CIVIL ENGINEERING DEPARTMENT

THESIS

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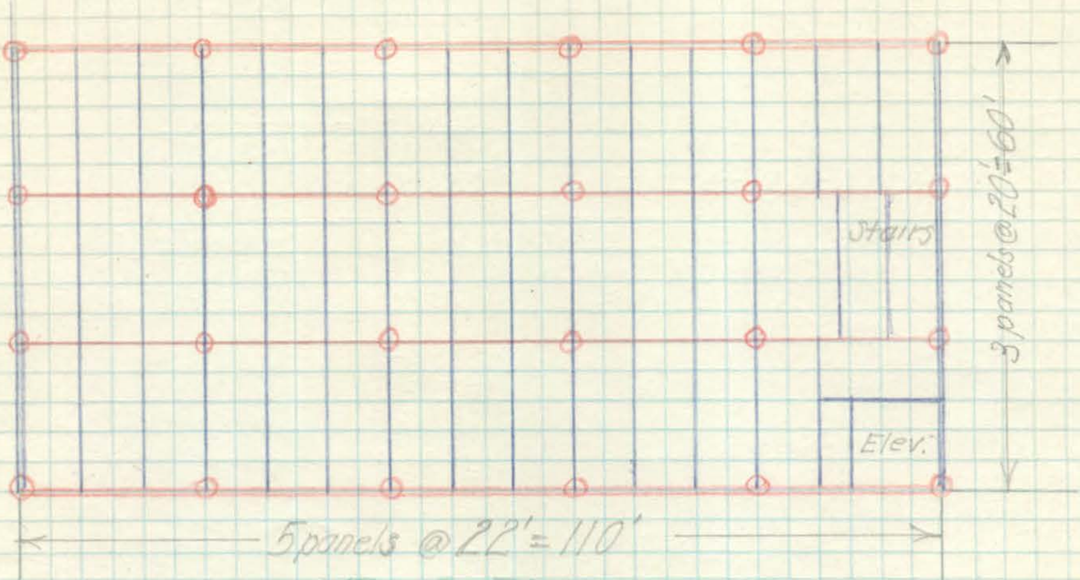
June 9, 1924.

"STRESS & LOAD CALCULATIONS for
UNKNOWN BUILDING"

INDEX

Floor System



Roof Plan

The elevator shaft is to be 10'x10' and the distance c-c of beams will be made $10\frac{1}{2}$' x $10\frac{1}{2}$'

The stairs will be considered to take a width of 12' and a length of 1 panel.

The panels will be reduced to $19\frac{3}{4}$' & $21\frac{3}{4}$' c-c by the width of the outer walls.

The floors and fireproofing will be made of concrete.

Wind Stress

Basement assumed 6' below ground level
2' fire wall at top

Effective height for wind pressure

$= 10 + 14 + 12 \times 2 + 2 - 6 = 44'$

Total pressure on side $= 44 \times 110 \times 30 =$ 145,200#

Pressure per vertical foot on one column assuming end columns to take as much as the center columns

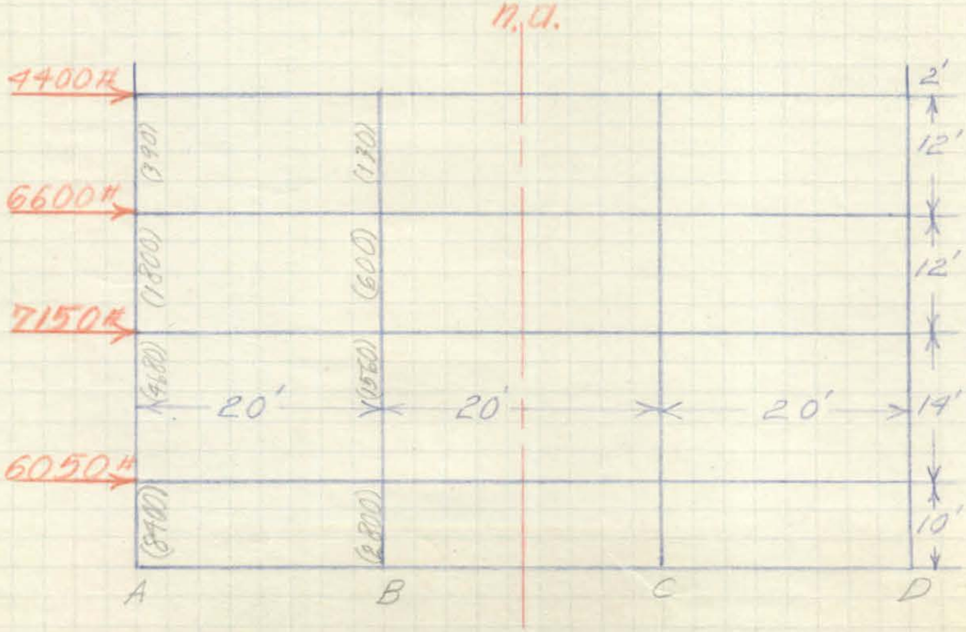
$= \frac{145,200}{6 \times 44} = 550\#$ 550#

Pressure at 1st floor $= 550 \times 11 =$ 6050#

" " 2nd floor $= 550 \times 13 =$ 7150#

" " 3rd floor $= 550 \times 12 =$ 6600#

Pressure " roof $= 550 \times 8 =$ 4400#



Wind Stress

Cantilever Method

3rd Floor

m_3 = stress in $B_3 + C_3$ + $3m_3$ = stress in $A_3 + D_3$

$2 \times 30 \times 3m_3 + 2 \times 10 \times m_3 = 4400 \times 6 = 26{,}400$

$200 m_3 = 26{,}400 \qquad m_3 = 132.0$

m_3	130
$3m_3$	390

2nd Floor

$200 m_2 = 4400 \times 18 + 6600 \times 6 = 118{,}800$

m_2	600
$3m_2$	1800

1st Floor

$200 m_1 = 4400 \times 31 + 6600 \times 19 + 7150 \times 7 = 311{,}500$

m_1	1560
m_1	4680

Basement

$200 m_b = 4400 \times 43 + 6600 \times 31 + 7150 \times 19 + 6050 \times 5 = 559{,}000$

m_b	2800
$3m_b$	8400

Shear

3rd Floor

Let S_2 = shear at inflection pt. of 2nd floor column
Inflection pt. ½ way up column

$\therefore S_3 = \frac{4400}{11000} S_2 + \frac{6600}{11000} S_2$ = shear taken by column at 3rd floor

Shear in girder $3_{A-B} = 1800 - 390 =$ 1410

$.4 S_{2A} \times 12 + .6 S_{2A} \times 6 = 1410 \times 10 = 14{,}100$

$\therefore S_{A2} = \frac{14100}{8.4} =$ 1680#

$S_{A3} = .4 S_{A2} =$ 680#

$.6 S_{A2} =$ 1000#

Shear in girder $3_{B-C} = 1410 + 600 - 130 =$ 1880

$8.4 S_{B2} = 18{,}800 + 14{,}100 = 32{,}900$

$S_{B2} =$ 3920

$S_{B3} = .4 S_{B2} =$ 1570

$.6 S_{B2} =$ 2350

Moments

Girder A-B (3) direct stress = 6600 - 1000 = 5600#
" " moment = 1410 × 10 = 14,100 ft.-lbs
Column A (3-4) " = 680 × 6 = 4080 ft.-lbs
" A (2-3) " = 1680 × 6 = 9780 ft.-lbs.

Girder B-C (3) direct stress = 5600 - 2300 = 3300#
" " moment = 1880 × 10 = 18,800 ft.-lbs
Column B (2-3) " = 3920 × 6 = 23,520 ft.-lbs
" B (3-4) " = 1570 × 6 = 9,420 ft.-lbs.

Wind Stress

2nd Floor

Let S_1 = shear at mid-point of 1st story column.

$S_2 = \frac{11}{18} S_1$ + Shear at 2nd floor $= \frac{7}{18} S_1$

Shear

Shear in girder 2_{A-B} = 4680 − 1800 = 2880

$\frac{11 \times 13 + 7 \times 7}{18} S_{A_1} = 28,800$ $\qquad S_{A_1} = 2700$

$\frac{11}{18} S_{A_1} = S_{A_2} = 1650$

$\frac{7}{11} S_{A_1} = 1050$

Shear in girder 2_{B-C} = 960 + 2880 = 3840

$\frac{192}{18} S_{B_1} = 38400 + 2880$ $\qquad S_{B_1} = 6300$

$\frac{11}{18} S_{B_1} = 3850$

$\frac{7}{18} S_{B_1} = 2450$

Moments

Girder A-B(2) direct stress = 7000 − 1050 = 5950 #
" " moment = 2880 × 10 = 28,800 ft. lbs.
Column A(2-3) " = 1650 × 6 = 9,900 ft. lbs.
" A(1-2) " = 2700 × 7 = 18,900 ft.-lbs.

Girder B-C(2) direct stress = 5950 − 2450 = 3500 #
" " moment = 3840 × 10 = 38,400 ft.-lbs.
Column B(2-3) " = 3850 × 6 = 22,300 ft.-lbs
" B(1-2) " = 6300 × 7 = 44,100 ft.-lbs

Basement

S_b = shear in basement columns.

$S_1 = \frac{18000}{24000} S_b$ = shear in col. $\frac{b}{24}$ = shear at girder.

Shear in girder A-B (b) = 3700

$\frac{3 \times 12 + 1 \times 5}{4} S_{A_b} = 37,000$ $\qquad S_{A_b} = 3600$

$\frac{3}{4} S_{A_b} = 2700$

$\frac{1}{4} S_{A_b} = 900$

Shear in girder B-C(b) = 3700 + 1300 = 5000

$4\frac{1}{4} S_{B_b} = 37000 + 50,000$ $\qquad S_{B_b} = 8500$

$\frac{3}{4} S_{B_b} = 6400$

$\frac{1}{4} S_{B_b} = 2100$

Moments

Girder A-B(1) direct stress = 6000 − 900 = 5100 #
" " moment = 3700 × 10 = 37,000 ft.-lbs.
Column A(1-2) " = 2700 × 7 = 18,900 ft.-lbs
" A(b-1) " = 3600 × 5 = 18,000 ft.-lbs.

Girder B-C(1) direct stress = 5100 − 2100 = 3000 #
" " moment = 5000 × 10 = 50,000 ft.-lbs.
Column B(1-2) " = 6400 × 7 = 44,800 ft.-lbs
" B(b-1) " = 8500 × 5 = 42,500 ft.-lbs.

Wind Stress

Roof

Shear is all taken by roof

shear in girder A-B = 390#

$S_r = \frac{0}{4400} = 0$

$S_{A_3} =$ 680

$S_{B_3} =$ 1570

Shear in girder B-C = 390+130 = 520

Moments

Girder A-B(r) direct stress = 4400 − 700 = 3700 #

" " moment = 390 × 10 = 3900 ft.-lbs.

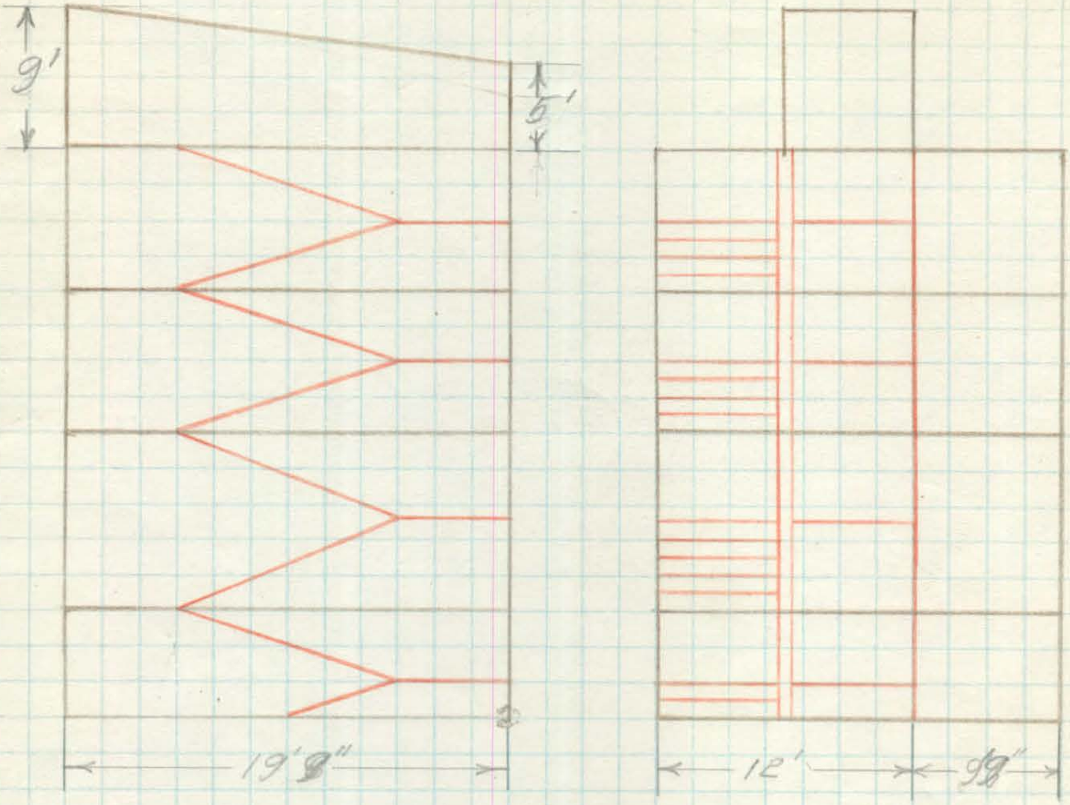
Column A(3-r) " = 680 × 6 = 4080 ft.-lbs

Girder B-C (r) direct stress = 3700 − 1500 = 2200 #

" " moment = 910 × 10 = 9100 ft.-lbs

Column B(3-r) " = 1570 × 6 = 9420 ft.-lbs.

Stairway - Outline



Pent-house roof

Use concrete slab. Try min. allowed by L.A. Ord.

$d = 2''$ $t = 3''$ $wt = 30 + 35 = 65\ \#/ft^2$

$M = \frac{1}{8} \times 66 \times (6.5)^2 \times 12 = 4120$

$\frac{M}{bd^2} = \frac{4120}{48} = 86$

for $f_s = 16000$ $f_c = 570$ + $K = 86$ (diagram)

$p = .0061$ use $p = .0065$

$A_s = .0065 \times 2 \times 12 = .163$

Use $\frac{1}{4}''$ $\boxed{S}$ $4\frac{1}{2}''$ c-c $A_s = .167$

Use $\frac{1}{4}''$ $\boxed{S}$ $12''$ c-c other direction.

Stairway Penthouse

Beams

load per ft. $= 65 \times \frac{6.5}{2} = 210\#$ span $= 19'9"$

Try 8"–18# I-beam. | 8"–18# I

$I = 56.9$. $A = 5.33 \square"$ web $t = .27$ $\frac{I}{c} = 14.2$

Fireproofing $= (8+2)(4+4) = 80\square"$

$80 - 5.33 = 74\square"$ $\frac{74}{144} \times 150 = 77\#/ft.$

$W = 210 + 18 + 77 = 305\#$

$M = \frac{1}{8} \times 305 \times 400 \times 12 = 183{,}000$ in-lbs.

$\frac{Mc}{I} = \frac{183000}{14.2} = 12{,}900\#/in^2$ stress in beam.

$v = \frac{305 \times 10}{.27 \times 8} = 1360\#/in^2$

This is safe and beam will be used.

A 3"–5.5# I beam is strong enough for the short span. | 3"–5.5# I

Wt of beam and fireproofing $= 35\#$

5"x5" = outside of f.p.

Columns

Wt. on one column $= 305 \times 10 + 35 \times 3 = 3150\#$

$L = 9'$

Use 5"–9.75# I-bar – larger than necessary but the size is needed for riveting of details. | 5"–9.75# I

Size of f.p. $= (5+4) + (3+4) = 9" \times 7" = 63\square"$

$63 - 3 = 60\square"$ $60 \times \frac{150}{144} = 63\#$

Total wt. $= 63 + 10 = 73\#/ft.$

Wt. from front columns = 3800# apiece.

" " back " = 3500# "

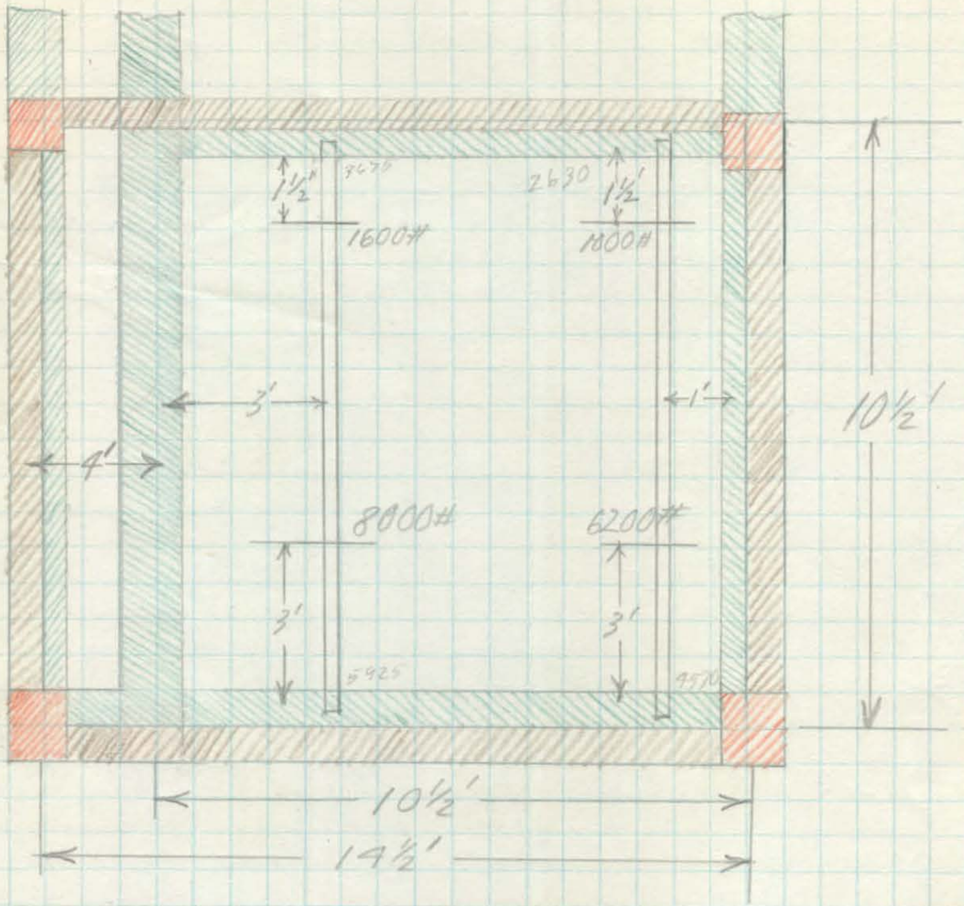
Wt. of each side wall $= 19 \times \frac{4+8}{2} \times 41 = 4670\#$

8" tile walls – plastered both sides wt = 41

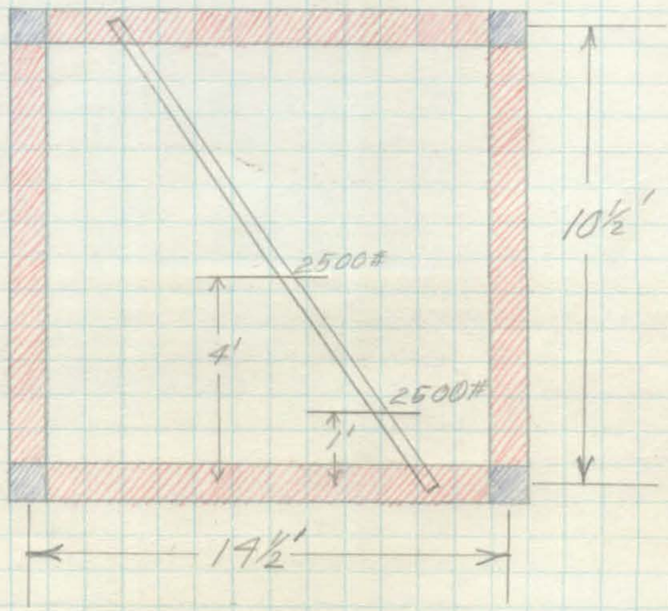
Wt. of rear wall $= 5 \times 5 \times 41 = 1025\#$

Elevator Penthouse

Plan at Roof



Plan 7' above Roof



Elevator Penthouse

Roof

Approximately $10\frac{1}{2}' \times 14\frac{1}{2}'$
Use $5\frac{1}{4}'$ panels.
Assume wt. = 35# 30# L.L.

$M = \frac{1}{10}\, 65 \times (5.25)^2 \times 12 = 2160$ in. lbs.

Use 3" slab $d = 2"$ — $d = 2"$, $h = 3"$

$K = \frac{2160}{4 \times 12} = 45$ Use $p = .5\%$ (excess) — $\frac{1}{4}"$ ◻ $6"$ c-c

$A_s = .005 \times 24 = .120$
use $\frac{1}{4}"$ ◻ $6"$ c-c and $\frac{1}{4}"$ ◻ $12"$ c-c other direction. — $\frac{1}{4}"$ ◻ $12"$ c-c

Bend up ½ of bars over support.
" " all bars at end " . and place same amount of stell in bottom

Cross beams at Roof of Penthouse

Load $= 65 \times 5.25 = 340$ # l.s. Span = 14.5'
Try 6"-12.25# I-beam. $S = 7.3$ in^3 — 6"-12.25#

$\frac{6 + 2 \times 6 - 4}{144} \times 150 = 47$#

wt. of f.p. + beam = 60# $w = 60 + 340 = 400$# — wt. = 60#/ft, L = 340#/ft

$M = \frac{1}{8} \times 400 \times (14.5)^2 \times 12 = 126,300$ in.-lbs

$S = \frac{126,300}{7.3} = 17,300$#/in^2

over stress but will be used since likelyhood of full load on roof is small.

Use 5"-9.75# beams at end as moment is about ½ as much $S = 4.8$ — 5"-9.75#

wt. f.p. $= \frac{7 \times 5 - 3}{144} \times 150 = 33$# — wt. = 43#

Total wt. of section = 43#

Girder

load $= 400 \times \frac{14.5}{2} = 2900$# at center.

$M_e = 1450 \times 6.25 \times 12 = 91,500$ in.-lbs.

Try 6"-12.25# beam $S = 7.3$ — 6"-12.25#

wt. f.p. + beam = 60# — wt. = 60#/ft

$M_G = \frac{1}{8} \times 60 \times (10.5)^2 \times 12 = 10000$

Total $M = 101,500$

$S = \frac{101,500}{7.3} = 14,000$#/in^2

This beam will be used

Elevator Penthouse

Beams 7' from Roof of Bldg.

2 beams $14\frac{1}{2}'$ long needed.

load = approx. 3500# 5' from end.

$R = \frac{9.5}{14.5} \times 3500 = 2300\#$

Equiv. load $= \frac{8 \times 2300 \times 5}{(14.5)^2} = 438.5\ \#/ft.$

Walls of 12" brick plastered inside.

Wt. $= 120 \times 2\frac{1}{3} = 280\ \#/ft.$

$W = 430 + 280 = 710\ \#/ft$

Try 9"-21# I-beam. $A = 6.31\square''$ $S = 18.9\ in^3$ — 9"-21# I

Wt. f.p. $= \frac{(9+2)(7)-7}{144} \times 150 = 73\#$ $W = 94\#$ — Wt. = 95# /ft.

Load $= 710 + 94 = 805\ \#/ft.$

$M = \frac{1}{8} \times 805 \times (14.5)^2 \times 12 = 243{,}000$ in.-lbs.

$S = \frac{243{,}000}{18.9} = 12{,}850\ \#/in^2$ safe & will be used.

Columns

Load at roof$_{(a)}$ $= 65 \times 5.25 \times 7.25 + 43 \times 7.25 + \frac{60 \times 7.25}{2} + 60 \times 5.25 = 3325\#$

Load 7' above Roof$_{(b)}$ $= 2300 + 280 \times 7.25 + 95 \times 7.25 = 5020\#$

Total load = 8350# for most stressed column. — 6"-12.25#

Wt. = 90#/ft.

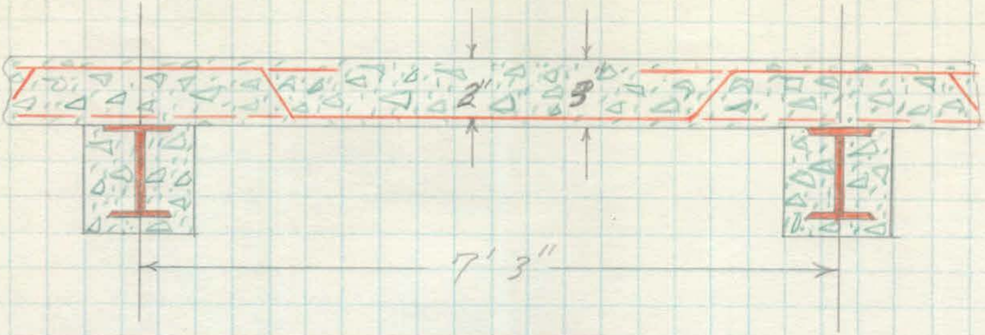
Use 6"-12.25# I-beam

Allowable load = 16.8 for 10' length.

Wt. $= \frac{(6+4)8-4}{144} \times 150 + 12 = 90\ \#/ft.$

Wt. of wall $= 6 \times 120 = 720\ \#/ft.$

Roof Slab



Fully Continuous

Load = 30#/ft. assume wt. = 35#/ft.

$M = \frac{1}{12} w l^2$

Use d = 2" t = 3" (L.A. Ord. min.)

$M = \frac{1}{12} \times 65 \times (7.25)^2 \times 12 = 3420$ in.-lbs

$\frac{M}{bd^2} = \frac{3420}{48} = 71.3$ $\boxed{f_s = 16000 \quad f_c = 500}$

$k = .319$ $j = .894$ $p = .005$ (from tables)

$A_s = 12 \times 2 \times .05 = .120$ □"

Use 1/4" [S] 6" c-c $a_s = .125$

" 1/4" [S] 18" c-c other direction

$u = \frac{3.62 \times 65}{.894 \times 2 \times 2} = 66$ #/in²

Bend 1/2 of bars 1'9" from ℄ and extend <u>all</u> bars 2' past ℄.

$M = \frac{1}{12} w l^2$
d = 2"
h = 3"
1/4" [S]
6" c-c
+ 18" c-c

End Span.

$M = \frac{12}{10} \times 3420 = 4100$ in-lbs.

$\frac{M}{bd^2} = 85.4$ for $f_s = 16000$, $f_c = 560$ $j = .885$

$p = .0060$ tables

$A_s = .0060 \times 12 \times 2 = .144$ □"

Use 1/4" [S] 5" c-c

" 1/4" " 18" c-c other direction.

$u = \frac{362 \times 65}{.885 \times 2 \times 2} = 66.5$ #/in².

Bend all bars 1'9" from end ℄ and extend 2' in wall or hook.

Add 1/4" [S] 5" c-c in bottom extending 2' either way from ℄ or hooked

$M = \frac{1}{10} w l^2$
d = 2"
h = 3"
1/4" [S]
5" c-c
+ 18" c-c

Roof - Cross - beams

Typical

$Load = (30+35)7.25 = 472\#/ft.$

$Span = 19.75'$

Try 10"-25# I-beam $A = 7.37$ w of flange = 4.66

$S = 24.4$

$Wt. \ of \ f.p = \frac{8\times(10+2)-7}{144}\times 150 = 94\#/ft.$

$Total \ w = 472+94+25 = 590\#/in^2$

$M = 1/8\times 590(19.75)^2\times 12 = 346,000 \ in. \ lbs.$

$f_s = \frac{346,000}{24.4} = 14,200\#/in^2$ (satisfactory)

$V = 5850\#$

10"-25#
$f = 14,200$
wt = 125#/ft

1st beam from stairs

$Load = 65\times(\frac{7.25}{2}+\frac{9.75}{2}) = 552\#/ft.$

$span = 19.75'$

Try 10"-30# I-beam. $s = 26.8$

$wt. \ of \ f.p. = 94\#/ft.$

$W = 552+94+30 = 676\#/ft.$

$M = 346,000\times\frac{676}{566} = 413,000 \ in\text{-}lbs.$

$f = \frac{413,000}{24.4} = 15,400\#/in^2$

$V = 6660\#$

10"-30#
$f = 15,400$
wt = 100#/ft

Beams at stairs

$Uniform \ load = \frac{9.75}{2}\times 65 + 4\times 41 = 482\#$

load of 164# 1 end @ other end.

$M = \frac{1}{8}\times 482\times(19.75)^2\times 12 + \frac{164\times\frac{19.75}{2}}{6\times 19.75}\left[(19.75)^2-(\frac{19.75}{2})^2\right]12 =$

$= 282,000 + 48,000 = 330,000 \ in\text{-}lbs.$

Try 10"-25# I-beam. $s = 24.4$

$wt. \ f.p. = \frac{7(10+1)-9}{144}\times 150 = 70\#/ft.$

$M = 282,000\times\frac{(70+25)}{482} = 55,500 \ in.\text{-}lbs.$

$Total \ M = 385,500 \ in.\text{-}lbs.$

$f = \frac{385,5000}{24.4} = 15,800\#/in^2$

$V = 7800\#$

10"-25#
$f = 15,800$
wt = 95#/ft

Roof Beams

Spandrel $Load = 2 \times 125 + \frac{7.26}{2} \times 66 = 495 \#/ft.$

Try 10"-25# I-beam $A = 7.37"$ $S = 24.4$ — 10"-25#, $f = 14,300 \#/in^2$, wt. = 100 #/ft

$wt.$ of fireproof $= \frac{7(10+1)-7}{144} \times 150 = 73 \#/ft.$

$wt. = 73 + 25 + 495 = 595 \#/in^2$

$M = \frac{1}{8} \times 595 (19.75)^2 12 = 348,000$ in.-lbs

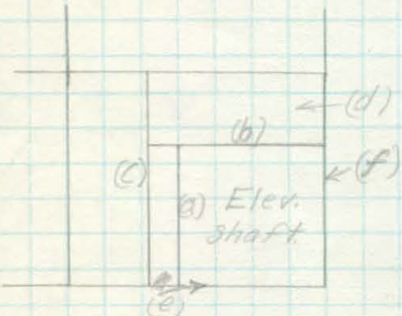
$f = \frac{348,000}{24.4} = 14,300$ in.-lbs.

$V = 5900 \#$

Beams at Elevator (a) Span = 10.5. Use 6"-12.50# I-beam. (from tables) — 6"-12.5# I

Wt. f.p. $= \frac{8 \times 7 - 3}{144} \times 150 = 55 \#$ — wt. = 67.5 #/ft

Total wt. = 67.5 #/ft.



(e) continue 3" slab - already designed.

(d) size $= 14.5' \times 9.25'$ $\frac{l}{b} = 1.57$

$(9.25)^4 = \overset{w_1}{7350}$ $(14.5)^4 = \overset{w_2}{44,500}$

$\frac{w_1}{w_2} = \frac{44,500}{7350}$ or $w_1 = 6.06 \, w_2$

$\therefore w_1 = 85\%$ $w_2 = 15\%$

~~assume $d = 2.25"$ $t = 3"$ $w = 37\#/ft.$~~

~~$M_1 = \frac{1}{8} .85 \times 67 (9.25)^2 12 = 7320$ in.-lbs.~~

Superseded

$d^2 = \frac{7320}{12 \times 107.5} = 5.67$ $d = 2.38$

take $d = 2.5"$ $t = 3.25"$

$M = 7320 \left(\frac{73}{67}\right) = 7980$ in.-lbs.

$\frac{M}{bd^2} = 105.5$ use $p = .0077$

$a_s = .0077 \times 12 \times 2.5 = .231$ Use 3/8" ⊡ 7" c-c $a_s = .240$

$M_2 = \frac{1}{8} \times .15 \times 73 (14.5)^2 12 = 3450$ in.-lbs.

$d^2 = \frac{3450}{12 \times 107.5} = 2.67$ $d = 1.63"$ Make $d = 1.75"$

$\frac{M}{bd^2} = \frac{3450}{12 \times 3.06} = 94$ Make $p = .007$ $a_s = .007 \times 12 \times 1.75 = .147$

Use 3/8" @ 9" c-c $a_s = .147$

Roof Beams

At Elevator (d)

14.5x9.25 $\ell/b = 1.57$

Reinforce 1 direction only

Assume wt. $= \frac{3.5}{12} \times 150 = 43\#/ft^2$

$M = \frac{1}{8} \times 73 \times (9.25)^2 \times 12 = 9400$ in-lbs.

$d^2 = \frac{9400}{12 \times 107.5} = 7.28$ $d = 2.7''$

make $d = 2.75''$ $t = 3.5''$ $p = .0077$

$a_s = .0077 \times 2.75 \times 12 = .254\,\square''$

Use 3/8" ⊡ 6½" c-c $a_s = .259\,\square''$

Use 1/4" ⊡ 12" c-c other direction

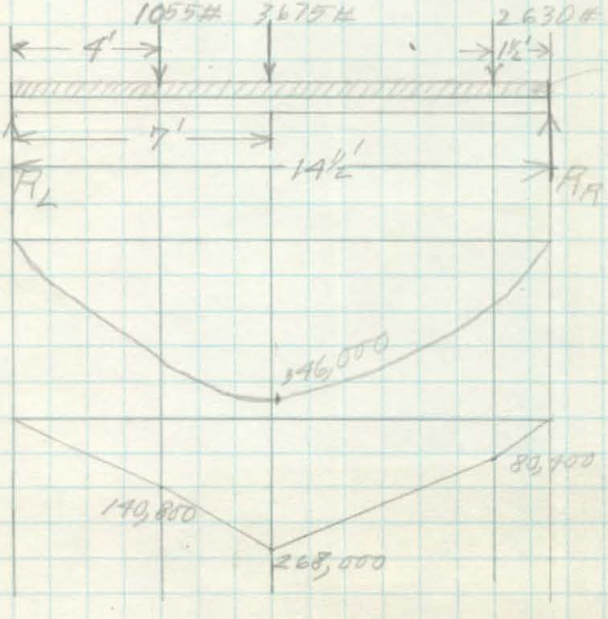
$d = 2.75''$
$t = 3.5''$
$a_s = 2.59\,\square''$
3/8" ⊡ 6½" c-c
1/4" ⊡ 12" c-c
wt = 73#/ft.

Beam (b)

Load $= \frac{9.25}{2} \times 73 + 6 \times 125 = 1100\ \#/ft.$

Load due to beam (a) $= (2 \times 67 + 67)\frac{10.5}{2} = 1055\#$

Other load = 3675# + 2630#



$M_U = \frac{12}{8} \times 1100 \times (14.5)^2$
$= 346,000$ in-lbs

$R_{LC} = \frac{10.5}{14.5} \times 1055 + \frac{7.5}{14.5} \times 3675 + \frac{1.5}{14.5} \times 2630 = 2935\#$

Total M = 614,000 in-lbs

Try 12"-40.8# $S = 44.8\ in^3$ $A = 11.84''$

Wt. of f.p. $= \frac{9 \times 14 - 12}{144} \times 150 = 109\#$

$M = 346,000 \times \frac{150}{1100} = 47,200$

Total M = 661,200 in.-lbs.

$f = \frac{661,200}{44.8} = 14,750\ \#/in^2$

$V = \frac{13,500}{.46 \times 12} = 2450\#/in^2$

$R_L = 2935 + 1250 \times 7.25 = 12,000\#$ $R_R = 13,500$

12"-40.8#
f = 14,750
Wt = 150#/ft.

Roof Beams

(c) at Elevator

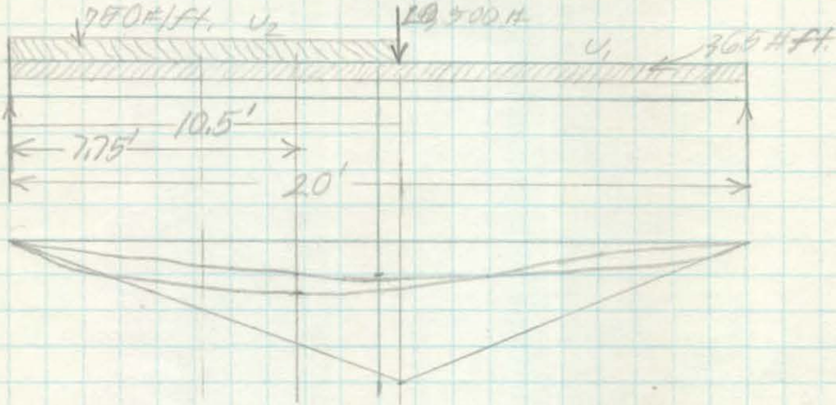
Slab load = $65 \times 5.62 = 365$ #/ft.

wall load = $6 \times 125 = 750$ #/ft.

Beam (b) load = 12,000 #

Column load = 8500 #

20,500 #



$M_{U_1} = \frac{1}{8} \times 365 \times (20)^2 = 219,000$ in.-lbs.

U_2 — $x_0 = \frac{14.75}{20} \cancel{\times \frac{750 \times 10.5}{750}} = 7.75'$

$M_{U_2} = [6810 \times 7.75 - 375(7.75)^2]12 = 364,000$ in lbs.

$M_c = 20500 \times \frac{9.5}{20} \times 10.5 \times 12 = 1,225,000$ in.-lbs.

Total M = approx. 1,725,000 in.-lbs.

Try 20" – 59.5 # Beth. I-beam

$S = 117.2$
$A = 17.36$
$t_{web} = .375$
8" flange

wt. f.p. $= \frac{22" \times 12 - 17}{144} \times 150 = 258$ #/ft.

60

320 #/ft.

$M = \frac{320}{365} \times 219000 = 192,000$ in lbs.

Total M = 1,917,000 in.-lbs.

$f = \frac{1,917,000}{117.2} = 16,300$ #/in²

$R_L = 23,400$ # $R_R = 18,800$ #

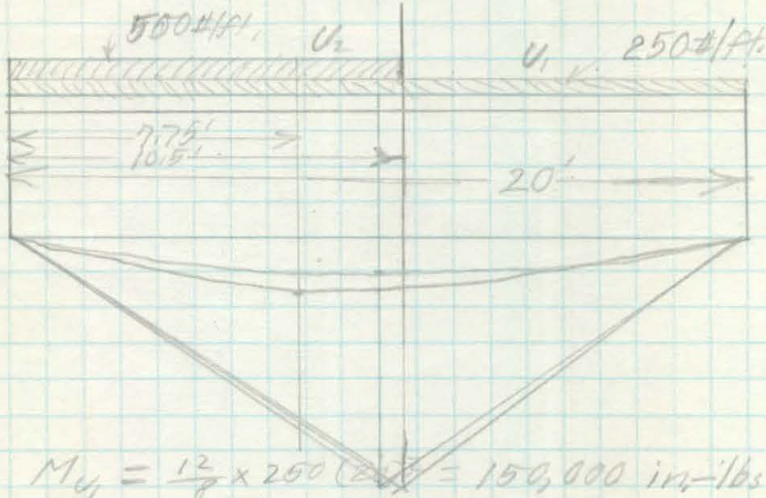
$v = \frac{23400}{.375 \times 20} = 2800$ #/in²

Roof Beams

(f) at Elevator Spandrel

Wall load (1) = 4 x 125 = 500 #/ft.
(2) = 2 x 125 = 250 #/ft.

Beam (b) = 13,500 #
Column = 7,200 #
20,700 #



$M_{U_1} = \frac{12}{8} \times 250 (20)^2 = 150,000$ in-lbs.

$M_{U_2} = 364,000 \times \frac{500}{750} = 236,000$ in-lbs.

$M_c = 1,225,000 \times \frac{20,700}{20,200} = 1,257,000$ in-lbs.

$M = 1,250,000 + 150,000 + 200,000 = 1,600,000$ in-lbs.

Try 20" - 89# Beth. I-beam flange = 8"
$S = 117.2$ in^3
$A = 17.36$ □"

Wt. of f.p. $= \frac{22 \times 12 - 17}{144} \times 100 = 260$ #/ft.
60
320 #/ft.

$M = \frac{320}{260} \times 150,000 = 192,000$ in-lbs.

$M = 1,792,000$ in.-lbs.

$f = \frac{1,792,000}{117.2} = 15,300$ #/in^2

$R_L = 22,200$ # $R_R = 21,100$ #

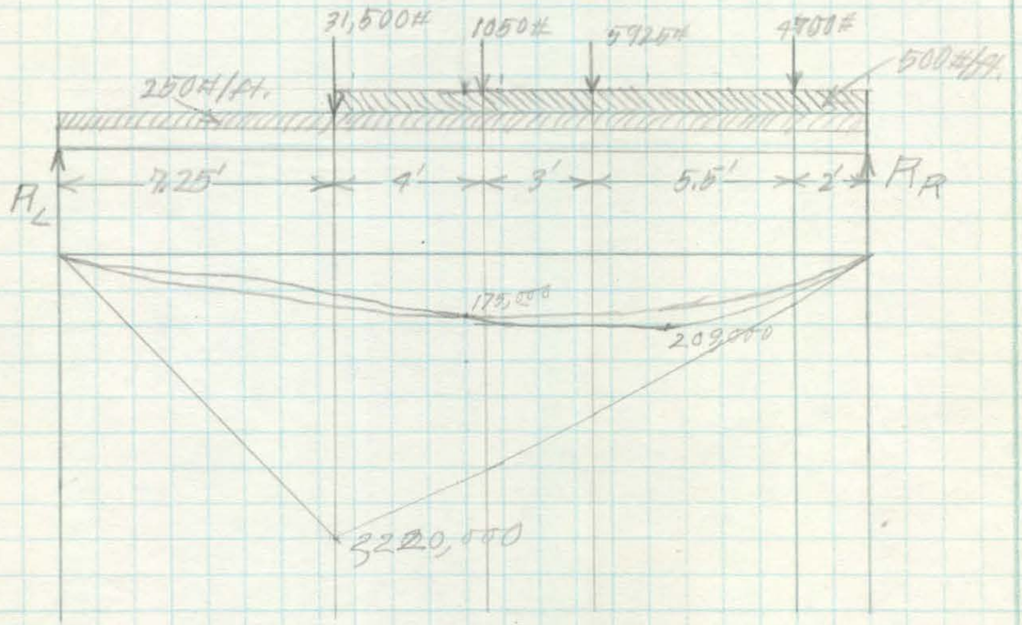
Roof Girder

(e) at Elevator

Wall load (1) = 4x125 = 500#/ft.
" " (2) = 2x125 = 250#/ft.

R_L beam (c) = 23,400# R_L beam (a) = 1050#
Column = 8,100# approx.
31,500#

Other loads - 5925# & 4570#



$M_{u_1} = \frac{12}{8} \times 250 \times (21.75)^2 = 177,000$ in.-lbs.

$x_0 = \frac{500 \times 14.5 \times 1/3}{500} = 4.8$ from Left.

$M_{u_2} = [500 \times 14.5 \times 2/3 \times 4.8 - 250 \times (4.8)^2]12 = 209,000$ in.-lbs.

$M_{c_{max}} = 25,475 \times 7.25 \times 12 = 2,220,000$ in.-lbs.

M_T = approx. 2,550,000 in.-lbs.

Try 24" Beth - 73.5 #/ft. S = 174.3 A = 21.5 □"
t_{web} = .39 flange = 9"

wt. of f.p. = $\frac{(26 \times 13 - 21.5)}{144} \times 150 = 330$ #/ft. 400#

$M = 177,000 \times \frac{400}{250} = 283,000$ in.-lbs.

M_{total} = approx 2,800,000 in.-lbs.

$f = \frac{2,800,000}{174.3} = 16,050$ #/in.²

$v = \frac{34300}{.39 \times 24} = 3680$ #/in.²

Roof Girders

Typical

$Load = 590 \times 19.75 = 11{,}650\#$ con. (cross-beam)

$M = 11{,}650 \times 7.25 \times 12 = 1{,}015{,}000$ in.-lbs.

Try 18" - 49# Beth $A = 14.25$ $t_{web} = .32"$
flange = 7.5" $S = 88.7 in^3$

$wt.\ f.p. = \frac{(20 \times 11.5 - 14)}{144} \times 150 = 225\#$

Total wt. = 275#

$M = \frac{12}{8} \times 275 \times (21.75)^2 = 195{,}000$ in.-lbs.

total $M = 1{,}210{,}000$ in.-lbs.

$f_s = \frac{1210{,}000}{88.7} = 13{,}650\#/in.^2$

Spandrel

$Load = \frac{11{,}650}{2} = 5825\#$ con. (crossbeam)
250 #/ft. (wall)

$M_c = \frac{1015000}{2} = 507{,}500$ in.-lbs.

$M_v = \frac{250}{275} \times 195{,}000 = 177{,}000$ in.-lbs.

Try 15" - 38# Beth. $A = 11.27$ $t_{web} = .29"$
flange = 6.6" $S = 59 in^3$

$wt.\ f.p. = \frac{(17 \times 10.5 - 11)}{144} \times 150 = 165\#/ft.$

total = 205#/ft.

$M = \frac{205}{250} \times 177{,}000 = 146{,}000$ in.-lbs.

Total $M = 830{,}500$ in.-lbs.

$f_s = \frac{830{,}500}{59} = 14{,}100\#/in.^2$

$v = \frac{10{,}775}{.29 \times 15} = 2480\#/in.^2$

Floor Slab ~ 3rd Floor

Load = 80# (live)
50# (assumed dead)
130# total
40# partition

End panel

$M_u = \frac{1}{10} \times 170 \times (7.25)^2\, 12 = 10{,}700$ in-lbs.

$d^2 = \frac{10700}{12 \times 107.5} = 8.28 \qquad d = 2.88''$

Make $d = 3.00''$ $\quad t = 3.75 \quad$ wt. = 47#

$\frac{M}{bd^2} = \frac{10,700}{12 \times 9} = 99.1$

$p = .0070 \qquad f_c = 620$ #/in.²

$a_s = .007 \times 3 \times 12 = .252$ □"

Use 3/8" ☐ 6½" c-c $\quad a_s = .259 \quad p = .00719$

Total wt. = 170#/ft²
$d = 3.''$
$t = 3.75''$
$a_s = .259$ □"
3/8" ☐
6½" c-c

Middle panels

assume w = 45#/ft.
Load = 165#/ft.

$M = \frac{1}{12} \times 165 \times (7.25)^2 \times 12 = 8{,}660$ in-lbs.

$d^2 = \frac{8660}{12 \times 107.5} = 6.71 \qquad d = 2.595$

make $d = 2.75''$ $\quad t = 3.5'' \quad$ wt = 44#/ft.

$\frac{M}{bd^2} = \frac{8660}{12 \times (2.75)^2} = 95.7$

$p = .0068 \qquad f_c = 610$ #/in²

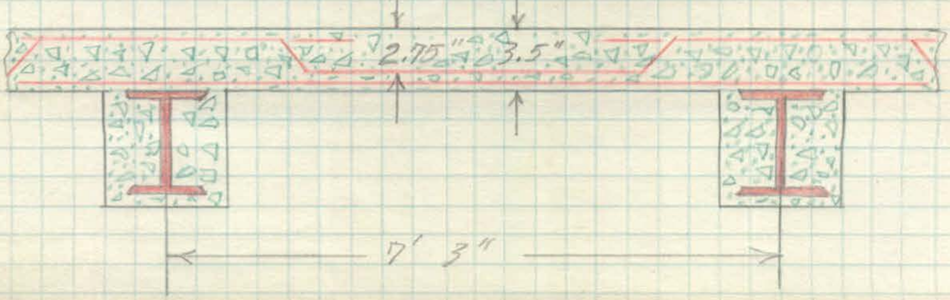
$a_s = .0068 \times 2.75 \times 12 = .225$

Use 3/8" ☐ 7½" c-c $\quad a_s = .225$

Total wt. = 165#/ft²
$d = 2.75''$
$t = 3.5''$
$a_s = .225$ □"
3/8" ☐
7½" c-c

End support Bend all bars 1' 9" from ℄ and hook

Center support Bend ½ of bars from each side 1' 9" from ℄ and extend 2' past ℄



Cross Beams ~ 3rd Floor

Load = 165 x 7.25 = 1190 #/ft.

Typical

$M = \frac{1}{8} \times 1190 (19.75)^2 12 = 700,000$ in.-lbs.

Try 15" - 38# Beth A = 11.27 t_{web} = .29" flange = 6.66 S = 59

Wt. of f.p. = $\frac{(17 \times 11 - 11)}{144}$ 144 = 175 #
38
213 #

$M_w = \frac{215}{1190} \times 700,000 = 126,000$ in.-lbs.

M_t = 826,000 in.-lbs

$f_s = \frac{826,000}{59} = 14,000$ #/in.² (satisfactory)

$V = 1405 \times \frac{19.75}{2} = 13,900\# = R$

15" - 38# B
load = 1190#
wt. = 215#
V = 13,900#

Spandrel

Load = $\frac{170 \times 7.25}{2}$ = 615# (slab)
11 x 125 = 1375# (wall)
1990 #/ft.

$M = \frac{12}{8} \times 1990 \times (19.75)^2 = 1,170,000$ in.-lbs

Try 18" - 49# Beth. A = 14.25 t = .320" flange = 7.5" S = 88.7

Wt. of f.p. = $\frac{(20 \times 11.5 - 14)}{144}$ 144 = 215 #
50
265 #

$M_w = \frac{265}{1990} \times 1,170,000 = 156,000$ in.-lbs.

M_T = 1,326,000 in.-lbs.

$f_s = \frac{1,326,000}{88.7} = 15,000$ #/in.²

$V = 2255 \times \frac{19.75}{2} = 22,300$ #

18" - 49# B
load = 1990#
wt. = 265#
V = 22,300#

Girders ~ 3rd Floor

Load = 2x13,900 = 27,800 # concentrated at 1/3 pts.

M_c = 27,800 x 7.25 x 12 = 2,420,000 in-lbs.

Typical

Try 24" - 73.5# Beth. A = 21.47 t = .39
flange = 9" S = 174.3

Wt of f.p. = (26 x 13 - 21.5) 144 / 144 = 316.5 #
73.5 #
390.0 #

M_w = 12/8 x 390 x (21.75)² = 277,000 in.-lbs.

M_T = 2,697,000 in.-lbs.

f_s = 2,697,000 / 174.3 = 15,500 #/in.²

V = 27,800 + 390 x 21.75/2 = 32,000 # allow. = 54,000 #

v = 32,000 / 24. x .39 = 3430 #/in.²

24" - 73.5# B
load = 27,800 # c
wt. = 390 #/ft. u.
V = 32,000 #

Spandrel

Load = 13,900 # concentrated at 1/3 pts.
10 1/3 x 125 = 1290 #/ft. (wall.

M_c = 1/2 x 2,420,000 = 1,210,000 in.-lbs.

M_w = 12/8 x 1290 x (21.75)² = 915,000 in-lbs.
2,125,000 in-lbs

Try 24" - 73.5 # B A = 21.47 t = .39
flange = 9" S = 174.3

Wt = 390 # M = 277,000 #

M_T = 2,402,000 in.-lbs.

f_s = 2,400,000 / 174.3 = 13,750 #/in.²

V = 13,900 + 1680 x 21.75/2 = 32,200 #
allowable = 54,100 #

24" - 73.5# B
load = 13,900 c
1290 #/ft. u
wt. = 390 #/ft. u.
V = 32,200 #

Floor Slab ~ 2nd Floor

Load = 200 #/ft. (live)
40 #/ft. (partitions)
56 #/ft (assumed dead)
296 #/ft.

End Span

$M = \frac{1}{10} \times 296\,(7.25)^2 \times 12 = 18{,}700$ in.-lbs

$d^2 = \frac{18{,}700}{12 \times 107.5} = 14.50 \qquad d = 3.81$

Make $d = 3.75''$ $\quad t = 4.5''$

$p = \frac{18{,}700 \times 8}{7 \times 16{,}000 \times 12 \times (3.75)^2} = .00792$

$a_s = .00792 \times 12 \times 3.75 = .357\ \square''$

Use 5/8" ◎ spaced 9½" c-c $\quad a_s = .388\ \square''$

Bend all bars 1'9" from ℄ and hook

Load = 240#
53 # (assumed)
293 #

Center Span

$M = \frac{12}{12} \times 293\,(7.25)^2 = 15{,}400$ in.-lbs.

$d^2 = \frac{15{,}400}{12 \times 107.5} = 11.94 \qquad d = 3.46$

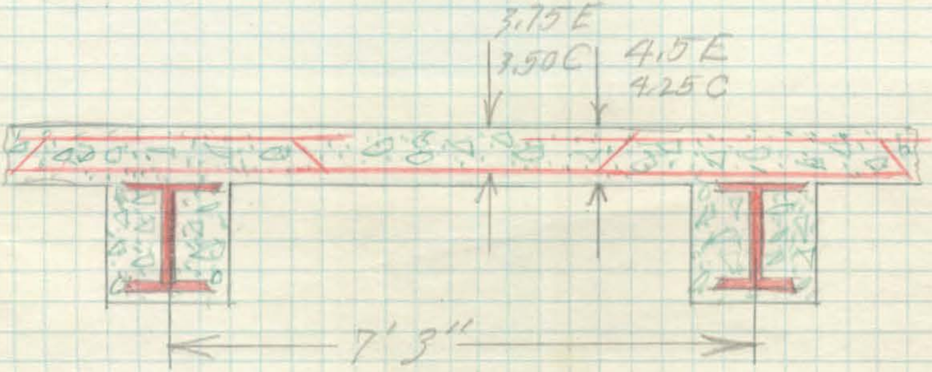
Make $d = 3.5''$ $\quad t = 4.25''$

$p = \frac{15{,}400 \times 8}{7 \times 16{,}000 \times 12 \times (3.5)^2} = .00748$

$a_s = .00748 \times 12 \times 3.5 = .314\ \square''$

Use 1/2" ▣ spaced 9½" c-c $\quad a_s = .316\ \square''$

Bend 1/2 of bars from each side 1'9" from ℄ of support and extend 2" past ℄



Beams - 2nd Floor

Typical

Load = 7.25(200+40+53) = 2125 #/ft.

$M = \frac{1}{8} \times 2125 \times (19.75)^2 \times 12 = 1{,}243{,}000$ in.-lbs.

Try 18" 49# Beth. A = 14.28 t = .32
flange = 7.5" S = 88.7
allow. V = 36,700#

wt. of f.p. = $\frac{20 \times 11.5 - 14}{144} \times 144 = 216$
49
265#

$M = \frac{265}{2125} \times 1{,}243{,}000 = 155{,}000$ in.-lbs.

$M_t = 1{,}398{,}000$ in.-lbs.

$f_s = \frac{1{,}398{,}000}{88.7} = 15{,}750$ #/in² (safe)

$V = \frac{19.75}{2} \times 2390 = 23{,}600$# (safe)

18"-49#B.
load = 2125#/ft.
wt. = 265#/ft.
V = 23,600#

Spandrel

Load = $\frac{2145}{2} = 1073$ #/ft (slab)
10×125 = 1250 #/ft. (wall)
2323 #/ft.

Try 18"-49# Beth. as above. wt. = 265#

$M = \frac{1}{8} \times 2588 \times (19.75)^2 \times 12 = 1{,}515{,}000$ in.-lbs.

$f_e = \frac{1{,}515{,}000}{88.7} = 17{,}100$ #/in² (too high)

Try 20"-59.5# Beth. A = 17.36 t = .375"
flange = 8" S = 117.2
V = 50,000

wt. f.p. = $\frac{22 \times 12 - 17}{144} \times 144 = 247$
59
306#

$M = \frac{2629}{2588} \times 1{,}515{,}000 = 1{,}540{,}000$ in.-lbs.

$f_s = \frac{1{,}540{,}000}{117.2} = 13{,}130$ #/in². (safe)

$V = 2630 \times \frac{19.75}{2} = 26{,}000$ #

20"-59.5#B.
load = 2323#/ft.
wt. = 306#/ft.
V = 26,000#

Girders – 2nd Floor

Typical

Load = 2 × 23,600 = 47,200 # at 1/3 points.

$M = 47,200 \times 7.25 \times 12 = 4,110,000$ in.-lbs.

Try 28" – 105.9 # Beth.

A = 30.88 t = .5
Flange = 10" S = 286.7
V = 89,000 #

wt. of f.p. $= \frac{30 \times 14 - 31}{144} \times 144 = 389$ #/ft.
106
495 #

$M = \frac{1}{8} \times (21.75)^2 \times 495 \times 12 = 350,000$ in.-lbs.

$M_t = 4,110,000 + 350,000 = 4,460,000$ in.-lbs.

$f_s = \frac{4,460,000}{286.7} = 15,550$ #/in.² (safe)

$V = \frac{21.75}{2} \times 495 + 47,200 = 52,600$ #

28" 105.9 # B
load = 47,200 # 1/3 pts.
wt. = 495 #/ft.
V = 52,600 #

Load = 23,600 # at 1/3 pts.

Spandrel

$9.5 \times 125 = 1190$ #/ft. wall.

$M_c = \frac{4,110,000}{2} = 2,055,000$ in.-lbs.

$M_u = \frac{1190}{495} \times 350,000 = 841,000$ in.-lbs.

M = 2,896,000 in.-lbs.

Try – 24" – 100 # Carnegie.

A = 29.41 t = .754"
flange = 7.254 S = 198.4

wt. of f.p. $= \frac{26 \times 11 - 29}{144} \times 144 = 256$ #/ft.
100
356.

$M = \frac{356}{495} \times 350,000 = 252,000$ in. lbs.

$M_t = 2,896,000 + 252,000 = 3,148,000$ in.-lbs.

$f_s = \frac{3,148,000}{198.4} = 15,900$ #/in.² (safe)

$V = 23,600 + 1546 \times \frac{21.75}{2} = 40,400$ #

24" 100 #
Load = 23,600 # 1/3 pts.
1190 #/ft.
Wt. = 356 #/ft.
V = 40,400 #

First Floor

Slabs and Typicals.

The slabs and typical beams and girders are the same for the 1st floor as for the 3rd floor.

Spandrel Beam

Load = 625 # (slab)
$12 \times 125 =$ 1500 # (wall)
2125 #

$M = \frac{1}{8}(19.75)^2 \times 2125 \times 12 = 1{,}243{,}000$ in-lbs.

18" – 49 # Beth. $A = 14.25$ $t = .32''$
flange = 7.5" $S = 88.7 \text{ in}^3$

Wt. of f.p. $= \frac{20 \times 11.5 - 14}{144} \times 144 = 216$ #
49
265 #

$M = \frac{265}{2125} \times 1{,}243{,}000 = 155{,}000$ in.-lbs.

$M_t = 1{,}398{,}000$ in.-lbs.

$f_s = \frac{1398000}{88.7} = 15{,}750$ #/in². (safe.

$V = 23{,}600$ #

18" – 49 # B
Load = 2125 #/ft.
Wt. = 265 #/ft.
V = 23,600 #

Spandrel Girder.

Load = $11.5 \times 125 = 1438$ #/ft. (wall.
13,900 # at $\frac{1}{3}$ points.

$M_c = 13900 \times 7.25 \times 12 = 1{,}210{,}000$ in. lbs.
$M_u = \frac{1}{8} \times 1438 \times (21.75)^2 \times 12 = 1{,}010{,}000$
$2{,}220{,}000$ in.-lbs.

Try ~~24~~" – 73 # Beth.
$A = 21.47 \square''$ $t = .390''$
flange = 9" $S = 174.3 \text{ in}^2$

Wt. of f.p. $= \frac{26 \times 13 - 21}{144} \times 144 = 317$ #/ft.
73
390 #/ft.

$M = \frac{390}{1438} \times 1{,}010{,}000 = 276{,}000$ in.-lbs.

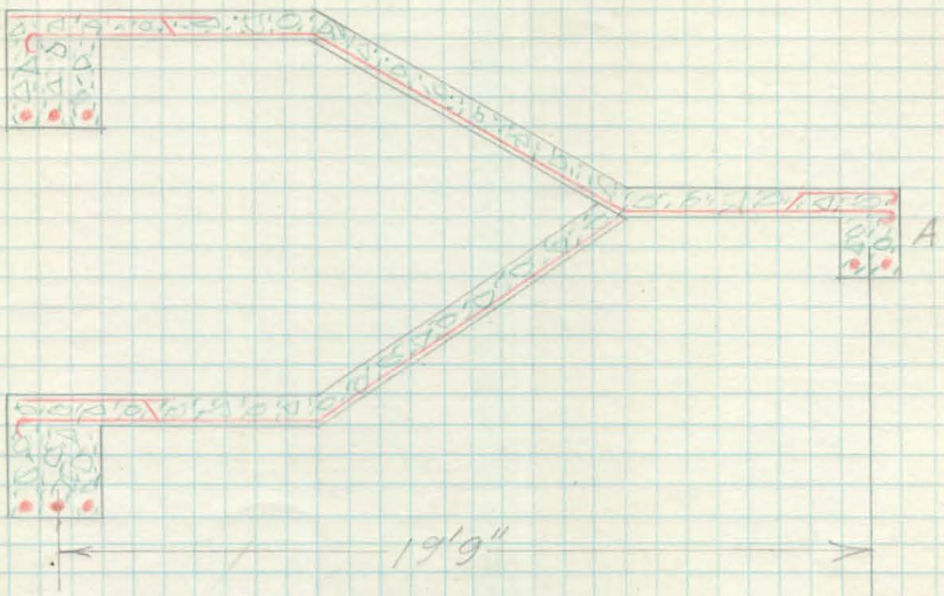
$M_t = 2{,}496{,}000$ in. lbs.

$f_s = \frac{2{,}496{,}000}{174.3} = 14{,}350$ #/in².

$V = 13{,}900 + 1828 \times \frac{21.75}{2} = 33{,}800$ #

24" – 73.5 # B
Load = 13,900 # at $\frac{1}{3}$ pts., 1438 #/ft.
Wt. = 390 #/ft.
V = 33,800 #

Stairs 3rd Floor - Roof



Stairway Slab

Assume L.L. = 100 #/ft.²
slab = 150 #/ft.²
stairs = 20 #/ft.²
270 #/ft.²

stairs =
$\frac{11 \times 7 \times 10}{2} = 385$ #
$\frac{385}{20} = 19$ #/ft.

$M = \frac{12}{10} \times 270 \, (19.75)^2 = 126,400$ in. lbs.

$d^2 = \frac{126,400}{12 \times 107.5} = 98 \quad d = 9.9''$

d = 9.5"
t = 10.5"

L.L. = 100# slab = 135 # stairs = 20 #

$a_s = .938$

$M = \frac{255}{270} \times 126,400 = 118,400$ in.-lbs.

5/8" ☐ 5" c-c

$d^2 = \frac{118,400}{12 \times 107.5} = 91.7 \quad d = 9.58''$

make d = 9.5" t = 10.5" $wt = \frac{10.5}{12} \times 150 = 131$#

$\frac{M}{bd^2} = \frac{118400}{12 \times (9.5)^2} = 109.7$

$p = .0082$

$f_c = 650$ #/in.² $f_s = 15,450$ $j = .871$ $k = .388$

$a_s = .0082 \times 9.5 \times 12 = .935$ ☐"

Use 5/8" ☐ 5" c-c $a_s = .938$ ☐"

Bend 1/3 of bars 4'6" from ℄ + 1/3 of bars 3'6" from ℄ on left.

Bend 1/3 of bars 4' + 1/3 of bars 3' from ℄ on right at intermediate support.

Use transverse bars 12 - c-c

Stairs - Beam A 3rd F to roof

$$Load = 270 \times \frac{10 + \overset{11.45}{\sqrt{36 + (9.75)^2}}}{2} = 2900 \ \#/ft.$$

Length = 12'

$$M = \frac{12}{8} \times 2900 \times 144 = 626{,}000 \text{ in.-lbs.}$$

Try 12"-36.5# Beth $A = 10.61 \square''$ $t = .31''$
flange = 6.3" $S = 44.9''$
$V = 32{,}200 \ \#$

$$\text{wt of f.p.} = \frac{14 \times 10.5 - 11}{144} \times 144 = 136 \ \#$$
$$\frac{+\ 36}{172 \ \#}$$

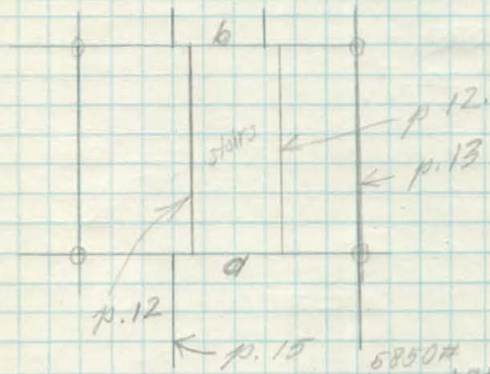
$$M = \frac{172}{2900} \times 626{,}000 = 37{,}200 \text{ in.-lbs.}$$

$$M_T = 663{,}000 \text{ in.-lbs.}$$

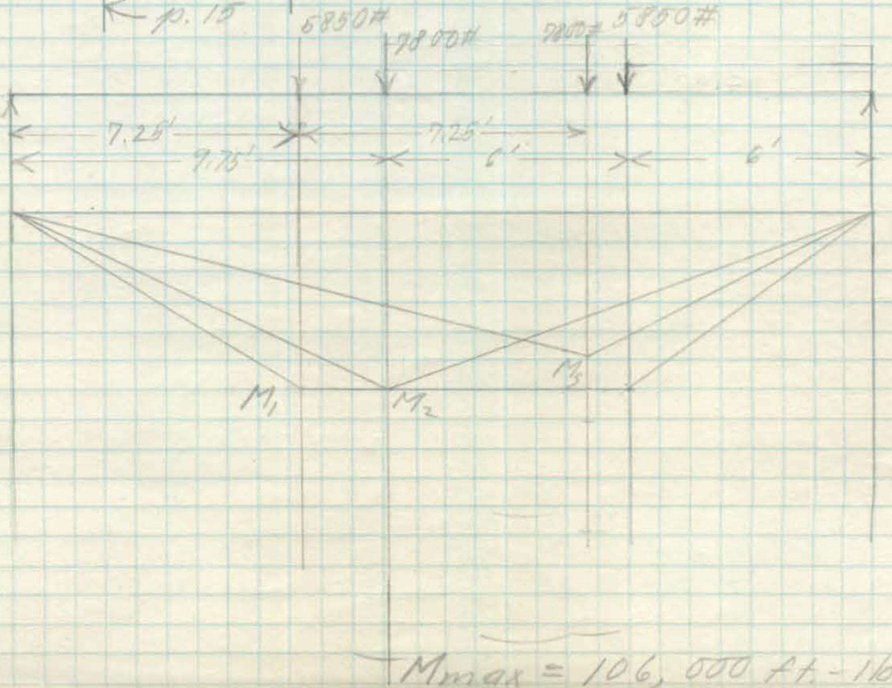
$$f_s = \frac{663{,}000}{44.9} = 14{,}800 \ \#/in^2 \ \text{(safe)}$$

$$V = 3072 \times \frac{21.45}{2} = 33{,}000 \ \#$$

12"-36.5# B
Load = 2900#/ft.
wt. = 172#/ft.
V = 33,000#



Beam (b)



$M_{max} = 106{,}000$ ft.-lbs.

Stairs

Beam (b)

$M_1 = 5850 \times 7.25 = 42,400$ ft.-lbs.

$M_2 = \cancel{7}800 \times \frac{12}{21.75} \times 9.75 = 42,000$ ft.-lbs

$M_3 = 7800 \times \frac{6}{21.75} \times 15.75 = 34,000$ ft.-lbs.

$M_{max} = 106,000 \times 12 = 1,260,000$ in.-lbs.

Try 18" – 49# Beth $A = 14.26\,\square"$ $t = .32"$ flange $= 9.5"$ $S = 88.7$ $V = 36,700\#$

18" – 49# B

V = 12,900 outside

= 15,200 center.

wt. of f.p. $= \frac{20 \times 11.5 - 14}{144} \times 144 = 216\#$

$\begin{array}{r} 216\# \\ \underline{49} \\ 265\# \end{array}$

$M = \frac{12}{8} \times 265 \times (21.75)^2 = 192,000$ in.-lbs.

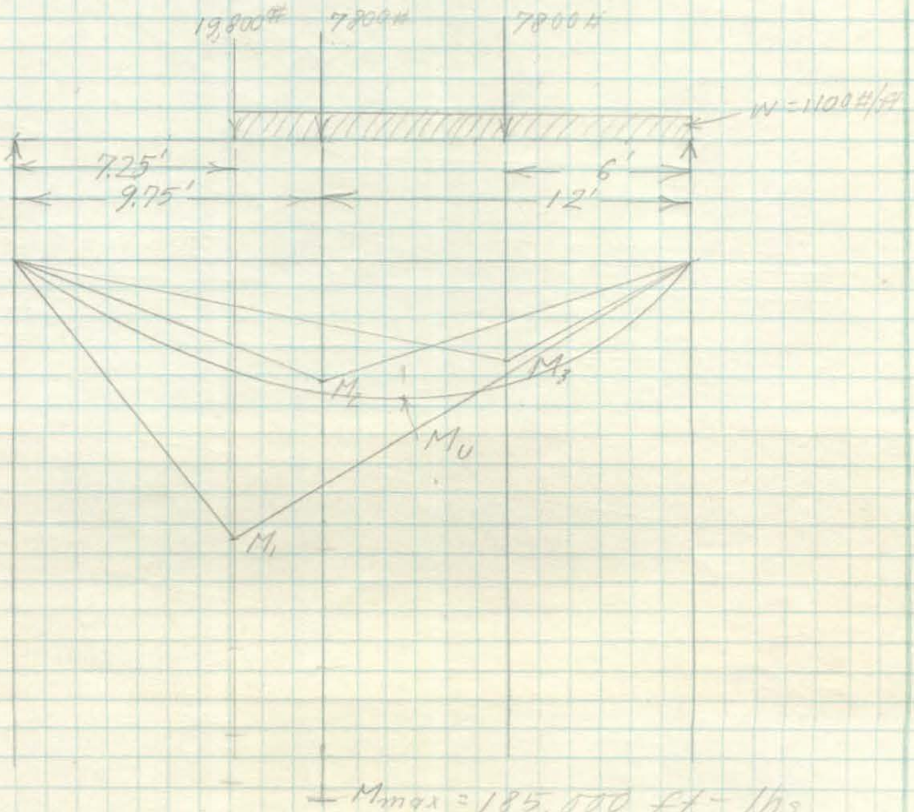
$M_T = 1,452,000$ in.-lbs.

$f_s = \frac{1,452,000}{88.7} = 16,350\#/in^2.$

$V_L = 265 \times \frac{21.75}{2} + 7800 \times \frac{18}{21.75} + 5850 = 15,200\#$

$V_R = 17,900\#$

Beam (a)



$M_{max} = 185,000$ ft-lbs.

Stairs

Beam (a)

$M_1 = 13{,}200 \times 7.25 = 95{,}700$ ft.-lbs.

$M_2 = 4300 \times 9.75 = 41{,}900$ ft.-lbs.

$M_3 = 2150 \times 15.75 = 33{,}900$ ft.-lbs.

$R_1 = \frac{1100}{43.5} \times 14.5 \times (\overset{29.0}{43.5} - 14.5) = 10{,}260\#$ $\quad R_2 = 5690\#$

$x_0 = \frac{10{,}260}{1100} = 9.33'$

$M_U = 10{,}260 \times 9.33 - \frac{1100}{2}(9.33)^2 = 48{,}000$ ft.-lbs.

$M_{max} = 185{,}000$ ft.-lbs.

Try 73.5# - 24" Beth. $\quad A = 21.47\square''$ $\quad t = .390''$
width = 9" $\quad S = 174.3\,in^3$
$V = 54{,}000\#$

wt. of f.p. $= \frac{26 \times 13 - 21.5}{144} \times 144 = 317\#/ft.$
$\quad 73$
$\quad 390\#/ft.$ total

$M = \frac{12}{8} \times 390 \times (21.75)^2 = 276{,}000$ in.-lbs.
$185{,}000 \times 12 = 2{,}220{,}000$ in.-lbs.
$2{,}290{,}000$ in.-lbs. approx. max.

$f_s = \frac{2{,}290{,}000}{174.3} = 13{,}120\ \#/in^2$ (safe)

$R_L = 30{,}250\#$ $\quad R_R = 29{,}600\#$

24" - 73.5 B

Wt. of beam
390#/ft.
M# =
2,290,000 in.-lbs

$R_L = 30{,}250\#$
$R_R = 29{,}600\#$

Central Columns

Column Schedule.

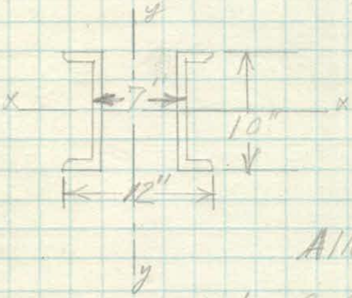
Floor area = 430 sq. ft.

Load	Roof	3rd Floor	2nd Floor	1st Floor
Floor Live Load	12,900	34,400	86,000	34,400
Floor Dead Load	29,100	48,000	57,700	48,000
Column + Covering	2 500	2 500	4 700	3 350
Total Load for story	44,500	84,900	148,400	85,750
Accumulated Total	44,500	129,400	277,800	363,550
Column Section	2-10" 20# [s single latticed		2-10" 20# [s 2-5/8" x 12" pls.	

Design of Columns.

Try 10" channels for both columns.

20# for upper column - single latticed



$r_{xx} = 3.66$ $\qquad r_{yy} = 3.80$

$A = 11.76$

$70\, l/r = 70 \times \frac{144}{3.66} = 2750$

$16000 - 2750 = 13,250$ #/in²

Allow. $P = 13,250 \times 11.76 = 155,000$ #

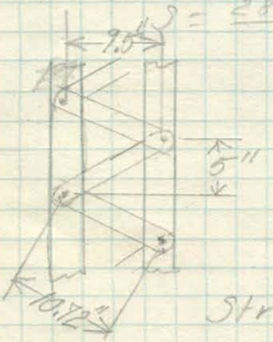
Wt of f.p. $= \frac{14 \times 12 - 12}{144} \times 144 = 156$

Wt. steel $= 40 + 9$ (lattice) $= 49$

205 #/ft.

$205 \times 12 = 2460$#

$S = \frac{280 A r}{c} = \frac{280 \times 11.76 \times 3.80}{6} = 2080$#



$A = \frac{10.72}{40} = .268$

Use 5/16" thick lacing $t = .315$

Make width = 2¼" for ¾" rivets.

Stress $= 1040 \times \frac{10.72 \times 16}{9.5 \times 2.25 \times 5} = 1675$ #/in²

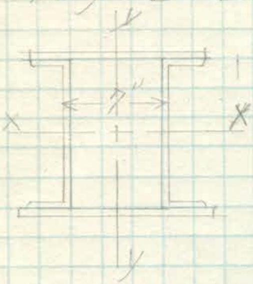
$r = .288 \times \frac{5}{16} = .09 \qquad l/r = \frac{10.72}{.09} = 120$ approx.

$P/A = 16000 - 70 \times 120 = 7600$ #/in² ← safe

Central Columns

1st + 2nd Floors

Try 2 - 20# -10" channels and 2-12"x 5/8" plates.



$$I_{x-x} = 2\left[\tfrac{1}{12} \times 12 \times (\tfrac{5}{8})^3 + \tfrac{15}{2} \times (\tfrac{101}{16})^2\right]$$
$$+ 2[157.4] = 915 \text{ in}^4$$

$$I_{yy} = 2\left[\tfrac{1}{12} \times \tfrac{5}{8} \times (12)^3\right] + 2[204.3]$$
$$= 589 \text{ in}^4$$

$$A = 11.76 + 2 \times \tfrac{5}{8} \times 12 = 26.76$$

$$r = \sqrt{\frac{589}{26.76}} = 4.7$$

$$P = 26.76\left(16000 - 70 \times \frac{14 \times 12}{4.7}\right) = 362,000\ \#$$ allowable load.

wt. = (17 x 16 - 27) + 91 = 335 #/ft.

335 x 10 = 3350 #

335 x 14 = 4700 #

Splice

Use standard Splice shown in Ketchum page 91. Use 1/2" thick plates.

Splice located 1' above top of 2nd Floor.

Base

~~Use C.I. plate 2'9" sq. 1'3" high.~~

~~Capacity 378,000 # wt. = 1,270 #~~

~~Shown in Ketchum p. 92.~~

Supersede

Total weight to foundation = 365,000

Side Columns

Column Schedule

Floor area = 215 sq. ft.

Load	Roof	3rd Floor	2nd Floor	1st Floor
Floor Live Load	6,450	17,200	43,000	17,200
Floor Dead Load	14,550	24,000	28,850	24,000
Wall Load	10,900	56,200	51,800	62,600
Colum and Cover.	3200	3,200	3400	4700
Total load - Story	35,100	100,600	127,150	108,500
Eccentric Effect	—	—	—	—
Accumulated Total	35,100	135,700	262,850	371,350
Column Section	2[s - 9" 13.25# 1 I - 9" 21#		2[s - 12" 25# 1 I - 9" 21# 2 Pls - 1/4" x 7"	

3rd floor & roof

Try 2 - 9" 13.25# channels and 1 - 9" 21# I-C.

$A = 7.78$ $\quad$ $A = 6.31$

$I_{y-y} = 206.5$ $\quad$ $I_{y-y} = 84.9$

$I_{x-x} = 94.6$ $\quad$ $I_{x-x} = 5.16$

$I_{x-x} = 99.76$ $\qquad r = \sqrt{\frac{99.76}{14.09}} = 2.66$

$P = 14.09\left(16000 - 70 \times \frac{12 \times 12}{2.66}\right) = 172,000\#$ = allowable load

$Wt. = (13 \times 18 - 14) + 47.5 = 265\#$

$12 \times 265 = 3180$

Side Columns

1st + 2nd Floors	Try 2 - 12" 25# [s and 1 - 9" 21# I-C 2 - 9" x $\frac{3}{4}$" pls on outside of channels. $I_{x-x} = 288$ $I_{xx} = .5$ $I_{x-x} = \underline{21}$ 304 $\quad r = \sqrt{\frac{317}{31.51}} = 3.16$ $P = 31.51\left(16000 - 70 \times \frac{14 \times 12}{3.15}\right) = 387,000\,\# =$ allowable load $wt = (16 \times 18 - 31) + 81.5 = 338\,\#$ $10 \times 338 = 3380 \quad 14 \times 338 = 4730\,\#$
Splice	Use plates to build up smaller section as in Ketchum p. 91. Splice I-web in addition. Total wt. = 372,000 #
Base	
End Columns	Make end columns same as the side columns. Total wt. = 372,000 #
Corner Columns	Make corner columns the same. Total weight = 285,000 #

Bases for Columns

Central Column

Column – 2-10" [s @ 20# + 2-$\frac{5}{8}$" × 12" pls.

wt = 363,000#.

$$Area = \frac{363\,000}{350} = 1040\ \square''$$

Make area ~~32.5"~~ × 32.5" $p = \frac{1}{2}(32.5 - 10) = 11.25''$

$$t = p\sqrt{\frac{3P}{Af}} = 11.25\sqrt{\frac{3 \times 363000}{1055 \times 16{,}000}} = 2.86''$$

Use plate 32.5" sq. and 2$\frac{7}{8}$" thick

2 ∠s 12" × $\frac{3}{4}$" × 6" × 6"   10 riv. vert + horiz

2 ∠s 8" × $\frac{3}{4}$" × 6" × 6"   8 riv. " " "

Outside Column

Column ~ 2-12" [s @ 25#

1-9" 21# I

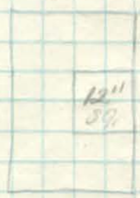
2-$\frac{3}{4}$" × 7" pls.

wt. = 372,000#

$$A = \frac{372{,}000}{350} = 1063\ \square''$$

Make plate 41" × 26" $p = 14''$

$$t = 14\sqrt{\frac{3 \times 372\,000}{1066 \times 16000}} = 3.58$$



Use plate 41" × 26" × 3$\frac{5}{16}$"

2 ∠s – 7" × $\frac{3}{4}$" × 6" × 6"   4 riv. vert. + horiz

2 ∠s – 9$\frac{1}{2}$" × $\frac{3}{4}$" × 6" × 6"   8 riv vert + horiz

Beams and their Connections

First Floor

Spandrel Girder $20' \; 11\frac{25}{32}''$ long $24'' - 73.5\#$ B.

End connection 2 ∟s $5'' \times 4'' \times \frac{3}{8}'' \times 1'\text{-}5\frac{1}{2}''$
Fld connect. 14 - $\frac{3}{4}''$ riv. single shear.
Beam " 9 - $\frac{3}{4}''$ riv. double shear.
staggered.

Connections for 2 - 15" 38# B - standards. 9'-5" c-c

Spandrel Beam $18' - 8\frac{9}{32}''$ long. 18" 49# B.

End Connection - standard connection
connections for 2 - spacers.

Typical Girder - $20' - 8\frac{7}{8}''$ 24" - 73.5 B.

Connection - same as spandrel.
connections for 2 - 15" 38# B - standards.

Typical Beam - 18'-11" 15" 38# B

Standard connections at end
standard connections for 2 spacers.

Second Floor

Spandrel Girder $20' - 11\frac{25}{32}''$ 24" - 100# C.

Connection - standard for 80# C
connections for 2 18" 49# B - standard

Spandrel Beam $18' - 8\frac{9}{32}''$ 20" 59.5# B

Standard end connection
connection for 2 spacers

Typical Girders $20' - 8\frac{7}{8}''$ 28" - 105.9# B

Standard end connections.
standard connections for 2 - 18" 49# B.

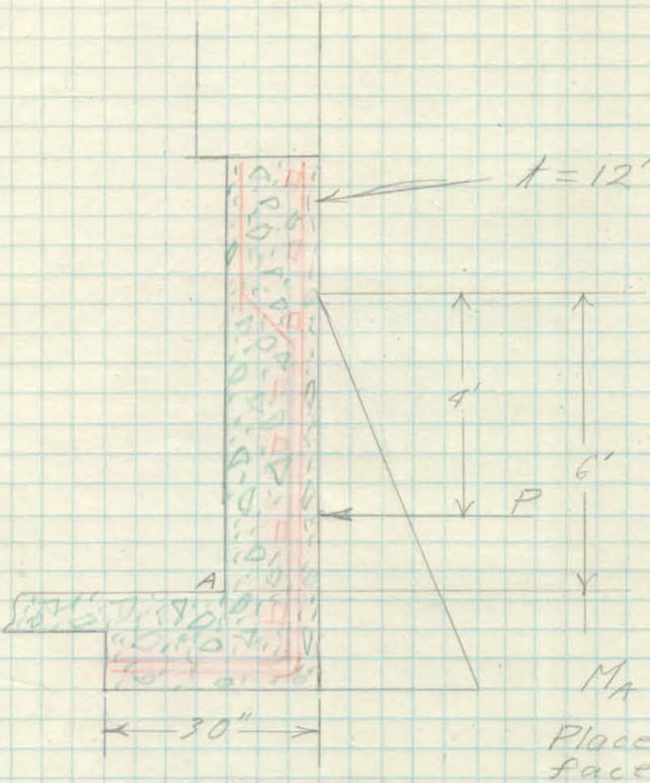
Typical Beam 18'-11" 18" - 49# B.

Standard end connections
connections for 2 spacers.

Beams and their Connections.

Third Floor	Spandrel Girder $20'-11\frac{17}{32}''$ $24''-73.5\#$ B.
	Same connection as first floor.
	Standard Connections for 2-15" 38# B
	Spandrel Beam $18'-8\frac{17}{32}''$ 18" 49# Beth.
	Same as first floor.
	Typical Girder $20'-8\frac{3}{8}''$ $24''-73.5\#$ B
	Same as first floor
	Typical Beam $18'-11''$ $15''-38\#$ B
	Same as first floor.
Roof	Spandrel Girder $20'-11\frac{17}{32}''$ $15''-38\#$ B
	Standard Connection.
	Standard connection for 2-10"-25# C.
	Spandrel Beam $18'-8\frac{17}{32}''$ $10''-25\#$ C
	Standard Connection.
	connections for 2 spacers.
	Typical Girder $20'-8\frac{7}{32}''$ $18''-49\#$ B
	Standard connections
	Standard connections for 2 10"-25# C
	Typical Beam $18'-11''$ $10''$ 25# C
	Standard connections
	Connections for 2 spacers.

Basement Wall



Assume basement floor 6' below ground level.

Use 12" thick walls and design as a cantilever retaining wall.

$P = \frac{1}{3} \times 6 \times 100 \times \frac{6}{2} = 600\#$

$M_A = 600 \times 2 \times 12 = 14{,}400$ in-lbs.

Place steel 3" in from face.

$a_s = \dfrac{14{,}400 \times 8}{16000 \times 7 \times 9} = .114 \square''$ $\quad p = .001055$

$k = \sqrt{30 \times .001055 + 225(.001055)^2} - 15(.001055) = .163$

$j = .947$

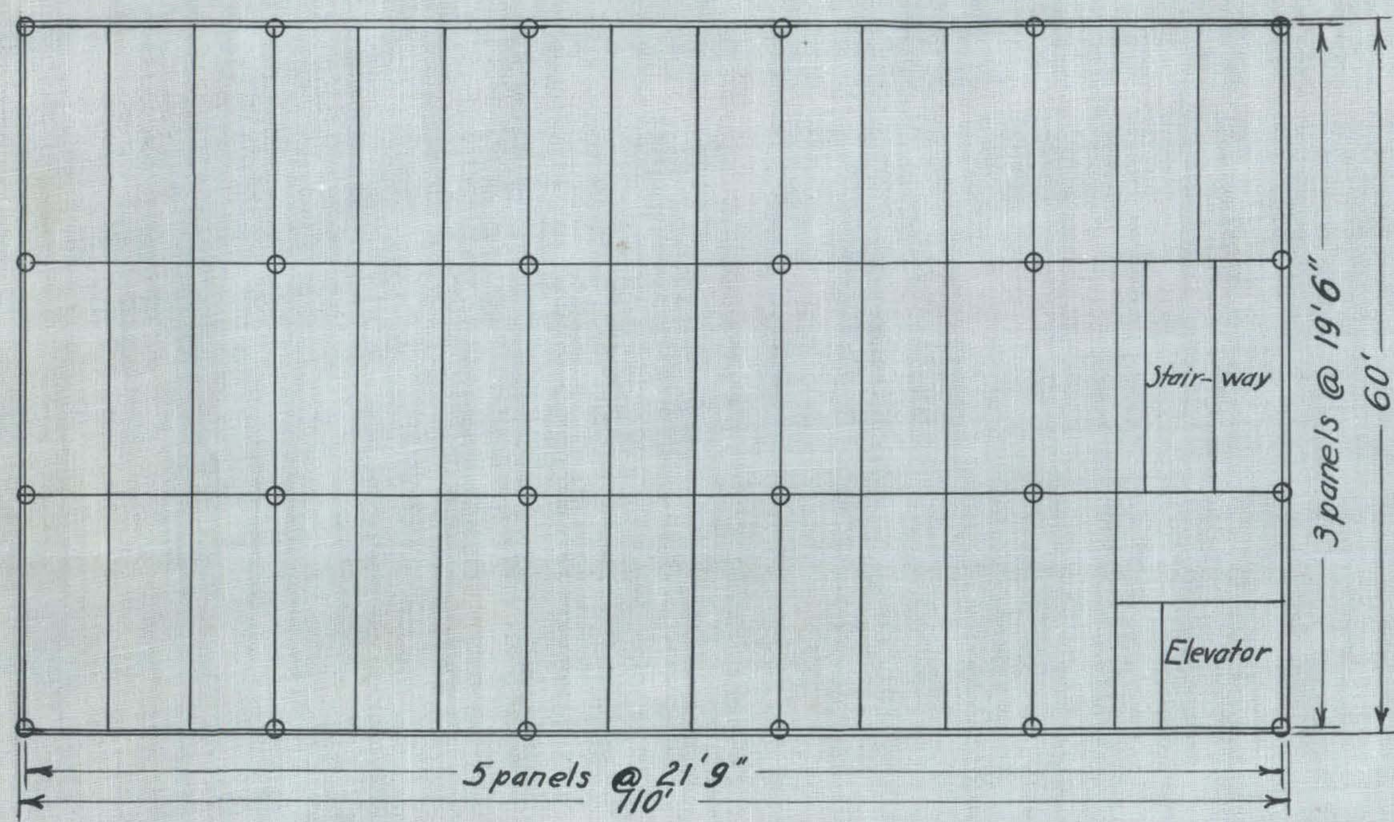
$a_s = \dfrac{14{,}400}{16000 \times 9 \times .947} = .1055 \square''$

Use 1/4" ⌀ - 6" c-c. $a_s = .125 \square''$

Use 1/4" ⌀ - 12" c-c $a_s = .0625 \square''$ longitudinally

Make bottom 12" thick d = 9" extending steel from wall into it and hook ends.

Bottom 2½' wide.



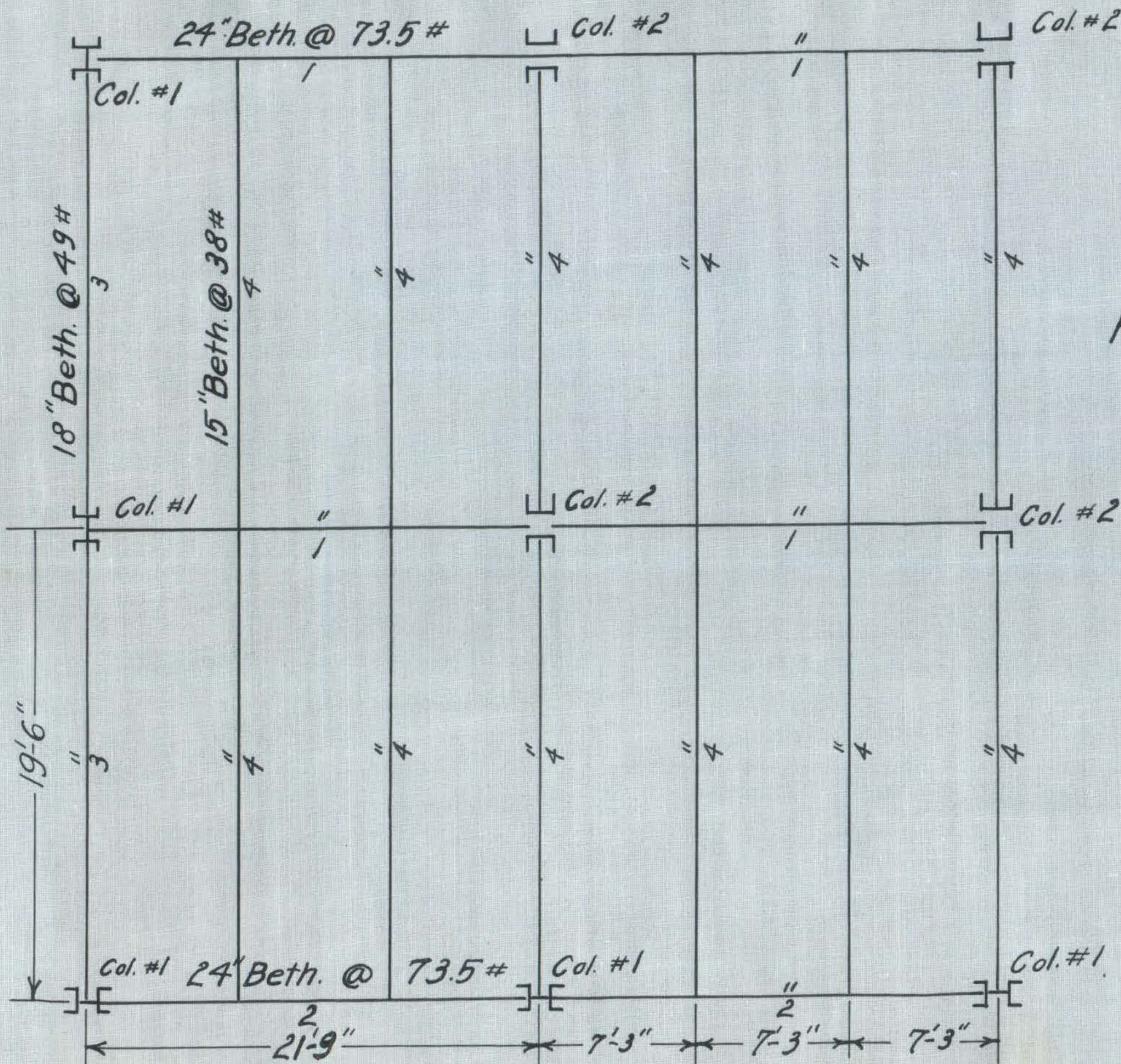
Floor Plan - Second Floor
Scale 1/16" = 1'

Height	Roof	1	2	3	4	5	6
12'		2[- 9" @ 13.25# 1I - 9" @ 21#	do	do	do	do	do
12'	3rd Floor						
14'	2nd Floor	2[- 12" @ 25# 1I - 9" @ 21# 2 Pls - 3/4" x 7"	do	do	do	do	do
10'	1st Floor						

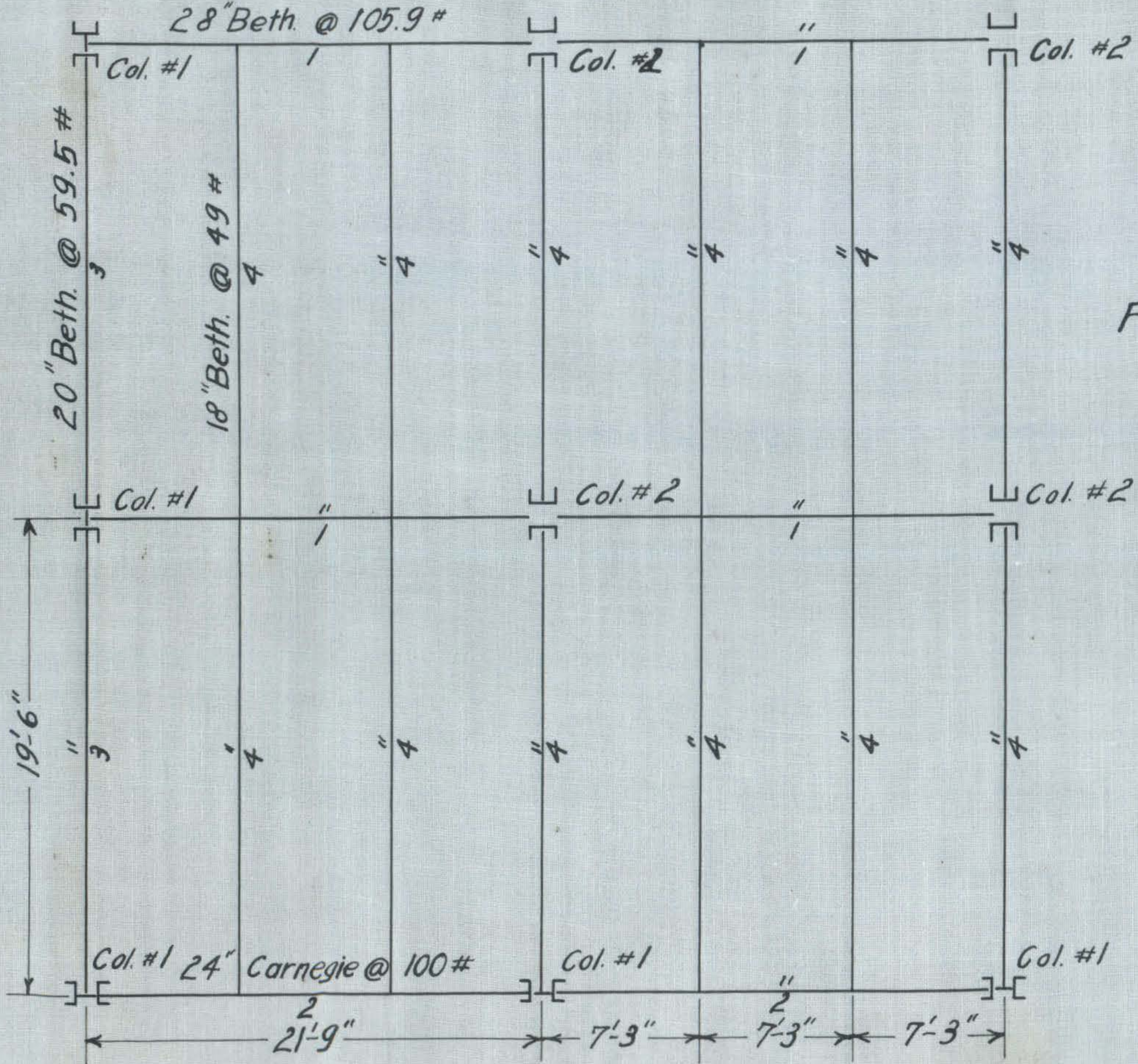
Column Schedule for Outer Rows

Height	Roof	1	2	3	4	5	6
12'		2[- 9" @ 13.25# 1I - 9" @ 21#	2[- 10" @ 20# 2 1/4" x 5/16" lacing	do	do	do	Same as #1
12'	3rd Floor						
14'	2nd Floor	2[- 12" @ 25# 1I - 9" @ 21# 2 Pls - 3/4" x 7"	2[- 10" @ 20# 2 Pls - 5/8" x 12"	do	do	do	Same as #1
10'	1st Floor						

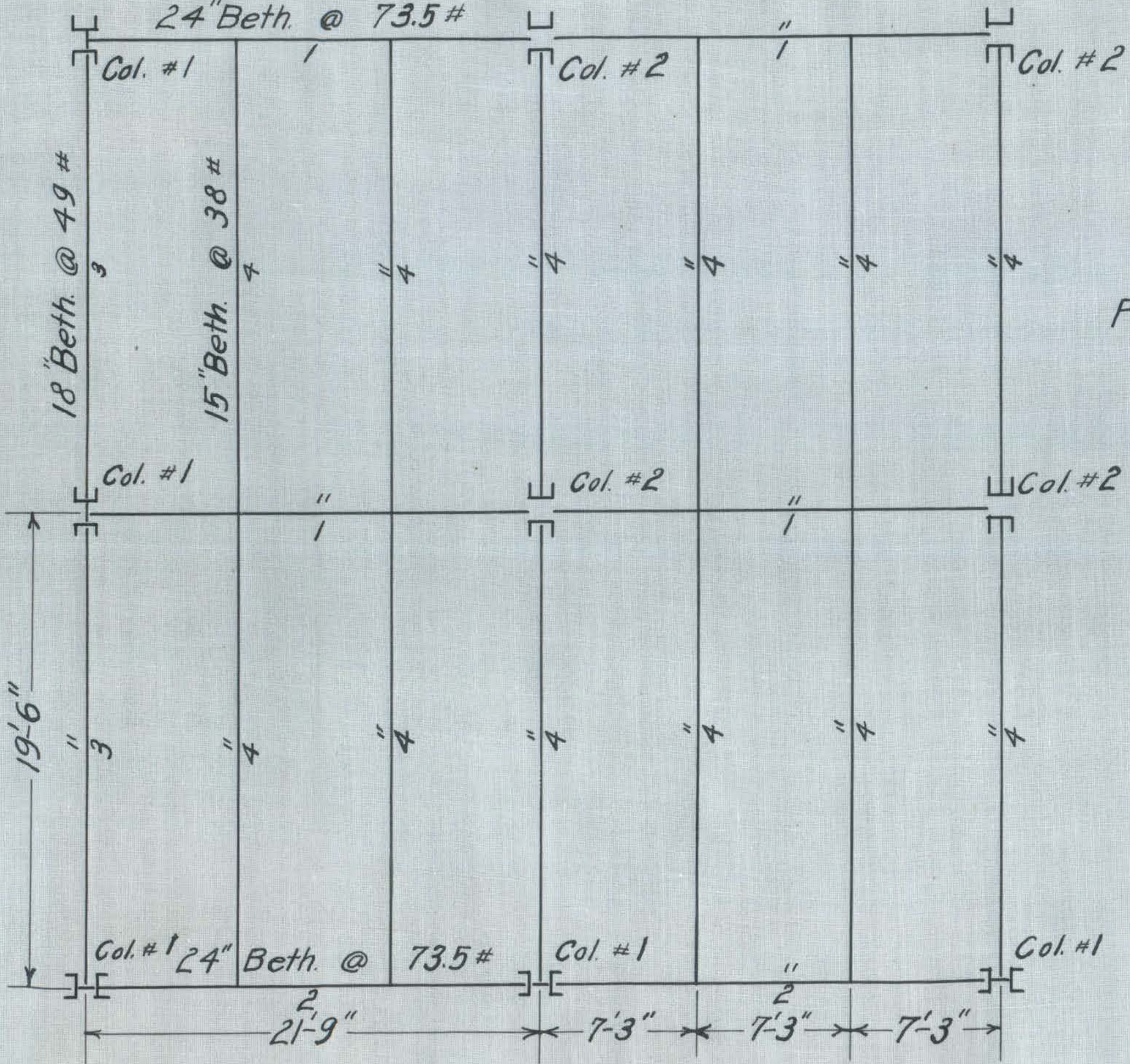
Column Schedule for Inner Rows



Floor Plan ~ First Floor
Scale $\frac{1}{8}$" = 1'



Floor Plan ~ Second Floor
Scale $\frac{1}{8}$" = 1'

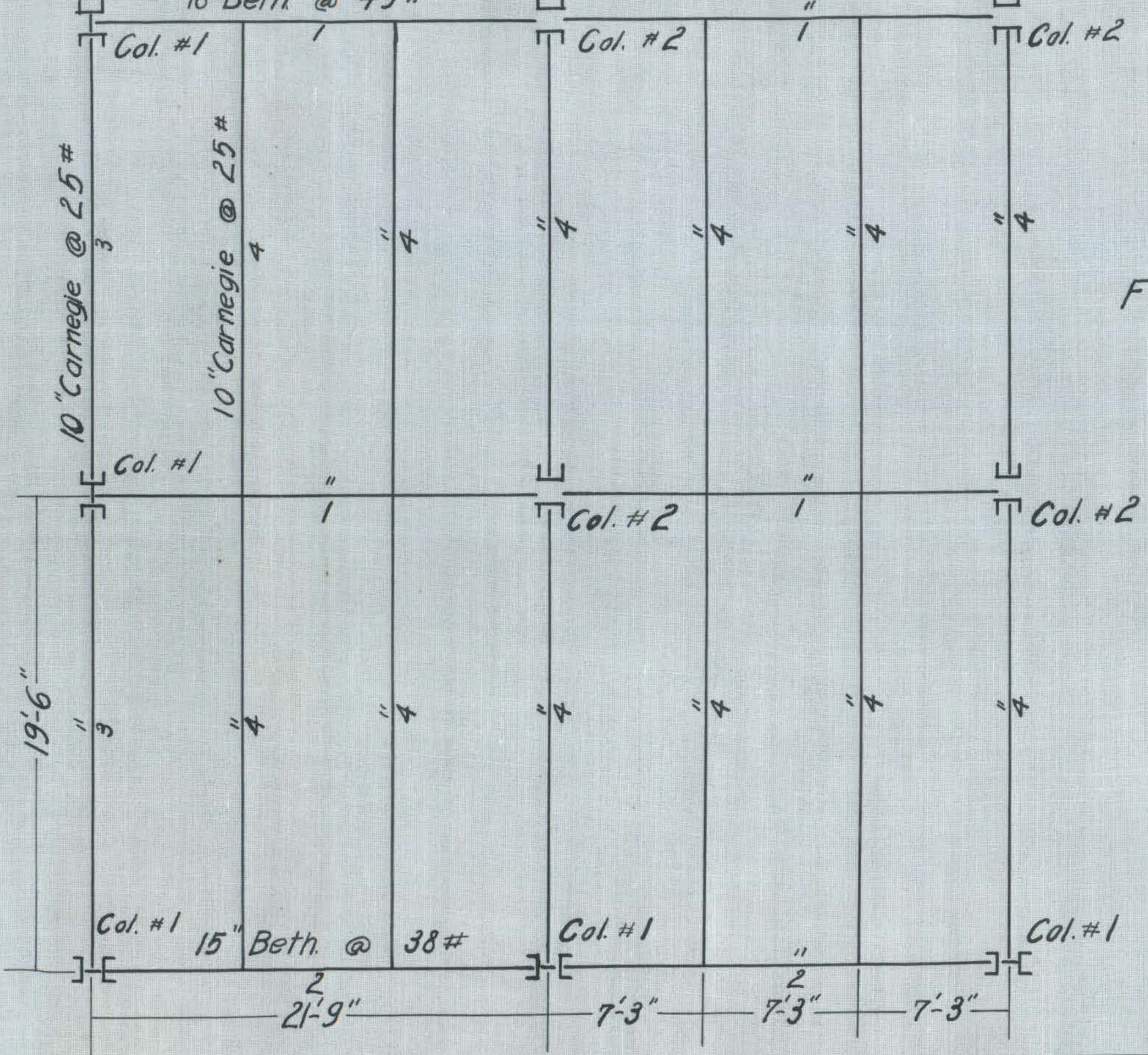


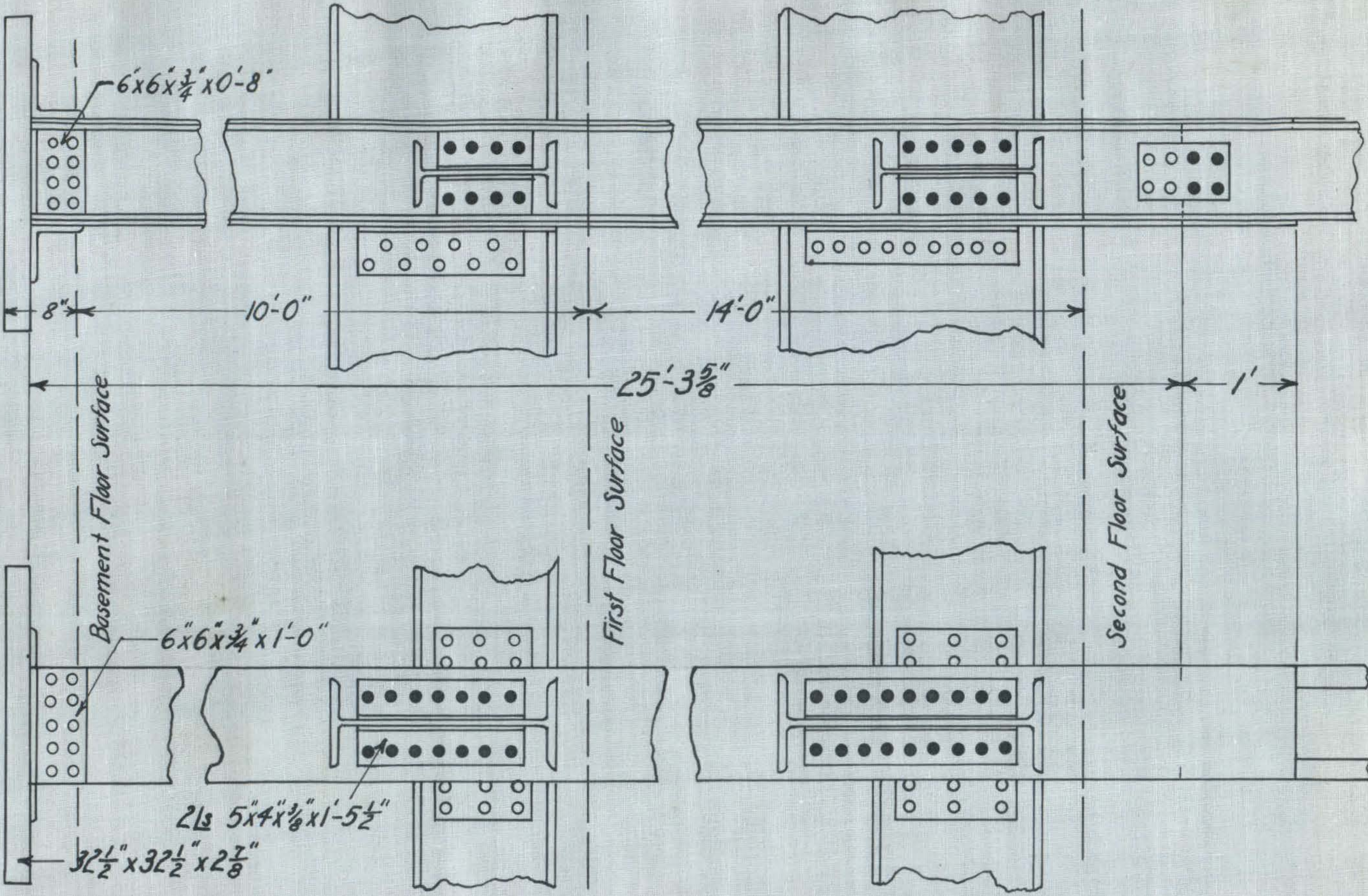
Floor Plan ~ Third Floor

Scale $\frac{1}{8}$" = 1'

Floor Plan ~ Roof

Scale $\frac{1}{8}$" = 1'



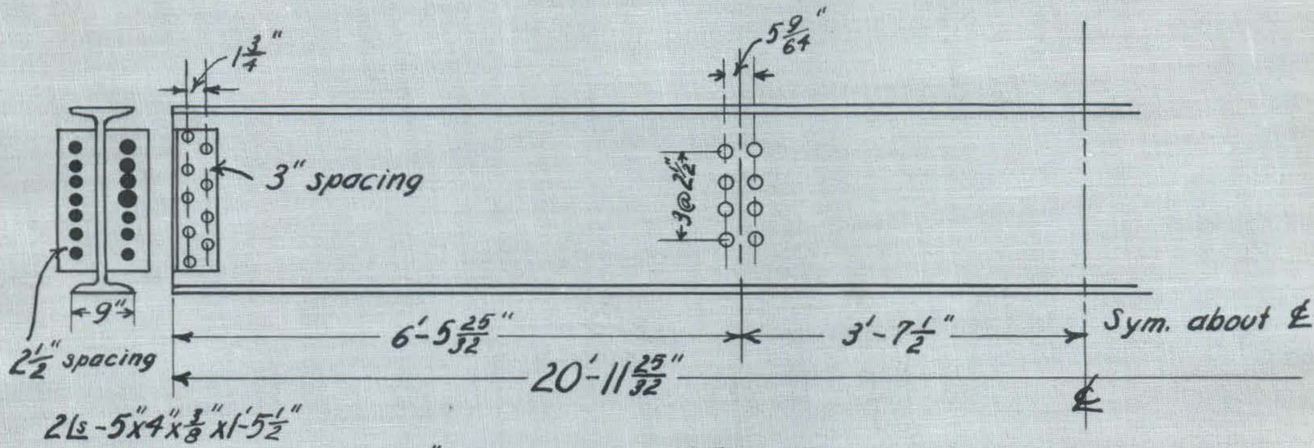


All other details are standard

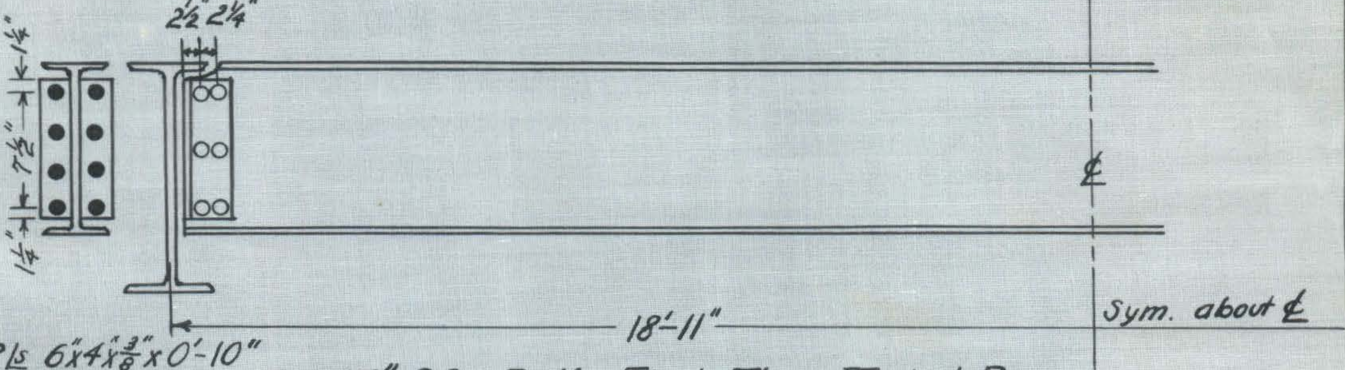
Composition ~ 2-10" [s @ 25# and 2 Pls - 12" x 5/8"

Central Column

Scale 3/4" = 1'



24"-73.5# Beth. ~ First Floor Spandrel



15"-38# Beth-First Floor Typical Beam

Other first floor girders and Beams are the same except as to length.