INNERBELT BRIDGE

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: Erection Bolt Revised - Diaphragm KB + connection PL check

Check Level (Mark One):

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

Enter description below:

		Print Name	Signature	Date
X	Originator	Paul Blasko	[signature]	9/8/11
X	Checker	SARAH LARSON	Sarah Larson	9-12-11
X	Backchecker	Paul Blasko	[signature]	9/12/11
X	Updater	SARAH LARSON	Sarah Larson	9-13-11
X	Validater	Kolbe Garrett	[signature]	10-3-11

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

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 for Erection Bolts

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

Bolt Cap : 28.7 kips
57.4 kips - double shear

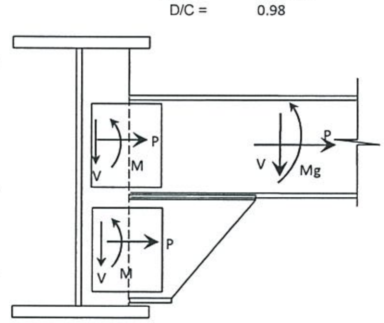
Mg =	-1086 ft-kips				
V =	138 kips	eh =	3.55 in	Veh =	40.7 ft-kips
P =	-71 kips	ev =	17.0 in	Pev =	-101 ft-kips
				Mc = Mg-Veh-Pev =	-1025 ft-kips

bolt	row	column	x	y	x-xb	y-yb	$(x-xb)^2$	$(y-yb)^2$	rv	rh	r
1	1	1	0.0	3.45	-1.55	-32.9	2.40	1081	4.33	50.6	50.8
2	2	1	0.0	10.35	-1.55	-26.0	2.40	675	4.33	40.7	40.9
3	3	1	0.0	17.25	-1.55	-19.1	2.40	364	4.33	30.8	31.1
4	4	1	0.0	24.15	-1.55	-12.2	2.40	148	4.33	20.9	21.3
5	5	1	0.0	31.05	-1.55	-5.3	2.40	28	4.33	11.0	11.8
6	7	1	0.0	43.50	-1.55	7.2	2.40	51	4.33	-6.9	8.2
7	8	1	0.0	47.38	-1.55	11.0	2.40	122	4.33	-12.5	13.2
8	9	1	0.0	51.25	-1.55	14.9	2.40	223	4.33	-18.0	18.5
9	10	1	0.0	55.13	-1.55	18.8	2.40	353	4.33	-23.6	24.0
10	11	1	0.0	59.00	-1.55	22.7	2.40	514	4.33	-29.2	29.5
11	12	1	0.0	62.88	-1.55	26.5	2.40	705	4.33	-34.7	35.0
12	1	2	3.25	0.00	1.70	-36.3	2.90	1320	9.00	55.5	56.2
13	2	2	3.25	6.90	1.70	-29.4	2.90	866	9.00	45.6	46.5
15	4	2	3.25	20.70	1.70	-15.6	2.90	244	9.00	25.8	27.3
16	5	2	3.25	27.60	1.70	-8.7	2.90	76	9.00	15.9	18.3
17	6	2	3.25	34.50	1.70	-1.8	2.90	3	9.00	6.0	10.8
18	7	2	3.25	43.50	1.70	7.2	2.90	51	9.00	-6.9	11.3
19	8	2	3.25	47.38	1.70	11.0	2.90	122	9.00	-12.5	15.4
21	10	2	3.25	55.13	1.70	18.8	2.90	353	9.00	-23.6	25.2
22	11	2	3.25	59.00	1.70	22.7	2.90	514	9.00	-29.2	30.5
23	12	2	3.25	62.88	1.70	26.5	2.90	705	9.00	-34.7	35.9
	Bot		16.3	176							
	Top		16.3	587							
	Total		33	763			55	8,519			

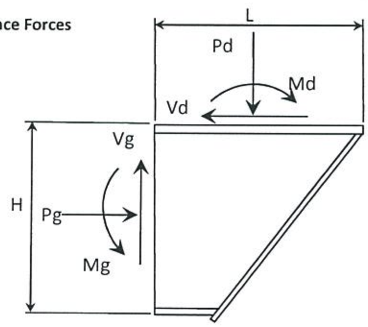
rv : vert comp of bolt force
rh : horiz comp of bolt force

	Bot	Top	Total
n	10	11	21
xb	1.63	1.48	1.55
yb	17.6	53.4	36.3
Ip	1,323	552	8,575
rp	30.3	-21.1	
rh-rp	24.3	-13.6	
P	303	-232	71
V	66.7	71.0	138
M	-1898	-793	-13,103

max r = 56.2 kips
D/C = 0.98



Knee Brace Forces



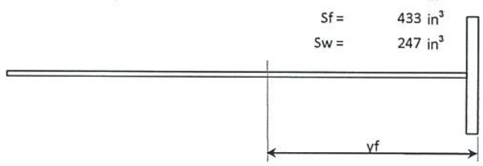
Positive Forces Shown

L :	40 in
H :	43.5 in
Pg =	303 kips
Vg =	66.7 kips
Mg =	1898 in-kips
Pd = Vg =	66.7 kips
Vd = Pg =	303 kips
Md = Mg+PgH/2+VgL/2 =	7,148 in-kips

Knee Brace Web

t : 0.625 in	@ Diaph	Vr =	725 kips	D/C =	0.42
	@ Gdr	Vr =	788 kips	D/C =	0.08

@ Diaph	bf :	12 in	A =	36 in^2	fa =	-1.88 ksi	axial stress		
	tf :	0.875 in	yf =	14.8 in	f_{bf} =	-16.5 ksi	bending stress in flange		
	Fy :	50 ksi	I =	6422 in^4	f_{bw} =	29.0 ksi	bending stress in web		D/C
			Sf =	433 in^3	f_f =	-18.4 ksi	Total flange stress		0.37
			Sw =	247 in^3	f_w =	27.1 ksi	Total web stress		0.54



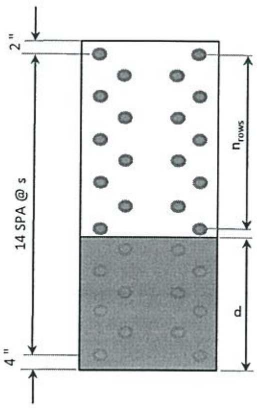
@ Girder	bf :	0 in	A =	27.2 in^2	fa =	-11.1 ksi
	tf :	0 in	yb =	21.8 in	fb =	9.6 ksi
			I =	4,287 in^4		
			S =	197 in^3		

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 =	0.51

N:\49633\Bridges\Design\Final Design\Unit 2\Excel\Diaphragms\final forces\LateralBracingRevision\RFI calcs\[FloorbeamConns 0908_3.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

fa =	-0.63 ksi
fb =	30.0 ksi
Max Bolt Stress =	29.4 ksi
Max Bolt Tension, Tu =	23.1 kips
Bolt Shear Cap, $\phi_s R_n$:	36.0 kips
ϕ_t :	0.80
Bolt Tensile Cap, $\phi_t T_n$ =	53.0 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 =	8407 in^4
St =	238 in^3
Sc =	1307 in^3
Bolt Shear, Pu =	13.8 kips
D/C =	0.44

Girder connection PL

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

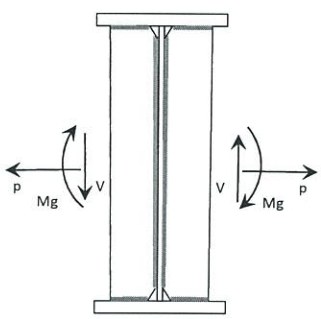
Girder Depth, H :	8 ft
F = M/H =	100 kips
L :	2 ft
M = FL/2 =	100 ft-kips

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 =	92 in
horiz clip :	1 in
vert clip :	2 in

L =	114 in
I =	90,235 in^3
Ss =	1,880 in^2 - of Stiff weld
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

Revised for Erection Bolts

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

Bolt Cap : 28.7 kips
57.4 kips - double shear

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

eh = 3.55 in
ev = 17.0 in

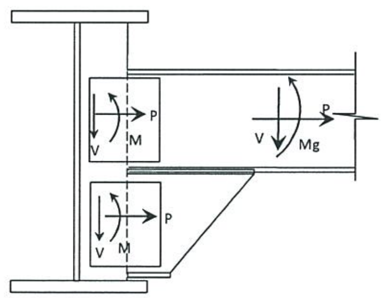
Veh = 12.5 ft-kips
Pev = -11 ft-kips
Mc = Mg-Veh-Pev = 1106 ft-kips

bolt	row	column	x	y	x-xb	y-yb	$(x-xb)^2$	$(y-yb)^2$	rv	rh	r
1	1	1	0.0	3.45	-1.55	-32.9	2.40	1081	4.41	-50.5	50.7
2	2	1	0.0	10.35	-1.55	-26.0	2.40	675	4.41	-39.8	40.1
3	3	1	0.0	17.25	-1.55	-19.1	2.40	364	4.41	-29.1	29.5
4	4	1	0.0	24.15	-1.55	-12.2	2.40	148	4.41	-18.5	19.0
5	5	1	0.0	31.05	-1.55	-5.3	2.40	28	4.41	-7.8	9.0
6	7	1	0.0	43.50	-1.55	7.2	2.40	51	4.41	11.5	12.3
7	8	1	0.0	47.38	-1.55	11.0	2.40	122	4.41	17.5	18.0
8	9	1	0.0	51.25	-1.55	14.9	2.40	223	4.41	23.5	23.9
9	10	1	0.0	55.13	-1.55	18.8	2.40	353	4.41	29.5	29.8
10	11	1	0.0	59.00	-1.55	22.7	2.40	514	4.41	35.5	35.7
11	12	1	0.0	62.88	-1.55	26.5	2.40	705	4.41	41.5	41.7
12	1	2	3.25	0.00	1.70	-36.3	2.90	1320	-0.62	-55.8	55.8
13	2	2	3.25	6.90	1.70	-29.4	2.90	866	-0.62	-45.2	45.2
15	4	2	3.25	20.70	1.70	-15.6	2.90	244	-0.62	-23.8	23.8
16	5	2	3.25	27.60	1.70	-8.7	2.90	76	-0.62	-13.1	13.1
17	6	2	3.25	34.50	1.70	-1.8	2.90	3	-0.62	-2.5	2.5
18	7	2	3.25	43.50	1.70	7.2	2.90	51	-0.62	11.5	11.5
19	8	2	3.25	47.38	1.70	11.0	2.90	122	-0.62	17.5	17.5
21	10	2	3.25	55.13	1.70	18.8	2.90	353	-0.62	29.5	29.5
22	11	2	3.25	59.00	1.70	22.7	2.90	514	-0.62	35.5	35.5
23	12	2	3.25	62.88	1.70	26.5	2.90	705	-0.62	41.5	41.5
	Bot		16.3	176							
	Top		16.3	587							
	Total		33	763			55	8,519			

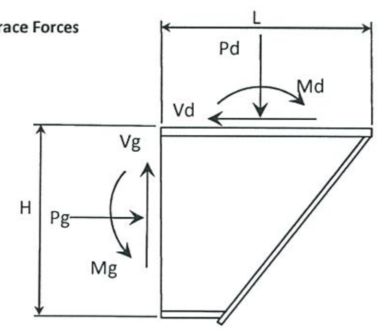
rv : vert comp of bolt force
rh : horiz comp of bolt force

	Bot	Top	Total
n	10	11	21
xb	1.63	1.48	1.55
yb	17.6	53.4	36.3
Ip	1,325	552	8,575
rp	-28.6	26.7	
rh-rp	-26.2	14.7	
P	-286	294	8
V	18.9	23.4	42
M	2051	855	13,180

max r = 55.8 kips
D/C = 0.97



Knee Brace Forces



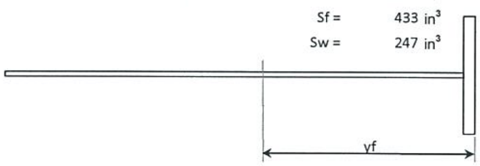
Positive Forces Shown

L : 40 in
H : 43.5 in
Pg = -286 kips
Vg = 18.9 kips
Mg = -2051 in-kips
Pd = Vg = 18.9 kips
Vd = Pg = -286 kips
Md = Mg+PgH/2+VgL/2 = -8,654 in-kips

Knee Brace Web

t :	0.625 in	@ Diaph	Vr =	725 kips	D/C =	0.39
		@ Gdr	Vr =	788 kips	D/C =	0.02

@ Diaph	bf :	12 in	A =	36 in^2	fa =	-0.53 ksi	axial stress	
	tf :	0.875 in	yf =	14.8 in	f_{bf} =	20.0 ksi	bending stress in flange	
	Fy :	50 ksi	I =	6422 in^4	f_{bw} =	-35.1 ksi	bending stress in web	D/C
			Sf =	433 in^3	f_f =	19.5 ksi	Total flange stress	0.39
			Sw =	247 in^3	f_w =	-35.6 ksi	Total web stress	0.71



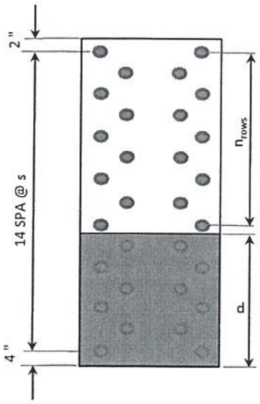
@ Girder	bf :	0 in	A =	27.2 in^2	fa =	10.5 ksi
	tf :	0 in	yb =	21.8 in	fb =	-10.4 ksi
			I =	4,287 in^4		
			S =	197 in^3		

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

Knee Brace - Diaphragm Connection



s : 3.75 in

row	h	d	d^2
1	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 =	-0.18 ksi
fb =	40.1 ksi
Max Bolt Stress =	39.9 ksi
Max Bolt Tension, Tu =	31.3 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

bolt D :	1 in
Ab =	0.785 in^2
Fub :	120 ksi
w :	14 in
d :	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 =	13.0 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.

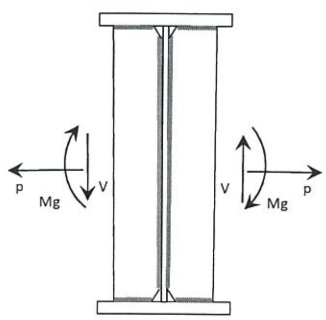
Girder Depth, H :	8.2 ft
F = M/H =	100 kips
L :	2 ft
M = FL/2 =	100 ft-kips

Conn PL

b :	12 in
t :	1.125 in
Fy :	50 ksi
tw :	1 in
beff :	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

For: Innerbelt Bridge
Made by: PDB
Date: 9/8/2011

Job no. 49633
Checked by: SJL
Date: 9/12/2011

Sheet no.
Backchecked by: PDB
Date: 9/12/2011

Revised for Erection Bolts

Floorbeam Type B, 3'-0" Kneebrace

Bolt Cap : 28.7 kips
57.4 kips - double shear

Mg = -3114 ft-kips
V = 240 kips
P = -386 kips

eh = 1.59 in
ev = 14.5 in

Veh = 32 ft-kips
Pev = -467 ft-kips
Mc = Mg-Veh-Pev = -2679 ft-kips

bolt	row	column	x	y	x-xb	y-yb	$(x-xb)^2$	$(y-yb)^2$	rv	rh	r
1	1	1	0	1.71	-1.59	-44.6	2.53	1992	3.55	51.8	51.9
2	2	1	0	5.14	-1.59	-41.2	2.53	1698	3.55	48.4	48.6
3	3	1	0	8.57	-1.59	-37.8	2.53	1427	3.55	45.1	45.2
4	4	1	0	12.00	-1.59	-34.4	2.53	1180	3.55	41.8	41.9
5	5	1	0	15.43	-1.59	-30.9	2.53	956	3.55	38.4	38.6
6	6	1	0	18.85	-1.59	-27.5	2.53	756	3.55	35.1	35.2
7	7	1	0	22.28	-1.59	-24.1	2.53	579	3.55	31.7	31.9
8	9	1	0	34.87	-1.59	-11.5	2.53	132	3.55	19.4	19.7
9	10	1	0	38.12	-1.59	-8.2	2.53	68	3.55	16.2	16.6
10	11	1	0	41.37	-1.59	-5.0	2.53	25	3.55	13.1	13.5
11	12	1	0	44.62	-1.59	-1.7	2.53	3	3.55	9.9	10.5
12	13	1	0	47.87	-1.59	1.5	2.53	2	3.55	6.7	7.6
13	14	1	0	51.12	-1.59	4.8	2.53	23	3.55	3.6	5.0
14	15	1	0	54.37	-1.59	8.0	2.53	64	3.55	0.4	3.6
15	16	1	0	57.62	-1.59	11.3	2.53	127	3.55	-2.8	4.5
16	17	1	0	60.87	-1.59	14.5	2.53	211	3.55	-6.0	6.9
17	18	1	0	64.12	-1.59	17.8	2.53	316	3.55	-9.1	9.8
18	19	1	0	67.37	-1.59	21.0	2.53	442	3.55	-12.3	12.8
19	20	1	0	70.62	-1.59	24.3	2.53	589	3.55	-15.5	15.9
20	21	1	0	73.87	-1.59	27.5	2.53	757	3.55	-18.7	19.0
21	22	1	0	77.12	-1.59	30.8	2.53	947	3.55	-21.8	22.1
22	23	1	0	80.37	-1.59	34.0	2.53	1157	3.55	-25.0	25.3
23	24	1	0	83.62	-1.59	37.3	2.53	1389	3.55	-28.2	28.4
24	25	1	0	86.87	-1.59	40.5	2.53	1642	3.55	-31.3	31.5
25	1	2	3.25	0.00	1.66	-46.3	2.75	2148	6.73	53.5	53.9
26	2	2	3.25	3.43	1.66	-42.9	2.75	1842	6.73	50.1	50.6
27	3	2	3.25	6.86	1.66	-39.5	2.75	1560	6.73	46.8	47.3
29	5	2	3.25	13.71	1.66	-32.6	2.75	1065	6.73	40.1	40.6
30	6	2	3.25	17.14	1.66	-29.2	2.75	853	6.73	36.7	37.3
31	7	2	3.25	20.57	1.66	-25.8	2.75	665	6.73	33.4	34.1
32	8	2	3.25	24.00	1.66	-22.4	2.75	500	6.73	30.0	30.8
33	9	2	3.25	34.87	1.66	-11.5	2.75	132	6.73	19.4	20.6
34	10	2	3.25	38.12	1.66	-8.2	2.75	68	6.73	16.2	17.6
35	11	2	3.25	41.37	1.66	-5.0	2.75	25	6.73	13.1	14.7
36	12	2	3.25	44.62	1.66	-1.7	2.75	3	6.73	9.9	12.0
37	13	2	3.25	47.87	1.66	1.5	2.75	2	6.73	6.7	9.5
38	14	2	3.25	51.12	1.66	4.8	2.75	23	6.73	3.6	7.6
39	15	2	3.25	54.37	1.66	8.0	2.75	64	6.73	0.4	6.7
40	16	2	3.25	57.62	1.66	11.3	2.75	127	6.73	-2.8	7.3
42	18	2	3.25	64.12	1.66	17.8	2.75	316	6.73	-9.1	11.3
43	19	2	3.25	67.37	1.66	21.0	2.75	442	6.73	-12.3	14.0
44	20	2	3.25	70.62	1.66	24.3	2.75	589	6.73	-15.5	16.9
45	21	2	3.25	73.87	1.66	27.5	2.75	757	6.73	-18.7	19.8
46	22	2	3.25	77.12	1.66	30.8	2.75	947	6.73	-21.8	22.8
47	23	2	3.25	80.37	1.66	34.0	2.75	1157	6.73	-25.0	25.9
48	24	2	3.25	83.62	1.66	37.3	2.75	1389	6.73	-28.2	29.0
49	25	2	3.25	86.87	1.66	40.5	2.75	1642	6.73	-31.3	32.1
	Bot		23	170							
	Top		52	2009							
	Total		75	2,178			124	32,800			

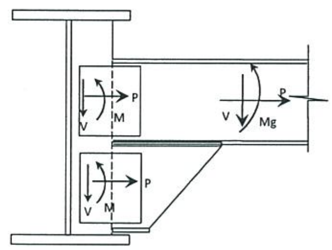
rv: vert comp of bolt force

rh: horiz comp of bolt force

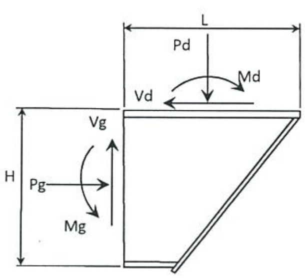
	Bot	Top	Total
n	14	33	47
xb	1.63	1.58	1.59
yb	12.1	60.9	46.3
Ip	859	8,706	32,924
rp	41.6	-6.0	
rh-rp	11.6	-25.4	
P	583	-197	386
V	72	168	240
M	839	8,500	13,795
		8,234	

rv = 6.7 kips
rh = 53.5 kips
r = 53.9 kips

D/C = 0.94



Knee Brace Forces



L : 54 in
H : 36 in
Pg = 583 kips
Vg = 72 kips
Mg = 839 in-kips
Pd = Vg = 72 kips
Vd = Pg = 583 kips
Md = Mg+PgH/2+VgL/2 = 9,387 in-kips

Knee Brace Web

t : 0.625 in

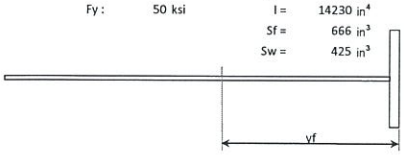
@ Diaph Vr = 979 kips D/C = 0.60
@ Gdr Vr = 653 kips D/C = 0.11

@ Diaph
bf : 12 in
tf : 0.875 in
Fy : 50 ksi

A = 44 in^2
yf = 21.4 in
I = 14230 in^4
Sf = 666 in^3
Sw = 425 in^3

fa = -1.63 ksi axial stress
f_{bf} = -14.1 ksi bending stress in flange
f_{bw} = 22.1 ksi bending stress in web
f_f = -15.7 ksi Total flange stress
f_w = 20.5 ksi Total web stress

D/C	
Total flange stress	0.31
Total web stress	0.41



@ Girder
bf : 0 in
tf : 0 in

A = 22.5 in^2
yb = 18.0 in
I = 2430 in^4
S = 135 in^3

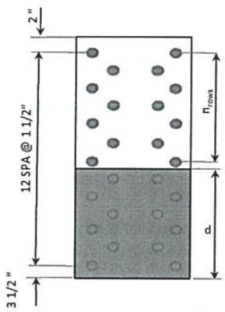
fa = -25.9 ksi
fb = 6.2 ksi

N:\49633\Bridges\Design\Final Design\Unit 2\Excel\Diaphragms\final forces\LateralBracingRevision\RFI calcs\[FloorbeamConns 0908_3.xlsx]B-5 Min M Revd

Kneebrace Connection PL - Web Weld

Double Fillet Size :	0.3125 in	Fexx :	70 ksi	ϕ_{e2} :	0.80		
Rr =	14.8 k/in	L =	54 in	Rr =	802 kips	D/C =	0.73

Knee Brace - Diaphragm Connection



s : 3.75 in

row	h	d	d^2
1	$\boxed{49.25}$	41.71	1,740
2	45.50	37.96	1,441
3	41.75	34.21	1,171
4	38.00	30.46	928
5	34.25	26.71	714
6	30.50	22.96	527
7	26.75	19.21	369
8	23.00	15.46	239
9	19.25	11.71	137
10	15.50	7.96	63
11	11.75	4.21	18
12	8.00	0.46	0
13	2.50	0.00	0

bolt D :	1 in
Ab =	0.785 in^2
Fub :	120 ksi
w :	14 in
d :	$\boxed{7.54}$ in
Ay =	398 in^3
n_{rows} =	12
n =	26
y =	21.1 in
Aby =	398 in^3
A =	124 in^2
I =	13539 in^4
St =	325 in^3
Sc =	1797 in^3

fa =	-0.58 ksi		
fb =	28.9 ksi		
Max Bolt Stress =	28.3 ksi		
Max Bolt Tension, Tu =	22.3 kips	Bolt Shear, Pu =	22.4 kips
Bolt Shear Cap, $\phi_s R_n$:	35.9 kips		
ϕ_t :	0.80		
Bolt Tensile Cap, $\phi_t T_n$ =	44.7 kips	D/C =	0.50

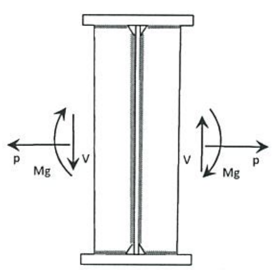
Girder connection PL

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

		Conn PL		
Girder Depth, H :	10.2 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 =	140 in
Lw =	118.4 in	I =	179,516 in^3
horiz clip :	1 in	Ss =	2,933 in^2 - of Stiff weld
vert clip :	2 in	Sw =	3,032 in^2 - of web weld

M =	3114 ft-kips		
V =	240 kips		
max. stiff weld shear due to M =	12.7 k/in	Stiff Weld D/C =	0.86
max. web weld shear due to M =	12.3 k/in		
max. web weld shear due to V =	2.0 k/in		
max resultant weld shear =	12.5 k/in	Web Weld D/C =	0.84

HNTB	For	Innerbelt Bridge	Job no.	49633	Sheet no.	
	Made by	PDB	Checked by	SJL	Backchecked by	PDB
	Date	9/8/2011	Date	9/15/2011	Date	9/15/2011

Revised for Erection Bolts

Floorbeam Type C, 2'-10 1/4" Kneebrace

Bolt Cap :	28.7 kips	Mg =	1719 ft-kips					
	57.4 kips - double shear	V =	-50 kips	eh =	1.58 in	Veh =	-7 ft-kips
		P =	-3 kips	ev =	10.2 in	Pev =	-3 ft-kips
						Mc = Mg-Veh-Pev =	1728 ft-kips

bolt	row	column	x	y	x-xb	y-yb	$(x-xb)^2$	$(y-yb)^2$	rv	rh	r
1	1	1	0	3.25	-1.58	-43.4	2.49	1880	0.42	-50.7	50.7
2	2	1	0	9.75	-1.58	-36.9	2.49	1358	0.42	-43.1	43.1
3	3	1	0	16.25	-1.58	-30.4	2.49	921	0.42	-35.5	35.5
4	4	1	0	22.75	-1.58	-23.9	2.49	569	0.42	-27.9	27.9
5	6	1	0	35.38	-1.58	-11.2	2.49	126	0.42	-13.1	13.1
6	7	1	0	38.63	-1.58	-8.0	2.49	64	0.42	-9.3	9.3
7	8	1	0	41.88	-1.58	-4.7	2.49	22	0.42	-5.5	5.5
8	9	1	0	45.13	-1.58	-1.5	2.49	2	0.42	-1.6	1.7
9	10	1	0	48.38	-1.58	1.8	2.49	3	0.42	2.2	2.2
10	11	1	0	51.63	-1.58	5.0	2.49	25	0.42	6.0	6.0
11	12	1	0	54.88	-1.58	8.3	2.49	68	0.42	9.8	9.8
12	13	1	0	58.13	-1.58	11.5	2.49	133	0.42	13.6	13.6
13	14	1	0	61.38	-1.58	14.8	2.49	218	0.42	17.4	17.4
14	15	1	0	64.63	-1.58	18.0	2.49	325	0.42	21.2	21.2
15	16	1	0	67.88	-1.58	21.3	2.49	452	0.42	25.0	25.0
16	17	1	0	71.13	-1.58	24.5	2.49	601	0.42	28.8	28.8
17	18	1	0	74.38	-1.58	27.8	2.49	771	0.42	32.6	32.6
18	19	1	0	77.63	-1.58	31.0	2.49	962	0.42	36.4	36.4
19	1	2	3.25	0.00	1.67	-46.6	2.79	2172	-3.39	-54.5	54.6
20	2	2	3.25	6.50	1.67	-40.1	2.79	1608	-3.39	-46.9	47.0
21	3	2	3.25	13.00	1.67	-33.6	2.79	1129	-3.39	-39.3	39.4
23	5	2	3.25	26.00	1.67	-20.6	2.79	425	-3.39	-24.1	24.3
24	6	2	3.25	35.38	1.67	-11.2	2.79	126	-3.39	-13.1	13.5
25	7	2	3.25	38.63	1.67	-8.0	2.79	64	-3.39	-9.3	9.9
26	8	2	3.25	41.88	1.67	-4.7	2.79	22	-3.39	-5.5	6.4
27	9	2	3.25	45.13	1.67	-1.5	2.79	2	-3.39	-1.6	3.8
29	11	2	3.25	51.63	1.67	5.0	2.79	25	-3.39	6.0	6.9
30	12	2	3.25	54.88	1.67	8.3	2.79	68	-3.39	9.8	10.3
31	13	2	3.25	58.13	1.67	11.5	2.79	133	-3.39	13.6	14.0
32	14	2	3.25	61.38	1.67	14.8	2.79	218	-3.39	17.4	17.7
33	15	2	3.25	64.63	1.67	18.0	2.79	325	-3.39	21.2	21.5
34	16	2	3.25	67.88	1.67	21.3	2.79	452	-3.39	25.0	25.2
35	17	2	3.25	71.13	1.67	24.5	2.79	601	-3.39	28.8	29.0
36	18	2	3.25	74.38	1.67	27.8	2.79	771	-3.39	32.6	32.8
37	19	2	3.25	77.63	1.67	31.0	2.79	962	-3.39	36.4	36.6
	Bot		13	98							
	Top		42	1534							
	Total		55	1631			92	17,607			

rv: vert comp of bolt force

rh: horiz comp of bolt force

	Bot	Top	Total
n	8	27	35
xb	1.63	1.56	1.58
yb	12.2	56.8	46.6
Ip	610	4,805	17,699
rp	-40.2	12.0	
rh-rp	-16.2	24.4	
P	-322	325	3
V	-11.9	-38.1	-50.0
M	-715	-5,630	-4,964

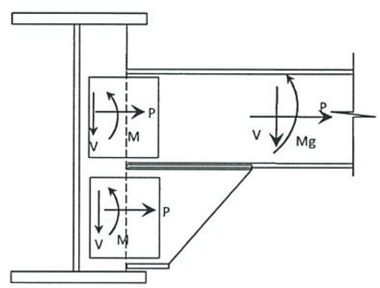
rv = 3.3 kips

rh = 54.7 kips

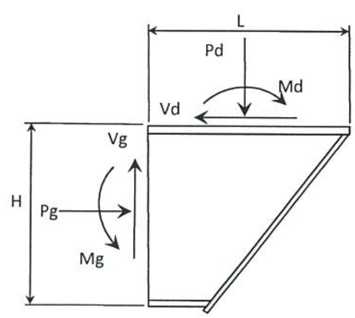
r = 54.8 kips

D/C = 0.95

actual max resultant bolt force



Knee Brace Forces

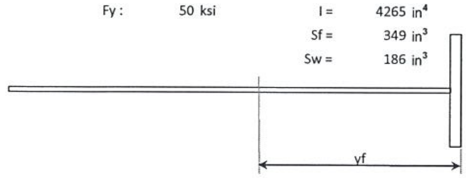


L :	34.25	in
H :	34.25	in
Pg =	-322	kips
Vg =	-12	kips
Mg =	-715	in-kips
Pd = Vg =	-12	kips
Vd = Pg =	-322	kips
Md = Mg+PgH/2+VgL/2 =	-6,024	in-kips

Knee Brace Web

t :	0.625 in	@ Diaph	Vr =	621 kips	D/C =	0.52
		@ Gdr	Vr =	621 kips	D/C =	0.02

@ Diaph	bf :	12 in	A =	32 in^2	fa =	0.37 ksi	axial stress		
	tf :	0.875 in	yf =	12.2 in	f_{bf} =	17.3 ksi	bending stress in flange		
	Fy :	50 ksi	I =	4265 in^4	f_{bw} =	-32.3 ksi	bending stress in web	D/C	
			Sf =	349 in^3	f_f =	17.6 ksi	Total flange stress	0.35	
			Sw =	186 in^3	f_w =	-32.0 ksi	Total web stress	0.64	

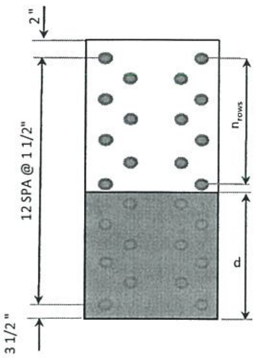


@ Girder	bf :	0 in	A =	21.4 in^2	fa =	15.0 ksi
	tf :	0 in	yb =	17.1 in	fb =	-5.8 ksi
			I =	2093 in^4		
			S =	122 in^3		

Kneebrace Connection PL - Web Weld

Double Fillet Size :	0.3125 in	Fexx :	70 ksi	ϕ_{e2} :	0.80		
Rr =	14.8 k/in	L =	34 in	Rr =	509 kips	D/C =	0.63

Knee Brace - Diaphragm Connection



s : 4 in

row	h	d	d^2
1	$\boxed{36.00}$	30.55	933
2	30.50	25.05	628
3	26.50	21.05	443
4	22.50	17.05	291
5	18.50	13.05	170
6	14.50	9.05	82
7	10.50	5.05	26
8	6.50	1.05	1
9	2.50	0.00	0

bolt D :	1 in
Ab =	0.785 in^2
Fub :	120 ksi
w :	14 in
d :	$\boxed{5.45}$ in
Ay =	208 in^3
n_{rows} =	8
n =	18
y =	16.6 in
Aby =	208 in^3
A =	88.9 in^2
I =	4798 in^4
St =	157 in^3
Sc =	880 in^3

fa =	0.13 ksi		
fb =	38.4 ksi		
Max Bolt Stress =	38.5 ksi		
Max Bolt Tension, Tu =	30.2 kips	Bolt Shear, Pu =	17.9 kips
Bolt Shear Cap, $\phi_s R_n$:	35.9 kips		
ϕ_t :	0.80		
Bolt Tensile Cap, $\phi_t T_n$ =	49.7 kips	D/C =	0.61

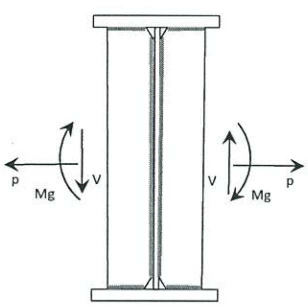
Girder connection PL

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

		Conn PL		
Girder Depth, H :	9.5 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 =	132 in
Lw =	110 in	I =	146,656 in^3
horiz clip :	1 in	Ss =	2,573 in^2 - of Stiff weld
vert clip :	2 in	Sw =	2,666 in^2 - of web weld

M = 1719 ft-kips
V = 50 kips

max. stiff weld shear due to M =	8.0 k/in	Stiff Weld D/C =	0.54
max. web weld shear due to M =	7.7 k/in		
max. web weld shear due to V =	0.5 k/in		
max resultant weld shear =	7.7 k/in	Web Weld D/C =	0.52

N:\49633\Bridges\Design\Final Design\Unit 2\Excel\Diaphragms\final forces\LateralBracingRevision\RFI calcs\[FloorbeamConns 0908_3.xlsx]C-S Max M Revd 3

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 for Erection Bolts

Floorbeam Type D, 1'-6" Top Kneebrace

Top Kneebrace is designed by treating it as a bottom kneebrace and flipping the sign of the shear and moment

Bolt Cap :	28.7 kips	Mg =	3114 ft-kips				
	57.4 kips - double shear	V =	-240 kips	eh =	3.14 in	Veh =	-63 ft-kips
		P =	-386 kips	ev =	7.3 in	Pev =	-235 ft-kips
						Mc = Mg-Veh-Pev =	3412 ft-kips

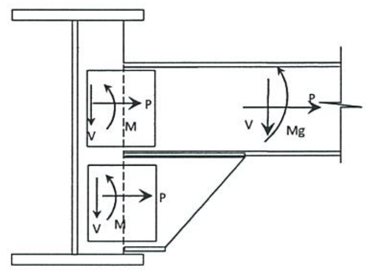
bolt	row	column	x	y	x-xb	y-yb	$(x-xb)^2$	$(y-yb)^2$	rv	rh	r
1	1	1	0	0	-3.14	-38.7	9.88	1496	0.60	-49.5	49.5
2	2	1	0	3.50	-3.14	-35.2	9.88	1238	0.60	-44.4	44.4
3	3	1	0	7.00	-3.14	-31.7	9.88	1004	0.60	-39.4	39.4
4	4	1	0	10.50	-3.14	-28.2	9.88	794	0.60	-34.3	34.3
5	5	1	0	20.00	-3.14	-18.7	9.88	349	0.60	-20.6	20.6
6	6	1	0	23.25	-3.14	-15.4	9.88	238	0.60	-15.9	15.9
7	7	1	0	26.50	-3.14	-12.2	9.88	148	0.60	-11.2	11.3
8	8	1	0	29.75	-3.14	-8.9	9.88	80	0.60	-6.6	6.6
9	9	1	0	33.00	-3.14	-5.7	9.88	32	0.60	-1.9	2.0
10	10	1	0	36.25	-3.14	-2.4	9.88	6	0.60	2.8	2.9
11	11	1	0	39.50	-3.14	0.8	9.88	1	0.60	7.5	7.5
12	12	1	0	42.75	-3.14	4.1	9.88	17	0.60	12.2	12.2
13	13	1	0	46.00	-3.14	7.3	9.88	54	0.60	16.9	16.9
14	14	1	0	49.25	-3.14	10.6	9.88	112	0.60	21.6	21.6
15	15	1	0	52.50	-3.14	13.8	9.88	191	0.60	26.3	26.3
16	16	1	0	55.75	-3.14	17.1	9.88	291	0.60	31.0	31.0
17	17	1	0	59.00	-3.14	20.3	9.88	413	0.60	35.6	35.6
18	18	1	0	62.25	-3.14	23.6	9.88	556	0.60	40.3	40.3
19	19	1	0	65.50	-3.14	26.8	9.88	719	0.60	45.0	45.0
20	20	1	0	68.75	-3.14	30.1	9.88	904	0.60	49.7	49.7
21	21	1	0	72.00	-3.14	33.3	9.88	1110	0.60	54.4	54.4
22	1	2	3.25	0.00	0.11	-38.7	0.01	1496	-4.09	-49.5	49.6
23	2	2	3.25	3.50	0.11	-35.2	0.01	1238	-4.09	-44.4	44.6
24	3	2	3.25	7.00	0.11	-31.7	0.01	1004	-4.09	-39.4	39.6
25	4	2	3.25	10.50	0.11	-28.2	0.01	794	-4.09	-34.3	34.6
26	5	2	3.25	20.00	0.11	-18.7	0.01	349	-4.09	-20.6	21.0
27	6	2	3.25	23.25	0.11	-15.4	0.01	238	-4.09	-15.9	16.4
28	7	2	3.25	26.50	0.11	-12.2	0.01	148	-4.09	-11.2	12.0
29	8	2	3.25	29.75	0.11	-8.9	0.01	80	-4.09	-6.6	7.7
30	9	2	3.25	33.00	0.11	-5.7	0.01	32	-4.09	-1.9	4.5
31	10	2	3.25	36.25	0.11	-2.4	0.01	6	-4.09	2.8	5.0
32	11	2	3.25	39.50	0.11	0.8	0.01	1	-4.09	7.5	8.6
33	12	2	3.25	42.75	0.11	4.1	0.01	17	-4.09	12.2	12.9
34	13	2	3.25	46.00	0.11	7.3	0.01	54	-4.09	16.9	17.4
35	14	2	3.25	49.25	0.11	10.6	0.01	112	-4.09	21.6	22.0
36	15	2	3.25	52.50	0.11	13.8	0.01	191	-4.09	26.3	26.6
37	16	2	3.25	55.75	0.11	17.1	0.01	291	-4.09	31.0	31.2
38	17	2	3.25	59.00	0.11	20.3	0.01	413	-4.09	35.6	35.9
39	18	2	3.25	62.25	0.11	23.6	0.01	556	-4.09	40.3	40.5
40	19	2	3.25	65.50	0.11	26.8	0.01	719	-4.09	45.0	45.2
41	20	2	3.25	68.75	0.11	30.1	0.01	904	-4.09	49.7	49.9
42	21	2	3.25	72.00	0.11	33.3	0.01	1110	-4.09	54.4	54.5
43	1	3	6.5	0.00	3.36	-38.7	11.27	1496	-8.78	-49.5	50.2
45	3	3	6.5	7.00	3.36	-31.7	11.27	1004	-8.78	-39.4	40.3
46	4	3	6.5	10.50	3.36	-28.2	11.27	794	-8.78	-34.3	35.4
47	5	3	6.5	20.00	3.36	-18.7	11.27	349	-8.78	-20.6	22.4
48	6	3	6.5	23.25	3.36	-15.4	11.27	238	-8.78	-15.9	18.2
49	7	3	6.5	26.50	3.36	-12.2	11.27	148	-8.78	-11.2	14.3
50	8	3	6.5	29.75	3.36	-8.9	11.27	80	-8.78	-6.6	11.0
51	9	3	6.5	33.00	3.36	-5.7	11.27	32	-8.78	-1.9	9.0
52	10	3	6.5	36.25	3.36	-2.4	11.27	6	-8.78	2.8	9.2
53	11	3	6.5	39.50	3.36	0.8	11.27	1	-8.78	7.5	11.6
54	12	3	6.5	42.75	3.36	4.1	11.27	17	-8.78	12.2	15.0
56	14	3	6.5	49.25	3.36	10.6	11.27	112	-8.78	21.6	23.3
57	15	3	6.5	52.50	3.36	13.8	11.27	191	-8.78	26.3	27.7
58	16	3	6.5	55.75	3.36	17.1	11.27	291	-8.78	31.0	32.2
59	17	3	6.5	59.00	3.36	20.3	11.27	413	-8.78	35.6	36.7
60	18	3	6.5	62.25	3.36	23.6	11.27	556	-8.78	40.3	41.3
61	19	3	6.5	65.50	3.36	26.8	11.27	719	-8.78	45.0	45.9
62	20	3	6.5	68.75	3.36	30.1	11.27	904	-8.78	49.7	50.5
63	21	3	6.5	72.00	3.36	33.3	11.27	1110	-8.78	54.4	55.1
	Bot		33	60							
	Top		159	2300							
	Total		192	2360			422	27,965			

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

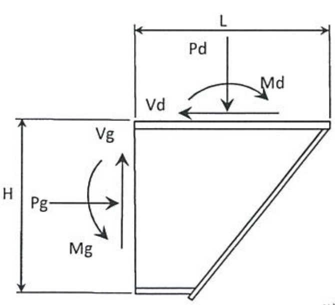
	Bot	Top	Total
n	11	50	61
xb	2.95	3.19	3.14
yb	5.4	46.0	38.7
Ip	253	13,277	28,386
rp	-41.7	16.9	
rh-rp	-7.3	37.5	
P	-458	844	386
V	-40	-200	-240
M	-366	-19,152	-15,132

r = 55.1 kips

D/C = 0.96



Knee Brace Forces



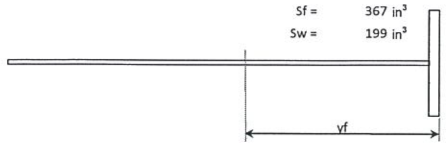
L :	35.5	in
H :	19.75	in
Pg =	-458	kips
Vg =	-40	kips
Mg =	-366	in-kips
Pd = Vg =	-40	kips
Vd = Pg =	-458	kips
Md = Mg+PgH/2+VgL/2 =	-4,177	in-kips

N:\49633\Bridges\Design\Final Design\Unit 2\Excel\Diaphragms\final forces\LateralBracingRevision\RFI calcs\[FloorbeamConns 0908_3.xlsx]D Top Min M Revd

Knee Brace Web

t :	0.625 in	@ Diaph	Vr =	643 kips		D/C =	0.71
		@ Gdr	Vr =	358 kips		D/C =	0.11

@ Diaph	bf :	12 in	A =	33 in^2	fa =	1.23 ksi	axial stress		
	tf :	0.875 in	yf =	12.8 in	f_{bf} =	11.4 ksi	bending stress in flange		
	Fy :	50 ksi	I =	4688 in^4	f_{bw} =	-21.0 ksi	bending stress in web	D/C	
			Sf =	367 in^3	f_f =	12.6 ksi	Total flange stress	0.25	
			Sw =	199 in^3	f_w =	-19.8 ksi	Total web stress	0.40	

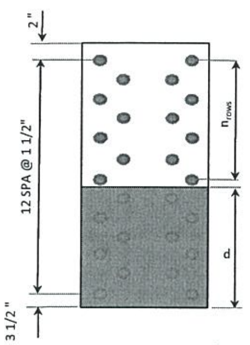


@ Girder	bf :	12 in	A =	32.3 in^2	fa =	14.2 ksi	
	tf :	0.875 in	yb =	9.9 in	fb =	-1.7 ksi	
			I =	2174 in^4			
			S =	220 in^3			

Kneebrace Connection PL - Web Weld

Double Fillet Size :	0.375 in	Fexx :	70 ksi	ϕ_{e2} :	0.80		
Rr =	17.8 k/in	L =	36 in	Rr =	632 kips	D/C =	0.72

Knee Brace - Diaphragm Connection



s : 3.25 in

row	h	d	d^2
1	37.25	31.16	971
2	31.75	25.66	658
3	28.50	22.41	502
4	25.25	19.16	367
5	22.00	15.91	253
6	18.75	12.66	160
7	15.50	9.41	89
8	12.25	6.16	38
9	9.00	2.91	8
10	5.75	0.00	0
11	2.50	0.00	0

bolt D :	1 in
Ab =	0.785 in^2
Fub :	120 ksi
w :	14 in
d :	6.09 in
Ay =	260 in^3
n_{rows} =	10
n =	22
y =	16.5 in
Aby =	260 in^3
A =	101 in^2
I =	5840 in^4
St =	187 in^3
Sc =	959 in^3

fa =	0.40 ksi		
fb =	22.3 ksi		
Max Bolt Stress =	22.7 ksi		
Max Bolt Tension, Tu =	17.8 kips	Bolt Shear, Pu =	20.8 kips
Bolt Shear Cap, $\phi_s R_n$:	35.9 kips		
ϕ_t :	0.80		
Bolt Tensile Cap, $\phi_t T_n$ =	46.7 kips	D/C =	0.38

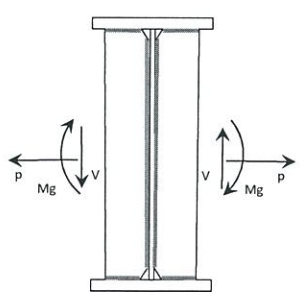
Girder connection PL

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 :	9 tw = 9 in

Conn PL Weld Capacity

Double Fillet Size :	0.4375 in	Fexx =	70 ksi	ϕ_{e2} :	0.80	Rr =	20.8 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 =	3114 ft-kips		
V =	240 kips		
max. stiff weld shear due to M =	19.9 k/in	Stiff Weld D/C =	0.96
max. web weld shear due to M =	19.0 k/in		
max. web weld shear due to V =	2.6 k/in		
max resultant weld shear =	19.2 k/in	Web Weld D/C =	0.93

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 for Erection Bolts

Floorbeam Type D, 1'-6" Bottom Kneebrace

Bolt Cap : 28.7 kips
57.4 kips - double shear

Mg = -3114 ft-kips
V = 240 kips
P = -386 kips

eh = 3.14 in
ev = 7.3 in

Veh = 63 ft-kips
Pev = -235 ft-kips
Mc = Mg-Veh-Pev = -2941 ft-kips

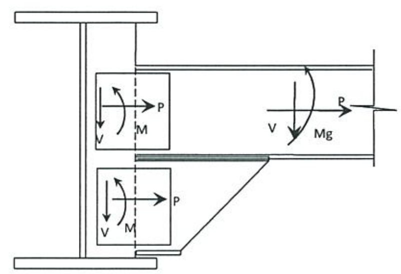
bolt	row	column	x	y	x-xb	y-yb	$(x\text{-}xb)^2$	$(y\text{-}yb)^2$	rv	rh	r
1	1	1	0	0	-3.14	-38.7	9.88	1496	0.03	54.4	54.4
2	2	1	0	3.50	-3.14	-35.2	9.88	1238	0.03	50.1	50.1
3	3	1	0	7.00	-3.14	-31.7	9.88	1004	0.03	45.7	45.7
4	4	1	0	10.50	-3.14	-28.2	9.88	794	0.03	41.4	41.4
5	5	1	0	20.00	-3.14	-18.7	9.88	349	0.03	29.6	29.6
6	6	1	0	23.25	-3.14	-15.4	9.88	238	0.03	25.5	25.5
7	7	1	0	26.50	-3.14	-12.2	9.88	148	0.03	21.5	21.5
8	8	1	0	29.75	-3.14	-8.9	9.88	80	0.03	17.4	17.4
9	9	1	0	33.00	-3.14	-5.7	9.88	32	0.03	13.4	13.4
10	10	1	0	36.25	-3.14	-2.4	9.88	6	0.03	9.3	9.3
11	11	1	0	39.50	-3.14	0.8	9.88	1	0.03	5.3	5.3
12	12	1	0	42.75	-3.14	4.1	9.88	17	0.03	1.3	1.3
13	13	1	0	46.00	-3.14	7.3	9.88	54	0.03	-2.8	2.8
14	14	1	0	49.25	-3.14	10.6	9.88	112	0.03	-6.8	6.8
15	15	1	0	52.50	-3.14	13.8	9.88	191	0.03	-10.9	10.9
16	16	1	0	55.75	-3.14	17.1	9.88	291	0.03	-14.9	14.9
17	17	1	0	59.00	-3.14	20.3	9.88	413	0.03	-18.9	18.9
18	18	1	0	62.25	-3.14	23.6	9.88	556	0.03	-23.0	23.0
19	19	1	0	65.50	-3.14	26.8	9.88	719	0.03	-27.0	27.0
20	20	1	0	68.75	-3.14	30.1	9.88	904	0.03	-31.1	31.1
21	21	1	0	72.00	-3.14	33.3	9.88	1110	0.03	-35.1	35.1
22	1	2	3.25	0.00	0.11	-38.7	0.01	1496	4.07	54.4	54.6
23	2	2	3.25	3.50	0.11	-35.2	0.01	1238	4.07	50.1	50.2
24	3	2	3.25	7.00	0.11	-31.7	0.01	1004	4.07	45.7	45.9
25	4	2	3.25	10.50	0.11	-28.2	0.01	794	4.07	41.4	41.6
26	5	2	3.25	20.00	0.11	-18.7	0.01	349	4.07	29.6	29.8
27	6	2	3.25	23.25	0.11	-15.4	0.01	238	4.07	25.5	25.8
28	7	2	3.25	26.50	0.11	-12.2	0.01	148	4.07	21.5	21.9
29	8	2	3.25	29.75	0.11	-8.9	0.01	80	4.07	17.4	17.9
30	9	2	3.25	33.00	0.11	-5.7	0.01	32	4.07	13.4	14.0
31	10	2	3.25	36.25	0.11	-2.4	0.01	6	4.07	9.3	10.2
32	11	2	3.25	39.50	0.11	0.8	0.01	1	4.07	5.3	6.7
33	12	2	3.25	42.75	0.11	4.1	0.01	17	4.07	1.3	4.3
34	13	2	3.25	46.00	0.11	7.3	0.01	54	4.07	-2.8	4.9
35	14	2	3.25	49.25	0.11	10.6	0.01	112	4.07	-6.8	7.9
36	15	2	3.25	52.50	0.11	13.8	0.01	191	4.07	-10.9	11.6
37	16	2	3.25	55.75	0.11	17.1	0.01	291	4.07	-14.9	15.4
38	17	2	3.25	59.00	0.11	20.3	0.01	413	4.07	-18.9	19.4
39	18	2	3.25	62.25	0.11	23.6	0.01	556	4.07	-23.0	23.3
40	19	2	3.25	65.50	0.11	26.8	0.01	719	4.07	-27.0	27.3
41	20	2	3.25	68.75	0.11	30.1	0.01	904	4.07	-31.1	31.3
42	21	2	3.25	72.00	0.11	33.3	0.01	1110	4.07	-35.1	35.3
43	1	3	6.5	0.00	3.36	-38.7	11.27	1496	8.11	54.4	55.0
45	3	3	6.5	7.00	3.36	-31.7	11.27	1004	8.11	45.7	46.4
46	4	3	6.5	10.50	3.36	-28.2	11.27	794	8.11	41.4	42.2
47	5	3	6.5	20.00	3.36	-18.7	11.27	349	8.11	29.6	30.6
48	6	3	6.5	23.25	3.36	-15.4	11.27	238	8.11	25.5	26.8
49	7	3	6.5	26.50	3.36	-12.2	11.27	148	8.11	21.5	23.0
50	8	3	6.5	29.75	3.36	-8.9	11.27	80	8.11	17.4	19.2
51	9	3	6.5	33.00	3.36	-5.7	11.27	32	8.11	13.4	15.7
52	10	3	6.5	36.25	3.36	-2.4	11.27	6	8.11	9.3	12.4
53	11	3	6.5	39.50	3.36	0.8	11.27	1	8.11	5.3	9.7
54	12	3	6.5	42.75	3.36	4.1	11.27	17	8.11	1.3	8.2
56	14	3	6.5	49.25	3.36	10.6	11.27	112	8.11	-6.8	10.6
57	15	3	6.5	52.50	3.36	13.8	11.27	191	8.11	-10.9	13.5
58	16	3	6.5	55.75	3.36	17.1	11.27	291	8.11	-14.9	17.0
59	17	3	6.5	59.00	3.36	20.3	11.27	413	8.11	-18.9	20.6
60	18	3	6.5	62.25	3.36	23.6	11.27	556	8.11	-23.0	24.4
61	19	3	6.5	65.50	3.36	26.8	11.27	719	8.11	-27.0	28.2
62	20	3	6.5	68.75	3.36	30.1	11.27	904	8.11	-31.1	32.1
63	21	3	6.5	72.00	3.36	33.3	11.27	1110	8.11	-35.1	36.0
	Bot		33	60							
	Top		159	2300							
	Total		192	2360			422	27,965			

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

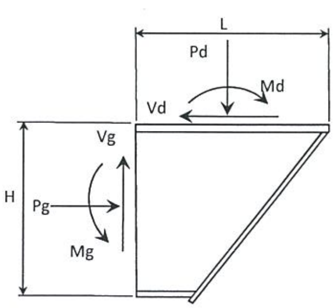
	Bot	Top	Total
n	11	50	61
xb	2.95	3.19	3.14
yb	5.4	46.0	38.7
Ip	253	13,277	28,386
rp	47.7	-2.8	
rh-rp	6.3	-32.3	
P	525	-139	386
V	41	199	240
M	315	16,509	21,690

r = 55.0 kips

D/C = 0.96



Knee Brace Forces



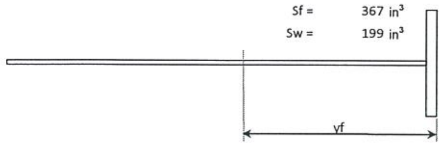
L : 35.5 in
H : 19.75 in
Pg = 525 kips
Vg = 41 kips
Mg = 315 in-kips
Pd = Vg = 41 kips
Vd = Pg = 525 kips
Md = Mg+PgH/2+VgL/2 = 4,774 in-kips

N:\49633\Bridges\Design\Final Design\Unit 2\Excel\Diaphragms\final forces\LateralBracingRevision\RFI calcs\[FloorbeamConns 0908_3.xlsx]D Bot Min M Revd

Knee Brace Web

t :	0.625 in	@ Diaph	Vr =	643 kips	D/C =	0.82
		@ Gdr	Vr =	358 kips	D/C =	0.11

@ Diaph	bf :	12 in	A =	33 in²	fa =	-1.24 ksi	axial stress	
	tf :	0.875 in	yf =	12.8 in	f_{bf} =	-13.0 ksi	bending stress in flange	
	Fy :	50 ksi	I =	4688 in⁴	f_{bw} =	24.0 ksi	bending stress in web	D/C
			Sf =	367 in³	f_f =	-14.3 ksi	Total flange stress	0.29
			Sw =	199 in³	f_w =	22.8 ksi	Total web stress	0.46

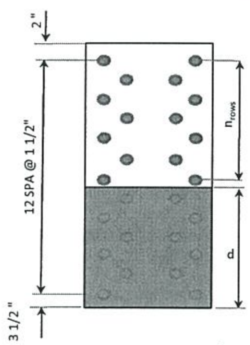


@ Girder	bf :	12 in	A =	32.3 in²	fa =	-16.3 ksi
	tf :	0.875 in	yb =	9.9 in	fb =	1.4 ksi
			I =	2174 in⁴		
			S =	220 in³		

Kneebrace Connection PL - Web Weld

Double Fillet Size :	0.375 in	Fexx :	70 ksi	ϕ_{e2} :	0.80		
Rr =	17.8 k/in	L =	36 in	Rr =	632 kips	D/C =	0.83

Knee Brace - Diaphragm Connection



s : 3.25 in

row	h	d	d²
1	$\boxed{37.25}$	31.16	971
2	34.00	27.91	779
3	30.75	24.66	608
4	27.50	21.41	458
5	24.25	18.16	330
6	21.00	14.91	222
7	17.75	11.66	136
8	14.50	8.41	71
9	11.25	5.16	27
10	8.00	1.91	4
11	2.50	0.00	0

bolt D :	1 in
Ab =	0.785 in²
Fub :	120 ksi
w :	14 in
d :	$\boxed{6.09}$ in
Ay =	260 in³
n_{rows} =	10
n =	22
y =	16.5 in
Aby =	260 in³
A =	101 in²
I =	6,718 in⁴
St =	216 in³
Sc =	1103 in³

fa =	-0.40 ksi		
fb =	22.1 ksi		
Max Bolt Stress =	21.7 ksi		
Max Bolt Tension, Tu =	17.1 kips	Bolt Shear, Pu =	23.8 kips
Bolt Shear Cap, $\phi_s R_n$:	35.9 kips		
ϕ_t :	0.80		
Bolt Tensile Cap, $\phi_t T_n$ =	42.8 kips	D/C =	0.40

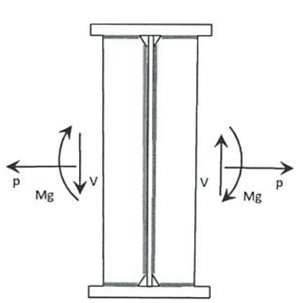
Girder connection PL

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 :	0.4375 in	Fexx =	70 ksi	ϕ_{e2} :	0.80	Rr =	20.8 k/in



Ls =	11 in	L =	114 in
Lw =	92 in	I =	90,235 in³
horiz clip :	1 in	Ss =	1,880 in² - of Stiff weld
vert clip :	2 in	Sw =	1,962 in² - of web weld

M =	3114 ft-kips		
V =	240 kips		
max. stiff weld shear due to M =	19.9 k/in	Stiff Weld D/C =	0.96
max. web weld shear due to M =	19.0 k/in		
max. web weld shear due to V =	2.6 k/in		
max resultant weld shear =	19.2 k/in	Web Weld D/C =	0.93

HNTB The HNTB Companies	Made	SJL	Date	8/4/2011	Job Number	49633
For Cleveland Innerbelt - Unit 2	Checked	GDH	Date	8/6/2011		
	Backchk'd	SJL	Date	8/7/2011	Sheet No.	
	Rev'd by :	PDB	Date	9/8/2011		
	Chk'd by :	SJL	Date	9/9/2011		
	BkCk'd by :	PDB	Date	9/12/2011		

1-Stringer Standard Diaphragm Design Connection Plate (Lateral Bracing Revision)

(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
A_{nt} =	7.94	in^2
Yielding ΦP_g =	555.16	kips
Fracture ΦP_n =	412.75	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

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

Rupture

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

S_g =	374.80	in^3	S_{net} =	286.50	in^3
A_g =	30.94	in^2	A_{net} =	23.44	in^2

(from Section Properties spreadsheet)

Gross Section Stress			Net Section Stress		
σ_{gross} =	28.4	ksi	σ_{net} =	37.2	ksi
$\Phi_y F_y$ =	47.5	ksi	$\Phi_u F_u$ =	52.0	ksi
Gross Section OK			**Net Section OK**		

D/C = 0.60 0.71

PL's have adequate overcapacity to accommodate oversized holes per HSS RFI - no need to revise

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 = $\Phi R_n = \Phi\ 2.4dF_ut$ (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 = $\Phi R_n = \Phi\ 1.2L_cF_ut$ (eqn. 6.13.2.9-2)

Φ =	0.80	
F_u =	65	(Grade 50, AASHTO 6.4.1-1)
t =	1.000	in
d =	1.000	in
End clear spacing =	1.38	in
clear Bolt spacing =	2.00	in
L_c=	1.38	(shear planes)
ΦR_n =	85.80	kips/bolt

Max Resultant force on bolt = 56.20

Check Bearing at Bolt Holes = **OK**

HNTB The HNTB Companies	Made	SJL	Date	8/5/2011	Job Number	49633
	Checked	GDH	Date	8/6/2011		
For: Cleveland Innerbelt - Unit 2	Backchk'd	SJL	Date	8/7/2011	Sheet No.	
	Rev'd by :	PDB	Date	9/8/2011		
	Rev'd by :	SJL	Date	9/15/2011		
	Chk'd by :	PDB	Date	9/15/2011		

2-Stringer Standard Diaphragm Design
Connection Plate (Lateral Bracing Revision)

(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 =	241.0	kips
Horizontal Force =	225.0	kips
Resultant factored force =	329.71	kips
Yielding in gross section, Φ_y =	0.95	
Fracture in net section, Φu =	0.80	
Gusset Plate thickness =	0.625	in
Bolt hole dia. =	1.2500	in
Rows of Bolts =	14	
Columns of Bolts =	2	
Gusset Plate width =	7.25	in
Gusset Plate Depth =	46.25	in
F_y =	50.0	ksi
Fu =	65.0	ksi

Gusset Plate

A_{gt} =	28.91	in^2
A_{nt} =	17.97	in^2
Yielding ΦP_g =	1373.05	kips
Fracture ΦP_n =	934.38	kips
	OK	

Check Tension and Bending on connection plate

Horizontal Force (ref. conn. Design) =	112.5	kips
Moment (ref. conn. Design) =	14904.0	in-kips

Yielding

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

Rupture

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

S_g =	678.50	in^3	S_{net} =	508.30	in^3
A_g =	47.66	in^2	A_{net} =	32.81	in^2

(from Section Properties spreadsheet)

Gross Section Stress			Net Section Stress		
σ_{gross} =	24.3	ksi	σ_{net} =	32.8	ksi
$\Phi_y F_y$ =	47.5	ksi	$\Phi_u F_u$ =	52.0	ksi
Gross Section OK			**Net Section OK**		

D/C = 0.51 0.63

PL's have adequate overcapacity to accommodate oversized holes per HSS RFI - edge distance increased from 2" to 2 1/4" to meet bearing resistance requirements

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 = $\Phi R_n = \Phi\ 2.4dF_ut$ (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 = $\Phi R_n = \Phi\ 1.2L_cF_ut$ (eqn. 6.13.2.9-2)

Φ =	0.80	
F_u =	65	(Grade 50, AASHTO 6.4.1-1)
t =	0.625	in
d =	1.000	in
End clear spacing =	1.63	in
clear Bolt spacing =	1.98	in
L_c=	1.63	(shear planes)
ΦR_n =	63.38	kips/bolt

Max Resultant force on bolt = 54.80

Check Bearing at Bolt Holes = **OK**

HNTB The HNTB Companies	Made	SJL	Date	8/5/2011	Job Number	49633
	Checked	GDH	Date	8/6/2011		
For Cleveland Innerbelt - Unit 2	Backchk'd	SJL	Date	8/7/2011	Sheet No.	
	Rev'd by :	PDB	Date	9/8/2011		
	Chk'd by :	SJL	Date	9/9/2011		
	BkCk'd by :	PDB	Date	9/12/2011		

Delta Diaphragm Design (Lateral Bracing Revison)
Connection Plate

(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 = 247.0 kips
Horizontal Force = 533.0 kips
Resultant factored force = 587.45 kips

Yielding in gross section, Φ_y = 0.95
Fracture in net section, Φu = 0.80

Gusset Plate thickness = 0.625 in
Bolt hole dia. = 1.2500 in
Rows of Bolts = 17
Columns of Bolts = 2
Gusset Plate width = 7.25 in
Gusset Plate Depth = 56 in

F_y = 50.0 ksi
Fu = 65.0 ksi

Gusset Plate

A_{gt} = 35.00 in^2
A_{nt} = 21.72 in^2

Yielding ΦP_g = 1662.50 kips
Fracture ΦP_n = 1129.38 kips
OK

Check Tension and Bending on connection plate

Horizontal Force (ref. conn. Design) = 266.5 kips
Moment (ref. conn. Design) = 18684.0 in-kips

Yielding	Rupture
$\frac{P}{A_g} + \frac{M}{S_g} \le \phi_y F_y$	$\frac{P}{A_{net}} + \frac{M}{S_{net}} \le \phi_u F_u$
S_g = 817.90 in^3	S_{net} = 482.10 in^3
A_g = 52.46 in^2	A_{net} = 30.82 in^2

(from Section Properties spreadsheet)

Gross Section Stress	Net Section Stress
σ_{gross} = 27.9 ksi	σ_{net} = 47.4 ksi
$\Phi_y F_y$ = 47.5 ksi	$\Phi_u F_u$ = 52.0 ksi
Gross Section OK	**Net Section OK**

D/C = 0.59 0.91

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 = $\Phi R_n = \Phi\ 2.4 d F_u t$ (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 = $\Phi R_n = \Phi\ 1.2 L_c F_u t$ (eqn. 6.13.2.9-2)

Φ =	0.80	
F_u =	65	(Grade 50, AASHTO 6.4.1-1)
t =	0.625	in
d =	1.000	in
End clear spacing =	1.63	in
clear Bolt spacing =	2.00	in
L_c=	1.63	(shear planes)
ΦR_n =	63.38	kips/bolt

Max Resultant force on bolt = 53.90

Check Bearing at Bolt Holes = OK

HNTB
The HNTB Companies

For	Cleveland Innerbelt - Unit 2	Made	SJL	Date	8/5/2011	Job Number 49633
		Checked	GDH	Date	8/6/2011	
		Backchk'd	SJL	Date	8/7/2011	Sheet No.
		Rev'd by :	PDB	Date	9/8/2011	
		Chk'd by :	SJL	Date	9/9/2011	
		BkCk'd by :	PDB	Date	9/12/2011	

N:\49633\Bridges\Design\Final Design\Unit 2\Excel\Diaphragms\final forces\LateralBracingRevision\RFI calcs\[PDB_Copy of Section_Properties_3.xlsx]delta Snet (2)

Delta Diaphragm - Connection Plate S_{net}
Section Properties

	b	h	y_{bar}	Area	$b*h*y_{bar}$	dy	$A(dy)^2$	Ixi
	in	in	in	in2	in3	in	in4	in4
	0.625	1.375	89.5025	0.86	76.916211	42.68	1565.4	0.14
1	0.625	2	87.19	1.25	108.9875	40.37	2036.9	0.42
2	0.625	2	83.94	1.25	104.925	37.12	1722.1	0.42
3	0.625	2	80.69	1.25	100.8625	33.87	1433.8	0.42
4	0.625	2	77.44	1.25	96.8	30.62	1171.8	0.42
5	0.625	2	74.19	1.25	92.7375	27.37	936.2	0.42
6	0.625	2	70.94	1.25	88.675	24.12	727.1	0.42
7	0.625	2	67.69	1.25	84.6125	20.87	544.3	0.42
8	0.625	1.1875	64.90875	0.74	48.174463	18.09	242.8	0.09
9	0.625	1.1875	60.72125	0.74	45.066553	13.90	143.4	0.09
10	0.625	2	57.94	1.25	72.425	11.12	154.5	0.42
11	0.625	2	54.69	1.25	68.3625	7.87	77.4	0.42
12	0.625	2	51.44	1.25	64.3	4.62	26.7	0.42
13	0.625	2	48.19	1.25	60.2375	1.37	2.3	0.42
14	0.625	2	44.94	1.25	56.175	-1.88	4.4	0.42
15	0.625	2	41.69	1.25	52.1125	-5.13	32.9	0.42
16	0.625	2	38.44	1.25	48.05	-8.38	87.8	0.42
	0.625	1.375	35.5025	0.86	30.509961	-11.32	110.1	0.14
				0.00	0	-46.82	0.0	0.00
				0.00	0	-46.82	0.0	0.00
	0.625	1.375	27.2525	0.86	23.420117	-19.57	329.1	0.14
1	0.625	2.17	24.23	1.36	32.861938	-22.59	692.3	0.53
2	0.625	2.17	20.81	1.36	28.223563	-26.01	917.7	0.53
3	0.625	2.17	17.39	1.36	23.585188	-29.43	1174.9	0.53
4	0.625	1.295	14.4075	0.81	11.66107	-32.42	850.4	0.11
5	0.625	1.295	10.1125	0.81	8.1848047	-36.71	1090.7	0.11
6	0.625	2.17	7.13	1.36	9.6700625	-39.69	2136.8	0.53
7	0.625	2.17	3.71	1.36	5.0316875	-43.11	2520.9	0.53
	0.625	1.375	0.6875	0.86	0.5908203	-46.14	1829.1	0.14

Total h	Total A	$\Sigma b*h*y_{bar}$
49.3	30.8	1443.2

from Bottom y_{bar} =	46.823	in
x_{bar} =		in
Area =	30.82	in^2
I_x =	22571	in^4
I_y =		in^4
S_x =	482.1	in^3
rx =	27.06	in
ry =		in
J=		in^4
Zx =		in^3

HNTB The HNTB Companies	Made	SJL	Date	8/7/2011	Job Number	49633
	Checked	GDH	Date	8/8/2011		
For **Cleveland Innerbelt - Unit 2**	Backchk'd	SJL	Date	8/8/2011	Sheet No.	
	Rev'd by :	PDB	Date	9/8/2011		
	Chk'd by :	SJL	Date	9/9/2011		
	BkCk'd by :	PDB	Date	9/12/2011		

Modified Delta Diaphragm Design
(Lateral Bracing Revision)
Connection Plate

(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 =	247.0	kips
Horizontal Force =	533.0	kips
Resultant factored force =	587.45	kips
Yielding in gross section, Φ_y =	0.95	
Fracture in net section, Φu =	0.80	
Gusset Plate thickness =	0.875	in
Bolt hole dia. =	1.2500	in
Rows of Bolts =	17	
Columns of Bolts =	3	
Gusset Plate width =	10.5	in
Gusset Plate Depth =	56	in
F_y =	50.0	ksi
Fu =	65.0	ksi

Gusset Plate

A_{gt} =	49.00	in^2
A_{nt} =	30.41	in^2
Yielding ΦP_g =	2327.50	kips
Fracture ΦP_n =	1581.13	kips
	OK	

Check Tension and Bending on connection plate

Horizontal Force (ref. conn. Design) =	266.5	kips
Moment (ref. conn. Design) =	19,421	in-kips

Yielding

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

Rupture

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

Yielding			Rupture		
S_g =	687.90	in^3	S_{net} =	482.33	in^3
A_g =	54.75	in^2	A_{net} =	37.84	in^2

(from Section Properties spreadsheet)

Gross Section Stress			Net Section Stress		
σ_{gross} =	33.1	ksi	σ_{net} =	47.3	ksi
$\Phi_y F_y$ =	47.5	ksi	$\Phi_u F_u$ =	52.0	ksi
Gross Section OK			**Net Section OK**		

D/C = 0.70 0.91

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 = $\Phi R_n = \Phi\ 2.4dF_ut$ (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 = $\Phi R_n = \Phi\ 1.2L_cF_ut$ (eqn. 6.13.2.9-2)

Φ =	0.80	
F_u =	65	(Grade 50, AASHTO 6.4.1-1)
t =	0.875	in
d =	1.000	in
End clear spacing =	1.38	in
clear Bolt spacing =	2.00	in
L_c=	1.38	(shear planes)
ΦR_n =	75.08	kips/bolt

Max Resultant force on bolt = 55.10

Check Bearing at Bolt Holes = **OK**

HNTB The HNTB Companies	Made	SJL	Date	8/7/2011	Job Number	49633
	Checked	GDH	Date	8/8/2011		
For Cleveland Innerbelt - Unit 2	Backchk'd	SJL	Date	8/8/2011	Sheet No.	
	Rev'd by :	PDB	Date	9/8/2011		
	Chk'd by :	SJL	Date	9/9/2011		
	BkCk'd by :	PDB	Date	9/12/2011		

N:\49633\Bridges\Design\Final Design\Unit 2\Excel\Diaphragms\final forces\LateralBracingRevision\RFI calcs\[PDB_Copy of Section_Properties_3.xlsx]8' delta Snet (2)

Modified Delta Diaphragm - Connection Plate S_{net}

Section Properties

	b	h	y_{bar}	Area	$b*h*y_{bar}$	dy	$A(dy)^2$	Ixi
	in	in	in	in2	in3	in	in4	in4
	0.875	1.875	76.3125	1.64	125.20	36.33	2165.6	0.48
1	0.875	2	73.125	1.75	127.97	33.14	1922.4	0.58
2	0.875	2	69.875	1.75	122.28	29.89	1563.9	0.58
3	0.875	2	66.625	1.75	116.59	26.64	1242.3	0.58
4	0.875	2	63.375	1.75	110.91	23.39	957.7	0.58
5	0.875	2	60.125	1.75	105.22	20.14	710.1	0.58
6	0.875	2	56.875	1.75	99.53	16.89	499.5	0.58
7	0.875	1.125	54.0625	0.98	53.22	14.08	195.2	0.10
8	0.875	1.125	49.9375	0.98	49.16	9.96	97.6	0.10
9	0.875	2	47.125	1.75	82.47	7.14	89.3	0.58
10	0.875	2	43.875	1.75	76.78	3.89	26.5	0.58
11	0.875	2	40.625	1.75	71.09	0.64	0.7	0.58
12	0.875	2	37.375	1.75	65.41	-2.61	11.9	0.58
13	0.875	2	34.125	1.75	59.72	-5.86	60.0	0.58
14	0.875	2	30.875	1.75	54.03	-9.11	145.1	0.58
15	0.875	2	27.625	1.75	48.34	-12.36	267.2	0.58
16	0.875	2	24.375	1.75	42.66	-15.61	426.2	0.58
	0.875	1.875	21.1875	1.64	34.76	-18.79	579.5	0.48
				0.00	0.00	-39.98	0.0	0.00
	0.875	2.125	14.9375	1.86	27.77	-25.04	1166.2	0.70
1	0.875	2.25	11.5625	1.97	22.76	-28.42	1590.0	0.83
2	0.875	1.375	8.5	1.20	10.23	-31.48	1192.4	0.19
3	0.875	1.375	4.125	1.20	4.96	-35.86	1546.8	0.19
	0.875	2.125	1.0625	1.86	1.98	-38.92	2816.3	0.70
		Total h		Total A	$\Sigma b*h*y_{bar}$			
		43.3		37.8	1513.0			

from Bottom y_{bar} =	39.981	in
x_{bar} =		in
Area =	37.84	in^2
I_x =	19284	in^4
I_y =		in^4
S_x =	482.3	in^3
rx =	22.57	in
ry =		in
J=		in^4
Zx =		in^3