Two-Span Continuous Bridge Steel Plate Girder Design

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Project Manager: Juan Alcantar, P.E. and Bradley Wagner, P.E.

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Submitted by

Upul Attanayake, Ph.D., P.E.
Presidential Innovation Professor
Western Michigan University
(269) 276 – 3217
upul.attanayake@wmich.edu

Haluk Aktan, Ph.D., P.E.
Professor
Western Michigan University
(269) 276 – 3206
haluk.aktan@wmich.edu

Kanchani Basnayake, M.Sc.
Graduate Research Assistant
Western Michigan University
(269) 276 – 3210
k.basnayakemudiyanselage@wmich.edu

Western Michigan University
Department of Civil & Construction Engineering
College of Engineering and Applied Sciences
Kalamazoo, MI 49008-5316
Fax: (269) 276 – 3211

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NOTATION

Text in Italics designates MDOT policies and AASHTO LRFD Specifications.

Text in Italics with shading designates MDOT practice that is not necessarily documented in published articles.

A	=	cross-sectional area of the girder (noncomposite section), constant taken from Table 6.6.1.2.5-1, moment arm
A_b	=	area of the bolt corresponding to the nominal diameter
A_{bf}	=	area of the bottom flange
A_{bot}	=	area of the bottom flange
A_c	=	total area of the transformed section, area of the compression flange
A_e	=	effective flange area of the flange under consideration
A_f	=	sum of the area of the fillers on both sides of the connected plate
A_g	=	gross area
A_{ht}	=	transformed haunch area
A_n	=	net area
A_{nc}	=	area of the noncomposite section
A_p	=	smaller of either the connected plate area or the sum of the splice plate areas on both sides of the connected plate
A_{pn}	=	area of the projecting elements for the stiffener outside of the web-to-flange fillet welds but not beyond the edge of the flange
A_s	=	gross area of the splice plate
A_{sc}	=	cross-sectional area of a stud shear connector
A_{st}	=	transformed concrete deck area
A_t	=	area of the tension flange
A_{tf}	=	area of the top flange
A_{tn}	=	net area along the plane resisting tension stress
A_{vg}	=	gross area along the plane resisting shear stress
A_{vn}	=	net area along the plane resisting shear stress
A_w	=	area of the web
$(ADTT)_{SL}$	=	single-lane ADTT as specified in LRFD Art. 3.6.1.4
b_c	=	full width of the compression flange
b_{eff}	=	effective flange width
b_f	=	flange width
b_{fc}	=	compression flange width
b_h	=	haunch width
b_t	=	width of each projecting stiffener element, full width of the tension flange
C	=	ratio of the shear-buckling resistance to the shear yield strength determined with the shear-buckling coefficient (k) of 5.0
C_b	=	moment gradient modifier
C_V	=	correction factor for live load shear
d	=	nominal diameter of the bolt, diameter of the stud, depth of the steel section, distance from the PNA to each component

d_e	=	roadway part of the overhang, horizontal distance from the exterior girder web centerline to the interior edge of curb or traffic barrier
d_h	=	bolt hole diameter
D	=	web depth, clear distance between flanges
D_c	=	depth of the web in compression in the elastic range
D_{cp}	=	depth of the web in compression at the plastic moment determined as specified in Art. D6.3.2
D_p	=	distance from the top of the concrete deck to the PNA
D_t	=	total depth of the composite section
DFL	=	distribution factor for live load deflection
DFM	=	distribution factor for moment
DFM_E	=	distribution factor for moment of an exterior girder
DFM_{EB}	=	distribution factor for moment of an exterior girder adjusted for skew
DFM_i	=	distribution factor for moment of a girder segment with uniform section properties
DFM_I	=	distribution factor for moment of an interior girder
DFM_{IB}	=	distribution factor for moment of an interior girder adjusted for skew
DFV	=	distribution factor for shear
DFV_I	=	distribution factor for shear of an interior girder
DFV_{IB}	=	distribution factor for shear of an interior girder adjusted for skew
DFV_E	=	distribution factor for shear of an exterior girder
DFV_{EB}	=	distribution factor for shear of an exterior girder adjusted for skew
e	=	eccentricity of a design truck or a design lane load from the center of gravity of the pattern of girders
e_g	=	distance between centers of gravity of the girder and slab
E	=	modulus of elasticity of the section under consideration
E_c	=	modulus of elasticity of concrete
E_s	=	modulus of elasticity of steel
f_1	=	stress without consideration of lateral bending at the brace point opposite to the one corresponding to f_2
f_2	=	largest compressive stress without consideration of lateral bending at either end of the unbraced length of the flange under consideration
f_{bu}	=	largest value of the compressive stress throughout the unbraced length in the flange under consideration, calculated without consideration of flange lateral bending
f_c	=	compression flange stress at the section under consideration due to the Service II loads calculated without consideration of flange lateral bending
f_f	=	flange stresses at the section under consideration due to Service II loads calculated without consideration of flange lateral bending
f_l	=	flange lateral bending stress
f_{l1}	=	first-order compression flange lateral bending stress at the section under consideration
f_{mid}	=	stress without consideration of lateral bending at the middle of the unbraced length of the flange under consideration
f_t	=	stress on the gross area of the tension flange due to the factored loads calculated without consideration of flange lateral bending
F_{cr}	=	elastic lateral-torsional buckling stress for the flange under consideration
F_{crw}	=	nominal bend-buckling resistance for webs with or without longitudinal stiffeners
F_{exx}	=	classification strength of the weld metal

F_{fat}	=	radial fatigue shear range per unit length
F_{nc}	=	nominal flexural resistance of the compression flange
F_p	=	total radial force in the concrete deck at the point of maximum positive live load plus impact moment
F_{rc}	=	net range of cross-frame or diaphragm force at the top flange
F_u	=	specified minimum tensile strength
F_{ub}	=	specified minimum bolt tensile strength
F_y	=	specified minimum yield strength
F_{yc}	=	specified minimum yield strength of the compression flange
F_{yf}	=	specified minimum yield strengths of the flanges
F_{yr}	=	compression-flange stress at the onset of nominal yielding within the cross-section, including residual stress effects, but not including compression flange lateral bending
F_{yt}	=	specified minimum yield strength of the tension flange
F_{ys}	=	specified minimum yield strength of the stiffener
F_{yw}	=	specified minimum yield strength of the web
g	=	gage between holes
$g_{interior}$	=	distribution factor for moment in interior girder
h	=	girder height, depth between the centerlines of the flanges
h_c	=	composite section height
h_e	=	ext. girder depth at maximum positive moment
h_i	=	int. girder depth at maximum positive moment
H_w	=	web force
I	=	moment of inertia
I_{bf}	=	bottom flange moment of inertia
I_{cn8}	=	short-term composite section moment of inertia
I_{cn24}	=	long-term composite section moment of inertia
I_{ht}	=	transformed haunch moment of inertia
I_{nc}	=	noncomposite girder moment of inertia
I_s	=	moment of inertia of the effective column section
I_{st}	=	transformed concrete deck moment of inertia
I_{tf}	=	top flange moment of inertia
I_w	=	web moment of inertia
I_{yc}	=	moment of inertia of the compression flange about the vertical axis
I_{yt}	=	moment of inertia of the tension flange about the vertical axis
J	=	St. Venant torsional constant
k	=	shear-buckling coefficient, bend-buckling coefficient, distance from the outer face of the flange resisting the bearing reaction to the web toe of the fillet
K	=	effective length factor in the plane of buckling
K_g	=	longitudinal stiffness parameter
K_{gi}	=	longitudinal stiffness parameter of girder segment with uniform section properties
K_h	=	hole size factor
K_s	=	surface condition factor
l	=	distance between brace points, unbraced length in the plane of buckling

L = span length, length of the girder shipping piece

L_1 = distance between truck load resultant and first interior girder

L_b = unbraced length

L_{bcl} = distance from girder end to abutment bearing centerline

L_{br} = total length of the bridge

L_c = clear distance between holes or between the hole and the end of the member in the direction of the applied bearing force

L_{ds} = design span (between abutment and pier bearing centerlines)

L_g = total length of the girder

L_i = girder segment length with uniform section properties

L_p = limiting unbraced length to achieve the nominal flexural resistance under uniform bending

L_r = limiting unbraced length to achieve the onset of nominal yielding in either flange under uniform bending with consideration of compression – flange residual stress effects

L_t = length of the tension plane

L_v = length of shear plane

m = multiple presence factor

M_0 = moment at the brace point opposite to the one corresponding to M_2, calculated from the moment envelope value that produces the largest compression at this point in the flange under consideration or the smallest tension if this point was never in compression. M_0 shall be due to the factored loads and shall be taken as positive when it causes compression and negative when it causes tension in the flange under consideration

M_1 = moment at the brace point opposite to the one corresponding to M_2

M_2 = largest major axis bending moment at either end of the unbraced length causing compression in the flange under consideration calculated from the critical moment envelope. M_2 shall be due to the factored loads and shall be taken as positive. If the moment is zero or causes tension in the flange under consideration at both ends of the unbraced length, M_2 shall be taken as zero.

M_{AD} = additional moment applied to the short-term composite section to cause the nominal yielding in either steel flange

M_b = moment due to barrier weight

$M_{construction}$ = construction load moment

M_D = moment due to concrete deck weight

M_{D1} = bending moment caused by factored permanent loads applied to the noncomposite section

M_{D2} = bending moment caused by factored permanent loads applied to the long-term composite section

M_{DC1} = moment due to permanent loads acting on noncomposite section

M_{DL} = moment due to permanent loads

M_e = critical elastic moment envelope value at the interior-pier section due to service II loads

M_f = moment due to SIP formwork weight

M_{flange} = moment resisted by the flanges

M_g = moment due to girder self-weight

M_H = moment due to haunch weight

$M_{HL\text{-}M}$ = HL – 93 Mod with impact moment

M_{L2L} = design moment per girder due to 90% of lane load

M_{L2T} = design moment per girder due to 90% of two-truck load

M_{LA} = design moment per girder due to single 60-kip load

M_{LFT} = design moment per girder due to fatigue truck load

M_{LL} = design moment per girder due to lane load

M_{LT}	=	design moment per girder due to truck load
M_{mid}	=	major axis bending moment at the middle of the unbraced length, calculated from the moment envelope values that produce the largest compression at this point in the flange under consideration, or the smallest tension if this point is never in compression. M_{mid} shall be due to the factored loads and shall be taken as positive when it causes compression and negative when it causes tension in the flange under consideration
M_n	=	nominal flexural resistance
M_{nc}	=	nominal flexural resistance based on the compression flange
$M_{nc\ (FLB)}$	=	nominal flexural resistance based on the compression flange due to flange local buckling
$M_{nc\ (LTB)}$	=	nominal flexural resistance based on the compression flange due to lateral-torsional buckling
M_p	=	plastic moment
M_{pe}	=	negative-flexure effective plastic moment for the service limit state
M_{rd}	=	redistribution moment for Service II loads at each interior-pier section
M_{SE}	=	moment due to settlement
M_u	=	bending moment about the major axis of the section
M_w	=	web moment
M_{ws}	=	moment due to FWS weight
M_y	=	yield moment
M_{yc}	=	yield moment with respect to the compression flange
M_{yt}	=	yield moment with respect to the tension flange
n	=	short-term modular ratio, modular ratio between girder and slab material, number of stress range cycles per truck passage taken from Table 6.6.1.2.5-2, number of studs in the cross-section, number of bolts along the chain
N	=	number of bolts, number of cycles, bearing length
N_b	=	number of girders
N_L	=	number of loaded lanes under consideration
N_s	=	number of shear planes per bolt
p	=	pitch of shear connectors
P	=	total nominal shear force
P_c	=	plastic force in the compression flange
P_e	=	elastic critical buckling resistance
P_{fy}	=	design yield resistance of each flange
$P_{fy\ (Bot)}$	=	design yield resistance of bottom flange
$P_{fy\ (Bot\text{-}inner)}$	=	design force carried by the bottom flange outer plate
$P_{fy\ (Bot\text{-}outer)}$	=	design force carried by the bottom flange outer plate
$P_{fy\ (Top)}$	=	design yield resistance of top flange
$P_{fy\ (Top\text{-}inner)}$	=	design force carried by the top flange inner plates
$P_{fy\ (Top\text{-}outer)}$	=	design force carried by the top flange outer plate
P_n	=	nominal compressive resistance
P_o	=	equivalent nominal yield resistance
P_p	=	total longitudinal force in the concrete deck at the point of maximum positive live load plus impact moment taken as the lesser of either P_{1p} and P_{2p}
P_r	=	factored yield resistance of the splice plate in tension
P_{rb}	=	plastic force in bottom longitudinal reinforcement

P_{rt}	=	plastic force in top longitudinal reinforcement
P_s	=	plastic force in the concrete slab
P_t	=	minimum required bolt tension, plastic force in the tension flange
P_w	=	plastic force in the web
Q	=	first moment of the transformed short-term area of the concrete deck about the neutral axis of the short-term composite section, statical moment of the area about the neutral axis
Q_n	=	nominal shear resistance of a single shear connector
Q_{bf_c8}	=	statical moment of the area of the bottom flange of the short-term composite section about the neutral axis
Q_{bf_c24}	=	statical moment of the area of the bottom flange of the long-term composite section about the neutral axis
Q_{bf_nc}	=	statical moment of the area of the bottom flange of the noncomposite section about the neutral axis
Q_r	=	factored shear resistance of a single shear connector
Q_{tf_c8}	=	statical moment of the area of the top flange of the short-term composite section about the neutral axis
Q_{tf_c24}	=	statical moment of the area of the top flange of the long-term composite section about the neutral axis
Q_{tf_nc}	=	statical moment of the area of the top flange of the noncomposite section about the neutral axis
r_s	=	radius of gyration about the axis normal to the plane of buckling
r_t	=	effective radius of gyration for lateral-torsional buckling
R	=	reaction on exterior girder in terms of lanes, reduction factor applied for the factored resistance of the bolts in shear at the strength limit state due to impact of filler plates, minimum girder radius within the panel
R_b	=	web load-shedding factor
R_h	=	hybrid factor
R_M	=	reduction factor for live load moment
R_n	=	nominal resistance of the bolt, connection, or the connected material, nominal resistance to the concentrated loading
R_p	=	reduction factor for holes
R_{pc}	=	web plastification factor for the compression flange
R_{pt}	=	web plastification factor for the tension flange
R_r	=	factored resistance of a bolted connection, factored shear resistance of the weld metal
R_{rr}	=	factored bolt shear resistance after incorporating the impact of the filler plates
R_u	=	factored concentrated load or bearing reaction
R_w	=	design web force
R_{weld}	=	required shear strength at each weld at the bottom flange-to-web connection
s	=	spacing between centers of bolts in a single bolt line, staggered spacing, required shear strength
s_{min}	=	minimum spacing between centers of bolts in standard bolt holes
S	=	girder spacing, section modulus
S_b	=	section modulus for bottom fiber
S_{nc}	=	section modulus of the noncomposite section
S_{lt}	=	section modulus of the long-term composite section
S_{st}	=	section modulus of the short-term composite section
S_t	=	section modulus for top fiber

S_{xc}	=	elastic section modulus about the major axis of the section to the compression flange taken as M_{yc}/F_{yc}
S_{xt}	=	elastic section modulus about the major axis of the section to the tension flange taken as M_{yt}/F_{yt}
t	=	thickness of the thinner outer plate or shape, thickness of the connected material
t_c	=	compression flange thickness
t_f	=	flange thickness
t_{fc}	=	compression flange thickness
t_h	=	haunch thickness
t_p	=	thickness of the projecting stiffener element
t_s	=	deck slab thickness
t_t	=	tension flange thickness
t_w	=	web thickness
U	=	reduction factor to account for shear lag
U_{bs}	=	reduction factor for block shear rupture resistance
V	=	shear force
V_b	=	shear due to barrier weight
V_{cr}	=	shear-yielding or shear-buckling resistance
V_D	=	shear due to concrete deck weight
V_{DC1}	=	shear due to permanent loads acting on noncomposite section
V_f	=	shear due to SIP formwork weight, vertical shear force range under the applicable fatigue load combination
V_{fat}	=	longitudinal fatigue shear range per unit length
V_g	=	shear due to girder self-weight
V_H	=	shear due to haunch weight
V_{HL-M}	=	HL – 93 Mod with impact shear
V_{L2L}	=	design shear per girder due to 90% of lane load
V_{L2T}	=	design shear per girder due to 90% of two-truck load
V_{LA}	=	design shear per girder due to single 60-kip load
V_{LL}	=	design shear per girder due to lane load
V_{LT}	=	design shear per girder due to truck load
V_n	=	nominal shear resistance
V_p	=	plastic shear force
V_{SE}	=	shear due to settlement
V_{sr}	=	horizontal fatigue shear range per unit length
V_u	=	factored shear in the web at the section under consideration
V_{ws}	=	shear due to FWS weight
w	=	clear roadway width, effective length of deck
w_b	=	type 4 – barrier or railing weight
w_c	=	unit weight of concrete
w_f	=	SIP formwork weight
w_s	=	unit weight of steel
w_{ws}	=	FWS weight on one girder
x	=	horizontal distance from the center of gravity of the pattern of girders to each girder

X_{ext}	=	horizontal distance from the center of gravity of the pattern of girders to the exterior girder
y_b	=	distance from the centroid to the extreme bottom fiber
y_{bc}	=	distance from the centroid of the composite section to the extreme bottom fiber
y_{gt}	=	distance from the centroid of the composite section to girder top
y_t	=	distance from the centroid of the section to the extreme top fiber
y_{tc}	=	distance from the centroid of the composite section to the extreme top fiber
Z_r	=	shear fatigue resistance of an individual shear connector
$\bar{y}$	=	distance to the PNA from the top of the girder top flange
θ	=	skew angle
θ_{RL}	=	plastic rotation at which the moment at an interior-pier section nominally begins to decrease with increasing plastic rotation (θ_p) at an interior-pier section
Δw	=	overhang width
f'_c	=	specified strength of concrete
η	=	load modifying factor for ductility, redundancy, and operational importance
η_Δ	=	load modifying factor for ductility
η_I	=	load modifying factor for operational importance
η_P	=	load modifying factor for redundancy
γ	=	load factor for fatigue load combination in Table 3.4.1-1
γ_p	=	load factor for permanent loading
γ_{SE}	=	load factor for settlement
ϕ	=	resistance factors
ϕ_b	=	resistance factor for bearing
ϕ_{bb}	=	resistance factor for bolts bearing on material
ϕ_{bs}	=	resistance factor for block shear rupture
ϕ_c	=	resistance factor for compression members
ϕ_{e2}	=	resistance factor for shear in throat of the weld metal
ϕ_f	=	resistance factor for flexure
ϕ_s	=	resistance factor for bolts in shear
ϕ_{sc}	=	resistance factor for shear connectors
ϕ_u	=	resistance factor for fracture in net section of tension members
ϕ_v	=	resistance factor for shear
ϕ_{vu}	=	resistance factor for shear rupture of the connected elements
ϕ_w	=	resistance factor for web crippling
ϕ_y	=	resistance factor for yielding in gross section of tension members
λ_f	=	slenderness ratio for the compression flange
λ_{pf}	=	limiting slenderness ratio for a compact flange
$\lambda_{pw\ (Dcp)}$	=	limiting slenderness ratio for a compact web
λ_{rf}	=	limiting slenderness ratio for a noncompact flange
λ_{rw}	=	limiting slenderness ratio for a noncompact web
(Δf)	=	force effect, live load stress range due to the passage of the fatigue load
$(\Delta F)_n$	=	nominal fatigue resistance as specified in Art. 6.6.1.2.5.
$(\Delta F)_{TH}$	=	constant-amplitude fatigue threshold taken from Table 6.6.1.2.5-3

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INTRODUCTION

This publication illustrates the design of a composite steel plate girder of a zero skew, 200 ft long, two-span, continuous interstate freeway bridge for flexure, shear, and deflection under dead, live, and construction loads (Figure 1). Each span consists of seven girders spaced at 9 ft - $8^5/_8$ in. on centers (Figure 2). The girder layout and dimensions of the trial girder sections are shown in Figure 3, Figure 4, and Figure 5. Girders are designed for composite behavior with a 9-in. thick cast-in-place concrete deck to resist superimposed dead, live, and impact loads.

Even though the AASHTO Standard Specifications, 17^{th} edition, Article 10.20.1, limited the maximum spacing of intermediate diaphragms to 25 ft, the 8^{th} edition of the AASHTO LRFD Specifications does not specify a minimum number of required interior diaphragms or the maximum spacing. The number of intermediate diaphragms needs to be defined after evaluating all the stages of construction. Considering past practice and potential positive benefits, such as the reduction in girder flange thickness, four interior diaphragms per span are selected for this bridge. The diaphragm layout is shown in Figure 3.

The design is implemented in accordance with the Michigan Department of Transportation (MDOT) policies documented in the Bridge Design Manual (BDM), Bridge Design Guides (BDG), and MDOT 2012 Standard Specifications for Construction (SSFC) as of 10/30/2020. As needed, the stipulations in the 8^{th} edition of AASHTO LRFD *Bridge Design Specifications* are considered. Certain material and design parameters are selected to be in compliance with MDOT practice reflected in the Bridge Design System (BDS) the MDOT legacy software.

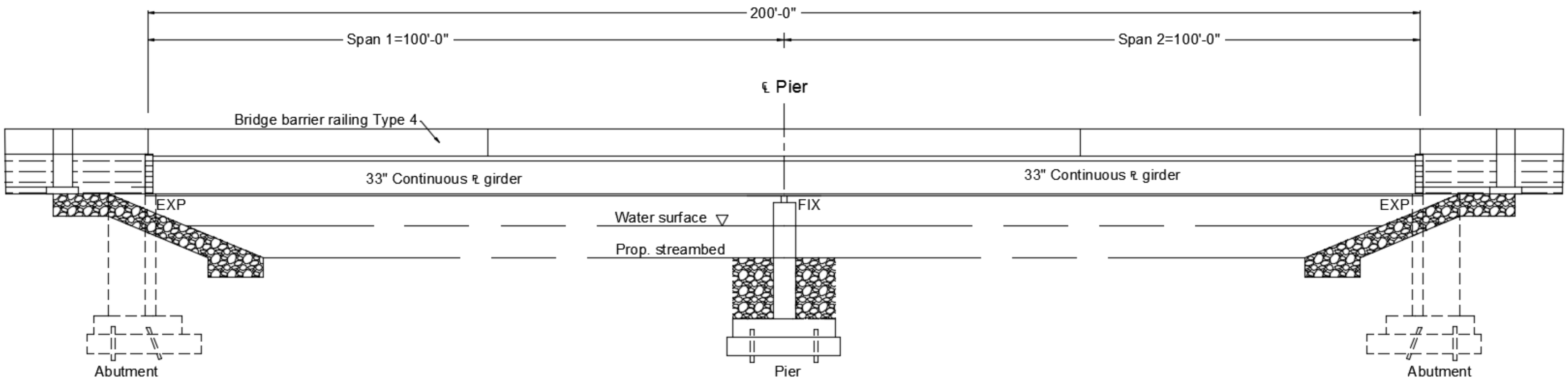


Figure 1. Bridge elevation

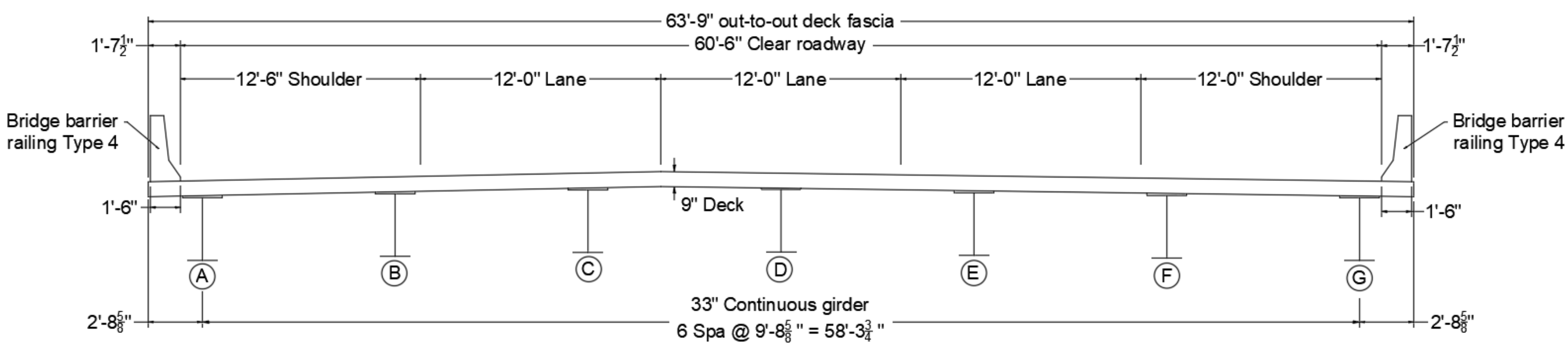


Figure 2. Bridge cross-section

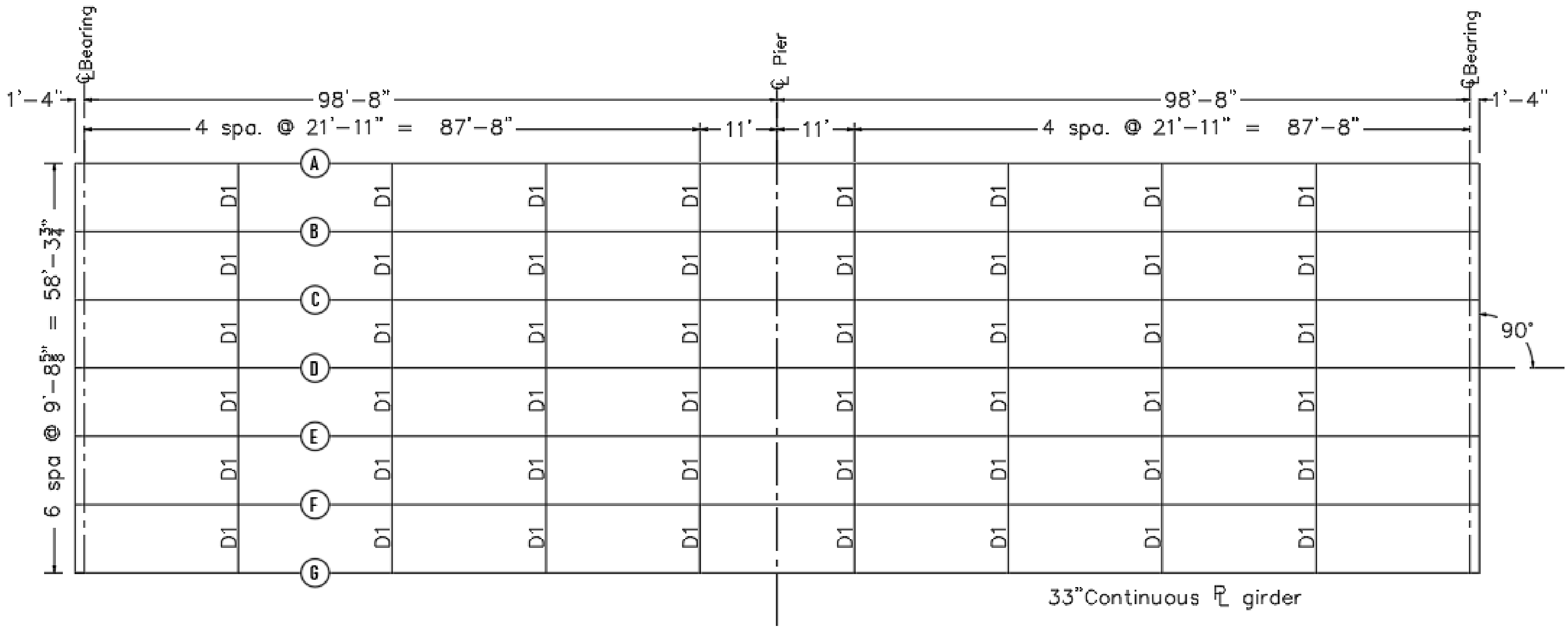


Figure 3. Girder and diaphragm layout

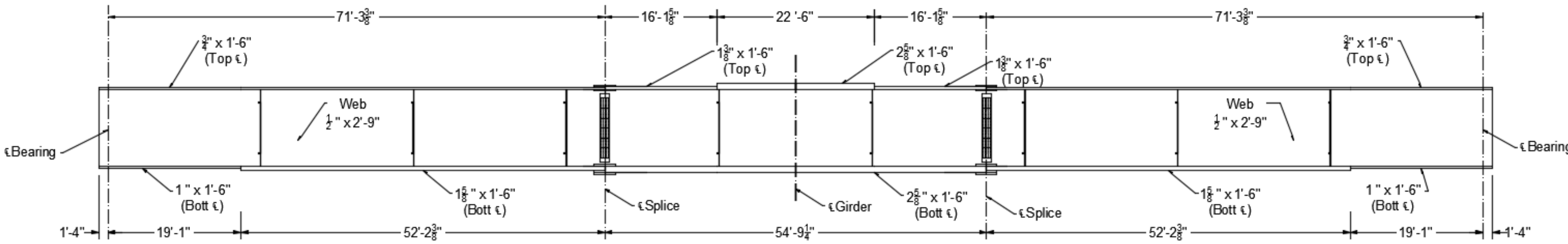


Figure 4. Exterior girder geometry selected for analysis

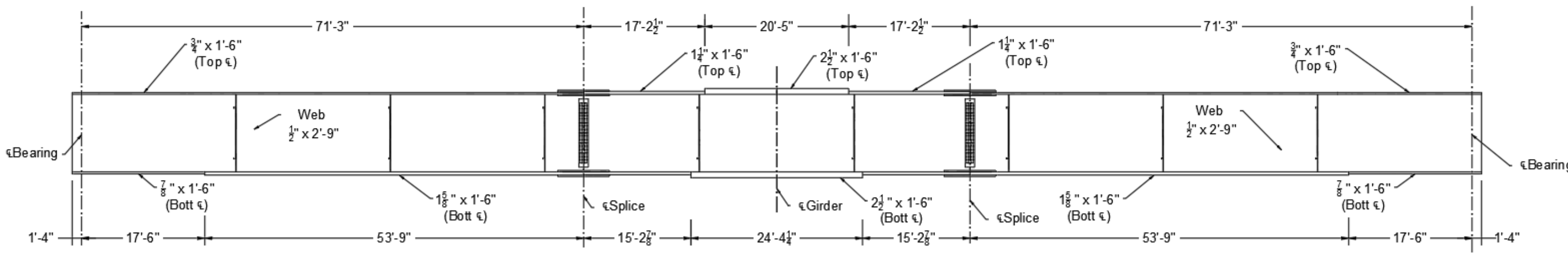


Figure 5. Interior girder geometry selected for analysis

The example is divided into 9 major steps as shown in **Figure 6**. Each step is adequately detailed with examples, explanations, and references.

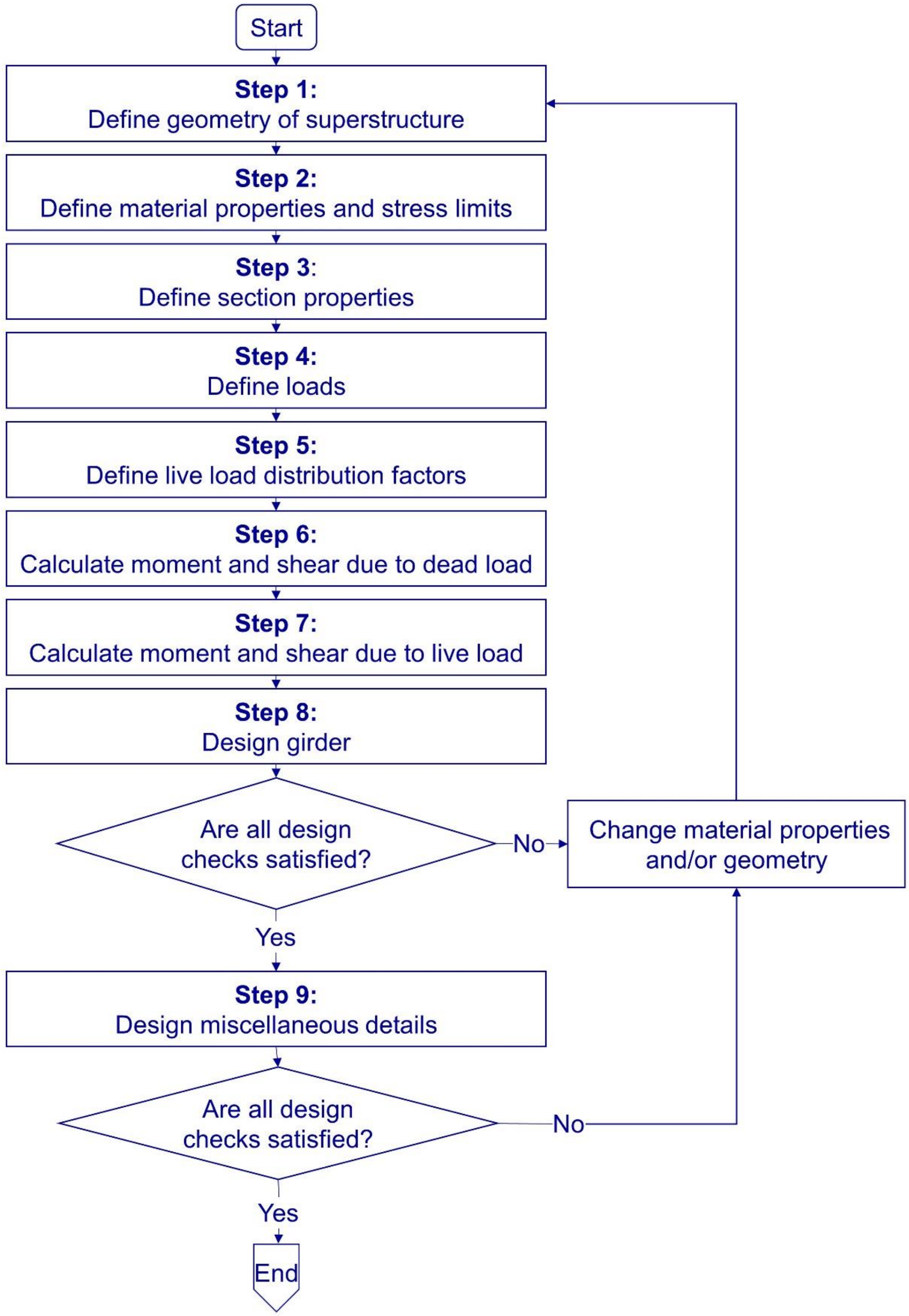


Figure 6. Major steps of the design example

STEP 1. GEOMETRY

Step 1.1. Superstructure

Total length of the bridge, L_{br}	= 200 ft	
Distance from girder end to abutment bearing centerline, L_{bcl}	= 1.33 ft	
Number of spans	= 2	
Design span (between abutment and pier bearing centerlines), L_{ds}	= 98.67 ft	
Design span of the two-span continuous girder = 2 L_{ds} = $L_{br} - 2 \times L_{bcl}$	= 197.34 ft	
Skew, θ	= 0 deg.	

Skew is the angle measured from a line perpendicular to the bridge's centerline to support the reference line. When skew exceeds 30 degrees, refined analysis methods should be utilized for load calculations. BDM Art. 7.01.14

Clear roadway width	= 60 ft – 6 in.	
Lane width	= 12 ft	BDG 6.05.01A
Outside shoulder width	= 12 ft	
Inside shoulder width	= 12 ft – 6 in.	

*For an interstate freeway with 3 lanes, shoulder width is 10' min, 12' if the Directional Design Hour Volume (DDHV) is greater than 250. These values result in a shoulder width of 12' on the structure (per note *).* BDG 6.05.01A

Overhang width, Δw = 2 ft – 8 ⅝ in.

BDG 6.41.01 (05/04/2006) presents a Standard Bridge Slab (load Factor Design). This detail does not include overhang limits. The bridge barrier railing details in the BDG shows a maximum of 2 ft – 6 in. for the overhangs. BDG 6.41.01

With a standard slab design, shear studs are not used within the negative moment region. BDM Art. 7.02.15.B1

Since this example does not use shear studs in the negative moment region, the overhang widths stated above are used, an exception to typical guidelines.

Type 4 barrier width	= 18 in.	STD Plan B-17-D
Distance from slab fascia to outside face of the barrier	= 1.5 in.	STD Plan B-17-D
Deck width	= 63 ft – 9 in.	
Ext. girder depth at maximum positive moment, h_e	= 35.375 in.	
Int. girder depth at maximum positive moment, h_i	= 35.125 in.	

A plate girder with a web depth of 33 in. is selected as a trial section. Total depth of the section changes based on the top and bottom flange plate thicknesses along the length.

Number of girders, N_b = 7

Designers should layout beam spacing to accommodate future part width reconstruction. In most cases beams at the centerline of the structure should be avoided. BDM Art. 7.01.15

An exemption can be requested considering the presence of a parallel bridge for future maintenance of traffic.

Girder spacing, S	= 9 ft – 8.625 in.	
	= 116.625 in.	
$S \leq 10$ ft for all the beams.		BDM Art. 7.02.02
Deck slab thickness, t_s	= 9 in.	
Standard deck slab thickness is 9 in.		BDG 6.41.01/ 6.41.02
Full slab thickness is included in dead load calculation as well as in the composite section calculations for the beam design. The deck slab design is based on 7.5 in. thickness, excluding the top 1.5 in. integrated wearing surface.		BDM Art. 7.02.08 B & 7.02.19 A4
Haunch thickness, t_h	= 1 in.	
$t_h \geq 1$ in.		BDM Art. 7.02.19-C
Haunch width, b_h	= 24 in.	

As a practice, haunch width is extended 3 in. from either side of the girder top flange even if the contractor later decides to use stay-in-place formwork. A note is included in the plans for the contractor to modify haunch width based on the type of formwork selected for construction.

STEP 2. MATERIAL PROPERTIES AND STRESS LIMITS

Step 2.1. Concrete

Unit weight, w_c = 0.145 kip/ft^3

MDOT BDS uses standard concrete density of 0.145 kip/ft^3 in modulus of elasticity and deflection calculations.

Unit weight, w_c = 0.150 kip/ft^3

MDOT uses a reinforced concrete density of 0.150 kip/ft^3 for dead load calculations.

Cast-in-place concrete slab, 28-day strength, f'_c = 4.0 ksi

f'_c = 4 ksi, Grade DM concrete. — BDM Art. 7.01.03

Modulus of elasticity, $E_c = 120{,}000\, K_1 (w_c)^{2.0} (f'_c)^{0.33}$ — LRFD Eq. 5.4.2.4-1

where:

- f'_c = specified strength of concrete, ksi
- K_1 = correction factor for source of aggregate = 1 — LRFD Art. 5.4.2.4
- w_c = unit weight of concrete, kip/ft^3 — LRFD Table 3.5.1.1

Elasticity modulus of cast-in-place slab:

$$E_c = 120{,}000(1)(0.145)^{2.0}(4)^{0.33} = 3{,}987 \text{ ksi}$$

Step 2.2. Steel

Structural steel: AASHTO M270 Grade 36, yield strength, F_y = 36 ksi — BDM Art. 7.01.03

Structural steel: AASHTO M270 Grade 50, yield strength, F_y = 50 ksi

Structural steel: AASHTO M270 Grade 50W, yield strength, F_y = 50 ksi

Structural steel, modulus of elasticity, E_s = 29,000 ksi

Reinforcing steel, yield strength, F_y = 60 ksi

Reinforcing steel, modulus of elasticity, E_s = 29,000 ksi — LFRD Art. 5.4.3.2

Steel unit weight, w_s = 0.490 kip/ft^3 — LRFD Table 3.5.1-1

STEP 3. SECTION PROPERTIES

Step 3.1. Cross-Section Proportion Limits

Step 3.1.1. Web proportions

Selection of an appropriate web depth has a significant influence on girder geometry. Thus, initial consideration should be given to the most appropriate web depth. In the absence of other criteria, the span-to-depth ratios may be used as a starting point for selecting a web depth. LRFD Art. 2.5.2.6.3

The minimum depth of the steel I-beam portion of a continuous-span composite section is 0.027L, where L is the span length. LRFD Table 2.5.2.6.3-1

Thus, the minimum steel depth = 0.027(98.67 ft)(12 in./ft) = 32 in.

Web thickness shall be a minimum of 7/16 in. BDM Art. 7.02.10A

For a section without longitudinal stiffeners: LRFD Art. 6.10.2.1.1

$$\frac{D}{t_w} \leq 150$$ LRFD Eq. 6.10.2.1.1-1

where:

D = clear distance between flanges

t_w = web thickness

Step 3.1.2. Flange proportions

Flange plate widths may be varied to achieve a more economical design when required. The minimum width shall be 12 in. The minimum thickness shall be ½ in. when shear connectors are not used and ¾ in. when shear connectors are welded to the flange in the field. BDM Art. 7.02.10B

Compression and tension flanges shall be proportioned such that: LRFD Art. 6.10.2.2

$$\frac{b_f}{2t_f} \leq 12.0$$ LRFD Eq. 6.10.2.2-1

$$b_f \geq D/6$$ LRFD Eq. 6.10.2.2-2

$$t_f \geq 1.1t_w$$ LRFD Eq. 6.10.2.2-3

$$0.1 \leq \frac{I_{yc}}{I_{yt}} \leq 10$$ LRFD Eq. 6.10.2.2-4

where:

b_f = flange width

I_{yc} = moment of inertia of the compression flange about the vertical axis

I_{yt} = moment of inertia of the tension flange about the vertical axis

t_f = flange thickness

Step 3.2. Exterior Girder

Table 1. Exterior Girder Dimensions

<table>
<tr><th rowspan="2"></th><th rowspan="2">Girder length (ft)</th><th rowspan="2">Bottom flange thickness (in.)</th><th rowspan="2">Top flange thickness (in.)</th><th rowspan="2">Top/bottom flange width (in.)</th><th colspan="2">Web</th></tr>
<tr><th>Depth (in.)</th><th>Thickness (in.)</th></tr>
<tr><td>Distance from girder end to CL of bearing</td><td>1.33</td><td rowspan="2">1.000</td><td rowspan="3">0.750</td><td rowspan="9">18</td><td rowspan="9">33</td><td rowspan="9">0.5</td></tr>
<tr><td rowspan="7">Distance between CL of bearings</td><td>19.08</td></tr>
<tr><td>52.20</td><td>1.625</td></tr>
<tr><td>16.14</td><td rowspan="3">2.625</td><td>1.375</td></tr>
<tr><td>22.50</td><td>2.625</td></tr>
<tr><td>16.14</td><td>1.375</td></tr>
<tr><td>52.20</td><td>1.625</td><td rowspan="3">0.750</td></tr>
<tr><td>19.08</td><td rowspan="2">1.000</td></tr>
<tr><td>Distance from CL of bearing to girder end</td><td>1.33</td></tr>
</table>

Step 3.2.1. Cross-section proportion limits

The following steps show the evaluation of cross-section proportion limits for the girder section at $0.4L_{ds}$ (39.468 ft) from the abutment bearing centerline of the exterior girder.

Web proportions

$$\frac{D}{t_w} = \frac{33}{0.5} = 66 < 150$$ **OK**

Flange proportions

Tension flange (bottom flange):

$$\frac{b_f}{2t_f} = \frac{18}{2 \times 1.625} = 5.54 < 12.0$$ **OK**

$$\frac{D}{6} = \frac{33}{6} = 5.5\ in. < b_f = 18\ in.$$ **OK**

$$1.1t_w = 1.1 \times 0.5 = 0.55\ in. < t_f = 1.625\ in.$$ **OK**

Compression flange (top flange):

$$\frac{b_f}{2t_f} = \frac{18}{2 \times 0.75} = 12 < 12.0$$ **OK**

$$\frac{D}{6} = \frac{33}{6} = 5.5\ in. < b_f = 18\ in.$$ **OK**

$$1.1t_w = 1.1 \times 0.5 = 0.55\ in. < t_f = 0.75\ in.$$ **OK**

Top and bottom flanges:

$$I_{yc} = \frac{1}{12}(0.75)(18)^3 = 364.5\ in.^4 \text{ (top flange)}$$

$$I_{yt} = \frac{1}{12}(1.625)(18)^3 = 789.75\ in.^4 \text{ (bottom flange)}$$

$$0.1 < \frac{I_{yc}}{I_{yt}} = \frac{364.5}{789.75} = 0.46 < 10 \qquad \textbf{OK}$$

Table 2 shows that the exterior girder dimensions satisfy the cross-section proportion limit requirements listed in **Step 3.1**.

Table 2. Cross-Section Proportion Limit Check for the Exterior Girder

Location ($\times L_{ds}$)	Distance from abutment bearing CL (ft)	Flange proportions							Web proportions		
		Top flange			Bottom flange			I_{yc}/I_{yt} [d]			
		b_f [a] (in.)	t_f [b] (in.)	$b_f/2t_f$ [c]	b_f [a] (in.)	t_f [b] (in.)	$b_f/2t_f$ [c]		D (in.)	t_w (in.)	D/t_w [e]
0.0	0	18	0.750	12	18	1.000	9.00	0.75	33	0.5	66
0.1	9.867	18	0.750	12	18	1.000	9.00	0.75	33	0.5	66
0.2	19.734	18	0.750	12	18	1.625	5.54	0.46	33	0.5	66
0.3	29.601	18	0.750	12	18	1.625	5.54	0.46	33	0.5	66
0.4	39.468	18	0.750	12	18	1.625	5.54	0.46	33	0.5	66
0.5	49.335	18	0.750	12	18	1.625	5.54	0.46	33	0.5	66
0.6	59.202	18	0.750	12	18	1.625	5.54	0.46	33	0.5	66
0.7	69.069	18	0.750	12	18	1.625	5.54	0.46	33	0.5	66
0.8	78.936	18	1.375	6.6	18	2.625	3.43	1.91	33	0.5	66
0.9	88.803	18	2.625	3.4	18	2.625	3.43	1.00	33	0.5	66
1.0	98.670	18	2.625	3.4	18	2.625	3.43	1.00	33	0.5	66
1.1	108.537	18	2.625	3.4	18	2.625	3.43	1.00	33	0.5	66
1.2	118.404	18	1.375	6.6	18	2.625	3.43	1.91	33	0.5	66
1.3	128.271	18	0.750	12	18	1.625	5.54	0.46	33	0.5	66
1.4	138.138	18	0.750	12	18	1.625	5.54	0.46	33	0.5	66
1.5	148.005	18	0.750	12	18	1.625	5.54	0.46	33	0.5	66
1.6	157.872	18	0.750	12	18	1.625	5.54	0.46	33	0.5	66
1.7	167.739	18	0.750	12	18	1.625	5.54	0.46	33	0.5	66
1.8	177.606	18	0.750	12	18	1.625	5.54	0.46	33	0.5	66
1.9	187.473	18	0.750	12	18	1.000	9.00	0.75	33	0.5	66
2.0	197.340	18	0.750	12	18	1.000	9.00	0.75	33	0.5	66

a - $b_f \geq D/6 = 5.5$ in.
b - $t_f \geq 1.1t_w = 0.55$ in.
c - $b_f/2t_f \leq 12.0$
d - $0.1 \leq I_{yc}/I_{yt} \leq 10$
e - $D/t_w \leq 150$

Step 3.2.2. Noncomposite and composite section properties

The elastic stress at any location on the composite section due to the applied load shall be the sum of the stresses caused by the loads applied separately to the steel section, short-term composite section, and long-term composite section.

LRFD Art. 6.10.1.1.1a

Transient loads (live loads) are applied on the short-term composite section and permanent loads are applied on the long-term composite section. Hence, short-term and long-term section properties are needed.

Short-term modular ratio, n = E_s/E_c — LRFD Art. 6.10.1.1.1b

Long-term modular ratio = 3n

$$n = \frac{E_s(\text{beam})}{E_c(\text{concrete})} = \frac{E_s}{E_c} = \frac{29000}{3987} = 7.27$$

To reflect the values used in BDS and the practice, use n = 8.

These modular ratios are used to transform deck slab and haunch widths to equivalent steel sections to calculate composite section properties.

Overhang width, Δw		= 2 ft – 8 $^5/_8$ in.	
		= 32.625 in.	
Effective flange width, b_{eff}	= S/2 + Δw	= 90.9375 in.	LRFD Art. 4.6.2.6.1
Transformed flange width with n	= b_{eff}/n = 90.9375/8	= 11.37 in.	
Transformed haunch width with n	= b_h /n = 24/8	= 3 in.	
Transformed flange width with 3n	= b_{eff}/3n = 90.9375/24	= 3.79 in.	
Transformed haunch width with 3n	= b_h/3n = 24/24	= 1 in.	

The moment of inertia (I) and the section modulus (S) of the girder at 1/10th of the design span (L_{ds}) are summarized in **Table 3** provided at the end of **Step 3.2**.

Step 3.2.2.1, **Step 3.2.2.2**, and **Step 3.2.2.3** show the calculation procedures for noncomposite and composite section properties of the exterior girder section at 0.4L_{ds}.

MDOT includes 1 in. haunch in the steel girder composite section property calculation.

Step 3.2.2.1. Noncomposite section property

Figure 7 shows the noncomposite girder section at $0.4L_{ds}$.

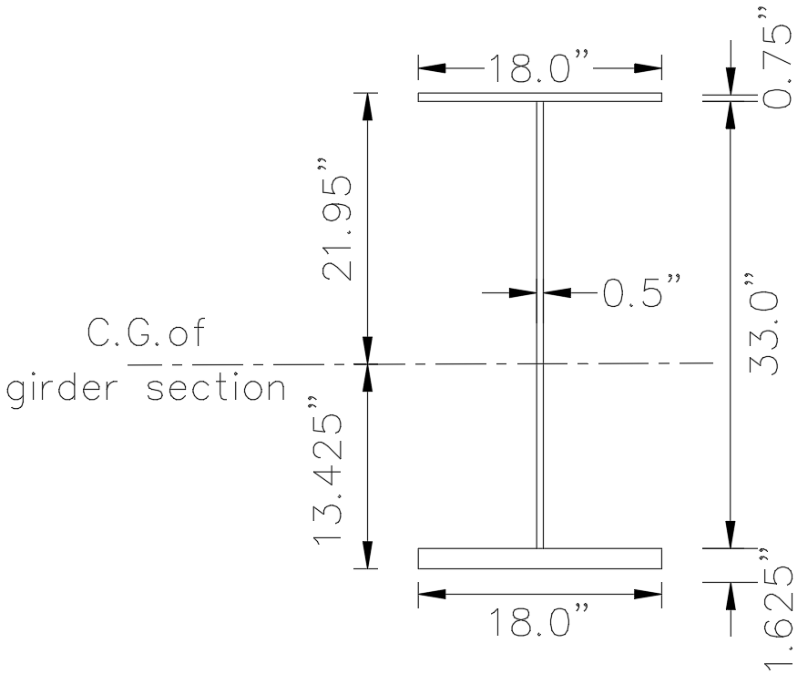


Figure 7. Exterior girder section at $0.4L_{ds}$

Girder height, h	$= 0.75 + 33 + 1.625$	$= 35.375$ in.
Area of the top flange, A_{tf}	$= 18 \times 0.75$	$= 13.5$ in.^{2}
Area of the web, A_w	$= 33 \times 0.5$	$= 16.5$ in.^{2}
Area of the bottom flange, A_{bf}	$= 18 \times 1.625$	$= 29.25$ in.^{2}
Area of the noncomposite section, A_{nc}	$= 13.5 + 16.5 + 29.25$	$= 59.25$ in.^{2}

Distance from the centroid to the extreme top fiber, y_t

$$= [13.5 \times (0.75/2) + 16.5 \times (0.75 + 33/2)] + 29.25 \times (0.75 + 33 + 1.625/2)/59.25 \quad = 21.95 \text{ in.}$$

Distance from the centroid to the extreme bottom fiber, y_b

$$= h - y_t = 35.375 - 21.95 \quad = 13.425 \text{ in.}$$

Top flange moment of inertia, I_{tf}	$= (18 \times 0.75^3)/12$	$= 0.633$ in.^{4}
Web moment of inertia, I_w	$= (0.5 \times 33^3)/12$	$= 1497.4$ in.^{4}
Bottom flange moment of inertia, I_{bf}	$= (18 \times 1.625^3)/12$	$= 6.437$ in.^{4}

Noncomposite girder moment of inertia, I_{nc}

$$= 0.633 + 13.5 \times (21.95 - 0.75/2)^2 + 1497.4 + 16.5 \times [21.95 - (0.75 + 33/2)]^2 + 6.437 + 29.25 \times (13.425 - 1.625/2)^2 \quad = 12{,}806 \text{ in.}^4$$

Section modulus for top fiber, S_t	$= I_{nc}/y_t$	$= 583$ in.^{3}
Section modulus for bottom fiber, S_b	$= I_{nc}/y_b$	$= 954$ in.^{3}

Step 3.2.2.2. Composite section property for short-term effects (n = 8)

Figure 8 shows the transformed section at $0.4L_{ds}$ for short-term load effects.

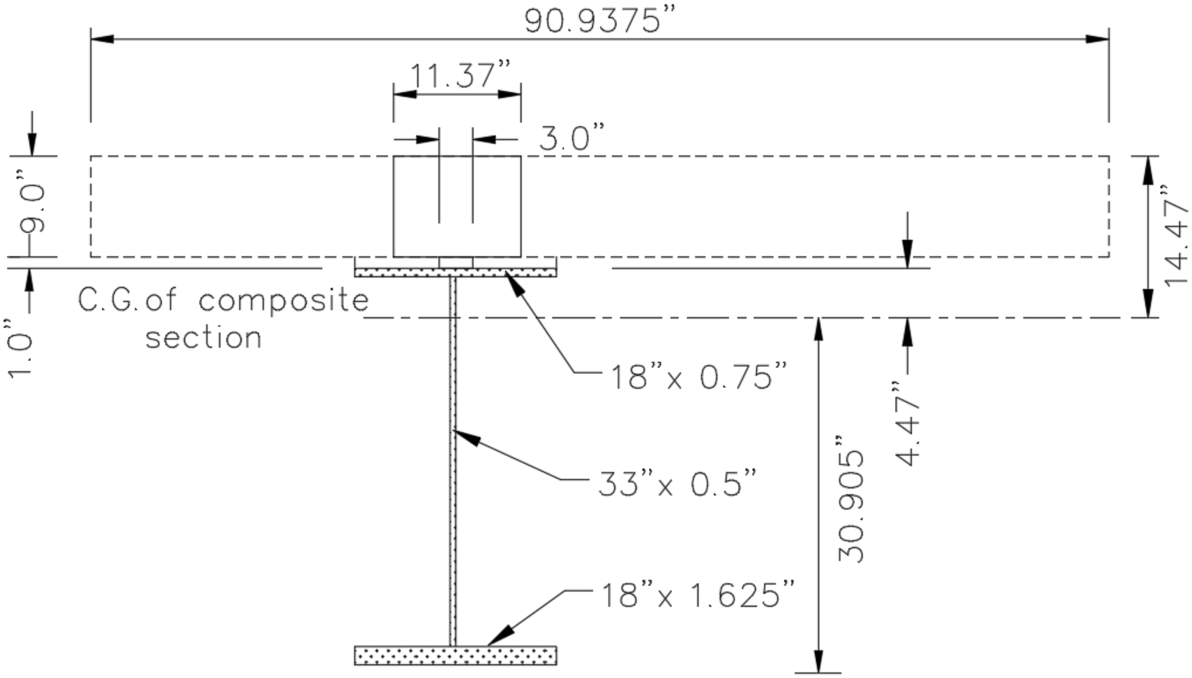


Figure 8. Transformed composite section at $0.4L_{ds}$ for short-term loading

Composite section height, h_c	= 1.625 + 33 + 0.75 + 1 + 9	= 45.375 in.
Transformed concrete deck area, A_{st}	= 11.37 × 9	= 102.33 in.^{2}
Transformed haunch area, A_{ht}	= 3 × 1	= 3 in.^{2}
Total area of the transformed section, A_c	= 102.33 + 3 + 59.25	= 164.58 in.^{2}

Distance from the centroid of the composite section to the extreme top

fiber, y_{tc} = [102.33 × (9/2) + 3 × (9 + 1/2) + 59.25 × (1 + 9 + 21.95)]/164.58 = 14.47 in.

Distance from the centroid of the composite section to the extreme bottom

fiber, y_{bc} = $h_c - y_{tc}$ = 45.375 – 14.47 = 30.905 in.

Distance from the centroid of the composite section to girder top, y_{gt}

= 14.47 – 1 - 9 = 4.47 in.

Transformed concrete deck moment of inertia, I_{st}	= $(11.37 \times 9^3)/12$	= 690.73 in.^{4}
Transformed haunch moment of inertia, I_{ht}	= $(3 \times 1^3)/12$	= 0.25 in.^{4}

Composite section moment of inertia, I_{cn8}

$$= 690.73 + 102.33 \times (14.47 - 9/2)^2 + 0.25 + 3 \times (14.47 - 9 - 1/2)^2 + 12806 + 59.25 \times (14.47 - (9 + 1 + 21.95)^2 = 41{,}847 \text{ in.}^4$$

The following section modulus values are used for girder design:

Section modulus for girder top fiber, S_t	= I_{cn8}/y_{gt}	= 9,362 in.^{3}
Section modulus for girder bottom fiber, S_b	= I_{cn8}/y_{bc}	= 1,354 in.^{3}

Step 3.2.2.3. Composite section property for long-term effects (3n = 24)

Figure 9 shows the transformed exterior girder section at $0.4L_{ds}$ for long-term effects.

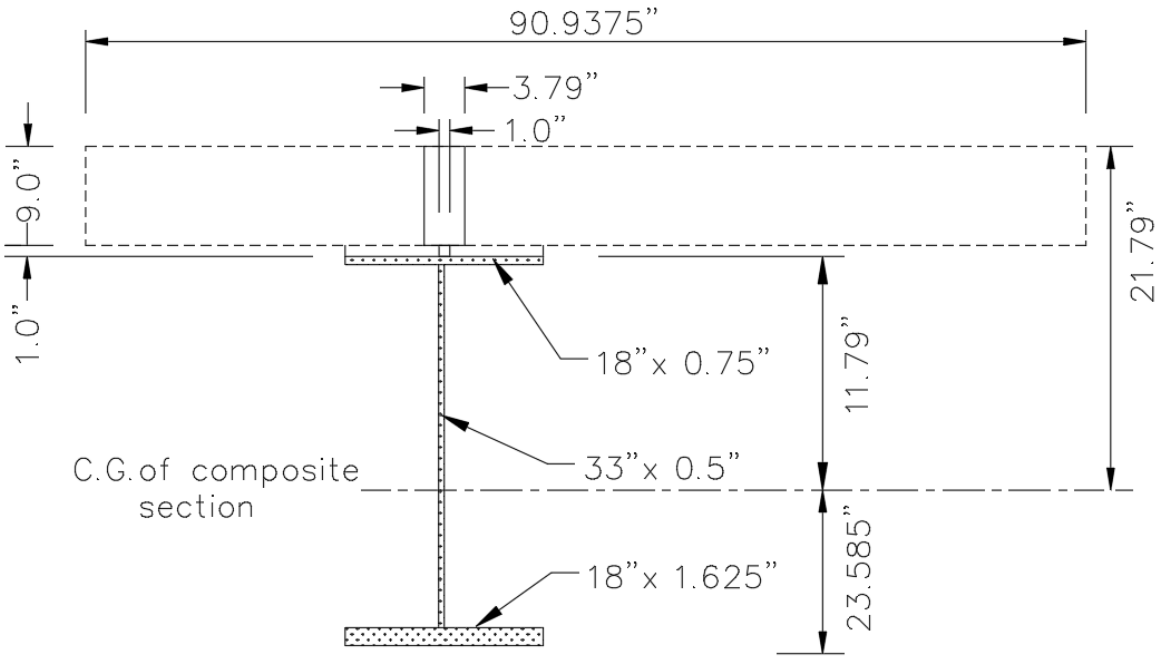


Figure 9. Transformed composite section at $0.4L_{ds}$ for long-term loading

Following a procedure similar to **Step 3.2.2.2**, the composite section properties for long-term effects are calculated.

Distance from the centroid of the composite section to the extreme top fiber, y_{tc}	= 21.79 in.
Distance from the centroid of the composite section to the extreme bottom fiber, y_{bc}	= 23.585 in.
Distance from the centroid of the section to girder top, y_{gt}	= 11.79 in.
Composite section moment of inertia, I_{cn24}	= 29,500 in.^{4}

The following section modulus values are used for girder design:

Section modulus for girder top fiber, S_t	$= I_{cn24}/y_{gt}$	= 2,502 in.^{3}
Section modulus for girder bottom fiber, S_b	$= I_{cn24}/y_{bc}$	= 1,251 in.^{3}

Table 3. Exterior Girder Section Properties

Location ($\times L_{ds}$)	Noncomposite				Composite (n = 8)			Composite (3n = 24)		
	A (in.^{2})	I (in.^{4})	S_t (in.^{3})	S_b (in.^{3})	I (in.^{4})	S_t (in.^{3})	S_b (in.^{3})	I (in.^{4})	S_t (in.^{3})	S_b (in.^{3})
0.0 (Abut A)	48.00	10418	552	656	30550	13694	939	22583	2613	865
0.1	48.00	10418	552	656	30550	13694	939	22583	2613	865
0.2	59.25	12806	583	954	41847	9362	1354	29500	2502	1251
0.3	59.25	12806	583	954	41847	9362	1354	29500	2502	1251
0.4	59.25	12806	583	954	41847	9362	1354	29500	2502	1251
0.5	59.25	12806	583	954	41847	9362	1354	29500	2502	1251
0.6	59.25	12806	583	954	41847	9362	1354	29500	2502	1251
0.7	59.25	12806	583	954	41847	9362	1354	29500	2502	1251
0.8	88.50	21874	969	1517	21874	969	1517	21874	969	1517
0.9	111.00	31535	1649	1649	31535	1649	1649	31535	1649	1649
1.0 (Pier)	111.00	31535	1649	1649	31535	1649	1649	31535	1649	1649
1.1	111.00	31535	1649	1649	31535	1649	1649	31535	1649	1649
1.2	88.50	21874	969	1517	21874	969	1517	21874	969	1517
1.3	59.25	12806	583	954	41847	9362	1354	29500	2502	1251
1.4	59.25	12806	583	954	41847	9362	1354	29500	2502	1251
1.5	59.25	12806	583	954	41847	9362	1354	29500	2502	1251
1.6	59.25	12806	583	954	41847	9362	1354	29500	2502	1251
1.7	59.25	12806	583	954	41847	9362	1354	29500	2502	1251
1.8	59.25	12806	583	954	41847	9362	1354	29500	2502	1251
1.9	48.00	10418	552	656	30550	13694	939	22583	2613	865
2.0 (Abut B)	48.00	10418	552	656	30550	13694	939	22583	2613	865

Note: Shaded cells indicate that the noncomposite I, S_t and S_b are used over the negative moment region in the absence of shear studs.

Step 3.3. Interior Girder

Table 4. Interior Girder Dimensions

<table>
<tr><th rowspan="2"></th><th rowspan="2">Girder length (ft)</th><th rowspan="2">Bottom flange thickness (in.)</th><th rowspan="2">Top flange thickness (in.)</th><th rowspan="2">Top/bottom flange width (in.)</th><th colspan="2">Web</th></tr>
<tr><th>Depth (in.)</th><th>Thickness (in.)</th></tr>
<tr><td>Distance from girder end to CL of bearing</td><td>1.33</td><td rowspan="2">0.875</td><td rowspan="3">0.75</td><td rowspan="11">18</td><td rowspan="11">33</td><td rowspan="11">0.5</td></tr>
<tr><td rowspan="9">Distance between CL of bearings</td><td>17.50</td></tr>
<tr><td>53.75</td><td>1.375</td></tr>
<tr><td>15.24</td><td>1.250</td><td rowspan="2">1.25</td></tr>
<tr><td>1.97</td><td rowspan="3">2.500</td></tr>
<tr><td>20.42</td><td>2.50</td></tr>
<tr><td>1.97</td><td rowspan="2">1.25</td></tr>
<tr><td>15.24</td><td>1.250</td></tr>
<tr><td>53.75</td><td>1.375</td><td rowspan="3">0.75</td></tr>
<tr><td>17.50</td><td rowspan="2">0.875</td></tr>
<tr><td>Distance from CL of bearing to girder end</td><td>1.33</td></tr>
</table>

Step 3.3.1. Cross-section proportion limits

The interior girder dimensions given in the above table satisfy the section dimension requirements listed in **Step 3.1**.

Table 5 shows that the interior girder dimensions satisfy the cross-section proportion limit requirements listed in **Step 3.1**.

Table 5. Cross-Section Proportion Limit Check for the Interior Girder

Location ($\times L_{ds}$)	Distance from abutment bearing CL (ft)	Flange proportions							Web proportions		
		Top flange			Bottom flange			I_{yc}/I_{yt} [d]			
		b_f [a] (in.)	t_f [b] (in.)	$b_f/2t_f$ [c]	b_f [a] (in.)	t_f [b] (in.)	$b_f/2t_f$ [c]		D (in.)	t_w (in.)	D/t_w [e]
0.0	0	18	0.75	12	18	0.875	10.29	0.86	33	0.5	66
0.1	9.867	18	0.75	12	18	0.875	10.29	0.86	33	0.5	66
0.2	19.734	18	0.75	12	18	1.375	6.55	0.55	33	0.5	66
0.3	29.601	18	0.75	12	18	1.375	6.55	0.55	33	0.5	66
0.4	39.468	18	0.75	12	18	1.375	6.55	0.55	33	0.5	66
0.5	49.335	18	0.75	12	18	1.375	6.55	0.55	33	0.5	66
0.6	59.202	18	0.75	12	18	1.375	6.55	0.55	33	0.5	66
0.7	69.069	18	0.75	12	18	1.375	6.55	0.55	33	0.5	66
0.8	78.936	18	1.25	7.2	18	1.250	7.20	1.00	33	0.5	66
0.9	88.803	18	2.50	3.6	18	2.500	3.60	1.00	33	0.5	66
1.0	98.670	18	2.50	3.6	18	2.500	3.60	1.00	33	0.5	66
1.1	108.537	18	2.50	3.6	18	2.500	3.60	1.00	33	0.5	66
1.2	118.404	18	1.25	7.2	18	1.250	7.20	1.00	33	0.5	66
1.3	128.271	18	0.75	12	18	1.375	6.55	0.55	33	0.5	66
1.4	138.138	18	0.75	12	18	1.375	6.55	0.55	33	0.5	66
1.5	148.005	18	0.75	12	18	1.375	6.55	0.55	33	0.5	66
1.6	157.872	18	0.75	12	18	1.375	6.55	0.55	33	0.5	66
1.7	167.739	18	0.75	12	18	1.375	6.55	0.55	33	0.5	66
1.8	177.606	18	0.75	12	18	1.375	6.55	0.55	33	0.5	66
1.9	187.473	18	0.75	12	18	0.875	10.29	0.86	33	0.5	66
2.0	197.340	18	0.75	12	18	0.875	10.29	0.86	33	0.5	66

a - $b_f \geq D/6 = 5.5$ in.
b - $t_f \geq 1.1t_w = 0.55$ in.
c - $b_f/2t_f \leq 12.0$
d - $0.1 \leq I_{yc}/I_{yt} \leq 10$
e - $D/t_w \leq 150$

Step 3.3.2. Noncomposite and composite section properties

Effective flange width, b_{eff}	= 9 ft - 8 $^5/_8$ in.	= 116.625 in.	LRFD Art. 4.6.2.6.1
Transformed flange width using n	= b_{eff}/n = 116.625/8	= 14.58 in.	
Transformed haunch width using n	= b_h/n = 24/8	= 3 in.	
Transformed flange width using 3n	= $b_{eff}/3n$ = 116.625/24	= 4.86 in.	
Transformed haunch width using 3n	= $b_h/3n$ = 24/24	= 1 in.	

The moment of inertia (I) and the section modulus (S) of the girder at 1/10th of the design span (L_{ds}) are summarized in **Table 6**, provided at the end of **Step 3.3**.

Step 3.3.2.1 and **Step 3.3.2.2** show the calculation procedures for noncomposite and composite section properties of the interior girder section over the pier (i.e. at 1.0L_{ds}).

Step 3.3.2.1. Noncomposite section property

Figure 10 shows the interior girder section over the pier ($1.0L_{ds}$).

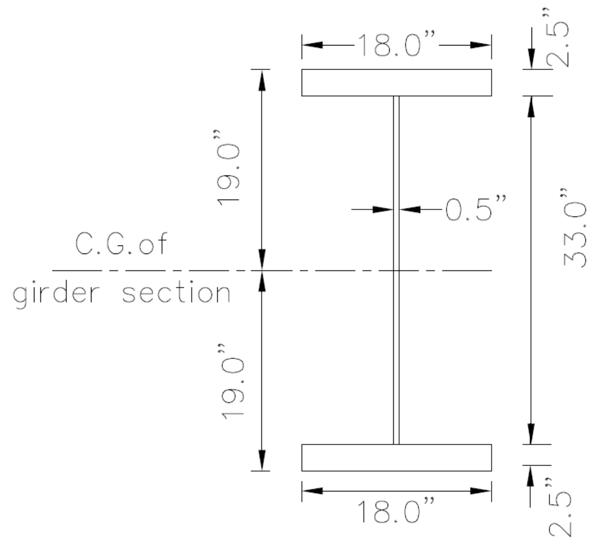


Figure 10. Interior girder section over the pier

Girder height, h	= 2.5 + 33 + 2.5	= 38 in.
Area of the top flange, A_{tf}	= 18 × 2.5	= 45 in.^{2}
Area of the web, A_w	= 33 × 0.5	= 16.5 in.^{2}
Area of the bottom flange, A_{bf}	= 18 × 2.5	= 45 in.^{2}
Area of the noncomposite section, A_{nc}	= 45 + 45 + 16.5	= 106.5 in.^{2}

Distance from the centroid to the extreme top fiber, y_t

$= [45 \times (2.5/2) + 16.5 \times (2.5 + 33/2) + 45 \times (2.5 + 33 + 2.5/2)]/106.5 = 19$ in.

Distance from the centroid to the extreme bottom fiber, y_b

$= h - y_t = 38 - 19 = 19$ in.

Top flange moment of inertia, I_{tf}	$= (18 \times 2.5^3)/12$	= 23.4375 in.^{4}
Web moment of inertia, I_w	$= (0.5 \times 33^3)/12$	= 1497.4 in.^{4}
Bottom flange moment of inertia, I_{bf}	$= (18 \times 2.5^3)/12$	= 23.4375 in.^{4}

Noncomposite girder moment of inertia, I_{nc}

$$= 23.4375 + 45 \times (19 - 2.5/2)^2 + 1497.4 + 16.5 \times [19 - (2.5 + 33/2)]^2 + 23.4375 + 45 \times (19 - 2.5/2)^2 = 29{,}900 \text{ in.}^4$$

Section modulus for top fiber, S_t	$= I_{nc}/y_t$	= 1,574 in.^{3}
Section modulus for bottom fiber, S_b	$= I_{nc}/y_b$	= 1,574 in.^{3}

Step 3.3.2.2. Composite section properties for short-term effects (n = 8) and long-term effects (3n = 24)

The section over the pier has no shear studs. Therefore, noncomposite section properties are used.

Table 6. Interior Girder Section Properties

Location (×L_{ds})	Noncomposite				Composite (n = 8)			Composite (3n = 24)		
	A (in.^{2})	I (in.^{4})	S_t (in.^{3})	S_b (in.^{3})	I (in.^{4})	S_t (in.^{3})	S_b (in.^{3})	I (in.^{4})	S_t (in.^{3})	S_b (in.^{3})
0.0 (Abut A)	45.75	9829	543	595	29613	50741	870	22628	3487	804
0.1	45.75	9829	543	595	29613	50741	870	22628	3487	804
0.2	54.75	11945	573	837	39692	17904	1206	29224	3227	1121
0.3	54.75	11945	573	837	39692	17904	1206	29224	3227	1121
0.4	54.75	11945	573	837	39692	17904	1206	29224	3227	1121
0.5	54.75	11945	573	837	39692	17904	1206	29224	3227	1121
0.6	54.75	11945	573	837	39692	17904	1206	29224	3227	1121
0.7	54.75	11945	573	837	39692	17904	1206	29224	3227	1121
0.8	61.50	14700	828	828	14700	828	828	14700	828	828
0.9	106.5	29900	1574	1574	29900	1574	1574	29900	1574	1574
1.0 (Pier)	106.5	29900	1574	1574	29900	1574	1574	29900	1574	1574
1.1	106.5	29900	1574	1574	29900	1574	1574	29900	1574	1574
1.2	61.50	14700	828	828	14700	828	828	14700	828	828
1.3	54.75	11945	573	837	39692	17904	1206	29224	3227	1121
1.4	54.75	11945	573	837	39692	17904	1206	29224	3227	1121
1.5	54.75	11945	573	837	39692	17904	1206	29224	3227	1121
1.6	54.75	11945	573	837	39692	17904	1206	29224	3227	1121
1.7	54.75	11945	573	837	39692	17904	1206	29224	3227	1121
1.8	54.75	11945	573	837	39692	17904	1206	29224	3227	1121
1.9	45.75	9829	543	595	29613	50741	870	22628	3487	804
2.0 (Abut B)	45.75	9829	543	595	29613	50741	870	22628	3487	804

Note: Shaded cells indicate that the noncomposite I, S_t and S_b are used over the negative moment region in the absence of shear studs.

STEP 4. LOADS

Figure 11 shows the loads and girder sections used for load response analysis. As shown in the figure, girder self-weight, stay-in-place (SIP) formwork load, and deck and haunch weight are applied on noncomposite girder sections. Deck placement analysis uses composite sections calculated using n = 8 for the girder segments with hardened concrete. Live load is applied on composite sections calculated using n = 8. Superimposed dead load (e.g. barrier and future wearing surface weights) and support settlement responses are evaluated using composite sections calculated with 3n = 24.

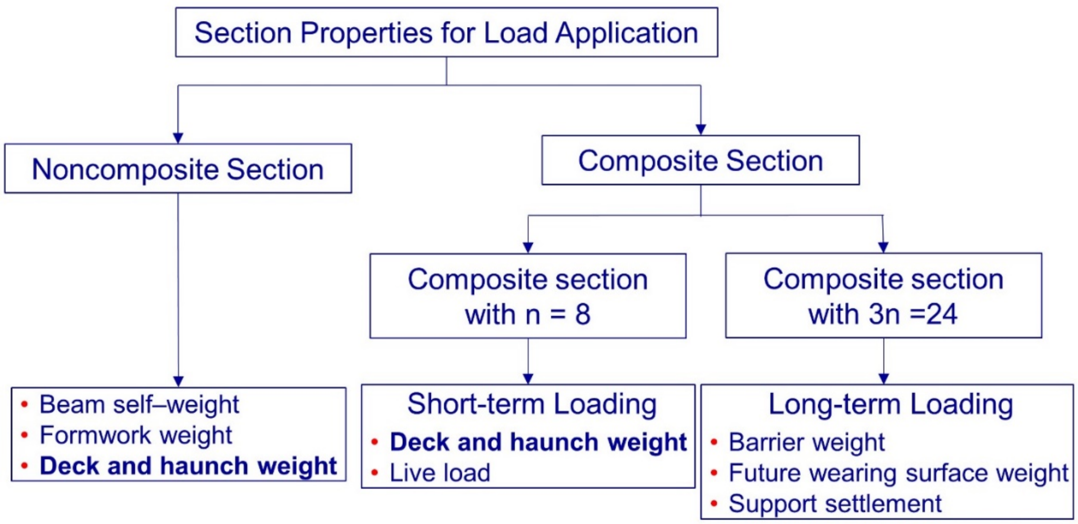


Figure 11. Section properties used in load response analysis

Step 4.1. Dead Loads (DC and DW)

The structural and nonstructural component dead loads are divided into two groups as shown below:

1. Dead loads acting on the girders before the concrete deck is hardened, and
2. Superimposed dead loads: dead loads acting on the composite section.

Dead loads acting on the girders before the concrete deck is hardened are resisted by the noncomposite steel girder. Loads acting on the noncomposite girder include girder self-weight, concrete slab and haunch weight, and stay-in-place formwork weight. Superimposed dead loads are resisted by the long-term composite section (3n = 24). Barrier and future wearing surface weights represent superimposed dead loads.

The following notations are used in this example:

DC - Dead loads acting on the girders before the concrete deck is hardened and barrier weight

DW - Future wearing surface weight

Step 4.1.1. Steel girder self-weight

Girder section changes along the length. Depending on the capabilities of the available analysis tools, the girder sections are explicitly modeled, or an average weight of the girder is used as a uniformly distributed load.

MDOT BDS uses an average steel girder weight as a uniformly distributed load. To account for the additional steel weight from cross-frames, diaphragms, stiffeners, etc., girder weight is multiplied by a factor of 1.2.

Step 4.1.2. Deck self-weight

Cast-in-place concrete deck weight on an interior girder

$= (b_{eff} \times t_s)\, w_c = [(116.625 \text{ in.} \times 9 \text{ in.}) / 12^2] \times 0.150 \text{ kip/ft}^3$ = 1.093 kip/ft

Cast-in-place concrete deck weight on an exterior girder

$= (b_{eff} \times t_s)\, w_c = [(90.9375 \text{ in.} \times 9 \text{ in.}) / 12^2] \times 0.150 \text{ kip/ft}^3$ = 0.853 kip/ft

Step 4.1.3. Haunch self-weight

Haunch width is maintained at the same width of the top flange when stay-in-place formwork is used. For removable formwork, haunch width is extended 3 in. from either side of the girder top flange.

This example considers stay-in-place formwork and haunch width of 18 in. for dead load calculation.

Concrete haunch weight on an interior or exterior girder

$= (b_{tf} \times t_h)\, w_c = [(18 \text{ in.} \times 1 \text{ in.})/12^2] \times 0.150 \text{ kip/ft}^3$ = 0.01875 kip/ft

Step 4.1.4. Stay-in-place (SIP) formwork weight

For new bridges or superstructure replacements, a design load of 15 LBS/SFT should be added for the use of stay in place metal forms. BDM Art. 7.01.04 I

If the following statement is included in the bridge plans, the design load of 15 psf is waived.

The contractor may use metal stay in place forms. If used, eliminating the polystyrene and filling the corrugations with concrete is prohibited. [Use with metal stay in place forms where design calculations show the increase in dead load will result in an overstress.] BDM Art. 8.07.01R

SIP formwork weight, w_f = 0.015 ksf BDM Art 7.01.04 I

SIP formwork weight on an interior girder

$= (b_{eff} - b_{tf})\, w_f = [(116.625 - 18)/12] \times 0.015 \text{ ksf}$ = 0.123 kip/ft

SIP formwork weight on an exterior girder

$= (b_{eff} - b_{tf})\, w_f = [(90.9375 - 18)/12] \times 0.015 \text{ ksf}$ = 0.091 kip/ft

The deck, haunch, and SIP formwork weights are applied as uniformly distributed loads on exterior and interior girders to calculate moments and shear forces acting on the noncomposite sections.

Step 4.1.5. Superimposed dead loads

Step 4.1.5.1. Barrier weight

Barrier loads are distributed equally to all girders. BDM Art. 7.01.04-J

- Width of the deck is constant **OK**
- Number of girders, $N_b \geq 4$ **OK**
- The roadway section of the overhang, $d_e \leq 3.0$ ft
  d_e = 32.625 in. – 19.5 in. = 1 ft – 1.125 in. **OK**
- Effect of curvature in plan can be ignored for the following reasons: LRFD Art. 4.6.1.2.4b
  - Girders are concentric.
  - Bearing lines are not skewed more than 10 degrees from radial.
  - The stiffness of the girders is similar.
  - The arc span divided by the girder radius in feet is less than 0.06 radians.
- Cross-section of the bridge is consistent with LRFD Table 4.6.2.2.1-1 **OK**

The criteria are satisfied and the barrier loads are equally distributed to all 7 girders.

Type 4 barrier or railing weight, w_b = 0.475 kip/ft — BDG 6.29 Series or BDG 1.21.01

Barrier weight on a girder, w_b
= (2 barriers)(0.475 kip/ft)/(7 girders) = 0.136 kip/ft

Step 4.1.5.2. Future wearing surface (FWS) weight

New bridges and bridge replacements shall be designed for a future wearing surface load of 25 LBS/SFT. BDM Art 7.01.04-H

FWS weight on one girder, w_{ws}
= (0.025 ksf)(60.5 ft)/(7 girders) = 0.216 kip/ft
where, clear roadway width is 60 ft – 6 in.

The barrier and FWS weights are applied as uniformly distributed loads on the long-term composite sections (3n = 24) of the interior and exterior girders to calculate the resulting moments and shear forces.

Step 4.2. Live Loads (LL)

Step 4.2.1. Design vehicular live load

Live loads are applied to the short-term composite girder section (n = 8).

The load modifying factor for ductility, redundancy, and operational importance, η = 1.0

The load modifying factor, η (eta), related to ductility, redundancy, and operational importance, shall be considered for less important roads. BDM Art. 7.01.04-B

Typically, 1.0 is used.

Vehicular live loading on the roadways of bridges designated HL-93 Mod, shall consist of 1.2 times the combination of the following: BDM Art. 7.01.04-A

- *Design truck or single 60-kip load*
- *Design lane load*

A design truck is shown in **Figure 12**. The design lane load is 0.64 kip/ft. LRFD Art. 3.6.1.2.1

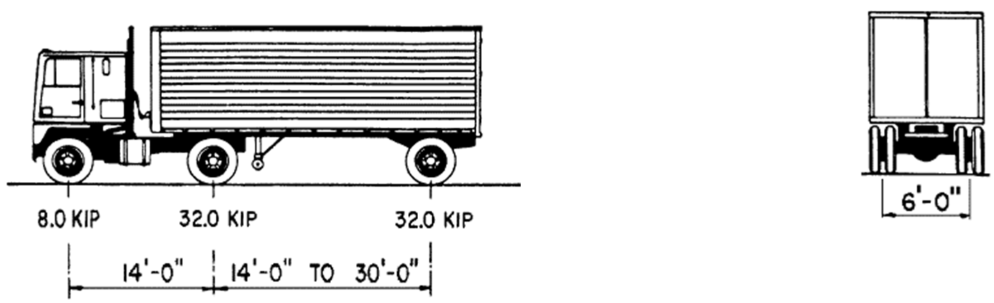


Figure 12. Characteristics of the design truck (LRFD Figure 3.6.1.2.2-1)

Where 90% of two design trucks is combined with 90% of the effect of a lane load for both negative moment and pier reactions per A.3.6.1.3, a 1.2 multiplier shall be applied to the resulting moment or load. Each design lane under consideration shall be occupied by either the design truck or single 60-kip load, coincident with the lane load, where applicable. The loads shall be assumed to occupy 10.0 ft transversely within a design lane. BDM Art. 7.01.04-A

Vehicular live loading on the roadways of bridges designated HL-93 Mod, shall consist of 1.2 times the combination of the following:

- *Design truck for continuity design*
- *Design lane load*

Design truck configuration for continuity design is shown in **Figure 13**. The design lane load is 0.64 kip/ft. LRFD Art. 3.6.1.3.1

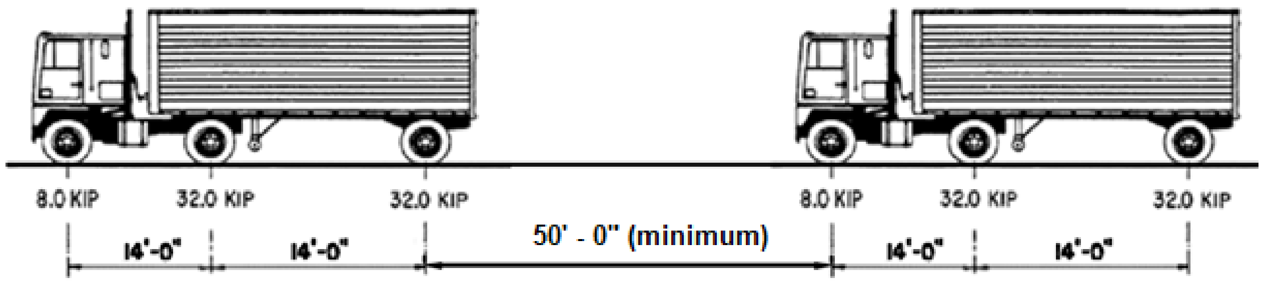


Figure 13. Characteristics of the design truck configuration for continuity design

The minimum headway between the front and rear axles of the two trucks is 50 ft. This spacing needs to be changed to generate the maximum load effects based on the span length of the continuous bridge. The distance between the two 32-kip axles of each truck is 14 ft.

When using software for live load modeling, the ability of the software to handle this two-truck configuration should be investigated.

Step 4.2.2. Dynamic load allowance

The design truck load is increased by a dynamic allowance factor. LRFD Table 3.6.2.1-1

Dynamic allowance, IM (for fatigue and fracture limit state) = 15%
Dynamic allowance, IM (for all other limit states) = 33%

Step 4.2.3. Loading for optional live-load deflection evaluation

The deflection should be taken as the larger of the following: LRFD Art. 3.6.1.3.2

- *That resulting from the design truck alone, or*
- *That resulting from 25% of the design truck taken together with the design lane load.*

In applying these criteria, the vehicular load shall include the dynamic load allowance of 33%. When investigating the maximum absolute deflection for straight girder systems, all design lanes should be loaded, and all supporting components should be assumed to deflect equally. LRFD Art. 2.5.2.6.2

Step 4.2.4. Fatigue load

The fatigue load is represented as a design truck with a constant spacing of 30 ft between the 32.0-kip axles as shown in **Figure 14**. LRFD Art. 3.6.1.4.1

The design truck used in the fatigue moment and shear force calculations is referred to as "HS20 fatigue truck" throughout this example.

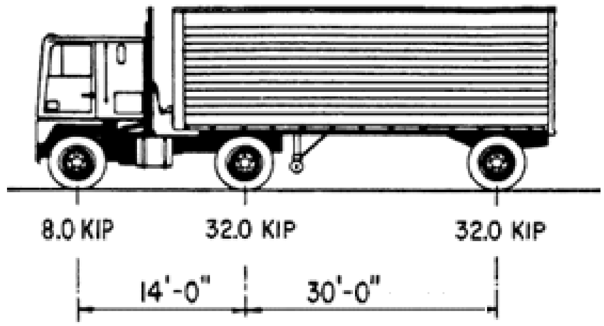


Figure 14. Characteristics of the fatigue design truck

Step 4.3. Support Settlement (SE)

A negative moment caused by a 1 in. settlement of a substructure unit is imposed when non-spread footings are used. A settlement of 2 in. is imposed when the bridge is on spread footings.

A support settlement of 1 in. is applied at both abutments simultaneously to calculate the resulting forces in the interior and exterior girders of the bridge.

STEP 5. LIVE LOAD DISTRIBUTION FACTORS

As shown later in this step, the girders satisfy the limitations defining the range of applicability of the approximate equations; these limitations are specified in the individual AASHTO LRFD tables containing the live load distribution factor equations. For example, the number of girders in the cross-section is greater than or equal to four, the transverse girder spacing is greater than or equal to 3.5 ft and less than or equal to 16 ft, and the span length is greater than or equal to 20 ft and less than or equal to 240 ft. The limitations on the slab thickness are also satisfied.

These standards allow for the determination of live load moments and shears using the simplified factor equations.

Bridge type for steel I- girder with a cast-in-place concrete deck is (a). LRFD Table 4.6.2.2.1-1

The number of 12 ft wide design lanes = integer part of the ratio of (w/12) LRFD Art 3.6.1.1.1
where, w is the clear roadway width, in ft, between the curbs.

w (see **Figure 2**)		= 60 ft - 6 in.
Number of design lanes	= integer part of (60.5/12)	= 5 lanes

Hence, 5 - 12 ft wide lanes can be accommodated within the clear roadway width.

Step 5.1. Interior Girder Live Load Distribution Factor

Figure 15 shows the procedure for calculating distribution factors for moment and shear of an interior girder. Notations are defined in the subsequent sections with calculations.

The live load distribution factors for an interior girder for checking the strength limit state are determined using the equations given in the indicated tables. Multiple presence factors (Article 3.6.1.1.2) are included in the equations. Separate factors are given to compute the bending moment and shear. LRFD Table 4.6.2.2.2b-1 & 4.6.2.2.3a-1

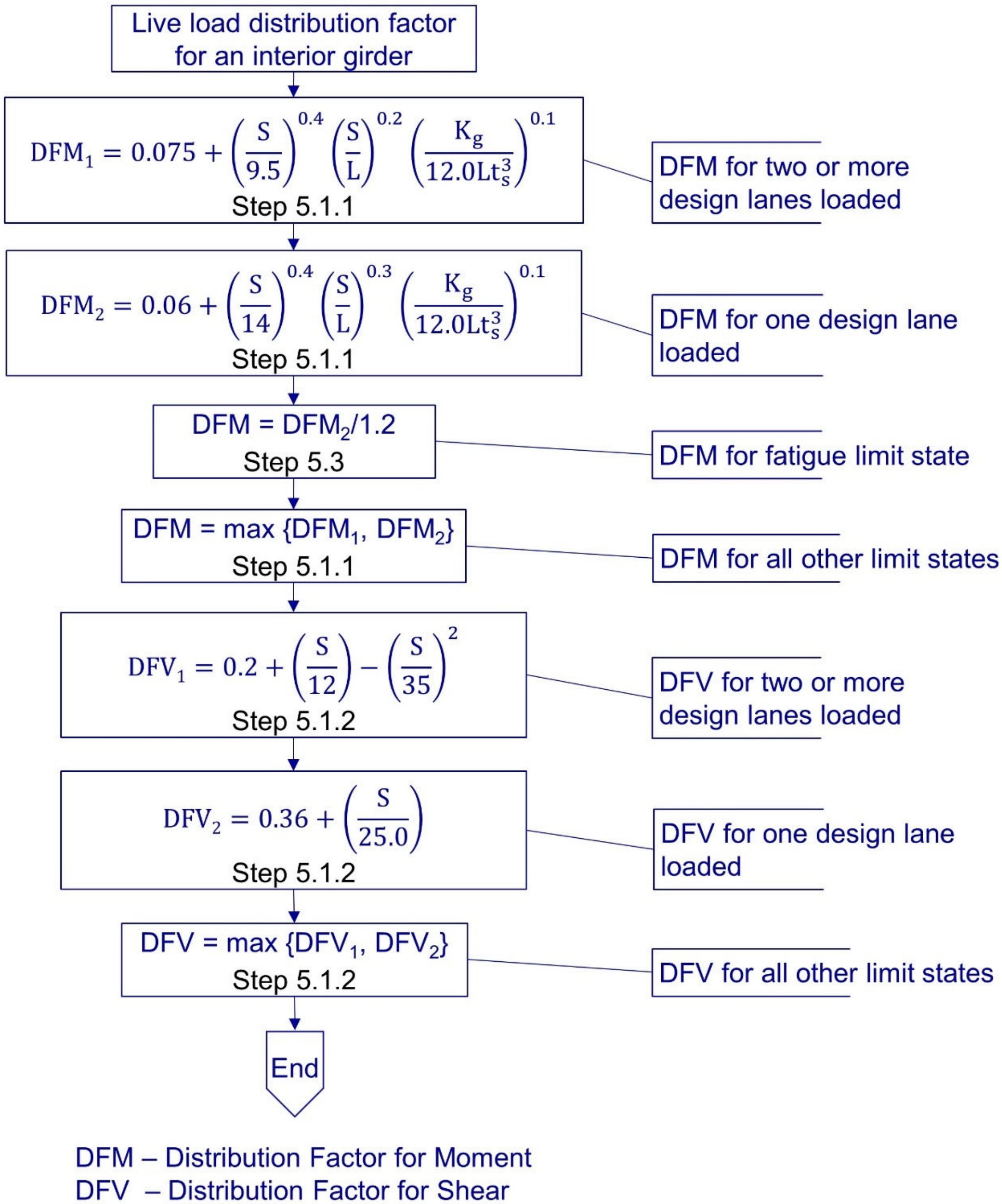


Figure 15. Live load moment and shear distribution factor calculation procedure for an interior girder

Step 5.1.1. Distribution factor for moment

For all limit states, except fatigue:

For two or more design lanes loaded:

$$\text{DFM} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$$

LRFD Table 4.6.2.2.2b-1

For one design lane loaded:

$$\text{DFM} = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$$

LRFD Table 4.6.2.2.2b-1

where:

DFM = distribution factor for moment

K_g = longitudinal stiffness parameter = n $(I + Ae_g^2)$ — LRFD Eq. 4.6.2.2.1-1

L = design span (= L_{ds}) = 98.67 ft

S = girder spacing = 9.71875 ft

t_s = deck slab thickness = 9 in.

where:

A = cross-sectional area of the girder (noncomposite section)

e_g = distance between centers of gravity of the girder and slab
= $t_s/2 + t_h + y_t$

I = moment of inertia of the girder (noncomposite section)

n = modular ratio between girder and slab material

t_h = haunch thickness

y_t = distance from the centroid of the section to the extreme top fiber

In this example, A, I, and e_g vary along the length of the girders, i.e. K_g changes. For preliminary design, the entire term containing K_g in the equation may be taken as 1.0.

After girder sections are selected, the following approaches are commonly used to calculate K_g: — LRFD Art. C4.6.2.2.1

(i) the value at the maximum positive moment sections

(ii) the average or a weighted-average value

(iii) the actual values at each section.

However, the distribution factor is typically not overly sensitive to the value of K_g.

Weighted-average $K_g = \sum(K_{gi} \times L_i)/\sum L_i$

where:

K_{gi} = longitudinal stiffness parameter of girder segment with uniform section properties

L_i = girder segment length with uniform section properties

K_g values for each section and the weighted-average values of the interior girder are shown in **Table 7**.

To use DFM equations, the following criteria need to be satisfied:

Range of Applicability LRFD Table 4.6.2.2.2b-1 and 4.6.2.2.3a-1	MDOT Policy BDG 6.41.01 and BDM Art. 7.02.02	Bridge Attributes
3.5 ft $\leq$ S $\leq$ 16 ft	S $\leq$ 10 ft for I beams	S = 9 ft – 8.625 in. **OK**
4.5 in. $\leq t_s \leq$ 12 in.	t_s = 9 in.	t_s = 9 in. **OK**
20 ft $\leq$ L $\leq$ 240 ft		L_{ds} = 98.67 ft **OK**
$N_b \geq 4$		N_b = 7 **OK**
10,000 in.^{4} $\leq K_g \leq$ 7,000,000 in.^{4}		K_g = 418,742 in.^{4} **OK**

The value of span length (L) to be used in the distribution factor equations is given in the AASHTO LRFD as follows: LRFD Table 4.6.2.2.1-2

Force Effect	Length, L (ft)	Value used in this example, (ft)
Positive Moment	Length of the span for which moment is being calculated	
Negative Moment – near interior supports of continuous spans between points of contraflexure under a uniform load on all spans	Average length of the two adjacent spans	
Negative Moment – other than near interior supports of continuous spans	Length of the span for which moment is being calculated	**98.67**
Shear	Length of the span for which shear is being calculated	
Exterior Reaction	Length of the exterior span	
Interior Reaction of Continuous Span	Average length of the two adjacent spans	

The span length of **98.67 ft** is used for positive and negative regions because of equal span lengths.

DFM is calculated using two approaches:

(1) DFM at each section

(2) DFM using the weighted-average K_g or weighted-average DFM at each section.

Weighted-average DFM = $\sum(DFM_i \times L_i)/\sum L_i$

where:

DFM_i = distribution factor for moment of a girder segment with uniform section properties

Table 7 presents (i) K_g and DFM at each section, (ii) the weighted-average K_g and DFM, and (iii) DFM selected for design.

Table 7. Distribution Factor for Moment (DFM) of an Interior Girder

Girder segment length (ft)	A (in.^{2})	y_t (in.)	I (in.^{4})	e_g (in.)	K_g (in.^{4})	DFM (lanes/girder) 2 or more design lanes loaded (a)	DFM (lanes/girder) One design lane loaded (b)	DFM for design (lanes/girder) Max [(a), (b)]
17.50 (from bearing CL)	45.75	18.10	9829	23.60	282479	0.645	0.446	0.645
53.75	54.75	20.86	11945	26.36	399904	0.665	0.459	0.665
15.24	61.50	17.75	14700	23.25	383557	0.663	0.458	0.663
1.97	84.00	22.67	20265	28.17	695385	0.699	0.482	0.699
10.21 (to pier bearing CL)	106.50	19.00	29900	24.50	750613	0.704	0.485	0.704
Weighted-average					**418742**	**0.666**	**0.460**	**0.666**
DFM with weighted-average K_g						**0.668**	**0.461**	**0.668**

As shown in the table, none of these methods results in significant differences in the DFM, since K_g is the only variable that changes along the length.

The following example demonstrates the calculation of DFM using weighted-averages:

With the weighted-average K_g:

For two or more design lanes loaded:

$$\mathrm{DFM} = 0.075 + \left(\frac{9.71875}{9.5}\right)^{0.6}\left(\frac{9.71875}{98.67}\right)^{0.2}\left(\frac{418742}{12.0 \times 98.67 \times 9^3}\right)^{0.1} = 0.668 \text{ lanes/girder}$$

LRFD Table 4.6.2.2.2b-1

For one design lane loaded:

$$\mathrm{DFM} = 0.06 + \left(\frac{9.71875}{14}\right)^{0.4}\left(\frac{9.71875}{98.67}\right)^{0.3}\left(\frac{418742}{12.0 \times 98.67 \times 9^3}\right)^{0.1} = 0.461 \text{ lanes/girder}$$

LRFD Table 4.6.2.2.2b-1

Two or more design lanes loaded governs:

A DFM with weighted-average K_g = 0.668 lanes/girder

With the weighted-average DFM:

For two or more design lanes loaded:

(0.645 × 17.50 + 0.665 × 53.75 + 0.663 × 15.24 + 0.699 × 1.97 + 0.704 × 10.21)/98.67 = 0.666 lanes/girder

For one design lane loaded:

(0.446 × 17.50 + 0.459 × 53.75 + 0.458 × 15.24 + 0.482 × 1.97 + 0.485 × 10.21)/98.67 = 0.460 lanes/girder

Two or more design lanes loaded governs:

A Weighted-average DFM = 0.666 lanes/girder

As shown above, both methods yield similar results.

A case with two or more design lanes loaded governs: A DFM of **0.668 lanes/girder** is used for further calculations.

Even though the weighted-average K_g of 418,742 in.^{4} is 4.5% greater than the K_g at the maximum positive moment location, the difference between DFM calculated with the weighted-average K_g and the DFM at the maximum positive moment is less than 0.5%.

Step 5.1.2. Distribution factor for shear

For all limit states, except fatigue:

For two or more design lanes loaded:

$$DFV = 0.2 + \left(\frac{S}{12}\right) - \left(\frac{S}{35}\right)^2$$

LRFD Table 4.6.2.2.3a-1

For one design lane loaded:

$$DFV = 0.36 + \left(\frac{S}{25.0}\right)$$

LRFD Table 4.6.2.2.3a-1

where:

DFV = distribution factor for shear in an interior girder
S = girder spacing = 9.71875 ft

For two or more design lanes loaded:

$$DFV = 0.2 + \left(\frac{9.71875}{12}\right) - \left(\frac{9.71875}{35}\right)^2$$ = 0.933 lanes/girder

For one design lane loaded:

$$DFV = 0.36 + \left(\frac{9.71875}{25.0}\right)$$ = 0.749 lanes/girder

A case with two or more design lanes loaded controls, and DFV = **0.933 lanes/girder.**

Step 5.2. Exterior Girder Live Load Distribution Factor

Figure 16 shows the procedure for calculating distribution factors for moment and shear of an exterior girder. Notations are defined in the subsequent sections with calculations.

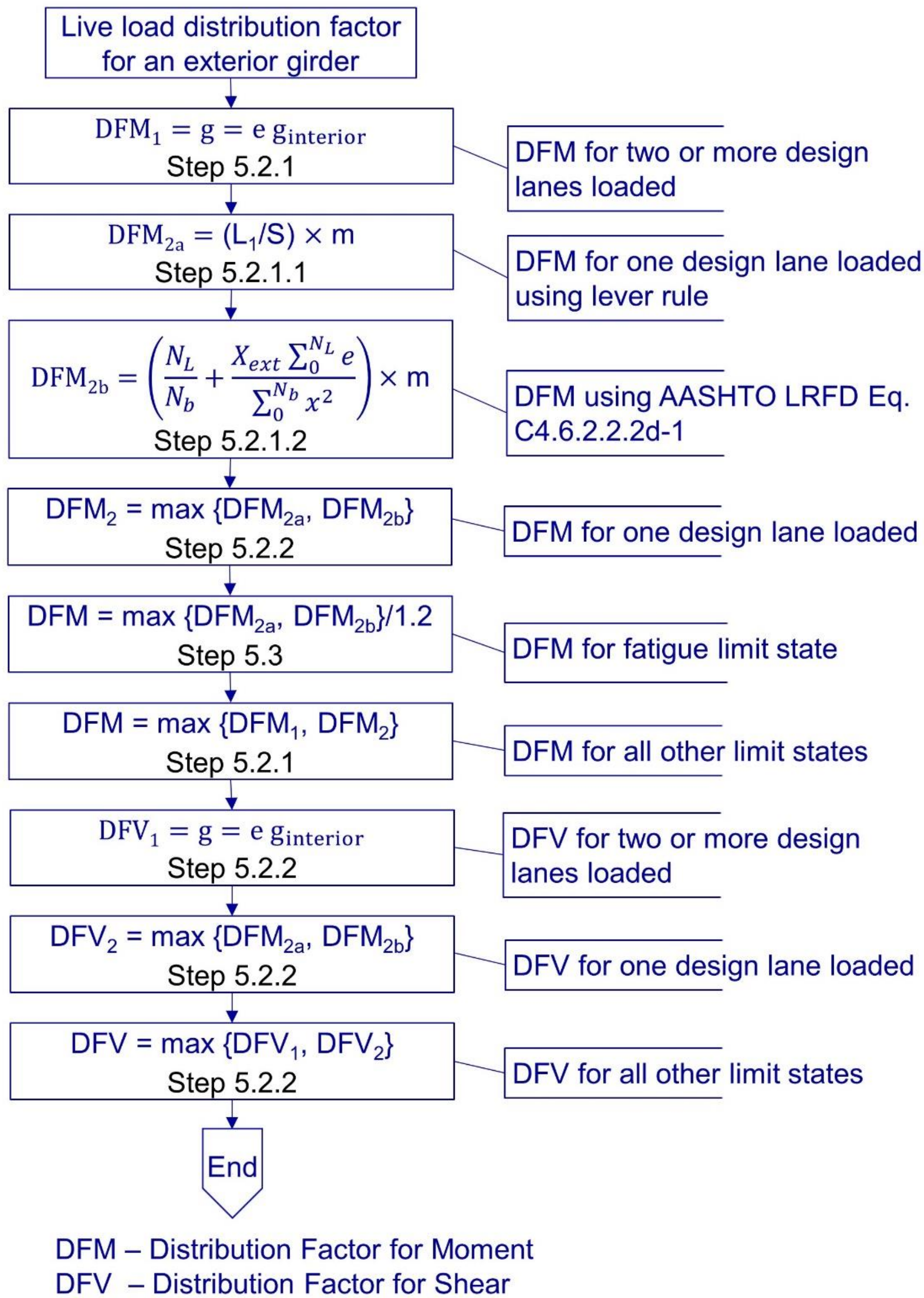


Figure 16. Live load moment and shear distribution factor calculation procedure for an exterior girder

Step 5.2.1. Distribution factor for moment

For two or more design lanes loaded:

$$g = e\, g_{interior}$$

$$e = 0.77 + \frac{d_e}{9.1}$$

LRFD Table 4.6.2.2.2d-1

where:

d_e = horizontal distance from the exterior girder web centerline to the interior edge of curb or traffic barrier

d_e = (32.625 - 18 - 1.5)/12 = 1.094 ft

$g_{interior}$ = distribution factor for moment in the interior girder

$$e = 0.77 + \frac{1.094}{9.1} = 0.890$$

The interior girder geometry is different from the exterior girder used in this example. Therefore, the distribution factor for moment of an interior girder ($g_{interior}$) in a case with two or more design lanes loaded cannot be used. This requires calculating new $g_{interior}$ values using the exterior girder as an interior girder.

Table 8 shows $g_{interior}$ values calculated using the AASHTO LRFD equation for a case with two or more design lanes loaded and the exterior girder section properties.

Table 8. Distribution Factor for Moment of an Interior Girder ($g_{interior}$) with Exterior Girder Dimensions

Girder segment length (ft)	A (in.^{2})	y_t (in.)	I (in.^{4})	e_g (in.)	K_g (in.^{4})	$g_{interior}$ (DFM for 2 or more design lanes loaded)	DFM_E ($e \times g_{interior}$)
19.08 (from bearing CL)	48.00	18.88	10418	24.38	311588	0.651	0.579
52.20	59.25	21.95	12806	27.45	459608	0.674	0.600
16.14	88.50	22.58	21874	28.08	733240	0.702	0.625
11.25 (to pier bearing CL)	111.00	19.13	31535	24.63	790974	0.707	0.629
				Weighted-average	**513493**	**0.678**	**0.603**
				DFM with weighted-average K_g		**0.680**	**0.606**

The DFM of **0.606 lanes/girder** for two or more design lanes loaded is used for further calculations.

For one design lane loaded:

LRFD Table 4.6.2.2.2d-1

The following two methods are considered for the calculation of live load distribution factor for a case with one design lane loaded:

- Lever rule based on the resultant of truck load
- Lever rule based on the wheel load.

Step 5.2.1.1. Lever rule

*The lever rule involves the use of static to determine the lateral distribution to the exterior girder by summing moment about the adjacent interior girder to find the wheel-load reaction at the exterior girder assuming the concrete deck is hinged at the interior girder (**Figure 17**). A wheel cannot be closer than 2 feet to the base of the curb (Article 3.6.1.3.1). For the specified transverse wheel spacing of 6 feet, the wheel-load distribution to the exterior girder is computed.*

LRFD Art. 4.6.2.2.1 & C4.6.2.2.1

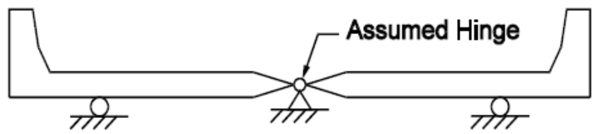


Figure 17. Notional model for applying the lever rule to three-girder bridges (LRFD Figure C4.6.2.2.1-1)

When both wheel lines are located between the barrier and the 1st interior girder, as shown in **Figure 18**, the lever rule based on the resultant of the truck load is applied. In bridges with narrow girder spacing, there is a possibility to have one of the wheel lines located between the 1st and 2nd interior girders; thus, the lever rule based on the wheel load is applied.

For this bridge, both wheel lines are located between the barrier and the 1st interior girder, as shown in **Figure 18.** Therefore, the lever rule based on the resultant of truck load is applied.

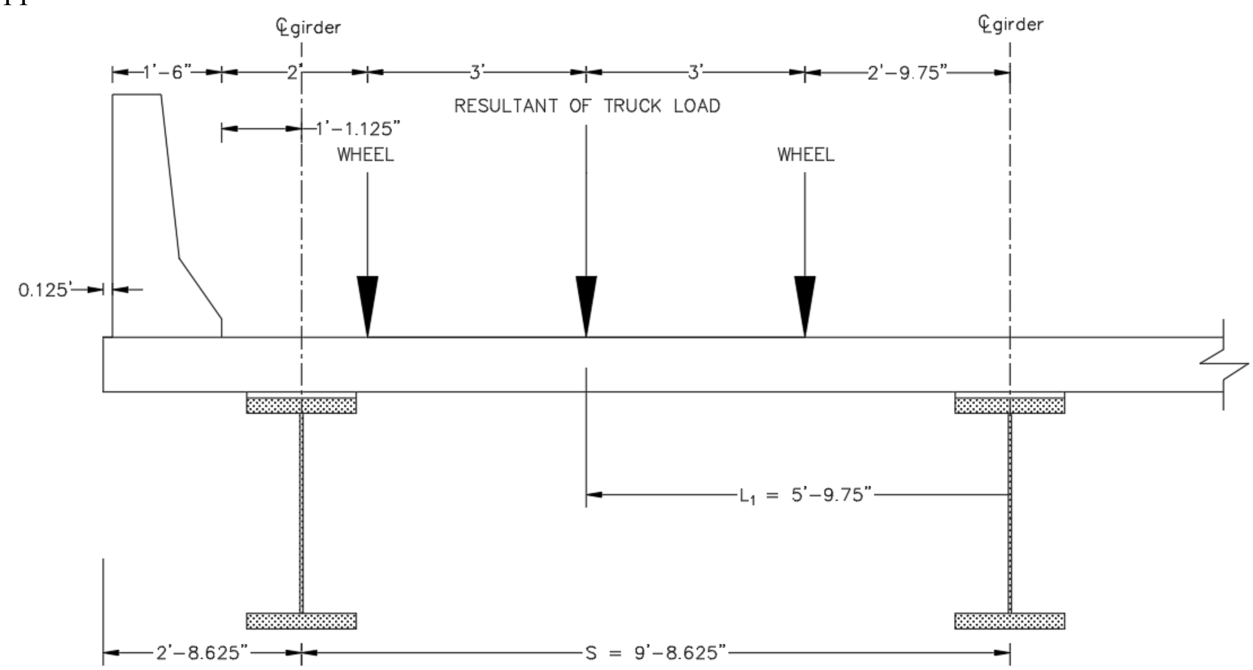


Figure 18. Dimensions required for the lever rule based on the resultant of truck load

Distance between wheel load and barrier	= 2 ft	LRFD Art. 3.6.1.3.1
Distance between wheel and the resultant of truck load	= 3 ft	
Girder spacing, S	= 9 ft – 8.625 in.	
	= 9.71875 ft	
Distance between truck load resultant and first interior girder, L_1	= 5 ft – 9.75 in.	
	= 5.8125 ft	
Multiple presence factor for one design lane loaded m	= 1.2	LRFD Table 3.6.1.1.2-1

Distribution factor for one design lane loaded case using the lever rule based on the resultant of truck load

$= (L_1/S) \times m$

$= (5.8125/9.71875) \times 1.2 = 0.718$ lanes/girder

Step 5.2.1.2. Using AASHTO LRFD Eq. C4.6.2.2.2d-1

In steel beam-slab bridge cross-sections with diaphragms or cross-frames, the distribution factor for the exterior beam shall not be taken to be less than that which would be obtained by assuming that the cross-section deflects and rotates as a rigid cross-section. (LRFD Art. 4.6.2.2.2d)

$$R = \frac{N_L}{N_b} + \frac{X_{ext} \sum_0^{N_L} e}{\sum_0^{N_b} x^2}$$

LRFD Eq. C4.6.2.2.2d-1

where:

- e = eccentricity of a design truck or a design lane load from the center of gravity of the pattern of girders
- N_b = number of girders
- N_L = number of loaded lanes under consideration
- R = reaction on exterior girder in terms of lanes
- x = horizontal distance from the center of gravity of the pattern of girders to each girder
- X_{ext} = horizontal distance from the center of gravity of the pattern of girders to the exterior girder

This additional investigation is required because the distribution factor for girders in a multigirder cross-section, Types "a," "e," and "k" in Table 4.6.2.2.1-1, was determined without consideration of diaphragm or cross-frames. (LRFD Art. C4.6.2.2.2d)

In this example, the maximum number of trucks that may be placed on half of the bridge cross-section is three. Thus, this procedure is used to calculate distribution factors for three cases: 1-lane loaded, 2-lanes loaded, and 3-lanes loaded. Use of more than 3 loaded lanes results in smaller distribution factors, and hence is not considered in this example.

For one design lane loaded:

Figure 19 shows the lane and truck positions.

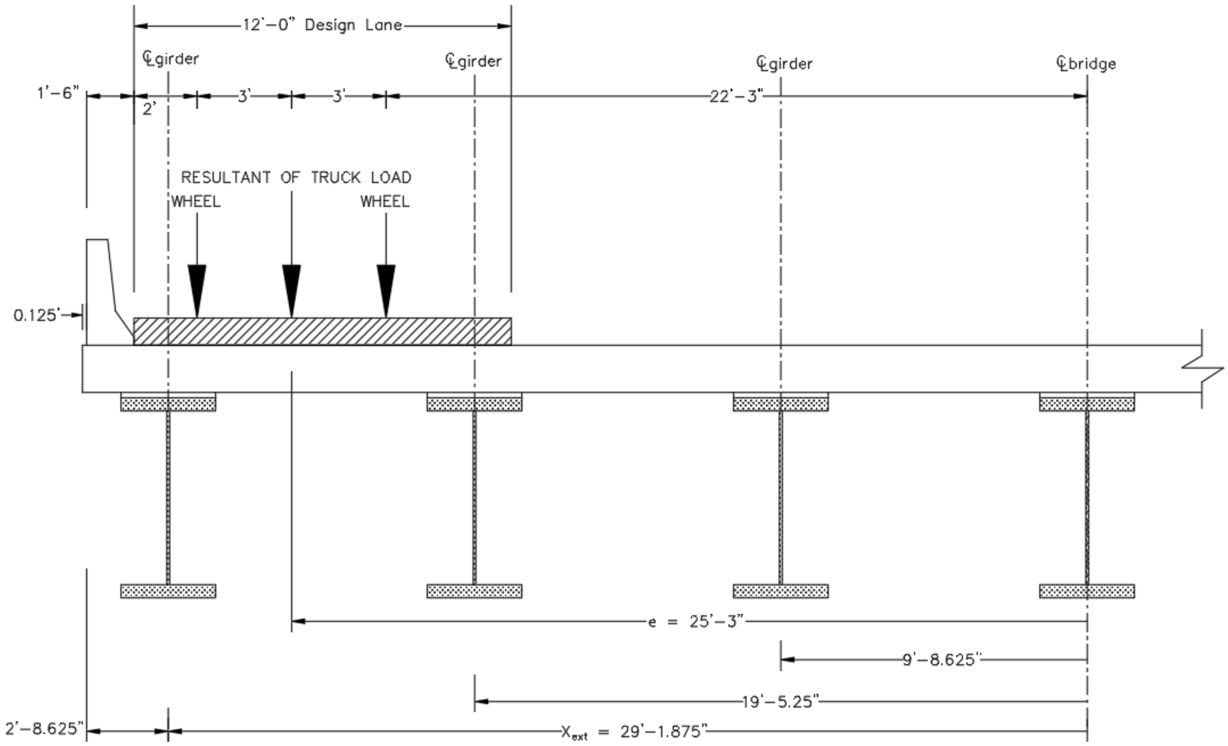


Figure 19. The lane and truck locations for a case with one design lane loaded

Number of loaded lanes, N_L	= 1
Distance from the center of gravity of the pattern of girders to the exterior girder, X_{ext}	= 29 ft – 1.875 in.
	= 29.15625 ft
Distance from the center of gravity of the pattern of girders to the interior edge of barrier = $w/2$ = 60.5/2	= 30.25 ft
where w = clear roadway width	
Eccentricity of the design truck from the center of gravity of the girders, e = $w/2$ - 5	= 25.25 ft
Number of girders, N_b	= 7

Girder location	x(ft)	x^2 (ft^2)
Exterior girder (Left)	29.15625	850.087
1st Interior girder (Left)	19.43750	377.816
2nd Interior girder (Left)	9.71875	94.454
Interior girder @ the CL of bridge	0	0
2nd Interior girder (Right)	9.71875	94.454
1st Interior girder (Right)	19.43750	377.816
Exterior girder (Right)	29.15625	850.087
	Σx^2 =	2644.714

$$R = \frac{1}{7} + \frac{29.15625 \times 25.25}{2644.714}$$ = 0.421 lanes/girder

Multiple presence factor for one design lane loaded = 1.2

R adjusted = 1.2 × 0.421 = 0.505 lanes/girder

For two design lanes loaded:

Figure 20 shows the lane and truck positions.

Number of loaded lanes, N_L = 2

Distance from the center of gravity of the pattern of girders to the exterior girder, X_{ext} = 29.15625 ft

Distance from the center of gravity of the pattern of girders to the interior edge of barrier = *w*/2 = 60.5/2 = 30.25 ft

where *w* = clear roadway width.

Eccentricity of the design truck from the center of gravity of the girders:

Truck 1: $e_1 = w/2 - 2 - 3$ = 25.25 ft

Truck 2: $e_2 = w/2 - 2 - 3 - 12$ = 13.25 ft

Note: The distance between the resultants of the two trucks is 12 ft. x remains similar to the one design lane loaded case.

Number of girders, N_b = 7

$$R = \frac{2}{7} + \frac{29.15625 \times (25.25 + 13.25)}{2644.714}$$ = 0.710 lanes/girder

Multiple presence factor for two design lanes loaded = 1.0

R adjusted = 1.0 × 0.710 = 0.710 lanes/girder

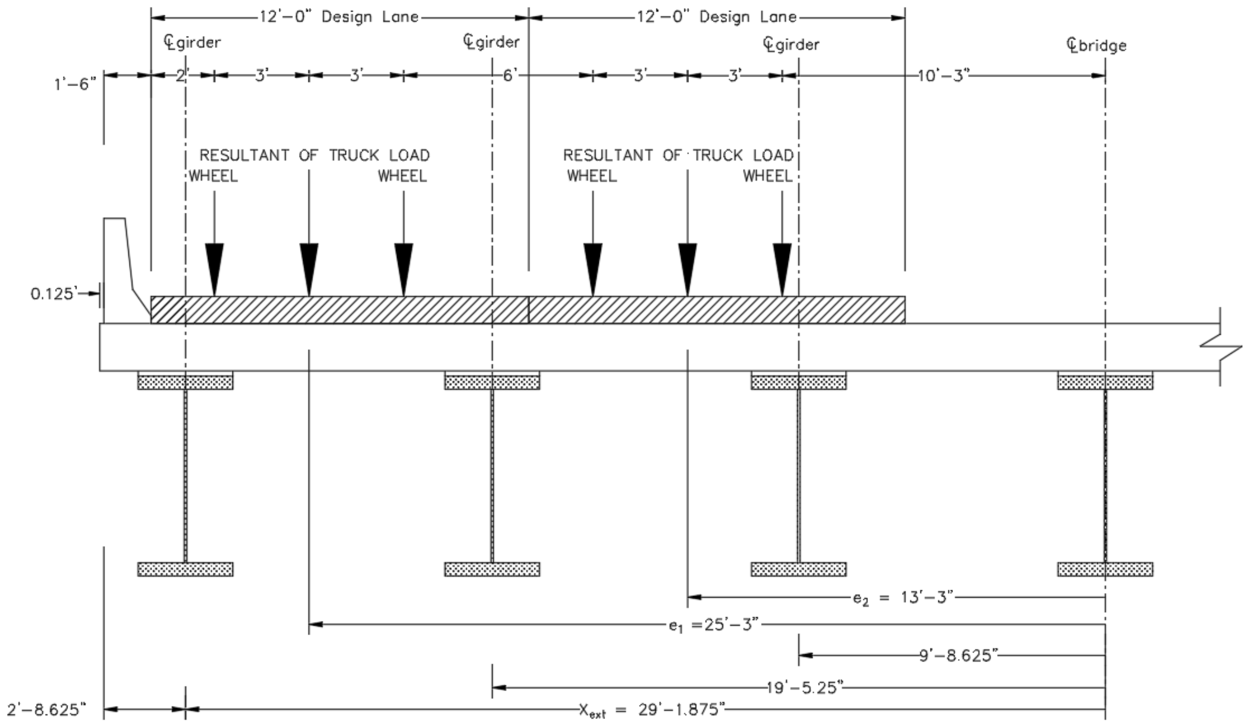


Figure 20. The lane and truck location for a case with two design lanes loaded

<u>For three design lanes loaded:</u>

Figure 21 shows the lane and truck positions.

Number of loaded lanes, N_L = 3

Distance from the center of gravity of the pattern of girders to the exterior girder, X_{ext} = 29.15625 ft

Distance from the center of gravity of the pattern of girders to the interior edge of barrier = $w/2$ = 60.5/2 = 30.25 ft

where w = clear roadway width

Eccentricity of the design truck from the center of gravity of the girders:

Truck 1: $e_1 = w/2 - 2 - 3$ = 25.25 ft

Truck 2: $e_2 = w/2 - 2 - 3 - 12$ = 13.25 ft

Truck 3: $e_2 = w/2 - 2 - 3 - 12 - 12$ = 1.25 ft

Note: The distance between the resultants of the two trucks is 12 ft. x remains similar to the one design lane loaded case.

Number of girders, N_b = 7

$$R = \frac{3}{7} + \frac{29.15625 \times (25.25 + 13.25 + 1.25)}{2644.714}$$ = 0.867 lanes/girder

Multiple presence factor for three design lanes loaded = 0.85

R adjusted = 0.85 × 0.867 = 0.737 lanes/girder

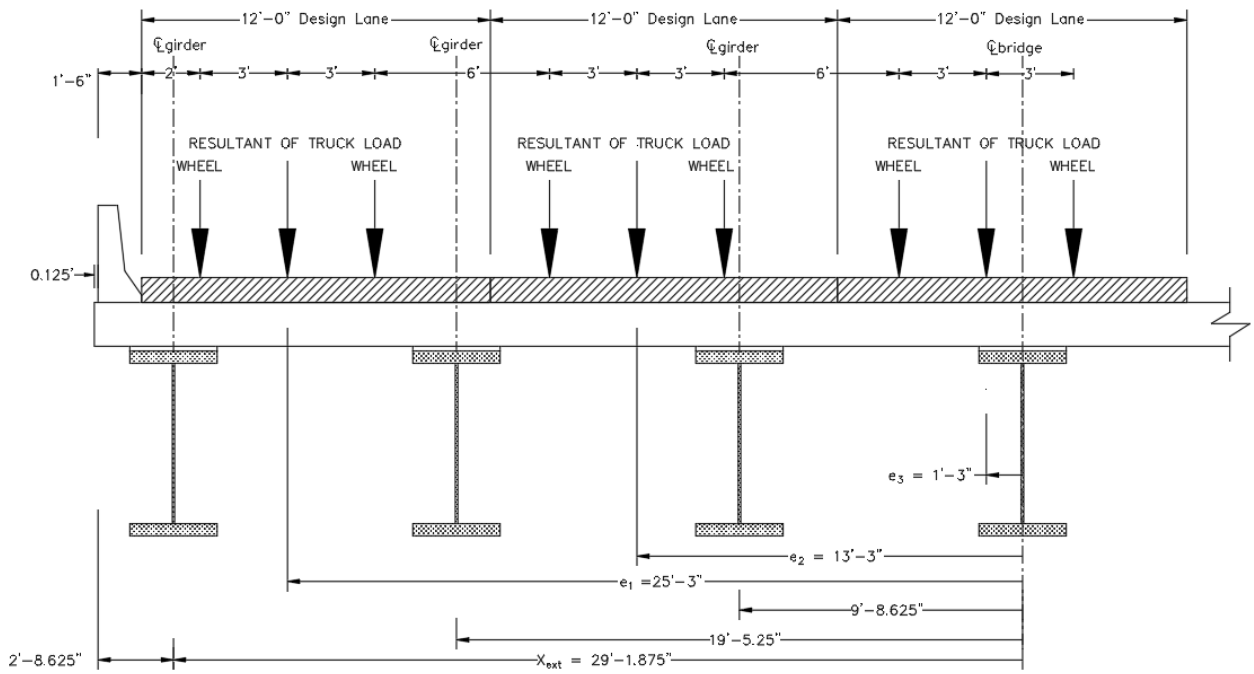


Figure 21. The lane and truck location for a case with three design lanes loaded

Step 5.2.1.3. DFM summary

The highest DFM at each section is selected as the DFM for design (see **Table 9**).

Table 9. Distribution Factor for Moment (DFM) of an Exterior Girder

Girder segment length (ft)	$g_{interior}$	DFM (lanes/girder)					
		2 or more design lanes loaded			One design lane loaded		Selected for design
		$eg_{interior}$	Eq. C4.6.2.2.2d-1		Lever rule	Eq. C4.6.2.2.2d-1	
			2-lanes	3-lanes			
19.08 (from bearing CL)	0.651	0.579	0.710	0.737	0.718	0.505	0.737
52.20	0.674	0.600	0.710	0.737	0.718	0.505	0.737
16.14	0.702	0.625	0.710	0.737	0.718	0.505	0.737
11.25 (to pier bearing CL)	0.707	0.629	0.710	0.737	0.718	0.505	0.737

A case with three design lanes loaded controls at every cross-section. A DFM of **0.737 lanes/girder** is used for further calculations.

Step 5.2.2. Distribution factors for shear

For two or more design lanes loaded:

$$g = e\, g_{interior}$$

LRFD Table 4.6.2.2.3b-1

$$e = 0.6 + \frac{d_e}{10}$$

$-1 \leq d_e \leq 5.5$

where:

d_e = horizontal distance from the exterior girder web centerline to the interior edge of curb or traffic barrier

= (32.625 - 18 - 1.5)/12 = 1.094 ft

$g_{interior}$ = distribution factor for shear in an interior girder

= 0.933 lanes/girder **Step 5.1.2**

$e = 0.6 + \frac{1.094}{10}$ = 0.709

g = (0.709) (0.933) = 0.661 lanes/girder

From **Step 5.2.1.1.**, distribution factor for one design lane loaded calculated using the lever rule = 0.718 lanes/girder (LRFD Art. 4.6.2.2.3b)

From **Step 5.2.1.2** and **5.2.1.3**, distribution factor for two or more design lanes loaded calculated using AASHTO LRFD Eq. C4.6.2.2.2d-1. (LRFD Art. 4.6.2.2.3b)

A three design lanes loaded case governs: A DFV of **0.737 lanes/girder** is used for calculations.

Step 5.3. Distribution Factors for Fatigue Limit State

The fatigue distribution factor is based on a one lane loaded case, and does not include the multiple presence factor, since the fatigue loading is specified as a single truck load. Because the distribution factors calculated from empirical equations incorporate the multiple presence factors, the fatigue distribution factors are equal to the strength distribution factors divided by the multiple presence factor for one lane. (LRFD Art. 3.6.1.1.2)

For an exterior girder:

Distribution factor for moment for one design lane loaded = 0.718

Distribution factor for shear for one design lane loaded = 0.718

For fatigue limit state:

Distribution factor for moment = 0.718/1.2 = 0.598

Distribution factor for shear = 0.718/1.2 = 0.598

For an interior girder:

Distribution factor for moment for one design lane loaded = 0.461

Distribution factor for shear for one design lane loaded = 0.749

For fatigue limit state:

Distribution factor for moment = 0.461/1.2 = 0.384

Distribution factor for shear = 0.749/1.2 = 0.624

Step 5.4. Distribution Factor for Live Load Deflection

All design lanes must be loaded when determining the live load deflection of the structure. In the absence of a refined analysis, an approximation of the live load deflection can be obtained by assuming that all girders deflect equally and applying the appropriate multiple presence factor. LRFD Art. 2.5.2.6.2

For a straight multibeam bridge, the distribution factor for deflection is equal to the number of lanes divided by the number of beams. LRFD Art. C2.5.2.6.2

The distribution factor for live load deflection:

$$DFL = m\left(\frac{N_L}{N_B}\right) = 0.65\left(\frac{5}{7}\right) = 0.464 \text{ lanes/girder}$$

Step 5.5. Skew Reduction/Correction Factors

Skew reduction/correction factor calculation is not needed for a bridge of zero skew.

Calculations shown in this step are for illustration purposes only.

Step 5.5.1. Reduction factor for live load moment

$$R_M = 1 - c_1(\tan\theta)^{1.5}$$

LRFD Table 4.6.2.2.2e-1

where:

R_M = reduction factor for live load moment

$$c_1 = 0.25\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5}$$

If $\theta < 30°$ then $c_1 = 0.0$

If $\theta > 60°$ use $\theta = 60°$

Range of Applicability LRFD Table 4.6.2.2.2e-1	MDOT Policy BDM Art. 7.01.14 and 7.02.02	Bridge Attributes
$30° \leq \theta \leq 60°$	$\theta \leq 30°$ (Approximate method) $30° < \theta \leq 45°$ (Refined method) $\theta > 45°$ (Need approval)	$\theta = 0°$
3.5 ft ≤ S ≤ 16 ft	S ≤ 10 ft for I beams	S = 9 ft – 8.625 in.
20 ft ≤ L ≤ 240 ft		L_{ds} = 98.67 ft
$N_b \geq 4$		N_b = 7

Since $\theta < 30°$, $c_1 = 0.0$ and $R_M = 1$

Step 5.5.2. Correction factor for support shear at obtuse corner

$$C_V = 1.0 + 0.20\left(\frac{12.0Lt_s^3}{K_g}\right)^{0.3}\tan\theta$$

LRFD Table 4.6.2.2.3c-1

where:

C_V = correction factor for live load shear

Use of this correction factor is subject to the same criteria shown above; except, θ range is between 0º and 60º. Since θ = 0°, C_V = 1.

Step 5.6. Moment and Shear Distribution Factors Adjusted for Skew

Distribution factor for moment in interior girder,

DFM_{IB} = (DFM_I) (R_M) = 0.668 lanes/girder

Distribution factor for moment in exterior girder,

DFM_{EB} = (DFM_E) (R_M) = 0.737 lanes/girder

Distribution factor for shear in interior girder,

DFV_{IB} = (DFV_I) = 0.933 lanes/girder

Distribution factor for shear in exterior girder,

DFV_{EB} = (DFV_E) (C_V) = 0.737 lanes/girder

Distribution factor for fatigue moment in interior girder,

DFM_{IB} = (DFM_I) (R_M) = 0.384 lanes/girder

Distribution factor for fatigue moment in exterior girder,

DFM_{EB} = (DFM_E) (R_M) = 0.598 lanes/girder

Distribution factor for fatigue shear in interior girder,

DFV_{IB} = (DFV_I) (C_V) = 0.624 lanes/girder

Distribution factor for fatigue shear in exterior girder,

DFV_{EB} = (DFV_E) (C_V) = 0.598 lanes/girder

STEP 6. MOMENT AND SHEAR FORCES DUE TO DEAD LOADS, SUPPORT SETTLEMENT, AND CONSTRUCTION LOADS (DECK POUR)

Step 6.1. Moment and Shear Forces in a Noncomposite Section

Noncomposite girder moment and shear are calcualted under steel girder self-weight, deck self-weight, haunch self-weight, and stay-in-place (SIP) formwork weight. Girder cross-section changes along the length. The self-weight of each segment is presented below. Instead of using discrete values, a weighted-average of 0.230 kip/ft and 0.204 kip/ft for exterior and interior girders can be used.

Load type	**Span**	**Exterior girder**		**Interior girder**	
		Load (kip/ft)	**Span range (ft)**	**Load (kip/ft)**	**Span range (ft)**
Girder self-weight	Span 1	0.1633 0.2016 0.3011 0.3777	0 - 19.08 ft 19.08 - 71.28 ft 71.28 - 87.42 ft 87.42 - 98.67 ft	0.1557 0.1863 0.2093 0.2858 0.3624	0 - 17.50 ft 17.50 - 71.25 ft 71.25 - 86.49 ft 86.49 - 88.46 ft 88.46 - 98.67 ft
	Span 2	Identical with span 1		Identical with span 1	
Note: Span range is defined from abutment bearing centerline to pier bearing centerline.					

Deck self-weight, haunch self-weight, and stay-in-place (SIP) formwork weight are:

Load type	**Span**	**Exterior girder load (kip/ft)**	**Interior girder load (kip/ft)**
Deck self-weight	Span 1/Span 2	0.85300	1.09300
Haunch self-weight	Span 1/Span 2	0.01875	0.01875
SIP formwork weight	Span 1/Span 2	0.09100	0.12300

In this example, SAP 2000 is used to calculate the moments and shear forces of interior and exterior girders. Girder sections were modeled in the software as per the plan details. The design span of the bridge is 197.34 ft. The design span of a single span is 98.67 ft.

Table 10 to **Table 13** show exterior and interior girder moments and shear forces.

Figure 22 to **Figure 25** show the moment and shear force diagrams.

Step 6.2. Moment and Shear Forces in Composite Section due to Barrier and FWS Weights

Barrier and FWS loads are applied on the long-term composite section defined with 3n = 24, except at regions without shear studs. In the region without shear studs, barrier and FWS loads are applied on noncomposite sections. The region without shear studs for an interior girder and an exterior girder are 69.2 ft – 128.07 ft and 69.3 ft – 128.07 ft, respectively. **Step 4.1.5** describes the barrier and FWS load calculation.

Load type	**Span**	**Exterior/Interior girder load (kip/ft)**
Barrier weight	Span 1/Span 2	0.136
FWS weight	Span 1/Span 2	0.216

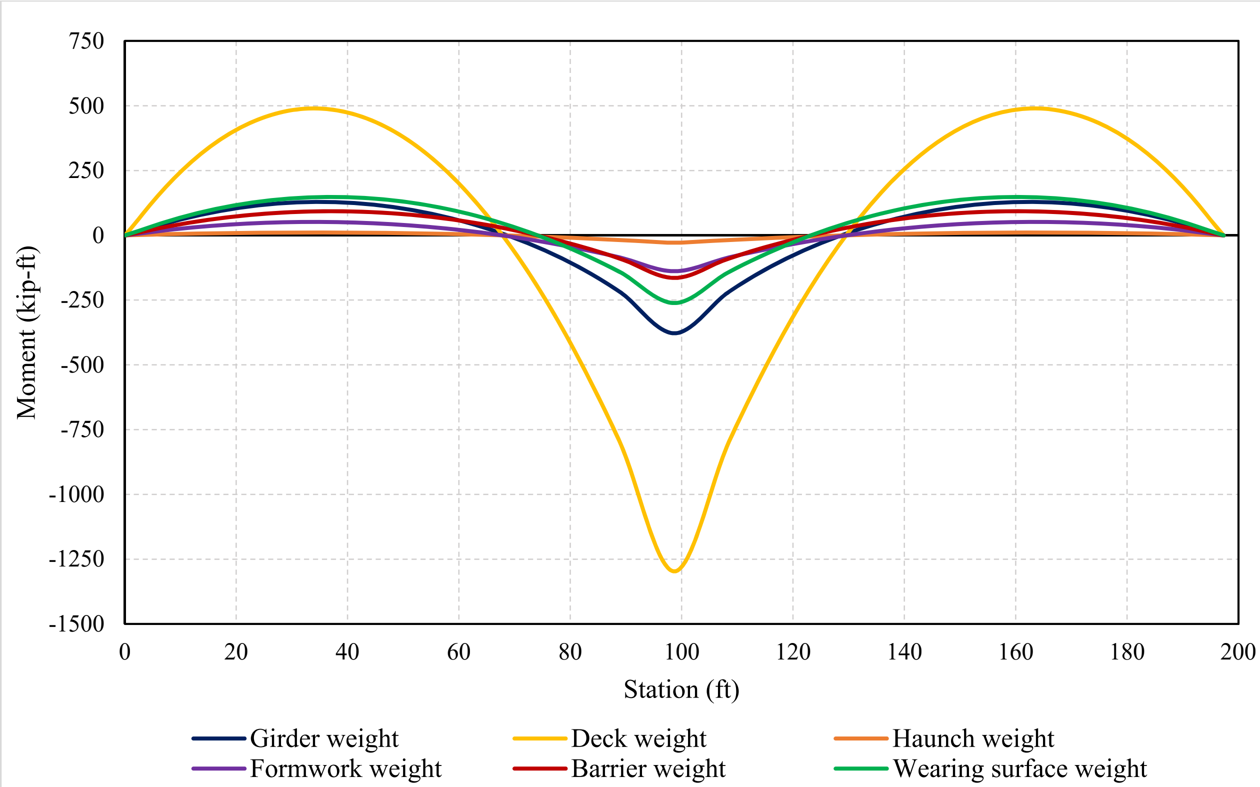


Figure 22. Unfactored dead load moment diagram for an exterior girder

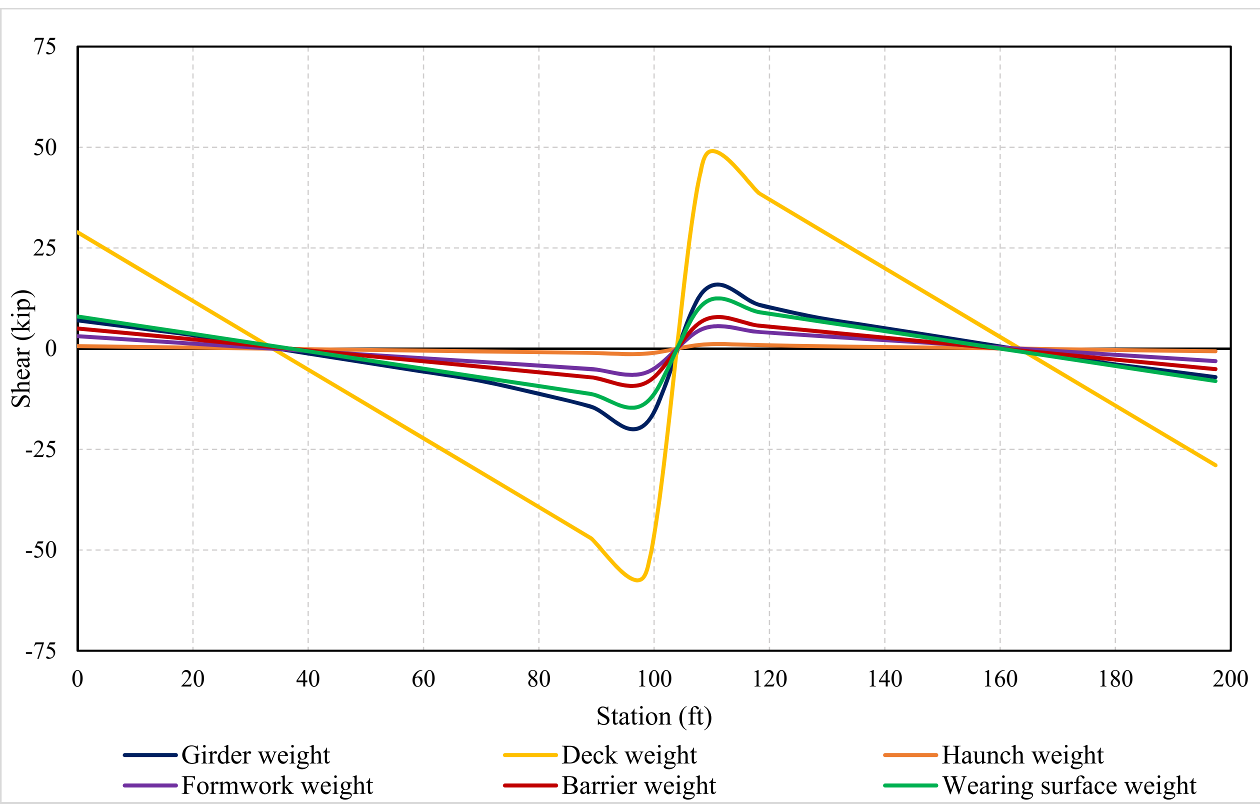


Figure 23. Unfactored dead load shear force diagram for an exterior girder

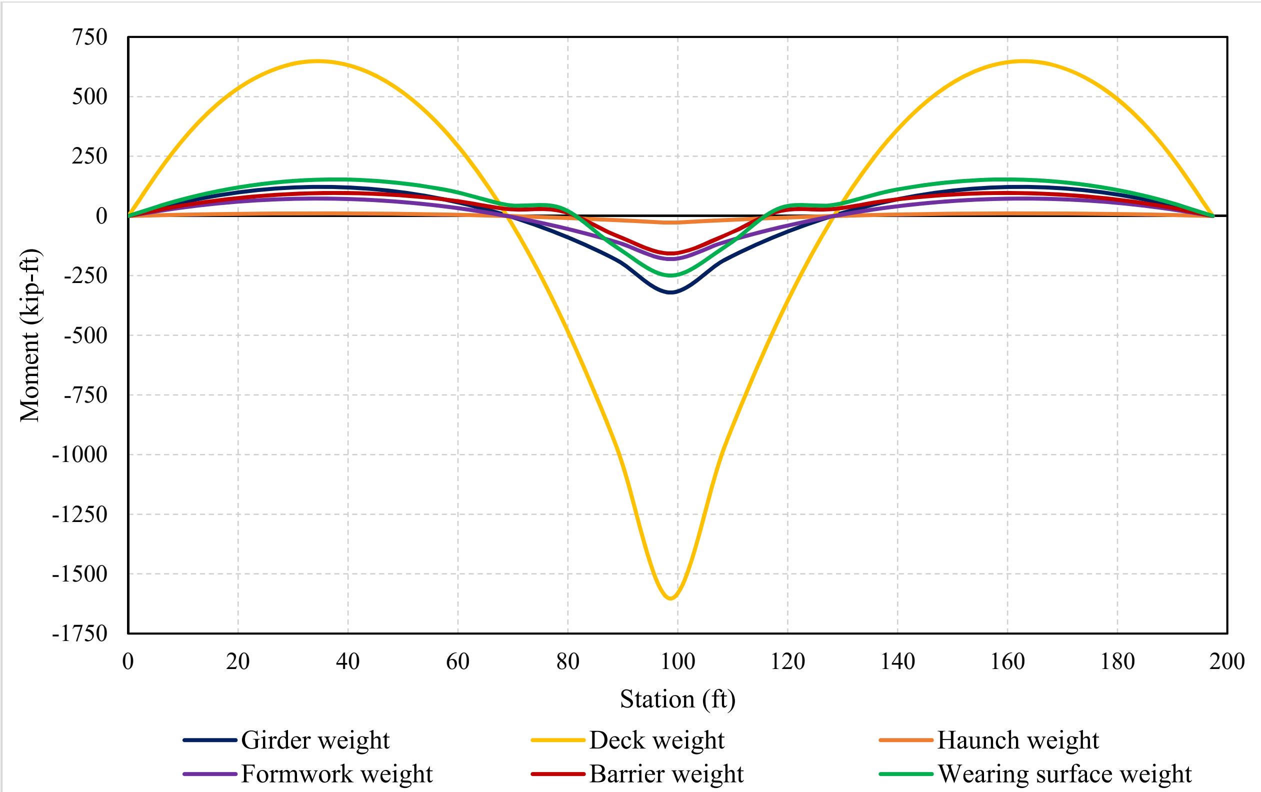


Figure 24. Unfactored dead load moment diagram for an interior girder

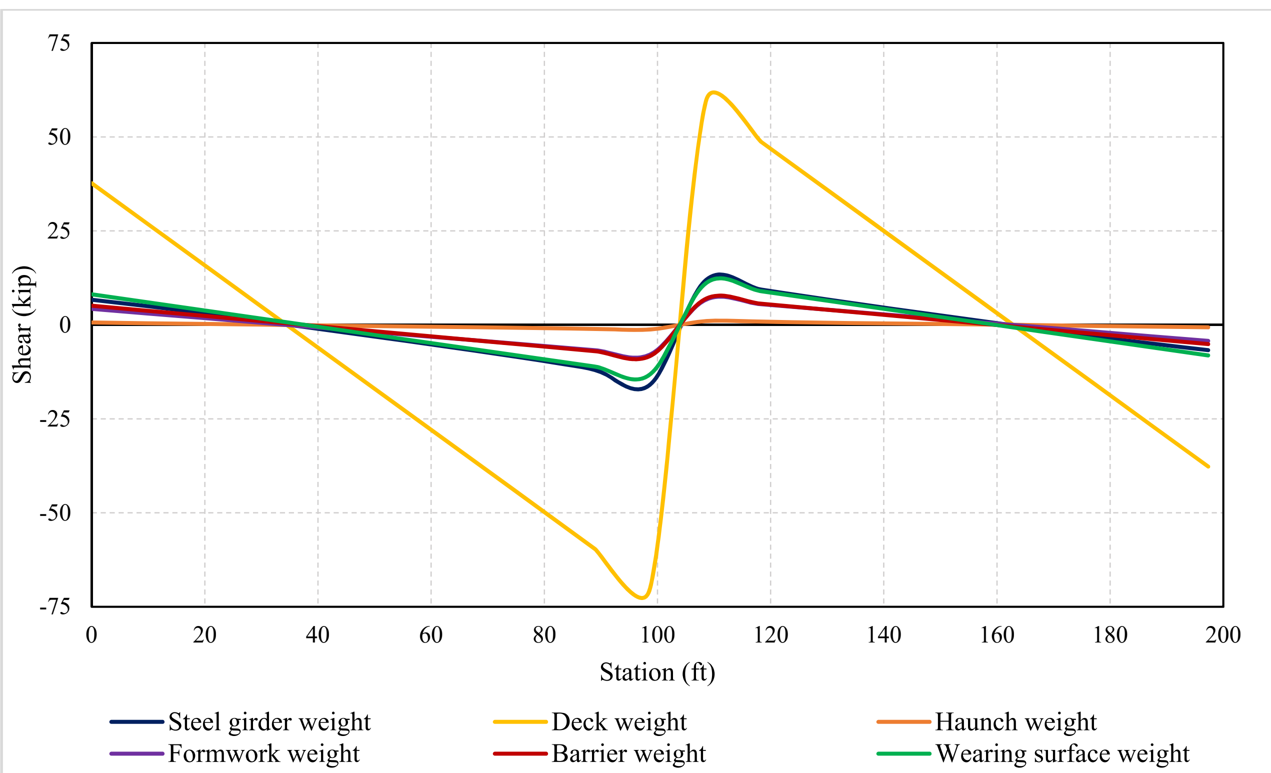
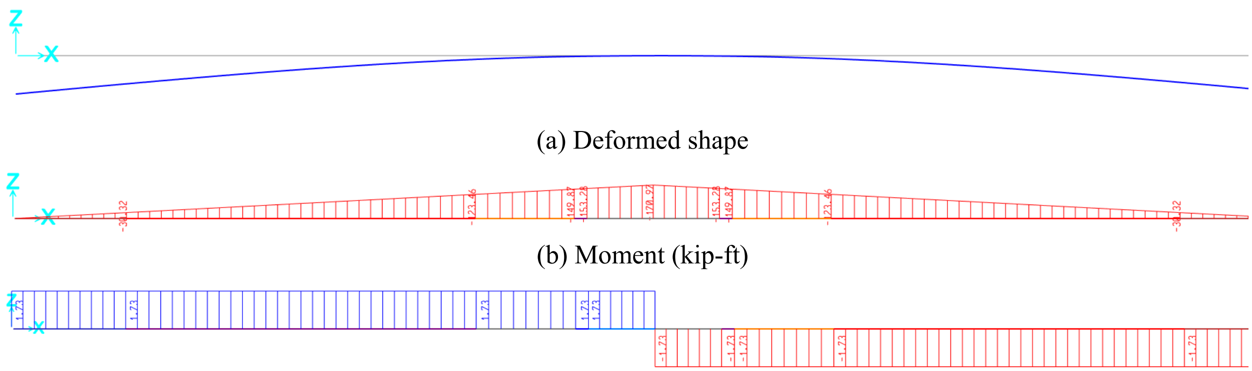


Figure 25. Unfactored dead load shear force diagram for an interior girder

Step 6.3. Moment and Shear Forces in a Composite Section due to Support Settlement

A negative moment caused by a 1 in. settlement of a substructure unit is imposed when non-spread footings are used. A settlement of 2 in. is imposed when the bridge is on spread footings.

Moments and shear forces developed in interior and exterior girders are calculated by imposing a 1 in. settlement simultaneously at both abutments, as shown in **Figure 26a**. The long-term composite section defined with 3n = 24 is used, except at regions without shear studs. In the region without shear studs, noncomposite section properties are used. The resulting moments and shear forces are presented in **Table 10** to **Table 13**. The resulting moments and shear forces of an interior girder are shown in **Figure 26b** and **Figure 26c**, respectively.



(a) Deformed shape

(b) Moment (kip-ft)

(c) Shear (kip)

Figure 26. Response of an interior girder due to a 1 in. settlement at both abutments

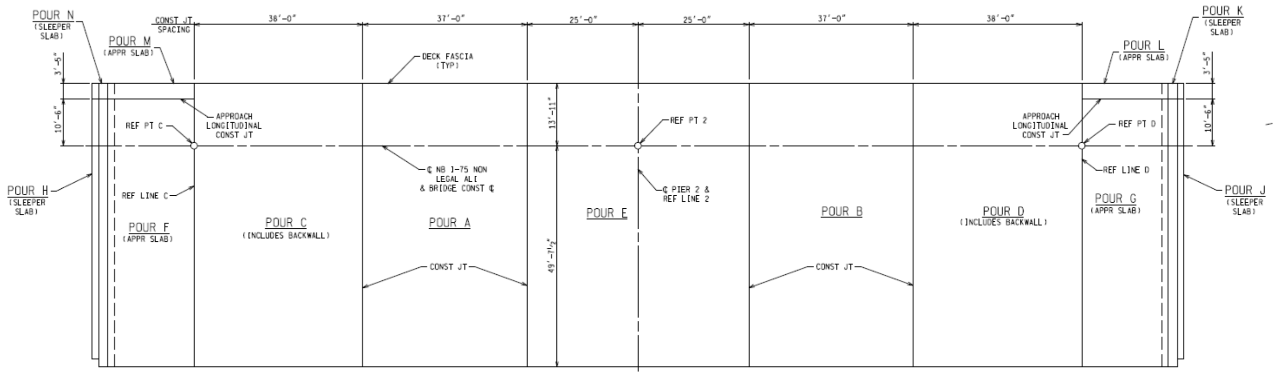
Step 6.4. Moment and Shear Forces due to Construction Loads (Deck Pour)

The final decision on the deck pour sequence is made after evaluating the resources available to the contractor. The typical approach is to pour the positive moment region first and the negative moment region the last, to avoid cracking within the negative moment region.

For highly skewed bridges, a deck over the abutments is poured first to induce additional dead loads on the bearings to avoid possible uplift.

Generally, the pour sequence is provided in the plans. If the Contractor proposes a different pour sequence, the proposed pour sequence is evaluated and approved or denied by MDOT.

Figure 27 shows the deck pour sequence.



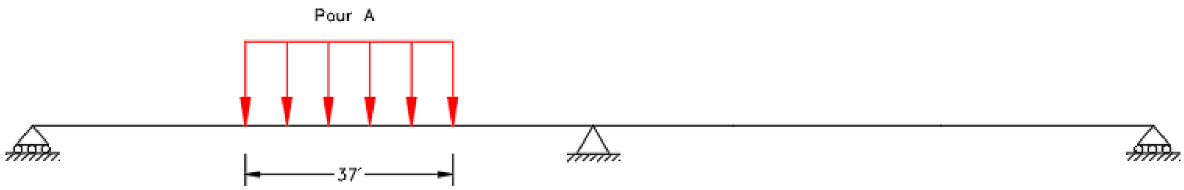
DECK POURS ARE TO BE MADE IN THE FOLLOWING SEQUENCE A AND B SHALL BE POURED THE SAME DAY, THEN POURS C AND D AND LASTLY POUR E. ALL OTHER POURS CAN BE PLACED AFTER POUR E. WHENEVER A DECK POUR IS MADE, AT LEAST 15 HOURS SHALL HAVE ELAPSED SINCE THE ADJACENT SECTION WAS PLACED. THIS INCLUDES SECTIONS SEPARATED BY LONGITUDINAL AS WELL AS TRANSVERSE JOINTS.

Figure 27. Deck pour sequence

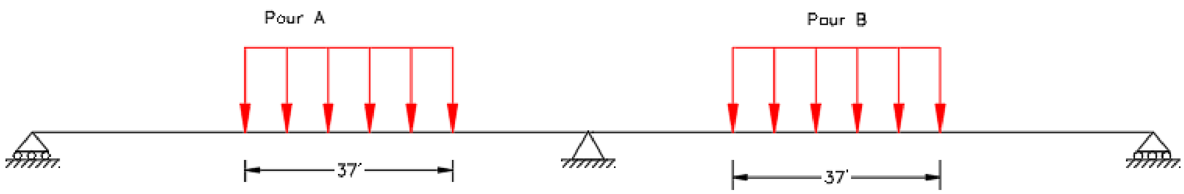
Figure 28 shows the loading sequence. In this example, the following deck pour sequence is considered.

1. A and B are poured on the same day. Both segments are noncomposite.
2. C, D, and E are poured on the same day after 15 hours from pour B. The pour sequence is in the order of C, D, and E. Segments A and B remain composite (n = 8).

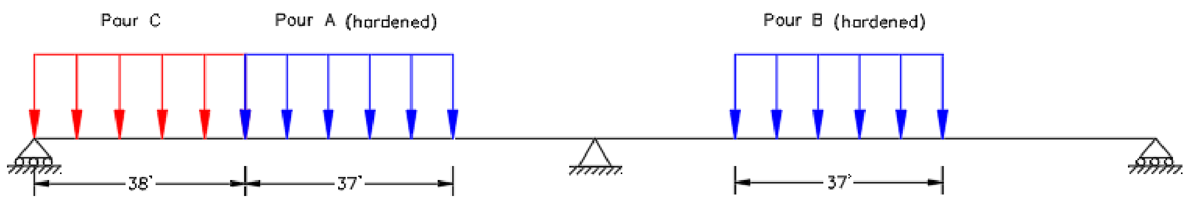
The resulting moments and shear forces are presented in **Table 10** to **Table 13**.



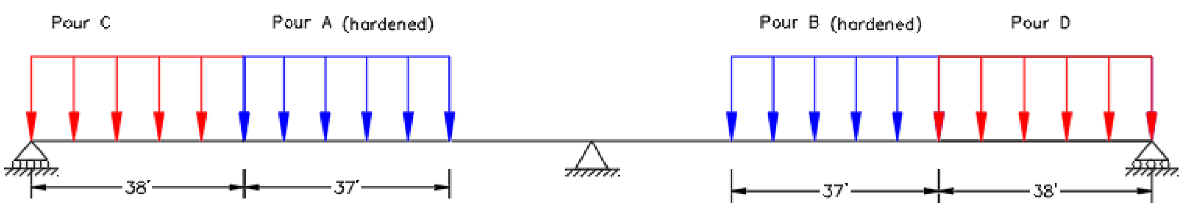
(a) Concrete placement of pour A



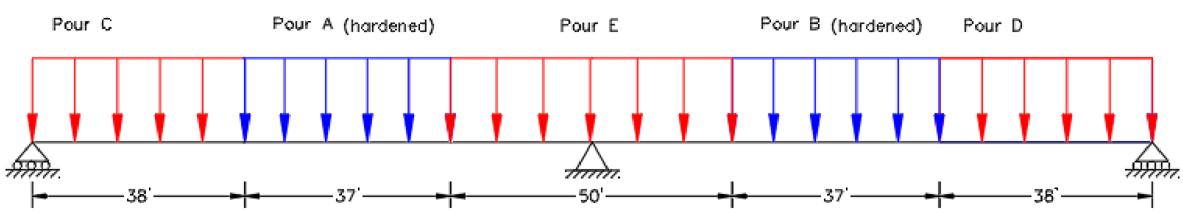
(b) Concrete placement of pour B



(c) Concrete placement of pour C



(d) Concrete placement of pour D



(e) Concrete placement of pour E

Figure 28. Deck dead load application based on the pour sequence

Table 10. Unfactored Exterior Girder Moment Under Dead Loads, Construction Loads, and Settlement (kip-ft)

Location ($\times L_{ds}$)	Girder weight, M_g	Deck weight, M_D	Haunch weight, M_H	Formwork weight, M_f	Barrier weight, M_b	FWS weight, M_{ws}	Construction loads (deck pour)					Settlement, M_{SE}
							A	A & B	A, B, & C	A, B, C, & D	A, B, C, D, & E	
0.0 (Abut A)	0	0	0	0	0	0	0	0	0	0	0	0
0.1	61	244	5	26	43	68	104	67	280	264	270	-13
0.2	103	404	9	43	73	116	207	134	475	443	456	-27
0.3	126	482	11	51	90	142	311	201	586	537	557	-40
0.4	127	477	10	51	93	148	411	264	612	547	573	-53
0.5	105	389	9	41	83	132	448	265	553	472	504	-67
0.6	62	218	5	23	60	96	400	181	409	312	350	-80
0.7	-3	-36	-1	-4	24	38	268	11	181	67	112	-93
0.8	-94	-374	-8	-40	-26	-41	63	-230	-121	-250	-212	-107
0.9	-217	-794	-17	-85	-88	-140	-152	-481	-431	-577	-620	-120
1.0 (Pier)	-378	-1297	-29	-138	-164	-261	-366	-732	-742	-904	-1112	-134
1.1	-217	-794	-17	-85	-88	-140	-330	-481	-525	-577	-620	-120
1.2	-94	-374	-8	-40	-26	-41	-293	-230	-238	-250	-212	-107
1.3	-3	-36	-1	-4	24	38	-256	11	5	67	112	-93
1.4	62	218	5	23	60	96	-220	181	175	312	350	-80
1.5	105	389	9	41	83	132	-183	265	260	472	504	-67
1.6	127	477	10	51	93	148	-146	264	260	547	573	-53
1.7	126	482	11	51	90	142	-110	201	198	537	557	-40
1.8	103	404	9	43	73	116	-73	134	132	443	456	-27
1.9	61	244	5	26	43	68	-37	67	66	264	270	-13
2.0 (Abut B)	0	0	0	0	0	0	0	0	0	0	0	0

Table 11. Unfactored Interior Girder Moment Under Dead Loads, Construction Loads, and Settlement (kip-ft)

Location ($\times L_{ds}$)	Girder weight, M_g	Deck weight, M_D	Haunch weight, M_H	Formwork weight, M_f	Barrier weight, M_b	FWS weight, M_{ws}	Construction loads (deck pour)					Settlement, M_{SE}
							A	A & B	A, B, & C	A, B, C, & D	A, B, C, D, & E	
0.0 (Abut A)	0	0	0	0	0	0	0	0	0	0	0	0
0.1	58	318	5	36	44	70	134	89	362	343	350	-11
0.2	98	531	9	60	74	118	268	178	616	578	593	-22
0.3	119	636	11	72	92	146	402	267	761	704	727	-33
0.4	120	635	11	72	96	152	531	351	799	722	753	-44
0.5	101	528	9	59	87	138	580	356	728	632	671	-55
0.6	61	315	5	35	65	103	521	252	549	434	480	-67
0.7	1	-5	0	-1	29	46	354	40	262	128	182	-78
0.8	-80	-432	-7	-49	20	32	94	-265	-118	-271	-225	-89
0.9	-184	-965	-17	-109	-82	-130	-178	-582	-510	-682	-740	-100
1.0 (Pier)	-321	-1603	-27	-180	-157	-250	-450	-899	-901	-1092	-1363	-111
1.1	-184	-965	-17	-109	-82	-130	-405	-582	-584	-682	-740	-100
1.2	-80	-432	-7	-49	20	32	-360	-265	-267	-271	-225	-89
1.3	1	-5	0	-1	29	46	-315	40	38	128	182	-78
1.4	61	315	5	35	65	103	-270	252	250	434	480	-67
1.5	101	528	9	59	87	138	-225	356	354	632	671	-55
1.6	120	635	11	72	96	152	-180	351	350	722	753	-44
1.7	119	636	11	72	92	146	-135	267	266	704	727	-33
1.8	98	531	9	60	74	118	-90	178	178	578	593	-22
1.9	58	318	5	36	44	70	-45	89	89	343	350	-11
2.0 (Abut B)	0	0	0	0	0	0	0	0	0	0	0	0

Table 12. Unfactored Exterior Girder Shear Under Dead Loads, Construction Loads, and Settlement (kip)

Location (×L_{ds})	Girder weight, V_g	Deck weight, V_D	Haunch weight, V_H	Formwork weight, V_f	Barrier weight, V_b	FWS weight, V_{ws}	Construction loads (deck pour)					Settlement, V_{SE}
							A	A & B	A, B, & C	A, B, C, & D	A, B, C, D, & E	
0.0 (Abut A)	7.0	28.9	0.6	3.1	5.0	8.0	10.5	6.8	32.7	31.1	31.7	-1.35
0.1	5.2	20.5	0.5	2.2	3.7	5.9	10.5	6.8	24.1	22.5	23.1	-1.35
0.2	3.4	12.1	0.3	1.3	2.4	3.7	10.5	6.8	15.5	13.9	14.5	-1.35
0.3	1.2	3.7	0.1	0.4	1.0	1.6	10.5	6.8	6.9	5.3	5.9	-1.35
0.4	-1.0	-4.7	-0.1	-0.5	-0.3	-0.5	8.1	4.4	-1.7	-3.3	-2.7	-1.35
0.5	-3.3	-13.1	-0.3	-1.4	-1.7	-2.6	-0.5	-4.2	-10.3	-11.9	-11.3	-1.35
0.6	-5.5	-21.6	-0.5	-2.3	-3.0	-4.8	-9.1	-12.8	-18.9	-20.5	-19.9	-1.35
0.7	-7.7	-30.0	-0.7	-3.2	-4.3	-6.9	-17.7	-21.4	-27.5	-29.1	-28.5	-1.35
0.8	-10.8	-38.4	-0.8	-4.1	-5.7	-9.0	-21.7	-25.4	-31.5	-33.1	-37.1	-1.35
0.9	-14.3	-46.8	-1.0	-5.0	-7.0	-11.2	-21.7	-25.4	-31.5	-33.1	-45.7	-1.35
1.0 (Pier)	-18.4	-55.2	-1.2	-5.9	-8.4	-13.3	-21.7	-25.4	-31.5	-33.1	-54.3	-1.35
1.1	14.3	46.8	1.0	5.0	7.0	11.2	3.7	25.4	25.5	33.1	45.7	1.35
1.2	10.8	38.4	0.8	4.1	5.7	9.0	3.7	25.4	25.5	33.1	37.1	1.35
1.3	7.7	30.0	0.7	3.2	4.3	6.9	3.7	21.4	21.5	29.1	28.5	1.35
1.4	5.5	21.6	0.5	2.3	3.0	4.8	3.7	12.8	12.9	20.5	19.9	1.35
1.5	3.3	13.1	0.3	1.4	1.7	2.6	3.7	4.2	4.3	11.9	11.3	1.35
1.6	1.0	4.7	0.1	0.5	0.3	0.5	3.7	-4.4	-4.3	3.3	2.7	1.35
1.7	-1.2	-3.7	-0.1	-0.4	-1.0	-1.6	3.7	-6.8	-6.7	-5.3	-5.9	1.35
1.8	-3.4	-12.1	-0.3	-1.3	-2.4	-3.7	3.7	-6.8	-6.7	-13.9	-14.5	1.35
1.9	-5.2	-20.5	-0.5	-2.2	-3.7	-5.9	3.7	-6.8	-6.7	-22.5	-23.1	1.35
2.0 (Abut B)	-7.0	-28.9	-0.6	-3.1	-5.0	-8.0	3.7	-6.8	-6.7	-31.1	-31.7	1.35

Table 13. Unfactored Interior Girder Shear Under Dead Loads, Construction Loads, and Settlement (kip)

Location ($\times L_{ds}$)	Girder weight, V_g	Deck weight, V_D	Haunch weight, V_H	Formwork weight, V_f	Barrier weight, V_b	FWS weight, V_{ws}	Construction loads (deck pour)					Settlement, V_{SE}
							A	A & B	A, B, & C	A, B, C & D	A, B, C, D, & E	
0.0 (Abut A)	6.7	37.7	0.6	4.2	5.1	8.1	13.6	9.0	42.2	40.3	41.0	-1.13
0.1	5.0	26.9	0.5	3.0	3.8	6.0	13.6	9.0	31.2	29.3	30.1	-1.13
0.2	3.2	16.1	0.3	1.8	2.4	3.9	13.6	9.0	20.3	18.3	19.1	-1.13
0.3	1.1	5.3	0.1	0.6	1.1	1.7	13.6	9.0	9.3	7.3	8.1	-1.13
0.4	-0.9	-5.5	-0.1	-0.6	-0.3	-0.4	10.5	5.9	-1.7	-3.6	-2.9	-1.13
0.5	-3.0	-16.3	-0.3	-1.8	-1.6	-2.5	-0.5	-5.1	-12.7	-14.6	-13.8	-1.13
0.6	-5.1	-27.0	-0.5	-3.0	-2.9	-4.7	-11.5	-16.0	-23.6	-25.6	-24.8	-1.13
0.7	-7.1	-37.8	-0.6	-4.3	-4.3	-6.8	-22.5	-27.0	-34.6	-36.5	-35.8	-1.13
0.8	-9.4	-48.6	-0.8	-5.5	-5.6	-8.9	-27.6	-32.1	-39.7	-41.7	-46.7	-1.13
0.9	-11.9	-59.4	-1.0	-6.7	-7.0	-11.1	-27.6	-32.1	-39.7	-41.7	-57.7	-1.13
1.0 (Pier)	-15.9	-70.2	-1.2	-7.9	-8.3	-13.2	-27.6	-32.1	-39.7	-41.7	-68.7	-1.13
1.1	11.9	59.4	1.0	6.7	7.0	11.1	4.6	32.1	32.1	41.7	57.7	1.13
1.2	9.4	48.6	0.8	5.5	5.6	8.9	4.6	32.1	32.1	41.7	46.7	1.13
1.3	7.1	37.8	0.6	4.3	4.3	6.8	4.6	27.0	27.0	36.5	35.8	1.13
1.4	5.1	27.0	0.5	3.0	2.9	4.7	4.6	16.0	16.1	25.6	24.8	1.13
1.5	3.0	16.3	0.3	1.8	1.6	2.5	4.6	5.1	5.1	14.6	13.8	1.13
1.6	0.9	5.5	0.1	0.6	0.3	0.4	4.6	-5.9	-5.9	3.6	2.9	1.13
1.7	-1.1	-5.3	-0.1	-0.6	-1.1	-1.7	4.6	-9.0	-9.0	-7.3	-8.1	1.13
1.8	-3.2	-16.1	-0.3	-1.8	-2.4	-3.9	4.6	-9.0	-9.0	-18.3	-19.1	1.13
1.9	-5.0	-26.9	-0.5	-3.0	-3.8	-6.0	4.6	-9.0	-9.0	-29.3	-30.1	1.13
2.0 (Abut B)	-6.7	-37.7	-0.6	-4.2	-5.1	-8.1	4.6	-9.0	-9.0	-40.3	-41.0	1.13

STEP 7. MOMENT AND SHEAR FORCES DUE TO LIVE LOADS

As discussed in **Step 4.2**, MDOT uses a modified version of the HL-93 loading. A single truck load, a single 60-kip load (axle load), a two design-truck load for continuous spans, and a lane load are multiplied by a factor of 1.2 to make the HL-93 Mod. BDM Art. 7.01.04-A

As per the specifications, only 90% of continuous span loads is considered for generating the maximum negative moment over the pier and pier reactions.

In this example, the load response due to each of these loads is calculated, and the response is multiplied by the

Factor for HL-93 Mod = 1.2 BDM Art. 7.01.04-A

Truck and axle load responses are also multiplied by the following factors to incorporate

Dynamic load allowance, IM = 0.33 LRFD Table 3.6.2.1-1

Reduction factor for continuous span loading = 0.90 BDM Art. 7.01.04-A

The typical procedure is to use a single line girder model to calculate moment and shear forces along the span under each load, and use the relationships given in **Step 7.1** and **Step 7.2** to calculate the moments and shear forces acting on interior and exterior girders.

Step 7.1. Moment and Shear Forces due to Single Design Truck or Axle Loads

For all limit states except fatigue:

A single line girder model is used to calculate moment and shear force under a single truck load and a single 60-kip load. The following relationships are used to calculate the moment per girder due to truck (M_{LT}) or axle load (M_{LA}), and shear force per girder due to truck (V_{LT}) or axle load (V_{LA}).

For an interior girder,

M_{LT} or M_{LA} = (moment per lane)(factor for HL-93 Mod)(DFM_{IB})(1 + IM)

= (moment per lane)(1.2)(DFM_{IB})(1.33)

= (moment per lane)(1.596)(DFM_{IB})

V_{LT} or V_{LA} = (shear force per lane)(factor for HL-93 Mod)(DFV_{IB})(1 + IM)

= (shear force per lane)(1.596)(DFV_{IB})

For an exterior girder,

M_{LT} or M_{LA} = (moment per lane)(factor for HL-93 Mod)(DFM_{EB})(1 + IM)
= (moment per lane)(1.596)(DFM_{EB})

V_{LT} or V_{LA} = (shear force per lane)(factor for HL-93 Mod)(DFV_{EB})(1 + IM)
= (shear force per lane)(1.596)(DFV_{EB})

Moment per lane and shear force per lane are presented in **Appendix A**.

The following labels are used to represent different truck load configurations used in this example:

- Single truck load: HS20-44
- Two design-truck load for continuous spans: 2-HS20-28.

M_{LT}, M_{LA}, V_{LT}, and V_{LA} values are presented in **Table 14** to **Table 17**. The following example demonstrates the calculation process.

For all other limit states except the fatigue limit state, DFM_{IB} = 0.668 **Step 5.6**

Interior girder positive moment per lane due to HS20-44 truck load at $0.5L_{ds}$ = 1232 kip-ft **Table A-3**

M_{LT} = (1232) (1.596) (0.668) = 1313 kip-ft

Interior girder positive moment per lane due to single 60-kip load at $0.5L_{ds}$ = 1239 kip-ft **Table A-3**

M_{LA} = (1239) (1.596) (0.668) = 1321 kip-ft

For all other limit states except the fatigue limit state, DFV_{IB} = 0.933 **Step 5.6**

Interior girder positive shear force per lane due to HS20-44 truck load at $0.0L_{ds}$ = 63.9 kips **Table A-4**

V_{LT} = (63.9) (1.596) (0.933) = 95.2 kips

Interior girder positive shear force per lane due to single 60-kip load at $0.0L_{ds}$ = 60 kips **Table A-4**

V_{LA} = (60) (1.596) (0.933) = 89.3 kips

Table 14 column a and b present M_{LT} and M_{LA} for an exterior girder.
Table 15 column a and b present M_{LT} and M_{LA} for an interior girder.
Table 16 column a and b present V_{LT} and V_{LA} for an exterior girder.
Table 17 column a and b present V_{LT} and V_{LA} for an interior girder.

Step 7.2. Moment and Shear Forces Due to Design Lane Load

For all limit states except fatigue:

A single line girder model is used to calculate moment and shear due to 0.64 kip/ft lane load. The lane load needs to be applied on a single span, both spans, and a portion of the span to generate the maximum moment and shear force response. This is best accomplished using software with inbuilt load configurations or an influence line diagram.

In this example, SAP 2000 is used to calculate the moment and shear force per lane under a lane load of 0.64 kip/ft. The moment per girder (M_{LL}) and shear force per girder (V_{LL}) under lane load are calculated using the following relationships.

For an interior girder,

M_{LL} = (moment per lane)(factor for HL-93 Mod)(DFM_{IB})
= (moment per lane)(1.2)(DFM_{IB})

V_{LL} = (shear force per lane)(factor for HL-93 Mod)(DFV_{IB})
= (shear force per lane)(1.2)(DFV_{IB})

For an exterior girder,

M_{LL} = (moment per lane)(factor for HL-93 Mod)(DFM_{EB})
= (moment per lane)(1.2)(DFM_{EB})

V_{LL} = (shear force per lane)(factor for HL-93 Mod)(DFV_{EB})
= (shear force per lane)(1.2)(DFV_{EB})

Moment per lane and shear force per lane are presented in **Appendix A**.

M_{LL} and V_{LL} valaues are presented in **Table 14** to **Table 17**. The following example demonstrates the calculation process.

Interior girder positive moment per lane due to the lane load of 0.64 kip/ft at $0.5L_{ds}$ = 603 kip-ft **Table A-3**

M_{LL} = (603) (1.2) (0.668) = 483 kip-ft

Interior girder positive shear force per lane due to the lane load of 0.64 kip/ft at $0.0L_{ds}$ = 28.1 kips **Table A-4**

V_{LL} = (28.1) (1.2) (0.933) = 31.5 kips

Table 14 column c presents M_{LL} for an exterior girder.
Table 15 column c presents M_{LL} for an interior girder.
Table 16 column c presents V_{LL} for an exterior girder.
Table 17 column c presents V_{LL} for an interior girder.

Step 7.3. Negative Moment and Shear Force Between Points of Contraflexure

For negative moment between points of contraflexure under a uniform load on all spans,one must use 90 percent of the effect of two design trucks spaced a minimum of 50.0 ft between the lead axle of one truck and the rear axle of the other truck, combined with 90 percent of the effect of the design lane load. LRFD Art. 3.6.1.3.1

Where multiple lanes of heavier vehicles are considered probable, consideration should be given to investigating negative moment at interior supports for pairs of the design tandem spaced from 26.0 ft to 40.0 ft apart, combined with the design lane load. LRFD Art. C3.6.1.3.1

MDOT practice excludes the application of design tandem loads on continuous structures.

Live load, described in **Step 4.2**, is applied to calculate a maximum negative moment and shear force between points of contraflexure. Special attention should be given to the spacing between trucks. AASHTO LRFD specified a minimum spacing of 50 ft. Depending on the design span length, the critical spacing between trucks to generate the maximum moment and shear force needs to be identified. This can be accomplished using influence lines or available software that allows defining a variable spacing between trucks.

Moment per lane and shear force per lane due to 2-HS20-28 trucks are presented in **Appendix A**.

Moment and shear force due to two trucks (M_{L2T} and V_{L2T}) are calculated using the following relationships and are presented in **Table 14** to **Table 17**.

For an interior girder due to 90% of the two-truck load,

M_{L2T} = (0.9)(moment per lane)(factor for HL-93 Mod)(DFM_{IB})(1 + IM)
= (0.9)(moment per lane)(1.2)(DFM_{IB})(1.33)
= (moment per lane)(1.436)(DFM_{IB})

V_{L2T} = (0.9)(shear force per lane)(factor for HL-93 Mod)(DFV_{IB})(1 + IM)
= (shear force per lane)(1.436)(DFV_{IB})

For an exterior girder due to 90% of the two-truck load,

M_{L2T} = (0.9)(moment per lane)(factor for HL-93 Mod)(DFM_{EB})(1 + IM)
= (0.9)(moment per lane)(1.2)(DFM_{EB})(1.33)
= (moment per lane)(1.436)(DFM_{EB})

V_{L2T} = (0.9)(shear force per lane)(factor for HL-93 Mod)(DFV_{EB})(1 + IM)
= (shear force per lane)(1.436)(DFV_{EB})

The following example demonstrates the calculation process.

Interior girder negative moment per lane due to 2-HS20-28 trucks at $1.0L_{ds}$ = -1159 kip-ft **Table A-3**

M_{L2T} = (-1159) (1.436) (0.668) = -1112 kip-ft

Interior girder negative shear force per lane due to 2-HS20-28 trucks at $1.0L_{ds}$ = -78.2 kips **Table A-4**

V_{L2T} = (-78.2) (1.436) (0.933) = -104.8 kips

Table 14 column d presents M_{L2T} for an exterior girder.
Table 15 column d presents M_{L2T} for an interior girder.
Table 16 column d presents V_{L2T} for an exterior girder.
Table 17 column d presents V_{L2T} for an interior girder.

Moment and shear force due to 90% of lane load (M_{L2L} and V_{L2L}) are calculated using the following relationships and are presented in **Table 14** to **Table 17**.

For an interior girder due to 90% of lane load,

M_{L2L} = (0.9)(moment per lane)(factor for HL-93 Mod)(DFM_{IB})
= (0.9)(moment per lane)(1.2)(DFM_{IB})
= (moment per lane)(1.08)(DFM_{IB})

V_{L2L} = (0.9)(shear force per lane)(factor for HL-93 Mod)(DFV_{IB})
= (shear force per lane)(1.08)(DFV_{IB})

For an an exterior girder due to 90% of lane load,

M_{L2L} = (0.9)(moment per lane)(factor for HL-93 Mod)(DFM_{EB})
= (0.9)(moment per lane)(1.2)(DFM_{EB})
= (moment per lane)(1.08)(DFM_{EB})

V_{L2L} = (0.9)(shear force per lane)(factor for HL-93 Mod)(DFV_{EB})
= (shear force per lane)(1.08)(DFV_{EB})

The following example demonstrates the calculation process.

Interior girder negative moment per lane due to lane load at $1.0L_{ds}$ = -683 kip-ft **Table A-3**

M_{L2L} = (-683) (1.08) (0.668) = -493 kip-ft

Interior girder negative shear force per lane due to lane load at $1.0L_{ds}$ = -40.1 kips **Table A-4**

V_{L2L} = (-40.1) (1.08) (0.933) = -40.4 kips

Table 14 column e presents M_{L2L} for an exterior girder.
Table 15 column e presents M_{L2L} for an interior girder.
Table 16 column e presents V_{L2L} for an exterior girder.
Table 17 column e presents V_{L2L} for an interior girder.

Step 7.4. Fatigue Load Moment and Shear Forces

The typical procedure is to use a single-line girder model to calculate moment and shear force along the span due to HS20 fatigue truck.

Impact factor for fatigue truck load = 15%

Appendix A presents moment and shear force per lane due to HS20 fatigue truck.

Interior and exterior girder moment and shear force due to HS20 fatigue truck load (M_{LFT} and V_{LFT}) are calculated using the following relationships and are presented in **Table 14** and **Table 15**.

For an interior girder,

M_{LFT} = (moment per lane) (DFM_{IB})(1 + IM)
= (moment per lane) ((DFM_{IB}) (1.15)

V_{LFT} = (shear force per lane) (DFV_{IB})(1 + IM)
= (shear force per lane) (DFV_{IB}) (1.15)

For an exterior girder,

M_{LFT} = (moment per lane) (DFM_{EB})(1 + IM)
= (moment per lane) (DFM_{EB}) (1.15)

V_{LFT} = (shear force per lane) (DFV_{EB})(1 + IM)
= (shear force per lane) (DFV_{EB}) (1.15)

The following example demonstrates the calculation process.

For fatigue limit state, DFM_{IB}	= 0.384	**Step 5.6**
Interior girder negative moment per lane due to HS20 fatigue truck at $1.0L_{ds}$	= -531 kip-ft	**Table A-3**
M_{LFT} = (-531) (0.384)(1.15)	= -234 kip-ft	
For fatigue limit state, DFV_{IB}	= 0.624	**Step 5.6**
Interior girder negative shear per lane due HS20 fatigue truck at $1.0L_{ds}$	= -62.1 kips	**Table A-4**
V_{LFT} = (-62.1) (0.624)(1.15)	= -44.6 kips	

Table 14 column j presents M_{LFT} for an exterior girder.
Table 15 column j presents M_{LFT} for an interior girder.

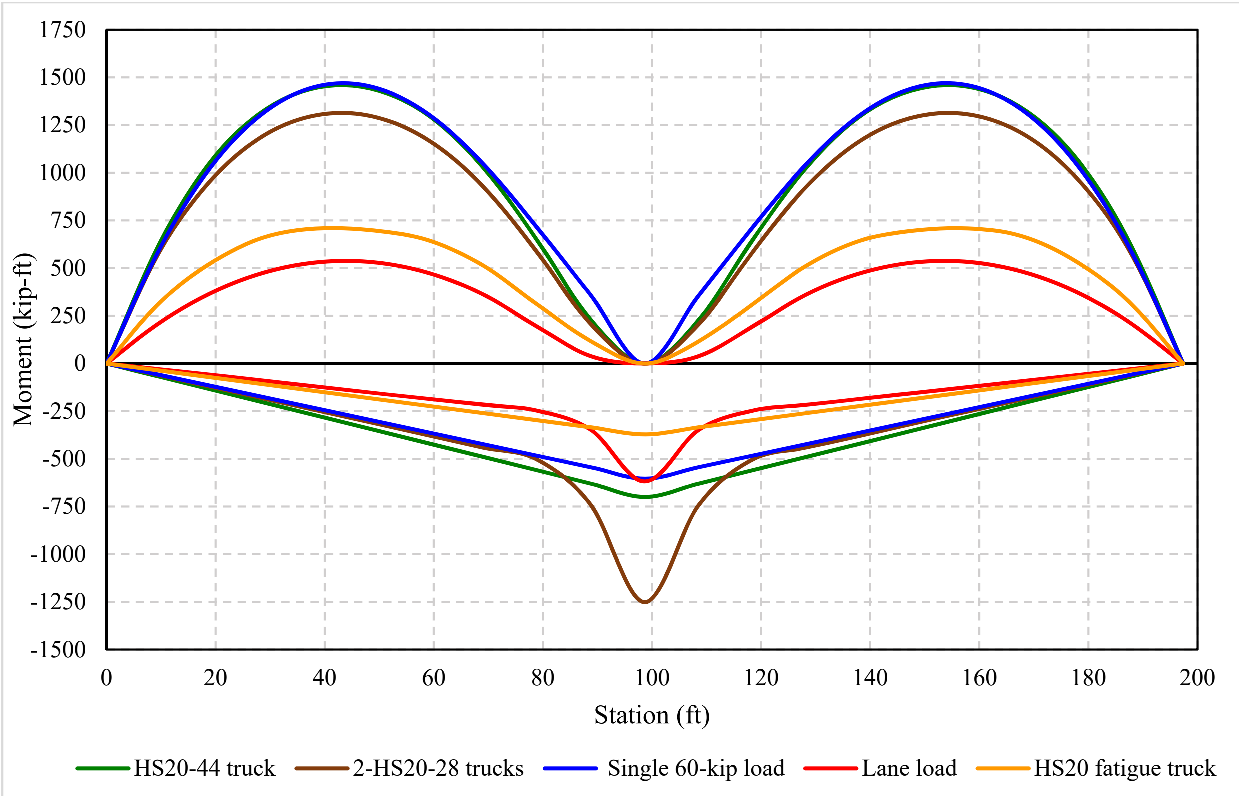


Figure 29. Unfactored live load moment envelopes for an exterior girder

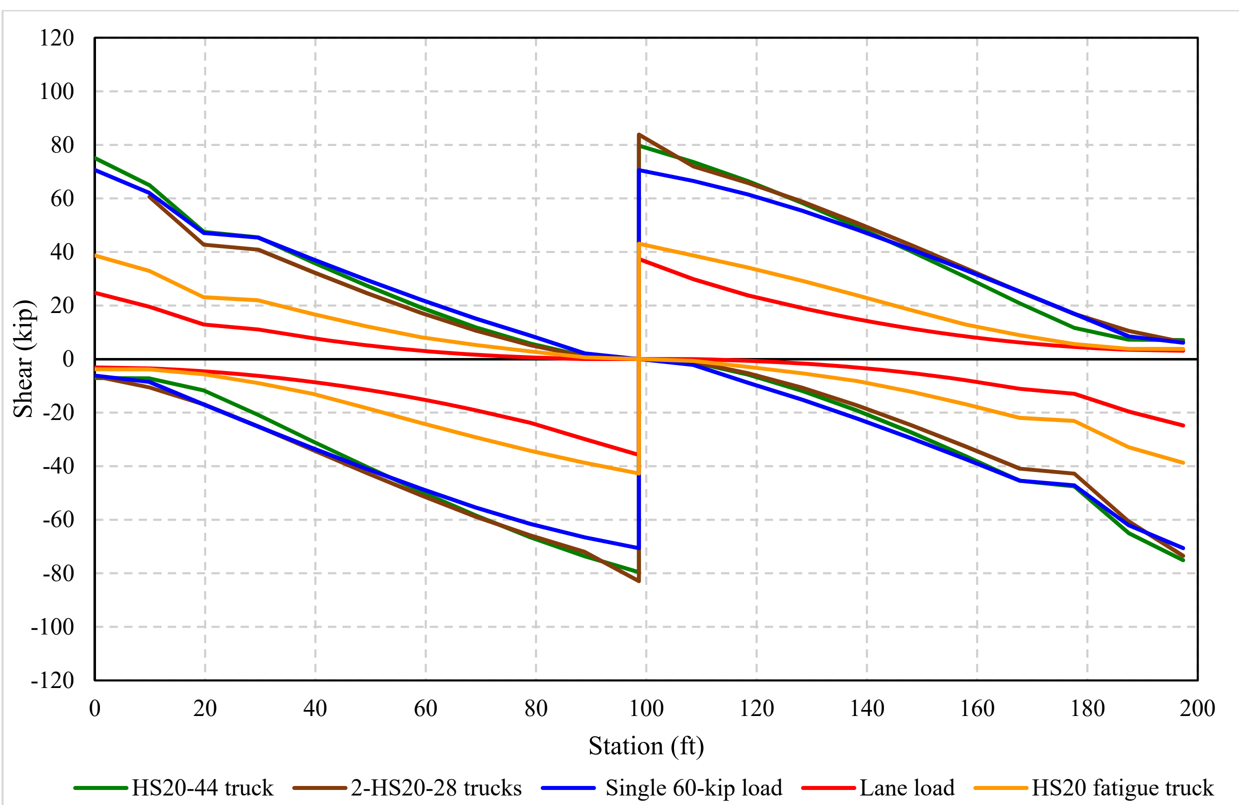


Figure 30. Unfactored live load shear envelopes for an exterior girder

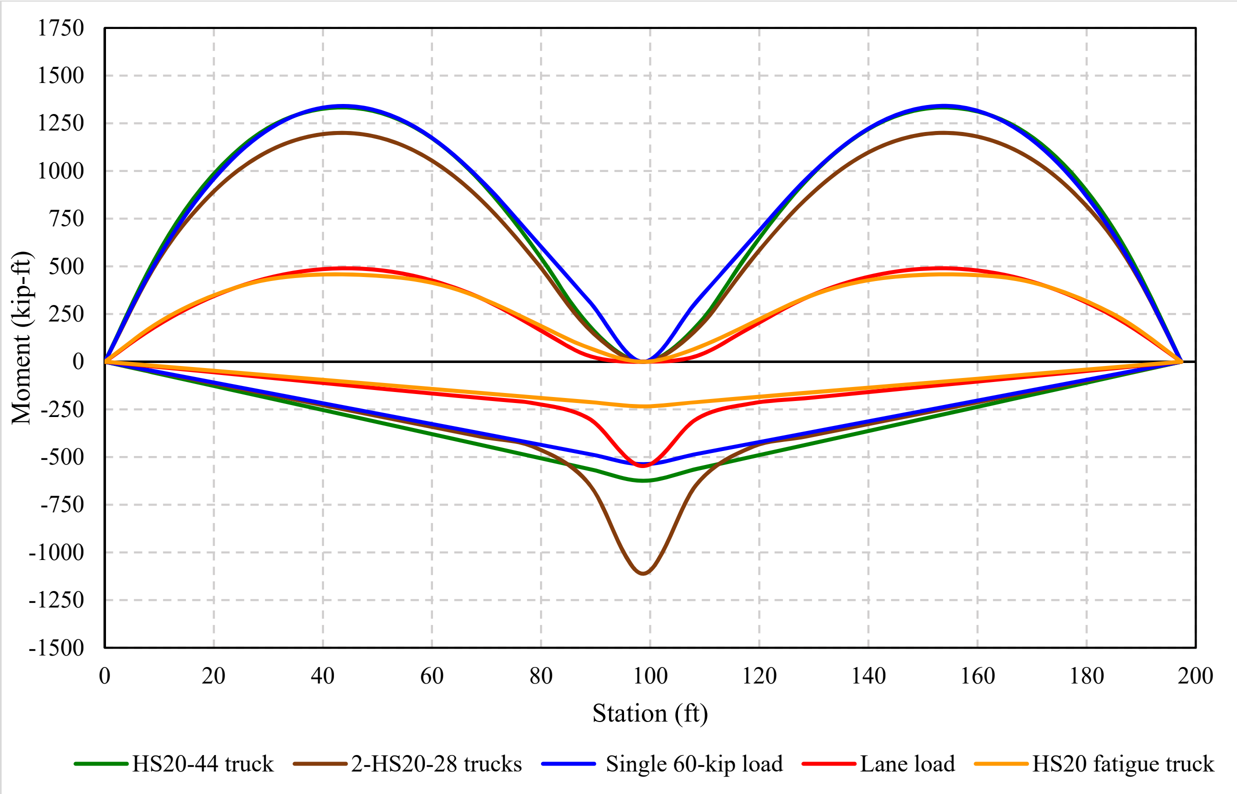


Figure 31. Unfactored live load moment envelopes for an interior girder

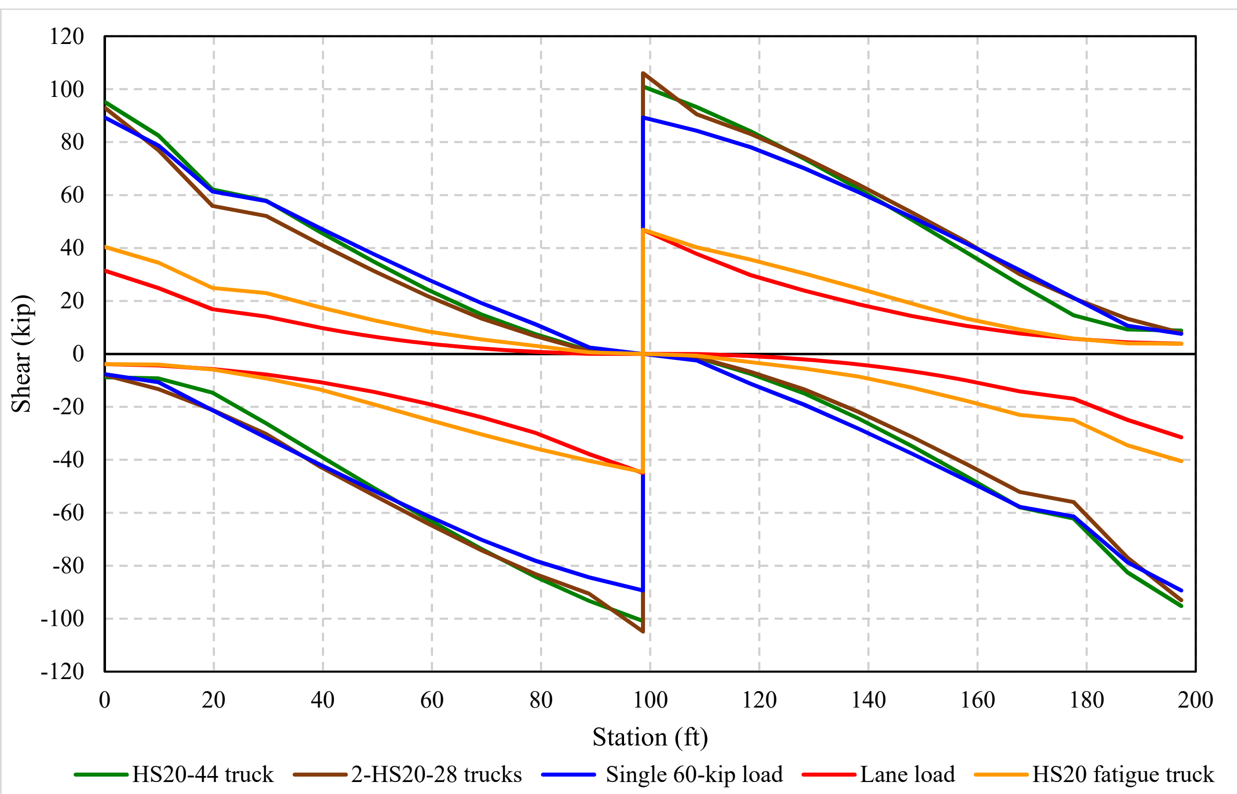


Figure 32. Unfactored live load shear envelopes for an interior girder

Table 14. Unfactored Live Load Moment on an Exterior Girder (kip-ft)

Location ($\times L_{ds}$)	M_{LT} (a)		M_{LA} (b)		M_{LL} (c)		M_{L2T} (d)		M_{L2L} (e)		$M_{LT} + M_{LL}$ (f)		$M_{L2T} + M_{L2L}$ (g)		$M_{LA} + M_{LL}$ (h)		HL – 93 Mod with impact, M_{HL-M} (i) = max[|(f), (g), (h)|]		M_{LFT} (j)	
	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL
0.0 (Abut A)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.1	638	-70	610	-60	216	-31	595	-63	195	-28	854	-101	789	-91	826	-91	854	-101	323	-37
0.2	1082	-140	1053	-121	378	-62	979	-126	340	-56	1460	-202	1319	-182	1431	-183	1460	-202	538	-74
0.3	1340	-210	1332	-181	482	-93	1208	-189	434	-84	1822	-303	1642	-272	1814	-274	1822	-303	668	-112
0.4	1451	-280	1459	-242	533	-124	1306	-252	479	-111	1984	-404	1786	-363	1992	-366	1992	-404	709	-149
0.5	1435	-350	1446	-302	530	-155	1291	-315	477	-139	1964	-505	1768	-454	1975	-457	1975	-505	697	-186
0.6	1297	-420	1303	-363	473	-186	1167	-378	425	-167	1770	-605	1593	-545	1776	-548	1776	-605	644	-223
0.7	1028	-490	1046	-423	362	-217	927	-441	326	-195	1389	-706	1252	-636	1408	-640	1408	-706	513	-260
0.8	647	-560	712	-483	193	-248	583	-504	173	-223	839	-807	757	-727	905	-731	905	-807	310	-297
0.9	228	-630	356	-544	38	-348	205	-743	34	-313	266	-978	239	-1056	394	-892	394	-1056	117	-335
1.0 (Pier)	0	-700	0	-604	0	-618	0	-1252	0	-557	0	-1318	0	-1808	0	-1223	0	-1808	0	-372
1.1	228	-630	356	-544	38	-348	205	-743	34	-313	266	-978	239	-1056	394	-892	394	-1056	117	-335
1.2	647	-560	712	-483	193	-248	583	-504	173	-223	839	-807	757	-727	905	-731	905	-807	310	-297
1.3	1028	-490	1046	-423	362	-217	927	-441	326	-195	1389	-706	1252	-636	1408	-640	1408	-706	513	-260
1.4	1297	-420	1303	-363	473	-186	1167	-378	425	-167	1770	-605	1593	-545	1776	-548	1776	-605	644	-223
1.5	1435	-350	1446	-302	530	-155	1291	-315	477	-139	1964	-505	1768	-454	1975	-457	1975	-505	697	-186
1.6	1451	-280	1459	-242	533	-124	1306	-252	479	-111	1984	-404	1786	-363	1992	-366	1992	-404	709	-149
1.7	1340	-210	1332	-181	482	-93	1208	-189	434	-84	1822	-303	1642	-272	1814	-274	1822	-303	668	-112
1.8	1082	-140	1053	-121	378	-62	979	-126	340	-56	1460	-202	1319	-182	1431	-183	1460	-202	538	-74
1.9	638	-70	610	-60	216	-31	595	-63	195	-28	854	-101	789	-91	826	-91	854	-101	323	-37
2.0 (Abut B)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

M_{LT} : Design moment per girder due to truck load
M_{LA} : Design moment per girder due to single 60-kip load
M_{LL} : Design moment per girder due to lane load
M_{L2T} : Design moment per girder due to 90% of two-truck load
M_{L2L} : Design moment per girder due to 90% of lane load
M_{LFT} : Design moment per girder due to fatigue truck load
Shaded cells indicate the values selected for design.

Table 15. Unfactored Live Load Moment on an Interior Girder (kip-ft)

Location ($\times L_{ds}$)	M_{LT} (a)		M_{LA} (b)		M_{LL} (c)		M_{L2T} (d)		M_{L2L} (e)		$M_{LT} + M_{LL}$ (f)		$M_{L2T} + M_{L2L}$ (g)		$M_{LA} + M_{LL}$ (h)		HL – 93 Mod with impact, $M_{HL\text{-}M}$ (i) = max[|(f), (g), (h)|]		M_{LFT} (j)	
	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL
0.0 (Abut A)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.1	575	-62	549	-54	195	-27	535	-56	176	-25	770	-90	711	-81	744	-81	770	-90	206	-23
0.2	977	-125	951	-107	341	-55	886	-112	307	-49	1318	-180	1193	-162	1292	-162	1318	-180	346	-47
0.3	1220	-187	1210	-161	438	-82	1098	-169	394	-74	1658	-269	1492	-242	1648	-243	1658	-269	430	-70
0.4	1324	-250	1331	-215	485	-109	1191	-225	437	-99	1809	-359	1628	-323	1816	-324	1816	-359	457	-94
0.5	1313	-312	1321	-269	483	-137	1182	-281	435	-123	1797	-449	1617	-404	1804	-405	1804	-449	452	-117
0.6	1187	-374	1190	-322	433	-164	1068	-337	389	-148	1619	-539	1458	-485	1622	-486	1622	-539	418	-141
0.7	940	-437	949	-376	333	-192	846	-393	299	-172	1273	-628	1145	-566	1281	-568	1281	-628	333	-164
0.8	587	-499	636	-430	181	-219	529	-450	163	-197	767	-718	692	-647	817	-649	817	-718	201	-188
0.9	189	-562	317	-483	31	-299	170	-641	28	-269	220	-861	198	-911	348	-782	348	-911	72	-211
1.0 (Pier)	0	-624	0	-537	0	-547	0	-1112	0	-493	0	-1171	0	-1604	0	-1084	0	-1604	0	-234
1.1	189	-562	317	-483	31	-299	170	-641	28	-269	220	-861	198	-911	348	-782	348	-911	72	-211
1.2	587	-499	636	-430	181	-219	529	-450	163	-197	767	-718	692	-647	817	-649	817	-718	201	-188
1.3	940	-437	949	-376	333	-192	846	-393	299	-172	1273	-628	1145	-566	1281	-568	1281	-628	333	-164
1.4	1187	-374	1190	-322	433	-164	1068	-337	389	-148	1619	-539	1458	-485	1622	-486	1622	-539	418	-141
1.5	1313	-312	1321	-269	483	-137	1182	-281	435	-123	1797	-449	1617	-404	1804	-405	1804	-449	452	-117
1.6	1324	-250	1331	-215	485	-109	1191	-225	437	-99	1809	-359	1628	-323	1816	-324	1816	-359	457	-94
1.7	1220	-187	1210	-161	438	-82	1098	-169	394	-74	1658	-269	1492	-242	1648	-243	1658	-269	430	-70
1.8	977	-125	951	-107	341	-55	886	-112	307	-49	1318	-180	1193	-162	1292	-162	1318	-180	346	-47
1.9	575	-62	549	-54	195	-27	535	-56	176	-25	770	-90	711	-81	744	-81	770	-90	206	-23
2.0 (Abut B)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

M_{LT} : Design moment per girder due to truck load
M_{LA} : Design moment per girder due to single 60-kip load
M_{LL} : Design moment per girder due to lane load
M_{L2T} : Design moment per girder due to 90% of two-truck load
M_{L2L} : Design moment per girder due to 90% of lane load
M_{LFT} : Design moment per girder due to fatigue truck load
Shaded cells indicate the values selected for design.

Table 16. Unfactored Live Load Shear on an Exterior Girder (kip)

Location ($\times L_{ds}$)	V_{LT} (a)		V_{LA} (b)		V_{LL} (c)		V_{L2T} (d)		V_{L2L} (e)		$V_{LT} + V_{LL}$ (f)		$V_{L2T} + V_{L2L}$ (g)		$V_{LA} + V_{LL}$ (h)		HL – 93 Mod with impact, V_{HL-M} (i) = max[|(f), (g), (h)|]	
	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL
0.0 (Abut A)	75.1	-7.1	70.6	-6.1	24.8	-3.1	73.4	-6.4	22.3	-2.8	99.9	-10.2	95.8	-9.2	95.4	-9.3	99.9	-10.2
0.1	65.0	-7.2	62.0	-8.5	19.6	-3.5	60.6	-10.5	17.6	-3.1	84.5	-10.7	78.2	-13.7	81.6	-12.0	84.5	-13.7
0.2	47.5	-11.7	47.1	-17.0	12.9	-4.6	42.8	-16.9	11.6	-4.2	60.5	-16.3	54.4	-21.1	60.1	-21.6	60.5	-21.6
0.3	45.5	-20.8	45.4	-25.2	11.1	-6.2	40.9	-25.1	10.0	-5.6	56.6	-27.0	50.9	-30.7	56.5	-31.4	56.6	-31.4
0.4	36.2	-30.7	37.3	-33.3	7.8	-8.5	32.6	-33.9	7.1	-7.7	44.0	-39.2	39.6	-41.6	45.2	-41.8	45.2	-41.8
0.5	27.4	-40.3	29.6	-41.0	5.2	-11.5	24.6	-42.6	4.7	-10.3	32.6	-51.7	29.3	-52.9	34.8	-52.5	34.8	-52.9
0.6	19.2	-49.5	22.2	-48.4	3.1	-15.0	17.3	-50.9	2.8	-13.5	22.3	-64.5	20.1	-64.4	25.3	-63.4	25.3	-64.5
0.7	11.8	-58.3	15.2	-55.4	1.6	-19.1	10.7	-58.8	1.5	-17.2	13.5	-77.4	12.1	-76.0	16.8	-74.5	16.8	-77.4
0.8	5.8	-66.5	8.8	-61.5	0.6	-23.8	5.2	-65.8	0.6	-21.4	6.4	-90.2	5.8	-87.2	9.4	-85.3	9.4	-90.2
0.9	1.2	-73.6	2.2	-66.5	0.1	-29.9	1.0	-72.0	0.0	-26.9	1.2	-103.4	1.1	-98.8	2.2	-96.4	2.2	-103.4
1.0 (Pier)	0.0	-79.6	0.0	-70.6	0.0	-35.8	0.0	-82.9	0.0	-32.2	0.0	-115.3	0.0	-115.1	0.0	-106.3	0.0	-115.3
1.1	73.6	-1.2	66.5	-2.2	29.9	-0.1	72.0	-1.0	26.9	0.0	103.4	-1.2	98.8	-1.1	96.4	-2.2	103.4	-2.2
1.2	66.5	-5.8	61.5	-8.8	23.8	-0.6	65.8	-5.2	21.4	-0.6	90.2	-6.4	87.2	-5.8	85.3	-9.4	90.2	-9.4
1.3	58.3	-11.8	55.4	-15.2	19.1	-1.6	58.8	-10.7	17.2	-1.5	77.4	-13.5	76.0	-12.1	74.5	-16.8	77.4	-16.8
1.4	49.5	-19.2	48.4	-22.2	15.0	-3.1	50.9	-17.3	13.5	-2.8	64.5	-22.3	64.4	-20.1	63.4	-25.3	64.5	-25.3
1.5	40.3	-27.4	41.0	-29.6	11.5	-5.2	42.6	-24.6	10.3	-4.7	51.7	-32.6	52.9	-29.3	52.5	-34.8	52.9	-34.8
1.6	30.7	-36.2	33.3	-37.3	8.5	-7.8	33.9	-32.6	7.7	-7.1	39.2	-44.0	41.6	-39.6	41.8	-45.2	41.8	-45.2
1.7	20.8	-45.5	25.2	-45.4	6.2	-11.1	25.1	-40.9	5.6	-10.0	27.0	-56.6	30.7	-50.9	31.4	-56.5	31.4	-56.6
1.8	11.7	-47.5	17.0	-47.1	4.6	-12.9	16.9	-42.8	4.2	-11.6	16.3	-60.5	21.1	-54.4	21.6	-60.1	21.6	-60.5
1.9	7.2	-65.0	8.5	-62.0	3.5	-19.6	10.5	-60.6	3.1	-17.6	10.7	-84.5	13.7	-78.2	12.0	-81.6	13.7	-84.5
2.0 (Abut B)	7.1	-75.1	6.1	-70.6	3.1	-24.8	6.4	-73.4	2.8	-22.3	10.2	-99.9	9.2	-95.8	9.3	-95.4	10.2	-99.9

V_{LT} : Design shear per girder due to truck load
V_{LA} : Design shear per girder due to single 60-kip load
V_{LL} : Design shear per girder due to lane load
V_{L2T} : Design shear per girder due to 90% of two-truck load
V_{L2L} : Design shear per girder due to 90% of lane load
Shaded cells indicate the values selected for design.

Table 17. Unfactored Live Load Shear on an Interior Girder (kip)

Location ($\times L_{ds}$)	V_{LT} (a)		V_{LA} (b)		V_{LL} (c)		V_{L2T} (d)		V_{L2L} (e)		$V_{LT} + V_{LL}$ (f)		$V_{L2T} + V_{L2L}$ (g)		$V_{LA} + V_{LL}$ (h)		HL – 93 Mod with impact, V_{HL-M} (i) = max[|(f), (g), (h)|]	
	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL
0.0 (Abut A)	95.2	-8.8	89.3	-7.6	31.5	-3.9	93.0	-8.0	28.3	-3.5	126.6	-12.7	121.3	-11.4	120.8	-11.5	126.6	-12.7
0.1	82.5	-9.3	78.7	-10.7	24.9	-4.3	76.9	-13.3	22.4	-3.9	107.4	-13.6	99.3	-17.2	103.5	-15.0	107.4	-17.2
0.2	62.2	-14.6	61.4	-21.2	16.9	-5.7	55.9	-21.1	15.2	-5.2	79.1	-20.4	71.2	-26.3	78.3	-27.0	79.1	-27.0
0.3	57.9	-26.2	57.7	-31.6	14.1	-7.7	52.1	-30.2	12.7	-7.0	72.0	-33.9	64.8	-37.1	71.8	-39.3	72.0	-39.3
0.4	46.1	-38.5	47.6	-41.8	10.0	-10.6	41.5	-42.5	9.0	-9.6	56.1	-49.1	50.5	-52.1	57.5	-52.4	57.5	-52.4
0.5	34.8	-50.6	37.7	-51.6	6.6	-14.3	31.3	-53.4	5.9	-12.9	41.4	-64.9	37.3	-66.3	44.3	-66.0	44.3	-66.3
0.6	24.3	-62.4	28.2	-61.1	3.9	-18.8	21.9	-64.0	3.5	-16.9	28.2	-81.1	25.4	-80.9	32.2	-79.9	32.2	-81.1
0.7	14.9	-73.7	19.2	-70.1	2.0	-23.9	13.4	-74.2	1.8	-21.5	16.9	-97.6	15.2	-95.7	21.2	-94.1	21.2	-97.6
0.8	7.5	-84.1	11.3	-78.1	0.8	-29.8	6.8	-83.1	0.7	-26.8	8.3	-113.9	7.5	-109.9	12.1	-107.9	12.1	-113.9
0.9	1.3	-93.2	2.5	-84.3	0.1	-37.8	1.2	-90.5	0.0	-34.0	1.4	-131.0	1.2	-124.5	2.5	-122.1	2.5	-131.0
1.0 (Pier)	0.0	-100.8	0.0	-89.3	0.0	-44.9	0.0	-104.8	0.0	-40.4	0.0	-145.7	0.0	-145.3	0.0	-134.3	0.0	-145.7
1.1	93.2	-1.3	84.3	-2.5	37.8	-0.1	90.5	-1.2	34.0	0.0	131.0	-1.4	124.5	-1.2	122.1	-2.5	131.0	-2.5
1.2	84.1	-7.5	78.1	-11.3	29.8	-0.8	83.1	-6.8	26.8	-0.7	113.9	-8.3	109.9	-7.5	107.9	-12.1	113.9	-12.1
1.3	73.7	-14.9	70.1	-19.2	23.9	-2.0	74.2	-13.4	21.5	-1.8	97.6	-16.9	95.7	-15.2	94.1	-21.2	97.6	-21.2
1.4	62.4	-24.3	61.1	-28.2	18.8	-3.9	64.0	-21.9	16.9	-3.5	81.1	-28.2	80.9	-25.4	79.9	-32.2	81.1	-32.2
1.5	50.6	-34.8	51.6	-37.7	14.3	-6.6	53.4	-31.3	12.9	-5.9	64.9	-41.4	66.3	-37.3	66.0	-44.3	66.3	-44.3
1.6	38.5	-46.1	41.8	-47.6	10.6	-10.0	42.5	-41.5	9.6	-9.0	49.1	-56.1	52.1	-50.5	52.4	-57.5	52.4	-57.5
1.7	26.2	-57.9	31.6	-57.7	7.7	-14.1	30.2	-52.1	7.0	-12.7	33.9	-72.0	37.1	-64.8	39.3	-71.8	39.3	-72.0
1.8	14.6	-62.2	21.2	-61.4	5.7	-16.9	21.1	-55.9	5.2	-15.2	20.4	-79.1	26.3	-71.2	27.0	-78.3	27.0	-79.1
1.9	9.3	-82.5	10.7	-78.7	4.3	-24.9	13.3	-76.9	3.9	-22.4	13.6	-107.4	17.2	-99.3	15.0	-103.5	17.2	-107.4
2.0 (Abut B)	8.8	-95.2	7.6	-89.3	3.9	-31.5	8.0	-93.0	3.5	-28.3	12.7	-126.6	11.4	-121.3	11.5	-120.8	12.7	-126.6

V_{LT} : Design shear per girder due to truck load
V_{LA} : Design shear per girder due to single 60-kip load
V_{LL} : Design shear per girder due to lane load
V_{L2T} : Design shear per girder due to 90% of two-truck load
V_{L2L} : Design shear per girder due to 90% of lane load
Shaded cells indicate the values selected for design.

Step 7.5. Combined Loads for Limit States

The force effects must be combined for each of the applicable limit state.

Step 7.5.1. Load combinations and load factors

LRFD Art. 3.4.1

Table 18. Load Combinations and Load Factors Used in the Design

Limit state	Load factors for load			
	DC	DW	LL	SE
Strength I	1.25/0.90	1.50/0.65	1.75	γ_{SE}
Service II	1.00	1.00	1.30	-
Fatigue I	-	-	1.75	-
Fatigue II	-	-	0.80	-

The load factor for settlement, γ_{SE}, should be considered on a project-specific basis. In lieu of project specific information to the contrary, γ_{SE}, may be taken as 1.0. Load combinations which include settlement shall also be applied without settlement. LRFD Art. 3.4.1

In this example, γ_{SE} =1.0 is used.

Step 7.5.2. Resistance factors, ϕ

1. Strength Limit State LRFD Art. 6.5.4.2
   - Flexure 1.00
   - Shear 1.00

2. Nonstrength Limit States 1.00 LRFD Art. C1.3.2.1

Step 7.5.3. Load modifiers

Table 19. Load Modifiers for Limit States

Load modifier	Limit state			Reference
	Strength	Service	Fatigue	
Ductility, η_D	1.0	1.0	1.0	A1.3.3
Redundancy, η_R	1.0	1.0	1.0	A1.3.4
Importance, η_I	1.0	N/A	N/A	A1.3.5
$\eta = \eta_D \eta_R \eta_I$	1.0	1.0	1.0	A1.3.2.1

The load combinations used in this example are shown below:

Strength I Limit State: $U = \eta[\gamma_P DC + \gamma_P DW + 1.75(LL+IM) + 1.0(SE)]$

Strength I Limit State: $U = \eta[\gamma_P DC + \gamma_P DW + 1.75(LL+IM)]$
Basic load combination relating to the normal vehicular use of the bridge without wind (Two combinations are considered with and without the impact of support settlement.)

Service II Limit State: $U = 1.00\ (DC + DW) + 1.30(LL+IM)$
Load combination intended to control yielding of steel structures and slip of slip-critical connections due to vehicular live load

Fatigue I limit State: $U = 1.75(LL + IM)$
Fatigue and fracture load combination related to infinite load-induced fatigue life

Fatigue II limit State: $U = 0.80(LL + IM)$
Fatigue and fracture load combination related to finite load-induced fatigue life

The load factor for permanent loading, γ_p, shall be selected to produce the total extreme factored force effect. For each load combination, both positive and negative extremes shall be investigated. LRFD Art. 3.4.1

BDS calculations use a factor of 0.75 for fatigue limit state II and soon will be updated to reflect the new values.

Table 10 to **Table 13** present the moments and shear forces due to dead loads (DC and DW), construction loads (DC), and support settlement (SE).

Table 14 to **Table 17** present the moments and shear forces due to HL–93 Mod and fatigue loads (LL).

Table 20 and **Table 21** present the combined moments and shear forces for Strength I and Service II limit states.

Table 22 and **Table 23** present the combined stresses for Strength I and Service II limit states.

The following example demostrates the calculation of combined moments, shear forces, and stresses at $0.4L_{ds}$ distance of an interior girder:

Strength I Limit State [**without** support settlement (**SE**)]

Maximum positive moment

$= 1.25\ (M_{DL}) + 1.25\ (M_b) + 1.50\ (M_{ws}) + 1.75\ (M_{HL\text{-}M(+)})$

$= 1.25\ (120 + 635 + 11 + 72) + 1.25\ (96) + 1.50\ (152) + 1.75\ (1816) \qquad = 4574$ kip-ft

Note: $M_{DL} = M_g + M_D + M_H + M_f$

Maximum negative moment

$= 0.90\ (M_{DL}) + 0.90\ (M_b) + 0.65\ (M_{ws}) + 1.75\ (M_{HL\text{-}M(-)})$

$= 0.90\ (120 + 635 + 11 + 72) + 0.90\ (96) + 0.65\ (152) + 1.75\ (-359) \qquad = 311$ kip-ft

Strength I Limit State [**with** support settlement(**SE**)]

Maximum positive moment

= 1.25 (M_{DL}) + 1.25 (M_b) + 1.50 (M_{ws}) + 1.75 ($M_{HL\text{-}M(+)}$) + 1.0 (M_{SE})

= 1.25 (120 + 635 + 11 + 72) + 1.25 (96) + 1.50 (152) + 1.75 (1816) + 1.0 (-44) = 4530 kip-ft

Note: A negative moment is developed at $0.4L_{ds}$ due to support settlement at the abutments.

Maximum negative moment

= 0.90 (M_{DL}) + 0.90 (M_b) + 0.65 (M_{ws}) + 1.75 ($M_{HL\text{-}M(-)}$) + 1.0 (M_{SE})

= 0.90 (120 + 635 + 11 + 72) + 0.90 (96) + 0.65 (152) + 1.75 (-359) + 1.0 (-44) = 267 kip-ft

The following steps show the stresses at the girder top and bottom under Strength I limit state. Sign convention:

Compressive stress is +ve. Tensile stress is -ve.

Tensile stress at girder top (**without** support settlement),

$$f_{top-T} = 0.90\frac{M_{DL}}{S_{t\_nc}} + 0.90\frac{M_b}{S_{t\_c24}} + 0.65\frac{M_{WS}}{S_{t\_c24}} + 1.75\frac{M_{HL-M(-)}}{S_{t\_c8}}$$

$$f_{top-T} = 0.90 \times \frac{(120 + 635 + 11 + 72) \times 12}{573} + 0.90 \times \frac{96 \times 12}{3227} + 0.65 \times \frac{152 \times 12}{3227} + 1.75 \times \frac{(-359) \times 12}{17904}$$

= 16.06 ksi

Compressive stress at girder top (**without** support settlement),

$$f_{top-C} = 1.25\frac{M_{DL}}{S_{t\_nc}} + 1.25\frac{M_b}{S_{t\_c24}} + 1.5\frac{M_{WS}}{S_{t\_c24}} + 1.75\frac{M_{HL-M(+)}}{S_{t\_c8}}$$

$$f_{top-C} = 1.25 \times \frac{(120 + 635 + 11 + 72) \times 12}{573} + 1.25 \times \frac{96 \times 12}{3227} + 1.5 \times \frac{152 \times 12}{3227} + 1.75 \times \frac{1816 \times 12}{17904}$$

= 25.36 ksi

Tensile stress at girder bottom (**without** support settlement),

$$f_{bot-T} = -\left(1.25\frac{M_{DL}}{S_{b\_nc}} + 1.25\frac{M_b}{S_{b\_c24}} + 1.5\frac{M_{WS}}{S_{b\_c24}} + 1.75\frac{M_{HL-M(+)}}{S_{b\_c8}}\right)$$

$$f_{bot-T} = -1.25 \times \frac{(120 + 635 + 11 + 72) \times 12}{837} - 1.25 \times \frac{96 \times 12}{1121} - 1.5 \times \frac{152 \times 12}{1121} - 1.75 \times \frac{1816 \times 12}{1206}$$

= -50.37 ksi

Compressive stress at girder bottom (**without** support settlement),

$$f_{bot-C} = -\left(0.90\frac{M_{DL}}{S_{b\_nc}} + 0.90\frac{M_b}{S_{b\_c24}} + 0.65\frac{M_{WS}}{S_{b\_c24}} + 1.75\frac{M_{HL-M(-)}}{S_{b\_c8}}\right)$$

$$f_{bot-C} = -0.90 \times \frac{(120 + 635 + 11 + 72) \times 12}{837} - 0.90 \times \frac{96 \times 12}{1121} - 0.65 \times \frac{152 \times 12}{1121} - 1.75 \times \frac{(-359) \times 12}{1206}$$

= -6.54 ksi

Tensile stress at girder top (**with** support settlement),

$$f_{top-T} = 0.90\frac{\mathrm{M_{DL}}}{S_{t\_nc}} + 0.90\frac{\mathrm{M}_b}{S_{t\_c24}} + 0.65\frac{\mathrm{M_{WS}}}{S_{t\_c24}} + 1.75\frac{\mathrm{M_{HL-M(-)}}}{S_{t\_c8}} + 1.0\frac{M_{SE}}{S_{t\_c24}}$$

$$f_{top-T} = 0.90 \times \frac{(120 + 635 + 11 + 72) \times 12}{573} + 0.90 \times \frac{96 \times 12}{3227} + 0.65 \times \frac{152 \times 12}{3227} + 1.75 \times \frac{(-359) \times 12}{17904} + 1.0 \times \frac{(-44) \times 12}{3227} = 15.90 \text{ ksi}$$

Compressive stress at girder top (**with** support settlement),

$$f_{top-C} = 1.25\frac{\mathrm{M_{DL}}}{S_{t\_nc}} + 1.25\frac{\mathrm{M}_b}{S_{t\_c24}} + 1.5\frac{\mathrm{M_{WS}}}{S_{t\_c24}} + 1.75\frac{\mathrm{M_{HL-M(+)}}}{S_{t\_c8}} + 1.0\frac{M_{SE}}{S_{t\_c24}}$$

$$f_{top-C} = 1.25 \times \frac{(120 + 635 + 11 + 72) \times 12}{573} + 1.25 \times \frac{96 \times 12}{3227} + 1.5 \times \frac{152 \times 12}{3227} + 1.75 \times \frac{1816 \times 12}{17904} + 1.0 \times \frac{(-44) \times 12}{3227} = 25.20 \text{ ksi}$$

Tensile stress at girder bottom (**with** support settlement),

$$f_{bot-T} = -\left(1.25\frac{\mathrm{M_{DL}}}{S_{b\_nc}} + 1.25\frac{\mathrm{M}_b}{S_{b\_c24}} + 1.5\frac{\mathrm{M_{WS}}}{S_{b\_c24}} + 1.75\frac{\mathrm{M_{HL-M(+)}}}{S_{b\_c8}} + 1.0\frac{M_{SE}}{S_{b\_c24}}\right)$$

$$f_{bot-T} = -1.25 \times \frac{(120 + 635 + 11 + 72) \times 12}{837} - 1.25 \times \frac{96 \times 12}{1121} - 1.5 \times \frac{152 \times 12}{1121} - 1.75 \times \frac{1816 \times 12}{1206} - 1.0 \times \frac{(-44) \times 12}{1121} = -49.89 \text{ ksi}$$

Compressive stress at girder bottom (**with** support settlement),

$$f_{bot-C} = -\left(0.90\frac{\mathrm{M_{DL}}}{S_{b\_nc}} + 0.90\frac{\mathrm{M}_b}{S_{b\_c24}} + 0.65\frac{\mathrm{M_{WS}}}{S_{b\_c24}} + 1.75\frac{\mathrm{M_{HL-M(-)}}}{S_{b\_c8}} + 1.0\frac{M_{SE}}{S_{b\_c24}}\right)$$

$$f_{bot-C} = -0.90 \times \frac{(120 + 635 + 11 + 72) \times 12}{837} - 0.90 \times \frac{96 \times 12}{1121} - 0.65 \times \frac{152 \times 12}{1121} - 1.75 \times \frac{(-359) \times 12}{1206} - 1.0 \times \frac{(-44) \times 12}{1121} = -6.07 \text{ ksi}$$

The maximum stress resulting from the two cases, with or without support settlement, is summarized under Strength I in **Table 22** and **Table 23**. Service II limit state stresses are calculated following a similar procedure and are presented in the tables.

Table 20. Factored Moments and Shear Forces on an Exterior Girder

Location (× L_{ds})	Moment (kip-ft)						Shear (kip)					
	Load Combinations						Load Combinations					
	Strength I (without SE)		Strength I (with SE)		Service II		Strength I (without SE)		Strength I (with SE)		Service II	
	+ve	- ve	+ve	- ve	+ve	- ve	+ve	- ve	+ve	- ve	+ve	- ve
0.0 (Abut A)	0	0	0	0	0	0	243	28	241	26	183	39
0.1	2071	209	2058	195	1558	316	197	9	196	7	148	20
0.2	3519	292	3492	265	2646	486	136	-18	134	-19	102	-5
0.3	4352	247	4312	207	3271	509	109	-48	108	-50	82	-33
0.4	4655	72	4602	19	3495	381	73	-82	71	-84	51	-62
0.5	4439	-232	4373	-299	3328	104	41	-121	40	-123	23	-91
0.6	3711	-666	3631	-746	2772	-323	12	-161	10	-162	-5	-121
0.7	2510	-1245	2417	-1339	1848	-901	-16	-203	-18	-205	-31	-153
0.8	1070	-2150	963	-2257	595	-1631	-43	-246	-45	-248	-57	-186
0.9	-483	-3561	-603	-3681	-829	-2715	-70	-290	-71	-292	-82	-220
1.0 (Pier)	-1975	-6063	-2108	-6197	-2267	-4618	-89	-333	-90	-335	-102	-252
1.1	-483	-3561	-603	-3681	-829	-2715	290	42	292	71	220	82
1.2	1070	-2150	963	-2257	595	-1631	246	16	248	45	186	57
1.3	2510	-1245	2417	-1339	1848	-901	203	-9	205	18	153	31
1.4	3711	-666	3631	-746	2772	-323	161	-37	162	-10	121	5
1.5	4439	-232	4373	-299	3328	104	121	-67	123	-40	91	-23
1.6	4655	72	4602	19	3495	381	82	-99	84	-71	62	-51
1.7	4352	247	4312	207	3271	509	48	-109	50	-108	33	-82
1.8	3519	292	3492	265	2646	486	18	-136	19	-134	5	-102
1.9	2071	209	2058	195	1558	316	-9	-197	-7	-196	-20	-148
2.0 (Abut B)	0	0	0	0	0	0	-28	-243	-26	-241	-39	-183

Shaded cells indicate the values selected for design under Strength I limit state considering the two cases, with and without support settlement.

Table 21. Factored Moments and Shear Forces on an Interior Girder

Location (× L_{ds})	Moment (kip-ft)						Shear (kip)					
	Load Combinations						Load Combinations					
	Strength I (without SE)		Strength I (with SE)		Service II		Strength I (without SE)		Strength I (with SE)		Service II	
	+ve	- ve	+ve	- ve	+ve	- ve	+ve	- ve	+ve	- ve	+ve	- ve
0.0 (Abut A)	0	0	0	0	0	0	302	32	301	31	227	46
0.1	2029	303	2018	292	1532	414	246	9	245	8	185	23
0.2	3450	457	3427	435	2604	657	174	-23	173	-24	130	-7
0.3	4282	460	4249	427	3231	726	139	-60	138	-61	104	-41
0.4	4574	311	4530	267	3447	620	94	-102	93	-103	67	-76
0.5	4345	10	4289	-45	3268	339	55	-149	54	-150	32	-112
0.6	3594	-443	3528	-509	2693	-116	19	-197	17	-198	-1	-149
0.7	2343	-1049	2266	-1127	1736	-746	-16	-249	-17	-250	-33	-188
0.8	958	-1894	869	-1983	546	-1450	-48	-300	-49	-301	-63	-227
0.9	-696	-3483	-796	-3583	-1034	-2669	-80	-353	-81	-354	-94	-267
1.0 (Pier)	-2223	-6044	-2334	-6155	-2539	-4625	-102	-404	-103	-405	-117	-306
1.1	-696	-3483	-796	-3583	-1034	-2669	353	50	354	81	267	94
1.2	958	-1894	869	-1983	546	-1450	300	18	301	49	227	63
1.3	2343	-1049	2266	-1127	1736	-746	249	-13	250	17	188	33
1.4	3594	-443	3528	-509	2693	-116	197	-48	198	-17	149	1
1.5	4345	10	4289	-45	3268	339	149	-84	150	-54	112	-32
1.6	4574	311	4529	267	3447	620	102	-123	103	-93	76	-67
1.7	4282	460	4249	427	3231	726	60	-139	61	-138	41	-104
1.8	3450	457	3427	435	2604	657	23	-174	24	-173	7	-130
1.9	2029	303	2018	292	1532	414	-9	-246	-8	-245	-23	-185
2.0 (Abut B)	0	0	0	0	0	0	-32	-302	-31	-301	-46	-227

Shaded cells indicate the values selected for design under Strength I limit state considering the two cases, with and without support settlement.

Table 22. Factored Stresses in an Exterior Girder (ksi)

Location ($\times L_{ds}$)	Load Combinations											
	Strength I (without SE)				Strength I (with SE)				Service II			
	Top-Tensile	Top-Comp	Bottom-Tensile	Bottom-Comp	Top-Tensile	Top-Comp	Bottom-Tensile	Bottom-Comp	Top-Tensile	Top-Comp	Bottom-Tensile	Bottom-Comp
0.0 (Abut A)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.1	6.79	11.15	-28.95	-4.42	6.73	11.09	-28.77	-4.24	7.69	8.78	-21.88	-6.01
0.2	10.59	18.95	-33.98	-4.56	10.46	18.82	-33.73	-4.30	12.09	14.86	-25.67	-6.53
0.3	12.57	22.89	-41.92	-4.55	12.38	22.70	-41.54	-4.17	14.40	17.94	-31.65	-7.17
0.4	12.28	23.20	-44.59	-2.99	12.02	22.94	-44.08	-2.48	14.17	18.16	-33.63	-6.03
0.5	9.72	19.88	-42.09	0.12	9.40	19.56	-41.45	0.76	11.39	15.52	-31.67	-3.10
0.6	4.90	12.95	-34.48	4.79	4.52	12.57	-33.71	5.56	6.07	10.04	-25.82	1.61
0.7	-2.50	2.75	-22.16	11.21	-2.95	2.30	-21.27	12.11	-1.79	1.73	-16.25	8.10
0.8	-26.63	13.25	-8.47	17.01	-27.95	11.93	-7.62	17.85	-20.20	7.36	-4.70	12.90
0.9	-25.91	-3.51	3.51	25.91	-26.79	-4.39	4.39	26.79	-19.76	-6.04	6.04	19.76
1.0 (Pier)	-44.12	-14.37	14.37	44.12	-45.09	-15.34	15.34	45.09	-33.60	-16.50	16.50	33.60
1.1	-25.91	-3.51	3.51	25.91	-26.79	-4.39	4.39	26.79	-19.76	-6.04	6.04	19.76
1.2	-26.63	13.25	-8.47	17.01	-27.95	11.93	-7.62	17.85	-20.20	7.36	-4.70	12.90
1.3	-2.50	2.75	-22.16	11.21	-2.95	2.30	-21.27	12.11	-1.79	1.73	-16.25	8.10
1.4	4.90	12.95	-34.48	4.79	4.52	12.57	-33.71	5.56	6.07	10.04	-25.82	1.61
1.5	9.72	19.88	-42.09	0.12	9.40	19.56	-41.45	0.76	11.39	15.52	-31.67	-3.10
1.6	12.28	23.20	-44.59	-2.99	12.02	22.94	-44.08	-2.48	14.17	18.16	-33.63	-6.03
1.7	12.57	22.89	-41.92	-4.55	12.38	22.70	-41.54	-4.17	14.40	17.94	-31.65	-7.17
1.8	10.59	18.95	-33.98	-4.56	10.46	18.82	-33.73	-4.30	12.09	14.86	-25.67	-6.53
1.9	6.79	11.15	-28.95	-4.42	6.73	11.09	-28.77	-4.24	7.69	8.78	-21.88	-6.01
2.0 (Abut B)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

\+ compression - tension

Table 23. Factored Stresses in an Interior Girder (ksi)

Location (× L_{ds})	Load Combinations											
	Strength I (without SE)				Strength I (with SE)				Service II			
	Top-Tensile	Top-Comp	Bottom-Tensile	Bottom-Comp	Top-Tensile	Top-Comp	Bottom-Tensile	Bottom-Comp	Top-Tensile	Top-Comp	Bottom-Tensile	Bottom-Comp
0.0 (Abut A)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.1	8.55	12.39	-31.49	-6.67	8.51	12.35	-31.32	-6.50	9.58	9.84	-23.92	-8.49
0.2	13.47	20.81	-38.35	-7.41	13.39	20.73	-38.12	-7.18	15.17	16.47	-29.12	-9.74
0.3	16.14	25.12	-47.46	-8.02	16.02	25.00	-47.10	-7.67	18.20	19.88	-36.00	-11.08
0.4	16.06	25.36	-50.37	-6.54	15.90	25.20	-49.89	-6.07	18.16	20.06	-38.16	-10.03
0.5	13.25	21.56	-47.30	-2.98	13.04	21.35	-46.70	-2.39	15.06	17.02	-35.75	-6.61
0.6	7.69	13.69	-38.23	2.67	7.44	13.44	-37.51	3.38	8.88	10.77	-28.75	-0.80
0.7	-0.64	1.81	-23.38	10.42	-0.93	1.52	-22.55	11.26	-0.36	1.30	-17.31	7.39
0.8	-27.45	13.88	-13.88	27.45	-28.74	12.59	-12.59	28.74	-21.01	7.92	-7.92	21.01
0.9	-26.55	-5.31	5.31	26.55	-27.32	-6.07	6.07	27.32	-20.35	-7.88	7.88	20.35
1.0 (Pier)	-46.08	-16.95	16.95	46.08	-46.92	-17.79	17.79	46.92	-35.26	-19.36	19.36	35.26
1.1	-26.55	-5.31	5.31	26.55	-27.32	-6.07	6.07	27.32	-20.35	-7.88	7.88	20.35
1.2	-27.45	13.88	-13.88	27.45	-28.74	12.59	-12.59	28.74	-21.01	7.92	-7.92	21.01
1.3	-0.64	1.81	-23.38	10.42	-0.93	1.52	-22.55	11.26	-0.36	1.30	-17.31	7.39
1.4	7.69	13.69	-38.23	2.67	7.44	13.44	-37.51	3.38	8.88	10.77	-28.75	-0.80
1.5	13.25	21.56	-47.30	-2.98	13.04	21.35	-46.70	-2.39	15.06	17.02	-35.75	-6.61
1.6	16.07	25.37	-50.37	-6.55	15.90	25.20	-49.89	-6.07	18.16	20.06	-38.16	-10.03
1.7	16.14	25.12	-47.46	-8.02	16.02	25.00	-47.10	-7.67	18.20	19.88	-36.00	-11.08
1.8	13.47	20.81	-38.35	-7.41	13.39	20.73	-38.12	-7.18	15.17	16.47	-29.12	-9.74
1.9	8.55	12.39	-31.49	-6.67	8.51	12.35	-31.32	-6.50	9.58	9.84	-23.92	-8.49
2.0 (Abut B)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

+ compression - tension

Step 7.6. Optional Live-Load Deflection

The flexural rigidity of the short-term (n = 8) composite section is used. The deflection should be the larger of that resulting from the following: LRFD Art. 3.6.1.3.2

(1) the design truck,

(2) the design lane plus 25 percent of the design truck.

The dynamic load allowance of 33% is applied to the design truck load. When investigating the maximum absolute deflection for straight girder systems, all design lanes should be loaded, and all supporting components should be assumed to deflect equally. LRFD Art. 2.5.2.6.2

The distribution factor for live load deflection = 0.464 lanes/girder **Step 5.4**

For composite design, the stiffness of the design cross-section used for the determination of deflection and frequency should include the entire width of the roadway and the structurally continuous portions of the railings, sidewalks, and median barriers. For straight girder systems, the composite bending stiffness of an individual girder may be taken as the stiffness determined as specified above, divided by the number of girders. LRFD Art. 2.5.2.6.2

SAP 2000 is used to calculate the deflection of exterior and interior girders. **Table 24** shows the results.

Step 7.6.1. Deflection of an exterior girder

At $0.4L_{ds}$:

Maximum deflection due to the design truck		= 1.04 in.
Maximum deflection due to 25% of the design truck		= 0.26 in.
Maximum deflection due to lane load		= 0.59 in.
Maximum deflection due to 25% of the design truck plus lane load	= 0.26 + 0.59 in.	= 0.85 in.
Live load deflection	= max (1.04 in., 0.85 in.)	= 1.04 in.

Step 7.6.2. Deflection of an interior girder

A similar calculation procedure was followed for an interior girder.

Live load deflection at $0.4L_{ds}$ = 1.13 in.

Table 24. Optional Live Load Deflection for an Exterior and an Interior Girder (in.)

	Exterior girder					Interior girder				
Location (× L_{ds})	Design Truck (a)	Lane Load (b)	25% Design Truck (c)	Lane Load +25% Design Truck (d) = (b) + (c)	LL deflection (e) = max [(a), (d)]	Design Truck (f)	Lane Load (g)	25% Design Truck (h)	Lane Load +25% Design Truck (i) = (g) + (h)	LL deflection (j) = max [(f), (i)]
0.0 (Abut A)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.1	0.35	0.21	0.09	0.30	0.35	0.38	0.22	0.09	0.32	0.38
0.2	0.66	0.38	0.17	0.55	0.66	0.70	0.41	0.18	0.59	0.70
0.3	0.89	0.51	0.22	0.74	0.89	0.96	0.56	0.24	0.80	0.96
0.4	1.04	0.59	0.26	0.85	1.04	1.13	0.65	0.28	0.93	1.13
0.5	1.08	0.62	0.27	0.89	1.08	1.18	0.68	0.30	0.98	1.18
0.6	1.02	0.59	0.25	0.84	1.02	1.14	0.65	0.28	0.94	1.14
0.7	0.87	0.50	0.22	0.72	0.87	1.00	0.58	0.25	0.83	1.00
0.8	0.62	0.36	0.16	0.52	0.62	0.73	0.42	0.18	0.61	0.73
0.9	0.31	0.18	0.08	0.26	0.31	0.36	0.21	0.09	0.30	0.36
1.0 (Pier)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1.1	0.31	0.18	0.08	0.26	0.31	0.36	0.21	0.09	0.30	0.36
1.2	0.62	0.36	0.16	0.52	0.62	0.73	0.42	0.18	0.61	0.73
1.3	0.87	0.50	0.22	0.72	0.87	1.00	0.58	0.25	0.83	1.00
1.4	1.02	0.59	0.25	0.84	1.02	1.14	0.65	0.28	0.94	1.14
1.5	1.08	0.62	0.27	0.89	1.08	1.18	0.68	0.30	0.98	1.18
1.6	1.04	0.59	0.26	0.85	1.04	1.13	0.65	0.28	0.93	1.13
1.7	0.89	0.51	0.22	0.74	0.89	0.96	0.56	0.24	0.80	0.96
1.8	0.66	0.38	0.17	0.55	0.66	0.70	0.41	0.18	0.59	0.70
1.9	0.35	0.21	0.09	0.30	0.35	0.38	0.22	0.09	0.32	0.38
2.0 (Abut B)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

STEP 8. GIRDER DESIGN

The design of an exterior girder is demonstrated in the following paragraphs.

The girder elevation is shown in **Figure 4**. The design of the exterior girder from the abutment to the point of contraflexure is primarily based on positive bending moments; thus, this section of the girder is referred to as the "positive bending region" throughout this example. Alternatively, the girder geometry between the points of contraflexure encompassing the pier is controlled by negative bending moments: consequently, referred to as the "negative bending region".

All types of I-section flexural members shall be designed as a minimum to satisfy the following: LRFD Art. 6.10.1

- The cross-section proportion limits specified in Article 6.10.2.
- The strength limit state requirements specified in Article 6.10.6.
- The constructibility requirements specified in Article 6.10.3.
- The service limit state requirements specified in Article 6.10.4.
- The fatigue and fracture limit state requirements specified in Article 6.10.5.

Step 3.1 presents cross-section proportion limits. As per the calculations presented in **Step 3.2.1**, the selected exterior girder sections satisfy the cross-section proportion limit requirements.

Step 8.1. Design of Negative Bending Region

The girder section over the pier ($1.0L_{ds}$) is selected to demonstrate the design procedure for the negative bending region. Shear connectors are not provided within the negative bending region. Thus, a noncomposite section is used for the design.

Step 8.1.1. Strength limit state

Figure 33 shows the strength limit state design procedure of the girder section over the pier.

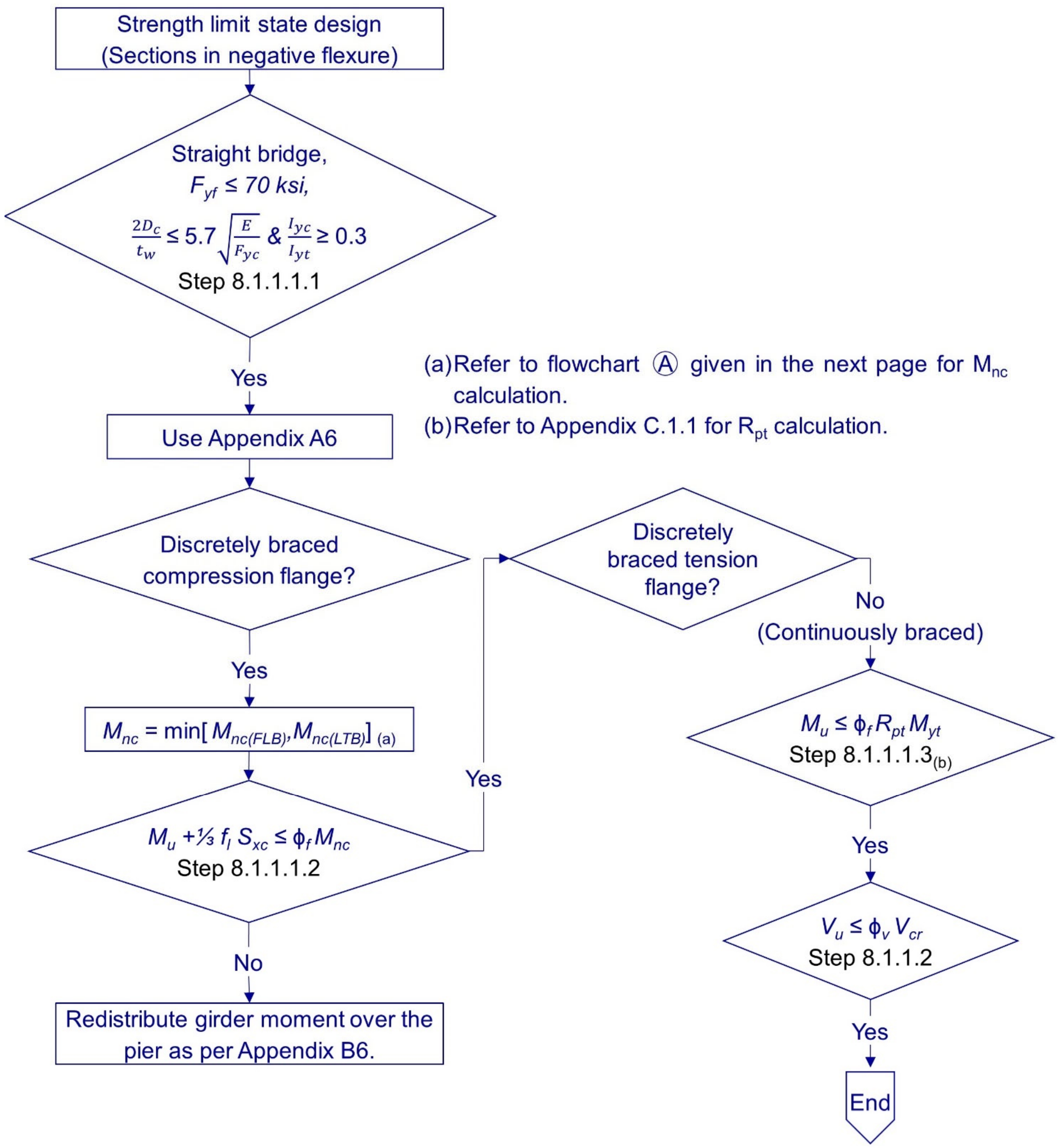


Figure 33. Strength limit state design of the girder section over the pier

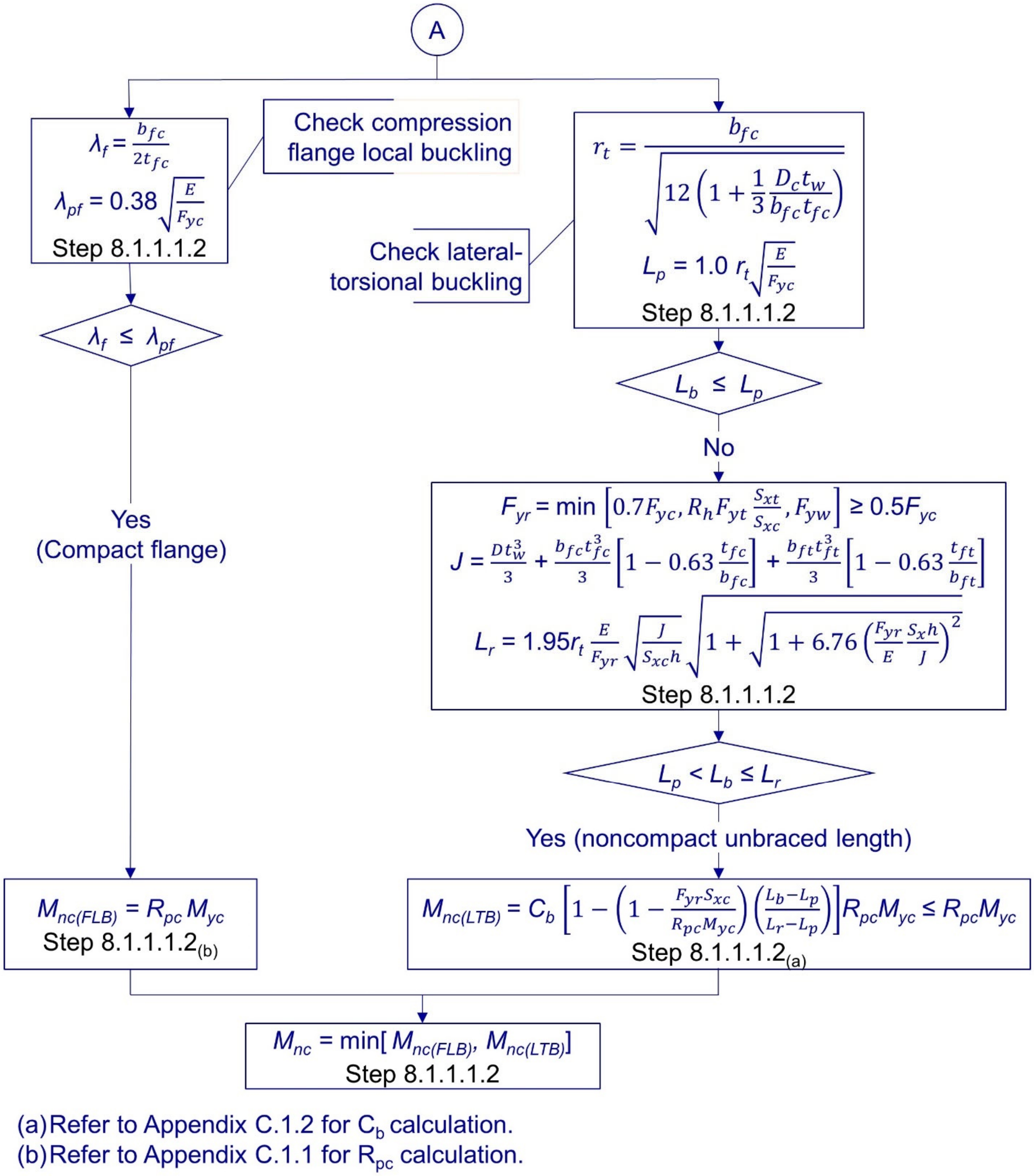


Figure 33. Strength limit state design of the girder section over the pier (contd.)

Step 8.1.1.1. Flexural resistance

The section is noncomposite over the pier. First, the specified steel grade requirements, LRFD Eq. 6.10.6.2.3-1 limits for web and LRFD Eq. 6.10.6.2.3-2 limits for flanges, are checked to identify the relevant procedures and equations for nominal flexural resistance calculation and the need for moment redistribution.

Step 8.1.1.1.1. Steel grade requirements and web and flange section limits

Sections in straight bridges whose supports are normal or skewed not more than 20 deg from normal, and with intermediate diaphragms or cross-frames placed in parallel to the supports, may be proportioned according to the provisions for compact or noncompact web sections specified in Appendix A6 if the following limits are satisfied. LRFD Art. 6.10.6.2.3

- The specified minimum yield strengths of the flanges,

$$F_{yf} = 50 \text{ ksi} < 70 \text{ ksi} \qquad \textbf{OK}$$

- Noncompact slenderness limit for the web:

$$\frac{2D_c}{t_w} \leq 5.7\sqrt{\frac{E}{F_{yc}}}$$

LRFD Eq. 6.10.6.2.3-1

where:

D_c = depth of the web in compression in the elastic range

E = modulus of elasticity of the section under consideration

F_{yc} = specified minimum yield strength of the compression flange

The section over the pier is noncomposite. The distance from the bottom of the girder to the elastic neutral axis, y_b, is 19.125 in. Subtracting the bottom flange thickness gives the web depth in compression in the elastic range (D_c).

$D_c = y_b$ – bottom flange thickness

$D_c = 19.125 - 2.625 = 16.50$ in.

$$\frac{2D_c}{t_w} = \frac{2 \times 16.50}{0.5} = 66 < 5.7\sqrt{\frac{E}{F_{yc}}} = 5.7\sqrt{\frac{29000}{50}} = 137.27 \qquad \textbf{OK}$$

- Limits for the flanges:

$$\frac{I_{yc}}{I_{yt}} \geq 0.3$$

LRFD Eq. 6.10.6.2.3-2

where:

I_{yc} = moment of inertia of the compression flange about the vertical axis

I_{yt} = moment of inertia of the tension flange about the vertical axis

At the negative bending region, the thickness and width of the top flange (tension flange) and the bottom flange (compression flange) are the same, i.e. t_f = 2.625 in. and b_f = 18 in. Therefore,

$$I_{yc} = I_{yt} = \frac{1}{12}(2.625)(18)^3 \qquad = 1275.75\ in.^4$$

$$\frac{I_{yc}}{I_{yt}} = 1 > 0.3 \qquad \textbf{OK}$$

Since all the requirements are satisfied, Appendix A6 is used to evaluate the girder section at the negative bending region under strength limit state. Otherwise, use provisions specified in Art. 6.10.8.

Step 8.1.1.1.2. Flexural resistance based on the compression flange

Since the compression flange is discretely braced over the pier, the flexural capacity of the compression flange must exceed the maximum negative moment plus one-third of the lateral bending stress due to the factored Strength I load multiplied by the compression flange section modulus: LRFD Art. A6.1.1

$$M_u + \frac{1}{3} f_l S_{xc} \leq \emptyset_f M_{nc}$$ LRFD Eq. A6.1.1-1

where:

ϕ_f = resistance factor for flexure = 1.0 (LRFD Art. 6.5.4.2)

f_l = flange lateral bending stress = 0 ksi (FHWA (2015))
The lateral bending stresses are negligible when the girders are straight and composite. Therefore, f_l = 0 ksi

M_{yc} = yield moment with respect to the compression flange = 6,871 kip-ft (**Appendix B.1.2**)

M_u = bending moment about the major axis of the section = -6,197 kip-ft (**Table 20**)

M_{nc} = nominal flexural resistance based on the compression flange

S_{xc} = elastic section modulus about the major axis of the section to the compression flange taken as M_{yc}/F_{yc}

Since the lateral bending stresses are zero at the Strength I limit state for the straight girders considered in this example, Eq. A6.1.1-1 reduces to

$$M_u \leq \emptyset_f M_{nc}$$

Calculation of nominal flexural resistance based on the compression flange, M_{nc}

The nominal flexural resistance based on the compression flange is taken as the minimum of the flange local buckling resistance (FLB) or the lateral-torsional buckling resistance (LTB). LRFD Art. A6.3.1

Nominal flange local buckling resistance, $M_{nc\,(FLB)}$:

To evaluate the nominal flange local buckling resistance, the flange slenderness classification is first determined. The flange is compact if $\lambda_f \leq \lambda_{pf}$. LRFD Art. A6.3.2

When $\lambda_f \leq \lambda_{pf}$,

$$M_{nc} = R_{pc}M_{yc}$$ LRFD Eq. A6.3.2-1

where:

λ_f = slenderness ratio for the compression flange

$= \dfrac{b_{fc}}{2t_{fc}}$ LRFD Eq. A6.3.2-3

λ_{pf} = limiting slenderness ratio for a compact flange

$= 0.38\sqrt{\dfrac{E}{F_{yc}}}$ LRFD Eq. A6.3.2-4

M_{yc} = yield moment with respect to the compression flange

= 6,871 kip-ft **Appendix B.1.2**

R_{pc} = web plastification factor for the compression flange = 1.103 **Appendix C.1.1**

Calculation of λ_f and λ_{pf}:

$$\lambda_f = \frac{18}{2 \times 2.625} = 3.429$$

$$\lambda_{pf} = 0.38\sqrt{\frac{29000}{50}} = 9.152$$

Since $\lambda_f < \lambda_{pf}$, the flange is compact.

Therefore, the nominal flange local buckling resistance is:

$M_{nc(FLB)} = M_{nc} = R_{pc}M_{yc}$ = 1.103 × 6871 = 7,579 kip-ft

$M_{nc(FLB)}$

Nominal lateral-torsional buckling resistance, $M_{nc\,(LTB)}$:

To evaluate the nominal lateral-torsional buckling resistance, the section compactness is first determined based on the unbraced length. The section is compact if $L_b \leq L_p$. LRFD Art. A6.3.3

When $L_b \leq L_p$,

$$M_{nc} = R_{pc}M_{yc}$$ LRFD Eq. A6.3.3-1

where:

L_b = unbraced length

L_p = limiting unbraced length to achieve the nominal flexural resistance of $R_{pc}M_{yc}$ under uniform bending

$$= 1.0 r_t \sqrt{\frac{E}{F_{yc}}}$$ LRFD Eq. A6.3.3-4

r_t = effective radius of gyration for lateral-torsional buckling

$$= \frac{b_{fc}}{\sqrt{12\left(1 + \frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\right)}}$$ LRFD Eq. A6.3.3-10

where:

b_{fc} = compression flange width

t_{fc} = compression flange thickness

When $L_p < L_b \leq L_r$,

$$M_{nc} = C_b\left[1 - \left(1 - \frac{F_{yr}S_{xc}}{R_{pc}M_{yc}}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right]R_{pc}M_{yc} \leq R_{pc}M_{yc}$$ LRFD Eq. A6.3.3-2

where:

C_b = moment gradient modifier

h = depth between the centerlines of the flanges

F_{yr} = compression-flange stress at the onset of nominal yielding within the cross-section, including residual stress effects, but not including compression flange lateral bending, taken as the smaller of $0.7F_{yc}$, $R_h F_{yt}\frac{S_{xt}}{S_{xc}}$, and F_{yw}, but not less than $0.5F_{yc}$

$$= \min\left(0.7F_{yc},\ R_h F_{yt}\frac{S_{xt}}{S_{xc}},\ F_{yw}\right) \geq 0.5F_{yc}$$ LRFD Art. A6.3.3

S_{xt} = elastic section modulus about the major axis of the section to the tension flange taken as M_{yt}/F_{yt}

F_{yt} = specified minimum yield strength of the tension flange

F_{yw} = specified minimum yield strength of the web

R_h = hybrid factor — LRFD Art. 6.10.1.10.1

L_r = limiting unbraced length to achieve the onset of nominal yielding in either flange under uniform bending with consideration of compression – flange residual stress effects

$$= 1.95 r_t \frac{E}{F_{yr}} \sqrt{\frac{J}{S_{xc}h}} \sqrt{1 + \sqrt{1 + 6.76\left(\frac{F_{yr}S_{xc}h}{EJ}\right)^2}}$$ LRFD Eq. A6.3.3-5

J = St. Venant torsional constant

$$= \frac{Dt_w^3}{3} + \frac{b_{fc}t_{fc}^3}{3}\left(1 - 0.63\frac{t_{fc}}{b_{fc}}\right) + \frac{b_{ft}t_{ft}^3}{3}\left(1 - 0.63\frac{t_{ft}}{b_{ft}}\right)$$ LRFD Eq. A6.3.3-9

Check if $L_b \leq L_p$:

L_b = 11 ft = 132 in.

Since the bottom flange movement over the pier is constrained by the bearings, L_b is taken as the distance from the pier to the nearest interior diaphragm, which is 11 ft.

$$r_t = \frac{18}{\sqrt{12\left(1 + \frac{1}{3}\frac{(16.5)(0.5)}{(18)(2.625)}\right)}} = 5.051 \text{ in.}$$

$$L_p = 1.0 \times 5.051\sqrt{\frac{29000}{50}} = 121.64 \text{ in.} < L_b = 132 \text{ in.}$$

The section is noncompact since $L_b > L_p$.

Check if $L_p < L_b \leq L_r$:

$0.7F_{yc}$ = 0.7 × 50 = 35 ksi

$S_{xc} = M_{yc}/F_{yc}$ = (6871 × 12)/50 = 1,649 in.^{3}

$S_{xt} = M_{yt}/F_{yt}$ = (6871 × 12)/50 = 1,649 in.^{3}

R_h = 1.0 LRFD Art. 6.10.1.10.1

$R_h F_{yt}\frac{S_{xt}}{S_{xc}}$ = (1.0) (50) (1649/1649) = 50 ksi

F_{yr} = min (35, 50, 50) = 35 ksi

h = 0.5 × 2.625 + 33 + 0.5 × 2.625 = 35.625 in.

$$J = \frac{(33)(0.5)^3}{3} + \frac{(18)(2.625)^3}{3}\left(1 - 0.63\frac{2.625}{18}\right) + \frac{(18)(2.625)^3}{3}\left(1 - 0.63\frac{2.625}{18}\right) = 198.5 \text{ in.}^4$$

$$L_r = 1.95 \times 5.051 \times \frac{29000}{35}\sqrt{\frac{198.5}{1649 \times 35.625}} \times \sqrt{1 + \sqrt{1 + 6.76\left(\frac{35 \times 1649 \times 35.625}{29000 \times 198.5}\right)^2}} = 729.5 \text{ in.}$$

Since L_p = 121.64 in. < L_b = 132 in. < L_r = 729.5 in., use LRFD Eq. A6.3.3-2 to calculate the nominal lateral-torsional buckling resistance.

C_b = 1.386 **Appendix C.1.2**

$$M_{nc(LTB)} = 1.386\left[1-\left(1-\frac{(35)(1649)}{(1.103)(6871\times 12)}\right)\left(\frac{132-121.64}{729.5-121.64}\right)\right](1.103)(6871)$$

= 10,439 kip-ft

Since $M_{nc(LTB)}$ = 10,439 kip-ft > $R_{pc}M_{yc}$ = (1.103)(6871) = 7,579 kip-ft

$M_{nc(LTB)}$ = 7,579 kip-ft

Flexural resistance of the compression flange, M_{nc}, shall be taken as the smaller resulting from local flange buckling and lateral-torsional buckling.

M_{nc} = minimum [$M_{nc(FLB)}$, $M_{nc(LTB)}$]
= minimum (7579, 7579) = 7,579 kip-ft

Thus, the moment resistance for the negative bending region of the girder,

$\emptyset_f M_{nc}$ = (1.0)(7579) = 7,579 kip-ft

$M_u = |-6{,}197|\ kip-ft < \emptyset_f M_{nc} = 7{,}579\ kip-ft$ **OK**

Since the Strength I factored moment over the pier is less than the moment resistance, redistribution of the moment is not required.

If the Strength I factored moment over the pier is greater than the moment resistance, moment redistribution shall be performed according to Appendix B6.4. The provisions of Art. B6.2.1 through B6.2.6 need to be satisfied for the section under consideration for the moments to be redistributed in accordance with Appendix B6. Specifically, the provisions may only be applied to straight continuous span I-section members whose bearing lines are not skewed more than 10 degrees from normal and along which there are no staggered (or discontinuous) cross-frames. LRFD Art. B6.2

Step 8.1.1.1.3. Flexural resistance based on the tension flange

The tension flange over the pier is continuously braced by the concrete deck at the strength limit state and must therefore satisfy: LRFD Art. A6.1.4

$$M_u \le \emptyset_f R_{pt} M_{yt}$$ LRFD Eq. A6.1.4-1.

where:

ϕ_f = resistance factor for flexure = 1.0 LRFD Art. 6.5.4.2

M_{yt} = yield moment with respect to the tension flange = 6,871 kip-ft **Appendix B.1.2**

R_{pt} = web plastification factor for the tension flange = 1.103 **Appendix C1.1**

$M_u = |-6197|\ kip - ft$

$< \emptyset_f R_{pt} M_{yt} = (1.0)(1.103)(6871) = 7{,}579\ kip - ft$ **OK**

Step 8.1.1.2. Shear resistance

The shear requirements at the strength limit state are expressed by: LRFD Art. 6.10.6.3 & 6.10.9

$V_u \leq \phi_v V_n$ LRFD Eq. 6.10.9.1-1

where:

ϕ_v = resistance factor for shear = 1.0 LRFD Art. 6.5.4.2

V_u = factored shear in the web at the section under consideration

V_n = $V_{cr} = CV_p$ LRFD Eq. 6.10.9.2-1

The nominal shear resistance of unstiffened webs, V_n, is defined by either shear-yielding or shear-buckling resistance, depending on the web slenderness ratio.

C = ratio of the shear-buckling resistance to the shear yield strength determined with the shear-buckling coefficient (k) of 5.0

V_{cr} = shear-yielding or shear-buckling resistance

V_p = plastic shear force

= $0.58F_{yw}Dt_w$ LRFD Eq. 6.10.9.2-2

= 0.58 (50) (33) (0.5) = 478.5 kips

The computation of C is based on the web slenderness classification.

If $\frac{D}{t_w} \leq 1.12\sqrt{\frac{Ek}{F_{yw}}}$, then C = 1.0 LRFD Eq. 6.10.9.3.2-4

If $1.12\sqrt{\frac{Ek}{F_{yw}}} < \frac{D}{t_w} \leq 1.40\sqrt{\frac{Ek}{F_{yw}}}$, then $C = \frac{1.12}{\frac{D}{t_w}}\sqrt{\frac{Ek}{F_{yw}}}$ LRFD Eq. 6.10.9.3.2-5

If $\frac{D}{t_w} > 1.40\sqrt{\frac{Ek}{F_{yw}}}$, then $C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2}\left(\frac{Ek}{F_{yw}}\right)$ LRFD Eq. 6.10.9.3.2-6

Since $\frac{D}{t_w} = \frac{33}{0.5} = 66 > 1.12\sqrt{\frac{29000\times5}{50}} = 60.3,$ $C \neq 1.0$

Since $1.12\sqrt{\frac{Ek}{F_{yw}}} = 60.3 < \frac{D}{t_w} = 66 < 1.40\sqrt{\frac{Ek}{F_{yw}}} = 1.40\sqrt{\frac{29000\times5}{50}} = 75.4,$

$$C = \frac{1.12}{\frac{D}{t_w}} \sqrt{\frac{Ek}{F_{yw}}} = \frac{1.12}{\frac{33}{0.5}} \sqrt{\frac{29000 \times 5}{50}} = 0.914$$

$V_n = CV_p = 0.914 \times 478.5$ = 437 kips

From **Table 20**, V_u = -335 kips

V_u is the factored shear under Strength I limit state in the web over the pier.

$V_u = |-335|$ kips $< \phi_v V_n = 1.0 \times 437 = 437$ kips **OK**

Therefore, the girder has adequate shear resistance.

Step 8.1.2. Constructibility Evaluation

Figure 34 shows the procedure for checking the adequacy of a girder section with discretely braced compression and tension flanges to support construction loads at both positive and negative bending regions. This step presents an overview of the calculation procedure for the exterior girder section over the pier ($1.0L_{ds}$). A detailed calculation procedure is presented in **Step 8.2.2.** for the exterior girder section at $0.40L_{ds}$.

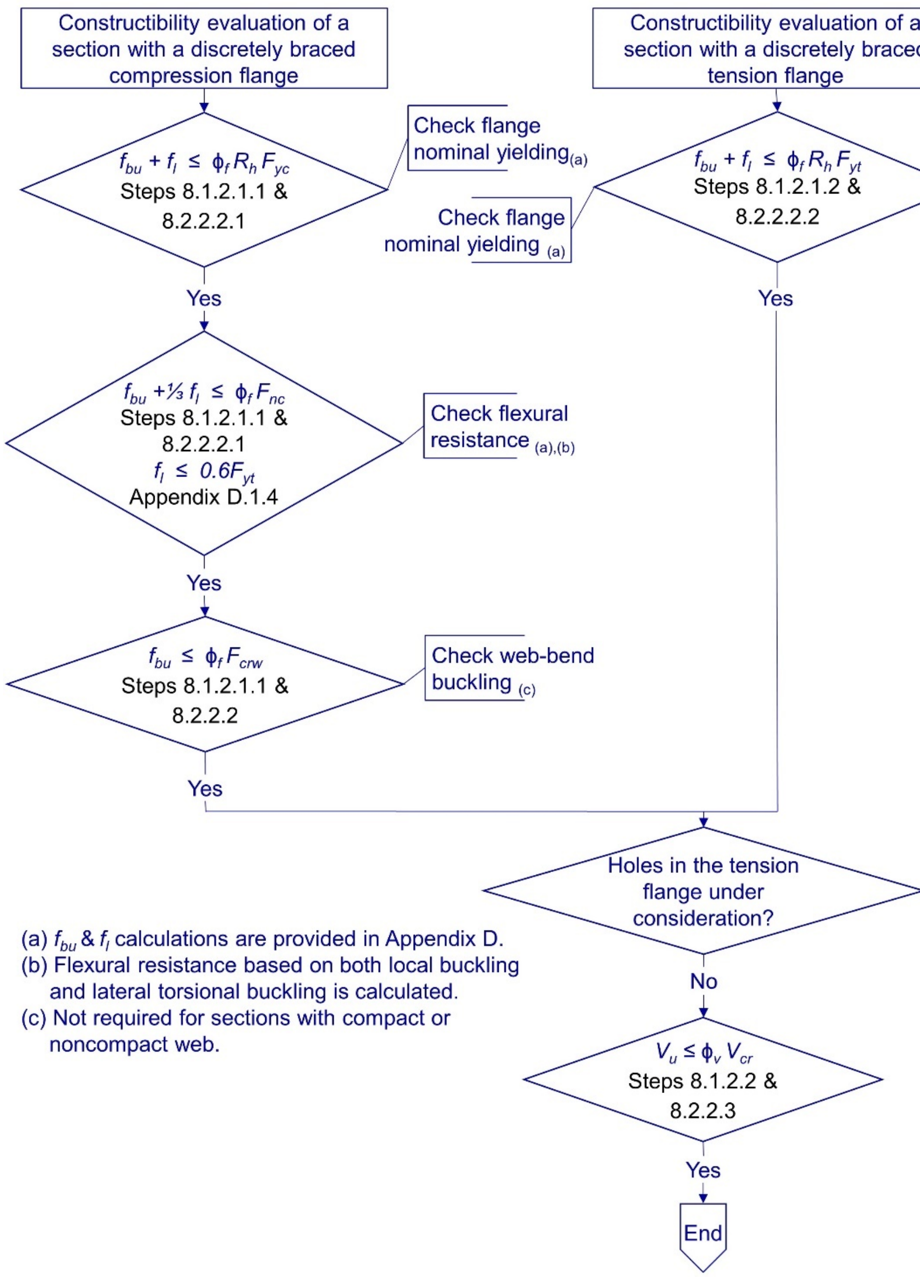


Figure 34. Procedure for checking the adequacy of a section with discretely braced compression and tension flanges to support construction loads

The engineer is required to design bridge systems to avoid construction difficulties and unacceptable locked-in forces. In addition, the main load-carrying members are not permitted to experience nominal yielding or reliance on post-buckling resistance during the construction phases. The sections must satisfy the requirements of Art. 6.10.3 at each construction stage. The applied loads to be considered are specified in Table 3.4.1-1, and the applicable load factors are provided in Article 3.4.2. The girders are considered to be noncomposite during the initial construction phase. The effects of forces from deck overhang brackets acting on the fascia girders are to be included in the constructibility checks. LRFD Art. 2.5.3 &. 6.10.3

Step 8.1.2.1. Flexural resistance

Within the negative bending region, the bottom flange is subjected to compression while the top flange is under tension. Both flanges are discretely braced during construction until the concrete deck is hardened or is made composite. LRFD Art. 6.10.3.2

Step 8.1.2.1.1. Compression flange

For critical stages of construction, each of the following requirements shall be satisfied.

$$f_{bu} + f_l \leq \phi_f R_h F_{yc}$$ LRFD Eq. 6.10.3.2.1-1

$$f_{bu} + \frac{1}{3} f_l \leq \phi_f F_{nc}$$ LRFD Eq. 6.10.3.2.1-2

where:

ϕ_f = resistance factor for flexure — LRFD Art. 6.5.4.2

f_{bu} = flange stress calculated without consideration of flange lateral bending — LRFD Art. 6.10.1.6

f_ℓ = flange lateral bending stress — LRFD Art. 6.10.1.6

F_{nc} = nominal flexural resistance of the flange (The calculation is similar to that at the positive bending region.)

F_{yc} = specified minimum yield strength of the compression flange

M_{yc} = yield moment with respect to the compression flange — LRFD Art. D6.2

R_h = hybrid factor — LRFD Art. 6.10.1.10.1

S_{xc} = elastic section modulus about the major axis of the section to the compression flange taken as M_{yc}/F_{yc}

As seen in **Step 8.1.1.2**, the web is noncompact over the pier. Therefore, Eq. 6.10.3.2.1-3 is not checked for web-bend buckling. LRFD Art. 6.10.3.2.1

Step 8.1.2.1.2. Tension flanges

For critical stages of construction:

$$f_{bu} + f_l \leq \phi_f R_h F_{yt}$$

LRFD Eq. 6.10.3.2.2-1

Regions of negative flexure, Eqs. 6.10.3.2.1-1, 6.10.3.2.1-2 and 6.10.3.2.2-1 specified in Art. 6.10.3.2 (which are to be checked for critical stages of construction) generally do not control because the sizes of the flanges in these regions are normally governed by the design checks at the strength limit state. Also, the maximum accumulated negative moments from the deck-placement analysis in these regions, plus the negative moments due to the steel weight, typically do not differ significantly from (or may be smaller than) the calculated dead load negative moments ignoring the effects of the sequential deck placement. The deck-overhang loads do introduce lateral bending stresses into the flanges in these regions, which can be calculated and used to check the above equations in a manner similar to that illustrated later on in this example for positive bending regions. Wind load, when considered for the construction case, also introduces lateral bending. FHWA (2015)

If there are holes in the tension flanges at the section under consideration, the tension flange shall satisfy the requirements specified in Art. 6.10.1.8. LRFD Art. 6.10.3.1.

Step 8.1.2.2. Shear resistance

Webs shall satisfy the following factors during critical stages of construction: LRFD Art. 6.10.3.3

$$V_u \leq \phi_v V_{cr}$$

LRFD Eq. 6.10.3.3-1

where:

V_u = shear in the web at the section under consideration due to factored permanent loads and factored construction loads applied to the noncomposite section

V_{cr} = shear-yielding or shear-buckling resistance

In **Step 8.1.1.2**, the unstiffened shear capacity of the girder is demonstrated to be sufficient to resist the applied shear at the strength limit state. Therefore, the section will have sufficient shear capacity to support construction loads.

Step 8.1.3. Service limit state

Figure 35 shows the service limit state design procedure implemented in this example.

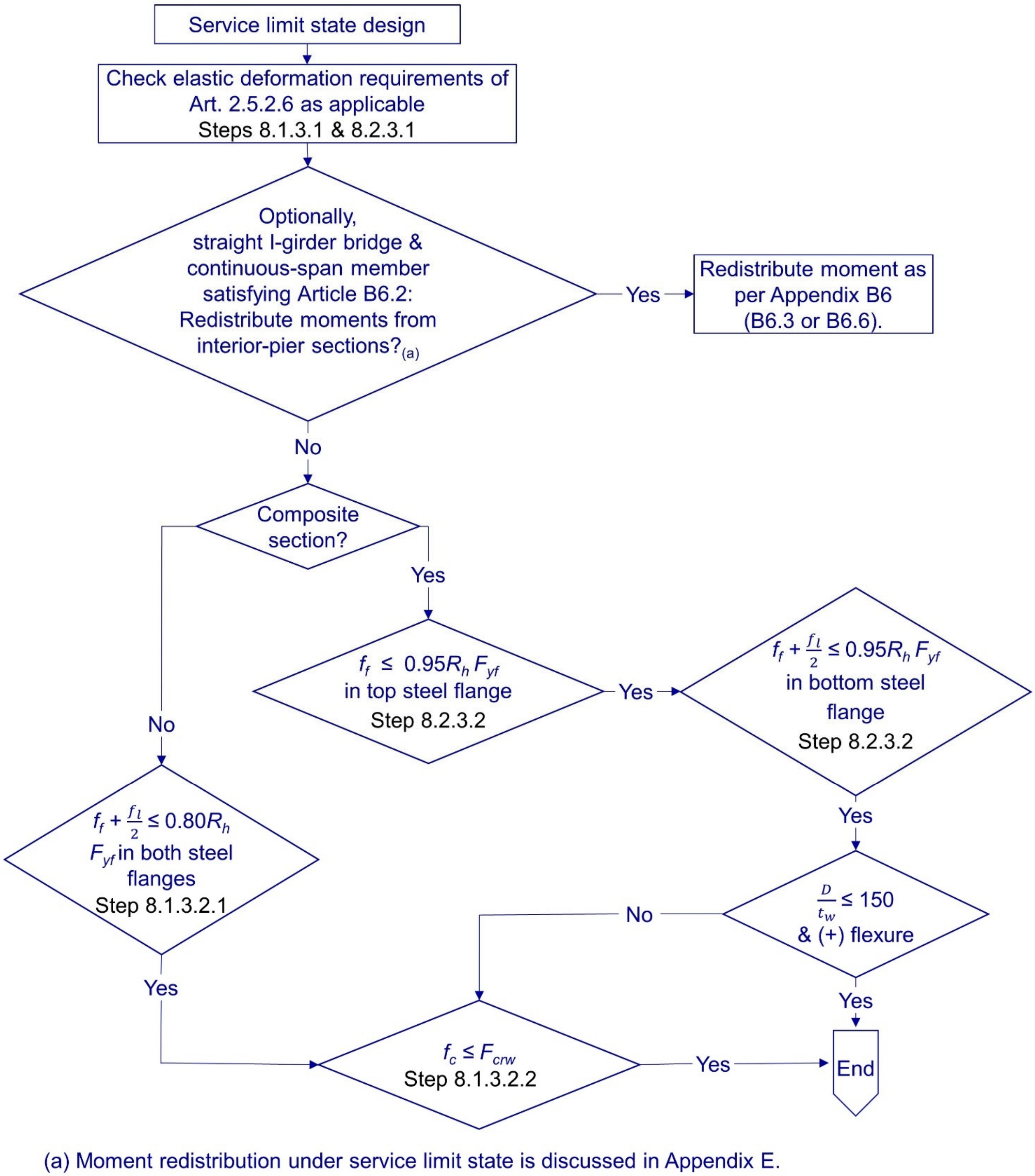


Figure 35. Service limit state design procedure

Step 8.1.3.1. Elastic deformations

Bridges should be designed to avoid undesirable structural or aesthetic effects due to their deformations. While deflection and depth limitations are made optional, any large deviation from past successful practice regarding slenderness and deflection should be cause for review of the design to determine that it will perform adequately. LRFD Art. 2.5.2.6.1

The elastic deformation is checked only at the positive bending region of the exterior girder.

Step 8.1.3.2. Permanent deformations

Since shear studs are not provided within the negative moment region, noncomposite section properties are used for calculation of the flexural stresses in the structural steel. LRFD Art. 6.10.4.2.1

Step 8.1.3.2.1. Flanges

For both steel flanges of noncomposite sections:

$$f_f + \frac{f_l}{2} \leq 0.80 R_h F_{yf}$$ LRFD Eq. 6.10.4.2.2-3

where:

f_f = flange stresses at the section under consideration due to Service II loads calculated without consideration of flange lateral bending

= -33.60 ksi for top flange **Table 22**

= 33.60 ksi for bottom flange **Table 22**

Note: Moment redistribution is not required over the pier. **Appendix E**

f_l = flange lateral bending stress at the section under consideration due to Service II loads = 0 ksi LRFD Art. C6.10.4.2.2

R_h = hybrid factor = 1.0 LRFD Art. 6.10.1.10.1

Applying LRFD Eq. 6.10.4.2.2-3 for both flanges:

For the top flange:

$$f_f + \frac{f_l}{2} = |-33.60| + 0 = 33.60\ ksi$$

$$< 0.80 R_h F_{yf} = 0.80(1.0)(50) = 40\ ksi$$ **OK**

For the bottom flange:

$$f_f + \frac{f_l}{2} = 33.60 + 0 = 33.60\ ksi$$

$$< 0.80 R_h F_{yf} = 0.80(1.0)(50) = 40\ ksi$$ **OK**

Step 8.1.3.2.2. Web

In addition, the compression flange stress at service loads is limited to the elastic bend-buckling resistance of the web by: LRFD Art. 6.4.4.2.2

$f_c \leq F_{crw}$ LRFD Eq. 6.10.4.2.2-4

where:

f_c = compression flange stress at the section under consideration due to the Service II loads calculated without consideration of flange lateral bending

= 33.60 ksi **Table 22**

F_{crw} = nominal bend-buckling resistance for webs with or without longitudinal stiffeners

For webs without longitudinal stiffeners: LRFD Art. 6.10.1.9.1

$$F_{crw} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} \leq min\left\{R_h F_{yc}, \frac{F_{yw}}{0.7}\right\}$$ LRFD Eq. 6.10.1.9.1-1

where:

D_c = depth of the web in compression in the elastic region

= 16.5 in. **Step 8.1.1.1.1**

k = bend-buckling coefficient

$= \frac{9}{(D_c/D)^2}$ LRFD Eq. 6.10.1.9.1-2

$k = \frac{9}{(16.5/33)^2}$ = 36

$F_{crw} = \frac{0.9 \times 29000 \times 36}{\left(\frac{33}{0.5}\right)^2}$ = 216 ksi

$min\left\{R_h F_{yc}, \frac{F_{yw}}{0.7}\right\} = min\left\{1.0 \times 50, \frac{50}{0.7}\right\}$ = 50 ksi

Therefore, F_{crw} = 50 ksi

f_c = 33.60 ksi < F_{crw} = 50 ksi **OK**

Step 8.1.4. Fatigue limit state

Figure 36 shows the fatigue limit state design procedure implemented in this example.

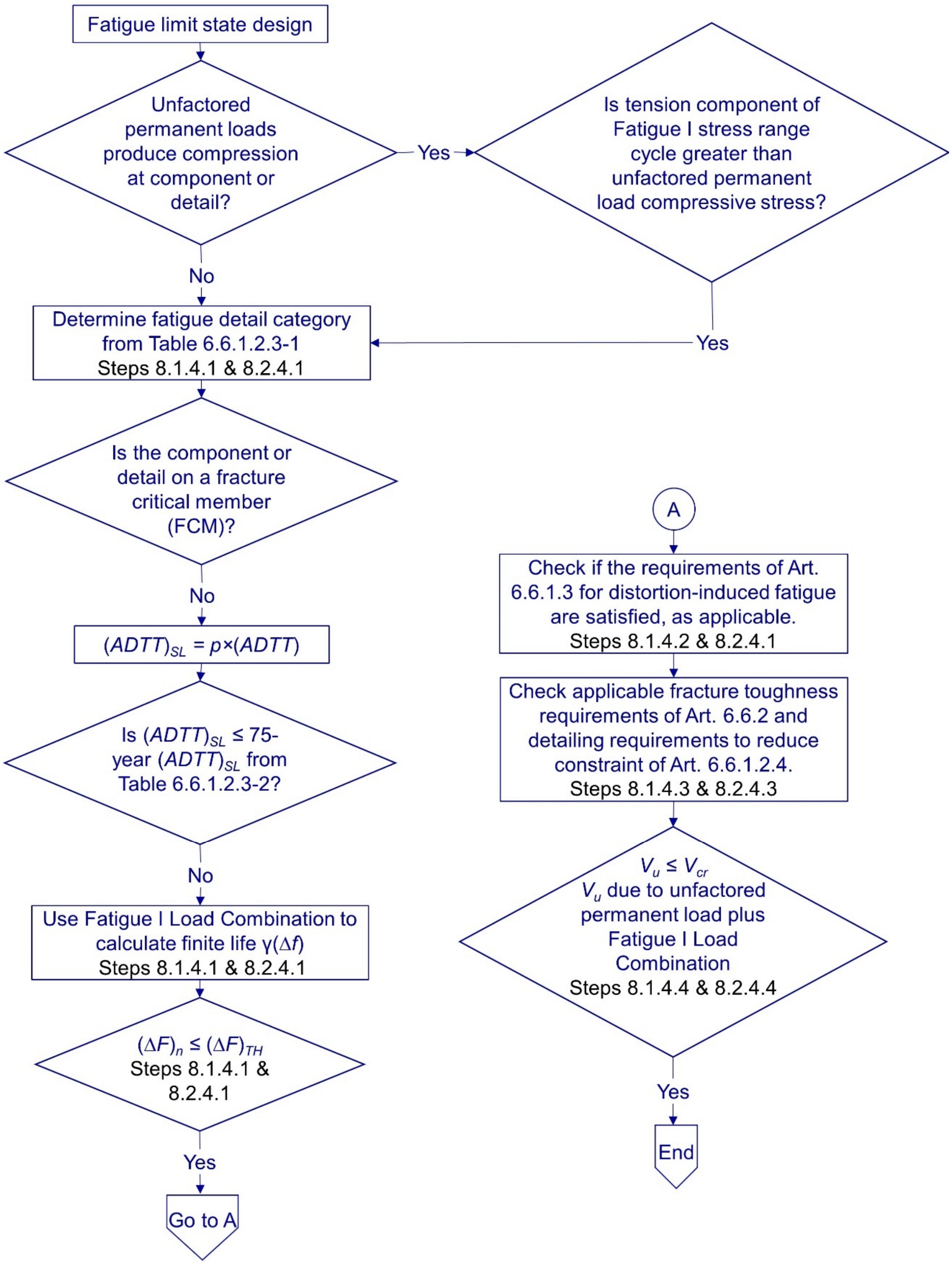


Figure 36. Fatigue limit state design procedure

The fatigue and fracture limit state incorporates three distinctive checks: fatigue resistance of details (Art. 6.10.5.1), fracture toughness (Art. 6.10.5.2), and a special fatigue requirement for webs (Art. 6.10.5.3). The first requirement involves the assessment of the fatigue resistance of details as specified in Art. 6.6.1 using the appropriate fatigue load combination specified in Table 3.4.1-1 and the fatigue live load specified in Art. 3.6.1.4. The fracture toughness requirements in Art. 6.10.5.2 specify that the fracture toughness must satisfy the requirements of Art. 6.6.2. The special fatigue requirement for the web controls the elastic flexing of the web to prevent fatigue cracking. LRFD Art. 6.10.5

Fatigue and fracture resistance need to be evaluated mainly in connection components in the bridge such as flange to web connection welds, stiffener welds, shear stud welds, connection plate welds, etc.

Step 8.1.4.1. Load induced fatigue

These provisions shall be applied only to details subjected to a net applied tensile stress. In regions where the unfactored permanent loads produce compression, fatigue shall be considered only if the compressive stress is less than the maximum live load tensile stress caused by the Fatigue I load combination. LRFD Art. 6.6.1.2

For load-induced fatigue considerations, each detail shall satisfy: LRFD Art. 6.6.1.2.2

$$\gamma(\Delta f) \le (\Delta F)_n$$ LRFD Eq, 6.6.1.2.2-1

where:

- γ = load factor for fatigue load combination in Table 3.4.1-1
- (Δf) = force effect, live load stress range due to the passage of the fatigue load
- $(\Delta F)_n$ = nominal fatigue resistance as specified in Art. 6.6.1.2.5.

For Fatigue I load combination and finite fatigue life:

$$(\Delta F)_n = (\Delta F)_{TH}$$ LRFD Eq. 6.6.1.2.5-1

For Fatigue II load combination and finite fatigue life:

$$(\Delta F)_n = (A/N)^{1/3}$$ LRFD Eq. 6.6.1.2.5-2

in which:

$$N = (365)(75)n(ADTT)_{SL}$$ LRFD Eq. 6.6.1.2.5-3

where:

A	=	constant taken from Table 6.6.1.2.5-1
$(ADTT)_{SL}$	=	single-lane ADTT as specified in LRFD Art. 3.6.1.4
$(\Delta F)_{TH}$	=	constant-amplitude fatigue threshold taken from Table 6.6.1.2.5-3.
n	=	number of stress range cycles per truck passage taken from Table 6.6.1.2.5-2

The single lane ADTT value, TRKS in BDS, if not input by the user, defaults to a value of 3,000 trucks per day. This assumed value is based upon statements made in C3.6.1.4.2 of the AASHTO LRFD specifications. Research shows that the average daily traffic (ADT) is physically limited to about 20,000 cars/day per lane and that ADTT can be found by multiplying by the appropriate fractional value found in Table C3.6.1.4.2-1. The BDS is using 0.15 to define the fraction of trucks in traffic in urban, interstate, and other rural highways.

MDOT considers this assumption to be appropriate, and it should be documented that this value of 3,000 trucks per day is considered to be per lane (i.e. $(ADTT)_{SL}$ = 3,000 trucks).

Except as specified herein for fracture critical members, where the projected 75-year single lane Average Daily Truck Traffic $(ADTT)_{SL}$ is less than or equal to that specified in Table 6.6.1.2.3-2 for the component or detail under consideration, that component or detail should be designed for finite life using the Fatigue II load combination specified in Table 3.4.1-1. Otherwise, the component or detail shall be designed for infinite life using the Fatigue I load combination. The single-lane Average Daily Truck Traffic $(ADTT)_{SL}$ shall be computed as specified in Article 3.6.1.4.2. LRFD Art. 6.6.1.2.2

The fatigue resistance of the base metal at the weld joining the intermediate diaphragm connection plate located at 11 feet to the left of the pier to the girder flanges is evaluated.

Fatigue detail = Category C′ LRFD Table 6.6.1.2.3-1

This is the welded stiffener connection 4.1 in LRFD Table 6.6.1.2.3-1.

For detail category C′: 75-yr $(ADTT)_{SL}$ = 975 trucks/day LRFD Table 6.6.1.2.3-2

Since this is less than the $(ADTT)_{SL}$ specified for the bridge (i.e. 3000 trucks/day per lane), use the Fatigue I load combination.

For category C′: $(\Delta F)_{TH}$ = 12 ksi LRFD Table 6.6.1.2.3-1

For Fatigue I load combination, $(\Delta F)_n = (\Delta F)_{TH}$ = 12 ksi

The applied stress range, (Δf), is taken as the result of the fatigue loading with a dynamic load allowance of 15% applied and distributed laterally by the previously calculated distribution factor for fatigue.

Loads at $0.8L_{ds}$ and $0.9L_{ds}$ are calculated during analysis. Moments due to dead loads on the noncomposite section's (DC_1), barrier weight (DC_2), and fatigue load at $0.8L_{ds}$ and $0.9L_{ds}$ are presented in **Table 10** and **Table 14.** Linear interpolation is used to calculate the moments at 11 ft to the left of the pier (i.e. 87.67 ft from the abutment bearing centerline) as follows:

Distance from the abutment bearing CL (ft)	DC_1 moment (kip-ft)	DC_2 moment (kip-ft)	Fatigue moment (+) (kip-ft)	Fatigue moment (-) (kip-ft)
78.936 ($0.8L_{ds}$)	-516	-26	310	-297
87.67	**-1045**	**-81**	**139**	**-330**
88.803 ($0.9L_{ds}$)	-1113	-88	117	-335

At 11 ft to the left of the pier (i.e., at 87.67 ft):

I_{nc} = 31535 in.^{4} **Table 3**

Distance to the top of the bottom flange = 19.125 – 2.625 = 16.50 in.

Distance to the bottom of the top flange = 19.125 – 2.625 = 16.50 in.

Note: Distance from the centroid to the girder top = 19.125 in.

Distance from the centroid to the girder bottom = 19.125 in.

At the section under consideration, the unfactored permanent loads produce tension at the top of the girder and compression at the bottom of the girder. In this example, the future wearing surface is not considered for the evaluation of net applied tensile stress.

At the top of the bottom flange:

Compressive stress produced by DC_1 and DC_2 loads are:

$$f_{DC1} = \frac{|-1045| \times 12}{(31535/16.50)} = 6.56 \text{ ksi (compression)}$$

$$f_{DC2} = \frac{|-81| \times 12}{(31535/16.50)} = 0.51 \text{ ksi (compression)}$$

$$\Sigma = 6.56 + 0.51 = 7.07 \text{ ksi (compression)}$$

Factored tensile stress produced by the fatigue load is:

$$f_{LL+IM} = 1.75 \times \frac{|139| \times 12}{(31535/16.50)} = 1.53 \text{ ksi (tension)}$$

$$7.07\ ksi > 1.53\ ksi$$

Therefore, the detail is subjected to a net compressive stress at the top of the bottom flange. Hence, a load induced fatigue check is not required. LRFD Art. 6.6.1.2

At the bottom of the top flange:

Since the unfactored permanent loads produce tensile stress at the bottom of the top flange, the detail is subjected to a net tensile stress. Therefore, a fatigue check is performed using the stress range due to fatigue load. LRFD Art. 6.6.1.2

$$\Delta f = \frac{(139 + |-330|) \times 12}{(31535/16.50)} = 2.94 \text{ ksi}$$

$$\gamma(\Delta f) = (1.75)(2.94) = 5.15 \text{ ksi}$$

$$\gamma(\Delta f) = 5.15\ ksi < (\Delta F)_n = 12\ ksi$$ **OK**

Step 8.1.4.2. Distortion induced fatigue

LRFD Art. 6.6.1.3

A positive connection is to be provided for all transverse connection-plate details to both the top and bottom flanges to prevent distortion induced fatigue. FHWA (2015)

Step 8.1.4.3. Fracture

LRFD Art. 6.6.2

The appropriate Charpy V-notch fracture toughness, found in Table 6.6.2-2, must be specified for main load-carrying components subjected to tensile stress under the Strength I load combination. FHWA (2015)

Step 8.1.4.4. Special fatigue requirement for webs

If the following condition is satisfied, significant elastic flexing of the web due to shear is not expected to occur, and the member is assumed to be able to sustain an infinite number of smaller loadings without fatigue cracking due to this effect. LRFD Art. C6.10.5.3

$$V_u \leq V_{cr}$$ LRFD Eq. 6.10.5.3-1

where:

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

V_{cr} = shear-yielding or shear-buckling resistance determined from LRFD Eq. 6.10.9.3.3-1

However, designs utilizing unstiffened webs at the strength limit state, as is the case here, automatically satisfy this criterion. Thus, this criterion is not explicitly evaluated herein.

Step 8.2. Design of Positive Bending Region

Step 8.2.1. Strength limit state

Figure 37 shows the strength limit state design procedure implemented in this example for the positive bending region.

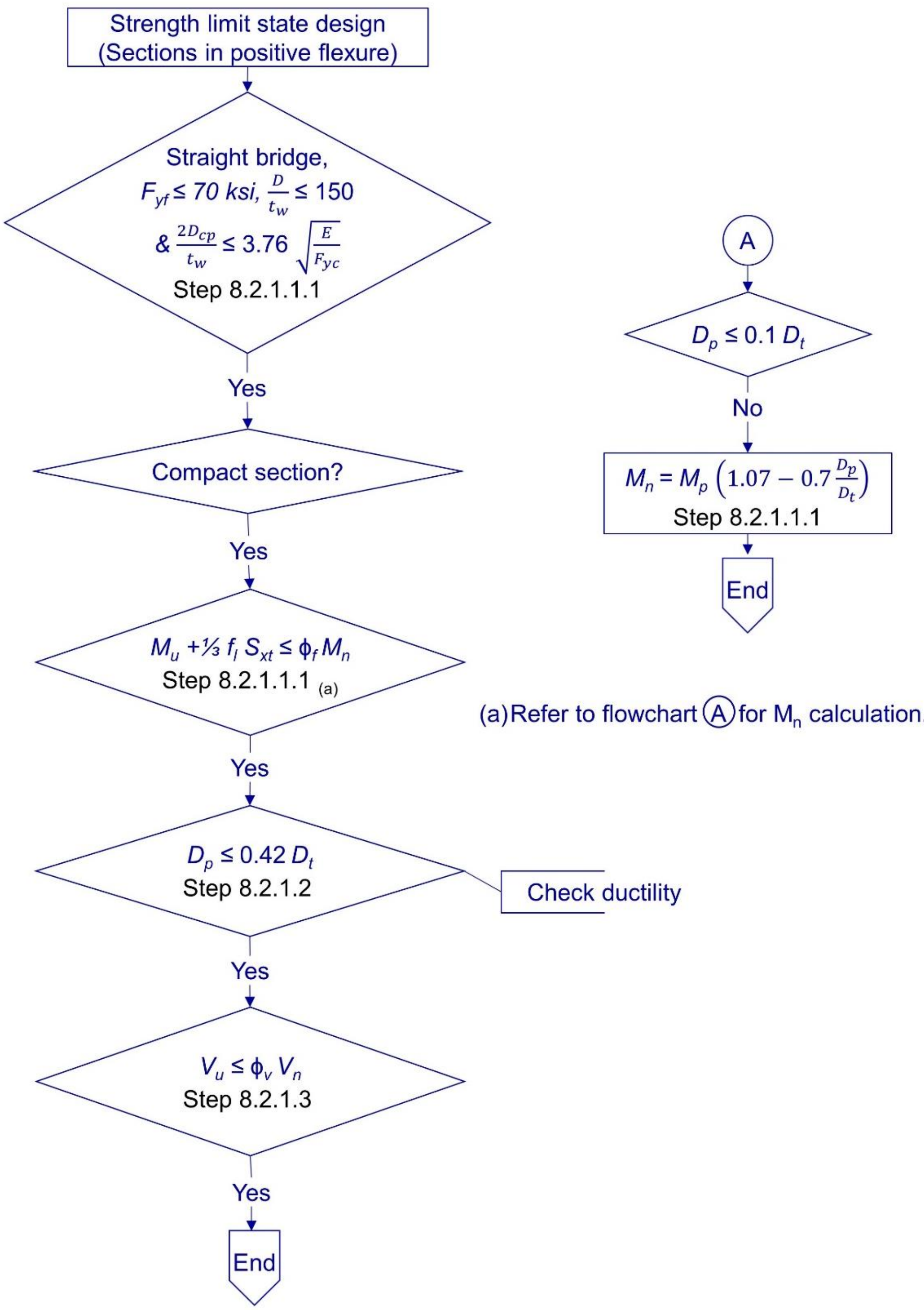


Figure 37. Strength limit state design procedure for the positive bending region

Step 8.2.1.1. Flexural resistance

Step 8.2.1.1.1. Evaluation of section compactness

Composite sections in straight bridges that satisfy the following requirements shall qualify as compact composite sections: LRFD Art. 6.10.6.2.2

- The specified minimum yield strengths of the flanges

F_{yf} = 50 ksi < 70 ksi **OK**

- The web satisfies the requirement

$$\frac{D}{t_w} = \frac{33}{0.5} = 66 < 150$$ LRFD Eq. 6.10.2.1.1-1

- The section satisfies the web slenderness limit

$$\frac{2D_{cp}}{t_w} \leq 3.76\sqrt{\frac{E}{F_{yc}}}$$ LRFD Eq. 6.10.6.2.2-1

where:

D_{cp} = depth of the web in compression at the plastic moment determined as specified in Art. D6.3.2

For all other composite sections in positive flexure except the case where the plastic neutral axis is in the web: D_{cp} shall be taken equal to zero. LRFD Art. D6.3.2

Since the plastic neutral axis (PNA) of this section is located within the girder top flange (PNA is located at 0.1 in. below the top of the girder top flange as calculated in **Appendix B.2.1**), D_{cp} = 0 in.

$$\frac{2 \times 0}{0.5} = 0 < 3.76\sqrt{\frac{E}{F_{yc}}}$$ **OK**

Therefore, the section is compact under positive flexure, and the nominal flexural resistance is evaluated as per Art. 6.10.7.1.

For compact sections in positive bending: LRFD Art. 6.10.7.1

$$M_u + \frac{1}{3}f_l S_{xt} \leq \phi_f M_n$$ LRFD Eq. 6.10.7.1.1-1

where:

ϕ_f = resistance factor for flexure = 1.0 LRFD Art. 6.5.4.2

f_l = flange lateral bending stress

Since the lateral bending stresses are negligible for the straight, composite girder considered herein, f_l = 0 ksi FHWA (2015)

F_{yt} = specified minimum yield strength of the tension flange

M_{yt} = yield moment with respect to the tension flange

M_n = nominal flexural resistance

S_{xt} = elastic section modulus about the major axis of the section to the tension flange

Calculation of nominal flexural resistance, M_n

If $D_p \leq 0.1D_t$, — LRFD Art. 6.10.7.1.2

$$M_n = M_p$$ — LRFD Eq. 6.10.7.1.2-1

Otherwise,

$$M_n = M_p\left(1.07 - 0.7\frac{D_p}{D_t}\right)$$ — LRFD Eq. 6.10.7.1.2-2

where:

D_p = distance from the top of the concrete deck to the PNA
= 9 + 1 + 0.1= 10.1 in.
(The PNA is at 0.1 in. below the top of the compression flange). — **Appendix B.2.1**

D_t = total depth of the composite section = 45.375 in.

M_p = plastic moment = 6,694 kip-ft — **Appendix B.2.1**

D_p = 10.1 in. > $0.1D_t$ = 0.1 × 45.375 = 4.5375 in.

$$M_n = 6694\left(1.07 - 0.7\frac{10.1}{45.375}\right) = 6{,}120 \text{ kip-ft}$$

In this continuous span bridge, M_n of the section shall satisfy: — LRFD Art. 6.10.7.1.2

$$M_n \leq 1.3R_hM_y$$ — LRFD Eq. 6.10.7.1.2-3

where:

M_y = yield moment = 5,266 kip-ft — **Appendix B2.2**

R_h = hybrid factor = 1.0 — LRFD Art. 6.1.1.10.1

unless the following two criteria are met:

- The span under consideration and all adjacent interior-pier sections satisfy the requirements of Art. B6.2, and — **OK** — **Appendix C.1.3**
- The appropriate value of θ_{RL} from Art. B6.6.2 exceeds 0.009 radians at all adjacent interior-pier sections.

where:

θ_{RL} = plastic rotation at which the moment at an interior-pier section nominally begins to decrease with increasing plastic rotation (θ_p) at an interior-pier section

θ_{RL} = 0.0603 rad > 0.009 rad **OK** **Appendix C.1.3**

Therefore, the section is not under the limitation of Eq. 6.10.7.1.2-3.

From elastic analysis procedures, the maximum positive moment under the Strength I load combination is 4,655 kip-ft (see **Table 20**), which is at a distance of 39.468 ft (0.4L_{ds}) from the left support.

If moment redistribution was performed at the interior-pier sections under the Strength I limit state, the redistributed moment must then be added to the moment at the section under consideration to determine the design moment.

$$M_u + \frac{1}{3} f_l S_{xt} = 4655 + 0 = 4{,}655\ kip - ft$$

$$< \emptyset_f M_n = (1.0)(6120) = 6{,}120\ kip - ft$$ **OK**

Therefore, the flexural resistance is satisfied at the maximum positive bending region (0.4L_{ds}) under the Strength I limit state.

Step 8.2.1.2. Ductility requirements

Ductility requirements are intended to protect the concrete deck from premature crushing. The D_p/D_t ratio is lowered to 0.42 to ensure significant yielding of the bottom flange when the crushing stain is reached at the top of concrete deck for all potential cases. In checking this requirement, D_t should be computed using a lower bound estimate of the actual thickness of the concrete haunch or may be determined conservatively by neglecting the thickness of the haunch. LRFD Art. C6.10.7.3

Compact and noncompact sections shall satisfy: LRFD Art. 6.10.7.3

$$D_p \leq 0.42 D_t$$ LRFD Eq. 6.10.7.3-1

where:

D_p = 10.1 in. **Step 8.2.1.1.1**

D_t = 45.375 in. **Step 8.2.1.1.1**

D_p = 10.1 in. < 0.42D_t = 0.42 × 45.375 = 19.06 in. **OK**

The ductility requirements are satisfied by the section at the maximum positive bending region (0.4L_{ds}) under the Strength I limit state.

Step 8.2.1.3. Shear resistance

The shear requirement at the Strength limit state is expressed by: LRFD Art. 6.10.9.1

$$V_u \le \phi_v V_n$$ LRFD Eq. 6.10.9.1-1

where:

ϕ_v = resistance factor for shear = 1.0 LRFD Art. 6.5.4.2

V_u = factored shear in the web at the section under consideration

V_n = nominal shear resistance

The nominal shear resistance of unstiffened webs shall be taken as shear yielding or shear buckling resistance as follows: LRFD Art. 6.10.9.2

$$V_n = V_{cr} = CV_p$$ LRFD Eq. 6.10.9.2-1

where:

C = ratio of the shear-buckling resistance to the shear-yield strength determined with the shear-buckling coefficient (k) of 5.0

V_{cr} = shear-yielding or shear-buckling resistance

V_p = plastic shear force

= $0.58F_{yw}Dt_w$ LRFD Eq. 6.10.9.2-2

= 0.58(50) (33) (0.5) = 478.5 kips

The computation of C is based on the web slenderness classification and is similar to the procedure used for the negative bending region.

Since $1.12\sqrt{\frac{Ek}{F_{yw}}} = 60.3 < \frac{D}{t_w} = 66 < 1.40\sqrt{\frac{Ek}{F_{yw}}} = 75.4$

$$C = \frac{1.12}{\frac{D}{t_w}}\sqrt{\frac{Ek}{F_{yw}}} = \frac{1.12}{\frac{33}{0.5}}\sqrt{\frac{29000 \times 5}{50}} = 0.914$$

$$V_n = CV_p = 0.914 \times 478.5 = 437 \text{ kips}$$

V_u is the factored shear in the web at $0.4L_{ds}$ from abutments.

From **Table 20,** V_u = -84 kips

$V_u = |-84|$ kips < $\phi_v V_n = 1.0 \times 437 = 437$ kips **OK**

Therefore, the shear resistance is adequate at the maximum positive bending location ($0.4L_{ds}$).

Step 8.2.2. Constructibility Evaluation

LRFD Art. 6.10.3

Figure 34 shows the procedure for checking the adequacy of an exterior girder section with discretely braced compression and tension flanges to support construction loads.

Step 8.2.2.1. Deck placement and deck overhang analysis

See **Appendix D.1.1** and **Appendix D.1.2.** for the deck placement and deck overhang analysis at the maximum positive bending region ($0.4L_{ds}$).

Analysis is performed to identify the maximum moment acting on the noncomposite section. The design calculations shown in this example at $0.4L_{ds}$ use a total construction moment of 665 kip-ft resulting from the concrete placement on the entire bridge deck in a single day. **Appendix D.1.1**

Step 8.2.2.2. Flexural resistance

Under the selected construction moment, both the compression and tension flanges are discretely braced. Therefore, the noncomposite section should satisfy Eqs. 6.10.3.2.1-1, 6.10.3.2.1-2, and 6.10.3.2.1-3, which ensure the flange stress is limited to the yield stress, the section has sufficient strength under the flange local buckling and lateral-torsional buckling limit states, and web bend-buckling does not occur during construction. LRFD Art. 6.10.3.2

First, determine if the noncomposite section satisfies the noncompact web slenderness limit as follows: LRFD Art. 6.10.6.2.3

$$\frac{2D_c}{t_w} \leq 5.7\sqrt{\frac{E}{F_{yc}}}$$

LRFD Eq. 6.10.6.2.3-1

where:

D_c = depth of the web in compression in the elastic range
= $y_t - t_f = 21.95 - 0.75 = 21.20$ in.

Note: the distance from the top of the girder to the neutral axis, $y_t = 21.95$ in. **Step 3.2.2.1**

$$\frac{2D_c}{t_w} = \frac{2 \times 21.2}{0.5} = 84.8 < 5.7\sqrt{\frac{29000}{50}} = 137.27 \qquad \textbf{OK}$$

The section is non-slender (i.e. the section has a compact or noncompact web). Therefore, the web bend-buckling evaluation using Eq. 6.10.3.2.1-3 is not required.

Step 8.2.2.2.1. Compression flange

The compression flange is checked under construction loads for nominal yielding and for flexural resistance under local buckling resistance and lateral-torsional buckling. LRFD Art. 6.10.3.2

Flange nominal yielding:

$$f_{bu} + f_l \leq \phi_f R_h F_{yc}$$ LRFD Eq. 6.10.3.2.1-1

where:

ϕ_f	=	resistance factor for flexure = 1.0	LRFD Art. 6.5.4.2
f_{bu}	=	largest value of the compressive stress throughout the unbraced length in the flange under consideration calculated without considering flange lateral bending	LRFD Art. 6.10.1.6
	=	17.1 ksi for Strength limit state	**Appendix D.1.3**
	=	19.2 ksi for Special load combination	**Appendix D.1.3**
f_ℓ	=	flange lateral bending stress	LRFD Art. 6.10.1.6
	=	11.4 ksi for Strength limit state	**Appendix D.1.4**
	=	11.6 ksi for Special load combination	**Appendix D.1.4**
F_{yc}	=	specified minimum yield strength of the compression flange = 50 ksi	
R_h	=	hybrid factor = 1.0	LRFD Art. 6.10.1.10.1

For Strength I limit state:

$$f_{bu} + f_l = 17.1 + 11.4 = 28.5\ ksi < \phi_f R_h F_{yc} = 1.0 \times 1.0 \times 50 = 50\ ksi$$ **OK**

For Special load combination:

$$f_{bu} + f_l = 19.2 + 11.6 = 30.8\ ksi < \phi_f R_h F_{yc} = 1.0 \times 1.0 \times 50 = 50\ ksi$$ **OK**

Flexural resistance due to local buckling:

$$f_{bu} + \frac{1}{3} f_l \leq \phi_f F_{nc}$$ LRFD Eq. 6.10.3.2.1-2

where:

f_{bu}	=	17.1 ksi for Strength limit state	**Appendix D.1.3**
	=	19.2 ksi for Special load combination	**Appendix D.1.3**
f_ℓ	=	11.4 ksi for Strength limit state	**Appendix D.1.4**
	=	11.6 ksi for Special load combination	**Appendix D.1.4**
F_{nc}	=	nominal flexural resistance	

Calculating nominal flexural resistance of the compression flange under flange local buckling, $(F_{nc})_{FLB}$

The nominal flexural resistance of the compression flange (F_{nc}) is determined as specified in Article 6.10.8.2. — LRFD Art. 6.10.3.2.1

If $\lambda_f \leq \lambda_{pf}$:

$$F_{nc} = R_b R_h F_{yc}$$ LRFD Eq. 6.10.8.2.2-1

Otherwise:

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

where:

λ_f = slenderness ratio for the compression flange

= $\frac{b_{fc}}{2t_{fc}}$ — LRFD Eq. 6.10.8.2.2-3

λ_{pf} = limiting slenderness ratio for a compact flange

= $= 0.38\sqrt{\frac{E}{F_{yc}}}$ — LRFD Eq. 6.10.8.2.2-4

λ_{rf} = limiting slenderness ratio for a noncompact flange

= $0.56\sqrt{\frac{E}{F_{yr}}}$ — LRFD Eq. 6.10.8.2.2-5

F_{yr} = compression flange stress at the onset of nominal yielding within the cross-section — LRFD Art. 6.10.8.2.2

= min $(0.7F_{yc}, F_{yw}) > 0.5\,F_{yc}$

R_b = web load-shedding factor = 1.0 — LRFD Art. 6.10.3.2.1

$F_{yr} = 0.7F_{yc} = 0.7(50) = 35\ ksi < F_{yw} = 50\ ksi$ **OK**

Also, $F_{yr} > 0.5F_{yc} = 0.5(50) = 25\ ksi$ **OK**

Therefore, $F_{yr} = 35\ ksi$

$$\lambda_{rf} = 0.56\sqrt{\frac{29000}{35}} = 16.1$$

$$\lambda_f = \frac{b_{fc}}{2t_{fc}} = \frac{18}{2\times 0.75} = 12$$

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_{yc}}} = 0.38\sqrt{\frac{29000}{50}} = 9.152$$

Since $\lambda_f = 12 > \lambda_{pf} = 9.152$, use LRFD Eq. 6.10.8.2.2-2:

$$F_{nc} = \left[1 - \left(1 - \frac{35}{(1.0)(50)}\right)\left(\frac{12 - 9.152}{16.1 - 9.152}\right)\right](1.0)(1.0)(50) = 43.9\ \text{ksi}$$

$(F_{nc})_{FLB}$ = 43.9 ksi

For Strength I limit state:

$$f_{bu} + \frac{1}{3} f_l = 17.1 + \frac{1}{3}(11.4) = 20.9\ ksi$$

$$< \emptyset_f (F_{nc})_{FLB} = (1.0)(43.9) = 43.9\ ksi \qquad \textbf{OK}$$

For Special load combination:

$$f_{bu} + \frac{1}{3} f_l = 19.2 + \frac{1}{3}(11.6) = 23.1\ ksi$$

$$< \emptyset_f (F_{nc})_{FLB} = (1.0)(43.9) = 43.9\ ksi \qquad \textbf{OK}$$

Flexural resistance due to lateral-torsional buckling:

$$f_{bu} + \frac{1}{3} f_l \le \emptyset_f F_{nc}$$ LRFD Eq. 6.10.3.2.1-2

where:

f_{bu}	=	17.1 ksi for Strength limit state	**Appendix D.1.3**
	=	19.2 ksi for Special load combination	**Appendix D.1.3**
f_ℓ	=	11.4 ksi for Strength limit state	**Appendix D.1.4**
	=	11.6 ksi for Special load combination	**Appendix D.1.4**

<u>Calculating nominal flexural resistance of the compression flange under lateral-torsional buckling, $(F_{nc})_{LTB}$</u>

For unbraced lengths in which the member is prismatic, the lateral-torsional buckling resistance of the compression flange shall be taken as: LRFD Art. 6.10.8.2.3

If $L_b \le L_p$,

$$F_{nc} = R_b R_h F_{yc}$$ LRFD Eq. 6.10.8.2.3-1

If $L_p < L_b \le L_r$,

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

If $L_b > L_r$,

$$F_{nc} = F_{cr} \le R_b R_h F_{yc}$$ LRFD Eq. 6.10.8.2.3-3

where:

C_b	=	moment gradient modifier = 1.0	**Appendix D.1.4**
F_{yr}	=	compression-flange stress at the onset of nominal yielding within the cross-section, including residual stress effects, but not including compression flange lateral bending	
	=	min $(0.7F_{yc}, F_{yw}) > 0.5\ F_{yc}$	LRFD Art. A6.3.3
L_b	=	unbraced length = 21 ft – 11 in. = 263 in.	

L_b is taken as the distance between interior diaphragms in the positive bending region, which is 21 ft – 11 in. (263 in.).

L_p = limiting unbraced length to achieve the nominal flexural resistance of $R_bR_hF_{yc}$ under uniform bending

= $1.0 r_t \sqrt{\frac{E}{F_{yc}}}$ LRFD Eq. 6.10.8.2.3-4

= 111.41 in. **Appendix D.1.4**

L_r = limiting unbraced length to achieve the onset of nominal yielding in either flange under uniform bending with consideration of compression – flange residual stress effects

= $\pi r_t \sqrt{\frac{E}{F_{yr}}}$ LRFD Eq. 6.10.8.2.3-5

r_t = effective radius of gyration for lateral-torsional buckling

= $\frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\right)}}$ LRFD Eq. 6.10.8.2.3-9

= 4.626 in. **Appendix D.1.4**

S_{xc} = elastic section modulus about the major axis of the section to the compression flange = M_{yc}/F_{yc}

S_{xt} = elastic section modulus about the major axis of the section to the tension flange = M_{yt}/F_{yt}

$L_r = \pi(4.626)\sqrt{\frac{29000}{35}}$ = 418.3 in.

Since L_p = 111.41 in. < L_b = 263 in. < L_r = 418.32 in.

Use LRFD Eq. 6.10.8.2.3-2:

$$F_{nc} = 1.0\left[1-\left(1-\frac{35}{(1.0)(50)}\right)\left(\frac{263-111.41}{418.32-111.41}\right)\right](1.0)(1.0)(50) = 42.6\ ksi$$

$$< (1.0)(1.0)(50) = 50\ ksi$$

$(F_{nc})_{LTB}$ = 42.6 ksi

For Strength I limit state:

$$f_{bu} + \frac{1}{3}f_l = 17.1 + \frac{1}{3}(11.4) = 20.9\ ksi$$

$$< \emptyset_f (F_{nc})_{LTB} = (1.0)(42.6) = 42.6\ ksi$$ **OK**

For Special load combination:

$$f_{bu} + \frac{1}{3}f_l = 19.2 + \frac{1}{3}(11.6) = 23.1\ ksi$$

$$< \emptyset_f (F_{nc})_{LTB} = (1.0)(42.6) = 42.6\ ksi$$ **OK**

Step 8.2.2.2.2. Tension flange

The tension flange is checked only for nominal yielding. LRFD Art. 6.10.3.2.2

$$f_{bu} + f_l \leq \phi_f R_h F_{yt}$$ LRFD Eq. 6.10.3.2.2-1

where:

f_{bu}	=	largest value of the compressive stress throughout the unbraced length in the flange under consideration calculated without considering flange lateral bending	LRFD Art. 6.10.1.6
	=	10.5 ksi for the Strength limit state	**Appendix D.1.3**
	=	11.7 ksi for the Special load combination	**Appendix D.1.3**
f_ℓ	=	flange lateral bending stress	LRFD Art. 6.10.1.6
	=	5.02 ksi for the Strength limit state	**Appendix D.1.4**
	=	4.89 ksi for the Special load combination	**Appendix D.1.4**
F_{yt}	=	specified minimum yield strength of tension flange = 50 ksi	
R_h	=	hybrid factor = 1.0	LRFD Art. 6.10.1.10.1

For Strength I limit state:

$$f_{bu} + f_l = 10.5 + 5.02 = 15.52\ ksi$$
$$< \phi_f R_h F_{yt} = 1.0 \times 1.0 \times 50 = 50\ ksi$$ **OK**

For Special load combination:

$$f_{bu} + f_l = 11.7 + 4.89 = 16.59\ ksi$$
$$< \phi_f R_h F_{yt} = 1.0 \times 1.0 \times 50 = 50\ ksi$$ **OK**

If there are holes in the tension flanges at the section under consideration, the tension flange shall satisfy the requirements specified in Art. 6.10.1.8. LRFD Art. 6.10.3.1.

The section at $0.4L_{ds}$ has no holes in the tension flange; therefore, this check is not performed.

Step 8.2.2.3. Shear resistance LRFD Art.6.10.3.3

In **Step 8.2.1.3**, the unstiffened shear capacity of the girder is demonstrated to be sufficient to resist the applied shear at the strength limit state. Therefore, the section will have sufficient shear capacity for the constructability check.

Step 8.2.3. Service limit state

LRFD Art. 6.10.4

Figure 35 shows the service limit state design procedure.

Step 8.2.3.1. Elastic deformations

Since the bridge is not designed to permit pedestrian traffic, the live load deflection will be limited to L/800, where L is span length. LRFD Art. 6.10.4.1 &. 2.5.2.6

L/800 = (98.67/800) × 12 = 1.48 in.

The optional live load deflection of an exterior girder at $0.4L_{ds}$ **Step 7.6**

= 1.04 in.< 1.48 in. **OK**

Therefore, the elastic deformation limit is satisfied at the maximum positive bending region ($0.4L_{ds}$).

Step 8.2.3.2. Permanent deformations

LRFD Art. 6.10.4.2.2

To control permanent deformations, flange stresses are limited as follows:

For the top steel flanges of composite sections:

$$f_f \leq 0.95 R_h F_{yf}$$

LRFD Eq. 6.10.4.2.2-1

For the bottom steel flanges of composite sections:

$$f_f + \frac{f_l}{2} \leq 0.95 R_h F_{yf}$$

LRFD Eq. 6.10.4.2.2-2

where:

f_f = flange stress at the section under consideration due to the Service II loads calculated without consideration of flange lateral bending

= 18.16 ksi for top flange **Table 22**

= - 33.63 ksi for bottom flange **Table 22**

f_l = flange lateral bending stress at the section under consideration due to the Service II loads = 0 ksi LRFD Art. C6.10.4.2.2

Lateral bending in the bottom flange is only a consideration at the service limit state for all horizontally-curved I-girder bridges and for straight I-girder bridges with discontinuous cross-frames or diaphragm lines in conjugation with skews exceeding 20 degrees. Wind load and deck overhang effects are not considered at the service limit state. LRFD Art. C6.10.4.2.2

No moment redistribution was performed over the pier. **Appendix E**

R_h = hybrid factor = 1.0 LRFD Art. 6.10.1.10.1

F_{yf} = specified minimum yield strength of the flange = 50 ksi

Top flange:

$$f_f = 18.16\ ksi < 0.95 R_h F_{yf} = 0.95(1.0)(50) = 47.5\ ksi \qquad \textbf{OK}$$

Bottom flange:

$$f_f + \frac{f_l}{2} = |-33.63| + 0 = 33.63\ ksi < 0.95(1.0)(50) = 47.5\ ksi \qquad \textbf{OK}$$

Step 8.2.4. Fatigue limit state

Figure 36 shows the fatigue limit state design procedure. The fatigue calculation procedure in the positive bending region is similar to that previously presented for the negative bending region in **Step 8.1.4**.

Step 8.2.4.1. Load induced fatigue

The fatigue resistance of the base metal at the weld joining the interior diaphragm connection plate located at 21 ft – 11 in. (i.e., 21.917 ft or 263 in.) from the abutment bearing centerline to the flanges is evaluated. **Figure 3**

Fatigue detail = Category C′

This is the welded stiffener connection 4.1 in LRFD Table 6.6.1.2.3-1.

$(\Delta F)_{TH} = (\Delta F)_n$ = 12 ksi **Step 8.1.4.1**

The concrete deck contribution to composite section properties are considered when computing barrier load (DC_2) and fatigue live load stresses. Fatigue live load stress ranges are applied to the short-term composite section in the subsequent fatigue calculations.

At the section under consideration, the unfactored permanent loads produce compression at the top of the girder and tension at the bottom of the girder. In this example, the effect of the future wearing surface is conservatively excluded when calculating the net applied tensile stress. The DC_1, DC_2, and fatigue loads at 21.917 ft are interpolated as follows:

Distance from the abutment bearing centerline(ft)	DC_1 (kip-ft)	DC_2 (kip-ft)	Fatigue moment (+) (kip-ft)	Fatigue moment (-) (kip-ft)
19.730 ft ($0.2L_{ds}$)	559	73	538	-74
21.917 ft	**584**	**77**	**567**	**-83**
29.600 ft ($0.3L_{ds}$)	670	90	668	-112

At 21.917 ft:

I_{nc} = 12,806 in.^{4}

I_{c8} = 41,847 in.^{4}

I_{c24} = 29,500 in.^{4}

Distance from the centroid to the girder top:

Noncomposite section	= 21.95 in.		**Step 3.2.2.1**
Composite (n = 8)	= 4.47 in.		**Step 3.2.2.2**
Composite (3n = 24)	= 11.79 in.		**Step 3.2.2.3**

Distance from the centroid to the girder bottom:

Noncomposite section	= 13.425 in.		**Step 3.2.2.1**
Composite (n = 8)	= 30.905 in.		**Step 3.2.2.2**
Composite (3n = 24)	= 23.585 in.		**Step 3.2.2.3**

Distance to the bottom of the top flange for:

Noncomposite section	= 21.95 – 0.75	= 21.20 in.
Composite (n = 8)	= 4.47– 0.75	= 3.72 in.
Composite (3n = 24)	= 11.79 – 0.75	= 11.04 in.

Distance to the top of the bottom flange for:

Noncomposite section	= 13.425 – 1.625	= 11.80 in.
Composite (n = 8)	= 30.905 – 1.625	= 29.28 in.
Composite (3n = 24)	= 23.585 – 1.625	= 21.96 in.

At bottom of top flange:

Compressive stress produced by DC_1 and DC_2 loads are:

$$f_{DC1} = \frac{|584| \times 12}{(12806/21.20)} \quad = 11.6 \text{ ksi (compression)}$$

$$f_{DC2} = \frac{|77| \times 12}{(29500/11.04)} \quad = 0.35 \text{ ksi (compression)}$$

$$\Sigma = 11.6 + 0.35 \quad = 11.95 \text{ ksi (compression)}$$

Factored tensile stress produced by the fatigue load is:

$$f_{LL+IM} = 1.75 \times \frac{|-83| \times 12}{(41847/3.72)} \quad = 0.15 \text{ ksi (tension)}$$

$$11.95\ ksi > 0.15\ ksi \quad \textbf{OK}$$

The detail is subjected to a net compressive stress at the bottom of the top flange. Therefore, the load induced fatigue is not checked. LRFD Art. 6.6.1.2

At top of the bottom flange:

DC_1 and DC_2 produce tensile stress at the top of the bottom flange. Therefore, the load induced fatigue needs to be checked.

$$\Delta f = \frac{(567 + |-83|) \times 12}{(41847/29.28)} \quad = 5.46 \text{ ksi}$$

$$\gamma(\Delta f) = (1.75)(5.46) \quad = 9.56 \text{ ksi}$$

$$\gamma(\Delta f) = 9.56\ ksi < (\Delta F)_n = 12\ ksi \quad \textbf{OK}$$

Step 8.2.4.2. Distortion induced fatigue

LRFD Art. 6.6.1.3

A positive connection is to be provided for all transverse connection-plate details to both the top and bottom flanges to prevent distortion induced fatigue.

FHWA (2015)

Step 8.2.4.3. Fracture

LRFD Art. 6.6.2

FHWA (2015)

The appropriate Charpy V-notch fracture toughness, found in Table 6.6.2-2, must be specified for main load-carrying components subjected to tensile stress under the Strength I load combination.

Step 8.2.4.4. Special fatigue requirement for webs

If the following condition is satisfied, significant elastic flexing of the web due to shear is not expected to occur, and the member is assumed to be able to sustain an infinite number of smaller loadings without fatigue cracking due to this effect.

LRFD Art. C6.10.5.3

$$V_u \leq V_{cr}$$

LRFD Eq. 6.10.5.3-1

where

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

V_{cr} = shear-yielding or shear-buckling resistance determined from LRFD Eq. 6.10.9.3.3-1

However, designs utilizing unstiffened webs at the strength limit state, automatically satisfy this criterion. Thus, this criterion is not explicitly evaluated herein.

STEP 9. MISCELLANEOUS STEEL DESIGN

Step 9.1. Bolted Splice and Fastener Design

Splices are typically used to connect girder sections together in the field to develop continuous spans. The design of bolted splices is covered in LRFD Art. 6.13.6.1.

Figure 38 shows a typical bolted field splice for an I-girder with top and bottom flange splice plates and web splice plates. When flange and/or web thickness on one side of the splice location is different from those on the other side, filler plates are used to match the thickness at the splice location.

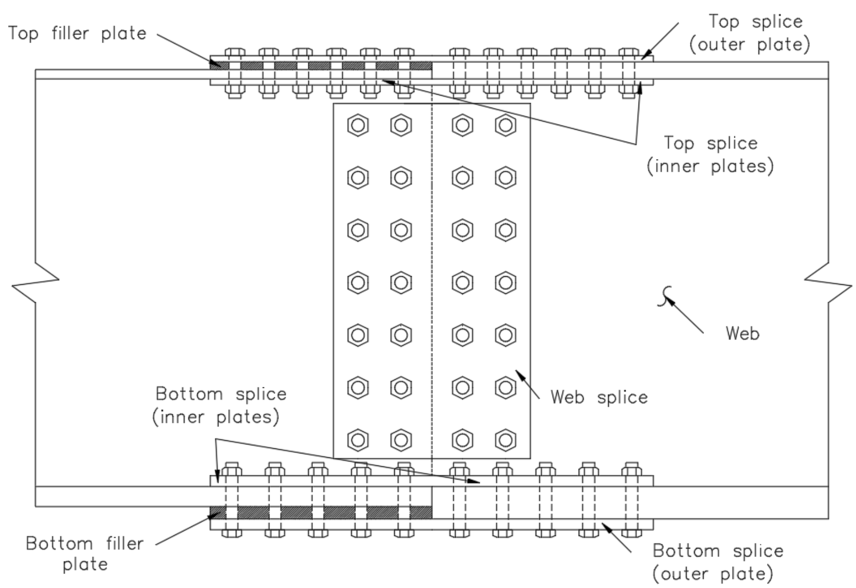


Figure 38. Typical bolted field splice for an I-section flexural member

The exterior girder field splice design is described in this step. **Figure 39** shows the design procedure implemented in this example.

The details shown in **Figure 40** to **Figure 42** are taken from an existing plan as the point of start to minimize design revisions. **Figure 40** and **Figure 41** show the splice detail at the top and bottom flanges. **Figure 42** shows the cross-section and the elevation at the connection.

In the absence of such details, bolt and plate dimensions need to be established by following the limits established by the DOT policies and/or AASHTO LRFD Specifications.

After calculating the required number of bolts as per the strength limit state, the bolt spacing, along with the end and edge distances, must first be established and checked before examining the factored bearing resistance of the bolt holes and the block shear rupture resistance of the splice plates and the flanges.

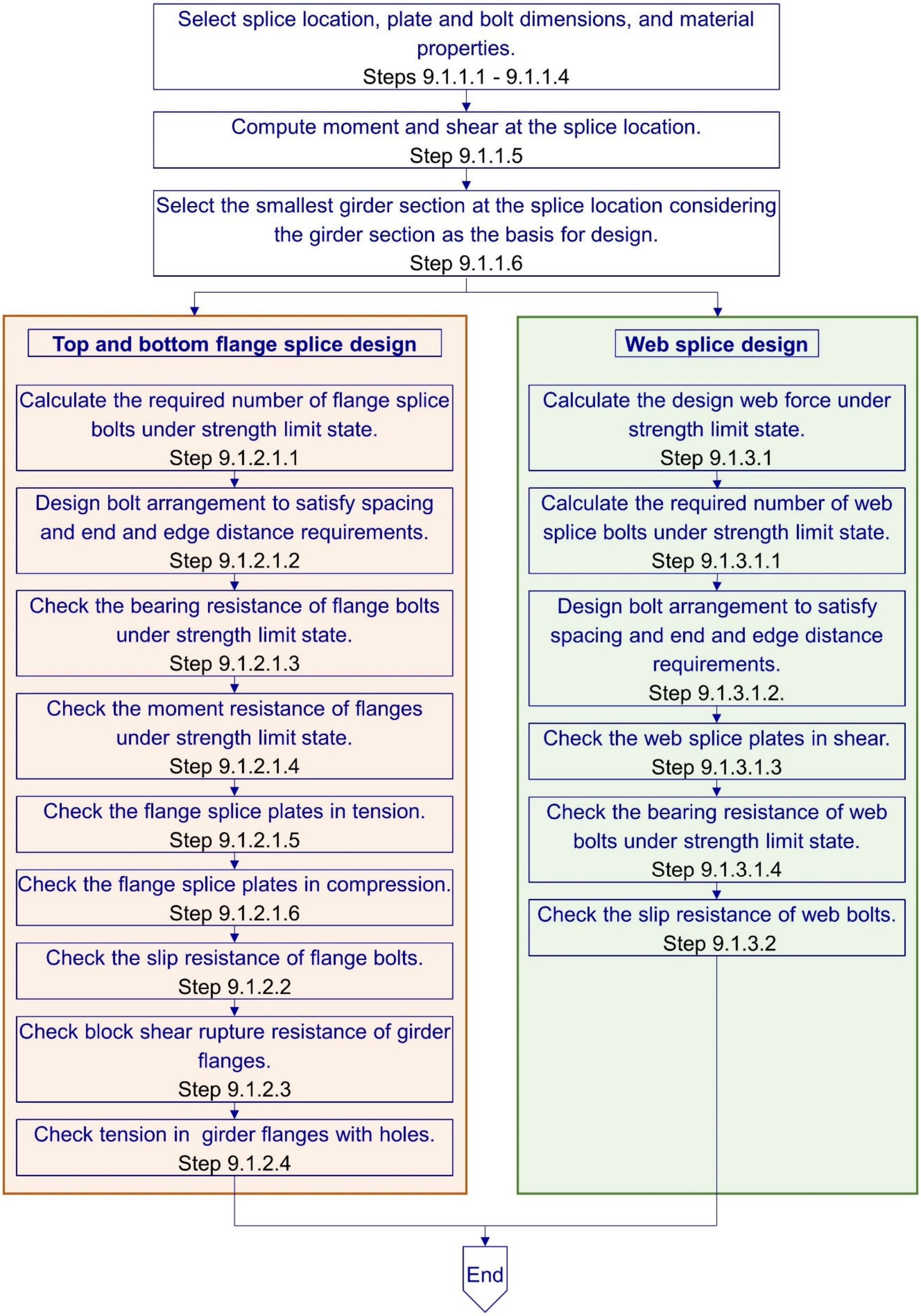


Figure 39. Field splice design procedure

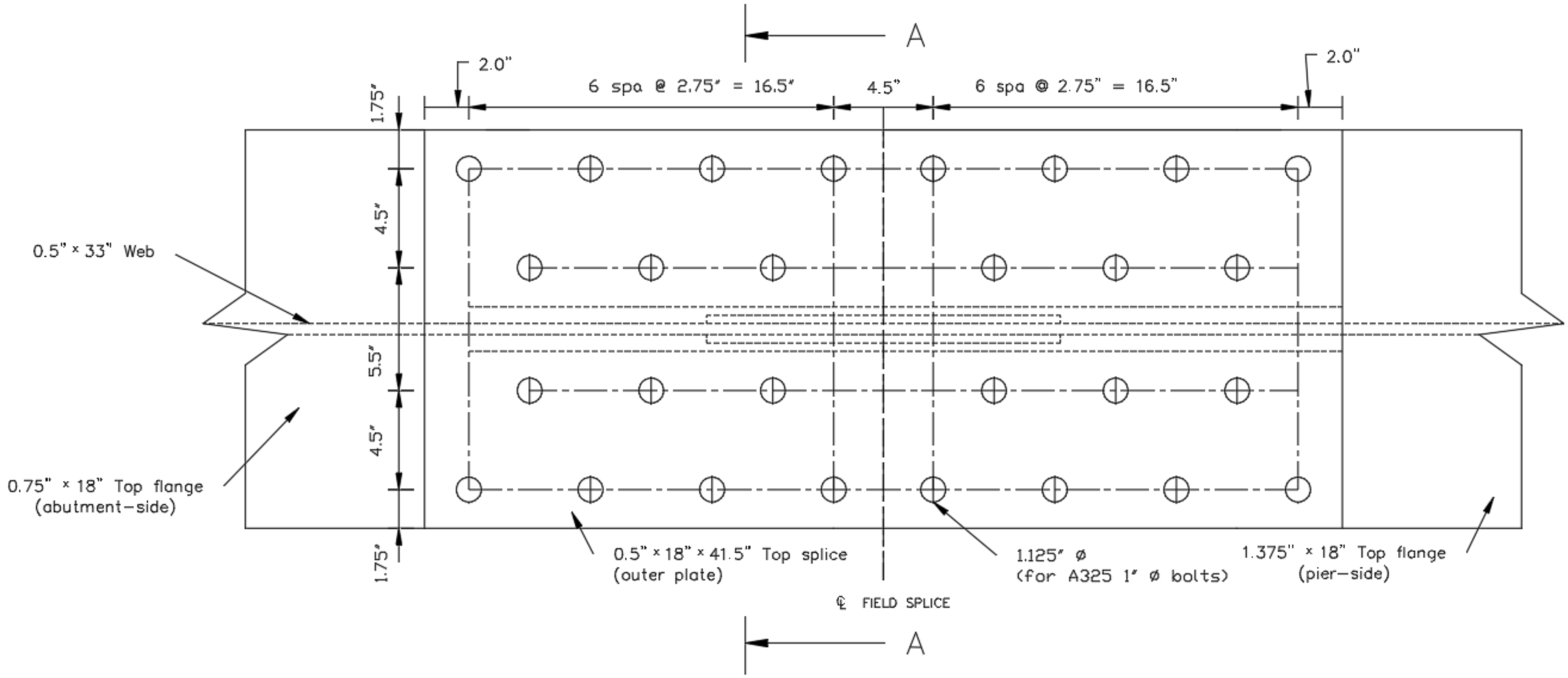


Figure 40. Top flange splice detail of an exterior girder

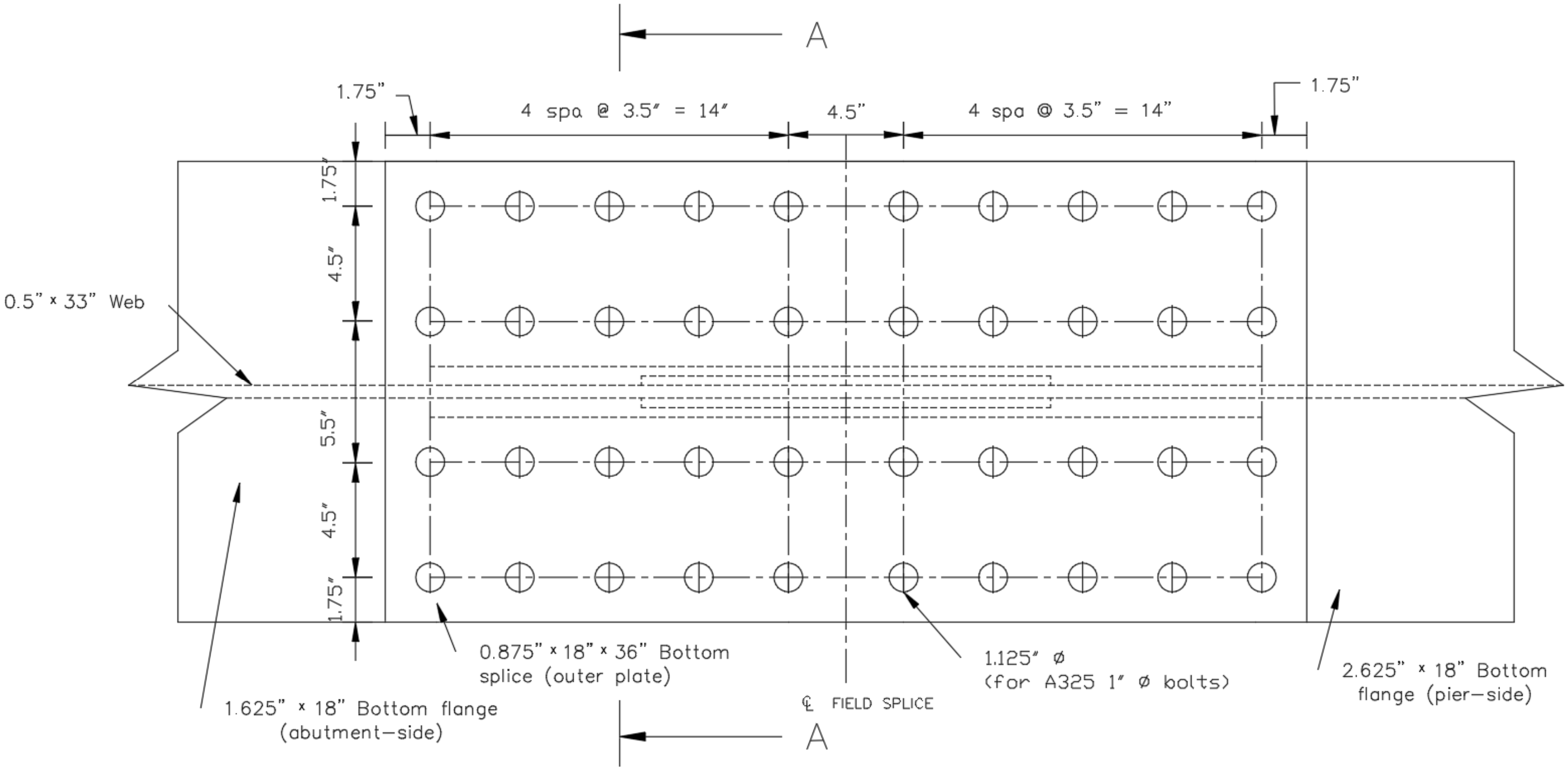


Figure 41. Bottom flange splice detail of an exterior girder

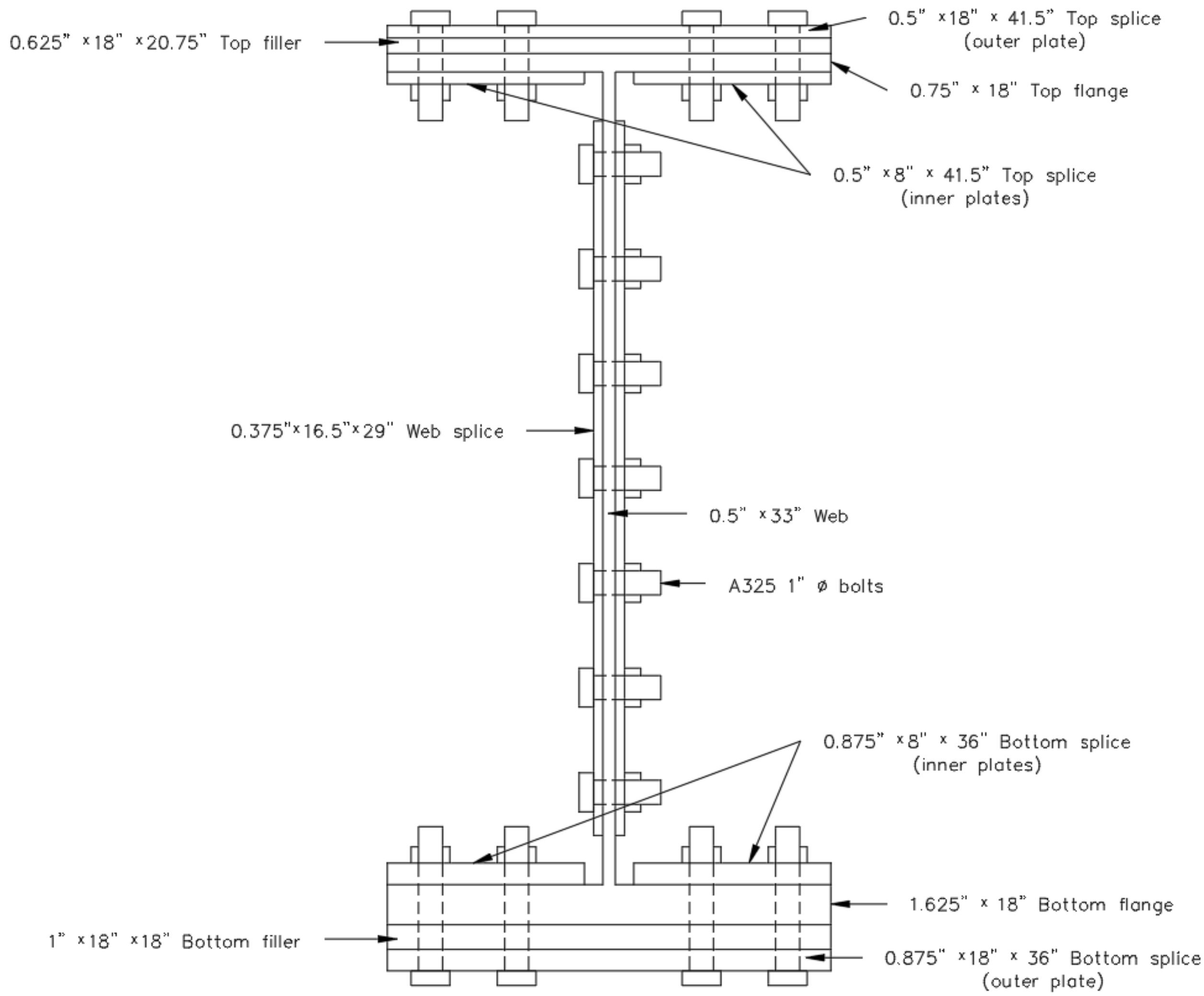


Figure 42. Cross-section of the splice connection (section A-A)

Step 9.1.1. Splice location, plate and bolt dimensions, and material properties

This step outlines the parameters for splice design.

Step 9.1.1.1. Selection of the splice location

Field splices are to be located at low-stress areas at or near the point of contraflexure for continuous spans.	BDM Art. 7.02.13B
In continuous spans, splices should be made at or near points of dead load contraflexure. Web and flange splices in areas of stress reversal shall be investigated for both positive and negative flexure.	LRFD Art. 6.13.6.1.3a

Points of superimposed dead load contraflexure are located at about $0.71L_{ds}$ (70.06 ft) from the abutment bearing centerline.

The distance to the point of contraflexure from the girder end

$$= 1.33 + 0.71(98.67) = 71.39 \text{ ft}$$

In this example, the bolt splices are provided at 71 ft – 3 3/8 in. (71.281 ft) from the abutment bearing center line. These points are closer to the points of superimposed dead load contraflexure.

Step 9.1.1.2. Geometry of the splice plates

Step 9.1.1.2.1. Flange splice plates

In this example, a single outer plate and 2 inner plates are used at the top and bottom flange splice locations. The splice plate dimensions are as follows:

	Dimensions	**Outer plate (in.)**	**Inner plates* (in.)**
Top flange plate	Thickness	0.5	0.5
	Width	18.0	8.0
	Length	41.5	41.5
Bottom flange plate	Thickness	0.875	0.875
	Width	18.0	8.0
	Length	36.0	36.0

* 2 inner plates are used.

Step 9.1.1.2.2. Web splice plates

Two plates are used on either side of the web. The plate dimensions are:

Thickness = 0.375 in.

Depth = 29 in.

Length = 16 in.

Step 9.1.1.3. Material properties of the splice plate

Yield strength, F_y = 50 ksi — LRFD Table 6.4.1-1

Tensile strength, F_u = 65 ksi — SSFC 906.05B

Step 9.1.1.4. Material properties of the splice bolts

All high strength bolts are to be hot dip galvanized. — BDM Art. 7.02.13C

In this example, similar bolts are used at both flange and web splices. — SSFC 906.07

Bolt type = A325

Bolt diameter, d = 1 in.

Bolt hole diameter, d_h = 1.125 in. — LRFD Table 6.13.2.4.2-1

For 1 in. diameter bolts, the standard hole size is 1-1/8 in. (1.125 in.). — LRFD Art. 6.13.2.4.1a

Specified minimum bolt tensile strength, F_{ub} = 120 ksi — LRFD Table 6.4.3.1.1-1

Step 9.1.1.5. Unfactored design moment and shear at the splice location

Step 9.1.1.5.1. Unfactored design moments

The moments at the splice location, 71.281 ft, were interpolated from the values at 69.069 ft ($0.7L_{ds}$) and 78.936 ft ($0.8L_{ds}$) given in **Table 10**, **Table 14**, and **Table D.1**.

$M_{DC1} = M_g + M_D + M_H + M_f$ = -150 kip-ft

M_b = 13 kip-ft

M_{ws} = 20 kip-ft

M_{SE} = -96 kip-ft

$M_{construction}$ = 84 kip-ft

$M_{HL\text{-}M\ (+)}$ = 1295 kip-ft

$M_{HL\text{-}M\ (-)}$ = -729 kip-ft

Step 9.1.1.5.2. Unfactored design shear

The shear forces at the splice location, 71.281 ft, were interpolated from the values at 69.069 ft ($0.7L_{ds}$) and 78.936 ft ($0.8L_{ds}$) given in **Table 12** and **Table 16**.

$V_{DC1} = V_g + V_D + V_H + V_f$ = -44 kips

V_b = -5 kips

V_{ws} = -7 kips

V_{SE} = -1.35 kips

$V_{construction}$ = -44 kips

$V_{HL\text{-}M\ (+)}$ = 15 kips

$V_{HL\text{-}M\ (-)}$ = -80 kips

Note: Shear force due to construction loads at $0.7L_{ds}$ and $0.8L_{ds}$ are -41.6 kips and -54.1 kips, respectively.

Step 9.1.1.6. Select girder section as the basis for field splice design

The girder plate sizes on the positive and negative moment regions under the superimposed dead loads and adjacent to the splice location are given below. In this two-span continuous structure, the girder sections towards the positive and negative moment regions are labeled as “abutment-side” and “pier-side”, respectively.

Girder dimensions	Abutment-side	Pier-side
Web thickness, t_w (in.)	0.5	0.5
Web depth, D (in.)	33	33
Top flange width, b_{flt} (in.)	18	18
Top flange thickness, t_{flt} (in.)	0.75	1.375
Bottom flange width, b_{flb} (in.)	18	18
Bottom flange thickness, t_{flb} (in.)	1.625	2.625

In this example, both flanges and web on either side of the splice location have the same F_u and F_y.

Yield strength, F_y	= 50 ksi
Tensile strength, F_u	= 65 ksi

Where a girder section changes at a splice, the smaller of the two connected sections shall be used in the design. Therefore, the abutment-side section dimensions are used for the bolted field splice design presented in this example. AISC (2017)

Step 9.1.2. Flange splice design

In the following steps, the adequacy of the splice details provided at the two flanges is evaluated.

Step 9.1.2.1. Strength limit state

The factored Strength I moments at the splice location are:

Positive moment = 0.9(-150) +1.25(13) + 1.5(20) + 1.75(1295) = 2178 kip-ft

Negative moment = 1.25(-150) + 0.9(13) + 0.65(20) + 1.75(-729) + 1.0(-96) = -1535 kip-ft

The negative moment under Strength I limit state with the settlement produces a higher negative moment at the splice location.

Step 9.1.2.1.1. Flange splice bolts

Flange splice plates and their connections shall be designed to develop the smaller design yield resistance of the flanges at the point of splice. LRFD Art. 6.13.6.1.3b

The design yield resistance of each flange, P_{fy}, at the point of splice is:

$$P_{fy} = F_{yf} A_e$$ LRFD Eq. 6.13.6.1.3b-1

where:

ϕ_u = resistance factor for fracture in net section of tension members = 0.80 — LRFD Art. 6.5.4.2

ϕ_y = resistance factor for yielding in gross section of tension members = 0.95 — LRFD Art. 6.5.4.2

A_e = effective flange area of the flange under consideration

$$= \left(\frac{\phi_u F_u}{\phi_y F_{yf}}\right) A_n \leq A_g$$ LRFD Eq. 6.13.6.1.3b-2

A_g = gross area of the flange under consideration

A_n = net area of the flange under consideration **Appendix F.1.1**

F_u = specified minimum tensile strength of the flange under consideration — **Step 9.1.1.6**

F_{yf} = specified minimum yield strength of the flange under consideration

For a flange splice with inner and outer splice plates, P_{fy} at the strength limit state shall be assumed to divide equally to the inner and outer splice plates and their connections when the areas of the inner and outer plates do not differ by more than ten percent. Should the areas differ by more than ten percent, the design force in each splice plate and its connection at the strength limit state should instead be determined by multiplying P_{fy} by the ratio of the area of the splice plate under consideration to the total area of the inner and outer splice plates. — LRFD Art. C6.13.6.1.3b

For each flange, the smaller design yield resistance at the splice location, P_{fy}, shall be divided by the factored shear resistance of the bolts, as specified in Art. 6.13.2.2., to determine the total number of flange splice bolts required on one side of the splice at the strength limit state. — LRFD Art. 6.13.6.1.3b

The factored resistance, R_r, of a bolted connection at the Strength limit is: — LRFD Art. 6.13.2.2

$$R_r = \phi R_n$$ LRFD Eq. 6.13.2.2-2

where:

ϕ = resistance factor for bolts — LRFD Art. 6.5.4.2 & 6.13.2.2

= ϕ_s = 0.80 (for bolts in shear)

R_n = nominal resistance of the bolt, connection, or the connected material

<u>Calculation of the nominal resistance of bolts, R_n</u>

For bolt diameters greater than or equal to 1.0 in., R_n of the bolts should be determined assuming the threads are excluded from the shear planes if the flange splice plate thickness closest to the nut is greater than 0.75 in. Otherwise, the threads should be included in the shear planes. — LRFD Art. C6.13.6.1.3b

When threads are excluded from the shear plane: — LRFD Art. 6.13.2.7

$$R_n = 0.56 A_b F_{ub} N_s$$ LRFD Eq. 6.13.2.7-1

When threads are included in the shear plane:

$$R_n = 0.45 A_b F_{ub} N_s$$ LRFD Eq. 6.13.2.7-2

where:

A_b = area of the bolt corresponding to the nominal diameter

F_{ub} = specified minimum bolt tensile strength — **Step 9.1.1.4**

N_s = number of shear planes per bolt

When the flange and web splices are symmetric: N_s = 2.0. — AISC (2017)

BDS conservatively uses the following equation:

$$R_n = 0.48 \times A_b \times F_{ub} \times N_s$$

BDS presents the equation in the following format:

$$R_n = 0.48 \times BOLTAREA \times 120000$$

F_{ub} of 120 ksi is used assuming A325 steel. N_s = 1 is used considering single shear.

Where filler plates are required, the provisions of Article 6.13.6.1.4 shall apply. LRFD Art. 6.13.6.1.3b

The specified minimum yield strength of the fillers 0.25 in. or greater in thickness shall not be less than the larger of 70 percent of the specified minimum yield strength of the connected plate and 36.0 ksi. When bolts carrying loads pass through fillers 0.25 in. or more in thickness in axially loaded connections, including girder flange splices, either: LRFD Art. 6.13.6.1.4

1. *the filler shall be extended beyond the splice material, and the filler extension shall be secured by enough additional bolts to distribute the total stress in the member uniformly over the combined section of the member and the filler or*
2. *the fillers need not be extended and developed provided that the factored resistance of the bolts in shear at the strength limit state, specified in Article 6.13.2.2, is reduced by the following factor:*

$$R = \left(\frac{1+\gamma}{1+2\gamma}\right)$$ LRFD Eq. 6.13.6.1.4-1

where:

$\gamma = A_f / A_p$

A_f = sum of the area of the fillers on both sides of the connected plate

A_p = smaller of either the connected plate area or the sum of the splice plate areas on both sides of the connected plate

The following steps show the calculation of the required number of bolts at the top flange and the bottom flange under the strength limit state.

Top flange:

A_n = 10.76 in.^{2} **Appendix F.1.1.1**

$A_g = 0.75 \times 18$ = 13.5 in.^{2}

$A_e = \left(\frac{\phi_u F_u}{\phi_{yf} F_{yf}}\right) A_n = \left(\frac{0.80 \times 65}{0.95 \times 50}\right) 10.76$ = 11.78 in.^{2}

$A_e = 11.78 \text{ in.}^2 < A_g = 13.5 \text{ in.}^2$ **OK**

Design yield resistance of top flange,

$P_{fy(Top)} = F_{yf} A_e = 50(11.78)$ = 589 kips

Area of the outer plate = 18 × 0.5 = 9 in.^{2}

Area of the inner plates = 2(8 × 0.5) = 8 in.^{2}

Deviation of the areas of the outer and inner plates

= [(9-8)/9] × 100 = 11% > 10%

For a flange with inner and outer splice plates, P_{fy} at the strength limit state may be assumed divided equally to the inner and outer plates and their connections when the areas of the inner and outer plates do not differ more than 10%. For this case, the connections are proportioned assuming double shear. LRFD Art. C6.13.6.1.3b

Should the areas of the inner and outer plates differ by more than 10%, the design force in each splice plate and its connection at the strength limit state should instead be determined by multiplying P_{fy} by the ratio of the area of the splice plate under consideration to the total area of the inner and outer splice plates. For this case, the connections are proportioned for the maximum calculated splice-plate force acting on a single shear plane.

Since the areas of the inner and outer plates differ by more than 10%, the design force carried by inner and outer plates are proportioned as shown below:

Design force carried by the top flange outer plate:

$$P_{fy(Top-outer)} = \left(\frac{9}{9+8}\right) \times 589 = 312 \text{ kips}$$

Design force carried by the top flange inner plates:

$$P_{fy(Top-inner)} = \left(\frac{8}{9+8}\right) \times 589 = 277 \text{ kips}$$

Since the bolt diameter is 1.0 in. and the flange splice plate thickness closest to the nut is 0.5 in. (< 0.75 in.), the threads should be included in the shear planes. LRFD Art. C6.13.6.1.3b

Therefore, LRFD Eq. 6.13.2.7-2 is used.

A_b = π(1.0)²/4 = 0.785 in.²

N_s = number of shear planes per bolt = 2 AISC 2017

R_n = $0.45 A_b F_{ub} N_s$ = 0.45 (0.785) (120) (2) = 84.8 kips

R_r = $\phi_s R_n$ = 0.80 × 84.8 = 67.9 kips

However, filler plates are required at the top flange.

Required thickness of the filler plate at the top flange

= 1.375 – 0.75 = 0.625 in. > 0.25 in.

Specified minimum yield strength of the filler plates

= 50 ksi > max (0.7 × 50 = 35, 36) = 36 ksi **OK** LRFD Art. 6.13.6.1.4

The factored resistance of the bolts in shear at strength limit state is reduced by *R*. LRFD Art. 6.13.6.1.4

A_f = 18 × 0.625 = 11.25 in.²

A_p = smaller (0.75 × 18, 0.5 × 18 + 2 × 0.5 × 8)

= smaller (13.5, 17) = 13.5 in.²

γ = A_f / A_p = 11.25/13.5 = 0.833

$$R = \left(\frac{1+\gamma}{1+2\gamma}\right) = \left(\frac{1+0.833}{1+2(0.833)}\right) = 0.688$$

Factored bolt shear resistance after incorporating the impact of the filler plates, R_{rr}:

R_{rr} = 0.688 × 46.7 = 46.7 kips

Number of bolts required, N = 589/46.7 ≈ 13

Provide 14 bolts to have a symmetric arrangement of the bolts.

Bottom flange:

A_n = 21.9375 in.^{2} **Appendix F.1.1.2**

$A_g = 1.625 \times 18$ = 29.25 in.^{2}

$$A_e = \left(\frac{\phi_u F_u}{\phi_{yf} F_{yf}}\right) A_n = \left(\frac{0.80 \times 65}{0.95 \times 50}\right) 21.9375$$ = 24.02 in.^{2}

$A_e = 24.02\ in.^2 < A_g = 29.25 in.^2$ **OK**

Design yield resistance of the bottom flange,

$P_{fy(Bott)} = F_{yf} A_e = 50(24.02)$ = 1,201 kips

Area of the outer plate = 18 × 0.875 = 15.75 in.^{2}

Area of the inner plates = 2(8 × 0.875) = 14 in.^{2}

Deviation of the areas of the outer and inner plates

= [(15.75 - 14)/15.75] × 100 = 11% > 10%

Therefore, P_{fy} is distributed according to the ratio of the splice plate under consideration to the total area of the splice plates. LRFD Art. C6.13.6.1.3b

Design force carried by the bottom flange outer plate:

$$P_{fy(Bot-outer)} = \left(\frac{15.75}{15.75 + 14}\right) \times 1201$$ = 636 kips

Design force carried by the bottom flange inner plates:

$$P_{fy(Bot-inner)} = \left(\frac{14}{15.75+14}\right) \times 1201$$ = 565 kips

Since the bolt diameter is 1.0 in. and bottom flange splice plate thickness closest to the nut is 0.875 in. (> 0.75 in.), the threads are excluded from the shear planes. LRFD Art. C6.13.6.1.3b

A_b = $\pi(1.0)^2/4$ = 0.785 in.^{2}

N_s = number of shear planes per bolt = 2

R_n = $0.56 A_b F_{ub} N_s$ = 0.56 (0.785) (120) (2) = 105.5 kips

R_r = $\phi_s R_n$ = 0.80 × 105.5 = 84.4 kips

However, filler plates are required at the bottom flange.

Required thickness of the filler plate at the bottom flange

= 2.625 – 1.625 = 1.0 in. > 0.25 in.

Specified minimum yield strength of the filler plates

= 50 ksi > max (0.7 × 50 = 35, 36) = 36 ksi **OK** LRFD Art. 6.13.6.1.4

The factored resistance of the bolts in shear at the strength limit state is reduced by *R*. LRFD Art. 6.13.6.1.4

A_f	$= 18 \times 1$	$= 18$ in.^{2}
A_p	= smaller $(1.625 \times 18,\ 0.875 \times 18 + 2 \times 0.875 \times 8)$	
	= smaller (29.25, 29.75)	$= 29.25$ in.^{2}
γ	$= A_f / A_p = 18/29.25$	$= 0.615$
R	$= \left(\frac{1+\gamma}{1+2\gamma}\right) = \left(\frac{1+0.615}{1+2(0.615)}\right)$	$= 0.724$

Factored bolt shear resistance after incorporating the impact of the filler plates:

R_{rr}	$= 0.724 \times 84.4$	$= 61.1$ kips
Number of bolts required, N = 1201/61.1		≈ 20

Provide at least 20 bolts on one side of the splice at the bottom flange.

The bolt spacing, along with end and edge distances, are checked for the details at the top and bottom and the web. The rest of the design calculations are performed after completing these checks.

Step 9.1.2.1.2. Spacing, edge distance, and end distance of bolts

Spacing of bolts

The minimum bolt spacing, $s_{min} = 3.0d$. LRFD Art. 6.13.2.6.1

where:

d = bolt diameter = 1.0 in.

$s_{min} = 3.0\,d = 3 \times 1.0 = 3.0$ in.

To seal against the penetration of moisture in joints, the spacing, s, of a single line of bolts adjacent to a free edge of an outer plate or shape must satisfy: LRFD Art. 6.13.2.6.1

$$s \leq (4.0 + 4.0t) \leq 7.0 \qquad \text{LRFD 6.13.2.6.2-1}$$

where:

t = thickness of the thinner outer plate or shape

When there is a second line of fasteners uniformly staggered with those in the line adjacent to the free edge at a gage less than 1.5 + 4.0 t, the staggered spacing, s, in two such lines, shall satisfy:

$$s \leq 4.0 + 4.0t - \left(\frac{3.0g}{4.0}\right) \leq 7.0 \qquad \text{LRFD Eq. 6.13.2.6.2-2}$$

where:

g = gage between holes

The staggered spacing need not be less than one-half the requirement for a single line. LRFD Art. 6.13.2.6.1

Edge distance of bolts

For 1 in. bolts, the minimum edge distance is 1.25 in. LRFD Table 6.13.2.6.6-1

The maximum edge distance shall not be more than eight times the thickness of the thinner outside plate or 5 in. LRFD Art. 6.13.2.6.6

End distance of bolts

The end distance for all types of holes measured from the center of the bolt shall not be less than the edge distances specified in Table 6.13.2.6.6-1. LRFD Art. 6.13.2.6.5

For 1 in. bolts, the minimum end distance is 1.25 in. LRFD Table 6.13.2.6.6-1

The maximum end distance shall be the maximum edge distance specified in LRFD Art. 6.13.2.6.6.

<u>Bolt spacing, edge and end distances at the top splice plate</u>

Minimum spacing between centers of bolts in a single bolt line,

$s_{min} = 3.0$ in.

Minimum spacing provided = 4.5 in. > 3.0 in. **OK**

The maximum spacing between bolts, s_{max}

$s \leq (4.0 + 4.0t)$ or 7.0

$s \leq (4.0 + 4.0 \times 0.5) = 6.0$ in. or 7.0 in.

Maximum spacing provided = 5.5 in. < 6.0 in. **OK**

The bolts at the top flange are staggered as shown in **Figure 40**.

Staggered spacing = 2.75 in.

Gage, g = 4.5 in. **Figure 40**

Since g = 4.5 in. > 1.5 + 4.0t = 1.5 + 4.0 (0.5) = 3.5 in., there is no need to perform the check for staggered bolts. Spacing requirement is checked considering a single bolt line. LRFD Art. 6.13.2.6.1

Maximum edge distance = min (8 × 0.5 = 4.0, 5.0) = 4.0 in.

Edge distance provided = 1.75 in. **Figure 40**

1.25 in. < edge distance = 1.75 in. < 4.0 in. **OK**

End distance provided = 2.0 in. **Figure 40**

1.25 in. < end distance = 2.0 in. < 4.0 in. **OK**

<u>Bolt spacing, edge and end distances of the bottom splice plate</u>

Minimum spacing between centers of bolts, $s_{min} = 3.0$ in.

Minimum spacing provided = 3.5 in. > 3.0 in. **OK**

The maximum spacing between bolts, s_{max}

$s \leq (4.0 + 4.0t)$ or 7.0

$s \leq (4.0 + 4.0 \times 0.875) = 7.5$ in. or 7.0 in.

Maximum spacing provided = 4.5 in. < 7.0 in. **OK**

Maximum edge distance = min (8 × 0.875 = 7.0, 5.0) = 5.0 in.

Edge distance provided = 1.75 in. **Figure 41**

1.25 in. < edge distance = 1.75 in. < 5.0 in. **OK**

End distance provided = 1.75 in. **Figure 41**

1.25 in. < end distance = 1.75 in. < 5.0 in. **OK**

Step 9.1.2.1.3. Bearing resistance check

The nominal bearing resistance of the connected member at the strength limit state is calculated as the sum of the bearing resistances of the individual bolts parallel to the line of applied force. LRFD Art. C6.13.2.9

For standard holes loaded in any direction, the nominal resistance of interior and end bolt holes at the strength limit state, R_n, is calculated based on the clear distance between holes and the edge distance of the connected member. LRFD Art. 6.13.2.9

With bolts spaced at a clear distance between holes not less than 2.0*d* and with a clear end distance not less than 2.0*d*:

$$R_n = 2.4dtF_u$$

LRFD Eq. 6.13.2.9-1

If either the clear distance between holes is less than 2.0*d*, or the clear end distance is less than 2.0*d*:

$$R_n = 1.2L_c tF_u$$

LRFD Eq. 6.13.2.9-2

where:

d = nominal diameter of the bolt

F_u = minimum specified tensile strength of the connected material

L_c = clear distance between holes or between the hole and the end of the member in the direction of the applied bearing force

t = thickness of the connected material

The factored resistance of a bolted connection at the strength limit state, R_r:

$$R_r = \phi R_n$$

LRFD Eq. 6.13.2.2-2

where:

ϕ = ϕ_{bb} = resistance factor for bolts bearing on material = 0.80 LRFD Art. 6.5.4.2 & 6.13.2.2

The current BDS code only determines the bearing resistance in cases where the clear distance between the bolt holes is less than 2.0d. The equation in the BDS is:

$$R_n = (\text{Edge distance - } d/2) \times 1.2 \times PLT \times F_u$$

where: PLT = thickness of the connected material

In this example, the bearing resistance of the most critical member at each flange, i.e. flange or the splice plate, is evaluated.

At the top flange splice connection:

½ in. thick splice plates are used.

The sum of thickness of the inner and outer splice plates = 2(0.5) = 1.0 in.

The flange thickness on the "abutment-side" and "pier-side" are 0.75 in. and 1.375 in., respectively.

Thickness of the thinner top flange = 0.75 in. (abutment-side)

F_u (the splice plates and top flange) = 65 ksi

At the top flange splice plate connection:

$\sum$ (inner and outer splice plate thicknesses) × F_u

> thickness of the thinner top flange × F_u

Therefore, the bearing capacity of the top flange is evaluated.

For the end bolts:

A 1/4 in. gap is provided between girder sections at the splice location.

Clear end distance, L_c = (Edge distance – hole diameter/2)

= (4.5 – 0.25 – 1.125)/2 = 1.5625 in.

Since the clear end distance of bolt holes is less than 2.0d = 2 × 1 = 2 in.

$R_n = 1.2 L_c t F_u$ = 1.2 (1.5625) (0.75) (65) = 91.4 kips

t is the top flange thickness and F_u is the minimum tensile strength of the top flange.

There are 2 bolts adjacent to the end of the top flange. **Figure 40**

Capacity (R_n) for the two end bolts = 2 (91.4) = 182.8 kips

Factored bearing resistance of the bolts, R_r

$R_r = \phi_{bb} R_n$ = (0.8) (182.8) = 146 kips

The factored shear resistance of a top flange bolt = 46.7 kip/bolt **Step 9.1.2.1.1**

The total factored shear resistance of the bolts in the two holes adjacent to the end of the flange = 2 (46.7) = 93.4 kips

R_r = 146 kips > 93.4 kips

Shear capacity of the bolts at the end of the plate controls the capacity.

For the interior bolts:

Clear distance between interior holes, L_c = 2 × 2.75 – 1.125 = 4.375 in.

Since the clear distance between two interior bolt holes is greater than 2.0d = 2 × 1 = 2 in.,

$R_n = 2.4dtF_u$ = 2.4 (1.0) (0.75) (65) = 117 kips

Figure 40

There are 12 interior bolts in the top flange.

Capacity (R_n) of the 12 interior bolts = 12 (117) = 1,404 kips

The factored bearing resistance of the interior bolts, R_r

$R_r = \phi_{bb}R_n$ = (0.80)(1404) = 1,123 kips

The total factored shear resistance of the 12 interior bolts = 12 (46.7) = 560 kips

R_r = 1,123 kips > 560 kips **OK**

Therefore, the flange interior bolt bearing capacity is greater than the bolt shear capacity.

For end and interior bolts:

The total factored bolt shear resistance at the top flange, R_r

= 93.4 + 560 = 653.4 kips

Design yield resistance of top flange, $P_{fy(Top)}$ = 589 kips

Step 9.1.2.1.1

R_r = 653.4 kips > P_{fy} = 589 kips **OK**

The top flange capacity is controlled by flange yielding.

<u>At the bottom flange splice plate connection:</u>

The sum of thickness of the inner and outer splice plates = 2(0.875) = 1.75 in.
Thickness of the thinner bottom flange = 1.625 in. (abutment-side)
F_u (the splice plates and bottom flange) = 50 ksi

At the bottom flange splice plate connection:

∑ (inner and outer splice plate thicknesses) × F_u

> thickness of the thinner bottom flange × F_u

Therefore, the bearing capacity of the bottom flange is evaluated.

For the end bolts:

A 1/4 in. gap is provided between girder sections at the splice location.

Clear end distance, L_c = (4.5 – 0.25 – 1.125)/2 = 1.5625 in.

Since the clear end distance of bolt holes < 2.0d = 2 × 1 = 2 in.,

$R_n = 1.2L_ctF_u$ = 1.2 (1.5625) (1.625) (65) = 198 kips

There are 4 bolts adjacent to the end of the bottom flange. **Figure 41**

Capacity (R_n) for the 4 end bolts = 4 (198) = 792 kips

Factored bearing resistance of the bolts, R_r

$R_r = \phi_{bb}R_n$ = (0.80) (792) = 634 kips

The factored shear resistance of the bottom flange bolt = 61.1 kip/bolt **Step 9.1.2.1.1**

The total factored shear resistance of 4 end bolts = 4 (61.1) = 244.4 kips

R_r = 634 kips > 244.4 kips **OK**

Shear capacity of bolts at the end of the plate controls the capacity.

For the interior bolts:

Clear distance between interior holes, L_c = 3.5 – 1.125 = 2.375 in.

Since the clear end distance between two interior bolt holes > 2.0d = 2 × 1 = 2 in.,

$R_n = 2.4dtF_u$ = 2.4 (1.0) (1.625) (65) = 253.5 kips

There are 16 interior bolts in the bottom flange. **Figure 41**

Capacity (R_n) of the 16 interior bolts = 16 (253.5) = 4,056 kips

The factored bearing resistance of the bolts, R_r

$R_r = \phi_{bb}R_n$ = (0.80) (4056) = 3,245 kips

The total factored shear resistance of 16 interior bolts = 16 (61.1) = 977.6 kips

R_r = 3,245 kips > 977.6 kips

Therefore, the flange interior bolt bearing capacity is greater than the bolt shear capacity.

For end and interior bolts:

The total factored bolt shear resistance at the bottom flange, R_r

$$= 244.4 + 977.6 \quad = 1{,}222 \text{ kips}$$

Design yield resistance of the bottom flange, $P_{fy\,(Bot)} = 1{,}201$ kips. **Step 9.1.2.1.1**

$R_r = 1{,}222$ kips $> P_{fy} = 1{,}201$ kips **OK**

The bottom flange capacity is controlled by flange yielding.

Step 9.1.2.1.4. Moment resistance

This step presents the moment resistance provided by the flanges and the moment expected to be resisted by the web splice.

For composite sections in negative flexure and noncomposite sections subject to positive or negative flexure, the moment resistance provided by the flanges at the splice location is computed as the design yield resistance of each flange (P_{fy}) for the top or bottom flange, whichever is smaller, times the moment arm, A. LRFD Art. C6.13.6.1.3b

The moment resistance provided by the flange splices shall be checked against the factored moment at the strength limit state at the splice location. When the factored moment exceeds the moment resistance provided by the flange splices, the additional moment shall be resisted by the web splices. LRFD Art. 6.13.6.1.3b

Figure 43 shows flange forces and the moment arm of a noncomposite section. The moment arm, A, is the distance between the mid-thicknesses of the top and bottom flanges.

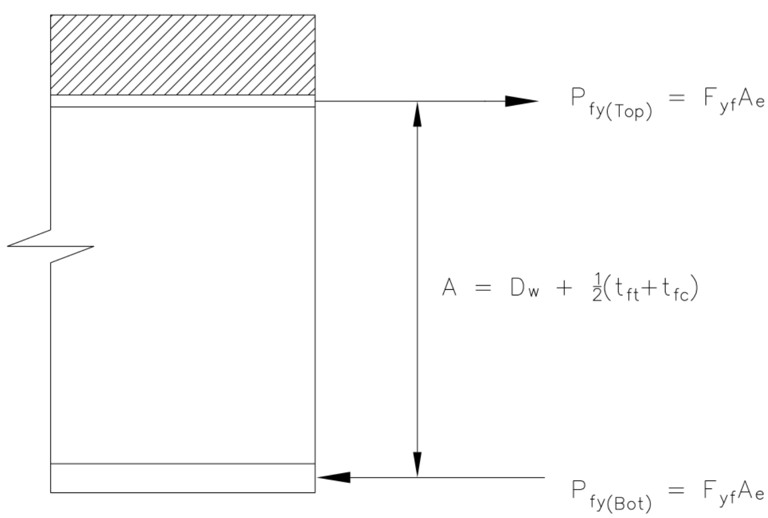


Figure 43. Calculation of the moment resistance provided by the flange splices for composite sections in positive flexure or noncomposite sections according to LRFD Art C6.13.6.1.3b-2

For positive moment:

Factored positive moment under strength limit state, M_u = 2178 kip-ft **Step 9.1.2.1**

Design yield resistance of the top flange, $P_{fy\ (Top)}$ = 589 kips **Step 9.1.2.1**

Design yield resistance of the bottom flange, $P_{fy\ (Bot)}$ = 1,201 kips **Step 9.1.2.1**

P_{fy} = smaller (589, 1201) = 589 kips LRFD Art. 6.13.6.1.3b

Flange moment arm,

$A = D_w + ½ (t_{ft} + t_{fc})$ = 33 + 1/2(0.75 + 1.625) = 34.1875 in.

The moment resisted by the flanges,

$M_{flange} = P_{fy} \times A$ = 589 × (34.1875/12) = 1,678 kip-ft

M_{flange} = 1,678 kip-ft < M_u = 2,178 kip-ft

Therefore, the additional moment of 500 kip-ft (i.e. 2178 – 1678 kip-ft) shall be resisted by the web splices. LRFD Art. 6.13.6.1.3b

For negative moment:

Factored negative moment under strength limit state, M_u = |−1535| kip-ft **Step 9.1.2.1**

= 1535 kip-ft

P_{fy} = smaller (589, 1201) = 589 kips LRFD Art. 6.13.6.1.3b

Flange moment arm,

$A = D_w + ½ (t_{ft} + t_{fc})$ = 33 + 1/2(0.75 + 1.625) = 34.1875 in.

The moment resisted by the flanges,

$M_{flange} = P_{fy} \times A$ = 589 × (34.1875/12) = 1,678 kip-ft

M_{flange} = 1,678 kip-ft > M_u = 1,535 kip-ft

Therefore, the capacity of the flanges is adequate, and no additional negative moment shall be resisted by the web splices. LRFD Art. 6.13.6.1.3b

As shown above, the web splice shall be designed to carry an additional positive moment of 500 kip-ft.

Step 9.1.2.1.5. Flange splice plates in tension

The factored resistance, R_r, in tension shall be taken as the least of the values given by yielding, fracture, or block shear rupture resistance. LRFD Art. 6.13.5.2

Gross section yielding:

The factored yield resistance of the splice plate in tension, P_r:

$$P_r = \phi_y F_y A_g$$ LRFD Eq. 6.8.2.1-1

where:

ϕ_y = resistance factor for yielding of tension members = 0.95 LRFD Art. 6.5.4.2

A_g = gross cross-sectional area of the splice plate

F_y = specified minimum yield strength of the splice plate = 50 ksi

The gross area shall be determined as the length of the plane multiplied by the thickness of the component. LRFD Art. 6.13.4

Net-section fracture:

The factored net section fracture resistance of a splice plate in tension, P_r:

$$P_r = \phi_u F_u A_n R_p U$$ LRFD Eq. 6.8.2.1-2

where:

ϕ_u = resistance factor for fracture of tension members = 0.80 LRFD Art. 6.5.4.2

A_n = net area of the splice plate LRFD Art 6.8.3

$\leq 0.85A_g$ LRFD Art. C6.13.5.2

F_u = tensile strength of the splice plate = 65 ksi

R_p = reduction factor for holes = 1.0 (drilled full size) LRFD Art. 6.8.2.1

U = reduction factor to account for shear lag LRFD Table 6.8.2.2-1
= 1.0 (for splice plates)

The net area, A_n, of an element is the product of the thickness of the element and its smallest net width. The width of each standard bolt hole shall be taken as the nominal diameter of the hole. The net width shall be determined for each chain of hole extending across the member or element along any transverse, diagonal, or zigzag line. LRFD Art. 6.8.3

The net width for each chain shall be determined by subtracting from the width of the element the sum of the widths of all holes in the chain and adding the quantity $s^2/4g$ for each space between consecutive holes in the chain, where:

s = pitch of any two consecutive holes
g = gage of the same two holes

Appendix F.1.1.1 and **F.1.1.2** show two examples. **Appendix F.1.1.1** shows the procedure to calculate the net area of the girder top flange. **Appendix F.1.1.2** shows the procedure to calculate the net area of the girder bottom flange.

Block shear rupture resistance:

The factored block shear rupture resistance, R_r: LRFD Art. 6.13.4

$$R_r = \phi_{bs} R_p (0.58 F_u A_{vn} + U_{bs} F_u A_{tn}) \leq \phi_{bs} R_p \left(0.58 F_y A_{vg} + U_{bs} F_u A_{tn}\right)$$ LRFD Eq. 6.13.4-1

where:

ϕ_{bs} = resistance factor for block shear rupture = 0.80 LRFD Art. 6.5.4.2

A_{tn} = net area along the plane resisting tension stress

A_{vg} = gross area along the plane resisting shear stress

A_{vn} = net area along the plane resisting shear stress

U_{bs} = reduction factor for block shear rupture resistance LRFD Art. 6.13.4
= 1.0 (for uniform tension stress)

The net area, A_n, shall be the gross area, A_g, minus the number of whole or fractional holes in the plane, multiplied by the nominal hole diameter times the thickness of the component. In determining the net section of cuts carrying tension stress, the effect of staggered holes adjacent to the cuts shall be determined as in Art. 6.8.3. For the net section carrying shear stress, the full effective diameter of holes centered within two diameters of the cut shall be deducted. Holes further removed may be disregarded. LRFD Art. 6.13.4

The A_n calculation for the splice plates is similar to that of the flanges presented in **Appendix F.1.1**.

The gross area is calculated by multiplying the length of the plane by the thickness of the component. LRFD Art. 6.13.4

In this example, both top and bottom splice plates can experience tension due to maximum positive and negative moments. Therefore, both top and bottom flange splice plates are checked for tension.

<u>Top flange splice plates</u>

Outer plate:

A_g = 18 × 0.5 = 9 in^2

Check the failure plane defined in **Appendix F.1.1.1.** for A_n.

A_n = [18 – 4 × 1.125 + 2(2.75^2/(4 × 4.5))] × 0.5 = 7.17 in^2

A_n = 7.17 in^2 < 0.85 A_g = 0.85(9) = 7.65 $in.^2$ **OK**

Design force carried by the top flange outer plate, $P_{fy\ (Top\text{-}outer)}$ = 312 kips **Step 9.1.2.1.1**

The factored yield resistance of the splice plate in tension, P_r,

$P_r = \phi_y F_y A_g = 0.95 \times 50 \times 9$ = 427.5 kips

P_r = 427.5 kips > $P_{fy\ (Top\text{-}outer)}$ = 312 kips **OK**

The factored net section fracture resistance of a splice plate in tension, P_r

$P_r = \phi_u F_u A_n R_p U = 0.80 \times 65 \times 7.17 \times 1.0 \times 1.0$ = 373 kips

P_r = 373 kips > $P_{fy\ (Top\text{-}outer)}$ = 312 kips **OK**

The connection shall be investigated by considering all possible failure planes. Such planes shall include those parallel and perpendicular to the applied forces. The planes parallel to the applied force shall be considered to resist only shear stresses. The planes perpendicular to the applied force shall be considered to resist only tension stresses. LRFD Art. 6.13.4

It is the Engineer's responsibility to evaluate all possible failure planes and block shear capacities. The failure pattern that provides the least capacity controls the design.

Figure 44 shows the block shear failure mode of the top flange outer plate at the splice connection. L_t is the length of the tension plane, and L_v is the length of shear plane.

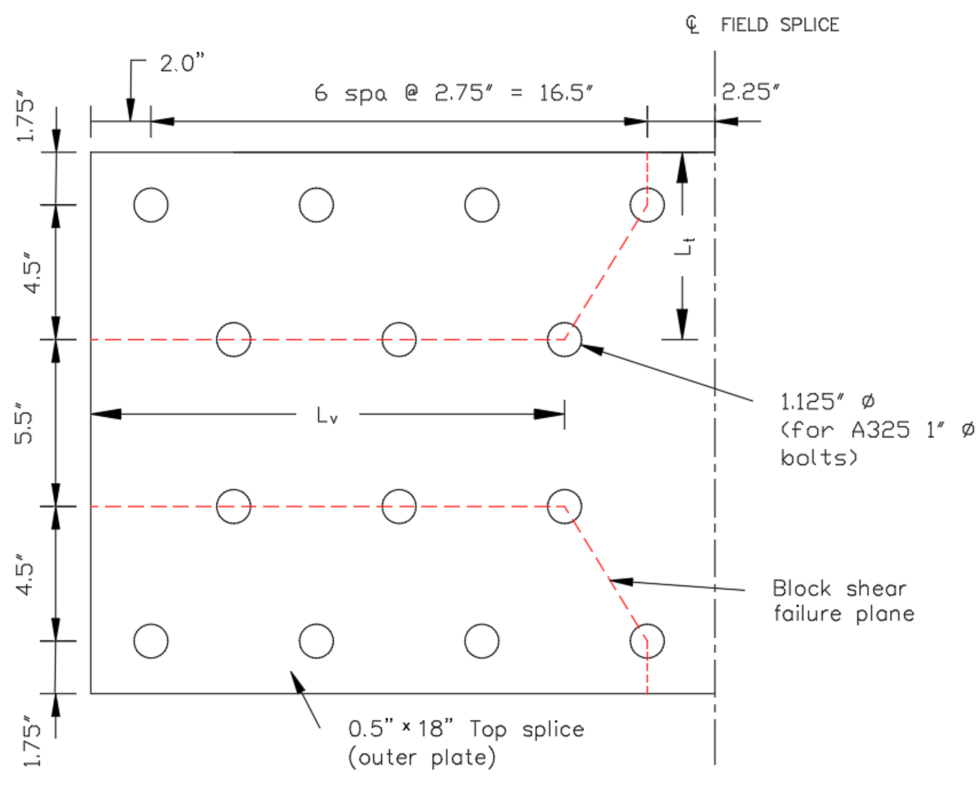


Figure 44. The block shear failure mode of the top flange outer splice plate

L_t	$= 1.75 + 4.5$	$= 6.25$ in.
A_{tn}	$= 2 \times [6.25 - 1.5(1.125) + (2.75^2/(4 \times 4.5))] \times 0.5$	$= 4.98$ in^2
L_v	$= 2 + 5(2.75)$	$= 15.75$ in.
A_{vg}	$= 2 \times 15.75 \times 0.5$	$= 15.75$ in^2
A_{vn}	$= 2 \times [15.75 - 2.5(1.125)] \times 0.5$	$= 12.9375$ in^2

$$R_r = \phi_{bs} R_p (0.58 F_u A_{vn} + U_{bs} F_u A_{tn})$$

$$= 0.80 \times 1.0 \times [0.58(65)(12.9375) + (1.0)(65)(4.98)] \quad = 649 \text{ kips}$$

$$> \phi_{bs} R_p (0.58 F_y A_{vg} + U_{bs} F_u A_{tn})$$

$$= 0.80 \times 1.0 \times [0.58(50)(15.75) + (1.0)(65)(4.98)] \quad = 624 \text{ kips}$$

$R_r = 624$ kips $> P_{fy\ (Top\text{-}outer)} = 312$ kips **OK**

Inner plates:

Figure 45 shows the top flange inner plates at the splice connection. The figure shows the net section of tension failure. This failure plane is defined by considering the bolt line that carries the maximum force acting on the plate.

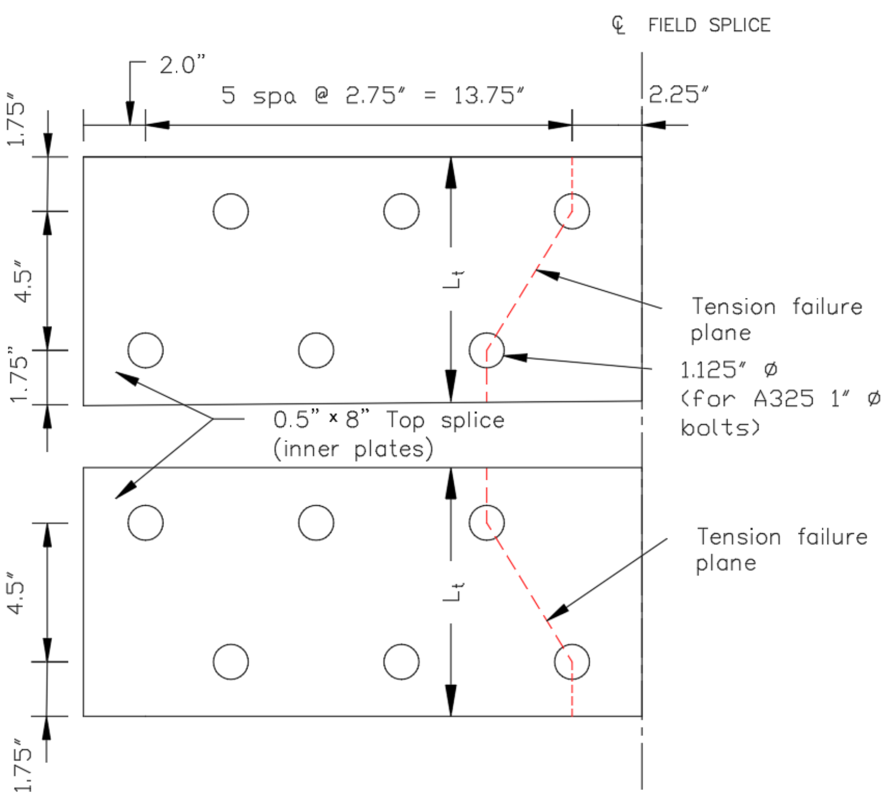


Figure 45. Net tension failure planes of the inner top splice plates

A_g	$= 2(8 \times 0.5)$	$= 8$ in^2
L_t		$= 8$ in.
A_n	$= 2\ [8 - 2 \times 1.125 + (2.75^2/(4 \times 4.5))] \times 0.5)$	$= 6.17$ in^2
A_n	$= 6.17$ in$^2 < 0.85\ A_g = 0.85(8) = 6.8$ in^2	**OK**

The design force carried by the top flange inner plates, $P_{fy\ (Top\text{-}inner)}$ = 277 kips **Step 9.1.2.1.1**

The factored yield resistance of the splice plate in tension, P_r,

$P_r = \phi_y F_y A_g = 0.95 \times 50 \times 8$ = 380 kips

P_r = 380 kips > $P_{fy\ (Top\text{-}inner)}$ = 277 kips **OK**

The factored net section fracture resistance of a splice plate in tension, P_r

$P_r = \phi_u F_u A_n R_p U = 0.80 \times 65 \times 6.17 \times 1.0 \times 1.0$ = 321 kips

P_r = 321 kips > $P_{fy\ (Top\text{-}inner)}$ = 277 kips **OK**

Figure 46 shows the block shear failure mode of the top flange inner plates.

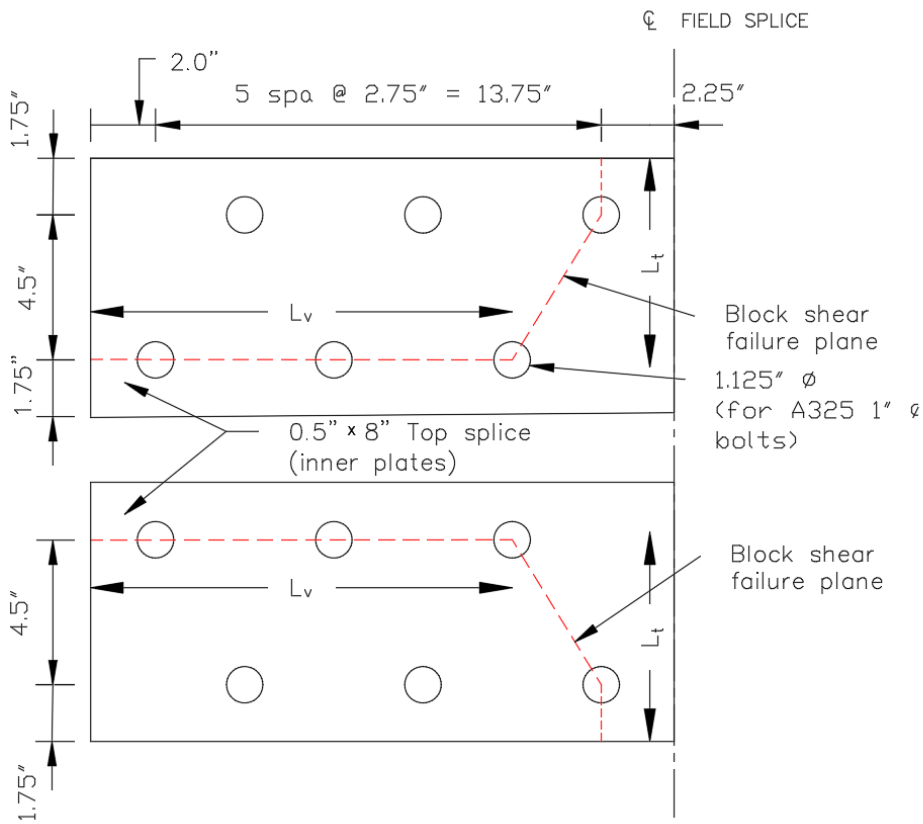


Figure 46. Block shear failure mode of the inner top splice plates

L_t = 1.75 + 4.5 = 6.25 in.

A_{tn} = 2 × [6.25 – 1.5(1.125) + (2.75²/(4 × 4.5))] × 0.5 = 4.98 in²

L_v = 2 + 5(2.75)] = 15.75 in

A_{vg} = 2 × 15.75 × 0.5 = 15.75 in²

A_{vn} = 2 × [15.75 – 2.5(1.125)] × 0.5 = 12.9375 in²

The factored block shear rupture resistance, R_r,

$R_r = \phi_{bs}R_p(0.58F_uA_{vn} + U_{bs}F_uA_{tn})$

$= 0.80 \times 1.0 \times [0.58(65)(12.9375) + (1.0)(65)(4.98)]$ = 649 kips

$> \phi_{bs}R_p(0.58F_yA_{vg} + U_{bs}F_uA_{tn})$

$= 0.80 \times 1.0 \times [0.58(50)(15.75) + (1.0)(65)(4.98)]$ = 624 kips

R_r = 624 kips > $P_{fy\ (Top\text{-}inner)}$ = 277 kips **OK**

<u>Bottom flange splice plates</u>

Outer plate:

A_g = 18 × 0.875 = 15.75 $in.^2$

Check the failure plane defined in **Appendix F.1.1.2.** for A_n.

A_n = (18 – 4 × 1.125) × 0.875 = 11.8125 $in.^2$

A_n = 11.8125 $in.^2$ < 0.85 A_g = 0.85(15.75) = 13.3875 $in.^2$ **OK**

The design force carried by the bottom flange outer plate, $P_{fy\ (Bot\text{-}outer)}$ = 636 kips **Step 9.1.2.1.1**

The factored yield resistance of the splice plate in tension, P_r,

P_r $= \phi_yF_yA_g = 0.95 \times 50 \times 15.75$ = 748.125 kips

P_r = 748.125 kips > $P_{fy\ (Bot\text{-}outer)}$ = 636 kips **OK**

The factored net section fracture resistance of a splice plate in tension, P_r

P_r $= \phi_uF_uA_nR_pU = 0.80 \times 65 \times 11.8125 \times 1.0 \times 1.0$ = 614.25 kips

P_r = 614.25 kips < $P_{fy\ (Bot\text{-}outer)}$ = 636 kips **NOT OK**

Plate fracture controls.

Use 1.0 in. thick splice plates at the bottom flange to satisfy the net section fracture resistance requirement. This will not alter the forces carried by each bottom flange splice plate if the inner and outer plate thicknesses are the same.

The revised calculation is shown below:

A_g = 18 × 1.0 = 18 $in.^2$

A_n = (18 – 4 × 1.125) × 1.0 = 13.5 $in.^2$

A_n = 13.5 $in.^2$ < 0.85 A_g = 0.85(18) = 15.3 $in.^2$ **OK**

The factored yield resistance of the splice plate in tension, P_r,

P_r $= \phi_yF_yA_g = 0.95 \times 50 \times 18$ = 855 kips

P_r = 855 kips > $P_{fy\ (Bot\text{-}outer)}$ = 636 kips **OK**

The factored net section fracture resistance of a splice plate in tension, P_r

P_r $= \phi_uF_uA_nR_pU = 0.80 \times 65 \times 13.5 \times 1.0 \times 1.0$ = 702 kips

P_r = 702 kips > $P_{fy\ (Bot\text{-}outer)}$ = 636 kips **OK**

Figure 47 shows the block shear failure mode of the bottom flange outer splice plates.

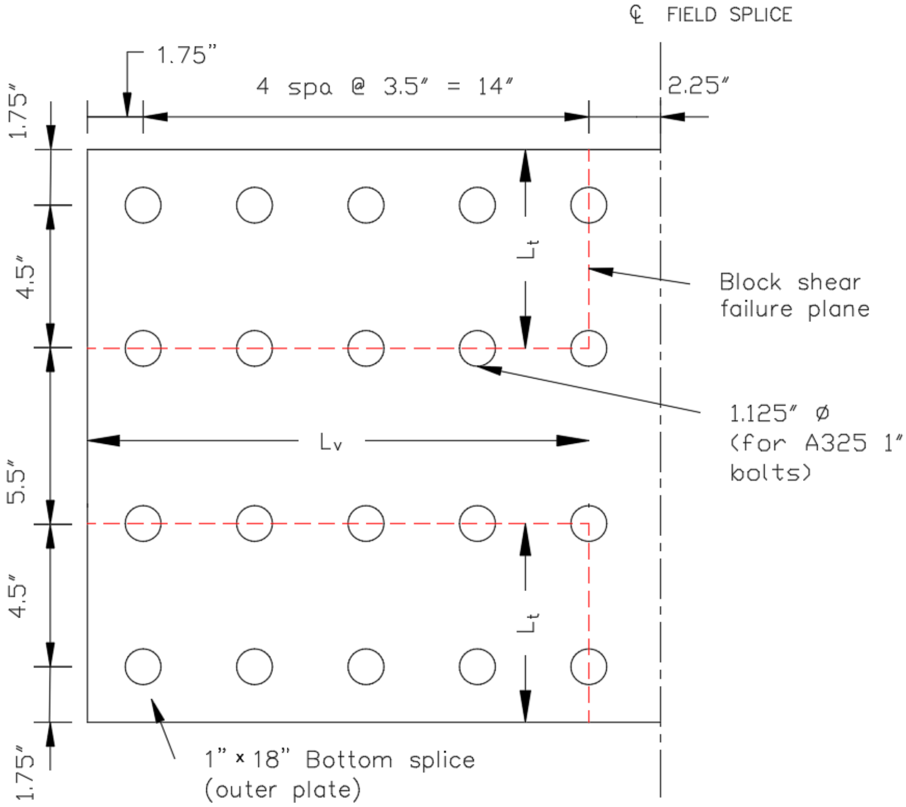


Figure 47. Block shear failure mode of the bottom flange outer splice plate

L_t = 1.75 + 4.5 = 6.25 in.

A_{tn} = 2 × [6.25 – 1.5(1.125)] × 1.0 = 9.125 in^2

L_v = 1.75 + 14 = 15.75 in.

A_{vg} = 2 × 15.75 × 1.0 = 31.5 in^2

A_{vn} = 2 × [15.75 – 4.5(1.125)] × 1.0 = 21.375 in^2

The factored block shear rupture resistance, R_r,

$$R_r = \phi_{bs}R_p(0.58F_uA_{vn} + U_{bs}F_uA_{tn})$$

$$= 0.80 \times 1.0 \times [0.58(65)(21.375) + (1.0)(65)(9.125)] = 1{,}119 \text{ kips}$$

$$< \phi_{bs}R_p\left(0.58F_yA_{vg} + U_{bs}F_uA_{tn}\right)$$

$$= 0.80 \times 1.0 \times [0.58(50)(31.5) + (1.0)(65)(9.125)] = 1{,}205 \text{ kips}$$

R_r = 1,119 kips > $P_{fy\ (Bot\text{-}outer)}$ = 636 kips **OK**

Inner plates:

Figure 48 shows the bottom flange inner plates at the splice connection. The figure shows the net tension failure plane.

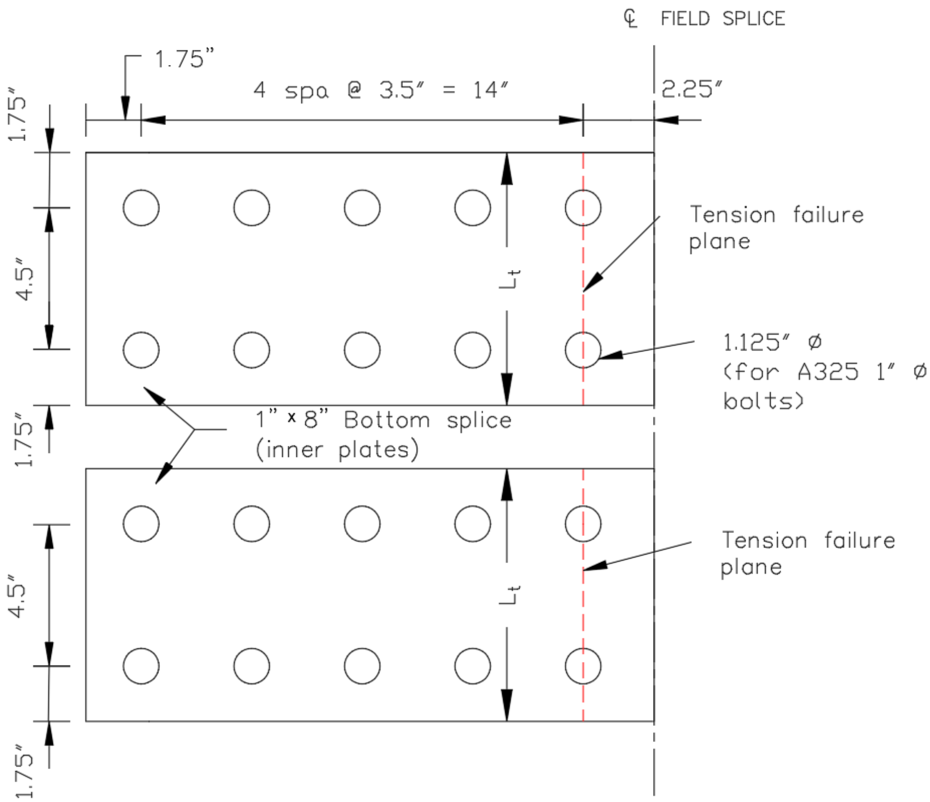


Figure 48. Net tension failure planes of the inner bottom splice plates

1.0 in. thick plates are used.

A_g = 2(8 × 1.0) = 16 in.^{2}

L_t = 8 in.

A_n = 2 [8 – 2 × 1.125] × 1.0 = 11.5 in.^{2}

A_n = 11.5 in.^{2} < 0.85 A_g = 0.85(16) = 13.6 in.^{2} **OK**

The design force carried by the bottom flange outer plate, $P_{fy\ (Bot\text{-}inner)}$ = 565 kips **Step 9.1.2.1.1**

The factored yield resistance of the splice plate in tension, P_r,

P_r $= \phi_y F_y A_g = 0.95 \times 50 \times 16$ = 760 kips

P_r = 760 kips > $P_{fy\ (Bot\text{-}outer)}$ = 565 kips **OK**

The factored net section fracture resistance of a splice plate in tension, P_r

P_r $= \phi_u F_u A_n R_p U = 0.80 \times 65 \times 11.5 \times 1.0 \times 1.0$ = 598 kips

P_r = 598 kips > $P_{fy\ (Bot\text{-}inner)}$ = 565 kips **OK**

Figure 49 shows the block shear failure mode of the bottom flange inner splice plates.

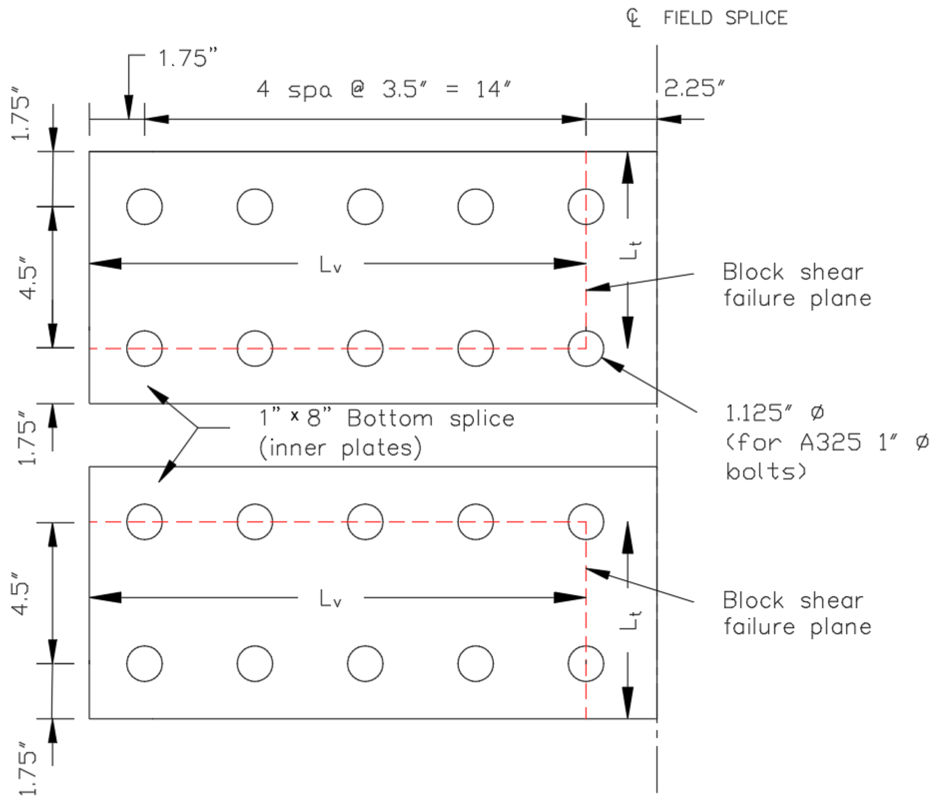


Figure 49. Block shear failure mode of the bottom flange inner splice plates

L_t	= 1.75 + 4.5	= 6.25 in.
A_{tn}	= 2 × [6.25 – 1.5(1.125)] × 1.0	= 9.125 in^2
L_v	= 1.75 + 14	= 15.75 in.
A_{vn}	= 2 × [15.75 – 4.5(1.125)] × 1.0	= 21.375 in^2
A_{vg}	= 2 × 15.75 × 1.0	= 31.5 in^2

The factored block shear rupture resistance, Rr,

$$R_r = \phi_{bs} R_p (0.58 F_u A_{vn} + U_{bs} F_u A_{tn})$$

$$= 0.80 \times 1.0 \times [0.58(65)(21.375) + (1.0)(65)(9.125)] \quad = 1{,}119 \text{ kips}$$

$$< \phi_{bs} R_p \left(0.58 F_y A_{vg} + U_{bs} F_u A_{tn}\right)$$

$$= 0.80 \times 1.0 \times [0.58(50)(31.5) + (1.0)(65)(9.125)] \quad = 1{,}205 \text{ kips}$$

R_r = 1,119 kips > $P_{fy\ (Bot\text{-}inner)}$ = 565 kips **OK**

Step 9.1.2.1.6. Splice plates in compression

Flange splice plates subjected to compression at the strength limit state are to be checked only for yielding on the gross section of the plates. LRFD Art. C6.13.6.1.3b

Yield resistance of splice plates in compression,

$$R_r = \phi_c F_y A_s$$

LRFD Eq. 6.13.6.1.3b-3

where:

ϕ_c = resistance factor for compression members = 0.95 — LRFD Art. 6.5.4.2

A_s = gross area of the splice plate

F_y = specified minimum yield strength of the splice plate = 50 ksi

In this example, both top and bottom splice plates can experience compression due to the maximum positive and negative moments. Therefore, splice plates at both flanges are checked for compression resistance.

<u>Top splice plates</u>

A_s	= 18 × 0.5 + 2 × 8 × 0.5	= 17 in.^{2}
$R_r = \phi_c F_y A_s$	= 0.95 × 50 × 17	= 808 kips

Design yield resistance of top flange, $P_{fy\,(Top)}$ = 589 kips — **Step 9.1.2.1.1**

R_r = 808 kips > $P_{fy\,(Top)}$ = 589 kips **OK**

<u>Bottom splice plates</u>

A_s	= 18 × 1.0 + 2 × 8 × 1.0	= 34 in.^{2}
$R_r = \phi_c F_y A_s$	= 0.95 × 50 × 34	= 1,615 kips

Design yield resistance of bottom flange, $P_{fy\,(Bot)}$ = 1,201 kips — **Step 9.1.2.1.1**

R_r = 1,615 kips > $P_{fy\,(Bot)}$ = 1,201 kips **OK**

Step 9.1.2.2. Slip resistance check

The moment resistance provided by the nominal slip resistance of the flange splice bolts is checked against the factored moment for checking slip. The factored moment for checking slip is taken as the moment at the splice location under the Service II limit state or due to the deck casting sequence, whichever governs. — LRFD Art. 6.13.6.1.3b

Factored moment under Service II limit state

Positive moment = 1.0 (-150 + 13) + 1.0 (20) + 1.30 (1295) = 1,567 kip-ft — **Step 9.1.1.5.1**

Negative moment = 1.0 (-150 +13) + 1.0(20) + 1.30 (-729) = -1,065 kip-ft — **Step 9.1.1.5.1**

Factored moment due to deck pour = 1.4 (87) = 121.8 kip-ft — **Step 9.1.1.5.1**

Therefore, the Service II limit state moments govern the slip resistance.

The nominal slip resistance of a bolt in a slip critical connection, R_n: — LRFD Art. 6.13.2.8

$$R_n = K_h K_s N_s P_t$$ — LRFD Eq. 6.13.2.8-1

where:

- K_h = hole size factor = 1.0 — LRFD Table 6.13.2.8-2
- K_s = surface condition factor = 0.5 (for surface condition B) — LRFD Table 6.13.2.8-3
- N_s = number of slip planes per bolt = 2.0
- P_t = minimum required bolt tension = 51 kips — LRFD Table 6.13.2.8-1

When checking for the slip of a connection for a flange splice with inner and outer splice plates, the slip resistance should always be determined by dividing the flange slip force equally to the two slip planes regardless of the ratio of the splice plate areas. Connection slip cannot occur unless the slip occurs on both planes. — LRFD Art. C6.13.6.1.3b

The Number of Slip Planes per Bolt used in BDS = 2.0
The Hole Size Factor, K_h used in BDS = 1.00, (assuming standard holes)

A surface condition B comprises unpainted blast-cleaned surfaces to SSPC-SP6 or better and blast cleaned surfaces with Class B coatings or unsealed pure zinc or 85/15 zinc/aluminum thermal sprayed coatings with a thickness less than or equal to 16 mils. — LRFD Art. 6.13.2.8

K_s = 0.50 is used in BDS, because Section 716.03.B.2.a of the Standard Specifications for Construction requires maintaining a Class B surface at slip critical connections. — SSFC 716.03.B.2.a

MDOT uses the following equation:

$$R_n = 1.0 \times 0.5 \times 2.0 \times P_t$$

MDOT currently uses only A325 bolts due to galvanization issues of A490 bolts.

The computed flange slip force shall be divided by the nominal slip resistance of the bolts, determined as in Art. 6.13.2.8, to determine the total number of flange splice bolts on one side of the splice to resist slip. The flange slip force is determined as the factored moment at the splice location divided by the appropriate moment arm. — LRFD Art. 6.13.6.1.3b

If the flange bolts are not sufficient to resist the factored moment for checking slip at the splice location under the Service II limit state, an additional moment is to be resisted by the web splices. Only if the web bolts are not sufficient to resist this additional moment, additional bolts are to be added to the flange splices. AISC (2017)

The slip resistance of the connection is not affected by the filler plates. Therefore, for slip critical connections, the nominal slip resistance of the bolts is not to be adjusted for the effect of the fillers. LRFD Art. C6.13.6.1.4

The following steps show the slip resistance check at the top and bottom flanges.

Nominal slip resistance of a bolt in a slip critical connection,

$R_n = K_h K_s N_s P_t = (1.0)(0.5)(2.0)(51)$ = 51 kips

In this example, the 14 splice bolts used in the top flange and the 20 splice bolts used in the bottom flange are checked for slip resistance.

R_n for top flange splice bolts	= 14 × 51	= 714 kips
R_n for bottom flange splice bolts	= 20 × 51	= 1,020 kips

For both positive and negative moments: **Step 9.1.2.1.4**

The moment arm	= 34.1875 in.	
Moment in the top flange	= 714 × (34.1875/12)	= 2,034 kip-ft
Moment in the bottom flange	= 1020 × (34.1875/12)	= 2,906 kip-ft

The section is noncomposite at the splice location. Therefore, the smaller of the nominal slip resistance of the top and bottom flange splice is used.

M_n = min (2034, 2906) = 2,034 kip-ft

Under Service II positive moment

M_u = 1,567 kip-ft < M_n = 2,034 kip-ft **OK**

Under Service II negative moment

$M_u = |-1{,}065|$kip-ft < M_n = 2,034 kip-ft **OK**

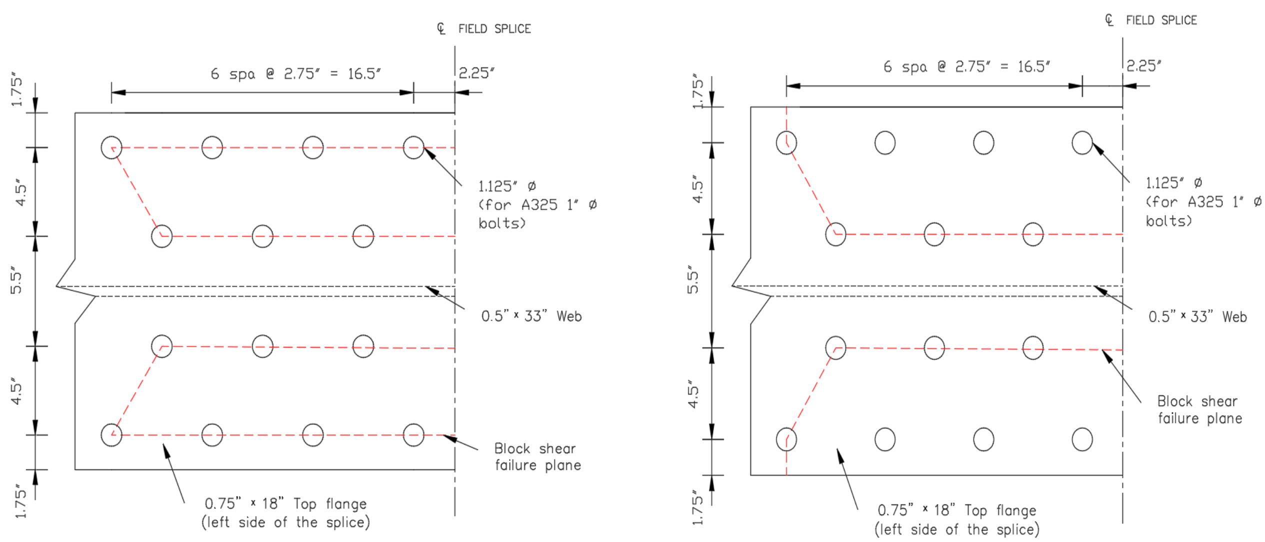
Step 9.1.2.3. Block shear rupture resistance of girder flanges

At the splice connection, the block shear rupture resistance of the critical girder flange under tension is checked. LRFD Art. 6.13.4

In this example, the block shear rupture resistance of the girder flanges on the "abutment-side" of the splice is checked since it has a smaller cross-section area compared to the "pier-side." Top and bottom flanges of the girder can sustain tension due to the applied maximum positive and negative moments. The block shear rupture resistance calculation for the flanges is similar to the block shear rupture resistance calculation for splice plates in **Step 9.1.2.1.5.**

Top flange:

Two potential failure modes, shown in **Figure 50**, are investigated for the top flange.



(a) Failure mode I **(b) Failure mode II**

Figure 50. Top flange block shear failure modes (Abutment-side)

For failure mode I:

$A_{tn} = 2 \times [4.5 - 1.125 + (2.75^2/(4 \times 4.5))] \times 0.75 = 5.693 \text{ in}^2$

$A_{vg} = 2 \times [6(2.75) + 2.25] \times 0.75 + 2 \times [5(2.75) + 2.25] \times 0.75 = 52.125 \text{ in}^2$

$A_{vn} = 2 \times [6(2.75) + 2.25 - 3.5(1.125)] \times 0.75 + 2[5(2.75) + 2.25 - 2.5(1.125)] \times 0.75 = 42 \text{ in}^2$

The factored block shear rupture resistance, R_r,

$$R_r = \phi_{bs} R_p (0.58 F_u A_{vn} + U_{bs} F_u A_{tn})$$

$$= 0.80 \times 1.0 \times [0.58(65)(42) + (1.0)(65)(5.693)] = 1{,}562 \text{ kips}$$

$$> \phi_{bs} R_p (0.58 F_y A_{vg} + U_{bs} F_u A_{tn})$$

$$= 0.80 \times 1.0 \times [0.58(50)(52.125) + (1.0)(65)(5.693)] = 1{,}505 \text{ kips}$$

The design yield resistance of the top flange, $P_{fy\ (Top)}$ = 589 kips **Step 9.1.2.1.1**

R_r = 1,505 kips > $P_{fy\ (Top)}$ = 589 kips **OK**

Flange yielding controls.

For failure mode II:

A_{tn} = 2 × [1.75 + 4.5 – 1.5(1.125) + (2.75²/(4 × 4.5))] × 0.75 = 7.474 in.²

A_{vg} = 2 × [5(2.75) + 2.25] × 0.75 = 24 in.²

A_{vn} = 2 × [5(2.75) + 2.25 – 2.5(1.125)] × 0.75 = 19.781 in.²

The factored block shear rupture resistance, R_r,

$$R_r = \phi_{bs} R_p (0.58 F_u A_{vn} + U_{bs} F_u A_{tn})$$

$$= 0.80 \times 1.0 \times [0.58(65)(19.781) + (1.0)(65)(7.474)] = 985 \text{ kips}$$

$$> \phi_{bs} R_p (0.58 F_y A_{vg} + U_{bs} F_u A_{tn})$$

$$= 0.80 \times 1.0 \times [0.58(50)(24) + (1.0)(65)(7.474)] = 945 \text{ kips}$$

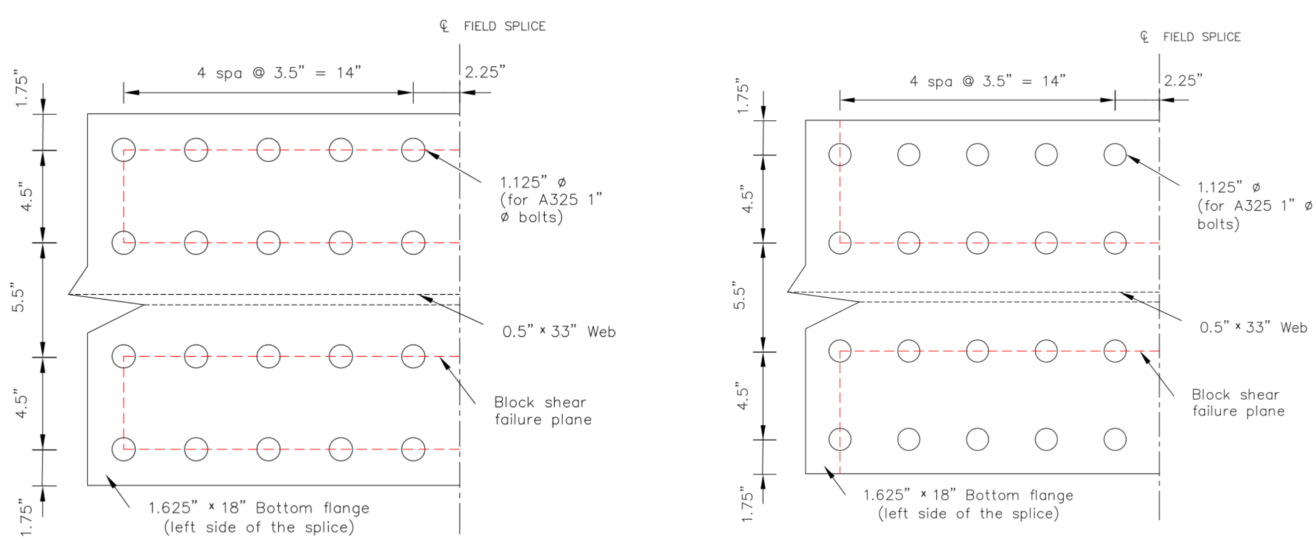
R_r = 945 kips > $P_{fy\ (Top)}$ = 589 kips **OK**

Flange yielding controls.

Since 945 kips < 1,505 kips, the block shear failure mode II controls.

<u>Bottom flange:</u>

Two potential failure modes, shown in **Figure 51**, are investigated for the bottom flange.



(a) Failure mode I **(b) Failure mode II**

Figure 51. Bottom flange b lock shear failure modes (Abutment-side)

For failure mode I:

A_{tn}	$= 2 \times [4.5 - 1.125] \times 1.625$	$= 10.97$ in.^{2}
A_{vg}	$= 4 \times (2.25 + 14) \times 1.625$	$= 105.625$ in.^{2}
A_{vn}	$= 4 \times [2.25 + 14 - 4.5(1.125)] \times 1.625$	$= 72.71$ in.^{2}

The factored block shear rupture resistance, R_r,

$$R_r = \phi_{bs} R_p (0.58 F_u A_{vn} + U_{bs} F_u A_{tn})$$

$$= 0.80 \times 1.0 \times [0.58(65)(72.71) + (1.0)(65)(10.97)] \qquad = 2{,}763 \text{ kips}$$

$$< \phi_{bs} R_p \left(0.58 F_y A_{vg} + U_{bs} F_u A_{tn}\right)$$

$$= 0.80 \times 1.0 \times [0.58(50)(105.625) + (1.0)(65)(10.97)] \qquad = 3{,}021 \text{ kips}$$

The design yield resistance of the bottom flange, $P_{fy\ (Bot)}$ = 1,201 kips **Step 9.1.2.1.1**

$R_r = 2{,}763$ kips $> P_{fy\ (Bot)} = 1{,}201$ kips **OK**

Flange yielding controls.

For failure mode II:

A_{tn}	$= 2 \times [1.75 + 4.5 - 1.5(1.125)] \times 1.625$	$= 14.83$ in.^{2}
A_{vg}	$= 2 \times (2.25 + 14) \times 1.625$	$= 52.81$ in.^{2}
A_{vn}	$= 2 \times [2.25 + 14 - 4.5(1.125)] \times 1.625$	$= 36.36$ in.^{2}

The factored block shear rupture resistance, R_r,

$$R_r = \phi_{bs} R_p (0.58 F_u A_{vn} + U_{bs} F_u A_{tn})$$

$$= 0.80 \times 1.0 \times [0.58(65)(36.36) + (1.0)(65)(14.83)] \qquad = 1{,}868 \text{ kips}$$

$$< \phi_{bs} R_p \left(0.58 F_y A_{vg} + U_{bs} F_u A_{tn}\right)$$

$$= 0.80 \times 1.0 \times [0.58(50)(52.81) + (1.0)(65)(14.83)] \qquad = 1{,}996 \text{ kips}$$

$R_r = 1{,}868$ kips $> P_{fy\ (Bot)} = 1{,}201$ kips **OK**

Flange yielding controls.

Since 1,868 kips < 2,763 kips, the block shear failure mode II controls.

Step 9.1.2.4. Tension in girder flanges with holes

The following check shall be satisfied for cross-sections with holes in the tension flange at strength limit state: LRFD Art. 6.10.1.8

$$f_t \leq 0.84 \left(\frac{A_n}{A_g}\right) F_u \leq F_{yt}$$ LRFD Eq. 6.10.1.8-1

where:

f_t = stress on the gross area of the tension flange due to the factored loads calculated without consideration of flange lateral bending

Since both flanges are in tension under the maximum negative or positive moments under strength limit state, tension in both flanges is checked as shown below:

<u>Top flange</u>

Maximum negative moment under the Strength I combination = -1535 kip-ft **Step 9.1.2.1**

S_t = 583 in.^{3} **Table 3**

$$f_t = \frac{|-1535| \times 12}{583} = 31.6 \text{ ksi}$$

A_n = [18 – 4 × 1.125 + 2 × (2.75^2)/ (4 × 4.5)] × 0.75 = 10.76 in.^{2}

A_g = 18 × 0.75 = 13.5 in.^{2}

$$0.84\left(\frac{A_n}{A_g}\right)F_u = 0.84\left(\frac{10.76}{13.5}\right)65 = 43.5 \text{ ksi}$$

$$f_t = 31.6\ ksi < 0.84\left(\frac{A_n}{A_g}\right)F_u = 43.5\ ksi < F_{yt} = 50\ ksi$$ **OK**

<u>Bottom flange</u>

Maximum positive moment under the Strength I combination = 2178 kip-ft **Step 9.1.2.1**

S_b = 954 in.^{3} **Table 3**

$$f_t = \frac{2178 \times 12}{954} = 27.4 \text{ ksi}$$

A_n = [18 – 4 × 1.125] × 1.625 = 21.9375 in.^{2}

A_g = 18 × 1.625 = 29.25 in.^{2}

$$0.84\left(\frac{A_n}{A_g}\right)F_u = 0.84\left(\frac{21.9375}{29.25}\right)65 = 40.95 \text{ ksi}$$

$$f_t = 27.4\ ksi < 0.84\left(\frac{A_n}{A_g}\right)F_u = 40.95\ ksi < F_{yt} = 50\ ksi$$ **OK**

The top flange splice details shown in **Figure 40** and **Figure 42** satisfy the design requirements. The revised bottom flange splice details satisfying all the design requirements are shown in **Figure 52** and **Figure 53**.

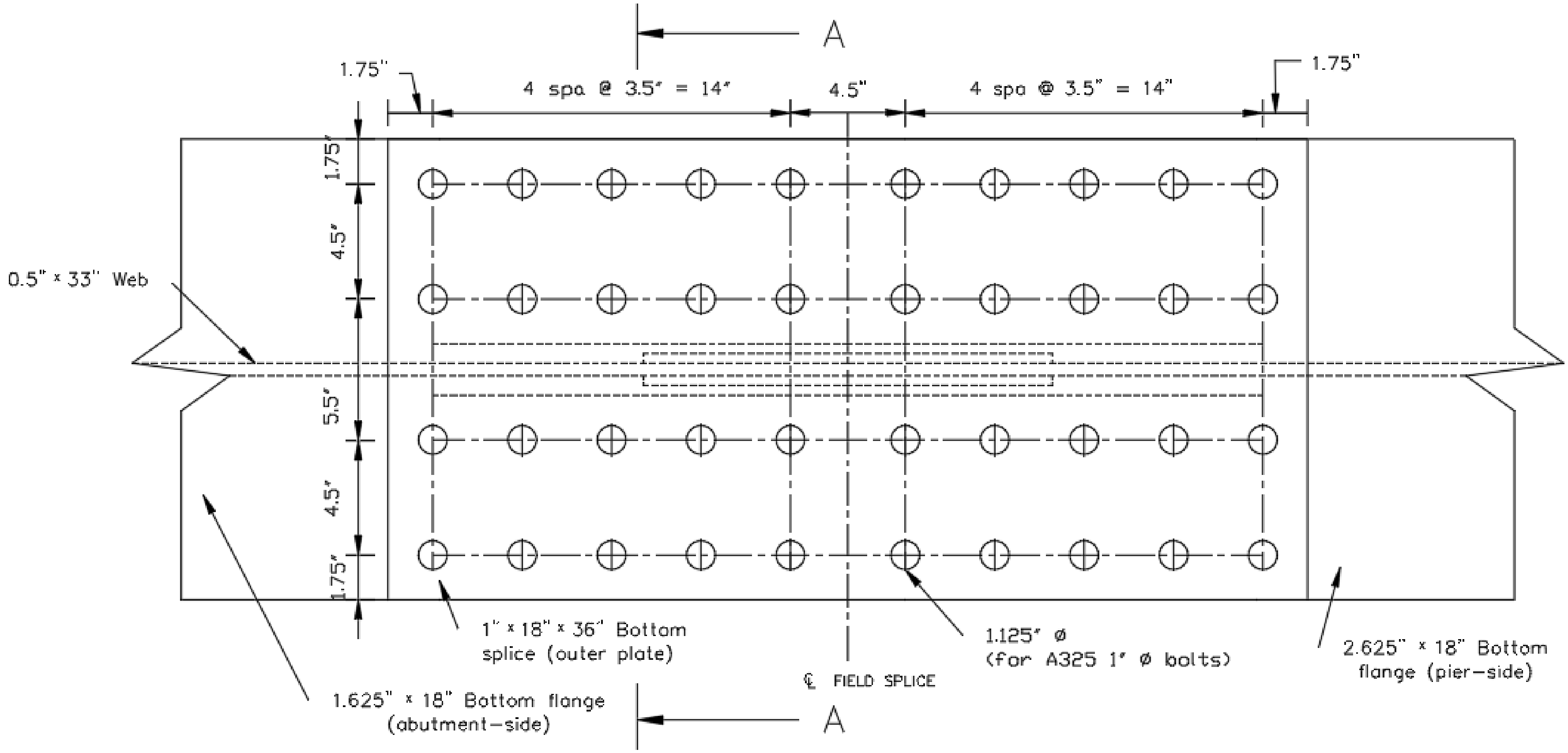


Figure 52. Revised bottom flange splice details

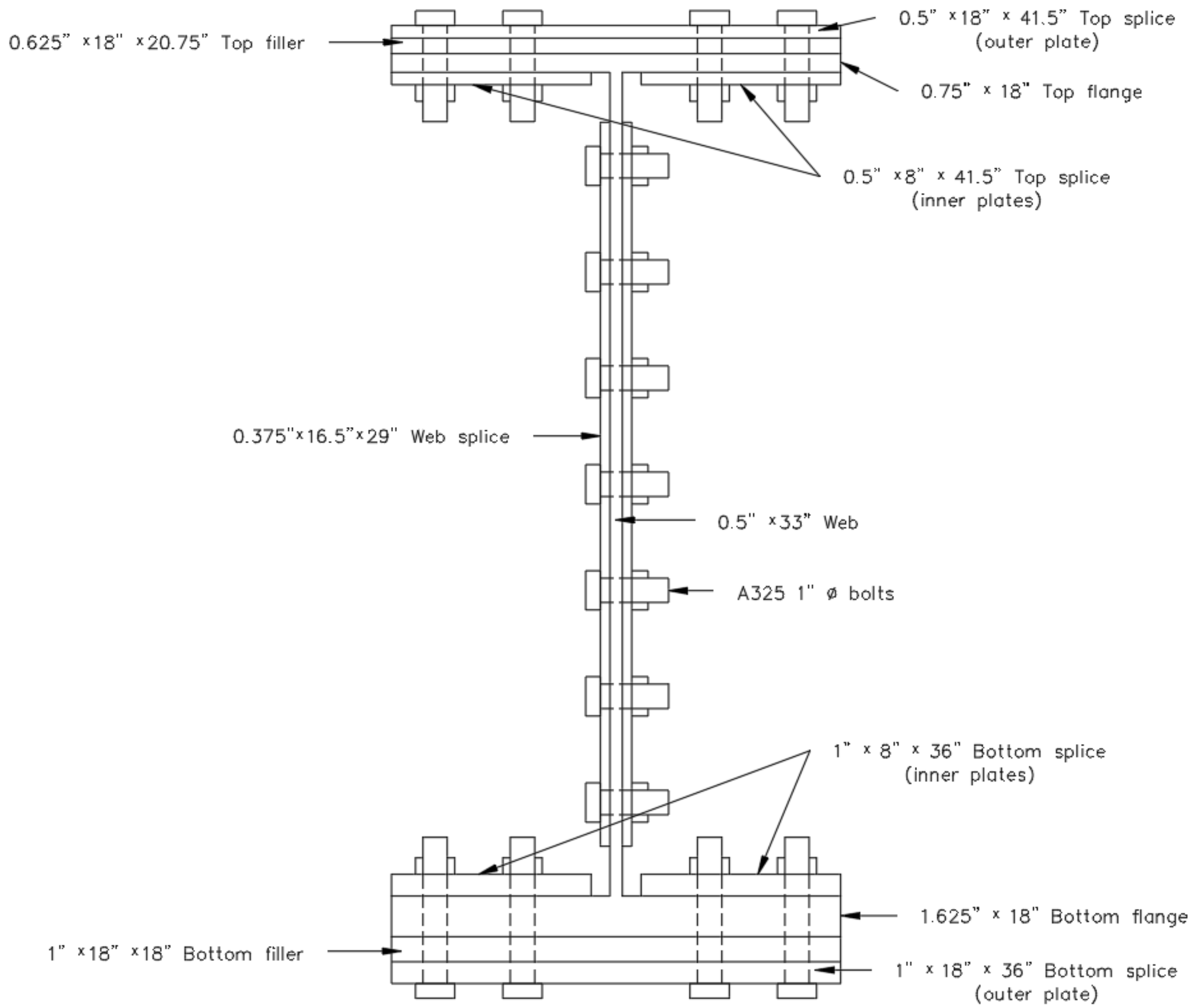


Figure 53. Cross-section of the splice connection with revised bottom splice details (section A-A)

Step 9.1.3. Web splice design

This example presents a detailed design of the exterior girder web splice. One can start the design without a candidate detail to use as a preliminary guide. In such cases, web splice plate dimensions need to be decided after considering the AASHTO LRFD requirements for the minimum thickness of structural steel and the required spacing for fabricating the assembly. After calculating the minimum number of bolts required, a bolt layout can be developed by considering the minimum number for vertical bolt rows, minimum spacing, maximum spacing dictated by the sealing requirements, and minimum edge and end distances.

In this example, a candidate detail is selected from a bridge with comparable dimensions as shown in **Figure 54**. The adequacy of the detail is evaluated for the bridge being designed.

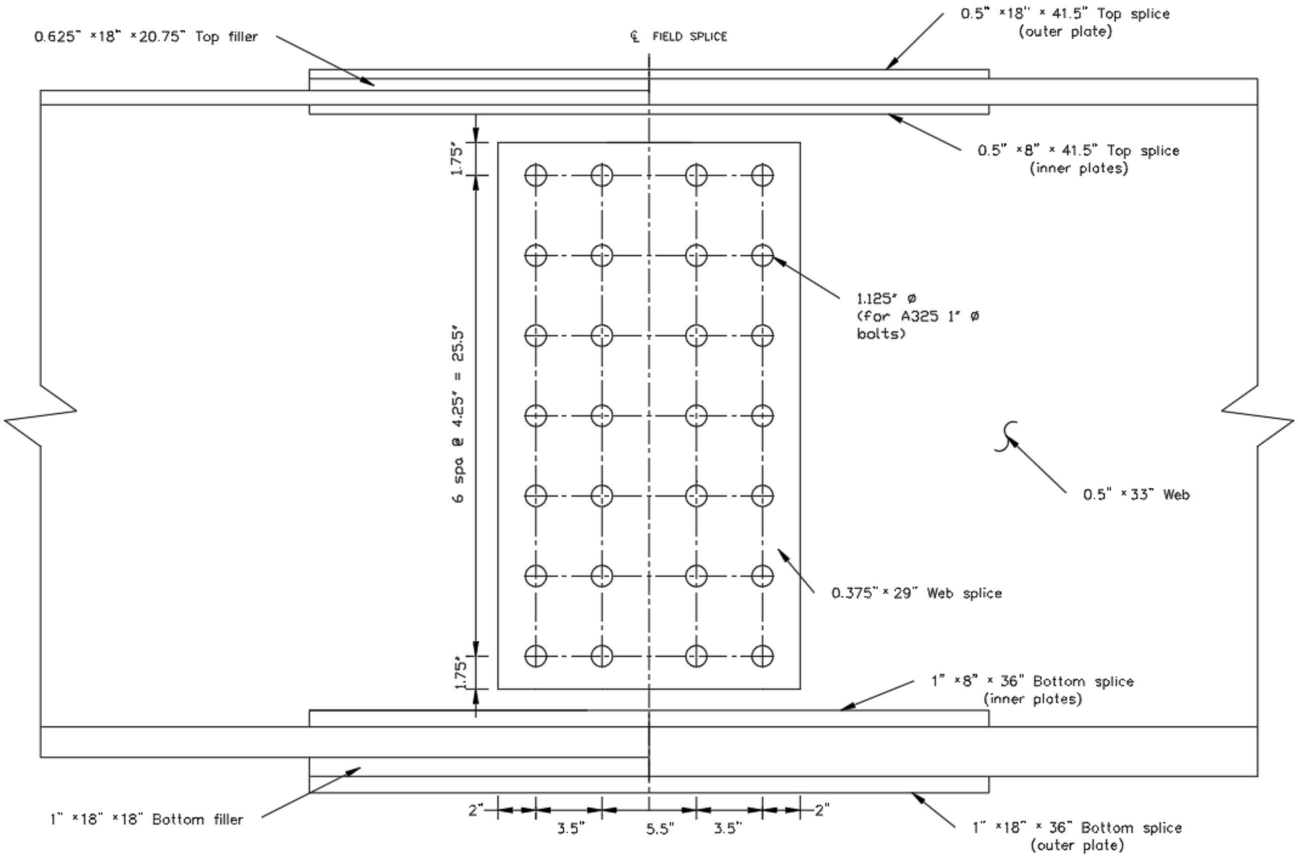


Figure 54. Web splice detail of an exterior girder

Step 9.1.3.1. Strength limit state

As a minimum, the web splice plates and their connections shall be designed at the strength limit state for a design web force taken equal to the smaller factored shear resistance of the web at the splice location. LRFD Art. 6.13.6.1.3c

$$V_u \leq V_r = \phi_v V_n$$

LRFD Eq. 6.10.9.1-1

where:

ϕ_v = resistance factor for shear = 1.0 LRFD Art. 6.5.4.2

V_n = nominal resistance as determined as in LRFD Art. 6.10.9.

V_n = 437 kips **Step 8.1.1.2**

V_u = factored shear in the web at the section under consideration

Factored shear resistance of the web, $V_r = \phi_v V_n = (1.0)(437)$ = 437 kips

Web dimensions on either side of the splice location (left girder and right girder) remain the same. Therefore, V_n is the same at either side of the splice location.

If the moment resistance provided by the flange splices is not adequate to resist the factored moment at the strength limit state at the splice location, the web splice plates and their connections shall instead be designed for a web force taken equal to the vector sum of the smaller factored shear resistance and a horizontal force located in the web that provides necessary moment resistance in conjugation with the flange splices. LRFD Art. 6.13.6.1.3c

As shown in **Step 9.1.2.1.4**, the moment resistance provided by the flange splices is not adequate to resist the splice location factored moment at the strength limit state.

The web moment is the portion of the splice location factored positive moment at the strength limit state that exceeds the moment resistance provided by the flange spices. LRFD Art. C6.13.6.1.3c

Positive web moment, M_w = 500 kip-ft **Step 9.1.2.1.4**

The web force, H_w, is taken as the web moment divided by the appropriate moment arm, A. LRFD Art. C6.13.6.1.3c

For noncomposite sections subjected to positive or negative flexure, the moment arm shall be taken as one quarter of the web depth: LRFD Art. C6.13.6.1.3c

Moment arm, A = D/4 = 33/4 = 8.25 in.

Figure 55 shows the moment arm calculation procedure for a noncomposite section**.**

$H_w = M_w/A$ = (500 × 12) / 8.25 = 727 kips

Design web force, R_w = $\sqrt{437^2 + 727^2}$ = 848 kips

Even though there is no resultant horizontal force acting on the entire section, the same resultant force can be used for the entire system when $H_w/2$ and $V_r/2$ are considered for the top and bottom portions of the web plate.

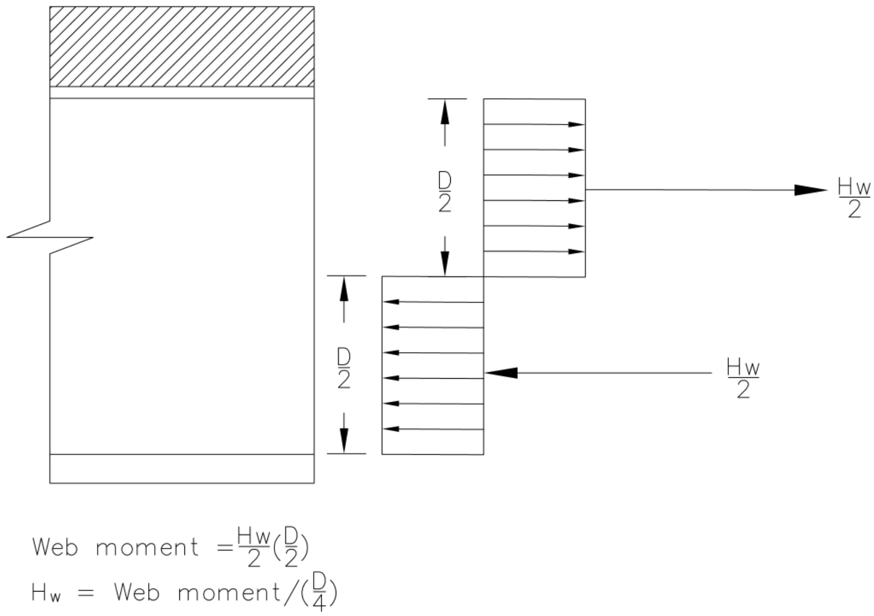


Figure 55. Calculation of horizontal force, H_w, for composite sections subjected to negative flexure and noncomposite sections (ASSHTO LRFD Figure C6.13.6.1.3c-2)

Step 9.1.3.1.1. Web splice bolts

The computed design web force shall be divided by the factored shear resistance of the bolts to determine the number of web splice bolts required on one side of the splice at the strength limit state. LRFD Art. 6.13.6.1.3c

The factored resistance, R_r, of a bolted connection at the strength limit state is: LRFD Art. 6.13.2.7.1

$$R_r = \phi R_n$$

LRFD Eq. 6.13.2.2-2

where:

ϕ = ϕ_s for bolts in shear = 0.8 LRFD Art. 6.5.4.2

R_n = nominal resistance of the bolt, connection, or the connected material

The factored shear resistance of the bolt should be based on the threads included in the shear planes, unless the web-splice plate thickness exceeds 0.5 in. LRFD Art. C6.13.6.1.3c

The thickness of the web splice plate is 0.375 in., which is less than 0.5 in. Therefore, threads are included in the shear plane while calculating the nominal resistance. LRFD Art. C6.13.6.1.3c

$$R_n = 0.45 A_b F_{ub} N_s \quad = 0.45(0.785)(120)(2) \quad = 84.8 \text{ kips}$$
$$R_r = \phi R_n \quad = 0.8(84.8) \quad = 67.8 \text{ kips}$$

In this example, filler plates are not required since the web geometry remains the same throughout the span. Therefore, bolt resistance is not reduced.

Minimum number of web splice bolts required, N = R_w/R_r = 848/67.8 = 13

The splice plates are to extend as near as practical for the full depth between flanges while providing adequate space for bolt assembly.

The required number of bolts is decided after evaluating the minimum thickness of structural steel, bolt spacing, along with end and edge distance requirements.

Step 9.1.3.1.2. Bolt spacing and end and edge distance check for the web splice plate

Splice plate thickness = 0.375 in. > 0.3125 in. **OK** LRFD Art. 6.7.3

The minimum spacing between bolts, s_{min} = 3d = 3.0 in. LRFD Art. 6.13.2.6.1

Minimum spacing provided = 3.5 in. > 3.0 in. **OK**

The maximum spacing between bolts, s_{max} LRFD Eq. 6.13.2.6.2-1

$s \leq (4.0 + 4.0t) \leq 7.0$ in.

$s < (4.0 + 4.0 \times 0.375) = 5.5$ in. < 7.0 in.

Maximum spacing provided = 5.5 in. = 5.5 in. **OK**

The minimum end distance for 1 in. diameter bolts ≥ 1.25 in. LRFD Table 6.13.2.6.6-1

End distance provided = 1.75 in. > 1.25 in. **OK**

The maximum end distance LRFD Art. 6.13.2.6.5

≤ 8 × the thickness of the thinner outside plate or 5 in.

≤ 8 × 0.375 = 3 in. or 5 in.

End distance provided = 1.75 in. < 3.0 in. **OK**

The minimum edge distance for 1 in. diameter bolts ≥ 1.25 in. LRFD Table 6.13.2.6.6-1

Edge distance provided = 2.0 in. > 1.25 in. **OK**

The maximum edge distance LRFD Art. 6.13.2.6.6

≤ 8 × the thickness of the thinner outside plate or 5 in.

≤ 8 × 0.375 = 3 in. or 5 in.

Edge distance provided = 2.0 in. < 3.0 in. **OK**

Considering the required minimum number of bolts, minimum and maximum bolt spacing, and end and edge requirements, ***provide 14 bolts*** on each side of the connection, as shown in **Figure 54**.

Step 9.1.3.1.3. Web splice plates in shear

The design web force at the strength limit state shall not exceed the lesser of the factored shear resistances of the web splice plates due to shear yielding, shear rupture, and block shear rupture. LRFD Art. 6.13.6.1.3c

As per the calculation presented in **Step 9.1.3.1**, the maximum design web force, R_w:

$$R_w = 848 \text{ kips}$$

The factored shear resistance, R_r, of the splice plate shall be taken as the smaller value based on the shear yielding and shear rupture.

Shear yielding:

The factored shear resistance against shear yielding, R_r: LRFD Art. 6.13.5.3

$$R_r = \phi_v 0.58 F_y A_{vg}$$ LRFD Eq. 6.13.5.3-1

where:

ϕ_v = resistance factor for shear = 1.0 LRFD Art. 6.5.4.2

A_{vg} = gross sectional area of the web splice plates subjected to shear

F_y = yield strength of the connected material = 50 ksi

Shear rupture:

The factored shear resistance against shear rupture, R_r: LRFD Art. 6.13.5.3

$$R_r = \phi_{vu} 0.58 R_p F_u A_{vn}$$ LRFD Eq. 6.13.5.3-2

where:

ϕ_{vu} = resistance factor for shear rupture of splice plates = 0.80 LRFD Art. 6.5.4.2

A_{vn} = net area of the web splice plate subjected to shear

F_u = specified tensile strength of the connected material = 65 ksi

R_p = reduction factor for holes

= 1.0 (assuming bolt holes are drilled full size) LRFD Art. 6.13.5.3

Block shear rupture resistance:

The factored shear resistance against block shear rupture, R_r:

$$R_r = \phi_{bs} R_p (0.58 F_u A_{vn} + U_{bs} F_u A_{tn}) \leq \phi_{bs} R_p \left(0.58 F_y A_{vg} + U_{bs} F_u A_{tn}\right)$$ LRFD Eq. 6.13.4-1

where:

ϕ_{bs} = resistance factor for block shear rupture = 0.80 LRFD Art. 6.5.4.2

A_{tn} = net area along the plane resisting tension stress

A_{vg} = gross area along the plane resisting shear stress

A_{vn} = net area along the plane resisting shear stress

U_{bs} = reduction factor for block shear rupture resistance = 1.0 LRFD Art. 6.13.4

The following steps show the capacity checks for the web splice plates in shear.

The factored shear resistance against shear yielding:

A_{vg} = 2 × 29 × 0.375 = 21.75 in.^{2}

R_r = $\phi_v 0.58 F_y A_{vg} = (1.0)(0.58)(50)(21.75)$ = 631 kips

R_r = 631 kips < R_w = 848 kips **FAIL**

Use a 0.675 in. thick web splice plate.

The web splice plate thickness of 0.675 in. is greater than 0.5 in. Therefore, the nominal resistance, R_n, calculation in **Step 9.1.3.1.1** should be revised by excluding the threads in the shear planes. LRFD Art. C6.13.6.1.3c

R_n = $0.56 A_b F_{ub} N_s$ = 0.56 (0.785)(120)(2) = 105.5 kips

R_r = ϕR_n = 0.8(105.5) = 84.4 kips

The required number of bolts = R_w/R_r = 848/84.4 = 10 bolts

Therefore, the previously selected 14 bolts are adequate for this web splice.

The factored shear resistance against shear yielding,

A_{vg} = 2 × 29 × 0.675 = 39.15 in.^{2}

R_r = $\phi_v 0.58 F_y A_{vg} = (1.0)(0.58)(50)(39.15)$ = 1,135 kips

R_r = 1,135 kips > R_w = 848 kips **OK**

The factored shear resistance against shear rupture,

A_{vn} = 2 × (29 – 7 × 1.125) × 0.675 = 28.52 in.^{2}

R_r = $\phi_{vu} 0.58 R_p F_u A_{vn} = (0.80)0.58(1.0)(65)(28.52)$ = 860 kips

R_r = 860 kips > R_w = 848 kips **OK**

Figure 56 shows the block shear failure mode of the web splice plates.

A_{vg} = 2 × (29 – 1.75) × 0.675 = 36.7875 in.^{2}

A_{vn} = 2 × [29 – 1.75 – 6.5(1.125)] × 0.675 = 26.92 in.^{2}

A_{tn} = 2 × [3.5 + 2.0 – 1.5 (1.125)] × 0.675 = 5.15 in.^{2}

The factored shear resistance against block shear rupture,

$$R_r = \phi_{bs} R_p (0.58 F_u A_{vn} + U_{bs} F_u A_{tn})$$

$$= 0.8 \times 1.0 \times (0.58 \times 65 \times 26.92 + 1.0 \times 65 \times 5.15) = 1{,}080 \text{ kips}$$

$$< \phi_{bs} R_p \left(0.58 F_y A_{vg} + U_{bs} F_u A_{tn}\right)$$

$$= 0.8 \times 1.0 \times (0.58 \times 50 \times 36.7875 + 1.0 \times 65 \times 5.15) = 1{,}121 \text{ kips}$$

R_r = 1,080 kips > R_w = 848 kips **OK**

Normally, the block shear resistance does not control the web splices of a typical proportion due to the overall length of the connection. LRFD Art. C6.13.6.1.3c

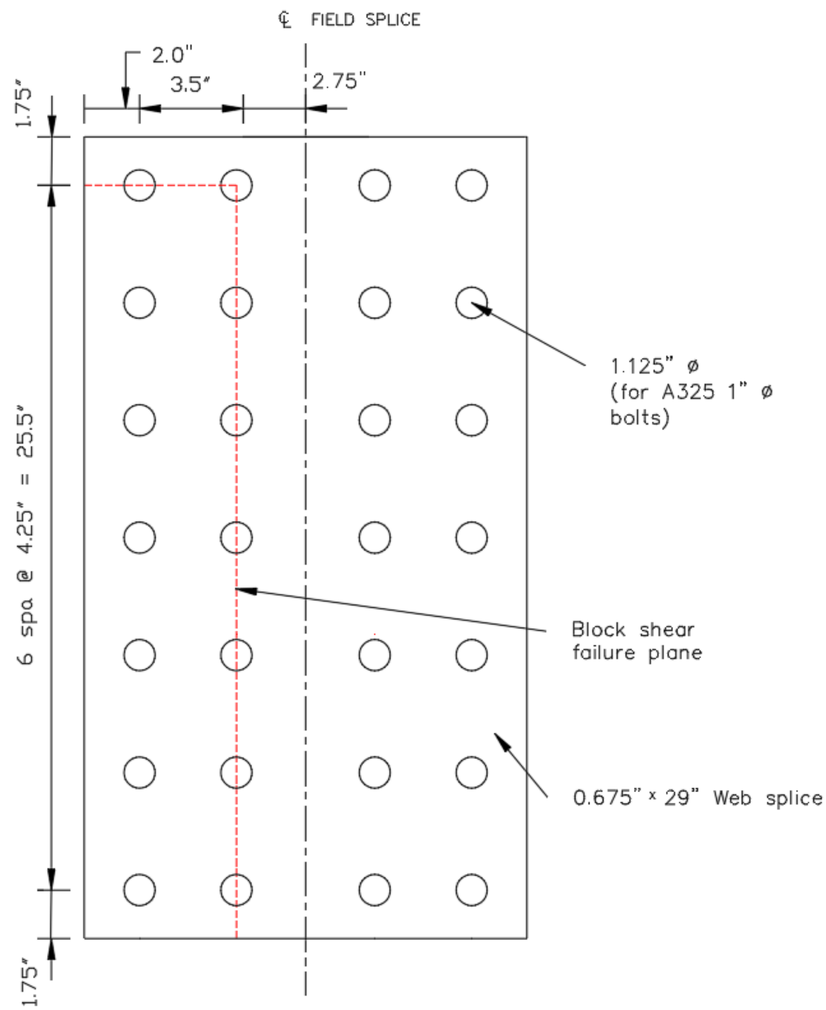


Figure 56. Block shear failure mode of the web splice plates

Step 9.1.3.1.4. Bearing resistance check

The bearing resistance of the web at bolt holes shall be checked at the strength limit state. LRFD Art. 6.13.6.1.3c

The bearing resistance of the connected material at the strength limit state is calculated as the sum of the bearing resistances of the individual bolts parallel to the line of the applied force. LRFD Art. 6.13.2.9

The bearing resistance of the critically connected element (i.e. web or web splice plate) is checked for bearing resistance.

At the web splice location:

The thickness of the web	= 0.5 in.
The thicknesses of the web splice plates	= 2(0.675) = 1.35 in.
F_u of the connected material (web and web splice plates)	= 65 ksi.

Web thickness × F_u < ∑ Web splice plate thickness × F_u

The web is critical, and the web bearing resistance needs to be evaluated.

Check the bearing resistance of the outermost web splice bolt hole for an inclined resultant design web force. LRFD Art. 6.13.6.1.3c LRFD Fig. C6.13.6.1.3c-3

A ¼ in. gap is provided between girder sections at the splice location.
The distance between bolt lines on either side of the centerline of the splice connection is 5.5 in.
Therefore, the distance from each bolt line to the end of the girder section is 2.625 in. [i.e., (5.5 – 0.25)/2 in.].

Clear end distance, L_c = 2.625 – 1.125/2 = 2.0625 in.

Since L_c > 2.0d = 2 × 1 = 2 in.,

$R_n = 2.4dtF_u = 2.4(1.0)(0.5)(65)$ = 78 kips

where:

F_u = minimum tensile strength of the web = 65 ksi
t = thickness of the web = 0.5 in.

Total bearing resistance of a bolt, R_r $= \phi_{bb}R_n = 0.80(78)$ = 62.4 kips/bolt

Design load per bolt, P_r = R_W/N = 848 kips/14 bolts = 60.6 kips/bolt

P_r = 60.6 kips < R_r = 62.4 kips **OK**

Step 9.1.3.2. Slip resistance check

Should the nominal slip resistance provided by the flange bolts not be sufficient to resist the flange slip force due to the factored moment at the splice location determined as per LRFD Art. 6.13.6.1.3b for flanges, the web splice bolts shall instead be checked for slip under a web slip force taken as equal to the vector sum of the factored shear and portion of the flange slip force that exceeds the nominal slip resistance of the flange bolts. LRFD Art. 6.13.6.1.3c

As shown in **Step 9.1.2.2**., the slip resistance provided by the flange bolts is sufficient.

Therefore, the bolted connections for web splices are checked for slip under a web slip force taken equal to the factored shear in the web at the splice location. The factored shear for checking slip is taken as the shear in the web at splice location under the Service II limit state or under the deck casting sequence, whichever governs. LRFD Art. 6.13.6.1.3c

Factored shear under the Service II limit state

Positive shear = 1.0 (-44 - 5) + 1.0 (-7) + 1.30 (15) = -36.5 kips
Negative shear = 1.0 (44 + 5) + 1.0(7) + 1.30 (80) = 160 kips

Factored shear due to deck casting sequence = 1.4(-44) = -61.6 kips

The Service II limit state governs.

V_u = 160 kips

The computed web slip force shall be divided by the nominal slip resistance of the bolts to determine the total number of web splice bolts on one side of the splice to resist slip. LRFD Art. 6.13.6.1.3b

The nominal slip resistance of a bolt in a slip critical connection, R_n, LRFD Art. 6.13.2.8

$$R_n = K_h K_s N_s P_t$$ LRFD Eq. 6.13.2.8-1

The nominal slip resistance of a bolt in a slip critical connection,

$R_n = K_h K_s N_s P_t = (1.0)(0.5)(2.0)(51)$ = 51 kips

The slip resistance check is similar to that for the flange splice bolts.

In this example, the 14 splice bolts used in the web are checked for slip resistance.

Capacity (R_n) for web bolts = 14 (51) = 714 kips

R_n = 714 kips > V_u = 160 kips **OK**

The revised details of the web splice satisfying all the design requirements are shown in **Figure 57.**

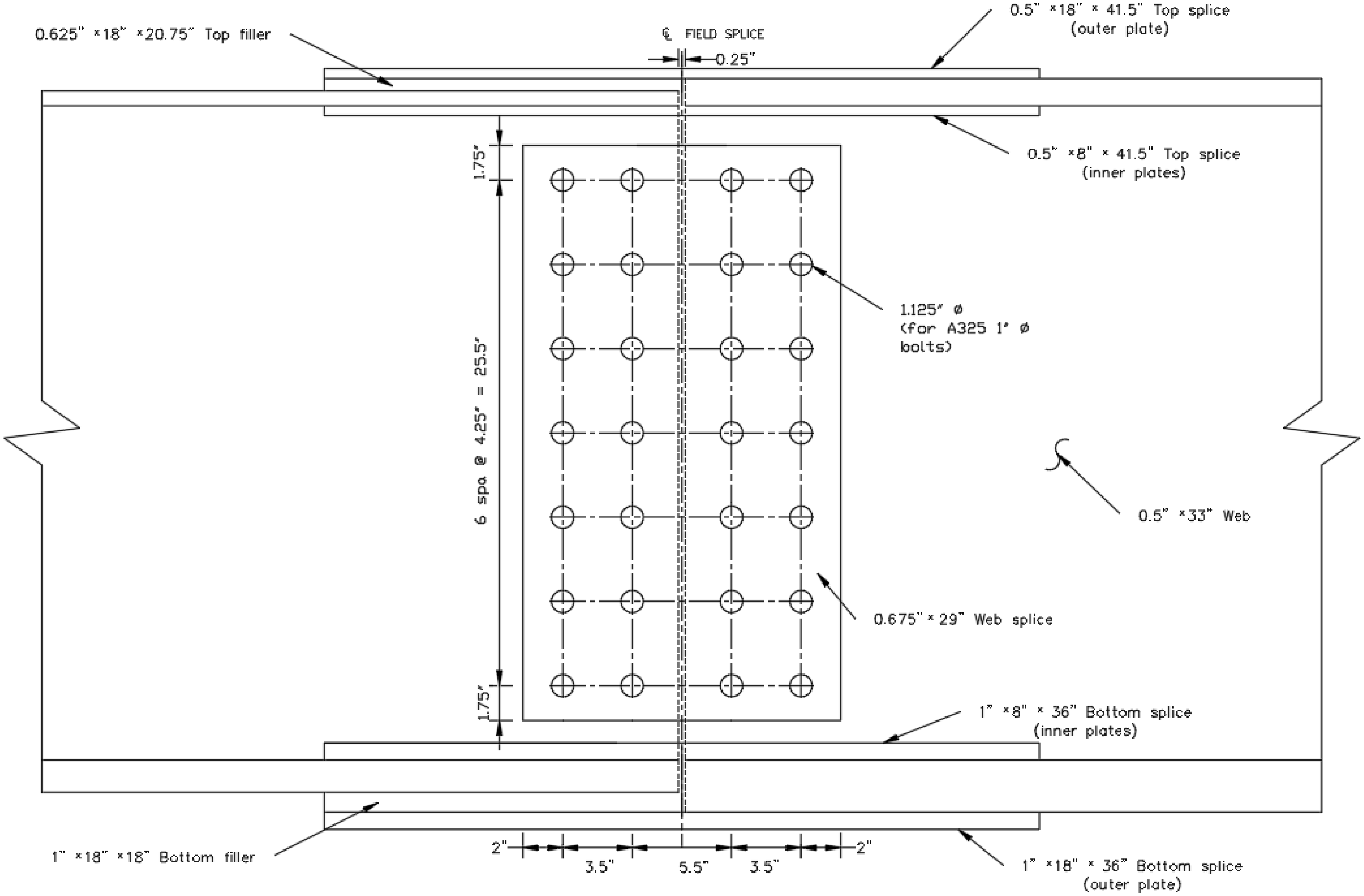


Figure 57. Revised web splice details

Step 9.2. Shear Connector Design

In this design example, stud shear connectors are being used only throughout the positive bending region.

Shear developers are not to be used in areas of negative moment. They should extend through the positive moment area and to, or slightly beyond, the point of contraflexure. This point should be determined for the loading condition that will place it closest to the support over which the negative moment will occur. In the event of a special case in which shear developers are used in negative moment areas, maximum tensile stress at the point of attachment is not to exceed that which is allowed by the current American Welding Society (AWS) specifications. Shear developers (acting as slab ties) shall be placed in at least one half of all spans regardless of contraflexure points and moment orientations. BDM Art. 7.02.15B

Straight continuous composite bridges should normally be provided with shear connectors throughout the entire length of the bridge. In the negative flexure regions, shear connectors shall be provided where the longitudinal reinforcement is considered to be a part of the composite section. Otherwise, shear connectors need not be provided in negative flexure regions, but additional connectors shall be placed in the region of the points of permanent load contraflexure as specified in Art. 6.10.10.3. LRFD Art. 6.10.10

When shear connectors are omitted in negative flexure regions, the longitudinal reinforcement shall be extended into the positive flexure region as specified in LRFD Art. 6.10.1.7.

Figure 58 shows the shear connector design steps implemented in this example.

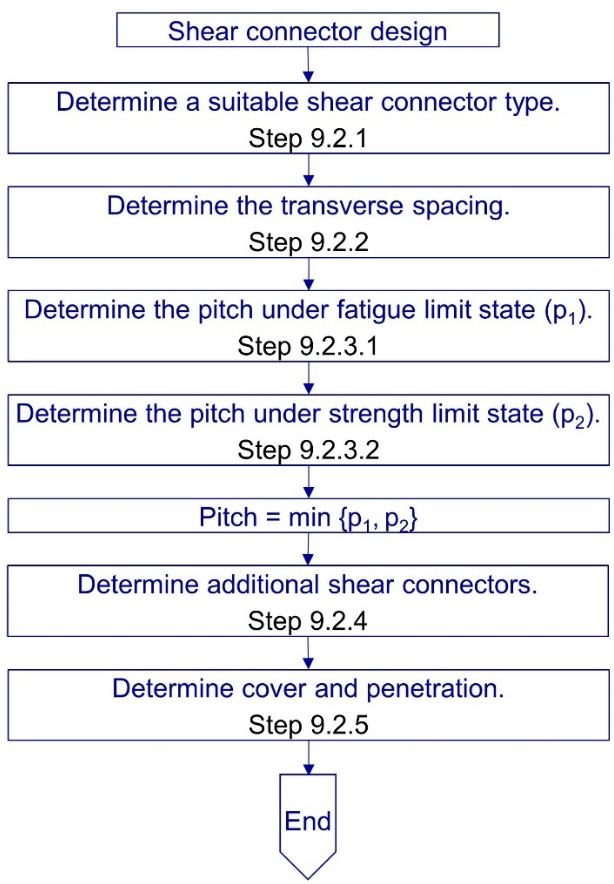


Figure 58. Shear connector design procedure

Step 9.2.1. Shear connector type

Shear developers shall be the stud type as shown in MDOT BDG 8.07.01. BDG 8.07.01

Figure 59 shows a typical stud detail.

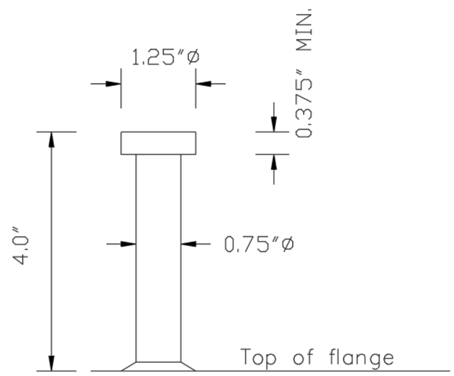


Figure 59. Detail of stud as per BDG 8.07.01

A minimum stud height of 4 in. is specified. BDG 8.07.01

Increase the length of the stud when necessary to maintain 2 in. minimum penetration of the stud into the deck slab (in 1 in. increments). The maximum stud length is limited to 16 in. BDG 8.07.01

For a stud shear connector, the height/diameter ≥ 4.0. LRFD Art. 6.10.10.1.1

Step 9.2.2. Transverse spacing

Shear connectors shall be placed transversely across the top flange of the steel section and may be spaced at regular or at variable intervals. Stud shear connectors shall not be closer than 4.0 stud diameters center-to-center transverse to the longitudinal axis of the supporting member. LRFD Art. 6.10.10.1.3

The clear distance between the edge of the top flange and the edge of the nearest shear connector shall not be less than 1.0 in.

Transverse spacing ≥ 3 in. BDG 8.07.01

Minimum edge distance ≥ 1.5 in.

Figure 60 shows the shear stud layout selected for this example.

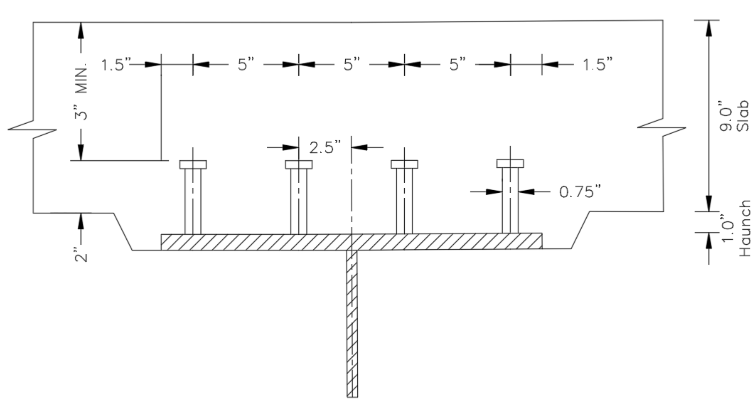


Figure 60. Transverse stud layout

Center-to-center transverse spacing of shear studs = 5 in. > 3.0 in. **OK**

The distance from the center of the shear stud to the edge of top flange = 1.5 in. **OK**

Stud height/diameter = 4 in./0.75 in. = 5.3 > 4.0 **OK**

Step 9.2.3. Pitch

The pitch of the shear connectors shall be determined to satisfy the fatigue limit state. The resulting number of shear connectors shall not be less than the number required to satisfy the strength limit state. LRFD Art. 6.10.10.1.2

For girders with web depth ≥ 24 in., LRFD Art. 6.10.10.1.2

6.0d ≤ The center-to-center pitch of shear connectors ≤ 48 in.
where d is the stud diameter.

Step 9.2.3.1. Fatigue limit state

There are two methods to calculate the fatigue shear resistance of an individual stud shear connector, Z_r.

Where the projected 75-year single lane Average Daily Truck Traffic $(ADTT)_{SL}$ is greater than or equal to 960 trucks per day, the fatigue I load combination shall be used and the fatigue shear resistance for infinite life shall be taken as LRFD Art. 6.10.10.2

$$Z_r = 5.5d^2$$ LRFD Eq. 6.10.10.2-1

Otherwise, the fatigue II load combination shall be used and the fatigue shear resistance for finite life shall be taken as: LRFD Art. 6.10.10.2

$$Z_r = \alpha d^2$$ LRFD Eq. 6.10.10.2-2

in which:

$$\alpha = 34.5 - 4.28logN$$ LRFD Eq. 6.10.10.2-3

where:

d = diameter of the stud

N = number of cycles

The pitch, P, of shear connectors shall be determined using Z_r and the shear force range V_{sr} as:

$$p \le \frac{nZ_r}{V_{sr}}$$ LRFD Eq. 6.10.10.1.2-1

in which:

V_{sr} = horizontal fatigue shear range per unit length

$= \sqrt{(V_{fat})^2 + (F_{fat})^2}$ LRFD Eq. 6.10.10.1.2-2

V_{fat} = longitudinal fatigue shear range per unit length

$= \frac{V_f Q}{I}$ LRFD Eq. 6.10.10.1.2-3

F_{fat} = radial fatigue shear range per unit length taken as the larger of either:

$F_{fat1} = \frac{A_{bot}\sigma_{flg}l}{wR}$ or: — LRFD Eq. 6.10.10.1.2-4

$F_{fat2} = F_{rc}/w$ — LRFD Eq. 6.10.10.1.2-5

where:

σ_{flg} = range of longitudinal fatigue stress in the bottom flange without consideration of flange lateral bending

A_{bot} = area of the bottom flange

F_{rc} = net range of cross-frame or diaphragm force at the top flange

I = moment of inertia of the short-term composite section

l = distance between brace points

n = number of studs in the cross-section

Q = first moment of the transformed short-term area of the concrete deck about the neutral axis of the short-term composite section

R = minimum girder radius within the panel

V_f = vertical shear force range under the applicable fatigue load combination

w = effective length of deck taken as 48.0 in., except at end supports where w maybe taken as 24.0 in.

Z_r = shear fatigue resistance of an individual shear connector

For straight spans or segments, the radial fatigue shear range from Eq. 6.10.10.1.2-4 may be taken equal to zero. For straight or horizontally curved bridges with skews not exceeding 20 degrees, the radial fatigue shear strength from Eq. 6.10.10.1.2-5 may be taken equal to zero. — LRFD Art. 6.10.10.1.2

In this example, the $(ADTT)_{SL}$ is 3000 trucks per day per lane. Therefore, the fatigue I load combination is used to determine the number of shear connectors required under fatigue limit state.

The fatigue shear resistance of an individual stud shear connector, Z_r

$$Z_r = 5.5d^2 \qquad = 5.5 \times 0.75^2 \qquad = 3.094 \text{ kips}$$

In the positive bending region, the maximum fatigue live load shear range, V_f, is located at the abutment.

At the abutment ($0.0L_{ds}$), shear force due to HS20 fatigue truck:

Positive shear force	= 56.2 kips	**Table A-2**
Negative shear force	= -5.5 kips	**Table A-2**

The DFV for an exterior girder under fatigue limit state, DFV_{EB} = 0.598 **Step 5.3**

Applying the IM of 15% and DFV,

Positive shear force	= 56.2 × 1.15 × 0.598	= 38.6 kips
Negative shear force	= -5.5 × 1.15 × 0.598	= -3.8 kips

Fatigue load range under the fatigue I load combination

V_f = 1.75 × [38.6 – (– 3.8)] = 74.2 kips

I and Q at the abutment are used in the calcualtion. The distance to the PNA from the girder top is 12.23 in. and the transformed short-term deck width is 11.37 in. **Step 3.2.2**

Q = (9 × 11.37) × (12.23 - 9/2) = 791 in.^{3}

I = 30,550 in.^{4} **Table 3**

F_{fat} = 0 kip/in. LRFD Art. 6.10.10.1.2

$$V_{fat} = \frac{74.2 \times 791}{30{,}550} = 1.921 \text{ kip/in.}$$

$$V_{sr} = \sqrt{\left(V_{fat}\right)^2 + \left(F_{fat}\right)^2} = \sqrt{(1.921)^2 + 0} = 1.921 \text{ kip/in.}$$

There are four rows of shear studs. Therefore, n = 4. **Figure 60**

The required pitch under the fatigue limit state:

$$p \le \frac{nZ_r}{V_{sr}} = \frac{(4)(3.094)}{(1.921)} = 6.44 \text{ in.}$$

Therefore, provide a pitch of 6 in. to satisfy the fatigue limit state requirements.

Step 9.2.3.2. Strength limit state

The number of studs and the pitch required as per the strength limit state are calculated. The pitch satisfying both fatigue and strength limit states is selected. LRFD Art. 6.10.10.4

The factored shear resistance of a single shear connector, Q_r, at the strength limit state: LRFD Art. 6.10.10.4.1

$$Q_r = \phi_{sc} Q_n$$

LRFD Eq. 6.10.10.4.1-1

where:

ϕ_{sc} = resistance factor for shear connectors = 0.85 LRFD Art. 6.5.4.2

Q_n = nominal shear resistance of a single shear connector

The nominal shear resistance of one stud shear connector embedded in a conrcete deck, Q_n, LRFD Art. 6.10.10.4.3

$$Q_n = 0.5A_{sc}\sqrt{f_c' E_c} \le A_{sc}F_u$$

LRFD Eq. 6.10.10.4.3-1

where:

A_{sc} = cross-sectional area of a stud shear connector

E_c = modulus of elasticity of the deck concrete

F_u = specified minimum tensile strength of a stud shear connector — LRFD Art. 6.4.4

At the strength limit state, the minimum number of shear connectors, *n*, between the maximum positive bending moment and each adjacent point of the zero moment shall be taken as: — LRFD Art. 6.10.10.4.1

$$n = \frac{P}{Q_r}$$ LRFD Eq. 6.10.10.4.1-2

where:

P = total nominal shear force

For simple spans and for continuous spans that are noncomposite for negative flexure in the final condition, the total nominal shear force, P, between the point of maximum positive design live load plu the impact moment and each adjacent point of zero moment shall be taken as — LRFD Art. 6.10.10.4.2

$$P = \sqrt{\left(P_p\right)^2 + \left(F_p\right)^2}$$ LRFD Eq. 6.10.10.4.2-1

where:

F_p = total radial force in the concrete deck at the point of maximum positive live load plus impact moment

For straight spans or segments, F_p may be taken equal to zero. — LRFD Art. 6.10.10.4.2

P_p = total longitudinal force in the concrete deck at the point of maximum positive live load plus impact moment taken as the lesser of either P_{1p} and P_{2p}.

P_{1p} = $0.85 f_c' b_s t_s$ — LRFD Eq. 6.10.10.4.2-2

P_{2p} = $F_{yw} D t_w + F_{yt} b_{ft} t_{ft} + F_{yc} b_{fc} t_{fc}$ — LRFD Eq. 6.10.10.4.2-3

b_s = effective width of the concrete deck

The following steps show the calculation of the pitch of shear connectors under the strength limit state.

$A_{sc} = \frac{\pi d^2}{4} = \frac{\pi (0.75)^2}{4}$ = 0.442 in.^{2}

E_c = 3987 ksi — **Step 2.1**

F_u = 60 ksi — LRFD Art.6.4.4

b_s = 90.9375 in. — **Step 3.2.2**

The nominal shear resistance of one stud shear connector, Q_n

$$Q_n = 0.5(0.442)\sqrt{(4)(3987)} = 27.9\ kips$$

$$> A_{sc}F_u = (0.442)(60) = 26.5 \text{ kips}$$

Therefore, Q_n = 26.5 kips.

The factored shear resistance of a single shear connector, Q_r

$$Q_r = \phi_{sc} Q_n = (0.85)(26.5) \qquad = 22.5 \text{ kips}$$

The section geometry at $0.4L_{ds}$ is considered when calcualting the total nominal shear force, P_p, since the positive live load plus the impact moment is the highest at $0.4L_{ds}$.

$P_{1p} = 0.85(4)(90.9375)(9) \qquad = 2783$ kips

$P_{2p} = (50)(33)(0.5) + (50)(18)(0.75) + (50)(18)(1.625) \qquad = 2963$ kips

P_p = min (P_{1p}, P_{2p}) = min (2783, 2963) = 2783 kips

F_p = 0 ksi (LRFD Art. 6.10.10.4.2)

$$P = \sqrt{(P_p)^2 + (F_p)^2} = \sqrt{(2783)^2 + 0} \qquad = 2783 \text{ kips}$$

$$n = \frac{P}{Q_r} = \frac{2783}{22.5} \qquad = 124$$

For continous spans that are noncomposite for negative flexure in the final condition, points of zero moment within the span should be taken as the points of steel dead load contraflexure. (LRFD Art. C6.10.10.4.2)

Point of steel dead load contraflexure = $0.659\ L_{ds} = 0.659 \times 98.67$ = 68.58 ft **Table 10**

The distance between the abutment centerline and the location of the maximum design live load plus impact (positive) moment approximately equals to 39.47 ft ($0.4L_{ds}$). Similarly, the distance between the section of the maximum design live load plus impact (positive) moment and the point of steel dead load contraflexure is approximately equal to 29.11 ft (68.58 ft - 39.47 ft). The number of shear connectors required at the strength limit state is calculated by selecting the least distance out of these two, i.e. 29.11 ft.

Assuming 124 shear connectors are used through the distance between the section of the maximum design live load plus the impact (positive) moment and the point of steel dead load contraflexure (29.11 ft), the required pitch under strength limit state is calculated as follows.

There are four rows of shear studs. Therefore, n = 4.

Required pitch = $4\left(\frac{29.11\times12}{124}\right)$ = 11 in.

The pitch determined using the fatigue limit state governs over the strength limit state. Therefore, use a pitch of 6 in. for the shear connectors.

Step 9.2.4. Additional shear connectors

For members that are noncomposite for negative flexure in the final conditions, additional shear connectors shall be provided in the region of points of permanent load contraflexure. LRFD Art. 6.10.10.3

Shear developers are not to be used in the areas of negative moment. They should extend through the positive moment area and to, or slightly beyond, the point of contraflexure. This point should be determined for the loading condition that will place it closest to the support over which the negative moment will occur. BDM Art. 7.02.15B

Therefore, additional shear studs are provided between the point of steel dead load contraflexure and the point of superimposed dead load contraflexure. Since the field splice is located at the point of superimposed dead load contraflexure, studs are provided up to the edge of the field splice top flange plate located towards the abutment side.

Step 9.2.5. Cover and penetration

Increase the length of stud when necessary to maintain a 2 in. minimum penetration of stud into the deck slab (1 in. increments). BDG 8.07.01

The clear depth of concrete cover over the tops of the shear connectors should not be less than 2.0 in. Shear connectors should penetrate at least 2.0 in. into the concrete deck. LRFD Art. 6.10.10.1.4

Clear depth of concrete cover over the tops of the shear connectors

= 3 in. (min) > 2 in. **OK** **Figure 60**

Therefore, using a pitch of 6 in. for each row, with four stud shear connectors across the girder cross-section, throughout the positive bending region of the girder satisfies both the fatigue limit state requirements and the strength limit state requirements.

Step 9.3. Bearing Stiffener Design

Bearing stiffeners shall be placed on the webs of built-up sections at all bearing locations. At bearing locations on rolled shapes and at other locations on built-up sections or rolled shapes subjected to concentrated loads, where the loads are not transmitted through a deck or deck system, either bearing stiffeners shall be provided or the web shall satisfy the provisions of Art. D6.5. LRFD Art. 6.10.11.2.1

Bearing stiffeners shall consist of one or more plates or angles welded or bolted to both sides of the web. The connections to the web shall be designed to transmit the full bearing force due to the factored loads. The stiffeners shall extend the full depth of the web and as closely as practical to the outer edges of the flanges.

Webs of built-up sections and rolled shapes without bearing stiffeners at the indicated locations must be investigated for the limit states of web local yielding and web crippling according to the procedures specified in Art. D6.5. LRFD Art. C6.10.11.2.1

Figure 61 shows the design procedure followed for the bearing stiffeners.

The bearing stiffener design at the pier and abutments is illustrated in this example.

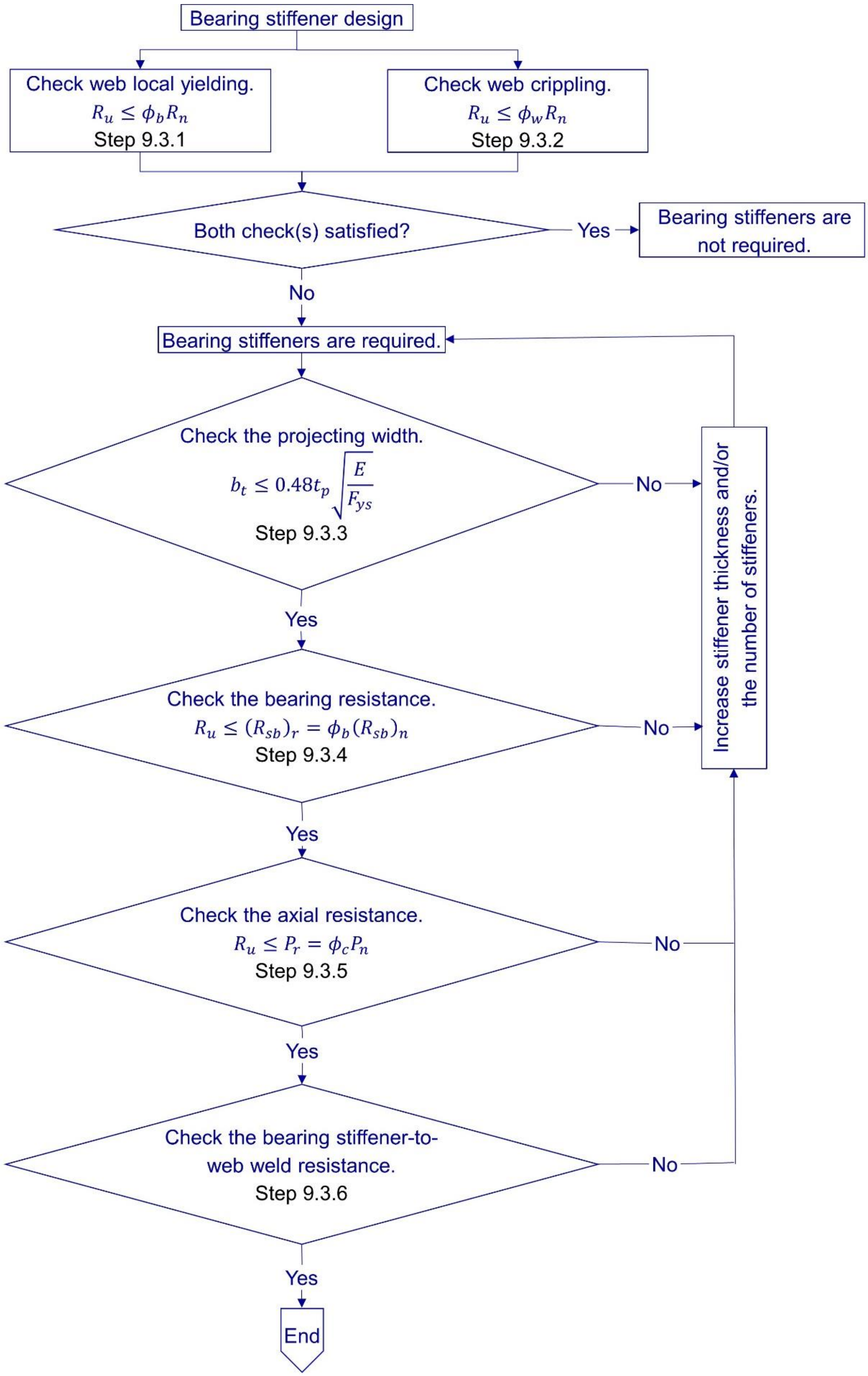


Figure 61. Bearing stiffener design procedure

Step 9.3.1. Web local yielding

Webs subjected to compressive or tensile concentrated loads shall satisfy:

$$R_u \leq \phi_b R_n$$ LRFD Eq. D6.5.2-1

where:

ϕ_b = resistance factor for bearing = 1.0 LRFD Art. 6.5.4.2

R_u = factored concentrated load or bearing reaction

R_n = nominal resistance to the concentrated loading

For interior pier reactions and for concentrated loads applied at a distance from the end of the member that is greater than d:

$$R_n = (5k + N)F_{yw}t_w$$ LRFD Eq. D6.5.2-2

Otherwise:

$$R_n = (2.5k + N)F_{yw}t_w$$ LRFD Eq. D6.5.2-3

where:

d = depth of the steel section

k = distance from the outer face of the flange resisting the bearing reaction to the web toe of the fillet
Assume k = flange thickness.

F_{yw} = yield strength of the web

N = bearing length

t_w = web thickness

Considering typical sole plate dimensions, the bearing length (N) at the pier and abutments is selected as 20.50 in. and 10.50 in, respectively.

At the pier (1.0L_{ds}):

R_u = 335 kips **Table 20**

k = 2.625 in. (bottom flange thickness)

N = 20.50 in.

F_{yw} = 50 ksi

t_w = 0.5 in.

Substituting the above values into Eq. D6.5.2-2:

R_n = [5(2.625) + 20.50] × 50 × 0.5 = 841 kips

Web yielding requirement at the pier:

R_u = 335 kips < $\phi_b R_n$ = (1.00) (841) = 841 kips **OK**

Web yielding capacity is greater than the factored reaction over the pier.

At the abutment (0.0L_{ds}):

R_u	= 243 kips	**Table 20**
k	= 1.0 in.	
N	= 10.50 in.	
F_{yw}	= 50 ksi	
t_w	= 0.5 in.	

Substituting the above values into Eq. D6.5.2-3:

R_n = [2.5(1.0) + 10.50] × 50 × 0.5 = 325 kips

Web yielding requirement at the abutment:

R_u = 243 kips < $\phi_b R_n$ = (1.00) (325) = 325 kips **OK**

Web yielding capacity is greater than the factored girder end reaction over abutment.

Step 9.3.2. Web crippling

Webs subjected to compressive concentrated loads shall satisfy:

$$R_u \leq \phi_w R_n$$ LRFD Eq. D6.5.3-1.

where:

ϕ_w = resistance factor for web crippling = 0.8 LRFD Art. 6.5.4.2

R_n = nominal resistance to the concentrated loading

For interior-pier reactions:

$$R_n = 0.8t_w^2\left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}$$ LRFD Eq. D6.5.3-2

Otherwise:

If N/d ≤ 0.2,

$$R_n = 0.4t_w^2\left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}$$ LRFD Eq. D6.5.3-3

If N/d > 0.2,

$$R_n = 0.4t_w^2\left[1 + \left(\frac{4N}{d} - 0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}$$ LRFD Eq. D6.5.3-4

where:

d = depth of the steel section

E = modulus of elasticity of the web

F_{yw} = specified yield strength of the web

N = bearing length

R_n = nominal resistance to the concentrated loading

t_f = thickness of the flange resisting the concentrated load

t_w = web thickness

At the pier (1.0L_{ds}):

d	= 2.625 + 33 + 2.625	= 38.25 in.
E		= 29000 ksi
F_{yw}		= 50 ksi
N		= 20.50 in.
R_u		= 335 kips
t_f		= 2.625 in.
t_w		= 0.5 in.

Substituting the above values into Eq. D6.5.3-2:

$$R_n = 0.8(0.5)^2\left[1 + 3\left(\frac{20.50}{38.25}\right)\left(\frac{0.5}{2.625}\right)^{1.5}\right]\sqrt{\frac{29000 \times 50 \times 2.625}{0.5}} \quad = 626 \text{ kips}$$

Web crippling resistance over the pier:

R_u = 335 kips < $\phi_w R_n$ =(0.80) (626) = 501 kips **OK**

The web crippling capacity is adequate to support the factored girder end reaction over the pier.

At the abutment (0.0L_{ds}):

All the input parameters are similar to that over the pier except for d, t_f, and N.

d = 1 + 33 + 0.75	= 34.75 in.
t_f	= 1.0 in.
R_u	= 243 kips
N	= 10.50 in.
N/d = 10.50 / 34.75	= 0.302 > 0.2

Substituting the above values into Eq. D6.5.3-4:

$$R_n = 0.4(0.5)^2\left[1 + \left(\frac{4 \times 10.50}{34.75} - 0.2\right)\left(\frac{0.5}{1.0}\right)^{1.5}\right]\sqrt{\frac{29000 \times 50 \times 1.0}{0.5}} \quad = 231 \text{ kips}$$

Web crippling resistance at the abutment:

R_u = 243 kips > $\phi_w R_n$ =(0.80) (231) = 185 kips **FAIL**

Web capacity is inadequate to prevent failure due to web crippling.

Therefore, bearing stiffeners are required at the abutment.

In this example, the 3/4 in. × 8 in. bearing stiffeners, as shown in **Figure 62,** are provided for an exterior girder at both pier and at abutments.

The following steps check if the 3/4 in. × 8 in. bearing stiffeners provided for an exterior girder over the abutment satisfy the LRFD requirements.

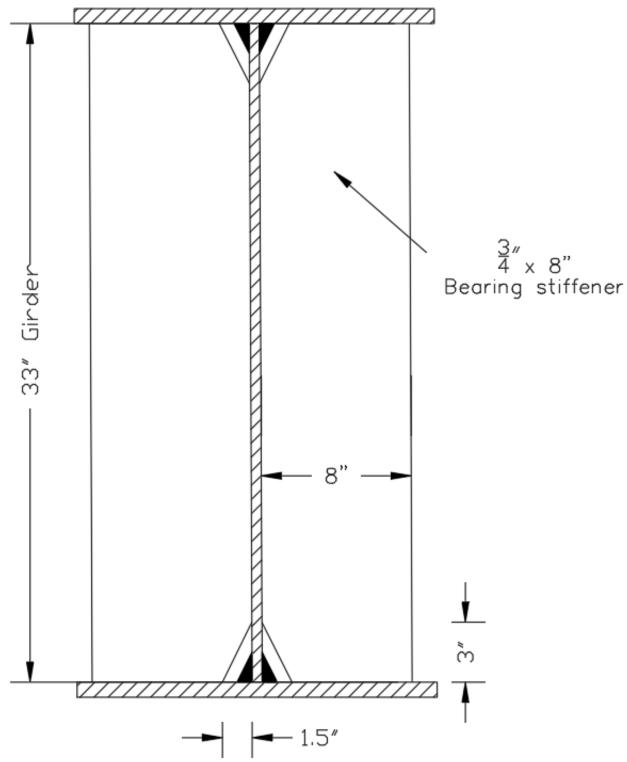


Figure 62. Bearing stiffener detail

Step 9.3.3. Projecting width

The width of projecting stiffener elements, b_t, must satisfy: LRFD Art. 6.10.11.2.2

$$b_t \leq 0.48 t_p \sqrt{\frac{E}{F_{ys}}}$$ LRFD Eq. 6.10.11.2.2-1

where:

b_t = width of each projecting stiffener element = 8 in. **Figure 62**

F_{ys} = specified minimum yield strength of the stiffener = 50 ksi

t_p = thickness of the projecting stiffener element = 0.75 in. **Figure 62**

≥ 7/16 in. **OK** BDM Art. 7.02.11A

b_t $= 8\ in. < 0.48(0.75)\sqrt{\frac{29000}{50}} = 8.67\ in.$ **OK**

Step 9.3.4. Bearing resistance

The factored bearing resistance of the bearing stiffeners is: LRFD Art. 6.10.11.2.3

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

where:

ϕ_b = resistance factor for bearing = 1.0 LRFD Art. 6.5.4.2

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

= $1.4A_{pn}F_{ys}$ LRFD Eq. 6.10.11.2.3-2

A_{pn} = area of the projecting elements for the stiffener outside of the web-to-flange fillet welds but not beyond the edge of the flange

F_{ys} = specified minimum yield strength of the stiffener

In this example, the clip provided at the base of the stiffener to clear the flange-to-web weld is 1.5 in. in length. **Figure 62**

The following steps show the bearing resistance over the abutment.

A_{pn}	= 2 × (8 – 1.5) × 0.75	= 9.75 in.^{2}
$(R_{sb})_n$	= 1.4 × 9.75 × 50	= 683 kips
$(R_{sb})_r$	= (1.0) (683)	= 683 kips

R_u = 243 kips < $(R_{sb})_r$ = 683 kips **OK**

The 3/4 in. × 8 in. bearing stiffeners have adequate bearing resistance.

Step 9.3.5. Axial resistance

The factored axial resistance, P_r, shall be determined as specified in Art. 6.9.2.1 using the specified minimum yield strength of the stiffener plates, F_{ys}. LRFD Art. 6.10.11.2.4a

The factored resistance of components in compression, P_r, shall be taken as

$$P_r = \phi_c P_n$$ LRFD Eq. 6.9.2.1-1

where:

ϕ_c = resistance factor for axial compression = 0.95 LRFD Art. 6.5.4.2

P_n = nominal compressive resistance LRFD Art. 6.9.4.1

The nominal compressive resistance, P_n, shall be taken as the smallest value based on the applicable modes of flexural buckling, torsional buckling, and flexural torsional buckling. Torsional buckling and flexural-torsional buckling shall not be applicable for bearing stiffeners. The only applicable buckling mode is flexural buckling. LRFD Art. 6.9.4.1.1

P_n shall be determined for compression members with nonslender element cross-sections as follows:

If $\frac{P_e}{P_o} \geq 0.44$,

$$P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] P_o$$ LRFD Eq. 6.9.4.1.1-1

If $\frac{P_e}{P_o} < 0.44$,

$$P_n = 0.877 P_e$$ LRFD Eq. 6.9.4.1.1-2

where:

P_e = elastic critical buckling resistance = 131,183 kips **Appendix F.2**

P_o = equivalent nominal yield resistance = $F_y A_g$

The following steps show the axial resistance check of the bearing stiffeners at the abutments.

A_g = 16.5 in. ^{2} **Appendix F.2**

P_o = $(1.0)(50)(16.5)$ = 825 kips

$\frac{P_e}{P_o} = \frac{131183}{825} = 159 > 0.44$

Then

$P_n = \left[0.658^{\left(\frac{825}{131183}\right)}\right](825)$ = 823 kips

$P_r = \phi_c P_n = (0.95)(823)$ = 782 kips

R_u = 243 kips < P_r = 782 kips **OK**

The 3/4 in. × 8 in bearing stiffeners have adequate axial resistance.

Step 9.3.6. Bearing stiffener-to-web welds

The resistance of fillet welds shall be taken as the smaller of the factored shear rupture resistance of the connected material adjacent to the weld leg (determined as in Art. 6.13.5.3), and the product of the effective area (specified in Art. 6.13.3.3) along with the factored shear resistance of the weld metal. LRFD Art. 6.13.3.2.4

The shear resistance of the weld metal, R_r:

$$R_r = 0.6 \phi_{e2} F_{exx}$$ LRFD Eq. 6.13.3.2.4-1

where:

ϕ_{e2} = resistance factor for shear in throat of the weld metal = 0.80 LRFD Art. 6.5.4.2

F_{exx} = classification strength of the weld metal = 70 ksi

R_r = 0.6 (0.80) (70) = 33.6 ksi

Since the thicknesses of the girder web and the stiffener are 0.5 in. and 0.75 in., the base weld thickness of the thicker part (at the abutment) is 0.75 in.

Since the thickness of the thicker part = 0.75 in., the minimum size of the permissible fillet weld is 0.25 in. — LRFD Table 6.13.3.4-1, SSFC Table 707-1

The effective area shall be the effective weld length multiplied by the effective throat. The effective throat shall be the shortest distance from the joint root to the weld face. — LRFD Art. 6.13.3.3

The size of the fillet weld		= 0.25 in.	
The effective throat	$= 0.25 \times \cos(45^0)$		
	$= 0.25 \times 0.707$	= 0.177 in.	
The shear strength per unit length of weld			
	$= 33.6 \times 0.177$	= 5.95 kip/in.	
The length of the weld, allowing 3 in. for clips at both the top and bottom of the stiffener	= 33 – 2(3)	= 28 in.	**Figure 62**
The total factored resistance of the welds connecting the stiffener to the web of the section	= 4 (28 × 5.95)	= 666 kips	
The shear rupture resistance of the web		= 860 kips	**Step 9.1.3.1.3.**
The resistance of the fillet weld	= min (860, 666)	= 666 kips	

Since the web shear rupture resistance > the shear resistance of the weld, the web shear resistance of 666 kips controls.

R_u = 243 kips < 666 kips **OK**

The 0.25 in. weld is adequate at the bearing stiffener-to-web connection and satisfies the minimum weld size requirement. — LRFD Table 6.13.3.4-1, SSFC Table 707-1

Step 9.4. Flange-to-Web Weld Design

This step outlines the design of the web-to-flange connection weld of an exterior girder over the abutment bearing (0.0L_{ds} location).

In this example, the adequacy of a flange-to-web weld of 5/16 in. (0.3125 in.) is evaluated.

The weld design resistance is checked against the required strength.

All welding details are to be according to AWS specifications, except for minimum fillet weld sizes, which should be as shown in the Standard Specifications. BDM Art. 7.02.12

Step 9.4.1. Required horizontal shear strength

The required horizontal shear strength:

$$s = \frac{VQ}{I}$$ FHWA (2015)

where:

I = moment of inertia

Q = statical moment of the area about the neutral axis

V = shear force

The shear flow is calculated using the relevant cross-sectional properties for the respective loading.

<u>Calculation of shear force (V)</u>

The shear forces at 0.0L_{ds} are as follows:

$V_{DC1} = V_g + V_D + V_H + V_f = 7 + 28.9 + 0.6 + 3.1$		= 38.7 kips	**Table 12**
V_b		= 5.0 kips	**Table 12**
V_{ws}		= 8.0 kips	**Table 12**
$V_{HL\text{-}M\,(+)}$		= 99.9 kips	**Table 16**

<u>Calculation of the moment of inertia (I) and statical moment (Q)</u>

I at 0.0L_{ds}:

I of noncomposite section, I_{nc}	= 10418 in.^{4}	**Table 3**
I of short-term composite section, I_{cn8}	= 30550 in.^{4}	**Table 3**
I of long-term composite section, I_{cn24}	= 22583 in.^{4}	**Table 3**

At 0.0L_{ds}:

Distance to the girder top from the neutral axis for:

noncomposite section	= 18.88 in.
short-term composite section	= 2.23 in.
long-term composite section	= 8.64 in.

Distance to the girder bottom from the neutral axis for:

noncomposite section	= 15.87 in.
short-term composite section	= 32.52 in.
long-term composite section	= 26.11 in.

Noncomposite section:

Top flange:	Q_{tf_nc}	= (18 × 0.75) (18.88 – 0.75/2)	= 249.82 in.^{3}
Bottom flange:	Q_{bf_nc}	= (18 × 1.0) (15.87 – 1.0/2)	= 276.66 in.^{3}

Short-term composite section:

Top flange:	Q_{tf_c8}	= (18 × 0.75) (2.23 – 0.75/2)	= 25.04 in.^{3}
Bottom flange:	Q_{bf_c8}	= (18 × 1.0) (32.52 – 1.0/2)	= 576.36 in.^{3}

Long-term composite section:

Top flange:	Q_{tf_c24}	= (18 × 0.75) (8.64 – 0.75/2)	= 111.58 in.^{3}
Bottom flange:	Q_{bf_c24}	= (18 × 1.0) (26.11 – 1.0/2)	= 460.98 in.^{3}

The required shear strength, *s*, under each loading is thus computed for the top flange and bottom flange as follows:

Top flange:

Noncomposite dead loads:

$$s = \frac{V_{DC1}Q_{tf\_nc}}{I_{nc}} = \frac{(1.25 \times 38.7)(249.82)}{10418} = 1.16 \text{ kip/in.}$$

Barrier loads:

$$s = \frac{V_b Q_{tf\_c24}}{I_{c24}} = \frac{(1.25 \times 5.0)(111.58)}{22583} = 0.03 \text{ kip/in.}$$

Wearing surface loads:

$$s = \frac{V_{ws} Q_{tf\_c24}}{I_{c24}} = \frac{(1.50 \times 8.0)(111.58)}{22583} = 0.06 \text{ kip/in.}$$

Live loads:

$$s = \frac{V_{HL-M} Q_{tf\_c8}}{I_{c8}} = \frac{(1.75 \times 99.9)(25.04)}{30550} = 0.14 \text{ kip/in.}$$

The required shear strength at the top flange

= 1.16 + 0.03 + 0.06 + 0.14 = 1.39 kip/in.

Bottom flange:

Noncomposite dead loads:

$$s = \frac{V_{DC1}Q_{bf\_nc}}{I_{nc}} = \frac{(1.25 \times 38.7)(276.66)}{10418} = 1.28 \text{ kip/in.}$$

Barrier loads:

$$s = \frac{V_b Q_{bf\_c24}}{I_{c24}} = \frac{(1.25 \times 5.0)(460.98)}{22583} = 0.13 \text{ kip/in.}$$

FWS loads:

$$s = \frac{V_{ws} Q_{bf\_c24}}{I_{c24}} \qquad = \frac{(1.50 \times 8.0)(460.98)}{22583} \qquad = 0.24 \text{ kip/in.}$$

Live loads:

$$s = \frac{V_{HL-M} Q_{bf\_c8}}{I_{c8}} \qquad = \frac{(1.75 \times 99.9)(576.36)}{30550} \qquad = 3.30 \text{ kip/in.}$$

The required shear strength at the bottom flange

= 1.28 + 0.13 + 0.24 + 3.30 = 4.95 kip/in.

The required shear strength is the highest at the bottom flange. Therefore, the required shear strength at the bottom flange is used to design the fillet welds at the top and bottom of the flange-to-web connection.

The required shear strength at each weld at the bottom flange-to-web connection,

R_{weld} = 4.95/2 = 2.475 kip/in.

The required shear strength must be evaluated in comparison to the shear resistance of a fillet weld and the shear rupture resistance of the base metal. LRFD Art. 6.13.3.2.4

Step 9.4.2. Shear resistance of the fillet weld, R_r

The calculation of the shear resistance of the fillet weld at the bottom flange-to-web is similar to that at the bearing stiffener-to-web described in **Step 9.3.6**.

At $0.0L_{ds}$, the base metal thickness of the thicker part joined at the weld is equal to the bottom flange thickness. Bottom flange thickness of 1.0 in. > web thickness of 0. 5 in.

Minimum size of the fillet weld = 5/16 in. = 0.3125 in. LRFD Table 6.13.3.4-1 SSFC Table 707-1

A fillet weld of 0.3125 in. is selected for each side of the web.

The factored shear resistance of the weld metal, $R_r = 0.6\phi_{e2}F_{exx}$ LRFD Eq. 6.13.3.2.4-1

where:

ϕ_{e2} = resistance factor for shear on the throat of the weld metal = 0.80 LRFD Art. 6.5.4.2

F_{exx} = classification strength of the weld metal = 70 ksi

The factored shear resistance of the weld metal

$$R_r = 0.6\phi_{e2}F_{exx} \qquad = 0.6(0.80)(70) \qquad = 33.6 \text{ ksi}$$

The effective throat of the weld = $0.3125 \times \cos 45^0$ = 0.221 in.

The factored shear resistance of a weld, $R_{r,weld}$ = 33.6 × 0.221 = 7.43 kip/in

$R_{r,weld}$ = 7.43 kip/in. > R_{weld} = 2.475 kip/in. **OK**

A 0.3125 in. fillet weld is adequate and satisfies the minimum size requirement. LRFD Table 6.13.3.4-1 SSFC Table 707-1

Step 9.4.3. Shear rupture resistance of the connected material, R_r

The factored shear rupture resistance of the connected material, R_r

$$R_r = \phi_{vu} 0.58 R_p F_u A_{vn}$$

LRFD Eq. 6.13.5.3-2

where:

ϕ_{vu} = resistance factor for shear rupture of the connected elements = 0.80 — LRFD Art. 6.5.4.2

A_{vn} = net area of the connection element subjected to shear

F_u = tensile strength of the connection element

R_p = reduction factor for holes = 1.0 (for welded connections)

Since the web is thinner than the top and bottom flanges, the factored shear rupture resistance of the web is evaluated.

A_{vn} = 0.5 in.^{2}/in.

F_u = 65 ksi

The factored shear rupture resistance of the web:

$$R_{r,web} = \phi_{vu} 0.58 R_p F_u A_{vn} = (0.80)(0.58)(1.0)(65)(0.5) = 15.08 \text{ kip/in.}$$

Therefore, the factored shear resistance of the weld governs.

$R_{r,web}$ = 15.08 kip/in. >

The required shear strength of 4.95 kip/in. **OK**

Since $R_{r,web} > R_{r,weld}$, the capacity of the weld controls the capacity at the connection.

A 0.3125 in. (5/16 in.) fillet weld is adequate at the flange-to-web connection and satisfies the minimum size requirement.

LRFD Table 6.13.3.4-1
SSFC Table 707-1

Step 9.5. Diaphragm Design

In this example, intermediate diaphragms are provided at the following locations (see **Figure 3**):

Span 1: 4 @ 21 ft -11 in. and 1 @ 11 ft

Span 2: 1 @ 11 ft and 4 @ 21 ft-11 in.

This step does not discuss the design procedure of a diaphragm in detail. Only the general procedure involved in the diaphragm design is discussed.

Diaphragms or cross-frames shall be provided at abutments, piers, and hinge joints. The need for diaphragms or cross-frames shall be investigated for all stages of assumed construction procedures and the final condition. Diaphragms or cross-frames required for conditions other than the final condition may be specified to be temporary bracing. BDM Art. 7.02.14A

Diaphragms and cross-frames may be placed at the following locations along the bridge: at the end of the structure, across interior supports, and intermittently along the span. The need for diaphragms or cross-frames must be investigated at all stages of assumed construction procedures and at the final condition. LRFD Art. 6.7.4.1

Step 9.5.1. Sizing

The cross-frame/diaphragm has not traditionally been treated as a primary member deserving special design attention in straight bridges with no skew; that is, structural actions are rarely calculated, and the standardized designs are generally used. FHWA (2015)

Figure 63 shows the standard diaphragm details. The depth of a diaphragm for a 33 in. deep girder is 22 in. BDG Art. 8.11.03

According to AASHTO, diaphragms or cross-frames should be a minimum of at least 0.75 of the girder depths for plate girders. LRFD Art. 6.7.4.2

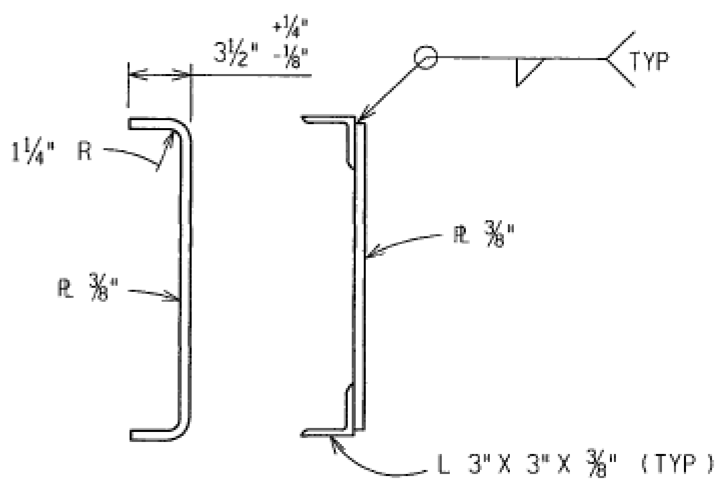


Figure 63. Recommended diaphragm details as per MDOT BDG Art. 8.11.03

End diaphragms or cross-frames are required at the ends of beams to support the end of the slab unless it is supported by other means. Diaphragms or cross-frames shall be no closer than 2' 0" from the beam end at independent back walls and shall have no less than 2'-0" of clearance at simple supports. BDM Art. 7.02.14B

End diaphragms shall be designed for forces and distortion transmitted by the deck and deck joint. End moments in diaphragms shall be considered in the design of the connection between the longitudinal component and the diaphragm. Diaphragms with span-to-depth ratios greater than or equal to 4.0 may be designed as beams. LRFD Art. 6.7.4.2

Step 9.5.2. Spacing

Where supports are not skewed more than 20 degrees from normal, an intermediate diaphragm or cross-frames may be placed in contiguous lines normal to the girders. LRFD Art. 6.7.4.2

Intermediate diaphragms or cross-frames should be provided at nearly uniform spacing in most cases for efficiency of the structural design, for constructibility, and/or to allow the use of simplified methods of analysis for calculation of flange lateral bending stresses. A tighter spacing may be desirable adjacent to interior-piers to reduce the unbraced length of the compression (bottom) flange. There are other reasons to vary the spacing of cross-frames/diaphragms.

The AASHTO Standard Specifications, 17th edition, Article 10.20.1 requirement for diaphragms spaced at not more than 25.0 ft has been replaced by a requirement for rational analysis in the AASHTO LRFD 8th Edition. LRFD Art. C6.7.4.1

When investigating the need for diaphragms or cross-frames and when designing them, the following must be considered but not limited to: LRFD Art. 6.7.4.1

- transfer of lateral wind loads from the bottom of the girder to the deck and from the deck to the bearings,
- provision of lateral support to the fascia girders between diaphragm locations to control torsional stresses and rotations due to loads applied to the overhangs, particularly during concrete deck placement,
- stability of the bottom flange for all loads when it is in compression,
- stability of the top flange in compression prior to curing of the deck,
- consideration of any flange lateral bending effects, and
- distribution of vertical dead and live loads applied to the structure.

Step 9.5.3. Connection plates

If the diaphragms, flanges or cross-frames chords are not directly attached to the girder flanges, forces from these elements are transferred through the connection plates. LRFD Art. 6.7.4.1
The connection plates shall be designed according to LRFD Art. 6.6.1.3.1.

APPENDIX A

UNFACTORED MOMENTS AND SHEAR FORCES IN EXTERIOR AND INTERIOR GIRDERS

Table A-1. Unfactored Moments in an Exterior Girder (kip-ft)

Location ($\times L_{ds}$)	Lane load		HS20-44 truck		2-HS20-28 trucks		Single 60-kip load		HS20 fatigue truck	
	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL
0.0 (Abut A)	0	0	0	0	0	0	0	0	0	0
0.1	244	-35	543	-59	562	-59	518	-51	470	-54
0.2	427	-70	920	-119	925	-119	896	-103	782	-108
0.3	545	-105	1139	-178	1141	-178	1132	-154	972	-162
0.4	602	-140	1234	-238	1234	-238	1241	-206	1030	-216
0.5	599	-175	1220	-297	1220	-297	1229	-257	1014	-270
0.6	534	-210	1103	-357	1103	-357	1108	-308	937	-324
0.7	409	-245	874	-416	875	-416	889	-360	746	-378
0.8	218	-280	550	-476	551	-476	606	-411	451	-433
0.9	43	-394	194	-535	194	-702	303	-462	170	-487
1.0 (Pier)	0	-699	0	-595	0	-1182	0	-514	0	-541
1.1	43	-394	194	-535	194	-702	303	-462	170	-487
1.2	218	-280	550	-476	551	-476	606	-411	451	-433
1.3	409	-245	874	-416	875	-416	889	-360	746	-378
1.4	534	-210	1103	-357	1103	-357	1108	-308	937	-324
1.5	599	-175	1220	-297	1220	-297	1229	-257	1014	-270
1.6	602	-140	1234	-238	1234	-238	1241	-206	1030	-216
1.7	545	-105	1139	-178	1141	-178	1132	-154	972	-162
1.8	427	-70	920	-119	925	-119	896	-103	782	-108
1.9	244	-35	543	-59	562	-59	518	-51	470	-54
2.0 (Abut B)	0	0	0	0	0	0	0	0	0	0

Table A-2. Unfactored Shear Forces in an Exterior Girder (kip)

Location ($\times L_{ds}$)	Lane load		HS20-44 truck		2-HS20-28 trucks		Single 60-kip load		HS20 fatigue truck	
	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL
0.0 (Abut A)	28.0	-3.5	63.8	-6.0	69.4	-6.0	60.0	-5.2	56.2	-5.5
0.1	22.1	-3.9	55.2	-6.1	57.3	-10.0	52.7	-7.3	47.9	-5.6
0.2	14.6	-5.2	40.4	-9.9	40.4	-16.0	40.1	-14.4	33.5	-8.2
0.3	12.5	-7.0	38.7	-17.7	38.7	-23.7	38.6	-21.4	31.9	-13.0
0.4	8.9	-9.6	30.8	-26.1	30.8	-32.0	31.7	-28.3	24.5	-18.8
0.5	5.9	-13.0	23.3	-34.2	23.3	-40.2	25.1	-34.9	17.7	-26.6
0.6	3.5	-16.9	16.3	-42.1	16.3	-48.1	18.8	-41.2	11.9	-34.7
0.7	1.8	-21.6	10.1	-49.6	10.1	-55.6	12.9	-47.1	7.7	-42.5
0.8	0.7	-26.9	4.9	-56.5	4.9	-62.2	7.5	-52.3	4.2	-49.7
0.9	0.1	-33.8	1.0	-62.5	1.0	-68.0	1.9	-56.6	1.0	-56.2
1.0 (Pier)	0.0	-40.5	0.0	-67.6	0.0	-78.3	0.0	-60.0	0.0	-62.1
1.1	33.8	-0.1	62.5	-1.0	68.0	-1.0	56.6	-1.9	56.2	-1.0
1.2	26.9	-0.7	56.5	-4.9	62.2	-4.9	52.3	-7.5	49.7	-4.2
1.3	21.6	-1.8	49.6	-10.1	55.6	-10.1	47.1	-12.9	42.5	-7.7
1.4	16.9	-3.5	42.1	-16.3	48.1	-16.3	41.2	-18.8	34.7	-11.9
1.5	13.0	-5.9	34.2	-23.3	40.2	-23.3	34.9	-25.1	26.6	-17.7
1.6	9.6	-8.9	26.1	-30.8	32.0	-30.8	28.3	-31.7	18.8	-24.5
1.7	7.0	-12.5	17.7	-38.7	23.7	-38.7	21.4	-38.6	13.0	-31.9
1.8	5.2	-14.6	9.9	-40.4	16.0	-40.4	14.4	-40.1	8.2	-33.5
1.9	3.9	-22.1	6.1	-55.2	10.0	-57.3	7.3	-52.7	5.6	-47.9
2.0 (Abut B)	3.5	-28.0	6.0	-63.8	6.0	-69.4	5.2	-60.0	5.5	-56.2

Table A-3. Unfactored Moments in an Interior Girder (kip-ft)

Location	Lane load		HS20-44 truck		2-HS20-28 trucks		Single 60-kip load		HS20 fatigue truck	
(× L_{ds})	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL
0.0 (Abut A)	0	0	0	0	0	0	0	0	0	0
0.1	244	-34	540	-59	558	-59	515	-50	467	-53
0.2	426	-68	917	-117	924	-117	892	-101	782	-106
0.3	546	-102	1144	-176	1144	-176	1135	-151	974	-159
0.4	605	-137	1242	-234	1242	-234	1248	-202	1034	-212
0.5	603	-171	1232	-293	1232	-293	1239	-252	1024	-266
0.6	540	-205	1113	-351	1113	-351	1116	-302	948	-319
0.7	415	-239	882	-410	882	-410	890	-353	754	-372
0.8	225	-273	550	-468	552	-469	597	-403	455	-425
0.9	38	-373	177	-527	177	-668	297	-453	163	-478
1.0 (Pier)	0	-683	0	-585	0	-1159	0	-504	0	-531
1.1	38	-373	177	-527	177	-668	297	-453	163	-478
1.2	225	-273	550	-468	552	-469	597	-403	455	-425
1.3	415	-239	882	-410	882	-410	890	-353	754	-372
1.4	540	-205	1113	-351	1113	-351	1116	-302	948	-319
1.5	603	-171	1232	-293	1232	-293	1239	-252	1024	-266
1.6	605	-137	1242	-234	1242	-234	1248	-202	1034	-212
1.7	546	-102	1144	-176	1144	-176	1135	-151	974	-159
1.8	426	-68	917	-117	924	-117	892	-101	782	-106
1.9	244	-34	540	-59	558	-59	515	-50	467	-53
2.0 (Abut B)	0	0	0	0	0	0	0	0	0	0

Table A-4. Unfactored Shear Forces in an Interior Girder (kip)

Location	Lane load		HS20-44 truck		2-HS20-28 trucks		Single 60-kip load		HS20 fatigue truck	
(× L_{ds})	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL	+ LL	- LL
0.0 (Abut A)	28.1	-3.5	63.9	-5.9	69.4	-5.9	60.0	-5.1	56.4	-5.4
0.1	22.2	-3.9	55.4	-6.2	57.4	-9.9	52.8	-7.2	48.1	-5.6
0.2	15.1	-5.1	41.7	-9.8	41.7	-15.8	41.2	-14.3	34.8	-8.2
0.3	12.6	-6.9	38.9	-17.6	38.9	-22.5	38.8	-21.2	32.1	-12.8
0.4	8.9	-9.5	31.0	-25.8	31.0	-31.7	31.9	-28.1	24.6	-18.7
0.5	5.9	-12.8	23.4	-34.0	23.4	-39.9	25.3	-34.7	17.8	-26.5
0.6	3.5	-16.8	16.3	-41.9	16.3	-47.8	19.0	-41.0	11.9	-34.5
0.7	1.8	-21.4	10.0	-49.5	10.0	-55.3	12.9	-47.1	7.6	-42.4
0.8	0.7	-26.6	5.0	-56.5	5.0	-62.0	7.6	-52.4	4.3	-49.6
0.9	0.0	-33.7	0.9	-62.6	0.9	-67.6	1.7	-56.6	0.9	-56.2
1.0 (Pier)	0.0	-40.1	0.0	-67.7	0.0	-78.2	0.0	-60.0	0.0	-62.1
1.1	33.7	0.0	62.6	-0.9	67.6	-0.9	56.6	-1.7	56.2	-0.9
1.2	26.6	-0.7	56.5	-5.0	62.0	-5.0	52.4	-7.6	49.6	-4.3
1.3	21.4	-1.8	49.5	-10.0	55.3	-10.0	47.1	-12.9	42.4	-7.6
1.4	16.8	-3.5	41.9	-16.3	47.8	-16.3	41.0	-19.0	34.5	-11.9
1.5	12.8	-5.9	34.0	-23.4	39.9	-23.4	34.7	-25.3	26.5	-17.8
1.6	9.5	-8.9	25.8	-31.0	31.7	-31.0	28.1	-31.9	18.7	-24.6
1.7	6.9	-12.6	17.6	-38.9	22.5	-38.9	21.2	-38.8	12.8	-32.1
1.8	5.1	-15.1	9.8	-41.7	15.8	-41.7	14.3	-41.2	8.2	-34.8
1.9	3.9	-22.2	6.2	-55.4	9.9	-57.4	7.2	-52.8	5.6	-48.1
2.0 (Abut B)	3.5	-28.1	5.9	-63.9	5.9	-69.4	5.1	-60.0	5.4	-56.4

APPENDIX B

PLASTIC MOMENT (M_P) AND YIELD MOMENT (M_Y)

B.1. NEGATIVE BENDING REGION

The plastic and yield moment calculation procedures for the section over the pier ($1.0L_{ds}$) are described.

B.1.1. Plastic moment capacity, M_p

The plastic moment, M_p, is calculated as the moment of the plastic forces about the plastic neutral axis (PNA). Plastic forces in steel portions of a cross-section are calculated using the yield strengths of the flanges, the web, and reinforcing steel, as appropriate. Plastic forces in concrete portions of the cross-section that are in compression are based on a rectangular stress block with the magnitude of the compressive stress equal to $0.85f'_c$. Concrete in tension is neglected. LRFD Art. D6.1.1

The PNA location is determined by the equilibrium condition that results in no net axial force.

Since the section over the pier in this example is noncomposite, the contribution of the concrete deck and longitudinal reinforcement are not included in the calculation of the plastic moment, M_p. LRFD Art. D6.1

The following steps show the calculation of element forces in the cross-section over the pier:

Plastic force in the tension flange:

$$P_t = F_{yt} b_t t_t$$ LRFD Art. D6.1

where:

b_t = full width of the tension flange = 18 in.

F_{yt} = specified minimum yield strength of tension flange = 50 ksi

t_t = tension flange thickness = 2.625 in.

$$P_t = 50 \times 18 \times 2.625 \quad = 2{,}362.5 \text{ kips}$$

Plastic force in the web:

$$P_w = F_{yw} D t_w$$ LRFD Art. D6.1

where:

D = web depth = 33 in

F_{yw} = specified minimum yield strength of web = 50 ksi

t_w = web thickness = 0.5 in

$$P_w = 50 \times 33 \times 0.5 \quad = 825 \text{ kips}$$

Plastic force in the compression flange:

$$P_c = F_{yc} b_c t_c$$

LRFD Art. D6.1

where:

b_c = full width of the compression flange = 18 in.

F_{yc} = specified minimum yield strength of compression flange = 50 ksi

t_c = compression flange thickness = 2.625 in.

$P_c = 50 \times 18 \times 2.625$ = 2,362.5 kips

LRFD Table D6.1-2 presents two conditions and relevant equations to determine the location of PNA and M_p for sections in negative flexure.

Case I: $P_c + P_w \geq P_t + P_{rb} + P_{rt}$ (see **Figure B.1**).

LRFD Table D6.1-2

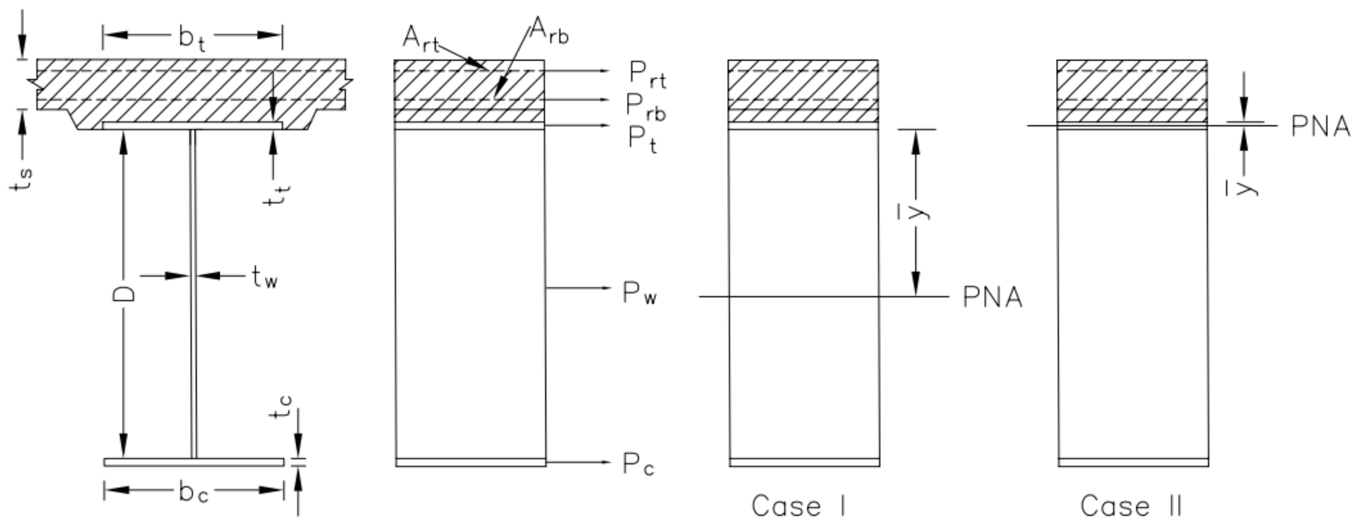


Figure B.1. Location of PNA for sections in negative flexure

Since the contribution of reinforcement is excluded, $P_{rb} = P_{rt} = 0$ kips.

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

$P_c + P_w$ = 2362.5 + 825 = 3187.5 kips

$P_t + P_{rb} + P_{rt}$ = 2362.5 + 0 + 0 = 2362.5 kips

3187.5 kips > 2362.5 kips **OK**

Therefore, PNA is in the girder web.

The distance to the PNA from the top of the web, $\bar{y}$, is computed as:

LRFD Table D6.1-2

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

$$\bar{y} = \left(\frac{33}{2}\right)\left(\frac{2362.5 - 2362.5}{825} + 1\right) = 16.5 \text{ in.}$$

Accordingly, the plastic moment capacity, M_p, LRFD Table D6.1-2

$$M_p = \frac{P_w}{2D}\left[\bar{y}^2 + (D - \bar{y})^2\right] + P_{rt}d_{rt} + P_{rb}d_{rb} + P_t d_t + P_c d_c$$

where:

d = distance from the PNA to each component

Since longitudinal reinforcement contribution is excluded, d_{rt} and d_{rb} are not calculated.

$$d_c = d_t = 16.5 + 0.5 \times 2.625 = 17.81 \text{ in.}$$

$$M_p = \frac{825}{2 \times 33}\left[16.5^2 + (33 - 16.5)^2\right] + 2362.5 \times 17.81 + 2362.5 \times 17.81 = 90{,}959 \text{ kip-in.}$$

= 7,580 kip-ft

B.1.2. Yield moment capacity, M_y

The yield moment, M_y, of a noncomposite section shall be taken as the smaller of the moment required to cause nominal first yielding in the compression flange, M_{yc}, and the moment required to cause nominal yielding in the tension flange, M_{yt}, at the strength limit state. Flange lateral bending in all types of sections shall be disregarded in this calculation. LRFD Art. D6.2.1

Symbolically,

$$F_{yf} = \frac{M_{D1}}{S_{nc}} + \frac{M_{D2}}{S_{lt}} + \frac{M_{AD}}{S_{st}}$$ LRFD Eq. D6.2.2-1

where:

F_{yf}	=	specified minimum yield strength of the flange	LRFD Art. D6.2.2
M_{D1}	=	bending moment caused by factored permanent loads applied to the noncomposite section (S_{nc})	LRFD Art. D6.2.2
M_{D2}	=	bending moment caused by factored permanent loads applied to the long-term composite section (S_{lt})	LRFD Art. D6.2.2
M_{AD}	=	additional moment applied to the short-term composite section (S_{st}) to cause the nominal yielding in either steel flange	LRFD Art. D6.2.2

Shear studs are not provided within the negative bending region. Thus, the girder is noncomposite over the pier. Therefore, LRFD Eq. D6.2.2-1 reduces to

$$F_{yf} = \frac{M_{D1} + M_{D2} + M_{AD}}{S_{nc}}$$

The yield moment is determined by taking the summation of M_{D1}, M_{D2}, and M_{AD}.

$$M_y = M_{D1} + M_{D2} + M_{AD}$$ LRFD Eq. D6.2.2-2

M_y is taken as the lesser of the moment calculated for the compression flange (M_{yc}) and tension flange (M_{yt}). LRFD Art. D6.2.1

At the girder section over the pier ($1.0L_{ds}$):

S_{nc}	= 1649 in.^{3}		**Table 3**
M_{D1}	= -1842 kip-ft	(= -378 – 1297 – 29 – 138)	**Table 10**
M_{D2} (barrier)	= -164 kip-ft		**Table 10**
M_{D2} (FWS)	= -261 kip-ft		**Table 10**

$$50 = 1.0\left(\frac{1.25 \times |-1842 - 164| + 1.50 \times \ |-261|}{1649} \times 12 + \frac{M_{AD}}{1649}\right)$$

M_{AD} = 47,662 kip-in. = 3,972 kip-ft

M_{yc} = 1.25(1842) + 1.25(164) + 1.50(261) + 3972 = 6,871 kip-ft

S_t and S_b values are the same over the pier. Therefore, $M_{yc} = M_{yt}$.

Thus, $M_y = M_{yc} = M_{yt}$ **= 6,871 kip-ft**

B.2. POSITIVE BENDING REGION

The plastic and yield moment calculation procedures for the section at $0.4L_{ds}$ distance from the abutment centerline are described.

B.2.1. Plastic moment capacity, M_p

The calculation of the plastic moment at the positive bending region is similar to the procedure implemented for the negative bending region, except that the forces in the concrete slab are calculated in addition to the forces in the tension flange, compression flange, and the web. Forces in longitudinal reinforcement are conservatively neglected, $P_{rb} = P_{rt} = 0$ kip.

LRFD Art. D6.2.1

Calculating forces in the elements at the positive bending region:

Plastic force in the tension flange:

$P_t = 50 \times 18 \times 1.625$ = 1,462.5 kips

Plastic force in the web:

$P_w = 50 \times 33 \times 0.5$ = 825 kips

Plastic force in the compression flange:

$P_c = 50 \times 18 \times 0.75$ = 675 kips

Plastic force in the concrete slab:

$P_s = 0.85f'_c b_s t_s$

where:

b_s = effective width of the concrete deck = 90.9375 in.

f'_c = design compressive strength of concrete deck = 4 ksi

t_s = concrete slab thickness = 9 in.

$P_s = 0.85 \times 4 \times 90.9375 \times 9$ = 2,782.7 kips

LRFD Table D6.1-1 presents seven conditions and relevant equations to determine the location of PNA and M_p for sections in positive flexure.

Figure B.2 shows the relevant forces and notations.

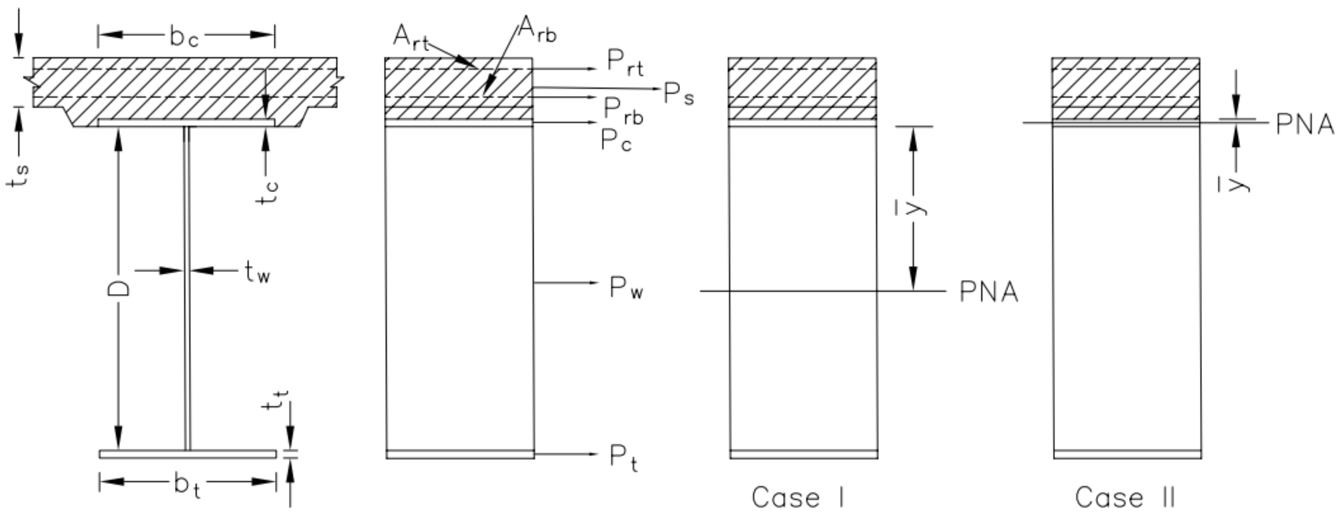


Figure B.2. Location of PNA for sections in positive flexure

<u>Case I:</u> $P_t + P_w \geq P_c + P_s + P_{rb} + P_{rt}$ — LRFD Table D6.1-1

$P_t + P_w$	= 1462.5 + 825	= 2,287.5 kips
$P_c + P_s + P_{rb} + P_{rt}$	= 675 + 2782.7 + 0 + 0	= 3,457.7 kips

2,287.5 kips < 3,457.7 kips **NOT OK**

Therefore, PNA is not in the web.

<u>Case II</u>: $P_t + P_w + P_c \geq P_s + P_{rb} + P_{rt}$ — LRFD Table D6.1-1

$P_t + P_w + P_c$	= 1462.5 + 825 + 675	= 2,962.5 kips
$P_s + P_{rb} + P_{rt}$	= 2782.7 + 0 + 0	= 2,782.7 kips

2,962.5 kips > 2,782.7 kips **OK**

Therefore, PNA is in the top flange.

The distance to the PNA from the top of the girder top flange, ȳ, is computed as: — LRFD Table D6.1-1

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

$$\bar{y} = \left(\frac{0.75}{2}\right)\left(\frac{825 + 1462.5 - 2782.7}{675} + 1\right) \quad \textbf{= 0.1 in.}$$

Plastic moment capacity, M_P: — LRFD Table D6.1-1

$$M_p = \frac{P_c}{2t_c}(\bar{y}^2 + (t_c - \bar{y})^2) + [P_s d_s + P_w d_w + P_t d_t]$$

d_w	$= 0.75 - 0.1 + 33/2$	= 17.15 in.
d_s	$= 0.1 + 1 + 9/2$	= 5.6 in.
d_t	$= 0.75 - 0.1 + 33 + 1.625/2$	= 34.46 in.

$$M_p = \frac{675}{2 \times 0.75}(0.1^2 + (0.75 - 0.1)^2) + 2782.7 \times 5.6 + 825 \times 17.15 + 1462.5 \times 34.46$$ = 80,324 kip-in.

= 6,694 kip-ft

B.2.2. Yield moment capacity, My

M_y calculation at the positive bending region is similar to **Step B.1.2**, except the section at $0.4L_{ds}$ is composite. Hence, LRFD Eq. D6.2.2.-1 is used to calculate M_{AD}.

At $0.4L_{ds}$:

M_{D1}	= 665 kip-ft	(= 127 + 477 + 10 + 51)	**Table 10**
M_{D2} (barrier)	= 93 kip-ft		**Table 10**
M_{D2} (fws)	= 148 kip-ft		**Table 10**

For the compression flange (top flange):

S_{nc}	= 583 in.^{3}	**Table 3**
S_{st}	= 9362 in.^{3}	**Table 3**
S_{lt}	= 2502 in.^{3}	**Table 3**

For the tension flange (bottom flange):

S_{nc}	= 954 in.^{3}	**Table 3**
S_{st}	= 1354 in.^{3}	**Table 3**
S_{lt}	= 1251 in.^{3}	**Table 3**

For the compression flange:

$$50 = 1.0\left(\frac{1.25 \times 665 \times 12}{583} + \frac{1.25 \times 93 \times 12 + 1.50 \times 148 \times 12}{2502} + \frac{M_{AD}}{9362}\right)$$

M_{AD} = 292,730 kip-in. = 24,394 kip-ft

Thus,

M_{yc} = 1.25(665) + 1.25(93) + 1.50(148) + 24394 = 25,564 kip-ft

For the tension flange:

$$50 = 1.0\left(\frac{1.25 \times 665 \times 12}{954} + \frac{1.25 \times 93 \times 12 + 1.50 \times 148 \times 12}{1251} + \frac{M_{AD}}{1354}\right)$$

M_{AD} = 49,149 kip-in. = 4,096 kip-ft

Thus,

M_{yt} = 1.25(665) + 1.25(93) + 1.50(148) + 4096 = 5,266 kip-ft

M_y = min (M_{yc}, M_{yt}) = min (25564, 5266) = **5,266 kip-ft**

APPENDIX C

STRENGTH LIMIT STATE SUPPLEMENTAL CALCULATIONS

C.1. AT THE NEGATIVE BENDING REGION

C.1.1. Calculation of web plastification factors (R_{pt} and R_{ct})

For compact web sections:

$$R_{pc} = \left(\frac{M_p}{M_{yc}}\right)$$ LRFD Eq. A6.2.1-4

$$R_{pt} = \left(\frac{M_p}{M_{yt}}\right)$$ LRFD Eq. A6.2.1-5

where:

M_{yc} = yield moment with respect to compression flange

= 6,871 kip-ft **Appendix B.1.2**

M_{yt} = yield moment with respect to tension flange = 6,871 kip-ft **Appendix B.1.2**

M_p = plastic moment = 7,580 kip-ft **Appendix B.1.1**

If the section has a web, which satisfies the compact web slenderness limit of Eq. A6.2.1-1, the section can reach M_p provided that other flange slenderness and lateral-torsional bracing requirements are satisfied. LRFD Art. CA6.2.1

$$\frac{2D_{cp}}{t_w} \leq \lambda_{pw(D_{cp})}$$ LRFD Eq. A6.2.1-1

where:

$\lambda_{pw(D_{cp})}$ = limiting slenderness ratio for a compact web corresponding to $\frac{2D_{cp}}{t_w}$

$$= \frac{\sqrt{\frac{E}{F_{yc}}}}{\left(0.54\frac{M_p}{R_h M_y} - 0.09\right)^2} \leq \lambda_{rw}\left(\frac{D_{cp}}{D_c}\right)$$ LRFD Eq. A6.2.1-2

λ_{rw} = limiting slenderness ratio for a noncompact web

$$= 5.7\sqrt{\frac{E}{F_{yc}}}$$ LRFD Eq. A6.2.1-3

D_c = depth of the web in compression in the elastic region

= 16.5 in. **Step 8.1.1.1.1**

D_{cp} = depth of the web in compression at the plastic moment (check the calculations shown below)

M_y = yield moment = 6,871 kip-ft **Appendix B.1.2**

R_h = hybrid factor = 1.0 LRFD Art. 6.10.1.10.1

Calculation of D_{cp}

For noncomposite sections where

$$F_{yw}A_w \geq \left|F_{yc}A_c - F_{yt}A_t\right|;$$ LRFD Eq. D6.3.2-3

$$D_{cp} = \frac{D}{2A_wF_{yw}}\left[F_{yt}A_t + F_{yw}A_w - F_{yc}A_c\right]$$ LRFD Eq. D6.3.2-4

where:

A_w	= area of the web	= 33 × 0.5	= 16.50 in.^{2}
A_c	= area of the compression flange	= 18 × 2.625	= 47.25 in.^{2}
A_t	= area of the tension flange	= 18 × 2.625	= 47.25 in.^{2}
F_{yw}	= F_{yc} = F_{yt} = 50 ksi		

For all other noncomposite sections, D_{cp} shall be taken as equal to D, the clear distance between flanges. LRFD Art. D6.3.2

$$F_{yw}A_w = 50 \times 16.50 = 825 \text{ kips} > |50 \times 47.25 - 50 \times 47.25| = 0$$ **OK**

$$D_{cp} = \frac{33}{2(50 \times 16.50)}[50 \times 47.25 + 50 \times 16.50 - 50 \times 47.25]$$ = 16.50 in.

Evaluation of the compact web slenderness limit

The following steps show the calculation of web plastification factors over the pier.

$$\lambda_{rw} = 5.7\sqrt{\frac{29000}{50}}$$ = 137.27

$$\lambda_{pw(D_{cp})} = \frac{\sqrt{\frac{29000}{50}}}{\left(0.54\frac{(7580)}{(1.0)(6871)} - 0.09\right)^2} = 94.17 < 137.27\left(\frac{16.5}{16.5}\right) = 137.27$$

Therefore, $\lambda_{pw(D_{cp})}$ = 94.17

$$\frac{2D_{cp}}{t_w} = \frac{2(16.5)}{0.5} = 66 < \lambda_{pw(Dcp)} = 94.17$$ **OK**

Check **Step 8.1.1.1.2** for the flange slenderness and the unbraced length requirements.

Since the required limits are satisfied, the section can reach M_p. Therefore,

$$R_{pt} = \left(\frac{7580}{6871}\right)$$ = 1.103

$$R_{pc} = \left(\frac{7580}{6871}\right)$$ = 1.103

C.1.2. Calculation of the moment gradient modifier, C_b

For unbraced cantilevers and for members where $M_{mid}/M_2 > 1$ or $M_2 = 0$:

$$C_b = 1.0$$ LRFD Eq. A6.3.3-6

For all other cases:

$$C_b = 1.75 - 1.05\left(\frac{M_1}{M_2}\right) + 0.3\left(\frac{M_1}{M_2}\right)^2 \leq 2.3$$ LRFD Eq. A6.3.3-7

where

M_{mid} = the major axis bending moment at the middle of the unbraced length, calculated from the moment envelope values that produce the largest compression at this point in the flange under consideration, or the smallest tension if this point is never in compression. M_{mid} shall be due to the factored loads and shall be taken as positive when it causes compression and negative when it causes tension in the flange under consideration.

M_0 = the moment at the brace point opposite to the one corresponding to M_2, calculated from the moment envelope value that produces the largest compression at this point in the flange under consideration or the smallest tension if this point was never in compression. M_0 shall be due to the factored loads and shall be taken as positive when it causes compression and negative when it causes tension in the flange under consideration.

M_1 = the moment at the brace point opposite to the one corresponding to M_2.

When variation of the moment between the brace points is concave in shape:

$$M_1 = M_0$$ LRFD Eq. A.6.3.3-11

Otherwise:

$$M_1 = 2M_{mid} - M_2 \geq M_0$$ LRFD Eq. A.6.3.3-12

M_2 = the largest major axis bending moment at either end of the unbraced length causing compression in the flange under consideration calculated from the critical moment envelope. M_2 shall be due to the factored loads and shall be taken as positive. If the moment is zero or causes tension in the flange under consideration at both ends of the unbraced length, M_2 shall be taken as zero.

From **Table 20**, factored moment under Strength I load combination at:

$1.0L_{ds}$ (i.e. over the pier, at 98.67 ft)	= - 6197 kip-ft
$1.1L_{ds}$ (i.e. at 108.537 ft)	= - 3681 kip-ft
$1.2L_{ds}$ (i.e. at 118.404 ft)	= - 2257 kip-ft

The brace point is 11 ft to the left or right of the pier (i.e. at 87.67 ft or 109.67 ft). The moment envelope is concave throughout the unbraced length for the critical moment location at the interior-pier. The moment at the brace point to the right of pier is calculated by interpolating the moments at $1.1L_{ds}$ and $1.2L_{ds}$.

Moment at brace point (i.e. at 109.67 ft)

$$= -3681 - \left(\frac{-3681-(-2257)}{118.404-108.537}\right)(118.404 - 109.67) \quad = -2{,}421 \text{ kip-ft}$$

Then:

M_2 (moment over the pier)	= - 6197 kip-ft
M_1	= - 2421 kip-ft
M_{mid} = (-6197 - 2421)/2	= - 4309 kip-ft

M_{mid}/M_2 = (-4309)/(-6197) = 0.695 < 1

$$C_b = 1.75 - 1.05\left(\frac{-2421}{-6197}\right) + 0.3\left(\frac{-2421}{-6197}\right)^2 = 1.386 < 2.3$$

Therefore, C_b = **1.386**

C.1.3. Evaluation of Appendix B6.2 and θ_{RL} Requirements

The provisions of Art. B6 shall be applied to only straight continuous span I-section members whose bearing lines are not skewed more than 10 degrees from normal and along which there are no staggered (or discontinuous) cross-frames. The specified minimum yield strength of the section must not exceed 70 ksi. In addition, the section must satisfy the web proportions (Article B6.2.1), compression flange proportions (Article B6.2.2), section transition (Article B6.2.3), compression flange bracing (Article B6.2.4), and shear (Article B6.2.5) requirements discussed below. LRFD Art. B6.2

Minimum yield strength*:*

The specified minimum yield strength of the section = 50 ksi < 70 ksi. **OK**

Web proportions: LRFD Art. B6.2.1

The web within the unbraced length shall be proportioned as:

$$\frac{D}{t_w} \leq 150$$ LRFD Eq. B6.2.1-1

$$\frac{2D_c}{t_w} \leq 6.8\sqrt{\frac{E}{F_{yc}}}$$ LRFD Eq. B6.2.1-2

$$D_{cp} \leq 0.75D$$ LRFD Eq. B6.2.1-3

where:

D_c = depth of the web in compression in the elastic range = 16.5 in. **Step 8.1.1.1.1**

D_{cp} = depth of the web in compression at the plastic moment = 16.5 in. **Appendix C.1.1**

$$\frac{D}{t_w} = \frac{33}{0.5} = 66 < 150$$ **OK**

$$\frac{2D_c}{t_w} = \frac{2 \times 16.50}{0.5} = 66 < 6.8\sqrt{\frac{E}{F_{yc}}} = 6.8 \times \sqrt{\frac{29000}{50}} = 163.77$$ **OK**

$$D_{cp} = 16.5\ in. < 0.75D = 0.75 \times 33 = 24.75\ in.$$ **OK**

Compression flange proportions: LRFD Art. B6.2.2

The compression flange within the unbraced length shall be proportioned as:

$$\frac{b_{fc}}{2t_{fc}} \leq 0.38\sqrt{\frac{E}{F_{yc}}}$$ LRFD Eq. B6.2.2-1

$$b_{fc} \geq \frac{D}{4.25}$$ LRFD Eq. B6.2.2-2

$$\frac{b_{fc}}{2t_{fc}} = \frac{18}{2 \times 2.625} = 3.43 < 0.38\sqrt{\frac{E}{F_{yc}}} = 0.38 \times \sqrt{\frac{29000}{50}} = 9.15 \qquad \textbf{OK}$$

$$b_{fc} = 18\ in. > \frac{D}{4.25} = \frac{33}{4.25} = 7.76\ in. \qquad \textbf{OK}$$

Section transitions: LRFD Art. B6.2.3

The steel I-section shall be prismatic within the unbraced length under consideration. **OK**

The unbraced length adjacent to the pier is 11 ft to the left and 11 ft to the right of the pier. The girder section remains the same within 11.25 ft to the left and 11.25 ft to the right of the pier with flange width, flange thickness, web thickness, and web depth as 18 in., 2.625 in., 0.5 in., and 33 in. Therefore, the girder is prismatic within the unbraced length.

Compression flange bracing: LRFD Art. B6.2.4

The unbraced length under consideration shall satisfy:

$$L_b \leq \left[0.1 - 0.06\left(\frac{M_1}{M_2}\right)\right]\frac{r_t E}{F_{yc}}$$ LRFD Eq. B6.2.4-1

$$L_b = 132\ in. < \left[0.1 - 0.06\left(\frac{-2421}{-6197}\right)\right]\frac{5.051 \times 29000}{50} = 224\ in. \qquad \textbf{OK}$$

See **Appendix C.1.2**. for M_1 and M_2, and **Step 8.1.1.1.2** for r_t.

Shear: LRFD Art. B6.2.5

Webs with or without transverse stiffeners within the unbraced length under consideration shall satisfy the following requirement at the strength limit state:

$$V_u \leq \phi_v V_n$$ LRFD Eq. B6.2.5-1

This check was performed in **Step 8.1.1.2**, and the condition was satisfied.

Evaluation of θ_{RL}:

For sections that satisfy the additional requirements specified in Art. B6.5.1:

$$\theta_{RL} = 0.137 - 0.143\frac{b_{fc}}{t_{fc}}\sqrt{\frac{F_{yc}}{E}} - 0.0216\frac{D}{b_{fc}} + 0.0241\frac{D}{b_{fc}}\frac{b_{fc}}{t_{fc}}\sqrt{\frac{F_{yc}}{E}}$$ LRFD Eq. B6.6.2-1

For all other sections:

$$\theta_{RL} = 0.128 - 0.143\frac{b_{fc}}{t_{fc}}\sqrt{\frac{F_{yc}}{E}} - 0.0216\frac{D}{b_{fc}} + 0.0241\frac{D}{b_{fc}}\frac{b_{fc}}{t_{fc}}\sqrt{\frac{F_{yc}}{E}}$$ LRFD Eq. B6.6.2-2

The requirements of Art. B6.5.1 are evaluated to select the equation for θ_{RL} calculation.

For interior-pier sections satisfying the requirements of Art. B6.2 and which contain: LRFD Art. B6.5.1

Transverse stiffeners spaced at D/2 or less over a minimum distance of D/2 on each side of the interior-pier section **NO**

or:

Ultracompact webs that satisfy:

$$\frac{2D_{cp}}{t_w} \leq 2.3\sqrt{\frac{E}{F_{yc}}}$$ LRFD Eq. B6.5.1-1

where:

D_{cp} = depth of the web in compression at the plastic moment = 16.5 in. **Appendix C.1.1**.

Even though this bridge does not include "ultracompact webs" (as described in LRFD Art. B6.5.1, Eq. B6.5.1-1) calculations are presented here.

$$\frac{2D_{cp}}{t_w} = \frac{2(16.50)}{0.5} = 66 < 2.3\sqrt{\frac{E}{F_{yc}}} = 2.3\sqrt{\frac{29000}{50}} = 55.4$$ **NO**

Since LRFD Art. B6.5.1 requirements are not satisfied, θ_{RL} is calculated using Eq. B6.6.2-2:

$$\theta_{RL} = 0.128 - 0.143 \times \frac{18}{2.625}\sqrt{\frac{50}{29000}} - 0.0216 \times \frac{33}{18} + 0.0241 \times \frac{33}{18} \times \frac{18}{2.625}\sqrt{\frac{50}{29000}} = 0.0603 \text{ rad}$$

APPENDIX D

CONSTRUCTIBILITY

D.1. AT THE POSITIVE BENDING REGION

The calculations are illustrated for the exterior girder section at $0.4L_{ds}$.

D.1.1. Deck placement analysis

Sections in positive flexure that are composite in the final condition, but are noncomposite during construction, shall be investigated for flexure according to the provisions of Article 6.10.3.2 during various stages of the deck placement. LRFD Art. 6.10.3.4

Geometric properties, bracing lengths and stresses used in calculating the nominal flexural resistance shall be for the steel section only.

Girder compression flange out-of-plane distortion potential can be minimized by having a compression flange that satisfies the flange proportion limits in Article 6.10.2.2 and under the following condition: LRFD Art. C6.10.3.4.1

$$b_{fc} \geq L/85$$ LRFD Eq. C6.10.3.4.1-1

where:

b_{fc} = compression flange width

L = length of the girder shipping piece

Flange proportion limits **OK** **Step 3.2.1** and **Table 2**

L is equal to the length of the girder from splice location to the abutment. **Figure 4**

L = 72 ft – 7 3/8 in. = 72.61 ft

b_{fc} = 18 in. > L/85 = 72.61 × 12/85 = 10 in. **OK**

The contractor shall complete bridge construction as per the approved plan set. The plan set includes the approved deck pour sequence. However, the contractor can propose a deck pour sequence based on available equipment and resources. The contractor's proposed methods are evaluated and approved or denied by MDOT. It is prudent to analyze alternative deck pour sequences and have the results available to the design unit to evaluate if the contractor proposes alternatives.

Table D.1 presents the exterior girder analysis results due to deck placement. In this example, the deck placement analysis is conducted by considering two different scenarios to identify the controlling cases:

(1) *Concrete is placed on the entire deck in a single day.* As shown in **Table D.1**, the total moment at $0.4L_{ds}$ is 665 kip-ft, and it acts on a noncomposite section.

(2) *Concrete is placed as per the pour sequence shown in **Figure 27**.* This is the deck pour sequence in the approved plan set. As shown in **Table D.1**, the maximum total moment acting on the noncomposite section at $0.4L_{ds}$ is 588 kip-

ft. Even though greater moments are acting at $0.4L_{ds}$ during concrete placement at C, D, and E segments, the deck at A is hardened and the girder top flange is continuously braced within that segment (i.e., from $0.4L_{ds}$ to $0.7L_{ds}$).

Table D.1. Moments from Deck Placement Analysis (kip-ft)

Location ($\times L_{ds}$)		0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
Steel girder self-weight moment, M_g (kip-ft) (a)		0	61	103	126	127	105	62	-3	-94	-217	-378
SIP formwork weight moment, M_f (kip-ft) (b)		0	26	43	51	51	41	23	-4	-40	-85	-138
Placement of the deck and haunch in a single pour												
Deck weight moment, M_D (kip-ft) (c)		0	244	404	482	477	389	218	-36	-374	-794	-1297
Haunch weight moment, M_H (kip-ft) (d)		0	5	9	11	10	9	5	-1	-8	-17	-29
Total moment at the end of concrete placement (kip-ft) (a)+(b)+(c)+(d)		0	336	560	671	**665**	545	309	-43	-516	-1112	-1841
Placement of the deck and haunch as per the defined pour sequence												
Deck and haunch moment due to concrete placement as per the pour sequence (kip-ft)	A (e)	0	104	207	311	411	448	400	268	63	-152	-366
	A & B (f)	0	67	134	201	264	265	181	11	-230	-481	-732
	A, B, & C (g)	0	280	475	586	612	553	409	181	-121	-431	-742
	A, B, C, & D (h)	0	264	443	537	547	472	312	67	-250	-577	-904
	A, B, C, D, & E (i)	0	270	456	557	573	504	350	112	-212	-620	-1112
Total moment at the end of each placement (kip-ft)	Cast A (a)+(b)+(e)	0	190	354	488	**588**	595	486	261	-71	-453	-883
	Cast B (a)+(b)+(f)	0	154	281	379	**442**	412	266	4	-364	-783	-1249
	Cast C (a)+(b)+(g)	0	367	622	764	789	700	495	174	-254	-733	-1258
	Cast D (a)+(b)+(h)	0	350	589	715	725	619	397	60	-384	-879	-1420
	Cast E (a)+(b)+(i)	0	357	602	734	750	651	436	105	-345	-921	-1629

Since the moment of 665 kip-ft acting at $0.4L_{ds}$ is critical for compression flange buckling, it is used in the subsequent constructability design checks to demonstrate the calculation process. Please note that ,to complete the entire design, every section needs to be evaluated using the moments acting on noncomposite and composite sections.

D.1.2. Deck overhang analysis

LRFD Art. C6.10.3.4

The loads applied to the deck overhang brackets induce torsion on the exterior girders, which introduces flange lateral bending stresses. This section illustrates the recommended approach to estimate these lateral bending stresses. **Figure D.1** shows the deck overhang bracket configuration used in this example.

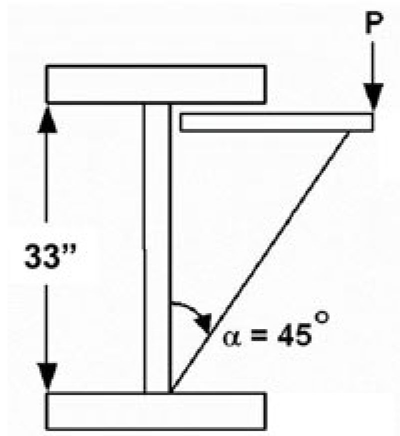


Figure D.1. Deck overhang bracket loads

Typically, the brackets are spaced between 3 and 4 ft, but the assumption is made here that the loads are uniformly distributed, except for the finishing machine. Half of the overhang weight is assumed to be supported by the exterior girder, and the remaining half is supported by the overhang brackets.

Weight of the concrete deck overhang and haunch acting on the overhang brackets,

$$\mathrm{P} = \frac{1}{2} \times (0.15)\left[\left(\frac{9}{12}\right)\left(\frac{32.625}{12}\right) + \left(\frac{1}{12}\right)\left(\frac{18/2}{12}\right)\right] \quad = 0.158 \text{ kip/ft}$$

The following is a list of typical construction loads assumed to act on the system before the concrete slab is hardened. The magnitudes of load listed are those that are applied to the overhang brackets. The finishing machine load shown here represents one-half of the finishing machine truss weight. FHWA (2015)

Overhang deck forms	= 40 lbs/ft
Screed rail	= 85 lbs/ft
Railing	= 25 lbs/ft
Walkway	= 125 lbs/ft
Finishing machine	= 3000 lbs

The lateral force acting on the girder section due to the vertical loading is:

$$\mathrm{F} = \mathrm{P} \times \tan\alpha$$

Since the overhang length is 32.625 in. from the exterior girder centerline and the girder web is 33 in., the fascia jack angle takes as 45 degrees.

In the absence of a more refined analysis, either of the following equations may be used to estimate the maximum flange lateral bending moments due to the eccentric loadings depending on how the lateral load is assumed to be applied to the top flange: LRFD Art. C6.10.3.4.1

$$M_l = \frac{F_l {L_b}^2}{12}$$ LRFD Eq. C6.10.3.4.1-2

$$M_l = \frac{P_l L_b}{8}$$ LRFD Eq. C6.10.3.4.1-3

where:

F_l	=	statically equivalent uniformly distributed lateral force from the brackets due to the factored loads
L_b	=	unbraced length
M_l	=	lateral bending moment in the top flange due to the eccentric loadings from the forming brackets
P_l	=	statically equivalent concentrated lateral bracket force placed at the middle of the unbraced length

The spacing between interior diaphragms at positive bending region is 21 ft – 11 in. (263 in.) This would be the unbraced length, L_b, at the positive bending region. **Figure 3**

D.1.3. Calculation of flexural resistance, f_{bu}

For checking of lateral-torsional buckling resistance, the correct value of the stress f_{bu} or moment M_u is generally the largest value causing compression in the flange under consideration throughout the unbraced length. LRFD Art. C6.10.1.6

f_{bu} is calculated as:

$$f_{bu} = \frac{\eta \gamma M_{construction}}{S}$$

where:

η	= redundancy factor = 1.0	**Step 7.5.3**
γ	= load factor	
	= 1.40 for Special load combination	LRFD Art. 3.4.2.1
	= 1.25 for Strength limit state	LRFD Art. 3.4.2.1
$M_{construction}$	= construction load moment	
S	= sectional modulus of the flange	

To ensure adequate stability and strength of the primary steel superstructure components during construction, an additional strength limit state load combination is specified for the investigation of loads applied to the fully erected steelwork. LRFD Art. C3.4.2.1

Unless otherwise specified by the owner, primary steel superstructure components shall be investigated for maximum force effects during construction for an additional load combination consisting of the applicable DC loads and any construction loads that are applied to the fully erected steelwork. For this additional load combination, the load factor for DC and construction loads, including dynamic effects (if applicable), shall not be less than 1.4. LRFD Art. 3.4.2.1

In this example, only the loads due to deck, haunch, formwork, and girder self-weight are used for $M_{construction}$ calculation.

$M_{construction}$ = 665 kip-ft **Table D-1**

For the strength I limit state*:*

Top flange:

$$f_{bu} = \frac{(1.0)(1.25)(665)(12)}{583} = 17.1 \text{ ksi (compression)}$$

Bottom flange:

$$f_{bu} = \frac{(1.0)(1.25)(665)(12)}{954} = 10.5 \text{ ksi (tension)}$$

For the special load combination:

Top flange:

$$f_{bu} = \frac{(1.0)(1.40)(665)(12)}{583} = 19.2 \text{ ksi (compression)}$$

Bottom flange:

$$f_{bu} = \frac{(1.0)(1.40)(665)(12)}{954} = 11.7 \text{ ksi (tension)}$$

D.1.4. Calculation of lateral bending stress, f_l

The first-order lateral bending stress, f_l, may be used if the following limit is satisfied. LRFD Art. 6.10.1.6

$$L_b \leq 1.2L_p\sqrt{\frac{C_bR_b}{f_{bu}/F_{yc}}}$$

LRFD Eq. 6.10.1.6-2

or equivalently:

$$L_b \leq 1.2L_p\sqrt{\frac{C_bR_b}{M_u/M_{yc}}}$$

LRFD Eq. 6.10.1.6-3

where:

C_b = moment gradient modifier (LRFD Art. 6.10.8.2.3)

f_{bu} = largest value of the compressive stress throughout the unbraced length in the flange under consideration, calculated without consideration of flange lateral bending

F_{yc} = specified minimum yield strength of the compression flange

L_b = unbraced length

L_p = limiting unbraced length to achieve the nominal flexural resistance of $R_bR_hF_{yc}$ under uniform bending (LRFD Art. 6.10.8.2.3)

	=	$1.0 r_t \sqrt{\frac{E}{F_{yc}}}$	LRFD Eq. 6.10.8.2.3-4
R_b	=	web load-shedding factor	LRFD Art. 6.10.1.10.2
r_t	=	effective radius of gyration for lateral-torsional buckling	
	=	$\frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\right)}}$	LRFD Eq. 6.10.8.2.3-9

At $0.4L_{ds}$:

D_c	= 21.20 in.	**Step 8.2.2.2**
$r_t = \frac{18}{\sqrt{12\left(1+\frac{1}{3}\frac{(21.20)(0.5)}{(18)(0.75)}\right)}}$	= 4.626 in.	
$L_p = 1.0 \times 4.626\sqrt{\frac{29000}{50}}$	= 111.41 in.	
f_{bu}	= 19.2 ksi	**Appendix D.1.3.**

f_{bu} is taken as the maximum stress in the top or bottom flanges calculated under the Strength I limit state and the Special load combination.

Calculation of moment gradient modifier, C_b

If $f_{mid}/f_2 > 1$ or $f_2 = 0$ — LRFD Art. 6.10.8.2.3

$$C_b = 1.0$$ LRFD Eq. 6.10.8.2.3-6

For all other cases:

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

where:

f_1	=	stress without consideration of lateral bending at the brace point opposite to the one corresponding to f_2
f_2	=	largest compressive stress without consideration of lateral bending at either end of the unbraced length of the flange under consideration
f_{mid}	=	stress without consideration of lateral bending at the middle of the unbraced length of the flange under consideration

The brace points located at either side from $0.4L_{ds}$ (39.468 ft) are at 21.917 ft and at 43.833 ft from the left abutment centerline. The sum of construction load moments at the two bracing points and at the middle of the unbraced length (32.875 ft) are interpolated as follows:

Location (ft)	19.734	**21.917**	29.601	32.875	39.468	**43.833**	49.335
Location (×Lds)	0.2	-	0.3	-	0.4	-	0.5
$M_{construction}$ (kip-ft)	622	**653**	764	**731**	665	**626**	595

The girder section remains the same throughout the braced length (i.e., S_t = 583 in.^{3}), and the top flange remains in compression throughout the unbraced length.

For the Strength I limit state:

Top flange compression, f_{tc}

At 21.917 ft: $f_{tc} = \frac{(1.0)(1.25)(653)(12)}{583}$ = 16.8 ksi

At 32.875 ft: $f_{tc} = \frac{(1.0)(1.25)(731)(12)}{583}$ = 18.8 ksi

At 43.833 ft: $f_{tc} = \frac{(1.0)(1.25)(626)(12)}{583}$ = 16.1 ksi

Therefore,

f_l = 16.8 ksi

f_2 = 16.1 ksi

f_{mid} = 18.8 ksi

Requirement for selecting an equation for C_b calculation.

f_{mid}/f_2 = 18.8/16.1 = 1.17 > 1

Therefore C_b = **1.0**

Similarly, the *special load combination* yields

f_l = 18.8 ksi

f_2 = 18.0 ksi

f_{mid} = 21.0 ksi

f_{mid}/f_2 = 21.0/18.0 = 1.17 > 1

Therefore C_b = **1.0**

Substituting values in LRFD Eq. 6.10.1.6-2:

$$L_b = 263\ in. > \ 1.2 \times 111.41 \sqrt{\frac{1.0 \times 1.0}{19.2/50}} = 216\ in.$$

Since Eq. 6.10.1.6-2 is not satisfied, second-order elastic compression-flange lateral bending stresses must be determined. LRFD Art. 6.10.1.6

The second-order compression flange lateral bending stress (f_l) may be determined by amplifying first-order values as follows:

$$f_l = \left(\frac{0.85}{1 - \frac{f_{bu}}{F_{cr}}}\right) f_{l1} \geq f_{l1}$$

LRFD Eq. 6.10.1.6-4

or

$$f_l = (AF) f_{l1} \geq f_{l1}$$

where:

AF = amplification factor = $\left(\frac{0.85}{1-\frac{f_{bu}}{F_{cr}}}\right)$

f_{l1} = first-order compression flange lateral bending stress at the section under consideration

F_{cr} = elastic lateral-torsional buckling stress for the flange under consideration

$$= \frac{C_b R_b \pi^2 E}{\left(\frac{L_b}{r_t}\right)^2}$$

LRFD Eq. 6.10.8.2.3-8

$$F_{cr} = \frac{(1.0)(1.0)\pi^2(29000)}{\left(\frac{263}{4.626}\right)^2}$$ = 88.6 ksi

The amplification factor (AF) is then determined as follows:

<u>For the strength I limit state</u>: Top flange compression

$$AF = \left(\frac{0.85}{1 - \frac{f_{bu}}{F_{cr}}}\right) = \left(\frac{0.85}{1 - \frac{17.1}{88.6}}\right) = 1.05 > 1.0$$

AF = 1.05

<u>For the special load combination</u>: Top flange compression

$$AF = \left(\frac{0.85}{1 - \frac{f_{bu}}{F_{cr}}}\right) = \left(\frac{0.85}{1 - \frac{19.2}{88.6}}\right) = 1.09 > 1.0$$

AF = 1.09

For tension flanges, AF = 1.0.

FHWA (2015)

For Strength I limit state:

When investigating Strength Load Combination I for maximum force effects during construction, load factors for the weight of the structure and appurtenances, DC and DW, shall not be less than 1.25. LRFD Art. 3.4.2.1

Unless otherwise specified by the owner, construction loads, including dynamic effects (if applicable), shall be added in Strength Load Combination I with a load factor not less than 1.5 when investigating for maximum force effect.

Dead loads are defined in **Step D.1.2.**

Dead loads

= 1.25(0.158) + 1.5(0.040 + 0.085 + 0.025 + 0.125) = 0.61 kip/ft

$F = F_l = P \times \tan\alpha = 0.61 \times \tan(45^0)$ = 0.61 kip/ft

$$M_l = \frac{F_l {L_b}^2}{12} = \frac{(0.61)(263/12)^2}{12}$$ = 24.4 kip-ft

Top flange: $f_l = \frac{M_l}{S_l} = \frac{M_l}{t_{tf} \times \frac{b_f^2}{6}} = \frac{24.4 \times 12}{0.75 \times {18^2}/{6}}$ = 7.23 ksi

Bottom flange: $f_l = \frac{M_l}{S_l} = \frac{M_l}{t_{bf} \times \frac{b_f^2}{6}} = \frac{24.4 \times 12}{1.625 \times {18^2}/{6}}$ = 3.34 ksi

Finishing machine load: P = 1.5 × (3) = 4.5 kips

$F = P_l = P \times \tan\alpha$ $= 4.5 \times \tan(45^0)$ = 4.5 kips

$$M_l = \frac{P_l L_b}{8} = \frac{(4.5)(263/12)}{8}$$ = 12.3 kip-ft

Top flange: $f_l = \frac{M_l}{S_l} = \frac{12.3 \times 12}{0.75 \times {18^2}/{6}}$ = 3.64 ksi

Bottom flange: $f_l = \frac{M_l}{S_l} = \frac{12.3 \times 12}{1.625 \times {18^2}/{6}}$ = 1.68 ksi

Total stress due to dead loads and finishing machine loads after amplification of the loads:

Top flange (compression): $f_l = (7.23 + 3.64) \times 1.05$ = 11.4 ksi

Bottom flange (tension): $f_l = (3.34 + 1.68) \times 1.00$ = 5.02 ksi

For Special load combinations:

For this additional load combination, the load factor for DC and construction loads including dynamic effects (if applicable) shall not be less than 1.4. LRFD Art. 3.4.2.1

Dead Loads = 1.4(0.158+ 0.040 + 0.085 + 0.025 + 0.125) = 0.606 kip/ft

$F = F_l = P \times \tan\alpha = 0.606 \times \tan(45^0)$ = 0.606 kip/ft

$$M_l = \frac{F_l {L_b}^2}{12} = \frac{(0.606)(263/12)^2}{12}$$ = 24.3 kip-ft

Top flange: $f_l = \frac{M_l}{S_l} = \frac{24.3 \times 12}{0.75 \times 18^2/6}$ = 7.20 ksi

Bottom flange: $f_l = \frac{M_l}{S_l} = \frac{24.3 \times 12}{1.625 \times 18^2/6}$ = 3.32 ksi

Finishing machine load: $P = 1.4 \times (3)$ = 4.2 kips

$F = P_l = P \times \tan\alpha = 4.2 \times \tan(45^0)$ = 4.2 kips

$$M_l = \frac{P_l L_b}{8} = \frac{(4.2)(263/12)}{8}$$ = 11.5 kip-ft

Top flange: $f_l = \frac{M_l}{S_l} = \frac{11.5 \times 12}{0.75 \times 18^2/6}$ = 3.41 ksi

Bottom flange: $f_l = \frac{M_l}{S_l} = \frac{11.5 \times 12}{1.625 \times 18^2/6}$ = 1.57 ksi

Total stress due to dead loads and finishing machine loads after amplification of the loads:

Top flange (compression): $f_l = (7.20 + 3.41) \times 1.09$ = 11.6 ksi

Bottom flange (tension): $f_l = (3.32 + 1.57) \times 1.00$ = 4.89 ksi

The following shows a summary of the f_{bu} and f_l used for evaluating the positive bending region stress state during construction:

Limit state	f_{bu} (ksi)		f_l (ksi)	
	Top flange (compression flange)	Bottom flange (tension flange)	Top flange (compression flange)	Bottom flange (tension flange)
Strength I	17.1	10.5	11.4	5.02
Special	19.2	11.7	11.6	4.89

The special load combination results in the highest compression flange lateral bending stress (f_l).

Since the compression flange lateral bending stress (f_l) must be less than 60 percent of the yield stress of the flange under consideration, LRFD Art. 6.10.1.6

$$f_l \leq 0.6F_{yf}$$ LRFD Eq. 6.10.1.6 -1

Top flange:

$f_l = 11.6\ ksi < 0.6F_{yf} = 0.6(50) = 30\ ksi$ **OK**

Bottom flange:

$f_l = 5.02\ ksi < 0.6F_{yf} = 0.6(50) = 30\ ksi$ **OK**

APPENDIX E

SERVICE LIMIT STATE: MOMENT REDISTRIBUTION FROM INTERIOR-PIER SECTIONS

For sections satisfying the requirements of LRFD Appendix B6.2, the redistribution of moment is permitted at the service limit state before evaluating section capacities. The redistribution of moment can be performed as per the procedures outlined in Art. B6.3 or B6.6. LRFD Art. 6.10.4.2.2

EVALUATION OF REQUIREMENTS TO REDISTRIBUTE MOMENTS

Appendix B6.2 requirements shall be satisfied to redistribute the moment from the interior-pier. This check is performed in **Appendix C.1.3.**

REDISTRIBUTION OF MOMENT AT INTERIOR-PIER SECTIONS

Since the provisions in Art. B6.2.1 - 6.2.5 are satisfied for the section over the pier, Art. B6.3 can be used to redistribute moments under the Service II limit state.

At each interior-pier section, the redistribution moment for service II loads, M_{rd}:

$$M_{rd} = |M_e| - M_{pe}$$ LRFD Eq. B6.3.3.1-1

$$0 \leq M_{rd} \leq 0.2|M_e|$$ LRFD Eq. B6.3.3.1-2

where:

M_e = critical elastic moment envelope value at the interior-pier section due to Service II loads

= - 4618 kip-ft **Table 20**

M_{pe} = negative-flexure effective plastic moment for the service limit state determined as specified in Art. B6.5

Article B6.5.2 specifies the M_{pe} to be used at the service limit state as follows:

$$M_{pe} = \left[2.90 - 2.3\frac{b_{fc}}{t_{fc}}\sqrt{\frac{F_{yc}}{E}} - 0.35\frac{D}{b_{fc}} + 0.39\frac{b_{fc}}{t_{fc}}\sqrt{\frac{F_{yc}}{E}}\frac{D}{b_{fc}}\right]M_n \leq M_n$$ LRFD Eq. B6.5.2-1

where:

M_n = smaller of M_{nc} and M_{nt} for sections with compact webs LRFD Art. B6.5.1

= 7579 kip-ft **Step 8.1.1.1**

$$M_{pe} = \left[2.90 - 2.3\frac{18}{2.625}\sqrt{\frac{50}{29000}} - 0.35\frac{33}{18} + 0.39\frac{18}{2.625}\sqrt{\frac{50}{29000}}\frac{33}{18}\right](7579)$$

= 13,696 kip-ft

$$M_{pe} = 13{,}696\ kip - ft > M_n = 7{,}579\ kip - ft$$

Therefore, M_{pe} = 7,579 kip-ft

Since M_{pe} = 7,579 kip-ft > M_e = 4,618 kip-ft, no moment redistribution is required under the Service II limit state. The elastic stresses under the Service II load combination are therefore computed without moment redistribution.

APPENDIX F

MISCELLANEOUS STEEL DESIGN

F.1. SPLICE DESIGN

F.1. 1. Calculation of the Net Area of Flanges, A_n

The net area, A_n, of an element is the product of the thickness of the element and its smallest net width. The net width shall be determined for each chain of holes extending across the member or element along any transverse, diagonal, or zigzag line. The net width for each chain shall be determined by subtracting from the width of the element, the sum of the widths of all holes in the chain and adding the quantity $s^2/4g$ for each space between consecutive holes in the chain. Here, *s* is the pitch of any two consecutive holes, and *g* is the gage of the same two holes.

LRFD Art. 6.8.3

Net area of the flange, A_n, with staggered holes:

$$A_n = t_f\left(b_f - nd_h + \sum \frac{s^2}{4g}\right)$$

Net area of the flange, A_n, for bolts in a row across the flange without staggered holes:

$$A_n = t_f\left(b_f - nd_h\right)$$

where:

- b_f = width of the flange under consideration
- d_h = diameter of standard size bolt hole as per LRFD Table 6.13.2.4.2-1
- g = gage
- n = number of bolts along the chain
- s = staggered spacing
- t_f = thickness of the flange under consideration

F.1.1.1. Top flange:

Figure F.1. shows the selection of a chain of bolt holes in the calculation of A_n for the splice plate at the top flange. Four rows of A325, 1" diameter bolts are used across the width of the flange at the splice connection.

b_f	= width of the flange under consideration	= 18 in.
d_h	= diameter of standard size bolt hole	= 1.125 in.
g	= gage	= 4.5 in.
n	= number of bolts along the chain	= 4
s	= staggered spacing	= 2.75 in.
t_f	= thickness of the flange under consideration	= 0.75 in.
A_n	$= 0.75\left(18 - 4 \times 1.125 + 2 \times \frac{2.75^2}{4 \times 4.5}\right)$	= 10.76 in.^{2}

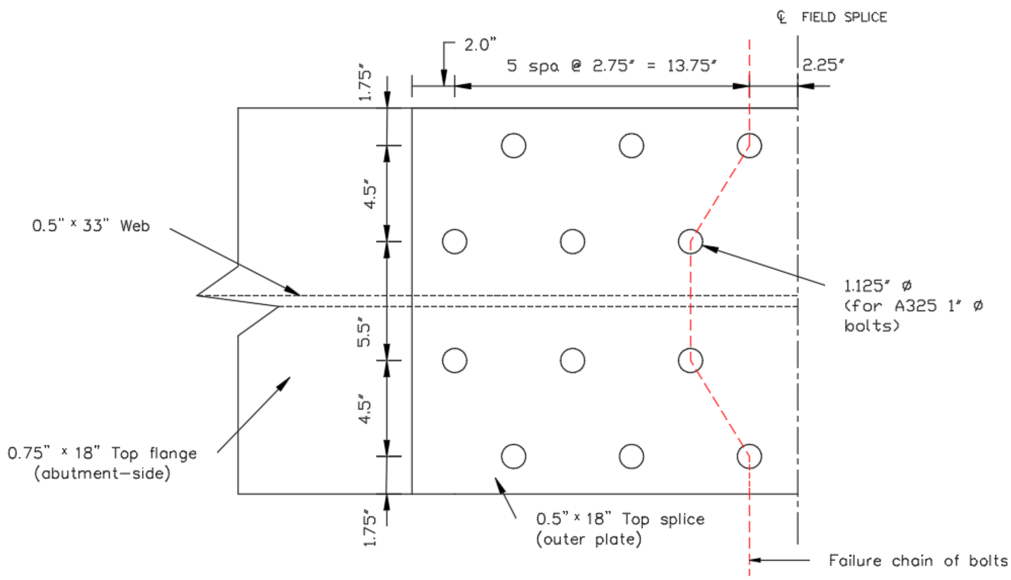


Figure F.1. Failure plane of the top flange splice plate

F.1.1.2. Bottom flange:

Figure F.2 shows the selection of a chain of bolt holes in the calculation of A_n for the splice plate at the bottom flange. Four rows of A325, 1" diameter bolts are used across the width of the flange at the splice connection.

b_f	= width of the flange under consideration	= 18 in.
d_h	= diameter of standard size bolt hole	= 1.125 in.
n	= number of bolts along the chain	= 4
t_f	= thickness of the flange under consideration	= 1.625 in.
A_n	$= 1.625(18 - 4 \times 1.125)$	= 21.9375 in.^{2}

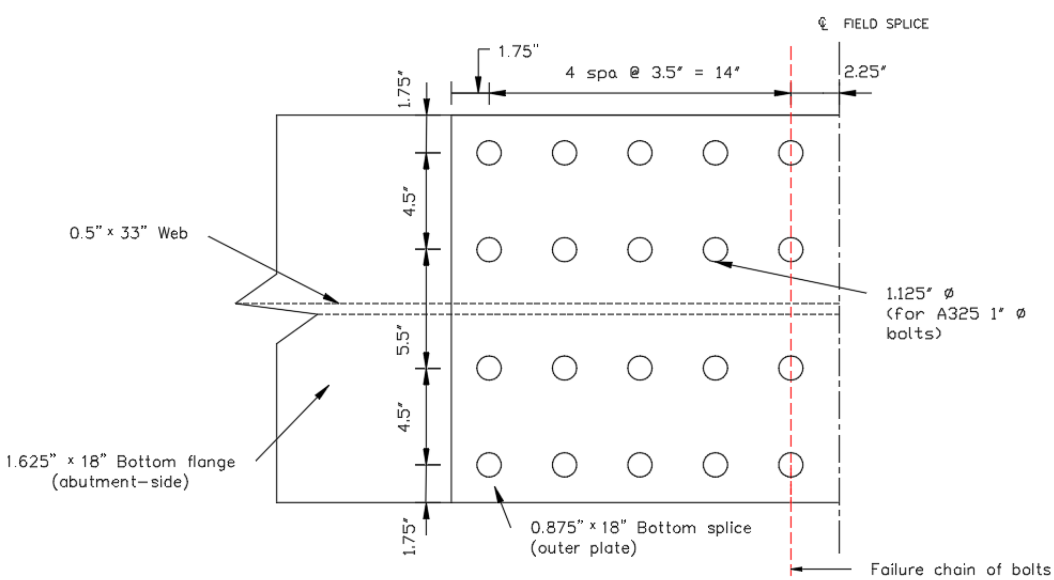


Figure F.2. Failure plane of the bottom flange splice plate

F.2. BEARING STIFFENERS

Calculation of elastic critical buckling load, P_e

The elastic critical buckling load, P_e, LRFD Art. 6.9.4.1.2

$$P_e = \frac{\pi^2 E}{\left(\frac{Kl}{r_s}\right)^2} A_g$$

LRFD Eq. 6.9.4.1.2-1

where:

A_g = gross cross-sectional area of the member

For stiffeners welded to the web, a portion of the web shall be included as part of the effective column section. For stiffeners consisting of two plates welded to the web, the effective column section shall consist of two stiffener elements, plus a concentrically located strip of web extending not more than $9t_w$ on each side of the stiffeners. LRFD Art. 6.10.11.2.4b

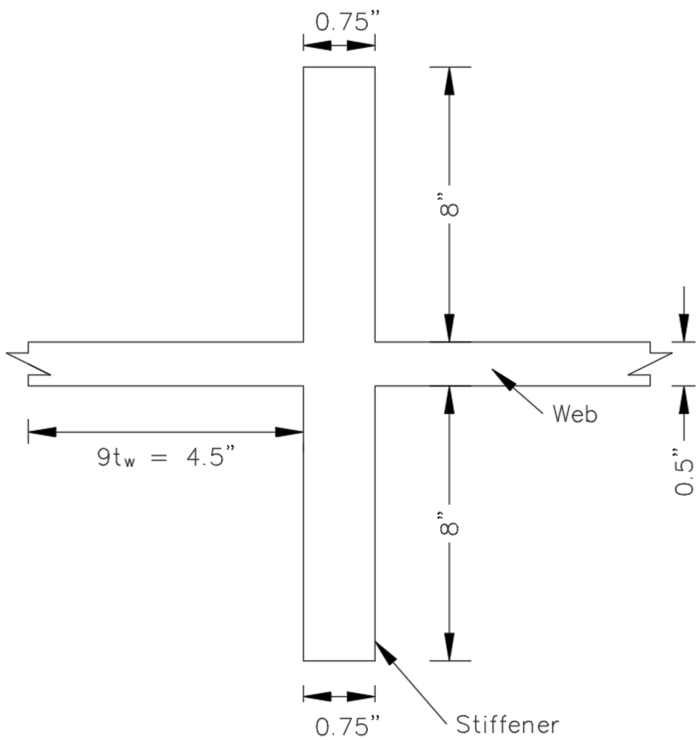


Figure F.3. Plan view of the effective column section

K = effective length factor in the plane of buckling

l = unbraced length in the plane of buckling

The effective length is taken as 0.75D, where D is the web depth. LRFD Art. 6.10.11.2.4a

r_s = radius of gyration about the axis normal to the plane of buckling

The radius of gyration is computed about the mid-thickness of the web. LRFD Art. 6.10.11.2.4a

$r_s \quad = \sqrt{\frac{I_s}{A_g}}$ FHWA (2015)

$I_s \quad$ = moment of inertia of the effective column section

The following steps show the calculation of P_e.

$A_g \quad = 2 \times (8 \times 0.75 + 9 \times 0.5 \times 0.5) \quad = 16.5 \text{ in.}^2$

$Kl \quad = 0.75 \text{ D} = 0.75 \text{ (33)} \quad = 24.75 \text{ in.}$

$I_s \quad = 0.75 \times (8 + 0.5 + 8)^3 / 12 \quad = 280.8 \text{ in.}^3$

(Neglecting the contribution of the extended web)

$$r_s = \sqrt{\frac{I_s}{A_g}} = \sqrt{\frac{280.8}{16.5}} = 4.125 \text{ in.}$$

$$P_e = \frac{\pi^2 E}{\left(\frac{Kl}{r_s}\right)^2} A_g = \frac{\pi^2 (29000)}{\left(\frac{24.75}{4.125}\right)^2} \times 16.5 = 131,\ 183 \text{ kips}$$

In addition, the bearing stiffeners must satisfy the limiting slenderness ratio, which is 120 for main members in compression. LRFD Art. 6.9.3

$$\frac{Kl}{r_s} = \frac{24.75}{4.44} = 6 < 120$$ **OK**

APPENDIX G

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