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 connection check

Check Level (Mark One):

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

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HNTB The HNTB Companies Engineers Architects	Made	SJL	Date	9/8/2011	Job Number	49633
	Checked	JBT	Date	9/13/2011		
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(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} =	21	
I_x =	8519.2	in^2
I_y =	55.3	in^2
I_p =	8574.6	in^2
eh =	1.55	in
ev =	16.86	in
x_{max} =	1.55	in
y_{max} =	36.33	in
r_{max} =	36.4	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 = $M_u + V_u * e$ =	9673	kip-in
$M*y_{max}/I_p$ =	41.0	kip
$F_x = P_u/n_{bolt}$ =	0.3	kip
F_x =	41.3	kip
$M*x_{max}/I_p$ =	1.7	kip
$F_y = V_u/n_{bolt}$ =	1.3	kip
F_y =	3.1	kip
Resultant F =	41.4	kip

41.4 kip < 43.4 kip, OK

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1-Stringer Standard Diaphragm Connection

Ip calculation

knee-brace bolt row = 3.4500 in

	column 1	column 2	column 3	distance from centroid (dy)	(from centroid) dx	Ixi	Iyi				$y_{bar}i$
row 1	1	1		26.54		1409.2	5.3	19.375	9	34.5	62.875
row 2	1	1		22.669		1027.8	5.3	15.50	9	34.5	59
row 3	1	1		18.794		706.4	5.3	11.63	9	34.5	55.125
row 4	1			14.919		222.6	2.4	7.75	9	34.5	51.25
row 5	1	1		11.044		243.9	5.3	3.88	9	34.5	47.375
row 6	1	1		7.169		102.8	5.3	0.00	9	34.5	43.5
row 7		1		-1.831		3.4	2.9			34.5	34.5
row 8	1			-5.281		27.9	2.4			31.05	31.05
row 9		1		-8.731		76.2	2.9			27.60	27.6
row 10	1			-12.181		148.4	2.4			24.15	24.15
row 11		1		-15.631		244.3	2.9			20.70	20.7
row 12	1			-19.081		364.1	2.4			17.25	17.25
row 13				-22.531		0.0	0.0			13.80	13.8
row 14	1			-25.981		675.0	2.4			10.35	10.35
row 15		1		-29.431		866.2	2.9			6.90	6.9
row 16	1			-32.881		1081.2	2.4			3.45	3.45
row 17		1		-36.331		1319.9	2.9			0.00	4.441E-15
row 18						0.0	0.0				0
row 19						0.0	0.0				0
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 =		21				8519.2	55.3			$\Sigma y_{bar}i*ni/n$=	36.330952

Ip = 8574.6

xbar= 1.548 in

1-Stringer Standard Diaphragm Design
Check web for connection (Final Forces)

(AASHTO 6.13.4)

Check for Block Shear

Horizontal Force =	153.0	kips
Resultant factored force =	153.00	kips

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

(Use thickness of kneebrace, conservative)

web thickness =	0.6250	in
Bolt hole dia. =	1.1250	in
Rows of Bolts =	6	
Columns of Bolts =	2	

Length of Block Shear Failure Planes

Gross Vertical Length =	19.375	in
Gross Horizonal Length =	5.25	in
Net Vertical Length =	13.75	in
Net Horizonal Length =	3.56	in
F_y =	50.0	ksi
Fu =	65.0	ksi

$$Rr = \Phi_{bs}R_p(0.58*F_uA_{vn}+U_{bs}F_uA_{tn}) \leq \Phi_{bs}R_p(0.58*F_yA_{vg}+U_{bs}F_uA_{tn})$$

Block Shear Factor, Φ_{bs} =	0.80
R_p =	1.00
U_{bs} =	0.50

Block Shear Horizontal Force

A_{gv} =	3.28	in^2
A_{nv} =	2.23	in^2
A_{nt} =	8.59	in^2
R_r =	290.6	k

OK for Block Shear

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:

$$\text{Bolt Capacity} = \Phi R_n = \Phi\ 2.4dF_ut \qquad \text{(eqn. 6.13.2.9-1)}$$

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

$$\text{Bolt Capacity} = \Phi R_n = \Phi\ 1.2L_cF_ut \qquad \text{(eqn. 6.13.2.9-2)}$$

Φ =	0.80	
F_u =	65	(Grade 50, AASHTO 6.4.1-1)
t =	0.6250	in
d =	1.000	in
End clear spacing =	1.69	in
clear Bolt spacing =	2.13	in
L_c=	1.69	(shear planes)
ΦR_n =	65.81	kips/bolt

Max Resultant force on bolt = 56.20 kips

Check Bearing at Bolt Holes = **OK**

Need to change web End Clear distance from 2" to 2.25"

1-Stringer Standard Diaphragm (Final Forces) Stiffener/Connection Plate Design

(AASHTO 6.13.4)

Check for Block Shear

Check failure plane of bolts in diaphragm only (not including knee-brace, conservative)

Vertical Force =	148.0	kips
Horizontal Force =	153.0	kips
Resultant factored force =	212.9	kips

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

(Assume 3/4" thickness, conservative)

thickness =	0.750	in
Bolt hole dia. =	1.2500	in
Rows of Bolts =	6	
Columns of Bolts =	2	

Length of Block Shear Failure Planes

Gross Vertical Length =	19.38	in
Gross Horizonal Length =	5.25	in
Net Vertical Length =	13.13	in
Net Horizonal Length =	3.38	in
F_y =	50.0	ksi
Fu =	65.0	ksi

$$Rr = \Phi_{bs}R_p(0.58*F_uA_{vn}+U_{bs}F_uA_{tn}) \le \Phi_{bs}R_p(0.58*F_yA_{vg}+U_{bs}F_uA_{tn})$$

Block Shear Factor, Φ_{bs} =	0.80
R_p =	1.00
U_{bs} =	0.50

Block Shear Horizontal Force

A_{gv} =	7.88	in^2
A_{nv} =	5.06	in^2
A_{nt} =	9.84	in^2
R_r =	408.6	k

OK for Block Shear

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 =	0.7500	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 =	64.35	kips/bolt

Max Resultant force on bolt = 56.20 kips

Check Bearing at Bolt Holes = **OK**

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NY check

Type B Diaphragm Connection
Unit 2 (Lateral Bracing Revision)

Connection Geometry

n_{col} =	3	
s_{col} =	3.25	in
s_{row} =	3.250	in
gap =	0.50	in
d_{edge} =	2.00	in

n_{bolt} =	47	
I_x =	32800.1	in^2
I_y =	121.5	in^2
I_p =	32921.6	in^2
eh =	1.59	in
ev =	32.40	in
x_{max} =	1.59	in
y_{max} =	46.35	in
r_{max} =	46.4	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 =	-176.4	kip
M_u =	2078.0	kip-ft
V_u =	-154.0	kip

Bolt Forces

Service II

Total M = $M_u + V_u * e$ =	27743	kip-in
$M*y_{max}/I_p$ =	39.1	kip
$F_x = P_u/n_{bolt}$ =	3.8	kip
F_x =	42.8	kip
$M*x_{max}/I_p$ =	1.3	kip
$F_y = V_u/n_{bolt}$ =	3.3	kip
F_y =	4.6	kip
Resultant F =	43.1	kip

43.1 kip < 43.4 kip, OK

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Type B Diaphragm Connection

Ip calculation

knee-brace bolt row = 1.7140 in

	column 1	column 2	column 3	distance from centroid (dy)	(from centroid) dx	Ixi	Iyi				$y_{bar}i$
row 1	1	1		40.52	1.63	3284.0	5.3	52	10.875	23.996	86.871
row 2	1	1		37.271	1.625	2778.3	5.3	48.75	10.875	23.996	83.621
row 3	1	1		34.021	1.625	2314.9	5.3	45.50	10.875	23.996	80.371
row 4	1	1		30.771	1.625	1893.8	5.3	42.25	10.875	23.996	77.121
row 5	1	1		27.521	1.625	1514.9	5.3	39.00	10.875	23.996	73.871
row 6	1	1		24.271	1.625	1178.2	5.3	35.75	10.875	23.996	70.621
row 7	1	1		21.021	1.625	883.8	5.3	32.50	10.875	23.996	67.371
row 8	1	1		17.771	1.625	631.6	5.3	29.25	10.875	23.996	64.121
row 9	1			14.521	1.625	210.9	0.0	26.00	10.875	23.996	60.871
row 10	1	1		11.271	1.625	254.1	5.3	22.75	10.875	23.996	57.621
row 11	1	1		8.021	1.625	128.7	5.3	19.50	10.875	23.996	54.371
row 12	1	1		4.771	1.625	45.5	5.3	16.25	10.875	23.996	51.121
row 13	1	1		1.521	1.625	4.6	5.3	13.00	10.875	23.996	47.871
row 14	1	1		-1.729	1.625	6.0	5.3	9.75	10.875	23.996	44.621
row 15	1	1		-4.979	1.625	49.6	5.3	6.50	10.875	23.996	41.371
row 16	1	1		-8.229	1.625	135.4	5.3	3.25	10.875	23.996	38.121
row 17	1	1		-11.479	1.625	263.5	5.3	0.00	10.875	23.996	34.871
row 18		1		-22.354	1.625	499.7	2.8			23.996	23.996
row 19	1			-24.068	1.625	579.2	2.5			22.28	22.282
row 20		1		-25.782	1.625	664.7	2.8			20.57	20.568
row 21	1			-27.496	1.625	756.0	2.5			18.85	18.85
row 22		1		-29.210	1.625	853.2	2.8			17.14	17.14
row 23	1			-30.924	1.625	956.3	2.5			15.43	15.43
row 24		1		-32.638	1.625	1065.2	2.8			13.71	13.71
row 25	1			-34.352	1.625	1180.0	2.5			12.00	12.00
row 26				-36.066	1.625	0.0	0.0			10.28	10.28
row 27	1			-37.780	1.625	1427.3	2.5			8.57	8.57
row 28		1		-39.494	1.625	1559.7	2.8			6.86	6.86
row 29	1			-41.208	1.625	1698.1	2.5			5.14	5.14
row 30		1		-42.922	1.625	1842.3	2.8			3.43	3.43
row 31	1			-44.636	1.625	1992.3	2.5			1.71	1.71
row 32		1		-46.350	1.625	2148.3	2.8			0.00	0.00
row 33											

Total # bolts = 47 32800.1 121.5 $\Sigma y_{bar}i*ni\ /n=$ 46.349553

Ip = 32921.6

xbar= 1.590 in

Delta Diaphragm Design (Lateral Bracing Revision)
Check web for connection

(AASHTO 6.13.4)

Check for Block Shear

Horizontal Force =	533.0	kips
Resultant factored force =	533.00	kips
Yielding in gross section, Φ_y =	0.95	
Fracture in net section, Φu =	0.80	
web thickness =	0.625	in
Bolt hole dia. =	1.1250	in
Rows of Bolts =	17	
Columns of Bolts =	2	

Length of Block Shear Failure Planes

Gross Vertical Length =	52.00	in
Gross Horizonal Length =	5.25	in
Net Vertical Length =	34.00	in
Net Horizonal Length =	3.56	in
F_y =	50.0	ksi
Fu =	65.0	ksi

$$Rr = \Phi_{bs}R_p(0.58*F_uA_{vn}+U_{bs}F_uA_{tn}) \le \Phi_{bs}R_p(0.58*F_yA_{vg}+U_{bs}F_uA_{tn})$$

Block Shear Factor, Φ_{bs} =	0.80
R_p =	1.00
U_{bs} =	0.50

Block Shear
Horizontal Force

A_{gv} =	3.28	in^2
A_{nv} =	2.23	in^2
A_{nt} =	21.25	in^2
R_r =	619.7	k

OK for Block Shear

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:

$$\text{Bolt Capacity} = \Phi R_n = \Phi\ 2.4dF_ut \qquad \text{(eqn. 6.13.2.9-1)}$$

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

$$\text{Bolt Capacity} = \Phi R_n = \Phi\ 1.2L_cF_ut \qquad \text{(eqn. 6.13.2.9-2)}$$

Φ =	0.80	
F_u =	65	(Grade 50, AASHTO 6.4.1-1)
t =	0.6250	in
d =	1.000	in
End clear spacing =	1.44	in
clear Bolt spacing =	2.13	in
L_c=	1.44	(shear planes)
ΦR_n =	56.06	kips/bolt

Max Resultant force on bolt = 53.90 kips

Check Bearing at Bolt Holes = OK

Delta Stiffener/Connection Plate Design (Lateral Bracing Revision)

(AASHTO 6.13.4)

Check for Block Shear

Check failure plane of bolts in diaphragm only (not including knee-brace, conservative)

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	
(Assume 3/4" thickness, conservative)		
thickness =	0.750	in
Bolt hole dia. =	1.2500	in
Rows of Bolts =	17	
Columns of Bolts =	2	

Length of Block Shear Failure Planes

Gross Vertical Length =	52.00	in
Gross Horizonal Length =	5.25	in
Net Vertical Length =	32.00	in
Net Horizonal Length =	3.38	in
F_y =	50.0	ksi
Fu =	65.0	ksi

$$Rr = \Phi_{bs}R_p(0.58*F_uA_{vn}+U_{bs}F_uA_{tn}) \le \Phi_{bs}R_p(0.58*F_yA_{vg}+U_{bs}F_uA_{tn})$$

Block Shear Factor, Φ_{bs} =	0.80
R_p =	1.00
U_{bs} =	0.50

Block Shear Horizontal Force

A_{gv} =	7.88	in^2
A_{nv} =	5.06	in^2
A_{nt} =	24.00	in^2
R_r =	776.7	k

OK for Block Shear

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:

$$\text{Bolt Capacity} = \Phi R_n = \Phi\ 2.4dF_ut \qquad \text{(eqn. 6.13.2.9-1)}$$

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

$$\text{Bolt Capacity} = \Phi R_n = \Phi\ 1.2L_cF_ut \qquad \text{(eqn. 6.13.2.9-2)}$$

Φ =	0.80	
F_u =	65	(Grade 50, AASHTO 6.4.1-1)
t =	0.7500	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 =	64.35	kips/bolt

Max Resultant force on bolt = 53.90 kips

Check Bearing at Bolt Holes = **OK**

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

Connection Geometry

n_{col} =	3	
s_{col} =	3.25	in
s_{row} =	3.250	in
gap =	0.50	in
d_{edge} =	2.00	in

n_{bolt} =	35	
I_x =	17607.0	in^2
I_y =	89.9	in^2
I_p =	17696.9	in^2
ev =	9.90	in
eh =	1.58	in
x_{max} =	1.58	in
y_{max} =	46.60	in
r_{max} =	46.6	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 =	-2.5	kip
M_u =	1276.3	kip-ft
V_u =	36.3	kip

Bolt Forces

Service II

Total M = $M_u + V_u * e$ =	15283	kip-in
$M*y_{max}/I_p$ =	40.2	kip
$F_x = P_u/n_{bolt}$ =	0.1	kip
F_x =	40.3	kip
$M*x_{max}/I_p$ =	1.4	kip
$F_y = V_u/n_{bolt}$ =	1.0	kip
F_y =	2.4	kip
Resultant F =	40.4	kip

40.4 kip < 43.4 kip, OK

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2-Stringer Standard Diaphragm Connection

Ip calculation

knee-brace bolt row = 3.2500 in

	column 1	column 2	column 3	distance from centroid (dy)	(from centroid) dx	Ixi	Iyi				$y_{bar}i$
row 1	1	1		31.02		1924.7	5.3	42.25	9.375	26	77.625
row 2	1	1		27.771		1542.5	5.3	39.00	9.375	26	74.375
row 3	1	1		24.521		1202.6	5.3	35.75	9.375	26	71.125
row 4	1	1		21.271		904.9	5.3	32.50	9.375	26	67.875
row 5	1	1		18.021		649.5	5.3	29.25	9.375	26	64.625
row 6	1	1		14.771		436.4	5.3	26.00	9.375	26	61.375
row 7	1	1		11.521		265.5	5.3	22.75	9.375	26	58.125
row 8	1	1		8.271		136.8	5.3	19.50	9.375	26	54.875
row 9	1	1		5.021		50.4	5.3	16.25	9.375	26	51.625
row 10	1			1.771		3.1	0.0	13.00	9.375	26	48.375
row 11	1	1		-1.479		4.4	5.3	9.75	9.375	26	45.125
row 12	1	1		-4.729		44.7	5.3	6.50	9.375	26	41.875
row 13	1	1		-7.979		127.3	5.3	3.25	9.375	26	38.625
row 14	1	1		-11.229		252.2	5.3	0.00	9.375	26	35.375
row 15		1		-20.604		424.5	2.8			26	26
row 16	1			-23.854		569.0	2.5			22.75	22.75
row 17				-27.104		0.0	0.0			19.50	19.5
row 18	1			-30.354		921.3	2.5			16.25	16.25
row 19		1		-33.604		1129.2	2.8			13.00	13
row 20	1			-36.854		1358.2	2.5			9.75	9.75
row 21		1		-40.104		1608.3	2.8			6.50	6.50
row 22	1			-43.354		1879.5	2.5			3.25	3.25
row 23		1		-46.604		2171.9	2.8			0.00	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 =		35				17607.0	89.9			$\Sigma y_{bar}i*ni\ /n=$	46.603571

Ip = 17696.9

xbar= 1.579

HNTB	Made	SJL	Date	9/15/2011	Job Number	49633
The HNTB Companies	Checked	JBT	Date	9/15/2011		
For Cleveland Innerbelt - Unit 2	Backchk'd	SJL	Date	9/15/2011	Sheet No.	

2-Stringer Standard Diaphragm Design
Check web for connection
(Lateral Bracing Revision)

(AASHTO 6.13.4)

Check for Block Shear

Horizontal Force =	225.0	kips
Resultant factored force =	225.00	kips
Yielding in gross section, Φ_y =	0.95	
Fracture in net section, Φu =	0.80	
web thickness =	0.4375	in
Bolt hole dia. =	1.1250	in
Rows of Bolts =	14	
Columns of Bolts =	2	

Length of Block Shear Failure Planes

Gross Vertical Length =	42.25	in
Gross Horizonal Length =	5.75	in
Net Vertical Length =	27.63	in
Net Horizonal Length =	4.06	in
F_y =	50.0	ksi
Fu =	65.0	ksi

$$Rr = \Phi_{bs}R_p(0.58*F_uA_{vn}+U_{bs}F_uA_{tn}) \le \Phi_{bs}R_p(0.58*F_yA_{vg}+U_{bs}F_uA_{tn})$$

Block Shear Factor, Φ_{bs} =	0.80
R_p =	1.00
U_{bs} =	0.50

Block Shear Horizontal Force

A_{gv} =	2.52	in^2
A_{nv} =	1.78	in^2
A_{nt} =	12.09	in^2
R_r =	367.8	k

OK for Block Shear

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:

$$\text{Bolt Capacity} = \Phi R_n = \Phi\ 2.4dF_ut \qquad \text{(eqn. 6.13.2.9-1)}$$

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

$$\text{Bolt Capacity} = \Phi R_n = \Phi\ 1.2L_cF_ut \qquad \text{(eqn. 6.13.2.9-2)}$$

Φ =	0.80	
F_u =	65	(Grade 50, AASHTO 6.4.1-1)
t =	0.4375	in
d =	1.000	in
End clear spacing =	2.19	in
clear Bolt spacing =	2.13	in
L_c=	2.13	(shear planes)
ΦR_n =	54.60	kips/bolt

Max Resultant force on bolt = 54.60 kips

Check Bearing at Bolt Holes = **OK**

Need to change web End Clear distance from 2.5" to 2.75"

HNTB The HNTB Companies						
	Made	SJL	Date	9/15/2011	Job Number	49633
	Checked	JBT	Date	9/15/2011		
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2-Stringer Standard Diaphragm Stiffener/Connection Plate Design (Lateral Bracing Revision)

(AASHTO 6.13.4)

Check for Block Shear

Check failure plane of bolts in diaphragm only (not including knee-brace, conservative)

Vertical Force =	241.0	kips
Horizontal Force =	225.0	kips
Resultant factored force =	329.7	kips

Yielding in gross section, Φ_y =	0.95	
Fracture in net section, Φu =	0.80	
(Assume 3/4" thickness, conservative)		
thickness =	0.750	in
Bolt hole dia. =	1.2500	in
Rows of Bolts =	14	
Columns of Bolts =	2	

Length of Block Shear Failure Planes

Gross Vertical Length =	42.25	in
Gross Horizonal Length =	5.25	in
Net Vertical Length =	26.00	in
Net Horizonal Length =	3.38	in
F_y =	50.0	ksi
Fu =	65.0	ksi

$$Rr = \Phi_{bs}R_p(0.58*F_uA_{vn}+U_{bs}F_uA_{tn}) \le \Phi_{bs}R_p(0.58*F_yA_{vg}+U_{bs}F_uA_{tn})$$

Block Shear Factor, Φ_{bs} =	0.80
R_p =	1.00
U_{bs} =	0.50

Block Shear Horizontal Force

A_{gv} =	7.88	in^2
A_{nv} =	5.06	in^2
A_{nt} =	19.50	in^2
R_r =	659.7	k

OK for Block Shear

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 =	0.7500	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 =	64.35	kips/bolt

Max Resultant force on bolt = 54.60 kips

Check Bearing at Bolt Holes = **OK**

HNTB	The HNTB Companies Engineers Architects Planners	Made	SJL	Date	9/8/2011	Job Number	49633
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Type D Diaphragm Connection
Unit 2 (Lateral Bracing Revision)

Connection Geometry

n_{col} =	3	
s_{col} =	3.25	in
s_{row} =	3.250	in
gap =	0.50	in
d_{edge} =	2.00	in

n_{bolt} =	61	
I_x =	27964.5	in^2
I_y =	421.8	in^2
I_p =	28386.3	in^2
eh =	3.14	in
ev =	25.19	in
x_{max} =	3.14	in
y_{max} =	38.68	in
r_{max} =	38.8	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 =	-176.4	kip
M_u =	2078.0	kip-ft
V_u =	-154.0	kip

Bolt Forces

Service II

Total M = $M_u + V_u * e$ =	26711	kip-in
$M*y_{max}/I_p$ =	36.4	kip
$F_x = P_u/n_{bolt}$ =	2.9	kip
F_x =	39.3	kip
$M*x_{max}/I_p$ =	3.0	kip
$F_y = V_u/n_{bolt}$ =	2.5	kip
F_y =	5.5	kip
Resultant F =	39.7	kip

39.7 kip < 43.4 kip, OK

HNTB	The HNTB Companies Engineers Architects Planners	Made	SJL	Date	9/8/2011	Job Number	49633
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Type D Diaphragm Connection

Ip calculation

knee-brace bolt row = 3.5000 in

	column 1	column 2	column 3	distance from centroid (dy)	(from centroid) dx	Ixi	Iyi				$y_{bar}i$
row 1	1	1	1	33.32	3.25	3330.6	21.2	52	9.5	10.5	72
row 2	1	1	1	30.070	3.25	2712.6	21.2	48.75	9.5	10.5	68.75
row 3	1	1	1	26.820	3.25	2157.9	21.2	45.50	9.5	10.5	65.5
row 4	1	1	1	23.570	3.25	1666.6	21.2	42.25	9.5	10.5	62.25
row 5	1	1	1	20.320	3.25	1238.7	21.2	39.00	9.5	10.5	59
row 6	1	1	1	17.070	3.25	874.1	21.2	35.75	9.5	10.5	55.75
row 7	1	1	1	13.820	3.25	573.0	21.2	32.50	9.5	10.5	52.5
row 8	1	1	1	10.570	3.25	335.2	21.2	29.25	9.5	10.5	49.25
row 9	1	1		7.320	3.25	107.2	9.9	26.00	9.5	10.5	46
row 10	1	1	1	4.070	3.25	49.7	21.2	22.75	9.5	10.5	42.75
row 11	1	1	1	0.820	3.25	2.0	21.2	19.50	9.5	10.5	39.5
row 12	1	1	1	-2.430	3.25	17.7	21.2	16.25	9.5	10.5	36.25
row 13	1	1	1	-5.680	3.25	96.8	21.2	13.00	9.5	10.5	33
row 14	1	1	1	-8.930	3.25	239.3	21.2	9.75	9.5	10.5	29.75
row 15	1	1	1	-12.180	3.25	445.1	21.2	6.50	9.5	10.5	26.5
row 16	1	1	1	-15.430	3.25	714.3	21.2	3.25	9.5	10.5	23.25
row 17	1	1	1	-18.680	3.25	1046.9	21.1	0.00	9.5	10.5	20
row 18	1	1	1	-28.180	3.25	2382.4	21.2			10.5	10.5
row 19	1	1	1	-31.680	3.25	3010.9	21.2			7.00	7
row 20	1	1		-35.180	3.25	2475.3	9.9			3.50	3.5
row 21	1	1	1	-38.680	3.25	4488.5	21.2			0.00	0.00
row 22					3.25	0.0	0.0				0.00
row 23					3.25	0.0	0.0				0.00
row 24					3.25	0.0	0.0				0.00
row 25					3.25	0.0	0.0				0.00
row 26					3.25	0.0	0.0				0.00
row 27					3.25	0.0	0.0				0.00
row 28					3.25	0.0	0.0				0.00
row 29					3.25	0.0	0.0				0.00
row 30					3.25	0.0	0.0				0.00
row 31					3.25	0.0	0.0				0.00
row 32					3.25	0.0	0.0				0.00
row 33											

HNTB	Made	SJL	Date	9/8/2011	Job Number	49633
The HNTB Companies	Checked	JBT	Date	9/13/2011		
For **Cleveland Innerbelt - Unit 2**	Backchk'd	SJL	Date	9/15/2011	Sheet No.	

Modified Delta Diaphragm Design (Lateral Bracing Revision) Check web for connection

(AASHTO 6.13.4)

Check for Block Shear

Horizontal Force =	533.0	kips
Resultant factored force =	533.00	kips
Yielding in gross section, Φ_y =	0.95	
Fracture in net section, Φu =	0.80	
web thickness =	0.625	in
Bolt hole dia. =	1.1250	in
Rows of Bolts =	17	
Columns of Bolts =	3	

Length of Block Shear Failure Planes

Gross Vertical Length =	52.00	in
Gross Horizonal Length =	8.50	in
Net Vertical Length =	34.00	in
Net Horizonal Length =	5.69	in
F_y =	50.0	ksi
Fu =	65.0	ksi

$$Rr = \Phi_{bs}R_p(0.58*F_uA_{vn}+U_{bs}F_uA_{tn}) \le \Phi_{bs}R_p(0.58*F_yA_{vg}+U_{bs}F_uA_{tn})$$

Block Shear Factor, Φ_{bs} =	0.80
R_p =	1.00
U_{bs} =	0.50

Block Shear Horizontal Force

A_{gv} =	5.31	in^2
A_{nv} =	3.55	in^2
A_{nt} =	21.25	in^2
R_r =	659.7	k

OK for Block Shear

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:

$$\text{Bolt Capacity} = \Phi R_n = \Phi\ 2.4dF_ut \qquad \text{(eqn. 6.13.2.9-1)}$$

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

$$\text{Bolt Capacity} = \Phi R_n = \Phi\ 1.2L_cF_ut \qquad \text{(eqn. 6.13.2.9-2)}$$

Φ =	0.80	
F_u =	65	(Grade 50, AASHTO 6.4.1-1)
t =	0.6250	in
d =	1.000	in
End clear spacing =	1.44	in
clear Bolt spacing =	2.13	in
L_c=	1.44	(shear planes)
ΦR_n =	56.06	kips/bolt

Max Resultant force on bolt = 55.10 kips

Check Bearing at Bolt Holes = **OK**

Modified Delta Stiffener/Connection Plate Design (Lateral Bracing Revision)

(AASHTO 6.13.4)

Check for Block Shear

Check failure plane of bolts in diaphragm only (not including knee-brace, conservative)

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	

thickness =	1.000	in
Bolt hole dia. =	1.2500	in
Rows of Bolts =	17	
Columns of Bolts =	3	

Length of Block Shear Failure Planes

Gross Vertical Length =	52.00	in
Gross Horizonal Length =	8.50	in
Net Vertical Length =	32.00	in
Net Horizonal Length =	5.38	in
F_y =	50.0	ksi
Fu =	65.0	ksi

$$Rr = \Phi_{bs}R_p(0.58*F_uA_{vn}+U_{bs}F_uA_{tn}) \leq \Phi_{bs}R_p(0.58*F_yA_{vg}+U_{bs}F_uA_{tn})$$

Block Shear Factor, Φ_{bs} =	0.80
R_p =	1.00
U_{bs} =	0.50

Block Shear Horizontal Force

A_{gv} =	17.00	in^2
A_{nv} =	10.75	in^2
A_{nt} =	32.00	in^2
R_r =	1156.2	k

OK for Block Shear

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 =	1.0000	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 = 55.10 kips

Check Bearing at Bolt Holes = **OK**

For CLEVELAND	Job no. 49633	Sheet no.
Made by SJL	Checked by DBJ	Backchecked by SJL
Date 9-14-11	Date 9/16/11	Date 9-16-11

HNTB

UNIT 2 - DIAPHRAGMS

ERECTION BOLT COVER PL

→ Determine Required thickness for Erection Bolt cover PL to resist bending under bolt tension force

→ Bolt Tension Force for 1"ø Bolt = 51k (AASHTO Tbl. 6.13.2.8.1)

Area under Washer:

$A = \pi (2")^2/4 = 3.14 in^2$

Area of 1 1/16"ø Hole:

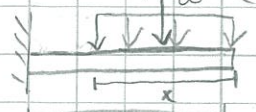
$A = \pi (1\ 1/16")^2/4 = 0.887 in^2$

Net Area under Washer:

$= 3.14 in^2 - 0.887 in^2 = 2.25 in^2$

Distributed Force $= \frac{51k}{2.25 in^2} = 22.7 ksi$

Assume cover PL is a cantilever with 1" width



$m_u = Pb = (22.7 k/in)(0.47")(0.72" - 0.47"/2)$

$m_u = 5.2 k \cdot in$

Assume 50 ksi steel

$50 ksi = \sigma = \frac{m}{S} = \frac{m_u}{bt^2/6} = \frac{5.2 k \cdot in}{(1")(t)^2/6}$

$t \geq 0.79"$

Check Shear:

Shear Area $= \pi (2") t = 6.28" t$

$\phi V_n = \phi 0.58 F_y A \qquad \phi = 1.0$

$\phi V_n = (1)(0.58)(50 ksi)(6.28")t > 51k$

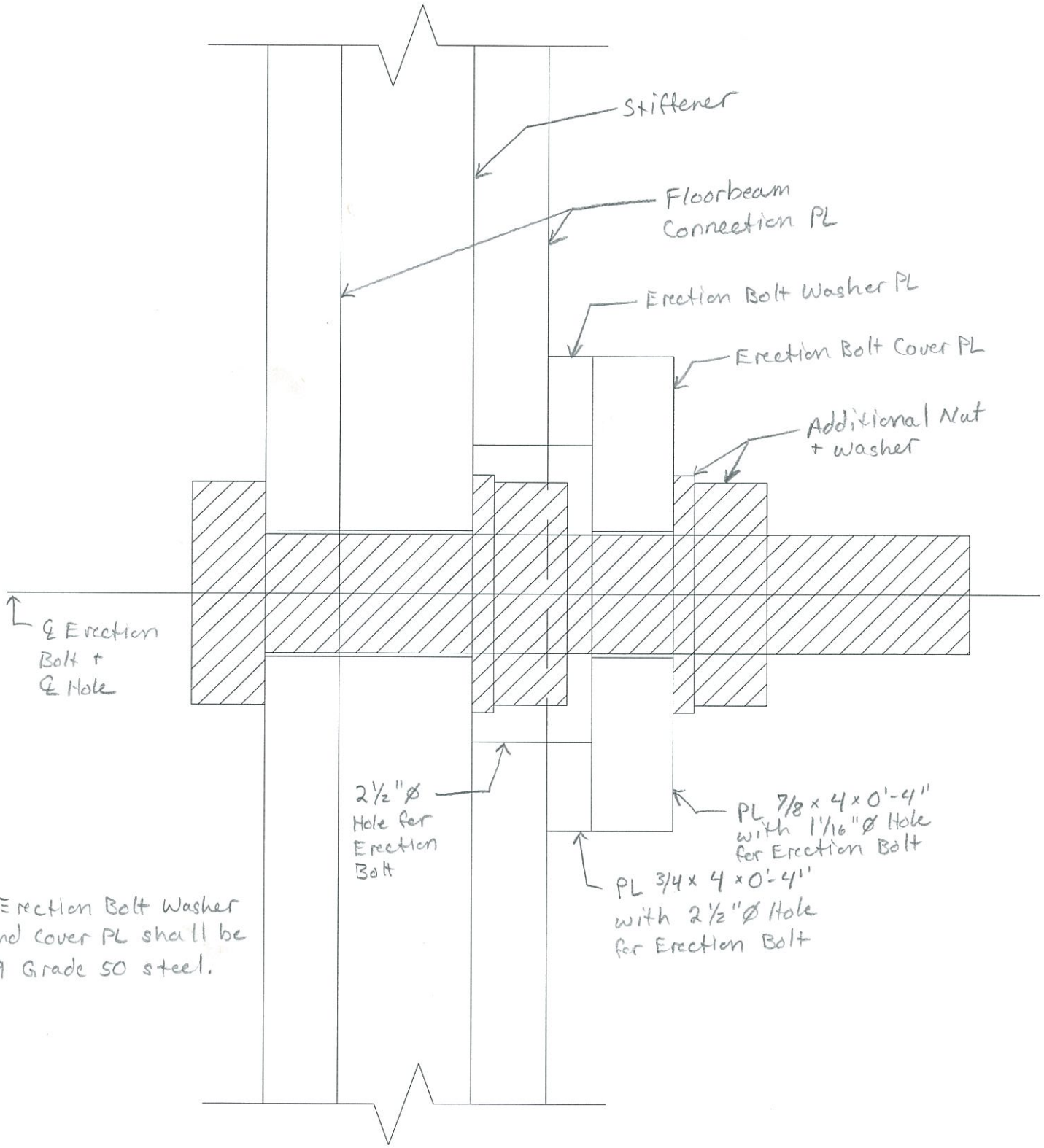
$\Rightarrow t > 0.28"$

→ Use $t = 7/8"$ for Erection Bolt cover PL

ME101-1204

SJL 9-15-11
dBT 9-16-11
SJL 9-16-11

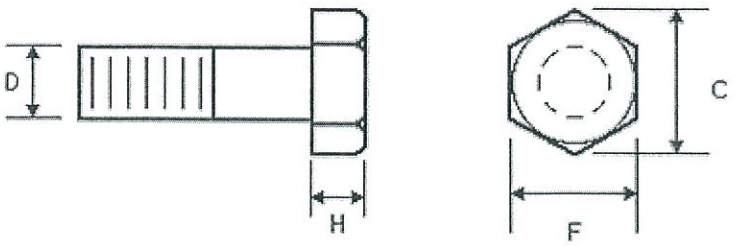
FLOORBEAM TYPE A, B, C, D, E, F, G
Erection Bolts



Note: Erection Bolt Washer PL and Cover PL shall be A709 Grade 50 steel.

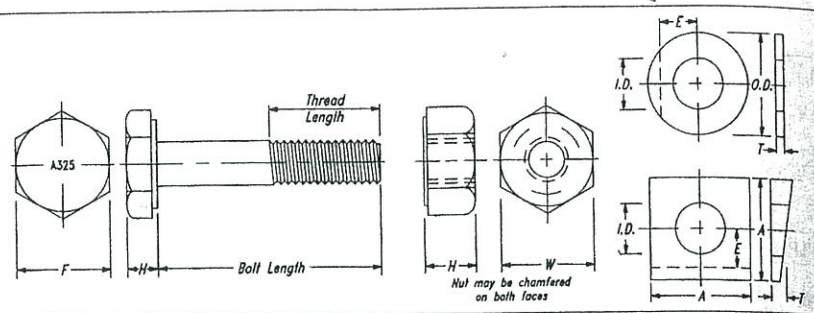
SECTION THRU ERECTION BOLT

Heavy Hex Heads



D	F	C	H
1/2	7/8	1	3/8
5/8	1-1/16	1-1/4	7/16
3/4	1-1/4	1-7/16	1/2
7/8	1-7/16	1-11/16	9/16
1	1-5/8	1-7/8	11/16
1-1/8	1-13/16	2-1/16	3/4
1-1/4	2	2-5/16	7/8
1-3/8	2-3/16	2-1/2	15/16
1-1/2	2-3/8	2-3/4	1
1-3/4	2-3/4	3-3/16	1-3/16
2	3-1/8	3-5/8	1-3/8
2-1/4	3-1/2	4-1/16	1-1/2
2-1/2	3-7/8	4-1/2	1-11/16
2-3/4	4-1/4	4-15/16	1-13/16
3	4-5/8	5-5/16	2
All dimensions in inches			

Table 7–15
Dimensions of High-Strength Fasteners, in.



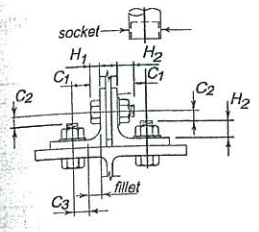
Measurement		Nominal Bolt Diameter, in								
		1/2	5/8	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2
A325 and A490 Bolts[a]	Width Across Flats, *F*	7/8	1 1/16	1 1/4	1 7/16	1 5/8	1 13/16	2	2 3/16	2 3/8
	Height, *H*	5/16	25/64	15/32	35/64	39/64	11/16	25/32	27/32	5/16
	Thread Length	1	1 1/4	1 3/8	1 1/2	1 3/4	2	2	2 1/4	2 1/4
	Bolt Length = Grip + Washer Thickness + →	11/16	7/8	1	1 1/8	1 1/4	1 1/2	1 5/8	1 3/4	1 7/8
A563 Nuts[b]	Width Across Flats, *W*	7/8	1 1/16	1 1/4	1 7/16	1 5/8	1 13/16	2	2 3/16	2 3/8
	Height, *H*	31/64	39/64	47/64	55/64	63/64	1 7/64	1 7/32	1 11/16	1 11/32
F436 Circular Washers[c]	Nom. Outside Diameter, *OD*	1 1/16	1 5/16	1 15/32	1 3/4	2	2 1/4	2 1/2	2 3/4	3
	Nom. Inside Diameter, *ID*	17/32	11/16	13/16	15/16	1 1/8	1 1/4	1 3/8	1 1/2	1 5/8
	Thckns., *T* Min.	0.097	0.122	0.122	0.136	0.136	0.136	0.136	0.136	0.136
	Thckns., *T* Max.	0.177	0.177	0.177	0.177	0.177	0.177	0.177	0.177	0.177
	Min. Edge Distance, *E*[d]	7/16	9/16	21/32	25/32	7/8	1	1 3/32	17/32	15/16
F436 Square or Rect. Washers[c,e]	Min. Side Dimension, *A*	1 3/4	1 3/4	1 3/4	1 3/4	1 3/4	2 1/4	2 1/4	2 1/4	2 1/4
	Mean Thickness, *T*	5/16	5/16	5/16	5/16	5/16	5/16	5/16	5/16	5/16
	Taper in Thickness	2:12	2:12	2:12	2:12	2:12	2:12	2:12	2:12	2:12
	Min. Edge Distance, *E*[d]	7/16	9/16	21/32	25/32	7/8	1	1 3/32	17/32	15/16

[a] Tolerances as specified in ASTM A325 and A490.
[b] Tolerances as specified in ASTM A563.
[c] ASTM F436 Washer Tolerances, in.:

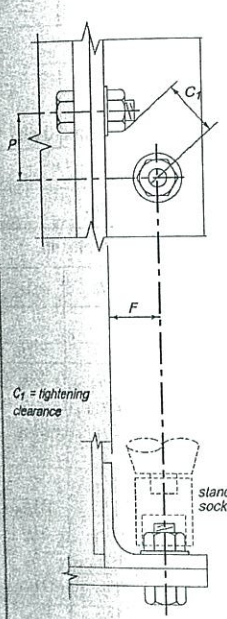
Nominal Outside Diameter	–1/32; +1/32
Nominal Diameter of Hole	–0; +1/32
Flatness: max. deviation from straight-edge placed on cut side shall not exceed	0.010
Concentricity: center of hole to outside diameter (full indicator runout)	0.030
Burr shall not project above immediately adjacent washer surface more than	0.010

[d] For clipped washers only.
[e] For use with American standard beams (S) and channel (C).

Ta
Entering and Tig
Conventional AST



Nomina Bolt Di
5/8
3/4
7/8
1
1 1/8
1 1/4
1 3/8
1 1/2



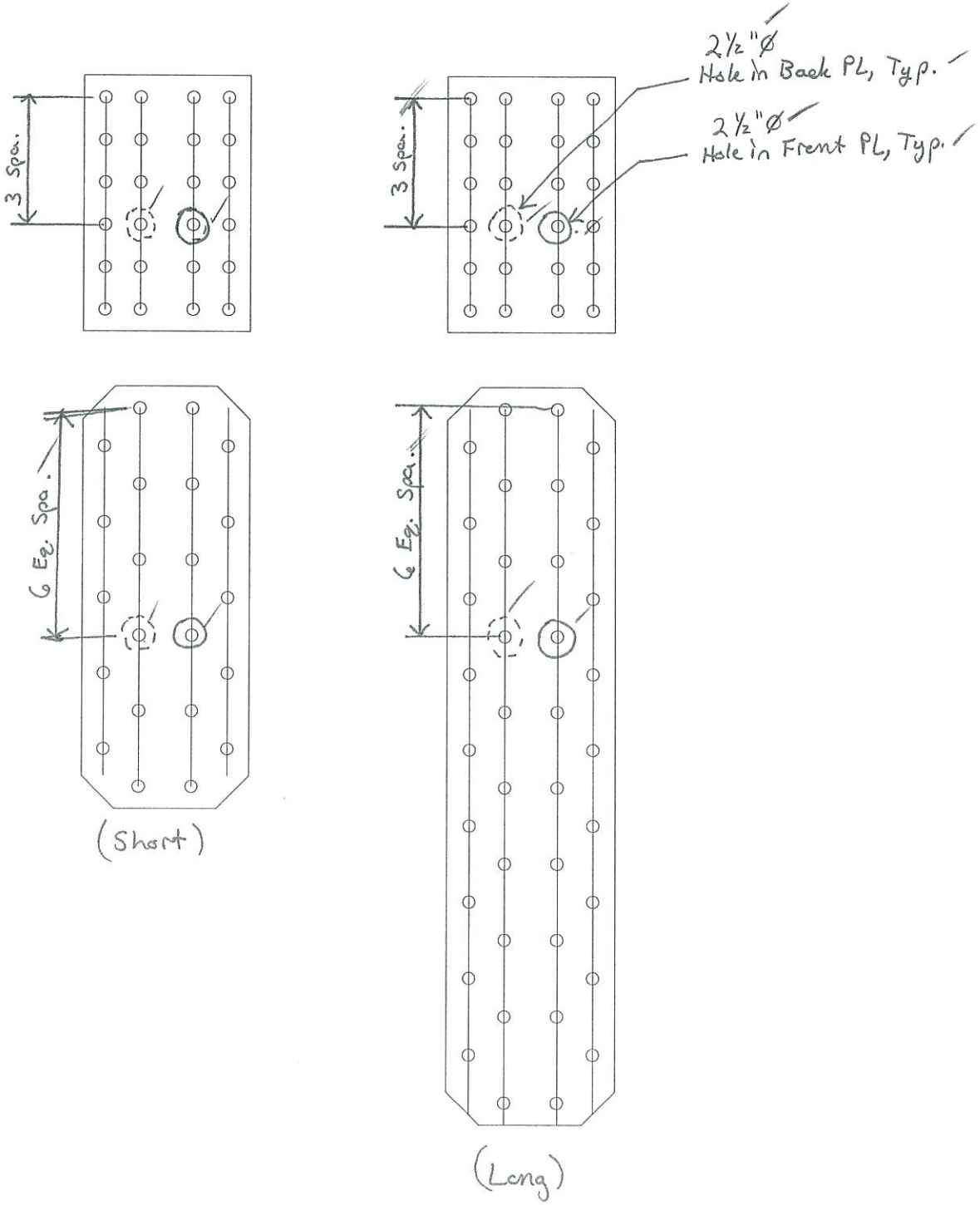
F	5/8
1	1 5/
1 1/8	1 1/
1 1/4	1 1/
1 3/8	1 7/
1 1/2	1 1/
1 5/8	1 1/
1 3/4	1 3/
1 7/8	1 1/
2	1
2 1/8	1 3/
2 1/4	
2 3/8	
2 1/2	
2 5/8	
2 3/4	
2 7/8	
3	
3 1/8	
3 1/4	
3 3/8	
3 1/2	
3 5/8	
3 3/4	
3 7/8	
4	

Notes:
H_1 = height o
H_2 = maximu
C_1 = clearanc
C_2 = clearanc

SJL 9-13-11
JBT 9/13/2011

Type A

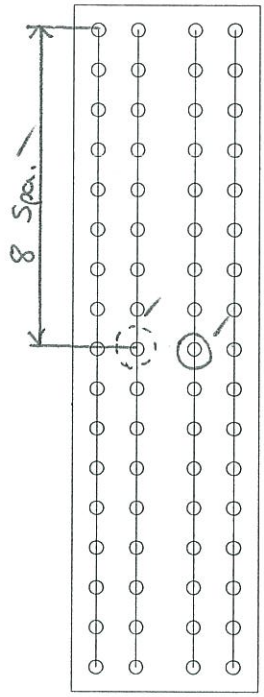
Location of Erection Bolts shown

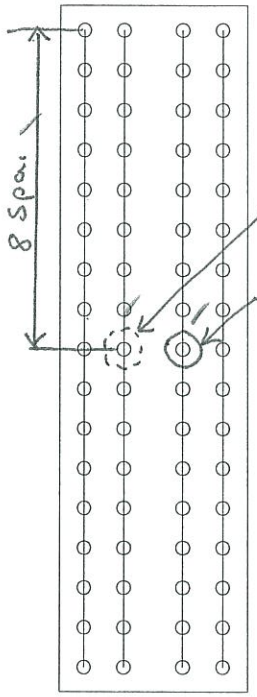


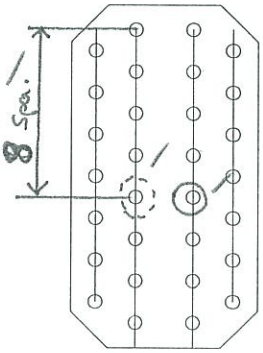
SJL 9-13-11
JBT 9/13/2011

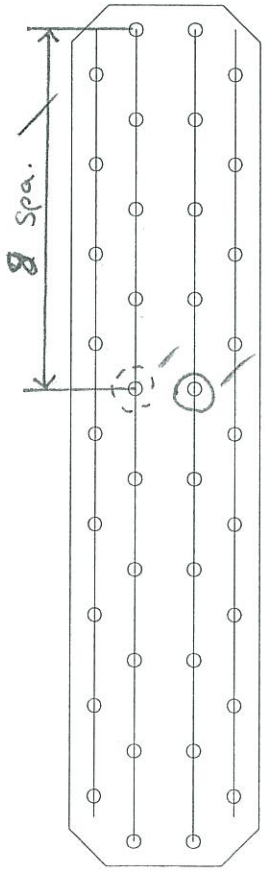
Type B

Location of Erection Bolts





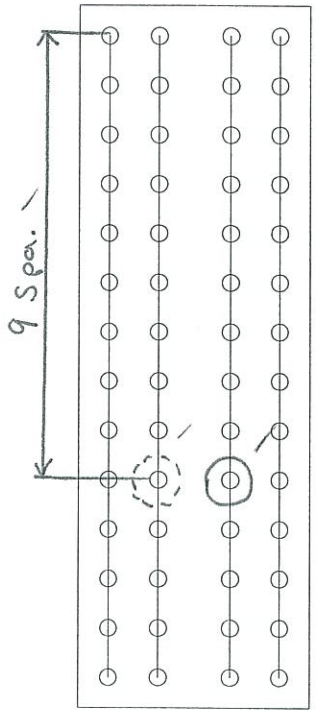


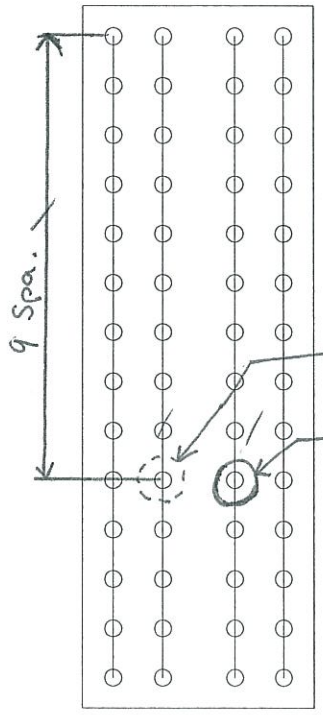


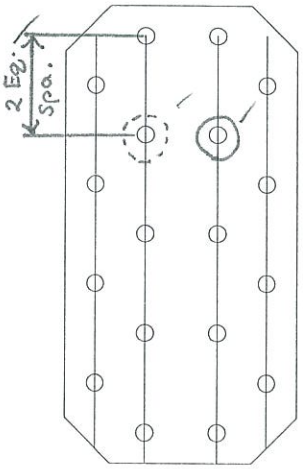
SJL 9-15-11
JBT 9-15-11

Type C

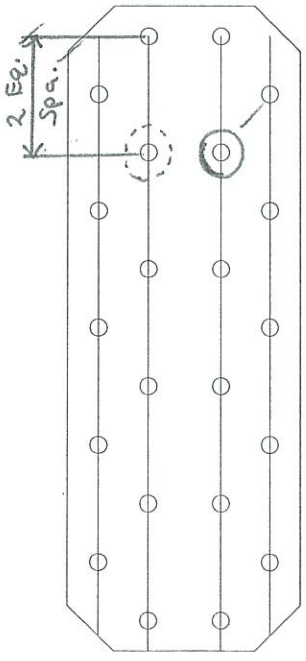
Location of Erection Bolts shown







(Short)

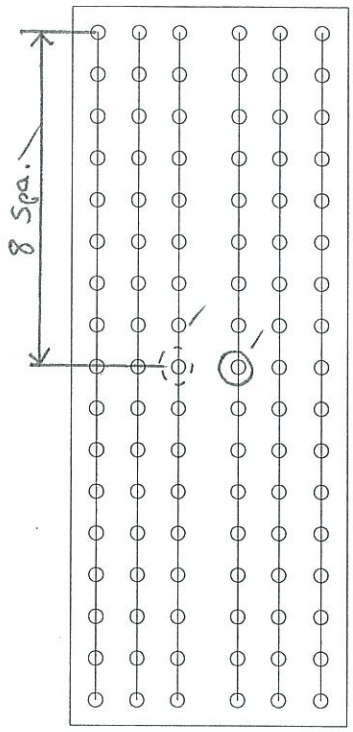


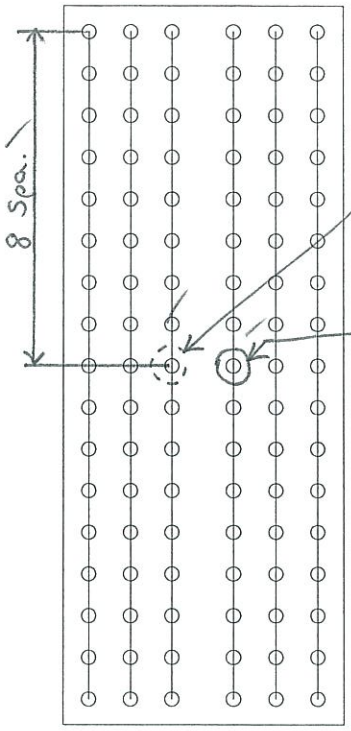
(Long)

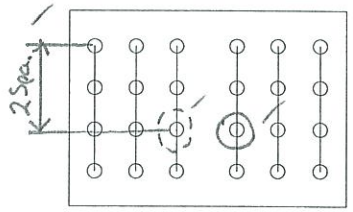
Type D, E, F

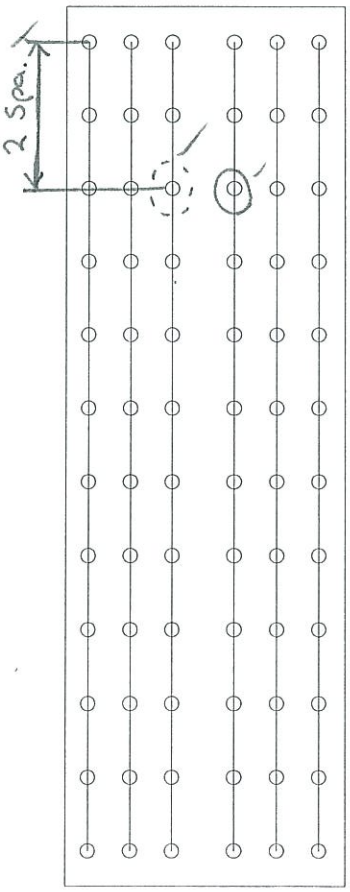
SJL 9-13-11
JBT 9/13/2011

Location of Erection Bolts.





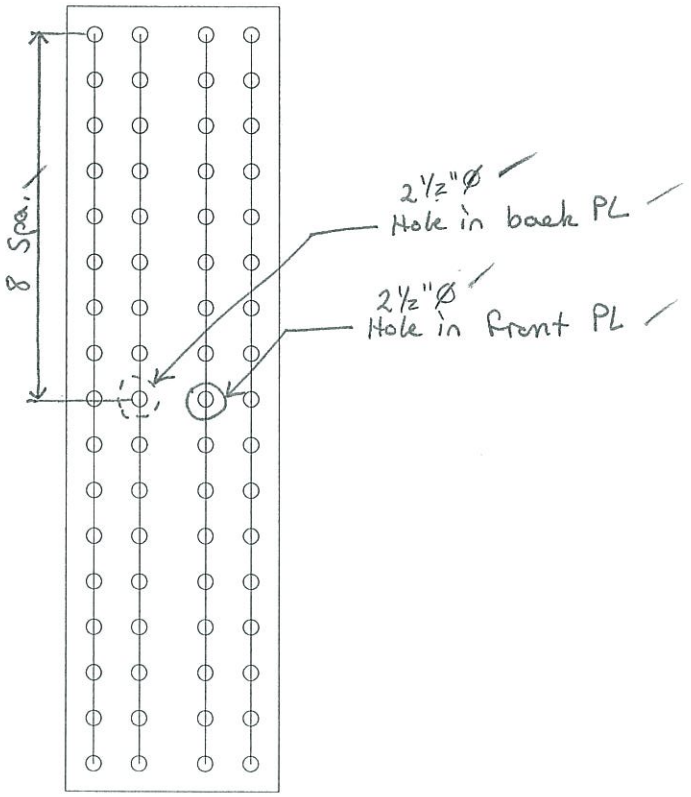




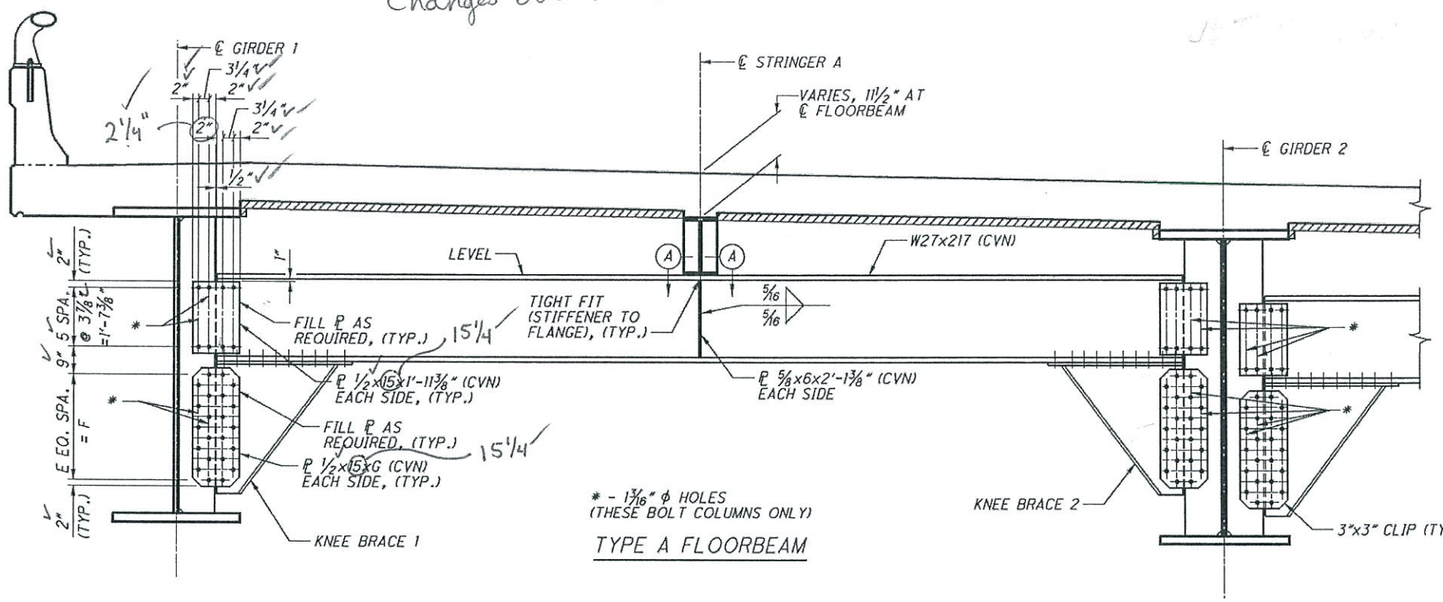
SJL 9-13-11
JBT 9/13/11

Type G

Location of Erection Bolts.

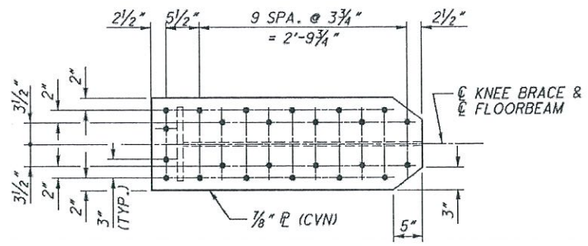


Changes due to Erection Bolt RFI

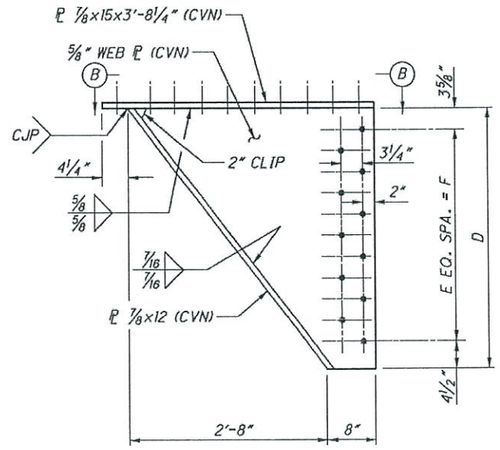


* - 1⅜" ϕ HOLES (THESE BOLT COLUMNS ONLY)

TYPE A FLOORBEAM



VIEW B-B

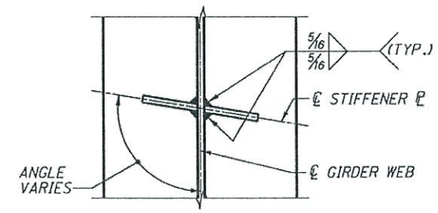


KNEE BRACE DETAIL

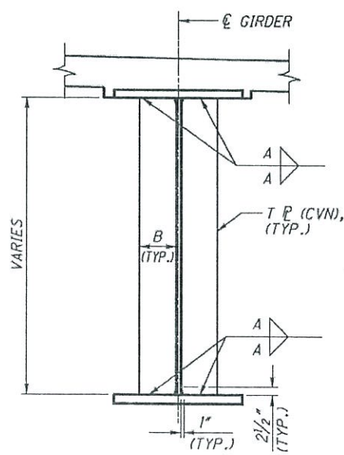
TABLE OF DIMENSIONS GIRDERS 2, 3 AND 4 STIFFENERS			
GIRDER WEB DEPTH	T	B	A
LESS THAN 11'-0"	⅞"	1'-0"	5/16"

TABLE OF DIMENSIONS GIRDERS 1 AND 5 STIFFENERS			
GIRDER WEB DEPTH	T	B	A
LESS THAN 10'-0"	⅞"	1'-0"	⅜"
10'-0" TO 11'-0"	1⅛"	1'-4½"	⅜"

GIRDERS 1 & 5 SHALL ONLY HAVE INSIDE STIFFENERS

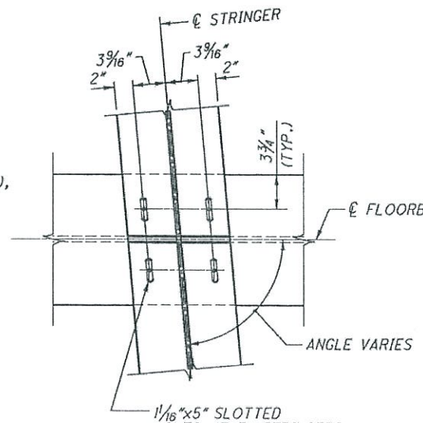


WELDING DETAIL



STIFFENER DETAIL

(GIRDERS 2-4 SHOWN) (CONNECTION TO FLOORBEAM NOT SHOWN FOR CLARITY)



SECTION A-A

see attached for Erection Bolt locations

TABLE OF DIMENSIONS - KNEE BRACES								
	D		E		F		G	
FLOORBEAM TYPE	KB 1	KB 2	KB 1	KB 2	KB 1	KB 2	KB 1	KB 2
A1	3'-10⅛"	3'-10⅛"	12	12	3'-2"	3'-2"	3'-6"	3'-6"
A2	5'-11⅛"	5'-11⅛"	16	16	5'-3"	5'-3"	5'-7"	5'-7"
A3	3'-6⅝"	3'-10⅛"	10	12	2'-10½"	3'-2"	3'-2½"	3'-6"
A4	3'-10⅛"	4'-11⅛"	12	14	3'-2"	4'-3"	3'-6"	4'-7"
A5	4'-4⅛"	4'-11⅛"	14	14	3'-8"	4'-3"	4'-0"	4'-7"
A6	4'-11⅛"	5'-11⅛"	14	16	4'-3"	5'-3"	4'-7"	5'-7"
A7	4'-11⅛"	4'-11⅛"	14	14	4'-3"	4'-3"	4'-7"	4'-7"
A8	3'-6⅝"	4'-4⅛"	10	14	2'-10½"	3'-8"	3'-2½"	4'-0"
A9	4'-4⅛"	5'-11⅛"	14	16	3'-8"	5'-3"	4'-0"	5'-7"

KB 1 DENOTES KNEE BRACE 1
KB 2 DENOTES KNEE BRACE 2

NOTES:

1. WHERE A SHAPE OR PLATE IS DESIGNATED (CVN), FURNISH MATERIAL THAT MEETS THE MINIMUM NOTCH TOUGHNESS REQUIREMENTS AS SPECIFIED IN 711.01.
2. FLOORBEAMS SHALL BE DETAILED, FABRICATED AND INSTALLED SO THAT THEY ARE UNLOADED AT ERECTION, ALLOWING GIRDER WEBS TO BE THEORETICALLY VERTICAL OR PLUMB UNDER THE NO LOAD CONDITION.
3. ALL STEEL SHALL BE A709 GRADE 50.
4. ALL STEEL SHALL BE PAINTED WITH AN IZEU COATING SYSTEM. SURFACES OF BOLTED CONNECTIONS SHALL BE PAINTED WITH A COATING THAT WILL PROVIDE CLASS B SLIP RESISTANCE AS DEFINED IN SECTION 6.13.2.8 OF THE AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS.
5. ALL BOLTS SHALL BE HIGH STRENGTH 1" ϕ, A325 GALVANIZED TYPE 1 BOLTS. ALL HOLES ARE 1⅛" ϕ UNLESS NOTED OTHERWISE.
6. FOR FLOORBEAM LOCATIONS, SEE SHEETS 15/84 THRU 18/84 .
7. STIFFENERS THAT EXTEND BEYOND EDGE OF FLANGE SHALL BE CLIPPED AT A 45° ANGLE.
8. BOLTS AT STRINGER TO FLOORBEAM CONNECTION SHALL BE SNUG TIGHT PRIOR TO DECK PLACEMENT. NUTS SHALL BE FULLY TIGHTENED AFTER DECK IS PLACED.
9. STRINGERS SHALL BE PLACED WITH BOLTS CENTERED IN THE SLOTTED HOLE.

PRELIMINARY NOT FOR CONSTRUCTION

ENGINEER'S SEAL

NO.	REVISIONS	DATE
B	FINAL SUBMITTAL	04/14/11
C	FINAL SUBMITTAL - REVISED	06/09/11
D	FINAL SUBMITTAL - REVISED	08/09/11

DESIGN AGENCY HNTB
WALSH CONSTRUCTION

CUYAHOGA INNERBELT BRIDGES

MAIN SPAN
UNIT 2
STRUCTURAL STEEL

SUPERSTRUCTURE DETAILS
BRIDGE NO. CUY-90-1532
I-90 WEST BOUND

DESIGNED	DRAWN	REVIEWED	DATE
SJL	TKT	HRH	08/09/11
CHECKED	CHECKED	STRUCTURE FILE NUMBER	
CDH	JPG	1809431	

CUY-90-14.90
PID No. 77332/85531

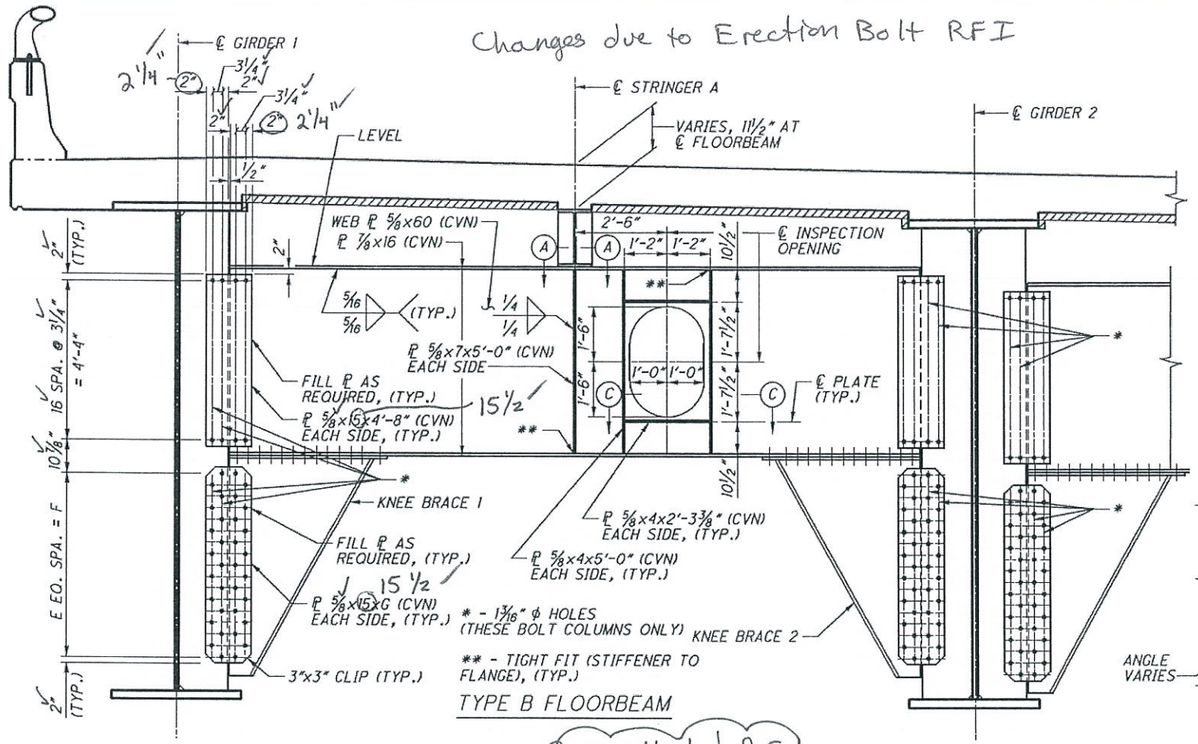
34/84

0/0

S.J.LARSON 9-12-11
UST 9-16-11

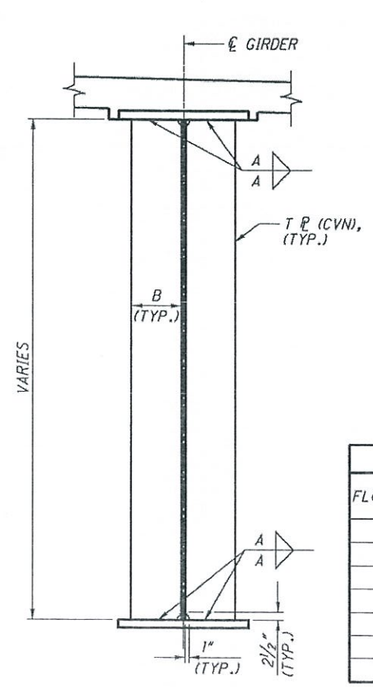
Date:9/9/2011
Model:Sheet1
File:49633-S-BR-SSD05-U2S1Steel.dgn

Changes due to Erection Bolt RFI



TYPE B FLOORBEAM

see attached for location of Erection Bolts



STIFFENER DETAIL

(GIRDERS 2-4 SHOWN)
(CONNECTION TO FLOORBEAM NOT SHOWN FOR CLARITY)

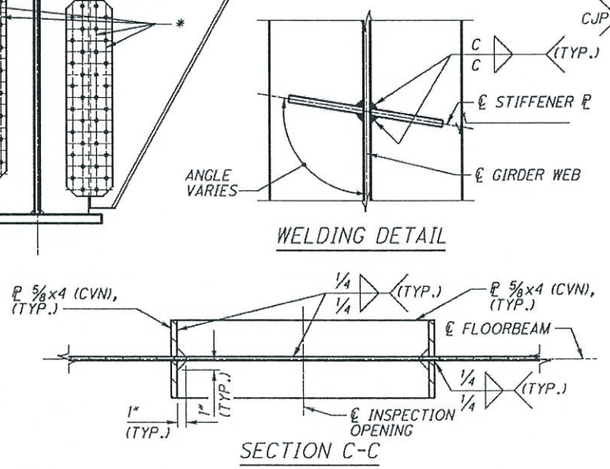
TABLE OF DIMENSIONS GIRDERS 2, 3 AND 4 STIFFENERS				
GIRDER WEB DEPTH	T	B	A	C
LESS THAN 15'-6"	½"	1'-3"	5/16"	5/16"

TABLE OF DIMENSIONS GIRDERS 1 AND 5 STIFFENERS				
GIRDER WEB DEPTH	T	B	A	C
LESS THAN 12'-0"	¾"	1'-6"	9/16"	7/16"
12'-0" TO 15'-6"	¾"	1'-6"	⅜"	5/16"

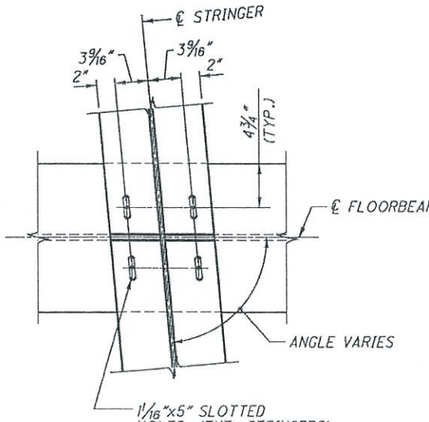
GIRDER 1 AND 5 SHALL ONLY HAVE INSIDE STIFFNERS

TABLE OF DIMENSIONS - KNEE BRACES								
FLOORBEAM TYPE	D		E		F		G	
	KB 1	KB 2	KB 1	KB 2	KB 1	KB 2	KB 1	KB 2
B1	6'-5⅛"	6'-5⅛"	18	18	5'-6"	5'-6"	5'-10"	5'-10"
B2	2'-11⅛"	3'-7⅛"	14	16	2'-0"	2'-8"	2'-4"	3'-0"
B3	3'-7⅛"	4'-11⅛"	16	16	2'-8"	4'-0"	3'-0"	4'-4"
B4	4'-11⅛"	6'-5⅛"	16	18	4'-0"	5'-6"	4'-4"	5'-10"
B5	6'-5⅛"	4'-11⅛"	18	16	5'-6"	4'-0"	5'-10"	4'-6"
B6	4'-11⅛"	3'-7⅛"	16	16	4'-0"	2'-8"	4'-4"	3'-0"
B7	3'-7⅛"	2'-11⅛"	16	14	2'-8"	2'-0"	3'-0"	2'-4"

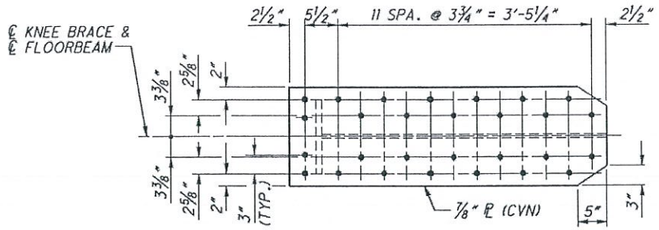
KB 1 DENOTES KNEE BRACE 1
KB 2 DENOTES KNEE BRACE 2



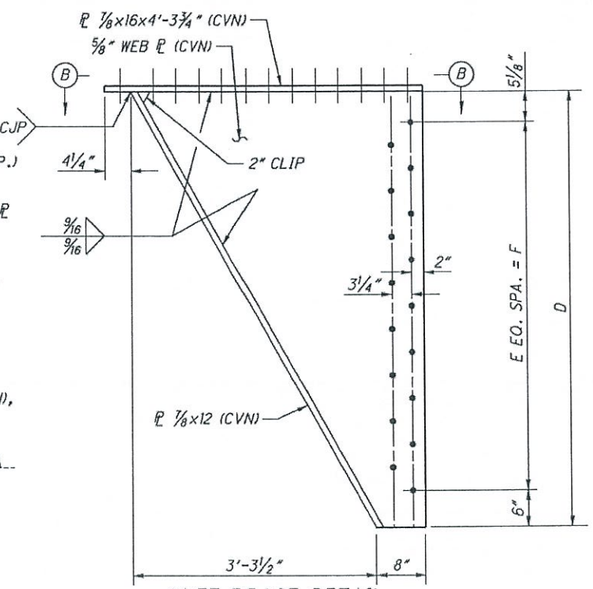
SECTION C-C



SECTION A-A



VIEW B-B



KNEE BRACE DETAIL

WELDING DETAIL

NOTES:

1. WHERE A SHAPE OR PLATE IS DESIGNATED (CVN), FURNISH MATERIAL THAT MEETS THE MINIMUM NOTCH TOUGHNESS REQUIREMENTS AS SPECIFIED IN 711.01.
2. FLOORBEAMS SHALL BE DETAILED, FABRICATED AND INSTALLED SO THAT THEY ARE UNLOADED AT ERECTION, ALLOWING GIRDER WEBS TO BE THEORETICALLY VERTICAL OR PLUMB UNDER THE NO LOAD CONDITION.
3. ALL STEEL SHALL BE A709 GRADE 50.
4. ALL STEEL SHALL BE PAINTED WITH AN IZEU COATING SYSTEM. SURFACES OF BOLTED CONNECTIONS SHALL BE PAINTED WITH A COATING THAT WILL PROVIDE CLASS B SLIP RESISTANCE AS DEFINED IN SECTION 6.13.2.8 OF THE AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS.
5. ALL BOLTS SHALL BE HIGH STRENGTH 1" ϕ, A325 GALVANIZED TYPE 1 BOLTS. ALL HOLES ARE 1⅛" ϕ UNLESS NOTED OTHERWISE.
6. FOR FLOORBEAM LOCATIONS, SEE SHEETS 15/84 THRU 18/84 .
7. STIFFENERS THAT EXTEND BEYOND EDGE OF FLANGE SHALL BE CLIPPED AT A 45° ANGLE.
8. BOLTS AT STRINGER TO FLOORBEAM CONNECTION SHALL BE SNUG TIGHT PRIOR TO DECK PLACEMENT. NUTS SHALL BE FULLY TIGHTENED AFTER DECK IS PLACED.
9. STRINGERS SHALL BE PLACED WITH BOLTS CENTERED IN THE SLOTTED HOLES.

ENGINEER'S SEAL

NO.	REVISIONS	DATE
B	FINAL SUBMITTAL	04/14/11
C	FINAL SUBMITTAL - REVISED	06/09/11
D	FINAL SUBMITTAL - REVISED	08/09/11

DESIGN AGENCY WALSH HNTB WALSH CONSTRUCTION

CLEVELAND BANNERBELT BRIDGE CO. PROFESSIONAL ENGINEERS

MAIN SPAN UNIT 2
STRUCTURAL STEEL
SUPERSTRUCTURE DETAILS
BRIDGE NO. CUY-90-1532
I-90 WEST BOUND

DESIGNED	DRAWN	REVIEWED	DATE
SJL	FWG	HRH	08/09/11
CHECKED	CHECKED	STRUCTURE FILE NUMBER	
GDH	JPG	1809431	

CUY-90-14.90
PID No. 77332/85531

35/84

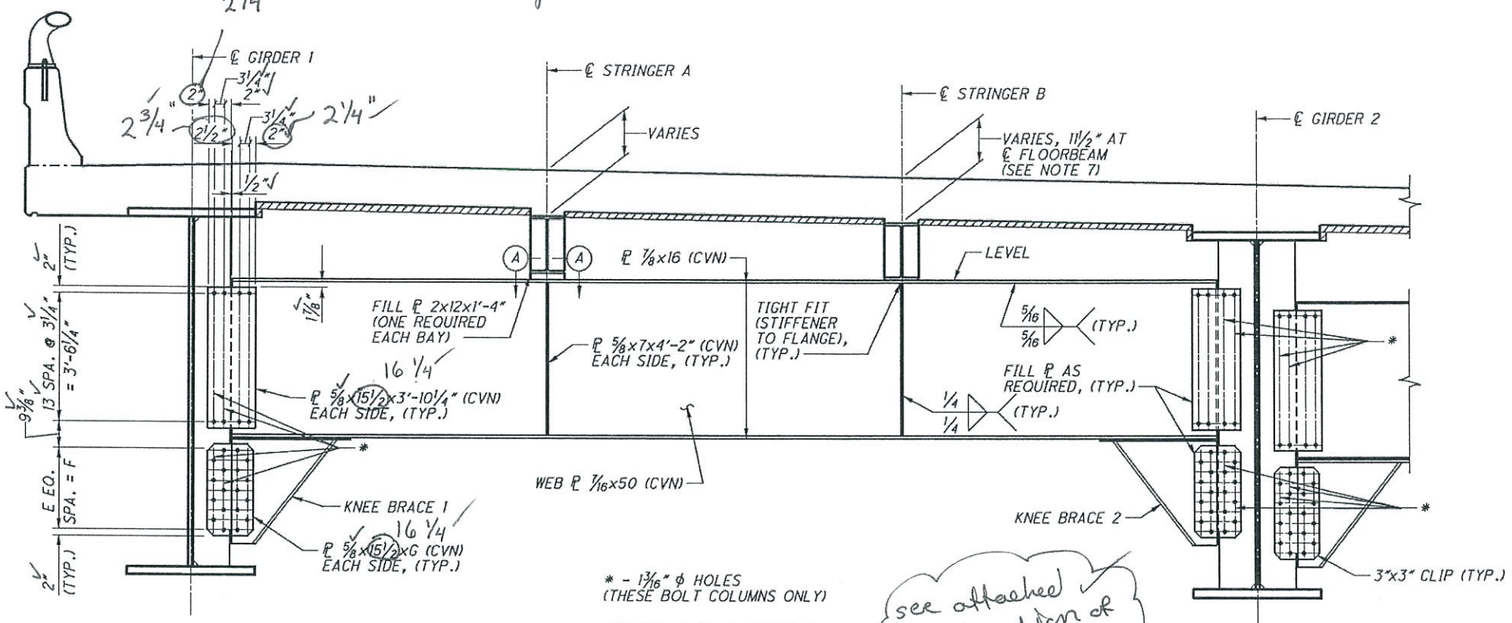
0/0

SJL ARSON 9-12-11
JBT 9-16-11

Date:9/9/2011
Model:Sheet1
File:49633-S-BR-SSD06-U2StSteel.dgn

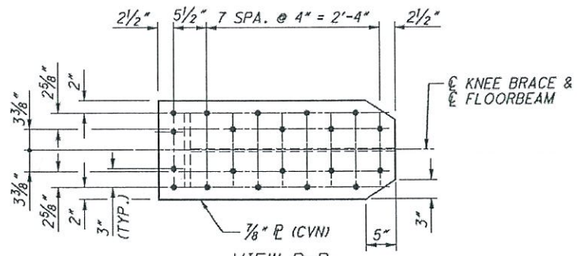
SJLARSON 9-12-11
DBT 9-15-2011

Changes due to Erection Bolt RFI

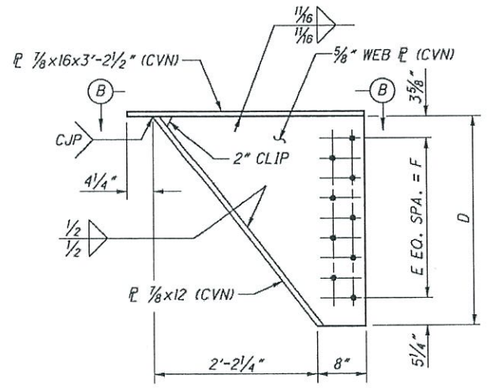


TYPE C FLOORBEAM

see attached for location of Erection Bolts



VIEW B-B

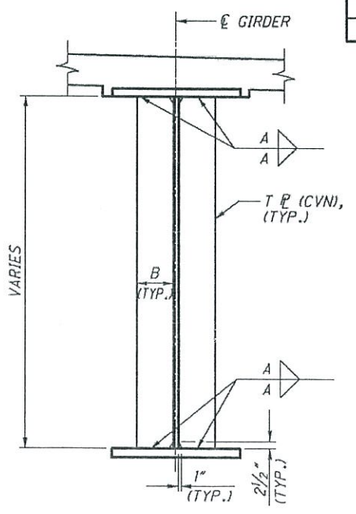


KNEE BRACE DETAIL

TABLE OF DIMENSIONS GIRDERS 2, 3 AND 4 STIFFENERS			
GIRDER WEB DEPTH	T	B	A
LESS THAN 11'-0"	1⅛"	1'-0"	5/16"

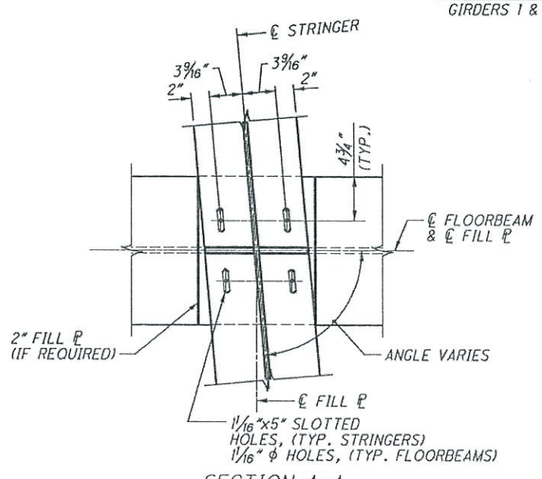
TABLE OF DIMENSIONS GIRDERS 1 AND 5 STIFFENERS			
GIRDER WEB DEPTH	T	B	A
LESS THAN 10'-0"	1⅛"	1'-0"	7/16"
10'-0" TO 11'-0"	1⅝"	1'-4½"	7/16"

GIRDERS 1 & 5 SHALL ONLY HAVE INSIDE STIFFENERS

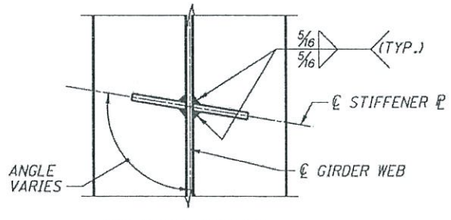


STIFFENER DETAIL

(GIRDERS 2-4 SHOWN)
(CONNECTION TO FLOORBEAM NOT SHOWN FOR CLARITY)



SECTION A-A



WELDING DETAIL

TABLE OF DIMENSIONS - KNEE BRACES								
FLOORBEAM TYPE	D		E		F		G	
	KB 1	KB 2	KB 1	KB 2	KB 1	KB 2	KB 1	KB 2
C1	2'-10⅛"	2'-10⅛"	8	8	2'-2"	2'-2"	2'-6"	2'-6"
C2	3'-11⅛"	3'-11⅛"	10	10	3'-2¼"	3'-2¼"	3'-6¼"	3'-6¼"

KB 1 DENOTES KNEE BRACE 1
KB 2 DENOTES KNEE BRACE 2

NOTES:

1. WHERE A SHAPE OR PLATE IS DESIGNATED (CVN), FURNISH MATERIAL THAT MEETS THE MINIMUM NOTCH TOUGHNESS REQUIREMENTS AS SPECIFIED IN 711.01.
2. FLOORBEAMS SHALL BE DETAILED, FABRICATED AND INSTALLED SO THAT THEY ARE UNLOADED AT ERECTION, ALLOWING GIRDER WEBS TO BE THEORETICALLY VERTICAL OR PLUMB UNDER THE NO LOAD CONDITION.
3. ALL STEEL SHALL BE A709 GRADE 50.
4. ALL STEEL SHALL BE PAINTED WITH AN IZEU COATING SYSTEM. SURFACES OF BOLTED CONNECTIONS SHALL BE PAINTED WITH A COATING THAT WILL PROVIDE CLASS B SLIP RESISTANCE AS DEFINED IN SECTION 6.13.2.8 OF THE AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS.
5. ALL BOLTS SHALL BE HIGH STRENGTH 1" ϕ, A325 GALVANIZED TYPE 1 BOLTS. ALL HOLES ARE 1⅛" ϕ UNLESS NOTED OTHERWISE.
6. FOR FLOORBEAM LOCATIONS, SEE SHEETS 15/84 THRU 18/84 .
7. MINIMUM SLAB + HAUNCH THICKNESS SHALL BE 11½" AT ℄ FLOORBEAM FOR STRINGERS B, D, F AND G.
8. STIFFENERS THAT EXTEND BEYOND THE EDGE OF FLANGE SHALL BE CLIPPED AT A 45° ANGLE.
9. BOLTS AT STRINGER TO FLOORBEAM CONNECTION SHALL BE SNUG TIGHT PRIOR TO DECK PLACEMENT. NUTS SHALL BE FULLY TIGHTENED AFTER DECK IS IN PLACE.
10. STRINGERS SHALL BE PLACED WITH BOLTS CENTERED IN THE SLOTTED HOLE.

PRELIMINARY NOT FOR CONSTRUCTION

ENGINEER'S SEAL

NO.	REVISIONS	DATE
B	FINAL SUBMITTAL	04/14/11
C	FINAL SUBMITTAL - REVISED	06/09/11
D	FINAL SUBMITTAL - REVISED	08/09/11

DESIGN AGENCY WALSH HNTB

CLEVELAND INNERBELT BRIDGE I-90 OHIO DEPARTMENT OF TRANSPORTATION

MAIN SPAN UNIT 2
STRUCTURAL STEEL
SUPERSTRUCTURE DETAILS
BRIDGE NO. CUY-90-1532
I-90 WEST BOUND

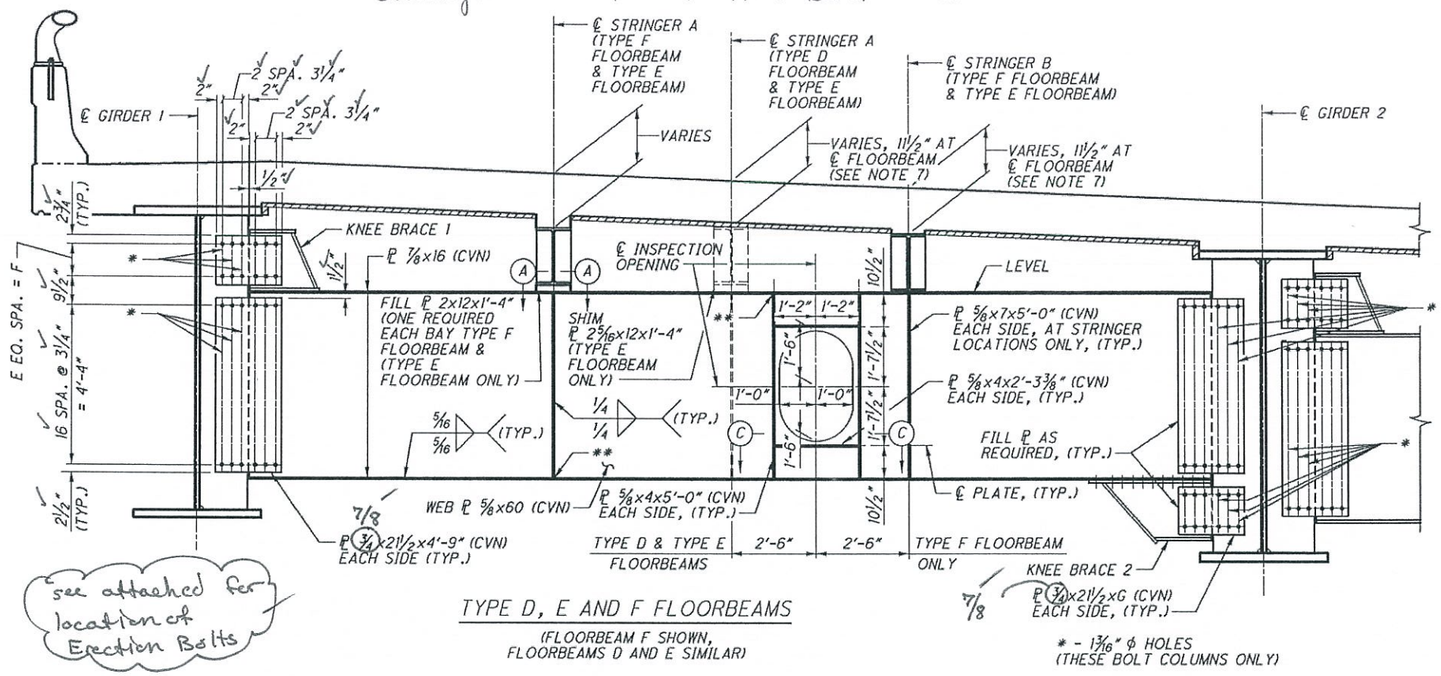
DESIGNED	DRAWN	CHECKED	REVIEWED	DATE	STRUCTURE FILE NUMBER
SJL	FWG	GDH	HRH	08/09/11	1809431
CHECKED GDH					

CUY-90-14.90
PID No. 77332/85531

36/84

Date:9/9/2011
Model:Sheet1
File=49633-S-BR-SSD07-U2StSteel.dgn

Changes due to Erection Bolt RFI



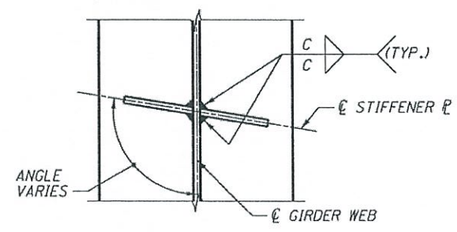
see attached for location of Erection Bolts

TYPE D, E AND F FLOORBEAMS

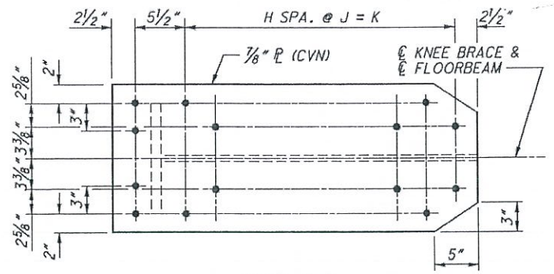
(FLOORBEAM F SHOWN, FLOORBEAMS D AND E SIMILAR)

* - 1 3/16" ϕ HOLES (THESE BOLT COLUMNS ONLY)

** - TIGHT FIT (STIFFENER TO FLANGE), (TYP.)

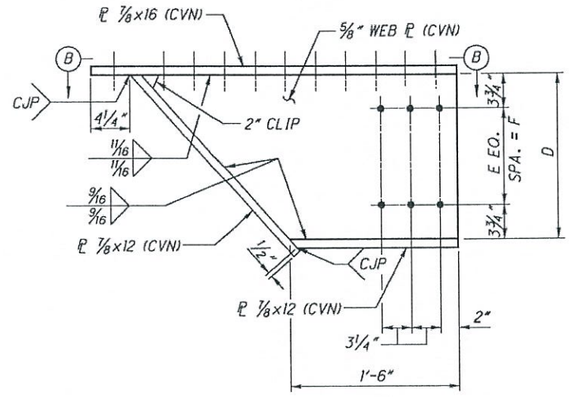


WELDING DETAIL



VIEW B-B

(KNEE BRACE 2 DETAIL SHOWN, KNEE BRACE 1 OPPOSITE HAND)



KNEE BRACE 2 DETAIL

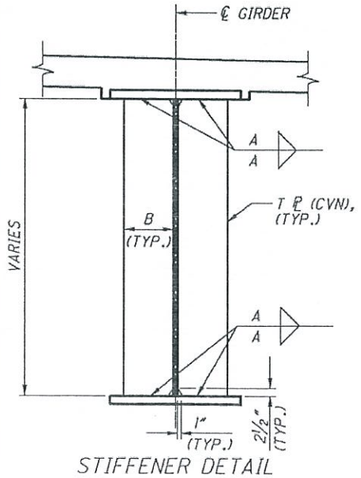
TABLE OF DIMENSIONS GIRDERS 2, 3 AND 4 STIFFENERS

GIRDER WEB DEPTH	T	B	A	C
8'-0"	1⅛"	1'-0"	7/16"	7/16"
8'-0" TO 10'-0"	1½"	1'-3"	7/16"	7/16"
10'-0" TO 15'-6"	1½"	1'-3"	5/16"	5/16"

TABLE OF DIMENSIONS GIRDERS 1 AND 5 STIFFENERS

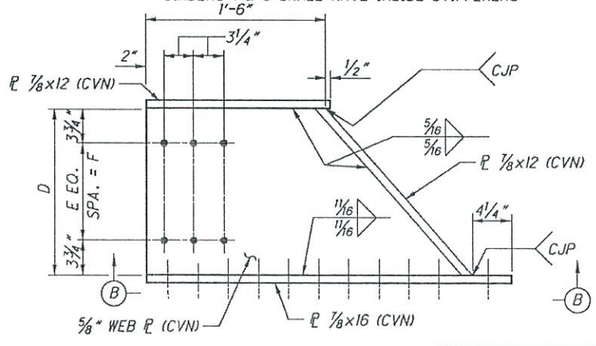
GIRDER WEB DEPTH	T	B	A	C
8'-0"	1⅛"	1'-0"	9/16"	7/16"
8'-0" TO 12'-0"	1¾"	1'-6"	9/16"	7/16"
12'-0" TO 15'-6"	1¾"	1'-6"	⅜"	5/16"

GIRDERS 1 & 5 SHALL HAVE INSIDE STIFFENERS



STIFFENER DETAIL

(GIRDERS 2-4 SHOWN) (CONNECTION TO FLOORBEAM NOT SHOWN FOR CLARITY)



KNEE BRACE 1 DETAIL

TABLE OF DIMENSIONS KNEE BRACE CONNECTION (KB1 & KB2)

D	H	J	K
1'-6"	9	3¼"	2'-5¼"
2'-10¼"	11	3¾"	3'-5¼"
5'-10¼"	11	3¾"	3'-5¼"

TABLE OF DIMENSIONS - KNEE BRACES

FLOORBEAM TYPE	D		E		F		G	
	KB 1	KB 2	KB 1	KB 2	KB 1	KB 2	KB 1	KB 2
D1, E1 AND F1	1'-6"	1'-6"	3	3	10½"	10½"	1'-4"	1'-4"
D2	1'-6"	2'-10¼"	3	5	10½"	2'-2¾"	1'-4"	2'-8¼"
F7	1'-6"	5'-10¼"	3	11	10½"	5'-2¾"	1'-4"	5'-8¼"

KB 1 DENOTES KNEE BRACE 1
KB 2 DENOTES KNEE BRACE 2

NOTES:

1. WHERE A SHAPE OR PLATE IS DESIGNATED (CVN), FURNISH MATERIAL THAT MEETS THE MINIMUM NOTCH TOUGHNESS REQUIREMENTS AS SPECIFIED IN 711.01.
2. FLOORBEAMS SHALL BE DETAILED, FABRICATED AND INSTALLED SO THAT THEY ARE UNLOADED AT ERECTION, ALLOWING GIRDER WEBS TO BE THEORETICALLY VERTICAL OR PLUMB UNDER THE NO LOAD CONDITION.
3. ALL STEEL SHALL BE A709 GRADE 50.
4. ALL STEEL SHALL BE PAINTED WITH AN IZEU COATING SYSTEM. SURFACES OF BOLTED CONNECTIONS SHALL BE PAINTED WITH A COATING THAT WILL PROVIDE CLASS B SLIP RESISTANCE AS DEFINED IN SECTION 6.13.2.8 OF THE AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS.
5. ALL BOLTS SHALL BE HIGH STRENGTH 1" ϕ, A325 GALVANIZED TYPE I BOLTS. ALL HOLES ARE 1 1/16" ϕ UNLESS NOTED OTHERWISE.
6. FOR FLOORBEAM LOCATIONS, SEE SHEETS 15|84 THRU 18|84 .
7. MINIMUM SLAB + HAUNCH THICKNESS SHALL BE 11½" FOR STRINGERS A, C, E AND G AT ℄ TYPE D FLOORBEAMS. MINIMUM SLAB + HAUNCH THICKNESS SHALL BE 11½" FOR STRINGERS B, D, F AND H AT ℄ TYPE E & F FLOORBEAMS. FOR TYPE E FLOORBEAMS, CENTER STRINGER SHALL BE SUPPORTED ON A SHIM ℄ TO MAINTAIN 11½" MINIMUM SLAB + HAUNCH THICKNESS.
8. STIFFENERS THAT EXTEND BEYOND THE EDGE OF FLANGE SHALL BE CLIPPED AT A 45° ANGLE.
9. BOLTS AT STRINGER TO FLOORBEAM CONNECTION SHALL BE SNUG TIGHT PRIOR TO DECK PLACEMENT. NUTS SHALL BE FULLY TIGHTENED AFTER DECK IS PLACED.
10. FOR SECTIONS A-A AND C-C, SEE SHEET 38|84 .
11. STRINGERS SHALL BE PLACED WITH BOLTS CENTERED IN THE SLOTTED HOLES.

PRELIMINARY NOT FOR CONSTRUCTION

ENGINEER'S SEAL

NO.	REVISIONS	DATE
B	FINAL SUBMITTAL	04/14/11
C	FINAL SUBMITTAL - REVISED	06/09/11
D	FINAL SUBMITTAL - REVISED	08/09/11

DESIGN AGENCY WILLIAM HNTB

CLEVELAND INNERBELT BRIDGE

DEPARTMENT OF TRANSPORTATION

MAIN SPAN UNIT 2

STRUCTURAL STEEL

SUPERSTRUCTURE DETAILS

BRIDGE NO. CUY-90-1532 I-90 WEST BOUND

DESIGNED	DRAWN	CHECKED	REVIEWED	DATE
SJL	FWG	GDH	HRH	06/09/11

CHECKED GDH

STRUCTURE FILE NUMBER 1809431

CUY-90-14.90

PID No. 77332/85531

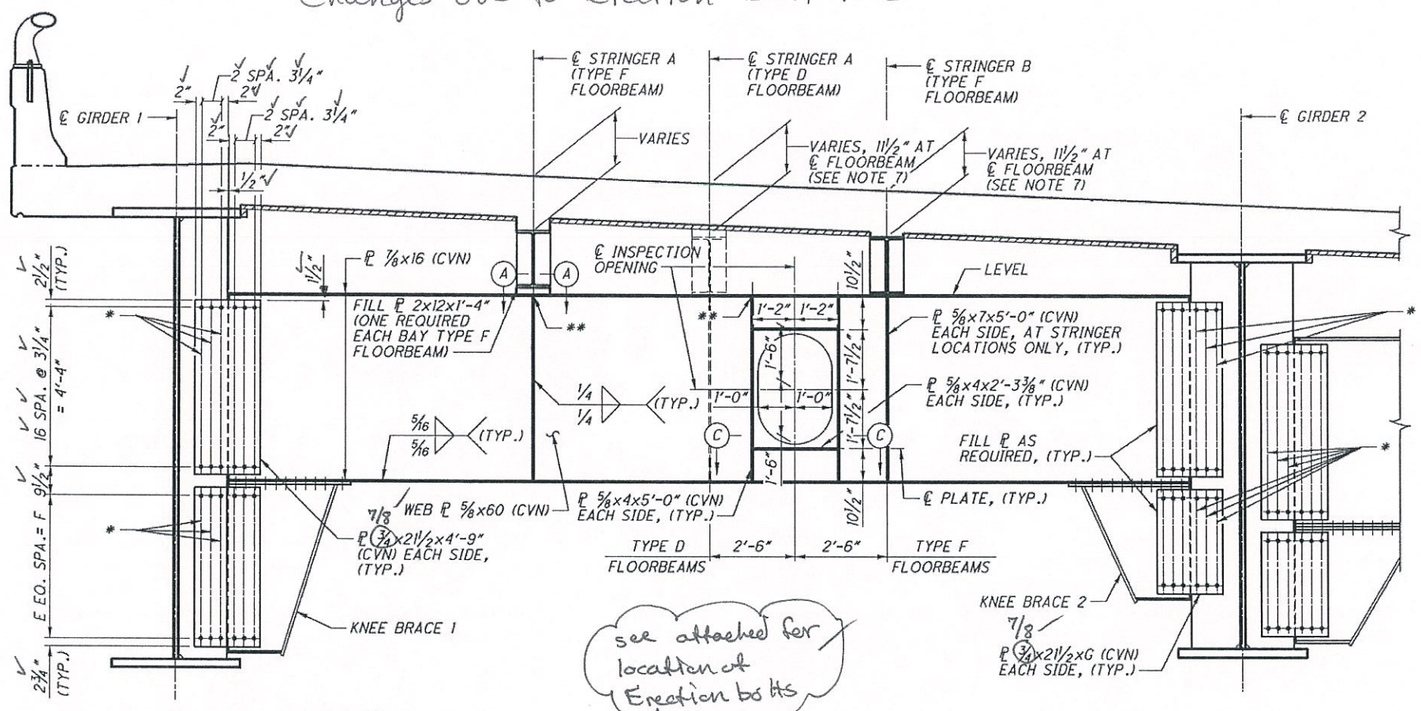
37/84

0/0

SJLARSON 9-12-11
UBT 9-16-11

Date:9/9/2011
Model:Sheet1
File:49633-S-BR-SSD08-U2StSteel.dgn

Changes due to Erection Bolt RFI



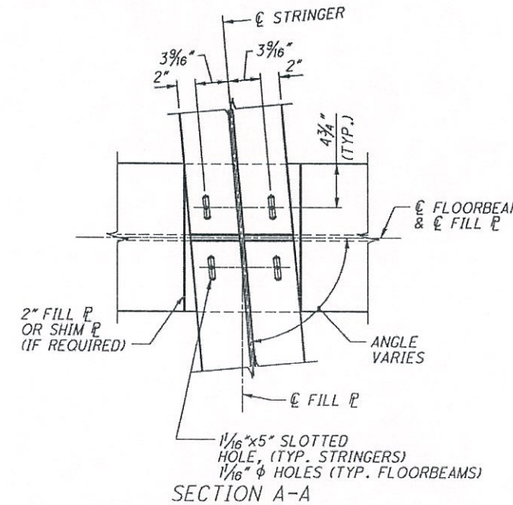
see attached for location of Erection bolts

TYPE D AND F FLOORBEAMS

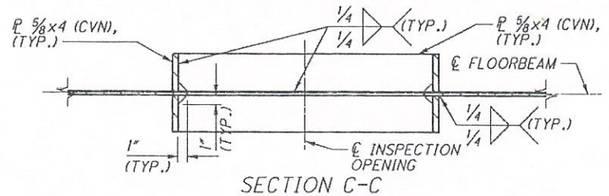
(FLOORBEAM TYPE F SHOWN, FLOORBEAM TYPE D SIMILAR)

* - 1¹³⁄₁₆" ϕ HOLES (THESE BOLT COLUMNS ONLY)

** - TIGHT FIT (STIFFENER TO FLANGE), (TYP.)



SECTION A-A

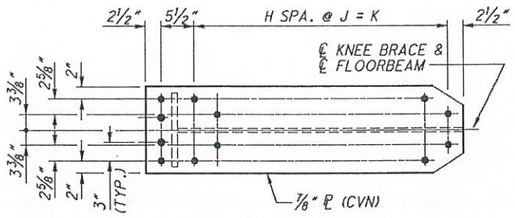


SECTION C-C

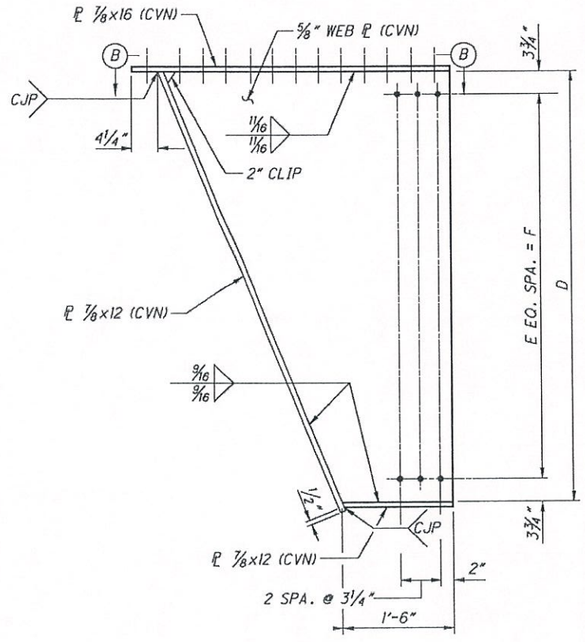
TABLE OF DIMENSIONS KNEE BRACE CONNECTION (KB1 AND KB2)			
D	H	J	K
1'-6"	9	3¼"	2'-5¼"
2'-3¾"	9	3¼"	2'-5¼"
2'-10¼"	11	3¾"	3'-5¼"
4'-4¼"	11	3¾"	3'-5¼"
5'-10¼"	11	3¾"	3'-5¼"

TABLE OF DIMENSIONS - KNEE BRACES								
FLOORBEAM TYPE	D		E		F		G	
	KB 1	KB 2	KB 1	KB 2	KB 1	KB 2	KB 1	KB 2
D3	2'-10¼"	4'-4¼"	5	9	2'-2¾"	3'-8¾"	2'-8¼"	4'-2¼"
D4	4'-4¼"	5'-10¼"	9	11	3'-8¾"	5'-2¾"	4'-2¼"	5'-8¼"
D5	5'-10¼"	1'-6"	11	3	5'-2¾"	10½"	5'-8¼"	1'-4"
D6	2'-3¾"	4'-4¼"	5	9	1'-8¼"	3'-8¾"	2'-1¾"	4'-2¼"
F8	5'-10¼"	5'-10¼"	11	11	5'-2¾"	5'-2¾"	5'-8¼"	5'-8¼"
F9	4'-4¼"	4'-4¼"	9	9	3'-8¾"	3'-8¾"	4'-2¼"	4'-2¼"
F10	5'-10¼"	4'-4¼"	11	9	5'-2¾"	3'-8¾"	5'-8¼"	4'-2¼"
F11	2'-10¼"	2'-3¾"	5	5	2'-2¾"	1'-8¼"	2'-8¼"	2'-1¾"
F12	2'-3¾"	2'-3¾"	5	5	1'-8¼"	1'-8¼"	2'-1¾"	2'-1¾"

KB 1 DENOTES KNEE BRACE 1
KB 2 DENOTES KNEE BRACE 2



VIEW B-B



KNEE BRACE DETAIL

NOTE:

1. FOR NOTES AND DETAILS, SEE SHEET 37/84.

PRELIMINARY NOT FOR CONSTRUCTION

ENGINEER'S SEAL

NO.	REVISIONS	DATE
B	FINAL SUBMITTAL	04/14/11
C	FINAL SUBMITTAL - REVISED	06/09/11
D	FINAL SUBMITTAL - REVISED	08/09/11

DESIGN AGENCY: WALSH HNTB

MAIN SPAN UNIT 2
STRUCTURAL STEEL

SUPERSTRUCTURE DETAILS

BRIDGE NO. CUY-90-1532
I-90 WEST BOUND

CUY-90-14.90
PID No. 77332/85531

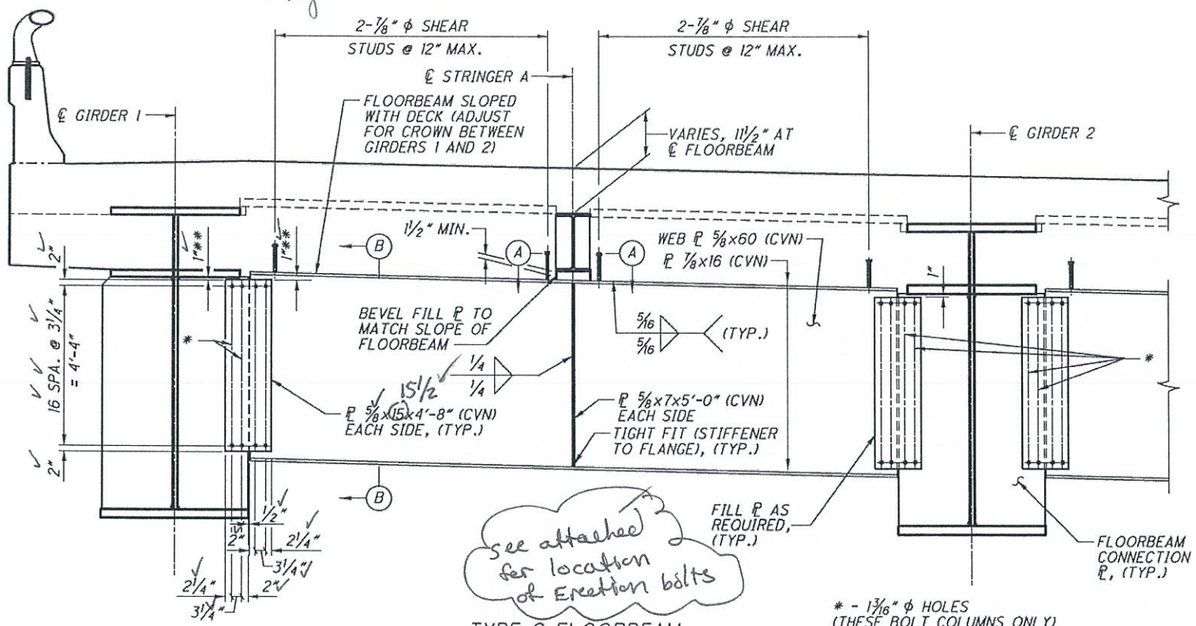
DESIGNED	DRAWN	CHECKED	CHECKED	DATE	REVIEWED	STRUCTURE FILE NUMBER
SJL	FWG	GDH	GDH	08/09/11	HRH	1809431

38/84

S JLARSON 9-12-11
JBT 9-16-11

Date:9/9/2011
Model:Sheet1
File:49633-S-BR-SSD08a-U2S1Steel.dgn

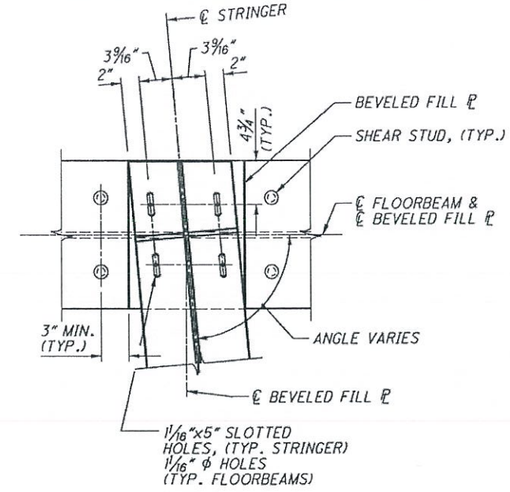
Changes due to Erection Bolt RFI



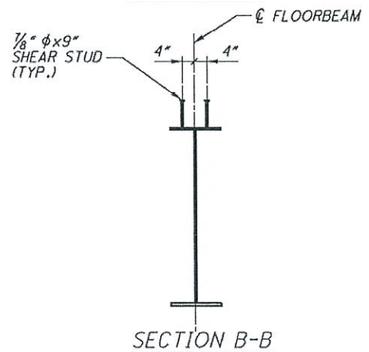
TYPE G FLOORBEAM

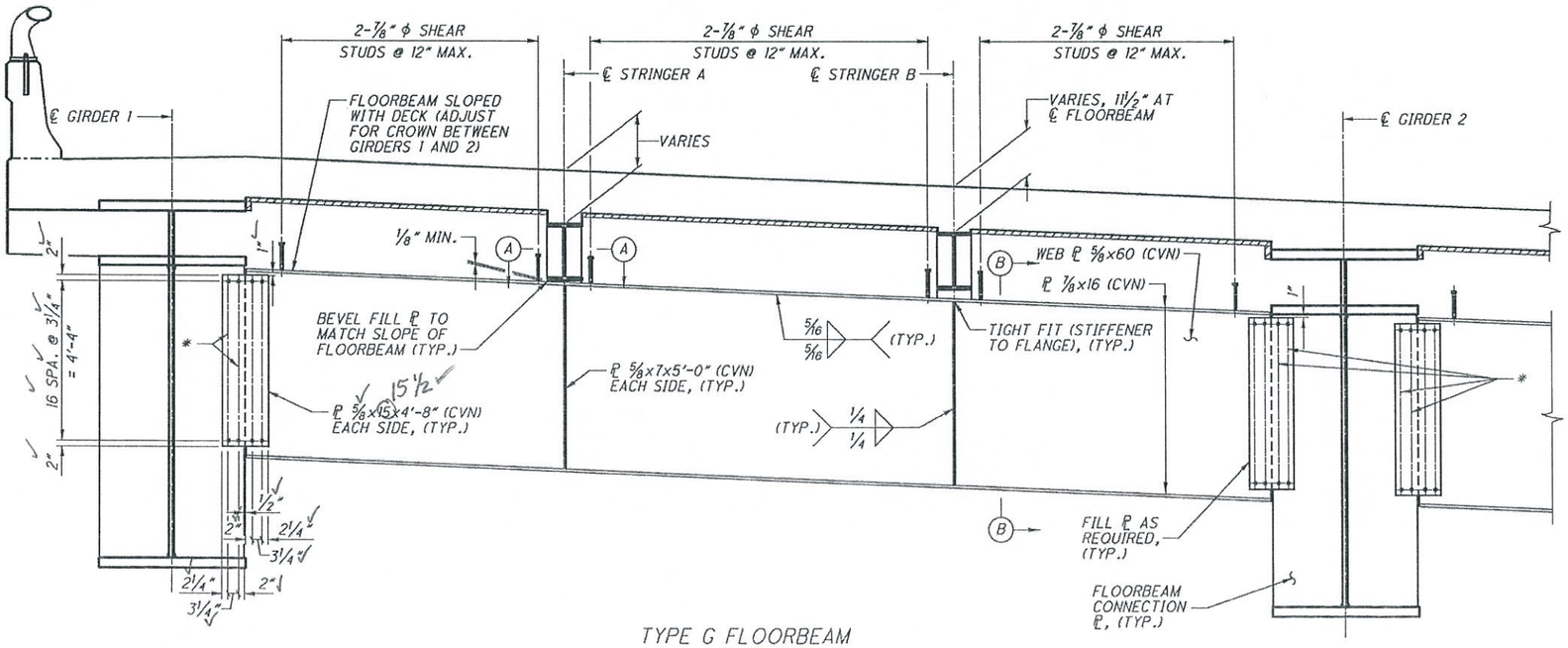
(SHOWN AT PIER 2)
(LOOKING UPSTATION)

* - 1 3/16" ϕ HOLES
(THESE BOLT COLUMNS ONLY)
** - 1" MINIMUM CLEAR FROM TOP OF CONNECTION ℙ TO BOTTOM OF FLOORBEAM TOP FLANGE OR BOTTOM OF SLOTTED FLANGE ℙ



SECTION A-A





TYPE G FLOORBEAM

(SHOWN AT PIER 11)
(LOOKING UPSTATION)

SECTION B-B

NOTES:

1. FLOORBEAMS SHALL BE DETAILED, FABRICATED, AND INSTALLED SO THEY ARE UNLOADED AT ERECTION, ALLOWING GIRDER WEBS TO BE THEORETICALLY VERTICAL OR PLUMB UNDER THE NO LOAD CONDITION.
2. WHERE A PLATE OR SHAPE IS DESIGNATED (CVN), FURNISH MATERIAL THAT MEETS THE MINIMUM NOTCH TOUGHNESS REQUIREMENTS AS SPECIFIED IN 711.01.
3. ALL BOLTS SHALL BE HIGH STRENGTH 1" ϕ, A325 GALVANIZED TYPE I BOLTS. ALL HOLES ARE 1 1/16" ϕ UNLESS NOTED OTHERWISE.
4. ALL STEEL SHALL BE A709 GRADE 50.
5. ALL STEEL SHALL BE PAINTED WITH AN IZEU COATING SYSTEM. SURFACES OF BOLTED CONNECTIONS SHALL BE PAINTED WITH A COATING THAT WILL PROVIDE CLASS B SLIP RESISTANCE AS DEFINED IN SECTION 6.13.2.8 OF THE AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS.
6. FOR FLOORBEAM LOCATIONS, SEE SHEETS 15|84 THRU 18|84.
7. FOR FLOORBEAM CONNECTION ℙ AND BEARING STIFFENER DETAILS, SEE SHEET 33|84.
8. BOLTS AT STRINGER TO FLOORBEAM CONNECTION SHALL BE SNUG TIGHT.
9. STRINGERS SHALL BE PLACED WITH BOLTS CENTERED IN THE SLOTTED HOLE.
10. FOR ADDITIONAL DETAILS, SEE BUILDABLE UNIT 1410 MAIN SPAN UNIT 2 SUPERSTRUCTURE.

PRELIMINARY NOT FOR CONSTRUCTION

ENGINEER'S SEAL

NO.	REVISIONS	DATE
B	FINAL SUBMITTAL	04/14/11
C	FINAL SUBMITTAL - REVISED	06/09/11
D	FINAL SUBMITTAL - REVISED	08/09/11

DESIGN AGENCY: WALSH CONSTRUCTION / HNTB

CLEVELAND'S INNERBELT BRIDGES — Rebuilding Transportation

MAIN SPAN UNIT 2
STRUCTURAL STEEL
SUPERSTRUCTURE DETAILS
BRIDGE NO. CUY-90-1532
I-90 WEST BOUND

DESIGNED	DRAWN	REVIEWED	DATE
SJL	FWC	HRH	08/09/11
CHECKED	CHECKED	STRUCTURE FILE NUMBER	
JPG	GDH	1809431	

CUY-90-14.90
PID No. 77332/85531

39/84

0/0

SJL 9-12-11
JBT 9/13/11

Date:9/9/2011
Model:Sheet1
File:49633-S-BR-SSD09-U2S1Steel.dgn