NCHRP Project 12-38 **COPY NO. 1**

DESIGN EXAMPLE
HORIZONTALLY CURVED STEEL I GIRDER BRIDGE

Appendix E

Prepared for
National Cooperative Highway Research Program
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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 I Girder Bridge design example has been developed to demonstrate the applicability of the **Recommended Specifications for Steel Curved-Girder Bridges.** There were three alternate designs for the web panels studied; unstiffened, transversely stiffened and longitudinally and transversely stiffened webs. 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 draft specifications based on the present AASHTO LFD specifications will be incorporated into the AASHTO Load and Resistance Factor Design (LRFD) specifications in the future. An extensive theoretical and experimental research program is being conducted on curved-girder bridges under sponsorship of the Federal Highway Administration (FHWA). This program should permit further improvements in the present curved-girder specifications.

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I. Objectives

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

2. Compare the critical stresses in the design by the Recommended Specifications to the stresses in a similar design by the Guide Specifications for Design of Horizontally Curved Highway Bridges (hereafter referred to as the Guide Spec).

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 similar positive dead load moments in the end and center spans.

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

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

Structural steel having a specified minimum yield stress of 50 ksi is used throughout. The deck is conventional cast-in-place concrete with a specified minimum 28-day compressive strength of 4,000 psi. A future wearing surface of 30 psf is specified.

Bridge underclearance is limited such that the total bridge depth may not exceed 120 inches at the low point on the cross section. 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 subjected to a temperature range from -40 degrees to 120 degrees Fahrenheit. The bridge is designed for a 75-year fatigue life.

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

Steel erection is examined, including the need for temporary supports. Sequential placement of the concrete deck is also considered. Permanent steel deck forms are assumed to be used between girders; the forms are assumed to weigh 15 psf.

III. Steel Framing

Proper layout of the steel framing is an important part of the design process. Five different framing plans considering different girder depths, cross frame spacings and with and without lateral flange bracing are examined.

A. Girder Spacing

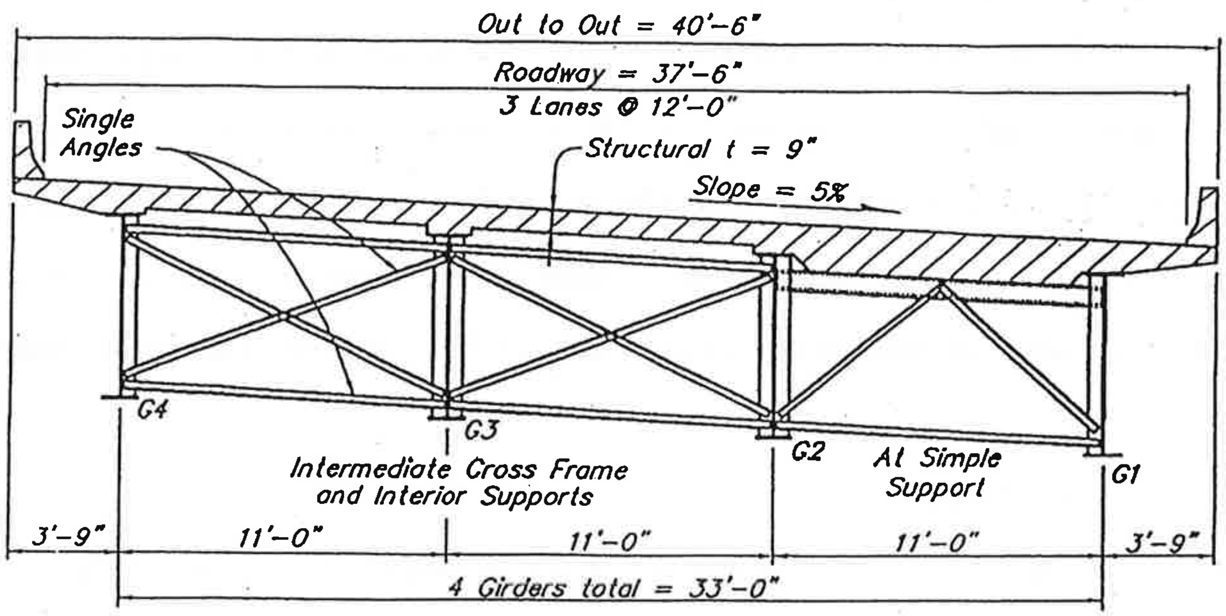
The four I girders are spaced at 11 feet with 3.75-foot deck overhangs. Reducing the girder spacing below 11 feet would lead to an increase in the size of the deck overhangs which would, in turn, lead to larger loading on the exterior girders, particularly the girder on the outside of the curve. A wider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

B. 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, G4, is 214.95 feet which is the girder with the longest effective span in this example. Therefore, the recommended girder depth is computed as follows:

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



I GIRDER CROSS SECTION

Deck Concrete--f'_c = 4000 psi E = 3.6×10^6 psi
Haunch--20 Inches wide, 4 inches deep from top of web.
Permanent deck forms
Total thickness--9.5 inches; Structural thickness = 9.0 inches.

Figure 1 I Girder Bridge Cross Section

A web depth of 84 inches is used.

C. Minimum Plate Sizes

A minimum thickness of one inch for the flange plates is arbitrarily chosen to minimize distortion due to welding. Article 9.1 recommends a minimum compression flange thickness of 1.5 times the web thickness. The unstiffened web (see below) is 0.875 inches thick. Therefore, Article 9.1 recommends a minimum compression flange thickness of 1.5 x 0.875" = 1.3 inches. When the top flange in positive-moment regions is partially braced, it is designed as a non-compact section and the recommended minimum compression flange thickness is infringed upon. The top flange is rigidified by the deck when the full stress is applied to the section. Therefore, in this example, a minimum flange thickness of one inch was deemed to be acceptable for the top flange in these regions. The bottom flanges meet the recommended thickness requirement in negative-moment regions. Article 9.1 specifies that the flange width not be less than 0.15 times the web depth (84 x 0.15 = 12.6 inches). The recommended minimum width is 0.2 times the web depth (84 x 0.2 = 18.8 inches). The minimum flange width is set at 15 inches.

Three options are investigated for the web design; an unstiffened web, a transversely stiffened web, and a longitudinally and transversely stiffened web (see Section D.3, page 22).

Article 6.4 limits the thickness of longitudinally stiffened webs to D/300. A 7/16-inch web is used throughout the girder for this option (84"/0.4375" = 192 < 300). Although a thinner web could have been used, it would have been difficult to fabricate and to maintain ANSI/AASHTO/AWS flatness requirements without costly straightening. If a thinner web had been used, more than one longitudinal stiffener would have been required in many locations.

The unstiffened web design has a 0.875-inch thick web throughout. Article 6.2 limits the slenderness of unstiffened webs with radii not greater than 700 feet to 100. The slenderness is 84"/0.875" = 96 < 100.

The slenderness of transversely stiffened webs is limited to 150 in Article 6.3. A 0.5625-in thick web is used in positive-moment regions of the transversely stiffened web design (84"/0.5625" = 149 < 150). The web thickness is increased to 0.625 inches in the field sections over the interior piers.

D. Cross Frames

The recommended cross frame spacing is 21 feet according to Equation (C9-1) in Article 9.3.2. Reduction of the cross frame spacing reduces cross frame forces since the load transferred between girders is a function of the curvature, and therefore is nearly constant. Reduction of cross frame spacing also reduces lateral flange bending moments and transverse deck stresses. By reducing lateral flange bending, flange sizes can be reduced, but at the expense of more cross frames. For the preliminary design, a constant cross frame spacing of approximately 16 feet was investigated. The final design uses a spacing of approximately 20 feet.

Cross frames are composed of single angles with an area of 5 square inches. Cross frames with an "X" configuration with top and bottom chords are used because they generally require the least labor to fabricate. If the girder spacing and or depth is large, a "K" configuration may be desired to reduce forces in the diagonals.

E. Field Section Sizes

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

IV. Framing for Final Design

A. Alternative Framing Schemes

Although not required by the Recommended Specifications, five alternative framing schemes were examined in this example in the preliminary design. All girders were assumed to have a constant depth. The cross frame spacing was approximately 16 feet in each of the five preliminary arrangements.

Design 1 - Standard - no lateral flange bracing; equal-depth girders.

Design 2 - Individual girder depths increased by six-inch increments from Girder 1 at 84 inches to Girder 4 at 102 inches.

Design 3 - Design 1 with single bottom flange lateral bracing in the exterior bays.

Design 4 - Design 1 with crossed bottom flange lateral bracing in exterior bays.

Design 5 - Design 1 with single top and bottom flange lateral bracing in exterior bays.

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

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

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

After comparing girder moments, shears, lateral flange moments, deflections, and cross frame forces, Design 1 was chosen to be carried to completion.

B. Cross Frames

The cross frame spacing is made nearly uniform over each span in the final design. The preliminary studies were made with 10 panels in the end spans and 14 panels in the center span creating a spacing of approximately 16 feet. A 16-foot spacing necessitates two intermediate transverse stiffeners in each panel to satisfy the minimum required stiffener spacing equal to the web depth, D. For this spacing the critical flange stress often was found to exceed the critical stress in the web. The cross frame spacing can be increased causing a reduction in the critical flange stress, thereby bringing it closer to the critical web stress, which is not affected by the cross frame spacing. This balancing of the critical web and flange stresses results in fewer cross frames without any increase in girder size. In the final design, there are 8 panels in the end spans and 11 panels in the center span. The number of intermediate transverse stiffeners per panel remains at two. Since the number of panels per girder is reduced to 27 from 34, the number of intermediate transverse stiffeners per girder is reduced by 14 ((34-27) x 2 = 14) or by 56 for the bridge. The number of cross frames is reduced by 21. The flange sizes are not increased since the critical web stress usually limits the design.

Figure 2 shows the final framing plan. The node numbering for the three-dimensional finite element model is also shown in this figure. These node numbers will be referred to frequently in the following narratives, tables and sample calculations.

C. Field Sections

The final girder field sections for the transversely stiffened girder design are given in Appendix A for all the girders. The longest field section, the center field section in Girder 4, is approximately 137 feet in length. Field section profiles for the transversely stiffened girder design are given in Appendix H.

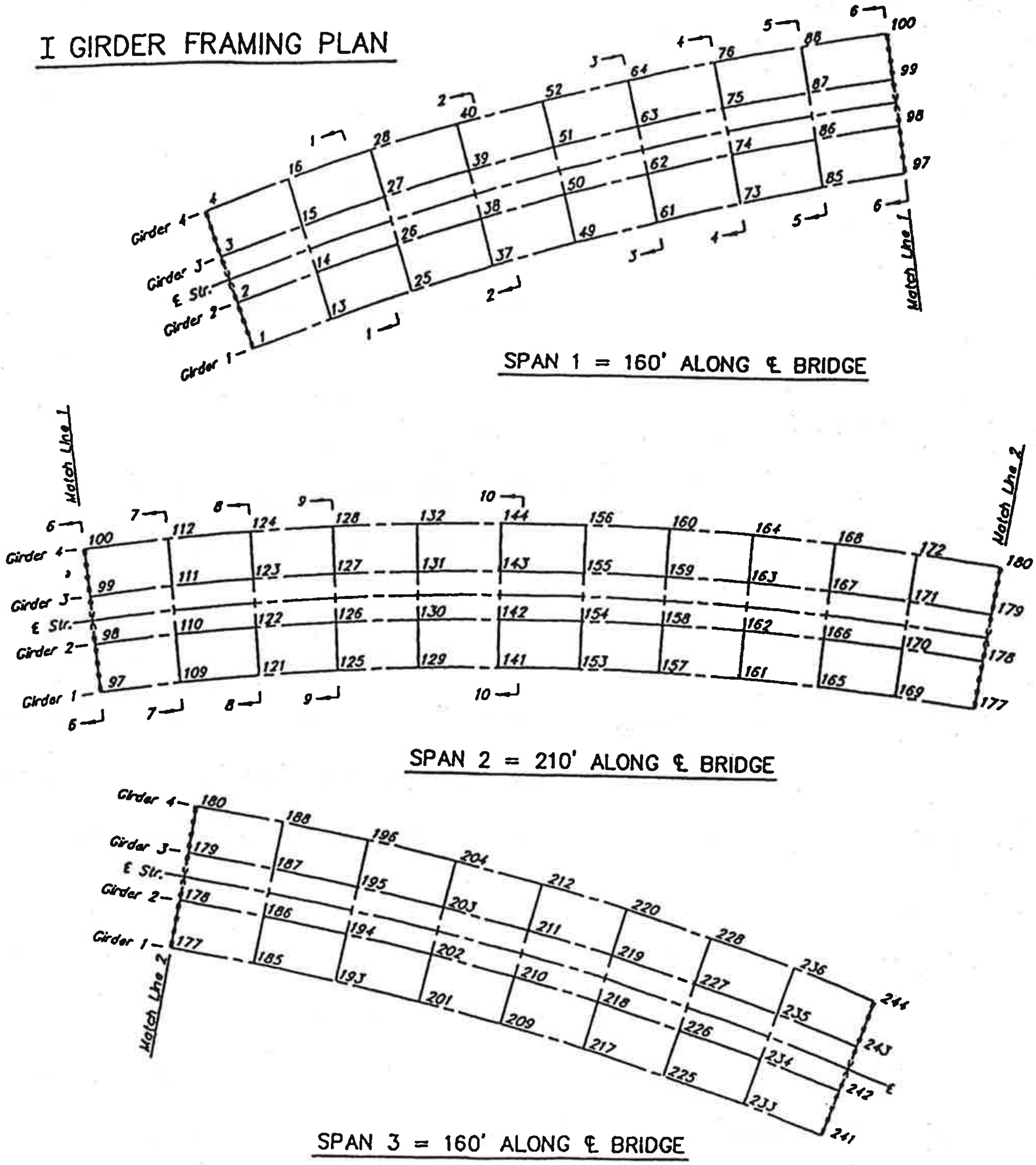


Figure 2 Framing Plan and Nodal Numbering

V. Final Design

A. Loads

1. Non-Composite Dead Load

The steel weight is applied as body forces to the non-composite structure in the analysis.

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

2. Constructibility

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

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

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

3. Superimposed Dead Load

The parapet loads are applied along the edges of the deck in the analysis. The future wearing surface is applied uniformly over the deck area. These superimposed dead loads are applied to the fully composite structure in the analysis.

4. Live Load

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

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

B. Analyses

1. 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. However, **Load Groups III, IV, V** and **VI** from **Table 3.22.1A** are also checked for temperature and wind loadings in combination with vertical loading.

The following load combinations and load factors are checked in this design example. Selected analysis results for these load groups are given in Tables D4 through D10, Appendix D. These results include the factored axial force in the cross-frame diagonal members between nodes 99 and 100 and between nodes 97 and 98, and the factored vertical, tangential and radial bearing reactions at node 98. In some design instances, other load cases may be critical, but for this example, these other load cases are assumed not to apply.

From **AASHTO Table 3.22.1A**:

Group I	1.3[D + 5/3(L + I) + CF]
Group II	1.3[D + W]
Group III	1.3[D + (L + I) + CF + .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 + .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 $\quad 1.4[D + C + W^*]$

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

2. 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 are modeled with shell elements. Flanges are modeled with beam elements. Curvature is represented by straight elements with small kinks at node points rather than by curved elements.

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

Bearings are represented by dimensionless elements called "foundation elements," which attach from a lower girder node to the "earth." For the thermal analyses and certain other analyses, proper lateral bearing restraints are specified for the foundation elements.

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

3. Comparison of Analyses

For the example bridge, a series of comparative analyses were made between the results from the three-dimensional finite element analysis; and the results from a two-dimensional grid analysis and a one-dimensional V-load analysis. These analysis results are given in Tables C1 through C5, Appendix C.

As can be seen from the comparisons shown in these tables, there exists a close correlation in the dead load analysis results among all three methods. Fairly close correlation exists between the finite element analysis and the grid analysis results for live load. However, the V-load method gives significantly different vertical bending moments in the live load analysis. The discrepancy between the V-load and finite element results is up to 70%. Much of this discrepancy is most probably due to the wheel load distribution factors (**AASHTO Article 3.23**) used to determine the primary vertical bending moments in the V-load analysis. This discrepancy in the live load analysis results will likely be improved if more accurate wheel load distribution factors are used in the V-load analysis. Although the results are not shown, the use of the wheel load distribution factors contained in the AASHTO Guide Specifications for Distribution of Loads for Highway Bridges (1994) appears to improve the correlation in this case.

C. Limit States

1. Strength

Live load responses for HS25 plus impact are generated for the Strength limit state. One, two and three traffic lanes are considered. Multiple presence reduction factors specified in **AASHTO Article 3.12** are applied. Centrifugal force effects are included. The impact factors specified in Article 3.5.6.1 for I girders are used. The deck is considered as placed at once on the non-composite steel.

2. Constructibility

The erection sequence is investigated to check both deflections and stress according to Article 13. Sequential deck placement is investigated to check deflections, Article 13.4, stress, Article 13.2, and concrete crack control, Articles 13.3 and 2.4.3. The effect of the concrete on the overhang brackets is considered according to Article 13.8.

3. Permanent Deflection

Live load responses for overload (Article 3.5.4) are created for multiple lanes of HS20 loading plus impact placed in the critical position for each girder. Both the lane loading and the truck 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**. Impact for overload is defined in Article 3.5.6.1 for I girders. The provisions of Article 9.5 are used to check the overload stress limits for control of permanent set and the applicable web and flange critical stresses to ensure stability. Overload stresses caused by loads acting on the composite section are to be determined using the appropriate uncracked transformed composite section according to Article 9.5.

4. Fatigue

The range of stress for fatigue is determined by computing the maximum and minimum stress due to one fatigue truck, defined in Article 3.5.7.1, 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 is held to marked traffic lanes. The fatigue truck is permitted to travel within two feet of the curb line. Article 4.5.2 specifies that the uncracked composite section is to be used to compute fatigue stresses.

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 9.6.2 requires that lateral bending stresses also be included when computing the stress ranges in the flanges. Lateral bending does not contribute to the stress range at the web-to-flange weld. However, if the connection plates receiving cross frames are welded to a tension flange, lateral bending contributes to the longitudinal flange stress range at the end of that weld and should be considered.

Cross frame members are fillet welded to gusset plates, which are bolted to the connection plates in this example. The base metal adjacent to the fillet welds at the end of the cross frame members must be checked for fatigue Category E. The stress range in these members is computed according to Article 3.5.7.2, which requires that the stress range be determined as the larger of either 75 percent of the stress range computed by positioning the factored fatigue truck in two different transverse positions or the stress range due to a single passage of the factored fatigue truck. The use of two transverse positions of the truck is synonymous with assuming that the stress range is determined by the separate passage of two trucks rather than one. The use of two passages of the truck is moderated by using 75 percent of the stress range to account for the reduced probability of two trucks being at their critical position at the same time.

5. 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 greatly at any one cross section.

Table 1 gives the preferred maximum live load deflections for the center span of each girder according to Article 12.4.

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

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

Computed maximum girder deflections in the center span due to the service live load plus impact (HS20 loading) are given in Table 2 and are based on the use of the uncracked composite section along the entire length of the bridge in the analysis. When multiple lanes are loaded to produce a deflection value given in Table 2, the multiple presence reduction factors specified in **AASHTO Article 3.12.1** are applied.

If a sidewalk were present, vehicular traffic would be constrained from a portion of the deck, which would cause the computed live load deflections to be reduced for either G1 or G4, depending on which side of the bridge the sidewalk was placed. Sidewalk load is discussed further in Article 3.5.5.

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

Girder	Lane Loading	Truck Loading
G1	1.97	1.49
G2	1.28	0.96
G3	2.08	1.50
G4	3.24	2.35

D. Design

1. Section Properties

As specifed in Article 4.5.2, composite properties are computed under the assumption that the entire deck area between girders is effective. A constant haunch height of 4 inches from the top of the web to the bottom of the deck is assumed. However, the concrete in the haunch is ignored in the computation of the section properties.

Concrete creep under dead load is accounted for by dividing the deck width by three times the normal modular ratio. The reinforcing steel is also adjusted for creep of the concrete by dividing its area by 3 since the concrete is assumed to transfer the force from the deck steel to the rest of the cross section. In the negative moment regions, an area of 8 square inches per girder is assumed for the longitudinal reinforcement. The neutral axis of the reinforcing is assumed to be 4 inches from the bottom of the deck. The cracked section is assumed for loads applied to the composite section at the strength limit state. Longitudinal deck reinforcing is considered to be effective for negative moment only.

Table D11, Appendix D, gives selected section properties for G4 for all three web designs. Locations from the neutral axis to the top (T) and bottom (B) extreme fiber of the steel section are given, as well as the depth of web in compression, D_c. These values are used in the selected sample calculations that follow in Appendix E.

2. Flanges

The size of curved I girder flanges is a function of girder depth, girder radius, cross frame spacing, and minimum specified yield stress of the flange. Article 5.2.2 defines a non-compact flange width-to-thickness ratio limit such that the tip stress in a partially braced compression flange may reach the yield stress prior to the onset of local buckling. The same non-compact width-to-thickness ratio limit, Equation (5-7), is also applied to partially braced tension flanges in Article 5.3. The vertical bending stress in partially braced non-compact compression flanges is limited by the smaller of the critical average flange stress from Equations (5-8) and (5-9).

Article 5.2.1 defines a compact flange width-to-thickness ratio limit such that a partially braced compression flange may undergo yielding prior to the onset of local buckling. The vertical bending stress in partially braced compact compression flanges and tension flanges is limited by the smaller of the critical average flange stress from Equations (5-4) and (5-6).

The Recommended Specifications do not require that a check of the flange strength be made at locations where the plate widths change between brace points. The smaller flange plate must be used to compute the strength of a partially braced flange between brace points when the flange size changes within a panel. The largest vertical bending stress at either brace point should be used in conjunction with the lateral flange bending stress at the more critical brace point and the smallest flange size within the panel to compute the critical flange stress (Article 5.1).

For the constructibility limit state, Article 13.2 requires that non-composite top flanges in compression be designed as non-compact flanges prior to hardening of the concrete to ensure that no yielding occurs, which tends to lead to the use of wider flanges. Lateral bending in top flanges is not considered after the deck has hardened for any limit state.

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

3. Webs

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

Table 3 Comparison of Lateral Flange Moments from 3D Analysis and Equation (4-1)

Loading	Lat. Mom. (k-ft) Top Flange		Lat. Mom. (k-ft) Bottom Flange	
	3D	Eq. (4-1)	3D	Eq. (4-1)
Steel	-4	-6	4	6
Deck	-16	-23	16	23
Supim DL	--	--	6	10
Total DL	-20	-29	26	39
Single truck	--	--	7	
Multiple truck	--	--	13	
Multiple lane	--	--	14	33
Total	-20	-29	40	72

Node 40 Near mid-span 1 Girder 4

Table 4 Comparison of Lateral Flange Moments from 3D Analysis and Equation (4-1)

Loading	Lat. Mom. (k-ft) Top Flange		Lat. Mom. (k-ft) Bottom Flange	
	3D	Eq. (4-1)	3D	Eq. (4-1)
Steel	8	16	-7	-16
Deck	35	60	-30	-60
SupimDL	--	--	-7	-25
Total DL	43	76	-44	-101
Single truck	--	--	-5	
Multiple truck	--	--	-9	
Multiple lane	--	--	-18	-51
Total	43	76	-62	-152

Node 100 Interior support Girder 4

Transversely stiffened webs without longitudinal stiffeners may have a slenderness, D/t_w, up to 150. This requirement differs from the Guide Spec Load Factor Design requirement, which permits a slenderness, $2D_c/t_w$, equal to $36{,}500/\sqrt{F_y}$ times a reduction factor that is a function of the girder radius and the transverse stiffener spacing.

In this example, the maximum allowable spacing of transverse stiffeners equals the web depth of 7 feet (Article 6.3). By limiting the maximum cross frame spacing to approximately 20 feet, only two intermediate transverse stiffeners per panel are required.

Although the final field section profiles given in Appendix H are for the transversely stiffened web design only, selected calculations are given in Appendix E for the unstiffened web design and for the longitudinally and transversely stiffened web design to show application of the Recommended Specifications to these web types.

4. Shear Connectors

The required pitch of the shear connectors is determined for fatigue and checked for strength. Three 7/8" x 6" shear studs per row are assumed in the design. The Recommended Specifications use the fatigue strength from **AASHTO LRFD Article 6.10.7.4.2** for the design of the shear connectors. The First Edition of **AASHTO LRFD** incorrectly gave the fatigue limit of a shear stud as $5.5d^2$ pounds per stud. The correct value is $(5.5/2)d^2$. This value was corrected in the 1996 Interims to AASHTO LRFD. The corrected value is also used in the Recommended Specifications and in this example.

The design longitudinal shear range in each stud is computed for a single passage of the factored fatigue truck. The analysis is made assuming that the heavy wheel of the truck is applied to both the positive and negative shear sides of the influence surfaces.

This computation tacitly assumes that the truck direction is reversed. In addition to vertical bending shear, Article 7.2.2 requires that the radial shear due to curvature or radial shear due to causes other than curvature (whichever is larger) be added vectorially to the bending shear for the fatigue check. The deck in the regions between points of dead load contraflexure is considered fully effective in computing the first moment for determining the required pitch for fatigue. This assumption requires tighter shear connector spacing in these regions than if only the longitudinal reinforcing is assumed effective, as is often done. There are several reasons the concrete is assumed effective. First, known field measurements indicate that it is effective at service loads. Second, the horizontal shear force in the deck is considered effective in the analysis and the deck must be sufficiently connected to the steel girders to be consistent with this assumption. Third, maximum shear range occurs when the truck is placed on each side of the point under consideration. Most often this produces positive bending so that the deck is in compression, even when the location is between the point of dead load contraflexure and the pier. The point of dead load contraflexure is obviously a poor indicator of positive or negative bending when moving loads are considered.

The strength check for shear connectors requires that a radial shear force due to curvature be considered. The deck strength in the negative-moment region is given as $0.45f_c^{\prime}$ in Article 7.2.1. This value is a conservative approximation to account for the combined contribution of both the longitudinal reinforcing steel and the concrete that remains effective in tension based on its modulus of rupture.

For both fatigue and strength checks, the effective width of deck is considered to be either the overhang plus half the distance to the adjacent girder for exterior girders, or the girder spacing for interior girders.

5. Bearing Orientation

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

In this example, the bearings at the piers are assumed fixed against translation in both the radial and tangential directions. The bearings at the abutments are assumed fixed against radial movement but free in the tangential direction. The pier stiffness in the tangential directions is considered and is simulated in the analysis by using a spring with a spring constant smaller than infinity. In the radial directions, the piers and abutments are assumed perfectly rigid. However, for the wind and temperature analyses, only Girder G2 is restrained in the radial direction. This is done in the temperature analysis to ensure that expansion between bearings at each pier and abutment does not create very large radial forces, which would not exist in reality because of "slop" in the bearings. The same assumption is made for the wind analyses. This is a very conservative assumption and it permits a very conservative design of the cross frame with regard to wind. The actual bearings may be designed by dividing the wind force between 2, 3 or 4 bearings.

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

6. Details

In this example, intermediate transverse web stiffeners are assumed to be fillet welded to one side of the web and to the compression flange. Article 6.5 states that when single transverse stiffeners are used, they are preferably to be attached to both flanges. In this example, the intermediate stiffeners are assumed to also be fillet welded to the tension flange. The termination of the stiffener-to-web weld adjacent to the tension flange is stopped a distance of 4 times the web thickness from the flange-to-web weld. The base metal adjacent to the stiffener weld to the tension flange is checked for fatigue Category C' (refer to **AASHTO LRFD Table 6.6.1.2.3-1**). Where the stiffener is fillet welded to the compression flange and the flange undergoes a net tension, the flange must also be checked for the Category C'. When the girder is curved, the lateral flange bending creates an additional stress at the tip of the stiffener-to-flange weld away from the web. Thus, the total stress range is computed from the sum of the lateral and vertical bending stress ranges.

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

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

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

7. Erection

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

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

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

8. Wind

a. Loading

Article 3.4 requires that wind intensities be taken from **AASHTO Article 3.15**. However, Article 3.4 requires that wind application be unidirectional rather than perpendicular to the bridge as specified in **AASHTO Article 3.15**, which assumes a bridge with girders parallel to a single plane. The wind force on a curved bridge, therefore, equals the wind intensity times the projected area of the bridge. Thus, the total force on the curved bridge is less than that computed if the wind is assumed to be applied along the arc length. According to the Recommended Specifications, the wind force must also be applied in various directions to determine the maximum force in the various elements of the structure. In the design example, the wind load is applied with respect to global axes. This requires that the force be separated into X and Y components, which are applied at nodes. Since there are nodes at the top and bottom of the girder, it is possible to divide the wind force between the top and bottom flange. The tributary area for the top of the windward girder equals half of the girder depth plus the height of the exposed deck and parapet concrete times the average spacing to each adjacent node. The tributary area for the bottom of the girder is simply half of the girder depth times the average spacing to each adjacent node.

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

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

b. Analysis

The completed bridge has an exposed height of approximately 10.5 feet. The design wind intensity of 50 psf results in a total wind force of 550 pounds per foot applied to the projected length of bridge. Load Group III includes wind on the live load in conjunction with wind on the structure. The wind on the live load is specified as 100 pounds per linear foot. For this load group, the wind on the structure is factored by 0.3 (0.3 x 550 = 165 pounds per linear foot). Thus, the net wind load is 265 pounds per foot.

If the wind load is applied such that all girders are exposed, such as across the end of the first phase of the erection process, wind load is applied to each girder. Since this bridge is superelevated 5 percent, the exposed height of bridge must be considered. If wind is coming from the outside of the curve, additional wind load must be applied to the bottom of the girders away from the windward side. If the wind is applied from the inside of the curve, additional wind load is applied to the opposite parapet. The girder spacing of 11 feet causes the elevation of adjacent girders to differ by 0.65 feet. Additional wind load is applied to the bottoms (0.65 feet x 100 = 65 pounds per linear foot of projected length) of each interior girder when the wind is applied from the Girder 4 side (outside of the curve). Girder 4 is the highest girder. The width of the bridge is approximately 40 feet, so the parapet on the outside of the curve is approximately 2 feet higher than on the inside of the curve. Thus, the parapet receives additional wind load on its projected area when the wind is applied from the inside of the curve.

A three-dimensional finite element analysis was made assuming that the wind load was applied from the Girder 4 side at 248 degrees clockwise from north. The first abutment is oriented north and south. This angle is orthogonal to a chord drawn between the abutments. The wind was applied to the outside of the curve at an angle which caused the largest possible total wind force. The majority of the wind force was applied to Girder 4. Another analysis applied the wind in the opposite direction (248 -180 = 68 degrees). Superelevation exposes the upper portion of the bridge so an additional wind force (2 feet times 50 pounds per square foot = 100 pounds per linear foot) was applied to the parapet on the outside of the curve in the analysis. Results from this analysis produced the largest cross bracing forces for the assumed bearing arrangement.

c. Construction

In addition to the AASHTO load groups, Article 2.5 of the Recommended Specifications requires that each critical phase of construction be examined. A load factor of 1.4 is used for this limit state.

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

The wind analysis for the construction condition was made assuming the wind to be acting perpendicular to the bridge at the first abutment. An additional wind load of 50 psf x 0.65 ft was applied to the top flange of Girder G2 to account for the superelevation. An additional wind load was also applied to the top and bottom flanges at the end of Girder G2 to account for the projection of Girder G2 three feet beyond Girder G1 in the X direction. The results from this analysis are discussed further in Appendix G.

9. Deck Staging

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

The unfactored moments from the deck staging analysis for the transversely stiffened girder design are presented in Table 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. Included in the “Deck” moments are the moments due to the deck haunch and the stay-in-place forms; “Cast” identifies the moments due to a particular deck cast; "SupImp" identifies the moments due to the superimposed dead load placed on the fully composite bridge. Reactions are accumulated sequentially in the analysis so that uplift can be checked at each stage. Accumulated deflections by stage are also computed. In each analysis of the deck placement, prior casts are assumed to be composite. The modular ratio for the deck is assumed to be 3n to account for creep. A somewhat smaller modular ratio may be desirable for the staging analyses since full creep usually takes approximately three years to occur. A modular ratio of n is used to check some of the deck stresses, as specified in Article 13.3.

E. Sample Calculations

Sample calculations at selected critical sections of the exterior girder, G4, are presented in Appendix E. Calculations are illustrated for all three web designs. 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 illustrate stiffener designs, a bolted field splice design, a cross-frame diagonal design and centrifugal force calculations. The calculations make use of the moments and shears contained in Tables D1 through D3 of Appendix D and the section properties contained in Table D11. The same moments and shears are used for all three designs in the sample calculations for simplicity and since the cross-sectional stiffnesses do not vary significantly in the three designs.

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APPENDIX A

Girder Field Sections

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                                    October  4, 1995     10:56 AM

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

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

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

BRIDGE-SYSTEMsm   3D Version ->  2.1

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Bridge Software Development International, Ltd.
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Girder --> 1 Field Section --> 1

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

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

Girder --> 1 Field Section --> 2

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

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

Girder --> 1 Field Section --> 3

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

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

Girder --> 1 Field Section --> 4

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

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

Girder --> 1 Field Section --> 5

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
49	197	9.76	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
50	201	9.76	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
51	205	9.76	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
52	209	9.76	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
53	213	9.76	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
54	217	9.76	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
55	221	9.76	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
56	225	9.76	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
57	229	9.76	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
58	233	9.76	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
59	237	9.76	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
60	241	9.76	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.

Sup ---> 156.23

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

Girder --> 2 Field Section --> 1

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
61	6	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
62	10	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
63	14	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
64	18	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
65	22	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
66	26	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
67	30	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
68	34	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
69	38	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
70	42	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
71	46	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
72	50	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
73	54	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
74	58	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
75	62	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
76	66	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
77	70	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.
78	74	6.62	15.00	1.0000	50.	16.00	1.0000	50.	84.00	.5625	50.

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

Girder --> 2 Field Section --> 2

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
79	78	6.62	18.00	1.2500	50.	19.00	1.5000	50.	84.00	.6250	50.
80	82	6.62	18.00	1.2500	50.	19.00	1.5000	50.	84.00	.6250	50.
81	86	6.62	18.00	1.2500	50.	19.00	1.5000	50.	84.00	.6250	50.
82	90	6.61	18.00	2.5000	50.	19.00	3.0000	50.	84.00	.6250	50.
83	94	6.61	18.00	2.5000	50.	19.00	3.0000	50.	84.00	.6250	50.
84	98	6.56	18.00	2.5000	50.	19.00	3.0000	50.	84.00	.6250	50.
Sup	--->	158.74									
85	102	6.31	18.00	2.5000	50.	19.00	3.0000	50.	84.00	.6250	50.
86	106	6.31	18.00	2.5000	50.	19.00	3.0000	50.	84.00	.6250	50.
87	110	6.31	18.00	2.5000	50.	19.00	3.0000	50.	84.00	.6250	50.
88	114	6.31	18.00	1.2500	50.	19.00	1.5000	50.	84.00	.6250	50.
89	118	6.31	18.00	1.2500	50.	19.00	1.5000	50.	84.00	.6250	50.
90	122	6.31	18.00	1.2500	50.	19.00	1.5000	50.	84.00	.6250	50.

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

Girder --> 2 Field Section --> 3

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
91	126	18.94	15.00	1.0000	50.	17.00	1.0000	50.	84.00	.5625	50.
92	130	18.94	15.00	1.0000	50.	17.00	1.0000	50.	84.00	.5625	50.
93	134	6.31	15.00	1.0000	50.	17.00	1.0000	50.	84.00	.5625	50.
94	138	6.31	15.00	1.0000	50.	17.00	1.0000	50.	84.00	.5625	50.
95	142	6.31	15.00	1.0000	50.	17.00	1.0000	50.	84.00	.5625	50.
96	146	6.31	15.00	1.0000	50.	17.00	1.0000	50.	84.00	.5625	50.
97	150	6.31	15.00	1.0000	50.	17.00	1.0000	50.	84.00	.5625	50.
98	154	6.31	15.00	1.0000	50.	17.00	1.0000	50.	84.00	.5625	50.
99	158	18.94	15.00	1.0000	50.	17.00	1.0000	50.	84.00	.5625	50.
100	162	18.94	15.00	1.0000	50.	17.00	1.0000	50.	84.00	.5625	50.
101	166	18.94	15.00	1.0000	50.	17.00	1.0000	50.	84.00	.5625	50.

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

Girder --> 2 Field Section --> 4

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

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

Girder --> 2 Field Section --> 5

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

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

Girder --> 3 Field Section --> 1

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
121	7	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
122	11	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
123	15	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
124	19	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
125	23	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
126	27	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
127	31	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
128	35	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
129	39	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
130	43	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
131	47	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
132	51	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
133	55	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
134	59	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
135	63	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
136	67	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
137	71	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
138	75	6.72	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.

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

Girder --> 3 Field Section --> 2

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
139	79	6.72	20.00	1.2500	50.	21.00	1.5000	50.	84.00	.6250	50.
140	83	6.72	20.00	1.2500	50.	21.00	1.5000	50.	84.00	.6250	50.
141	87	6.72	20.00	1.2500	50.	21.00	1.5000	50.	84.00	.6250	50.
142	91	6.71	20.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
143	95	6.71	20.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
144	99	6.66	20.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
Sup	--->	161.26									
145	103	6.41	20.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
146	107	6.41	20.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
147	111	6.41	20.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
148	115	6.41	20.00	1.2500	50.	21.00	1.5000	50.	84.00	.6250	50.
149	119	6.41	20.00	1.2500	50.	21.00	1.5000	50.	84.00	.6250	50.
150	123	6.41	20.00	1.2500	50.	21.00	1.5000	50.	84.00	.6250	50.

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

Girder --> 3 Field Section --> 3

	Rght		-----Top Flange----			---Bottom Flange--			---- Web ------		
Mem.	Node	Length	Width	Thick.	Fy	Width	Thick.	Fy	Depth	Thick.	Fy
151	127	19.24	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
152	131	19.24	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
153	135	6.41	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
154	139	6.41	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
155	143	6.41	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
156	147	6.41	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
157	151	6.41	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
158	155	6.41	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
159	159	19.24	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
160	163	19.24	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
161	167	19.24	15.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.

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

Girder --> 3 Field Section --> 4

	Rght		-----Top Flange----			---Bottom Flange--			---- Web ------		
Mem.	Node	Length	Width	Thick.	Fy	Width	Thick.	Fy	Depth	Thick.	Fy
162	171	19.24	20.00	1.2500	50.	21.00	1.5000	50.	84.00	.6250	50.
163	175	9.65	20.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
164	179	9.65	20.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
Sup	--->	211.65									
165	183	10.08	20.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
166	187	10.08	20.00	2.5000	50.	21.00	3.0000	50.	84.00	.6250	50.
167	191	10.08	20.00	1.2500	50.	21.00	1.5000	50.	84.00	.6250	50.
168	195	10.08	20.00	1.2500	50.	21.00	1.5000	50.	84.00	.6250	50.

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

Girder --> 3 Field Section --> 5

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
169	199	10.08	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
170	203	10.08	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
171	207	10.08	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
172	211	10.08	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
173	215	10.08	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
174	219	10.08	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
175	223	10.08	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
176	227	10.08	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
177	231	10.08	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
178	235	10.08	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
179	239	10.08	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.
180	243	10.08	16.00	1.0000	50.	18.00	1.0000	50.	84.00	.5625	50.

Sup ---> 161.26

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

Girder --> 4 Field Section --> 1

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
181	8	6.83	20.00	1.0000	50.	21.00	1.0000	50.	84.00	.5625	50.
182	12	6.83	20.00	1.0000	50.	21.00	1.0000	50.	84.00	.5625	50.
183	16	6.83	20.00	1.0000	50.	21.00	1.0000	50.	84.00	.5625	50.
184	20	6.83	20.00	1.0000	50.	21.00	1.0000	50.	84.00	.5625	50.
185	24	6.83	20.00	1.0000	50.	21.00	1.0000	50.	84.00	.5625	50.
186	28	6.83	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
187	32	6.83	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
188	36	6.83	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
189	40	6.83	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
190	44	6.83	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
191	48	6.83	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
192	52	6.83	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
193	56	6.83	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
194	60	6.83	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
195	64	6.83	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
196	68	6.83	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
197	72	6.83	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
198	76	6.83	20.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.

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

Girder --> 4 Field Section --> 2

Mem.	Rght Node	Length	Top Flange Width	Top Flange Thick.	Top Flange Fy	Bottom Flange Width	Bottom Flange Thick.	Bottom Flange Fy	Web Depth	Web Thick.	Web Fy
199	80	6.83	28.00	1.2500	50.	27.00	1.5000	50.	84.00	.6250	50.
200	84	6.83	28.00	1.2500	50.	27.00	1.5000	50.	84.00	.6250	50.
201	88	6.83	28.00	1.2500	50.	27.00	1.5000	50.	84.00	.6250	50.
202	92	6.82	28.00	2.5000	50.	27.00	3.0000	50.	84.00	.6250	50.
203	96	6.82	28.00	2.5000	50.	27.00	3.0000	50.	84.00	.6250	50.
204	100	6.77	28.00	2.5000	50.	27.00	3.0000	50.	84.00	.6250	50.
Sup --->		163.77									
205	104	6.51	28.00	2.5000	50.	27.00	3.0000	50.	84.00	.6250	50.
206	108	6.51	28.00	2.5000	50.	27.00	3.0000	50.	84.00	.6250	50.
207	112	6.51	28.00	2.5000	50.	27.00	3.0000	50.	84.00	.6250	50.
208	116	6.51	28.00	1.2500	50.	27.00	1.5000	50.	84.00	.6250	50.
209	120	6.51	28.00	1.2500	50.	27.00	1.5000	50.	84.00	.6250	50.
210	124	6.51	28.00	1.2500	50.	27.00	1.5000	50.	84.00	.6250	50.

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

Girder --> 4 Field Section --> 3

Mem.	Rght Node	Length	Top Flange Width	Top Flange Thick.	Top Flange Fy	Bottom Flange Width	Bottom Flange Thick.	Bottom Flange Fy	Web Depth	Web Thick.	Web Fy
211	128	19.54	17.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
212	132	19.54	17.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
213	136	6.51	17.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
214	140	6.51	17.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
215	144	6.51	17.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
216	148	6.51	17.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
217	152	6.51	17.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
218	156	6.51	17.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
219	160	19.54	17.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
220	164	19.54	17.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.
221	168	19.54	17.00	1.0000	50.	21.00	1.5000	50.	84.00	.5625	50.

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

Girder --> 4 Field Section --> 4

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

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

Girder --> 4 Field Section --> 5

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

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

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APPENDIX B

Girder Moments and Shears at Tenth Points

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                                        September 23, 1995      8:31 PM


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

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

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


BRIDGE-SYSTEMsm   3D Version ->  2.1

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Bridge Software Development International, Ltd.
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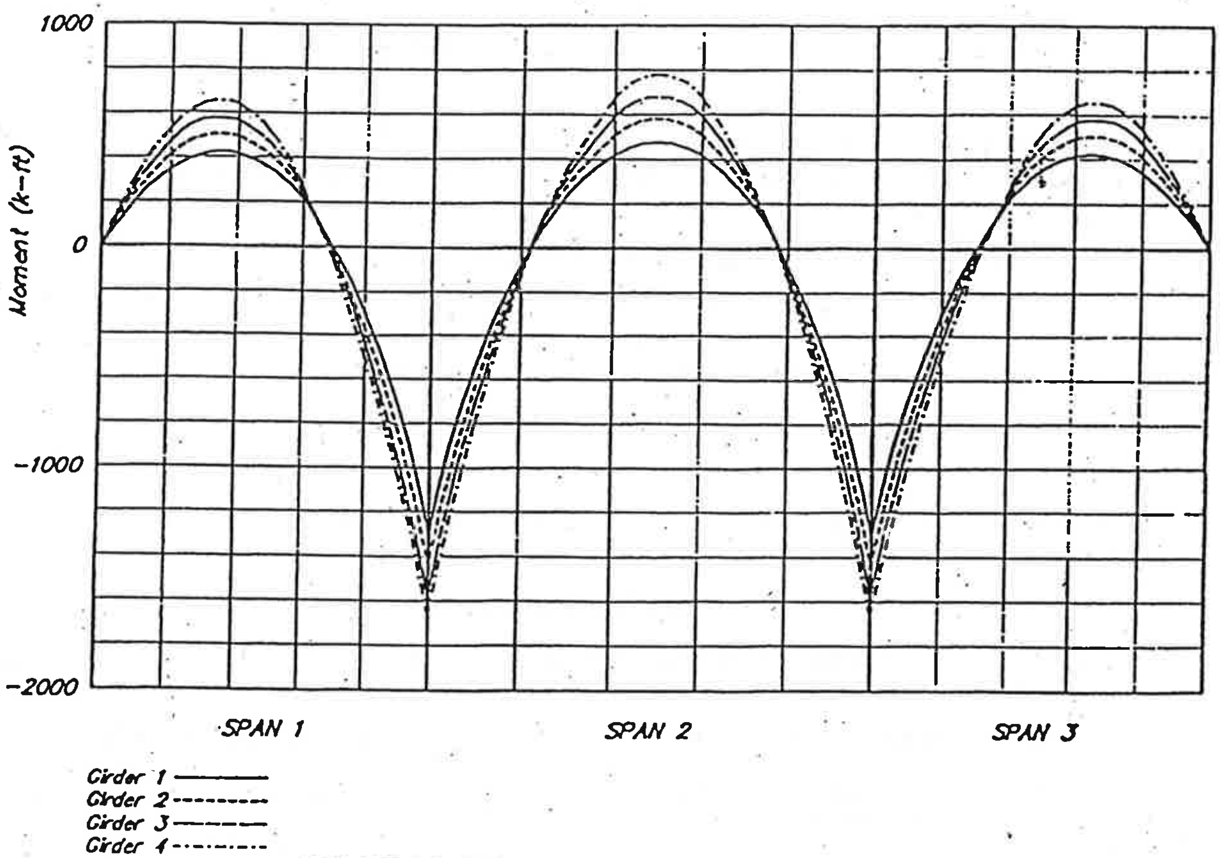
<u>Stage Definition</u>

Stg1	=	Load due to weight of structural steel including girders and cross frames
Stg6	=	Load due to weight of concrete deck placed at one time
Stg7	=	Load due to weight of parapets and wearing surface placed on composite bridge



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

Figure B1 Dead Load (Structural Steel) Moment

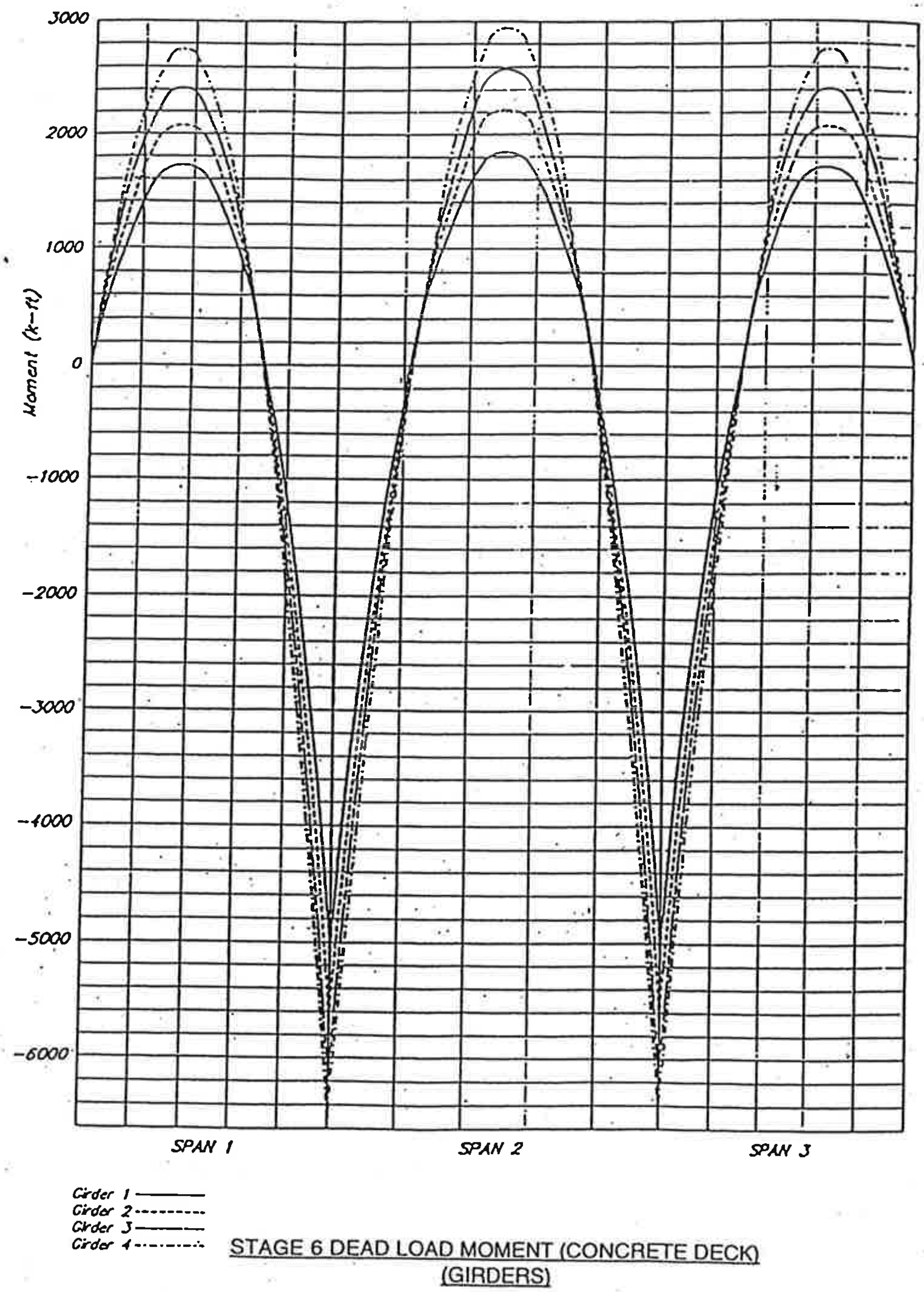
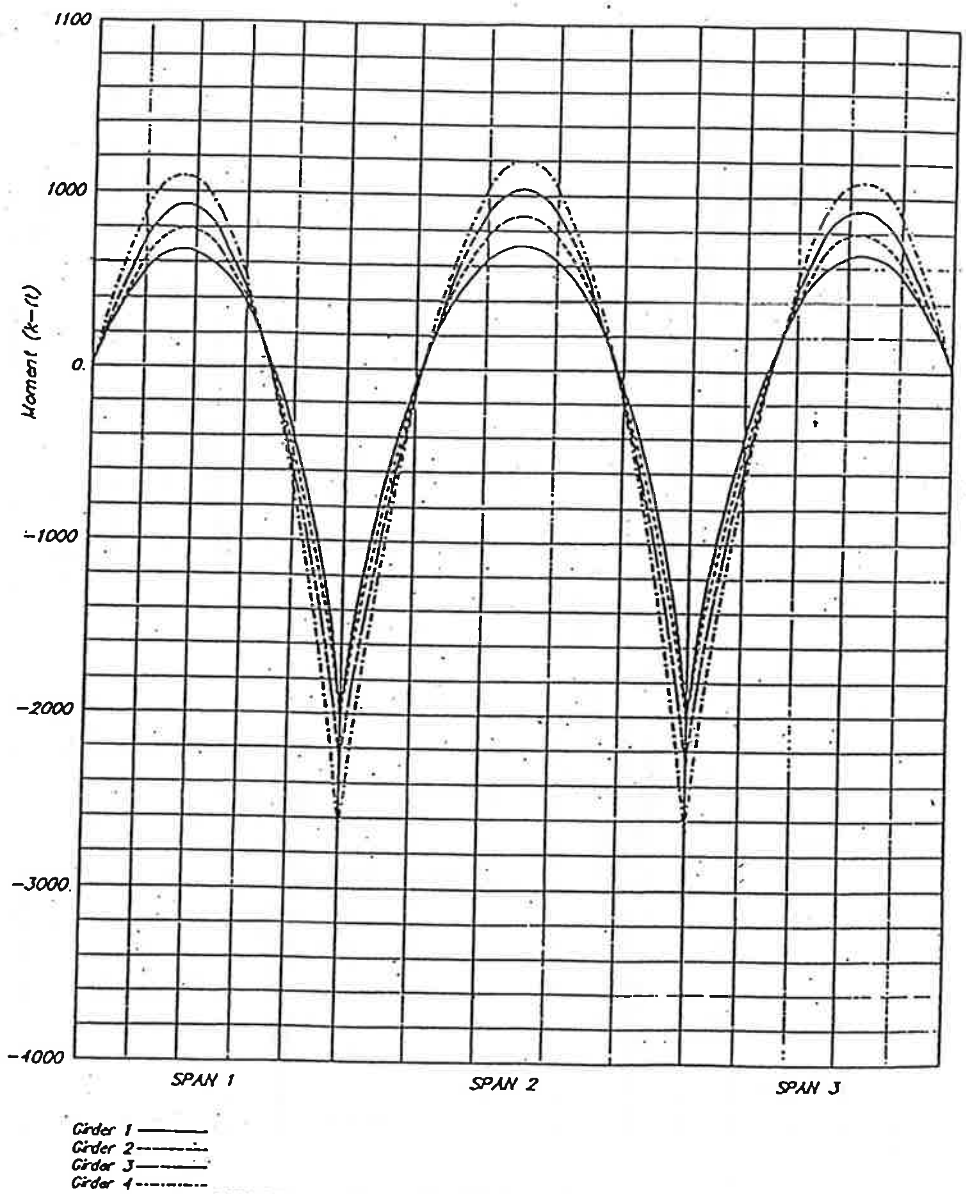
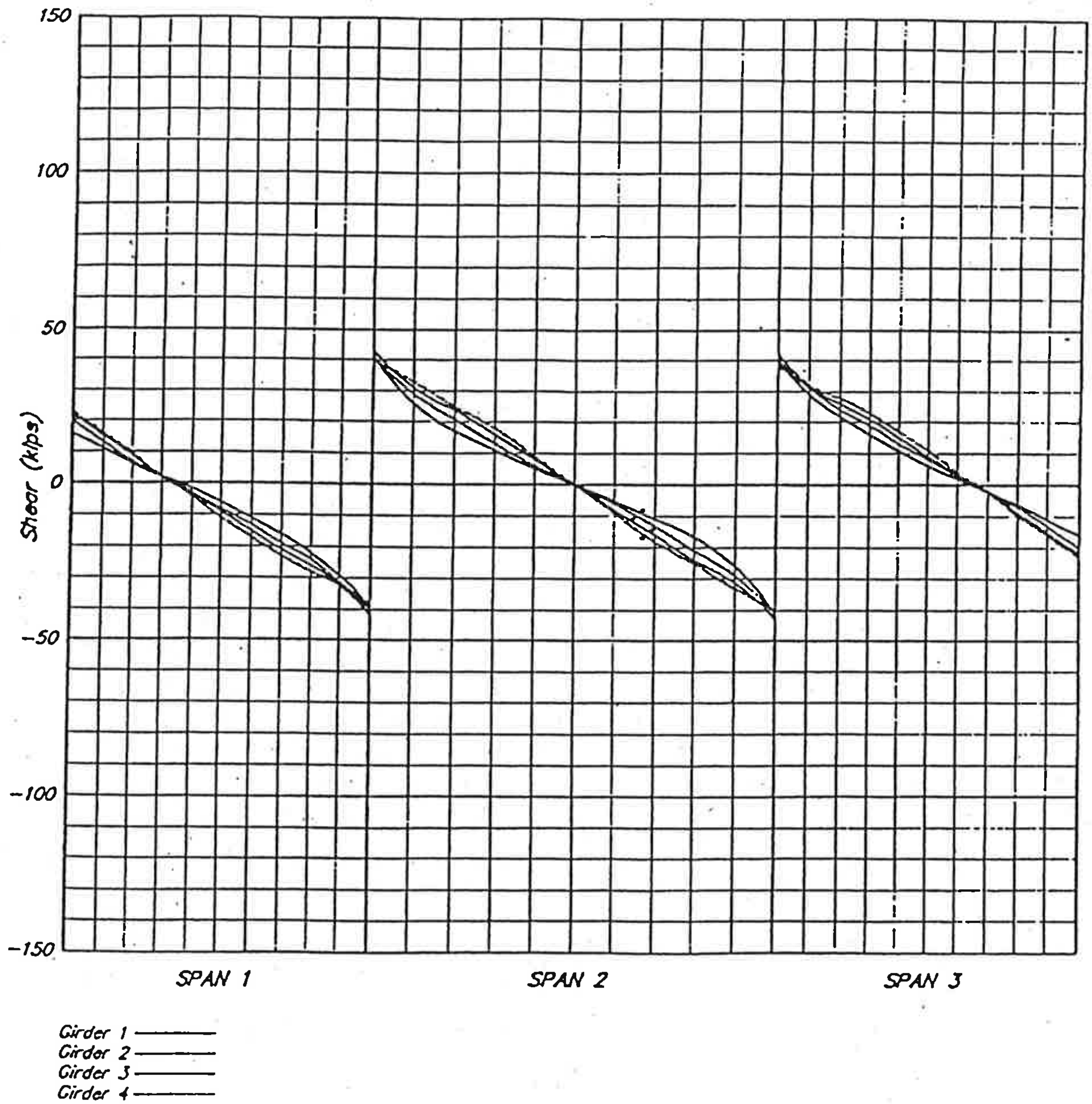


Figure B2 Dead Load (Concrete Deck) Moment



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

Figure B3 Dead Load (Superimposed Dead Load) Moment



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

Figure B4 Dead Load (Structural Steel) Shear

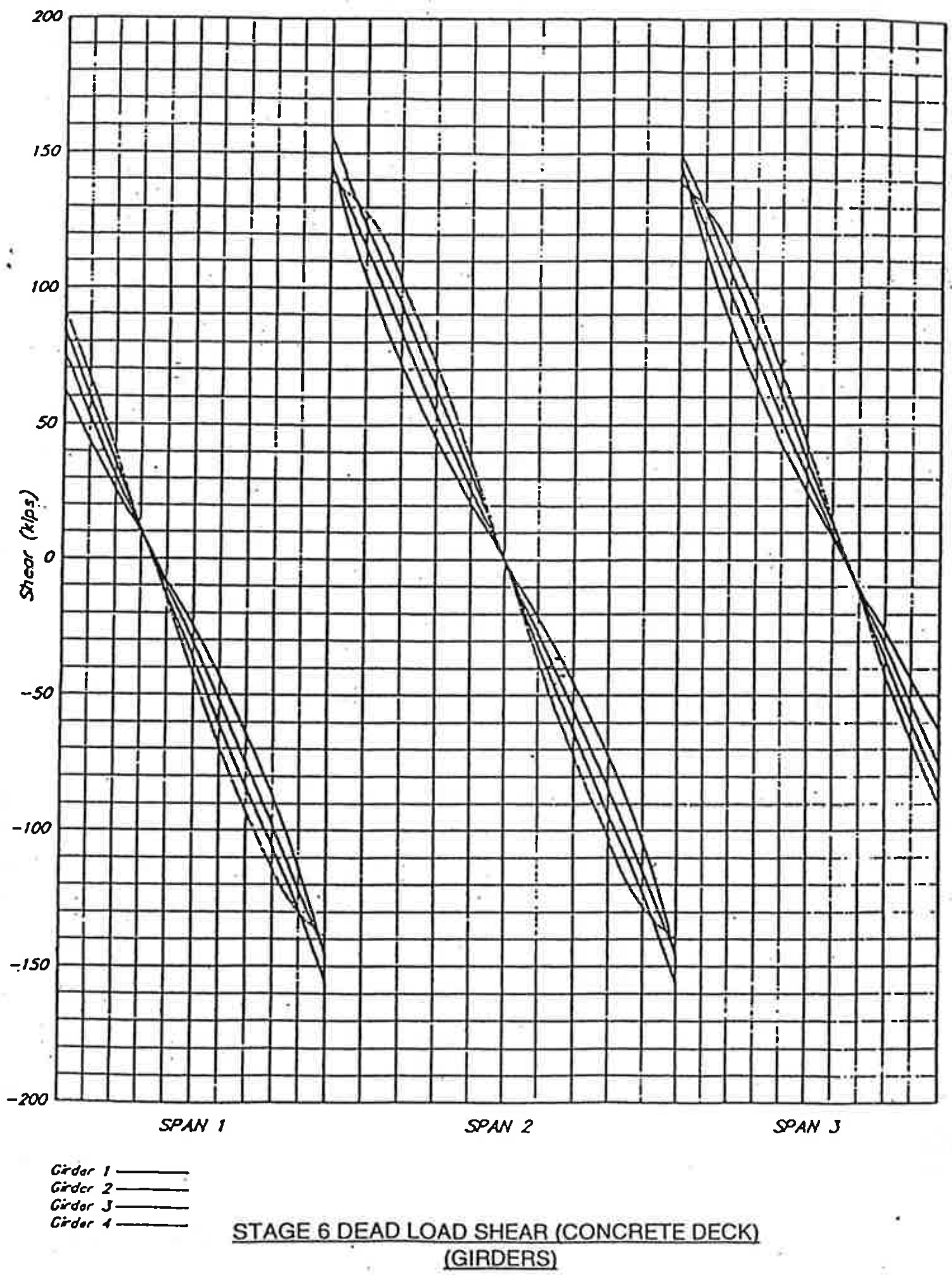


Figure B5 Dead Load (Concrete Deck) Shear

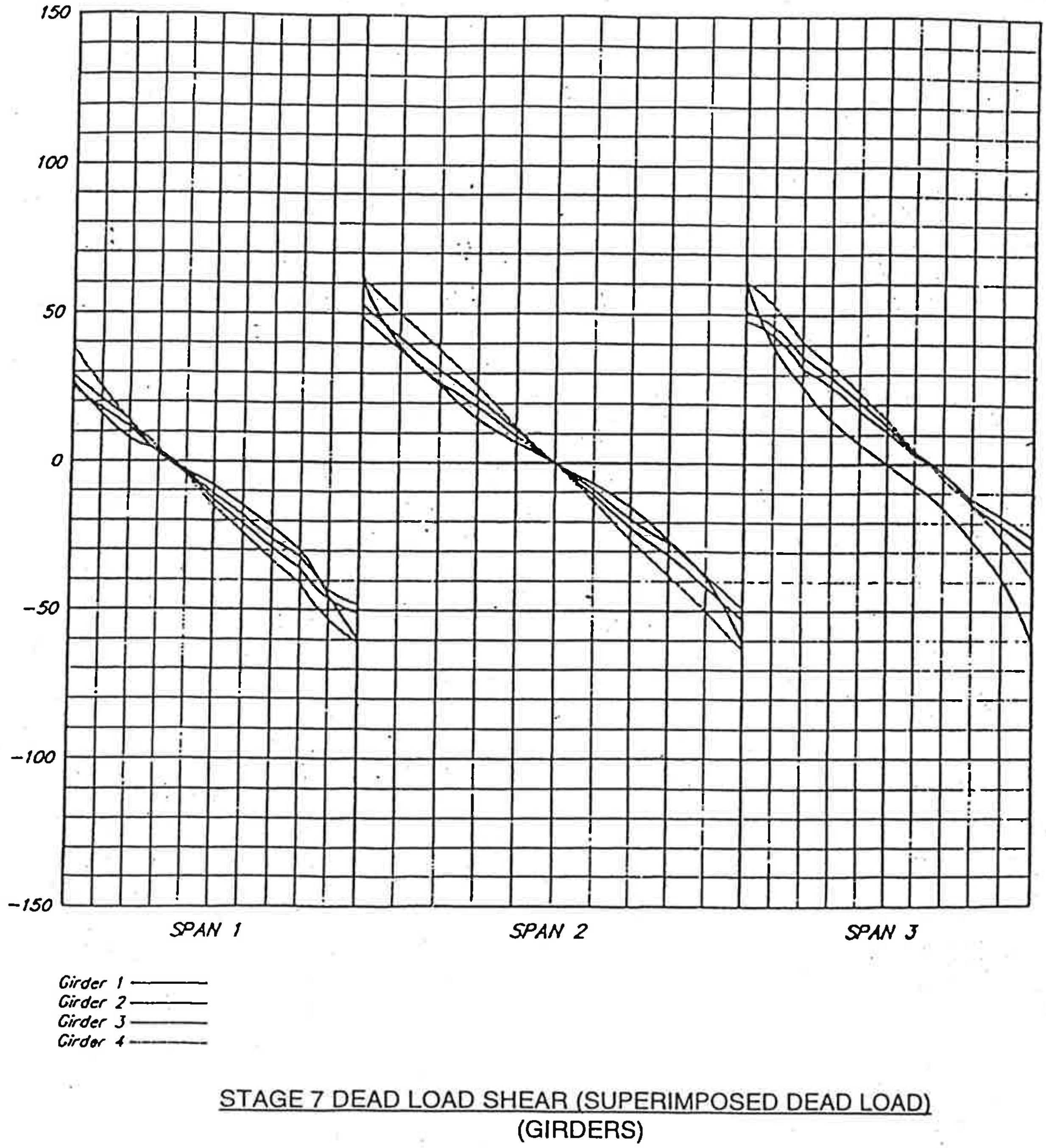


Figure B6 Dead Load (Superimposed Dead Load) Shear

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

DEAD LOADS

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

	LIVE LOAD MOMENTS				LIVE LOAD SHEARS			
	Lane		Truck		Lane		Truck	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	0	0	90	-25	85	-17
15.62	1024	-298	1069	-214	69	-19	68	-13
31.25	1773	-570	1775	-404	55	-26	57	-21
46.87	2250	-815	2158	-556	44	-33	46	-29
62.49	2471	-1030	2297	-673	34	-40	36	-37
69.74 MAX	2493	-1122	2298	-711				
78.11	2456	-1224	2219	-763	27	-50	29	-47
93.74	2235	-1407	1902	-951	22	-62	25	-55
109.36	1706	-1590	1502	-1082	21	-77	18	-65
124.98	1157	-1874	996	-1229	19	-93	13	-75
140.61	667	-2732	440	-1499	16	-111	11	-83
156.23	791	-4641	424	-1953	17	-135	9	-94

	LIVE LOAD MOMENTS				LIVE LOAD SHEARS			
	Special		Maximum		Special		Maximum	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	0	0	0	0	90	-25
15.62	0	0	1069	-298	0	0	69	-19
31.25	0	0	1775	-570	0	0	57	-26
46.87	0	0	2250	-815	0	0	46	-33
62.49	0	0	2471	-1030	0	0	36	-40
78.11	0	0	2456	-1224	0	0	29	-50
93.74	0	0	2235	-1407	0	0	25	-62
109.36	0	0	1706	-1590	0	0	21	-77
124.98	0	0	1157	-1874	0	0	19	-93
140.61	0	0	667	-2732	0	0	16	-111
156.23	0	0	791	-4641	0	0	17	-135

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

D E A D L O A D S

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

		---- LIVE LOAD MOMENTS ----				---- LIVE LOAD SHEARS ----			
		--- Lane ---		-- Truck --		--- Lane ---		-- Truck --	
Length		POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00		791	-4641	424	-1953	134	-17	94	-9
20.50		664	-2304	531	-1212	109	-22	79	-15
41.01		1228	-1408	1258	-1015	89	-29	70	-23
61.51		1986	-1280	1792	-891	71	-29	59	-24
82.02		2535	-1313	2182	-755	56	-33	47	-29
102.52		2731	-1325	2287	-727	44	-45	38	-40
103.52	MAX	2731	-1325	2288	-727				
123.03		2558	-1315	2193	-754	36	-57	31	-48
143.53		2008	-1301	1810	-909	31	-70	26	-58
164.04		1242	-1450	1262	-1045	29	-88	21	-68
184.54		692	-2256	584	-1198	21	-104	16	-76
205.05		760	-4482	429	-1906	17	-137	9	-99

	---- LIVE LOAD MOMENTS ----				---- LIVE LOAD SHEARS ----			
	-- Special--		-- Maximum -		-- Special -		-- Maximum -	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	791	-4641	0	0	134	-17
20.50	0	0	664	-2304	0	0	109	-22
41.01	0	0	1258	-1408	0	0	89	-29
61.51	0	0	1986	-1280	0	0	71	-29
82.02	0	0	2535	-1313	0	0	56	-33
102.52	0	0	2731	-1325	0	0	44	-45
123.03	0	0	2558	-1315	0	0	36	-57
143.53	0	0	2008	-1301	0	0	31	-70
164.04	0	0	1262	-1450	0	0	29	-88
184.54	0	0	692	-2256	0	0	21	-104
205.05	0	0	760	-4482	0	0	17	-137

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

DEAD LOADS

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

	---- LIVE LOAD MOMENTS ----				---- LIVE LOAD SHEARS ----			
	--- Lane ---		-- Truck --		--- Lane ---		-- Truck --	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	760	-4482	429	-1906	139	-17	101	-9
15.62	665	-2676	454	-1478	108	-18	82	-11
31.25	1146	-1851	999	-1215	94	-18	75	-16
46.87	1693	-1570	1498	-1070	79	-20	67	-21
62.49	2221	-1389	1898	-942	65	-23	58	-26
78.11	2449	-1210	2227	-753	53	-28	49	-31
86.49 MAX	2489	-1110	2311	-702				
93.74	2469	-1018	2309	-665	43	-35	41	-38
109.36	2249	-805	2167	-548	34	-44	32	-47
124.98	1783	-566	1793	-399	28	-54	24	-56
140.61	1042	-300	1095	-214	25	-68	18	-68
156.23	0	0	0	0	24	-90	17	-85

	---- LIVE LOAD MOMENTS ----				---- LIVE LOAD SHEARS ----			
	-- Special--		-- Maximum -		-- Special -		-- Maximum -	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	760	-4482	0	0	139	-17
15.62	0	0	665	-2676	0	0	108	-18
31.25	0	0	1146	-1851	0	0	94	-18
46.87	0	0	1693	-1570	0	0	79	-21
62.49	0	0	2221	-1389	0	0	65	-26
78.11	0	0	2449	-1210	0	0	53	-31
93.74	0	0	2469	-1018	0	0	43	-38
109.36	0	0	2249	-805	0	0	34	-47
124.98	0	0	1793	-566	0	0	28	-56
140.61	0	0	1095	-300	0	0	25	-68
156.23	0	0	0	0	0	0	24	-90

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

DEAD LOADS

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

		LIVE LOAD MOMENTS				LIVE LOAD SHEARS			
		Lane		Truck		Lane		Truck	
Length		POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00		0	0	0	0	77	-10	83	-5
15.87		882	-161	946	-115	58	-14	61	-7
31.75		1507	-325	1542	-233	45	-21	51	-17
47.62		1893	-493	1862	-355	36	-29	42	-27
63.50		2069	-669	1980	-480	28	-36	34	-36
70.85	MAX	2086	-753	1974	-541				
79.37		2054	-854	1923	-614	22	-44	28	-43
95.25		1867	-1056	1719	-758	16	-54	23	-50
111.12		1439	-1278	1362	-920	10	-65	15	-58
126.99		966	-1599	900	-1094	5	-78	7	-66
142.87		512	-2341	425	-1294	2	-95	1	-74
158.74		448	-3826	381	-1604	6	-113	5	-89

	LIVE LOAD MOMENTS				LIVE LOAD SHEARS			
	Special		Maximum		Special		Maximum	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	0	0	0	0	83	-10
15.87	0	0	946	-161	0	0	61	-14
31.75	0	0	1542	-325	0	0	51	-21
47.62	0	0	1893	-493	0	0	42	-29
63.50	0	0	2069	-669	0	0	34	-36
79.37	0	0	2054	-854	0	0	28	-44
95.25	0	0	1867	-1056	0	0	23	-54
111.12	0	0	1439	-1278	0	0	15	-65
126.99	0	0	966	-1599	0	0	7	-78
142.87	0	0	512	-2341	0	0	2	-95
158.74	0	0	448	-3826	0	0	6	-113

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

D E A D L O A D S

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

Length		LIVE LOAD MOMENTS Lane POS	Lane NEG	Truck POS	Truck NEG	LIVE LOAD SHEARS Lane POS	Lane NEG	Truck POS	Truck NEG
.00		448	-3826	381	-1604	112	-6	87	-5
20.83		502	-1949	458	-992	91	-9	68	-8
41.67		957	-1084	1064	-790	73	-16	60	-16
62.50		1531	-805	1548	-622	60	-23	53	-21
83.34		1958	-752	1829	-463	47	-29	45	-28
104.17	MAX	2101	-738	1928	-347	37	-37	36	-37
125.01		1965	-753	1836	-466	29	-47	28	-44
145.84		1527	-802	1548	-620	23	-58	22	-51
166.68		958	-1107	1059	-798	16	-73	15	-59
187.51		526	-1913	495	-974	12	-86	9	-66
208.35		444	-3711	378	-1581	6	-114	5	-94

Length	LIVE LOAD MOMENTS Special POS	Special NEG	Maximum POS	Maximum NEG	LIVE LOAD SHEARS Special POS	Special NEG	Maximum POS	Maximum NEG
.00	0	0	448	-3826	0	0	112	-6
20.83	0	0	502	-1949	0	0	91	-9
41.67	0	0	1064	-1084	0	0	73	-16
62.50	0	0	1548	-805	0	0	60	-23
83.34	0	0	1958	-752	0	0	47	-29
104.17	0	0	2101	-738	0	0	37	-37
125.01	0	0	1965	-753	0	0	29	-47
145.84	0	0	1548	-802	0	0	23	-58
166.68	0	0	1059	-1107	0	0	16	-73
187.51	0	0	526	-1913	0	0	12	-86
208.35	0	0	444	-3711	0	0	6	-114

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

DEAD LOADS

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

		LIVE LOAD MOMENTS				LIVE LOAD SHEARS			
		Lane		Truck		Lane		Truck	
Length		POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00		444	-3711	378	-1581	116	-6	97	-5
15.87		540	-2302	458	-1285	89	-4	74	-5
31.75		985	-1596	921	-1094	77	-8	66	-11
47.62		1450	-1282	1374	-922	66	-12	60	-17
63.50		1876	-1060	1729	-760	56	-17	53	-24
79.37		2063	-858	1934	-617	47	-23	46	-29
87.90	MAX	2094	-756	1984	-543				
95.25		2077	-670	1989	-482	38	-30	38	-36
111.12		1898	-495	1869	-356	30	-37	30	-43
126.99		1520	-326	1557	-235	24	-46	22	-52
142.87		902	-162	969	-117	19	-58	14	-62
158.74		0	0	0	0	10	-77	5	-84

	LIVE LOAD MOMENTS				LIVE LOAD SHEARS			
	Special		Maximum		Special		Maximum	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	444	-3711	0	0	116	-6
15.87	0	0	540	-2302	0	0	89	-5
31.75	0	0	985	-1596	0	0	77	-11
47.62	0	0	1450	-1282	0	0	66	-17
63.50	0	0	1876	-1060	0	0	56	-24
79.37	0	0	2063	-858	0	0	47	-29
95.25	0	0	2077	-670	0	0	38	-36
111.12	0	0	1898	-495	0	0	30	-43
126.99	0	0	1557	-326	0	0	24	-52
142.87	0	0	969	-162	0	0	19	-62
158.74	0	0	0	0	0	0	10	-84

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

D E A D L O A D S

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

Length		LIVE LOAD MOMENTS Lane POS	Lane NEG	Truck POS	Truck NEG	LIVE LOAD SHEARS Lane POS	Lane NEG	Truck POS	Truck NEG
.00		0	0	0	0	84	-15	83	-10
16.13		1035	-256	1031	-181	67	-19	65	-11
32.25		1752	-496	1690	-350	52	-25	52	-20
48.38		2188	-722	2049	-510	39	-31	43	-28
64.50		2396	-944	2185	-668	31	-38	36	-36
71.95	MAX	2419	-1047	2182	-741				
80.63		2389	-1166	2127	-825	24	-47	28	-43
96.75		2180	-1395	1897	-986	18	-57	22	-51
112.88		1729	-1633	1504	-1154	11	-68	14	-60
129.01		1187	-1954	991	-1330	6	-83	7	-69
145.13		654	-2703	465	-1530	3	-100	3	-77
161.26		596	-4298	503	-1807	8	-122	5	-90

Length	LIVE LOAD MOMENTS Special POS	Special NEG	Maximum POS	Maximum NEG	LIVE LOAD SHEARS Special POS	Special NEG	Maximum POS	Maximum NEG
.00	0	0	0	0	0	0	84	-15
16.13	0	0	1035	-256	0	0	67	-19
32.25	0	0	1752	-496	0	0	52	-25
48.38	0	0	2188	-722	0	0	43	-31
64.50	0	0	2396	-944	0	0	36	-38
80.63	0	0	2389	-1166	0	0	28	-47
96.75	0	0	2180	-1395	0	0	22	-57
112.88	0	0	1729	-1633	0	0	14	-68
129.01	0	0	1187	-1954	0	0	7	-83
145.13	0	0	654	-2703	0	0	3	-100
161.26	0	0	596	-4298	0	0	8	-122

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

D E A D L O A D S

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

	---- LIVE LOAD MOMENTS ----				---- LIVE LOAD SHEARS ----			
	--- Lane ---		-- Truck --		--- Lane ---		-- Truck --	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	596	-4298	503	-1807	120	-8	89	-5
21.16	570	-2230	467	-1116	97	-13	70	-10
42.33	1042	-1252	1120	-929	78	-20	62	-17
63.49	1671	-1002	1656	-762	64	-23	55	-20
84.66	2208	-1002	1987	-590	50	-30	46	-26
105.82 MAX	2369	-1008	2106	-466	39	-39	36	-38
126.99	2215	-1004	1995	-593	30	-50	28	-46
148.15	1671	-1003	1656	-762	24	-62	21	-53
169.32	1043	-1272	1120	-942	19	-78	16	-61
190.48	593	-2175	491	-1099	15	-92	10	-68
211.65	586	-4159	494	-1772	8	-122	6	-95

	---- LIVE LOAD MOMENTS ----				---- LIVE LOAD SHEARS ----			
	-- Special--		-- Maximum -		-- Special -		-- Maximum -	
Length	POS	NEG	POS	NEG	POS	NEG	POS	NEG
.00	0	0	596	-4298	0	0	120	-8
21.16	0	0	570	-2230	0	0	97	-13
42.33	0	0	1120	-1252	0	0	78	-20
63.49	0	0	1671	-1002	0	0	64	-23
84.66	0	0	2208	-1002	0	0	50	-30
105.82	0	0	2369	-1008	0	0	39	-39
126.99	0	0	2215	-1004	0	0	30	-50
148.15	0	0	1671	-1003	0	0	24	-62
169.32	0	0	1120	-1272	0	0	19	-78
190.48	0	0	593	-2175	0	0	15	-92
211.65	0	0	586	-4159	0	0	8	-122

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

DEAD LOADS

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

Length		LIVE LOAD MOMENTS Lane POS	Lane NEG	Truck POS	Truck NEG	LIVE LOAD SHEARS Lane POS	Lane NEG	Truck POS	Truck NEG
.00		586	-4159	494	-1772	125	-8	97	-6
16.13		675	-2649	480	-1511	93	-5	76	-5
32.25		1199	-1941	1004	-1324	82	-9	69	-9
48.38		1733	-1628	1514	-1151	71	-14	62	-16
64.50		2183	-1391	1904	-984	59	-19	54	-23
80.63		2393	-1165	2134	-824	49	-24	47	-29
89.30	MAX	2423	-1044	2190	-739				
96.75		2398	-941	2192	-666	41	-32	38	-37
112.88		2182	-717	2047	-508	34	-40	30	-44
129.01		1750	-490	1693	-347	28	-51	22	-52
145.13		1046	-256	1051	-181	23	-67	16	-65
161.26		0	0	0	0	15	-84	10	-83

Length	LIVE LOAD MOMENTS Special POS	Special NEG	Maximum POS	Maximum NEG	LIVE LOAD SHEARS Special POS	Special NEG	Maximum POS	Maximum NEG
.00	0	0	586	-4159	0	0	125	-8
16.13	0	0	675	-2649	0	0	93	-5
32.25	0	0	1199	-1941	0	0	82	-9
48.38	0	0	1733	-1628	0	0	71	-16
64.50	0	0	2183	-1391	0	0	59	-23
80.63	0	0	2393	-1165	0	0	49	-29
96.75	0	0	2398	-941	0	0	41	-37
112.88	0	0	2182	-717	0	0	34	-44
129.01	0	0	1750	-490	0	0	28	-52
145.13	0	0	1051	-256	0	0	23	-67
161.26	0	0	0	0	0	0	15	-84

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

DEAD LOADS

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

Length		LIVE LOAD MOMENTS Lane POS	Lane NEG	Truck POS	Truck NEG	LIVE LOAD SHEARS Lane POS	Lane NEG	Truck POS	Truck NEG
.00		0	0	0	0	116	-32	99	-19
16.38		1463	-439	1377	-291	94	-32	83	-20
32.75		2622	-867	2423	-588	78	-34	71	-21
49.13		3453	-1272	3104	-885	61	-41	58	-27
65.51		3861	-1665	3384	-1168	44	-52	44	-40
73.06	MAX	3897	-1855	3391	-1289				
81.89		3835	-2072	3303	-1424	31	-67	31	-55
98.26		3510	-2401	2923	-1664	21	-80	21	-67
114.64		2684	-2690	2235	-1888	16	-95	13	-79
131.02		1824	-3043	1399	-2077	15	-111	8	-90
147.39		1038	-3880	576	-2249	15	-125	7	-97
163.77		1190	-5954	698	-2513	17	-137	9	-102

Length	LIVE LOAD MOMENTS Special POS	Special NEG	Maximum POS	Maximum NEG	LIVE LOAD SHEARS Special POS	Special NEG	Maximum POS	Maximum NEG
.00	0	0	0	0	0	0	116	-32
16.38	0	0	1463	-439	0	0	94	-32
32.75	0	0	2622	-867	0	0	78	-34
49.13	0	0	3453	-1272	0	0	61	-41
65.51	0	0	3861	-1665	0	0	44	-52
81.89	0	0	3835	-2072	0	0	31	-67
98.26	0	0	3510	-2401	0	0	21	-80
114.64	0	0	2684	-2690	0	0	16	-95
131.02	0	0	1824	-3043	0	0	15	-111
147.39	0	0	1038	-3880	0	0	15	-125
163.77	0	0	1190	-5954	0	0	17	-137

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

D E A D L O A D S

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

Length		LIVE LOAD MOMENTS Lane POS	LIVE LOAD MOMENTS Lane NEG	LIVE LOAD MOMENTS Truck POS	LIVE LOAD MOMENTS Truck NEG	LIVE LOAD SHEARS Lane POS	LIVE LOAD SHEARS Lane NEG	LIVE LOAD SHEARS Truck POS	LIVE LOAD SHEARS Truck NEG
.00		1190	-5954	698	-2513	135	-17	99	-9
21.49		934	-3276	587	-1672	127	-26	88	-16
42.99		1610	-2008	1628	-1486	114	-27	88	-16
64.48		2662	-1771	2512	-1292	95	-29	76	-20
85.98		3640	-1867	3077	-1069	75	-36	62	-26
107.47		3924	-1903	3283	-878	56	-56	45	-46
108.47	MAX	3925	-1903	3284	-878				
128.97		3631	-1859	3098	-1050	42	-74	33	-60
150.46		2695	-1775	2563	-1295	32	-91	23	-74
171.96		1624	-2031	1632	-1516	26	-112	16	-84
193.45		956	-3169	659	-1644	26	-123	13	-91
214.95		1135	-5750	674	-2433	17	-133	9	-101

Length	LIVE LOAD MOMENTS Special POS	LIVE LOAD MOMENTS Special NEG	LIVE LOAD MOMENTS Maximum POS	LIVE LOAD MOMENTS Maximum NEG	LIVE LOAD SHEARS Special POS	LIVE LOAD SHEARS Special NEG	LIVE LOAD SHEARS Maximum POS	LIVE LOAD SHEARS Maximum NEG
.00	0	0	1190	-5954	0	0	135	-17
21.49	0	0	934	-3276	0	0	127	-26
42.99	0	0	1628	-2008	0	0	114	-27
64.48	0	0	2662	-1771	0	0	95	-29
85.98	0	0	3640	-1867	0	0	75	-36
107.47	0	0	3924	-1903	0	0	56	-56
128.97	0	0	3631	-1859	0	0	42	-74
150.46	0	0	2695	-1775	0	0	32	-91
171.96	0	0	1632	-2031	0	0	26	-112
193.45	0	0	956	-3169	0	0	26	-123
214.95	0	0	1135	-5750	0	0	17	-133

APPENDIX C

Comparison of Analyses

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A. Modeling for Grid Analysis

1. 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 a classical strength of materials assumption. 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 used in the analysis has six degrees of freedom at each node, three degrees of freedom were suppressed to simulate the classical grid analysis output.

2. 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 for 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 is required to approximate the girder curvature. The grid element, CBEND, used in the analysis is based on polar coordinates. As a result, there is no need to provide kinks, or many elements to simulate the curvature.

3. 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.

4. 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.

5. 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 I 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 loads (DL2) consisted of the future wearing surface load of 30 psf and the concrete parapets assumed to weight 530 plf. The parapet dead loads were resolved into equivalent vertical loads and the resulting torques were applied at the node points along the exterior girders.

6. Live Load

Wheel loads and equivalent lane loads 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 HS 25 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 to put the maximum wheel load over the particular girder under investigation. Impact factors used were those given in Article 3.5.6.1 of the Recommended Specifications.

B. Modeling for V-Load Analysis

1. General

The theory and application of the V-load method are best illustrated in "V-Load Analysis, Chapter 12, Highway Structures Design Handbook, Volume 1," available through the National Steel Bridge Alliance (NSBA). As demonstrated in the "Current Practice (NCHRP Project 12-38) Report" and in "V-Load Analysis", the dead load analysis is reasonably accurate. However, the accuracy of the live load analysis is seemingly affected by the accuracy of the wheel load distribution factors that are used. The V-load analysis presented herein was performed using VANCK (V-load Analysis and Check), a computer program also available from the NSBA. The V-load method is used by VANCK to compute vertical bending moments, shearing forces and lateral flange bending moments at each tenth point in each span for all girders.

2. Coordinates

VANCK computes coordinates for each tenth point in each span for all girders based on benchmark input data.

3. Boundary Conditions

As the primary analysis in the V-load method is performed on individual straight girders with the same arc span lengths (developed lengths) as the individual curved girders, boundary conditions are the same as for the curved-girder bridge. Both ends of the bridge are simply supported and the girders are continuous over the interior supports.

4. Dead Load

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

5. Live Load

The live load distribution factors used in the V-load analysis were based on **AASHTO Article 3.23.2.3**. The distribution factor for the interior girders was S/5.5 = 11/5.5 = 2.0 wheels. S/(4+0.25S) = 11/(4+0.25x11) = 1.63 wheels was used for the exterior girders. As in the case of the grid analysis, trucks (or equivalent lane loads) were placed at the approximate locations to produce maximum positive moments and/or maximum negative moments. The direction of the truck was determined from the ordinates of the straight-girder influence lines. Impact factors used were those given in Article 3.5.6.1 of the Recommended Specifications.

Table C1 Dead Load (Structural Steel) Analysis Comparison

		Max Moment			Reaction	
		+ M, Side Span (k-ft)	-M, Interior Pier (k-ft)	+ M, Center Span (k-ft)	Abutment (k)	Interior Pier (k)
G1	FEM	351	1,333	384	14.0	81.0
	Grid	341	1,305	347	13.5	85.4
	V-load	390	1,349	404	15.8	79.5
G2	FEM	404	1,324	416	16.0	74.0
	Grid	368	1,204	357	14.6	68.0
	V-load	431	1,416	513	16.8	75.8
G3	FEM	478	1,517	461	18.0	84.0
	Grid	418	1,370	378	15.6	82.7
	V-load	495	1,601	605	19.5	77.0
G4	FEM	678	1,917	635	23.0	89.0
	Grid	691	1,742	587	24.1	87.6
	V-load	689	2,028	811	25.2	90.6

Reaction Sum (kips): FEM=399, Grid=391.5, V-load=400.2

Table C2 Dead Load (Concrete Deck) Analysis Comparison

		Max Moment			Reaction	
		+ M, Side Span (k-ft)	-M, Interior Pier (k-ft)	+ M, Center Span (k-ft)	Abutment (k)	Interior Pier (k)
G1	FEM	1,767.0	5,897.0	1,812.0	66.0	318.0
	Grid	1,720.7	5,962.0	1,617.6	68.1	341.6
	V-load	1,604.0	5,356.0	1,596.0	64.7	312.6
G2	FEM	1,875.0	5,605.0	1,879.0	71.0	278.0
	Grid	1,705.4	5,198.3	1,705.4	67.2	251.0
	V-load	1,840.0	5,422.0	1,812.0	70.2	292.0
G3	FEM	2,080.0	6,112.0	2,006.0	78.0	301.0
	Grid	1,795.7	5,646.9	1,636.9	67.6	304.0
	V-load	2,141.0	5,867.0	2,008.0	78.5	285.5
G4	FEM	2,775.0	7,272.0	2,658.0	92.0	286.0
	Grid	2,793.4	6,823.4	2,469.6	98.2	288.5
	V-load	2,569.0	6,539.0	2,367.0	90.7	292.6

Reaction Sum (kips): FEM=1,490, Grid=1,486.2, V-load=1,486.8

Table C3 Dead Load (Superimposed Dead Load) Analysis Comparison

		Max Moment			Reaction	
		+ M, Side Span (k-ft)	-M, Interior Pier (k-ft)	+ M, Center Span (k-ft)	Abutment (k)	Interior Pier (k)
G1	FEM	702.0	2,336.0	735.0	30.0	131.0
	Grid	560.0	2,106.0	525.0	26.5	125.4
	V-load	644.8	1,722.3	684.1	24.8	109.6
G2	FEM	711.0	2,003.0	743.0	22.0	91.0
	Grid	625.0	1,791.0	584.5	19.7	76.3
	V-load	790.2	1,866.2	895.8	28.7	110.5
G3	FEM	778.0	2,214.0	801.0	24.0	97.0
	Grid	701.0	2,161.0	635.8	24.6	103.5
	V-load	1,042.2	2,085.6	911.4	32.1	111.8
G4	FEM	1,128.0	3,015.0	1,183.0	42.0	129.0
	Grid	1,163.0	2,994.0	1,015.8	43.8	146.6
	V-load	1,032.9	2,320.5	1,183.1	35.4	113.2

Reaction Sum (kips): FEM=566.0, Grid=566.4, V-load=566.1

Table C4 Live Load (Truck) Analysis Comparison

		Max Moment			Reaction	
		+ M, Side Span (k-ft)	-M, Interior Pier (k-ft)	+ M, Center Span (k-ft)	Abutment (k)	Interior Pier (k)
G1	FEM	2,298.0	1,953.0	2,288.0	-	-
	Grid	2,052.0	1,945.0	1,963.7	-	-
	V-load	2,241.3	1,475.5	2,278.3	-	-
G2	FEM	1,980.0	1,604.0	1,928.0	-	-
	Grid	2,078.0	1,647.0	2,039.2	-	-
	V-load	2,904.9	1,822.7	2,984.6	-	-
G3	FEM	2,185.0	1,807.0	2,106.0	-	-
	Grid	2,249.0	1,918.0	2,188.0	-	-
	V-load	3,377.4	2,069.0	3,516.5	-	-
G4	FEM	3,391.0	2,513.0	3,284.0	-	-
	Grid	3,709.0	2,513.0	3,576.7	-	-
	V-load	3,269.3	1,918.2	3,479.4	-	-

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

Table C5 Live Load (Lane) Analysis Comparison

		Max Moment			Reaction	
		+ M, Side Span (k-ft)	-M, Interior Pier (k-ft)	+ M, Center Span (k-ft)	Abutment (k)	Interior Pier (k)
G1	FEM	2,493.0	4,641.0	2,731.0	-	-
	Grid	1,862.0	4,782.0	1,953.8	-	-
	V-load	2,303.1	3,504.1	2,500.1	-	-
G2	FEM	2,086.0	3,826.0	2,101.0	-	-
	Grid	2,050.0	4,039.0	2,096.1	-	-
	V-load	3,030.6	4,345.4	3,345.2	-	-
G3	FEM	2,419.0	4,298.0	2,369.0	-	-
	Grid	2,420.0	4,517.0	2,358.0	-	-
	V-load	3,681.1	4,862.7	4,086.0	-	-
G4	FEM	3,897.0	5,954.0	3,925.0	-	-
	Grid	4,296.0	5,736.0	4,090.0	-	-
	V-load	3,739.3	4,454.9	4,177.2	-	-

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

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Appendix D

Selected Design Forces and Girder 4 Section Properties

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Table D1 Girder 4 Selected Moments (k-ft)

Section Node	Steel	Deck	Cast(#)[1]	SupImp[2]	Ovrlod[3]	LLmax[4]	Fat_{min}[5]	Fat_{max}[5]
1-1 24	574	2,367	3,018(1)	968	2,210	2,763	-217	896
2-2 44	661	2,682	3,932(1) -3,035(2)	1,093	3,174	3,968	-431 V=-15k	1,177 V= 15k
3-3 64	213	802	2,554(1) -3,113(2)	381	2,720 -2,072	3,400 -2,590	-627 V=-26k	966 V=5k
4-4 76	-290	-1,282	1,023(1) -3,469(2)	-411	1,838 -2,338	2,297 -2,923	-726	634
5-5 88	-958	-3,921	-549(1) -3,685(2)	-1,457	970 -2,977	1,212 -3,721	-816	230
6-6 100	-1,917	-7,272	---	-3,015	-4,924	-6,155	-954 V=-41k	251 V=2k
7-7 112	-1,010	-4,082	---	-1,501	-2,859	-3,574	-632	175
8-8 124	-382	-1,585	-1,910(1) -169(2)	-487	-1,768	1,583 -2,210	-555	484
10-10 148	634	2,652	-1,045(1) 4,089(2)	1,179	3,270	4,087	-288	1,152

[1](#) denotes Deck Cast

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

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

Steel, Deck and Cast moments are unfactored. Deck includes 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 3.5.6.1.

[4]LLmax - Unfactored live-load plus impact moment due to multiple lanes of HS25. Impact is according to Article 3.5.6.1.

[5]Fat - Maximum and minimum fatigue moments due to one fatigue vehicle plus 15% impact times the load factor of 0.75 specified in Article 3.5.7.1.

All live load moments, including fatigue moments, include centrifugal force effects. Multiple presence reduction factors (**AASHTO Article 3.12**) were considered in determining Ovrlod and LLmax.

Table D2 Shear (kips), Girder 4 Span 1

Tenth Point	Steel	Deck	SupImp	Tot DL	LLoad	(5/3)LL	1.3(TotDL +5/3[LL])
0	23	92	42	157	116	193	455
1	16	69	24	109	94	157	345
2	11	44	15	70	78	130	260
3	3	10	5	18	61	102	156
4	-4	-19	-5	-28	-53	-87	-150
5	-10	-47	-16	-73	-67	-112	-241
6	-18	-74	-27	-119	-80	-133	-328
7	-24	-101	-36	-161	-95	-158	-418
8	-30	-121	-43	-194	-111	-185	-493
9	-36	-134	-53	-223	-125	-208	-560
10	-45	-144	-64	-253	-137	-243	-645

Table D3 Shear (kips), Girder 4 Span 2

Tenth Point	Steel	Deck	SupImp	Tot DL	LLoad	5/3LL	1.3(TotDL +5/3[LL])
0	44	142	65	251	135	225	619
1	33	131	54	218	127	212	559
Sect 8-8 Node 124 (Splice)	27	112 7#1 92#2	41	180	126 101OV -21 OV	210	507
2	25	107	38	170	114	190	468
3	18	77	27	122	95	158	364
4	9	38	14	51	75	125	241
5	0	-1	0	-1	56	93	120
6	-9	-38	-14	-51	-75	-125	-241

OV - Multiple lanes of HS20 (unfactored)
Appropriate allowance for impact is included in live load shears.
#1 and #2 under Deck denote casts (see Table D1).

Table D4 Load Combinations for Cross Frame Member 99 top - 100 bottom

Group	γ	D 22	L+I 10 -2	CF -2	W 7 -9	WL 0	LF 0	T 1 -1	Total (kips)
I	1.30	1.0 22	1.67 17	1.0 (-2)	0.0	0.0	0.0	0.0	51
II	1.30	1.0 22	0.0	0.0	1.0 7	0.0	0.0	0.0	38
III	1.30	1.0 22	1.0 10	1.0 (-2)	0.3 2	1.0 0	1.0 0	0.0	44
IV	1.30	1.0 22	1.0 10	1.0 (-2)	0.0	0.0	0.0	1.0 0	42
V	1.25	0.0	0.0	0.0	1.0 7	0.0	0.0	1.0 0	9
VI	1.25	1.0 22	1.0 10	1.0 (-2)	0.3 2	1.0 0	1.0	1.0 0	43

Table D5 Load Combinations for Cross Frame Member 99 bottom - 100 top

Group	Y	D -43	L+I 1 -15	CF 3	W 15 -14	W L 0	LF 0	T 4 -4	Total (kips)
I	1.30	1.0 -43	1.67 2 -25	1.0 (3)	0.0	0.0	0.0	0.0	-88
II	1.30	1.0 -43	0.0	0.0	1.0 -14	0.0	0.0	0.0	-74
III	1.30	1.0 -43	1.0 -15	1.0 (3)	0.3 -4	1.0 0	1.0	0.0	-81
IV	1.30	1.0 -43	1.0 -15	1.0 (3)	0.0	0.0	0.0	1.0 -4	-81
V	1.25	0.0	0.0	0.0	1.0 -14	0.0	0.0	1.0 -4	-23
VI	1.25	1.0 -43	1.0 -15	1.0 (3)	0.3 -4	1.0 0	1.0	1.0 -4	-83

Table D6 Load Combinations for Cross Frame Member 97 top - 98 bottom

Group	γ	D 11	L+I 0 -1	CF -7	W 37 -37	WL 0	LF 0	T 6 -6	Total (kips)
I	1.3 0	1.0 11	1.67 (-2)	1.0 (-7)	0.0	0.0	0.0	0.0	14
II	1.3 0	1.0 11	0.0	0.0	1.0 37	0.0	0.0	0.0	62
III	1.3 0	1.0 11	1.0 (-1)	1.0 (-7)	0.3 11	1.0 0	1.0	0.0	29
IV	1.3 0	1.0 11	1.0 (-1)	1.0 (-7)	0.0	0.0	0.0	1.0 6	22
V	1.25 5	0.0	0.0	0.0	1.0 37	0.0	0.0	1.0 6	54
VI	1.25 5	1.0 11	1.0 (-1)	1.0 (-7)	0.3 11	1.0 0	1.0	1.0 6	35

Table D7 Load Combinations for Cross Frame Member 97 bottom - 98 top

Group	Y	D -34	L+I 1 -13	CF 5	W 21 -22	WL 0	LF 0	T 1 -1	Total (kips)
I	1.30	1.0 -34	1.67 -22	1.0 (5)	0.0	0.0	0.0	0.0	-73
II	1.30	1.0 -34	0.0	0.0	1.0 -22	0.0	0.0	0.0	-73
III	1.30	1.0 -34	1.0 -13	1.0 (5)	0.3 -7	1.0 0	1.0 0.0	0.0	-70
IV	1.30	1.0 -34	1.0 -13	1.0 (5)	0.0	0.0	0.0	1.0 -1	-62
V	1.25	0.0	0.0	0.0	1.0 -22	0.0	0.0	1.0 -1	-29
VI	1.25	1.0 -34	1.0 -13	1.0 (5)	0.3 -7	1.0 0	1.0 0.0	1.0 -1	-69

Table D8 Load Combinations for Vertical Reaction Bearing 98

Group	Y	D -443	L+I 14 -204	CF 0	W -4 9	WL 0.0	LF 0.0	T 11 -11	Total (Kips)
I	1.30	1.0 -443	1.67 -341	1.0 0	0.0	0.0	0.0	0.0	-1,019
II	1.30	1.0 -443	0.0	0.0	1.0 -4	0.0	0.0	0.0	-581
III	1.30	1.0 -443	1.0 -204	1.0 0	0.3 -1	1.0 0	1.0 0.0	0.0	-842
IV	1.30	1.0 -443	1.0 -204	1.0 0	0.0 0	0.0	0.0	1.0 -11	-855
V	1.25	0.0	0.0	0.0	1.0 9	0.0	0.0	1.0 11	25
VI	1.25	1.0 -443	1.0 -204	1.0 0	0.3 -1	1.0 0	1.0 0	1.0 -11	-824

Table D9 Load Combinations for Tangential Reaction Bearing 98

Group	Y	D -2	L+I 13 -16	CF 0 0	W 1 -1	WL 0 0	LF 8 -8	T 50 -50	Total (Kips)
I	1.30	1.0 -2	1.67 -27	1.0 0	0.0 0	0.0	0.0	0.0 0.0	-38
II	1.30	1.0 -2	0.0 0.0	0.0	1.0 -1	0.0	0.0	0.0	-4
III	1.30	1.0 -2	1.0 -16	1.0 0	0.3 0	1.0 0	1.0 -8	0.0 0	-34
IV	1.30	1.0 -2	1.0 -16	1.0 0	0.0	0.0	0.0	1.0 -50	-88
V	1.25	0.0	0.0	0.0	1.0 -1	0.0	0.0	1.0 -50	-64
VI	1.25	1.0 -2	1.0 -16	1.0 0	0.3 0	1.0 0	1.0 -8	1.0 -50	-95

Table D10 Load Combinations for Radial Reaction Bearing 98

Group	γ	D 10	L+I 5 -1	CF 24	W 121 -121	WL 30 -30	LF 0	T 7 -7	Total (Kips)
I	1.30	1.0 10	1.67 8	1.0 24	0.0	0.0	0.0	0.0	55
II	1.30	1.0 10	0.0	0.0	1.0 121	0.0	0.0	0.0	170
III	1.30	1.0 10	1.0 5	1.0 24	0.3 36	1.0 30	1.0 0	0.0	136
IV	1.30	1.0 10	1.0 5	1.0 24	0.0	0.0	0.0	1.0 7	60
V	1.25	0.0	0.0	0.0	1.0 121	0.0	0.0	1.0 7	160
VI	1.25	1.0 10	1.0 5	1.0 24	0.3 36	1.0 30	1.0 0	1.0 7	140

Table D11 Selected Girder 4 Section Properties--Transversely Stiffened Web

Point Node	Section Size	Section Type	Moment of Inertia	Neutral Axis B	Neutral Axis T	D_c
1 24	20 x 1.0 84 x .5625 21 x 1.0 A=88.3 in^2	Noncomp	101,818	42.52	43.48	42.48
		Comp DL	181,371	60.28	25.72	24.72
		Comp DL Bars				
		Comp LL	242,459	73.79	12.21	11.21
		Comp LL Bars				
2 44 3 64 4 76	20 x 1.0 84 x .5625 21 x 1.5 A=98.7 in^2	Noncomp	118,978	38.47	48.03	47.03
		Comp DL	217,079	56.44	30.06	29.06
		Comp DL Bars	126,842	39.92	46.58	38.42
		Comp LL	297,525	71.08	15.42	14.42
		Comp LL Bars	141,391	42.60	43.90	41.10
5 88	28 x 1.25 84 x .625 27 x 1.5 A=128.0 in^2	Noncomp	168,029	41.63	45.12	40.13
		Comp DL				
		Comp DL Bars	175,058	42.69	44.06	41.19
		Comp LL				
		Comp LL Bars	188,290	44.68	42.07	43.18

Legend:
B = to the outermost edge of the bottom flange
T = to the outermost edge of the top flange
D_c = depth of the web in compression; where two values are shown, the top value is for positive moment and the bottom value is for negative moment.
Noncomp = steel section only
Comp DL = steel section plus concrete deck transformed using modular ratio of 3n.
Comp DL Bars= steel section plus longitudinal reinforcement area divided by 3.
Comp LL = steel section plus concrete deck transformed using modular ratio of n.
Comp LL Bars = steel section plus longitudinal reinforcement
Lstiff = distance of the longitudinal stiffener from the top flange

Table D11 Girder 4 Section Properties (continued)--Transversely Stiffened Web

Point Node	Section Size	Section Type	Moment of Inertia	Neutral Axis B	Neutral Axis T	D_c
6 100	28 x 2.5 84 x .625 27 x 3.0 A=203.5 in^2	Noncomp	313,872	42.56	46.94	39.56
		Comp DL	420,273	52.49	37.01	49.49
		Comp DL Bars	321,111	43.24	46.26	40.24
		Comp LL	545,757	64.17	25.33	61.17
		Comp LL Bars	335,040	44.55	44.95	41.55
7 112	17 x 1.0 84 x .5625 21 x 1.5 A=98.7 in^2	Noncomp	168,029	41.63	45.12	40.13
		Comp DL				
		Comp DL Bars	175,058	42.69	44.06	41.19
		Comp LL				
		Comp LL Bars	188,290	44.68	42.07	43.18
8 124	17 x 1.0 84 x .5625 21 x 1.5 A=95.8 in^2	Noncomp	111,989	36.98	49.52	48.52 35.48
		Comp DL	213,901	55.73	30.77	29.77 54.23
		Comp DL Bars	120,277	38.51	47.99	37.01
		Comp LL	296,306	70.80	15.70	14.70 69.30
		Comp LL Bars	135,575	41.34	45.16	39.84

Table D11 Girder 4 Section Properties (continued)--Unstiffened Web

Point Node	Section Size	Section Type	Moment of Inertia	Neutral Axis B	Neutral Axis T	D_c
2 44	20 x 1.0 84 x .875 21 x 1.25 A=119.7 in^2	Noncomp	126,432	41.00	45.25	44.25
		Comp DL	220,250	55.93	30.32	29.32
		Comp DL Bars				
		Comp LL	306,031	69.50	16.75	15.75
		Comp LL Bars				
6 100	27 x 2.5 84 x .875 26 x 3.0 A=219.0 in^2	Noncomp	316,052	42.84	46.66	39.84
		Comp DL	422,769	52.14	37.36	49.14
		Comp DL Bars	323,221	43.46	46.04	40.46
		Comp LL	552,380	63.41	26.09	60.41
		Comp LL Bars	337,054	44.68	44.82	41.68

Table D11 Girder 4 Section Properties (continued)--Longitudinally Stiffened Web

Point Node	Section Size	Section Type	Moment of Inertia	Neutral Axis B	Neutral Axis T	D_c
2 44	21 x 1.0 84 x .4375 22 x 1.5 A=90.75 in^2 Lstiff=18"	Noncomp	116,890	37.79	48.71	47.71
		Comp DL	214,623	57.03	29.47	28.47
		Comp DL Bars				
		Comp LL	290,835	71.93	14.57	13.57
		Comp LL Bars				
3 64	21 x 1.0 84 x .4375 22 x 1.5 A=90.75 in^2 Lstiff=42"	Noncomp	116,890	37.79	48.71	47.71 36.29
		Comp DL	214,623	57.03	29.47	28.47
		Comp DL Bars				
		Comp LL	290,835	71.93	14.57	13.57 70.43
		Comp LL Bars				
4 76	21 x 1.0 84 x .4375 22 x 1.5 A=90.75 in^2 Lstiff=56"	Noncomp	116,890	37.79	48.71	47.71 36.29
		Comp DL	214,623	57.03	29.47	55.53
		Comp DL Bars	124,931	39.37	47.12	37.88
		Comp LL	290,835	71.93	14.57	13.57 70.43
		Comp LL Bars	139,710	42.30	44.20	40.80
5 88	29 x 1.25 84 x .4375 28 x 1.5 A=115.0 in^2 Lstiff=66"	Noncomp	163,684	41.32	45.43	39.82
		Comp DL				
		Comp DL Bars	170,780	42.50	44.24	41.00
		Comp LL				
		Comp LL Bars	184,047	44.72	42.03	43.22
6 100	29 x 2.5 84 x .4375 28 x 3.0 A=193.3 in^2 Lstiff=66"	Noncomp	314,783	42.32	47.18	39.32
		Comp DL	421,112	52.72	36.78	49.72
		Comp DL Bars	322,084	43.03	46.47	40.03
		Comp LL	543,850	64.69	24.81	61.69
		Comp LL Bars	336,106	44.41	45.09	41.41

Appendix E

Sample Calculations

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Girder Stress Check Section 0-0 G4 Node 4
Transversely Stiffened Web - Shear Strength - Web

Determine the required transverse stiffener spacing in Field Section 1 of G4 according to the provisions of Article 6.3.2. $t_w = 0.5625$ in.

$$V_{cr} = CV_p \qquad \text{Eq (6-4)}$$
$$V_p = 0.58 F_y D t_w$$
$$V_{cr} = C \times 0.58 \times 50 \times 84 \times 0.5625 = C \times 1{,}370$$

Compute the elastic buckling coefficient C according to the provisions of Article 6.2.2. Try a required stiffener spacing, d = 84 in, which is equal to the web depth D. k_w is determined from Eq (6-9).

$$k_w = 5 + \frac{5}{(d/D)^2} = 5 + \frac{5}{(84/84)^2} = 10$$

Determine which equation is to be used to compute C.

$$\frac{D}{t_w} = \frac{84}{0.5625} = 149$$

$$1.38\sqrt{\frac{Ek_w}{F_y}} = 1.38\sqrt{\frac{29{,}000 \times 10}{50}} = 105 < 149$$

Therefore, use Eq (6-7).

$$C = \frac{1.52 E k_w}{(D/t_w)^2 F_y} = \frac{1.52 \times 29{,}000 \times 10}{(149)^2 \times 50} = 0.40$$

From Table D2, the maximum factored shear within this field section is at the end support (Section 0-0), V = 455 kips.

$$V_{cr} = CV_p = 0.40 \times 1{,}370 = 548 \text{ k} > 455 \text{ k} \quad \text{OK}$$

Therefore, transverse stiffener spacings up to D=84 in, which is the maximum permitted stiffener spacing according to Article 6.3, may be used in Field Section 1 of G4 (t_w=0.5625 in) since there is no point within this field section where the factored shear exceeds 455 kips. However, at Section 0-0, which is a simple end support, the spacing of the first stiffener adjacent to the support is limited to 0.5D=42 in according to the provisions of Article 6.3.

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

The girder must be checked for steel weight and for Cast#1 of the concrete deck on the 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 G4 is an exterior girder.

Overhang Bracket Load

Since G4 is an exterior girder, half of the overhang weight is assumed placed on the girder and the other half is placed on the overhang brackets as shown in Figure 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 20.5 feet. Assume that the average deck thickness in the overhang is 10 inches. The weight of the deck finishing machine is not considered.

Compute the vertical load on the overhang brackets.

$$\text{Deck} = \frac{1}{2} \text{ x } 3.75 \text{ ft x } \frac{10 \text{ in.}}{12 \text{ in/ft}} \text{ x } 150 \text{ lbs/ft}^3 = 234 \text{ lbs/ft}$$

$$\text{Deck forms + Screed rail} = \underline{240 \text{ lbs/ft}}$$

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

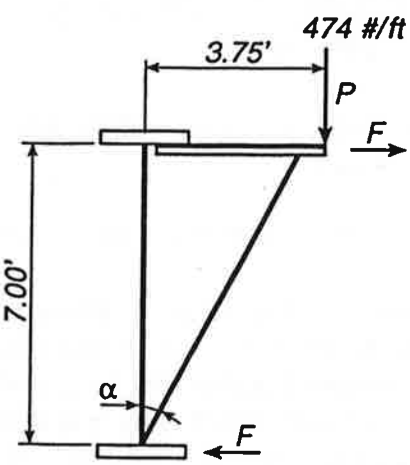

Figure E1 Overhang Bracket Loading

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

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

$$\alpha = \arctan\left(\frac{3.75 \text{ ft}}{7.00 \text{ ft}}\right) = 28^{o}$$

$$F = 474 \tan(28^{o}) = 252 \text{ lbs/ft}$$

Compute the lateral flange moment 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 G4 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 at the brace points.

$$M_{lat} = 0.08F\ell^2 = \frac{0.08 \times 252 \times 20.5^2}{1{,}000} = -8.5 \text{ k-ft} \qquad \text{Eq (C13-1)}$$

From Table D1, the moment due to the steel weight plus Cast #1 is 661+3,932=4,593 k-ft. The load factor for constructibility checks is 1.4 according to the provisions of Article 3.3. Using the section properties for the transversely stiffened web design from Table D11, the vertical bending stress, f_b, in the top flange is computed as:

$$f_{top\ flg} = f_b = \frac{4{,}593 \times 12 \times 48.03}{118{,}978} \times 1.4 = -31.15 \text{ ksi}$$

Top flange: 20 in x 1 in; $S = \frac{1}{6}(1)(20)^2 = 66.7 \text{ in}^3$

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.

$$f_m = \frac{-8.5 \times 12}{66.7} = -1.53 \text{ ksi; Load Factor} = 1.4$$

$$f_m = -1.53 \times 1.4 = -2.14 \text{ ksi}; \ \frac{f_m}{f_b} = 0.07 \text{ (the ratio is positive)}$$

Check the non-compact condition for constructibility according to Article 13.2. 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 I girder flanges. 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 2-2 will be conservatively used in the strength check. The approximate Eq (4-1) is used below to compute the lateral flange bending

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

moment due to curvature. Eq (4-1) assumes the presence of a cross frame at the point under investigation and that M is constant over the distance between brace points. Although the use of Eq (4-1) is not theoretically pure at locations in-between brace points, it can conservatively be used.

$$M_{lat} = \frac{6}{5}\frac{M\ell^2}{RD} = \frac{6}{5}\frac{4{,}593 \times 20.5^2}{717 \times 84} = -38.46 \text{ k-ft} \qquad \text{Eq (4-1)}$$

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

$$M_{tot\,lat} = -38.46 + (-8.5) = -46.96 \text{ k-ft}$$

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

$$f_\ell = \frac{-46.96 \times 12}{66.7} = -8.45 \text{ ksi}; \; |f_\ell| < 0.5F_y = 25.0 \text{ ksi OK} \qquad \text{Eq (5-1)}$$

$$f_b + f_\ell = -31.15 + (-8.45) = -39.60 \text{ ksi}$$

Since f_b exceeds $0.33F_y = 16.5$ ksi:

$$|f_\ell/f_b| \leq 0.5 \qquad \text{Eq (5-2)}$$

$$|f_\ell/f_b| = 8.45/31.15 = 0.27 < 0.5 \text{ OK}$$

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

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

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

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

$$\lambda = \frac{1}{\pi}\frac{20.50 \times 12}{0.9 \times 20}\sqrt{\frac{50}{29{,}000}} = 0.18$$

<u>Girder Stress Check Section 2-2 G4 Node 44</u>
<u>Transversely Stiffened Web - Constructibility - Top Flange</u>

$$F_{bs} = 50.0 \times [1 - 3(0.18)^2] = 45.14 \text{ ksi}$$

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

$$\rho_b = \frac{1}{1 + \frac{\ell}{R}\frac{12\ell}{b_f}} = \frac{1}{1 + \frac{20.50}{717} \times \frac{20.50 \times 12}{20}} = 0.74$$

$$\rho_{w1} = \frac{1}{1 - \frac{f_m}{f_b}\left(1 - \frac{12\ell}{75 b_f}\right)} = \frac{1}{1 - .07\left(1 - \frac{20.50 \times 12}{75 \times 20}\right)} = 1.06$$

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

$$= \frac{0.95 + \dfrac{\frac{20.50 \times 12}{20}}{30 + 8{,}000 \times \left(0.10 - \frac{20.50}{717}\right)^2}}{1 + 0.60 \times 0.07} = 1.08$$

$$\rho_b\rho_w = 0.74 \times 1.06 = 0.78$$

$$F_{cr1} = 45.14 \times 0.78 = 35.21 \text{ ksi}$$

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

$$F_{cr2} = 50 - 8.45 = 41.55 \text{ ksi} > F_{cr1} = 35.21 \text{ ksi}$$

$$\therefore F_{cr} = 35.21 \text{ ksi}$$

$$\frac{|-31.15|}{35.21} = 0.88 < 1.00 \text{ OK}$$

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

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

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

$$1.02\sqrt{\frac{29{,}000}{39.60}} = 27.60 > 23$$

$$\frac{b_f}{t_f} = \frac{20}{1} = 20 < 23 \text{ OK}$$

Check Eq (5-3):

$$\ell \le 25b_f \le R/10 \qquad \text{Eq (5-3)}$$

$$20.50\,(12) = 246 \text{ in} < 25\,(20) = 500 \text{ in OK}$$

$$20.50 \text{ ft} < 717/10 = 71.7 \text{ ft OK}$$

<u>Girder Stress Check Section 2-2 G4 Node 44</u>
<u>Transversely Stiffened Web - Constructibility - Web</u>

The girder must be checked for the steel weight and for Cast#1 of the concrete deck acting on the non-composite section.

Load	Moment (k-ft)	
Steel	661	Table D1
Cast #1	3,932	Table D1
Total Moment	4,593	

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

Using the section properties for the transversely stiffened web design from Table D11, compute the vertical bending stress at the top of the web for constructibility.

$$f_{top\ web} = \frac{4{,}593 \times 47.03}{118{,}978} \times 12 \times 1.4 = -30.50 \text{ ksi}$$

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 web bend buckling stress according to Article 6.3.1 because the web is transversely stiffened.

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

$$k = 9 \times \left(\frac{84}{47.03}\right)^2 = 28.71 > 7.2 \text{ OK}$$

$$F_{cr} = \frac{0.9 \times 29{,}000 \times 28.71}{\left(\frac{84}{0.5625}\right)^2} = 33.60 \text{ ksi}$$

$$\frac{|-30.50|}{33.60} = 0.91 < 1.00 \quad \text{OK}$$

The bottom flange is not critical for constructibility in this case.

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

Check the deck tensile stress at this section in Span 1 due to Cast #2 according to the provisions of Article 13.3. Moment due to Cast#2 = -3,035 k-ft from Table D1. This section is checked since it lies within the Cast #1 composite section, which is 100 feet long and assumed to be hardened for Cast #2. Assume no creep: n = 7.56.

$$f_{deck} = \frac{-3{,}035 \text{ x } 27.42}{297{,}525} \text{ x } \frac{12}{7.56} \text{ x } 1.4 = 0.62 \text{ ksi}$$

Compute the deck stress considering creep: 3n = 22.68.

$$f_{deck} = \frac{-3{,}035 \text{ x } 42.06}{217{,}079} \text{ x } \frac{12}{22.68} \text{ x } 1.4 = 0.44 \text{ ksi}$$

Because the calculated tensile stress in the deck based on the modular ratio of n is larger than the stress based on 3n, Article 13.3 specifies that n be used to compute stresses in the concrete due to factored construction loads, although the actual stress is probably somewhere between these two values. Article 13.3 further states that whenever the tensile stress in the deck exceeds 0.9 times the modulus of rupture defined in **AASHTO Article 8.15.2.1**, longitudinal reinforcement equal to at least one percent to the total cross-sectional area of the deck must be placed in the deck according to the provisions of Article 2.4.3. Assume the compressive strength of the hardened concrete from Cast #1 is 3,000 psi at the time Cast #2 is made. The modulus of rupture is:

$$f_r = 7.5\sqrt{f_c'} = \frac{7.5\sqrt{3{,}000}}{1{,}000} = 0.41 \text{ ksi}$$

$$0.9f_r = 0.9(0.41) = 0.37 \text{ ksi} < 0.62 \text{ ksi}$$

Therefore, one percent longitudinal reinforcement is required at this section. The reinforcement is to be No. 6 bars or smaller, spaced at not more than 12 inches. Article 2.4.3 also requires this reinforcement wherever the tensile stress in the deck due to the overload defined in Article 3.5.4 exceeds $0.9f_r$. For this case, the computed dead load stress in the deck is unfactored.

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

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

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

Check the top flange at this section for strength according to the provisions of Article 5.4 because the flange is continuously braced after the deck has hardened. Cross sectional properties are given in Table D11.

Load	Moment	
Steel	661 k-ft	Table D1
Deck	2,682 k-ft	Table D1
Total Non-composite	3,343 k-ft	
Superimposed DL	1,093 k-ft	Table D1
Live load HS25	3,968 k-ft	Table D1

Superimposed dead load includes the future wearing surface. Live load is due to three lanes of HS25 lane load plus the appropriate centrifugal force effects specified in Article 3.5.2. Impact has been applied to the live load according to Article 3.5.6.1. The overturning effect of the centrifugal force has been considered by increasing the exterior wheel load and decreasing the interior wheel load in each lane, as computed on pages 199 through 201. Although centrifugal force need be considered only for truck loads, the vertical effect also has been conservatively included for lane load in this example. The live loads were also multiplied by 0.90 in the analysis to account for the probability of multiple presence, as specified in **AASHTO Article 3.12.**

Compute the factored vertical bending stress in the top flange due to dead and live load.

$$f_{top\ flg} = \left[\frac{3{,}343 \times 48.03}{118{,}978} + \frac{1{,}093 \times 30.06}{217{,}079} + \frac{3{,}968(5/3) \times 15.42}{297{,}525}\right] \times 12 \times 1.3$$

$$= -28.76 \text{ ksi}$$

$$F_{cr} = F_y = 50 \text{ ksi}$$

$$\frac{|-28.76|}{50.0} = 0.58 < 1.00 \quad \text{OK}$$

According to the provisions of Article 5.4, lateral flange bending need not be considered after the deck has hardened.

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

Check the web strength at this section according to the provisions of Article 6.3.1.

Compute the factored vertical bending stress at the top and bottom of the web due to dead and live load.

$$f_{top\ web} = \left[\frac{3{,}343 \times 47.03}{118{,}978} + \frac{1{,}093 \times 29.06}{217{,}079} + \frac{3{,}968(5/3) \times 14.42}{297{,}525}\right] \times 12 \times 1.3 = -27.90 \text{ ksi}$$

$$f_{bot\ web} = \left[\frac{3{,}343 \times 36.97}{118{,}978} + \frac{1{,}093 \times 54.94}{217{,}079} + \frac{3{,}968(5/3) \times 69.58}{297{,}525}\right] \times 12 \times 1.3$$

$$= 44.65 \text{ ksi} < F_y \text{ OK}$$

Locate D_c using the accumulated factored web stresses according to the provisions of Article 6.1.

$$D_c = D\left(\frac{|f_{top\ web}|}{|f_{bot\ web}| + |f_{top\ web}|}\right) = 84 \times \left(\frac{27.90}{44.65 + 27.90}\right) = 32.30 \text{ in}$$

Compute the critical stress according to Article 6.3.1.

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

$$k = 9 \times \left(\frac{84}{32.30}\right)^2 = 60.87 > 7.2 \text{ OK}$$

$$F_{cr} = \frac{0.9 \times 29{,}000 \times 60.87}{\left(\frac{84}{0.5625}\right)^2} = 71.24 \text{ ksi}$$

71.24 > 50.0 ksi, therefore, F_{cr} = 50.0 ksi

$$\frac{|-27.90|}{50.0} = 0.56 < 1.00 \quad \text{OK}$$

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

Check the bottom flange strength at this section according to the provisions of Article 5.3.

Compute the factored bottom flange vertical bending stress due to dead and live load.

$$f_{bot\,flg} = \left[\frac{3{,}343 \times 38.47}{118{,}978} + \frac{1{,}093 \times 56.44}{217{,}079} + \frac{3{,}968(5/3) \times 71.08}{297{,}525}\right] \times 12 \times 1.3 = 45.94 \text{ ksi}$$

Compute the lateral bending stress at the cross frame due to curvature, f_w, by the V-load method Eq (4-1) (see earlier discussion regarding the use of this equation at this section, page 102).

$$M_{lat} = \frac{6}{5}\frac{M\ell^2}{RD} = \frac{6}{5} \times \frac{M \times (20.50)^2}{717 \times 84} = 0.0084M$$

Load		M_{lat}
Steel + Deck	= 0.0084 x 3,343 k-ft	= 28.1 k-ft
Superimp DL	= 0.0084 x 1,093 k-ft	= 9.2 k-ft
DL M_{lat}		= 37.3 k-ft
LL M_{lat}	= 0.0084 x 3,968 k-ft	= 33.3 k-ft

The lateral flange moment at the brace points due to curvature is positive in the bottom flange of all four girders whenever the bottom flange is subjected to tension because the stress due to the lateral moment is tensile on the convex side of the flange at the brace points (see Article 5.1). The opposite is true whenever the bottom flange is subjected to compression.

Compute the section modulus of the bottom flange plate, 1.5 x 21 in, about a vertical axis in the plane of the web.

$$S = \frac{b_f^2 t_f}{6} = \frac{21^2 \times 1.50}{6} = 110.3 \text{ in}^3$$

Compute the total factored lateral flange bending stress.

$$f_w = \left[\frac{37.3 + 33.3(5/3)}{110.3}\right] \times 12 \times 1.3 = 13.12 \text{ ksi}$$

Since there are no other sources of lateral flange moment in this case, $f_m = 0$. Thus, $f_\ell = 0 + f_w = f_w$.

$$f_\ell \le 0.5F_y \qquad \text{Eq (5-1)}$$

<u>Girder Stress Check Section 2-2 G4 Node 44</u>
<u>Transversely Stiffened Web - Strength - Bottom Flange</u>

$13.12 \text{ ksi} < 0.5(50) = 25 \text{ ksi OK}$

Since f_b exceeds $0.33F_y = 16.5$ ksi:

$$|f_\ell/f_b| \le 0.5 \qquad \text{Eq (5-2)}$$

$$|f_\ell/f_b| = 13.12/45.94 = 0.29 < 0.5 \text{ OK}$$

$$F_{cr2} = F_y - \frac{|f_\ell|}{3} \qquad \text{Eq (5-11)}$$

$$50.0 - \frac{13.12}{3} = 45.63 \text{ ksi}$$

$$\frac{45.94}{45.63} = 1.01 \cong 1.00 \quad \text{Say OK}$$

Eq (5-10) is not critical because the product of the $\bar{\rho}$ values exceeds 1.0, but is limited to 1.0 in Article 5.2.1. Therefore, $F_{cr1} = F_y$ according to Eq (5-10), which is greater than F_{cr2}.

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

Check the fatigue stress in the bottom flange at this section according to the provisions of Article 3.5.7 and 9.6.2. The fatigue design life is 75 years.

Base metal at transverse stiffeners must be checked for Category C' (refer to **Table 6.6.1.2.3-1** 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 and Article 9.6.1, uncracked concrete section properties are to be used for fatigue checks.

M_{min}	-431 k-ft	Table D1
M_{max}	1,177 k-ft	Table D1
M_{range}	1,608 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:

$$f_{range} = \frac{1{,}608 \times (71.08 - 1.50)}{297{,}525} \times 12 = 4.51 \text{ ksi}$$

The lateral flange bending stress in the flange at the connection plate must be considered since the stiffeners are welded to the tension flange (Article 9.6.2). Assume that the connection plates are 6 in wide.

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

<u>Girder Stress Check Section 2-2 G4 Node 44</u>
<u>Transversely Stiffened Web - Fatigue - Bottom Flange</u>

$$I_{flg} = \frac{1}{12} \times 21^3 \times 1.5 = 1{,}158 \text{ in}^4$$

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

$$M_{lat} = \frac{6}{5}\frac{M\ell^2}{RD} = \frac{6}{5}\frac{1{,}608 \times 20.5^2}{717 \times 84} = 13.46 \text{ k-ft}$$

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

$$c = 6 + .5625/2 = 6.3 \text{ in}$$

$$f_{lat} = \frac{13.46 \times 6.3}{1{,}158} \times 12 = 0.88 \text{ ksi}$$

Total stress range $= 4.51 + 0.88 = 5.39$ ksi

$$\frac{5.39}{6.0} = 0.90 < 1.0 \text{ OK}$$

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

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

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" x 7/8"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 shear connectors/row = $n Z_r = 3 \times 2.105$ = 6.315 kips/row
From Table D1, the bending shear range due to one factored fatigue truck = 15 + |-15| = 30 kips.

According to the provisions of Article 4.5.2, the entire deck cross sectional area is assumed to be effective. Deck thickness, t = 9.0 in. Modular ratio n equals 7.56.

$$\text{Effective deck width} = \left(\frac{11}{2} + 3.75\right) \times 12 = 111 \text{ in}$$

$$\text{Transformed deck area} = \frac{\text{Area}}{n} = \frac{111 \times 9}{7.56} = 132.1 \text{ in}^2$$

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 D11.
Neutral axis of the section is 15.42 in from the top of the steel.
Moment arm of the deck = Neutral axis - t_{flg} + haunch + $t_{deck}/2$
Moment arm = 15.42 in - 1.0 in + 4.0 in + 9 in/2 = 22.92 in

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

$$Q = 132.1 \times 22.92 = 3{,}028 \text{ in}^3$$

$$V_{fat} = \frac{VQ}{I} = \frac{30 \times 3{,}028}{297{,}525} = 0.31 \text{ k/in}$$

Compute the radial shear range, F_{fat}, due to the factored fatigue vehicle by the two methods specified in Article 7.2.2.

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

Method 1

Use the cross frame forces from the 3D analysis to compute F_{fat} from Eq (7-13):

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

Force range in cross frame diagonal going from bottom of G3 to top of G4
$F = +5.6 + |-1.2| = 6.8$ k
Diagonal forms a 33-degree angle from the horizontal.
Horizontal component of force = $\cos 33° \times 6.8$ k = 5.7 k
Force range in top chord = $+0.9 + |-0.2| = 1.1$ k
Top chord force acts in opposite direction to the horizontal component in the diagonal.
Therefore, the net range of force transferred from the cross frame to the top flange is F_{CR}.
$F_{CR} = 5.7\text{ k} - 1.1\text{ k} = 4.6\text{ k}$

According to the provisions of Article 7.2.2, compute the radial force according to Eq (7-13). According to Article 7.2.2, the effective length of deck w = 48 inches at an interior section.

$$F_{fat} = \frac{F_{CR}}{w} \qquad \text{Eq (7-13)}$$

$$F_{fat} = \frac{4.6}{48} = 0.10 \text{ k/in}$$

Method 2

Use the V-load assumption to compute F_{fat} due to curvature from Eq (7-12):

Compute the radial component of the fatigue shear range due to curvature, F_{fat}, according to the V-load assumption, as specified in the provisions of Article 7.2.2.

Use the range of fatigue moment 1,608 k-ft (computed earlier, page 111) to compute the bottom flange stress range at the mid-thickness of the flange.

$$F_{fat} = \frac{A_{bot}\sigma_{flg}\ell}{wR} \qquad \text{Eq (7-12)}$$

<u>Girder Stress Check Section 2-2 G4 Node 44</u>
<u>Transversely Stiffened Web - Fatigue - Shear Connectors</u>

$$\sigma_{flg} = \frac{1{,}608 \times (71.08 - 0.75)}{297{,}525} \times 12 = 4.56 \text{ ksi}$$

ℓ = 20.5 ft; A_{bot} = 31.5 in^2; R = 717 ft

$$F_{fat} = \frac{31.5 \times 4.56 \times 20.5}{48 \times 717} = 0.09 \text{ k/in}$$

The above value compares well with 0.10 k/in calculated by Method 1.

This similarity can be interpreted to indicate that all of the torsion is due to curvature. If other sources of torsion were present, such as skew, the radial shear range computed from the net range of cross frame force would be significantly greater than the radial shear range due to curvature computed according to the V-load assumption.

Using the radial component of shear range from Method 1, compute the net range of shear for fatigue.

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

$$V_{sr} = \sqrt{V_{fat}^{\ 2} + F_{fat}^{\ 2}} \qquad \text{Eq (7-11)}$$

$$V_{sr} = \sqrt{0.31^2 + 0.10^2} = 0.33 \text{ k/in}$$

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

$$\text{Shear stud pitch} = \frac{nZ_r}{V_{sr}} = \frac{6.315}{0.33} = 19.1 \text{ in/row}$$

Although not illustrated here, the number of shear connectors that is provided must also be checked for ultimate strength according to the provisions of Article 7.2.1. An ultimate strength check for shear connectors is illustrated later in this example.

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

Determine the required pitch of the shear connectors at this section (Section 3-3) according to the fatigue provisions of Article 7.2.2.

Compute the first moment of the deck with respect to the neutral axis of the uncracked live load composite section.
Section properties are from Table D11.
Determine the distance from the center of the deck to the neutral axis.
NA is 15.42 in from the top of the steel.
Neutral axis - flange thick. + haunch + deck thickness/2
15.42 - 1.0 + 4.0 + 9/2 = 22.92 in.
Bending shear range due to the factored fatigue vehicle = +5 +|-26|
= 31 kips (Table D1).

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

$$Q = 132.1 \text{ x } 22.92 = 3{,}028 \text{ in}^3$$

$$V_{fat} = \frac{VQ}{I} = \frac{31 \text{ x } 3{,}028}{297{,}525} = 0.32 \text{ k/in}$$

Compute the radial shear range due to the factored fatigue vehicle by the two methods specified in Article 7.2.2.

Method 1

Use the cross frame forces from the 3D analysis to compute F_{fat} from Eq (7-13):

Cross frame force range due to the factored fatigue vehicle (from separate calculations):
Force range in cross frame diagonal going from bottom of G3 to top of G4
F = +1.2 + |-4.3| = 5.5 k
Diagonal forms a 33-degree angle from the horizontal.
Horizontal component of force = cos 33° x 5.5 k = 4.6 k
Force range in top chord = +0.2 + |-0.9| = 1.1 k
Top chord force acts in opposite direction to the horizontal component in the diagonal.
Therefore, the net range of force transferred from the cross frame to the top flange is F_{CR}.
F_{CR} = 4.6 - 1.1 = 3.5 k

According to the provisions of Article 7.2.2, compute the radial force according to Eq (7-13).

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

According to Article 7.2.2, the effective length of deck w = 48 inches at an interior section.

$$F_{fat} = \frac{F_{CR}}{w} \qquad \text{Eq (7-13)}$$

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

Method 2

Use the V-load assumption to compute F_{fat} due to curvature from Eq (7-12):

Moment range due to the factored fatigue vehicle = $|-627| + 966 = 1{,}593$ k-ft (from Table D1). Use the range of fatigue moment 1,593 k-ft to compute the bottom flange stress range at the mid-thickness of the flange.

$$F_{fat} = \frac{A_{bot}\sigma_{flg}\ell}{wR} \qquad \text{Eq (7-12)}$$

$$\sigma_{flg} = \frac{1{,}593 \times (71.08 - 0.75)}{297{,}525} \times 12 = 4.52 \text{ ksi}$$

$\ell = 20.5$ ft; $A_{bot} = 31.5$ in^2; $R = 717$ ft

$$F_{fat} = \frac{31.5 \times 4.52 \times 20.5}{48 \times 717} = 0.08 \text{ k/in}$$

The above value compares well with 0.07 k/in calculated by Method 1.

This similarity can be interpreted to indicate that all of the torsion is due to curvature. If other sources of torsion were present, such as skew, the radial shear range computed from the net range of cross frame force would be significantly greater than the radial shear range due to curvature computed according to the V-load assumption.

Using the radial component of shear range from Method 2, compute the net range of shear for fatigue.

$$V_{sr} = \sqrt{V_{fat}^2 + F_{fat}^2} \qquad \text{Eq (7-11)}$$

$$V_{sr} = \sqrt{0.32^2 + 0.08^2} = 0.33 \text{ k/in}$$

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

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

$$\text{Shear stud pitch} = \frac{nZ_r}{V_{sr}} = \frac{6.315}{0.33} = 19.1 \text{ in/row}$$

Although not illustrated here, the number of shear connectors that is provided must also be checked for ultimate strength according to the provisions of Article 7.2.1. An ultimate strength check for shear connectors is illustrated later in this example.

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

Check the top and bottom flange for strength at this section according to the provisions of Article 5. The section will be checked for the Group I load combination in the following computations.

Load	Moment (k-ft)	
Steel	-1,917	Table D1
Deck	-7,272	Table D1
Total non-composite	-9,189	
Superimposed DL	-3,015	Table D1
Live load HS25	-6,155	Table D1

Top Flange

Check the top flange according to the provision of Article 5.4 since the flange is continuously braced after the deck has hardened. Compute the vertical bending stress in the top flange. For loads applied to the composite section, assume a cracked section, as specified in Article 4.5.2. Section properties are from Table D11.

$$f_{top\ flg} = \left[\frac{-9{,}189 \times 46.94}{313{,}872} + \frac{-3{,}015 \times 46.26}{321{,}111}\right] + \left[\frac{-6{,}155(5/3) \times 44.95}{335{,}040}\right] \times 12 \times 1.3 = 49.68 \text{ ksi}$$

$$F_{cr} = F_y = 50.0 \text{ ksi}; \quad \frac{49.68}{50.0} = 0.99 < 1.00 \quad \text{OK}$$

According to the provisions of Article 5.4, lateral flange bending need not be considered after the deck has hardened.

Bottom Flange

Compactness check

Compute the allowable width-to-thickness ratio of the bottom flange according to Article 5.2.1.

$$\frac{b_f}{t_f} = \frac{27}{3} = 9 < 18, \text{ therefore the flange is compact.}$$

The flange size is constant between brace points. The largest vertical bending stress, f_b, between brace points is at this section. The factored vertical bending stress is calculated as:

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

$$f_{bot\ flg} = \left[\frac{-9{,}189 \times 42.56}{313{,}872} + \frac{-3{,}015 \times 43.24}{321{,}111}\right] + \left[\frac{-6{,}155\,(5/3) \times 44.55}{335{,}040}\right] \times 12 \times 1.3 = -47.05 \text{ ksi}$$

Compute the critical stress for the bottom flange according to Article 5.2.1.

$$F_{cr1} = F_{bs}\ \overline{\rho_b}\,\overline{\rho_w} \qquad \text{Eq (5-4)}$$

$$F_{bs} = F_y(1 - 3\lambda^2) \qquad \text{Eq (5-5)}$$

$$\lambda = \frac{1}{\pi}\left(\frac{12\ell}{b_f}\right)\sqrt{\frac{F_y}{E}} = \frac{1}{\pi}\ \frac{20.50 \times 12}{0.9 \times 27}\sqrt{\frac{50}{29{,}000}} = 0.13$$

b_f is taken as $0.9\,b_f$ in computing λ if the section is not doubly symmetric.

$$F_{bs} = 50.0 \times [1 - 3(0.13)^2] = 47.47 \text{ ksi}$$

$$\overline{\rho_b} = \frac{1}{1 + \frac{12\ell}{b_f}\left(1 + \frac{2\ell}{b_f}\right)\left(\frac{\ell}{R} - 0.01\right)^2}$$

$$\overline{\rho_b} = \frac{1}{1 + \frac{12 \times 20.50}{27}\left(1 + \frac{2 \times 20.50}{27}\right) \times \left(\frac{20.50}{717} - 0.01\right)^2} = 0.99$$

Since there is no lateral flange bending other than that due to curvature, $f_m = 0.0$

$$\overline{\rho_w} = 0.95 + 18\left(0.1 - \frac{\ell}{R}\right)^2 + \frac{f_m}{f_b}\ \frac{\left(0.3 - 1.2\frac{\ell}{R}\frac{\ell}{b_f}\right)}{\overline{\rho_b}\left(\frac{F_{bs}}{F_y}\right)}$$

$$\overline{\rho_w} = 0.95 + 18\left(0.1 - \frac{20.50}{717}\right)^2 + 0 \times \frac{\left(0.3 - 1.2 \times \frac{20.50}{717} \times \frac{20.50}{27}\right)}{0.99\left(\frac{47.47}{50}\right)} = 1.04$$

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Transversely Stiffened Web - Strength - Top and Bottom Flange

$$\overline{\rho_b}\,\overline{\rho_w} = 0.99 \times 1.04 = 1.03$$

But $\overline{\rho_b}\,\overline{\rho_w} \le= 1.0 \therefore \overline{\rho_b}\,\overline{\rho_w} = 1.0$

$$F_{cr1} = F_{bs}\overline{\rho_b\rho_w} = 47.47 \times 1.0 = 47.47 \text{ ksi}$$

$$\frac{|f_{bot\ flg}|}{F_{cr1}} = \frac{|-47.05|}{47.47} = 0.99 < 1.00 \text{ OK}$$

Compute the lateral flange bending stress at the cross frame due to curvature, f_w, by the V-load method Eq (4-1).

$$M_{lat} = \frac{6}{5}\,\frac{M\ell^2}{RD} = \frac{6}{5}\,\frac{M \times (20.50)^2}{717 \times 84} = .0084M \qquad \text{Eq (4-1)}$$

Load		M_{lat} (k-ft)
Steel + Deck	= 0.0084 x (-9,189)	= -77.2
Superimp DL	= 0.0084 x (-3,015)	= -25.3
DL M_{lat}		= -102.5
LL M_{lat}	= 0.0084 x (-6,155)	= -51.7

Compute the section modulus of the bottom flange plate about a vertical axis in the plane of the web.

$$S_{bot\ flg} = \frac{1}{6} \times 3 \times 27^2 = 364.5 \text{ in}^3$$

Compute the total factored lateral flange bending stress.

$$f_w = \left[\frac{-102.5 - 51.7(5/3)}{364.5}\right] \times 12 \times 1.3 = -8.07 \text{ ksi}$$

Since there are no other sources of lateral flange bending moment other than curvature, $f_m = 0.0$ Thus, $f_\ell = f_w$.

$$|f_\ell| \le 0.5F_y \qquad \text{Eq (5-1)}$$

$$|-8.07| \text{ ksi} < 0.5(50) = 25 \text{ ksi OK}$$

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

Since f_b exceeds $0.33 F_y = 16.5$ ksi:

$$|f_\ell / f_b| \le 0.5 \qquad \text{Eq (5-2)}$$

$$|f_\ell / f_b| = 8.07/47.05 = 0.17 < 0.5 \text{ OK}$$

$$F_{cr2} = F_y - \frac{|f_\ell|}{3} \qquad \text{Eq (5-6)}$$

$$50 - \frac{|-8.07|}{3} = 47.31 \text{ ksi} < F_{cr1} \quad \therefore F_{cr} = F_{cr2} = 47.31 \text{ ksi}$$

$$\frac{|-47.05|}{47.31} = 0.99 < 1.00 \text{ OK}$$

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

Girder Stress Check Section 6-6 G4 Node 100
Transversely Stiffened Web - Overload - Bottom Flange

The live load for overload is multiple lanes of HS20 in this example.
The unfactored HS20 live load moment plus impact at this section is -4,924 k-ft (from Table D1). 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.

The unfactored lateral flange moment due to DL is -102.5 k-ft as given on page 121. Compute the live load lateral flange moment due to curvature with the V-load equation (see page 121) along with the factored lateral flange stress.

$$M_{lat} = 0.0084 \times (-4{,}924) = -41.4 \text{ k-ft}$$

$$f_w = \frac{-102.5 + (-41.4 \times 5/3)}{364.5} \times 12 = -5.65 \text{ ksi}$$

Compute the bottom flange stress due to vertical bending.
According to the provisions of Article 9.5, assume the section is uncracked for loads applied to the composite section.

$$f_{bot\,flg} = \left(\frac{-9{,}189 \times 42.56}{313{,}872} + \frac{-3{,}015 \times 52.49}{420{,}273} + \frac{-4{,}924(5/3) \times 64.17}{545{,}757} \right) \times 12 = -31.05 \text{ ksi}$$

According to the provisions of Article 9.5, the vertical bending stress at overload in a partially braced compression flange must be less than F_{cr} determined from Eq (5-8) of Article 5.2.2.

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

Use F_{bs} = 47.47 ksi from the previous calculation given on page 120. Since there is no other lateral flange bending other than that due to curvature, $f_m = 0.0$.

$$\rho_b = \frac{1}{1 + \dfrac{\ell}{R}\dfrac{12\ell}{b_f}} = \frac{1}{1 + \dfrac{20.5}{717} \times \dfrac{12 \times 20.5}{27}} = 0.79$$

$$\rho_{w1} = \frac{1}{1 - \dfrac{f_m}{f_b}\left(1 - \dfrac{12\ell}{75 b_f}\right)} = \frac{1}{1 - 0 \times \left(1 - \dfrac{12 \times 20.5}{75 \times 27}\right)} = 1.00$$

Girder Stress Check Section 6-6 G4 Node 100
Transversely Stiffened Web - Overload - Bottom Flange

$$\rho_{w2} = \frac{0.95 + \dfrac{\dfrac{12\ell}{b_f}}{30 + 8{,}000\left(0.1 - \dfrac{\ell}{R}\right)^2}}{1 + 0.6\left(\dfrac{f_m}{f_b}\right)}$$

$$\rho_{w2} = \frac{0.95 + \dfrac{\dfrac{12\text{x}20.5}{27}}{30 + 8{,}000\text{x}\left(0.1 - \dfrac{12\text{x}20.5}{717}\right)^2}}{1 + 0.6\text{x}0} = 0.97$$

Therefore, $\rho_w = 0.97$; smaller of ρ_{w1} or ρ_{w2}

$$\rho_b \rho_w = 0.79 \text{ x } 0.97 = 0.77$$

$$F_{cr1} = 47.47 \text{ x } 0.77 = 36.55 \text{ ksi}$$

$$\frac{|-31.05|}{36.55} = 0.85 < 1.00 \text{ OK}$$

Since the top flange is continuously braced at overload, the vertical bending stress in the top flange is limited to $0.95F_y$ according to Article 9.5 (calculations not shown). For partially braced tension flanges, the vertical bending stress is limited to the critical stress given by Eq (5-8).

Although not illustrated here, the web must also be checked at overload to ensure that the maximum compressive stress in the web does not exceed the bend-buckling stress (Article 9.5). The composite section is assumed to be uncracked in this check. Web checks for overload are illustrated elsewhere in this example.

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

Fatigue of the base metal at the bottom of the top flange adjacent to the bearing stiffener/connection plate weld to the flange at this section will be checked for Category C'. Stresses are computed using the factored fatigue vehicle defined in Article 3.5.7.1. The vehicle is placed in an adjacent span to create a negative moment and in the third span to create a positive moment at this section.

M_{min}	|-954| k-ft	Table D1
M_{max}	251 k-ft	Table D1
M_{range}	1,205 k-ft	

According to the provisions of Article 5.4, the lateral flange bending stress in the top flange is assumed to be zero since the top flange is considered continuously braced after the deck has hardened. According to the provisions of Article 4.5.2, uncracked concrete section properties are to be used for the fatigue checks.

$$f_{fat} = \frac{1{,}205 \times (25.33 - 2.5)}{545{,}757} \times 12 = 0.60 \text{ ksi}$$

One-half of the fatigue threshold for Category C' = 6 ksi, as discussed previously.

$$\frac{0.60}{6} = 0.1 < 1.0 \quad \text{OK}$$

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

Determine the required transverse stiffener spacing in Field Section 2 of G4 according to the provisions of Article 6.3.2.

The maximum factored shear within this field section is at the pier (Section 6-6), V = 645 kips per Table D2.

t_w= 0.625 in.

$$V_{cr} = CV_p \qquad \text{Eq (6-4)}$$

$$V_p = 0.58F_yDt_w = 0.58 \times 50 \times 84 \times 0.625 = 1{,}523 \text{ k}$$

Try a required spacing d=84 in, which is equal to the maximum permissible spacing of D specified in Article 6.3.

$$k_w = 5 + 5\left(\frac{D}{d}\right)^2 = 5 + 5\left(\frac{84}{84}\right)^2 = 10 \qquad \text{Eq (6-9)}$$

$$\frac{D}{t_w} = \frac{84}{0.625} = 134$$

$$1.38\sqrt{\frac{Ek_w}{F_y}} = 1.38\sqrt{\frac{29{,}000 \times 10}{50}} = 105 < 134$$

$$C = \frac{1.52Ek_w}{\left(\frac{D}{t_w}\right)^2 F_y} = \frac{1.52 \times 29{,}000 \times 10}{\left(\frac{84}{0.625}\right)^2 \times 50} = 0.49 \qquad \text{Eq (6-7)}$$

$$V_{cr} = CV_p = 0.49 \times 1{,}523 = 746 \text{ k} > 645 \text{ k} \quad \text{OK} \qquad \text{Eq (6-4)}$$

Therefore, transverse stiffener spacings up to the maximum permitted spacing of D=84 in may be used in Field Section 2 of G4 (t_w= 0.625 in).

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

Check the web strength at this section according to the provisions of Article 6.3.1. Use the moments from Table D1 and the section properties from Table D11. The composite section is assumed cracked for this condition according to the provisions of Article 4.5.2. Compute the factored vertical bending stress at the top and bottom of the web due to dead and live load.

$t_w = 0.625$ in

$$f_{top\ web} = \left[\frac{-9{,}189 \times 44.44}{313{,}872} + \frac{-3{,}015 \times 43.76}{321{,}111}\right] + \left[\frac{-6{,}155\,(5/3) \times 42.45}{335{,}040}\right] \times 12 \times 1.3 = 46.98 \text{ ksi} < F_y \text{ OK}$$

$$f_{bot\ web} = \left[\frac{-9{,}189 \times 39.56}{313{,}872} + \frac{-3{,}015 \times 40.24}{321{,}111}\right] + \left[\frac{-6{,}155\,(5/3) \times 41.55}{335{,}040}\right] \times 12 \times 1.3 = -43.81 \text{ ksi}$$

$$D_c = 84\left(\frac{|-43.81|}{|-43.81| + 46.98}\right) = 40.53 \text{ in}$$

$$k = 9.0\left(\frac{D}{D_c}\right)^2 = 9.0\left(\frac{84}{40.53}\right)^2 = 38.7 > 7.2 \text{ OK}$$

$$F_{cr} = \frac{0.9Ek}{\left(\frac{D}{D_c}\right)^2} = \frac{0.9 \times 29{,}000 \times 38.7}{\left(\frac{84}{0.625}\right)^2} = 55.92 \text{ ksi} > 50 \text{ ksi} \qquad \text{Eq (6-8)}$$

$$\therefore F_{cr} = 50 \text{ ksi}$$

$$\frac{|-43.81|}{50.00} = 0.88 < 1.00 \text{ OK}$$

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

Determine the required shear connector pitch for fatigue at this section according to the provisions of Article 7.2.2.

Compute the first moment of the deck with respect to the neutral axis of the uncracked live load composite section.
Determine the distance from the center of the deck to the neutral axis.
NA is 25.33 in. from the top of the steel (Table D11).
Neutral axis - flange thick. + haunch + deck thickness/2

$$25.33 - 2.5 + 4.0 + 9/2 = 31.33 \text{ in.}$$

Bending shear range due to the factored fatigue vehicle = +2+|-41| = 43 kips from Table D1.

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

$$Q = 132.1 \times 31.33 = 4{,}139 \text{ in}^3$$

$$V_{fat} = \frac{VQ}{I} = \frac{43 \times 4{,}139}{545{,}757} = 0.33 \text{ k/in}$$

Compute the radial shear range, F_{fat}, due to the factored fatigue vehicle by the two methods specified in Article 7.2.2.

Method 1

Use the cross frame forces from the 3D analysis to compute F_{fat} from Eq (7-13):

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

Force range in cross frame diagonal going from bottom of G3 to top of G4
F = +0.8 + |-5.3| = 6.1 k
Diagonal forms a 33-degree angle from the horizontal.
Horizontal component of force = cos 33° x 6.1 = 5.1 k.
Force range in top chord = +0.2 + |-0.6| = 0.8 k
Top chord force acts in opposite direction to the horizontal component in the diagonal.
Therefore, the net range of force transferred from the cross frame to the top flange is F_{CR}.
$F_{CR} = 5.1 - 0.8 = 4.3$ k

According to the provisions of Article 7.2.2, compute the radial force according to Eq (7-13).

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

According to Article 7.2.2, the effective length of deck w = 48 inches at an interior section.

$$F_{fat} = \frac{F_{CR}}{w} \qquad \text{Eq (7-13)}$$

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

<u>Method 2</u>

Use the V-load assumption to compute F_{fat} due to curvature from Eq (7-12):

Use the range of fatigue moment 1,205 k-ft to compute the bottom flange stress range at the mid-thickness of the flange.

$$F_{fat} = \frac{A_{bot}\sigma_{flg}\ell}{wR} \qquad \text{Eq (7-12)}$$

$$\sigma_{flg} = \frac{1{,}205 \times (64.17 - 1.5)}{545{,}757} \times 12 = 1.66 \text{ ksi}$$

$\ell = 20.5$ ft; $A_{bot} = 81$ in^2; R = 717 ft

$$F_{fat} = \frac{81 \times 1.66 \times 20.5}{48 \times 717} = 0.08 \text{ k/in}$$

The above value compares well with 0.09 k/in calculated by Method 1.

This similarity can be interpreted to indicate that all of the torsion is due to curvature. If other sources of torsion were present, such as skew, the radial shear range computed from the net range of cross frame force would be significantly greater than the radial shear range due to curvature computed according to the V-load assumption.

Using the radial component of shear range from Method 1, compute the net range of shear for fatigue.

$$V_{sr} = \sqrt{V_{fat}^2 + F_{fat}^2} \qquad \text{Eq (7-11)}$$

$$V_{sr} = \sqrt{0.33^2 + 0.09^2} = 0.34 \text{ k/in}$$

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

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

$$\text{Shear stud pitch} = \frac{nZ_r}{V_{sr}} = \frac{6.315}{0.34} = 18.6 \text{ in/row}$$

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

Compute the number of shear connectors required for ultimate strength between the end of the girder in Span 1 and the point of maximum positive live load moment in Span 1 according to the provisions of Article 7.2.1.

Shear connectors are 6 inches long x 7/8" in diameter.

Compute the ultimate strength of one shear connector.

H/d = 6.0/0.875 = 6.86 > 4.0; Use: **AASHTO Eq (10-67).**

$$S_u = 0.4d^2\sqrt{f_c' E_c} \le 60{,}000 A_{sc}$$ **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.)

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

$$E_c = 3.6 \times 10^6 \text{ psi}$$

$$S_u = \frac{0.4 \times 0.875^2\sqrt{4{,}000 \times 3.6 \times 10^6}}{1{,}000} = 37 \text{ kips} > 60(0.6) = 36 \text{ kips}$$

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

Compute the critical longitudinal force according to the provisions of Article 7.2.1.

The distance L_p between the point of maximum positive live load moment and the end support is 73 feet.

According to Article 4.5.2, the entire width of deck is assumed effective. Although G4 is an exterior girder with an overhang less than half of the girder spacing, the width of the deck is assumed to be equal to the girder spacing of 11 feet so that all girders will have the same stud spacing.

$$P_{1p} = A_s F_y = 98.7 \times 50 = 4{,}935 \text{ kips}$$ Eq (7-3)

$$P_{2p} = 0.85 f_c' b_d t_d = 0.85 \times 4.0 \times (11 \times 12) \times 9 = 4{,}039 \text{ kips}$$ Eq (7-4)

therefore, $\overline{P_p} = 4{,}039$ kips

$$\overline{F_p} = \overline{P_p}\frac{L_p}{R} = 4{,}039 \times \frac{73}{717} = 411 \text{ kips}$$ Eq (7-5)

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

$$P = \sqrt{\overline{P_p}^2 + \overline{F_p}^2} = \sqrt{4{,}039^2 + 411^2} = 4{,}060 \text{ kips} \qquad \text{Eq (7-2)}$$

$$N = \frac{P}{\phi_{sc} S_u} = \frac{4{,}060}{0.85 \times 36} = 133 \qquad \text{Eq (7-1)}$$

Compute the required pitch p with 3 studs per row.

$$\text{No. of rows} = \frac{133}{3} = 45 \text{ rows}$$

$$p = \frac{73 \times 12}{(45 - 1)} = 19.9 \text{ in}$$

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

Compute the required pitch between the point of maximum negative live load moment and the adjacent point of maximum positive live load moment.

Compute the tension force in the deck at the support according to the provisions of Article 7.2.1.

$$P_{1n} = A_s F_y = 203.5 \times 50 = 10{,}175 \text{ kips} \qquad \text{Eq (7-8)}$$

$$P_{2n} = 0.45 f_c' b_d t_d = 0.45 \times 4 \times (11 \times 12) \times 9 = 2{,}138 \text{ kips} \qquad \text{Eq (7-9)}$$

therefore, $P_n = 2{,}138$ kips

$$\overline{P_T} = \overline{P_p} + \overline{P_n} = 4{,}039 + 2{,}138 = 6{,}177 \text{ kips} \qquad \text{Eq (7-7)}$$

$L_n = 164 - 73 = 91$ ft

$$\overline{F_T} = \overline{P_T} \frac{L_n}{R} = 6{,}177 \times \frac{91}{717} = 784 \text{ kips} \qquad \text{Eq (7-10)}$$

$$P = \sqrt{\overline{P_T}^2 + \overline{F_T}^2} = \sqrt{6{,}177^2 + 784^2} = 6{,}227 \text{ kips} \qquad \text{Eq (7-6)}$$

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

$$N = \frac{P}{\phi_{sc} S_u} = \frac{6{,}227}{0.85 \times 36} = 204 \qquad \text{Eq (7-1)}$$

Compute the required pitch p with 3 studs per row.

$$\text{No. of rows} = \frac{204}{3} = 68 \text{ rows}$$

$$p = \frac{91 \times 12}{(68 - 1)} = 16.3 \text{ in}$$

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

Girder Stress Check Section 2-2 G4 Node 44
Unstiffened Web - Constructibility - Web

Check the unstiffened web at this section according to the provisions of Article 6.2.1 for the non-composite load due to the steel weight and Cast#1.

Use the non-composite moments for the transversely stiffened web design at Node 44 from Table D1.

Compute the constructibility vertical bending stress at the top and bottom of the web with a load factor = 1.4, as specified in Article 3.3. Use the section properties for the unstiffened web design from Table D11.

$$f_{top\ web} = \frac{4{,}593 \times 44.25}{126{,}432} \times 12 \times 1.4 = -27.01 \text{ ksi}$$

$$f_{bot\ web} = \frac{4{,}593 \times 39.75}{126{,}432} \times 12 \times 1.4 = 24.26 \text{ ksi}$$

Compute D_c using the effective factored stresses.

$$D_c = 84 \times \left(\frac{|-27.01|}{|-27.01| + 24.26} \right) = 44.25 \text{ in}$$

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 web bend-buckling stress for the unstiffened web using the provisions of Article 6.2.1.

$$F_{cr} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} \le F_y; \; k = 7.2\left(\frac{D}{D_c}\right)^2 \ge 7.2 \qquad \text{Eq (6-3)}$$

$$k = 7.2 \times \left(\frac{84}{44.25} \right)^2 = 25.95 > 7.2 \text{ OK}$$

$$F_{cr} = \frac{0.9 \times 29{,}000 \times 25.95}{\left(\frac{84}{0.875}\right)^2} = 73.49 \text{ ksi} > 50.0 \text{ ksi, therefore } F_{cr} = 50.0 \text{ ksi}$$

$$\frac{|-27.01|}{50.0} = 0.54 < 1.00 \text{ OK}$$

Girder Stress Check Section 2-2 G4 Node 44
Unstiffened Web - Bending Strength - Web

Check the strength of the unstiffened web at this section in bending according to the provisions of Article 6.2.1. Compute the factored vertical bending stress at the top and bottom of the web due to dead and live load.

Use the moments from Table D1. Use the section properties from Table D11.

$$f_{top\ web} = \left[\frac{3{,}343 \times 44.25}{126{,}432} + \frac{1{,}093 \times 29.32}{220{,}250} + \frac{3{,}968\,(5/3) \times 15.75}{306{,}031}\right] \times 12 \times 1.3 = -25.83\ \text{ksi}$$

$$f_{bot\ web} = \left[\frac{3{,}343 \times 39.75}{126{,}432} + \frac{1{,}093 \times 54.68}{220{,}250} + \frac{3{,}968\,(5/3) \times 68.25}{306{,}031}\right] \times 12 \times 1.3 = 43.64\ \text{ksi} < F_y\ \text{OK}$$

Locate the effective D_c from the factored stresses in the top and bottom of the web.

$$D_c = 84 \times \left(\frac{|-25.83|}{|-25.83| + 43.64}\right) = 31.23\ \text{in}$$

Compute the critical stress according to Article 6.2.1.

$$F_{cr} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} \leq F_y;\ k = 7.2\left(\frac{D}{D_c}\right)^2 \geq 7.2 \qquad \text{Eq (6-3)}$$

$$k = 7.2 \times \left(\frac{84}{31.23}\right)^2 = 52.09 > 7.2\ \text{OK}$$

$$F_{cr} = \frac{0.9 \times 29{,}000 \times 52.09}{\left(\frac{84}{0.875}\right)^2} = 147\ \text{ksi} > 50.0\ \text{ksi} \therefore F_{cr} = 50\ \text{ksi}$$

$$\frac{|-25.83|}{50.0} = 0.52 < 1.00\ \text{OK}$$

Girder Stress Check Section 2-2 G4 Node 44
Unstiffened Web - Overload - Web

Check the unstiffened web for bend-buckling at overload at this section according to the provisions of Article 9.5, which refers to the provisions of Article 6.2.1.

Use the moments from Table D1. Use the section properties from Table D11. Compute the overload vertical bending stress at the top and bottom of the web.

$$f_{top\ web} = \left[\frac{3{,}343 \times 44.25}{126{,}432} + \frac{1{,}093 \times 29.32}{220{,}250} + \frac{3{,}174(5/3) \times 15.75}{306{,}031}\right] \times 12 = -19.05 \text{ ksi}$$

$$f_{bot\ web} = \left[\frac{3{,}343 \times 39.75}{126{,}432} + \frac{1{,}093 \times 54.68}{220{,}250} + \frac{3{,}174(5/3) \times 68.25}{306{,}031}\right] \times 12 = 30.03 \text{ ksi}$$

Locate the effective D_c from the factored stresses in the top and bottom of the web.

$$D_c = 84 \times \left(\frac{|-19.05|}{|-19.05| + 30.03}\right) = 32.60 \text{ in}$$

$$F_{cr} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} \le F_y; \; k = 7.2\left(\frac{D}{D_c}\right)^2 \ge 7.2 \qquad \text{Eq (6-3)}$$

$$k = 7.2 \times \left(\frac{84}{32.60}\right)^2 = 47.80 > 7.2 \text{ OK}$$

$$F_{cr} = \frac{0.9 \times 29{,}000 \times 47.80}{\left(\frac{84}{0.875}\right)^2} = 135 \text{ ksi} > 50.0 \text{ ksi} \therefore F_{cr} = 50 \text{ ksi}$$

$$\frac{|-19.05|}{50.0} = 0.38 < 1.00 \text{ OK}$$

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

Check the strength of the unstiffened web at this section in bending according to the provisions of Article 6.2.1.

Use the moments from Table D1. Use the section properties from Table D11. The composite section is assumed cracked for this case, as specified in Article 4.5.2. Compute the factored vertical bending stress at the top and bottom of the web due to dead and live load.

$$f_{top\ web} = \left[\frac{-9{,}189 \times 44.16}{316{,}052} + \frac{-3{,}105 \times 43.54}{323{,}221} + \frac{-6{,}155(5/3) \times 42.32}{337{,}054}\right] \times 12 \times 1.3 = 46.46\ \text{ksi} < F_y\ \text{OK}$$

$$f_{bot\ web} = \left[\frac{-9{,}187 \times 39.84}{316{,}052} + \frac{-3{,}015 \times 40.46}{323{,}221} + \frac{-6{,}155(5/3) \times 41.68}{337{,}054}\right] \times 12 \times 1.3 = -43.75\ \text{ksi}$$

Locate the effective D_c from the factored stresses in the top and bottom of the web.

$$D_c = 84 \times \left(\frac{|-43.75|}{|-43.75| + 46.46}\right) = 40.74\ \text{in}$$

$$F_{cr} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} \le F_y;\ k = 7.2\left(\frac{D}{D_c}\right)^2 \ge 7.2 \qquad \text{Eq (6-3)}$$

$$k = 7.2 \times \left(\frac{84}{40.74}\right)^2 = 30.61 > 7.2\ \text{OK}$$

$$F_{cr} = \frac{0.9 \times 29{,}000 \times 30.61}{\left(\frac{84}{0.875}\right)^2} = 86.7\ \text{ksi} > 50.0\ \text{ksi} \therefore F_{cr} = 50\ \text{ksi}$$

$$\frac{|-43.75|}{50.0} = 0.88 < 1.00\ \text{OK}$$

The web at this section must also be checked for overload (not illustrated). Web checks for overload are illustrated elsewhere in this example.

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

Determine if transverse stiffeners are required at the point of maximum factored shear (Section 6-6). From Table D2, the maximum factored shear at this section is 645 kips, which is the maximum shear in the girder.

Compute the shear strength of the unstiffened web according to the provisions of Article 6.2.2. $t_w = 0.875$ in.

$$V_{cr} = CV_p \qquad \text{Eq (6-4)}$$

$$V_p = 0.58F_yDt_w$$

$$V_{cr} = C \times 0.58 \times 50 \times 84 \times 0.875 = C \times 2{,}131$$

Determine C.

$$C = \frac{1.52Ek_w}{(D/t_w)^2F_y} \quad \text{for } \frac{D}{t_w} > 1.38\sqrt{\frac{Ek_w}{F_y}} \qquad \text{Eq (6-7)}$$

$$k_w = 5$$

$$1.38\sqrt{\frac{Ek_w}{F_y}} = 1.38\sqrt{\frac{29{,}000 \times 5}{50}} = 74$$

$$\frac{D}{t_w} = \frac{84}{0.875} = 96 > 74$$

$$\text{Therefore } C = \frac{1.52Ek_w}{(D/t_w)^2F_y} = \frac{1.52 \times 29{,}000 \times 5}{(96)^2 \times 50} = 0.48$$

Compute the shear strength of the unstiffened web.

$$V_{cr} = CV_p = 0.48 \times 2{,}131 = 1{,}023 \text{ k} > 645 \text{ k OK}$$

Therefore, intermediate transverse stiffeners are not required anywhere along the girder.

<u>Girder Stress Check Section 0-0 G4 Node 4</u>
<u>Transversely Stiffened Web - Bearing Stiffener Design</u>

This location has the largest total reaction at a simple end support (Appendix B).

Load	Reaction (kips)
Steel	23 x 1.3 = 30
Deck	92 x 1.3 = 120
Superimposed	42 x 1.3 = 55
HS25 LL	116 x 1.3 x 5/3 = 251
Total	273 456

Design the bearing stiffeners according to the provisions of Article 6.7. Use bars with F_y = 50 ksi. Compute the maximum permissible b/t according to Eq (6-13).

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

Compute the effective area of the web (t_w = 0.5625 in).

$$A_w = 18t_w^2 = 18 \times 0.5625^2 = 5.70 \text{ in}^2$$

Try 2-bars 7 in x 0.625 in. Bearing area = 2(7 - 1.0)(0.625) = 7.50 in^2 (Assume 1 in. for the stiffener clip). Bearing strength of milled stiffeners = 1.35F_y= 67.5 ksi

$$\frac{b_s}{t_s} = \frac{7.0}{0.625} = 11.2 < 11.6 \text{ OK}$$

$$\frac{456}{67.5} = 6.76 \text{ in}^2 < 7.50 \text{ in}^2$$

$$A = 5.70 + 7.50 = 13.2 \text{ in}^2$$

$$I = \frac{14.5625^3 \times 0.625}{12} = 160.8 \text{ in}^4;\ r = \sqrt{\frac{I}{A}} = \sqrt{\frac{160.8}{13.2}}$$

$$r = 3.49 \text{ in};\ K = 0.75;\ L = 84 \text{ in};\ \frac{KL}{r} = \frac{0.75 \times 84}{3.49} = 18.1 < \sqrt{\frac{2\pi^2 E}{F_y}} = 107.0$$

Girder Stress Check Section 0-0 G4 Node 4
Transversely Stiffened Web - Bearing Stiffener Design

Assume the concentrated load is applied concentrically to the stiffener in this case.
From **AASHTO Article 10.54.1**,

$$F_{cr} = F_y\left[1 - \frac{F_y}{4\pi^2 E}\left(\frac{KL}{r}\right)^2\right] = 50\left[1 - \frac{50}{4\pi^2 E}(18.1)^2\right] = 49.28 \text{ ksi}$$

$$\frac{456}{13.2} = 34.55; \ \frac{34.55}{49.28} = 0.70 < 1.00 \text{ OK}$$

Girder Stress Check Section 6-6 G1 Node 97
Transversely Stiffened Web - Bearing Stiffener Design

This location has the largest total reaction at an interior support (Appendix B).

Load	Reaction (kips)			
Steel	81	x 1.3	=	105
Deck	319	x 1.3	=	415
Supim.	132	x 1.3	=	172
HS25	226	x 1.3 x 5/3	=	490
Total Reaction	758			1,182

Design the bearing stiffeners according to the provisions of Article 6.7.
Try 2-Bars 11" x 1"; Bearing area = 2 x (11 - 1)(1.0) = 20.0 in^2
Compute the effective area of the web (t_w = 0.625 in).

$$18 \times 0.625^2 = 7.03 \quad (A_w)$$
$$+ \underline{20.00}$$
$$A = 27.03 \text{ in}^2$$

$$\frac{b_s}{t_s} = \frac{11.0}{1} = 11.0 < 11.6 \text{ OK}$$

Compute the required bearing area. Bearing strength of milled stiffeners is $1.35 F_y = 67.5$ ksi.

$$\frac{1{,}182}{67.5} = 17.51 \text{ in}^2 < 20.00 \text{ in}^2 \text{ OK}$$

Compute the radius of gyration of the pair of stiffeners.

$$I = \frac{22.625^3 \times 1.00}{12} = 965 \text{ in}^4; \; r = \sqrt{\frac{I}{A}} = \sqrt{\frac{965}{27.03}} = 5.98 \text{ in}$$

Set K = 0.75; L = 84 in; KL/r = 0.75 x 84/5.98 = 10.54 < $\sqrt{\frac{2\pi^2 E}{F_y}} = 107.0$

<u>Girder Stress Check Section 6-6 G1 Node 97</u>
<u>Transversely Stiffened Web - Bearing Stiffener Design</u>

Assume the concentrated load is applied concentrically to the stiffener in this case:

$$F_{cr} = F_y\left[1 - \frac{F_y}{4\pi^2 E}\left(\frac{KL}{r}\right)^2\right] = 50\left[1 - \frac{50}{4\pi^2 E}(10.54)^2\right] = 49.76 \text{ ksi}$$

$$\frac{1{,}182}{27.03} = 43.73 \text{ ksi}; \ \frac{43.73}{49.76} = 0.88 < 1.00 \text{ OK}$$

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

Check the longitudinally stiffened web at this section for the non-composite load due to the steel weight and Cast#1. Use the moments for the transversely stiffened web design from Table D1.

Compute the vertical bending stress at the top of the web with a load factor = 1.4, as specified in Article 3.3. Use the section properties for the longitudinally stiffened web design from Table D11.

The longitudinal stiffener is located 18 in. from the top flange; d_s = 18 in.

Check the web in positive bending according to the provisions of Article 6.4.1.

$$f_{top\ web} = \frac{4{,}593 \times (48.71 - 1.0)}{116{,}890} \times 12 \times 1.4 = -31.49\ \text{ksi}$$

Compute the critical web bend-buckling stress.

$$F_{cr} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} \le F_y \qquad \text{Eq (6-10)}$$

$$\frac{d_s}{D_c} = \frac{18}{47.71} = 0.38 < 0.40\text{, hence use Eq (6-12)}$$

$$k = 11.64\left(\frac{D}{D_c - d_s}\right)^2 \ge 9\left(\frac{D}{D_c}\right)^2 \ge 7.2 \qquad \text{Eq (6-12)}$$

$$k = 11.64\left(\frac{84}{47.71 - 18.00}\right)^2 = 93.05 > 9\left(\frac{84}{47.71}\right)^2 = 27.90 > 7.2\ \text{OK}$$

$$F_{cr} = \frac{0.9 \times 29{,}000 \times 93.05}{\left(\frac{84}{0.4375}\right)^2} = 65.88 > 50.0\ \text{ksi} \quad \therefore F_{cr} = 50\ \text{ksi}$$

$$\frac{|-31.49|}{50.0} = 0.63 < 1.00$$

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

Check the strength of the longitudinally stiffened web at this section in bending according to the provisions of Article 6.4.1. The longitudinal stiffener is located 18 inches from the top flange; d_s = 18 in.

Use the moments from Table D1. Use the section properties from Table D11. Compute the factored vertical bending stress at the top and bottom of the web due to dead and live load.

$$f_{top\ web} = \left(\frac{3{,}343 \times 47.71}{116{,}890} + \frac{1{,}093 \times 28.47}{214{,}623} + \frac{3{,}968(5/3) \times 13.57}{290{,}835} \right) \times 12 \times 1.3 = -28.36 \text{ ksi}$$

$$f_{bot\ web} = \left(\frac{3{,}343 \times 36.29}{116{,}890} + \frac{1{,}093 \times 55.53}{214{,}623} + \frac{3{,}968(5/3) \times 70.43}{290{,}835} \right) \times 12 \times 1.3 = 45.59 \text{ ksi} < F_y \text{ OK}$$

Locate the effective D_c from the factored stresses in the top and bottom of the web.

$$D_c = 84 \times \left(\frac{|-28.36|}{|-28.36| + 45.59} \right) = 32.21 \text{ in}$$

$$\frac{d_s}{D_c} = \frac{18}{32.21} = 0.56 > 0.40, \text{ hence use Eq (6-11)}$$

$$F_{cr} = \frac{0.9Ek}{\left(\frac{D}{t_w} \right)^2} \leq F_y;\ k = 5.17\left(\frac{D}{d_s} \right)^2 \geq 9\left(\frac{D}{D_c} \right)^2 \geq 7.2 \qquad \text{Eq (6-10); Eq (6-11)}$$

$$k = 5.17 \times \left(\frac{84}{18} \right)^2 = 112.6 \geq 9\left(\frac{84}{32.21} \right)^2 = 61.21 > 7.2 \text{ OK}$$

$$F_{cr} = \frac{0.9 \times 29{,}000 \times 112.6}{\left(\frac{84}{0.4375} \right)^2} = 79.7 \text{ ksi} > 50.0 \text{ ksi} \therefore F_{cr} = 50 \text{ ksi}$$

$$\frac{|-28.36|}{50.0} = 0.57 < 1.00 \text{ OK}$$

Girder Stress Check Section 3-3 G4 Node 64
Longitudinally Stiffened Web - Fatigue - Top Flange

Since stress reversal occurs at this section, the top and bottom of the girder must be examined. To determine if a single longitudinal stiffener is sufficient at this section, constructibility, fatigue, overload and strength must to be checked at this point.

The uncracked composite section is to be used in the computation of fatigue stresses according to the provisions of Article 4.5.2. Use the section properties from Table D11.

Load	Moment (k-ft)	Source
Steel	213	Table D1
Deck	802	Table D1
Total Non-Composite DL	1,015	
Superimposed DL	381	Table D1
Live load HS25	3,400	Table D1
M_{min}	-627	
M_{max}	966	
M_{range}	1,593	

Compute the maximum unfactored dead load vertical bending stresses in the top and bottom of the web of the steel girder.

$$f_{top\ web} = \left[\frac{1{,}015 \times 47.71}{116{,}890} + \frac{381 \times 28.47}{214{,}623}\right] \times 12 = -5.58 \text{ ksi}$$

$$f_{bot\ web} = \left[\frac{1{,}015 \times (37.79-1.50)}{116{,}890} + \frac{381 \times (57.03-1.50)}{214{,}623}\right] \times 12 = 4.96 \text{ ksi}$$

Compute the vertical bending stress at the bottom of the web due to the positive fatigue moment.

$$f_{bot\ web} = \frac{966 \times (71.93 - 1.50)}{290{,}835} \times 12 = 2.81 \text{ ksi}$$

Compute the vertical bending stress at the top of the web due to the negative fatigue moment.

$$f_{top\ web} = \frac{-627 \times 13.57}{290{,}835} \times 12 = 0.35 \text{ ksi}$$

Check if two times the tensile fatigue stress overcomes the dead load stress in the top of the web.

Two times the tensile fatigue stress = 2 x 0.35 = 0.70 ksi

Girder Stress Check Section 3-3 G4 Node 64
Longitudinally Stiffened Web - Fatigue - Bottom Flange

$|-5.58|$ ksi > 0.70 ksi, therefore fatigue need not be checked at the bottom of the top flange.

Check the base metal at the top of the bottom flange at this section adjacent to the transverse stiffener fillet weld to the flange for fatigue (Category C').

Compute the live load fatigue stress range due to vertical bending at the bottom of the web.

$$f_{bot\ range} = \frac{1{,}593 \times (71.93 - 1.50)}{290{,}835} \times 12 = 4.63 \text{ ksi}$$

The lateral flange bending stress at the transverse stiffener must be considered since the stiffeners are welded to the tension flange. Assume that the transverse stiffeners are 6 in wide. Compute the fatigue stress range at 6 in from the center of the web due to lateral flange bending.

Compute the moment of inertia of the flange about a vertical axis in the plane of the web.

$$I_{flg} = \frac{1}{12} 22^3 \times 1.5 = 1{,}331 \text{ in}^4$$

Compute the range of lateral flange moment due to curvature.

$$M_{lat} = \frac{6}{5}\frac{M\ell^2}{RD} = \frac{6}{5}\frac{1{,}593 \times 20.5^2}{717 \times 84} = 13.34 \text{ k-ft} \qquad \text{Eq (4-1)}$$

Compute the stress range due to lateral flange bending.

$$f_{fat} = \frac{13.34 \times 6}{1{,}331} \times 12 = 0.72 \text{ ksi}$$

Total stress range = 4.63 + 0.72 = 5.35 ksi

$$\frac{5.35}{6.0} = 0.89 < 1.0 \text{ OK}$$

Girder Stress Check Section 3-3 G4 Node 64
Longitudinally Stiffened Web - Constructibility - Web

Check the longitudinally stiffened web for Cast #1, which ends at this point, Section 3-3. The section is non-composite for this condition.
From Table D1, the non-composite moment = 213 + 2,554 = 2,767 k-ft
The section properties are from Table D11. The neutral axis is 47.71 in from the top of the web. Compute the factored vertical bending stress at the top of the web.

$$f_{top\ web} = \left(\frac{2{,}767 \times 47.71}{116{,}890}\right) \times 12 \times 1.4 = -18.97 \text{ ksi}$$

Locate the longitudinal stiffener 42 in from the top flange; d_s = 42 in.

$$F_{cr} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} \le F_y \qquad \text{Eq (6-10)}$$

$$\frac{d_s}{D_c} = \frac{42}{47.71} = 0.88 > 0.40\text{, hence use Eq (6-11)}$$

$$k = 5.17\left(\frac{D}{d_s}\right)^2 = 5.17 \times \left(\frac{84}{42}\right)^2 = 20.7 \qquad \text{Eq (6-11)}$$

Compute k from Equation (6-8) for comparison and use the larger value according to the provisions of Article 6.4.1.

$$k = 9\left(\frac{D}{D_c}\right)^2 = 9 \times \left(\frac{84}{47.71}\right)^2 = 27.9 > 20.7$$

$$F_{cr} = \frac{0.9 \times 29{,}000 \times 27.9}{\left(\frac{84}{0.4375}\right)^2} = 19.75 \text{ ksi} < F_y$$

$$\frac{|-18.97|}{19.75} = 0.96 < 1.00 \text{ OK}$$

<u>Girder Stress Check Section 3-3 G4 Node 64</u>
<u>Longitudinally Stiffened Web - Constructibility - Deck</u>

Cast #2

Check the deck stress due to Cast #2, which ends at Section 3-3.
Assume the section is composite for this condition. The moment due to Cast#2 acts on the composite section. Ignore the stress in the girder due to non-composite loads.

From Table D1, M = -3,113 k-ft. Compute the stress in the top of the deck. Assume no creep. Use n = 7.56, as specified in Article 13.3. The distance from the neutral axis to the extreme fiber = 14.57 + 3 + 9 = 26.57 in.

$$f_{deck} = \left(\frac{-3{,}113 \ \text{x} \ 26.57}{290{,}835} \right) \ \text{x} \ 12 \ \text{x} \ \frac{1.4}{7.56} = 0.63 \ \text{ksi}$$

The stress in the deck probably is not this high at the free end of the deck placed in Cast#1. It is likely that the composite section does not remain plane at this location and some slip occurs between the deck and the top flange. Some overstress likely occurs in the shear connectors.

Nevertheless, since the tensile stress in the deck due to the factored construction loads exceeds 0.9 times the modulus of rupture, Article 2.4.3 requires that longitudinal reinforcement equal to at least one percent of the total cross sectional area of the deck be placed in the deck at this location. The reinforcement is to be No. 6 bars or smaller, spaced at not more than 12 inches.

Girder Stress Check Section 3-3 G4 Node 64
Longitudinally Stiffened Web - Bending Strength - Web

Positive live load bending case

Try the longitudinal stiffener at 42 in from the top flange; d_s=42 in. Check the strength of the web at this section for the positive live load bending case according to the provisions of Article 6.4.1.

Use the moments from Table D1 and the section properties from Table D11.
The composite section is assumed uncracked for this condition according to the provisions of Article 4.5.2.

Compute the factored vertical bending stress at the top and bottom of the web due to dead and live load.

$$f_{top\ web} = \left[\frac{1{,}015 \times 47.71}{116{,}890} + \frac{381 \times 28.47}{214{,}623} + \frac{3{,}400(5/3) \times 13.57}{290{,}835}\right] \times 12 \times 1.3 = -11.38 \text{ ksi}$$

$$f_{bot\ web} = \left[\frac{1{,}015 \times 36.29}{116{,}890} + \frac{381 \times 55.53}{214{,}623} + \frac{3{,}400(5/3) \times 70.43}{290{,}835}\right] \times 12 \times 1.3 = 27.86 \text{ ksi} < F_y \text{ OK}$$

Compute the effective D_c.

$$D_c = 84 \times \left(\frac{|-11.38|}{|-11.38| + 27.86}\right) = 24.36 \text{ in from the top flange}$$

Compute the ratio of the stiffener distance from the compression flange, d_s, to the depth of the web in compression, D_c, to determine the method of computing k.

$$\frac{d_s}{D_c} = \frac{42}{24.36} = 1.72 > 0.40$$

Compute k according to the provisions of Article 6.4.1.

$$k = 5.17\left(\frac{D}{d_s}\right)^2 = 5.17 \times \left(\frac{84}{42}\right)^2 = 20.7 \qquad \text{Eq (6-11)}$$

Check k for the transversely stiffened case according to the provisions of Article 6.4.1.

Girder Stress Check Section 3-3 G4 Node 64
Longitudinally Stiffened Web - Bending Strength - Web

$$k = 9 \times \left(\frac{84}{24.36}\right)^2 = 107.0 > 20.7$$

Use k for the transversely stiffened case.

$$F_{cr} = \frac{0.9 \times 29{,}000 \times 107.0}{\left(\frac{84}{0.4375}\right)^2} = 75.8 \text{ ksi} > 50.0 \text{ ksi} \therefore F_{cr} = 50 \text{ ksi}$$

$$\frac{|-11.38|}{50.0} = 0.23 < 1.00 \text{ OK}$$

Negative live load bending case

Longitudinal stiffener is located 42 in from the top flange; d_s=42 in. Check the strength of the web at this section for the negative live load bending case according to the provisions of Article 6.4.1.

Use the moments from Table D1 and the section properties from Table D11. The composite section is assumed cracked for this condition according to the provisions of Article 4.5.2.

The compressive deck stress due to the factored superimposed dead load is overcome by the tensile deck stress due to the factored live load plus impact. Compute the factored vertical bending stress at the top and bottom of the web due to dead and live load.

$$f_{top\ web} = \left[\frac{1{,}015 \times 47.71}{116{,}890} + \frac{381 \times 47.71}{116{,}890} + \frac{-2{,}590(5/3) \times 47.71}{116{,}890}\right] \times 12 \times 1.3 = 18.60 \text{ ksi} < F_y \text{ OK}$$

$$f_{bot\ web} = \left[\frac{1{,}015 \times 36.29}{116{,}890} + \frac{381 \times 36.29}{116{,}890} + \frac{-2{,}590(5/3) \times 36.29}{116{,}890}\right] \times 12 \times 1.3 = -14.14 \text{ ksi}$$

$$D_c = 84 \times \left[\frac{|-14.14|}{|-14.14| + 18.60}\right] = 36.28 \text{ in from the bottom flange}$$

Girder Stress Check Section 3-3 G4 Node 64
Longitudinally Stiffened Web - Bending Strength - Web

$$\frac{d_s}{D_c} = \frac{42.0}{36.28} = 1.16 > 0.40$$

$$k = 5.17 \times \left(\frac{D}{d_s}\right)^2 = 5.17 \times \left(\frac{84}{42}\right)^2 = 20.7$$ Eq (6-11)

Check k for the transversely stiffened case.

$$k = 9 \times \left(\frac{84}{36.28}\right)^2 = 48.25 > 20.7$$

$$F_{cr} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} \leq F_y$$ Eq (6-10)

$$F_{cr} = \frac{0.9 \times 29{,}000 \times 48.25}{\left(\frac{84}{0.4375}\right)^2} = 34.16 \text{ ksi}$$

$$\frac{|-14.14|}{34.16} = 0.41 < 1.00 \text{ OK}$$

Girder Stress Check Section 3-3 G4 Node 64
Longitudinally Stiffened Web - Overload - Web

Positive live load bending case

Use the uncracked composite section to check the web for overload at this section for the positive live load bending case according to Article 9.5. The longitudinal stiffener is located 42 in from the top flange; d_s=42 in.

Compute the top and bottom web vertical bending stress due to the dead load plus the positive live load overload moment.

$$f_{top\ web} = \left[\frac{1{,}015 \times 47.71}{116{,}890} + \frac{381 \times 28.47}{214{,}623} + \frac{2{,}720(5/3) \times 13.57}{290{,}835}\right] \times 12 = -8.12 \text{ ksi}$$

$$f_{bot\ web} = \left[\frac{1{,}015 \times 36.29}{116{,}890} + \frac{381 \times 55.53}{214{,}623} + \frac{2{,}720(5/3) \times 70.43}{290{,}835}\right] \times 12 = 18.14 \text{ ksi}$$

Find the effective D_c.

$$D_c = 84 \times \left(\frac{|-8.12|}{|-8.12| + 18.14}\right) = 26.0 \text{ in from the top flange}$$

$$\frac{d_s}{D_c} = \frac{42}{26.0} = 1.62 > 0.40\text{, hence}$$

$$k = 5.17 \times \left(\frac{D}{d_s}\right)^2 = 5.17 \times \left(\frac{84}{42}\right)^2 = 20.7 \qquad \text{Eq (6-11)}$$

Check k for the transversely stiffened case.

$$k = 9 \times \left(\frac{84}{26.0}\right)^2 = 93.9 > 20.7$$

$$F_{cr} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} \le F_y \qquad \text{Eq (6-10)}$$

Girder Stress Check Section 3-3 G4 Node 64
Longitudinally Stiffened Web - Overload - Web

Article 9.5 states that the vertical bending stress at overload is not to exceed $0.95F_y$ in continuously braced flanges of composite girders. This limitation is extended to the web compressive stress for this case.

$$F_{cr} = \frac{0.9 \times 29{,}000 \times 93.9}{\left(\frac{84}{0.4375}\right)^2} = 66.5 \text{ ksi} > 0.95F_y = 47.5 \text{ ksi} \therefore F_{cr} = 47.5 \text{ ksi}$$

$$\frac{|-8.12|}{47.5} = 0.17 < 1.00 \text{ OK}$$

Girder Stress Check Section 3-3 G4 Node 64
Longitudinally Stiffened Web - Overload - Web

Negative live load bending case

Use the uncracked composite section to check the web for overload at this section for the negative live load bending case according to Article 9.5. The longitudinal stiffener is located 42 in from the top flange; d_s=42 in.

Compute the top and bottom web vertical bending stress due to the dead load plus the negative live load overload moment.

$$f_{top\ web} = \left[\frac{1{,}015 \times 47.71}{116{,}890} + \frac{381 \times 28.47}{214{,}623} + \frac{-2{,}072(5/3) \times 13.57}{290{,}835}\right] \times 12 = -3.64 \text{ ksi}$$

$$f_{bot\ web} = \left[\frac{1{,}015 \times 36.29}{116{,}890} + \frac{381 \times 55.53}{214{,}623} + \frac{-2{,}072(5/3) \times 70.43}{290{,}835}\right] \times 12 = -5.07 \text{ ksi}$$

Since the web is entirely in compression, set k = 7.2 (see C6.3.1). Compute the critical web bend-buckling stress. The bottom flange is used because it is partially braced rather than the top flange, which is encased in concrete. Also, the stress in the bottom of the web is larger than in the top of the web. The largest stress magnitude is assumed to be critical in this case because the equation for k is based on the assumption that a uniform compressive stress is applied to the web.

$$F_{cr} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} = \frac{0.9 \times 29{,}000 \times 7.2}{\left(\frac{84}{0.4375}\right)^2} = 5.10 \text{ ksi}$$

$$\frac{|-5.07|}{5.10} = 0.99 < 1.00 \text{ OK}$$

Girder Stress Check Section 4-4 G4 Node 76
Longitudinally Stiffened Web - Constructibility - Web

The longitudinal stiffener is located 56 in from the top flange at this section; d_s=56 in. The section is non-composite.

Compute the factored vertical bending stress in the top and bottom of the web due to Cast #1. The constructibility load factor equals 1.4 according to the provisions of Article 3.3.

Moments are from Table D1. Moment = -290 + 1,023 = 733 k-ft
Section properties are from Table D11.

$$f_{top\ web} = \left(\frac{733 \times 47.71}{116{,}890}\right) \times 12 \times 1.4 = -5.03 \text{ ksi}$$

$$f_{bot\ web} = \left(\frac{733 \times 36.29}{116{,}890}\right) \times 12 \times 1.4 = 3.82 \text{ ksi}$$

Compute the effective D_c.

$$D_c = 84 \times \left(\frac{|-5.03|}{|-5.03| + 3.82}\right) = 47.74 \text{ in from the top flange}$$

Compute the ratio of d_s to D_c to determine the method of computing k.

$$\frac{d_s}{D_c} = \frac{56}{47.74} = 1.17 > 0.40$$

Therefore, compute k as follows from Eq (6-11)

$$k = 5.17\left(\frac{D}{d_s}\right)^2 = 5.17 \times \left(\frac{84}{56}\right)^2 = 11.63 \qquad \text{Eq (6-11)}$$

Compute k for the transversely stiffened case.

$$k = 9\left(\frac{D}{D_c}\right)^2 = 9 \times \left(\frac{84}{47.74}\right)^2 = 27.86 > 11.63$$

Girder Stress Check Section 4-4 G4 Node 76
Longitudinally Stiffened Web - Constructibility - Web

$$F_{cr} = \frac{0.9 \times 29{,}000 \times 27.86}{\left(\dfrac{84}{0.4375}\right)^2} = 19.73 \text{ ksi} \qquad \text{Eq (6-8)}$$

$$\frac{|-5.03|}{19.73} = 0.25 < 1.00 \text{ OK}$$

Girder Stress Check Section 4-4 G4 Node 76
Longitudinally Stiffened Web - Bending Strength - Web

Positive live load bending case

Since this section is potentially subjected to stress reversal, check both positive and negative live load bending conditions. First, check the strength of the longitudinally stiffened web at this section for the positive live load bending case according to the provisions of Article 6.4.1.

Use the moments from Table D1 and the section properties from Table D11.
The composite section is assumed uncracked for the positive live load moment according to the provisions of Article 4.5.2. Compute the factored vertical bending stress in the top and bottom of the web due to dead and live load.

$$f_{top\ web} = \left[\frac{-1{,}572 \times 47.71}{116{,}890} + \frac{-411 \times 46.12}{124{,}931} + \frac{2{,}297\,(5/3) \times 13.57}{290{,}835}\right] \times 12 \times 1.3 = 9.59 \text{ ksi} < F_y \text{ OK}$$

$$f_{bot\ web} = \left[\frac{-1{,}572 \times 36.29}{116{,}890} + \frac{-411 \times 37.88}{124{,}931} + \frac{2{,}297(5/3) \times 70.43}{290{,}835}\right] \times 12 \times 1.3 = 4.90 \text{ ksi} < F_y \text{ OK}$$

The web is entirely in tension. Therefore, web bend-buckling need not be checked for this case. A second longitudinal stiffener is not required.

Negative live load bending case

The longitudinal stiffener is located 56 inches from the top flange; $d_s = 28$ in. Check the strength of the web at this section for the negative live load bending case according to the provisions of Article 6.4.1.

Use the moments from Table D1 and the section properties from Table D11.
The composite section is assumed cracked for the negative live load moment according to the provisions of Article 4.5.2. Compute the factored vertical bending stress in the top and bottom of the web due to dead and live load.

$$f_{top\ web} = \left[\frac{-1{,}572 \times 47.71}{116{,}890} + \frac{-411 \times 46.12}{124{,}931} + \frac{-2{,}923(5/3) \times 43.20}{139{,}710}\right] \times 12 \times 1.3 = 35.88 \text{ ksi} < F_y \text{ OK}$$

$$f_{bot\ web} = \left[\frac{-1{,}572 \times 36.29}{116{,}890} + \frac{-411 \times 37.88}{124{,}931} + \frac{-2{,}923(5/3) \times 40.80}{139{,}710}\right] \times 12 \times 1.3 = -31.75 \text{ ksi}$$

<u>Girder Stress Check Section 4-4 G4 Node 76</u>
<u>Longitudinally Stiffened Web - Bending Strength - Web</u>

Compute D_c using the factored stresses in the top and bottom of the web.

$$D_c = 84 \times \left(\frac{|-31.75|}{|-31.75| + 35.88} \right) = 39.44 \text{ in from the bottom flange}$$

$$\frac{d_s}{D_c} = \frac{28.0}{39.44} = 0.71 > 0.40, \text{ hence}$$

$$k = 5.17\left(\frac{D}{d_s}\right)^2 = 5.17 \times \left(\frac{84}{28}\right)^2 = 46.53$$

Check k for the transversely stiffened case.

$$k = 9 \times \left(\frac{84}{39.44}\right)^2 = 40.83 < 46.53$$

$$F_{cr} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} \le F_y \qquad \text{Eq (6-10)}$$

$$F_{cr} = \frac{0.9 \times 29{,}000 \times 46.53}{\left(\frac{84}{0.4375}\right)^2} = 32.94 \text{ ksi}$$

$$\frac{|-31.75|}{32.94} = 0.96 < 1.00 \text{ OK}$$

Girder Stress Check Section 4-4 G4 Node 76
Longitudinally Stiffened Web - Overload - Web

Positive live load bending case

Check the longitudinally stiffened web for bend-buckling at overload at this section according to the provisions of Article 9.5, which refers to the provisions of Article 6.4.1. First, check the positive live load bending case.

The longitudinal stiffener is located 56 in. from the top flange. The composite section is assumed uncracked at overload according to the provisions of Article 9.5. Compute the overload vertical bending stress in the top and bottom of the web.

$$f_{top\ web} = \left[\frac{-1{,}572 \times 47.71}{116{,}890} + \frac{-411 \times 28.47}{214{,}623} + \frac{1{,}838(5/3) \times 13.57}{290{,}835}\right] \times 12 = 6.64 \text{ ksi}$$

$$f_{bot\ web} = \left[\frac{-1{,}572 \times 36.29}{116{,}890} + \frac{-411 \times 55.53}{214{,}623} + \frac{1{,}838(5/3) \times 70.43}{290{,}835}\right] \times 12 = 1.77 \text{ ksi}$$

The web is entirely in tension. Therefore, web bend-buckling need not be checked for this case.

Girder Stress Check Section 4-4 Node 76
Longitudinally Stiffened Web - Overload - Web

Negative live load bending case

Next, check the negative live load bending case. The longitudinal stiffener is located 56 in from the top flange; $d_s = 28$ in. The composite section is assumed uncracked at overload according to the provisions of Article 9.5. Compute the overload vertical bending stress in the top and bottom of the web.

$$f_{top\ web} = \left[\frac{-1{,}572 \times 47.71}{116{,}890} + \frac{-411 \times 28.47}{214{,}623} + \frac{-2{,}338(5/3) \times 13.57}{290{,}835}\right] \times 12 = 10.54 \text{ ksi}$$

$$f_{bot\ web} = \left[\frac{-1{,}572 \times 36.29}{116{,}890} + \frac{-411 \times 55.53}{214{,}623} + \frac{-2{,}338(5/3) \times 70.43}{290{,}835}\right] \times 12 = -18.46 \text{ ksi}$$

Find the effective D_c.

$$D_c = 84 \times \left(\frac{|-18.46|}{|-18.46| + 10.54}\right) = 53.47 \text{ in from the bottom flange}$$

$$\frac{d_s}{D_c} = \frac{28}{53.47} = 0.52 > 0.40$$

Therefore, compute k for the longitudinally stiffened web from Eq (6-11).

$$k = 5.17\left(\frac{D}{d_s}\right)^2 = 5.17 \times \left(\frac{84}{28}\right)^2 = 46.53 \qquad \text{Eq (6-11)}$$

Compute k for the transversely stiffened case from Eq (6-8).

$$k = 9\left(\frac{D}{D_c}\right)^2 = 9 \times \left(\frac{84}{53.47}\right)^2 = 22.21 < 46.53$$

$$F_{cr} = \frac{0.9 \times 29{,}000 \times 46.53}{\left(\frac{84}{0.4375}\right)^2} = 32.94 \text{ ksi}$$

$$\frac{|-18.46|}{32.94} = 0.56 < 1.00 \text{ OK}$$

Girder Stress Check Section 5-5 G4 Node 88
Longitudinally Stiffened Web - Bending Strength - Web

The longitudinal stiffener is located 66 inches from the top flange at this section; d_s = 18 in. Check the strength of the web at this section for the negative live load bending case according to the provisions of Article 6.4.1.

The composite section is assumed cracked for this condition according to the provisions of Article 4.5.2. Moments are from Table D1. Section properties are from Table D11. Compute the factored vertical bending stress in the top and bottom of the web due to dead and live load.

$$f_{\text{top web}} = \left[\frac{-4{,}879 \times 44.18}{163{,}684} + \frac{-1{,}457 \times 43.00}{170{,}780} + \frac{-3{,}721(5/3) \times 40.78}{184{,}047}\right] \times 12 \times 1.3 = 47.70 \text{ ksi} < F_y \text{ OK}$$

$$f_{\text{bot web}} = \left[\frac{-4{,}879 \times 39.82}{163{,}684} + \frac{-1{,}457 \times 41.00}{170{,}780} + \frac{-3{,}721(5/3) \times 43.22}{184{,}047}\right] \times 12 \times 1.3 = -46.69 \text{ ksi}$$

Compute D_c using the factored stresses in the top and bottom of the web.

$$D_c = 84 \times \left(\frac{|-46.69|}{|-46.69| + 47.70}\right) = 41.55 \text{ in from the bottom flange}$$

Compute the ratio of d_s to D_c to determine the method of computing k.

$$\frac{d_s}{D_c} = \frac{18}{41.55} = 0.43 > 0.40\text{, hence use Eq (6-11)}$$

$$F_{cr} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} \le F_y;\ k = 5.17\left(\frac{D}{d_s}\right)^2 \ge 9\left(\frac{D}{D_c}\right)^2 \ge 7.2 \qquad \text{Eq (6-10); Eq (6-11)}$$

$$k = 5.17\left(\frac{D}{d_s}\right)^2 = 5.17 \times \left(\frac{84}{18}\right)^2 = 112.6 > 9\left(\frac{84}{41.55}\right)^2 = 36.78 > 7.2$$

$$F_{cr} = \frac{0.9 \times 29{,}000 \times 112.6}{\left(\frac{84}{0.4375}\right)^2} = 79.7 \text{ ksi} > 50.0 \text{ ksi} \therefore F_{cr} = 50 \text{ ksi}$$

$$\frac{|-46.69|}{50.0} = 0.93 < 1.00 \text{ OK}$$

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

The longitudinal stiffener is located 66 inches from the top flange at this section; d_s = 18 in. Check the strength of the web at this section according to the provisions of Article 6.4.1.

The composite section is assumed cracked according to the provisions of Article 4.5.2. Moments are from Table D1. Section properties are from Table D11.

Compute the factored vertical bending stress in the top and bottom of the web due to dead and live load.

$$f_{top\ web} = \left[\frac{-9{,}189 \times 44.68}{314{,}783} + \frac{-3{,}015 \times 43.97}{322{,}084} + \frac{-6{,}155(5/3) \times 42.59}{336{,}106}\right] \times 12 \times 1.3 = 47.05 \text{ ksi} < F_y \text{ OK}$$

$$f_{bot\ web} = \left[\frac{-9{,}189 \times 39.32}{314{,}783} + \frac{-3{,}015 \times 40.03}{322{,}084} + \frac{-6{,}155(5/3) \times 41.41}{336{,}106}\right] \times 12 \times 1.3 = -43.47 \text{ ksi}$$

Compute D_c using the factored stresses in the top and bottom of the web.

$$D_c = 84 \times \left(\frac{|-43.47|}{|-43.47| + 47.05}\right) = 40.34 \text{ in from the bottom flange}$$

Compute the ratio of d_s to D_c to determine the method of computing k.

$$\frac{d_s}{D_c} = \frac{18}{40.34} = 0.45 > 0.40, \text{ hence use Eq (6-11)}$$

$$F_{cr} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} \le F_y;\ k = 5.17\left(\frac{D}{d_s}\right)^2 \ge 9\left(\frac{D}{D_c}\right)^2 \ge 7.2 \qquad \text{Eq (6-10); Eq (6-11)}$$

$$k = 5.17\left(\frac{D}{d_s}\right)^2 = 5.17 \times \left(\frac{84}{18}\right)^2 = 112.6 > 9\left(\frac{84}{40.34}\right)^2 = 39.02 > 7.2$$

$$F_{cr} = \frac{0.9 \times 29{,}000 \times 112.6}{\left(\frac{84}{0.4375}\right)^2} = 79.7 \text{ ksi} > 50.0 \text{ ksi} \therefore F_{cr} = 50 \text{ ksi}$$

$$\frac{|-43.47|}{50.0} = 0.87 < 1.00 \text{ OK}$$

<u>Girder Stress Check Section 6-6 G4 Node 100</u>
<u>Longitudinally Stiffened Web - Overload - Web</u>

The longitudinal stiffener is located 66 inches from the top flange at this section; d_s = 18 in. Check the web for bend-buckling at overload at this section according to the provisions of Article 9.5, which refers to the provisions of Article 6.4.1.

The composite section is assumed uncracked at overload according to the provisions of Article 9.5. Compute the overload vertical bending stress in the top and bottom of the web.

$$f_{top\ web} = \left[\frac{-9{,}189 \times 44.68}{314{,}783} + \frac{-3{,}015 \times 34.28}{421{,}112} + \frac{-4{,}924(5/3) \times 22.31}{543{,}850}\right] \times 12 = 22.64 \text{ ksi}$$

$$f_{bot\ web} = \left[\frac{-9{,}189 \times 39.32}{314{,}783} + \frac{-3{,}015 \times 49.72}{421{,}112} + \frac{-4{,}924(5/3) \times 61.69}{543{,}850}\right] \times 12 = -29.22 \text{ ksi}$$

Compute D_c using the overload stresses in the top and bottom of the web.

$$D_c = 84 \times \left(\frac{|-29.22|}{|-29.22| + 22.64}\right) = 47.33 \text{ in from the bottom flange}$$

Compute the ratio of d_s to D_c to determine the method of computing k.

$$\frac{d_s}{D_c} = \frac{18}{47.33} = 0.38 < 0.40, \text{ hence use Eq (6-12)}$$

$$k = 11.64\left(\frac{D}{D_c - d_s}\right)^2 = 11.64 \times \left(\frac{84}{47.33 - 18}\right)^2 = 95.47 \geq 9\left(\frac{D}{D_c}\right)^2 = 28.35 > 7.2$$

Article 9.5 states that the vertical bending stress at overload in partially braced compression flanges is not to exceed the critical flange stress given by Eq (5-8). This limitation is extended to the web compressive stress for this case. Separate calculations similar to those illustrated on pages 123-124 are used to compute F_{cr1} = 36.84 ksi for the bottom flange at this section.

$$F_{cr} = \frac{0.9 \times 29{,}000 \times 95.47}{\left(\frac{84}{0.4375}\right)^2} = 67.6 \text{ ksi} > F_{cr1} = 36.84 \text{ ksi} \quad \therefore F_{cr} = 36.84 \text{ ksi}$$

$$\frac{|-29.22|}{36.84} = 0.79 < 1.00 \text{ OK}$$

Girder Stress Check G4
Longitudinally Stiffened Web - Longitudinal Stiffener Design

Design the longitudinal stiffener according to the provisions of Article 6.6. Size the longitudinal stiffener assuming the actual transverse stiffener spacing d_o is equal to the maximum permissible spacing of D, which will require the maximum size longitudinal stiffener.

$$I_{\ell s} \geq Dt_w^3(2.4a^2 - 0.13)\beta \qquad \text{Eq (6-19)}$$

Since the stiffener is on the side of the web away from the center of curvature,

$\beta = \frac{Z}{6} + 1.$

$$Z = \frac{0.079 d_o^2}{Rt_w} \leq 10 \qquad \text{Eq (6-18)}$$

$$Z = \frac{0.079 \times 84^2}{717 \times 0.4375} = 1.78 < 10$$

$$\beta = \frac{1.78}{6} + 1 = 1.30$$

$$a = \frac{d_o}{D} = \frac{84}{84} = 1.0$$

The required moment of inertia of the longitudinal stiffener is therefore:

$$I_{\ell s} = 84 \times 0.4375^3 \times (2.4 \times 1^2 - 0.13) \times 1.30 = 20.76 \text{ in}^4$$

According to the provisions of Article 6.6, consider a bar section acting with a web width equal to $18t_w$.

Try: Bar 7" x 0.625":

Check the width-to-thickness ratio of the stiffener according to Eq (6-13):

$$\frac{b_s}{t_s} = \frac{7}{0.625} = 11.2$$

$$\frac{b_s}{t_s} = 0.48\sqrt{\frac{E}{F_y}} = 0.48\sqrt{\frac{29{,}000}{50}} = 11.56 > 11.2 \text{ OK} \qquad \text{Eq (6-13)}$$

Girder Stress Check G4
Longitudinally Stiffened Web - Longitudinal Stiffener Design

$A_{web} = 18 \times 0.4375^2 = 3.45$ in; $A_s = 7 \times 0.625 = 4.38$ in^2

$A_{tot} = 3.45 + 4.38 = 7.83$ in^2

Compute the moment of inertia of the longitudinal stiffener.

First, determine the location of the neutral axis from the outside tip of the longitudinal stiffener.

$$\bar{y} = \frac{4.38 \times 3.5 + 3.45 \times 7.219}{7.83} = 5.14 \text{ in}$$

Compute the moment of inertia.

$$I = \frac{1}{12}(0.625 \times 7^3) + 4.38 \times (5.14 - 3.5)^2 + 3.45 \times (7.219 - 5.14)^2 = 44.56 \text{ in}^4$$

44.56 in^4 > 20.76 in^4 OK

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

Compute the shear strength of the longitudinally stiffened web at this section according to the provisions of Article 6.4.2.

The maximum factored shear at this section = 645 kips per Table D2. $t_w = 0.4375$ in.

$$V_{cr} = CV_p \qquad \text{Eq (6-4)}$$

$$V_p = 0.58\, F_y D t_w = 0.58 \times 50 \times 84 \times 0.4375 = 1{,}066 \text{ k}$$

Try a required spacing d = 41 in.

$$k_w = 5 + 5\left(\frac{D}{d}\right)^2 = 5 + 5\left(\frac{84}{41}\right)^2 = 26 \qquad \text{Eq (6-9)}$$

$$\frac{D}{t_w} = \frac{84}{0.4375} = 192$$

$$1.38\sqrt{\frac{Ek_w}{F_y}} = 1.38\sqrt{\frac{29{,}000 \times 26}{50}} = 169 < 192$$

$$C = \frac{1.52Ek_w}{\left(\frac{D}{t_w}\right)^2 F_y} = \frac{1.52 \times 29{,}000 \times 26}{\left(\frac{84}{0.4375}\right)^2 \times 50} = 0.62 \qquad \text{Eq (6-7)}$$

$$V_{cr} = CV_p = 0.62 \times 1{,}066 = 661 \text{ k} > 645 \text{ k OK}$$

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

Design the transverse stiffeners in the Span 1 portion of Field Section 2 of the transversely stiffened web design according to the provisions of Article 6.5.

The entire length of all 4 girders may use an 84" stiffener spacing, as determined previously. Since the panel length between cross frames is less than 3 times the girder depth, two transverse stiffeners spaced at some distance less than 84 in may be used per panel. There are 8 panels in the end spans and 11 panels in the center span.

Number of transverse stiffeners:

Span 1	8 x 2 =	16	
Span 2	11 x 2 =	22	
Span 3	8 x 2 =	16	
Total		54	transverse stiffeners per girder

R = 717 ft, t_w = 0.625 in, d = 84 in, D = 84 in. The actual transverse stiffener spacing d_o in Span 1 is computed as [(163.77/8)/3] x 12 = 81.89 in.

Determine the minimum required stiffener size according to the provisions of Article 6.5. The minimum required width equals the larger of one-fourth of the flange width = 27/4 = 6.75 in, or

$$2" + \frac{D}{30} = 2 + \frac{84}{30} = 4.80 < 6.75 \therefore \text{ Use } 6.75 \text{ in}$$

The minimum required thickness of the stiffener is determined from Eq (6-13).

$$\frac{b_s}{t_s} \le 0.48\sqrt{\frac{E}{F_y}} = 0.48\sqrt{\frac{29{,}000}{50}} = 11.56$$

$$\text{Minimum } t_s = \frac{6.75}{11.56} = 0.58 \text{ in}$$

Try a Bar 6.75" x 0.625".

Compute the required moment of inertia of the stiffener according to Eq (6-14).

$$I_{ts} = d_o t_w^3 J \qquad \text{Eq (6-14)}$$

$$J = \left[\left(\frac{1.58}{d/D}\right)^2 - 2\right] X \ge 0.5 \qquad \text{Eq (6-15)}$$

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

$$a = \frac{d_o}{D} = \frac{81.89}{84} = 0.97$$

$$Z = \frac{0.079\,d_o^2}{Rt_w} = \frac{0.079 \times 81.89^2}{717 \times 0.625} = 1.182 < 10 \qquad \text{Eq (6-18)}$$

$$X = 1 + \left(\frac{a - 0.78}{1{,}775}\right) Z^4 \quad \text{for } a > 0.78 \qquad \text{Eq (6-17)}$$

$$X = 1 + \left(\frac{0.97 - 0.78}{1{,}775}\right) \times 1.182^4 = 1.00$$

The required stiffener spacing d was determined previously to be 84 in. Therefore:

$$\frac{d}{D} = \frac{84}{84} = 1.0$$

$$J = \left[\left(\frac{1.58}{1.0}\right)^2 - 2\right] \times 1.00 = 0.50$$

$$I_{ts} = d_o t_w^3 J = 81.89 \times 0.625^3 \times 0.50 = 10.00 \text{ in}^4$$

$t_s = 0.625$ in, $b_s = 6.75$ in.

For single stiffeners:

$$I = \frac{1}{3} b_s^3 t_s = \frac{1}{3} \times 6.75^3 \times 0.625 = 64.07 \text{ in}^4 > 10.00 \text{ in}^4 \text{ OK}$$

The actual stiffener spacing d_o was used to compute X and Z because these curvature parameters are dependent on the restraint actually provided by the transverse stiffeners. d_o is also used to compute I_{ts} to allow a smaller stiffener to be used. If desired, the required stiffener spacing d may be substituted for d_o in these equations. However, larger required sizes for the stiffeners will generally result.

Girder Stress Check G4 Spans 1 & 2
Longitudinally Stiffened Web - Transverse Stiffener Spacing

Calculations similar to those shown on page 166 reveal that the following transverse stiffener spacings are satisfactory in Spans 1 and 2 for the longitudinally stiffened web design. The factored shears and stiffener spacings are graphically shown in Figure E2 for Span 1 and in Figure E3 for Span 2.

Span 1

Panel	Spacing (in)	No. Transverse Stiffeners
1	50	4
2	61	3
3	82	2
4	82	2
5	61	3
6	50	4
7	41	5
8	41	5
	Total	28

Span 2

Panel	Spacing (in)	No. Transverse Stiffeners
1	41	5
2	41	5
3	50	4
4	61	3
5	61	3
6	61	3
7	51	3
8	61	3
9	50	4
10	41	5
11	41	5
	Total	43

Total for girder
28 x 2 + 43 = 99

Girder Stress Check G4 Span 1
Longitudinally Stiffened Web - Transverse Stiffener Spacing

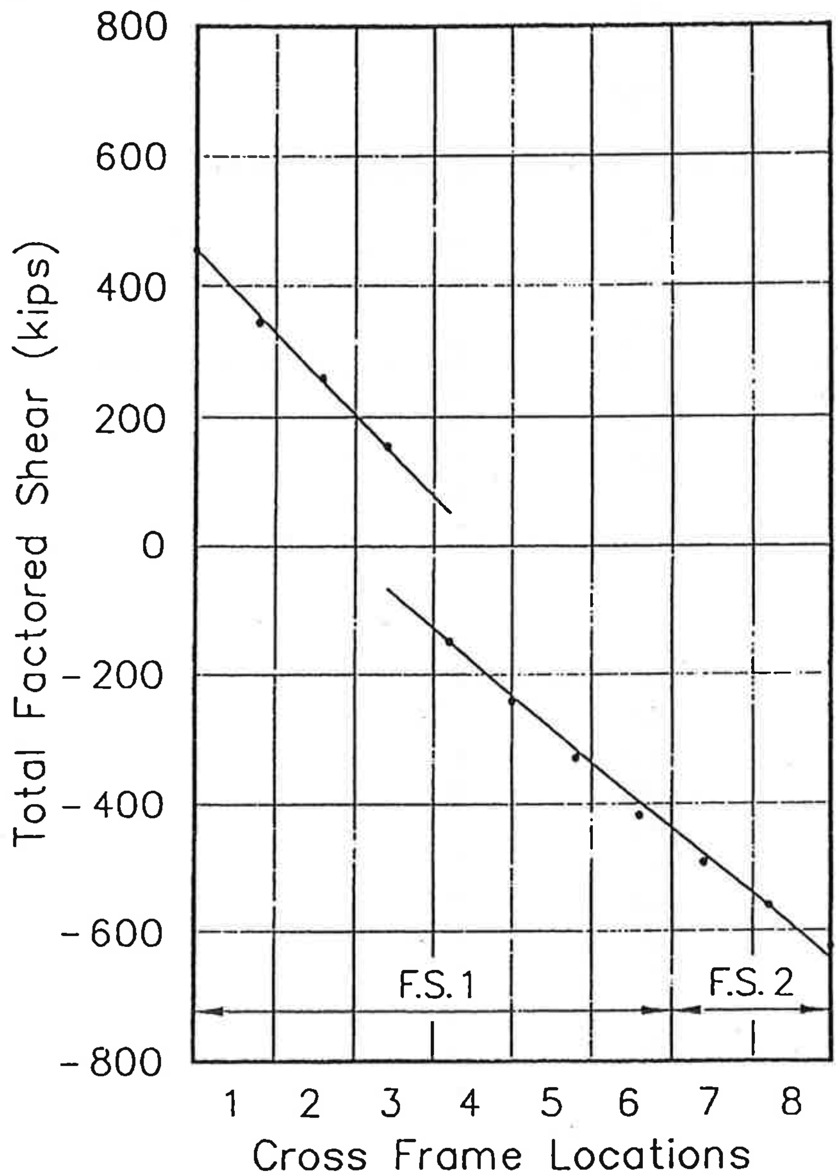


Figure E2 Factored Shear and Transverse Stiffener Spacing - Span 1

Girder Stress Check G4 Span 2
Longitudinally Stiffened Web - Transverse Stiffener Spacing

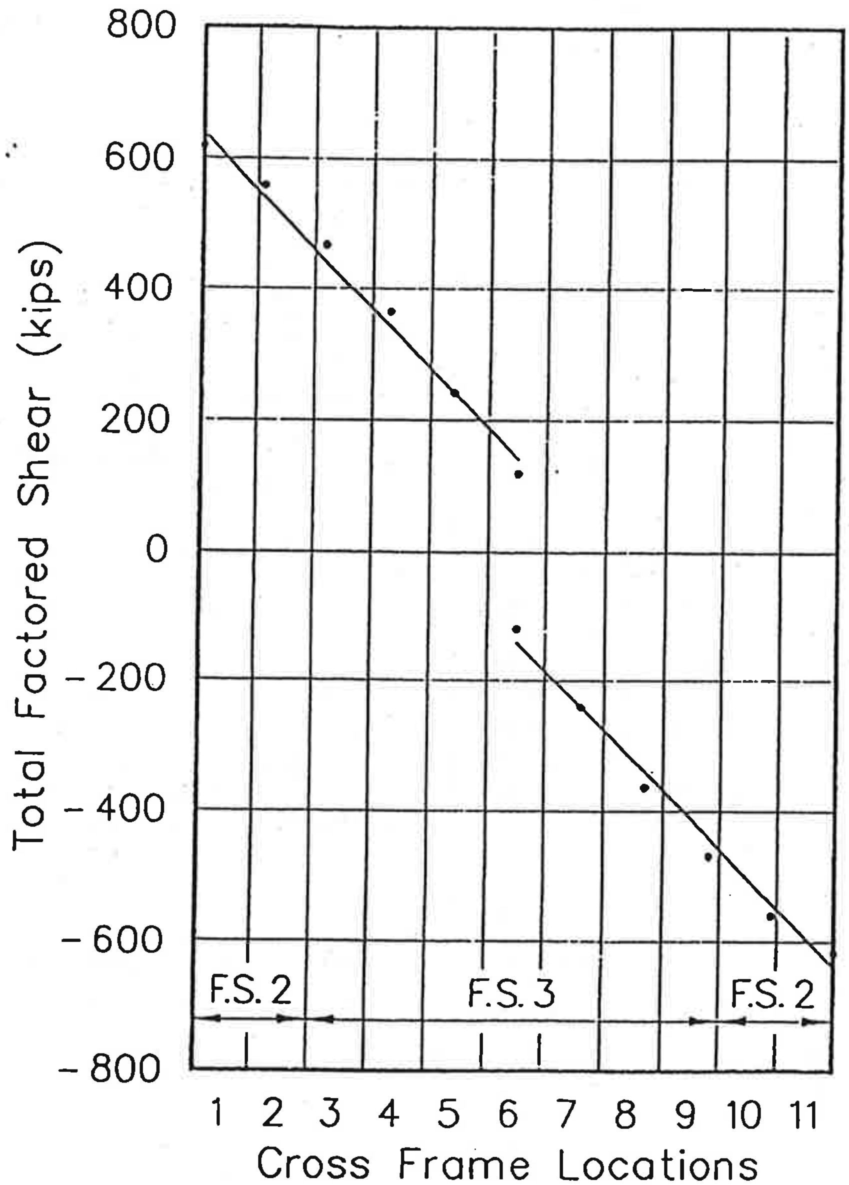


Figure E3 Factored Shear and Transverse Stiffener Spacing - Span 2

<u>Bolted Splice Design Section 8-8 G4 Node 124</u>
<u>Design Action Summary and Section information</u>

Design the bolted field splice for the transversely stiffened girder 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 E1 Unfactored Actions

Load	Moment (k-ft)		Lateral Flange Moment (k-ft)		Shear (kips)	
Steel	-382		±2.9		27	
Deck	-1,585		±12.1		112	
Comp DL	-487		±3.7		41	
Cast #1	-1,910		±17.8		7	
Overload HS20 With impact	Truck	Lane	Truck	Lane	Truck	Lane
	1,266	1,190	±9.6	±9.0	81	101
	-1,311	-1,768	±10.0	±13.4	-13	-21
Strength HS25 With impact	1,583	1,488	±12.0	±11.3	101	126
	-1,639	-2,210	±12.5	±16.8	-16	-26

Table E2 Cross Section

Component	Size (in)	Area (in^2)	Yield (F_y)	Tensile (F_u)
Top flange	17 x 1.0	17.00	50	65
Web	84 x 0.5625	47.25	50	65
Bottom flange	21 x 1.5	31.50	50	65

Note: Other section properties for the gross section may be found in Table D11.

Bolted Splice Design Section 8-8 G4 Node 124
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

Section 8-8 G4 Node 124 Bolted Splice
Top Flange - Constructibility and Overload

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 negative 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 girder splices. Since the flange is partially braced for this case, lateral flange bending must be considered. To account for the effects of lateral flange bending, the flange splice bolts will be designed for the combined effects of shear and moment using the traditional elastic vector method. The shear on the bolts is caused by the flange force calculated from the average vertical bending stress in the flange and the moment on the bolts is caused by the lateral flange bending.

Compute the polar moment of inertia of the top flange bolt pattern shown in Figure E4.

$$I_p = A_b\left[2 \times 4(3.5^2 + 6.5^2) + 2 \times 4(1.5^2 + 4.5^2)\right] = 616A_b \text{ in}^4$$

where:

A_b = area of bolt

Moment = -382 + (-1,910) = -2,292 k-ft
Lateral flange moment = 2.9 + 17.8 = 20.7 k-ft

$$f_{top\ flg} = \frac{-2{,}292 \times 49.52}{111{,}989} \times 12 \times 1.4 = 17.03 \text{ ksi}$$

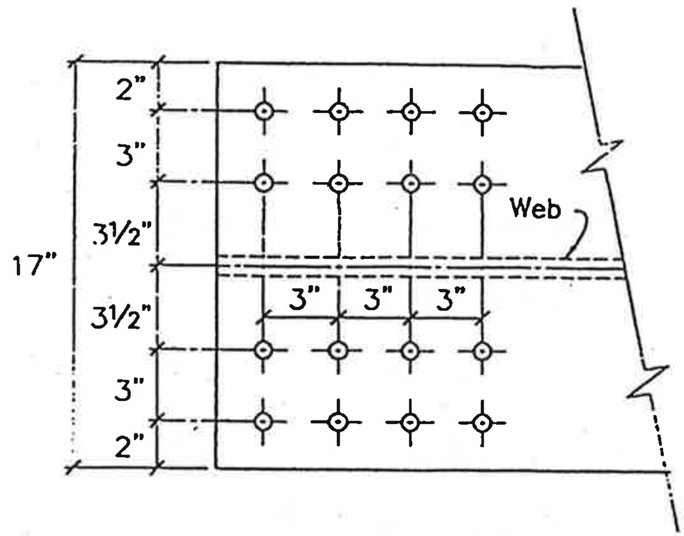
$$f_{top\ web} = \frac{-2{,}292 \times 48.52}{111{,}989} \times 12 \times 1.4 = 16.68 \text{ ksi}$$

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.

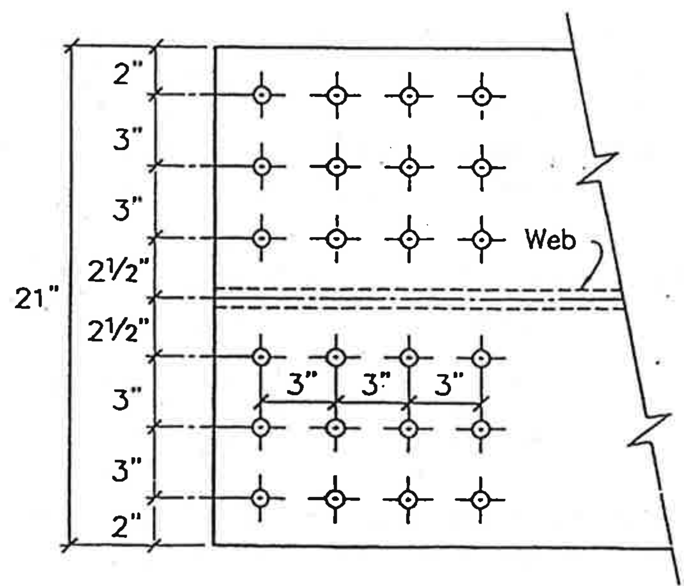
$$F_{top} = \left(\frac{17.03 + 16.68}{2}\right) \times 17.00 = 287 \text{ kips}$$

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

Section 8-8 G4 Node 124 Bolted Splice
Bolt Patterns for Top and Bottom Flanges



Top Flange Bolt Pattern



Bottom Flange Bolt Pattern

Figure E4 Bolt Patterns for Top and Bottom Flanges

Section 8-8 G4 Node 124 Bolted Splice
Top Flange - Constructibility and Overload

$$F_L = \frac{287}{16} = 17.94 \text{ k/bolt}$$

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

$$F_{L\,lat} = \frac{20.7 \times 6.5}{616} \times 12 \times 1.4 = 3.67 \text{ k/bolt}$$

Compute the transverse component of force in the critical bolt.

$$F_{Tr} = \frac{20.7 \times 4.5}{616} \times 12 \times 1.4 = 2.54 \text{ k/bolt}$$

$$F_{L\,tot} = 17.94 + 3.67 = 21.61 \text{ k/bolt}$$

Compute the resultant force on the critical bolt.

$$\Sigma_F = \sqrt{2.54^2 + 21.61^2} = 21.76 \text{ k/bolt;}\ \frac{21.76}{38.4} = 0.57 < 1.0 \text{ OK}$$

Overload

Compute the average vertical bending stress in the top flange at overload (Article 3.5.4). According to the provisions of Article 9.5, 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. By inspection, however, only the negative live load bending condition need be checked.

$$f_{top\ flg} = \left[\frac{-1{,}967 \times 49.52}{111{,}989} + \frac{-487 \times 30.77}{213{,}901} + \frac{-1{,}768(5/3) \times 15.70}{296{,}306}\right] \times 12 = 13.15 \text{ ksi}$$

$$f_{top\ web} = \left[\frac{-1{,}967 \times 48.52}{111{,}989} + \frac{-487 \times 29.77}{213{,}901} + \frac{-1{,}768(5/3) \times 14.70}{296{,}306}\right] \times 12 = 12.79 \text{ ksi}$$

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.

$$F_{top} = \left(\frac{13.15 + 12.79}{2}\right) \times 17.00 = 220 \text{ kips}$$

Lateral bending in the top flange is not considered after the deck has hardened (Article 5.4). By inspection, overload slip does not control.

Section 8-8 G4 Node 124 Bolted Splice
Bottom Flange - Constructibility

Since Cast #1 causes a larger negative moment than the entire deck, Steel + Cast #1 controls constructibility. Load factor = 1.4 (Article 3.3).

$$f_{bot\ flg} = \frac{-2{,}292 \times 36.98}{111{,}989} \times 12 \times 1.4 = -12.71 \text{ ksi}$$

$$f_{bot\ web} = \frac{-2{,}292 \times 35.48}{111{,}989} \times 12 \times 1.4 = -12.20 \text{ ksi}$$

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

$$F_{bot} = \left[\frac{-12.71 + (-12.20)}{2}\right] \times 31.50 = -392 \text{ kips}$$

Compute the polar moment of inertia of the bottom flange bolt pattern shown in Figure E4.

$$I_p = A_b\left[2 \times 6\left(1.5^2 + 4.5^2\right) + 2 \times 4\left(2.5^2 + 5.5^2 + 8.5^2\right)\right] = 1{,}140A_b \text{ in}^4$$

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

$$F_{L\ lat} = \frac{20.7 \times 8.5}{1{,}140} \times 12 \times 1.4 = 2.59 \text{ k/bolt}$$

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

$$F_L = \frac{392}{24} = 16.33 \text{ k/bolt}$$

$$F_{L\ tot} = 16.33 + 2.59 = 18.92 \text{ k/bolt}$$

Compute the transverse component of force in the critical bolt.

$$F_{Tr} = \frac{20.7 \times 4.5}{1{,}140} \times 12 \times 1.4 = 1.37 \text{ k/bolt}$$

Compute the resultant force in the critical bolt.

$$\Sigma_F = \sqrt{18.92^2 + 1.37^2} = 18.97 \text{ k/bolt;}\ \frac{18.97}{38.4} = 0.49 < 1.0 \text{ OK}$$

Section 8-8 G4 Node 124 Bolted Splice
Bottom Flange - Overload

Compute the average vertical bending stress in the bottom flange at overload (Article 3.5.4). The unfactored dead load lateral flange moment = -2.9 + (-12.1) + (-3.7) = -18.7 k-ft

According to the provisions of Article 9.5, the composite section is assumed uncracked at overload. By inspection, only the negative live load bending condition need be checked.

$$f_{bot\ flg} = \left[\frac{-1{,}967 \times 36.98}{111{,}989} + \frac{-487 \times 55.73}{213{,}901} + \frac{-1{,}768(5/3) \times 70.80}{296{,}306}\right] \times 12 = -17.77 \text{ ksi}$$

$$f_{bot\ flg} = \left[\frac{-1{,}967 \times 35.48}{111{,}989} + \frac{-487 \times 54.23}{213{,}901} + \frac{-1{,}768(5/3) \times 69.30}{296{,}306}\right] \times 12 = -17.23 \text{ ksi}$$

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.

$$P_{fo} = \left[\frac{-17.77 + (-17.23)}{2}\right] \times 31.50 = -551 \text{ kips}$$

Compute the maximum lateral moment in the bottom flange due to overload.

$$M_{lat} = -18.7 + (5/3)(-13.4) = -41.0 \text{ k-ft}$$

$$F_{L\ lat} = \frac{41.0 \times 8.5}{1{,}140} \times 12 = 3.67 \text{ k/bolt}$$

$$F_L = \frac{551}{24} = 22.96 \text{ k/bolt}$$

$$F_{L\ tot} = 22.96 + 3.67 = 26.63 \text{ k/bolt}$$

$$F_{Tr} = \frac{41.0 \times 4.5}{1{,}140} \times 12 = 1.94 \text{ k/bolt}$$

$$\Sigma_F = \sqrt{1.94^2 + 26.63^2} = 26.70 \text{ k/bolt}; \quad \frac{26.70}{38.4} = 0.70 < 1.0 \text{ OK}$$

Section 8-8 G4 Node 124 Bolted Splice
Top and Bottom Flange - Strength

The effective area of the top flange is computed from **AASHTO Article 10.18.2.2.4** as follows:

$$A_e = W_n t + \beta A_g \leq A_g$$

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

$$A_e = [17.0 - 4(0.875 + 0.125)](1.0) + (0.15)(17.0)(1.0) = 15.6 \text{ in}^2$$

$$A_g = (17.0)(1.0) = 17.0 \text{ in}^2 > 15.6 \text{ in}^2$$

$$\therefore A_e = 15.6 \text{ in}^2$$

The effective width of the top flange is computed as:

$$(b_f)_{eff} = \frac{A_e}{t} = \frac{15.6}{1.0} = 15.6 \text{ in}$$

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:

$$A_e = [21.0 - 6(0.875 + 0.125)](1.5) + 0.15(21.0)(1.5) = 27.2 \text{ in}^2$$

$$A_g = (21.0)(1.5) = 31.5 \text{ in}^2$$

$$\therefore A_e = 27.2 \text{ in}^2$$

The effective width of the bottom flange is computed as:

$$(b_f)_{eff} = \frac{A_e}{t} = \frac{27.2}{1.5} = 18.1 \text{ in}$$

Section properties computed using the effective bottom flange width 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 for both the positive and negative live load bending conditions. The provisions of Article 4.5.2 are followed to determine which composite section (cracked or uncracked) to use.

Negative live load bending case

$$F_{topflgavg} = \left[\frac{-1{,}967 \times 49.75}{108{,}575} + \frac{-487 \times 48.17}{117{,}075} + \frac{-2{,}210(5/3) \times 45.27}{132{,}743}\right] \times 12 \times 1.3 = 36.78 \text{ ksi}$$

$$F_{botflgavg} = \left[\frac{-1{,}967 \times 35.50}{108{,}575} + \frac{-487 \times 37.08}{117{,}075} + \frac{-2{,}210(5/3) \times 39.98}{132{,}743}\right] \times 12 \times 1.3 = -29.75 \text{ ksi}$$

Positive live load bending case

$$F_{topflgavg} = \left[\frac{-1{,}967 \times 49.75}{108{,}575} + \frac{-487 \times 48.17}{117{,}075} + \frac{1{,}582(5/3) \times 14.61}{272{,}106}\right] \times 12 \times 1.3 = 14.98 \text{ ksi}$$

$$F_{botflgavg} = \left[\frac{-1{,}967 \times 49.75}{108{,}575} + \frac{-487 \times 37.08}{117{,}075} + \frac{1{,}582(5/3) \times 70.64}{272{,}106}\right] \times 12 \times 1.3 = -1.76 \text{ ksi}$$

Separate calculations show that the negative live load bending case is critical. The top 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:

$$F_{cu} = \frac{|f_{cu}/R| + \alpha F_y}{2} \geq 0.75\alpha F_y$$

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.

Section 8-8 G4 Node 124 Bolted Splice
Top and Bottom Flange - Strength

$$F_{cu} = \frac{|36.78/1.0| + 1.0(50)}{2} = 43.39 \text{ ksi (controls)}$$

$$0.75\alpha F_y = 0.75(1.0)(50) = 37.50 \text{ ksi}$$

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.

$$P_{cu} = 43.39(15.6) = 677 \text{ kips (tension)}$$

The minimum design stress for the non-controlling (bottom) flange for this case is specified in **AASHTO Article 10.18.2.2.1** as:

$$F_{ncu} = R_{cu}(|f_{ncu}/R|) \geq 0.75\alpha F_y$$

For a partially braced horizontally curved compression flange, α may be taken equal to the ratio of the critical average flange stress, F_{cr}, to the yield stress F_y. From separate calculations, F_{cr} for the bottom flange at the splice is 46.16 ksi. Therefore:

$$\alpha = \frac{F_{cr}}{F_y} = \frac{46.16}{50} = 0.92$$

$$R_{cu} = |F_{cu}/f_{cu}| = |43.39/36.78| = 1.18$$

f_{ncu} is the average factored vertical bending stress in the non-controlling flange at the splice concurrent with f_{cu}.

$$R_{cu}(|f_{ncu}/R|) = 1.18(|-29.75/1.0|) = 35.11 \text{ ksi (controls)}$$

$$0.75\alpha F_y = 0.75(0.92)(50) = 34.50 \text{ ksi}$$

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.

$$P_{ncu} = (35.11)(21.0)(1.5) = 1{,}106 \text{ kips (compression)}$$

Section 8-8 G4 Node 124 Bolted Splice
Top and Bottom Flange - Strength

Top Flange

Lateral flange bending is not considered in the top flange after the deck has hardened and the flange is continuously braced. Therefore:

$$\text{No. bolts req'd} = \frac{677}{52.6} = 12.87 \text{ bolts, use } 16 \text{ bolts}$$

$$\frac{677}{16} = 42.31 \text{ k/bolt}; \quad \frac{42.31}{52.6} = 0.80 < 1.0 \text{ OK}$$

Since the required fill plate for the top flange splice is 1/4-in. thick, no reduction in the bolt design shear strength is required per the requirements of **AASHTO Article 10.18.1.2.1**.

Bottom Flange

For the bottom flange, lateral flange bending must be considered since the flange is partially braced.

$$M_{lat} = [-2.9 + (-12.1) + (-3.7) + (5/3)(-16.8)] \text{ x } 1.3 = -60.8 \text{ k-ft}$$

The lateral flange moment is then factored up by R_{cu} to be consistent with the computation of F_{ncu} and P_{ncu}.

$$M_{lat} = -60.8 \text{ x } 1.18 = -71.7 \text{ k-ft}$$

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

$$F_{L\ lat} = \frac{71.7 \text{ x } 8.5}{1{,}140} \text{ x } 12 = 6.42 \text{ k/bolt}$$

Compute the transverse component of force in the critical bolt.

$$F_{Tr} = \frac{71.7 \text{ x } 4.5}{1{,}140} \text{ x } 12 = 3.40 \text{ k/bolt}$$

Compute the force in each bolt due to the minimum design force, P_{ncu}.

$$F_L = \frac{1{,}106}{24} = 46.08 \text{ k/bolt}$$

$$F_{L\ tot} = 46.08 + 6.42 = 52.50 \text{ k/bolt}$$

Section 8-8 G4 Node 124 Bolted Splice
Top and Bottom Flange - Strength

Compute the resultant force on the critical bolt.

$$\Sigma_F = \sqrt{3.40^2 + 52.50^2} = 52.61 \text{ k/bolt; } \frac{52.61}{52.6} = 1.00 \text{ OK}$$

Note that a fill plate is not required for the bottom flange splice. Therefore, no reduction in the bolt design shear strength is necessary.

Section 8-8 G4 Node 124 Bolted Splice
Web - Constructibility and Overload

A pattern of two rows of 7/8-inch bolts spaced vertically at 3.5 inches will be tried for the web splice. There are 46 bolts on each side of the web splice. The pattern is shown in Figure E5. Note that there is 3.5 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.

$$I_p = A_b[2 \times 2(3.5^2 + 7.0^2 + 10.5^2 + 14.0^2 + 17.5^2 + 21.0^2 + 24.5^2 + 28.0^2 + 31.5^2 + 35.0^2 + 38.5^2)] + (46 \times 1.5^2) = 24{,}898A_b \text{ in}^4$$

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

Compute the factored shear at the splice due to Steel plus Cast #1 and #2. The Cast #2 shear is taken from Table D3 and is conservatively included in the calculation.

$$V = (27 + 7 + 92) \times 1.4 = 176 \text{ kips}$$

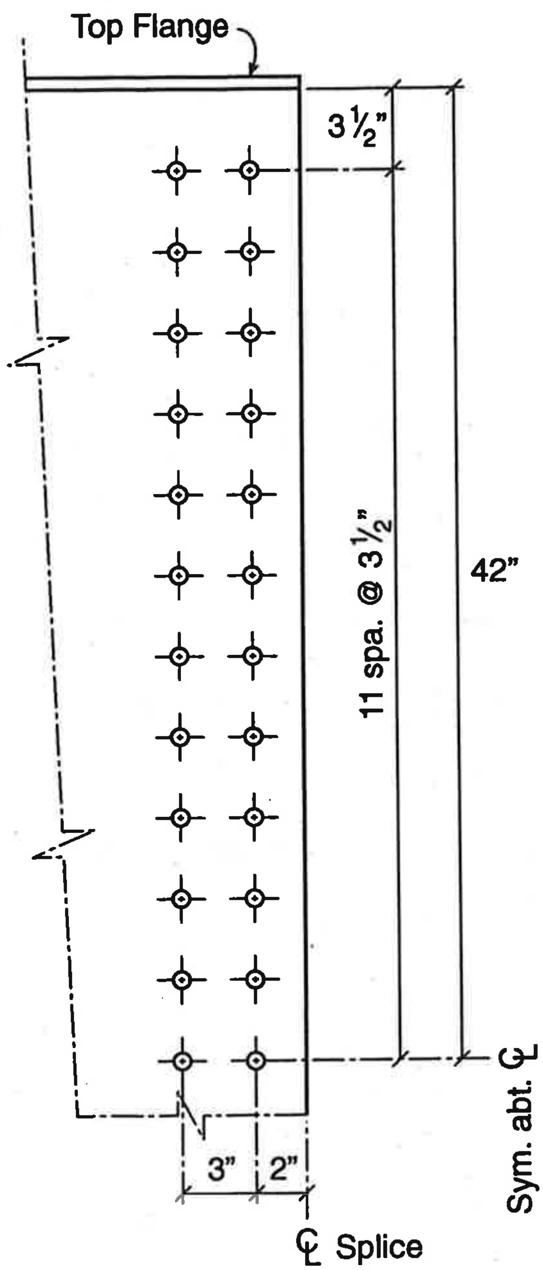
Compute the moment, M_v, due to the eccentricity of the factored shear about the centroid of the connection (refer to the web bolt pattern in Figure E5).

$$M_v = Ve = 176(3/2 + 2.125)(1/12) = 53.2 \text{ k-ft}$$

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 174), the average factored vertical bending stress in the top flange for Steel plus Cast #1 is computed as:

$$f_{tf} = \left(\frac{17.03 + 16.68}{2}\right) = 16.86 \text{ ksi}$$

<u>Section 8-8 G4 Node 124 Bolted Splice</u>
<u>Web - Bolt Pattern</u>



Note: 1/4" gap assumed between edges of field pieces.

Figure E5 Web Bolt Pattern

Section 8-8 G4 Node 124 Bolted Splice
Web - Constructibility and Overload

The average factored vertical bending stress in the bottom flange is (see page 177)

$$f_{bf} = \left[\frac{-12.71 + (-12.20)}{2}\right] = -12.46 \text{ ksi}$$

Using these stresses

$$M_w = \frac{t_w D^2}{12}|f_{tf} - f_{bf}| = \frac{0.5625(84)^2}{12}|16.86 - (-12.46)|(1/12) = -808.1 \text{ k-ft}$$

$$H_w = \frac{t_w D}{2}(f_{tf} + f_{bf}) = \frac{0.5625(84)}{2}[16.86 + (-12.46)] = 104.0 \text{ kips}$$

The total moment on the web splice is computed as:

$$M_{tot} = M_v + |M_w| = 53.2 + |-808.1| = 861.3 \text{ k-ft}$$

Compute the vertical bolt force due to the factored shear.

$$F_s = \frac{V}{N_b} = \frac{176}{46} = 3.83 \text{ k/bolt}$$

Compute the bolt force due to the horizontal force resultant.

$$F_H = \frac{H_w}{N_b} = \frac{104.0}{46} = 2.26 \text{ k/bolt}$$

Compute the horizontal and vertical components of the force on the extreme bolt due to the total moment on the splice.

$$F_{Mv} = \frac{M_{tot}x}{I_p} = \frac{861.3(12)(3/2)}{24{,}898} = 0.62 \text{ k/bolt}$$

$$F_{Mh} = \frac{M_{tot}y}{I_p} = \frac{861.3(12)(38.5)}{24{,}898} = 15.98 \text{ k/bolt}$$

Compute the resultant bolt force.

$$F_r = \sqrt{(F_s + F_{Mv})^2 + (F_H + F_{Mh})^2} = \sqrt{(3.83 + 0.62)^2 + (2.26 + 15.98)^2} = 18.77 \text{ k/bolt}$$

Section 8-8 G4 Node 124 Bolted Splice
Web - Constructibility and Overload

$$\frac{18.77}{38.4} = 0.49 < 1.0 \text{ OK}$$

The preceding check is obviously conservative since the maximum factored moment after Cast #1 is assumed to be concurrent with the maximum factored shear after Cast #2.

Overload

Compute the factored overload design shear, V_{wo}, at the splice, which is simply taken equal to the maximum 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**.

$$V_{wo} = V_o = [27 + 112 + 41 + 5/3(101)] = 348 \text{ kips}$$

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 E5).

$$M_{vo} = V_{wo}e = 348(3/2 + 2.125)(1/12) = 105.1 \text{ k-ft}$$

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 negative live load bending case controls.

Using the results from earlier calculations (pages 176 and 178), the average vertical bending stress in the top flange due to overload, f_o, is computed as:

$$f_o = \left(\frac{13.15 + 12.79}{2}\right) = 12.97 \text{ ksi}$$

The average vertical bending stress in the bottom flange due to overload, f_{of}, is

$$f_{of} = \left[\frac{-17.77 + (-17.23)}{2}\right] = -17.50 \text{ ksi}$$

Using these stresses

$$M_{wo} = \frac{t_w D^2}{12}|f_o - f_{of}| = \frac{0.5625(84)^2}{12}|12.97 - (-17.50)|(1/12) = -839.8 \text{ k-ft}$$

$$H_{wo} = \frac{t_w D}{2}(f_o + f_{of}) = \frac{0.5625(84)}{2}[12.97 + (-17.50)] = -107.0 \text{ kips}$$

Section 8-8 G4 Node 124 Bolted Splice
Web - Constructibility and Overload

The total moment on the web splice is computed as:

$$M_{tot} = M_v + |M_w| = 105.1 + |-839.8| = 944.9 \text{ k-ft}$$

Compute the vertical bolt force due to V_{wo}.

$$F_s = \frac{V_{wo}}{N_b} = \frac{348}{46} = 7.57 \text{ k/bolt}$$

Compute the bolt force due to the overload horizontal design force resultant, H_{wo}.

$$F_H = \frac{H_{wo}}{N_b} = \frac{107.0}{46} = 2.33 \text{ k/bolt}$$

Compute the horizontal and vertical components of the force on the extreme bolt due to the total moment on the splice.

$$F_{Mv} = \frac{M_{tot}x}{I_p} = \frac{944.9(12)(3/2)}{24,898} = 0.68 \text{ k/bolt}$$

$$F_{Mh} = \frac{M_{tot}y}{I_p} = \frac{944.9(12)(38.5)}{24,898} = 17.53 \text{ k/bolt}$$

Compute the resultant bolt force.

$$F_r = \sqrt{(F_s + F_{Mv})^2 + (F_H + F_{Mh})^2} = \sqrt{(7.57 + 0.68)^2 + (2.33 + 17.53)^2} = 21.51 \text{ k/bolt}$$

$$\frac{21.51}{38.4} = 0.56 < 1.0 \text{ OK}$$

Section 8-8 G4 Node 124 Bolted Splice
Web - Strength

Determine the design shear, V_{wu}, for the web splice according to the provisions of **AASHTO Article 10.18.2.3.2**.

The factored shear at the splice is computed as:

$$V = [27 + 112 + 41 + 5/3(126)] \times 1.3 = 507 \text{ kips}$$

The shear capacity of the 0.5625-in thick web at the splice (the smaller web) was computed earlier (page 99) according to the provisions of Article 6.3.2 as:

$$V_u = V_{cr} = 548 \text{ kips}$$

$$0.5V_u = 0.5(548) = 274 \text{ kips} < 507 \text{ kips}$$

Therefore, according to **AASHTO Article 10.18.2.3.2**, since $V > 0.5V_u$:

$$V_{wu} = \frac{(V + V_u)}{2} = \frac{(507 + 548)}{2} = 528 \text{ kips}$$

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 E5):

$$M_{vu} = V_{wu}e$$

$$M_{vu} = 528(3/2 + 2.125)(1/12) = 159.5 \text{ k-ft}$$

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 negative live load bending condition controls.

As computed earlier (page 181) for the negative live load bending case:

$$f_{cu} = 36.78 \text{ ksi}$$
$$F_{cu} = 43.39 \text{ ksi}$$
$$f_{ncu} = -29.75 \text{ ksi}$$
$$R_{cu} = 1.18$$

From the equations in **AASHTO Article 10.18.2.3.4**:

<u>Section 8-8 G4 Node 124 Bolted Splice</u>
<u>Web - Strength</u>

$$M_{wu} = \frac{t_w D^2}{12} \left| RF_{cu} - R_{cu} f_{ncu} \right|$$

$$= \frac{0.5625(84)^2}{12} \left| 1.0(43.39) - 1.18(-29.75) \right| (1/12) = -2,163.5 \text{ k-ft}$$

$$H_{wu} = \frac{t_w D}{2}(RF_{cu} + R_{cu} f_{ncu}) = \frac{0.5625(84)}{2}[1.0(43.39) + 1.18(-29.75)] = 195.7 \text{ kips}$$

The total moment on the web splice is computed as:

$$M_{tot} = M_{vu} + \left| M_{wu} \right| = 159.5 + \left| -2,163.5 \right| = 2,323 \text{ k-ft}$$

Compute the vertical bolt force due to the design shear.

$$F_s = \frac{V_{wu}}{N_b} = \frac{528}{46} = 11.48 \text{ k/bolt}$$

Compute the bolt force due to the horizontal design force resultant.

$$F_H = \frac{H_{wu}}{N_b} = \frac{195.7}{46} = 4.25 \text{ k/bolt}$$

Compute the horizontal and vertical components of the force on the extreme bolt due to the total moment on the splice.

$$F_{Mv} = \frac{M_{tot} x}{I_p} = \frac{2,323(12)(3/2)}{24,898} = 1.68 \text{ k/bolt}$$

$$F_{Mh} = \frac{M_{tot} y}{I_p} = \frac{2,323(12)(38.5)}{24,898} = 43.10 \text{ k/bolt}$$

Compute the resultant bolt force.

$$F_r = \sqrt{(F_s + F_{Mv})^2 + (F_H + F_{Mh})^2}$$

$$= \sqrt{(11.48 + 1.68)^2 + (4.25 + 43.10)^2} = 49.14 \text{ k/bolt}$$

$$\frac{49.14}{52.6} = 0.93 < 1.0 \text{ OK}$$

Section 8-8 G4 Node 124 Bolted Splice
Splice Plates

Web Splice Plate Design

Use nominal 0.375-in. thick splice plates. As permitted in **AASHTO Article 10.18.1.2.2**, a fill plate is not included since the difference in thickness of the web plates on either side of the splice is only 1/16".

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 thinner 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:

$$L_{c2} = 3.5 - 1.0 = 2.5 \text{ in}$$

$$\phi R = 0.9(2.5)(0.5625)(65) = 82.27 \text{ kips}$$

The maximum force on the extreme bolt was computed earlier (page 190) for strength to be

$$F_r = 49.14 \text{ kips}$$

$$\frac{49.14}{49.36} = 0.99 < 1.0 \text{ OK}$$

Section 8-8 G4 Node 124 Bolted Splice
Splice Plates

Check for flexural yielding on the gross section of the web splice plates.

$$A_g = 2(0.375)(81.0) = 60.75 \text{ in}^2$$

$$S_{PL} = \frac{2(0.375)(81)^2}{6} = 820.1 \text{ in}^3$$

$$f = \frac{M_{vu} + |M_{wu}|}{S_{PL}} + \frac{H_{wu}}{A_g}$$

$$= \frac{(159.5 + |-2{,}163.5|)(12)}{820.1} + \frac{195.7}{60.75} = 37.21 \text{ ksi} < 50 \text{ ksi OK}$$

Check for shear yielding on the gross section of the web splice plates.

$$V_y = 0.58A_gF_y = 0.58(60.75)(50) = 1{,}762 \text{ kips}$$

$$\frac{V_{wu}}{V_y} = \frac{528}{1{,}762} = 0.30 < 1.0 \text{ OK}$$

Flange Splice Plate Design

The width of the outside splice plate should be at least as wide as the width of the narrowest flange at the splice.

Top Flange

Try: 17 x 0.5 in outer plate $A_g = 8.50 \text{ in}^2$ Try: 2-7 x 0.625 in inner plates $A_g = 8.75 \text{ in}^2$

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).

The effective areas of the inner and outer splice plates are computed as:

$$A_e = W_n t + \beta A_g \leq A_g$$

Outer: $A_e = [17.0 - 4(0.875 + 0.125)](0.5) + 0.15(8.50) = 7.78 \text{ in}^2$

<u>Section 8-8 G4 Node 124 Bolted Splice</u>
<u>Splice Plates</u>

$$\text{Inner: } A_e = [2(7.0) - 4(0.875 + 0.125)](0.625) + 0.15(8.75) = 7.56 \text{ in}^2$$

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.

The top flange is subjected to tension. The minimum design force, P_{cu}, for the top flange was computed earlier (page 181) to be 677 kips. Lateral flange bending need not be considered in the top flange after the deck has hardened. The capacity of the splice plates to resist tension is computed as:

$$P_y = F_y A_e$$

$$\text{Outer: } P_y = 50(7.78) = 389 \text{ kips} > 677/2 = 339 \text{ kips OK}$$
$$\text{Inner: } P_y = 50(7.56) = 378 \text{ kips} > 677/2 = 339 \text{ kips OK}$$

Check bearing of the bolts on the connected material under the minimum design force. 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 four bolts adjacent to the edge of the splice plate, the edge distance is assumed to be 1.5 in. Therefore, the clear distance between the edge of the holes and the end of the splice plate is:

$$L_{c1} = 1.5 - \frac{1.0}{2} = 1.0 \text{ in}$$

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

<u>Section 8-8 G4 Node 124 Bolted Splice</u>
<u>Splice Plates</u>

$$\phi R = 4(0.9L_{c1}tF_u) = 4[0.9(1.0)(1.0)(65)] = 234.0 \text{ kips (controls)}$$

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

$$\phi R = 12(0.9L_{c2}tF_u) = 12[0.9(2.0)(1.0)(65)] = 1{,}404 \text{ kips}$$

or

$$\phi R = 12(1.8dtF_u) = 12[1.8(0.875)(1.0)(65)] = 1{,}229 \text{ kips (controls)}$$

$$\phi R_{TOTAL} = 234.0 + 1{,}229 = 1{,}463 \text{ kips}$$

$$\frac{P_{cu}}{\phi R_{TOTAL}} = \frac{677}{1{,}463} = 0.46 < 1.0 \text{ OK}$$

<u>Bottom Flange</u>

Try: 21 x 0.75 in outer plate
$A_g = 15.75 \text{ in}^2$

Try: 2-9.5 x 0.875 in inner plates
$A_g = 16.63 \text{ in}^2$

Since the splice plates are on a partially braced flange and subjected to compression, check for yielding on the gross section of the splice plates under their portion of the minimum design force, P_{ncu}, plus the factored-up lateral flange bending moment.

The combined area of the inner splice plates is within 10 percent of the area of the outside plate. Therefore, **AASHTO Article 10.18.1.3** permits both the inner and outer plates to be designed for one-half the flange design force. The minimum flange design force, P_{ncu}, was computed earlier (page 181) to be 1,106 kips (compression). The factored-up lateral flange moment for strength was computed earlier (page 182) to be -71.7 k-ft.

Check the outer splice plate. The lateral section modulus of the splice plate is

$$S = \frac{0.75(21)^2}{6} = 55.1 \text{ in}^3$$

$$f = \frac{(1{,}106/2)}{0.75(21.0)} + \frac{71.7(12)}{55.1} = 50.73 \text{ ksi}$$

$$\frac{50.73}{50} = 1.01 \text{ but say OK since all actions are arbitrarily factored up}$$

Section 8-8 G4 Node 124 Bolted Splice
Splice Plates

Check the inner splice plates. The lateral section modulus of the splice plates is:

$$S = \frac{0.875(2 \times 9.5)^2}{6} = 52.6 \text{ in}^3$$

$$f = \frac{(1{,}106/2)}{2(0.875)(9.5)} + \frac{71.7(12)}{52.6} = 49.62 \text{ ksi}$$

$$\frac{49.62}{50} = 0.99 < 1.0 \text{ OK}$$

Separate calculations similar to those illustrated previously (pages 193 and 194) show that bearing of the bolts on the flange is not critical.

<u>Girder Stress Check G4 Node 99-100</u>
<u>Cross Frame Diagonal - Strength</u>

Evaluation of the cross frame results shows that the diagonal member between G4 and G3 at Node 100 has the largest force. The forces in this member for the various Load Groups are shown in Table D5. The largest factored load for the Load Groups that were examined is -88 kips.

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

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

$$F_{cr} = \frac{\pi^2E}{\left(\frac{KL_c}{r}\right)^2} \quad \text{If} \quad \frac{KL_c}{r} > \sqrt{\frac{2\pi^2E}{F_y}}$$ **AASHTO Eq (10-153)**

Determine the effective length of the diagonal.

$$L_c = \sqrt{11^2 + 7^2} = 13 \text{ ft}$$

Try: L 6 x 6 x 5/8; $F_y = 50$ ksi; $r_{zz} = 1.18$ in.; $A = 7.11$ in^2

Article 9.3.1 requires that K = 1 for single angle members. Although the ends of the member may be restrained, there are eccentricities within the connections that are not considered. ASCE Manual No. 52 on Steel Transmission Tower Design recommends K = 1.0 for single angles for this reason based on available test data.

$$\sqrt{\frac{2\pi^2E}{F_y}} = \sqrt{\frac{2\pi^2 \times 29{,}000}{50}} = 107$$

$$\frac{KL_c}{r} = \frac{1 \times 13 \times 12}{1.18} = 132 > 107$$

$$F_{cr} = \frac{\pi^2 \times 29{,}000}{132^2} = 16.43 \text{ ksi}$$

$$P_u = 0.85 \times 7.11 \times 16.43 = 99 \text{ k}$$

$$\frac{|-88|}{99} = 0.89 < 1.0 \text{ OK}$$

Girder Stress Check G4 Node 99-100
Diagonal - Strength and Connection

A more rigorous design of an eccentrically loaded single-angle member may be performed utilizing the AISC Specification for Allowable Stress Design of Single-Angle Members contained in the Ninth Edition of the AISC ASD Manual of Steel Construction.

Check fatigue of the member assuming the diagonal is connected to a gusset plate with fillet welds. The range of force in this member due to the factored fatigue loading is 7 kips. However, there are other members with a range of force of 17 kips. The larger range will be used for the fatigue design.

Article 3.5.7.2 requires that 75 percent of the range be used for checking fatigue of transverse members.

17 x 0.75 = 12.75 kips.

$$\text{Fatigue stress range} = \frac{12.75}{7.11} = 1.79 \text{ ksi}$$

Material adjacent to fillet discontinuous welds is Category E.

1.79 ksi < 2.25 ksi permitted 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 here.

Assume a 5/16" fillet weld; F_u = 70 ksi. According to **AASHTO Table 10.56A**:

Weld capacity = 0.45 x 0.707 x 0.3125 x 70 = 7.0 k/in
Length of weld = 88/7.0 = 12.57 in

In order to minimize eccentricity, the welds should be proportioned on each side of the angle leg such that the resultant force is along the neutral axis of the angle.
The neutral axis is 1.73 in. from the heel. Compute the required length of weld along the heel, ℓ_1. ℓ_2 is the required length of weld along the connected leg.

$$\ell_1(1.73) = \ell_2(6-1.73)$$

$$\ell_2 = 12.57 - \ell_1$$

$$\ell_1 = \frac{4.27(12.57 - \ell_1)}{1.73}$$

Girder Stress Check G4 Node 99-100
Diagonal - Strength and Connection

$\ell_1 = 8.95$ in; $\ell_2 = 12.57 - 8.95 = 3.62$ in

The gusset plate should be of at least the same thickness as the angle and have at least the same equivalent net area. The gusset plate is bolted to the connection plate, which is welded to the girder web and flanges. The angle diagonal is attached near the bottom of G4. The bottom chord carries 40 kips out of the connection so the resultant force is approximately 48 kips (88-40) that is transferred into the girder through bolts. Since the minimum number of bolts in a connection is two, there is no need to examine the bolt design. Welds between the connection plate and bottom flange must be able to transfer 48 kips of shear. Since this location is at a bearing, the welds are deemed to be adequate.

Centrifugal Force Calculations

According to Article 3.5.2, the centrifugal force is to be determined according to **AASHTO Article 3.10**. The radial component of the centrifugal force is assumed to be transmitted from the deck through the end cross frames or diaphragms and the bearings to the substructure. According to Article 3.5.2, a load path must be provided to carry this radial force to the substructure.

Centrifugal force also causes an overturning effect. The center of gravity of the design truck is assumed to be 6 ft above the roadway (**AASHTO Article 3.10.3**). The spacing of the wheels is 6 ft (**AASHTO Article 3.7**). Figure E6 shows the relationship between the centrifugal force and superelevation effect graphically. An overturning effect occurs because the radial component of centrifugal force is applied 6 feet above the top of the deck.

The overturning component causes the exterior (with respect to curvature) wheel line to be more that half the weight of the truck and the interior wheel line to be less than half the weight of the truck by the same amount. Thus, the outside of the bridge is more heavily loaded. The overturning force must be adjusted for the roadway superelevation.

Centrifugal force is proportional to the mass and the tangential speed squared, and inversely proportional to the bridge radius.

$$C.F. = \frac{MV^2}{R};\ C.F. = \frac{W}{g}\left(\frac{V^2}{R}\right);\ C.F. = \frac{W}{32.2}\left(\frac{V^2}{R}\right)$$

where:

V = Design speed(ft/sec) = 1.47 x V(mph)
R = Bridge radius (ft)
Wt = Vehicle weight (kips)
g = Acceleration of gravity (32.2 ft/sec^2)

$$C.F. = \frac{W}{32.2}\left(\frac{1.47V}{R}\right)^2 = 0.068\left(\frac{V^2}{R}\right)$$ **AASHTO Eq (3-2)**

Design speed = 35 mph

$$C.F. = 0.068W\left(\frac{35^2}{700}\right) = 0.119W$$

Centrifugal Force Calculations

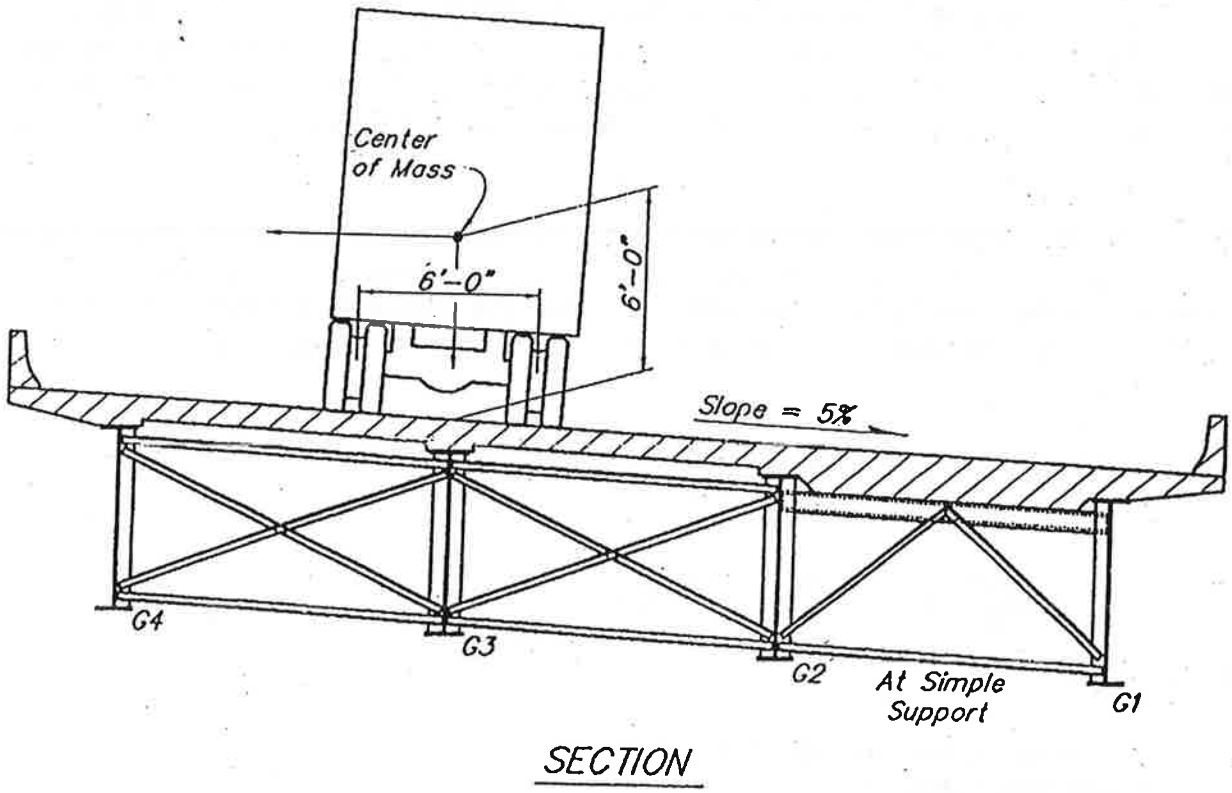


Figure E6 Centrifugal Force and Superelevation

Centrifugal Force Calculations

Compute the radial force.

Truck in one lane = $0.119 \times 90^k = 10.71^k$
Truck in two lanes = $0.119 \times 90^k \times 2 = 21.42^k$
Truck in three lanes = $0.119 \times 90^k \times 3 \times 0.90 = 28.92^k$
The three-lane case is reduced by the 0.90 factor according to **AASHTO Article 3.12**.

The largest centrifugal force is due to three trucks. The force will be applied to the deck in the radial direction. The force is resisted by the shear in the deck and is transferred to the bearings through the cross frames at the bearings. Other cross frames are not affected by the radial force.

Compute the overturning force.

Sum the moments about the inside wheel.

The 5 percent cross slope of the deck must be taken into account.
Vehicle gravity acts 3 ft - (6 ft x 0.05) = 2.70 ft from the inside wheel.

Σ Moments about inside wheel equal 0.
$Wt \times [2.70 \text{ ft} + 0.119 \times 6 \text{ ft}] - F_o [6 \text{ ft}] = 0$
where: F_o = Reaction of the outside wheel.
$F_o = 0.57\ Wt$
F_i = Force on inside wheel
$F_i = Wt(1.00 - 0.57) = 0.43\ Wt$

The wheel loads in each lane that are applied to the influence surfaces are adjusted by these factors. The result is that the exterior girder will receive slightly higher load and the interior girder will receive slightly lower load. Thus, it is also necessary to compute the condition with no centrifugal force, i.e., a stationary vehicle with equal wheel loads, and select the worst case. Otherwise, the stationary vehicle condition may be ignored.

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Appendix F

Tabulation of Stress Checks, Girder 4

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Introduction

The following tabulations of comparative stress checks (for Girder 4) between the Recommended Specifications and the current AASHTO LFD Guide Specifications have not been updated to reflect the provisions in the most recent version of the Recommended Specifications at sections other than the sections checked in the Sample Calculations (Appendix E). As such, the tabulated numbers at sections not included in Appendix E should not necessarily be relied upon since the accuracy cannot be guaranteed. However, the tabulations at those sections are still provided and remain useful to identify general comparative trends since the revised numbers are not anticipated to be significantly different in most cases. The procedures illustrated in Appendix E can be used to generate the exact numbers at those sections, if desired.

Table F1 Constructibility - Top Flange, Girder 4

Section /Node	Recommended Specs			Current Guide Spec	
	F_{cr}	f_b	f_b+f_ℓ	F_{cr}	f_b
1-1 24	-35.69	-25.77 0.72	-35.52 0.71	**	
2-2 44	-35.21	-31.15 0.88	-39.60 0.79	**	
2-2 44#	-35.71	-27.61 0.77	-34.48 0.79		
2-2 44##	-35.98	-32.16 0.89	-42.92 0.86		
3-3 64	-35.08	-18.77 0.53	-26.76 0.54		
3-3 64#	-34.79	-16.64 0.48	-24.63 0.49		
3-3 64##	-35.90	-19.37 0.54	-26.63 0.53		
4-4 76	-29.72	-4.97 0.17	-8.69 0.17		
4-4 76#	-28.95	-4.41 0.15	-8.12 0.16		
4-4 76##	-30.95	-5.13 0.17	-8.50 0.17		
10-10 148	-34.11	-27.32 0.80	-40.08 0.80	-29.90	-27.81 0.93

** b/t failed $4{,}400/\sqrt{F_y}$
# Unstiffened web
Longitudinally stiffened web

Table F2 Constructibility - Bottom Flange, Girder 4

Section /Node	Recommended Specs			Current Guide Spec	
	F_{cr}	f_b	f_b+f_ℓ	F_{cr}	f_b
1-1 24	39.91	25.20 0.63	34.04 0.68		
2-2 44	39.19	24.95 0.64	32.12 0.64		
2-2 44#	39.55	25.03 0.63	33.64 0.67		
2-2 44##	39.49	24.95 0.63	31.48 0.63		
3-3 64	39.71	15.03 0.38	19.87 0.40		
3-3 64#	39.32	15.08 0.38	20.88 0.42		
3-3 64##	40.07	15.03 0.38	19.44 0.39		
4-4 76	34.89	3.98 0.11	6.23 0.12		
4-4 76#	33.79	3.99 0.12	6.69 0.13		
4-4 76##	35.57	3.98 0.11	6.03 0.12		
10-10 148	40.56	20.40 0.52	25.98 0.52		

Unstiffened web
Longitudinally stiffened web

Table F3 Constructibility - Web, Girder 4

Section /Node	Recommended Specs		Current Guide Spec	
	F_{cr}	f_b	F_{cr}	f_b
1-1 24	-41.18T	-25.18 0.61		
2-2 44	-33.60T	-30.50 0.91		
2-2 44#	-50.00T	-27.01 0.54		
2-2 44##	-50.00T	-31.49 0.63		
3-3 64	-33.60T	-18.37 0.55		
3-3 64#	-50.00T	-16.27 0.33		
3-3 64## *	-14.02T -19.73T	-18.97 1.35 0.96		
4-4 76	-33.60T	-4.87 0.14		
4-4 76#	-50.00T	-4.31 0.09		
4-4 76## *	-6.87T -19.73	-5.03 0.76 0.25		
10-10 148	-31.57B	-26.77 0.85		

* Transverse stiffener only criteria used to determine k
\# Unstiffened Web
\## Longitudinally Stiffened Web
T = Top of web in compression
B = Bottom of web in compression

Table F4 Strength - Bottom Flange, Girder 4

Section /Node	Recommended Specs			Current Guide Spec		
	F_{cr}	f_b	f_b+f_ℓ	F_{cr}	f_b	f_b+f_w
1-1 24	50.00	46.04 0.92	51.08 1.02	50.00	45.80 0.92	60.74 1.21
2-2 44	45.63	45.94 1.01	50.30 1.01	50.00	45.65 0.91	58.56 1.17
2-2 44#	50.00	44.67 0.89	49.90 1.00		0.91	1.17
2-2 44##	50.00	46.91 0.94	50.88 1.02			
3-3 64	50.00 -45.57	27.78 0.56 -14.73 0.32	30.57 0.61 -15.88 0.32			
3-3 64#	50.00 -45.57	26.72 0.53 -14.77 0.32	30.06 0.60 -16.16 0.32			
3-3 64##	50.00 -45.96	28.56 0.57 - 14.73 0.32	31.10 0.62 -15.78 0.32			
4-4 76	-45.57	-32.84 0.72	-35.55 0.71	-45.60	-32.10	-40.03
4-4 76#	-45.56	-32.88 0.72	-36.13 0.72		0.70	0.80
4-4 76##	-45.96	-32.96 0.72	-35.43 0.71			
5-5 88	-47.32	-47.36 1.00	-50.35 1.01	-36.40	-46.62	-55.45
5-5 88#	-47.10	-46.60 0.97	-48.83 0.98		1.28	1.11
5-5 88##	-47.10	-45.67 0.97	-48.44 0.97			

 # Unstiffened web
Longitudinally stiffened web

Table F5 Strength - Bottom Flange, Girder 4

Section /Node	Recommended Specs			Current Guide Spec		
	F_{cr}	f_b	f_b+f_ℓ	F_{cr}	f_b	f_b+f_w
6-6 100	-47.31	-47.05 0.99	-49.71 0.99	-47.3	-46.36 0.98	-54.22 1.08
6-6 100#	-47.13	-46.96 1.00	-49.83 1.00		0.98	1.08
6-6 100##	-47.53	-46.70 0.98	-49.17 0.98			
7-7 112	-47.56	-47.44 1.00	-50.16 1.00	-37.30	-46.72 1.25	-54.77 1.10
8-8 124	-45.97	-30.09 0.65	-32.29 0.65	-46.00	-29.52 0.64	-36.01 0.72
10-10 144	50.0	47.10 0.94	51.14 1.02	50.00	46.24 0.92	58.06 1.16

\# Unstiffened web
\## Longitudinally stiffened web

Table F6 Strength - Top Flange, Girder 4

Section /Node	Recommended Specs		Current Guide Spec	
	F_{cr}	f_b	F_{cr}	f_b
1-1 24	-50.00	-25.35 0.51	-50.0	-25.92 0.52
2-2 44	-50.00	-28.76 0.58	-50.0	-29.47 0.59
2-2 44#	-50.00	-26.66 0.53		
2-2 44##	-50.00	-29.24 0.58		
3-3 64	-50.00 50.00	-11.79 0.24 18.39 0.37		
3-3 64#	-50.00 50.00	-11.32 0.23 16.30 0.33		
3-3 64##	-50.00 50.00	-11.84 0.24 18.98 0.38		
4-4 76	50.00	35.85 0.72	-50.0	-35.10
4-4 76#	50.00	32.61 0.65		0.70
4-4 76##	50.00	36.68 0.73		
5-5 88	50.00	47.78 0.96	50.0	47.08
5-5 88#	50.00	45.75 0.91		0.94
5-5 88##	50.00	49.88 1.00		

 # Unstiffened web
Longitudinally stiffened web

Table F7 Strength - Top Flange, Girder 4

Section /Node	Recommended Specs		Current Guide Spec	
	F_{cr}	f_b	F_{cr}	f_b
6-6 100	50.00	49.68 0.99	50.0	48.99 0.98
6-6 100#	50.00	49.14 0.98		
6-6 100##	50.00	49.74 0.99		
7-7 112	50.00	47.98 0.96	50.0	47.32 0.95
8-8 124	50.00	35.74 0.71	50.0	35.13 0.70
10-10 148	-50.00	-30.88 0.62	-50.00	-31.52 0.63

Unstiffened web
Longitudinally stiffened web

Table F8 Strength - Web (Compression), Girder 4

Section /Node	Recommended Specs		Current Guide Spec	
	F_{cr}	f_b	F_{cr}	f_b
1-1 24	-50.00T	-24.52 0.49		
2-2 44	-50.00T	-27.90 0.56		
2-2 44#	-50.00T	-25.83 0.52		
2-2 44##	-50.00T	-28.36 0.57		
3-3 64	-50.00T -50.00B	-11.34 0.23 -14.15 0.28		
3-3 64#	-50.00T -50.00B	-10.88 0.22 -14.32 0.29		
3-3 64##	-14.02T -14.02B -50.00T * -34.16B	-11.38 0.81 -14.14 1.01 -11.38 0.23 -14.14 0.41		
4-4 76	-46.79B	-31.65 0.68		
4-4 76#	-50.00B	-31.93 0.64		
4-4 76##	-32.94B	-31.75 0.96		

\# Unstiffened web
\## Longitudinally stiffened web
* Transverse stiffener only criteria used to determine k
T = top of web in compression
B = bottom of web in compression

Table F9 Strength - Web (Compression), Girder 4

Section /Node	Recommended Specs		Current Guide Spec	
	F_{cr}	f_b	F_{cr}	f_b
5-5 88	-50.00B	-45.71 0.91		
5-5 88#	-50.00B	-44.02 0.88		
5-5 88##	-50.00B	-46.69 0.93		
6-6 100	-50.00B	-43.81 0.88		
6-6 100#	-50.00B	-43.75 0.88		
6-6 100##	-50.00B	-43.47 0.87		
7-7 112	-50.00B	-45.79 0.92		
8-8 124	-50.00B	-28.95 0.58		
10-10 148	-50.00T	-29.98 0.60		

\# Unstiffened web
\## Longitudinally stiffened web
T = top of web in compression
B = bottom of web in compression

Table F10 Fatigue - Category C' and Stud Spacing, Girder 4

Section /Node	Category C'		Stud Spac (in.)	
	Recommended Specs	Current Guide Spec	Recommended Specs	Current Guide Spec
1-1 24	0.67*	0.93 0.66S	14.9	17.3
2-2 44	0.75	1.09 0.76S	19.1	18.3
2-2 44#	0.72		16.5	
2-2 44##	0.78		17.3	
3-3 64	0.75b - - t	1.10b 0.77Sb 0.77t	16.0	18.2
3-3 64##	0.89b - - t		16.3	
6-6 100	0.10	0.46 0.23	16.3	19.7
6-6 100#	0.10		14.4	
6-6 100##	0.10		14.8	
9-9 128	0.60b -- t	0.89b 0.63Sb 0.66t 0.45St	14.3	17.1
10-10 148	0.67	0.97b 0.67Sb	15.3	17.4

Unstiffened web
Longitudinally stiffened web
S = single truck loading
b = bottom flange
t = top flange
- - t = no fatigue check at top flange required
* = ratio of computed to allowable stress range

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Appendix G

Two Erection Schemes

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The superstructure is designed based on analyses that assume the bridge steel is erected under zero-gravity, and that the deck is placed on the bridge at one time. In fact, neither assumption is true. The bridge is erected piece wise under gravity and the deck is placed sequentially. Thus, each portion of the superstructure is added under gravity and the rules of superposition do not apply since the structure changes stiffness as it is built. It is incumbent on the erector to erect the steel such that the final stresses and deflections of the steel are consistent with the design analysis.

To accomplish this, steel is normally fabricated to fit together under the zero stress condition. Dead load deflections are computed and the girders are cambered such that the dead load deflection from the zero-stress state is accounted for. To ensure that the camber is present in the fabricated girders, it is necessary to place the girders in the cambered position under zero stress for final drilling or reaming of bolt holes for girder splices and cross frame connections. Usually, curved girders are fitted in the vertical position so that girder splices can be more easily made. Cross frames are fitted and clamped to the girders. It is assured that the bolt holes match the zero-gravity assumption made in the analyses by assembling the steel under zero-stress prior to reaming bolt holes. Unfortunately, this requires complete fabrication of the bridge prior to shipping.

Bolt holes may be sub-punched prior to fit-up. After the girder field sections are placed with respect to each other and splice plates are clamped solid, bolt holes are reamed. Bolt holes should not be over-sized on curved girders because that makes it impossible to control the position of the girders during erection. If standard one-sixteenth inch oversize holes are used and reaming in the field is not permitted and, if holes are not elongated during the erection process, the erector is assured that the erected steel will have deflections and stresses close to those determined in the analyses for self-weight.

Bearing stiffeners are often detailed so they will rotate to perpendicular when dead load is applied. When torsion due to dead load is applied, girders tend to rotate out-of-plum with respect to the longitudinal axis of the girder. This may lead to problems with bearings and even with bridge geometry. Applying a "camber" about the longitudinal axis, although not common, has been done to mitigate such problems. Girder rotations are more critical on larger span curved girders, and those with skewed supports.

During the steel erection process, deflections are likely to differ significantly from the zero stress deflections (cambers) and from the final deflections due to steel self-weight. The erection should be performed according to a plan that will permit the steel to return to the correct deflected shape. This problem is exacerbated with curved girders because the girder weight tends to shift from the inside of the curve toward the outside girder, making it difficult to adjust one girder without lifting all girders in the cross section. Frequently, more temporary supports are required on smaller curved girder bridges than would be required on straight bridges of similar spans.

There are advantages of examining the erection process during the design phase. The need for temporary supports can be identified as well as the location and magnitude of any supports. The location of field splices may be optimized. Cambers may be adjusted to correct for an erection scheme that permits other than the zero-stress assumption.

There are two obvious ways that the steel can be erected for the three-span design example. The end spans can be erected first and the center span erected last.

Alternatively, the center span can be erected first and the end spans added last. Only the first alternative will be examined.

There are two ways for the erection to proceed with the end-span alternative. It may be done without temporary supports, or temporary supports may be used. Calculations will show that if temporary supports are not used, cranes must be available to lift the steel in the end-spans so that the center span may be fit-up properly. Otherwise, the deflections and stresses related to the steel self-weight will be in error.

Step 1 entails splicing Field Sections 1 and 2 and 4 and 5 of Girders 1 and 2 on the ground prior to erection. Connections are made under zero stress, which is obtained by blocking the girders in the vertical position. The erected girders will extend into Span 2.

Step 2 is attaching the assembled lengths of Girders 1 and 2 together by bolting the cross frames between the girders. A second unit composed of Field Sections 4 and 5 is also assembled.

Step 3 is to form a similar pair of units for Girders 3 and 4. These units are also assembled on the ground.

Step 4 is erection of the sub-assemblages with two cranes. The curved girders are made stable by bracing the girders to each other with the cross frames. However, there is no shear bracing between the girders except that provided by the bending resistance of the cross frames. It is desirable to provide some type of shear bracing between girders such as temporary diagonal bracing clamped to the flanges prior to their erection. The two Girder 1 and 2 end pairs would be erected last because the cross frames between Girders 2 and 3 may be attached to Girder 3 prior to erection. Attaching these cross frames to

Girder 3 of the outer pairs may be desirable to provide extra weight on the inside of the curve to balance the natural tendency of the outer pairs to twist toward the outside of the curve.

Step 5 is connection of the cross frames between Girders 2 and 3 in the erected sub-assemblages. Since the pairs of girders erected in Step 4 are acting independently, it is necessary to adjust their elevation to make up the cross frame connections.

Step 6 is connection of the cross frames between Girders 1 and 2 and Girders 3 and 4 in Field Section 3. As in Step 1, these connections are made on the ground under zero stress. The zero stress condition is again obtained by blocking the girders in the vertical position.

Step 7 is erection of the sub-assemblages of Girders 1 and 2 and Girders 3 and 4 of Field Section 3. Splices between Field Section 3 and Field Sections 2 and 4 are the only ones that are made in the air and under other than a zero stress condition; thus, they are the most critical.

The two erection schemes will be studied: No temporary supports and; temporary supports in Spans 1 and 3. In each step of the erection analysis, it is assumed that the self-weight of the steel being erected is considered. Subsequent stages only consider the self-weight of the additional steel. Thus, the sum of the vertical reactions for all stages will equal the weight of the erected steel.

Reactions, deflections and rotations of the field splices in Span 2 will be examined. Comparisons of these values during the erection process will be made to the computed deflected shape from the analysis used for design, which is identified as RUN2. It should be noted that the analysis in RUN2 differs slightly from that actually used in the design in that the lateral bearing restraints are considered, whereas in the design analysis, they are ignored to ensure that design moments are not reduced due to the arching effect of the bearing restraints.

The first erection scheme with no temporary supports is modeled by RUN1, RUN1A, RUN3B and RUN3C described below.

RUN1 -- Structure after Step 5. Cross frames between Girders 2 and 3 are connected. Gravity is applied to the completed structure. This represents the condition expected after the cross frames between Girders 2 and 3 have been connected.

RUN1A -- Structure after Step 4. Weight of cross frames between Girders 2 and 3 is not considered; thus, the cross frames are assumed not to be effective.

RUN3B -- Sub-assemblage of Field Section 3 of Girders 1 and 2 is erected.

RUN3C -- Sub-assemblage of Field Section 3 of Girders 3 and 4 is erected after the sub-assemblage from RUN3B is in place.

The inner Girder 1 and 2 pair is erected prior to the outer Girder 3 and 4 pair in the center span in an attempt to help limit the overall deflection and twist of the system. Predicted final deformations of the erected steel are the sum of results from the above runs. Figure G1 shows a plots of the deflected shape of the structure at one end at the end of Step 4. Deflections are amplified by 100. Figure G1(a) is a plot from the top. Note the lateral movement of the girders. Figure G1(b) is an oblique plot with deflections again amplified by 100. Although it appears that the girders have deflected toward the center of curvature, it is actually a combination of vertical and lateral deflection to the outside that gives this appearance. The sharp upward angle at the field splice locations is apparent. It is tacitly assumed that the girder splices are properly made at the angle provided by the already erected steel. Also, the Field Section 3 pieces are assumed to be fit up with the initial zero-stress camber. In fact, the rotations of the erected sub-assemblages proscribe this. Splices must be made by elongating the holes. This causes a kink or "dog-leg" at the field splices in Span 2.

The second erection scheme making use of temporary supports in Spans 1 and 3 is represented by RUN1D, RUN3E and RUN3G described as follows.

RUN1D -- Sub-assemblages described in the first scheme are erected in the end spans and temporary supports are used in Span 1 at Nodes 37 through 40 and in Span 3 at Nodes 217 through 220. These temporary support points are located approximately 50 feet from the ends of the bridge.

RUN3E -- This run considers the sub-assemblages in the end spans in-place with cross frames installed between Girders 2 and 3 and with the temporary supports still present. Sub-assemblages of Field Section 3 are erected without cross frames between Girders 2 and 3.

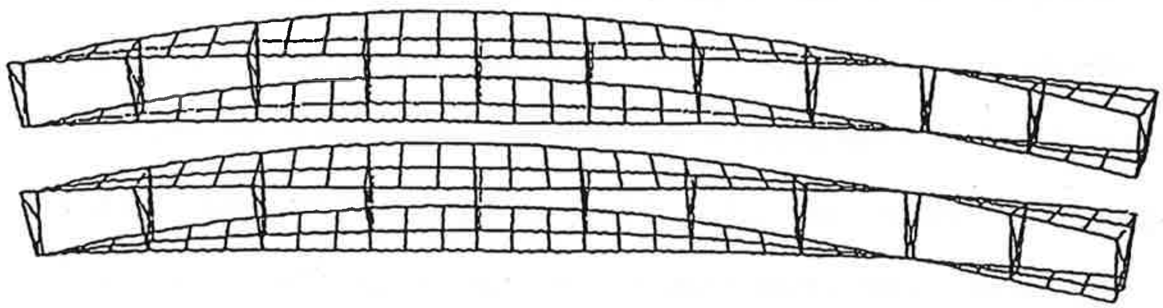
RUN3G-- This run considers installation of the cross frames between Girders 2 and 3 in Field Section 3 followed by removal of the temporary supports in Spans 1 and 3. This analysis is accomplished by first assuming that the cross frames between Girders 2 and 3 in Field Section 3 are effective and then by applying the opposite sum of the temporary reactions from RUN1D and RUN3E.

The results from the analysis of each of these erection schemes are described below.

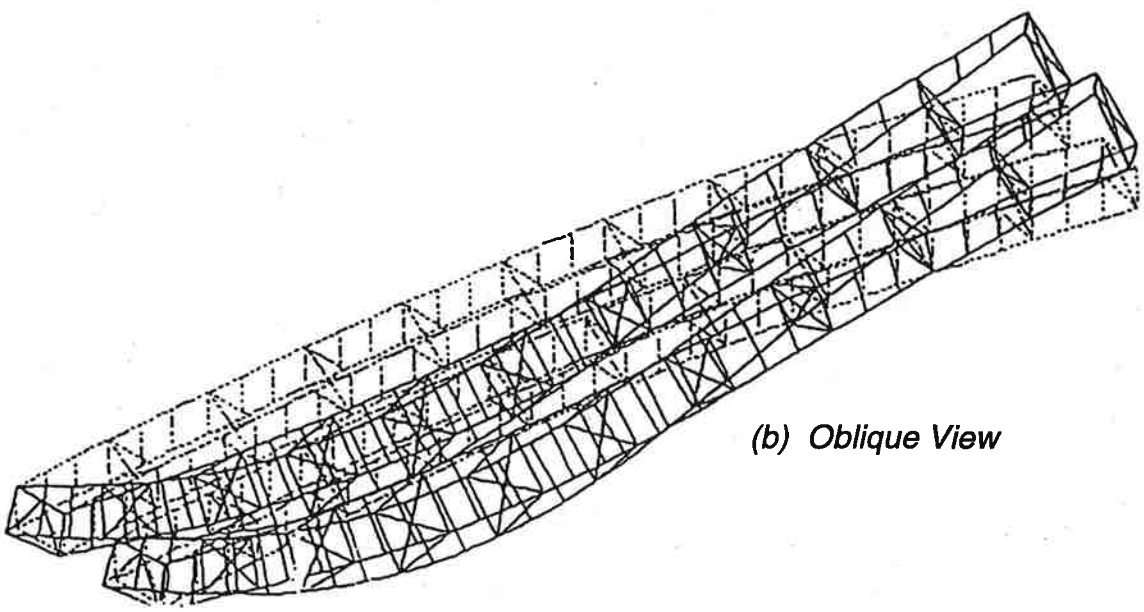
A. Steel Erection without Temporary Supports in End Spans

After the sub-assemblages in Span 1 and Span 3 are erected, it is necessary to connect Girders 2 and 3 with cross frames. Table G1 gives the magnitude of the maximum deflections before and after connection of the cross frames. RUN1 is the maximum final deflection of the connected girders. RUN1A is the maximum deflection of the unconnected girder pairs. It is evident that two cranes are required to lift the two sub-assemblages to fit the cross frames. RUN1 considers the elastic changes in the cross frame forces so the actual adjustments that need to be made prior to making the cross frame connections are slightly different. Although not checked here, it may be necessary to check girder stresses during this operation when the spans are large and the girders are heavy. Table G2 shows the changes in reactions between the two runs, RUN1 and RUN1A. RUN2 is also shown for comparison. This table indicates the approximate lift required to adjust the girders to make the cross frame connections. Note that the reactions between RUN1 and RUN1A differ by the weight of the cross frames between Girders 2 and 3.

One of the most important issues in this process is the rotation of the girders at the field splices in Span 2. Table G5 gives the computed "X" deformations at the top and bottom of the girders at the field splice locations for RUN1. The deformations are shown for one end only (Nodes 121-124). These "X" deformations are referred to as "longitudinal" deformations. They are actually the global x direction deflections. Since the model at the left end was oriented in the x direction, the bridge has been rotated through some angle such that the x direction is no longer parallel to the tangent of the girders at the splice



(a) Top View



(b) Oblique View

Figure G1 Deformed Shapes

locations (Nodes 121-124). However, for simplicity, the "X" values are used without resolving them to tangent values. The differences between the top and bottom of the girder "X" deflections are not true rotations since the dimensions are in inches. The differences in "X" deflections represent the measurement that can be made using a plumb line. Table G3 gives the similar computations for RUN2, which is from the design analysis of the completely erected steel. It is obvious that the girder end has rotated prior to placing the center field section. When the center field sections are connected, their web splice line should match the web splice line of the erected pieces. This is not possible since each end span would have the same attitude and the ends of the center girder sections can not be rotated sufficiently to match the splice lines. Although not shown, analysis of the lifted center field sections indicated that self-weight would cause negligible rotations.

One option is to stretch the bolt holes in the splice so that a "dog-leg" is created. This is represented with RUN3B and RUN3C. These runs assume that the center pairs of girder field sections are connected to the first sub-assemblages with the only additional rotation being that due to the weight of the newly added sections. The additional rotations are computed in Tables G6 and G7. Table G8 shows the differences in rotations at the splice for RUN1+RUN3B+RUN3C compared to RUN2.

Table G9 gives accumulated deflections at Nodes 37 through 40 in Span 1 for RUN1+RUN3B+RUN3C compared to RUN2. It is apparent that the accumulated deflections are in error compared to RUN2. A similar comparison is presented in Table G10 for Nodes 145 through 148 in Span 2 (at midspan). In the second case, the error is much larger. These errors mean that the computed deflections and related stresses from

RUN2 are different from those resulting from the selected erection scheme.

B. Steel Erection with Temporary Supports in End Spans

Alternatively, temporary supports could be used in the end spans. In this scheme, rigid temporary supports are assumed placed at Nodes 37 through 40 in Span 1 and at Nodes 217 through 220 in Span 3. The steel is to be erected in much the same manner as before, but after it has been erected, the temporary supports will be removed.

RUN1D was made assuming that the cross frames between Girders 2 and 3 are not in place. The temporary supports were assumed to engage the girders at the zero stress condition. It was assumed that the supports could be raised or lowered to give a different deflected shape to the girders. Table G11 shows the computed rotation of the webs at the field splice locations due to this loading. It is significant to note that the web has rotated very little.

RUN3E was made assuming that cross frames between Girders 2 and 3 in the sub-assemblages that were erected earlier are effective. The Field Section 3 sub-assemblages of Girders 1 and 2 and of Girders 3 and 4 are then added to the previously erected sub-assemblages with their temporary supports still effective in the end spans. Table G12 shows the computed rotation of the webs at the field splice locations due to this loading.

RUN3G was made assuming that cross frames between Girders 2 and 3 in Field Section 3 are effective. This run considers the removal of the temporary supports. The loading is accomplished by applying downward forces at the locations of the temporary supports equal to the computed net temporary reactions from RUN1D and RUN3E. Table G13 shows the computed rotation of the webs at the field splice locations due to this loading. Table G14 shows the computation of the sum of the girder rotations at Nodes 121 through 124 for RUN1D + RUN3E and RUN3G compared to RUN2. The net is close to the rotation computed for RUN2. Tables G15 and G16 show the sum of deflections at the temporary supports in Span 1 and the net deflections at mid-span of Span 2 (respectively) compared to those from RUN2. The deflections in Span 2 are computed assuming that Field Section 3 was inserted with no deflection from the zero stress condition. This assumption is most likely violated since the Field Section 3 assemblage is lifted at pick-points and most likely will deflect due to its self-weight. This deflection will cause a slight error that was not considered. However, an analysis not shown here revealed that pick-points can be easily located that permit this assumption to be nearly valid. This assumption must be accompanied with the assumption that the web splice is vertical at both ends of the field section when it is attached to the other steel.

The key to the good comparison of the final deflections to those from RUN2 is that the webs of the end-span sub-assemblages and the Field Section 3 sub-assemblages were both nearly in the same attitude as they were under the zero-stress state. The splice joint was assumed to be vertical in this case when the splice was fabricated. A vertical joint can be accomplished in the field by proper selection of the overhang distance and/or by adjusting the height of the temporary supports in the end spans. A trial and error approach accomplished by lifting the end-spans is often adequate when the crane capacity is available and when there are not a large number of girders in the cross section. However, larger deflections during the erection stages generally dictate the need for temporary supports. When temporary supports are utilized, computations of deflections and stresses are needed to ensure that the girders are not overstressed; to ascertain the temporary support reactions and; to determine the amount of deflection at the temporary supports when they are removed so that adequate stroke is provided in the jacks for their removal.

Table G1 Difference between Maximum Deflections (in)

Girder	RUN1A	RUN1	Delta
1	-1.15	-0.99	-0.16
2	-2.62	-1.37	1.25
3	-1.14	-1.75	-0.61
4	-2.57	-2.13	0.44

Table G2 Comparison of Reactions (kips) between RUN1 and RUN1A and RUN2

Girder	Supp	RUN2	RUN1	RUN1A	Difference
1	1	-13.6	-15.2	-11.6	3.6
	2	-81.8	-45.6	-40.0	5.6 9.2
2	1	-15.7	-20.2	-31.6	-11.4
	2	-73.7	-47.4	-57.9	-10.5 -21.9
3	1	-18.1	-25.8	-11.2	14.6
	2	-82.6	-54.7	-40.4	14.3 28.9
4	1	-22.5	-35.1	-38.3	-3.2
	2	-89.9	-70.3	-74.0	-3.7 -6.9

Table G3 Slope (in) at Splice for RUN2

Girder/Node	Top of Girder	Bot of Girder	Difference
G1/121	0.036	-0.024	-0.060
G2/122	0.038	-0.026	-0.064
G3/123	0.040	-0.027	-0.067
G4/124	0.045	-0.025	-0.070

Table G4 Slope (in) at Splice RUN1A

Girder/Node	Top of Girder	Bot of Girder	Difference
G1/121	-0.219	0.064	-0.155
G2/122	-0.382	0.063	-0.319
G3/123	-0.221	0.054	-0.167
G4/124	-0.375	0.052	-0.323

Table G5 Slope (in) at Splice RUN1

Girder/Node	Top of Girder	Bot of Girder	Difference
G1/121	-0.132	0.013	0.145
G2/122	-0.176	0.013	0.189
G3/123	-0.218	0.013	0.231
G4/124	-0.259	0.013	0.272

Table G6 Slope (in)at Splice for RUN3C

Girder/Node	Top of Girder	Bot of Girder	Difference
G1/121	0.058	0.025	0.033
G2/122	0.076	0.009	0.067
G3/123	0.108	-0.003	0.111
G4/124	0.134	-0.017	0.151

Table G7 Slope[1] (in) at splice for RUN3B

Girder/Node	Top of Girder	Bot of Girder	Difference
G1/121	0.123	0.031	0.092
G2/122	0.116	0.036	0.080
G3/123	0.080	0.050	0.030
G4/124	0.050	0.055	-0.005

[1] Table gives the rotation of the girders after only Span 1 has been erected, including the cross frames.

Table G8 Difference in Rotations (in) at Splice for RUN1+RUN3B+RUN3C Compared to RUN2

Girder/Node	RUN1	RUN3B	RUN3C	Total	RUN2
G1/121	-0.119	0.092	0.033	0.006	0.060
G2/122	-0.163	0.080	0.067	-0.016	0.064
G3/123	-0.205	0.030	0.111	-0.064	0.067
G4/124	-0.246	-0.005	0.151	-0.100	0.070

Table G9 Deflections (in) at Nodes 37-40 without Temporary Supports
No Cross Frames between Girders 2 and 3

Girder/Node	RUN1	RUN3B	RUN3C	Total	RUN2
G1/37	-0.930	0.412	-.043	-0.561	-.431
G2/38	-1.286	0.325	.279	-0.682	-.507
G3/39	-1.640	0.236	.605	-0.799	-.580
G4/40	-2.000	0.147	.933	-0.920	-.654

Table G10 Net Deflection (in) at Nodes 145-148 without Temporary Supports
No Cross Frames between Girders 2 and 3

Girder/Node	RUN1	RUN3B	RUN3C	Total	RUN2
G1/145	0.0	-1.270	0.044	-1.226	-.640
G2/146	0.0	-1.276	-0.580	-0.696	-.702
G3/147	0.0	-0.000	-1.478	-1.478	-.761
G4/148	0.0	-0.000	-2.436	-2.436	-.821

Table G11 Slope (in) at Splice for RUN1D

Girder/Node	Top of Girder	Bot of Girder	Difference
G1/121	-0.004	-0.002	-0.002
G2/122	-0.009	-0.002	-0.007
G3/123	-0.005	-0.002	-0.003
G4/124	-0.010	-0.002	-0.008

Table G12 Slope (in) at Splice for RUN3E

Girder/Node	Top of Girder	Bot of Girder	Difference
G1/121	0.111	0.016	0.095
G2/122	0.122	0.005	0.117
G3/123	0.124	0.008	0.116
G4/124	0.142	0.002	0.140

Table G13 Slope (in) at Splice for RUN3G

Girder/Node	Top of Girder	Bot of Girder	Difference
G1/121	-0.068	-0.032	-0.036
G2/122	-0.073	-0.027	-0.046
G3/123	-0.079	-0.023	-0.056
G4/124	-0.087	-0.021	-0.066

Table G14 Difference in Rotations (in) at Splice for RUN1D+RUN3E+RUN3G Compared to RUN2

Girder/Node	RUN1D	RUN3E	RUN3G	Total	RUN2
G1/121	-0.002	0.095	-0.036	0.057	0.060
G2/122	-0.007	0.117	-0.046	0.064	0.064
G3/123	-0.003	0.116	-0.056	0.057	0.067
G4/124	-0.008	0.140	-0.066	0.066	0.070

Table G15 Net Deflection (in) with Temporary Supports at Nodes 37-40

Girder/Node	RUN1D	RUN3E	RUN3G	Total	RUN2
G1/37	0.0	0.0	-.410	-0.410	-.431
G2/38	0.0	0.0	-.487	-0.487	-.507
G3/39	0.0	0.0	-.561	-0.561	-.580
G4/40	0.0	0.0	-.638	-0.638	-.654

Table G16 Net Deflection (in) at Nodes 145-148 with Temporary Supports No Cross Frames between Girders 2 and 3

Girder/Node	RUN1D	RUN3E	RUN3G	Total	RUN2
G1/145	0.0	-.906	0.287	-0.619	-.640
G2/146	0.0	-1.225	0.406	-0.819	-.702
G3/147	0.0	-1.167	0.525	-0.642	-.761
G4/148	0.0	-1.513	0.646	-0.867	-.821

C. Wind on Steel During Erection

Although it is not the responsibility of the designer to consider all wind conditions during construction of the bridge, some conditions should be considered during a study of the erection scheme. In this example, only one wind direction has been studied. Wind from the inside of the curve and perpendicular to the beginning of the bridge is considered. The intensity used in the analysis is 50 pounds per square foot. However, the intensity can be reduced if desired. Wind is applied to the first stage of construction. Temporary supports (or falsework) are assumed to be in place to resist vertical but not lateral forces. Girders 1 and 2 are framed together, but Girders 3 and 4 are not connected and are ignored.

Wind is applied to Girder 1. Curvature is considered in reducing the exposed projection and thereby reducing the amount of wind. The superelevation of the girders causes Girder 2 to be 0.55 feet higher than Girder 1 and to receive additional wind on the top flange. Girder 2 also extends approximately 3 feet beyond Girder 1 in the direction of the wind due to curvature. An additional force is applied to the end of the that girder (Node 122) at both the top and bottom flange.

Analysis results include reactions, deflections, axial forces in the top and bottom of the girders, lateral flange bending and the axial force in all bracing members. The first analysis did not consider any lateral bracing between girders. That analysis indicated lateral deflections due to wind of several feet. However, such bracing is required when lifting the pairs of girder sections. The presence of lateral bracing was therefore assumed in subsequent analyses. The bracing is composed of top lateral members connecting at the cross frame locations. A single member is used in each panel between Girders 1 and 2. These members will be removed when placing the deck forms.

Wind produces an uplift of 21.7 kips on Girder 1, Node 1. The gravity reaction due to the steel weight is only 5.6 kips down. This indicates an uplift condition. Using a dead load factor of 0.9, as specified in Article 3.3 for an uplift condition, and a load factor of 1.4 for the wind during construction as specified in Article 3.4, the maximum uplift is computed to be [1.4(21.7) - 0.9(5.6)] = 25.3 kips. Thus, the contractor needs to provide a temporary tie-down with 25.3 kip capacity for a 100 mph design wind during construction if the girders are left in pairs without cross frames added between Girders 2 and 3. The case with cross frames added is not checked, but would be much less critical and may not require tied-down bearings.

1. Girder Stresses Including Lateral Flange Bending

The top flanges act as a truss with the top flange lateral bracing. This action creates very little lateral flange bending. At Support 2, lateral flange bending is more critical. However, girder axial stresses are not very large at that location.

2. Bracing Members

Cross frame bracing is most critical at bearings under wind loading. At these points wind force is taken from the top flanges to the bearings. There is little force in these members due to gravity. At Support 1, the force is 25.05 kips. At Support 2, the critical force is 23.50 kips. Since the temporary supports are assumed to provide no lateral restraint, no wind force is taken. These cross frame members must be checked for forces in the completed bridge before they are designed for these wind forces to ensure that the critical loading case is considered.

Top lateral bracing is most critical at supports because the lateral shear is largest at the supports. At Support 1, the critical force is 48 kips and at Support 2, the critical force is 49 kips. Gravity contributes essentially no force in these members. Since this is the only loading condition for these members, they can be designed for these forces factored by 1.4, which is the load factor for wind during construction specified in Article 3.4.

Appendix H

Field Section Profiles

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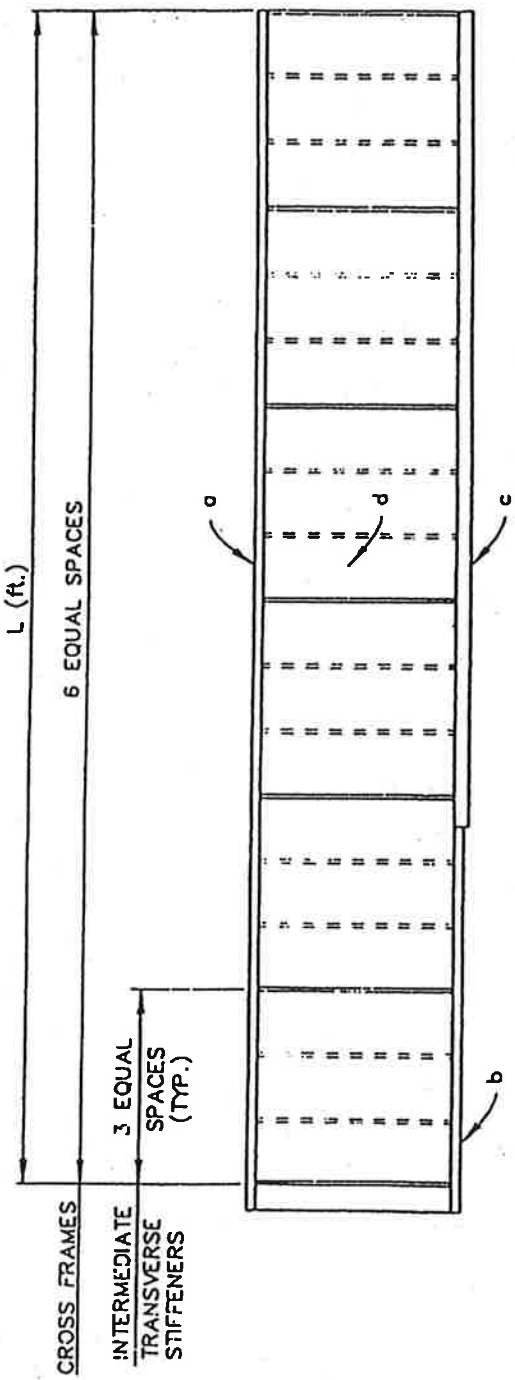
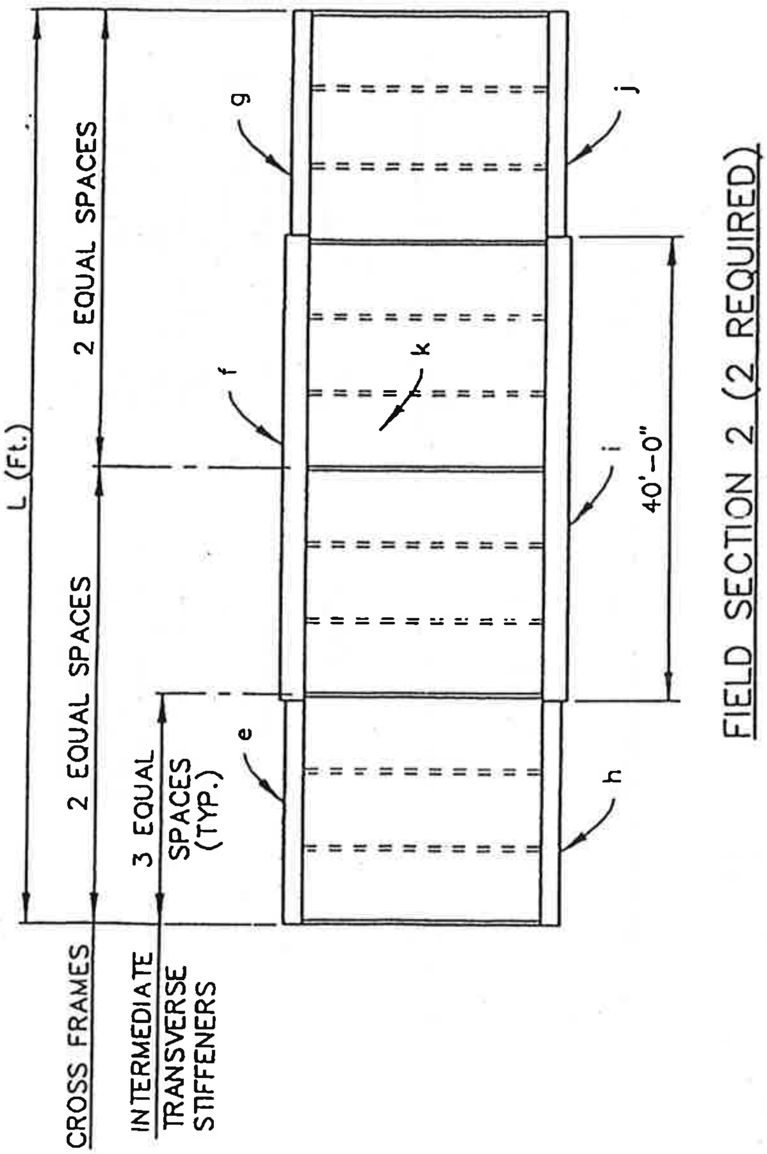


FIELD SECTION 1 (2 REQUIRED)

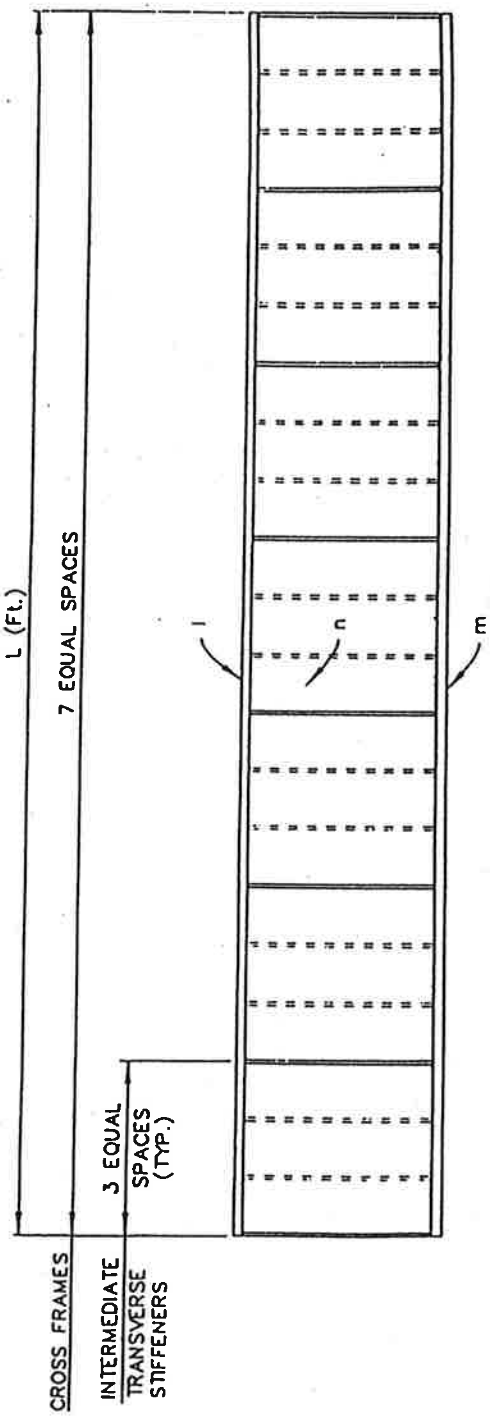
	a	b	c	d	L	Wt. (Lb.)
G1	15x1	No change	16x1	84 x 9/16	117	31,215
G2	15x1	No change	16x1	84 x 9/16	119	31,717
G3	16x1	No change	18x1	84 x 9/16	121	33,454
G4	20x1	21x1	21 x 1 1/2	84 x 9/16	123	40,074

Figure H1 Field Section 1



	e	f	g	h	i	j	k	L	Wt.(Lb.)
G1	21x1 1/4	21x2 1/2	21x1 1/4	21x1 1/2	21x3	21x1 1/2	84x 5/8	77.5	36,094
G2	18x1 1/4	18x2 1/2	18x1 1/4	19x1 1/2	19x3	19x1 1/2	84x 5/8	77.6	34,006
G3	20x1 1/4	20x2 1/2	20x1 1/4	21x1 1/2	21x3	21x1 1/2	84x 5/8	78.7	36,754
G4	28x1 1/4	28x2 1/2	28x1 1/4	27x1 1/2	27x3	27x1 1/2	84x 5/8	80.0	45,077

Figure H2 Field Section 2



FIELD SECTION 3 (1 REQUIRED)

	l	m	n	L	Wt. (Lb.)
G1	15 x 1	18 x 1	84 x 9/16	130.5	35,625
G2	15 x 1	17 x 1	84 x 9/16	132.6	35,748
G3	15 x 1	20 x 1	84 x 9/16	134.7	36,772
G4	17 x 1	28 x 1½	84 x 9/16	136.8	44,558

Figure H3 Field Section 3