Report on Techniques for Bridge Strengthening

Design Example – Plate Girder Shear and Flexural Strengthening

September 2018
**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.

This report is 1 of 5 reports, including a main report, funded under Task 6 of the FHWA Cooperative Agreement DTFH61-11-H-0027.

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This design example, plate girder shear and flexural strengthening, 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 is based on AASHTO LRFD Bridge Design Specifications, 7th Edition.
# SI* (MODERN METRIC) CONVERSION FACTORS

## APPROXIMATE CONVERSIONS TO SI UNITS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>When You Know</th>
<th>Multiply By</th>
<th>To Find</th>
<th>Symbol</th>
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</thead>
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**NOTE:** Volumes greater than 1000 L shall be shown in m³

## MASS

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## ILLUMINATION

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## APPROXIMATE CONVERSIONS FROM SI UNITS

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<tr>
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## TEMPERATURE (exact degrees)

<table>
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<th>°F</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.8°C+32</td>
<td>5°F</td>
</tr>
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</table>

## ILLUMINATION

<table>
<thead>
<tr>
<th>Symbol</th>
<th>When You Know</th>
<th>Multiply By</th>
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## FORCE and PRESSURE or STRESS

<table>
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<th>Symbol</th>
<th>When You Know</th>
<th>Multiply By</th>
<th>To Find</th>
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</thead>
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</tbody>
</table>

*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.

<table>
<thead>
<tr>
<th>Publication Title</th>
<th>Publication Year</th>
<th>Publication Number</th>
<th>Available for Download</th>
</tr>
</thead>
</table>

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.\(^2\))
\( A_{gbp} \) = gross area of new bottom cover plate (in.\(^2\))
\( A_{gp} \) = total gross area of new cover plates (in.\(^2\))
\( A_{gtp} \) = gross area of new top cover plates (in.\(^2\))
\( A_{grd} \) = gross area of built-up girder (in.\(^2\))
\( A_{nbp} \) = net area of new bottom cover plate (in.\(^2\))
\( A_{np} \) = total net area of new cover plates (in.\(^2\))
\( A_{ntp} \) = 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.)
\( b_{tp} \) = 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.)
\( D_{M1} \) = moment live load distribution for single lane
\( D_{M2} \) = moment live load distribution for two lanes
\( D_{V1} \) = shear live load distribution for single lane
\( D_{V2} \) = shear live load distribution for two lanes
\( d_w \) = 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)
\( e_{g} \) = distance between centers of gravity of the beam and the deck (in.)
\( E_s \) = modulus of elasticity of steel (ksi)
\( f_{b,DCbf} \) = 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:

- $f_{y_{LL}}$: 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)
- $F_{ot}$: 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.$^4$)
- $I_{s}$: 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.)
- $L_{u}$: 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.)
- $L_{y}$: 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 stringers 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.)
\[ P_{rp} \] = 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/rivet 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)
\[ S_{xmin} \] = 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)
- $t_{hr}$ = thickness of the bottom flange plate of a built-up girder (in.)
- $t_{hauch}$ = 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.)
- $u_{wc}$ = uniform density weight of concrete (lb./ft$^3$)
- $u_{wDC1}$ = uniform weight of non-composite dead load (lb./ft)
- $u_{wDC2}$ = uniform weight of composite dead load (lb./ft)
- $u_{wDW}$ = uniform density weight of wearing surface (lb./ft)
- $u_{wp}$ = uniform weight of parapet (lb./ft)
- $u_{ws}$ = uniform density weight of steel (lb./ft$^3$)
- $u_{wSIP}$ = uniform weight of stay-in-place forms (lb./ft$^2$)
- $u_{wws}$ = 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.)
- $W_{c-c}$ = the curb-to-curb 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.)
- $w_{t\,grd}$ = uniform weight of girder (lb./ft)
- $w_{t\,hanch}$ = uniform weight of haunch per girder (lb./ft)
- $w_{t\,m}$ = uniform weight of miscellaneous dead loads per girder (lb./ft)
- $w_{t\,p}$ = uniform weight of parapet per girder (lb./ft)
- $w_{t\,SIP}$ = uniform weight of stay-in-place forms per girder (lb./ft)
- $w_{t\,slab}$ = uniform weight of slab per girder (lb./ft)
- $w_{t\,ws}$ = 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 and the c.g. of the built-up member (in.)
- $y_2$ = distance between the c.g. of section 2 and 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.)
- $\phi M_n$ = factored moment resistance (k-in.)
- $\phi P_{n\,y\,p}$ = factored resistance of new plates for yielding on gross section (kip)
- $\phi P_{n\,u\,p}$ = factored resistance of new plates for fracture on net section (kip)
- $\phi R_n$ = factored shear resistance of bolt (kip)
- $\phi T_n$ = factored tension resistance of bolt (kip)
- $\phi V_n$ = factored shear resistance (kip)
Symbols and Notation

Variables used throughout the design example are listed alphabetically below:

- $\phi_{bb}$ = resistance factor for bolt bearing on connected material
- $\phi_f$ = resistance factor for flexure
- $\phi_s$ = resistance factor for shear on bolt
- $\phi_u$ = resistance factor for fracture on net section of tension member
- $\phi_v$ = resistance factor for shear
- $\phi_y$ = resistance factor for yielding on gross section of tension member
- $\gamma_{DC}$ = load factor for dead load, non-composite and composite
- $\gamma_{DW}$ = load factor for future wearing surface
- $\gamma_{LL}$ = load factor for live load and live load impact
- $\eta_D$ = load modifier for ductility
- $\eta_i$ = load modifier relating to ductility, redundancy and operational classification
- $\eta_I$ = load modifier for operational classification
- $\eta_R$ = load modifier for redundancy
- $\lambda_f$ = slenderness ratio for the compression flange
- $\lambda_{pf}$ = limiting slenderness ratio for a compact flange
- $\lambda_{rf}$ = limiting slenderness ratio for a non-compact flange
- $\sigma_1$ = primary stress in the $x$-direction (ksi)
- $\sigma_2$ = primary stress in the $y$-direction (ksi)
- $\tau_{xy}$ = shear stress in the $xy$-plane (ksi)
- $\theta$ = $\frac{1}{2}$ 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 an 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. 13/8” x 15”
Web Plate: PL. 3/8” x 59”
Bottom Flange: PL. 13/8” 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: $u_{ws} = 490$ lb./ft$^3$
Unit Weight of Concrete: $u_{wc} = 150$ lb./ft$^3$
Unit Weight of Overlay: $u_{wos} = 145$ lb./ft$^3$
PLATE GIRDER SHEAR AND FLEXURAL STRENGTHENING

DESIGN EXAMPLE

CROSS SECTION

ELEVATION

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PLATE GIRDER SHEAR AND FLEXURAL STRENGTHENING

DESIGN EXAMPLE

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)

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LRFD Factors:

For this example use: \( \eta_D = 1.0 \quad \eta_R = 1.0 \quad \eta_I = 1.0 \) therefore: \( \eta_i = 1.0 \)

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<thead>
<tr>
<th>Resistance Factors</th>
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<tr>
<td><strong>Type of Resistance</strong></td>
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<tr>
<td>Flexure</td>
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<td>Axial Compression</td>
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<tr>
<td>Tension, fracture in An</td>
</tr>
<tr>
<td>Tension, yielding in Ag</td>
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<tr>
<td>A325 bolt in shear</td>
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<tr>
<td>A141 rivet in shear</td>
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<td>Fastener bearing on material</td>
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<table>
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<th>Load Combinations and Load Factors</th>
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<tr>
<td><strong>Limit State</strong></td>
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<td>DC</td>
</tr>
<tr>
<td>Strength I</td>
</tr>
<tr>
<td>Strength II</td>
</tr>
<tr>
<td>Service II</td>
</tr>
</tbody>
</table>

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

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

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

\[ w_{tgrd} = A_{grd} (12 \text{ in./ft.})(1,728 \text{ in.}^3/\text{ft.}^3)(u_{w,c}) = 63.375 \text{ in.}^2 (12 \text{ in./ft.})(1,728 \text{ in.}^3/\text{ft.}^3)(490 \text{ lb./ft.}^3) \]

\[ w_{tgrd} = 215.65 \text{ lb./ft.} \]

Deck Slab

\[ t_{slab} = 8 \text{ in.} \quad \text{deck slab thickness (without overlay)} \]

\[ S = 7.5 \text{ ft.} \quad \text{girder spacing (tributary width for slab weight on girder)} \]

\[ w_{t_{slab}} = t_{slab} S u_{w,c} (1 \text{ ft.}/12 \text{ in.}) = (8 \text{ in.})(7.5 \text{ ft.})(150 \text{ lb./ft.}^3) (1 \text{ ft.}/12 \text{ in.}) = 750 \text{ lb./ft.} \]
Dead Loads (continued):

Deck Haunch

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

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

\[ w_{hnch} = t_{hnch} \times w_{hnch} \times uw_c \left( 1 \text{ ft.}^2/144 \text{ in.}^2 \right) = (8 \text{ in.})(18 \text{ in.})(150 \text{ lb./ft.}^3) (1 \text{ ft.}^2/144 \text{ in.}^2) = 51.56 \text{ lb./ft.} \]

Stay-in-Place Forms

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

\[ w_{SIP} = 6.25 \text{ ft. tributary width of S.I.P. forms (S – b_{ft})} \]

\[ wt_{SIP} = uw_{SIP} \times w_{SIP} = (15 \text{ psf})(6.25 \text{ ft.}) = 93.75 \text{ lb./ft.} \]

Concrete Parapet

\[ uw_p = 520 \text{ lb./ft. weight of one f-shape parapet per linear foot} \]

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

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

Miscellaneous

\[ wt_m = 50 \text{ lb./ft. assumed weight for miscellaneous items: stiffeners, cross frame, etc.} \]

Deck Overlay (Wearing Surface)

\[ t_{ws} = 2.5 \text{ in. overlay thickness} \]

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

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

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

\[ wt_{ws} = 197.1 \text{ lb./ft.} \]

Summary

\[ u_{DC1} = wt_{grd} + wt_{slab} + wt_{hnch} + wt_{SIP} + wt_p + wt_m \]

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

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

\[ u_{DW} = wt_{ws} = 197.1 \text{ lb./ft.} \]
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):

<table>
<thead>
<tr>
<th>Piece</th>
<th>t (in.)</th>
<th>w (in.)</th>
<th>A (in.²)</th>
<th>y (in.)</th>
<th>Ay (in.²)</th>
<th>Ad² (in.⁴)</th>
<th>Io (in.⁴)</th>
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</thead>
<tbody>
<tr>
<td>Top Flange</td>
<td>1.375</td>
<td>15</td>
<td>20.625</td>
<td>30.1875</td>
<td>622.62</td>
<td>18,795</td>
<td>3.25</td>
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<td>Web</td>
<td>0.375</td>
<td>59</td>
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<td>0.00</td>
<td>0.00</td>
<td>6,418</td>
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<tr>
<td>Bottom Flange</td>
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<td>15</td>
<td>20.625</td>
<td>-30.1875</td>
<td>-622.62</td>
<td>18,795</td>
<td>3.25</td>
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<tr>
<td><strong>Totals</strong></td>
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<td>63.375</td>
<td>0.00</td>
<td>37,590</td>
<td>6,425</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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

\[ I_x = \Sigma Io + \Sigma Ad^2 = 37,590 \text{ in.}^4 + 6,425 \text{ in.}^4 = 44,015 \text{ in.}^4 \]

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

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

Live Load Distribution Factors:

from above:

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

\[ S = 7.5 \text{ ft.} \quad A_{grd} = 63.375 \text{ in.}^2 \]

also calculate:

\[ n = \frac{E_s}{E_c} = \frac{29,000 \text{ ksi}}{3,640 \text{ ksi}} = 7.97, \text{ use 8.0} \]

\[ e_g = D_w/2 + t_{tf} + t_{nch} + t_{slab}/2 = 59 \text{ in.}/2 + 1.375 \text{ in.} + 2.75 \text{ in.} + 8 \text{ in.}/2 = 37.625 \text{ in.} \]

\[ K_g = n \left( I_x + A_{grd} e_g^2 \right) = 8 \left( 44,015 \text{ in.}^4 + 63.375 \text{ in.}^2 \left( 37.625 \text{ in.} \right)^2 \right) = 1,069,850 \text{ in.}^4 \]

\[
\frac{K_g}{12 L t_{slab}^3} = \frac{1,069,850\text{in.}^4}{12(86\text{ ft.})(8\text{ in.})^3} = 2.025
\]
Live Loads (continued):

Interior Beam – Moment Distribution Factor:  

LRFD Table 4.6.2.2.2b-1

One Lane

\[ \text{DF}_{M1} = 0.06 + \left( \frac{S}{14} \right)^{0.4} \left( \frac{S}{L} \right)^{0.3} \left( \frac{K_g}{12 L t_{slab}} \right)^{0.1} = 0.06 + \left( \frac{7.5 \text{ ft.}}{14} \right)^{0.4} \left( \frac{7.5 \text{ ft.}}{86 \text{ ft.}} \right)^{0.3} (2.025)^{0.1} = 0.462 \]

Two Lanes

\[ \text{DF}_{M2} = 0.075 + \left( \frac{S}{9.5} \right)^{0.6} \left( \frac{S}{L} \right)^{0.2} \left( \frac{K_g}{12 L t_{slab}} \right)^{0.1} = 0.075 + \left( \frac{7.5 \text{ ft.}}{9.5} \right)^{0.6} \left( \frac{7.5 \text{ ft.}}{86 \text{ ft.}} \right)^{0.2} (2.025)^{0.1} = 0.647 \]

Interior Beam – Moment Distribution Factor:  

LRFD Table 4.6.2.2.3a-1

One Lane

\[ \text{DF}_{V1} = 0.36 + \frac{S}{25.0} = 0.36 + \frac{7.5 \text{ ft.}}{25.0} = 0.66 \]

Two Lanes

\[ \text{DF}_{V2} = 0.2 + \frac{S}{12} \left( \frac{S}{35} \right)^{2.0} = 0.2 + \frac{7.5 \text{ ft.}}{12.0} \left( \frac{7.5 \text{ ft.}}{35} \right)^{2.0} = 0.779 \]

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

Dynamic Load Allowance, IM (impact factor):  

LRFD Table 3.6.2.1-1

\[ \text{IM} = 33\% \]

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.

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### Design Moments

<table>
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<tr>
<th>20&lt;sup&gt;th&lt;/sup&gt; Point</th>
<th>x-dist. ft.</th>
<th>M&lt;sub&gt;UL&lt;/sub&gt; k-ft</th>
<th>M&lt;sub&gt;DC1&lt;/sub&gt; k-ft</th>
<th>M&lt;sub&gt;DW&lt;/sub&gt; k-ft</th>
<th>M&lt;sub&gt;LL&lt;/sub&gt; k-ft</th>
<th>M&lt;sub&gt;LL+I&lt;/sub&gt; k-ft</th>
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</thead>
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<td>0</td>
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<td>0</td>
<td>0</td>
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</tr>
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<td>0.05</td>
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<td>32</td>
<td>261</td>
<td>224</td>
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<tr>
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</table>

- **M<sub>UL</sub>** = Moment on simple span with 1 k./ft. uniform unit load.
- **M<sub>DC1</sub>** = Non-composite dead load moment, **M<sub>UL</sub>** DC1 (1 kip/1,000 lb.)
- **M<sub>DW</sub>** = Dead load moment from wearing surface, **M<sub>UL</sub>** DW (1 kip/1,000 lb.)
- **M<sub>LL</sub>** = Live load moment for HS-20 Truck
- **M<sub>LL+I</sub>** = Live load moment plus impact, (M<sub>LL</sub> DFM) (1+IM)
### DESIGN SHEARS

<table>
<thead>
<tr>
<th>x-dist. ft.</th>
<th>V\textsubscript{UL} kip.</th>
<th>V\textsubscript{UL} DC\textsubscript{1} kip.</th>
<th>V\textsubscript{UL} DW kip.</th>
<th>V\textsubscript{LL} kip.</th>
<th>V\textsubscript{LL}+l kip.</th>
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<td>53.39</td>
</tr>
<tr>
<td>0.85</td>
<td>77.4</td>
<td>34.00</td>
<td>46.54</td>
<td>6.20</td>
<td>56.99</td>
</tr>
<tr>
<td>0.90</td>
<td>81.7</td>
<td>38.00</td>
<td>52.02</td>
<td>6.92</td>
<td>60.59</td>
</tr>
<tr>
<td>0.95</td>
<td>86.0</td>
<td>43.00</td>
<td>58.87</td>
<td>7.84</td>
<td>64.19</td>
</tr>
</tbody>
</table>

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

- \( V\textsubscript{UL} \) = Shear on simple span with 1 k./ft. uniform unit load.
- \( V\textsubscript{UL} \textsubscript{DC1} \) = Non-composite dead load shear, \( V\textsubscript{UL} \textsubscript{DC1} \) (1 kip/1,000 lb.)
- \( V\textsubscript{UL} \textsubscript{DW} \) = Dead load shear from wearing surface, \( V\textsubscript{UL} \textsubscript{DW} \) (1 kip/1,000 lb.)
- \( V\textsubscript{LL} \) = Live load shear for HS-20 Truck
- \( V\textsubscript{LL}+l \) = Live load shear plus impact, \( V\textsubscript{LL} \textsubscript{DFV} \) (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, CONTINUOUS 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:

$$\lambda_f = \frac{b_{tf}}{2 t_{tf}} = \frac{15\text{ in.}}{2(1.375\text{ in.})} = 5.455$$  \hspace{1cm} \text{LRFD Eqn. 6.10.8.2.2-3}

$$\lambda_{pf} = 0.38 \sqrt{\frac{E_s}{F_y}} = 0.38 \sqrt{\frac{29,000\text{ ksi}}{36\text{ ksi}}} = 10.785$$  \hspace{1cm} \text{LRFD Eqn. 6.10.8.2.2-4}

$$\lambda_{rf} = 0.56 \sqrt{\frac{E_s}{F_y}} = 0.56 \sqrt{\frac{29,000\text{ ksi}}{36\text{ ksi}}} = 15.894$$  \hspace{1cm} \text{LRFD Eqn. 6.10.8.2.2-5}

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

Compression Flange Flexural Resistance – Lateral Torsional Buckling:

$$r_r = \frac{b_{tf}}{\sqrt{12(1 + \frac{1}{3} \frac{D_{tw}}{b_{tf} t_{tf}})}} = \frac{15\text{ in.}}{\sqrt{12(1 + \frac{1}{3} \frac{(29.5\text{ in.})(0.375\text{ in.})}{(15\text{ in.})(1.375\text{ in.})}})} = 3.988\text{ in.}$$ \hspace{1cm} \text{LRFD Eqn. 6.10.8.2.3-9}

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

if $L_b \leq L_p$, 0.0 in. $\leq$ 113.2 in. yes, then $F_{nc(LTB)} = F_y = 36\text{ ksi}$ \hspace{1cm} \text{LRFD Eqn. 6.10.8.2.3-1}
As-Built Flexural Resistance:

Compression Flange Flexural Resistance:

\[ F_{nc} = \text{minimum of } F_{nc(FLB)} \text{ and } F_{nc(LTB)} = 36 \text{ ksi} \]

Tension Flange Flexural Resistance:

\[ F_{nc} = F_{ye} = 36 \text{ ksi} \quad \text{LRFD Eqn. 6.10.8.3-1} \]

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

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

\[ M_{DC1} = 1,266 \text{ k-ft} \quad M_{DW} = 168 \text{ k-ft} \quad M_{LL+I} = 1,091 \text{ k-ft} \]

\[ M_u = 1.25 M_{DC1} + 1.5 M_{DW} + 1.75 M_{LL+I} = 1.25(1,266 \text{ k-ft}) + 1.5(168 \text{ k-ft}) + 1.75(1,091 \text{ k-ft}) = 3,744 \text{ k-ft} \]

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

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

As-Inspected Flexural Resistance:

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

\[ F_{nc} \text{ and } F_{nt} = 36 \text{ ksi} \]

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

Strength-I

\[ M_u = 3,744 \text{ k-ft} \text{ (same as As-Built)} \]
As-Inspected Flexural Resistance:

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

<table>
<thead>
<tr>
<th>Piece</th>
<th>t</th>
<th>w</th>
<th>A</th>
<th>y</th>
<th>Ay</th>
<th>A(y-y')²</th>
<th>Io</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top Flange</td>
<td>1.375</td>
<td>15</td>
<td>20.625</td>
<td>30.1875</td>
<td>622.62</td>
<td>15,527</td>
<td>3.25</td>
</tr>
<tr>
<td>Web</td>
<td>0.375</td>
<td>59</td>
<td>22.125</td>
<td>0.00</td>
<td>0.00</td>
<td>198</td>
<td>6,418</td>
</tr>
<tr>
<td>Bottom Flange</td>
<td>1.375</td>
<td>15</td>
<td>20.625</td>
<td>-30.1875</td>
<td>-622.62</td>
<td>22,702</td>
<td>3.25</td>
</tr>
<tr>
<td>Section Loss</td>
<td>-0.375</td>
<td>15</td>
<td>-5.625</td>
<td>-30.6875</td>
<td>172.62</td>
<td>-6,379</td>
<td>-.07</td>
</tr>
</tbody>
</table>

\[ y' = \sum Ay / A = 172.62 \text{ in.}^3 / 57.75 \text{ in.}^2 = 2.989 \text{ in. upwards} \]

\[ A_{\text{grad}} = 57.75 \text{ in.}^2 \]

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

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

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

\[ S_{x_{\text{min}}} = 1,141 \text{ in.}^3 \text{ (tension flange controls)} \]

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

Determine maximum factored stress in flanges:

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

\[ f_{bu} = 32.79 \text{ ksi} \leq \phi_f F_y = 1.0(36 \text{ ksi}) = 36 \text{ ksi}, \text{ OK} \]

LRFD Eqn. 6.10.8.1.3-1

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

\[ f_{bu} = 39.38 \text{ ksi} > \phi_f F_{ye} = 1.0(36 \text{ ksi}) = 36 \text{ ksi}, \text{ NG} \]

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: \( t_w = 0.375 \text{ in.} \quad D_w = 59 \text{ in.} \quad \phi_v = 1.0 \)

Also: \( d_o = 48 \text{ in.} \) web stiffener spacing in end panel.

\( h_{sl} = 36 \text{ in.} \) height of section loss

\( t_{sl} = 0.25 \text{ in.} \) total web thickness loss in \( h_{sl} \).

\[
k = 5.0 + \frac{5}{\left(\frac{d_o}{D_w}\right)^2} = 5 + \frac{5}{\left(\frac{48 \text{ in.}}{59 \text{ in.}}\right)^2} = 12.554
\]

LRFD Eqn. 6.20.9.3.2-7

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

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

\[
V_p = 0.58 t_w D_w F_y = 0.58(0.375 \text{ in.})(59 \text{ in.})(36 \text{ ksi}) = 461.97 \text{ kip} \quad \text{LRFD Eqn. 6.20.9.3.2-3}
\]

\[
\phi_v V_n = \phi_v V_p C = 1.0(461.97 \text{ kip})(0.642) = 296.5 \text{ kip} \quad \text{LRFD Eqn. 6.20.9.3.2-7}
\]

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.

\[
V_p = 0.58(t_w D_w - h_{sl} t_{sl}) F_y = 0.58(0.375 \text{ in.} 59 \text{ in.} - 36 \text{ in.} 0.25 \text{ in.})(36 \text{ ksi}) = 274.1 \text{ kip}
\]

\[
\phi_v V_n = \phi_v V_p C = 1.0(274.1 \text{ kip})(0.642) = 175.9 \text{ kip}
\]

Strength-I

\[
V_{DC1} = 58.87 \text{ kip} \quad V_{DW} = 7.84 \text{ kip} \quad V_{LL+I} = 64.19 \text{ kip}
\]

\[
V_u = 1.25 V_{DC1} + 1.5 V_{DW} + 1.75 V_{LL+I} = 1.25(58.87 \text{ kip}) + 1.5(7.84 \text{ kip}) + 1.75(64.19 \text{ kip}) = 197.68 \text{ kip}
\]

\[
\phi_v V_n = 296.5 \text{ kip} \geq V_u = 198 \text{ kip} \quad \text{OK (As-Built)}
\]

\[
\phi_v V_n = 175.9 \text{ kip} < V_u = 198 \text{ kip} \quad \text{NG (As-Inspected)}
\]
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:

\(\gamma_{DC}, \gamma_{DW}\) and \(\gamma_{LL}\) are the LRFD load factors

DC, DW and LL+I are the force effects

C is the member factored capacity

Flexural Ratings:

recall:

\(\gamma_{DC} = 1.25\)
\(\gamma_{DW} = 1.5\)
\(\gamma_{LL} = 1.75\)

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

\(\phi M_n = 4,278\) k-ft (As-Built)
\(\phi M_n = 3,423\) k-ft (As-Inspected)

As-Built:

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

As-Inspected:

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

Flexural strengthening is required.

Shear Ratings (End Panel):

recall:

\(\gamma_{DC} = 1.25\)
\(\gamma_{DW} = 1.5\)
\(\gamma_{LL} = 1.75\)

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

\(\phi V_n = 296.3\) kip (As-Built)
\(\phi V_n = 175.8\) kip (As-Inspected)
Shear Ratings (continued):

As-Built:
\[
RF = \frac{\phi V_n - \gamma_{DC} V_{DC} - \gamma_{DW} V_{DW}}{\gamma_V V_{LL+1}} = \frac{296.3 \text{ kip} - (1.25)(58.87 \text{ kip}) - (1.5)(7.84 \text{ kip})}{(1.75)(64.19 \text{ kip})} = 1.878
\]

As-Inspected:
\[
RF = \frac{\phi V_n - \gamma_{DC} V_{DC} - \gamma_{DW} V_{DW}}{\gamma_V V_{LL+1}} = \frac{175.8 \text{ kip} - (1.25)(58.87 \text{ kip}) - (1.5)(7.84 \text{ kip})}{(1.75)(64.19 \text{ kip})} = 0.805
\]

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)

\[ A_{g\text{bp}} = 0.375 \text{ in.} \times 15 \text{ in.} = 5.625 \text{ in.}^2 \]
\[ A_{g\text{tp}} = 2 \times 0.5 \text{ in.} \times 5.625 \text{ in.} = 5.625 \text{ in.}^2 \]
\[ \frac{A_{g\text{tp}}}{A_{g\text{bp}}} = \frac{5.625 \text{ in.}^2}{5.625 \text{ in.}^2} = 1.0 \text{ OK} \]

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

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.

\[ F_{b\text{ne}} = \phi_f F_y = 1.0 \cdot 36 \text{ ksi} = 36.0 \text{ ksi} \quad \text{(existing steel)} \]
\[ F_{b\text{nn}} = \phi_f F_{yn} = 1.0 \cdot 50 \text{ ksi} = 50.0 \text{ ksi} \quad \text{(new steel)} \]

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:

\[ s^2 / 4g = (3.5 \text{ in.})^2 / (4 \cdot 2.625 \text{ in.}) = 1.167 \text{ in.} > 1.0 \text{ in.} \text{ (bolt hole diameter)} \]

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

\[ W_{\text{net}} = 15 \text{ in.} - (2 \text{ holes})(1 \text{ in./hole}) = 13 \text{ in.} \geq 0.85 (W_g) = 0.85(15 \text{ in.}) = 12.75 \text{ in.}, \text{ use } 12.75 \text{ in.} \]

\[ W_b = W_g - W_{\text{net}} = 15 \text{ in.} - 12.75 \text{ in.} = 2.25 \text{ in.} \text{ (effective width of bolt holes for section properties.)} \]

<table>
<thead>
<tr>
<th>Piece</th>
<th>T (in.)</th>
<th>W (in.)</th>
<th>A (in.²)</th>
<th>Y (in.)</th>
<th>Ay (in.²)</th>
<th>A(y-y')² (in.⁴)</th>
<th>Io (in.⁴)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top Flange</td>
<td>1.375</td>
<td>15</td>
<td>20.625</td>
<td>30.1875</td>
<td>622.62</td>
<td>13,837</td>
<td>3.25</td>
</tr>
<tr>
<td>Web</td>
<td>0.375</td>
<td>59</td>
<td>22.125</td>
<td>0.00</td>
<td>0.00</td>
<td>406</td>
<td>6,418</td>
</tr>
<tr>
<td>Bottom Flange</td>
<td>1.375</td>
<td>15</td>
<td>20.625</td>
<td>-30.1875</td>
<td>-622.62</td>
<td>24,511</td>
<td>-0.07</td>
</tr>
<tr>
<td>Section Loss</td>
<td>-0.375</td>
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<td>-6,880</td>
<td>-2,493</td>
</tr>
<tr>
<td>Bolt Holes</td>
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<td>-2.25</td>
<td>-29.000</td>
<td>65.25</td>
<td>-2,493</td>
<td></td>
</tr>
<tr>
<td><strong>Totals</strong></td>
<td>55.50</td>
<td>237.87</td>
<td>29,382</td>
<td>6,425</td>
<td></td>
<td></td>
<td>6,425</td>
</tr>
</tbody>
</table>

\[ y' = \sum A_y / A = 237.87 \text{ in.}^3 / 55.50 \text{ in.}^2 = 4.286 \text{ in.} \text{ upwards} \]

\[ I_x = \sum Io + \sum Ad^2 = 29,382 \text{ in.}^4 + 6,425 \text{ in.}^4 = 35,806 \text{ in.}^4 \]

\[ S_{xc} = I_x / y_{if} \text{, where } y_{if} = D_w/2 + t_{if} - y', \text{ then } S_{xc} = 35,806 \text{ in.}^4 / 26.589 \text{ in.} = 1,347 \text{ in.}^3 \]

\[ S_{xt} = I_x / y_{br}, \text{ where } y_{br} = D_w/2 + t_{bf} + y' - SL, \text{ then } S_{xt} = 35,806 \text{ in.}^4 / 35.161 \text{ in.} = 1,018 \text{ in.}^3 \]
Determine the Strengthened Member Net Section Properties:

Net Width of Strengthening Plates

\[ W_{n\,bp} = 12.75 \text{ in.} \quad \text{Bottom Plate (same as existing bottom flange)} \]
\[ W_{n\,tp} = 5.625 \text{ in.} \quad (1 \text{ holes})(1 \text{ in./hole}) = 4.625 \text{ in.} < 0.85 \left(W_g\right) = 0.85(5.625 \text{ in.}) = 4.7813 \text{ in.} \]
\[ W_{n\,tp} = 4.625 \text{ in.} \quad \text{each Top Plate.} \]
\[ W_{n\,ef} = W - W_{n\,ef} = 15 \text{ in.} - 12.75 \text{ in} = 2.25 \text{ in.} \quad \text{(existing flange)} \]
\[ W_{b\,bp} = W - W_{n\,bp} = 15 \text{ in.} - 12.75 \text{ in} = 2.25 \text{ in.} \quad \text{(bottom plate)} \]
\[ W_{b\,tp} = 2(W - W_{n\,tp}) = 2(5.625 \text{ in.} - 4.625 \text{ in.}) = 2.00 \text{ in.} \quad \text{(top plates)} \]
\[ W_b = 2.25 \text{ in.} \quad \text{(conservative, effective width of bolts holes for section properties.)} \]

<table>
<thead>
<tr>
<th>Piece</th>
<th>( t )</th>
<th>( w )</th>
<th>( A )</th>
<th>( y )</th>
<th>( A_y )</th>
<th>( A(y-y')^2 )</th>
<th>( I_o )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top Flange</td>
<td>1.375</td>
<td>15</td>
<td>20.625</td>
<td>30.1875</td>
<td>622.62</td>
<td>21,978</td>
<td>3.25</td>
</tr>
<tr>
<td>Web</td>
<td>0.375</td>
<td>59</td>
<td>22.125</td>
<td>0.00</td>
<td>0.00</td>
<td>133</td>
<td>6,418</td>
</tr>
<tr>
<td>Top Cov. PLs</td>
<td>0.5</td>
<td>11.25</td>
<td>5.625</td>
<td>-29.750</td>
<td>-167.3</td>
<td>4,190</td>
<td>0.12</td>
</tr>
<tr>
<td>Bottom Flange</td>
<td>1.375</td>
<td>15</td>
<td>20.625</td>
<td>-30.1875</td>
<td>-622.62</td>
<td>15,862</td>
<td>3.25</td>
</tr>
<tr>
<td>Bott. Cov. PLs</td>
<td>0.375</td>
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<td>-31.0625</td>
<td>-174.7</td>
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<td>-30.6875</td>
<td>172.62</td>
<td>-4,483</td>
<td>-0.07</td>
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<td>-4.219</td>
<td>-30.125</td>
<td>127.1</td>
<td>-3,664</td>
<td>-</td>
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<td>64.78</td>
<td>-42.36</td>
<td>38,843</td>
<td>6,425</td>
<td></td>
</tr>
</tbody>
</table>

\[ y' = \Sigma A_y / A = -42.36 \text{ in.}^3 / 64.78 \text{ in.}^2 = -0.654 \text{ in.} \quad \text{(downwards)} \]
\[ I_x = \Sigma I_o + \Sigma A d^2 = 38,843 \text{ in.}^4 + 6,425 \text{ in.}^4 = 45,268 \text{ in.}^4 \]
\[ y_{tf} = D_w/2 + t_{tf} - y' = 31.529 \text{ in.} \quad \text{(centroid to top of top flange, existing)} \]
\[ y_{bf} = D_w/2 + t_{bf} + y' = 30.221 \text{ in.} \quad \text{(centroid to bottom of bottom flange, existing)} \]
\[ y_{bp} = D_w/2 + t_{br} + t_{bp} + y' = 30.596 \text{ in.} \quad \text{(centroid to bottom of bottom cover plate, new)} \]
\[ S_{tf} = 1 / y_{tf} = 38,843 \text{ in.}^4 / 31.529 \text{ in.} = 1,436 \text{ in.}^3 \quad \text{(top flange, existing)} \]
\[ S_{bf} = 1 / y_{bf} = 38,843 \text{ in.}^4 / 30.221 \text{ in.} = 1,498 \text{ in.}^3 \quad \text{(bottom flange, existing)} \]
\[ S_{bp} = 1 / y_{bp} = 38,843 \text{ in.}^4 / 30.596 \text{ in.} = 1,480 \text{ in.}^3 \quad \text{(bottom cover plate, new)} \]
Determine the Member Flexural Stresses:

from above:

\[
\begin{align*}
M_{DC} &= 1,266 \text{ k-ft} \\
M_{DW} &= 168 \text{ k-ft} \\
M_{LL+I} &= 1,091 \text{ k-ft} \\
S_{ebf} &= 1,266 \text{ in}^3 \\
S_{sbf} &= 1,498 \text{ in}^3 \\
S_{sbp} &= 1,480 \text{ in}^3
\end{align*}
\]

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:

\[
f_b \text{ DCebf} = \frac{M_{DC}}{S_{ebf}} = \frac{1,266 \text{ k-ft}(12 \text{ in/ft})}{1,018 \text{ in}^3} = 14.92 \text{ ksi}
\]

flexural stress on net section of strengthened member, existing bottom flange from \(M_{DW}\):

\[
f_b \text{ DWsbf} = \frac{M_{DW}}{S_{sbf}} = \frac{168 \text{ k-ft}(12 \text{ in/ft})}{1,498 \text{ in}^3} = 1.35 \text{ ksi}
\]

flexural stress on net section of strengthened member, new bottom plate from \(M_{DW}\):

\[
f_b \text{ DWsbp} = \frac{M_{DW}}{S_{sbp}} = \frac{168 \text{ k-ft}(12 \text{ in/ft})}{1,480 \text{ in}^3} = 1.36 \text{ ksi}
\]

flexural stress on net section of strengthened member, existing bottom flange from \(M_{LL+I}\):

\[
f_b \text{ LLsbf} = \frac{M_{LL+I}}{S_{sbf}} = \frac{1,091 \text{ k-ft}(12 \text{ in/ft})}{1,498 \text{ in}^3} = 8.74 \text{ ksi}
\]

flexural stress on net section of strengthened member, new bottom plate from \(M_{LL+I}\):

\[
f_b \text{ LLsbp} = \frac{M_{LL+I}}{S_{sbp}} = \frac{1,091 \text{ k-ft}(12 \text{ in/ft})}{1,480 \text{ in}^3} = 8.85 \text{ ksi}
\]

Factored Flexural Stresses:

Existing Bottom Flange

\[
f_u \text{ ebf} = 1.25 (f_b \text{ DCebf}) + 1.5(f_b \text{ DWsbf}) + 1.75(f_b \text{ LLsbf})
\]

\[
f_u \text{ ebf} = 1.25(14.92 \text{ ksi}) + 1.5(1.35 \text{ ksi}) + 1.75(8.74 \text{ ksi}) = 35.97 \text{ ksi} < F_{be} = 36 \text{ ksi} \text{ OK}
\]

New Bottom Strengthening Plate

\[
f_u \text{ nbp} = 1.25 (f_b \text{ DCebp}) + 1.5(f_b \text{ DWsbp}) + 1.75(f_b \text{ LLsbp})
\]

\[
f_u \text{ nbp} = 1.25(0.0 \text{ ksi}) + 1.5(1.36 \text{ ksi}) + 1.75(8.85 \text{ ksi}) = 17.52 \text{ ksi} < F_{bn} = 50 \text{ ksi} \text{ OK}
\]
Determine the Strengthened Member Flexural Rating Factor:

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

Flexural Ratings:
recall:

\[ \gamma_{DC} = 1.25 \quad \gamma_{DW} = 1.5 \quad \gamma_{L} = 1.75 \]

Existing Member:

\[ f_{b, DCebf} = 14.92 \text{ ksi} \quad f_{b, DWsbf} = 1.35 \text{ ksi} \quad f_{b, LLsbf} = 8.74 \text{ ksi} \]

\[ RF = \frac{F_{b_{ne}} - \gamma_{DC}f_{b, DCebf} - \gamma_{DW}f_{b, DWsbf}}{\gamma_{L}f_{b, LLsbf}} = \frac{36.0 \text{ ksi} - (1.25)(14.95 \text{ ksi}) - (1.5)(1.35 \text{ ksi})}{(1.75)(8.74 \text{ ksi})} = 1.00 \quad \text{OK} \]

Strengthening Material:

\[ f_{b, DCsbp} = 0.0 \text{ ksi} \quad f_{b, DWsbp} = 1.36 \text{ ksi} \quad f_{b, LLsbp} = 8.84 \text{ ksi} \]

\[ RF = \frac{F_{b_{nn}} - \gamma_{DC}f_{b, DCsbp} - \gamma_{DW}f_{b, DWsbp}}{\gamma_{L}f_{b, LLsbp}} = \frac{36.0 \text{ ksi} - (1.25)(0.0 \text{ ksi}) - (1.5)(1.36 \text{ ksi})}{(1.75)(8.85 \text{ ksi})} = 2.19 \quad \text{OK} \]

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:

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

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

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

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

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

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

\[ R_r = R_n = K_h K_s N_s P_t = 1.0 (0.5)(2 \text{ shear plane})(39 \text{ kip}) = 39 \text{ kip/bolt} \]
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:

\[ s_{\text{min}} = 3.0 \ D_b = 3.0 \times (0.875 \text{ in.}) = 2.625 \text{ in.} \text{, use 3.0 in. where possible} \quad \text{LRFD 6.13.2.6} \]

Maximum Bolt Spacing for Sealing:

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

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

\[ s_{\text{max}} = 4.0 + 4.0 \times (0.375 \text{ in.}) - 0.75 \times (2.625 \text{ in.}) = 3.5 \text{ in. pitch} \quad \text{LRFD Eq. 6.13.2.6.2-2} \]

Maximum Bolt Spacing for Stitching:

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

\[ s_{\text{max}} = 4.0 \ D_b = 4.0 \times (0.875 \text{ in.}) = 3.5 \text{ in. for end 1.5 w = 9 in.} \quad \text{LRFD 6.13.2.6.4} \]

Sealing Controls. Use 3.5 in. pitch full length of cover plate.

Determine number of bolts to develop the cover plates:

Recall:

\[ W_{g \ bp} = 15.0 \text{ in. gross width of new bottom plate} \]
\[ W_{n \ bp} = 12.75 \text{ in. net width of new bottom plate} \]
\[ t_{bp} = 0.375 \text{ in. thickness of new bottom plate} \]
\[ W_{g \ tp} = 5.625 \text{ in. gross width of each new top plate} \]
\[ W_{n \ tp} = 4.625 \text{ in. net width of each new top plate} \]
\[ t_{ip} = 0.5 \text{ in. thickness each new top plate} \]
PLATE GIRDER SHEAR AND FLEXURAL STRENGTHENING

DESIGN EXAMPLE

\[ A_{gbp} = W_{gbp} t_{bp} = (15.0 \text{ in.})(0.375 \text{ in.}) = 5.625 \text{ in.}^2 \]
\[ A_{nbp} = W_{nbp} t_{bp} = (12.75 \text{ in.})(0.375 \text{ in.}) = 4.78125 \text{ in.}^2 \]
\[ A_{gtp} = W_{gtp} t_{tp} = (5.625 \text{ in.})(0.5 \text{ in.}) = 2.8125 \text{ in.}^2 \]
\[ A_{ntp} = W_{ntp} t_{tp} = (4.625 \text{ in.})(0.5 \text{ in.}) = 2.3125 \text{ in.}^2 \]
\[ A_g = A_{gbp} + 2 A_{gtp} = 5.625 \text{ in.}^2 + 2(2.8125 \text{ in.}^2) = 11.25 \text{ in.}^2 \quad \text{gross area of new plates} \]
\[ A_n = A_{nbp} + 2 A_{ntp} = 4.7813 \text{ in.}^2 + 2(2.313 \text{ in.}^2) = 9.41 \text{ in.}^2 \quad \text{net area of new plates} \]

\[ \phi_y = 0.95, \text{ resistance factor for tension, yielding on gross section} \]
\[ \phi_u = 0.80, \text{ resistance factor for tension, yielding on gross section} \]

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

\[ P_r = 489.32 \text{ kip} \]

Determine the number of bolts to develop the new plates:

\[ P_r / R_r = 489.32 \text{ kip} / (55.4 \text{ kip/bolt}) = 8.83 \text{ bolts, use 10 bolts} \]

\[ L_d = (\text{No. bolts} / (\text{bolts/column}) - 1) p + 0.5 p + 0.5 p \]
\[ L_d = (10 \text{ bolts} / (2 \text{ bolts/column}) - 1) 3.5 \text{ in.} + 0.5(3.5 \text{ in.}) + 0.5(3.5 \text{ in.}) = 17.5 \text{ in.} \]
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

\[ M_{DC1} = 1,063 \text{ k-ft} \quad M_{DW} = 142 \text{ k-ft} \quad M_{LL+I} = 945 \text{ k-ft} \]
\[ M_u = 1.25 M_{DC1} + 1.5 M_{DW} + 1.75 M_{LL+I} = 1.25(1,063 \text{ k-ft}) + 1.5(142 \text{ k-ft}) + 1.75(945 \text{ k-ft}) = M_u = 3,196 \text{ k-ft} \]
Determine the moment resistance at 0.3 L (1/4 in. bottom flange thickness loss):

Recall As-Built Section Properties:

\[
\begin{align*}
& t_{bf} = 1.375 \text{ in.} \quad b_{bf} = 15.0 \text{ in.} \quad t_{tf} = 1.375 \text{ in.} \quad b_{tf} = 15.0 \text{ in.} \\
& D_w = 59.0 \text{ in.} \quad t_w = 0.375 \text{ in.} \quad A_{grd} = 63.375 \text{ in.}^2 \quad I_x = 44,015 \text{ in.}^4
\end{align*}
\]

Section Loss Properties:

\[
t_{SL} = 0.25 \text{ in.} \quad \text{thickness loss amount in region being investigated.}
\]

\[
A_{SL} = (-0.25 \text{ in.})(15 \text{ in.}) = -3.75 \text{ in.}^2
\]

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

\[
y_{SL} = \frac{D_w}{2} + t_{bf} - \frac{t_{SL}}{2} = \frac{59 \text{ in.}}{2} + 1.375 \text{ in.} - \frac{0.25 \text{ in.}}{2} = -30.75 \text{ in.}
\]

\( y' = \text{centroid of section loss from center of web} \)

\[
y' = \frac{\sum Ay}{A} = \frac{(0 \text{ in.})(63.375 \text{ in.}^2) + (-30.75 \text{ in.})(-3.75 \text{ in.}^2)}{(63.375 \text{ in.}^2 - 3.75 \text{ in.}^2)} = 1.934 \text{ in. from center of web towards top flange}
\]

\[
I_x = \sum Io + \sum Ad^2
\]

\[
= 44,015 \text{ in.}^4 + (63.375 \text{ in.}^2)(1.934 \text{ in.})^2 + (-3.75 \text{ in.}^2)(30.75 \text{ in.} + 1.934 \text{ in.})^2
\]

\[
= 40,246 \text{ in.}^4
\]

\[
S_{xt} = \frac{I_x}{y_{bf}}, \text{ where } y_{bf} = \frac{D_w}{2} + t_{bf} + y' - SL, \text{ then } S_{xt} = \frac{40,246 \text{ in.}^4}{32.559 \text{ in.}} = 1,236 \text{ in.}^3
\]

\[
\phi M_n = \phi F_{nc} S_x = (1.0)(36 \text{ ksi})(1,236 \text{ in.}^3)(1 \text{ ft.} / 12 \text{ in.}) = 3,708 \text{ k-ft}
\]

\[
\phi M_n = 3,708 \text{ k-ft} > M_u = 3,196 \text{ k-ft} \quad \text{OK}
\]

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

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Check Connections to the Bottom Flange

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

<table>
<thead>
<tr>
<th>Piece</th>
<th>t</th>
<th>w</th>
<th>A</th>
<th>y</th>
<th>Ay</th>
<th>A(y-y')²</th>
<th>Io</th>
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<tbody>
<tr>
<td></td>
<td>(in.)</td>
<td>(in.)</td>
<td>(in.²)</td>
<td>(in.)</td>
<td>(in.²)</td>
<td>(in.⁴)</td>
<td>(in.⁴)</td>
</tr>
<tr>
<td>Top Flange</td>
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<td>20.625</td>
<td>30.1875</td>
<td>622.62</td>
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<td>-622.62</td>
<td>4,190</td>
<td>0.12</td>
</tr>
<tr>
<td>New Web PLs</td>
<td>1.0</td>
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<td>28.000</td>
<td>-15.500</td>
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<tr>
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<td>-</td>
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<td>-28.5130</td>
<td>-270.87</td>
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<td>3.25</td>
</tr>
<tr>
<td><strong>Totals</strong></td>
<td></td>
<td></td>
<td><strong>100.88</strong></td>
<td><strong>-705.87</strong></td>
<td><strong>47,116</strong></td>
<td><strong>8,266</strong></td>
<td></td>
</tr>
</tbody>
</table>

\[ y' = \Sigma Ay / A = -705.87 \text{ in.}^3 / 100.88 \text{ in.}^2 = -6.988 \text{ in. downwards} \]

\[ I_x = \Sigma Io + \Sigma Ad^2 = 8,266 \text{ in.}^4 + 47,116 \text{ in.}^4 = 55,382 \text{ in.}^4 \]

\[ A_1 = A_{bf} + A_{Ls} = 20.625 \text{ in}^2 + 9.50 \text{ in}^2 = 30.125 \text{ in}^2 \]

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

\[ = -29.659 \text{ in}^2 \]

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

\[ A_2 = A_{bf} = 20.625 \text{ in}^2 \]

\[ y_2 = y_{bf} = -30.1875 \text{ in.} \]

\[ Q_2 = A_2 (y_2 - y') = 20.625 \text{ in}^2 (-30.1875 \text{ in.} - -6.988 \text{ in.}) = 478.498 \text{ in.}^3 \]
Check Connections to the Bottom Flange (cont.)

Recall:

\[ V_u = 197.68 \text{ kip (Strength-I)} \]
\[ R_r = 55.40 \text{ kip/bolt (Double Shear) or 27.7 kip/bolt (Single Shear)} \]

\[ q_1 = \frac{V_u Q_1}{I} = \frac{(197.68 \text{ kip} \cdot (689.989 \text{ in.}^3))}{55,382 \text{ in.}^4} = 2.438 \text{ kip/in.} \]
\[ r_1 = q_1 \cdot p = 2.438 \text{ kip/in. (6 in./bolt)} = 14.63 \text{ kip/bolt} \quad \text{where } p \text{ is the max bolt spacing.} \]
\[ r_1 = 14.63 \text{ kip/bolt} < R_r = 55.40 \text{ kip/bolt (Double Shear)} \quad \text{OK} \]

\[ q_2 = \frac{V_u Q_2}{I} = \frac{(197.68 \text{ kip} \cdot (478.498 \text{ in.}^3))}{55,382 \text{ in.}^4} = 1.708 \text{ kip/in.} \]
\[ r_2 = q_2 \cdot p = 1.708 \text{ kip/in. (6 in./bolt)} = 10.25 \text{ kip/bolt} \]
\[ r_2 = 10.25 \text{ kip/bolt} < R_r = 27.7 \text{ kip/bolt (Single Shear)} \quad \text{OK} \]

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

Check the New Web Plates Splice Connection.

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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: \( V_{uv} = 1.5V_u \), otherwise LRFD Eqn. 6.13.6.1.4b-1
\[ V_{uv} = 0.5 \phi_v V_n + 0.5 V_u \quad \text{LRFD Eqn. 6.13.6.1.4b-2} \]

\[ V_p = 0.58 F_y t D 2 = 0.58 (50 \text{ ksi})(0.5 \text{ in.})(28 \text{ in.}) 2 = 812 \text{ kip} \]
\[ \phi_v V_n = \phi_v V_p = 1.0 (812 \text{ kip}) = 812 \text{ kip} \]
\[ V_u = 197.68 \text{ kip} < 0.5 \phi_v V_n = 406 \text{ kip}, \text{ then } V_{uv} = 1.5 V_u = 1.5 (197.68 \text{ kip}) = 296.52 \text{ kip} \]

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:

\[ e = 4 \text{ in.}/2 + 3 \text{ in.}/2 = 3.5 \text{ in.}, \text{ centroid of bolt group (left side) to the centerline of splice.} \]
\[ d_1 = \sqrt{(3.5)(3 \text{ in.})^2 + (1.5 \text{ in.})^2} = 10.607 \text{ in.} \]
\[ d_2 = \sqrt{(2.5)(3 \text{ in.})^2 + (1.5 \text{ in.})^2} = 7.649 \text{ in.} \]
\[ d_3 = \sqrt{(1.5)(3 \text{ in.})^2 + (1.5 \text{ in.})^2} = 4.743 \text{ in.} \]
\[ d_4 = \sqrt{(0.5)(3 \text{ in.})^2 + (1.5 \text{ in.})^2} = 2.121 \text{ in.} \]
\[ J_{bg} = 4d_1^2 + 4d_2^2 + 4d_3^2 + 4d_4^2 = 4(10.607 \text{ in.})^2 + 4(7.649 \text{ in.})^2 + 4(4.743 \text{ in.})^2 + 4(2.121 \text{ in.})^2 \]
\[ J_{bg} = 792 \text{ in}^4/\text{in}^2 \]
\[ V_{uv} = 296.52 \text{ kip} \]
\[ M_{uv} = e V_{uv} = 296.52 \text{ kip} (3.5 \text{ in.}) = 1,037.8 \text{ k-in} \]
\[ N_b = 16 \text{ bolts per side of splice} \]
Check the New Web Plates Splice Connection (cont.).

Determine maximum shear force in corner bolt.

\[ V_{vv} = \frac{V_{uv}}{N_b} = \frac{296.52 \text{ kip}}{16 \text{ bolts}} = 18.53 \text{ kip/bolt, direct vertical shear} \]

\[ V_{vm} = \frac{M_x}{J} = \frac{(1,037.8 \text{ k-in (1.5 in.)})}{792 \text{ in}^2} = 1.966 \text{ kip/bolt, vertical shear from moment} \]

\[ V_{hm} = \frac{M_x}{J} = \frac{(1,037.8 \text{ k-in (10.5 in.)})}{792 \text{ in}^2} = 13.76 \text{ kip/bolt, horizontal shear from moment} \]

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

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

Check the splice plate capacity:

\[ t = 0.5 \text{ in.} \quad h = 24 \text{ in.} \quad A = 12 \text{ in}^2 \quad S_x = 48 \text{ in}^3 \quad V_{uv} = 296.52 \text{ kip} \]

\[ \phi_v V_n = 2(1.0) 0.58 A_g F_y = 2(0.58)(12 \text{ in}^2)(50 \text{ ksi})^2 = 696 \text{ kip} \geq V_{uv} = 296.52 \text{ kip} \text{ OK} \]

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

Therefore, the splice design is sufficient
Design Connection based on capacity of the larger of the new web plates.

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

\[ \phi_V \cdot V_n = 1.0 \cdot (1,160 \text{ kip}) = 1,160 \text{ kip}, \text{ new web plate factored shear resistance} \]

\[ V_u = 197.68 \text{ kip}, \text{ factored shear for Strength-I Limit State} \]

\[ 0.5V_u + 0.5 \phi_V \cdot V_n = 0.5(197.68 \text{ kip}) + 0.5(1,160 \text{ kip}) = 678.84 \text{ kip} \]

\[ V_p = 461.97 \text{ kip}, \text{ Plastic capacity of as-built web plate} \]

Design for the minimum of the average of \( V_u \) and \( \phi_V \cdot V_n \) or \( V_p \) of the existing Web \[ \text{LRFD 6.13.1} \]

\[ V_{uv} = 461.97 \text{ kip} \]

Check 6 in. leg connection.

\[ e = 0.987 \text{ in.}, \text{ center of gravity of L6x4 from the back of the angle in the 4 in. direction.} \]

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

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

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

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

Check combined shear and tension in controlling bolt. \[ \text{LRFD Eqn. 6.13.2.11-2} \]

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

\[ \phi_s T_n = 0.8(31.58 \text{ kip}) = 25.26 \text{ kip} > T_b = 2.92 \text{ kip} \text{ OK} \]

Check 4 in. leg connection.

\[ e = 0.987 \text{ in.}, \text{ center of gravity of L6x4 from the back of the angle in the 4 in. direction.} \]

\[ V_{uv} = 461.97 \text{ kip} \]

\[ M_{uv} = 455.96 \text{ k-in} \]

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

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

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

\[ V_{bs} = \sqrt{V_{vv}^2 + V_{hm}^2} = \sqrt{(35.54 \text{ kip})^2 + (5.01 \text{ kip})^2} = 35.89 \text{ kip} < R_e = 55.4 \text{ kips (Double Shear) OK} \]

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

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

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

Maximum Bolt Spacing for Stitching:  
\[ s_{\text{max}} = 12 \, t_{\text{min}} = 12 \, (0.5 \, \text{in.}) = 6 \, \text{in. pitch} \geq 6 \, \text{in max. provided OK} \]

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 its 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 load 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.
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