Report on Techniques for Bridge Strengthening

Design Example – Plate Girder Shear and Flexural Strengthening

September 2018

U.S. Department of Transportation

Federal Highway Administration

FHWA-HIF-18-043

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Foreword

This design example is targeted at bridge owners and bridge engineers who have been tasked with strengthening an existing bridge. It is intended to be an aid in designing appropriate bridge strengthening retrofits. Each example, in the set of examples, covers a different situation for which strengthening is commonly needed.

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<tr><td colspan="5">16. Abstract
This design example, plate girder shear and flexural strengthening, involves the addition of steel strenghening material to an existing steel plate girder. The existing bridge was designed for HS-20 live loading. The girder is to be strengthened due to section loss from corrosion. The design criteria is to strengthen the girder to obtain a HS-20 live load rating factor equal to or greater than 1.0. This example is based on AASHTO LRFD Bridge Design Specifications, 7th Edition.</td></tr>
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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
	NOTE: volumes greater than 1000 L shall be shown in m^3			
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

Design Procedure

The following American Association of State Highway and Transportation Officials (AASHTO) documents were used for this example.

Publication Title	Publication Year	Publication Number	Available for Download
AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014	2014	---	No
Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges, 2003	2003	—	Yes

Summary of Design/Analysis Procedure:

First, the bridge data, material properties, section properties and existing dead load member forces must be defined. It is also necessary to identify the standard or specification that will be used for the analysis/design along with the required design live loading and applicable load combinations and design factors.

The solution of the example will follow the following general steps:

Step 1. Calculate nominal resistance of members.

Step 2. Calculate existing bridge member load rating factors.

Step 3. Design member strengthening,

Step 4. Calculate strengthened member load rating factors.

A summary will be given at the end of the example, listing the dimensions and location of the strengthening system and the final capacity provided.

Symbols and Notation

Variables used throughout the design example are listed alphabetically below:

A = gross area of individual piece of built-up member (in.^{2})
A_1 = area of section 1 for determining the first moment of area, Q_1 (in.^{2})
A_2 = area of section 2 for determining the first moment of area, Q_2 (in.^{2})
A_b = gross area of new high strength bolt (in.^{2})
$A_{g\ bp}$ = gross area of new bottom cover plate (in.^{2})
$A_{g\ p}$ = total gross area of new cover plates (in.^{2})
$A_{g\ tp}$ = gross area of new top cover plates (in.^{2})
A_{grd} = gross area of built-up girder (in.^{2})
$A_{n\ bp}$ = net area of new bottom cover plate (in.^{2})
$A_{n\ p}$ = total net area of new cover plates (in.^{2})
$A_{n\ tp}$ = net area of new top cover plates (in.^{2})
A_{Ls} = area of new strengthening angles (in.^{2})
A_{SL} = area of section loss (in.^{2})
b_{bf} = width of the bottom flange of built-up girder (in.)
b_{tf} = width of the top flange of built-up girder (in.)
C = ratio of shear-buckling resistance to the shear specified minimum yield strength
d = distance between c.g. of built-up member and c.g. of an individual piece (in.)
d_i = distance of i^{th} bolt from the centroid of the bolt group (in.)
D_b = diameter of new high strength bolt (in.)
D_c = depth of web in compression in the elastic range (in.)
DF_{M1} = moment live load distribution for single lane
DF_{M2} = moment live load distribution for two lanes
DF_{V1} = shear live load distribution for single lane
DF_{V2} = shear live load distribution for two lanes
d_o = spacing of transverse web stiffeners on a built-up plate girder (in.)
D_w = depth of web plate of built-up member (in.)
e = distance between the center of the bolt group on one side of splice and the center of splice (in.)
E_c = modulus of elasticity of deck slab concrete (ksi)
eg = distance between centers of gravity of the beam and the deck (in.)
E_s = modulus of elasticity of steel (ksi)
$f_{b\ DCebf}$ = flexural stress in bottom flange from existing locked-in dead loads, M_{DC} (ksi)
$f_{b\ DWsbf}$ = flexural stress in bottom flange of strengthened member from M_{DW} (ksi)
$f_{b\ DWsbp}$ = flexural stress in new bottom plate of strengthened member from M_{DW} (ksi)
$f_{b\ LLsbf}$ = flexural stress in bottom flange of strengthened member from M_{LL+I} (ksi)

Symbols and Notation

Variables used throughout the design example are listed alphabetically below:

$f_{b\ LLsbp}$ = flexural stress in new bottom plate of strengthened member from M_{LL+I} (ksi)
$F_{b\ ne}$ = nominal flexural resistance stress of existing steel (ksi)
$F_{b\ nn}$ = nominal flexural resistance stress of new steel (ksi)
f_{bu} = factored stress in flange (ksi)
f'_c = compressive strength of concrete deck slab (ksi)
F_{nc} = nominal resistance of the compression flange (ksi)
$F_{nc(FLB)}$ = nominal compression flange local buckling flexural resistance (ksi)
$F_{nc(LTB)}$ = nominal compression flange lateral torsional buckling flexural resistance (ksi)
F_{nt} = nominal resistance of the tension flange (ksi)
$f_{u\ ebf}$ = factored stress in existing member bottom flange (ksi)
$f_{u\ nbp}$ = factored stress in new bottom cover plate (ksi)
F_{ub} = tensile strength of new H.S. bolt (ksi)
F_{ue} = tensile strength of existing steel (ksi)
F_{un} = tensile strength of new steel (ksi)
F_{ye} = yield strength of existing steel (ksi)
F_{yn} = yield strength of new steel (ksi)
F_{yr} = compression flange stress at onset of nominal yielding (ksi)
g = bolt or rivet gage (in.)
h_{sl} = height of section loss (in.)
IM = live load impact factor
I_o = moment of inertia of piece about its major principal axis (in.^{4})
I_x = moment of inertia of built-up member about the major principal axis (in.^{4})
J_{bg} = polar moment of inertia of bolt group (in.^{4})
k = shear buckling coefficient for webs
K_g = longitudinal stiffness parameter (in.^{4})
K_h = hole size factor (LRFD Table 6.13.2.8-2)
K_s = coefficient of friction on faying surface
L = span length of girder (in.)
L_b = unbraced member length (in.)
L_c = clear distance between edge of bolt hole and end of member (in.)
L_d = development length of bolted connection (in.)
L_p = limiting unbraced length to achieve nominal flexural resistance of M_p (in.)
L_r = limiting unbraced length to achieve onset of nominal yielding in flange (in.)
M_{DC} = moment due to dead load (k-in.)
M_{DW} = moment due to wearing surfaces and utilities (k-in.)
M_{LL+I} = moment due to live load (k-in.)
M_p = plastic moment (k-in.)
M_n = nominal moment resistance (k-in.)

Symbols and Notation

Variables used throughout the design example are listed alphabetically below:

M_r = factored moment resistance (k-in.)
M_u = moment due to factored loads (k-in.)
M_{UL} = moment from 1 kip/ft uniform load (k-in.)
M_{uv} = factored moment in splice connection due to eccentrically applied shear (k-in.)
n = modular ratio, E_s / E_c
N_b = number of bolts in a connection
n_{cs} = number of stringer s in the cross section which share a DC2 or DW uniform dead load
N_s = number of shear/slip planes in a connection
p = pitch of bolts in a connection (in.)
$P_{r\,p}$ = factored tension resistance to new cover plates (kip)
P_t = minimum bolt pretension (kip)
q_1 = shear flow at interface with section 1 (kip/in.)
Q_1 = first moment of area of section 1, about the c.g. of strengthened member (in.^{3})
q_2 = shear flow at interface with section 2 (kip/in.)
Q_2 = first moment of area of section 2, about the c.g. of strengthened member (in.^{3})
r_1 = shear force per bolt due to shear flow at interface with section 1 (kip/bolt)
r_2 = shear force per bolt due to shear flow at interface with section 1 (kip/bolt)
RF = live load rating factor
R_n = nominal bolt/rivet resistance (kip)
R_r = factored bolt/rive resistance (kip)
r_T = radius of gyration of compression flange plus 1/3 of the compression web area (in.)
S = stringer spacing in within cross section (in.)
s = spacing of bolts in a connection (in.)
S_{bf} = section modulus for bottom flange (in.^{3})
S_{bp} = section modulus for new bottom cover plate (in.^{3})
S_{ebf} = section modulus for existing bottom flange in existing condition (in.^{3})
SL = thickness of corrosion section loss (in.)
s_{min} = minimum allowable bolt spacing (in.)
s_{max} = maximum allowable bolt spacing (in.)
S_{sbf} = section modulus for existing bottom flange in strengthened condition (in.^{3})
S_{sbp} = section modulus for new bottom cover plate in strengthened condition (in.^{3})
S_{tf} = section modulus for top flange (in.^{3})
S_x = section modulus built-up member (in.^{3})
$S_{x\,min}$ = minimum section modulus between top and bottom flanges (in.^{3})
S_{xc} = section modulus for compression flange (in.^{3})
S_{xt} = section modulus for tension flange (in.^{3})

Symbols and Notation

Variables used throughout the design example are listed alphabetically below:

t = thickness of individual piece in built-up section (in.)
t_b = minimum thickness of connected material (in.)
t_{bp} = thickness of new bottom cover plate (in.)
T_b = factored tension in bolt (kip)
t_{bf} = thickness of the bottom flange plate of a built-up girder (in.)
t_{hnch} = thickness of the haunch (in.)
t_{min} = minimum thickness of connected parts (in.)
T_n = nominal tension resistance of a bolt (kip)
t_{sl} = thickness of section loss (in.)
t_{slab} = thickness the deck slab (in.)
t_{tf} = thickness of the top flange plate of a built-up girder (in.)
t_{tp} = thickness of the new top cover plates (in.)
t_w = thickness of the web plate of a built-up girder (in.)
uw_c = uniform density weight of concrete (lb./ft^3)
uw_{DC1} = uniform weight of non-composite dead load (lb./ft)
uw_{DC2} = uniform weight of composite dead load (lb./ft)
uw_{DW} = uniform density weight of wearing surface (lb./ft)
uw_p = uniform weight of parapet (lb./ft)
uw_s = uniform density weight of steel (lb./ft^3)
uw_{SIP} = uniform weight of stay-in-place forms (lb./ft^2)
uw_{ws} = uniform density weight of wearing surface dead load (lb./ft^3)
V_b = factored shear per bolt (kip/bolt)
V_{br} = factored resultant shear per bolt (kip/bolt)
V_{DC} = shear due to dead load (kip)
V_{DW} = shear due to wearing surfaces and utilities (kip)
V_{hm} = factored horizontal shear in bolt due to moment (kip/bolt)
V_{LL} = shear due to live load (kip)
V_{LL+I} = shear due to live load plus impact (kip)
V_n = nominal shear resistance (kip.)
V_p = plastic shear resistance (kip.)
V_r = factored shear resistance (kip)
V_{UL} = shear due to 1 k/ft uniform load (kip)
V_{uv} = factored shear for splice connection due to direct shear (kip)
V_{vm} = factored vertical shear in bolt due to moment (kip/bolt)
V_{vv} = factored vertical shear in bolt due to direct shear (kip/bolt)
w = width of individual piece in built-up section (in.)

Symbols and Notation

Variables used throughout the design example are listed alphabetically below:

W_b = total effective width of bolts holes for section properties (in.)
$W_{b\ bp}$ = effective width of bolts holes in new bottom cover plate for section properties (in.)
$W_{b\ tp}$ = effective width of bolts holes in new top cover plates for section properties (in.)
W_{bef} = effective width of bolts holes in existing bottom flange for section properties (in.)
$W_{c\text{-}c}$ = the curb-to-curd width for wearing surface dead load (in.)
W_g = gross width of plate (in.)
$W_{g\ bp}$ = gross width of new bottom cover plate (in.)
$W_{g\ tp}$ = gross width of new top cover plates (in.)
$W_{n\ bp}$ = net width of new bottom cover plate(in.)
$W_{n\ tp}$ = net width of new top cover plates (in.)
W_{nef} = net width of existing bottom flange (in.)
wt_{grd} = uniform weight of girder (lb./ft)
wt_{hnch} = uniform weight of haunch per girder (lb./ft)
wt_m = uniform weight of miscellaneous dead loads per girder (lb./ft)
wt_p = uniform weight of parapet per girder (lb./ft)
wt_{SIP} = uniform weight of stay-in-place forms per girder (lb./ft)
wt_{slab} = uniform weight of slab per girder (lb./ft)
wt_{ws} = uniform weight of wearing surface per girder (lb./ft)
y = distance between the c.g. of an individual piece and the c.g. of the member (in.)
y' = distance between the mid-height of the web and c.g. of the member (in.)
y_1 = distance between the c.g. of section 1 the c.g. of the built-up member (in.)
y_2 = distance between the c.g. of section 2 the c.g. of the built-up member (in.)
y_b = thickness of the flange of a rolled shape (in.)
y_{bf} = distance to the extreme fiber of the bottom flange to the c.g. of the member (in.)
y_{bp} = distance to the extreme fiber of the new bottom plate to the c.g. of the member (in.)
y_{SL} = distance to the c.g. of the section loss to the c.g. of the member (in.)
y_{tf} = distance to the extreme fiber of the top flange to the c.g. of the member (in.)
ϕM_n = factored moment resistance (k-in.)
$\phi P_{ny\ p}$ = factored resistance of new plates for yielding on gross section (kip)
$\phi P_{nu\ p}$ = factored resistance of new plates for fracture on net section (kip)
ϕR_n = factored shear resistance of bolt (kip)
ϕT_n = factored tension resistance of bolt (kip)
ϕV_n = factored shear resistance (kip)

Symbols and Notation

Variables used throughout the design example are listed alphabetically below:

ϕ_{bb} = resistance factor for bolt bearing on connected material
ϕ_f = resistance factor for flexure
ϕ_s = resistance factor for shear on bolt
ϕ_u = resistance factor for fracture on net section of tension member
ϕ_v = resistance factor for shear
ϕ_y = resistance factor for yielding on gross section of tension member
γ_{DC} = load factor for dead load, non-composite and composite
γ_{DW} = load factor for future wearing surface
γ_{LL} = load factor for live load and live load impact
η_D = load modifier for ductility
η_i = load modifier relating to ductility, redundancy and operational classification
η_I = load modifier for operational classification
η_R = load modifier for redundancy
λ_f = slenderness ratio for the compression flange
λ_{pf} = limiting slenderness ratio for a compact flange
λ_{rf} = limiting slenderness ratio for a non-compact flange
σ_1 = primary stress in the x-direction (ksi)
σ_2 = primary stress in the y-direction (ksi)
τ_{xy} = shear stress in the xy-plane (ksi)
θ = ½ angle between x and y axis on mohr's circle (degrees)

Worked Design Example

Introduction:

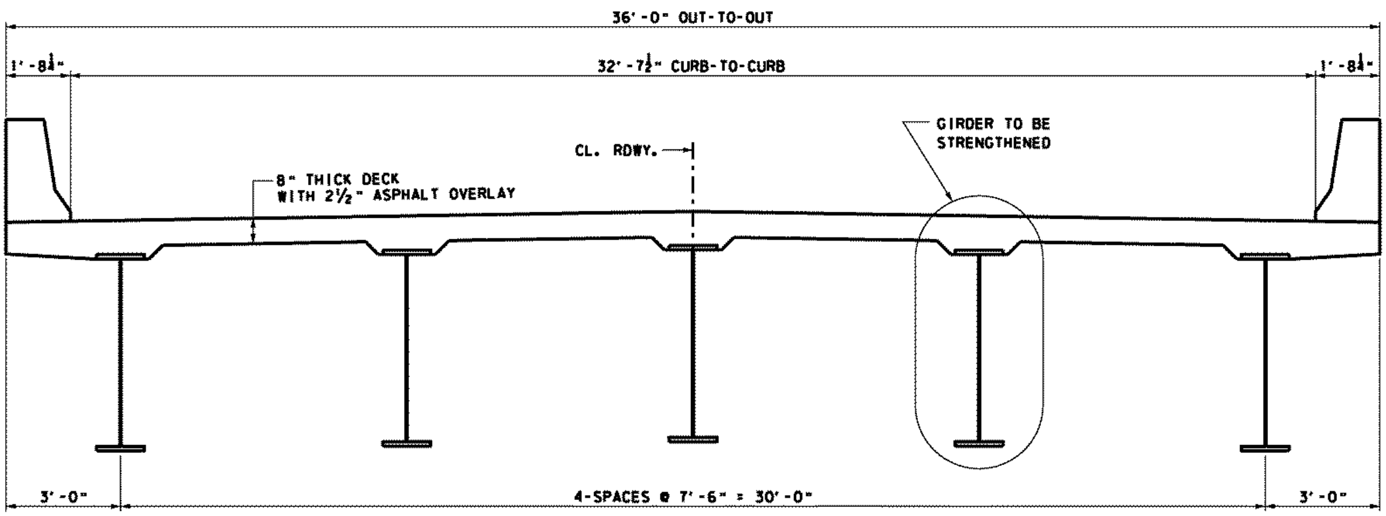
This example involves the addition of steel strengthening material to an existing steel plate girder. The existing bridge was designed for HS-20 live loading. The girder is to be strengthened due to section loss from corrosion. The design criteria is to strengthen the girder to obtain a HS-20 live load rating factor equal to or greater than 1.0. This example will be based on AASHTO LRFD Bridge Design Specifications, 7th Edition.

Bridge Data:

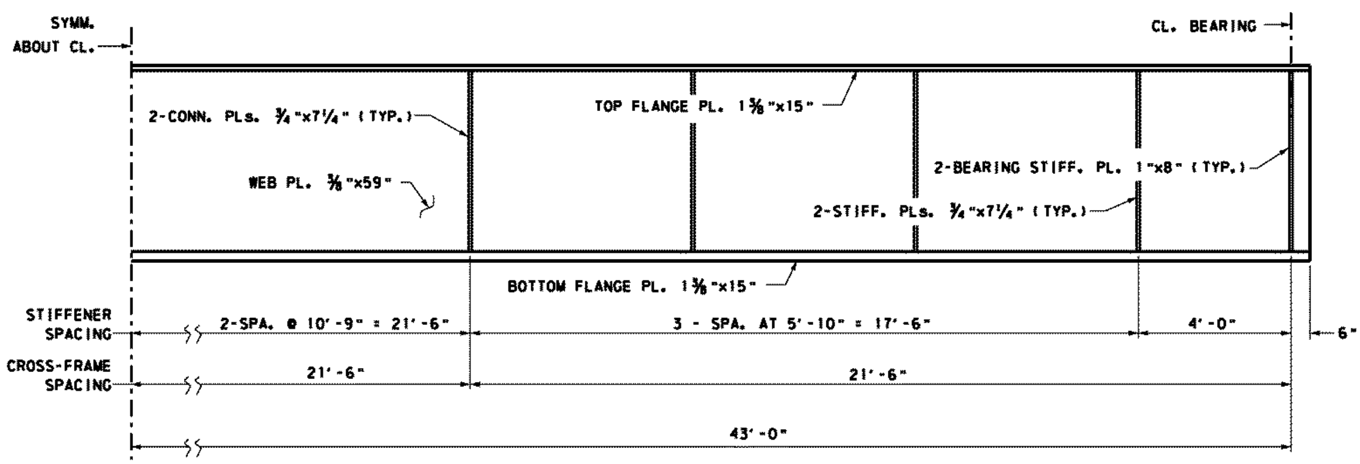
Bridge Type:	Simple span, multi steel girder bridge.
Span length:	86 ft between centerline of bearings
Year Built:	1975
Location:	State of Pennsylvania
Girder:	Non-Composite Steel Plate Girder
Barrier Type:	F-Shape (520 lb./ft.)
Out-to -Out of Bridge:	36'-0"
Curb-to -Curb Width:	32'-7$^1/_2$"
Slab Thickness:	8.0 in.
Overlay Thickness:	2.5 in.
Haunch Height:	2.75 in.
Girder Spacing:	7'-6"
Unbraced Length:	n/a. (top flange considered braced by deck slab)
Top Flange:	PL. 1$^3/_8$" x 15"
Web Plate:	PL. $^3/_8$" x 59"
Bottom Flange:	PL. 1$^3/_8$" x 15"

Material Properties:

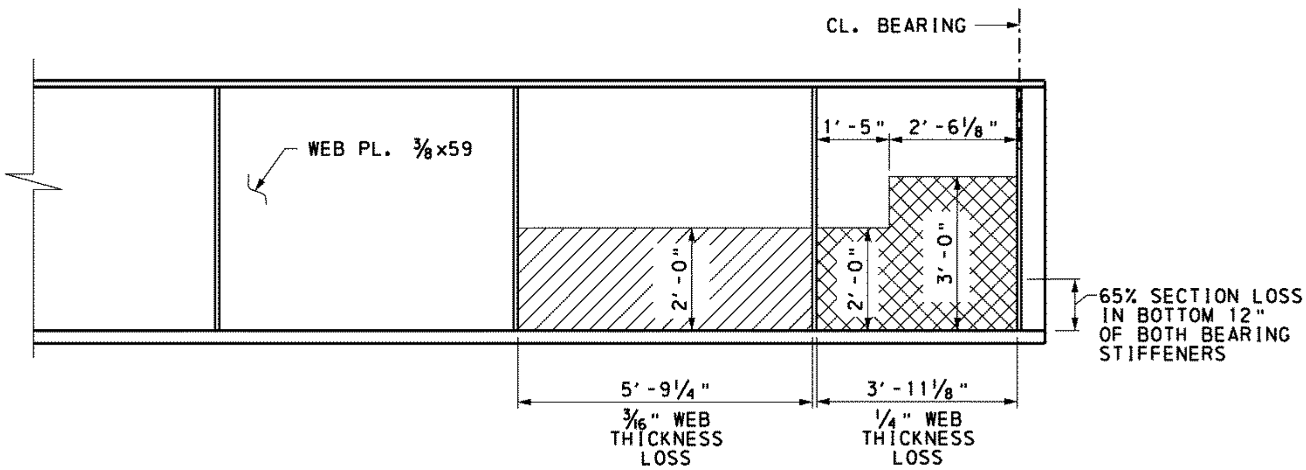
Steel Modulus of Elasticity:	E_s	= 29,000 ksi
Concrete Modulus of Elasticity:	E_c	= 3,640 ksi
Existing Steel Yield Strength:	F_{ye}	= 36 ksi (ASTM A36)
Existing Steel Tensile Strength:	F_{ue}	= 58 ksi
New Steel Yield Strength:	F_{yn}	= 50 ksi (ASTM A709, Gr. 50)
New Steel Tensile Strength:	F_{un}	= 65 ksi
New H.S. Bolt Tensile Strength:	F_{unb}	= 120 ksi (ASTM A325)
Concrete Compressive Strength:	f_c'	= 3.5 ksi
Unit Weight of Steel:	uw_s	= 490 lb./ft^3
Unit Weight of Concrete:	uw_c	= 150 lb./ft^3
Unit Weight of Overlay:	uw_{ws}	= 145 lb./ft^3



CROSS SECTION

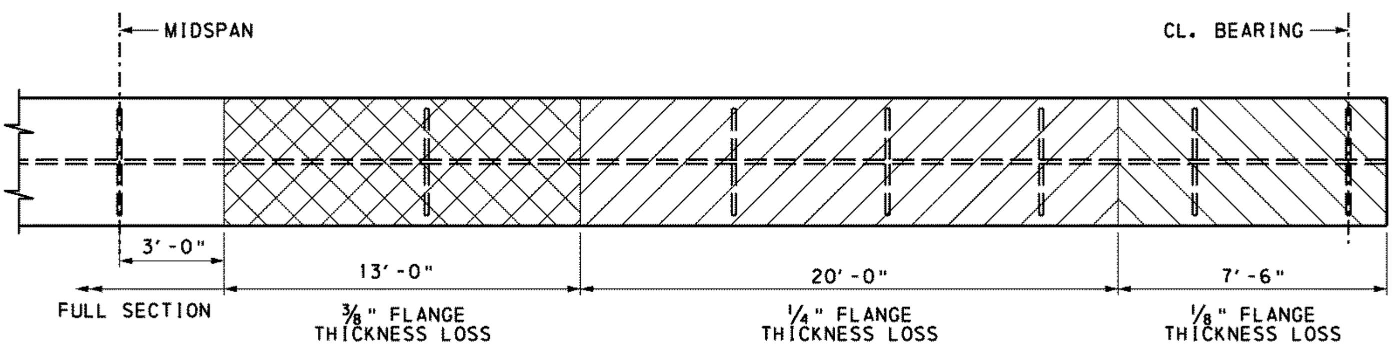


ELEVATION



ELEVATION - GIRDER WEB SECTION LOSS

(SECTION LOSS NOTED IS THE TOTAL THICKNESS LOSS IN WEB)



PLAN VIEW - BOTTOM FLANGE SECTION LOSS

(SECTION LOSS ON BOTTOM SURFACE OF BOTTOM FLANGE PLATE)

LRFD Factors:

For this example use: $\eta_D = 1.0$ $\eta_R = 1.0$ $\eta_I = 1.0$ therefore: $\eta_i = 1.0$

Resistance Factors	
Type of Resistance	**Factor, f**
Flexure	$\phi_f = 1.00$
Axial Compression	$\phi_c = 0.95$
Tension, fracture in An	$\phi_u = 0.80$
Tension, yielding in Ag	$\phi_y = 0.95$
A325 bolt in shear	$\phi_s = 0.80$
A141 rivet in shear	$\phi_s = 0.80$
Fastener bearing on material	$\phi_{bb} = 0.80$

Load Combinations and Load Factors				
Limit State	**Load Factors**			
	DC	**DW**	**LL**	**IM**
Strength I	1.25	1.50	1.75	1.75
Strength II	1.25	1.50	1.35	1.35
Service II	1.00	1.00	1.30	1.30

As-Built Condition:

Calculate the deal and live load force effects and the shear and moment resistances for the original as-built condition of the girder as a starting point. These values will be used in the strengthening design, since the design criteria is to restore the girder capacity to match or exceed the original as-built condition.

Dead Loads:

Beam self weight

$A_{grd} = t_{tf}\, b_{tf} + t_{bf}\, b_{bf} + t_w\, D_w$ = (1.375 in.)(15 in.) + (1.375 in.)(15 in.) + (0.375 in.)(59 in.)

$A_{grd} = 63.375\ \text{in.}^2$

$wt_{grd} = A_{grd}$ (12 in./ft.)(1,728 in.^{3}/ft.^{3})(uw_s) = 63.375 in.^{2} (12 in./ft.)(1,728 in.^{3}/ft.^{3})(490 lb./ft.^{3})

wt_{grd} = 215.65 lb./ft.

Deck Slab

t_{slab} = 8 in. deck slab thickness (without overlay)

S = 7.5 ft. girder spacing (tributary width for slab weight on girder)

$wt_{slab} = t_{slab}\, S\, uw_c$ (1 ft./12 in.) = (8 in.)(7.5 ft.)(150 lb./ft.^{3}) (1 ft./12 in.) = 750 lb./ft.

Dead Loads (continued):

Deck Haunch

t_{hnch} = 2.75 in. typical haunch height (top of top flange to bottom of deck slab)

w_{hnch} = 18 in. typical width of haunch (1.5 in beyond top flange on each side)

$wt_{hnch} = t_{hnch}\, w_{hnch}\, uw_c\, (1\ ft.^2/144\ in.^2) = (8\ in.)(18\ in.)(150\ lb./ft.^3)\ (1\ ft.^2/144\ in.^2) = 51.56\ lb./ft.$

Stay-in-Place Forms

uw_{SIP} = 15 psf weight of S.I.P forms per square foot

w_{SIP} = 6.25 ft. tributary width of S.I.P. forms ($S - b_{tf}$)

$wt_{SIP} = uw_{SIP}\, w_{SIP}$ = (15 psf)(6.25 ft.) = 93.75 lb./ft.

Concrete Parapet

uw_P = 520 lb./ft. weight of one f-shape parapet per linear foot

n_{cs} = 5 girders (number of girders to share the weight of the two parapets)

$wt_p = 2(uw_p)/\ n_{cs}$ = 2(520 lb./ft.)(5 girders) = 208 lb./ft.

Miscellaneous

wt_m = 50 lb./ft. assumed weight for miscellaneous items: stiffeners, cross frame, etc..

Deck Overlay (Wearing Surface)

t_{ws} = 2.5 in. overlay thickness

$W_{c\text{-}c}$ = 32.625ft. Curb-to-curb width

n_{cs} = 5 girders (number of girders to share the weight of the overlay)

$wt_{ws} = t_{ws}\, W_{c\text{-}c}\, uw_{ws}\, (1\ ft./12\ in.)\ /\ n_{cs} = (2.5\ in.)(32.625\ ft.)(145\ lb./ft.^3)\ (1\ ft./12\ in.)\ /\ (5)$

wt_{ws} = 197.1 lb./ft.

Summary

$u_{DC1} = wt_{grd} + wt_{slab} + wt_{hnch} + wt_{SIP} + wt_p + wt_m$

u_{DC1} = (215.65 + 750 + 51.56+ 93.75 + 208 + 50) lb./ft. = 1369 lb./ft.

u_{DC2} = 0 lb./ft. (assumed non-composite beam)

$u_{DW} = wt_{ws}$ = 197.1 lb./ft.

Live Loads:

Non-Composite Section Properties:

To calculate the live load distribution factors, first determine the girder section properties.

As-Built Section Properties (about major x-axis):

Piece	t (in.)	w (in.)	A ($in.^2$)	y (in.)	Ay ($in.^2$)	Ad^2 ($in.^4$)	Io ($in.^4$)
Top Flange	1.375	15	20.625	30.1875	622.62	18,795	3.25
Web	0.375	59	22.125	0.00	0.00	0.00	6,418
Bottom Flange	1.375	15	20.625	-30.1875	-622.62	18,795	3.25
Totals			63.375		0.00	37,590	6,425

A_{grd} = 63.375 $in.^2$

I_x = Σ Io + Σ Ad^2 = 37,590 $in.^4$ + 6,425 $in.^4$ = 44,015 $in.^4$

S_{xc} = I_x / y_{tf}, where $y_{tf} = D_w/2 + t_{tf}$, then S_{xc} = 44,015 $in.^4$ / 30.875 in. = 1,426 $in.^3$

S_{xt} = 1,426 $in.^3$, since in the as-built condition the girder is symmetrical about the x-axis.

Live Load Distribution Factors:

from above:

t_{slab} = 8 in L = 86.0 ft. E_s = 29,000 ksi E_c = 3,640 ksi I_x = 44,015 $in.^4$

S = 7.5 ft. A_{grd} = 63.375 $in.^2$

also calculate:

$n = E_s / E_c$ = 29,000 ksi / 3,640 ksi = 7.97, use 8.0

$eg = D_w/2 + t_{tf} + t_{hnch} + t_{slab}/2$ = 59 in./2 + 1.375 in. + 2.75 in. + 8 in./2 = 37.625 in.

$K_g = n (I_x + A_{grd}\, eg^2)$ = 8 (44,015 in.4 + 63.375 $in.^2$ (37.625 in.$)^2$) = 1,069,850 $in.^4$

$$\left(\frac{K_g}{12\,L\,t_{slab}^{\;3}}\right) = \left(\frac{1{,}069{,}850\text{in.}^4}{12\,(86\,\text{ft.})(8\,\text{in.})^3}\right) = 2.025$$

Live Loads (continued):

Interior Beam – Moment Distribution Factor: LRFD Table 4.6.2.2.2b-1

One Lane

$$DF_{M1} = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12\,L\,t_{slab}{}^{3}}\right)^{0.1} = 0.06 + \left(\frac{7.5\,ft.}{14}\right)^{0.4}\left(\frac{7.5\,ft.}{86\,ft.}\right)^{0.3}(2.025)^{0.1} = 0.462$$

Two Lanes

$$DF_{M2} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12\,L\,t_{slab}{}^{3}}\right)^{0.1} = 0.075 + \left(\frac{7.5\,ft.}{9.5}\right)^{0.6}\left(\frac{7.5\,ft.}{86\,ft.}\right)^{0.2}(2.025)^{0.1} = 0.647$$

Interior Beam – Moment Distribution Factor: LRFD Table 4.6.2.2.3a-1

One Lane

$$DF_{V1} = 0.36 + \frac{S}{25.0} = 0.36 + \frac{7.5\,ft.}{25.0} = 0.66$$

Two Lanes

$$DF_{V2} = 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^{2.0} = 0.2 + \frac{7.5\,ft.}{12.0} - \left(\frac{7.5\,ft.}{35}\right)^{2.0} = 0.779$$

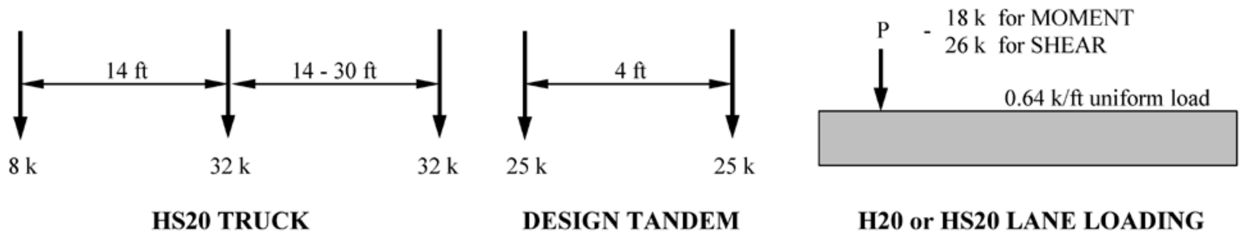
Note, multiple presence factors are included in the distribution factor equations.

Dynamic Load Allowance, IM (impact factor):

IM = 33% LRFD Table 3.6.2.1-1

Design Vehicular Live Load: LRFD Table 3.6.1.2

For this span length the design truck will control for both moment and shear. When span lengths get larger, or for continuous spans, the lane loading and tandem design loading also needs to be checked.



Shear and Moments Values:

DESIGN MOMENTS						
20th Point	x-dist. ft.	M_{UL} k-ft	M_{DC1} k-ft	M_{DW} k-ft	M_{LL} k-ft	M_{LL+I} k-ft
0.00	0	0	0	0	0	0
0.05	4.3	176	240	32	261	224
0.10	8.6	333	456	61	490	422
0.15	12.9	471	645	86	689	592
0.20	17.2	592	810	108	856	737
0.25	21.5	693	949	126	993	854
0.30	25.8	777	1,063	142	1,099	945
0.35	30.1	841	1,152	153	1,179	1,014
0.40	34.4	888	1,215	162	1,240	1,066
0.45	38.7	915	1,253	167	1,269	1,092
0.50	43.0	925	1,266	168	1,268	1,091
0.55	47.3	915	1,253	167	1,269	1,092
0.60	51.6	888	1,215	162	1,240	1,066
0.65	55.9	841	1,152	153	1,179	1,014
0.70	60.2	777	1,063	142	1,099	945
0.75	64.5	693	949	126	993	854
0.80	68.8	592	810	108	856	737
0.85	73.1	471	645	86	689	592
0.90	77.4	333	456	61	490	422
0.95	81.7	176	240	32	261	224
1.00	86.0	0	0	0	0	0

M_{UL} = Moment on simple span with 1 k./ft. uniform unit load.
M_{DC1} = Non-composite dead load moment, M_{UL} DC1 (1 kip/1,000 lb.)
M_{DW} = Dead load moment from wearing surface, M_{UL} DW (1 kip/1,000 lb.)
M_{LL} = Live load moment for HS-20 Truck
M_{LL+I} = Live load moment plus impact, (M_{LL} DF_M) (1+IM)

Shear and Moment Values (continued):

DESIGN SHEARS						
20^{th} Point	x-dist. ft.	V_{UL} kip.	V_{DC1} kip.	V_{DW} kip.	V_{LL} kip.	V_{LL+I} kip.
0.00	0.0	43.00	58.87	7.84	64.19	66.51
0.05	4.3	38.00	52.02	6.92	60.59	62.78
0.10	8.6	34.00	46.54	6.20	56.99	59.05
0.15	12.9	30.00	41.07	5.47	53.39	55.32
0.20	17.2	25.00	34.22	4.56	49.79	51.59
0.25	21.5	21.00	28.75	3.83	46.19	47.86
0.30	25.8	17.00	23.27	3.10	42.59	44.13
0.35	30.1	12.00	16.43	2.19	38.99	40.40
0.40	34.4	8.00	10.95	1.46	35.39	36.67
0.45	38.7	4.00	5.48	0.73	31.79	32.94
0.50	43.0	0.00	0.00	0.00	28.19	29.21
0.55	47.3	4.00	5.48	0.73	31.79	32.94
0.60	51.6	8.00	10.95	1.46	35.39	36.67
0.65	55.9	12.00	16.43	2.19	38.99	40.40
0.70	60.2	17.00	23.27	3.10	42.59	44.13
0.75	64.5	21.00	28.75	3.83	46.19	47.86
0.80	68.8	25.00	34.22	4.56	49.79	51.59
0.85	73.1	30.00	41.07	5.47	53.39	55.32
0.90	77.4	34.00	46.54	6.20	56.99	59.05
0.95	81.7	38.00	52.02	6.92	60.59	62.78
1.00	86.0	43.00	58.87	7.84	64.19	66.51

Note: The shear values in the table are the absolute values of the maximum shear at that location.

V_{UL} = Shear on simple span with 1 k./ft. uniform unit load.

V_{DC1} = Non-composite dead load shear, V_{UL} DC1 (1 kip/1,000 lb.)

V_{DW} = Dead load shear from wearing surface, V_{UL} DW (1 kip/1,000 lb.)

V_{LL} = Live load shear for HS-20 Truck

V_{LL+I} = Live load shear plus impact, (V_{LL} DF_V) (1+IM)

Shear and Moment Values (continued):

The shear and moments values for M_{UL}, V_{UL}, M_{LL} and V_{LL} were determined using, CONTINOUS BEAM ANALYSIS (CBA) software, for PennDOT. The unit dead loads were then factored using calculated dead load uniform weights for DC1 and DW. The basic live loads were factored using calculated distribution factors and impact factors.

Determine As-Built and As-Inspected Factored Resistances:

The top flange of the girders on this bridge are fully embedded into the concrete deck slab. Per AASHTO Manual for Condition Evaluation of Bridges, Section 6.6.9.3 and C6.6.9.3, the compression flange may be assumed to be adequately braced by the concrete deck. The flexural resistance calculation will be based on a continually braced compression flange.

As-Built Flexural Resistance:

Compression Flange Flexural Resistance – Flange Local Buckling:

$$\lambda_f = \frac{b_{tf}}{2t_{tf}} = \frac{15\,\text{in.}}{2(1.375\,\text{in.})} = 5.455$$ LRFD Eqn. 6.10.8.2.2-3

$$\lambda_{pf} = 0.38\sqrt{\frac{E_s}{F_y}} = 0.38\sqrt{\frac{29{,}000\,\text{ksi}}{36\,\text{ksi}}} = 10.785$$ LRFD Eqn. 6.10.8.2.2-4

$$\lambda_{rf} = 0.56\sqrt{\frac{E_s}{F_y}} = 0.56\sqrt{\frac{29{,}000\,\text{ksi}}{36\,\text{ksi}}} = 15.894$$ LRFD Eqn. 6.10.8.2.2-5

if $\lambda_f \le \lambda_{pf}$, $5.454 \le 10.784$ yes, then $F_{nc(FLB)} = F_y = 36\,\text{ksi}$ LRFD Eqn. 6.10.8.2.2-1

Compression Flange Flexural Resistance – Lateral Torsional Buckling:

$$r_T = \frac{b_{tf}}{\sqrt{12(1+\frac{1}{3}\frac{D_c t_w}{b_{tf} t_{tf}})}} = \frac{15\,\text{in.}}{\sqrt{12(1+\frac{1}{3}\frac{(29.5\,\text{in.})(0.375\,\text{in.})}{(15\,\text{in.})(1.375\,\text{in.})})}} = 3.988\,\text{in.}$$ LRFD Eqn. 6.10.8.2.3-9

$$L_p = 1.0\,r_T\sqrt{\frac{E}{F_y}} = 3.988\,\text{in.}\sqrt{\frac{29{,}000\,\text{ksi}}{36\,\text{ksi}}} = 113.2\,\text{in.}$$ LRFD Eqn. 6.10.8.2.3-1

if $L_b \le L_p$, $0.0\,\text{in.} \le 113.2\,\text{in.}$ yes, then $F_{nc(LTB)} = F_y = 36\,\text{ksi}$ LRFD Eqn. 6.10.8.2.3-1

As-Built Flexural Resistance:

Compression Flange Flexural Resistance:

F_{nc}= minimum of $F_{nc(FLB)}$ and $F_{nc(LTB)}$ = 36 ksi

Tension Flange Flexural Resistance:

F_{nc}= F_{ye} = 36 ksi LRFD Eqn. 6.10.8.3-1

$\phi M_n = \phi_b F_{nc} S_x =$ (1.0)(36 ksi)(1,426 in.^{3}) (1 ft. / 12 in.) = 4,278 k-ft

Determine maximum factored stress in flanges:

Since the As-Built Section is symmetrical, the stress in the top flange will be the same magnitude, but the reverse sign as the bottom flange stress.

Use moment values at 0.5 L:

Strength-I

M_{DC1} = 1,266 k-ft M_{DW} = 168 k-ft M_{LL+I} = 1,091 k-ft

$M_u = 1.25\ M_{DC1} + 1.5\ M_{DW} + 1.75\ M_{LL+I}$ = 1.25(1,266 k-ft) + 1.5(168 k-ft) + 1.75(1,091 k-ft) =

M_u = 3,744 k-ft

$$f_{bu} = \frac{M_u\,(D_w/2 + t_{tf})}{I_x} = \frac{(3{,}744\,\text{k}\cdot\text{ft})(12\,\text{in./ft.})(\frac{59\,\text{in.}}{2} + 1.375\,\text{in.})}{44{,}015\,\text{in.}^4} = 31.52\,\text{ksi}$$

$f_{bu} = 31.52\,\text{ksi} \le \phi_f\ F_{ye} = 1.0\,(36\,\text{ksi}) = 36\,\text{ksi}$, OK LRFD Eqn. 6.10.8.1.3-1

As-Inspected Flexural Resistance:

Since the top, compression, flange is continually braced, as seen above:

F_{nc} and F_{nt} = 36 ksi.

Use moment values at 0.5 L since the section loss in the bottom flange extends to within 3 ft. of midspan.

Strength-I

M_u = 3,744 k-ft (same as As-Built)

As-Inspected Flexural Resistance:

As-Inspected Section Properties (about major x-axis):

Piece	t (in.)	w (in.)	A (in.^{2})	y (in.)	Ay (in.^{2})	A(y-y')^2 (in.^{4})	Io (in.^{4})
Top Flange	1.375	15	20.625	30.1875	622.62	15,527	3.25
Web	0.375	59	22.125	0.00	0.00	198	6,418
Bottom Flange	1.375	15	20.625	-30.1875	-622.62	22,702	3.25
Section Loss	-0.375	15	-5.625	-30.6875	172.62	-6,379	-.07
Totals			57.75		172.62	31,777	6,425

$y' = \Sigma Ay / A = 172.62 \text{ in.}^3 / 57.75 \text{ in.}^2 = 2.989$ in. upwards

$A_{grd} = 57.75 \text{ in.}^2$

$I_x = \Sigma\, Io + \Sigma\, Ad^2 = 31{,}777 \text{ in.}^4 + 6{,}425 \text{ in.}^4 = 38{,}202 \text{ in.}^4$

$S_{xc} = I_x / y_{tf}$, where $y_{tf} = D_w/2 + t_{tf} - y'$, then $S_{xc} = 38{,}202 \text{ in.}^4 / 27.886 \text{ in.} = 1{,}370 \text{ in.}^3$

$S_{xt} = I_x / y_{bf}$, where $y_{bf} = D_w/2 + t_{bf} + y' - SL$, then $S_{xt} = 38{,}202 \text{ in.}^4 / 33.489 \text{ in.} = 1{,}141 \text{ in.}^3$

$S_{xmin} = = 1{,}141 \text{ in.}^3$ (tension flange controls)

$\phi M_n = \phi_b F_{nc} S_x = (1.0)(36 \text{ ksi})(1{,}141 \text{ in.}^3)(1 \text{ ft.} / 12 \text{ in.}) = 3{,}423$ k-ft

Determine maximum factored stress in flanges:

$$f_{bu} = \frac{M_u}{S_{xc}} = \frac{(3{,}744\, k \cdot ft)(12\, in./ft.)}{1{,}370\, in.^3} = 32.79\, ksi \text{ (compression flange)}$$

$$f_{bu} = 32.79\, ksi \leq \phi_f F_y = 1.0\,(36\, ksi) = 36\, ksi, \text{ OK}$$ LRFD Eqn. 6.10.8.1.3-1

$$f_{bu} = \frac{M_u}{S_{xt}} = \frac{(3{,}744\, k \cdot ft)(12\, in./ft.)}{1{,}141 in.^3} = 39.38\, ksi \text{ (tension flange)}$$

$$f_{bu} = 39.38\, ksi > \phi_f F_{ye} = 1.0\,(36\, ksi) = 36\, ksi, \text{ NG}$$ LRFD Eqn. 6.10.8.1.3-1

Strengthening of the Tension Flange is required.

Determine As-Built and As-Inspected End Panel Shear Resistance:

recall: t_w = 0.375 in. D_w =59 in. ϕ_v = 1.0

Also: d_o = 48 in. web stiffener spacing in end panel.

h_{sl} = 36 in. height of section loss

t_{sl} = 0.25 in. total web thickness loss in h_{sl}.

$$k = 5.0 + \frac{5}{\left(\frac{d_o}{D_w}\right)^2} = 5 + \frac{5}{\left(\frac{48\,in.}{59\,in.}\right)^2} = 12.554$$ LRFD Eqn. 6.20.9.3.2-7

$$\text{if } \frac{D_w}{t_w} = \frac{59\,in.}{0.375\,in.} = 157.33 \le 1.4\sqrt{\frac{E\,k}{F_y}} = 1.4\sqrt{\frac{29{,}000\,ksi(12.554)}{36\,ksi}} = 140.8,\ \text{Yes}$$

$$\text{then}: C = \frac{1.57}{(D_w/t_w)^2}\frac{E\,k}{F_y} = \frac{1.57}{(157.3)^2}\left(\frac{140.8}{1.4}\right)^2 = 0.642$$ LRFD Eqn.6.10.9.3.2-6

$$V_p = 0.58\,t_w\,D_w\,F_y = 0.58(0.375\,in.)(59\,in.)(36\,ksi) = 461.97\,kip$$ LRFD Eqn. 6.20.9.3.2-3

$$\phi_v V_n = \phi_v\,V_p\,C = 1.0(461.97\,kip)(0.642) = 296.5\,kip$$ LRFD Eqn. 6.20.9.3.2-7

For As-Inspected resistance, use the same k and C values as for the As-Built. The section loss is localized and the k and C values are based off of global web dimensions.

$$V_p = 0.58(t_w\,D_w - h_{sl}t_{sl})\,F_y = 0.58\left(0.375\,in.\,59\,in. - 36\,in.\,0.25\,in.\right)(36\,ksi) = 274.1\,kip$$

$$\phi_v V_n = \phi_v\,V_p\,C = 1.0(274.1\,kip)(0.642) = 175.9\,kip$$

Strength-I

V_{DC1} = 58.87 kip V_{DW} = 7.84 kip V_{LL+I} = 64.19 kip

V_u = 1.25 V_{DC1} + 1.5 V_{DW} + 1.75 V_{LL+I} = 1.25(58.87 kip) + 1.5(7.84 kip) + 1.75(64.19 kip) =

V_u = 197.68 kip

ϕ_v V_n = 296.5 kip >= V_u = 198 kip OK (As-Built)

ϕ_v V_n = 175.9 kip < V_u = 198 kip NG (As-Inspected)

Determine As-Built and As-Inspected Flexural Load Rating:

The general load rating equation is as follows (simplified LRFR Eqn. 6-1):

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL + I)}$$ where:

γ_{DC}, γ_{DW} and γ_{LL} are the LRFD load factors
DC, DW and LL+I are the force effects
C is the member factored capacity

Flexural Ratings:

recall:

γ_{DC} = 1.25 | γ_{Dw} = 1.5 | γ_L = 1.75

M_{DC1} = 1,266 k-ft | M_{DW} = 168 k-ft | M_{LL+I} = 1,091 k-ft

ϕM_n = 4,278 k-ft (As-Built) | ϕM_n = 3,423 k-ft (As-Inspected)

As-Built:

$$RF = \frac{\phi M_n - \gamma_{DC} M_{DC} - \gamma_{DW} M_{DW}}{\gamma_L M_{LL+I}} = \frac{4{,}278\,k\cdot ft - (1.25)(1{,}266\,k\cdot ft) - (1.5)(168\,k\cdot ft)}{(1.75)(1{,}091\,k\cdot ft)} = 1.279$$

As-Inspected:

$$RF = \frac{\phi M_n - \gamma_{DC} M_{DC} - \gamma_{DW} M_{DW}}{\gamma_L M_{LL+I}} = \frac{3{,}423\,k\cdot ft - (1.25)(1{,}266\,k\cdot ft) - (1.5)(168\,k\cdot ft)}{(1.75)(1{,}091\,k\cdot ft)} = 0.832 < 1.0 \quad NG$$

Flexural strengthening is required.

Shear Ratings (End Panel):

recall:

γ_{DC} = 1.25 | γ_{Dw} = 1.5 | γ_L = 1.75

V_{DC1} = 58.87 kip | V_{DW} = 7.84 kip | V_{LL+I} = 64.19 kip

ϕV_n 296.3 kip (As-Built) | ϕV_n = 175.8 kip (As-Inspected)

Shear Ratings (continued):

As-Built:

$$RF = \frac{\phi V_n - \gamma_{DC} V_{DC} - \gamma_{DW} V_{DW}}{\gamma_L V_{LL+I}} = \frac{296.3\,\text{kip} - (1.25)(58.87\,\text{kip}) - (1.5)(7.84\,\text{kip})}{(1.75)(64.19\,\text{kip})} = 1.878$$

As-Inspected:

$$RF = \frac{\phi V_n - \gamma_{DC} V_{DC} - \gamma_{DW} V_{DW}}{\gamma_L V_{LL+I}} = \frac{175.8\,\text{kip} - (1.25)(58.87\,\text{kip}) - (1.5)(7.84\,\text{kip})}{(1.75)(64.19\,\text{kip})} = 0.805$$

Shear strengthening is required.

Design The Member Strengthening:

Assume strengthening for both shear and flexure will consist of bolted cover plates.

Factors to consider:

- The AASHTO minimum plate thickness is 0.3125” LRFD 6.7.3
- For a bolted cover plate, the cost of the plate material is often minor compared to the labor involved with the bolting operations
- Increasing the plate thickness will increase the allowable bolt spacing for stitching and sealing requirements.
- New material installed on the outside faces of the existing member, typically provides the best increase in section properties for flexure. If cover plates are installed on both the top and bottom sides of the flange, the area of the strengthening plates above and below should be nearly equal for even distribution, similar to splice plates.
- Web plates for shear strengthening should be connected to the flange(s), to resist horizontal shear forces.
- The strengthening plates will need to be fully developed beyond the point they are required for strength to be effective.
- Bolting to an existing tension member, may cause a reduction in the effective net area of the existing member.
- There will be significant locked-in dead load stresses in the existing member. The dead load forces in the existing member at the time of strengthening, will remain in the existing member as locked-in stresses. The new material will contribute to carrying a portion of the live load, as well as any change in dead loads after strengthening.

Design Flexural Strengthening:

Determine the size of the bottom flange cover plates required.

Trial 1: 1 - Plate 3/8” x 15” (bottom side) and 2 – Plates 1/2” x 5-5/8” (top side)

$A_{g\ bp}$ = 0.375 in. x 15 in. = 5.625 in.^{2}

$A_{g\ tp}$ = 2 x 0.5 in. x 5.625 in. = 5.625 in.^{2}

$A_{g\ tp}$ / $A_{g\ bp}$ = 5.625 in.^{2} / 5.625 in.^{2} = 1.0 OK

($A_{g\ tp}$ and $A_{g\ bp}$ should be with +/- 10% of each other to be consider having equal distribution)

Check if Trail Strengthening is Adequate:

Note: With the use of spreadsheets, MathCAD sheets or other automated software, the design checks can be setup to automatically update with changes in input values. This allows for a trial and error approach, which is often easier than deriving elaborate equations to calculate the required strengthening directly.

When checking the strengthened member, the capacity of both the new material and the existing material of the strengthened member needs to be evaluated. To do this, it is often easier to deal in stress values, rather than member forces and capacities. The stress values can easily be computed using the factored loads, factored resistance and the section properties of the member.

Determine the Allowable Flexural Stresses in the Flanges and Strengthening Material.

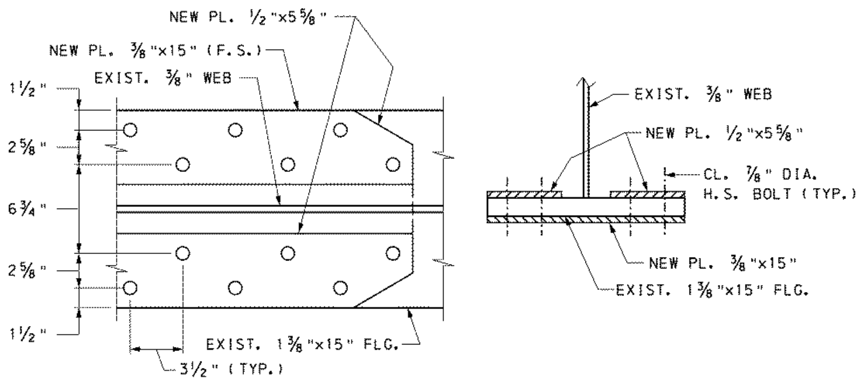
From previous calculations, it was shown that the nominal flexural resistance stress, F_{nc}, for both the tension and compression flanges is equal to F_y. This allows the flexural resistance stress to be determined directly for both the existing and new steel.

$F_{b\ ne} = \phi_f\ F_{ye}$ = 1.0 · 36 ksi = 36.0 ksi (existing steel)

$F_{b\ nn} = \phi_f\ F_{yn}$ = 1.0 · 50 ksi = 50.0 ksi (new steel)

The actual flexural stresses can be determined from the section modulus of the existing and strengthened members. For the new material the section modulus should be based on the extreme fiber distance, y, to the new material. For the existing member the section modulus should be based on the y-distance to the outer fiber of the existing material. Since the new cover plates are to be bolted to the existing flange, the net section properties should be used. Use the effective area requirements of LRFD 6.13.5.2, which limits the net area to, no greater than 85% of the gross area.

Determine the Existing Member Net Section Properties:



STRENGTHENING DETAILS

Net Width of Existing Bottom Flange

$s^2 / 4g =$ (3.5 in.)^2 / (4 · 2.625 in.) = 1.167 in. > 1.0 in. (bolt hole diameter)

will not control if there is 1-stagger per additional bolt hole, such as 4-bolts with 2-staggers.

W_{nef} = 15 in. – (2 holes)(1 in./hole) = 13 in. ≥ 0.85 (W_g) = 0.85(15 in.) = 12.75 in., use 12.75 in.

$W_b = W_g – W_{nef}$ = 15 in.–12.75 in = 2.25 in. (effective width of bolt holes for section properties.)

Piece	T (in.)	W (in.)	A (in.^{2})	Y (in.)	Ay (in.^{2})	A(y-y')^2 (in.^{4})	Io (in.^{4})
Top Flange	1.375	15	20.625	30.1875	622.62	13,837	3.25
Web	0.375	59	22.125	0.00	0.00	406	6,418
Bottom Flange	1.375	15	20.625	-30.1875	-622.62	24,511	3.25
Section Loss	-0.375	15	-5.625	-30.6875	172.62	-6,880	-0.07
Bolt Holes	-1.000	2.25	-2.25	-29.000	65.25	-2,493	
Totals			55.50		237.87	29,382	6,425

y' = ΣAy / A = 237.87 in.^{3} / 55.50 in.^{2} = 4.286 in. upwards

I_x = Σ Io + Σ Ad^2 = 29,382 in.^{4} + 6,425 in.^{4} = 35,806 in.^{4}

$S_{xc} = I_x / y_{tf}$, where $y_{tf} = D_w/2 + t_{tf} – y'$, then S_{xc} = 35,806 in.^{4} / 26.589 in. = 1,347 in.^{3}

$S_{xt} = I_x / y_{bf}$, where $y_{bf} = D_w/2 + t_{bf} + y'$ -SL, then S_{xt} = 35,806 in.^{4} / 35.161 in. = 1,018 in.^{3}

Determine the Strengthened Member Net Section Properties:

Net Width of Strengthening Plates

$W_{n\ bp}$ = 12.75 in. Bottom Plate (same as existing bottom flange)

$W_{n\ tp}$ = 5.625 in. – (1 holes)(1 in./hole) = 4.625 in. < 0.85 (W_g) = 0.85(5.625 in.) = 4.7813 in.

$W_{n\ tp}$ = 4.625 in. each Top Plate.

$W_{bef} = W - W_{nef}$ = 15 in.–12.75 in = 2.25 in. (existing flange)

$W_{b\ bp} = W - W_{nbp}$ = 15 in.–12.75 in = 2.25 in. (bottom plate)

$W_{b\ tp} = 2(W - W_{ntp})$ = 2(5.625 in.–4.625 in) = 2.00 in. (top plates)

W_b = 2.25 in. (conservative, effective width of bolts holes for section properties.)

Piece	**t** (in.)	**w** (in.)	**A** (in.^{2})	**y** (in.)	**Ay** (in.^{2})	**A(y-y')^2** (in.^{4})	**Io** (in.^{4})
Top Flange	1.375	15	20.625	30.1875	622.62	21,978	3.25
Web	0.375	59	22.125	0.00	0.00	133	6,418
Top Cov. PLs	0.5	11.25	5.625	-29.750	-167.3	4,190	0.12
Bottom Flange	1.375	15	20.625	-30.1875	-622.62	15,862	3.25
Bott. Cov. PLs	0.375	15	5.625	-31.0625	-174.7	4,603	0.07
Section Loss	-0.375	15	-5.625	-30.6875	172.62	-4,483	-0.07
Bolt Holes	-1.875	2.25	-4.219	-30.125	127.1	-3,664	-
Totals			64.78		-42.36	38,843	6,425

y' = $\Sigma Ay / A$ = -42.36 in.^{3} / 64.78 in.^{2} = -0.654 in. (downwards)

I_x = Σ Io + Σ Ad^2 = 38,843 in.^{4} + 6,425 in.^{4} = 45,268 in.^{4}

y_{tf} = $D_w/2 + t_{tf} - y'$ = 31.529 in. (centroid to top of top flange, existing)

y_{bf} = $D_w/2 + t_{bf} + y'$ = 30.221 in. (centroid to bottom of bottom flange, existing)

y_{bp} = $D_w/2 + t_{bf} + t_{bp} + y'$ = 30.596 in. (centroid to bottom of bottom cover plate, new)

S_{tf} = I / y_{tf} = 38,843 in.^{4} / 31.529 in. = 1,436 in.^{3} (top flange, existing)

S_{bf} = I / y_{bf} = 38,843 in.^{4} / 30.221 in. = 1,498 in.^{3} (bottom flange, existing)

S_{bp} = I / y_{bp} = 38,843 in.^{4} / 30.596 in. = 1,480 in.^{3} (bottom cover plate, new)

Determine the Member Flexural Stresses:

from above:

M_{DC} = 1,266 k-ft M_{DW} = 168 k-ft M_{LL+I} = 1,091 k-ft

S_{ebf} = 1,266 in^3 S_{sbf} = 1,498 in^3 S_{sbp} = 1,480 in^3

The existing, DC, dead load stresses, are locked-in to the existing member material. These stresses are determined from the DC member forces and the existing member section properties. Since the section modulus of the as-inspected existing bottom flange is significantly lower than the top flange, and the allowable stresses are the same, the existing bottom flange will control over the top flange.

locked-in dead load flexural stress on the net section of the existing member bottom flange:
$f_{b\ DCebf} = M_{DC} / S_{ebf}$ = 1,266 k-ft (12 in/ft) / 1,018 in.^{3} = 14.92 ksi

flexural stress on net section of strengthened member, existing bottom flange from M_{DW}:
$f_{b\ DWsbf} = (M_{DW} / S_{sbf})$ = 168 k-ft (12 in/ft) / 1,498 in.^{3} = 1.35 ksi

flexural stress on net section of strengthened member, new bottom plate from M_{DW}:
$f_{b\ DWsbp} = (M_{DW} / S_{sbp})$ = 168 k-ft (12 in/ft) / 1,480 in.^{3} = 1.36 ksi

flexural stress on net section of strengthened member, existing bottom flange from M_{LL+I}:
$f_{b\ LLsbf} = (M_{LL+I} / S_{sbf})$ = 1,091 k-ft (12 in/ft) / 1,498 in.^{3} = 8.74 ksi

flexural stress on net section of strengthened member, new bottom plate from M_{LL+I}:
$f_{b\ LLsbp} = (M_{LL+I} / S_{sbp})$ = 1,091 k-ft (12 in/ft) / 1,480 in.^{3} = 8.85 ksi

Factored Flexural Stresses:

Existing Bottom Flange

$f_{u\ ebf} = 1.25\,(f_{b\ DCebf}) + 1.5(f_{b\ DWsbf}) + 1.75(f_{b\ LLsbf})$
$f_{u\ ebf}$ = 1.25 (14.92 ksi) + 1.5(1.35 ksi) + 1.75(8.74 ksi) = 35.97 ksi < F_{be} = 36 ksi OK

New Bottom Strengthening Plate

$f_{u\ nbp} = 1.25\,(f_{b\ DCebp}) + 1.5(f_{b\ DWsbp}) + 1.75(f_{b\ LLsbp})$
$f_{u\ nbp}$ = 1.25 (0.0 ksi) + 1.5(1.36 ksi) + 1.75(8.85 ksi) = 17.52 ksi < F_{bn} = 50 ksi OK

Determine the Strengthened Member Flexural Rating Factor:

From the previous member stress calculations, the bottom flange material will control the rating.

Flexural Ratings:

recall:

γ_{DC} = 1.25 γ_{Dw} = 1.5 γ_{L} = 1.75

Existing Member:

$f_{b\ DCebf}$ = 14.92 ksi $f_{b\ DWsbf}$ = 1.35 ksi $f_{b\ LLsbf}$ = 8.74 ksi

$$RF = \frac{F_{b\ ne} - \gamma_{DC} f_{b\ DCebf} - \gamma_{DW} f_{b\ DWsbf}}{\gamma_L f_{b\ LLsbf}} = \frac{36.0\ \text{ksi} - (1.25)(14.95\ \text{ksi}) - (1.5)(1.35\ \text{ksi})}{(1.75)(8.74\ \text{ksi})} = 1.00\ \ \text{OK}$$

Strengthening Material:

$f_{b\ DCsbp}$ = 0.0 ksi $f_{b\ DWsbp}$ = 1.36 ksi $f_{b\ LLsbp}$ = 8.84 ksi

$$RF = \frac{F_{b\ nn} - \gamma_{DC} f_{b\ DCsbp} - \gamma_{DW} f_{b\ DWsbp}}{\gamma_L f_{b\ LLsbp}} = \frac{36.0\ \text{ksi} - (1.25)(0.0\ \text{ksi}) - (1.5)(1.36\ \text{ksi})}{(1.75)(8.85\ \text{ksi})} = 2.19\ \ \text{OK}$$

Flexural strengthening is adequate.

Note that the existing steel controls the rating by a significant amount due to the locked-in non-composite dead load forces. If these locked-in dead load forces are high enough, it might not be practical to add a sufficient amount of new strengthening material to reach the desire live load rating level. In these cases it might be possible to jack the existing structure from temporary supports, to reduced the locked-in dead loads while the new strengthening material is installed. Recall that the locked-in forces are any forces in the existing material, just before the new strengthening material is installed and the bolts are fully tightened, at which point the new and existing materials shares any forces applied at any time after.

Design Connections of Strengthening Material:

Calculate the Factored Resistance of New Bolts:

New ASTM A325, High Strength Bolts:

D_b =	0.875 in.	diameter of bolt
A_b =	0.601 in.^{2}	cross-sectional area of bolt
F_{ub} =	120 ksi	tensile strength of bolt
L_c =	1.5 in.	minimum clear distance from bolt hole to edge of connected material
t_b =	0.875 in.	minimum thickness of connected material (new plates combined)
N_s =	2 planes	number of slip plane in connection
K_h =	1.0	hole size factor (LRFD Table 6.13.2.8-2)
K_s =	0.5	coefficient of friction on faying surface (Class B surface condition)
P_t =	39 kip	minimum bolt pretension (LRFD Table 6.13.2.8-1)
ϕ_s =	0.80	resistance factor for bolts in shear
ϕ_{bb} =	0.80	resistance factor for bolts bearing on connected material

Shear Resistance where Threads are Excluded from the Shear Plane: LRFD Eq. 6.13.2.7-1

$R_n = 0.48\ A_b\ F_{ub}\ N_s =$ 0.48 (0.601 in.^{2})(120 ksi)(2 shear plane) = 69.24 kip/bolt
$R_r = \phi_s\ R_n =$ 0.80 (69.24 kip/bolt) = 55.40 kip/bolt

Bolt Bearing on Connected Material: LRFD Eq. 6.13.2.9-2

$R_n = 1.2\ L_c\ t\ F_{ue} =$ 1.2 (1.5 in.)(0.875 in.)(60 ksi) = 94.5 kip/bolt
$R_r = \phi_s\ R_n =$ 0.80 (94.5 kip/bolt) = 75.6 kip/bolt

Slip Resistance of Service II Load Case Checks: LRFD Eq. 6.13.2.8-1

$R_r = R_n = K_h\ K_s\ N_s\ P_t =$ 1.0 (0.5)(2 shear plane)(39 kip) = 39 kip/bolt

Design Connections of Strengthening Material (continued):

Determine Bolt Spacing and Edge Distance Requirements:

Minimum Edge Distance:

From LRFD Table 6.13.2.6.6-1, the minimum required edge distance is 1.5 in.,

Minimum Bolt Spacing:

s_{min} = 3.0 D_b = 3.0 (0.875 in.) = 2.625 in. , use 3.0 in. where possible   LRFD 6.13.2.6

Maximum Bolt Spacing for Sealing:

Bottom Cov. Plate: t_{min} = 0.375 in. (thickness of new bottom strengthening plate)
g = 2.625 in. (minimum outer gage)

$s_{max} \leq 4.0 + 4.0\ t_{min}$ - $(0.75\ g) \leq 7.0$, for staggered line of fasteners adjacent to free edge

s_{max} = 4.0 + 4.0 (0.375 in.) – 0.75 (2.625 in.) = 3.5 in. pitch   LRFD Eq. 6.13.2.6.2-2

Maximum Bolt Spacing for Stitching: LRFD Eq. 6.13.2.6.3-1

$s_{max} \leq 15\ t_{min}$ - $(0.375\ g) \leq 12\ t_{min}$,   12 t_{min} Controls, 12 (0.375 in.) = 4.5 in. pitch

s_{max} = 4.0 D_b = 4.0 (0.875 in.) = 3.5 in. for end 1.5 w = 9 in.   LRFD 6.13.2.6.4

Sealing Controls. <u>Use 3.5 in. pitch full length of cover plate</u>.

Determine number of bolts to develop the cover plates:
Recall:

$W_{g\ bp}$ = 15.0 in. gross width of new bottom plate

$W_{n\ bp}$ = 12.75 in. net width of new bottom plate

t_{bp} = 0.375 in. thickness of new bottom plate

$W_{g\ tp}$ = 5.625 in. gross width of each new top plate

$W_{n\ tp}$ = 4.625 in. net width of each new top plate

t_{tp} = 0.5 in. thickness each new top plate

$A_{g\ bp} = W_{g\ bp}\, t_{bp}$ = (15.0 in.)(0.375 in.) = 5.625 in.^{2}

$A_{n\ bp} = W_{n\ bp}\, t_{bp}$ = (12.75 in.)(0.375 in.) = 4.78125 in.^{2}

$A_{g\ tp} = W_{g\ tp}\, t_{tp}$ = (5.625 in.)(0.5 in.) = 2.8125 in.^{2}

$A_{n\ tp} = W_{n\ tp}\, t_{tp}$ = (4.625 in.)(0.5 in.) = 2.3125 in.^{2}

$A_{g\ p} = A_{g\ bp} + 2\ A_{g\ tp}$ = 5.625 in.^{2} + 2 (2.8125 in.^{2}) = 11.25 in.^{2} gross area of new plates
$A_{n\ p} = A_{n\ bp} + 2\ A_{n\ tp}$ = 4.7813 in.^{2} + 2 (2.313 in.^{2}) = 9.41 in.^{2} net area of new plates

ϕ_y = 0.95, resistance factor for tension, yielding on gross section
ϕ_u = 0.80, resistance factor for tension, yielding on gross section

$\phi P_{ny\ p} = \phi_y\ F_{ny}\ A_{g\ p}$ = 0.95 (50 ksi)(11.25 in.^{2}) = 534.375 kip LRFD Eq. 6.8.2.1-1
$\phi P_{nu\ p} = \phi_u\ F_{nu}\ A_{n\ p}$ = 0.80 (65 ksi)(9.41 in.^{2}) = 489.32 kip controls LRFD Eq. 6.8.2.1-2

$P_{r\ p}$ = 489.32 kip

Determine the number of bolts to develop the new plates:

$P_{r\ p}\ /\ R_r$ = 489.32 kip / (55.4 kip/bolt) = 8.83 bolts, use 10 bolts

L_d = (No. bolts / (bolts/column) – 1) p + 0.5 p + 0.5 p
L_d = (10 bolts / (2 bolts/column) - 1) 3.5 in. + 0.5(3.5 in.) + 0.5(3.5 in.) = 17.5 in.
Assume 0.5 p for end distance at both ends, where p is the bolt spacing between columns.

The new cover plates must extend a minimum of 17.5 in. past the point where they are no longer required for strength.

The ends of the 3/8 in. thickness loss region are between 0.465 L and 0.314 L. Consider extending the plates beyond this region. By inspection, 0.5 L which is in the full thickness region of the bottom flange, and is greater than 17.5 in. past the point of section loss, is adequate. Try 0.3 L as the other termination point, which is in the 1/4 in. thickness loss region, where the moment resistance is larger and the factored moments are lower.

Strength-I at 0.3 L

M_{DC1} = 1,063 k-ft M_{DW} = 142 k-ft M_{LL+I} = 945 k-ft

$M_u = 1.25\ M_{DC1} + 1.5\ M_{DW} + 1.75\ M_{LL+I}$ = 1.25(1,063 k-ft) + 1.5(142 k-ft) + 1.75(945 k-ft) =

M_u = 3,196 k-ft

Determine the moment resistance at 0.3 L (1/4 in. bottom flange thickness loss):

Recall As-Built Section Properties:

t_{bf} = 1.375 in.	b_{bf} = 15.0 in.	t_{tf} = 1.375 in.	b_{tf} = 15.0 in.
D_w = 59.0 in.	t_w = 0.375 in.	A_{grd} = 63.375 in.^{2}	I_x = 44,015 in.^{4}

Section Loss Properties:
t_{SL} = 0.25 in. thickness loss amount in region being investigated.

A_{SL} = (-0.25 in.)(15 in.) = -3.75 in.^{2}

Neglect $I_{o\ SL}$ and consider all thickness loss occurring on the bottom of the bottom flange.

y_{SL} = - (D_w / 2 + t_{bf} - t_{SL} /2) = -(59 in./2 + 1.375 in. – 0.25 in./2) = -30.75 in.

= centroid of section loss from center of web

y' = ΣAy / A = ((0 in.)(63.375 in.^{2}) + (-30.75 in.)(-3.75 in.^{2})) / (63.375 in.^{2} – 3.75 in.^{2})
y' = 1.934 in. from center of web towards top flange

I_x = Σ Io + Σ Ad^2
= 44,015 in.^{4} + (63.375 in.^{2})(1.934 in.)^2 + (-3.75 in.^{2})(30.75 in.+1.934 in.)^2
= 40,246 in.^{4}

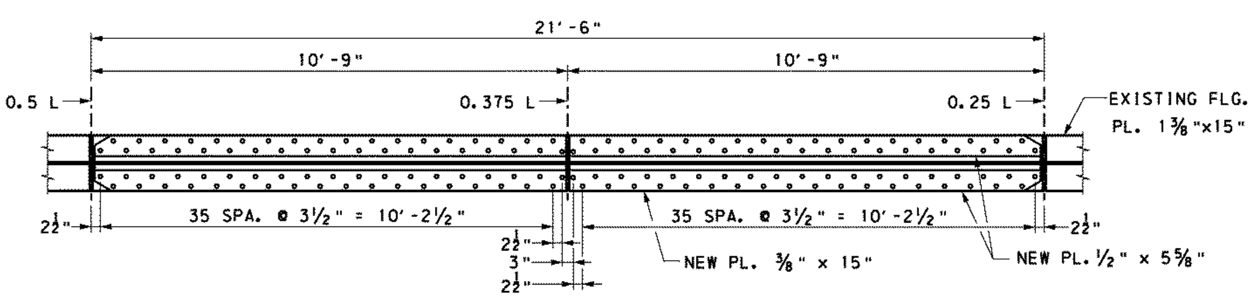
S_{xt} = I_x / y_{bf} , where y_{bf} = D_w/2 + t_{bf} + y' - SL, then S_{xt} = 40,246 in.^{4} / 32.559 in.

= 1,236 in.^{3}

ϕM_n = $\phi_b F_{nc} S_x$ = (1.0)(36 ksi)(1,236 in.^{3}) (1 ft. / 12 in.) = 3,708 k-ft

ϕM_n = 3,708 k-ft > M_u = 3,196 k-ft OK

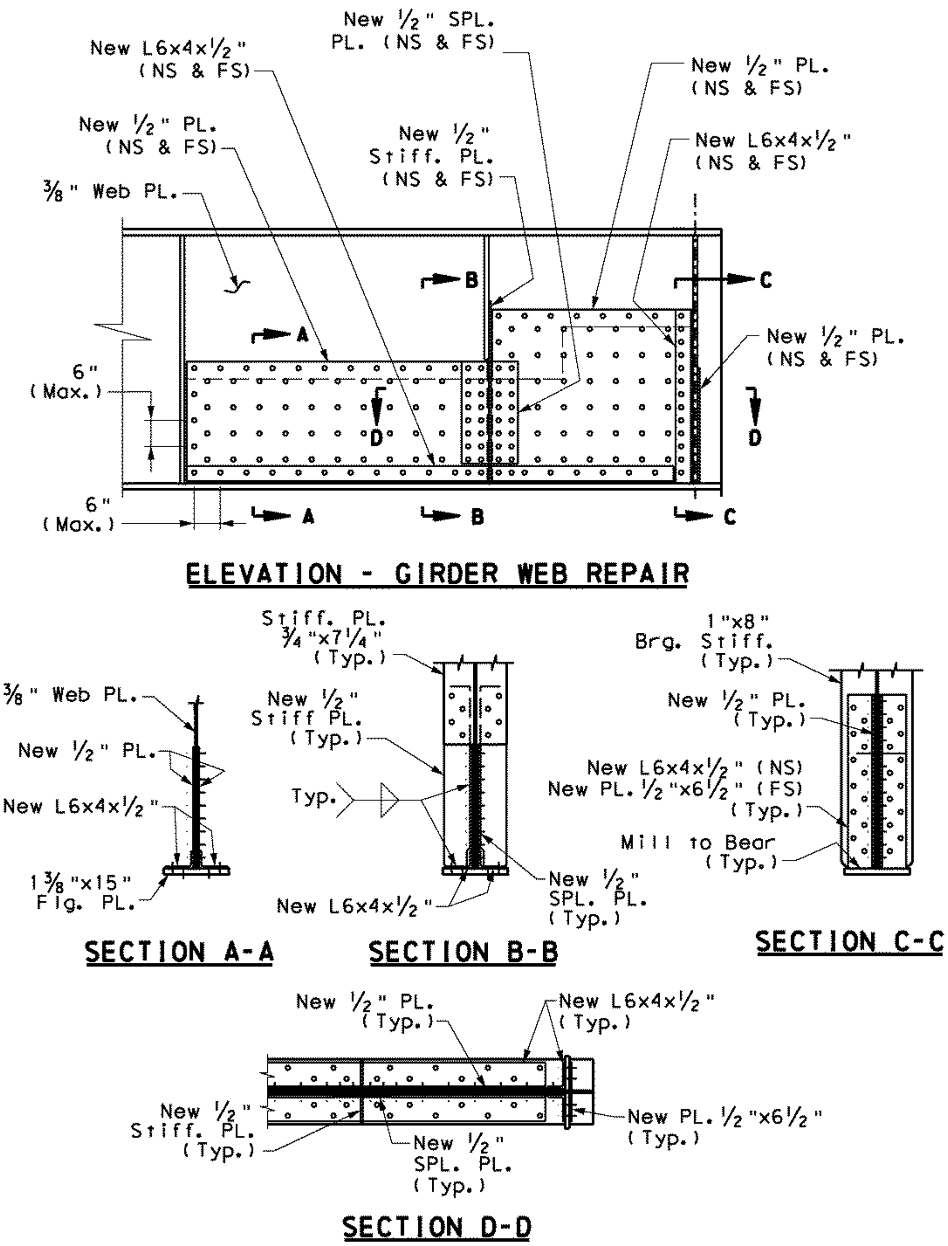
Extend new strengthening plates from 0.25 L to 0.5 L, +/- 3 in.



NEW BOTTOM FLANGE STRENGTHENING PLATES

Design Shear Strengthening:

The basic design concept is to provide new web plates on both sides of the web in the regions of web section loss. The strengthening plate should extend beyond the area of section loss and bolt into full web section, if possible. Also the strengthening plates should connect into the bottom flange due to the horizontal component of the shear forces. The bolted connections should be adequate for all strength requirement plus stitching and sealing requirements.



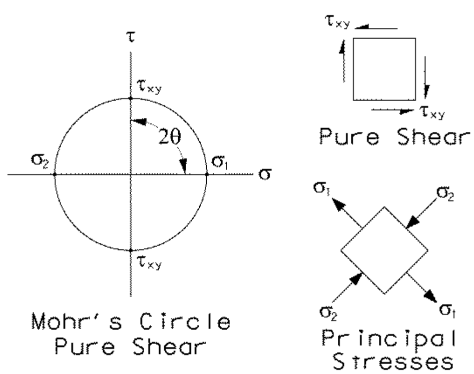
Shear Strengthening:

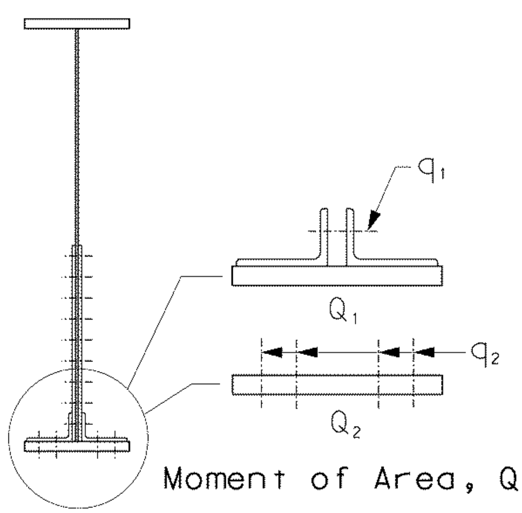
By inspection the new 1/2 in. web plates provide sufficient material area to compensate for the thickness loss in the existing web. However, the bolted connection still need to be checked. The following checks will be made on the shear strengthening:

1. Connection of the strengthening material to the bottom flange.
2. Splice between web plates in adjacent panels.
3. Connection of strengthening material to the bearing stiffener
4. Stitching and Sealing Requirements

Check Connections to the Bottom Flange

To provide proper shear strengthening, the new material needs to be connected to the bottom flange to provide resistance to the horizontal component of the shear. From Mohr's Circle, shown below, pure shear has an equal vertical and horizontal component.





Check Connections to the Bottom Flange

Check shear flow using VQ/I for the New L6x4x1/2 horizontal (q1) and vertical (q2) bolts.

Piece	t (in.)	w (in.)	A (in.^{2})	y (in.)	Ay (in.^{2})	A(y-y')^2 (in.^{4})	Io (in.^{4})
Top Flange	1.375	15	20.625	30.1875	622.62	21,978	3.25
Web	0.375	59	22.125	0.00	0.00	133	6,418
Bott. Flange	1.375	15	20.625	-30.1875	-622.62	4,190	0.12
New Web PLs	1.0	28	28.000	-15.500	-434.00	15,862	3.25
New Ls 6x4	-	-	9.500	-28.5130	-270.87	15,862	3.25
Totals			100.88		-705.87	47,116	8,266

y' = ΣAy / A = -705.87 in.^{3} / 100.88 in.^{2} = -6.988 in. downwards

I_x = Σ Io + Σ Ad^2 = 8,266 in.^{4} + 47,116 in.^{4} = 55,382 in.^{4}

$A_1 = A_{bf} + A_{Ls}$ = 20.625 in^2 + 9.50 in^2 = 30.125 in.^{2}

$y_1 = (A_{bf}\, y_{bf} + A_{Ls}\, y_{Ls}) / A_1$ = ((20.625 in^2)(-30.1875 in) + (9.50 in^2)(-28.5130 in.)) / 30.125 in.^{2}

= -29.659 in^2

$Q_1 = A_1 (y_1 - y')$ = 30.125 in.^{2} (-29.659 in. - -6.988 in.) = 682.989 in.^{3}

$A_2 = A_{bf}$ = 20.625 in^2

$y_2 = y_{bf}$ = -30.1875 in.

$Q_2 = A_2 (y_2 - y')$ = 20.625 in.^{2} (-30.1875 in. - -6.988 in.) = 478.498 in.^{3}

Check Connections to the Bottom Flange (cont.)

Recall:

V_u = 197.68 kip (Strength-I)

R_r = 55.40 kip/bolt (Double Shear) or 27.7 kip/bolt (Single Shear)

$q_1 = V_u Q_1 / I$ = (197.68 kip (689.989 in.^{3})) / 55,382 in.^{4} = 2.438 kip/in.

$r_1 = q_1 p$ = 2.438 kip/in. (6 in./bolt) = 14.63 kip/bolt where p is the max bolt spacing.

r_1 = 14.63 kip/bolt < R_r = 55.40 kip/bolt (Double Shear) OK

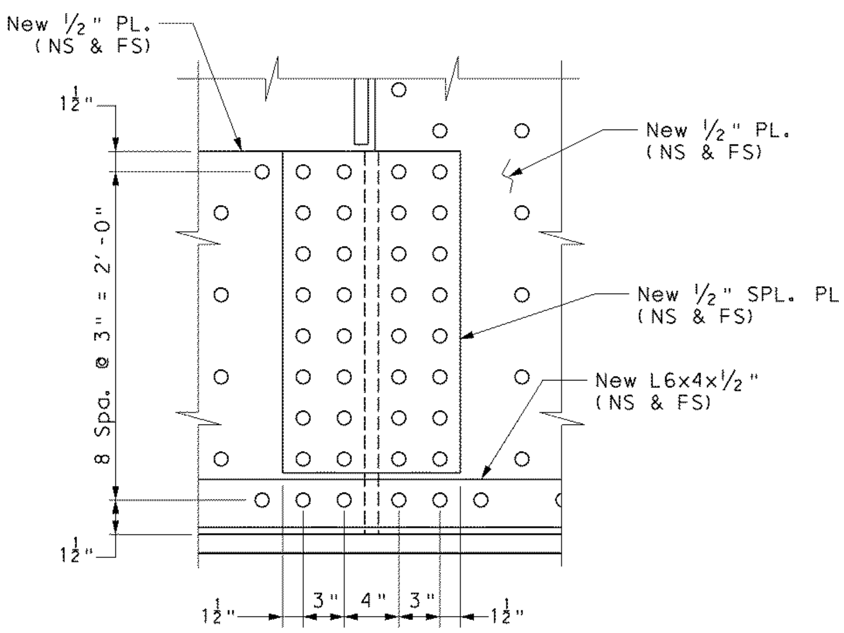
$q_2 = V_u Q_2 / I$ = (197.68 kip (478.498 in.^{3})) / 55,382 in.^{4} = 1.708 kip/in.

$r_2 = q_2 p$ = 1.708 kip/in. (6 in./bolt) = 10.25 kip/bolt

r_2 = 10.25 kip/bolt < R_r = 27.7 kip/bolt (Single Shear) OK

The New L6x4x1/2 connections to the new web plates and to the bottom flange are sufficient.

Check the New Web Plates Splice Connection.



SPLICE DETAIL

Check the New Web Plates Splice Connection.

Design Splice based on capacity of the smaller of the new web plates.

If $V_u < 0.5\ \phi_v V_n$ then: $V_{uv} = 1.5 V_u$, otherwise LRFD Eqn. 6.13.6.1.4b-1

$V_{uv} = 0.5\ \phi_v V_n + 0.5\ V_u$ LRFD Eqn. 6.13.6.1.4b-2

$V_p = 0.58\ F_y\ t\ D\ 2 = 0.58\ (50\ \text{ksi})(0.5\ \text{in.})(28\ \text{in.})\ 2 = 812\ \text{kip}$

$\phi_v V_n = \phi_v\ V_p = 1.0\ (812\ \text{kip}) = 812\ \text{kip}$

$V_u = 197.68\ \text{kip} < 0.5\ \phi_v V_n = 406\ \text{kip}$, then $V_{uv} = 1.5\ V_u = 1.5\ (197.68\ \text{kip}) = 296.52\ \text{kip}$

Check the splice plate bolts. Assume the full design shear is resisted by the splice plates and neglect the contribution of the new L6x4 angles in the splice.

Determine bolt group properties:

e = 4 in./2 +3 in./2 = 3.5 in., centroid of bolt group (left side) to the centerline of splice.

$$d_1 = \sqrt{[(3.5)(3\ \text{in.})]^2 + (1.5\ \text{in.})^2} = 10.607\ \text{in.}$$

$$d_2 = \sqrt{[(2.5)(3\ \text{in.})]^2 + (1.5\ \text{in.})^2} = 7.649\ \text{in.}$$

$$d_3 = \sqrt{[(1.5)(3\ \text{in.})]^2 + (1.5\ \text{in.})^2} = 4.743\ \text{in.}$$

$$d_4 = \sqrt{[(0.5)(3\ \text{in.})]^2 + (1.5\ \text{in.})^2} = 2.121\ \text{in.}$$

$$J_{bg} = 4{d_1}^2 + 4{d_2}^2 + 4{d_3}^2 + 4{d_4}^2 = 4(10.607\ \text{in.})^2 + 4(7.649\ \text{in.})^2 + 4(4.743\ \text{in.})^2 + 4(2.121\ \text{in.})^2$$

$$J_{bg} = 792\ \text{in}^4/\text{in}^2$$

$V_{uv} = 296.52\ \text{kip}$

$M_{uv} = e\ V_{uv} = 296.52\ \text{kip}\ (3.5\ \text{in.}) = 1{,}037.8\ \text{k-in}$

$N_b = 16$ bolts per side of splice

Check the New Web Plates Splice Connection (cont.).

Determine maximum shear force in corner bolt.

V_{vv} = V_{uv} / N_b = 296.52 kip / 16 bolts = 18.53 kip/bolt, direct vertical shear

V_{vm} = M x / J = (1,037.8 k-in (1.5 in.)) / 792 in.^{2} = 1.966 kip/bolt, vertical shear from moment

V_{hm} = M x / J = (1,037.8 k-in (10.5 in.)) / 792 in.^{2} = 13.76 kip/bolt, horizontal shear from moment

$$V_{br} = \sqrt{(V_{vv} + V_{vm})^2 + (V_{hm})^2} = \sqrt{(18.53\ \text{kip} + 1.97\ \text{kip})^2 + (13.76\ \text{kip})^2} = 24.69\ \text{kip}$$

V_{br} = 24.69 kip/bolt < R_r = 55.40 kip/bolt (Double Shear) OK

Check the splice plate capacity:

t = 0.5 in. h = 24 in. A = 12 in.^{2} Sx = 48 in.^{3} V_{uv} = 296.52 kip

$\phi_v V_n$ = 2(1.0) 0.58 A_g F_y = 2(0.58)(12 in.^{2})(50 ksi)2 = 696 kip >= V_{uv} = 296.52 kip OK

$\phi_f M_n$ = 2(1.0) F_y S_x = 2 (50 ksi)(48 in.^{3}) = 4,800 k-in > M_{uv} = 1,037.8 k-in OK

Therefore, the splice design is sufficient

Check the New L6x4 Connection to the Existing Bearing Stiffeners.

Design Connection based on capacity of the larger of the new web plates.

$V_p = 0.58\ F_y\ t\ D\ 2 = 0.58\ (50\ \text{ksi})(0.5\ \text{in.})(40\ \text{in.})\ 2 = 1{,}160\ \text{kip}$

$\phi_v V_n = \phi_v\ V_p = 1.0\ (1{,}160\ \text{kip}) = 1{,}160\ \text{kip}$, new web plate factored shear resistance

$V_u = 197.68$ kip, factored shear for Strength-I Limit State

$0.5V_u + 0.5\ \phi_v V_n = 0.5(197.68\ \text{kip}) + 0.5\ (1{,}160\ \text{kip}) = 678.84\ \text{kip}$

$V_p = 461.97$ kip, Plastic capacity of as-built web plate

Design for the minimum of the average of V_u and $\phi_v V_n$ or V_p of the existing Web LRFD 6.13.1

$V_{uv} = 461.97$ kip

Check 6 in. leg connection.

e = 0.987 in., center of gravity of L6x4 from the back of the angle in the 4 in. direction.

$M_{uv} = e\ V_{uv} = 461.97\ \text{kip}\ (0.987\ \text{in.}) = 455.96\ \text{k-in}$

$\Sigma y_b^2 = 4(16.5\ \text{in.})^2 + 4(13.5\ \text{in.})^2 + 4(10.5\ \text{in.})^2 + 4(7.5\ \text{in.})^2 + 4(4.5\ \text{in.})^2 + 4(1.5\ \text{in.})^2 = 2{,}574\ \text{in.}^2$

$T_b = M_{uv}\ y / \Sigma y_b^2 = ((455.96\ \text{k-in})(16.5\ \text{in.}))/\ 2{,}574\ \text{in.}^2 = 2.92\ \text{kip/bolt}$

$V_b = V_{uv} / N_b = 455.96\ \text{kip} / 24\ \text{bolts} = 19.0\ \text{kip/bolt}$

Check combined shear and tension in controlling bolt. LRFD Eqn. 6.13.2.11-2

$$T_n = 0.76\ A_b\ F_{ub}\sqrt{1-\frac{P_u}{\phi_s R_n}} = 0.76(0.601\ \text{in.}^2)(125\ \text{ksi})\sqrt{1-\frac{19.0\ \text{kip}}{27.7\ \text{kip}}} = 31.58\ \text{kip}$$

$\phi_t T_n = 0.8\ (31.58\ \text{kip}) = 25.26\ \text{kip} > T_b = 2.92\ \text{kip}$ OK

Check 4 in. leg connection.

e = 0.987 in., center of gravity of L6x4 from the back of the angle in the 4 in. direction.

$V_{uv} = 461.97$ kip $\qquad M_{uv} = 455.96$ k-in

$\Sigma y_b^2 = 2(18\ \text{in.})^2 + 2(15\ \text{in.})^2 + 2(12\ \text{in.})^2 + 2(9\ \text{in.})^2 + 2(6\ \text{in.})^2 + 2(3\ \text{in.})^2 = 1{,}638\ \text{in.}^2$

$V_{vv} = V_{uv} / N_b = 461.97\ \text{kip} / 13\ \text{bolts} = 35.54\ \text{kip/bolt}$

$V_{hm} = ((M_{uv})\ y_b) / \Sigma y_b^2 = ((455.96\ \text{k-in})\ 18\ \text{in.})/1{,}638\ \text{in.}^2 = 5.01\ \text{kip/bolt}$

$$V_{br} = \sqrt{V_{vv}^{\ 2} + V_{hm}^{\ 2}} = \sqrt{(35.54\ \text{kip})^2 + (5.01\ \text{kip})^2} = 35.89\ \text{kip} < R_r = 55.4\ \text{kips (Double Shear) OK}$$

Check Stitching and Sealing Requirements.

All plates and angles are 1/2 in. thick, angles have a 2-1/2 in. gage and the plates have a 3 in. gage at staggered locations.

Maximum Bolt Spacing for Sealing: LRFD Eq. 6.13.2.6.2-2

$s_{max} \leq 4.0 + 4.0\ t_{min}\ - (0.75\ g\ ) \leq 7.0$, for staggered line of fasteners adjacent to free edge

6 in. leg of angle:
$s_{max}\ = 4.0 + 4.0\ (0.5\text{ in.}) – 0.75\ (2.5\text{ in.}) = 4.125$ in. pitch > 3 in. provided OK

Free Edge of Plates:
$s_{max}\ = 4.0 + 4.0\ (0.5\text{ in.}) – 0.75\ (3\text{ in.}) = 3.75$ in. pitch > 3 in. provided OK

Maximum Bolt Spacing for Stitching: LRFD Eq. 6.13.2.6.3-1

$s_{max} = 12\ t_{min} = 12\ (0.5\text{ in.}) = 6$ in. pitch >= 6 in max. provided OK

Discussion:

The intent of the shear strengthening design was to provide additional material to compensate for the loss in thick of the web plate due to corrosion section loss. Providing addition strength beyond the As-Built condition was not considered as part of the design. Therefore, the strengthened girder can be considered to have a shear resistance equal to the As-Built condition and the Live Load Ratings can also be considered the same as the As-Built condition. For this to be the case, the following condition must be met:

1. The new web plate material should be at least sufficient to compensate for the section loss.
2. The new web plates should extend beyond the section loss and connect with at least one line of bolts into full web section, if possible.
3. The new web plates should connect to the bottom flange, the bearing stiffener, if applicable, and be continuous through adjacent web panels
4. The strength of the connections should be sufficient and the stitching and sealing requirement should be met.

Summary

There are many factors that must be considered when designing plate girder strengthening. A summary of the major considerations are as follows:

The existing member will carry all of the dead load forces in the member at the time of strengthening. These are considered locked-in stresses. The only forces the new strengthening material will resist are it's proportional share of the live loads and the changes in dead loads after strengthening. These are referred to as the shared stresses. This fact means that the new strengthening will have a limit to its effectiveness, particularly when there are high dead load to live loads stresses in the member. There are few methods of increasing the effectiveness of the strengthening. One method involves jacking out a portion of the existing member dead loads during the strengthening. Another method involves optimizing the timing of the strengthening when the existing dead load forces are minimized. This is most appropriate during re-decking work on the bridge.

When designing the girder flexural strengthening, both the existing member material and the new strengthening material must be checked for all limit states. Typically the existing member material will control due to the locked-in dead load stresses and the fact that the existing material will often be composed of lower strength material than the new strengthening material. However, if there are high live load to dead load stresses, or there are high flexural stresses and the new material is installed on the outer extremes of the member, the new material can easily control over the existing material. This is why both the new and existing material should be checked.

The connections for the new flexural strengthening materials should be designed to develop the full capacity of the material. This will often involve the removal of existing fasteners. The existing member connection capacity needs to be checked for the removal of these fasteners. If the existing connection capacity is insufficient, new higher strength fasteners can be used to replace the existing fasteners, typically one at a time. Another method is to restrict live load during the new material connection work.

Finally, the cost of this type of strengthening is overwhelmingly based on the installation costs and not the cost of the material. It may turn out that a thicker strengthening plate will cost less due to an increase in allowable bolt spacing.

Summary (continued)

The connections for the new shear strengthening materials should be designed to develop the capacity of the new material or the average of the capacity of the new material and the actual factored shear, but does not necessarily need to exceed the existing as-built web capacity, unless addition strength beyond the as-built condition is desired. Developing the full capacity may result in an excess number of bolts. The web plates should be connected to the bottom flange, and vertical stiffeners or be continuous through adjacent web panels. This will often involve the removal of a portion of existing vertical stiffeners. The portion of the existing stiffener can be replaced with a new stiffener attached to the new web plates.

If the new web plates are simply to compensate for section loss and the as-inspected factored shear resistance is not significantly lower than the factored shear force, the locked-in dead load forces in the existing web can likely be neglected in the strengthening design.

Finally, the cost of these types of strengthening is overwhelmingly based on the installation costs and not the cost of the material. It may turn out that a thicker strengthening plate will cost less due to an increase in allowable bolt spacing.

References Page

AISC, 1989, *Manual of Steel Construction – Allowable Stress Design, 9th Edition,* AISC, Chicago, IL.

AASHTO (2014). *AASHTO LRFD Bridge Design Specifications, Customary U.S. Units,* 7th Ed., AASHTO, Washington, D.C.

AASHTO (2003). *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges, First Edition, with 2005 Interim Revisions,* AASHTO, Washington, D.C.

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