ATTACHMENT III
STEEL SET MEMBER AND COMPONENT DESIGN

filename: atch3.wp5

96Ø919Ø1Ø8 - Part 3

PURPOSE III.A STEEL SET CALCULATION

NOTE: REFER TO ATTACHMENT IX, PAGE IX-3 & IX-5 FOR STEEL SET SKETCHES.

CRITICAL STRESS COMBINATIONS
{LOOK @ WORST COMBINATIONS OF HIGH AXIAL COMPRESSION & HIGH MOMENTS. NOTE: AXIAL COMPRESSION GIVES GREATEST CONTRIBUTION TO INTERACTION EQUATIONS OF AISC CHAPTER H}

FILE: P10K2dy - WITH SEISMIC LOAD STEEL SETS @ 2'-0

W8×31

F-axial N	MOMENT N-M	ELEM. NO.
1,324,000	15,050	36
1,282,000	16,320	29
1,349,000	12,430	12
KIPS ($*2.2481\times10^{-4}$)	Ft-K ($.73756\times10^{-3}$)	
297.65	11.10	
288.21	12.04	
303.27	9.17	

a) MULTIPLY FORCE IN (N) by $2.2481*10^{-4}$ TO obtain FORCE IN KIPS.

b) MULTIPLY MOMENTS IN (N-M) by $0.73756*10^{-3}$ TO OBTAIN MOMENTS IN FT-K.

THE STEEL SETS IN THIS RUN ARE AT 2'-0 c/c * .3048 m/ft = .61 M.

MULTIPLY STRESSES IN PREVIOUS TABLES - given for one METER! by .61 TO OBTAIN STRESSES PER ONE STEEL SET

F-axial K/STEEL SET	MOMENT FT-K/STEEL SET	CASE
181.57	6.77	1
175.81	7.34	2
185.00	5.60	3

CHECK INTERACTION COEFFICIENT FOR EACH OF THE ABOVE.

UNBRACED LENGTH — ALLOWABLE STRESS

L_x = DISTANCE BET'N BLOCKING (SUPPORT) = 4 ELEMENTS ≅ 75" < L_Y -- NOT GOVERN.

$L_Y = \pi\left(12.42' - \frac{45''}{12}\right)\left(\frac{35°}{360°}\right)(2) = 7.36' = 88.29''$ --- TIE ROD SPACING

Length used is a conservative assurance against buckling. W-shape used when fully loaded is braced by contact with the tunnel wall. (BABEAB000-01717-2100-41101, Rev 03)

$F_a = 18.89$ ksi - (AISC 3-16, TABLE C-36)

$$\frac{KL_y}{r_y} = \frac{1.0 \times 88.29}{2.02} = 43.71$$

$L_c = 8.4'$ --- AISC PAGE 2-12, $L_y = 7.36' < L_c \Rightarrow F_{bx} = 0.66 F_y = 24^{ksi}$

$F_b = 24.0$ ksi

@ ELEM. 36 & 29, $K\ell_b/r_b = \frac{(1.0)(18.66)(12)}{3.47} = 64.5 \Rightarrow F_e' = 35.9$

@ ELEM 12, $K\ell_b/r_b = \frac{(1.0)(17.54)(12)}{3.47} = 60.7 \Rightarrow F_e' = 40.54$

$F_e' = 35.9$ --- AISC PAGE 5-122, TABLE 8

Case 3 AISC 5-54 H1-1

$$f_a = \frac{185}{9.13} = 20.26 \text{ ksi}$$

$$f_b = \frac{5.60 \times 12}{27.5} = 2.44$$

$$\frac{20.26}{18.89} + \frac{2.44}{\left(1 - \frac{20.26}{40.54}\right) \times 24} =$$

$$1.073 + .203 = 1.276 < 1.33 \quad OK$$

CASE 2

$$f_a = \frac{175.81}{9.13} = \underline{19.26} \text{ ksi}$$

$$f_b = \frac{7.34 \times 12}{27.5} = \underline{3.2} \text{ ksi}$$

$$\frac{19.26}{18.89} + \frac{3.2}{\left(1 - \frac{19.26}{35.9}\right) \times 24.0}$$

$$1.02 + .288 = \underline{1.31 < 1.33} \quad \text{OK}$$

CASE 1

$$f_a = \frac{181.57}{9.13} = \underline{19.89} \text{ ksi}$$

$$f_b = \frac{6.77 \times 12}{27.5} = \underline{2.95} \text{ ksi}$$

$$\frac{19.89}{18.89} + \frac{2.95}{\left[1 - \left(\frac{19.89}{35.9}\right)\right] \times 24.0} = 1.053 + 0.276 = \underline{1.328}$$

$$\underline{\leq 1.33}$$

FILE: P34K2dy. W8x31 @ 4'-0
W/SEISMIC.

CASE	Faxial (N)	M (N-M)	ELEM. No
1	603,100	10,110	11
2	869,000	1,849	33

	Faxial (K)	M (K-M)
	4' SPACING x 0.3048 m/ft = 1.22	
1	135.58 * 1.22 = 165.41	7.46 * 1.22 = 9.10
2	195.36 * 1.22 = 238.34	1.36 * 1.22 = 1.66

	f_a	f_b	$C = \frac{f_a}{F_a} + \frac{f_b}{\left(1 - \frac{f_a}{F'_e}\right) F_b}$ *
1	18.117	3.971	.96 + .242 = 1.202
2	26.11	.724	1.382 + .053 = 1.433

1.4 > 1.33 W8x31 NOT GOOD
@ 4'-0 IN THIS AREA

RERUN FOR 2'-0 c/c.

* @ ELEM 11: $\frac{K l_b}{r_b} = \frac{(1.0)(12.91)(12)}{3.47} = 44.6$, $F'_e = 75.09$

@ ELEM 33: $\frac{K l_b}{r_b} = \frac{(1.0)(13.23)(12)}{3.47} = 45.8$, $F'_e = 71.20$

NEW FILE ; p34k2dyx   W 8x31 @ 2'-0
W/ SEISMIC.

CASE	Faxial (N)	M (N-M)	ELEM. №
1	1,446,000	4,630	32
2	1,125,000	7,202	30
3	1,137,000	12,270	11

	Faxial (K)	M (Ft-K)
1	325.08 * .61 = 198.30	3.415 * .61 = 2.083
2	252.91 * .61 = 154.28	5.312 * .61 = 3.24
3	255.61 * .61 = 155.92	9.050 * .61 = 5.52

	f_a	f_b	$C = \frac{f_a}{F_a} + \frac{f_b}{\left(1 - \frac{f_a}{F'_e}\right) F_b}$, L_*
1	21.72	0.909	1.150 + .06 = 1.210 < 1.33
2	16.90	1.414	0.89 + .086 = .980 < 1.33
3	17.08	2.409	0.904 + 0.144 = 1.048 < 1.33

W8 x 31 - OK FOR 2'-0 c/c

* ELEM 32 & 30 $kl_b/r_b = 45.8$, $F'_e = 71.2$
ELEM 11 , $F'_e = 75.1$

FILE : P07K2dy. W8x31 @ 4'-0 W/SEISMIC.

	Faxial (N)	M (N-M)	ELEM. No
1	341,600	1,277	33
2	344,900	949	9
3	280,000	1,669	6
4	282,500	6,124	11

	Faxial (k)	M (Ft-k)
1	76.80 * 1.22 = 93.70	.942 * 1.22 = 1.15
2	77.54 * 1.22 = 94.60	.700 * 1.22 = .854
3	62.95 * 1.22 = 76.80	1.231 * 1.22 = 1.50
4	63.51 * 1.22 = 77.48	4.517 * 1.22 = 5.51

	fa	fb	C
1	10.26	0.50	.543 + .028 = .571
2	10.36	0.37	.548 + .018 = .566
3	8.41	0.65	.445 + .032 = .477
4	8.49	2.40	0.449 + 0.122 = 0.571

all < 1.33

F_e' ; @ ELEM 33 $Kl_b/r_b = \frac{13.30 \times 12}{3.47} = 46.0$, $F_e' = 70.6$

@ ELEM 6, 9 & 11 $Kl_b/r_b = \frac{8.96 \times 12}{3.47} = 30.3$, $F_e' = 162.8$

FILE: P18K2dy - SEISMIC W8x31 @ 4'-0

CASE	Faxial (N)	M (N-M)	ELEM. No
1	557,700	3,650	36
2	776,100	2,113	33
3	587,500	9,528	11
4	706,000	3,308	10

@ ELEM. 36 & 33 : $K\ell_b/r_b = \frac{(1.0)(10.12)(12)}{3.47} = 35.0$, $F_e' = 121.9$

ELEM 11 & 10 : $K\ell_b/r_b = \frac{(1.0)(6.07)(12)}{3.47} = 21.0$, $F_e' = 338.6$

	Faxial (K)	M (Ft-K)
1	125.38 × 1.22 = 152.96	2.69 × 1.22 = 3.28
2	174.48 × " = 212.86	1.56 " = 1.90
3	132.08 × " = 161.14	7.03 " = 8.58
4	158.72 × " = 193.64	2.44 " = 2.98

CASE 2

$$f_a = \frac{212.86}{9.13} = 23.31 \; ; \; f_b = \frac{1.9 \times 12}{27.5} = .83$$

$$\frac{23.31}{18.89} + \frac{.83}{\left(1 - \frac{23.31}{121.9}\right) 21.6} = 1.23 + .048$$

$$= \underline{1.278 < 1.33}$$

CASE 3

$$f_a = \frac{161.13}{9.13} = 17.65 \; ; \; f_b = \frac{8.58 \times 12}{27.5} = 3.74$$

$$\frac{17.65}{18.89} + \frac{3.74}{\left(1 - \frac{17.65}{338.6}\right) 21.6} = .934 + .183$$

$$= \underline{1.117 < 1.33}$$

OTHER CASES ARE LESS CRITICAL

BY INSPECTION -

FILE m34k250x. W8x31 @ 2'
NO SEISMIC.

CASE	Faxial (N)	M (N-M)	ELEM. NO
1	1,163,000	3,805	32
2	897,800	10,770	11
3	1,048,000	6,292	10

	Faxial (K)	M (Ft-K)
1.	261.45 * .61 = 159.48	2.81 * .61 = 1.72
2.	201.83 * .61 = 123.12	7.94 * .61 = 4.84
3.	235.60 * .61 = 143.7	4.64 * .61 = 2.83

F_e' : @ ELEM 32, L = 13.27 Kl_b/r_b = 45.9, F_e' = 70.9
@ ELEM 10, 11 L = 9.86 Kl_b/r_b = 34.1, F_e' = 128.5

	fa	fb	C
1	17.47	.75	.925 + .046 = .971 < 1
3	13.49	2.11	.714 + .109 = .823 < 1
4	15.74	1.235	.833 + .065 = .898 < 1

OK

FILE M10K250W. W8×31 @ 2'-0

NO SEISMIC

CASE	Faxial (N)	M (N-M).	ELEM NO
1	922,000	12,840	29
2	868,100	18,060	11

	Faxial (K)	M (Ft-K).
1	207.27 * .61 = 126.43	9.47 * .61 = 5.78
2	195.16 * .61 = 119.05	13.32 * .61 = 8.13

	f_a	f_b	C
1	13.85	2.52	0.733 + 0.176 = .909
2	13.04	3.55	0.690 + 0.221 = .911

all < 1.00 OK.

F_e': @ ELEM 29, $\frac{K l_b}{r_b} = \frac{(1.0)(17.48)(12)}{3.47} = 60.4$, $F_e' = 40.94$

@ ELEM 11, $\frac{K l_b}{r_b} = \frac{(1.0)(15.70)(12)}{3.47} = 54.3$, $F_e' = 50.66$

FILE: M 18 K 250 W8x31 4'-0 c/c NO SEISMIC

CASE	F axial (N)	M. (N-M)	ELEM No
1	522,500	3,371	10
2	416,600	8,389	11
3	583,300	1,492	8

CASE	F (K)	M (Ft-K)
1	117.46 * 1.22 = 143.3	2.49 * 1.22 = 3.04
2	93.66 * 1.22 = 114.3	6.19 * 1.22 = 7.55
3	131.13 * 1.22 = 160.0	1.10 * 1.22 = 1.34

CASE	$f_a = \frac{F}{A}$	$f_b = \frac{M}{S}$	$C = \frac{f_a}{F_a} + \frac{f_b}{\left(1 - \frac{f_a}{F'_e}\right) F_b}$
	$Kl/r = 8.74 \times 12 / 3.47 = 30.2$,	$F'_e = 163.8$	
1	15.696	1.33	.831 + .068 = .899
2	12.515	3.29	.662 + .165 = .827
3	17.525	.59	.928 + .031 = .959
			< 1. OK

FILE M07K250W W8x31 @ 4'-0

NO SEISMIC

	Faxial (N)	M (N-M)	ELEM. No
1	212,500	720	33
2	163,800	5,419	11
3	196,500	3,808	10

	Faxial (K)	M (Fz-K)
1	47.77 * 1.22 = 58.28	.53 * 1.22 = .65
2	36.82 * 1.22 = 44.92	4.00 * 1.22 = 4.88
3	44.18 * 1.22 = 53.90	2.81 * 1.22 = 3.43

	f_a	f_b	C
1	6.38	.28	.338 + .014 = .352
2	4.92	2.13	.260 + .100 = .360
3	5.90	1.50	.312 + .071 = .383

F_e' : @ ELEM 33, $Kl/r = \frac{13.37(12)}{3.47} = 46.3$, $F_e' = 69.7$ ALL < 1.0

@ ELEM 10 & 11, $\frac{Kl}{r} = \frac{6.62(12)}{3.47} = 21.6$, $F_e' = 320.6$

FILE M18_K2dy W6x20 @ 4'-0

w/SEISMIC

CASE	Faxial (N)	M (N-M)	ELEM. No
1	170,500	4,049	12
2	153,400	5,926	11
3	271,500	724	9

	Faxial (K)	M (Ft-K)
1	38.33 * 1.22 = 46.76	2.99 * 1.22 = 3.65
2	34.49 * 1.22 = 42.08	4.37 * 1.22 = 5.33
3	61.04 * 1.22 = 74.47	.534 * 1.22 = .652

W6x20 $A = 5.87 \text{ in}^2$ $S = 13.4 \text{ in}^3$ $\frac{K\ell_y}{r_y} = \frac{88.29}{1.46} = 58.9$

$F_a = 17.54$, $\frac{K\ell_b}{r_b} = \frac{7.54 \times 12}{2.66} = 34.0$, $F'_e = 129.18$ $L_c = 6.4'$

	fa	fb	C
1	7.96	3.27	.454 + .161 = .615
2	7.17	4.77	.409 + .234 = .643
3	12.69	.584	.723 + .044 = .767

ALL < 1.33 OK

FILE : M07- K2 dy   W6x20 @ 4-0

W/SEISMIC

CASE	Faxial (N)	M (N-M)	ELEM No
1	121,300	197	33
2	95,300	5,490	11
3	118,900	427	8

	Faxial (K)	M (Ft-K)
1	27.27 × 1.22 = 33.27	.15 × 1.22 = .183
2	21.42 × 1.22 = 26.13	4.05 × 1.22 = 4.94
3	26.73 × 1.22 = 32.61	.32 × 1.22 = .39

	fa	fb	C
1	5.67	.164	.323 + .009 = .332
2	4.45	4.42	.254 + .212 = .466
3	5.56	.135	.317 + .017 = .334

ALL < 1.0 ... O.K.

F_e' : @ ELEM 33 $Kl_b/r_b = \frac{12.68(12)}{2.66} = 57.2$, $F_e' = 45.6$

@ ELEM 11 $Kl_b/r_b = \frac{7.36(12)}{2.66} = 33.2$, $F_e' = 135.5$

@ ELEM 8 $Kl_b/r_b = \frac{7.33 \times 12}{2.66} = 33.0$, $F_e = 137.13$

FILE M27_kzdy W6x20 @ 4'-0
W/ SEISMIC.

Case	Faxial (N)	M(N·M)	ELEM. No
1	202,400	888	35
2	270,500	436	33
3	268,100	613	8

	Faxial (K)	M (Ft-K)
1	45.50 * 1.22 = 55.51	.655 * 1.22 = .80
2	60.81 * 1.22 = 74.19	.336 * 1.22 = .41
3	60.27 * 1.22 = 73.53	.452 * 1.22 = .55

	fa	fb	C
1	9.46	.716	.539 + .043 = .582
2	12.64	.367	.721 + .021 = .745
3	12.53	.493	.714 + .025 = .739

ALL < 1.33 --- O.K.

F_e' : @ ELEM 33 & 35 , $K\ell_b/r_b = \frac{13.07 \times (12)}{2.66} = 59.0$, $F_e' = 42.9$

@ ELEM 8 , $K\ell_b/r_b = \frac{7.61 \times 12}{2.66} = 34.3$, $F_e' = 127.0$

FILE M34-C2K2d. W6×20 @ 4'-0
WITH SEISMIC.

CASE	FAXIAL (N)	M (N-M)	ELEM.
1	303,000	2,308	36
2	388,900	782	33
3	320,800	3228	12

	FAXIAL (K)	M (FT-K)
1	68.12 * 1.22 = 83.11	1.70 * 1.22 = 2.074
2	87.43 * 1.22 = 106.66	0.58 * 1.22 = 0.708
3	72.13 * 1.22 = 88.00	2.38 * 1.22 = 2.904

	f_a	f_b	C
1	14.16	1.86	0.807 + 0.128 = 0.94
2	18.17	0.63	1.036 + 0.051 = $\underline{\underline{1.087}}$
3	15.00	2.60	0.855 + 0.123 = 0.978

ALL ARE < 1.33 ∴ OK

FROM PAGE III-17 (SIMILAR)

@ NODES 33 & 36 : $F_e' = 42.9$ @ $l_x = 13.1'$

@ NODE 12 : $F_e' = 127.0$ @ $l_x = 7.54'$

FILE: M53_K2dy W6×20 @ 4'-0 W/ SEISMIC

CASE	Faxial (N)	M (N-M)	ELEM. No
1	202,400	5,463	11
2	245,200	3,614	10
3	271,200	552	9

	Faxial (K)	M (Ft-K)
1	45.50 × 1.22 = 55.51	4.03 × 1.22 = 4.92
2	55.12 × 1.22 = 67.25	2.67 × 1.22 = 3.26
3	60.97 × 1.22 = 74.38	.407 × 1.22 = .497

	f_a	f_b	C
1	9.46	4.41	0.593 + 0.217 = 0.756
2	11.46	2.92	0.653 + 0.146 = 0.800
3	12.67	.445	0.722 + 0.022 = 0.744

ALL < 1.33 --- O.K.

F_e' : @ ELEM. 9, 10 & 11 $Kl_b/r_b = \frac{6.89\,(12)}{2.66} = 31.1$, $F_e' = 154.4$

FILE M18_K2. W6x20 @ 4'-0

NO SEISMIC

	Faxial (N)	M (N-M)	ELEM. No
1	153,500	3,602	10
2	198,800	782	9

	Faxial (K)	M (Ft-K)
1	34.51 * 1.22 = 42.10	2.66 * 1.22 = 3.25
2	44.69 * 1.22 = 54.52	.58 * 1.22 = .71

	fa	fb	C
1	7.17	2.91	.409 + .143 = .552
2	9.29	.64	.530 + .032 = .562

ALL < 1.0 --- O.K.

F_e' : $K l_b / r_b = \frac{(7.52)(12)}{2.66} = 34$; $F_e' = 129.2$

FILE m07_K2. W6x20 @ 4'-0

NO SEISMIC

CASE	Faxial (N)	M (N-M)	ELEM. No
1	49,250	5,390	12
2	63,010	3,575	10
3	67,980	783	9

	Faxial (K)	M (Ft-K)
1	11.07 * 1.22 = 13.51	3.98 * 1.22 = 4.86
2	14.17 * 1.22 = 17.29	2.64 * 1.22 = 3.22
3	15.28 * 1.22 = 18.64	.578 * 1.22 = .705

	fa	fb	C
1	2.30	4.35	.131 + .205 = .336
2	2.95	2.88	.168 + .136 = .304
3	3.18	.63	.181 + .030 = .211

F_e' : $K l_b / r_b = \frac{7.5 \times 12}{2.66} = 34$, $F_e' = 129.2$

ALL < 1.0 ---- O.K.

FILE M27_K2 W6x20 @ 4'-0

NO SEISMIC

CASE	Faxial (N)	M (N-M)	ELEM NO
1	215,000	404	33
2	159,300	4,962	12
3	158,200	5,396	11
4	188,100	3,688	10

	Faxial (K)	M (Ft-K)
1	48.33 * 1.22 = 58.96	0.30 * 1.22 = .366
2	35.81 * 1.22 = 43.69	3.66 * 1.22 = 4.46
3	35.56 * 1.22 = 43.38	3.98 * 1.22 = 4.86
4	42.29 * 1.22 = 51.59	2.72 * 1.22 = 3.32

F_e' : @ ELEM 33, $kl_b/r_b = 13.20 \times 12 / 2.66 = 60$, $F_e' = 41.5$

@ ELEM 10,12, $kl_b/r_b = 32$, $F_e' = 145.8$

	fa	fb	C
1	10.04	.328	.572 + .020 = .592 < 1 OK
2	7.44	3.99	.424 + .195 = .619 < 1
3	7.39	4.35	.421 + .212 = .633 < 1
4	8.79	2.97	.501 + .146 = .647 < 1 OK

FILE M 34 - K2. W6x20 @ 4'-0
-NO SEISMIC.

CASE	Faxial (N)	M (N-M)	ELEM. No
1	181,300	4,672	12
2	170,900	5,453	11
3	201,000	3,632	10
4	226,100	533	9

	Faxial (K)	M (Ft-K)
1	40.76 * 1.22 = 49.74	3.45 * 1.22 = 4.21
2	38.42 * 1.22 = 46.87	4.02 * 1.22 = 4.90
3	45.19 * 1.22 = 55.13	2.68 * 1.22 = 3.27
4	50.83 * 1.22 = 62.01	.415 * 1.22 = 0.506

F_e' : $Kl_b/r_b = 32$, $F_e' = 145.8$

	f_a	f_b	C
1	8.47	3.77	.483 + .185 = .668 < 1
2	7.98	4.39	.455 + .215 = .670 < 1
3	9.39	2.93	.535 + .145 = .68 < 1
4	10.56	.453	.602 + .023 = .625 < 1

DR

FILE MSBK2. W6x20 @ 4'-0
NO SEISMIC

CASE	Faxial (N)	M (N-M)	ELEM. No
1	163,300	4,888	12
2	160,800	5,458	11
3	196,500	3,627	10

	Faxial (K)	M (Ft-K)
1	36.71 * 1.22 = 44.79	3.61 * 1.22 = 4.40
2	36.15 * 1.22 = 44.10	4.03 * 1.22 = 4.92
3	44.18 * 1.22 = 53.90	2.68 * 1.22 = 3.27

	fa	fb	C
1	7.63	3.94	.435 + 0.192 = .627
2	7.51	4.41	.428 + .215 = .643
3	9.18	2.93	.523 + .145 = .668

all < 1.0 OK

F'_e : $Kl_b/r_b = 32$, $F'_e = 145.8$

STEEL SET CALCULATION FOR
DETERMINING JACK SIZE.

CHECK W8x31 FOR 30T JACK.

FROM COMPUTER OUTPUT FILE: STLRV2
LOAD COMBINATION 6 CORRESPONDS
TO 30T JACKING LOAD -
(SEE ATTACHMENT I, PAGE I-29 THRU I-34)

COMPUTER INTERACTION COEFF. FROM AISC
CODE CHECK = 0.892 @ MEMBERS 4, 41, 42 AND
= 0.893 @ MEMBER 3

MEMBER	P_{max}	M_{max}	
3	55.08	33.42	GOVERNS
4	54.91	33.42	
41	54.95	33.38	
42	55.10	33.38	

MEMBER 3 IS CRITICAL

$P = 55.08$ $\qquad M = 33.42$

$f_a = \dfrac{55.08}{9.13} = \underline{6.03}^{ksi}$ $\qquad f_b = \dfrac{33.42 \times 12}{27.5} = \underline{14.58}^{ksi}$

UNBRACED LENGTHS

$L_y = 88.29'' = 3.5^{\circ}$ TIE ROD SPACING

$L_x \simeq 3.5 * 18.79'' = 65.77''$

(FROM MOMENT DIAGRAM, PAGE I-43)
FOR LOADCOMB. No-7 - ASSUME SIMILAR FOR L.C. No 6

ALLOWABLE STRESSES

TABLE C-36 AISC P. 3-16

$\frac{K L_y}{r_y} = \frac{88.29}{2.02} = 43.71$ $\quad F_a = \underline{18.89^{KSI}}$

$\frac{K l_x}{r_x} = \frac{65.77}{3.47} = 18.95$ $\quad F'_{e_x} = 338.62$

AISC 5-45 F1-2

$L_c = \frac{76\, b_f}{\sqrt{F_y}} = \frac{76 \times 8}{6} = \underline{101.33''}$ GOVERNS

$L_c = \frac{20{,}000}{\frac{d}{A_f} F_y} = \frac{20{,}000}{2.3 \times 36} = \underline{241}$

$L_c = 101.33'' > 65.67'' = L_x$

$\therefore$ FROM AISC 5-45 F1-1 $\rightarrow F_b = \underline{\underline{24^{KSI}}}$

$(= .66 F_y)$ -

DETERMINE INTERACTION COEFFICIENTS

AISC 5-54, H1

H1-1 $\frac{6.03}{18.89} + \frac{14.58}{\underbrace{\left(1 - \frac{6.03}{338.62}\right)}_{.982} 24}$

$.319 + .619 = \underline{.938} < 1$. but too close.

H1-2 DOES NOT GOVERN - BY INSPECTION

USE 25 TON JACK ± 2 TON

HAND CALCULATIONS FOR 27 TON JACK ARE NOT REQUIRED SINCE ALL COMPUTER INTERACTION COEFFICIENTS ARE LOWER

SEE ATTACHMENT I - SUMMARY OF COMPUTER ANALYSES FOR JACKING LOADS -

STEEL SET CALCULATIONS FOR 25^T
JACKING WITHOUT FOOT SEGMENTS.

1) STZRV 4C - MEMBER 3 GOVERNS
SEE ATTACHMENT I, PAGE I-163 THRU I-171

$F_{axial} = \underline{45.84^K}$ $M = \underline{27.95}^{1K}$

$L_y = 88.29'' \doteq 35°$ TIE ROD SPACING

$L_x \cong 3.5 \times 18.79'' = 65.77''$

(FROM MOMENT DIAGRAM).
ATTACHMENT I, PAGE I-170

$\frac{K l_y}{r_y} = \frac{88.29}{2.02} = 43.71 \longrightarrow F_a = \underline{18.89}$ ksi

$\frac{K l_x}{r_x} = \frac{65.77}{3.47} = 18.92 \longrightarrow F_e' = 338.62$ ksi

$F_b = 21.6$

$f_a = \frac{45.84}{9.13} = 5.02$ $f_b = \frac{27.95 \times 12}{27.5}$

AISC 5-54 H1-1 $= 14.76$

$$\frac{5.02}{18.89} + \frac{12.20}{\left(1 - \frac{5.02}{338.62}\right) 21.6} = .266 + .573$$

$$= .839 < 1.0 \text{ OK}$$

* 25 TON JACKING BOTH ENDS @ 47'

2) STZRV3AZ MEMBERS 37 42 GOVERN

SEE ATTACHMENT I, PAGE I-108 THRU I-119

$M = 25^{1K}$

$F_{axial} = 60.65^{K}$

$$f_a = \frac{57.94}{9.13} = 6.64 \text{ Ksi}$$

$$f_b = \frac{25 \times 12}{27.5} = 10.91 \text{ Ksi}$$

AISC 5-54 . H1-1.

$$\frac{6.64}{18.89} + \frac{10.01}{\left(1 - \frac{6.64}{251}\right) 21.6} = .352 + .473$$

$$= .825 < 1.0 \text{ --- O.K.}$$

W6X20 STEEL SET @ 1.8 m (6 ft) SPACING:

FLAC Runs for a W6x20 steel set at 1.83 meter (6 ft) spacing using Welded Wire Fabric (WWF) and no lagging for Support Class I and Rock Property Category 5 (Ref 5.20, Table 14) produced the following max. reaction forces per meter in the steel sets (values from Ref 5.20, Pages 283-286):

Load Case	Axial (N)	Moment (N-m)	Shear (N)	Axial[1] (kip)	Moment[2] (kip-ft)	Shear[1] (kip)
Static	7762	5585	24770	3.19	7.53	10.18
Dynamic	9670	5580	24560	3.97	7.53	10.09

Footnotes:
[1] N x 6 ft spacing/3.2808 ft/m x .0002248 kip/N = N x .000411 = Force (kip)
[2] N-m x 6 ft spacing /3.2808 ft/m x .0007376 k-ft/N-m = N-m x .001349 = Moment (k-ft)

By inspection, the maximum values from the above table will result in interaction values less than unity since the combined axial forces and moments are less than the combined axial forces and moments used from the worst case W6x20 static & seismic load calcs of Attachment III.A (Pages III-18 & 23) and the jacking loads of Attachment VIII (Page VIII-2). Therefore, the W6x20 at 6 ft spacing with WWF (no lagging) is adequate for the above rock load reaction forces from Ref 5.20.

PURPOSE III. B.

STEEL SET LAGGING CALCULATION.

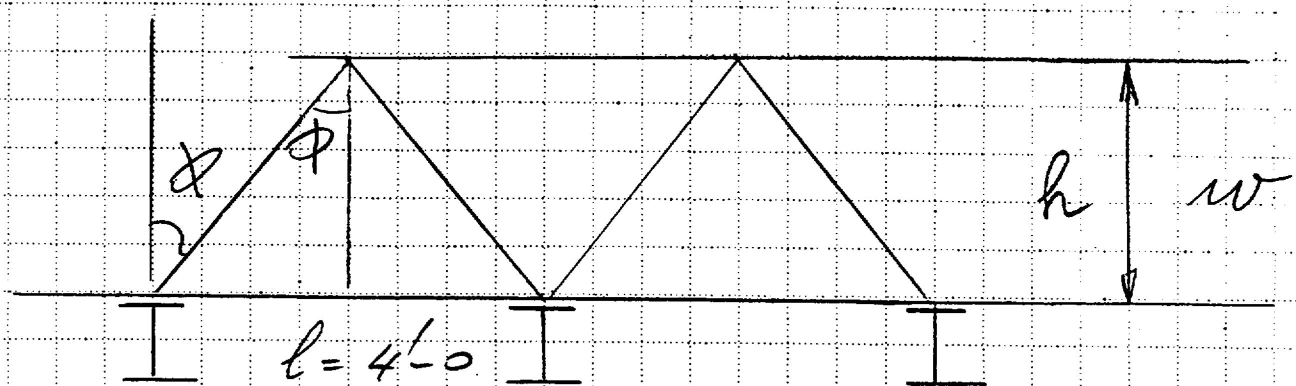
FROM: ESF GROUND SUPPORT DESIGN-ANALYSIS
BABEE0000-01717-0200-00002, REV. 00 (REF. 5,20)

The following data are available

UNIT	MEAN DENSITY Kg/m^3	DENSITY $\#/FT^3$	MINIMUM-FRICTION ANGLE ϕ
TCw	2150	134	53°
PTn	1299	81	40°
TSw1	2162	135	41°
TSw2	2274	142	49°

THE MINIMUM FRICTION ANGLE IS USED because a larger angle will give a smaller value for w (SEE LOAD DIAGRAM NEXT PAGE)

BY INSPECTION TSw1 OR TSw2 WILL DETERMINE THE LARGER w



NOTE: ℓ + CONSTRUCTION TOLERANCE = ℓ ± 2" = 4.33' MAX.
EFFECT OF TOLERANCE ON DESIGN IS FACTORED IN f_b

$$w = h * \text{DENSITY } (D)$$

$$\frac{2'}{h} = tg\,\phi$$

FOR TSW1

$$\frac{2'}{h} = tg\,41^\circ = .8692867$$

$$h = \frac{2'}{.8692867} = 2.3'$$

$$w = 2.3' * D = 2.3' * 140 = \underline{\underline{322}}\ \#/'$$

($D = 135\ \#/\text{CU FT} \longrightarrow = 140\ \#/\text{ft}^3$ FOR BOUNDING)

FOR TSw2

$$\frac{2'}{h} = tg\,49^\circ = 1.15$$

$$h = \frac{2'}{1.15} = 1.74'$$

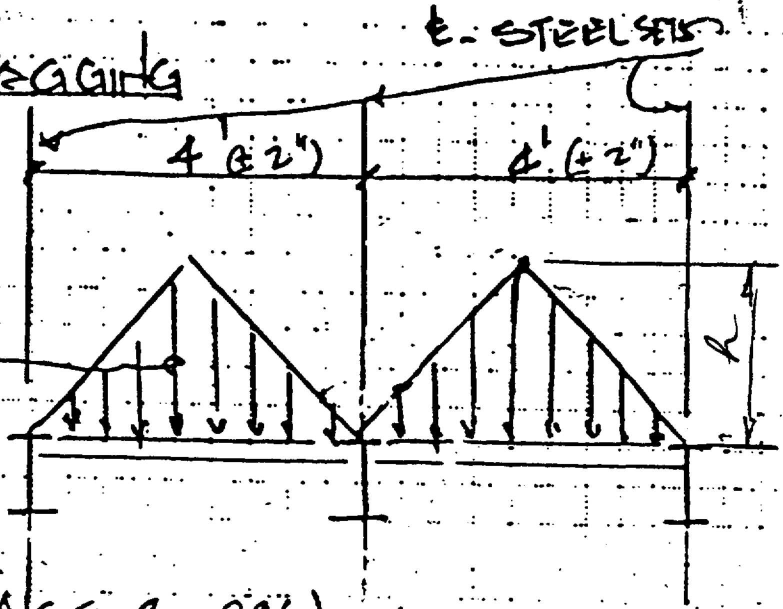
$$w = 1.74 * D = 1.74 * 145 = \underline{\underline{252^{\#/'}}}$$

$(D = 142\ ^{\#}/FT^3$ - USE $145\ ^{\#}/ft^3$ FOR BOUNDING$)$

$w = \underline{\underline{322^{\#/'}}}$ GOVERNS -

CALCULATE MOMENT AND SHEAR IN LAGGING MEMBER with $w = 322$

DESIGN STEEL LAGGING



ROCK LONG TERM LOADS FOR LAGGING w

$M_{max} = \frac{WL}{6} \rightarrow$ (AISC 2-296)

α = ROCK DENSITY

h = RAVELING ROCK HIEGHT

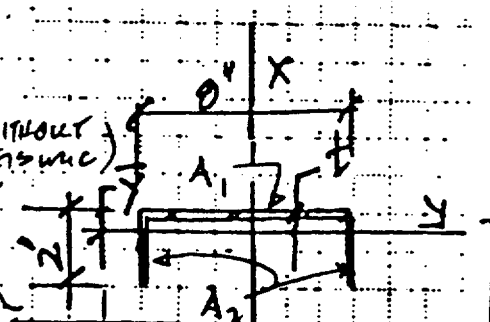
$w = h \times D$; $w = 2.3' \times 140 = 322$ #/'

$W = \frac{322(4)}{2} = 644$

$M_{max} = \frac{644(4)}{6} = 429$ FT-lb

$V = R = \frac{W}{2} = \frac{644}{2} = 322$ lb (WITHOUT SEISMIC)

Try standard cold formed section with GAGE 7 METAL



GAGE 7 = t = 0.1793 IN (6-2, AISC) [CONSTRUCTOR STANDARD PRACTICE OR RECOMMENDATION

$A_1 = 0.1793(8)$ $\quad \bar{Y}_1 = \frac{0.1793}{2}$

$A_2 = 0.1793(2-0.1793)(2)$ $\quad \bar{Y}_2 = \frac{1}{2}(2-0.1793) + 0.1793 = 1.09$

$A_2 = 0.653$

$$\bar{y} = \frac{A_1\bar{y}_1 + A_2\bar{y}_2}{A_1 + A_2}$$

$$\bar{y} \cong \frac{0.1793(8)\left(\frac{0.1793}{2}\right) + 0.653 \times 1.09}{0.1793(8) + 0.653}$$

$$\bar{Y} = \frac{0.84}{2.087} = 0.403''$$

$$I_y = \left[\frac{(0.1793)^3(8)}{12} + 0.1793(8)\left(0.403 - \frac{0.1793}{2}\right)^2\right] + \left[\frac{(2-.1793)^3(0.1793)(2)}{12} + 0.1793(2-0.1793)(2)(1.09)^2\right]$$

$$= 0.0038 + 0.141 + 0.18 + 0.7757$$

$$= 1.1 \text{ in}^4$$

$F_{by} = 21{,}600$ psi ALLOWABLE (F2-2 AISC) (Worse case: LAGGING WILL BE INSTALLED CONTINUOUS ON THE ENTIRE STEEL SET)

$f_{b_1} = \frac{Mc}{I_y}$, $c = 2 - 0.403 = 1.6''$

429 FT-lb IS PER ONE FOOT. BENT PL IS 8" WIDE.

M_y = MOMENT IN ⊓ PLATE

$$= 429\left(\frac{8}{12}\right) = 286 \text{ FT-lb}$$

$$f_{b_1} = \frac{286(1.6)(12)}{1.1} \times \left(\frac{4.33}{4}\right) = 5404 \text{ psi} < 21{,}600 \text{ psi}$$

∴ GAGE 7 IS O.K.

TRY C8 × 11.5 (ASTM A36)

$t_w = 0.22''$ $S_y = 0.781 \text{ in}^3$ (AISC p1-40441)

$$f_{b_1} = \frac{M}{S_y} = \frac{286 \times 12}{0.781} \times \left(\frac{4.33'}{4}\right) = 4{,}757 \text{ psi}$$

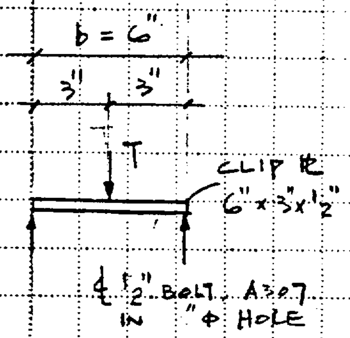
$$f_{b_1} < 21{,}600 \text{ psi}$$

CHECK CONNECTION DETAIL $\frac{8}{IX-12}$, ATTACHMENT IX, Option-1, p. IX-12

a. CHECK CARRIAGE BOLT ~

T = MAX. BOLT TENSION

$= 322^{lb}$ --- D.L.



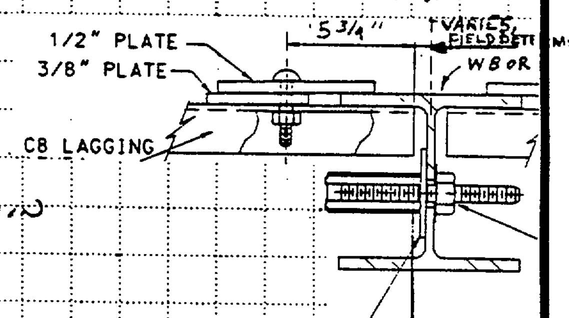
T_a = ALLOWABLE BOLT TENSION

$= 20\ (0.142\ in^2$ TENSILE STRESS AREA)

$= 2.8^k$ ---- AISC 5-73 & 4-147

$\gg T = 0.322^k$ --- O.K.

USE 1/2" Φ A307 BOLT MINIMUM.



b. CHECK CLIP PLATE ~

TRY PLATE SIZE $6" \times 3" \times t$

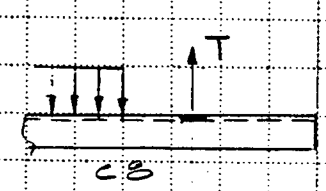
S_x = SECTION MODULUS AT CRITICAL SECTION

$= (3" - \frac{9}{16}\Phi \text{ HOLE}) \times (t)^2/6 = 0.406\, t^2$

M = MAX. MOMENT

$= T \times 6"/4 = 0.322 \times 6/4 = 0.483^{"k}$

$S_x = \frac{M}{F_b}$, $F_b = 0.75 F_y = 27^{ksi}$ -- AISC F2-1



$0.406\, t^2 = \frac{0.483}{27}$

$t = 0.211"$ USE 1/2"

c. CHECK C8 WEB ~

NUT PERIMETER $\cong 4 \times (3/4) = 3"$

v = SHEAR IN C8 WEB $= \frac{0.322^k}{3" \times 0.22} = 0.488^{ksi}$

V_a = ALLOWABLE SHEAR $= 0.4 F_y$ --- AISC 5-49

$= 14.4^{ksi} \gg 0.488^{ksi}$ --- O.K.

USE $6" \times 3" \times 1/2$ CLIP PL, 1/2" Φ CARRIGE BOLT, A307 AND ASME B 18.5

3/8" FILLER PL (COMPATIBLE WITH W8x31 FLG) AS SHOWN IN DETAL $\frac{8}{IX-12}$

CHECK (INCLUDE SEISMIC)

$S_V = 0.37W$ (TBV-193) —— (REF 5.16, APPENDIX A.5)

$S_V = 0.37(644) = 238$ lb

$M = \frac{(644 + 238)4}{6} = 588$ FT-lb/PER FT., $S_y = 0.781\ \text{in}^2$ (AISC 1-41)

M_c = BENDING MOMENT IN C8

$M_c = 588 \left(\frac{8''}{12''}\right) = 392$ FT-lb

$\therefore f_b = \frac{392(12)}{0.781} \times \left(\frac{4.33}{4}\right) = 6{,}522$ psi $= 6.52$ ksi < 27.0 ksi $\times 1.$

USE C8×11.5 FOR LAGGING

AND FILLER PLATE 3/8" × 4" × 4"

USE 9/16" SQUARE HOLES IN 1/2" AND 3/8" PLATES
W/ SLOTTED HOLE IN C8 - FOR TOLERANCE
ADJUSTMENT

T_2 = BOLT TENSION WITH SEISMIC (PAGE III-35)

$T_2 = 322(1.37)\left(\frac{8}{12}\right) \times \left(\frac{4.33}{4}\right) = 318^{lb} = 0.318^k < T_a = 2.8^k \times 1.33$

f_{b2} = BENDING STRESS WITH 1/2" PLATE & w = 3"

$S = \frac{(3 - 9/16)t^2}{6} = \frac{2.44(0.5)^2}{6} = 0.102\ \text{in}^3$

$f_{b2} = \frac{318^{lb} \times 3''}{0.102} = 9352.9$ psi $= 9.35$ ksi < 27.0 ksi $\times 1.3$

1/2" DIA BOLT AND 1/2" PLATE IS OK

Option-2 (p. III 44) is Constructor's option and subject to A/E review. For Option-3 see Attachment IX, p. IX 12 & p. III-45

CALCS. FOR OPTION-3 (LAGGING CONNECTION TO STEEL SET)
(SEE PAGE III-45)

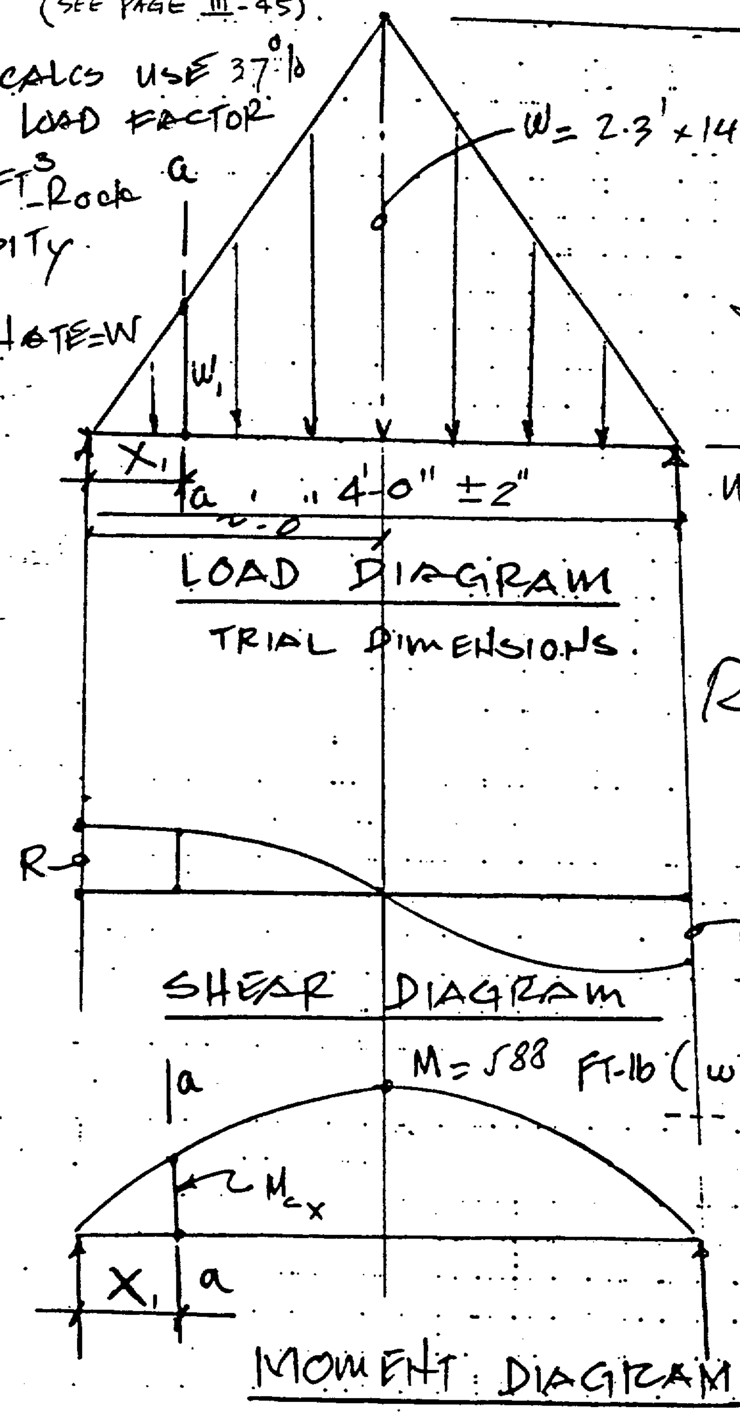
FOR THIS CALCS USE 37% SEISMIC LOAD FACTOR

$\alpha = 140\ lb/FT^3$ Rock DENSITY

LOAD ORDINATE = W

$140 \times 1.37 = 191.8$

$W = 2.3' \times 140 \times 1.37 = 2.3 \times 168$
$= 441\ lb/FT$ WITH SEISMIC



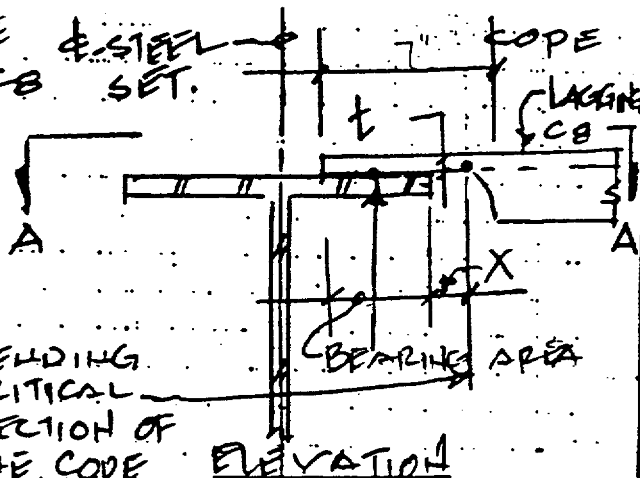
$W = \frac{441 \times 4'}{2}$
$= 882\ lb/ft$

$R = \frac{882}{2} = 441\ lb/ft$

R = 441 lb/FT (WITH SEISMIC)

$t_w = .22''$ (AISC 1-40)

X = DISTANCE FROM EDGE OF BEARING AREA OF C8 TO BENDING MOMENT CRITICAL SECTION OF THE COPE (a-a).



ELEVATION
TRIAL DIMENSIONS

M_{c_x} = BENDING MOMENT ON C8 AT CRITICAL SECTION.

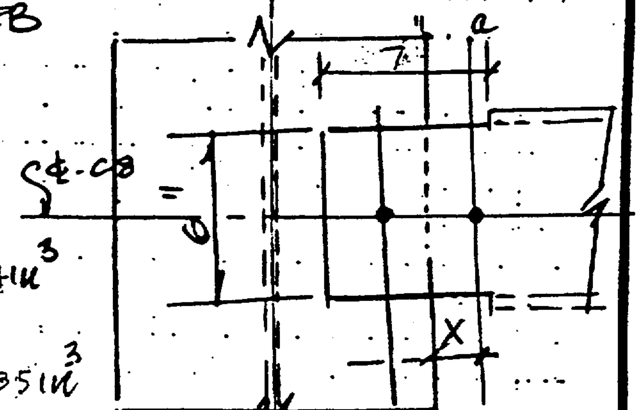
S_6 = SECTION MODULUS AT CRITICAL SECTION FOR COPE AT THE WEB (ALTERNATE 1)

S_8 = SECTION MODULUS AT CRITICAL SECTION FOR COPE AT FLANGES (ALTERNATE 2)

CONSIDER CRITICAL CASE BY LETTING BOLT HOLE AT CRITICAL SECTION

$$S_6 = \frac{(6-\frac{3}{4})(t)^2}{6} = \frac{5.25\times(0.22)^2}{6} = 0.0424\,in^3$$

$$S_8 = \frac{(8-\frac{3}{4})\times(t)^2}{6} = \frac{7.25\cdot(0.22)^2}{6} = 0.0585\,in^3$$

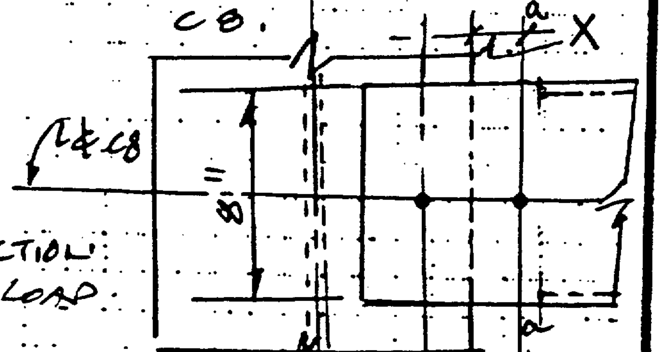


SECTION A (ALTERNATE 1)
COPE AT THE WEB OF C8.

CHECK BENDING STRESS AT CRITICAL SECTION FOR THE FOLLOWING, VALUE OF X

$X = 3''$, $X = 3\frac{1}{2}''$, $X = 4''$

AND $X = 7'' - \frac{3}{4}'' = 6.25''$ (MIN BEARING LENGTH)



w_1 = LOAD AT CRITICAL SECTION OF THE COPE. SEISMIC LOAD

IF $X = 3'' = 0.25'$

$$w_1 = \frac{2.3\,(191.8)}{2}(0.25) = 55\ lb/ft$$

SECTION A (ALTERNATE-2
COPE AT THE FLANGES OF C8

IF $X = 3\frac{1}{2}'' = 0.29'$

$$w_1 = \frac{2.3(191.8)}{2}(0.29) = 64 \text{ lb}$$

IF $X = 4'' = 0.33'$

$$w_1 = \frac{2.3(191.8)}{2}(0.33) = 72.8 \text{ lb}$$

IF $X = 6\frac{1}{4}'' = 0.521'$

$$w_1 = \frac{2.3(191.8)}{2}(0.521) = 114.88 \text{ lb}$$

$$\Sigma M_{a\text{-}a} = 0$$

$$M_{c_3} = \left[441(3) - \frac{55(3)}{2}\left(\frac{3}{3}\right)\right]\frac{8}{12}$$

$$= 827 \text{ in-lb}$$

$$M_{c_{3\frac{1}{2}}} = \left[441(3.5) - \frac{64}{2}(3.5)\left(\frac{3.5}{3}\right)\right]\frac{8}{12}$$

$$= 942.0 \text{ in-lb}$$

$$M_{c_4} = \left[441(4) - \frac{72.8}{2}(4)\left(\frac{4}{3}\right)\right]\frac{8}{12}$$

$$= 1046.6 \text{ in-lb}$$

$$M_{c_{4\frac{1}{2}}} = \left[441 \times (6.25) - \frac{114.88}{2}(6.25)\left(\frac{6.25}{3}\right)\right]\frac{8}{12}$$

747.92

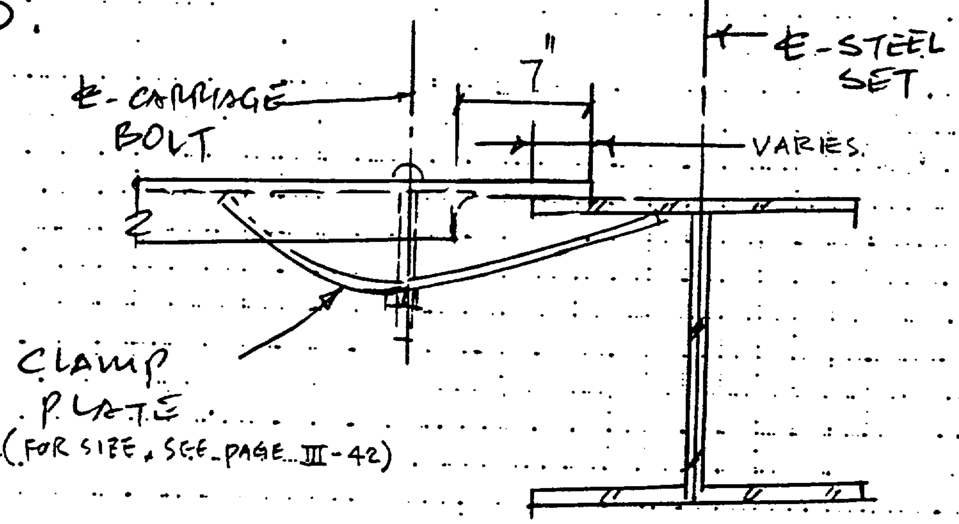
$$= 2008.3 \text{ in-lb}$$

CALCULATE BENDING STRESS AT CRITICAL SECTION & USE $0.75 F_y$ FOR ALLOWABLE STRESS AISC (F2-1)

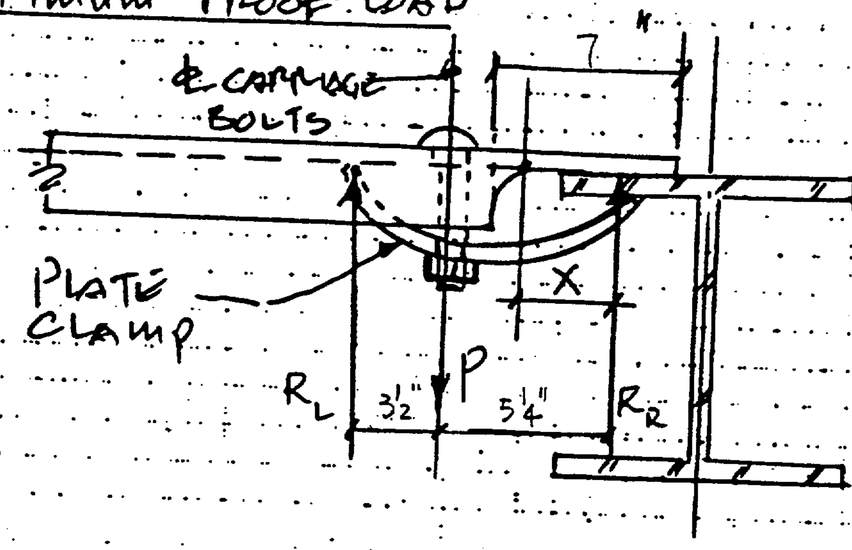
C8 x 11.5		BENDING STRESS @ CRITICAL SECTION $\frac{M}{S}$	
X	M_{cx}	SECTION 6"-COPE AT WEB $S_0 = 0.0424 in^3$	SECTION 8" COPE AT FLANGES $S_0 = 0.0585 in^3$
3"	827 in-lb	19.5 ksi < 1.33 x 0.75 Fy = 35.9 ksi	14.14 ksi < 1.33 x 0.75 Fy = 35.9 ksi
3 1/2"	942 in-lb	22.2 ksi < 1.33 x 0.75 Fy	16.1 ksi < 1.33 x 0.75 Fy
4"	1046.6 in-lb	24.7 ksi < 1.33 x 0.75 Fy	17.9 ksi < 1.33 x 0.75 Fy
6 1/4"	2008.3 in-lb	47.4 ksi > 1.33 x 0.75 Fy --- N.G.	34.3 ksi < 1.33 x 0.75 Fy * ---- O.K.

* ∴ USE C8X11.5 WITH 7" MAX COPE AT THE FLANGES AND SHALL BE INSTALLED AS SHOWN BELOW.

FOR SAFETY DURING CONSTRUCTION OF THE LAGGING A CLAMP PLATE IS REQUIRED.



CHECK MAXIMUM PROOF LOAD



P = PROOF LOAD THAT CRITICAL SECTION CAN WITHSTAND.

CLAMP DETAIL

TRIAL DIMENSIONS.

$\Sigma M_{R_L} = 0$

$8 R_R = 3P$

$R_R = 0.375P$

$M_{MAX} = R_R \left(5\tfrac{1}{4}\right)$ ---- MAX. BENDING MOMENT IN CHANNEL AT CRITICAL SECTION OF THE CHANNEL.

$= 0.375\,P(5.25) = 1.97\,P$ in-lb

$F_b = 27.0^{ksi} = 0.75\,F_y$ ALLOWABLE BENDING STRESS
$= 27{,}000$ psi (AISC F2-1, p. 5-48)

$F_b = \dfrac{Mc_A}{S_B}$

$27000 = \dfrac{1.97P}{0.0585}$

$P = 801.8$ lb $\quad \therefore M_{MAX} = 1.97(801.8) = 1579.5$ in-lb

FROM PREVIOUS PAGE, IT DEMONSTRATES THAT BOLT TENSION IS LIMITED TO 1579.5 lb TO ENSURE THAT THE BENDING IN CLAMP PL IS WITHIN ALLOWABLE, THE CARRIAGE BOLT SHALL BE SNUG TIGHT AND NOT BE TIGHTENED TO A BOLT PRETENSION.

THE CLAMP PLATE STIFFNESS SHALL NOT EXCEED THE STIFFNESS OF C8 CRITICAL SECTION SO THAT THE BACK OF CHANNEL WILL NOT BEND AWAY FROM ROCK SURFACE.

TRY $3/16 \times 8^{+''} \times 3''$

$$S = \frac{3 \times (\frac{3}{16})^2}{6}$$

$$= 0.0176 \text{ in}^3$$

$$f_b = \frac{M_{3/16}}{S_{3/16}}$$; $M_{3/16}$ = MOMENT CAPACITY FOR CLAMP PLATE

$$27{,}000 = \frac{M_{3/16}}{0.0176}$$

$$M_{3/16} = 475.0 \text{ in-lb}$$

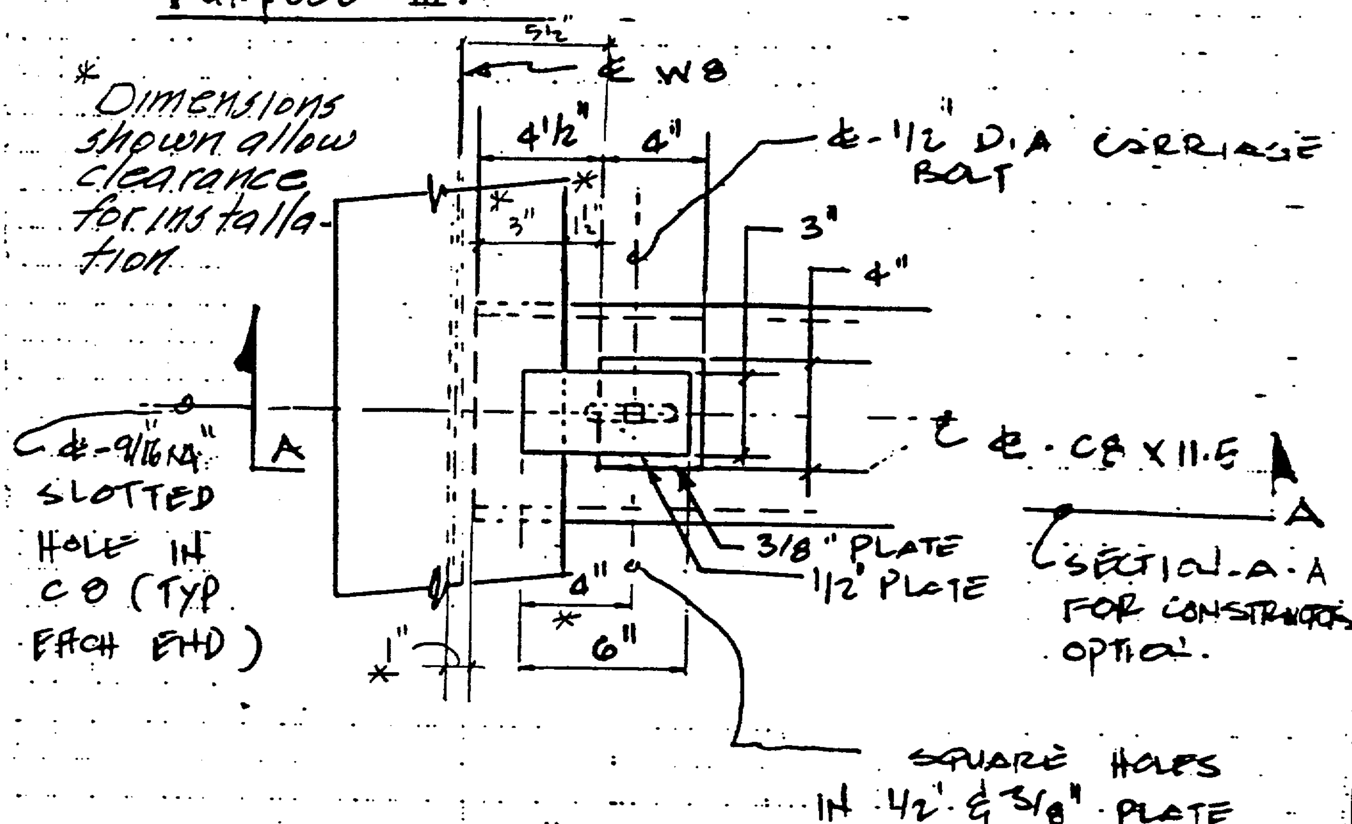
$$\therefore M_{3/16} < M_{c4}, \quad \text{RATIO} = \frac{1579.5}{475.0} = 3.3 \text{ ok}$$

USE CURVE $3/16'' \times 8^{+} \times 3''$ PLATE, Option-3

OR $8'' \times 3''$ PIECE CUT FROM 20" Φ SCH 10 PIPE (WALL THICKNESS = 0.25")

SUMMARY

PURPOSE - III.B



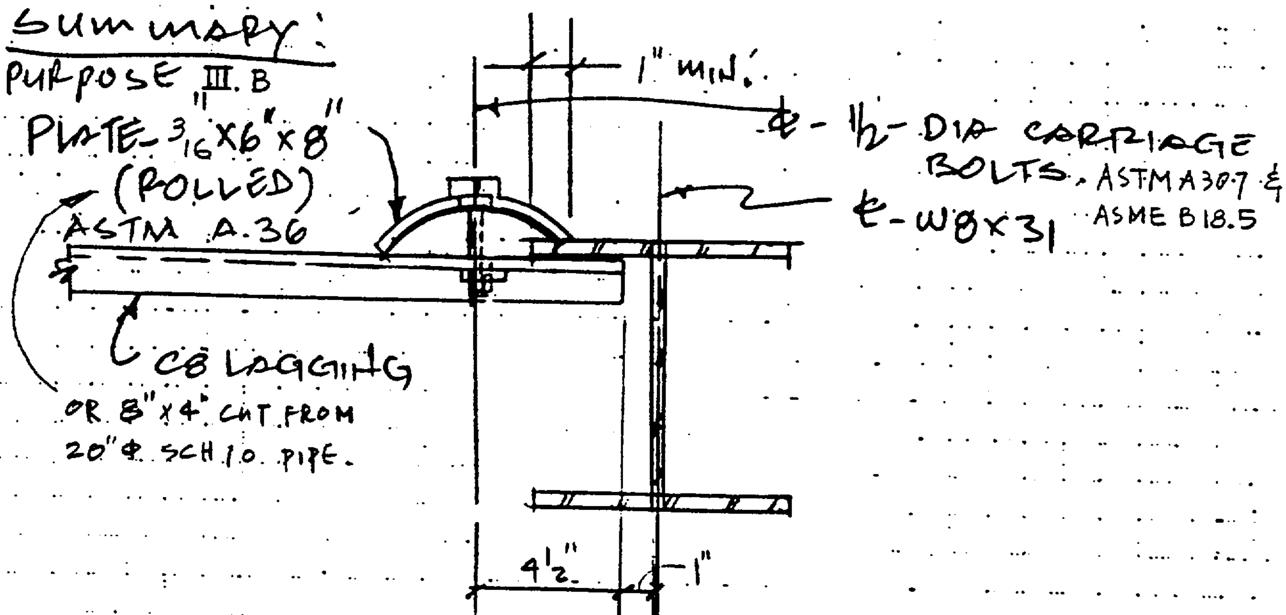
C8 X 11.5, ½" AND 3/8" PLATES SHALL BE IN ACCORDANCE WITH ASTM A36

½" CARRIAGE BOLT SHALL BE IN ACCORDANCE WITH ASTM A 307 AND ASME-B 18.5

OPTION - 1 (p. III-35)

SUMMARY:

PURPOSE III.B



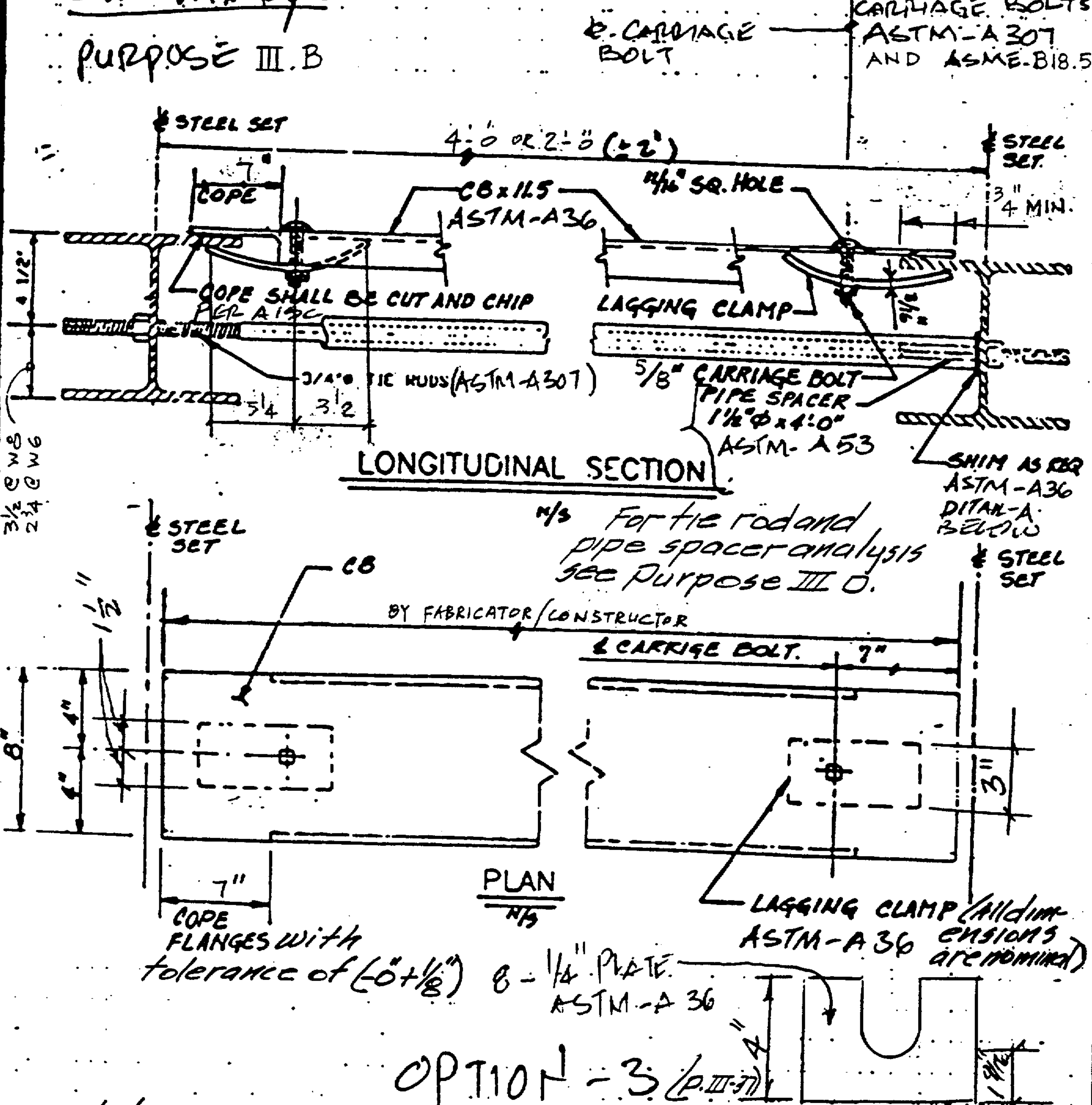
SECTION - A - A

(CONSTRUCTOR OPTION)

CONSTRUCTORS SHALL SUBMIT SHOP DRAWING FOR THIS OPTIONAL - 2 OF LAGGING AND PLATE CONNECTION AND SHALL BE REVIEWED BY THE A/E.

OPTION - 2 (p. III-37)

SUMMARY - PURPOSE III.B



OPTION - 3 (p.III-37)

III-40, III-41 & III-42

(see p.III-62)

DET. A

Note: For this option lagging clamp, carriage bolt, nut and washer are classified QA: NONE.

PURPOSE III.C A) JACKING BRACKET ASSEMBLY CALCULATION FOR W8x31 STEEL SETS

DESIGN JACKING BRACKET : (25 TON JACK CAPACITY) DESIGN FOR 27 TON FORCE.

1. DESIGN OF CONNECTION BOLTS TO STEEL SET:

JACKING BRACKET SHALL BE REMOVED AFTER A COMPLETE RING OR SEGMENTS ASSEMBLY OF STEEL SET IS IN PLACE.

TRY 8-1" A307 BOLTS (SEE AISC 4-3 & 4-5)

T_a = ALLOWABLE BOLT TENSION

$= 15.70^k \rightarrow$ TENSION ON NOMINAL AREA AREA - AISC 4-3

OR $20(.606) = 12.12^k$ TENSION ON NET AREA - AISC 4-147

$\therefore$ USE $T_a = 12.12^k$ ALLOWABLE

V_a = ALLOWABLE BOLT SHEAR

$V_a = 7.9^k$ – AISC 4-5

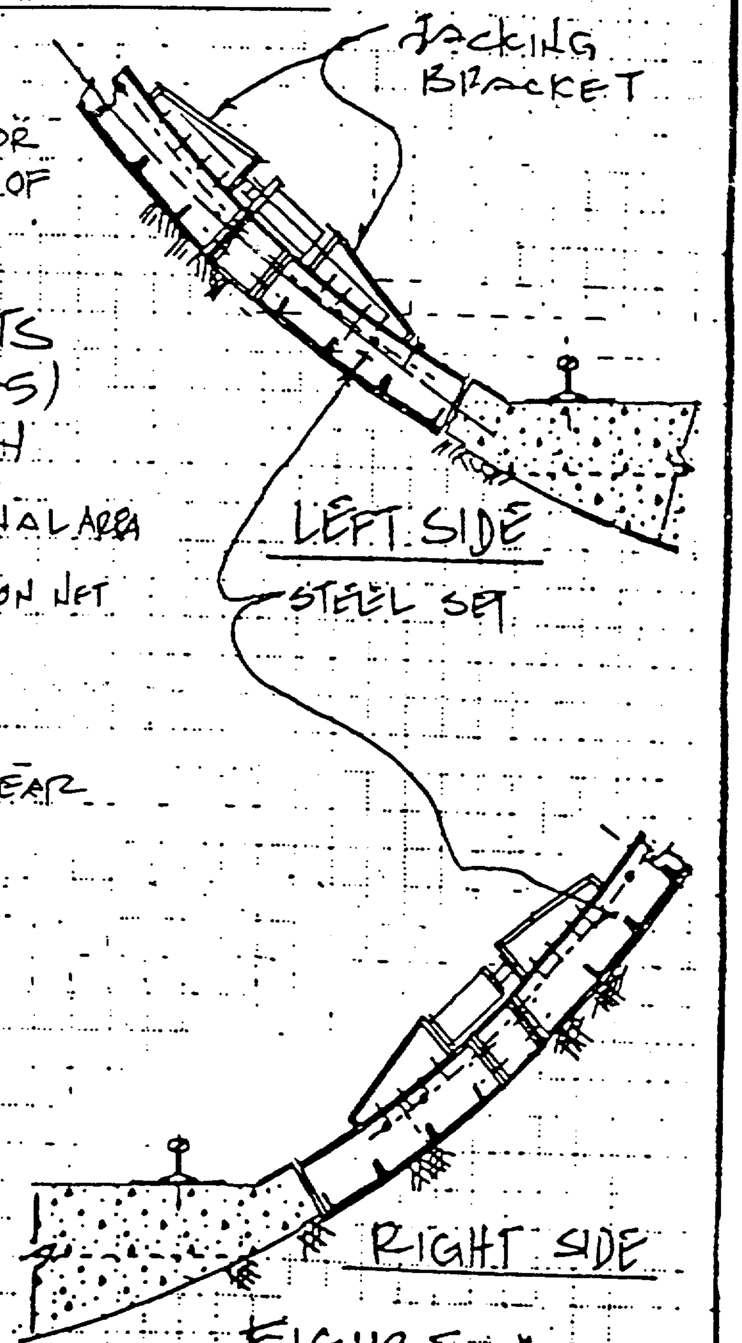
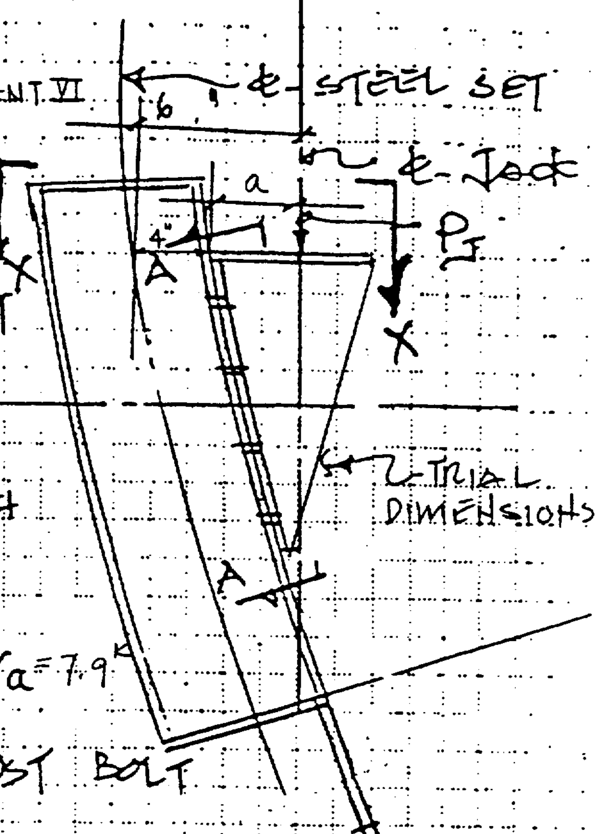


FIGURE-A

a = ℄ JACK TO W8 FLANGE
= 1/2 JACK BODY DIAMETER --- ATTACHMENT VI
= $1\frac{11}{16}$" ; USE 2"

DESIGN CAPACITY $= P_J = 54^K$



M = MOMENT ON THE BRACKET

$M = 54(2) = 108^{IN\text{-}K}$

V_a = SHEAR FORCE ON EACH BOLT

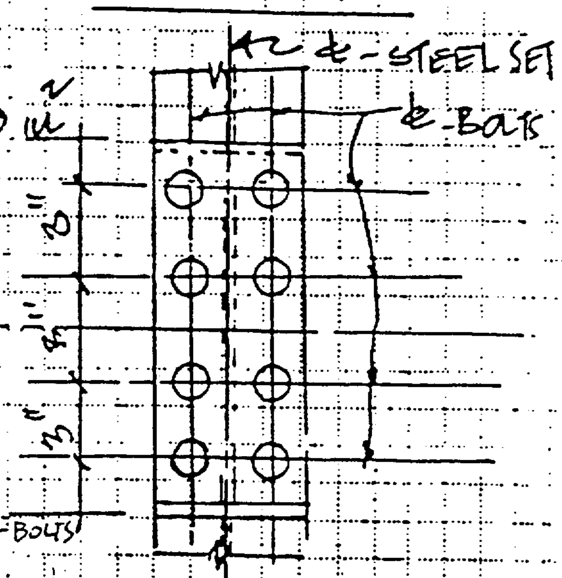
$V_a = \frac{54}{8} = 6.75^{K/BOLT} < V_a = 7.9^K$

T_a = TENSION ON THE TOP MOST BOLT

$T_a = \frac{Mc_1}{I}$

$\begin{cases} I_o = \frac{\pi}{4}(.5)^4 = 0.049 \text{ IN}^4 \\ A = 0.7854 \text{ in}^2 \end{cases}$

FIGURE_B



SECTION A-A

$c_1 = 4.5"$

$\Sigma c^2 = (1.5^2 + 4.5^2) = 22.50 \text{ in}^2$

$I = 4[I_o + A\Sigma c^2]$

$I = 4[0.049 + 0.7854(22.5)]$

$= 70.88 \text{ in}^4$

$T_a = \frac{108(4.5)}{70.88} = 6.86^K$ FOR (2)-BOLTS

$= 3.43^K$ PER BOLT $< 12.12^K$ --- O.K.

(SEE NEXT PAGE)

$V_a = 7.9^k$ --- AISC 4-5

$T = 20^{ksi} \times 0.606 = 12.12^k$ --- AISC 4-3 & 4-147

$f_v = 6.75^k / 0.7854 = 8.6^{ksi}$

$26 - 1.8 f_v = 10.5^{ksi} < 20^{ksi}$ --- O.K. (AISC 5-74, TABLE J3.3)

USE 8-1" Φ ASTM A307 (60,000 PSI) MINIMUM WITH ASTM A563, GRADE DH HEAVY HEX NUTS AND ASTM F436 WASHERS, TYPE I, UNLESS SHOWN OTHERWISE. ALL BOLTS SHALL BE SNUG TIGHT.

2. DESIG OF JACK PLATE:

(ALTERNATE DESIGN BY CONTRACTORS OPTION.)

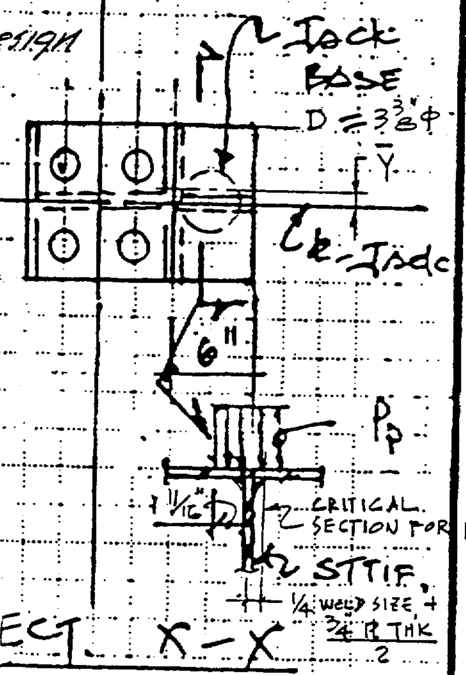
REPRESENTATIVE PARAMETERS SHOWN ON SECT X-X.
For details of jack see Section 4.1.5, Design Inputs.

A_J = JACK CROSS SECTIONAL AREA @ BASE

$A_J = \frac{\pi (3\frac{3}{8})^2}{4} = 8.95 \text{ IN}^2$

P_p = PRESSURE FROM JACK ON PLATE UNDER JACK BASE

$P_p = \frac{54}{A_J} = 6.04$ KSI



SECT. X-X
FROM FIG. B

M_J = MOMENT IN TOP JACKING PLATE (6" STRIP)

$= P_p \left(\frac{A_J}{2}\right)(\bar{Y})$; $\bar{Y} = R - R\left(1 - \frac{4}{3\pi}\right) = 0.72"$ --- (AISC 6-20)

$M_J = 6.04 \left(\frac{8.95}{2}\right)\left(0.72" - 0.25 - \frac{0.75}{2}\right) = 2.57^{\text{IN-K}} / 6"$ STRIP

0.095

DETERMINE THICKNESS OF CAP PLATE,

t = THICKNESS OF CAP PLATE.

$$S = \frac{b\,t^2}{6} = \frac{6\,t^2}{6} = t^2$$

$$S = \frac{M_J}{F_b}$$

$$t^2 = \frac{M_J}{F_b}$$

$F_b = 0.75\,F_y = 27^{KSI}$ ---- (AISC PAGE 5-48, F2-1)

$$t^2 = \frac{2.57}{27} = 0.095''$$

$$t = 0.308''$$

USE 3/4" PL - HORIZONTAL PLATE FOR JACKING BRACKET.

DESIGN OF VERTICAL PLATE OF THE JACKING BRACKET

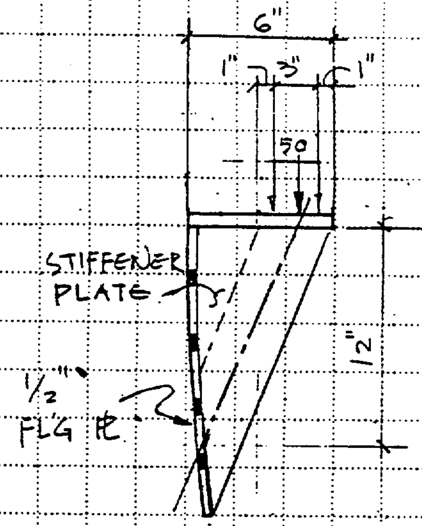
(a) STIFFENER PLATE ~

WIDTH TO THICKNESS RATIO

$d/t \le 127/\sqrt{F_y}$ --- (AISC, p 5-36)

$d = 6''$ MAX.

$t_{MIN} = \dfrac{d}{127/\sqrt{F_y}} = \dfrac{6}{21.17} = 0.283''$



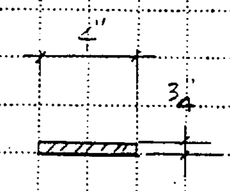
TRY 3/4" STIFFENER PLATE W/
1/2" FLANGE PLATE.

CHECK STIFFENER PL FOR JACK LOAD.
THE STIFFENER PLATE CAN BE IDEALIZED
AS A COMPRESSION ELEMENT 3/4" x 4" x 12" LG

$I_{STIFF. PL} = \dfrac{4 \times (0.75)^3}{12} = 0.14 \text{ IN}^4$

$A = 0.75 \times 4 = 3.0 \text{ IN}^2$

$r = \sqrt{\dfrac{0.14}{3.0}} = 0.216$



$Kl = \frac{3}{4} \times 12'' = 9''$ --- EFFECTIVE LENGTH
(AISC K1.8, PAGE 5-B2)

$Kl/r = 9.0/0.216 = 41.67 \Rightarrow F_a = 19.0^{ksi}$ --- AISC, TABLE C-36

$P_a = 19.0^{ksi} \times 3.0 = 57^k > 54^k$ ---- O.K.

USE 3/4" STIFFENER PL AS SHOWN.

(b) WELDING OF STIFFENER PL TO FLG PLATE OF JACKING BRACKET ~

TRY $\frac{1}{4}''$ FILLET WELD $\frac{1}{4}$ / $\frac{1}{4}$ 14" LG.

A_w = WELD AREA $= (2 \times \frac{1}{4})(14) = 7''\ in^2$

$M = 54^k \times 4'' = 216^{''k}$

$P/A = 54/7 = 7.71^{ksi}$

$S_w = \dfrac{bd^2}{6} = (2 \times \frac{1}{4}) \times \dfrac{14^2}{6} = 16.33\ in^2$

$\dfrac{M}{S_w} = 216^{''k}/16.33 = 13.23^{ksi}$

RESULTANT STRESS IN WELD

$= [(7.71)^2 + (13.23)^2]^{1/2} = 15.3^{ksi}$

ALLOWABLE WELD STRESS $= 0.3 \times 70^{ksi}$ ----(AISC PAGE 5-70)

$= 21.0^{ksi} > 15.3^{ksi}$ ---- O.K.

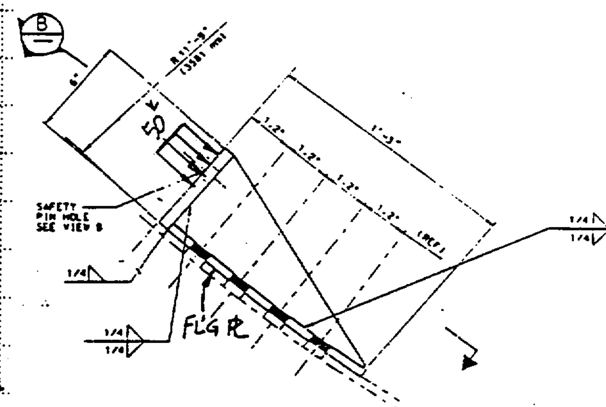
MINIMUM SIZE OF FILLET WELD FOR BASE METAL PLATE OF $\frac{1}{2}$ TO $\frac{3}{4}''$ IS $\frac{1}{4}''$ WELD --- O.K. (AISC TABLE J2.4)

USE $\frac{1}{4}$ / $\frac{1}{4}$ FILLET WELD E70XX MINIMUM ELECTRODES.

<C> FLG PL OF JACKING BRACKET IN CONTACT WITH W8x31 FLG :

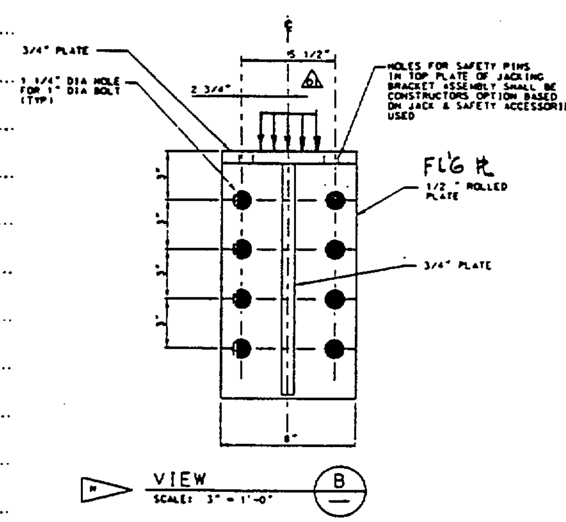
TRY 1/2" PLATE, ASTM A36 MATERIAL,

MIN. BOLT EDGE DISTANCE ℓ
PARALLEL TO THE LINE OF
FORCE $= 3'' > 1\frac{1}{2}d = 1.5''$
AND BOLT SPACING
$= 3'' \nless 3d = 3''$ (AISC P. 5-75)



ALLOWABLE BEARING
$F_p = 1.2 F_u$ --- AISC J3-1, P.5-74
$= 1.2 \times 58^{ksi} = 69.6^{ksi}$

BEARING AREA
$= 1'' \times \frac{1}{2}'' = 0.5'' \; in^2$



SHEAR ON EACH BOLT
$= \frac{54^k}{8} = 6.75^k$

BEARING STRESS
$= \frac{6.75}{0.5} = 13.5^{ksi} < F_p = 69.6^{ksi}$
--- O.K.

USE 1/2" FLG PLATE
ASTM A36 MATERIAL

PURPOSE III.C B) JACKING BRACKET ASSEMBLY CALCULATION FOR W6X20 STEEL SETS

DESIGN JACKING BRACKET : (15 TON JACK FORCE)

1. DESIGN OF CONNECTION BOLTS TO STEEL SET : (DESIGN FOR 17 TONS)

JACKING BRACKET SHALL BE REMOVED AFTER
A. COMPLETE RING OR SEGMENTS ASSEMBLY OF STEEL SET IS IN PLACE.

TRY 6 - 1" A 307 BOLTS
(SEE AISC 4-3 & 4-5

T_a = ALLOWABLE BOLT TENSION

$= 15.70^k \rightarrow$ TENSION ON NOMINAL AREA AREA - AISC 4-3

OR $20(.606) = 12.12^k$ TENSION ON NET AREA - AISC 4-147

$\therefore$ USE $T_a = 12.12^k$ ALLOWABLE

V_a = ALLOWABLE BOLT SHEAR

$V_a = 7.9^k$ - AISC 4-5

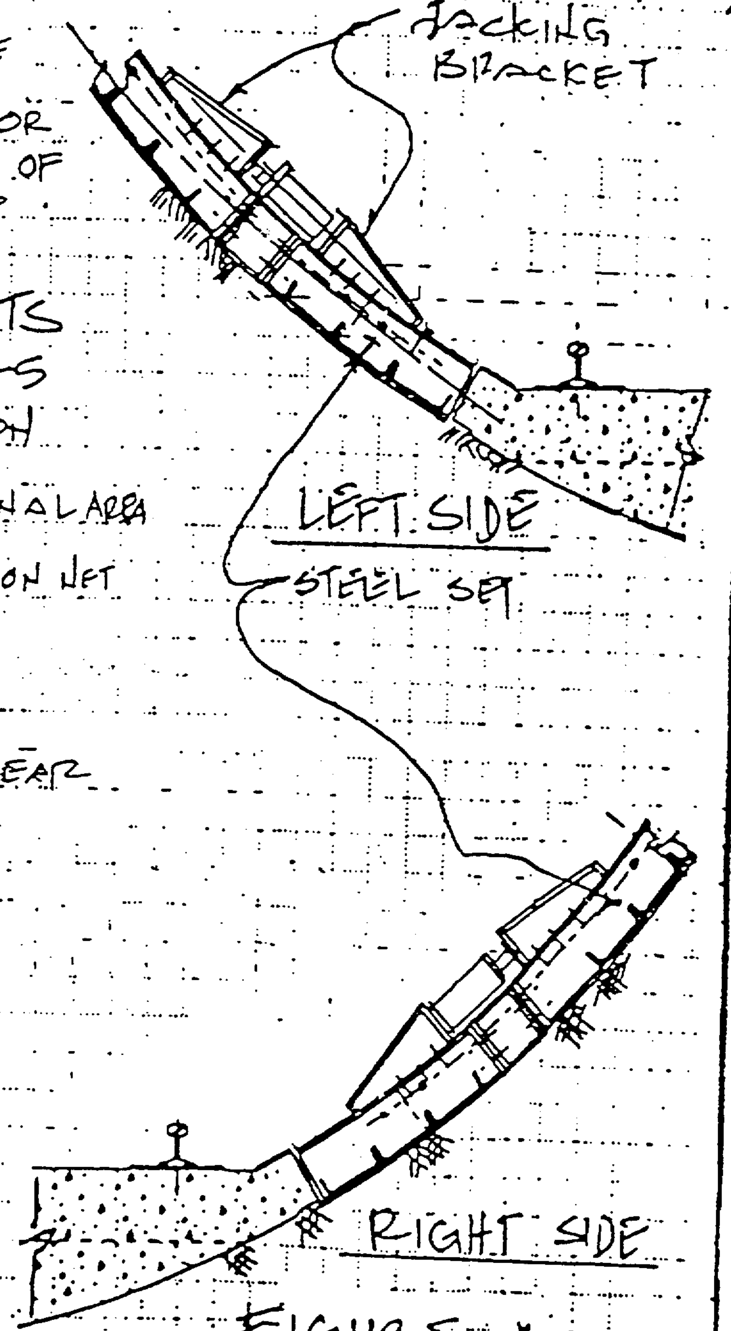


FIGURE - A

a = ℄ JACK TO W6 FLANGE
= 1/2 JACK BODY DIAMETER --- ATTACHMENT VI
= $1\frac{3}{8}$" ; USE 2"

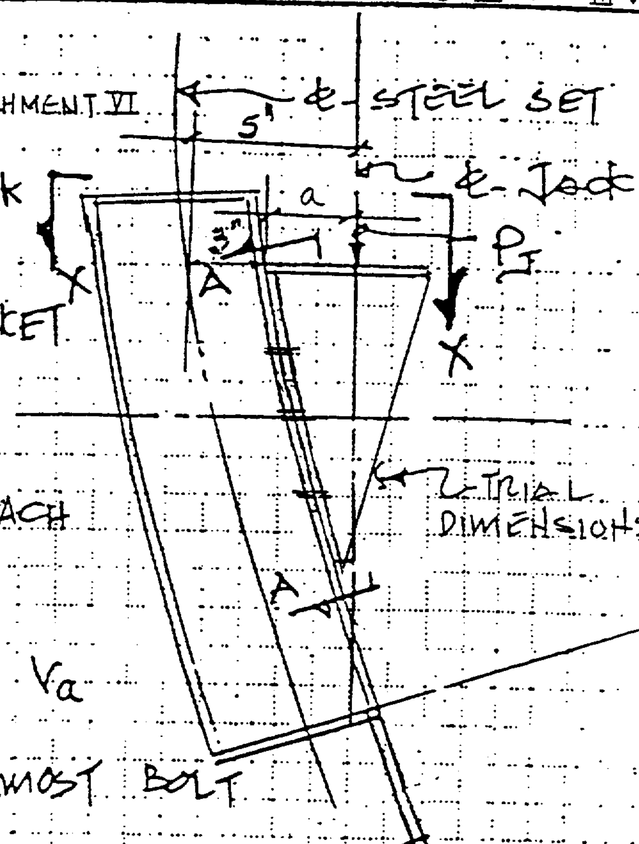
DESIGN CAPACITY = $P_J = 34^k$

M = MOMENT ON THE BRACKET

$M = 34(2) = 68^{\text{in-k}}$

V_6 = SHEAR FORCE ON EACH BOLT

$V_6 = \dfrac{34}{6} = 5.67^{\text{K/BOLT}} < V_a$



T_6 = TENSION ON THE TOP MOST BOLT

$T_6 = \dfrac{Mc_1}{I}$

$\begin{cases} I_o = \frac{\pi}{4}(.5)^4 = 0.049 \text{ IN}^4 \\ A = 0.7854 \text{ in}^2 \end{cases}$

FIGURE_B

$c_1 = 3$"

$\Sigma c^2 = (\bar{0.0}^2 + \bar{3.0}^2) = 9.0 \text{ in}^2$

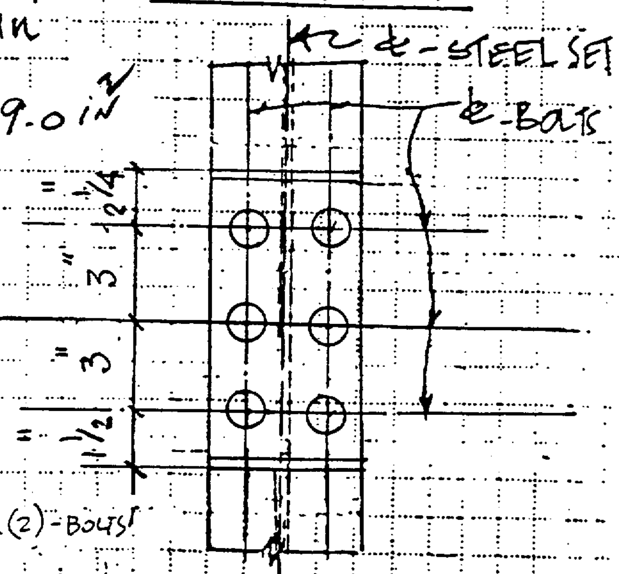
$I = 2[I_o + A\Sigma c^2]$

$I = 2[0.049 + 0.7854(9.0)]$

$= 14.24 \text{ in}^4$

$T_6 = \dfrac{68(3.0)}{14.24} = 14.36^k$ FOR (2)-BOLTS

$= 7.16^k$ PER BOLT < 12.12^k --- O.K.
(SEE NEXT PAGE)



SECTION A-A

$V_a = 7.9^k$ --- AISC 4-5

$T_a = 20^{ksi} \times 0.606 = 12.12^k$ --- AISC 4-3 & 4-147

$f_v = 5.67^k / 0.7854 = 7.22^{ksi}$

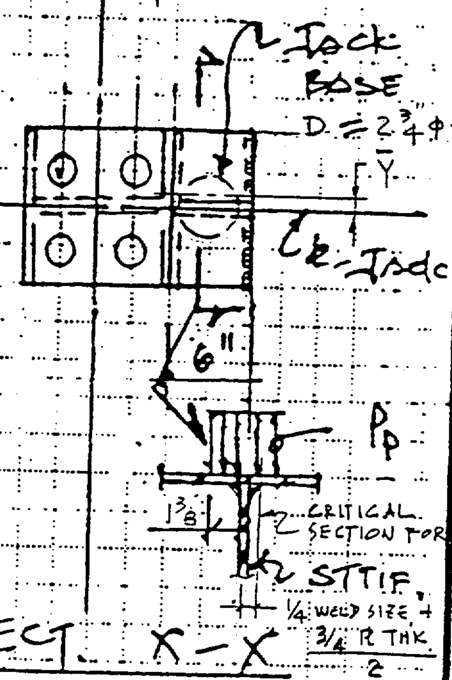
$26 - 1.8 f_v = 13.0^{ksi} < 20^{ksi}$ --- O.K. (AISC 5-74, TABLE J3.3)

USE 6 - 1" φ ASTM A307 (60,000 PSI) MINIMUM WITH ASTM A563, GRADE DH HEAVY HEX NUTS AND ASTM F436 WASHERS, TYPE 1, UNLESS SHOWN OTHERWISE. ALL BOLTS SHALL BE SNUG TIGHT.

2. DESIGN OF JACK PLATE :

(ALTERNATE DESIGN BY CONTRACTORS OPTION.)

REPRESENTATIVE PARAMETERS SHOWN ON SECT X-X



PAGE VI-8

A_J = JACK CROSS SECTIONAL AREA @ BASE

$A_J = \frac{\pi (2.75)^2}{4} = 5.94 \ in^2$

P_p = PRESSURE FROM JACK ON PLATE UNDER JACK BASE

$P_p = \frac{34}{A_J} = 5.72$ KSI

M_J = MOMENT IN TOP JACKING PLATE (6" - STRIP) SECT. X-X FROM FIG. B

$= P_p \left(\frac{A_J}{2}\right)(\bar{Y})$; $\bar{Y} = R - R\left(1 - \frac{4}{3\pi}\right) = 0.584$ --- (AISC 6-20)

$M_J = 5.72 \times \left(\frac{5.94}{2}\right)\left(0.584 - 0.25 - \frac{0.75}{2}\right) \cong 0$ "-K / 6" STRIP

$= 0.041$

DETERMINE THICKNESS OF CAP PLATE,

t = THICKNESS OF CAP PLATE.

$$S = \frac{b\,t^2}{6} = \frac{6\,t^2}{6} = t^2$$

$$S = \frac{M_J}{F_b}$$

$$t^2 = \frac{M_J}{F_b}$$

$F_b = 0.75 F_y = 27^{KSI}$ --- (AISC PAGE 5-48, F2-1)

$$t^2 = \frac{0.0}{27} = 0''$$

$t = 0''$

USE 3/4" PL - HORIZONTAL PLATE FOR JACKING BRACKET.

DESIGN OF VERTICAL PLATE OF THE JACKING BRACKET

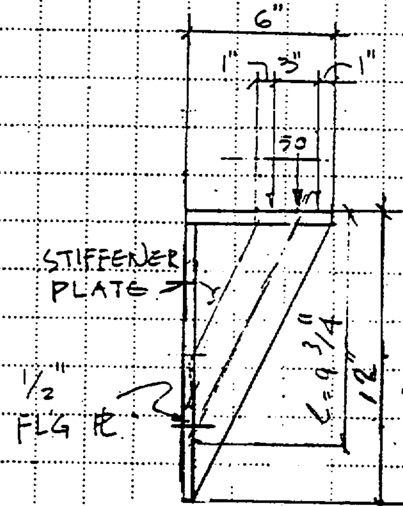
(a) STIFFENER PLATE ~

WIDTH TO THICKNESS RATIO

$d/t \le 127/\sqrt{F_y}$ --- (AISC, p 5-36)

$d = 6''$ MAX.

$t_{MIN} = \dfrac{d}{127/\sqrt{F_y}} = \dfrac{6}{21.17} = 0.283''$



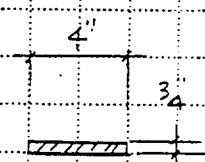
TRY 3/4" STIFFENER PLATE W/
1/2" FLANGE PLATE.

CHECK STIFFENER PL FOR JACK LOAD.
THE STIFFENER PLATE CAN BE IDEALIZED
AS A COMPRESSION ELEMENT 3/4" x 4" x 9 3/4" LG

$I_{STIFF\ PL} = \dfrac{4 \times (0.75)^3}{12} = 0.14 \ IN^4$

$A = 0.75 \times 4 = 3.0 \ IN^2$

$r = \sqrt{\dfrac{0.14}{3.0}} = 0.216$



$Kl = \frac{3}{4} \times 9.75'' = 7.31$ --- EFFECTIVE LENGTH
(AISC K1.8, PAGE 5-32)

$Kl/r = 7.31/0.216 = 33.85 \rightarrow F_a = 19.6^{ksi}$ --- AISC, TABLE C-36

$P_a = 19.6^{ksi} \times 3.0 = 58.8^k > 34^k$ --- O.K.

USE 3/4" STIFFENER PL AS SHOWN.

(b) WELDING OF STIFFENER PL TO FLG PLATE OF JACKING BRACKET ~

TRY 1/4" FILLET WELD 1/4 / 1/4 14" LG.

A_w = WELD AREA = $(2 \times \frac{1}{4})(12'') = 6'' \text{ in}^2$

$M = 34^k \times 4'' = 68^{''k}$

$P/A = 34/6 = 5.67^{ksi}$

$S_w = \frac{bd^2}{6} = (2 \times \frac{1}{4}) \times \frac{12^2}{6} = 12.0 \text{ in}^2$

$\frac{M}{S_w} = 68^{''k}/12 = 5.67^{ksi}$

RESULTANT STRESS IN WELD

$= [(5.67)^2 + (5.67)^2]^{1/2} = 8.02^{ksi}$

ALLOWABLE WELD STRESS ≐ BASE METAL ---(AISC PAGE 5-70)

$= 21.6^{ksi} > 8.02^{ksi}$ --- O.K.

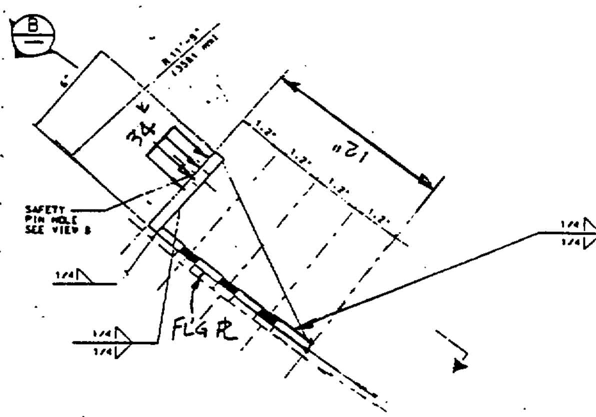
MINIMUM SIZE OF FILLET WELD FOR BASE METAL PLATE OF 1/2 TO 3/4" IS 1/4" WELD --- O.K. (AISC TABLE J2.4)

USE 1/4 / 1/4 FILLET WELD E 70XX MINIMUM ELECTRODES.

<C> FLG PL OF JACKING BRACKET IN CONTACT WITH W6×20 FLG :

TRY 1/2" PLATE, ASTM A36 MATERIAL,

∴ MIN. BOLT EDGE DISTANCE l PARALLEL TO THE LINE OF FORCE = 3" > $1\frac{1}{2}d$ = 1.5" AND BOLT SPACING = 3" ≮ 3d = 3"



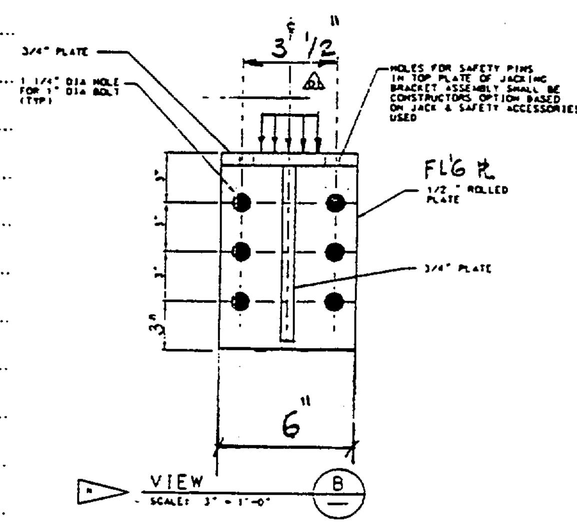
ALLOWABLE BEARING

$F_p = 1.2 F_u$ ---- AISC P 4-6

$= 1.2 \times 58^{ksi} = 69.6^{ksi}$

BEARING AREA

$= 1" \times 1/2" = 0.5" \ in^2$



SHEAR ON EACH BOLT

$= \frac{34^k}{6} = 5.67^k$

BEARING STRESS

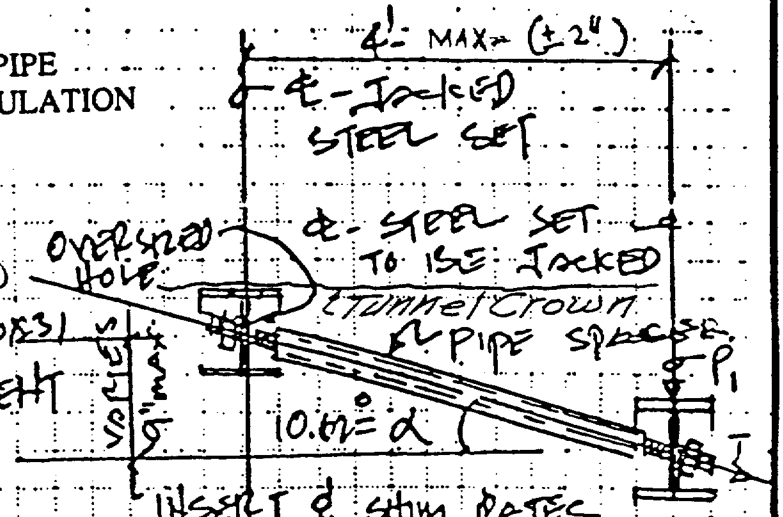
$= \frac{5.67}{0.5} = 11.34^{ksi} < F_p = 69.6^{ksi}$

---- O.K.

USE 1/2" FLG PLATE
ASTM A36 MATERIAL

PURPOSE III.D

TIE ROD AND PIPE SPACER CALCULATION



DESIGN TIE ROD

P_1 = CONSTRUCTION/ERECTION DEAD LOAD OF W8X31 STEEL SET SEGMENT THAT COULD INDUCE MAXIMUM TENSION = (JACK STROKE + SHIMS) --- ATTACHMENT VI TO TIE ROD. (3-TIE RODS PER SETS FOR CALCS ONLY) WORST CASE FOR TIE ROD

$P_1 = 19(31)(1.25)$ INCREASED FOR ATTACHMENTS

$= 736.25^{lb}$

T = ROD TENSION = $P_1 / \sin\alpha$

$$T = \frac{736.25^{lb}}{\sin 10.62°} = \underline{3{,}995\ lb}$$

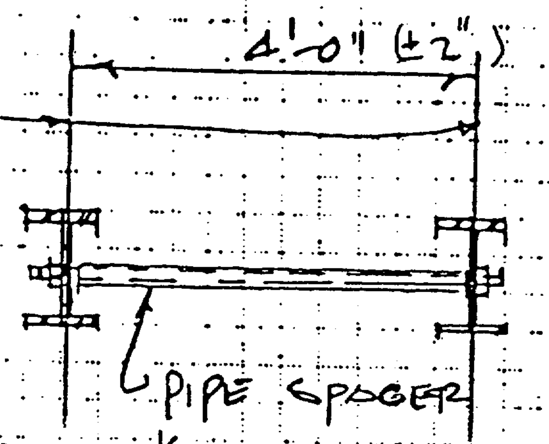
USE 3/4" DIA TIE ROD A-307 MINIMUM

T_A = ALLOWABLE TENSION FOR 3/4" A-307

$T_A = 8.8^k \rightarrow$ (AISC 4-3)
OR $20(0.334) = 6.68^k$ (AISC 4-147)

$T < T_A = 6.68^k$

USE MAXIMUM 15/16Ø OVERSIZED HOLES IN W8 OR W6 WEB FOR TIE ROD 3/4" TO AVOID THE BENDING OF RODS.



DESIGN PIPE SPACER

MAXIMUM AXIAL LOAD
TO PIPE SPACER EQUAL
TO BOLT TENSION CAPACITY = 6.68^{k}

$P_a = 6.68^{K}$

$L = 4'-4"$

TRY $1\frac{1}{2}"$ Φ PIPE SCHEDULE 40

$A = 0.799 \text{ in}^2$ } AISC P 1-93
$r = 0.623$

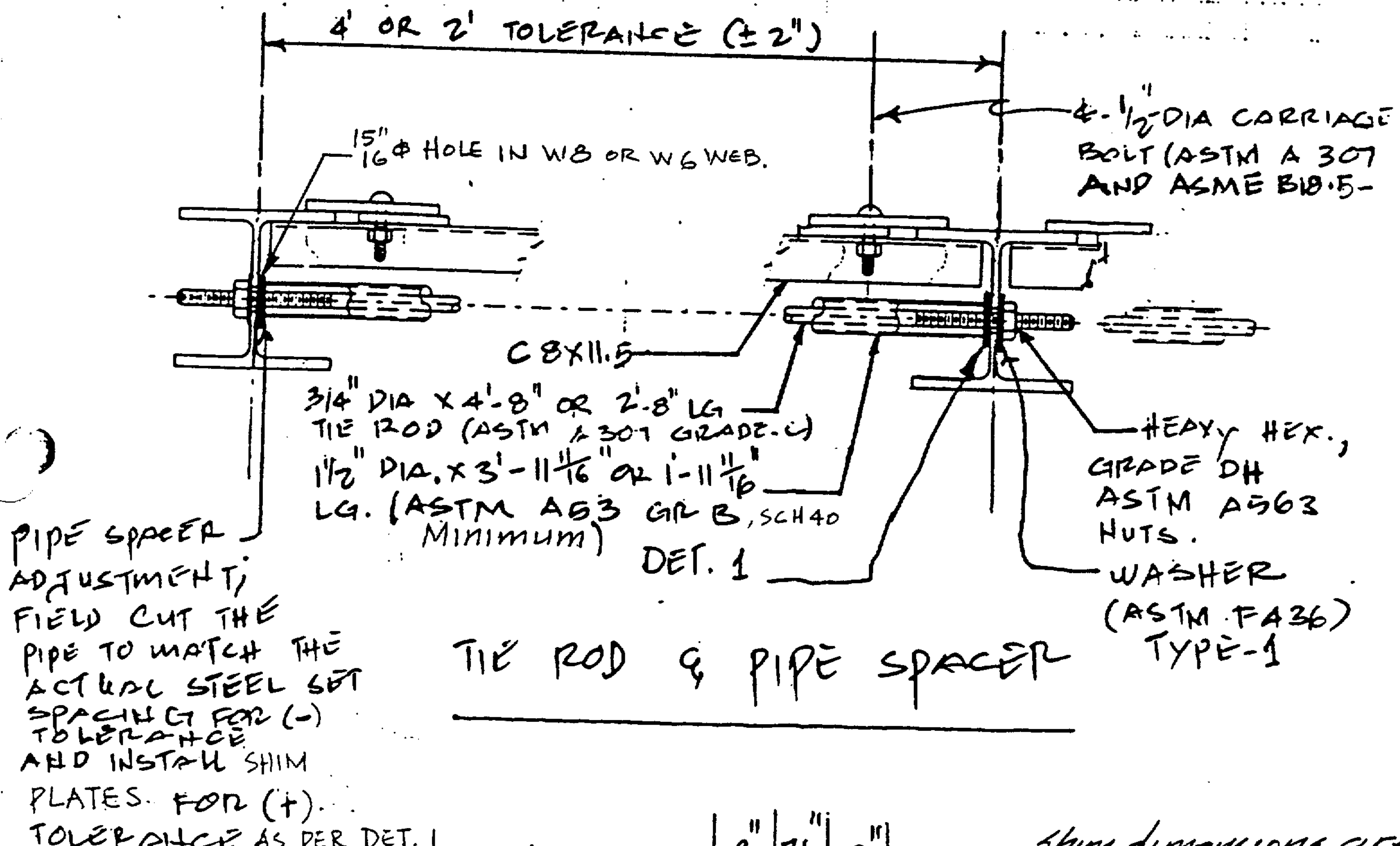
$\frac{KL}{r} = \frac{(1)\,4.33 \times 12}{0.623} = 83.4$

$\therefore F_a = 14.97^{ksi}$ ----- AISC P3-16, TABLE C-36

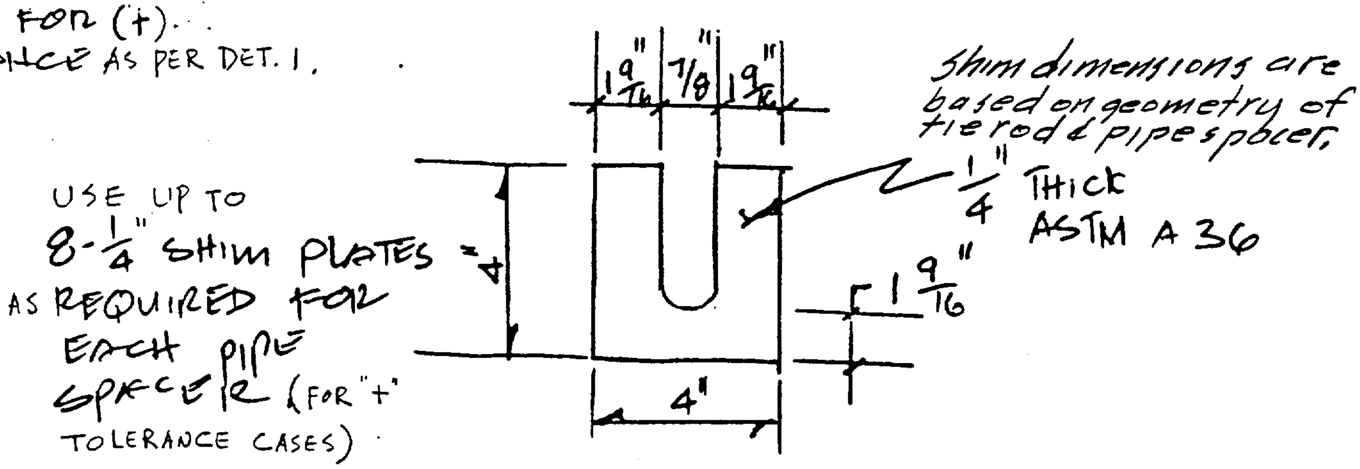
P_c = CAPACITY FOR $1\frac{1}{2}"$ Φ PIPE
$= 0.799\,(14.97) = 11.96^{k} > P_a = 6.68^{k}$

USE $1\frac{1}{2}"$Φ SCH 40 PIPE - ASTM - A 53 GR B, MIN.

SUMMARY III.D



TIE ROD & PIPE SPACER



DET. 1 (PIPE SPACER SHIM PLATES)

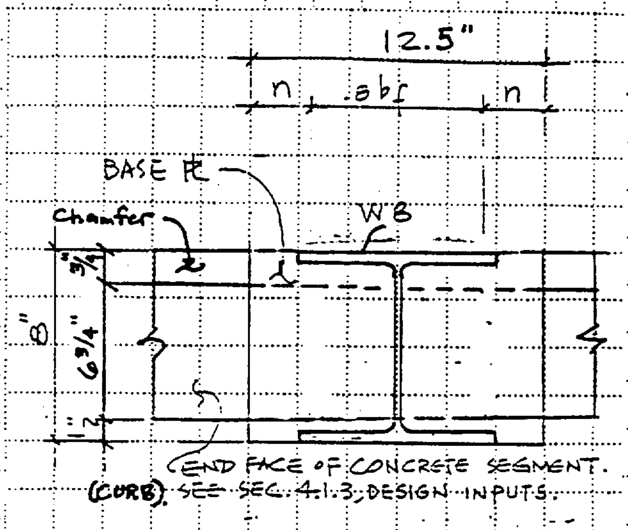
(NOT USED) Replaced by page III-65.
Previous analysis on pages III-63 & III-64 developed rectangular dimensions of the foot plate based on element axial loads ($P_a = 113.6^k$ at Node 41). Subsequent analysis use the average axial load of 121.8 from File p1okzdy.sav (see page III-108) to design the foot plate.

(NOT USED) Replaced by page III-65.

Purpose III. E Steel Set Foot Plate Calculation

Design of Foot Plate

(a) W8×31



For the W8×31, max $P_{a(ave)}$ > max P_a @ foot PL (See pages III-108 & III-78) ⇒ Design using P_a = 121.8 k (D+R+U+Seismic). Static Axial Ave = 79.36* (Page III-108) is 65% of P_a. P_a will control.

A_b ≡ Bearing Area = 12.5" × 6.75" = 84.38 in²

f_p ≡ Actual bearing pressure

$= \frac{P_a}{A_b} = \frac{121.8\,(1000)}{84.38} = 1443.6$ psi

F_p ≡ Allowable bearing pressure = $0.35 f_c'$ (AISC 3-107)

f_c' = 5000 psi (Ref. 5.21) ⇒ $F_p = 0.35(5000) = 1750$ psi

$f_p < F_p$ ⇒ OK for bearing

Thickness of plate = 3/4" to match existing design

The base plate is not fully supported by the invert curb and cannot be analyzed by the conventional AISC methods (AISC 3-106). Therefore, a finite element analysis was performed (see Attachment X) to check the actual stresses developed in the plate. As shown in Attachment X, the actual stresses developed are within the 36 ksi allowable stress:

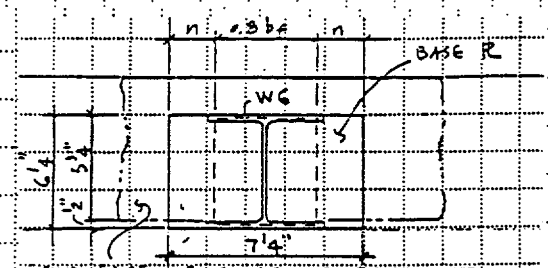
$F_b = 0.75 F_y = 0.75(36) = 27$ ksi (AISC 5-48)

Seismic increase = $1.33\bar{3} F_b = 1.33\bar{3} \times 27 = 36$ ksi

⇒ $\boxed{\text{Use } 12.5'' \times 8'' \times 3/4'' \text{ PL, ASTM A36}}$

Purpose III.E (con't)

(b) W6x20



END FACE OF CONCRETE SEGMENT, SEE SECTION 4.1.3, DESIGN INPUTS.

For the W6x20, max $P_{a(ave)}$ < max P_a @ foot ℙ (See pages III-109 & III-79) ⇒ Design using max @ foot ℙ :

$P_a = 41.96$ k (D+R+U+Seismic)

$A_b = 7.25" \times 5.75" = 41.69$ in^2

(Dimensions match connection ℙ design, pages III-102 & IX-15)

$f_p = \frac{P_a}{A_b} = \frac{41.96(1000)}{41.69} = 1006$ psi $< F_p = 1750$ psi ⇒ OK

Check bending (AISC 3-106):

$m \cong 0$; $n = (7.25 - 0.8 \times 6)/2 = 1.225"$

$t_p = 2n\sqrt{\frac{f_p}{F_y}} = 2(1.225)\sqrt{\frac{1.006}{36}} = 0.41"$ ⇒ Use 5/8" to match existing design

Since 1/3 allowable seismic increase not used, no need to check static case.

⇒ $\boxed{\text{Use } 5/8" \times 6\frac{1}{4}" \times 7\frac{1}{4}" \text{ ℙ, ASTM A36}}$

DETERMINE WELD ¼ LENGTH REQ'D AT STEEL SET WEB :

MAXIMUM SHEAR = $P_v = 7.8^k$ --- w/SEISMIC

TRY 3" ¼ OF WELD ON STEEL SET WEB.

ALLOWABLE SHEAR = $(0.3 \times 70^{ksi}) \times (0.25" \cos 45°) \times (2 \times 3") \times 1.33$

$= 29.6^k > P_v = 7.8^k$ --- O.K.

BY INSPECTION, WELD IS ADEQUATE FOR $P_v = 6.81^k$ w/o SEISMIC

1/4" IS THE MINIMUM WELD SIZE ALLOWED

AISC 5-67 TABLE J2.4

TRY 4" OF 1/4" FILLET WELD ON ONE SIDE

$(.3 \times 70)(.25 \cos 45)(\times 4") \times 1.33 = 19.74^k > 7.8^k$ OK

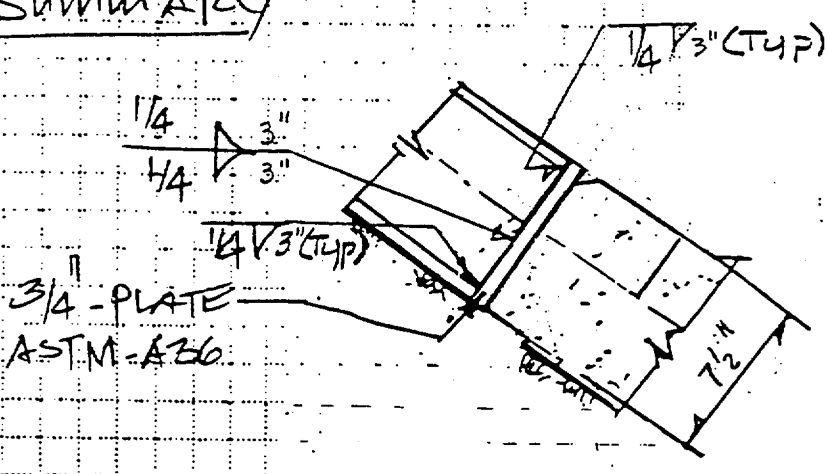
MINIMUM WELD SIZE GOVERNS

PURPOSE III. E - ALTERNATE I, (SEE ATTACHMENT IX)

SINCE PRIMARY FORCES INDUCED TO FOOT PLATE ARE COMPRESSIVE, THE WELD REQUIREMENT IS MINIMUM

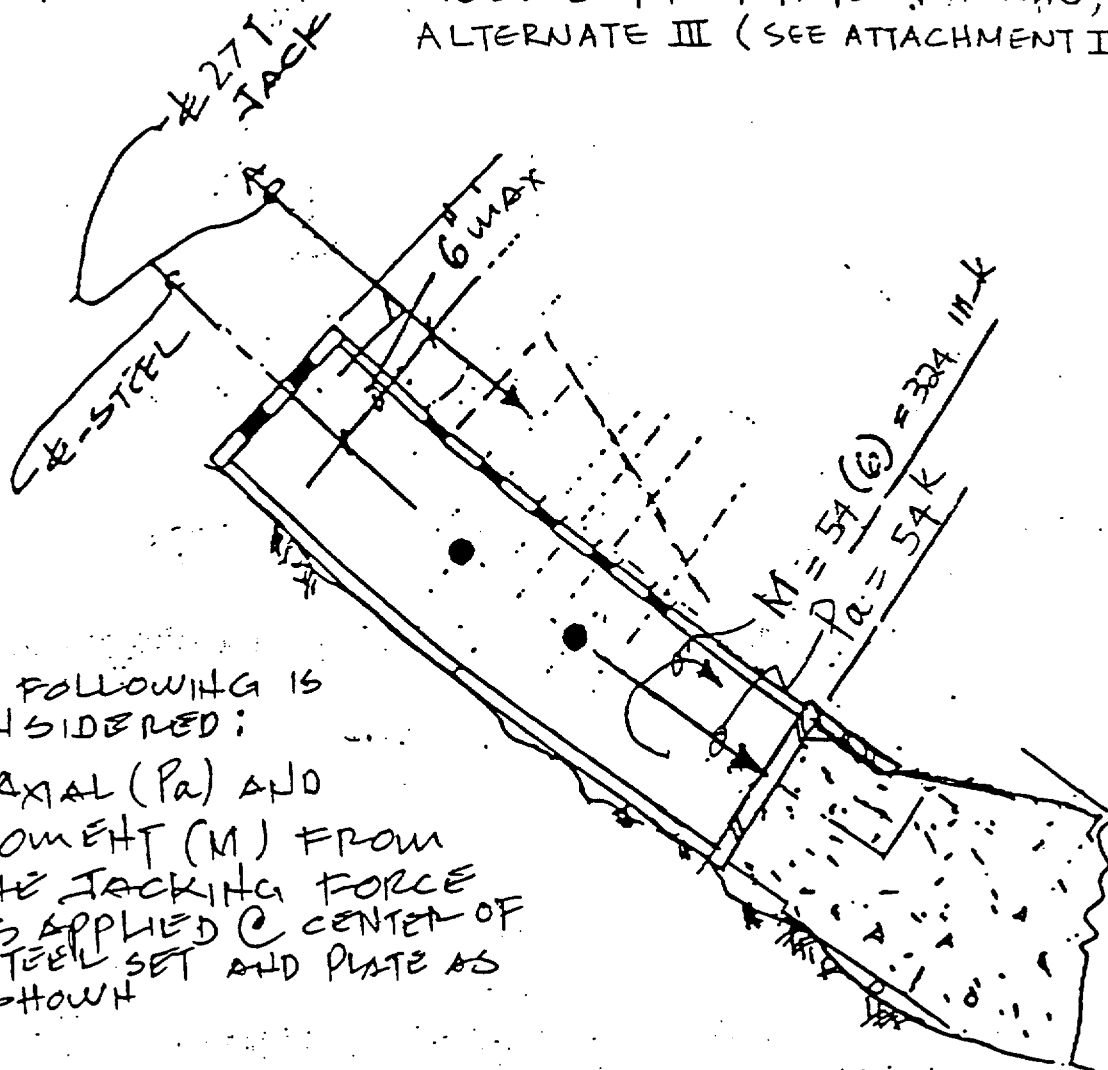
FOR WELD PLATE TO W8 USE FILLET WELDS E70XX (minimum) ELECTRODES USE 1/4" WELD AS MIN. AISC TABLE J2.4.

SUMMARY



STEEL SET FOOT PLATE
(ALTERNATE I)

PURPOSE III.E — STEEL SET FOOT PLATE DESIGN, ALTERNATE III (SEE ATTACHMENT IX)



THE FOLLOWING IS CONSIDERED:

AXIAL (Pa) AND MOMENT (M) FROM THE JACKING FORCE IS APPLIED @ CENTER OF STEEL SET AND PLATE AS SHOWN

P_a = AXIAL LOAD FROM JACKING FORCE

$= 54^{kips}$

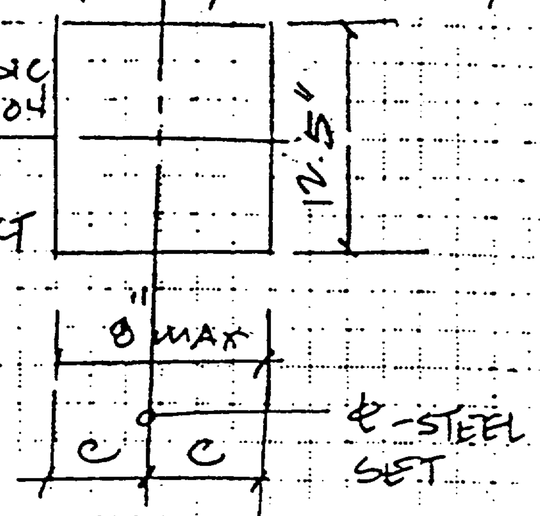
M = MOMENT FROM JACKING FORCE WITH ECCENTRICITY OF 6"

= 324 IN-K

$F_p = 0.35 (f'_c) = 0.35 (5000) = 1750$ psi — AISC P 5-79

Fp = ALLOWABLE BEARING CAPACITY FOR F'c=500 psi CONCRETE.

$$\boxed{f_p = \frac{P_{au}}{A} \pm \frac{Mc}{I}}$$ → BASIC EQUATION



A = FOOT PLATE AREA IN CONTACT WITH CONCRETE SEGMENT.

I = MOMENT OF INERTIA FOR FOOT PLATE / RESPECT TO ℄-STEEL SET.

TRIAL DIMENSION BASE

TRY 8" x 12.5" PLATE ✓

$A = 6\frac{3}{4} \times 12.5 = 84.38 \, in$

$C = 4"$

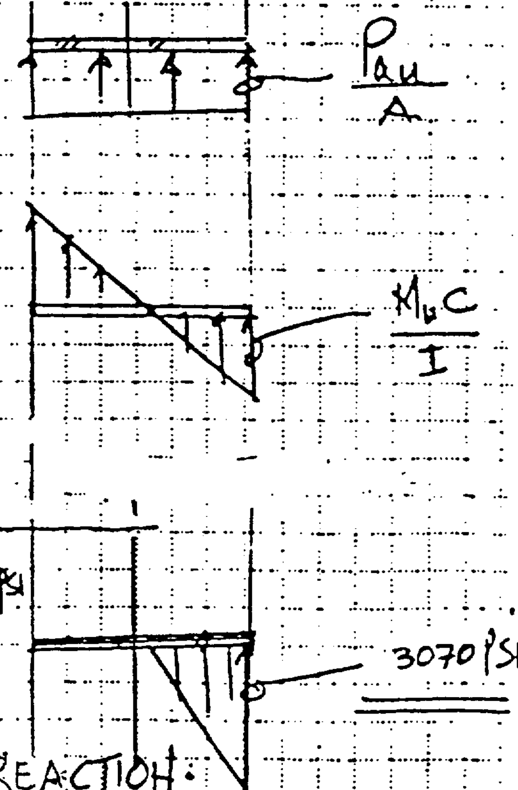
$I = \frac{12.5(8)^3}{12} = 533.33 \, in^4$

$f_p = \frac{54}{84.38} \pm \frac{324 \times (4)}{533.33}$

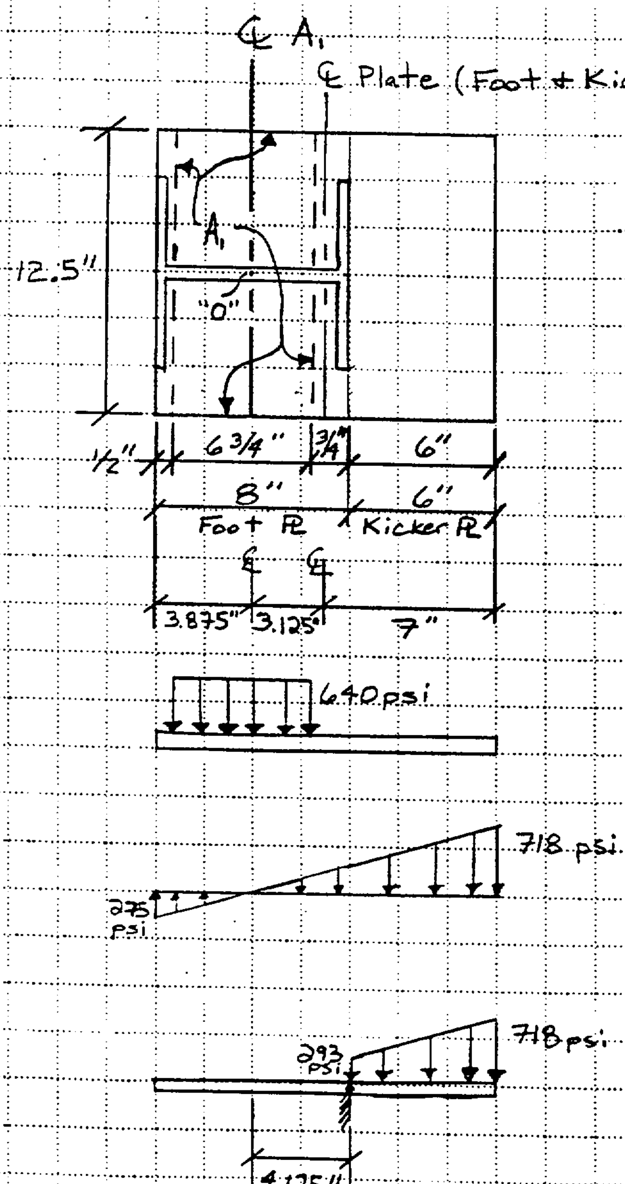
$= 0.64 \pm 2.43$

$= 3.07 \, ksi = 3070 \, psi > F_p = 1750 \, psi$

N.G



REVISE TRIAL DIMENSION TO INCLUDE VERTICAL PLATE REACTION.



A_1 = Bearing Area
$= (6\,^3/_4)(12.5) = 84.38\ in^2$

$P_a = 54\ k$

$\frac{P_A}{A_1} = \frac{54}{84.38} = 0.64\ ksi$

I (about ℄ of A_1 @ "0")

$I = \frac{12.5(14)^3}{12} + (12.5)(14)(3.125)^2$
$= 2858.33 + 1708.98$
$= 4567.31\ in^4$

$c_1 = 3.875"\quad c_2 = 10.125"$
$M = 324\ in\text{-}k$

$\frac{Mc_1}{I} = \frac{324(3.875)}{4567.31} = 0.275\ psi$

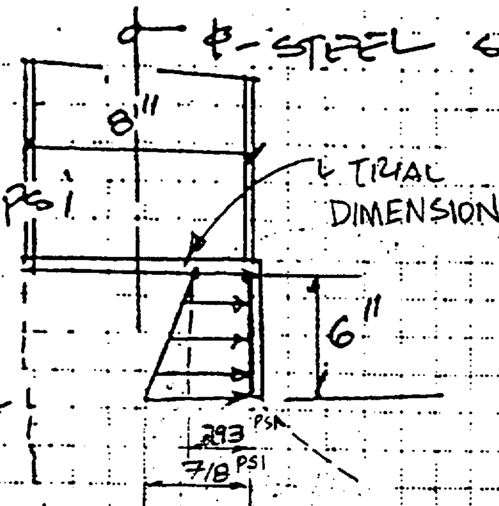
$\frac{Mc_2}{I} = \frac{324(10.125)}{4567.31} = 0.718\ psi$

Trap. loading for design of kicker PL; from similar Δs,

$\frac{718}{10.125} = \frac{x}{4.125} \Rightarrow x = 293\ psi$

∴ MAXIMUM PRESSURE ON CONCRETE EQUAL TO

718 PSI MAXIMUM < F_p=1750 PSI



DESIGN VERTICAL PLATE

M_{MAX} = MOMENT @ TOP OF PLATE

$= 293(6)(3) + \frac{(718-293)}{2}(6)(4)$

$= 5274 + 5100$

$= 10374$ IN-lb

F_b = 27 KSI ALLOWABLE FOR PLATE (AISC 5-48, F2-1)

$S_x = \frac{bt^2}{6} = \frac{12.5t^2}{6} = 2.08\bar{3}t^2$

$F_b = \frac{M}{S_x} \Rightarrow S_x = \frac{M}{F_b} \Rightarrow 2.08\bar{3}t^2 = \frac{10374}{27000} = 0.384$

$\Rightarrow t = \underline{0.429"}$

⇒ Use 1/2" plate to match existing design

No stiffener plates are required; however, to match existing drawings, the 2 1/2" stiffeners shown are OK.

Check weld size :

Check $\overset{3/4}{\underset{1/4}{}}$ weld:

Max Shear on Weld $= \frac{293 + 718}{2}(6)(12.5) / 1000$

$= 37.91$ k

Weld Effective area $= (1/4 \times 0.707)(2)(12.5)$

$= 4.42 \text{ in}^2$

Weld Capacity $= (0.3)(70)(4.42)$ [AISC 5-70]

$= 92.8$ k > 37.9 k $\Rightarrow$ OK

(NOT USED) Stiffners not required.

(NOT USED) Stiffners not required.

(NOT USED) Stiffners not required.

PURPOSE III. E. – ALTERNATE III (SEE ATTACHMENT IX-16)

SINCE PRIMARY FORCES INDUCED TO FOOT PLATE ARE COMPRESSIVE, THE WELD REQUIREMENT IS MINIMUM.

FOR WELD PLATE TO W8 USE FILLET WELDS E70XX MINIMUM ELECTRODES

USE 1/4" WELD AS MIN. AISC TABLE J2.4.

SUMMARY

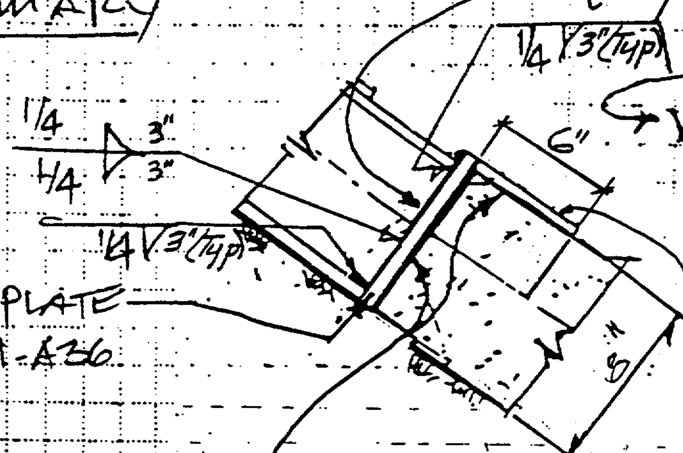
THIS PLATE IS ENLARGED FROM 3/4" x 8" x 12.5" TO 3/4" x 8.5" x 12.5" TO ACCOMODATE.

Vertical Plate REQD FOR JACKING LOAD.

WELD ELECTRODES SHALL BE E70XX MIN.

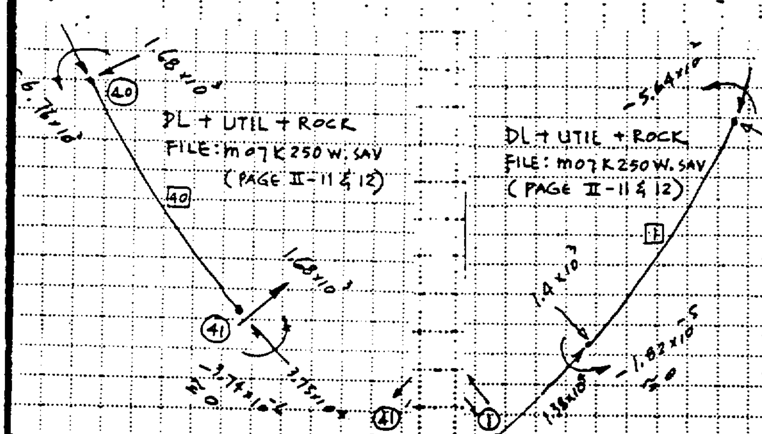
1/2" x 6" x 12.5" Plate

3/4" - PLATE ASTM-A36



THESE PLATE SURFACES SHALL BE WITH POSITIVE CONTACT TO CONCRETE DURING JACKING.

SLIDING STABILITY:



REFERING TO PAGE III-113, THE CRITICAL SLIDING RESISTANCE AT SUPPORT / FOOT PLATE IS AT RUN № 11. ON PAGE III-113, THE SLIDING RESISTANCE WAS CALCULATED CONSERVATIVELY BY AVERAGING FORCES IN ELEMENT 40 AND 41. ACTUAL CRITICAL FORCES CAN BE FOUND AT NODE 41 (THE FOOT PLATE) AND ARE REPRODUCED ON THE LEFT. SLIDING STABILITY RATIO

$$= \frac{0.3\,(1.38\times10^4)}{1.4\times10^3} = 2.96 > 1.0 \quad \text{OK}$$

PURPOSE III E (CONT.)

TABLE 1A. SEISMIC COMPUTER RUN IDENTIFIERS (ATTACHMENT II)

RUN	FILENAME	ROCK TYPE	STATION	MEMBER	SPACING	LOADING[1]
1	p10k2dy.sav	PTn	10+00	W8x31	2 ft	D+U+R+E
2	p07k2dy.sav	TCW	07+00	W8x31	4 ft	D+U+R+E
3	p18k2dy.sav	TSW_1	18+00	W8x31	4 ft	D+U+R+E
4	p34k2dyx.sav	TSW_2	34+00	W8x31	2 ft	D+U+R+E
5	m07_k2dy.sav	TCW	07+00	W6x20	4 ft	D+U+R+E
6	m18_k2dy.sav	TSW_1	18+00	W6x20	4 ft	D+U+R+E
7	m27_k2dy.sav	TSW	27+00	W6x20	4 ft	D+U+R+E
8	m34_k2dy.sav	TSW_2	34+00	W6x20	4 ft	D+U+R+E
9	m53_k2dy.sav	TSW_2	53+00	W6x20	4 ft	D+U+R+E

[1] Dead Load (D) + Utility Loads (U) + Rock Load (R) + Seismic Load (E)

TABLE 1B. STATIC COMPUTER RUN IDENTIFIERS (ATTACHMENT II)

RUN	FILENAME	ROCK TYPE	STATION	MEMBER	SPACING	LOADING[2]
10	m10k250w.sav	PTn	10+00	W8x31	2 ft	D+U +R
11	m07k250w.sav	TCW	07+00	W8x31	4 ft	D+U +R
12	m18k250w.sav	TSW_1	18+00	W8x31	4 ft	D+U +R
13	m34k250x.sav	TSW_2	34+00	W8x31	2 ft	D+U +R
14	m07_k2.sav	TCW	07+00	W6x20	4 ft	D+U +R
15	m18_k2.sav	TSW_1	18+00	W6x20	4 ft	D+U +R
16	m27_k2.sav	TSW_2	27+00	W6x20	4 ft	D+U +R
17	m34_k2.sav	TSW_2	34+00	W6x20	4 ft	D+U +R
18	m53_k2.sav	TSW_2	53+00	W6x20	4 ft	D+U +R

[2] Dead Load (D) + Utility Loads (U) + Rock Load (R) (No seismic)

Purpose III-E (Cont.)

TABLE 2A. SUMMARY OF REACTIONS AT BASE OF STEEL SETS[1] (SEISMIC LOAD CASE -W8x31)

RUN	ELEM.	NODE[5]		FORCES (SI UNITS)[2]		CONVER. FACTOR[3]	FORCES (ENGLISH UNITS)[4]	
		Nod1	Nod2	F-Shear	F-Axial		F-Shear	F-Axial
1	48	48	01	+2.946E+04	+5.693E+05	1.3704E-04	4.04	78.02
	01	01	02	-1.025E+04	+7.314E+05	"	-1.40	100.23
	40	40	41	+1.350E+04	+8.288E+05	"	1.85	113.58
	41	41	42	-2.737E+04	+6.597E+05	"	-3.75	90.41
2	48	48	01	+6.615E+03	+4.946E+04	2.7409E-04	1.81	13.56
	01	01	02	+1.695E+03	+9.691E+04	"	0.46	26.56
	40	40	41	-1.342E+03	+1.048E+05	"	-0.37	28.72
	41	41	42	+6.926E+03	+7.383E+04	"	1.89	20.24
3	48	48	01	+1.809E+04	+2.272E+05	2.7409E-04	4.96	62.27
	01	01	02	+5.482E+03	+2.344E+05	"	1.50	64.25
	40	40	41	-5.996E+03	+2.688E+05	"	-1.64	73.68
	41	41	42	-1.776E+04	+3.082E+05	"	-4.87	84.47
4	48	48	01	+4.106E+04	+2.219E+05	1.3704E-04	5.63	30.41
	01	01	02	+5.026E+03	+3.794E+05	"	0.69	52.00
	40	40	41	-3.537E+03	+4.062E+05	"	-0.48	55.67
	41	41	42	-2.736E+04	+2.459E+05	"	-3.75	33.70

FOOOTNOTES:

1. From Computer Runs 1 thru 4 identified in Table 1A (Attachment II)
2. SI Units = Newtons
3. Conversion Factor = F(newtons) x Spacing(ft)/3.28084(ft/m) x 0.22481(lbf/newton) x 0.001(kip/lbf) = F(kip)
4. English Units = Kips
5. Pinned Nodes (Moment = 0) are at Nodes 01 & 41

Purpose III E (Cont.)

TABLE 2B. SUMMARY OF REACTIONS AT BASE OF STEEL SETS[1] (SEISMIC LOAD CASE -W6x20)

RUN	ELEM.	NODE[5]		FORCES (SI UNITS)[2]		CONVER. FACTOR[3]	FORCES (ENGLISH UNITS)[4]	
		Nod1	Nod2	F-Shear	F-Axial		F-Shear	F-Axial
5	48	48	01	+9.245E+03	+2.151E+04	2.7409E-04	2.53	5.90
	01	01	02	-1.525E+02	+3.071E+04	"	-0.04	8.42
	40	40	41	+1.162E+02	+3.195E+04	"	0.03	8.76
	41	41	42	-4.998E+03	+2.083E+04	"	-1.37	5.71
6	48	48	01	+2.114E+04	+1.051E+05	2.7409E-04	5.79	28.81
	01	01	02	+2.872E+02	+5.643E+04	"	0.08	15.47
	40	40	41	-2.530E+02	+7.635E+04	"	-0.07	20.93
	41	41	42	-2.202E+04	+1.531E+05	"	6.04	41.96
7	48	48	01	+2.517E+04	+1.300E+04	2.7409E-04	6.90	3.56
	01	01	02	+8.836E+02	+7.251E+04	"	0.24	19.87
	40	40	41	-1.020E+03	+8.315E+04	"	-0.28	22.79
	41	41	42	-2.845E+04	+3.529E+04	"	-7.80	9.67
8	48	48	01	+1.236E+04	+9.559E+03	2.7409E-04	3.39	2.62
	01	01	02	+9.089E+02	+7.749E+04	"	0.25	21.24
	40	40	41	-7.561E+02	+8.131E+04	"	-0.21	22.29
	41	41	42	-6.258E+03	+7.554E+04	"	-1.72	20.70
9	48	48	01	+2.456E+04	+1.328E+04	2.7409E-04	6.73	3.64
	01	01	02	+9.359E+02	+7.082E+04	"	0.26	19.41
	40	40	41	-5.696E+02	+7.827E+04	"	-0.16	21.45
	41	41	42	-2.250E+04	+4.287E+04	"	-6.17	11.75

FOOOTNOTES:
1. From Computer Runs 5 thru 9 identified in Table 1A (Attachment II)
2. SI Units = Newtons
3. Conversion Factor = F(newtons) x Spacing(ft)/3.28084(ft/m) x 0.22481(lbf/newton) x 0.001(kip/lbf) = F(kip)
4. English Units = Kips
5. Pinned Nodes (Moment = 0) are at Nodes 01 & 41

Purpose III-E (cont.)

TABLE 2C. SUMMARY OF REACTIONS AT BASE OF STEEL SETS[1] (STATIC LOAD CASE - W8x31)

RUN	ELEM.	NODE[5]		FORCES (SI UNITS)[2]		CONVER. FACTOR[3]	FORCES (ENGLISH UNITS)[4]	
		Nod1	Nod2	F-Shear	F-Axial		F-Shear	F-Axial
10	48	48	01	+1.805E+04	+3.410E+05	1.3704E-04	2.47	46.73
	01	01	02	-7.099E+03	+4.667E+05	"	-0.97	63.96
	40	40	41	+1.246E+04	+4.875E+05	"	1.71	66.81
	41	41	42	-1.513E+04	+3.543E+05	"	-2.07	48.55
11	48	48	01	-1.207E+03	-5.178E+04	2.7409E-04	-0.33	-14.19[6]
	01	01	02	-1.403E+03	+1.379E+04	"	-0.38	3.78
	40	40	41	-1.682E+03	+3.749E+04	"	-0.46	10.28
	41	41	42	-1.576E+04	-2.465E+04	"	-4.32	- 6.76[6]
12	48	48	01	+1.655E+04	+1.207E+05	2.7409E-04	4.54	33.08
	01	01	02	+2.345E+03	+1.462E+05	"	0.64	40.07
	40	40	41	-1.143E+03	+1.442E+05	"	-0.31	39.52
	41	41	42	-1.358E+04	+1.309E+05	"	-3.72	35.87
13	48	48	01	+3.282E+04	+1.111E+05	1.3704E-04	4.50	15.23
	01	01	02	+2.449E+03	+2.539E+05	"	0.34	34.80
	40	40	41	-2.068E+03	+2.647E+05	"	-0.28	36.28
	41	41	42	-1.538E+04	+1.082E+05	"	-2.11	14.82

FOOOTNOTES:

1. From Computer Runs 10 thru 13 identified in Table 1B (Attachment II)
2. SI Units = Newtons
3. Conversion Factor = F(newtons) x Spacing(ft)/3.28084(ft/m) x 0.22481(lbf/newton) x 0.001(kip/lbf) = F(kip)
4. English Units = Kips
5. Pinned Nodes (Moment = 0) are at Nodes 01 & 41
6. Negative axial forces (tension) for Run No. 11 indicate that the rock is moving away from the steel set/invert & are a result of modelling assumptions in FLAC runs to provide conservative rock loads. In reality, the steel set remains in compression at all times.

PURPOSE III.E (CONT.)

TABLE 2D. SUMMARY OF REACTIONS AT BASE OF STEEL SETS[1] (STATIC LOAD CASE - W6x20)

RUN	ELEM.	NODE[5]		FORCES (SI UNITS)[2]		CONVER. FACTOR[3]	FORCES (ENGLISH UNITS)[4]	
		Nod1	Nod2	F-Shear	F-Axial		F-Shear	F-Axial
14	48	48	01	+4.352E+03	-1.685E+04	2.7409E-04	1.19	- 4.62[6]
	01	01	02	-2.331E+02	+1.043E+04	"	-0.06	2.86
	40	40	41	+2.307E+02	+9.822E+03	"	0.06	2.69
	41	41	42	-7.839E+02	-1.633E+04	"	-0.21	-4.48[6]
15	48	48	01	+1.645E+04	+4.356E+04	2.7409E-04	4.51	11.94
	01	01	02	+5.369E+01	+3.370E+04	"	-0.01	9.24
	40	40	41	+1.172E+02	+3.892E+04	"	-0.03	10.66
	41	41	42	-1.879E+04	+4.925E+04	"	-5.15	13.50
16	48	48	01	+2.040E+04	-2.092E+04	2.7409E-04	5.59	-5.73[6]
	01	01	02	+8.914E+02	+5.366E+04	"	0.24	14.71
	40	40	41	-8.040E+02	+5.721E+04	"	-0.22	15.68
	41	41	42	-2.483E+04	-2.027E+04	"	-6.81	-5.56[6]
17	48	48	01	+8.100E+03	-2.764E+04	2.7409E-04	2.22	-7.58[6]
	01	01	02	+8.765E+02	+5.838E+04	"	0.24	16.00
	40	40	41	-5.700E+02	+5.361E+04	"	-0.16	14.69
	41	41	42	-1.458E+03	+2.165E+04	"	-0.40	5.93
18	48	48	01	+2.058E+04	-2.727E+04	2.7409E-04	5.64	-7.47[6]
	01	01	02	+8.523E+02	+5.076E+04	"	0.23	13.91
	40	40	41	-4.445E+02	+5.212E+04	"	-0.12	14.29
	41	41	42	-1.820E+04	-1.464E+04	"	-4.99	-4.01

FOOOTNOTES:

1. From Computer Runs 14 thru 18 identified in Table 1B (Attachment II)
2. SI Units = Newtons
3. Conversion Factor = F(newtons) x Spacing(ft)/3.28084(ft/m) x 0.22481(lbf/newton) x 0.001(kip/lbf) = F(kip)
4. English Units = Kips
5. Pinned Nodes (Moment = 0) are at Nodes 01 & 41
6. Negative axial forces (tension) for Runs No. 14 & 16-18 indicate that the rock is moving away from the steel set/invert & are a result of modelling assumptions in the FLAC runs to provide conservative rock loads. In reality, the steel set remains in compression at all times.

STEEL SET FOOT PLATE OFFSET FROM FACE OF INVERT CURB.

MAX. OFFSET REQUIRED FOR BEARING (SEE SKETCH BELOW)

$P_u = P_R = 121.8^k$ (PAGE III-108). { USE UNFACTORED LOAD SINCE SEISMIC LOAD BASED ON DYNAMIC ANALYSIS & CONSERVATIVE "DBE" ZPA OF 0.37g H&V WHICH IS > UBC SEISMIC (BY APPROX 2X

$P_u \le \phi P_n$ = BEARING STRENGTH $\le \phi\,(.85) f_c' A_1$ ACI 318, SECT. 10.15

$\phi = 0.70$ ACI 318, SECT. 9.3.2.4

$f_c' = 5000$ psi $A_1 = [6.75 - (X - 0.5)](12.5)$
$= (7.25 - X)(12.5)$

$P_u = \phi P_n = 121.8^k \le \underbrace{0.70\,(.85)(5000)}_{2.975\text{ksi}}[(7.25-X)(12.5)]$

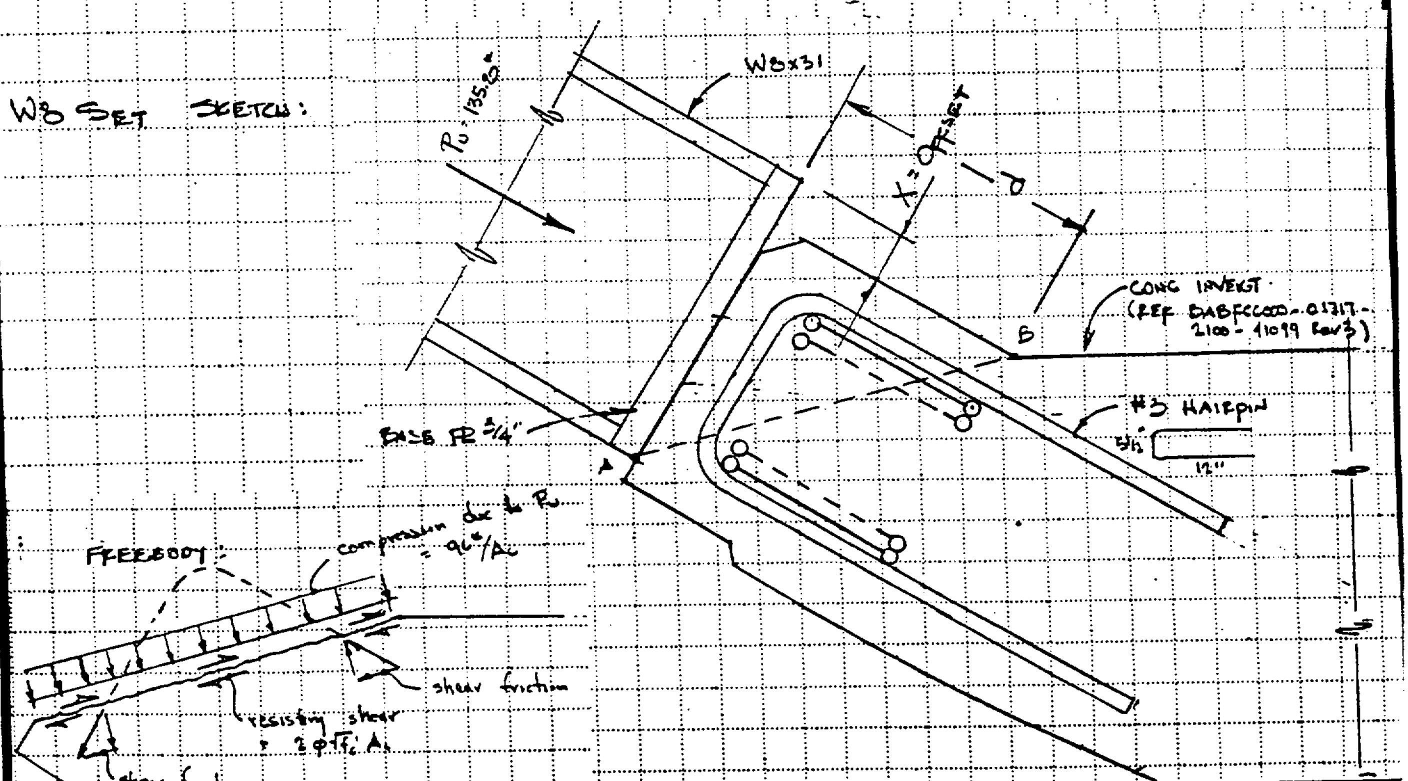
$121.8^k \le 2.975\,(7.25 - X)(12.5)$

$\frac{121.8}{2.975(12.5)} = 3.275'' = 7.25 - X$

$X = 7.25 - 3.275 = 3.975$ SAY $\underline{\underline{4''}}$

CHECK, $\frac{121.8}{(8-4.0-.75)(12.5)} = 2.998$ ksi brng pressure ≈ 2.975 ∴ assume OK

W8 SET SKETCH:



FOOT PLATE OFFSET (CONTINUED)

SHEAR FRICTION (REF ACI 318, SECTION 11.7):

FOR BOTH E & F, $\alpha = 45°$. ASSUMING #3 HAIRPIN ACTS AS SHEAR-FRICTION REINFORCEMENT, $A_{vf} = 0.11\ in^2 \times 2 = 0.22\ in^2$

ASSUME EFFECTIVE WIDTH OF CURB (IN SHEAR) IS $\cong b + 2d$ so $W_{eff} = 12.5 + 2(6.25) = 25''$ WHICH IS APPROX $\frac{1}{2}$ OF INVERT WIDTH. CONSERVATIVELY USE THIS WIDTH (DON'T FACTOR e 45°).

FOR SHEAR FRICTION REINF. ALONE (2 #3 BARS):

$V_n = A_{vf} f_y (\mu \sin \alpha_f + \cos \alpha_f)$ (ACI 318, SECTION 11.7.4.2)

$= (0.22)(60)(1.4(1.0) \sin 45° + \cos 45°)$

$= 13.2^k (.990 + .707) = 22.4^k << 121.8^k$ ∴ ALONE, A_{vf} CAN'T RESIST SHEAR.

FOR SHEAR ON SECTION, $V_c = 2\left(1 + \frac{N_u}{2000 A_g}\right)\sqrt{f_c'}\,A_c$ (ACI 318, SECTION 11.3.1.2)

ASSUME $1\frac{1}{4}''$ OFFSET (SELECTED TO HAVE FAILURE PLANE ABOVE PT B):

$A_c \cong 6.5''\left(\frac{1}{\sin 45°}\right)(25'') + \underbrace{\tfrac{1}{2}(6.5'')(6.5)(2)\left(\tfrac{1}{\cos 45°}\right)}_{\text{AT SIDES OF SHEAR "CONE"}} = 289.6\ in^2 = A_g$

$V_c = 2\left(1 + \frac{N_u}{2000 A_g}\right)\sqrt{f_c'}$ w/ $N_u = 121.8^k$ $A_g = 289.6$

$= 2\left(1 + \frac{121.8^k}{2000(289.6)}\right)\sqrt{f_c'} = 0.171$ ksi

$V_{c_1} = 0.171(289.6) = 49.5^k$

ADD NET COMPRESSION ACROSS SECTION TO V_c

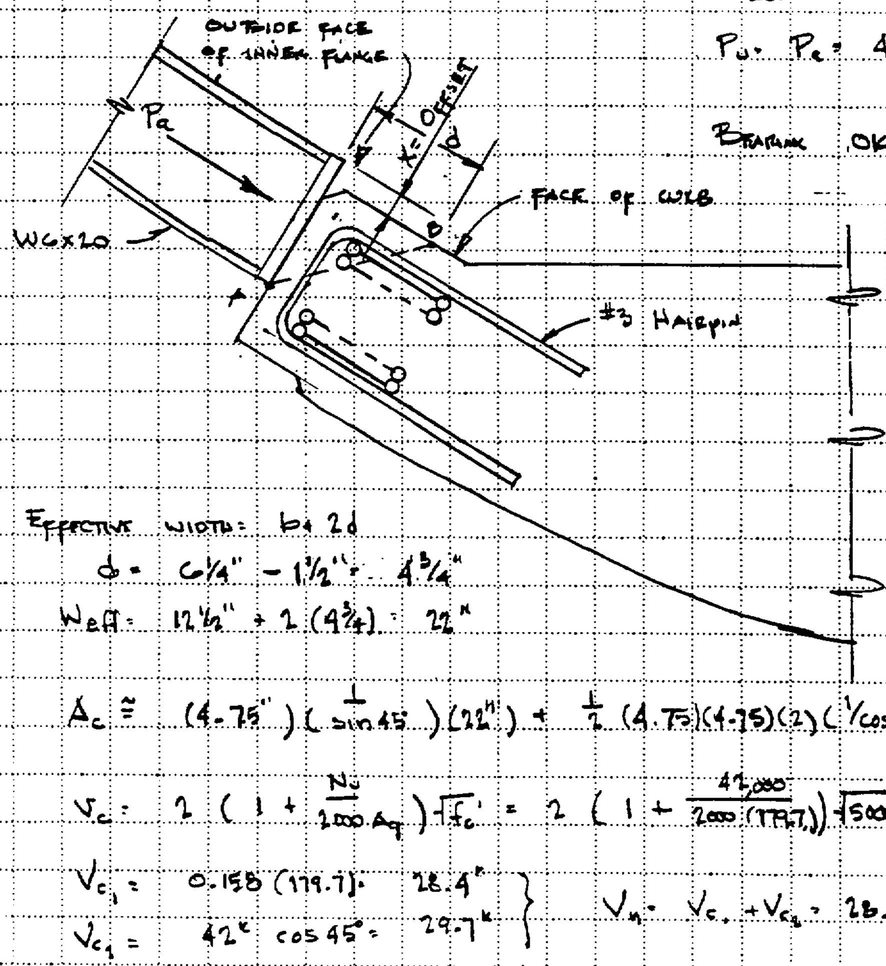
$V_{c_2} = 121.8^k \cos 45 = 86.1^k$ ... (ACI 318, SECT 11.7.7)

$V_n = 49.5^k + 86.1^k = 135.6^k$

$\phi V_n = 0.85(135.6) = 115.3^k \cong V_u = 121.8$ (5% UNDER)

∴ MAX. OFFSET = $1\frac{1}{4}''$ (95%) $\cong 1\frac{1}{8}''$ ∴ USE $\underline{\underline{1''}}$ MAX. OFFSET

FOOT PLATE OFFSET (CONTINUED)



CHECK W6 STEEL SET

$P_u = P_a = 41.96^k$ (w/ SEISMIC) say $= 42^k$

BEARING OK BY INSPECTION

START BY ASSUMING:
OFFSET = X = $1\frac{1}{2}''$

CHECK SHEAR ON SECTION A-B

FROM PREVIOUS PAGE, SHEAR-FRICTION REINF. IS NOT SUFFICIENT.

EFFECTIVE WIDTH: $b + 2d$

$d = 6\frac{1}{4}'' - 1\frac{1}{2}'' = 4\frac{3}{4}''$

$W_{eff} = 12\frac{1}{2}'' + 2(4\frac{3}{4}) = 22''$

$$A_c \cong (4.75'')\left(\frac{1}{\sin 45}\right)(22'') + \frac{1}{2}(4.75)(4.75)(2)(1/\cos 45°) = 179.7\ in^2$$

$$V_c = 2\left(1 + \frac{N_u}{2000 A_g}\right)\sqrt{f_c'} = 2\left(1 + \frac{42{,}000}{2000\,(179.7)}\right)\sqrt{5000} = 0.158\ ksi$$

$$\left.\begin{array}{l} V_{c_1} = 0.158\,(179.7) = 28.4^k \\ V_{c_2} = 42^k \cos 45° = 29.7^k \end{array}\right\} \quad V_n = V_{c_1} + V_{c_2} = 28.4 + 29.7 = 58.1^k$$

$$\phi V_n = 0.85\,(58.1) = 49.4^k > 42.0^k \quad \therefore \ 1\frac{1}{2}'' \text{ OFFSET OK FOR W6}$$

SUMMARY:

BASED ON SHEAR FAILURE OF THE INVERT CURB, AN ALLOWANCE OF $\underline{\underline{1''}}$ WILL BE SPECIFIED FOR THE MAX. OFFSET OF THE BASEPLATE TOWARD THE TUNNEL CENTERLINE FOR BOTH THE W6 & W8 STEEL SETS. OFFSET TO BE MEASURED FROM FACE OF INVERT CURB TO OUTSIDE FACE OF INNER FLANGE OF THE W6 OR W8.

PURPOSE III.F A) STEEL SET FOOT SEGMENT CALCULATION FOR W 8x31 FOOT SEGMENT

DESIGN STEEL SET-FOOT SEGMENT

W8x31 IS "OK" FOR ROCK LONG TERM LOADS (SEE COMPUTER OUTPUT).

CHECK THE CONSTRUCTION LOADS - USING 25 TONS JACK AND 27 TONS DESIGN LOAD.

$P_a = 27^T \times 2^{k/T}$

$P_a = 54\ k$

$M = \frac{54(6)}{12}$

$= 27$ FT-K

FOR W8x31

$A_1 = 9.13\ in^2$

$I_1 = 110\ in^4$, $S = 27.5\ in^3$

$f_{bx} = \frac{27^{1K} \times 12}{27.5} = 11.78^{KSI}$

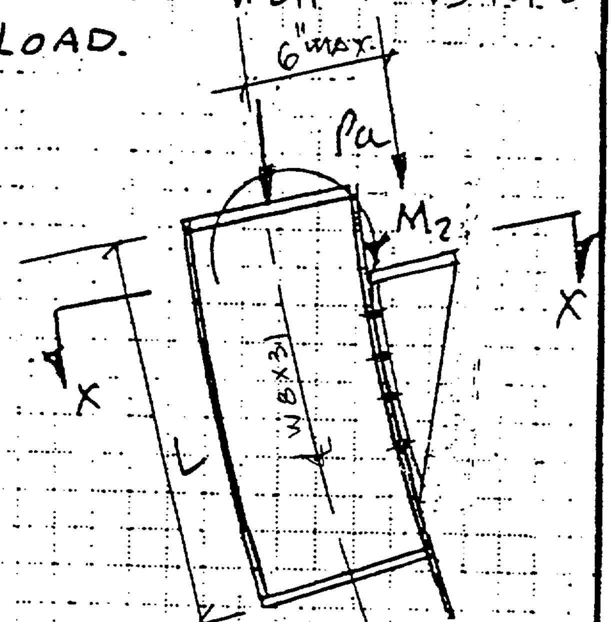
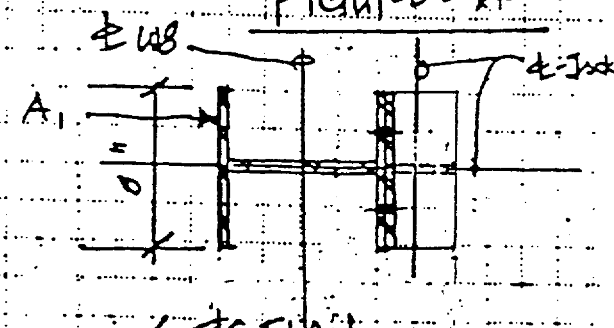


FIGURE-A



SECTION X-X

PURPOSE III. F

W8x31 FOOT SEGMENT

DESIGN PARAMETERS

$L = 12'-0''$ MAX (TRIAL DIMENSION)

$A = 9.13 \, in^2$, $I = 110.0 \, in^4$,

$r_x = 3.47$

$r_y = 2.02$

$$f_a = \frac{P_a}{A} = \frac{54.00}{9.13} = 5.91 \text{ ksi}$$

$$\frac{KL}{r_y} = \frac{(1)(12 \times 12)}{2.02} = 71.3 \qquad F_a = 16.3 \text{ ksi}$$

(AISC TABLE C-36)

$$f_a/F_a = \frac{5.91}{16.3} = 36.3\% > 15\% \quad \text{CHECK AISC (H1-1\&2)}$$

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} \leq 1.0 \text{ ---- AISC (H1-1)}$$

$$K l_b / r_b = \frac{1.0 \times 144}{3.47} = 41.5 \quad F'_{ex} = 86.7 \text{ --- AISC P5-122}$$

$$0.336 + \frac{1.0 \times \cancel{11.78}\ 0.585}{\left(1 - \frac{5.91}{86.7}\right) \times 21.6} = 0.875 < 1.0 \text{ ---- O.K.}$$

PURPOSE III.F FOR W8X31 FOOT SEGMENT

$$\frac{f_a}{0.6F_y} + \frac{f_{b_x}}{F_{b_x}} \text{ ---- AISC (H1-2)}$$

$$\frac{5.91}{21.6} + \frac{11.78}{21.6} = 0.82 < 1.0 \text{ --- O.K.}$$

- USE W8X31 WITH MINIMUM JACKING BRACKET AS SHOWN ON FIGURE A AND MAXIMUM LENGTH 12'-0".

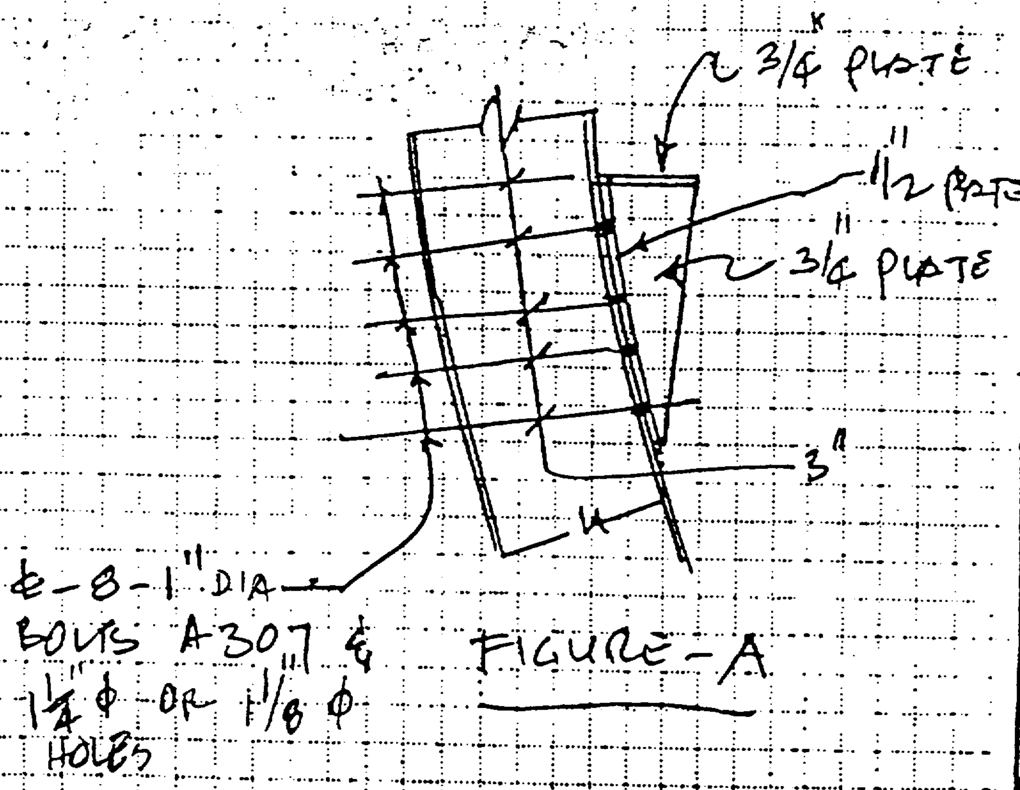
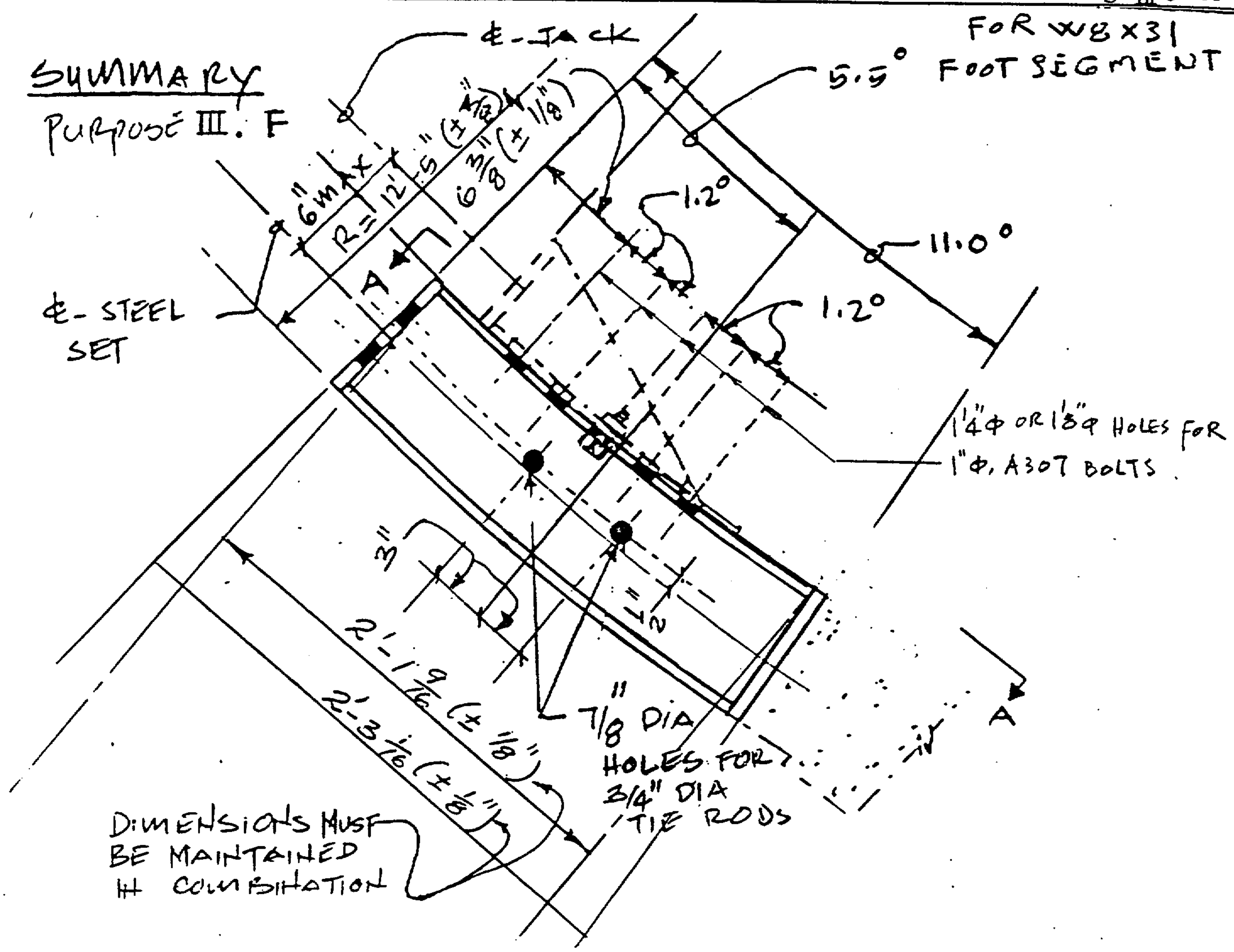


FIGURE-A

SUMMARY
PURPOSE III. F

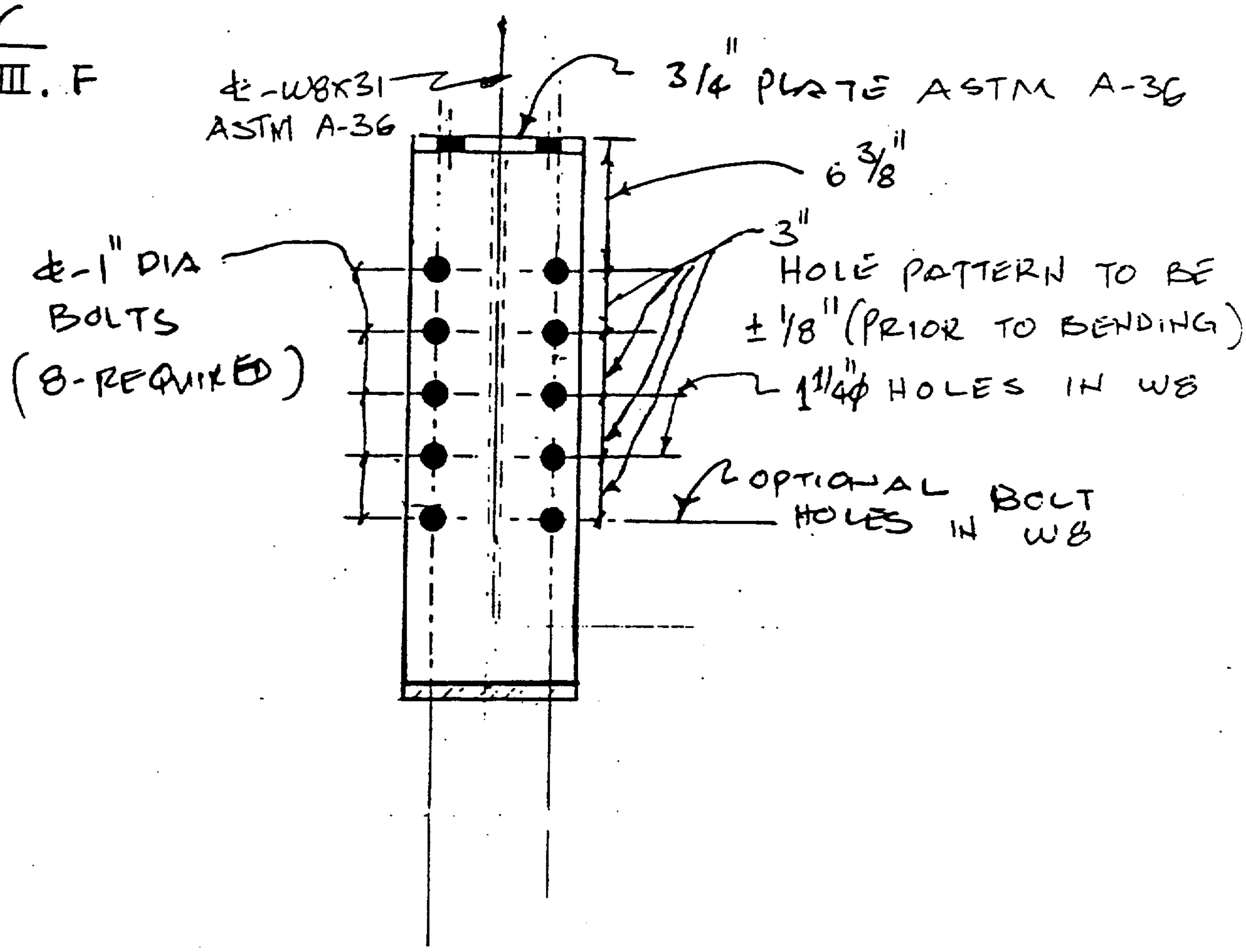


STEEL SET FOOT SEGMENT

FOR W8x31 FOOT SEGMENT

SUMMARY

PURPOSE III. F



SECTION A-A

PURPOSE III.F B) STEEL SET FOOT SEGMENT CALCULATION - FOR W6 x 20 FOOT SEGMENT

DESIGN STEEL SET - FOOT SEGMENT

W8 x 31 IS "OK" FOR LONG TERM LOADS (SEE COMPUTER OUTPUT).

CHECK THE CONSTRUCTION LOADS - USING 15 TONS JACK AND 17 TONS DESIGN LOAD.

$P_a = 17^T \times 2^{k/T}$

$P_a = 34\ k$

$M = \frac{34(5)}{12}$

$= 14.17$ FT-K

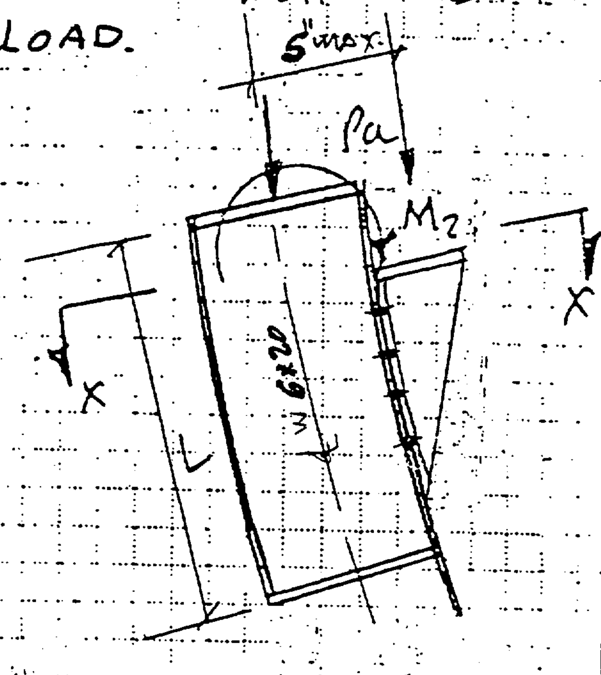


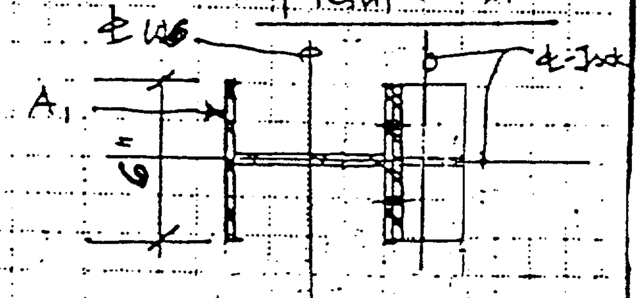
FIGURE - A

FOR W 6 x 20

$A_1 = 5.87\ in^2$

$I_1 = 41.4\ in^4$, $S = 13.4\ in^3$

$f_{bx} = \frac{14.17^{1K} \times 12}{13.4} = 12.69^{KSI}$



SECTION X - X

PURPOSE III. F

W6x20 FOOT SEGMENT

DESIGN PARAMETERS

L = 12'-0" MAX (TRIBL DIMENSION)

$A = 5.87\ in^2$, $I = 41.4\ in^4$,

$r_x = 2.66$

$r_y = 1.5$

$f_a = \frac{P_a}{A} = \frac{34.00}{5.87} = 5.79$ ksi

$\frac{KL}{r_y} = \frac{(1)(12 \times 12)}{1.5} = 96$ $F_a = 13.48$ ksi (AISC TABLE C-36)

$f_a/F_a = \frac{5.79}{13.48} = 42.95\% > 15\%$ CHECK AISC (H1-1&2)

$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{(1 - \frac{f_a}{F'_{ex}}) F_{bx}} \leq 1.0$ ---AISC (H1-1)

$K l_b / r_b = \frac{1.0 \times 144}{2.66} = 54.14$, $F'_{ex} = 50.8$ --- AISC P5-122

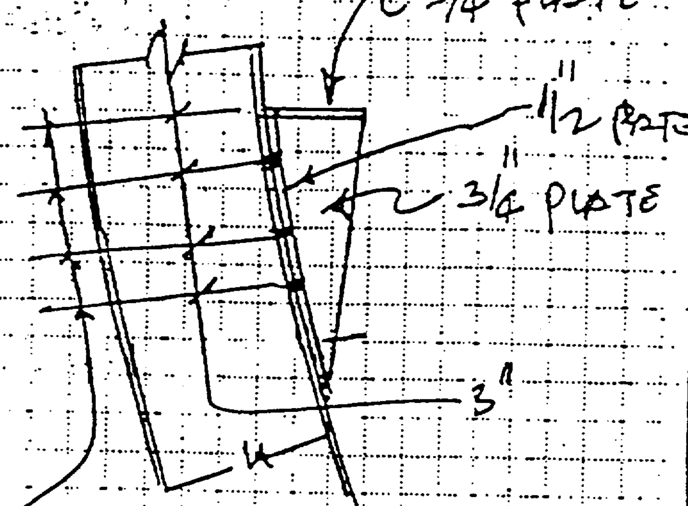
$0.237 + \frac{1.0 \times \overset{0.663}{12.69}}{(1 - \frac{5.79}{50.8}) \times 21.6} = 0.95 < 1.0$ --- O.K.

PURPOSE III. F — FOR W6X20 FOOT SEGMENT

$f_a/0.6F_y + \dfrac{f_{b_x}}{F_{b_x}}$ ---- AISC (H1-2)

$\dfrac{5.79}{21.6} + \dfrac{12.69}{21.6} = 0.856 < 1.0$ ---- O.K.

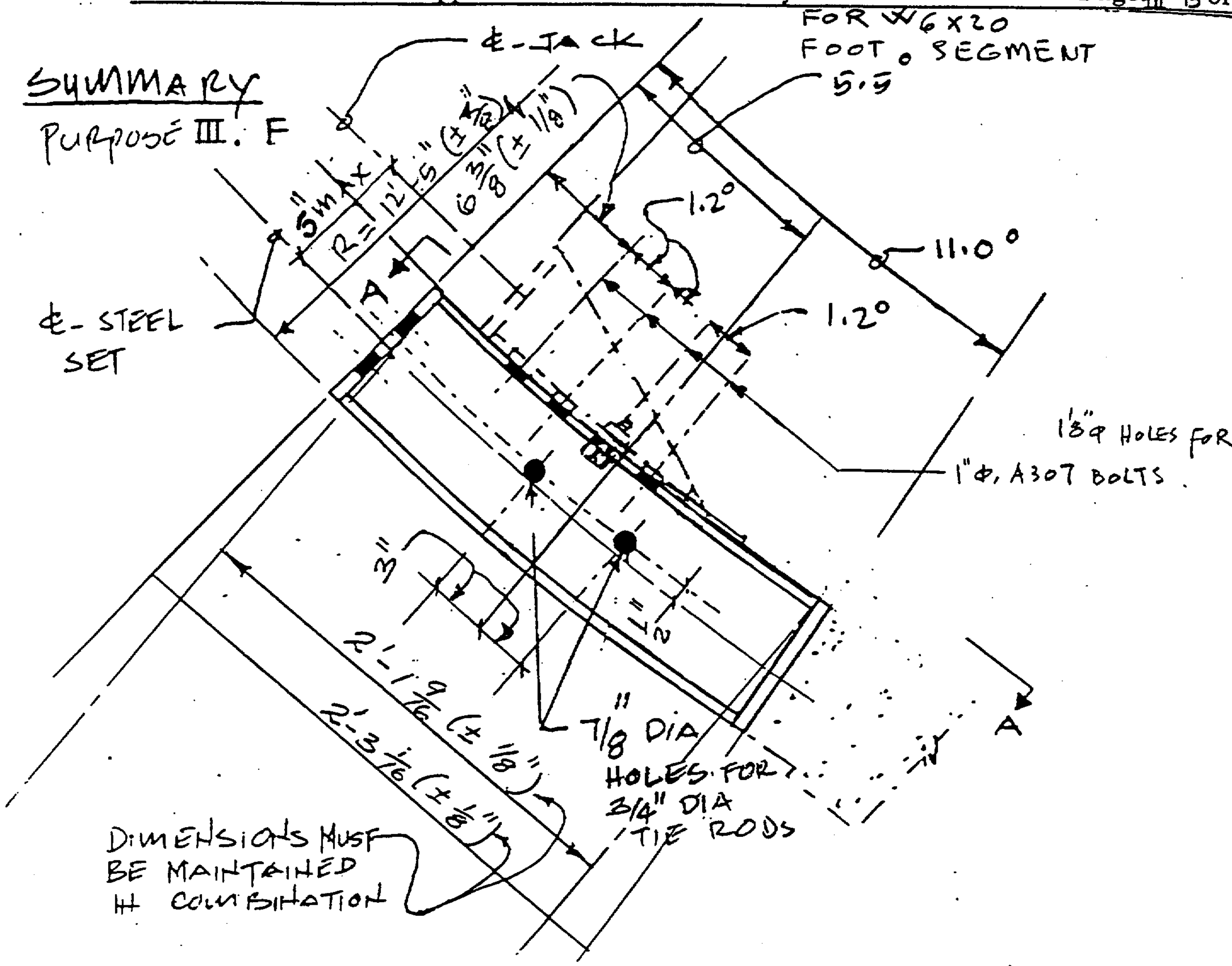
• USE W6X20 WITH MINIMUM JACKING BRACKET AS SHOWN ON FIGURE A AND MAXIMUM LENGTH 12'-0".



6-1" DIA BOLTS A307 & 1 1/8" ϕ HOLES.

FIGURE-A

SUMMARY
PURPOSE III. F

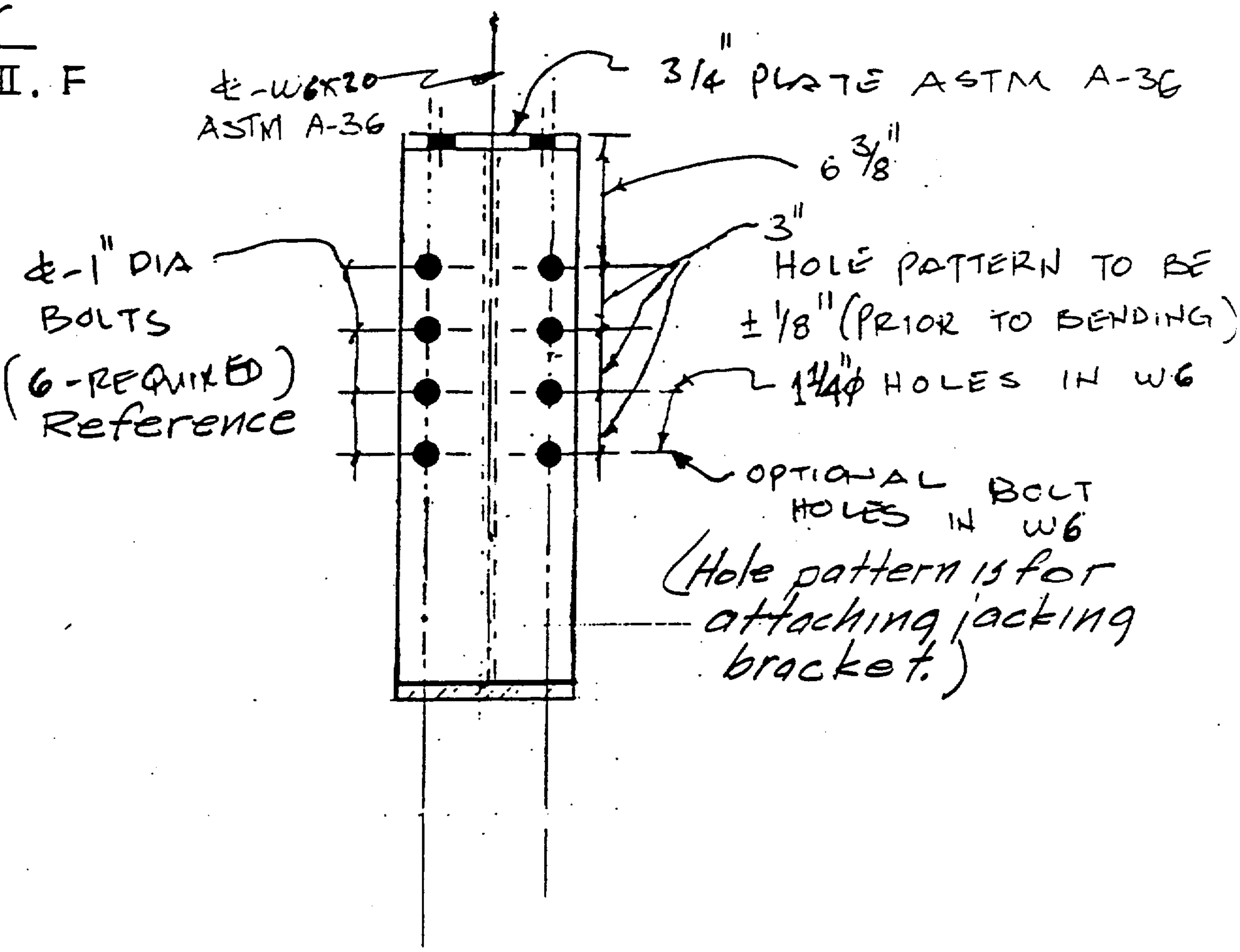


STEEL SET FOOT SEGMENT

FOR W6x20 FOOT SEGMENT

SUMMARY

Purpose III. F



SECTION A-A

PURPOSE III G CONNECTION BETWEEN STEEL SET SEGMEMTS CALCULATION

A) FOR W8x31 STEEL SETS

SPLICE LOCATIONS ARE AT NODES 27 AND 15.

CRITICAL LOADS AND LOCATIONS:

CASE 1) SEISMIC LOADING AT STATION 18+00, NODE 15:
SPACING 4'-0"

SHEAR = 12,970 N. FACTORED = 12.970 × 1.22 × 0.225 = 3.56^k
AXIAL = 252.10 N. " = 252.1 × 1.22 × 0.225 = 69.20^k
MOMENT = 2.14 N m " = 2.14 × 1.22 × 0.738 = 1.93$^{k'}$

CASE 2) STATIC LOADING AT STATION 7+00, NODE 15:
SPACING 4'-0"

SHEAR = 1.876 N " = 1.876 × 1.22 × 0.225 = 0.51^k
AXIAL = 5.86 N = 5.86 × 1.22 × 0.225 = 1.61^k
MOMENT = 0.289 NM = .289 × 1.22 × 0.738 = 0.26$^{k'}$

CASE 3) SEISMIC LOADING AT STATION 10+00, NODE 27
SPACING 2'-0"

SHEAR = 15.11 N " = 15.11 × 0.61 × 0.225 = 2.07^k
AXIAL = 1,069.00 N " = 1069.00 × 0.61 × 0.225 = 146.72
MOMENT = 4.773 NM " = 4.773 × 0.61 × 0.738 = 2.15$^{k'}$

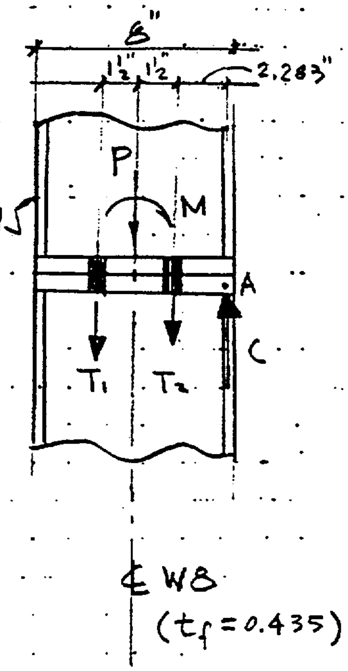
REVIEW OF THE ABOVE LOADING AND COMPARISON OF THE ABOVE LOADING WITH THE JACKING LOADS INDICATES THAT JACKING LOADS GOVERNS THE DESIGN OF BOLTS FOR THE SPLICE CONNECTION. (SEE THE FOLLOWING CALCULATION SHEETS.

PURPOSE III.G CONNECTION BETWEEN STEEL SET SEGMENTS CALCULATION

The splice design is controlled by the jacking loading conditions. The joint at the connection point is modeled in the computer input as a fixed joint. The axial force, shear, and bending moment used for design of the connection are based on the file STLRV3A (Attachment I) which produces the maximum tensile stresses in the connection bolts.

PURPOSE III. 6 A) CONNECTION BETWEEN STEEL SET SEGMENTS CALCULATION FOR W8×31

DESIGN OF STEEL SET SPLICE CONNECTION :



THE CRITICAL DESIGN LOADS AT THE CONNECTION ARE PRODUCED BY JACKING LOADS (STLRV3A) JOINT 16 MEMBER 15

$P = -41.16^k$ COMPRESSION

$V = 9.44^k$ SHEAR

$M = 9.11^{1k}$ MOMENT

$\Sigma M_{@A} = 0$

$9.11^{1k} \times 12 - (5.283\,T_1 + 2.283\,T_2) - 41.16^k \times 3.783 = 0$

$T_1 + 0.432\,T_2 + 8.78 = 0$ ----(1)

AND,

$T_1 / T_2 = 5.283 / 2.283$

$T_1 = 2.314\,T_2$

FROM EQ. (1)

$2.314\,T_2 + 0.432\,T_2 + 8.78 = 0$

$T_2 = -3.2^k$ (COMPRESSION)

$T_1 = -7.4^k$ (COMPRESSION)

THE COMPRESSION FORCES AT BOLTS INDICATE NO BOLT TENSION AND THE COMPRESSION FORCES ARE ACTURALY TAKEN BY BEARING BETWEEN PLATES.

PURPOUSE III G A) CONTIN'D.

CHECK STRESS IN BOLT ~

USING 1"Φ A307 BOLTS, AREA FOR TENSILE STRESS = 0.606^{in^2} --- P.4-147 AISC

TENSION STRESS IN BOLT = 0

$$\text{SHEAR STRESS IN BOLT} = \frac{9.44^{k}}{(4)\text{ BOLTS} \times 0.7854} = 3.0^{\text{ksi}} = f_v$$

ALLOWABLE SHEAR $F_v = 10^{\text{ksi}} > f_v = 3.0^{\text{ksi}}$ --- O.K.
--(AISC PAGE 4-5)

USE (4) - 1"Φ A-307 BOLTS.

CHECK END ℞ AT SPLICE CONNECTION ~ (END ℞ THICKNESS = 3/4")

MAX. BENDING IN PLATE = 0.

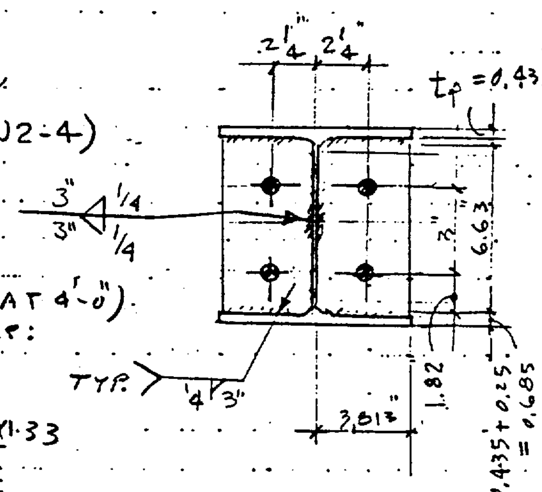
WELD OF END ℞ TO STEEL STEEL SET,

USE MINIMUM 1/4 1/4 --- (AISC TABLE J2-4)

WELD OF WEB TO END PLATE:

MAX. SHEAR = 3.56^{K} (SEE. LOAD AT STATION 12+00 - NODE 15 - STEEL SET AT 4'-0").
FOR 1/4" FILLET WELD x 3" LONG - 2 SIDES:
ALLOW WELD CAP = V_A

$V_A = (0.3 \times 70\text{ ksi})(0.25\text{ in} \times \cos 45°)(2 \times 3\text{ in}) \times 1.33$
$= 29.6\text{ K} >> 3.56\text{ K}$ O.K.



USE 3/4 ℞, A36 AT EACH END OF W8
AND 1/4" FILLET WELD AS SHOWN.

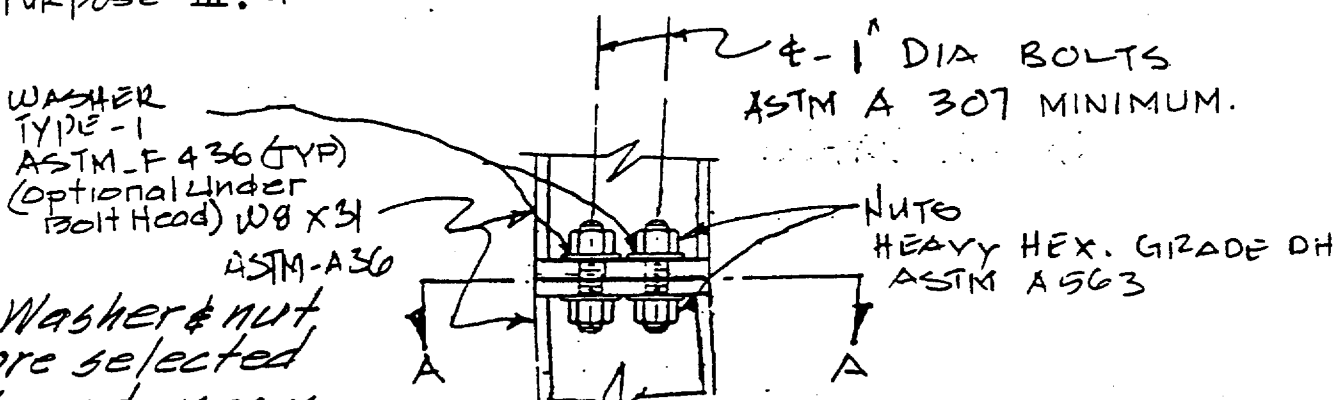
WELD OF FLANGE TO END PLATE:

THE STEEL SET IS IN FULL COMPRESSION AND REQUIRES NO STRENGTH WELD AT FLANGE
HOWEVER, FOR GOOD DESIGN PRACTICE USE 1/4 V3 AS SHOWN.

SUMMARY

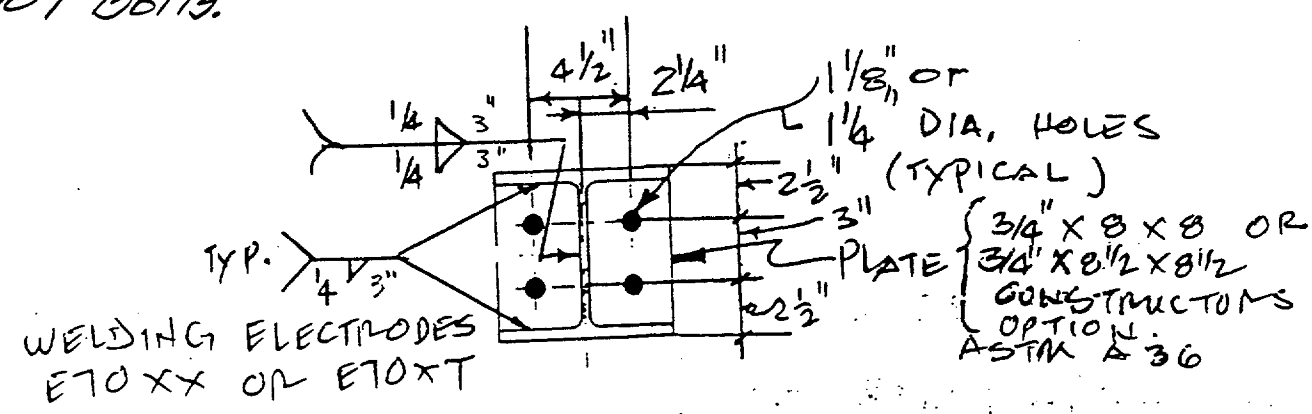
PURPOSE III.G

A) FOR W8X31 STEEL SETS



Washer & nut are selected based on compatibility with ASTM A307 Bolts.

CONNECTION BETWEEN STEEL SET SEGMENT



NOTE: HOLE DIAMETER 1 1/8" OR 1 1/4" SHALL BE AT CONSTRUCTOR'S OPTION.

SECTION A-A

PURPOSE III G B.) CONNECTION BETWEEN STEEL SET

SEGMENT CALCULATIONS FOR W6X20 STEEL SETS

MOST CRITICAL LOAD CONDITION FOR THE SPLICE:

LOCATIONS: NODES 27 AND NODE 15

CRITICAL LOADING CONDITION:

CASE 1) STATIC LOADING AT STATION 53+00, NODE 15
SPACING 4'-0"
SHEAR 2566 N. FACTORED = 2.566 X 1.22 X 0.225 = 0.704 K
AXIAL 52,550 " = 52.55 X 1.22 X 0.225 = 14.42 K
MOMENT 573.4 N M " = 0.573 X 1.22 X 0.738 = 0.52 K'

CASE 2) STATIC LOADING AT STATION 7+00, NODE 15
SPACING 4'-0"
SHEAR 691.4 N " = 0.691 X 1.22 X 0.225 = 0.190 K
AXIAL 18440 N " = 18.44 X 1.22 X 0.225 = 5.06 K
MOMENT 246.7 N M " = 0.247 X 1.22 X 0.738 = 0.222 K'

CASE 3) STATIC LOADING AT STATION 53.0, NODE 15
SPACING 4'-0"
SHEAR 2041 N " = 2.04 X 1.22 X 0.225 = 0.560 K
AXIAL 37,750 N " = 37.75 X 1.22 X 0.225 = 10.36 K
MOMENT 654.4 N M " = 0.654 X 1.22 X 0.738 = 0.589 K'

PURPOSE III G B) CONNECTION BETWEEN STEEL SET SEGMENT CALCULATIONS FOR W6x20 STEEL SETS (CONT'D)

JACKING CONDITION LOADS:

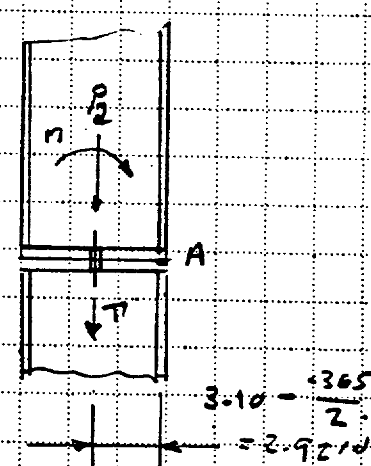
REF. COMPUTER RUN FOR 15 TON JACKING ATTACHMENT VIII "PROGRAM STLRV3". JOINT 16 MEMB. 15

$P_z = -23.81^k \quad V_y = -5.54^k \quad M_z = 5.51^{k'}$

BY COMPARISON OF THE JACKING LOADS TO STATIC AND SEISMIC LOADING, JACKING LOAD WILL GOVERN THE CONNECTION DESIGN.

CONNECTION DESIGN

TRY 2 - 1"φ BOLTS AND 5/8" THK END PLATES.



$\Sigma M_A = 0$

$23.81 \times 2.92 + T \times 2.92 = 5.51 \times 12$

$2.92T = 66.12 - 69.52$

$T = -1.17^k$ ∴ BOLTS ARE IN TENSION

CHECK BOLTS FOR SHEAR;

$V_b = \dfrac{5.54^k}{2 \text{ BOLTS}} = 2.77^k/\text{BOLT} < 7.9^k/\text{BOLT}$ * O.K.

* ALLOW SHEAR FOR 1"φ A307 BOLT PER AISC TABLE I-D

∴ 2 - 1"φ A307 BOLTS O.K. FOR SPLICE LOCATION

PURPOSE III.G B) SPLICE CONNECTION FOR W6x20 (CONT'D)

CHECK END PLATE FOR SPLICE CONNECTION: (5/8" THICK PL)

MAXIMUM BENDING IN PLATE = 0

∴ 5/8" THICK PLATE O.K. FOR BENDING.

WELD OF PLATE TO STEEL SET:

MAX. WELD SIZE ≤ W6x20 WEB THICKNESS

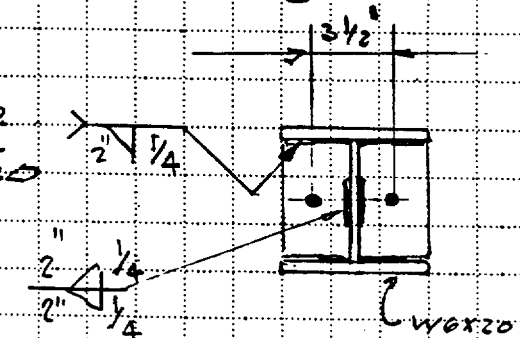
$t_w = 1/4"$ (AISC 5-67)

TRY 3/16 FILLET WELD 2" LONG EACH SIDE OF THE WEB TO RESIST SHEAR FORCES

CAPACITY OF 2" WELD = V

$V = (0.3 \times 70\ ksi)(\frac{3}{16} \cos 45°) \times$
$2\ SIDES \times 2\ IN) = 11.14^k$

MAX. SHEAR = $5.54^k < 11.14^k$

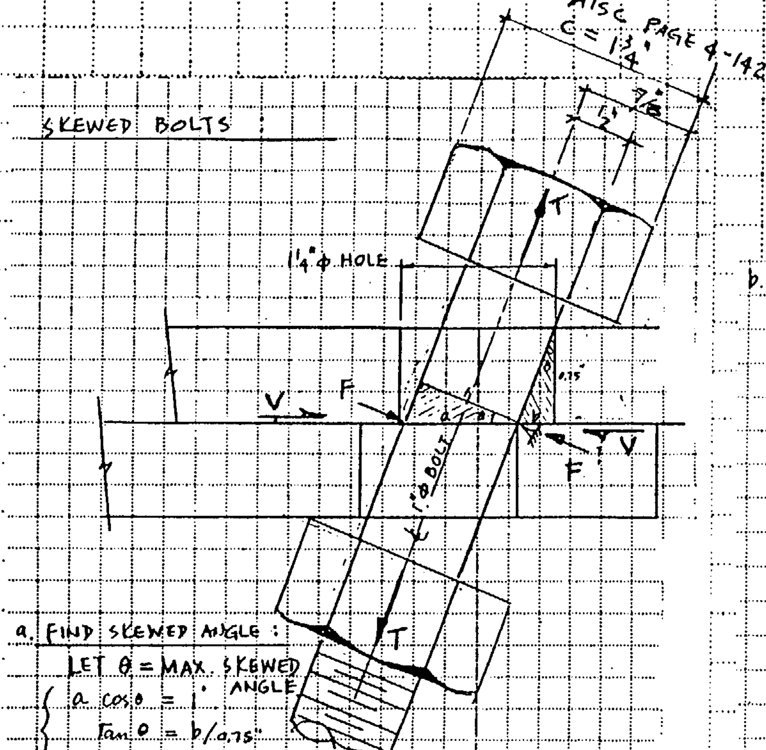


WELD OF FLG TO END PL:
(SEE PAGE III-97)

∴ 3/16 WELD x 2" LONG AT EACH SIDE OF THE WEB MIN. TO RESIST MAXIMUM SHEAR FORCES

USE 5/8" THICK A36 PLATE WITH 1/4" FILLET WELD AS SHOWN ON EACH SIDE OF SPLICE CONNECTION

SKEWED BOLTS :



NOTE :

$F = V\cos\theta = 2.36\cos 15.85° = 2.27^k$

$T = V\sin\theta = 2.36\sin 15.85° = 0.64^k$

$M = Fa\sin\theta =$ (SEE BELOW) $= 0.64^{"k}$

a. FIND SKEWED ANGLE :

LET θ = MAX. SKEWED ANGLE

$\left\{\begin{array}{l} a\cos\theta = 1" \\ \tan\theta = b/0.75" \\ a + b = 1.25" \end{array}\right.$

$\frac{1}{\cos\theta} + 0.75\tan\theta = 1.25"$

$\frac{1}{\cos\theta} + \frac{0.75\sin\theta}{\cos\theta} = 1.25"$

$1 + 0.75\sin\theta = 1.25\cos\theta$

$(0.75)^2\sqrt{1-\cos^2\theta} = 1.25\cos\theta - 1$

$0.5625(1-\cos^2\theta) = 1.5625\cos^2\theta - 2.5\cos\theta + 1$

$2.125\cos^2\theta - 2.5\cos\theta + 0.4375 = 0$

$\cos\theta = \frac{2.5 \pm \sqrt{6.25 - 3.719}}{4.25} = \frac{2.5 \pm 1.59}{4.25}$

$\cos\theta = 0.962$ or 0.214

$\theta = 15.85°$

$a = 1/\cos\theta = 1.04"$

$b = 1.25" - 1.04 = 0.21"$

MAX. SKEWED ANGLE = 15.85°

b. BENDING IN SKEWED BOLT :

USING 1" φ A307 BOLT IN $1\frac{1}{4}"$ φ MAX. HOLE.

THE TENSION IN BOLT AT SNUG TIGHT CONDITION ≅ 0

(PER TELEPHONE CONVERSATION WITH AISC ENGINEER STAFF) (SEE VI-9)

THE BENDING IN THE SKEWED BOLT DUE TO BOLT TENSION IS NEGLIGIBLE.

CONSIDER BENDING IN THE SKEWED BOLT DUE TO SHEAR.

MAX. SHEAR AT CONNECTION = 9.44^k ---- PAGE III-96
(JACKING CONDITION)

MAX. SHEAR PER BOLT $= 9.44^k / 4$ BOLTS $= 2.36^k = V$

FROM THE FREE BODY OF THE UPPER HALF OF THE BOLT (SEE SKETCH)

F = MAX. BOLT SHEAR $= V\cos\theta = 2.36\cos 15.85°$

$F = 2.27^k$

MOMENT IN BOLT $= F \times (a\sin\theta)$

$= 2.27^k \times (1.04"\sin 15.85°) = 0.64^{"k}$

SECTION MODULUS OF THREAD BOLT

$= \frac{\pi d^3}{32} = \frac{\pi(0.847)^3}{32} = 0.06\ in^3$

$f_t = f_b + \frac{T}{A} = 10.7 + 0.8 = 11.5_{ksi} < F_t = 20^{ksi}$ --- AISC PG 4-3

where $\left\{\begin{array}{l} f_b = \frac{M}{S} = \frac{0.64"}{0.06} = 10.7_{ksi}\ \text{TENSION} \\ T/A = 0.64^k/0.7854 = 0.8_{ksi} \end{array}\right\}$ ALLOWABLE TENSILE STRESS (ALSO SEE BELOW)

CHECK COMBINED STRESS (AISC J3.5 & TABLE J3.3)

$f_v = \frac{V}{A} = \frac{2.27^k}{.7854} = 2.89_{ksi} < F_v = 10_{ksi}$

$F_t = 26 - 1.8 f_v \leq 20$ (AISC TABLE J3.3)

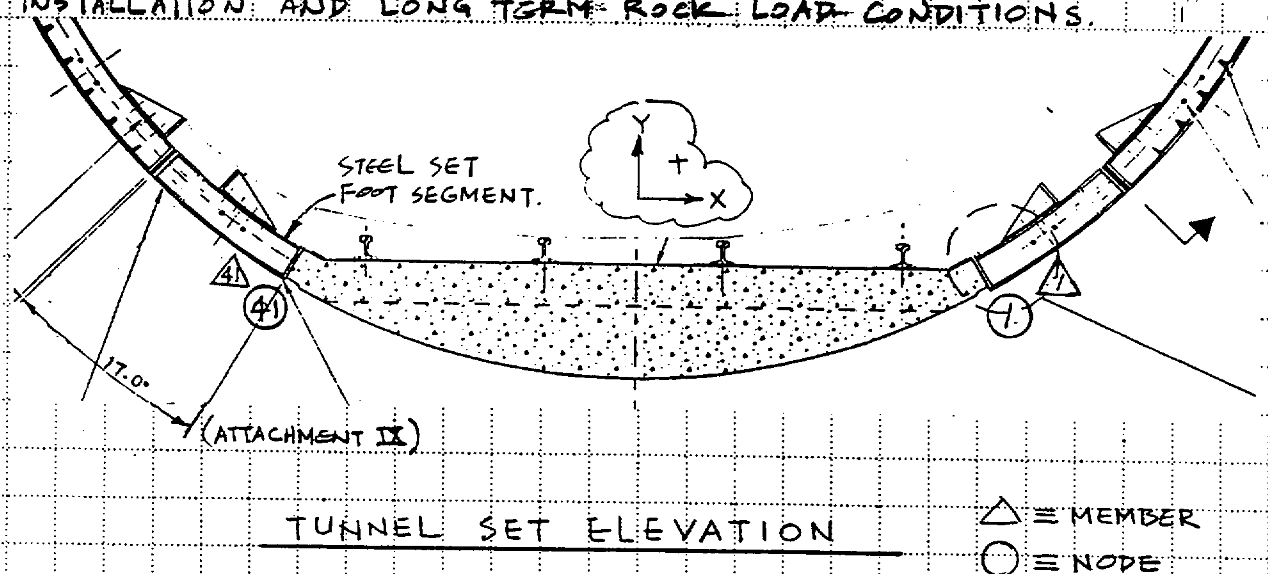
$= 26 - 1.8(2.89) = 20.8_{ksi}$ ∴ USE $F_t = 20_{ksi}$ AS SHOWN ABOVE

THE SKEWED BOLT IS OK AS SHOWN

PURPOSE III.H.

STABILITY OF STEEL SET FOOT SEGMENT:

THE STEEL SET FOOT SEGMENTS ARE SUPPORTED ON CONCRETE INVERT AS SHOWN BELOW. THE FOOT SEGMENT MUST BE STABLE - DURING JACKING INSTALLATION AND LONG TERM ROCK LOAD CONDITIONS.



TUNNEL SET ELEVATION

〈A〉 LONG TERM ROCK LOAD CONDITION:

THE STEEL SET FOOT SUPPORT POINTS ON CONCRETE INVERT ARE AT NODES 01 AND 41 IN COMPUTER MODEL (SEE ATTACHMENT II). THE STEEL SET FOOT SEGMENT STABILITY ANALYSIS AGAINST SLIDING IS PRESENTED IN THE FOLLOWING PAGES.

This analysis compares the average axial force in the steel set and concrete invert with the maximum shear force generated in the lower elements of the steel set. The average axial force is used because the steel set and invert exhibit ring compression. As modeled in FLAC, the rock squeezes the steel set into an oval shape, developing compressive forces in the sides of the set and tensile forces in the top of the set and in the invert. The average of these forces represents the axial force distributed throughout the steel set and invert. The maximum shear in the lower elements of the steel set was selected from the shear diagrams shown in Reference 5.20. As shown in the diagrams, the maximum shear forces in the steel set occur at locations above the springline where utilities are supported. These shear forces are absorbed by the surrounding rock and are not applicable to determine the sliding of the steel set on the concrete invert. To be conservative, the maximum shear in an area near the steel / concrete interface was found by looking at the FLAC data in Attachment II and determining the cutoff point where the shear begins to decrease in value. For this analysis, the lower elements of the steel set refer to the segments between elements 35 and 40 and between elements 1 and 6.

FILE : m07K250W.SAV. AT STATION TCW 7+00 (II-10 to II-12)
STEEL SET W8×31 @ 4'-0" O.C.
LOADING ; STATIC DL + UTIL

TOTAL AXIAL FORCE OF 48 ELEMENTS

$\Sigma F = 236.05 \times 10^4$ NEWTONS PER SPACING @ ONE METER

AVERAGE AXIAL FORCE ALONG THE STEEL SET

$$= \frac{\Sigma F}{48 \text{ ELEMENTS}}$$

$$= \frac{236.05 \times 10^4}{48} = (4.918 \times 10^4 \text{ NEWTONS}) \times (2.2481 \times 10^{-4} \text{ KIPS/NEWTONS})$$

$$= 11.06 \frac{\text{KIPS}}{\text{METER SPACING}} \times \frac{1m}{3.281ft} \times 4ft = 13.98^k @ 4' \text{ SPACING}$$

MAX SHEAR IN BOTTOM ELEMENTS OF STEEL SET (BETWEEN NODES 35 TO 40 AND NODES 1 TO 6) = 4.972×10^3 N (ELEM 38)

= 1.36 kips @ 4' spacing

FRICTION FORCE AT BOTTOM OF STEEL SET FOOT SEGMENT

$= \mu N$; $\mu_k = 0.3$ BETWEEN CONCRETE & STEEL (REF. 5.17, p.275)

$= 0.3 \times 13.98 = 4.04^k \gg 1.36^k$ MAX. SHEAR

---- O.K.

NO SLIDING @ STATION TCW 7+00 (W8 SET @ 4'0")

OVERALL FACTOR OF SLIDING RESISTANCE = $\frac{\text{FRICTION FORCE}}{\text{MAX SHEAR}}$

$$F.S.R. = \frac{4.04}{1.36} = 2.97$$

ALL OTHER STATIONS ALONG THE MAIN DRIFT USING W8×31 STEEL SETS ARE SUMMARIZED IN PAGE III-107. ALL FRICTION FORCES ARE LARGER THAN THE SHEAR IN THE FOOT SEGMENT.

NO ANCHORAGE IS REQUIRED. THE LOWEST FACTOR OF SLIDING RESISTANCE FOR THE W8×31 STEELSETS IS 2.97 (m07K250W). THIS IS A VERY CONSERVATIVE FACTOR BASED ON STANDARD ENGINEERING PRACTICE AND IS THEREFORE REASONABLE.

FILE: M07_K2.sav AT STATION TCW 7+00 (III-96 & III-50)
STEEL SET W6 x 20 @ 4'0" O.C.
LOADING: STATIC: DL + UTIL

TOTAL AXIAL FORCE OF 48 ELEMENTS

$\Sigma F = 4.491 \times 10^5$ NEWTONS PER ONE METER SPACING

AVERAGE AXIAL FORCE ALONG THE STEEL SET

$$= \frac{\Sigma F}{48 \text{ ELEMENTS}}$$

$$= \frac{4.491 \times 10^5}{48} = (9.357 \times 10^3 \, N)(2.2481 \times 10^{-4} \, kips/N)$$

$$= \frac{2.103 \text{ kips}}{1m \text{ Spacing}} * \frac{1m}{3.281 ft} * 4 ft = 2.564 \text{ kips @ 4' SPACING}$$

MAX SHEAR IN BOTTOM ELEMENTS OF STEEL SET (BETWEEN NODES 35 TO 40 AND NODES 1 TO 6) = 1.119×10^3 N (ELEM 5)

= 0.307 kips @ 4' spacing

FRICTION FORCE AT BOTTOM OF STEEL SET FOOT SEGMENT

= μN ; $\mu = 0.3$ BETWEEN CONCRETE & STEEL (REF 5.17, p. 275)

= 0.3 * 2.564 k = 0.769 k > 0.307 (Max Shear)

⇒ O.K.

⇒ NO SLIDING @ STATION TCW 7+00 (W6 SET @ 4'0")

OVERALL FACTOR OF SLIDING RESISTANCE = $\frac{\text{FRICTION FORCE}}{\text{MAX SHEAR}}$

$$F.S.R. = \frac{0.769}{0.307} = 2.51$$

ALL OTHER STATIONS ALONG THE MAIN DRIFT USING W6 X 20 STEEL SETS ARE SUMMARIZED IN PAGE III-108. ALL FRICTION FORCES ARE LARGER THAN THE SHEAR IN THE FOOT SEGMENT.

NO ANCHORAGE IS REQUIRED. THE LOWEST FACTOR OF SLIDING RESISTANCE FOR THE W6X20 STEEL SETS IS 2.51 (M07_K2). THIS IS A VERY CONSERVATIVE FACTOR BASED ON STANDARD ENGINEERING PRACTICE AND IS THEREFORE REASONABLE.

DETERMINATION OF SLIDING					
STEEL SET / SPACING	W8x31, 4'0" o.c.	W8x31, 2'0" o.c.	W8x31, 4'0" o.c.	W8x31, 2'0" o.c.	W8x31, 4'0" o.c.
FILE	m07k250w.sav	m10k250w.sav	m18k250w.sav	m34k250x.sav	p07k2dy.sav
STATION	TCW @ 7+00	PTN @ 10+00	TSW1 @ 18+00	TSW2 @ 34+00	TCW @ 7+00
LOADING	Static	Static	Static	Static	Seismic+Static
Axial sum (N/1 m spacing)	2.360E+06	2.780E+07	1.153E+07	1.999E+07	6.355E+06
Axial ave (N/1 m spacing)	4.918E+04	5.791E+05	2.402E+05	4.165E+05	1.324E+05
Axial ave (kips/spacing)	1.348E+01	7.936E+01	6.583E+01	5.707E+01	3.629E+01
Controlling shear, N	4.972E+03	9.315E+03	1.187E+04	1.222E+04	7.011E+03
Controlling shear, kips	1.363E+00	2.553E+00	3.253E+00	3.348E+00	1.922E+00
Friction force (kips)	4.043E+00	2.381E+01	1.975E+01	1.712E+01	1.089E+01
Result (friction>shear)	O.K.	O.K.	O.K.	O.K.	O.K.
Overall F.S.R	2.97	9.33	6.07	5.11	5.67

Overall Summary: There is no sliding at any of the above stations.

Axial and shear forces at each station are from Attachment II.
Controlling shear refers to the maximum shear in the lower elements of the steel set.
Calculations for this analysis are obtained by the process outlined on pages III-105 and III-106.

DETERMINATION OF SLIDING			
STEEL SET / SPACING	W8x31, 2'0" o.c.	W8x31, 4'0" o.c.	W8x31, 2'0" o.c.
FILE	p10k2dy.sav	p18k2dy.sav	p34k2dyx.sav
STATION	PTN @ 10+00	TSW1 @ 18+00	TSW2 @ 34+00
LOADING	Seismic+Static	Seismic+Static	Seismic+Static
Axial sum (N/1 m spacing)	4.265E+07	1.776E+07	2.737E+07
Axial ave (N/1 m spacing)	8.886E+05	3.701E+05	5.702E+05
Axial ave (kips/spacing)	1.218E+02	1.014E+02	7.814E+01
Controlling shear, N	1.206E+04	1.497E+04	1.568E+04
Controlling shear, kips	3.305E+00	4.103E+00	4.296E+00
Friction force (kips)	3.653E+01	3.043E+01	2.344E+01
Result (friction>shear)	O.K.	O.K.	O.K.
Overall F.S.R.	11.05	7.42	5.46

Overall Summary: There is no sliding at any of the above stations.

Axial and shear forces at each station are from Attachment II.
Controlling shear refers to the maximum shear in the lower elements of the steel set.
Calculations for this analysis are obtained by the process outlined on pages III-105 and III-106.

DETERMINATION OF SLIDING					
STEEL SET / SPACING	W6x20, 4'0" o.c.	W6x20, 4'0" o.c.	W6x20, 4'0" o.c.	W6x20, 4'0" o.c.	W6x20, 4'0" o.c.
FILE	m07_k2.sav	m18_k2.sav	m27_k2.sav	m34_k2.sav	m53_k2.sav
STATION	TCW @ 7+00	TSW1 @ 18+00	TSW2 @ 27+00	TSW2 @ 34+00	TSW2 @ 53+00
LOADING	Static	Static	Static	Static	Static
Axial sum (N/1 m spacing)	4.491E+05	3.141E+06	1.764E+06	2.286E+06	1.861E+06
Axial ave (N/1 m spacing)	9.357E+03	6.544E+04	3.675E+04	4.762E+04	3.877E+04
Axial ave (kips/spacing)	2.564E+00	1.793E+01	1.007E+01	1.305E+01	1.063E+01
Controlling shear, N	1.119E+03	4.490E+03	3.538E+03	2.929E+03	3.398E+03
Controlling shear, kips	3.067E-01	1.231E+00	9.697E-01	8.028E-01	9.313E-01
Friction force (kips)	7.693E-01	5.380E+00	3.022E+00	3.915E+00	3.188E+00
Result (friction>shear)	O.K	O.K	O.K	O.K	O.K
Overall F.S.R.	2.51	4.37	3.12	4.88	3.42

Overall Summary: There is no sliding at any of the above stations.

Axial and shear forces at each station are from Attachment II.
Controlling shear refers to the maximum shear in the lower elements of the steel set.
Calculations for this analysis are obtained by the process outlined on pages III-105 and III-106.

DETERMINATION OF SLIDING					
STEEL SET / SPACING	W6x20, 4'0" o.c.	W6x20, 4'0" o.c.	W6x20, 4'0" o.c.	W6x20, 4'0" o.c.	W6x20, 4'0" o.c.
FILE	m07_k2dy.sav	m18_k2dy.sav	m27_k2dy.sav	m34c2k2d.sav	m53_k2dy.sav
STATION	TCW @ 7+00	TSW1 @ 18+00	TSW2 @ 27+00	TSW2 @ 34+00	TSW2 @ 53+00
LOADING	Seismic+Static	Seismic+Static	Seismic+Static	Seismic+Static	Seismic+Static
Axial sum (N/1 m spacing)	1.675E+06	5.270E+06	3.166E+06	5.029E+06	3.274E+06
Axial ave (N/1 m spacing)	3.489E+04	1.098E+05	6.596E+04	1.048E+05	6.821E+04
Axial ave (kips/spacing)	9.562E+00	3.009E+01	1.808E+01	2.872E+01	1.870E+01
Controlling shear, N	1.576E+03	6.242E+03	4.392E+03	1.045E+04	4.199E+03
Controlling shear, kips	4.319E-01	1.711E+00	1.204E+00	2.864E+00	1.151E+00
Friction force (kips)	2.869E+00	9.027E+00	5.423E+00	8.615E+00	5.609E+00
Result (friction>shear)	O.K	O.K	O.K	O.K	O.K
Overall F.S.R.	6.64	5.28	4.51	3.01	4.87

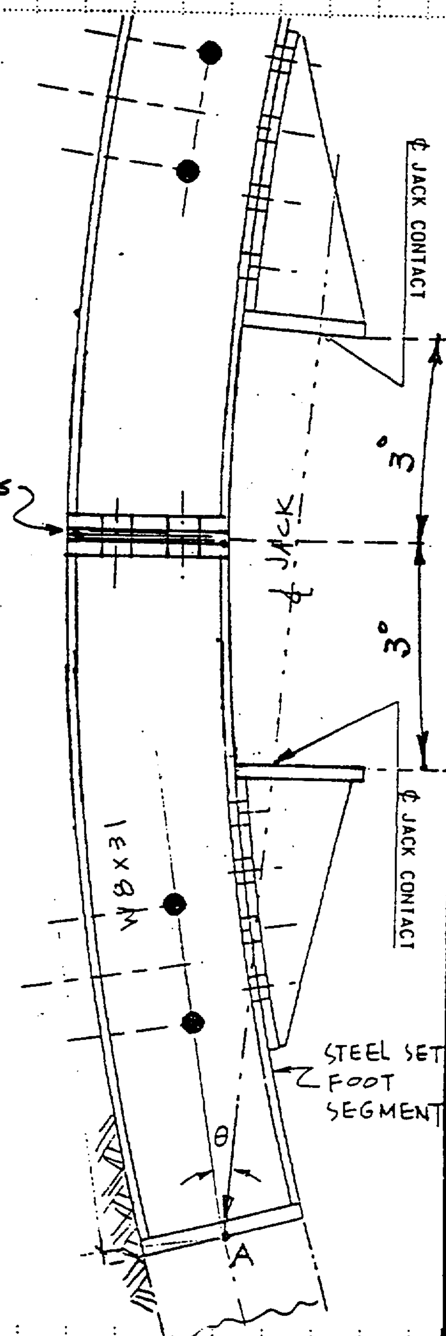
Overall Summary: There is no sliding at any of the above stations.

Axial and shear forces at each station are from Attachment II.
Controlling shear refers to the maximum shear in the lower elements of the steel set.
Calculations for this analysis are obtained by the process outlined on pages III-105 and III-106.

PURPOSE III. H

⟨B⟩ JACKING CONDITION DURING STEEL SET INSTALLATION :
(ALTERNATE I ONLY, SEE ATTACHMENT IX-5)

THE STEEL SET FOOT SEGMENT DETAIL IS SHOWN ON THE RIGHT. THE STABILITY OF THE FOOT SEGMENT DURING JACKING DEPENDS ON THE POSITION OF THE JACK. AN ANALYSIS TO DETERMINE STABILITY OF THE FOOT SEGMENT IS SHOWN ON THE FOLLOWING PAGES.



STABILITY DETERMINATION OF THE STEEL SET, FOOT SEGMENT AT JACKING LOCATION: W8×31 STEEL SET.

WITH JACKING BRACKETS LOCATED AS SHOWN ON THE "STEEL SET DIMENSIONS" SKETCH, FOLLOWING SHEET, DETERMINE THE LOCATION OF INTERSECTION OF THE LINE OF JACKING FORCE AND THE BASE PLATE. IF THIS LOCATION FALLS TO THE LEFT OF 1/3 SECTON OF THE BASE PLATE, STEEL SET AT JACK LOCATION WILL BE STABLE DURING THE JACKING OPERATION. SEE DETAIL 1 FOLLOWING PAGES.

IN DET. 1:

R = RADIUS TO ℄ STEEL SET = 12'-5" − 4" = 12'-1" = 145"

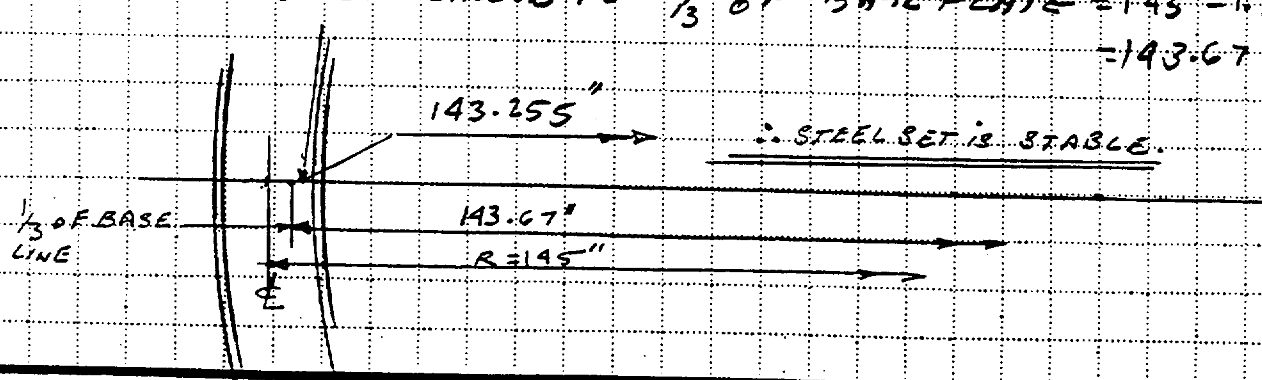
θ = ANGLE BTWN TOP OF LOWER JACK BRACKET & BASE

$\theta = 17^\circ - 3^\circ = 14^\circ \qquad \cos 14^\circ = 0.9703$

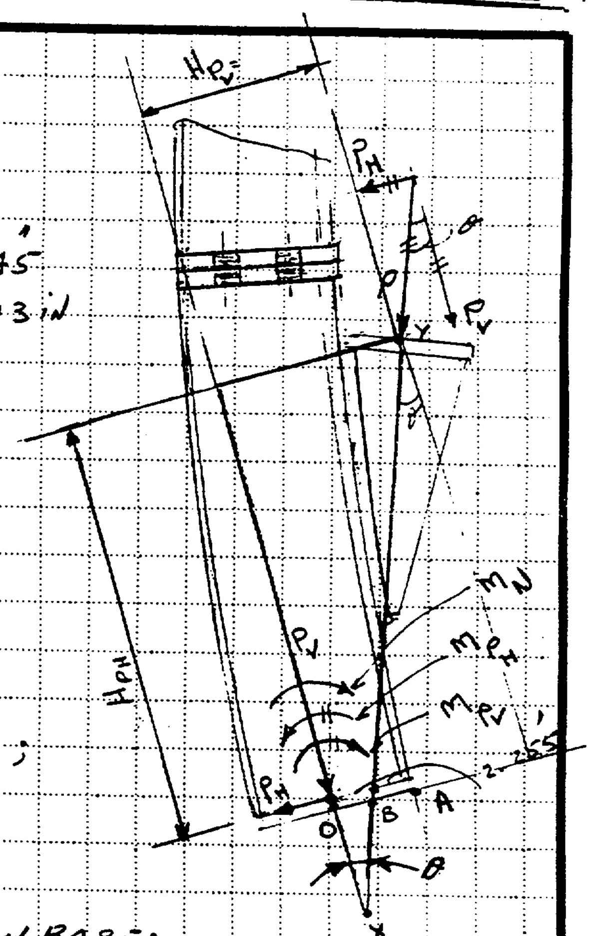
$$\cos\theta = \frac{R - (4'' + 2'')}{D} = \frac{145 - 6}{D} = 0.9703$$

D = 143.255" = DISTANCE BTWN. CENTER OF STEEL SET TO LINE OF JACKING FORCE ON THE BASE PLATE

℄ STEEL SET CIRCLE TO 1/3 OF BASE PLATE = 145" − 1.33 = 143.67"



∴ STEEL SET IS STABLE.



DETERMINE STABILITY RATIO

$\angle\theta = 17^\circ - 3^\circ = 14^\circ$ (SEE DET. 1) SKETCH

$\Delta(OBX)$; $\theta = 14^\circ$; $\overline{OB} = (145 - 143.255) = 1.745''$

$\overline{BX} = \overline{OB} \div \sin\theta = 1.745 \div 0.242 = 7.213$ in

$\overline{YB} = \tan 14 (145 - 6) = 34.66$ in

$\overline{YX} = 7.213 + 34.66 = 41.873$ in

$\overline{OX} = \frac{OB}{\tan\theta} = \frac{1.745}{.2493} = 7.00$

$\frac{\overline{YX}}{\overline{BX}} = \frac{H_{PH}}{\overline{OX}}$; $H_{P_H} = \frac{7 \times 41.873}{7.213} = 40.636$

$H_{P_V} = \overline{YX} \sin\theta = 41.873 \times 0.2419 = 10.13$ in

RESOLVE JACKING FORCES "P" INTO P_H & P_V;

$P_H = P \sin 14^\circ = 0.219\,P$

$P_V = P \cos 14^\circ = 0.97\,P$

SUPERIMPOSE P_V & P_H TO POINT "O" ON BASE:

$M_{P_V} = P_V \times H_{P_V} = 10.13\,P_V$; $M_{P_H} = P_H \times H_{P_H} = 40.636 \times P_H$

$= 10.13(0.97P) = 9.83\,P$; $= 40.636 \times 0.219\,P = 8.90\,P$

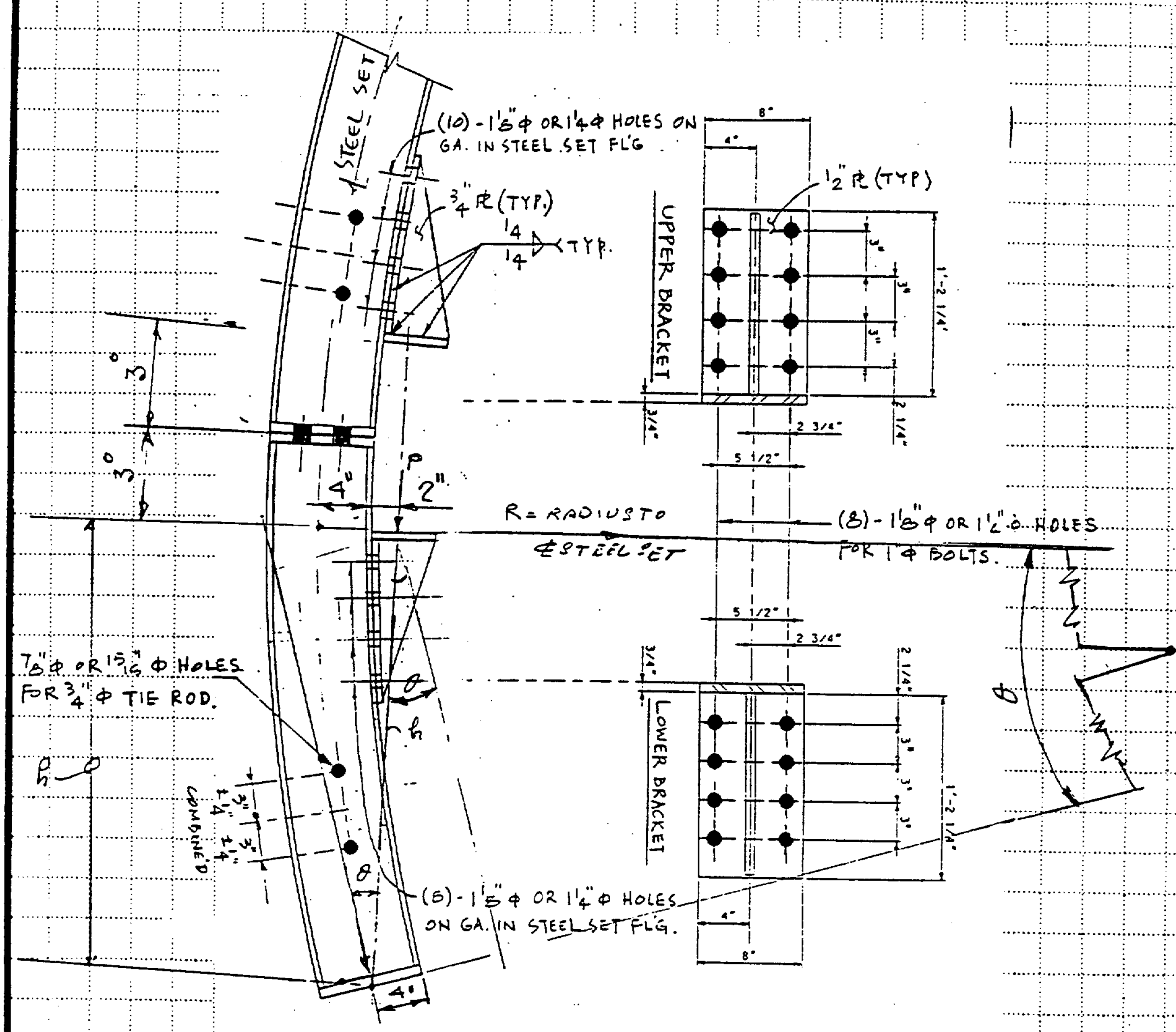
$OTM = 9.83\,P - 8.90\,P = 0.93\,P$

STABILITY MOMENT ABOUT TOE (POINT "A") = $P_V \times 4'' = 0.97P \times 4'' = 3.88\,P$

STABILITY RATIO = $\frac{3.88\,P}{0.93\,P}$ = 4.17 > 1.0 STABILITY RATIO O.K.

CHECK SLIDING: $P_H = 0.219\,P$

FRICTION AT BASE = $\mu N = 0.3\,P > 0.219\,P$ ∴ SLIDING O.K.



DETAIL 1 / IX-5 & 6

STABILITY DETERMINATION OF THE STEEL SET AT JACKING LOCATION: W6X20 STEEL SET.

FOR REFERENCE OF THIS ENGINEERING CALCULATIONS SEE STABILITY CALCULATIONS FOR W8X31 STEEL SETS;

R = RADIUS TO ℄ STEEL SET = 12'-5" - 3" = 12-2" = 146"

θ = ANGLE BTWN TOP OF LOWER JACK BRACKET & BASE PLATE.

$\theta = 17° - 3° = 14°$; $\cos 14° = 0.9703$

D = DISTANCE BTWN. CENTER OF STL-SET TO LINE OF JACKING FORCE ON THE BASE PLATE.

$\cos\theta = \frac{R - (3'' + 2'')}{D} = \frac{146'' - 5''}{D} = 0.9703$

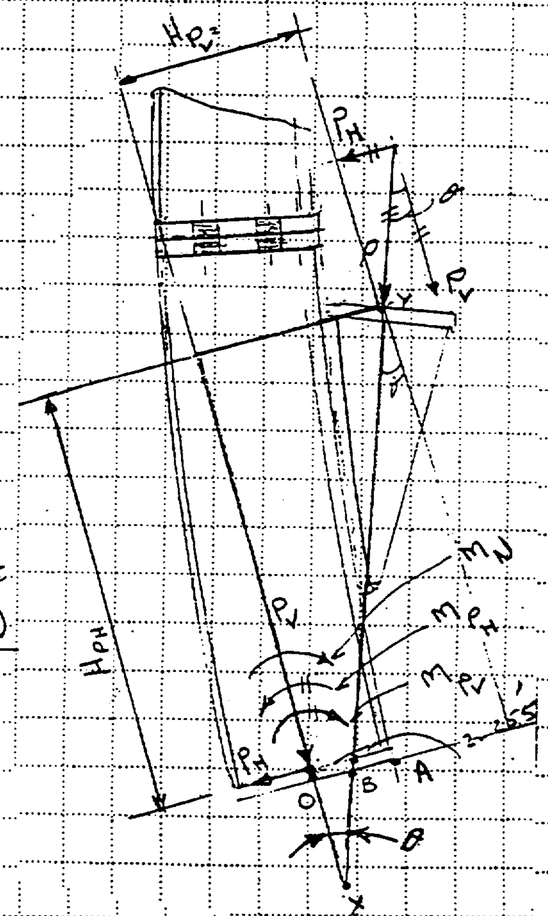
$D = 145.3\frac{1}{2}''$

℄ STEEL SET TO 1/3 OF BASE ℗

$= 145 - 3'' + \frac{6}{3} = 144$ in

∴ LINE OF JACKING IS TO THE LEFT OF 1/3 OF BASE ℗ LINE

∴ STEEL SET FOR W6X20 SECTION IS STABLE.



DETERMINE STABILITY RATIO FOR W6x20 STEEL SECTION

$\Delta(OBx)$ $\theta = 14^\circ$ $\overline{OB} = (146 - 145.32) = 0.68$ in

$\overline{Bx} = \frac{OB}{\sin\theta} = \frac{0.68}{.242} = 2.81$ in

$\overline{YB} = \tan 14 (146 - 5) = 35.155$ in

$\overline{Yx} = 2.81 + 35.155 = 37.965$

$\overline{Ox} = \frac{OB}{\tan\theta} = \frac{0.68}{0.2493} = 2.73$ in

$\frac{\overline{Yx}}{\overline{Bx}} = \frac{H_{PH}}{Ox}$; $H_{PH} = \frac{2.73 \times 37.965}{2.81} = 36.88$ in

$H_{PV} = \overline{Yx} \sin\theta = 37.965 \times 0.2419 = 10.13$ in

RESOLVE JACKING FORCE "P" INTO P_H & P_V;

$P_H = P \sin 14^\circ = 0.219 P$

$P_V = P \cos 14^\circ = 0.97 P$

SUPER IMPOSE P_V & P_H TO POINT "O" ON THE BASE PLATE:

$M_{PV} = P_V \times H_{PV} = 10.13 \times 0.97 P = 9.83 P$

$M_{PH} = P_H \times H_{PH} = 36.88 \times 0.291 = 10.73 P$

$OTM = 10.73 P - 9.83 P = 0.90 P$ ∴ OTM IS IN DIRECTION AGINST THE TUNNEL WALL + $P_V \times 3'' = 0.97 P \times 3'' = 2.91 P$

IN ADDITION STABILITY MOMENT = $P_V \times 3' = 0.97 \times 3 \times P = 2.91 P$

∴ STEEL SET IS PUSHED AGAINST WALL DURING JACKING

(PURPOSE III.I AND III.J NOT USED)

PURPOSE K   A) SHIM PLATE CALCULATION FOR W8X31 STEEL SETS

MOST CRITICAL LOADING CONDITION FROM COMPUTER OUTPUT:

STATIC LOADING - STATION 10+00 STEEL SET AT 2'-0' SPACING.
MEMBER #39.

SHEAR = -10.49 KN; AXIAL = 548.8 KN; MOMENT = 5.01 KN.M

$\therefore V = 10.49\,KN \times 0.225\,{}^{K}/_{KN} \times 0.61^{*} = 1.44^{K}$

$P_d = 548.8\,KN \times 0.225\,{}^{K}/_{KN} \times 0.61^{*} = 75.26^{K}$

$M = 5.01\,KNM \times 0.738\,{}^{KFT}/_{KN} \times 0.61^{*} = 2.26^{K'}$

* COMPUTER RUN IS BASED ON TRIBUTARY WIDTH OF 1.0 METER, WHILE STEEL SETS ARE TO BE PLACED AT 2'-0" $\therefore$ 2.0' ÷ 3.28' = 0.61 LOAD FACTOR.

CONTACT AREA OF THE END PLATE = $A_c = 8'' \times 8'' = 64\,in^2$

I = MOMENT OF INERTIA OF THE PLATE $= \dfrac{8(8)^3}{12} = 341.33\,in^4$

$c = 4''$; $\dfrac{Mc}{I} = \dfrac{2.26(12)(4)}{341.33} = 0.318$ ksi

$\dfrac{P_d}{A_c} = \dfrac{75.26}{64} = 1.18$ ksi

P = MAXIMUM BEARING ON END PLATE $= \dfrac{P_d}{A_c} \pm \dfrac{Mc}{I}$

$P = 1.18 \pm 0.318 = \begin{cases} 1.498 \simeq 1.5\,ksi \\ 0.862 \end{cases}$

PURPOSE III,K A) SHIM PLATE CALCULATION FOR W8X31 STEEL SETS

LOADS:

P = MAXIMUM BEARING PRESSURE ON END PLATE = 1.5 ksi

P_S = MAX. BRG ON SHIM PLATE

$= 1.5 \text{ ksi } \dfrac{\text{AREA END PLATE}}{\text{AREA SHIM PLATE}}$

AREA OF SLOTS = A_{SL}

$A_{SL} = 2 \times \left[\left(6.25 - \frac{1.25}{2}\right) \times 1.25 + \frac{1}{2}\left(\pi \times \frac{1.25^2}{4}\right)\right] = 15.28 \text{ in}^2$

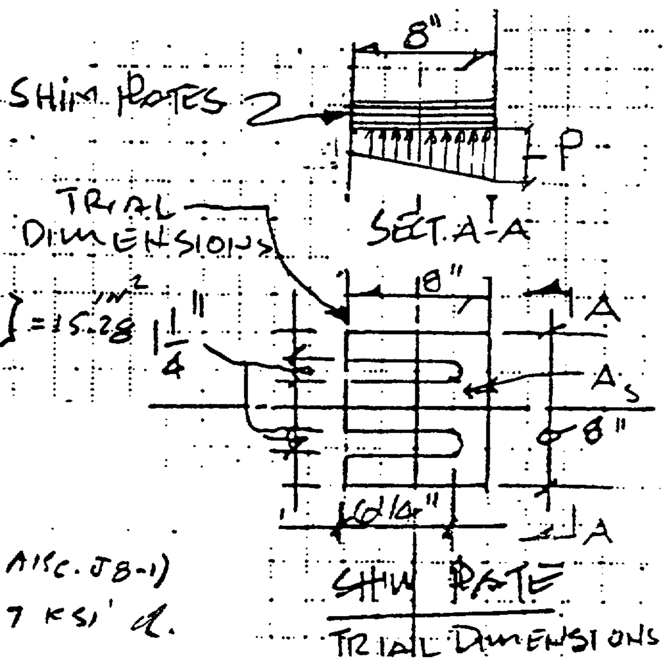
AREA OF SHIM PLATE = A_S

$A_S = 64 \text{ in}^2 - 15.28 = 48.72 \text{ in}^2$

$P_S = 1.5 \text{ ksi} \times \dfrac{64}{48.72} = 1.97 \text{ ksi}$

ALLOW BEARING = 0.9 x 36 ksi (AISC. J8-1)

= 32.4 ksi >> 1.97 ksi ✓.



SHIM PLATES IS OK FOR MAXIMUM PRESSURE

NOTES;

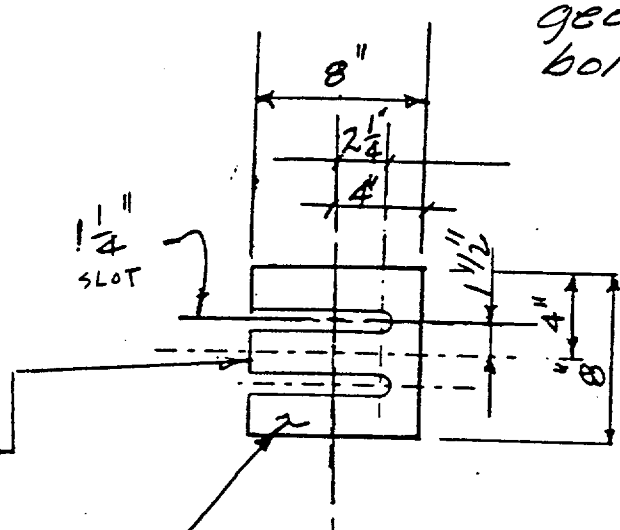
SHIM PLATES PACK RECOMMENDED:
- 1/8" - PLATE
- 1/4" - "
- 1/2" - "
- 1" - "

TOTAL THICKNESS OF THE SHIM PLATES PACK WILL BE CONTRACTOR'S OPTION AND WILL BE BASED ON THE INSERT HEIGHT.

SUMMARY

PURPOSE III.K A)

Shim plate design is based on the geometry of the bolted connections.



SHIM PLATE PACK
2 EA. 1/8" PLATE
2 EA. 1/4" PLATE
2 EA 1/2" PLATE
1 1" PLATE
CONSTRUCTOR OPTION.

PLATES ASTM - A 36 (minimum

SHIM PLATES

PURPOSE K 8) SHIM PLATE CALCULATION FOR W6x20 STEEL SET.

MOST CRITICAL LOADING CONDITION FROM COMPUTER OUT PUT.

STATION 27+00 ; STATIC LOADING ; NODE 39 ;

SHEAR 1776 N ; AXIAL 55460 N ; MOM 425.3 N.M

COMPUTER OUTPUT IS BASED ON 1 METER SPACING WHILE STEEL SETS ARE SPACED AT 4'-0"

$\frac{4.0'}{3.28} = 1.22 \Rightarrow$ FACTOR LOADS

1 KIP = 0.225 KN ; 1 KIP FT = 0.738 KNM

$V = 1.776 \text{ KN} \times 0.225 \times 1.22 = 0.49^{K}$

$P = 55.46 \text{ KN} \times 0.225 \times 1.22 = 15.22^{K}$,

$M = 0.425 \text{ KNM} \times 0.738 = 0.38^{K}$

p = PRESSURE ON END PLATE $= \frac{P}{A_c} \pm \frac{Mc}{I}$

$A_c = 6.25'' \times 7.0'' = 43.75 \text{ in}^2$; $c = 7'' \div 2 = 3.5''$

$I = \frac{1}{12} \times 7.5 \, (6.25)^3 = 152.59 \text{ in}^4$

$p = \frac{15.22^{K}}{43.75} \pm \frac{0.38 \times 12 \times 3.5}{152.59} = 0.348 \pm 0.105 = \begin{cases} 0.243 \text{ KSI} \\ 0.453 \text{ KSI} \end{cases}$

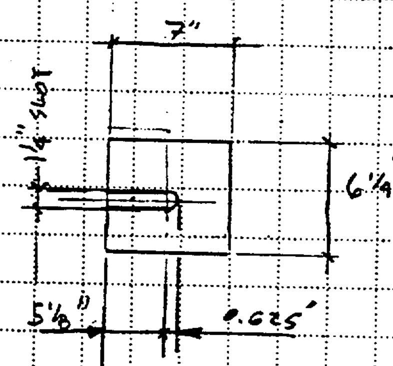
PURPOSE III, K (B) SHIM PLATE CALCULATION FOR W6x20 STEEL SETS.

LOADS; (FROM PREVIOUS SHEET).

$P_2 = 0.453$ ksi = MAXIMUM BEARING PRESSURE ON 6.25" x 7.0" END PLATE = 43.75 in²

AREA OF THE SHIM PLATE = 43.75 in² − AREA OF THE SLOT.

AREA OF THE SLOT = A_S



$A_S = 2 \times [1.25 \times 3.125 + (.5 \times \pi \times .625^2)]$

$= 2(3.906 + 0.613) = 9.04\ in^2$

AREA SHIM PLATE = 43.75 − 9.04
= 34.71 in²

MAXIMUM BEARING PRESSURE ON THE SHIM PLATE

$= P_2 \times \dfrac{\text{AREA END PLATE}}{\text{AREA OF SHIM PLATE}} = \dfrac{43.75}{34.71} \times 0.453 = 0.571$ ksi

0.571 ksi < ALLOW. BEARING PRESSURE FOR STEEL

∴ SHIM PLATES ok. FOR BEARING PRESSURE

USE 6¼ x 7" PLATES - ASTM A-36, MINIMUM.

Purpose III.L Steel Wedges

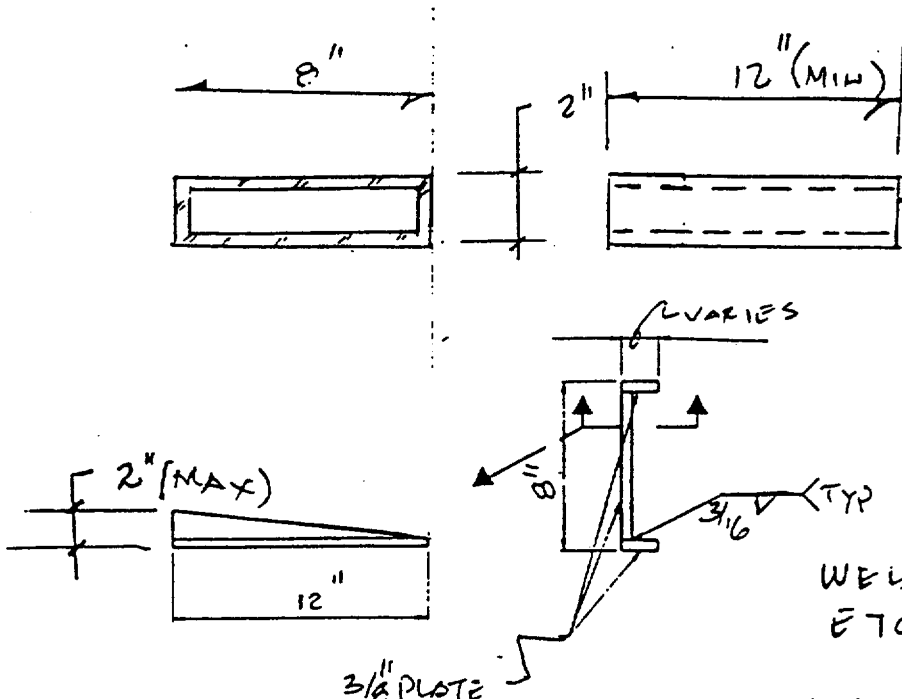
Wedges and blocking are installed behind the steel set as required to ensure that the steel set is in positive contact with the excavated surface. (See Section 2.3) Wedge and blocking material can be of wood or metal. A typical steel wedge made from plate or tubing is shown on p. III-124. Other suggested options are shown in Attachment IX, p. IX-14. Alternate designs by the Constructor are subject to A/E approval.

Crushing of the wedge material may occur during rock loading, however positive contact between the excavated surface and the steel set will remain. Numerical analysis of the wedge is not required.

Blocking, backfill or other materials placed in voids shall be selected by the Constructor and are subject to A/E approval.

(NOT USED)

SUMMARY



CUT FROM TS 8 x 2 x 3/8 (STRUCTURAL TUBING.) with minimum yield stress > 36 KSI

WELDING ELECTRODE E70XX OR E70XT

3/8" PLATE ASTM A 36, MIN.

ALTERNATE DESIGN BY CONSTRUCTOR SHALL BE REVIEWED BY A/E.

STEEL WEDGES

Structural Tubing or Plate Shown