FORM DQP 2.01-1
LEVEL 1 CHECK PRINT SIGN-OFF SHEET

Client Name: **Ohio Department of Transportation**

Job Title: **Cleveland Innerbelt Design-Build Contract**

Job Number: **CUY-90-14.90**

Document Title: RFI 00358 - Type A Floorbeam connection check

Check Level (Mark One):

- [ ] 1A 100% Document Check
- [X] 1B 100% Input Check

Enter description below:

		Print Name	Signature	Date
X	Originator	SARAH LARSON	Sarah Larsen	7-13-12
X	Checker	Jeff Wheeler	[signature]	7-17-12
X	Backchecker	SARAH LARSON	Sarah Larsen	7-17-12
X	Updater	SARAH LARSON	Sarah Larsen	7-17-12
X	Validater	Jeff Wheeler	[signature]	7-17-12

Insert an "X" in the box to indicate a required QC activity.

For CLEVELAND	Job no. 49633	Sheet no.
Made by SJL	Checked by JCW	Backchecked by SJL
Date 7-13-12	Date 7-17-12	Date 7-17-12

HNTB

CLEVELAND INNERBELT
UNIT 2 - STRUCTURAL STEEL
RFI 00358

→ While drilling the bolt pattern in the top connection plate of a Type A1 diaphragm, the drill bit "walked" 1/8" resulting in a hole elongated by 1/8".

→ It is not known in which direction the elongation occured.

→ Excel spreadsheets used in the diaphragm design have been updated to evaluate the connection for bolt shear strength and slip considering the bolt in the elongated hole is missing. In addition, an Excel spreadsheet from design has been updated that checks the connection plate considering an additional 1/4" of material has been taken from the elongated hole.

→ In addition to capacity checks, geometry checks must be considered:

- minimum Edge Distance

From AASHTO Tbl. 6.13.2.6.6-1

Min. Edge Dist. = 1.75"

From ODOT BDM

Min. Edge Dist. = 1.75" + 0.25" = 2.0"

However, the ODOT BDM states that the 0.25" increase is to allow for fabrication tolerances. Therefore since

2" - 0.125" = 1.875" > 1.75"

→ Say OK

- minimum Bolt spacing

From AASHTO 6.13.2.6

min. spacing = $3d_b$ = 3(1") = 3"

min. clear dist. = $2d_b$ = 2(1") = 2"

Spacing provided:

Vertical = 3.875"

Horizontal = 3.25" ← Controls.

min. spacing = 3.25" - 1/8"

= 3.125" > 3" ⇒ OK

min. clear = 3.25" - 1.0625" - 0.125"

= 2 1/16" $\cancel{<}$ 2.0"

→ OK

ME101-1204

302.4.1.12 BASELINE REQUIREMENTS FOR CURVED AND DOG-LEGGED STEEL STRUCTURES

CMS 513 requires the fabricator to include in the shop drawings an overall layout with dimensions showing the horizontal position of beam or girder segments with respect to a full-length base or workline. Offsets from this full-length base line are to be provided by the fabricator for each 10 feet [3000 mm] of length. The designer shall provide this baseline in the plans along with enough information for the fabricator to be able to readily calculate the required offsets. The requirement for this information is especially critical on structures located on a curve or spiral or having other complex geometry.

302.4.1.13 INTERMEDIATE EXPANSION DEVICES

Intermediate expansion devices for a structure, if required, shall be located over a pier and the structural members shall be designed to be discontinuous at that pier.

302.4.1.14 BOLTED SPLICES

For galvanized structures, the bolt hole size requires a 1/16 inch [1.5 mm] increase over a standard hole size to allow for the additional thickness of the zinc coating. This increase in hole size decreases the splice capacity.

Bolt allowable stresses for painted surfaces or unpainted weathering steel surfaces shall be based on AASHTO 's values for Class A, Contact Surface, Standard Hole Type.

Bolt allowable stressed for metallized surfaces shall be based on AASHTO's values for Class C, Contact Surface, Standard Hole Type.

Bolt allowable stressed for galvanized surfaces shall be based on AASHTO's values for Class C, Contact Surface, Oversized Hole Type.

Beams having bolted splices at bend points shall have additional details incorporated in the plans to completely detail the joint requirements. The minimum edge distances specified in AASHTO shall be provided at the edges of all main members and splice plates.

For splices at bend points the lines of holes in the beam or girder flanges should be parallel to the centerline of the web. If the bend angle is small enough use rectangular splice plates (splice plates should not overhang flange by more than ½ inch [13 mm] and inside splice plates should not have to be trimmed to clear web or web to flange radius). When the angle is too large to allow rectangular splice plates the plates should be trimmed to align with the flange edges. In either case minimum edge distances shall be met.

Bolted compression splices, such as in a column, while designed as a friction type connection, also require the ends of the spliced members to be in full bearing by milling of the ends. For compression splice members with milled ends the requirements of *LRFD 6.13.6.1.3* shall be met.

The designer should recognize that "FULL BEARING" of beams and girders is not defined by AASHTO. "FULL BEARING" has been generally defined by ODOT as 75 percent of the bearing surface in contact and the other 25 percent with no gap greater than 1/32 inch [0.8 mm]. The designer shall specify the required fit definition when designing in conformance to the AASHTO design requirements for bolted splices in compression members.

Refer to Figure 302.4.1.14-1 for additional bolted splice details.

302.4.1.14.a BOLTS

Field splices in beams and girders shall be bolted connections using high strength bolts, ASTM A325[M].

The designer shall specify the diameter of the bolts and check that the type (Type I for Galvanized or Type III for Weathering) of A325[M] bolts is described in the coating notes or bolt material specifications.

Coating systems that are zinc based, such as OZEU, IZEU, Galvanizing or Metallizing require galvanized Type I bolts.

Un-coated weathering steel structures shall have A325[M], Type III bolts. If the faying surfaces under both the head and nut of every bolt of a weathering steel member are coated, specify galvanized A325[M] Type I bolts. Otherwise, specify A325[M], Type III bolts.

Generally, bolted splices should be designed using 1 inch [25 mm] or 1⅛ inch [29 mm] diameter bolts. No metric bolts or studs are available in the small quantities required for bridges.

302.4.1.14.b EDGE DISTANCES

The edge distances provided in *LRFD 6.13.2.6.6* are absolute minimums allowed during fabrication. For design and detailing purposes, 0.25 in. shall be added to the minimum edge distances listed in *Table 6.13.2.6.6-1.*

This increase will allow for fabrication tolerances when drilling bolt holes in splice plates, especially the inside flange splice plates.

302.4.1.14.c LOCATION OF FIELD SPLICES

Generally bolted splices should be located at points of dead load contraflexure on a continuous structure. Splices may also be supplied to help meet shipping and handling limitations. Plans should show optional field splice locations.

HNTB

For	Innerbelt Bridge	Job no.	49633	Sheet no.	
Made by	PDB	Checked by	SJL	Backchecked by	PDB
Date	9/8/2011	Date	9/12/2011	Date	9/12/2011
Revised by	SJL	Checked by	JCW	Backchecked by	SJL
Date	7/13/2012	Date	7/17/2012	Date	7/17/2012

Revised for RFI 358

Floorbeam Type A, 3'-7 1/2" Kneebrace

Bolt Cap : 28.7 kips
57.4 kips - double shear

Mg = -1086 ft-kips
V = 138 kips
P = -71 kips

eh = 3.63 in
ev = 20.3 in

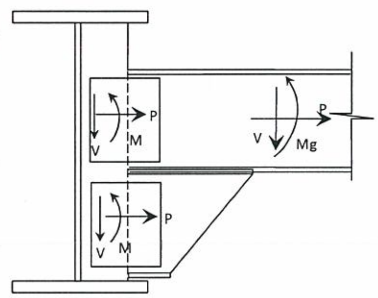
Veh = 41.6 ft-kips
Pev = -120 ft-kips
Mc = Mg-Veh-Pev = -1007 ft-kips

bolt	row	column	x	y	x-xb	y-yb	$(x-xb)^2$	$(y-yb)^2$	rv	rh	r
1	1	1	0.0	3.17	-1.63	-32.8	2.64	1075	4.29	43.0	43.2
2	2	1	0.0	9.50	-1.63	-26.5	2.64	700	4.29	35.3	35.6
3	3	1	0.0	15.84	-1.63	-20.1	2.64	405	4.29	27.6	28.0
4	4	1	0.0	22.17	-1.63	-13.8	2.64	190	4.29	20.0	20.4
5	5	1	0.0	28.50	-1.63	-7.5	2.64	56	4.29	12.3	13.0
6	6	1	0.0	34.84	-1.63	-1.1	2.64	1	4.29	4.6	6.3
7	7	1	0.0	47.00	-1.63	11.0	2.64	122	4.29	-10.2	11.0
8	8	1	0.0	50.88	-1.63	14.9	2.64	223	4.29	-14.9	15.5
9	9	1	0.0	54.75	-1.63	18.8	2.64	353	4.29	-19.6	20.0
10	10	1	0.0	58.63	-1.63	22.7	2.64	514	4.29	-24.3	24.6
12	12	1	0.0	66.38	-1.63	30.4	2.64	925	4.29	-33.7	33.9
13	1	2	3.25	0.00	1.63	-36.0	2.64	1293	8.23	46.8	47.6
14	2	2	3.25	6.33	1.63	-29.6	2.64	878	8.23	39.2	40.0
15	3	2	3.25	12.67	1.63	-23.3	2.64	542	8.23	31.5	32.5
17	5	2	3.25	25.34	1.63	-10.6	2.64	113	8.23	16.1	18.1
18	6	2	3.25	31.67	1.63	-4.3	2.64	18	8.23	8.4	11.8
19	7	2	3.25	38.00	1.63	2.0	2.64	4	8.23	0.7	8.3
20	8	2	3.25	47.00	1.63	11.0	2.64	122	8.23	-10.2	13.1
21	9	2	3.25	50.88	1.63	14.9	2.64	223	8.23	-14.9	17.0
23	11	2	3.25	58.63	1.63	22.7	2.64	514	8.23	-24.3	25.6
24	12	2	3.25	62.50	1.63	26.5	2.64	705	8.23	-29.0	30.1
25	13	2	3.25	66.38	1.63	30.4	2.64	925	8.23	-33.7	34.7
	Bot		19.5	228							
	Top		16.3	563							
	Total		36	791			58	9,902			

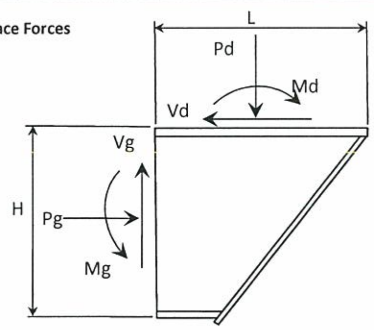
rv : vert comp of bolt force
rh : horiz comp of bolt force

	Bot	Top	Total
n	12	10	22
xb	1.63	1.63	1.63
yb	19.0	56.3	36.0
Ip	1,857	3686	9,960
rp	23.8	-21.5	
rh-rp	23.0	-12.2	
P	286	-215	71
V	75.1	62.6	138
M	-2253	-4470	-12,974

max r = 47.6 kips
D/C = 0.83



Knee Brace Forces



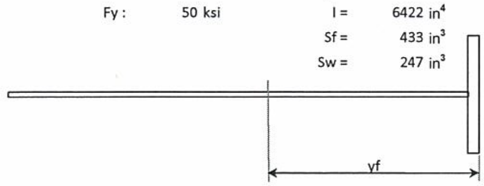
Positive Forces Shown

L : 40 in
H : 43.5 in
Pg = 286 kips
Vg = 75.1 kips
Mg = 2253 in-kips
Pd = Vg = 75.1 kips
Vd = Pg = 286 kips
Md = Mg+PgH/2+VgL/2 = 6,961 in-kips

Knee Brace Web

t : 0.625 in	@ Diaph	Vr =	725 kips	D/C =	0.39
	@ Gdr	Vr =	788 kips	D/C =	0.10

@ Diaph	bf :	12 in	A =	36 in^2	fa =	-2.11 ksi	axial stress		
	tf :	0.875 in	yf =	14.8 in	f_{bf} =	-16.1 ksi	bending stress in flange		
	Fy :	50 ksi	I =	6422 in^4	f_{bw} =	28.2 ksi	bending stress in web		D/C
			Sf =	433 in^3	f_f =	-18.2 ksi	Total flange stress		0.36
			Sw =	247 in^3	f_w =	26.1 ksi	Total web stress		0.52



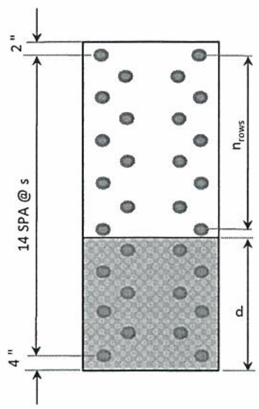
@ Girder	bf :	0 in	A =	27.2 in^2	fa =	-10.5 ksi
	tf :	0 in	yb =	21.8 in	fb =	11.4 ksi
			I =	4,287 in^4		
			S =	197 in^3		

Kneebrace Connection PL - Web Weld

Double Fillet Size :	0.3125 in	Fexx :	70 ksi	ϕ_{e2} :	0.80		
Rr =	14.8 k/in	L =	40 in	Rr =	594 kips	D/C =	0.48

\\kcow00\Jobs\49633\Bridges\Design\Final Design\Unit 2\Excel\Diaphragms\final forces\LateralBracingRevision\RFI calcs\[FloorbeamConns 071312.xlsx]A-S Min M Revd

Knee Brace - Diaphragm Connection



s : 3.75 in

row	h	d	d^2
1	$\boxed{41.75}$	35.32	1,247
2	38.00	31.57	996
3	34.25	27.82	774
4	30.50	24.07	579
5	26.75	20.32	413
6	23.00	16.57	274
7	19.25	12.82	164
8	15.50	9.07	82
9	11.75	5.32	28
10	8.00	1.57	2
11	2.50	0.00	0

bolt D :	1 in
Ab =	0.785 in^2
Fub :	120 ksi
w :	14 in
d :	$\boxed{6.43}$ in
Ay =	290 in^3
n_{rows} =	10
n :	22
y =	18.4 in
Aby =	290 in^3
A =	106 in^2
I =	8407 in^4
St =	238 in^3
Sc =	1307 in^3

fa =	-0.71 ksi
fb =	29.2 ksi
Max Bolt Stress =	28.5 ksi
Max Bolt Tension, Tu =	22.4 kips
Bolt Shear, Pu =	13.0 kips
Bolt Shear Cap, $\phi_s R_n$:	36.0 kips
ϕ_t :	0.80
Bolt Tensile Cap, $\phi_t T_n$ =	53.4 kips
D/C =	0.42

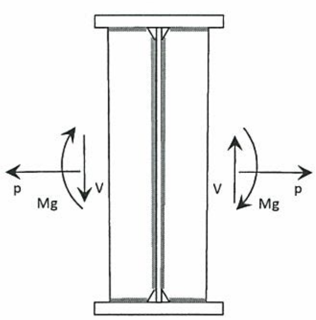
Girder connection PL

At interior girders, most of the floorbeam moment will be transferred directly from one floobeam to the next.

		Conn PL	
Girder Depth, H :	8 ft	b :	12 in
F = M/H =	100 kips	t :	1.125 in
L :	2 ft	Fy :	50 ksi
M = FL/2 =	100 ft-kips	tw :	1 in
		beff :	$\boxed{9}$ tw = 9 in

Conn PL Weld Capacity

Double Fillet Size :	0.3125 in	Fexx =	70 ksi	ϕ_{e2} :	0.80	Rr =	14.8 k/in



Ls =	11 in	L =	114 in
Lw =	92 in	I =	90,235 in^3
horiz clip :	1 in	Ss =	1,880 in^2 - of Stiff weld
vert clip :	2 in	Sw =	1,962 in^2 - of web weld

M =	-1086 ft-kips
V =	138 kips

max. stiff weld shear due to M =	6.9 k/in	Stiff Weld D/C =	0.47
max. web weld shear due to M =	6.6 k/in		
max. web weld shear due to V =	1.5 k/in		
max resultant weld shear =	6.8 k/in	Web Weld D/C =	0.46

HNTB

For	Innerbelt Bridge	Job no.	49633	Sheet no.	
Made by	PDB	Checked by	SJL	Backchecked by	PDB
Date	9/8/2011	Date	9/12/2011	Date	9/12/2011
Revised by	SJL	Checked by	JCW	Backchecked by	SJL
Date	7/13/2012	Date	7/17/2012	Date	7/17/2012

Revised for RFI 358

Floorbeam Type A, 3'-7 1/2" Kneebrace

Bolt Cap : $\boxed{28.7}$ kips
57.4 kips - double shear

Mg = 1107 ft-kips
V = 42 kips
P = -8 kips

eh = 3.63 in
ev = 20.3 in

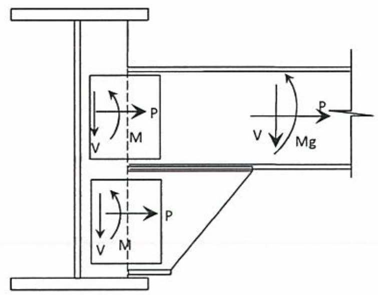
Veh = 12.8 ft-kips
Pev = -14 ft-kips
Mc = Mg-Veh-Pev = 1108 ft-kips

bolt	row	column	x	y	x-xb	y-yb	$(x-xb)^2$	$(y-yb)^2$	rv	rh	r
1	1	1	0.0	3.17	-1.63	-32.8	2.64	1075	4.09	-43.4	43.6
2	2	1	0.0	9.50	-1.63	-26.5	2.64	700	4.09	-34.9	35.2
3	3	1	0.0	15.84	-1.63	-20.1	2.64	405	4.09	-26.5	26.8
4	4	1	0.0	22.17	-1.63	-13.8	2.64	190	4.09	-18.0	18.5
5	5	1	0.0	28.50	-1.63	-7.5	2.64	56	4.09	-9.6	10.4
6	6	1	0.0	34.84	-1.63	-1.1	2.64	1	4.09	-1.1	4.2
7	7	1	0.0	47.00	-1.63	11.0	2.64	122	4.09	15.1	15.7
8	8	1	0.0	50.88	-1.63	14.9	2.64	223	4.09	20.3	20.7
9	9	1	0.0	54.75	-1.63	18.8	2.64	353	4.09	25.5	25.8
10	10	1	0.0	58.63	-1.63	22.7	2.64	514	4.09	30.6	30.9
12	12	1	0.0	66.38	-1.63	30.4	2.64	925	4.09	41.0	41.2
13	1	2	3.25	0.00	1.63	-36.0	2.64	1293	-0.25	-47.6	47.6
14	2	2	3.25	6.33	1.63	-29.6	2.64	878	-0.25	-39.2	39.2
15	3	2	3.25	12.67	1.63	-23.3	2.64	542	-0.25	-30.7	30.7
17	5	2	3.25	25.34	1.63	-10.6	2.64	113	-0.25	-13.8	13.8
18	6	2	3.25	31.67	1.63	-4.3	2.64	18	-0.25	-5.4	5.4
19	7	2	3.25	38.00	1.63	2.0	2.64	4	-0.25	3.1	3.1
20	8	2	3.25	47.00	1.63	11.0	2.64	122	-0.25	15.1	15.1
21	9	2	3.25	50.88	1.63	14.9	2.64	223	-0.25	20.3	20.3
23	11	2	3.25	58.63	1.63	22.7	2.64	514	-0.25	30.6	30.6
24	12	2	3.25	62.50	1.63	26.5	2.64	705	-0.25	35.8	35.8
25	13	2	3.25	66.38	1.63	30.4	2.64	925	-0.25	41.0	41.0
	Bot		19.5	228							
	Top		16.3	563							
	Total		36	791			58	9,902			

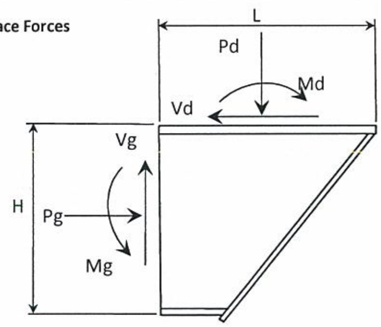
rv : vert comp of bolt force
rh : horiz comp of bolt force

	Bot	Top	Total
n	12	10	22
xb	1.63	1.63	1.63
yb	19.0	56.3	36.0
Ip	1,857	3686	9,960
rp	-22.3	27.5	
rh-rp	-25.4	13.4	
P	-267	275	8
V	23.1	19.2	42
M	2479	4919	13,193

max r = 47.6 kips
D/C = 0.83



Knee Brace Forces



Positive Forces Shown

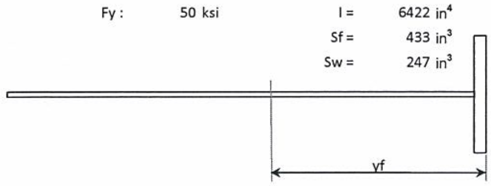
L : $\boxed{40}$ in
H : $\boxed{43.5}$ in
Pg = -267 kips
Vg = 23.1 kips
Mg = -2479 in-kips
Pd = Vg = 23.1 kips
Vd = Pg = -267 kips
Md = Mg+PgH/2+VgL/2 = -8,752 in-kips

Knee Brace Web

t : 0.625 in

@ Diaph	Vr =	725 kips	D/C =	0.37
@ Gdr	Vr =	788 kips	D/C =	0.03

@ Diaph	bf : 12 in	A =	36 in^2	fa =	-0.65 ksi	axial stress	
	tf : 0.875 in	yf =	14.8 in	f_{bf} =	20.2 ksi	bending stress in flange	
	Fy : 50 ksi	I =	6422 in^4	f_{bw} =	-35.5 ksi	bending stress in web	D/C
		Sf =	433 in^3	f_f =	19.6 ksi	Total flange stress	0.39
		Sw =	247 in^3	f_w =	-36.1 ksi	Total web stress	0.72



@ Girder	bf : 0 in	A =	27.2 in^2	fa =	9.8 ksi
	tf : 0 in	yb =	21.8 in	fb =	-12.6 ksi
		I =	4,287 in^4		
		S =	197 in^3		

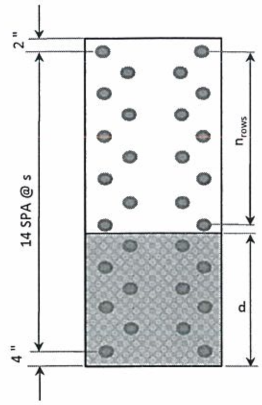
Kneebrace Connection PL - Web Weld

Double Fillet Size : 0.3125 in   Fexx : 70 ksi ϕ_{e2} : 0.80

Rr = 14.8 k/in   L = 40 in   Rr = 594 kips   D/C = -0.45

\\kcow00\Jobs\49633\Bridges\Design\Final Design\Unit 2\Excel\Diaphragms\final forces\LateralBracingRevision\RFI calcs\[FloorbeamConns 071312.xlsx]A-S Max M Revd

Knee Brace - Diaphragm Connection



s : 3.75 in

row	h	d	d^2
1	$\boxed{41.75}$	35.32	1,247
2	36.25	29.82	889
3	32.50	26.07	679
4	28.75	22.32	498
5	25.00	18.57	345
6	21.25	14.82	220
7	17.50	11.07	122
8	13.75	7.32	54
9	10.00	3.57	13
10	6.25	0.00	0
11	2.50	0.00	0

fa =	-0.22 ksi
fb =	40.5 ksi
Max Bolt Stress =	40.3 ksi
Max Bolt Tension, Tu =	31.6 kips
Bolt Shear Cap, $\phi_s R_n$:	36.0 kips
ϕ_t :	0.80
Bolt Tensile Cap, $\phi_t T_n$ =	53.9 kips

bolt D :	1 in
Ab =	0.785 in^2
Fub :	120 ksi
w :	14 in
d :	$\boxed{6.43}$ in
Ay =	290 in^3
n_{rows} =	10
n :	22
y =	18.4 in
Aby =	290 in^3
A =	106 in^2
I =	7631 in^4
St =	216 in^3
Sc =	1186 in^3
Bolt Shear, Pu =	12.1 kips

D/C = 0.59

Girder connection PL

At interior girders, most of the floorbeam moment will be transferred directly from one floobeam to the next.

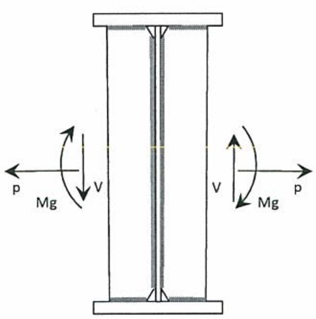
Girder Depth, H :	8.2 ft
F = M/H =	100 kips
L :	2 ft
M = FL/2 =	100 ft-kips

Conn PL

b :	12 in
t :	1.125 in
Fy :	50 ksi
tw :	1 in
beff :	$\boxed{9}$ tw = 9 in

Conn PL Weld Capacity

Double Fillet Size : 0.3125 in   Fexx = 70 ksi ϕ_{e2} : 0.80   Rr = 14.8 k/in



Ls =	11 in
Lw =	94.4 in
horiz clip :	1 in
vert clip :	2 in

L =	116.4 in
I =	96,730 in^3
Ss =	1,966 in^2 - of Stiff weld
Sw =	2,049 in^2 - of web weld

M = 1107 ft-kips
V = 42 kips

max. stiff weld shear due to M =	6.8 k/in	Stiff Weld D/C =	0.46
max. web weld shear due to M =	6.5 k/in		
max. web weld shear due to V =	0.4 k/in		
max resultant weld shear =	6.5 k/in	Web Weld D/C =	0.44

HNTB The HNTB Companies Engineers Architects	Made	SJL	Date	7/13/2012	Job Number	49633
	Checked	JCW	Date	7/17/2012		
For Cleveland Innerbelt	Backchk'd	SJL	Date	7/17/2012	Sheet No.	

(TYPE A)
1-Stringer Standard Diaphragm Connection
Unit 2 (Lateral Bracing Revision)

Connection Geometry

n_{col} = 3
s_{col} = 3.25 in
s_{row} = 3.875 in
gap = 0.50 in
d_{edge} = 2.00 in

n_{bolt} = 22
I_x = 9901.6 in^2
I_y = 58.1 in^2
I_p = 9959.7 in^2
eh = 1.63 in
ev = 20.73 in
x_{max} = 1.63 in
y_{max} = 35.96 in
r_{max} = 36.0 in

Bolt Capacity

Slip Resistance (6.13.2.8)

$R_n = K_h*K_s*N_s*P_t$

K_h = 0.85 — Oversize holes (Table 6.13.2.8-2)
K_s = 0.5 — Class B Surface (Table 6.13.2.8-3)
N_s = 2
P_t = 51 kip — (Table 6.13.2.8-1)
ϕ_s = 1.00 — (6.13.2.2)

ϕR_n = 43.35

Factored Member Forces

Service II

P_u = -6.9 kip
M_u = 788.3 kip-ft
V_u = -27.5 kip

Bolt Forces

Service II

Total $M = M_u + V_u*e$ = 9702 kip-in
$M*y_{max}/I_p$ = 35.0 kip
$F_x = P_u/n_{bolt}$ = 0.3 kip
F_x = 35.3 kip
$M*x_{max}/I_p$ = 1.6 kip
$F_y = V_u/n_{bolt}$ = 1.3 kip
F_y = 2.8 kip
Resultant F = 35.5 kip

35.5 kip < 43.4 kip, OK

1-Stringer Standard Diaphragm Connection

Ip calculation

knee-brace bolt row = 3.1670 in

	column 1	column 2	column 3	distance from centroid (dy)	(from centroid) dx	Ixi	Iyi				$y_{bar}i$
row 1	1	1		30.42		1850.9	5.3	19.375	9	38.004	66.379
row 2		1		26.547		704.7	2.6	15.50	9	38.004	62.504
row 3	1	1		22.672		1028.0	5.3	11.63	9	38.004	58.629
row 4	1			18.797		353.3	2.6	7.75	9	38.004	54.754
row 5	1	1		14.922		445.3	5.3	3.88	9	38.004	50.879
row 6	1	1		11.047		244.1	5.3	0.00	9	38.004	47.004
row 7		1		2.047		4.2	2.6			38.004	38.004
row 8	1			-1.120		1.3	2.6			34.84	34.837
row 9		1		-4.287		18.4	2.6			31.67	31.67
row 10	1			-7.454		55.6	2.6			28.50	28.503
row 11		1		-10.621		112.8	2.6			25.34	25.336
row 12	1			-13.788		190.1	2.6			22.17	22.169
row 13				-16.955		0.0	0.0			19.00	19.002
row 14	1			-20.122		404.9	2.6			15.84	15.835
row 15		1		-23.289		542.4	2.6			12.67	12.668
row 16	1			-26.456		699.9	2.6			9.50	9.501
row 17		1		-29.623		877.5	2.6			6.33	6.334
row 18	1			-32.790		1075.2	2.6			3.17	3.167
row 19		1		-35.957		1292.9	2.6			0.00	-1.07E-14
row 20						0.0	0.0				0
row 21						0.0	0.0				0.00
row 22						0.0	0.0				0.00
row 23						0.0	0.0				0.00
row 24						0.0	0.0				0.00
row 25						0.0	0.0				0.00
row 26						0.0	0.0				0.00
row 27						0.0	0.0				
row 28						0.0	0.0				

Total # bolts = 22 | 9901.6 | 58.1 | $\Sigma y_{bar}i*ni/n=$ 35.957455

Ip = 9959.7

xbar= 1.625 in

HNTB The HNTB Companies	Made	SJL	Date	7/13/2012	Job Number	49633
	Checked	JCW	Date	7/17/2012		
For Cleveland Innerbelt - Unit 2	Backchk'd	SJL	Date	7/17/2012	Sheet No.	

1-Stringer Standard Diaphragm Design
Connection Plate (RFI 358)

(AASHTO 6.8.2 and 6.13.4)

Check for yielding in gross section, fracture in net section
(Block Shear does not apply)

Vertical Force =	148.0	kips
Horizontal Force =	153.0	kips
Resultant factored force =	212.87	kips
Yielding in gross section, Φ_y =	0.95	
Fracture in net section, Φu =	0.80	
Gusset Plate thickness =	0.500	in
Bolt hole dia. =	1.2500	in
Rows of Bolts =	6	
Columns of Bolts =	2	
Gusset Plate width =	7.25	in
Gusset Plate Depth =	23.375	in
F_y =	50.0	ksi
Fu =	65.0	ksi

Gusset Plate

A_{gt} = 11.69 in^2

(modified for erection bolt, 1", and RFI 358, 0.25") A_{nt} = 7.31 in^2

Yielding ΦP_g = 555.16 kips

Fracture ΦP_n = 380.25 kips

OK

Check Tension and Bending on connection plate

Horizontal Force (ref. conn. Design) = 76.5 kips

Moment (ref. conn. Design) = 9714.0 in-kips

Yielding

$$\frac{P}{A_g}+\frac{M}{S_g}\le \phi_y F_y$$

S_g = 374.80 in^3

A_g = 30.94 in^2

Rupture

$$\frac{P}{A_{net}}+\frac{M}{S_{net}}\le \phi_u F_u$$

S_{net} = 273.28 in^3

A_{net} = 22.19 in^2

(from Section Properties spreadsheet)

Gross Section Stress

σ_{gross} = 28.4 ksi

$\Phi_y F_y$ = 47.5 ksi

Gross Section OK

Net Section Stress

σ_{net} = 39.0 ksi

$\Phi_u F_u$ = 52.0 ksi

Net Section OK

Check for Bearing Resistance at Bolt Holes

For bolts spaced at a clear distance between holes not less than 2.0d and with a clear end distance not less than 2.0 d:

Bolt Capacity = $\Phi R_n = \Phi\ 2.4dF_u t$ (eqn. 6.13.2.9-1)

If either the clear distance between holes is less than 2.0d, or the clear end distance is less than 2.0d:

Bolt Capacity = $\Phi R_n = \Phi\ 1.2L_c F_u t$ (eqn. 6.13.2.9-2)

Φ =	0.80	
F_u =	65	(Grade 50, AASHTO 6.4.1-1)
t =	1.000	in (5/8" plate each side)
d =	1.000	in
End clear spacing =	1.13	in*
clear Bolt spacing =	1.75	in*
L_c=	1.13	(shear planes)
ΦR_n =	70.20	kips/bolt

Max Resultant force on bolt = 56.20

Check Bearing at Bolt Holes = **OK**

* modified for RFI 358 assuming additional 0.25" of hole width

HNTB The HNTB Companies	Made	SJL	Date	7/13/2012	Job Number	49633
	Checked	JCW	Date	7/17/2012		
For **Cleveland Innerbelt - Unit 2**	Backchk'd	SJL	Date	7/17/2012	Sheet No.	

\\kcow00\Jobs\49633\Bridges\Design\Final Design\Unit 2\Excel\Diaphragms\final forces\LateralBracingRevision\RFI calcs\[Diaphragm_connections_071312.xlsx]1-Str Std Snet

1-Stringer Standard Diaphragm - Connection Plate S_{net}

Section Properties (for RFI 358)

	b	h	y_{bar}		Area	$b*h*y_{bar}$	dy	$A(dy)^2$	Ixi
	in	in	in		in2	in3	in	in4	in4
	0.5	1.375	66.1875	1	0.69	45.503906	35.20	851.7	0.11
1	0.5	2.5	63	2	1.25	78.75	32.01	1280.7	0.65
2	0.5	2.5	59	3	1.25	73.75	28.01	980.6	0.65
3	0.5	2	55.5	4	1.00	55.5	24.51	600.7	0.33
4	0.5	2	51	5	1.00	51	20.01	400.3	0.33
5	0.5	2.625	47.4375	6	1.31	62.261719	16.45	355.0	0.75
	0.5	1.375	44.1875	7	0.69	30.378906	13.20	119.7	0.11
					0.00	0	-30.99	0.0	0.00
					0.00	0	-30.99	0.0	0.00
					0.00	0	-30.99	0.0	0.00
					0.00	0	-30.99	0.0	0.00
					0.00	0	-30.99	0.0	0.00
					0.00	0	-30.99	0.0	0.00
					0.00	0	-30.99	0.0	0.00
					0.00	0	-30.99	0.0	0.00
					0.00	0	-30.99	0.0	0.00
					0.00	0	-30.99	0.0	0.00
1	0.5	1.375	37.8125		0.69	25.996094	6.82	32.0	0.11
2	0.5	5.65	33.05		2.83	93.36625	2.06	12.0	7.52
3	0.5	5.65	26.15		2.83	73.87375	-4.84	66.2	7.52
4	0.5	5.15	19.5		2.58	50.2125	-11.49	340.0	5.69
5	0.5	5.15	12.1		2.58	31.1575	-18.89	919.0	5.69
6	0.5	5.65	5.45		2.83	15.39625	-25.54	1842.9	7.52
7	0.5	1.375	0.6875		0.69	0.4726563	-30.30	631.3	0.11
					0.00	0	-30.99	0.0	0.00
					0.00	0	-30.99	0.0	0.00

	Total h		Total A	$\Sigma b*h*y_{bar}$
	44.4		22.2	687.6

from Bottom y_{bar} =	**30.991**	**in**
x_{bar} =		in
Area =	22.19	in^2
I_x =	8469.2	in^4
I_y =		in^4
S_x =	273.3	in^3
rx =	19.54	in
ry =		in
J=		in^4
Zx =		in^3