Report on Techniques for Bridge Strengthening

Design Example – Steel Truss Member Strengthening

September 2018

U.S. Department of Transportation
Federal Highway Administration

FHWA-HIF-18-042

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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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16. Abstract This design example, steel truss member strengthening, involves the addition of steel cover plates to steel truss members, one-tension member and one-compression member. The bridge is to be strengthened to meet HL-93 design loading (Existing bridge designed for HS-15 live load). This example is based on AASHTO LRFD Bridge Design Specifications, 7th Edition.		
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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)

STEEL TRUSS MEMBER STRENGTHENING
DESIGN EXAMPLE

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	—	No

Summary of Design/Analysis Procedure:

Define the bridge data, material properties, section properties and existing dead load member forces. Identify the standard or specification to 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 follows the general steps below:

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	=	distance between connectors (in.)
A_b	=	gross area of new high strength bolt (in.^{2})
A_{ge}	=	gross area of existing member (in.^{2})
A_{gn}	=	gross area of strengthening material (in.^{2})
A_{gs}	=	gross area of strengthened member (in.^{2})
A_{ne}	=	net area of existing member (in.^{2})
A_{nn}	=	net area of strengthening material (in.^{2})
A_{ns}	=	net area of strengthened member (in.^{2})
A_r	=	gross area of existing rivet (in.^{2})
b	=	distance between the centerline of the webs of a boxed-shape member (in.)
b_f	=	width of the flange of a rolled shape (in.)
b-to-b	=	back to back distance between shapes in a built-up member (in.)
C_m	=	moment gadient modifier
d	=	depth of a rolled shape (in.)
D_b	=	diameter of new high strength bolt (in.)
D_r	=	diameter of existing rivet (in.)
E	=	modulus of elasticity of steel (ksi)
$f_{a\ DCge}$	=	dead load axial stress on gross section of existing member (ksi)
$f_{a\ DCgs}$	=	dead load axial stress on gross section of new strengthening material (ksi)
$f_{a\ DCne}$	=	dead load axial stress on net section of existing member (ksi)
$f_{a\ DCns}$	=	dead load axial stress on net section of new strengthening material (ksi)
$f_{a\ DWge}$	=	wearing surface dead load axial stress on gross section of existing member (ksi)
$f_{a\ DWgs}$	=	wearing surface dead load axial stress on gross section of new material (ksi)
$f_{a\ DWne}$	=	wearing surface dead load axial stress on net section of existing member (ksi)
$f_{a\ DWns}$	=	wearing surface dead load axial stress on net section of new material (ksi)
$f_{a\ LLge}$	=	live load plus impact axial stress on gross section of existing member (ksi)
$f_{a\ LLgs}$	=	live load plus impact axial stress on gross section of new material (ksi)
$f_{a\ LLne}$	=	live load plus impact axial stress on net section of existing member (ksi)
$f_{a\ LLns}$	=	live load plus impact axial stress on net section of new material (ksi)
$f_{au\ ge}$	=	factored axial stress on gross area of existing member (ksi)
$f_{au\ gn}$	=	factored axial stress on gross area of new material (ksi)
$f_{au\ ne}$	=	factored axial stress on net area of existing member (ksi)
$f_{au\ nn}$	=	factored axial stress on net area of new material (ksi)
$f_{b\ DCne}$	=	dead load flexural stress on net section of existing member (ksi)
$f_{b\ DCns}$	=	dead load flexural stress on net section of new material (ksi)

Symbols and Notation

Variables used throughout the design example are listed alphabetically below:

$f_{b\ DWne}$ = wearing surface dead load flexural stress on net section of existing member (ksi)
$f_{b\ DWns}$ = wearing surface dead load flexural stress on net section of new material (ksi)
$f_{b\ LLne}$ = live load plus impact flexural stress on net section of existing member (ksi)
$f_{b\ LLns}$ = live load plus impact flexural stress on net section of new material (ksi)
$f_{bu\ ge}$ = factored flexural stress on gross area of existing member (ksi)
$f_{bu\ gn}$ = factored flexural stress on gross area of new material (ksi)
$f_{bu\ ne}$ = factored flexural stress on net area of existing member (ksi)
$f_{bu\ nn}$ = factored flexural stress on net area of new material (ksi)
$F_{a\ ge}$ = equivalent allowable axial stress on gross section of existing material (ksi)
$F_{a\ gn}$ = equivalent allowable axial stress on gross section of new material (ksi)
$F_{a\ gse}$ = equivalent allowable axial stress for existing material on strengthened member (ksi)
$F_{a\ gsn}$ = equivalent allowable axial stress for new material on strengthened member (ksi)
$F_{a\ ne}$ = equivalent allowable axial stress on net section of existing material (ksi)
$F_{a\ nn}$ = equivalent allowable axial stress on net section of new material (ksi)
$F_{b\ ne}$ = equivalent allowable flexural stress on net section of existing material (ksi)
$F_{b\ nn}$ = equivalent allowable flexural stress on net section of new material (ksi)
$F_{a\ nse}$ = equivalent allowable flexural stress for exist material on strengthened member (ksi)
$F_{a\ nsn}$ = equivalent allowable flexural stress for new material on strengthened member (ksi)
F_{ye} = yield strength of existing steel (ksi)
F_{yn} = yield strength of new steel (ksi)
F_{ue} = tensile strength of existing steel (ksi)
F_{un} = tensile strength of new steel (ksi)
F_{unb} = tensile strength of new high strength bolt (ksi)
F_{uer} = tensile strength of existing rivet (ksi)
ϕF_r = factored strength of existing rivet (ksi)
g = bolt or rivet gage (in.)
G = shear modulus of steel (ksi)
h = depth between the centerlines of the flanges (in.)
IM = live load impact factor
I_{xe} = moment of inertia of existing member about the major principal axis (in.^{4})
I_{xs} = moment of inertia of existing member about the major principal axis (in.^{4})
I_{ye} = moment of inertia of existing member about the minor principal axis (in.^{4})
I_{ys} = moment of inertia of existing member about the minor principal axis (in.^{4})
J = St. Venant torsional constant (in.^{4})
K = effective column length factor
K_h = hole size factor (LRFD Table 6.13.2.8-2)
K_s = coefficient of friction on faying surface

Symbols and Notation

Variables used throughout the design example are listed alphabetically below:

KL/r = slenderness ratio (in.)
$(KL/r)_m$ = modified slenderness ratio of built-up member (in.)
L_c = clear distance between edge of bolt hole and end of member (in.)
L_x = unbraced member length about x-axis (in.)
L_y = unbraced member length about y-axis (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 load (k-in.)
M_u = moment due to factored loads (k-in.)
M_n = nominal moment resistance (k-in.)
M_r = factored moment resistance (k-in.)
M_{CR} = elastic lateral-torsional buckling moment (k-in.)
M_y = yield moment (k-in.)
M_p = plastic moment (k-in.)
N_s = number of shear/slip planes in a connection
P_{DC} = axial tension or compression due to dead load (kip)
P_{DW} = axial tension or compression due to wearing surfaces and utilities (kip)
P_e = elastic critical buckling resistance (kip)
P_{LL+I} = axial tension or compression due to live load load (kip)
P_o = equivalent nominal yield resistance (kip)
P_n = nominal axial tension or compression resistance (kip)
P_r = factored axial tension or compression resistance (kip)
P_t = minimum required pretension in bolt (kip)
P_u = axial tension or compression due to factored loads (kip)
Q = slender element reduction factor
r_i = minimum radius of gyration of an individual component shape (in.)
r_{xe} = radius of gyration of existing member about the x-x major principal axis (in.)
r_{ye} = radius of gyration of existing member about the y-y minor principal axis (in.)
r_{xs} = radius of gyration of strengthened member about the x-x major principal axis (in.)
r_{ys} = radius of gyration of strengthened member about the y-y minor principal axis (in.)
RF = live load rating factor
R_n = nominal bolt/rivet resistance (kip)
R_r = factored bolt/rivet resistance (kip)

Symbols and Notation

Variables used throughout the design example are listed alphabetically below:

s_{min} = minimum allowable bolt spacing (in.)
s_{max} = maximum allowable bolt spacing (in.)
S_{xe} = section modulus of existing member about the x-x major principal axis (in.^{3})
S_{ye} = section modulus of existing about the y-y minor principal axis (in.^{3})
S_{xse} = section modulus of existing material on strengthened member about x-axis (in.^{3})
S_{yse} = section modulus of existing material on strengthened member about y-axis (in.^{3})
S_{xsn} = section modulus of new material on strengthened member about x-axis (in.^{3})
S_{ysn} = section modulus of new material on strengthened member about y-axis (in.^{3})
t_f = thickness of the flange of a rolled shape (in.)
t_{min} = minimum thickness of connected parts (in.)
t_w = thickness of the web of a rolled shape (in.)
U = reduction factor to account for shear lag in connections subject to a tension load
$W_{out\text{-}out}$ = the out to out distance of webs in a builtup member (in.)
Z_{xe} = plastic section modulus of existing member about the major principal axis (in.^{3})
Z_{ye} = plastic section modulus of existing member about the minor principal axis (in.^{3})
Z_{ys} = plastic section modulus of strengthened member about the major principal axis (in.^{3})
δ_b = beam column moment modification factor
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 noncompact flange
ϕ_{bb} = resistance factor for bolt bearing on connected material
ϕ_c = resistance factor for compression
ϕ_f = resistance factor for flexure
ϕ_k = stiffness reduction factor
ϕ_s = resistance factor for shear on bolt or rivet
ϕ_u = resistance factor for fracture on net section of tension member
ϕ_y = resistance factor for yielding on gross section of tension member

Worked Design Example

Introduction: These examples involve the addition of steel cover plates to steel truss members, one-tension member and one-compression member. The bridge is to be strengthened to meet HL-93 design loading (Existing bridge designed for HS-15 live load). This example is be based on AASHTO LRFD Bridge Design Specifications, 7^{th} Edition.

Bridge Data:

Bridge Type:	Steel Deck Truss
Span length:	Two four span continuous trusses and one simple span steel girder approach span at each end equal 2411' -0" between end bearings.
Year Built:	1958
Location:	State of Tennessee

Material Properties:

Existing Steel Yield Strength:	F_{ye} = 33 ksi	(ASTM A7)
Existing Steel Tensile Strength:	F_{ue} = 60 ksi	
New Steel Yield Strength:	F_{yn} = 50 ksi	(ASTM A709, Gr. 50)
New Steel Tensile Strength:	F_{un} = 65 ksi	
Existing Rivet Tensile Strength:	F_{uer} = 52 ksi	(ASTM A141)
New H.S. Bolt Tensile Strength:	F_{unb} = 120 ksi	(ASTM A325)

Existing Member Properties:

	U19-L20 (Tension)	**L20-U21 (Compression)**
Member:	Rolled Wide Flange 12WF72	2-C15x33.9 ($12^1/_4$" b-to-b channels) w/ $^5/_{16}$"x$2^1/_4$" lacing bars (top & bottom)
End Connections:	Riveted Gusset Plates	Riveted Gusset Plates
Unbraced Length:	$L_y = L_x$ = 47'-$4^{13}/_{16}$"	$L_y = L_x$ = 47'-$4^{13}/_{16}$"
Gross Area:	A_{ge} = 21.16 in.^{2}	A_{ge} = 19.80 in.^{2}
Net Area:	A_{ne} = 16.87 in^2	A_{ne} = n/a
Moment of Inertia (y-axis):	I_{ye} = 166.09 in.^{4}	I_{xe} = 553.5 in.^{4}
Section Modulus (y-axis):	S_{ye} = 25.57 in.^{3}	I_{ye} = 579.12 in.^{4}
Plastic Modulus (y-axis):	Z_{ye} = 49.2 in.^{3}	S_{xe} = 79.1 in.^{3}
Radius of gyration (y-axis):	r_{ye}= n/a	r_{ye} = 5.62 in.
Radius of gyration (x-axis):	r_{xe} = n/a	r_{xe} = 5.41 in.
Effective length factor:	K = n/a	K= 0.75 (riveted)
Slenderness ratio:	K L/r = n/a	K L/r = 78.86

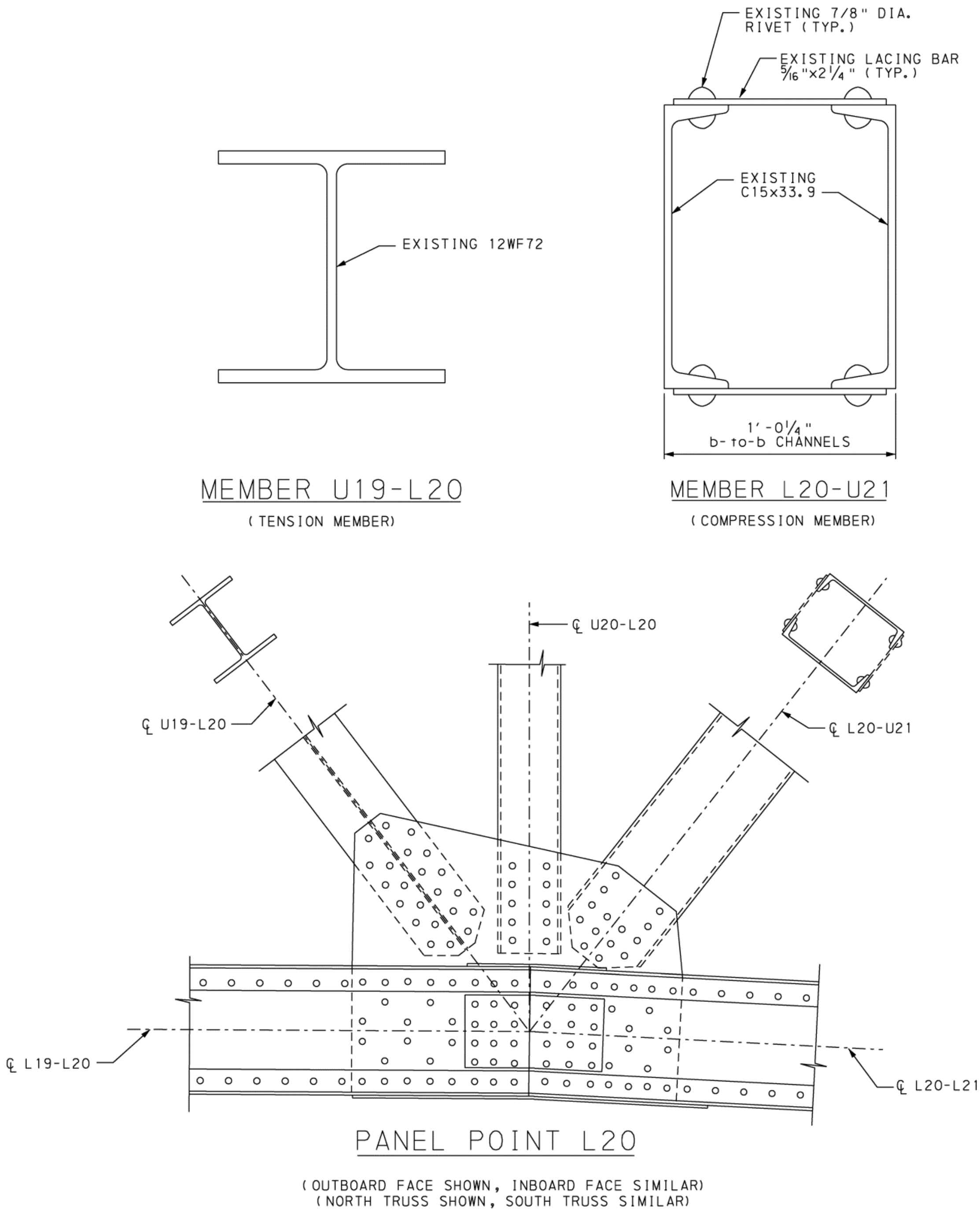


FIGURE 1 - EXISTING TRUSS MEMBERS

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
Service II	1.00	1.00	1.30	1.30
Fatigue	-	-	0.75	0.75

Member Forces:

The following table shows the member forces for both the existing and the new condition. Note that the DC dead load remains the same for both condition, since the truss is being strengthened in the in-service condition. An additional DW is added in the new condition for future wearing surface. The live load is also the same since the existing members are being checked and strengthened for the final HL-93 design live load condition..

Unfactored Member Forces						
Member	**Member Forces**					
	DC		**DW**		**LL+I**	
	P_{DCx}	M_{DCy}	P_{DWx}	M_{DWy}	P_{LLx}	M_{LLy}
	(kip)	(k-ft.)	(kip)	(k-ft.)	(kip)	(k-ft.)
U19-L20 (Existing)	+130	+4	+0	+0	+266	+6
U19-L20 (New)	+130	+4	+10	+1	+266	+6
L20-U21 (Existing)	-147	-11	-0	-0	-133	-4
L20-U21 (New)	-147	+11	-13	+1	-133	-4

+ indicates tension

- indicates compression

due to symmetry, bending moments are all noted as positive (maximum member stress is independent of the sign of the moment)

Calculate Resistance of Existing Member U19-L20 (Tension):

Axial Tension Resistance: LRFD 6.8.2.1

$A_{ge} = 21.16$ in.^{2} $A_{ne} = 16.87$ in.^{2} $F_{ye} = 33$ ksi $F_{ue} = 60$ ksi $\phi_y = 0.95$ $\phi_u = 0.8$

12WF72: $d = 12.25$ in. $t_w = 0.430$ in. $b_f = 12.040$ in. $t_f = 0.671$ in.

Limit State: yielding on gross area: LRFD Eq. 6.8.2.1-1

$P_r = \phi_y F_{ye} A_{ge}$ $P_r = 0.95 \text{ x } 33 \text{ ksi x } 21.16 \text{ in.}^2$ $P_r = 663.4$ kip

Limit State: fracture on net area: LRFD Eq. 6.8.2.1-2

Table 6.8.2.2-1, Case 7 bf = 12.04 in. > 2/3 d = 8.167 in. therefore U = 0.90

$P_r = \phi_u F_{ue} A_{ne} U$ $P_r = 0.80 \cdot 60 \text{ ksi} \cdot 16.87 \text{ in.}^2 \cdot 0.90$ $P_r = 728.8$ kip

$P_r = 663.4$ kip minimum of LRFD Eqs. 6.8.2.1-1 and 6.8.2.1-2

Flexural Resistance (non-composite I-Shape bent about weak axis): LRFD 6.12.2.2.1

from above: $Z_{ye} = 49.2$ in.^{3} $F_{ye} = 33$ ksi $F_{ue} = 60$ ksi $\phi_f = 1.00$

12WF72: $b_f = 12.040$ in. $t_f = 0.671$ in.

$\lambda_f = b_f / 2\, t_f$ $\lambda_f = 12.04 \text{ in.} / (2 \text{ x } 0.671 \text{ in.})$ $\lambda_f = 8.97$ LRFD Eq. 6.12.2.2.1-3

$\lambda_{pf} = 0.38\sqrt{E/F_{ye}}$ $\lambda_{pf} = 0.38\sqrt{29{,}000 \text{ ksi} / 33 \text{ ksi}}$ $\lambda_{pf} = 11.26$ LRFD Eq. 6.12.2.2.1-4

$\lambda_{rf} = 0.83\sqrt{E/F_{ye}}$ $\lambda_{rf} = 0.83\sqrt{29{,}000 \text{ ksi} / 33 \text{ ksi}}$ $\lambda_{rf} = 24.6$ LRFD Eq. 6.12.2.2.1-5

If: $\lambda_f = 8.97 \leq \lambda_{pf} = 11.26$ then: $M_n = M_p$ LRFD Eq. 6.12.2.2.1-1

$\phi_f M_n = \phi_f \cdot Z_{ye} \cdot F_{ye}$ $\phi_f M_n = 1.00 \cdot 49.2 \text{ in.}^3 \cdot 33 \text{ ksi x } (1 \text{ ft.}/ 12 \text{ in.})$ $\phi_f M_n = 135.3$ k-ft.

Calculate Resistance of Existing Member L20-U21 (Compression):

Axial Compression Resistance: LRFD 6.9

$F_{ye} = 33$ ksi $F_{ue} = 60$ ksi $\phi_y = 0.95$ $\phi_c = 0.95$

$A_{ge} = 19.8$ in.^{2} $S_{xe} = 79.1$ in.^{3} $r_{ye} = 5.62$ in. $r_{xe} = 5.41$ in. $K = 0.75$

C15x33.9: $r_i = 0.904$ in. $t_w = 0.40$ in. $b_f = 3.40$ in. $t_f = 0.65$ in.

Check Slenderness of Channel: LRFD 6.9.4.2

From LRFD Table 6.9.4.2.1-1: k = 0.56 (flange of channel) and 1.49 (web of channel)

$$\frac{b}{t} \le k\sqrt{\frac{E}{Fy}}$$ LRFD Eqn. 6.9.4.2.1-1

Check Flange: $\dfrac{3.4\,\text{in.}}{0.65\,\text{in.}} = 5.23 \le 0.56\sqrt{\dfrac{29{,}000\,\text{ksi}}{33\,\text{ksi}}} = 16.6$ OK

Check web: $\dfrac{11.85\,\text{in.}}{0.4\,\text{in.}} = 29.625 \le 1.49\sqrt{\dfrac{29{,}000\,\text{ksi}}{33\,\text{ksi}}} = 44.17$ OK

Channel is nonslender, therefore Q = 1.0

Determine Slenderness Ratio of Built-up Member: LRFD 6.9.4.3

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2}$$ LRFD Eqn. C6.9.4.3.1-1

where a = 12 in. (rivet spacing for lacing bars)

$r_i = 0.904$ in. (min. radius of gyration of single channel)

L = 568.8125 in. (unbraced length of member)

K = 0.75 (riveted member)

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{0.75\cdot 568.8\,\text{in}}{5.41\,\text{in}}\right)_o^2 + \left(\frac{12\,\text{in}}{0.904\,\text{in}}\right)^2} = 79.96$$

Calculate Equivalent Nominal Yield Resistance:

$P_o = Q\,F_{ye}\,A_{ge}$ $P_o = 1.0$ (33 ksi)(19.8 in.^{2}) $P_o = 653.4$ kip

Calculate Resistance of Existing Member L20-U21 (Compression): Continued

Calculate Elastic Critical Buckling Resistance: LRFD Eqn. 6.9.4.1.2-1

$$P_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} Ag \qquad P_e = \frac{\pi^2 29{,}000\,\text{ksi}}{(79.96)^2} 19.8\,\text{in.}^2 = 886.4\,\text{kip}$$

Calculate the Axial Compression Resistance:

If $\frac{P_e}{P_o} \geq 0.44$ then $P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] P_o$ LRFD Eqn. 6.9.4.1.1-1

if $\frac{P_e}{P_o} < 0.44$ then $P_n = 0.877 P_e$ LRFD Eqn. 6.9.4.1.1-2

If $\frac{P_e}{P_o} = \frac{653.4\,\text{kip}}{886.4\,\text{kip}} = 0.737 > 0.44$ then $P_n = \left[0.658^{\left(\frac{653.4}{886.4}\right)}\right] 653.4 = 479.9\,\text{kip}$

Calculate the Flexural Resistance:

LRFD 6.12 – Miscellaneous Flexural Members (6.12.2.2.2 – Box-Shaped Members)

$$M_{CR} = \frac{\pi}{L}\sqrt{EI_x GJ}$$

LRFD Eqn. C6.12.2.2.2-4

$$M_n = F_y S\left[1 - \frac{F_y S}{4M_{CR}}\right]$$

LRFD Eqn. C6.12.2.2.2-4

where:

$F_{ye} = 33$ ksi $E = 29{,}000$ ksi $I_{xe} = 553.5$ in.^{4} $S_{xe} = 79.1$ in.^{3} $L = 568.81$ in.

$G = 0.385$ $E = 11{,}165$ ksi   LRFD Eqn. C6.12.2.2.2-2

LRFD Eqn. C6.12.2.2.2-3 $J = \frac{4A^2}{\sum \frac{b}{t}}$ for a boxed shape member.

Since this member is consists of two channels with lacing bars, we will determine an equivalent box-shape and then determine it’s Polar Moment of Inertia, J.
(see Figure 2).

Calculate Resistance of Existing Member L20-U21 (Compression): Continued

Determine equivalent box-shape:

$$t_{fe} = \frac{2t_f b_f}{W_{out.to.out}} = \frac{2(0.65\,\text{in.})(3.4\,\text{in.})}{12.25\,\text{in.}} = 0.36\,\text{in.}$$ equivalent top flange thickness

$$h = d - t_{fe} = 15\,\text{in.} - 0.36\,\text{in.} = 14.64\,\text{in.}$$ height, center-to-center of flanges

$$b = W_{out.to.out} - t_w = 12.25\,\text{in.} - 0.4\,\text{in.} = 11.85\,\text{in.}$$ width, center-to-center of webs

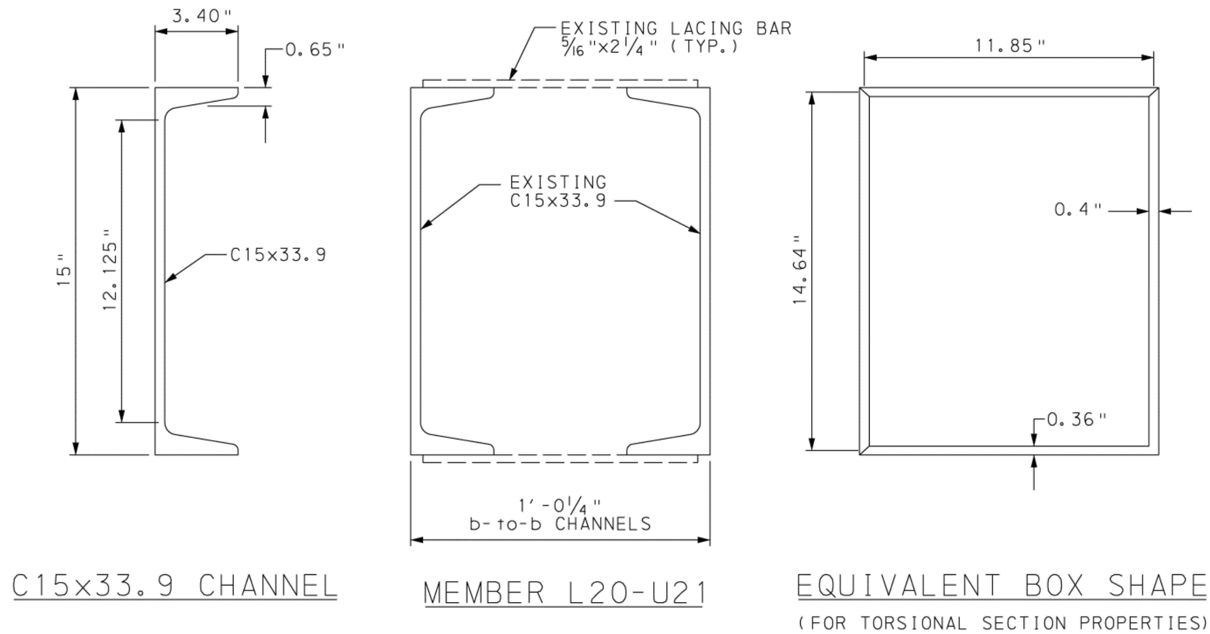


 Figure 2.

from AISC, Design Guide 09- Torsional Analysis of Structural Steel Members, Table 3.1

$$J = \frac{2t_{fe}t_w b^2 h^2}{t_w b + t_{fe} h} = \frac{2(.36\,\text{in.})(0.4\,\text{in.})(11.85\,\text{in.})^2(14.64\,\text{in.})^2}{(11.85\,\text{in.})(0.4\,\text{in.}) + (14.64\,\text{in.})(0.36\,\text{in.})} = 865.9\,\text{in.}^4$$

then:

$$M_{CR} = \frac{\pi}{L}\sqrt{EI_x GJ} = \frac{\pi}{568.8\,\text{in.}}\sqrt{(29{,}000\,\text{ksi})(553.5\,\text{in.}^4)(11{,}165\,\text{ksi})(865.9\,\text{in.}^4)} = 68{,}803\,\text{ksi}$$

$$M_n = F_y S_x\left[1 - \frac{F_y S_x}{4M_{CR}}\right] = (33\,\text{ksi})(79.1\,\text{in.}^3)\left[1 - \frac{(33\,\text{ksi})(79.1\,\text{in.}^3)}{4(68{,}803\,\text{ksi})}\right] = 2{,}586\,\text{k}\cdot\text{in.}$$

$$\phi_f M_n = 1.0(2{,}586\,\text{k}\cdot\text{in.}) = 2{,}586\,\text{k}\cdot\text{in.}$$

Summary of Existing Members Axial and Flexural Resistances and Factored Forces:

Existing Member Resistances and Unfactored Forces								
Member	Resistance		Unfactored Forces					
	Axial	Flexural	Axial			Flexural		
			DC	DW	LL+I	DC	DW	LL+I
	(kip)	(k-in.)	(kip)	(kip)	(kip)	(k-in.)	(k-in.)	(k-in.)
U19-L20	663.4	1,623.6	130	10	266	48	12	72
L20-U21	-479.9	2,723.7	-147	-13	-133	132	12	48

+ indicates tension

- indicates compression

due to symmetry, bending moments are all noted as positive (maximum member stress is independent of the sign of the moment)

Calculate Demand to Capacity (D/C) Ratios

Member U19-L20:

$$\frac{P_u}{P_r} = \frac{1.25(130\,\text{kip}) + 1.5(10\,\text{kip}) + 1.75(266\,\text{kip})}{663.4\,\text{kip}} = \frac{643\,\text{kip}}{663.4\,\text{kip}} = 0.969 \qquad \leq 1.0 \quad \text{OK}$$

$$\frac{M_u}{M_r} = \frac{1.25(48\,\text{k}\cdot\text{in.}) + 1.5(12\,\text{k}\cdot\text{in.}) + 1.75(72\,\text{k}\cdot\text{in.})}{1{,}624\,\text{k}\cdot\text{in.}} = \frac{204\,\text{k}\cdot\text{in.}}{1{,}624\,\text{k}\cdot\text{in.}} = 0.126 \quad \leq 1.0 \quad \text{OK}$$

Member L20-U21:

$$\frac{P_u}{P_r} = \frac{1.25(-147\,\text{kip}) + 1.5(-13\,\text{kip}) + 1.75(-133\,\text{kip})}{-480\,\text{kip}} = \frac{-436\,\text{kip}}{-480\,\text{kip}} = 0.908 \quad \leq 1.0 \quad \text{OK}$$

$$\frac{M_u}{M_r} = \frac{1.25(132\,\text{k}\cdot\text{in.}) + 1.5(12\,\text{k}\cdot\text{in.}) + 1.75(48\,\text{k}\cdot\text{in.})}{2{,}586\,\text{k}\cdot\text{in.}} = \frac{267\,\text{k}\cdot\text{in.}}{2{,}586\,\text{k}\cdot\text{in.}} = 0.103 \leq 1.0 \quad \text{OK}$$

Check Combined Axial and Bending Interations

Member U19-L20 : LRFD 6.8.2.3 - Combined Tension and Flexure

From above: if: $\frac{P_u}{P_r} = 0.969 \quad \geq 0.2,$ then use LRFD Eqn. 6.8.2.3-2

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) = 0.969 + \frac{8}{9}(0.126) = 1.08 \quad >1.0 \quad \text{No Good (8\% overstressed)}$$

Member U19-L20 : LRFD 6.9.2.2 - Combined Axial Compression and Flexure

From above: if: $\frac{P_u}{P_r} = 0.908 \quad \geq 0.2,$ then use LRFD Eqn. 6.9.2.2-2

before the interaction equation (LRFD Eqn. 6.9.2.2-2) is used, the beam column moment magnification factor, δ_b, from LRFD 4.5.3.2.2b, must be calculated.

$$\delta_b = \frac{C_m}{1 - \frac{P_u}{\phi_K P_e}} \qquad \text{LRFD Eqn. 4.5.3.2.2b-3}$$

where:

$\phi_K = 1.0$ (stiffness reduction factor, 1.0 for steel and aluminum and 0.75 for concrete)

$$P_e = \frac{\pi^2 E I_y}{(KL)^2} = \frac{\pi^2 (29{,}000\,\text{ksi})(579.2\,\text{in.}^4)}{[(0.75)(568.8\,\text{in.})]^2} = 910.9\,\text{kip} \qquad \text{LRFD Eqn.4.5.3.2.2b-5}$$

$C_m = 0.6 + 0.4\frac{M_{1b}}{M_{2b}}$ where: M_{1b} is the smaller end moment LRFD Eqn.4.5.3.2.2b-6
M_{2b} is the larger end moment

Since end moments are relatively small, minor differences in the end moment result in large differences in the M_{1b} / M_{2b} ratios. Therefore, conservatively assume $C_m = 1.0$

$$\delta_b = \frac{C_m}{1 - \frac{P_u}{\phi_K P_e}} = \frac{1.0}{1 - \frac{436\,\text{kip}}{1.0(911\,\text{kip})}} = 1.92$$

Check Combined Axial and Bending Interations: (Continued)

$$\frac{P_u}{P_r}+\frac{8.0}{9.0}\left(\delta_b\frac{M_{ux}}{M_{rx}}\right)=0.908+\frac{8}{9}(1.92)(0.103)=1.08$$ >1.0 No Good (8% overstressed)

Calculate Load Rating Factors for the Existing Members:

This is similar to the LRFD interaction checks for combined axial and flexure. Load rating factors are very typical calculation used to evaluate existing bridge members. The Load Rating Factor, RF, is the ratio of the design live load vehicle's effect that the member can safely carry for the give limit state investigated. The AASHTO Guide Manual For Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges, June 2003, is used for the load rating equations.

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

Member U19-L20: Combine LRFD Eqn. 6.8.2.3-2 with LRFR Eqn. 6-1
(see LRFR Appendix C.6.2 for derivation of the following equation)

$$RF=\frac{1-\gamma_{DC}\left[\frac{P_{DC}}{P_r}+\frac{8}{9}\frac{M_{DC}}{M_r}\right]-\gamma_{DW}\left[\frac{P_{DW}}{P_r}+\frac{8}{9}\frac{M_{DW}}{M_r}\right]}{\gamma_L\left[\frac{P_{LL+I}}{P_r}+\frac{8}{9}\frac{M_{LL+I}}{M_r}\right]}$$

$$RF=\frac{1-1.25\left[\frac{130\,kip}{663\,kip}+\frac{8}{9}\frac{48\,k\cdot in.}{1{,}624\,k\cdot in.}\right]-1.5\left[\frac{10\,kip}{663\,kip}+\frac{8}{9}\frac{12\,k\cdot in.}{1{,}624\,k\cdot in.}\right]}{1.75\left[\frac{266\,kip}{663\,kip}+\frac{8}{9}\frac{72\,k\cdot in.}{1{,}624\,k\cdot in.}\right]}=0.894$$ (HL-93 at STR-I)

A RF less than 1.0, indicates the member has insufficient capacity to resist the full factored design live load effect.

Calculate Load Rating Factors for the Existing Members: (Continued)

Member L20-U21: Combine LRFD Eqn. 6.9.2.2-1 with LRFR Eqn. 6-1 (see LRFR Appendix C.6.2 for derivation of the following equation) This is similar to the previous equation except for the addition of the moment magnification factor, δ_b.

$$RF = \frac{1 - \gamma_{DC}\left[\frac{P_{DC}}{P_r} + \frac{8}{9}\delta_b \frac{M_{DC}}{M_r}\right] - \gamma_{DW}\left[\frac{P_{DW}}{P_r} + \frac{8}{9}\delta_b \frac{M_{DW}}{M_r}\right]}{\gamma_L\left[\frac{P_{LL+I}}{P_r} + \frac{8}{9}\delta_b \frac{M_{LL+I}}{M_r}\right]}$$

$$\delta_b = \frac{C_m}{1 - \frac{P_u}{P_e}} = \frac{1.0}{1 - \frac{\gamma_{DC}P_{DC} + \gamma_{DW}P_{DW} + (RF)\gamma_L P_{LL+I}}{P_e}}$$

since δ_b depends on RF, set RF = 1.0 and refine with several iterations. δ_b = 1.92 calculated previously. This is the same as setting RF = 1.0

First Iteration:

$$RF = \frac{1 - 1.25\left[\frac{147\,\text{kip}}{480\,\text{kip}} + \frac{8}{9}(1.92)\frac{132\,\text{k}\cdot\text{in.}}{2{,}586\,\text{k}\cdot\text{in.}}\right] - 1.5\left[\frac{13\,\text{kip}}{480\,\text{kip}} + \frac{8}{9}(1.92)\frac{12\,\text{k}\cdot\text{in.}}{2{,}586\,\text{k}\cdot\text{in.}}\right]}{1.75\left[\frac{133\,\text{kip}}{480\,\text{kip}} + \frac{8}{9}(1.92)\frac{48\,\text{k}\cdot\text{in.}}{2{,}586\,\text{k}\cdot\text{in.}}\right]} = 0.843$$

$$\delta_b = \frac{1.0}{1 - \frac{(1.25)(147\,\text{kips}) + (1.5)(13\,\text{kips}) + 0.843(1.75)(133\,\text{kips})}{911\,\text{kips}}} = 1.781$$

After Four Iterations:

RF = 0.863 (HL-93 at STR-I) and δ_b = 1.797

Both members, U19-U20 and L20-U21, will require strengthening to obtain a Load Rating Factor greater than 1.0 for the HL-93 design live loading, for the Strength-I Limit State.

Design The Member Strengthening:

Assume strengthening for both members will consist of bolted cover plates.

Factors to consider:

- The AASHTO minimum plate thickness is 0.3125” LRFD 6.7.3
- For a long bolted cover plate, the cost of the plate material is 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 most effective method to increase the section properties.
- The strengthening plates will need to be fully developed into the gusset plate connection 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.
- There will be a temporary reduction in the member capacity when removing rivets at the gusset plates, to allow connection of the new plates.

Member U19-L20 (Tension) Strengthening:

Try 5/8” x 12” ASTM A709, Grade 50, cover plates bolted to the outside of the flanges.

(see Figure 3.)

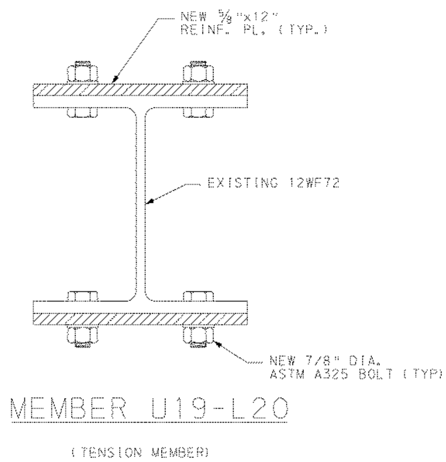


Figure 3

Member U19-L20 (Tension) Strengthening

(continued): Section Properties:

	Existing	Strengthened
Member:	12WF72	12WF72 w/ 2-5/8”x12” Cov. PLs.
End Connections:	Riveted Gusset Plates	Riveted Gusset Plates
Unbraced Length:	$L_y = L_x = 47'\text{-}4^{13}/_{16}''$	$L_y = L_x = 47'\text{-}4^{13}/_{16}''$
Gross Area:	$A_{ge} = 21.16$ in.^{2}	$A_{gs} = 36.16$ in.^{2}
Net Area:	$A_{ne} = 16.87$ in.^{2}	$A_{ns} = 27.87$ in.^{2}
Moment of Inertia (y-axis):	$I_{ye} = 166.09$ in.^{4}	$I_{ys} = 320.45$ in.^{4}
Section Modulus (y-axis):	$S_{ye} = 25.57$ in.^{3}	$S_{ys} = 48.8$ in.^{3}
Plastic Modulus (y-axis):	$Z_{ye} = 49.2$ in.^{3}	$Z_{ys} = 104.7$ in.^{3}

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 work with 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 Member Stresses:

from above:

$P_{DC} = 130$ kip $\quad P_{DW} = 10$ kip $\quad P_{LL+I} = 266$ kip
$M_{DC} = 48$ k-in. $\quad M_{DW} = 12$ k-in. $\quad M_{LL+I} = 72$ k-in.

The existing, DC, composite and noncomposite 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.

locked-in dead load tension stress on the gross section of the existing member:
$f_{a\,DCge} = P_{DC} / A_{ge} = 130 \text{ kip} / 21.16 \text{ in.}^2 = 6.14 \text{ ksi}$

locked-in dead load tension stress on the net section of the existing member:
$f_{a\,DCne} = P_{DC} / A_{ne} = 130 \text{ kip} / 16.87 \text{ in.}^2 = 7.71 \text{ ksi}$

locked-in dead load flexural stress on the net section of the existing member:
$f_{b\,DCne} = M_{DC} / S_{xe} = 48 \text{ k-in} / 25.57 \text{ in.}^3 = 1.88 \text{ ksi}$

Member U19-L20 (Tension) Strengthening (continued):

Shared Stresses in New and Existing Material of Strengthened Member:

Any change in dead load, and the live load plus impact member forces is considered to be shared between the new strengthening material and the existing member. These stresses are determined based on ratios of the new material and existing member section properties with the total strengthened member section properties.

Axial Tension Stresses on Gross Area:

A_{ge} = 21.16 in.^{2} A_{gs} = 36.16 in.^{2} $A_{gn} = A_{gs} - A_{ge}$ = 36.16 in.^{2} – 21.16 in.^{2} = 15 in.^{2}

ratio of area of gross section of existing member to total gross section area of strengthened member:
A_{ge} / A_{gs} = 21.16 in.^{2} / 36.16 in.^{2} = 0.585

ratio of area of gross section of new material to total gross section area of strengthened member:
A_{gn} / A_{gs} = 15 in.^{2} / 36.16 in.^{2} = 0.414

axial stress on gross section of existing member from wearing surface dead loads:
$f_{a\,DWge}$ = (P_{DW} / A_{gs}) 0.585 = (10 kip / 36.16 in.^{2}) 0.585 = 0.16 ksi

axial stress on gross section of new material from wearing surface dead loads:
$f_{a\,DWgn}$ = (P_{DW} / A_{gs}) 0.415 = (10 kip / 36.16 in.^{2}) 0.415 = 0.12 ksi

axial stress on gross section of existing member from live load plus impact:
$f_{a\,LLge}$ = (P_{LL} / A_{gs}) 0.585 = (266 kip / 36.16 in.^{2}) 0.585 = 4.30 ksi

axial stress on gross section of new material from live load plus impact :
$f_{a\,LLgn}$ = (P_{LL} / A_{gs}) 0.415 = (266 kip / 36.16 in.^{2}) 0.415 = 3.05 ksi

Axial Tension Stresses on Net Area:

A_{ne} = 16.87 in.^{2} A_{ns} = 27.87 in.^{2} $A_{nn} = A_{ns} - A_{ne}$ = 27.87 in.^{2} – 16.87 in.^{2} = 11 in.^{2}

ratio of area of net section of existing member to total net section area of strengthened member:
A_{ne} / A_{ns} = 16.87 in.^{2} / 27.87 in.^{2} = 0.605

ratio of area of net section of new material to total net section area of strengthened member:
A_{nn} / A_{ns} = 11 in.^{2} / 27.87 in.^{2} = 0.395

Member U19-L20 (Tension) Strengthening (continued):

Axial Tension Stresses on Net Area (cont.):

axial stress on net section of existing member from wearing surface dead loads:
$f_{a\ DWne} = (P_{DW} / A_{ns})\ 0.605 = (10\text{ kip} / 27.87\text{ in.}^2)\ 0.605 = 0.217\text{ ksi}$

axial stress on net section of new material from wearing surface dead loads:
$f_{a\ DWnn} = (P_{DW} / A_{ns})\ 0.395 = (10\text{ kip} / 27.87\text{ in.}^2)\ 0.395 = 0.142\text{ ksi}$

axial stress on net section of existing member from live load plus impact:
$f_{a\ LLne} = (P_{LL} / A_{ns})\ 0.605 = (266\text{ kip} / 27.87\text{ in.}^2)\ 0.605 = 5.77\text{ ksi}$

axial stress on net section of new material from live load plus impact :
$f_{a\ LLnn} = (P_{LL} / A_{ns})\ 0.395 = (266\text{ kip} / 27.87\text{ in.}^2)\ 0.395 = 3.77\text{ ksi}$

Flexural Stresses:

Flexural stresses are determined for the section modulus of the strengthened member. For the new material the section modulus should be based on the extreme fiber distance, c, to the new material. For the existing member the section modulus should be based on the c distance to the outer fiber of the existing material. The net section moment of inertias is used for this example. Since the bending is about the member y-axis, and the new material is the full flange width, the net section modulus is the same for both the new and the existing material.

$S_{ys} = 48.8\text{ in.}^4$

flexural stress on net section of existing member from wearing surface dead loads:
$f_{b\ DWne} = (M_{DW} / S_{ys}) = (12\text{ k-in.} / 48.8\text{ in.}^4) = 0.246\text{ ksi}$

flexural stress on net section of new material from wearing surface dead loads:
$f_{b\ DWnn} = f_{b\ Dwne} = 0.246\text{ ksi}$

flexural stress on net section of existing member from live load plus impact:
$f_{b\ LLne} = (M_{LL}\ S_{ys}) = (72\text{ k-in.} / 48.8\text{ in.}^4) = 1.48\text{ ksi}$

flexural stress on net section of new material from live load plus impact :
$f_{b\ LLnn} = f_{b\ LLne} = 1.48\text{ ksi}$

Member U19-L20 (Tension) Strengthening (continued):

Determine Equivalent Allowable Stresses from Factored Resistances and Section Properties:

To work with stress values, the member resistances need to be converted into equivalent allowable stress values, based on the member factor resistances and the member section properties.

Tension on Gross Area:

$P_r = \boxed{\boldsymbol{\phi_y F_y}} A_g$ LRFD Eq. 6.8.2.1-1

Allowable Stress of Tension on Gross Area

$F_{a\,ge} = \phi_y F_{ye} = 0.95 \cdot 33 \text{ ksi} = 31.4 \text{ ksi}$ (existing steel)

$F_{a\,gn} = \phi_y F_{yn} = 0.95 \cdot 50 \text{ ksi} = 47.5 \text{ ksi}$ (new steel)

Fracture on Net Area:

$P_r = \boxed{\boldsymbol{\phi_u F_u U}} A_n$ LRFD Eq. 6.8.2.1-2

$F_{a\,ne} = \phi_u F_{ue} U = 0.80 \cdot 60 \text{ ksi} \cdot 0.90 = 43.2 \text{ ksi}$ (existing steel)

$F_{a\,nn} = \phi_u F_{un} U = 0.80 \cdot 65 \text{ ksi} \cdot 0.90 = 46.8 \text{ ksi}$ (new steel)

Flexure:

From previous calculations, it was shown that the slenderness ratio for the flange, λ_f, is below the limiting slenderness ratio for a compact flange, λ_{pf}. This allows the flexural resistance to be computed based on the plastic moment, LRFD Eq. 6.12.2.2.1-1. However, for ease of stress value computations for the new and existing steel, this example uses the conservative flexural resistance based on the yield moment. Note that since the bending is about the member's weak axis, the section modulus for the strengthened section is used for both the new and the existing material

$M_r = \boxed{\boldsymbol{\phi_f F_y}} S_y$

$F_{b\,ne} = \phi_f F_{ye} S_{ys} = 1.0 \cdot 33 \text{ ksi} = 33.0 \text{ ksi}$ (existing steel)

$F_{b\,nn} = \phi_f F_{yn} S_{ys} = 1.0 \cdot 50 \text{ ksi} = 50.0 \text{ ksi}$ (new steel)

Member U19-L20 (Tension) Strengthening (continued):

Determine Factored Stresses for New and Existing Material:

Existing Member:

Axial Tension on Gross Section:

$f_{au\ ge} = 1.25\ (f_{a\ DCge}) + 1.5\ (f_{a\ DWge}) + 1.75\ (f_{a\ LLge})$

$f_{au\ ge} = 1.25\ (6.14\ \text{ksi}) + 1.5\ (0.16\ \text{ksi}) + 1.75\ (4.30\ \text{ksi}) = 15.44\ \text{ksi}$

Axial Tension on Net Section:
$f_{au\ ne} = 1.25\ (f_{a\ DCne}) + 1.5\ (f_{a\ DWne}) + 1.75\ (f_{a\ LLne})$

$f_{au\ ne} = 1.25\ (7.71\ \text{ksi}) + 1.5\ (0.22\ \text{ksi}) + 1.75\ (5.77\ \text{ksi}) = 20.07\ \text{ksi}$

Flexure on Net Section:

$f_{bu\ ne} = 1.25\ (f_{b\ DCne}) + 1.5\ (f_{b\ DWne}) + 1.75\ (f_{b\ LLne})$

$f_{bu\ ne} = 1.25\ (1.88\ \text{ksi}) + 1.5\ (0.25\ \text{ksi}) + 1.75\ (1.48\ \text{ksi}) = 5.32\ \text{ksi}$

New Material:

Axial Tension on Gross Section:

$f_{au\ gn} = 1.25\ (f_{a\ DCgn}) + 1.5\ (f_{b\ DWgn}) + 1.75\ (f_{a\ LLgn})$

$f_{au\ gn} = 1.25\ (0\ \text{ksi}) + 1.5\ (.12\ \text{ksi}) + 1.75\ (3.05\ \text{ksi}) = 5.52\ \text{ksi}$

Axial Tension on Net Section:
$f_{au\ nn} = 1.25\ (f_{a\ DCnn}) + 1.5\ (f_{b\ DWnn}) + 1.75\ (f_{a\ LLnn})$

$f_{au\ nn} = 1.25\ (0\ \text{ksi}) + 1.5\ (0.14\ \text{ksi}) + 1.75\ (3.77\ \text{ksi}) = 6.81\ \text{ksi}$

Flexure on Net Section:

$f_{bu\ nn} = 1.25\ (f_{b\ DCnn}) + 1.5\ (f_{b\ DWnn}) + 1.75\ (f_{b\ LLnn})$

$f_{bu\ nn} = 1.25\ (0\ \text{ksi}) + 1.5\ (0.25\ \text{ksi}) + 1.75\ (1.48\ \text{ksi}) = 2.97\ \text{ksi}$

Member U19-L20 (Tension) Strengthening (continued):

Calculate Demand to Capacity (D/C) Ratios:

Existing Material:

$$\frac{f_{au \cdot ge}}{F_{a \cdot ge}} = \frac{15.44\,\text{ksi}}{31.4\,\text{ksi}} = 0.492 \quad \leq 1.0 \quad \text{OK} \qquad \text{axial tension on gross area (controls over net area)}$$

$$\frac{f_{au \cdot ne}}{F_{a \cdot ne}} = \frac{20.07\,\text{ksi}}{43.2\,\text{ksi}} = 0.465 \quad \leq 1.0 \quad \text{OK} \qquad \text{axial tension on net area}$$

$$\frac{f_{bu \cdot ne}}{F_{b \cdot ne}} = \frac{5.32\,\text{ksi}}{33\,\text{ksi}} = 0.161 \quad \leq 1.0 \quad \text{OK} \qquad \text{flexure on net section}$$

From above: if: $\frac{f_{au \cdot ge}}{F_{a \cdot ge}} = 0.492 \geq 0.2$, then use LRFD Eqn. 6.8.2.3-2

$$\frac{f_{au \cdot ge}}{F_{a \cdot ge}} + \frac{8.0}{9.0}\left(\frac{f_{bu \cdot ne}}{F_{b \cdot ne}}\right) = 0.492 + \frac{8}{9}(0.161) = 0.635 \leq 1.0 \quad \text{OK (Existing Member not Overstressed)}$$

New Material:

$$\frac{f_{au \cdot gn}}{F_{a \cdot gn}} = \frac{5.52\,\text{ksi}}{47.5\,\text{ksi}} = 0.116 \quad \leq 1.0 \quad \text{OK} \qquad \text{axial tension on gross area (controls over net area)}$$

$$\frac{f_{au \cdot nn}}{F_{a \cdot nn}} = \frac{6.81\,\text{ksi}}{46.8\,\text{ksi}} = 0.146 \quad \leq 1.0 \quad \text{OK} \qquad \text{axial tension on net area (controls over gross area)}$$

$$\frac{f_{bu \cdot nn}}{F_{b \cdot nn}} = \frac{2.97\,\text{ksi}}{50\,\text{ksi}} = 0.059 \quad \leq 1.0 \quad \text{OK} \qquad \text{flexure on net section}$$

From above: if: $\frac{f_{au \cdot gn}}{F_{a \cdot gn}} = 0.116 < 0.2$, then use LRFD Eqn. 6.8.2.3-1

$$\frac{f_{au \cdot nn}}{2 \cdot F_{a \cdot nn}} + \left(\frac{f_{bu \cdot nn}}{F_{b \cdot nn}}\right) = 0.073 + (0.059) = 0.132 \quad \leq 1.0 \quad \text{OK (New Material not Overstressed)}$$

By Inspection, the controlling live load rating is controlled by stresses in the existing member. This is typically the case when there are significant locked-in dead load stresses in the existing member.

Member U19-L20 (Tension) Strengthening (continued):

Calculate the Controlling Live Load Rating of the Strengthened Member:

$$RF = \frac{1 - \gamma_{DC}\left[\frac{f_{a \cdot DCge}}{F_{a \cdot ge}} + \frac{8}{9}\frac{f_{b \cdot DCne}}{F_{b \cdot ne}}\right] - \gamma_{DW}\left[\frac{f_{a \cdot DWge}}{F_{a \cdot ge}} + \frac{8}{9}\frac{f_{b \cdot DWne}}{F_{b \cdot ne}}\right]}{\gamma_{LL}\left[\frac{f_{a \cdot LLge}}{F_{a \cdot ge}} + \frac{8}{9}\frac{f_{b \cdot LLne}}{F_{b \cdot ne}}\right]}$$

$$RF = \frac{1 - 1.25\left[\frac{6.14}{31.4\,ksi} + \frac{8}{9}\frac{1.88\,ksi}{33\,ksi}\right] - 1.5\left[\frac{0.16\,ksi}{31.4\,ksi} + \frac{8}{9}\frac{0.25\,ksi}{33\,ksi}\right]}{1.75\left[\frac{4.3\,ksi}{31.4\,ksi} + \frac{8}{9}\frac{1.48\,ksi}{33\,ksi}\right]} = 2.18 \qquad \text{(HL-93 at STR-I)}$$

Therefore, strengthening of tension Member U19-L20 is sufficient.

Member L20-U21 (Compression) Strengthening:

Try 1/2" x 6" ASTM A709, Grade 50, cover plates bolted to the outside of the webs.

(see Figure 4.)

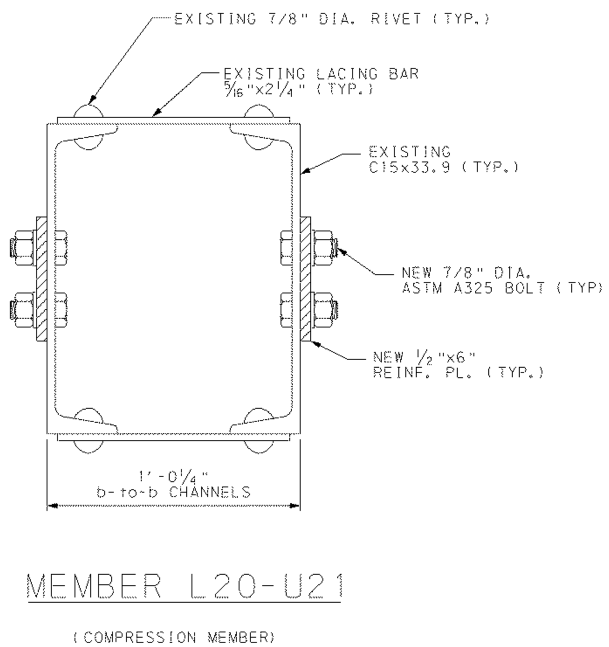


Figure 4

Member L20-U21 (Compression) Strengthening (continued):

Section Properties:

	Existing	Strengthened
	2-C15x33.9	Existing Section
	($12^{1}/_{4}$" b-to-b channels)	with ½"x6" Web PLs
End Connections:	Riveted Gusset Plates	Riveted Gusset Plates
Unbraced Length:	$L_y = L_x = 47'\text{-}4^{13}/_{16}"$	$L_y = L_x = 47'\text{-}4^{13}/_{16}"$
Gross Area:	A_{ge} = 19.80 in.^{2}	A_{gs} = 25.80 in.^{2}
Moment of Inertia (x-axis):	I_{xe} = 553.5 in.^{4}	I_{xs} = 565.76 in.^{4}
Moment of Inertia (y-axis):	I_{ye} = 579.1 in.^{4}	I_{ys} = 823.16 in.^{4}
Section Modulus (x-axis):	S_{xe} = 79.1 in.^{3}	S_{xs} = 81.17 in.^{3}
Radius of gyration (y-axis):	r_{ye} = 5.62 in.	r_{ys} = 5.65 in.
Radius of gyration (x-axis):	r_{xe} = 5.41 in.	r_{xs} = 4.99 in.
Effective length factor:	K = 0.75 (riveted)	K= 0.75 (riveted)
Slenderness ratio:	K L/r = 78.86	K L/r = 85.45

Determine the Member Stresses:

from above:

P_{DC} = 147 kip P_{DW} = 13 kip P_{LL+I} = 133 kip
M_{DC} = 132 k-in. M_{DW} = 12 k-in. M_{LL+I} = 48 k-in.

The existing, DC, composite and noncomposite 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.

locked-in dead load tension stress on the gross section of the existing member:
$f_{a\,DCge} = P_{DC} / A_{ge}$ = 147 kip / 19.80 in.^{2} = 7.42 ksi

locked-in dead load flexural stress on the net section of the existing member:
$f_{b\,DCne} = M_{DC} / S_{xe}$ = 132 k-in / 79.1 in.^{3} = 1.67 ksi

Member L20-U21 (Compression) Strengthening (continued):

Shared Stresses in New and Existing Material of Strengthened Member:

Axial Compression Stresses on Gross Area:

A_{ge} = 19.80 in.^{2} A_{gs} = 25.80 in.^{2} $A_{gn} = A_{gs} - A_{ge}$ = 25.80 in.^{2} – 19.80 in.^{2} = 6.00 in.^{2}

ratio of area of gross section of existing member to total gross section area of strengthened member:
A_{ge} / A_{gs} = 19.80 in.^{2} / 25.80 in.^{2} = 0.767

ratio of area of gross section of new material to total gross section area of strengthened member:
A_{gn} / A_{gs} = 6.00 in.^{2} / 25.80 in.^{2} = 0.233

axial stress on gross section of existing member from wearing surface dead loads:
$f_{a\ DWge}$ = (P_{DW} / A_{gs}) 0.767 = (13 kip / 25.80 in.^{2}) 0.767 = 0.39 ksi

axial stress on gross section of new material from wearing surface dead loads:
$f_{a\ DWgn}$ = (P_{DW} / A_{gs}) 0.233 = (13 kip / 25.80 in.^{2}) 0.233 = 0.12 ksi

axial stress on gross section of existing member from live load plus impact:
$f_{a\ LLge}$ = (P_{LL} / A_{gs}) 0.767 = (133 kip / 25.80 in.^{2}) 0.767 = 3.96 ksi

axial stress on gross section of new material from live load plus impact :
$f_{a\ LLgn}$ = (P_{LL} / A_{gs}) 0.233 = (133 kip / 25.80 in.^{2}) 0.233 = 1.20 ksi

Flexural Stresses:

S_{xse} = 81.17 in.^{3} (top flange of channel) S_{xsn} = 188.59 in.^{3} (top of new plate)

flexural stress on net section of existing member from wearing surface dead loads:
$f_{b\ DWne}$ = (M_{DW} / S_{xse}) = (12 k-in. / 81.17 in.^{3}) = 0.148 ksi

flexural stress on net section of new material from wearing surface dead loads:
$f_{b\ DWnn}$ = (M_{DW} / S_{xsn}) = (12 k-in. / 188.59 in.^{3}) = 0.064 ksi

flexural stress on net section of existing member from live load plus impact:
$f_{b\ LLne}$ = ($M_{LL}\ S_{xse}$) = (48 k-in. / 81.17 in.^{3}) = 0.591 ksi

flexural stress on net section of new material from live load plus impact :
$f_{b\ DWnn}$ = (M_{LL} / S_{xsn}) = (48 k-in. / 188.59 in.^{3}) = 0.255 ksi

Member L20-U21 (Compression) Strengthening (continued):

Determine Equivalent Allowable Stresses from Factored Resistances and Section Properties:

Allowable Axial Compression Stress for Existing Material for Locked-in DC Loading:

$F_{a\,ge} = \phi_c P_n/A_{ge} = 0.95\ (479.9\ \text{kip} / 19.8\ \text{in}^2) = 24.24\ \text{ksi}$ (existing member)

Allowable Axial Compression Stress for Existing Material for Shared DW and LL Loading:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{0.75 \cdot 568.8\,\text{in}}{4.99\,\text{in}}\right)_o^2 + \left(\frac{12\,\text{in}}{0.904\,\text{in}}\right)^2} = 86.48 \quad \text{LRFD Eqn. C6.9.4.3.1-1}$$

$Q = 1.0$

$P_o = Q\,F_{ye}\,A_{ge} \qquad P_o = 1.0\ (33\ \text{ksi})(25.8\ \text{in.}^2) \qquad P_o = 851.4\ \text{kip}$

$$P_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} A_g \qquad P_e = \frac{\pi^2\, 29{,}000\,\text{ksi}}{(86.48\,\text{in.})^2} 25.8\,\text{in.}^2 = 987.4\,\text{kip}$$

$$\frac{P_e}{P_o} = \frac{987.4\,\text{kip}}{851.4\,\text{kip}} = 1.16 > 0.44 \quad \text{then } P_n = \left[0.658^{\left(\frac{987.4\ \text{kip}}{851.4\ \text{kip}}\right)}\right] 851.4\,\text{kip} = 523.9\,\text{kip}$$

$F_{a\,gse} = \phi_c P_n/A_{gs} = 0.95\ (523.9\ \text{kip} / 25.8\ \text{in}^2) = 19.29\ \text{ksi}$

Allowable Flexural Stress for Existing Material for DC, DW and LL Loading:

Since the new strengthening plates are installed, centered on the channel webs, they provide very little contribution to the flexural resistance. Therefore, conservatively use the previously calculated moment resistance for the existing member without strengthening.

$\phi_f M_n = 1.0(2{,}586\ \text{k} \cdot \text{in.}) = 2{,}586\ \text{k} \cdot \text{in.}$

$F_{b\,ne} = \phi_f M_n/S_{xe} = (2{,}586\ \text{kip-in.} / 79.10\ \text{in}^3) = 32.69\ \text{ksi}$

Member L20-U21 (Compression) Strengthening (continued):

Calculate Demand to Capacity (D/C) Ratios:

Existing Material:

axial compression on gross area, $\frac{f_{au \cdot ge}}{F_{a \cdot ge}} =$

$$\frac{1.25 f_{a \cdot DCge}}{F_{a \cdot ge}} + \frac{1.5 f_{a \cdot DWge} + 1.75 f_{a \cdot LLge}}{F_{a \cdot gse}} = 1.25 \frac{7.42\,\text{ksi}}{24.24\,\text{ksi}} + \frac{1.5(0.39\,\text{ksi}) + 1.75(3.96\,\text{ksi})}{19.29\,\text{ksi}} = 0.77 \leq 1.0 \text{ OK}$$

flexure on net section, $\frac{f_{bu \cdot ne}}{F_{b \cdot ne}} =$

$$\frac{1.25 f_{b \cdot DCge}}{F_{b \cdot ge}} + \frac{1.5 f_{b \cdot DWge} + 1.75 f_{b \cdot LLge}}{F_{b \cdot nse}} = 1.25 \frac{1.67\,\text{ksi}}{32.69\,\text{ksi}} + \frac{1.5(0.15\,\text{ksi}) + 1.75(0.59\,\text{ksi})}{32.69\,\text{ksi}} = 0.10 \quad \leq 1.0 \text{ OK}$$

New Material:

axial compression on gross area:

$$\frac{1.25 f_{a \cdot DCgn}}{F_{a \cdot gsn}} + \frac{1.5 f_{a \cdot DWgn} + 1.75 f_{a \cdot LLgn}}{F_{a \cdot gsn}} = 1.25 \frac{0.0\,\text{ksi}}{19.29\,\text{ksi}} + \frac{1.5(0.06\,\text{ksi}) + 1.75(1.20\,\text{ksi})}{19.29\,\text{ksi}} = 0.12 \quad \leq 1.0 \text{ OK}$$

flexure on net section:

$$\frac{1.25 f_{b \cdot DCnn}}{F_{b \cdot nsn}} + \frac{1.5 f_{b \cdot DWnn} + 1.75 f_{b \cdot LLnn}}{F_{b \cdot nsn}} = 1.25 \frac{0.0\,\text{ksi}}{32.69\,\text{ksi}} + \frac{1.5(0.06\,\text{ksi}) + 1.75(0.25\,\text{ksi})}{32.69\,\text{ksi}} = 0.02 \quad \leq 1.0 \text{ OK}$$

By Inspection, the controlling live load rating will be controlled by the existing member.

From above: if: $\frac{f_{au \cdot ge}}{F_{a \cdot ge}} = 0.77 \quad \geq 0.2,$ then use LRFD Eqn. 6.8.2.3-2

$$\frac{f_{au \cdot ge}}{F_{a \cdot ge}} + \frac{8.0}{9.0}\left(\frac{f_{bu \cdot ne}}{F_{b \cdot ne}}\right) = 0.77 + \frac{8}{9}(0.10) = 0.859 \quad \leq 1.0 \quad \text{OK (Existing Member not Overstressed)}$$

Member L20-U21 (Compression) Strengthening (continued):

Calculate the Controlling Live Load Rating of the Strengthened Member:

$$RF = \frac{1-\gamma_{DC}\left[\frac{f_{a\cdot DCge}}{F_{a\cdot ge}}+\frac{8}{9}\delta_b\frac{f_{b\cdot DCne}}{F_{b\cdot ne}}\right]-\gamma_{DW}\left[\frac{f_{a\cdot DWge}}{F_{a\cdot ge}}+\frac{8}{9}\delta_b\frac{f_{b\cdot DWne}}{F_{b\cdot ne}}\right]}{\gamma_{LL}\left[\frac{f_{a\cdot LLge}}{F_{a\cdot ge}}+\frac{8}{9}\delta_b\frac{f_{b\cdot LLne}}{F_{b\cdot ne}}\right]}$$

$$\delta_b = \frac{C_m}{1-\frac{P_u}{P_e}} = \frac{1.0}{1-\frac{\gamma_{DC}P_{DC}+\gamma_{DW}P_{DW}+(RF)\gamma_L P_{LL+I}}{P_e}}$$

First Iteration:

Set $\delta_b = 1.92$ use P_e for strengthened condition.

$$RF = \frac{1-1.25\left[\frac{7.42\,\text{ksi}}{24.24\,\text{ksi}}+\frac{8}{9}(1.92)\frac{1.67\,\text{ksi}}{32.69\,\text{ksi}}\right]-1.5\left[\frac{0.39\,\text{ksi}}{19.29\,\text{ksi}}+\frac{8}{9}(1.92)\frac{0.15\,\text{ksi}}{32.69\,\text{ksi}}\right]}{1.75\left[\frac{3.96\,\text{ksi}}{19.29\,\text{ksi}}+\frac{8}{9}(1.92)\frac{0.59\,\text{ksi}}{32.69\,\text{ksi}}\right]} = 1.13$$

$$\delta_b = \frac{C_m}{1-\frac{P_u}{P_e}} = \frac{1.0}{1-\frac{1.25(147\,\text{kip})+1.5(13\,\text{kip})+(1.13)(1.75)(133\,\text{kip})}{987.4\,\text{kip}}} = 1.89$$

After Four Iterations:

RF = 1.13 (HL-93 at STR-I) and $\delta_b = 1.90$

Therefore, strengthening of compression Member L20-U21 is sufficient.

The next step is to design and check the connection of the strengthening material into the gusset plates.

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 =	1.25 in.	minimum clear distance from bolt hole to edge of connected material
t_b =	0.4 in.	minimum thickness of connected material (L20-U21 channel web)
N_s =	1 plane	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)(1 shear plane) = 34.62 kip/bolt
$R_r = \phi_s\ R_n =$ 0.80 (34.62 kip/bolt) = 27.69 kip/bolt

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

$R_n = 1.2\ L_c\ t\ F_{une} =$ 1.2 (1.25 in.)(0.4 in.)(60 ksi) = 36.0 kip/bolt
$R_r = \phi_s\ R_n =$ 0.80 (36.0 kip/bolt) = 28.8 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 (0.33)(1 shear plane)(39 kip) = 19.5 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 Db = 3.0 (0.875 in.) = 2.625 in. , use 3.0 in. where possible LRFD 6.13.2.6

Maximum Bolt Spacing for Sealing:

Member U19-L20: t_{min} = 0.625 in. (thickness of new strengthening plates)
g = 2.5 in. (gage of existing rivets)

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

s_{max} = 4.0 + 4.0 (0.625 in.) – 0.75 (2.5 in.) = 4.625 in. pitch LRFD Eq. 6.13.2.6.2-2

Member L20-U21: t_{min} = 0.40 in. (thickness of existing channel web)

$s_{max} \leq 4.0 + 4.0\ t_{min} \leq 7.0$, for single line of fasteners adjacent to free edge

s_{max} = 4.0 + 4.0 (0.625 in.) = 5.6 in., say 5.625 in. LRFD Eq. 6.13.2.6.2-1

Maximum Bolt Spacing for Stiching:

Member U19-L20 (Tension): LRFD Eq. 6.13.2.6.3-1

$s_{max} \leq 15\ t_{min}$ - (0.375 g) ≤ 12 t_{min}, 12 t_{min} Controls, 12 (0.625 in.) = 7.5 in. pitch

Member L20-U21 Compression:

s_{max} = 12 t_{min} = 12 (0.40 in.) = 4.8 in., say 4.75 in. (typical) LRFD 6.13.2.6.3

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

Design Connections of Strengthening Material (continued):

Calculate the Factored Resistance of Existing Rivets:

Existing ASTM A141 Rivets:

D_r =	0.875 in.	diameter of bolt
A_r =	0.601 in.^{2}	cross-sectional area of bolt
ϕF_r =	21 ksi	factored shear strength of rivet
N_s =	1 plane	number of shear planes in connection
ϕ_s =	0.80	resistance factor for rivets in shear
ϕ_{bb} =	0.80	resistance factor for bolts bearing on connected material

From: Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges

Shear Resistance of Rivets: LRFR Eq. 6.5 and Table 6-14

$\phi_s R_n = \phi F_r N_s A_r =$ (21 ksi)(1 shear plane)(0.601 in.^{2}) = 12.62 kip/rivet

Determine the Number of Bolts Required to Develop the Strength of the New Strengthening Material:

Member L20-U21 with New ½ in. x 6 in. Strengthening Plates (Compression):

A_{gn} = (0.5 in.)(6 in.) = 3 in.^{2}

$P_r = \phi_c A_{gn} F_{yn} =$ 0.9 (3 in.^{2})(50 ksi) = 135 kip (nominal yield resistance)

Strength I :
Number of Bolts Required = P_r / R_r = (135 kip) / (27.69 kip/bolt) = 4.88, say 6 bolts

Service II:
P_u = 1.00 P_{DC} + 1.00 P_{DW} + 1.3 P_{LL}, where P_{DC} = 0 since existing member carries all of P_{DC}.

P_u = 1.00 $(f_{a\ DWgn})(A_{gn})$ +1.3 $(f_{a\ LLgn})(A_{gn})$ = [1.0(0.12 ksi)+1.3(1.2 ksi)] 3 in.^{2} = 5.04 kips

Number of Bolts Required = P_u / R_r = (5.04 kip) / (19.5 kip/bolt) = 0.25, say 1 bolt

Therefore, STR-I controls. Use six bolts.

Design Connections of Strengthening Material (continued):

Member U19-L20 with New 5/8 in. x 12 in. Strengthening Plates (Tension):

A_{gn} = (0.625 in.)(12 in.) = 7.5 in.^{2} (gross area)

W_{nn} = 12 in. – 1 in./hole (4 holes) + 2 staggers (2 in.^{2}/ 4· 2.5 in.) = 8.8 in. (net width)

A_{nn} = (0.625 in.)(8.8 in.) = 5.5 in.^{2} (net area)

Limit State: yielding on gross area: LRFD Eq. 6.8.2.1-1

$P_r = \phi_y F_{yn} A_{gn}$ P_r = 0.95 x 50 ksi x 7.5 in.^{2} P_r = 356.25 kip

Limit State: fracture on net area: LRFD Eq. 6.8.2.1-2

$P_r = \phi_u F_{un} A_{nn} U$ P_r = 0.80 · 65 ksi · 5.5 in.^{2} · 1.0 P_r = 286.0 kip

P_r = 286.0 kip minimum of LRFD Eqs. 6.8.2.1-1 and 6.8.2.1-2

Strength I :
Number of Bolts Required = P_r / R_r = (286 kip) / (27.69 kip/bolt) = 10.3, say 10 Bolts

Service II:
P_u = 1.00 ($f_{a\ DWgn}$)(A_{gn}) +1.3 ($_{fa\ LLgn}$)(A_{gn}) = [1.0(0.12 ksi)+1.3(3.05 ksi)] 7.5 in.^{2} = 30.64 kips

Number of Bolts Required = P_u / R_r = (30.64 kip) / (19.5 kip/bolt) = 1.6, say 2 bolts

Therefore, STR-I controls. Use ten bolts.

Check the Existing Member Connections:

Member U19-L20:

10 existing rivets out of 22 rivets/ per sided will need to be removed to connect the new strengthening plates. Determine the live load resistance the member has during this rivet removal.

R_r = (2 sides)(12 rivets/side)(12.62 kip/rivet) = 302 kips

$$\frac{P_u}{R_r} = \frac{1.25(130\,\text{kip}) + 1.75(266\,\text{kip})}{302\,\text{kip}} = \frac{628\,\text{kip}}{302\,\text{kip}} = 2.08 > 1.0\,\text{NG}$$

Design Connections of Strengthening Material (continued):

By removing 20 existing rivets, the connection resistance is less than one half of the factored load. Check connection if the existing rivets are replaced with new high strength bolts, one at a time.

R_r = (2 sides)(12 rivets/side)(27.69 kip/bolt) = 665 kips > P_u OK

Member L20-U21:

Check existing riveted connection first.

R_r = (2 sides)(12 rivets/side)(12.62 kip/rivet) = 302 kip < P_u = 436 kip

The existing connection is not sufficient to resist the factored loading.

Similar to member U19-L20, consider replacing the existing rivets in the connection with new high strength bolts, one at a time. The new strengthening plates will also have four additional bolts per plates.

R_r = (2 sides)(16 bolts/side)(27.69 kip/bolt) = 886 kip > P_u = 436 kip

R_r = 886 kip > P_o = 851.4 kips

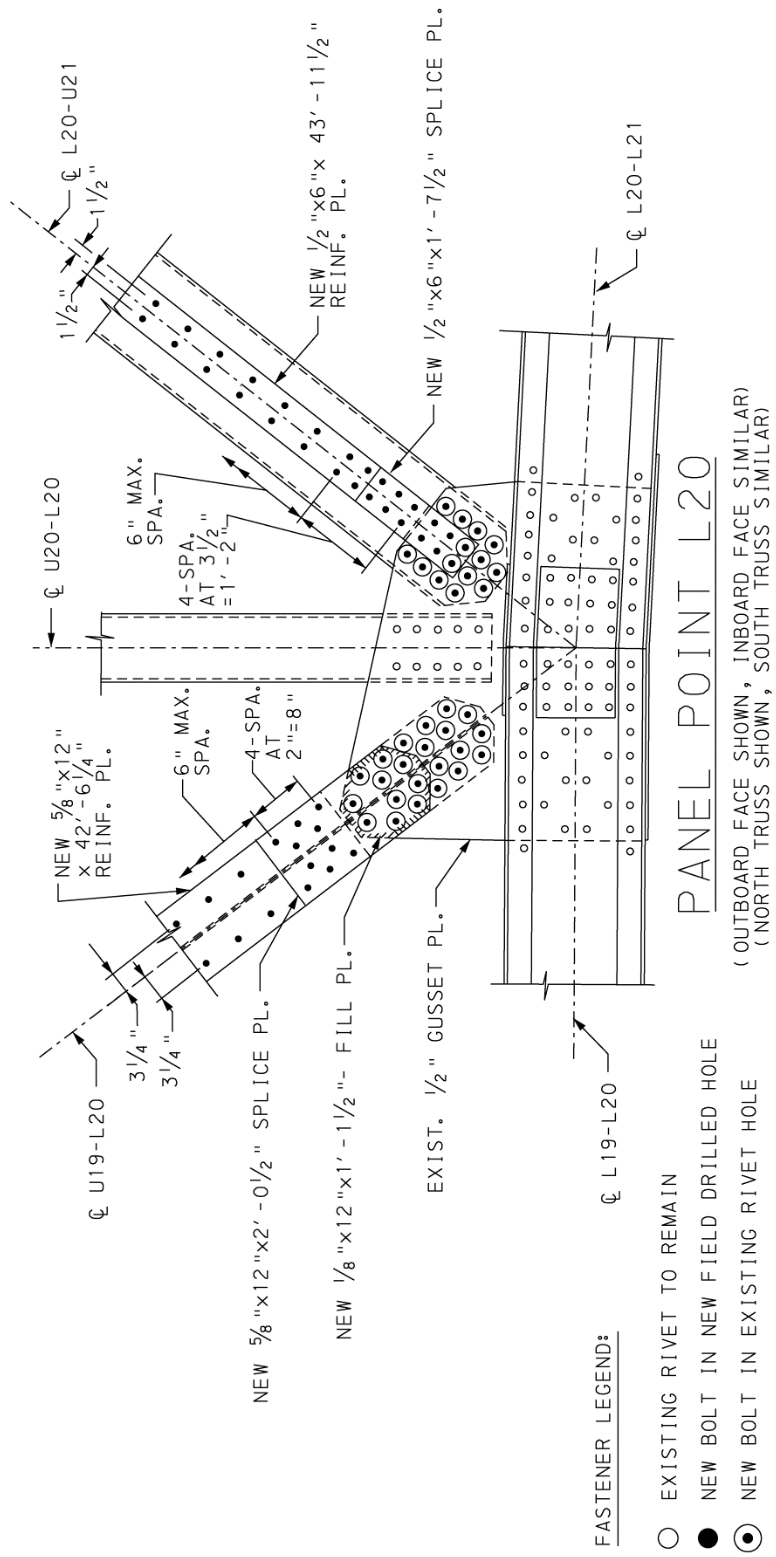
The connection is sufficient to develop the strengthened member capacity.

Additional Comments:

Both connection will require splice plates to splice the strengthening plates into the gusset plate connection. The number of bolts on either side of the splice will be the same number of bolts calculated to develop the full capacity of the strengthening plates.

The follow sketch is a summary of the strengthening and connection design.

Strengthening Design Sketch:



Summary

There are many factors that must be considered when designing steel truss member 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 truss member 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 must be checked.

The connections for the new strengthening materials must 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 for stitching and sealing, resulting in fewer new fasteners to install.

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