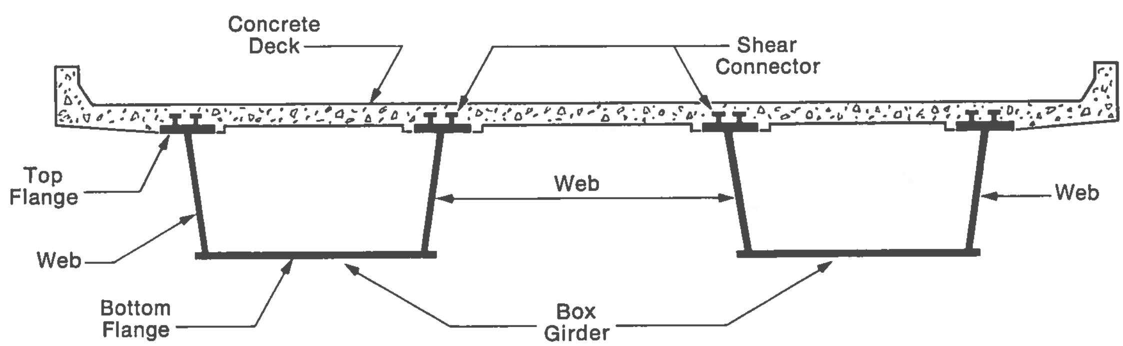
II/7

Composite: Box Girder Load Factor Design

Introduction

Box girders are an alternative form of welded steel plate girders. Each of the plate girders discussed in Chapters 4 and 4A has a single vertical web and is I-shaped in cross section. A box girder, however, has two or more webs, which may be vertical or inclined, and its vertical cross section is a hollow rectangle or trapezoid.

The bottom flange of a box girder usually is a continuous, horizontal plate extending between and connected to the bottom of the webs. The top flange may be a similar steel plate between the tops of the webs or a combination of a narrow steel plate on each web and a composite reinforced concrete deck.



CROSS SECTION OF TYPICAL BOX-GIRDER BRIDGE

The box-shaped cross section of the girders has many advantages for bridge construction. As a result, box-girder highway bridges are economical for simple spans of 75 ft or more and continuous spans of 100 ft or more. Such bridges are often used as grade-separation and elevated structures in urban areas where aesthetics is important. They have also proven advantageous in rural applications, such as stream crossings.

A pleasing appearance is often one of the most important reasons for selection of box-girder construction. They look good because they have a smooth, uninterrupted profile and because they can be given attractive shapes compatible with high structural efficiency.

Maintenance is easier and less costly for box-girder bridges than for plate-girder structures. If the box girders are sealed, their interior need not be painted. When exterior painting is desired, box girders present a smaller exposed surface than do plate girders. And the uninterrupted, continuous exterior surface of a box girder makes this area easier to paint and also less subject to corrosion.

From a structural viewpoint, box girders offer the advantage of a more efficient cross section for resisting torsion than that of plate girders. The high torsional resistance makes box sections particularly advantageous for curved bridges.

In addition, box girders generally can compete favorably in construction cost with plate girders. While the fabrication cost per pound of steel for a box girder may be larger than that for a plate girder, box girders generally require less steel. So the total cost of fabrication of box girders may be about the same as or less than that of plate girders. Erection costs of box girders also may be less. For example, box girders may be advantageous where erection must be performed under difficult conditions or on a limited time schedule. Because one box girder is equivalent to two or more plate girders, placement of a box girder in a single erection lift accomplishes the equivalent of lifting and connecting two plate girders. Also, when laterally braced, a box girder is much stabler during handling and erection than a plate girder of the same length.

This chapter illustrates the design of a two-span, composite, box-girder highway bridge with geometry and general arrangement conforming to Interstate System requirements. The example bridge is typical of a structure that carries one roadway of an Interstate Highway over both roadways of another Interstate Highway. Both spans of the bridge are 120 ft. ASTM A36, A588 and A572, Grade 50, steels are used for the steel portions of the superstructure.

At mid-length of the bridge, the box girders are supported on and rigidly connected to a single, center steel pier. Consequently, the girders and pier act as a rigid frame. (The girders, if desired, could be simply supported at the center pier and analyzed as ordinary continuous beams.)

The chapter also presents an alternate design with a reinforced concrete pier at mid-length of the bridge. This pier also is rigidly connected to the girders.

Criteria for load factor design used in the example are in accordance with the American Association of State Highway and Transportation Officials "Standard Specifications for Highway Bridges," 1973, and 1974, 1975, and 1976 "Interim Specifications." These specifications are referred to for brevity in this chapter as AASHTO followed by an article and section reference.

General Design Considerations

As shown in the cross section of a typical box-girder bridge, a composite box girder usually consists of:

1. Two steel web plates.
2. A steel bottom-flange plate joining the two webs and forming the underside of the box.
3. Two steel top-flange plates.
4. The reinforced concrete bridge deck made composite with the steel by embedment in the deck concrete of shear connectors welded to the steel top flanges.

Because the bottom flange of a box girder is usually wide, compressive stresses in it could cause it to buckle. To prevent this, the bottom flange, where compression may occur, is stiffened by longitudinal stiffeners welded at equal intervals across the width of the flange.

The steel top flanges for composite box girders need be no wider than necessary to provide adequate bearing for the concrete deck that they support and to allow sufficient space for welding of shear connectors to the flanges.

Webs of box girders are similar to webs of plate girders. Like such webs, box-girder webs may be stiffened transversely or both transversely and longitudinally. They are, however, often sloped rather than vertical. This is done not only to improve the appearance of the bridge but also to reduce the width of the bottom flange.

Box girders also differ from plate girders in that cross bracing between webs is concealed within box girders. Internal diaphragms or cross frames are required within a box girder at each support to resist transverse rotation, displacement and excessive distortion of the girder cross section. Diaphragms or cross frames are also occasionally positioned at other locations to stabilize box girders during handling and erection.

In addition, in continuous spans with field splices, cross frames are usually installed on each side of the splice. When box girders are curved, internal cross frames should be provided at regular intervals along the span, with lateral bracing between the cross frames at the top-flange level. For a tangent alignment, diaphragms or cross frames along the span are unnecessary.

In design of a composite box girder with a vertical axis of symmetry, each half of the cross section may be considered equivalent to a plate-girder section. Principles of composite design presented in detail in Chapters 3 and 4 for wide-flange beams and plate girders therefore may be applied to the box-girder sections.

Box girders that are unsymmetrical with respect to the vertical centroidal axis are a special case that requires a more vigorous analysis. Such girders are not treated in this chapter.

CONCRETE DECK

In a composite box-girder bridge, the roadway slab spans from web to web of each box girder and between the webs of adjacent box girders. Often, the slab cantilevers beyond the outer webs of exterior box girders. The slab may be designed in the same manner as for a series of plate girders with composite construction throughout the full length of the girders.

LATERAL DISTRIBUTION OF DEAD LOAD

When the bridge deck is supported by a single box girder, the dead load on the girder equals its own weight plus the weight of the deck, which consists of the weights of slab, haunches, parapets, wearing surface and railings. When two box girders support the deck, each girder carries its own weight and the weight of half the bridge deck.

In unshored composite design, the dead load on a box girder is divided into two parts, the initial loads, or loads applied before the deck concrete has hardened, and superimposed loads, those applied after the concrete has hardened. The initial dead load is made up of the weights of the girder and slab and is assumed to be carried by the steel portions of the girder alone. The superimposed dead load is made up of the weights of parapets, wearing surface and railings and is assumed to be carried by the steel portions of the girder acting compositely with the concrete slab.

When three or more box girders support the deck, each girder carries as initial dead load its own weight plus the weight of the part of the slab immediately above it. In addition, each girder is assumed to support the portion of the slab extending halfway to the nearest web of an adjacent girder and, for exterior girders, slab cantilevers. The superimposed dead load may be distributed equally to all the girders.

For shear design, the total dead load acting on a girder may be distributed equally to each web of the box.

LATERAL DISTRIBUTION OF LIVE LOAD

For computation of live-load bending moments, a box girder may be assumed to carry the fraction of a wheel load W_L computed from

$$W_L = 0.1 + 1.7R + \frac{0.85}{N_w}$$

where $N_w = W_c/12$, *reduced* to the nearest whole number

W_c = roadway width, ft, between curbs

$R = N_w$ divided by the number of box girders, but not less than 0.5 nor more than 1.5

The reduction in load intensity for multiple lane loading required by Art. 1.2.9 of the AASHTO Specifications is not applicable to box girders, because it has been taken into account in the development of the preceding equation. The reduction should be applied, however, for design of other bridge components, such as the substructure.

The wheel-load distribution determined by the equation is applicable to bridges for which the distance center to center of adjacent top flanges is within the approximate range of 80 to 120% of the width of the box girders and for which the deck cantilever does not exceed either 6 ft or 60% of the distance center to center of adjacent top flanges, and to continuous bridges for which the box girders are composite throughout their entire length.

One-half the distribution factor for moment should be used, in general, in calculation of the live-load vertical shear in each box-girder web. In calculation of shears at points of support of the girders, however, the wheel load immediately adjacent to the support should be distributed as if the deck acted as a simple beam between the webs.

The distribution factor for live-load deflection may be obtained by dividing the number of lanes by the number of girders.

STRUCTURAL ANALYSIS

The longitudinal variations of moments, shears and deflections are calculated from an analysis of the structure as a rigid frame, simply supported at the end bearings of the girders and fixed at the base of the center pier. In the analysis, the variation in moments of inertia of the cross sections of girders and pier along their lengths should be taken into account in determination of the stiffness of frame members. The analysis should also provide for the change in stiffness after the deck concrete hardens. For the initial dead load, the stiffness is that of the steel section alone. For the superimposed dead load and the live load, the stiffness is that of the composite section, based, respectively, on the modular ratios $3n$ and n, where n is the ratio of the modulus of elasticity of the steel E_s to the modulus of elasticity of the concrete E_c. For the stiffness calculations, the concrete slab may be considered effective over the full length of the structure.

As for all statically indeterminate structures, the stiffness of the members of the box-girder bridge is not known accurately until the structure has been designed and therefore has to be assumed initially. As a result, after member sizes have been determined from stresses calculated from the initial analysis, the stiffness of the frame members should be calculated and the structure analyzed and designed again with the new values of stiffness. The procedure should be repeated until the stiffness on which an analysis is based agrees reasonably with the stiffness of the designed members.

The example bridge was analyzed by computer. Final member sizes were selected after three cycles of analysis and design. The results of the final cycle are presented in this chapter.

DESIGN LOADS

Members designed by the Load Factor method are required to meet certain criteria for three theoretical load levels: Maximum Design Load, Overload and Service Load.

Service Loads are the design loads used in working-stress design. They are applied in design calculations to keep live-load deflections and fatigue life (for assumed fatigue loading) of structural members within acceptable limits.

The Maximum Design Load and the Overload are computed from the service loads by multiplying by a factor of unity or larger the dead, live and impact service loads. Maximum Design Load is applied in design calculations to insure that the structure can withstand in emergencies (simultaneously in more than one lane) a few passages of very heavy vehicles that may induce significant permanent deformations. An Overload is applied to limit permanent deformations that may be caused by occasional overweight vehicles and that would impair riding quality of the deck. The weight of these vehicles is taken as 5/3 the live and impact service loads (simultaneously in more than one lane).

In determination of moments, shears and other forces, the structure is assumed to act elastically under the three loading levels. The loads for these levels are defined as follows:

Service Load: $D+(L+I)$

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

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

where D = dead load
L = live load
I = impact load

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

DESIGN FOR MAXIMUM DESIGN LOADS

Box girders usually do not have as much bending strength as a compact section, because the webs do not normally meet compactness criteria. The maximum-moment capacity at any section therefore should be computed for positive bending from

$$M_u = F_y S$$

where F_y = specified minimum yield stress, psi, of the steel
S = elastic section modulus

The capacity usually need not be reduced to allow for overall buckling. Both flanges of a box girder may be considered braced against lateral torsional buckling. The top flange is braced by the concrete deck, and the bottom flange normally is too wide to buckle in its plane.

For positive bending, with the bottom flange in tension, the effective width of that flange for calculation of the section modulus may not be taken as more than one-fifth the girder span. This limitation accounts for the phenomenon of shear lag in the box section.

Hence, for positive bending, a box girder should be so proportioned 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.

In negative bending, the moment capacity at any section is governed by the critical local buckling stress F_{cr} of the bottom flange, which is in compression. The section, therefore, should be proportioned so that

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

The critical bottom-flange buckling stress F_{cr} is a function of the width-thickness ratio w/t for the flange plate and a buckling coefficient k:

When $w/t \leq 3{,}070\sqrt{k}/\sqrt{F_y}$,

$$F_{cr} = F_y$$

When $3{,}070\sqrt{k}/\sqrt{F_y} < w/t \leq 6{,}650\sqrt{k}/\sqrt{F_y}$,

$$F_{cr} = 0.592 F_y (1 + 0.687 \sin c\pi/2)$$

where $c = \left(6{,}650\sqrt{k} - \frac{w}{t}\sqrt{F_y}\right)/3{,}580\sqrt{k}$

When $w/t > 6{,}650\sqrt{k}/\sqrt{F_y}$,

$$F_{cr} = 26.2 \times 10^6 k\left(\frac{t}{w}\right)^2$$

In the preceding equations, w is the spacing of the longitudinal stiffeners on the flange, and t is the plate thickness.

When there are no longitudinal stiffeners on the bottom flange b, the spacing of the girder webs, is substituted for w and k should be taken as 4. For a bottom flange with n longitudinal stiffeners with equal spacing w, k may be computed from the following:

When $n = 1$,

$$k = \sqrt[3]{8I_s/wt^3}$$

where I_s = moment of inertia, in.^{4}, of a longitudinal stiffener about an axis parallel to the bottom flange and at the base of the stiffener

When $n = 2$, 3, 4 or 5,

$$k = \sqrt[3]{14.3I_s/wt^3n^4}$$

The value of k, however, should not exceed 4.

CHANGES IN FLANGE-PLATE THICKNESS

The same principles that govern design for changes in flange-plate thickness of plate girders also apply to box girders. Because the bottom flange of a box girder is very wide and the steel stop flanges usually are narrow, changes in thickness of the bottom-flange plate will be smaller than for the top-flange plates.

FLANGE WIDTH-THICKNESS RATIOS

To prevent local buckling of compression flanges, AASHTO Specifications require that the width-thickness ratio of projecting compression flanges not exceed

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

where b' = width of projecting flange element

t = flange thickness

When the bending moment M on a section is less than the moment capacity M_u of the section, b'/t may be increased in the ratio $\sqrt{M_u/M}$.

The b'/t requirement need not be satisfied for compression flanges of composite girders in positive-bending regions after the deck concrete has hardened. Before the deck concrete has hardened, however, the steel top flange is subject to local buckling under the initial dead load. For this condition, the b'/t limit of $2{,}200/\sqrt{F_y}$ may be increased in the ratio $\sqrt{F_y/f_{bD}}$. Here, f_{bD} is the actual stress in the flange due to the initial, factored dead-load moment and F_y is the yield stress or the top-flange stress used in calculation of the moment capacity of the section.

WEBS

For a box girder to qualify as a braced noncompact section with a moment capacity under maximum design load of $M_u = F_yS$ or $M_u = F_{cr}S$, each web must satisfy the following requirements:

1. The web depth-thickness ratio should not exceed

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

where D = web depth measured in the plane of the web (along the slope of inclined webs)

t_w = thickness of web measured normal to the plane of the web

2. At any section, shear due to maximum design load should not exceed

$$V_p = 0.58 F_y D t_w$$

When the web is sloped, V_p is the shear along the slope. The permissible vertical shear then is the vertical component of V_p.

3. If there are no transverse stiffeners on the web, the design shear in the plane of the web also must not exceed the buckling capacity of an unstiffened web:

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

where E = steel modulus of elasticity

When the shear exceeds V_b, the web should be stiffened transversely. In that case, the web depth-thickness ratio should not be greater than

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

or, when D_c, the clear distance between the neutral axis and the compression flange, exceeds $D/2$,

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

The shear capacity V_u of a transversely stiffened web fulfilling the preceding requirements may be computed from

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

where d_o = spacing of transverse stiffeners and

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

If a section is subjected to simultaneous shear V and bending moment M and V exceeds $0.6V_u$, then the concurrent bending moment for the maximum design load is limited to

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

where M_u = moment capacity of the section when not subject to design shears exceeding $0.6V_u$

(Many designers conservatively use the maximum shear on the section in computation of M, although that shear may not occur for the same loading condition as for maximum moment on the section. This procedure is used in the example in this chapter.)

When the web depth-thickness ratio is larger than that permitted for a web with transverse stiffeners, then a longitudinal stiffener is required in addition to transverse stiffeners. The longitudinal stiffener should be placed at an averaged clear distance equal to approximately $2D_c/5$ from the compression flange. This distance should be adjusted to accommodate welding. Under the preceding conditions, the web depth-thickness ratio may be as large as

$$\frac{D}{t_w} = \frac{73{,}000}{\sqrt{F_y}}$$

or, when D_c exceeds $D/2$,

$$\frac{D_c}{t_w} \leq \frac{36{,}500}{\sqrt{F_y}}$$

The shear capacity of a web with a longitudinal stiffener may be calculated in the same way as for a web with transverse stiffeners only. Similarly, if the shear on the section exceeds $0.6V_u$, the maximum simultaneous moment is limited by the same equation as for a web with transverse stiffeners only.

WEB STIFFENERS

Bearing and intermediate transverse stiffeners are designed by the procedures given in Chapter 4A.

When longitudinal stiffeners are used in combination with transverse stiffeners, the criteria given in Chapter 4A for stiffeners still apply, except that the depth of the subpanel, $0.8D$, rather than D, should be used in all equations. In addition, the section modulus of each transverse stiffener should be at least

$$S_t=\frac{1}{3}(D/d_o)S_l$$

where D = clear unsupported depth between flange components measured in the plane of the web

d_o = spacing of transverse stiffeners

S_l = section modulus of the longitudinal stiffener

Longitudinal stiffeners must satisfy the same requirement for width-thickness ratio as transverse stiffeners. For rigidity, the moment of inertia of each longitudinal stiffener should be at least

$$I=Dt_w^3[2.4(d_o/D)^2-0.13]$$

Also, the radius of gyration should not be less than

$$r=\frac{d_o\sqrt{F_y}}{23{,}000}$$

I and r should be computed for an axis through the mid-plane of the web, and the section should include both the longitudinal stiffener and a strip of web $18t_w$ wide centrally located with respect to the stiffener.

HYBRID SECTIONS

In a hybrid girder, the steel in one or both flanges has a higher yield strength than the web plate. The bending strength of a girder section is based on the properties of the flange steel, which are modified by a reduction factor, R.

In positive-moment regions of a composite box girder, the area of the steel compression flange should be equal to or smaller than the area of the tension flange. In negative-moment regions, the area of the compression flange should be equal to the area of the steel tension flange or larger by an amount not exceeding 25%. Also, the minimum specified yield strength of the web should not be less than 35% of the minimum specified yield strength of the tension flange.

The moment capacity M_u at any section of a noncompact, hybrid box girder is given by

$$M_u=F_{yf}SR$$

where F_{yf} = the minimum specified yield strength of a flange steel

S = elastic section modulus

$$R=1-\frac{\beta\psi(1-\rho)^2(3-\psi+\rho\psi)}{6+\beta\psi(3-\psi)}$$

$$\rho=\frac{\text{yield strength of web}}{\text{yield strength of tension flange}}$$

$$\beta = \frac{\text{area of web}}{\text{area of tension flange}}$$

ψ = distance from the outer fiber of the tension flange to the neutral axis of the composite section divided by the depth of the steel section

The expression for M_u shall be applied to both flanges.

Tension-field action is not taken into account in design of hybrid sections with stiffened webs. (See Commentary, "Tentative Criteria for Load Factor Design of Steel Highway Bridges," American Iron and Steel Institute Bulletin No. 15, March, 1969.) The shear capacity of a stiffened web is therefore

$$V_u = V_p C$$

where V_p and C are defined as for sections with flanges and webs made of the same steel. Also, the area requirement given in Chapter 4A for transverse stiffeners is not applicable to hybrid girders.

DESIGN FOR OVERLOAD

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

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

For the same purpose, composite sections of a homogeneous girder in positive bending must satisfy the relationship

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

Objectional deformations in a hybrid girder, under the Overload, will occur at a lower moment level than in a homogeneous girder because of premature yielding in the web. To account for this, the above moment relationships must be modified by the reduction factor, R.

For noncomposite sections and negative bending of composite sections of a hybrid girder,

$$0.8F_{yf} SR \geq \left[D + \frac{5}{3}(L+I)\right]$$

For composite sections in positive bending of a hybrid girder,

$$0.95F_{yf} SR \geq \left[D + \frac{5}{3}(L+I)\right]$$

For an unsymmetrical section, stresses in both flanges shall be checked.

DESIGN FOR SERVICE LOADS

Fatigue should be investigated in the same manner as for working-stress design, with Service loads, to satisfy the provisions of AASHTO Art. 1.7.3. The strength of longitudinal reinforcing steel of the concrete deck, in tension in negative-moment regions, should be taken into account in computation of section properties for sections in those regions. For fatigue computations, the stress range in the reinforcing steel is limited to 20,000 psi.

Fatigue becomes critical under tension or stress reversal at the following locations in box girders with groove-welded flange transitions, stud shear connectors, fillet-welded transverse web stiffeners and fillet-welded bottom-flange longitudinal stiffeners:

1. Base metal adjacent to a fillet weld at the end of a longitudinal flange or web stiffener (AASHTO Category E).

2. Base metal adjacent to stud shear connectors (AASHTO Category C).

3. Base metal in the girder web at the toe of a transverse-stiffener fillet weld or at the toe of a fillet weld for a cross-frame connection plate (AASHTO Category C).

4. Base metal adjacent to full-penetration groove-welded flange transitions (AASHTO Category B).

Groove-welded splices at transitions in width or thickness of flanges may be assigned to AASHTO fatigue Category B if transition slopes not exceeding 1 to 2½ are used and the welds are finished smooth and flush.

SHEAR CONNECTORS

Shear connectors should be designed in the same way as for working-stress design.

DEFLECTIONS

Dead-load and live-load deflections should be calculated in the same way as for working-stress design.

DESIGN OF PIER

The girders and pier of the following design example are designed for vertical dead, live and impact loads, wind and longitudinal force from braking and traction. AASHTO Specifications call for a transverse wind load of 50 psf and a simultaneous longitudinal wind load of 12 psf acting on the surface of the bridge as seen in elevation. Also, simultaneous transverse and longitudinal wind forces of 100 lb per lin ft and 40 lb per lin ft, respectively, are specified for wind on live load. In addition, a longitudinal braking and traction force equal to 5% of the live load should be considered applied to the bridge 6 ft above the deck.

For transverse loads, the structure is analyzed as a grid; that is, as a structure loaded normal to its plane. For vertical and longitudinal loads, the structure is analyzed as a rigid frame.

In design of the pier, loads are combined in accordance with the following groupings:

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

$$\text{Group II: } 1.30(D+W)$$

$$\text{Group III: } 1.30(D+L+I+0.3W+WL+LF)$$

where W = wind on the structure

WL = wind on the live load

LF = longitudinal force

The pier is initially designed with a steel, rectangular, hollow-box cross section. Unit stresses in the section are calculated from

$$f=\frac{P}{A}+\frac{M_x y}{I_x}+\frac{M_y x}{I_y}$$

where P = vertical load on the pier

A = cross-sectional area of the pier

M_x = bending moment about principal axis XX of the section

M_y = bending moment about principal axis YY of the section

x = distance from point where stress is to be computed to the YY axis

y = distance from the point to the XX axis

I_x = moment of inertia of the section about the XX axis

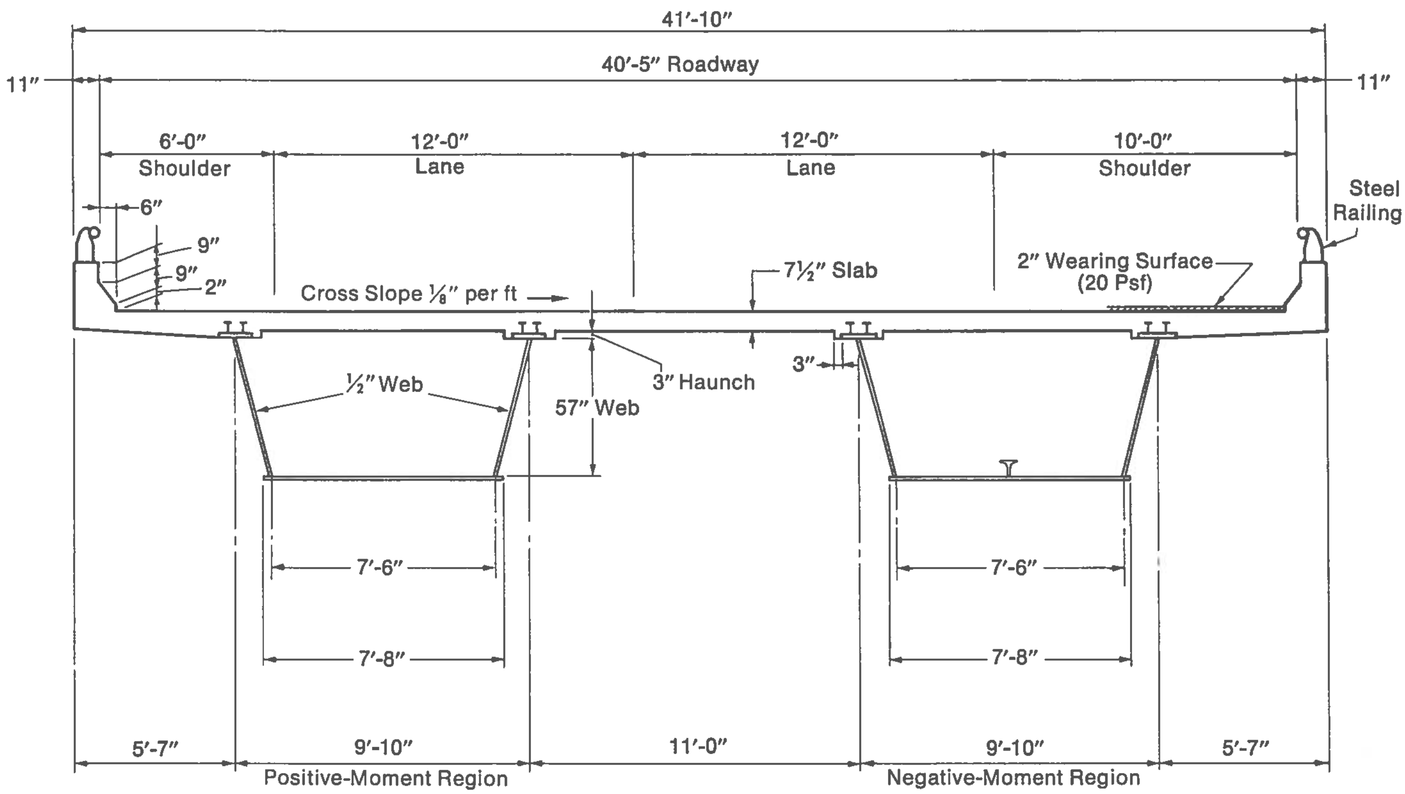
I_y = moment of inertia of the section about the YY axis

Each plate of the box section is assumed to be analogous to the bottom compression plate of a box girder and is designed for a critical buckling stress F_{cr}.

Design Example—Two-Span Rigid-Frame Box Girder (120-120 Ft) Composite for Positive and Negative Bending

The following data apply to this design:

Roadway Section: See typical bridge cross section.



TYPICAL CROSS SECTION OF EXAMPLE BRIDGE

Specifications: 1973 AASHTO Standard Specifications for Highway Bridges and Interim Specifications 1974, 1975 and 1976.

Loading: HS20-44.

Structural Steel: ASTM A36, A588 and A572, Grade 50.

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) (parapets and railings) supported by the composite section with the modular ratio $n = 8$. (Used in design of web-to-flange fillet welds.)

Case 3—Superimposed dead load (DL_2) (parapets 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 $n = 8$.

Fatigue—500,000 cycles of truck loading
100,000 cycles of lane loading

Loading Combinations:

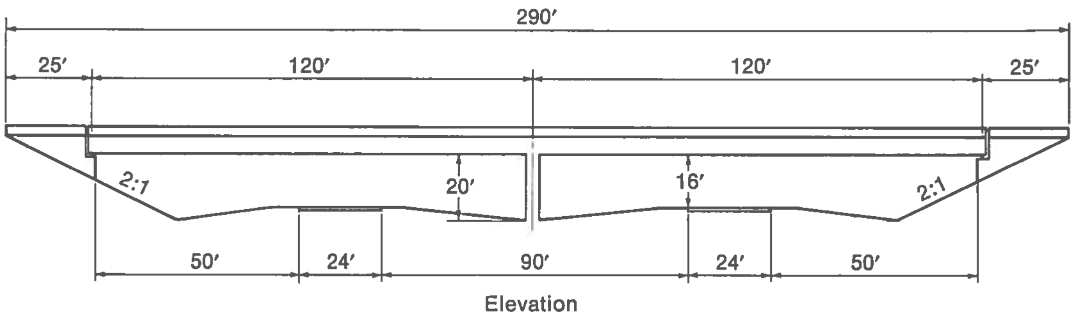
Combination A = Case 1+3+4

Combination B = Case 2+4

Combination C = Case 1+2+4

GEOMETRY OF BRIDGE

The geometric layout of the example structure is shown in an elevation view.



ELEVATION OF TWO-LANE OVERPASS STRUCTURE

LOADS, SHEARS AND MOMENTS FOR BOX GIRDER

The initial dead load (DL_1) consists of an assumed weight of 475 lb per lin ft for the box girder, plus the weight of a 7½-in. concrete slab and haunches. An average depth and width is assumed for the haunch, because the actual depth and width varies along the girder.

The superimposed dead load (DL_2) carried by the composite section consists of the weights of the parapet, 2-in. future wearing surface and single-tube steel railings.

Dead Load on Steel Box Girder

$$\begin{aligned}\text{Slab} &= 0.63\times 20.9\times 0.150 = 1.976\\ &\ 0.12\times 4.83\times 0.150 = 0.087\\ \text{Haunches} &= 0.19\times 1.67\times 0.150\times 2 = 0.095\\ \text{Girder (assumed weight)} &= \underline{0.475}\\ DL_1 \text{ per girder} &= 2.633 \text{ k/ft}\end{aligned}$$

Dead Load Carried by Composite Section

$$\begin{aligned}\text{Parapet} &= 1.50\times 0.92\times 0.150 = 0.207\\ &\ 0.37\times 0.50\times 0.150 = 0.028\\ &\ 0.17\times 1.42\times 0.150 = 0.036\\ \text{Wearing surface} &= 0.020\times 19.5 = 0.390\\ \text{Railing} &= \underline{0.020}\\ DL_2 \text{ per girder} &= 0.681 \text{ k/ft}\end{aligned}$$

Live Load on Box Girder

The live load distribution factor is calculated from the AASHTO Specification formula previously discussed. For a roadway width $W_c = 40$ ft,

$$N_w = \frac{W_c}{12} = 3.33$$

Reduced to the integer, 3. Because there are two box girders,

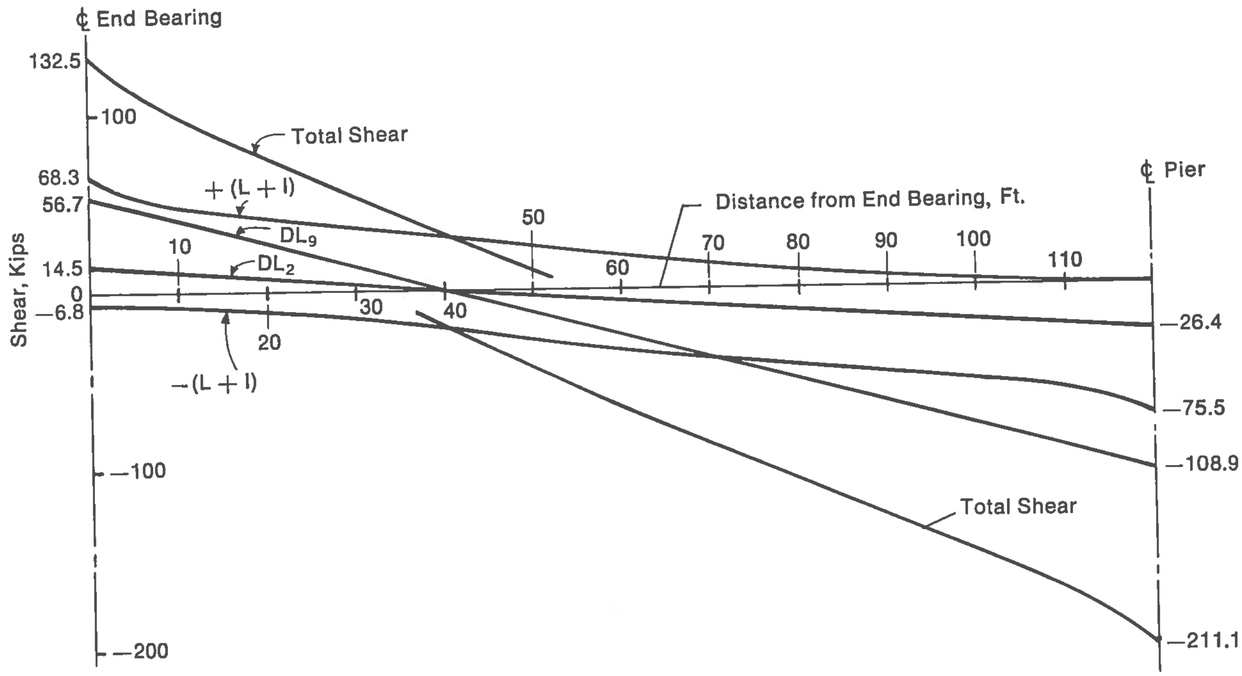
$$R = \frac{N_w}{2} = \frac{3}{2}$$

The distribution factor for live load per girder then is

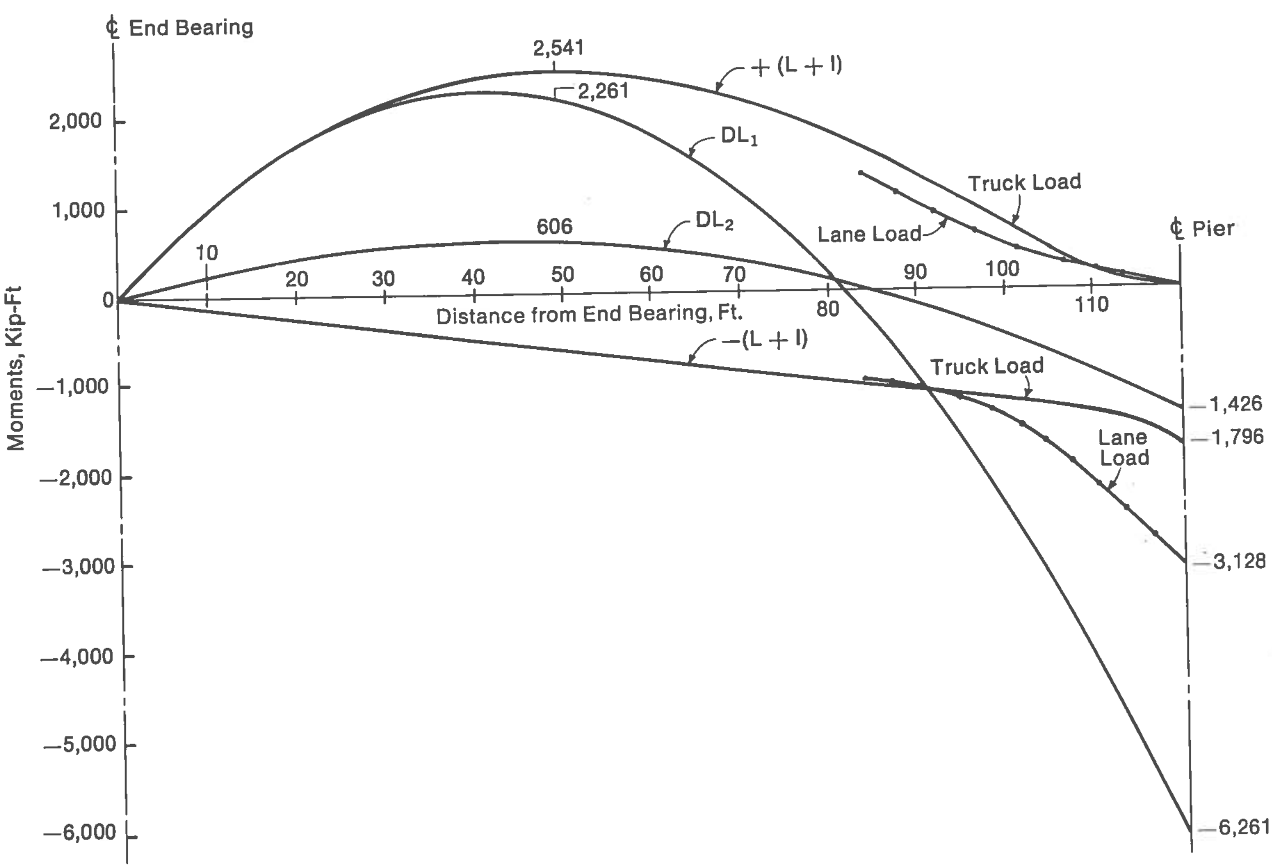
$$W_L = 0.1 + 1.7R + \frac{0.85}{N_w} = 0.1 + 1.7\times\frac{3}{2} + \frac{0.85}{3} = 2.933 \text{ wheels} = 1.467 \text{ axles}$$

$$\text{Impact} = \frac{50}{100+120} = 0.227$$

Maximum moment and maximum shear may be calculated by any convenient method. The following curves were obtained by including the effect of the center pier, which is rigidly connected to the box girder, on girder shears and moments.



MAXIMUM VERTICAL SHEAR PER WEB

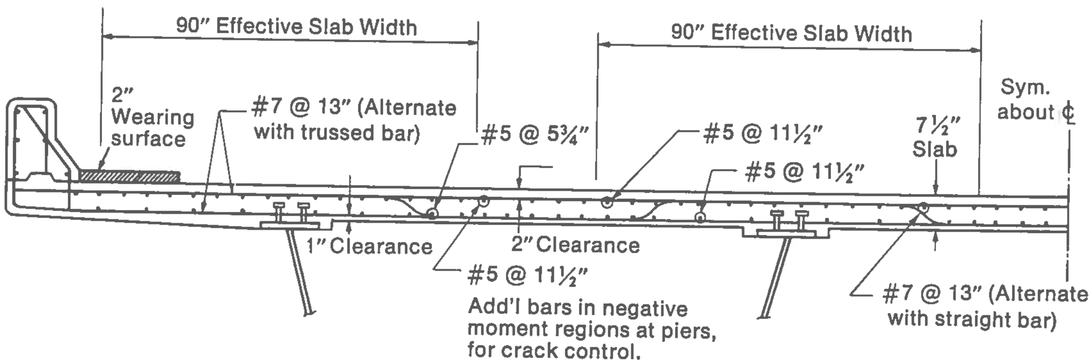


MAXIMUM-MOMENT CURVES FOR BOX GIRDER

DESIGN OF GIRDER SECTIONS

In determination of the effective width of the concrete slab for the composite section, each half of the box girder is considered equivalent to a plate girder and the usual AASHTO criteria for effective slab width are applied. Hence, the effective slab width for the box girder equals the sum of the effective slab widths for each flange.

For negative bending, the longitudinal slab reinforcement is considered part of the composite section. This steel consists of the normal distribution reinforcement and the additional bars for crack control. The area of the reinforcement and location of its center of gravity with respect to the bottom of the slab are calculated from data shown on the slab half section.



SLAB HALF SECTION

Effective Slab Width

1. One-fourth the span: $\frac{1}{4} \times \frac{3}{4} \times 120 \times 12 \times 2 = 540$ in.
2. Center to center of girders: $12[9.83 + \frac{1}{2}(9.83 + 11)] = 243$ in.
3. $12 \times$ slab thickness: $12 \times 7.5 \times 2 = 180$ in. (governs)

Area of Slab Reinforcement for Negative-Moment Section

Bar Location	No. of Bars	Area per Bar	Total Area	*d*	*Ad*
Top row	31	0.31	9.61	4.313	41.45
Bottom row	18	0.31	5.58	2.188	12.21
			15.19 in.²		53.66 in.³

$$d_{\text{Reinf.}} = \frac{53.66}{15.19} = 3.63 \text{ in.}$$

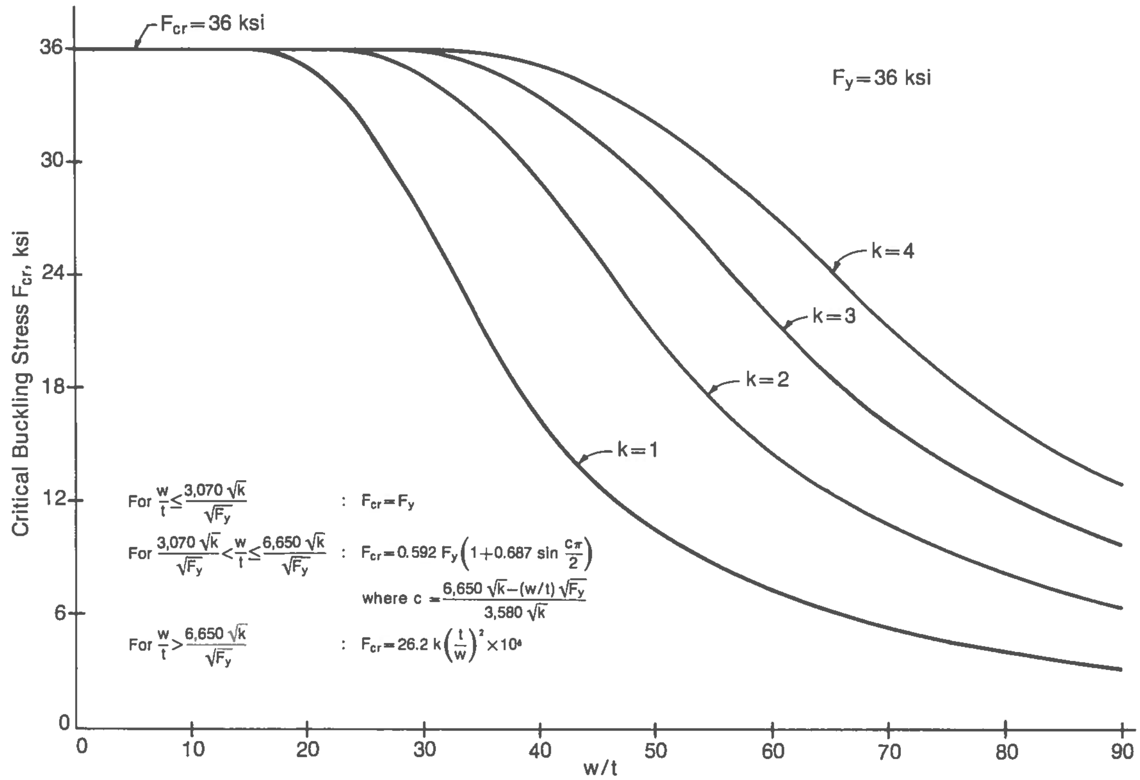
Dimensions of Bottom Flange

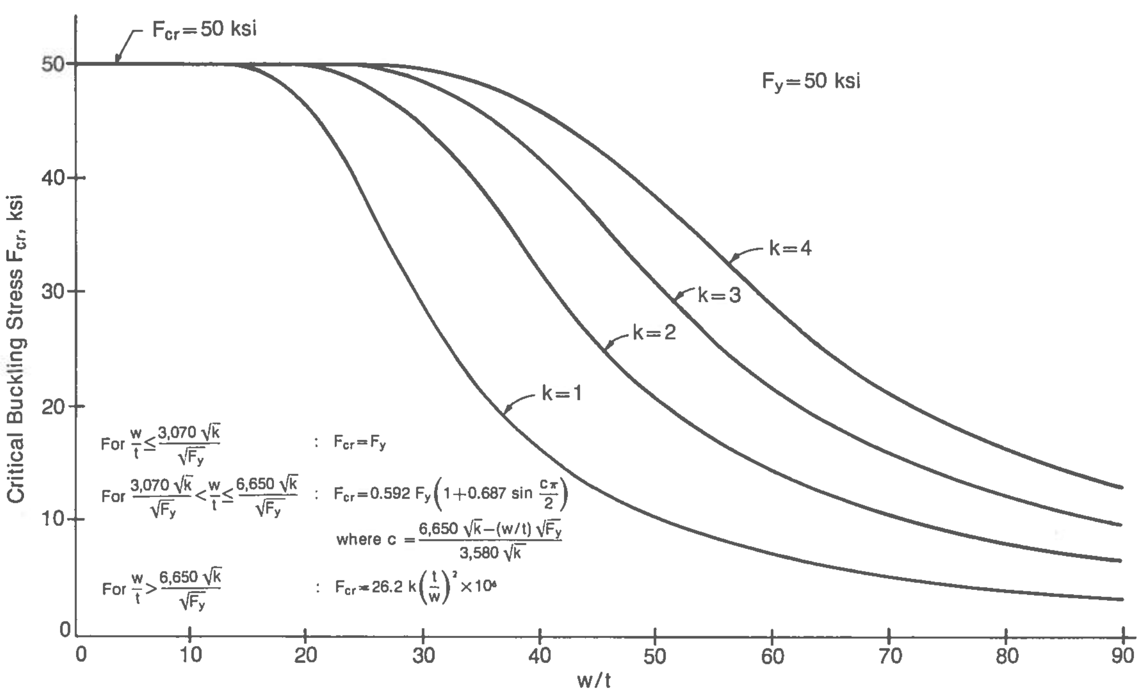
A check of the effective width of the bottom flange of the box girder in positive-moment regions indicates that the maximum effective width is greater than the full width of the flange plate. Hence, the full width is used.

$$\text{Max. Effective Width} = \tfrac{1}{5}\text{ Span} = \tfrac{1}{5} \times \tfrac{3}{4} \times 120 \times 12 = 216 > 92 \text{ in.}$$

Therefore, use the actual plate width.

In design of the bottom flange in negative-moment regions, where the flange is in compression, it is convenient to read values of the critical buckling stress F_{cr} directly from a graph rather than to calculate F_{cr} from the equations given previously. The following sets of curves, which show the variation of F_{cr} with w/t for k values of 1 to 4, may be used to obtain F_{cr}. One set of curves is based on $F_y = 36$ ksi, and the second set on $F_y = 50$ ksi. Linear interpolation may be used to determine F_{cr} for values of k between the values plotted.





BUCKLING STRESS FOR BOTTOM PLATE OF LONGITUDINALLY STIFFENED BOX GIRDER

GIRDER DEPTH AND WEB DESIGN

Because of limitations in AASHTO Specifications on depth-span ratios of girders, the box girder should be at least 36 in. deep. For greater economy, however, a 57-in. depth is selected.

AASHTO requires that the depth-span ratio H/L not exceed 1/30 for the box girder alone nor 1/25 for the box girder plus the slab. The span L is determined by the distance from the girder support at the abutment to the point of contraflexure.

Minimum Depth of Structure

$$\text{Span} = \tfrac{3}{4} \times 120 = 90 \text{ ft}$$

$$\frac{H_{\min}}{90} = \frac{1}{30} \qquad H_{\min} = \frac{90}{30} = 3 \text{ ft} = 36 \text{ in.}$$

$$\frac{H_{\min} + 10.5/12}{90} = \frac{1}{25} \qquad H_{\min} = \frac{90}{25} - \frac{10.5}{12} = 2.725 \text{ ft} = 32.7 \text{ in.}$$

The minimum permissible depth, therefore, is 36 in. A deeper section, however, will be more economical. Costs will increase, though, if the depth exceeds that at which the thickness of the flange is governed by minimum-thickness requirements rather than by stress.

Another consideration affecting economy is fabrication costs. The best current design practice prefers minimization of detail material, such as stiffeners, despite increase in main material.

Accordingly, the web for the example girders is arbitrarily designed as unstiffened in the positive-moment region. The web thickness required for this condition is maintained through the negative-moment region. In this region, however, the web is transversely stiffened where it is subject to high shear. No longitudinal stiffeners are used.

On this basis then, a ½-in.-thick web with a depth when projected on the vertical of 57 in. is selected. The web is sloped at 57 in. on 14 in., or 4.071:1. The 57-in. vertical depth is about the maximum at which, in this structure, design of most of the flange material is controlled by stress rather than minimum permissible thickness. Studies have shown that such a depth is most economical for spans of this range.

In all calculations for the web, the shear is computed for the maximum design load $1.30[D + (5/3)(L+I)]$ and resolved in the direction of the slope. The web depth D is measured along the slope.

$$D = \sqrt{57^2 + 14^2} = 58.69$$

$$\frac{D}{t_w} = \frac{58.69}{\frac{1}{2}} = 117 < 150$$

Hence, the depth-thickness requirements for an unstiffened web are satisfied.

Unstiffened Web—Positive-Moment Region

At the end bearing, the maximum design shear along the slope is

$$V' = \frac{58.69}{57} \times 1.30 \left[56.7 + 14.5 + \frac{5}{3}(68.3) \right] = 248 \text{ kips}$$

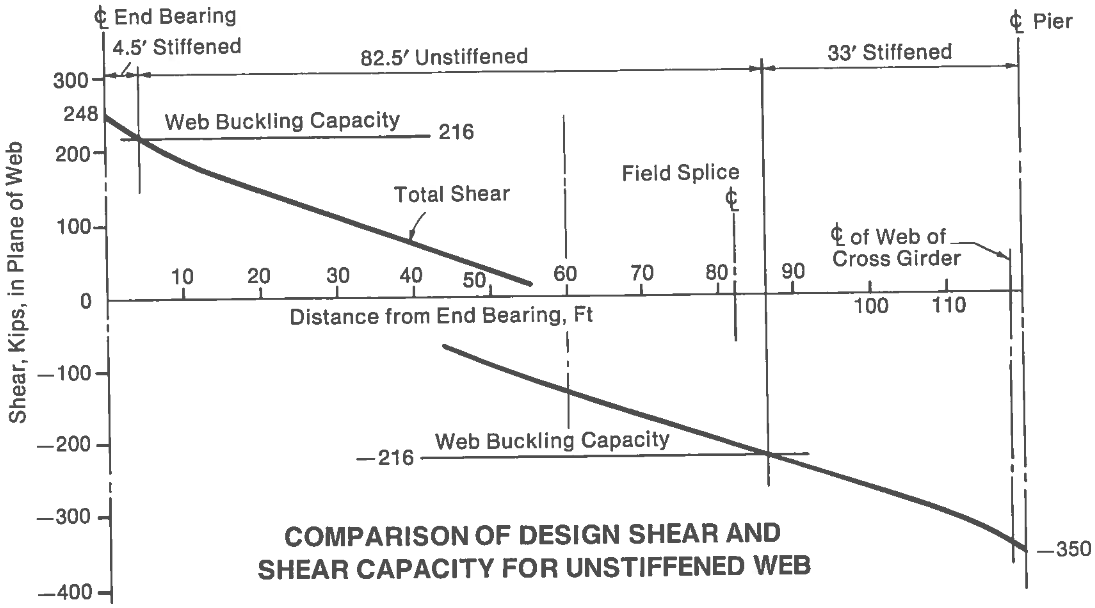
Maximum shear strength of the web is

$$V_p = 0.58 F_y D t_w = 0.58 \times 36 \times 58.69 \times \tfrac{1}{2} = 613 > 248 \text{ kips}$$

Maximum capacity of the unstiffened web for buckling is

$$V_b = \frac{3.5 E t_w^3}{D} = \frac{3.5 \times 29{,}000 (\frac{1}{2})^3}{58.69} = 216 < 248 \text{ kips}$$

While the ½-in. web satisfies ultimate strength and D/t_w requirements, the buckling capacity of the unstiffened web is less than the end shear. Rather than use a thicker web, it is more economical to add one or two stiffeners adjacent to the end bearing. Superposition of the 216-kip buckling capacity on the shear diagram indicates that the web should be transversely stiffened for a distance of 4.5 ft from the end bearing and of 33 ft from the pier. (Web-stiffener design is presented after design of the girder sections for bending stresses.)



COMPARISON OF DESIGN SHEAR AND SHEAR CAPACITY FOR UNSTIFFENED WEB

FATIGUE REQUIREMENTS

Before design of the girder sections for positive and negative-bending moment is begun, it will be helpful to summarize the fatigue checks that should be made.

At locations of maximum negative moment and at flange transitions in negative-bending regions, the top flange is in tension. If stud shear connectors are welded to this flange, as they are in this example, AASHTO fatigue Category C determines the maximum stress range permitted at those locations.

At locations of maximum positive moment and at flange transitions in positive-bending regions, the bottom flange is in tension. If a transverse stiffener or cross-frame connection plate is nearby, the stress range in the web is determined by fatigue Category C. Also, if the section being investigated is at a groove-welded flange splice, the maximum stress range in the bottom flange may not exceed that for fatigue Category B.

Box-girder sections near points of contraflexure, where stress reversals are likely to occur, should be checked for the stress range for fatigue Category C, at the top flange where shear connectors are likely to be attached and at the bottom of the web where a transverse stiffener or cross-frame connection plate is attached. Also, the bottom flange should be checked for fatigue Category B if the section is close to a transition groove weld. If a longitudinal flange stiffener is terminated in this region, the flange base metal at the end of the stiffener-to-flange fillet weld should be checked for the stress range for fatigue Category E.

AASHTO Specifications assign the following allowable ranges of stress to Categories B, C and E:

Allowable Stress Range, Psi

Category	500,000 Cycles (Truck Loading)	100,000 Cycles (Lane Loading)
B	27,500	45,000
C	19,000	32,000
E	12,500	21,000

CRITICAL BUCKLING STRESSES AT NEGATIVE-MOMENT SECTIONS

A single, longitudinal, structural tee (ST shape) is used to stiffen the bottom flange in the negative-moment region. For the box girders in this example, the single stiffener is more economical than several stiffeners.

A structural tee is an efficient shape for a longitudinal stiffener for the flange, because the tee provides a high ratio of stiffness to cross-sectional area. Other shapes, such as plates, angles or channels, however, may also be used.

The following ST shapes are chosen as possible longitudinal stiffeners, and the moment of inertia I_s about the base of the stem of each stiffener is calculated.

Moments of Inertia of Longitudinal ST Stiffeners

ST Shape	Moment of Inertia, In.^{4}
9 ×35	$84.7+10.3(6.06)^2=463.0$
7.5×25	$40.6+7.35(5.25)^2=243.2$
6 ×25	$25.2+7.35(4.16)^2=152.4$
6 ×20.4	$18.9+6.00(4.42)^2=136.1$
5 ×17.5	$12.5+5.15(3.44)^2=73.4$
4 ×11.5	$5.03+3.38(2.85)^2=32.5$
3.5×10	$3.36+2.94(2.46)^2=21.2$

With I_s known and the stiffener spacing chosen as $w=90/2=45$ in., the value of the flange buckling coefficient k furnished by the stiffener is calculated for various plate thicknesses from the equation previously given for a flange with a single stiffener.

$$k=\sqrt[3]{\frac{8I_s}{wt^3}}=\sqrt[3]{\frac{8I_s}{45t^3}}=\frac{0.562}{t}\sqrt[3]{I_s}$$

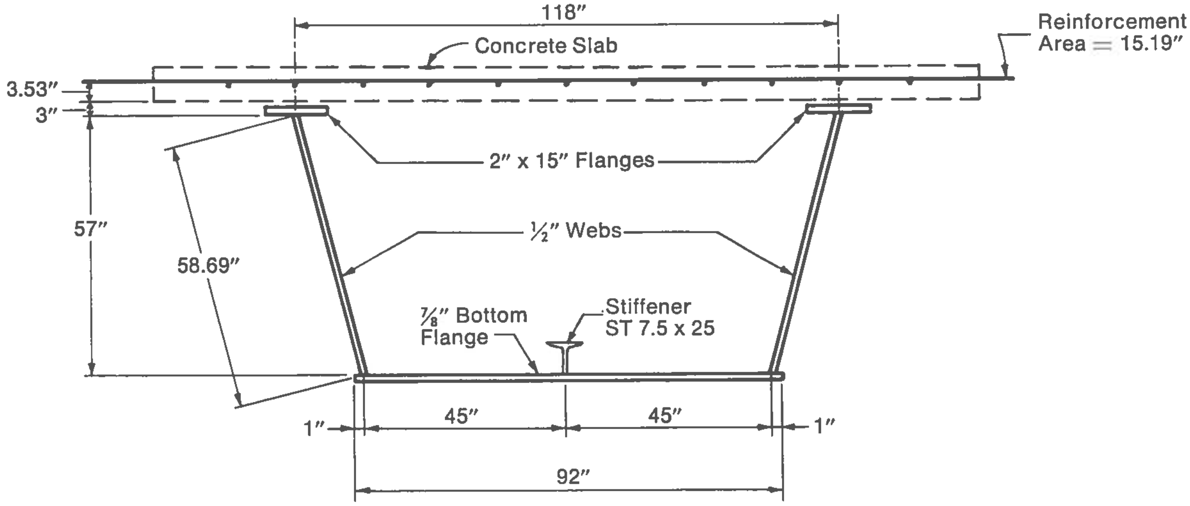
With k known, the critical buckling stress F_{cr} is obtained from the curves previously presented and listed in the following table for several plate thicknesses. Stiffeners listed in the table provide a k value as near to 4 as practicable.

Critical Flange Buckling Stress with One Longitudinal Stiffener, Ksi

t	w/t	ST Stiffener	I_s	k	F_{cr} with $F_y=36$	F_{cr} with $F_y=50$
13/16	55.4	6 ×25	152.4	3.70	28.5	31.5
		6 ×20.4	136.1	3.56	27.6	30.1
		5 ×17.5	73.4	2.90	24.2	24.7
		4 ×11.5	32.5	2.20	18.6	18.7
7/8	51.4	7.5×25	243.2	4.00	31.6	37.4
		6 ×25	152.4	3.43	29.3	33.1
		6 ×20.4	136.1	3.30	28.8	32.1
		5 ×17.5	73.4	2.69	25.2	26.7
15/16	48.0	7.5×25	243.2	3.74	32.1	38.6
		6 ×25	152.4	3.20	30.3	34.7
		6 ×20.4	136.1	3.08	30.0	34.1
		5 ×17.5	73.4	2.51	26.3	28.4
1	45.0	7.5×25	243.2	3.50	32.6	39.9
		6 ×25	152.4	3.00	31.3	36.9
		6 ×20.4	136.1	2.27	30.6	35.7
		5 ×17.5	73.4	2.35	27.3	29.8

FLANGE TRANSITION 2 FT FROM INTERIOR SUPPORT

Adjacent to the center pier, the section chosen is hybrid, with the yield stress $F_{yf}=50$ ksi for the top and bottom flange plates and $F_{yw}=36$ ksi for the web plates. In this case, F_{yw} is much larger than the minimum of 35% of F_{yf} required by AASHTO for a hybrid section.



NEGATIVE-MOMENT SECTION 2 FT FROM THE INTERIOR SUPPORT

Properties are calculated for the steel section alone where a flange transition occurs 2 ft from the center of the interior support and for that steel section plus the slab reinforcement. The moment of inertia of each inclined web I_{ow} with respect to a horizontal axis at mid-depth of the web is computed from

$$I_{ow}=\frac{S^2}{S^2+1}I_w$$

where S = web slope with respect to the horizontal = 57/14 = 4.071

I_w = moment of inertia with respect to an axis normal to the web. In the calculation of section properties, d is measured vertically from a horizontal axis through the mid-depth of the web to the centroid of each element of the box girder.

At the interior support, the bottom flange of the box girder should be designed for a biaxial state of stress at the connection to the cross girder over the pier. For this reason, design of the maximum-moment section is included with the design of the section at the negative-moment flange transition 2 ft from the interior support. This section is investigated first. The biaxially stressed flange is investigated later in this chapter.

Steel Section at Transition 2 Ft from Center of Interior Support

Material	A	d	Ad	Ad^2	I_o	I
2 T. Flg. Pl. 2×15	60.00	29.50	1,770	52,215	20	52,235
2 Web Pl. ½×58.69	58.69				15,891	15,891
Bot. Flg. Pl. ⅞×92	80.50	−28.94	−2,330	67,421		67,421
Stiff. ST 7.5×25	7.35	−23.25	−171	3,973	41	4,014
	206.54 in.²		−731 in.³			139,561

$$d_s=\frac{-731}{206.54}=-3.54\text{ in.}$$

$-3.54\times 731=$ $-2,588$

$I_{NA}=$ 136,973 in.⁴

$d_{\text{Top of steel}}=30.50+3.54=34.04$ in.

$d_{\text{Bot. of steel}}=29.38-3.54=25.84$ in.

$$S_{\text{Top of steel}}=\frac{136{,}973}{34.04}=4{,}024\text{ in.}^3$$

$$S_{\text{Bot. of steel}}=\frac{136{,}973}{25.84}=5{,}301\text{ in.}^3$$

Steel Section, with Reinforcing Steel, 2 Ft from Interior Support

Material	A	d	Ad	Ad^2	I_o	I
Steel Section	206.54		−731			139,561
Reinforcement	15.19	35.03	532	18,640		18,640
	221.73 in.^{2}		−199 in.^{3}			158,201

$$d_c = \frac{-199}{221.73} = -0.90 \text{ in.}$$

$$-0.90 \times 199 = -179$$

$$I_{NA} = 158{,}022 \text{ in.}^4$$

$$d_{\text{Top of steel}} = 30.50 + 0.90 = 31.40 \text{ in.} \qquad d_{\text{Bot. of steel}} = 29.38 - 0.90 = 28.48 \text{ in.}$$

$$S_{\text{Top of steel}} = \frac{158{,}022}{31.40} = 5{,}033 \text{ in.}^3 \qquad S_{\text{Bot. of steel}} = \frac{158{,}022}{28.48} = 5{,}549 \text{ in.}^3$$

$$d_{\text{Reinf.}} = 35.03 + 0.90 = 35.93 \text{ in.}$$

$$S_{\text{Reinf.}} = \frac{158{,}022}{35.93} = 4{,}398 \text{ in.}^3$$

As discussed previously, a hybrid section is designed for the higher strength of the flange steel but reduced by a factor R.

Investigation of Hybrid Section

In negative-moment regions, the area of the compression flange may not exceed the area of the tension flanges by more than 25%. The area of the tension flanges 2 ft from the interior support, including the area of the slab reinforcing steel, is

$$A_{ft} = 2 \times 15 \times 2 + 15.19 = 75.19 \text{ in.}^2$$

The area of the compression flange, including the area of the longitudinal stiffener, is

$$A_{fc} = 92 \times \tfrac{7}{8} + 7.35 = 87.85 \text{ in.}^2$$

The ratio of the compression-flange area to the tension-flange area is

$$\frac{87.85}{75.19} = 1.168 < 1.25$$

For determination of R for the section 2 ft from the interior support, the parameters ρ, ψ and β are calculated. For yield strength of web $F_{yw} = 36$ ksi and yield strength of flanges $F_{yf} = 50$ ksi,

$$\rho = \frac{F_{yw}}{F_{yf}} = \frac{36}{50} = 0.72$$

$$\psi = \frac{31.40}{31.40 + 28.28} = 0.524$$

$$\beta = \frac{A_w}{A_f} = \frac{58.69 \times \tfrac{1}{2}}{15 \times 2} = 0.978$$

The reduction factor for the hybrid section then is

$$R = 1 - \frac{\beta\psi(1-\rho)^2(3-\psi+\rho\psi)}{6+\beta\psi(3-\psi)}$$

$$= 1 - \frac{(0.978)(0.534)(1-0.72)^2[3-0.524+(0.72)(0.524)]}{6+(0.978)(0.524)(3-0.524)} = 0.984$$

The design relationship for Maximum Design Load on a hybrid section is

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

When the bottom flange is in compression, as it is 2 ft from the interior support, the flange yield stress in the preceding relationship should be replaced by the critical buckling stress F_{cr}. Thus, the maximum allowable bending stress becomes:

$$\text{Top flange: } RF_{yf} = 0.984 \times 50 = 49.2 \text{ ksi (tension)}$$

$$\text{Bot. flange: } RF_{cr} = 0.984 \times 37.4 = 36.8 \text{ ksi (compression)}$$

The value of F_{cr} is obtained from the table of critical buckling stresses previously presented.

Maximum Service-Load Moments 2 Ft from Interior Support

	DL_1	DL_2	$-(L+I)$	$+(L+I)$
M, kip-ft	−5,850	−1,320	−2,980	+50

Steel Stresses 2 Ft from Interior Support Due to Maximum Design Loads

Top of Steel (Tension)

For DL_1: $F_b = \dfrac{5{,}850 \times 12}{4{,}024} \times 1.30 = 22.7$

For DL_2: $F_b = \dfrac{1{,}320 \times 12}{5{,}033} \times 1.30 = 4.1$

For $L+I$: $F_b = \dfrac{2{,}980 \times 12}{5{,}033} \times 1.30 \times \dfrac{5}{3} = \underline{15.4}$

$42.2 < 49.2$ ksi

Bottom of Steel (Compression)

$F_b = \dfrac{5{,}850 \times 12}{5{,}301} \times 1.30 = 17.2$

$F_b = \dfrac{1{,}320 \times 12}{5{,}549} \times 1.30 = 3.7$

$F_b = \dfrac{2{,}980 \times 12}{5{,}549} \times 1.30 \times \dfrac{5}{3} = \underline{14.0}$

$36.8 > 34.9$ ksi

Reinforcing Steel Stress (Tension) 2 Ft from Interior Support

$$f_r = \frac{1.3 \times 12\left(1{,}320 + \frac{5}{3} \times 2{,}980\right)}{4{,}398} = 22.3 < 40 \text{ ksi}$$

Check of Fatigue-Stress Range

The fatigue-stress range in the reinforcing steel due to Service Loads is limited to 20 ksi. The live-load stress range at the interior support is computed from the Service-Load moments with a section modulus in tension of 4,398 in.^{3}

$$f_{sr} = \frac{12(2{,}980+50)}{4{,}398} = 8.27 < 20 \text{ ksi}$$

In addition to the check of the Maximum Design Load, the transition section should also be investigated for fatigue at the weld of the stud shear connector. On the assumption that a row of connectors will be placed on the top flange near the transition, the live-load stress range for the top of the steel girder at this location is determined to be

$$f_{sr} = \frac{12(2{,}980+50)}{5{,}033} = 7.22 < 32 \text{ ksi (lane load controls)}$$

The section is satisfactory for fatigue near the interior support.

Although not presented in this chapter, calculations indicate that the following arrangements could have been used as alternates for the bottom flange of the hybrid girder:

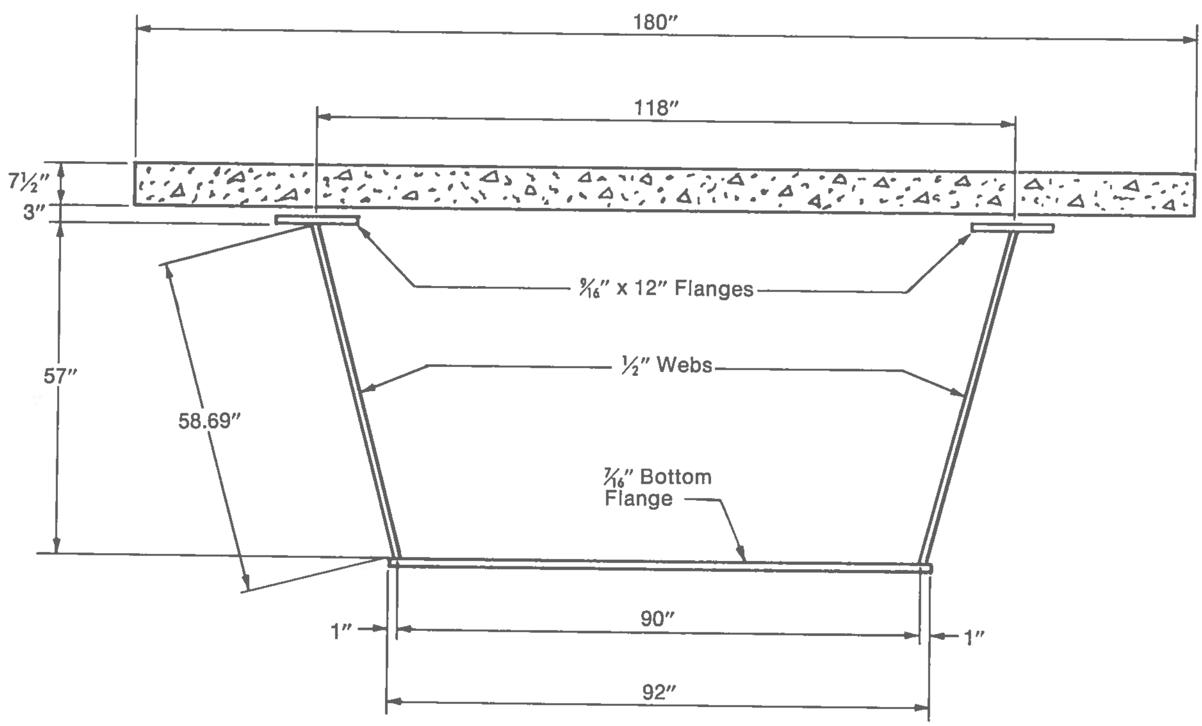
13/16-in. plate with two ST7.5×25 stiffeners
3/4-in. plate with three ST7.5×25 stiffeners

But the amount of steel saved with each 1/16-in. reduction in flange thickness does not offset the additional amount of steel added by another flange stiffener. Hence, use of a single flange stiffener is the most economical.

If the section is designed entirely of A36 steel, about 10% more material is required. But A36 steel is less expensive than the higher-strength steel required by the hybrid design. Current data indicate that A572 steel costs about 10% more than A36 steel. One-quarter of the hybrid-girder section (the webs), however, is A36 steel. Consequently, the hybrid girder costs slightly less than the A36 girder. Therefore, the hybrid section is used with a single, longitudinal, bottom-flange stiffener for the region near the center pier.

POSITIVE-MOMENT SECTION

The section for maximum positive-bending moment, which is located 48 ft from the end bearing (0.4L) is fabricated entirely of A36 steel and is designed for composite action with the concrete slab. The bottom flange of this section does not require a longitudinal stiffener.



SECTION FOR MAXIMUM POSITIVE MOMENT

Steel Section

Material	A	d	Ad	Ad^2	I_o	I
2 T. Flg. Pl. $\frac{9}{16} \times 12$	13.50	28.78	389	11,182		11,182
2 Web Pl. $\frac{1}{2} \times 58.69$	58.69				15,891	15,891
Bot. Flg. $\frac{7}{16} \times 92$	40.25	−28.72	−1,156	33,200		33,200
	112.44 in.^{2}		−767			60,273

$$d_s = \frac{-767}{112.44} = -6.82 \text{ in.}$$

$$-6.82 \times 767 = -5,231$$

$$I_{NA} = 55,042 \text{ in.}^4$$

$$d_{\text{Top of steel}} = 29.06 + 6.82 = 35.88 \text{ in.} \qquad d_{\text{Bot. of steel}} = 28.94 - 6.82 = 22.12 \text{ in.}$$

$$S_{\text{Top of steel}} = \frac{55,042}{35.88} = 1,534 \text{ in.}^3 \qquad S_{\text{Bot. of steel}} = \frac{55,042}{22.12} = 2,488 \text{ in.}^3$$

Composite Section, $3n = 24$

Material	A	d	Ad	Ad^2	I_o	I
Steel Section	112.44		−767			60,273
Conc. 180×7.5/24	56.25	35.25	1,983	69,894	267	70,161
	168.69 in.^{2}		1,216 in.^{3}			130,434

$$d_{24} = \frac{1{,}216}{168.69} = 7.21 \text{ in.} \qquad -7.21 \times 1{,}216 = -8{,}767$$

$$I_{NA} = 121{,}667 \text{ in.}^4$$

$d_{\text{Top of steel}} = 29.06 - 7.21 = 21.85$ in. $\qquad d_{\text{Bot. of steel}} = 28.94 + 7.21 = 36.15$ in.

$$S_{\text{Top of steel}} = \frac{121{,}667}{21.85} = 5{,}568 \text{ in.}^3 \qquad S_{\text{Bot. of steel}} = \frac{121{,}667}{36.15} = 3{,}366 \text{ in.}^3$$

Composite Section, $n = 8$

Material	A	d	Ad	Ad^2	I_o	I
Steel Section	112.44		−767			60,273
Conc. 180×7.5/8	168.75	35.25	5,948	209,682	791	210,473
	281.19 in.^{2}		5,181 in.^{3}			270,746

$$d_8 = \frac{5{,}181}{281.19} = 18.43 \text{ in.} \qquad 18.43 \times 5{,}181 = -95{,}486$$

$$I_{NA} = 175{,}260 \text{ in.}^4$$

$d_{\text{Top of steel}} = 29.06 - 18.43 = 10.63$ in. $\qquad d_{\text{Bot. of steel}} = 28.94 + 18.43 = 47.37$ in.

$$S_{\text{Top of steel}} = \frac{175{,}260}{10.63} = 16{,}487 \text{ in.}^3 \qquad S_{\text{Bot. of steel}} = \frac{175{,}260}{47.37} = 3{,}700 \text{ in.}^3$$

$$d_{\text{Top of conc.}} = 39.00 - 18.43 = 20.57 \text{ in.}$$

$$S_{\text{Top of conc.}} = \frac{175{,}260}{20.57} = 8{,}520 \text{ in.}^3$$

The relationship for Maximum Design Load

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

governs the design of the maximum-positive-moment section.

Bending Moments 48 Ft from End Support

	DL_1	DL_2	$-(L+I)$	$+(L+I)$
M, kip-ft	2,261	606	−607	2,541

Steel Stresses—Combination A

Top of Steel (Compression)

For DL_1: $F_b = \frac{2{,}261 \times 12}{1{,}534} \times 1.30 = 23.0$

For DL_2: $F_b = \frac{606 \times 12}{5{,}568} \times 1.30 = 1.7$

For $L+I$: $F_b = \frac{2{,}541 \times 12}{16{,}487} \times 1.30 \times \frac{5}{3} = \underline{4.0}$

$28.7 < 36$ ksi

Bottom of Steel (Tension)

$F_b = \frac{2{,}261 \times 12}{2{,}488} \times 1.30 = 14.2$

$F_b = \frac{606 \times 12}{3{,}366} \times 1.30 = 2.8$

$F_b = \frac{2{,}541 \times 12}{3{,}700} \times 1.30 \times \frac{5}{3} = \underline{17.9}$

$36 > 34.9$ ksi

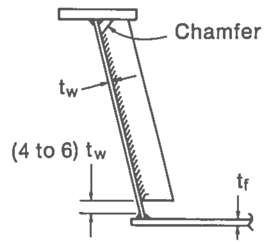
Stress at Top of Concrete—Combination B

$$f_c=\frac{1.3\times12\left(606+\frac{5}{3}\times2{,}541\right)}{8{,}520\times8}=1.11<(0.85\times4.0=3.4\text{ ksi})$$

Check for Fatigue at Web Fillet Welds

As pointed out previously, box girders often are braced by cross frames at intervals throughout the span, to stabilize the sections during handling. In this example, cross frames are placed at about the one-third points of each span. At each cross frame, a connection plate is fillet welded to the box-girder webs like a transverse-stiffener connection. The webs, therefore, should be investigated for fatigue at the toe of the connection-plate fillet weld.

SECTION AT WEB STIFFENER



It is recommended practice to terminate the fillet weld that connects a transverse stiffener to the web at a distance of four to six times the web thickness t_w from the inner face of the tension flange. With the end of the weld at a distance of $4t_w$, the maximum bending stress at the toe of the stiffener fillet weld is

$$f_b=\frac{M(y-4t_w-t_f)}{I}$$

where y = distance from centroidal axis of girder to bottom of steel section
t_f = flange thickness

The cross-frame connection plate in the positive-moment region is located 41 ft from the end support. The range of the live-load moments at this location is

$$M_L\text{ Range}=2{,}470+520=2{,}990\text{ kip-ft}$$

The range of tensile stress at the connection-plate fillet weld is then calculated as

$$f_b=\frac{2{,}990\times12}{175{,}260}(47.37-4\times0.5-0.438)=9.2<19\text{ ksi}$$

The positive-moment section therefore is satisfactory.

FLANGE-PLATE TRANSITION 25 FT FROM END SUPPORT

The thickness of the bottom flange is reduced from 7/16 in. to 5/16 in. at a distance of 25 ft from the end support. The thickness of the steel top flanges is maintained at 9/16 in. The section at the transition is investigated.

Steel Section

Material	A	d	Ad	Ad^2	I_o	I
2 T. Flg. Pl. 9/16×12	13.50	28.78	389	11,182		11,182
2 Web Pl. ½×58.69	58.69				15,891	15,891
Bot. Flg. 5/16×92	28.75	−28.66	−824	23,615		23,615
	100.94 in.²		−435 in.³			50,688

$$d_s=\frac{-435}{100.94}=-4.31\text{ in.}$$

$$-4.31\times435=-1{,}875$$

$$I_{NA}=48{,}813\text{ in.}^4$$

$$d_{\text{Top of steel}}=29.06+4.31=33.37\text{ in.}\qquad d_{\text{Bot. of steel}}=28.81-4.31=24.50\text{ in.}$$

$$S_{\text{Top of steel}}=\frac{48{,}813}{33.37}=1{,}463\text{ in.}^3\qquad S_{\text{Bot. of steel}}=\frac{48{,}813}{24.50}=1{,}992\text{ in.}^3$$

Composite Section, $3n = 24$

Material	A	d	Ad	Ad^2	I_o	I
Steel Section	100.94		−435			50,688
Conc. 180×7.5/24	56.25	35.25	1,983	69,894	267	70,161
	157.19 in.^{2}		1,548 in.^{3}			120,849

$$d_{24} = \frac{1{,}548}{157.19} = 9.85 \text{ in}$$

$$-9.85 \times 1{,}548 = \underline{-15{,}248}$$
$$I_{NA} = 105{,}601 \text{ in.}^4$$

$$d_{\text{Top of steel}} = 29.06 - 9.85 = 19.21 \text{ in.} \qquad d_{\text{Bot. of steel}} = 28.81 + 9.85 = 38.66 \text{ in.}$$

$$S_{\text{Top of steel}} = \frac{105{,}601}{19.21} = 5{,}497 \text{ in.}^3 \qquad S_{\text{Bot. of steel}} = \frac{105{,}601}{38.66} = 2{,}732 \text{ in.}^3$$

Composite Section, $n = 8$

Material	A	d	Ad	Ad^2	I_o	I
Steel Section	100.94		−435			50,688
Conc. 180×7.5/8	168.75	35.25	5,948	209,682	791	210,473
	269.69 in.^{2}		5,513 in.^{3}			261,161

$$d_8 = \frac{5{,}513}{269.69} = 20.44 \text{ in.}$$

$$-20.44 \times 5{,}513 = \underline{-112{,}686}$$
$$I_{NA} = 148{,}475 \text{ in.}^4$$

$$d_{\text{Top of steel}} = 29.06 - 20.44 = 8.62 \text{ in.} \qquad d_{\text{Bot. of steel}} = 28.81 + 20.44 = 49.25 \text{ in.}$$

$$S_{\text{Top of steel}} = \frac{148{,}475}{8.62} = 17{,}224 \text{ in.}^3 \qquad S_{\text{Bot. of steel}} = \frac{148{,}475}{49.25} = 3{,}015 \text{ in.}^3$$

$$d_{\text{Top of conc.}} = 39.00 - 20.44 = 18.56 \text{ in.}$$

$$S_{\text{Top of conc.}} = \frac{148{,}475}{18.56} = 7{,}999 \text{ in.}^3$$

As with the maximum-positive-moment section, the relationship for Maximum Design Load governs the design of the section 25 ft from the end support. Fatigue in the base metal adjacent to the butt-welded flange transition should be checked. Fatigue in the girder webs at the toe of the transverse-stiffener fillet welds need not be investigated, because stiffeners are used only in the vicinity of the end bearing, where the live-load stress range is small.

Bending Moments 25 Ft from End Support

	DL_1	DL_2	$-(L+I)$	$+(L+I)$
M, kip-ft	1,970	510	−310	1,960

Steel Stresses 25 Ft from End Support Due to Maximum Design Loads

Top of Steel (Compression)

For DL_1: $F_b = \dfrac{1{,}970 \times 12}{1{,}463} \times 1.30 = 21.0$

For DL_2: $F_b = \dfrac{510 \times 12}{5{,}497} \times 1.30 = 1.4$

For $L+I$: $F_b = \dfrac{1{,}960 \times 12}{17{,}224} \times 1.30 \times 5 = \underline{3.0}$

$25.4 < 36$ ksi

Bottom of Steel (Tension)

$F_b = \dfrac{1{,}970 \times 12}{1{,}992} \times 1.30 = 15.4$

$F_b = \dfrac{510 \times 12}{2{,}732} \times 1.30 = 2.9$

$F_b = \dfrac{1{,}960 \times 12}{3{,}015} \times 1.30 \times \dfrac{5}{3} = \underline{16.9}$

$36 > 35.2$ ksi

Stress at Top of Concrete (Compression)

$$f_c=\frac{1.3\times 12\left(510+\frac{5}{3}\times 1,960\right)}{7,999\times 8}=0.92<(0.85\times 4.0=3.4\text{ ksi})$$

The section therefore is satisfactory for Maximum Design Load.

Check for Fatigue at Flange Transition

The range of live-load stress in the bottom flange at the transition is

$$f_{sr}=\frac{12(1,960+310)}{3,015}=9.0<27.5\text{ ksi}$$

Resistance to fatigue therefore is satisfactory at the transition 25 ft from the end bearing.

SECTION TRANSITION 17 FT FROM INTERIOR SUPPORT

A hybrid section is used for the box girder from the interior support to a point on the girder 17 ft away. There, the top and bottom flanges are changed to A36 steel. There are two reasons for fabricating the section entirely of A36 steel rather than hybrid. One reason is that the bending moment decreases rapidly with distance from the pier. As a result, the strength of a hybrid section more than 17 ft from the interior support would be excessive. The second reason is that a compression flange of suitable thickness of steel with $F_y=50,000$ psi would require tension flanges larger than necessary to satisfy the criteria:

$$\frac{\text{Compression-flange area}}{\text{Tension-flange area}}\leq 1.25$$

The transition section is investigated in the same manner as for the transition section 2 ft from the pier, except that the former section is not hybrid. At the transition, the top flanges are made 1 in. thick, and the bottom flange is made 11⁄16 in. thick. The ST7.5×25 bottom-flange longitudinal stiffener, continued from the pier to the inflection region, is stiff enough to provide the maximum k value of 4.

Steel Section 17 Ft from Interior Support

Material	A	d	Ad	Ad^2	I_o	I
2 T. Flg. Pl. 1×15	30.00	29.00	870	25,230		25,230
2 Web Pl. ½×58.69	58.69				15,891	15,891
Bot. Flg. Pl. 11⁄16×92	63.25	−28.84	−1,824	52,608		52,608
Stiff. ST7.5×25	7.35	−23.25	−171	3,973	41	4,014
	159.29 in.²		−1,125 in.³			97,743

$$d_s=\frac{-1,125}{159.29}=-7.06\text{ in.}$$

$$-7.06\times 1,125=\underline{-7,942}$$

$$I_{NA}=89,801\text{ in.}^4$$

$$d_{\text{Top of steel}}=29.50+7.06=36.56\text{ in.}$$

$$d_{\text{Bot. of steel}}=29.19-7.06=22.13\text{ in.}$$

$$S_{\text{Top of steel}}=\frac{89,801}{36.56}=2,456\text{ in.}^3$$

$$S_{\text{Bot. of steel}}=\frac{89,801}{22.13}=4,058\text{ in.}^3$$

Steel Section, with Reinforcing Steel, 17 Ft from Interior Support

Material	A	d	Ad	Ad^2	I_o	I
Steel Section	159.29		−1,125			97,743
Reinforcement	15.19	35.03	532	18,640		18,640
	174.48 in.^{2}		−593 in.^{3}			116,383

$$d_c=\frac{-593}{174.48}=-3.40 \text{ in.} \qquad -3.40\times 593 = \underline{-2{,}016}$$

$$I_{NA}=114{,}367 \text{ in.}^4$$

$$d_{\text{Top of steel}}=29.50+3.40=32.90 \text{ in.} \qquad d_{\text{Bot. of steel}}=29.19-3.40=25.79 \text{ in.}$$

$$S_{\text{Top of steel}}=\frac{114{,}367}{32.90}=3{,}476 \text{ in.}^3 \qquad S_{\text{Bot. of steel}}=\frac{114{,}367}{25.79}=4{,}435 \text{ in.}^3$$

$$d_{\text{Reinf.}}=35.03+3.40=38.43 \text{ in.}$$

$$S_{\text{Reinf.}}=\frac{114{,}367}{38.43}=2{,}976 \text{ in.}^3$$

Service-Load Moments 17 Ft from Interior Support

	DL_1	DL_2	Lane Load $-(L+I)$	Truck Load $-(L+I)$	Truck Load $+(L+I)$
M, kip-ft	−3,000	−630	−1,650	−1,290	580

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

Top of Steel (Tension)

For DL_1: $F_b=\dfrac{3{,}000\times 12}{2{,}456}\times 1.30 = 19.1$

For DL_2: $F_b=\dfrac{630\times 12}{3{,}476}\times 1.30 = 2.8$

For $L+I$: $F_b=\dfrac{1{,}650\times 12}{3{,}476}\times 1.30\times\dfrac{5}{3}=\underline{12.3}$

34.2 < 36 ksi

Bottom of Steel (Compression)

$F_b=\dfrac{3{,}000\times 12}{4{,}058}\times 1.30 = 11.5$

$F_b=\dfrac{630\times 12}{4{,}435}\times 1.30 = 2.2$

$F_b=\dfrac{1{,}650\times 12}{4{,}435}\times 1.30\times\dfrac{5}{3}=\underline{9.7}$

23.4 ksi

Reinforcing Steel Stress (Tension) 17 Ft from Interior Support

$$f_r=\frac{1.3\times 12\left(630+\frac{5}{3}\times 1{,}650\right)}{2{,}2976}=17.7<40 \text{ ksi}$$

Check for Buckling

$$k=\sqrt[3]{\frac{8I_s}{wt^3}}=\sqrt[3]{\frac{8\times 243.2}{45\,(^{11}\!/_{16})^3}}=5.1>4$$

Use $k=4$. The ratio of stiffener spacing to flange thickness $w/t=45/(^{11}\!/_{16})=65.5$. From the curves for allowable buckling stress, $F_{cr}=24.0>23.4$. Resistance of the bottom flange to buckling therefore is satisfactory.

Check for Fatigue

Because shear connectors are welded to the top flange, a fatigue check should be made at the transition section to insure that the tensile-stress range in the top flange is within the allowable. The range of live-load stress in the top flange is

$$f_{sr}=\frac{12(1{,}290+580)}{3{,}476}=6.46<19.0 \text{ ksi}$$

The fatigue-stress range in the reinforcing steel is investigated next.

$$f_{sr}=\frac{12(1{,}290+580)}{2{,}976}=7.54<20 \text{ ksi}$$

Resistance of the reinforcement to fatigue is therefore satisfactory.

SECTION NEAR FIELD SPLICE

A field splice is placed 37 ft from the interior support. Here, a transition is made from the negative-moment section made of A36 steel to the positive-moment section used through the maximum-positive-moment region.

For some distance into the dead-load positive-moment region, the bending moment resulting from the sum of the dead-load moment and the negative live-load moment is negative and produces compression in the bottom flange. The 7/16-in. bottom flange used for the maximum-positive-moment section would not have sufficient buckling resistance under this condition unless it is stiffened longitudinally. Therefore, the ST7.5×25 longitudinal stiffener used in the negative-moment region is extended through the field splice and into the dead-load positive-moment region. The stresses on the gross section at the field splice are checked as follows:

Steel Section 37 Ft from Interior Support

Material	A	d	Ad	Ad^2	I_o	I
2 T. Flg. Pl. 9/16×12	13.50	28.78	389	11,182		11,182
2 Web Pl. 1/2×58.69	58.69				15,891	15,891
Bot. Flg. Pl. 7/16×92	40.25	−28.72	−1,156	33,200		33,200
Stiff. ST7.5×25	7.35	−23.25	−171	3,973	41	4,014
	119.79 in.²		−938 in.³			64,287

$$d_s=\frac{-938}{119.79}=-7.83 \qquad -7.83\times 938=\underline{-7{,}345}$$

$$I_{NA}=56{,}942 \text{ in.}^4$$

$$d_{\text{Top of steel}}=29.06+7.83=36.89 \text{ in.} \qquad d_{\text{Bot. of steel}}=28.94-7.83=21.11 \text{ in.}$$

$$S_{\text{Top of steel}}=\frac{56{,}942}{36.89}=1{,}544 \text{ in.}^3 \qquad S_{\text{Bot. of steel}}=\frac{56{,}942}{21.11}=2{,}697 \text{ in.}^3$$

Steel Section, with Reinforcing Steel, 37 Ft from Interior Support

Material	A	d	Ad	Ad^2	I_o	I
Steel Section	119.79		−938			64,287
Reinforcement	15.19	35.03	532	18,640		18,640
	134.98 in.²		−406 in.³			82,927

$$d_c=\frac{-406}{134.98}=-3.01 \text{ in.} \qquad -3.01\times 406=\underline{-1{,}222}$$

$$I_{NA}=81{,}705 \text{ in.}^4$$

$$d_{\text{Top of steel}}=29.06+3.01=32.07 \text{ in.} \qquad d_{\text{Bot. of steel}}=28.94-3.01=25.93 \text{ in.}$$

$$S_{\text{Top of steel}}=\frac{81{,}705}{32.07}=2{,}548 \text{ in.}^3 \qquad S_{\text{Bot. of steel}}=\frac{81{,}705}{25.93}=3{,}151 \text{ in.}^3$$

$$d_{\text{Reinf.}}=35.03+3.01=38.04 \text{ in.}$$

$$S_{\text{Reinf.}}=\frac{81{,}705}{38.04}=2{,}148 \text{ in.}^3$$

Service-Load Moments at Field Splice 37 Ft from Interior Support

	DL_1	DL_2	$+(L+I)$	$-(L+I)$
M, kip-ft	−100	50	1,690	−1,050

Steel Stresses 37 Ft from Interior Support Due to Maximum Design Loads

Top of Steel (Tension)

For DL_1: $F_b = \frac{100\times 12}{1{,}544}\times 1.30 = 1.0$

For DL_2: $F_b = \frac{-50\times 12}{2{,}548}\times 1.30 = -0.3$

For $L+I$: $F_b = \frac{1{,}050\times 12}{2{,}548}\times 1.30\times\frac{5}{3} = \underline{10.7}$

$11.4 < 36$ ksi

Bottom of Steel (Compression)

$F_b = \frac{100\times 12}{2{,}697}\times 1.30 = 0.6$

$F_b = \frac{-50\times 12}{3{,}151}\times 1.30 = -0.2$

$F_b = \frac{1{,}050\times 12}{3{,}151}\times 1.30\times\frac{5}{3} = \underline{8.7}$

9.1 ksi

Reinforcing Steel Stress (Tension) 37 Ft from Interior Support

$$f_r = \frac{1.3\times 12\left(-50+\frac{5}{3}\times 1{,}050\right)}{2{,}148} = 12.3 < 40 \text{ ksi}$$

Check for Buckling

$$k = \sqrt[3]{\frac{8I_s}{wt^3}} = \sqrt[3]{\frac{8\times 243.2}{45(7/16)^3}} = 8.02 > 4$$

Use $k=4$. The ratio of stiffener spacing to flange thickness $w/t = 45/(7/16) = 102.9$. The curves for critical buckling stress presented previously do not extend to a value of w/t this large. Hence, F_{cr} must be calculated.

$$\frac{w}{t} = \frac{6{,}650\sqrt{k}}{\sqrt{F_y}} = \frac{6{,}650\sqrt{4}}{\sqrt{36{,}000}} = 70.1 < 102.9$$

$$F_{cr} = 26.2\times 10^3 k\left(\frac{t}{w}\right)^2 = 26{,}200\times 4\left(\frac{1}{102.9}\right)^2 = 9.9 > 9.1 \text{ ksi}$$

Check for Fatigue

Because the fillet weld of the longitudinal stiffener to the bottom flange is interrupted at the field splice, fatigue of the base metal adjacent to the weld should be checked for tension at the bottom of the stiffener stem, where the weld passes across the thickness of the stem. On the assumption that the longitudinal stiffener is made continuous at the field splice by some sort of splice arrangement, the section properties of the girder with the longitudinal stiffener may be used for calculation of stresses. Section properties therefore are computed for the composite section, including the longitudinal flange stiffener.

Composite Section, $n=8$

Material	A	d	Ad	Ad^2	I_o	I
Steel section	119.79		−938			64,287
Conc. 180×7.5/8	168.75	35.25	5,948	209,682	791	210,473
	288.54 in.²		5,010 in.³			274,760

$$d_s = \frac{5{,}010}{288.54} = 17.36 \text{ in.}$$

$$-17.36\times 5{,}010 = \underline{-86{,}974}$$

$$I_{NA} = 187{,}786 \text{ in.}^4$$

$$d_{\text{Top of steel}} = 29.06 - 17.36 = 11.70 \text{ in.}$$

$$d_{\text{Bot. of steel}} = 28.94 + 17.36 = 46.30 \text{ in.}$$

$$S_{\text{Top of steel}} = \frac{187{,}786}{11.70} = 16{,}050 \text{ in.}^3$$

$$S_{\text{Bot. of steel}} = \frac{187{,}786}{46.30} = 4{,}056 \text{ in.}^3$$

The maximum range of live-load stress in the bottom flange at the end of the stiffener-to-flange fillet weld is

$$f_{sr}=\frac{1{,}050\times12\times25.49}{81{,}705}+\frac{1{,}690\times12\times45.86}{187{,}786}=3.9+5.0=8.9<12.5\text{ ksi}$$

An additional fatigue check is made for tension in the top flange of the girder. On the assumption that shear connectors are welded to the top flange near the splice, the stress range at that section may not exceed 19 ksi. The stress range is

$$f_{sr}=\frac{1{,}050\times12}{2{,}548}+\frac{1{,}690\times12}{16{,}050}=4.9+1.3=6.2<19\text{ ksi}$$

TERMINATION OF LONGITUDINAL STIFFENER

Next, an investigation is made to determine the location of the section at which the ST7.5×25 may be terminated in the positive-bending region. The following calculations indicate that at a distance of 10 ft from the field splice, or 47 ft from the interior support, the compressive stress in the bottom flange is less than the critical buckling stress for the bottom-flange plate, so that the longitudinal stiffener is no longer needed.

Service-Load Moments 47 Ft from Interior Support

	DL_1	DL_2	$+(L+I)$	$-(L+I)$
M, kip-ft	930	300	2,110	925

Steel Stresses 47 Ft from Interior Support Due to Maximum Design Loads

	Top of Steel (Tension)		Bottom of Steel (Compression)	
For DL_1:	$F_b=\frac{-930\times12}{1{,}534}\times1.30$	$=-9.5$	$F_b=\frac{-930\times12}{2{,}488}\times1.30$	$=-5.8$
For DL_2:	$F_b=\frac{-300\times12}{1{,}534}\times1.30$	$=-3.1$	$F_b=\frac{-300\times12}{2{,}488}\times1.30$	$=-1.9$
For $-(L+I)$:	$F_b=\frac{925\times12}{1{,}534}\times1.30\times\frac{5}{3}=$	15.7	$F_b=\frac{925\times12}{2{,}488}\times1.30\times\frac{5}{3}=$	9.7
		$3.1<36$ ksi		2.0 ksi

Check for Buckling

For the bottom flange without a stiffener, $k=4$. The ratio of flange width to thickness $w/t=90/(7/16)=205.7$. The curves for allowable buckling stress presented previously do not extend to a value of w/t this large. Consequently, F_{cr} must be calculated.

$$\frac{w}{t}=\frac{6{,}650\sqrt{k}}{\sqrt{F_y}}=\frac{6{,}650\sqrt{4}}{\sqrt{36{,}000}}=70.1<205.7$$

$$F_{cr}=26.2\times10^3k\left(\frac{t}{b}\right)^2=26{,}200\times4\left(\frac{7/16}{90}\right)^2=2.5>2.0\text{ ksi}$$

Fatigue Check at End of Longitudinal Stiffener

The maximum range of stress in the bottom flange at the end of the longitudinal stiffener is

$$f_{sr}=\frac{12(2{,}110+925)46.93}{175{,}260}=9.8<12.5\text{ ksi}$$

LATERAL FLANGE BENDING

The change along the girder span, kips per ft, in the horizontal component of the web shear acts as a lateral horizontal force on the flange of the box girder. Under the initial dead load DL_1, the lateral force due to shear is assumed to be equally distributed to the top and bottom flanges. The lateral force on the top flange causes lateral bending of that flange. The change in vertical shear, and therefore the lateral load on both flanges, is constant. It is equal to the difference between the shears at the girder supports divided by the span.

Let ΔV_V be the change in DL_1 vertical shear, kips per ft, along the girder. With the shear at the end bearing equal to 56.7 kip-ft and the shear at the interior support equal to -108.9 kip-ft,

$$\Delta V_V = \frac{56.7+108.9}{120} = 1.38 \text{ kips per ft}$$

The horizontal component of the web shear then is

$$\Delta V_H = \frac{14}{57} \times 1.38 = 0.34 \text{ kips per ft}$$

One-half of ΔV_H, or 0.17 kips per ft, is applied to the top flange as a uniformly distributed lateral force.

To support the top flange under this loading, a strut is placed at appropriate intervals between the webs of the box girder, just below the top flange. The spacing of the struts, the forces acting on them and the deflection of the top flange midway between struts are determined.

Lateral Bracing in Positive-Moment Region

The top flange is checked first in the positive-moment region for the combination of lateral and vertical bending. Previous investigations of vertical bending indicated that this flange is understressed. The stress in the flange at the section of maximum positive moment is 28.7 ksi. The capacity available at the section for resisting lateral bending is

$$f_L = 36.0 - 28.7 = 7.3 \text{ ksi}$$

Factored $\Delta V_H = 1.30 \times 0.17 = 0.22$ kips per ft.

Assume that the lateral bending moment at a strut is

$$M = \frac{\Delta V_H d^2}{12}$$

where d = spacing, ft, of struts

The section modulus of the $\frac{9}{16} \times 12$-in. top flange is

$$S_f = \frac{tw^2}{6} = \tfrac{1}{6} \times \tfrac{9}{16}(12)^2 = 13.5 \text{ in.}^3$$

The lateral bending stress then is

$$f_L = \frac{12M}{S_f} = \frac{12M}{13.5} = 0.889M = 0.889\frac{\Delta V_H d^2}{12} = 0.074 \times 0.22d^2 = 0.0163d^2$$

With $f_L = 7.3$ ksi, solving for d yields

$$d \leq \sqrt{\frac{7.3}{0.0163}} = 21.2 \text{ ft}$$

Bracing of the top flange is provided by a strut incorporated in the cross frames, which are placed at about the third points of the girder span. In addition, a strut is placed midway between the end bearing and the cross frame in the positive-moment region. Spacing of the struts then is 20.5 ft.

The force in the struts $= wd = \Delta V_H d = 0.22 \times 20.5 = 4.5$ kips.

The deflection midway between struts is

$$\Delta=\frac{wd^4}{384EI}=\frac{(0.22/12)(20.5\times 12)^4}{384\times 28{,}000(1/12)(9/16)(12)^3}=0.07 \text{ in.}$$

Lateral Bracing in Negative-Moment Region

The top flange is checked next in the negative-moment region. A strut is placed about midway between the cross frame 36 ft from the interior support, near the field splice, and the cross girder 1.5 ft from the interior support. The total stress in the top flange is computed first for the unbraced span between the strut and the cross frame 36 ft from the interior support and then for the unbraced span between the strut and the cross girder. The unbraced spans are ½(36−1.5)=17.25 ft long.

$$M=\frac{0.22(17.25)^2}{12}=5.5 \text{ kip-ft}$$

To simplify calculations for the stress at the strut, the vertical bending stress is taken as that at the flange transition 17 ft from the interior support. The resulting stress is on the conservative side, because the strut is actually 18 ft 3 in. from the interior support, where the stress is smaller.

The section modulus of the section at the transition is

$$S_f=\frac{1(15)^2}{6}=37.5 \text{ in.}^3$$

$$f_L=\frac{5.5\times 12\times 6}{37.5}=1.8 \text{ ksi}$$

$$\text{Stress from vertical bending}=\frac{34.2}{36.0}=F_y$$

Similarly, to simplify calculations for stress at the cross girder, the vertical bending stress is taken as that at the flange transition 2 ft from the interior support. The lateral bending stress is on the conservative side, because the cross girder is actually 1.5 ft from the interior support. Hence, the unbraced span is 18.25−1.50=16.75 <17.25 ft. The section modulus is

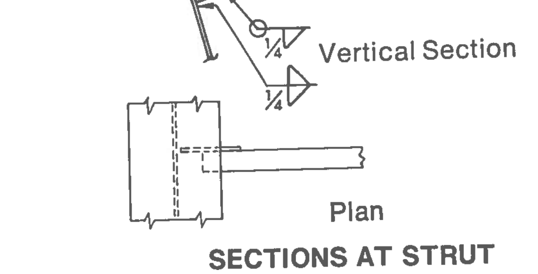
$$S_f=\frac{2(15)^2}{6}=75 \text{ in.}^3$$

$$f_L=\frac{5.5\times 12}{75}=0.9 \text{ ksi}$$

$$\text{Stress from vertical bending}=\frac{42.2}{43.1}<(0.984\times 50=49.2 \text{ ksi})$$

Design of Struts

The strut are all the same size and designed for the maximum lateral force, $R=4.5$ kips. Considered to be secondary members, the struts are always in tension.



SECTIONS AT STRUT

The area required for a strut is

$$A=\frac{R}{F_y}=\frac{4.5}{36}=0.125 \text{ in.}^2$$

For the struts as secondary tension members, the largest permissible slenderness ratio $L/r=240$.

Try a $3\times2\frac{1}{2}\times\frac{1}{4}$-in. angle.

$$A=1.31>0.125 \text{ in.}^2$$

$$\frac{L}{r}=\frac{114}{0.528}=216<240$$

The angle is satisfactory.

FLANGE-TO-WEB WELDS

The flanges are fillet welded to the webs on both sides of each web. Each pair of welds must resist the horizontal shear at the interface of the web and flange. The welds are checked at the end bearing and near the interior support, where the vertical shear is largest. The horizontal shear flow S, kips per lin in., may be computed from

$$S=\frac{VQ}{I}$$

where V = vertical shear, kips
I = moment of inertia of girder, in.^{4}
Q = moment of flange area about centroidal axis of girder, in.^{3}

Load factor design specifies a maximum strength F_v of $0.45F_u$ for fillet welds, where F_u is the specified minimum tensile strength of the welding rod. But F_u may not exceed the tensile strength of the connected parts. Fillet welds are designed for F_v under the Maximum Design Load. Calculations indicate that the size of the weld is governed by the thickness of the flange, rather than by stress.

With $F_u=58$ ksi for A36 steel, the design relationship for strength of a fillet weld is

$$f_v \text{ due to } 1.30\left[D+\frac{5}{3}(L+I)\right]\leq(0.45\times58\times0.707=18.5 \text{ ksi})$$

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

Investigation of the welds at the end bearing begins with a tabulation of the vertical shears and calculation of horizontal shear flow, kips per lin in. of weld width.

Service-Load Shears at End Support

	DL_1	DL_2	$+(L+I)$	$-(L+I)$
V, kips	56.7	14.5	68.3	−6.8

Section Properties at End Support

Steel Section Only

$$I=48{,}813 \text{ in.}^4$$

Top Flg.: $Q=\frac{9}{16}\times12\times33.09=223 \text{ in.}^3$

Bot. Flg.: $Q=\frac{1}{2}\times\frac{5}{16}\times92\times24.34=350 \text{ in.}^3$

Composite Section, $n=8$

$$I=148{,}475 \text{ in.}^4$$

Top Flg.: $Q=\frac{9}{16}\times12\times8.34 \quad = \quad 56$

Conc.: $Q=\frac{1}{2}\times\frac{180}{8}\times7.5\times14.81=\underline{1{,}250}$

$1{,}306 \text{ in.}^3$

Bot. Flg.: $Q=\frac{1}{2}\times\frac{5}{16}\times92\times49.07 \quad = \quad 705 \text{ in.}^3$

Shear Flow $S=\frac{VQ}{I}$ Due to Maximum Design Loads

	Top Weld		Bottom Weld	
For DL_1:	$S=\frac{56.7\times 223}{48,813}\times 1.30$	$=0.337$	$S=\frac{56.7\times 350}{48,813}\times 1.30$	$=0.529$
For DL_2:	$S=\frac{14.5\times 1,306}{148,475}\times 1.30$	$=0.165$	$S=\frac{14.5\times 705}{148,475}\times 1.30$	$=0.090$
For $L+I$:	$S=\frac{68.3\times 1,306}{148,475}\times 1.30\times\frac{5}{3}$	$=1.302$	$S=\frac{68.3\times 705}{148,475}\times 1.30\times\frac{5}{3}$	$=0.702$
		1.804 kips per in.		1.321 kips per in.

The AASHTO Specifications require that the web be fully developed by the flange-to-web weld, to insure adequate fatigue resistance with respect to transverse distortional stresses. The provisions therefore state that the total effective thickness (based on the throat dimension in the case of fillet welds) must be at least equal to the web thickness.

Shear in the top weld governs. For two welds, the shear flow in each weld is 1.804/2 $=0.902$ kips per in.

$$\text{Weld size required}=\frac{0.902}{18.5}=0.049\text{ in.}$$

This, however, is less than the minimum weld size required by either the thickness of flange or thickness of web. The weld size required by thickness of flange, AASHTO 1.7.21(B)

$=\frac{1}{4}$ in.

The weld size required by thickness of web, AASHTO 1.7.49(E)

$$=\frac{\text{web thickness}}{0.707\times 2}=\frac{\frac{1}{2}}{1.414}=\frac{3}{8}\text{ in.}-\text{governs}$$

Next, the flange-to-web welds are designed at the transition 2 ft from the interior support in the same manner as at the end bearing.

Service-Load Shears 2 Ft from the Interior Support

	DL_1	DL_2	$L+I$
V, kips	106	26	72

Section Properties 2 Ft from Interior Support

Steel Section Only

Top Flg.: $Q=2\times 15\times 33.04 \quad = 991$ in.^{3}

Bot. Pl.: $Q=\frac{1}{2}\times\frac{7}{8}\times 92\times 25.40=1,022$

Stiff.: $Q=\frac{1}{2}\times 7.35\times 19.71 \quad = \underline{72}$

1,094 in.^{3}

Steel Plus Reinforcing

Top Flg.: $Q=2\times 15\times 30.40 \quad = 912$

Reinf.: $Q=\frac{1}{2}\times 15.19\times 35.93 \quad = \underline{273}$

1,185 in.^{3}

Bot. Pl.: $Q=\frac{1}{2}\times\frac{7}{8}\times 92\times 28.04=1,129$

Stiff.: $Q=\frac{1}{2}\times 7.35\times 22.35 \quad = \underline{82}$

1,211 in.^{3}

Shear Flow $S=\frac{VQ}{I}$ Due to Maximum Design Loads

	Top Weld		Bottom Weld	
For DL_1:	$S=\frac{106\times 991}{136{,}973}\times 1.30$	$=0.997$	$S=\frac{106\times 1{,}094}{136{,}973}\times 1.30$	$=1.101$
For DL_2:	$S=\frac{26\times 1{,}185}{158{,}022}\times 1.30$	$=0.253$	$S=\frac{26\times 1{,}211}{158{,}022}\times 1.30$	$=0.259$
For $L+I$:	$S=\frac{72\times 1{,}185}{158{,}022}\times 1.30\times\frac{5}{3}$	$=1.170$	$S=\frac{72\times 1{,}211}{158{,}022}\times 1.30\times\frac{5}{3}$	$=1.195$
		2.420 kips per in.		2.555 kips per in.

Shear in the bottom weld governs. For two welds, the shear flow in each weld is $2.555/2=1.278$ kips per in.

$$\text{Weld size required}=\frac{1.278}{18.5}=0.069\text{ in.}$$

Again the weld size is governed by the ½-in. web thickness rather than by stress or by flange thickness. A ⅜-in. web-to-flange weld is required throughout the length of the box girder.

Shear-Stress Range in Bottom Weld Due to Service Loads

The maximum shear range at the transition 2 ft from the interior support equals

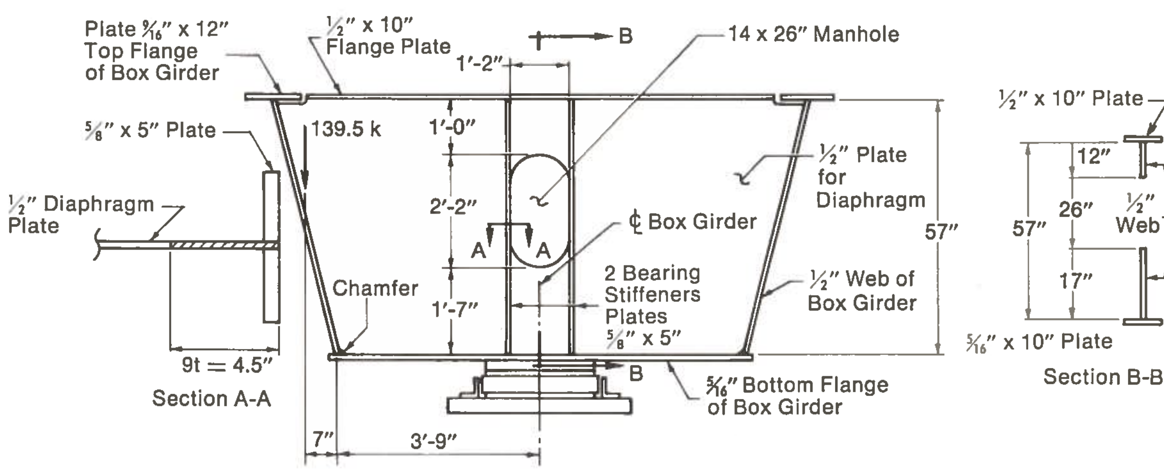
$$S_r=\frac{72\times 1{,}211}{158{,}022}=0.552\text{ kips per in.}$$

The actual stress range in the ⅜-in. fillet weld equals

$$f_{vr}=\frac{0.552}{2\times 0.707\times \frac{3}{8}}=1.04<12\text{ ksi}$$

END DIAPHRAGM AND EXPANSION DAM

At the end support, each box girder is supported at the center of the bottom flange on a single bearing. An end diaphragm is required at this support to retain the shape of the cross section and to transfer the loads on the girder into the bearing. A ½-in. plate is used for the web and a ½×10-in. plate as the top flange of the diaphragm. The bottom flange of the box girder serves as the bottom flange of the diaphragm.



SECTIONS AT END DIAPHRAGM

Design of Bearing Stiffeners at End Support

For access to the bridge-seat area between the diaphragm and the backwall of the abutment, a 14×26-in. screen-covered manhole is provided in the center of the diaphragm. The manhole is flanked by two bearing stiffeners. These stiffeners are designed as columns in accordance with load-factor-design provisions for compression members.

Assume that each stiffener consists of two 5-in.-wide plates welded to opposite sides of the diaphragm web. The minimum thickness required by width-thickness-ratio limitations for a stiffener is

$$t=\frac{b'}{12}\sqrt{\frac{F_y}{33{,}000}}=\frac{5}{12}\sqrt{\frac{36{,}000}{33{,}000}}=0.435 \text{ in.}$$

End Reactions

	DL_1	DL_2	$L+I$	Total
V, kips	56.7	14.5	68.3	139.5

The allowable bearing stress for a stiffener under service loads is 29 ksi. The minimum stiffener thickness required for bearing then is

$$t=\frac{139.5/2}{29(5-0.25-0.25)}=0.534>0.435 \text{ in.}$$

Try two ⅝×5-in. plates for each bearing stiffener. (See Section A-A of Section at End Diaphragm.)

The stiffener column consists of the two ⅝×5-in. plates plus a length of web equal to

$$L_w=9t_w=9\times \tfrac{1}{2}=4.5 \text{ in.}$$

Area of the equivalent column is

$$A_s=2\times \tfrac{5}{8}\times 5+\tfrac{1}{2}\times 4.5=8.5 \text{ in.}$$

Moment of inertia of the equivalent column is

$$I_s=\frac{(\tfrac{5}{8})(5+0.5+5)^3}{12}+60.3 \text{ in.}^4$$

and the radius of gyration is

$$r=\sqrt{\frac{I}{A}}=\sqrt{\frac{60.3}{8.5}}=2.66 \text{ in.}$$

Consequently, the slenderness ratio of the stiffener equals

$$\frac{KL_c}{r}=\frac{D}{r}=\frac{57}{2.66}=21.4$$

$$\sqrt{\frac{2\pi^2E}{F_y}}=\sqrt{\frac{2\pi^2\times 29{,}000}{36}}=126>21.4$$

The allowable stress then is

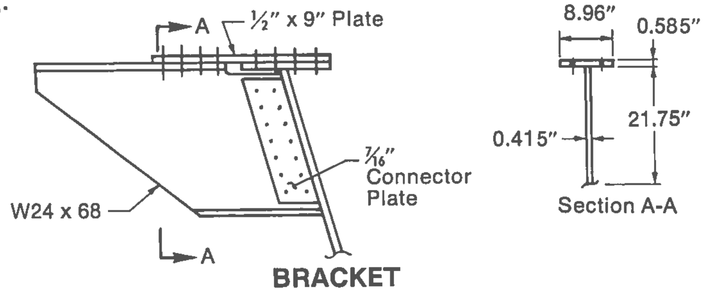
$$F_{cr}=F_y\left[1-\frac{F_y}{4\pi^2E}\left(\frac{D}{r}\right)^2\right]=36\left[1-\frac{36}{4\pi^2\times 29{,}000}\left(\frac{57}{2.66}\right)^2\right]=35.5 \text{ ksi}$$

The Maximum Design Load on the columns is

$$V_u=1.3\left(56.7+14.5+\frac{5}{3}\times 68.3\right)=241 \text{ kips}$$

Bracket Design

The maximum bending stress in the bracket, which is connected to the outer web of the box girder, occurs at Section A-A through the bracket where the bracket stem is 21.75 in. long.



In calculation of section properties for design of the bracket, neglect the area of web holes and deduct the area of flange holes exceeding 15% of the top-flange area.

$$\text{Flange area } A_f = 0.585 \times 8.96 = 5.21 \text{ in.}^2$$

$$15\%\ A_f = 0.15 \times 5.24 = 0.79 \text{ in.}^2$$

$$\text{Area of two flange holes} = 2 \times 1 \times 0.585 = 1.170 \text{ in.}^2$$

$$\text{Areas of holes over } 15\%\ A_f = 1.17 - 0.79 = 0.38 \text{ in.}^2$$

Properties of Section A-A Through Bracket

Material	A	d	Ad	Ad^2	I_o	I
Top Flg. 0.585×8.96	5.24	11.17	58.5	654		654
Flg. Holes	−0.38	11.17	−4.2	−47		−47
Stem 0.415×21.75	9.03				356	356
	13.89 in.²		54.3 in.³			963

$$d_s = \frac{54.0}{13.88} = 3.89 \text{ in.}$$

$$-3.91 \times 54.3 = \quad -212$$

$$I_{NA} = \quad 751 \text{ in.}^4$$

$$d_{\text{Bot.}} = \frac{21.75}{2} + 3.91 = 14.79 \text{ in.}$$

$$S_{\text{Bot.}} = \frac{751}{14.79} = 50.8 \text{ in.}^3$$

If dead load on the bracket is neglected, the maximum-design-load moment is

$$M = 1.30 \times \frac{5}{3} \times 65.9 = 142.8 \text{ kip-ft}$$

The bending stress in the stem under maximum design load is

$$f_b = \frac{142.8 \times 12}{50.8} = 33.7 < 36 \text{ ksi}$$

The W18×35 bracket is satisfactory for bending.

Bracket Web Connection

Try two vertical rows of seven ⅞-in.-dia A325 bolts in the bracket web, 3 in. c. to c., and two horizontal rows of four ⅞-in.-dia bolts in the bracket flange. Thus, there are a total of 22 bolts. The location y of the horizontal axis of the 22 bolts with respect to the horizontal axis of the web bolts is calculated as follows, noting that the sum of the moments of the web bolts about their axis equals zero:

$$y = \frac{8 \times 11.875}{22} = 4.32 \text{ in.}$$

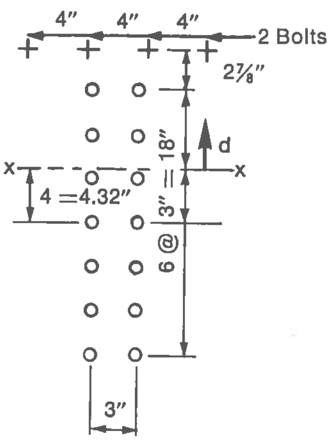
The 14 bolts in the bracket-web connection must carry the vertical shear. All 22 bolts are subjected to the bracket maximum bending moment. The forces imposed on the bolts are produced by the Overload $5/3(L+I)$, DL being ignored.

$$\text{Shear}=5/3\times 20.8=\ 34.7 \text{ kips}$$

$$\text{Moment}=5/3\times 65.9=109.8 \text{ kip-ft}$$

Moments of Inertia of Bolts

BOLT HOLES IN BRACKET



d^2 for Bolts

$(1.68)^2=$	2.8
$(4.68)^2=$	21.9
$(1.32)^2=$	1.7
$(4.32)^2=$	18.7
$(7.32)^2=$	53.6
$(10.32)^2=$	106.5
$(13.32)^2=$	177.4
$\frac{1}{2}\Sigma=$	382.6
Web $\Sigma d^2=$	765.2
Flange	
$8(7.56)^2=$	457.2
$I_x=\Sigma d^2=$	1,222.4

$$I_y\approx 14(1.5)^2+4[(2)^2+(6)^2]=192$$

The polar moment of inertia of the bracket bolts is

$$I=I_x+I_y=1{,}222+192=1{,}414$$

The distance from the centroid to the outermost bolt is

$$d=\sqrt{(12+4.32)^2+(2)^2}=16.44 \text{ in.}$$

The load per bolt due to shear is

$$P_s=\frac{34.7}{14}=2.5 \text{ kips}$$

The load on the outermost bolt due to moment is

$$P_m=\frac{109.8\times 12\times 16.44}{1{,}414}=15.32 \text{ kips}$$

The vertical component of this load is

$$P_v=\frac{15.32\times 1.5}{16.44}=1.4 \text{ kips}$$

The horizontal component is

$$P_h=\frac{15.32(9+4.32)}{16.44}=12.4 \text{ kips}$$

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

$$P=\sqrt{(2.5+1.4)^2+(12.4)^2}=13.0\approx 12.6 \text{ kips}$$

Use fourteen ⅞-in.-dia bolts in the web in two rows and 8 bolts in the flange in two rows.

Bracket Flange Connection

The force due to bending on the flange bolts is

$$P=\frac{109.8\times 12\times 7.56}{1{,}414}=7.04 \text{ kips per bolt}<12.6 \text{ kips}$$

Top Splice Plate on Bracket

The average top-flange bending stress under maximum design load is

$$F_b = \frac{142.8 \times 12(11.17-3.89)}{750} = 16.6 \text{ ksi}$$

The force in the top flange is

$$P = F_b A_f = 16.6 \times 5.21 = 86.5 \text{ kips}$$

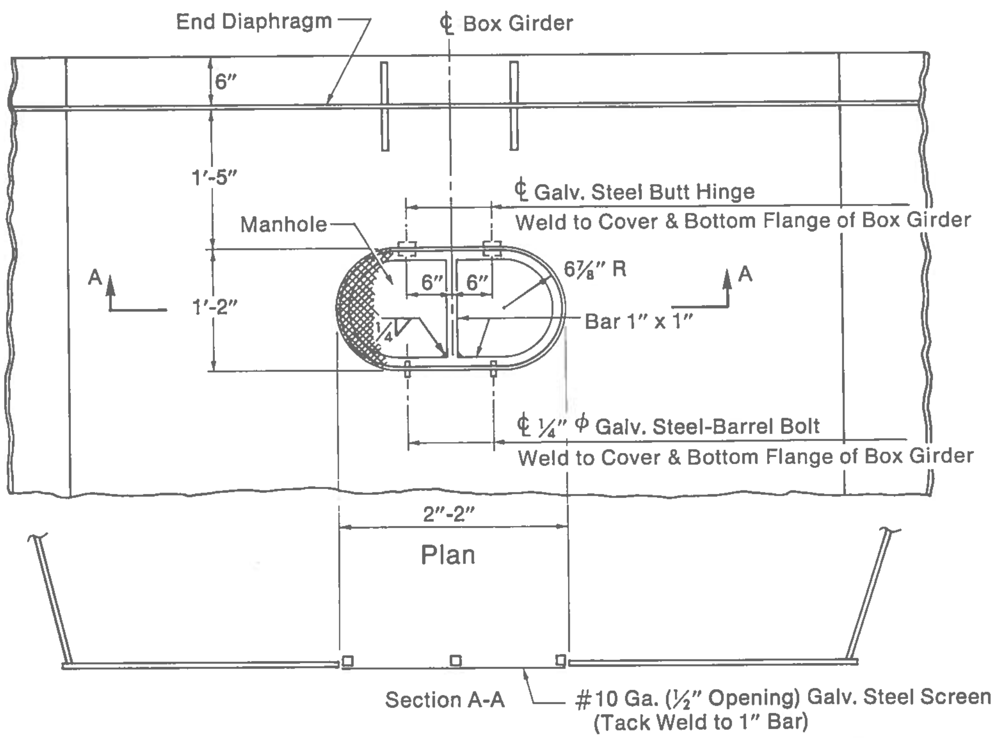
The required area of splice plate therefore is

$$A = \frac{86.5}{36} = 2.40 \text{ in.}^2$$

Use Pl. ½×9 in. Net area$=0.5(9-2\times1)=3.50>2.40$ in.^{2}

BOX-GIRDER ACCESS MANHOLE

A manhole is provided near the end bearing for access to the box-girder interior. Because the stress is small in the bottom flange, the manhole is placed in that flange rather than in the web, which is subject to high shearing stress. With the manhole close to the end bearing, the bottom flange needs no reinforcement around the hole. Details of the manhole are shown in horizontal and vertical box-girder sections.



HORIZONTAL AND VERTICAL GIRDER SECTIONS AT MANHOLE

WEB STIFFENERS

The distance, d_o, of the first stiffener from the end bearing is governed either by D, the depth of the web, or by the formula

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

For this girder the web depth, D, is 58.69 inches and

$$d_o = 14{,}500\sqrt{\frac{58.69(\frac{1}{2})^3}{248{,}000}} = 78.8 \text{ in.} > 58.69 \text{ in.}$$

Therefore, one stiffener is placed 58 in. from the end bearing.

Calculations indicate that stiffeners are required over almost all of the negative-moment region. If the actual shear exceeds 60% of the shear capacity, the actual moment is limited by

$$\frac{M}{M_u} \leq 1.375 - 0.625\frac{V}{V_u}$$

A reasonable way of spacing the web stiffeners in the negative-moment region is to use the maximum possible spacing that does not reduce the moment capacity of the box girder.

The first stiffener space adjacent to the pier is measured from the cross-girder web, which is 18 in. from the centerline of the pier. The bending moment in the box girder at the cross girder is much smaller than the moment capacity of the box girder, because the bottom-flange thickness is increased for a biaxial state of stress.

At the bottom-flange transition of the hybrid section 2 ft from the pier, however, the moment capacity of the section is much closer to the design moment. A 57-in. stiffener spacing is selected and checked to insure that the ratio of design moment M to the moment capacity M_u is satisfactory.

At the flange transition, the Maximum-Design-Load shear along the sloped web is 335 kips. For calculation of the web shear capacity V_u with the stiffener spacing $d_o = 57$ in.,

$$C = 18{,}000\frac{t_w}{D}\sqrt{\frac{1+(D/d_o)^2}{F_y}} - 0.3 = 18{,}000 \times \frac{1/2}{58.69}\sqrt{\frac{1+(58.69/57)^2}{36{,}000}} - 0.3 = 0.860$$

$$V_p = 0.58F_yDt_w = 0.58 \times 36 \times 58.69 \times 1/2 = 613 \text{ kips}$$

For a hybrid girder,

$$V_u = V_pC = 613 \times 0.860 = 527 \text{ kips}$$

$$\frac{V}{V_u} = \frac{335}{527} = 0.636 > 0.6$$

Therefore, the maximum permissible value of M/M_u is

$$\frac{M}{M_u} = 1.375 - 0.625\frac{V}{V_u} = 1.375 - 0.625 \times 0.636 = 0.978$$

Previous calculations of the flange stress at the transition determined that under the Maximum Design Load the stress in the top flange is 42.2 ksi and in the bottom flange 34.9 ksi. The design strength in tension F_u is 49.2 ksi and the critical buckling stress F_{cr} is 36.8 ksi.

The actual value of M/M_u is .

Top Flange

$$\frac{M}{M_u} = \frac{F_b}{F_u} = \frac{42.2}{49.2} = 0.858 < 0.978$$

Bottom Flange

$$\frac{F_b}{F_{cr}} = \frac{34.9}{36.8} = 0.948 < 0.978$$

The 57-in. stiffener spacing is satisfactory at the flange transition 2 ft from the interior support.

Stiffeners are placed on opposite sides of the field splice at a distance of 12 in. from the centerline of the splice. Try four stiffeners spaced at 71.4 in. c. to c. between the stiffener adjacent to the field splice and the stiffener 57 in. from the cross-girder web, or 6.25 ft from the pier. A check of the stiffener spacing starts with the second stiffener space from the pier.

At 6.25 ft from the pier, the Maximum Design Shear is 311 kips.

Bending Moments 6.25 Ft from Interior Support

	DL_1	DL_2	$-(L+I)$
M, kip-ft	−5,000	−1,120	−2,570

Steel stresses 6.25 ft from the interior support are computed with the section moduli calculated for the section 2 ft from that support.

Steel Stresses 6.25 Ft from Interior Support Due to Maximum Design Loads

	Top of Steel		Bottom of Steel	
For DL_1:	$F_b=\frac{5{,}000\times12}{4{,}024}\times1.30$	$=19.4$	$F_b=\frac{5{,}000\times12}{5{,}301}\times1.30$	$=14.7$
For DL_2:	$F_b=\frac{1{,}120\times12}{5{,}033}\times1.30$	$=3.5$	$F_b=\frac{1{,}120\times12}{5{,}549}\times1.30$	$=3.1$
For $L+I$:	$F_b=\frac{2{,}570\times12}{5{,}033}\times1.30\times\frac{5}{3}$	$=13.3$	$F_b=\frac{2{,}570\times12}{5{,}549}\times1.30\times\frac{5}{3}$	$=12.0$
		36.2 ksi		29.8 ksi

For calculation of the shear capacity V_u of the web, with $d_o=71.4$ in. and $V_p=$ 613 kips.

$$C=18{,}000\times\frac{\frac{1}{2}}{58.69}\sqrt{\frac{1+(58.69/71.4)^2}{36{,}000}}-0.3=0.746$$

$$V_u=V_pC=613\times0.746=457 \text{ kips}$$

$$\frac{V}{V_u}=\frac{311}{457}=0.681>0.6$$

Therefore, the maximum permissible value of M/M_u is

$$\frac{M}{M_u}=1.375-0.625\times0.681=0.949$$

The actual value of M/M_u is

Top Flange

$$\frac{M}{M_u}=\frac{F_b}{F_u}=\frac{36.2}{49.2}=0.736<0.949$$

Bottom Flange

$$\frac{F_b}{F_{cr}}=\frac{29.8}{36.8}=0.810<0.949$$

For the second stiffener space from the pier, 71.4 in. is satisfactory.

Another check is made of the stiffener space measured from the transition section 17 ft from the interior support. Calculations show that with the 71.4-in. spacing, the shear is less than 60% of the shear capacity. Hence, no reduction in moment capacity is required. Thus, the spacing of 71.4 in. is satisfactory at the transition.

From previous design calculations for this transition section, the maximum design shear is 268 kips and $V_u=457$ kips.

$$\frac{V}{V_u}=\frac{268}{457}=0.586<0.6$$

Therefore, moment capacity need not be reduced.

On the basis of the preceding calculations, the stiffener spacing in the negative-moment region is set as follows:

1. First stiffener: 75 in. from interior support.
2. Next four stiffeners: Equally spaced at 71.4 in.
3. Sixth and seventh stiffeners: 12 in. on either side of the field splice centerline.

Design of Intermediate Stiffeners

The web stiffeners are attached to the inside face of the web. They must satisfy requirements for minimum area and moment of inertia, as described in the Introduction of this chapter.

A plate $\frac{3}{8}\times5$ in. is selected for all the stiffeners. The stiffener adjacent to the end bearing is located at a section that is subjected to larger shears than the sections at the other stiffeners. Therefore, it is checked, and because it is satisfactory, the other stiffeners also are.

For calculation of the shear capacity of the web 58 in. from the end bearing, with $V_p = 613$ kips,

$$C = 18{,}000 \times \frac{1/2}{58.69}\sqrt{\frac{1+(58.69/58)^2}{36{,}000}} - 0.3 = 0.850$$

$$V_u = V_p\left[C + \frac{0.87(1-C)}{\sqrt{1+(d_o/D)^2}}\right] = 613\left[0.850 + \frac{0.87(1-0.850)}{\sqrt{1+(58/58.69)^2}}\right] = 578 \text{ kips}$$

The area of the stiffener should be at least

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

where $B = 2.4$ for a single-plate stiffener

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

$$A = \frac{36}{36}\left[0.15 \times 2.4 \times 58.69 \times \frac{1}{2}(1-0.850)\frac{248}{578} - 18\left(\frac{1}{2}\right)^2\right] = -3.82$$

The negative result indicates that the web contribution, $18t_w^2$, is more than enough in itself to satisfy the area requirement.

The width-thickness ratio b'/t of the stiffener plate is $5/(3/8) = 13.3$. The permissible maximum ratio is

$$\frac{b'}{t} = \frac{2{,}600}{\sqrt{F_y}} = \frac{2{,}600}{\sqrt{36{,}000}} = 13.7 > 13.3$$

The moment of inertia of the stiffener plate about the edge connected to the web is

$$I = \frac{td^3}{3} = \frac{(3/8)(5)^3}{3} = 15.6 \text{ in.}^4$$

The minimum moment of inertia required is computed as follows:

$$J = 2.5(D/d_o)^2 - 2 = 2.5(58.69/58.0)^2 - 2 = 0.56$$

$$I = d_o t_w^3 J = 58(1/2)^3 0.56 = 4.06 < 15.6 \text{ in.}^4$$

The $3/8 \times 5$-in. plate also satisfies width-to-thickness and moment of inertia requirements.

SHEAR CONNECTORS

A 7/8-in.-dia, 5-in.-high, welded stud is placed on each side of the web at intervals along both top flanges of the box girder, to serve as a shear connector between the flanges and the concrete slab. The shear-connector spacing is calculated in exactly the same manner as for the composite wide-flange beam of Chapter 3A. The spacing of the connectors is governed by fatigue under service loads in the positive-moment regions. Maximum spacing is 24 in. in the negative-moment region.

Allowable stud loads are determined for a ratio of stud height H to diameter d greater than 4.

$$\frac{H}{d} = \frac{5}{0.875} = 5.7 > 4$$

For $H/d > 4$, AASHTO Specifications give the ultimate strength of a shear connector as

$$S_u = 0.4d^2\sqrt{f'_c E_c}$$

where E_c = modulus of elasticity of concrete = $57{,}000\sqrt{f'_c}$

f'_c = 28-day strength of concrete, psi = 4,000 psi

$S_u = 0.4d^2\sqrt{57{,}000}(f'_c)^{3/4} = 0.4(7/8)^2\sqrt{57{,}000}(4{,}000)^{3/4} = 36{,}800$ psi

With α given in AASHTO Specifications as 10.6 for 500,000 cycles of load, the load range per shear connector is

$$Z_r = \alpha d^2 = 10.6(7/8)^2 = 8.11 \text{ kips per stud}$$

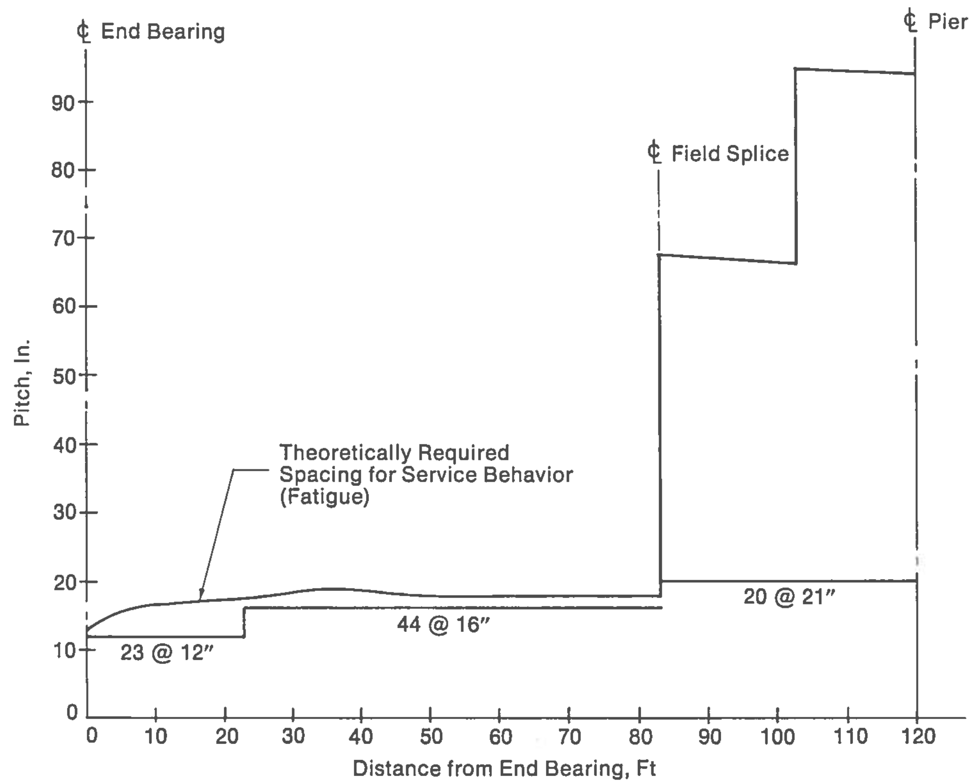
Shear-Connector Spacing for Service Behavior (Fatigue)

Shear-connector spacing is computed initially at the section at the end bearing. A similar computation is made for each tenth point of the span, and a curve of theoretical spacing for service behavior is plotted. (This curve is relatively flat. As a result, it is not necessary to calculate the spacing at every point.) The actual, stepped, shear-connector spacing diagram is drawn below the theoretical curve.

At the end support, the shear range $V_r = 68.3 + 6.8 = 75.1$ kips per web. Section properties for computation of the shear per inch S_r between the concrete slab and the top flanges of the girder at the end support are the same as the section properties determined previously for the transition section 25 ft from the end support, with $n = 8$. The area of the effective concrete slab is 168.75 in.^{2} The distance from the centroidal axis of the girder to the centroidal axis of the slab is $35.25 - 20.44 = 14.81$ in. The moment of inertia of the girder $I = 148{,}475$ in.^{4} Hence,

$$S_r = \frac{V_r Q}{I} = \frac{75.1 \times 168.75 \times 14.81}{148{,}475} = 1.26 \text{ kips per in.}$$

$$\text{Spacing required (4 studs, 2 webs)} = \frac{4 \times 8.11}{2 \times 1.26} = 12.9 \text{ in. Use 12 in.}$$



SHEAR-CONNECTOR SPACING

Shear Connectors—Strength Requirements

The number of connectors provided for fatigue is checked to insure that adequate connectors are provided for ultimate strength. The number of connectors between the point of maximum positive moment and the end support must equal or exceed

$$N_1 = \frac{P}{0.85 S_u}$$

where P is the smaller of the following two forces computed at the point of maximum moment, 48 ft from the end support:

$$P_1 = A_s F_y = 112.44 \times 36 = 4{,}048 \text{ kips}$$

$$P_2 = 0.85 f'_c bc = 0.85 \times 4 \times 180 \times 7.5 = 4{,}590 > 4{,}048 \text{ kips}$$

P_1 governs. Thus, the number of connectors required is

$$N_1 = \frac{4{,}048}{0.85 \times 36.8} = 129.4 \text{ or } \frac{129.4}{4} = 33 \text{ rows}$$

Service-Load design provides 42 rows of shear connectors between the end support and the maximum-positive-moment section. The ultimate-strength requirement for this region therefore is satisfied.

The number of connectors required between the point of maximum positive moment and the interior support is determined for the section at the interior support from

$$N_2 = \frac{P + P_3}{0.85 S_u}$$

where $P_3 = A_s^r F_y^r = 15.19 \times 40 = 608$ kips

Therefore, the number of connectors required is

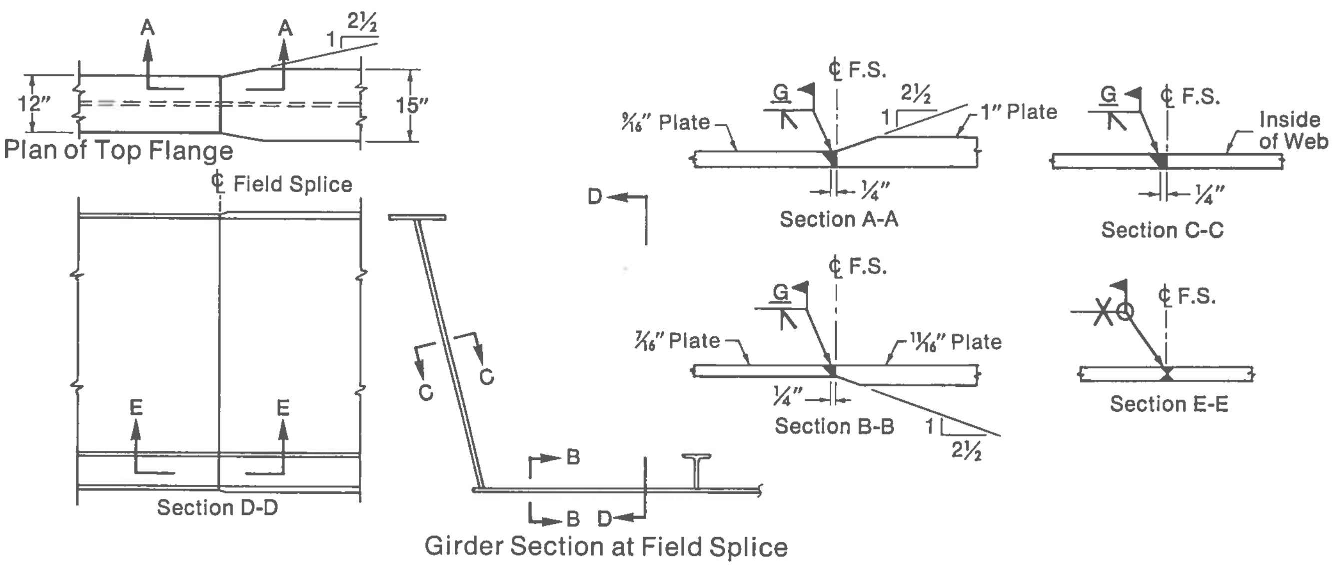
$$N_2 = \frac{4{,}048 + 608}{0.85 \times 36.8} = 148.8 \text{ or } \frac{148.8}{4} = 38 \text{ rows}$$

The number of rows furnished for Service Loads between the maximum-positive-moment section and the interior support is 47. This number satisfies strength requirements.

WELDED FIELD SPLICE

A field splice is placed at the inflection point of each span, 37 ft from the interior support. The splice is made with full-penetration butt welds. Because there is a thickness change in both the bottom and top flanges, fatigue restrictions for base metal adjacent to a butt weld must be satisfied. The condition of fatigue in base metal adjacent to a fillet weld, previously investigated at the cut-off of the longitudinal bottom-flange stiffener, however, is more severe and the section was found to be satisfactory for that condition. Hence, no further investigation for fatigue is necessary at the section 37 ft from the interior support.

The change in width of the top-flange is made in accordance with the taper required by Art. 1.7.10 of the AASHTO Specifications. Details of the welded splice are illustrated.



WELDED FIELD SPLICE

BOLTED FIELD SPLICE

For Load-Factor design of a bolted field splice, AASHTO Specifications require that the splice material be proportioned for the Maximum Design Load and resistance to fatigue under Service Loads. Because friction connections must resist slip under Overload, fastener size must be selected for an allowable stress $F_v = 21$ ksi under the Overload of $D+5/3(L+I)$.

The allowable load for a ⅞-in.-dia., A325 bolt in double shear is

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

For design of the splice material for the Maximum Design Load, the design moment is chosen 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 loss in flange area due to bolt holes with area exceeding 15% of each flange area.

Bending Moments 37 Ft from Interior Support, Kip-Ft

	For Service Loads	Factor	For Overload	Factor	Maximum Design Loads
DL_1	−100	1	−100	1.30	−130
DL_2	50	1	50	1.30	65
$+(L+I)$	1,690	$\frac{5}{3}$	2,817	1.30	3,662
$-(L+I)$	−1,050				
Maximum	1,640		2,767		3,597
Minimum	−1,100				

Shears 37 Ft from Interior Support, Kips

	For Service Loads	Factor	For Overload	Factor	Maximum Design Loads
DL_1	58.0	1	58.0	1.30	75.4
DL_2	13.8	1	13.8	1.30	17.9
$L+I$	47.8	$\frac{5}{3}$	79.7	1.30	103.6
Total					196.9

The section at the splice is subject to the following moments:

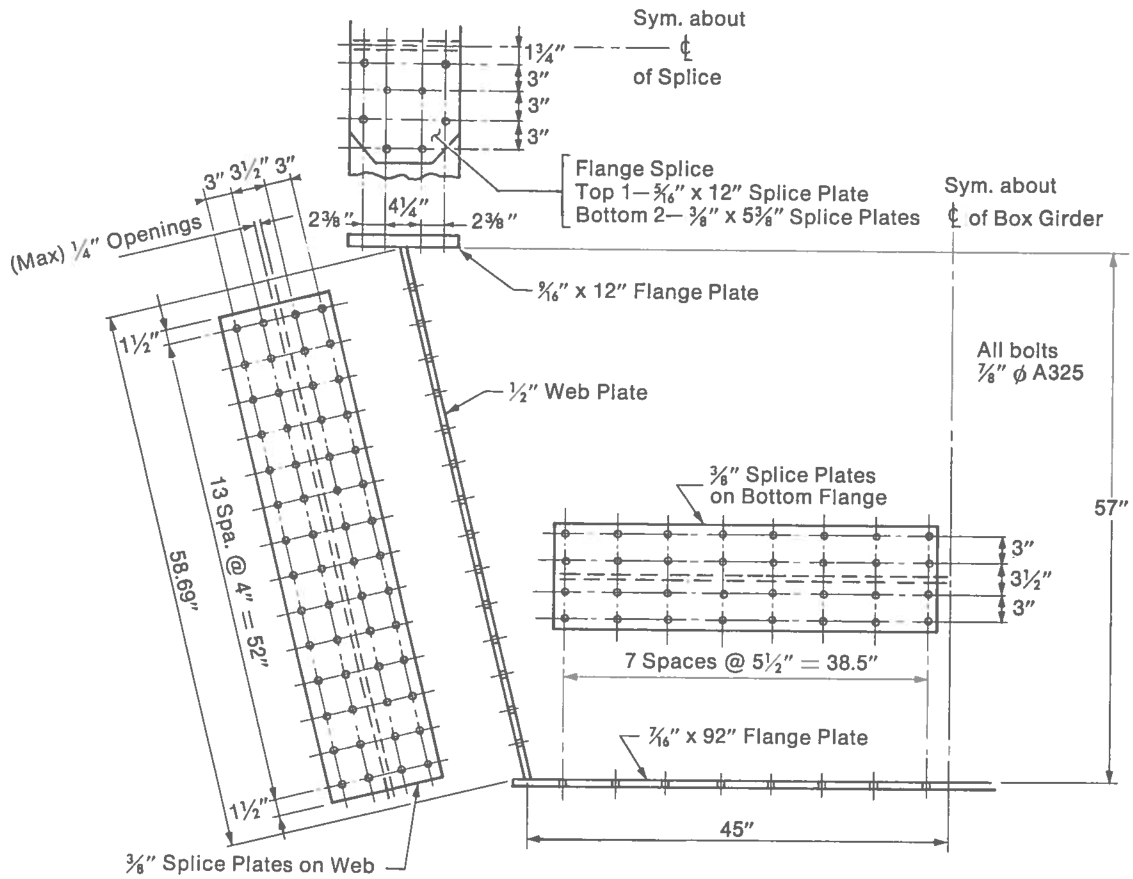
Negative moment that acts only on the steel section.

Positive moment that acts on the composite steel-concrete section.

Negative moment resisted by the steel section and the concrete reinforcement.

Because the effects of positive moment dominate at the splice, splice material is designed for positive moment. Also, to simplify the design procedure, the composite concrete slab is neglected.

Net section properties at the splice are those for the smaller section, on the positive-moment side of the splice.

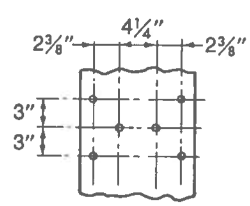


BOLTED FIELD SPLICE

Net Section at Top-Flange Splice

The splice of each top flange is made with ⅞-in.-dia, A325 bolts, arranged staggered in four rows. Pitch of the bolts longitudinally is $s=3$ in. Gage $g=2\frac{3}{8}$ in.

$$\frac{s^2}{4g}=\frac{(3)^2}{4\times 2.375}=0.947$$



BOLT HOLES IN TOP FLANGE

The deduction from the flange width at the section across the flange through two holes equals $2\times 1=2.00$ in.

The deduction from the flange width at a section through a chain of four holes equals $4\times 1-2\times 0.947=2.106>2.00$ in. Use 2.106 in. for the deduction in computing the net flange area.

Flange Area and Deductions

Gross Area $=\frac{9}{16}\times 12=6.75$ in.^{2}

$$\begin{aligned}
\text{Area deducted for bolt holes} &= \tfrac{9}{16}\times 2.106 = 1.18\\
-15\%\text{ of gross area} &= -0.15\times 6.75 = \underline{-1.01}\\
\text{Net deduction for two flanges} &= 0.17\times 2=0.34\text{ in.}^2
\end{aligned}$$

Net Section at Bottom Flange and Stiffener Splices

Assume that the center of gravity of the stiffener coincides with the center of gravity of the bolt holes. Deduct the following areas: 16 holes in the bottom-flange plate, two holes from the flange of the stiffener and two holes from the stiffener stem.

Flange Area and Deductions

$$\text{Gross area of bottom flange and stiffener} = \tfrac{7}{16} \times 92 + 7.35 = 47.60 \text{ in.}^2$$

$$\text{Area deducted for bolt holes} = \tfrac{7}{16} \times 16 + 2 \times 0.622 + 2 \times 0.55 = \quad 9.34$$

$$-15\% \text{ of gross area} = -0.15 \times 47.60 \qquad = \underline{-7.14}$$

$$\text{Net deduction for bottom flange and stiffener} = \quad 2.20 \text{ in.}^2$$

Properties of the gross cross section of the box girder are obtained from previous calculations for the section 37 ft from the interior support. The bolt holes in the flanges are deducted in the computation of properties of the net section.

Net Section at the Splice—Steel Section Only

Material	A	d	Ad	Ad^2	I_o	I
Gross Section	119.79		−938			64,287
Top Flg. Bolt Holes	−0.34	28.78	− 10	− 282		− 282
Bot. Flg. Bolt Holes	−2.20		63	−1,815		−1,815
	117.25 in.²		−885 in.³			62,190

$$d_s = \frac{-885}{117.25} = -7.55 \text{ in.}$$

$$-7.55 \times 885 = \underline{-6,682}$$

$$I_{NA} = 55,508 \text{ in.}^4$$

$$d_{\text{Top of steel}} = 29.06 + 7.55 = 36.61 \qquad d_{\text{Bot. of steel}} = 28.94 - 7.55 = 21.39 \text{ in.}$$

$$S_{\text{Top of steel}} = \frac{55,508}{36.61} = 1,516 \text{ in.}^3 \qquad S_{\text{Bot. of steel}} = \frac{55,508}{21.39} = 2,595 \text{ in.}^3$$

Design Moments and Shears at the Field Splice

The capacity of the net section is based on the minimum section modulus of the steel section. For $F_y = 36$ ksi, the net section capacity is

$$M_{net} = \frac{36 \times 1,516}{12} = 4,548 \text{ kip-ft}$$

$$75\% \ M_{net} = 0.75 \times 4,548 = 3,411 \text{ kip-ft}$$

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

$$M_{av} = \frac{3,597 + 4,548}{2} = 4,073 > 3,411 \text{ kip-ft}$$

The design moment, therefore, is 4,073 kip-ft.

The design shear is determined by multiplying the calculated shear for the design loads, 196.9 kips, by the ratio of the design moment to the calculated moment on the section, 3,597 kip-ft.

$$\text{Design Shear} = 196.9 \times \frac{4,073}{3,597} = 233 \text{ kips}$$

In the plane of each web,

$$\text{Design Shear} = \frac{223}{2} \times \frac{58.69}{57} = 115 \text{ kips}$$

Web Splice

The web splice plates must carry the design shear, a moment M_v due to the eccentricity of the 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. The gross moment of inertia is obtained from the earlier calculation of section properties 37 ft from the interior support and adjusted for the change in position of the centroidal axis because of deductions for bolt holes in the flanges.

$$I_w = 15{,}891 + 58.69(7.55)^2 = 19{,}236 \text{ in.}^4$$

Web Moments for Design Loads

$$M_v = \frac{223 \times 3.25}{12} = \quad 60$$

$$M_w = 4{,}073 \times \frac{19{,}366}{55{,}508} = \frac{1{,}411}{1{,}471} \text{ kip-ft, or 736 kip-ft per web}$$

Try two $\frac{3}{8} \times 55$-in. web splice plates. Assume two rows of $\frac{7}{8}$-in.-dia, A325 bolts, with 14 bolts per row, on each side of the joint. The area of one hole is 0.375 in.^{2} The holes remove from each splice plate the following percentage of its cross-sectional area:

$$\% \text{ of plate} = \frac{14 \times 0.375}{0.375 \times 55} \times 100 = 25.5\%$$

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

$$\frac{25.5 - 15.0}{22.5} = 0.41$$

With 4-in. spacing of bolts along the slope of the web,

$$d^2 \text{ for holes} = 2^2 + 6^2 + 10^2 + 14^2 + 18^2 + 22^2 + 26^2 = 1{,}820$$

$$\Sigma Ad^2 = 4 \times 0.41 \times \tfrac{3}{8} \times 1{,}820 = 1{,}119 \text{ in.}^4$$

or, with respect to a horizontal axis

$$\Sigma Ad^2 = 1{,}119 \left(\frac{57}{58.69}\right)^2 = 1{,}055 \text{ in.}^4$$

Assume that the neutral axis of the splice coincides with the neutral axis of the net section of the box girders. The bending properties of the web splice plates with respect to a horizontal axis are then computed as follows:

The area of two bolts holes to be deducted equals $2 \times 4 \times 0.375 \times 0.41 = 4.31$ in.^{2}

Web-Splice Section

Material	A	d	Ad^2	I_o	I
2 Splice Pl. $\frac{3}{8} \times 55$	41.25	7.55	2,351	9,807	12,158
Area of Holes	−4.31	7.55	−246	−1,055	−1,301

10,857 in.^{4}

$$d_{\text{Top of splice}} = 27.50 + 7.55 = 35.05 \text{ in.} \qquad d_{\text{Bot. of splice}} = 27.50 - 7.55 = 19.95 \text{ in.}$$

$$S_{\text{Top of splice}} = \frac{10{,}857}{35.05} = 310 \text{ in.}^3 \qquad S_{\text{Bot. of splice}} = \frac{10{,}857}{19.95} = 544 \text{ in.}^3$$

The maximum bending stress in the plates for the Maximum Design Load therefore is

$$f_b = \frac{736 \times 12}{310} = 28.5 < 36 \text{ ksi}$$

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

$$F_v = 0.55F_y = 0.55 \times 36 = 19.8 \text{ ksi}$$

The shear stress for the Maximum Design Shear is

$$f_v = \frac{115}{41.25} = 2.79 < 19.8 \text{ ksi}$$

The $\frac{3}{8} \times 55$-in. web splice plates are stisfactory for Maximum Design Load requirements. The plates are next checked for fatigue under service loads.

The range of moment carried by the web equals

$$M_w = (1{,}640 + 1{,}100)\frac{19{,}236}{55{,}508} = 950, \text{ or } \frac{950}{2} = 475 \text{ kip-ft per web}$$

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

$$f_{br} = \frac{475 \times 12 \times 35.05}{12{,}158} = 16.4 \text{ ksi}$$

Check for Fatigue

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

Use two $\frac{3}{8} \times 55$-in. web splice plates.

Web Bolts

The 28 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

$$I = 2 \times 2 \times 1{,}820 + 28\left(7.55 \times \frac{58.69}{57}\right)^2 + 28(1.5)^2 = 9{,}035$$

Web Moments for Overload

$$M_v = \frac{151.5 \times 3.25}{12} \quad = \quad 41$$

$$M_w = 2{,}767 \times \frac{19{,}236}{55{,}508} = \frac{959}{1{,}000} \text{ or } \frac{1{,}000}{2} = 500 \text{ kip-ft per web}$$

Load per bolt due to shear is

$$P_s = \frac{151.5}{2(28)} \times \frac{58.69}{57} = 2.79 \text{ kips}$$

Load on the outermost bolt due to moment is

$$\text{Vertical in-plane component} = \frac{500 \times 12 \times 1.5}{9{,}035} \times \frac{58.69}{57} = 1.03 \text{ kips}$$

$$\text{Horizontal in-plane component} = \frac{500(58.69/57)12(26 + 7.55 \times 58.69/57)}{9{,}035} = 23.09 \text{ kips}$$

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

$$P=\sqrt{(2.79+1.03)^2+(23.09)^2}=23.4<25.3 \text{ kips}$$

Use fourteen ⅞-in.-dia, A325 bolts in two rows.

Flange-Splice Design

The flange splice plates are proportioned for the Maximum Design Load and checked for fatigue.

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

$$f_{b\ \text{Top}}=\frac{4{,}073\times 12(28.78+7.55)}{55{,}508}=32.0 \text{ ksi}$$

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

$$P_{\text{Top}}=32.0\left(\frac{13.50-0.34}{2}\right)=211 \text{ kips}$$

The required net area of the top-flange splice plates then becomes

$$A_{\text{Top}}=\frac{211}{36}=5.86 \text{ in.}^2$$

This value exceeds 75% of the net area of the top flange:

$$0.75\left(\frac{13.50-0.34}{2}\right)=4.94<5.86 \text{ in.}^2$$

Try a ⁵⁄₁₆-in. outer splice plate and two ⅜×5⅜-in. inner splice plates. The net area of these plates after deduction of bolt holes in excess of 15% of the plate area is

$$\begin{aligned}\text{Top plate}&=(\tfrac{5}{16}\times 12)-2.106(1\times\tfrac{5}{16})+0.15(\tfrac{5}{16}\times 12) &&=3.65\\ \text{Bot. plate}&=2[(\tfrac{3}{8}\times 5\tfrac{3}{8})-1.053(1\times\tfrac{3}{8})+0.15(\tfrac{3}{8}\times 5\tfrac{3}{8})] &&=\underline{3.85}\\ &&&\quad 7.50>5.86 \text{ in.}^2\end{aligned}$$

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

$$f_{b\ \text{Bot.}}=\frac{4{,}073\times 12(28.72-7.55)}{55{,}508}=18.6 \text{ ksi}$$

The total flange force is

$$P_{\text{Bot.}}=18.6[40.25-16\times\tfrac{7}{16}+0.15\times 40.25)]=18.6\times 39.29=731 \text{ kips}$$

The required net area of the bottom flange becomes

$$A_{\text{Bot.}}=\frac{731}{36}=20.3 \text{ in.}^2$$

This is less than 75% of the net area of the bottom flange. Therefore, the required area of the bottom plate is

$$A_{\text{Bot.}}=0.75\times 39.39=29.5 \text{ in.}^2$$

Try two ⅜×41½-in. outer plates and two ⅜×41½-in. inner plates. The net area after deduction of bolt holes in excess of 15% of the plate area is

$$4[\tfrac{3}{8}\times 41.5-8(1\times\tfrac{3}{8}+0.15(\tfrac{3}{8}\times 41.5)]=59.6>29.5 \text{ in.}^2$$

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

The range of live-load moment at the splice equals

$$M_{Lr}=1{,}640+1{,}100=2{,}740 \text{ kip-ft}$$

And the range of average stress in the flanges is

$$\text{Top Flange: } f_{sr}=\frac{2{,}740\times12(28.78+7.55)}{55{,}508}=21.5 \text{ ksi}$$

$$\text{Bot. Flange: } f_{sr}=\frac{2{,}740\times12(28.78-7.55)}{55{,}508}=12.6 \text{ ksi}$$

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

$$\text{Top Flange: } f_{sr}=\frac{21.5(\frac{1}{2})(13.50-0.34)}{\frac{5}{16}\times12+2\times\frac{3}{8}\times5.375}=18.2<27.5 \text{ ksi}$$

$$\text{Bot. Flange: } f_{sr}=\frac{12.6\times39.29}{4\times\frac{3}{8}\times41.5}=7.95<27.5 \text{ ksi}$$

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 2,767 kip-ft.

The average stress in the top flange is

$$f_b=\frac{2{,}767\times12(28.78+7.55)}{55{,}508}=21.7 \text{ ksi}$$

And the flange force becomes

$$P_{\text{Top}}=21.7\left(\frac{13.50-0.34}{2}\right)=143 \text{ kips}$$

For this flange force, the number of bolts required is

$$\frac{143}{25.3}=5.7 \text{ bolts}$$

Use 8 bolts.

The average stress in the bottom flange is

$$f_b=\frac{2{,}767\times12(28.72-7.55)}{55{,}508}=12.7 \text{ ksi}$$

And the bottom-flange force is

$$P_{\text{Bot.}}=12.7\times39.29=499 \text{ kips}$$

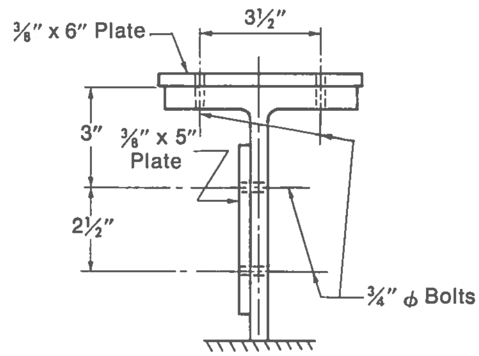
For this flange force, the number of bolts required is

$$\frac{499}{25.3}=19.7 \text{ bolts}$$

Use 64 bolts. Details of the splice are shown on page 54.

Stiffener Splice

Next, a splice is designed for the ST7.5×25, longitudinal, bottom-flange stiffener. A splice in the stiffener is desirable to assure that the interruption of the stiffener at the field splice does not become a node for buckling. The splice is designed for the axial capacity of the ST7.5×25. This capacity equals the product of the critical buckling stress for the bottom flange and the area of the stiffener.



SPLICE OF ST7.5×25

The allowable bottom-flange stress at the field splice was determined previously for the section 37 ft from the interior support to be 9.9 ksi. The force on the stiffener therefore is

$$P_{st} = 9.9 \times 7.35 = 72.8 \text{ kips}$$

Use ¾-in.-dia, A325 bolts, with an allowable stress in single shear of $0.442 \times 21 = 9.3$ kips per bolt. The number of bolts required is

$$\frac{72.8}{1.3 \times 9.3} = 6.0 \text{ bolts}$$

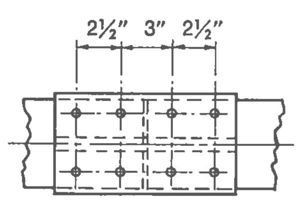
Use 8 bolts.

The area required for the splice plates is

$$A_{st} = \frac{72.8}{36} = 2.02 \text{ in.}^2$$

Try a ⅜×6-in. splice plate on top of the flange and a ⅜×5-in. plate on the stem, each with two longitudinal rows of bolts. The net area of the plates is

$$\begin{aligned} \text{Flange: } & 6 \times \tfrac{3}{8} - 2(\tfrac{7}{8} \times \tfrac{3}{8}) + 0.15(6 \times \tfrac{3}{8}) = 1.93 \\ \text{Stem: } & 5 \times \tfrac{3}{8} - 2(\tfrac{7}{8} \times \tfrac{3}{8}) + 0.15(5 \times \tfrac{3}{8}) = \underline{1.50} \\ & \qquad\qquad 3.43 > 2.02 \text{ in.}^2 \end{aligned}$$



STIFFENER-FLANGE SPLICE

DESIGN OF PIER

The pier that serves as the interior support of the box girder is investigated at the bottom and at the top, just below the cross girder. Two types of load, in addition to ordinary dead, live and impact loads, influence the design of the pier:

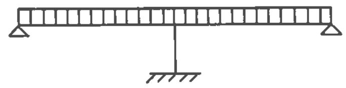
1. *Wind on the structure and on the live load.* These loads induce transverse bending moments, which are computed by treating the bridge frame as a structure loaded normal to its plane.

2. *Longitudinal loads from traction and braking.* These loads produce longitudinal bending moments, which are obtained from an analysis of the bridge as a vertical frame.

Three group loadings are considered in design of the pier:

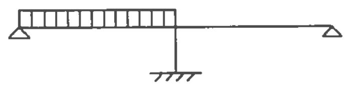
Group I loading is $1.30[D + 5/3(L+I)]$. This group includes four cases of loading:

Case 1. Three lanes of live load, symmetrically placed on the box girder, and applied on both spans for maximum axial load on the pier. (Apply a 10% reduction because of the small probability of coincident maximum loading.)



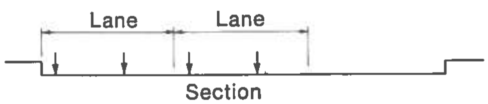
CASE 1 LOADING

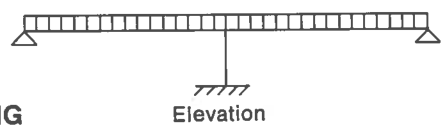
Case 2. Three lanes of live load, symmetrically placed on the box girder, and applied on only one span to produce maximum stresses from axial load and longitudinal moment on the pier. (Apply a 10% reduction because of the small probability of coincident maximum loading.)



CASE 2 LOADING

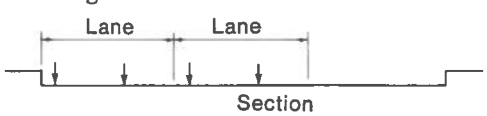
Case 3. Two lanes of live load, eccentrically positioned as shown in the diagram, and applied on two spans to produce maximum stresses from axial load and transverse moment on the pier.

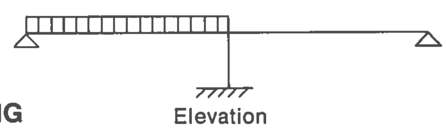




CASE 3 LOADING

Case 4. Two lanes of live load, eccentrically positioned, as shown in the diagram, and applied on only one span to produce maximum stresses from axial load, longitudinal moment and transverse moment on the pier.





CASE 4 LOADING

Group II loading is $1.30(D+W)$, where W is wind on the structure.

Group III loading is $1.30(D+L+I+0.3W+WL+LF)$, where WL is wind on the live load and LF is the longitudinal force due to live loads.

Axial loads and bending moments obtained from the application of the four loading cases of Group I and from Groups II and III are listed in a table.

LOADS AT BOTTOM OF COLUMN

Service Loads				Maximum Design Loads Group I: $1.30[D+5/3(L+I)]$		
Load	P, kips	M_x, kip-ft	M_y, kip-ft	P, kips	M_x, kip-ft	M_y, kip-ft
Case 1						
DL_1*	891	0	0	1,158	0	0
DL_2	211	0	0	274	0	0
$L+I$	406	0	0	880	0	0
	1,508	0	0	2,312	0	0

*Includes 20 kips for weight of cross girder and column.

LOADS AT BOTTOM OF COLUMN

Service Loads				Maximum Design Loads Group I: 1.30[$D+5/3(L+I)$]		
Load	P, kips	M_x, kip-ft	M_y, kip-ft	P, kips	M_x, kip-ft	M_y, kip-ft
Case 2						
DL_1*	891	0	0	1,158	0	0
DL_2	211	0	0	274	0	0
$L+I$	230	497	0	498	1,077	0
	1,332	497	0	1,930	1,077	0
Case 3						
DL_1*	891	0	0	1,158	0	0
DL_2	211	0	0	274	0	0
$L+I$	301	0	2,556	652	0	5,538
	1,403	0	2,556	2,084	0	5,538
Case 4						
DL_1*	891	0	0	1,158	0	0
DL_2	211	0	0	274	0	0
$L+I$	171	368	1,449	370	797	3,139
	1,273	368	1,449	1,802	797	3,139

Service Loads				Group II: 1.30 ($D+W$)		
Load	P, kips	M_x, kip-ft	M_y, kip-ft	P, kips	M_x, kip-ft	M_y, kip-ft
DL_1*	891	0	0	1,158	0	0
DL_2	211	0	0	274	0	0
W	0	257	396	0	334	515
	1,102	257	396	1,432	334	515

Service Loads				Group III: 1.30($D+L+I+0.3W+WL+LF$)		
Load	P, kips	M_x, kip-ft	M_y, kip-ft	P, kips	M_x, kip-ft	M_y, kip-ft
Case 3						
DL_1*	891	0	0	1,158	0	0
DL_2	211	0	0	274	0	0
$L+I$	301	0	2,556	391	0	3,323
$0.3W$	0	77	119	0	100	155
WL	0	112	93	0	146	121
LF	0	199	0	0	259	0
	1,403	388	2,768	1,823	505	3,599

*Includes 20 kips for weight of cross girder and column.

Service Loads				Group III: 1.30(D+L+I+0.3W+WL+LF)		
Load	P, kips	M_x, kip-ft	M_y, kip-ft	P, kips	M_x, kip-ft	M_y, kip-ft
Case 4						
DL_1*	891	0	0	1,158	0	0
DL_2	211	0	0	274	0	0
$L+I$	171	368	1,449	222	478	1,884
$0.3W$	0	77	119	0	100	155
WL	0	112	93	0	146	121
LF	0	110	0	0	143	0
	1,273	667	1,661	1,654	867	2,160

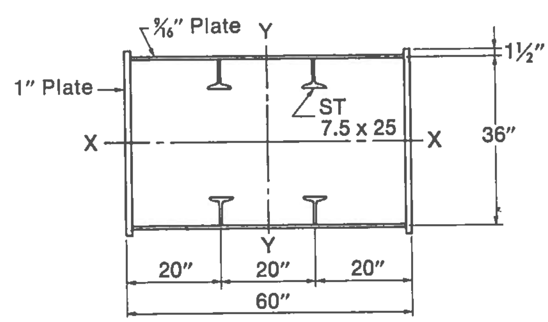
*Includes 20 kips for weight of cross girder and column.

The pier cross section is a rectangular steel box made up of four plates. The two plates that form the sides of the pier, or column, and are perpendicular to the longitudinal axis of the bridge are stiffened inside the box by two ST shapes, spaced equally across the width of each plate. The plates of the column section are designed for a critical buckling stress F_{cr} in the same manner as the bottom compression flange of the box girder. All steel is A36.

For stiffeners, an ST7.5×25 is selected. This is the same shape used for stiffening the box-girder compression flange. For computation of F_{cr} for the column plates, a k value is computed based on the stiffener and compression-plate properties. The critical buckling stress is then determined from the graphs previously presented in the Design of Girder Sections.

For the unstiffened plates, k is taken as 4, and F_{cr} is obtained as for a stiffened plate.

Try a 9/16-in.-thick plate for the stiffened plates and a 1-in.-thick plate for the unstiffened plates of the pier. The critical buckling stresses and section properties are calculated as follows:



SECTION AT BOTTOM OF COLUMN

For a 9/16-in. plate,

$$k = \sqrt[3]{\frac{2{,}432}{0.07(2)^4(9/16)^3 20}} = 3.93$$

For a 1-in. plate, $k=4$. The width-thickness ratio of the 9/16-in. plate is

$$\frac{w}{t} = \frac{20}{9/16} = 35.6$$

From the graph of buckling stresses, $F_{cr}=35.6$ ksi. For the 1-in. plate, $w/t=36/1=36$ and $F_{cr}=35.7$ ksi.

Section at Bottom of Column

Material	A	d_x	Ad_x^2	I_{ox}	I_x	d_y	Ad_y^2	I_{oy}	I_y
2 Pl. $\frac{9}{16}\times58$	65.25	18.00	21,141		21,141			18,292	18,292
2 Pl. 1×39	78.00			9,887	9,887	29.50	67,880		67,880
4 ST7.5×25	29.40	12.47	4,572	162	4,734	10.00	2,940	31	2,971
	172.65 in.^{2}				35,762 in.^{4}				89,143 in.^{4}

Stresses at the corners of the box are calculated taking into account axial and bending effects. The governing condition is found to be Group I, Case 3.

Group I

Case 1: $f=\dfrac{2,312}{172.65}=13.4$ ksi

Case 2: $f=\dfrac{1,930}{172.65}+\dfrac{1,077\times12\times19.5}{35,762}=18.2$ ksi

Case 3: $f=\dfrac{2,084}{172.65}+\dfrac{5,538\times12\times30}{89,143}=34.5<35.6$ ksi

Case 4: $f=\dfrac{1,802}{172.65}+\dfrac{797\times12\times19.5}{35,762}+\dfrac{3,139\times12\times30}{89,143}=28.3$ ksi

Group II

$$f=\frac{1,432}{172.65}+\frac{334\times12\times19.5}{35,762}+\frac{515\times12\times30}{89,143}=12.6 \text{ ksi}$$

Group III

Case 3: $f=\dfrac{1,823}{172.65}+\dfrac{505\times12\times19.5}{35,762}+\dfrac{3,599\times12\times30}{89,143}=28.4$ ksi

Case 4: $f=\dfrac{1,654}{172.65}+\dfrac{867\times12\times19.5}{35,762}+\dfrac{2,160\times12\times30}{89,143}=24.0$ ksi

Next, stresses are checked at the top of the pier. The loads at this section are shown in a table.

LOADS AT TOP OF COLUMN

Service Loads				Maximum Design Loads Group I: $1.30[D+D+5/3(L+I)]$		
Load	P, kips	M_x, kip-ft	M_y, kip-ft	P, kips	M_x, kip-ft	M_y, kip-ft
Case 1						
DL_1*	876	0	0	1,139	0	0
DL_2	211	0	0	274	0	0
$L+I$	406	0	0	880	0	0
	1,493	0	0	2,293	0	0
Case 2						
DL_1*	876	0	0	1,139	0	0
DL_2	211	0	0	274	0	0
$L+I$	230	497	0	498	1,077	0
	1,317	497	0	1,911	1,077	0

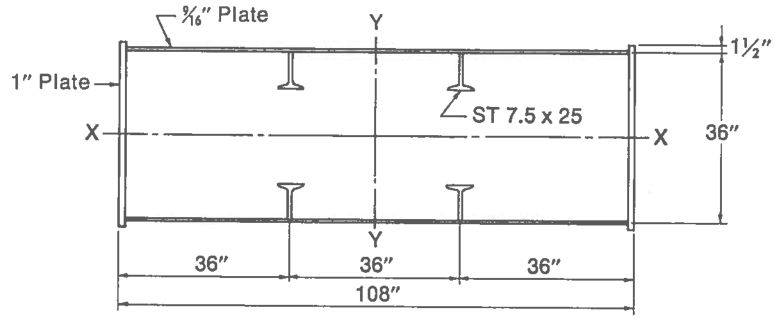
*Includes 5 kips for the weight of the cross girder.

LOADS AT TOP OF COLUMN

Service Loads				Maximum Design Loads Group I: 1.30[$D+D+5/3(L+I)$]		
Load	P, kips	M_x, kip-ft	M_y, kip-ft	P, kips	M_x, kip-ft	M_y, kip-ft
Case 3						
DL_1*	876	0	0	1,139	0	0
DL_2	211	0	0	274	0	0
$L+I$	301	0	2,556	652	0	5,538
	1,388	0	2,556	2,065	0	5,538
Case 4						
DL_1*	876	0	0	1,139	0	0
DL_2	211	0	0	274	0	0
$L+I$	171	368	1,449	370	797	3,139
	1,258	368	1,449	1,783	797	3,139
Service Loads				Group II: 1.30($D+W$)		
Load	P, kips	M_x, kip-ft	M_y, kip-ft	P, kips	M_x, kip-ft	M_y, kip-ft
DL_1*	876	0	0	1,139	0	0
DL_2	211	0	0	274	0	0
W	0	257	607	0	334	789
	1,087	257	607	1,413	334	789
Service Loads				Group III: 1.30($D+L+I+0.3W+WL+LF$)		
	P, kips	M_x, kip-ft	M_y, kip-ft	P, kips	M_x, kip-ft	M_y, kip-ft
Case 3						
DL_1*	876	0	0	1,139	0	0
DL_2	211	0	0	274	0	0
$L+I$	301	0	2,556	391	0	3,323
$0.3W$	0	77	182	0	100	237
WL	0	107	150	0	139	195
LF	0	190	0	0	247	0
	1,388	374	2,888	1,804	486	3,755
Case 4						
DL_1*	876	0	0	1,139	0	0
DL_2	211	0	0	274	0	0
$L+I$	171	368	1,449	222	478	1,884
$0.3W$	0	77	182	0	100	237
WL	0	107	150	0	139	195
LF	0	104	0	0	135	0
	1,258	656	1,781	1,635	852	2,316

*Includes 5 kips for the weight of the cross girder.

The plates comprising the section at the top of the pier have the same thickness as those at the bottom, but the width of the stiffened plate is 9 ft, rather than 5 ft as at the bottom.



SECTION AT TOP OF COLUMN

The critical buckling stress F_{cr} for the 1×39-in. unstiffened plates is the same at the top as at the bottom of the pier. For the $\frac{9}{16}$-in. plates, however, F_{cr} is considerably smaller at the top than at the bottom of the pier, because w/t is larger at the top. But the decrease in F_{cr} is compensated for by the larger area and moment of inertia at the top. The result is that stresses are smaller in the plates at the top. In the investigation of the section at the top, the governing loading condition, as at the bottom, is Group I, Case 3.

For a $\frac{9}{16}$-in. plate,

$$k = \sqrt[3]{\frac{243.2}{0.07(2)^4(\frac{9}{16})^3 36}} = 3.23$$

For a 1-in. plate, $k=4$. The width-thickness ratio of the $\frac{9}{16}$-in. plate is

$$\frac{w}{t} = \frac{36}{\frac{9}{16}} = 64$$

From the graphs presented earlier in this chapter, $F_{cr} = 20.6$ ksi. For the 1-in. plate, $w/t = 36/1 = 36.0$ and $F_{cr} = 35.7$ ksi

Section at Top of Column

Material	A	d_x	Ad_x^2	I_{ox}	I_x	d_y	Ad_y^2	I_{oy}	I_y
2 Pl. $\frac{9}{16} \times 106$	119.25	18.00	38,637		38,637			111,658	111,658
2 Pl. 1×39	78.00			9,887	9,887	53.50	223,256		223,256
4 ST 7.5×25	29.40	12.47	4,572	162	4,734	18.00	9,526	31	9,557
	226.65 in.^{2}				53,258 in.^{4}				344,471 in.^{4}

Group I

Case 1: $f = \frac{2,293}{226.65} = 10.1$ ksi

Case 2: $f = \frac{1,911}{226.65} + \frac{1,077 \times 12 \times 19.5}{53,258} = 13.1$ ksi

Case 3: $f = \frac{2,065}{226.65} + \frac{5,538 \times 12 \times 54}{344,471} = 19.5 < 20.6$ ksi

Case 4: $f = \frac{1,783}{226.65} + \frac{797 \times 12 \times 19.5}{53,258} + \frac{3,139 \times 12 \times 54}{344,471} = 17.3$ ksi

Group II

$$f=\frac{1{,}413}{226.65}+\frac{334\times12\times19.5}{53{,}258}+\frac{789\times12\times54}{344{,}471}=9.2\text{ ksi}$$

Group III

$$\text{Case 3: } f=\frac{1{,}804}{226.65}+\frac{486\times12\times19.5}{53{,}258}+\frac{3{,}755\times12\times54}{344{,}471}=17.2\text{ ksi}$$

$$\text{Case 4: } f=\frac{1{,}635}{226.65}+\frac{852\times12\times19.5}{53{,}258}+\frac{2{,}316\times12\times54}{344{,}471}=15.3\text{ ksi}$$

DESIGN OF CROSS GIRDER

A cross girder is mounted on top of the pier to transfer the load from the two box girders to the column. The cross girder is integrally joined to the column and extends to the exterior webs of the box girders.

A field splice is made in the cross girder at the interior webs of the box girders. Such an arrangement allows the box girders to pass through the cross girder without interruption.

Loads on Cross Girder

Four loading cases are considered in design of the cross girder:

Loading 1. Dead, live and impact loads are applied, with live load on both box-girder spans, to produce maximum vertical load on the cross girder.

Loading 2. Dead, live and impact loads are applied, with the live load on one box-girder span, to produce maximum torque in the cross girder.

Loading 3. Longitudinal force from live load LF, taken as 5% of the live load, is combined with Loading 1.

Loading 4. Longitudinal force LF is combined with Loading 2.

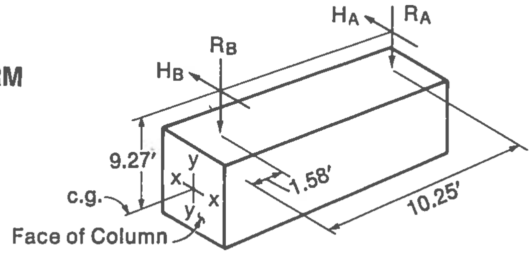
The box girder vertical reactions R_B and R_B at the cross girder, for Loadings 1 and 2, are listed in a table. These reactions are assumed to act through the mid-depth of the exterior and interior box-girder webs, respectively. (Although the calculations are not shown here, the web live loads were determined by utilizing the influence lines for vertical reactions at the center pier.) Lane loading governs.

Service-Load Reactions, Kips

	Loading 1		Loading 2	
	R_A	R_B	R_A	R_B
DL_1	218	218	218	218
DL_2	53	53	53	53
$L+I$	138	108	78	61
Total	409	379	349	332

Shears and moments in the cross girder at the face of the column for $1.30[D+5/3(L+I)]$ for Loadings 1 and 2 are listed in a table. The moment arms, ft, for calculation of the moments are the distances from the center of the box-girder webs to the nearest faces of the column.

CROSS-GIRDER ARM



Cross-Girder Shears and Moments

Loading 1: $1.30[D+5/3(L+I)]$

	Shear, Kips	Arm, Ft	Moment, Kip-ft
R_A	651	10.25	6,676
R_B	586	1.58	926
	1,237		7,602

Loading 2: $1.30[D+5/3(L+I)]$

	Shear, Kips	Arm, Ft	Moment, Kip-ft	Torque, Kip-ft
R_A	521	10.25	5,340	
R_B	484	1.58	765	
	1,005		6,106	654

The torque of 654 kip-ft shown in the table is obtained by considering the behavior of the structure with the live load on one span only. Under the loading condition, the rotation of the box girders in a vertical plane, as they deflect under the unsymmetrical loading, twists the cross girder. The sum of the resulting torques in both arms of the cross girder equals the longitudinal moment in the pier. Thus, an influence line for longitudinal moment in the pier also is an influence line for the sum of the torques in the arms of the cross girder. Therefore, with the use of the influence line for longitudinal moment in the pier, the maximum torque in the cross girder may be calculated for the portion of the live load applied to one arm of the cross girder.

Similarly, moments, shears and torques in the cross girder for Loadings 3 and 4 are calculated and listed in a table. In the table, the total moment M_y about the vertical axis is reduced 50% for the following reason: The cross-girder arms are actually not free cantilevers in the horizontal direction. The transverse rigidity of the box girders restrains rotation of the far end of the cross girder. As a result, the bending moment M_y in the cross girder at the face of the column is reduced by about one-half.

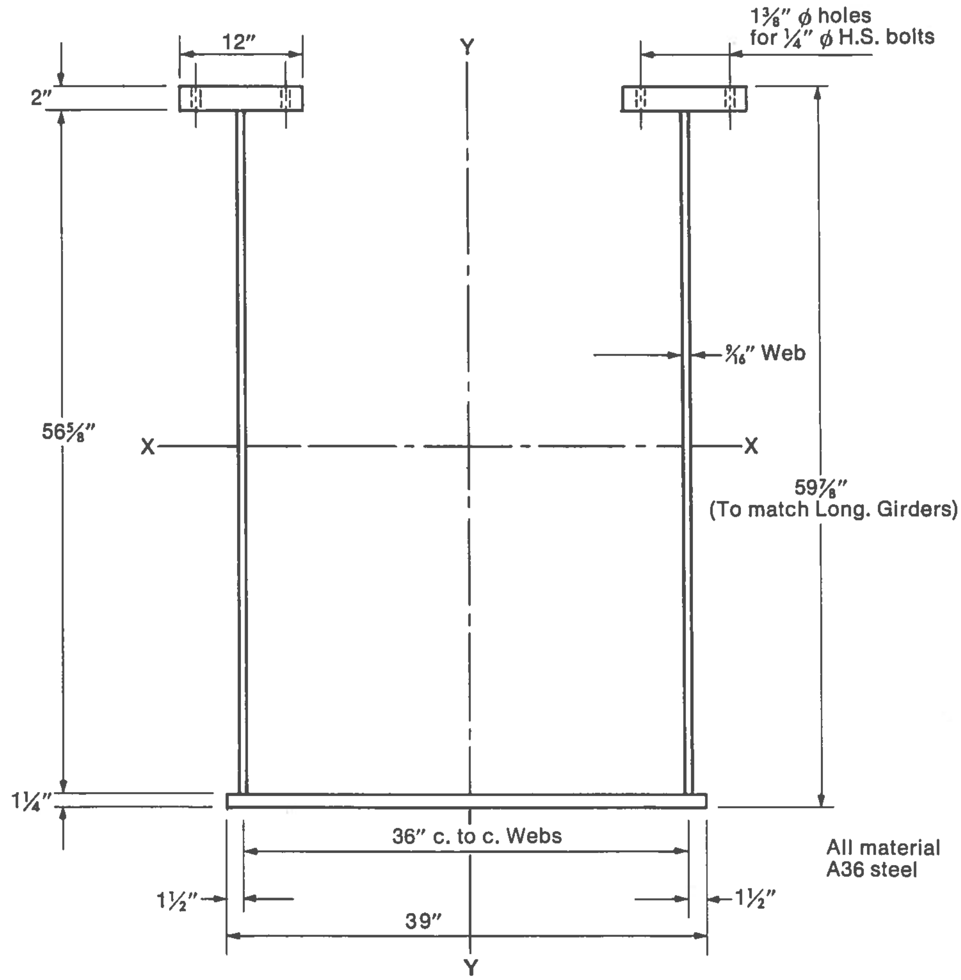
Cross-Girder Shears, Moments and Torques

Loading 3: $1.30(D+L+I+LF)$

	Vertical Loads $1.3(D+L+I)$ Kips	Horizontal Loads, $1.3LF$ Kips	Moment Arm, Ft	M_x Kip-ft	M_y Kip-ft	Torque Arm, Ft, about C.G. of Box Girder	Torque, Kip-ft
R_A	532		10.25	5,453			
H_A		10.3	10.25		106	9.27	95
R_B	493		1.58	779			
H_B		8.1	1.58		13	9.27	75
	1,025	18.4		6,232	119 Use 59		170

Loading 4: $1.30(D+L+I+LF)$

	Vertical Loads $1.3(D+L+I)$ Kips	Horizontal Loads, $1.3LF$ Kips	Moment Arm, Ft	M_x Kip-ft	M_y Kip-ft	Torque Arm, Ft, about C.G. of Box Girder	Torque, Kip-ft
R_A	454		10.25	4,654			246
H_A		5.6	10.25		57	9.27	52
R_B	432		1.58	683			194
H_B		4.4	1.58		7	9.27	42
	886	10.0		5,337	64 Use 32		534



CROSS-GIRDER SECTION AT FACE OF COLUMN

In cross section, the cross girder is a hollow box, with two ⁹⁄₁₆-in.-thick webs, a 1¼ × 39-in. bottom flange and two 2 × 12-in. top flanges. A drawing shows the cross section at the face of the column. The section satisfies requirements for a braced, noncompact section.

The ⁹⁄₁₆-in. webs of the section match the thickness of abutting plates in the same plane in the pier. Properties about the horizontal and vertical axes of the section are calculated for the gross and net section.

Cross-Girder Section Properties about X-X Axis

Gross Section

Material	A	d_x	Ad_x	Ad_x^2	I_o	I_x
2 Flg. Pl. 2×12	48.00	29.31	1,407	41,236		41,236
2 Web Pl. $\frac{9}{16}\times56\frac{5}{8}$	63.70				17,021	17,021
Flg. Pl. $1\frac{1}{4}\times39$	48.75	−28.94	−1,411	40,829		40,829
	160.45 in.^{2}		−4 in.^{3}			99,086

$$d_s = \frac{-4}{160.45} = -0.02$$

$-0.02\times4 =$ 0

$I_{\text{Gross}} =$ 99,086 in.^{4}

Net Section

Material	A	d_x	Ad_x^2	I_o	I_x
Gross Section	160.45			99,086	99,086
4 Holes $1\frac{7}{8}\times2$	−11.00	29.33	−9,463		−9,463
15% of Top Flg. Area	7.20	29.33	6,194		6,194
	156.65 in.^{2}				$I_{\text{Net}}=95{,}817$ in.^{4}

$d_{\text{Top of steel}} = 30.31+0.02=30.33$ in.

$d_{\text{Bot. of steel}} = 29.56-0.02=29.54$ in.

$$S_{\text{Top of steel}} = \frac{95{,}817}{30.33} = 3{,}159 \text{ in.}^3$$

$$S_{\text{Bot. of steel}} = \frac{95{,}817}{29.54} = 3{,}244 \text{ in.}^3$$

The properties of the net section are obtained by deducting the area of the flange holes in excess of 15% of the flange area. The center of gravity of the gross section is used as the reference axis in locating the center of gravity of the net section.

Cross-Girder Section Properties about Y-Y Axis

Gross Section

Material	A	d_y	Ad_y^2	I_o	I_y
2 Flg. Pl. 2×12	48.00	18.00	15,552	576	16,128
2 Web Pl. $\frac{9}{16}\times56\frac{5}{8}$	63.70	18.00	20,639		20,639
Flg. Pl. $1\frac{1}{4}\times39$	48.75			6,179	6,179
	160.45 in.^{2}				$I_{\text{Gross}}=42{,}946$ in.^{4}

Net Section

Material	A	d_y	Ad_y^2	I_y
Gross Section	160.45			42,946
2 Outer Holes $1\frac{3}{8}\times2$	−5.50	21.00	−2,426	−2,426
2 Inner Holes $1\frac{3}{8}\times2$	−5.50	15.00	−1,238	−1,238
15% of Top Flg. Area	7.20	18.00	2,333	2,333
				$I_{\text{Net}}=41{,}615$ in.^{4}

Because the top flange is in tension, there is no restriction on the width-thickness ratio of this flange.

Web Thickness and Stiffeners at Face of Column

The web-thickness ratio of the cross girder at the face of the column is

$$\frac{D}{t_w}=\frac{56.63}{9/16}=101<150$$

This ratio is satisfactory for an unstiffened web. The maximum design shear, however, exceeds the buckling capacity for an unstiffened web. Consequently, transverse stiffeners are necessary on the web.

The maximum shear permissible is

$$V_b=\frac{3.5Et_w^3}{D}=\frac{3.5\times 29{,}000(9/16)^3}{56.63}=315 \text{ kips per web}$$

For Loading 1, the maximum shear is

$$V=\frac{1{,}237}{2}=619>315 \text{ kips per web}$$

Hence, transverse stiffeners are needed. But a longitudinal stiffener is not required because

$$\frac{D}{t_w}=\frac{36{,}500}{\sqrt{F_y}}=\frac{36{,}500}{\sqrt{36{,}000}}=192>101$$

The shear capacity is calculated with the assumption that stiffeners will be spaced about 3 ft apart. A reduction in moment capacity is required where the shear exceeds 60% of the web shear capacity.

The maximum shear capacity of one web is

$$V_p=0.58F_yDt_w=0.58\times 36\times 56.63\times 9/16=665 \text{ kips}$$

For computation of the web shear capacity V_u with the web stiffener spacing $d_o=36$ in.,

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

$$=18{,}000\times\frac{9/16}{56.63}\sqrt{\frac{1+(56.63)^2}{36{,}000}}-0.3=1.45>1$$

Because C is larger than unity, the shear strength of the web $V_u=V_p=665$ kips. Hence, where the shear on both webs V exceeds $0.60\times 2\times 665=798$ kips, the bending-moment capacity is reduced as required by

$$\frac{M}{M_u}=1.375-0.625\frac{V}{V_u}=1.375-0.625\times\frac{V}{2\times 665}=\frac{1.375}{1.375}-\frac{V}{2{,}128}$$

Check of Bending Stresses in Cross Girder—Loading 1

Next, bending stresses are checked for Loading 1. The ultimate moment capacity M_u of the section is controlled by the allowable tensile strength F_y of the top flange. (Note that the critical buckling stress in the bottom flange also is F_y, as obtained for a w/t ratio of $36/1.25=28.8$ from the curves for F_{cr} previously presented.

For Loading 1, at the face of the column,

$$V=1{,}237 \text{ kips}>(0.60V_u=798 \text{ kips})$$

Therefore, the moment capacity is reduced by the fraction:

$$\frac{M}{M_u}=1.375-\frac{1{,}237}{2{,}128}=0.794$$

For the bending moment of 7,602 kip-ft at the face of the column, the stress at the top of the steel section is

$$f_b=\frac{7{,}602\times 12}{3{,}159}=28.9\approx(36\times 0.794=28.6 \text{ ksi})$$

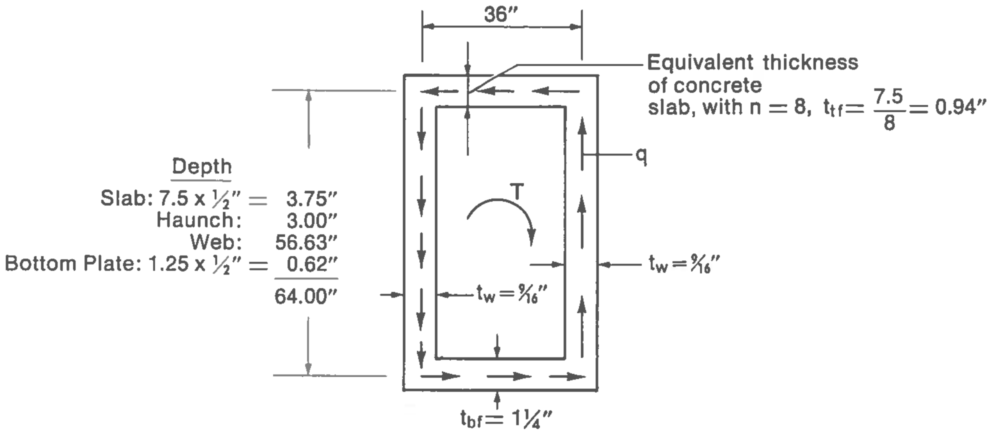
The stress at the bottom of the section is

$$f_b = \frac{7{,}602 \times 12}{3{,}244} = 28.1 < 28.6 \text{ ksi}$$

The section is satisfactory in bending under Loading 1.

Shear and Torque in Cross Girder

Under Loading 2, the cross girder is subjected to torque as well as to shear and moment. The torque is resisted by shear stresses in the box section, including the concrete slab. The total shear stress equals the sum of the torsional shear and the shear stress due to flexure.



CROSS-GIRDER TORSIONAL STRESSES

The following calculations indicate that for Loading 2 the total shear in a web, from vertical loads and torsion, is less than the shear capacity of the web, $V_u = 665$ kips. The torsional shear in the web is

$$q = \frac{T}{2bd} = \frac{654 \times 12}{2 \times 36 \times 64} = 1.70 \text{ kips per in.}$$

The shear in a web due to flexure for Loading 2 is

$$V = \frac{1{,}005}{2} = 502.5 \text{ kips}$$

The total web shear then is

$$V_T = 502.5 + 1.70 \times 56.63 = 599 < 665 \text{ kips}$$

The section is satisfactory for the combination of torsional and flexural shears.

Cross-Girder Bending about Two Axes

The section next is checked for bending about both the X-X and Y-Y axes under Loading 3. The shear stress in the webs is observed to be not critical and therefore need not be checked.

The shear due to flexure for Loading 3 is

$$V = 1{,}025 > (0.60V_u = 798 \text{ kips})$$

The bending moment about the X-X and Y-Y axes are, respectively, $M_x = 6{,}232$ kip-ft and $M_y = 59$ kip-ft. The moment capacity of the section because of the high shear is reduced as required by

$$\frac{M}{M_u} = 1.375 - \frac{1{,}025}{2{,}128} = 0.893$$

The bending stress at the bottom of the section is

$$f_b = \frac{6{,}232 \times 12}{3{,}244} + \frac{59 \times 12 \times 19.50}{41{,}615} + 23 = 0.3 = 23.3 < (36 \times 0.893 = 32.2 \text{ ksi})$$

Loading 4 is eliminated by inspection.

Check of Cross Girder for Fatigue

Fatigue is checked at the top of the cross-girder web where the flange-to-web fillet weld is terminated for the field splice at the interior webs of the box girders. The fatigue stress range in the web adjacent to the terminated fillet weld is governed by AASHTO Category E. The allowable range is 21 ksi for 100,000 cycles of lane loading. Because the section being checked is at a point of high stress produced by combined shear and bending, the range of principal stress is determined instead of the range of normal stresses due to bending.

For Loading 1, the live-load moment range is

$$M_L = 138 \times 10.25 + 108 \times 1.58 = 1{,}585 \text{ kip-ft}$$

The bending-stress range at the top of the web is

$$f_{br} = \frac{M_L c}{I} = \frac{1{,}585 \times 12 \times 28.33}{95{,}817} = 5.62 \text{ ksi}$$

The live-load shear range is $V_L = 138 + 108 = 246$ kips. The shear-stress range at the top of the web is

$$f_{vr} = \frac{V_L Q}{I_b} = \frac{246(1.15 \times 48 - 11.0)29.33}{95{,}817 \times 0.56} = 5.94 \text{ ksi}$$

The principal-stress range then is

$$f_r = \frac{f_{br}}{2} + \sqrt{\left(\frac{f_{br}}{2}\right)^2 + f_{vr}^2} = \frac{5.62}{2} + \sqrt{\left(\frac{5.62}{2}\right)^2 + 5.94^2} = 9.38 < 21 \text{ ksi}$$

Next, the flange-to-web weld is investigated for fatigue in the usual manner and found to be governed by thickness of material, rather than strength under maximum design loads. Also, fatigue in the weld metal is not critical.

Check of Weld at Top Flange

The section at the face of the column is investigated for Loading 1, for which the maximum shear is 1,237 kips. The horizontal shear flow in each web is

$$S = \frac{VQ}{I} = \frac{1{,}237(1.15 \times 48 - 11.0)29.33}{95{,}817 \times 2} = 8.37 \text{ kips per in.}$$

For two welds, the shear flow in each weld is $8.37/2 = 4.19$ kips per in. The weld capacity is $0.45F_u \times 0.707 = 0.45 \times 58 \times 0.707 = 18.5$ ksi.

$$\text{Weld size required} = \frac{4.19}{18.5} = 0.23 \text{ in.}$$

This, however, is less than the minimum weld size required by AASHTO Specifications for thickness of the top flange. Therefore, use a ⅜-in. fillet weld.

The shear range at the weld is $V_L = 138 + 108 = 246$ kips. The range of horizontal shear flow in each web is

$$s_r = \frac{V_L Q}{I} = \frac{246(1.15 \times 48 - 11.0)29.33}{95{,}817 \times 2} = 1.66 \text{ kips per in.}$$

For 100,000 cycles of lane loading, the allowable shear-stress range on the throat of the fillet welds is 15 ksi. The actual stress range in the ⅜-in. weld is

$$f_{vr} = \frac{1.66}{0.375 \times 0.707 \times 2} = 3.13 < 15 \text{ ksi}$$

Check of Weld at Bottom Flange

The horizontal shear flow in each web at the bottom flange is

$$S=\frac{1{,}237\times 39\times 1.25\times 28.92}{95{,}817\times 2}=9.10 \text{ kips per in.}$$

For two welds, the shear flow in each weld is 9.10/2 = 4.55 kips per in.

$$\text{Weld size required}=\frac{4.55}{18.5}=0.25 \text{ in.}$$

Because this is less than the minimum weld size required by AASHTO Specifications for the thickness of the bottom flange, use that minimum, $\frac{5}{16}$ in.

The shear-flow range at the weld is

$$s_r=\frac{V_L Q}{I}=\frac{246\times 39\times 1.25\times 28.92}{95{,}817}=3.62 \text{ kips per in. per web}$$

For two welds, the stress range in each weld is

$$f_{vr}=\frac{3.62}{0.313\times 0.707\times 2}=8.18<15 \text{ ksi}$$

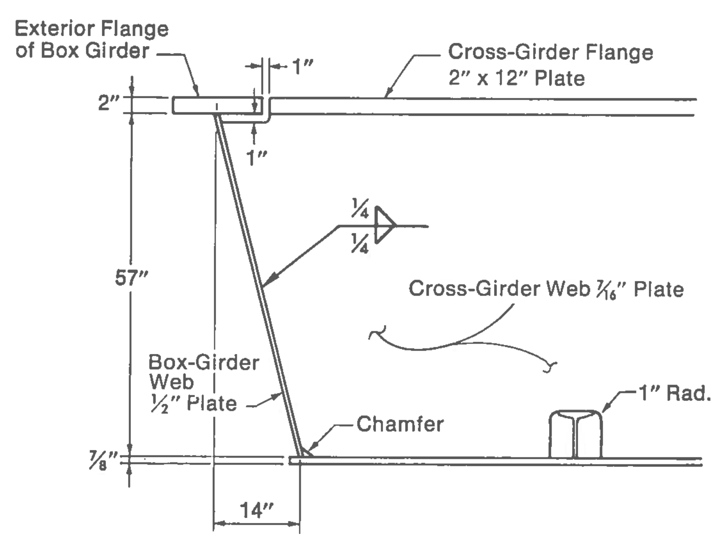
Web Thickness within Box Girders

In the preceding calculations, the design section for the cross girder is at the face of the pier. The shear capacity was computed for a $\frac{9}{16}$-in.-thick web and 36-in. stiffener spacing for the region of the cross girder between the box girders. For the region of the cross girder within a box girder, however, the web thickness is reduced to $\frac{7}{16}$ in. This can be done because the cross-girder web in this region carries only the load from the exterior web of the box girder. Thus, the maximum shear is only about one-half the shear at the face of the pier.

Connection of Cross Girder at Exterior Webs of Box Girders

No flange splice is provided at the junction of the cross-girder flanges with the exterior top flange of a box girder, because there is no stress at the ends of the cross-girder flanges. But even if the cross-girder flanges carried stress, welding into the side of the box-girder flange, which is in tension, should be avoided.

The connection of the cross-girder web to the box-girder exterior web must transmit the shear from the box-girder web to the cross-girder web. A $\frac{1}{4}$-in. fillet weld on each side of the cross-girder webs is more than adequate. Details and calculations for this region follow.



CROSS-GIRDER CONNECTION AT BOX-GIRDER EXTERIOR WEB

Welds between Webs of Girders

The vertical shear in the box-girder web for maximum design loads is

$$V=\frac{1.3}{2}\left[218+53+\frac{5}{3}\times 138\right]=326 \text{ kips}$$

The shear along the slope of the web is

$$V'=\frac{58.69}{57}\times 326=336 \text{ kips}$$

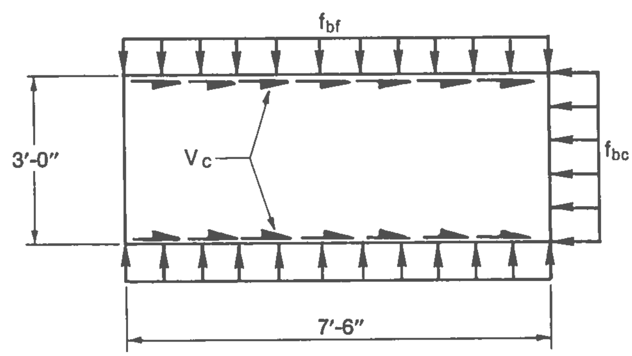
For two welds, the shear on each weld is $336/2=168$ kips. The length of the welds is $58.69-1.5\times 58.69/57=57$ in. Capacity of a weld is 18.5 ksi.

$$\text{Weld size required}=\frac{168}{57\times 18.5}=0.16 \text{ in.}$$

The minimum size of weld permitted for the ½-in. box-girder web is ¼-in. Therefore, use a ¼-in. fillet weld on each side of the cross-girder web at the junction with the exterior web of the box girder.

Biaxial Stresses at Interior Support

In the discussion earlier in this chapter of design of negative-moment sections of the box girders, it was pointed out that the bottom flange of those box girders is subjected to a biaxial state of stress at the cross-girder connection. Actually, the state of stress in the flange is complicated, as can be see from a sketch of the box-girder bottom flange at the connection. In the sketch:



PLAN OF BOTTOM FLANGE OF BOX GIRDER AT CROSS GIRDER

f_{bf} = longitudinal bending stress in the box girder under maximum design load

f_{bc} = bending stress in the bottom flange of the cross girder under maximum design load

V_c = shear stress delivered to the flange plate from the cross-girder webs under maximum design load

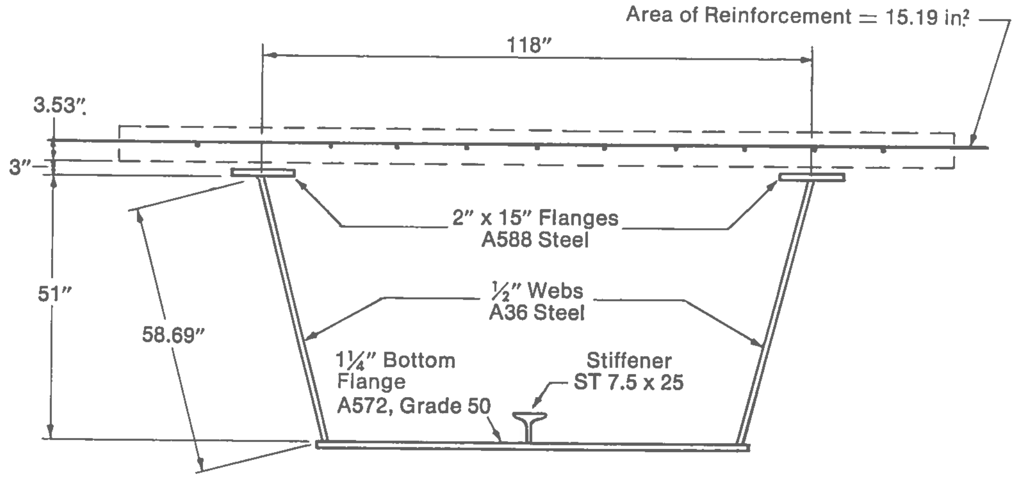
To insure stability of the bottom flange, the following interaction equation should be satisfied:

$$\frac{f_{bf}}{F_{bf}}+\frac{f_{bc}}{F_{bc}}\leq 1$$

where F_{bf} = critical buckling stress in the longitudinal direction of the box girder

F_{bc} = maximum allowable compressive stress in the cross-girder bottom flange

A drawing shows a section of a box girder at the interior support. The section is hybrid, with $F_y=50$ ksi for the flanges and $F_y=36$ ksi for the webs. A 1¼-in.-thick bottom flange with a single longitudinal ST7.5×25 stiffener is assumed.



BOX-GIRDER SECTION AT INTERIOR SUPPORT

For use in the interaction equation, the critical buckling stress of the bottom flange is determined without the reduction factor R normally employed in hybrid design. This approach is justified when the yield strength of the lower-strength webs of a hybrid section is not exceeded under maximum design loads.

Loading 1 produces the most critical state of biaxial stress in the bottom-flange plate at the cross-girder connection.

Moments in Box Girder 1.5 Ft from Interior Support

	DL_1	DL_2	$L+I$
M, kip-ft	5,950	1,350	2,211

Steel Section at Interior Support

Material	A	d	Ad	Ad^2	I_o	I
2 T. Flg. Pl. 2×15	60.00	29.50	1,770	52,215	20	52,235
2 Web Pl. ½×58.69	58.69				15,891	15,891
Bot. Flg. Pl. 1¼×92	115.00	−29.13	−3,350	97,584	15	97,599
Stiff. ST7.5×25	7.35	−23.25	−171	3,973	41	4,014
	241.04 in.²		−1,751 in.³			169,739

$$d_s = \frac{-1{,}751}{241.04} = -7.26 \text{ in.}$$

$$-7.26 \times 1{,}751 = -12{,}712$$

$$I_{NA} = 157{,}027 \text{ in.}^4$$

$$d_{\text{Top of steel}} = 30.50 + 7.26 = 37.76 \text{ in.} \qquad d_{\text{Bot. of steel}} = 29.75 - 7.26 = 22.49 \text{ in.}$$

$$S_{\text{Top of steel}} = \frac{157{,}027}{37.76} = 4{,}159 \text{ in.}^3 \qquad S_{\text{Bot. of steel}} = \frac{157{,}027}{22.49} = 6{,}982 \text{ in.}^3$$

$$d_{\text{Top of web}} = 28.50 + 7.26 = 35.76 \text{ in.}$$

$$S_{\text{Top of web}} = \frac{157{,}027}{35.76} = 4{,}391 \text{ in.}^3$$

Steel Section with Reinforcing Steel at Interior Support

Material	A	d	Ad	Ad^2	I_o	I
Steel Section	241.04		−1,751			169,739
Reinforcement	15.19	35.03	532	18,640		18,640
	256.23 in.²		−1,219 in.³			188,379

$$d_c = \frac{-1{,}219}{256.23} = -4.76 \text{ in.}$$

$$-4.76 \times 1{,}219 = -5{,}802$$

$$I_{NA} = 182{,}577 \text{ in.}^4$$

$$d_{\text{Top of steel}} = 30.50 + 4.76 = 35.26 \text{ in.} \qquad d_{\text{Bot. of steel}} = 29.75 - 4.76 = 24.99 \text{ in.}$$

$$S_{\text{Top of steel}} = \frac{182{,}577}{35.26} = 5{,}178 \text{ in.}^3 \qquad S_{\text{Bot. of steel}} = \frac{182{,}577}{24.99} = 7{,}306 \text{ in.}^3$$

$$d_{\text{Top of web}} = 28.50 + 4.76 = 33.26 \text{ in.}$$

$$S_{\text{Top of web}} = \frac{182{,}577}{33.26} = 5{,}489 \text{ in.}^3$$

Stresses 1.5 Ft from Interior Support Due to Maximum Design Loads

Top of Web

$$DL_1:\ F_b = \frac{5{,}950 \times 12}{4{,}391} \times 1.30 = 21.1$$

$$DL_2:\ F_b = \frac{1{,}350 \times 12}{5{,}489} \times 1.30 = 3.8$$

$$L+I:\ F_b = \frac{2{,}211 \times 12}{5{,}489} \times 1.30 \times \frac{5}{3} = 10.5$$

$$35.4 < 36 \text{ ksi}$$

Bottom of Steel

$$F_b = \frac{5{,}950 \times 12}{6{,}982} \times 1.30 = 13.3$$

$$F_b = \frac{1{,}350 \times 12}{7{,}306} \times 1.30 = 2.9$$

$$F_b = \frac{2{,}211 \times 12}{7{,}306} \times 1.30 \times \frac{5}{3} = 7.9$$

$$f_{bf} = 24.1 \text{ ksi}$$

Because the maximum bending stress in the web is less than $F_y = 36$ ksi for the web, no reduction in allowable stress is required for the hybrid section. From the preceding calculations, for use in the interaction equation, $f_{bf} = 24.1$ ksi.

For determination of the critical buckling stress in the bottom flange,

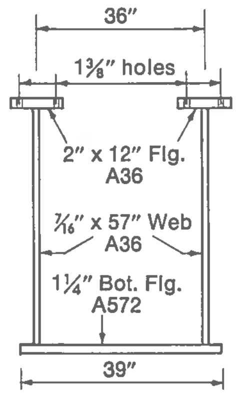
$$k = \sqrt[3]{\frac{8I_s}{wt^3}} = \sqrt[3]{\frac{8 \times 243.2}{45(1.25)^3}} = 2.81$$

The width-thickness ratio of the bottom flange is $w/t = 45/1.25 = 36$. From the curves for critical buckling stress presented previously, $F_{cr} = 44.3$ ksi $= F_{bf}$ in the interaction equation.

The bending moment in the cross girder at the edge of the box-girder bottom flange equals the product of the reaction R_A at the outer web of the box girder and the horizontal distance between the center of gravity of the web and the edge of the bottom flange. For maximum design loads and loading 1, this moment is

$$M = 651 \times 8.08 = 5{,}260 \text{ kip-ft}$$

Properties of the cross-girder section within the box girder are computed next. The width of the bottom flange of the section is set equal to the width of the cross-girder bottom flange between the box girder and the column.



SECTION OF CROSS GIRDER WITHIN BOX GIRDER

Cross-Girder Steel Section Within Box Girder

Material	A	d	Ad	Ad^2	I_o	I
2 T. Flg. Pl. 2×12	48.00	29.5	1,416	41,772	16	41,788
4 Holes	−11.00	29.5	−324	−9,573		−9,573
15% of Flange Area	7.20	29.5	212	6,266		6,266
2 Web Pl. ⁷⁄₁₆×57	49.88				13,504	
Bot. Flg. Pl. 1¼×39	48.75	−29.13	−1,420	41,367	6	41,373
	142.83 in.²		−116 in.³			93,358

$$d_s = \frac{-116}{142.83} = -0.81 \text{ in.} \qquad -0.81 \times 116 = -94$$

$$I_{NA} = 93{,}264 \text{ in.}^4$$

$$d_{\text{Top of steel}} = 30.50 + 0.81 = 31.31 \text{ in.} \qquad d_{\text{Bot. of steel}} = 29.75 - 0.81 = 28.94 \text{ in.}$$

$$S_{\text{Top of steel}} = \frac{93{,}263}{31.31} = 2{,}979 \text{ in.}^3 \qquad S_{\text{Bot. of steel}} = \frac{93{,}264}{28.94} = 3{,}223 \text{ in.}^3$$

$$d_{\text{Top of web}} = 28.50 + 0.81 = 29.31 \text{ in.}$$

$$S_{\text{Top of web}} = \frac{93{,}264}{29.31} = 3{,}182 \text{ in.}^3$$

Stresses in Cross Girder Due to Maximum Design Loads

Top of Web

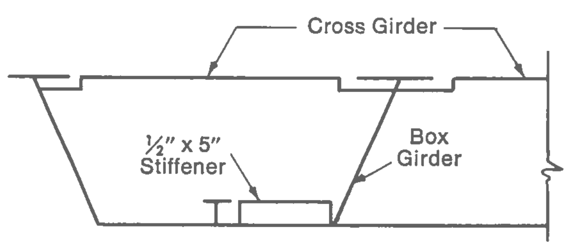
$$F_b = \frac{5{,}269 \times 12}{3{,}182} = 19.8 < 36 \text{ ksi}$$

Bottom of Steel

$$F_b = f_{bc} = \frac{5{,}260 \times 12}{3{,}223} = 19.6 \text{ ksi}$$

The maximum bending stress in the cross-girder web is less than $F_y = 36$ ksi. Hence, no reduction in allowable flange stress is required. From the preceding calculations, for use in the interaction equation, $f_{bc} = 19.6$ ksi.

To increase the stiffness of the cross-girder bottom flange, a ½×5-in. plate, to serve as a longitudinal stiffener, is fillet welded to the bottom flange between the box-girder longitudinal stiffener and the box-girder web.



STIFFENER ON CROSS GIRDER WITHIN BOX GIRDER

The moment of inertia of the stiffener is

$$I_s = \frac{0.5(5)^3}{3} = 20.8 \text{ in.}^4$$

For calculation of the critical buckling stress in the bottom flange,

$$k = \sqrt[3]{\frac{8I_s}{wt^3}} = \sqrt[3]{\frac{8 \times 20.8}{18(1.25)^3}} = 1.68$$

The width-thickness ratio of the bottom flange is 18/1.25 = 14.4. From the curves for critical buckling stress, $F_{cr} = 50$ ksi $= F_{bc}$ in the interaction equation.

All required stresses for the interaction equation have now been determined. For the bottom flange plate then,

$$\frac{f_{bf}}{F_{bf}} + \frac{f_{bc}}{F_{bc}} = \frac{24.1}{44.3} + \frac{19.6}{50} = 0.936 < 1$$

Check of Shear in Bottom Flange

The cross-girder webs apply shear to the cross-girder bottom flange. The shear flow in each web of the cross girder at the junction of the webs and flange is

$$S = \frac{651 \times 39 \times 1.25 \times 28.32}{2 \times 93{,}264} = 4.81 \text{ kips per in.}$$

The corresponding flange shear is

$$v_c = \frac{4.81}{1.25} = 3.85 \text{ ksi}$$

The maximum allowable bottom-flange shear is assumed to be given by

$$F_v = 0.58F_y = 0.58 \times 50 = 29.0 > 3.85 \text{ ksi}$$

Check of Fatigue in Bottom Flange

Fatigue should be checked in the bottom flange at the end of the fillet welds for the cross-girder longitudinal stiffener. The allowable stress range for 100,000 cycles of lane loading is 21 ksi. In the direction of the stiffener, the range of live-load moment due to service loads is the product of the change in box-girder reaction R_A and the horizontal distance between the center of gravity of the web and the edge of the bottom flange nearest the column.

$$M_L = 138 \times 8.08 = 1{,}115 \text{ kip-ft}$$

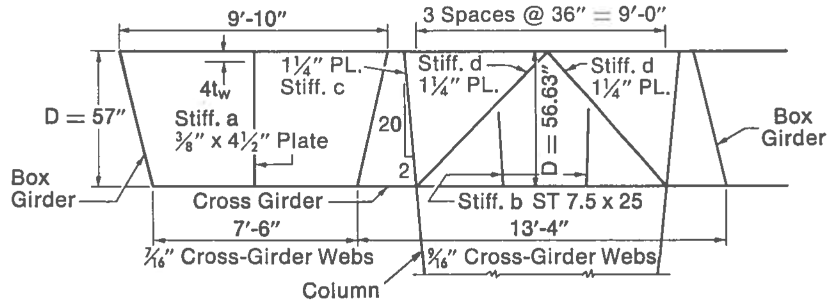
The corresponding stress range in the bottom flange at the end of the stiffener fillet welds is

$$f_{sr} = \frac{1{,}115 \times 12 \times 27.69}{93{,}264} = 3.97 < 21 \text{ ksi}$$

The cross-girder section therefore is satisfactory.

Cross-Girder Transverse Web Stiffeners

The transverse web stiffeners for the cross girder are designed next. Calculations are made for four stiffeners, designated *a*, *b*, *c* and *d*.



CROSS-GIRDER TRANSVERSE STIFFENERS

Stiffener *a*

The shear under Loading 1 is found to exceed the buckling capacity of the unstiffened 7/16-in. web of the cross girder. Hence, transverse stiffeners are necessary. A single stiffener *a* is tried at mid-width of the box girder.

The shear per web at the stiffener location is

$$V=\frac{R_A}{2}=\frac{651}{2}=362 \text{ kips}$$

The shear capacity of the 7/16-in. web is

$$V_b=\frac{3.5\times 29{,}000(0.44)^3}{57}=152<326 \text{ kips}$$

Therefore, a stiffener is necessary. A stiffener at mid-width of the box girder satisfies the spacing requirement for the first stiffener near the end of the cross girder. A plate 3/8 × 4½ in. provides a satisfactory section for the stiffener.

The maximum permissible spacing for the stiffener is

$$d_o=14{,}500\sqrt{\frac{Dt_w^3}{V}}=14{,}500\sqrt{\frac{57(0.44)^3}{326{,}000}}=56.0 \text{ in.}$$

The actual spacing measured from the end of the cross girder at mid-depth is

$$d_o=\tfrac{1}{2}\times\frac{118+90}{2}=52<56 \text{ in.}$$

For determination of the ultimate shear capacity,

$$C=18{,}000\times\frac{0.44}{57}\sqrt{\frac{1+(57/52)^2}{36{,}000}}-0.3=0.786<1$$

$$V_p=0.58F_yDt_w=0.58\times 36\times 57\times 0.44=524 \text{ kips}$$

$$V_u=V_p\left[C+\frac{0.87(1-C)}{\sqrt{1+(d_o/D)^2}}\right]=524\left[0.786+\frac{0.87(1-0.786)}{\sqrt{1+(52/57)^2}}\right]=484>326 \text{ kips}$$

The depth-thickness ratio of the web is limited by

$$\frac{D}{t_w}=\frac{36{,}500}{\sqrt{F_y}}=\frac{36{,}500}{\sqrt{36{,}000}}=192$$

The actual web depth-thickness ratio is $57/0.44=130<192$.

Required area of stiffener is

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

where $B=2.4$ for a single-plate stiffener

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

$$A=\frac{36}{36}\left[0.15\times 2.4\times 57\times 0.44(1-0.786)\frac{326}{484}-18(0.44)^2\right]=-2.18 \text{ in.}^2$$

The negative result indicates that the web contribution is larger than the required area of stiffener.

The width-thickness ratio of the 3/8 × 4½-in. stiffener plate is

$$\frac{b'}{t}=\frac{4.5}{3/8}=12$$

The maximum permissible ratio is

$$\frac{b'}{t}=\frac{2{,}600}{\sqrt{F_y}}=\frac{2{,}600}{\sqrt{36{,}000}}=13.7>12$$

The moment of inertia of the stiffener plate about the edge connected to the web is

$$I = \frac{0.375(4.5)^3}{3} = 11.4 \text{ in.}^4$$

The minimum moment of inertia required is computed as follows:

$$J = 2.5\left(\frac{D}{d_o}\right)^2 - 2 = 2.5\left(\frac{57}{52}\right)^2 - 2 = 1.0$$

$$I = d_o t_w^3 J = 52(0.44)^3 1.0 = 4.43 < 11.4 \text{ in.}_1$$

Stiffener *b*

In design of stiffener *b*, which is placed over the column, the presence of stiffener *d*, which is placed diagonally over the column, will be ignored. Also, it is assumed that stiffener *b* traverses the full height of the panel, 56.63 in. Because it has already been shown that a spacing of 36 in. provides adequate shear capacity for the region of the cross girder over the pier, only the required properties of this stiffener need be calculated.

For the stiffener, assume that an ST7.5×25 stiffener of the column is extended above the column. The ST7.5×25 provides a moment of inertia equal to

$$I = 40.6 + 7.35(5.25)^2 = 243 \text{ in.}^4$$

The required moment of inertia is calculated as follows:

$$J = 2.5\left(\frac{56.63}{36}\right)^2 - 2 = 4.19$$

$$I = 36(0.56)^3 4.19 = 26.5 < 243$$

Hence, the ST7.5×25 is satisfactory.

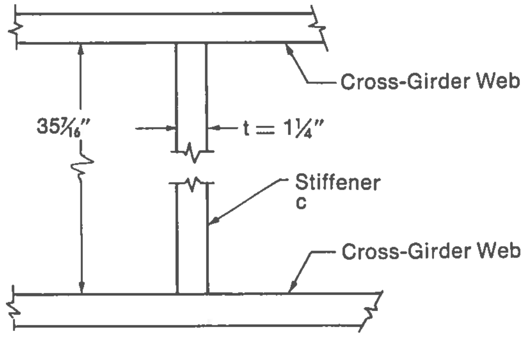
Stiffener *c*

Stiffener *c* is, in effect, an extension of an unstiffened side of the column. A solid diaphragm is used for the stiffener. It serves both as a bearing stiffener, to transfer the load from the cross girder to the column, and as a transverse stiffener of the $\frac{9}{16}$-in. cross-girder web. Stiffener *c* is assumed to transmit all the load from the cross girder into the column, because of the tendency of the cross girder to rotate about the column face under negative moment. (The assumption is conservative, because some of the cross-girder load will be transferred into the column through the cross-girder web.)

The stiffener is designed as a column to carry maximum design loads. It is checked for local buckling with the width-thickness-ratio criterion for bottom compression flanges of a box girder.

For the stiffener, try a $1\frac{1}{4}$-in.-thick plate. Width of the diaphragm is $36 - \frac{9}{16} = 35\frac{7}{16}$ in. Width-thickness ratio is $35.44/1.25 = 28.4$. The maximum permissible ratio is

$$\frac{b}{t} = \frac{6{,}140}{\sqrt{F_y}} = \frac{6{,}140}{\sqrt{36{,}000}} = 32.4 > 28.4$$



HORIZONTAL SECTION THROUGH CROSS GIRDER AT STIFFENER *c*

Because the region of the cross girder in which stiffener c is located is subject to high shear and bending, the web will not be included with the stiffener as part of a column. The area of the stiffener alone is $1.25 \times 35.44 = 44.3$ in. The moment of inertia of the stiffener is

$$I = \frac{1.25(35.44)^3}{12} = 4{,}637 \text{ in.}^4$$

The radius of gyration of the stiffener is

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{4{,}637}{44.3}} = 10.23 \text{ in.}$$

The length of the diaphragm is

$$L' = 56.63 - 4 \times 9/16 = 54.38 \text{ in.}$$

Consequently, the slenderness ratio of the stiffener is

$$\frac{L'}{r} = \frac{54.38}{10.23} = 1.23$$

The critical strength of the stiffener as a column then is

$$F_{cr} = F_y\left[1 - \frac{F_y}{4\pi^2 E}\left(\frac{L'}{r}\right)^2\right] = 36\left[1 - \frac{36}{4\pi^2 \times 29{,}000}(1.23)^2\right] = 36.0 \text{ ksi}$$

For Loading 1, the shear at the face of the column is 1,237 kips. The capacity of the stiffener as a column is

$$P_u = 0.85AF_{cr} = 0.85 \times 44.3 \times 36.0 = 1{,}356 > 1{,}237 \text{ kips}$$

The 1¼-in. plate is satisfactory.

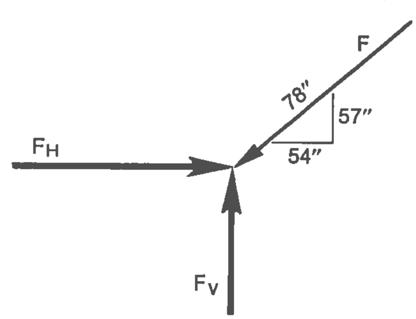
Stiffener *d*

The cross-girder web near the column face is subject to shear, bending and axial compression stresses simultaneously from two directions. The high principal stresses that would normally occur in this region can be reduced, however, by use of a compression stiffener that acts like a truss diagonal. Hence, stiffener d is incorporated as an inclined, solid diaphragm. This plate stiffens the cross-girder web and reduces principal web stresses in the combined-stress region. In design of such a member, the assumption is made that the flange forces carried by the cross girder and the unstiffened side of the column are resisted by compression in the diagonal stiffener. For computation of these forces, the web is neglected and the structure is assumed to behave essentially as a truss.

For stiffener d, try a 1¼-in. plate. Width of the diaphragm is 35.44 in. and length is

$$L' = \sqrt{\left(\frac{108}{2}\right)^2 + 56.63^2} = 78.2 \text{ in.}$$

As for stiffener c, the area of the stiffener is 44.3 in.², $r = 10.23$ in. and $L'/r = 7.64$.



FORCES AT STIFFENER *d*

The force in the bottom flange of the cross girder is

$$F_{cg}=f_bA=28.1\times1.25\times39=1{,}370 \text{ kips}$$

The force in the unstiffened side of the column is

$$F_{col}=f_bA=19.5\times1\times39=761 \text{ kips}$$

Assume that stiffener d carries the smaller of the following:

One-half of the horizontal load F_{cg}. (The cross-girder bottom flange above the column carries the other half.)

$$F=\frac{78.2}{54}F_H=\frac{78.2}{54}\times\frac{1{,}370}{2}=992 \text{ kips}$$

The entire vertical load on the unstiffened side of the column.

$$F=\frac{78.2}{56.63}F_v=\frac{78.2}{56.63}\times761=1{,}051>992 \text{ kips}$$

The stiffener is then designed as a column in the same manner as stiffener c. Again, the web is neglected in determining the radius of gyration. The critical stress in the stiffener as a column is

$$F_{cr}=F_y\left[1-\frac{F_y}{4\pi^2E}\left(\frac{L'}{r}\right)^2\right]=36\left[1-\frac{36}{4\pi^2\times29{,}000}(7.64)^2\right]=35.9 \text{ ksi}$$

The capacity of the stiffener therefore is

$$P_u=0.85AF_{cr}=0.85\times44.3\times35.9=1{,}352>992 \text{ kips}$$

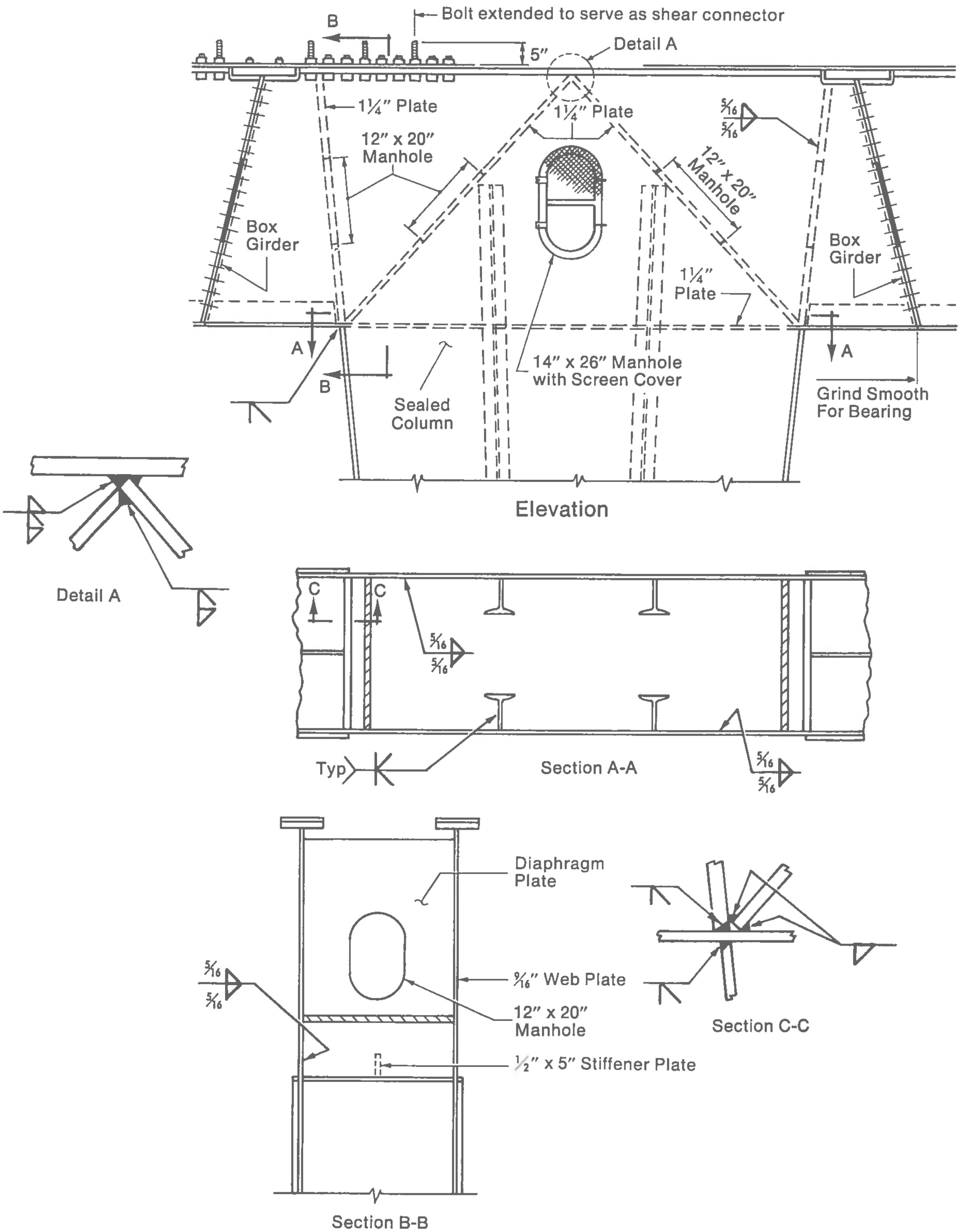
The 1¼-in. plate is satisfactory.

Access to Cross-Girder Interior

Provision should be made for access to the cross-girder interior for inspection and maintenance. There is no need to provide access to the column interior, because the column will be fabricated as a completely sealed unit.

For entrance into the cross girder, a 14×26-in., screen-covered manhole is centered between the box girders in the 9/16-in. web of the cross girder. Also, open 12×20-in. manholes are provided at the center of stiffeners c and d and in the box-girder interior webs.

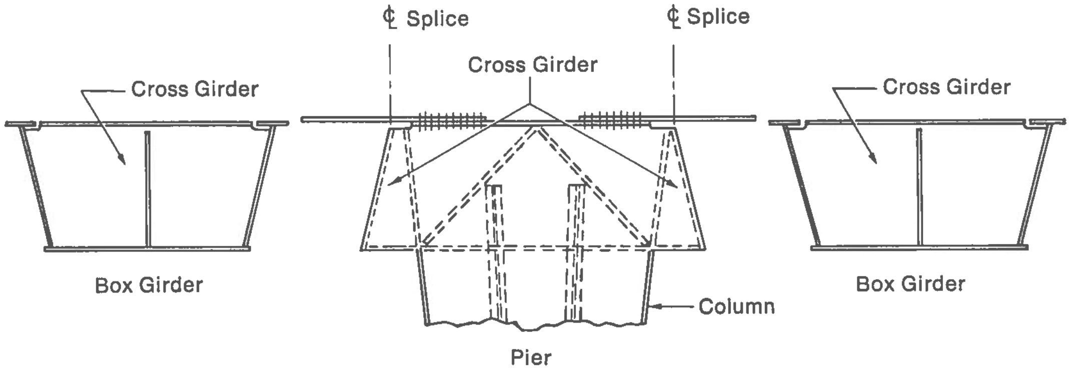
The 1¼-in.-thick stiffeners are assumed to be one-third unloaded at the manhole. Thus, 67% of the load is carried by the net section, which is $100(35.44-12)/35.44=66\%$ of the gross section. As a result, no increase in the stiffener thickness is necessary to make up for the loss in section because of the opening. Details of the region are shown in a drawing.



CROSS-GIRDER SECTIONS AT COLUMN

FIELD SPLICE

As shown in a drawing, the column and the portion of the cross girder directly above it are shop fabricated for erection as a unit. When the box girders are fabricated, the portion of the cross girder within them is incorporated. The remainder of the cross girder is connected between the box girders and the pier with field splices.



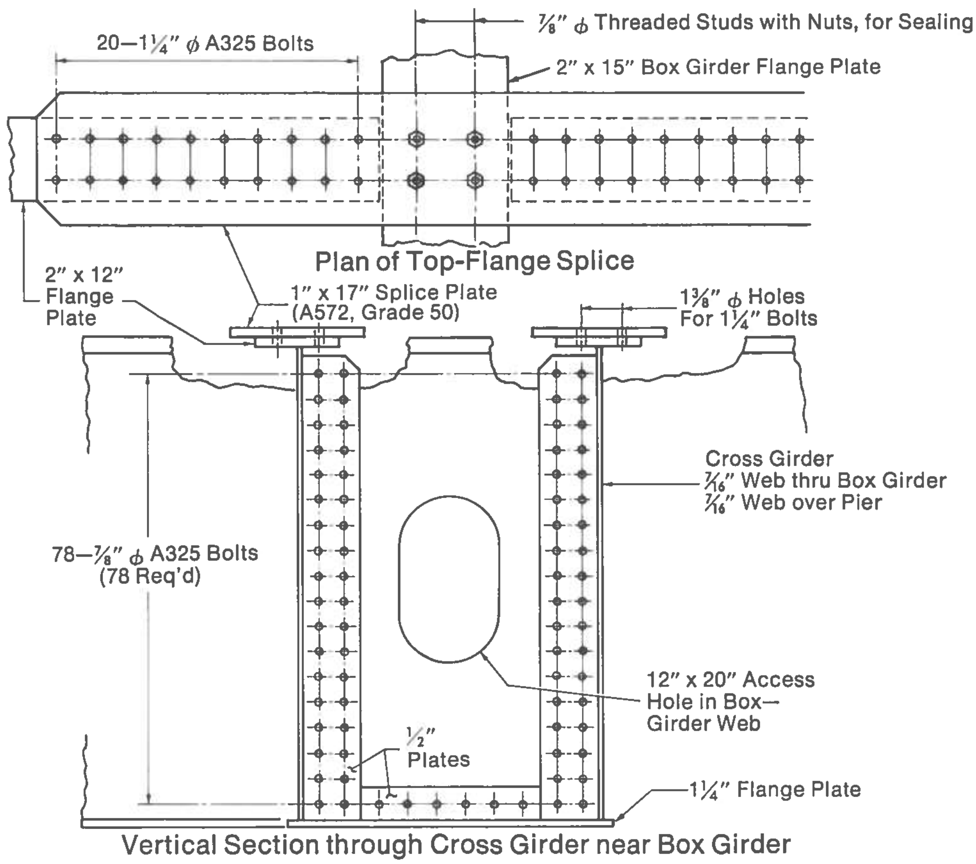
FIELD-SPLICE SCHEME—BOX GIRDERS TO PIER

In design of the field splices, it is assumed that all the bending moment is taken by the flange splices and that all of the shear is taken by the connection of the box-girder web to the cross-girder webs.

For the top-flange splice, a splice plate connecting the pier and box-girder segments passes over the top flange of the box girder but is not attached to it structurally. The flange of the box girder passes through this region without interruption.

At the bottom flange, a positive connection is not needed. Because this flange is always in compression, the plates may be simply butted together, and the stress is transferred in bearing. This joint will be discussed later.

The field splice is designed in the same manner as the field splice for the box girder, described previously. The splice material is proportioned to carry the larger of 75% of the member capacity or the average of the member capacity and the bending moment due to maximum design loads. The fasteners are designed for overload, with a maximum shear stress of 21 ksi. Finally, the splice material is investigated for fatigue in base metal adjacent to friction-type fasteners. Details of the splice are shown in a drawing.



CROSS-GIRDER BOLTED FIELD SPLICE

Design of the splice begins with a tabulation of the applied shears and bending moments. The moments are computed for a section through the middle of the inner top flange of the box girder. The box-girder reactions R_A and R_B are assumed to act at middepth of the box-girder webs. Loading 1 controls for connector and splice plate designs.

Service Loads

	Shear, Kips	Moment, Kip-ft
DL_1:	$218+218=436$	$218\times9.25+218\times0.58=2{,}143$
DL_2:	$53+53=106$	$53\times9.25+53\times0.58=521$
$L+I$:	$138+108=246$	$138\times9.25+108\times0.58=1{,}339$
	788	4,003

Maximum Design Loads: $1.30[D+5/3(L+I)]$

	Shear, Kips	Moment, Ft-kips
DL_1:	$436\times1.30=567$	$2{,}143\times1.30=2{,}786$
DL_2:	$106\times1.30=138$	$521\times1.30=677$
$L+I$:	$246\times1.30\times\frac{5}{3}=533$	$1{,}339\times1.30\times\frac{5}{3}=2{,}901$
	1,238	6,364

To determine the design moment for the splice, the moment capacity of the section is calculated. It is controlled by the section modulus and allowable stress for the top flange.

Maximum Strength of Member

Previous calculations indicated that at the section at the face of the column the cross girder has a section modulus $S=3{,}159$ in.^{3} The moment capacity of the section therefore is

$$M_u=\frac{36\times3{,}159}{12}=9{,}477 \text{ kip-ft}$$

$$0.75M_u=7{,}108 \text{ kip-ft}$$

The design moment is the larger of 75% of the moment capacity or the average of this capacity and the moment due to the Maximum Design Load.

$$M_{av}=\frac{9{,}477+6{,}364}{2}=7{,}921>7{,}108 \text{ kip-ft}$$

The design moment for the splice therefore is 7,921 kip-ft.

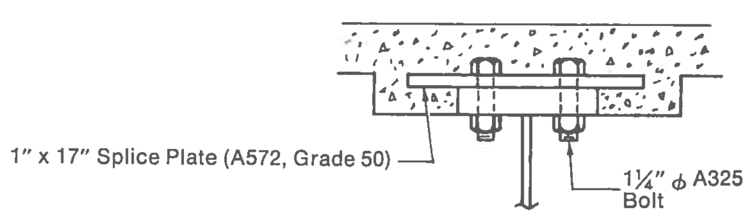
Design of Bolted Top-Flange Splice Plates

The two top-flange splice plates are to be made of A572, Grade 50, steel. The splice-plate area required for each flange is

$$A=\frac{M_{av}}{F_yd}=\tfrac{1}{2}\times\frac{7{,}921\times12}{50\times59.75}=15.9 \text{ in.}^2$$

Try a 1×17-in. plate on each flange with a gross area per plate of 17-in.^{2} The net area is

$$A=17-(2\times1\tfrac{3}{8}\times1-0.15\times17)=16.8>15.9 \text{ in.}^2$$



SECTION THROUGH TOP-FLANGE SPLICE

The number of 1¼-in.-dia, A325 bolts required in the flange splice for Overload is determined next. The Overload moment $D+5/3(L+I)=2{,}143+521+5/3\times1{,}339=4{,}896$ kip-ft, or $4{,}896/2=2{,}448$ kip-ft per flange. The force on the flange is

$$F=\frac{M}{d}=\frac{2{,}448\times12}{58.25}=504 \text{ kips}$$

Allowable stress on a bolt under Overload is 21 ksi. The bolt area is 1.23 in.^{2} The total number of bolts required then is

$$N=\frac{504}{21\times1.23}=20 \text{ bolts}$$

Use twenty 1¼-in.-dia bolts on each side of the joint.

Web Splice

The connection of the cross girder to the box-girder web is assumed to carry all the shear on the splice but no bending moment. As shown in the drawing of the cross-girder field splice, ½-in. connection plates are welded to the webs and bottom flange of the cross girder. These plates are to be field bolted to the box-girder interior web at the field splice.

The Overload shear $D+5/3(L+I)=436+106+5/3\times246=952$ kips. The shear on the sloped box-girder web then is

$$V'=952\times\frac{58.69}{57}=980 \text{ kips}$$

For the connection, ⅞-in.-dia, A325 bolts will be used. Bolt area is 0.60 in.^{2} Allowable stress in the bolts for Overload is 21 ksi. The number of bolts required is

$$N=\frac{980}{21\times0.60}=77 \text{ bolts}$$

Use 78 bolts, ⅞ in. in diameter, in two rows of 18 bolts each along each cross-girder web and six along the bottom flange.

Check of Fatigue in Bolted Top-Flange Splice

Fatigue under Service Loads is checked in the metal adjacent to friction-type fasteners in the top-flange splice plates. Fatigue for this condition is classified by AASHTO as Category B. For 100,000 cycles of lane loading, the associated allowable stress range is 45 ksi. The range of live-load moment at the field splice is

$$M_L=1{,}339-0=1{,}339 \text{ kip-ft}$$

The range of force in a flange splice plate is

$$F_r=\frac{M_L}{d}=\frac{1}{2}\times\frac{1{,}339\times12}{59.75}=135 \text{ kips}$$

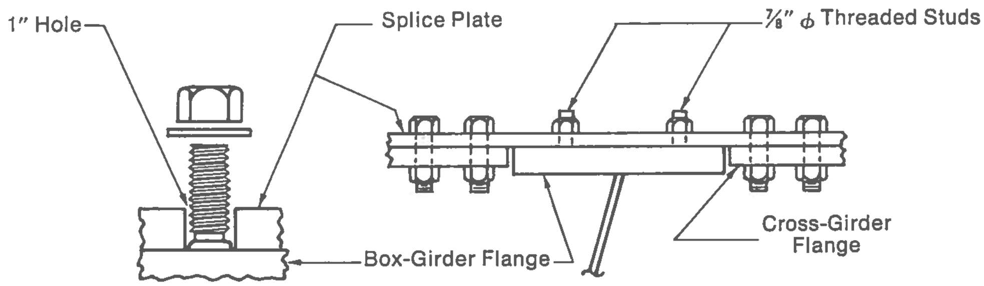
The actual stress range in the gross section of the splice plate therefore is

$$f_{br}=\frac{F_r}{A}=\frac{135}{17\times1}=7.9<45 \text{ ksi}$$

The plate is satisfactory for fatigue.

Sealing Studs

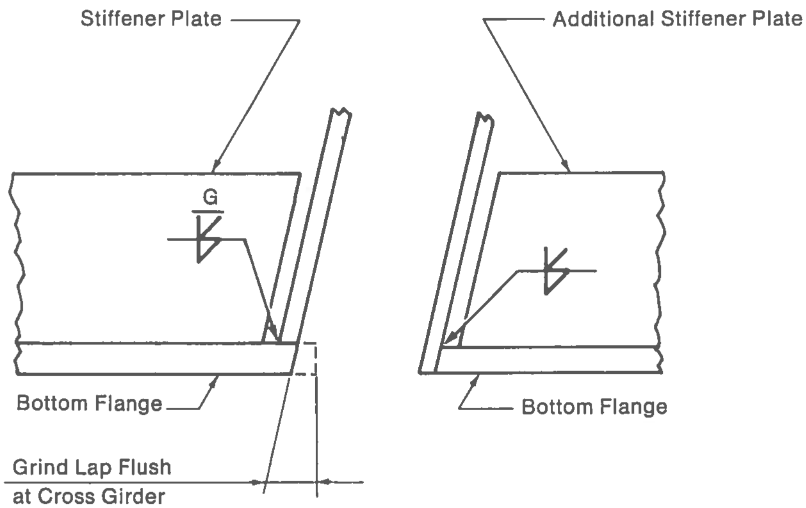
For sealing purposes, four ⅞-in.-dia, 2½-in.-long, threaded studs are placed on the upper side of the box-girder flange for bolting to the splice plate, as shown in a drawing.



SEALING STUDS ON BOX-GIRDER TOP FLANGE

Bottom-Flange Joint

As noted previously, the bottom flange is always in compression, so that a positive connection between the cross-girder and box-girder bottom flanges is not needed. The flange compressive force is assumed to be transmitted in bearing. For the purpose, abutting plate edges are ground smooth.



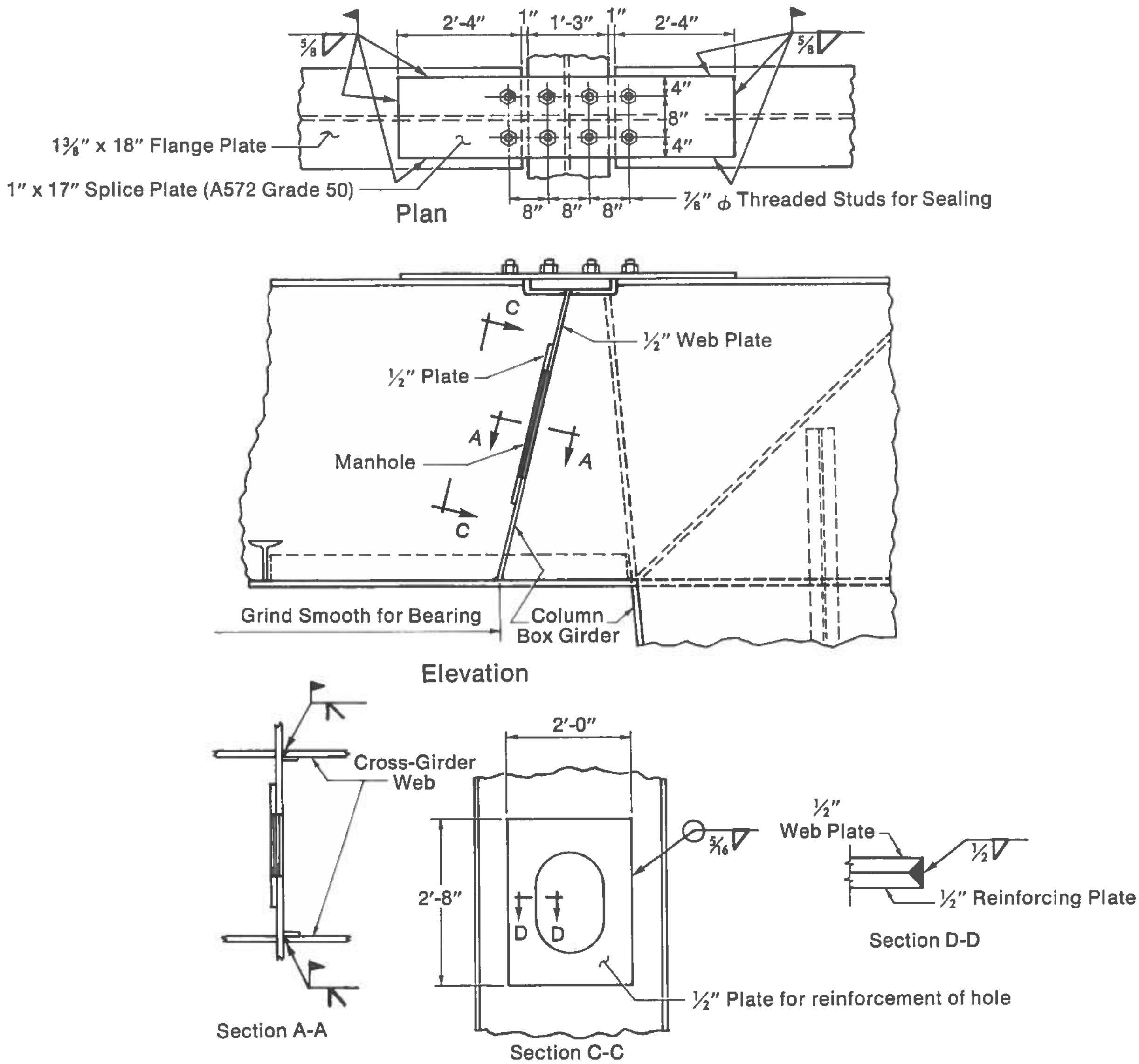
DETAIL AT BOTTOM FLANGE

To guard against buckling due to possible nonuniform bearing stress in the abutting flange plates, an additional ½ × 5-in. stiffener plate is provided at midwidth of the cross-girder bottom flange between the inner web of the box girder and the face of the column.

Alternative Welded Field Splice

As an alternative to the bolted splice, a welded splice may be used, as shown in a drawing.

Welding to the side of the top flange of the box girder creates an undesirable fatigue condition. To avoid it, the welded splice utilizes at the top flange a single 1 × 17-in. splice plate, fillet welded to the cross-girder flanges on each side of the splice and passing over but not connected to the box-girder flange. The cross-girder flanges are widened from 12 to 18 in. to accommodate the 17-in.-wide splice plates. To maintain about the same flange area, a 1⅜ × 18-in. plate is used for the flanges. Eight ⅞-in.-dia, threaded studs are used for sealing purposes over the box-girder flange.



WELDED FIELD SPLICE

Design of Flange-Splice Weld

For the design moment for the splice of 7,921 kip-ft, the force on the flange splice plate is

$$F = \tfrac{1}{2} \times \frac{7{,}921 \times 12}{59.12} = 804 \text{ kips}$$

For a weld along each 28-in.-long side and along the 17-in.-wide end of the splice plate, the shear flow is

$$S = \frac{804}{2 \times 28 + 17} = 11.0 \text{ kips per in.}$$

The weld capacity is $0.45F_u \times 0.707 = 0.45 \times 58 \times 0.707 = 18.5$ ksi.

$$\text{Weld size required} = \frac{11.0}{18.5} = 0.60 \text{ in.}$$

Use a ⅝-in. fillet weld.

Check of Fatigue in Welded Top-Flange Splice

Fatigue under Service Loads is investigated at the splice plate fillet weld. Fatigue in base metal adjacent to a transverse flange fillet weld is classified by AASHTO as Category E. For 100,000 cycles of lane loading, the allowable stress range is 21 ksi. The range of force in the flange at the splice plate is

$$F_r = \frac{M_L}{d} = \tfrac{1}{2} \times \frac{1{,}339 \times 12}{57.94} = 139 \text{ kips}$$

The actual stress range in the flange at the fillet weld is

$$f_{sr} = \frac{139}{0.375 \times 18} = 5.6 < 21 \text{ ksi}$$

Fatigue stress range in the longitudinal fillet weld is limited to 15 ksi for 100,000 cycles of lane loading. Actual stress range in the fillet weld is

$$f_{sr} = \frac{139}{0.625 \times 0.707(2 \times 28 + 17)} = 4.31 < 15$$

The splice is satisfactory in fatigue.

Other Welded-Splice Details

The web splice is made with full-penetration butt welds.

As with the bolted design, the connection of the bottom flanges of the box girders to the bottom flange of the cross girder is an unwelded butt joint, stiffened to prevent buckling from possible uneven bearing pressure.

The welded splice is completed by reinforcing the region around the 12×20-in. manhole in the box-girder web.

PIER ALTERNATIVE—REINFORCED CONCRETE

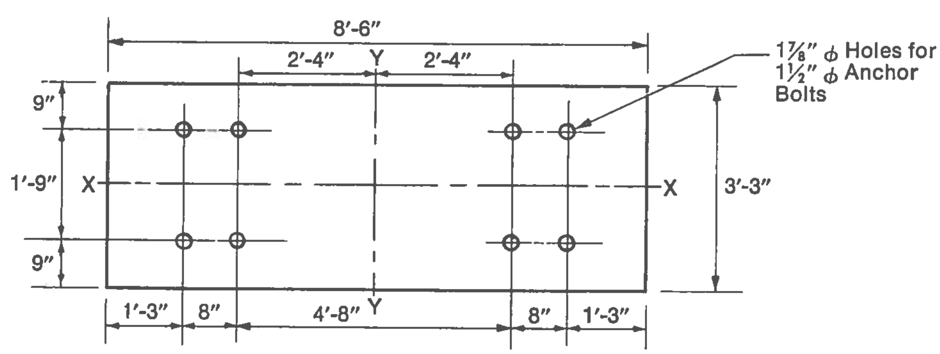
Next, an alternative pier design is prepared for reinforced concrete with the working-stress method of design. With this type of pier, the bottom flange of the cross girder is greatly increased in thickness in the region over the pier. Serving primarily as a masonry plate, the bottom flange transfers the load from the cross girder to the pier in bearing and is restrained against uplift by anchor bolts embedded in the pier concrete.

The design of the cross girder is similar to that previously covered in the steel-pier design calculations and is not treated in the following. One notable difference in the cross girder of the alternative design is that a thicker web is employed over the pier instead of a diagonal stiffener. High principal stresses in the cross-girder webs require the use of additional material.

The dimensions of the concrete pier at the top are set at 9 ft by 4.75 ft to accommodate a masonry plate of 8.5 ft by 3.25 in.

Design of Anchor Bolts

Anchor bolts are designed for maximum uplift forces under the larger of the loadings $1.5(D+L+I)$ or $D+2(L+I)$. Allowable stresses for elements designed for either of these loadings may be increased by 50%. Eight anchor bolts are used, as shown in a drawing.



PLAN OF MASONRY PLATE

The net area of the masonry plate is

$$A=102\times39-8\times2.76=3{,}956\ \text{in.}^2$$

The moment of inertia and section modulus of the plate with respect to the Y-Y axis are

$$I_y=\frac{39(102)^3}{12}-4\times2.76(28)^2-4\times2.76(36)^2=3{,}426{,}000\ \text{in.}^4$$

$$S_y=\frac{3{,}426{,}000}{51}=67{,}176\ \text{in.}^3$$

Group I loading, Case 3, controls the anchor-bolt design. The anchorage should be capable of resisting the larger of the following:

150% of the calculated uplift caused by Service Loads: $D+L+I$.

100% of the calculated uplift with double the live plus impact loads: $D+2(L+I)$.

Loads at Top of Concrete Pier

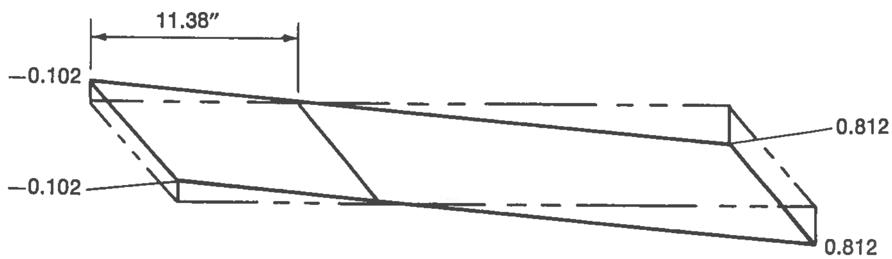
	DL_1	DL_2	$L+I$	**Total**
P, kips	891	211	301	1,403
M_y, kip-ft			2,556	2,556

In the table, DL_1 includes 20 kips for the weight of the cross girder.

Stresses beneath the cross-girder masonry plate are then computed as follows (minus indicates uplift):

Under $D+L+I$,

$$f_b=\frac{P}{A}\pm\frac{M_y}{S_y}=\frac{1{,}403}{3{,}956}\pm\frac{2{,}556\times12}{67{,}176}=0.355\pm0.457=-0.102;\ 0.812\ \text{ksi}$$



STRESSES UNDER MASONRY PLATE FOR $D+L+I$

The distance of the neutral axis from the uplift end is

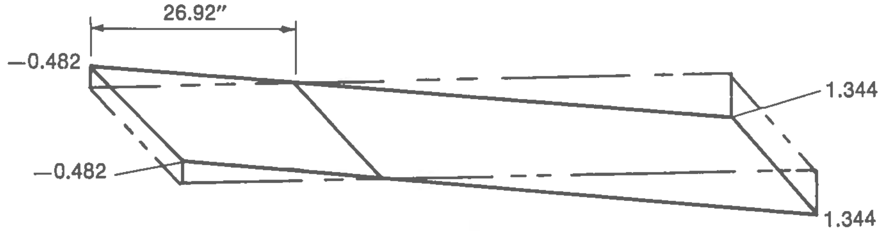
$$d_u=\frac{0.102}{0.102+0.812}\times102=11.38\ \text{in.}$$

The uplift force for $D+L+I$ is

$$F_{up}=\tfrac{1}{2}\times0.102\times11.38\times39=22.6\ \text{kips}$$
$$1.5F_{up}=1.5\times22.6=33.9\ \text{kips}$$

Under $D+2(L+I)$,

$$f_b=\frac{1{,}403+301}{3{,}956}\pm\frac{2\times2{,}556\times12}{67{,}176}=0.431\pm0.913=-0.482;\ 1.344\ \text{ksi}$$



STRESSES UNDER MASONRY PLATE FOR $D+2(L+I)$

The distance of the neutral axis from the uplift end is

$$d_u = \frac{0.482}{1.344+0.482} \times 102 = 26.92 \text{ in.}$$

The uplift force for $D+2(L+I)$ is

$$F_{up} = \tfrac{1}{2} \times 0.482 \times 26.92 \times 39 = 253 > 33.9 \text{ kips}$$

The anchorage should therefore be designed to resist an uplift of 253 kips.

The 1½-in.-dia, A325 anchor bolts have a yield strength of 105 ksi and may be pretensioned up to 70% of this. After being pretensioned, the bolts can sustain additional service-load tensile stress up to 36 ksi.

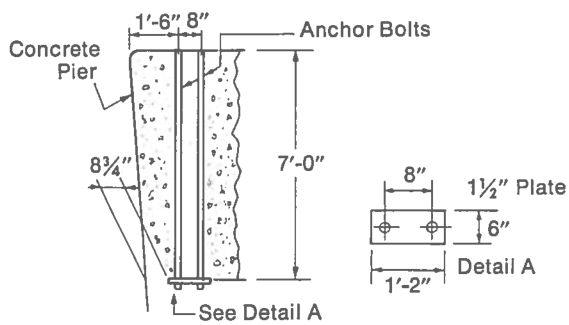
To eliminate uplift of the cross-girder masonry plate, each bolt should be pretensioned to 253/4 = 63 kips, say 65 kips. The bolt pretension stress then is

$$f_t = \frac{65}{1.77} = 36.7 < (0.7 \times 105 = 73.5 \text{ ksi})$$

Additional bolt tension under uplift is

$$f_t = \frac{253}{4 \times 1.77} = 35.7 < (1.5 \times 36 = 54 \text{ ksi})$$

A sufficient length of the anchor bolts should be embedded in the concrete of the pier to develop the uplift force in the bolts.



ANCHOR BOLTS IN PIER

Assume that the pier has no shear reinforcement. The allowable shear stress then is, for 4,000-psi concrete,

$$v_c = 1.8\sqrt{f'_c} = 1.8\sqrt{4{,}000} = 114 \text{ psi}$$

This may be increased 50% for the loading $D+2(L+I)$ to $1.5 \times 114 = 171$ psi. The maximum load from four anchor bolts for this loading is

Prestress:	4 × 65 = 260
Uplift:	253
Total:	513 kips

The embedment length required for the anchor bolts then is

$$L = \frac{513{,}000}{171 \times 39} = 76.9 \text{ in.}$$

Use a 7-ft embedment length for the anchor bolts.

Check of Bearing Stresses on Concrete Pier

AASHTO Specifications state that an allowable concrete bearing stress of $0.3f'_c$ may be used for the loaded area. When the supporting surface is wider on all sides than the loaded area, however, the allowable stress may be increased by a factor equal to the square root of the ratio of supporting area to loaded area, but not more than two. When the loaded area is subject to high edge stresses, the allowable bearing stress should be also multiplied by 0.75.

The allowable bearing stress for 4,000-psi concrete not subject to high edge stresses is for the 3-ft 9-in. × 9-ft pier top and 8-ft 6-in. × 3-ft 3-in. masonry plate:

$$F_b = 0.3 \times 4{,}000\sqrt{\frac{3.75 \times 9.0}{3.25 \times 8.5}} = 1{,}326 \text{ psi}$$

For high edge stresses, however,

$$F_b = 0.75 \times 1{,}326 = 995 \text{ psi}$$

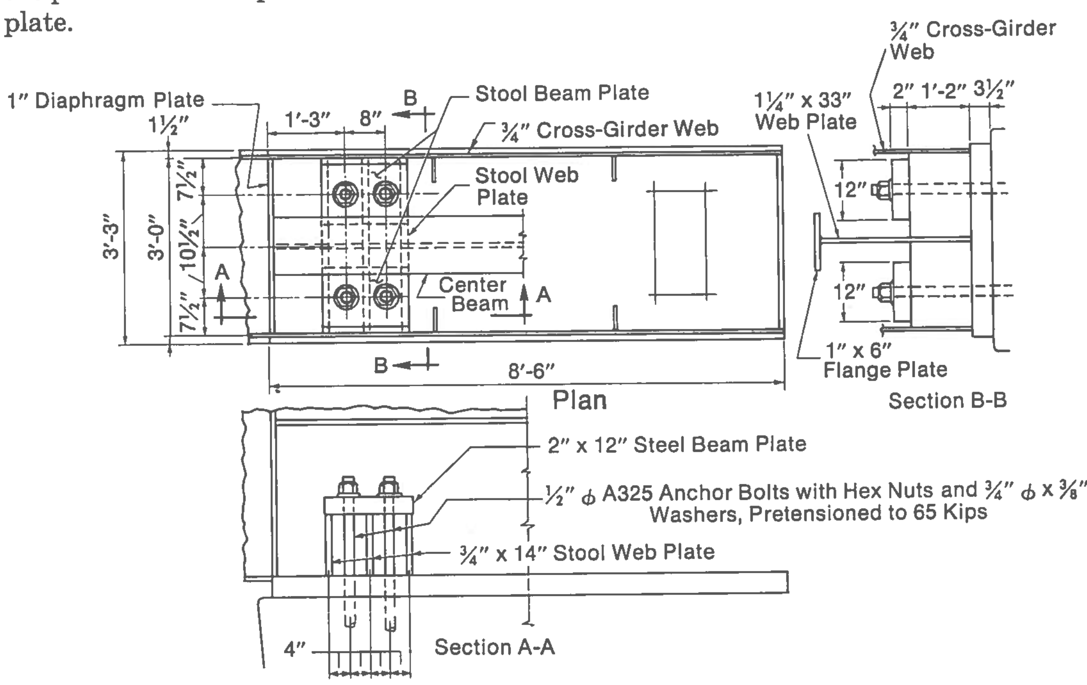
Because of high edge stresses, the allowable bearing stress on the concrete pier is taken as 995 psi. Concrete stresses under the cross-girder masonry plate are as follows:

$$\begin{aligned} \text{Pretension: } & \frac{8 \times 65}{3{,}956} = 0.131 \\ D+L+I: & \quad = \underline{0.812} \\ \text{Total: } & \quad 0.943 < 0.995 \text{ ksi} \\ \text{Pretension: } & \quad 0.131 \\ D+2(L+I): & \quad \underline{1.344} \\ \text{Total: } & \quad 1.475 < (1.5 \times 0.995 = 1.493 \text{ ksi}) \end{aligned}$$

The masonry plate is satisfactory in bearing on the pier.

Design of Anchor-Bolt Stools

The nuts of the anchor bolts are tightened against steel stools, 16 in. above the top of the masonry plate. In addition, a built-up plate section, or center beam, is attached to the masonry plate along its midwidth, as shown in a drawing. There are several reasons for this arrangement. One reason is that it should be easier to pretension the bolts from a higher vantage point within the cross girder. A more important reason is that the stools and center beam serve as a stiff grillage to distribute the pretension and uplift forces more uniformly, thus reducing stresses in the masonry plate.

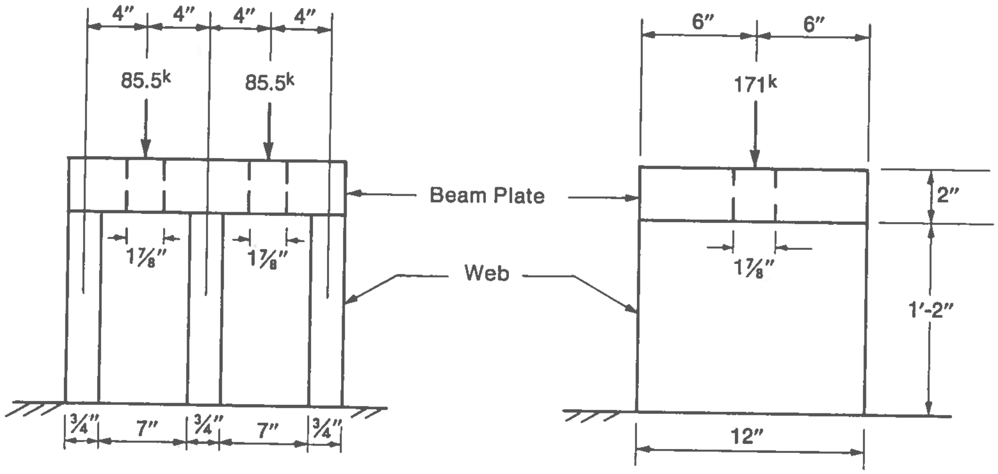


ANCHOR-BOLT STOOL

Each stool has a top seat, or beam plate, and three legs, or webs. A stool may be considered conservatively to act in two different ways:

1. The entire load from the bolts, taken as the interior reaction of a two-span continuous beam, is transmitted directly to the masonry plate in axial compression.

2. The entire load from the bolts is transmitted in shear to the cross-girder web or to the center beam.



LOADS ON STOOL

Try ¾-in. plates for the stool webs and a 2-in. plate for the stool beam plate. Allowable compressive stress in the webs is 20 ksi and allowable shear stress is 12 ksi.

The maximum bolt force equals the sum of the pretension and uplift forces. For $D+L+I$,

$$F_B = 65 + \frac{22.6}{4} = 70.7 \text{ kips}$$

For $D+2(L+I)$, with the allowable compressive stress increased 50%,

$$F_B = \frac{65+253/4}{1.5} = 85.5 > 70.7 \text{ kips}$$

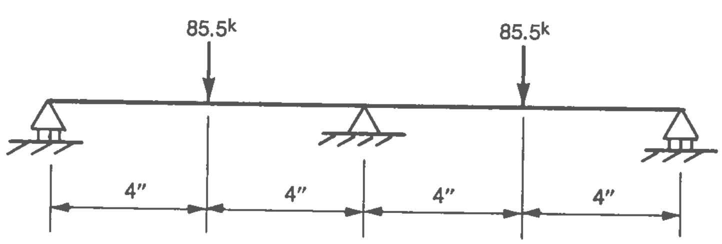
With the beam plate spanning the three webs treated as a two-span continuous beam with equal loads P at the middle of each span, the load on the center web is $11P/8 = 11 \times 85.5/8 = 117.6$ kips. The axial stress in the center web then is

$$f_{cw} = \frac{117.6}{0.75 \times 12} = 13.1 < 20 \text{ ksi}$$

The shear stress in the center web is

$$f_{vw} = \frac{117.6}{0.75 \times 14} = 11.2 < 12.0 \text{ ksi}$$

Next, the beam plate is investigated as a two-span continuous beam. Stresses are checked in the net section at the bolt locations and in the gross section at the center web. Try a 2×12-in. plate. Allowable bending stress is 20 ksi.



BEAM PLATE

The moments of inertia of the gross and net section are

$$I_{\text{Gross}} = \frac{12(2)^3}{12} = 8.0 \text{ in.}^4$$

$$I_{\text{Net}} = \frac{(12-1.88)(2)^3}{12} = 6.75 \text{ in.}^4$$

The maximum positive bending moment is $5PL/32$ and it produces a maximum stress in the net section of

$$f_b = \frac{M_c}{I} = \frac{(5/32)85.5 \times 8 \times 1.0}{6.75} = 15.8 < 20 \text{ ksi}$$

The maximum negative bending moment is $3PL/16$ and it produces a maximum stress in the gross section of

$$f_b = \frac{(3/16)85.5 \times 8 \times 1.0)}{8.0} = 16.0 < 20 \text{ ksi}$$

Use a 2×12-in. beam plate.

Design of Center Beam

The built-up plate section welded to the masonry plate is investigated as a center beam. This beam acts as the center support for the masonry plate, which is treated as a two-span continuous beam spanning between the webs of the cross girder and under the center beam. Group I loading, Case 1, controls the center-beam design. The load at the top of concrete below the cross girder for Case 1 is $D+L+I = 1{,}508$ kips, including 20 kips for the weight of the cross girder.

The bearing stress on the bottom of the masonry plate is

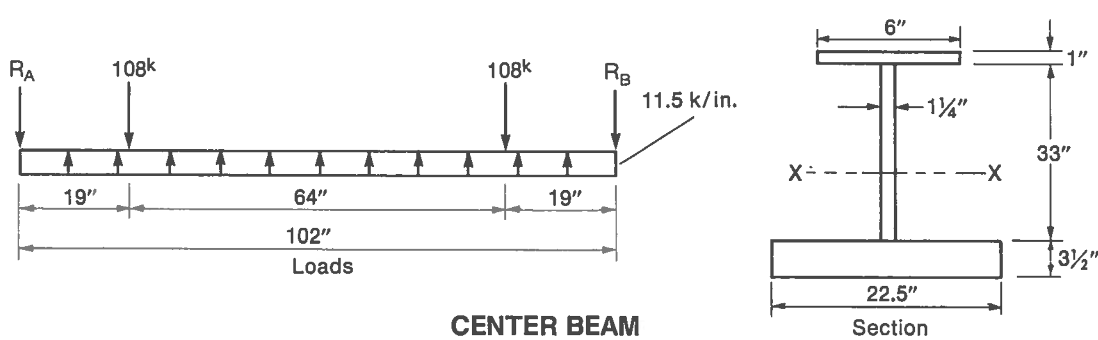
$$f_b = \frac{P}{A} = \frac{1{,}508 + 8 \times 65}{3{,}956} = 0.513 \text{ ksi}$$

With the masonry plate treated as a two-span continuous beam with uniform load f_bL on each span, the upward load on the center beam is $1.25 f_bL$. The upward load per unit length on the beam therefore is

$$w = 1.25 \times 0.513 \times 18 = 11.5 \text{ kips per in.}$$

The downward loads consist of the reactions R_A and R_B at diaphragms at the ends of the center beam and the bolt loads, which are assumed to be concentrated $15+4 = 19$ in. from each end of the beam. The bolt load taken by the center beam is

$$P = 4 \times 65 \times \frac{7.5}{18} = 108 \text{ kips}$$



CENTER BEAM

The reactions of the center beam are

$$R_A = R_B = \tfrac{1}{2}(11.5 \times 102 - 2 \times 108) = 478 \text{ kips}$$

The bending moment at midspan is

$$M = 478 \times 51 + 108 \times 32 - \tfrac{1}{2} \times 11.5(51)^2 = 12{,}878 \text{ kip-in.}$$

Use a 1×6-in. flange plate and a 1¼×33-in. web plate welded to the masonry plate, as shown in a drawing. Assume an effective bottom-flange width for the center beam of 1.25×18=22.5 in. The section modulus for the top portion of the beam is computed to be 628 in.^{3} The tensile bending stress in the top flange is

$$f_b = \frac{12{,}878}{628} = 20.5 \approx 20 \text{ ksi}$$

Shear stress in the web at the end of the beam is

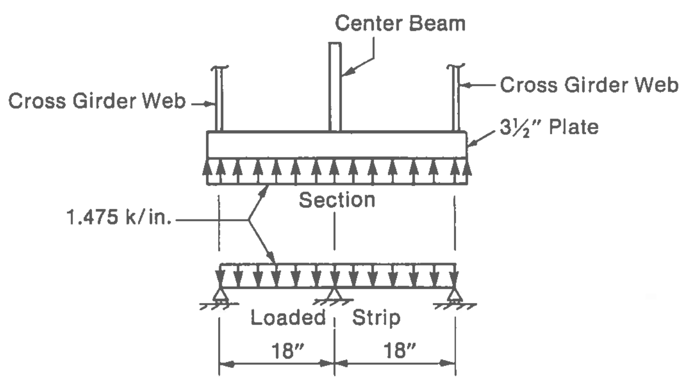
$$f_v = \frac{478}{1.25 \times 33} = 11.6 < 12 \text{ ksi}$$

Design of Masonry Plate

The masonry plate is analyzed next. Consider a 1-in.-wide transverse strip of the plate. As computed previously, under $D+2(L+I)$, the strip has a maximum bearing stress of 1.475 ksi, which causes bending stresses along the strip. Axial stresses delivered to the ends of the masonry plate from the bottom flange of the cross girder acts at right angles to the bending stresses. By inspection, the axial stresses are not critical.

Try a 3½-in.-thick masonry plate. The maximum moment in the 1-in. strip is

$$M = \frac{1.475(18)^2}{8} = 59.7 \text{ kip-in.}$$



MASONRY PLATE

The moment of inertia of the strip is

$$I = \frac{1.0(3.5)^3}{12} = 3.57 \text{ in.}^4$$

The bending stress in the strip therefore is

$$f_b = \frac{59.7 \times 1.75}{3.57} = 29.3 < (1.5 \times 20 = 30 \text{ ksi})$$

Use a 3½-in. masonry plate.

FINAL DESIGN

Drawings of the box-girder bridge of this design example are shown on the following sheets.