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FIRM:Your Firm                              JOB NO.2004-0028   SHEET NO:  1
MADE BY:KJH  DATE:06-03-2005                CHECKED BY:        DATE:
TITLE:Example SPLICE output
========================================================================
BRIDGE GIRDER FIELD SPLICE DESIGN:F.S. # 1 IN SPAN 1  AT X= 62 '
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(Splice design file:C:\DCALC\Taylor Street\DCALC49.IN)

DESIGN DATA:
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Design Specifications: 2002 AASHTO and IDOT ABD Memorandum 03.8
Design method: Load factor design
               Ultimate load = 1.3(DL + 5/3*LL)
               Overload = DL + 5/3*LL = Ultimate/1.3

Ultimate Moment at centerline of splice=   776.68 k*ft
Overload Moment at centerline of splice=   776.68/1.3 =   597.4 k*ft
Ultimate Shear at centerline of splice =   172.27 k
Overload Shear at centerline of splice =   172.27/1.3 =   132.52 k

Fatigue moment at centerline of splice =   532.70 k*ft
Number of fatigue cycles = 2,000,000
Structure has redundant load path.

MATERIAL DATA:
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Steel type: M270 Grade 50
Yield stress, Fy= 50.0 ksi
Ultimate stress, Fu= 65.0 ksi

Class A contact surface (slip coeff. 0.33)

Allowable bending stress, Fb= Fy = 50.00 ksi (AASHTO 10.48.2)
Allowable shear stress,   Fv= 0.58*Fy = 29.00 ksi (AASHTO Eq. 10-114)
Overload allowable bending, Fb=0.8*Fy =  40.0 ksi (AASHTO 10.57.1)

Web bolts:  .875 in. dia. A325  high strength friction bolts
Bolt area =  .601 sq. in.

Flange bolts:  .875 in. dia. A325  high strength friction bolts
Bolt area =  .601 sq. in.

Allowable bolt shear stress, Fvbolt= 35.00 ksi (AASHTO Table 10.56A)

Allowable bolt bearing stress, Fp=1.8*Fu=117.00 ksi (AASHTO 10.56.1.3)

Overload slip critical bolt shear, FvboltOverload= 21.00 ksi (Table 10.57A)

Allowable fatique stress, Fsr= 18.00 ksi (Cat. B, Table 10.3.1A)
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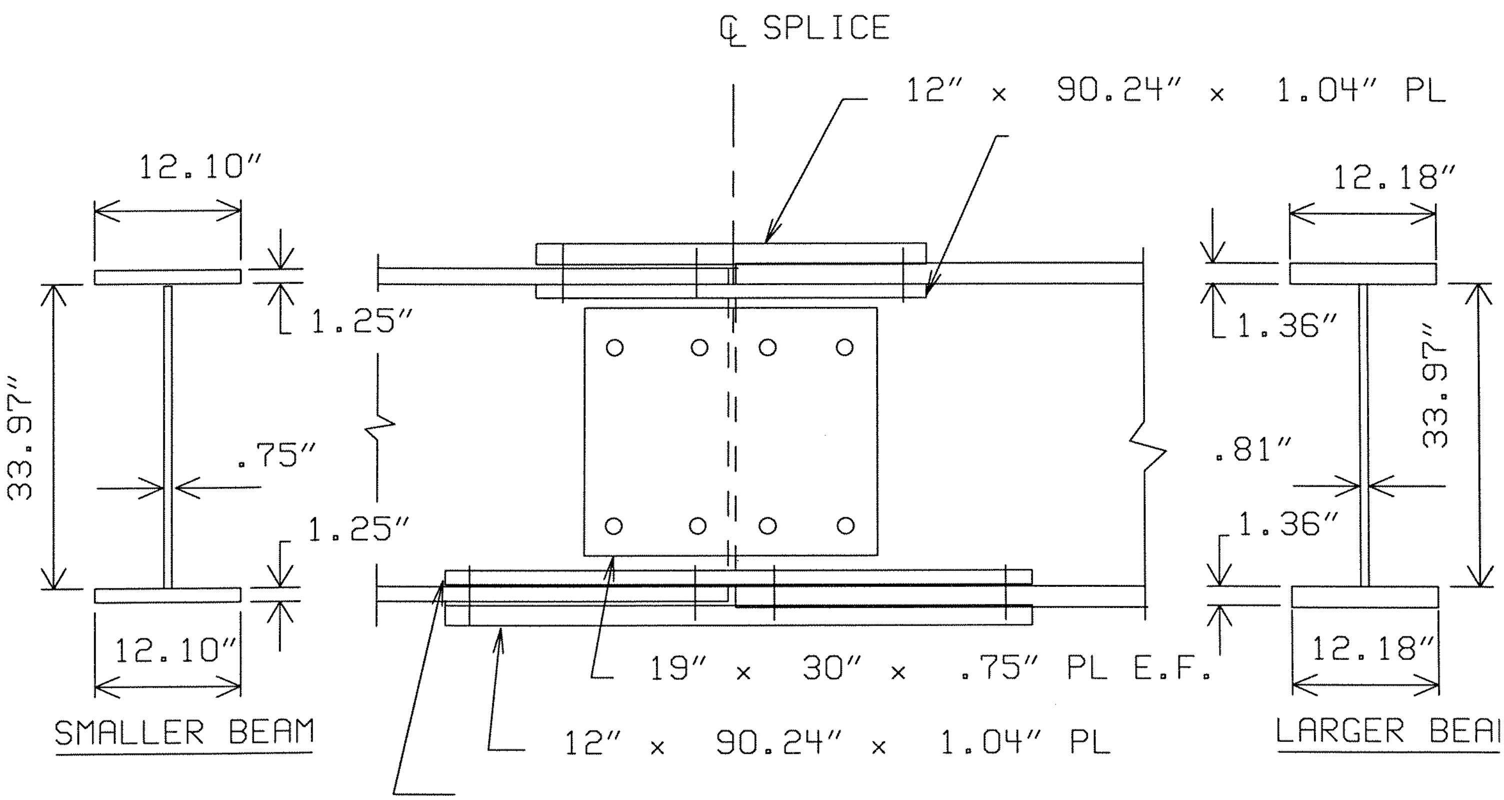
BRIDGE GIRDER FIELD SPLICE DESIGN - COMPUTE DESIGN MOMENT AND SHEAR



Compute section properties of smaller beam:

Item	A	Y	AY	AY^2	I0
Top Flange	15.13	35.84	542.60	19449.63	1.97
Web	25.81	18.23	470.77	8584.61	2482.67
Bot Flange	15.13	.62	9.46	5.91	1.97
	56.09		1022.84	28040.15	2486.61

N.A. at Y= 1022.84/ 56.09 = 18.23 in

$I = 28040.15 + 2486.61 - 56.09 * (18.23)^2$

$= 11875.25$ in^4

Stresses computed at mid-thickness of flanges:

Top flange average stress = 12*(776.6)*(17.61 in)/11875.25
= 13.82 ksi (Controlling flange)

Bot flange average stress = 12*(776.6)*(-17.61 in)/11875.25
= -13.82 ksi (Controlling flange)

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BRIDGE GIRDER FIELD SPLICE DESIGN - COMPUTE DESIGN MOMENT AND SHEAR (Cont'd)
---------------------------------------------------------------------------
(For clarity, from this point both flange stresses will be considered
 positive. AASHTO equations will have sign conventions adjusted.)

Top flange splice controlling design stress, AASHTO Eq. 10-4b:
Top flange Fcu = ( 13.8 ksi +  50.0 ksi)/2 =  31.91 ksi
Or Top flange Fcu =  0.75 * 50.00 =  37.50 ksi (Governs)

Bot flange fncu =  13.82 ksi
Rcu = Fcu/fcu =  37.50 ksi/ 13.82 ksi =  2.71
Since stresses in top and bottom flanges are equal
the design stress for the bottom flange =  37.50 ksi

Design flange forces:
Top Flange Design Force =  37.50 ksi *  15.13 sq. in. =  567.6 k
Bot Flange Design Force =  37.50 ksi *  15.13 sq. in. =  567.6 k

Fatigue flange forces are computed by using the flange stresses due to M
and prorating by Msr divided by M:
Top Flange Fatigue Force =  13.82 ksi *  15.13 sq. in.
                              * ( 532.7 k*ft/ 776.6 k*ft)
                         =  143.4 k
Bot Flange Fatigue Force =  13.82 ksi *  15.13 sq. in.
                              * ( 532.7 k*ft/ 776.6 k*ft)
                         =  143.4 k

Overload flange forces are computed by using the flange stresses due
to 1.3*(DL + 5/3*LL) and dividing by 1.3:
Top Flange Overload Force =  13.82 ksi *  15.13 sq. in. / 1.3
                          =  160.9 k
Bot Flange Overload Force =  13.82 ksi *  15.13 sq. in. / 1.3
                          =  160.9 k

The design shear is determined by AASHTO 10.18.2.3.2:
Vu = 29.00 ksi *  33.97 in *  .759 in =   748.6 k
V =  172.2 k< 0.5Vu =  374.3 k
Vdesign = 1.5 * V           (Eq. 10-4i)
        = 1.5 *  172.2 k =  258.4 k

The web plates shall be designed for a moment of Mv = Vdesign * e,
where "e" is the eccentricity of the bolt group to centerline of splice.
Also, there is a moment and horizontal force given by AASHTO Eqs 10-4l & 4m:
Mweb = tw*D^2  /12*(R*Fcu - Rcu*fncu)
     =  .759 in*( 33.97 in)^2  /12*( 37.50 ksi +  2.71* 13.821 ksi)
     =  5481.3 k*in
Hweb = tw*D/2 *(R*Fcu + Rcu*fncu)
     =  .759 in*( 33.97 in)/2*( 37.50 ksi -  2.71* 13.82 ksi)
     =   0.0 k
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BRIDGE GIRDER FIELD SPLICE DESIGN - COMPUTE DESIGN MOMENT AND SHEAR (Cont'd)

Overload web moments are computed by using flange stresses due to 1.3*(DL + 5/3*LL) and dividing by 1.3:

```
Mweboverload =  .759 in * ( 33.97 in)^2  /12 *( 13.82 ksi +  13.82 ksi)/1.3
             =    1554.0 k*in
Hweboverload =  .759 in * ( 33.97 in)/2 * ( 13.82 ksi -  13.82 ksi)/1.3
             =     0.0 k
```

The portion of fatigue moment carried by the web is computed by using flange stresses due to M and prorating by Msr divided by M:

```
Mwebfatigue =  .759 in *( 33.97 in)^2  /12 *( 13.82 ksi +  13.82 ksi)
               * ( 532.7 k*ft /  776.6 k*ft)
            =  1385.6 k*in
Hwebfatigue =  .759 in *( 33.97 in)/2 * ( 13.82 ksi -  13.82 ksi)
               * ( 532.7 k*ft /  776.6 k*ft)
            =   0.0 k
```

FIRM:Your Firm
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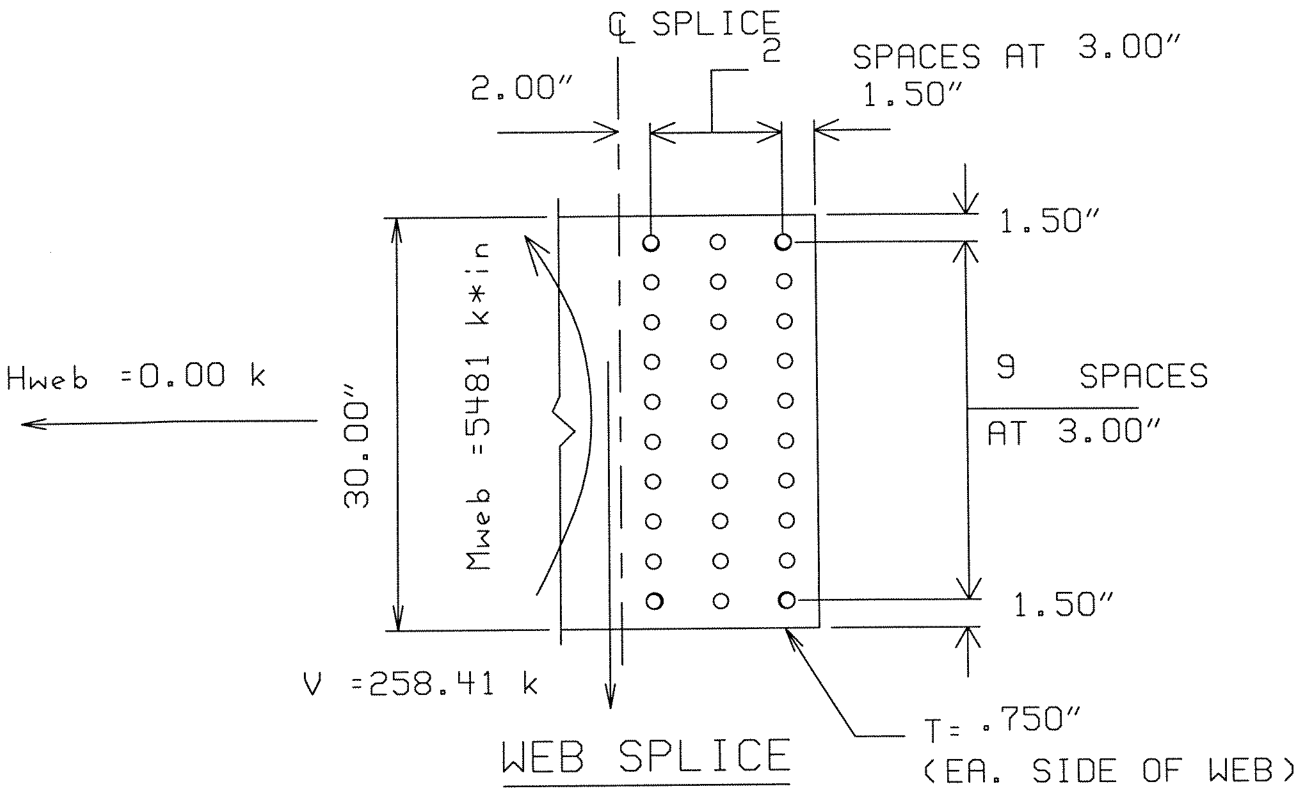
BRIDGE GIRDER FIELD SPLICE DESIGN - WEB SPLICE DESIGN



Compute moment of inertia of bolt group:

$Ix= 10 * [(3.00)^2 + (0.00)^2 + (3.00)^2]$
$= 180 \text{ bolt*in}^2$

$Iy= 3 * [(13.50)^2 + (10.50)^2 + (7.50)^2 + (4.50)^2 + (1.50)^2 + (1.50)^2 + (4.50)^2 + (7.50)^2 + (10.50)^2 + (13.50)^2]$
$= 2227 \text{ bolt*in}^2$

$Ip= Ix + Iy = 180 + 2227 = 2407 \text{ bolt*in}^2$

Moment at center of bolt group,
M= 5481 k*in + 258 k * 5.00 in = 6773 k*in

Bolt reactions, $Rx= 6773\text{k*in*}13.50 \text{ in}/ 2407 \text{ bolt*in}^2 + 0.0 \text{ k}/30 \text{ bolt}$
= 37.98 k/bolt
$Ry= 6773\text{k*in*} 3.00 \text{ in}/ 2407 \text{ bolt*in}^2 + 258.4\text{k}/30 \text{ bolt}$
= 17.05 k/bolt

Resultant, $R= [(37.98)^2 + (17.05)^2]^{.5} = 41.63 \text{ k}$

Bolt diameter= .875 in, Bolt area= .601 sq. in.

Bolt shear= 41.63 k/(2 * .601 sq.in.) = 34.61 ksi < 35.00 ksi (OK)

Bearing stress in splice plates,
= 41.63 k/(2 * .875 in * .750 in) = 31.72 ksi <117.00 ksi (OK)

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BRIDGE GIRDER FIELD SPLICE DESIGN - WEB SPLICE DESIGN (Cont'd)

Bearing stress in girder web plate,
= 41.63 k/(.875 in * .759 in) = 62.60 ksi <117.00 ksi (OK)

Splice plate gross moment of inertia,
Ig= 1/12 *(2* .75 in) * $(30.00\ \text{in})^3$ = 3375.00 in^4

Hole diameter = .8750 in. + 1/8 in. (for design purposes) = 1.0000 in

Moment of inertia of holes in splice plates,
Iholes= 2* .750 * 1.000 *[$(13.50)^2$ + $(10.50)^2$ + $(7.50)^2$
+ $(4.50)^2$ + $(1.50)^2$ + $(1.50)^2$ + $(4.50)^2$
+ $(7.50)^2$ + $(10.50)^2$ + $(13.50)^2$]
= 1113.75 in^4

Net moment of inertia of splice plates,
Inet= 3375.00 - 1113.75 = 2261.25 in^4

Gross area of splice = 2* .750in * 30.000in= 45.00 sq. in.

Splice PL combined bending stress and stress due to Hweb,
fb= 6773.39k*in* 15.00in/ 2261.25in^4 + 0.0 k/ 45.00 sq. in
= 44.93 ksi< 50.00 ksi (OK)

Splice PL shear stress,
fv = 258.41k/ 45.00 sq. in. = 5.74 ksi< 29.00 ksi (OK)

Check fatigue stress in splice plates using gross moment of inertia:
fsr= 1385 k*in * 15.00 in/ 3375 in^4 + 0.0 k/ 45.00 sq. in.
= 6.15 ksi < 18.00 ksi (OK)

Check overload stresses:

Overload moment at center of bolt group,
M= 1554 k*in + 132 k * 5.00 in = 2216 k*in

Overload splice PL combined bending stress plus stress due to Hweboverload,
fb= 2216.60k*in* 15.00in/ 2261.25in^4 + 0.0 k/ 45.00 sq/in
= 14.70 ksi < 40.0 ksi (OK)

Bolt reactions, Rx= 2216k*in*13.50 in/ 2407 bolt*in^2 + 0.0 k/30bolt
= 12.42 k/bolt
Ry= 2216k*in* 3.00 in/ 2407 bolt*in^2 + 132.5 k/30bolt
= 7.17 k/bolt

Resultant, R= $[(12.42)^2 + (7.17)^2]^{.5}$ = 14.35 k

Overload Bolt shear= 14.35 k/(2* .601 sq.in.)= 11.93 ksi < 21.0 ksi (OK)

BRIDGE GIRDER FIELD SPLICE DESIGN - WEB SPLICE DESIGN PER IDOT CRITERIA:

The following calculation uses the criteria set forth in the Illinois Department of Transportation's Memorandum to All Bridge Designers, 03.8, dated December 30, 2003.

The design shear force for LFD is,
Vuw = 0.58*Fy*D*tw =0.58 * 50 * 33.97 * .759 = 748.6 k
V web (ult) = 1.0 * Vuw = 1.0 * 748.6 k = 748.6 k
V web (service) = 748.6 k/1.25 = 598.9 k

The design moment for LFD is computed as follows:
M web (ult) = 5.00 in * 748.6 k= 3743.4 k*in
M web (service) = 3743.4 k*in/1.25 = 2994.7 k*in

Check web bolts for ultimate strength:
Bolt reactions, Rx= 3743k*in*13.50 in/ 2407 bolt*in^2
= 20.99 k/bolt
Ry= 3743k*in* 3.00 in/ 2407 bolt*in^2 + 748.6k/30 bolt
= 29.62 k/bolt
Resultant, R= $[(20.99)^2 + (29.62)^2]^{.5}$ = 36.30 k
Bolt shear (ult)= 36.30 k/(2 * .601 sq.in.) = 30.18 ksi < 35.00 ksi (OK)

Bearing stress in splice plates,
= 36.30 k/(2 * .875 in * .750 in) = 27.66 ksi <117.00 ksi (OK)

Bearing stress in girder web plate,
= 36.30 k/(.875 in * .759 in) = 54.59 ksi <117.00 ksi (OK)

Splice PL bending stress (using the net moment of inertia),
fb= 3743.49k*in* 15.00in/ 2261.25in^4
= 24.83 ksi< 50.00 ksi (OK)

Splice PL shear stress,
fv = 748.69k/ 45.00 sq. in. = 16.63 ksi< 29.00 ksi (OK)

Check web bolts for slip:
Bolt reactions, Rx= 2994k*in*13.50 in/ 2407 bolt*in^2
= 16.79 k/bolt
Ry= 2994k*in* 3.00 in/ 2407 bolt*in^2 + 598.9 k/30bolt
= 23.69 k/bolt
Resultant, R= $[(16.79)^2 + (23.69)^2]^{.5}$ = 29.04 k

Bolt shear = 29.04 k/(2* .601 sq.in.)= 24.15 ksi > 21.0 ksi (OK)

(Note: IDOT's procedure does not require a fatigue check.)

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MADE BY:KJH   DATE:06-03-2005   CHECKED BY: DATE:
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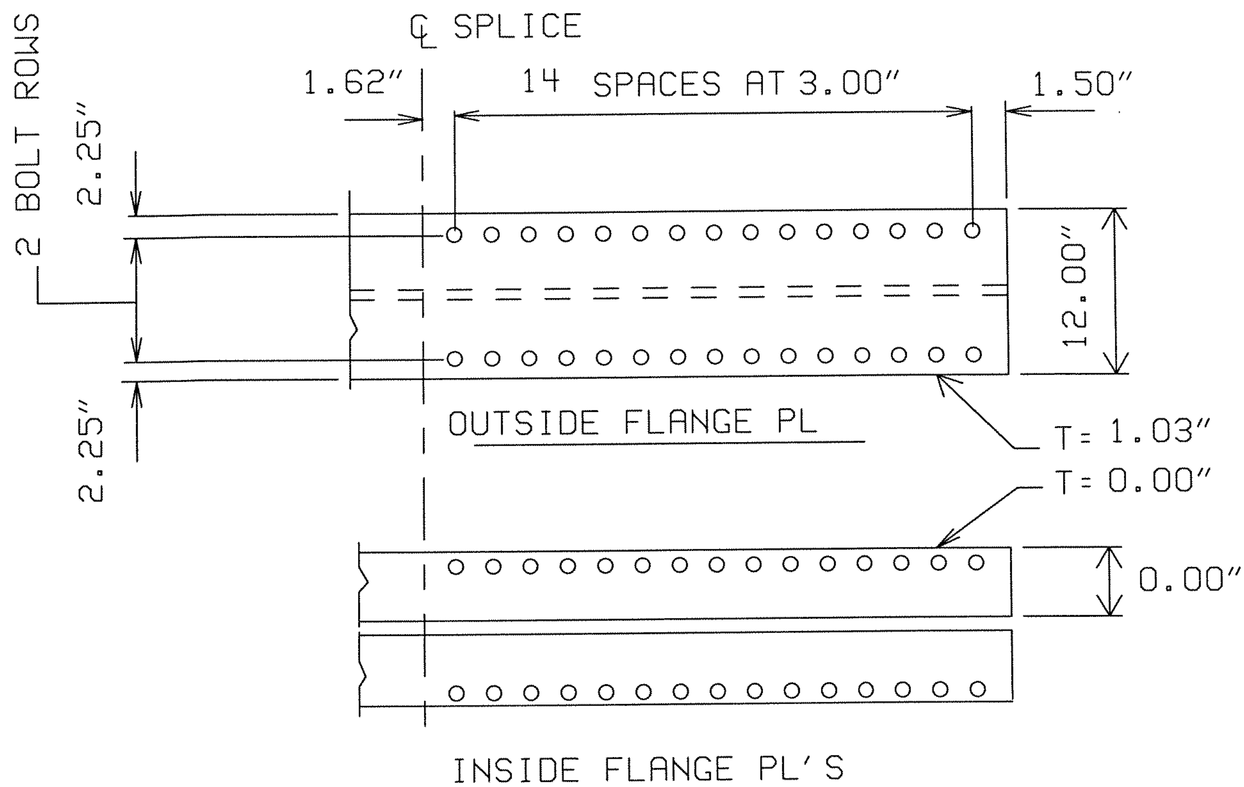
BRIDGE GIRDER FIELD SPLICE DESIGN - FLANGE SPLICE



Force in top flange =   567 k
Load per bolt =   567 k/ 30 bolts = 18.92 k/bolt

Flange bearing stress= 18.92 k per bolt/(1.250 in * .875 in)
                     = 17.29 ksi < 117.00 ksi (OK)

Compute net area of plates, deducting holes in excess of 15% of gross area:
Outside plate= 1.039 * (12.00 * 1.15 - 2 * 1.000)   = 12.27 sq.in.
Inside plates= 0.000 * (2* 0.00*1.15 - 2 * 1.000)   =   0.00 sq.in.

Total net area = 12.27 sq.in.

Splice stress= 567 k/ 12.27 sq.in.= 46.25 ksi

Splice fatigue stress = 143.4 k/ 12.27 sq. in
                      = 11.69 ksi < 18.00 ksi (OK)

Outside plate force= 567 k * (12.27 sq.in./ 12.27 sq.in.)= 567 k
Inside plate force = 567 k - 567 k =   0 k

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BRIDGE GIRDER FIELD SPLICE DESIGN  - FLANGE SPLICE
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Filler plates will not be greater than 1/4 inch, therefore
the allowable bolt shear does not need to be reduced (AASHTO 10.18.1.2).

Maximum bolt shear =  567 k/(  30 bolts *  .601 sq.in. per bolt)
                   =  31.46 ksi < 35.00 ksi (OK)

Bearing stress,
(Outside PL) = 567 k/( 30 bolts*1.039 in* .875 in)= 20.79 ksi<117.00 ksi(OK)

Check overload stresses:
-----------------------
Overload splice stress,
 fb =  160.9 k / 12.27 sq. in. =  13.11 ksi <=  40.0 ksi (OK)

Overload forces in splice plate,
(Outside plate) =  160.9 k *  12.2 sq. in/ 12.2 sq. in = 160.9 k
(Inside plate ) =  160.9 k -  160.9 k =    0.0 k

Overload bolt shear at outside splice plate governs,
 fv =  160.9 k /(30 bolts *  .601 sq. in)=   8.9 ksi <=  21.0 ksi (OK)
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BRIDGE GIRDER FIELD SPLICE DESIGN  - FLANGE SPLICE DESIGN PER IDOT:
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The procedure used by the IDOT Memo 03.8 designs for 100% of the capacity
of the effective area (Ae) of the flange.

Compute LFD effective area of flange:

Ae = Wn * t + Beta*Ag < Ag
Wn = 12.10 -  2 *  1.00 (holes)
   = 10.10 inches (net width)
t = 1.250 in,    Beta = 0.15
Ag =  1.25 in * 12.10 in =  15.13 sq. in.
Ae = 10.10 * 1.250 + 0.15* 15.13 =  14.90 sq. in.< Ag

Compute flange splice plate design force:

P flange (ult) = 14.90 sq. in. * 50 ksi =  745.4 k
P flange (service) =  745.4 k/1.25 =  596.3 k

Maximum bolt shear =  745 k/(  30 bolts *  .601 sq.in. per bolt)
                   =  41.32 ksi > 35.00 ksi (NG)

Bearing stress,
(Outside PL) = 745 k/( 30 bolts*1.039 in* .875 in)= 27.30 ksi<117.00 ksi(OK)

Check flange bolts for slip:

Overload forces in splice plate,
(Outside plate) =  596.3 k *  12.2 sq. in/ 12.2 sq. in = 596.3 k
(Inside plate ) =  596.3 k _   596.3 k =    0.0 k

Overload bolt shear at outside splice plate governs,
 fv =  596.3 k /(30 bolts *  .601 sq. in)=  33.0 ksi >  21.0 ksi (NG)

(Note: IDOT's procedure does not use a bolt shear capacity
reduction factor for filler plates (AASHTO 10.18.2) nor is a
fatigue check required.)
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