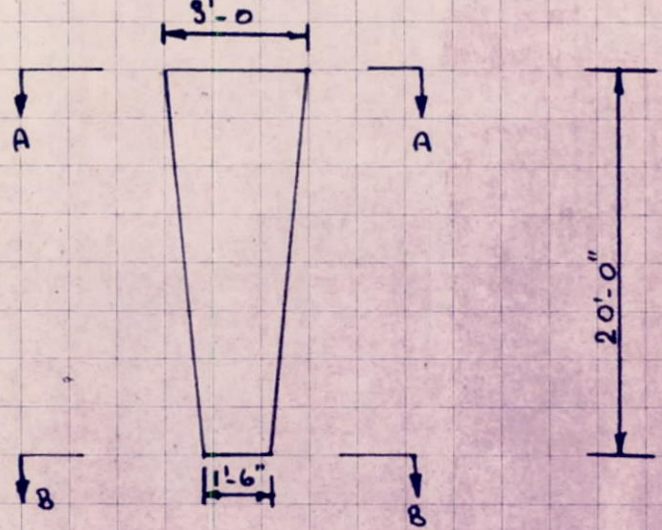
COLUMN UNITS

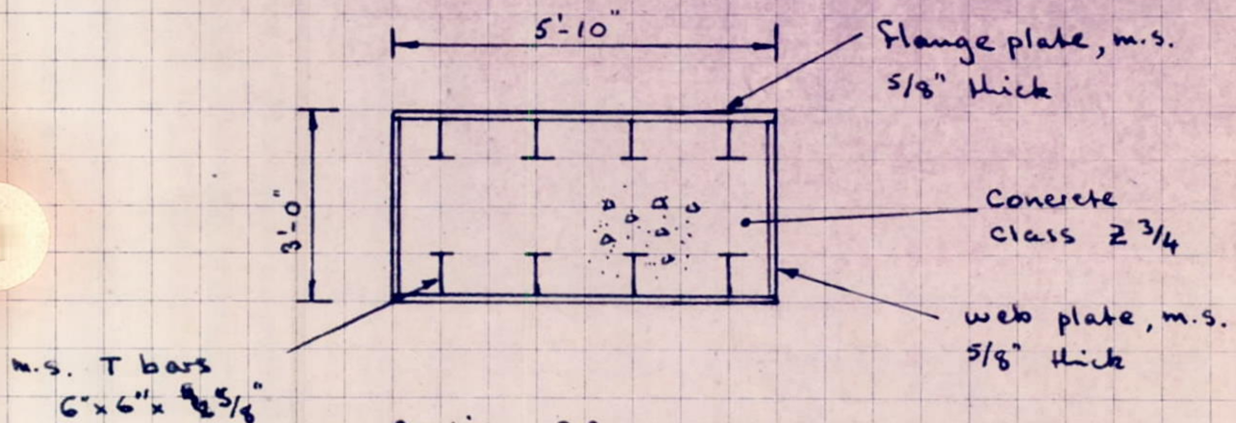
A type of steel box column with composite concrete is suggested, and section properties analysed. Forces on the column, including those due to wind are evaluated, and the column anlysed for stress conditions. The Lee McCall anchorage between column and cantilever unit is designed. Column deflections and the necessary vertical adjustments are calculated to allow for dead load deflections.

CONTENTS

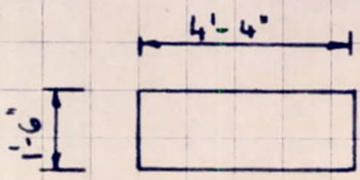
Design of Central Column - Unit 7

Preliminary calculations have suggested a column thus:-





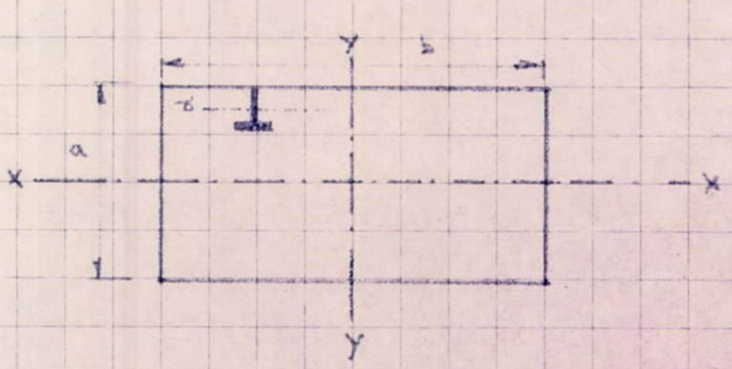
Section AA.



Section BB

otherwise similar to Section AA.

Column Section Properties



Typical cross section of column, x ft up from base.

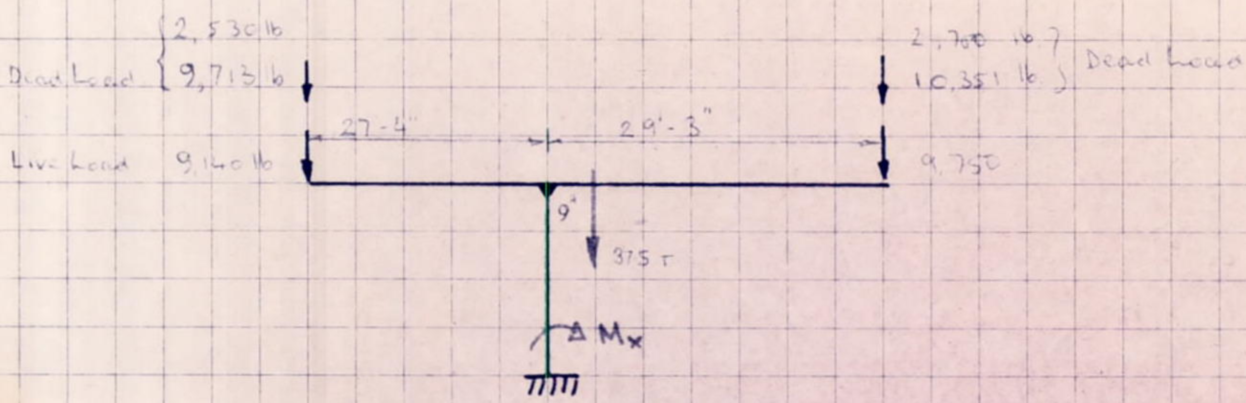
Area of T bar = 7·13"
d = 4·31

Assume composite modular ratio = 10
i.e. concrete E = 3×10^{-6} lb/in²

x ft from base	a ins	b ins	Area flange, in²	I_{xx} flange in⁴	I_{xx} T bars in⁴	Total Steel I_{xx} in⁴	Concrete I_{xx} in⁴	Total equiv. concrete I_{xx} in⁴ m = 10
0	17·5	53·0	33·125	4,893	830	5,723	18,952	76,182
2	19·35	54·7	34·1875	6,193	1,281	7,474	27,029	101,769
4	21·20	56·4	35·250	7,690	1,830	9,520	37,319	132,519
6	23·05	58·1	36·3125	9,387	2,477	11,864	50,160	168,740
8	24·90	59·8	37·375	11,297	3,221	14,518	65,920	211,100
10	26·75	61·5	38·4375	13,117	4,063	17,180	84,980	256,780
12	28·60	63·2	39·50	15,456	5,002	20,458	107,747	312,327
14	30·45	64·9	40·5625	18,041	6,010	24,051	134,652	375,162
16	32·30	66·6	41·6250	20,881	7,174	28,055	166,141	446,691
18	34·15	68·3	42·6875	23,989	8,406	32,395	202,687	526,637
20	36·0	70·0	43·750	27,374	9,736	37,110	244,783	615,883

I_{yy} at x = 2' $= 19.35 \times 5/8 \times (27)^2 \times 2$
$= \underline{17,632 \text{ in}^4}$ (Webs only)

Forces on Unit 7 due to Dead and Live Loads.



Eccentric Dead Load Moments, (including Superload)

$$M_x = -\left[12,243 \times 27.33 + \frac{166 \times (27.33)^2}{2}\right] + 375 \times 2240 \times 0.75 + 13,051 \times 29.25 + \frac{166 \times (29.25)^2}{2}$$

$$= 119,156 \text{ lb.ft}$$

$$= \underline{1,430,000 \text{ lb.in}} \quad \text{say.}$$

Live Load Moments, Long Span Loaded.

$$M_x = 9,750 \times 29.25 + \frac{(29.25)^2 \times 600}{2}$$

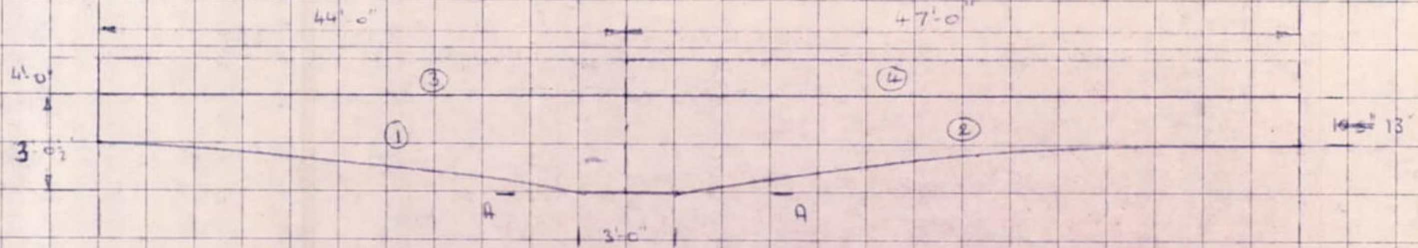
$$= 541,855 \text{ lb.ft.}$$

$$= \underline{6,502,260 \text{ lb.in}}.$$

Total possible $M_x = 6,502,260 + 1,430,000$

$$= \underline{7,932,260 \text{ lb.in}}.$$

Wind Loadings (Central portion)



Area ① $= \left(\frac{1}{3} \times 42.5 \times 36.5 \times 12 + 42.5 \times 13 \times 12 + 18 \times 36.5\right)\frac{1}{144} = \underline{93.7}\ ft^2$

Area ② $= \left(\frac{1}{3} \times 45.5 \times 36.5 \times 12 + 45.5 \times 13 \times 12 + 18 \times 36.5\right)\frac{1}{144} = \underline{100}\ ft^2$

Area ③ $= 44 \times 4 = \underline{176}\ ft^2$

Area ④ $= 47 \times 4 = \underline{188}\ ft^2$

Dist of Centroids from ℄

$\bar{x}$ of Area ①, $94.0\,\bar{x} = \left(6205 \times \frac{1}{4} \times 44 + 6630 \times 22.0 + 657 \times \frac{1.5}{2}\right)\frac{1}{144}$ $\bar{x} = 16.8$ ft

$\bar{x}$ of Area ② $100\,\bar{x} = \left(6643 \times \frac{1}{4} \times 47.0 + 7098 \times \frac{47.0}{2} + 657 \times \frac{1.5}{2}\right)\frac{1}{144}$ $\underline{\bar{x} = 17.04\ ft}$

$\bar{x}$ of Area ③ $= \frac{44.0}{2}$ $\underline{\bar{x} = 22.0\ ft}$

$\bar{x}$ of Area ④ $= \frac{47.0}{2}$ $\underline{\bar{x} = 23.5\ ft}$

Dist of Centroids from Top of Column (line AA)

$\bar{y}$ of ① $\quad \overset{78.3}{\cancel{94.0}}\,\bar{y} = \left(\frac{1}{10}\,23.5 \times 3995 + \overset{6630}{\cancel{13.0 \times 62.5}} \times 300 + \frac{36.5}{2} \times \cancel{30}\,657\right)\frac{1}{144 \times 12} \quad \bar{y} = 2.04\text{ ft.}$

$\bar{y}$ of ② $\quad 83.56\,\bar{y} = \left(\frac{1}{10} \times 23.5 \times 4277 + 70.98 \times 30.0 + 18.25 \times 657\right)\frac{1}{144 \times 12} \quad \bar{y} = 2.05\text{ ft}$

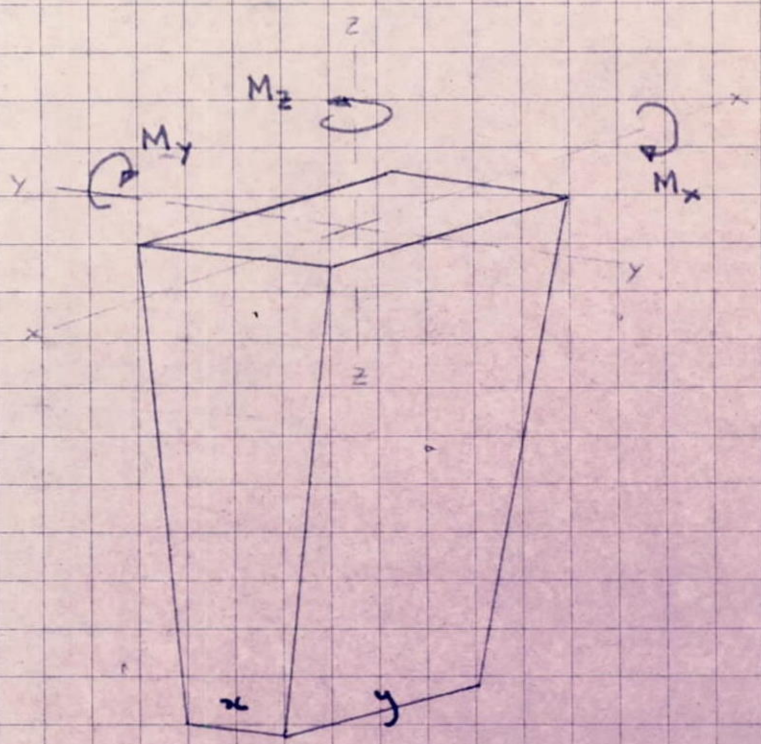
$\bar{y}$ of ③ $\quad \bar{y} = 2.0 + 3.04 \quad \bar{y} = 5.04\text{ ft}$

$\bar{y}$ of ④ $\quad \bar{y} = 2.0 + 3.04 \quad \bar{y} = 5.04\text{ ft}$

Summary.

Item	Area ft^2	Centroid to ℄ Column, $\bar{x}$ ft	Centroid to top of Column, $\bar{y}$ ft	Description of Loaded Area
①	78.3	16.8	2.04	Small Span — Superstructure
②	83.6	18.08	2.05	Large Span
③	176.0	22.0	5.04	Small Span — Live Load
④	188.0	23.5	5.04	Large Span

Forces on Unit 6 due to Wind (Central Column)



~~Mx at Base of Column~~

~~Longitudinal force~~

Lateral Wind forces

$$\text{Max} = (78.3 + 83.56 + 176 + 188) \times 15 = \cancel{7{,}890} \text{ lbf}$$
$$+ \frac{7 \times 12}{22 \times 16}(7{,}890) = 9773 \text{ lbf}$$

∴ At Base of Column

$$M_y = \cancel{7{,}890 \times 20 \times 12} = \cancel{1.90 \times 10^6} \text{ lb in}$$
$$= 9773 \times 20 \times 12 = \underline{2.35 \times 10^6 \text{ lb in}}$$

Long. Wind forces

$$\text{Max} = (78.3 + 83.56) \times 15 \times \frac{1.24}{4} + (176 + 188) \times 15 \times \frac{1}{4}$$

$$= 753 + 1365 = 2118 \text{ lbf}$$

$$M_x = 2118 \times 20 \times 12 = \underline{508{,}320 \text{ lb in}}$$

Long Span Only Loaded.

Long. Wind forces

$$M_x \simeq \frac{508,320}{2} \simeq 275,000 \text{ lb in say.}$$

M_z =

	Area	force	arm	Moment
1	78.3	1174.5	-16.8×12	$-236,779$
2	83.56	1253	$+18.08 \times 12$	271,830
4	188	2820	$+23.5 \times 12$	795,140

$$M_z = 830,311 \text{ lb in}$$

Stresses at Base of Central Column, regarding only the box section

I_{xx} box (at 2' above base for Unit 6) $= 7,474 \text{ in}^4$ (steel only)

M_x max = 7,932,260 lb.in (D.L & L.L.)
508,320 lb.in (Wind)
8,440,580 lb.in

$$\sigma_x = \frac{8,440,580}{7,474} \times 8.75 = \underline{\underline{9,881}} \text{ p.s.i.} \quad - \text{ O.K.}$$

$$\sigma_y = \frac{2.35 \times 10^6}{17,632} \times 27 = \underline{\underline{3,600}} \text{ p.s.i} \quad - \text{ O.K.}$$

By St. Venant (occurs at y_2)

$$\sigma_z = \frac{M_z\,(3y + 1.8x)}{x^2 \times y^2} = \frac{830,311\,(3 \times 54.7 + 1.8 \times 19.35)}{(19.35)^2 \times (54.7)^2}$$

$$= \underline{\underline{150 \text{ p.s.i}}} \text{ say.} \quad - \text{ O.K.}$$

Max permissible stresses in mild steel plate is 10.5 t/sq in = 23,500 p.s.i

Examination shows that no combination of stresses is going to exceed this figure, and the column unit 7, is safe in bending.

Design of Anchorage between Units 3 and 7

It is proposed to stress unit 3 to unit 7 by Lee McCall bars

Dead Load M_x = + 1,430,000 lb.in

Live Load M_x = + 6,502,260 lb.in (long span loaded)
or − 5,690,000 lb.in (short span loaded)

Long. Wind Load M_x = 508,320 lb.in

Stresses on top section of column (20'-0" up from base)

Live ~~Dead~~ Load Stresses

$$\sigma_L = \pm \frac{6,502,260 \times 12 \times 18}{70 \times (36)^3} = \pm \underline{430} \text{ p.s.i}$$

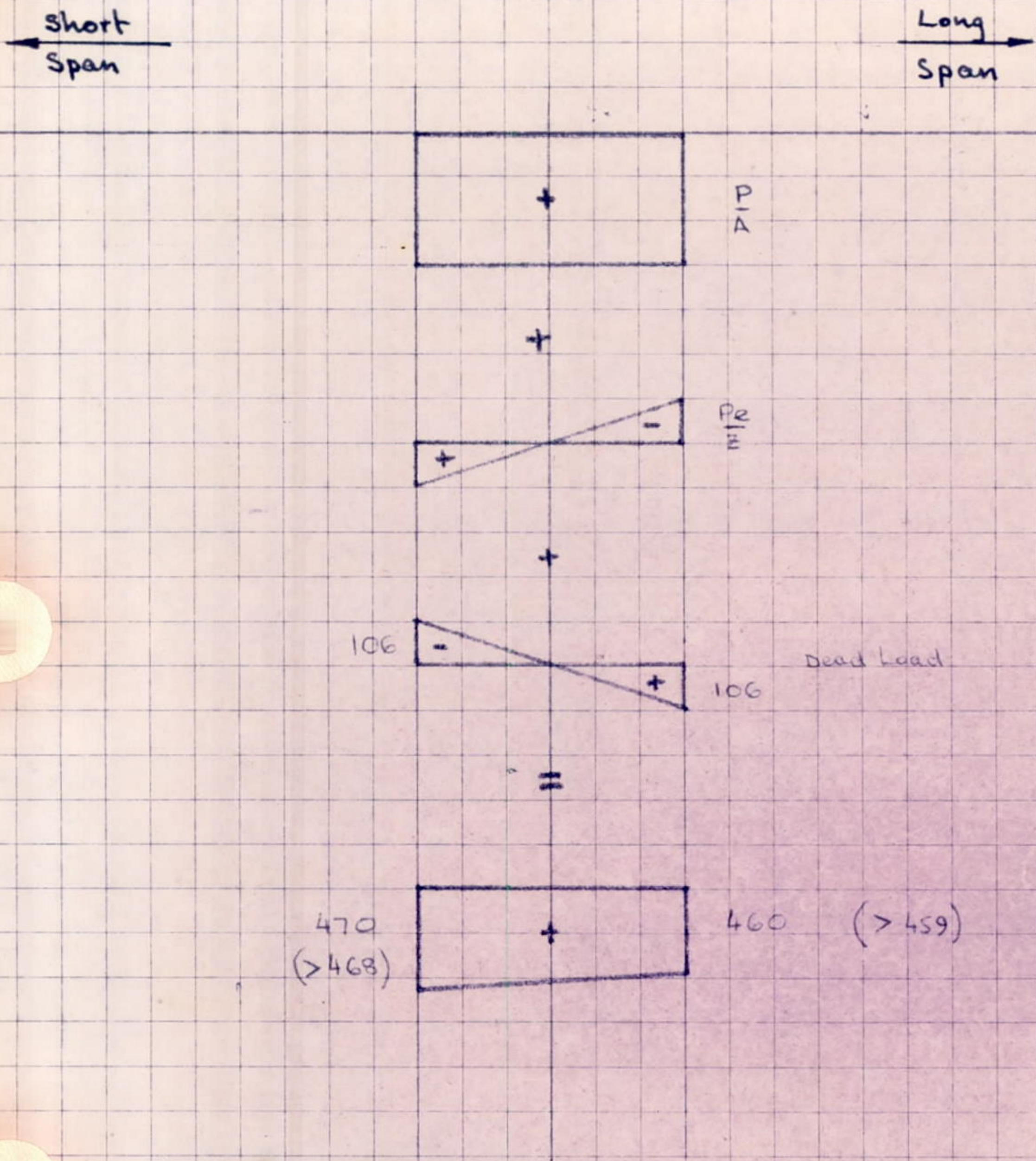
$$\text{or } \frac{5,690,000}{13,500} = \pm \underline{421}, \text{ p.s.i}$$

Wind Load $= \dfrac{508,320}{13,500} = \pm \underline{38} \text{ p.s.i.}$

Dead Load Stresses

$$\sigma_D = \pm \underline{106} \text{ p.s.i.}$$

Prestressing Diagrams



Prestressing Equations

$$\frac{P}{A} - \frac{Pe}{Z} + 106 = 460 \qquad - (1)$$

$$\frac{P}{A} + \frac{Pe}{Z} - 106 = 470 \qquad - (2)$$

[We are limited by geometrical details to e = 1.5" max.]

in (1) $\quad \frac{P}{A} - \frac{Pe}{Z} = 354$ $\qquad A = 70 \times 36 = 2,520 \text{ in}^2$

$\qquad Z = 13,500 \text{ in}^3$

$$P\left(\frac{1}{A} - \frac{e}{Z}\right) = 354 \qquad - (3)$$

ADD (1) + (2) $\quad 2\frac{P}{A} = 930$

$\underline{\underline{P = 1,172,000 \text{ lbf.}}}$ — 10 No. Lee McCall bars. $1\frac{3}{8}$" ϕ

Sub in (3) $\quad \underline{e = 1.3"}$ — O.K.

Estimate of Prestress Losses in Lee McCall Bars. (C.P. 115 & Lee McCall Design Manual)

	Loss. p.s.i.
(a) Steel creep. (Manual, Table 11)	4000
(b) Concrete Creep (") 6×460	1840
(c) Concrete shrinkage (" ") (two-3 weeks after concreting)	4,800
(d) Anchoring	0
(e) Friction loss in anchorage & Jack	0
(f) Friction in duct. $(K = 4 \times 10^{-4})(x = 30")$	0

$K.x = 4 \times 10^{-4} \times 2.5 = 10^{-3}$

Then $P_{\text{final}} = (1 - 10^{-3}) P_{\text{initial}}$

i.e. negligible (duct is also 2" wide)

(g) Elastic deformation (no overstressing procedure) $\dfrac{24 \times 10^6}{5.75 \times 10^6} \times 460$	1,920
Total Losses	12,560 p.s.i.

Initial tendon stress $= 45\ \text{T}/\text{in}^2 = 100{,}800$ p.s.i.

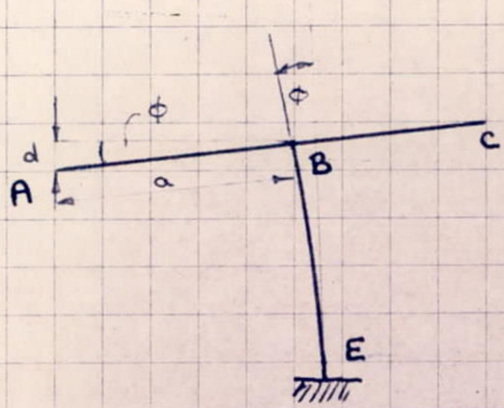
$\therefore$ % losses $= \dfrac{12{,}560}{100{,}800} = 11.6\%$

i.e. well within initial assumption of 15%. Since the concrete stresses are low there is no danger of overstressing.

(g) Saving if overstressing procedure adopted. $\dfrac{1{,}920}{100{,}800} = 1.9\%$

i.e. not necessary.

Vertical deflection of Point A due to Column Flexure

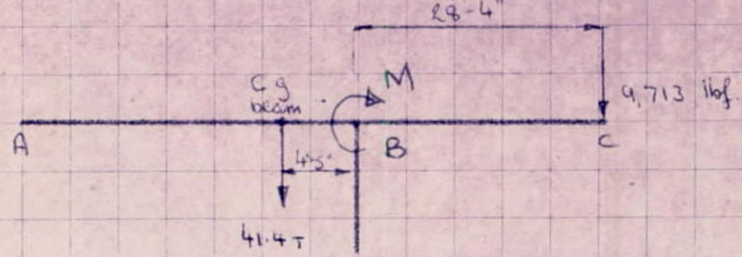


$d = a\phi$

where $a = 43'\text{-}0''$

Using Mohr's 1st Theorem of Area Moments ~ ϕ = Area $\frac{M}{EI}$ diag. BE

Moment on Column



$M = 41.4 \times 2240 \times 53 \; - \; 9{,}713 \times 28.33 \times 12$

$M = 1{,}612{,}977 \; = \; \underline{\underline{1{,}613{,}000 \text{ lb in } \text{say}}}.$

This moment is constant from top to bottom of the column, the column cross section properties are taken from the column analysis.

ϕ is evaluated in tabular form overleaf.

$M = 1,613,000$ lb in

x ft from base	Equiv Conc. I (m=10) in^4	$\frac{M}{I}$ lb/in^2	Length of Strip ins	Area Strip lb/in^2
0	76,182	21·17	12"	254·04
2	101,769	15·85	24"	380·40
4	132,519	12·17	24"	292·08
6	168,740	9·56	24"	231·60
8	211,100	7·64	24"	183·36
10	256,780	6·28	24"	150·72
12	312,327	5·16	24"	123·84
15	375,162	4·30	24"	103·20
16	446,691	3·61	24"	86·64
18	526,637	3·06	24"	73·44
20	615,883	2·62	12"	31·44
			ΣA =	1,911

Then $\phi = \frac{\Sigma A}{E_c} = \frac{1911}{3 \times 10^6} = 637 \times 10^{-6}$ rads

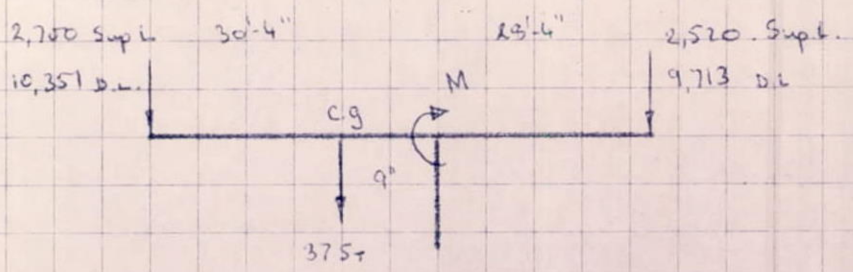
And $d = a\phi = 637 \times 10^{-6} \times 43 \times 12 = 0.329''$

Deflection of top of Column $= 0.329 \times \frac{20}{43} = 0.153''$

~~From Defln. Deg of AB and Dead loads = 0.158"~~
~~Then column deflection = 0.158" x 20/43 = 0.073"~~

We require to set the column approx. $\frac{5}{32}''$ out of plumb.

Eccentric D.L. B.M. on Central Cantilever – Unit 3



Superload is included in this calculation because this frame remains statically determinate for both Dead Load and Live Load.

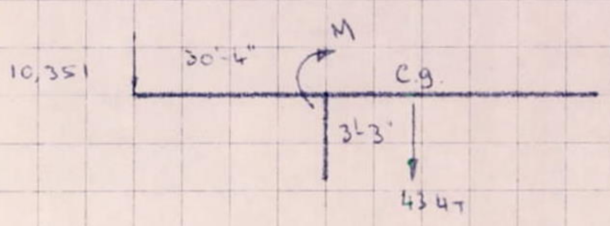
$$M = 13{,}051 \times 30.33 + 37.5 \times 2240 \times 0.75 + 166 \times 30.33 \times 15.18 - 12{,}233 \times 28.33 - 166 \times 28.33 \times 14.18$$

$$M = 122{,}018 \text{ lb ft} = 1{,}464{,}216 \text{ lb in}$$

Likely column deflection is $\frac{1{,}464{,}216}{1{,}613{,}000} \times 0.153 = 0.1389''$

Column Set Say $\underline{\underline{1/8''}}$ out of Plumb

Eccentric D.L. B.M on Unit 5



$$M = 43.4 \times 2240 \times 39 - 10{,}351 \times 30.33 \times 12$$

$$M = 24{,}074 \text{ lb in}$$

Likely column deflection is $\frac{24{,}000}{1{,}613{,}000} \times 0.153 = 0.0023''$

<u>Insignificant</u>