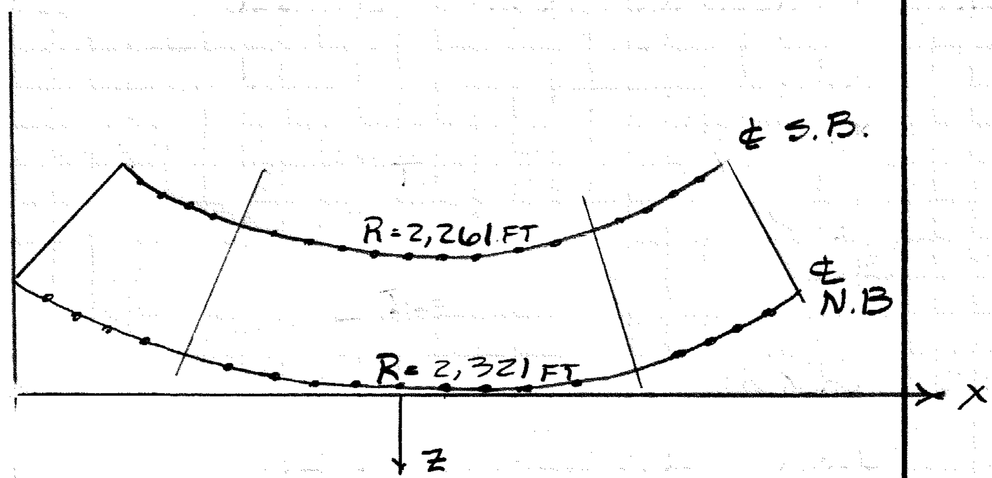
COUNTY: CROSSING: NOTES

Modelling for Substructure Analysis



		S.B.		N.B.	
	Node	X	Z	X	Z
ABUT	1	-220.21	-70.74	-226.09	-11.03
	2	-196.93	-68.59	-202.16	-8.82
	3	-173.62	-66.67	-178.23	-6.85
	4	-150.29	-65.00	-154.28	-5.13
	5	-126.95	-63.56	-130.32	-3.66
BENT	6	-103.59	-62.37	-106.34	-2.44
	7	-82.88	-61.52	-85.11	-1.56
	8	-62.17	-60.85	-63.82	-0.88
	9	-41.45	-60.38	-42.55	-0.39
	10	-20.72	-60.09	-21.27	-0.10
	11	0.00	-60.00	0.00	0.00
	12	20.72	-60.09	21.27	-0.10
	13	41.45	-60.38	42.55	-0.39
	14	62.17	-60.85	63.82	-0.88
	15	82.88	-61.52	85.11	-1.56
BENT	16	103.59	-62.37	106.34	-2.44
	17	126.95	-63.56	130.32	-3.66
	18	150.29	-65.00	154.28	-5.13
	19	173.62	-66.67	178.23	-6.85
	20	196.93	-68.59	202.16	-8.82
ABUT	21	220.21	-70.74	226.09	-11.03

DESIGN DATE

CHECK DATE

CHECK DATE

COUNTY: CROSSING: NOTES

ABUTMENT NODE : $201^{K} + \frac{1}{10}(117' \times 11.87\,KLF)$
(SB)
$= 340$ KIPS

SPAN 1 NODE : $\frac{1}{5}(117' \times 11.87\,KLF)$
(SB)
$= 278$ KIPS

BENT NODE : $\frac{1}{10}(117' \times 11.87\,KLF)$
(SB)
$\frac{1}{20}(207' \times 11.78\,KLF)$
269 KIPS
$8.48\,KLF \times 28/4$
$= 589$ KIPS

SPAN 2 NODE : $\frac{1}{10}(207' \times 11.78\,KLF)$
(SB)
$= 244$ KIPS

ABUTMENT NODE : $201^{K} + \frac{1}{10}(120' \times 11.87\,KLF)$
(NB)
$= 344$ KIPS

SPAN 1 NODE : $\frac{1}{5}(120 \times 11.87)$
(NB)
$= 285$ KIPS

BENT NODE : $\frac{1}{10}(120 \times 11.87)$
(NB)
$\frac{1}{20}(213 \times 11.78)$
269^{K}
$8.48 \times 28/4$
597 KIPS

SPAN 2 NODE : $\frac{1}{10}(213 \times 11.78)$
(NB)
$= 251$ KIPS

MID-HEIGHT COLUMN NODE :
$\frac{1}{2} \times 8.48 \times 28 = 119$ KIPS

DESIGN	DATE
CHECK	DATE
CHECK	DATE

COUNTY: CROSSING: | NOTES

LOCAL AXIS ORIENTATION:

COLUMNS: $\alpha = \dfrac{210/2}{2,291.8312} \cdot \dfrac{180}{\pi} = 2.625^{\circ}$

ABUTMENTS: $\alpha = \dfrac{118.5 + 210/2}{2,291.8312} \cdot \dfrac{180}{\pi} = 5.5875^{\circ}$

ARTIFICIAL ABUTMENT COLUMN TO MODEL STIFFNESS:

$K = 6,240^{K}/IN$ LONG.
$240\ ^{K}/IN$ TRANS.

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USE STEEL COLUMN PINNED AT TOP:

$K = \dfrac{3EI}{L^3} = 3 \times 29,000 \dfrac{I}{L^3}$

USE $L = 20''$

LONGITUDINAL:

$\dfrac{3 \times 29,000 I_L}{20^3} = 6,240$

$I_L = 574.\ IN^4$

TRANSVERSE:

$\dfrac{3 \times 29,000 I_T}{20^3} = 240$

$I_T = 22.07\ IN^4$

DESIGN	DATE
CHECK	DATE
CHECK	DATE

COUNTY: CROSSING: NOTES

MODE	FREQ.	PERIOD	% MASS	NOTES
1	0.78 Hz	1.28 SEC	47%	SB - TRANS.
2	0.78 Hz	1.28 SEC	47%	NB - TRANS.
3	1.41 Hz	0.71 SEC	2%	NB - TRANS.
4	1.69 Hz	0.59 SEC	49%	NB - LONG.
5	1.41 Hz	0.71 SEC	2%	SB - TRANS.
6	1.69 Hz	0.59 SEC	48%	SB - LONG.
7	2.63 Hz	0.38 SEC	1%	SB - TRANS.
10	2.56 Hz	0.39 SEC	1%	NB - TRANS.

TOTAL TRANS. = 100%
TOTAL LONG. = 97%

ABUTMENT REACTIONS:

EQ - LONGITUDINAL: $V_L = 1{,}245$ KIPS, $V_T = 148$ KIPS — PER ABUT.

EQ - TRANSVERSE: $V_L = 340$ KIPS, $V_T = 617$ KIPS

ENDWALL TAKES

$$(V_{L1} + V_{L2})\frac{1{,}200}{1{,}680} = 1.43\,V_L$$ PAGE 86

$$(V_L)_{MAX} = 1{,}245 + .3 \times 340 = 1{,}347 \text{ KIPS}$$

$$(V_L)_{WALL} = 1.43 \times 1{,}347 = 1{,}926 \text{ KIPS}$$

$$P_{EW} = \frac{1{,}926^K}{60FT \times 7.23FT} = 4.5 \text{ KSF} < 7.7 \text{ KSF}$$

OK

DESIGN	DATE
TEHuff	
CHECK	DATE
CHECK	DATE

COUNTY: CROSSING:

NOTES

FACTORED REACTIONS:

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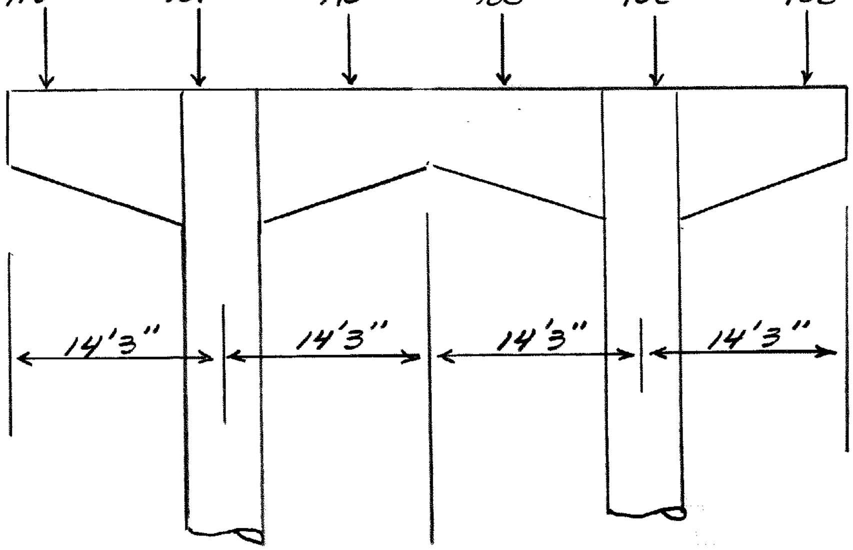
499^K	482^K	490^K	490^K	492^K	554^K	LL+I
425^K	406^K	414^K	414^K	416^K	472^K	LL
139^K	128^K	131^K	130^K	134^K	135^K	SDL
413^K	381^K	390^K	388^K	402^K	408^K	DL



UNFACTORED: DIVIDE DL BY 1.25
DIVIDE SDL BY 1.50
DIVIDE LL BY 1.75

FOR INPUT TO RC-PIER

COLUMN FACE CHECK POINTS:

11.75', 16.75', 40.25', 45.25'

8 BARS PER ROW = 7 SP. @ 5"

1ST ROW @ 4 1/4" FROM TOP
2ND ROW @ 8 3/4"
3RD ROW @ 13 1/4"
4th ROW @ 17 3/4"
5TH ROW @ 22 1/4"

PRELIMINARY COLUMN DESIGN:

4' φ COLUMNS
28 No. 9 BARS (1.5%)
No. 5 SPIRALS

DESIGN	DATE
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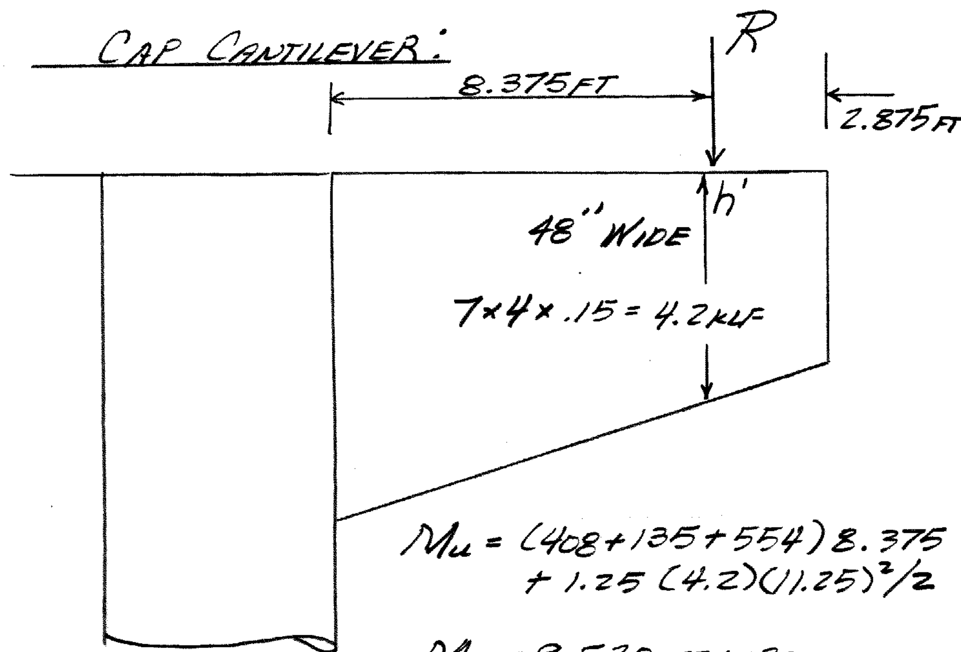
CAP CANTILEVER:



$M_u = (408+135+554)\,8.375$
$\quad + 1.25\,(4.2)(11.25)^2/2$

$M_u = 9{,}520$ FT·KIPS

$V_u = 408+135+554$
$\quad + 1.25 \times 4.2 \times 11.25$

$V_u = 1{,}156$ KIPS

$d_e = 108'' - 3\frac{1}{2}'' - 9'' = 95\frac{1}{2}'' = h - 12\frac{1}{2}''$ — @ FACE

$\phi = 0.90$ FOR SHEAR — 5.5.4

$d_v = .9 d_e$ — 5.8.2.9

$V_c = 0.0316\,(2)\sqrt{3}\,(48)\,d_v = 5.25\,d_v$ — 5.8.3.3

$V_s = \dfrac{A_v(60)d_v}{s}$ USE No. 5 ⊔ (DOUBLE)
$A_v = 4 \times .32 = 1.28\,\mathrm{in}^2$

@ BEAM, $V_u \cong 408+135+554 = 1{,}097$k

$0.9(5.25 d_v + 76.8 d_v/s) \geq 1{,}097$

$d_v(1 + 14.63/s) \geq 233$

s	d_v	d_e	h'
6"	68"	75"	88"
4½"	55"	61"	73"

DESIGN	DATE
TEHuff	
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CHECK	DATE

COUNTY: CROSSING: NOTES

USE END $h = 6$ FT

h @ FACE OF COLUMN

$$= \frac{1+\sqrt{5}}{2} \cdot 6 = 9.71 \text{ FT}$$

SAY 9'9"

THEN $h' = 6 + 3.75(0.255)$

$= 6.96$ FT

$= 83.5$"

$d_e = 83.5 - 12 = 71.5$"

$d_v = .9 \times 71.5 = 64.4$"

$64.4(1 + 14.63/s) \geq 233$

$s \leq 5.6$"

DOUBLE No. 5 U

FROM RC-PIER RESULTS:

SPACING BETWEEN COLUMNS $= \frac{2.83}{2.42} \times 5.17 = 6$"

AT FACE OF COLUMN: $h = 117$"

$d_e = 117 - 12$

$= 105$"

$$a/2 = \frac{60 A_s}{2 \times .85 \times 3 \times 48} = 0.245 A_s$$

$$\phi M_n = .9 \times 60 A_s (105 - .245 A_s)/12 \geq 9{,}520$$

$$1.103 A_s^2 - 472.5 A_s + 9{,}520 = 0$$

$$\Rightarrow A_s = 21.2 \text{ IN}^2$$

4 ROWS OF 6 No.9 BARS PER ROW IN TOP

DESIGN	DATE
TEHuff	
CHECK	DATE
CHECK	DATE

COUNTY: CROSSING:

NOTES

AVWIN MODEL

LOADING	V_L	V_T
WS	6 KIPS	44 KIPS
WL	2 KIPS	7 KIPS
CE	0 KIPS	31 KIPS
BR	24 KIPS	0 KIPS
TU	57 KIPS	23 KIPS
EQ-L	194 KIPS	24 KIPS
EQ-T	31 KIPS	245 KIPS
100% EQ-L + 30% EQ-T	203 KIPS	98 KIPS
30% EQ-L + 100% EQ-T	89 KIPS	252 KIPS

LOADS SHOWN ARE "PER BENT", NOT "PER COLUMN"

DESIGN	DATE
TEHuff	
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CHECK	DATE

COUNTY: CROSSING: NOTES

EXTREME EVENT CONSIDERATIONS:

FOOTING DESIGN: $R = 1.0$ — PAGE 19

SEISMIC SHEARS WITH $R = 1.0$: — PAGE 90

$V_L = 1{,}190$ KIPS MAX.
$V_T = 698$ KIPS MAX.

DETERMINE PLASTIC SHEARS:

EXTREME EVENT LIMIT STATE:

$$1.25(DC) + 1.50(DW) + 0.50(LL+I) + 0.50(CE) + 0.50(BR) + 1.00(EQ)$$

GRAVITY LOAD ON BENT = — PAGE 91

$2{,}382^K = 413 + 381 + 390 + 388 + 402 + 408$
$797^K = 139 + 128 + 131 + 130 + 134 + 135$
$355^K = 1.25\,(4' \times 8.3' \times 57')(.15)$ — CAP WT.
$276^K = 1.25\,(26' \times 28.3\ \text{SF} \times 2\ \text{COLS})(.15)$ — COL. WT.

$\boxed{3{,}810^K = \text{FACTORED BENT DL}}$

LIVE LOADS: VALUES ON PAGE 91 ARE MAXIMUMS FOR THE INDIVIDUAL BEAMS & DO NOT OCCUR SIMULTANEOUSLY. RERUN DESCUS WITH LANE ASSIGNMENTS AS SHOWN ON PAGE 7 FOR ALL GIRDERS (NOT GIRDER 1 ONLY)

RESULTS:

LL:	124^K	189^K	258^K	326^K	401^K	472^K
LL+I:	145^K	222^K	304^K	383^K	471^K	555^K

$\sum R_{LL} = 1{,}770$ KIPS $(LF = 1.75)$

$\sum R_{LL+I} = 2{,}080$ KIPS $(LF = 1.75)$

DESIGN	DATE
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COUNTY: CROSSING:

NOTES

COLUMN DESIGN:

PAGE 98

4'6" ⌀ COLUMN
28 No. 10 BARS (1.55%)

PILE CAP DESIGN:

PERMIT 100 K/PILE UNDER SERVICE LEVEL DL + LL AND CHECK FOR OTHER CASES.

DRIVE TO: 200 KIPS/PILE
(FS = 2.0 AGAINST FAILURE OF SUPPORTING MATERIAL)

SERVICE DL = 3,810/1.25
= 3,048 K/BENT

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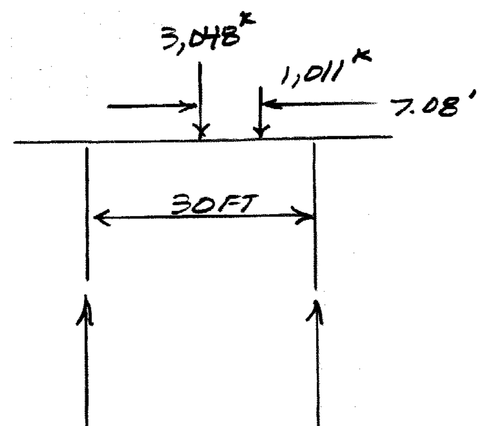
SERVICE LL = 1,770/1.75
= 1,011 K/BENT

@ 7.08 FT FROM ℄ BENT



3048/2 =

$+ 1{,}011\left(\frac{7.92}{30}\right)$

= 1,791 KIPS

3,048/2 =

$+ 1{,}011\left(\frac{22.08}{30}\right)$

= 2,268 KIPS
÷ 100 K/PILE

$\boxed{= 23 \text{ PILES}}$

DESIGN	DATE
TEHuff	
CHECK	DATE
CHECK	DATE

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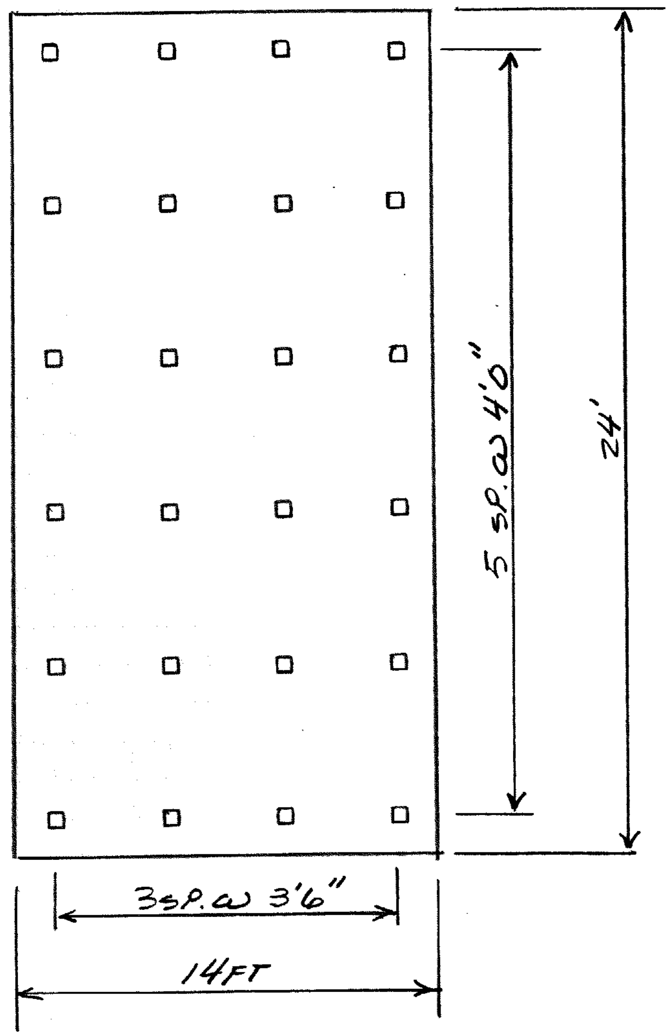
COUNTY: CROSSING:

NOTES

TRY



$2{,}268^{k} / 24$ PILES
$= 94.5\ k$ / PILE

DRIVE TO 200^{k}

F.S. = 2.12

DESIGN	DATE
TEHoff	
CHECK	DATE
CHECK	DATE

PROJECT: I55 over MALLORY AVENUE

COLUMN DESIGN - Column: 1

Column Type: Round D = 54.00 in

Code: AASHTO LRFD
Units: US
Design/Analysis Method: No Slenderness Considered.

Design Parameters:

f'c = 3000.0 psi fy = 60000.0 psi
phi flex = 0.90 phi axial = 0.75
Ec = 3320.6 ksi Es = 29000 ksi
Concrete Type : Normal Weight.

Reinforcement:

Reinforcement Pattern:

Layer	Dir	Size	No. bars	Bar Dist. in
1	X	10	28	3.26

Main bars summary: Spiral size: # 5
28 # 10 bars

Total number of bars in the column: 28

$V_u = \sqrt{194.4^2 + 50.4^2}$
$= 201$ kips

Design values used - (e-min effect included).

(global coordinates)

Loc ft	Comb	Fx kips	Fy kips	Fz kips	Mx kips-ft	My kips-ft	Mz kips-ft
0.00	60	-109.6	2075.4	-107.5	-3219.0	-35.9	1276.2
26.00	60	109.6	1997.8	107.5	425.1	-35.9	1573.4
0.00	61	-194.4	1987.6	-50.4	-1510.6	-16.9	2407.4
26.00	61	194.4	1910.1	50.4	353.4	-16.9	2646.8

Column Design

Loc ft	Comb	Pu kips	Mux kips-ft	Muz kips-ft	pMn kips-ft	C in	Incl deg	pPn/Pu	pMn/Mu
0.00	60	2075.4	3219.0	1276.2	3690.6	29.49	21.6	1.00	1.07
26.00	61	1910.1	353.4	2646.8	3712.4	28.31	82.4	1.00	1.39

Note:
 * The provided reinforcement is not adequate.
 ** Minimum/Maximum requirement for reinforcement ratio or bar spacing violated.

PROJECT: I55 over MALLORY AVENUE

ISOLATED FOOTING DESIGN

Code: AASHTO LRFD
Units: US

Geometry:
=========

Name : LW-PILECAP
Shape : Rectangular, Type : Pile/Shaft Cap

Bf(X) = 14.00 ft, Hf(Z) = 24.00 ft, Thickness(Y) = 66.00 in

Start at X = -7.00 ft from centerline of column.
Columns located on the footing:
Column No. 2 at x = 0.00 ft, Round D = 54.00 in

Ag = 336.00 ft^2, Ix = 1120.00 ft^2, Iz = 367.50 ft^2

Surcharge = 0.00 ksf

Piles: Square Size: 14.00 in Capacity: 225.00 kips

Design Parameters:
==================

f'c = 3000.00 psi fy = 60000.00 psi
phi flex = 0.90 phi shear = 0.90
Ec = 3320.6 ksi Es = 29000.0 ksi
Crack control factor z = 130.00 kips/in
Concrete Type : Normal Weight.

Pile Reactions, Service:
========================

Pile	Loc(X) ft	X in	Z in	Column Loads comb	Ovs	P, kips	Mxx, kft	Mzz, kft	Pile Reac. kips
1	-5.25	21.0	-120.0	37	1.000	-2747.44	382.80	42.55	123.22
				38	1.000	-2496.36	1144.25	-80.18	104.20
2	-1.75	63.0	-120.0	37	1.000	-2747.44	382.80	42.55	122.81
				38	1.000	-2496.36	1144.25	-80.18	104.97
3	1.75	105.0	-120.0	35	1.000	-2771.28	608.29	-257.81	122.82
				38	1.000	-2496.36	1144.25	-80.18	105.73
4	5.25	147.0	-120.0	35	1.000	-2771.28	608.29	-257.81	125.27
				38	1.000	-2496.36	1144.25	-80.18	106.49
5	-5.25	21.0	-72.0	37	1.000	-2747.44	382.80	42.55	124.58
				38	1.000	-2496.36	1144.25	-80.18	108.29
6	-1.75	63.0	-72.0	37	1.000	-2747.44	382.80	42.55	124.18
				38	1.000	-2496.36	1144.25	-80.18	109.05
7	1.75	105.0	-72.0	35	1.000	-2771.28	608.29	-257.81	124.99
				38	1.000	-2496.36	1144.25	-80.18	109.82
8	5.25	147.0	-72.0	35	1.000	-2771.28	608.29	-257.81	127.44
				38	1.000	-2496.36	1144.25	-80.18	110.58
9	-5.25	21.0	-24.0	37	1.000	-2747.44	382.80	42.55	125.95
				38	1.000	-2496.36	1144.25	-80.18	112.38
10	-1.75	63.0	-24.0	37	1.000	-2747.44	382.80	42.55	125.55
				38	1.000	-2496.36	1144.25	-80.18	113.14
11	1.75	105.0	-24.0	35	1.000	-2771.28	608.29	-257.81	127.16
				38	1.000	-2496.36	1144.25	-80.18	113.90

PROJECT: I55 over MALLORY AVENUE

12	5.25	147.0	-24.0	35	1.000	-2771.28	608.29	-257.81	129.62
				38	1.000	-2496.36	1144.25	-80.18	114.67
13	-5.25	21.0	24.0	36	1.000	-2747.44	1103.59	42.55	128.61
				39	1.000	-2496.36	567.62	-80.18	115.43
14	-1.75	63.0	24.0	36	1.000	-2747.44	1103.59	42.55	128.20
				39	1.000	-2496.36	567.62	-80.18	116.20
15	1.75	105.0	24.0	34	1.000	-2771.28	1329.08	-257.81	130.62
				39	1.000	-2496.36	567.62	-80.18	116.96
16	5.25	147.0	24.0	34	1.000	-2771.28	1329.08	-257.81	133.08
				39	1.000	-2496.36	567.62	-80.18	117.72
17	-5.25	21.0	72.0	36	1.000	-2747.44	1103.59	42.55	132.55
				39	1.000	-2496.36	567.62	-80.18	117.46
18	-1.75	63.0	72.0	34	1.000	-2771.28	1329.08	-257.81	132.91
				39	1.000	-2496.36	567.62	-80.18	118.22
19	1.75	105.0	72.0	34	1.000	-2771.28	1329.08	-257.81	135.37
				39	1.000	-2496.36	567.62	-80.18	118.99
20	5.25	147.0	72.0	34	1.000	-2771.28	1329.08	-257.81	137.82
				39	1.000	-2496.36	567.62	-80.18	119.75
21	-5.25	21.0	120.0	36	1.000	-2747.44	1103.59	42.55	136.49
				39	1.000	-2496.36	567.62	-80.18	119.49
22	-1.75	63.0	120.0	34	1.000	-2771.28	1329.08	-257.81	137.66
				39	1.000	-2496.36	567.62	-80.18	120.25
23	1.75	105.0	120.0	34	1.000	-2771.28	1329.08	-257.81	140.11
				39	1.000	-2496.36	567.62	-80.18	121.01
24	5.25	147.0	120.0	34	1.000	-2771.28	1329.08	-257.81	142.57
				39	1.000	-2496.36	567.62	-80.18	121.78

Pile Reactions, Factored:
========================

Pile	Loc(X) ft	X in	Z in	Column Loads comb	Ovs	P, kips	Mxx, kft	Mzz, kft	Pile Reac. kips
1	-5.25	21.0	-120.0	2	---	-4163.61	-202.72	-77.89	188.62
				15	---	-1263.60	659.79	-383.27	55.72
2	-1.75	63.0	-120.0	2	---	-4163.61	-202.72	-77.89	189.36
				15	---	-1263.60	659.79	-383.27	59.37
3	1.75	105.0	-120.0	2	---	-4163.61	-202.72	-77.89	190.10
				16	---	-1192.58	196.15	540.36	59.80
4	5.25	147.0	-120.0	2	---	-4163.61	-202.72	-77.89	190.84
				16	---	-1192.58	196.15	540.36	54.66
5	-5.25	21.0	-72.0	2	---	-4163.61	-202.72	-77.89	187.89
				15	---	-1263.60	659.79	-383.27	58.08
6	-1.75	63.0	-72.0	2	---	-4163.61	-202.72	-77.89	188.64
				15	---	-1263.60	659.79	-383.27	61.73
7	1.75	105.0	-72.0	2	---	-4163.61	-202.72	-77.89	189.38
				16	---	-1192.58	196.15	540.36	60.50
8	5.25	147.0	-72.0	2	---	-4163.61	-202.72	-77.89	190.12
				16	---	-1192.58	196.15	540.36	55.36
9	-5.25	21.0	-24.0	2	---	-4163.61	-202.72	-77.89	187.17
				15	---	-1263.60	659.79	-383.27	60.43
10	-1.75	63.0	-24.0	2	---	-4163.61	-202.72	-77.89	187.91
				15	---	-1263.60	659.79	-383.27	64.08
11	1.75	105.0	-24.0	2	---	-4163.61	-202.72	-77.89	188.65
				16	---	-1192.58	196.15	540.36	61.20
12	5.25	147.0	-24.0	2	---	-4163.61	-202.72	-77.89	189.40
				16	---	-1192.58	196.15	540.36	56.06
13	-5.25	21.0	24.0	1	---	-4163.61	1058.66	-77.89	188.70
				15	---	-1263.60	659.79	-383.27	62.79

PROJECT: I55 over MALLORY AVENUE

14	-1.75	63.0	24.0	1	---	-4163.61	1058.66	-77.89	189.44
				15	---	-1263.60	659.79	-383.27	66.44
15	1.75	105.0	24.0	1	---	-4163.61	1058.66	-77.89	190.18
				16	---	-1192.58	196.15	540.36	61.91
16	5.25	147.0	24.0	1	---	-4163.61	1058.66	-77.89	190.92
				16	---	-1192.58	196.15	540.36	56.76
17	-5.25	21.0	72.0	1	---	-4163.61	1058.66	-77.89	192.48
				15	---	-1263.60	659.79	-383.27	65.15
18	-1.75	63.0	72.0	1	---	-4163.61	1058.66	-77.89	193.22
				16	---	-1192.58	196.15	540.36	67.75
19	1.75	105.0	72.0	1	---	-4163.61	1058.66	-77.89	193.96
				16	---	-1192.58	196.15	540.36	62.61
20	5.25	147.0	72.0	1	---	-4163.61	1058.66	-77.89	194.71
				16	---	-1192.58	196.15	540.36	57.46
21	-5.25	21.0	120.0	1	---	-4163.61	1058.66	-77.89	196.26
				15	---	-1263.60	659.79	-383.27	67.50
22	-1.75	63.0	120.0	1	---	-4163.61	1058.66	-77.89	197.00
				16	---	-1192.58	196.15	540.36	68.45
23	1.75	105.0	120.0	1	---	-4163.61	1058.66	-77.89	197.74
				16	---	-1192.58	196.15	540.36	63.31
24	5.25	147.0	120.0	1	---	-4163.61	1058.66	-77.89	198.49
				16	---	-1192.58	196.15	540.36	58.16

Note:
Only max. force in piles is considered for design.
Pile coordinates X and Z are from the most left edge of the footing.

Max. Pile Reaction Used in Design: (without selfweight and surcharge)
==================================

200/131 = 1.53

Factored pile reaction = 184.05 kips

Service pile reaction = 131.02 kips

FS = 1.53 > 1.5 OK (LATERAL LOADS)

Fatigue pile reaction = 42.57 kips

FS = 2.12 > 2.0 OK (GRAVITY ONLY)
(P. 96)

Reinforcement Schedule:
=======================:

Dir	Quantity	Size	Bar dist. in	As total in^2	Spacing in	Hook	
X	23	# 8	3.50	18.17	12.77	None	
X	23	# 8	62.50	18.17	12.77	None	
Z	43	# 8	4.50	33.97	3.83	None	— 27 #10 = 34.29
Z	13	# 8	61.50	10.27	13.42	None	— 27 # 6 = 11.88

Flexure:
========

Dir	Loc ft	d in	Mmax kft	Comb	Asb_req in^2	Asb_prv in^2	Asb_eff in^2	Ast_req in^2	Ast_prv in^2	Ast_eff in^2
X	-1.99	62.50	3595.6	1	17.42	18.17	18.17	17.42	18.17	18.17

PROJECT: I55 over MALLORY AVENUE

X	1.99	62.50	3595.6	1	17.42	18.17	18.17	17.42	18.17	18.17
Z	-1.99	61.50	8847.6	1	33.23	33.97	33.97	10.16	10.27	10.27
Z	1.99	61.50	8847.6	1	33.23	33.97	33.97	10.16	10.27	10.27

27 No. 10 = 34.29 IN^2 > 33.23, OK

Note:
* The provided reinforcement is not adequate, either less than required or larger than maximum allowed.
** Required spacing of bars violated.

Cracking/Fatigue

Dir	Loc ft	d in	Mmax kft	Comb	Cracking fs ksi	ratio fs	Mmax kft	Comb	Fatigue fs ksi	ratio fs
X	-1.99	62.50	2559.6	34	28.20	1.17 *	831.6	33	9.16	0.46
X	1.99	62.50	2559.6	34	28.20	1.17 *	831.6	33	9.16	0.46
Z	-1.99	61.50	6298.4	34	38.83	1.09 *	2046.2	33	12.62	0.68
Z	1.99	61.50	6298.4	34	38.83	1.09 *	2046.2	33	12.62	0.68

Note:
* Cracking / fatigue ratio exceeds allowable.

One Way Shear:

Col	Dir	Dist ft	Comb	dv in	Vu kips	phi*Vc kips
1	X	-7.14	----	------	Outside of Footing	
	X	7.14	----	------	Outside of Footing	
	Z	6.92	1	59.12	736.2	979.2
	Z	-6.92	1	59.12	736.2	979.2

Note:
* Shear resistance is less than applied shear force.
You may increase the footing depth or provide stirrups.

Two Way Shear:

#	Bo ft	Ao ft^2	Comb	Avg. dv in	Vu kips	phi*Vc kips
Columns:						
1	29.96	71.43	1	60.44	3681.0	4268.0
Piles - max:						
6	24.81	38.48	1	60.44	184.0	3534.7
Piles - min:						
1	9.95	24.75	1	60.44	184.0	1417.9

COUNTY: CROSSING: NOTES

COLUMN SPIRALS:

SEISMIC ZONE IS 3 ($A = 0.22$) 3.10.4

$\rho_s \geq 0.12 f'_c / f_y$ 5.10.11.4.1d

$$\rho_s \geq 0.45 \left(\frac{A_g}{A_c} - 1 \right) \frac{f'_c}{f_{yh}}$$

A_c = AREA OF CORE MEASURED TO OUTSIDE OF SPIRAL

$d_c = 54'' - 6'' = 48''$

$A_c = \pi (48)^2/4 = 1{,}810 \text{ IN}^2$

$$\rho_s \geq 0.45 \left(\frac{2{,}290}{1{,}810} - 1 \right) \frac{3}{60}$$

$= 0.119 (3/60)$

(0.12 CONTROLS)

$\rho_s \geq 0.12 (3/60) = 0.006$

$$\rho_s = \frac{A_{sp} (\pi d_s)}{s A_c}$$

No. 6 SPIRALS: $A_{sp} = 0.44 \text{ IN}^2$
$d_s = 48 - 6/8$
$= 47.25''$

$$\rho_s = \frac{0.44 (\pi)(47.25)}{s (1{,}810)} \geq 0.006$$

$s \leq 6.01''$

END REGION = $D = 4.5$ FT
$H/6 = 18/6 = 3$ FT
$18'' = 1.5$ FT

$\boxed{\text{4.5 FT CONTROLS END REGION}}$

(5.10.11.4.1c)

DESIGN	DATE
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CHECK	DATE

SPIRALS TO EXTEND $D/2 = 2.25$ FT INTO CAP & FOOTING. — 5.10.11.4.3

$S_{MAX} = 6''$ — 5.10.6.2

WITH NO 5 SPIRALS: $A_{sp} = 0.32$
$d_s = 48 - 5/8$
$= 47.4''$

$$\rho_s = \frac{0.32(\pi)(47.4)}{s\,(1{,}810)} \geq 0.006$$

$$s \leq 4.4''$$

USE No. 5 SPIRALS @ 4″ IN END REGIONS;

USE No. 5 SPIRALS @ 6″ ELSEWHERE

$b_v = 54''$ — 5.8.2.9

$D_r = 54'' - 6'' - 5/8'' - 5/8'' - 10/8'' = 45.5''$ — 5.8.3.3, 5.8.2.4

$$d_e = \frac{D}{2} + \frac{D_r}{\pi} = \frac{54}{2} + \frac{45.5}{\pi} = 41.5''$$

$d = 0.9\,d_e = 37.3''$

$\phi V_c = .9\,(.0316)(2\sqrt{3})(54 \times 37.3)$

$= \underline{\underline{199 \text{ KIPS}}} \approx (V_u)_{MAX} = 201 \text{ KIPS}$ — PAGE 98

$\therefore$ MIN SPIRALS WILL SUFFICE

DESIGN	DATE
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COUNTY: CROSSING: NOTES

CROSS-FRAME FORCES:

ABUTMENT MEMBERS

1	21
22	42
43	63
64	84
85	105

SERVICE $M = 0.3\ FT{\cdot}K$ DL
$8.8\ FT{\cdot}K$ LL
$V = 0.1\ KIPS$ DL
$1.1\ KIPS$ LL

BENT MEMBERS

6	16
27	37
48	58
69	79
90	100

SERVICE $M = 65.1\ FT{\cdot}K$ DL
$20.8\ FT{\cdot}K$ SDL
$4.2\ FT{\cdot}K$ LL

$V = 8.9\ KIPS$ DL
$2.9\ KIPS$ SDL
$0.5\ KIPS$ LL

INTERMEDIATE CROSS-FRAMES

SERVICE $M = 54.4\ FT{\cdot}K$ DL
$10.7\ FT{\cdot}K$ SDL
$80.1\ FT{\cdot}K$ LL

$V = 0.6\ KIPS$ DL
$0.6\ KIPS$ SDL
$6.7\ KIPS$ LL

$M_{RANGE} = 163\ FT{\cdot}KIPS$
$V_{RANGE} = 14.2\ KIPS$

WS: $V_T = (0.685\ KLF/2)(24\ FT)$
$= 8.22\ KIPS$

DESIGN	DATE
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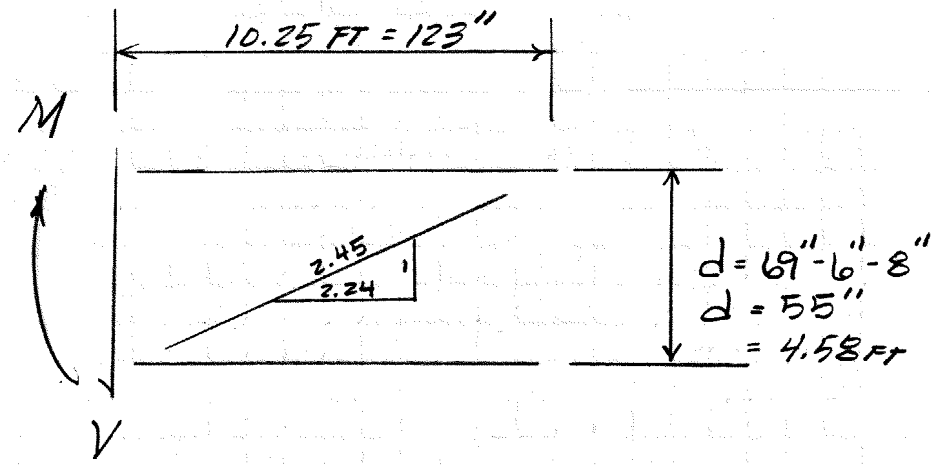
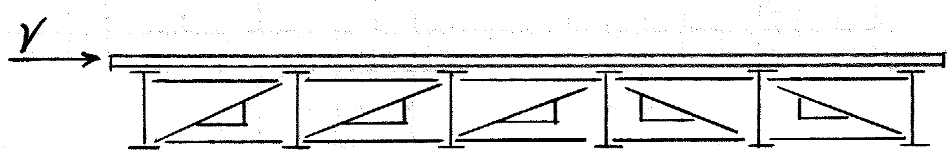


$P_{DIAG} \cong 2.45\,V$

$P_{CHORD} \cong M/d$

FROM LATERAL LOADS: (AT BENTS)



$P_{CHORD} \cong V/5$

$P_{DIAG} \approx (2.45/2.24)(V/5)$
$= V/4.57$

LOAD	V	P_{CHORD}	P_{DIAG}
WS	44^K	8.8^K	9.7^K
WL	7^K	1.4^K	1.6^K
CE	31^K	6.2^K	6.8^K
TU	23^K	4.6^K	5.1^K
EQ-1	98^K	19.6^K	21.5^K
EQ-2	252^K	50.4^K	55.2^K

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THIS TABLE IS FOR CROSS-FRAMES AT BENTS ONLY.

DESIGN	DATE
TEH	
CHECK	DATE
CHECK	DATE

COUNTY: CROSSING: NOTES

STRENGTH I LIMIT STATE:

P_{CHORD} = 1.25 [65.1/4.58] = 17.8^k DL
1.25 [20.8/4.58] = 5.7^k SDL
1.75 [4.2/4.58] = 1.6^k LL
1.75 [6.2^k] = 10.9^k CE
0.50 [4.6^k] = 2.3^k TU

P_{CHORD} = 38.3 KIPS
(BENT CROSS-FRAME)

P_{CHORD} = 1.25 [54.4/4.58] = 14.9^k DL
1.25 [10.7/4.58] = 3.0^k SDL
1.75 [80.1/4.58] = 30.6^k LL

P_{CHORD} = 48.5 KIPS
(INTERMEDIATE CROSS-FRAME)

P_{DIAG} = 1.25 [2.45 × 8.9] = 27.3^k DL
1.25 [2.45 × 2.9] = 8.9^k SDL
1.75 [2.45 × 0.5] = 2.2^k LL
1.75 [6.8] = 11.9^k CE
0.5 [5.1] = 2.6^k TU

P_{DIAG} = 52.9 KIPS
(BENT CROSS-FRAME)

P_{DIAG} = 2.45[1.25(0.6+0.6)+1.75(6.7)]

P_{DIAG} = 32.4 KIPS
(INTERMEDIATE CROSS-FRAMES)

EXTREME EVENT I LIMIT STATE

P_{CHORD} = 1.25 [65.1/4.58] = 17.8^k DL
1.25 [20.8/4.58] = 5.7^k SDL
0.50 [4.2/4.58] = 0.5^k LL
0.50 [6.2] = 3.1^k CE
1.00 [50.4] = 50.4^k EQ

P_{CHORD} = 77.5^k
(BENT CROSS-FRAME)

DESIGN	DATE
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CHECK	DATE

COUNTY: CROSSING: | NOTES

$$P_{DIAG} = 1.25\,[2.45 \times 8.9] = 27.3^{k} \quad DL$$
$$1.25\,[2.45 \times 2.9] = 8.9^{k} \quad SDL$$
$$0.5\,[2.45 \times 0.5] = 0.6^{k} \quad LL$$
$$0.5\,[6.8] = 3.4^{k} \quad CE$$
$$1.0\,[55.2] = 55.2^{k} \quad EQ$$

$P_{DIAG} = 95.4$ KIPS
(BENT CROSS-FRAME)

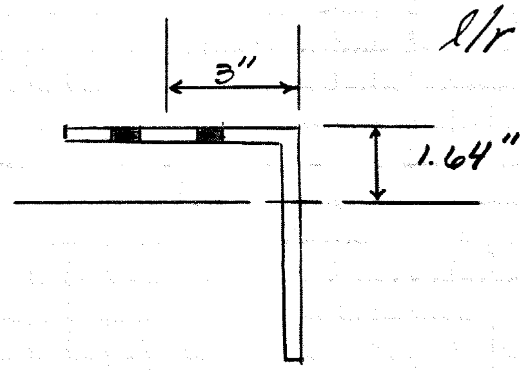
FACTORED DESIGN FORCES:

INTERMEDIATE CROSS-FRAMES	BENT CROSS-FRAMES
$P_{CHORD} = 48.5$ KIPS	$P_{CHORD} = 77.5$ KIPS
$P_{DIAG} = 52.9$ KIPS	$P_{DIAG} = 95.4$ KIPS

CHORD: $L = 123''$

TRY $\angle 6 \times 6 \times 3/8$ $r_{min} = 1.19''$

$\ell / r = 103 < 140$, OK 6.9.3



$$M_1 = P \times 1.64''$$
$$M_2 = P \times (3 - 1.64) = P \times 1.36''$$ 6.9.4

$K = 0.75$ (4.6.2.5)

$$\lambda = \left(\frac{0.75 \times 123}{1.19 \times \pi}\right)^2 \frac{50}{29{,}000} = 1.05$$

$$\therefore P_n = 0.66^{1.05} \times 50 \times 4.36 = 141^{k}$$

$$\boxed{\phi P_n = .9 \times 141 = 127 \text{ KIPS}}$$

DESIGN	DATE
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FLEXURAL CAPACITY OF ANGLES: — 6.12

$F_{ob} = \frac{85{,}900}{(l/b)^2}\left[(1+.78\, l^2t^2/b^4)^{1/2} - 1\right]$ — AISC 9th P. 5-312

FOR $C_b = 1.0$

$l = 123$
$t = 3/8$
$b = 6''$

$F_{ob} = 104\text{ KSI}$, LTB STRESS
$> F_y = 50\text{KSI}$

∴ PERMIT YIELD

$M_y = 50\text{KSI} \times 3.53\text{ IN}^3/12 = 14.7\text{ FT·K}$

$M_{rx} = M_{ry} = 14.7\text{ FT·K}$

$P_r = \phi P_n = 127\text{ KIPS}$

$\frac{P_u}{P_r} = \frac{48.5}{127} = 0.38 > 0.20$ — 6.9.2.2

$M_{ux} = 48.5 \times 1.64''/12 = 6.6\text{ FT·K}$
$M_{uy} = 48.5 \times 1.36''/12 = 5.5\text{ FT·K}$

$$\frac{P_u}{P_r} + \frac{8}{9}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right)$$

$$= 0.38 + \frac{8}{9}(5.5/14.7 + 6.6/14.7)$$

$= \underline{1.11 > 1.0\text{, NO GOOD}}$

TRY $\angle 6 \times 6 \times 7/16$ $A = 5.06\text{ IN}^2$
$S = 4.08\text{ IN}^3$

$P_r = .9 \times .66^{1.05} \times 50 \times 5.06 = 147^K$
$M_{rx} = M_{ry} = 50 \times 4.08/12 = 17.0^{'-K}$

$$\frac{48.5}{147} + \frac{8}{9}\left(\frac{5.5}{17} + \frac{6.6}{17}\right) = 0.96 < 1.0$$
OK

DESIGN	DATE
TEH/	6/06
CHECK	DATE
CHECK	DATE

$\angle 6\times6\times 7/16$ OK
FOR CHORD OF
INTERMEDIATE
CROSS-FRAMES
(GR. 50W)

CHORD AT BENT:

$$A_{req'd} \approx 5.06 \times .96 \times \frac{77.5}{48.5}$$

$$= 7.76\ IN^2$$

TRY: $\angle 8\times8\times\frac{1}{2}$ $\quad r_{min} = 1.59$, $A = 7.75$, $S = 8.36$

$$\lambda = \left(\frac{.75\times123}{1.59\,\pi}\right)^2 \frac{50}{29{,}000} = 0.59$$

$$P_n = 0.66^{.59} \times 50 \times 7.76$$
$$= 304\ KIPS$$

$$M_{ux} = P_u \times 2.19''$$
$$M_{uy} = P_u \times 1.81''$$

$$M_{rx} = M_{ry} = 50\times8.36 = 418''\text{-}K$$

$$\frac{77.5}{.9\times304} + \frac{8}{9}\left(\frac{77.5\times2.19}{418} + \frac{77.5\times1.81}{418}\right)$$

$$= 0.94 < 1.0,\ OK$$

$\angle 8\times8\times\frac{1}{2}$ OK
FOR CHORD OF
BENT CROSS-
FRAMES (GR 50W)

DESIGN	DATE
TEHuff	
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CHECK	DATE

COUNTY: CROSSING: | NOTES

DIAGONAL: $l = (123^2 + 55^2)^{1/2} = 135''$

TRY $\angle 6 \times 6 \times 9/16$ $r_{min} = 1.18$
$A = 6.43$
$S = 5.14$

$\lambda = 1.29$

$\phi P_n = .9 \times 50 \times 6.43 \times .66^{1.29}$
$= 169$ KIPS

$M_{rx} = M_{ry} = 50 \times 5.14 = 257''^{k}$

$$\frac{52.9}{169} + \frac{8}{9}\left(\frac{52.9 \times 1.64}{257} + \frac{52.9 \times 1.36}{257}\right)$$

$= 0.86 < 1.0$, OK

$\angle 6 \times 6 \times 9/16$ OK FOR DIAGONAL OF INTERMEDIATE CROSS-FRAMES, GR. 50W

BENT DIAGONAL

$A_{req'd} \cong .86 \times 6.43 \times \frac{95.4}{52.9}$
$= 10 \text{ IN}^2$

$\angle 8 \times 8 \times 3/4$ OK FOR DIAGONAL OF BENT CROSS FRAME, GR. 50W

DESIGN	DATE
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CHECK	DATE

COUNTY:	CROSSING:	NOTES

CONNECTION BOLTS — 6.13

FACTORED LOADS: — PAGE 108

	CHORD	DIAGONAL
INTERMEDIATE CROSS-FRAME	48.5^K	52.9^K
BENT CROSS-FRAME	77.5^K	95.4^K

A325 BOLTS, $F_{ub} = 120$ ksi $(\frac{1}{2}" - 1")$ — 6.4.3.1
105 ksi $(1\frac{1}{8}" - 1\frac{1}{2}")$

A490 BOLTS, $F_{ub} = 150$ ksi

SHEAR RESISTANCE: — 6.13.2.7

WITH 7/8" BOLTS, $A_b = 0.6013$ IN^2

$\phi_s = 0.80$ — 6.5.4.2

$\phi R_n = 0.80(0.48)(.6013)(120)(1)$ — SINGLE SHEAR
$= 27.7$ KIPS

2 BOLTS, $\phi R_n = 2 \times 27.7$
$= 55.4$ KIPS

> 2, 7/8" A325 BOLTS
> OK FOR INTERMEDIATE
> CROSS-FRAME CHORDS
> & DIAGONALS
> (STRENGTH LIMIT STATE)

4 BOLTS, $\phi R_n = 4 \times 27.7$
$= 110.8$ KIPS

> 4, 7/8" A325 BOLTS
> OK FOR CROSS-FRAMES
> AT BENTS

DESIGN	DATE
TEHuff	
CHECK	DATE
CHECK	DATE

COUNTY: CROSSING: NOTES

CONNECTIONS FOR CROSS-FRAME MEMBERS IN CURVED GIRDER BRIDGES SHOULD BE DESIGNED AS SLIP-CRITICAL SINCE THEY ARE SUBJECT TO FATIGUE LOADING, WHEREAS STRAIGHT GIRDER CROSS-FRAMES ARE NOT. 6.13.2.1.1

SERVICE II LOAD COMBINATION 6.13.2.2

$$DC + DW + 1.3LL + 1.3CE + TU$$

INTERMEDIATE CROSS-FRAME

$$\begin{aligned}
P_{CHORD} &= 54.48/4.58 &= 11.9^K \quad DL\\
&\ 10.7/4.58 &= 2.3^K \quad SDL\\
&\ 1.3(80.1)/4.58 &= \underline{22.7^K} \quad LL\\
& & 36.9^K
\end{aligned}$$

$$\begin{aligned}
P_{DIAG} &= 2.45 \times .6 &= 1.5^K \quad DL\\
&\ 2.45 \times .6 &= 1.5^K \quad SDL\\
&\ 1.3 \times 2.45 \times 6.7 &= \underline{21.3^K} \quad LL\\
& & 24.3^K
\end{aligned}$$

BENT CROSS-FRAME:

$$\begin{aligned}
P_{CHORD} &= 65.1/4.58 &= 14.2^K \quad DL\\
&\ 20.8/4.58 &= 4.6^K \quad SDL\\
&\ 1.3 \times 4.2/4.58 &= 1.2^K \quad LL\\
&\ 1.3 \times 6.2 &= 8.1^K \quad CE\\
&\ 1.0 \times 4.6 &= \underline{4.6^K} \quad TU\\
& & 32.7^K
\end{aligned}$$

$$\begin{aligned}
P_{DIAG} &= 2.45 \times 8.9^K &= 21.8^K \quad DL\\
&\ 2.45 \times 2.9^K &= 7.1^K \quad SDL\\
&\ 1.3 \times 2.45 \times .5^K &= 1.6^K \quad LL\\
&\ 1.3 \times 6.8^K &= 8.9^K \quad CE\\
&\ 1.0 \times 5.1^K &= \underline{5.1^K} \quad TU\\
& & 44.5^K
\end{aligned}$$

DESIGN	DATE
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CHECK	DATE

COUNTY:	CROSSING:	NOTES

SLIP RESISTANCE — 6.13.2.8

$K_h = 1.0$, USE STANDARD HOLES

$K_s = 0.50$, SPECIFY BLAST-CLEANED SURFACE

$N_s = 1$ SLIP PLANE

$P_T = 39$ KIPS FOR 7/8" A325 BOLTS

$R_n = 1.0 \times 0.50 \times 1 \times 39$
$= \underline{19.5 \text{ KIPS}}$

NO. OF BOLTS REQ'D:

INTERMEDIATE CROSS-FRAME : CHORD - 36.9/19.5 = 1.89 — 2 BOLTS OK

DIAG - 24.3/19.5 = 1.25 — 2 BOLTS, OK

BENT CROSS-FRAME : CHORD - 32.7/19.5 = 1.68 — 4 BOLTS OK

DIAG - 44.5/19.5 = 2.28 — 4 BOLTS OK

DESIGN	DATE
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CHECK	DATE

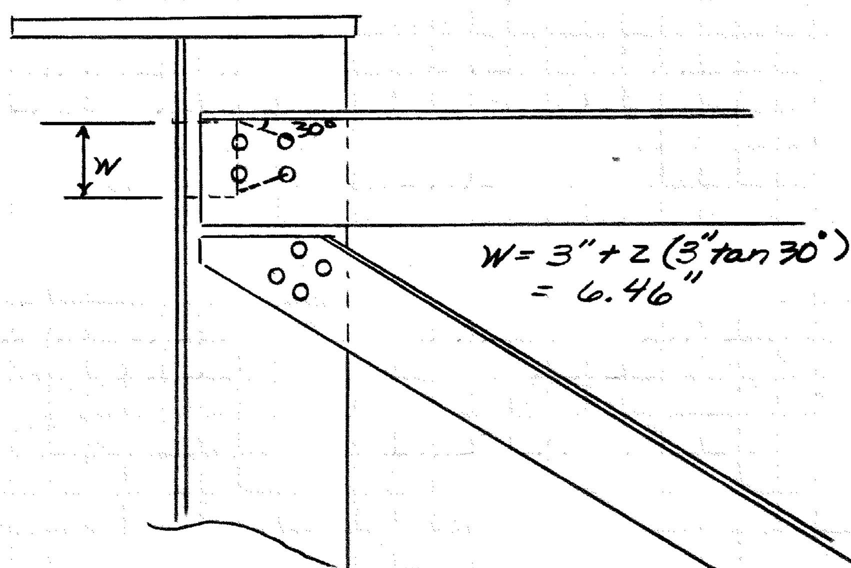
COUNTY: CROSSING: NOTES

GUSSET PL DESIGN:

TERMINATE BRACES A MINIMUM OF 2t FROM RE-ENTRANT CORNERS TO PERMIT OUT-OF-PLANE PLASTIC HINGING DURING SEISMIC EVENT

REF.: "SEISMIC DESIGN & BEHAVIOR OF GUSSET PLATES", BY ABOLHASSAN ASTANEH-ASL, STEEL TIPS, STRUCTURAL STEEL EDUCATION COUNCIL, DECEMBER, 1998, PAGE 7.

WHITMORE EFFECTIVE WIDTH:



$W = 3'' + 2(3'' \tan 30°)$
$= 6.46''$

GUSSET CAPACITY $= 6.46t(F_y)$

$6.46t(50) \geq 95.4^k$

$t \geq 0.295''$

PAGE 108 P_{MAX}

TO QUALIFY AS TRANSVERSE STIFFENER, NEED $A_s > 5.03\ IN^2$ (PAGE 71)

TO QUALIFY AS BEARING STIFFENER, NEED 9" × 1 1/8" GUSSET (P. 73)

INTERMEDIATE GUSSETS: 8" × 3/4"

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CHECK	DATE

COUNTY: CROSSING: NOTES

BEARINGS: 14.7.5

AT BENT: $R_{DL} = 413.0$ KIPS / BEAM
$R_{SDL} = 139.0$ KIPS / BEAM
$R_{LL+I} = 554.0$ KIPS / BEAM
(FACTORED)

$R_{DL} = 330.4$ KIPS / BEAM
$R_{SDL} = 111.2$ KIPS / BEAM
$R_{LL+I} = 316.6$ KIPS / BEAM
(UNFACTORED)

$\theta_{DL+SDL} = 0.0041$ RAD
$\theta_{LL+I} = 0.0017$ RAD
(UNFACTORED)

TRY 27" WIDE x 18" LONG BEARING

4, 3/4" INTERNAL LAYERS
2, 3/8" COVER LAYERS
5, 1/8" STEEL LAYERS

NO HOLES IN BEARING:

$$S = \frac{27 \times 18}{2(3/4)(27+18)} = 7.2$$

SHORE A HARDNESS = 60
$G_{MIN} = 0.130$ ksi
$G_{MAX} = 0.200$ ksi

COMPRESSION:

$\sigma_s \leq 2.00 \times .130 \times 7.2 = 1.87$ ksi
$\sigma_s \leq 1.75$ ksi, CONTROLS

$$\sigma_s = \frac{330.4 + 111.2 + 316.6}{27 \times 18} = 1.56 \text{ ksi}$$

< 1.75 ksi

OK

$h_{rt} = 4 \times 3/4 + 2 \times 3/8 = 3.75"$

$\Delta = 0 < h_{rt}/2 = 1.875$, OK FOR SHEAR DEFL.

DESIGN	DATE
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COUNTY: CROSSING: NOTES

LIVE LOAD COMPRESSION:

$$\sigma_L = \frac{316.6}{27 \times 18} = 0.65\ ksi < 1.0\ ksi$$

ok

ROTATIONAL CAPACITY:

$$(\sigma_s)_{MIN} = 1.0(0.20)(7.2)\left(\frac{.0058}{4}\right)\left(\frac{18}{3/4}\right)^2$$

$$= 1.203\ ksi < \sigma_s = 1.56\ ksi$$

ok

$$(\sigma_s)_{MAX} = 2.25(0.13)(7.2)K$$

$$K = 1 - .167\left(\frac{.0058}{4}\right)\left(\frac{18}{3/4}\right)^2$$

$$= 0.8605$$

$$(\sigma_s)_{MAX} = 2.25 \times 0.13 \times 7.2 \times 0.8605$$

$$= 1.812\ ksi > \sigma_s = 1.56\ ksi$$

ok

STABILITY:

$$A = \frac{1.92 \times 3.75/18}{(1 + 2 \times 18/27)^{1/2}} = 0.2619$$

$$B = \frac{2.67}{9.2\ (1 + 18/108)} = 0.2487$$

$2A > B \Rightarrow$ NEED TO CHECK:

$$\sigma_s \le \frac{0.13 \times 7.2}{0.2619 - 0.2487} = 71.4\ ksi$$

$> 1.56\ ksi$

ok

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P_{DL} = :	441.6 kips	σ_s = :	1.560 ksi
P_{LL} = :	316.6 kips	σ_L = :	0.651 ksi
P_{TL} = :	758.2 kips	σ_{BAL} = :	1.944 ksi

B = : 0.0345497

h_s req'd = : 0.098 inches

h_s used = : 0.125

Δ = :	0.00 inches
θ_{DL} = :	0.00410 radians
θ_{LL} = :	0.00170 radians
θ_{ML} = :	0.00000 radians
θ_{TL} = :	0.00580 radians
Shore A Hardness = :	60
G_{MIN} = :	130 psi
G_{MAX} = :	200 psi
F_y = :	36 ksi
F_{TH} = :	24 ksi
W = :	27 inches
L = :	18 inches
# of holes in pad = :	0
hole diameter = :	1 inches
net area = :	486 in^2
h_{ri} = :	0.75 inches
h_{ro} = :	0.375 inches
Shape Factor, S = : (C14.7.5.1-1)	7.20

	# of internal elastomer layers								
	1	2	3	4	5	6	7	8	9
h_{rt} = :	1.500	2.250	3.000	3.750	4.500	5.250	6.000	6.750	7.500
A = :	0.01455	0.02182	0.02910	0.03637	0.04364	0.05092	0.05819	0.06547	0.07274
h_{total} = :	1.750	2.625	3.500	4.375	5.250	6.125	7.000	7.875	8.750
Shear, V (kips) = :	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Moment, M (ft-k) = :	210.4	140.3	105.2	84.2	70.1	60.1	52.6	46.8	42.1
Method A: AASHTO LRFD Section 14.7.6									
$(h_{total})_{MAX}$ = :	STABILITY CHECK: AASHTO LRFD Section 14.7.6.3.6 (LFD 17th Edition – Section 14.6.6.3.6)								
6.000	OK	OK	OK	OK	OK	[redacted]			
$(\sigma_s)_{MAX}$ = :	COMPRESSION CHECK: AASHTO LRFD Section 14.7.6.3.2 (LFD 17th Edition – Section 14.6.6.3.2)								
1.030	[redacted]								
$(\sigma_s)_{MIN}$ = :	ROTATION CAPACITY CHECK: AASHTO LRFD Section 14.7.6.3.5d (LFD 17th Edition – Section 14.6.6.3.5b)								
	1.203	0.802	0.601	0.481	0.401	0.344	0.301	0.267	0.241
$(h_{rt})_{MIN}$ = :	SHEAR DEFORMATION CHECK: AASHTO LRFD Section 14.7.6.3.4 (LFD 17th Edition – Section 14.6.6.3.4)								
0.000	OK	OK	OK	OK	OK	OK	OK	OK	OK
Method B: AASHTO LRFD Section 14.7.5									
$(h_{rt})_{MIN}$ = :	SHEAR DEFORMATION CHECK: AASHTO LRFD Section 14.7.5.3.4 (LFD 17th Edition – Section 14.6.5.3.4)								
0.000	OK	OK	OK	OK	OK	OK	OK	OK	OK
$(\sigma_s)_{MAX}$ = :	COMPRESSION CHECK: AASHTO LRFD Section 14.7.5.3.2 (LFD 17th Edition – Section 14.6.5.3.2)								
1.750	OK	OK	OK	OK	OK	OK	OK	OK	OK
$(\sigma_L)_{MAX}$ = :	LIVE LOAD COMPRESSION CHECK: AASHTO 17th Edition – Section 14.6.5.3.2								
0.936	OK	OK	OK	OK	OK	OK	OK	OK	OK
$(\sigma_s)_{MAX}$ = :	ROTATIONAL CAPACITY CHECK 1: AASHTO LRFD Section 14.7.5.3.5 (LFD 17th Edition – Section 14.6.5.3.5)								
	[redacted]	1.714	1.812	1.871	1.910	1.938	1.959	1.975	1.989
$(\sigma_s)_{MIN}$ = :	ROTATIONAL CAPACITY CHECK 2: AASHTO LRFD Section 14.7.5.3.5 (LFD 17th Edition – Section 14.6.5.3.5								
	[redacted]		1.203	0.962	0.802	0.687	0.601	0.535	0.481
$(\sigma_s)_{MAX}$ = :	STABILITY CHECK: AASHTO LRFD Section 14.7.5.3.6 (LFD 17th Edition – Section 14.6.5.3.6)								
	-6.499	-10.214	-23.836	71.430	14.295	7.942	5.499	4.205	3.404
L_{BAL} = :	PAD LENGTH "L" TO GIVE BALANCED DESIGN								
	12.60	17.81	21.82	25.19	28.17	30.85	33.33	35.63	37.79
W_{BAL} = :	PAD LENGTH "W" TO GIVE BALANCED DESIGN								
	30.97	21.90	17.88	15.49	13.85	12.64	11.71	10.95	10.32

COUNTY: CROSSING: NOTES

FIELD SPLICES: FACTORED M & V

FT·KIPS
KIPS

GIRDER	(COMPOSITE) SPLICE #1	(NON-COMP.) SPLICE #2
1	48' OF SPAN 1 18×3/4 : 21×1 M_{DL} = 134 V_{DL} = 46 M_{SDL} = 93 V_{SDL} = 16 M_{LL+I} = 6,201 V_{LL+I} = 137 V_r = 95	48.25' OF SPAN 2 18×3/4 : 21×1 M_{DL} = -848 V_{DL} = 136 M_{SDL} = -113 V_{SDL} = 49 M_{LL+I} = -2,880 V_{LL+I} = 273 V_r = 118
2	47.79' OF SPAN 1 18×3/4 : 21×7/8 M_{DL} = 296 V_{DL} = 38 M_{SDL} = 138 V_{SDL} = 14 M_{LL+I} = 4,494 V_{LL+I} = 122 V_r = 88	48.04' OF SPAN 2 18×3/4 : 21×7/8 M_{DL} = -648 V_{DL} = 114 M_{SDL} = -91 V_{SDL} = 42 M_{LL+I} = -2,001 V_{LL+I} = 212 V_r = 110
3	47.58' OF SPAN 1 18×3/4 : 21×7/8 M_{DL} = 387 V_{DL} = 37 M_{SDL} = 166 V_{SDL} = 13 M_{LL+I} = 4,545 V_{LL+I} = 124 V_r = 90	47.83' OF SPAN 2 18×3/4 : 21×7/8 M_{DL} = -656 V_{DL} = 114 M_{SDL} = -104 V_{SDL} = 42 M_{LL+I} = -2,134 V_{LL+I} = 213 V_r = 114
4	47.375' OF SPAN 1 18×3/4 : 21×7/8 M_{DL} = 410 V_{DL} = 36 M_{SDL} = 174 V_{SDL} = 13 M_{LL+I} = 4,526 V_{LL+I} = 124 V_r = 90	47.63' OF SPAN 2 18×3/4 : 21×7/8 M_{DL} = -657 V_{DL} = 114 M_{SDL} = -106 V_{SDL} = 42 M_{LL+I} = -2,133 V_{LL+I} = 214 V_r = 114
5	47.17' OF SPAN 1 18×3/4 : 21×7/8 M_{DL} = 373 V_{DL} = 37 M_{SDL} = 169 V_{SDL} = 13 M_{LL+I} = 4,325 V_{LL+I} = 120 V_r = 88	47.42' OF SPAN 2 18×3/4 : 21×7/8 M_{DL} = -655 V_{DL} = 115 M_{SDL} = -101 V_{SDL} = 42 M_{LL+I} = -1,981 V_{LL+I} = 210 V_r = 109
6	46.96' OF SPAN 1 18×3/4 : 21×7/8 M_{DL} = 295 V_{DL} = 39 M_{SDL} = 152 V_{SDL} = 13 M_{LL+I} = 4,908 V_{LL+I} = 115 V_r = 82	47.21' OF SPAN 2 18×3/4 : 21×7/8 M_{DL} = -657 V_{DL} = 120 M_{SDL} = -91 V_{SDL} = 44 M_{LL+I} = -2,212 V_{LL+I} = 226 V_r = 101

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COUNTY: CROSSING: NOTES

GIRDER 1, SPLICE 2, NON-COMPOSITE

$M_u = 113 + 848 + 2{,}880$
$= 3{,}841$ FT·KIPS

18" × 3/4" FLANGES
69" × 1/2" WEB

ASSUME 4 ROWS OF BOLTS ACROSS FLANGE, 7/8" BOLTS, 15/16" HOLES, 1" HOLES FOR CALC. — 6.13.2.4.2

$A_n = (18 - 4 \times 1)(3/4) = 10.5 \text{ IN}^2$ — 6.10.3.6 &
$A_g = 18 \times 3/4 = 13.5 \text{ IN}^2$ — 6.8.3

$\phi_u = 0.80$ — 6.5.4.2
$\phi_y = 0.95$

$F_u = 70$ KSI — 6.4.1.1
$F_{yf} = 50$ KSI

$$B = \frac{10.5}{13.5}\left(\frac{0.80 \times 70}{0.95 \times 50} - 1\right) \geq 0.0$$

$B = 0.139$

$A_e = 10.5 + .139(13.5)$
$= 12.4 \text{ IN}^2$

$I = 1/2(69)^3/12 + 2(12.4)(34.875)^2$
$= 43{,}851 \text{ IN}^4$

WEB SPLICE SHEAR: — 6.13.6.1.4b

$d_o = 10.25$ FT, STIFFENER SPACING AT SPLICE

$d_o/D = 10.25/5.75 = 1.78$

$D/t_w = 69/.5 = 138$

$$k = 5 + \frac{5}{1.78^2} = 6.58$$

$(29{,}000 \times 6.58/50)^{1/2} = 62$

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COUNTY: CROSSING: | NOTES

$$1.10\sqrt{\frac{Ek}{F_{yw}}} = 1.10 \times 62 = 68$$ 6.10.7.3.3

$$1.38\sqrt{\frac{Ek}{F_{yw}}} = 1.38 \times 62 = 86$$

$D/t_w = 138 > 86$, THUS:

$$C = \frac{1.52}{138^2}\left(\frac{29{,}000 \times 6.58}{50}\right) = 0.305$$

$$V_n = .58 \times 50 \times 69 \times \tfrac{1}{2}\left(0.305 + \frac{0.87 \times .695}{(1 + 1.78^2)^{1/2}}\right)$$

$\underline{V_n = 601 \text{ KIPS}}$ $\quad V_r = \phi V_n = V_n$ $\quad$ $\phi_v = 1.0$

$V_u = 136 + 49 + 273 = 458 \text{ KIPS}$
$> .5 V_r = 300 \text{ KIPS}$

$$V_{uw} = \left(\frac{458 + 601}{2}\right) \doteq \boxed{V_{uw} = 530 \text{ KIPS}}$$ 6.13.6.1.4b-2

$$f_{cf} = \frac{(3{,}841 \text{ FT·K} \times 12 \text{ IN/FT})(34.875 \text{ IN.})}{43{,}851}$$

$f_{cf} = 36.7 \text{ ksi}$

$$F_{cf} = \frac{36.7 + 50}{2} = \boxed{43.3 \text{ ksi, CONTROLS}}$$ 6.13.6.1.4c

$\geq .75 \times 50 = 37.5 \text{ ksi}$

$$M_{uw} = \frac{\tfrac{1}{2}(69)^2}{12}(43.3 - 36.7) = \underline{\underline{1{,}309 \text{ FT·KIPS}}}$$

$$H_{uw} = \frac{\tfrac{1}{2}(69)}{2}(43.3 - 36.7) = \underline{\underline{114 \text{ KIPS}}}$$

ECCENTRICITY OF WEB SPLICE
$= 1\tfrac{1}{2} + 1\tfrac{1}{2} = 3''$

$$M_{uw} = 1{,}309 \text{ FT·K} + 530 \text{ K} (3''/12)$$

$$\underline{\underline{M_{uw} = 1{,}442 \text{ FT·KIPS}}}$$

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DT-1420 (Structures Design Form 3)

COUNTY: CROSSING: NOTES

FACTORED SHEAR RESISTANCE OF BOLTS — 6.13.2.7

$R_n = .48\,A_b\,F_{ub}\,N_s$

$N_s = 2$, DOUBLE SHEAR

$F_{ub} = 120$ ksi — 6.4.3.1

$A_b = \pi (7/8)^2/4 = 0.601$ IN^2

$R_n = .48 \times .601 \times 120 \times 2$
$= 69.3$ KIPS /BOLT
$\phi R_n = .8 \times 69.3 = 55.4$ K /BOLT

SLIP RESISTANCE OF BOLTS — 6.13.2.8

$K_h = 1.0$, STD HOLES

$K_s = 0.5$, SPECIFY CLASS B SURFACE COND. (UNPAINTED, BLAST-CLEANED)

$P_T = 39$ KIPS, A325 7/8" PRETENSION

$R_n = 1.0 \times .5 \times 39 \times 2$
$= 39$ KIPS /BOLT — DOUBLE SHEAR

BOLTS REQ'D FOR SHEAR:

$N_b = \dfrac{43.3\text{ksi} \times 12.4\text{IN}^2}{55.4\text{K/BOLT}} = 9.7$

$\boxed{\text{USE 12 BOLTS EACH SIDE OF ℄ SPLICE IN EACH FLANGE 7/8" A325}}$

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COUNTY: CROSSING: | NOTES

FOR SLIP-CRITICAL SERVICE LIMIT STATE II:

$$M_u = \frac{113}{1.25} + \frac{848}{1.25} + \frac{2{,}880}{1.75} \times 1.30$$

$M_u = 2{,}908$ FT·KIPS

GROSS SECTION IS TO BE USED FOR STRESS CALC'S FOR SLIP-CRITICAL CONDITIONS — 6.13.6.1.4a

$I = 69^3(1/2)/12 + 2(18 \times 3/4)(34.875)^2$
$= 46{,}527 \text{ IN}^4$

$f = 2{,}908 \times 12 \times 34.875/46{,}527$
$= 26.2$ KSI

$F = 26.2 \times 18 \times 3/4 = 353$ KIPS

$N_b = 353^k / 39^k/\text{BOLT} = 9.1$ BOLTS

(12 STILL OK)

FLANGE SPLICE PLATES:

TENSION $= 43.3\text{KSI} \times 12.4\text{IN}^2$
$= 537$ KIPS

$U = 1.00$ — 6.8.2.2

YIELD: $\phi_y P_n = \phi_y F_y A_g$
$= .95(50) A_g = 47.5 A_g$

FRACTURE: $\phi_u P_n = \phi_u F_u A_n U$
$= .80(70)(1.00) A_n$
$= 56.0 A_n$

$A_n \geq 537/56 = 9.59 \text{ IN}^2$

$A_g \geq 537/47.5 = 11.31 \text{ IN}^2$

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OUTER PL WIDTH = 18" (1 PLATE)
INNER PL WIDTH = 8 1/4" (2 PLATES)

TRY 5/16" OUTER PL
3/8" INNER PL'S

$$A_n = (18-4\times1)(5/16)+2(8\tfrac{1}{4}-2\times1)(3/8)$$
$$= 4.375+4.6875$$
$$= 9.06\ IN^2 < 9.59\ IN^2,\ \text{No Good}$$

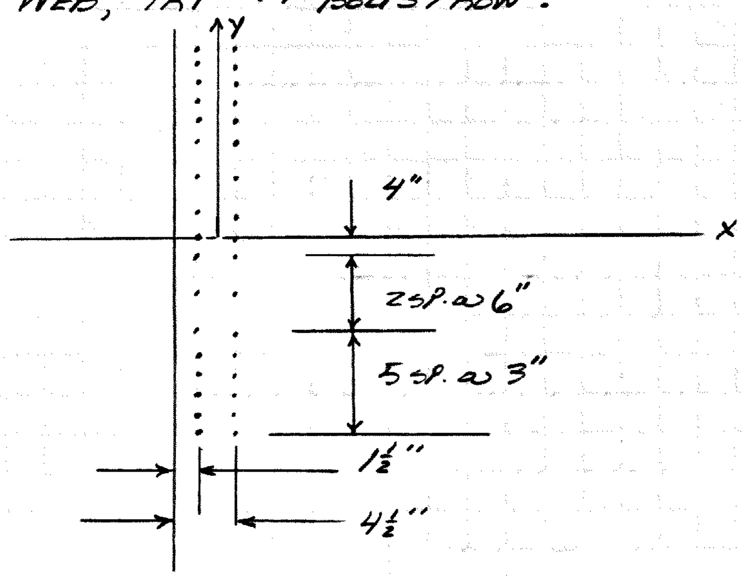
TRY 3/8" OUTER PL
7/16" INNER PL'S

$$A_n = (18-4\times1)(3/8)+2(8\tfrac{1}{4}-2\times1)(7/16)$$
$$= 5.25+5.47$$
$$= 10.72\ IN^2 > 9.59\ IN^2,\ OK$$

$$A_g = 18\times 3/8 + 2\times 8\tfrac{1}{4}\times 7/16$$
$$= 13.97\ IN^2 > 11.31\ IN^2,\ OK$$

18" × 3/8" × 19" OUTER PL
2, 8 1/4" × 7/16" × 19" INNER PL'S

WEB, TRY 17 BOLTS/ROW:



$$I_y = 17(1\tfrac{1}{2})^2(2) = 76.5\ \text{BOLT}\cdot IN^2$$

$$I_x = 4(4^2+10^2+16^2+19^2+22^2+25^2+28^2+31^2)$$
$$= 14{,}348\ \text{BOLT}\cdot IN^2$$

$$J = I_x + I_y = 14{,}425\ \text{BOLT}\cdot IN^2$$

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COUNTY: CROSSING: NOTES

WEB BOLT SHEAR, STRENGTH I LIMIT STATE:

$f_{VV} = 530^k / 34 \text{ BOLTS} = 15.6^k/\text{BOLT}$

$f_{VM} = 1{,}442 \times 12 \times 1\tfrac{1}{2} / 14{,}425 = 1.8^k/\text{BOLT}$

$f_{HV} = 114k / 34 \text{ BOLTS} = 3.35^k/\text{BOLT}$

$f_{HM} = 1{,}442 \times 12 \times 31 / 14{,}425 = 37.2^k/\text{BOLT}$

$f_V = 15.6 + 1.8 = 17.4k/\text{BOLT}$

$f_h = 3.35 + 37.2 = 40.6^k/\text{BOLT}$

$f_r = (17.4^2 + 40.6^2)^{1/2} = 44.1^k/\text{BOLT} < 55.4^k$ OK

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$\frac{\text{SERVICE II}}{\text{STRENGTH I}} = \frac{2{,}908}{3{,}841} = 0.757$

$f_r = .757 \times 44.1 = 33.4^k/\text{BOLT} < 39^k/\text{BOLT}$ OK FOR SLIP

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WEB ℞'s: $I = t(65)^3/12 - (t \times 1)(4^2 + 10^2 + 16^2 + 19^2 + 22^2 + 25^2 + 28^2 + 31^2)$

$A = t(65-17) = 48t$

$= 22{,}885t - 3{,}587t$
$= 19{,}298t$

$S = 19{,}298t / 32.5$
$= 594t$

$f_{℞} = \frac{1442 \times 12}{594t} + \frac{114}{48t} \leq 50 \text{ ksi}$

$t \approx 0.63''$ TOTAL
$\div 2$ ℞'s
$= 0.32''$
WEB ℞

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WEB ℞ SHEAR:

$$f_v = \frac{530^K}{48t} \leq .58 \times 50$$

$t \geq 0.38''$ TOTAL
$\div 2$ ℞'s
$= 0.19''$
WEB ℞

FLEXURE CONTROLS,
$t \geq 0.32''$

USE 3/8" WEB
SPLICE ℞'S
WITH 34, 7/8"
A325 BOLTS
EACH SIDE OF
℄ SPLICE

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GIRDER:	I-55 / G1	SPLICE:	1

Top Flange

b = :	18	inches
t = :	0.75	inches
F_y = :	50	ksi

Bottom Flange

b = :	18	inches
t = :	0.75	inches
F_y = :	50	ksi

Web

D = :	69	inches
t = :	0.5	inches
F_y = :	50	ksi

Spacings

Cross-Frames = :	20	ft
Transverse Stiffeners = :	10	ft
Select Compression Flange :	Top	

Moments (Service Level)

DC (Non-Composite) = :	107	ft-k
DW (Composite) = :	62	ft-k
LL+I = :	3,543	ft-k

Shears (Service Level)

DC (Non-Composite) = :	37	kips
DW (Composite) = :	11	kips
LL+I = :	78	kips

Slab

Eff. Width = :	109	inches
Thickness = :	8.5	inches
Eccentricity = :	5.25	inches
Rebar Area = :	0	in^2
Rebar d = :	3	inches
Rebar F_y = :	60	ksi
f'_c = :	3.0	ksi
n = :	9.29	
3n = :	27.87	
Esteel = :	29,000	ksi

Y_{CG} @ : 35.25 inches

Section Properties	Girder	Comp (n)	Comp(3n)	Actual Stresses ksi	Stress Limit ksi
S_{top}, in^3	1,334	16,979	4,776	5.82	48.07
S_{bot}, in^3	1,334	1,933	1,759	40.33	50.00
J = :	8				

Spacing arrangement of Web Splice Bolts:
Starting at c.l. Splice and working "out":

Bearing Case			ix =	9,532.7	bolt-in^2
0.5	spaces @	4 3/16	iy =	54.0	bolt-in^2
5.0	spaces @	6	j =	9,586.7	bolt-in^2
0.0	spaces @	3	Force =	67.1	kips
Slip Case			ix =	14,127.5	bolt-in^2
1.0	spaces @	4 3/16	iy =	67.5	bolt-in^2
3.0	spaces @	6	j =	14,195.0	bolt-in^2
3.0	spaces @	3	Force =	36.6	kips

Design Parameters

R_{HYB} = : 1.000		f_{cu} = :	40.33 ksi
α_{TOP} = : 0.121		F_{cu} = :	45.17 ksi
α_{BOTT} = : 0.807		f_{ncu} = :	5.82 ksi
Bottom Flange Controls		F_{ncu} = :	37.50 ksi
No. of slip planes =:	2	Top Flange Design Stress = :	37.50 ksi
		Bottom Flange Design Stress = :	45.17 ksi
		Section Shear Capacity = :	612.04 kips
SERVICE II Shear =:	149.24	STRENGTH I Shear = :	199.00 kips
STR-I/SER-II =:	1.33	Splice Design Shear = :	298.50 kips
		Web Moment from Applied Loads = :	854.42 ft-kips
		Horizontal Design Web Force = :	666.66 kips
		Bolt Size = :	7/8 inches
		Bolt Type = :	A325
		Threads in Shear Plane = :	NO
		Edge Distance = :	1.500 inches
		Minimum Bolt Spacing = :	3.000 inches
		# bolt rows each side of Web Splice = :	2
		# bolt rows each side of Flange Splice = :	4
		Web Moment from eccentricity of Shear = :	87.06 ft-kips
		Bolt Capacity for STRENGTH I Limit State = :	69.27 kips
		Contact Surface Finish = :	Class B
		Friction Coefficient = :	0.50
		Hole Type = :	STD
		Bolt Slip-Resistance for SERVICE II Limit State = :	39.00 kips

Top Flange Splice :	W, in.	t, in.	L, in.	Grade	Wt, lbs
Outer Plate :	18.000	5/16	19.000	50	30
Inner Plates (2) :	8.250	3/8	19.000	50	33
Filler Plate :	18.000	1/4	9.500	50	12
# bolts/row :	1.83 (bearing)				
# bolts/row :	2.41 (slip)				
# bolts/row :	3				
total # bolts :	24	7/8	4 7/16	A325	18
Bottom Flange Splice :	**W, in.**	**t, in.**	**L, in.**	**Grade**	**Wt, lbs**
Outer Plate :	18.000	3/8	19.000	50	36
Inner Plates (2) :	8.250	7/16	19.000	50	39
Filler Plate :	18.000	1/4	9.500	50	12
# bolts/row :	2.20 (bearing)				
# bolts/row :	2.90 (slip)				
# bolts/row :	3				
total # bolts :	24	7/8	4 9/16	A325	19
Web Splice :	**W, in.**	**t, in.**	**L, in.**	**Grade**	**Wt, lbs**
Web Plates (2) :	13.000	3/8	67.188	50	186
# bolts/row :	12 (bearing)				
# bolts/row :	14 (slip)				
total # bolts :	56	7/8	4 1/4	A325	41
					426

GIRDER: I-55 / G2 **SPLICE:** 1

Top Flange		
b = :	18	inches
t = :	0.75	inches
F_y = :	50	ksi
Bottom Flange		
b = :	18	inches
t = :	0.75	inches
F_y = :	50	ksi
Web		
D = :	69	inches
t = :	0.5	inches
F_y = :	50	ksi
Spacings		
Cross-Frames = :	20	ft
Transverse Stiffeners = :	10	ft
Select Compression Flange :	Top	

Moments (Service Level)		
DC (Non-Composite) = :	237	ft-k
DW (Composite) = :	92	ft-k
LL+I = :	2,568	ft-k
Shears (Service Level)		
DC (Non-Composite) = :	30	kips
DW (Composite) = :	9	kips
LL+I = :	70	kips
Slab		
Eff. Width = :	109	inches
Thickness = :	8.5	inches
Eccentricity = :	5.25	inches
Rebar Area = :	0	in^2
Rebar d = :	3	inches
Rebar F_y = :	60	ksi
f'_c = :	3.0	ksi
n = :	9.29	
3n = :	27.87	
Esteel = :	29,000	ksi

Y_{CG} @ : 35.25 inches

Section Properties	Girder	Comp (n)	Comp(3n)	Actual Stresses ksi	Stress Limit ksi
S_{top}, in^3	1,334	16,979	4,776	6.19	48.07
S_{bot}, in^3	1,334	1,933	1,759	31.50	50.00
J = :	8				

Spacing arrangement of Web Splice Bolts:
Starting at c.l. Splice and working "out":

Bearing Case			ix =	9,532.7	bolt-in^2
0.5	spaces @	4 3/16	iy =	54.0	bolt-in^2
5.0	spaces @	6	j =	9,586.7	bolt-in^2
0.0	spaces @	3	Force =	60.3	kips
Slip Case			ix =	12,918.5	bolt-in^2
0.5	spaces @	4 3/16	iy =	63.0	bolt-in^2
4.0	spaces @	6	j =	12,981.5	bolt-in^2
2.0	spaces @	3	Force =	35.5	kips

Design Parameters				
R_{HYB} = : 1.000		f_{cu} = :	31.50	ksi
α_{TOP} = : 0.129		F_{cu} = :	40.75	ksi
α_{BOTT} = : 0.630		f_{ncu} = :	6.19	ksi
Bottom Flange Controls		F_{ncu} = :	37.50	ksi
No. of slip planes =:	2	Top Flange Design Stress = :	37.50	ksi
		Bottom Flange Design Stress = :	40.75	ksi
		Section Shear Capacity = :	612.04	kips
SERVICE II Shear =:	130.36	STRENGTH I Shear = :	174.00	kips
STR-I/SER-II =:	1.33	Splice Design Shear = :	261.00	kips
		Web Moment from Applied Loads = :	805.92	ft-kips
		Horizontal Design Web Force = :	564.91	kips
		Bolt Size = :	7/8	inches
		Bolt Type = :	A325	
		Threads in Shear Plane = :	NO	
		Edge Distance = :	1.500	inches
		Minimum Bolt Spacing = :	3.000	inches
		# bolt rows each side of Web Splice = :	2	
		# bolt rows each side of Flange Splice = :	4	
		Web Moment from eccentricity of Shear = :	76.13	ft-kips
		Bolt Capacity for STRENGTH I Limit State = :	69.27	kips
		Contact Surface Finish = :	Class B	
		Friction Coefficient = :	0.50	
		Hole Type = :	STD	
		Bolt Slip-Resistance for SERVICE II Limit State = :	39.00	kips

Top Flange Splice :	W, in.	t, in.	L, in.	Grade	Wt, lbs
Outer Plate :	18.000	5/16	19.000	50	30
Inner Plates (2) :	8.250	3/8	19.000	50	33
Filler Plate :	18.000	1/8	9.500	50	6
# bolts/row :	1.83 (bearing)				
# bolts/row :	2.41 (slip)				
# bolts/row :	3				
total # bolts :	24	7/8	4 7/16	A325	18
Bottom Flange Splice :	**W, in.**	**t, in.**	**L, in.**	**Grade**	**Wt, lbs**
Outer Plate :	18.000	3/8	19.000	50	36
Inner Plates (2) :	8.250	7/16	19.000	50	39
Filler Plate :	18.000	1/8	9.500	50	6
# bolts/row :	1.99 (bearing)				
# bolts/row :	2.62 (slip)				
# bolts/row :	3				
total # bolts :	24	7/8	4 9/16	A325	19
Web Splice :	**W, in.**	**t, in.**	**L, in.**	**Grade**	**Wt, lbs**
Web Plates (2) :	13.000	5/16	67.188	50	155
# bolts/row :	12 (bearing)				
# bolts/row :	13 (slip)				
total # bolts :	52	7/8	4 1/8	A325	37
					379

HUFF

GIRDER:	I-55 / 63	SPLICE:	1

Top Flange		
b = :	18	inches
t = :	0.75	inches
F_y = :	50	ksi
Bottom Flange		
b = :	18	inches
t = :	0.75	inches
F_y = :	50	ksi
Web		
D = :	69	inches
t = :	0.5	inches
F_y = :	50	ksi
Spacings		
Cross-Frames = :	20	ft
Transverse Stiffeners = :	10	ft
Select Compression Flange :	Top	

Moments (Service Level)		
DC (Non-Composite) = :	310	ft-k
DW (Composite) = :	111	ft-k
LL+I = :	2,597	ft-k
Shears (Service Level)		
DC (Non-Composite) = :	30	kips
DW (Composite) = :	9	kips
LL+I = :	71	kips
Slab		
Eff. Width = :	109	inches
Thickness = :	8.5	inches
Eccentricity = :	5.25	inches
Rebar Area = :	0	in^2
Rebar d = :	3	inches
Rebar F_y = :	60	ksi
f'_c = :	3.0	ksi
n = :	9.29	
3n = :	27.87	
Esteel = :	29,000	ksi

Y_{CG} @ : 35.25 inches

Section Properties	Girder	Comp (n)	Comp(3n)	Actual Stresses ksi	Stress Limit ksi
S_{top}, in^3	1,334	16,979	4,776	7.11	48.07
S_{bot}, in^3	1,334	1,933	1,759	32.83	50.00
J = :	8				

Spacing arrangement of Web Splice Bolts:
Starting at c.l. Splice and working "out":

Bearing Case			ix =	9,532.7	bolt-in²
0.5	spaces @	4 3/16	iy =	54.0	bolt-in²
5.0	spaces @	6	j =	9,586.7	bolt-in²
0.0	spaces @	3	Force =	61.1	kips
Slip Case			ix =	12,918.5	bolt-in²
0.5	spaces @	4 3/16	iy =	63.0	bolt-in²
4.0	spaces @	6	j =	12,981.5	bolt-in²
2.0	spaces @	3	Force =	36.0	kips

Design Parameters

R_{HYB} = : 1.000		f_{cu} = :	32.83 ksi
α_{TOP} = : 0.148		F_{cu} = :	41.41 ksi
α_{BOTT} = : 0.657		f_{ncu} = :	7.11 ksi
Bottom Flange Controls		F_{ncu} = :	37.50 ksi
No. of slip planes =:	2	Top Flange Design Stress = :	37.50 ksi
		Bottom Flange Design Stress = :	41.41 ksi
		Section Shear Capacity = :	612.04 kips
SERVICE II Shear =:	130.38	STRENGTH I Shear = :	174.00 kips
STR-I/SER-II =:	1.33	Splice Design Shear = :	261.00 kips
		Web Moment from Applied Loads = :	832.90 ft-kips
		Horizontal Design Web Force = :	559.64 kips
		Bolt Size = :	7/8 inches
		Bolt Type = :	A325
		Threads in Shear Plane = :	NO
		Edge Distance = :	1.500 inches
		Minimum Bolt Spacing = :	3.000 inches
		# bolt rows each side of Web Splice = :	2
		# bolt rows each side of Flange Splice = :	4
		Web Moment from eccentricity of Shear = :	76.13 ft-kips
		Bolt Capacity for STRENGTH I Limit State = :	69.27 kips
		Contact Surface Finish = :	Class B
		Friction Coefficient = :	0.50
		Hole Type = :	STD
		Bolt Slip-Resistance for SERVICE II Limit State = :	39.00 kips

Top Flange Splice :	W, in.	t, in.	L, in.	Grade	Wt, lbs
Outer Plate :	18.000	5/16	19.000	50	30
Inner Plates (2) :	8.250	3/8	19.000	50	33
Filler Plate :	18.000	1/8	9.500	50	6
# bolts/row :	1.83 (bearing)				
# bolts/row :	2.42 (slip)				
# bolts/row :	3				
total # bolts :	24	7/8	4 7/16	A325	18
Bottom Flange Splice :	**W, in.**	**t, in.**	**L, in.**	**Grade**	**Wt, lbs**
Outer Plate :	18.000	3/8	19.000	50	36
Inner Plates (2) :	8.250	7/16	19.000	50	39
Filler Plate :	18.000	1/8	9.500	50	6
# bolts/row :	2.02 (bearing)				
# bolts/row :	2.67 (slip)				
# bolts/row :	3				
total # bolts :	24	7/8	4 9/16	A325	19
Web Splice :	**W, in.**	**t, in.**	**L, in.**	**Grade**	**Wt, lbs**
Web Plates (2) :	13.000	5/16	67.188	50	155
# bolts/row :	12 (bearing)				
# bolts/row :	13 (slip)				
total # bolts :	52	7/8	4 1/8	A325	37
					379

GIRDER: I-55 / 64 **SPLICE:** 1

Top Flange		
b = :	18	inches
t = :	0.75	inches
F_y = :	50	ksi
Bottom Flange		
b = :	18	inches
t = :	0.75	inches
F_y = :	50	ksi
Web		
D = :	69	inches
t = :	0.5	inches
F_y = :	50	ksi
Spacings		
Cross-Frames = :	20	ft
Transverse Stiffeners = :	10	ft
Select Compression Flange :	Top	

Moments (Service Level)		
DC (Non-Composite) = :	328	ft-k
DW (Composite) = :	116	ft-k
LL+I = :	2,586	ft-k
Shears (Service Level)		
DC (Non-Composite) = :	29	kips
DW (Composite) = :	9	kips
LL+I = :	71	kips
Slab		
Eff. Width = :	109	inches
Thickness = :	8.5	inches
Eccentricity = :	5.25	inches
Rebar Area = :	0	in^2
Rebar d = :	3	inches
Rebar F_y = :	60	ksi
f'_c = :	3.0	ksi
n = :	9.29	
3n = :	27.87	
Esteel = :	29,000	ksi

Y_{CG} @ : 35.25 inches

Section Properties	Girder	Comp (n)	Comp(3n)	Actual Stresses ksi	Stress Limit ksi
S_{top}, in^3	1,334	16,979	4,776	7.32	48.07
S_{bot}, in^3	1,334	1,933	1,759	32.97	50.00
J = :	8				

Spacing arrangement of Web Splice Bolts:
Starting at c.l. Splice and working "out":

Bearing Case			ix =	9,532.7	bolt-in²
0.5	spaces @	4 3/16	iy =	54.0	bolt-in²
5.0	spaces @	6	j =	9,586.7	bolt-in²
0.0	spaces @	3	Force =	61.2	kips
Slip Case			ix =	12,918.5	bolt-in²
0.5	spaces @	4 3/16	iy =	63.0	bolt-in²
4.0	spaces @	6	j =	12,981.5	bolt-in²
2.0	spaces @	3	Force =	36.0	kips

Design Parameters				
R_{HYB} = : 1.000		f_{cu} = :	32.97	ksi
α_{TOP} = : 0.152		F_{cu} = :	41.48	ksi
α_{BOTT} = : 0.659		f_{ncu} = :	7.32	ksi
Bottom Flange Controls		F_{ncu} = :	37.50	ksi
No. of slip planes =:	2	Top Flange Design Stress = :	37.50	ksi
		Bottom Flange Design Stress = :	41.48	ksi
		Section Shear Capacity = :	612.04	kips
SERVICE II Shear =:	129.58	STRENGTH I Shear = :	173.00	kips
STR-I/SER-II =:	1.34	Splice Design Shear = :	259.50	kips
		Web Moment from Applied Loads = :	838.13	ft-kips
		Horizontal Design Web Force = :	556.65	kips
		Bolt Size = :	7/8	inches
		Bolt Type = :	A325	
		Threads in Shear Plane = :	NO	
		Edge Distance = :	1.500	inches
		Minimum Bolt Spacing = :	3.000	inches
		# bolt rows each side of Web Splice = :	2	
		# bolt rows each side of Flange Splice = :	4	
		Web Moment from eccentricity of Shear = :	75.69	ft-kips
		Bolt Capacity for STRENGTH I Limit State = :	69.27	kips
		Contact Surface Finish = :	Class B	
		Friction Coefficient = :	0.50	
		Hole Type = :	STD	
		Bolt Slip-Resistance for SERVICE II Limit State = :	39.00	kips

Top Flange Splice :	W, in.	t, in.	L, in.	Grade	Wt, lbs
Outer Plate :	18.000	5/16	19.000	50	30
Inner Plates (2) :	8.250	3/8	19.000	50	33
Filler Plate :	18.000	1/8	9.500	50	6
# bolts/row :	1.83 (bearing)				
# bolts/row :	2.42 (slip)				
# bolts/row :	3				
total # bolts :	24	7/8	4 7/16	A325	18
Bottom Flange Splice :	**W, in.**	**t, in.**	**L, in.**	**Grade**	**Wt, lbs**
Outer Plate :	18.000	3/8	19.000	50	36
Inner Plates (2) :	8.250	7/16	19.000	50	39
Filler Plate :	18.000	1/8	9.500	50	6
# bolts/row :	2.02 (bearing)				
# bolts/row :	2.67 (slip)				
# bolts/row :	3				
total # bolts :	24	7/8	4 9/16	A325	19
Web Splice :	**W, in.**	**t, in.**	**L, in.**	**Grade**	**Wt, lbs**
Web Plates (2) :	13.000	5/16	67.188	50	155
# bolts/row :	12 (bearing)				
# bolts/row :	13 (slip)				
total # bolts :	52	7/8	4 1/8	A325	37
					379

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GIRDER: I-55 / G5 **SPLICE:** 1

Top Flange		
b = :	18	inches
t = :	0.75	inches
F_y = :	50	ksi
Bottom Flange		
b = :	18	inches
t = :	0.75	inches
F_y = :	50	ksi
Web		
D = :	69	inches
t = :	0.5	inches
F_y = :	50	ksi
Spacings		
Cross-Frames = :	20	ft
Transverse Stiffeners = :	10	ft
Select Compression Flange :	Top	

Moments (Service Level)		
DC (Non-Composite) = :	298	ft-k
DW (Composite) = :	113	ft-k
LL+I = :	2,471	ft-k
Shears (Service Level)		
DC (Non-Composite) = :	30	kips
DW (Composite) = :	9	kips
LL+I = :	69	kips
Slab		
Eff. Width = :	109	inches
Thickness = :	8.5	inches
Eccentricity = :	5.25	inches
Rebar Area = :	0	in^2
Rebar d = :	3	inches
Rebar F_y = :	60	ksi
f'_c = :	3.0	ksi
n = :	9.29	
3n = :	27.87	
Esteel = :	29,000	ksi

Y_{CG} @ : 35.25 inches

Section Properties	Girder	Comp (n)	Comp(3n)	Actual Stresses ksi	Stress Limit ksi
S_{top}, in^3	1,334	16,979	4,776	6.84	48.07
S_{bot}, in^3	1,334	1,933	1,759	31.36	50.00
J = :	8				

Spacing arrangement of Web Splice Bolts:
Starting at c.l. Splice and working "out":

Bearing Case						
0.5	spaces @	4 3/16		ix =	9,532.7	bolt-in^2
5.0	spaces @	6		iy =	54.0	bolt-in^2
0.0	spaces @	3		j =	9,586.7	bolt-in^2
				Force =	60.0	kips
Slip Case				ix =	12,918.5	bolt-in^2
0.5	spaces @	4 3/16		iy =	63.0	bolt-in^2
4.0	spaces @	6		j =	12,981.5	bolt-in^2
2.0	spaces @	3		Force =	35.3	kips

Design Parameters				
R_{HYB} = : 1.000		f_{cu} = :	31.36	ksi
α_{TOP} = : 0.142		F_{cu} = :	40.68	ksi
α_{BOTT} = : 0.627		f_{ncu} = :	6.84	ksi
Bottom Flange Controls		F_{ncu} = :	37.50	ksi
No. of slip planes =: 2		Top Flange Design Stress = :	37.50	ksi
		Bottom Flange Design Stress = :	40.68	ksi
		Section Shear Capacity = :	612.04	kips
SERVICE II Shear =:	127.41	STRENGTH I Shear = :	170.00	kips
STR-I/SER-II =:	1.33	Splice Design Shear = :	255.00	kips
		Web Moment from Applied Loads = :	819.06	ft-kips
		Horizontal Design Web Force = :	548.70	kips
		Bolt Size = :	7/8	inches
		Bolt Type = :	A325	
		Threads in Shear Plane = :	NO	
		Edge Distance = :	1.500	inches
		Minimum Bolt Spacing = :	3.000	inches
		# bolt rows each side of Web Splice = :	2	
		# bolt rows each side of Flange Splice = :	4	
		Web Moment from eccentricity of Shear = :	74.38	ft-kips
		Bolt Capacity for STRENGTH I Limit State = :	69.27	kips
		Contact Surface Finish = :	Class B	
		Friction Coefficient = :	0.50	
		Hole Type = :	STD	
		Bolt Slip-Resistance for SERVICE II Limit State = :	39.00	kips

Top Flange Splice :	W, in.	t, in.	L, in.	Grade	Wt, lbs
Outer Plate :	18.000	5/16	19.000	50	30
Inner Plates (2) :	8.250	3/8	19.000	50	33
Filler Plate :	18.000	1/8	9.500	50	6
# bolts/row :	1.83 (bearing)				
# bolts/row :	2.42 (slip)				
# bolts/row :	3				
total # bolts :	24	7/8	4 7/16	A325	18
Bottom Flange Splice :	**W, in.**	**t, in.**	**L, in.**	**Grade**	**Wt, lbs**
Outer Plate :	18.000	3/8	19.000	50	36
Inner Plates (2) :	8.250	7/16	19.000	50	39
Filler Plate :	18.000	1/8	9.500	50	6
# bolts/row :	1.98 (bearing)				
# bolts/row :	2.62 (slip)				
# bolts/row :	3				
total # bolts :	24	7/8	4 9/16	A325	19
Web Splice :	**W, in.**	**t, in.**	**L, in.**	**Grade**	**Wt, lbs**
Web Plates (2) :	13.000	5/16	67.188	50	155
# bolts/row :	12 (bearing)				
# bolts/row :	13 (slip)				
total # bolts :	52	7/8	4 1/8	A325	37
					379

HUFF

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GIRDER: I-55 / 66 **SPLICE:** 1

Top Flange		
b = :	18	inches
t = :	0.75	inches
F_y = :	50	ksi
Bottom Flange		
b = :	18	inches
t = :	0.75	inches
F_y = :	50	ksi
Web		
D = :	69	inches
t = :	0.5	inches
F_y = :	50	ksi
Spacings		
Cross-Frames = :	20	ft
Transverse Stiffeners = :	10	ft
Select Compression Flange :	Top	

Moments (Service Level)		
DC (Non-Composite) = :	236	ft-k
DW (Composite) = :	101	ft-k
LL+I = :	2,805	ft-k
Shears (Service Level)		
DC (Non-Composite) = :	31	kips
DW (Composite) = :	9	kips
LL+I = :	66	kips
Slab		
Eff. Width = :	109	inches
Thickness = :	8.5	inches
Eccentricity = :	5.25	inches
Rebar Area = :	0	in^2
Rebar d = :	3	inches
Rebar F_y = :	60	ksi
f'_c = :	3.0	ksi
n = :	9.29	
3n = :	27.87	
Esteel = :	29,000	ksi

Y_{CG} @ : 35.25 inches

Section Properties	Girder	Comp (n)	Comp(3n)	Actual Stresses ksi	Stress Limit ksi
S_{top}, in^3	1,334	16,979	4,776	6.50	48.07
S_{bot}, in^3	1,334	1,933	1,759	34.16	50.00
J = :	8				

Spacing arrangement of Web Splice Bolts:
Starting at c.l. Splice and working "out":

Bearing Case			ix =	9,532.7	bolt-in^2
0.5	spaces @	4 3/16	iy =	54.0	bolt-in^2
5.0	spaces @	6	j =	9,586.7	bolt-in^2
0.0	spaces @	3	Force =	61.9	kips
Slip Case			ix =	12,918.5	bolt-in^2
0.5	spaces @	4 3/16	iy =	63.0	bolt-in^2
4.0	spaces @	6	j =	12,981.5	bolt-in^2
2.0	spaces @	3	Force =	36.6	kips

Design Parameters

R_{HYB} = :	1.000	f_{cu} = :	34.16	ksi
α_{TOP} = :	0.135	F_{cu} = :	42.08	ksi
α_{BOTT} = :	0.683	f_{ncu} = :	6.50	ksi
Bottom Flange Controls		F_{ncu} = :	37.50	ksi
No. of slip planes =:	2	Top Flange Design Stress = :	37.50	ksi
		Bottom Flange Design Stress = :	42.08	ksi
		Section Shear Capacity = :	612.04	kips
SERVICE II Shear =:	125.30	STRENGTH I Shear = :	167.00	kips
STR-I/SER-II =:	1.33	Splice Design Shear = :	250.50	kips
		Web Moment from Applied Loads = :	828.06	ft-kips
		Horizontal Design Web Force = :	587.63	kips
		Bolt Size = :	7/8	inches
		Bolt Type = :	A325	
		Threads in Shear Plane = :	NO	
		Edge Distance = :	1.500	inches
		Minimum Bolt Spacing = :	3.000	inches
		# bolt rows each side of Web Splice = :	2	
		# bolt rows each side of Flange Splice = :	4	
		Web Moment from eccentricity of Shear = :	73.06	ft-kips
		Bolt Capacity for STRENGTH I Limit State = :	69.27	kips
		Contact Surface Finish = :	Class B	
		Friction Coefficient = :	0.50	
		Hole Type = :	STD	
		Bolt Slip-Resistance for SERVICE II Limit State = :	39.00	kips

Top Flange Splice :	W, in.	t, in.	L, in.	Grade	Wt, lbs
Outer Plate :	18.000	5/16	19.000	50	30
Inner Plates (2) :	8.250	3/8	19.000	50	33
Filler Plate :	18.000	1/8	9.500	50	6
# bolts/row :	1.83 (bearing)				
# bolts/row :	2.41 (slip)				
# bolts/row :	3				
total # bolts :	24	7/8	4 7/16	A325	18
Bottom Flange Splice :	**W, in.**	**t, in.**	**L, in.**	**Grade**	**Wt, lbs**
Outer Plate :	18.000	3/8	19.000	50	36
Inner Plates (2) :	8.250	7/16	19.000	50	39
Filler Plate :	18.000	1/8	9.500	50	6
# bolts/row :	2.05 (bearing)				
# bolts/row :	2.71 (slip)				
# bolts/row :	3				
total # bolts :	24	7/8	4 9/16	A325	19
Web Splice :	**W, in.**	**t, in.**	**L, in.**	**Grade**	**Wt, lbs**
Web Plates (2) :	13.000	5/16	67.188	50	155
# bolts/row :	12 (bearing)				
# bolts/row :	14 (slip)				
total # bolts :	56	7/8	4 1/8	A325	39
					382

GIRDER:	I-55 / 61	SPLICE:	2

Top Flange		
b = :	18	inches
t = :	0.75	inches
F_y = :	50	ksi
Bottom Flange		
b = :	18	inches
t = :	0.75	inches
F_y = :	50	ksi
Web		
D = :	69	inches
t = :	0.5	inches
F_y = :	50	ksi
Spacings		
Cross-Frames = :	20	ft
Transverse Stiffeners = :	10	ft
Select Compression Flange :	Bottom	

Moments (Service Level)		
DC (Non-Composite) = :	678	ft-k
DW (Composite) = :	75	ft-k
LL+I = :	1,646	ft-k
Shears (Service Level)		
DC (Non-Composite) = :	109	kips
DW (Composite) = :	33	kips
LL+I = :	156	kips
Slab		
Eff. Width = :	109	inches
Thickness = :	8.5	inches
Eccentricity = :	5.25	inches
Rebar Area = :	0	in^2
Rebar d = :	3	inches
Rebar F_y = :	60	ksi
f'_c = :	3.0	ksi
n = :	9.29	
3n = :	27.87	
Esteel = :	29,000	ksi

Y_{CG} @ : 35.25 inches

Section Properties	Girder	Comp (n)	Comp(3n)	Actual Stresses ksi	Stress Limit ksi
S_{top}, in^3	1,334	16,979	4,776	9.95	50.00
S_{bot}, in^3	1,334	1,933	1,759	26.28	48.07
J = :	8				

Spacing arrangement of Web Splice Bolts:
Starting at c.l. Splice and working "out":

Bearing Case					
0.5	spaces @	4 1/4	ix =	9,558.4	bolt-in^2
5.0	spaces @	6	iy =	54.0	bolt-in^2
0.0	spaces @	3	j =	9,612.4	bolt-in^2
			Force =	63.0	kips
Slip Case			ix =	12,951.4	bolt-in^2
0.5	spaces @	4 1/4	iy =	63.0	bolt-in^2
4.0	spaces @	6	j =	13,014.4	bolt-in^2
2.0	spaces @	3	Force =	36.7	kips

Design Parameters				
R_{HYB} = : 1.000		f_{cu} = :	26.28	ksi
α_{TOP} = : 0.199		F_{cu} = :	38.14	ksi
α_{BOTT} = : 0.547		f_{ncu} = :	9.95	ksi
Bottom Flange Controls		F_{ncu} = :	37.50	ksi
No. of slip planes =:	2	Top Flange Design Stress = :	37.50	ksi
		Bottom Flange Design Stress = :	38.14	ksi
		Section Shear Capacity = :	612.04	kips
SERVICE II Shear =:	344.27	STRENGTH I Shear = :	458.00	kips
STR-I/SER-II =:	1.33	Splice Design Shear = :	535.02	kips
		Web Moment from Applied Loads = :	869.13	ft-kips
		Horizontal Design Web Force = :	408.84	kips
		Bolt Size = :	7/8	inches
		Bolt Type = :	A325	
		Threads in Shear Plane = :	NO	
		Edge Distance = :	1.500	inches
		Minimum Bolt Spacing = :	3.000	inches
		# bolt rows each side of Web Splice = :	2	
		# bolt rows each side of Flange Splice = :	4	
		Web Moment from eccentricity of Shear = :	156.05	ft-kips
		Bolt Capacity for STRENGTH I Limit State = :	69.27	kips
		Contact Surface Finish = :	Class B	
		Friction Coefficient = :	0.50	
		Hole Type = :	STD	
		Bolt Slip-Resistance for SERVICE II Limit State = :	39.00	kips

Top Flange Splice :	W, in.	t, in.	L, in.	Grade	Wt, lbs
Outer Plate :	18.000	5/16	19.000	50	30
Inner Plates (2) :	8.250	3/8	19.000	50	33
Filler Plate :	18.000	1/4	9.500	50	12
# bolts/row :	1.83 (bearing)				
# bolts/row :	2.44 (slip)				
# bolts/row :	3				
total # bolts :	24	7/8	4 7/16	A325	18
Bottom Flange Splice :	**W, in.**	**t, in.**	**L, in.**	**Grade**	**Wt, lbs**
Outer Plate :	18.000	5/16	19.000	50	30
Inner Plates (2) :	8.250	3/8	19.000	50	33
Filler Plate :	18.000	1/4	9.500	50	12
# bolts/row :	1.86 (bearing)				
# bolts/row :	2.49 (slip)				
# bolts/row :	3				
total # bolts :	24	7/8	4 7/16	A325	18
Web Splice :	**W, in.**	**t, in.**	**L, in.**	**Grade**	**Wt, lbs**
Web Plates (2) :	13.000	5/16	67.250	50	155
# bolts/row :	12 (bearing)				
# bolts/row :	14 (slip)				
total # bolts :	56	7/8	4 1/8	A325	39
					382

HUFF

GIRDER: I-55 / 62 **SPLICE:** 2

Top Flange		
b = :	18	inches
t = :	0.75	inches
F_y = :	50	ksi
Bottom Flange		
b = :	18	inches
t = :	0.75	inches
F_y = :	50	ksi
Web		
D = :	69	inches
t = :	0.5	inches
F_y = :	50	ksi
Spacings		
Cross-Frames = :	20	ft
Transverse Stiffeners = :	10	ft
Select Compression Flange :	Bottom	

Moments (Service Level)		
DC (Non-Composite) = :	518	ft-k
DW (Composite) = :	61	ft-k
LL+I = :	1,143	ft-k
Shears (Service Level)		
DC (Non-Composite) = :	91	kips
DW (Composite) = :	28	kips
LL+I = :	121	kips
Slab		
Eff. Width = :	109	inches
Thickness = :	8.5	inches
Eccentricity = :	5.25	inches
Rebar Area = :	0	in^2
Rebar d = :	3	inches
Rebar F_y = :	60	ksi
f'_c = :	3.0	ksi
n = :	9.29	
3n = :	27.87	
Esteel = :	29,000	ksi

Y_{CG} @ : 35.25 inches

Section Properties	Girder	Comp (n)	Comp(3n)	Actual Stresses ksi	Stress Limit ksi
S_{top}, in^3	1,334	16,979	4,776	7.47	50.00
S_{bot}, in^3	1,334	1,933	1,759	18.87	48.07
J = :	8				

Spacing arrangement of Web Splice Bolts:
Starting at c.l. Splice and working "out":

Bearing Case			ix =	9,558.4	bolt-in^2
0.5	spaces @	4 1/4	iy =	54.0	bolt-in^2
5.0	spaces @	6	j =	9,612.4	bolt-in^2
0.0	spaces @	3	Force =	60.9	kips
Slip Case			ix =	11,433.4	bolt-in^2
1.0	spaces @	4 1/4	iy =	58.5	bolt-in^2
4.0	spaces @	6	j =	11,491.9	bolt-in^2
1.0	spaces @	3	Force =	38.6	kips

Design Parameters				
R_{HYB} = : 1.000		f_{cu} = :	18.87	ksi
α_{TOP} = : 0.149		F_{cu} = :	37.50	ksi
α_{BOTT} = : 0.393		f_{ncu} = :	7.47	ksi
Bottom Flange Controls		F_{ncu} = :	37.50	ksi
No. of slip planes =:	2	Top Flange Design Stress = :	37.50	ksi
		Bottom Flange Design Stress = :	37.50	ksi
		Section Shear Capacity = :	612.04	kips
SERVICE II Shear =:	276.69	STRENGTH I Shear = :	368.00	kips
STR-I/SER-II =:	1.33	Splice Design Shear = :	490.02	kips
		Web Moment from Applied Loads = :	865.37	ft-kips
		Horizontal Design Web Force = :	390.76	kips
		Bolt Size = :	7/8	inches
		Bolt Type = :	A325	
		Threads in Shear Plane = :	NO	
		Edge Distance = :	1.500	inches
		Minimum Bolt Spacing = :	3.000	inches
		# bolt rows each side of Web Splice = :	2	
		# bolt rows each side of Flange Splice = :	4	
		Web Moment from eccentricity of Shear = :	142.92	ft-kips
		Bolt Capacity for STRENGTH I Limit State = :	69.27	kips
		Contact Surface Finish = :	Class B	
		Friction Coefficient = :	0.50	
		Hole Type = :	STD	
		Bolt Slip-Resistance for SERVICE II Limit State = :	39.00	kips

Top Flange Splice :	W, in.	t, in.	L, in.	Grade	Wt, lbs
Outer Plate :	18.000	5/16	19.000	50	30
Inner Plates (2) :	8.250	3/8	19.000	50	33
Filler Plate :	18.000	1/8	9.500	50	6
# bolts/row :	1.83 (bearing)				
# bolts/row :	2.45 (slip)				
# bolts/row :	3				
total # bolts :	24	7/8	4 7/16	A325	18
Bottom Flange Splice :	**W, in.**	**t, in.**	**L, in.**	**Grade**	**Wt, lbs**
Outer Plate :	18.000	5/16	19.000	50	30
Inner Plates (2) :	8.250	3/8	19.000	50	33
Filler Plate :	18.000	1/8	9.500	50	6
# bolts/row :	1.83 (bearing)				
# bolts/row :	2.45 (slip)				
# bolts/row :	3				
total # bolts :	24	7/8	4 7/16	A325	18
Web Splice :	**W, in.**	**t, in.**	**L, in.**	**Grade**	**Wt, lbs**
Web Plates (2) :	13.000	5/16	67.250	50	155
# bolts/row :	12 (bearing)				
# bolts/row :	13 (slip)				
total # bolts :	52	7/8	4 1/8	A325	37
					367

HUFF

GIRDER: I-55 / G3 **SPLICE:** 2

Top Flange		
b = :	18	inches
t = :	0.75	inches
F_y = :	50	ksi
Bottom Flange		
b = :	18	inches
t = :	0.75	inches
F_y = :	50	ksi
Web		
D = :	69	inches
t = :	0.5	inches
F_y = :	50	ksi
Spacings		
Cross-Frames = :	20	ft
Transverse Stiffeners = :	10	ft
Select Compression Flange :	Bottom	

Moments (Service Level)		
DC (Non-Composite) = :	525	ft-k
DW (Composite) = :	69	ft-k
LL+I = :	1,219	ft-k
Shears (Service Level)		
DC (Non-Composite) = :	91	kips
DW (Composite) = :	28	kips
LL+I = :	121	kips
Slab		
Eff. Width = :	109	inches
Thickness = :	8.5	inches
Eccentricity = :	5.25	inches
Rebar Area = :	0	in^2
Rebar d = :	3	inches
Rebar F_y = :	60	ksi
f'_c = :	3.0	ksi
n = :	9.29	
3n = :	27.87	
Esteel = :	29,000	ksi

Y_{CG} @ : 35.25 inches

Section Properties	Girder	Comp (n)	Comp(3n)	Actual Stresses ksi	Stress Limit ksi
S_{top}, in^3	1,334	16,979	4,776	7.67	50.00
S_{bot}, in^3	1,334	1,933	1,759	19.86	48.07
J = :	8				

Spacing arrangement of Web Splice Bolts:
Starting at c.l. Splice and working "out":

Bearing Case					
0.5	spaces @	4 1/4	ix =	9,558.4	bolt-in^2
5.0	spaces @	6	iy =	54.0	bolt-in^2
0.0	spaces @	3	j =	9,612.4	bolt-in^2
			Force =	61.0	kips
Slip Case			ix =	11,433.4	bolt-in^2
1.0	spaces @	4 1/4	iy =	58.5	bolt-in^2
4.0	spaces @	6	j =	11,491.9	bolt-in^2
1.0	spaces @	3	Force =	38.6	kips

Design Parameters				
R_{HYB} = : 1.000		f_{cu} = :	19.86	ksi
a_{TOP} = : 0.153		F_{cu} = :	37.50	ksi
a_{BOTT} = : 0.413		f_{ncu} = :	7.67	ksi
Bottom Flange Controls		F_{ncu} = :	37.50	ksi
No. of slip planes =: 2		Top Flange Design Stress = :	37.50	ksi
		Bottom Flange Design Stress = :	37.50	ksi
		Section Shear Capacity = :	612.04	kips
SERVICE II Shear =:	276.69	STRENGTH I Shear = :	368.00	kips
STR-I/SER-II =:	1.33	Splice Design Shear = :	490.02	kips
		Web Moment from Applied Loads = :	859.38	ft-kips
		Horizontal Design Web Force = :	397.01	kips
		Bolt Size = :	7/8	inches
		Bolt Type = :	A325	
		Threads in Shear Plane = :	NO	
		Edge Distance = :	1.500	inches
		Minimum Bolt Spacing = :	3.000	inches
		# bolt rows each side of Web Splice = :	2	
		# bolt rows each side of Flange Splice = :	4	
		Web Moment from eccentricity of Shear = :	142.92	ft-kips
		Bolt Capacity for STRENGTH I Limit State = :	69.27	kips
		Contact Surface Finish = :	Class B	
		Friction Coefficient = :	0.50	
		Hole Type = :	STD	
		Bolt Slip-Resistance for SERVICE II Limit State = :	39.00	kips

Top Flange Splice :	W, in.	t, in.	L, in.	Grade	Wt, lbs
Outer Plate :	18.000	5/16	19.000	50	30
Inner Plates (2) :	8.250	3/8	19.000	50	33
Filler Plate :	18.000	1/8	9.500	50	6
# bolts/row :	1.83 (bearing)				
# bolts/row :	2.44 (slip)				
# bolts/row :	3				
total # bolts :	24	7/8	4 7/16	A325	18
Bottom Flange Splice :	W, in.	t, in.	L, in.	Grade	Wt, lbs
Outer Plate :	18.000	5/16	19.000	50	30
Inner Plates (2) :	8.250	3/8	19.000	50	33
Filler Plate :	18.000	1/8	9.500	50	6
# bolts/row :	1.83 (bearing)				
# bolts/row :	2.44 (slip)				
# bolts/row :	3				
total # bolts :	24	7/8	4 7/16	A325	18
Web Splice :	W, in.	t, in.	L, in.	Grade	Wt, lbs
Web Plates (2) :	13.000	5/16	67.250	50	155
# bolts/row :	12 (bearing)				
# bolts/row :	13 (slip)				
total # bolts :	52	7/8	4 1/8	A325	37
					367

HUFF

GIRDER: I-55 / 64 SPLICE: 2

Top Flange		
b = :	18	inches
t = :	0.75	inches
F_y = :	50	ksi
Bottom Flange		
b = :	18	inches
t = :	0.75	inches
F_y = :	50	ksi
Web		
D = :	69	inches
t = :	0.5	inches
F_y = :	50	ksi
Spacings		
Cross-Frames = :	20	ft
Transverse Stiffeners = :	10	ft
Select Compression Flange :	Bottom	

Moments (Service Level)		
DC (Non-Composite) = :	526	ft-k
DW (Composite) = :	71	ft-k
LL+I = :	1,219	ft-k
Shears (Service Level)		
DC (Non-Composite) = :	91	kips
DW (Composite) = :	28	kips
LL+I = :	122	kips
Slab		
Eff. Width = :	109	inches
Thickness = :	8.5	inches
Eccentricity = :	5.25	inches
Rebar Area = :	0	in^2
Rebar d = :	3	inches
Rebar F_y = :	60	ksi
f'_c = :	3.0	ksi
n = :	9.29	
3n = :	27.87	
Esteel = :	29,000	ksi

Y_{CG} @ : 35.25 inches

Section Properties	Girder	Comp (n)	Comp(3n)	Actual Stresses ksi	Stress Limit ksi
S_{top}, in^3	1,334	16,979	4,776	7.68	50.00
S_{bot}, in^3	1,334	1,933	1,759	19.87	48.07
J = :	8				

Spacing arrangement of Web Splice Bolts:
Starting at c.l. Splice and working "out":

Bearing Case					
0.5	spaces @	4 1/4	ix =	9,558.4	bolt-in^2
5.0	spaces @	6	iy =	54.0	bolt-in^2
0.0	spaces @	3	j =	9,612.4	bolt-in^2
			Force =	61.0	kips
Slip Case			ix =	11,433.4	bolt-in^2
1.0	spaces @	4 1/4	iy =	58.5	bolt-in^2
4.0	spaces @	6	j =	11,491.9	bolt-in^2
1.0	spaces @	3	Force =	38.7	kips

Design Parameters				
R_{HYB} = : 1.000		f_{cu} = :	19.87	ksi
α_{TOP} = : 0.154		F_{cu} = :	37.50	ksi
α_{BOTT} = : 0.413		f_{ncu} = :	7.68	ksi
Bottom Flange Controls		F_{ncu} = :	37.50	ksi
No. of slip planes =: 2		Top Flange Design Stress = :	37.50	ksi
		Bottom Flange Design Stress = :	37.50	ksi
		Section Shear Capacity = :	612.04	kips
SERVICE II Shear =:	278.17	STRENGTH I Shear = :	370.00	kips
STR-I/SER-II =:	1.33	Splice Design Shear = :	491.02	kips
		Web Moment from Applied Loads = :	859.59	ft-kips
		Horizontal Design Web Force = :	396.78	kips
		Bolt Size = :	7/8	inches
		Bolt Type = :	A325	
		Threads in Shear Plane = :	NO	
		Edge Distance = :	1.500	inches
		Minimum Bolt Spacing = :	3.000	inches
		# bolt rows each side of Web Splice = :	2	
		# bolt rows each side of Flange Splice = :	4	
		Web Moment from eccentricity of Shear = :	143.21	ft-kips
		Bolt Capacity for STRENGTH I Limit State = :	69.27	kips
		Contact Surface Finish = :	Class B	
		Friction Coefficient = :	0.50	
		Hole Type = :	STD	
		Bolt Slip-Resistance for SERVICE II Limit State = :	39.00	kips

Top Flange Splice :	W, in.	t, in.	L, in.	Grade	Wt, lbs
Outer Plate :	18.000	5/16	19.000	50	30
Inner Plates (2) :	8.250	3/8	19.000	50	33
Filler Plate :	18.000	1/8	9.500	50	6
# bolts/row :	1.83 (bearing)				
# bolts/row :	2.44 (slip)				
# bolts/row :	3				
total # bolts :	24	7/8	4 7/16	A325	18
Bottom Flange Splice :	**W, in.**	**t, in.**	**L, in.**	**Grade**	**Wt, lbs**
Outer Plate :	18.000	5/16	19.000	50	30
Inner Plates (2) :	8.250	3/8	19.000	50	33
Filler Plate :	18.000	1/8	9.500	50	6
# bolts/row :	1.83 (bearing)				
# bolts/row :	2.44 (slip)				
# bolts/row :	3				
total # bolts :	24	7/8	4 7/16	A325	18
Web Splice :	**W, in.**	**t, in.**	**L, in.**	**Grade**	**Wt, lbs**
Web Plates (2) :	13.000	5/16	67.250	50	155
# bolts/row :	12 (bearing)				
# bolts/row :	13 (slip)				
total # bolts :	52	7/8	4 1/8	A325	37
					367

HUFF

GIRDER:	I-55 / 65	SPLICE:	2

Top Flange

b = :	18	inches
t = :	0.75	inches
F_y = :	50	ksi

Bottom Flange

b = :	18	inches
t = :	0.75	inches
F_y = :	50	ksi

Web

D = :	69	inches
t = :	0.5	inches
F_y = :	50	ksi

Spacings

Cross-Frames = :	20	ft
Transverse Stiffeners = :	10	ft
Select Compression Flange :	Bottom	

Moments (Service Level)

DC (Non-Composite) = :	524	ft-k
DW (Composite) = :	67	ft-k
LL+I = :	1132	ft-k

Shears (Service Level)

DC (Non-Composite) = :	92	kips
DW (Composite) = :	28	kips
LL+I = :	120	kips

Slab

Eff. Width = :	109	inches
Thickness = :	8.5	inches
Eccentricity = :	5.25	inches
Rebar Area = :	0	in^2
Rebar d = :	3	inches
Rebar F_y = :	60	ksi
f'_c = :	3.0	ksi
n = :	9.29	
3n = :	27.87	
Esteel = :	29,000	ksi

Y_{CG} @ : 35.25 inches

Section Properties	Girder	Comp (n)	Comp(3n)	Actual Stresses ksi	Stress Limit ksi
S_{top}, in^3	1,334	16,979	4,776	7.55	50.00
S_{bot}, in^3	1,334	1,933	1,759	18.88	48.07
J = :	8				

Spacing arrangement of Web Splice Bolts:
Starting at c.l. Splice and working "out":

Bearing Case					
0.5	spaces @	4 1/4	ix =	9,558.4	bolt-in^2
5.0	spaces @	6	iy =	54.0	bolt-in^2
0.0	spaces @	3	j =	9,612.4	bolt-in^2
			Force =	60.9	kips
Slip Case					
1.0	spaces @	4 1/4	ix =	11,433.4	bolt-in^2
4.0	spaces @	6	iy =	58.5	bolt-in^2
1.0	spaces @	3	j =	11,491.9	bolt-in^2
			Force =	38.6	kips

Design Parameters

R_{HYB} = : 1.000		f_{cu} = :	18.88 ksi
a_{TOP} = : 0.151		F_{cu} = :	37.50 ksi
a_{BOTT} = : 0.393		f_{ncu} = :	7.55 ksi
Bottom Flange Controls		F_{ncu} = :	37.50 ksi
No. of slip planes =:	2	Top Flange Design Stress = :	37.50 ksi
		Bottom Flange Design Stress = :	37.50 ksi
		Section Shear Capacity = :	612.04 kips
SERVICE II Shear =:	276.00	STRENGTH I Shear = :	367.00 kips
STR-I/SER-II =:	1.33	Splice Design Shear = :	489.52 kips
		Web Moment from Applied Loads = :	867.70 ft-kips
		Horizontal Design Web Force = :	388.32 kips
		Bolt Size = :	7/8 inches
		Bolt Type = :	A325
		Threads in Shear Plane = :	NO
		Edge Distance = :	1.500 inches
		Minimum Bolt Spacing = :	3.000 inches
		# bolt rows each side of Web Splice = :	2
		# bolt rows each side of Flange Splice = :	4
		Web Moment from eccentricity of Shear = :	142.78 ft-kips
		Bolt Capacity for STRENGTH I Limit State = :	69.27 kips
		Contact Surface Finish = :	Class B
		Friction Coefficient = :	0.50
		Hole Type = :	STD
		Bolt Slip-Resistance for SERVICE II Limit State = :	39.00 kips

Top Flange Splice :	W, in.	t, in.	L, in.	Grade	Wt, lbs
Outer Plate :	18.000	5/16	19.000	50	30
Inner Plates (2) :	8.250	3/8	19.000	50	33
Filler Plate :	18.000	1/8	9.500	50	6
# bolts/row :	1.83 (bearing)				
# bolts/row :	2.45 (slip)				
# bolts/row :	3				
total # bolts :	24	7/8	4 7/16	A325	18
Bottom Flange Splice :	**W, in.**	**t, in.**	**L, in.**	**Grade**	**Wt, lbs**
Outer Plate :	18.000	5/16	19.000	50	30
Inner Plates (2) :	8.250	3/8	19.000	50	33
Filler Plate :	18.000	1/8	9.500	50	6
# bolts/row :	1.83 (bearing)				
# bolts/row :	2.45 (slip)				
# bolts/row :	3				
total # bolts :	24	7/8	4 7/16	A325	18
Web Splice :	**W, in.**	**t, in.**	**L, in.**	**Grade**	**Wt, lbs**
Web Plates (2) :	13.000	5/16	67.250	50	155
# bolts/row :	12 (bearing)				
# bolts/row :	13 (slip)				
total # bolts :	52	7/8	4 1/8	A325	37
					367

GIRDER: I-55 / 66 **SPLICE:** 2

Top Flange		
b = :	18	inches
t = :	0.75	inches
F_y = :	50	ksi
Bottom Flange		
b = :	18	inches
t = :	0.75	inches
F_y = :	50	ksi
Web		
D = :	69	inches
t = :	0.5	inches
F_y = :	50	ksi
Spacings		
Cross-Frames = :	20	ft
Transverse Stiffeners = :	10	ft
Select Compression Flange :	Bottom	

Moments (Service Level)		
DC (Non-Composite) = :	526	ft-k
DW (Composite) = :	61	ft-k
LL+I = :	1,264	ft-k
Shears (Service Level)		
DC (Non-Composite) = :	96	kips
DW (Composite) = :	29	kips
LL+I = :	129	kips
Slab		
Eff. Width = :	109	inches
Thickness = :	8.5	inches
Eccentricity = :	5.25	inches
Rebar Area = :	0	in^2
Rebar d = :	3	inches
Rebar F_y = :	60	ksi
f'_c = :	3.0	ksi
n = :	9.29	
3n = :	27.87	
Esteel = :	29,000	ksi

Y_{CG} @ : 35.25 inches

Section Properties	Girder	Comp (n)	Comp(3n)	Actual Stresses ksi	Stress Limit ksi
S_{top}, in^3	1,334	16,979	4,776	7.70	50.00
S_{bot}, in^3	1,334	1,933	1,759	20.26	48.07
J = :	8				

Spacing arrangement of Web Splice Bolts:
Starting at c.l. Splice and working "out":

Bearing Case					
0.5	spaces @	4 1/4	ix =	9,558.4	bolt-in^2
5.0	spaces @	6	iy =	54.0	bolt-in^2
0.0	spaces @	3	j =	9,612.4	bolt-in^2
			Force =	61.3	kips
Slip Case			ix =	12,951.4	bolt-in^2
0.5	spaces @	4 1/4	iy =	63.0	bolt-in^2
4.0	spaces @	6	j =	13,014.4	bolt-in^2
2.0	spaces @	3	Force =	35.1	kips

Design Parameters				
R_{HYB} = : 1.000		f_{cu} = :	20.26	ksi
α_{TOP} = : 0.154		F_{cu} = :	37.50	ksi
α_{BOTT} = : 0.422		f_{ncu} = :	7.70	ksi
Bottom Flange Controls		F_{ncu} = :	37.50	ksi
No. of slip planes =: 2		Top Flange Design Stress = :	37.50	ksi
		Bottom Flange Design Stress = :	37.50	ksi
		Section Shear Capacity = :	612.04	kips
SERVICE II Shear =:	293.22	STRENGTH I Shear = :	390.00	kips
STR-I/SER-II =:	1.33	Splice Design Shear = :	501.02	kips
		Web Moment from Applied Loads = :	855.56	ft-kips
		Horizontal Design Web Force = :	400.99	kips
		Bolt Size = :	7/8	inches
		Bolt Type = :	A325	
		Threads in Shear Plane = :	NO	
		Edge Distance = :	1.500	inches
		Minimum Bolt Spacing = :	3.000	inches
		# bolt rows each side of Web Splice = :	2	
		# bolt rows each side of Flange Splice = :	4	
		Web Moment from eccentricity of Shear = :	146.13	ft-kips
		Bolt Capacity for STRENGTH I Limit State = :	69.27	kips
		Contact Surface Finish = :	Class B	
		Friction Coefficient = :	0.50	
		Hole Type = :	STD	
		Bolt Slip-Resistance for SERVICE II Limit State = :	39.00	kips

Top Flange Splice :	W, in.	t, in.	L, in.	Grade	Wt, lbs
Outer Plate :	18.000	5/16	19.000	50	30
Inner Plates (2) :	8.250	3/8	19.000	50	33
Filler Plate :	18.000	1/8	9.500	50	6
# bolts/row :	1.83 (bearing)				
# bolts/row :	2.44 (slip)				
# bolts/row :	3				
total # bolts :	24	7/8	4 7/16	A325	18
Bottom Flange Splice :	**W, in.**	**t, in.**	**L, in.**	**Grade**	**Wt, lbs**
Outer Plate :	18.000	5/16	19.000	50	30
Inner Plates (2) :	8.250	3/8	19.000	50	33
Filler Plate :	18.000	1/8	9.500	50	6
# bolts/row :	1.83 (bearing)				
# bolts/row :	2.44 (slip)				
# bolts/row :	3				
total # bolts :	24	7/8	4 7/16	A325	18
Web Splice :	**W, in.**	**t, in.**	**L, in.**	**Grade**	**Wt, lbs**
Web Plates (2) :	13.000	5/16	67.250	50	155
# bolts/row :	12 (bearing)				
# bolts/row :	13 (slip)				
total # bolts :	52	7/8	4 1/8	A325	37
					367

HUFF

COUNTY:	CROSSING:	NOTES

GIRDER 1: 32, 7/8" A325 BOLTS PER FLANGE (16 PER SIDE)

7/16" × 18" × 2'1" OUTER PL
1/2" × 8 1/4" × 2'1" INNER PL'S
1/4" FILLER PL'S

68, 7/8" A325 BOLTS IN WEB (4 ROWS OF 17) — 1 SP. @ 4", 2 SP. @ 6", 5 SP. @ 3"

7/16" × 13" × 5'7" WEB PL'S

GIRDERS 2-6: 24, 7/8" A325 BOLTS PER FLANGE

3/8" × 18" × 1'7" OUTER PL
7/16" × 8 1/4" × 1'7" INNER PL'S
1/8" FILLER PL'S

60, 7/8" A325 BOLTS IN WEB (4 ROWS OF 15) — 1 @ 4 3/16", 3 @ 6", 3 @ 3"

DESIGN	DATE
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COUNTY: CROSSING: NOTES

LONGITUDINAL STRESS IN DECK:

6.10.3.2.4
6.10.1.7

LOADINGS TO BE CONSIDERED:
(1) FACTORED CONSTRUCTION LOADS
(2) SERVICE II LIMIT STATE

$\phi = 0.90$
$f_r = 0.24\sqrt{3} = 0.42 \text{ KSI}$

5.5.4.2.1
5.4.2.6

SERVICE II: DC + DW + 1.3LL + TU

GIRDER 1:

X FT	M_{DL} FT·K	M_{SDL} FT·K	M_{LL} FT·K	M_u FT·K	FLANGES
47.25	107	74	-2,258	-2,755	18"×3/4" ①
55.75	-448	-118	-3,384	-4,965	21"×1" ⑦

$S_c = 150,626/22.68 = 6,641 \text{ IN}^3$

$f = 4,965 \times 12/6,641/9$
$= 1.00 \text{ KSI} > .42$

$S_c = 121,405/20.43 = 5,942 \text{ IN}^3$

$f = 2,755 \times 12/5,942/9 = 0.62 \text{ KSI}$
$> .42$

$M_u(12)/5,942/9 \le 0.42 \Rightarrow M_u \le 1,872 \text{ FT·K}$

FROM END: AT $x = 24$ FT, $M_u = -750$, OK
AT $x = 36$ FT, $M_u = -1609$, OK

FROM BENT 1: AT $x = 54$ FT, $M_u = -854$, OK
$x = 43$ FT, $M_u = -2907$, N.G.

USE $.01 \times 8.5 \times 12 = 1.02 \text{ IN}^2/\text{FT}$
$= 0.51 \text{ IN}^2/\text{FACE}$

$\boxed{\text{No. 5 @ 7" TOP \& BOTTOM FROM 84 FT LT OF BENT 1 TO 54 FT RT OF BENT 1}}$

DESIGN | DATE
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CHECK | DATE

COUNTY: CROSSING:

NOTES

CHECK DISTRIBUTION REINFORCEMENT — 9.7.3.2

PRIMARY REINFORCEMENT = No. 6 @ 7" — PAGE 24
= $0.75\ IN^2$

MIN BOTTOM LONGITUDINAL
STEEL = $.67 \times .75 = 0.50\ IN^2/FT$

$\boxed{\text{No. 5 @ 7" OK}}$

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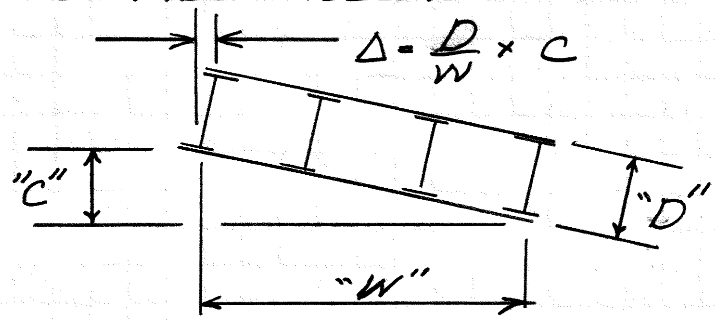
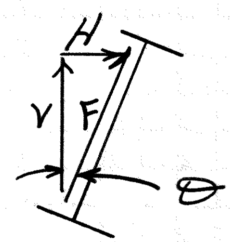
DT-1420 (Structures Design Form 3)

OUT-OF-PLUMB GIRDER WEBS RESULT IN ADDITIONAL LATERAL FLANGE BENDING EFFECTS AS DESCRIBED BELOW:



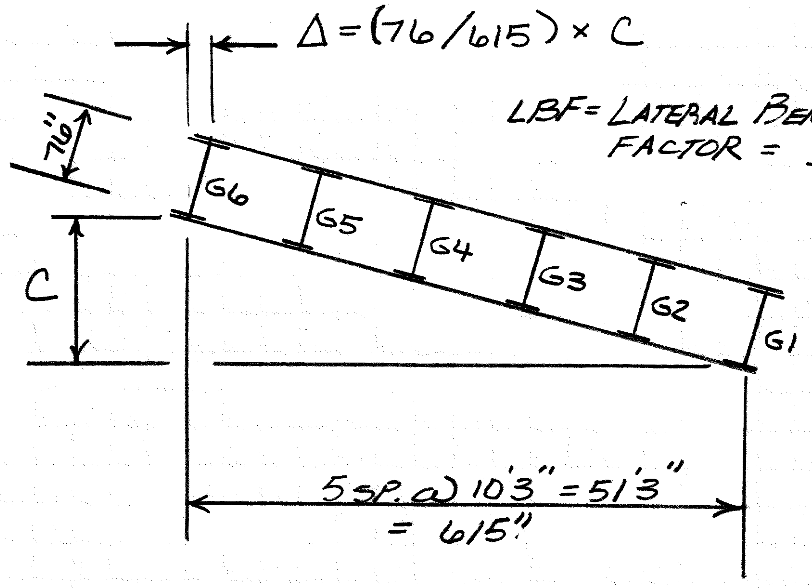


$\sin\theta \approx \theta$ (small θ)

LATERAL BENDING FACTOR

$$= LBF = \frac{H}{F} = \frac{\Delta}{D} = \frac{C}{W}$$

COUNTY: CROSSING: NOTES



LBF = LATERAL BENDING FACTOR = $\frac{C}{615}$

DL GIRDER + DECK:

SPAN PT.	G1	G6	C	LBF^{-1}
	DEFL. (IN.)			
1.0	0.000	0.000	0.000	∞
1.1	-0.011	-0.040	0.029	21,207
1.2	0.034	-0.027	0.061	10,082
1.3	0.156	0.067	0.089	6,910
1.4	0.342	0.233	0.109	5,642
1.5	0.541	0.431	0.110	5,591
1.6	0.692	0.596	0.096	6,406
1.7	0.750	0.673	0.077	7,987
1.8	0.673	0.617	0.056	10,982
1.9	0.430	0.399	0.031	19,839
2.0	0.000	0.000	0.000	∞
2.1	-1.255	-1.191	0.064	9,609
2.2	-3.084	-2.952	0.132	4,659
2.3	-5.144	-4.915	0.229	2,686
2.4	-6.635	-6.344	0.291	2,113
2.5	-7.150	-6.853	0.297	2,071

DESIGN	DATE
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COUNTY: CROSSING: | NOTES

VERTICAL BENDING MOMENT IN G1:

$M = 4,310$ FT·KIPS
GIRDER + DECK
UNFACTORED

SPAN 2 MIDSPAN — WORST CASE

$M_\ell = 4,310 / 2,071 = 2.08$ FT·K FROM OUT-OF-PLUMB WEB

$$M_\ell = \frac{4,310\,(25)^2}{10\,(2,347)(5.75)} = 19.96 \text{ FT·K}$$ FROM HORIZONTAL CURVATURE

THE OUT-OF-PLUMB WEB EFFECT ON LATERAL FLANGE BENDING IS ABOUT 10.4% OF THE HORIZONTAL CURVATURE EFFECT ON LATERAL FLANGE BENDING.

THE OUT-OF-PLUMP EFFECT WAS DISREGARDED IN THE DESIGN OF THIS STRUCTURE.

IN GENERAL, BOTH EFFECTS SHOULD BE INCLUDED IN CURVED GIRDER DESIGNS.

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DT-1420 (Structures Design Form 3)

V-LOAD ANALYSIS:

$$V = \frac{\Sigma M}{CK}$$

ΣM = SUMMATION OF VERTICAL BENDING MOMENTS ACROSS DIAPHRAGM LINE.

$C = 7/5$ FOR 6-GIRDER SECTION

$$K = \frac{R_i D}{d_i}$$

R_i = GIRDER RADIUS
D = GIRDER SPACING
d_i = DIAPHRAGM SPACING ALONG GIRDER

V = V-LOAD SHEAR ON EXTERIOR GIRDERS

FOR GIRDERS OUTSIDE ℄:

V-LOAD ACTS DOWNWARD IN M^+ REGIONS

V-LOAD ACTS UPWARD IN M^- REGIONS

FOR GIRDERS INSIDE ℄:

V-LOAD ACTS UPWARD IN POSITIVE MOMENT REGIONS

V-LOAD ACTS DOWNWARD IN M^- REGIONS.

FOR INTERIOR GIRDERS, V-LOAD IS PROPORTIONAL TO DISTANCE FROM ℄.

DESIGN	DATE
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COUNTY: CROSSING: NOTES

GIRDER LENGTHS, NORTHBOUND BRIDGE

GIRDER 1, $R_1 = 2{,}347.4562$ FT
$d_1 = 24.28$ FT
$D = 10.25$ FT

$L_{11} = 5 \times 24.28 = 121.4$ FT
$L_{21} = 10 \times 21.51 = 215.1$ FT
$L_{31} = \quad 121.4$ FT

$$\frac{RD}{d} = \frac{2{,}347.4562 \times 10.25}{24.48} \qquad = K$$

$= 982.9$, SPAN 1

$$\frac{2{,}347.4562 \times 10.25}{21.51}$$

$= 1{,}118.6$, SPAN 2

GIRDER	L_1	L_2	L_3
2	$5 \times 24.17 = 120.85$	$10 \times 21.42 = 214.2$	120.85
3	$5 \times 24.06 = 120.30$	$10 \times 21.32 = 213.2$	120.30
4	$5 \times 23.96 = 119.8$	$10 \times 21.23 = 212.3$	119.8
5	$5 \times 23.85 = 119.25$	$10 \times 21.13 = 211.3$	119.25
6	$5 \times 23.75 = 118.75$	$10 \times 21.04 = 210.4$	118.75

SLAB LOADS: 1.2532 KLF, GIRDERS 2-5
1.1735 KLF, GIRDERS 1, 6

(TO MATCH DESCUS)

DETAIL FACTOR = 1.15
(APPLY TO STEEL WT.)

DESIGN DATE

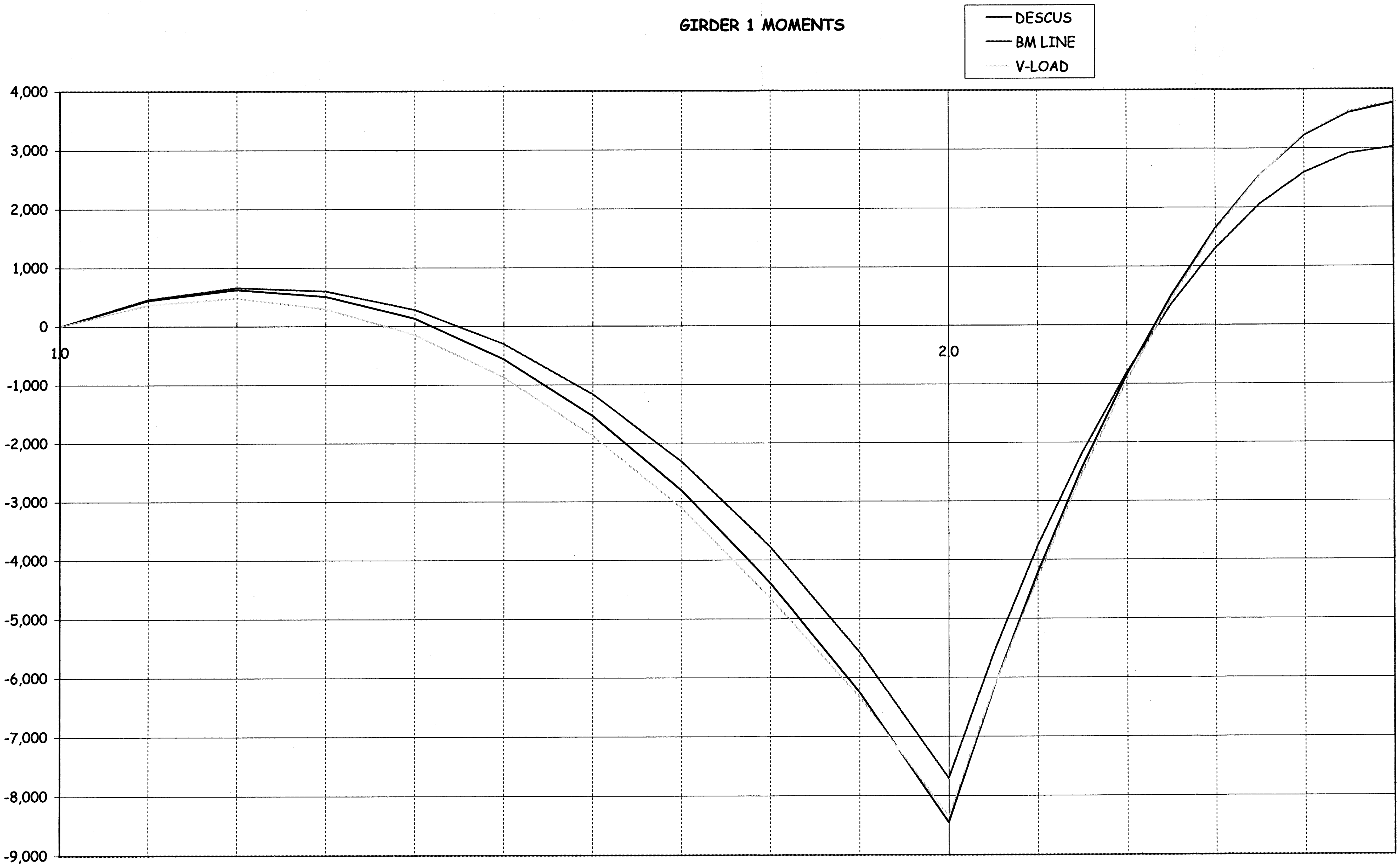
CHECK DATE

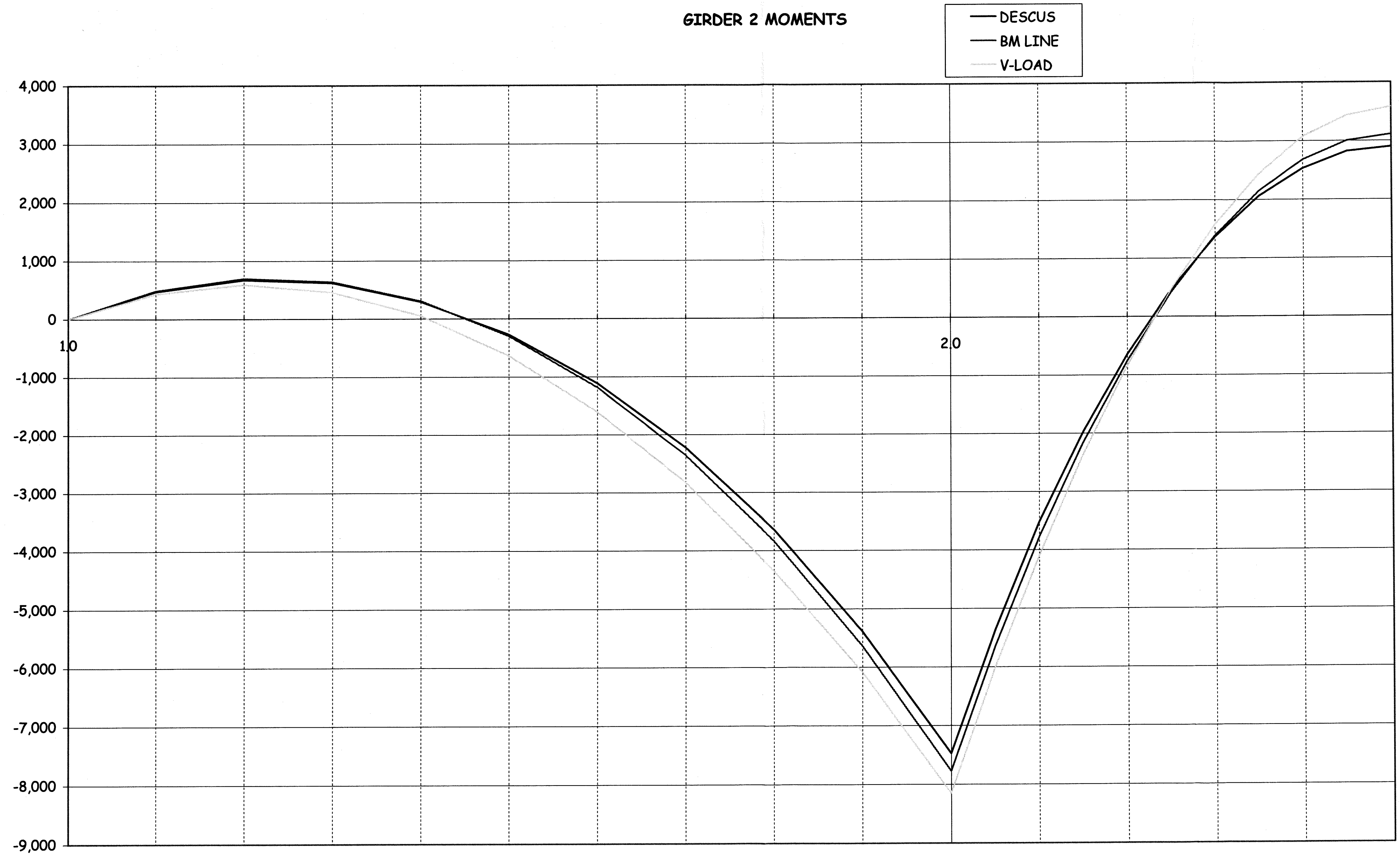
CHECK DATE

DT-1420 (Structures Design Form 3)

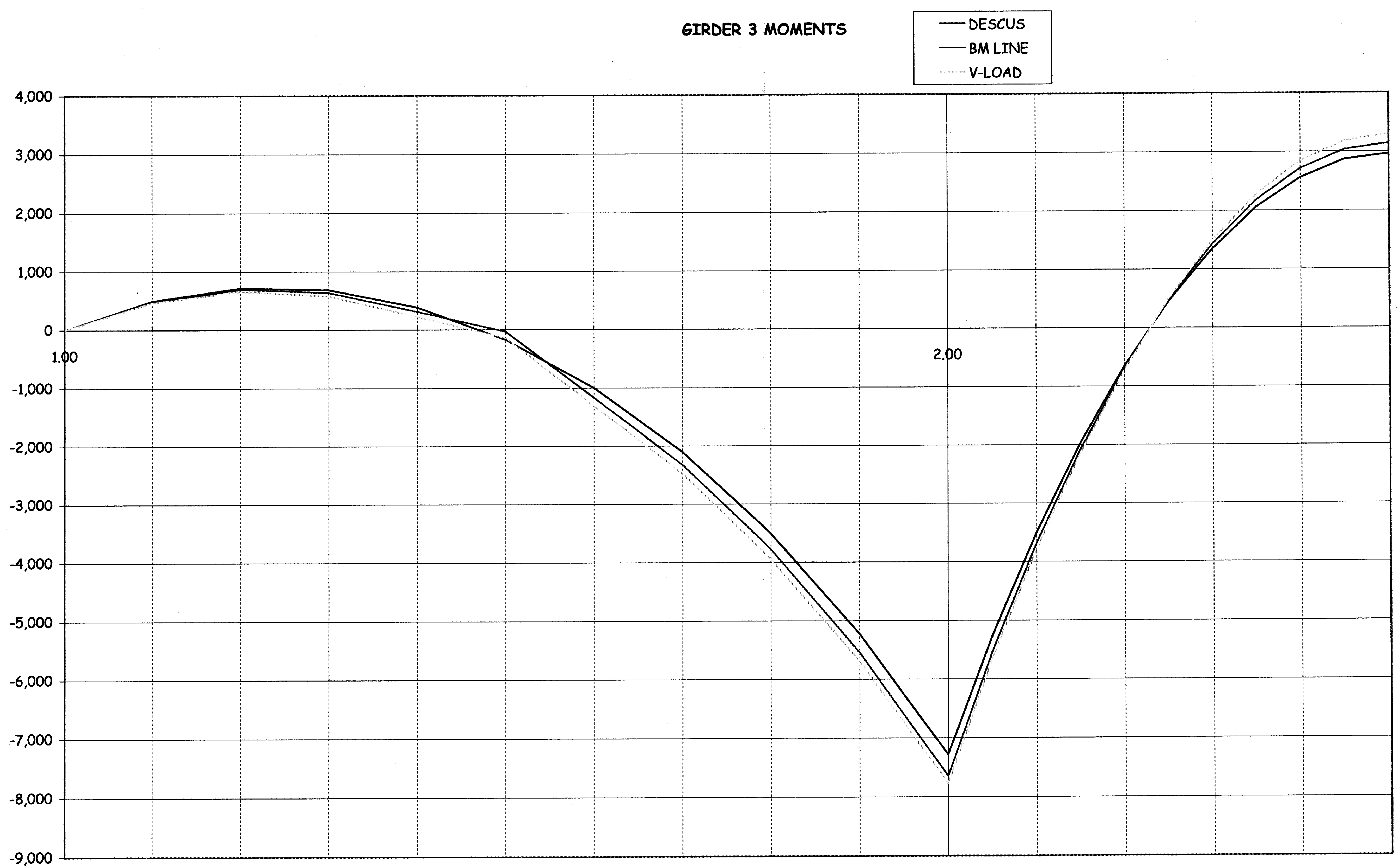
I-55 / MALLORY AVENUE
V-LOAD ANALYSIS
DEAD LOAD GIRDER AND SLAB

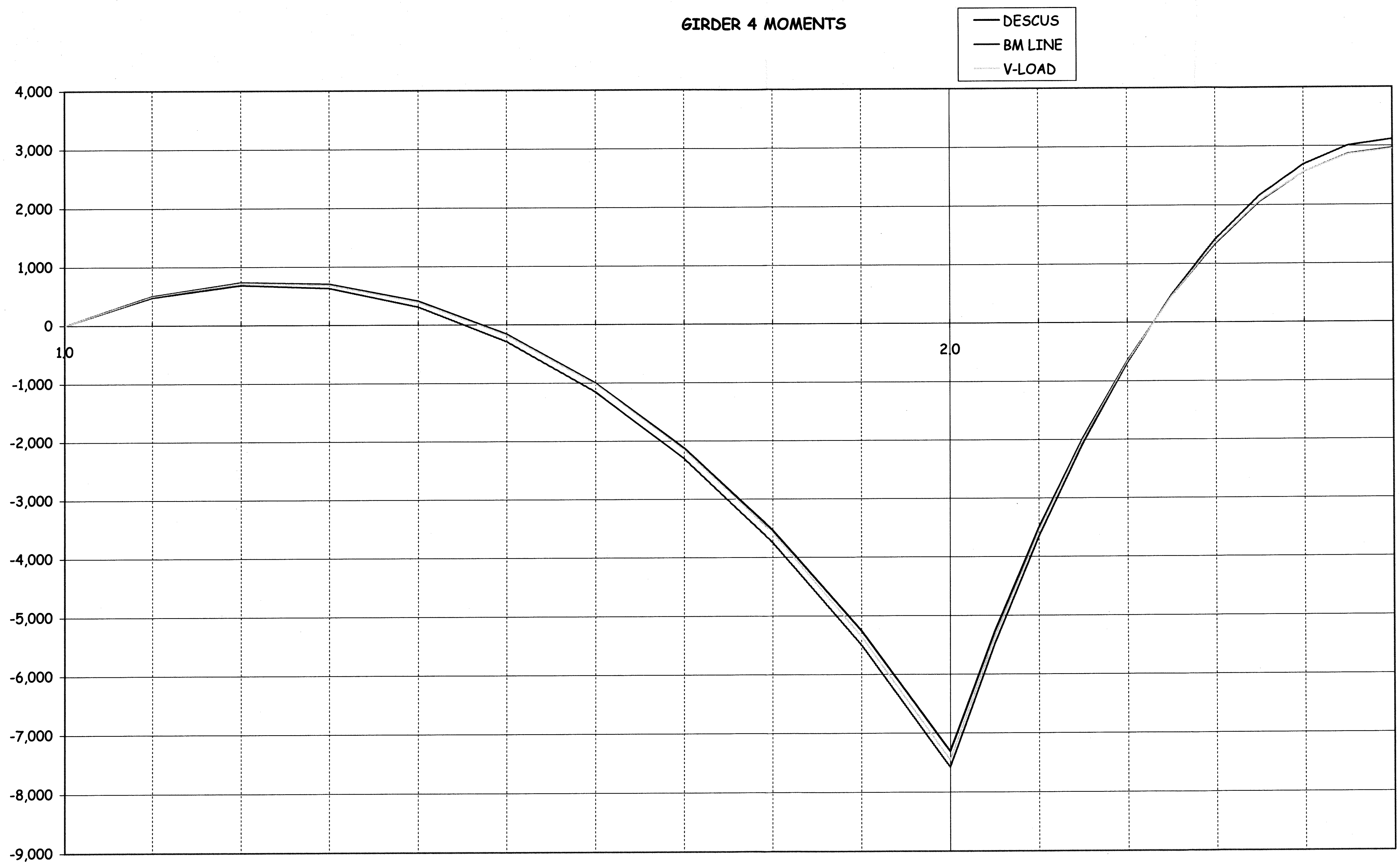
		GIRDER 1 MOMENTS, FT-KIPS				GIRDER 2 MOMENTS, FT-KIPS				GIRDER 3 MOMENTS, FT-KIPS				GIRDER 4 MOMENTS, FT-KIPS				GIRDER 5				GIRDER 6			
			V-LOAD				V-LOAD				V-LOAD				V-LOAD				V-LOAD				V-LOAD		
	PT	DESCUS	BM LINE	ADDED	V-LOAD	DESCUS	BM LINE	ADDED	V-LOAD	DESCUS	BM LINE	ADDED	V-LOAD	DESCUS	BM LINE	ADDED	V-LOAD	DESCUS	BM LINE	ADDED	V-LOAD	DESCUS	BM LINE	ADDED	V-LOAD
	1.00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	1.10	444	461	-90	371	473	486	-53	433	495	483	-17	466	501	478	17	495	486	474	52	526	454	447	86	533
DIAPH.	1.20	628	662	-179	483	673	699	-106	593	719	695	-35	660	734	689	35	724	707	683	103	786	658	645	171	816
	1.30	511	603	-304	299	621	640	-180	460	688	638	-59	579	706	633	59	692	673	627	176	803	601	594	292	886
DIAPH.	1.40	134	285	-429	-144	296	309	-254	55	387	312	-84	228	409	309	83	392	373	306	248	554	295	295	412	707
	1.50	-560	-299	-570	-869	-267	-297	-338	-635	-168	-28	-111	-139	-158	-286	111	-175	-178	-283	330	47	-290	-257	548	291
DIAPH.	1.60	-1,533	-1,161	-711	-1,872	-1,111	-1,183	-422	-1,605	-999	-1,165	-139	-1,304	-1,000	-1,154	138	-1,016	-1,001	-1,144	412	-732	-1,132	-1,065	684	-381
	1.70	-2,809	-2,316	-792	-3,108	-2,221	-2,356	-469	-2,825	-2,104	-2,323	-154	-2,477	-2,120	-2,303	154	-2,149	-2,095	-2,282	459	-1,823	-2,271	-2,139	762	-1,377
DIAPH.	1.80	-4,395	-3,779	-873	-4,652	-3,638	-3,835	-517	-4,352	-3,505	-3,776	-170	-3,946	-3,533	-3,743	169	-3,574	-3,479	-3,710	505	-3,205	-3,692	-3,494	839	-2,655
	1.90	-6,249	-5,568	-758	-6,326	-5,384	-5,633	-448	-6,081	-5,225	-5,543	-147	-5,690	-5,253	-5,495	146	-5,349	-5,166	-5,446	436	-5,010	-5,467	-5,147	725	-4,422
DIAPH.	2.00	-8,454	-7,707	-644	-8,351	-7,474	-7,774	-380	-8,154	-7,282	-7,645	-124	-7,769	-7,307	-7,578	123	-7,455	-7,183	-7,511	368	-7,143	-7,562	-7,119	611	-6,508
	2.05	-6,199	-5,598	-595	-6,193	-5,364	-5,643	-351	-5,994	-5,264	-5,536	-114	-5,650	-5,275	-5,487	114	-5,373	-5,199	-5,439	339	-5,100	-5,453	-5,154	564	-4,590
DIAPH.	2.10	-4,216	-3,761	-547	-4,308	-3,521	-3,778	-321	-4,099	-3,507	-3,687	-104	-3,791	-3,503	-3,655	104	-3,551	-3,471	-3,623	311	-3,312	-3,593	-3,438	516	-2,922
	2.15	-2,419	-2,171	-347	-2,518	-1,971	-2,155	-202	-2,357	-1,969	-2,079	-65	-2,144	-1,968	-2,060	65	-1,995	-1,953	-2,042	193	-1,849	-2,021	-1,947	321	-1,626
DIAPH.	2.20	-848	-806	-148	-954	-648	-759	-83	-842	-656	-694	-25	-719	-657	-688	25	-663	-655	-682	76	-606	-657	-664	126	-538
	2.25	502	352	81	433	466	421	53	474	453	475	20	495	450	471	-20	451	450	467	-59	408	476	422	-97	325
DIAPH.	2.30	1,647	1,305	309	1,614	1,366	1,388	190	1,578	1,350	1,433	65	1,498	1,347	1,421	-65	1,356	1,347	1,408	-194	1,214	1,414	1,313	-321	992
	2.35	2,540	2,052	481	2,533	2,056	2,141	293	2,434	2,060	2,180	100	2,280	2,057	2,160	-99	2,061	2,040	2,141	-295	1,846	2,142	2,008	-489	1,519
DIAPH.	2.40	3,222	2,589	653	3,242	2,531	2,680	395	3,075	2,558	2,714	133	2,847	2,558	2,690	-133	2,557	2,525	2,666	-397	2,269	2,674	2,507	-657	1,850
	2.45	3,604	2,912	717	3,629	2,824	3,004	433	3,437	2,868	3,035	146	3,181	2,867	3,009	-146	2,863	2,826	2,982	-434	2,548	2,986	2,807	-719	2,088
DIAPH.	2.50	3,770	3,020	780	3,800	2,900	3,112	471	3,583	2,965	3,142	158	3,300	2,963	3,115	-158	2,957	2,916	3,087	-471	2,616	3,097	2,907	-781	2,126

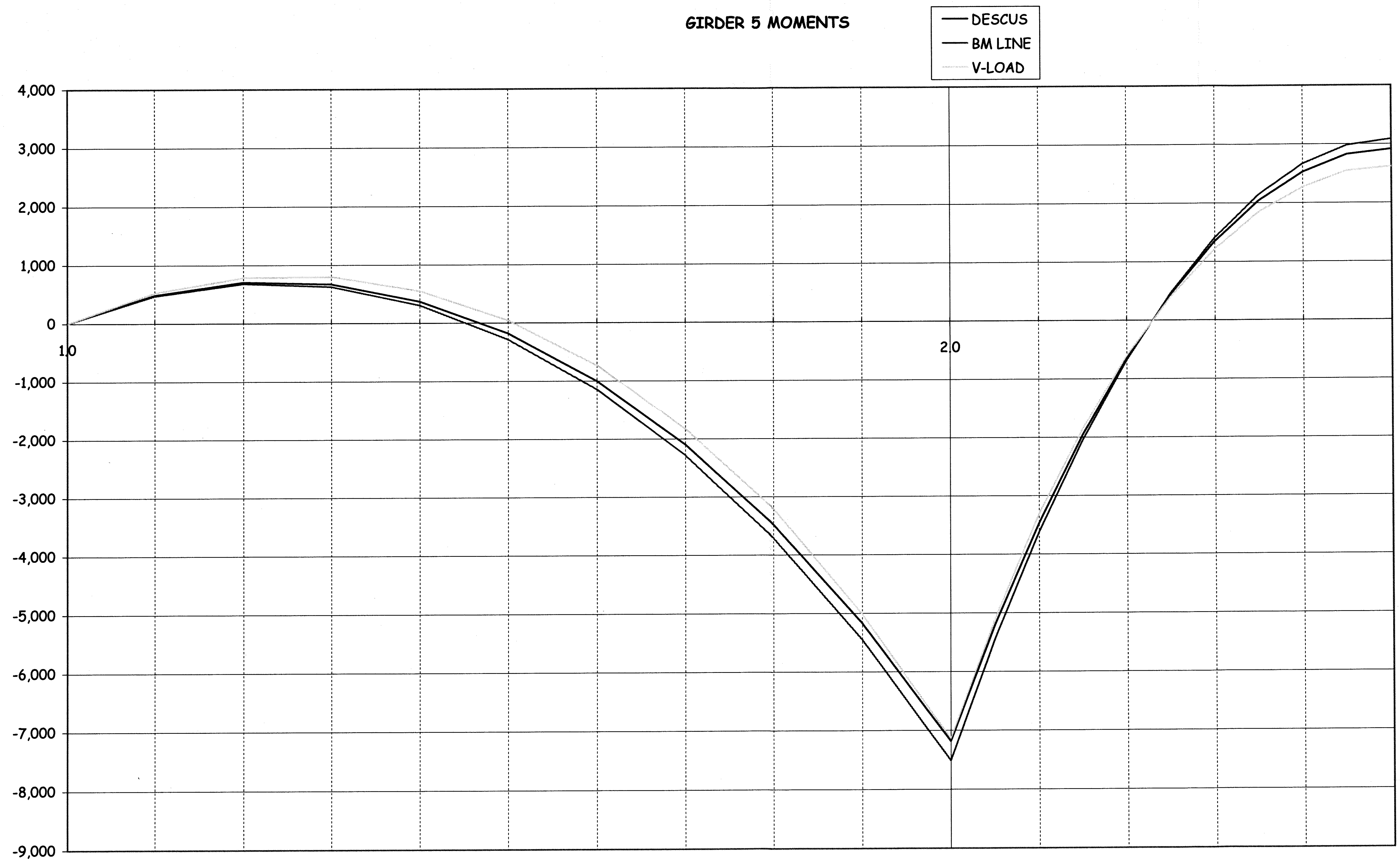

GIRDER 1 MOMENTS
DESCUS
BM LINE
V-LOAD
4,000
3,000
2,000
1,000
0
-1,000
-2,000
-3,000
-4,000
-5,000
-6,000
-7,000
-8,000
-9,000
1.0
2.0

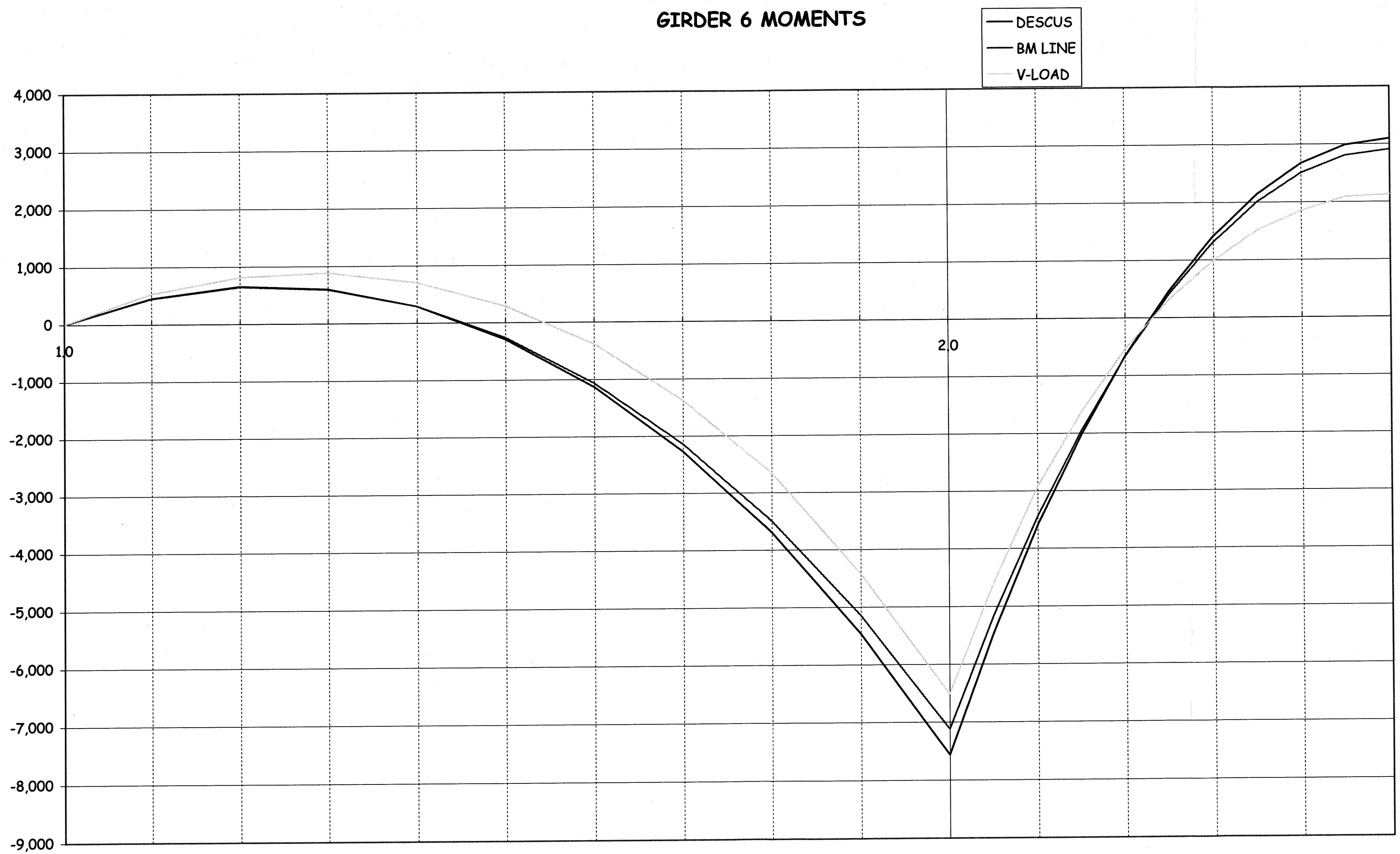


GIRDER 2 MOMENTS
DESCUS
BM LINE
V-LOAD
4,000
3,000
2,000
1,000
0
-1,000
-2,000
-3,000
-4,000
-5,000
-6,000
-7,000
-8,000
-9,000
1.0
2.0


GIRDER 3 MOMENTS
DESCUS
BM LINE
V-LOAD
4,000
3,000
2,000
1,000
0
-1,000
-2,000
-3,000
-4,000
-5,000
-6,000
-7,000
-8,000
-9,000
1.00
2.00


GIRDER 4 MOMENTS
DESCUS
BM LINE
V-LOAD
4,000
3,000
2,000
1,000
0
-1,000
-2,000
-3,000
-4,000
-5,000
-6,000
-7,000
-8,000
-9,000
1.0
2.0


GIRDER 5 MOMENTS
DESCUS
BM LINE
V-LOAD
4,000
3,000
2,000
1,000
0
-1,000
-2,000
-3,000
-4,000
-5,000
-6,000
-7,000
-8,000
-9,000
1.0
2.0


GIRDER 6 MOMENTS
DESCUS
BM LINE
V-LOAD
4,000
3,000
2,000
1,000
0
-1,000
-2,000
-3,000
-4,000
-5,000
-6,000
-7,000
-8,000
-9,000
1.0
2.0

Left lane: PI Sta 265+75 El 261.50
PC Sta 263+25 El. 258.75
PT Sta 268+25 El. 256.03
$G1 = +1.10$ $G2 = -2.19$ $L = 500$ ft

Right lane: PI Sta 265+98 El. 256.85
PC Sta 263+53 El. 253.84
PT Sta 268+43 El. 252.09
$G1 = +1.23$ $G2 = -1.94$ $L = 490$ ft

SE = 0.065 ft/ft left & down

Begin Bridge Sta 263+65
℄ Brg. Abut 1 Sta 263+67.25
℄ Bent 1 Sta 264+85
℄ Bent 2 Sta 266+95
℄ Brg Abut 2 Sta 268+12.75
End Bridge Sta 268+15

Max. Top flange $t = 4''$
Use $1\frac{3}{4}'' + 4'' + 8\frac{1}{2}'' = 14.25''$ from tos to tow
Abutment: G1-NB thru G6-SB Seat $= 14.25'' + 69'' + \frac{3}{4}'' + \frac{1}{2}''$ PAD
$= 84\frac{1}{2}''$ below tos
$= 7.042$ ft below tos

Bottom flange $t = 4''$, Girder 1-NB, Girder 1-SB
$t = 3\frac{1}{2}''$, G2NB, G6NB, G2SB, G6SB
$t = 3\frac{1}{4}''$, G3, G4, G5 NB & SB

G1 : $14.25'' + 4'' + 69'' + 6.375''$ brg $= 93.625''$ tos-seat
$= 7.802$ ft

G2, G6 : $93.125'' = 7.760$ ft tos-seat
G3, G4, G5 : $92.875'' = 7.740$ ft tos-seat

Left Lane: (Southbound Bridge)

G6: 44.625' left of f.g. (-2.901 ft)
G5: 34.375' left of f.g. (-2.234 ft)
G4: 24.125' left of f.g. (-1.568 ft)
G3: 13.875' left of f.g. (-0.902 ft)
G2: 3.625' left of f.g. (-0.236 ft)
G1: 6.625' right of f.g. (+0.431 ft)

Right Lane: (Northbound Bridge)

G6: 6.625' left of f.g. (-0.431 ft)
G5: 3.625' right of f.g. (+0.236 ft)
G4. 13.875' right of f.g. (+0.902 ft)
G3. 24.125' right of f.g. (+1.568 ft)
G2: 34.375' right of f.g. (+2.234 ft)
G1: 44.625' right of f.g. (+2.901 ft)

Abutment No. 1

℄ Bearing 263+67.25 f.g. left (SB) = 259.16
f.g. right (NB) = 254.01

Beam Seats

	G1	G2	G3	G4	G5	G6
SB (left)	252.549	251.882	251.216	250.549	249.883	249.217
NB (right)	249.869	249.203	248.536	247.870	247.204	246.538

Bent No. 1

℄ Bearing 264+85 f.g. left (SB) = 259.67
f.g. right (NB) = 254.90

	G1	G2	G3	G4	G5	G6
SB (left)	252.299	251.674	251.028	250.362	249.695	249.009
NB (right)	249.999	249.374	248.728	248.062	247.396	246.709

Bent No. 2

℄ Bearing 266+95 f.g. left (SB) = 258.32
f.g. right (NB) = 254.26

	G1	G2	G3	G4	G5	G6
SB (left)	250.949	250.324	249.678	249.012	248.345	247.659
NB (right)	249.359	248.734	248.088	247.422	246.755	246.069

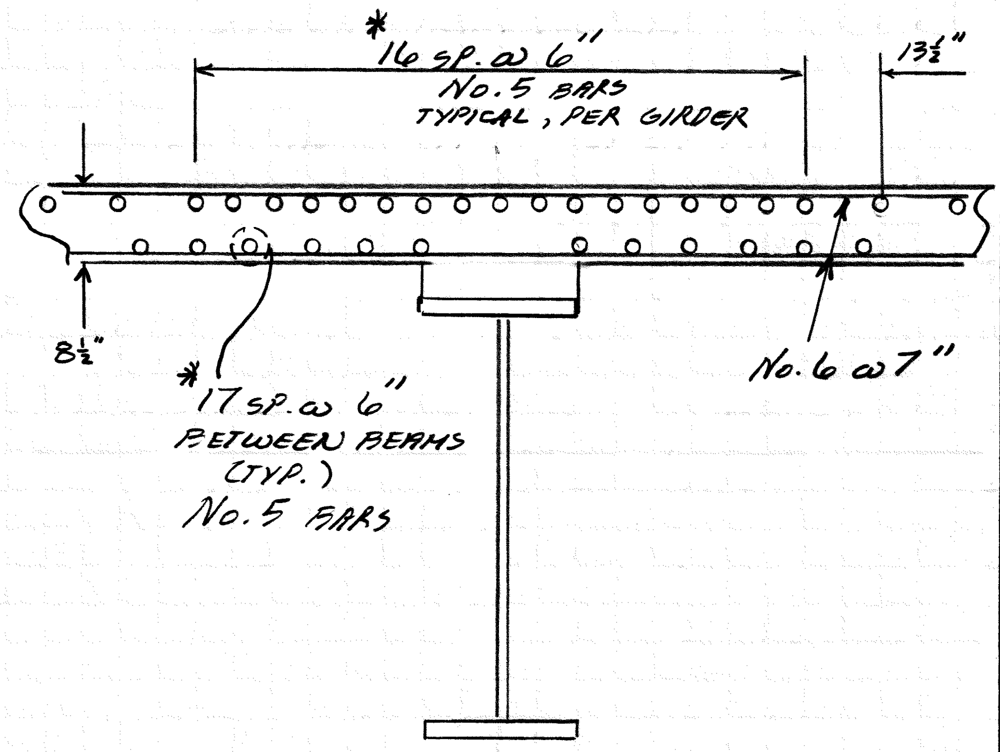
Abutment No. 2

℄ Bearing 268+12.75 f.g. left (SB) = 256.29
f.g. right (NB) = 252.65

	G1	G2	G3	G4	G5	G6
SB (left)	249.679	249.013	248.346	247.680	247.014	246.348
NB (right)	248.509	247.843	247.176	246.510	245.844	245.178

COUNTY: CROSSING:

NOTES



* NOTE TO DETAILER: CUT OFF
EVERY OTHER BAR:
84' LEFT OF BENT 1
54' RIGHT OF BENT 1
54' LEFT OF BENT 2
84' RIGHT OF BENT 2

DESIGN	DATE
CHECK	DATE
CHECK	DATE

DL CAMBER, INCHES

SPAN	GIRDER 1 SB		GIRDER 2 SB		GIRDER 3 SB		GIRDER 4 SB		GIRDER 5 SB		GIRDER 6 SB	
PT	DL STEEL	DL TOTAL	DL STEEL	DL TOTAL	DL STEEL	DL TOTAL	DL STEEL	DL TOTAL	DL STEEL	DL TOTAL	DL STEEL	DL TOTAL
1.00	0	0	0	0	0	0	0	0	0	0	0	0
1.10	0	0	0	0	0	1/8	0	1/8	0	1/8	0	0
1.20	0	-0	0	0	0	1/8	0	1/8	0	1/8	0	0
1.30	0	- 1/4	0	- 1/8	0	-0	0	-0	0	-0	0	- 1/8
1.40	0	- 3/8	0	- 1/4	0	- 1/4	0	- 1/4	0	- 1/4	0	- 1/4
1.50	-0	- 5/8	-0	- 1/2	-0	- 1/2	-0	- 1/2	-0	- 1/2	-0	- 1/2
1.60	-0	- 7/8	-0	- 3/4	-0	- 3/4	-0	- 3/4	-0	- 3/4	-0	- 3/4
1.70	- 1/8	- 7/8	- 1/8	- 7/8	- 1/8	- 7/8	- 1/8	- 7/8	- 1/8	- 7/8	- 1/8	- 7/8
1.80	- 1/8	- 7/8	- 1/8	- 3/4	- 1/8	- 3/4	- 1/8	- 3/4	- 1/8	- 3/4	- 1/8	- 3/4
1.90	-0	- 1/2	-0	- 1/2	-0	- 1/2	-0	- 1/2	-0	- 1/2	-0	- 1/2
2.00	0	0	0	0	0	0	0	0	0	0	0	0
2.05	1/8	3/4	1/8	5/8	1/8	5/8	1/8	5/8	1/8	5/8	1/8	5/8
2.10	1/4	1 5/8	1/4	1 5/8	1/4	1 5/8	1/4	1 1/2	1/4	1 1/2	1/8	1 4/8
2.15	3/8	2 5/8	3/8	2 5/8	3/8	2 5/8	3/8	2 5/8	3/8	2 5/8	3/8	2 1/2
2.20	1/2	3 7/8	1/2	3 7/8	1/2	3 7/8	1/2	3 7/8	1/2	3 3/4	1/2	3 5/8
2.25	3/4	5 1/8	5/8	5 1/8	5/8	5 1/8	5/8	5 1/8	5/8	5	5/8	4 7/8
2.30	7/8	6 3/8	7/8	6 3/8	7/8	6 3/8	7/8	6 3/8	3/4	6 1/4	3/4	6
2.35	1	7 3/8	1	7 1/2	1	7 1/2	1	7 3/8	1	7 1/4	7/8	7
2.40	1 1/8	8 1/8	1 1/8	8 1/4	1 1/8	8 1/4	1 1/8	8 1/8	1	8	1	7 3/4
2.45	1 1/4	8 5/8	1 1/8	8 3/4	1 1/8	8 3/4	1 1/8	8 5/8	1 1/8	8 1/2	1 1/8	8 1/4
2.50	1 1/4	8 3/4	1 1/4	8 7/8	1 1/4	8 7/8	1 1/8	8 7/8	1 1/8	8 5/8	1 1/8	8 3/8

(SYMMETRIC WITH RESPECT TO MIDSPAN OF SPAN NO. 2)

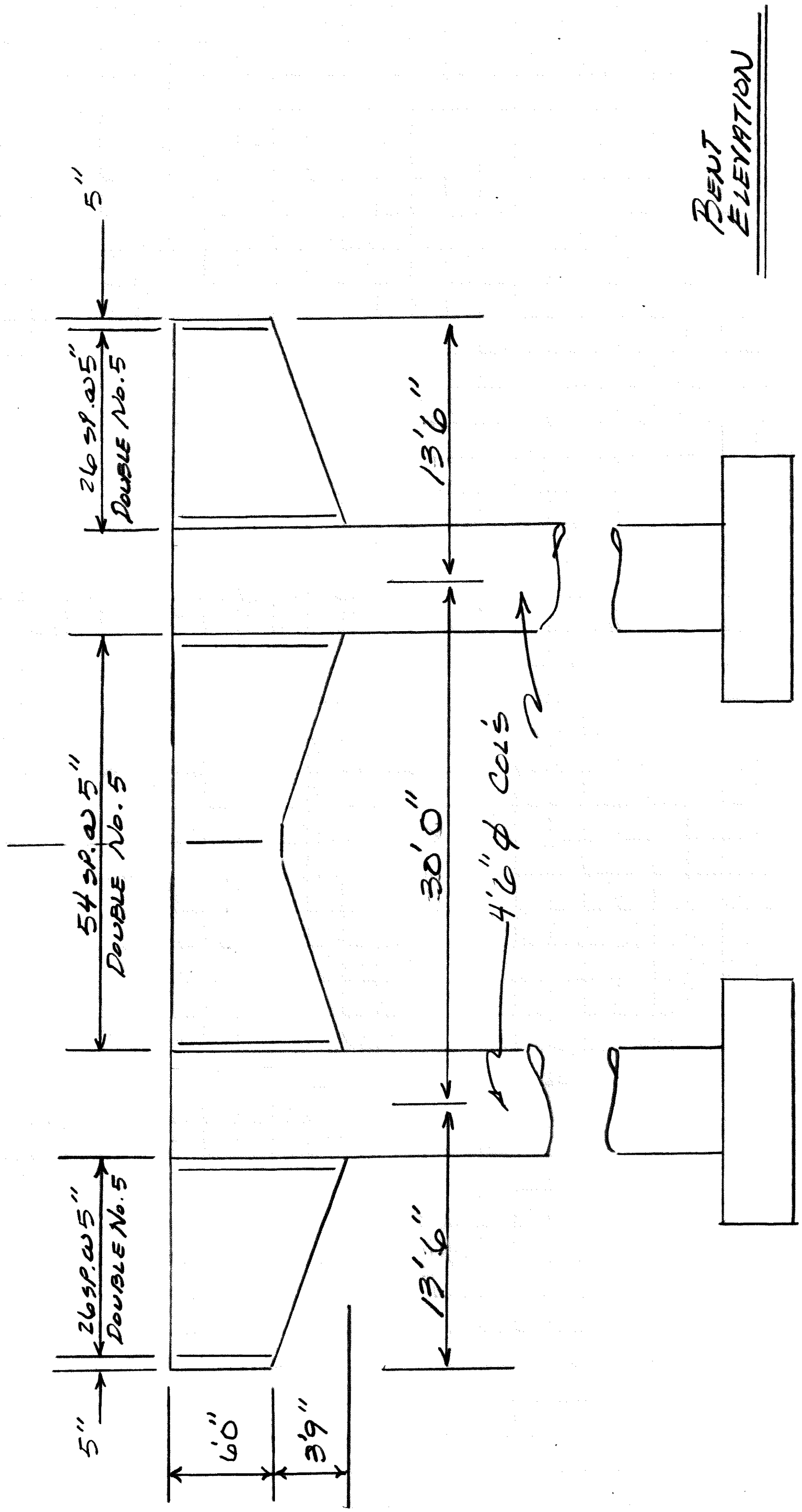
DL CAMBER, INCHES

SPAN	GIRDER 1 NB		GIRDER 2 NB		GIRDER 3 NB		GIRDER 4 NB		GIRDER 5 NB		GIRDER 6 NB	
PT	DL STEEL	DL TOTAL	DL STEEL	DL TOTAL	DL STEEL	DL TOTAL	DL STEEL	DL TOTAL	DL STEEL	DL TOTAL	DL STEEL	DL TOTAL
1.00	0	0	0	0	0	0	0	0	0	0	0	0
1.10	0	0	0	0	0	1/8	0	1/8	0	1/8	0	0
1.20	0	-0	0	0	0	1/8	0	1/8	0	1/8	0	0
1.30	0	- 1/4	0	- 1/8	0	-0	0	-0	0	-0	0	- 1/8
1.40	0	- 1/2	0	- 3/8	0	- 1/4	0	- 1/4	0	- 1/4	0	- 1/4
1.50	-0	- 3/4	-0	- 5/8	-0	- 1/2	-0	- 1/2	-0	- 1/2	-0	- 5/8
1.60	-0	-1	-0	- 7/8	-0	- 3/4	-0	- 3/4	-0	- 3/4	-0	- 3/4
1.70	- 1/8	-1	- 1/8	-1	- 1/8	- 7/8	- 1/8	- 7/8	- 1/8	- 7/8	- 1/8	- 7/8
1.80	- 1/8	- 7/8	- 1/8	- 7/8	- 1/8	- 7/8	- 1/8	- 7/8	- 1/8	- 7/8	- 1/8	- 7/8
1.90	- 1/8	- 5/8	- 1/8	- 5/8	- 1/8	- 5/8	- 1/8	- 5/8	-0	- 1/2	-0	- 1/2
2.00	0	0	0	0	0	0	0	0	0	0	0	0
2.05	1/8	3/4	1/8	3/4	1/8	3/4	1/8	3/4	1/8	3/4	1/8	3/4
2.10	1/4	1 3/4	1/4	1 3/4	1/4	1 3/4	1/4	1 3/4	1/4	1 3/4	1/4	1 5/8
2.15	3/8	2 7/8	3/8	2 7/8	3/8	2 7/8	3/8	2 7/8	3/8	2 7/8	3/8	2 3/4
2.20	5/8	4 1/4	1/2	4 1/4	1/2	4 1/4	1/2	4 1/4	1/2	4 1/8	1/2	4
2.25	3/4	5 3/4	3/4	5 3/4	3/4	5 3/4	3/4	5 5/8	3/4	5 5/8	3/4	5 3/8
2.30	1	7 1/8	1	7 1/8	7/8	7 1/8	7/8	7	7/8	6 7/8	7/8	6 3/4
2.35	1 1/8	8 1/4	1 1/8	8 1/4	1 1/8	8 1/4	1 1/8	8 1/4	1	8	1	7 7/8
2.40	1 1/4	9 1/8	1 1/4	9 1/8	1 1/4	9 1/8	1 1/4	9 1/8	1 1/8	8 7/8	1 1/8	8 5/8
2.45	1 3/8	9 5/8	1 3/8	9 5/8	1 1/4	9 3/4	1 1/4	9 5/8	1 1/4	9 3/8	1 1/4	9 1/8
2.50	1 3/8	9 3/4	1 3/8	9 7/8	1 3/8	9 7/8	1 1/4	9 3/4	1 1/4	9 5/8	1 1/4	9 3/8

(SYMMETRIC WITH RESPECT TO MIDSPAN OF SPAN NO. 2)

COUNTY: CROSSING:

NOTES



DESIGN	DATE
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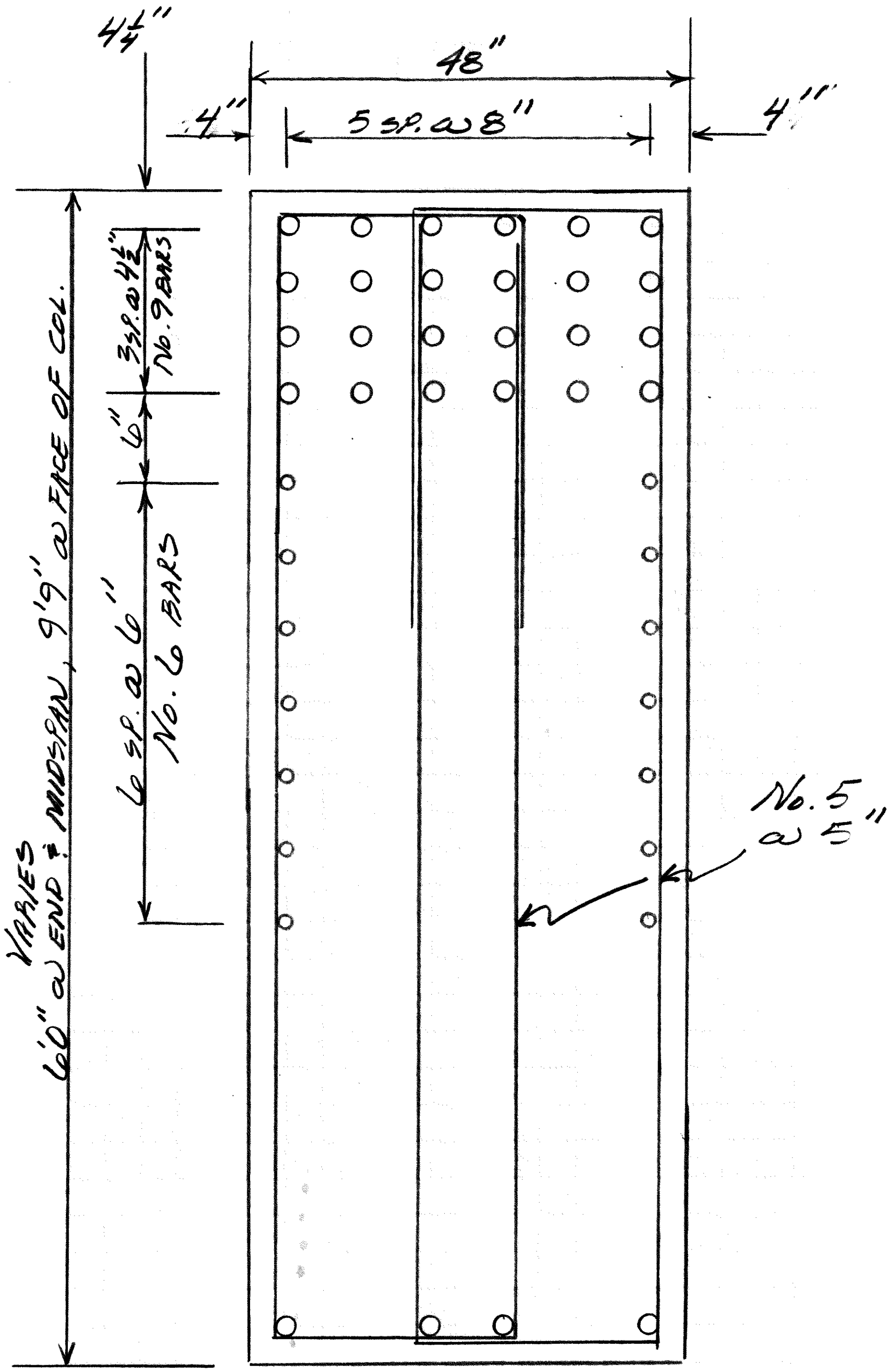
CHECK	DATE

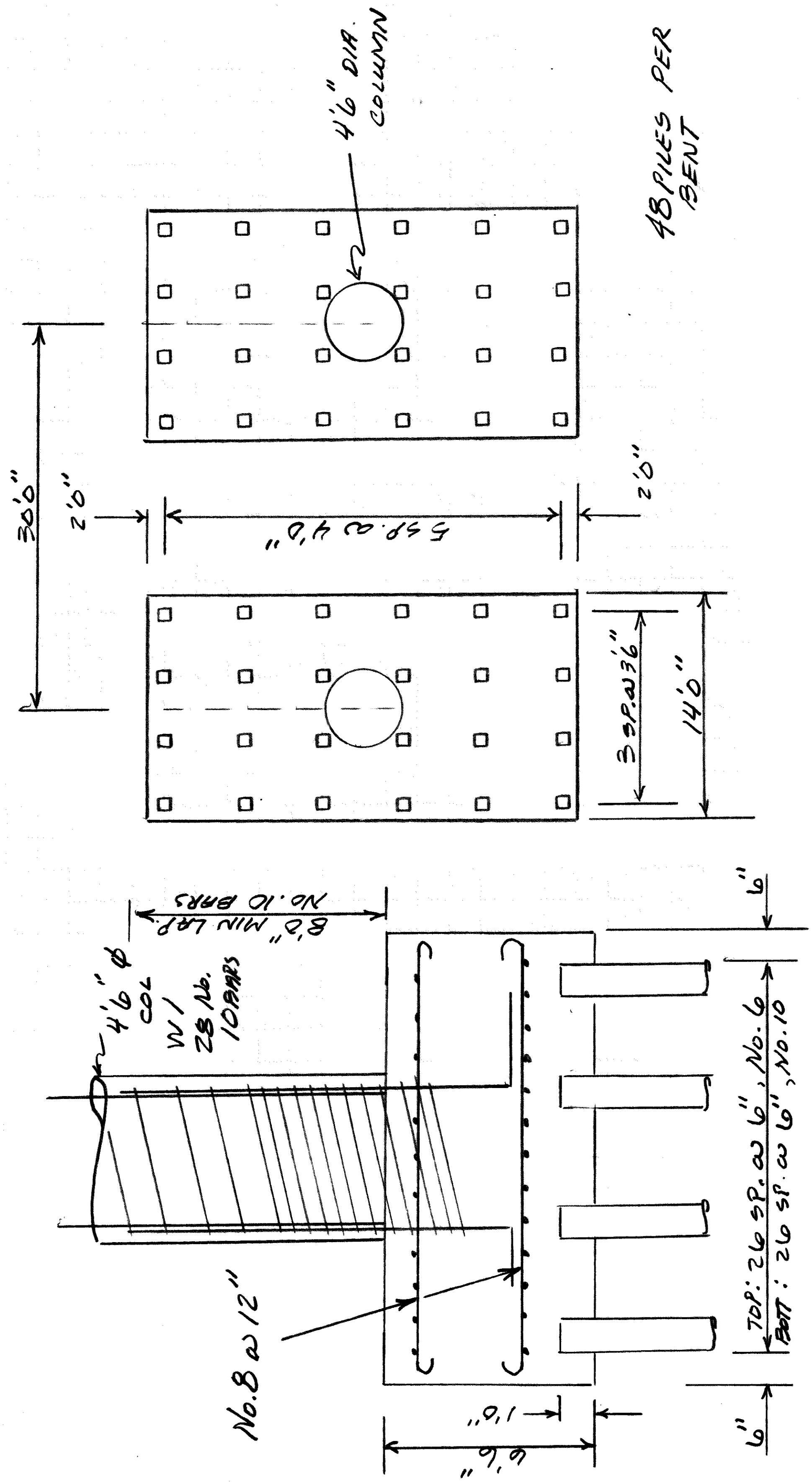
NOTES



BENT CAP SECTION

DESIGN	DATE
CHECK	DATE
CHECK	DATE

COUNTY: CROSSING:



NOTES

14" SQ. PRESTRESSED CONCRETE FRICTION PILES. DRIVE TO 100 TONS.

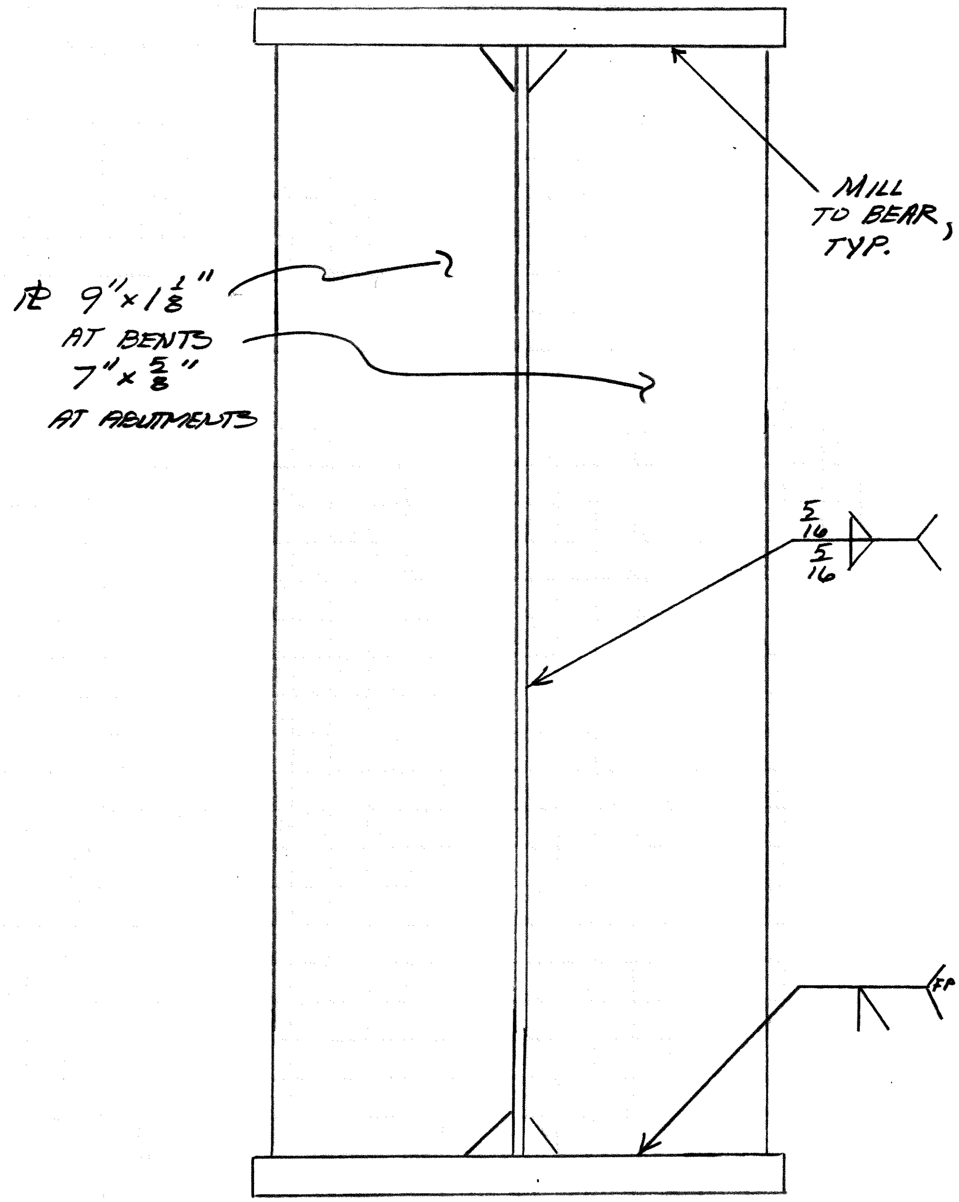
COLUMN SPIRALS:
No. 5 @ 4" PITCH (END REGIONS)
No. 5 @ 6" PITCH (MID REGIONS)
SPIRALS EXTEND 2'3" INTO CAP & FOOTING WITH 4" PITCH.
"END REGION" IS 4'6" BELOW TOP OF COL. & 4'6" ABOVE BOTTOM OF COL.

DESIGN	DATE
CHECK	DATE
CHECK	DATE

COUNTY: CROSSING:

NOTES

BEARING STIFFENERS



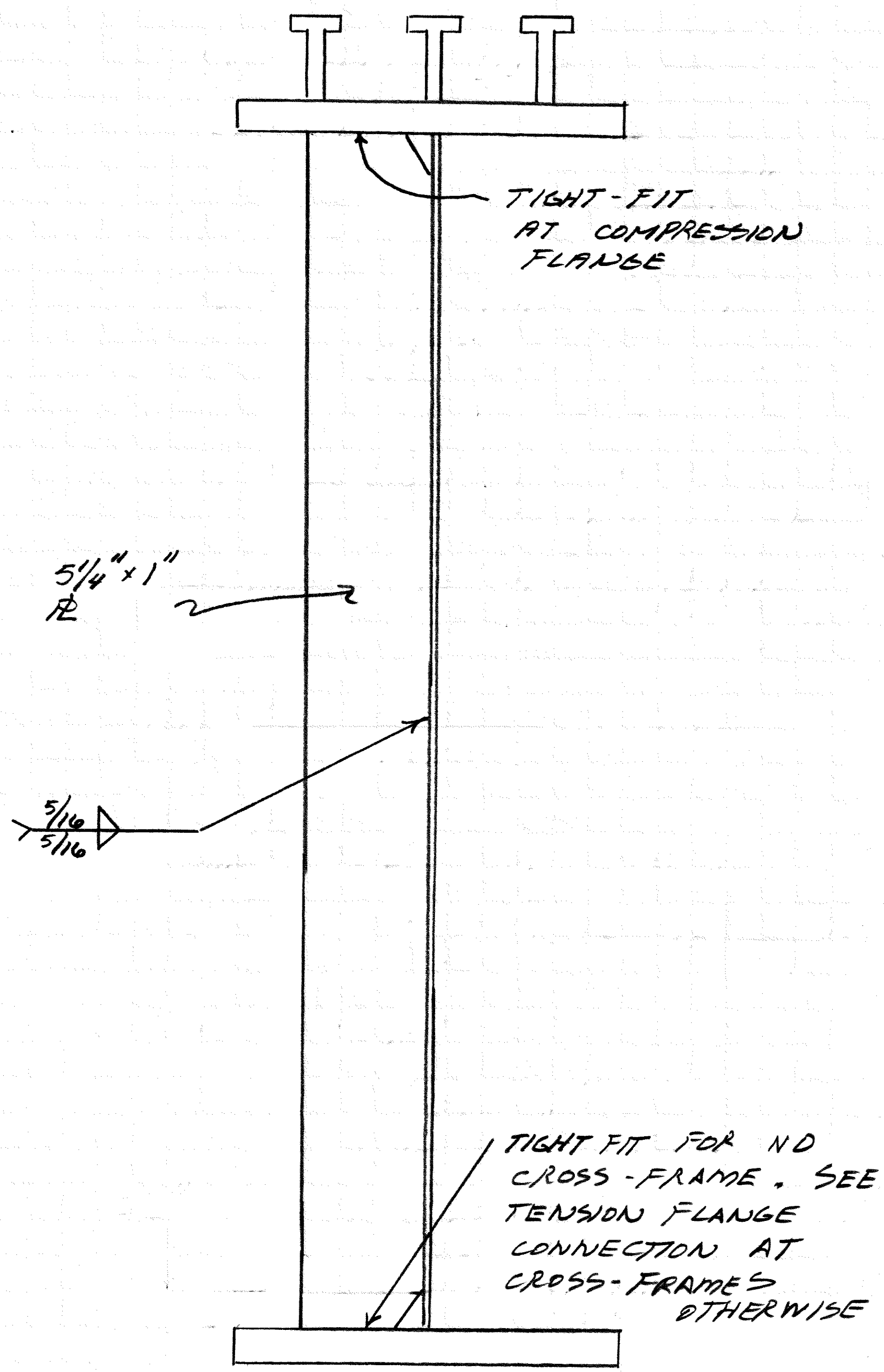
DESIGN	DATE
CHECK	DATE
CHECK	DATE

COUNTY: CROSSING:

NOTES

TRANSVERSE STIFFENERS

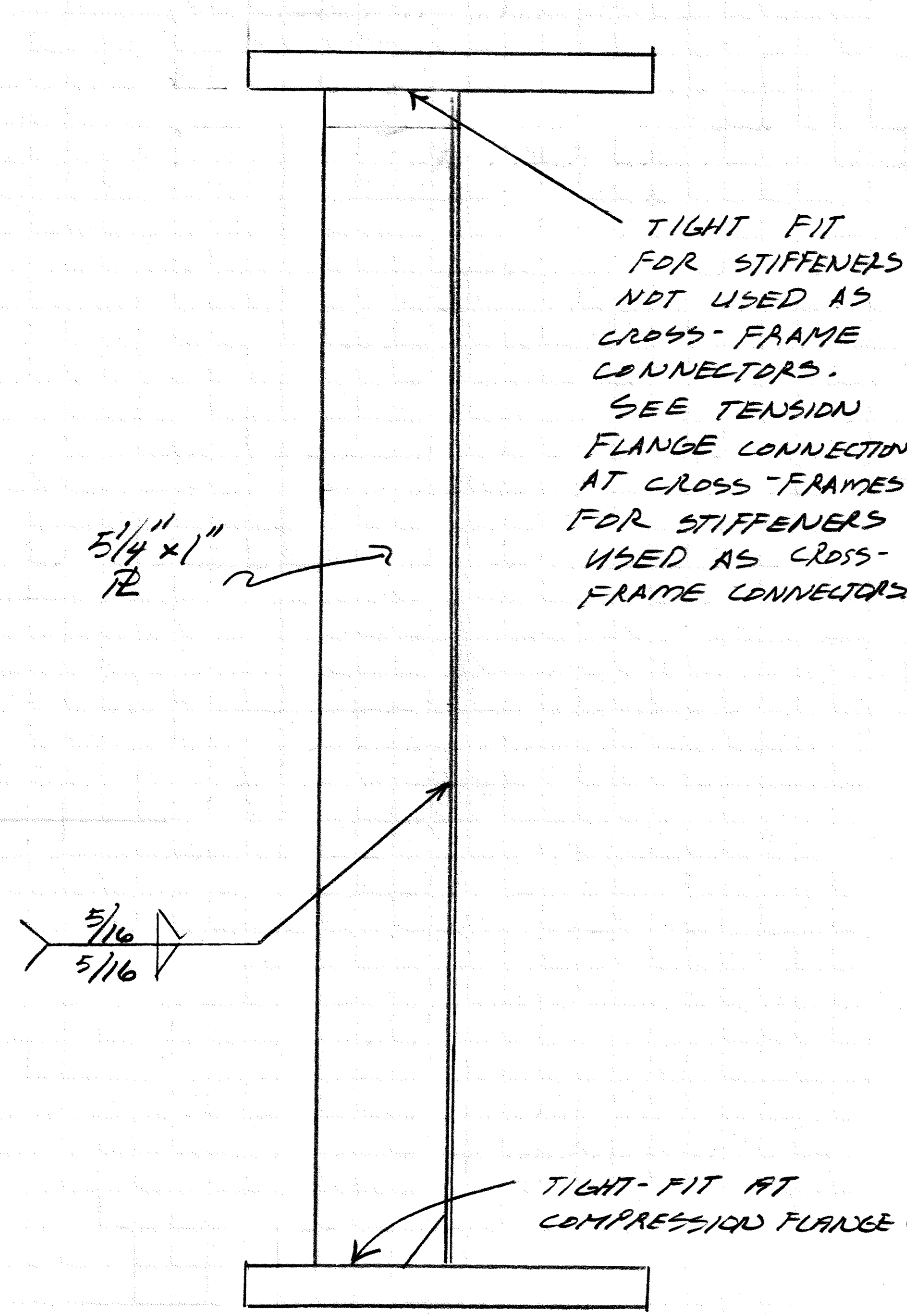
POSITIVE MOMENT REGION



DESIGN	DATE
CHECK	DATE
CHECK	DATE

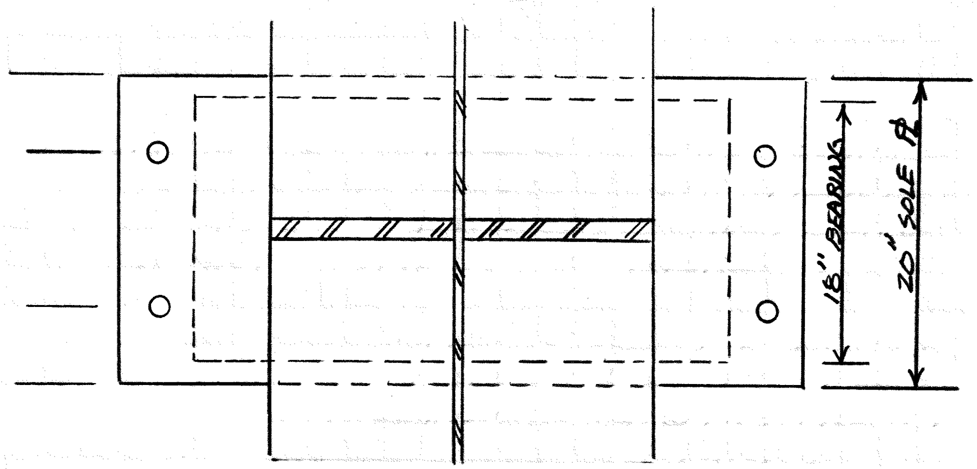
COUNTY: CROSSING: NOTES

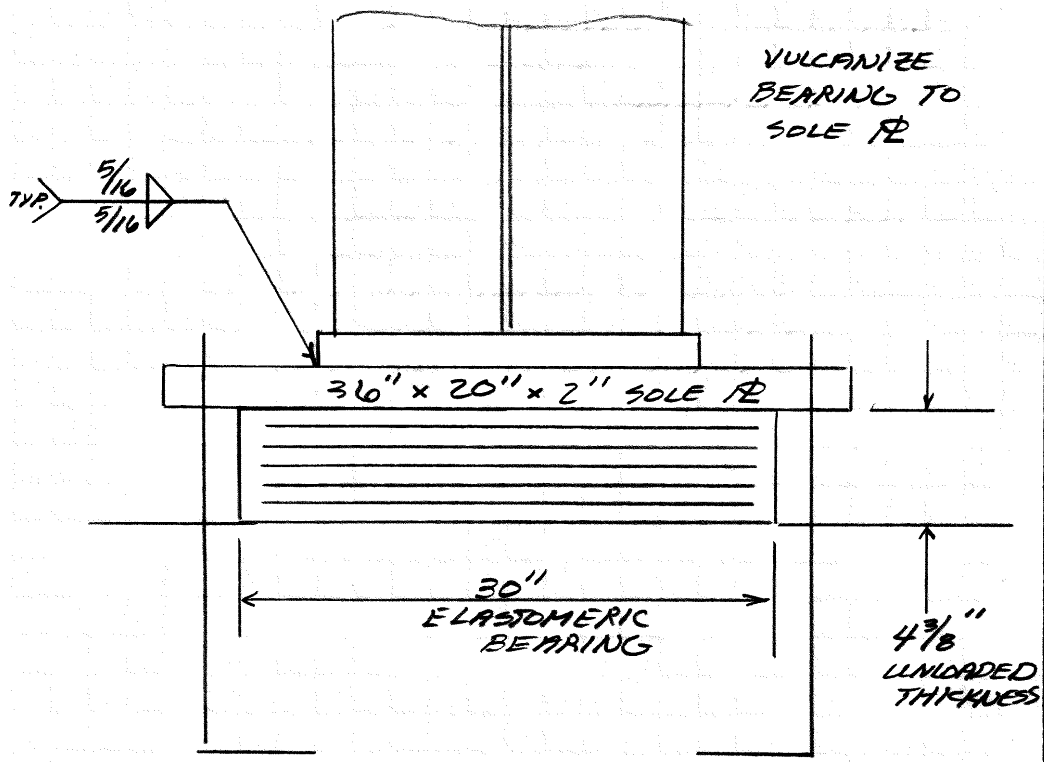
TRANSVERSE STIFFENERS
NEGATIVE MOMENT REGION



DESIGN	DATE
CHECK	DATE
CHECK	DATE

COUNTY:	CROSSING:	NOTES





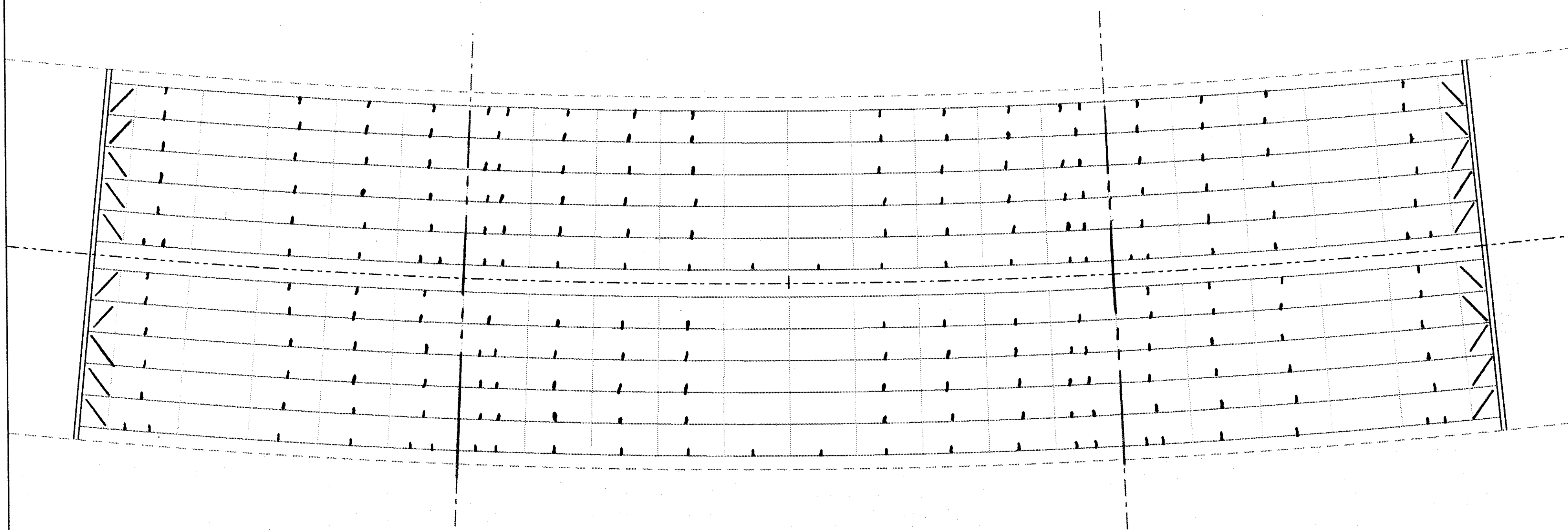
BEARINGS AT BENTS
(ALL GIRDERS)

BEARING SPEC.: 4, 3/4" INTERNAL LAYERS
2, 3/8" COVER LAYERS
5, 1/8" STEEL LAYERS

ELASTOMERIC LAYERS: SHORE A HARDNESS = 60

STEEL LAYERS: F_y = 36ksi (MIN.)

DESIGN	DATE
CHECK	DATE
CHECK	DATE



TRANSVERSE STIFFENER LAYOUT

STIFFENERS TO BE SPACED EQUALLY BETWEEN CROSS-FRAMES.

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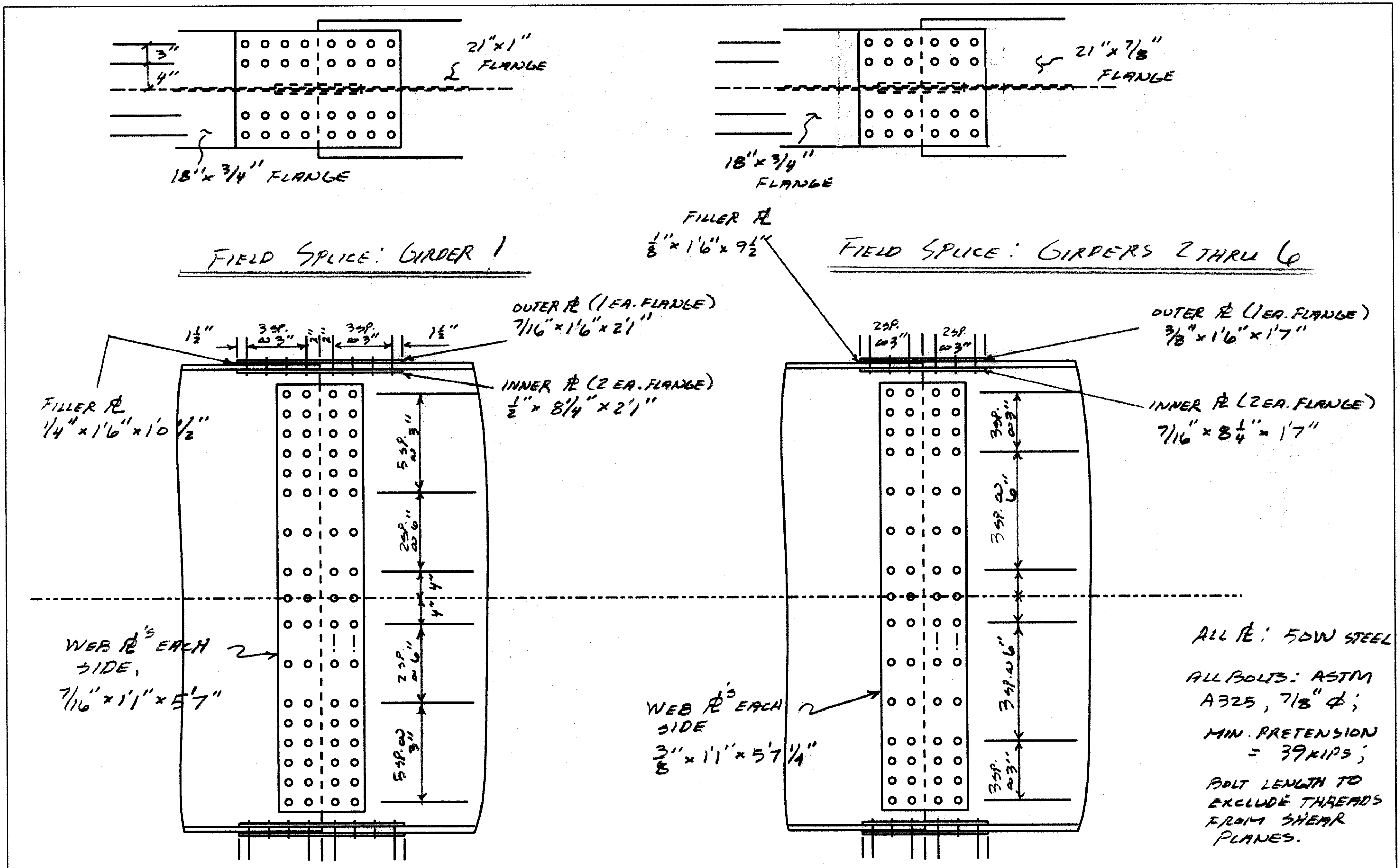

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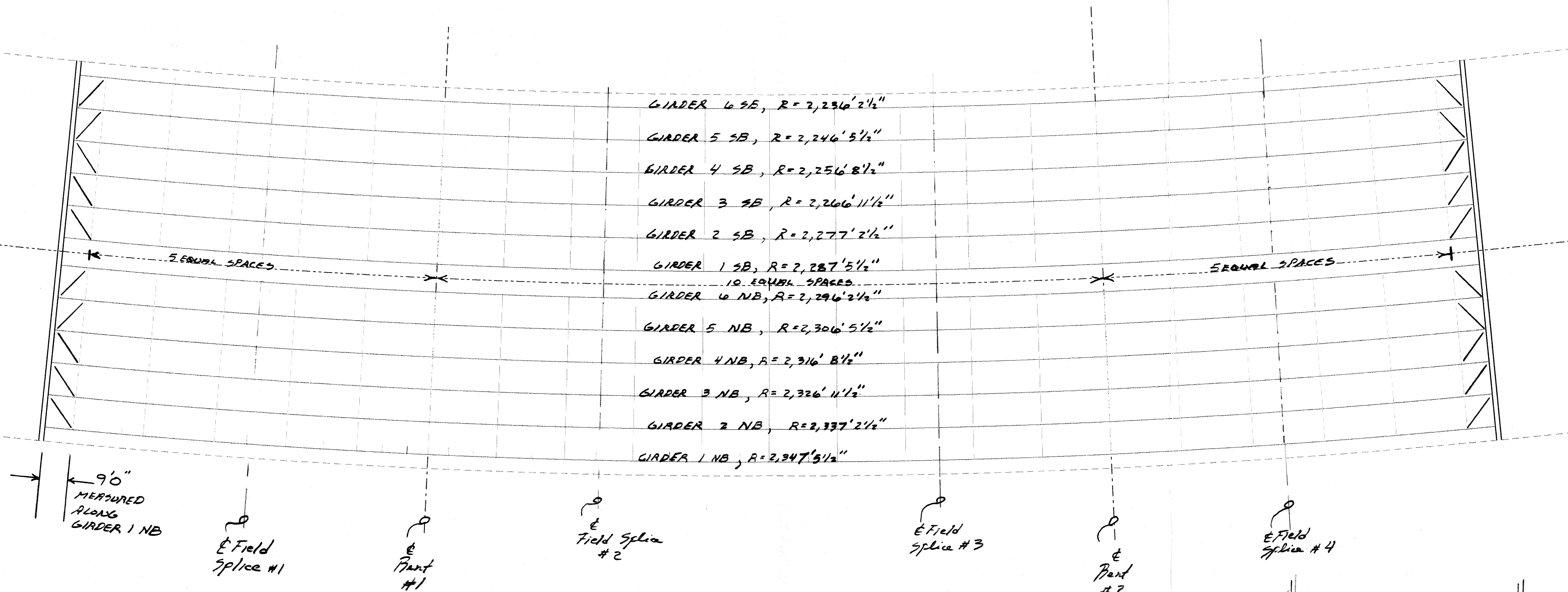
BLAST CLEAN SURFACES AT ALL FIELD SPLICE LOCATIONS PRIOR TO SPLICE INSTALLATION.



Framing Plan

I-55 OVER
MALLORY AVENUE
SHELBY COUNTY
HUFF

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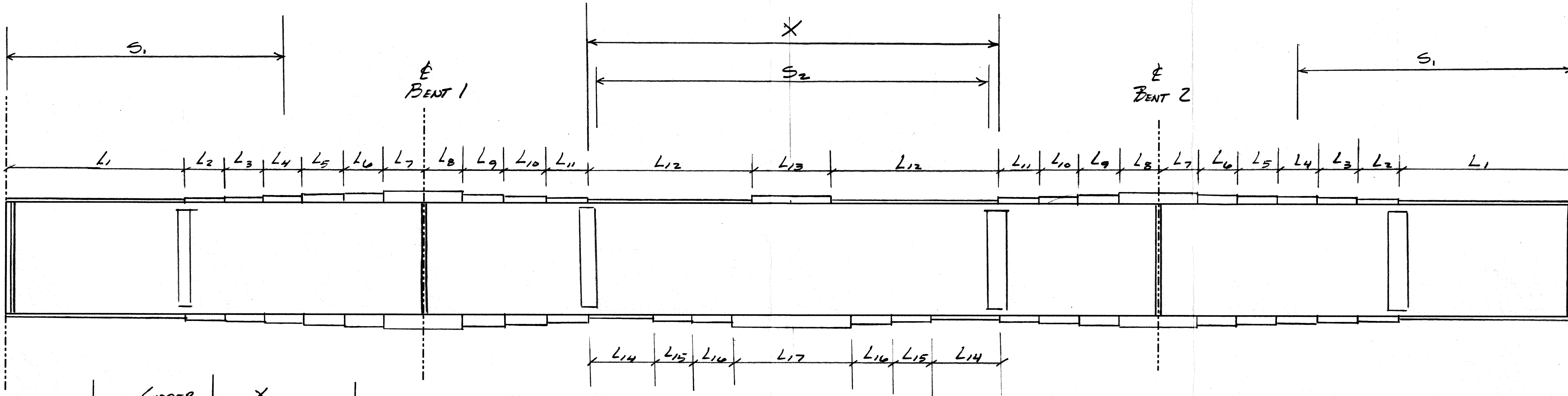
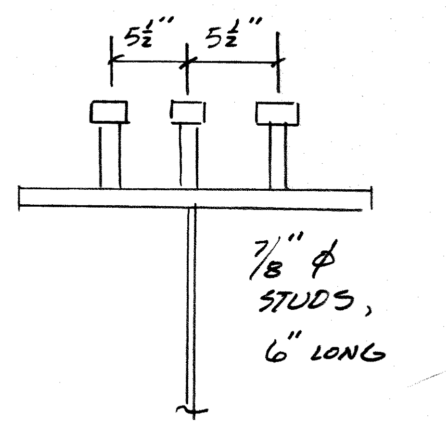


GIRDER	X
1 NB	118' 7"
2 NB	118' 0 3/4"
3 NB	117' 6 1/2"
4 NB	117' 0 3/8"
5 NB	116' 6"
6 NB	116' 0"
1 SB	115' 6 5/8"
2 SB	115' 0 1/2"
3 SB	114' 6 1/4"
4 SB	114' 0"
5 SB	113' 5 3/4"
6 SB	112' 11 1/2"



SHEAR STUD DETAIL

GIRDER	SHEAR STUDS S_1	SHEAR STUDS S_2	GIRDER	SHEAR STUDS S_1	SHEAR STUDS S_2
1 NB	100 SP. @ 9"	154 SP. @ 9"	1 SB	100 SP. @ 9"	153 SP. @ 9"
2 NB	92 SP. @ 1'1"	108 SP. @ 1'1"	2 SB	92 SP. @ 1'1"	106 SP. @ 1'1"
3 NB	92 SP. @ 1'1"	108 SP. @ 1'1"	3 SB	92 SP. @ 1'1"	105 SP. @ 1'1"
4 NB	92 SP. @ 1'1"	107 SP. @ 1'1"	4 SB	92 SP. @ 1'1"	105 SP. @ 1'1"
5 NB	92 SP. @ 1'1"	107 SP. @ 1'1"	5 SB	92 SP. @ 1'1"	104 SP. @ 1'1"
6 NB	101 SP. @ 11"	125 SP. @ 11"	6 SB	101 SP. @ 11"	123 SP. @ 11"

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	R=2,347.4562'		R= 2,337.2062'		R= 2,326.9562'		R= 2,316.7062'		R= 2,306.4562'		R= 2,296.2062'	
	GIRDER 1 NB		GIRDER 2 NB		GIRDER 3 NB		GIRDER 4 NB		GIRDER 5 NB		GIRDER 6 NB	
	TOP FLANGE	BOTTOM FLANGE	TOP FLANGE	BOTTOM FLANGE	TOP FLANGE	BOTTOM FLANGE	TOP FLANGE	BOTTOM FLANGE	TOP FLANGE	BOTTOM FLANGE	TOP FLANGE	BOTTOM FLANGE
L_1	18"× 3/4" 48'0"	18"× 3/4" 48'0"	18"× 3/4" 47'9½"	18"× 3/4" 47'9½"	18"× 3/4" 47'7"	18"× 3/4" 47'7"	18"× 3/4" 47'4½"	18"× 3/4" 47'4½"	18"× 3/4" 47'2"	18"× 3/4" 47'2"	18"× 3/4" 46'11½"	18"× 3/4" 46'11½"
L_2	21"× 1" 10'10½"	21"× 1" 10'10½"	—	—	—	—	—	—	—	—	21"× 7/8" 9'3¼"	21"× 7/8" 9'3¼"
L_3	21"× 1½" 10'0"	21"× 1½" 10'0"	21"× 7/8" 20'6¾"	21"× 7/8" 20'6¾"	21"× 7/8" 20'2¾"	21"× 7/8" 20'2¾"	21"× 7/8" 19'11"	21"× 7/8" 19'11"	21"× 7/8" 19'7"	21"× 7/8" 19'7"	21"× 1" 10'0"	21'× 1" 10'0"
L_4	21"× 2" 15'0"	21"× 2" 15'0"	21"× 1½" 15'0"	21"× 1½" 15'0"	21"× 1¼" 15'0"	21"× 1¼" 15'0"	21"× 1¼" 15'0"	21"× 1¼" 15'0"	21"× 1¼" 15'0"	21"× 1¼" 15'0"	21"× 1½" 15'0"	21"× 1½" 15'0"
L_5	21"× 2½" 10'0"	21"× 2½" 10'0"	21"× 2" 10'0"	21"× 2" 10'0"	21"× 1¾" 10'0"	21"× 1¾" 10'0"	21"× 1¾" 10'0"	21"× 1¾" 10'0"	21"× 1¾" 10'0"	21"× 1¾" 10'0"	21"× 2" 10'0"	21"× 2" 10'0"
L_6	21"× 3¼" 15'0"	21"× 3¼" 15'0"	21"× 2½" 15'0"	21"× 2½" 15'0"	21"× 2½" 15'0"	21"× 2½" 15'0"	21"× 2½" 15'0"	21'× 2½" 15'0"	21"× 2½" 15'0"	21"× 2½" 15'0"	21"× 2¾" 15'0"	21"× 2¾" 15'0"
L_7	21"× 4" 12'6"	21"× 4" 12'6"	21"× 3½" 12'6"	21"× 3½" 12'6"	21"× 3¼" 12'6"	21"× 3¼" 12'6"	21"× 3¼" 12'6"	21"× 3¼" 12'6"	21"× 3¼" 12'6"	21"× 3¼" 12'6"	21"× 3½" 12'6"	21"× 3½" 12'6"
L_8	21"× 4" 10'9"	21"× 4" 10'9"	21"× 3½" 10'9"	21"× 3½" 10'9"	21"× 3¼" 10'9"	21"× 3¼" 10'9"	21"× 3¼" 10'9"	21"× 3¼" 10'9"	21"× 3¼" 10'9"	21"× 3¼" 10'9"	21"× 3½" 10'9"	21"× 3½" 10'9"
L_9	21"× 3" 15'0"	21"× 3" 15'0"	21"× 2¼" 15'0"	21"× 2¼" 15'0"	21"× 2¼" 15'0"	21"× 2¼" 15'0"	21"× 2¼" 15'0"	21"× 2¼" 15'0"	21"× 2¼" 15'0"	21"× 2¼" 15'0"	21"× 2½" 15'0"	21"× 2½" 15'0"
L_{10}	21"× 2" 7'6"	21"× 2" 7'6"	21"× 1½" 7'6"	21"× 1½" 7'6"	21"× 1½" 7'6"	21"× 1½" 7'6"	21"× 1½" 7'6"	21"× 1½" 7'6"	21"× 1½" 7'6"	21"× 1½" 7'6"	21"× 1½" 7'6"	21"× 1½" 7'6"
L_{11}	21"× 1" 15'0"	21"× 1" 15'0"	21"× 7/8" 14'9½"	21"× 7/8" 14'9½"	21"× 7/8" 14'7"	21"× 7/8" 14'7"	21"× 7/8" 14'4½"	21"× 7/8" 14'4½"	21"× 7/8" 14'2"	21"× 7/8" 14'2"	21"× 7/8" 13'11½"	21"× 7/8" 13'11½"
L_{12}	18"× 3/4" 48'6½"		18"× 3/4" 59'0⅜"		18"× 3/4" 58'9¼"		18"× 3/4" 58'6¼"		18"× 3/4" 58'3"		18"× 3/4" 58'0"	
L_{13}	18"× 1" 21'6"		—		—		—		—		—	
L_{14}		18"× 3/4" 14'3½"		18"× 3/4" 34'0⅜"		18"× 3/4" 33'9¼"		18"× 3/4" 33'6¼"		18"× 3/4" 33'3"		18"× 3/4" 13'0"
L_{15}		18"× 1¼" 12'6"		—		—		—		—		18"× 1" 20'0"
L_{16}		18"× 1½" 7'6"		—		—		—		—		—
L_{17}		18"× 1¾" 50'0"		18"× 1" 50'0"		18"× 1" 50'0"		18"× 1" 50'0"		18"× 1" 50'0"		18"× 1¼" 50'0"

NORTHBOUND BRIDGE

GIRDER SECTIONS

ALL WEBS: 69"× 1/2"

WEBS, FLANGES: GRADE 50W

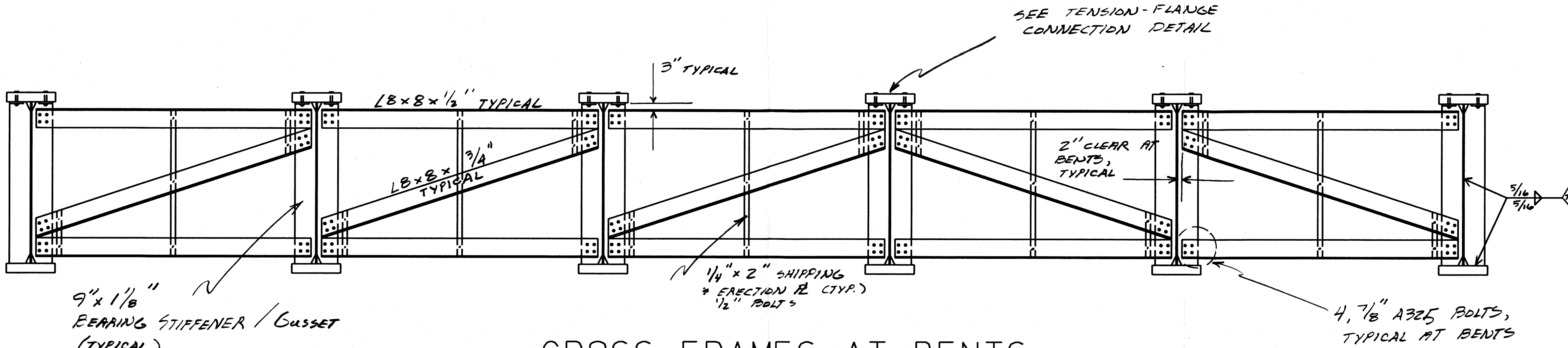
	R = 2,287.4562'		R = 2,277.2062'		R = 2,266.9562'		R = 2,256.7062'		R = 2,246.4562		R = 2,236.2062'	
	GIRDER 1 SB		GIRDER 2 SB		GIRDER 3 SB		GIRDER 4 SB		GIRDER 5 SB		GIRDER 6 SB	
	TOP FLANGE	BOTTOM FLANGE	TOP FLANGE	BOTTOM FLANGE	TOP FLANGE	BOTTOM FLANGE	TOP FLANGE	BOTTOM FLANGE	TOP FLANGE	BOTTOM FLANGE	TOP FLANGE	BOTTOM FLANGE
L_1	18"×3/4" 47'3"	18"×3/4" 47'3"	18"×3/4" 47'0"	18"×3/4" 47'0"	18"×3/4" 46'10"	18"×3/4" 46'10"	18"×3/4" 46'7"	18"×3/4" 46'7"	18"×3/4" 46'4"	18"×3/4" 46'4"	18"×3/4" 46'2"	18"×3/4" 46'2"
L_2	21"×1" 8'6¼"	21"×1" 8'6¼"	—	—	—	—	—	—	—	—	21"×7/8"	21"×7/8" 6'11½"
L_3	21"×1½" 10'0"	21"×1½" 10'0"	21"×7/8" 18'3"	21"×7/8" 18'3"	21"×7/8" 17'10½"	21"×7/8" 17'10½"	21"×7/8" 17'7¼"	21"×7/8" 17'7¼"	21"×7/8" 17'4"	21"×7/8" 17'4"	21"×1" 10'0"	21"×1" 10'0"
L_4	21"×2" 15'0"	21"×2" 15'0"	21"×1½" 15'0"	21"×1½" 15'0"	21"×1¼" 15'0"	21"×1¼" 15'0"	21"×1¼" 15'0"	21"×1¼" 15'0"	21"×1¼" 15'0"	21"×1¼" 15'0"	21"×1½" 15'0"	21"×1½" 15'0"
L_5	21"×2½" 10'0"	21"×2½" 10'0"	21"×2" 10'0"	21"×2" 10'0"	21"×1¾" 10'0"	21"×1¾" 10'0"	21"×1¾" 10'0"	21"×1¾" 10'0"	21"×1¾" 10'0"	21"×1¾" 10'0"	21"×2" 10'0"	21"×2" 10'0"
L_6	21"×3¼" 15'0"	21"×3¼" 15'0"	21"×2½" 15'0"	21"×2½" 15'0"	21"×2½" 15'0"	21"×2½" 15'0"	21"×2½" 15'0"	21"×2½" 15'0"	21"×2½" 15'0"	21"×2½" 15'0"	21"×2¾" 15'0"	21"×2¾" 15'0"
L_7	21"×4" 12'6"	21"×4" 12'6"	21"×3½" 12'6"	21"×3½" 12'6"	21"×3¼" 12'6"	21"×3¼" 12'6"	21"×3¼" 12'6"	21"×3¼" 12'6"	21"×3¼" 12'6"	21"×3¼" 12'6"	21"×3½" 12'6"	21"×3½" 12'6"
L_8	21"×4" 10'9"	21"×4" 10'9"	21"×3½" 10'9"	21"×3½" 10'9"	21"×3¼" 10'9"	21"×3¼" 10'9"	21"×3¼" 10'9"	21"×3¼" 10'9"	21"×3¼" 10'9"	21"×3¼" 10'9"	21"×3½" 10'9"	21"×3½" 10'9"
L_9	21"×3" 15'0"	21"×3" 15'0"	21"×2¼" 15'0"	21"×2¼" 15'0"	21"×2¼" 15'0"	21"×2¼" 15'0"	21"×2¼" 15'0"	21"×2¼" 15'0"	21"×2¼" 15'0"	21"×2¼" 15'0"	21"×2½" 15'0"	21"×2½" 15'0"
L_{10}	21"×2" 7'6"	21"×2" 7'6"	21"×1½" 7'6"	21"×1½" 7'6"	21"×1½" 7'6"	21"×1½" 7'6"	21"×1½" 7'6"	21"×1½" 7'6"	21"×1½" 7'6"	21"×1½" 7'6"	21"×1½" 7'6"	21"×1½" 7'6"
L_{11}	21"×1" 13'9"	21"×1" 13'9"	21"×7/8" 13'6¾"	21"×7/8" 13'6¾"	21"×7/8" 13'4¼"	21"×7/8" 13'4¼"	21"×7/8" 13'1¾"	21"×7/8" 13'1¾"	21"×7/8" 12'11¼"	21"×7/8" 12'11¼"	21"×7/8" 12'8¾"	21"×7/8" 12'8¾"
L_{12}	18"×3/4" 47'0½"		18"×3/4" 57'6¼"		18"×3/4" 57'3⅛"		18"×3/4" 57'0"		18"×3/4" 56'8⅞"		18"×3/4" 56'5¾"	
L_{13}	18"×1" 21'6		—		—		—		—		—	
L_{14}		18"×3/4" 12'9¼"		18"×3/4" 32'6¼"		18"×3/4" 32'3⅛"		18"×3/4" 32'0"		18"×3/4" 31'8⅞"		18"×3/4" 11'5¾"
L_{15}		18"×1¼" 12'6"		—		—		—		—		18"×1" 20'0"
L_{16}		18"×1½" 7'6"		—		—		—		—		—
L_{17}		18"×1¾" 50'0"		18"×1" 50'0"		18"×1" 50'0"		18"×1" 50'0"		18"×1" 50'0"		18"×1¼" 50'0"

ALL WEBS: 69"×½"

WEBS, FLANGES: GRADE 50W

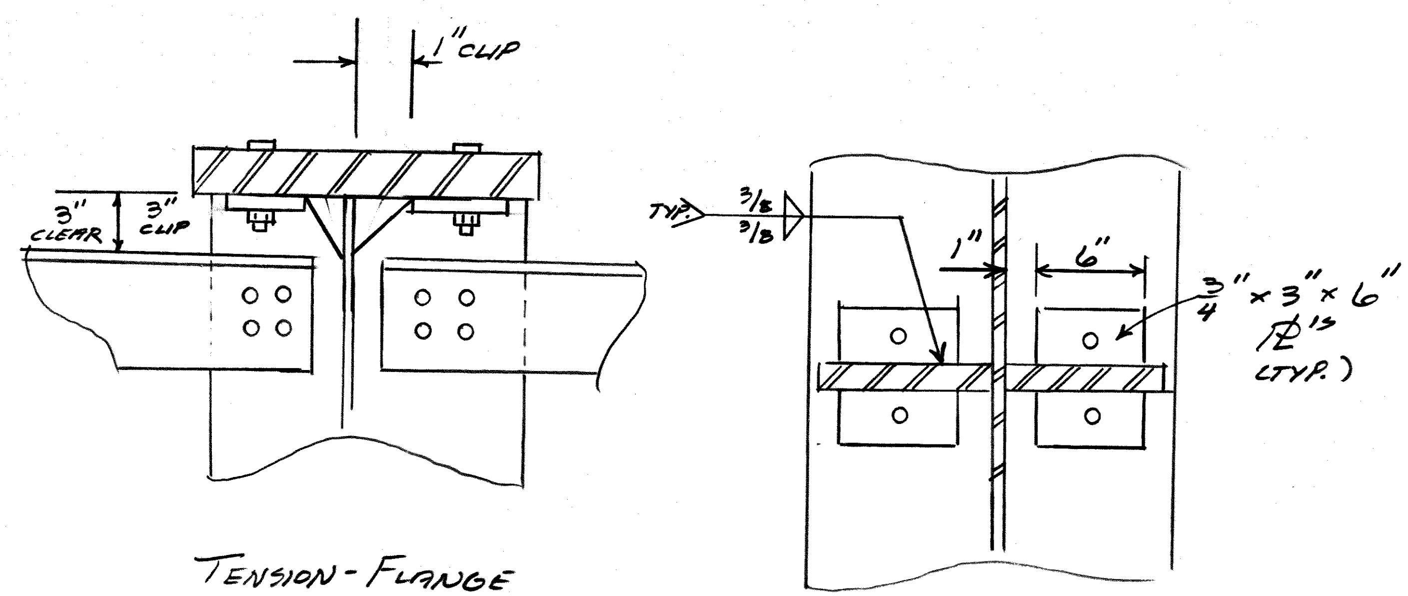
SOUTHBOUND BRIDGE

GIRDER SECTIONS



CROSS-FRAMES AT BENTS

MINIMUM BOLT PRETENSION FOR 7/8" A325 BOLTS IS 39 KIPS



TENSION-FLANGE CONNECTION DETAIL

XSECTION.DGN

REV. 3-10-03

DEFINITION OF TERMS

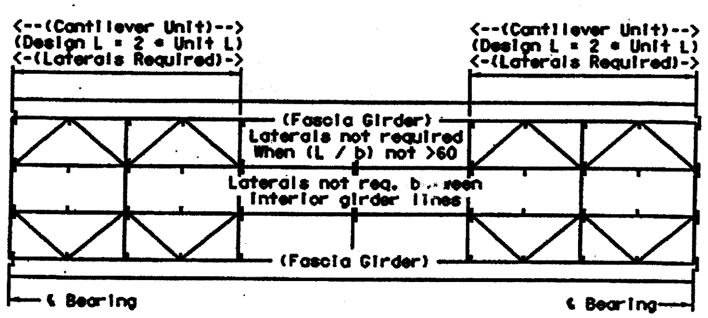
"NO-LOAD" is the "ZERO DEFLECTION" position for the "SHOP CAMBER" shown on the approved shop plans due to: 1.) Total Dead Load
3.) Profile Elevation
4.) Cross Slope Correction

GENERAL NOTES TO DESIGNER

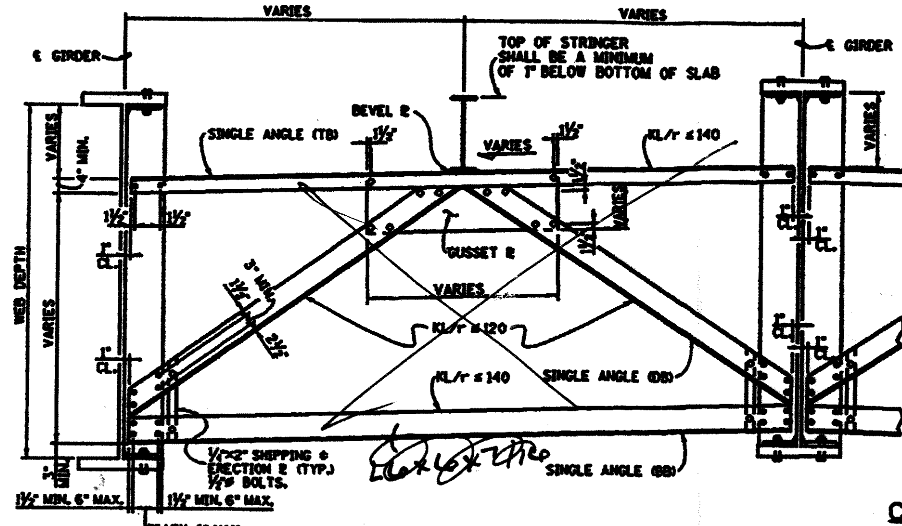
WIDENING OF EXISTING STRUCTURES WITHOUT CLOSURE POURS:
Initial bracing between the new and existing structure for straight or curved girders must be one top and one bottom member with one bolt at each end of each member. The remaining bolts and diagonal frame members must be in place and all bolts torqued to the required plans value after the slab pour.
---> The bracing between girders for multiple girder widening must conform to the fabrication and erection requirements for either a curved girder system or a straight girder system as follows:
2.) WIDENING OF EXISTING STRUCTURE WHEN THE EXISTING DECK IS REMOVED
All transverse frames shall be in place and all bolts torqued to the required plans value. The deflection values of the existing girders without the slab is ambiguous. It would be difficult to obtain proper bar cover without full cross frame installation.

STRAIGHT GIRDER NOTES TO DESIGNER

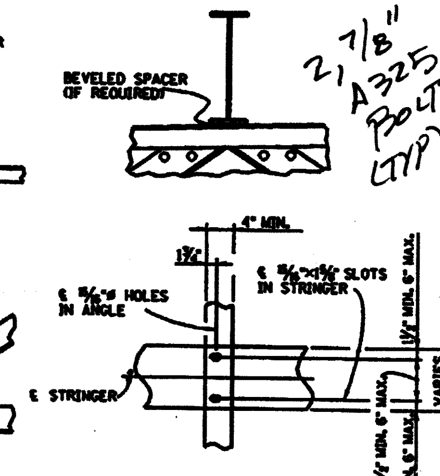
**** Straight girder "DESIGN"
Each girder is designed as an independent single line elastic member. PHASE design and PHASE construction is allowed.
**** Straight girder "LATERAL BRACING" requirements when falsework not provided.
1.) Lateral bracing must be provided in the top flange plane of all open box girder systems...(also known as TUB GIRDERS)
2.) The transverse bracing web stiffener must be tab connected to the top flange and the bottom flange of the box.
3.) Lateral bracing must be provided in the bottom flange plane of both exterior bays for straight girder framing systems when L / b is greater than 60.
Where: L = unbraced length,(in.)
b = flange width,(in.),at the max. stress point in the unbraced range.
*---> Note: lateral bracing in the bottom flange plane will laterally fix the transverse bracing frame. The transverse frame will provide lateral bracing for the top compression flange.
Lateral bracing in both exterior bays is necessary so that the contractor can start the girder placement from either side of the bridge. Lateral bracing is not required in the mid range of the span if adequate bracing is provided in the side ranges so that the unbraced length of the mid span is not greater than 60. See sketch below.
4.) Cantilever erection: L = 2 times the actual cantilever length.
5.) Maximum unbraced compression stress = Fb = 13,960,000 / (L / b)^2 when 60 < L / b < 80. When L / B is > 80 bracing shall be provided.



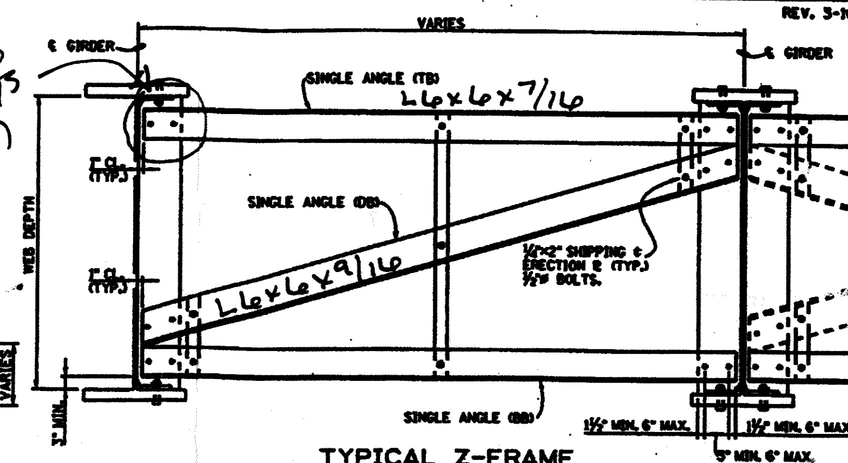
PARTIAL FRAMING PLAN



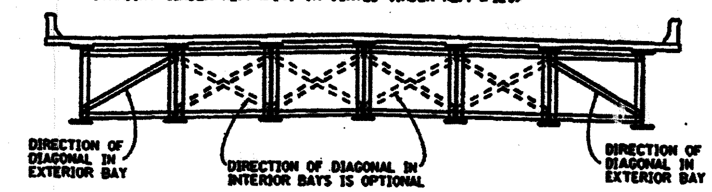
TYPICAL K-FRAME
(GUSSET PLATE MAY BE WELDED IN LIEU OF BOLTED CONNECTION)



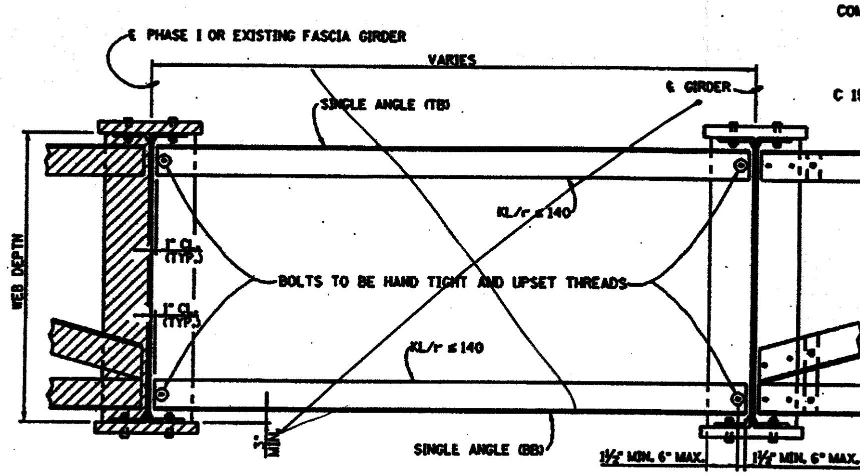
STRINGER TO CROSS-FRAME CONNECTION
(NOTE: 7/8"ø A325 BOLTS TO BE USED)
(TYPICAL ALL LOCATIONS)



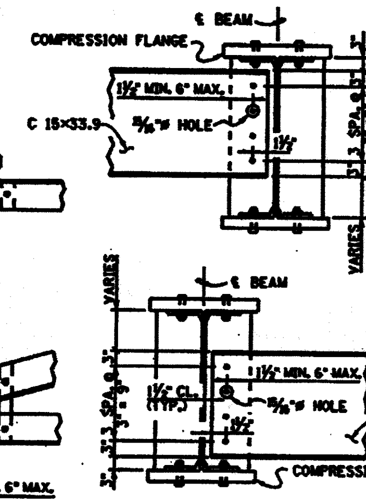
TYPICAL Z-FRAME
(NOT TO BE USED WITH ROLLED BEAMS)
(STRAIGHT GIRDER KL/r ≤ 140 OR CURVED GIRDER KL/r ≤ 120)



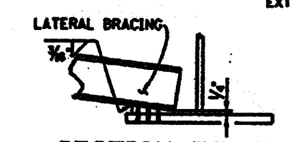
TYPICAL CROSS-SECTION
(SHOWING Z-FRAME DIAGONAL ORIENTATION)



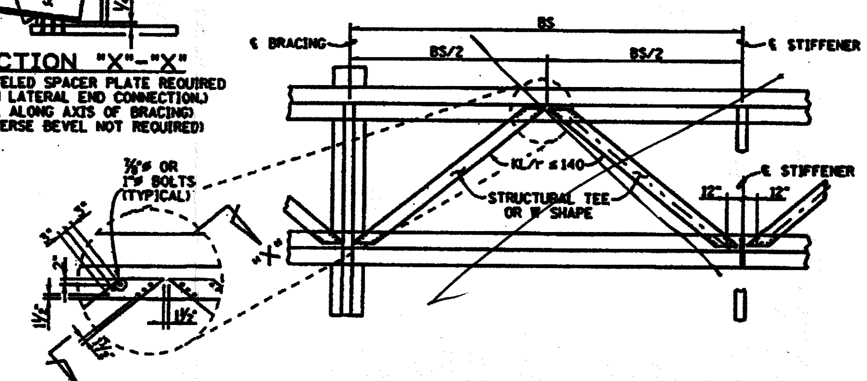
TYPICAL LINE BRACING
(PHASE CONSTRUCTION OR WIDENING OF EXISTING STRUCTURE)



TYPICAL CHANNEL BRACING
(NOTE: 7/8"ø A325 BOLTS TO BE USED)
(BOLTS TO BE HAND TIGHT AND UPSET THREADS)
(1"ø HOLES IN CHANNELS UNLESS OTHERWISE NOTED)
(STIFFENERS SHALL MAINTAIN A 15/16"ø HOLE)



SECTION "X"-"X"
(NOTE: BEVELED SPACER PLATE REQUIRED AT EACH LATERAL END CONNECTION.)
(BEVEL ALONG AXIS OF BRACING)
(TRANSVERSE BEVEL NOT REQUIRED)



LATERAL CONNECTION DETAIL
(NOT TO BE USED WITH CURVED GIRDERS)

CURVED GIRDER NOTES TO DESIGNER

**** Curved girder "GENERAL"
1.) Phase construction should be avoided if possible.
2.) Phase cambers will not be correct. The cross slope of each phase will not match the required cross slope for the full deck width and the closure pour cannot correct this problem.
3.) Phase design and construction is allowed but the results will never be correct.
4.) Phase construction must always have a PHASE-3 closure slab pour.
**** Curved girder "LATERAL BRACING"
1.) The distance from the end bracing at each abutment to the first interior bracing line should not be greater than the beam spacing. Single line lateral bracing must be attached to the top and bottom flanges in the external bays between the end bracing and the first interior bracing...(End bracing is always required)
The distance as measured along the girder from the center line of the lateral member to the transverse bracing center line shall not be greater than 12 inches.
2.) Lateral bracing of I-girder systems at any other location will invalidate lateral flange bending analysis.
3.) Lateral bracing must be provided in the top flange plane of all open box girder systems...(also known as TUB GIRDERS)
4.) The transverse bracing web stiffener must be tab connected to the top flange and the bottom of the box.
**** Curved girder "DESIGN"
1.) Each phase is an independent analysis for dead load and live load.
2.) Live and dead load to be checked for overstress in both PHASE-1 and PHASE-2...(Overstress can occur in either design phase)
3.) All non-composite loads,(including closure slab load for PHASE-3 is to be included in PHASE-1 and PHASE-2 analysis.
4.) Ignore all composite stresses in PHASE-1 and PHASE-2 analysis.
5.) Live load and overlay load to be included in PHASE-3 analysis.
6.) Ignore all non-composite stresses in PHASE-3 analysis.
7.) Final stress on any girder line will be PHASE-1 or PHASE-2 plus the added stress from PHASE-3 for each girder line.

REQUIRED CONTRACT PLANS NOTES

STRAIGHT GIRDER TRANSVERSE BRACING:<--(plans note for single or multiple phase)
Straight girder transverse bracing,(including closure bracing), shall be fabricated for TOTAL CAMBER minus BEAM SELF WEIGHT CAMBER.

CURVED GIRDER TRANSVERSE BRACING:<---(plans note for single or multiple phase)
Transverse bracing shall be fabricated to fit the "NO-LOAD" position for each phase and the closure frame fabricated to match the deflected position of each PHASE.
***** NOTE to Designer: The webs will not be vertical in the "NO-LOAD" position.

STRAIGHT GIRDER ERECTION:|<---(plans note for single phase construction)
or CURVED GIRDER ERECTION:
Beams may have to be crane lifted, jacked, needle beamed, etc. to achieve proper cross slope and hole alignment during transverse bracing installation. The Contractor will not be allowed to ream holes until 50 % of the holes in the pattern at that location are filled with bolts of plans specified size. All transverse bracing shall be in place and all bolts torqued to the plans specified values prior to the slab pour. All cost of transverse bracing complete in place to be included in the lump sum cost of steel structures.
STRAIGHT GIRDER ERECTION:|<---(plans note for multiple phase construction)
or CURVED GIRDER ERECTION:
Beams may have to be crane lifted, jacked, needle beamed, etc. to achieve cross slope and proper hole alignment during PHASE-1 and PHASE-2 transverse bracing installation. The Contractor will not be allowed to ream holes until 50 % of the holes in the pattern at that location are filled with bolts of plans specified size. All transverse bracing in PHASE-1 or PHASE-2 shall be in place and all bolts torqued to the plans specified values prior to the slab pour in that phase. All related cost of transverse bracing installation to be included in the lump sum cost of steel structures.
NOTE to Designer:
The interior fascia girder of PHASE-1 and PHASE-2 may not be aligned after PHASE-1 and PHASE-2 pour. The closure pour may not correct this error due to the composite section of PHASE-1 and PHASE-2.
It may be necessary to field drill the diagonal of the transverse bracing in the bay of the closure pour.

STRAIGHT GIRDER CHANNEL BRACING:|<--(plans note for multiple phase construction)
or CURVED GIRDER CHANNEL BRACING:
Channel bracing for PHASE-3 closure pour to be connected during PHASE-2 beam erection. The center bolt shall be installed hand tight at each end of the channel. After the deck slab of PHASE-2 has been poured, the Contractor shall install the remaining bolts and torque all bolts to the required plans value.
NOTE to Designer: Use channel bracing when the diagonal rise to run is less than 1 to 4 or when the beam is a rolled beam. All holes in the channel shall be the standard "1/16 inch oversize".
STRAIGHT GIRDER Z-FRAME BRACING:|<--(plans note for multiple phase construction)
or CURVED GIRDER Z-FRAME BRACING:
The top and bottom members for transverse bracing in the PHASE-3 closure bay to be connected during PHASE-2 beam erection, one bolt shall be installed hand tight at each end of each top and bottom member. Bolts shall not be installed in the diagonal member at this time. After the deck slab of PHASE-2 has been poured, the Contractor shall install the remaining frame bolts and torque all bolts to the required plans value.
NOTE to Designer: Z-Frame bracing when the rise to run is equal to or greater than 1 to 4.
STRAIGHT GIRDER K-FRAME BRACING:|<--(plans note for multiple phase construction)
or CURVED GIRDER K-FRAME BRACING:
The top and bottom member for transverse bracing in the PHASE-3 closure bay to be connected during PHASE-2 beam erection. One bolt shall be installed hand tight at each end of each top and bottom member. Bolts shall not be installed in the diagonal member at this time. After the deck slab of PHASE-2 has been poured, the Contractor shall install the remaining frame bolts and torque all bolts to the required plans value.
NOTES to Designer:
1.) K-Frames should only be used for Girder-Stringer systems.
2.) K-Frames should be single angle members to accommodate erection.
3.) The bottom member of the K-Frame must float on hanger straps
*****> Caution: Girder-Stringer systems shall not be used with curved girders.

GENERAL NOTES

NOTE: ALL BOLTS SHALL BE EITHER 7/8"ø OR 1"ø AND MAY BE EITHER A325 OR A490. SIZE AND TYPE DETERMINED BY DESIGN.
NOTE: LOAD INDICATOR WASHERS SHALL NOT BE USED.
NOTE: ALL BOLTS SHALL BE TORQUED TO AASHTO SPECIFICATIONS.

STATE OF TENNESSEE
DEPARTMENT OF TRANSPORTATION
CROSS-FRAME
AND
LATERAL CONNECTION
DETAILS
INTERMEDIATE CROSS-FRAMES

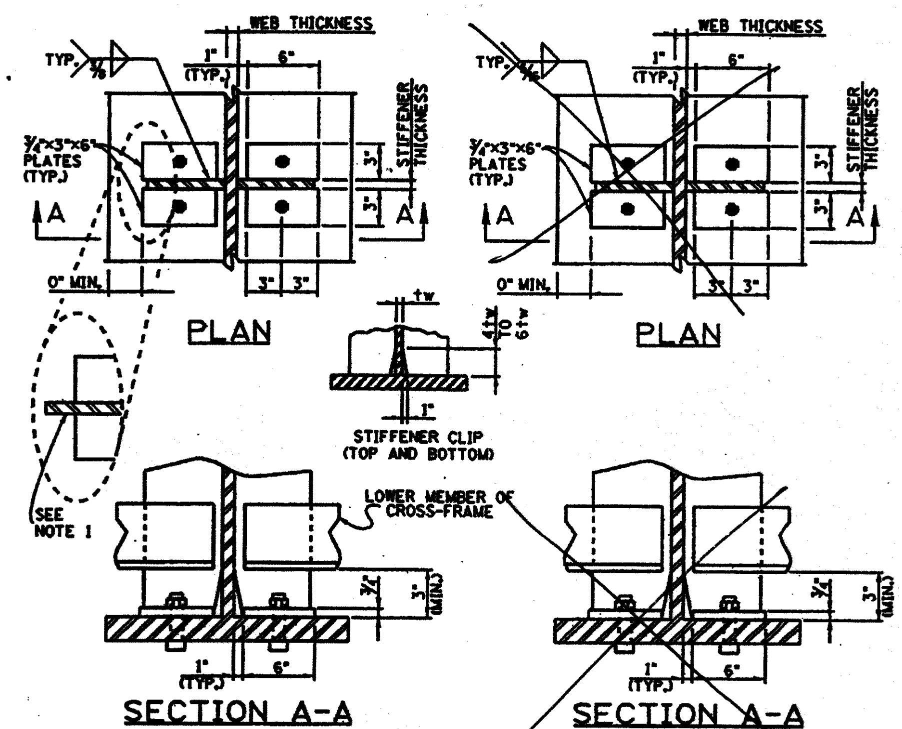
CORRECT Edward P. Wasserman
ENGINEER OF STRUCTURES

DESIGNED BY C.M. HILES DATE 3-03
DRAWN BY K.L. FRANKENFIELD DATE 3-03

4/ /2003

STANDARD CONNECTION DETAILS

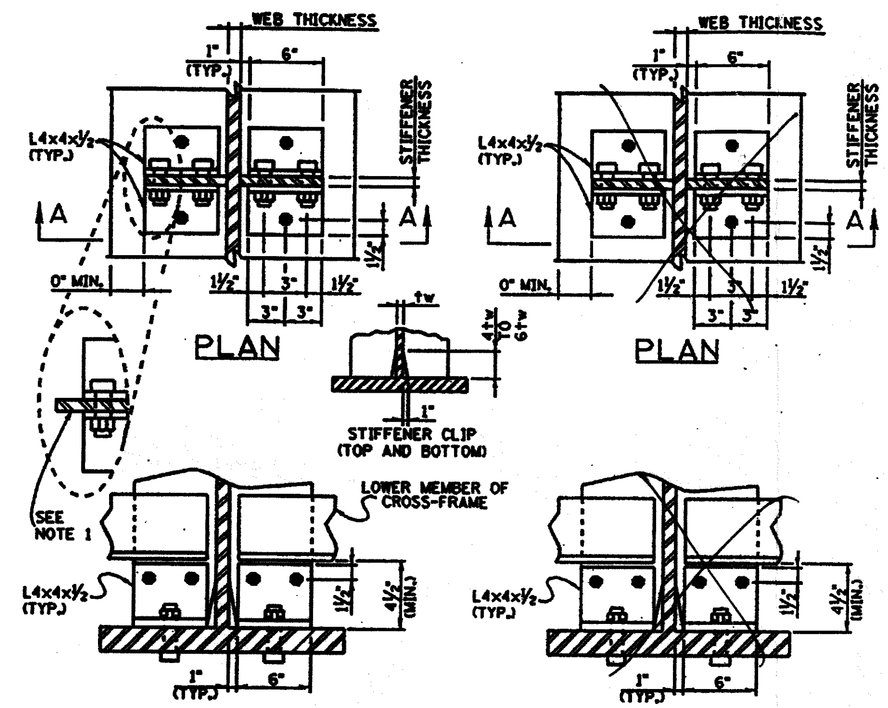
TENSION FLANGE CONNECTION AT CROSS-FRAME LOCATIONS



1 NOTE: TAB YIELD STRENGTH SHALL MATCH STIFFENER YIELD STRENGTH. TAB SIZE AND LOCATION SHALL BE AS SHOWN REGARDLESS OF STIFFENER WIDTH.
2 NOTE: STIFFENER MAY BE WELDED DIRECTLY TO COMPRESSION FLANGE UTILIZING ⅜" FILLET WELDS FOR CURVED GIRDERS OR 5⁄16" FILLET WELDS FOR STRAIGHT GIRDERS APPLIED FULL WIDTH BOTH SIDES OF STIFFENER.
3 NOTE: ⅞"ø - A325 BOLTS TO BE TORQUED TO AASHTO SPECIFICATION FOR FRICTION CONNECTION PRIOR TO TAB TO STIFFENER WELD FOR STRAIGHT GIRDERS.
4 NOTE: 1"ø - A490 BOLTS TO BE TORQUED TO AASHTO SPECIFICATION FOR FRICTION CONNECTION PRIOR TO TAB TO STIFFENER WELD FOR CURVED GIRDERS.
5 NOTE: WEB STIFFENER FIT SHALL BE WITHIN AWS D1.1 FABRICATION TOLERANCE.
6 NOTE: IF PROJECTING FLANGE WIDTH IS LESS THAN 7", A SPECIAL DETAIL WILL BE REQUIRED.
7 NOTE: THE FABRICATOR MAY SUBSTITUTE A SINGLE ¾" PLATE PROVIDED: THE LOCATION OF BOLTS AND THE LENGTH PARALLEL TO THE STIFFENER DOES NOT CHANGE. THE PLATE LENGTH PARALLEL TO THE WEB PLATE MUST BE 6" PLUS THE STIFFENER PLATE THICKNESS.
8 NOTE: TENSION FLANGE TAB PLATES NOT APPLICABLE @ BEARING STIFFENER CROSS-FRAMES PARALLELING ℄ OF BEARING.
9 NOTE: IF ROLLED BEAM'S $K_1 - tw/2$ IS GREATER THAN 1" THEN TAB GRINDING IS REQUIRED.

ALTERNATE CONNECTION DETAILS

TENSION FLANGE CONNECTION AT CROSS-FRAME LOCATIONS



1 NOTE: ANGLE YIELD STRENGTH SHALL NOT BE LESS THAN 36 KSI. ANGLE SIZE AND LOCATION SHALL BE AS SHOWN REGARDLESS OF STIFFENER WIDTH.
2 NOTE: ⅞"ø - A325 BOLTS TO BE TORQUED TO AASHTO SPECIFICATION FOR FRICTION CONNECTION FOR STRAIGHT GIRDERS.
3 NOTE: 1"ø - A490 BOLTS TO BE TORQUED TO AASHTO SPECIFICATION FOR FRICTION CONNECTION FOR CURVED GIRDERS.
4 NOTE: WEB STIFFENER FIT SHALL BE WITHIN AWS D1.1 FABRICATION TOLERANCE.
5 NOTE: IF PROJECTING FLANGE WIDTH IS LESS THAN 7", A SPECIAL DETAIL WILL BE REQUIRED.
6 NOTE: TENSION FLANGE TAB PLATES NOT APPLICABLE @ BEARING STIFFENER CROSS-FRAMES PARALLELING ℄ OF BEARING.
7 NOTE: IF ROLLED BEAM'S $K_1 - tw/2$ IS GREATER THAN 1" THEN ANGLE GRINDING IS REQUIRED.

STATE OF TENNESSEE
DEPARTMENT OF TRANSPORTATION
GIRDER CONNECTION DETAILS

CORRECT Edward P. Wasserman
ENGINEER OF STRUCTURES

DESIGNED BY C.M. HILES DATE 3-03
DRAWN BY K.L. FRANKENFIELD DATE 3-03