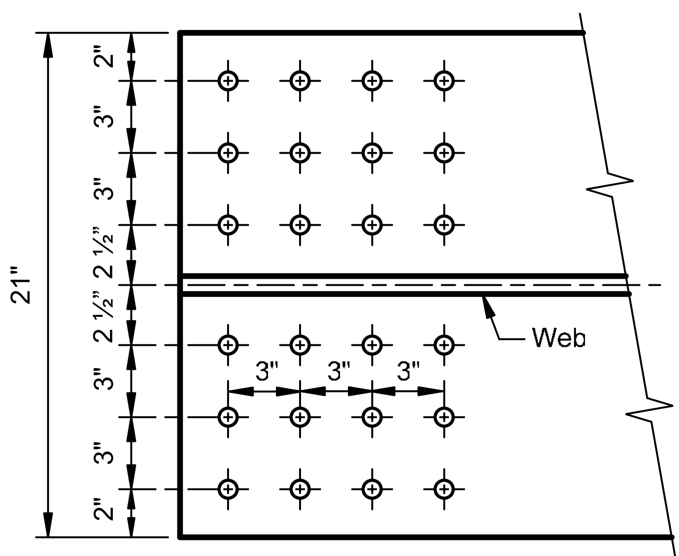


Figure D-3. Factored Shear and Transverse Stiffener Spacing - Span 2



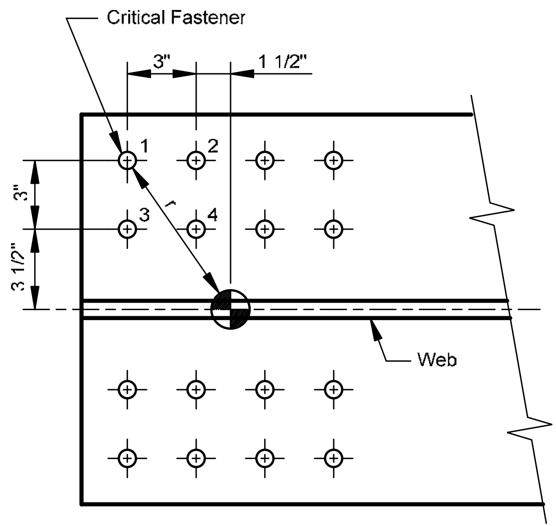
Top Flange Bolt Pattern



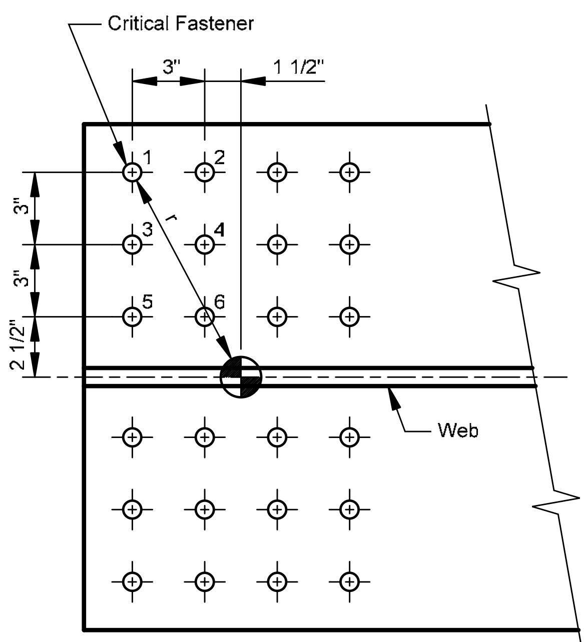
Bottom Flange Bolt Pattern

(a)

Figure D-4. Bolt Patterns for Top and Bottom Flanges

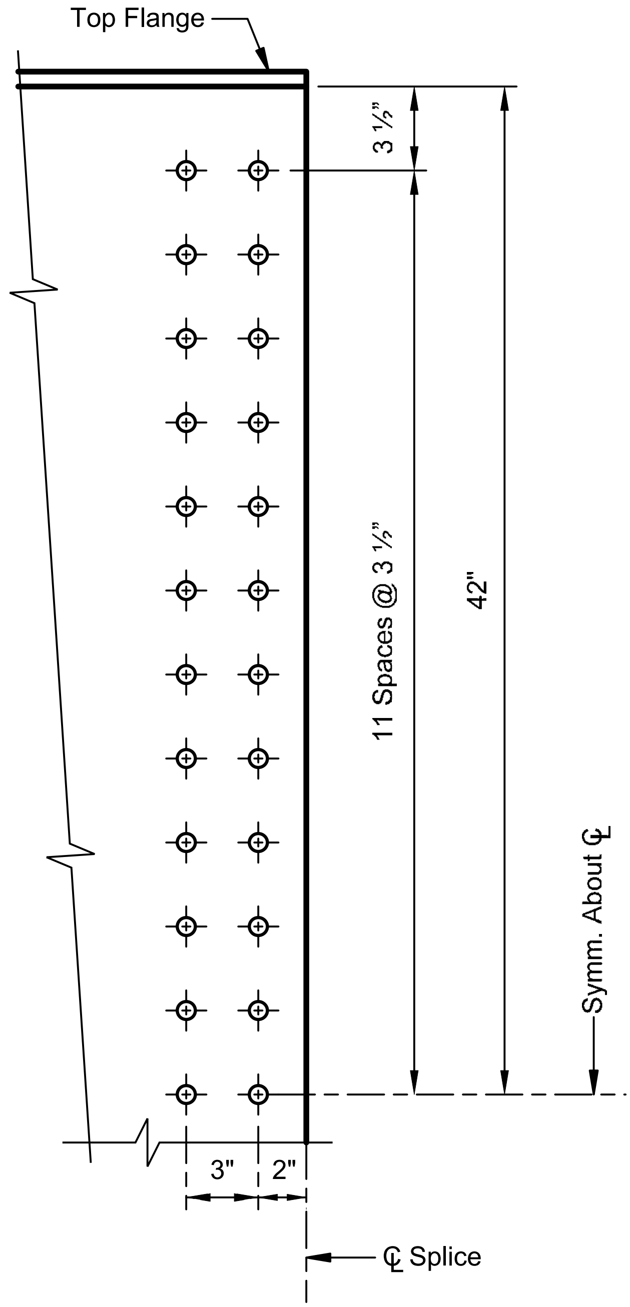


(b) Critical Bolt Location in Top Flange Splice Plate



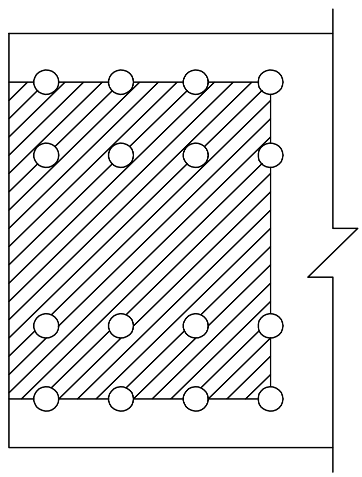
(c) Critical Bolt Location in Bottom Flange Splice Plate

Figure D-4. Bolt Patterns for Top and Bottom Flanges (continued)

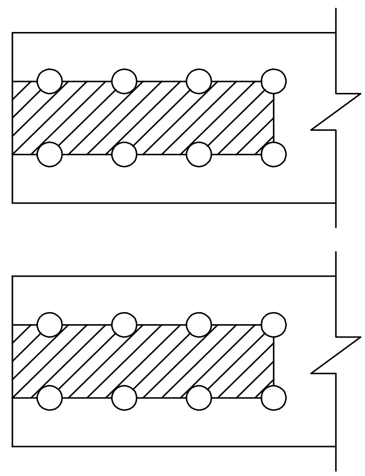


Note: 1/4" gap assumed between edges of field pieces.

Figure D-5. Web Bolt Pattern

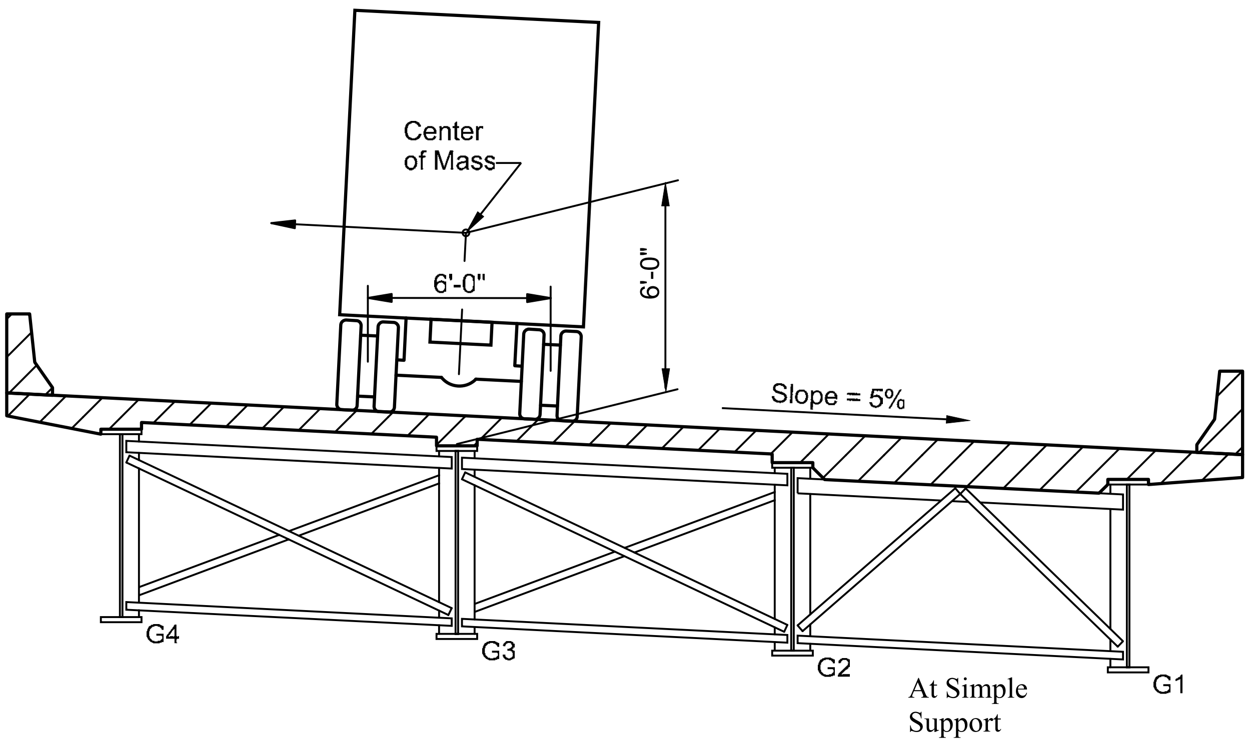


Top Flange - Outside Plates



Top Flange - Inside Plates

Figure D-6. Controlling Flange Failure Paths



Section

Figure D-7. Centrifugal Force and Superelevation

Table D-1. Unfactored Loads

Load	Moment (k-ft)		Lateral Flange Moment (k-ft)		Shear (kips)	
Steel	-382		+/-2.9		27	
Deck	-1,585		+/-12.1		112	
Comp DL	-487		+/-3.7		41	
Cast #1	-1,910		+/-17.8		7	
Strength HL-93 with Impact	Pos	Neg	Pos	Neg	Pos	Neg
	2,054	-2,772	-	-	139	-26

Table D-2. Cross-Section

Component	Size (in.)	Area (in^2)	Yield, F_y (ksi)	Tensile, F_u (ksi)
Top Flange	17 x 1.0	17.0	50	65
Web	84 x 0.5625	47.25	50	65
Bottom Flange	21 x 1.5	31.5	50	65

Note: Other section properties for the gross section may be found in Table C-4.

APPENDIX E

Tabulation of Stress Checks, Girder 4

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Introduction

The following tables show the comparative stress checks (for Girder 4) between the AASHTO-LFD Guide Specifications (NCHRP12-38) and the AASHTO-LRFD Specifications, Third Edition, 2004 with 2005 Interims.

Table E-1. Constructibility – Top Flange, Girder 4

Section/ Node	NCHRP 12-38 (LFD)			2004 LRFD Specifications with 2005 Interims									
				Eq (6.10.3.2.1-1)			Eq (6.10.3.2.1-2)			Eq (6.10.3.2.1-3)			Controlling
	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	ratio
1-1 24	-35.69	-25.77	0.72	-50.0	-31.8	0.64	-44.33	-25.94	0.59	-41.19	-23.01	0.56	0.64
2-2 44	-35.21	-31.15	0.88	-50.0	-39.17	0.78	-44.16	-31.6	0.72	-33.60	-27.81	0.83	0.83
2-2 44#	-35.98	-32.16	0.89	-50.0	-38.58	0.77	-45.12	-32.0	0.71	-50.00	-28.71	0.57	0.77
3-3 64	-35.08	-18.77	0.54	-50.0	-23.61	0.47	-44.16	-19.04	0.43	-33.60	-16.76	0.50	0.50
3-3 64#	-35.90	-19.37	0.54	-50.0	-23.51	0.47	-45.12	-19.37	0.43	-19.75	-17.30	0.88	0.88
4-4 76	-29.72	-4.97	0.17	-50.0	-7.64	0.15	-44.16	-5.51	0.12	-33.60	-4.44	0.13	0.15
4-4 76#	-30.95	-5.13	0.17	-50.0	-7.49	0.15	-45.12	-5.55	0.12	-19.75	-4.58	0.23	0.23
10-10 148	-34.11	-27.32	0.80	-50.0	-51.7	1.03	-41.75	-38.12	0.91	-31.57	-31.33	0.99	1.03

* Applied stress divided by the resistance
# Longitudinally stiffened web

Table E-2. Constructibility – Bottom Flange, Girder 4

Section/ Node	NCHRP 12-38 (LFD)			2004 LRFD Specifications with 2005 Interims Eq (6.10.3.2.2-1)		
	F_{cr}	f_b	ratio *	$\phi_f R_h F_{yt}$	$f_{bu} + f_l$	ratio
1-1 24	39.91	25.2	0.63	50.0	28.34	0.57
2-2 44	39.19	24.95	0.64	50.0	27.26	0.55
2-2 44#	39.49	24.95	0.63	50.0	26.81	0.54
3-3 64	39.71	15.03	0.38	50.0	16.42	0.33
3-3 64#	40.07	15.03	0.38	50.0	16.15	0.32
4-4 76	34.89	3.98	0.11	50.0	4.35	0.09
4-4 76#	35.57	3.98	0.11	50.0	4.28	0.09
10-10 148	40.56	20.4	0.50	50.0	28.52	0.57

* Applied stress divided by the resistance
# Longitudinally stiffened web

Table E-3. Constructibility – Web, Girder 4

Section/ Node	NCHRP 12-38 (LFD)			2004 LRFD Specifications with 2005 Interims		
	F_{cr}	f_b	ratio *	F_{crw}	f_{cw}	ratio
1-1 24	-41.18	-25.18	0.61	-41.19	-23.01	0.56
2-2 44	-33.6	-30.5	0.91	-33.6	-27.81	0.83
2-2 44#	-50	-31.49	0.63	-50.0	-28.71	0.57
3-3 64	-33.6	-18.37	0.55	-33.6	-16.76	0.50
3-3 64#	-14.02	-18.97	1.35	-19.75	-17.3	0.88
	-19.73	-18.97	0.96	-34.14	-14.06	0.41
4-4 76	-33.6	-4.87	0.14	-33.6	-4.44	0.13
4-4 76#	-6.87	-5.03	0.73	-19.75	-4.58	0.23
	-19.73	-5.03	0.25	-34.14	-18.23	0.53
10-10 148	-31.57	-26.77	0.85	-31.57	-31.33	0.99

* Applied stress divided by the resistance
# Longitudinally stiffened web

Table E-4. Strength – Top Flange, Girder 4

Section/ Node	NCHRP 12-38 (LFD)			2004 LRFD Specifications with 2005 Interims		
	F_{cr}	f_b	ratio *	$\phi_f F_{nt}$ or $\phi_f F_{nc}$	f_{bu}	ratio
1-1 24	-50.0	-25.35	0.51	-50.0	-24.98	0.50
2-2 44	-50.0	-28.76	0.58	-50.0	-28.33	0.57
2-2 44#	-50.0	-29.24	0.58	-50.0	-28.78	0.58
3-3 64	-50.0	-11.79	0.24	-50.0	-11.74	0.23
	50.0	18.39	0.37	50.0	11.61	0.23
3-3 64#	-50.0	-11.84	0.24	-50.0	-11.78	0.24
	50.0	18.98	0.38	50.0	11.73	0.23
4-4 76	50.0	35.85	0.72	50.0	36.69	0.73
4-4 76#	50.0	36.68	0.73	50.0	40.59	0.81
5-5 88	50.0	47.78	0.96	50.0	48.5	0.97
5-5 88#	50.0	49.88	0.998	50.0	49.84	1.00
6-6 100	50.0	49.68	0.99	50.0	46.72	0.93
6-6 100#	50.0	49.74	0.99	50.0	46.77	0.94
7-7 112	50.0	47.98	0.96	50.0	45.99	0.92
8-8 124	50.0	35.74	0.71	50.0	35.64	0.71
10-10 148	-50.0	-30.88	0.62	-50.0	-28.88	0.58

* Applied stress divided by the resistance
\# Longitudinally stiffened web

Table E-5. Strength – Bottom Flange, Girder 4

Section/ Node	NCHRP 12-38 (LFD)			2004 LRFD Specifications with 2005 Interims		
	F_{cr}	f_b	ratio *	$\phi_f F_{nt}$ or $\phi_f F_{nc}$	f_{bu}	ratio
1-1 24	44.94	46.04	1.02	50.0	47.14	0.94
2-2 44	45.63	45.94	1.01	50.0	46.64	0.93
2-2 44#	46.01	46.91	1.02	50.0	47.65	0.95
3-3 64	50	27.78	0.56	50.0	28.33	0.57
	-45.57	-14.73	0.32	-45.10	-12.57	0.28
3-3 64#	50	28.56	0.57	50.00	29.14	0.58
	-45.96	-14.73	0.32	-44.55	-12.67	0.28
4-4 76	-45.57	-32.84	0.72	-45.10	-36.62	0.81
4-4 76#	-45.96	-32.96	0.72	-44.55	-36.75	0.82
5-5 88	-46.99	-47.36	1.01	-46.46	-48.21	1.04
5-5 88#	-36.84	-45.67	1.24	-46.46	-49.25	1.06
6-6 100	-47.31	-47.05	0.99	-48.12	-44.16	0.92
6-6 100#	-47.53	-46.70	0.98	-47.68	-43.83	0.92
7-7 112	-47.56	-47.44	1.00	-47.71	-45.41	0.95
8-8 124	-45.97	-30.09	0.65	-45.32	-30.06	0.66
10-10 148	45.97	47.10	1.02	50.0	46.08	0.92

* Applied stress divided by the resistance
# Longitudinally stiffened web

Table E-6. Strength - Web (Compression), Girder 4

Section/ Node	NCHRP 12-38 (LFD)			2004 LRFD Specifications with 2005 Interims		
	F_{cr}	f_b	ratio *	F_{crw} **	f_{bu}	ratio
1-1 24	-50.0	-24.52	0.49	-	-	-
2-2 44	-50.0	-27.9	0.56	-	-	-
2-2 44#	-50.0	-28.36	0.57	-50.0	-28.78	0.58
3-3 64	-50.0	-11.34	0.23	-	-	-
	-50.0	-14.15	0.28	-44.0	-12.57	0.29
3-3 64#	-50.0	-11.38	0.23	-50.0	-11.78	0.24
	-34.2	-14.14	0.41	-23.9	-12.67	0.53
4-4 76	-46.79	-31.65	0.68	-44.0	-36.62	0.83
4-4 76#	-32.94	-31.75	0.96	-32.94	-36.75	1.12
5-5 88	-50.0	-45.71	0.91	-49.2	-48.21	0.98
5-5 88#	-50.0	-46.69	0.93	-50.0	-49.25	0.99
6-6 100	-50.0	-43.81	0.88	-50.0	-44.16	0.88
6-6 100#	-50.0	-43.47	0.87	-50.0	-43.83	0.88
7-7 112	-50.0	-45.79	0.92	-49.2	-45.41	0.92
8-8 124	-50.0	-28.95	0.58	-46.8	-30.06	0.64
10-10 148	-50.0	-29.98	0.60	-	-	-

* Applied stress divided by the resistance
** Composite sections subjected to positive flexure need not be checked in the final condition when the web does not require longitudinal stiffeners (Article 6.10.1.9.1)
\# Longitudinally stiffened web

Table E-7. Fatigue - Category C' and Stud Spacing, Girder 4

Section/ Node	Shear Connector Spacing (in.)	
	NCHRP 12-38 (LFD)	2004 LRFD Specifications with 2005 Interims
1-1 24	14.9	-
2-2 44	19.1	20.4
2-2 44#	17.3	-
3-3 64	16.0	19.7
3-3 64#	16.3	-
6-6 100	16.3	19.7
6-6 100#	14.8	-
9-9 128	14.3	-
10-10 148	15.3	-

Longitudinally stiffened web

APPENDIX F

Field Section Profiles

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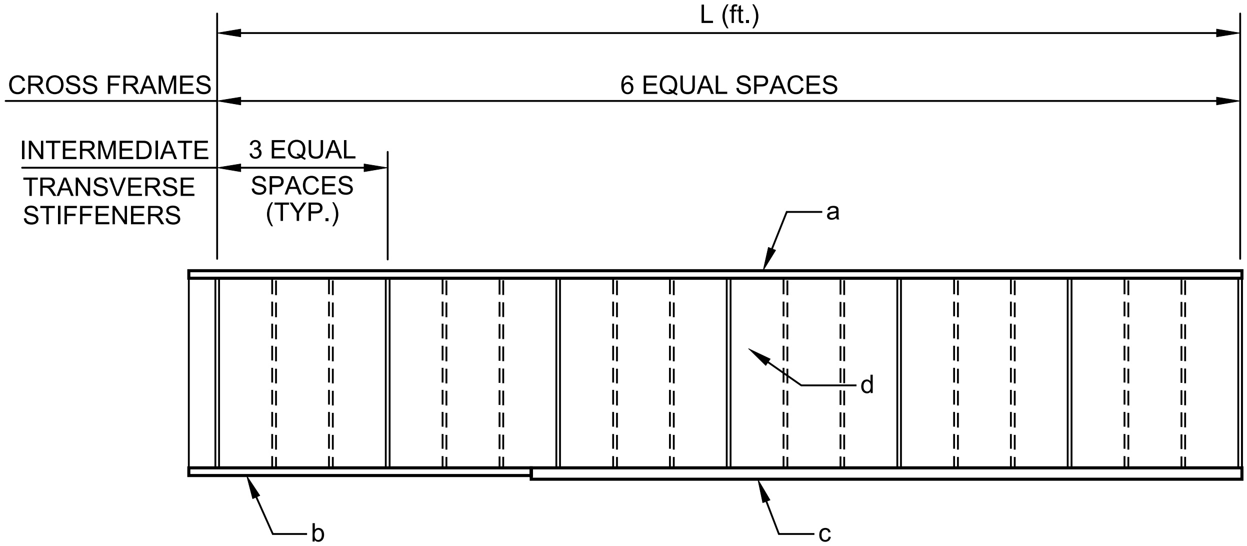
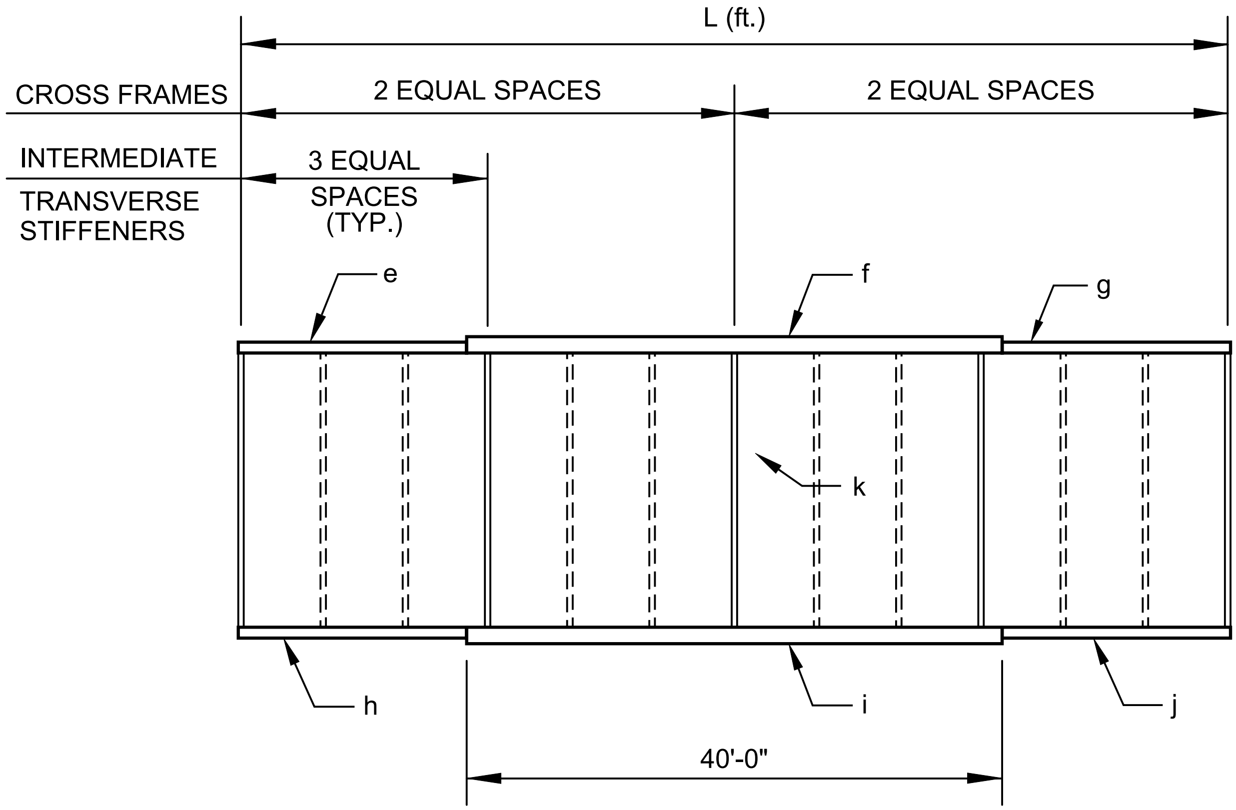


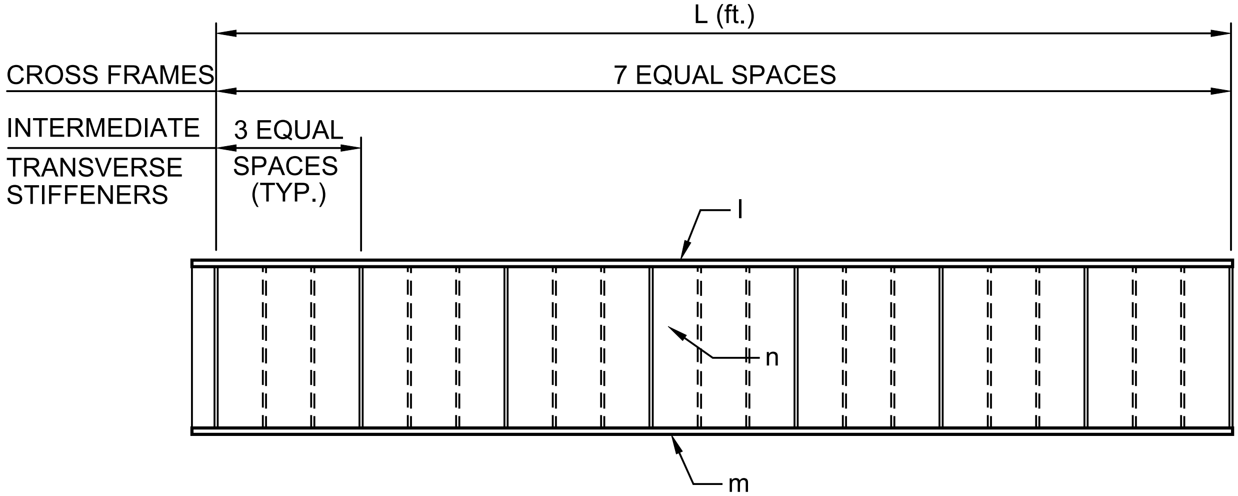
	a	b	c	d	L	Wt. (Lb.)
G1	15x1	No change	16x1	84x9/16	117	31,215
G2	15x1	No change	16x1	84x9/16	119	31,717
G3	16x1	No change	18x1	84x9/16	121	33,454
G4	20x1	21x1	21x1.5	84x9/16	123	40,074

Figure F-1. Field Section 1



	e	f	g	h	i	j	k	L	Wt. (Lb.)
G1	21x1.25	21x2.5	21x1.25	21x1.5	21x3	21x1.5	84x5/8	77.5	36,094
G2	18x1.25	18x2.5	18x1.25	19x1.5	19x3	19x1.5	84x5/8	77.6	34,006
G3	20x1.25	20x2.5	20x1.25	21x1.5	21x3	21x1.5	84x5/8	78.7	36,754
G4	28x1.25	28x2.5	28x1.25	27x1.5	27x3	27x1.5	84x5/8	80.0	45,077

Figure F-2. Field Section 2



	l	m	n	L	Wt. (Lb.)
G1	15x1	18x1	84x9/16	130.5	35,625
G2	15x1	17x1	84x9/16	132.6	35,748
G3	15x1	20x1	84x9/16	134.7	36,772
G4	17x1	28x1.5	84x9/16	136.8	44,558

Figure F-3. Field Section 3

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Wind on Steel During Erection

Although it is not the responsibility of the designer to consider all wind conditions during construction of the bridge, some conditions should be considered during a study of the erection scheme. In this example, only one wind direction has been studied. Wind from the inside of the curve and perpendicular to the beginning of the bridge is considered. The intensity used in the analysis is 50 pounds per square foot. However, the intensity can be reduced if desired. Wind is applied to the first stage of construction. Temporary supports (or falsework) are assumed to be in place to resist vertical but not lateral forces. Girders 1 and 2 are framed together, but Girders 3 and 4 are not connected and are ignored.

Wind is applied to Girder 1. Curvature is considered in reducing the exposed projection and thereby reducing the amount of wind. The superelevation of the girders causes Girder 2 to be 0.55 feet higher than Girder 1 and to receive additional wind on the top flange. Girder 2 also extends approximately 3 feet beyond Girder 1 in the direction of the wind due to curvature. An additional force is applied to the end of the that girder (Node 122) at both the top and bottom flange.

Analysis results include reactions, deflections, axial forces in the top and bottom of the girders, lateral flange bending and the axial force in all bracing members. The first analysis did not consider any lateral bracing between girders. That analysis indicated lateral deflections due to wind of several feet. However, such bracing is required when lifting the pairs of girder sections. The presence of lateral bracing was therefore assumed in subsequent analyses. The bracing is composed of top lateral members connecting at the cross frame locations. A single member is used in each panel between Girders 1 and 2. These members will be removed when placing the deck forms.

Wind produces an uplift of 21.7 kips on Girder 1, Node 1. The gravity reaction due to the steel weight is only 5.6 kips down. This indicates an uplift condition. Using a dead load factor of 0.9, as specified in Article 3.3 for an uplift condition, and a load factor of 1.4 for the wind during construction as specified in Article 3.4, the maximum uplift is computed to be [1.4(21.7) - 0.9(5.6)] = 25.3 kips. Thus, the contractor needs to provide a temporary tie-down with 25.3 kip resistance for a 100 mph design wind during construction if the girders are left in pairs without cross frames added between Girders 2 and 3. The case with cross frames added is not checked, but would be much less critical and may not require tied-down bearings.

Girder Stresses Including Lateral Flange Bending

The top flanges act as a truss with the top flange lateral bracing. This action creates very little lateral flange bending. At Support 2, lateral flange bending is more critical. However, girder axial stresses are not very large at that location.

Bracing Members

Cross frame bracing is most critical at bearings under wind loading. At these points wind force is taken from the top flanges to the bearings. There is little force in these members due to gravity. At Support 1, the force is 25.05 kips. At Support 2, the critical force is 23.50 kips.

Since the temporary supports are assumed to provide no lateral restraint, no wind force is taken. These cross frame members must be checked for forces in the completed bridge before they are designed for these wind forces to ensure that the critical loading case is considered.

Top lateral bracing is most critical at supports because the lateral shear is largest at the supports. At Support 1, the critical force is 48 kips and at Support 2, the critical force is 49 kips. Gravity contributes essentially no force in these members. Since this is the only loading condition for these members, they can be designed for these forces factored by 1.4, which is the load factor for wind during construction specified in Article 3.4.