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Registered Professional Engineer State of Nebraska

Robert A. Christiansen E-4623

LOAD AND STRESS ANALYSIS

TELEX/HYGAIN PRODUCT 125 STEEL CRANK UP TOWER

BY
MORRIS STOVER

NOVEMBER 16, 1982

REVIEWED & APPROVED
Robert A Christiansen
10/16/82

THIS REPORT PROVIDES AN ANALYSIS OF LOADS AND RESULTING STRESSES FOR TELEX/HYGAIN PRODUCT 125 CRANK UP TOWER.

DESIGN CRITERIA

1. STRUCTURAL DESIGN: UBC
2. WIND LOADS: UBC 20
3. DESIGN STRESS: AISC
4. FOOTINGS: UBC/ACI

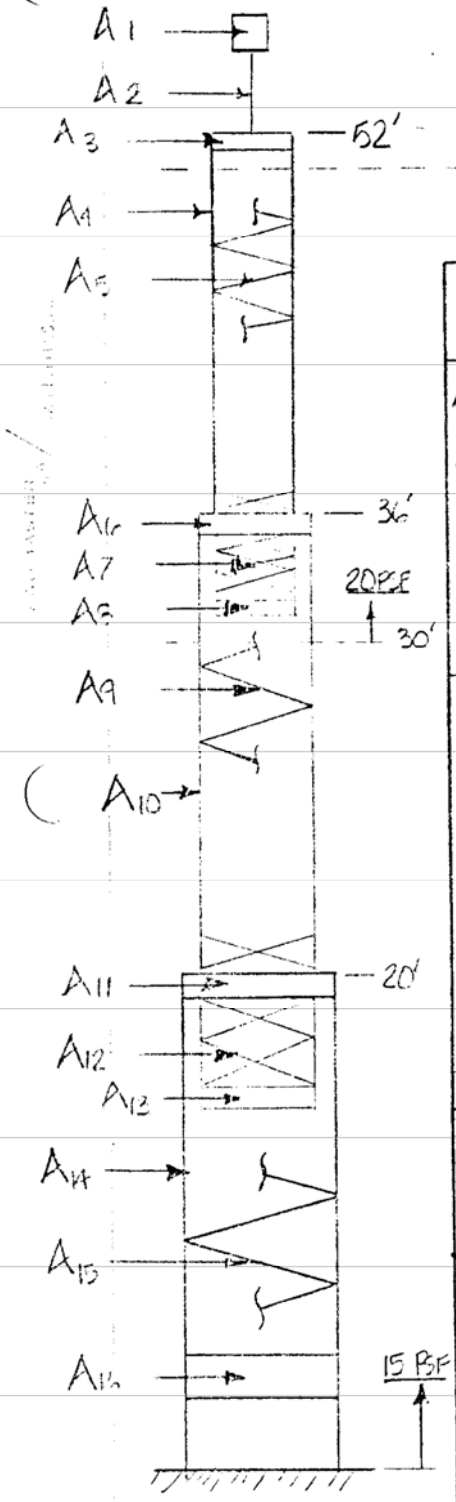
REFERENCES:

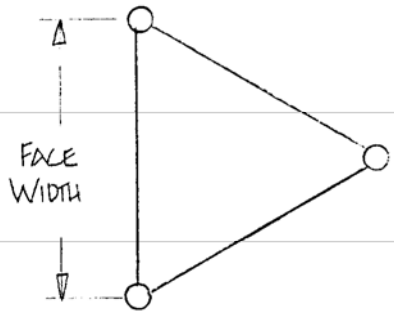
1. STRUCT. ENGR. HNDBK.
2. ASTM STANDARDS
3. AISC STEEL CONSTRUCTION MANUAL
4. ACI 318-77
5. TELEX/HYGAIN TOWER DESIGN & INSTALLATION DRAWING NO. 125-1

REVIEWED & APPROVED
PAGES 1-19, 11-16-82

11/16/82

WIND LOADS - UBC



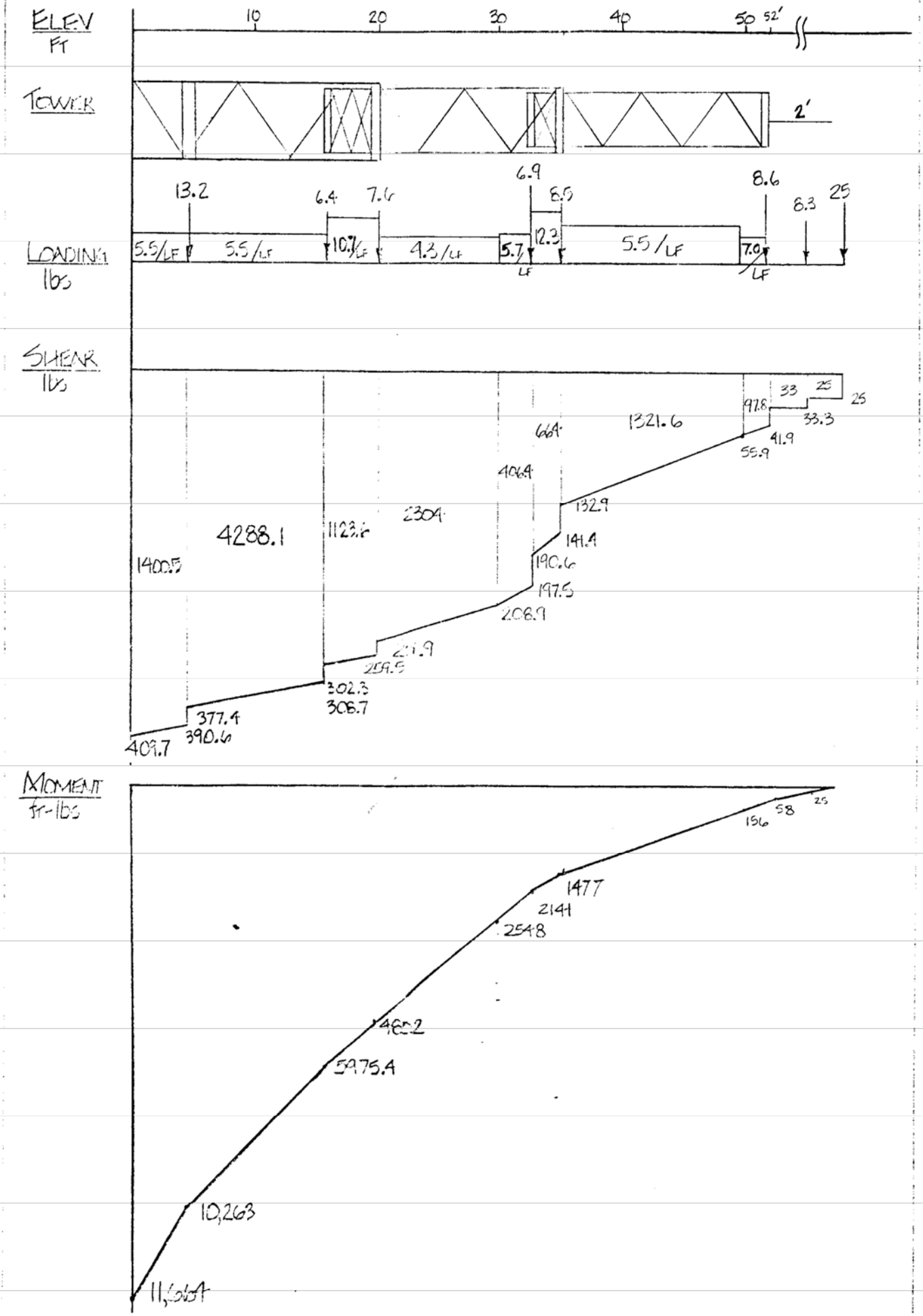


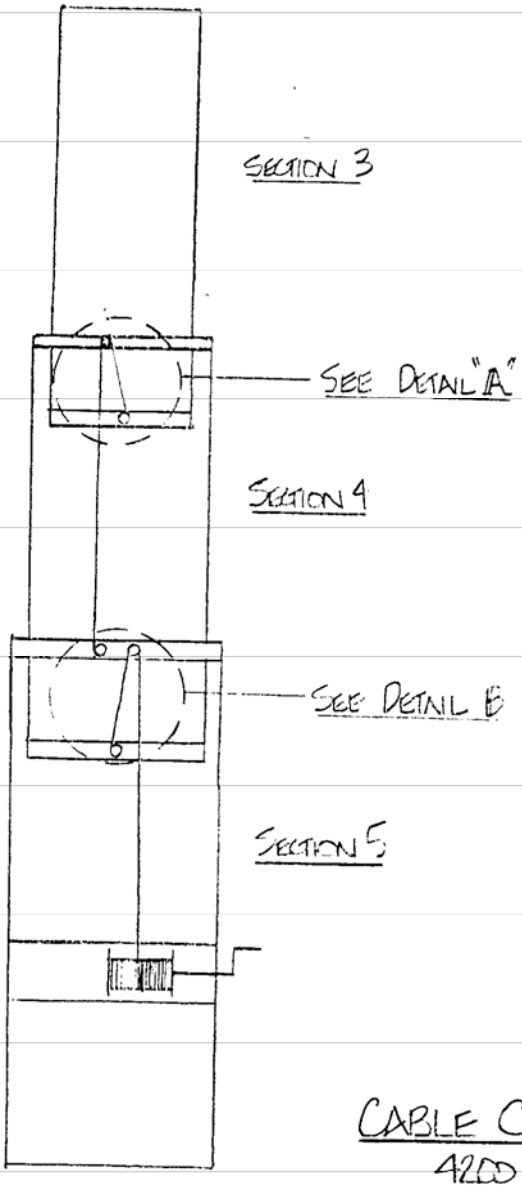
SECTION	FACE WIDTH
3	10.5"
4	12.75"
5	15.187"

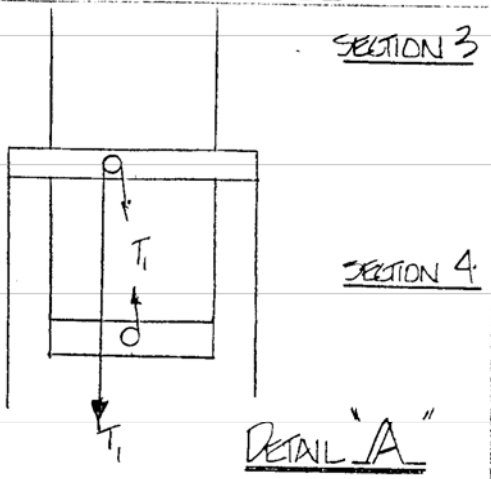
TOWER ELEMENT	ELEMENT SIZE - IN	PROJECTED AREA - ft^2	FACTORS	WIND AREA - ft^2	PSF	WIND LOAD
A_1 - ANTENNA	—	—	—	1.0 ft^2	25	25 lb
A_2 - MAST	24 x 24"	.33	1.0	.33	↓	8.3 lb
A_3 - MTG PL.	3" x 9.5"	.198	1.3 x 1.34	.34	↓	8.6 lb
A_4 - LEG	1" ϕ	.167/ft	1.34	.22/ft	↓	5.6 lb/ft
A_5 - SNGL BRACE	5/16" ϕ	.042/ft	1.34	.056/ft	↓	1.4 lb/ft
A_4 - LEG	1" ϕ	.167/ft	1.34	.22/ft	20	4.4 lb/ft
A_5 - SNGL BRACE	5/16" ϕ	.042/ft	1.34	.056/ft	↓	1.1 lb/ft
A_6 CABLE BRKT	3" x 11.75"	.245	1.3 x 1.34	.43	↓	8.5 lb
A_7 - DBL BRACE	5/16" ϕ	.084/ft	1.34	.112/ft	↓	2.2 lb/ft
A_8 - CABLE BRKT	3" x 9.5	.198	1.3 x 1.34	.34	↓	6.9 lb
A_9 - SNGL BRACE	5/16" ϕ	.048/ft	1.34	.064/ft	↓	1.3 lb/ft
A_{10} - LEG	1" ϕ	.167/ft	1.34	.22/ft	↓	4.4 lb/ft
A_9 - SNGL BRACE	5/16" ϕ	.048/ft	1.34	.064/ft	15	1.0 lb/ft
A_{10} - LEG	1" ϕ	.167	1.34	.22/ft	↓	3.3 lb/ft
A_{11} - SHEAVE BRKT	3" x 13.9"	.290	1.3 x 1.34	.51	↓	7.6 lb
A_{12} - DBL BRACE	5/16" ϕ	.096/ft	1.34	.13/ft	↓	1.9 lb/ft
A_{13} - CABLE BRKT	3" x 11.75	.245	1.3 x 1.34	.43	↓	6.4 lb
A_{14} - LEG	1¼" ϕ	.208/ft	1.34	.278/ft	↓	4.2 lb/ft
A_{15} - SNGL BRACE	5/16" ϕ	.066/ft	1.34	.088/ft	↓	1.3 lb/ft
A_{16} - WINCH PL.	7" x 13.9"	.676	1.3	.879	↓	13.2 lb

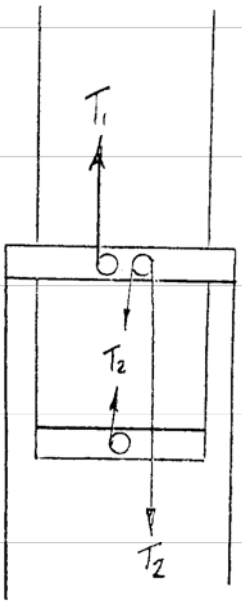
PRODUCT 125
CRANK-UP TOWER

MSS 10-21-82









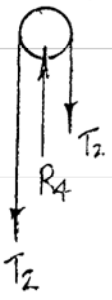
CABLE CAPACITY

4200 lb BREAK STR MAX LOAD 3731 lbs OKAY

WINCH CAPACITY

1500 lb CAPACITY MAX LOAD 3731 lbs OKAY

T_1 = Wt of Section 3
= 136.2#
$R_2 = (2)T_1 = 272.4$ #



T_2 = Wt of Section 4 + R_1
= 100.5 + 272.4
= 372.9 #
$R_4 = 2(T_2) = 746$ lbs

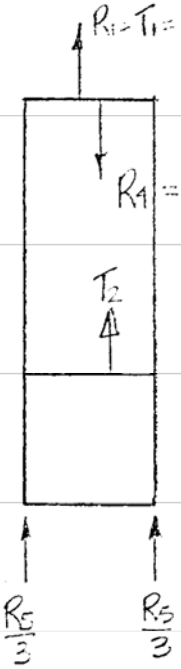
FREE BODY DIAGRAMS OF TOWER SECTIONS FOR THE VERTICAL LOADS

SECTION 3

SECTION 4

R_2 = FORCE ON PULLEY
$= 272.4^{\#}$

SECTION 5



R_5 = R_4 + DEAD LOAD OF SECTION 5 $- R_1 - T_2$
$= 746^{\#} + 163.2^{\#} - 136.2^{\#} - 373^{\#}$
$= 400$ lbs

ALLOWABLE STRESSES
AISC & UBC

MEMBER	SIZE	AREA	L	r	k	kl/r	Fy (KSI)	Fa (KSI)	Fa' (KSI)
SECTION 3									
LEG	1"ϕ x .065	.191	15'	.53"	1.0	45.0	45	22.90	30.46
DIAGONAL SINGLE	5/16 ϕ	.077	12.[illegible]	.378	.8	127.4	36	9.20	12.24
DIAGONAL DOUBLE	5/16 ϕ	.077	12.[illegible]	.378	.8	127.4	36	9.20	12.24
SECTION 4									
LEG	1"ϕ x .095	.270	15'	.522	1.0	46.6	45	22.70	30.27
DIAGONAL SINGLE	5/16" ϕ	.077	12.[illegible]	.378	.8	143.0	36	7.30	9.71
DIAGONAL DOUBLE	5/16" ϕ	.077	12.[illegible]	.378	.8	143.0	36	7.30	9.71
SECTION 5									
LEG	1¼" ϕ x .120	.426	15'	.[illegible]	1.0	37.3	45	23.82	31.68
DIAGONAL SINGLE	3/8" ϕ	.110	15.5'	.374	.8	134.5	36	8.25	10.97
DIAGONAL DOUBLE	3/8" ϕ	.110	15.5	.374	.8	134.5	36	8.25	10.97

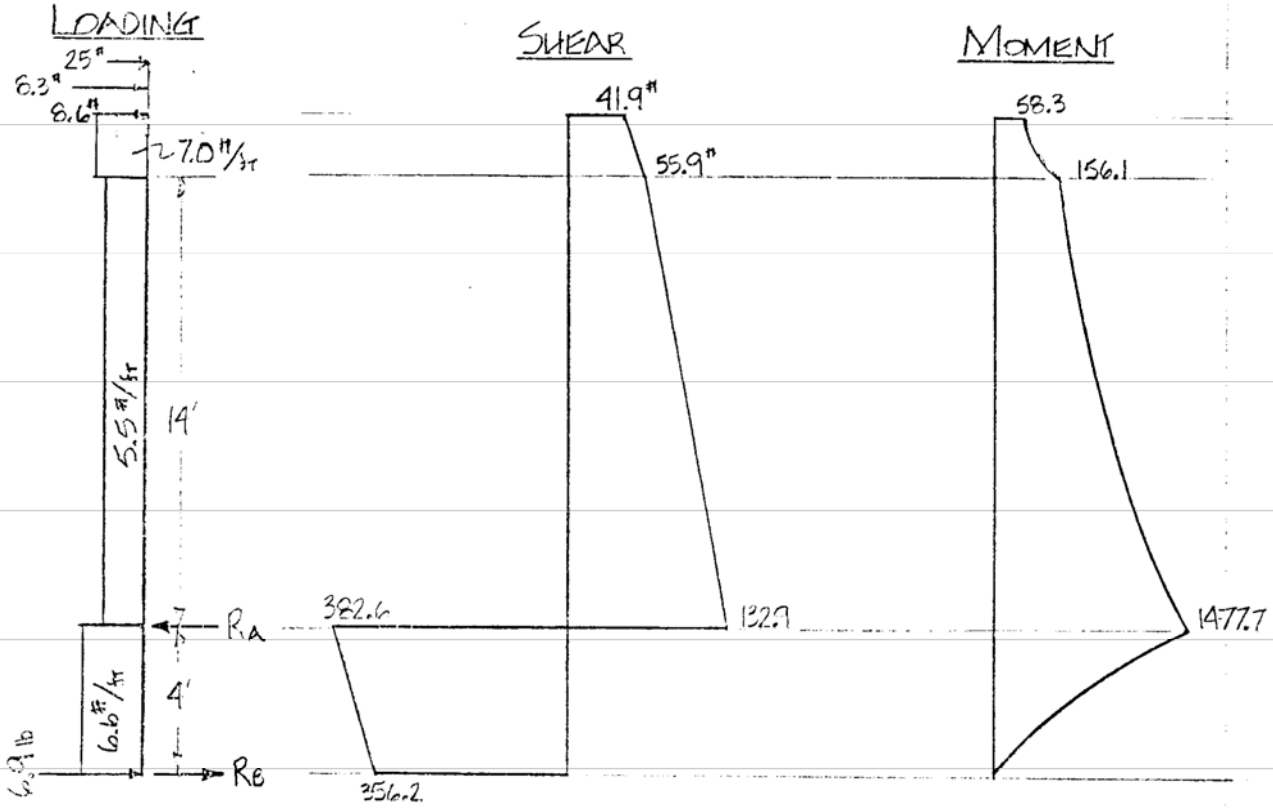
NOTES:

① K FACTOR
- K = 1.0 LEGS
- K = 0.8 SINGLE DIAGONALS
- K = 0.8 DOUBLE DIAGONALS

② Fa - ALLOWABLE STRESS FOR COMPRESSIVE MEMBERS - AISC 1.5.1.3.1 & 1.5.1.3.2

③ Fa' - ALLOWABLE STRESS INCREASED BY 1.33 - UBC

FREE BODY DIAGRAMS
SECTION 3

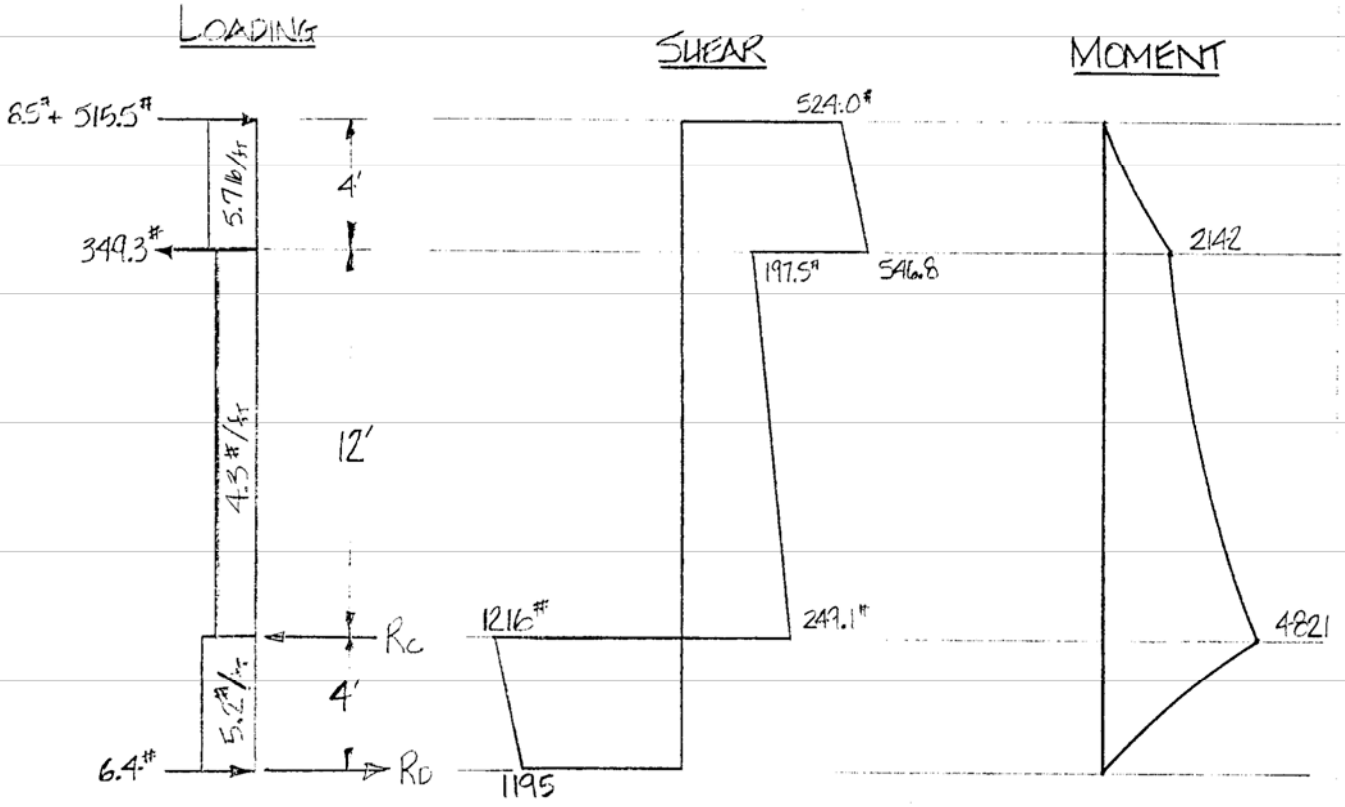


$$4R_A = 25^{\#}(22) + 8.3(21) + 8.6(20) + 7.0(19)2 + 14(5.5)11 + 4(6.6)2$$

$R_A = 515.5$ lbs

$R_B = 349.3$ lbs

FREE BODY DIAGRAM
SECTION 4

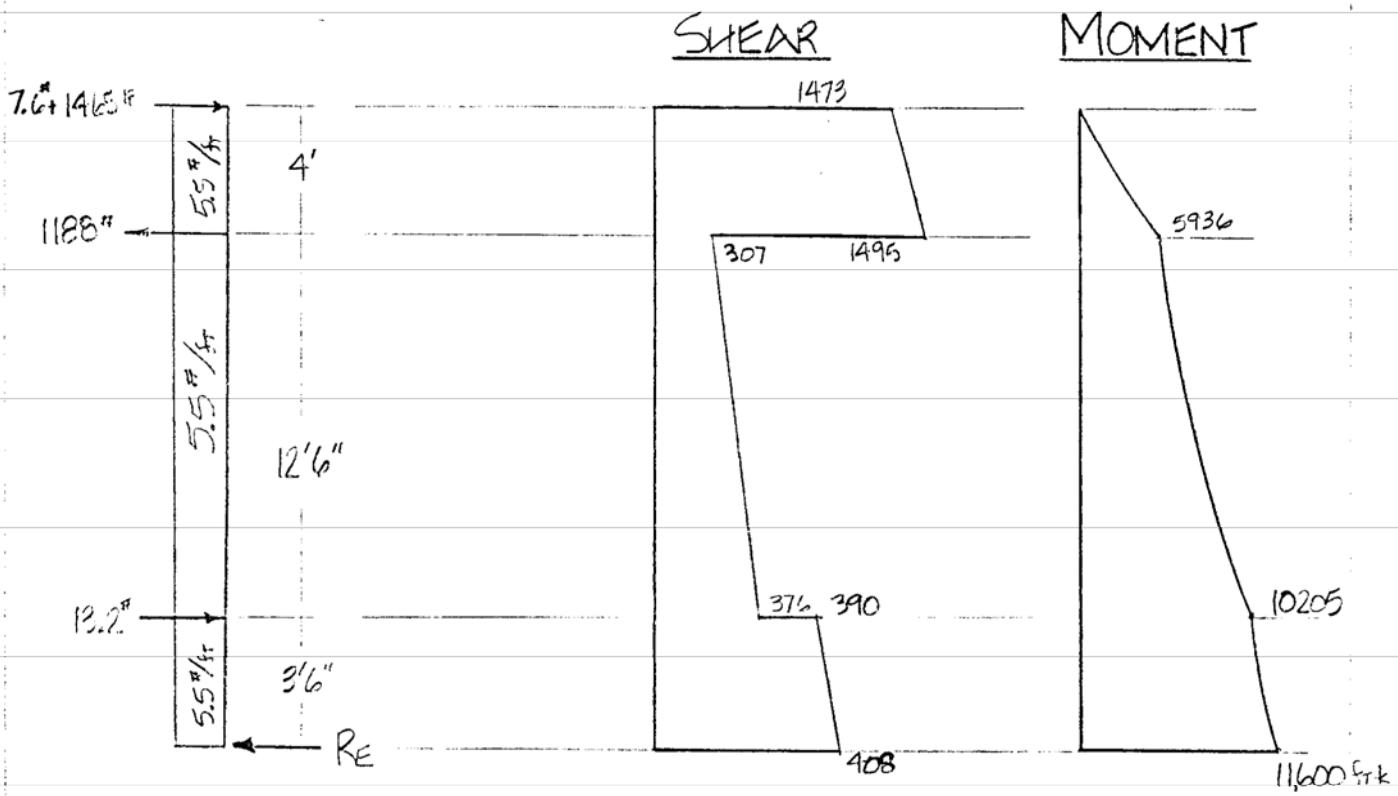


$$4R_C = 524(20) + 5.7(4)18 - 349.3(16) + 4.3(12)10 + 5.2(4)(2)$$

$R_C = 1465$ lbs

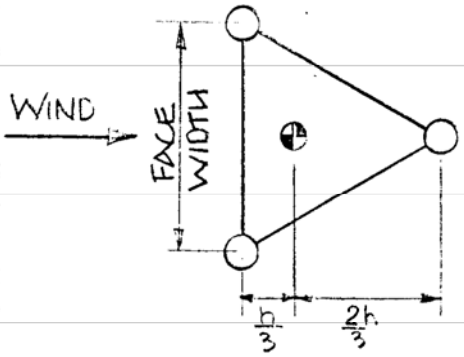
$R_D = 1186$ lbs

FREE BODY DIAGRAMS
SECTION 5



$R_E = 408$ lbs

ACTUAL STRESSES & COMPRESSIVE LOADS - LEGS



h = FACE WIDTH x SIN 60°

M = MOMENT DUE TO WIND

$= F \times \frac{2}{3}h + 2 \times \frac{F}{2} \times \frac{h}{3}$

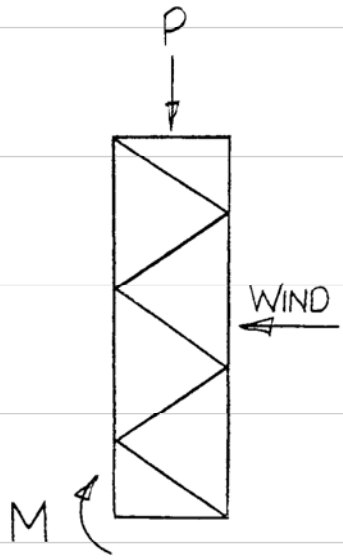
$= F \times h$

HENCE:

$F = \frac{M}{h}$

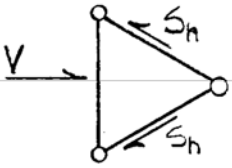
$P = \frac{P_{DL}}{3} + \frac{M \times 12}{h}$

$f_a = \frac{P}{A}$



MEMBER	MOMENT ft-lbs	LOAD - $\frac{P_{DL}}{3}$ lbs	h IN	AREA IN^2	P lbs	f_a PSI	F_a PSI	F_a / f_a
SECTION 3 ELEV 36'	1478	45	9.1	.191	1994	10,440	30,460	2.92
SECTION 4 ELEV 20'	4821	124	11.0	.270	5,383	19,940	30,270	1.52
SECTION 5 ELEV 0'	11,600	133	13.2	.426	10,680	25,070	31,680	1.26

ACTUAL STRESSES & COMPRESSIVE LOADS – DIAGONAL BRACES

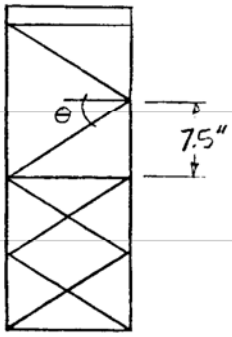


V = MAX SHEAR ON TOWER SECTION

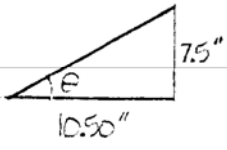
$S_h = V/2\cos 30° = .577V$ = SHEAR ON ONE FACE

$S = S_h/\cos\theta$ = SHEAR INTO THE DIAGONALS

$f_a = S/A$

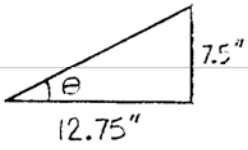


SECTION 3



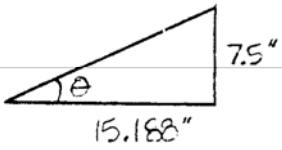
$\theta_3 = 35.5°$

SECTION 4



$\theta_4 = 31.0°$

SECTION 5



$\theta_5 = 26.3°$

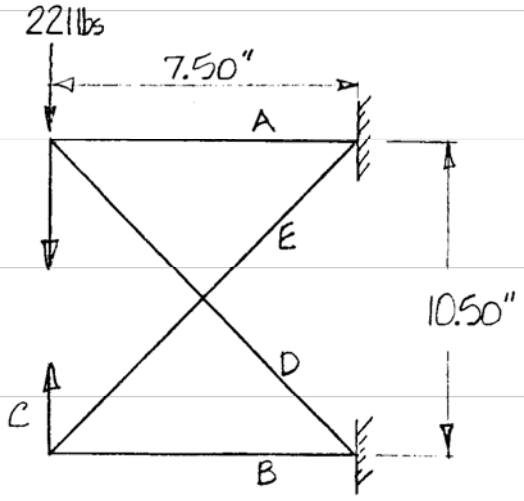
MEMBER	V lbs	S lbs	AREA IN^2	f_a PSI	F'_a PSI	F'_a/f_a
SECTION 3						
SINGLE BRACE	133	94.3	.077	1225	12,240	10.0
DOUBLE BRACE	383	143	.077	1857	12,240	6.60
SECTION 4						
SINGLE BRACE	547	368	.077	4779	9,710	2.03
DOUBLE BRACE	1216	432	.077	5610	9,710	1.73
SECTION 5						
SINGLE BRACE	408	263	.110	2391	10,970	4.59
DOUBLE BRACE	1495	524	.110	4764	10,970	2.30

ANALYSIS OF THE DOUBLE BRACING

SECTION 3

APPLYING THE METHOD OF LEAST WORK

$V_{MAX} = 383$ lbs
$S_h = 221$ lbs



MEMBER	A IN^2	ℓ IN	S' lb	S' ℓ/A	U	US' ℓ/A	U^2 ℓ/A	RU	S' + RU ①
A	.191	7.5	+158	+6204	+.71	+4405	19.79	-74.76	+83.2
B	.191	7.5	0	0	+.71	0	19.79	-74.76	-74.8
C	.38	10.5	0	0	+1.0	0	27.63	-105.3	-105.3
D	.077	12.9	-272	-45,570	-1.23	+56,050	253.5	+129.5	-142.5
E	.077	12.9	0	0	-1.23		253.5	+129.5	+129.5
						60,455	574.1		

$$R = -\frac{60455}{574.1} = -105.3$$

NOTES:

(+) — TENSION
(−) — COMPRESSION

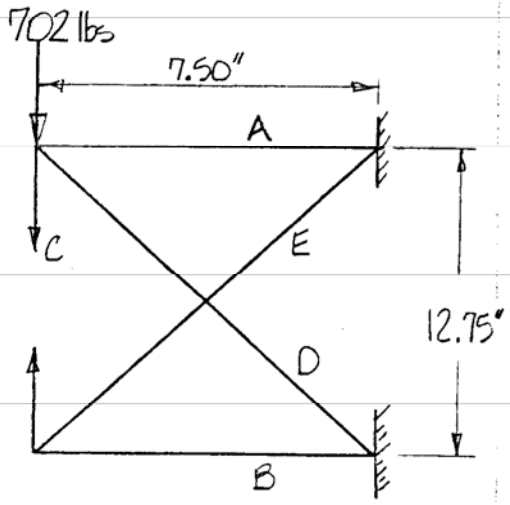
① S' + RU = S = SHEAR INTO MEMBER (REFER TO PAGE 11/19)

ANALYSIS OF THE DOUBLE BRACING
SECTION 4

METHOD OF LEAST WORK

$V_{MAX} = 1216$ lbs

$S_h = 702$ lbs



MEMBER	A IN^2	ℓ IN	S' lbs	$S'\ell/A$	u	$uS'\ell/A$	$u^2\ell/A$	Ru	S' + Ru ①
A	.27	7.5	+413	+11,470	+.59	+6770	9.67	-195.1	+217.9
B	.27	7.5	0	0	+.59	0	9.67	-195.1	-195.1
C	.38	12.75	0	0	+1.0	0	33.55	-330.6	-330.6
D	.077	14.8	-815	-156,650	-1.16	+181,710	258.6	+383.5	-431.5
E	.077	14.8	0	0	-1.16	0	258.6	+383.5	+383.5
						188,480	570.2		

$$R = -\frac{188{,}480}{570.2} = -330.6$$

NOTES:

(+) - TENSION

(−) - COMPRESSION

① $S' + Ru = S =$ SHEAR INTO MEMBER (REFER TO PAGE 11/19)

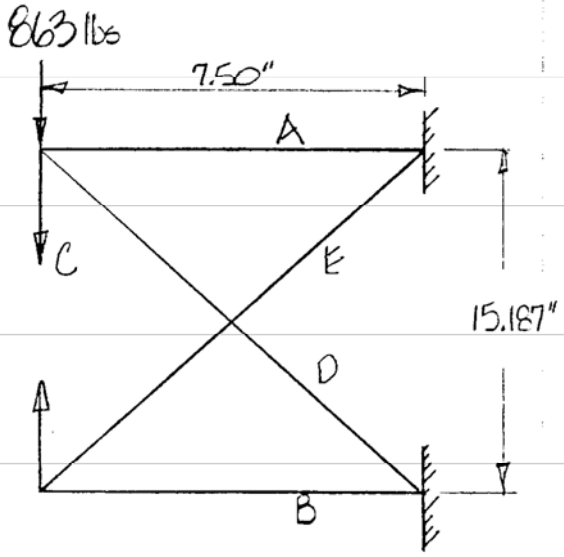
ANALYSIS OF THE DOUBLE BRACING

SECTION 5

METHOD OF LEAST WORK

$V_{MAX} = 1495$ lbs

$S_h = 863$ lbs



MEMBER	A IN^2	ℓ IN	S' lbs	$S'\ell/A$	μ	$\mu S'\ell/A$	$\mu^2\ell/A$	R_μ	$S'+R_\mu$ ①
A	.426	7.5"	+425	7480	+.494	+3695	4.30	-192.2	+232.8
B	.426	7.5"	0	0	+.494	0	4.30	-192.2	-192.2
C	.38	15.19"	0	0	1.0	0	40.0	-389.1	-389.1
D	.110	16.9"	-960	-147,490	-1.12	+165,190	192.7	+437.8	-524.2
E	.110	16.9"	0	0	-1.12	0	192.7	+437.8	+437.8
						168885	434.0		

$$R = -\frac{168,885}{434.0} = -389.1$$

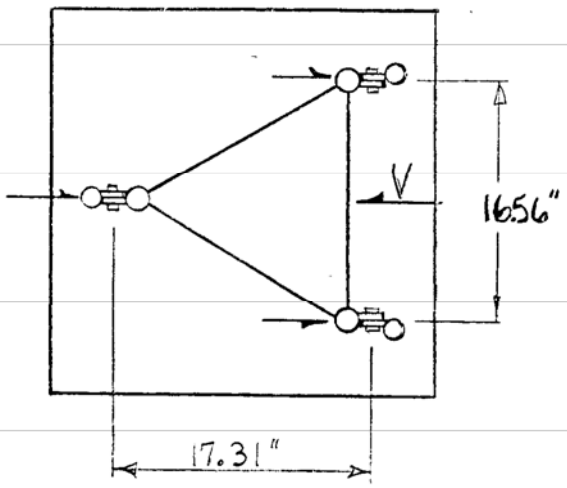
NOTES:

(+) — TENSION

(−) — COMPRESSION

① $S' + R_\mu = S$ = SHEAR INTO MEMBER (REFER TO PAGE 11/19)

FOUNDATION – BOLTS IN FRICTION CONNECTION
USING A307 BOLTS



VERTICAL LOADS

$P_{DL} = 133 \text{ lbs/LEG}$

$M = 11{,}600 \text{ ft-lbs}$

$P = \frac{11{,}600 \times 12}{17.31} \pm 133$

$= 8173 \text{ lbs} \quad (8040 \text{ TENSION})$

HORIZONTAL FORCE

$V = 408 \text{ lbs}$

RESULTANT FORCE

$= (408^2 + 8173^2)^{1/2}$

$= 8184 \text{ lbs}$

CHECK THE CAPACITY 2-¾" A307 BOLTS

$\text{FORCE/BOLT} = F_v \frac{\pi d^2}{4}$

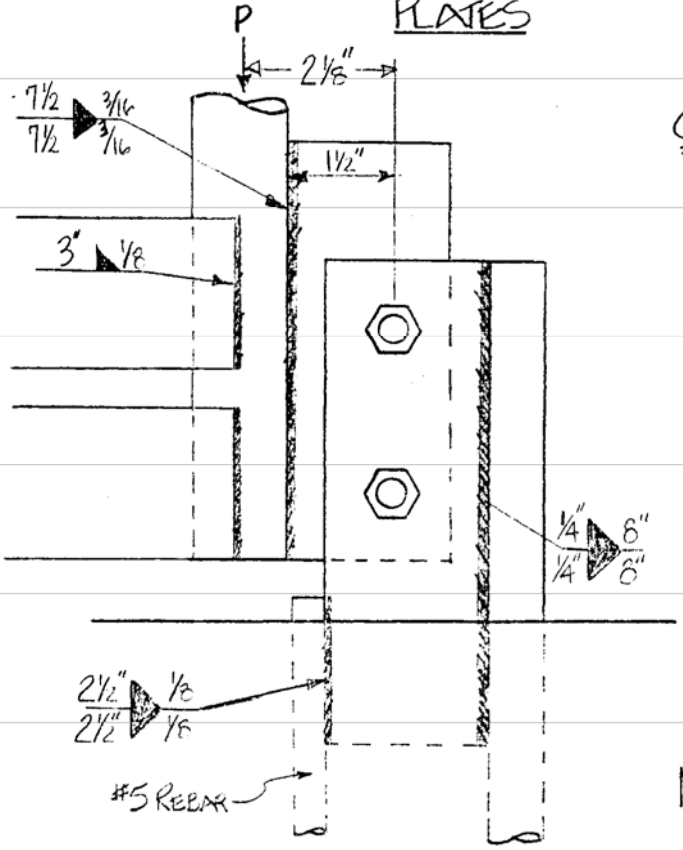
$= 10 \text{ KSI} \frac{\pi (.75)^2}{4}$

$= 4418 \text{ lbs/BOLT} \times 2 \text{ BOLTS} = 8836 \text{ lbs} \times 1.33 \text{ (UBC WIND FACTOR)}$

$= 11750 \text{ lbs}$

$\text{MARGIN} = \frac{\text{ALLOWABLE FORCE}}{\text{ACTUAL FORCE}} = \frac{11{,}750}{8184} = 1.44$ OKAY

FOUNDATION – WELDS ON BASE CONNECTION PLATES



CHECK CAPACITY OF TOWER EARS

ECCENTRIC LOAD – Pe CONDITIONS FROM AISC PG 4-76 TABLE XIX

P = 8184 lbs ℓ = 7.5" D = 3

$a\ell$ = 1.5" a = .20 C_1 = 1.0

k = 0.0

FROM CHART C = 1.39

$P_{ALLOWABLE} = CC_1D\ell \times 1.33$ (WIND FACTOR)

$= 1.0(1.39)3(7.5) \times (1.33)$

$= 41.6$ KIPS

$MARGIN = \frac{41.6\ K}{8.2\ K} = 5.1$ OKAY

CHECK CAPACITY OF FOUNDATION EARS

TOTAL WELD CAPACITY – E70

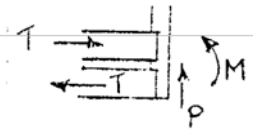
$P_{ALLOWABE} = .707(.3)70(.125(2\frac{1}{2}")2 + .25(8")2)$

$= 68.7$ KIPS $\times$ 1.33

$= 91.3$ KIPS

$MARGIN = \frac{91.3\ K}{8.2\ K} = 11.1$ OKAY

CHECK THE CAPACITY OF THE BASE BRACKETS



M = MOMENT CAUSED BY ECCENTRIC LOAD = P x 2⅛"

M = 17,391 IN-lbs

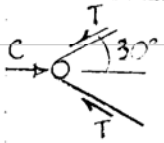
$P_{ALLOW} = .707(70).3(3")\ ⅛$

$= 5568 \times 1.33$

$MARGIN = \frac{7405}{2869} = 2.6$ OKAY

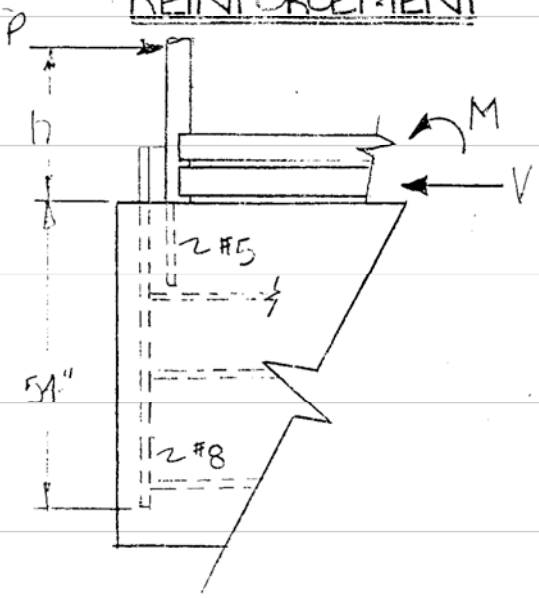
COUPLE T RESISTS M

$COUPLE = \frac{17391\ in\text{-}lbs}{3.5"} = 4969$ lbs



$T = \frac{4969}{2 \times \cos 30} = 2869$ lbs

FOUNDATION & FOUNDATION REINFORCEMENT



CHECK IMBEDMENT LENGTHS - ℓ_d

#8 BARS REF. ACI

$\ell_d = .04 A_b f_y / \sqrt{f'_c}$

$\ell_d = .04(.79)\,40{,}000/\sqrt{2000}$

$\ell_d = 28.33''$

ACTUAL $\ell_d = 54''$ OKAY

#5 BARS

$\ell_d = 10.7''$

ACTUAL $\ell_d = 16''$ OKAY

CHECK BAR TENSION

$\frac{P}{2 \text{ BARS}} = T = \frac{8040}{2} = 4020$ lbs/PER BAR

$P_{ALLOW} = AREA_{\#5} \times 40{,}000$ PSI

$= .31 \times 40{,}000$ PSI

$= 12{,}400$ lbs

MARGIN $= \frac{12{,}400}{4020} = 3.1$ OKAY

CHECK FOUNDATION DEPTH

M = 11,600 ft-lbs V = 408 lbs V = P

$h = \frac{M}{V} + .5 = \frac{11600}{408} + .5 = 28.93'$

$d = \frac{A}{2}\left(1 + \sqrt{1 + \frac{4.36h}{A}}\right)$ REF. UBC

$d_{ALLOW} = \frac{.339}{2}\left(1 + \sqrt{1 + \frac{4.36(28.93)}{.339}}\right)$

$= 3.44$ ft

SPECIFIED DEPTH = 5.50 ft OKAY

FOR CLASS 3 SOILS
ISOLATED FOOTING

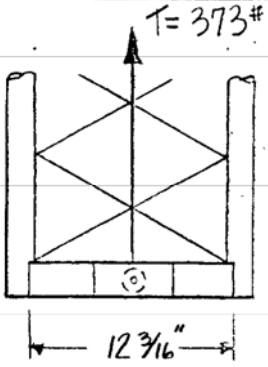
$S_1 = 200(2)1.42 = 568$

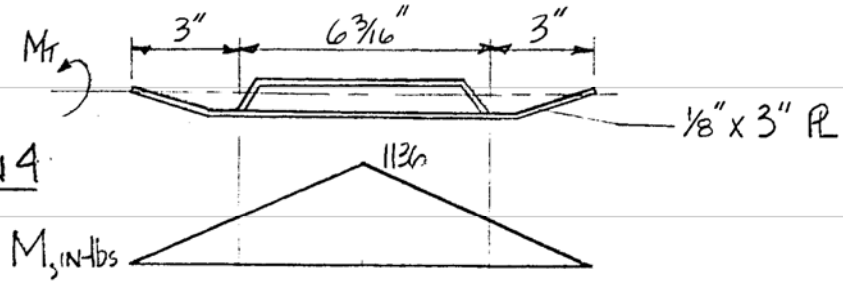
$B = \frac{42(1.414)}{12} = 4.950$

$A = \frac{2.34P}{S_1 B} = \frac{2.34(408)}{568(4.950)}$

$A = .339$

CABLE BRACKETS





SECTION 4

$M_{MAX} = PL/4 = 1136$ IN lbs

$M_{3"} = \left(\frac{3"}{6\,3/32"}\right) 1136 = 560$ IN lbs

$S_{x\,DOUBLE\ PLATE} = \frac{(1/8)\,3^2}{6} \times 2 = .375\,IN^3$ $\quad f_{bx} = \frac{1136\ IN\ lbs}{.375} = 3.03\ KSI$

$S_{x\,SINGLE\ PLATE} = \frac{(1/8)3^2}{6} = .1875\,IN^3$ $\quad f_{bx} = \frac{560\ IN\cdot lbs}{.1875\ IN^3} = 2.99\ KSI$

M_T = MOMENT DUE TO ECCENTRICITY = Pe

$e = 0"$ $\quad \therefore M_T = 0$

MARGIN

$\frac{F_b}{f_b} = \frac{24\ KSI}{3.03\ KSI} = 7.9$ OKAY

WELD CAPACITY

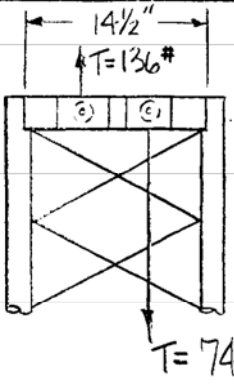
$P_{ALLOW} = .707\,(1/8).3\,(70) \times 6"$

$= 11.1$ KIPS

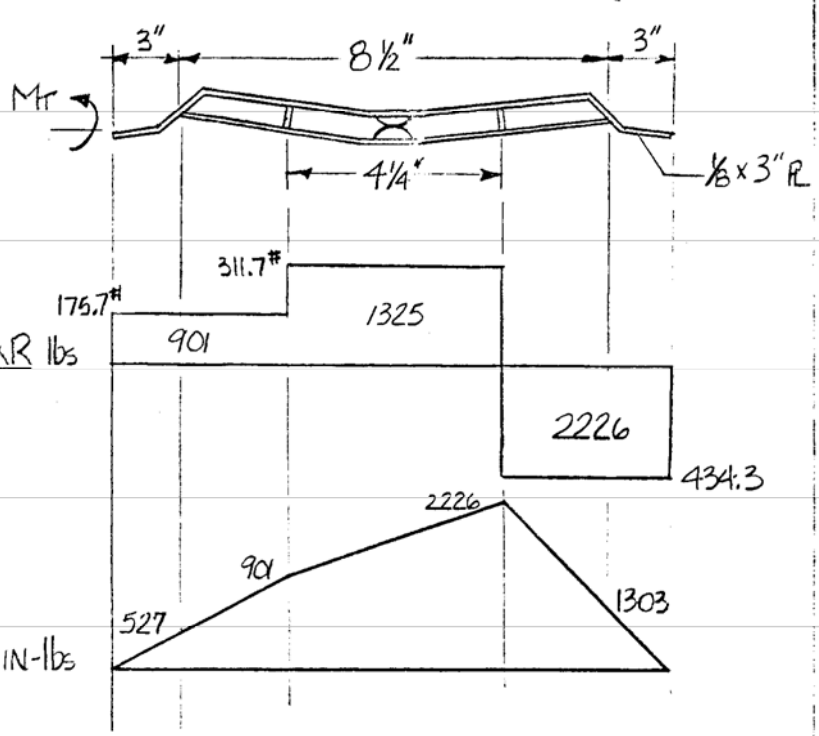
MARGIN (WELDS)

$\frac{P_{ALLOW}}{P_{ACTUAL}} = \frac{11.1\ KIPS}{.373} = 29.9$ OKAY

SHEAVE BRACKETS



SECTION 5



SHEAR lbs

MOMENT, IN-lbs

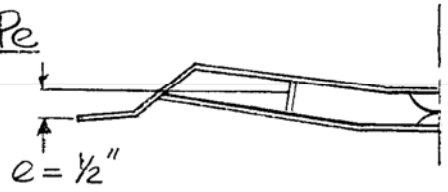
$f_{bx\ SINGLE\ PLATE} = \frac{1303\ IN\ lbs}{.1875\ IN^3} = 6.95\ KSI$

$f_{bx\ DOUBLE\ PLATE} = \frac{2226\ IN\ lbs}{.375\ IN^3} = 5.94\ KSI$

Mt = MOMENT DUE TO TORSION = Pe

$Mt = 434.3^{\#} \times \frac{1}{2}"$

$= 217.2\ IN\ lbs$



① f_T = SHEAR STRESS = $\frac{Mt}{.333\,b^2 d}$

$= \frac{217.2}{.333(⅛)^2 3} = 13.9\ KSI$

② $f_b = \frac{f_{bx}}{2} + \sqrt{\left(\frac{f_{bx}}{2}\right)^2 + f_T^2} = \frac{6.95}{2} + \sqrt{\left(\frac{6.95}{2}\right)^2 + 13.9^2}$

$f_b = 17.8\ KSI$

WELD CAPACITY = .707(⅛).3(70) 3
= 5.57 KIPS

MARGIN

$\frac{F_b}{f_b} = \frac{24\ KSI}{17.8\ KSI} = 1.35$ OKAY

MARGIN - WELDS

$\frac{P_{ALLOW}}{P_{ACTUAL}} = \frac{5.57\ KIPS}{.434} = 12.8$ OKAY

NOTES:

① - FROM - STD HANDBOOK FOR MECH. ENGINEERS by BAUMEISTER & MARKS

② - FROM - ELEMENTS OF STRENGTH OF MATERIALS by TIMOSHENKO & YOUNG

SUPPLEMENT TO [illegible] 12[illegible] LOADS & [illegible] ANALYSIS DATED 11/[illegible]/83

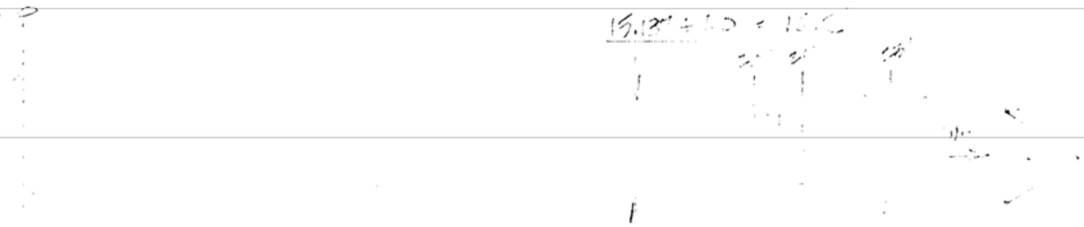
- CONSIDER LOADS [illegible] WITH [illegible] WIND [illegible] DUE TO [illegible] ON TOWER AND ANTENNA, WITH [illegible]

- ASSUME:

1. THE [illegible] [illegible] [illegible] [illegible] [illegible] (> 1/2 [illegible]) [illegible] CONSERVATIVE

2. ANTENNA [illegible] [illegible] TOWER [illegible] WITH 1/2 [illegible]

TILE: ?



PROJECTED AREA OF [illegible] TAB

= 16[illegible] × 1.[illegible] = 1.7[illegible] FT² [illegible]

FLAT SURFACE

PROJECTED AREA OF NOMINAL ANTENNA

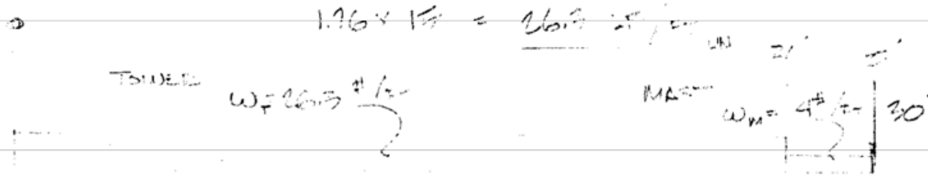
= 1[illegible] × [illegible] = [illegible]

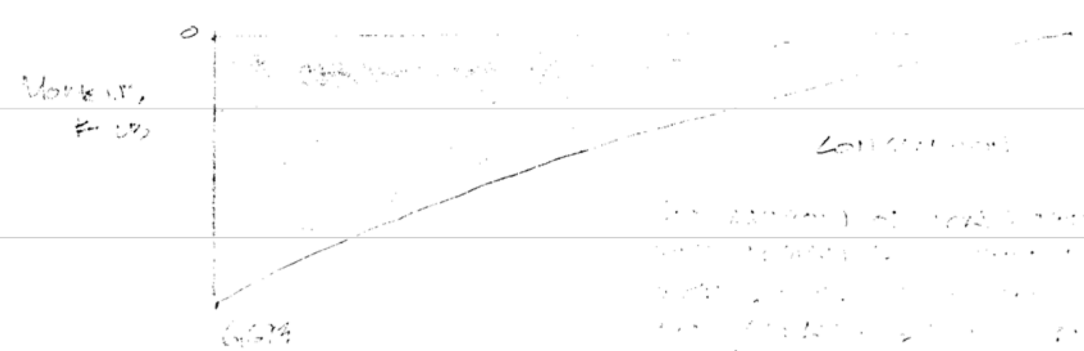
WIND LOAD ON TOWER [illegible] 30

1.76 × 15 = 26.5 [illegible]

WIND LOAD ON MAST [illegible]

2 × 15 = 30[illegible]





CONCLUSIONS

[illegible]

7/8/83