NCHRP Project 12-38

COPY No. 1

DESIGN EXAMPLE
HORIZONTALLY CURVED STEEL BOX GIRDER BRIDGE

Appendix F

Prepared for
National Cooperative Highway Research Program
Transportation Research Board
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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 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.

This Horizontally Curved Steel Box Girder Bridge Design Example has been developed to demonstrate the applicability of the **Recommended Specifications for Steel Curved-Girder Bridges**. In this example, a composite bottom flange option is provided for the bottom flange in the negative moment regions. The Design Example was compiled as a part of the deliverables in National Cooperative Highway Research Program Project 12-38.

The following terms are used to identify particular specifications:

- ANSI/AASHTO/AWS refers to the 1996 edition of D1.5-96 *Bridge Welding Code,* American Welding Society and Interim Specifications,
- "previous curved-girder specifications" or Guide Spec refer to the 1993 AASHTO *Guide Specifications for Horizontally Curved Highway Bridges*,
- LFD/ASD refers to the 1996 AASHTO *Standard Specifications for Highway Bridges*, 16th edition and Interim Specifications and
- LRFD refers to the 1998 AASHTO *LRFD Bridge Design Specifications* and Interim Specifications.

It is expected that curved-girder provisions based on the **Recommended Specifications for Steel Curved-Girder Bridges** will be incorporated into the AASHTO Load and Resistance Factor Design (LRFD) specifications in the future.

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I. Objective

Using the Recommended Specifications for Steel Curved-Girder Bridges (hereafter referred to as the Recommended Specifications), design a three-span horizontally curved steel box girder bridge with two tub girders in the cross section.

II. Design parameters

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

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

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

Structural steel having a specified minimum yield stress of 50 ksi is used throughout. The deck is conventional cast-in-place concrete with a specified minimum 28-day compressive strength of 4,000 psi. The haunch is 4.0 inches deep measured from the top of the web to the bottom of the deck. The width of the haunch is assumed to be 20.0 inches. A future wearing surface of 30 psf is specified.

The roadway is superelevated 5 percent.

Live load is HS25 for the strength limit state. Live load for overload and service load is taken as HS20 in this example. Live load for fatigue is taken as defined in Article 3.5.7.1. The bridge is designed for a 75-year fatigue life. The bridge site is assumed to be located in earthquake Zone A so earthquake loading need not be considered.

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

III. Steel Framing

The steel framing consists of two trapezoidal tub girders with the tops of the webs in each tub spaced 10 feet apart and with a clear deck span of 12.5 feet between the interior webs of each tub. The cross section is shown in Figure 1. Two bearings set one foot inside of each web are used under each box at each support, as permitted in Article 10.2.1.

A. Girder Depth

Article 12.2 provides for a preferred minimum depth limit of one-twenty-fifth of the span of the girder when the steel has a specified minimum yield stress not greater than 50 ksi. In checking this requirement, the effective length of girder spans continuous on both ends is defined as eighty percent of the longest span between bearings. The effective length is defined as ninety percent of the longest span between bearings of girder spans continuous on only one end. The longest effective span length (either end or interior span) controls. The length of the center span of the outside girder, G2, is 213.38 feet (measured along the longitudinal centerline of the box), which is the girder with the longest effective span in this example. Therefore, the recommended girder depth is computed as follows:

$$0.8 \times 213.38 \times 12/25 = 81.9 \text{ in.}$$

The actual vertical girder depth is 78 inches, which is slightly less than the preferred minimum depth. However, box girders are generally stiffer than I girders because an individual box nearly acts as two I girders for vertical bending. For torsion, an individual box girder is significantly stiffer than two I girders.

The slope of the webs is one-on-four, which is the limit given in Article 10.1. As a result, the width of the bottom flange of each tub is 81 inches between webs. The actual box flange width is 83 inches to provide a 1-inch lip outside of each web, which is needed for welding of the webs to the bottom flange.

B. Internal and External Bracing

The boxes are braced internally at intermediate locations with K-frames. The internal K-frames are spaced longitudinally at approximately 17 feet, which is below the maximum limit of 30 feet specified in Article 10.2.2.3. At locations where a longitudinal flange stiffener is not used, Article 10.2.2.3 requires that transverse bracing members be attached to the bottom flange. At these locations, the bottom strut of the K-frame is assumed in this example to be welded to the bottom flange and bolted to the connection plates on the webs. At locations where a longitudinal flange stiffener is used, the bottom strut is assumed to be bolted to the top of the longitudinal stiffener (Article 10.2.2.3 requires attachment of the transverse member to the longitudinal stiffener by bolting) and to the connection plates on the webs. The cross frames are assumed to be single-angle members bolted to connection plates. The working points are assumed to be located as close to the flange-web intersections as practical, except where the longitudinal flange stiffening causes the bracing to be offset from the flange.

Design of the internal cross bracing members is not shown in this example. It was determined from the analysis that the largest factored load in any of the internal cross frame members on the bridge is 80 kips in the diagonal members located at Nodes 11 and 12 in Span 1. Cross frame members were modeled as truss members in the analysis, with a cross-sectional area of 8.0 square inches. Article 10.2.2.3 specifies that the cross-sectional area and stiffness of the top and bottom transverse bracing members not be smaller than the area and stiffness of the diagonal members. In addition, at locations where a longitudinal flange stiffener is present, the moment of inertia of the transverse bracing member should equal or exceed the moment of inertia of the longitudinal stiffener taken about the base of the stiffener.

The largest range of stress due to fatigue loading in the internal cross frames was found to be approximately 15 ksi. This maximum stress range was determined by passing the factored fatigue truck defined in Article 3.5.7.1 over the left and right web of a tub, resulting in a reversal of stress in each member. The sum of the absolute values of the maximum tensile and compressive stresses was 15 ksi. According to Article 3.5.7.2, only 75 percent of the stress range so determined is used to check fatigue for transverse members. Thus, the design fatigue stress range is approximately 11 ksi. The design stress range exceeds the nominal fatigue resistance of 2.25 ksi specified for a Category E detail according to **AASHTO LRFD Article 6.6.1.2**. The value of 2.25 ksi is equal to one-half of the constant-amplitude fatigue threshold of 4.5 ksi specified for a Category E detail in **Table 6.6.1.2.5-3** of **AASHTO LRFD**. This value is used whenever the fatigue strength is governed by the constant-amplitude fatigue threshold, which is assumed to be the case in this example. Since the design fatigue stress range exceeds the nominal fatigue resistance for a fatigue Category E detail, fillet welds cannot be used for these member connections in this particular case.

As required in Articles 10.2.2.2 and 10.2.3, there are full-depth internal and external diaphragms provided at support lines, but there are no other external braces provided between the boxes in this example.

C. Bracing of Tub Flanges

The top flanges of the individual tubs are braced with single members placed diagonally between the tub flanges. Figure 2 shows the arrangement of the top diagonal bracing in each girder. Figure 2 also gives the node numbers for part of Span 1 so that the locations can be related to subsequent sample design calculations given in Appendix E. The bracing is assumed to be directly connected to the flanges at each internal cross frame, as recommended in Article 10.2.4. These top-flange bracing members provide torsional continuity to the box before the deck cures, and therefore, must have adequate capacity to resist the torsional shear flow in the non-composite section at the constructibility limit state. One end of each internal cross frame does not have lateral bracing attached. The tub flanges tend to develop larger lateral flange bending stresses at the points where the lateral bracing is not connected because the top flange must provide the majority of the torsional resistance. Top flange bracing should be continuous along the length of the girder to ensure that the top flanges are not required to resist the entire torsion at any one location.

There are several causes of the lateral moments in the top flanges including curvature, inclination of the webs and overhang bracket loads. The effect of curvature can be conservatively estimated using Equation (4-1). The inclination of the webs causes a radial force, which must be resisted by the flanges. On the exterior of the bridge, a portion of the deck weight is applied to overhang brackets, which results in a radial tensile force on the outside top flanges and an opposite force on the bottom flange.

The single top flange lateral bracing members used in the design example cause the lateral flange moments to vary depending on whether or not the brace is connected to an interior or exterior flange. To illustrate, both single-diagonal and double-diagonal (or X) top-flange bracing arrangements were analyzed using a 3D finite-element model assuming both inclined and vertical webs. For the case of vertical webs, the bottom flange width is 120 inches. The lateral flange moments in the two top flanges, and in some cases, the forces in the top flange bracing members in part of Span 1 due to the entire deck weight and Cast #1 (with the effect of the overhang brackets considered in each case) are reported in Figures 3 through 7. Half of the overhang weight was assumed to be applied to the brackets in the analysis, as shown in Figure E1 (Appendix E). In Figures 3 through 7, the lateral flange moments are shown above and below the top flanges of each girder, whereas the axial forces in the top chord of the internal K-braces and in the top lateral bracing are shown near the appropriate members. Note that the inverted K-bracing inside the boxes results in two top chord members across the tub in the finite-element model.

Figure 3 shows the results for the case of the entire deck weight applied to the boxes and overhang brackets assuming double top flange lateral bracing and inclined webs. Figure 4 shows similar results for the case assumed in the design example (single-diagonal top flange lateral bracing and inclined webs) under the loading due to the entire deck weight. Figure 5 shows the results due to the entire deck weight for the single-diagonal bracing case with vertical webs. Figure 6 shows the results due to Cast #1 for the single-diagonal bracing case with inclined webs (again the case assumed in the design example). This loading case causes larger girder moments and bracing forces in Span 1 than does the entire deck load because the load in Span 2 tends to counter the load in Span 1. Finally, Figure 7 shows the results (lateral flange bending moments only) due to the entire deck weight for the case assuming double top flange lateral bracing and vertical webs.

From examination of the results shown in Figures 3 through 7, the single-diagonal bracing pattern chosen for the design example results in the largest lateral flange bending moments and bracing member forces. While these effects are reduced somewhat when double-diagonal bracing is utilized, additional bracing members and connections are required. A suggested solution is to utilize parallel single-diagonal bracing members in each bay, which would result in lower lateral flange bending moments in combination with fewer members and connections.

D. Longitudinal Flange Stiffener

A single longitudinal flange stiffener is used on the box flanges over the negative moment sections. The longitudinal stiffener is terminated at the bolted field splices in Spans 1, 2 and 3. By terminating the longitudinal flange stiffener at the bolted splices, there is no need to consider fatigue at the terminus of the stiffener. The bottom flange splice plates inside the box must be split to permit the stiffener to extend to the free edge of the flange where the longitudinal stress is zero, as shown in Figure E6 (Appendix E).

E. Field Sections

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

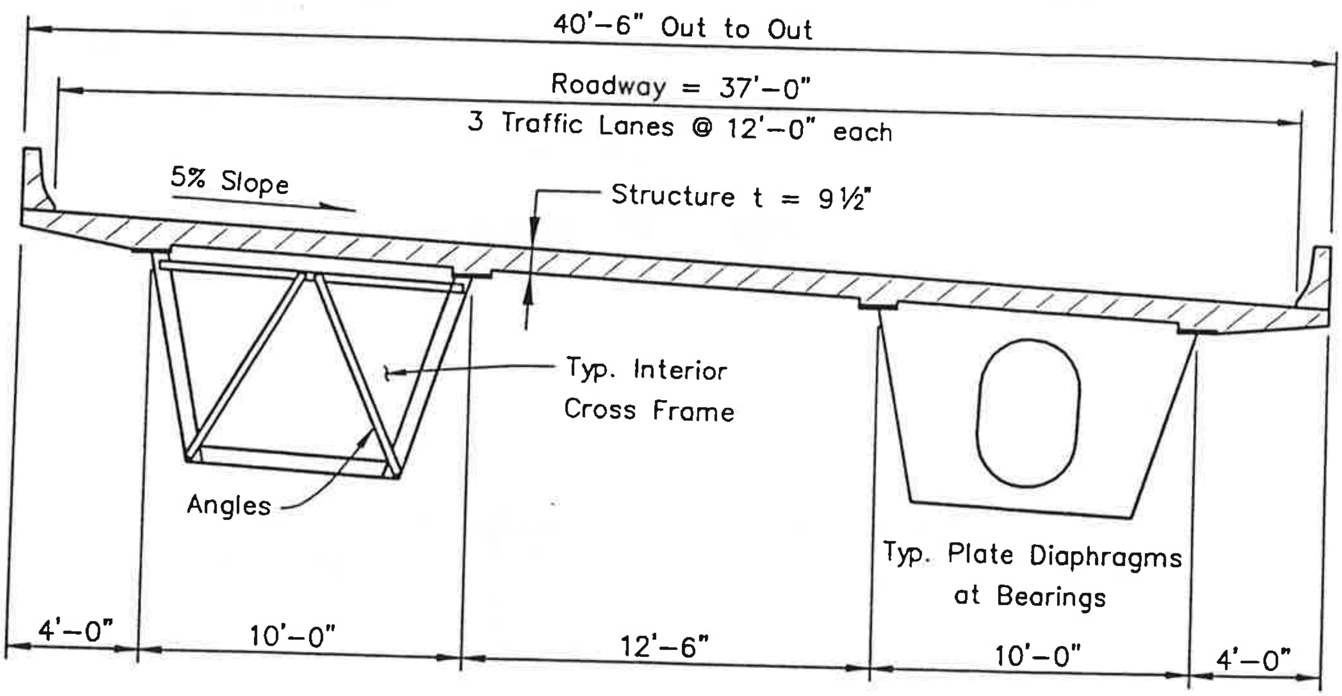
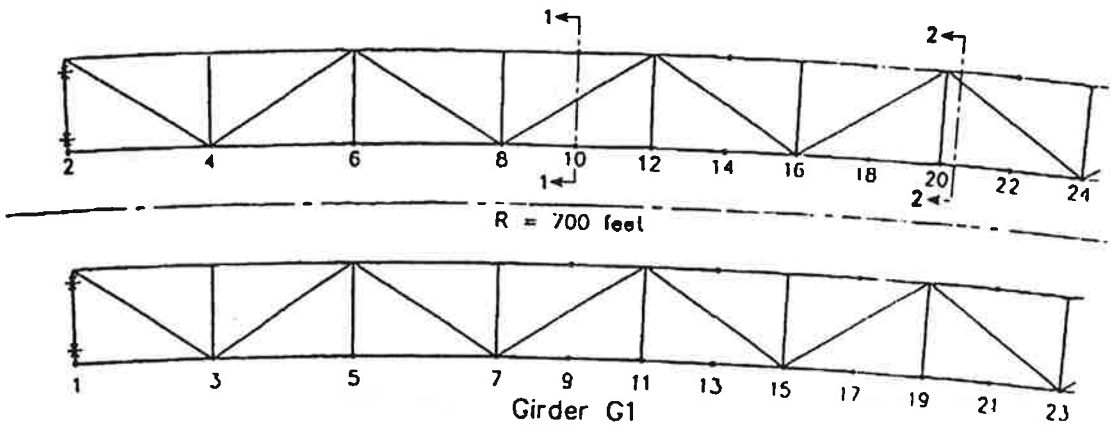


Figure 1 Box Girder Bridge Cross Section



*Bearing Locations

Figure 2 Node Numbers

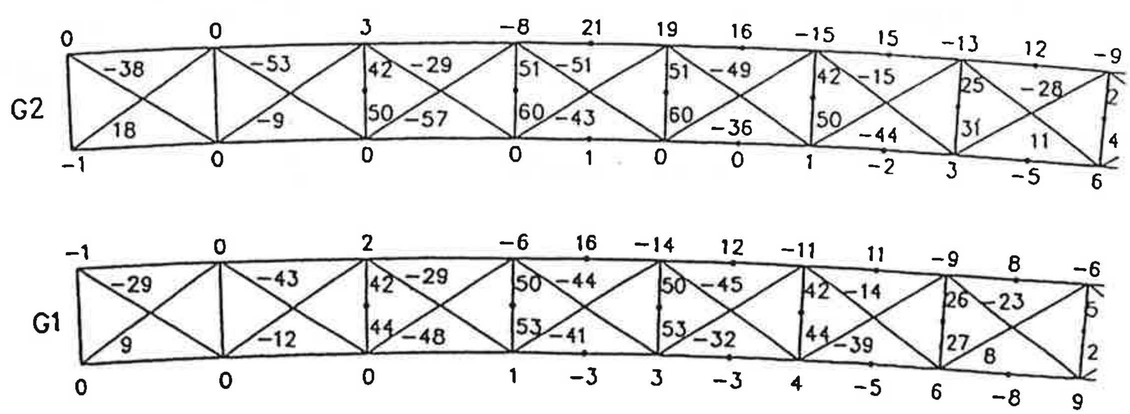


Figure 3 Lateral Flange Moments (k-ft) and Bracing Forces (kips) Due to Entire Deck Weight with Overhang Brackets, Inclined Webs

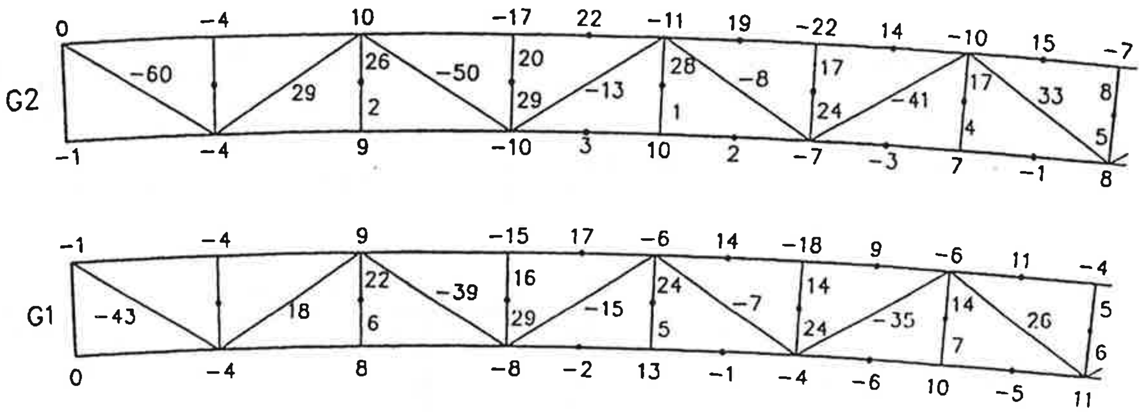


Figure 4 Lateral Flange Moments (k-ft) and Bracing Forces (kips) Due to Entire Deck Weight with Overhang Brackets, Inclined Webs

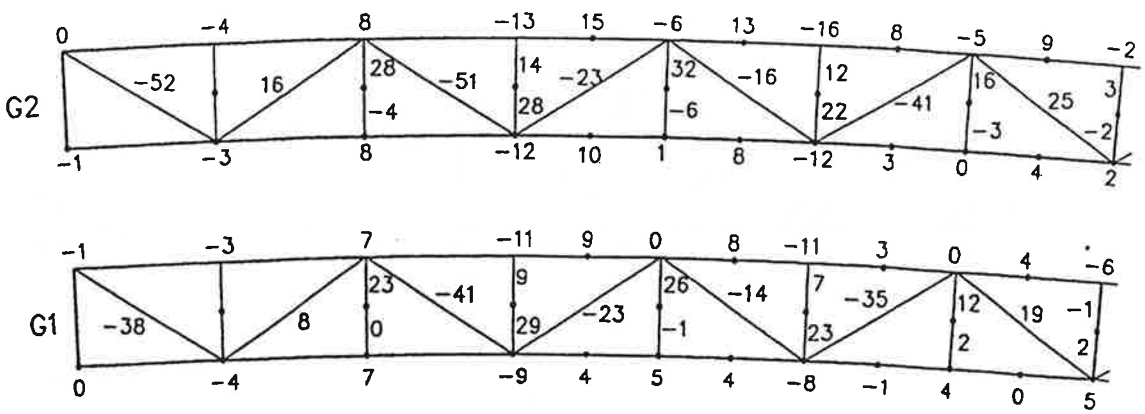


Figure 5 Lateral Flange Moments (k-ft) and Bracing Forces (kips) Due to Entire Deck Weight with Overhang Brackets, Vertical Webs

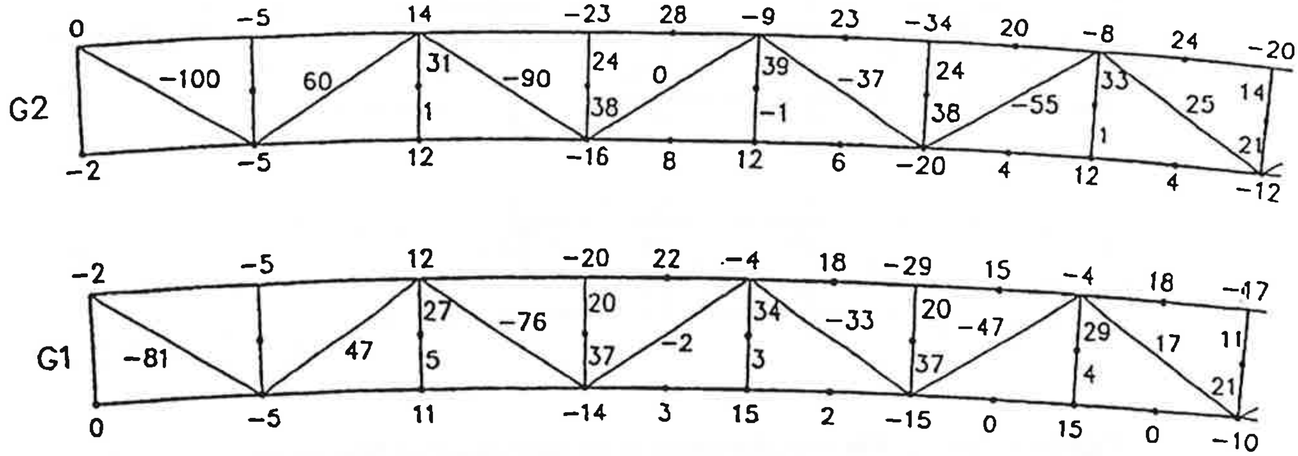


Figure 6 Lateral Flange Moments (k-ft) and Bracing Forces (kips) Due to Cast #1 with Overhang Brackets, Inclined Webs

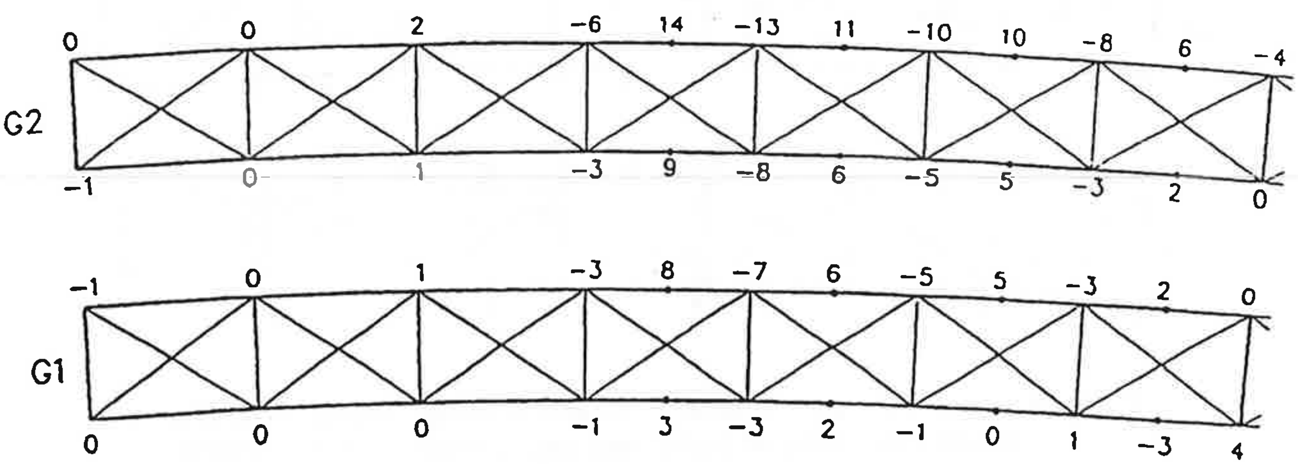


Figure 7 Lateral Flange Moments (k-ft) Due to Entire Deck Weight with Overhang Brackets, Vertical Webs

IV. Analyses

A. Loading Combinations

AASHTO Section 3 is used to determine load combinations for strength according to Article 3.1 of the Recommended Specifications. **Group I** loading is used for design of most members for the strength limit state. For temperature and wind loadings in combination with vertical loading, **Load Groups III, IV, V and VI** from **Table 3.22.1A** must also be checked. These load groups are defined as follows:

Group I 1.3[D + 5/3(L + I) + CF]
Group II 1.3[D + W]
Group III 1.3[D + (L + I) + CF + 0.3W + WL + LF]
Group IV 1.3[D + (L + I) + CF + T]
Group V 1.25[W + T]
Group VI 1.25[D + (L + I) + CF + 0.3W + WL + LF + T]

where:

D = Dead load
L = Live load
I = Impact
CF = Centrifugal force
W = Wind
WL = Wind on live load
T = Temperature
LF = Longitudinal force from live load

In addition to the above load Groups, the Recommended Specifications include a Group loading for the constructibility limit state defined in Article 3.3 as follows:

Group C 1.4[D + C + W*]

where:

D = Dead load
W* = wind load for construction conditions from an assumed critical direction.

Magnitude of wind may be less than that used for final bridge design.

C = Construction loads

In this example, only the **Group I** and **Group C** (minus the wind load) load combinations are checked. Other load cases may be critical, but for simplicity, these other load cases are not considered in this example. Selected analysis results for these two load groups are given in Tables D2 and D4, Appendix D. Table D2 gives the **Group I** shears for Girder 2 at the tenth points of Span 1. Table D4 gives the **Group I** and **Group C** top flange bracing forces in Span 1 of Girder 2.

B. Three-Dimensional Finite Element Analyses

Article 4.1 requires that the analysis be performed using a rational method that accounts for the interaction of the entire superstructure. Small-deflection elastic theory is acceptable.

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

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

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

Cross frames are modeled as individual truss elements connected to the nodes at the top and bottom of the girders. Internal solid-plate diaphragms at supports are modeled with a single plate element and external solid-plate diaphragms at supports are modeled utilizing three plate elements along the length for the web and six beam elements representing the top and bottom flanges of the diaphragm.

C. Comparison of Analyses

In order to make a quantitative comparison of the most common analytical methods employed for box-girder structures, two other types of analyses were made in addition to the three-dimensional finite element analysis; a two-dimensional grid analysis and a one-dimensional M/R analysis (Tung and Fountain, "Approximate Torsional Analysis of Curved Box Girders by the M/R Method," *Engineering Journal*, AISC, Vol. 7, No. 3, July 1970). The results of these comparative analyses are given in Tables C1 through C5 of Appendix C.

There is a close correlation of the vertical bending moments obtained from all three analysis methods in all cases considered. Good correlation of these values is expected because the radial geometry and the relatively high torsional rigidity of the box girders in this particular example minimize the increase in the vertical bending moments due to curvature. For cases where the width of the box section at mid-depth is larger than the girder depth, the ratio of relative torsional rigidity to bending rigidity will typically be higher than 0.4, which is the upper limit on this rigidity ratio recommended by Tung and Fountain for the M/R method to be considered valid for larger subtended angles. Hence, Article 4.2.2 specifies the limitation that the girder depth be less than the width of the box at mid-depth in order to utilize approximate methods to determine the vertical bending moments. The arc span divided by the girder radius must also be less than 0.3 radians.

As the spacing between centers of the two box girders exceeds 14 feet, the wheel load distribution is determined according to footnote "f" of **AASHTO Table 3.23.1** (based on the computation of simple beam reactions). Therefore, the total number of truck wheel loads is identical in all three analysis methods.

The concrete deck is modeled as a continuum in the 3D finite element model. In both the grid analysis method and the M/R method, girders are represented as one-dimensional elements and the rigid concrete deck cannot be represented as a continuum. The 3D finite element analysis also properly recognizes the physical location of the two bearings at each support point. Two bearings at each support point cannot be physically represented in the grid model, or in the M/R method, due to the limitations imposed by one-dimensional modeling of the girders. Although the vertical bending moments compared well among the three analysis methods, the difference in torsional moments and shears between the 3D finite element analysis and the other analysis methods is more significant, particularly in the case of parapet loading, as shown in Table C3. For the parapet loading, this would be expected since the parapet loads are applied at their actual locations at the edges of the deck in the finite-element model. The grid and M/R methods tended to underestimate the torques in each girder at the supports to a degree and overestimate the shear in the diaphragms between the boxes at the supports in this case.

V. Loads

A. Dead Load

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

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

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

Dead load moments, shears and torques in each girder from the 3D analysis are given in Appendix B.

B. Live Load

Analysis for live load is accomplished by first applying a series of unit vertical loads, one at a time, to the deck surface in the 3D model. Numerous responses are determined for each unit load, including girder moments, shears, torques, deflections, reactions, cross frame forces, etc. The magnitude of the response for a particular unit load is the magnitude of the ordinate of the influence surface for that response at the point on the deck where that unit load is applied. Curve fitting is used to determine responses between points on the influence surfaces. The specified live loads are applied mathematically to each influence surface and a search is then made to determine the maximum and minimum value of each response for each live load position. An impact allowance is applied according to Article 3.5.6.2.

Live load plus impact moments, shears and torques in each girder for HS25 loading from the 3D analysis are also given in Appendix B.

VI. Limit States

A. Strength

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

B. Constructibility

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

The unfactored moments from the deck staging analysis are presented in Table D1, Appendix D. "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. Included in the "Deck" and "Cast" moments are the moments due to the deck haunch and the stay-in-place forms; "SupImp" identifies the moments due to the superimposed dead load placed on the fully composite bridge.

Reactions are accumulated sequentially in the staging analysis to check for uplift at each stage. Accumulated deflections by stage are also computed. In each analysis of the deck placement, prior casts are assumed to be composite. The modular ratio for the deck is assumed to be 3n to account for creep. A somewhat smaller modular ratio may be desirable for the staging analysis since full creep usually takes approximately three years to occur. As specified in Article 13.3, a modular ratio of n should be used to check deck stresses. Moments and other actions determined from the deck-staging analysis are not considered for the strength limit state checks.

C. Fatigue

The fatigue limit state is checked by using the stress ranges due to the passage of one fatigue vehicle, defined in Article 3.5.7.1, traversing the length of the bridge in the critical transverse position on the deck for each response. The load factor is 0.75 for the fatigue truck, as specified in Article 3.5.7.1. Impact is 15 percent for the fatigue truck (Article 3.5.6.3). Centrifugal force effects are included. The transverse position of the truck may be different for each response and for positive and negative values of the same response. The fatigue truck is assumed to travel in either direction, or in opposite directions, to produce the maximum stress range. Marked traffic lanes are not considered. This assumption provides larger fatigue stresses than would be obtained if the fatigue truck were held to marked traffic lanes. The fatigue truck is permitted to travel within two feet of the curb line. As specified in Article 4.5.2, stress ranges are computed using the composite section for both positive and negative bending.

Article 2.3 specifies that twice the factored fatigue live load defined in Article 3.5.7.1 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 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 10.6.1 requires that longitudinal stress ranges be computed as the sum of the stress ranges due to vertical bending and warping. In addition, the through-thickness bending stress range due to cross-sectional distortion at flange-to-web fillet welds and at the termination of fillet welds connecting transverse elements must be checked for fatigue. Computation of these through-thickness bending stresses is illustrated in the Sample Calculations given in Appendix E.

D. Live Load Deflection

Article 12.4 requires that live load deflection be checked using the service live load plus impact. The limiting live load deflection is specified as the fraction of the span defined in Article 12.4. Different live load positions must be examined for each girder and span since the deflections of curved girders usually differ significantly at any one cross section.

Table 1 gives the preferred maximum live load deflections for the end and center spans of Girder 2 according to Article 12.4.

Table 1 Preferred Maximum Live Load Deflections (in.)

Girder	Span	L (ft)	L/800	L/1,000
G2	End	162.6	2.44	1.95
	Center	213.4	3.20	2.56

Computed maximum girder deflections in Girder 2 due to the service live load plus impact (HS20 loading) are 1.19 inches in the end span (Span 1) and 1.89 inches in the center span (Span 2). The computed deflections are based on the use of the uncracked composite section along the entire length of the bridge in the analysis. The multiple presence factors specified in **AASHTO Article 3.12** were considered.

If a sidewalk were present, vehicular traffic would be constrained from a portion of the deck (unless vehicles were permitted to mount the sidewalk), which would cause the computed live load deflections to be reduced depending on which side of the bridge the sidewalk was placed. Sidewalk load is discussed further in Article 3.5.5.

E. Permanent Deflection

Live load responses for overload (Article 3.5.4) are created for multiple lanes of HS20 live loading plus impact placed in the critical position for each girder. Both truck and lane loading are considered. Multiple presence reduction factors and centrifugal force effects are included. The load factors for overload are 5/3 on live load and 1.0 on dead load, as specified in **AASHTO Article 10.57** for Group I loading. Impact for overload is defined in Article 3.5.6.2 for tub girders. The provisions of Article 10.5 are used to check the overload flange stress limits for control of permanent set and that the compressive flange stress in the non-composite box flange does not exceed the applicable critical flange stress. Web bending stresses are checked according to the provisions of Article 6. Overload stresses caused by loads acting on the composite section are to be determined using the appropriate uncracked transformed composite section according to Article 10.5.

VII. Design

A. Section Properties

Table D5, Appendix D, gives selected section properties for Girder 2. Locations from the neutral axis to the top (T) and bottom (B) extreme fiber of the steel section are given. The section properties include the longitudinal component of the top-flange bracing area. Longitudinal flange stiffeners are also included in the section properties.

When the section is composite, the entire overhang, the concrete between the tub webs, and half of the concrete between girders is considered effective, as specified in Article 4.5.2. The haunch depth is considered in computing the section properties, but the area of the haunch is not included. The longitudinal reinforcing steel area equal to 20.0 square inches per box is assumed placed at the mid-thickness of the deck. The longitudinal reinforcing steel within the effective portion of the concrete is considered effective when the section is subjected to negative bending at the strength limit state. The deck area is divided by 3n and the reinforcing steel area is divided by 3 (for positive and negative bending, respectively) for computing the transformed section properties to account for creep in the concrete under superimposed dead load. The reinforcing steel area is adjusted since the concrete is assumed to transfer the force from the deck steel to the rest of the cross section.

Table E1 in the Sample Calculations (Appendix E) gives section properties for Girder 2 for the case where the bottom flange is composed of composite steel and concrete, as an alternative to a conventional longitudinally stiffened bottom flange. The Sample Calculations in Appendix E discuss the computation of the section properties given in Table E1 in more detail.

B. Shear Connectors

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

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

C. Flanges

According to Article 13.2, the top flanges of the tubs must meet the criteria for non-compact flanges at the constructibility limit state.

Three types of bottom (box) flanges are used in this example. In positive moment regions, the bottom flange is an unstiffened plate. A single longitudinal stiffener is used to increase the compressive strength of the bottom flange in the negative moment regions. Alternatively, an 8-inch thick concrete slab made composite with the flange plate is also investigated in the negative moment regions. The composite concrete permits a reduction in the flange plate thickness and the elimination of the longitudinal flange stiffener. At the constructibility limit state, Article 13.2 states that the critical stress for box flanges is to be determined from the provisions of Article 10.4.1.

D. Webs

In this example, transversely stiffened webs are used throughout. Transverse stiffener designs are not shown, but are similar to the designs illustrated in the I-girder design example. Transverse stiffeners are required throughout most of the girder length. The spacing of the transverse stiffeners near the interior supports is 62 inches. According to the provisions of Article 6.3, the maximum spacing is limited to 80.5 inches, which is the inclined depth of the web.

E. Diaphragms

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

F. Sample Calculations

Sample calculations at selected critical locations of Girder 2 are presented in Appendix E. The calculations are intended to illustrate the application of some of the more significant provisions contained in the Recommended Specifications. As such, complete calculations are not shown at all sections for each design. The sample calculations illustrate calculations to be made at the Strength, Fatigue, Constructibility and Serviceability limit states. The calculations also include longitudinal flange stiffener and bearing stiffener designs, a top flange bracing member design, a diaphragm design, transverse bending stress computations, a composite bottom flange option and a bolted field splice design. The calculations make use of the moments, shears, torques, and top flange bracing forces contained in Tables D1 through D4 of Appendix D and the section properties contained in Table D5.

APPENDIX A

Girder Field Sections

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June 21, 1997 9:25 AM

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

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

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

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


Box girder cross section

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

Girder --> 1 Field Section --> 1

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

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

Girder --> 1 Field Section --> 2

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

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

Girder --> 1 Field Section --> 3

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

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

Girder --> 1 Field Section --> 4

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

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

Girder --> 1 Field Section --> 5

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

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

Girder --> 2 Field Section --> 1

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

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

Girder --> 2 Field Section --> 2

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

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

Girder --> 2 Field Section --> 3

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

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

Girder --> 2 Field Section --> 4

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

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

Girder --> 2 Field Section --> 5

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

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

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

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

APPENDIX B

Girder Moments, Shears, and Torques at Tenth-Points

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STRENGTH -- HS25 PLUS IMPACT

April 5, 1997 10:51 AM

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

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

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


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


Stage Definition

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

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

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

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

DEAD LOADS

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

LIVE LOADS

Length	Moments: Lane POS	Lane NEG	Truck POS	Truck NEG	Special POS	Special NEG	1-Lane Truck POS	1-Lane Truck NEG
.00	0	0	0	0	0	0	0	0
15.74	2142	-477	2177	-338	0	0	1144	-133
31.49	3801	-954	3770	-676	0	0	1916	-267
47.23	4835	-1430	4641	-1013	0	0	2267	-399
62.97	5332	-1908	4972	-1352	0	0	2390	-525
78.71	5327	-2380	4865	-1687	0	0	2340	-641
94.46	4831	-2849	4367	-2020	0	0	2137	-758
110.20	3870	-3331	3484	-2374	0	0	1768	-926
125.94	2459	-3896	2276	-2747	0	0	1228	-1138
141.69	1256	-5372	980	-3133	0	0	481	-1385
157.43	950	-8230	804	-3605	0	0	302	-1719

Length	Shears: Lane POS	Lane NEG	Truck POS	Truck NEG	Special POS	Special NEG	Torque Maximums POS	Torque Maximums NEG
.00	133	-26	127	-16	0	0	615	-365
15.74	111	-33	105	-22	0	0	717	-443
31.49	91	-39	86	-29	0	0	693	-442
47.23	71	-43	74	-37	0	0	544	-372
62.97	47	-50	53	-46	0	0	357	-294
78.71	32	-63	39	-57	0	0	295	-326
94.46	23	-81	32	-74	0	0	271	-404
110.20	21	-99	26	-88	0	0	390	-542
125.94	19	-117	21	-99	0	0	541	-700
141.69	18	-130	14	-107	0	0	660	-820
157.43	16	-179	11	-130	0	0	1079	-897

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

DEAD LOADS

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

LIVE LOADS

Length	Moments Lane POS	Lane NEG	Truck POS	Truck NEG	Special POS	Special NEG	1-Lane Truck POS	1-Lane Truck NEG
.00	950	-8230	804	-3605	0	0	302	-1719
20.66	1161	-4412	1171	-2354	0	0	639	-1075
41.33	2421	-2419	2730	-1962	0	0	1476	-813
61.99	4193	-2145	4030	-1593	0	0	2015	-612
82.65	5390	-2152	4835	-1251	0	0	2340	-473
103.31	5786	-2157	5089	-913	0	0	2440	-356
123.98	5388	-2152	4834	-1250	0	0	2340	-472
144.64	4191	-2145	4028	-1593	0	0	2015	-613
165.30	2418	-2417	2729	-1960	0	0	1476	-814
185.96	1159	-4412	1168	-2351	0	0	639	-1076
206.63	949	-8229	803	-3607	0	0	302	-1719

Length	Shears Lane POS	Lane NEG	Truck POS	Truck NEG	Special POS	Special NEG	Torque Maximums POS	Torque Maximums NEG
.00	189	-17	138	-12	0	0	1079	-897
20.66	145	-30	108	-18	0	0	995	-714
41.33	130	-29	102	-21	0	0	899	-593
61.99	102	-35	85	-34	0	0	671	-438
82.65	77	-42	69	-40	0	0	493	-360
103.31	58	-56	51	-51	0	0	402	-388
123.98	42	-78	40	-69	0	0	386	-486
144.64	35	-102	34	-85	0	0	467	-658
165.30	29	-130	21	-102	0	0	622	-885
185.96	30	-144	18	-108	0	0	741	-987
206.63	16	-186	12	-134	0	0	901	-1077

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

D E A D L O A D S

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

L I V E L O A D S

	Moments							
	Lane		Truck		Special		1-Lane Truck	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	949	-8229	803	-3607	0	0	302	-1719
15.74	1256	-5368	980	-3133	0	0	481	-1383
31.49	2465	-3897	2278	-2747	0	0	1228	-1136
47.23	3875	-3331	3487	-2375	0	0	1767	-926
62.97	4837	-2849	4369	-2022	0	0	2134	-758
78.71	5331	-2381	4868	-1687	0	0	2338	-641
94.46	5335	-1910	4974	-1352	0	0	2388	-526
110.20	4837	-1432	4642	-1013	0	0	2265	-400
125.94	3802	-955	3771	-676	0	0	1915	-268
141.69	2143	-478	2178	-338	0	0	1144	-134
157.43	0	0	0	0	0	0	0	0

	Shears						Torque Maximums	
	Lane		Truck		Special			
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	182	-16	134	-12	0	0	901	-1077
15.74	131	-19	107	-14	0	0	834	-634
31.49	117	-19	100	-21	0	0	719	-517
47.23	100	-19	88	-26	0	0	558	-368
62.97	81	-23	74	-32	0	0	425	-256
78.71	64	-32	58	-39	0	0	355	-268
94.46	50	-47	46	-53	0	0	318	-329
110.20	43	-71	37	-72	0	0	394	-535
125.94	37	-89	28	-86	0	0	469	-683
141.69	32	-111	21	-106	0	0	469	-712
157.43	28	-135	18	-130	0	0	379	-635

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

DEAD LOADS

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

LIVE LOADS

Length	Moments Lane POS	Lane NEG	Truck POS	Truck NEG	Special POS	Special NEG	1-Lane Truck POS	1-Lane Truck NEG
.00	0	0	0	0	0	0	0	0
16.26	2252	-498	2273	-353	0	0	1206	-133
32.51	3994	-993	3937	-704	0	0	2016	-265
48.77	5078	-1484	4847	-1052	0	0	2380	-398
65.03	5598	-1980	5192	-1403	0	0	2502	-538
81.29	5595	-2477	5080	-1756	0	0	2442	-689
97.54	5062	-2978	4553	-2110	0	0	2216	-855
113.80	4049	-3489	3632	-2473	0	0	1826	-1046
130.06	2591	-4083	2380	-2844	0	0	1264	-1268
146.31	1310	-5611	1000	-3234	0	0	503	-1533
162.57	990	-8565	838	-3700	0	0	337	-1881

Length	Shears Lane POS	Lane NEG	Truck POS	Truck NEG	Special POS	Special NEG	Torque Maximums POS	Torque Maximums NEG
.00	116	-26	117	-18	0	0	496	-511
16.26	98	-30	97	-24	0	0	572	-504
32.51	81	-35	79	-30	0	0	583	-475
48.77	64	-39	67	-39	0	0	507	-417
65.03	44	-47	49	-44	0	0	350	-384
81.29	33	-61	39	-56	0	0	315	-402
97.54	28	-77	36	-70	0	0	295	-445
113.80	26	-92	33	-82	0	0	377	-544
130.06	22	-106	23	-92	0	0	525	-630
146.31	15	-121	11	-95	0	0	701	-688
162.57	14	-163	10	-117	0	0	854	-966

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

D E A D L O A D S

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

L I V E L O A D S

	-----------------	Moments	-----------------					
	--- Lane ---		-- Truck --		-- Special--		1-Lane Truck	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	990	-8565	838	-3700	0	0	337	-1881
21.34	1218	-4607	1211	-2431	0	0	653	-1185
42.68	2571	-2562	2849	-2036	0	0	1495	-907
64.01	4391	-2238	4194	-1660	0	0	2046	-685
85.35	5638	-2224	5032	-1297	0	0	2381	-507
106.69	6053	-2220	5298	-940	0	0	2486	-356
128.03	5639	-2225	5031	-1299	0	0	2380	-509
149.36	4391	-2239	4193	-1663	0	0	2047	-687
170.70	2575	-2568	2849	-2039	0	0	1497	-910
192.04	1219	-4611	1210	-2433	0	0	655	-1188
213.38	990	-8569	838	-3700	0	0	337	-1880

	----------	Shears	----------				-- Torque --	
	--- Lane ---		-- Truck --		-- Special--		--Maximums--	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	170	-15	124	-11	0	0	854	-966
21.34	129	-24	94	-18	0	0	842	-751
42.68	116	-30	92	-26	0	0	805	-594
64.01	94	-38	77	-38	0	0	650	-453
85.35	73	-41	63	-41	0	0	514	-428
106.69	53	-53	46	-46	0	0	440	-476
128.03	41	-73	41	-63	0	0	410	-544
149.36	38	-94	38	-77	0	0	455	-636
170.70	30	-115	26	-92	0	0	621	-789
192.04	24	-127	18	-93	0	0	765	-827
213.38	15	-168	10	-119	0	0	938	-877

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

D E A D L O A D S

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

L I V E L O A D S

Length	Moments: Lane POS	Lane NEG	Truck POS	Truck NEG	Special POS	Special NEG	1-Lane Truck POS	1-Lane Truck NEG
.00	990	-8569	838	-3700	0	0	337	-1880
16.26	1310	-5613	1000	-3233	0	0	502	-1532
32.51	2591	-4086	2378	-2843	0	0	1264	-1267
48.77	4045	-3488	3630	-2471	0	0	1823	-1043
65.03	5057	-2975	4550	-2108	0	0	2213	-854
81.29	5590	-2475	5075	-1754	0	0	2439	-688
97.54	5594	-1979	5188	-1402	0	0	2498	-538
113.80	5074	-1483	4842	-1051	0	0	2377	-398
130.06	3990	-992	3932	-703	0	0	2012	-264
146.31	2249	-497	2270	-352	0	0	1203	-132
162.57	0	0	0	0	0	0	0	0

Length	Shears: Lane POS	Lane NEG	Truck POS	Truck NEG	Special POS	Special NEG	Torque Maximums POS	Torque Maximums NEG
.00	166	-14	121	-10	0	0	938	-877
16.26	121	-15	95	-11	0	0	706	-689
32.51	107	-22	92	-23	0	0	649	-517
48.77	92	-26	82	-32	0	0	556	-379
65.03	77	-28	70	-36	0	0	438	-295
81.29	61	-33	56	-39	0	0	367	-334
97.54	47	-44	44	-49	0	0	352	-387
113.80	39	-64	39	-67	0	0	402	-495
130.06	35	-81	29	-78	0	0	495	-584
146.31	31	-99	22	-95	0	0	528	-586
162.57	28	-117	18	-117	0	0	512	-498

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APPENDIX C

Comparison of Analyses

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1. Modeling for Grid Analysis

a. General

The MSC/NASTRAN ver. 68 was used for the grid analysis. The finite element analysis using grid elements is considered exact within the confinement of classical strength of materials assumptions. The element stiffness matrices of the structural members, which are represented as one-dimensional entities are exact as they are based on an exact displacement shape function. Therefore, the analysis results are exact regardless of the grid refinement. Most of the commercially available grid analysis programs give twisting moments (pure torsion), vertical bending moments, and shearing forces. Although the CBEND element of MSC/NASTRAN utilized in the analysis has six degrees of freedom at each node, three degrees of freedom associated with horizontal bending about the vertical axis and axial behavior were suppressed to simulate the classical grid analysis output.

b. Coordinates

There are two types of coordinate systems employed in developing grid elements, i.e., rectangular Cartesian coordinates and cylindrical (or polar) coordinates. Grid elements based on cylindrical coordinates are ideally suited to be used in horizontally curved bridge girder analyses. Although grid elements based on rectangular coordinates may be used in curved girder analyses, a minimum of ten elements per span are usually needed to satisfactorily approximate the girder curvature. The grid element, CBEND, used in the analysis is formulated based on polar coordinates. As a result, a large number of elements is not needed to simulate the curvature.

c. Kinematic Degrees of Freedom

General purpose structural analysis computer program packages, including NASTRAN, ABAQUS, and SAP, assign six kinematic degrees of freedom at each node; i.e., three translations and three rotations. Special purpose grid analysis programs generally assign only three kinematic degrees of freedom at each node, i.e., two rotations with respect to two axes within the plane of the structure and one translation perpendicular to the plane of the deck. Therefore, grid analyses generally do not evaluate warping functions.

d. Boundary Conditions

The grid element based on cylindrical coordinates presents no special difficulties in modeling girder end boundary conditions whether the abutments or the interior piers are in the radial direction or skewed. However, the grid element based on rectangular coordinates almost always presents more difficulties in modeling boundary conditions regardless of the actual support condition. As the CBEND element used in the analysis is based on polar coordinates, no special modeling difficulties were encountered in defining the radially oriented supports. However, as discussed previously, the two bearings located at each support could not be physically represented in the grid model.

e. Dead Load

The non-composite dead loads (DL1) applied in the grid analysis were computed using the field sections given in Appendix A and the box girder bridge cross section shown in Figure 1. The distributed load (steel weight or deck weight) was lumped at each node using a single or double tributary area concept. The composite dead load (DL2) consisted of the future wearing surface load of 30 psf and the concrete parapets assumed to weigh 530 psf. The parapet dead loads were resolved into equivalent vertical loads and torques applied at the node points along the girder.

f. Live Load

Truck wheel loads and equivalent lane loadings were distributed to adjacent grid points using a double-interpolation scheme. Work equivalent bending moments and torques were neglected. Sample examples run with these fixed-end actions did not show any noticeable differences. The middle wheel of the HS25 truck was placed at the approximate location for maximum positive moment in Span 1 (at $0.4L_1$) and the middle wheels of two HS25 trucks were placed at $0.4L_1$ and $0.4L_2$ measured from the interior support for maximum negative moment. The direction of the truck was determined from the ordinates of the straight-girder influence lines. The minimum rear-axle spacing of 14 ft was assumed to govern. The trucks were shifted laterally within their design lanes to put the maximum wheel loads over the particular girder under investigation. Impact factors used were those given in Article 3.5.6.2 of the Recommended Specifications. Multiple presence factors specified in **AASHTO Article 3.12.1** were applied to the live loads to account for the probability of coincident loading.

2. Modeling for M/R Method Analysis

a. General

The original concept and procedures presented by Tung and Fountain (in "Approximate Torsional Analysis of Curved Box Girders by the M/R Method," *Engineering Journal*, AISC, Vol. 7, No. 3, July 1970) were followed to perform the approximate analysis.

To ensure a reasonable degree of accuracy in the analysis of continuous curved girders, the M/R method recommends that the central angle of each span and the weighted average flexural and torsional rigidity ratio, EI/GJ, in each span not exceed the following limits: (a) 30° and 2.5, or (b) 25° and 4.0. As the largest central angle for the example box girder is 17° and the flexural and torsional rigidity ratio, EI/GJ, is 4.0 for non-composite dead loads and 2.0 for composite dead and live loads, the example box girder meets the recommended limitations. Therefore, the results from the M/R method should compare reasonably well with the results from more refined methods.

b. Coordinates

The vertical bending moments and shears are computed by first straightening each curved box girder to its full developed length followed by calculation of the moments and shears in the equivalent straight girders by any conventional method of analysis. As there are three different sections used along the length of each box girder, vertical bending moments and shears were determined using a grid analysis computer program. The girder torques are computed by applying a distributed M/R loading to developed straight conjugate beams, where M is the vertical bending moment determined according to the above procedure and R is the girder radius.

c. Boundary Conditions

Since the vertical bending moments and shears are evaluated for an equivalent straight girder with the same arc span length (developed length) as the curved girder, the boundary conditions for determining these actions are the same as for an individual straight girder; that is, the girder is assumed to be simply supported at the abutments and continuous over the interior pier. For determining the girder torques from the conjugate beam analogy, the M/R method gives much better results when the supports for the box are torsionally-fixed, which is the case when the box is supported on two bearings as is the case in this example. However, it is not unusual to note a difference of up to 30% between the torsion at the abutments evaluated from a finite element analysis and the M/R method when the end support boundary conditions are assumed to be torsionally simple (supported on a single bearing). The Tung and Fountain paper recommends that there be at least one torsionally-fixed support in each span to successfully apply the M/R method. Furthermore, if an interior support point is torsionally free, it is recommended that the span of the conjugate beam be taken between two adjacent torsionally-fixed supports in the torsional analysis.

d. Dead Load

The uniform non-composite dead load (DL1) applied to each girder in the M/R analysis was computed using the field sections given in Appendix A and the box girder bridge cross section shown in Figure 1. The composite dead load (DL2), including the parapet dead load and future wearing surface load, was assumed to be uniformly distributed to each girder.

e. Live Load

As the spacing between the centers of the two box girders exceeds 14 feet, the wheel load distribution for the straight-girder analysis is determined according to footnote "f" of **AASHTO Table 3.23.1** (see also the note after Table C5). As in the case of the grid analysis, trucks or equivalent lane loads were placed at the approximate locations for producing the maximum positive and negative moments. The direction of the truck was determined from the ordinates of the straight girder influence lines. Impact factors used were taken from Article 3.5.6.2 of the Recommended Specifications.

Table C1 Dead Load (Structural Steel) Analysis Comparison

		Max Moment			Max Shear per Web*	
		+ M, End Span (k-ft)	-M, Interior Pier (k-ft)	+ M, Center Span (k-ft)	Abutment (kips)	Interior Pier (kips)
G1	FEM	1,049	2,969	1,118	27.0	44.0
	Grid	1,024	2,619	1,103	23.0	37.4
	M/R	1,012	2,523	1,105	23.0	33.0
G2	FEM	1,116	3,155	1,190	31.0	46.0
	Grid	1,097	2,787	1,176	23.5	38.7
	M/R	1,104	2,754	1,206	23.0	34.6

*The maximum shear per web includes the additional shear due to torsion.

Table C2 Dead Load (Concrete Deck) Analysis Comparison

		Max Moment			Max Shear per Web*	
		+ M, End Span (k-ft)	-M, Interior Pier (k-ft)	+ M, Center Span (k-ft)	Abutment (kips)	Interior Pier (kips)
G1	FEM	4,321	11,629	4,442	114.0	171.0
	Grid	4,368	11,096	4,682	96.1	158.9
	M/R	4,394	10,956	4,798	95.0	141.6
G2	FEM	4,633	12,272	4,826	110.0	185.0
	Grid	4,658	11,832	4,993	99.7	165.4
	M/R	4,690	11,685	5,114	98.0	146.2

*The maximum shear per web includes the additional shear due to torsion.

Table C3 Dead Load (Superimposed Dead Load) Analysis Comparison

		Max Moment			Max Shear per Web*	
		+ M, End Span (k-ft)	-M, Interior Pier (k-ft)	+ M, Center Span (k-ft)	Abutment (kips)	Interior Pier (kips)
G1	FEM	1,706	4,098	1,944	58.0	94.0
	Grid	1,602	3,873	1,781	34.0	55.1
	M/R	1,582	3,816	1,828	31.0	50.7
G2	FEM	1,733	4,473	1,890	91.0	102.0
	Grid	1,708	4,130	1,899	35.2	56.9
	M/R	1,721	4,084	1,934	32.5	52.4

*The maximum shear per web includes the additional shear due to torsion.

Table C4 Live Load (HS25 Truck) Analysis Comparison

		Max Moment			Max Shear per Web	
		+ M, End Span (k-ft)	-M, Interior Pier (k-ft)	+ M, Center Span (k-ft)	Abutment (k)	Interior Pier (k)
G1	FEM	4,972	3,607	5,089	-	-
	Grid	5,111	3,438	5,338	-	-
	M/R	5,101	3,413	5,333	-	-
G2	FEM	5,192	3,700	5,298	-	-
	Grid	5,278	3,550	5,512	-	-
	M/R	5,268	3,525	5,507	-	-

As the truck positions are varied for each category of Max Moment, web shears are not listed.

Table C5 Live Load (Lane) Analysis Comparison

		Max Moment			Max Shear per Web	
		+ M, End Span (k-ft)	-M, Interior Pier (k-ft)	+ M, Center Span (k-ft)	Abutment (k)	Interior Pier (k)
G1	FEM	5,332	8,230	5,786	-	-
	Grid	5,446	8,235	6,018	-	-
	M/R	5,452	8,166	6,054	-	-
G2	FEM	5,598	8,569	6,053	-	-
	Grid	5,759	8,728	6,368	-	-
	M/R	5,718	8,579	6,354	-	-

As the lane positions are varied for each category of Max Moment, web shears are not listed.

Note:

Vertical bending moments computed by the M/R method based on the live-load lateral distribution factor for straight box girders specified in **AASHTO Article 10.39.2** are 92 percent of those given in Tables C4 and C5. The wheel load distribution factor, W_L, according to **AASHTO Equation 10-70** is 2.93 wheels, which is 92 percent of that determined by the simple beam assumption specified in footnote "f" of **AASHTO Table 3.23.1**.

Appendix D

Selected Design Forces and Girder 2 Section Properties

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Table D1 Girder 2 Selected Moments (k-ft)
and Web Fatigue Shears (kips)

Section Node	Steel	Deck	Cast(#)[1]	SupImp[2]	Ovrlod[3]	LLmax[4]	Fat_{min}[5]	Fat_{max}[5]
1-1 10	1,144	4,747	7,062(1) 2,073(2)	1,771	4,318 -1,386	5,398 -1,733	-239 V=-14k	1,258 V=23k
2-2 20.3 Splice	462	1,941	5,830(1) -3,554(2)	754	3,952 -2,443 V=-62k	4,940 -3,054	-451 V=-26k	1,108 V=12k
3-3 28	-892	-3,579		-1,166	2,073 -3,267	2,591 -4,084	-649 V=-34	646 V=8k
4-4 32	-1,896	-7,599		-2,610	1,049 -4,490	1,311 -5,612	-784 V=-36	258 V=2k
5-5 36	-3,154	-12,272		-4,473	793 -6,853	991 -8,566	-961 V=-44k	173 V=4k
6-6 40	-1,956	-7,866		-2,693	797 -4,425	996 -5,531	-656 V=-5k	192 V=37k
7-7 44	-979	-4,015		-1,303	1,546 -2,818	1,933 -3,522	-542 V=-8k	554 V=35k
8-8 48	184	634	-2,939(1) 1,985(2)	328	2,866 -1,842	3,582 -2,303	-403 V=-11k	917 V=32k
9-9 62	1,190	4,826	-2,811(1) 5,941(2)	1,890	4,842 -1,777	6,053 -2,221	-182 V=-18k	1,271 V=18K

[1](#) denotes Deck Cast number
Cast #1 begins at Section 1-1 and ends at Section 3-3
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]Ovrlod - Unfactored live-load plus impact moment due to multiple lanes of HS20. Impact is according to Article 3.5.6.2.

[4]LLmax - Unfactored live-load plus impact moment due to multiple lanes of HS25. Impact is according to Article 3.5.6.2.

[5]Fat - Maximum and minimum fatigue moment due to one fatigue vehicle plus 15% impact times the load factor of 0.75 specified in Article 3.5.7.1. Vertical shears in the critical web (V) due to the factored fatigue vehicle are given in the Fat columns. Fatigue moments and shears are increased by 10 percent to allow for warping.

All live load moments and shears, including fatigue moments and shears, include centrifugal force effects.

Multiple presence reduction factors (**AASHTO Article 3.12**) were considered in determining Ovrlod and LLmax.

The location of nodes and sections may be found by referring to Figure 2 and Appendix A.

Table D2 Girder 2, Span 1 Tenth-Point Shear (kips)

Tenth Point	Steel	Deck	SupImp	Tot DL	LL (LL + I)	(5/3)LL	1.3(TotDL +5/3[LL])
0	31	110	91	232	117	195	555
1	19	74	39	132	98	163	384
2	11	44	26	81	81	135	281
3	5	21	14	40	67	112	198
4	-7	-26	-8	-41	-47	-78	-155
5	-11	-45	-14	-70	-61	-102	-224
6	-17	-69	-28	-114	-77	-128	-315
7	-24	-97	-40	-161	-92	-153	-408
8	-29	-117	-51	-197	-106	-177	-486
9	-35	-137	-62	-234	-121	-202	-567
10	-46/47	-185/185	-96/102	-327/334	-163/170	-272/283	-779/802

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

Table D3 Girder 2, Selected Torques (k-ft)

Section Node	Steel	Deck	SupImp[1]	Ovrlod[2]	LL_{max}[3]	Fat[4]	Fat[4]
1-1 10	59	205 464[5]	95	350 -318	437 -398	-85	174
2-2 20.3 Splice	-36	-125 -188	-134	236 -356	295 -445	-165	96
3-3 28	-28	-53 352	-247	420 -504	525 -630	-238	108
4-4 32	-10	63 -420	-273	561 -550	701 -688	-241	132
5-5 36	-22/36	48/-33	-346/447	683[6] -765	854[6] -966	-232[6]	254[6]
6-6 40	23	-52 -335	378	680 -672	850 -840	-171	260
7-7 44	28	13 -305	336	658 -540	822 -675	-105	264
8-8 48	72	211 -298	289	576 -432	720 -540	-90	244
9-9 62	0	0 0	0	352 -381	440 -476	-100	116

[1]SupImp - Unfactored superimposed dead load
[2]Ovrlod - Unfactored live-load plus impact torque due to multiple lanes of HS20. Impact is according to Article 3.5.6.2.
[3]LL_{max} - Unfactored live-load plus impact torque due to multiple lanes of HS25. Impact is according to Article 3.5.6.2.
[4]Fat - Maximum and minimum torques due to one fatigue vehicle plus 15% impact times the load factor of 0.75 specified in Article 3.5.7.1.
[5]Bottom value, where listed, is the torque due to Cast #1.
[6]Only the minimum and maximum live-load torques are reported at the pier section.

All live load torques, including fatigue torques, include centrifugal force effects. Multiple presence reduction factors (**AASHTO Article 3.12**) were considered in determining Ovrlod and LL_{max}.
The location of nodes and sections may be found by referring to Figure 2 and Appendix A.

Table D4 Top Flange Bracing Forces G2 Span 1 (kips)

Element	Steel	Deck	SupImp	LL+I	Fact	*Cast #1	Cast #2	Const
1	-13	-40	-7	-2	-82	-100	7	-158
2	6	12	-4	-3	19	60	0	92
3	-11	-39	-11	-5	-90	-90	13	-141
4	-4	-20	-9	-6	-56	0	7	4
5	-2	-7	-9	-6	-36	-37	18	-55
6	-10	-38	-9	-6	-87	-55	15	-91
7	7	25	-3	-5	27	25	23	77
8	-6	-15	-4	-4	-41	-76	31	-115
9	11	31	6	5	73	70	15	134
10	9	46	6	5	90	-51	64	-59
11	7	42	5	4	79	71	-31	109
12	12	33	7	4	76	-25	82	97
13	-8	-16	-5	-3	-44	43	-76	-57

Notes:
1. Casts consider overhang bracing forces
2. Fact = 1.3[DL + 5/3(LL+I)]
3. Const = 1.4[Steel + Cast #1]
   or 1.4[Steel + Cast #1 + Cast #2]

*These values are taken from Figure 6

Table D5 Selected Section Properties for Girder 2

Section Node	Section Size	Section Type	Moment of Inertia	Neutral Axis B	Neutral Axis T
1-1 10 2-2 20.3	2-16 x 1 2-78 x 0.5625 83 x 0.625 A=181 in^2	Noncomp	185,187	36.83	42.80
		Comp DL	354,925	55.35	24.27
		CompLL	479,646	68.84	10.78
3-3 32	2-18 x 1.5 2-78 x 0.5625 83 x 1 LS WT8x28.5 A=243 in^2	Noncomp	275,175	35.32	45.18
		CompDL	475,329	51.05	29.45
		CompDL Bars	292,858	36.72	43.78
		CompLL	650,889	64.77	15.73
		CompLL Bars	325,531	39.30	41.20
5-5 36	2-18 x 3 2-78 x 0.5625 83 x 1.5 LS WT8x28.5 A=338 in^2	Noncomp	438,966	38.81	43.69
		CompDL	633,467	50.44	32.06
		CompDL Bars	454,805	39.76	42.74
		CompLL	836,080	62.50	20.00
		CompLL Bars	484,714	41.55	40.95

Legend:

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

Composite section properties are computed using the deck area including the overhang and half of the deck width between girders. The area of haunch is not included. The haunch depth is considered. The longitudinal reinforcing steel area equal to 20.0 square inches per box is assumed placed at mid-thickness of the deck.

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

The area and moment of inertia of the box section include the longitudinal component of the top flange bracing area, the longitudinal flange stiffener (where present) and the 1-inch bottom flange lips. A single top-flange bracing member of 8.0 in^2 placed at an angle of 30 degrees from tangent to the girder is assumed.

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Appendix E

Sample Calculations

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

In accordance with Article 13.2, the web of the non-composite section must be checked for steel weight and for the Cast#1 of the concrete deck. The web check for strength is not shown, but is similar to the check illustrated below. Moment values are from Table D1.

Load	Moment
Steel	1,144
Cast #1	7,062
Total Moment	8,206 k-ft

Constructibility Load Factor = 1.4 according to the provisions of Article 3.3.

Compute the vertical bending stress at the top of the web due to the above moment using the section properties for the non-composite section from Table D5.

$D = 78/\cos 14.3° = 80.5$ in.

$D_c = 42.80 - 1.00 = 41.80$ in. $41.80/\cos 14.3° = 43.14$ in.

$$f_{top\ web} = \frac{8{,}206 \times 41.80}{185{,}187} \times 12 \times 1.4 = -31.12 \text{ ksi}$$

As specified in Article 13.2, critical stresses in girder webs for constructibility are to be determined according to the provisions of Article 6.

Compute the critical bend buckling stress for the transversely stiffened web according to Article 6.3.1.

$$F_{cr} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} \le F_y; \quad \text{where: } k = 9\left(\frac{D}{D_c}\right)^2 \ge 7.2 \qquad \text{Eq (6-8)}$$

$$k = 9 \times \left(\frac{80.5}{43.14}\right)^2 = 31.3 > 7.2 \text{ OK}$$

$$F_{cr} = \frac{0.9 \times 29{,}000 \times 31.3}{\left(\frac{80.5}{0.5625}\right)^2} = 39.89 \text{ ksi} < F_y$$

$$\frac{|-31.12|}{39.89} = 0.78 < 1.00 \text{ OK}$$

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

The girder must be checked for steel weight and for Cast #1 of the concrete deck on the non-composite section according to the provisions of Article 13.7. The factored steel stresses during the sequential placement of the concrete are not to exceed the critical stresses specified in Article 13.2. The effect of the overhang brackets on the flanges must also be considered according to Article 13.8 since G2 is an outside girder.

Overhang Bracket Load

Since G2 is an outside girder, half of the overhang weight is assumed placed on the girder and the other half is placed on the overhang brackets, as shown in Figure E1.

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

Compute the vertical load on the overhang brackets.

$$\text{Deck } \frac{1}{2} \text{ x } 4.0 \text{ ft x } \frac{10 \text{ in.}}{12 \text{ in/ft}} \text{ x } 150 \text{ lbs/ft}^3 = 250 \text{ lbs/ft}$$

$$\text{Deck forms + Screed rail} = \underline{224 \text{ lbs/ft}}$$

$$\text{Uniform load on brackets} = 474 \text{ lbs/ft}$$

Compute the lateral force on the flanges due to overhang brackets. See Figure E1.

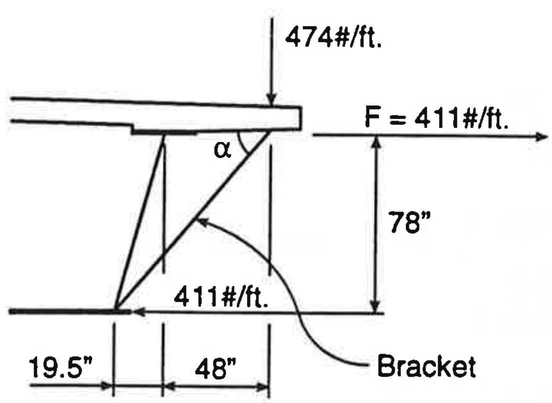

Figure E1 Overhang Bracket Loading

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

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

$$F = 474 \,/\, \tan(49.1^\circ) = 411 \text{ lbs/ft}$$

Compute the lateral flange moment on the outermost tub flange due to the overhang forces in accordance with Article 13.8. The lateral flange moment at the brace points due to the overhang forces is negative in the top flange of G2 on the outside of the curve because the stress due to the lateral moment is compressive on the convex side of the flange at the brace points (see Article 5.1). The opposite would be true on the convex side of the G1 top flange on the inside of the curve at the brace points, as illustrated in later calculations.

$$M_{lat} = 0.08F\ell^2 = 0.08 \times 411 \times 16.3^2/1{,}000 = -8.74 \text{ k-ft} \qquad \text{Eq (C13-1)}$$

In addition to the moment due to the overhang brackets, the inclined webs of the box cause a lateral force on the top flanges. This force is relatively small in this case and will be ignored. A third source of lateral bending is due to curvature, which can be conservatively estimated by the approximate V-load Equation (4-1) given in the Recommended Specifications, as illustrated below.

As specified in Articles 10.1 and 13.2, the strength of a flange with a single web (including tub flanges) at the constructibility limit state is to be computed according to the criteria for non-compact flanges given in Article 5. The strength of a flange according to Article 5 and the approximate V-load Equation (4-1) both assume that the lateral bending is equal at each end of a panel. As can be seen from an examination of Figures 3 through 7, this is obviously not the case.

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

From Table D1, the moment due to the steel weight plus Cast #1 is 1,144+7,062=8,206 k-ft. The load factor for constructibility checks is 1.4 according to the provisions of Article 3.3. Using the section properties from Table D5, the vertical bending stress, f_b, in the top flange is computed as:

$$f_{top\ flg} = f_b = \frac{8{,}206 \times 12 \times 42.80}{185{,}187} \times 1.4 = -31.86 \text{ ksi}$$

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

Top flange: 16 in x 1 in; $S = \frac{1}{6}(1)(16)^2 = 42.7 \text{ in}^3$

As defined in Article 5.2.1, f_m, is the factored lateral flange bending stress at the critical brace point due to effects other than curvature.

$$f_m = \frac{-8.74 \times 12}{42.7} = -2.46 \text{ ksi; Load Factor} = 1.4$$

$$f_m = -2.46 \times 1.4 = -3.44 \text{ ksi; } \frac{f_m}{f_b} = \frac{-3.44}{-31.86} = 0.11 \text{ (the ratio is positive)}$$

The top flange size is constant between brace points in this region. Article 5.1 specifies that f_b be taken as the largest factored average flange stress at either brace point when checking the strength of flanges with one web. The section under investigation is not located at a brace point. In positive-moment regions, the largest value of f_b may not necessarily be at either brace point. Generally though, f_b will not be significantly larger than the value at adjacent brace points, which is the case in this example. Therefore, the computed value of f_b at Section 1-1 will be conservatively used in the strength check. The approximate Eq (4-1) is used below to compute the lateral flange bending moment due to curvature. Eq (4-1) assumes the presence of a cross frame at the point under investigation and, as mentioned previously, that M is constant over the distance between brace points. Although the use of Eq (4-1) is not theoretically pure for tub girders or at locations in-between brace points, it may conservatively be used. For a single flange, consider only half of the girder moment due to steel and Cast #1. M = 8,206/2 = 4,103 k-ft.

$$M_{lat} = \frac{6}{5}\frac{M\ell^2}{RD} = \frac{6}{5}\frac{(4{,}103 \times 16.3^2)}{716.3 \times 78} = -23.41 \text{ k-ft} \qquad \text{Eq (4-1)}$$

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

$$M_{tot\ lat} = -23.41 + (-8.74) = -32.15 \text{ k-ft; } -32.15 \times 1.4 = -45.01 \text{ k-ft}$$

f_ℓ is defined as the sum of f_m and the factored lateral flange bending stress due to curvature, f_w.

$$f_\ell = \frac{-45.01 \times 12}{42.7} = -12.65 \text{ ksi; } |f_\ell| < 0.5F_y = 25.0 \text{ ksi OK} \qquad \text{Eq (5-1)}$$

$$f_b + f_\ell = -31.86 + (-12.65) = -44.51 \text{ ksi}$$

<u>Girder Stress Check Section 1-1 G2 Node 10</u>
<u>Constructibility - Top Flange (continued)</u>

Since f_b exceeds $0.33F_y = 16.5$ ksi:

$$|f_\ell / f_b| \leq 0.5$$

$$|f_\ell / f_b| = 12.65/31.86 = 0.4 < 0.5 \text{ OK}$$

In order to limit the factored stress in the compression flange to the yield stress during construction, Article 5.2.2 must be used.

$$F_{cr1} = F_{bs}\rho_b\rho_w \qquad \text{(Eq 5-8)}$$

$$F_{bs} = F_y\,(1 - 3\lambda^2) \quad \text{where: } \lambda = \frac{1}{\pi}\frac{12\ell}{b_f}\sqrt{\frac{F_y}{E}} \qquad \text{(Eq 5-5)}$$

b_f is to be taken as $0.9b_f$ in computing λ if the section is not doubly symmetric (Article 5.2.1).

$$\lambda = \frac{1}{\pi}\frac{12 \times 16.3}{0.9 \times 16}\sqrt{\frac{50 \text{ ksi}}{29{,}000 \text{ ksi}}} = 0.18$$

$$F_{bs} = 50\,(1 - 3 \times 0.18^2) = 45.14 \text{ ksi}$$

Compute the ρ factors according to the provisions of Article 5.2.2.

$$\rho_b = \frac{1}{1 + \frac{\ell}{R}\frac{12\ell}{b_f}} = \frac{1}{1 + \frac{16.3}{716.3} \times \frac{12 \times 16.3}{16}} = 0.78$$

$$\rho_{w1} = \frac{1}{1 - \frac{f_m}{f_b}\left(1 - \frac{12\ell}{75b_f}\right)} = \frac{1}{1 - 0.11\left(1 - \frac{12 \times 16.3}{75 \times 16}\right)} = 1.10$$

$$\rho_{w2} = \frac{0.95 + \dfrac{\frac{12\ell}{b_f}}{30 + 8{,}000\left(0.10 - \frac{\ell}{R}\right)^2}}{1 + 0.60\left(\frac{f_m}{f_b}\right)}$$

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

$$= \frac{0.95 + \dfrac{\dfrac{12 \times 16.3}{16}}{30 + 8{,}000 \times \left(0.1 - \dfrac{16.3}{716.3}\right)^2}}{1 + 0.60 \times 0.11} = 1.04$$

$\rho_w = \rho_{w1}$ or ρ_{w2} whichever is smaller since $f_m/f_b \geq 0$

$\rho_b \rho_w = 0.78 \times 1.04 = 0.81$

$F_{cr1} = F_{bs} \rho_b \rho_w = 45.14 \times 0.81 = 36.56$ ksi

$F_{cr2} = F_y - |f_\ell|$ Eq (5-9)

$F_{cr2} = 50 - |-12.65| = 37.35 \text{ ksi} > F_{cr1} = 36.56$ ksi

$\therefore F_{cr} = 36.56$ ksi

$$\frac{|-31.86|}{36.56} = 0.87 < 1.00 \text{ OK}$$

Check the width-to-thickness ratio of the top flange:

$$\frac{b_f}{t_f} \leq 1.02\sqrt{\frac{E}{(f_b + f_\ell)}} \leq 23$$ Eq (5-7)

$$1.02\sqrt{\frac{29{,}000}{44.51}} = 26.04 > 23$$

$$\frac{b_f}{t_f} = \frac{16}{1} = 16 < 23 \text{ OK}$$

Check Eq (5-3):

$\ell \leq 25b_f \leq R/10$ Eq (5-3)

16.3(12) = 196 in < 25(16) = 400 in OK

16.3 ft < 716.3/10 = 71.6 ft OK

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

Since the load on the overhang bracket produces a lateral flange moment at the brace points on the convex side of the G1 inner top flange of the opposite sense from that on the convex side of the G2 outer top flange, the ratio, f_m / f_b, for G1 due to the overhang force is negative.

Check the constructibility stress in the G1 top flange on the inside of the curve at this section.

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

Load	Moment
Steel	1,075
Cast #1	6,476
Total Moment	7,551 k-ft

Compute the lateral flange bending moment due to curvature using Equation (4-1).

$$M_{lat} = \frac{6}{5}\frac{M\ell^2}{RD}$$

For a single flange, consider only half of the girder moment due to Steel plus Cast #1.

$$M = 7{,}551/2 = 3{,}776 \text{ k-ft}$$

Unbraced length of flange = 15.7 feet.

$$M_{lat} = \frac{6}{5}\frac{3{,}776 \times 15.7^2}{683.8 \times 78} = -20.94 \text{ k-ft}$$

Compute the factored lateral flange bending stress due to curvature.

$$f_w = \frac{M_{lat}}{S} = \frac{-20.94}{42.67} \times 12 \times 1.4 = -8.24 \text{ ksi}$$

Compute the lateral flange moment and factored lateral flange bending stress due to the overhang bracket load.

$$M_{lat} = 0.08F\ell^2 = 0.08 \times 411 \times 15.7^2/1{,}000 = 8.10 \text{ k-ft}$$

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

$$f_m = \frac{8.10}{42.7} \times 12 \times 1.4 = 3.19 \text{ ksi}$$

$$f_\ell = f_w + f_m = -8.24 + 3.19 = -5.05 \text{ ksi}; \ |f_\ell| < 0.5F_y = 25 \text{ ksi OK}$$

Compute the vertical bending stress at the top of the steel.

$$f_b = \frac{7{,}551 \times 42.80}{185{,}187} \times 12 \times 1.4 = -29.32 \text{ ksi}$$

Compute the ratio of lateral bending stress to vertical bending stress, $|f_\ell/f_b|$.

The ratio, $|f_\ell/f_b|$, must be less than 0.5 since f_b exceeds $0.33F_y$ = 16.5 ksi according to the provisions of Article 5.1.

$$\left|\frac{f_\ell}{f_b}\right| = \left|\frac{-5.05}{-29.32}\right| = 0.17 < 0.5 \text{ OK}$$

Compute the critical vertical bending stress according to the provisions of Article 5.2.2.

Eq (5-8)

$$F_{cr1} = F_{bs}\rho_b\rho_w$$

From Section 1-1 G2 Node 10, Constructibility - Top Flange , page 69,

$$F_{bs} = 45.14 \text{ ksi}$$

Compute the ρ factors according to the provisions of Article 5.2.2.

$$\frac{f_m}{f_b} = \frac{3.19}{-29.32} = -0.11$$

$$\rho_b = \frac{1}{1 + \frac{\ell}{R}\frac{12\ell}{b_f}} = \frac{1}{1 + \frac{15.7}{683.8} \times \frac{12 \times 15.7}{16}} = 0.79$$

$$\rho_{w1} = \frac{1}{1 - \frac{f_m}{f_b}\left(1 - \frac{12\ell}{75b_f}\right)} = \frac{1}{1 - (-0.11)\left(1 - \frac{12 \times 15.7}{75 \times 16}\right)} = 0.92$$

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

Since $f_m/f_b < 0$, $\rho_w = \rho_{w1} = 0.92$

$\rho_b \rho_w = 0.79 \times 0.92 = 0.73$

$F_{cr1} = 45.14 \times 0.73 = 32.95$ ksi

$F_{cr2} = F_y - |f_\ell|$ Eq (5-9)

$F_{cr2} = 50 - |-5.05| = 44.95 \text{ ksi} > F_{cr1} = 32.95$ ksi

$\therefore F_{cr} = 32.95$ ksi

$$\frac{|-29.32|}{32.95} = 0.89 < 1.00 \text{ OK}$$

Check the width-to-thickness ratio of the top flange:

$$\frac{b_f}{t_f} \leq 1.02\sqrt{\frac{E}{(f_b + f_\ell)}} \leq 23$$ Eq (5-7)

$$1.02\sqrt{\frac{29{,}000}{|-29.32 + (-5.05)|}} = 29.63 > 23$$

$$\frac{b_f}{t_f} = \frac{16}{1} = 16 < 23 \text{ OK}$$

Separate calculations indicate that Eq (5-3) is also satisfied.

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

Check the fatigue stress in the bottom flange at this section according to the provisions of Articles 3.5.7 and 10.6.1. The fatigue design life is 75 years.

Base metal at transverse stiffener weld terminations and at stiffener-connection plate welds at locations subject to a net tensile stress must be checked for Category C' (refer to **Table 6.6.1.2.3-1** of **AASHTO LRFD**). 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** in **AASHTO LRFD**).

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 3.5.6.3, the impact allowance is 0.15. One-half of the fatigue threshold is specified as the limiting stress range for this case since it is assumed that at some time in the life of the bridge, a truck loading of twice the magnitude of the factored fatigue truck will occur. By using half of the fatigue threshold, twice the factored truck is actually considered. According to the provisions of Article 4.5.2, uncracked concrete section properties are to be used for fatigue checks. As specified in Article 10.6.1, the longitudinal stress range in tub girders is to be computed as the sum of the stress ranges due to vertical bending and warping. In this example, the fatigue moments have been increased by 10 percent to allow for warping.

M_{min}	-239 k-ft	Table D1
M_{max}	1,258 k-ft	Table D1
M_{range}	1,497 k-ft	

According to **AASHTO LRFD Article 6.6.1.2**, the limiting stress range for Category C'=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** of **AASHTO LRFD**.

Compute the range of vertical bending stress at the top of the bottom flange (section properties are taken from Table D5):

$$f_{range} = \frac{1{,}497 \times (68.84 - 0.625)}{479{,}646} \times 12 = 2.55 \text{ ksi}$$

$$\frac{2.55}{6.0} = 0.43 < 1.00 \text{ OK}$$

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

Determine the required pitch of the shear connectors for fatigue at this section according to the provisions of Articles 7.2.2 and 7.3.3.

The fatigue threshold for one stud shear connector in kips, Z_r, is defined in **AASHTO LRFD Article 6.10.7.4.2** as $(5.5/2)d^2$.

Use: 3 - 6"x7/8"ϕ studs/row.
Fatigue threshold for one 7/8"ϕ shear stud=$(5.5/2) \times 0.875^2 = 2.105$ kips.
Fatigue threshold for 3 such connectors/row=$nZ_r = 3 \times 2.105 = 6.315$ kips/row.

From Table D1, the bending plus torsional shear range due to one factored fatigue truck = $23 + |-14| = 37$ kips. The shear values in Table D1 are vertical shears and are for the critical web, which is subject to additive bending and torsional shears. The values are increased by 10 percent to account for warping. As specified in Article 7.3.3, the shear connector arrangement determined for the critical web will also be used for the top flange attached to the non-critical web.

According to the provisions of Articles 4.5.2 and 7.3.1, the entire deck cross sectional area is assumed to be effective. Deck thickness, t=9.5 in. Modular ratio, n=7.56.

$$\text{Effective deck width} = \frac{1}{2}(10 + 12.5) \times 12 = 135 \text{ in. (over critical web)}$$

$$\text{Transformed deck area} = \frac{\text{Area}}{n} = \frac{135 \times 9.5}{7.56} = 169.6 \text{ in}^2$$

Compute the first moment of the deck with respect to the neutral axis of the uncracked live load composite section.
Determine the distance from the center of the deck to the neutral axis.
Section properties are from Table D5.
Neutral axis of the section is 10.78 in from the top of the steel.
Moment arm of the deck = Neutral axis - t_{flg} + haunch + $t_{deck}/2$
Moment arm = 10.78 - 1.0 + 4.0 + 9.5/2 = 18.53 in.

Compute the longitudinal fatigue shear range, V_{fat}. Use one-half of the moment of inertia.

$$Q = 169.6 \times 18.53 = 3{,}143 \text{ in}^3$$

$$V_{fat} = \frac{VQ}{I} = \frac{37 \times 3{,}143}{479{,}646 \times 0.5} = 0.48 \text{ k/in}$$

As specified in Article 7.3.3, the radial shear range, F_{fat}, due to curvature may be ignored in the design of box girders.

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

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

$$\text{Shear stud pitch} = \frac{nZ_r}{V_{sr}} = \frac{6.315}{0.48} = 13.1 \text{ in/row}$$

Although not illustrated here, the number of shear connectors that is provided must also be checked for ultimate strength according to the provisions of Articles 7.2.1 and 7.3.2.

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

Check the bottom (box) flange for strength at this section according to the provisions of Article 10.4.2.4. The section will be checked for the Group I load combination in the following computations. Assume one longitudinal flange stiffener.

Load	Moment (k-ft)	
Steel	-3,154	Table D1
Deck	-12,272	Table D1
Total non-composite	-15,426	
Superimposed DL	-4,473	Table D1
Live load HS25	-8,566	Table D1

Compute the factored vertical bending stress in the bottom flange due to dead and live load. For loads applied to the composite section, assume a cracked section, as specified in Article 4.5.2. Section properties are from Table D5. Shear lag need not be considered since the box flange width does not exceed one-fifth of the distance between the points of contraflexure on either side of the pier section (Article 10.3.1). The longitudinal vertical bending stress is, therefore, assumed to be uniform across the flange because shear lag need not be considered and because it is assumed that the spacing of the internal bracing is such that the longitudinal warping stress at the strength limit state is limited to 10 percent of the vertical bending stress (Article 10.2.2.3).

$$f_{bot\ flg} = f_b = \left[\frac{-15{,}426 \times 38.81}{438{,}966} + \frac{-4{,}473 \times 39.76}{454{,}805} + \frac{-8{,}566(5/3) \times 41.55}{484{,}714}\right] \times 12 \times 1.3 = -46.47 \text{ ksi}$$

Compute the factored St. Venant torsional shear stress, f_v, in the bottom flange due to the non-composite loads. Torques are taken from Table D3.

Load	Torque (k-ft)
Steel	-22
Deck	48
Total Torque	26

Compute the enclosed area of the non-composite box.

$$A_o = \frac{(120 + 81)}{2} \times 80.25 \times \frac{1}{144} = 56.0 \text{ ft}^2$$

$$f_v = \frac{T}{2A_o t_f} = \frac{26}{2 \times 56.0 \times 1.5} \times \frac{1}{12 \text{ in/ft}} \times 1.3 = 0.017 \text{ ksi}$$

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

where: T = torque; A_o = enclosed area of box; t_f = bottom flange thickness

Compute the factored torsional shear stress in the bottom flange due to the composite loads. Torques are taken from Table D3.

Load		Torque (-)		Torque (+)
SupImp DL		-346		447
Live Load	5/3 (-966)	-1,610	5/3 (854)	1,423
Total Torque		-1,956 k-ft		1,870 k-ft

Since $|-1{,}956| > 1{,}870$, use negative torque.

Compute the enclosed area of the composite box.

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

$$f_v = \frac{T}{2A_ot_f} = \frac{-1{,}956}{2 \times 61.1 \times 1.5} \times \frac{1}{12 \text{ in/ft}} \times 1.3 = 1.16 \text{ ksi}$$

$$f_{v\ tot} = 0.017 + 1.16 = 1.18 \text{ ksi}$$

Although the torques on the non-composite and composite box act in opposite directions, the resulting shear flows are conservatively added together in determining the total factored torsional shear stress since the dead-load torque on the composite box includes the effect of the future wearing surface. The torsional shear stress is well below the critical shear stress, $F_v = 0.75F_y/\sqrt{3} = 21.65$ ksi, given by Eq (10-1) in Article 10.4.2.2.

As specified in Article 10.4.2.4.2, the strength of non-composite longitudinally stiffened box flanges in compression is to be determined according to the provisions of Article 10.4.2.4.1, with the spacing between longitudinal stiffeners (or between the stiffener and the web), b_s, substituted for the flange width, b_f, and using the buckling coefficients, k and k_s, defined by Equations (10-10) and (10-9), respectively. Determine which equation to use to compute the critical stress. Start with Equation (10-4).

$$F_{cr} = F_y\Delta \qquad \text{Eq (10-4)}$$

when $\sqrt{F_y}\dfrac{b_f}{t_f} \le R_1$

where: F_y = specified minimum yield stress of the flange (ksi)

<u>Girder Stress Check Section 5-5 G2 Node 36</u>
<u>Strength - Bottom Flange (continued)</u>

b_f = b_s = distance between the longitudinal stiffener and the web (in)
t_f = flange thickness (in)

$$b_s = \frac{81}{2} = 40.5 \text{ in.}$$

$$t_f = 1.5 \text{ in.}$$

$$\sqrt{F_y}\frac{b_s}{t_f} = \sqrt{50}\frac{40.5}{1.5} = 191$$

$$\Delta = \sqrt{1 - 3\left(\frac{f_v}{F_y}\right)^2} \qquad \text{Eq (10-3)}$$

where: F_y = 50 ksi

$$\Delta = \sqrt{1-3\left(\frac{1.18}{50}\right)^2} \fallingdotseq 1.0$$

R_1 shall be taken as:

$$R_1 = \frac{97\sqrt{k}}{\sqrt{\frac{1}{2}\left[\Delta + \sqrt{\Delta^2 + 4\left(\frac{f_v}{F_y}\right)^2\left(\frac{k}{k_s}\right)^2}\right]}} \qquad \text{Eq (10-5)}$$

where: k = plate buckling coefficient
k_s = shear buckling coefficient

Try: k = 4.0 and k_s=5.34

Since the denominator of Eq (10-5) is approximately 1.0 in this case, $R_1 = 97\sqrt{k} = 97 \times \sqrt{4.0} = 194$. 191 < 194, therefore, use Equation (10-4).

$$F_{cr} = 50 \times 1.0 = 50 \text{ ksi}$$

A very rigid longitudinal flange stiffener is required to provide k=4.0 to ensure that a node forms at the stiffener. Since a critical flange stress less than 50 ksi would be satisfactory for this case, try k = 2.0 instead of 4.0, which will result in a lower critical flange buckling

<u>Girder Stress Check Section 5-5 G2 Node 36</u>
<u>Strength - Bottom Flange (continued)</u>

stress, but which will also result in a significantly smaller longitudinal flange stiffener. Since the denominator of Eq (10-5) is again approximately equal to 1.0 with k taken equal to 2.0:

$$R_1 = 97 \times \sqrt{2.0} = 137 < 191$$

Therefore, compute R_2.

$$R_2 = \frac{210\sqrt{k}}{\sqrt{\frac{1}{1.2}\left[\Delta - 0.4 + \sqrt{(\Delta - 0.4)^2 + 4\left(\frac{f_v}{F_y}\right)^2\left(\frac{k}{k_s}\right)^2}\right]}} \qquad \text{Eq (10-7)}$$

$$R_2 = \frac{210\sqrt{2.0}}{\sqrt{\frac{1}{1.2}\left[1.0 - 0.4 + \sqrt{(1.0 - 0.4)^2 + 4\left(\frac{1.18}{50}\right)^2\left(\frac{2.0}{5.34}\right)^2}\right]}}$$

$$R_2 = \frac{297}{\sqrt{0.833\left[0.6 + \sqrt{(1.0 - 0.4)^2 + 4\left(\frac{1.18}{50}\right)^2\left(\frac{2.0}{5.34}\right)^2}\right]}} = 297$$

Check $R_1 < \frac{b_s}{t_f}\sqrt{F_y} \le R_2$. 137 < 191 < 297, therefore, use Equation (10-6) to compute the critical flange stress.

$$F_{cr} = F_y\left[\Delta - 0.4\left(1 - \sin\left(\frac{\pi}{2}\left[\frac{R_2 - \frac{b_s}{t_f}\sqrt{F_y}}{R_2 - R_1}\right]\right)\right)\right] \qquad \text{Eq (10-6)}$$

$$F_{cr} = 50\left[1.0 - 0.4\left(1 - \sin\left(\frac{\pi}{2}\left[\frac{297 - \frac{40.5}{1.5}\sqrt{50}}{297 - 137}\right]\right)\right)\right] = 47.26 \text{ ksi}$$

$$\frac{|-46.47|}{47.26} = 0.98 < 1.00 \text{ OK}$$

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

The actual value of k_s will be determined in the next section on the design of the longitudinal flange stiffener. The critical stress will then be checked using the actual value of k_s to determine if there is a significant change in the stress.

The bottom flange at interior supports acting in combination with the internal diaphragm is subject to bending in two directions plus the torsional and diaphragm shear (ignoring through-thickness bending of the plate under its own self-weight). Therefore, Article 10.4.2.1 also requires that the principal stress in non-composite box flanges at supports due to vertical bending in the box girder in combination with the diaphragm plus torsional shear not exceed the critical stress given by Eq (10-2). Eq (10-2) is derived from the von Mises-Hencky yield criterion for combined stress. The equation in this form is valid for the case of bending in one direction in combination with shear. For a box supported on two bearings (the case in this example), bottom-flange bending stresses due to vertical bending of the diaphragm over the bearing sole plates are relatively small and will be neglected for simplicity in this example.

From previous calculations, the total factored St. Venant torsional shear stress in the bottom flange is equal to 1.18 ksi.

To estimate the shear stress in the bottom flange due to the diaphragm shear, assume a 1"x12" top flange for the diaphragm. As specified in Article 10.4.2.1, assume that 18 times the thickness of the bottom (box) flange (18x1.5=27 in) is effective with the diaphragm. The diaphragm is assumed to be 78 inches deep and 1 in thick. From separate calculations, the moment of inertia of the effective section is 112,375 in^4 and the neutral axis is located 31.05 in above the mid-thickness of the bottom flange. Subsequent calculations on page 87 indicate that the total factored vertical component of the diaphragm shear is 802.5 kips. The maximum shear stress in the effective bottom flange due to the diaphragm shear is therefore approximated as:

$$f_v = \frac{VQ}{It_f} = \frac{802.5(27/2)(1.5)(31.05)}{112{,}375(1.5)} = 2.99 \text{ ksi}$$

$$f_{v\ tot} = 1.18 + 2.99 = 4.17 \text{ ksi}$$

The factored vertical bending stress in the bottom flange, f_b, was computed earlier to be -46.47 ksi. The maximum principal stress is therefore computed to be

$$\sigma = \frac{f_b}{2} - \frac{1}{2}\sqrt{(f_b)^2 + 4f_v^2} = \frac{-46.47}{2} - \frac{1}{2}\sqrt{(-46.47)^2 + 4(4.17)^2} = -46.84 \text{ ksi}$$

$$F_{cr} = F_y\Delta \qquad \text{Eq (10-2)}$$

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

where:

$$\Delta = \sqrt{1 - 3\left(\frac{f_v}{F_y}\right)^2} \qquad \text{Eq (10-3)}$$

For this check, f_v in Eq (10-2) is taken as the total shear stress in the flange.

$$\Delta = \sqrt{1 - 3\left(\frac{4.17}{50}\right)^2} = 0.99$$

$$F_{cr} = 50(0.99) = 49.50 \text{ ksi}$$

$$\frac{|-46.84|}{49.50} = 0.95 < 1.0 \text{ OK}$$

For a box supported on a single bearing, the effect of bending in the plane of the diaphragm is likely to be more significant and should be considered. The effective section specified in Article 10.4.2.1 may be used to compute the flange bending stress about the tangential z-axis due to bending of the internal diaphragm over the sole plate. In this case, the resulting minimum and maximum principal stresses in the flange should be input into the more general form of the von Mises-Hencky yield criterion given as follows:

$$\sigma_1^2 - \sigma_1\sigma_2 + \sigma_2^2 = F_y^2$$

where:

σ_1, σ_2 are the maximum and minimum principal stresses

$$\sigma_1, \sigma_2 = \frac{1}{2}[(f_b)_x + (f_b)_z] \pm \sqrt{\left[\frac{(f_b)_x - (f_b)_z}{2}\right]^2 + f_v^2}$$

(Ref: Ugural, A.C. and Fenster, S.K. (1975). Advanced Strength and Applied Elasticity, Elsevier North Holland Publishing Co., Inc., New York, NY, pp. 105-107)

Although not illustrated here, bend buckling of the web must also be checked at the strength limit state according to the provisions of Article 6. The critical compressive flange stresses, computed above, should not exceed the critical compressive web stress (adjusted for the thickness of the flange).

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

Compute the required moment of inertia of the longitudinal bottom flange stiffener, I_s', about an axis parallel to the flange at the base of the stiffener.

Assume a plate buckling coefficient, k = 2.0.

$$I_s' = \phi t_f^3 b_s \qquad \text{Eq (10-10)}$$

where: $\phi = 0.125k^3$ for n=1

$$\phi = 0.125 \times 2^3 = 1.0$$
$$I_s' = 1 \times 1.5^3 \times 40.5 = 136.7 \text{ in}^4$$

Try: WT8X28.5 with stem welded to the flange. From the AISC Manual: I = 48.7 in^4; A = 8.38 in^2; NA = 6.275 in. from the tip of the stem.

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

$$I_s = 48.7 + 8.38(6.275)^2 = 378.7 \text{ in}^4 > 136.7 \text{ in}^4\text{, say OK}$$

$$k_s = \frac{5.34 + 2.84\sqrt[3]{\dfrac{I_s}{b_s t_f^3}}}{(n+1)^2} \le 5.34 \qquad \text{Eq (10-9)}$$

where:
- n = number of equally spaced longitudinal flange stiffeners
- I_s = actual moment of inertia of one longitudinal flange stiffener about an axis parallel to the flange at the base of the stiffener (in^4)
- t_f = thickness of flange plate (in)
- b_s = distance between longitudinal stiffeners or web and adjacent longitudinal stiffener (in)

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

The use of k_s = 2.33 in the equations for R_1 and R_2 on the preceding pages does not affect the calculated critical bottom flange stress.

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

Article 10.4.2.4.2 requires that transverse stiffening of the flange be provided at the point of maximum compressive flexural stress in the flange. According to Article 10.2.2.3, transverse top and bottom bracing members (i.e. top and bottom struts of internal cross frames) are required to ensure that the cross section shape is retained. Whenever longitudinal flange stiffeners are present, the bottom transverse members are to be attached to the longitudinal stiffener(s) by bolting. At other locations, the bottom transverse member is to be attached directly to the box flange. The cross sectional area and stiffness of the top and bottom transverse bracing members is not to be less than the area and stiffness of the diagonal members. In addition, at locations where a longitudinal flange stiffener is present, the moment of inertia of the bottom transverse bracing member should equal or exceed the moment of inertia of the longitudinal stiffener taken about the base of the stiffener. At the pier section (the point of maximum compressive flexural stress in a box flange in most cases), the bottom transverse bracing member, when properly attached to the longitudinal flange stiffener, can be assumed to provide the required transverse stiffening of the box flange. Use a W10x68 ($I=394\ in^4$) for the bottom transverse bracing member.

The longitudinal flange stiffener should be attached to the internal diaphragm with a pair of clip angles as shown in Figure E2 (page 90).

<u>Girder Stress Check Section 5-5 G2 Node 36</u>
<u>Overload - Web</u>

The live load for overload is multiple lanes of HS20 in this example. According to the provisions of **AASHTO Article 10.57**, the dead load factor is 1.0 and the live load factor is 5/3 for overload.

Check the web for bend-buckling at overload at this section according to the provisions of Article 10.5, which refers to the provisions of Article 6. Use the moments from Table D1. Use the section properties from Table D5. The composite section is assumed uncracked at overload according to the provisions of Article 10.5. Compute the overload vertical bending stress in the top and bottom of the web.

$$f_{top\ web} = \left[\frac{-15{,}426 \times 40.69}{438{,}966} + \frac{-4{,}473 \times 29.06}{633{,}467} + \frac{-6{,}853(5/3) \times 17.00}{836{,}080}\right] \times 12 = 22.41 \text{ ksi}$$

$$f_{bot\ web} = \left[\frac{-15{,}426 \times 37.31}{438{,}966} + \frac{-4{,}473 \times 48.94}{633{,}467} + \frac{-6{,}853(5/3) \times 61.00}{836{,}080}\right] \times 12 = -29.88 \text{ ksi}$$

Locate the portion of the web in compression, D_c, from the factored stresses in the top and bottom of the web.

$$D_c = 78 \times \left(\frac{|-29.88|}{|-29.88| + 22.41}\right) = 44.57 \text{ in}; \quad \frac{44.57}{\cos 14.3°} = 46.00 \text{ in}$$

Compute the critical stress according to the provisions of Article 6.3.1.

$$F_{cr} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} \le F_y \qquad \text{Eq (6-8)}$$

where: $k = 9\left(\frac{D}{D_c}\right)^2 \ge 7.2$

$$k = 9 \times \left(\frac{80.5}{46.00}\right)^2 = 27.56$$

Girder Stress Check Section 5-5 G2 Node 36
Overload - Web (continued)

Article 10.5 states that the compressive vertical bending stress at overload in non-composite box flanges must not exceed the lesser of $0.95F_y$ (for composite sections) or $0.80F_y$ (for non-composite sections) and F_{cr} defined by Equations (10-4), (10-6) or (10-8), as applicable. This limitation is extended to the web compressive stress for this case (an adjustment for the thickness of the flange can be made, but is not made here). Separate calculations (similar to those shown earlier to compute F_{cr} for the bottom flange at the strength limit state) indicate that F_{cr} for the flange at overload is also equal to 47.26 ksi, which is less than $0.95F_y$ = 47.5 ksi.

$$F_{cr} = \frac{0.9 \times 29{,}000 \times 27.56}{\left(\frac{80.5}{0.5625}\right)^2} = 35.12 \text{ ksi} < F_{cr} = 47.26 \text{ ksi}$$

$$\frac{|-29.88|}{35.12} = 0.85 < 1.00 \text{ OK}$$

The vertical bending stress in the top flange at overload at this section is limited to $0.95F_y$ since the section is composite and the flange is continuously braced.

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

Try a 1-inch thick A36 diaphragm plate.

Compute the maximum factored vertical shear in the diaphragm.

Load	Shear (kips)	Source
Steel	47	3D Finite Element Analysis
Deck	185	(in critical web)
SupImp	102	
Total DL	334	
Live Load	170	

$$V = 1.3(47 + 185 + 102) + 1.3(5/3)(170) = 802.5 \text{ kips}$$

Compute the shear capacity according to **AASHTO Equation (10-113)**. Separate calculations indicate that C=1.0.

$$V_u = CV_p$$ **AASHTO Eq (10-113)**

$$V_p = 0.58F_yDt_w = 0.58(36)(78)(1.0) = 1{,}629 \text{ kips}$$

$$V_u = 1.0(1{,}629) = 1{,}629 \text{ kips}$$

$$\frac{802.5}{1{,}629} = 0.49 < 1.0 \text{ OK}$$

The internal diaphragm is subject to vertical bending over the bearing sole plates in addition to shear. Therefore, Article 10.2.2.2 requires that the principal stresses in support diaphragms not exceed the critical stress given by Equation (10-2), which is a yield criterion for combined stress.

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

The sum of the total Steel plus Deck shears is equal to 47+185 = 232 kips. Referring to the calculations on page 77, the shear flow in the non-composite box is computed as:

$$SF = \frac{T}{2A_o} = \frac{26}{2(56.0)(12)} = 0.0193 \text{ kips/in}$$

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

$$V_T = 0.0193(80.5) = 15.54 \text{ kips}$$

The vertical component of V_T is computed as:

$$(V_T)_v = 15.54\left(\frac{78}{80.5}\right) = 15.06 \text{ kips}$$

$$V_b = 232 - 15.06 = 216.9 \text{ kips}$$

The sum of the total Superimposed Dead Load plus Live Load shears is equal to 102+5/3(170)=385 kips. Referring to the calculations on page 78, the shear flow in the composite box is computed as:

$$SF = \frac{T}{2A_o} = \frac{|-1{,}956|}{2(61.1)(12)} = 1.33 \text{ kips/in}$$

$$V_T = 1.33(80.5) = 107.1 \text{ kips}$$

The vertical component of V_T is computed as:

$$(V_T)_v = 107.1\left(\frac{78}{80.5}\right) = 103.8 \text{ kips}$$

$$V_b = 385 - 103.8 = 281.2 \text{ kips}$$

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

$$(f_v)_T = 1.3(0.0193/1.0 + 1.33/1.0) = 1.75 \text{ ksi}$$

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

$$(f_v)_b = \frac{1.3(216.9 + 281.2)}{78(1.0)} = 8.3 \text{ ksi}$$

As mentioned previously, for a box supported on two bearings, the bending stresses in the plane of the diaphragm due to vertical bending of the diaphragm over the bearing sole plates are relatively small and will be neglected in this example for simplicity. For a box supported on a single bearing, the effect of the bending stresses in the plane of the diaphragm are likely to be more significant and should be considered. As specified in Article 10.4.2.1, a width of the bottom (box) flange equal to 18 times its thickness may be considered effective with the diaphragm in resisting bending.

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

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

$$\sigma = f_v = (f_v)_T + (f_v)_b = 1.75 + 8.3 = 10.05 \text{ ksi}$$

$$F_{cr} = F_y \Delta \qquad \text{Eq (10-2)}$$

where:

$$\Delta = \sqrt{1 - 3\left(\frac{f_v}{F_y}\right)^2} \qquad \text{Eq (10-3)}$$

For this check, f_v in Eq (10-3) is taken as the total shear stress (bending plus torsional shear stress) in the diaphragm.

$$\Delta = \sqrt{1 - 3\left(\frac{10.05}{36}\right)^2} = 0.875$$

$$F_{cr} = 36(0.875) = 31.5 \text{ ksi}$$

$$\frac{10.05}{31.5} = 0.32 < 1.0 \text{ OK}$$

A conservative design is chosen since bending in the plane of the diaphragm is ignored. Further more detailed investigation of the state of stress in the diaphragm may allow for the use of a thinner plate.

For the two-bearing arrangement selected in this example, the section through the access hole is not critical. However, the designer should ensure that sufficient section is provided around the access hole to carry the torsional shear flow without reinforcement of the hole. For a box supported on a single bearing, the section through the access hole is critical and additional stiffening and/or reinforcement around the hole may be necessary.

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

Compute the factored reactions.

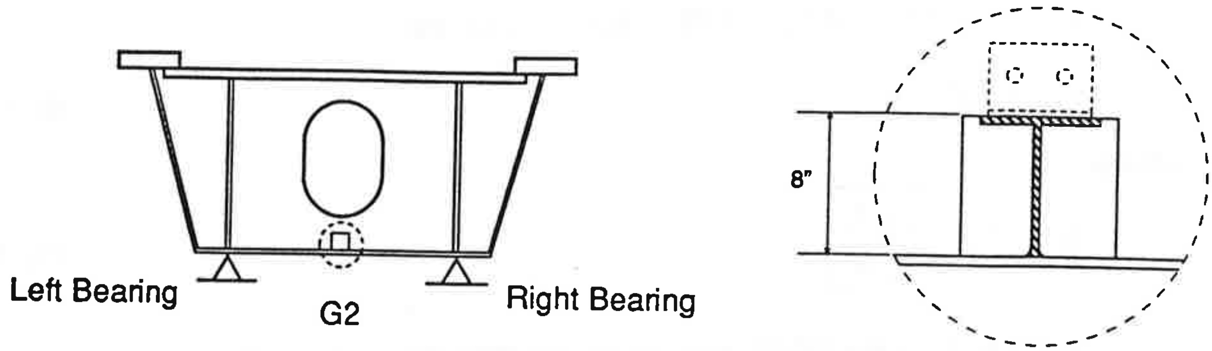

Figure E2 Internal Diaphragm and Bearing Stiffeners at Pier of Girder 2
Looking Upstation

Load	Reaction Location Left	Right	Source
Steel	79	93	3D Finite Element Analysis
Deck	238	370	(Not tabulated)
SupImp	198	26	
Total DL	515 k	489 k	
Live Load	252 k	288 k	
	-78 k	-17 k	uplift

$$R_{left} = 1.3\left[515 + \frac{5}{3}(252)\right] = 1{,}216 \text{ kips}$$

$$R_{right} = 1.3\left[489 + \frac{5}{3}(288)\right] = 1{,}260 \text{ kips (controls)}$$

Ignore uplift.

Assume that the bearings are fixed at the piers. Thus, there will be no expansion causing eccentric loading on the bearing stiffeners. Design the bearing stiffeners at this location according to the provisions of Article 6.7.

Use bars with F_y=50 ksi. Compute the maximum permissible width-to-thickness ratio of the stiffener plates according to Eq (6-13).

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

$$\frac{b_s}{t_s} \le 0.48\sqrt{\frac{E}{F_y}} = 0.48\sqrt{\frac{29{,}000}{50}} = 11.6$$

Compute the effective area of the diaphragm to which the stiffeners are attached (t_w=1.0 in) according to the provisions of Article 6.7.

$$A_d = 18t_w^2 = 18 \times 1.0^2 = 18.0 \text{ in}^2$$

Try 2-Bars 11" x 1"; Bearing area = 2(11-1.0)(1.0) = 20.0 in^2 (Assume 1 in. for stiffener clip). Bearing strength of milled stiffeners=1.35F_y = 67.5 ksi.

$$\frac{b_s}{t_s} = \frac{11.0}{1.0} = 11.0 < 11.6 \text{ OK}$$

$$\frac{1{,}260}{67.5} = 18.67 \text{ in}^2 < 20.0 \text{ in}^2 \text{ OK}$$

$$A = 18.0 + 20.0 = 38.0 \text{ in}^2$$

$$I = \frac{23.0^3 \times 1.0}{12} = 1{,}014 \text{ in}^4;\ r = \sqrt{\frac{I}{A}} = \sqrt{\frac{1{,}014}{38.0}}$$

r = 5.17 in.; K = 0.75 (Article 6.7); L = 78 in

$$\frac{KL}{r} = \frac{0.75 \times 78}{5.17} = 11.3 < \sqrt{\frac{2\pi^2 E}{F_y}} = 107.0$$

From **AASHTO Article 10.54.1,**

$$F_{cr} = F_y\left[1 - \frac{F_y}{4\pi^2 E}\left(\frac{KL}{r}\right)^2\right] = 50\left[1 - \frac{50}{4\pi^2 E}(11.3)^2\right] = 49.72 \text{ ksi}$$

$$\frac{1{,}260}{38.0} = 33.16 \text{ ksi};\ \frac{33.16}{49.72} = 0.67 < 1.00 \text{ OK}$$

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

Design the top (tub) flange single diagonal bracing member in Span 1 of Girder 2 in the first bay adjacent to the abutment (Element 1 in Table D4). Tub flange bracing is designed to satisfy the constructibility limit state only (Article 10.2.4). The bracing is designed according to the provisions of Article 9.3.1.

Load	Force	Source
Steel	-13	Table D4
Cast #1	-100	(from 3D finite element analysis)
	-113 kips	

Load Factor = 1.4 (Article 3.3); Design load = -113 x 1.4 = -158 kips

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

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

Try a structural tee (WT) section with the stem down with the flange of the tee bolted to the bottom of the tub flanges, which is the preferable method of connection according to Article 10.2.4. Assume that a timber will brace the member at mid-length in the vertical plane during construction. Therefore, the unbraced length with respect to the x-axis equals 222/2 = 111 in. The unbraced length with respect to the y-axis = 222 in.

Try: WT 9 x 38.
From AISC Manual: A = 11.2 in^2; y = 1.80 in.; S_x = 9.83 in^3; r_x = 2.54 in.; r_y = 2.61 in.

Check buckling about the y-axis.
Compute the effective length according to Article 9.3.1. K = 0.9

$$\frac{KL_c}{r_y} = \frac{0.9 \text{ x } 222}{2.61} = 76.55$$

Using **Equation (10-151)** from **AASHTO Article 10.54.1**, compute the critical stress. There is no eccentricity with respect to the y-axis.

$$F_{cr} = F_y\left[1 - \frac{F_y}{4\pi^2 E}\left(\frac{KL_c}{r_y}\right)^2\right]$$ **AASHTO Eq (10-151)**

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

$$F_{cr} = 50\left[1 - \frac{50}{4\pi^2(29{,}000)}(76.55)^2\right] = 37.20 \text{ ksi}$$

$$P_u = 0.85 A_s F_{cr}$$ **AASHTO Eq (10-150)**

$$P_u = 0.85(11.2)(37.20) = 354 \text{ kips}$$

$$\frac{|-158|}{354} = 0.45 < 1.0 \text{ OK}$$

Check buckling about the x-axis.

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

Compute the effective length according to the provisions of Article 9.3.1. Use half of the unbraced length of the member since a timber brace is used at mid-length.

$$M_{ecc} = 158 \text{ k} \times 1.80 \text{ in.} = 284 \text{ k-in}$$

Use the provisions of **AASHTO Equation (10-155)** to check the capacity of the member under a combined moment and axial force.

$$\frac{KL_c}{r_x} = \frac{0.9(111)}{2.54} = 39.33$$

$$F_{cr} = 50\left[1 - \frac{50}{4\pi^2(29{,}000)}(39.33)^2\right] = 46.62 \text{ ksi}$$ **AASHTO Eq (10-151)**

$$\frac{P}{0.85 A_s F_{cr}} + \frac{MC}{M_u\left(1 - \frac{P}{A_s F_e}\right)} \le 1.0$$ **AASHTO Eq (10-155)**

$M_{ecc} = 284$ k-in; $M_u = 9.83 \times 50$ ksi $= 491.5$ k-in

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

C is conservatively taken equal to 1.0.

$$F_e = \frac{E\pi^2}{\left(\frac{KL_c}{r_x}\right)^2} = \frac{29{,}000 \text{ x } \pi^2}{(39.33)^2} = 185 \text{ ksi}$$

AASHTO Eq (10-157)

$$\frac{158}{0.85 \text{ x } 11.2 \text{ x } 46.62} + \frac{284}{491.5\left(1 - \frac{158}{11.2 \text{ x } 185}\right)} = 0.36 + 0.63 = 0.99$$

0.99 < 1.0 OK

<u>Girder Stress Check Section 5-5 G2 Node 36</u>
<u>Transverse Bending Stress</u>

Article 10.3.1 requires that the transverse bending stresses in webs and flanges be investigated using a rational method. Article 10.2.2.3 limits the transverse bending stresses at the strength limit state to 20 ksi. Article 10.6.1 requires that the through-thickness (transverse) bending stress range due to cross section distortion at flange-to-web fillet welds (at the corner of the box) and at the termination of fillet welds connecting transverse elements be checked for fatigue.

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

The "Design Guide to Box Girder Bridges," Bethlehem Steel Corporation, 1981, presents a method developed by Wright and Abdel-Samed (1968) to estimate transverse bending stresses using the Beam on Elastic Foundation Analogy (BEF). Five pertinent pages of the Guide are included on pages 101 to 105. In this method, the deflection of the BEF is analogous to the transverse bending stress.

The fatigue loading produces a positive torque of 254 k-ft and a negative torque of -232 k-ft at the pier, Section 5-5 Node 36, as given in Table D3. The total range of torque is 486 k-ft. Since this range is produced by placing the truck in two different transverse locations, 75 percent of the range is used according to Article 3.5.7.2.

486 x 0.75 = 365 k-ft = 4,374 k-in

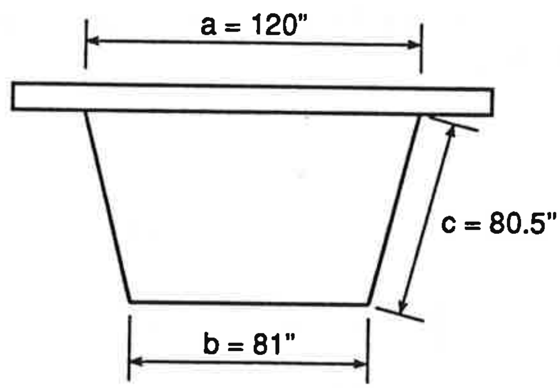

Figure E3 Composite Box Cross Section, G2

I_{comp} = 836,080 in^4 (from Table D5)

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

Minimum transverse stiffener spacing = 62 in. (Calculations not shown)
Cross frame spacing = 16.3 feet = 196 in.
t_c = 0.5625 in.; t_b = 1.50 in.; t_a = 9.5 in.
E_c = 3,834 ksi; E_s = 29,000 ksi

Transverse stiffener - bar $5\frac{1}{2}'' \times \frac{1}{2}''$

Poisson's ratio for concrete, $\mu_c = 0.16$; Poisson's ratio for steel, $\mu_s = 0.30$ (Article 1.2.3)

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

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

$$D_a = \frac{E\,t_a^3}{12(1-\mu_c^2)} = \frac{3{,}834(9.5)^3}{12(1-0.16^2)} = \frac{3.287 \times 10^6}{11.69} = 281{,}180 \text{ k-in}^2\text{/in}$$

$$D_b = \frac{E\,t_b^3}{12(1-\mu_s^2)} = \frac{29{,}000(1.5)^3}{12(1-0.30^2)} = \frac{9.78 \times 10^4}{10.92} = 8{,}963 \text{ k-in}^2\text{/in}$$

D_c = flexural rigidity of web

Compute D_c considering the transverse stiffeners according to Bethlehem Guide Equation (A3d) since Article 10.3.1 permits transverse stiffeners to be considered effective in resisting transverse bending.

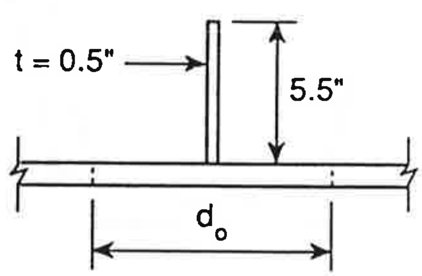

Figure E4 Effective Width of Web Plate, d_o, Acting with Transverse Stiffener

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

Compute d_o in Figure E4 using Equation (A4) from the Bethlehem Guide.

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

d = 62 in. spacing of transverse stiffeners.
c = 80.5 in.
h = c

$$d_o = \frac{(62)\tanh\left[5.6\left(\frac{62}{80.5}\right)\right]}{5.6\left(\frac{62}{80.5}\right)(1-0.3^2)} = 15.8 \text{ in.}$$

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

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

$$NA = \frac{2.75\,(0.5625 + 5.5/2) + 8.89\,(0.5625/2)}{11.64} = 1.0 \text{ in.}$$

$$I_s = \frac{1}{12}(5.5)^3 + 2.75\,(5.5/2 + 0.5625 - 1.0)^2 + 8.89\,(0.5625/2 - 1.0)^2 + \frac{1}{12}(0.5625^3 \times 15.8) = 33.40 \text{ in}^4$$

$$D_c = \frac{E_s I_s}{d} = \frac{29{,}000 \times 33.40}{62} = 15{,}623 \text{ k-in}^2\text{/in}$$ Bethlehem Guide Eq (A3d)

The stiffness of the transverse stiffener is assumed to be distributed evenly along the web. Compute the compatibility shear at the center of the box (bottom) flange according to Bethlehem Guide Equation (A2).

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

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

$$v = \frac{\frac{1}{15,623}[(2 \times 120 + 81)(120 \times 81 \times 80.5)] + \frac{1}{281,180}(81 \times 120^3)}{(120+81)\left[\frac{120^3}{281,180} + \frac{2 \times 80.5\,(120^2 + 120 \times 81 + 81^2)}{15,623} + \frac{81^3}{8,963}\right]}$$

$v = 0.22$

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

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

$$\delta_1 = \frac{120 \times 81}{24(120 + 81)}\left\{\frac{80.5}{15,623}\left[\frac{2 \times 120 \times 81}{120 + 81} - 0.22(2 \times 120 + 81)\right] + \frac{120^2}{281,180}\left(\frac{81}{120 + 81} - 0.22\right)\right\}$$

$$\delta_1 = 0.29 \text{ in}^2/\text{k}$$

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

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

where: I = moment of inertia of box

$$\beta = \sqrt[4]{\frac{1}{29,000 \times 836,080 \times 0.29}}$$

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

β = 0.00345
$\beta \ell$ = 0.00345 x 196 = 0.68

where: ℓ = distance between cross frames

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

$$f_t = C_t F_d \beta \frac{1}{2a}(m\ell \text{ or } T)$$ Bethlehem Guide Eq (A8)

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

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

$$S = \frac{I}{c} = \frac{33.40}{(5.5 + 0.5625 - 1.0)} = 6.60 \text{ in}^3$$

Compute S per inch.

$$S = \frac{6.60}{62} = 0.106 \text{ in}^3\text{/in.}$$

$$F_d = \frac{bv}{2S} = \frac{81 \times 0.22}{2 \times 0.106} = 84 \text{ in}^{-2}$$

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

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

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

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

<u>Girder Stress Check Section 5-5 G2 Node 36</u>
<u>Transverse Bending Stress (continued)</u>

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

$$F_d = \frac{120}{2 \times 0.106}\left(\frac{81}{120+81} - 0.22\right) = 104 \text{ in}^{-2}$$

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

Compute f_t using Bethlehem Guide Equation (A8).

$$f_t = C_t F_d \beta\left(\frac{1}{2a}\right)T \qquad \text{Bethlehem Guide Eq (A8)}$$

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

$$f_t = 0.03 \times 208 \times 0.00345 \times \left(\frac{1}{2 \times 120}\right) \times 4{,}374 = 0.39 \text{ ksi}$$

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

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

Designer's Guide to Steel Box Girder Bridges

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Notations

A	– enclosed area of box
A_b	– area of one diaphragm bracing member
A_{fl}	– bottom flange area
a	– width of box at top
Bi	– bimoment (k-in^2)
b	– width of box at base
C_b	– BEF factor for determining the diaphragm force
C_t	– BEF factor for determining the transverse distortional bending stress
C_w	– BEF factor for determining the normal distortional warping stress
c	– inclined height of box
D	– transverse flexural rigidity
d	– stiffener spacing
d_o	– effective width of web or flange plate acting with stiffener
E	– Young's modulus
E_b	– Young's modulus of diaphragm bracing
F_b	– force in diaphragm
F_d	– transverse bending stress in box plate due to an applied torque
F_y	– minimum specified yield stress
f_b	– natural frequency (Hz)
f'_c	– minimum specified compression strength of concrete
G	– shear modulus
g	– acceleration of gravity
h	– box depth
I	– moment of inertia of box section
I_s	– moment of inertia of stiffener bar and effective portion of web or flange
I_w	– warping constant (in^6)
K	– effective length factor
K_f	– constant
K_T	– torsional constant (in^4)
k	– buckling coefficient
L	– simple span length
L_b	– length of diaphragm bracing member
l	– diaphragm spacing
M	– in-plane moment
m	– uniformly applied torque
n	– modular ratio
NB	– number of box girders in the bridge
P	– load
Q	– statical moment (in^3)
q	– diaphragm brace stiffness/box stiffness/length (nondimensional)
R	– bridge radius
$R_{A,B,C,D}$	– reactions
r	– radius of gyration
S	– section modulus
S_w	– warping statical moment (in^4)
T	– concentrated torque
t	– plate thickness
V	– shear
v	– compatibility shear at center of box bottom
V_b	– bending shear
V_s	– shear connector force
V_{st}	– Saint-Venant's torsional shear
W	– bridge weight per length
W_L	– AASHTO wheel load factor
W_n	– normalized warping function (in^2)
$\tilde{W}, \mathcal{A} \cdot \tilde{\mathcal{S}}$	– see References 4 and 12
X	– 1/R
x	– distance from a diaphragm
y	– vertical distance to extreme fiber from neutral axis
α	– angle of skew
β	– BEF stiffness parameter (in^{-1})
γ	– distortional angle (radians)
γ'	– first derivative of distortion angle
γ''	– second derivative of distortion angle
Δ_b	– deflection due to flexure
δ_b	– deformation of bracing member due to applied torque (in^2/k)
δ_t	– box distortion per kip per inch of load without diaphragms (in^2/k)
μ	– Poisson's ratio
σ_b	– normal bending stress
σ_{dw}	– normal distortional warping stress
σ_t	– transverse bending stress
σ_{tw}	– normal torsional warping stress
τ	– Saint-Venant shearing stress
τ_b	– bending shear stress
τ_{dw}	– distortional warping shear stress
τ_{tw}	– torsional warping shear stress
ϕ	– angle of rotation
$\bar{\phi}$	– subtended angle between radial piers
ψ	– $L\sqrt{G K_T / E I_w}$

Appendix

Beam on Elastic Foundation Analogy for Determining Distortional Stresses in Box Girders

This presentation is based on work performed at the University of Illinois under the direction of R. N. Wright, and sponsored by the American Iron and Steel Institute. The example is also taken from this work, with minor modifications.

The deflection, δ_1, shown in Figure Alc is due to a torsional load shown in Figure Ala. Deflection, δ_1, is the reciprocal of the torsional stiffness of the box, and analogous to the reciprocal of the foundation modulus in the BEF problem. It is computed as follows:

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

where:

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

v = compatibility shear at center of bottom flange

$D_a = Et_a^3/12\,(1-\mu^2)$ **(A3a)**

$D_b = Et_b^3/12\,(1-\mu^2)$ **(A3b)**

$D_c = Et_c^3/12\,(1-\mu^2)$ **(A3c)**

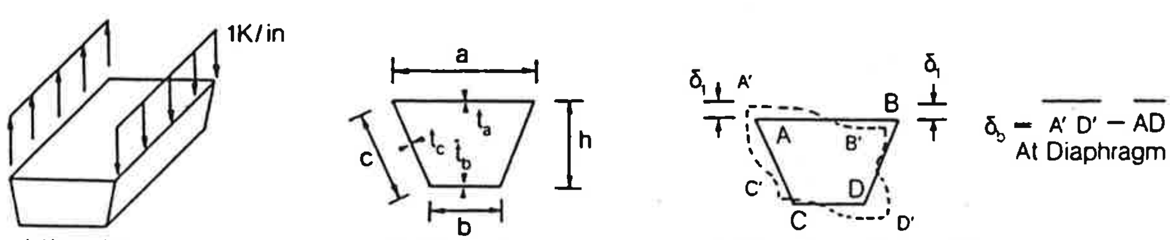
} transverse flexural rigidity of an unstiffened plate (k-in^2/in)

where: t_a, t_b, t_c = top flange, bottom flange, and web thickness in.

μ = Poisson's ratio

The term, v, is the compatibility shear at the center of the bottom flange when unit loads are applied at the top corners of the box section of unit length as shown in Figure Ala. The center of the bottom flange was chosen by Wright (30) because the transverse bending moment and thrust are zero at this point. Dimensions used in Equation 8 are shown in Figure Alb.

Figure A1/Box under uniform torsional loading



(a)Loading (b)Dimensions (c)Distortional Deformation

When transverse stiffeners are present on either flanges or webs, they should be considered in calculating transverse flexural rigidities. The rigidity of the stiffened plate is calculated as follows:

$$D = \frac{EI_s}{d} \quad \textbf{(A3d)}$$

where: I_s = moment of inertia of stiffened plate
d = stiffener spacing

The effective width of plate, d_o, acting with a stiffener can be determined as follows:

$$d_o = \frac{d \tanh (5.6\, d/h)}{\frac{5.6\, d}{h}(1 - \mu^2)} \quad \textbf{(A4)}$$

where: h = transverse length of element, "b" or "c"

Equation A4 is a semiempirical relationship which Wright et al found to give reliably accurate results (30).

The BEF stiffness parameter, β, in the analogy is calculated as follows:

$$\beta \approx \sqrt[4]{\frac{1}{EI\delta_l}} \quad \textbf{(A5)}$$

where: I = moment of inertia of the box section

Stiffness parameter, β, is a measure of the torsional stiffness of the beam, and is analogous to the beam-foundation parameter in the Beam on Elastic Foundation problem. The diaphragms in the box girder restrict box deformation, and are analogous to supports in the BEF. They are incorporated in the solution by the term "q", which is the dimensionless ratio of diaphragm stiffness to the box stiffness per unit. It is defined as follows:

$$q = \left[\frac{E_b A_b}{L_b\, l\, \delta_l}\right] \delta_b^2 \quad \textbf{(A6)}$$

where: E_b = Young's modulus of diaphragm material
A_b = cross-sectional area of one diaphragm bracing member
L_b = length of diaphragm brace

$$\delta_b = \frac{2\,(1 + a/b)}{\sqrt{1 + \left[\frac{a+b}{2h}\right]^2}}\,[\delta_l] \quad \textbf{(A7)}$$

where: δ_b = deformation of the bracing member (see Figure A1c)

Equation A6 tacitly assumes that cross bracing is effective in both compression and tension. If the bracing slenderness is large, the bracing is only effective in tension, and A_b in Equation A6 should be one-half the area of one brace.

The stresses derived from distortion of the box can be determined analogously by solving the BEF problem. Moment in the BEF is analogous to normal distortional stress, σ_{dw}, and deflection in the BEF is analogous to distortional transverse bending stress, σ_t. The reactions in the BEF are analogous to the forces in cross bracing, F_b. Solutions for these three components are presented in graphical form in Figures A2 through A10. These figures give a "C" value which is used in appropriate equations–A8, A10, A11. These graphs show relationships for uniform torque, m, or concentrated torque, T, at midpanel or diaphragms. The figures give the appropriate "C" values for a given box geometry, β, loading, diaphragm stiffness, q, and spacing, l. The designer is able to determine the distortion-related stresses, and estimate how diaphragm spacing and stiffness may be best modified if necessary.

Equation A8 gives transverse bending stresses at the top or bottom corners of the box section, depending on the determination of F_d in Equations A9a and A9b. The critical stress may be in either the web or flange. The AASHTO Specification limits the range of the transverse bending stresses to 20,000 psi. Therefore, the torsion in both directions often must be determined. The stress range is the sum of absolute values of stresses due to opposite torques.

$$\sigma_t = C_t\, F_d\, \beta \frac{1}{2a}\,(ml \text{ or } T) \quad \textbf{(A8)}$$

where: m = uniform torque per unit length
T = concentrated torque

$$F_d = \frac{bv}{2S} \text{ for bottom corner of box} \quad \textbf{(A9a)}$$

$$F_d = \frac{a}{2S}\left(\frac{b}{a+b} - v\right) \text{ for top corner of box} \quad \textbf{(A9b)}$$

where: S = section modulus of transverse member (see Figure A1c)

Figure A4/Uniform torque on continuous beam–distortional transverse bending stress at midpanel

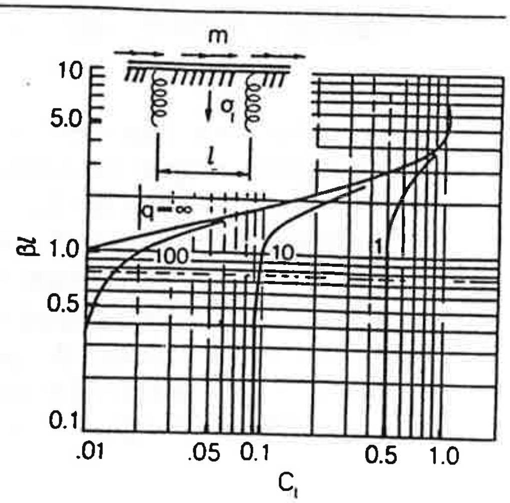


Figure A5/Uniform torque on continuous beam–diaphragm force

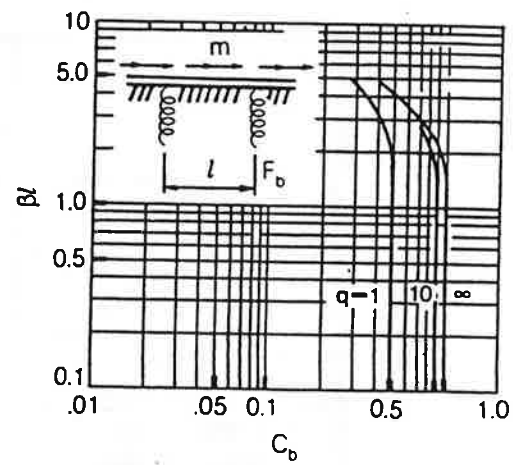
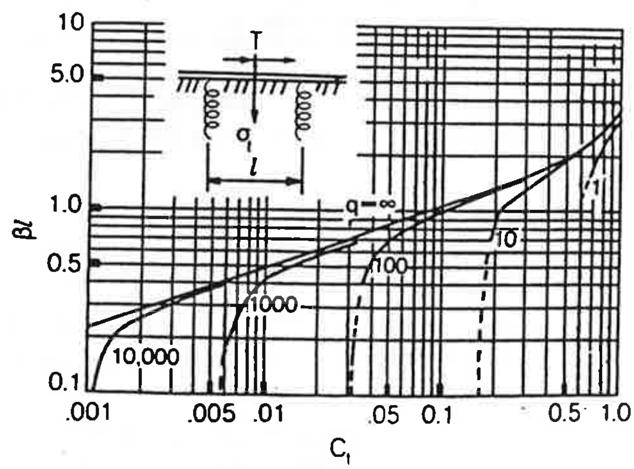


Figure A6/Concentrated torque at midpanel on continuous beam–distortional transverse bending stress at load



Girder Stress Check Section 5-5 G2 Node 36
Composite Bottom Flange Option

Assume that the bottom box flange in the negative moment regions will be redesigned using an unstiffened plate with 8 inches of composite concrete according to the provisions of Article 10.4.3. The girder moment due to the added weight of the flange concrete is considered. It is assumed that the concrete is placed after the steel has been erected and before the deck is cast. Thus, the bottom flange will be composite for the deck load. The superimposed dead load is added after the deck and the bottom flange concrete have hardened. Live load will be applied to the composite section. Only the rebars in the deck will be considered to act compositely with the steel section (with the composite box flange) at the strength limit state.

Table E1 Section Properties of G2 with 8 inches of 6,000 psi Concrete in Bottom Flange

Section	Top Flg 18 x 3 Bot Flg 81 x 1.25 (Node 36)		Top Flg 18 x 1.5 Bot Flg 81 x 0.75 (Node 32)	
	I (in^4)	NA (in)	I (in^4)	NA (in)
1. Noncomp	396,155	41.99	238,466	39.59
2. Comp flg 3n w/o deck rebars	438,765	38.26	275,140	34.69
3. Comp flg n w/o deck rebars	502,477	32.68	324,442	28.11
4. Comp flg 3n w/deck rebars	454,801	39.20	293,094	36.06
5. Comp flg n w/deck rebars	560,391	35.22	390,313	31.61

NA is distance from bottom of section to the neutral axis.
Effective deck rebar area for 3n equals 20.0/3 = 6.67 in^2

1. Non-composite
2. Steel with bottom flange concrete at 3n without deck rebars
3. Steel with bottom flange concrete at n without deck rebars
4. Steel with bottom flange concrete at 3n and deck rebars at 3n
5. Steel with bottom flange concrete at n and deck rebars at n

Properties of bottom flange concrete:

f'_c = 6,000 psi; E_c = 4,696 ksi; n = 6.2

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

The bottom flange concrete is assumed to be continuous through the interior support diaphragm as required in Article 10.2.2.2. Bottom transverse bracing members (i.e. the bottom struts of the interior cross frames) are assumed to be located above the concrete in this region.

Try a 1.25-inch thick unstiffened bottom flange plate.

Check the transverse bending stress in the bottom flange plate due to the self-weight of the plate and the wet concrete.

Compute the section modulus of a one-foot wide section of the bottom flange plate.

$$S = \frac{1}{6} b t_f^2 = \frac{1}{6} \text{ x } 12 \text{ x } 1.25^2 = 3.13 \text{ in}^3/\text{ft}$$

Compute the moment applied to the flange plate due to the weight of the steel and flange concrete. Assume a simple span between webs.

$$\text{Steel weight per foot of width, } w_s = \frac{1.25 \text{ x } 12'' \text{ x } 3.4}{12} = 4.3 \text{ pounds/inch/ft}$$

$$M_{steel} = \frac{1}{8} w_s \, l^2 = \frac{1}{8} \text{ x } 0.0043 \text{ x } 81^2 = 3.53 \text{ k-in/ft}$$

$$\text{Concrete weight per foot of width, } w_c = 1' \text{ x } 150 \text{ x } \frac{8}{12}\frac{1}{12} = 8.33 \text{ pounds/inch/ft}$$

$$M_{conc} = w_c \, l^2 = \frac{1}{8} \text{ x } 0.00833 \text{ x } 81^2 = 6.83 \text{ k-in/ft}$$

Compute the maximum transverse bending stress in the flange plate at the constructibility limit state. Load factor = 1.4 (Article 3.3).

$$f_{tran} = \frac{M}{S} = \frac{(3.53 + 6.83)}{3.13} \text{ x } 1.4 = 4.63 \text{ ksi} < 50 \text{ ksi OK}$$

Although not checked here, Article 10.4.2.1 also limits the maximum vertical deflection of the box flange due to self-weight and the applied permanent loads to 1/360 times the transverse span between webs.

The concrete is to be placed on the bottom flange of the field section over each pier.

The bottom flange concrete causes a longitudinal girder moment of -880 k-ft at Section 5-5 in G2 from the finite element analysis. The longitudinal girder moment due to steel = -3,154 k-ft from Table D1.

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

Compute the total moment applied to the steel section.

$$M = -3{,}154 + (-880) = -4{,}034 \text{ k-ft}$$

Check the vertical bending stress in the unstiffened plate at the constructibility limit state.

Compute the vertical bending stress in the bottom flange due to steel weight and concrete. Section properties are from Table E1.

As specified in Article 10.4.3.1, for loads applied prior to hardening of the concrete, composite box flanges are to be designed as non-composite box flanges according to the provisions of Article 10.4.2.

$$f_{bot\ flg} = \frac{-4{,}034 \times 41.99 \times 12 \times 1.4}{396{,}155} = -7.18 \text{ ksi}$$

The critical buckling stress for the non-composite bottom flange with no stiffening according to Article 10.4.2.4.1 equals 24.9 ksi. Calculations are not shown.

$$\frac{|-7.18|}{24.9} = 0.29 < 1.00 \text{ OK}$$

Compute the factored bottom flange vertical bending stress in the non-composite section at the strength limit state.

$$f_b = -7.18 \times \left(\frac{1.3}{1.4}\right) = -6.67 \text{ ksi}$$

Compute the factored bottom flange vertical bending stress in the composite section due to the deck weight.

$$M_{deck} = -12{,}272 \text{ k-ft (Table D1).}$$

As specified in Article 10.4.3.3, concrete creep is to be considered when checking the compressive steel stress. Concrete compressive stress are to be checked without considering creep. Therefore, use 3n section properties to check the steel stress and n section properties to check the concrete stress.

Compute the factored vertical bending stress in the extreme fiber of the steel flange. Use creep properties (3n) from Table E1.

$$f_{bot} = \frac{-12{,}272 \times 38.26}{438{,}765} \times 12 \times 1.3 = -16.69 \text{ ksi}$$

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

Compute the factored vertical bending stress in the extreme fiber of the flange concrete. Use no creep properties (n) from Table E1.

$$f_{bot} = \frac{-12{,}272 \times (32.68 - 1.25)}{502{,}477} \times 12 \times 1.3 \times \frac{1}{6.2} = -1.93 \text{ ksi}$$

Compute the factored vertical bending stress in the composite bottom flange due to the superimposed dead load after the deck has hardened.

Superimposed dead load moment = -4,473 k-ft from Table D1.

Compute the factored vertical bending stress in the extreme fiber of the flange. Use the appropriate creep properties (3n) from Table E1.

$$f_{bot} = \frac{-4{,}473 \times 39.20}{454{,}801} \times 12 \times 1.3 = -6.01 \text{ ksi}$$

Compute the factored vertical bending stress in the extreme fiber of the flange concrete. Use the appropriate no creep properties (n) from Table E1.

$$f_{bot} = \frac{-4{,}473 \times (35.22 - 1.25)}{560{,}391} \times 12 \times 1.3 \times \frac{1}{6.2} = -0.68 \text{ ksi}$$

Compute the factored vertical bending stress in the composite bottom flange due to the live load. Use the appropriate n section properties from Table E1.

Moment due to live load = -8,566 k-ft from Table D1

$$f_{bot} = \frac{-8{,}566(5/3) \times 35.22}{560{,}391} \times 12 \times 1.3 = -14.00 \text{ ksi}$$

Compute the factored vertical bending stress in the composite bottom flange concrete due to the live load. Use the appropriate n section properties from Table E1.

$$f_{bot} = \frac{-8{,}566(5/3) \times (35.22 - 1.25) \times 12 \times 1.3}{560{,}391} \times \frac{1}{6.2} = -2.18 \text{ ksi}$$

<u>Girder Stress Check Section 5 -5 G2 Node 36</u>
<u>Composite Bottom Flange Option (continued)</u>

Check the total factored vertical bending stress in the steel bottom flange at the strength limit state.

$$f_{tot} = -6.67 + (-16.69) + (-6.01) + (-14.00) = -43.37 \text{ ksi}$$

As specified in Article 10.4.3.3, the critical stress for the steel flange in compression at the strength limit state is given by Equation (10-4) as follows:

$$F_{cr} = F_y\Delta \qquad \text{Eq (10-4)}$$

$$\text{where: } \Delta = \sqrt{1 - 3\left(\frac{f_v}{F_y}\right)^2} \qquad \text{Eq (10-3)}$$

From Table D3, the torque due to the steel weight is -22 k-ft (the torque due to the wet bottom concrete is neglected since the load is symmetrical and the curvature effect is relatively small). Using calculations similar to those shown on page 77, the enclosed area of the non-composite box is computed to be $A_o = 55.9 \text{ ft}^2$.

$$f_v = \frac{T}{2A_ot_f} = \frac{|-22|}{2(55.9)(1.25)12} \times 1.3 = 0.017 \text{ ksi}$$

Subsequent calculations show that the factored torque on the composite section at the strength limit state is equal to -2,481 k-ft. To account for the possibility of concrete creep, it is conservatively assumed that all of this torque is resisted by the steel flange in this computation, as required by Article 10.4.3.4 at the strength limit state. The enclosed area of the composite box is computed to be $A_o = 61.0 \text{ ft}^2$.

$$f_v = \frac{T}{2A_ot_f} = \frac{|-2{,}481|}{2(61.0)(1.25)(12)} = 1.36 \text{ ksi}$$

$$(f_v)_{tot} = 0.017 + 1.36 = 1.38 \text{ ksi}$$

The total shear stress satisfies Eq (10-1) in Article 10.4.2.2 by inspection.

$$\Delta = \sqrt{1 - 3\left(\frac{1.38}{50}\right)^2} = 0.999$$

$$F_{cr} = 50(0.999) = 49.95 \text{ ksi}$$

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

$$\frac{|-43.37|}{49.95} = 0.87 < 1.00 \quad \text{OK}$$

Compute the factored vertical bending stress in the bottom of the concrete at the strength limit state assuming no creep.

$$F_{cr\ conc} = 0.85 \times 6 = 5.10 \text{ ksi (Article 10.4.3.3)}$$

$$f_{tot} = -1.93 + (-0.68) + (-2.18) = -4.79 \text{ ksi}$$

$$\frac{|-4.79|}{5.10} = 0.94 < 1.00 \text{ OK}$$

<u>Girder Stress Check Section 4-4 G2 Node 32</u>
<u>Composite Bottom Flange Option</u>

Try a 0.75 inch thick flange plate at this section with 8 inches of concrete.

Check the transverse bending stress in the bottom flange plate due to the self-weight of the plate and the wet concrete.

Compute the section modulus of a one-foot wide section of the bottom flange plate.

$$S = \frac{1}{6}bt_f^2 = \frac{1}{6} \times 12 \times 0.75^2 = 1.13 \text{ in}^3\text{/ft}$$

Compute the moment applied to the flange plate due to the weight of the steel and flange concrete. Assume a simple span between webs.

$$\text{Steel weight per foot of width, } w_s = \frac{0.75 \times 12'' \times 3.4}{12} = 2.6 \text{ pounds/inch/ft}$$

$$M_{steel} = \frac{1}{8}w_s\, l^2 = \frac{1}{8} \times 0.0026 \times 81^2 = 2.13 \text{ k-in/ft}$$

Compute the maximum transverse bending stress in the flange plate at the constructibility limit state. Load factor=1.4.

M_{conc} = 6.83 k-in/ft due to the weight of the concrete from page 107.

$$f_{tran} = \frac{M}{S} = \frac{(2.13 + 6.83)}{1.13 \text{ in}^3} \times 1.4 = 11.10 \text{ ksi} < 50 \text{ ksi OK}$$

Longitudinal girder moment due to additional concrete in the bottom flange = -358 k-ft from finite element analysis (not shown). Longitudinal girder moment due to steel = -1,896 k-ft (Table D1).

M = -1,896 + (-358) = -2,254 k-ft

Check the vertical bending stress in the bottom flange due to the total moment applied to the non-composite section at the constructibility limit state. Section properties are from Table E1.

$$f_{bot\ flg} = \frac{-2,254 \times 39.59 \times 12 \times 1.4}{238,466} = -6.29 \text{ ksi}$$

Determine the critical stress for the unstiffened box flange according to Article 10.4.2.4.1.

F_{cr} = 8.8 ksi (Calculations not shown)

<u>Girder Stress Check Section 4-4 G2 Node 32</u>
<u>Composite Bottom Flange Option (continued)</u>

$$\frac{|-6.29|}{8.8} = 0.71 < 1.00 \quad \text{OK}$$

Adjust the stress for the strength limit state load factor.

$$-6.29 \times \frac{1.3}{1.4} = -5.84 \text{ ksi}$$

Compute the factored vertical bending stress in the composite bottom flange due to the deck weight. Use 3n section properties from Table E1.

Moment due to deck = -7,599 k-ft from Table D1.

$$f_{bot} = \frac{-7{,}599 \times 34.69}{275{,}140} \times 12 \times 1.3 = -14.95 \text{ ksi}$$

Compute the factored vertical bending stress in the extreme fiber of the composite bottom concrete due to the deck weight. Use n section properties from Table E1.

$$f_{bot} = \frac{-7{,}599 \times (28.11 - 0.75)}{324{,}442} \times 12 \times 1.3 \times \frac{1}{6.2} = -1.61 \text{ ksi}$$

Compute the factored vertical bending stress in the composite bottom flange due to the superimposed dead load. Use the appropriate 3n section properties from Table E1.

Superimposed dead load moment = -2,610 k-ft from Table D1

$$f_{bot} = \frac{-2{,}610 \times 36.06}{293{,}094} \times 12 \times 1.3 = -5.01 \text{ ksi}$$

Compute the factored vertical bending stress in the composite bottom flange concrete due to superimposed dead load. Use the appropriate n section properties from Table E1.

$$f_{bot} = \frac{-2{,}610 \times (31.61 - 0.75)}{390{,}313} \times 12 \times 1.3 \times \frac{1}{6.2} = -0.52 \text{ ksi}$$

Compute the factored vertical bending stress in the composite bottom flange due to the live load. Use the appropriate n section properties from Table E1.

Girder Stress Check Section 4-4 G2 Node 32
Composite Bottom Flange Option (continued)

Moment due to live load = -5,612 k-ft from Table D1

$$f_{bot} = \frac{-5{,}612(5/3) \times 31.61}{390{,}313} \times 12 \times 1.3 = -11.82 \text{ ksi}$$

Compute the factored vertical bending stress in the composite bottom flange concrete due to live load. Use the appropriate n section properties from Table E1.

$$f_{bot} = \frac{-5{,}612(5/3) \times (31.61 - 0.75)}{390{,}313} \times 12 \times 1.3 \times \frac{1}{6.2} = -1.86 \text{ ksi}$$

Compute the total factored vertical bending stress in the steel bottom flange at the strength limit state.

$$f_{tot} = -5.84 + (-14.95) + (-5.01) + (-11.82) = -37.62 \text{ ksi}$$

The torque due to the steel weight is -10 k-ft (Table D3). The enclosed area of the non-composite box is computed to be $A_o = 55.2 \text{ ft}^2$.

$$f_v = \frac{T}{2A_o t_f} = \frac{|-10|}{2(55.2)(0.75)(12)} \times 1.3 = 0.013 \text{ ksi}$$

The factored torque on the composite section is computed to be

Loading	Torque (Table D3)	
Deck	63 x 1.3	82 k-ft
Superimposed DL	-273 x 1.3	-355 k-ft
Live Load	-688(5/3) x 1.3	-1,491 k-ft
		-1,764 k-ft

The enclosed area of the composite box is computed to be $A_o = 60.8 \text{ ft}^2$.

$$f_v = \frac{T}{2A_o t_f} = \frac{|-1{,}764|}{2(60.8)(0.75)(12)} = 1.61 \text{ ksi}$$

$$(f_v)_{tot} = 0.013 + 1.61 = 1.62 \text{ ksi}$$

which satisfies Eq (10-1) in Article 10.4.2.2 by inspection.

Girder Stress Check Section 4-4 G2 Node 32
Composite Bottom Flange Option (continued)

$$\Delta = \sqrt{1 - 3\left(\frac{1.62}{50}\right)^2} = 0.998$$

$$F_{cr} = 50(0.998) = 49.90 \text{ ksi}$$

$$\frac{|-37.62|}{49.90} = 0.75 < 1.00 \text{ OK}$$

Compute the total factored vertical bending stress in the bottom flange concrete at the strength limit state.

$$f_{tot} = -1.61 + (-0.52) + (-1.86) = -3.99 \text{ ksi}$$

$F_{cr\ conc}$ = 5.10 ksi from page 111

$$\frac{|-3.99|}{5.10} = 0.78 < 1.00 \text{ OK}$$

Girder Stress Check Section 5-5 G2 Node 36
Composite Bottom Flange Option Design of Shear Connectors - Strength

Check the ultimate strength of the shear connectors on the composite bottom flange according to the provisions of Article 10.4.3.5, which refer back to the provisions of Article 7.2.1. As specified in Article 10.4.3.5, the radial force due to curvature is ignored. The longitudinal force to be developed is given by Eq (7-4), with b_d taken as the full width of the bottom flange concrete.

$$\text{Required capacity} = 0.85 f'_c b_d t_d \qquad \text{Eq (7-4)}$$

$$P = 0.85 \times 6 \text{ ksi} \times 81 \text{ in.} \times 8 \text{ in.} = 3{,}305 \text{ kips}$$

Compute the capacity of one shear connector.

$$\frac{H}{d} = \frac{6}{0.875} = 6.86 > 4.0$$

$$S_u = 0.4 d^2 \sqrt{f'_c E_c} \le 60{,}000 A_{sc} \qquad \textbf{AASHTO Eq (10-67)}$$

(note: the upper limit of $60{,}000 A_{sc}$ in the above equation will be incorporated in a future Interim to the Standard Specifications and is included here.)

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

$$S_u = 0.4 \times 0.875^2 \sqrt{6 \times 4{,}696} = 51.4 \text{ k} > 60(0.6) = 36 \text{ kips}$$

$$\therefore S_u = 36 \text{ kips}$$

Compute the minimum number of shear connectors required on each side of the pier.

$$\text{No. of shear connectors reqd.} = \frac{P}{\phi_{sc} S_u} = \frac{3{,}305}{0.85(36)} = 108.0$$

Try six studs uniformly spaced across the flange (Figure E5) with 18 rows on each side of pier (for 108 shear connectors per flange on each side of the pier). The studs must be spaced transversely so that the steel plate slenderness limit of R_1 in Eq (10-4) is satisfied, where b_f is taken as the transverse spacing between the shear connectors (Article 10.4.3.5).

Check the computed force on critical studs at Node 36.

Compute the axial force in the bottom flange concrete due to the vertical moment. Compute the stresses in the top of the flange concrete due to the deck weight without

Girder Stress Check Section 5-5 G2 Node 36
Composite Bottom Flange Option Design of Shear Connectors - Strength (continued)

creep. Use the ratio of distances to the neutral axis.

$$f_{top} = -1.93 \left(\frac{32.68 - 9.25}{32.68 - 1.25} \right) = -1.44 \text{ ksi}$$

Compute the stress in the top of the flange concrete due to superimposed dead load and live load without creep.

$$f_{top} = [-0.68 + (-2.18)] \left(\frac{35.22 - 9.25}{35.22 - 1.25} \right) = -2.19 \text{ ksi}$$

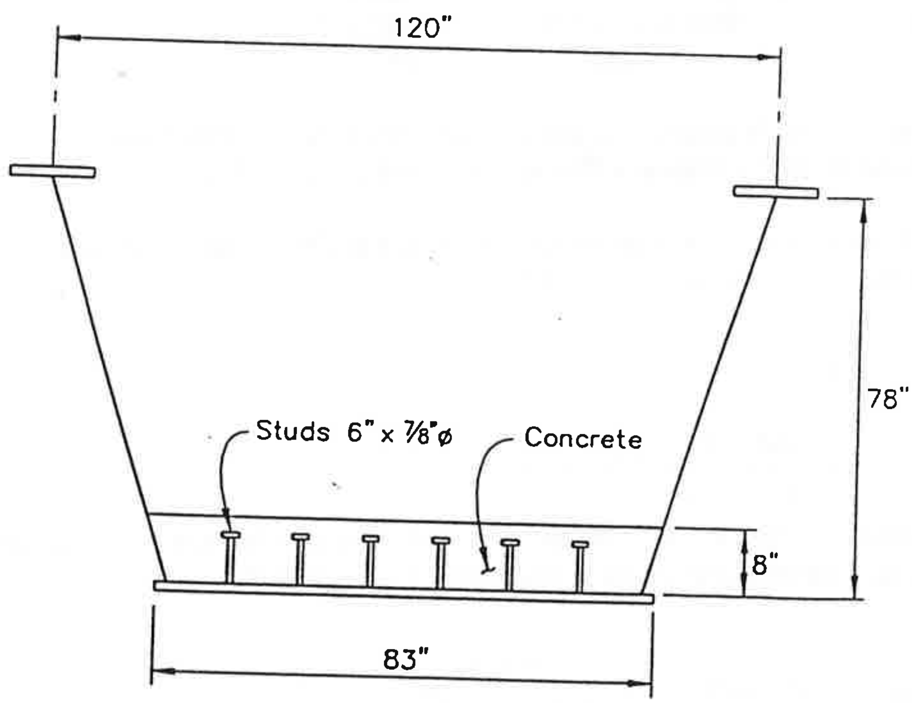


Figure E5 Shear Studs in Composite Bottom Flange

Girder Stress Check Section 5-5 G2 Node 36
Composite Bottom Flange Option Design of Shear Connectors - Strength (continued)

Compute the longitudinal force in the flange concrete due to vertical bending. Use the average bending stress in the concrete times the area of the concrete.

$$F = \left(\frac{1.93 + 1.44}{2} + \frac{2.86 + 2.19}{2} \right) \times 81 \times 8 = 2{,}728 \text{ kips}$$

Compute the longitudinal force per stud.

$$F_L = 2{,}728/108 = 25.26 \text{ kips/stud}$$

Compute the St. Venant torsional shear in the concrete. Assume that a single row of studs across the flange will resist the torsional shear in the flange concrete.

Loading	Torque (Table D3)	
Deck	48 x 1.3	62 k-ft
Superimposed DL	-346 x 1.3	-450 k-ft
Live load	-966(5/3) x 1.3	-2,093 k-ft
	Total	-2,481 k-ft

Assume all torsion is applied to the uncracked section without creep.
Effective concrete thickness = Thickness/n = 8 in./6.2 = 1.29 in.

Using calculations similar to those shown on page 78, the enclosed area of the composite box is computed to be $A_o = 61.0 \text{ ft}^2$.

$$V = \frac{T}{2A_o} b_f$$

$$V = \frac{|-2{,}481|}{2 \times 61.0} \frac{81}{12} = 137 \text{ kips}$$

Compute the portion of the torsional shear resisted by the concrete by taking the ratio of the effective concrete thickness to the total thickness of the steel flange plus the effective concrete.

$$V_{conc} = 137 \text{ kips} \times \frac{1.29 \text{ in.}}{(1.25 \text{ in.} + 1.29 \text{ in.})} = 70 \text{ kips}$$

As specified in Article 10.4.3.4, adequate orthogonal reinforcement must be provided to resist this computed torsional shear in the concrete.

Compute the transverse shear per stud assuming six shear connectors per row across the

<u>Girder Stress Check Section 5-5 G2 Node 36</u>
<u>Composite Bottom Flange Option Design of Shear Connectors - Strength (continued)</u>

flange.

$$V_T = \frac{70\ \text{k}}{6\ \text{studs}} = 11.67\ \text{kips/stud}$$

Compute the vector sum of the shears per stud, as required by Article 10.4.3.5.

$$\text{Force} = \sqrt{25.26^2 + 11.67^2} = 27.8\ \text{kips}$$

$$\frac{27.8}{0.85(36)} = 0.91 < 1.00\ \text{OK}$$

In this example, the ultimate strength of the shear connectors is checked only at the pier. The shear connectors should also be checked for the thinner plate where the concrete resists a larger portion of a smaller shear. Also, as specified in Article 10.4.3.5, the number of shear connectors should be increased where the concrete terminates to satisfy the requirements of **AASHTO Article 10.38.5.1.3**. Finally, the shear connectors should also be checked for fatigue according to the provisions of Article 7.2.2, with F_{fat} in Eq (7-11) taken as the portion of the torsional shear range due to the factored fatigue vehicle resisted by the box flange concrete determined in a fashion similar to that demonstrated above (refer again to Article 10.4.3.5).

Girder Stress Check Section 2-2 G2 Node 20.3
Stresses

Check the bottom flange bending stress at Section 2-2, which is located 100 feet from the abutment. Since this is the location of the bolted field splice in Span 1, it is desirable to terminate the longitudinal flange stiffener at this location where the longitudinal stress at the free edge of the flange is zero. By terminating the longitudinal flange stiffener at the free edge of the flange (at the bolted splice) and not extending it further into the end span, fatigue of the base metal at the terminus of the stiffener-to-flange weld need not be considered. The bottom flange splice plate inside the box must be split to permit the stiffener to extend to the free edge of the flange (Figure E6). Also, the compressive strength of the unstiffened bottom (box) flange on the side of the field splice directly across from the stiffener termination must be checked at the strength limit state to ensure that the stiffener can be terminated at this section. The section properties of the section without the flange stiffener are used below.

Compute the vertical bending stresses in the top extreme fiber of the steel at this section. Moments are from Table D1. Section properties are from Table D5. In this particular case, the girder sections immediately to the left and right of Section 2-2 are the same (except for the flange stiffener).

$$f_{top}\ (\text{Steel}) = \frac{462 \times 42.80}{185{,}187} \times 12 = -1.28 \text{ ksi}$$

$$f_{top}\ (\text{Deck}) = \frac{1{,}941 \times 42.80}{185{,}187} \times 12 = -5.38 \text{ ksi}$$

$$f_{top}\ (\text{Superimposed DL}) = \frac{754 \times 24.27}{354{,}925} \times 12 = -0.62 \text{ ksi for } 3n$$

$$f_{top}\ (\text{Superimposed DL}) = \frac{754 \times 42.80}{185{,}187} \times 12 = -2.09 \text{ ksi for cracked section w/o rebars}$$

$$f_{top}\ (L + I) = \frac{4{,}940(5/3) \times 10.78}{479{,}646} \times 12 = -2.22 \text{ ksi for } n$$

$$f_{top}\ (L + I) = \frac{|-3{,}054|(5/3) \times 42.80}{185{,}187} \times 12 = 14.12 \text{ ksi for cracked section w/o rebars}$$

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

$$f_{top} = 1.3(-1.28 - 5.38 - 0.62 - 2.22) = -12.35 \text{ ksi}$$

$$f_{top} = 1.3(-1.28 - 5.38 - 2.09 + 14.12) = 6.98 \text{ ksi}$$

By similar computations, Tables E2, E3 and E4 are created. Overload stresses in Table E3 due to loads acting on the composite section are computed assuming an uncracked section, as specified in Article 10.5.

Girder Stress Check Section 2-2 G2 Node 20.3
Stress Summary (ksi)

Table E2 Strength Limit State at 100 feet from Left Abutment

Location	Steel	Deck	Superimp DL	HS25 (L + I) x (5/3)		1.3 x Sum
Top Flange	-1.28	-5.38	-0.62	Positive	-2.22	-12.35
			-2.09	Negative	14.12	6.98
Top Web	-1.25	-5.25	-0.59	Positive	-2.01	-11.83
			-2.04	Negative	13.79	6.83
Bottom Flange	1.10	4.63	1.41	Positive	14.18	27.72
			1.80	Negative	-12.15	-6.01
Bottom Web	1.08	4.55	1.39	Positive	14.05	27.39
			1.77	Negative	-11.94	-5.90

Table E3 Overload at 100 feet from Left Abutment

Location	Steel	Deck	Superimp DL	HS20 (L + I) x (5/3)		Sum
Top Flange	-1.28	-5.38	-0.62	Positive	-1.78	-9.06
				Negative	1.10	-6.18
Top Web	-1.25	-5.25	-0.59	Positive	-1.61	-8.70
				Negative	1.00	-6.09
Bottom Flange	1.10	4.63	1.41	Positive	11.34	18.48
				Negative	-7.01	0.13
Bottom Web	1.08	4.55	1.39	Positive	11.24	18.26
				Negative	-6.95	0.07

Table E4 Constructibility Limit State at 100 feet from Left Abutment

Location	Steel	Cast #1	1.4 x Sum
Top Flange	-1.28	-16.17	-24.43
Top Web	-1.25	-15.79	-23.86
Bottom Flange	1.10	13.91	21.01
Bottom Web	1.08	13.68	20.66

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

Check the compressive strength of the unstiffened bottom flange directly across from the flange stiffener termination according to the provisions of Article 10.4.2.4.1.

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

Load	Torque (Table D3)
Steel	-36
Deck	-125
Total Torque	-161 k-ft

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

The enclosed area of the non-composite box is computed to be $A_o = 55.0 \text{ ft}^2$.

$$f_v = \frac{T}{2A_o t_f} = \frac{|-161|}{2 \times 55.0 \times 0.625} \times \frac{1}{12} = 0.20 \text{ ksi}$$

where: T = torque; A_o = enclosed area of box; t_f = flange thickness

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

Load	Torque (Table D3)
SupImp DL	-134
Live Load 5/3 (-445) =	-742
Total Torque	-876 k-ft

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

The enclosed area of the composite box is computed to be $A_o = 60.8 \text{ ft}^2$.

$$f_v = \frac{T}{2A_o t_f} = \frac{|-876|}{2 \times 60.8 \times 0.625} \times \frac{1}{12} = 0.96 \text{ ksi}$$

$$f_v = 1.3(0.20 + 0.96) = 1.51 \text{ ksi}$$

Compute the critical stress for the bottom flange at the strength limit state.

<u>Girder Stress Check Section 2-2 G2 Node 20.3</u>
<u>Strength - Bottom Flange (continued)</u>

Compute Δ according to Article 10.4.2.3.

$$\Delta = \sqrt{1-3\left(\frac{f_v}{F_y}\right)^2} \qquad \text{Eq (10-3)}$$

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

$$F_{cr} = F_y\Delta,\ F_{cr} = 50 \times 0.999 = 49.95 \text{ ksi}$$

Check Equation (10-4):

b_f= 81 in.; t_f= 0.625 in.

$$\sqrt{F_y}\frac{b_f}{t_f} = \sqrt{50}\frac{81}{0.625} = 916 \qquad \text{Eq (10-4)}$$

where: b_f = flange width between webs (in.)
t_f = flange thickness (in.)

R_1 is taken as:

$$R_1 = \frac{97\sqrt{k}}{\sqrt{\frac{1}{2}\left[\Delta + \sqrt{\Delta^2 + 4\left(\frac{f_v}{F_y}\right)^2\left(\frac{k}{k_s}\right)^2}\right]}} \qquad \text{Eq (10-5)}$$

where: k = plate buckling coefficient = 4.0
k_s = shear buckling coefficient = 5.34

Since the denominator of Eq (10-5) is approximately 1.0 in this case,
$R_1 = 97\sqrt{k} = 97 \times \sqrt{4.0} = 194$

$R_2 = 210 \times \sqrt{4.0} = 420 < 916$ Eq (10-7)

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

Therefore, use Equation (10-8) to determine the critical flange stress.

$$F_{cr} = (26.21)(10^3)k\left(\frac{t_f}{b_f}\right)^2 - \frac{f_v^2 k}{26.21(10^3)\ k_s^2 \left(\frac{t_f}{b_f}\right)^2} \qquad \text{Eq (10-8)}$$

$$F_{cr} = (26.21)(10^3)(4.0)\left(\frac{0.625}{81}\right)^2 - \frac{1.51^2 \text{ x } 4.0}{26.21(10^3)(5.34)^2\left(\frac{0.625}{81}\right)^2} = 6.04 \text{ ksi}$$

From Table E2, the computed factored compressive stress in the bottom flange for strength = -6.01 ksi.

$$\frac{|-6.01|}{6.04} = 0.99 < 1.00 \text{ OK}$$

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

<u>Bolted Splice Design Section 2-2 G2 Node 20.3</u>
<u>Design Action Summary and Section Information</u>

Design the bolted field splice at this section according to the provisions of Article 11 of the Recommended Specifications in conjunction with the revised provisions for the design of bolted splices appearing in the 1999 Interims to **AASHTO Article 10.18**.

Table E5 Unfactored Actions

<table>
<tr><th>Load</th><th>Moment (k-ft)</th><th>Torque (k-ft)</th><th colspan="2">Top Flange Lateral Moment (k-ft)</th><th colspan="2">Shear (kips)</th></tr>
<tr><td>Steel</td><td>462</td><td>-36</td><td colspan="2">-1</td><td colspan="2">-17</td></tr>
<tr><td>Deck</td><td>1,941</td><td>-125</td><td colspan="2">-7</td><td colspan="2">-69</td></tr>
<tr><td>Comp DL</td><td>754</td><td>-134</td><td colspan="2">-3</td><td colspan="2">-28</td></tr>
<tr><td>Cast #1</td><td>5,830</td><td>-188</td><td colspan="2">-15</td><td colspan="2">-58</td></tr>
<tr><td rowspan="3">Overload HS20 with Impact</td><td>Truck</td><td>Lane</td><td colspan="2">Torque (k-ft)</td><td>Truck</td><td>Lane</td></tr>
<tr><td>3,554</td><td>3,952</td><td rowspan="2">236</td><td rowspan="2">-356</td><td>29</td><td>22</td></tr>
<tr><td>-1,731</td><td>-2,443</td><td>-56</td><td>-62</td></tr>
<tr><td rowspan="2">Strength HS25 with Impact</td><td>4,442</td><td>4,940</td><td rowspan="2">295</td><td rowspan="2">-445</td><td>36</td><td>28</td></tr>
<tr><td>-2,164</td><td>-3,054</td><td>-70</td><td>-77</td></tr>
</table>

Note: Reported shears are vertical shears and are for bending plus torsion in the critical web.

Table E6 Tub Cross Section

Component	Size (in)	Area (in^2)	Yield (F_y)	Tensile (F_u)
Top Flanges	2-16 x 1	32.00	50	65
Web	2-78 x 0.5625	90.56	50	65
Bottom Flange	83 x 0.625	51.88	50	65

Note: Other section properties for the gross section may be found in Table D5. The cross section is the same on both sides of the splice (except for the presence of a bottom flange longitudinal stiffener on one side).

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

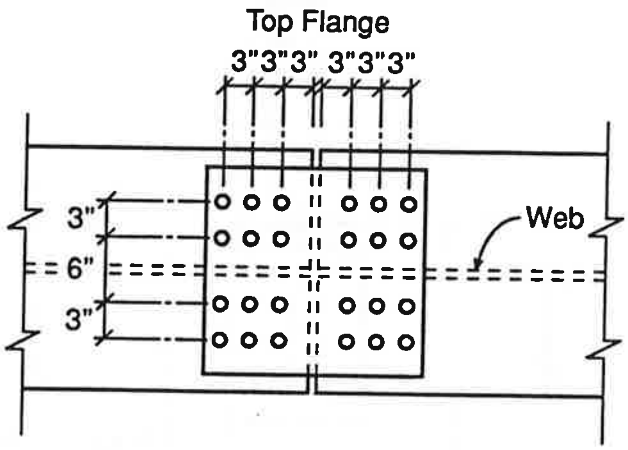
Bolt capacities
Use: 7/8" ϕ A325 bolts. Use standard size holes 1/16" larger than the bolt diameter (Article 11.2). According to **AASHTO Article 10.16.14.6**, the diameter of a standard hole is to be taken as 1/8" greater than the diameter of the bolt for design.

Use a Class B surface condition. Bolts are in double shear and threads are not permitted in the shear planes.

Service and Constructibility
Slip limit = 32.0 ksi **(AASHTO Table 10.57A)** for a Class B surface condition
0.60 in^2 x 32.0 ksi x 2 Planes = 38.4 k/bolt

Strength
Shear - **(AASHTO Table 10.56A)**:
Shear limit = 1.25 x 35 = 43.8 ksi; 0.60 x 43.8 x 2 Planes = 52.6 k/bolt

Bolted Splice Design Section 2-2 G2 Node 20.3
Bolt Patterns for Top and Bottom Flanges



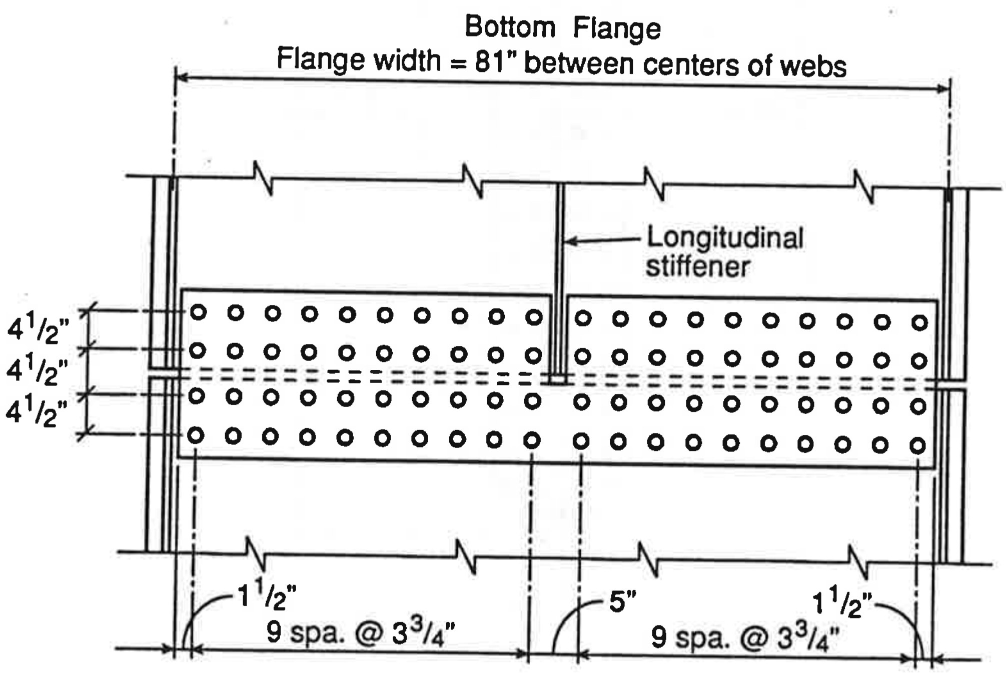
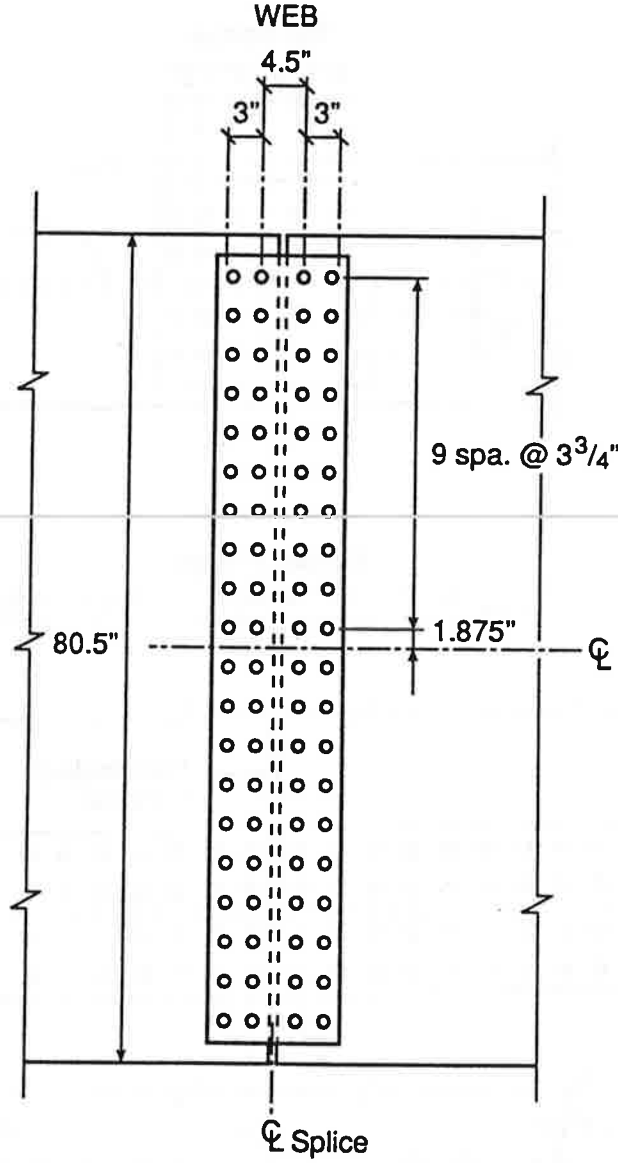


Figure E6 Bolt Patterns for Top and Bottom Flange

Bolted Splice Design Section 2-2 G2 Node 20.3
Bolt Pattern for Web



Notes: (1) 1/2" gap assumed between the edges of the field pieces.
(2) The indicated distances are along the web slope.

Figure E7 Bolt Pattern for Web

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

AASHTO Article 10.18.2.2.2 requires that high-strength bolted connections for flange splices be designed to prevent slip under an overload design force. In addition, **AASHTO Article 10.18.2.1.4** requires that high-strength bolted connections be proportioned to prevent slip for constructibility. These same requirements are stated in Article 11.2 of the Recommended Specifications.

Constructibility

Since Cast #1 causes a larger positive moment than the entire deck, Steel + Cast #1 controls. Constructibility: Load factor = 1.4 (Article 3.3).

Article 11.1 requires that lateral bending be considered in the design of curved girder splices. Lateral flange bending must be considered for the top flanges of tub girders prior to hardening of the deck. To account for the effects of lateral flange bending, the flange splice bolts will be designed for the combined effects of shear and moment using the traditional elastic vector method. The shear on the bolts is caused by the flange force calculated from the average vertical bending stress in the flange and the moment on the bolts is caused by the lateral flange bending. Article 11.1 also requires that warping be considered when checking for slip of bolted connections in curved box splices. Examination of the warping stresses in the top flange for this load condition from the analysis indicates that they are negligible at this section in this example.

Compute the polar moment of inertia of the top flange bolt pattern shown in Figure E6.

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

where:

A_b = area of one bolt (in^2)

Moment = 462 + (5,830) = 6,292 k-ft from Table E5
Lateral flange moment = -1 + (-15) = -16 k-ft from Table E5

The factored vertical bending stresses are taken from Table E4.

$$f_{top\ flg} = -24.43 \text{ ksi}$$

$$f_{top\ web} = -23.86 \text{ ksi}$$

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.

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

$$F_{top} = \left(\frac{-24.43 - 23.86}{2}\right) \times 16.00 = -386 \text{ kips}$$

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

$$F_L = \frac{386}{12} = 32.17 \text{ k/bolt}$$

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

$$F_{L\ lat} = \frac{16 \times 6.0}{342} \times 12 \times 1.4 = 4.72 \text{ k/bolt}$$

Compute the transverse component of force in the critical bolt.

$$F_{Tr} = \frac{16 \times 3.0}{342} \times 12 \times 1.4 = 2.36 \text{ k/bolt}$$

$$F_{L\ tot} = 32.17 + 4.72 = 36.89 \text{ k/bolt}$$

Compute the resultant force on the critical bolt.

$$\Sigma_F = \sqrt{2.36^2 + 36.89^2} = 36.97 \text{ k/bolt}; \frac{36.97}{38.4} = 0.96 < 1.0 \text{ OK}$$

Overload

Compute the average vertical bending stress in the top flange at overload (Article 3.5.4). According to the provisions of Article 10.5, the composite section is to be considered uncracked at overload. Since the splice is located in an area of potential stress reversal, both positive and negative live load bending conditions must be considered. Examination of Table E3 indicates that the positive live load bending condition controls.

$$f_{top\ flg} = -9.06 \text{ ksi}$$
$$f_{top\ web} = -8.70 \text{ ksi}$$

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.

$$F_{top} = \left(\frac{-9.06 - 8.70}{2}\right) \times 16.00 = -142 \text{ kips}$$

Article 11.1 specifies that warping be considered when checking for slip of bolted connections in curved box splices. Warping in the top flange is considered to be negligible after the deck has hardened. Also, lateral flange bending is not considered after the deck

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

has hardened and the section is closed. St. Venant torsional shears are also not considered in the top flanges of tub girders. By inspection, overload slip does not control.

Since Cast #1 causes a larger positive moment than the entire deck, Steel + Cast #1 controls constructibility. Load factor = 1.4 (Article 3.3). The factored vertical bending stresses are taken from Table E4.

$$f_{bot\ flg} = 21.01 \text{ ksi}$$
$$f_{bot\ web} = 20.66 \text{ ksi}$$

Examination of the warping stresses in the bottom flange for Steel + Cast #1 from the analysis indicates that they are negligible at this section in this example.

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

$$F_{bot} = \left(\frac{21.01 + 20.66}{2}\right) \times 51.88 = 1{,}081 \text{ kips}$$

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

Compute the polar moment of inertia of the bottom flange bolt pattern shown in Figure E6.

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

Compute the factored St. Venant torsional shear in the bottom flange. From Table E5, the unfactored torque due to Steel plus Cast #1 = -36 + (-188) = -224 k-ft. The enclosed area of the non-composite box, A_o, is computed to be 55.0 ft^2.

$$V = \frac{T}{2A_o} b_f = \frac{|-224|}{2(55.0)} \times \frac{81}{12} \times 1.4 = 19.2 \text{ kips}$$

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

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

$$M = 19.2 \times (2.25 + 2.25) = 86.4 \text{ k-in}$$

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

$$F_{LM} = \frac{86.4 \times 36.25}{19{,}859} = 0.16 \text{ k/bolt}$$

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

$$F_L = \frac{1{,}081}{40} = 27.03 \text{ k/bolt}$$

$$F_{L\,tot} = 27.03 + 0.16 = 27.19 \text{ k/bolt}$$

Compute the transverse component of force in the critical bolt.

$$F_{Tr} = \frac{86.4 \times 2.25}{19{,}859} = 0.01 \text{ k/bolt}$$

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

$$F_v = \frac{19.2}{40} = 0.48 \text{ k/bolt}$$

$$F_{T\,tot} = 0.01 + 0.48 = 0.49 \text{ k/bolt}$$

Compute the resultant force in the critical bolt.

$$\Sigma_F = \sqrt{27.19^2 + 0.49^2} = 27.19 \text{ k/bolt}; \quad \frac{27.19}{38.4} = 0.71 < 1.0 \text{ OK}$$

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

Compute the average vertical bending stress in the bottom flange at overload (Article 3.5.4).

According to the provisions of Article 10.5, the composite section is assumed uncracked at overload. Examination of Table E3 indicates that the positive live load bending condition controls.

$$f_{bot\ flg} = 18.48 \text{ ksi}$$

$$f_{bot\ web} = 18.26 \text{ ksi}$$

Warping stresses at overload in the bottom flange are negligible at this section in this example. Compute the overload design force, P_{fo}, in the bottom flange from the average overload vertical bending stress in the flange. The gross section of the flange is used to check for slip.

$$P_{fo} = \left(\frac{18.48 + 18.26}{2}\right) \times 51.88 = 953 \text{ kips}$$

Compute the overload St. Venant torsional shear in the bottom flange. From Table E5, the torques are as follows:

Load	Torque
Steel	-36
Deck	-125
Non-composite torque	-161 k-ft
SupImp DL	-134
(5/3)HS20 (LL+I)	-593
Composite torque	-727 k-ft

$$V = \frac{T}{2A_o} b_f$$

$$V_{non\text{-}comp} = \frac{|-161|}{2(55.0)} \times \frac{81}{12} = 9.9 \text{ kips}$$

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

$$V_{comp} = \frac{|-727|}{2(60.8)} \times \frac{81}{12} = 40.4 \text{ kips}$$

<u>Bolted Splice Design Section 2-2 G2 Node 20.3</u>
<u>Overload - Bottom Flange (continued)</u>

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

$$M = (9.9 + 40.4) \times (2.25 + 2.25) = 226.4 \text{ k-in}$$

$$F_{LM} = \frac{226.4 \times 36.25}{19,859} = 0.41 \text{ k/bolt}$$

$$F_L = \frac{953}{40} = 23.83 \text{ k/bolt}$$

$$F_{L\,tot} = 23.83 + 0.41 = 24.24 \text{ k/bolt}$$

$$F_{Tr} = \frac{226.4 \times 2.25}{19,859} = 0.03 \text{ k/bolt}$$

$$F_v = \frac{(9.9 + 40.4)}{40} = 1.26 \text{ k/bolt}$$

$$F_{T\,tot} = 0.03 + 1.26 = 1.29 \text{ k/bolt}$$

$$\Sigma_F = \sqrt{1.29^2 + 24.24^2} = 24.27 \text{ k/bolt}; \ \frac{24.27}{38.4} = 0.63 < 1.0 \text{ OK}$$

Bolted Splice Design Section 2-2 G2 Node 20.3
Strength - Top and Bottom Flange

The effective area of the top flange is computed from **AASHTO Article 10.18.2.2.4** as follows:

$$A_e = W_n t + \beta A_g \le A_g$$

where

W_n	=	least net width of the flange
t	=	flange thickness
β	=	0.15 (for this case)
A_g	=	gross area of the flange

$$A_e = [16.0 - 4(0.875 + 0.125)](1.0) + (0.15)(16.0)(1.0) = 14.4 \text{ in}^2$$

$$A_g = (16.0)(1.0) = 16.0 \text{ in}^2 > 14.4 \text{ in}^2$$

$$\therefore A_e = 14.4 \text{ in}^2$$

The effective width of the top flange is computed as:

$$(b_f)_{eff} = \frac{A_e}{t} = \frac{14.4}{1.0} = 14.4 \text{ in}$$

Section properties computed using the effective top flange width are used to calculate the vertical bending stresses in the flange at the splice for strength whenever the top flange is subjected to tension. The gross area is used for the bottom flange.

Similarly, the effective area of the bottom flange is computed as:

$$A_e = [83.0 - 20(0.875 + 0.125)](0.625) + 0.15(83.0)(0.625) = 47.2 \text{ in}^2$$

$$A_g = (83.0)(0.625) = 51.9 \text{ in}^2 > 47.2 \text{ in}^2$$

$$\therefore A_e = 47.2 \text{ in}^2$$

For the bottom (box) flange, an effective flange thickness rather than an effective flange width will be computed in order to maintain the same web slope. The effective thickness of the bottom flange is computed as:

$$(t_f)_{eff} = \frac{A_e}{b_f} = \frac{47.2}{83.0} = 0.57 \text{ in}$$

Bolted Splice Design Section 2-2 G2 Node 20.3
Strength - Top and Bottom Flange (continued)

Section properties computed using the effective bottom flange thickness are used to calculate the vertical bending stresses in the flange at the splice for strength whenever the bottom flange is subjected to tension. The gross area is used for the top flange in this case. If yielding on the effective area is prevented in a flange or splice plate subjected to tension, then fracture on the net section will theoretically not occur (for typical ratios of net to gross area ≥ 0.5 and yield strengths of 70 ksi or below) and need not be explicitly checked. 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.

Using the effective section properties (from separate calculations), calculate the average factored vertical bending stress in the top and bottom flange at the strength limit state for both the positive and negative live load bending conditions. The longitudinal component of the top flange bracing area is again included in the effective section properties. Deck rebars are not included at this section. The smaller section is to be used to design the splice, therefore, the longitudinal flange stiffener is not included. The provisions of Article 4.5.2 are followed to determine which composite section (cracked or uncracked) to use.

Negative live load bending case

$$F_{topflgavg} = \left[\frac{2{,}403 \times 41.29}{179{,}050} + \frac{754 \times 43.02}{179{,}740} + \frac{-3{,}054(5/3) \times 43.02}{179{,}740}\right] \times 12 \times 1.3 = 7.55 \text{ ksi}$$

$$F_{botflgavg} = \left[\frac{2{,}403 \times 37.52}{179{,}050} + \frac{754 \times 35.79}{179{,}740} + \frac{-3{,}054(5/3) \times 35.79}{179{,}740}\right] \times 12 \times 1.3 = -5.61 \text{ ksi}$$

Positive live load bending case

$$F_{topflgavg} = \left[\frac{2{,}403 \times 41.29}{179{,}050} + \frac{754 \times 23.18}{338{,}310} + \frac{4{,}940(5/3) \times 9.91}{456{,}064}\right] \times 12 \times 1.3 = -12.24 \text{ ksi}$$

$$F_{botflgavg} = \left[\frac{2{,}403 \times 37.52}{179{,}050} + \frac{754 \times 55.63}{338{,}310} + \frac{4{,}940(5/3) \times 68.90}{456{,}064}\right] \times 12 \times 1.3 = 29.19 \text{ ksi}$$

Separate calculations (similar to subsequent calculations) show that the positive live load bending case is critical. The bottom flange is the controlling flange since it has the largest average flexural stress for this loading case. **AASHTO Article 10.18.2.2.1** defines the design stress, F_{cu}, for the controlling flange as follows:

$$F_{cu} = \frac{|f_{cu}/R| + \alpha F_y}{2} \geq 0.75\alpha F_y$$

Bolted Splice Design Section 2-2 G2 Node 20.3
Strength - Top and Bottom Flange (continued)

f_{cu} is the average factored vertical bending stress in the controlling flange at the splice. The hybrid factor R is taken as 1.0 since horizontally curved hybrid girders are not permitted and α is taken as 1.0 for flanges in tension. For a non-composite box flange subject to compression, α may be taken equal to the ratio of the critical flange compressive stress, F_{cr}, to the yield stress, F_y.

$$F_{cu} = \frac{|29.19/1.0| + 1.0(50)}{2} = 39.60 \text{ ksi (controls)}$$

$$0.75\alpha F_y = 0.75(1.0)(50) = 37.50 \text{ ksi}$$

The minimum design force for the controlling flange, P_{cu}, is taken equal to F_{cu} times the smaller effective flange area, A_e, on either side of the splice. The area of the smaller flange is used to ensure that the design force does not exceed the strength of the smaller flange. In this case, the effective flange areas are the same on both sides of the splice.

$$P_{cu} = 39.60(47.2) = 1{,}869 \text{ kips (tension)}$$

The minimum design stress for the non-controlling (top) flange for this case is specified in **AASHTO Article 10.18.2.2.1** as:

$$F_{ncu} = R_{cu}(|f_{ncu}/R|) \geq 0.75\alpha F_y$$

where α is taken as 1.0 for flanges in compression that are continuously braced (in this case by the hardened concrete deck).

$$R_{cu} = |F_{cu}/f_{cu}| = |39.60/29.19| = 1.36$$

f_{ncu} is the average factored vertical bending stress in the non-controlling flange at the splice concurrent with f_{cu}.

$$R_{cu}(|f_{ncu}/R|) = 1.36(|-12.24/1.0|) = 16.65 \text{ ksi}$$

$$0.75\alpha F_y = 0.75(1.0)(50) = 37.50 \text{ ksi (controls)}$$

The minimum design force for the non-controlling flange, P_{ncu}, is computed as:

$$P_{ncu} = F_{ncu}A_e$$

where the effective flange area, A_e, is taken equal to the smaller gross flange area, A_g, on either side of the splice since the flange is subjected to compression. In this case, the

Bolted Splice Design Section 2-2 G2 Node 20.3
Strength - Top and Bottom Flange (continued)

gross flange areas are the same on both sides of the splice.

$$P_{ncu} = (37.50)(16.0)(1.0) = 600 \text{ kips (compression)}$$

Top Flange

St. Venant torsional shears are not considered in the top flanges of tub girders. Lateral flange bending in the top flange is also not considered after the deck has hardened and the section is closed. Warping also need not be considered in top flanges when checking the strength limit state after the deck has hardened. Therefore:

$$\text{No. bolts req'd} = \frac{600}{52.6} = 11.41 \text{ bolts, use 12 bolts}$$

$$\frac{600}{12} = 50.00 \text{ k/bolt}; \ \frac{50.00}{52.6} = 0.95 < 1.0 \text{ OK}$$

Since a fill plate is not required for the top flange splice, no reduction in the bolt design shear strength is required per the requirements of **AASHTO Article 10.18.1.2.1**.

Bottom Flange

Compute the factored St. Venant torsional shear in the bottom flange at the strength limit state. Warping torsion is ignored since it is assumed in this example that the spacing of the internal bracing is sufficient to limit the warping stress to 10 percent of the vertical bending stress at the strength limit state. Further, the specifications do not require warping to be considered in the design of bolted box flange splices at the strength limit state. From Table E5, the torques are as follows:

Load	Torque
Steel	-36
Deck	-125
Non-composite torque	-161 k-ft
SupImp DL	-134
5/3 x HS25 (LL+I)	-742
Composite torque	-876 k-ft

$$V = \frac{T}{2A_o} b_f$$

$$V_{non-comp} = \frac{|-161|}{2(55.0)} \times \frac{81}{12} \times 1.3 = 12.84 \text{ kips}$$

Bolted Splice Design Section 2-2 G2 Node 20.3
Strength - Top and Bottom Flange (continued)

$$V_{comp} = \frac{|-876|}{2(60.8)} \times \frac{81}{12} \times 1.3 = 63.21 \text{ kips}$$

$$V_{tot} = 12.84 + 63.21 = 76.05 \text{ kips}$$

The total torsional shear is then factored up by R_{cu}= 1.36 (see earlier calculations) to be consistent with the computation of F_{cu} and P_{cu}.

$$V_{fact} = 76.05 \times 1.36 = 103.4 \text{ kips}$$

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

$$M = 103.4 \times (2.25 + 2.25) = 465.3 \text{ k-in}$$

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

$$F_{LM} = \frac{465.3 \times 36.25}{19{,}859} = 0.85 \text{ k/bolt}$$

Compute the transverse component of force in the critical bolt.

$$F_{Tr} = \frac{465.3 \times 2.25}{19{,}859} = 0.05 \text{ k/bolt}$$

Compute the force in each bolt due to the minimum design force, P_{cu}.

$$F_L = \frac{1{,}869}{40} = 46.73 \text{ k/bolt}$$

$$F_{L\ tot} = 46.73 + 0.85 = 47.58 \text{ k/bolt}$$

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

$$F_v = \frac{103.4}{40} = 2.59 \text{ k/bolt}$$

$$F_{T\ tot} = 2.59 + 0.05 = 2.64 \text{ k/bolt}$$

Compute the resultant force on the critical bolt.

$$\Sigma_F = \sqrt{2.64^2 + 47.58^2} = 47.65 \text{ k/bolt}; \quad \frac{47.65}{52.6} = 0.91 < 1.00 \text{ OK}$$

Bolted Splice Design Section 2-2 G2 Node 20.3
Constructibility and Overload - Web

Note that a fill plate is also not required for the bottom flange splice. Therefore, no reduction in the bolt design shear strength is necessary.

A pattern of two rows of 7/8-inch bolts spaced vertically at 3.75 inches will be tried for the web splice. There are 40 bolts on each side of the web splice. The pattern is shown in Figure E7. Note that there is 4.625 inches 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 connection.

$$I_p^1 = A_b \frac{nm}{12} \times [s^2(n^2 - 1) + g^2(m^2 - 1)]$$

where:

- n = number of lines
- m = number of rows
- s = pitch of rows
- g = spacing of rows
- A_b = area of one bolt

[1]Equation from: F.I. Sheikh-Ibrahim, Ph.D. dissertation, "Development of Design Procedure for Steel Girder Splices," U. Texas, 1995.

For n=20; m=2; s=3.75 in; g=3 in.,

$$I_p = \frac{20 \times 2}{12}[3.75^2(20^2 - 1) + 3^2(2^2 - 1)]A_b = 18{,}793A_b \text{ in}^4$$

AASHTO Article 10.18.2.1.4 requires that high-strength bolted connections be proportioned to prevent slip for constructibility. **AASHTO Article 10.18.2.3.5** further requires that bolted web splices be designed to prevent slip under the most critical combination of the design actions at overload. These same requirements are stated in Article 11.2 of the Recommended Specifications.

Constructibility

From Table E5, compute the factored vertical shear at the splice (bending plus torsional shear in the critical web) due to Steel plus Cast #1.

Bolted Splice Design Section 2-2 G2 Node 20.3
Constructibility and Overload - Web (continued)

$$V = (-17 - 58) \times 1.4 = -105 \text{ kips}$$

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 E7).

$$M_v = Ve = 105(3/2 + 2.25)(1/12) = 32.8 \text{ k-ft}$$

Determine the portion of the vertical bending moment resisted by the web, M_w, and the horizontal force resultant in the web, H_w, using equations similar to those provided in **AASHTO Article 10.18.2.3.5** for overload. M_w and H_w are assumed to be applied at the centroid of the connection. Using the results from earlier calculations (page 129), the average factored vertical bending stress in the top flange for Steel plus Cast #1 is computed as:

$$\left(\frac{-24.43 - 23.86}{2}\right) = -24.15 \text{ ksi}$$

The average factored vertical bending stress in the bottom flange is (see page 131)

$$f_{bf} = \left(\frac{21.01 + 20.66}{2}\right) = 20.84 \text{ ksi}$$

Using these stresses

$$M_w = \frac{t_w D^2}{12}\left|f_{tf} - f_{bf}\right| = \frac{0.5625(78)^2}{12}\left|-24.15 - (20.84)\right|(1/12) = 1{,}069.2 \text{ k-ft}$$

$$H_w = \frac{t_w D}{2}(f_{tf} + f_{bf}) = \frac{0.5625(78)}{2}(-24.15 + 20.84) = -72.6 \text{ kips}$$

The total moment on the web splice is computed as:

$$M_{tot} = M_v + M_w = 32.8 + 1{,}069.2 = 1{,}102 \text{ k-ft}$$

Compute the in-plane bolt force due to the factored vertical shear.

$$F_s = \frac{V}{N_b} = \frac{105}{40} = 2.63 \text{ k/bolt; } 2.63/\cos 14.3° = 2.71 \text{ k/bolt}$$

Compute the in-plane bolt force due to the horizontal force resultant.

$$F_H = \frac{H_w}{N_b} = \frac{72.6}{40} = 1.82 \text{ k/bolt; } 1.82/\cos 14.3° = 1.88 \text{ k/bolt}$$

Bolted Splice Design Section 2-2 G2 Node 20.3
Constructibility and Overload - Web (continued)

Compute the in-plane horizontal and vertical components of the force on the extreme bolt due to the total moment on the splice.

$$F_{Mv} = \frac{M_{tot}x}{I_p} = \frac{1{,}102(12)(3/2)}{18{,}793} = 1.06 \text{ k/bolt; } 1.06/\cos 14.3° = 1.09 \text{ k/bolt}$$

$$F_{Mh} = \frac{M_{tot}y}{I_p} = \frac{1{,}102(12)(35.63)}{18{,}793} = 25.07 \text{ k/bolt; } 25.07/\cos 14.3° = 25.87 \text{ k/bolt}$$

Compute the resultant in-plane bolt force.

$$F_r = \sqrt{(F_s + F_{Mv})^2 + (F_H + F_{Mh})^2} = \sqrt{(2.71 + 1.09)^2 + (1.88 + 25.87)^2} = 28.01 \text{ k/bolt}$$

$$\frac{28.01}{38.4} = 0.73 < 1.0 \text{ OK}$$

Overload

Compute the factored vertical overload design shear, V_{wo}, at the splice (bending plus torsional shear in the critical web), which is simply taken equal to the maximum vertical shear at the splice, V_o, due to the overload (Article 3.5.4) according to the provisions of **AASHTO Article 10.18.2.3.5**. From Table E5:

$$V_{wo} = V_o = [-17 - 69 - 28 + 5/3(-62)] = -217 \text{ kips}$$

Compute the moment, M_{vo}, due to the eccentricity of the overload design shear about the centroid of the connection (refer to the web bolt pattern in Figure E7).

$$M_{vo} = V_{wo}e = 217(3/2 + 2.25)(1/12) = 67.8 \text{ k-ft}$$

Determine the overload design moment, M_{wo}, and the overload horizontal design force resultant, H_{wo}, using the equations provided in **AASHTO Article 10.18.2.3.5**. M_{wo} and H_{wo} are assumed to be applied at the centroid of the connection. Separate calculations indicate that the positive live load bending case controls.

Using the results from earlier calculations (page 130 and 133), the average vertical bending stress in the top flange due to overload, f_o, is computed as:

$$f_o = \left(\frac{-9.06 - 8.70}{2}\right) = -8.88 \text{ ksi}$$

Bolted Splice Design Section 2-2 G2 Node 20.3
Constructibility and Overload - Web (continued)

The average vertical bending stress in the bottom flange due to overload, f_{of}, is

$$f_{of} = \left(\frac{18.48 + 18.26}{2}\right) = 18.37 \text{ ksi}$$

Using these stresses

$$M_{wo} = \frac{t_w D^2}{12}|f_o - f_{of}| = \frac{0.5625(78)^2}{12}|-8.88 - 18.37|(1/12) = 647.6 \text{ k-ft}$$

$$H_{wo} = \frac{t_w D}{2}(f_o + f_{of}) = \frac{0.5625(78)}{2}(-8.88 + 18.37) = 208.2 \text{ kips}$$

The total moment on the web splice is computed as:

$$M_{tot} = M_{vo} + M_{wo} = 67.8 + 647.6 = 715.4 \text{ k-ft}$$

Compute the in-plane bolt force due to V_{wo}.

$$F_s = \frac{V_{wo}}{N_b} = \frac{217}{40} = 5.43 \text{ k/bolt; } 5.43/\cos 14.3° = 5.60 \text{ k/bolt}$$

Compute the in-plane bolt force due to the overload horizontal design force resultant, H_{wo}.

$$F_H = \frac{H_{wo}}{N_b} = \frac{208.2}{40} = 5.21 \text{ k/bolt; } 5.21/\cos 14.3° = 5.38 \text{ k/bolt}$$

Compute the in-plane horizontal and vertical components of the force on the extreme bolt due to the total moment on the splice.

$$F_{Mv} = \frac{M_{tot}x}{I_p} = \frac{715.4(12)(3/2)}{18{,}793} = 0.69 \text{ k/bolt; } 0.69/\cos 14.3° = 0.71 \text{ k/bolt}$$

$$F_{Mh} = \frac{M_{tot}y}{I_p} = \frac{715.4(12)(35.63)}{18{,}793} = 16.28 \text{ k/bolt; } 16.28/\cos 14.3° = 16.80 \text{ k/bolt}$$

Compute the resultant in-plane bolt force.

$$F_r = \sqrt{(F_s + F_{Mv})^2 + (F_H + F_{Mh})^2} = \sqrt{(5.60 + 0.71)^2 + (5.38 + 16.80)^2} = 23.06 \text{ k/bolt}$$

$$\frac{23.06}{38.4} = 0.60 < 1.00 \text{ OK}$$

Bolted Splice Design Section 2-2 G2 Node 20.3
Strength - Web

Determine the vertical design shear, V_{wu}, for the web splice for strength according to the provisions of **AASHTO Article 10.18.2.3.2**.

From Table E5, the factored vertical shear at the splice (bending plus torsional shear in the critical web at the strength limit state) is computed as:

$$V = [-17 - 69 - 28 - 5/3(77)] \times 1.3 = -315 \text{ kips}$$

Compute the shear capacity of the 0.5625-in. thick web at the splice according to the provisions of Article 6.3.2.

Separate calculations indicate that transverse stiffeners are required.
Try a required stiffener spacing, d=80.5 in., which is equal to the web depth D along the inclined slope and is equal to the maximum permitted spacing according to Article 6.3. k_w is determined from Eq (6-9).

$$k_w = 5 + \frac{5}{(d/D)^2} = 5 + \frac{5}{(80.5/80.5)^2} = 10$$

Determine which equation is to be used to compute C.

$$\frac{D}{t_w} = \frac{80.5}{0.5625} = 143$$

$$1.38\sqrt{\frac{Ek_w}{F_y}} = 1.38\sqrt{\frac{29{,}000 \times 10}{50}} = 105 < 143$$

Therefore, use Eq (6-7).

$$C = \frac{1.52Ek_w}{(D/t_w)^2F_y} = \frac{1.52 \times 29{,}000 \times 10}{(143)^2 \times 50} = 0.43$$

$$V_{cr} = CV_p$$

$$V_p = 0.58F_yDt_w = 0.58 \times 50 \times 80.5 \times 0.5625 = 1{,}313 \text{ kips}$$

$$\therefore V_{cr} = V_u = 0.43 \times 1{,}313 = 565 \text{ kips} > 315/\cos 14.3° = 325 \text{ kips OK}$$

$$0.5V_u = 0.5(565) = 283 \text{ kips} < 315 \text{ kips}$$

Bolted Splice Design Section 2-2 G2 Node 20.3
Strength - Web (continued)

Therefore, according to **AASHTO Article 10.18.2.3.2**, since $V > 0.5V_u$:

$$V_{wu} = \frac{(V + V_u)}{2} = \frac{(315 + 565)}{2} = 440 \text{ kips}$$

The moment, M_{vu}, due to the eccentricity of V_{wu} from the centerline of the splice to the centroid of the web splice bolt group is computed from **AASHTO Article 10.18.2.3.3** as follows (refer to web bolt pattern in Figure E7):

$$M_{vu} = V_{wu}e$$

$$M_{vu} = 440(3/2 + 2.25)(1/12) = 137.5 \text{ k-ft}$$

Determine the portion of the vertical bending moment resisted by the web, M_{wu}, and the horizontal design force resultant in the web, H_{wu}, according to the provisions of **AASHTO Article 10.18.2.3.4**. M_{wu} and H_{wu} are assumed to act at the centroid of the connection. Separate calculations indicate that the positive live load bending condition controls.

As computed earlier (pages 136-137) for the positive live load bending case:

f_{cu} = 29.19 ksi
F_{cu} = 39.60 ksi
f_{ncu} = -12.24 ksi
R_{cu} = 1.36

From the equations in **AASHTO Article 10.18.2.3.4**:

$$M_{wu} = \frac{t_w D^2}{12}\left|RF_{cu} - R_{cu}f_{ncu}\right|$$

$$= \frac{0.5625(78)^2}{12}\left|1.0(39.60) - 1.36(-12.24)\right|(1/12) = 1{,}336.7 \text{ k-ft}$$

$$H_{wu} = \frac{t_w D}{2}(RF_{cu} + R_{cu}f_{ncu}) = \frac{0.5625(78)}{2}[1.0(39.60) + 1.36(-12.24)] = 503.5 \text{ kips}$$

The total moment on the web splice is computed as:

$$M_{tot} = M_{vu} + M_{wu} = 137.5 + 1{,}336.7 = 1{,}474.2 \text{ k-ft}$$

Compute the in-plane bolt force due to the vertical design shear.

Bolted Splice Design Section 2-2 G2 Node 20.3
Strength - Web (continued)

$$F_s = \frac{V_{wu}}{N_b} = \frac{440}{40} = 11.00 \text{ k/bolt; } 11.00/\cos 14.3° = 11.35 \text{ k/bolt}$$

Compute the in-plane bolt force due to the horizontal design force resultant.

$$F_H = \frac{H_{wu}}{N_b} = \frac{503.5}{40} = 12.59 \text{ k/bolt; } 12.59/\cos 14.3° = 12.99 \text{ k/bolt}$$

Compute the in-plane horizontal and vertical components of the force on the extreme bolt due to the total moment on the splice.

$$F_{Mv} = \frac{M_{tot}x}{I_p} = \frac{1{,}474.2(12)(3/2)}{18{,}793} = 1.41 \text{ k/bolt; } 1.41/\cos 14.3° = 1.46 \text{ k/bolt}$$

$$F_{Mh} = \frac{M_{tot}y}{I_p} = \frac{1{,}474.2(12)(35.63)}{18{,}793} = 33.54 \text{ k/bolt; } 33.54/\cos 14.3° = 34.61 \text{ k/bolt}$$

Compute the resultant in-plane bolt force.

$$F_r = \sqrt{(F_s + F_{Mv})^2 + (F_H + F_{Mh})^2}$$
$$= \sqrt{(11.35 + 1.46)^2 + (12.99 + 34.61)^2} = 49.29 \text{ k/bolt}$$

$$\frac{49.29}{52.6} = 0.94 < 1.0 \text{ OK}$$

Bolted Splice Design Section 2-2 G2 Node 20.3
Splice Plates

Web Splice Plate Design

Use nominal 0.375-in. thick splice plates. Fill plates are not required in this case.

The maximum permissible spacing of the bolts for sealing = $4 + 4t \leq 7.0 = 4 + 4(0.375) = 5.5$ in OK

Check bearing of the bolts on the connected material assuming the bolts have slipped and gone into bearing. The resultant force acting on the extreme bolt of the web splice is compared to the bearing strength of the web along two orthogonal shear failure planes. This is conservative since the components of the resultant force parallel to the failure surface are smaller than the maximum resultant force.

The clear distance between the edge of the hole and the edge of the field piece is computed as:

$$L_{c1} = 2.0 - \frac{1.0}{2} = 1.5 \text{ in.}$$

According to **AASHTO Article 10.56.1.3.2**, the bearing strength is computed as the lesser of

$$\phi R = 0.9 L_c t F_u = 0.9(1.5)(0.5625)(65) = 49.36 \text{ kips (controls)}$$

or

$$\phi R = 1.8 d t F_u = 1.8(0.875)(0.5625)(65) = 57.59 \text{ kips}$$

In the vertical direction between horizontal bolt rows:

$$L_{c2} = 3.75 - 1.0 = 2.75 \text{ in}$$

$$\phi R = 0.9(2.75)(0.5625)(65) = 90.49 \text{ kips}$$

The maximum resultant in-plane force on the extreme bolt was computed earlier (page 146) for strength to be

$$F_r = 49.29 \text{ kips}$$

$$\frac{49.29}{49.36} = 0.999 < 1.0 \text{ OK}$$

Bolted Splice Design Section 2-2 G2 Node 20.3
Splice Plates (continued)

Check for flexural yielding on the gross section of the web splice plates.

$$A_g = 2(0.375)(75.25) = 56.44 \text{ in}^2$$

$$S_{PL} = \frac{2(0.375)(75.25 \times \cos 14.3°)^2}{6} = 664.6 \text{ in}^3$$

$$f = \frac{M_{vu} + M_{wu}}{S_{PL}} + \frac{H_{wu}}{A_g}$$

$$= \frac{(137.5 + 1{,}336.7)(12)}{664.6} + \frac{503.5}{56.44} = 35.54 \text{ ksi} < 50 \text{ ksi OK}$$

Check for shear yielding on the gross section of the web splice plates.

$$V_y = 0.58 A_g F_y = 0.58(56.44)(50) = 1{,}637 \text{ kips}$$

$$\frac{V_{wu}}{V_y} = \frac{440/\cos 14.3°}{1{,}637} = 0.28 < 1.0 \text{ OK}$$

Flange Splice Plate Design

Top Flange

The width of the outside splice plate should be at least as wide as the width of the narrowest flange at the splice. In this case, however, the width of the top flange is the same on either side of the splice. Therefore;

Try: 15 x 0.5 in outer plate
$A_g = 7.50 \text{ in}^2$

Try: 2-6 x 0.625 in inner plates
$A_g = 7.50 \text{ in}^2$

As specified in **AASHTO Article 10.18.2.2.1**, the effective area, A_e, of each splice plate is to be sufficient to prevent yielding of each splice plate under its calculated portion of the minimum flange design force. If yielding on the effective area is prevented in splice plates subjected to tension, then fracture need not be explicitly checked (for typical ratios of net to gross area $\geq$ 0.5 and yield strengths not exceeding 70 ksi). For splice plates subjected to compression, the effective area is equal to the gross area.

For the negative live load bending case, the controlling flange is the top flange. The flange is subjected to tension under this live load bending condition (see page 136). Compute the minimum design force, P_{cu}, in the top flange for this load case.

Bolted Splice Design Section 2-2 G2 Node 20.3
Splice Plates (continued)

$$F_{cu} = \frac{|7.55/1.0 + 1.0(50)|}{2} = 28.78 \text{ ksi}$$

$$0.75\alpha F_y = 0.75(1.0)(50) = 37.50 \text{ ksi (controls)}$$

$$P_{cu} = F_{cu}A_e = 37.50(14.4) = 540 \text{ kips}$$

The effective areas of the inner and outer splice plates are computed as:

$$A_e = W_n t + \beta A_g \le A_g$$

$$\text{Outer: } A_e = [15.0 - 4(0.875 + 0.125)](0.5) + 0.15(7.50) = 6.63 \text{ in}^2$$

$$\text{Inner: } A_e = [2(6.0) - 4(0.875 + 0.125)](0.625) + 0.15(7.50) = 6.13 \text{ in}^2$$

As specified in **AASHTO Article 10.18.1.3**, if the combined area of the inner splice plates is within 10 percent of the area of the outside splice plate, then both the inner and outer plates may be designed for one-half the flange design force (which is the case here). Double shear may then be assumed in designing the 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.

As discussed previously, St. Venant torsional shear and lateral flange bending are not considered in the top flange at the strength limit state. Warping torsion is also ignored. The capacity of the splice plates to resist tension is therefore computed as:

$$P_y = F_y A_e$$

$$\text{Outer: } P_y = 50(6.63) = 332 \text{ kips} > 540/2 = 270 \text{ kips OK}$$
$$\text{Inner: } P_y = 50(6.13) = 307 \text{ kips} > 540/2 = 270 \text{ kips OK}$$

Under the positive live load bending case, the top flange is the non-controlling flange and is subjected to compression. The minimum design force, P_{ncu}, for the top flange for this load case was computed earlier (see page 138) to be 600 kips. The capacity of the splice plates to resist compression is computed as:

$$P_y = F_y A_e = F_y A_g$$

$$\text{Outer: } P_y = 50(7.50) = 375 \text{ kips} > 600/2 = 300 \text{ kips OK}$$

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Splice Plates (continued)

Inner: $P_y = 50(7.50) = 375 \text{ kips} > 600/2 = 300 \text{ kips OK}$

Check bearing of the bolts on the connected material under the minimum design force, P_{ncu}= 600 kips. The design bearing strength is taken as the sum of the bearing strengths of the individual bolt holes parallel to the line of the applied force. By inspection, the top flange governs the bearing strength of the connection.

For the three 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:

$$L_{c1} = 1.5 - \frac{1.0}{2} = 1.0 \text{ in}$$

The center-to-center distance between the bolts in the direction of the force is 3.0 in. Therefore

$$L_{c2} = 3.0 - 1.0 = 2.0 \text{ in}$$

According to **AASHTO Article 10.56.1.3.2**, the bearing strength for the end row of bolts is computed as the lesser of

$$\phi R = 4(0.9L_{c1}tF_u) = 4[0.9(1.0)(1.0)(65)] = 234.0 \text{ kips (controls)}$$

or

$$\phi R = 4(1.8dtF_u) = 4[1.8(0.875)(1.0)(65)] = 409.5 \text{ kips}$$

For the remaining bolt holes, the design bearing strength is taken as the lesser of

$$\phi R = 8(0.9L_{c2}tF_u) = 8[0.9(2.0)(1.0)(65)] = 936 \text{ kips}$$

or

$$\phi R = 8(1.8dtF_u) = 8[1.8(0.875)(1.0)(65)] = 819 \text{ kips (controls)}$$

$$\phi R_{TOTAL} = 234.0 + 819 = 1{,}053 \text{ kips}$$

$$\frac{P_{ncu}}{\phi R_{TOTAL}} = \frac{600}{1{,}053} = 0.57 < 1.0 \text{ OK}$$

Bottom Flange

Try: 75.5 x 0.375 in outer plate
$A_g = 28.3 \text{ in}^2$

Try: 2 - 36.75 x 0.375 in inner plates
$A_g = 27.6 \text{ in}^2$

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Splice Plates (continued)

Note: Since the inner splice plate must be partially split to accommodate the longitudinal flange stiffener (Figure E6), it will conservatively be treated as two separate plates in the subsequent calculations although this is physically not the case.

The minimum flange design force, P_{cu}, was computed earlier to be 1,869 kips (tension) (page 137). The factored-up moment for strength due to the St. Venant torsional shear was computed earlier (page 139) to be 465.3 k-in. Warping torsion in the bottom flange is not considered at the strength limit state for reasons discussed previously.

The effective areas of the inner and outer splice plates are computed as:

$$A_e = W_n t + \beta A_g \leq A_g$$

$$\text{Outer: } A_e = [75.5 - 20(0.875 + 0.125)](0.375) + 0.15(28.3) = 25.06 \text{ in}^2$$

$$\text{Inner: } A_e = 2[36.75 - 10(0.875 + 0.125)](0.375) + 0.15(27.6) = 24.20 \text{ in}^2$$

Since the flange is subjected to a net tension, the holes will be considered in computing a net section modulus for the splice plates. The holes remove the following percentage of cross-sectional area from each splice plate:

$$\text{Outer: } \frac{20(0.875 + 0.125)(0.375)}{75.5(0.375)} \times 100 = 26.5\%$$

$$\text{Inner: } \frac{10(0.875 + 0.125)(0.375)}{36.75(0.375)} \times 100 = 27.2\%$$

Only hole area in excess of 15 percent of the gross area of the plates must be removed. Therefore, the fraction of hole area that must be deducted in determining the net section modulus is

$$\text{Outer: } \frac{26.5 - 15.0}{26.5} = 0.43$$

$$\Sigma Ad^2 = 2 \times 0.43(0.875 + 0.125)(0.375) \times (2.5^2 + 6.25^2 + 10^2 + 13.75^2 + 17.5^2 + 21.25^2 + 25^2 + 28.75^2 + 32.5^2 + 36.25^2) = 1{,}585 \text{ in}^4$$

$$\text{Inner: } \frac{27.2 - 15.0}{27.2} = 0.45$$

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Splice Plates (continued)

$$\Sigma Ad^2 = 2 \times 0.45(0.875 + 0.125)(0.375) \times (1.875^2 + 5.625^2 + 9.375^2 + 13.125^2 + 16.875^2) = 195.8 \text{ in}^4$$

The net section modulus of the inner and outer splice plates together is therefore equal to:

$$S_{net} = \frac{(1/12)(0.375)(75.5)^3 - 1{,}585}{(75.5/2)} + 2\left(\frac{[(1/12)(0.375)(36.75)^3 - 195.8}{(36.75/2)}\right) = 461.8 \text{ in}^3$$

The combined stress in the splice plates is equal to:

$$f = \frac{1{,}869}{(25.06 + 24.20)} + \frac{465.3}{461.8} = 38.95 \text{ ksi}$$

$$\frac{38.95}{50} = 0.78 < 1.00 \text{ OK}$$

If the combined area of the equivalent inner splice plates had not been within 10 percent of the area of the outside splice plate, the minimum design force and factored-up moment would be proportioned to the inner and outer plates accordingly.

Separate calculations similar to those illustrated previously (page 150) show that bearing of the bolts on the bottom flange is not critical.