

II/4A

Composite: Welded Plate Girder Load Factor Design

Introduction

Chapter 4 illustrates the design of a composite, welded plate girder bridge by the working stress method. This chapter illustrates load factor design for the same type of construction.

The example presented in this chapter deals with the design of a two-span, continuous girder (100 ft.—100 ft.), composite for positive and negative moments similar to Design III of Chapter 4. The load factor design is in accordance with the 1973 Standard Specifications for Highway Bridges of the American Association of State Highway and Transportation Officials and their Interim Specifications dated 1974, 1975 and 1976. These specifications will be referred to for brevity as AASHTO followed by an article and section reference. USS COR-TEN B (ASTM A588, Grade A) is used for the steel portion of the composite beam. This is a high-strength low-alloy structural steel that is widely used in unpainted bridges where its enhanced atmospheric corrosion resistance is desired to help minimize maintenance costs.

The procedures for dead-load distribution, lateral distribution of live load, computation of reactions, shears, moments and deflections, determination of effective slab widths, section properties (except for plastic section modulus and related properties) and stresses in composite sections are the same for load factor and working stress designs. Descriptive text and illustrative calculations similar to those presented in Chapter 4 are not repeated but the similarity is pointed out.

General Design Considerations

Members designed by the Load Factor method are proportioned for multiples of the design loads. They are required to meet certain criteria for three theoretical load levels: 1) Maximum Design Load 2) Overload and 3) Service Load. The Maximum Design Load and Overload requirements are based on multiples of the service loads with certain other coefficients necessary to insure the required capabilities of the structure. Service loads are defined as the same loads as used in working stress design.

The Maximum Design Load criteria insures the structure's capability of withstanding a few passages of exceptionally heavy vehicles (simultaneously in more than one lane), in times of extreme emergency, that may induce significant permanent deformations.

The Overload criteria insures control of permanent deformations in a member, caused by occasional overweight vehicles equal to $5/3$ the design live and impact loads (simultaneously in more than one lane), that would be objectionable to riding quality of the structure.

The Service Load criteria insures that the live load deflection and fatigue life (for assumed fatigue loading) of a member are controlled within acceptable limits.

Moments, shears and other forces are determined by assuming elastic behavior of the structure except for a continuous beam of compact section where negative moments over supports, determined by elastic analysis, may be reduced by a maximum of 10%. This reduction, however, must be accompanied by an increase in the maximum positive moment equal to the average decrease of the negative moments in the span.

DESIGN LOADS

The moments, shears or forces to be sustained by a stress-carrying steel member are computed from the following formulas for the three loading levels.

$$\text{Service Load: } D + (L + I)$$

$$\text{Overload: } D + \frac{5}{3}(L + I)$$

$$\text{Maximum Design Load: } 1.30 \left[D + \frac{5}{3}(L + I) \right]$$

where D = dead load

L = live load

I = impact load

The factor 1.30 is included to compensate for uncertainties in strength, theory, loading, analysis and material properties and dimensions. The factor 5/3 is incorporated to allow for overloads. Factors for other group loading combinations are given in AASHTO Art. 1.2.22.

DESIGN FOR MAXIMUM DESIGN LOADS

Welded plate girders of normal proportions are not likely to satisfy the requirements for compactness described in Chapter 3A. Therefore, the maximum strength or maximum moment capacity of a girder section is less than it would be if the fully plastic bending strength could be developed.

If a girder meets the requirements for a symmetrical, braced, noncompact section, the maximum strength may be computed from

$$M_u = F_y S$$

where F_y = specified minimum yield point or yield strength, psi, of the type of steel being used

S = elastic section modulus

The section consequently must be proportioned so that

$$F_y S \geq 1.30 \left[D + \frac{5}{3}(L + I) \right]$$

Here, D , L , and I represent moments induced by the service loads.

For this relationship to be permitted, the following criteria must be satisfied:

1. Width-thickness ratio of the compression flange projection, when the bending moment M induced by the Maximum Design Load equals the maximum strength M_u , should not exceed

$$\frac{b'}{t} = \frac{2,200}{\sqrt{F_y}}$$

where b' = width of projecting flange element

t = flange thickness

When $M < M_u$, b'/t may be increased in the ratio $\sqrt{M_u/M}$.

The b'/t requirement need not be satisfied for the compression flanges of composite girders in the positive bending regions.

2. Depth-thickness ratio of the web should not exceed

$$\frac{D}{t_w} = 150$$

where D = clear unsupported distance between flange components

t_w = web thickness

3. Spacing of lateral bracing of the compression flange should not exceed

$$L_b = \frac{20,000,000 A_f}{F_y d}$$

where A_f = cross-sectional area of compression flange

d = depth of girder

The displacement or twisting of girders, called lateral buckling, may also be prevented by embedment of the top and sides of the compression flange in concrete.

4. Axial compression should not exceed

$$P = 0.15 F_y A$$

where A = cross-sectional area of girder

5. Shear should not exceed either of the following values:

$$V = \frac{3.5 E t_w^3}{D}$$

$$V = 0.58 F_y D t_w$$

where E = steel modulus of elasticity

If a girder section acting together with the longitudinal slab reinforcing steel meets the preceding requirements, it may be designed as a braced, noncompact section, though it is unsymmetrical about its horizontal centroidal axis. Its maximum strength is the moment inducing yielding at an extreme surface under Maximum Design Loads composed of the initial dead load, superimposed dead load, and live load plus impact, taking into account whether the construction is shored or unshored when the concrete slab is cast.

When a member does not meet Criterion 3 for spacing of lateral bracing of braced, noncompact sections, it is considered an unbraced section. For symmetrical sections, the calculated maximum strength is reduced to

$$M_u = F_y S \left[1 - \frac{3 F_y}{4 \pi^2 E} \left(\frac{L_b}{b'} \right)^2 \right]$$

When the ratio of the smaller moment to the larger moment at the ends of the braced length L_b is less than 0.7, this value of M_u may be increased 20% but may not exceed $F_y S$.

For sections unsymmetrical about the horizontal axis but symmetrical about the vertical axis, along the web, maximum strength may be computed from the appropriate formula previously given, except that when the preceding formula for M_u is used, b' should be replaced by $0.9b'$. Because the girder section with longitudinal slab steel is unsymmetrical about the horizontal axis, this modification applies to the calculation of maximum bending strength when the section does not qualify as braced, in accordance with Criterion 3.

The above AASHTO lateral buckling equation for maximum strength was developed for prismatic compression flanges. In the case where there is a transition in compression flange width or thickness within an unbraced length, the compression flange section throughout this length is no longer prismatic and the AASHTO lateral

buckling requirements are not directly applicable. However, it can be shown that by a modification of application the AASHTO lateral buckling formula can be applied conservatively to girders with stepped flanges.* This can be done by rearranging the AASHTO formula and computing the critical buckling stress of the braced panel, in which the transition occurs as that of the girder in the stepped-down region. This stress may then be increased by 20% providing the ratio of compression flange axial forces at the ends of the braced panel are equal to or less than 0.7. Although concurrent axial forces are theoretically correct, maximum axial forces, as obtained from the moment envelopes, may conservatively be used.

The critical buckling stress is determined from the following rearrangement of the AASHTO buckling formula:

$$F_{cr} = \frac{M_u}{S} = F_y \left[1 - \frac{3F_y}{4\pi^2 E} \left(\frac{L_b}{b'} \right)^2 \right]$$

where b' = projecting compression flange width of the girder in the stepped-down region
 S = section modulus of the steel section in the stepped-down region

The maximum strength at any point in the panel is expressed as:

$$M_u = F_{cr} S_x$$

where S_x = section modulus at the point considered.

BEARING AND INTERMEDIATE STIFFENERS

In the section of the AASHTO Specifications dealing with load factor design, there are no provisions for bearing stiffeners, though intermediate transverse stiffeners and longitudinal stiffeners are covered. AASHTO Art. 1.7.73, however, requires stiffeners to be placed over bearings of welded plate girders. These stiffeners preferably should be made of plates and should satisfy the following requirements:

They should extend as nearly as practicable to the outer edges of the flange plates. The plates should be placed on both sides of the web.

The stiffeners should be designed as columns. For stiffeners composed of a pair of plates, the column section should be assumed to comprise those plates plus a centrally located strip of web with width not exceeding 18 times the web thickness.

The connection of the stiffeners to the web should be capable of transmitting the entire end reaction to the bearings.

The stiffeners should be ground to fit against the flange through which they receive their reaction or attached to the flange by full-penetration groove welds.

Only the portion of the stiffeners outside the flange-to-web plate welds should be considered effective in bearing.

Thickness of the stiffener plates should be at least

$$t = \frac{b'}{12} \sqrt{\frac{F_y}{33,000}}$$

AASHTO 1.7.134 contains load factor-design provisions for compression members. Presumably, these would apply to design of bearing stiffeners as columns, whereas the bearing pressure would be limited by the allowable stress in bearing. The total end reaction transmitted to the bearings and caused by the Maximum Design Loads, therefore, should not exceed the maximum strength of the bearing stiffeners as a column. By AASHTO 1.7.134, the maximum strength may be computed from

$$P_u = 0.85 A_s F_{cr}$$

*United States Steel Research reviewed the basis for the AASHTO requirements and analyzed the buckling loads of stepped columns with various geometries. Based on these results a design procedure was developed which relates the strength of a stepped flange to that of a prismatic flange. For additional information on this procedure contact a U.S.S. Construction Representative through the nearest USS Sales Office.

where A_s = gross effective area of the column cross section

F_{cr} = critical stress, determined from whichever of the following formulas is appropriate

$$F_{cr} = F_y \left[1 - \frac{F_y}{4\pi^2 E} \left(\frac{KL_c}{r} \right)^2 \right] \quad \frac{KL_c}{r} \leq \sqrt{\frac{2\pi^2 E}{F_y}}$$

$$F_{cr} = \frac{\pi^2 E}{(KL_c/r)^2} \quad \frac{KL_c}{r} > \sqrt{\frac{2\pi^2 E}{F_y}}$$

K = effective length factor, which may be taken as unity for bearing stiffeners

L_c = length of member between points of support = D for bearing stiffeners

r = radius of gyration of the column section in the plane of buckling

Intermediate stiffeners must be provided if a girder section does not satisfy Criterion 5, for shear. The depth-thickness ratio of the web with transverse stiffeners should not exceed

$$\frac{D}{t_w} = \frac{36,500}{\sqrt{F_y}}$$

For composite girders and other girders unsymmetrical about the horizontal centroidal axis, if D_c , the clear distance between neutral axis and compression flange, exceeds $D/2$, the depth-thickness ratio should not exceed

$$\frac{D_c}{t_w} = \frac{18,250}{\sqrt{F_y}}$$

The shear capacity of girder webs with transverse stiffeners is given by

$$V_u = V_p \left[C + \frac{0.87(1-C)}{\sqrt{1+(d_o/D)^2}} \right]$$

where $V_p = 0.58F_y D t_w$

d_o = distance between transverse stiffeners

$$C = 18,000 \frac{t_w}{D} \sqrt{\frac{1+(D/d_o)^2}{F_y}} - 0.3 \leq 1.0$$

The effect of shear on the bending strength of a girder can usually be ignored. If, however, the shear V on any panel of a girder with transverse stiffeners exceeds $0.6V_u$, the moment M at that section should be limited to

$$M = M_u \left(1.375 - 0.625 \frac{V}{V_u} \right)$$

where M_u is the bending strength of the section unreduced for shear.

Spacing of transverse stiffeners along a girder should not exceed the distance d_o determined from the preceding formula for V_u nor $1.5D$. At simply supported ends of girders, though, the first stiffener space may not be larger than D nor

$$d_o = 14,500 \sqrt{\frac{D t_w^3}{V}}$$

Transverse stiffeners should be proportioned so that the width-thickness ratio does not exceed

$$\frac{b'}{t} = \frac{2,600}{\sqrt{F_y}}$$

Also, the gross cross-sectional area of each one-sided stiffener or pair of two-sided stiffeners should be at least

$$A = Y \left[0.15 B D t_w (1-C) \frac{V}{V_u} - 18 t_w^2 \right]$$

where Y = ratio of web yield strength to stiffener yield strength

$B = 1.0$ for stiffener pairs

$= 1.8$ for single angles

$= 2.4$ for single plates

C is the same as for the computation of V_u .

In addition, the required moment of inertia of each stiffener with respect to the mid-plane of the web is

$$I = d_o t_w^3 J$$

where $J = 2.5 \left(\frac{D}{d_o} \right)^2 - 2 \geq 0.5$

Transverse stiffeners need not bear on a tension flange, but the maximum distance between a face of that flange and the nearest web-to-stiffener weld should not exceed $6t_w$. When stiffeners are provided on only one side of the web, they should bear on the compression flange, but need not be attached to it.

DESIGN FOR OVERLOAD

To guard against objectionable deformation under occasional overloads, the following moment relationship must be observed for noncomposite sections and negative bending of composite sections.

$$0.8F_y S \geq \left[D + \frac{5}{3}(L + I) \right]$$

For the same purpose, composite sections in positive bending must satisfy the relationship

$$0.95F_y S \geq \left[D + \frac{5}{3}(L + I) \right]$$

DESIGN FOR SERVICE LOADS

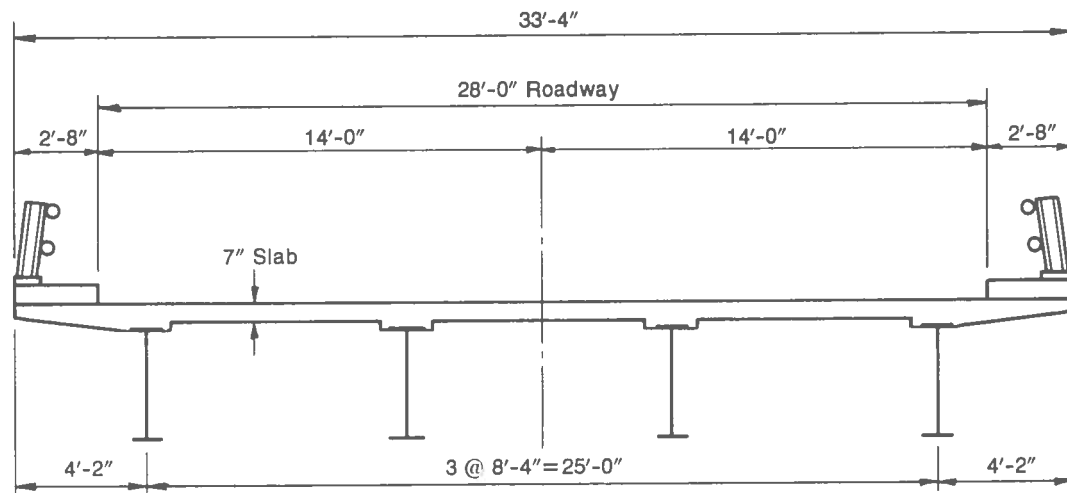
Fatigue is investigated in the same manner as in working stress design, using service loads and the provisions of AASHTO Art. 1.7.3. If the longitudinal reinforcing steel in tension over the negative moment region is considered in computing section properties, the stress range in the reinforcing steel is limited to 20,000 psi.

SHEAR CONNECTORS

Provisions for shear connectors in load factor design are identical to provisions for working-stress design. These are illustrated in Chapter 3A.

Design Example—Two-Span Continuous Girder (100-100 Ft) Composite for Positive and Negative Moment

To illustrate load factor design, an interior girder of a two-span bridge, similar to Design III of Chapter 4, will be designed. The section in the positive-moment region consists of the steel girder acting compositely with the concrete slab. In the negative-moment region, the section consists of the steel girder and the longitudinal slab reinforcing steel. The following data apply to this design:



TYPICAL SECTION

Roadway Section: The same as that shown for Design I in Chapter 4 (see Typical Section).

Specifications: 1973 AASHTO Standard Specifications for Highway Bridges, Interims 1974, 1975 & 1976

Loading: HS20-44

Structural Steel: ASTM A588, Grade A, with $F_y = 50,000$ psi

Concrete: $f'_c = 4,000$ psi, modular ratio $n = 8$

Slab Reinforcing Steel: ASTM A615, Grade 40 with $F_y = 40,000$ psi

Loading Conditions:

Case 1—Weight of girder and slab (DL_1) supported by the steel girder alone.

Case 2—Superimposed dead load (DL_2) (curbs and railings) supported by the composite section with the modular ratio $n = 8$.

Case 3—Superimposed dead load (DL_2) (curbs and railings) supported by the composite section with the increased modular ratio $3n = 3 \times 8 = 24$.

Case 4—Live load plus impact ($L+I$) supported by the composite section with the modular ratio $n = 8$.

Loading Combinations:

Combination A = Case 1+3+4.

Combination B = Case 1+2+4.

Stress Cycles for Fatigue:

500,000 cycles of truck loading.

100,000 cycles of lane loading.

LOADS, SHEARS AND MOMENTS

Analysis is based on the assumption of constant moment of inertia throughout the length of the girder.

The initial dead load DL_1 consists of an estimated weight of 0.170 kips per ft for the girder and framing details, plus the weight of the 7-in.-thick concrete deck slab. The dead load DL_2 carried by the composite section is made up of the weight of the curbs and railings. The live load is AASHTO HS20-44 truck loading with impact for a 100-ft span.

Dead Load Carried by Steel

$$\text{Slab} = 7/12 \times 8.33 \times 0.150 = 0.730$$

$$\text{Steel girder, details and conc. haunch} = 0.170$$

$$DL_1 \text{ per girder} = 0.900 \text{ k/ft}$$

Dead Load Carried by Composite Section*

$$\text{Curbs and railings, } DL_2 = 0.660 \text{ k/ft}$$

$$DL_2 \text{ per girder} = 0.660/4 = 0.165 \text{ k/ft}$$

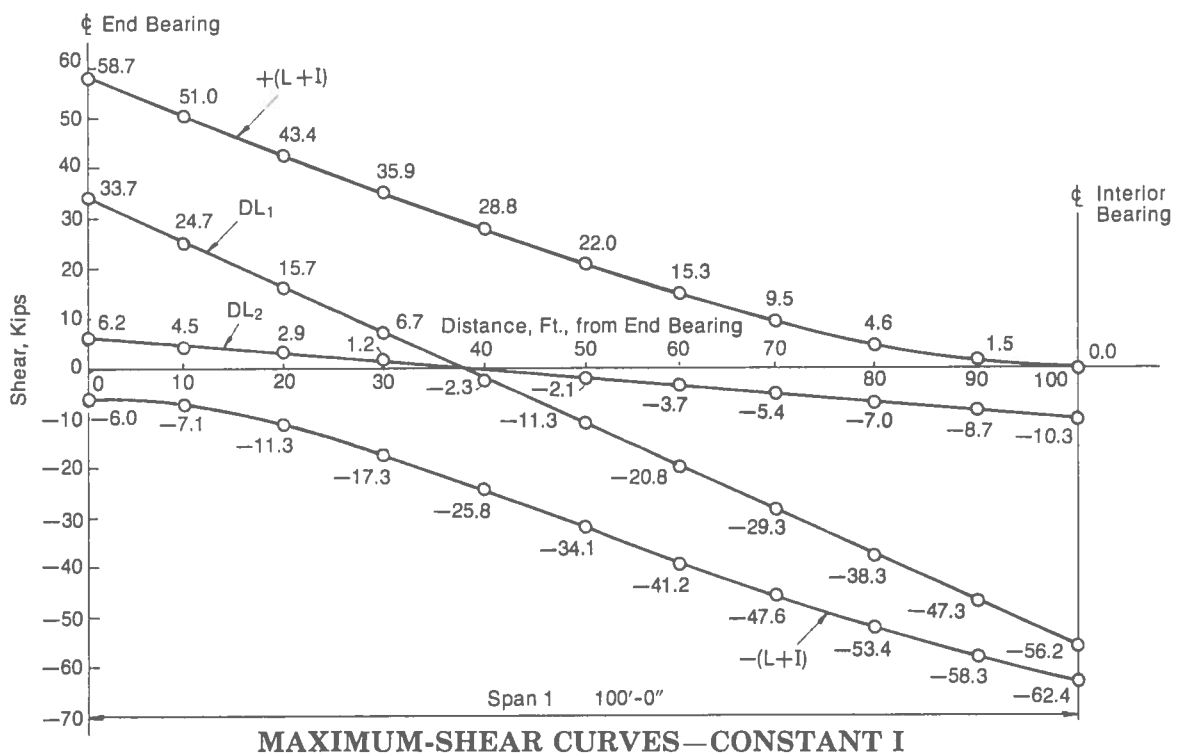
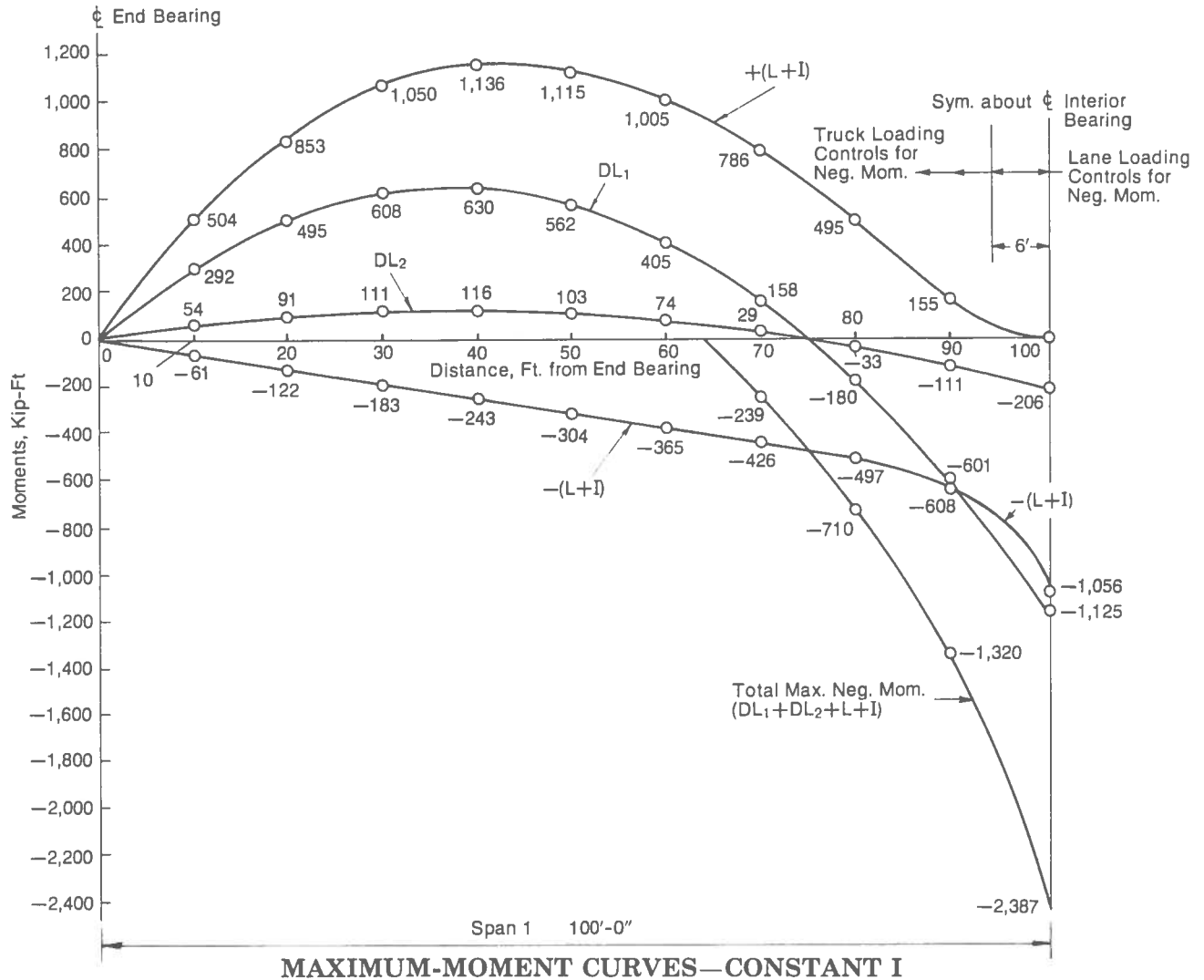
Live Load

$$\text{Live-load distribution} = \frac{S}{5.5} = \frac{8.33}{5.5} = 1.51 \text{ wheels} = 0.755 \text{ axles}$$

$$\text{Impact} = \frac{50}{100 + 125} = 0.222$$

*No future wearing surface is anticipated for this bridge. If a future wearing surface will be required, its weight must be included in the dead load carried by the composite section and distributed equally to all stringers.

The curves shown for maximum moment and maximum shear may be calculated by any convenient method.



DESIGN OF GIRDER SECTION

In recognition of the fact that normal welded plate girders never satisfy the requirements for compactness, the section is assumed to be either a braced or unbraced, noncompact section. Minimum material requirements for either classification of section are calculated for a trial section with anticipated flange widths of 10 and 14 in. and a web depth of 42 in.

Thicknesses for Noncompact Section

For $F_y = 50,000$ psi, the criteria for width-thickness ratio of the projecting compression flanges becomes $b'/t \leq 9.8$ assuming $M = M_u$. Hence, the minimum thickness for a 14-in. compression flange in negative bending, with $b' = 7$ in., is

$$t = \frac{7}{9.8} = 0.714: \text{ use } \frac{3}{4} \text{ in.}$$

For a 10-in. compression flange in negative bending, with $b' = 5$ in., the minimum thickness is

$$t = \frac{5}{9.8} = 0.510: \text{ use } \frac{9}{16} \text{ in.}$$

To satisfy the criterion for unstiffened web depth-thickness ratio $D/t_w \leq 150$, the web thickness for $D = 42$ in. must be at least

$$t_w = \frac{42}{150} = 0.280: \text{ use } \frac{5}{16} \text{ in.}$$

If in subsequent computations it is found that $M < M_u$, these thicknesses may be reduced by the ratio $\sqrt{\frac{M}{M_u}}$

STIFFENED WEB

Webs may be designed as fully stiffened, partly stiffened, or unstiffened. The best current design practice seeks to achieve maximum economy by minimizing fabrication. Thus, an optimum design would likely incorporate either a partly stiffened or unstiffened web. Initially, a $\frac{3}{8}$ -in. stiffened web is investigated.

The maximum shear permitted on a $\frac{3}{8}$ -in. web without stiffeners is the smaller of the following:

$$V = \frac{3.5Et_w^3}{D} = \frac{3.5 \times 29,000 \left(\frac{3}{8}\right)^3}{42} = 127 \text{ kips}$$

$$V = 0.58F_yDt_w = 0.58 \times 50 \times 42 \times \frac{3}{8} = 457 > 127 \text{ kips}$$

Maximum Shearing Stress at Supports, Kips

	DL_1	DL_2	$L+I$
At Interior Support	56.2	10.3	62.4
At End Bearing	33.7	6.2	58.7

The shear for Maximum Design Loads at the interior support is

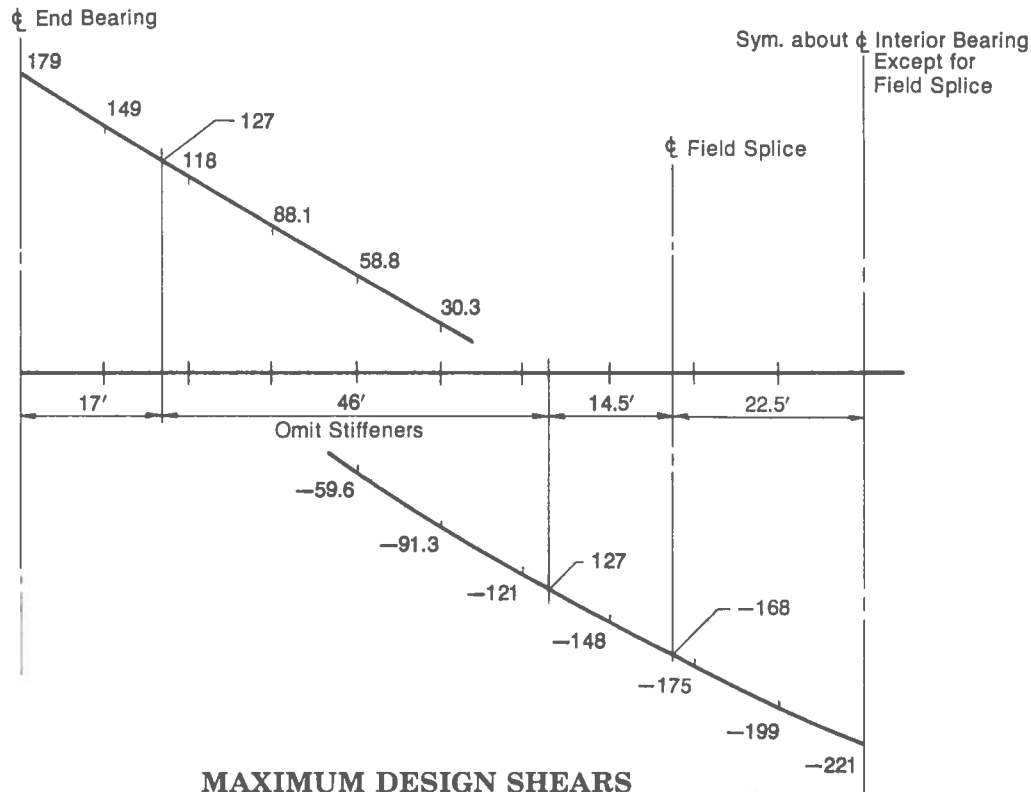
$$V = 1.30 \left(56.2 + 10.3 + \frac{5}{3} \times 62.4 \right) = 222 > 127 \text{ kips}$$

Similarly, the shear at the end bearing is

$$V = 1.30 \left(33.7 + 6.2 + \frac{5}{3} \times 58.7 \right) = 179 > 127 \text{ kips}$$

Because the shear exceeds that permitted at both the interior support and the end bearing, stiffeners are required at both locations.

If, however, the shears for Maximum Design Loads are plotted along the girder span, there is a region within the span for which the maximum shear at any section is less than the 127 kip capacity of the section. As indicated in the Maximum Design Shear Diagram, web stiffeners may be omitted over a 46-ft-long length in the positive-moment region.



Stiffener Spacing at Interior Support

Next, the shear capacity is calculated for the stiffened web at the interior support, with the assumptions that the diaphragms will be spaced at 20 ft and that the stiffeners will be equally spaced within the diaphragm panels. The shear capacity depends on the stiffener spacing. Also, the moment capacity depends on the stiffener spacing, because the permitted moment is reduced if the shear exceeds 0.6 the shear capacity. A reasonable procedure, therefore, is to space stiffeners in the negative-moment region, where shear and moment are large, so that there is no reduction in moment capacity. A trial stiffener spacing $d_o = 54$ in. $= 1.286D < 1.50D$ is investigated.

For use in the formula for V_u ,

$$V_p = 0.58F_y D t_w = 0.58 \times 50 \times 42 \times \frac{3}{8} = 457 \text{ kips}$$

Also, for use in the formula for V_u ,

$$C = 18,000 \frac{t_w}{D} \sqrt{\frac{1 + (D/d_o)^2}{F_y}} - 0.3 = 18,000 \times \frac{3/8}{42} \sqrt{\frac{1 + (42/54)^2}{50,000}} - 0.3 = 0.610$$

Substitution of these values gives the shear capacity as

$$V_u = V_p \left[C + \frac{0.87(1-C)}{\sqrt{1 + (d_o/D)^2}} \right] = 457 \left[0.610 + \frac{0.87(1-0.610)}{\sqrt{1 + (54/42)^2}} \right] = 374 \text{ kips}$$

$$0.6V_u = 0.6 \times 374 = 224 > 222 \text{ kips}$$

No reduction in moment capacity is required for stiffener spacings up to 54 in.

Use five spaces at 48 in. in the diaphragm panel adjacent to the pier.

Stiffener Spacing at Field Splice

A field splice, 22.5 ft from the pier, lies in the second diaphragm panel from that support. With the decrease in shear with distance from the pier, stiffeners can be placed farther apart than in the first diaphragm panel. The maximum permissible stiffener spacing is $1.5D = 1.5 \times 42 = 63$ in. Calculations indicate that at 60 in. the shear capacity at the field splice is more than adequate. Shear due to Maximum Design Loads at the splice is 168 kips.

For use in the formula for V_u , $V_p = 457$ kips and

$$C = 18,000 \times \frac{3}{8} \sqrt{\frac{1 + (42/60)^2}{50,000}} - 0.3 = 0.577$$

Substitution of these values gives the shear capacity at the splice as

$$V_u = 457 \left[0.577 + \frac{0.87(1 - 0.577)}{\sqrt{1 + (60/42)^2}} \right] = 360 \text{ kips}$$

$$0.6V_u = 0.6 \times 360 = 216 > 168 \text{ kips}$$

Stiffener Spacing at End Bearing

AASHTO Specifications require that the first stiffener space at the end bearing be limited for a $\frac{3}{8}$ -in. web to

$$d_o = 14,500 \sqrt{\frac{Dt_w^3}{V}} = 14,500 \sqrt{\frac{42(\frac{3}{8})^3}{179,100}} = 51.0 \text{ in.}$$

Since this spacing exceeds the web depth, the first stiffener spacing is set equal to 42 in. The shear capacity of the web with this stiffener spacing is then investigated. For use in the formula for V_u , $V_p = 457$ kips and

$$C = 18,000 \times \frac{3}{8} \sqrt{\frac{1 + (42/42)^2}{50,000}} - 0.3 = 0.716$$

Substitution of these values gives the shear capacity at the end bearing as

$$V_u = 457 \left[0.716 + \frac{0.87(1 - 0.716)}{\sqrt{1 + (42/42)^2}} \right] = 407 > 179 \text{ kips}$$

Stiffener Spacing 42 In. from End Bearing

The remaining stiffeners within the first diaphragm panel are placed at 49½ in. intervals, and the shear in the web is investigated for this spacing at a distance of 42 in. from the end bearing. Shear due to the factored loads is 168 kips. For use in the formula for V_u , $V_p = 457$ kips and

$$C = 18,000 \times \frac{3}{8} \sqrt{\frac{1 + (42/49.5)^2}{50,000}} - 0.3 = 0.643$$

Substitution of these values gives the shear capacity as

$$V_u = 457 \left[0.643 + \frac{0.87(1 - 0.643)}{\sqrt{1 + (49.5/42)^2}} \right] = 386$$

$$0.6V_u = 0.6 \times 386 = 231 > 168 \text{ kips}$$

UNSTIFFENED WEB

As an alternate design, an unstiffened web is investigated. Calculations show that without stiffeners a thickness of $\frac{1}{16}$ in. is required for the positive-moment region and a thickness of $\frac{1}{2}$ in. for the negative-moment region.

Required minimum thickness of web is determined from the criterion for shear capacity of an unstiffened web:

$$t = \sqrt[3]{\frac{VD}{3.5E}}$$

At the end bearing, the shear has previously been calculated to be 179 kips. Hence the web thickness there must be at least

$$t = \sqrt[3]{\frac{179 \times 42}{3.5 \times 29,000}} = 0.420: \text{ use } \frac{7}{16} \text{ in.}$$

At the interior support, the shear has previously been computed to be 222 kips, for which the web thickness must be at least

$$t = \sqrt[3]{\frac{222 \times 42}{3.5 \times 29,000}} = 0.451: \text{ use } \frac{1}{2} \text{ in.}$$

FATIGUE REQUIREMENTS

Before the design of positive and negative moment sections is begun, it may be helpful to determine in general what fatigue checks should be made. For this welded plate girder with butt welded flange transitions, fillet welded web stiffeners or diaphragm connection plates and stud shear connectors, fatigue should be checked under tension or reversal at the following locations:

AASHTO Category C

1. Base metal adjacent to stud-type shear connectors.
2. Base metal in the girder web at the toe of transverse stiffener (or diaphragm connection plate) fillet welds.

AASHTO Category B

3. Base metal adjacent to full penetration groove welded flange transitions.
4. Base metal adjacent to continuous fillet welds parallel to applied stress in bottom flanges.

Groove welded flange splices at transitions in width or thickness can be assigned AASHTO fatigue category B providing that transition slopes not exceeding 1 to 2½ are used and that the welds are finished smooth and flush.

At maximum negative moment locations or at transitions in negative moment regions, the top flange is in tension. If shear studs are connected to this flange, AASHTO fatigue category C may govern the maximum stress range there, if less than that permitted by rebar fatigue.

At maximum positive moment locations or at transitions in positive moment regions, the bottom flange is in tension. If a transverse stiffener or diaphragm connection plate is near, the stress range in the web is governed by fatigue category C. Also, if the section is at a groove welded flange splice, the maximum bottom flange stress range cannot exceed that of fatigue category B.

Girder sections near points of contraflexure where stress reversals are likely to occur must be checked for category C allowables at the top flange where shear connectors are likely to be attached and in the bottom of the web if a transverse stiffener or diaphragm connection plate is located there. Also, the bottom flange must be checked for fatigue category B if the section is adjacent to a groove welded splice.

The AASHTO Specifications assign the following allowable ranges of stress to categories B and C:

	500,000 cycles (Truck Loading)	100,000 cycles (Lane Loading)
Category B	27.5 ksi	45.0 ksi
Category C	19.0 ksi	32.0 ksi

MAXIMUM NEGATIVE MOMENT AT INTERIOR SUPPORT

The girder will be designed with a stiffened web. The negative-moment section at the interior support is designed first. The section will be noncompact and either braced or unbraced. The design relationship for Maximum Design Load is

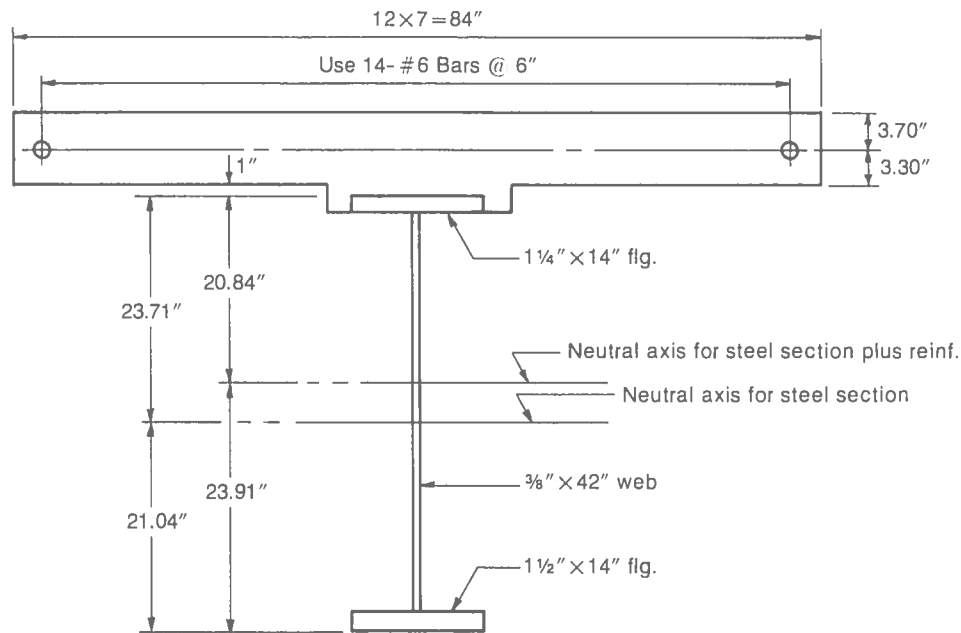
$$F_y S \geq 1.30 \left[D + \frac{5}{3}(L + I) \right]$$

and that for Overload is

$$0.80 F_y S \geq \left[D + \frac{5}{3}(L + I) \right] \text{ or } F_y S \geq 1.25 \left[D + \frac{5}{3}(L + I) \right]$$

By inspection, the Maximum Design Load relationship governs.

A section made up of a $1\frac{1}{2} \times 14$ -in. bottom flange, a $1\frac{1}{4} \times 14$ -in. top flange and a $\frac{3}{8} \times 42$ -in. web plate is tried. The slab contains 14 No. 6 longitudinal bars at 6 in. spacing.



MAXIMUM-NEGATIVE-MOMENT SECTION

With diaphragms spaced at 20 ft, the unbraced length of compression flange exceeds the maximum unbraced length for a braced section.

$$L_b = \frac{20,000,000 A_f}{F_y d} = \frac{20,000,000 \times 21}{50,000 \times 44.75} = 188 \text{ in.} = 15.7 \text{ ft} < 20 \text{ ft}$$

The negative-moment section, therefore, is an unbraced, noncompact section. As a result, the calculated maximum bending strength is defined by

$$M_u = F_y S \left[1 - \frac{3 F_y}{4 \pi^2 E} \left(\frac{L_b}{0.9 b'} \right)^2 \right]$$

or, dividing through by S , the critical lateral buckling stress for the compression flange is expressed as

$$F_{cr} = \frac{M_u}{S} = F_y \left[1 - \frac{3 F_y}{4 \pi^2 E} \left(\frac{L_b}{0.9 b'} \right)^2 \right]$$

Assuming that the compression flange does not change in width within the unbraced length, the critical allowable compression stress becomes

$$F_v \left[1 - \frac{3 \times 50,000}{4\pi^2 \times 29,000,000} \left(\frac{20 \times 12}{0.9 \times 6.81} \right)^2 \right] = 0.799F_v$$

If the ratio of moments at the two ends of the unbraced length is less than 0.7, however, this stress may be increased by 20% but not to more than F_v .

Maximum Negative Moments, Kip-Ft

	DL_1	DL_2	$L+I$	Total
At Interior Support	-1,125	-206	-1,056	-2,387
At Diaphragm	-180	-33	-497	-710

The ratio of the total factored moments is

$$R = \frac{1.30 \left[-180 - 33 + \frac{5}{3}(-497) \right]}{1.30 \left[-1,125 - 206 + \frac{5}{3}(-1,056) \right]} = 0.3 < 0.7$$

Hence, the allowable stress may be increased up to 20%.

$$1.20 \times 0.799F_v = 0.959F_v$$

From the preceding, the design relationship for Maximum Design Load at the maximum-negative-moment location is given by

$$1.30 \left[D + \frac{5}{3}(L+I) \right] \leq 0.959F_v S$$

and the design relationship for Overload is given by

$$D + \frac{5}{3}(L+I) \leq 0.80F_v S$$

By inspection, the Maximum Design Load relationship governs.

Section properties are calculated for the girder section alone and for the girder section plus the longitudinal reinforcing bars in the concrete slab. The allowable stresses in the girder section are

$$\begin{aligned} F_b &= 0.959F_v = 0.959(50) = 48.0 \text{ ksi for compression flange} \\ F_b &= F_v &= 50.0 \text{ ksi for tension flange} \\ F_v^r & &= 40.0 \text{ ksi for rebars} \end{aligned}$$

Steel Section at Interior Support

Material	A	d	Ad	Ad^2	I_o	I
T. Flg. $1\frac{1}{4} \times 14$	17.50	21.62	378	8,184		8,184
B. Flg. $1\frac{1}{2} \times 14$	21.00	-21.75	-457	9,934		9,934
Web $\frac{3}{8} \times 42$	15.75				2,315	2,315

$$54.25 \text{ in.}^2$$

$$-79 \text{ in.}^3$$

$$20,433$$

$$d_s = \frac{-79}{54.25} = -1.46 \text{ in.}$$

$$-1.46(79) = -\frac{115}{I_{NA}} = 20,318 \text{ in.}^4$$

$$d_{\text{Top of steel}} = 22.25 + 1.46 = 23.71 \text{ in.}$$

$$d_{\text{Bot. of steel}} = 22.50 - 1.46 = 21.04 \text{ in.}$$

$$S_{\text{Top of steel}} = \frac{20,318}{23.71} = 857 \text{ in.}^3$$

$$S_{\text{Bot. of steel}} = \frac{20,318}{21.04} = 966 \text{ in.}^3$$

Steel Section with Reinforcing Steel at Interior Support

Material	A	d	Ad	Ad ²	I _o	I
Girder	54.25		- 79			20,433
Reinf. 14- #6	6.16	26.55	164	4,342		4,342

$$60.41 \text{ in.}^2$$

$$85 \text{ in.}^3$$

$$24,775$$

$$d_s = \frac{85}{60.41} = 1.41 \text{ in.}$$

$$-1.41(85) = -\frac{120}{I_{NA}} = 24,655 \text{ in.}^4$$

$$d_{\text{Top of steel}} = 22.25 - 1.41 = 20.84 \text{ in.}$$

$$d_{\text{Bot. of steel}} = 22.50 + 1.41 = 23.91 \text{ in.}$$

$$S_{\text{Top of steel}} = \frac{24,655}{20.84} = 1,183 \text{ in.}^3$$

$$S_{\text{Bot. of steel}} = \frac{24,655}{23.91} = 1,031 \text{ in.}^3$$

$$d_{\text{Reinf.}} = 20.84 + 1 + 3.30 = 25.14 \text{ in.}$$

$$S_{\text{Reinf.}} = \frac{24,655}{25.14} = 981 \text{ in.}^3$$

Steel Stresses for Maximum Negative Moment Due to Maximum Design Loads

Top of Steel (Tension)

Bottom of Steel (Compression)

$$\text{For } DL_1: F_b = \frac{1,125(12)}{857} \times 1.3 = 20.5$$

$$F_b = \frac{1,125(12)}{966} \times 1.3 = 18.2$$

$$\text{For } DL_2: F_b = \frac{206(12)}{1,183} = 2.7$$

$$F_b = \frac{206(12)}{1,031} \times 1.3 = 3.1$$

$$\text{For } LL + I: F_b = \frac{1,056(12)}{1,183} \times 1.3 \times \frac{5}{3} = 23.2$$

$$F_b = \frac{1,056(12)}{1,031} \times 1.3 \times \frac{5}{3} = 26.6$$

$$46.4 < 50 \text{ ksi}$$

$$47.9 < 48.0 \text{ ksi}$$

Reinforcing Steel Stress (Tension)

$$F_b = \frac{1.3 \times 12 \left[206 + \frac{5}{3} \times 1,056 \right]}{981} = 31.3 < 40 \text{ ksi}$$

Fatigue stress range in the reinforcing steel is limited to 20 ksi. The Service Load stress range is computed to be

$$f_{sr} = \frac{1,056(12)}{981} = 12.9 \text{ ksi} < 20 \text{ ksi}$$

In addition to Maximum Design Load, the maximum-negative-moment section should be checked for fatigue at the stud shear connector weld. Assuming that a row of connectors will be placed on the top flange near the interior support, the live load stress range at this location is determined to be

$$f_{sr} = \frac{1,056(12)}{1,183} = 10.7 < 32 \text{ ksi (Lane Load Controls)}$$

The section is satisfactory in fatigue near the interior support. By inspection, the connection of the stud shear connector is more critical than the top flange connection of the bearing stiffener.

MAXIMUM POSITIVE MOMENT

The maximum-positive-moment section qualifies as a braced, noncompact section, because the compression flange is braced throughout by the concrete slab. A trial section comprising a $\frac{9}{16} \times 10$ -in. top flange (minimum material), $\frac{3}{8} \times 42$ -in. web, and

1½ × 10-in. bottom flange is selected to meet strength requirements. Properties are calculated for the steel section alone, the composite section with modular ratio 3n = 24, and the composite section with modular ratio n = 8.

Steel Section

Material	A	d	Ad	Ad ²	I _o	I
Top Flg. ¾ × 10	5.63	21.28	120	2,549	2,315	2,549
Web ¾ × 42	15.75					2,315
Bot. Flg. 1½ × 10	15.00	-21.75	-326	7,096		7,096
	36.38 in. ²		-206 in. ³			11,960

$$d_s = \frac{-206}{36.38} = -5.66 \text{ in.} \quad -5.66 \times 206 = -1,166$$

$$I_{NA} = \frac{10,794}{10,794} \text{ in.}^4$$

$$d_{\text{Top of steel}} = 21.56 + 5.66 = 27.22 \text{ in.} \quad d_{\text{Bot. of steel}} = 22.50 - 5.66 = 16.84 \text{ in.}$$

$$S_{\text{Top of steel}} = \frac{10,794}{27.22} = 397 \text{ in.}^3 \quad S_{\text{Bot. of steel}} = \frac{10,794}{16.84} = 641 \text{ in.}^3$$

Composite Section, 3n = 24

Material	A	d	Ad	Ad ²	I _o	I
Steel Section	36.38		-206		100	11,960
*Conc. 84 × 7/24	24.50	26.75	655	17,531		17,631
	60.88 in. ²		449 in. ³			29,591

$$d_{24} = \frac{449}{60.88} = 7.38 \text{ in.} \quad -7.38 \times 449 = -3,314$$

$$I_{NA} = \frac{26,277}{26,277} \text{ in.}^4$$

$$d_{\text{Top of steel}} = 21.56 - 7.38 = 14.18 \text{ in.} \quad d_{\text{Bot. of steel}} = 22.50 + 7.38 = 29.88 \text{ in.}$$

$$S_{\text{Top of steel}} = \frac{26,277}{14.18} = 1,853 \text{ in.}^3 \quad S_{\text{Bot. of steel}} = \frac{26,277}{29.88} = 879 \text{ in.}^3$$

Composite Section, n = 8

Material	A	d	Ad	Ad ²	I _o	I
Steel Section	36.38		-206		300	11,960
Conc. 84 × ¾	73.50	26.75	1,966	52,594		52,894
	109.88 in. ²		1,760 in. ³			64,854

$$d_8 = \frac{1,760}{109.88} = 16.02 \text{ in.} \quad -16.02 \times 1,760 = -28,195$$

$$I_{NA} = \frac{36,659}{36,659} \text{ in.}^4$$

$$d_{\text{Top of steel}} = 21.56 - 16.02 = 5.54 \text{ in.} \quad d_{\text{Bot. of steel}} = 22.50 + 16.02 = 38.52 \text{ in.}$$

$$S_{\text{Top of steel}} = \frac{36,659}{5.54} = 6,617 \text{ in.}^3 \quad S_{\text{Bot. of steel}} = \frac{36,659}{38.52} = 952 \text{ in.}^3$$

The design relationship for Maximum Design Loads is

$$F_y S \geq 1.30 \left[D + \frac{5}{3} (L + I) \right]$$

*When the design does not include a wearing surface on the bridge deck, one-half inch is sometimes subtracted from the slab depth to account for wear from traffic. This was not done in this example design.

The design relationship for Overload, however, is

$$0.95F_y S \geq \left[D + \frac{5}{3}(L+I) \right] \text{ or } F_y S \geq 1.053 \left[D + \frac{5}{3}(L+I) \right]$$

By inspection, the Maximum Design Load governs, and the allowable stress is $F_y = 50$ ksi. Maximum positive moment occurs 40 ft from the end support.

Bending Moments 40 Ft from End Support

	DL_1	DL_2	$L+I$
M , kip-ft	630	116	1,136

Steel Stresses—Combination A

Top of Steel (Compression)

$$\text{For } DL_1: F_b = \frac{630 \times 12}{397} \times 1.30 = 24.8$$

$$\text{For } DL_2: F_b = \frac{116 \times 12}{1,853} \times 1.30 = 1.0$$

$$\text{For } L+I: F_b = \frac{1,136 \times 12}{6,617} \times 1.30 \times \frac{5}{3} = 4.5$$

$$30.3 < 50 \text{ ksi}$$

Bottom of Steel (Tension)

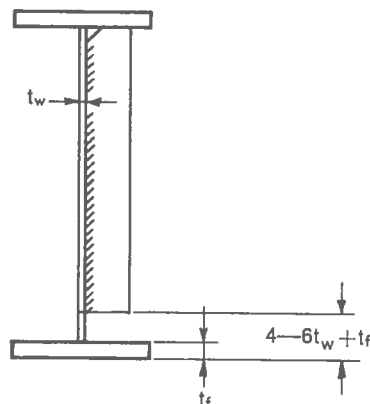
$$F_b = \frac{630 \times 12}{641} \times 1.30 = 15.3$$

$$F_b = \frac{116 \times 12}{879} \times 1.30 = 2.1$$

$$F_b = \frac{1,136 \times 12}{952} \times 1.30 \times \frac{5}{3} = 31.0$$

$$48.4 < 50 \text{ ksi}$$

In addition to Maximum Design Loads, the section used for maximum positive moment must also be investigated for fatigue at the toe of the transverse stiffener fillet weld. To accommodate painting and drainage of moisture, the transverse stiffeners are normally cut back at least 1 in. from the inside face of the tension flange. Furthermore, to lessen fatigue stresses during transportation, it is recommended practice to terminate the stiffener-to-web weld a distance of four to six times the web thickness from the inside face of the tension flange. Using four times the web thickness, the maximum bending stress at the toe of the stiffener fillet weld is given by



$$f'_b = \frac{M[y - (4t_w + t_f)]}{I}$$

The maximum range of live load moments in the positive-moment region occurs at the diaphragm connection stiffener at the 0.4 point of span 1.

$$M_{LL \text{ Range}} = 1,136 - (-243) = 1,379 \text{ kip-ft.}$$

The range of tensile stress at the connection plate fillet weld can then be calculated as

$$f_b = \frac{1,379(12)}{36,659} \times [38.52 - \{ (4)(\frac{3}{8}) + 1.5 \}] = 16.0 < 19 \text{ ksi (Truck Load Controls)}$$

The trial section is satisfactory for maximum positive moment.

FLANGE-PLATE TRANSITION 17 FT FROM END SUPPORT

With the two main sections designed, attention is next directed to the transition sections. At 17 ft from the end bearing, the thickness of the bottom flange of the maximum-positive-moment section is stepped down from $1\frac{1}{2}$ to 1 in. Properties are calculated for the steel and composite sections.

Steel Section

Material	A	d	Ad	Ad ²	I _o	I
Top Flg. $\frac{9}{16} \times 10$	5.63	21.28	120	2,549	2,315	2,549
Web $\frac{3}{8} \times 42$	15.75					2,315
Bot. Flg. 1×10	10.00	-21.50	-215	4,623		4,623

$$31.38 \text{ in.}^2$$

$$-95 \text{ in.}^3$$

$$9,487$$

$$d_s = \frac{-95}{31.38} = -3.03 \text{ in.}$$

$$-3.03 \times 95 = -288$$

$$I_{NA} = \frac{288}{9,199} \text{ in.}^4$$

$$d_{\text{Top of steel}} = 21.56 + 3.03 = 24.59 \text{ in.}$$

$$d_{\text{Bot. of steel}} = 22.0 - 3.03 = 18.97 \text{ in.}$$

$$S_{\text{Top of steel}} = \frac{9,199}{24.59} = 374 \text{ in.}^3$$

$$S_{\text{Bot. of steel}} = \frac{9,199}{18.97} = 485 \text{ in.}^3$$

Composite Section, $3n = 24$

Material	A	d	Ad	Ad ²	I _o	I
Steel Section	31.38		-95		100	9,487
Conc. $84 \times 7/24$	24.50	26.75	655	17,531		17,631

$$55.88 \text{ in.}^2$$

$$560 \text{ in.}^3$$

$$27,118$$

$$d_{24} = \frac{560}{55.88} = 10.02 \text{ in.}$$

$$-10.02 \times 560 = -5,611$$

$$I_{NA} = \frac{5,611}{21,507}$$

$$d_{\text{Top of steel}} = 21.56 - 10.02 = 11.54 \text{ in.}$$

$$d_{\text{Bot. of steel}} = 22.00 + 10.02 = 32.02 \text{ in.}$$

$$S_{\text{Top of steel}} = \frac{21,507}{11.54} = 1,864 \text{ in.}^3$$

$$S_{\text{Bot. of steel}} = \frac{21,507}{32.02} = 672 \text{ in.}^3$$

Composite Section, $n = 8$

Material	A	d	Ad	Ad ²	I _o	I
Steel Section	31.38		-95		300	9,487
Conc. $84 \times \frac{7}{8}$	73.50	26.75	1,966	52,594		52,894

$$104.88 \text{ in.}^2$$

$$1,871 \text{ in.}^3$$

$$62,381$$

$$d_8 = \frac{1,871}{104.88} = 17.84 \text{ in.}$$

$$-17.84 \times 1,871 = -33,379$$

$$I_{NA} = \frac{33,379}{29,002} \text{ in.}^4$$

$$d_{\text{Top of steel}} = 21.56 - 17.84 = 3.72 \text{ in.}$$

$$d_{\text{Bot. of steel}} = 22.00 + 17.84 = 39.84 \text{ in.}$$

$$S_{\text{Top of steel}} = \frac{29,002}{3.72} = 7,796 \text{ in.}^3$$

$$S_{\text{Bot. of steel}} = \frac{29,002}{39.84} = 728 \text{ in.}^3$$

As with previous sections, Maximum Strength is more critical than Overload and is investigated. Also, fatigue in base metal adjacent to the butt-welded flange transition and fatigue in the web at the toe of transverse stiffener fillet welds must be checked.

Bending Moments 17 Ft from End Support

	DL ₁	DL ₂	-(L+I)	+(L+I)
M, kip-ft	440	80	-104	739

Steel Stresses 17 Ft from End Support Due to Maximum Design Loads

Top of Steel (Compression)	Bottom of Steel (Tension)
For DL_1 : $F_b = \frac{440 \times 12}{374} \times 1.30 = 18.3$	$F_b = \frac{440 \times 12}{485} \times 1.30 = 14.1$
For DL_2 : $F_b = \frac{80 \times 12}{1,864} \times 1.30 = 0.7$	$F_b = \frac{80 \times 12}{672} = 1.9$
For $L+I$: $F_b = \frac{739 \times 12}{7,796} \times 1.30 \times \frac{5}{3} = 2.5$	$F_b = \frac{739 \times 12}{728} \times 1.30 \times \frac{5}{3} = 26.4$
$21.5 < 50$ ksi	$42.4 < 50$ ksi

Strength of the section is satisfactory. Fatigue is investigated next.

The range of stress in the bottom flange at the transition is

$$f_{cr} = \frac{(739 + 104)(12)}{728} = 13.9 < 27.5 \text{ ksi (Truck Load Controls)}$$

A transverse web stiffener is located 1'-1 1/2" away from the flange transition where the bottom flange has been reduced to a 1 in. thickness. By observation of the stress range at the transition, it can be concluded that the stress range in the girder web at the toe of the stiffener fillet weld does not exceed the 19 ksi allowable for such a detail.

Resistance to fatigue is therefore satisfactory at the transition 17 ft from the end support.

FLANGE TRANSITIONS 6 FT FROM INTERIOR SUPPORT

A transition in section in the negative-moment region is located 6 ft from the interior support. The top 14-in.-wide flange plate is reduced in thickness from 1 1/4 to 3/4 in. and the bottom flange plate is reduced in thickness from 1 1/2 to 1 in. Properties are calculated for the steel section alone and for the steel section plus the longitudinal reinforcing in the slab.

Steel Section 6 Ft from Interior Support

Material	A	d	Ad	Ad ²	I _o	I
Top Flg. 3/4 × 14	10.50	21.37	224	4,795	2,315	4,795
Web 3/8 × 42	15.75					2,315
Bot. Flg. 1 × 14	14.00	-21.50	-301	6,472		6,472
	40.25 in. ²		- 77 in. ³			13,582

$$d_s = \frac{-77}{40.25} = -1.91 \text{ in.} \quad -1.91(77) = -147 \quad I_{NA} = 13,435 \text{ in.}^4$$

$$d_{\text{Top of steel}} = 21.75 + 1.91 = 23.66 \text{ in.} \quad d_{\text{Bot. of steel}} = 22.00 - 1.91 = 20.09 \text{ in.}$$

$$S_{\text{Top of steel}} = \frac{13,435}{23.66} = 568 \text{ in.}^3 \quad S_{\text{Bot. of steel}} = \frac{13,435}{20.09} = 669 \text{ in.}^3$$

Steel Section with Reinforcing Steel 6 Ft from Interior Support

Material	A	d	Ad	Ad ²	I _o	I
Steel Section	40.25		- 77			13,582
Reinf. 14- #6	6.16	26.55	164	4,342		4,342
	46.41 in. ²		87 in. ³			17,924

$$d_s = \frac{87}{46.41} = 1.87 \text{ in.} \quad -1.87(87) = -163 \quad I_{NA} = 17,761 \text{ in.}^4$$

$$d_{\text{Top of steel}} = 21.75 - 1.87 = 19.88 \text{ in.} \quad d_{\text{Bot. of steel}} = 22.00 + 1.87 = 23.87 \text{ in.}$$

$$S_{\text{Top of steel}} = \frac{17,761}{19.88} = 893 \text{ in.}^3 \quad S_{\text{Bot. of steel}} = \frac{17,761}{23.87} = 744 \text{ in.}^3$$

$$d_{\text{Reinf.}} = 22.25 - 1.87 + 1.00 + 3.30 = 24.68 \text{ in.}$$

$$S_{\text{Reinf.}} = \frac{17,761}{24.68} = 720 \text{ in.}^3$$

Bending Moments 6 Ft from Interior Support

	DL_1	DL_2	$-(L+I)$	$+(L+I)$
M , kip-ft	-805	-150	-721	+63

The allowable compressive stress for the 14-in.-wide girder flange which is not prismatic within the unbraced length was previously computed to be $0.972F_y = 0.972(50) = 48.6$ ksi.

Steel Stresses 6 Ft from Interior Support Due to Maximum Design Loads

Top of Steel (Tension)	Bottom of Steel (Compression)
For DL_1 : $F_b = \frac{805(12)}{568} \times 1.3 = 22.1$	$F_b = \frac{805(12)}{669} \times 1.3 = 18.8$
For DL_2 : $F_b = \frac{150(12)}{893} \times 1.3 = 2.6$	$F_b = \frac{150(12)}{744} \times 1.3 = 3.1$
For $L+I$: $F_b = \frac{721(12)}{893} \times 1.3 \times \frac{5}{3} = 21.0$	$F_b = \frac{721(12)}{744} \times 1.3 \times \frac{5}{3} = 25.2$
$45.7 < 50$ ksi	$47.1 < 48.6$ ksi

Since shear connectors are attached to the top flange, a fatigue check should be made at the transition to insure that the tensile stress range in the top flange is within the allowable.* The range of live load tensile stress in the top flange is determined to be

$$f_{sr} = \frac{(721+63)(12)}{893} = 10.5 < 19 \text{ ksi}$$

Reinforcing Steel Stress (Tension) 6 Ft from Interior Support

$$f_r = \frac{1.3 \times 12 \left(150 + \frac{5}{3} \times 721 \right)}{720} = 29.3 < 40 \text{ ksi}$$

The fatigue stress range in the reinforcing steel is

$$f_{sr} = \frac{12(721+63)}{720} = 13.1 < 20 \text{ ksi}$$

Thus the reinforcing steel is satisfactory for fatigue

WELDED FIELD SPLICE

A field splice is located 22.5 ft from the pier. This location places the splice midway between a diaphragm and web stiffener and close to the dead-load inflection point.

For the positive-moment side of the splice, a section with a $\frac{3}{4} \times 10$ -in. top flange and a 1×10 -in. bottom flange is tried. These flange plates then have the same thickness as the 14-in.-wide flanges on the negative-moment side of the splice. For a welded field splice, this offers the advantage that transition slopes are not required and, if the splice were to be bolted, filler plates would not be required.

A welded field splice is investigated 22.5 ft from the pier. Section properties are calculated for the steel, steel with slab reinforcing steel and composite sections.

*Connection of studs to the top flange governs over connection of stiffeners to the top flange.

Steel Section on Positive-Moment Side of Splice

Material	A	d	Ad	Ad ²	I _o	I
T. Flg. ¾ × 10	7.50	21.37	160	3,425	2,315	3,425
Web ⅝ × 42	15.75					2,315
B. Flg. 1 × 10	10.00	-21.50	-215	4,622		4,622

$$33.25 \text{ in.}^2$$

$$- 55 \text{ in.}^3$$

$$10,362$$

$$d_s = \frac{-55}{33.25} = -1.65 \text{ in.}$$

$$-1.65(55) = - \frac{91}{I_{NA} = 10,271}$$

$$d_{\text{Top of steel}} = 21.75 + 1.65 = 23.40 \text{ in.}$$

$$d_{\text{Bot. of steel}} = 22.00 - 1.65 = 20.35 \text{ in.}$$

$$S_{\text{Top of steel}} = \frac{10,271}{23.40} = 439 \text{ in.}^3$$

$$S_{\text{Bot. of steel}} = \frac{10,271}{20.35} = 505 \text{ in.}^3$$

Steel Section with Slab Reinforcing Steel on Positive-Moment Side of Splice

Material	A	d	Ad	Ad ²	I _o	I
Steel Section	33.25		- 55			10,362
Reinf. 14- # 6	6.16	26.55	164	4,342		4,342

$$39.41 \text{ in.}^2$$

$$109 \text{ in.}^3$$

$$14,704$$

$$d_s = \frac{109}{39.41} = 2.77 \text{ in.}$$

$$-2.77(109) = - \frac{302}{I_{NA} = 14,402 \text{ in.}^4}$$

$$d_{\text{Top of steel}} = 21.75 - 2.77 = 18.98 \text{ in.}$$

$$d_{\text{Bot. of steel}} = 22.00 + 2.77 = 24.77 \text{ in.}$$

$$S_{\text{Top of steel}} = \frac{14,402}{18.98} = 759 \text{ in.}^3$$

$$S_{\text{Bot. of steel}} = \frac{14,402}{24.77} = 581 \text{ in.}^3$$

$$d_{\text{Reinf.}} = 22.25 - 2.77 + 1.00 + 3.30 = 23.78 \text{ in.}$$

$$S_{\text{Reinf.}} = \frac{14,402}{23.78} = 606 \text{ in.}^3$$

Composite Section, 3n = 24

Material	A	d	Ad	Ad ²	I _o	I
Steel Section	33.25		- 55		100	10,362
Conc. 84 × 7/24	24.50	26.75	655	17,531		17,631

$$57.75 \text{ in.}^2$$

$$600 \text{ in.}^3$$

$$27,993$$

$$d_{24} = \frac{600}{57.75} = 10.39 \text{ in.}$$

$$-10.39 \times 600 = - \frac{6,234}{I_{NA} = 21,759 \text{ in.}^4}$$

$$d_{\text{Top of steel}} = 21.75 - 10.39 = 11.36 \text{ in.}$$

$$d_{\text{Bot. of steel}} = 22.00 + 10.39 = 32.39 \text{ in.}$$

$$S_{\text{Top of steel}} = \frac{21,759}{11.36} = 1,915 \text{ in.}^3$$

$$S_{\text{Bot. of steel}} = \frac{21,759}{32.39} = 672 \text{ in.}^3$$

Composite Section, $n = 8$

Material	A	d	Ad	Ad^2	I_o	I
Steel Section	33.25		- 55			10,362
Conc. $84 \times \frac{7}{8}$	73.50	26.75	1,966	52,594	300	52,894

$$106.75 \text{ in.}^2$$

$$1,911 \text{ in.}^3$$

$$63,256$$

$$d_8 = \frac{1,911}{106.75} = 17.90 \text{ in.}$$

$$-17.90 \times 1,911 = -34,210$$

$$I_{NA} = \frac{29,046}{29,046} \text{ in.}^4$$

$$d_{\text{Top of steel}} = 21.75 - 17.90 = 3.85 \text{ in.}$$

$$d_{\text{Bot. of steel}} = 22.00 + 17.90 = 39.90 \text{ in.}$$

$$S_{\text{Top of steel}} = \frac{29,046}{3.85} = 7,544 \text{ in.}^3$$

$$S_{\text{Bot. of steel}} = \frac{29,046}{39.90} = 728 \text{ in.}^3$$

Maximum Moments at Field Splice

With Positive Live-Load Moment

$$DL_1: M = -95$$

$$DL_2: M = -20$$

$$L+I: M = +575$$

With Negative Live-Load Moment

$$DL_1: M = -95$$

$$DL_2: M = -20$$

$$L+I: M = -480$$

Because of the location of the section close to the dead-load inflection point, bending strength is not critical. But fatigue must be investigated at the butt welds in the bottom flanges and at the shear-connector welds to the top flange.

The maximum range of stress in the bottom flange at the field splice is determined to be

$$f_{sr} = \frac{480(12)}{581} + \frac{575(12)}{728} = 9.9 + 9.5 = 19.4 < 27.5 \text{ ksi}$$

Assuming that shear connectors are welded to the top flange of the girder near the splice, the stress range at that point cannot exceed 19 ksi. The actual stress range is

$$f_{sr} = \frac{480(12)}{759} + \frac{575(12)}{7,544} = 7.59 + 0.91 = 8.50 < 19 \text{ ksi}$$

BOLTED FIELD SPLICE

A bolted field splice is designed as an alternative to the welded splice. The bolted splice is a friction-type connection made with $\frac{7}{8}$ -in.-dia A325 bolts. Design calculations and details of this splice are given later, to permit continuation of the investigation of flange-plate transitions.

FLANGE TRANSITION 28 FT FROM INTERIOR SUPPORT

Next, the location must be determined at which the transition from the section with $\frac{3}{4} \times 10$ -in. top flange and 1 \times 10-in. bottom flange to the maximum-positive-moment section, with $\frac{9}{16} \times 10$ -in. top flange and 1 $\frac{1}{2} \times 10$ -in. bottom flange can be made. For this purpose, the transition is assumed and stresses are checked at 28 ft from the pier. Fatigue is the governing condition. Section properties are the same as those calculated for the transition at the field splice.

Maximum Moments 28 Ft from Interior Support

With Positive Live-Load Moment

$$DL_1: M = 95$$

$$DL_2: M = 20$$

$$L+I: M = 735$$

With Negative Live-Load Moment

$$DL_1: M = 95$$

$$DL_2: M = 20$$

$$L+I: M = -445$$

Steel Stresses 28 Ft from Interior Support Due To Maximum Design Loads

Top of Steel (Compression)	Bottom of Steel (Tension)
For DL_1 : $F_b = \frac{95 \times 12}{439} \times 1.30 = 3.4$	$F_b = \frac{95 \times 12}{505} \times 1.30 = 2.9$
For DL_2 : $F_b = \frac{20 \times 12}{1,915} \times 1.30 = 0.2$	$F_b = \frac{20 \times 12}{672} \times 1.30 = 0.5$
For $L+I$: $F_b = \frac{735 \times 12}{7,544} \times 1.30 \times \frac{5}{3} = 2.5$	$F_b = \frac{735 \times 12}{728} \times 1.30 \times \frac{5}{3} = 26.2$
$\overline{6.1} < 50 \text{ ksi}$	$\overline{29.6} < 50 \text{ ksi}$

Strength of the composite section is much greater than needed. Fatigue is investigated in the bottom flange butt weld and in shear connector welds to the top flange. There is no transverse stiffener at the transition but fatigue should be investigated at the toe of the stiffener-to-web weld 25 ft from the interior support.

The maximum range of stress in the bottom flange at the transition is determined to be

$$f_{sr} = \frac{445(12)}{581} + \frac{735(12)}{728} = 9.2 + 12.1 = 21.3 < 27.5 \text{ ksi}$$

The stress range in the top flange near stud shear connector welds is

$$f_{sr} = \frac{445(12)}{759} + \frac{735(12)}{7,544} = 7.0 + 1.2 = 8.2 < 19 \text{ ksi}$$

Earlier calculations for stiffener spacing place a stiffener 3.0 ft away from the flange transition or 25 ft from the interior support. Live-load moments at this position are determined to be

$$+(L+I) = 645 \text{ kip-ft}$$

$$-(L+I) = -462 \text{ kip-ft}$$

and the live load stress range is

$$f_{sr} = \frac{462(12)(20+2.77)}{14,402} + \frac{645(12)(20+17.90)}{29,046} = 8.8 + 10.1 = 18.9 < 19 \text{ ksi}$$

Bolted-Splice Alternative

With a bolted field splice, it may, in some cases, be more economical to extend the heavier maximum-positive-moment section all the way to the splice and eliminate the butt-welded transition. Filler plates would then be required at the splice.

FLANGE-TO-WEB WELDS

The flange-to-web welds must have sufficient strength to transfer the horizontal shear between the girder flange and web plus adequate resistance to fatigue. AASHTO Art. 1.7.135 limits the maximum strength of the welds to 0.45 of the specified minimum tensile strength of the welding-rod metal. (The ultimate strength of the weld metal in fillet welds need not match the strength of the base metal.) Fatigue limitations are the same as those for weld metal in working-stress design.

With $F_u = 70$ ksi for A588 steel, the design relationship for strength of a fillet weld is

$$1.30 \left[D + \frac{5}{3}(L+I) \right] \leq (0.45 \times 70 \times 0.707 = 22.3 \text{ ksi})$$

Here, D , L and I are the shear stresses due to dead, live and impact loads, respectively.

FLANGE-TO-WEB WELDS AT END SUPPORT

The flange-to-web welds are checked initially at the end bearing. Section properties needed for shear calculations are determined first. The moments of inertia of the steel

section alone and the composite section with $n=8$ are the same as those computed for the flange-plate transition 17 ft from the end support. Calculations indicate that the weld size is governed by material thickness rather than maximum-strength requirements.

Section Properties at End Support

Steel Section Only

$$I = 9,199 \text{ in.}^4$$

Composite Section, $n=8$

$$I = 29,002 \text{ in.}^4$$

$$\text{Top Flg.: } Q = \frac{9}{16} \times 10 \times 24.31 = 137 \text{ in.}^3$$

$$\text{Top Flg.: } Q = \frac{9}{16} \times 10 \times 3.44 = 19$$

$$\text{Bot. Flg.: } Q = 1 \times 10 \times 18.47 = 185 \text{ in.}^3$$

$$\text{Conc.: } Q = \frac{7}{8} \times 84 \times 8.91 = \frac{655}{674} \text{ in.}^3$$

$$\text{Bot. Flg.: } Q = 1 \times 10 \times 39.34 = 393 \text{ in.}^3$$

Maximum Shears at End Support

With Positive Live-Load Shear

$$DL_1: V = 33.7$$

$$DL_2: V = 6.2$$

$$L+I: V = 58.7$$

With Negative Live-Load Shear

$$DL_1: V = 33.7$$

$$DL_2: V = 6.2$$

$$L+I: V = -6.0$$

Shear Flow $S = VQ/I$ Due to Maximum Design Loads

Top Weld

Bottom Weld

$$\text{For } DL_1: S = \frac{33.7 \times 137}{9,199} \times 1.30 = 0.65 \quad S = \frac{33.7 \times 185}{9,199} \times 1.30 = 0.88$$

$$\text{For } DL_2: S = \frac{6.2 \times 674}{29,002} \times 1.30 = 0.19 \quad S = \frac{6.2 \times 393}{29,002} \times 1.30 = 0.11$$

$$\text{For } L+I: S = \frac{58.7 \times 674}{29,002} \times 1.30 \times \frac{5}{3} = 2.96 \quad S = \frac{58.7 \times 393}{29,002} \times 1.30 \times \frac{5}{3} = 1.72$$

3.80 kips per in. 2.71 kips per in.

Shear in the top weld governs. For two welds the shear flow in each weld is $3.80/2 = 1.90$ kips per in.

$$\text{Weld size required} = \frac{1.90}{22.3} = 0.085 \text{ in.}$$

This, however, is less than the minimum size of weld required by AASHTO Specifications for the thickness of flange plate. Therefore, use the following minimum-size welds:

For $\frac{9}{16}$ -in. top flange, $\frac{1}{4}$ -in. fillet welds.

For 1-in. bottom flange, $\frac{5}{16}$ -in. fillet welds.

Shear Flow Range in Top Weld Due to Service Loads

Maximum shear range occurs at the end-support of the girder and is equal to

$$s_r = \frac{(58.7 + 6.0) \times 674}{29,002} = 1.50 \text{ kips per in.}$$

Shear stress on the throat of fillet welds falls into AASHTO fatigue category F. For 500,000 cycles of truck loading, the associated allowable shear stress range is 12 ksi. The actual stress range in the top weld at the end-support section equals

$$f_{vr} = \frac{1.50}{2 \times 0.707 \times \frac{1}{4}} = 4.24 < 12 \text{ ksi}$$

The web-to-flange welds have adequate resistance to fatigue.

FLANGE-TO-WEB WELDS AT INTERIOR SUPPORT

Calculations similar to those for the welds at the end bearing are made to determine flange-to-web weld size at the pier. They also show that material thickness rather than strength governs. Additional computations indicate that this also is the case throughout the length of the girder.

Section properties are computed first at the interior support. The moments of inertia for the girder and the girder plus reinforcing steel were calculated previously for determination of bending strength.

Section Properties at Interior Support

Steel Section Only

$$I = 20,318 \text{ in.}^4$$

Steel plus Reinforcing

$$I = 24,655 \text{ in.}^4$$

$$\text{Top Flg.: } Q = 1.25(14)(23.09) = 404 \text{ in.}^3$$

$$\text{Top Flg.: } Q = 1.25(14)(20.22) = 354$$

$$\text{Reinf.: } Q = 6.16(25.14) = 155$$

$$\text{Bot. Flg.: } Q = 1.5(14)(20.29) = 426 \text{ in.}^3$$

$$509 \text{ in.}^3$$

$$\text{Bot. Flg.: } Q = 1.5(14)(23.16) = 486 \text{ in.}^3$$

Maximum Shears at Interior Support

	DL_1	DL_2	$L+I$
V, kips	56.2	10.3	62.4

Shear Flow $S = VQ/I$ Due to Maximum Design Loads

Top Weld

Bottom Weld

$$\text{For } DL_1: S = \frac{56.2 \times 404}{20,318} \times 1.3 = 1.45$$

$$S = \frac{56.2 \times 426}{20,318} \times 1.3 = 1.53$$

$$\text{For } DL_2: S = \frac{10.3 \times 509}{24,655} \times 1.3 = 0.28$$

$$S = \frac{10.3 \times 486}{24,655} \times 1.3 = 0.26$$

$$\text{For } L+I: S = \frac{62.4 \times 509}{24,655} \times 1.3 \times \frac{5}{3} = 2.79$$

$$S = \frac{62.4 \times 486}{24,655} \times 1.3 \times \frac{5}{3} = 2.67$$

$$4.52 \text{ kips per in.}$$

$$4.46 \text{ kips per in.}$$

Shear in the top weld governs. As before, the allowable stress on the fillet weld is 22.3 ksi. For two welds,

$$\text{weld size required} = \frac{4.52}{2(22.3)} = 0.101 \text{ in.}$$

Minimum size of fillet weld permitted for the $1\frac{1}{4}$ and $1\frac{1}{2}$ -in. thick flanges, however, is $\frac{5}{16}$ in. Next, this size of weld is investigated for fatigue.

The range of shear flow in the top weld due to Service Loads equals

$$s_r = \frac{62.4 \times 509}{24,655} = 1.29 \text{ kips per in.}$$

The corresponding stress range on a $\frac{5}{16}$ -in. web-to-flange weld is

$$f_{vr} = \frac{1.29}{2 \times 0.707 \times \frac{5}{16}} = 2.92 < 12 \text{ ksi}$$

The web-to-flange welds at the interior support have adequate resistance to fatigue.

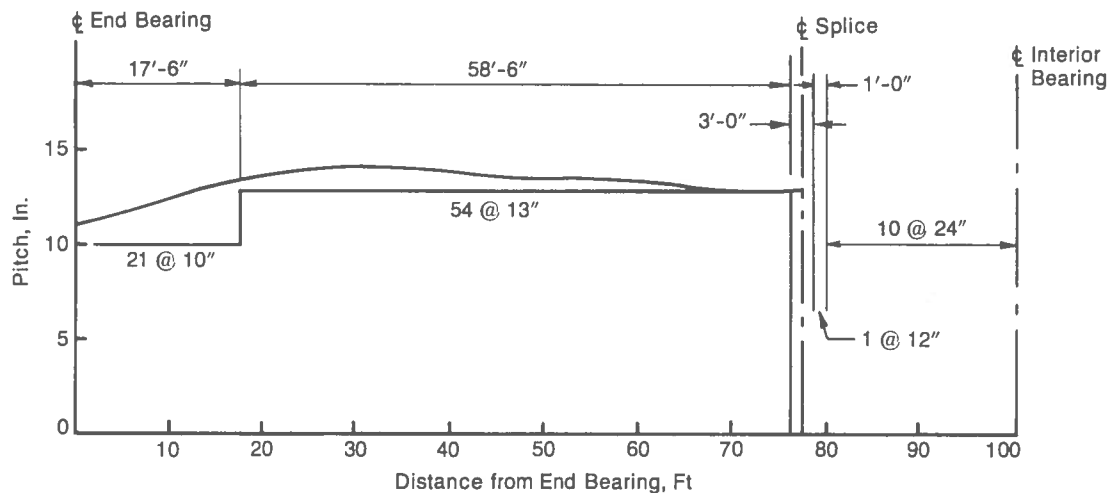
FLANGE-TO-WEB WELDS FOR VARIOUS FLANGE THICKNESSES

Minimum weld size for material thickness governs throughout the length of the girder.

Weld sizes are as follows: $\frac{1}{4}$ in. where the top flange is $\frac{5}{16}$ or $\frac{3}{4}$ in. thick and $\frac{5}{16}$ in. for the top and bottom flanges elsewhere.

SHEAR CONNECTORS

Welded studs, $\frac{7}{8}$ in. in diameter by 5 in. long, are selected for use as shear connectors. Because the calculations for this example are similar to the calculations made in Chapter 3A, only the shear-connector spacing diagram is shown here. Fatigue governs the spacing in the positive-moment region while the 24-in. maximum spacing controls in the negative-moment region.



SHEAR CONNECTOR SPACING

TRANSVERSE INTERMEDIATE STIFFENERS

Each transverse intermediate stiffener consists of a plate welded to one side of the web. As is the rest of the girder, the stiffener is made of A588 steel. A 4-in.-wide stiffener is tried.

Thickness required is determined from the requirement for maximum width-thickness ratio of stiffeners. Minimum thickness thus is

$$t = \frac{b' \sqrt{F_y}}{2,600} = \frac{4 \sqrt{50,000}}{2,600} = 0.344 \text{ in.}$$

Use a $\frac{3}{8}$ -in.-thick stiffener plate. Cross-sectional area of the plate is

$$A = 4 \times \frac{3}{8} = 1.50 \text{ in.}^2$$

and the moment of inertia is

$$I = \frac{(\frac{3}{8})(4)^3}{3} = 8.0 \text{ in.}^4$$

Area and moment-of-inertia requirements are checked for the stiffener near the interior support, where shear is largest and equals 222 kips for the Maximum Design Loads. Distance of the stiffener from the support is $d_o = 48$ in. For calculation of the minimum stiffener area required by AASHTO Specifications, $t_w = \frac{3}{8}$ in., $D = 42$ in., $Y = 1$ and $B = 2.4$. Also needed are C and V_u , the shear capacity of the web. For computation of V_u , V_p previously has been found to be 457 kips.

$$C = 18,000 \frac{t_w}{D} \sqrt{\frac{1 + (D/d_o)^2}{F_y}} - 0.3 = 18,000 \times \frac{\frac{3}{8}}{42} \sqrt{\frac{1 + (42/48)^2}{50,000}} - 0.3 = 0.655$$

The shear capacity of the web then is

$$V_u = V_p \left[C + \frac{0.87(1-C)}{\sqrt{1 + (d_o/D)^2}} \right] = 457 \left[0.655 + \frac{0.87(1-0.655)}{\sqrt{1 + (48/42)^2}} \right] = 390 \text{ kips}$$

Substitution in the formula for minimum cross-sectional area of stiffener required gives

$$A = Y \left[0.15BDt_w(1-C)\frac{V}{V_u} - 18t_w^2 \right]$$

$$= 0.15 \times 2.4 \times 42 \times \frac{3}{8} (1 - 0.655) \frac{222}{390} - 18 \left(\frac{3}{8} \right)^2 = -1.42 \text{ in.}^2$$

The negative result indicates that the area requirement can be ignored. The area formula is based on the assumption that a portion of the web assists the stiffener. The quantity $18t_w^2$ in the area formula represents the contribution of the web. When this quantity predominates; that is, when the formula yields a negative number, the web itself contributes more than the required area.

For computation of the required moment of inertia of the stiffener,

$$J = 2.5 \left(\frac{D}{d_o} \right)^2 - 2 = 2.5 \left(\frac{42}{48} \right)^2 - 2 = -0.086 < 0.5$$

The minimum value permitted for J is 0.5. The required moment of inertia then is

$$I = d_o t_w^3 J = 48 \left(\frac{3}{8} \right)^3 0.5 = 1.3 < 8.0 \text{ in.}^4$$

Use $\frac{3}{8} \times 4$ -in. stiffeners.

BEARING STIFFENERS AT END SUPPORT

The bearing stiffeners are designed as columns to carry the reaction forces at points of support. A stiffener consisting of two $4\frac{1}{2}$ -in.-wide plates welded to opposite sides of the girder web is investigated at the end support. Minimum thickness of stiffener required is

$$t = \frac{b'}{12} \sqrt{\frac{F_u}{33,000}} = \frac{4.5}{12} \sqrt{\frac{50,000}{33,000}} = 0.46 \text{ in.}$$

Use a $\frac{1}{2}$ -in.-thick stiffener plate.

End Reaction

	DL_1	DL_2	$L + I$	Total
R , kips	33.7	6.2	58.7	98.6

$$\text{Bearing stress} = \frac{98.6}{4.0 \times \frac{1}{2} \times 2} = 24.7 < 40 \text{ ksi (allowable)}$$

The stiffener column consists of the two $\frac{1}{2} \times 4\frac{1}{2}$ -in. plates and a length of web equal to

$$L_w = 18 \times \frac{3}{8} = 6.75 \text{ in.}$$

Area of the equivalent column is

$$A_s = 2 \times \frac{1}{2} \times 4\frac{1}{2} + \frac{3}{8} \times 6.75 = 7.03 \text{ in.}^2$$

Moment of inertia of the equivalent column is

$$I_s = \frac{(\frac{1}{2})(4.5 + 0.375 + 4.5)^3}{12} = 34.3 \text{ in.}^4$$

and the radius of gyration is

$$r = \sqrt{\frac{34.3}{7.03}} = 2.21 \text{ in.}$$

Consequently, the slenderness ratio of the stiffener equals

$$\frac{KL_c}{r} = \frac{D}{r} = \frac{42}{2.21} = 19.00$$

$$\sqrt{\frac{2\pi^2 E}{F_y}} = \sqrt{\frac{(2)(\pi^2)(29,000)}{50}} = 107.0 > 19.0$$

The allowable stress then is

$$F_{cr} = F_y \left[1 - \frac{F_y}{4\pi^2 E} \left(\frac{D}{r} \right)^2 \right] = 50 \left[1 - \frac{50}{4\pi^2 \times 29,000} (19.00)^2 \right] = 49.2 \text{ ksi}$$

The Maximum Design Load on the columns was previously computed for the investigation of the stiffened web at the end bearing to be 179 kips. The capacity of the equivalent column is

$$P_u = 0.85 A_s F_{cr} = 0.85 \times 7.03 \times 49.2 = 294 > 179 \text{ kips}$$

Therefore, the two $\frac{1}{2} \times 4\frac{1}{2}$ -in. plates are satisfactory as bearing stiffeners.

BEARING STIFFENERS AT INTERIOR SUPPORT

The bearing stiffeners at the interior support are designed in the same way as those at the end support. A stiffener consisting of two 6-in.-wide plates welded to opposite sides of the girder web is investigated at the interior support. Minimum thickness of stiffener required is

$$t = \frac{6}{12} \sqrt{\frac{50,000}{33,000}} = 0.615 \text{ in.}$$

Use $\frac{5}{8} \times 6$ -in. stiffeners.

Interior Reaction

	DL_1	DL_2	$L+I$	Total
R , kips	112.4	20.6	92.0	225.0

$$\text{Bearing stress} = \frac{225.0}{5.5 \times \frac{5}{8} \times 2} = 32.7 < 40 \text{ ksi (allowable)}$$

The reaction due to the Maximum Design Loads is

$$R = 1.30 \left(112.4 + 20.6 + \frac{5}{3} \times 92.0 \right) = 372 \text{ kips}$$

The stiffener column consists of the two $\frac{5}{8} \times 6$ -in. plates and a portion of the web 6.75 in. long, as at the end support. The area of the equivalent column is

$$A_s = 2 \times \frac{5}{8} \times 6 + \frac{3}{8} \times 6.75 = 10.0 \text{ in.}^2$$

Moment of inertia of the equivalent column is

$$I_s = \frac{(\frac{5}{8})(6+0.375+6)^3}{12} = 98.7 \text{ in.}^4$$

and the radius of gyration is

$$r = \sqrt{\frac{98.7}{10.0}} = 3.14 \text{ in.}$$

Therefore, the slenderness ratio is

$$\frac{D}{r} = \frac{42}{3.14} = 13.4$$

The allowable stress then is

$$F_{cr} = 50 \left[1 - \frac{50}{4\pi^2 \times 29,000} (13.4)^2 \right] = 49.6 \text{ ksi}$$

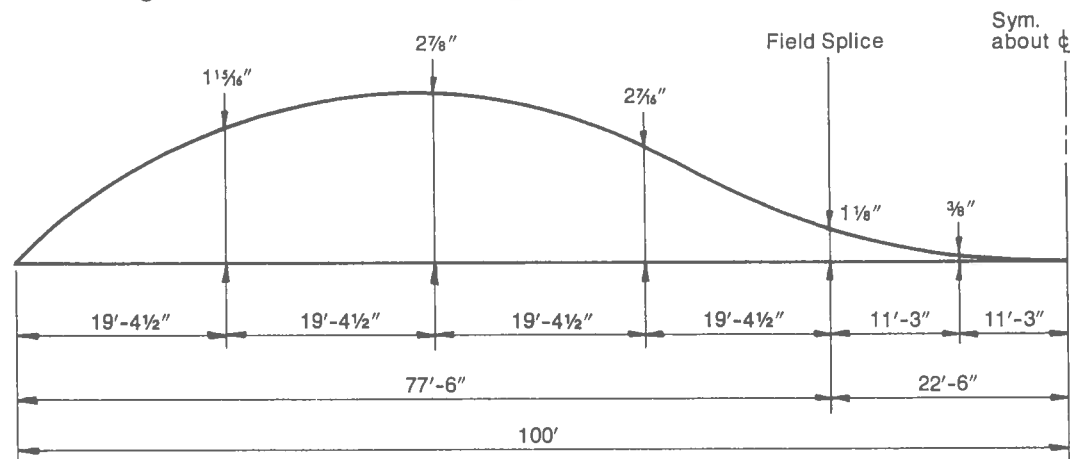
The capacity of the equivalent column consequently is

$$P_u = 0.85 \times 10.0 \times 49.6 = 422 > 372 \text{ kips}$$

Therefore, the two $\frac{5}{8} \times 6$ -in. plates are satisfactory as bearing stiffeners.

DEFLECTIONS

Dead-load and live-load deflections are determined in the same manner as for working-stress design. The dead-load camber diagram is shown below.



CAMBER DIAGRAM

The maximum live-load deflection is 0.995 in. and the deflection-span ratio is $1/1,204 < 1/800$.

DESIGN OF BOLTED FIELD SPLICE

The bolted alternate to the welded field splice, 22.5 ft from the pier, is to be a friction-type connection made with $\frac{7}{8}$ -in.-dia A325 bolts. The web on both sides of the splice is $\frac{3}{8} \times 42$ in. Flange plates on the positive-moment side of the splice are $\frac{3}{4} \times 10$ -in. top flange and 1×10 -in. bottom flange, whereas those on the negative-moment side are $\frac{3}{4} \times 14$ -in. top flange and 1×14 -in. bottom flange.

Shears 22.5 Ft from Pier, Kips

	For Service Loads	Factor	For Overload	Factor	Max. Design Loads
DL_1 :	36.1	1	36.1	1.30	46.9
DL_2 :	6.6	1	6.6	1.30	8.6
$L+I$:	52.0	$\frac{5}{3}$	86.7	1.30	112.7
	<u>94.7</u>		<u>129.4</u>		<u>168.2</u>

Moments 22.5 Ft from Pier, Kip-Ft

	For Service Loads	Factor	For Overload	Factor	Max. Design Loads
DL_1 :	— 95	1	— 95	1.30	— 124
DL_2 :	— 20	1	— 20	1.30	— 26
$-(L+I)$:	— 480	$\frac{5}{3}$	— 800	1.30	— 1,040
$+(L+I)$:	<u>575</u>				
Maximum:	— 595		<u>— 915</u>		<u>— 1,190</u>
Minimum:	460				

For load factor design of a bolted field splice, AASHTO Specifications require that the splice material be designed for Maximum Design Loads and resistance to fatigue under service loads. Friction connections must resist slip deformation under Overload, and therefore the fasteners must be proportioned for an allowable stress $F_v = 21$ ksi for an Overload of $D + (5/3)(L + I)$. The allowable bolt load in double shear is

$$P = 2 \times 0.6013 \times 21 = 25.3 \text{ kips per bolt}$$

For design of the splice material for Maximum Design Loads, the design moment is computed as the greater of:

Average of the calculated moment on the section and maximum capacity of the section

75 % of the maximum capacity of the section

The calculated moment is that induced by the Maximum Design Load $1.30[D + (5/3)(L + I)]$. Splice material should have a capacity equal at least to the design moment. The section capacity is based on the gross section minus any flange-area loss due to bolt holes in excess of 15 % of each flange area.

The section at the splice is subjected to negative moment which acts on the girder section only, negative moment which act on the girder section plus slab reinforcing steel and positive moment which acts on the girder plus concrete section. Because the effects of negative moment predominate at the splice, splice material will be designed for a negative-moment section. Also, because the DL_1 moment which acts on the girder section only is small in relation to the moment which acts on the girder section plus slab reinforcing steel, the calculation for net section at the splice will include the slab reinforcing steel.

The bolt holes remove from the $1\frac{1}{8} \times 10$ -in. flange

$$\% \text{ of flange} = \frac{2 \times 1}{10} \times 100 = 20 \%$$

Therefore, $20\% - 15\% = 5\%$ of the flange area must be deducted for determination of the net section. With this deduction, the net moment of inertia including the slab reinforcing steel is determined below.

Net Section Properties at Splice

Material	A	d	Ad	Ad ²	I _o	I
T. Flg. .95($\frac{3}{4} \times 10$)	7.13	21.37	152	3,256	2,315	3,256
Web $\frac{3}{8} \times 42$	15.75					2,315
B. Flg. .95(1×10)	9.50	-21.50	-204	4,391		4,391
Reinf. 14- #6	6.16	26.55	164	4,342		4,342

$$38.54 \text{ in.}^2$$

$$112 \text{ in.}^3$$

$$14,304$$

$$d_s = \frac{112}{38.54} = 2.91 \text{ in.}$$

$$-2.91(112) = -\frac{326}{13,978 \text{ in.}^4}$$

$$d_{\text{Top of steel}} = 21.75 - 2.91 = 18.84 \text{ in.}$$

$$d_{\text{Bot. of steel}} = 22.00 + 2.91 = 24.91 \text{ in.}$$

$$S_{\text{Top of steel}} = \frac{13,978}{18.84} = 742 \text{ in.}^3$$

$$S_{\text{Bot. of steel}} = \frac{13,978}{24.91} = 561 \text{ in.}^3$$

Base metal fatigue should be investigated at the gross girder section near friction type fasteners.

Design Moments and Shears

For $F_v = 50$ ksi, the net section capacity is

$$M_{\text{net}} = \frac{50 \times 561}{12} = 2,337 \text{ kip-ft}$$

$$75\% M_{\text{net}} = 0.75 \times 2,337 = 1,753 \text{ kip-ft}$$

The average of the calculated moment for the design loads and the net capacity of the section is

$$M_{av} = \frac{1,190 + 2,337}{2} = 1,764 > 1,753 \text{ kip-ft}$$

The design moment therefore is 1,764 kip-ft.

The design shear is determined by multiplying the calculated shear for the design loads by the ratio of the design moment to the calculated moment on the section.

$$\text{Design shear} = 168.2 \times \frac{1,764}{1,190} = 249 \text{ kips}$$

Web Splice

The web splice plates must carry the design shear, a moment M_v due to the eccentricity of this shear, and a portion M_w of the design moment on the section. The portion of the design moment to be resisted by the web is obtained by multiplying the design moment by the ratio of the moment of inertia of the web to the net moment of inertia of the entire section.

$$I_{web} = 2,315 + 15.75(2.91)^2 = 2,448 \text{ in.}^4$$

Web Moments for Design Loads

$$M_v = \frac{249 \times 3.5}{12} = 73$$

$$M_w = 1,764 \times \frac{2,448}{13,978} = 309$$

382 kip-ft

Try two $\frac{5}{16} \times 38$ -in. web splice plates and two rows of bolts with eight bolts per row on each side of the joint. The area of one hole is $0.312 \times 1 = 0.312$ sq in. The holes remove

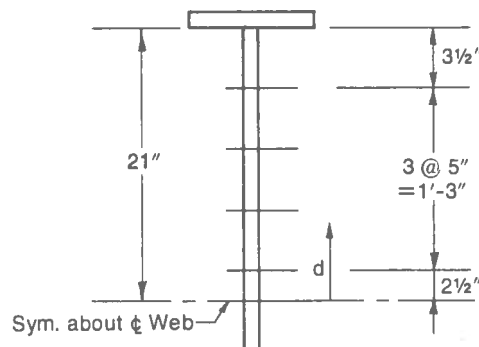
$$\% \text{ of plate} = \frac{8 \times 0.312}{0.312 \times 38} \times 100 = 21.05 \%$$

Consequently, the fraction of the hole area that must be deducted in determination of the net section is

$$\frac{21.05 - 15}{21.05} = 0.2874$$

d^2 for Holes

$$\begin{aligned} (2.5)^2 &= 6.25 \\ (7.5)^2 &= 56.25 \\ (12.5)^2 &= 156.25 \\ (17.5)^2 &= 306.25 \\ \Sigma d^2 &= 525.00 \text{ in.}^2 \end{aligned}$$



$$\Sigma A d^2 \text{ web holes} = 0.2874 \times 4 \times 0.312 \times 525 = 188 \text{ in.}^4$$

The assumption is made that the neutral axis of the splice material is in the same position as it is on the net section. The bending properties of the web splice plates are then computed as follows.

Material	A	d	Ad ²	I _o	I
2 Splice Pl. $\frac{5}{16}$ " \times 38"	23.75	2.91	201	2,853	3,054
Holes 2(8)(0.312)(0.287)	– 1.43	2.91	– 12	– 188	– 200

$$2,854 \text{ in.}^4$$

$$d_{\text{Top of splice}} = 19.0 - 2.91 = 16.09 \text{ in.}$$

$$d_{\text{Bot. of splice}} = 19.0 + 2.91 = 21.91 \text{ in.}$$

$$S_{\text{Top of splice}} = \frac{2,854}{16.09} = 177 \text{ in.}^3$$

$$S_{\text{Bot. of splice}} = \frac{2,854}{21.91} = 130 \text{ in.}^3$$

Hence, the maximum bending stress in the plates for design loads is

$$f_b = \frac{382 \times 12}{130} = 35.3 < 50 \text{ ksi}$$

The plates are satisfactory for bending. The allowable shear stress is

$$F_r = 0.55F_u = 0.55 \times 50 = 27.5 \text{ ksi}$$

The shear stress for the maximum design shear is

$$f_r = \frac{249}{23.75} = 10.5 < 27.5 \text{ ksi}$$

The $\frac{5}{16} \times 38$ -in. web splice plates are satisfactory for strength requirements. The plates are next checked for fatigue under service loads.

Web Bending Stress Range for Service Loads

The range of moment carried by the web equals

$$M_w = (575 + 480) \times \frac{2,448}{13,978} = 185 \text{ kip-ft}$$

The maximum bending stress range in the gross section of the web splice plates then is

$$f_b = \frac{185 \times 12 \times 21.91}{3,054} = 15.9 \text{ ksi}$$

Allowable Fatigue Stresses for Splice Material

Fatigue in base metal adjacent to friction-type fasteners is classified by AASHTO as Category B. For 500,000 cycles, the associated allowable stress range is 27.5 ksi. The plates are satisfactory for fatigue.

Use two $\frac{5}{16} \times 38$ -in. web splice plates

Web Bolts

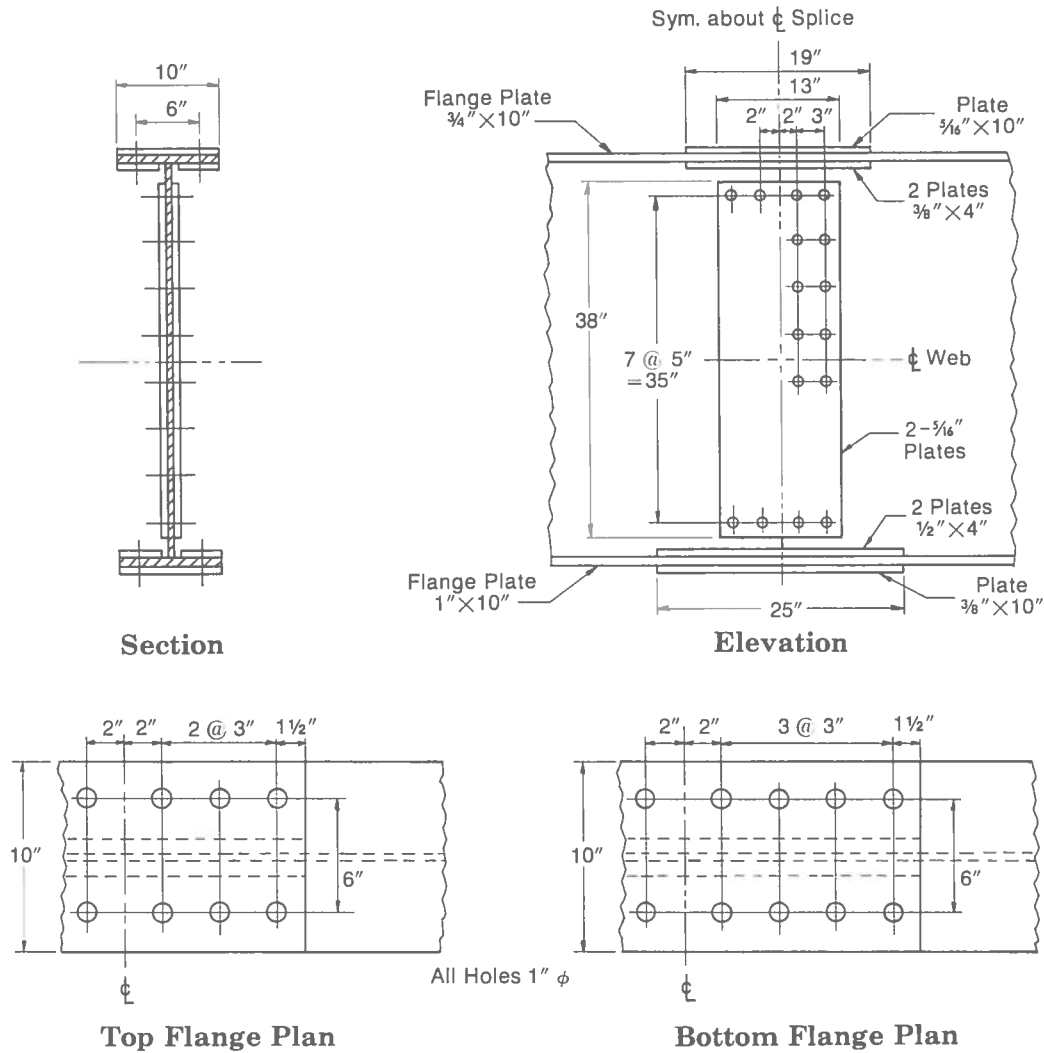
The 16 bolts in the web splice must carry the vertical shear, the moment due to the eccentricity of this shear about the centroid of the bolt group, and the portion of the beam moment taken by the web. These forces are induced by the Overload $D + (5/3)(L + I)$. The allowable load in double shear was previously computed to be $P = 25.3$ kips per bolt.

The polar moment of inertia of the bolt group about the assumed location of the neutral axis is

$$I = 2 \times 2 \times 525 + 16(1.5)^2 + 16(2.91)^2 = 2,271 \text{ in.}^4$$

The distance from the centroid to the outermost bolt is

$$d = \sqrt{(17.5 + 2.91)^2 + (1.5)^2} = 20.5 \text{ in.}$$



SPLICE DETAILS

Web Moments for Overload

$$M_r = \frac{129.4 \times 3.5}{12} = 38$$

$$M_w = 915 \times \frac{2,448}{13,978} = 160$$

198 kip-ft

Load per bolt due to shear is

$$P_s = \frac{129.4}{16} = 8.1 \text{ kips}$$

Load on the outermost bolt due to moment is

$$P_m = \frac{198 \times 12 \times 20.5}{2,271} = 21.4 \text{ kips}$$

The vertical component of this load is

$$P_v = \frac{21.4 \times 1.5}{20.5} = 1.6 \text{ kips}$$

The horizontal component is

$$P_h = \frac{21.4 \times (17.5 + 2.91)}{20.5} = 21.3 \text{ kips}$$

Therefore, the total load on the outermost bolt is the resultant

$$P = \sqrt{(8.1 + 1.6)^2 + (21.3)^2} = 23.4 < 25.3 \text{ kips}$$

Use 16 $\frac{1}{8}$ -in.-dia A325 bolts in two rows.

Flange-Splice Design

The flange splice plates are proportioned for Maximum Design Loads and checked for fatigue. The average stress in the top flange under the Maximum Design Load is

$$F_{b \text{ Top}} = \frac{1,764(12)(21.37 - 2.91)}{13,978} = 28.0 \text{ ksi}$$

The total flange force is determined by multiplying the average stress by the net flange area

$$P_{\text{Top}} = 28.0 \times 10 \times 0.75 \times 0.95 = 199 \text{ kips}$$

The required net area of top flange splice plate then becomes

$$A_{\text{Top}} = \frac{199}{50} = 3.98 \text{ in.}^2$$

Since this value is less than 75% of the net area of the top flange, the minimum required area will be

$$A_{\text{Top}} = 0.75(7.13) = 5.35 \text{ in.}^2$$

Try a $\frac{5}{16} \times 10$ -in. outer plate and two $\frac{3}{8} \times 4$ -in. inner plates. The net area after deduction of bolt holes in excess of 15% of the plate area is

$$\begin{aligned} A_f &= (\frac{5}{16} \times 10) + (2 \times \frac{3}{8} \times 4) - [(2 \times 1 \times \frac{5}{16}) + (2 \times 1 \times \frac{3}{8}) - 0.15 \{ (\frac{5}{16} \times 10) + (2 \times \frac{3}{8} \times 4) \}] \\ &= 5.67 \text{ in.}^2 > 5.35 \text{ in.}^2 \end{aligned}$$

The average stress in the bottom flange under the Maximum Design Load is

$$F_{b \text{ Bot.}} = \frac{1,764(12)(21.5 + 2.91)}{13,978} = 37.0 \text{ ksi}$$

The total flange force is

$$P_{\text{Bot.}} = 37.0 \times 10 \times 1 \times .95 = 351 \text{ kips}$$

The required net area of the bottom plate becomes

$$A_{\text{Bot.}} = \frac{351}{50} = 7.02 \text{ in.}^2$$

which is less than 75% of the net area of the bottom flange, or

$$A_{\text{Bot.}} = 0.75(9.50) = 7.13 \text{ in.}^2$$

Try a $\frac{3}{8} \times 10$ -in. outer plate and two $\frac{1}{2} \times 4$ -in. inner plates. The net area after deduction of bolt holes in excess of 15% of the plate area is

$$\begin{aligned} A_f &= (\frac{3}{8} \times 10) + (2 \times \frac{1}{2} \times 4) - [(2 \times 1 \times \frac{3}{8}) + (2 \times 1 \times \frac{1}{2}) - .15 \{ (\frac{3}{8} \times 10) + (2 \times \frac{1}{2} \times 4) \}] \\ &= 7.16 \text{ in.}^2 > 7.13 \text{ in.}^2 \end{aligned}$$

The flange splice plates are then checked for fatigue under Service Load.

The range of live-load moment at the splice equals

$$460 + 595 = 1,055 \text{ kip-ft}$$

And the range of average stress in the flanges is calculated to be

$$\text{Top flange: } f_{sr} = \frac{1,055(12)(21.37 - 2.91)}{13,978} = 16.7 \text{ ksi}$$

$$\text{Bot. flange: } f_{sr} = \frac{1,055(12)(21.50 + 2.91)}{13,978} = 22.1 \text{ ksi}$$

The corresponding range of stress in the gross section of the flange splice plates is

$$\text{Top flange: } f_{sr} = \frac{16.7(10)(0.75)(0.95)}{(\frac{5}{16} \times 10) + (2 \times \frac{3}{8} \times 4)} = 19.4 < 27.5 \text{ ksi}$$

$$\text{Bot. flange: } f_{sr} = \frac{22.1(10)(1)(0.95)}{(\frac{3}{8} \times 10) + (2 \times \frac{1}{2} \times 4)} = 27.1 < 27.5 \text{ ksi}$$

Flange Bolts

The number of bolts required in the flange splice is determined by the capacity needed for transmitting the flange force under the Overload $D + 5/3(L + I)$. The total moment on the section is 915 kip-ft.

The average stress in the top flange is

$$F_b = \frac{915(12)(21.37 - 2.91)}{13,978} = 14.5 \text{ ksi}$$

And the flange force becomes

$$P_{\text{Top}} = 14.5(10 \times 0.75 \times 0.95) = 103 \text{ kips}$$

For this flange force

$$\frac{103}{25.3} = 4.1 \text{ bolts are required}$$

Use 6 bolts in two rows.

The average stress in the bottom flange is

$$F_b = \frac{915(12)(21.50 + 2.91)}{13,978} = 19.2 \text{ ksi}$$

And the bottom flange force is

$$P_{\text{Bot.}} = 19.2(10 \times 1 \times 0.95) = 182 \text{ kips}$$

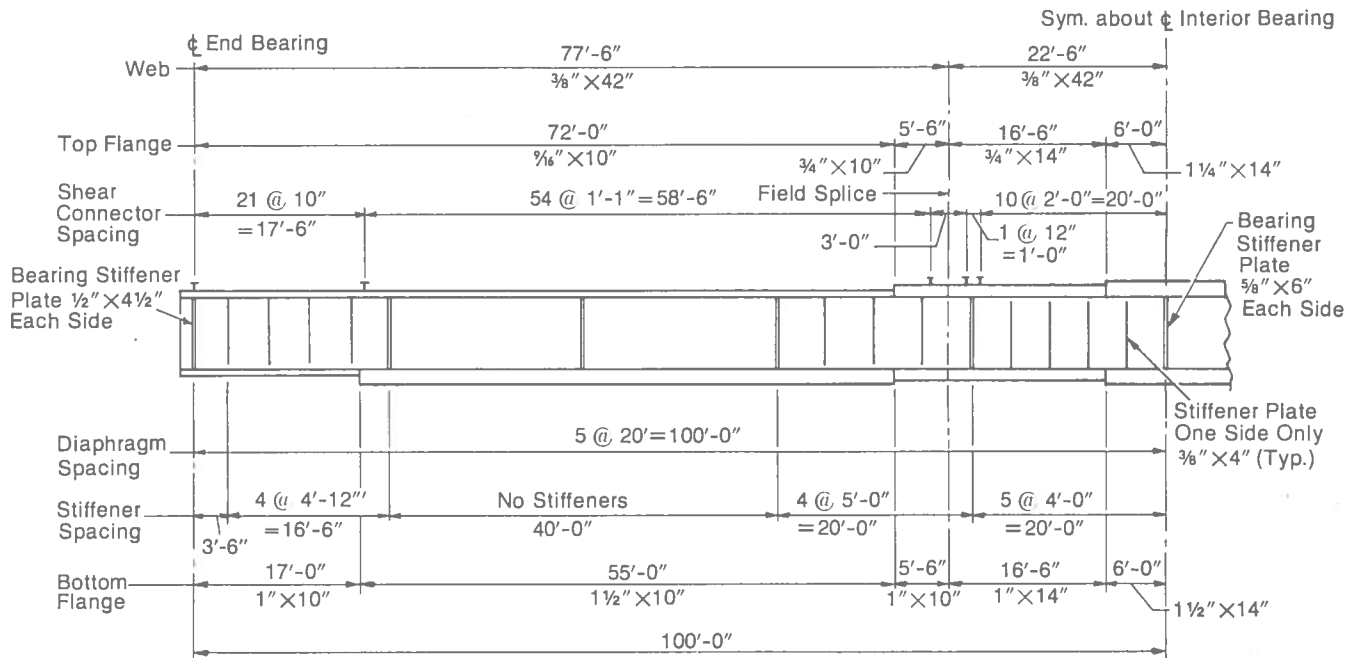
For this flange force

$$\frac{182}{25.3} = 7.2 \text{ bolts are required}$$

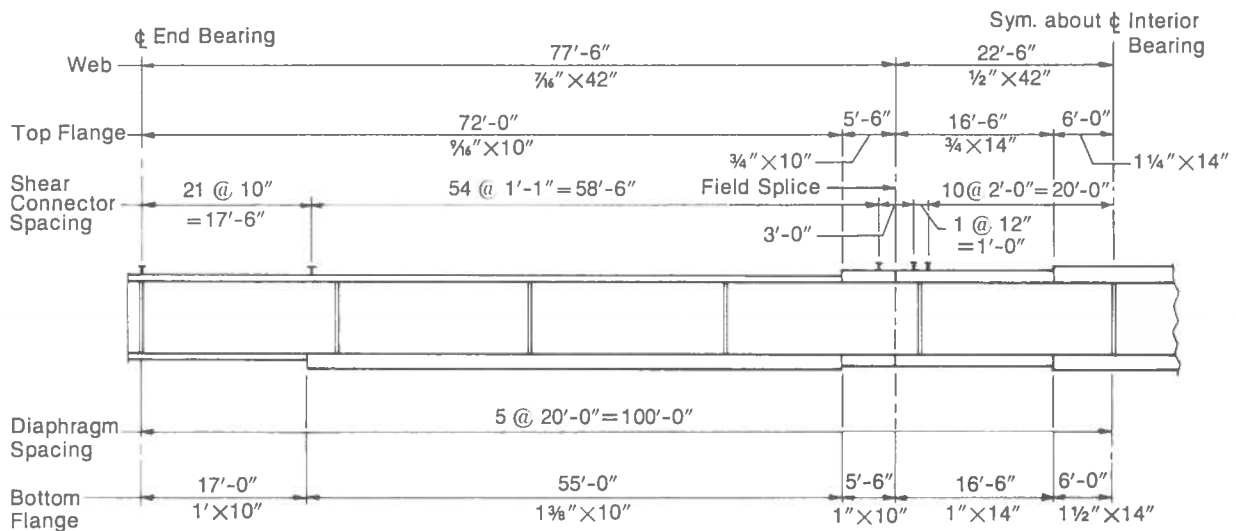
Use 8 bolts in two rows. Details of the splice are shown on page 33.

FINAL DESIGN

An elevation of the two-span, continuous girder with a stiffened web is shown below. An elevation of the girder with an unstiffened web follows. A detail drawing of the complete design is shown on the following page.



ELEVATION OF GIRDER WITH STIFFENED WEB



ELEVATION OF GIRDER WITH UNSTIFFENED WEB