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CALCULATIONS FOR SELECTION OF A ROOF FRAME FOR A GYNASIUM

by

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A THESIS SUBMITTED TO THE GRADUATE COMMITTEE
FOR THE DEGREE OF

MASTER OF SCIENCE

IN

ARCHITECTURAL ENGINEERING

GIFT OF JOHN F. POULTON.

THREE HINGED CONCRETE FRAME

SIZE:
24" UNIFORM THICKNESS.
BEAM DEPTH VARIES FROM 28" TO 60"
COLUMN " " " 26 TO 50"

ECH 538 THESIS. Glover PAGE 1

SNC. 3 HINGED ARCH

Bldg is 90' sq. c-c with FRAMES 15' o-c.
IT IS 25' AT EXTERIOR & 33' High AT CENTER
$f_s = 20{,}000$ $f_c = 30{,}000$, $n = 10$ $f_c = 1350$

30,000,000

DESIGN OF CONCRETE slab

L = 15'

LOADING

ROOFING	METAL + 1 LAYER FELT	2 # / sq. ft.
slab	ASSUMN 4" slab	48 # / sq. ft.
CORK FINISH	(sound board)	2 # / sq. ft.
LIVE LOAD	-	25 # / sq. ft
		77 # / sq. ft.

$$+M = \frac{wl^2}{14} = \frac{77 \times 15^2}{14} = 1.24$$

$$+M = \frac{wl^2}{16} = \frac{77 \times 15^2}{16} = 1.08$$

$$-M = \frac{wl^2}{10} = \frac{77 \times 15^2}{10} = 1.73$$

$$-M = \frac{wl^2}{11} = \frac{77 \times 15^2}{11} = 1.58$$

$$-M = \frac{wl^2}{28} = \frac{77 \times 15^2}{28} = .62$$

$$K = \frac{M}{bd^2} = \frac{1.73 \times 12}{12 \times 3^2} = 192$$ o.k. check proved 3½" slab would exceed 235 allowable.

$$A_s = \frac{M}{f_s jd} = \frac{1.24 \times 12}{20 \times .866 \times 3} = .286$$ sq in STEEL REQ'd

$$= \frac{1.08 \times 12}{20 \times .866 \times 3} = .249$$ " " " "

$$\frac{1.73 \times 12}{20 \times .866 \times 3} = .4$$ " " " "

$$\frac{1.58 \times 12}{20 \times .866 \times 3} = .365$$ " " " "

$$\frac{.62 \times 12}{20 \times .866 \times 3} = .143$$ " " " "

A_s MIN & TEMP STEEL $= .0025\,bd = .0025 \times 12 \times 3 = .09$
¼ φ 7" oc.

Shear at SUPPORT ENDS

$$\frac{wl}{2} = \frac{.077 \times 15}{2} = .578 \text{ K}$$

Shear at 1st INT. SUPPORT

$.578 \times 1.15 = .665$ K.

$$s_o = \frac{V}{u_j d} = \frac{.578}{150 \times .866 \times 3} = 1.48 \text{ \#/IN}$$

$$\frac{.665}{150 \times .866 \times 3} = 1.70 \text{ \#/IN}$$

$$u = \frac{V}{b_j d} = \frac{.665}{12 \times .866 \times 3} = 21.4 \text{ \#/sq. in} \quad \text{O.K. 90 Allowable.}$$

		3/4 @ "	3/8 φ @ "	3/8 φ @ "	
3/8 φ - 4 1/2" o.c.	3/8 φ - 5 1/4" o.c.	3/8 φ - 5 1/2" o.c.	3/8 φ 5 1/4" o.c.	3/8 φ 5 1/4" o.c.	3/8 φ 4 1/2" o.c.

USE 4" S.S. USE. ←

Slope on roof LESS THAN 2"/FT ∴ WIND Could NOT CONTROL. ∴ NOT checked.

DESIGN OF 3 HINGED CONCRETE FRAME.
SPAN 96' - SPACING OF FRAMES

WEIGHTS DEAD.

ROOFING	METAL + 1 LAYER FELT	2 # / sq. ft.
Slab		48 # / sq. ft
Sound board		2 # / sq. ft.
		52 # / sq. ft.

52 x 15' BAYS = 780 # / LN. FT.
* wt. of FRAME 3 x 2 x 144 = 864 # / LN FT.
1.644 K / LN FT.

* IT WAS ASSUMED THAT THE FRAME would be 24" wide & VARY IN DEPTH FROM 24" TO 48" MAKING 3' A ROUGH AVERAGE BUT SAFE.

VERTICAL REACTION DEAD

BEAM PORTION 1.644 x 46 = 75.6
wt. OF COLOMN 3 x 2 x 25' x 144 = 21.6
97.2 K

HORIZONTAL REACTION

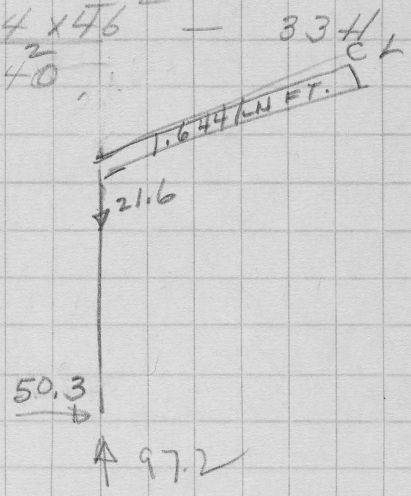
$\Sigma M_c = 75.6(45) - \frac{1.644 \times 46^2}{2} - 33 H_L$

$33 H_L = 3400 - 1740$

$H_L = \frac{1660}{33}$

$H_L = 50.3$

$H_L = H_R$



COLUMN PORTION (DEAD ONLY)

POSITION	THRUST	MOMENT	SHEAR
BASE	- 93.3	0	- 50.3 K
5' from BASE	- 89.0	+ 251.5	- 50.3 K
10' " "	- 84.7	+ 503.0	- 50.3 K
15' " "	- 80.4	+ 754.5	- 50.3 K
20' " "	- 76.1	+ 1006.0	- 50.3 K

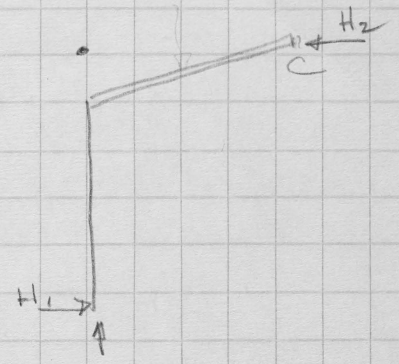
BEAM PORTION (DEAD ONLY)

POSITION	THRUST	MOMENT	SHEAR
	+ 6.84		
40 FROM ℄	+ 8.27	- 901.2	+ 64.8
35 " "	+ 9.71	- 641	+ 56.7
30 " "	+ 11.14	- 505	+ 48.6
25 " "	+ 12.58	- 259	+ 40.5
20 " "	+ 14.01	- 122	+ 32.4
15 " "	+ 15.45	- 35	+ 24.3
10 " "	+ 16.88	+ 18	+ 16.2
5 " "	+ 18.32	+ 25	+ 8.1
℄	+ 19.75	0	0

LIVE LOAD - 3 HINGED CONCRETE FRAME.

L = 46'

ALLOWED - 25 #/sq. ft.



VERTICAL REACTION

25 x 15 x 46 = 17.25 K.

HORIZONTAL REACTION

ΣM_o ABOUT "C"

$$\Sigma M_C = \frac{wl^2}{2} - 17.25(45) + 33H_1$$

$$33H_1 = -\frac{.375 \times 46^2}{2} + 17.25(45)$$

$$= -396.75 + 776.25 = 379.51$$

$$H_1 = 11.5$$

$$H_1 = H_2$$

COLUMN PORTION LIVE LOAD ONLY

POSITION	THRUST	MOMENT	SHEAR
BASE	- 17.25 K	0	- 11.5 K
5' FROM BASE	- 17.25 K	+ 57.5	- 11.5 K
10' " "	- 17.25 K	+ 115.0	- 11.5 K
15' " "	- 17.25 K	+ 172.5	- 11.5 K
20' " "	- 17.25 K	+ 230.0	- 11.5 K
25' " "	- 17.25 K.	+ 287.5	- 11.5 K

BEAM PORTION LIVE LOAD ONLY

POSITION	THRUST	MOMENT	SHEAR
40' " "	+ 11.61	- 205.0	+ 14.80
35' " "	+ 11.28	- 144.6	+ 12.95
30' " "	+ 10.96	- 92.0	+ 11.10
25' " "	+ 10.63	- 56.6	+ 9.25
20' " "	+ 10.31	- 24.8	+ 7.40
15' " "	+ 9.98	- 4.0	+ 5.55
10' " "	+ 9.65	+ 9.1	+ 3.700
5' " "	+ 9.33	+ 11.1	+ 1.850
2' " "	+ 9.00	0	0

WIND LOAD check on 3 HINGED CONCRETE FRAME.

NAT'L BOARD OF FIRE UNDERWRITERS Allows 15# / sq ft. for Bldg's up to 40' High.

$\therefore P = 15\#/\text{sq ft} \times 15' = .225\#$ / LN FT OF VERTICAL SURFACE

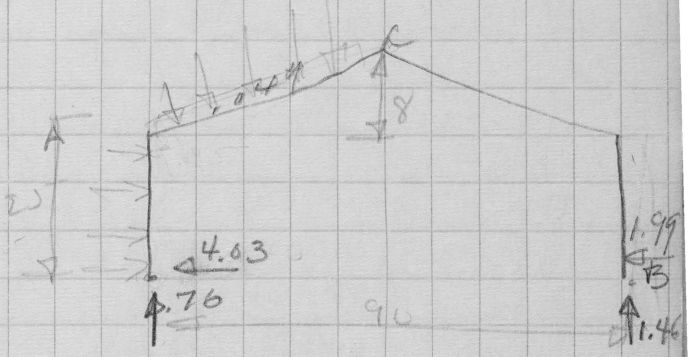
WIND ON ROOF

KETCHUM' FORMULAE

$P_N = P \times \frac{\theta}{45}$

$P_N = .225 \times \frac{90-52}{45}$ 10-6'

$P_N = .049$ should be .0505



FINDING REACTIONS
(whole FRAME)

$\Sigma M_A = \frac{.225 \times 25^2}{2} + .049 \times 46 \times 27.3 - 90 R_{VL}$

$90 R_{VL} = 70.31 + 61.53$

$90 R_{VL} = 131.84$

$R_{VL} = 1.46$

(whole FRAME.

$\Sigma M_B = \frac{.225 \times 25^2}{2} - .049 \times 46 \times 61.4 + 90 R_{VR}$

$90 R_{VR} = -70.31 + 138.4$

$90 R_{VR} = 68.09$

$R_{VR} = .76$

(Half FRAME)

$\Sigma M_C = .76 \times 45 - .225 \times 25 \times 20.5 - .049 \times 46 \times 23$

$+ 33 R$

$33 R = 34.20 - 115.31 - 51.54$

$33 R = 132.95$

$R_{HL} = 4.03$

(whole frame)

$$\Sigma M_c = .76 \times 45 + 4.03(33) - .225 \times 25 \times 20.5 - .049 \times 46 \times 23 - 1.46(45) + 33\, R_{H_R}$$

$$33\, R_{H_R} = -34.20 - 132.95 + 115.31 + 51.54 + 65.70$$

$$R_{H_R} = 1.99$$

Column Portion Lt. (wind load Lt.)

Position	Thrust	Moment	Shear
Base	-.76	0	+4.03
5' from base	-.76	-17.34	+2.90
10' " "	-.76	-29.05	+1.78
15' " "	-.76	-35.2	+.65
17.91 " "	-.76	-36.10	0
20 " "	-.76	-35.60	-.47
25 " "	-.76	-30.80	-1.60

Column Portion Rt. (wind load Lt.)

Position	Thrust	Moment	Shear
Base	-1.46	0	+1.99
5' from base	-1.46	-9.95	+1.99
10' " "	-1.46	-19.90	+1.99
15' " "	-1.46	-29.85	+1.99
20' " "	-1.46	-39.80	+1.99
25' " "	-1.46	-49.75	+1.99

BEAM PORTION LT. (WIND LEFT.)

POSITION	THRUST	MOMENT	SHEAR
43' FROM ℄	+1.61	-31.56	+.63
40' " "	+1.64	-32.38	+.49
35' " "	+1.68	-32.90 (slight	+.24
30' " "	+1.72	-32.0 INACCURACY	0
25 " "	+1.76	-31.89 EXIST HERE)	-.25
20 " "	+1.80	-30	-.49
15 " "	+1.84	-26.4	-.74
10 " "	+1.88	-21.6	-.99
5 " "	+1.92	-15.7	-1.23
℄ "	+1.96	-8.7	-1.48 ℄
		0	

BEAM PORTION RT (WIND LEFT)

POSITION	THRUST	MOMENT	SHEAR
43' FROM ℄	+1.96	-46.4	+1.44
40 " "	+1.96	-44.5	+1.44
35 " "	+1.96	-38.9	+1.44
30 " "	+1.96	-33.2	+1.44
25 " "	+1.96	-27.8	+1.44
20 " "	+1.96	-22.2	+1.44
15 " "	+1.96	-16.7	+1.44
10 " "	+1.96	-11.3	+1.44
5 " "	+1.96	-5.4	+1.44
℄	+1.96	0	+1.44

NOTE: THE WIND STRESSES ARE SO SMALL IN COMPARISON TO DEAD & LIVE STRESSES - THEY CAN NOT POSSIBLY CONTROL & WILL NOT BE checked further.

ACTUAL DESIGN OF 3 HINGED FRAME

Check on Column Portion

Column 20' from base

$f_s = 20{,}000 \quad n = 10$

$f_c = 1350$

$d = 48' \quad d' = 2$

$d'' = 23$

$b = 24$

$M = 1006 + 230.0 = 1236$ 'K

$N = 76.1 + 17.25 = 93.35$ K

$e = \frac{12M}{N} + d'' = \frac{12(1236)}{93.35} + 23 = 159 + 23 = 182$ IN

$E = \frac{e}{12} = \frac{182}{12} = 15.2$ FT.

TABLE 1 (RCDH) $\quad K = 236$

TABLE 4 " $\quad b \times d = 24 \times 48 \quad F = 4.61$

$NE = 93.35 \times 15.2 = 1420$

$KF = 236 \times 4.61 \quad 1090$

330 COMP REINF REQ'D.

TABLE 7 $20{,}000 / 10 / 1350$ & $\frac{d'}{d} = .0417 \quad C = .89$

$A'_s = \frac{NE - KF}{CD} = \frac{330}{.89 \times 48} = 7.73$ SQ. IN REQ'D

5 - 1 1/4 ⟸

TABLE 10 $\frac{e}{d} = \frac{178.5}{48} = 3.72 \quad c = 1.30 \quad a = 1.44$

$A_s = \frac{NE}{adc} = \frac{1398}{1.44 \times 48 \times 1.30} = 15.5$ 10 - 1 1/4 ⟸

2 layers

CHECK ON COLUMN PORTION

COLUMN 15' FROM BASE

$f_s = 20{,}000 \quad n = 10$
$f_c = 1350$
$d = 42' - d' = 2$
$d'' = 20$
$b = 24$
$M = 754.5 + 172.5 = 927$
$N = 80.4 + 17.25 = 97.65$

$e = \frac{12M}{N} + d'' = \frac{12(927)}{97.65} + 20 = 114 + 20 = 134$
$E = \frac{e}{12} = \frac{134}{12} = 11.2$

Table 1 (RCDH) $\quad K = 236$
Table 4 " $\quad b \times d = 24 \times 42 \quad F = 3.53$

$NE = 97.65 \times 11.2 = 1090$
$KF = 236 \times 3.53 = 833$
$NE - KF \quad 257$ COMP REINF REQ'D.

Table 7 $\quad 20{,}000 / 10 / 1350 \quad \& \quad \frac{d'}{d} = .048 \quad c = .85$

$A'_s = \frac{NE - KF}{cD} = \frac{257}{.85 \times 42} = 7.2 \square''$ REQ'D
5 - 1 1/4 □

Table 10 $\quad \frac{e}{d} = \frac{134}{42} = 3.19 \quad i = 1.41 \quad a = 1.44$

$A_s = \frac{1090}{1.44 \times 42 \times 1.41} = 12.8 \square''$ REQ'D
6 - 1 1/4 □

check on Column Portion

Column 10' from Base.

$f_s = 20,000$ $n = 10$
$f_c = 1350$
$d = 36$ $d' = 2$
$d'' = 17$
$b = 24$
$M = 503.0 + 115.0 = 618$
$N = 84.7 + 17.25 = 101.95$

$e = \frac{12M}{N} + d'' = \frac{12(618)}{101.95} + 17 = 73 + 17 = 90$ in.
$E = \frac{e}{12} = \frac{90}{12} = 7.5$ ft.

Table 1 (R'c OH) $K = 236$
Table 4 " $b \times d = 24 \times 36$ $F = 2.59$

$NE = 101.95 \times 7.5 = 762$
$KF = 236 \times 2.59 = 612$
$NE - KF$ 150 comp. reinf. req'd.

Table 7 $20,000/10/1350$ & $\frac{d'}{d} = .0555$ $c = .84$

$A'_s = \frac{NE - KF}{cD} = \frac{150}{.89 \times 36} = 4.68$ □" 3 - 1 1/4 □ ⟸

Table 10 $\frac{e}{d} = \frac{90}{36} = 2.5$ $i = 1.53$ $a = 1.44$

$A_s = \frac{NE}{adi} = \frac{762}{1.44 \times 36 \times 1.53} = 9.54$ □" 2 - 1 1/4 □ ⟸

CHECK ON COLUMN PORTION

COLUMN 5' FROM BASE

$f_s = 20,000 \quad n = 10$
$f_c = 1350$
$d = 30 \quad d' = 2$
$d'' = 14$
$b = 24$
$M = 251.5 + 57.5 = 309'K$
$N = 89.0 + 17.25 = 106.25\ K$

$e = \frac{12M}{N} + d'' = \frac{12(309)}{106.25} + 14 = 34.9 + 14 = 48.9$ IN.
$E = \frac{e}{12} = \frac{48.9}{12} = 4.07$ FT.

TABLE 1 (RCDH) $K = 236$
TABLE 4 (RCDH) $b \times d = 24 \times 30$ $F = 1.80$

$NE = 106.25 \times 4.07 = 433$
$KF = 236 \times 1.80 = 425$
8 COMP STEEL REQ'D

TABLE 7 $20,000 / 10 / 1350$ & $\frac{d'}{d} = .067$ $c = .80$

$A'_s = \frac{NE - KF}{cd} = \frac{8}{.80 \times 30} = .33$ □" REQ'D 1 - 3/4 ϕ

TABLE 10 $\frac{e}{d} = \frac{48.9}{30} = 1.63$ $i = 2.15$ $a = 1.44$

$A_s = \frac{NE}{a\,d\,i} = \frac{433}{1.44 \times 30 \times 2.15} = 4.68$" □ REQ'D.
3 - 1 1/4 □

BASE OF COLUMN

d = 24 ; b = 24

MOMENT = 0

THRUST = 93.3 + 17.25 = 110.55

$P = A_g (.225 f'_c + f_s p_g)$

$93.3 = 624(.225(3000) + 20{,}000 p_g)$

$93.3 = 421{,}200 + 11{,}520{,}000 p_g$

$P_g = \frac{93{,}300 - 421{,}200}{11{,}520{,}000}$

NO STEEL REQ'D. BY ABOVE OPERATION

USE MINIMUM .0025 x bd = 1.56

USE 2 - 1" φ

Check shear, ON ENTIRE COLUMN BASE IS USED AS AREA SMALLEST & shear IS UNIFORM THROUGH OUT COLUMN. SIZE 24 x 26 ACT. SIZE

V = 50.3 + 11.5 = 61.8

$v = \frac{V}{bjd} = \frac{61.8}{24 \times .866 \times 24} = 123.5$ DESIGN STIRRUPS

STIRRUPS NEEDED TO DEPTH = 33.1 − 24 = 9.1 = 28.3 = d

DESIGN OF STIRRUPS

123.5
90
33.5

shear 93.3

$V' = 93.3 \times \frac{33.5}{123.5} = 25.3$

$\frac{6}{s} s = 4.3$

$s = 4.3 \times \frac{s}{6}$

$s = 3.58'$

Redo ?

$$A_v = \frac{V's}{2 f_s jd} = \frac{25.3 \times 3.58 \times 2}{2 \times 20 \times .866 \times 24} = 1.31$$

$$\frac{1.31}{.22} = 6$$

USE 6 - 3/8 ϕ STIRRUPS STARTING BASE OF COLUMN.

FIRST STIRRUP = .143' FROM BASE.
3 - .358'
1 - .537
1 - .787

DESIGN OF BEAM PORTION

40' FROM ℄

$f_s = 20{,}000 \quad n = 10$
$f_c = 1350$
$d = 48 \quad d' = 2$
$d'' = 23$
$b = 24$
$M = 901.2 + 205.0 = 1106.2$
$N = 8.27 + 11.61 = 19.88$

$e = \frac{12M}{N} + d'' = \frac{12(1106.2)}{19.88} + 23 = 668 + 23 = 691$ IN.
$E = \frac{e}{12} = \frac{691}{12} = 57.6$ FT.

TABLE 1 (RCDH) $K = 236$
TABLE 4 " $b \times d = 24 \times 48$ $F = 4.61$

NE 19.88×57.6 1142
KF 236×4.61 1090
52 COMP REINF REQ'D.

TABLE 7 20,000 | 10 | 1350 & $\frac{d'}{d} = .0417$ $c = .89$

$A'_s = \frac{NE - KF}{cD} = \frac{52}{.89 \times 48} = 1.22 \square''$ 1 - 1 1/8 □ ⟸

TABLE 10 $\frac{e}{d} = \frac{691}{48} = 14.4$ $i = 1.06$ $a = 1.44$

$A_s = \frac{NE}{adi} = \frac{1141}{1.44 \times 48 \times 1.06} = 15.6$ 10 - 1 1/4 □ ⟸
2 LAYERS

check shear

$V = 64.8 + 14.8 = 79.6$ K.

$v = \frac{V}{bjd} = \frac{79.6}{48 \times .866 \times 24} = 68$ O.K. NO STIRRUP REQ'D.

BOND.
$\Sigma_0 = \frac{V}{u j d} = \frac{79.6}{150 \times .866 \times 48} = 12.8$ O.K.

BEAM DESIGN.

35' FROM ℄.

$f_s = 20,000$ $n = 10$
$f_c = 1350$
$d = 45$ $d' = 2$
$d'' = 21.5$
$b = 24$
$M = 641 + 144.6 = 785.6$
$N = 9.71 + 11.28 = 20.99$

$e = \frac{12M}{N} + d'' = \frac{12(785.6)}{20.99} + 21.5 = 469 + 21.5 = 490.5$ IN.
$E = \frac{490.5}{12} = 40.9$ FT.

TABLE 1 (RCDH) $K = 236$
TABLE 4 " $b \times d = 24 \times 45$ F 4.06

$NE = 20.99 \times 40.9 = 856$
$KF = 236 \times 4.06 = 960$
NO COMP. STEEL

TABLE 10 $\frac{e}{d} = \frac{490.5}{45} = 10.9$ $i = 1.09$ $a = 1.44$

$A_s = \frac{NE}{adi} = \frac{856}{1.44 \times 45 \times 1.09} = 12.1$ 8-1 1/4 ⊡
TWO LAYERS.

Check on shear.
$V = 56.7 + 12.95 = 69.65$

$U = \frac{V}{bjd} = \frac{69.65}{24 \times .866 \times 45} = 74.5$ O.K. NO STIRRUPS

$T_{BOND} = \frac{V}{\Sigma ojd} = \frac{69.65}{15.0 \times .866 \times 45} = 11.9$ O.K.

DESIGN OF BEAM.

30' FROM ℄.

$f_s = 20{,}000 \quad n = 10$
$f_c = 1350$
$d = 42 \quad d' = 2$
$d'' \quad 20$
$b = 24$
$M = 505 + 92.0 = 597.0$
$N = 11.14 + 10.96 = 22.1$

$e = \frac{12M}{N} + d'' = \frac{12(597)}{22.1} + 20 = 325 + 20 = 345$ IN.
$E = \frac{e}{12} = \frac{345}{12}$ 28.7 FT.

TABLE 1 $\quad K = 236$
TABLE A $\quad b \times d = 24 \times 42 \quad F = 3.53$

$NE = 22.1 \times 28.7 = 634$
$KF = 236 \times 3.53 = 833$

NO COMP STEEL REQ'D

TABLE 10 $\quad \frac{e}{d} = \frac{345}{42} = 8.22 \quad i = 1.12 \quad a = 1.44$

$A_s = \frac{NE}{adi} = \frac{634}{1.44 \times 42 \times 1.12} = 9.35 \quad 6 - 1\frac{1}{4}$ SQ ⟸

check shear:
$V = 48.6 + 11.10 = 59.7$ K.

$u = \frac{V}{bjd} = \frac{59.7}{24 \times .866 \times 42} = 68.2$ OK. NO STIRRUPS

BOND
$\xi_o = \frac{V}{ujd} = \frac{59.7}{150 \times .866 \times 42} = 10.9$ OK

DESIGN OF BEAM
25 FROM ℄

$f_s = 20{,}000 \quad n = 10$
$f_c = 1350$
$d = 39 \quad d' = 2$
$d'' = 18.5$
$b = 24$
$M = 56.6 + 259 = 315.6$
$N = 10.63 + 12.58 = 23.21$

$e = \frac{12M}{N} + d'' = \frac{12(315.6)}{23.21} + 18.5 = 163 + 18.5 = 181.5$ IN.
$E = \frac{e}{12} = \frac{181.5}{12} = 15.1$ FT.

TABLE 1 $\qquad K = 236$
TABLE A $\quad b \times d = 24 \times 39 \qquad F = 2.80$

$NE = 23.21 \times 15.1 = 350$
$KF = 236 \times 2.80 = \underline{661}$

NO COMP STEEL REQ'D

TABLE 10 $\frac{e}{d} = \frac{181.5}{39} = 4.66 \quad i = 1.24 \qquad a = 1.44$

$A_s = \frac{NE}{adi} = \frac{350}{1.44 \times 39 \times 1.24} \quad 5.02 \quad 4 - 1\frac{1}{8}$ ⊡

<u>check shear</u>

$V = 40.5 + 9.25 = 49.75$

$v = \frac{V}{bjd} = \frac{49.75}{24 \times .866 \times 39} = 61.2$ O.K. NO STIRRUPS REQ'D

<u>BOND</u>

$\Sigma_o = \frac{V}{ujd} = \frac{49.75}{150 \times .866 \times 39} = 9.8$ O.K.

DESIGN OF BEAM
20' FROM ℄

$f_s = 20{,}000 \quad n = 10$
$f_c = 1350$
$d = 36 \quad d' = 2$
$d'' = 17$
$b = 24$
$M = 122 + 24.8 = 146.8$
$N = 14.01 + 10.31 = 24.32$

$e = \frac{12M}{N} + d'' = \frac{12(146.8)}{24.32} + 17 = 72.5 + 17 = 89.5$ INCH
$E = \frac{e}{12} = \frac{89.5}{12} = 7.46$ FT.

TABLE 1 (RCDH) $\quad K = 236$
TABLE 4 " $\quad b \times d = 24 \times 36 \quad F = 2.97$

NE $\quad 24.32 \times 7.46 = 181$
KF $\quad 236 \times 2.97 = 700$

NO COMP STEEL REQ'D

TABLE 10 $\frac{e}{d} = \frac{89.5}{36} = 2.49 \quad i = 1.53 \quad a = 1.44$

$A_s = \frac{NE}{adi} = \frac{181}{1.44 \times 36 \times 1.53} = 2.29$

2 - 1 1/8 ⊓ ⟸

MIN $= .0025 \times 24 \times 38 = 2.28$ O.K —

Check shear
$V = 32.4 + 7.40 = 39.8$

$v = \frac{V}{bjd} = \frac{39.8}{24 \times .866 \times 36} = 53$. O.K. NO STIRRUPS NEEDED

BOND
$\Sigma_o = \frac{V}{\omega jd} = \frac{39.8}{150 \times .866 \times 36} = 8.5$ O.K.

DESIGN OF BEAM
15' FROM ℄

$f_s = 20{,}000 \qquad n = 10$
$f_c = 1350$
$d = 33 \qquad d' = 2$
$d'' = 15.5$
16.5 $b = 24$
$M = 35 + 4.0 = 39.0$
$N = 15.45 + 9.98 = 25.43$

$e = \frac{12M}{N} + d'' = \frac{12(39.0)}{25.43} + 15.5 = 18.4 + 15.5 = 33.9$ IN.
$E = \frac{33.9}{12} = 3.32$

TABLE 1 (RCDH) $\qquad K = 236$
TABLE 4 " $\quad b \times d = 24 \times 33 \qquad F = 2.18$

$NE = 25.43 \times 3.32 = 84.5$
$KF = 236 \times 2.18 = \underline{514.0}$

NO COMP. STEEL REQ'D

TABLE 10 $\frac{e}{d} = \frac{33.9}{33} = 1.03 \qquad j = 5$ (ESTIMATED) $\qquad a = 1.44$

$A_s = \frac{NE}{adj} = \frac{84.5}{1.44 \times 33 \times 5} = .355$

MIN. STEEL $.0025 \times 24 \times 33 = 2.1$ ⟵ USE 3 - 1" ϕ

<u>check shear</u>
$V = 5.55 + 24.3 = 29.85$

$v = \frac{29.85}{24 \times .866 \times 33} = 43.5$ O.K. NO STIRRUPS REQ'D.

<u>BOND</u>
$\Sigma_0 = \frac{V}{\Sigma o j d} = \frac{29.85}{150 \times .866 \times 33} = 6.97$ O.K.

DESIGN OF BEAM
10' FROM ℄

$f_s = 20{,}000 \quad n = 10$
$f_c = 1350$
$d = 30 \quad d' = 2$
$d'' = 14$
$b = 24$
$M = 18 + 9 = 27$
$N = 16.88 + 9.65 = 26.53$

$e = \frac{12M}{N} + d'' = \frac{12(27)}{26.53} + 14 = 12.2 + 14 = 26.2$ IN.
$E = \frac{26.2}{12} = 2.18$ FT

TABLE 1 (RCDH) $K = 236$
TABLE 4 " $b \times d = 24 \times 30$ $F = 1.80$

$NE = 26.53 \times 2.18 = 57.7$
$KF = 236 \times 1.80 = \underline{425}$

NO COMP STEEL REQ'D

TABLE 10 $\frac{e}{d} = \frac{26.2}{30} = .873$ $i = 5$ (est.) $a = 1.44$.

$A_s = \frac{NE}{adi} = \frac{57.7}{1.44 \times 30 \times 5} = .268$

MIN STEEL $= .0025 \times 24 \times 32 = 1.92 \Leftarrow$ USE 2 - 1" ⌀.

check shear
$V = 16.2 + 3.70 = 19.9$

$v = \frac{V}{bjd} = \frac{19.9}{24 \times .866 \times 30} = 32.4$ O.K. NO STIRRUPS REQ'D

BOND
$\Sigma_o = \frac{V}{u j d} = \frac{19.9}{150 \times .866 \times 30} = 5.1$ O.K.

DESIGN OF BEAM
5' FROM ℄

$f_s = 20{,}000 \quad n = 10$
$f_c = 1350$
$d = 27 \quad d' = 2$
$d'' = 12.5$
$b = 24$
$M = 11 + 25 = 36$
$N = 9.33 + 19.75 = 29.08$

$e = \frac{12M}{N} + d'' = \frac{12(36)}{29.08} + 12.5 = 14.5 + 12.5 = 27.0$ IN.
$E = \frac{27.0}{12} = 2.25$ FT.

TABLE 1 (RCDH) $K = 236$
TABLE 4 " $b \times d = 24 \times 27$ $E = 1.46$

$NE = 29.08 \times 2.25 = 65.4$
$KF = 236 \times 1.46 = \underline{345.0}$

NO COMP. STEEL NEEDED

TABLE 10 $\frac{e}{d} = \frac{27}{27} = 1$ $i = 5$ (EST.) $a = 1.44$

$A_s = \frac{NE}{adi} = \frac{65.5}{1.44 \times 27 \times 5} = .325$

MIN STEEL $= .0045 \times 24 \times 29 = 1.74$
2 - 1" ⌀.

<u>check</u> <u>shear</u>
$V = 1.85 + 8.1 = 9.95$

$v = \frac{V}{bjd} = \frac{9.95}{24 \times .866 \times 27} = 17.7$ o.k. NO STIRRUPS

<u>BOND</u>

$\Sigma_o = \frac{V}{ujd} = \frac{9.95}{150 \times .866 \times 27} = 2.84$ o.k.

DESIGN OF TBEAM
℄

$f_s = 20{,}000 \quad n = 10$
$f_c = 1350$
$d = 24 \quad d' = 2$
$d'' = 11$
$b = 24$
$M = 0$
$N = 19.75 + 9.00 = 28.75$

$P = A_g\,(.225\,f'_c + f_s\,p_g)$
$28.75 = 624\,(.225\,3000 + 20{,}000\,p_g)$
$28{,}750 = 421{,}200 + 12{,}480{,}000\,P_g$
$P_g = \dfrac{28{,}750 - 421{,}200}{12{,}480{,}000}$

NO STEEL REQ'D BY ABOVE OPERATION

USE MIN STEEL $.0025 \times bd = 1.56$
USE, 2-1" ϕ

No shear.

DESIGN OF FOOTING

40 sq. ft

2.5 K / sq. FT soil pressure ALLOWED.

3 Hinged Concrete

DESIGN OF TIE Rod:-

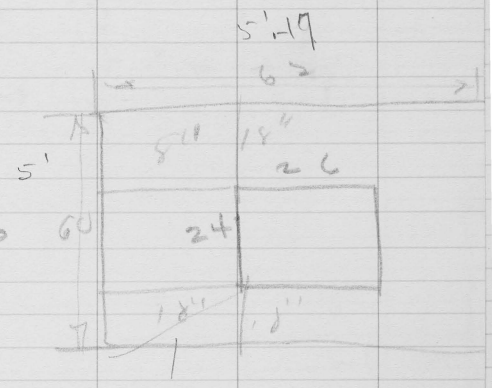
Shear = 11.5 + 50.3 = 61.8

$\frac{61.8}{20} = 3.1$ 2 - 1 1/4 □

DESIGN OF PEDISTAL

24 x 26

USE 4 - 1 1/4 □
TIES 1/4 ϕ - 12" o.c.



DESIGN OF Footing

D + 20% LL 93.3 + 4.3 = 97.6
D + L.L. 93.3 17.3 = 110.6

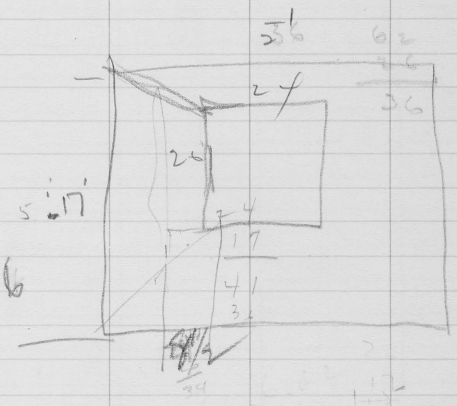
Area D = 97.6 ÷ 3.85 = 25.3
P.2 = 110.6 ÷ 13 = 8.5

Shortway:-

$M = 8.5 \times 5.17 \times \frac{1.5^2}{2} = 49.5$

$V_b = 8.5 \times 5.17 \times 1.5 = 65.8$

$V_v = 8.5 \times \frac{5.17 + 3.58}{2} \times .58 = 21.6$



Longway

$M = 8.5 \times 5.0 \times \frac{1.5^2}{2} = 47.9$

V_b 8.5 x 5.0 x 1.5 = 63.6

V_v 8.5 x $\frac{5.0 + 3.42}{2}$ x .58 = 20.7

M all 17.1 x 5.0 = 85.5 > 49.5

$v = \frac{21.6}{52 \times 7/8 \times 8 1/2} = 56.4 < 60$

$v = \frac{20.7}{50 \times 7/8 \times 8 1/2} = 56.4 < 60$

~~MAKE Footing~~
~~15"~~

short way

$49.5 \times .693 \div 8\frac{1}{2} = 4.05$

$z_0 = 65.8 \div (.168 \times 7/8 \times 8\frac{1}{2}) = 53.$

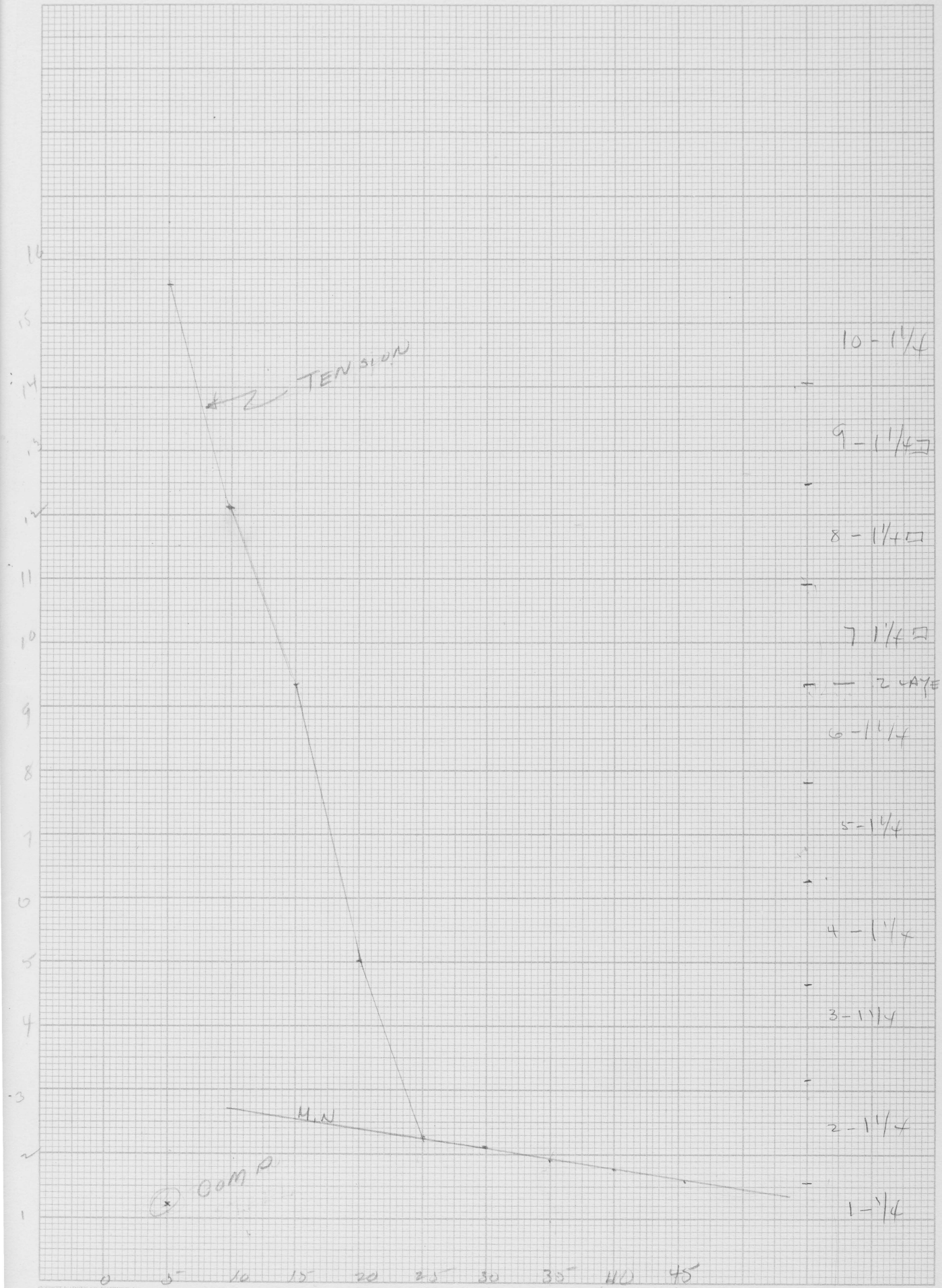
} 24 – 3/4 ϕ BARS

long way

$A_s = 47.9 \times .693 \div 8\frac{1}{2} = 3.93$

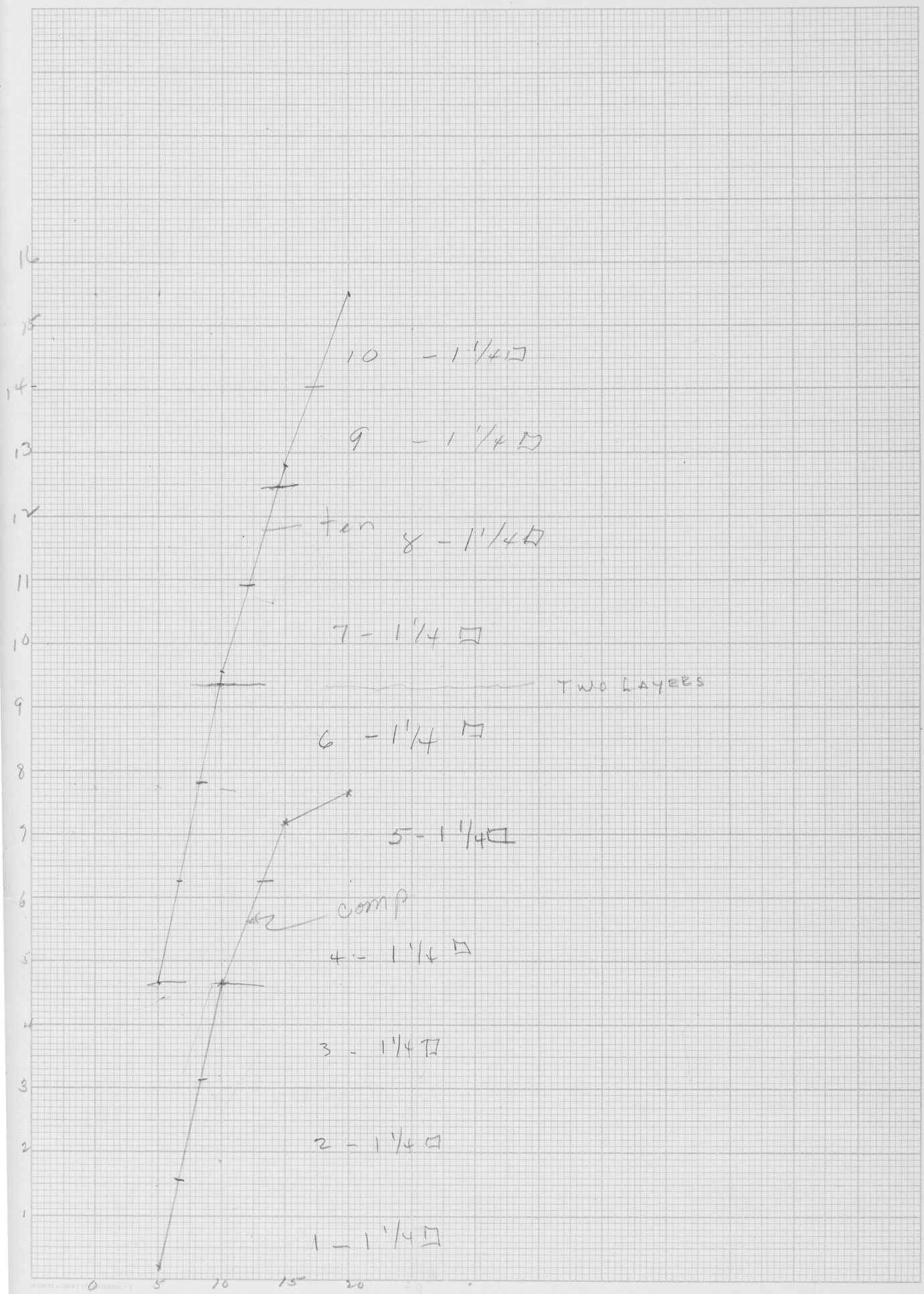
$z_0 = 63.6 \div (.168 \times 7/8 \times 8\frac{1}{2}) = 51.4$

} 24 – 3/4



EVERETT WADDEY CO. RICHMOND, VA.

3 HINGED CONCRETE BEAM PORTION



COLUMN PORTION. 3 HINGED CONCRETE

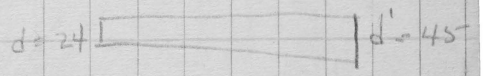
TWO HINGED CONCRETE FRAME

A. ORIGINAL DESIGN
24" UNIFORM THICKNESS
COLUMN DEPTH VARIES 24" TO 34"
BEAM UNIFORM DEPTH OF 36"

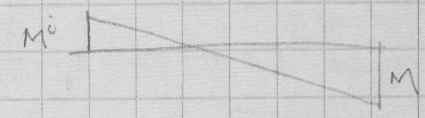
DESIGN OF TWO HINGED FRAME.

CALCULATION OF STIFFNESS FACTORS FOR COLUMN.

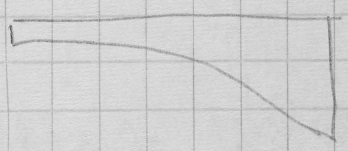
FRAME DIMENSIONS ASSUMED TO BE



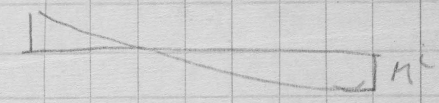
Column 5'-d at base
3.75 = d at top
24" = thickness of b.
L = 25



(1) $$\Delta_{\frac{M}{EI}} = \frac{12}{E}\int_0^{\ell} \frac{\left(M - \frac{(M+M')}{L}x\right)dx}{(d'-ax)^3} = 1$$



$$M_{\frac{M}{EI}} = \frac{12}{E}\int_0^{\ell} \frac{\left(M - \frac{M+M'}{L}x\right)x}{(d'-ax)^3} = 0$$



solving for (1)

$$\frac{12}{E}\int_0^{\ell} \overset{(a)}{\frac{M\,dx}{(d'-ax)^3}} - \overset{(b)}{\frac{M\,x\,dx}{L(d'-ax)^3}} - \overset{(c)}{\frac{M'\,x\,dx}{L(d'-ax)^3}} = 1$$

PART A solving.

$$\frac{12}{E}M\int_0^{\ell}\frac{dx}{(d'-ax)^3} = \frac{12}{E}\left[\frac{M}{2a(d'-ax)^2}\right]_0^{\ell}$$

PART B solving

$$\frac{12M}{EL}\int_0^{\ell}\frac{x\,dx}{(d'-ax)^3} = \frac{12M}{EL}\left[\frac{d'}{2a^2(d'-ax)^2} - \frac{1}{a^2(d'-ax)}\right]_0^{\ell}$$

PART C solving

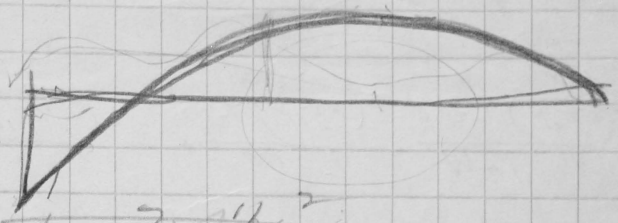
$$\frac{12M'}{EL}\int_0^{\ell}\frac{x\,dx}{(d'-ax)^3} = \frac{12M'}{EL}\left[\frac{d'}{2a^2(d'-ax)^2} - \frac{1}{a^2(d'-ax)}\right]_0^{\ell}$$

COMBINING

$$\frac{12}{E}\left[\frac{M}{2a(d'-ax)^2} - \frac{Md'}{2a^2L(d'-ax)^2} + \frac{M}{a^2L(d'-ax)} - \frac{M'd'}{2a^2L(d'-ax)^2} + \frac{M'}{a^2L(d'-ax)}\right]_0^{\ell} = 1$$

$M_0 = \int_0^e x\,y\,dy$

$M_0 = \frac{x^2}{2}\,y$



$1 - 2x + \frac{1}{2}x^2 = y$

328,05000
314481.37
13,568.63

231,167.37
83,314
314481.37

217,460.7
10,350
3356.64
231,16734

$A = \int 81x - 4.6\,(25 + .179x - \frac{wx^2}{2}$

$y = x^2 + 2x + 3$

$\int = x^2 + 3x + 2$

$\int y\,dy$

328,050,00
83,314
41,364
231,167.37
170,196.63

231,167.37

178
46
1068
712
8188

$y = 115 + .8188x + .895x^2 - 81e$

54
6

18
6
4

18
6

26.727
729,000
240,543,000
534,54
187089
19483.3.54
6

57.5
8100
57500

80,000 1,600,000
160000

20

1,556,640
465,750
2,016,390
240,543
1,775,847

4600
465,750,0

170,196.63
83314.00
86,882.63

6461,0000
224
2584400 00
12922
1,550,640,000
14,470,640.000

AREA OF MOMENT:

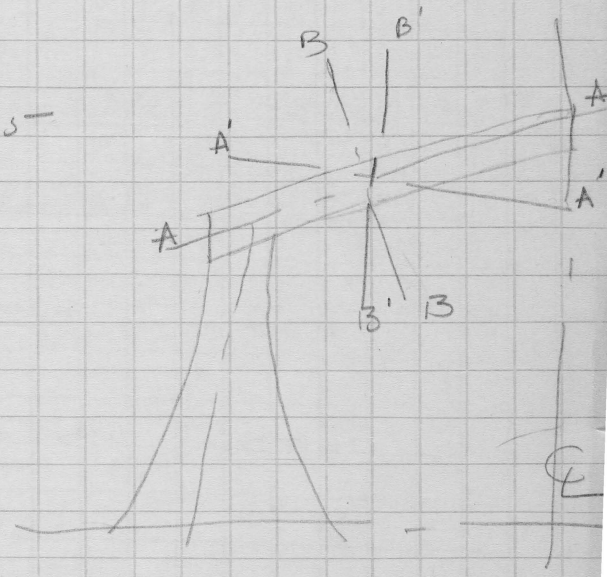
$$P = \int_0^{l} \left(74.5\,l - \frac{1.67\,l^2}{2}\right) dl$$

$$= \left(\frac{74.5\,l^2}{2} - \frac{1.67\,l^3}{6}\right)_0^{90}$$

$$= 301{,}725 - 196{,}830 = 104{,}895$$

CENTROID AT CENTER.

MOMENT OF INERTIA OF ANALY. FRAME.



BEAM

$$I = \frac{bd^3}{12} = \frac{2 \times 3^3}{12} = 4.5$$

ANALY. BEAM

ABOUT AXIS A-A

$$I_x = \frac{bd^3}{12} \; ; \; d = \frac{1}{4.5E}$$

$$I_x = \frac{1}{12} \times \frac{1}{(4.5E)^3} \times \frac{46}{1}$$

$$I_x = \frac{.042066758}{E^3}$$

ABOUT AXIS B-B.

$$I_y = \frac{bd^3}{12}$$

$$I_y = \frac{1}{12} \times \frac{46^3}{4.5E} = \frac{97{,}336}{54E} = \frac{1802.44}{E} \checkmark$$

MOVING AXIS THRU 10°-6" TO A'A'

$$I'_x = I_x \cos^2\theta + I_y \sin^2\theta - 2P_{xy} \sin\theta \cos\theta$$

$$I'_x = \frac{.042066758}{E^3}(.98450)^2 + \frac{1802.44}{E}(.17537)^2 - 0$$

$$I'_x = \frac{.042066758}{E^3}(.96924024) + \frac{1802.44}{E}(.0307546369)$$

$$I'_x = \frac{.040772794619941 92}{E^3} + \frac{55.433387834036}{E}$$

MOVING AXIS THRU 10°-6" TO B'-B'

$$I'_y = I_x \sin^2\theta + I_y \cos^2\theta + 2P_{xy} \sin\theta \cos\theta$$

$$I'_y = \frac{.042066758}{E^3}(.17537)^2 + \frac{1802.44}{E}(.98450)^2 + 0$$

$$I'_y = \frac{.042066758}{E^3}(.0307546369) + \frac{1802.44}{E}(.96924025)$$

$$I'_y = \frac{.00129374787205 6846}{E^3} + \frac{1748.097047 1}{E}$$

MOVING TO AXIS X-X

$$I''_x = I'_x + Ad^2$$

$$I''_y = \frac{.0407729946199419 2}{E^3} + \frac{55.433387834036}{E} + \left(\frac{46}{4.5E}\right)(27)^2$$

$$= \text{TOO SMALL DROPPED} + \frac{55.433387834036}{E} + \frac{33534}{4.5E}$$

$$I''_x = \frac{7505.433387834036}{E} \quad (\tfrac{1}{2}\text{ frame})$$

MOVING TO AXIS Y-Y

$$I''_y = I'_y + Ad^2$$

$$I''_y = \frac{.00129374787205 6846}{E^3} + \frac{1748.897047 1}{E} + \left(\frac{46}{4.5E}\right)(22.5)^2$$

$$I''_y = \text{TOO SMALL DROPPED} \quad \frac{1748.897047 1}{E} + \frac{5157}{E}$$

$$I''_y = \frac{6905.897047}{E} \quad (\tfrac{1}{2}\text{ FRAME})$$

MOMENT OF INERTIA OF COLUMN PORTION

REAL COLUMN.

$$I = \frac{bd^3}{12} \qquad d = (2 + .04y)$$

$$I = \frac{1}{6}(2 + .04y)^3$$

I_{y-y} too SMALL to be bothered with!

$$I_{R-R} \int_0^{22} xy^2\,dy = \int_0^{22} \frac{6}{E(2+.04y)^3}\,y^2\,dy$$

$$I_{R-R} \quad \frac{6}{E}\left(\frac{1}{.04}\right)^3\left(\log_e(2+.88) + \frac{4}{2.88} - \frac{4}{16.5888}\right)$$

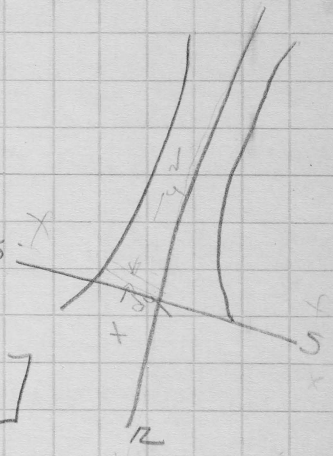
$$- \frac{6}{E}\left(\frac{1}{.04}\right)^3\left(\log_e(2) + \frac{4}{2} - \frac{4}{8}\right]$$

$$\frac{6}{E}\left(\frac{1}{.000064}\right)\left[1.05779 + 1.388 - .24113\right]$$

$$- \frac{6}{E}\left(\frac{1}{.000064}\right)\left[.69315 + 2 - .5\right]$$

$$I_{R-R} \quad \frac{93750}{E}\left[.01151\right]$$

$$I_{RR} = \frac{1079.062.5}{E}$$



90,000
.01
900.00

MOVING AXIS THRU 1'-9"

$I'_{R-R} = I_{R-R} \cos^2\theta + 0$ (note I_{s-s} would have been too small to bother with)

$I'_{R-R} = \frac{1079.0625}{E} (.99980)^2$

$.9 \times .9 = .81$

$I'_{R-R} = \frac{1079.0625}{E} \; .99960000 \; 40$

I_{R-R} or $I_{y-y} = \frac{1078.6309181625}{E}$ (1/2 FRAME

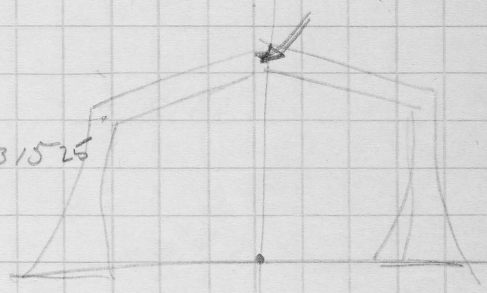
Total $I_{x-x} = 2\left[\frac{7505.43387834036}{E} + \frac{1078.6309181625}{E}\right]$

$I_{x-x} = \frac{17168.2959300572}{E}$

LOAD FRAME with 104,895K Applied AT TOP CENTER OF FRAME

$M_a = \frac{P}{A} + \frac{Mc}{I} + \frac{P}{A} = 0$ AS $A = \infty$

45' FROM END $\frac{Mc}{I} = \frac{\frac{104,895}{4.5E} \times 33 \times 33}{\frac{17168.2959300572}{E}} = 1478.573613$ ℄



END $\frac{Mc}{I} = \frac{\frac{104,895}{4.5E} \times 33 \times 25}{\frac{17168.2959300572}{E}} = 1120.131525$

5' FROM END. $\frac{\frac{104,895}{4.5E} \times 33 \times 25.889}{\frac{17168.2959300572}{E}} = 1159.9634020229$

10' FROM END. $\frac{\frac{104,895}{4.5E} \times 33 \times 26.78}{\frac{17168.2959300572}{E}} = 1199.88488958$

44.80526

15' FROM END $\frac{\frac{104,895}{4.5E} \times 33 \times 27.67}{\frac{17168.2959300572}{E}} = 1239.76157187$

20' FROM END $\frac{\frac{104,895}{4.5} \times 33 \times 28.56}{17168.2959300572} = 1279.6382541 6$

25' $\frac{\frac{104,895}{4.5} \times 33 \times 29.45}{17168.2959300572} = 1319.51493645$

30' $\frac{\frac{104,895}{4.5} \times 33 \times 30.34}{17168.2959300572} = 1359.39161874$

35' $\frac{\frac{104,890}{4.5} \times 33 \times 31.23}{17168.2959300572} = 1399.26830103$

40' $\frac{\frac{104,895}{4.5} \times 33 \times 32.12}{17168.2959300572} = 1439.14498332$

COLUMN

20 $\frac{23310 \times 33 \times 20}{17168.2959300572} = 896.0$

15' $\frac{23310 \times 33 \times 15}{17168.2959300572} = 672.0$

10' $\frac{23310 \times 33 \times 10}{17168.2959300572} = 448.0$

5' $\frac{23310 \times 33 \times 10}{17168.2959300572} = 224.0$

0' = 0

LIVE LOAD

25 #/sq. ft = 25 x 15 = .375 K/LIN FT

VERTICAL REACTION

.375 x 46 = 17.25

MOMENT (NO FIXITY)

@ B = 0.

5' FROM B = 80.38 'K

10' FROM B = 157.08 'K

15' " " = 212.5 'K

20' " " = 265.3 'K

25' " " = 309.0 'K

30' " " 342.2 'K

35' " " 368 'K

40' " " 383 'K

46' " " 396 'K

SYMETRICAL ABOUT ℄.

NO MOMENTS ON COLUMN

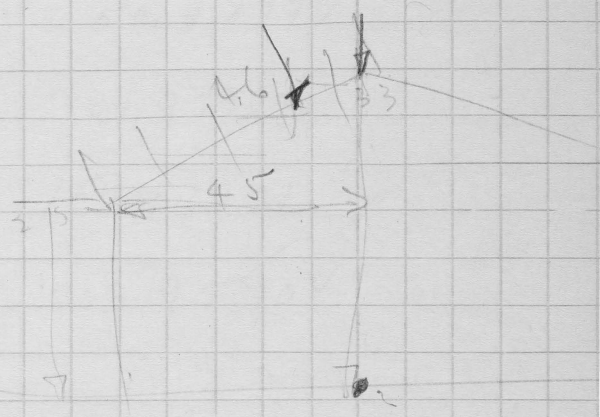
AREA OF MOMENTS

$$P = \int_0^{\ell} \left(16.95\ell - \frac{.3685\ell^2}{2}\right) d\ell$$

$$= \left(\frac{16.95\ell^2}{2} - \frac{.3685\ell^3}{6}\right)\Bigg|_0^{90}$$

$$= 68,647.5 - 44,772.75$$

$$P = 23,874.75$$



$$B = \frac{Mc}{I} = \frac{\frac{23,874.75}{4.5} \times 33 \times 25}{17168.29593005 72} = 254.948850$$

5' $\frac{5305.50 \times 33 \times 25.889}{17168.29593005 72}$ 264.014831106'

10' $\frac{5305.50 \times 33 \times 26.78}{17168.29593005 72}$ 273.10120812

15' $\frac{5305.50 \times 33 \times 27.67}{17168.29593005 72}$ 282.17738718

20' $\frac{5305.50 \times 33 \times 28.56}{17168.29593005 72}$ 291.25356624

25' $\frac{5305.50 \times 33 \times 29.45}{17168.295930057 2}$ 300.3297453

30' $\frac{5305.50 \times 33 \times 30.34}{17168.295930057 2}$ 309.40592436

35' $\frac{5305.50 \times 33 \times 31.23}{17168.295930057 2}$ 318.48210342

40' $\frac{5305.50 \times 33 \times 32.12}{17168.295930057 2}$ 327.55828248

46' $\frac{5305.50 \times 33 \times 33}{17168.295930057 2}$ 336.532482

COLUMN.

BASE = 0

5' = $\frac{5305.50 \times 33 \times 5}{17168.295930057 2}$ = 50.98977

10' = $\frac{5305.50 \times 33 \times 10}{17168.295930057 2}$ = 101.97954

15' = $\frac{5305.50 \times 33 \times 15}{17168.295930057 2}$ = 152.96931

20' = $\frac{5305.50 \times 33 \times 20}{17168.295930057 2}$ = 203.95908

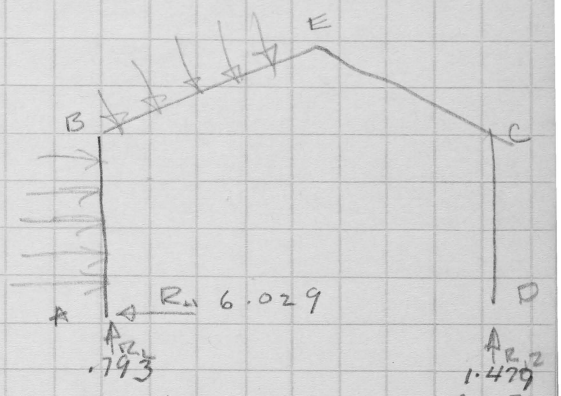
WIND LOAD

15 x 15 = .225 K / LN FT ON VERTICAL SURFACE

WIND ON ROOF

SEE PAGE 6 OF 3 HINGED FRAME

= .0505 K / LN FT.



VERTICAL REACTION

$\Sigma M_A = \frac{.225 \times 25^2}{2}$ + HOR. COMP OF WIND ON ROOF + VERTICAL COMP − $90 R_R$

$90 R_R = 70.3 + .17508 \times .0505 \times 45.7 \times 29 + .98455 \times .0505 \times 45.7 \times 22.5$

$90 R_R = 70.3 + 11.718 + 51.124$

$R_R = 1.479$

$\Sigma M_D = \frac{.225 \times 25^2}{2} + .17508 \times .0505 \times 45.7 \times 29 - .98455 \times .0505 \times 45.7 \times 67.5 - 90 R_L$

$90 R_L = -70.3 - 11.718 + 153.373$

$R_L = .793$

HORIZONTAL REACTION

$\Sigma H = 0$

TOTAL HORIZONTAL = $.225 \times 25 + .17508 \times .0505 \times 45.7$

$= 5.625 + .404$

$= 6.029$

MOMENTS

COLUMN

BASE = 0

5' FROM " = 27.557

10' " " = 49.04

15' " " = 65.122

20' " " = 75.58

25' " " = 80.412

AREA OF MOMENT DIA ON COLUMN

$$P = \int_0^{l} 6.029l - \frac{.225l^2}{2}\, dl$$

$$= \frac{6.029l^2}{2} - \frac{.225l^3}{6}\Big|_0^{25}$$

$$= 1884.063 - 587.936$$

$$= 1296.127$$

$$\bar{x} = \frac{M}{A}$$

$$M = \int_0^{25} 6.029l^2 - \frac{.225l^3}{2}\, dl$$

$$= \frac{6.029l^3}{3} - \frac{.225l^4}{8}\Big|_0^{25}$$

$$= 31401.042 - 10985.770$$

$$= 20415.272$$

$$\bar{x} = \frac{20415.272}{1296.127}$$

$$\bar{x} = 15.75$$

WIND LT.
MOMENTS ON BEAM

LT. END =	80.412
5' FROM " " =	84.046
10	86.408
15	87.527
20	87.373
25	85.967
30	82.279
35	79.338
40	74.133
MIDDLE FROM RTEND =	65.595
40	58.669
35	50.739
30	42.807
25	34.876
20	26.944
15	19.014
10	11.072
5	5.051
RTEND	-0-

AREA OF MOMENT DIA. ON BEAM

$$P = \int_0^{45.7} .793x + 6.029(25 + .178x) - .225(25)(12.5 + .178x) - \frac{.0505x^2}{2}\,dx + \int_{45.7}^{91.4} .793x + 6.029[25 + .178(91.4 - x)]$$

$$- .225(25)(12.5 + .178(91.4 - x)) - .0505(45.7(22.85 + .978(x - 45.7)$$

$$P = \left(\frac{.793x^2}{2} + 150.725x + \frac{1.073162x^2}{2} - 70.3125x - \frac{1.00125x^2}{2} - \frac{.0505x^3}{6}\right)_0^{45.7} + \left(\frac{.793x^2}{2} + 150.725x + 108.0897488x - \frac{1.073192x^2}{2} - 70.3125x - 91.51425x + \frac{1.00125x^2}{2} - 52.2343725x - \frac{2.1670116x^2}{2} + 99.03515155x\right)_{45.7}^{91.4}$$

.500625
.0084166
.500625
1.0835356

$$P = 828.086 + 6888.133 + 1120.644 - 3213.281 - 1045.550$$
$$- 803.370 - 0 + 3312.345 + 13,776.265 + 9879.403$$
$$- 4474.578 - 6426.563 - 8364.402 + 4182.201$$
$$- 4819.922 - 9051.813 + 9051.813 - 828.086$$
$$- 6888.133 - 4939.702 + 1120.644 + 5213.281$$
$$+ 4182.201 - 1045.550 + 2409.961 + 2262.953$$
$$- 4525.907$$

$$P = 5801.120$$

$$x = \frac{M}{A}$$

$$M = \int_{0}^{45.7} .793x^2 + 150.725x + 1.073162x^2 - 70.3125x - 1.00125x^2$$
$$- \frac{.0505}{2}x^3 + \int_{45.7}^{91.4} .793x^2 + 150.725x + 108.0897488x$$
$$- 1.073192x^2 - 70.3125x - 91.51425x + 1.00125x^2$$
$$- 52.73437 25x - 2.16707116x^2 + 99.03515155x \Big)_{45.7}^{91.4}$$

$$M = \frac{.793x^3}{3} + \frac{150.725x^2}{2} + \frac{1.073162x^3}{3} - \frac{70.3125x^2}{2} - \frac{1.00125x^3}{3}$$
$$- \frac{.0505x^4}{8} \Big)_{0}^{45.7} + \Big(\frac{.793x^3}{3} + \frac{150.725x^2}{2} + \frac{108.0897488x^2}{2}$$
$$- \frac{1.073192x^3}{3} - \frac{70.3125x^2}{2} - \frac{91.51425x^2}{2} + \frac{1.00125x^3}{3}$$
$$- \frac{52.73437 25x^2}{2} - \frac{2.16707116x^3}{3} + \frac{99.03515155x^2}{2} \Big)_{45.7}^{91.4}$$

$$M = +25,229.03 + 157,393.83 + 34,142.28 - 73,423.48 - 31,854.43$$
$$- 27,533.80 - 0 + 201,832.23 + 629,575.31 + 451,488.716$$
$$- 273,138.26 - 293,698.90 - 382,253.19 + 254,835.46$$
$$- 220,270.42 - 551,557.09 + 413,667.85 - 25,229.03$$
$$- 157,393.83 - 112,872.18 + 34,142.28 + 73,423.48$$
$$+ 95,563.30 - 31,854.43 + 55,067.60 + 68,944.64$$
$$- 183,416.96$$

$$M = 211,410.$$

36.6

$\bar{X} = \frac{M}{A}$

$\bar{X} = \frac{211,410.01}{5801.12}$

$\bar{X} = 36.6$

$\frac{M}{I}$ dia for column

$$= \frac{12}{2}\int_0^{25} \frac{6.029\ell - .225\ell^2}{(.04\ell+2)^3}\,d\ell = 6\left\{6.029\frac{2}{.0032(.04\ell+2)^2} - \frac{1}{.0016(.04+2)^2} - .1125\left[\frac{1}{.000064}\left(\log(.04\ell+2) + \frac{4}{(.04\ell+2)} - \frac{4}{2(.04\ell+2)^2}\right)\right]\right\}_0^{25}$$

$$= \frac{72.348}{.0288} - \frac{36.174}{.0048} - \frac{.675(0.47712)}{.000064} - \frac{2.7}{3(.000064)} + \frac{2.7}{8(.000064)}$$

$$- \frac{36.174}{.0064} + \frac{36.174}{.0032} + \frac{.675(.30103)}{.000064} + \frac{2.7}{2(.000064)} - \frac{2.7}{8(.000064)}$$

$$= 2512.08 - 7535.41 - 5032.125 - 14062.5 + 2343.75$$
$$- 5652.18 + 11304.37 + 3174.926 + 2109.75 - 5273.445$$

$\frac{M}{I}$ D.A. $= +2873.216$

BEAM $\cancel{\frac{P}{A}}^{0} \pm \frac{MC}{I}$

@ end $= \frac{2873.216}{17168.29593} \times 15.75 \times 25 + \frac{5801.12}{17168.29593} \times 31.51 \times 25 = 125.047$

@ 5' FROM " $= \frac{2873.216 \times 15.75 \times 23.89}{17168.29593} + \frac{1289.1377 \times 31.51 \times 23.89}{17168.29593} = 129.448$

@ 10' " " $\frac{2873.316 \times 15.75 \times 26.78}{17168.29593} + \frac{1289.1377 \times 31.51 \times 26.78}{17168.29593} = 133.951$

@ 15' " " $\frac{2873.316 \times 15.75 \times 27.67}{17168.29593} + \frac{1289.1377 \times 31.51 \times 27.67}{17168.29593} = 138.402$

@ 20' " " $\frac{2873.316 \times 15.75 \times 28.56}{17168.29593} + \frac{1289.1377 \times 31.51 \times 28.56}{17168.29593} = 142.854$

@ 25' $\frac{2873.316 \times 15.75 \times 29.45}{17168.29593} + \frac{1289.1377 \times 31.51 \times 29.45}{17168.29593} = 147.306$

@ 30' $\frac{2873.316 \times 15.75 \times 30.34}{17168.29593} + \frac{1289.1377 \times 31.51\ 30.34}{17168.29593} = 151.757$

@ 35' $\frac{2873.316 \times 15.75 \times 31.23}{17168.29593} + \frac{1289.1377 \times 31.51 \times 31.23}{17168.29593} = 156.209$

@ 40' $\frac{2873.316 \times 15.75 \times 32.12}{17168.29593} + \frac{1289.1377 \times 31.51\ 32.12}{17168.29593} = 160.661$

BEAM CONT.

MIDDLE : $\frac{287.316 \times 15.75 \times 33}{17168.29593} + \frac{1289.1377 \times 31.51 \times 33}{17168.29593} = 165.062$

COLUMN

Distance from BASE.

20' = $\frac{287.316 \times 15.75 \times 20}{17168.29593} + \frac{1289.1377 \times 31.51 \times 20}{17168.29593} = 100.038$

15' = $\frac{287.316 \times 15.75 \times 15}{17168.29593} + \frac{1289.1377 \times 31.51 \times 15}{17168.29593} = 75.028$

10' = $\frac{287.316 \times 15.75 \times 10}{17168.29593} + \frac{1289.1377 \times 31.51 \times 10}{17168.29593} = 50.019$

5' $\frac{287.316 \times 15.75 \times 5}{17168.29593}$ $\frac{1289.1377 \times 31.51 \times 5}{17168.29593} = 25.0094$

0 = $\underline{\underline{0}}$

MOMENT DUE TO DEAD (symetrical about ℄

COLUMN =

BASE = 0
5' " = -224.0
10' " = -448.0
15' " = -672.0
20' " = -896.0
25' " = -1120.131525

BEAM =
END = -1120.131525
5' " = -1159.963402029 + 351.8 = -808.16
10' " = -1199.884889958 + 664.0 = -535.88
15' " = -1239.76157187 + 935.5 = -304.26
20' " = -1279.63825416 + 1166.0 = -113.64
25' " = -1319.51493645 + 1356 = +36.49
30' " = -1359.39161874 + 1501 = +141.61
35' " = 1399.26830102 + 1620 = +220.73
40' " = 1439.14498332 + 1685 = +245.86
45' " = 1478.573613 + 1885 = +406.43

LIVE LOAD (symetrical about ℄)

COLUMN
BASE = 0
5' " = -50.98977
10' " = -101.97954
15' " = 152.96931
20' " = 203.95908
25' " = -254.948850

BEAM
END = -254.948850
5' END = -264.01483 1106 + 80.38 = -183.63
10' " = -273.10120812 + 151.08 = -122.02
15' " = -282.17738718 + 212.5 = -69.68
20' " = -291.25356624 + 265.3 = -25.95
25' " = -300.329745 3 + 309.0 = + 8.67
30' " = -309.405 92436 + 342.2 = + 32.59
35' " = -318.48210 342 + 368. = + 49.52
40' " = -327.55828248 + 383 = + 55.44
45' " = -336.53248 2 + 396 = + 59.47

MOMENTS DUE TO WINDLOAD

LEFT COLUMN.

BASE = 0

5' "	= +27.557	− 25.0094	=	+2.348
10' "	= +49.04	− 50.019	=	− .979
15' "	= 65.122	− 75.028	=	−9.906
20' "	= 75.58	− 100.038	=	−24.458
25' "	= 80.412	− 125.047	=	−44.635

RIGHT COLUMN

BAS = 0

5' "	= 25.0094
10' "	= 50.019
15' "	= 75.028
20' "	= 100.038
25' "	= 125.047

BEAM

LT. END.	= +80.412	−	125.047	=	−44.635	1.785
5' "	= +84.046	−	129.498	=	−45.452	
10'	= +86.408	−	133.951	=	47.543	
15'	= +87.527	−	138.402	=	50.875	
20'	= +87.373	−	142.854	=	55.481	
25'	= +85.967	−	147.306	=	61.339	
30'	= +83.279	−	151.757	=	67.478	
35'	= +79.338	−	156.209	=	76.871	
40'	= +74.133	−	160.661	=	85.528	
℄	= +65.595	−	165.062	=	99.467	
50'	= 58.669	−	160.661	=	−101.992	
55'	= +50.739	−	156.209	=	105.470	
60'	= +42.807	−	151.757	=	108.950	
65'	= +34.876	−	147.306	=	112.430	
70'	= +26.944	−	142.854	=	115.860	
75'	= +19.014	−	138.402	=	119.388	
80'	= +11.072	−	133.951	=	122.879	
85'	= +5.051	−	129.498	=	124.447	
RT. END.	0	−	125.047	=	−125.047	5.06488

SHEAR DUE TO DEAD LOAD

COLUMN

BASE	=	44.805261
5' "	=	44.805261
10' "	=	44.805261
15 "	=	44.805261
20' "	=	44.805261
25' "	=	44.805261

THRUST COLUMN

BASE	93.6
5' "	90.5
10' "	87.2
15 "	83.5
20' "	79.6
25 "	75.6

BEAM

END	75.6
5' "	67.2
10' "	58.8
15' "	50.4
20' "	42.0
25' "	33.6
30' "	25.2
35' "	16.8
40' "	8.4
45' "	0

symmetrical about q.

BEAM

44.805261

↓

Shear due to Live Load

COLUMN.

BASE	=	10.197950
5' "	=	10.197950
10 "	=	10.197950
15 "	=	10.197950
20 "	=	10.197950
25 "	=	10.197950

THRUST

17.25
17.25
17.25
17.25
17.25

BEAM

END	=	17.25
5 "	=	15.334
10 "	=	13.417
15 "	=	11.501
20 "	=	9.584
25 "	=	7.668
30 "	=	5.751
35 "	=	3.834
40 "	=	1.927
45	=	0

10.197950

↓

WIND SHEAR
COLUMN LT.

BASE	= -1.027
5' "	= +.098
10'	= +1.223
15'	= 2.348
20'	= 3.473
25'	= 4.598

THRUST

.793

BEAM

END	= .793	
5'	= .545	
10'	= .296	
15'	= .047	
20'	= -.201	
25'	= -.449	
30'	= -.698	
35'	= -.946	
40'	= -1.195	
℄	= -1.479	→ 90

THRUST

4.435
4.478
4.521
4.564
4.607
4.650
4.693
4.736
4.079
4.825 → 90

RT.
BASE - 5.002

1.479

Note wind
at pt max stress

MAX STRESSES (CAUSED BY DEAD + LIVE)

Lt. COLUMN & RT. COLUMN

	MOMENTS	SHEAR	THRUST
BASE	= 0	55.01	110.85
5' FROM "	= 274.99	55.01	107.75
10' " "	= 549.98	55.01	104.45
15' " "	= 824.97	55.01	100.75
20' " "	= 1099.96	55.01	96.85
25' " "	= 1375.18	55.01	92.85

BEAM

END =	−1375.18	92.85	55.01
5' FROM "	−991.79	82.53	55.01
10' " "	−675.90	72.22	55.01
15'	−373.94	61.90	55.01
20'	−139.59	51.58	55.01
25'	+ 45.16	41.27	55.01
30'	+ 174.20	30.95	55.01
35'	+ 270.25	19.63	55.01
40'	+ 301.30	10.33	55.01
℄	+ 465.90	0	55.01

DESIGN OF 2 HINGED FRAME:

BASE OF COLUMN

2' SQUARE ; d = 22"

M = 0

V = 55.01

THRUST = 110.85

Check FOR THRUST -

$P = A_g (.225 f'_c + f_s p_g)$

$110.85 = 576 (.225(3000) + 20,000 p_g)$

$110,850 = 382,725 + 11,520,000 p_g$

$p_g = \frac{110,850 - 382,725}{11,520,000}$

No steel REQ'D BY ABOVE OPERATION

USE MINIMUM .0025 x bd = 1.32

USE 2 - 1" ⌀

check shear ON ENTIRE COLUMN

BASE IS USED AS AREA IS SMALLEST & Shear IS UNIFORM THROUGHOUT COLUMN.

d = 22 b = 24

V = 55.01

$v = \frac{V}{bjd} = \frac{55.01}{24 \times .866 \times 22} = 120.05$

STIRRUPS NEEDED to D = 29.4

5' FROM BASE OF COLUMN

$d = 24.4 \quad b = 24$

$f_s = 20{,}000 \quad n = 10$

$f_c = 1350$

$d = 24.4 \quad d' = 2$

$d'' = 11.2$

$b = 24$

$M = 274.99$

$N = 107.75$

$e = \frac{12M}{N} + d'' = \frac{12(274.99)}{107.75} + 11.2 = 30.62 + 11.2 = 41.82$ IN

$E = \frac{e}{12} = \frac{41.82}{12} = 3.48'$

TABLE 1 (RCHD $\quad K = 236$

" 4 (RCHD) $\quad b \times d = 24 \times 24.4 \quad K = 1.15$

NE $107.75 \times 3.48 = 375$

KF $236 \times 1.15 = 271$

104 COMP. STEEL REQ'D

TABLE 7 $\quad 20{,}000/10/1350$ & $\frac{d'}{d} = .082 \quad C = .75$

$A'_s = \frac{NE - KF}{cd} = \frac{104}{.75 \times 24.4} = 5.7\square''$ REQ'D $- 4 - 1\frac{1}{4}\square$ USE

TABLE 10 $\quad \frac{e}{d} = \frac{41.82}{24.4} = 1.70 \quad i = 2.04 \quad a = 1.44$

$A_s = \frac{NE}{adi} \quad \frac{375}{1.44 \times 24.4 \times 2.04} = 5.24\square''$ REQ'D $4 - 1\frac{1}{4}\square$ USE

10' FROM BASE OF COLUMN

$d = 26.8 \quad b = 24$

Redesign for 2 layers of comp. steel

$f_s = 20{,}000$

$f_c = 1350$

$d = 26.8 \quad d' = 2$

$d'' = 12.4$

$b = 24$

$M = 549.98$

$N = 107.75$

$e = \frac{12M}{N} + d'' = \frac{12(549.98)}{107.75} + 12.4 = 61.2 + 12.4 = 73.6$ IN

$E = \frac{e}{12} = \frac{73.6}{12} = 6.14'$

TABLE 1 (RCHD) $\quad K = 236$

" 4 (RCHD) $\quad b \times d = 24 \times 26.8 \quad F = 1.46$

$NE = 107.75 \times 6.14 = 661$

$KF = 236 \times 1.46 = 345$

316 comp steel needed.

Table 7 - 20,000/10/1350 & $\frac{d'}{d} = .0746 \quad c = .78$

$A'_s = \frac{NE - KF}{cd} = \frac{316}{.78 \times 26.8} = 15.1 \square''$ REQ'D $\quad$ 10 - 1¼" BARS - 2 LAYERS

Table 10 $\frac{e}{d} = \frac{73.6}{26.8} = 2.75 \quad i = 1.46 \quad a = 1.44$

$A_s = \frac{NE}{adi} = \frac{661}{1.44 \times 26.8 \times 1.46} = 12.25$ $\quad$ 8 - 1¼" BARS 2 LAYERS

10'-1" FROM BASE OF COLUMN

29.2 = d b = 24

f_s = ~~20000~~

f_c = 1350

d = 29.2 d' = 2

d' = 13.7

b = 24

M = 824.97

N = 100.95

little mis -

$e = \frac{12M}{N} + d'' = \frac{12(824.97)}{100.95} + 13.7 = 98.4 + 13.7 = 108.3$ IN

$E = \frac{e}{12} = \frac{108.3}{12} = 9.04$

Table 1 (RCHD K = 236

" 4 (RCHD b × d = 24 × 29.2 F = 1.68

NE = 104.45 × 9.04 = 945

KF = 236 × 1.68 = 397

548 COMP steel REQ'D)

Table 7 - 20,000/10/1350 d'/d = .146 c = .55

$A'_s = \frac{NE - KF}{cd} = \frac{548}{.55 \times 29.2} = 34.1$

3 layers of steel

Table 16 (13) $\frac{e}{d} = \frac{108.3}{29.2} = 3.71$ i = 1.30 a = 1.44

$A'_s = \frac{NE}{cdi} = \frac{945}{1.44 \times 29.2 \times 1.30} = 17.3$ □" REQ'D - 6 - 1 1/4 -

2 LAYERS.

20' FROM BASE

168
139.6

$f_s = 20000$
$f_c = 1350$
$d = 31.6 \quad d = 2$
$d'' = 14.8$
$b = 24$
$M = 1099.96$ 1500
$N = 96.85$ 120

$e = \frac{12M}{N} + d'' = 12\frac{(1099.96)}{96.85} + 14.8 = 136 + 14.8 = 150.8$ IN
$E = \frac{e}{12} = \frac{150.8}{12} = 12.51$

Table 1 (RCHD) $\quad K = 236$
" 4 (RCHD) $b \times d = 24 \times 31.6 \quad F = 1.49$

$NE = 96.85 \times 12.51 = 1210$
$KF = 236 \times 1.49 = 352$
758 COMP steel —

COMP 4 layers —

TENT. 3 layers

IN THE COMPUTER'S MIND THE USE OF 7 LAYERS OF STEEL IS NOT ECONOMICAL USE OF CONCRETE. —

RE DESIGNED

TWO HINGED CONCRETE FRAME

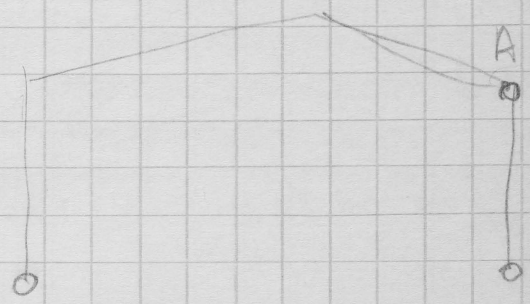
B- SECOND DESIGN:
24" UNIFORM THICKNESS.
COLUMN DEPTH VARIES FROM 37" TO 67"
BEAM " " " 55" TO 67"

ADJUSTMENT MADE:-
BEAM WIDTH 19½"
DEPTH - COLUMN VARIES FROM 29" TO 52"
BEAM " " 43" TO 52"

TWO HINGED FRAME CONCRETE

METHOD COLUMN ANALAGY

① MAKE DETERMINATE.
PLACE HINGE AT A.



② FIND MOMENTS ON FRAME

LOADING - DEAD

ROOFING METAL + 1 LAYER FELT	2# / sq. ft
SLAB	48# / sq ft
Sound Board	2# / sq. ft.
	52# / sq ft.

52 X 15' BAYS 780# / LIN FT.

VERTICAL REACTION

BEAM = 780 X 45.7 = 35.6

2 X 42.2 X 5 X 144 = 60.7

CORNER $\frac{5.5 \times 5.5}{2} \times 2 \times 144$ = 4.35

COLUMN

CORNER = $\frac{5.5 \times 5.5}{2} \times 2 \times 144$ = 4.35

$\frac{3 + 5.5}{2} \times 2 \times 19.5 \times 144$ = 93.84

128.84 K

NO HORIZONTAL REACTION

MOMENTS @ B = 0

" 5' FROM " = 471.0

" 10' = 847.5

" 15' = 1249.5

" 20 = 1557.00

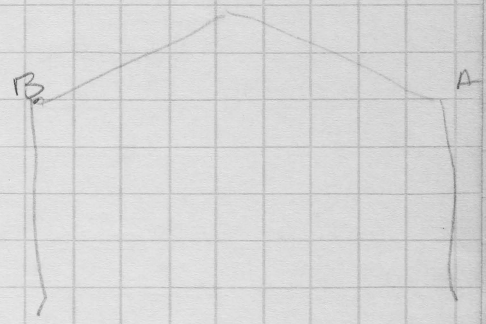
" 25 = 1810.00

" 30 = 2008.50

" 35 = 2152.50

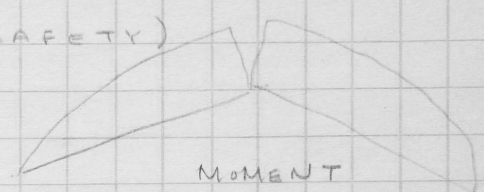
" 40 = 2224.00

" ℄ " " = 2277.54



SYMTRICAL ABOUT ℄ & NO MOMENTS ON COLUMNS

AREA OF MOMENTS (Following EQUATION APPROX but ON SIDE OF SAFETY)



MOMENT AREA

$$P = \int_0^{l} \left(99.65l - \frac{2.18l^2}{2}\right) dl$$

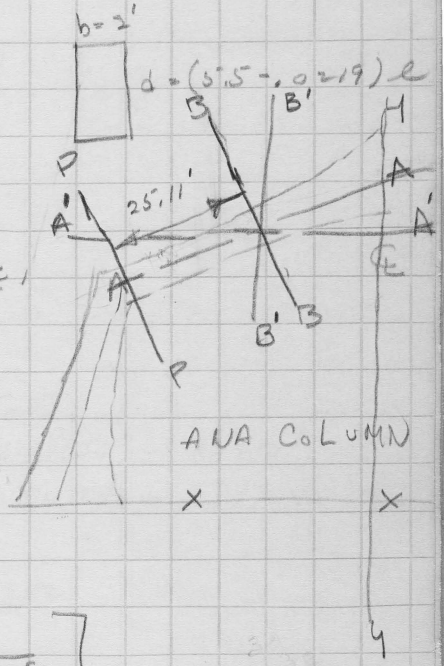
$$P = \left(\frac{99.65l^2}{2} - \frac{2.18l^3}{6}\right)\Big|_0^{91.4}$$

$$P = 416,653.755 - 277,423.869 = 139,229.886$$

CENTROID OF MOMENT AREA AT CENTER

MOMENT OF INERTIA OF ANAL. BEAM

BEAM REAL



$$I = \frac{1}{12} bd^3 \; ; \; d = (5.5 - .0219l$$

$$I = \frac{(5.5 - .0219l)^3}{6}$$

$$d = \frac{bd^3}{12(5.5 - .0219l)^3 E I}$$

ANAL BEAM ABOUT AXIS A-A

$$I_x = \int_0^{45.7} \frac{y^3}{3} dx = \frac{2 \times 27}{3} \int_0^{45.7} \frac{dx}{(5.5 - .0219x)^9 E^3} =$$

$$= -\frac{18}{E^3} \left[\frac{1}{.1032(5.5 - .0219x)^8}\right]_0^{45.7}$$

$$= -\frac{18}{E^3} \left[\frac{1}{.1032\,(4.5)^8} - \frac{1}{.1032\,(5.5)^8}\right]$$

$$= -\frac{18}{E^3} \left[\frac{1}{17,377.013112} - \frac{1}{86,413.42416}\right]$$

$$= -\frac{18}{E^3} \left[.000057553723 - .000001157227\right]$$

$$= -\frac{18}{E^3} \left[.000045984496\right]$$

$$= \frac{.00082772928}{E^3}$$

FINDING CENTROID OF ANAL COLUMN (1/2)

$$\bar{x} = \frac{M}{A}$$

AREA OF ANAL COLUMN (1/2)

$$A = \frac{1}{E}\int_0^{45.7} \frac{6}{(5.5-.0219x)}\,dx = \frac{6}{E}\left(-\frac{1}{2(.0219)(5.5-.0219x)^2}\right)_0^{45.7}$$

$$= \frac{6}{E}\left[\frac{1}{.88695} - \frac{1}{1.32495}\right] = \frac{6}{E}\left[1.127459 - .754745\right]$$

$$A = \frac{6}{E}\left[.372714\right] = \frac{2.236284}{E}$$

MOMENT ABOUT LEFT END

$$M = \int_0^{45.7} xy\,dx = \frac{6}{E}\int_0^{45.7} \frac{x\,dx}{(5.5-.0219x)^3} =$$

$$= \frac{6}{E}\left[\frac{5.5}{2(.0219)^2(45.5)^2} - \frac{1}{(.0219)^2(45.5)} - \frac{5.5}{2(.0219)^2(55.5)^2} + \frac{1}{(.0219)^2(55.5)}\right]$$

$$= \frac{6}{E}\left[283.15187 - 463.3394 - 189.5479 + 379.09585\right]$$

$$= \frac{6}{E}\left[9.36037\right] = \frac{56.16222}{E}$$

CENTROID

$$\bar{x} = \frac{M}{A}$$

$$= \frac{56.16222}{E} \times \frac{E}{2.236284}$$

$$\bar{x} = 25.114093$$

I_{y-y} ABOUT P-P

$$I_{yp} = \int_0^{45.7} x^2 y\,dx = \frac{6}{E}\int_0^{45.7} \frac{x^2\,dx}{(55.5-.0219)^3}$$

$$= \frac{6}{E}\,\frac{1}{(.0219)^3}\left[\log 4.5 + \frac{2(5.5)}{4.5} - \frac{(5.5)^2}{2(4.5)^2} - \log 5.5 - \frac{2(5.5)}{5.5} + \frac{(5.5)^2}{2(5.5)^2}\right]$$

$$= -\frac{6}{E\,.00001050345^9}\left[1.50408 + 2.44444 - .7469135 - 1.70475 - 2 + .5\right]$$

$$= -\frac{6}{.00001050345^9 E}\left[-.0031391\right]$$

$$I_{yp} = \frac{1793.1807}{E}$$

MOVING Y AXIS TO CENTROID AXIS B-B

$$I_y = I_{yp} - Ad^2$$

$$= \frac{1793.1807}{E} - \frac{2.236284}{E}(25.11)^2$$

$$= \frac{1793.1807}{E} - \frac{1410.00412}{E}$$

$$I_y = \frac{383.1766}{E}$$

MOVING AXIS THRU 10°-5' APPROX to A'-A'

$$I'_x = I_x \cos^2\theta + I_y \sin^2\theta - 2P_x \sin\theta \cos\theta$$

$$= \frac{.000827729}{E^3}(.98455)^2 + \frac{383.1766}{E}(.17508)^2 - 0$$

$$= \text{TOO SMALL} + \frac{11.745515}{E}$$

$$I'_x = \frac{11.745515}{E}$$

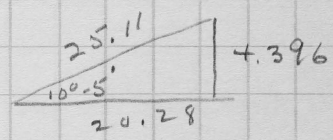
MOVING AXIS THRU 10°-5' TO B'-B'

$$I'_y = I_x \sin^2\theta + I_y \cos^2\theta + 2P_x \sin\theta \cos\theta$$

$$= \frac{.0008277298}{E^3}(.17508)^2 + \frac{383.1766}{E}(.98455)^2 + 0$$

TOO SMALL $+ \frac{371.4279}{E}$

$$I'_y = \frac{371.4279}{E}$$



FIND AMT TO MOVE AXIS

$\sin\theta = \frac{y}{25.11}$

$y = (25.11)(.17508)$

$y = 4.396$

$+25$

29.40

$\cos\theta = \frac{x}{25.11}$

$x = (25.11)(.98455)$

$x = 24.72$

FROM 45.00

20.28

MOVING AXIS TO X-X

$$I''_x = I'_x + Ad^2$$

$$= \frac{11.7455}{E} + \frac{2.236284}{E}(29.4)^2$$

$$= \frac{11.7455}{E} + \frac{1932.961012 9}{E}$$

$$I''_x = \frac{1944.7065}{E}$$

MOVING AXIS TO Y-Y

$$I''_y = I'_y + Ad^2$$

$$= \frac{371.4279}{E} + \frac{2.236284}{E}(20.28)^2$$

$$= \frac{371.4279}{E} + \frac{919.7353}{E}$$

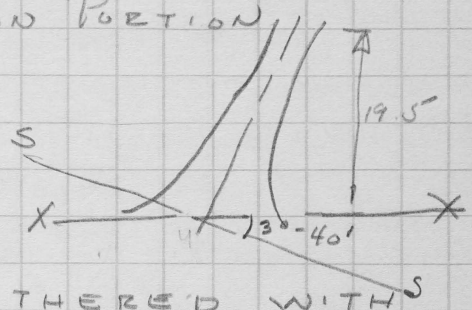
$$I''_y = \frac{1291.1632}{E}$$

X

MOMENT OF INERTIA OF COLUMN PORTION

REAL COLUMN

$I = \frac{1}{12} bd^3 \qquad d = (3 + .12824)$



$I = \frac{2}{12} (3 + .12824)^3$

I_{4-4} TOO SMALL TO BE BOTHERED WITH

$$I_{s-s} = \int_0^{19.5} x\, y^2 dy = \frac{6}{E} \int_0^{19.5} \frac{y^2 dy}{(3 + .12824)^3}$$

$$= \frac{6}{E} \left[\frac{1}{(.1282)^3} \left(\log 5.5 + \frac{2(3)}{5.5} - \frac{9}{2(5.5)^2} - \log 3 - \frac{2(3)}{3} + \frac{9}{2(9)} \right) \right.$$

$$= \frac{6}{.0023642\,E} \left[1.70475 + 1.090909 - .14876 - 1.09861 - 2.0 + .5 \right]$$

$$= \frac{6}{.0023642\,E} \left[.04828 \right]$$

$$I_{s-s} = \frac{122.5277}{E}$$

MOVING AXIS THRU 3°-40'

$$I''_x = I_{s-s} \cos^2\theta + I_y \sin^2\theta - 2P_x \sin\theta \cos\theta$$

$$= \frac{122.5277}{E} (.99795)^2 + \text{too small} - 0$$

$$I''_x = \frac{122.0259}{E}$$

$$\text{TOTAL } I''_x = 2 \left[\frac{1944.7065}{E} + \frac{122.0259}{E} \right]$$

$$= 2 \left[\frac{2066.7324}{E} \right]$$

$$= \frac{4133.4648}{E}$$

$\frac{M}{EI}$ DIAGRAM AREA

$$\frac{M}{EI}_{DIA} = \frac{6}{E}\int_0^{45.7} \frac{99.65\ell - 1.09\ell^2}{(5.5 - .0219\ell)^3}\, d\ell$$

$$= \frac{6}{E}\left(\int \frac{99.65\ell\, d\ell}{5.5 - .0219\ell)^3} - \int \frac{1.09\ell^2\, d\ell}{(5.5 - .0219\ell)^3}\right)$$

$$= \frac{6}{E}\left[99.65\left(\frac{5.5}{2(-.0219)^2(4.5)^2} - \frac{1}{(-.0219)^2(4.5)} - \frac{5.5}{2(-.0219)^2(5.5)^2} + \frac{1}{2(.0219)^2(5.5)}\right.\right.$$

$$- \frac{1.09}{(-.0219)^3}\left(\log 4.5 + \frac{2(5.5)}{(4.5)} - \frac{(5.5)^2}{2(4.5)^2} - \log 5.5\right.$$

$$\left.\left. - \frac{2(5.5)}{(5.5)} + \frac{(5.5)^2}{2(5.5)^2}\right)\right]$$

$$= \frac{597.90}{E}\left(283.39878 - 566.47 - 189.54794 + 379.0958\right)$$

$$- \frac{6.54}{(-.00001050345 9)E}\left(1.50408 + 2.44444 - .746913 5 - 1.70475 - 2.0 + .5\right)$$

$$= \frac{597.90}{E}\left[9.36037\right] - \frac{-6.54}{(-.00001050345 9)E}\left[.0031391\right]$$

$$= \frac{5596.565223}{E} - \frac{1954.5731}{E} = \frac{3641.9921}{E}$$

ABOVE IS $\frac{M}{EI}$ for HALF FRAME

$$\therefore \frac{7283.9842}{E}$$

FINDING CENTROID OF $\frac{M}{EI}$ diagram

$$\bar{X} = \frac{M}{A}$$

$$M \text{ of } \frac{M}{EI} = \frac{6}{E}\int_0^{45.7} \frac{99.65L^2 - 1.09L^3}{(5.5 - .0219L)^3}\,dL =$$

$$= \frac{6}{E}\left[\int_0^{45.7} \frac{99.65L^2\,dL}{(5.5-.0219L)^3} - \int_0^{45.7} \frac{1.09L^3\,dL}{(5.5-.0219L)^3}\right.$$

$$= \frac{6}{E}\left\{99.65\left[\frac{.0031391}{.00001050345 9}\right] - 1.09\left[\frac{1}{.0219}\left(\frac{L^3}{(5.5-.0219)^2}\right.\right.\right.$$

$$\left.\left.\left. - 3(5.5)\int_0^{45.7} \frac{L^2\,dL}{(5.5-.0219L)^3}\right)\right]\right\}$$

$$= \frac{6}{E}\left\{99.65\left[\frac{.0031391}{.00001050345 9}\right] + \frac{1.09\,(45.7)^3}{.0219\,(4.5)^2} - 16.5\left[\frac{.0031391}{.00001050345 9}\right]\right.$$

$$= \frac{597.90}{E}\left[\frac{.0031391}{.00001050345 9}\right] + \frac{1,407,528.4668}{E}$$

$$- \frac{4,329.497}{E}\left[\frac{.0031391}{.00001050345 9}\right]$$

$$= -\frac{1,293,928.8431}{E} + \frac{1,407,528.4668}{E}$$

$$M \text{ of } \frac{M}{EI} = 113,599.7$$

$$\bar{X} = \frac{113,599.7}{3641.9921}$$

$$\bar{X} = 31.19$$

$\bar{y}$ distance to ABOVE CENTROID.

$$\sin\theta = \frac{y}{31.19}$$

$$y = 31.19\,(.17508)$$

$$y = 5.46$$

TOTAL Y = 25.00 + 5.46 = 30.46

Column MOMENTS (FIXED END)

$$\frac{7283.9842 \times 30.46}{4133.4648} \times 25 = 1341.91375$$

$$\frac{7283.9842 \times 30.46}{4133.4648} \times 20 = 1073.531$$

$$\frac{7283.9842 \times 30.46}{4133.4648} \times 15 = 805.14825$$

$$\frac{7283.9842 \times 30.46}{4133.4648} \times 10 = 536.76555$$

$$\frac{7283.9842 \times 30.46}{4133.4648} \times 5 = 268.38275$$

BEAM

$$\frac{7283.9842 \times 30.46}{4133.4648} \times 25 =$$

$$\frac{7283.9842 \times 30.46}{4133.4648} \times 25.89 = 1389.6858745 \quad 5$$

$$\frac{7283.9842 \times 30.46}{4133.4648} \times 26.78 = 1437.458009 \quad 10$$

$$\frac{7283.9842 \times 30.46}{4133.4648} \times 27.67 = 1485.2301385 \quad 15$$

$$\frac{7283.9842 \times 30.46}{4133.4648} \times 28.56 = 1533.002268 \quad 20$$

$$\frac{7283.9842 \times 30.46}{4133.4648} \times 29.45 = 1580.7743975 \quad 25$$

$$\frac{7283.9842 \times 30.46}{4133.4648} \times 30.34 = 1628.546527 \quad 30$$

$$\frac{7283.9842 \times 30.46}{4133.4648} \times 31.23 = 1676.3186565 \quad 35$$

$$\frac{7283.9842 \times 30.46}{4133.4648} \times 32.12 = 1724.090786 \quad 40$$

$$\frac{7283.9842 \times 30.46}{4133.4648} \times 33 = 1771.32615 \quad 4$$

DEAD LOADS ONLY

FINAL MOMENTS

BEAM

END	-1341.9138
5' "	-918.8659
10' "	-549.9580
15' "	-235.7301
20' "	+23.9977
25' "	+229.2256
30' "	+379.9535
35' "	+476.1813
40' "	+499.9192
℄ "	+506.2178

COLUMN

BASE	-0-
5' FROM "	268.3828
10' " "	536.7655
15' " "	805.1483
20' " "	1073.531
25' " "	1341.9138

Shear

END	99.65
5	88.75
10	77.85
15	66.94
20	55.04
25	44.14
30	33.23
35	22.33
40	11.43
℄	.43

COLUMN

BASE	= 53.67676
5' "	= 53.67676
10' "	= 53.67676
15' "	= 53.67676
20' "	= 53.67676
25' "	= 53.67676

THRUST

END	98.09
5	98.09
10	98.09
15	98.09
20	98.09
25	98.09
30	98.09
35	98.09
40	98.09
℄	98.09

COLUMN

BASE	128.85
5'	122.85
10'	118.65
15'	112.65
20'	105.65
25'	99.65

LIVE LOAD

25 #/sq. ft = 25 x .15 = .375 K/LNFT.

VERTICAL REACTION

.375 x 45.7 = 17.1375

MOMENT (No FIXITY)

@ B	=	0
5' FROM B	=	80.8812
10' " "	=	152.625
15' " "	=	214.7562
20' " "	=	267.750
25' " "	=	311.1312
30' " "	=	345.375
35' " "	=	365.9562
40' " "	=	385.500
℄	=	401.5919

SYMETRICAL ABOUT ℄

NO MOMENTS ON COLUMN.

AREA OF $\frac{M}{EI}$ DIAGRAM

$$\frac{6}{E}\int_0^{45.7} \frac{17.1375(.98455)\ell - \frac{.375(.98455)\ell^2}{2}}{(5.5 - .0219\ell)^3}\,d\ell$$

$$= \frac{6}{E}\int_0^{45.7} \frac{17.111\,\ell\,d\ell}{(5.5 - .0219\ell)^3} - \int_0^{45.7} \frac{.181751\,\ell^2\,d\ell}{(5.5 - .0219\ell)^3}$$

$$= \frac{6}{E}\left\{17.111\left[9.36037\right] - .181751\left[\frac{.0031391}{.0000105^{-03459}}\right]\right.$$

298.86345

$$= \frac{960.991746\,42}{E} - \frac{325.912386}{E}$$

$$= \frac{635.07936}{E}$$ (1/2 FRAME

CENTROID OF $\frac{M}{EI}$

$$= \frac{6}{E}\int_0^{45.7} \frac{17.111\, l^2\, dl}{(5.5-.0219l)^3} - \int_0^{45.7} \frac{.181751\, l^3\, dl}{(5.5-.0219l)^3}$$

$$= \frac{6}{E}(17.111)\left(\frac{.0031391}{.00001050 3459}\right) - \frac{6(.181751)}{-.0219E}\left[\frac{(45.7)^3}{(4.5)^2} - 3(5.5)\int_0^{45.7} \frac{l^2\, dl}{(5.5-.0219l)^3}\right]$$

$$= \frac{6(17.111)(298.86345)}{E} + \frac{6(.181751)}{.0219E}\left\{\left[\frac{(45.7)^3}{(4.5)^2} - 16.5\right]\left[298.86345\right]\right\}$$

$$= \frac{30,683.115}{E} + \frac{245,926.507}{E} - \frac{257,299.247}{E}$$

$$= \frac{19,310.375}{E}$$

$$\bar{X} = \frac{M}{A}$$

$$= \frac{19,310.375}{E} \times \frac{E}{635.07936}$$

$$\bar{X} = 30.41$$

$\cos\theta = \frac{4}{30.41}$

$4 = .17508(30.41)$

$4 = 5.32 + 25 = 30.32$

9.316932

Column

$$\frac{\frac{1270.1587}{E} \times 30.32}{\frac{4133.4648}{E}} \times 25 = 232.9233$$

$$\frac{1270.1587 \times 30.32}{4133.4648} \times 20 \quad 186.3864$$

$$\frac{1270.1587 \times 30.32}{4133.4648} \times 15 \quad 139.75398$$

$$\frac{1270.1587 \times 30.32}{4133.4648} \times 10 \quad 93.16932$$

$$\frac{1270.1587 \times 30.32}{4133.4648} \times 5 \quad 46.58466$$

BEAM

5 $\frac{1270.1587 \times 30.32}{4133.4648} \times 25.89 = 241.2153694 8$

10 $\frac{1270.1587 \times 30.32}{4133.4648} \times 26.78 = 249.5074389 6$

15 $\frac{1270.1587 \times 30.32}{4133.4648} \times 27.67 = 257.7995084 4$

20 $\frac{1270.1587 \times 30.32}{4133.4648} \times 28.56 = 266.0.9157792$

25 $\frac{1270.1587 \times 30.32}{4133.4648} \times 29.45 = 276.2480338$

30 $\frac{1270.1587 \times 30.32}{4133.4648} \times 30.34 = 284.5391032 8$

35 $\frac{1270.1587 \times 30.32}{4133.4648} \times 31.23 = 294.6945591 60$

40 $\frac{1270.1587 \times 30.32}{4133.4648} \times 32.12 = 298.6628640$

℄ $\frac{1270.1587 \times 30.32}{4133.4648} \times 33 = 307.4587 56$

LIVE LOAD ONLY

MOMENTS

BEAM

END −232.92
5' FROM " −160.43
10' " " − 96.88
15 " " − 43.03
20' " " + 1.66
25' " " + 34.88
30' " " + 60.74
35' " " + 71.27
40' " " + 86.84
℄ + 94.13

COLUMN

BASE — 0 —
5' FROM BASE 46.58
10' " " 93.17
15' " " 139.75
20' " " 186.34
25' " " 232.92

Shear

BEAM

B	= 17.13
5' FROM	= 15.25
10	13.38
15	11.50
20	9.63
25	7.75
30	5.88
35	4.00
40	2.13
℄	.13

Column.

BASE	9.3168
5' FROM "	9.3168
10' " "	9.3168
15' " "	9.3168
20' " "	9.3168
25' " "	9.3168

THRUST

BEAM

B	9.30
5' "	9.30
10'	9.30
15	9.30
20	9.30
25	9.30
30	9.30
35	9.30
40	9.30
℄	9.30

Column

BASE	17.1375
5' FROM "	17.1375
10' " "	17.1375
15' " "	17.1375
20' " "	17.1375
25' " "	17.1375

DESIGN OF COLUMN.

BASE

M = 0

shear = 53.68 + 9.32 = 63.0

THRUST = 128.85 + 17.14 = 145.99

$P = A_g(.225 f'_c + f_s p_g)$

$146{,}000 = 864(.225(3000 + 20{,}000 p_g))$

$p_g = \frac{-583{,}200 + 146{,}000}{3{,}888{,}000}$

No steel REQ'D BY ABOVE OPERATION.

USE MINIMUM .0025 x bd = .0025 x 816 = 2.04

3 - 1" ϕ ⟸ USE

check shear ON ENTIRE column

BASE IS USED AS AREA SMALLEST & shear uniform THROUGHOUT column.

$v = \frac{V}{bjd} = \frac{63}{24 \times 866 \times 34} = 89.4$ o.k. NO STIRRUPS NEEDED

5' FROM BASE

$f_s = 20{,}000$ n = 0

$f_c = 1350$

d = 41.5 d' = 2

d'' = 19.8

b = 24

M = 268.38 + 46.58 = 314.96

N = 122.85 + 17.14 = 139.99

$e = \frac{12M}{N} + d'' = \frac{12(314.96)}{139.99} + 19.8 = 27 + 19.8 = 46.8$ IN.

$E = \frac{e}{12} = \frac{46.8}{12} = 3.9$ FT.

TABLE 1 (RCDH) K = 236

" 4 " b x d = 24 x 41.5 F 3.53

NE 139.99 x 3.9 = 545

KF 236 x 3.53 = 833

No COMP STEEL NEEDED

TABLE 10 $\frac{e}{d} = \frac{46-8}{41.5} = 1.13$ $i =$ APPROX 4.0 $a = 1.44$

$A_s = \frac{NE}{adi} = \frac{545}{1.44 \times 41.5 \times 4.0} = 2.28$

2 - 1 1/8 ϕ USE

MIN $= .0025 \times 24 \times 41.5 = 2.49$

covered by above steel

10' FROM BASE

$f_s = 20{,}000$ $n = 10$

$f_c = 1350$

$d = 49$ $d' = 2$

$d'' = 23.5$

$M = 536.77 + 93.17 = 629.94$

$N = 118.65 + 17.14 = 135.79$

$e = \frac{12M}{N} + d'' = \frac{12(629.94)}{135.79} + 23.5 = 55.6 + 23.5 = 79.1$ INCH

$E = \frac{e}{12} = \frac{79.1}{12} = 6.6$ FT.

TABLE 1 (RCHD) $K = 236$

TABLE 4 $b \times d = 24 \times 49$ $F = 4.82$

$NE = 135.79 \times 6.6 = 890$

$KE = 236 \times 4.82 = 1130$

NO COMP STEEL NEEDED

TABLE 10 $\frac{e}{d} = \frac{79.1}{49} = 1.615$ $i = 2.17$ $a = 1.44$

$A_s = \frac{NE}{adi} = \frac{890}{1.44 \times 49 \times 2.17} = 5.8$

USE 4 - 1 1/4 ϕ BARS

MIN $= .0025 \times 49 \times 2 = 2.94$

15' FROM BASE

$f_s = 20{,}000$ $n = 10$

$f_c = 1350$

$d = 56.5$ $d' = 2$

$d'' = 27.2$

$M = 805.15 + 139.75 = 944.9$

$N = 112.65 + 17.14 = 129.79$

$e = \frac{12M}{N} + d'' = \frac{12(944.9)}{129.79} + 27.2 = 87.5 + 27.2 = 114.7''$

$E = \frac{114.7}{12} = 9.56$ FT.

TABLE 1 (RCHD) $K = 236$

TABLE 4 (RCHD) $b \times d = 24 \times 56.5$ $F = 6.27$

NE 129.79 x 9.56 = 1200
KF 236 x 6.27 = 1480

NO COMP STEEL NEEDED.

Table 10 $\frac{e}{d} = \frac{114.7}{56.5} = 2.03$ $i = 1.76$ $a = 1.44$

$\frac{NE}{adi} = \frac{1200}{1.44 \times 56.5 \times 1.76} = 8.38$

6 – 1¼" □ USE

20' FROM BASE

$f_s = 20{,}000$ $n = 10$
$f_c = 1350$
$d = 64$ $d' = 2$
$d'' = 31$
$M = 1073.53 + 168.34 = 1241.87$
$N = 105.65 + 17.14 = 122.79$

$e = \frac{12M}{N} + d'' = \frac{12(1241.87)}{122.79} + 31 = 121.5 + 31 = 152.5$ in.
$E = \frac{152.5}{12} = 12.7'$

Table 1 (RCHD) $K = 236$
Table 4 (RCHD) $b \times d = 24 \times 64$ $F = 8.19$

NE = 122.79 x 12.7 = 1558
KF = 236 x 8.19 = 1932

NO COMP STEEL

TABLE 10 $\frac{e}{d} = \frac{152.5}{64} = 2.39$ $i = 1.56$ $a = 1.44$

$A'_s = \frac{NE}{adi} = \frac{1558}{1.44 \times 64 \times 1.56} = 10.8$

USE 7 - 1¼ □
TWO ROWS Steel

DESIGN OF BEAM PORTION

5' FROM OUTER END

$f_s = 20{,}000 \quad n = 10$

$f_c = 1350$

$d = 64 \quad d' = 2$

$d'' = 31$

$M = 918.87 + 160.43 = 1079.30$

$N = 98.08 + 9.30 = 107.38$

$e = \frac{12M}{N} + d'' = \frac{12(1079.30)}{107.38} + 31 = 120.5 + 31 = 151.5$ IN

$E = \frac{151.5}{12} = 12.65$

TABLE 1 (RCHD) $\quad K = 236$

TABLE 4 (RCHD) $b \times d \quad F = 8.19$

$NE \quad 107.38 \times 12.65 = 1360$

$KF \quad 236 \times 8.19 = 1932$

NO COMP steel

Table 10 $\frac{e}{d} \quad \frac{151.5}{64} = 2.37 \quad i = 1.56 \quad a = 1.44$

$A'_s = \frac{NE}{adi} = \frac{1360}{1.44 \times 64 \times 1.56} = 9.48$

USE 7 - 1¼ □

TWO ROWS OF STEEL

10' FROM OUTER END

$f_s = 20{,}000 \quad n = 10$

$f_c = 1350$

$d = 62.5 \quad d' = 2$

$d'' = 31.5$

$M = 549.96 + 96.88 = 646.84$

$N = 98.09 + 9.30 = 107.39$

$e = \frac{12M}{N} + d'' = \frac{12(646.84)}{107.39} + 31.5 = 72.2 + 31.5 = 103.7$ IN.

$E = \frac{103.7}{12} = 8.65$

TABLE 1 (RCHD) $\quad K = 236$

TABLE 4 (RCHD) $b \times d = 24 \times 62.5 \quad F = 7.90$

$NE = 107.39 \times 8.65 = 930$

$KF = 236 \times 7.90 = 1865$ NO COMP STEEL

TABLE 10: $\frac{e}{d} = \frac{103.7}{62.5} = 1.66 \quad i = 2.10 \quad a = 1.44$

$\frac{NE}{adi} \quad \frac{930}{1.44 \times 62.5 \times 2.10} = 4.92$ 4 - 1¼ □ BARS

15' FROM OUTER END

$f_s = 20,000 \quad n = 10$

$f_c = 1350$

$d = 61 \quad d' = 2$

$d'' = 29.5$

$M = 235.73 + 43.03 = 278.76$

$N = 98.08 + 9.30 = 107.38$

$e = \frac{12M}{N} + d'' = \frac{12(278.76)}{107.38} + 29.5 = 31.2 + 29.5 = 60.7$

$E = \frac{60.7}{12} = 5.07$

TABLE 1 (REND) $\quad K = 236$

TABLE 4 (REND) $\quad b \times d = 24 \times 61 \quad F = 7.30$

$NE = 107.38 \times 5.07 = 545$

$KF = 236 \times 7.30 = 1722$ NO COMP STEEL

TABLE 10 $\quad \frac{e}{a} = \frac{60.7}{60} = .99 \quad i = 4$ (Maybe) $\quad a = 1.44$

$\frac{NE}{adu} = \frac{545}{1.44 \times 61 \times 4} = 1.55$

$MIN = .0025 \times 24 \times 61 = 3.66 \quad 3 - 1\frac{1}{8}$

20' FROM OUTER END

$f_s = 20,000 \quad n = 10$

$f_c = 1350$

$d = 59.5 \quad d' = 2$

$d'' = 28.7$

$M = 24.00 \quad 1.66 = 25.66$

$N = 98.08 + 9.30 = 107.38$

$e = \frac{12M}{N} + d' = \frac{12(25.66)}{107.38} + 28.7 = 2.88 + 28.7 = 31.58$

$E = \frac{e}{12} = \frac{31.58}{12} = 2.63$

TABLE 1 $\quad K = 236$

TABLE 4 $\quad F = 7.20$

$NE = 107.38 \times 2.63 = 282.2$

$KF = 236 \times 7.20 = 1700$ NO COMP STEEL

TABLE 10 $\quad \frac{e}{a} = \frac{31.58}{59.5} \quad i = 3$

DOES NOT MATTER AS MIN CONTROLS.

$MIN = .0025 \times 24 \times 59.5 = 3.57$

$3 - 1\frac{1}{8}$

25' FROM OUTEREND

$f_s = 20{,}000 \quad n = 10$

$f_c = 1350$

$d = 58 \quad d = 2$

$d'' = 28$

$M = 229.23 + 34.88 = 264.11$

$N = 98.09 + 9.30 = 107.39$

$e = \frac{12M}{N} + d'' = \frac{12(264.11)}{107.39} + 28 = 29.5 + 28 = 57.5$

$E = \frac{57.5}{12} = 4.78$

TABLE 1 (RCHD) $\quad K = 236$

TABLE 4 $\quad 24 \times 58 \quad F = 6.73$

$NE = 107.39 \times 4.78 = 514$

$KF = 236 \times 6.73 = 1590$ No COMP STEEL

TABLE $\frac{e}{d} = 1$ approx $\quad i = 4 \quad a = 1.44$

$A_s = \frac{NE}{adi} = \frac{514}{1.44 \times 58 \times 4} = 1.53$

MIN STEEL $= .0025 \times 24 \times 58 = 3.48 \quad 3\ 1/8\ \square$

30' FROM OUTER END

$f_s = 20{,}000 \quad n = 10$

$f_c = 1350$

$d = 56.5 \quad d = 2$

$d'' = 27.2$

$M = 60.74 + 379.95 = 440.69$

$N = 107.39$

$e = \frac{12M}{N} + d'' = \frac{12(440.69)}{107.36} + 27.2 = 49.3 + 27.2 = 76.5$

$E = \frac{76.5}{12} = 6.38$

TABLE 1 $\quad K = 236$

TABLE 4 $\quad 24 \times 56.5 \quad F = 6.27$

$NE = 107.39 \times 6.38 = 685$

$KF = 236 \times 6.27 = 1480$ No COMP STEEL

TABLE $\frac{e}{d} = \frac{76.5}{56.5} = 1.36 \quad i = 2.90 \quad a = 1.44$

$A_s \frac{NE}{adi} = \frac{685}{1.44 \times 56.5 \times 2.9} = 2.91$

MIN STEEL $.0025 \times 25 \times 56.5 = 3.39 \quad 3 - 1\ 1/8\ \square$

35' FROM OUTER EDGE

$f_s = 20{,}000 \quad n = 10$

$f_c = 1350$

$d = 55 \quad d'2$

$d'' = 26.5$

$M = 476.18 + 71.27 = 547.45$

$N = 107.39$

$e = \frac{12M}{N} + d'' = \frac{12(547.45)}{107.39} + 26.5 = 61 + 26.5 = 87.5$

$E = \frac{87.5}{12} = 7.3$

$K = 236 \qquad F = 6.46$

$NE = 107.39 \times 7.3 = 783$

$KF = 236 \times 6.46 = 1525$ NO COMP STEEL.

$\frac{e}{d} = 1.59 \quad i = 2.17 \quad a = 1.44$

$A_s = \frac{783}{1.44 \times 55 \times 2.17} = 4.56 \qquad 3 - 1\frac{1}{4}$ □

40' FROM OUTER EDGE

$f_s = 20{,}000 \quad n = 10$

$f_c = 1350$

$d = 53.5 \quad d'$

$d'' = 25.8$

$M = 86.84 + 499.92 = 586.76$

$N = 107.39$

$e = \frac{12M}{N} + d'' = \frac{12(586.76)}{107.39} + 25.8 = 65.6 + 25.8 = 91.4$

$E = \frac{91.4}{12} = 7.6$

$K = 236 \qquad F = 5.84$

$NE = 107.39 \times 7.6 = 815$

$KF = 236 \times 5.84 = 1380$ NO COMP. STEEL

$\frac{e}{d} = \frac{91.4}{53.5} = 1.7 \qquad i = 2.04 \qquad a = 1.44$

$\frac{NE}{adi} = \frac{815}{1.44 \times 53.5 \times 2.04} = 5.2 \qquad 4 - 1\frac{1}{4}$ ⟸

℄ BEAM check.

$f_s = 20{,}000 \quad n = 16$

$f_c = 1350$

$d = 52 \quad d = 2$

$d'' = 25$

$M = 94.13 + 506.22 = 600.35$

$N = 107.39$

$e = \frac{12M}{N} + d'' = \frac{12(600.35)}{107.39} + 25 = 67.2 + 2.5 = 92.2$ IN.

$E = \frac{92.2}{12} = 7.7$

$K = 236 \quad F = 6.19$

$NE = 107.39 \times 7.7 = 827$

$KF = 236 \times 6.19 = 1460$ NO COMP STEEL

$\frac{e}{d} = \frac{92.2}{52} = 1.78$ $c = 1.92$ $a = 1.44$

$A_s = \frac{NE}{adc} = \frac{827}{1.44 \times 52 \times 1.92} = 5.77$ 4 - 1 1/4 ⟸

check shear on BEAM & Bond

℄ $\begin{cases} U = \frac{V}{bjd} = \frac{0}{24 \times .866 \times 52} = 0 \quad \text{O.K} \\ \varepsilon_0 = \frac{V}{u_j d} = \frac{0}{150 \times .866 \times 52} = \text{O.K} \end{cases}$

40' $\begin{cases} V = 11.43 + 2.13 = 13.56 \\ U = \frac{13.56}{24 \times .866 \times 53.5} = 12.3 \\ \varepsilon_0 = \frac{13.56}{150 \times .866 \times 53.5} = 1.945 \end{cases}$

35' $\begin{cases} U = 4.00 + 22.33 = 26.33 \\ U = \frac{26.33}{24 \times .866 \times 55} = 23.0 \\ \varepsilon_0 = \frac{26.33}{150 \times .866 \times 55} = 3.69 \end{cases}$

30 $\begin{cases} U = 33.23 + 5.88 = 39.11 \\ U = \frac{39.11}{24 \times .866 \times 56.5} = 33.4 \\ \varepsilon_0 = \frac{39.11}{150 \times .866 \times 56.5} = 5.32 \end{cases}$

CON'T Check BEAM SHEAR & BOND.

25

$V = 44.14 + 7.75 = 51.89$

$v = \frac{V}{bjd} = \frac{51.89}{24 \times .866 \times 58} = 43$

$\tau_0 = \frac{V}{\Sigma jd} = \frac{51.89}{150 \times .866 \times 58} = 6.9$

20

$V = 55.04 + 9.63 = 64.67$

$v = \frac{64.67}{24 \times .866 \times 59.5} = 52.40$

$\tau_0 = \frac{64.67}{150 \times .866 \times 59.5} = 8.29$

15

$V = 66.44 + 11.50 = 77.94$

$v = \frac{77.94}{24 \times .866 \times 61} = 61.5$

$\tau_0 = \frac{77.94}{150 \times .866 \times 61} = 9.85$

10

$V = 77.85 + 13.38 = 91.23$

$v = \frac{91.23}{24 \times .866 \times 62.5} \quad 70.3$

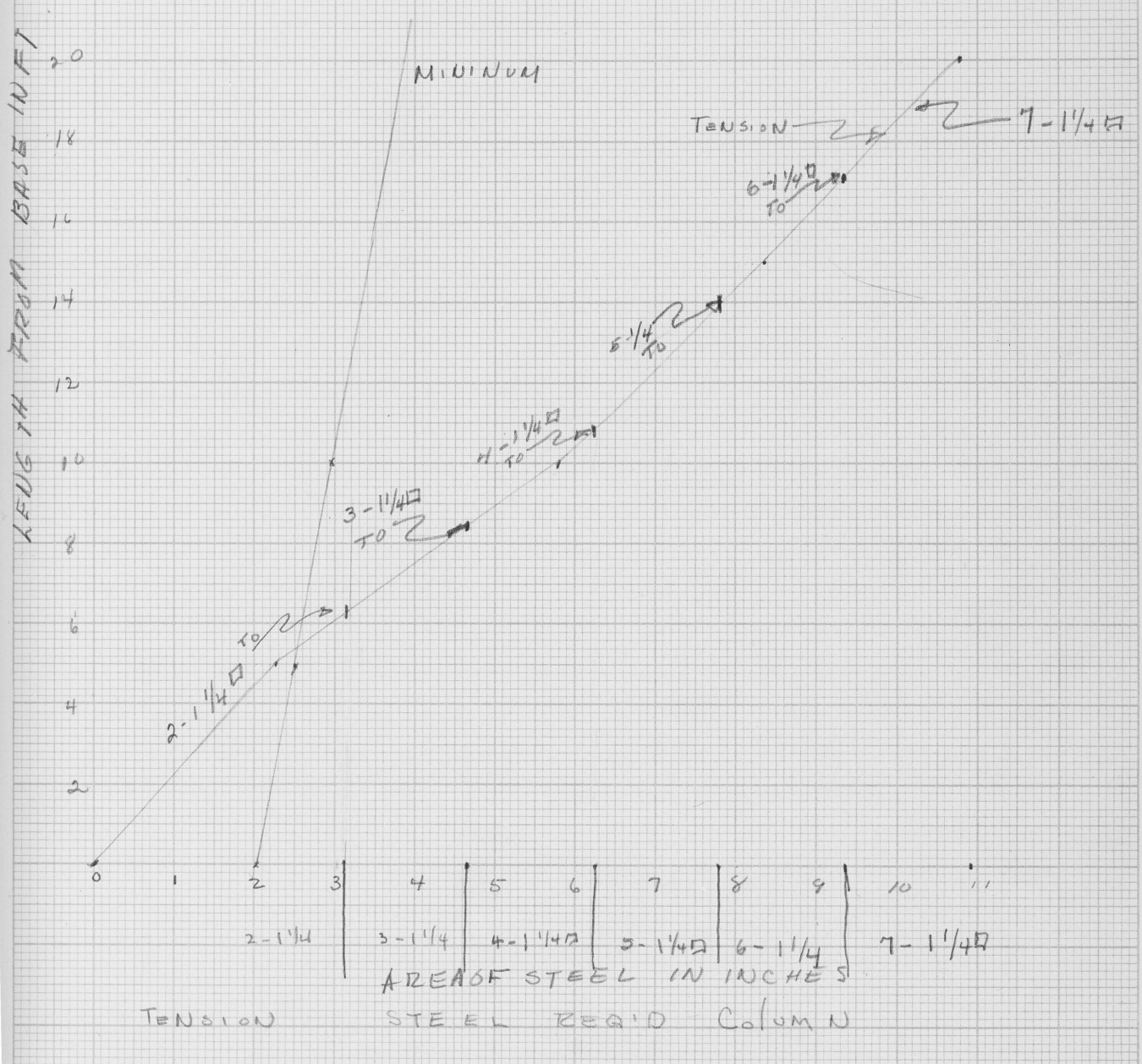
$\tau_0 = \frac{91.23}{150 \times .866 \times 62.5} \quad 11.25$

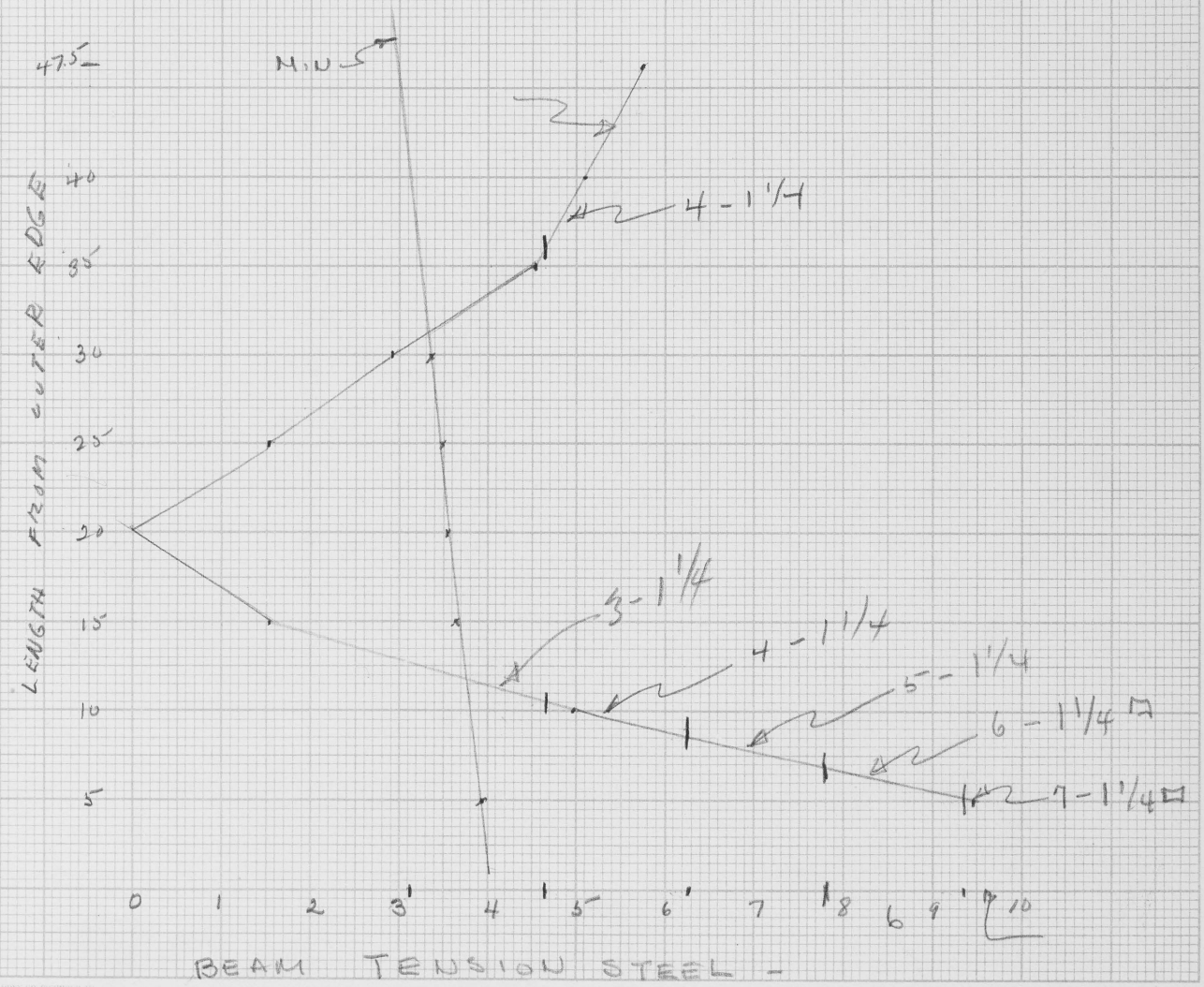
5

$V = 88.75 + 15.25 = 104.00$

$v = \frac{104.00}{24 \times .866 \times 64} = 78.2$

$\tau_0 = \frac{104.00}{150 \times .866 \times 64} = 12.5$


LENGTH FROM BASE IN FT
20
18
16
14
12
10
8
6
4
2
MINIMUM
TENSION
7-1¼□
6-1¼□ TO
5-1¼□ TO
4-1¼□ TO
3-1¼□ TO
2-1¼□ TO
0
1
2
3
4
5
6
7
8
9
10
11
2-1¼
3-1¼
4-1¼□
5-1¼□
6-1¼
7-1¼□
AREA OF STEEL IN INCHES
TENSION STEEL REQ'D COLUMN



EVERETT WARDEN CO. RICHMOND, VA

It occurred to me the shears should be adjusted on beam to meet change in frame size so rough adjustment made -

Change of width from 24 to 19.5 -		4.5"
change of depth at base from 37 to 29		8"
" " " at haunch "	67 to 52	12"
" " " at center "	55 to 43	12"

An approx change of 1" in depth throughout.

Weight change in column
1 x 23 x 4.5 x 144 = 14.9 or about 15 K

Weight change in beam
1 x 49.7 x 4.5 x 144 = 28.3 or about 28.4

This make in approx change of 2/3 on beam

	Dead	+	L.L	=	Total
End	66.5	+	17.1	=	83.6
5'	59.2	+	15.3	=	74.5
10'	51.8	+	13.4	=	65.2
15'	44.6	+	11.5	=	56.1
20	36.7	+	9.6	=	46.3
25	29.4	+	7.8	=	37.2
30	22.2	+	5.9	=	28.1
35	14.9	+	4.0	=	18.9
40	7.6	+	2.1	=	9.7
℄	.06	+	.13	=	.14

26
3

$f_s = 20{,}000 \quad n = 10$

$f_c = 1350$

$d = 49 \quad d' = 3$

$d'' = 23$

$b = 19.5$

$M = 1241.87$

$N = 122.79$

$e = \frac{12M}{N} + d'' = \frac{12(1241.87)}{122.79} + 23 = 121.5 + 23 = 144.5$

$E = \frac{144.5}{12} = 12.1$

$K = 236 \quad F = 3.80$

$NE = 122.79 \times 12.1 = 1480$

$KF = 236 \times 3.80 = 896$

584

$\frac{d'}{d} = \frac{3}{49} = .0613 = c = .81$

$A'_s = \frac{NE - KF}{cd} = \frac{584}{.81 \times 49} = 14.70$

$\frac{E}{d} = \frac{144.5}{49} = 2.95 \quad i = 1.41 \quad a = 1.44$

$A_s = \frac{NE}{adi} = \frac{1480}{1.44 \times 49 \times 1.41} = 14.90$

PROPORTION SET UP FOR OTHER LIMITS - OF D

$\frac{49^3}{64^3} = \frac{x^3}{52^3} \quad x = 40$

$\frac{49^3}{64^3} = \frac{x^3}{34^3} = \quad x = 26$

NEW FRAME $b = 19.5$

$d = 26$ (AT BASE OF COLUMN OVER ALL 29

$d = 49$ (AT TOP OF COLUMN) " " 52

$d = 40$ (AT ℄) 43

$25 \overline{)23.8} = .0767$

225

50

BASE OF COLUMN

MIN STEEL REQ'D = .0025 × 19.5 × 26 = 1.27

check shear on entire column

$v = \frac{V}{b_o d} = \frac{63}{19.5 \times .866 \times 26} = 143.2$ STIRRUPS NEEDED

.0767 in/in

DESIGN OF STIRRUPS.

143.2 − 90 = 53.2 ✓

$s = \frac{.44 \times 20 \times .866 \times 27}{53.2} = 3.86''$

27.29
.34

137 − 90 = 47

$\frac{.44 \times 20 \times .866 \times 27.29}{47} = 4.42$

.07
.28

27.63

135 − 90 = 45

$\frac{44 \times 20 \times .866 \times 27.63}{45} = 4.68$

.36
27.99

134 − 90 = 44

$\frac{44 \times 20 \times .866 \times 27.99}{44} = 4.85$

.37
28.36

132 − 90 = 42

$\frac{44 \times 20 \times .866 \times 28.36}{42} = 5.15$

.40
28.76

130 − 90 = 40

$\frac{44 \times 20 \times .866 \times 28.76}{40} = 5.48$

.42
29.18

128 − 90 = 38

$\frac{44 \times 20 \times .866 \times 29.18}{38} = 5.86$

.45
29.63

125.5 − 90 = 35.5

$\frac{44 \times 20 \times .866 \times 29.63}{35.5} = 6.37$

.48
30.11

124 − 90 = 34

$\frac{44 \times 20 \times .866 \times 30.11}{34} = 6.77$

.52
30.68

122 − 90 = 32

$\frac{44 \times 20 \times .866 \times 30.63}{32} = 7.33$

.56
31.19

120 − 90 = 30

$\frac{31.19}{30} = 7.9$

.61
31.80

117.5 − 90 = 27.5

$\frac{44 \times 20 \times .866 \times 31.80}{27.5} = 8.84$

.68
32.48

115 − 90 = 25

$\frac{44 \times 20 \times .866 \times 32.48}{25} = 9.92$

.76
33.24

$$\frac{44 \times 20 \times .866 \times 32.24}{22} = 11.55$$

$$\frac{44 \times 20 \times .866 \times 33.3}{20} = 12.55$$

$$\frac{44 \times 20 \times .866 \times 33.99}{18} = 14.4$$

$$\frac{44 \times 20 \times .866 \times 35}{16} = 22.3$$

$$\frac{44 \times 20 \times .866 \times 36.81}{12} = 23.4$$

$$\frac{44 \times 20 \times .866 \times 38.61}{7} = 42.2$$

5' FROM BASE

$f_s = 20000 \quad n = 10$

$f_c = 1350$

$b = 19.0$

$d = 32.75 \quad d' = 2$

$d'' = 15.38$

$M = 314.96$

$N \quad 139.94$

$e = \frac{12M}{N} + d'' = \frac{12(314.96)}{139.94} + 15.38 = 27 + 15.38 = 42.38$

$E = \frac{42.38}{12} = 3.53'$

$K = 236 \quad F = 1.83$

$NE = 139.94 \times 3.53 = 494$

$KF = 236 \times 1.83 = 432$

62 COMP STEEL NEEDED

$\frac{d'}{d} = \frac{2}{32.75} = .061 \quad C = .81$

$A'_s = \frac{NE - KF}{Cd} = \frac{62}{.81 \times 32.75} = 2.34$

$\frac{e}{d} = \frac{42.38}{32.75} = 1.29 \quad i = 3.0 \quad a = 1.44$

$A_s = \frac{NE}{adi} = \frac{494}{3.0 \times 31.75 \times 1.44} = 3.6$

10' FROM BASE

$f_s = 20{,}000 \quad n = 10$

$f_c = 1350$

$b = 19.5$

$d = 37.50 \quad d' = 3$

$d'' = 17.25$

$M = 629.94$

$N = 135.79$

$e = \frac{12M}{N} + d'' = \frac{12(629.94)}{135.79} + 18.25 = 55.5 + 17.25 = 72.75$

$E = \frac{e}{12} = \frac{72.75}{12} = 6.06$

$K = 236 \quad F = 2.23$

$NE = 135.79 \times 6.06 = 822$

$KF = 236 \times 2.23 = 526$

$NE - KF \quad 296$ COMP. STEEL NEEDED

$\frac{d'}{d} = \frac{3}{37.50} = .08 \quad c = .75$

$A'_s = \frac{NE - KF}{cd} = \frac{296}{.75 \times 37.50} = 10.5$

$\frac{e}{d} = \frac{72.75}{37.50} = 1.94 \quad a = 1.44 \quad i = 1.79$

$A_s = \frac{NE}{adi} = \frac{822}{1.44 \times 37.50 \times 1.79} = 8.52$

15' FROM BASE

$f_s = 20{,}000 \quad n = 10$

$f_c = 1350$

$b = 19.5$

$d = 43.25 \quad d' = 3$

$d'' = 20.12$

$M = 944.9$

$N = 129.79$

$e = \frac{12M}{N} + d'' = \frac{12(944.9)}{129.79} + 20.12 = 87.5 + 20.12 = 107.62$

$E = \frac{e}{12} = \frac{107.62}{12} = 8.95$

$K = 236 \quad F = 2.96$

$NE = 129.79 \times 8.95 = 1160$

$KF = 236 \times 2.96 = 698$

$NE - KF \qquad 462$

$\frac{d'}{d} = \frac{3}{43.25} = .0694 \qquad c = .78$

$A'_s = \frac{NE - KF}{cd} = \frac{462}{.78 \times 43.25} = 13.55$

$\frac{e}{d} = \frac{107.62}{43.25} = 2.49 \qquad i = 1.52 \qquad a = 1.44$

$A_s = \frac{NE}{adi} = \frac{1160}{1.44 \times 43.25 \times 1.52} = 12.8$

DESIGN OF BEAM 5' FROM OUTER EDGE

$f_s = 20{,}000 \quad n = 10$

$f_c = 1350$

$b = 19.5$

$d = 49, \quad d' = 3$

$d'' = 23$

$M = 1079.30$

$N = 107.38$

$e = \frac{12M}{N} + d'' = \frac{12(1079.30)}{107.38} + 23 = 120.5 + 23 = 143.5$

$E = \frac{143.5}{12} = 11.95$

$K = 236 \quad F = 3.80$

$NE = 107.38 \times 11.95 = 1405$

$KF = 236 \times 3.80 = 896$

509

$\frac{d'}{d} = \frac{3}{49} = .0612 \quad c = .81$

$A'_s = \frac{NE - KF}{cd} = \frac{509}{.81 \times 49} = 12.8$

$\frac{e}{d} = \frac{143.5}{49} = 2.93 \quad a = 1.44 \quad i = 1.42$

$A_s = \frac{NE}{adi} = \frac{1405}{1.44 \times 49 \times 1.42} = 14.05$

check shear & BOND

$v = \frac{V}{bjd} = \frac{74.5}{19.5 \times .866 \times 49} = 90$

$\xi_o = \frac{V}{\Sigma jd} = \frac{7405}{20 \times .866 \times 49} = 8.7$

DESIGN OF BEAM 10' FROM OUTER EDGE

$f_s = 20{,}000$ $n = 10$

$f_c = 1350$

$b = 19.5$

$d = 48.9$ $d' = 2$

$d'' = 23.45$

$M = 646.84$

$N = 107.39$

$e = \frac{12M}{N} + d'' = \frac{12(646.84)}{107.39} + 23.45 = 72.2 + 23.45 = 95.65$

$E = \frac{e}{12} = \frac{95.65}{12} = 7.96$

$K = 236$ $F = 3.80$

$NE = 107.39 \times 7.96 = 855$

$KF = 236 \times 3.80 = 874$

NO COMP STEEL

$\frac{e}{d} = \frac{95.65}{48.9} = 1.96$ $a = 1.44$ $i = 1.79$

$A_s = \frac{NE}{adi} = \frac{855}{1.44 \times 48.9 \times 1.79} = 6.79$

check FOR SHEAR & BOND:

$v = \frac{V}{bjd} = \frac{65.2}{48.9 \times .866 \times 19.5} = 78$

$\Sigma_0 = \frac{V}{\omega jd} = \frac{65.2}{48.9 \times .866 \times 20} = 7.7$

DESIGN OF BEAM 15' FROM OUTER EDGE

$f_s = 20{,}000 \quad n = 10$

$f_c = 1350$

$b = 19.5$

$d = 47.8 \quad d' = 2$

$d'' = 22.9$

$M = 27876$

$N = 107.38$

$e = \frac{12M}{N} + d'' = \frac{12(27876)}{107.38} + 22.9 = 31.2 + 22.9 = 54.1$

$E = \frac{54.1}{12} = 4.52$

$K = 236 \quad F = 3.65$

$NE = 107.38 \times 4.52 = 485$

$KF = 236 \times 3.65 = \underline{860}$

NO COMP STEEL

$\frac{e}{d} = \frac{54.1}{47.8} = 1.13$ i = about 4 $\quad a = 1.44$

$A_s = \frac{NE}{adi} = \frac{485}{1.44 \times 47.8 \times 4} = 1.76$

$M.N = .0025 \times 19.5 \times 47.8 = 2.33$

check for shear & BOND

$v = \frac{V}{bjd} = \frac{56.1}{47.8 \times .866 \times 19.5} = 69.5$

$\Sigma_o = \frac{V}{\mu jd} = \frac{56.1}{47.8 \times .866 \times 20} = 6.77$

DESIGN OF BEAM 20 FT. FROM OUTER EDGE

$f_s = 20{,}000$ $n = 10$

$f_c = 1350$

$b = 19.5$

$d = 46.7$ $d' = 2$

$d'' = 22.3$

$M = 25.66$

$N = 107.38$

MIN. STEEL REQ'D $= .0025 \times 19.5 \times 46.7 = 2.27$

Chear for shear & BOND.

$$v = \frac{V}{bjd} = \frac{46.3}{46.7 \times .866 \times 19.5} = 58.5$$

$$\Sigma o = \frac{V}{\Sigma j d} = \frac{46.3}{46.7 \times .866 \times 20} = 5.7$$

DESIGN OF BEAM 25' FROM OUTER EDGE

$f_s = 20,000 \quad n = 10$

$f_c = 1350$

$b = 19.5$

$d = 45.6 \quad d' = 2$

$d'' = 21.8$

$M = 264.11$

$N = 107.39$

$e = \frac{12M}{N} + d'' = \frac{12(264.11)}{107.39} + 21.8 = 29.5 + 21.8 = 51.3''$

$E = \frac{51.3}{12} = 4.27'$

$K = 236 \quad F = 3.28$

$NE = 107.39 \times 4.27 = 458$

$KF = 236 \times 3.24 = 764$ NO COMP STEEL

$\frac{e}{d} = \frac{51.3}{45.6} = 1.13$ i = about 4 $a = 1.44$

$A = \frac{NE}{adi} = \frac{458}{1.44 \times 45.6 \times 4} = 1.75$

$M.W = .0025 \times 19.5 \times 45.6 = 2.22$

Check Shear & BOND

$v = \frac{V}{bjd} = \frac{37.2}{45.6 \times .866 \times 19.5} = 48.2$

$v = \frac{V}{ujd} = \frac{37.2}{45.6 \times .866 \times 20} = 4.7$

DESIGN OF BEAM 30' FROM OUTER EDGE

$f_s = 20{,}000 \quad n = 10$

$f_c = 1350$

$b = 19.5$

$d = 44.5 \quad d = 2$

$d'' = 21.2$

$M = 440$

$N = 107.39$

$e = \frac{12M}{N} + d'' = \frac{12(440)}{107.39} + 21.2 = 49.3 + 21.2 = 70.5''$

$E = \frac{70.5}{12} = 5.88'$

$K = 236 \quad F = 3.14$

$NE = 107.39 \times 5.88 = 631$

$KF = 236 \times 3.14 = \underline{731}$ NO COMP STEEL NEEDED

$\frac{e}{d} = \frac{70.5}{44.5} = 1.585 \quad i = 2.19 \quad a = 1.44.$

$A_s = \frac{NE}{adi} = \frac{631}{1.44 \times 44.5 \times 2.19} = 4.52$

Check for shear & BOND

$v = \frac{V}{bjd} = \frac{28.1}{44.5 \times .866 \times 19.5} = 37.4$

$\sigma_o = \frac{V}{\Sigma jd} = \frac{28.1}{44.5 \times .866 \times 20} = 3.65$

DESIGN OF BEAM 35' FROM OUTER EDGE

$f_s = 20{,}000 \qquad n = 10$

$f_c = 1350$

$b = 19.5$

$d = 43.3 \qquad d' = 2$

$d'' = 20.6$

$M = 547.45$

$N = 107.39$

$e = \frac{12M}{N} + d'' = \frac{12(547.45)}{107.39} + 20.6 = 61 + 20.6 = 81.6$

$E = \frac{81.6}{12} = 6.8$

$K = 236 \qquad F = 2.93$

$NE = 107.39 \times 6.8 = 730$

$KF = 236 \times 2.93 = 692$

38 COMP STEEL NEEDED

$\frac{d'}{d} = \frac{2}{43.3} = .046 \qquad c = .86$

$A'_s = \frac{NE - KF}{cd} = \frac{38}{.86 \times 43.3} = 1.02$

$\frac{e}{d} = \frac{81.6}{43.3} = 1.885 \qquad a = 1.44 \quad i = 1.83$

$A_s = \frac{NE}{adi} = \frac{730}{1.44 \times 43.3 \times 1.83} = 6.4$

Check. BOND & Shear.

$v = \frac{V}{bjd} = \frac{18.9}{43.3 \times .866 \times 19.5} = 25.8$

$\varepsilon_0 = \frac{V}{\omega j d} = \frac{18.9}{43.3 \times .866 \times 20} = 2.52$

DESIGN OF BEAM 40' FROM outer edge

$f_s = 20{,}000$ $n = 10$

$f_c = 1350$

$b = 19.5$

$d = 41.2$ $d' = 2$

$d'' = 20.1$

$M = 586.76$

$N = 107.39$

$e = \frac{12M}{N} + d'' = \frac{12(586.76)}{107.39} + 20.1 = 65.6 + 20.1 = 85.7'$

$E = \frac{85.7}{12} = 7.14$

$K = 236$ $F = 2.66$

$NE = 107.39 \times 7.14 = 795$

$KF = 236 \times 2.66 = 627$

168 COMP STEEL NEEDED.

$\frac{d'}{d} = \frac{2}{41.2}$ $.047$ $c = .87$

$A'_s = \frac{NE - KF}{cd} = \frac{168}{.87 \times 41.2} = 4.7$

$\frac{e}{d} = \frac{85.7}{41.2} = 2.08$ $j = 1.69$ $a = 1.44$

$A_s = \frac{NE}{adj} = \frac{795}{1.44 \times 41.2 \times 1.69} = 7.95$

check shear & BOND –

$v = \frac{V}{bjd} = \frac{9.7}{41.2 \times .866 \times 19.5} = 13.9$

$\varepsilon_o = \frac{V}{ujd} = \frac{9.7}{41.2 \times .866 \times 20} = 1.36$

DESIGN OF BEAM AT ℄. (45.7 FROM END OF BEAM)

$f_s = 20{,}000 \qquad n = 10$

$f_c = 1350$

$b = 19.5$

$d = 40 \qquad d' = 2$

$d'' = 19.5$

$M = 600.35$

$N = 107.39$

$e = \frac{12M}{N} + d'' = \frac{12(600.35)}{107.39} + 19.5 = 67.2 + 19.5 = 86.7''$

$E = \frac{86.7}{12} = 7.22$

$K = 236 \quad F = 2.53$

$NE = 107.39 \times 7.22 = 776$

$KF = 236 \times 2.53 = 597$

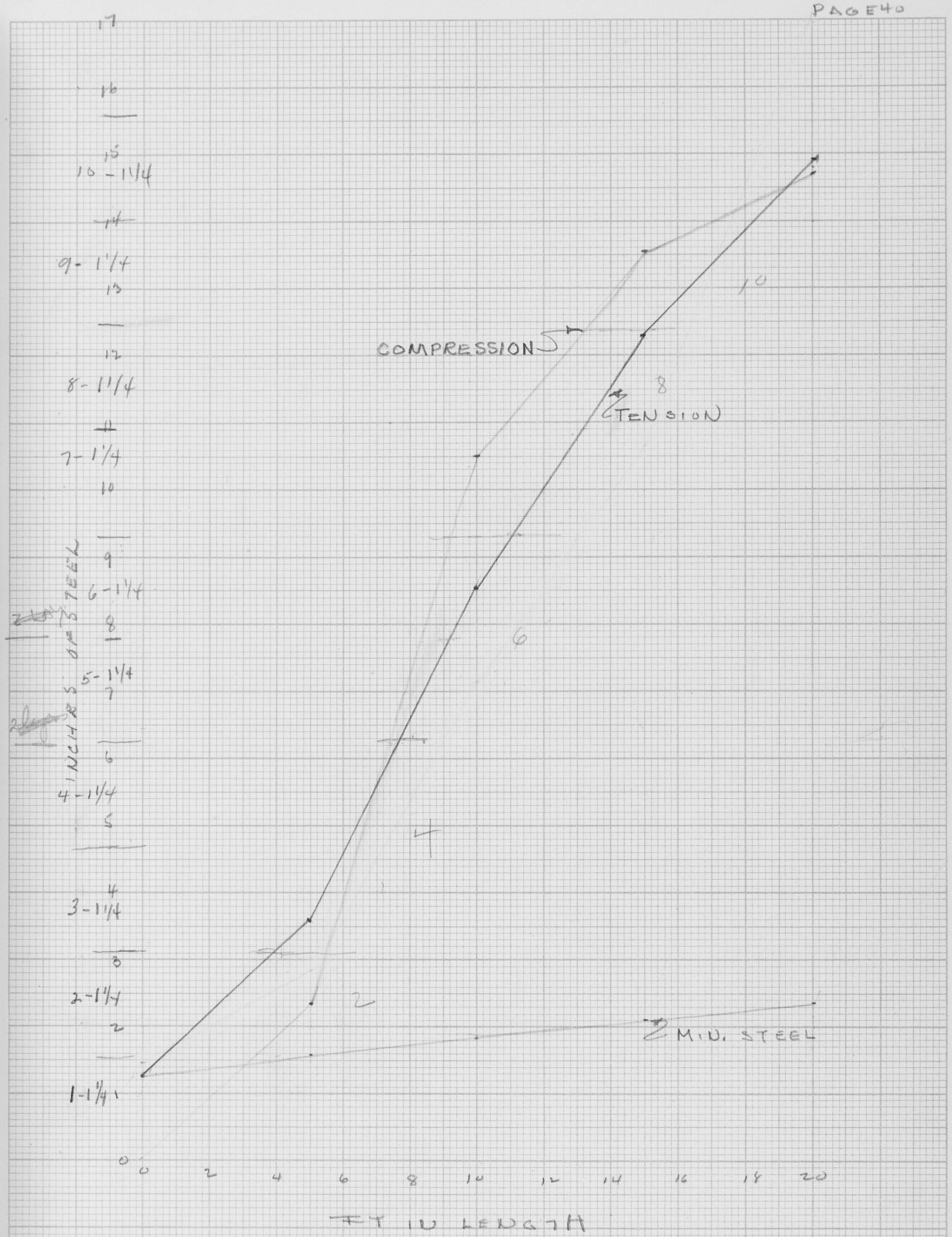
179 COMP STEEL REQ'D

$\frac{d'}{d} = \frac{2}{40} = .05 \quad c = .84$

$A'_s = \frac{NE - KF}{cd} = \frac{179}{.84 \times 40} = 5.33$

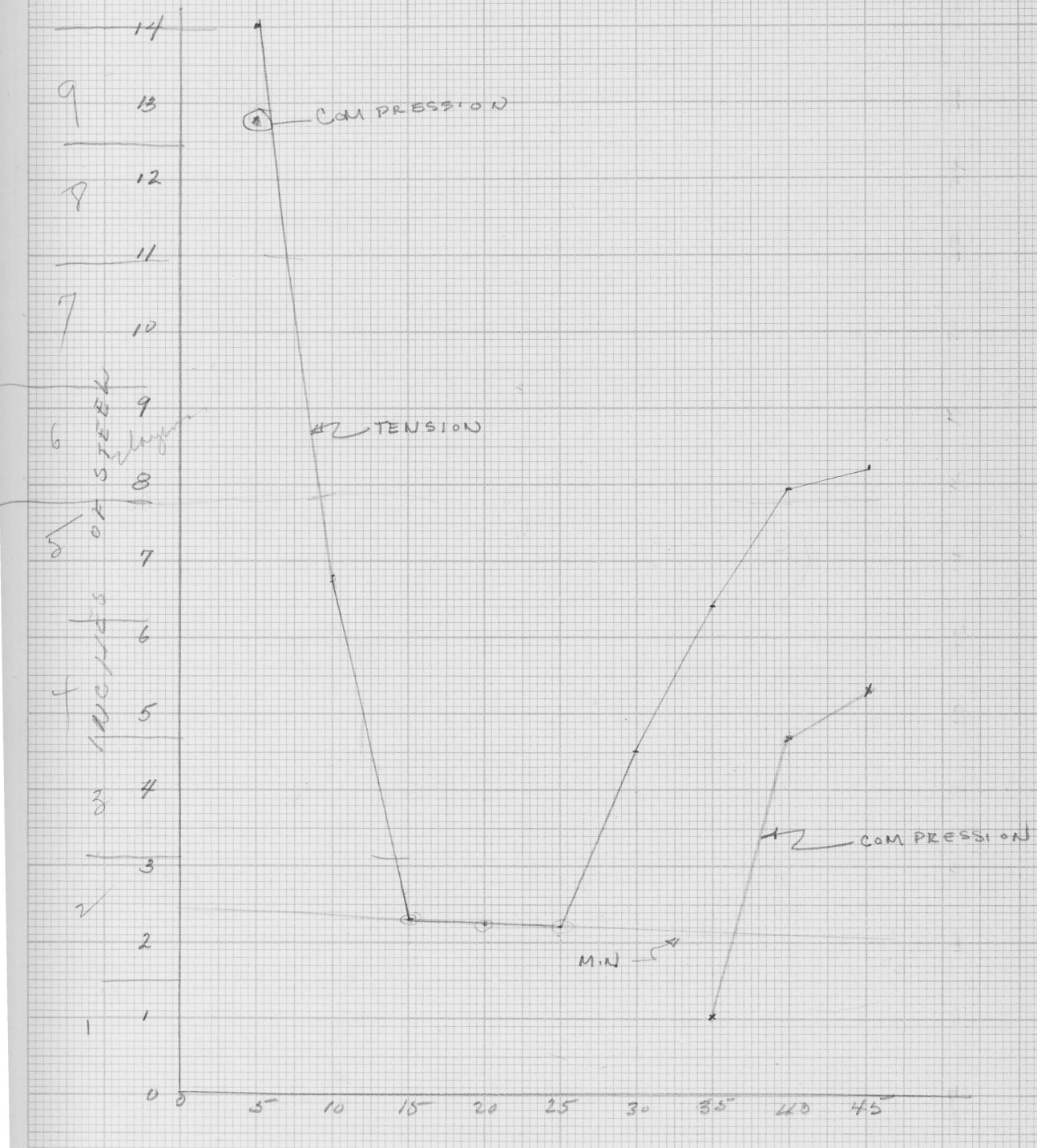
$\frac{e}{d} = \frac{86.7}{40} = 2.17 \qquad a = 1.44 \quad i = 1.64$

$A_s = \frac{NE}{adi} = \frac{776}{1.44 \times 40 \times 1.64} = 8.22$



STEEL REQ'D COLUMN

EVERETT WADDEY CO. RICHMOND, VA



BEAM STEEL NEED

EVERETT WADDEY CO. RICHMOND, VA.

DESIGN OF Footing FOR Two hinged CONCRETE :-

TIE ROD TO TAKE shear at base of column

Shear = $\frac{63.0}{20}$ = 3.15 5 - 1 1/8 □

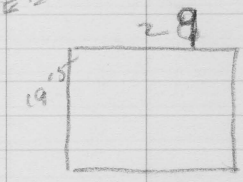
DESIGN OF Footing

~~116 - 15 = 130~~

88.6 + 27.1 = 115.7 D + L.L

98.6 + 3.5 = 102.1 D + 20% C.L

PEDISTAL



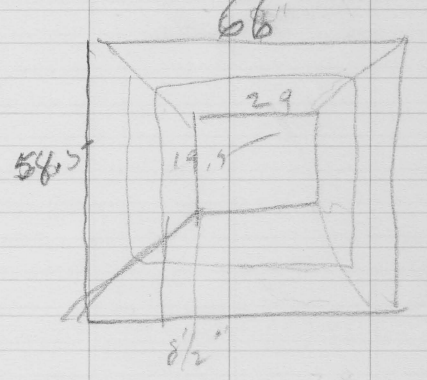
4 - 1 1/4 □

1/4 □ TIE - 12" o.c

AREA REQ'D = 102.1 ÷ 3.85 = 2.66

P_R = 115.7 ÷ 13 = 8.9

Shortway



$M = 8.9 \times 5.5 \times \frac{1.5^2}{2} = 55$

$V_b = 8.9 \times 5.5 \times 1.5 = 73.3$

$V_v = 8.9 \times \frac{5.5 + 3.83}{2} \times .79 = 32.8$

69 59.0
29 19.5
40 40.5

LONG WAY

$M = 8.9 \times 4.87 \times \frac{1.5^2}{2} = 48.8$

$V_b = 8.9 \times 4.87 \times 1.5 = 65.2$

7.91

$V_v = 8.9 \times \frac{4.87 + 3.04}{2} \times .79 = 27.75$

63.

$M_{all} = 17.5 \times 4.87 = 85.4 > 55$ o.k

$V = \frac{36.8}{66 \times 7/8 \times 8 1/2} = 7.57$

$\frac{27.7}{58.5 \times 7/8 \times 8 1/2} = 63.3$

MAKE Footing 15" DEEP.

SHORT WAY

55 x .693 ÷ 11½ = 3.3
ε_u = 73.3 ÷ (.168 x 7/8 x 11½) = 43.7

LONGWAY
A_s = 48.8 x .693 ÷ 11½ = 2.95
ε_u = 65.2 ÷ (.168 x 7/8 x 11½) = 38.9

FOOTING 15" DEEP 5'-5" x 4'-16½"

THREE HINGED STEEL FRAME

SIZE:

FLANGE: UNIFORM 16x1

WEB 3/8" THICK

COLUMN VARIES IN DEPTH FROM 10" TO 31"

BEAM " " " " 10" TO 32½"

GUESS FOR 3 HINGED STEEL FRAME.

DEAD weight assumned TO BE 15% LESS THAN CONCRETE FRAME

wt OF CONCRETE = .864 K/LNFT OF BEAM
LESS 15% OF 846 .130
.734 K/LNFT OF BEAM
wt OF ROOF ETC CARRIED .784 K/LN FT
1.514

VERTICAL REACTION
1.514 K/LN FT x 45.7 69.2 K
21.6 x .15 = -3.2 + 21.6 (COLUMN) 18.4 K
87.6 K VERTICAL REACTION

$\Sigma M_{\mathbb{C}}$ (HALF FRAME)

$$69.2(45) - \frac{1.514 \times 45.7^2}{2} - 33 H_L = 0$$

$$2810 - 1580 = 33 H_L$$

$$H_L = \frac{1230}{33} = 37.3$$

$$H_L = H_R$$

AT TOP OF COLUMN
MOMENT = 860 FT K OR - 10,520,000 IN #.
THRUST = 69.2 K - 69,200 #
SHEAR = 37.3 K - 37300 #

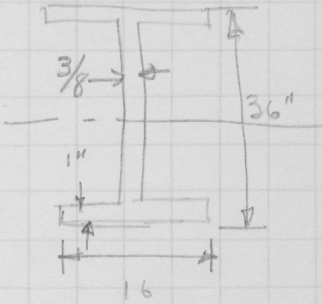
SHEAR THROUGHOUT COLUMN = 37300 #
CONTROLS web

web = $\frac{\text{shear}}{\text{area}}$ = allowable

$$\frac{37,300}{A} = 13,000$$

$$A = 3 \text{ sq}''$$

MAKE web at BASE = 3/8" x 8"



check FOR MOMENT

$f = \frac{Mc}{I} = \frac{10,520,000 \times 18}{10588.3} = 17,883.89$

$\frac{ld}{bt} \quad \frac{22 \times 12 \times 26}{16 \times 1} = 429$

allowable = 20,000

check FOR COMP.

$f = \frac{P}{A} = \frac{69,200}{44.5} = 1,555.2$

$\frac{l}{r} = \frac{22 \times 12}{15.42} = 17.12$

allowable COMP = 16,860

$I = 34 \times 3/8 - 1228.3$

$= 685 \times 16 - 9360$

10588.3

$A = 3/8 \times 34 = 12.5 + 32 = 44.5$

$I = ar^2$

$10588.3 = 44.5\, r^2$

$r^2 = 237.9393$

$r = 15.42$

$\frac{f_a}{F_b} + \frac{f_b}{F_b} \leq 1$

$\frac{1,555}{16,860} + \frac{17,883}{20,000} =$

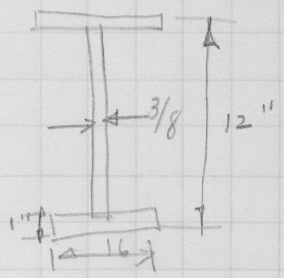
$.092 + .894 = .986$

check FOR COMPRESSION AT BASE

$f = \frac{P}{a} = \frac{87,600}{36.5} = 2,400$

$\frac{l}{r} = \frac{22 \times 12}{4.8} = 55$

$C_{allowing} = 15,530$



$A = \frac{3}{16} \times \frac{3}{8} = 4.5$

$= 32.$

36.5

$I = 31.3$

$50 \times 16 \quad 800.$

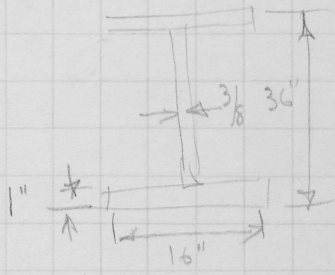
831.3

$I = ar^2$

$831.3 = 36.5\, r^2 \quad r = 4.8$

Approximate DESIGN OF BEAM PORTION

MOMENT = 812 FTK. = 9,744,000 IN #
THRUST = 37.3 K = 37,300 #
SHEAR = 69.2 K = 69,200 #



check shear

$$13{,}000 = \frac{69{,}200}{3/8\ x} =$$

$$x = 14.24875$$

check FOR MOMENT

$$f = \frac{MC}{I} = \frac{9{,}744{,}000 \times 18}{10588.3} = 16{,}565$$

I = 10588.3
see previous page

$$\frac{ld}{bt} = \frac{33 \times 12 \times 26}{1 \times 16} = 643.5$$

Allowable = 18,648 o.k.

check FOR COMPRESSION

$$f = \frac{P}{A} = \frac{37{,}300}{36.5} = 1{,}022$$

$$\frac{l}{r} = \frac{90 \times 12}{4.8} = 22.5$$

Allowable = 16,750

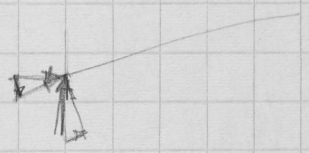
$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$$

$$\frac{1{,}022}{16{,}750} + \frac{16{,}565}{18{,}648}$$

$$.061 + .888 = .949$$

DESIGN OF 3 HINGED STEEL FRAME

DEAD LOAD CARRIED 780 #/LN FT.



Column weight (each)

plate 25 x 2 x 54.4 = 2720 #
web 55 x 2 x 12.8 = 1408
41 28 #

1/2 BEAM weight.

plate 45.7 x 2 x 54.4 = 4972
web 100.5 x 2 x 12.8 = 2573
7545 #

Approx 165.1 #/LN.FT.
780
945

TOTAL VERTICAL REACTION

Column	4128#
BEAM	7545
IMPOSED 780 x 45.7	35,646
	47,319 #

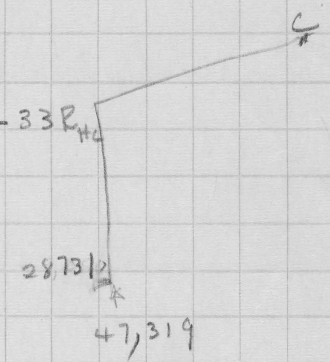
FIND HORIZONAL REACTION

FRAME ΣM_c = 43,191 (45) - 45.9 x 472.5 x 22.5 - 33 R_{HL}

33 R_{HL} = 1,943,595 - 995,468

33 R_{HL} = 948,127

R_{HL} 28,731



MOMENT		Shear	THRUST
BASE	= 0	28,731	(47,319)
5' FROM BASE	= 143,655	28,731	46,519
10' " "	= 287,310	28,731	45,719
15' " "	= 430,965	28,731	44,877
20'	= 574,620	28,731	44,034
22' " "	= 632,082	28,731	43,191

BEAM PORTION (DEAD ONLY

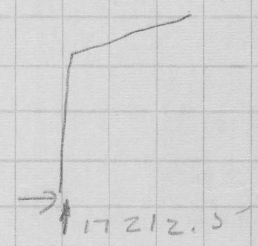
	MOMENT	SHEAR	THRUST
END	- 718,275	42,551 #	29,728
3'	- 607,361	40,662 #	29,685
5'	- 540,463	37,827 #	29,678
10'	- 387,940	33,103 #	29,508
15'	- 258,404	28,379 #	29,398
20'	- 154,446	23,655 #	29,288
25	- 74,672	18,931 #	29,178
30	- 18,793	14,207 #	29,068
35	+ 10,782	9,483 #	28,958
40	+ 12,063	4,759 #	28,848
℄	- 0 -	22 #	27,738

LIVE LOAD

Allowable - 25 #/sq ft

VERTICAL REACTION

$25 \times 15 \times 45.9 = 17212.5$ #



HORIZONTAL REACTION

$\Sigma M_C = 45(17212.5) - \frac{375 \times 45.9^2}{2} - 33 R_{HL}$

$33 R_{HL} = 774,562.5 - 387,281.25$

$R_{HL} = \frac{387,281.25}{33}$

$R_{HL} = 11,735.8$

$R_{HL} = R_{HR}$

LIVE LOAD ONLY COLUMN

POSITION -	MOMENT	SHEAR	THRUST
BASE	- 0 -	11,736	17212.5
5'	58,679	11,736	17212.5
10'	117,358	11,736	17212.5
15'	176,037	11,736	17212.5
20'	234,716	11,736	17212.5
22'	258,187	11,736	17212.5

LIVE LOAD ONLY

BEAM PORTION	MOMENT	SHEAR	THRUST
END	- 293,395	16,935	12,133
3'	- 249,267	16,185	12,107
5'	- 222,769	15,061	12,089
10'	- 161,894	13,187	12,045
15'	- 110,711	11,313	12,041
20'	- 69,393	9,439	11,997
25'	- 37,671	7,565	11,953
30'	- 15,433	5,691	11,909
35'	- 3,632	3,817	11,865
40'	779	1943	11,821
℄	- 0 -	- 267	11,769

TOTAL LOAD COLUMN

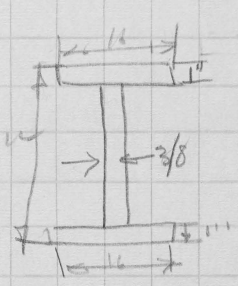
POSITION	MOMENTS	SHEAR	THRUST
BASE	- 0 -	40467	64,532
5' FROM BASE	202,334	40467	63,732
10' " "	404,668	40467	62,932
15' " "	607,002	40467	62,090
20' " "	809,336	40467	61,247
22 " "	890,269	40467	60,404
	1,200,000		
	310,207		
		2,000	
		÷ 81,000	

BEAM END			
END	- 1,011,670 IN #	59,486	41,861
3'	- 856,628	56,847	41,792
5	- 763,232	52,888	41,707
10	- 549,834	46,290	41,553
15	- 396,115	39,692	41,439
20	- 223,839	33,094	41,285
25	- 112,343	26,496	41,131
30	- 34,216	19,898	40,977
35	+ 17,150	13,300	40,823
40	+ 11,283	6,702	40,660
℄	- 0 -	245	39,507

Check For Column Design

L = 25'

MEAN D = $\frac{2}{3} \times 26 = 18 + 10 = 28$



BASE

Shear = 40467 #

THRUST 64,532 #

Allowable = 13,000 # SHEAR

AREA 3/8 X 10 = 3.75 web

32

35.75

$\frac{\text{shear}}{\text{area of web}} = \frac{40,467}{3.75} = 10,791$ LESS THAN 13,000

Check For Compression

Allowable = 15,750

$f = \frac{P}{A} = \frac{64,532}{35.75} = 1,805$ O.K

$l = 25 \times 12$

$I = 31.3 + 976 = 1007.3$

$A = 35.75$

No BENDING.

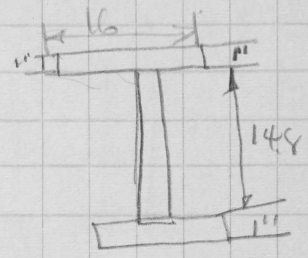
$I = ar^2$

$1007.3 = 35.75 r^2$

$r^2 = 28.18$

$r = 5.31$

$\frac{l}{r} = \frac{300}{5.31} = 56.5$



Column 5' FROM BASE:

M = 202,334 #FT.

V = 40467 #

N = 63,732 #

No FURTHER check ran for shear as BASE SMALLEST SECTION & shear UNIFORM THROUGHOUT COLUMN.

Check FOR COMP.

Allowable = 15,480

$f = \frac{P}{A} = \frac{63,732}{37.55} = 1,697$

$l = 300$

$A = 32,$ PLATES

5.55 web

37.55

$I = 125(16) = 2000.0$

101.6

2101.6

Check FOR BENDING. 16.8

allowable = 20,000

$f = \frac{Mc}{I} = \frac{202,334 \times 8.4 \times 12}{2101.6} = 9704.6$

$I = ar^2$

$2101.2 = 37.55 r^2$

$r^2 = 55.96$

$f = \frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$

$\frac{1,679}{15,440} + \frac{9704.6}{20,000} =$

$.108 + .485 = .593$

$\frac{hd}{bt} = \frac{300 \times 16.8}{16 \times 1}$

$= 315$

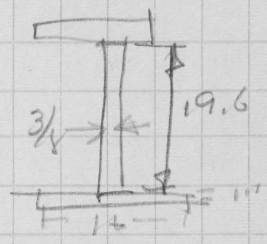
less than 600

$\therefore$ allowable = 20,000

COLUMN 10' FROM BASE:

$M = 404{,}668 \times 12 = 4{,}856{,}256$

$N = 62{,}932$



check FOR COMP.

allowable = 16,530

$f = \frac{P}{A} = \frac{62{,}932}{39.35} = 1{,}599$

$l = 300$

$A = 32.00$
$\quad 7.35$
$\quad 39.35$

check for BENDING.

allowable = 20,000

$f = \frac{MC}{I} = \frac{4{,}856{,}256 \times 9.8}{3627.2}$

$= 13{,}121.$

$I = 2357$
$\quad 212(16)3392$
$\quad 3627.7$

COMBINED

$\frac{f_a}{F_A} + \frac{f_B}{F_B} \leq$

$\frac{1{,}599}{16{,}530} + \frac{13{,}121}{20{,}000} =$

$.097 + .656 = .756$

$I = ar^2$

$3627.7 = 39.35 r^2$

$r^2 = 92.19$

$r = 9.6$

$\frac{l}{r} = \frac{300}{9.6} = 31.25$

∴ allowable 16,530

$\frac{Ld}{bt} = \frac{300 \times 21.6}{16 \times 1}$

$= 405 < 600$

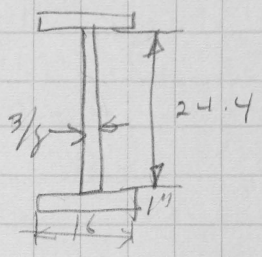
∴ allowable = 20,000

$\frac{10.8}{21.6}$

COLUMN 15' FROM BASE

$M = 607{,}002 \times 12 = 7{,}284{,}024$

$N = 62{,}090$



check FOR COMP.

allowable = 16,680

$f = \frac{P}{A} = \frac{62090}{41.15} = 1{,}509$

$l = 300$ in.

$A = 9.15$

32

41.15

check FOR BENDING

allowable = 20,000

$f = \frac{Mc}{I} = \frac{7{,}284{,}024 \times 13.2}{5622.5}$

$= 17{,}101$

$I = 454.5$

$323(16)\ 5168$

5622.5

$I = ar^2$

$r^2 = \frac{5622.5}{41.15}$

$r^2 = 136.63$

$r = 11.7$

$\frac{l}{r} = \frac{300}{11.7}\ 25.64$

∴ allowable 16,680

$\frac{Ld}{bt} = \frac{300 \times 26.4}{16 \times 1}$

$= 495 < 600$

allowable = 20,000

COMBINED

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$$

$$\frac{1509}{16{,}680} + \frac{17{,}101}{20{,}000}$$

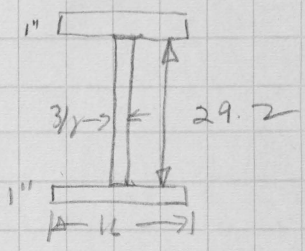
$$.090 + .855 = .945$$

13.2

COLUMN 20' FROM BASE

M = 809,336 X 12 = 9,712,032

N = 61,247



check FOR COMP

allowable 16,770

$f = \frac{P}{A} = \frac{61.247}{42.95} = 1,438$

$l = 300$ in.

A = 10.95
32
42.95

check FOR BENDING

allowable = 20,000

$f = \frac{Mc}{I} = \frac{9,712,032 \times 15.6}{8074.6}$

= 18,763

I = 7786
16(456) 7296
8074.6

COMBINED

$\frac{f_a}{F_A} + \frac{f_B}{F_B} \leq 1$

$\frac{1,438}{16,770} + \frac{18,763}{20,000} =$

.086 + .938 = 1.024

within 5% - OK

$r^2 = \frac{I}{A}$

$r^2 = \frac{8074.6}{42.95}$

$r^2 = 188$

$r = 13.71$

$\frac{l}{r} = \frac{300}{13.72}$

$\frac{l}{r} = 21.88$

allowable = 16,770

$\frac{hd}{bt} = \frac{300 \times 31.2}{16 \times 1}$

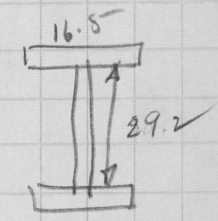
= 585 < 600

allowable = 20,000

COLUMN 20' ABOVE BASE

M = 809,336 x 12 = 9,712,032

N = 61,247



l = 300 in.

A = 10.92
33
43.92

check FOR COMP.

allowable 16,770

$f = \frac{P}{A} = \frac{61,247}{43.92} = 1,395$

check FOR BENDING

allowable 20,000

$f = \frac{Mc}{I} = \frac{9,712,032 \times 15.6}{8302.6}$

$= 18,248$

I = 778.6
16.5(45)7524.0
8302.6

COMBINED

$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$

$\frac{1,395}{16,770} + \frac{18,248}{20,000}$

.083 + .912 = .995

$r^2 = \frac{I}{A}$

$r^2 = \frac{8302.6}{43.92}$

$r^2 = 189.$

$r = 13.7$

$\frac{l}{r} = \frac{300}{13.7}$

21.89

allowable = 16,770

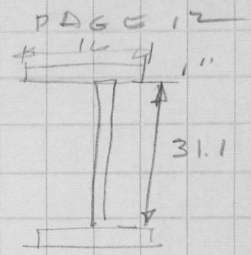
THIS EXTRA CHECK WAS RUN ALTHOUGH NOT NEEDED HERE

31.2

$\frac{hd}{bt} = \frac{300 \times 32.2}{16.5 \times 1}$

= 579

allowable 20,000



MOMENT 890,269 x 12
THRUST 60,404

CHECK FOR COMP.
allowable 16,790
$f = \frac{P}{A} = \frac{890,269}{43.66}$
$= 2,039$

CHECK FOR BENDING.
allowable - 19,324
$f = \frac{MC}{I} = \frac{890,269 \times 16 \times 12}{9123}$
$= \cancel{19322}\ 18736$

$\frac{2,039}{16,790} + \frac{19322}{19324} =$

$.121 + .999 = 1.090$

TOO SMALL
TRY 16 1/2" FLANGE.

$l = 300$

$A = 11.66$
32.0
43.66

I 931.0
$512(16)^3 = 8192$
$9123.$

$r^2 = \frac{I}{A}$
$r^2 = \frac{9123}{43.66}$
$r^2 = 208.95$
$r = 14.45$
$\frac{l}{r} = \frac{300}{14.45} =$

20.76
allowable 16,790

$\frac{ld}{bt} = \frac{300 \times 33.1}{16 \times 1}$

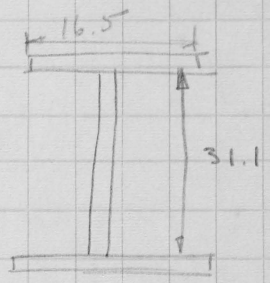
$= 62$

allowable
19,322

22' FROM BOTTOM

M = 896,269 x 12 = 10,758,228

THRUST 60,404



$l = 300$

check FOR COMP.

allowable = 16,790

$f = \frac{P}{A} = \frac{60,404}{44.66}$

$f = 1,353$

A = 11.66
33.00
44.66

check FOR BENDING

allowable 19,967

$f = \frac{Mc}{I} = \frac{896,269 \times 12 \times 16.5}{9379}$

$= 18,618$

I = 931.0
512(16.5)448
9379.0

COMBINED

$\frac{f_a}{F_A} + \frac{f_b}{F_B} = \leq 1$

$\frac{1,353}{16,790} + \frac{18,618}{19,967}$

$.081 + .932 = 1.013$

ok

$r^2 = \frac{I}{A} = \frac{9379}{44.66}$

$r^2 = 210$

$r = 14.5$

$\frac{l}{r} = \frac{300}{14.5} = 20.69$

All. 16,790

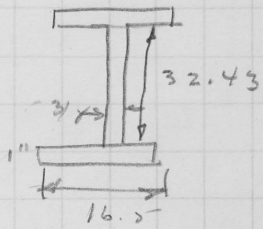
$\frac{ld}{bt} = \frac{300 \times 33.1}{16.5 \times 1}$

$= 602$

allowable
19,967

BEAM DESIGN 3' FROM OUTER EDGE

Note: STIFTNER PLACED 17' FROM OUTER EDGE:



M = 856,528 x 12 = 10,279,536

N = 41,792

check FOR COMP

ALLowable 14,420

$f = \frac{P}{A} = \frac{41,792}{45.12} = 926$

A = 33.0
12.12
45.12

check FOR BENDING.

ALLowable 20,000

$f = \frac{Mc}{I} = \frac{10,279,536 \times 17.2}{9731.5} = 18,168$

I = 525(16.5) 8662.5
1069
9731.5

$r^2 = \frac{I}{A} = \frac{9731.5}{45.12}$

$r^2 = 216$

$r = 14.7$

$\frac{l}{r} = \frac{90 \times 12}{14.7} = 73.5$

allowable = 14,420

COMBINED

$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$

$\frac{926}{14,420} + \frac{18,168}{20,000} =$

.064 + .908 = .972 o.K.

L for bending

17 x 12 = 204

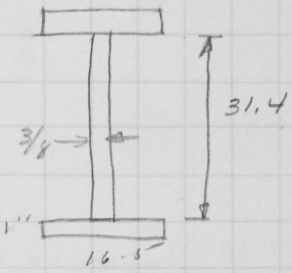
$\frac{Ld}{bt} = \frac{204 \times 34.4}{16.5 \times 1}$

425

allowable

20,000

BEAM DESIGN 5' FROM OUTER EDGE.



M = 763,232 x 12 =
N = 41,707

check FOR COMPRESSION
Allowable:
$f = \frac{P}{A} = \frac{41,707}{43.73}$

A = 11.77
33
43.73

check FOR MOMENT
allowable.
$f = \frac{MC}{I} = \frac{\quad \times 16.7}{95^{-}93.5^{-}}$ 334

I = 931
I (16.5)(5^-25) 8662.5^-
9593.5

$r^2 = \frac{I}{A} = \frac{95^{-}93.5^{-}}{43.73}$
$r^2 =$
$r =$

Check FOR COMBINED

$\frac{f_a}{F_A} + \frac{f_B}{F_B} \leq 1$

$\frac{\quad}{\quad} + \frac{\quad}{\quad} =$

$\frac{l}{r} = \frac{90 \times 12}{}$

allowable =

$+ \quad =$

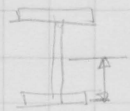
$\frac{ld}{bt} = \frac{17 \times 12 \times 33.4}{16.5^{-}}$

=

allowable.

COLUMN: DESIGN OF WELDS.

$\frac{VQ}{I} =$



BASE.

$\frac{40467 \times 5.5 \times 16.5}{1037.8} = 3{,}538$

$I = 61(16.5)1006.5$
31.3
1037.8

5' BASE

$\frac{40467 \times 7.9 \times 16.5}{2164.1}$ 2,437

$I = 125(16.5)2062.5$
101.6
2164.1

10' FROM BASE

$\frac{40467 \times 10.3 \times 16.5}{3733.7}$ 1,842

$I = 212(16.5) = 3498$
235.7
3733.7

15' FROM BASE

$\frac{40467 \times 12.7 \times 16.5}{5784.0} = 1{,}466$

$I = 323(16.5)\ 5329.5$
454.5
5784.0

20' FROM BASE

$\frac{40467 \times 15.6 \times 16.5}{8302.6} = 1{,}255$

$I = 8302.6$
See pg. 11a

22' FROM BASE

$\frac{40467 \times 16.6 \times 16.5}{9379} = 1{,}182$

$I = 9379$
see pg. 11a.

BEAM DESIGN OF Welds. -

$\frac{VQ}{I}$

3' FROM OUTER EDGE

$\frac{56,847 \times 16.5 \times 16.7}{9731.5} = 1,610$

$I = 9731.5$

5' FROM OUTER EDGE.

$\frac{52,488 \times 16.5 \times 16.2}{9593.5} = 1,474$

$I = 16.5(525)\ 8662.5$
$931.$
9593.5

10' FROM OUTER EDGE

$\frac{46,290 \times 16.5 \times 14.9}{8082.2} = 1,408$

$I = 16.5(444)\ 7320$
762.2
8082.2

15' FROM OUTER EDGE

$\frac{39,692 \times 16.5 \times 13.6}{6604.8} = 1,349$

$I = 16.5(367)\ 6055.5$
549.3
6604.8

20' FROM OUTER EDGE

$\frac{33,096 \times 16.5 \times 12.3}{5382} = 1,248$

$I = 16.5(300)\ 4950$
432
5382

25' FROM OUTER EDGE

$\frac{26,496 \times 16.5 \times 11}{4292.8} = 1,120$

$I = 16.5(240)\ 3960.0$
332.8
4292.8

30' FROM OUTER EDGE

$\frac{19,848 \times 16.5 \times 9.7}{3319.3} = 962$

$I = 16.5(180)\ 2986.5$
332.8
3319.3

35' FROM outer edge

$$\frac{13{,}300 \times 16.5 \times 8.3}{2421.5} = 752$$

$I = 16.5(139)$ 2293.5
128.0
2421.5

40' FROM outer edge

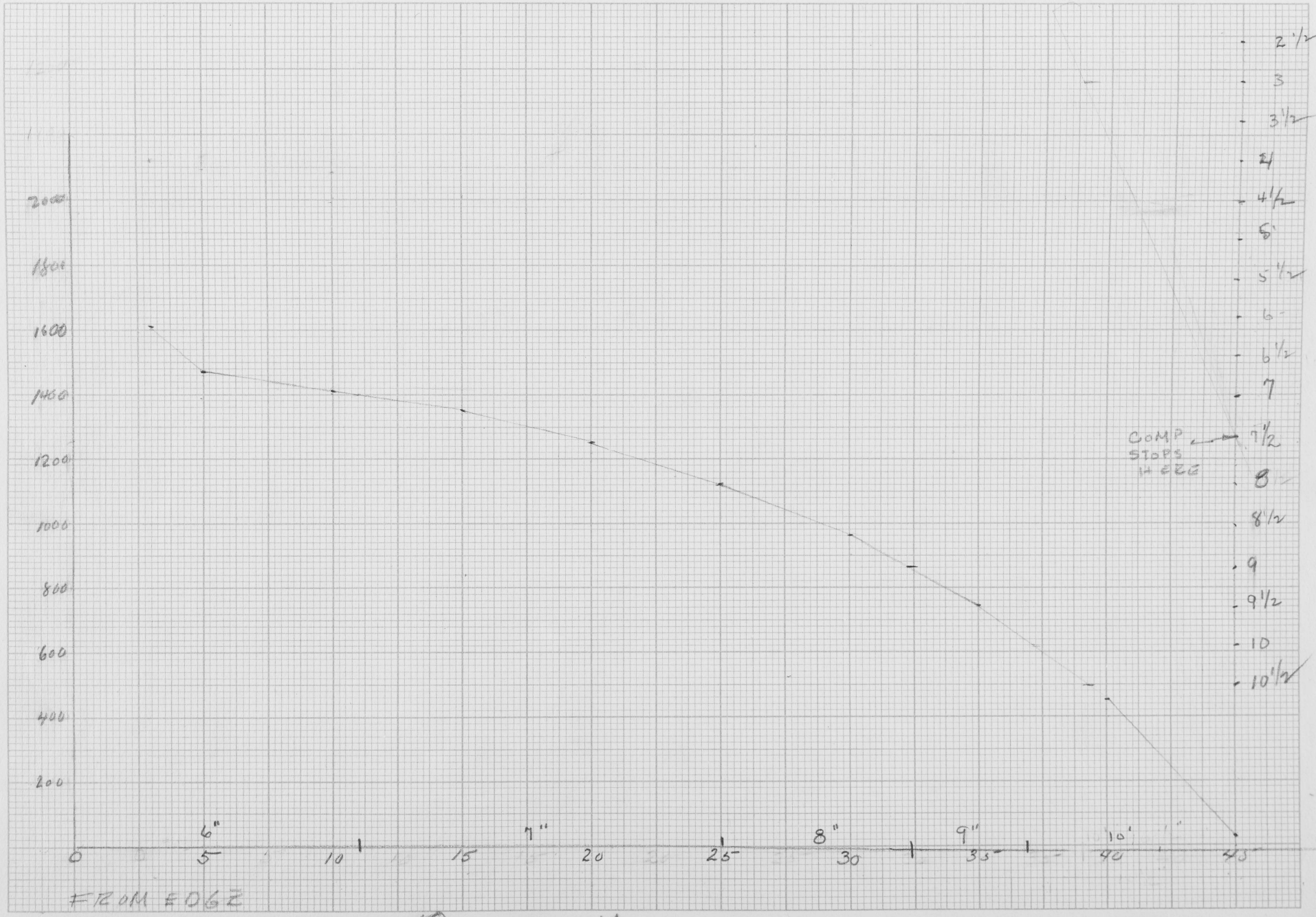
$$\frac{6{,}702 \times 16.5 \times 7}{1702.8} = 455$$

$I = 98(16.5) =$ 1617.0
85.8
1702.8

@

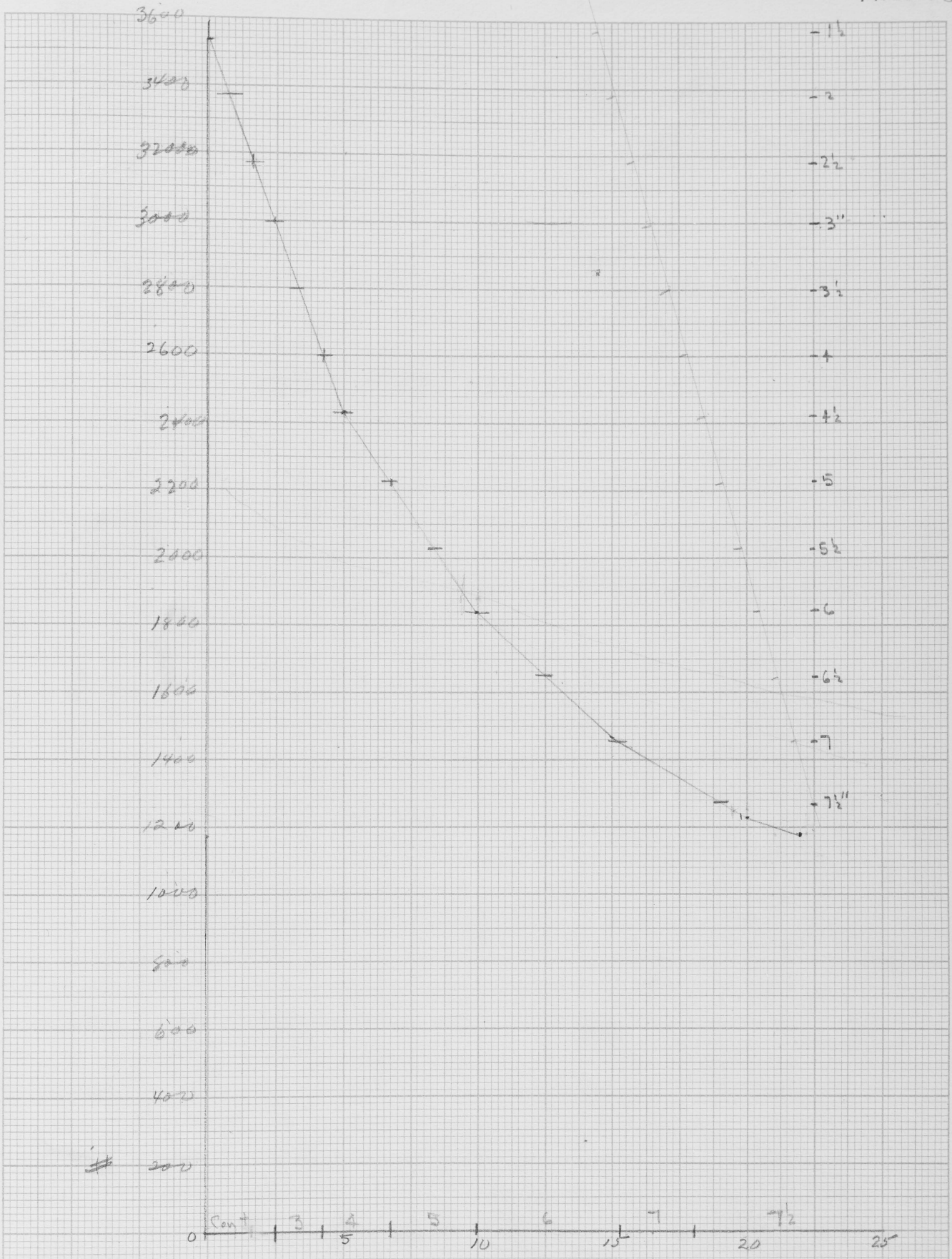
$$\frac{245 \times 16.5 \times 5.5}{1037.8} = 21$$

$I = 6.5(16.5) =$ 1006.5
31.3
1037.8



RQ BEAM

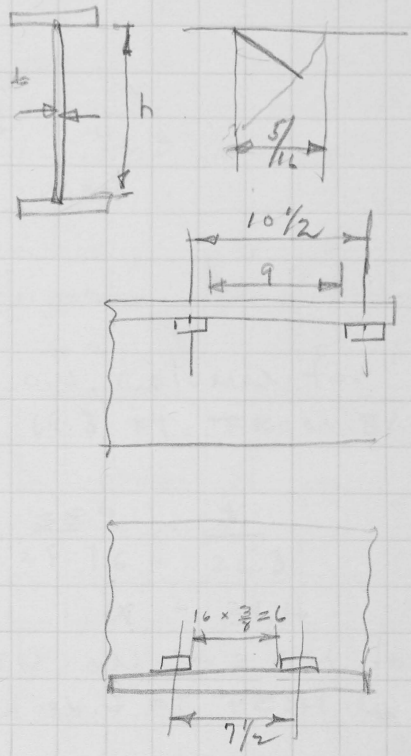
LEGAL PAGE 17



COLUMN

$\sim = \frac{VQ}{I}$ DIAGRAM

EVERETT WADDEY CO. RICHMOND, VA.



$F = 13.6 \times L \times 5/16 \times .707$

$F = 3.0\ L$

$= 3.0 \times 1\frac{1}{2} = 4.5$

BECAUSE 2 SIDE $= 9.0$

for $U = 3.54$

$S = \frac{9.0}{3.54} = 2.83''$ o.c.

for $V = 1.18$

$S = \frac{9.0}{1.18} = 7.6''$ o.c.

for $S = 3''$

$V = \frac{9.0}{3.0} = 3.0$

for $S = 7\frac{1}{2}$

$V = \frac{9.0}{7.5} = 1.265$

---

for $S = 4.5$

$U = \frac{9.0}{4.5} = 2.0$

for $S = 7.5$

$V = \frac{9.0}{7.5} = 1.2$

$16 \times \frac{3}{8} = 6$

STIFTNERS:-

$\frac{h}{t} \geq 70$ if: $\frac{25}{34} = \frac{x}{26.3}$

$\frac{h}{3/8} = 70$ l. 19.34

$h = 26.3$

∴ STIFTNERS ARE NEEDED ON COLUMN for above CONDITION FROM 19.3 FT FROM BASE ON.

if $\frac{35.9}{28.76} = \frac{x}{26.3}$

$x = 34.4$

∴ STIFTNERS ARE NEEDEN ON BEAM for above CONDITION FROM 34.4 FT. FROM ℄.

$\frac{64,000,000}{(70)^2} = 13,061$

COLUMN

@ 19.3 $v = 1,250$

@ 20 -

$\frac{h}{t} = \frac{29.2}{3/8} = 77.87$

$\frac{64,000,000}{(77.87)^2} = 10,555$

@ 22

$\frac{31.1}{3/8} = 82.93$

$\frac{64,000,000}{(82.93)^2} = 9,306$

BEAM

@ 3' FROM EDGE

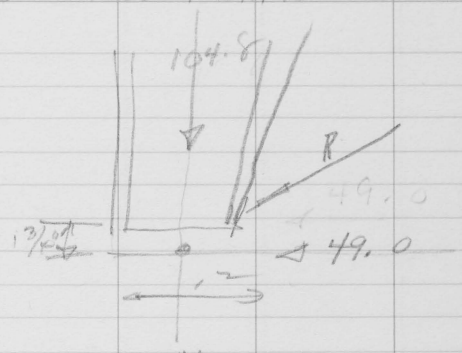
$\frac{h}{t}$ $\frac{32.43}{3/8} = 86.48$

$\frac{64,000}{(86.48)^2} = 8,557$

No stiffners needed

DESIGN OF foot CONNECTION 3 HINGED STEEL FRAME

$R = \sqrt{104.8^2 + 49.0^2} = 115.2$



PIN

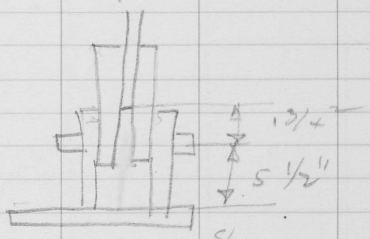
$115.2 \div 15 = 7.68$

3 1/4 ϕ PIN

HINGE PLATES

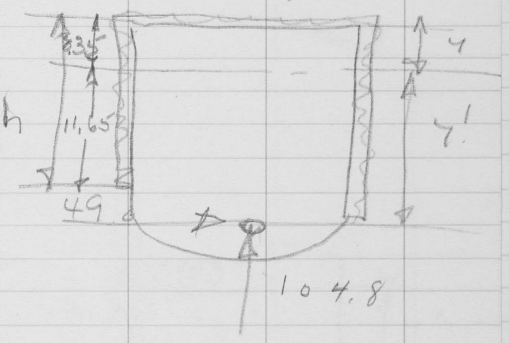
$t = \frac{115.2}{40.0 \times 3\frac{1}{4}} = .885$

USE 2 - 1/2" bottom plates
2 - 1/2" Top Plates.



Assume h

LENGTH req'd = 104.8 ÷ 2.30 = 46'
Approx double 92"
16
76 ÷ 4 = 19'
say 20' = h



$c.g. = \frac{8 \times 0 + 20 \times 2 \times 10}{8 + (20 \times 2)} = 8.34'' = y$

$\therefore y' = 20 + 1.75 - 8.35 = 13.40$

$M = 49.0 \times 13.40 = 655$ INK.

USE 1/4 fillet
$t = 13.0 \times .25 \times .707$
$= 2.30$

$J = 1 \times 2 \times \left(\sqrt{4^2 + 11.15^2}\right)^2 = 280$
$= 1 \times 2 \times \left(\sqrt{4^2 + 10.15^2}\right)^2 = 238$
$= 1 \times 2 \times \left(\sqrt{4^2 + 9.15^2}\right)^2 = 200$
$1 \times 4 \times \left(\sqrt{4^2 + 8.00^2}\right)^2 = 320$
$1 \times 4 \times \left(\sqrt{4^2 + 7.00^2}\right)^2 = 260$
$1 \times 4 \left(\sqrt{4^2 + 6.00^2}\right)^2 = 208$
$1 \times 4 \left(\sqrt{4^2 + 5.00^2}\right)^2 = 164$
$1 \times 4 \left(\sqrt{4^2 + 4.00^2}\right)^2 = 128$
$1 \times 4 \left(\sqrt{4^2 + 3.00^2}\right)^2 = 100$
$1 \times 4 \left(\sqrt{4^2 + 2.00^2}\right)^2 = 80$
$1 \times 4 \left(\sqrt{4^2 + 1.00^2}\right)^2 = 68$
$1 \times 2 \left(\sqrt{3.5^2 + 8.35^2}\right)^2 = 164$
$1 \times 2 \left(\sqrt{2.5^2 + 8.35^2}\right)^2 = 152$
$1 \times 2 \left(\sqrt{1.5^2 + 8.35^2}\right)^2 = 144$
$1 \times 2 \left(\sqrt{.5^2 + 8.35^2}\right)^2 = 140$
2646

$f_M = \frac{655 \times \sqrt{4^2 + 11.15^2}}{5292} = 1.47$

$f_N = 104.8 \div 96 = \frac{109}{2.56}$

MAKE h = 24'

DESIGN OF CENTER HINGE

NO MOMENT

Shear = 245 #

THRUST 39,507 #

$R = \sqrt{39.5^2 + .2^2} = 39.6$

PIN

$A = 39.6 \div 15 = 2.64$

use 2" dia pin

Hinge PLATES

$t = \frac{39.6}{40.0 \times 2} = .495$

USE 2 - 1/2" PLATES

5/16 = 4/16 3/16

Assume h

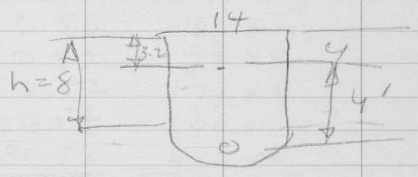
LENGTH REQ'D = 39.5 ÷ 2.30 = 17.2

say about 22

14

8 ÷ 4 = 2"

MAKE 8"



$cg = \frac{(14 \times 0) + (8 \times 2 \times 4)}{14 + (8 \times 2)}$

$= \frac{64}{20}$ 3.2

$y' = 8 + 1.5 - 3.2 = 6.3$

$M = .245 \times 6.3$ 1.54 INK

NEG.

$f_N = \frac{39.5}{80 \times 2} = .66$

MAKE PLATES 1/4 X 14 X 12

DESIGN OF SPLICE AT CORNER
MOMENT = 12,150 IN.K.
V = 59.5 K

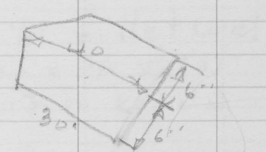
web splice

I = 1275(16.5) = 21100
1300
22400

$f = \frac{12,50 \times 26}{22400} = 14.1$

$M = \frac{14.1 \times 1300}{26} = 706$ IN K.

LENGTH splice plate = 40" - 30"



I REQ'd $\frac{706 \times 20}{20.0} = 706$

A reqd = 59.5 ÷ 13 = 4.6

2 PLATES 3/8 x 35 - A = 26.2 I = 2679.6

DESIGN OF 1"Φ BOLTS :-

1 @	3.70	6.85
1 @	9.70	40.00
1 @	15.70	123.00
1 @	21.70	236.00
1 @	27.70	385.00
1 @	33.70	567.50
		1363.35

1 @	6.52	21.25
1 @	12.52	78.50
1 @	18.52	171.50
1 @	24.52	300.00
1 @	30.52	466.00
		1037.25
	+	1363.35
		2400.60

$f_M = \frac{706 \times 16.85}{2400.6}$ 4.96

$f_v = \frac{59.5}{22}$ 2.71

Check For weld

$\frac{59.5}{2.3} = 25.9$

about 80"

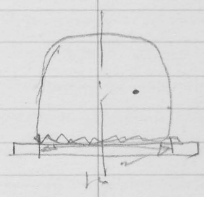
$\frac{59.5}{80} = .75$

MORE THAN Adequate for added moment

DESIGN OF bottom plates

1/2" THICK by previous design.

MOMENT TO BE DEVELOPED
49.0 x 5 1/2 = 269 INK



$I = 4 \times 1 \times \frac{d^3}{12} + a\left(\frac{d}{2}\right)^2$

$I = 4 \times 1 \times \frac{13^3}{12} + 2(8)^2$

1365 + 128

I = 1493

USE 1/4" filletwelds
$F_A = 13.0 \times 1/4 \times .707$
$= 2.30$ K/LN IN

$f = \frac{269 \times 8}{1493} = 1.44$

$f = \frac{P}{A} = \frac{49}{66} \quad .74$
2.18

DESIGN OF Bolts to TRANSMIT shear & MOMENT FROM BASE PLATE TO Footing:

M = 269 INK
V = 49 K

allow = 15,000 # for shear

USE 4 1 1/8" ϕ Bolts

for tension 20 K / sq. IN. $= (2)(.993) = 1.986$

$\frac{269}{8} = \frac{33.6}{20}$ K = 1.7 O.K.

DESIGN OF TIE ROD

$\frac{49}{20} = 2.45$ 2 - 1 1/8 □

DESIGN OF BASE PLATE

P = 105

P = .6

$P = \frac{105}{.6} = 175$ sq.in MAKE PLATE 18x10

$t^2 = .15\ pn^2 = .15 \times .6\ (3.5)^2 = 1.1025$

$t = 1.04$ MAKE 1 1/4" BASE PLATE

18x10x1 1/4 BASE PLATE

DESIGN OF Footing

DEAD N = 47,319 #

L.L. N = 17,213 #

DESIGN OF PEDISTAL

SIZE DETERMINED BY BASE PLATE 12" x 20" x 24"

THRUST 64.5

FROM ACI Code:-

REQ'D 4 - 7/8 ϕ
1/4" TIES - 12" O.C.

DESIGN OF FLANGE PLATE

AT FACE OF OUTER PL $f = \frac{12,150 \times 24}{22,400} = 13$

AT C.G. OF PS $= 13 \times \frac{23\frac{1}{2}}{24} = 12.75$

total force IN 1 PL $= 12.75 \times 16.5 = 210.5$

AREA OF A SPLICE PLATE $= 210.5 \div 20 = 10.5$

USE PLATE $16\frac{1}{2} \times 1$

No OF BOLTS $= 210.5 \div 9.38 = 24$ - 1" ϕ Bolts - 6 ROWS
MAKE BOLTED PORTION 14" LONG.

LENGTH OF WELD

$\frac{210.5}{2.3} = 92"$

92 - 16 = 76"

MAKE WELDED PORTION 38" LONG

DESIGN OF Footing.

D.L + 20 % L.L = 47.3 + 3.4 = 50.4
DL + LL 47.3 + 17.2 = 64.5

AREQ'D = 50.4 ÷ 3.85 = 13.1

P_r = 64.5 ÷ 13 = 4.96

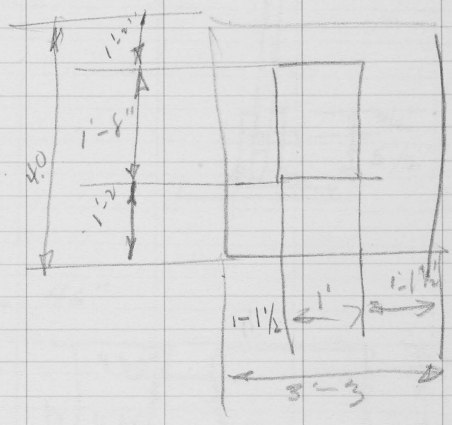
4 x 3'-3"
AR = 13.0

short way

$M = 4.96 \times 4.0 \times \frac{1.12^2}{2} = 12.5$

$V_B = 4.96 \times 4.0 \times 1.12 = 22.2$

$V_v = 4.96 \times \frac{3.93 \times 3.08}{2} \times .042 = 7.3$



LONG WAY

$M = 4.96 \times 3.25 \times \frac{1.17^2}{2} = 14.0$

$V_B = 4.96 \times 3.25 \times 1.17 = 18.9$

$V_v = 4.96 \times .04 \times 3.25 = .65$

$+ 4.96 \times \frac{3.25 \times 2.42 \times .46}{2} = \frac{6.45}{7.10}$

$M_{ALL} = 17.1 \times 3.25 = 55.6 > 11.0$ o.k.

$v = \frac{7.3}{37 \times 7/8 \times 8\frac{1}{2}} = 26.4$ o.k. < 60

$v = \frac{7.1}{29 \times 7/8 \times 8\frac{1}{2}} = 32.8$ o.k < 60

short way

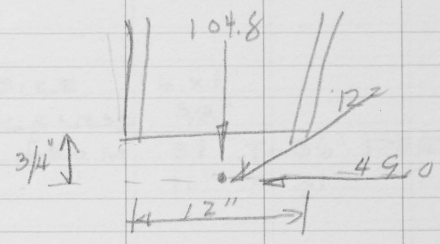
$A_s = 12.5 \times .693 \div 8\frac{1}{2} = 1.02$
$s_o = 22.2 \div (.168 \times 7/8 \times \ ) = 17.78$
} 11 - 1/2 ϕ

LONG WAY

$A_s = 11.0 \times .693 \times 8\frac{1}{2} = .90$
$s_o = 18.9 \div (.168 \times 7/8 \times 8\frac{1}{2}) = 15.16$
} 10 - 1/2 ϕ

DESIGN OF foot CONNECTION 3 HINGED STEEL FRAME

$R = \sqrt{104.8^2 + 49.0^2} = 115.2$

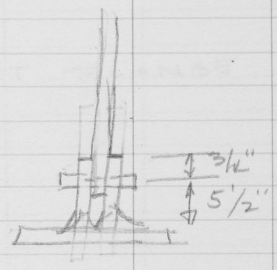


PIN A = 115.2 ÷ 15.0 = 7.68

3/4 ϕ PIN

HINGE PLATES

$t = \frac{115.2}{40.0 \times 3.25} = .885$



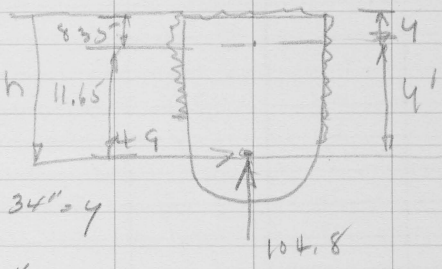
USE 2 - 1/2" BOTH PLS
2 - 1/2" TOP PLS

ASSUME h

LENGTH REQ'D = 104.8 ÷ 2.30 = 46"

APP. DOUBLE 92"
16
46" ÷ 4 = 19"

SAY 20" h



$c.g. = \frac{(8 \times 0) + (20 \times 2 \times 10)}{8 + (20 \times 2)} = 8.34" = y$

$\therefore y' = 20 + 1.75 - 8.35 = 13.40$

$M = 49.0 \times 13.40 = 655$ INK.

USE 1/4" FILLET.
$F = 13.0 \times .25 \times .707 = 2.30$

$J = 1 \times 2 \times (\sqrt{4^2 + 11.15^2})^2 = 280$
$1 \times 2 \times (\sqrt{4^2 + 10.15^2})^2 = 238$
$1 \times 2 \times (\sqrt{4^2 + 9.15^2})^2 = 200$
$1 \times 4 \times (\sqrt{4^2 + 8.00^2})^2 = 320$
$1 \times 4 \times (\sqrt{4^2 + 7.00^2})^2 = 260$
$1 \times 4 \times (\sqrt{4^2 + 6.00^2})^2 = 208$
$1 \times 4 \times (\sqrt{4^2 + 5.00^2})^2 = 164$
$1 \times 4 \times (\sqrt{4^2 + 4.00^2})^2 = 128$
$1 \times 4 \times (\sqrt{4^2 \times 3.00^2})^2 = 100$
$1 \times 4 \times (\sqrt{4^2 \times 2.00^2})^2 = 80$
$1 \times 4 \times (\sqrt{4^2 + 1.00^2})^2 = 68$
$1 \times 2 \times (\sqrt{3.5^2 + 8.35^2})^2 = 164$
$1 \times 2 \times (\sqrt{3.5^2 \quad 8.35^2})^2 = 152$
$1 \times 2 \times (\sqrt{1.5^2 + 8.35^2})^2 = 144$
$1 \times 2 \times (\sqrt{.52^2 + 8.35^2})^2 = 140$
2646

$f_M = \frac{655 \times \sqrt{4^2 + 18.15^2}}{5292} = 17.4$

$f_M = 104.8 \div 96 = 1.09$

1.47
256

MAKE H = 24"

TWO HINGED STEEL

ATTEMPTED SIZE

CONSTANT FLANGE SIZE 16 x 1

" WEB THICKNESS 3/8"

BEAM WEB VARIES FROM 31" TO 36" DEEP

COLUMN " " " 10" TO 31" "

FINAL DESIGN

SAME AS ABOVE EXCEPT FLANGE REDUCED TO 15 x 1

TWO HINGED FRAME STEEL

METHOD OF ANALYSIS:

① MAKE DETERMINATE
PLACE HINGE AT A

② FIND MOMENTS ON FRAME

LOADING DEAD

ROOFING, SLAB & SOUND BOARD 52 #/sq. ft

OR.

780 #/LN FT.

VERTICAL REACTION DEAD ONLY

web BEAM	3/8 x 34 x 45.7	= 1980 #
from above	780 x 45.7	35600 #
PLATE	108.8 x 45.7	4900 #
		42,480 #

COLUMN

PLATE	108.8 x 23	2500
		510
		3010

TOTAL 45,490 #

NO FIXITY ∴ NO HORIZONTAL REACTION :-

MOMENTS IN # IN

@ B	-0-
3' FROM "	1,003,200
5' " "	2,409,240
10' " "	4,539,600
15' " "	6,390,840
20' " "	7,963,200
25' " "	9,268,440
30' " "	10,270,800
35' " "	11,006,040
40' " "	11,462,400
℄ " "	11,721,600

AREA OF MOMENTS (FOLLOWING EQUASION APPROX BUT ONSIDE OF SAFETY)

$$P \int_0^{\ell} = 42,480\ell - 38.75\ell^2 \, d\ell$$

$$P = 709,063,800 \text{ IN \#}$$

MOMENT OF INERTIA

REAL BEAM

$$I = \frac{bh^3}{12} + 2ay^2$$

$$I = 3/8(34 - .219\ell)^3 + 32(17.5 - .109\ell)^2$$

$$I \quad \frac{(34 - .219\ell)^3 + 1024(17.5 - .109\ell)^2}{32}$$

ANAL BEAM ABOUT AXIS A-A.

$$d = \frac{1}{EI} = \frac{32}{E(34 - .219\ell)^3 + 1024(17.5 - .109\ell)^2}$$

$$I_x = \int_0^{45.7} \frac{y^3 dx}{3} = \frac{32^3}{3E^3} \int_0^{45.7} \frac{d\ell}{[(34 - .219)^3 + 1024(17.5 - .109\ell)^2]^3}$$

UN-INTERGRATABLE ∴ PLOTTED

ℓ =	c	ℓ	ℓ^2	ℓ^3	[ ]	$\frac{32^3}{[\ \ ]^3}$
ℓ = 0	352,904	-0-	-0-	-0-	352,904	.00000000000248
5	"	23,330	94.92	1.31	329,670	.00000000000305
10	"	46,661	379.68	10.45	306,634	.00000000000379
15	"	69,991	854.28	35.45	283,802	.00000000000477
20	"	93,321	1518.73	84.03	261,186	.00000000000612
25	"	116,651	2373.01	164.12	238,790	.00000000000802
30	"	139,982	3417.14	283.59	216,623	.00000000000 1075
35	"	163,311	4651.10	450.34	194,693	.00000000000 1480
40	"	186,642	6074.91	672.22	173,008	.00000000000 2109
ℓ	"	213,239	7929.62	1002.49	149,597	.00000000000 3263

TOTAL = $\frac{.00000000\ 672}{E^3}$ [SEE GRAPH 1A]

FINDING CENTROID OF ANAL COLUMN (1/2)

$$\bar{X} = \frac{M}{A}$$

AREA OF ANAL. COLUMN

$$A = \frac{1}{E} \int_0^{45.7} \frac{32\,d\ell}{[352,904 - 4666.052\ell + 3.7968182\ell^2 + .0105034592\ell^3]}$$

ℓ = 0	352,904	.0000907
5	329,670	.0000971
ℓ = 10	306,634	.0001044
15	283,802	.0001127
20	261,186	.0001225
25	238,796	.0001340
30	216,623	.0001477
35	194,693	.0001644
40	174,008	.0001850
ℓ	149,594	.0002139

[SEE GRAPH 2A]

TOTAL $\frac{.073884}{E}$

163
11 4.1

3200
3000
2800
2600
2400
2200
2000
1800
1600
1400
1200
1000
800
600
400
200

0 5 10 15 20 25 30 35 40 45

6 60

everything to inches

4666 X .0000000000002 X 6 X 12

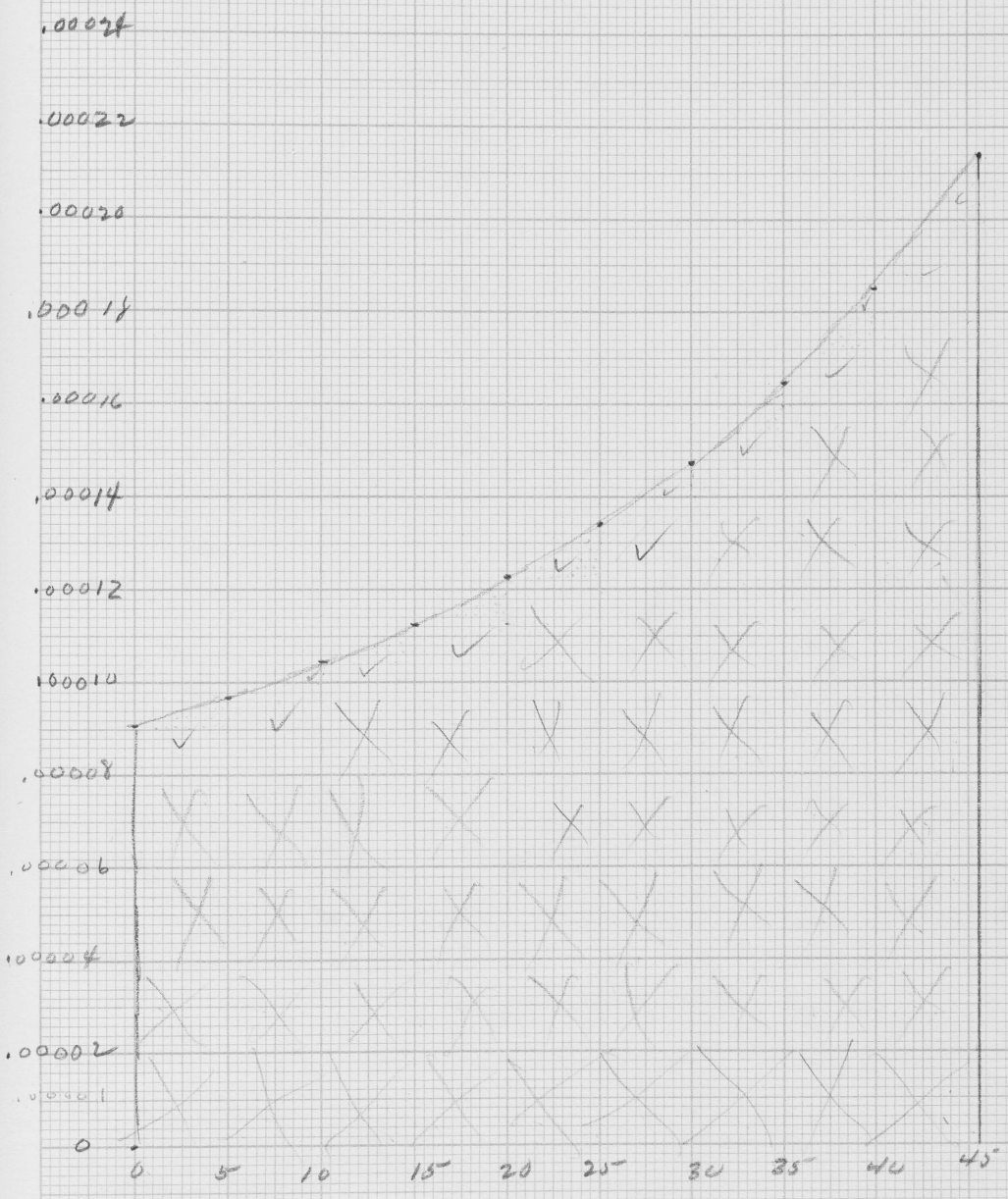
.0000000067 1904

3400
114
99
136
70
12
67
51
66
86
33
99
5
55
26
100
52
70
43
20
4666

INERTIA

EVERETT WAIDLEY CO. RICHMOND, VA.

6157 x .000002 x 6 = .073884

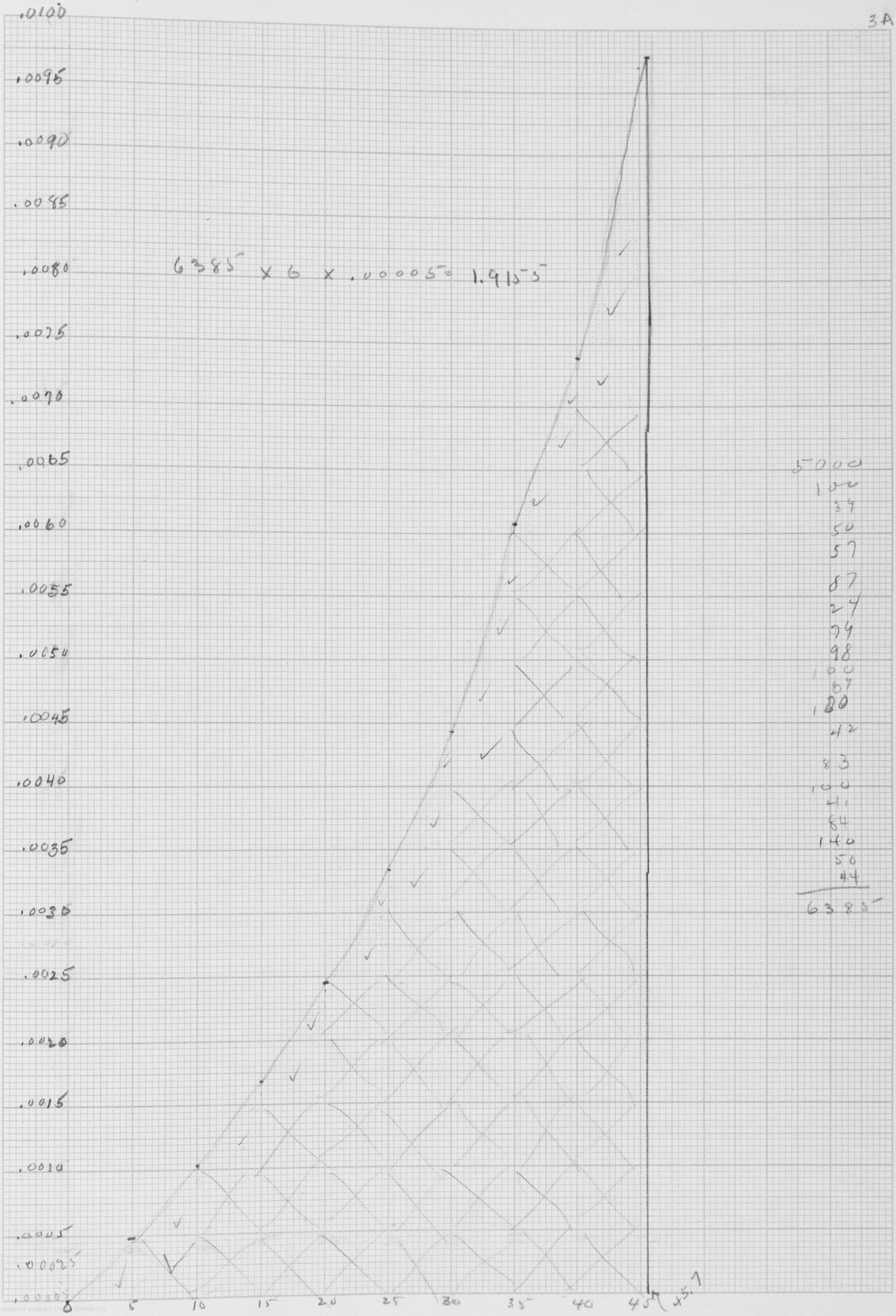


105
7
73.5

5400
74
100
79
38
84
100
41
72
100
71
6157

AREA OF ANAL COLUMN.

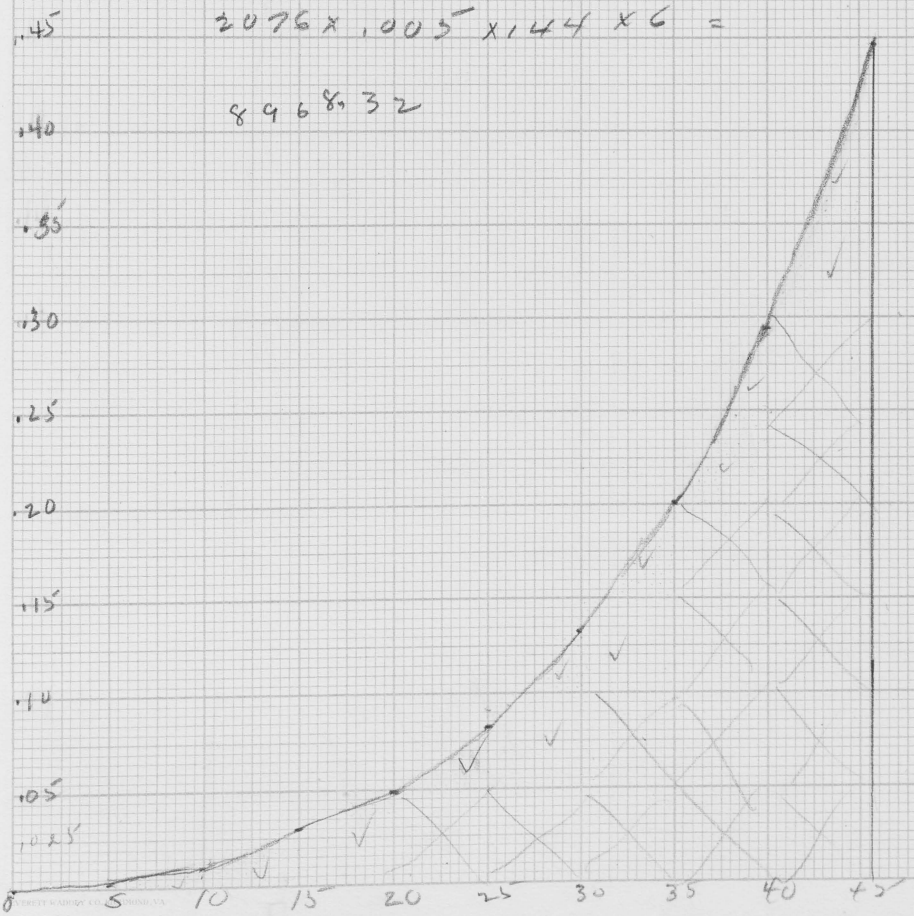
EVERETT WADDEY CO. RICHMOND, VA.



MOMENT OF AREA OF ANAL COLUMN

2076 x .005 x .144 x 6 =

8968.32

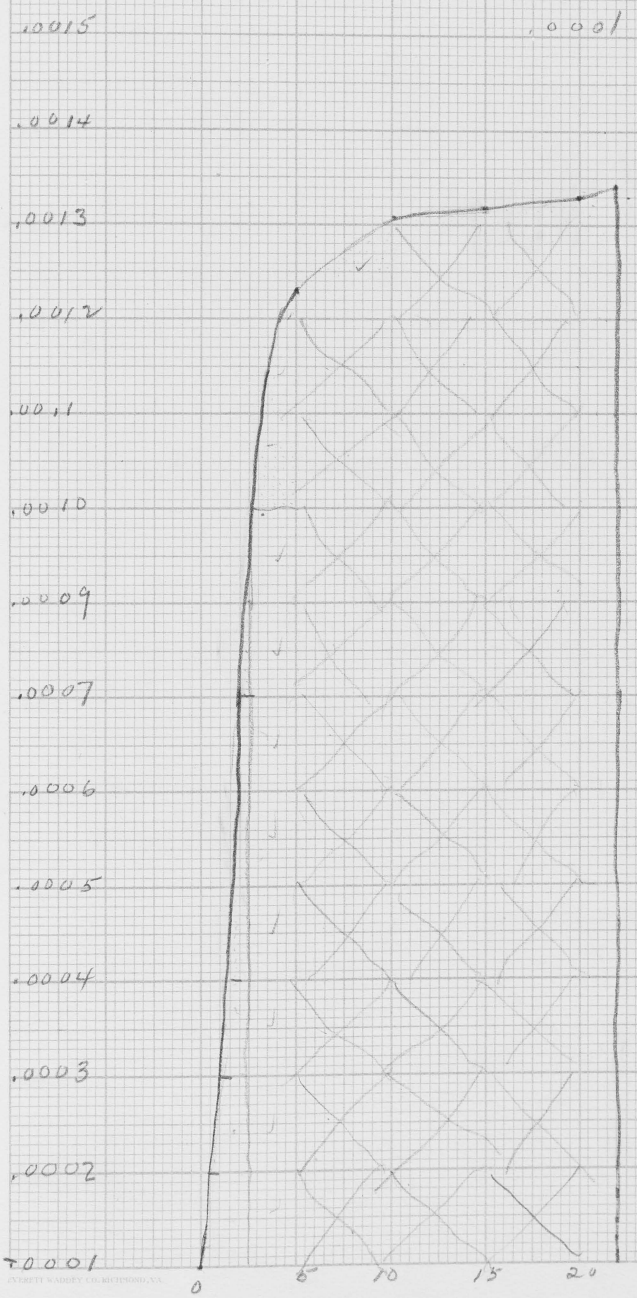


84
7
62.3

1400
4
62
13
37
28
24
43
19
96
36
100
75
35
2076

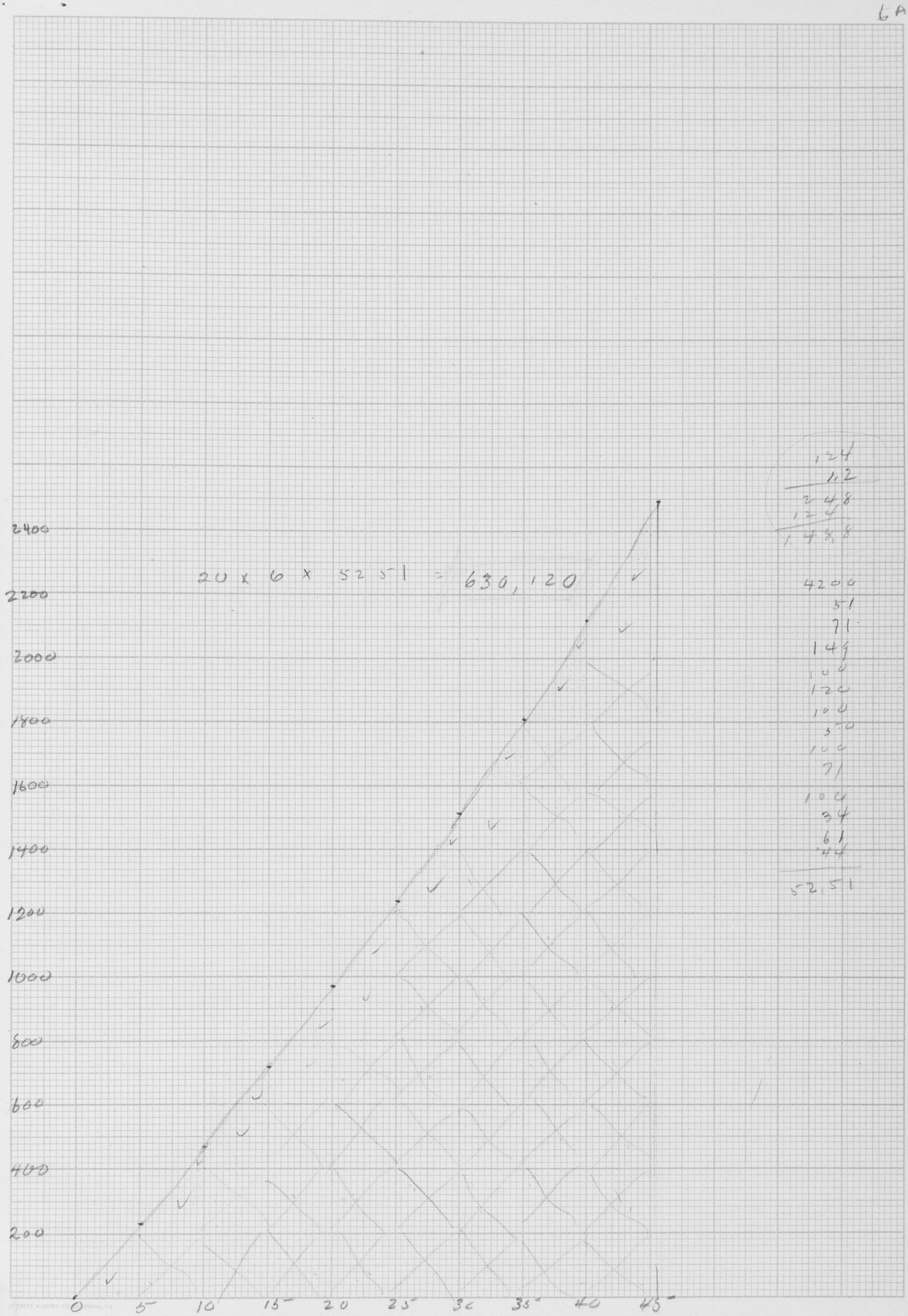
.0001 x 6 x 144

$$\frac{374,544}{E}$$



3200
452
400
60
30
20
10
12
27
72
30
4335

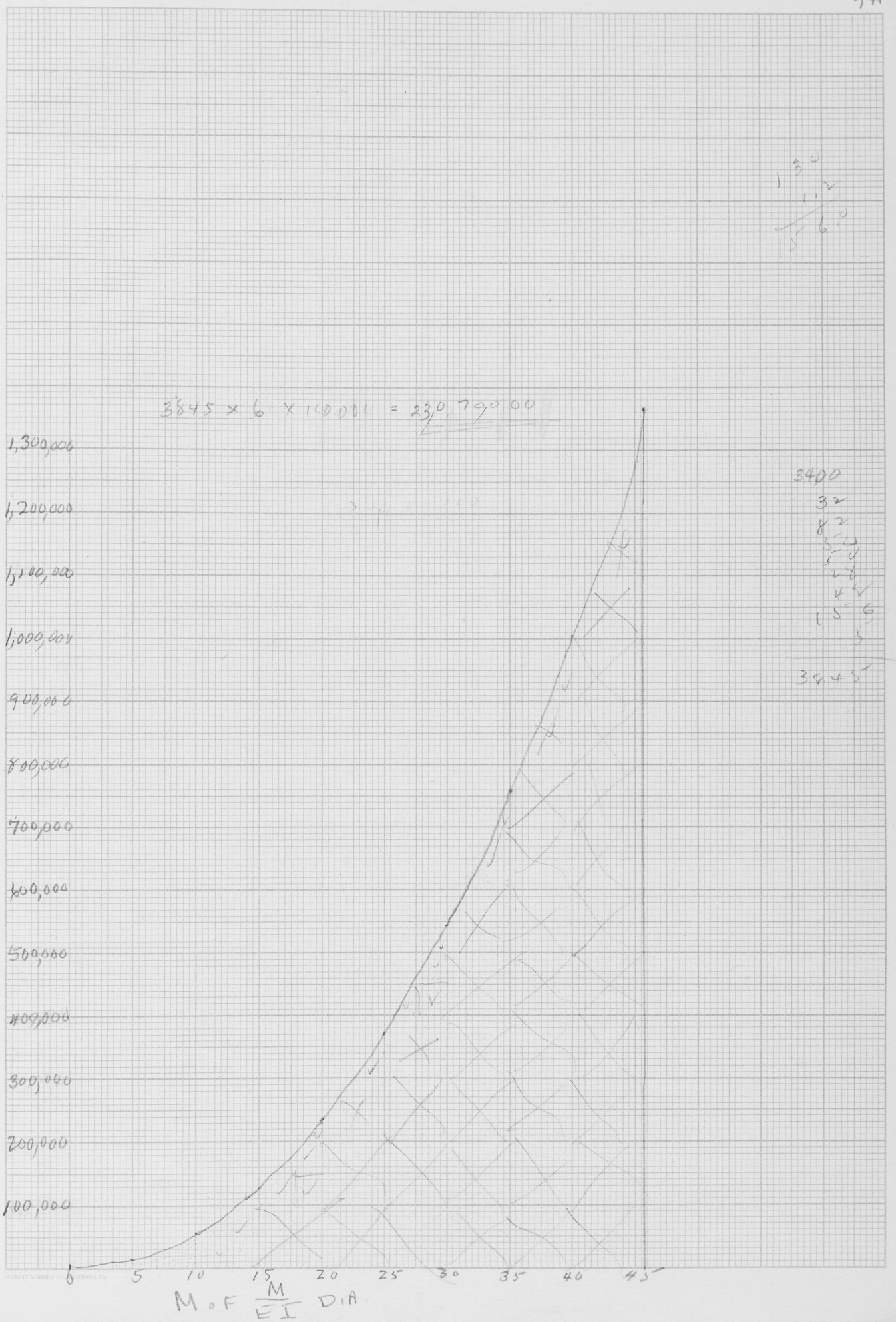
EVERETT WADDEY CO. RICHMOND, VA.



$\frac{M}{EI}$ DIA. AREA.

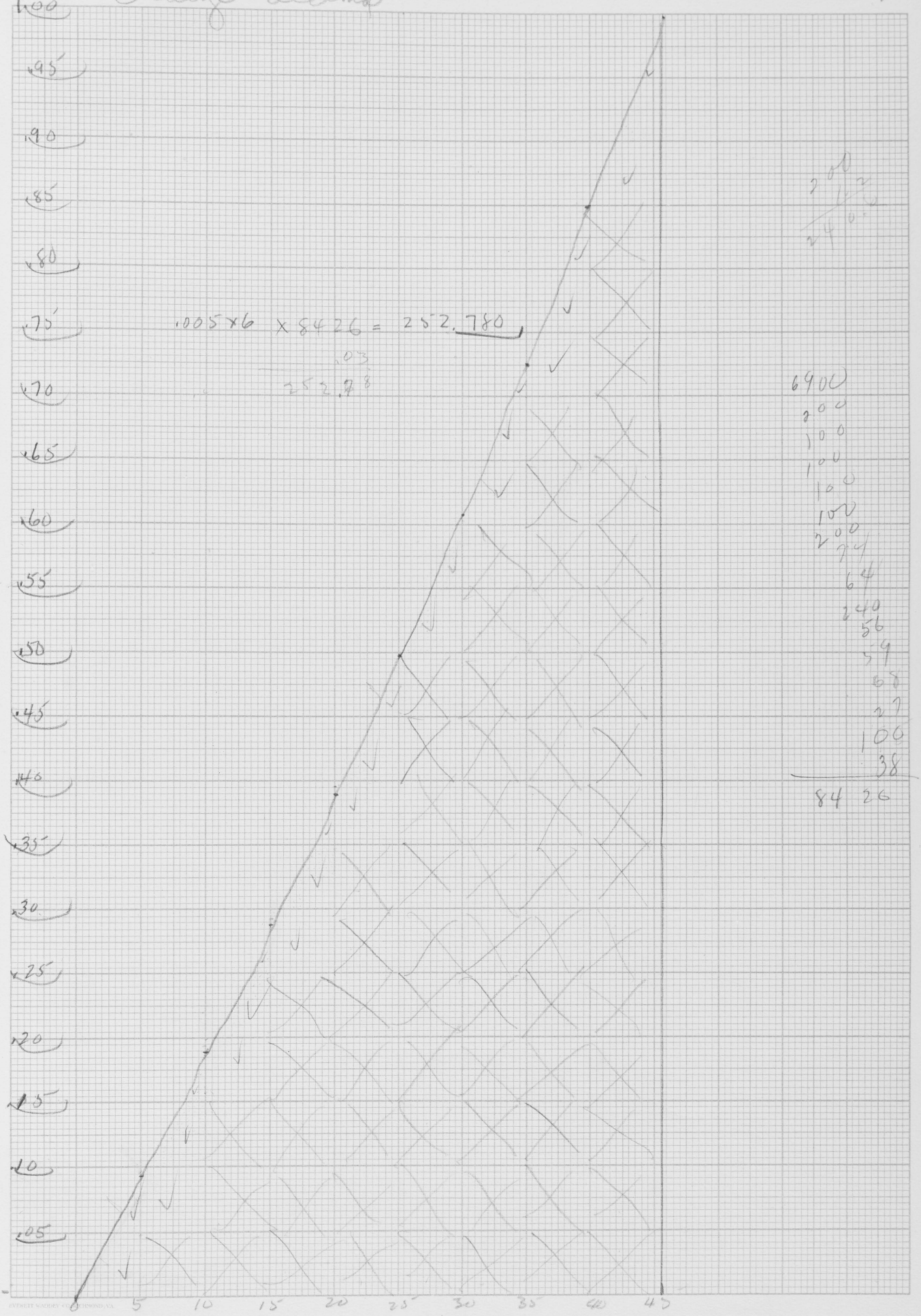
EVERETT WADDEY CO. RICHMOND, VA

7A

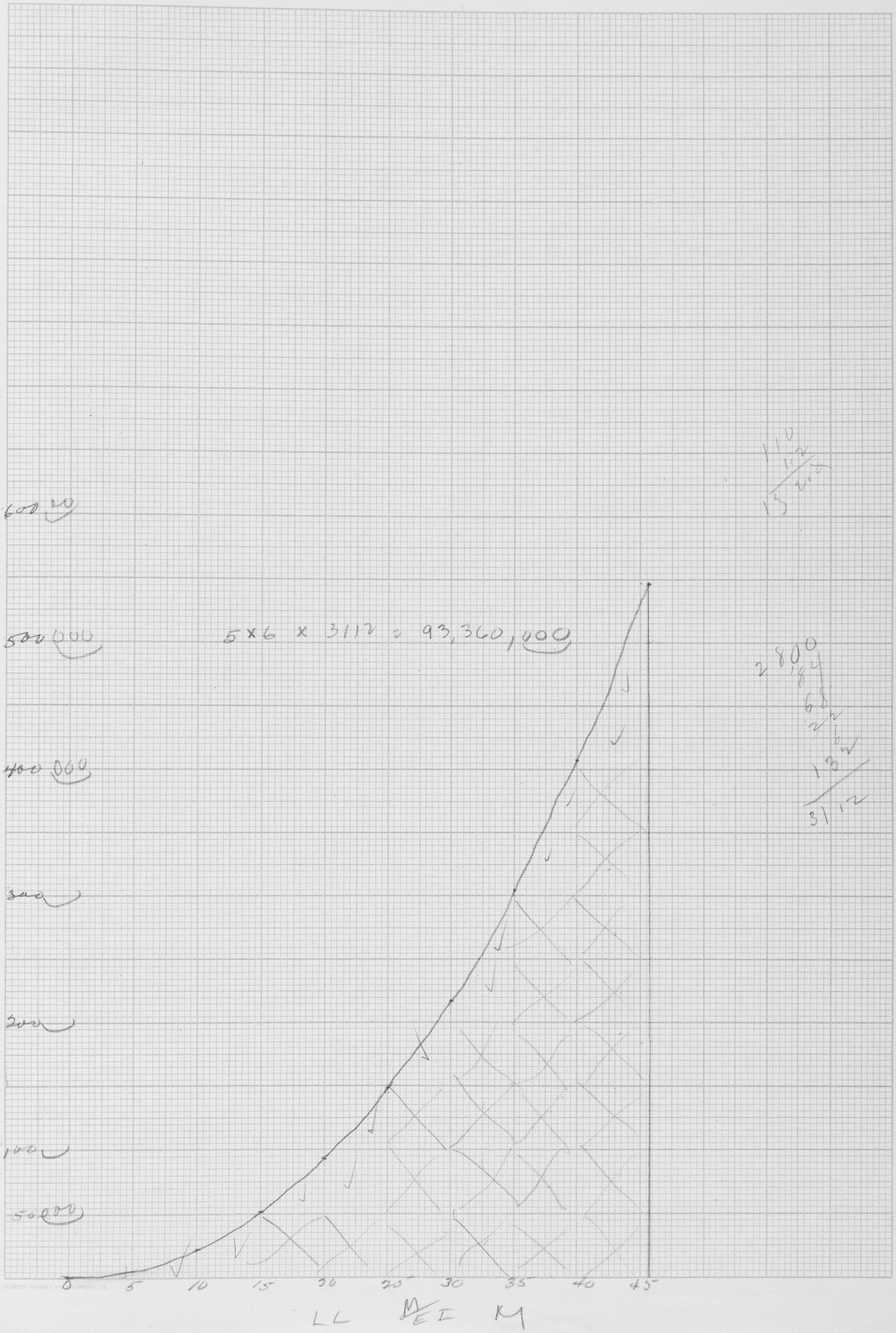

3845 x 6 x 100001 = 2307900
1,300,000
1,200,000
1,100,000
1,000,000
900,000
800,000
700,000
600,000
500,000
400,000
300,000
200,000
100,000
0
5
10
15
20
25
30
35
40
45
M of M/EI DIA
3400
32
87
520
28
4
15 6
3
3845

Change decimal

84



L.L. M/EI AREA.


600 00
500 000
400 000
300
200
100
50 000
5 × 6 × 3112 = 93,360,000
0
5
10
15
20
25
30
35
40
45
LL
M/EI
M
2800
84
60
22
132
3112

MOMENT ABOUT L END (OF HALF FRAME)

$$M = \int x y \, dx = \frac{32}{E} \int_0^{45.7} \frac{12 l \, dl}{[352,904 - 4666.05 l - 3.796818 l^2 + .01050345 l^3]}$$

PLOTTED SEE GRAPH 3A.

	DEM	$\frac{12 l}{[\ ]}$
$l = 0$	352,904	— 0 —
5	349,670	.000485
10	306,634	.001044
15	283,802	.001691
20	261,186	.002450
25	238,790	.003350
30	216,623	.004432
35	194,693	.006064
40	173,008	.007399
℄	149,594	.009776

TOTAL $= \frac{1.9155}{E}$

$$\bar{x} = \frac{M}{A} = \frac{1.9155}{.073884} = 25.93$$

I_{y-y} ABOUT P-P

$$I_{yp} = \int x^2 y \, dx = \frac{32}{E} \int_0^{45.7} \frac{144 l^2 \, dl}{[352,901 - 4666.05 l - 3.796818 l^2 + .01050345 l^3]}$$

	DEM.	
$l = 0$	352,904	— 0 —
5	399,670	.00243
10	306,634	.01044
15	283,802	.03151
20	261,186	.04901
25	238,790	.08376
30	216,623	.13295
35	194,693	.20134
40	173,008	.29594
℄	149,594	.44675

SEE GRAPH 4A

TOTAL $= \frac{8968.32}{E}$

MOVING AXIS TO CENTROID AXIS B-B

$$I_y = I_{yp} - A d^2$$

$$= \frac{8968.32}{E} - \frac{.073884}{E} \times \overline{25.93}^2 \times 144$$

$$= \frac{8968.32}{E} - \frac{7153.49}{E} = \frac{1814.83}{E}$$

MOVING AXIS THRU 10°-36' (APPROX), TO A-A.

$I'_x = I_x \cos^2\theta + I_y \sin^2\theta - 2P_x \sin\theta \cos\theta$.

< Too small $+ \frac{1814.83}{E} (.18395)^2 - 0$

$I'_x = \frac{61.409}{E}$

MOVING AXIS THRU 10°-36' (APPROX), TO B-B

$I'_y = I_x \sin^2\theta + I_y \cos^2\theta + 2P_x \sin\theta \cos\theta$

= Too small $+ 1814.83 (.98294)^2 + 0$

$I'_y = 1753.44$

FIND AMT. TO MOVE AXIS

$\sin\theta = \frac{y}{25.93}$

$y = (25.93)(.18395)$

$y = 4.77$

$+ 25$

29.77

$\cos\theta = \frac{x}{25.93}$

$x = (25.93)(.98294)$

$x = 25.48$

FROM 45.00

19.52

MOVING AXIS TO X-X

$I''_x = I'_x + Ad^2$

$= \frac{61.409}{E} + \frac{.073884}{E} \times (29.77)^2 \times 144$

$\frac{61.409}{E} + \frac{9429.11}{E}$

$I''_x = \frac{9490.52}{E}$

MOVING AXIS TO Y-Y

$I''_y = I'_y + Ad^2$

$= \frac{1753.44}{E} + \frac{.073884}{E} (19.52)^2 \times 144$

$= \frac{1753.44}{E} + \frac{4053.90}{E}$

$= 5807.34$

MOMENT OF INERTIA OF COLUMN PORTION

REAL

$$I = \frac{bh^3}{12} + 2ay^2$$

$$I = \frac{3/8(10+.96\ell)^3}{32} + 32(5.5+.48\ell)^2$$

$$I = \frac{(10+.96\ell)^3 + 1024(5.5+.48\ell)^2}{32}$$

I_{y-y} too small to be bothered with

$$I_{s-s} = \int_0^{22\times 12} xy\,dy = \frac{32}{E}\int \frac{y^2\,dy}{(10+.96y)^3 + 1024(5.5+.48y)^2}$$

$$= \frac{y^2}{31,976 + 5,694.72y + 23,620.61y^2 + .884736y^3}$$

PLOTTED SEE GRAPH 5A.

	C	ℓ	ℓ^2	ℓ^3	SUM	$32y^2$	$\frac{32y^2}{DEM.}$
$\ell = 0$	31,976	-0-	-0-	-0-	31,976	-0-	-0-
5	"	28,473.6	590,515	111	651,076	800	.00123
10	"	56,947.2	2,363,061	885	2,451,069	3200	.00131
15	"	85,420.8	5,314,637	2985	5,435,019	7200	.00132
20	"	113,8944	9,448,244	7078	9,601,192	12,800	.00133
22	"	125,2838	11,432,323	9421	11,599,056	15,488	.00134

TOTAL = $\frac{374.544}{E}$

$$I''_x = I_{ss}\cos^2\theta + I_{y-y}\sin^2\theta - 2P_x\sin\theta\cos\theta$$

$$= \frac{374.544}{E}(.99919)^2 \text{ too small}$$

$$I''_x = 373.873$$

$$\text{TOTAL } I''_x = 2\left[\frac{373.873}{E} + \frac{9490.5}{E}\right] = \frac{19,728.746}{E}$$

$\frac{M}{EI}$ DIAGRAM AREA

$$A \text{ of } \frac{M}{EI} = \int_0^{457\times 12} \frac{32(42,480\,\ell - 38.75\ell^2)\,d\ell}{E(352,904 - 4666.052\,\ell + 3.796818\ell^2 + .0105034 59\ell^3)}$$

	DEM	ℓ	ℓ^2	$\ell - \ell^2$	$\times 32$	$\frac{A}{D}$
$\ell = 0$	352,904	-0-	-0-	-0-	-0-	-0-
5	329,670	2,548,800	134,500	2,404,300	77,097,600	233.86
10	306,634	5,097,600	538,000	4,539,600	145,267,200	473.75
15	283,802	7,646,400	1,255,000	6,390,900	204,508,800	720.60
20	261,186	10,195,200	2,232,000	7,963,200	254,822,400	975.64
25	238,790	12,774,000	3,487,500	9,256,500	296,208,000	1240.45
30	216,623	15,292,800	5,022,000	10,270,800	328,665,600	1517.22
35	194,693	17,841,600	6,835,500	11,006,100	352,195,200	1808.98
40	173,008	20,390,400	8,928,000	11,462,400	366,796,800	2120.11
E	149,544	23,296,030	11,653,750	11,642,280	372,552,960	2490.42

TOTAL AREA = $\frac{630,190}{E}$

SEE GRAPH 6A

$M_o = \frac{M}{EI}$ D.A. $\frac{32}{E} \int_0^{45.7 \times 12} \frac{42,480\ell^2 - 38.75\ell^3}{[352,904 - 4666.052\ell + 3.796818\ell + .0105034 59\ell^3]}$

$\ell = 0$	-0-
5	14,032
10	56,850
15	129,708
20	234,415
25	372,135
30	546,199
35	759,772
40	1,017,653
ℓ	1,365,746

SEE GRAPH 7A.

TOTAL = $\frac{23,070,000}{E}$

$\bar{X} = \frac{23,070,000}{E} \times \frac{E}{630,190}$

$\bar{X} = 366.1$ IN.

$\bar{y}$ = distance to ABOVE CENTROID

$\sin\theta = \frac{y}{366.1}$

$y = 366.1\ (.18395)$

$= 67.4 + 300 = 367.4$ IN.

COLUMN MOMENTS FIXED END

@25' BASE FROM	$\frac{1,260,386}{E} \times 367.4 \div \frac{19,729}{E}$		× 300	= 7,041,364
@22' "	"		× 264	= 6,196,400
@20' "	"		× 240	= 5,636,309
@15' "	"		× 180	= 4,224,819
@10' "	"		× 120	= 2,816,581
@5' "	"		× 60	= 1,408,273
@0				= -0-

BEAM MOMENTS FIXED END

@3' FROM LT	1,260,380 X367.4 / 19,729	x	363.83	= 7,139,946
5' " "	"	x	310.68	= 7,292,037
10' " "	"	x	324.36	= 7,542,710
15' " "	"	x	332.04	= 7,793,822
20' " "	"	x	342.72	= 8,044,655
25' " "	"	x	353.40	= 8,294,727
30' " "	"	x	364.08	= 8,545,400
35' " "	"	x	374.76	= 8,796,073
40' " "	"	x	385.44	= 9,046,745
℄	"	x	396.00	= 9,294,601

FINAL DEAD LOAD

Column

	MOMENT	SHEAR	THRUST
BASE	— 0 —	20,655	45,490
5' FROM "	1,408,273	"	45,000
10' " "	2,816,531	"	44,500
15' " "	4,224,819	"	43,900
20' " "	5,630,309	"	43,000
22' " "	6,196,400	"	42,800

BEAM

OUTER END	7,041,364		
3' " "	6,136,740	41,918	20,300
5' " "	4,882,740	40,058	20,265
10' " "	3,003,110	35,268	20,215
15' " "	1,402,982	30,618	20,129
20' " "	80,855	27,968	20,044
25' " "	− 973,713	23,318	19,958
30' " "	− 1,725,400	16,668	19,873
35' " "	− 2,209,967	12,018	19,787
40 " "	− 2,415,655	7,368	19,702
℄	− 2,426,899	2,718	19,616
		− 583	19,519

LIVE LOAD

25# /sq ft = 25 x 15 = 375# /LNFT.

VERTICAL REACTION

375 x 45.7 = 17,137.5

MOMENTS NO FIXITY

OUTEREDGE	-0-
3' " "	402,300
5' " "	970,572
10' " "	1,830,312
15' " "	2,577,072
20' " "	3,213,000
25' " "	3,733,572
30' " "	4,144,500
35' " "	4,391,472
40 " "	4,626,000
℄	4,819,184

SYMETRICAL ABOUT ℄

NO MOMENTS ON COLUMNS

$$\text{AREA OF } \frac{M}{EI} = \frac{32}{E}\int_0^{45.7} \frac{17{,}137.5(.98294)\,l \times 12 - .1875(.98294)\times\frac{1}{2}\,l^2 \times 144}{(352{,}904 - 4666.052\,l + 3.796818\,l^2 + .01050345\,l^3)}$$

$$= \frac{32}{E}\int \frac{17017 \times l \times 12 - 15.514687\,l^2 \times 144}{(\text{DENOMMATOR})}$$

	DEN.	l	l^2	$32(l-l^2)$	$\frac{M}{D}$
$l = 0$	352,904	-0-	-0-	-0-	-0-
5	324,670	1,021,020	55,850	30885,440	94
10	306,634	3,042,040	223,410	58,196,160	190
15	283,802	3,063,060	502,680	81,932,160	289
20	261,186	4,084,080	893,650	102,693,760	391
25	238,790	5,105,100	1,396,320	118,680,960	497
30	216,023	6,126,120	2,010,700	131,693,440	608
35	194,693	7,147,140	2,736,790	141,131,200	725
40	170,008	8,168,160	3,574,580	146,898,560	849
℄	149,594	9,332,120	4,665,930	149,318,000	998

TOTAL $\frac{252{,}780}{E}$

$$\text{M OF } \frac{M}{EI} \text{ DIA} = \frac{32}{E}\int \frac{17017 \times l^2 \times 144 - 15.514687\,l^3 \times 1728}{352{,}904 - 4666.052\,l + 3.796818\,l^2 + .01050345\,l^3}$$

$l = 0$	-0-
5	5,640
10	22,800
15	52,020
20	93,840
25	149,100
30	218,880
35	304,500
40	407,520
℄	547,300

TOTAL = $\frac{93{,}360{,}000}{E}$

$\bar{x} = \frac{93,360,000}{E} \times \frac{E}{252,780} = 369.333''$

$\bar{y}$ distance

$\sin\theta = \frac{y}{369.3}$

$y = 369.3(.18395)$

$y = 67.9 + 300 = 367.9$

Column (Fixed) Live Load

	$\frac{505,560 \times 367.9}{19729}$				
@25' from base		×	300	=	3,820,000
22' " "	"	×	264	=	2,482,000
20' " "	"	×	240	=	2,260,000
15' " "	"	×	180	=	1,692,000
10' " "	"	×	120	=	1,130,000
5' " "	"	×	60	=	565,000
—0— " "	"	×	0	=	—0—

Beam

3' from L end	"	×	304.20	=	2,865,000
5' " "	"	×	310.68	=	2,920,000
10' " "	"	×	321.36	=	3,029,000
15' " "	"	×	332.04	=	3,120,000
20 " "	"	×	342.72	=	3,225,000
25 " "	"	×	353.40	=	3,325,000
30 " "	"	×	364.08	=	3,422,000
35 " "	"	×	374.76	=	3,530,000
40 " "	"	×	385.44	=	3,630,000
℄	"	×	396.00	=	3,730,000

Live Load	Total M	V	N
Column			
Base	—0—	9,420	17,138
5' from "	565,000	"	17,138
10' " "	1,130,000	"	17,138
15' " "	1,692,000	"	17,138
20' " "	2,260,000	"	17,138
22' " "	2,482,000	"	17,138
Beam			
End	2,820,000	17,000	9,650
3' from end	2,462,700	16,400	9,629
5' " "	1,949,428	15,295	9615
10' " "	1,189,688	13,452	9584
15' " "	542,928	11,609	9549
20' " "	112,000	9766	9515
25' " "	—408,572	7,923	9480
30' " "	—722,500	6,082	9446
35' " "	—861,472	4240	9411
40' " "	—996,000	2398	9377
℄ " "	—1,089,188	—308	9341

Check Column AT 22' FROM BASE

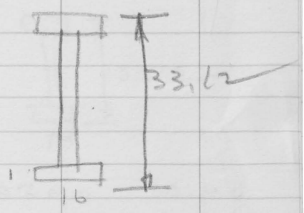
PAGE 11

M = 8,678,400
N = 59,938

$f = \frac{P}{A} = \frac{59,938}{4367} = 1,370$

$f = \frac{Mc}{I} = \frac{8,678,400 \times 16.56}{9123}$

$= 15,750$



AREA = 32.00
11.67
43.67

INERTIA 512(16) 8192
931.0
9123.0

$r^2 = \frac{I}{A} = \frac{9123}{43.67}$

$r^2 = 209$

$r = 14.45$

$\frac{l}{r} = \frac{25 \times 12}{14.45}$

20.375

Allowable 16,810

$\frac{ld}{bt} = \frac{300 \times 33.12}{16 \times 1}$

$= 620$

allowable 19,350

COMBINED

$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$

$\frac{1,370}{16,810} + \frac{15,750}{19,350}$

$.082 + .814 = .896$

11 A

TRY 15" FLANGE

M = 8,678,400

N = 59,938

$f = \frac{P}{A} = \frac{59,938}{41.64} = 1,440$

$f = \frac{Mc}{I} = \frac{8,678,400 \times 16.56}{8611} = 16,700$

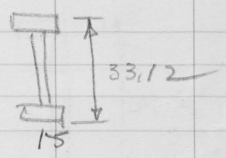
COMBINED

$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$

$\frac{1,440}{16,790} + \frac{16,700}{18,200} =$

$.086 + .918 = 1.004$

USE 15' flange



AREA 30.00
11.64
41.64

INERTIA 512(15)7680
931.0
8611.0

$r^2 = \frac{I}{A} = \frac{8611}{41.64}$

$r^2 = 207$

$r = 14.4$

$\frac{L}{r} \frac{25 \times 12}{14.4} = 20.8$

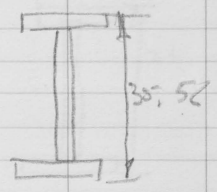
allowable 16,790

$\frac{ld}{bt} = \frac{300 \times 33.12}{15}$

660

allowable = 18,200

check BEAM 3' FROM outer edge



M = 8,599,444
N = 29,894

$f = \frac{P}{A} = \frac{29,894}{44.59} = 665$

$f = \frac{Mc}{I} = \frac{8,599,444 \times 17.78}{10712.3}$

$f = 14,250$

AREA 32.00
12.59
44.59

INERTIA 1176.3
(596) (16) 9536
10712.3

$r^2 = \frac{I}{A} = \frac{10712.3}{44.59}$

$r^2 = 235$

$r = 15.35$

$\frac{90 \times 12}{15.35} =$

70.4

allowable 14,620

COMBINED

$\frac{f_a}{F_A} + \frac{f_b}{F_b} \leq 1$

$\frac{665.0}{14,620} + \frac{14,250}{18,000} =$

$.0455 + .792 = .838$

.046
.838

$\frac{ld}{bt} = \frac{25 \times 12 \times 35.56}{16 \times 1}$

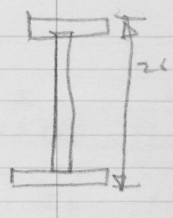
$= 667$

allowable 18,000

$M = 3,516,087$
$N = 28,860$

$f = \frac{P}{A} = \frac{28,860}{41} = 705$

$f = \frac{MC}{I} = \frac{3,516,087 \times 13}{5440} = 8,400$



AREA = 32.00
9.00
41.00

INERTIA 432.0
313(16) 5008
5440.0

$r^2 = \frac{I}{A} = \frac{5440}{41} = 133$

$r = 11.55$

$\frac{90 \times 12}{11.55} = 93.6$

allowable 12,720

allowable for bending
20,000

$\frac{705}{12,720} + \frac{8,400.0}{20,000}$

$.055 + .420 = .475$

$\frac{th^3}{12} + a y^2$

COLUMN DESIGN OF WELDS

$\frac{VQ}{I}$

BASE	$\frac{30,075 \times 5.5 \times 15}{946.3}$	2620	I = 15(61)	I = 31.3 915 946.3
5' FROM BASE	$\frac{30,075 \times 7.9 \times 15}{1980.5}$	1780	15(123)	I = 105.5 1875 1980.5
10' FROM BASE	$\frac{30,075 \times 10.3 \times 15}{3385.5}$	1372	15(212)	I = 205.0 3180 3385.5
15' FROM BASE	$\frac{30,075 \times 12.7 \times 15}{5282}$	1082	15(323)	I = 432 4850 5282
20' FROM BASE	$\frac{30,075 \times 15.1 \times 15}{7612.2}$	895	15(456)	I = 762.2 6850 7612.2
22' FROM BASE	$\frac{30,075 \times 16.1 \times 15}{8661}$	838	15(515)	I = 931 7730 8661

BEAM

3' FROM outer edge	$\frac{56,458 \times 17.3 \times 15}{10036.3}$	1455	15(598)	I 1176.3 8860 10036.3
5' " " "	$\frac{50,563 \times 17.0 \times 15}{9763.0}$	1318	15(575)	I 1123.0 8640 9763.0
10' " " "	$\frac{44,070 \times 16.4 \times 15}{9104}$	1190	15(538)	I 1024 8080 9104
15' " " "	$\frac{39,577 \times 15.9 \times 15}{8471}$	1115	15(502)	I 931 7540 8471
20' " " "	$\frac{33,084 \times 15.3 \times 15}{7883.8}$	964	15(468)	I 843.8 7040 7883.8
25' " " "	$\frac{24,591 \times 14.8 \times 15}{7292.2}$	748	15(435)	I 762.2 6530 7292.2
30' " " "	$\frac{18,100 \times 14.2 \times 15}{6665.1}$	579	15(403)	I 615.1 6050 6665.1
35' " " "	$\frac{11,600 \times 13.7 \times 15}{6149.3}$	389	15(375)	I 549.3 5600 6149.3
40' " " "	$\frac{5116 \times 13.1 \times 15}{5638.3}$	178.5	15(343)	I 484.3 5150 5638.3
	$\frac{891 \times 12.5 \times 15}{5132}$	32.6	15(313)	I 432 4700 5132

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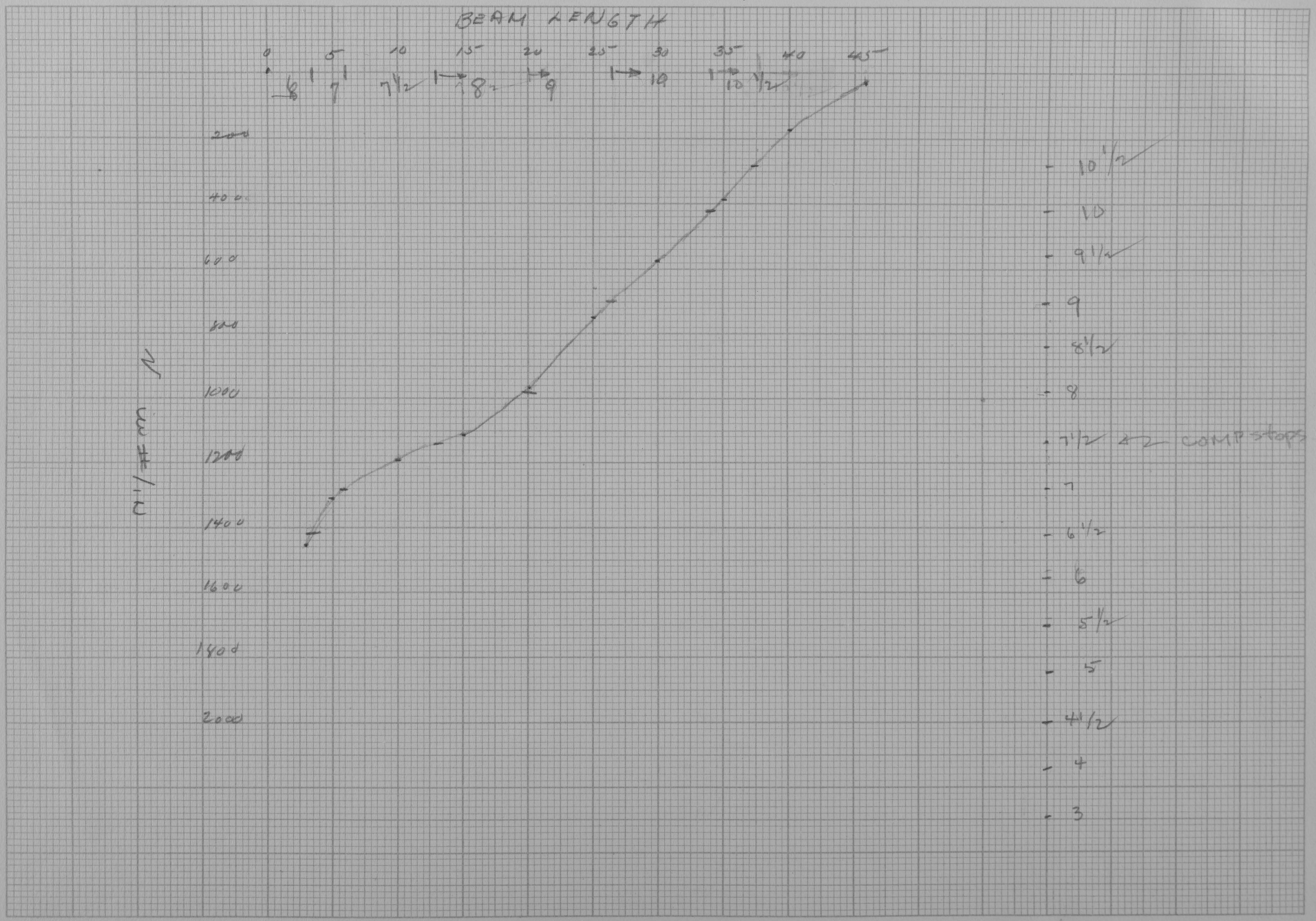
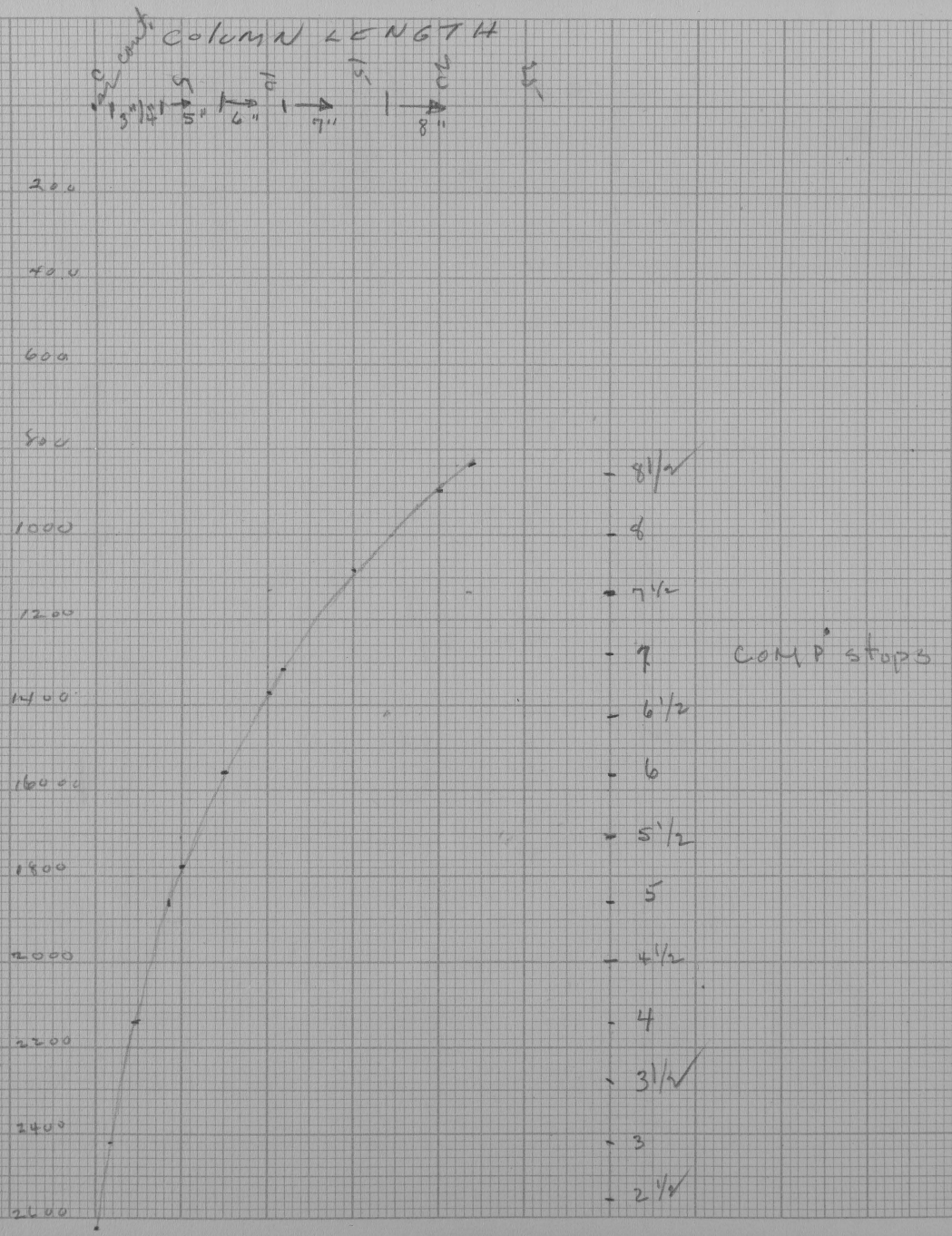

BEAM LENGTH
0
5
10
15
20
25
30
35
40
45
6
7
7 1/2
8
9
10
10 1/2
200
400
600
800
1000
1200
1400
1600
1800
2000
N cm #/in
10 1/2
10
9 1/2
9
8 1/2
8
7 1/2 ← 2 COMP stops
7
6 1/2
6
5 1/2
5
4 1/2
4
3


COLUMN LENGTH
3"
5"
6"
7"
8"
200
400
600
800
1000
1200
1400
1600
1800
2000
2200
2400
2600
8 1/2
8
7 1/2
7
6 1/2
6
5 1/2
5
4 1/2
4
3 1/2
3
2 1/2
COMP stops

$F = 13.6 \times L \times 5/16 \times .707$

$F = 3.0L$

$= 3.0 \times 1\frac{1}{2} = 4.5$

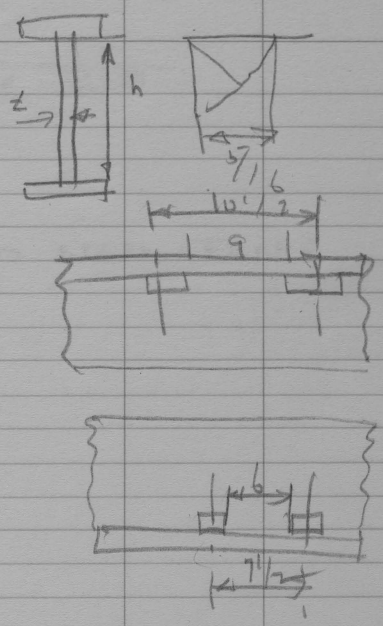
BECAUSE $2 \times 10 \times 3 = 9.0$

for $s = 4.5$

$N = \frac{9.0}{4.5} = 2$

for $s = 7.5$

$N = \frac{9.0}{7.5} = 1.265$



STIFFNER

$\frac{h}{t} = 70$

$\frac{25}{34} = \frac{l}{26.3} = l$ 19.34 Base of column

when $h = 26.3$ & greater, $\frac{h}{t}$ exceeds 70

$\frac{45.7}{34} = \frac{l}{26.3}$ $l = 34$ from ℄

$\frac{(64,000,000)}{\left(\frac{34}{3/8}\right)^2} = 7,700$

$\frac{64,000,000}{(70)^2} = 13,061$

NO STIFFNERS NEEDED

design 2 splices
~~change 1/2~~
~~1 tension member~~
~~Base plate~~
footing & pedistal
Bolts

DESIGN OF TENSION MEMBER TIE ROD

$\frac{30.075}{20} = 1.5$ USE 1-1" ϕ

DESIGN OF FOOT CONNECTION & HINGED STEEL FRAME

$R = \sqrt{62.6^2 + 30.1^2} = 69.7$

PIN:

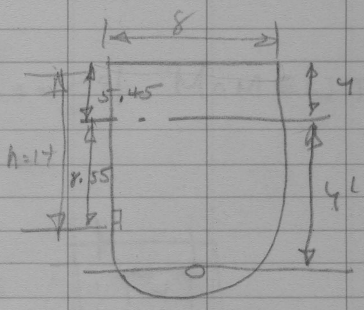
$69.7 \div 15 = 4.65$

2½" ϕ pin.

Hinge plates:

$t = \frac{69.7}{40.0 \times 2\frac{1}{2}} = .697$

USE 2 — 3/8 plates bottom & TOP.



ASSUME h

LENGTH req'd = 62.6 ÷ 2.3 = 27.25

APPROX. 2½ = say 70

16

54 ÷ 4 = say 14

$cg = \frac{8 \times 0 + (14 \times 2 \times 7)}{8 + (14 \times 2)} = \frac{196}{36} = 5.45 = y$

$\therefore y' = 14 - 5.45 + 1.75 = 6.80 = y'$

$M = 30 \times 6.80 = 204$ IN K.

$J = 1 \times 2 \left(\sqrt{4^2 + 8.05^2}\right)^2 = 161.4$

$= 1 \times 2 \left(\sqrt{4^2 + 7.05^2}\right)^2 = 131.4$

$1 \times 2 \left(\sqrt{4^2 + 6.05^2}\right)^2 = 105.2$

$1 \times 4 \left(\sqrt{4^2 + 5.05^2}\right)^2 = 166.0$

$1 \times 4 \left(\sqrt{4^2 + 4.05^2}\right)^2 = 129.6$

$1 \times 4 \left(\sqrt{4^2 + 3.05^2}\right)^2 = 101.2$

$1 \times 4 \left(\sqrt{4^2 + 2.05^2}\right)^2 = 89.2$

$1 \times 4 \left(\sqrt{4^2 + 1.05^2}\right)^2 = 68.4$

$1 \times 2 \left(\sqrt{5.45^2 + 3.5^2}\right)^2 = 84.0$

$1 \times 2 \left(\sqrt{5.45^2 + 2.5^2}\right)^2 = 72.0$

$1 \times 2 \left(\sqrt{5.45^2 + 1.5^2}\right)^2 = 64.0$

1752.4

$f_M = \frac{204 \times \sqrt{7^2 + 8.05^2}}{2304.8} .79$

$f_N = 62.6 \div 72 = .87$

1.66

MAKE h = 12

DESIGN OF LOWER HINGE PLATES

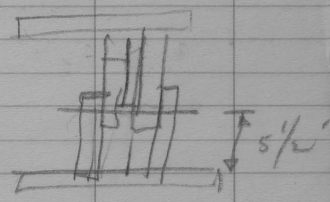
MOMENT = 30.1 x 5.5 = 165.5

$I = 4 \times 1 \times \frac{d^3}{12} + a\left(\frac{d}{2}\right)^2$

$4 \times 1 \times \frac{12^3}{12} + 2(6)^2$

$4 \times 144 + 72$

$576 + 72 = 648$



$f = \frac{165.5 \times 6}{648} = 1.53$

$f = \frac{P}{A} = \frac{30.1}{50} = .63$

2.16

MAKE 2-PLATE 3/8" with 12" CONTACT TO BASE PL

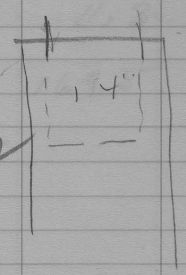
DESIGN OF Bolts to transmit shear & MOMENT from BASE plate to footing.

M = 165.5 in k

V = 30.1

all. for shear = 15,000 #

$\frac{30.1}{15} = 2.07$ — $\frac{2.07}{4} = .52$



for tension 20k/sq in

$\frac{165.5}{12} = 13.8$ kips

4 - 7/8 ϕ BOLTS

shear controls

4 - 1" Bolts

DESIGN OF TIE ROD.

$\frac{30.1}{20} = 1.57$ = USE 1 - 1¼ TIE ROD

$f_m = \frac{572 \times 16.80}{2400.6} = 4.01$

$f_N = \frac{58.9}{22} = 2.68$

$6.69 < 9.38$ o.k.

check for weld

$\frac{58.9}{2.3} = 25.6$

ABOUT 80" supplied.

$\frac{58.9}{25.6}$

MORE THAN Adequate for added moment

DESIGN OF FLANGE SPLICE.

AT FACE OF OUTER PL $f = \frac{9861 \times 24}{20420} = 11.5$

At cg OF PLs $= 11.5 \times \frac{23.5}{24} = 11.3$

total force ON PLs $= 11.3 \times 15 = 169.5$

A req'd OF Spice PL $= 169.5 \div 20 = 8.5$

USE PLATE 15 x 1

No. OF Bolts $= 169.5 \div 9.38 = 18.1$ USE 20 - 5 ROWS

MAKE BOLTED PORTION 15'- LONG

LENGTH OF WELD

$\frac{169.5}{2.3} = 73.7$

$73.7 - 16 = 57.7$

MAKE WELDED PORTION 30" LONG

DESIGN OF CENTER Splice.

M = 3,516
V = .891
N = 28.8

$I = 15(313) = 4695.0$
432.0
5127.0

$f = \frac{3,516 \times 13}{5127} = 8.9$

$M = \frac{8.9 \times 432}{26} = 148$ INK.

LENGTH OF splice pt = 22"

$I_{REQ'D} \frac{148 \times 11}{2} = 81.6$

$A_{REQ'D} = \frac{28.8}{13} = 2.22$

2 plates 3/8 x 15

DESIGN OF Bolts.

1@ 3 1/2 16.13
1@ 6 1/2 85
1@ 9 1/2 181
282.13 x 2

12 - 1" ϕ Bolts

$f_m = \frac{148 \times 13}{564.26} = 3.91$

$f_N = \frac{28.8}{6} = 4.8$

$8.71 < 9.38$ o.k

check for weld
$\frac{28.8}{2.3} = 12.5$ Supplied 3ϕ"

MORE THAN NECESSARY for moment

DESIGN of BASE PLATE.

$P = 62.6$

$F = 6$

$A = \frac{62.6}{.6} = 10.43$ sq. in. MAKE PLATE 8 x 14

$t^2 = .15\,pn = .15(6)(3)^2 = .810$

$t = .9$

MAKE BASE PLATE 8' x 14" x 1"

DESIGN OF CORNER SPLICE

$M = 9861$

$V = 58,918$

say d = 40 at ℄ of splice

web splice

$I \quad 1275(15) = 19120$

$\quad\quad\quad\quad\quad\quad 1300$

$\quad\quad\quad\quad\quad\quad 20420$

$f = \frac{9861 \times 26}{20420} = 12.55$

$M = \frac{12.55 \times 1300}{26} = 6.28$ INK.

$\begin{array}{r} 9.9 \\ 6\overline{)59} \end{array}$

LENGTH OF SPLICE PLATES 40-30

$I_{REQ'D} \frac{572 \times 20}{20} = 572$

$A_{REQ'd} \frac{58.9}{13} = 4.52$

2 PLATES 3/8 x 35 A = 26.2 I = 2679.6

DESIGN OF BOLTS

1 @	3.70	= 6.85
1 @	9.70	40.00
	15.70	123.00
	21.70	236.00
	27.70	385.00
	33.70	567.50
		1363.35

1 @	6.52	21.25
1 @	12.52	78.50
1 @	18.52	171.50
1 @	24.52	300.00
1 @	30.52	466.00
		1037.25

Flange Splice for center.

At face of outer Pl $f_2 = \frac{3,514 \times 13}{5,127} = 8.92$

At cg of outer Pl $f = 8.92 \times \frac{12.5}{13} = 8.57$

total force in 1 Pl = $8.57 \times 16.5 = 139.5$

AREA OF SPLICE PLATE = $139.5 \div 20 = 6.95$

USE PLATE 15 x 1

No. OF BOLTS. $139.5 \div 9.38 = 14.9$ USE 16 - 1" ϕ Bolts 4 ROWS.

MAKE Bolted Section 12" long

LENGTH OF WELD

$\frac{139.5}{2.3} = 60.7$

$60.7 - 15 = \frac{45.7}{2} =$ Approx - 23"

MAKE welded SECTION 23"

DESIGN OF PEDISTAL

SIZE DETERMINED BY BASE PLATE 10 X 16

TWO FT DEEP

THRUST =

FROM ACI CODE

3 - 3/4 ϕ
1/4 TIES - 12" o.c.

48
16
232 (16)

38
10
228 (14)

3'-11"

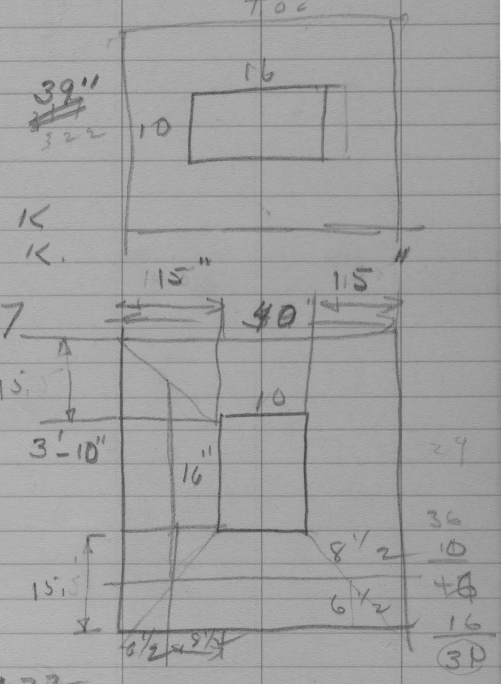
DESIGN OF Footings:-

D.L. 45.5
L.L. 17.1

D.L + 20% LL = 45.5 + 3.4 = 48.9 K
D.L + L.L. = 45.5 + 17.1 = 62.6 K.

AREA REQ'D = 48.9 ÷ 3.85 = 12.7

(3) Pr = 62.6 ÷ 13 = 4.82



short way:-

$M = 4.82 \times 3.92 \times \frac{1.25^2}{2} = 14.75$

$V_b = 4.82 \times 3.92 \times 1.25 = 23.6$

$V_v = 4.82 \times \frac{3.83 \times 3.25}{2} \times .54 = 9.22$

LONG WAY

$M = 4.82 \times 3.33 \times \frac{1.25^2}{2} = 12.5$

$V_b = 4.82 \times 3.33 \times 1.25 = 20.2$

$V_v = 4.82 \times \frac{3.34}{2} \times .54 = 7.26$

$M_{all} = 14.75 \times 3.33 = 49.2$

$v = \frac{9.22}{33 \times .866 \times 8\frac{1}{2}} = 38.4$ o.k. < 60

$v = \frac{7.26}{27 \times .866 \times 8\frac{1}{2}} = 36.6$ o.k. < 60

<u>shortway</u>

$A_s = 14.75 \times .693 \div 8\frac{1}{2} = 1.20$

$\Sigma_o = 23.6 \div (.168 \times .866 \times 8\frac{1}{2}) = 19.1$

$13 - \frac{1}{2}\phi$

LONG WAY

$A_s = 12.5 \times .693 \div 8\frac{1}{2} = 1.02$

$\Sigma_o = 20.2 \times (.168 \times .866 \times 8\frac{1}{2}) = 16.35$

$11 - \frac{1}{2}\phi$