THE EFFECT OF EARTHQUAKES ON REINFORCED CONCRETE BUILDING DESIGN

for

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by

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THE EFFECT OF EARTHQUAKES ON REINFORCED CONCRETE BUILDING DESIGN

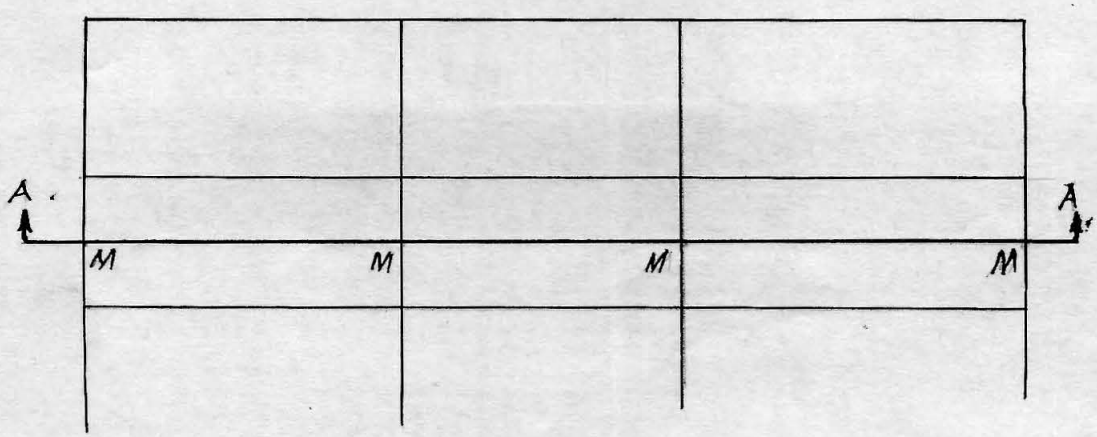
The building under consideration was the Pasadena Furniture Building, on East Colorado St., Pasadena, California. It is a modern reinforced concrete building that may be classed as a warehouse of flat-slab construction.

Instead of checking up over the entire building, one wall column and one interior column of one row of columns was taken as an example of the existing design, and calculations made on these, from the roof to the basement. It was thought that this would be a fair assumption, for the building is essentially a warehouse with regular column spacing.

In place of the forces that would result from earthquakes, a horizontal force of ten percent of the dead weight of the building was used. This was thought to be conservative, because the earthquakes are not always of the same magnitude. The average quake has an accelaration of from two to

six feet per second per second. Computing the accelaration that would result from a force of ten percent of the dead weight, it would be $F = \frac{m\,a}{g}$, or $a = \frac{F\,g}{m}$, $= \frac{1}{10}\ 32.2 = 3.22$ ft. per second per second. This is seen to be a conservative figure. It was found after obtaining the dead weights, that they differed very little from each other on each floor (roof excepted). So an even figure was used to simplify calculations, 20,000 lbs. for each floor and 40,000 lbs. for the roof.

The forces were assumed to act similarly to wind forces. That is, one tenth of the dead weight above one floor was assumed to be the shear on the center of the column directly below that floor. The stresses and moments were found by the Portal Method. The assumption (1) is made that the moment



in the columns is zero at the point M. Also (2) the horizontal shear in any plane A-A is divided by the number of aisles, therefore the outer columns take one-half the shear of an interior column. In this method of computing stresses, the difference in crossection of the columns does not affect the result. A diagram of the bending moments and direct stresses in the various members for columns 7, 18, 29, and 39, will be found on page 23 and for the plane of columns passing north and south through column 18, on page 48.

The roof beams and girders were checked and found to be sufficiently strong for bending. The required size of flat slab for the bending moment caused by earthquakes was found to be 28.2" deep. Such a depth would be out of the question. It was therefore necessary to change the style of floor construction from flat slab to beam and girder construction.

This was one of the most important results brought out by the investigation. This means that if reinforced concrete buildings are to be designed

to withstand earthquake stresses, it means that it would be necessary to design for beams and girders. This would mean deeper head room than for the flat slab construction as designed, neglecting horizontal forces, but as flat slab construction, considering horizontal forces is out of the question, it is the only way out.

A typical design of the floor system neglecting horizontal forces is given on page 29. It was assumed that the live load of 125 pounds per square foot as not acting, for this would put an unnecessary burden on the members. The bending moment caused by dead weight was added to that caused by the horizontal forces. The negative bending moment caused by the dead weight could have been calculated so as to neutralize an equal amount of positive moment, but this was considered to be negligible.

The shear at the center of the columns is the greatest and the shear at the interior columns is twice as much as that on the wall columns. As the columns are, (with hooping) they are strong

enough to stand the shear. The longitudinal steel will almost be sufficient to stand the bending moment and the direct stress. The method followed in finding the direct stresses gave values for all columns, but the direct stress on the interior columns are neutralized because of the difference in direction of the forces.

All columns are square, that is the core is circular with two inches of concrete outside of each end of a diameter, this makes the side of the square column four inches greater than the core diameter. In calculating for the necessary steel,"d" will be taken as equal to the core diameter plus two inches, "b" equal to one side of the column. Taking into account a reversal of stress, it necessary to have the same amount of steel on opposite sides of the column. The bending moments is twice as much on the interior columns as it is on the wall columns.

The above calculations were made also with horizontal forces acting north and south directions,

as far as they affect columns 7 and 18. No change will be made in the columns to stand the direct stress, for the forces can only act either north and south or east and west. It will be necessary however, to add steel on the north and south sides of the columns to stand the bending stresses.

The amount of stress that the steel that was originally in the columns would stand was considered and applied to that necessary to stand the bending moment. The amount of stress that was in excess of that which the original steel would stand was assumed to act at 16,000 pounds per square inch and the extra steel placed at thet point. With the forces acting east and west extra steel was needed on all floors for column 18 and on floors 8, 7, 6, and 5, for column7. For the north and south forces steel was needed on all except the first floor and the basement for col 18 and on floors 8, 7, 6, and 5, for column 7.

In the calculation of the steel needed in all beams and girders steel is needed both in the top and bottom of the members. Only half of the

steel was carried to the one quarter point on each beam and girder. As very little steel was needed to be added to each column, excepting columns on the 7th, and 8th floors, the extra steel was only extended to the quarter point on each side of the floors, excepting on floors 8, 7, and the basement. The volume of concrete in the columns was assumed to be the same in the old and new design.

The final results show that there is very little difference in the amount of concrete in both designs, but the amount of steel needed in the new design is more than twice as great as in the old design.

There is the question that if the original building had been of beam and girder design, whether there would have been as much difference in the amounst of steel needed. Off hand it seems that the amount of steel needed would be about one and one half times as much, as when neglecting the horizontal forces.

INDEX

Dead Weight of Members
(Col 7, 18, 29, 39)

Roof Dead Load

Column #7

$L.L = \left(\frac{18' + 19'}{2}\right) \times \frac{24.5'}{2} \times 30 = 6.798$

$DL = 39.2 - 6.8 = 32.4$ Kips

Column #18

$L.L. = \left(\frac{18 \times 19}{2}\right) \times \frac{24.5}{2} \times 30 = 13.596$

$D.L = 61.2 - 13.6 = 47.6$ Kips.

Column #29

$L.L = 13.6$

$DL = 169.7 - 13.6 = 156.1$ Kips.

Column #39

$L.L. = 6.798$

$DL. = 168.3 - 6.8 = 161.5$ Kips.

Total D.L on Roof

$32.4 + 47.6 + 156.1 + 161.5 = 397.6$ Kips

Column	#7	#18	#29	#39
Column Demensions	16X16	16X16	20X20	20X20

D.L. of Roof = 397.6 Kips

8th floor. Column #7

$L.L = \left(\frac{18+19}{2}\right)\frac{24.5}{2} \times 155 = 35.126$ — 35.126

$D.L. = 121.1 - 35.1 = 86.0$ Kips

Column #18

$L.L. = 70.2$

$DL = 176.7 - 70.2 = 106.5$ Kips

Column # 29

$L.L. = 70.2$

$DL = 281.7 - 70.2 = 211.5$ Kips

Column #39

$L.L. = 35.1$

$DL = 239.7 - 35.1 = 204.6$ Kips

Total

$86.0 + 106.5 + 211.5 + 204.6 = 608.6$ Kips

Column	#7	#18	#29	#39
Column Demensions	18X18	20X20	24X24	23X23

D.L. of 8th floor

$608.6 - 397.6 = 211.0$ Kips

7th floor

Column #7

LL = 22.5 x 22.5 x (155+125) = 63.455

DL = 201.5 - 63.5 = 138.0

Column #18

LL = 126.8

DL = 289.7 - 126.8 = 162.9

Column #29

LL = 126.8

DL = 389.7 - 126.8 = 262.9

Column #39

LL = 63.5

DL = 310.0 - 63.5 = 246.5

Total

138.0 + 162.9 + 262.9 + 246.5 = 810.3 Kips.

	7	18	29	39
Dimensions	22X22	24X24	27X27	25X25

DL. of 7th floor

810.3 - 608.6 = 201.7 Kips

6th Floor

Column #7

LL = 226.625 x (280 + 125) = .91783

DL = 280.1 - 91.9 = 188.2

Column #18

LL = 183.8

DL = 394.2 - 183.8 = 210.4

Column #29

LL = 183.8

DL = 493.7 - 183.8 = 309.9

Column #39

LL = 91.9

DL = 378.8 - 91.9 = 276.9

Total

188.2 + 210.4 + 309.9 + 276.9 = 985.4 Kips

	7	18	29	39
Dimensions	24 X 24	27 X 27	28 X 28	27 X 27

D.L. of 6th floor

985.4 - 810.3 = 175.1 Kips.

5th floor

Column #7

LL = 226.625(405+125) = 120.111

DL = 361.0 - 120.1 = 240.9

Column #18

LL = 240.2

DL = 504.7 - 240.2 = 264.5

Column #29

LL = 240.2

DL = 601.2 - 240.2 = 361.0

Column #39

LL = 120.1

DL = 450.8 - 120.1 = 330.7

Total

240.9 + 264.5 + 361.0 + 330.7 = 1197.1

7	18	29	39
26X26	28X28	30X30	29X29

D.L. of 5th floor

1197.1 - 985.4 = 211.7 Kips.

4th floor

Column #1

LL = 226.625(530+125) = 148.439

DL = 440.2 − 148.4 = 291.8

Column #18

LL = 296.8

DL = 611.9 − 296.8 = 315.1

Column #29

LL = 296.8

DL = 705.0 − 296.8 = 408.2

Column #39

LL = 148.4

DL = 521.0 − 148.4 = 372.6

Total

291.8 + 315.1 + 408.2 + 372.6 = 1387.7 Kips.

7	18	29	39
28X28	30X30	32X32	30X30

D.L. on 4th floor

1387.7 − 1197.1 = 190.6 Kips

3rd floor

Column #7

LL = 226,625(605+125) = 176,767

DL = 518.1 − 176.8 = 341.3

Column #18

LL = 353.6

DL = 718.0 − 353.6 = 354.4

Column #29

LL = 353.6

DL = 809.0 − 353.6 = 445.4

Column #39

LL = 176.8

DL = 592.0 − 176.8 = 415.2

Total

341.3 + 354.4 + 445.4 + 415.2 = 1556.3

7	18	29	39
30x30	32x32	34x34	34x34

D.L. on 3rd floor

1556.3 − 1387.7 = 168.6 Kips.

2nd floor

Column # 7

LL = 226.625 (780+125) = 205.096

DL = 594.6 − 205.1 = 389.5

Column #18

LL = 410.2

DL = 825.3 − 410.2 = 415.1

Column #29

LL = 410.2

DL = 914.0 − 410.2 = 503.8

Column #39

LL = 205.1

DL = 664.0 − 205.1 = 458.9

Total

389.5 + 415.1 + 503.8 + 458.9 = 1767.3

7	18	29	39
31X31	34X34	34X34	33X33

D.L on 2nd floor

1767.3 − 1556.3 = 2110 Kips.

1st floor

Column #7

LL = 226.625(905+125) = 233,42 #

DL = 675.4 - 233.4 = 442.0

Column #18

LL = 466.8

DL = 935.3 - 466.8 = 468.5

Column #29

LL = 466.8

DL = 1023.0 - 466.8 = 556.2

Column #39

LL = 233.4

DL = 746.8 - 233.4 = 506.6

Total

442.0 + 468.5 + 556.2 + 506.6 = 1973.3 Kips.

7	18	29	39
33X33	34X34	36X36	34X34

D.L. on 1st floor

1973.3 - 1767.3 = 206.0 Kips.

Stresses & Moments to be found by Portal Method.

Horizontal Shear in any plane is divided by number of aisles.

∴ An outer column takes but half the shear of an interior column.

In this method of computing stresses the difference in cross-section of the columns does not effect results.

Direct stress in columns

5th floor (Any aisle)

$$\frac{40}{3} \times \frac{14.33}{2} \div \frac{73}{3} = 3.92$$

direct stresses coming on interior columns from the adjacent aisles are equal in amount but opposite in direction. Therefore their algebraic sum is zero

7th floor

$$\left[40\left(14.33+\frac{11.75}{2}\right)+20\left(\frac{11.75}{2}\right)\right] \div 73 = 12.65$$

6th floor

$$\left[40\left(14.33+11.75+\frac{11.75}{2}\right)+20\left(11.75+\frac{11.75}{2}\right)+20\left(\frac{11.75}{2}\right)\right] \div 73 = 23.95$$

5th floor

$$\left[40\left(14.33+2\times 11.75+\frac{11.75}{2}\right)+20\left(2\times 11.75+\frac{11.75}{2}\right)+20\left(11.75+\frac{11.75}{2}\right)+20\left(\frac{11.75}{2}\right)\right] \div 73 =$$

38.49

4th floor

$[40(14.33+3.5\times11.75)+20(3.5\times11.75)+20(2.5\times11.75)+20(1.5\times11.75)$
$+20(.5\times11.75)]\div 73 = 56.2$

3rd floor

$[40(14.33+4.5\times11.75)+20(4.5\times11.75)+20(3.5\times11.75)+20(2.5\times11.75)$
$+20(1.5\times11.75)+20(.5\times11.75)]\div 73 = 72.2$

2nd floor

$[40(14.33+5.5\times11.75)+20(5.5\times11.75)+20(4.5\times11.75)+20(3.5\times11.75)$
$+20(2.5\times11.75)+20(1.5\times11.75)+20(.5\times11.75)]\div 73 = 101.0$

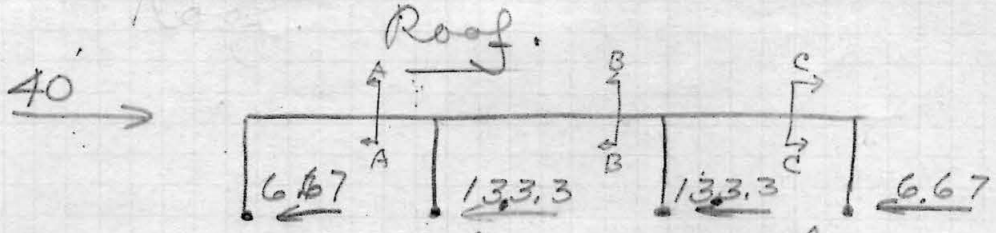
1st floor

$[40(14.33+6\times11.75+.5\times12.75)+20(6\times11.75+.5\times12.75)+20(5\times11.75+.5\times12.75)$
$+20(4\times11.75+.5\times12.75)+20(3\times11.75+12.75\times.5)+20(2\times11.75+.5\times12.75)$
$+20(1\times11.75+.5\times12.75)+20(.5\times12.75)]\div 73 = 129.6$

basement

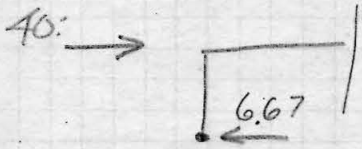
$[40(14.33+6\times11.75\times1.5\times12.75)+20(6\times11.75+1.5\times12.75)+20(5\times11.75+1.5\times12.75)$
$+20(4\times11.75+1.5\times12.75)+20(3\times11.75+12.75\times1.5)+20(2\times11.75+1.5\times12.75)$
$+20(1\times11.75+1.5\times12.75)+20(1.5\times12.75)+20(.5\times12.75)]\div 73 = 162.7$

Direct Stresses in Beams



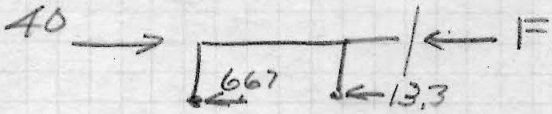
all aisles take equal shears.

Section A-A



$F = 40 - 6.67 = 33.3$

Section B-B

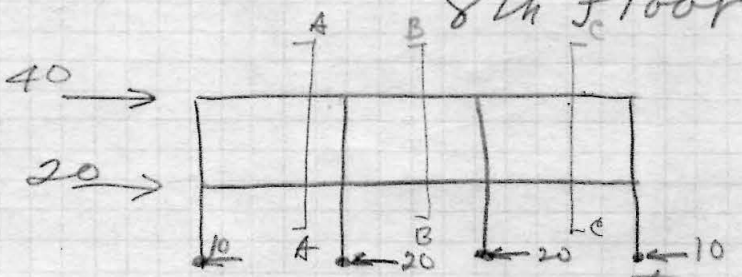


$F = 40 - 6.67 - 13.3 = 20$

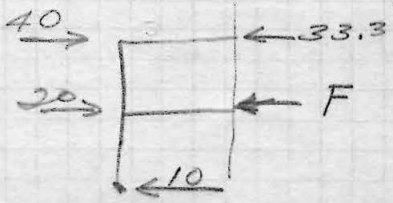
section C-C

$F = 6.67$

8th floor



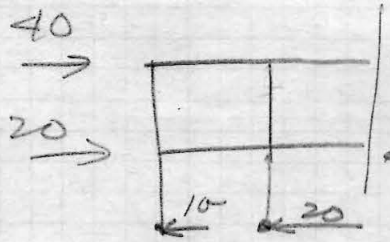
Section A-A



$F = 60 - 33.3 - 10$

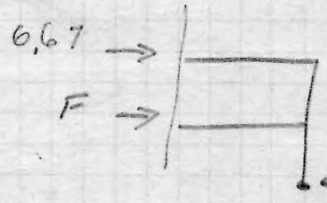
$= 16.7$

Section BB



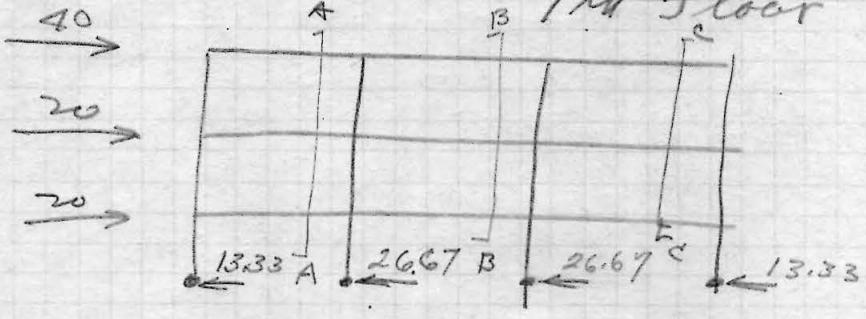
$F = 60 - 20 - 30 = 10$

Section CC

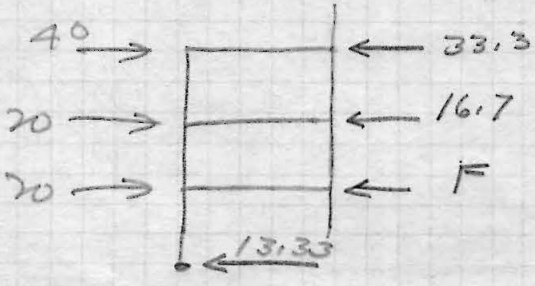


$F = 10 - 6.67 = 3.33$

7th Floor



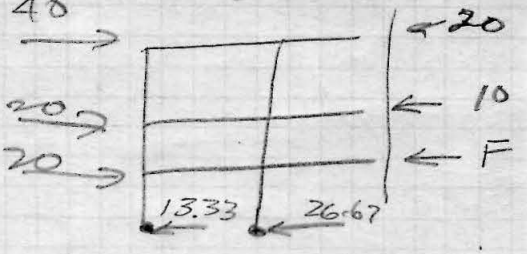
Section A.A.



$F = 80 - 33.3 - 16.7 - 13.3$

$= 16.7$

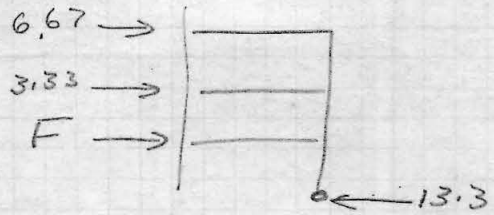
Section BB.



$F = 80 - 20 - 10 - 13.3 - 16.7$

$= 10$

Section C



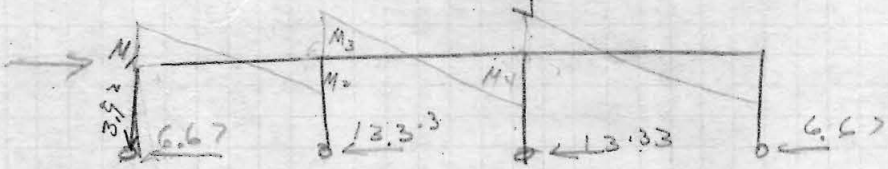
$F = 13.3 - 6.67 - 3.33 -$

$= 3.33$

These stresses will be the same in all girders of the remaining floors, that is in floors 6, 5, 4, 3, 2, 1 for sections A-A corresponding to those used in above illustrations, the stresses will be 6,7 section B-B = 10, Section C-C = 3.33

Bending Moments in Girders

Roof.



$M_1 = 6.67 \times \frac{14.33}{2} = +47.6$ ft lbs.

$M_2 = 6.67 \times \frac{14.33}{2} - 3.92 \times 24.33 = -47.8$

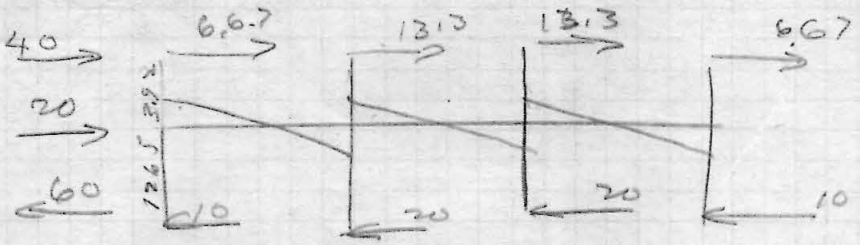
$M_3 = (6.67 + 13.33)\frac{14.33}{2} - 3.92 \times 24.33 = +47.9$

$M_4 = (6.67 + 13.33)\frac{14.33}{2} - 3.92 \times 48.66 = -47.5$

$M_5 = (6.67 + 13.33 + 13.33)\frac{14.33}{2} - 3.92 \times 48.66 = +48.2$

$M_6 = (6.67 + 13.33 + 13.33)\frac{14.33}{2} - 3.92 \times 73 = -47.7$

8th Floor



$M_1 = 6,67 \times \frac{14,33}{2} + 10 \times \frac{11,75}{2} = +106.$

$M_2 = 6,67 \times \frac{14,3}{2} + 10 \times \frac{11,75}{2} - (12,65 - 39,2)24,33 = -106.$

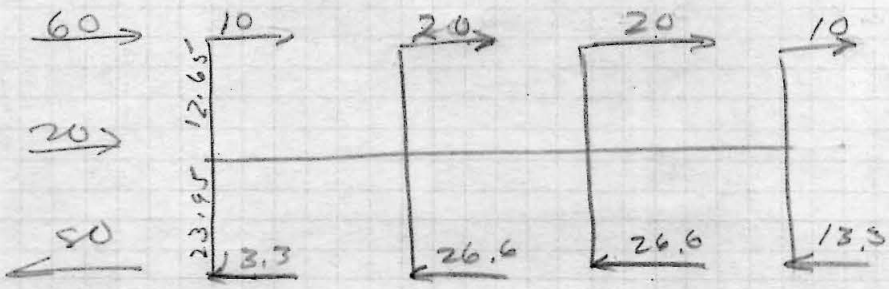
$M_3 = (6,67 + 13,3)\frac{14,3}{2} + (10+20)\frac{11,75}{2} - (12,65 - 39,2)24,33 = +106$

$M_4 = (6,67 + 13,3)\frac{14,3}{2} + (10+20)\frac{11,75}{2} - (12,65 - 39,2)48,66 = -106$

$M_5 = (6,67 + 13,3 + 13,3)\frac{14,3}{2} + (10+20+20)\frac{11,75}{2} - (12,65 - 39,2)48,66 = +106$

$M_6 = (6,67 + 13,3 + 13,3)\frac{14,3}{2} + (10+20+20)\frac{11,75}{2} - (12,65 - 39,2)73 = -106$

7th Floor



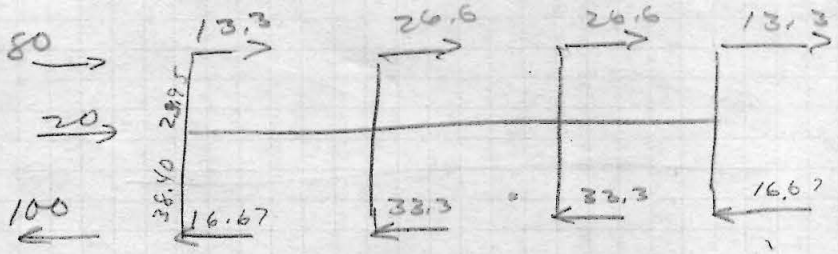
$M_1 = (10 + 13,3)\frac{11,75}{2} = +137,0$

$M_2 = (10 + 13,3)\frac{11,75}{2} - (23,95 - 12,65)24,33 = -138$

$M_3 = (10 + 13,3 + 20 + 26,6)\frac{11,75}{2} - (23,95 - 12,65)24,33 = -137$

$-M_4 = +M_5 = -M_6 = 137$

6th Floor



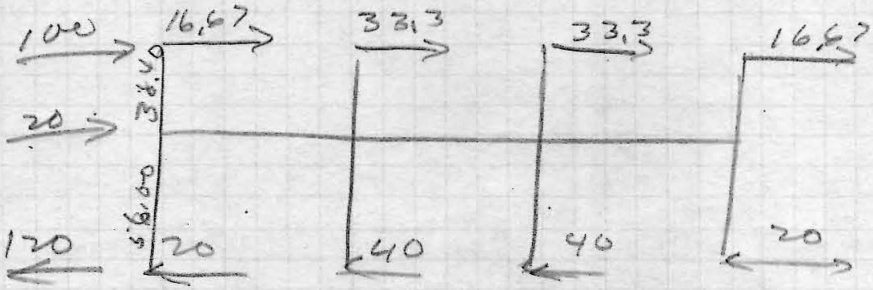
$M_1 = (13.3 + 16.67) \frac{11.75}{2} = +176.3$

$M_2 = (13.3 + 16.67) \frac{11.75}{2} - (38.40 - 23.95)\,24.33 = -175.7$

$M_3 = (13.3 + 16.67 + 26.6 + 33.33) \frac{11.75}{2} - (38.40 - 23.95)\,24.33 = +176.5$

$-M_4 = M_5 = -M_6 = 176.5$

5th Floor



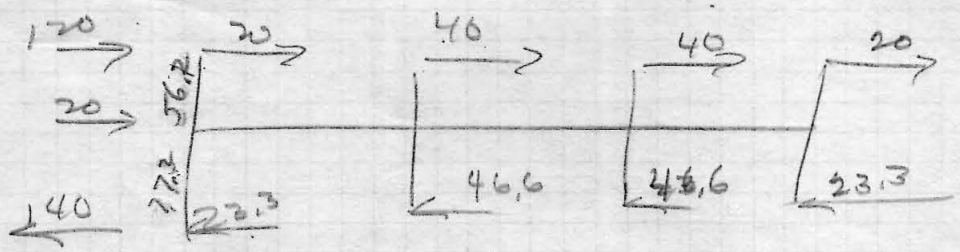
$M_1 = (16.67 + 20) \frac{11.75}{2} = +215.0$

$M_2 = (16.67 + 20) \frac{11.75}{2} - (56.730 - 38.40)\,24.33 = -216$

$M_3 = (16.67 + 20 + 33.3 + 40) \frac{11.75}{2} - (56.730 - 38.40)\,24.33 = +215$

$-M_4 = M_5 = -M_6 = 215$

4th Floor



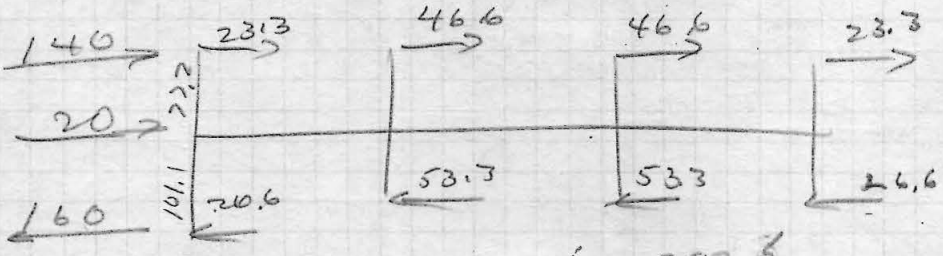
$M_1 = (20+23.3)\frac{11.25}{2} = +254.5$

$M_2 = (20+23.3)\frac{11.25}{2} - (77.2-56.20)24.33 = -255$

$M_3 = (20+23.3+46.+46.6)\frac{11.25}{2} - (77.2-56.2)24.33 = +254$

$-M_4 = M_5 - M_6 = 254.5$

3rd Floor



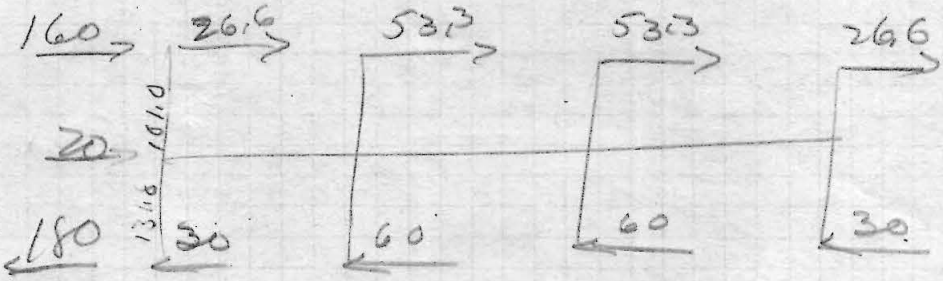
$M_1 = (23.3+26.6)\frac{11.25}{2} = +293.5$

$M_2 = (23.3+26.6)\frac{11.25}{2} - (101.0-77.2)24.33 = -294.5$

$M_3 = (23.3+26.6+46.6+50.2)\frac{11.25}{2} - (101.0-77.2)24.33 = +294$

$-M_4 + M_5 = -M_6 = 294.0$

2nd Floor.



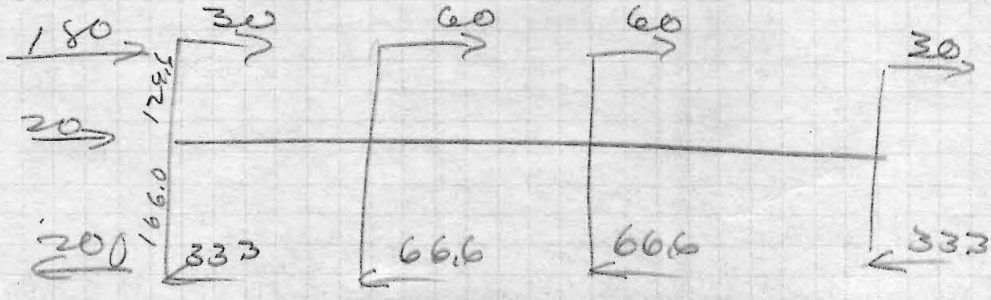
$M_1 = 26.6 \times \frac{11.25}{2} + 30 \times \frac{12.25}{2} = -347$

$M_2 = 26.6 \times \frac{11.25}{2} + 30 \times \frac{12.25}{2} - (129.6 + 101.0)\,24.33 = -347.0$

$M_3 = (26.6 + 53.3)\frac{11.25}{2} + (30+60)\frac{12.25}{2} - (129.6 - 101.0)\,24.33 = +348.0$

$-M_4 = M_5 = -M_6 = 347.7$

1st Floor.



$M_1 = (30 + 33.3)\frac{12.25}{2} = +403.0$

$M_2 = (30 + 33.3)\frac{12.25}{2} - (162.7 - 129.6)\,24.33 = -402$

$M_3 = (30 + 33.3 + 60 + 66.6)\frac{12.25}{2} - (162.7 - 129.6)\,24.33 = 405$

$-M_4 = M_5 = -M_6 = 403$

B.M. in Columns (at floor girders)

Roof

8th story col

outer col. = $6.67 \times \frac{14.33}{2} = 47.8$ } Page 15

inner col. = $13.3 \times \frac{14.33}{2} = 95.6$

8th floor

8th story col.

outer col = $6.67 \times \frac{14.33}{2} = 47.8$

inner col = $13.3 \times \frac{14.33}{2} = 95.6$

7th story col

outer col = $10 \times \frac{11.25}{2} = 58.75$ } P. 16

inner col = $20 \times \frac{11.25}{2} = 117.5$

7th Floor

7th story col

outer col = $10 \times \frac{11.25}{2} = 58.75$

inner col = $20 \times \frac{11.25}{2} = 117.5$

6th story col

outer col = $13.3 \times \frac{11.25}{2} = 78.3$ } Page 16.

inner col = $26.6 \times \frac{11.25}{2} = 156.6$

6th Floor

6th story col.

outer col = $13.3 \times \frac{11.25}{2} = 78.3$

in col = $26.6 \times \frac{11.25}{2} = 156.6$

5th story col

outer col = $16.67 \times \frac{11.25}{2} = 98.4$ } P. 17

inner col = $33.33 \times \frac{11.25}{2} = 196.8$

5th Floor

5th story col

outer col = $16.67 \times \frac{14.5}{2} = 98.4$

in col = $33.3 \times \frac{11.75}{2} = 196.8$

4th story col.

outer col = $20 \times \frac{11.75}{2} = 117.5$

in col = $40 \times \frac{11.75}{2} = 235.0$ } Pag 19

4th Floor

4th story col

outer col = $20 \times \frac{11.75}{2} = 117.5$

inner " = $40 \times \frac{11.75}{2} = 235.0$

3rd story col

outer col = $23.3 \times \frac{16.75}{2} = 137.0$

inner col = $46.6 \times \frac{16.75}{2} = 274.0$ } P. 18

3rd Floor

3rd story col

outer col = $23.3 \times \frac{11.75}{2} = 137.0$

inner col = $46.6 \times \frac{11.75}{2} = 274.0$

2nd story col

outer col = $26.6 \times \frac{11.75}{2} = 156.5$

innercol = $53.3 \times \frac{11.75}{2} = 313.0$ } P. 18

2nd Floor

2nd story col

outer col = $26.6 \times \frac{11.75}{2} = 156.5$

inner col = $53.3 \times \frac{11.75}{2} = 313.0$

1st story col

outer col = $30 \times \frac{12.75}{2} = 191.4$

inner col = $50 \times \frac{12.75}{2} = 382.8$

} P. 19

1st Floor

1st + 2nd story col

outer col = $30 \times \frac{12.75}{2} = 191.4$

inner col = $50 \times \frac{12.75}{2} = 382.8$

basement col

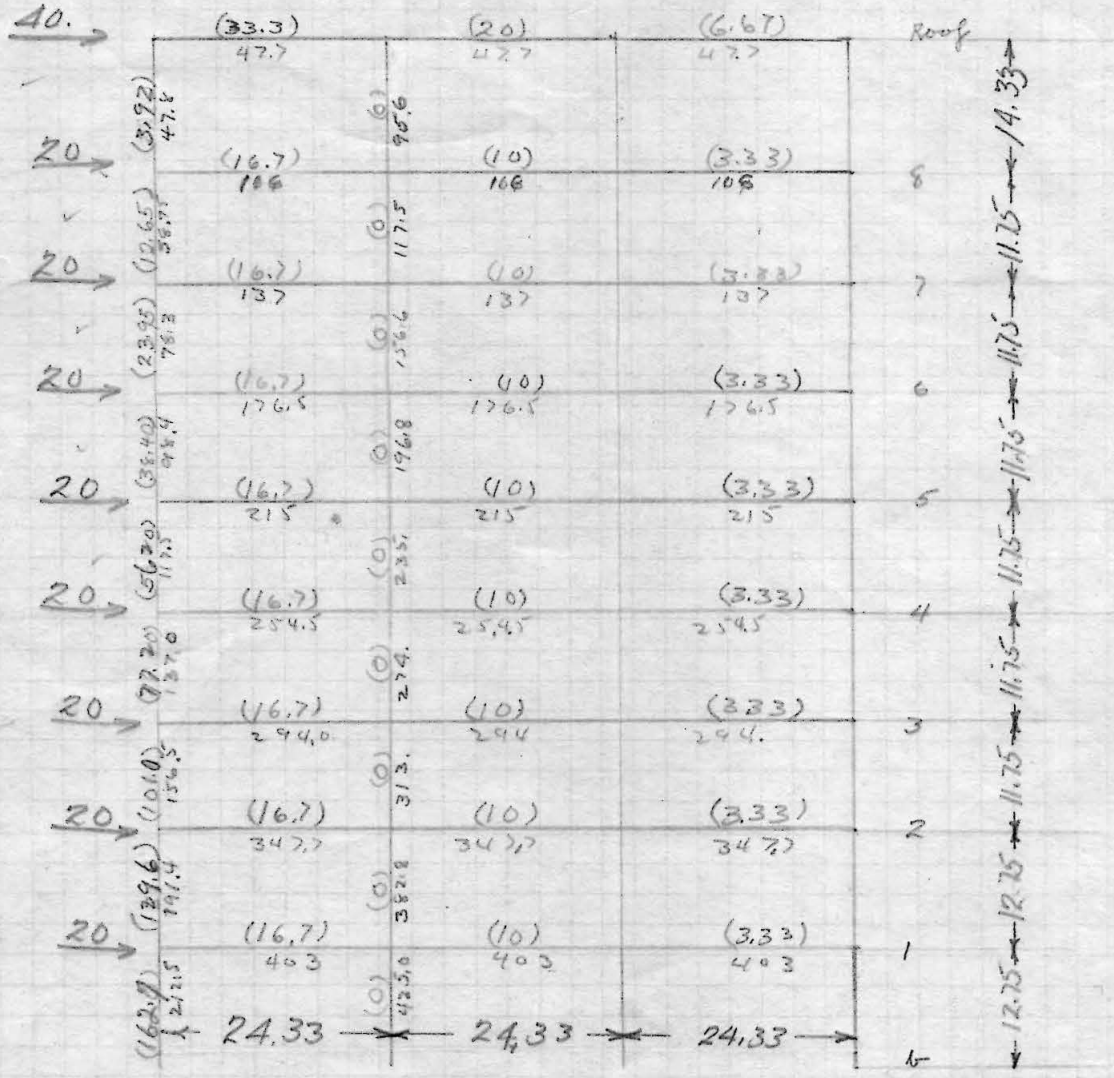
outer col = $33.3 \times \frac{12.75}{2} = 212.5$

inner col = $66.6 \times \frac{12.75}{2} = 425.0$

} P. 19.

Diagram Showing Direct Stresses and B.M.

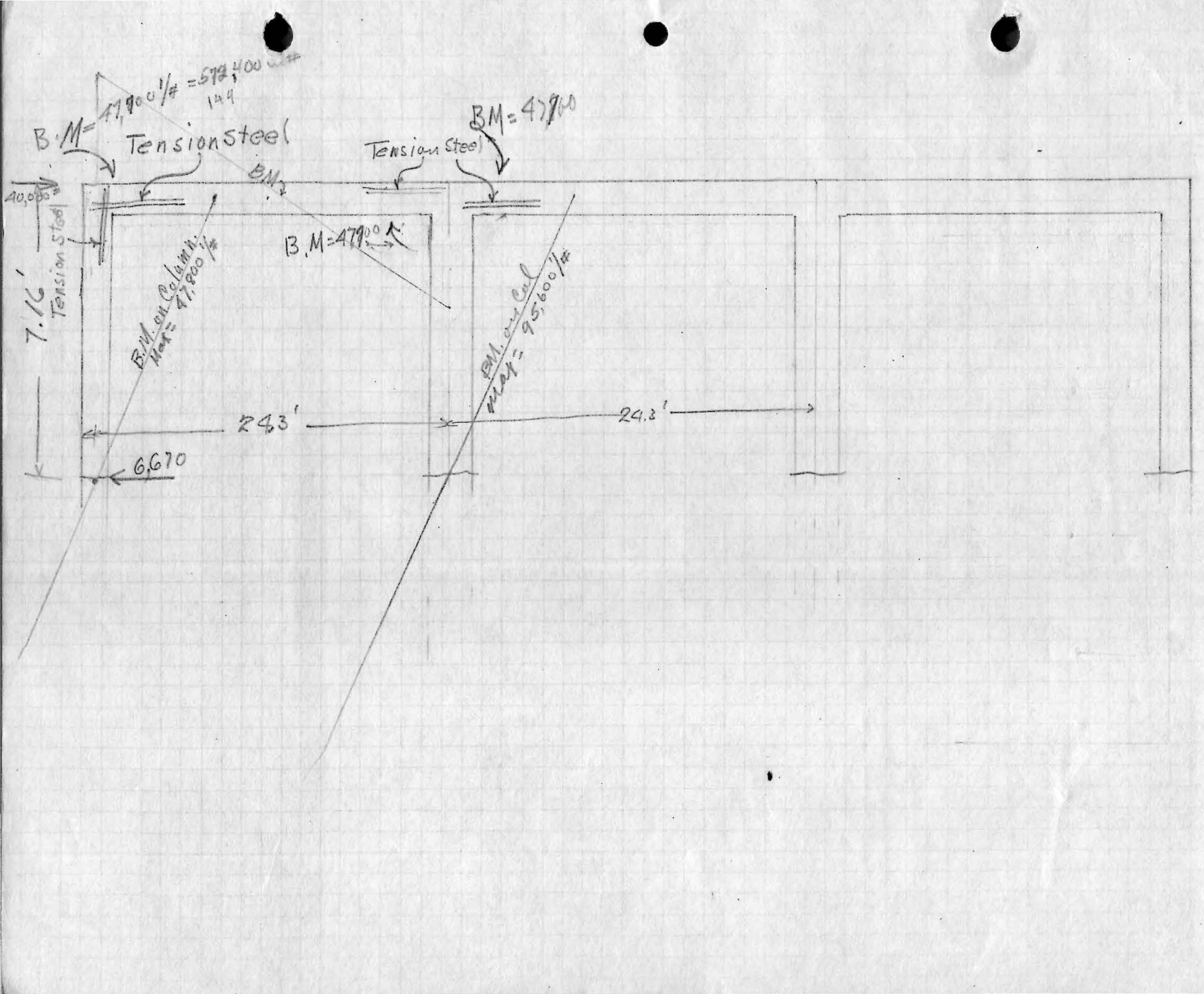
Columns #7, #18, #29, #39



#7 #18 #29 #39

(33.3) = direct stress of 33,300 lbs.
47.7 = B.M. of 47,700 ft lbs.

An average of 20 kips is used on floors
+
40 kips on roof.


B.M. = 47,900 ft#
Tension steel
B.M. = 47,900
Tension steel
B.M.
40,000 #
Tension steel
7.16'
B.M. = 47,900 ft#
B.M. on Column Max = 47,800 ft#
B.M. on Col Max = 95,600 ft#
24.3'
24.3'
6.670

Roof Beam End Span B 23

BM = 149,000

RM = 133.8 × 12 × 28 × 28 ÷ 2 = 1,260,000 in #

$a_s = .0077 \times 12 \times 28 = 1.817$ sq in.

∴ Amt. of steel necessary to carry 149000 in # = 1.817

but 4-1⅛" bars or 5.06 sq in are used.

The beam will stand 1260000 in # (p=.0077)

∴ the beam is O.K. to stand the moment due to the horizontal force (B.M. = 572,400 in #)

As shown on preceding page steel must be placed in the tension and compression side of the beam (over the supports) because of the reversal of force due to the earthmovements. But as there are 4-1⅛" bars at these points the beams will be of sufficient size + strength to carry the Bending moment induced by horizontal forces

B 22

The beam would have ultimate R.M = 1,260,000.

The steel is still more than necessary

BM caused by Horizontal force = 572,000 in #

∴ B 22 is of sufficient size + strength.

B 21

Has same characteristics as B 23 with regards to horizontal force.

Eight floor Flat slab

Strength of Original beam

$$M = \frac{19 \times 24.5 \times (125+150) \times 24.5}{30 \times 9.5} = 11000.$$

$$d = \sqrt{\frac{11000}{133.8}} = 9.1''$$ $\quad K = \frac{M}{bd^2}$

$$a_s = \frac{11000 \times 12}{16000 \times 9/8 \times 8.5} = 1.088 \text{ sq}'' \text{ per ft.}$$

use $\frac{5}{8}$ @ $4\frac{1}{2}''$ or 21 bars over the 8 ft.

18 bars were used.

Size & Strength required

B. M. = $106,500 \times 12 = 1,270,000$ in#

$$d = \sqrt{\frac{106,000}{133.8}} = 28.12''$$

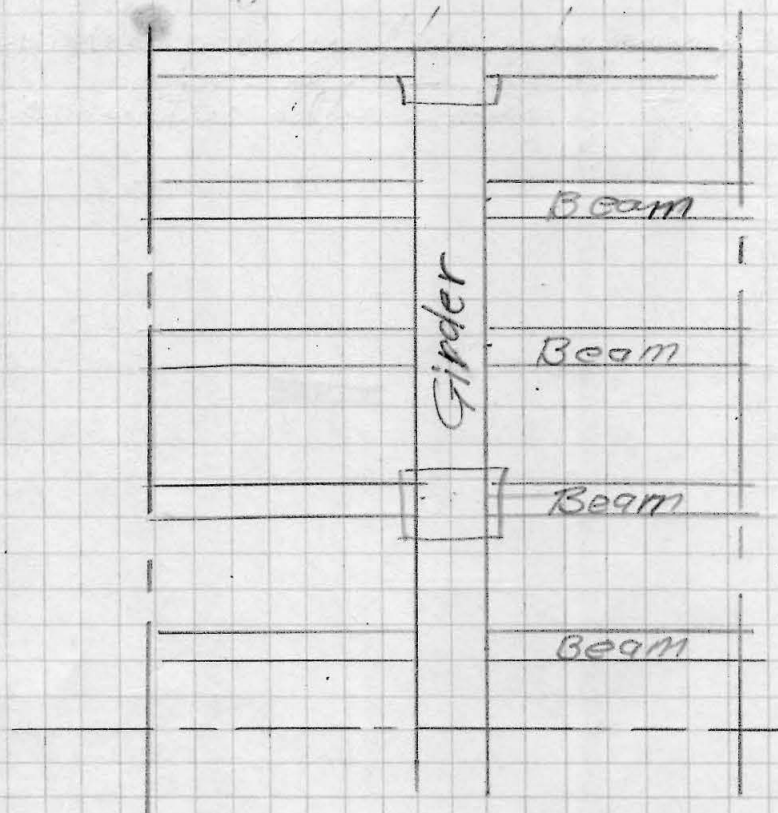
Such a depth as this would be out of the question.

It is necessary therefore to change the style of floor construction from flat slab construction to beam and girder construction.

Beam & Girder design of floor system.

8th floor.

The beams will run parallel with the longest side of the building, with the girders across the building.



Assumptions —

Floor load all goes into beams.

Beam loads act as concentrated loads on girders

Floor

D. Load = 50 #

L. Load = 125 "

175

$$BM = \frac{1 \times 175 \times 8 \times 8 \times 12}{10} = 13400 \text{ in \#}$$

$$d = \sqrt{\frac{13400}{135.8 \times 12}} = 2.9$$

use 4" slab.

$a_s = \frac{13400}{16000 \times 3 \times .875} = .32$ sq in per ft.

use $\frac{3}{8}$" sq. @ $5\frac{1}{4}$"

Beam Calculated for DL & LL.

Load = 175 x 8 = 1550/ft

$BM = \frac{1}{10} \times 1500 \times 19 \times 19 \times 12 = 672000$

at support

$BM = 672000$

$RM = 1074 \times 10 \times 17 \times 17 = 311000$

$115 \overline{)361400}$

24000

$a'_s = .0077 \times 10 \times 17 = 1.31$

$a_s = \frac{24000}{16000} = \frac{1.50}{2.81}$ T

$C_1 = \frac{24000}{6670} = 3.60$

$\frac{.0275}{.0650} = \frac{.215}{x}$ $f_s = \frac{275}{375} \times 650 = 476$

$f'_s = 476 \times (15-1) = 6670$

Stirrups

$V = 9.5 \times 1550 = 14700$ $v = \frac{14700}{11 \times 17 \times .81} = 10.5$

$\frac{W}{d} = \frac{1550}{19} = 97$

dist = 5'9" Ns of $\frac{3}{8}$" = 5

Cantilever section

assuming "j" = .92

$$a_s = \frac{692000}{16000 \times .92 \times 16} = 2.87 \text{ sq in.}$$

∴ steel in support would limit

Girder

$$B.M. = 1550 \times 19 \times 8 \times 17 + \frac{400}{10} \times 24 \times 24 \times 17$$

$$= 2,830,000 + 277,000$$

$$= 3,107,000$$

over support

$$BM = 3,107,000$$

$$RM = 133.8 \times 14 \times 30 \times 30 = 1690000$$

$$30 \overline{)1417000} \quad 47200$$

$$a'_s = .0097 \times 14 \times 30 = 4.07$$

$$a''_s = \frac{472000}{16000} = 2.95$$

$$a''_s = 7.02 \text{ Tension}$$

$$f'_c = \frac{275}{375} \times 750 = 550$$

$$f'_s = 550 \times 14 = 7700$$

$$a''_s = \frac{472000}{7700} = 6.13 \text{ Comp.}$$

center

assuming $j = .92$

$$a_s = \frac{3107000}{16000 \times .92 \times 30} = 7.05 \text{ sq. in.}$$

Stirrups

$$V = 19 \times 1550 + 400 \times 12 = 39300 \qquad 30 \times 14 \times 12 = 5040$$

$$\frac{3}{8} \times \frac{3}{4} \times 2 \times 16000 = 45000$$

$$N\,s\,\tfrac{3}{8} = \frac{39300 - 5040}{4500} = 8$$

This floor would be typical of all the floors except the roof & 1st & 2nd floors, that is:

	Concrete	Steel
Floor	4"	$\frac{3}{8}$" @ $5\frac{1}{4}$"
Beams	10 x 19 outside	2.86 sq in
Girder	14 x 32 outside	7.05 sq in

This design would be all right without considering earthquake stresses. In considering earthquake stresses it would be necessary to change the girder on each floor.

Assume live load is not acting when horizontal force is acting on bldg. (600# assume not)

$$BM \text{ then} = \frac{5}{10} \times \left(\frac{10 \times 19 \times 150}{144}\right) \times 19 \times 8 \times 12 + \frac{400 \times 24 \times 24 \times 12}{10}$$

$$= 289000 + 270000$$

$$= 559{,}000 \text{ in \#}$$

This value of BM should be added to BM caused by horizontal force.

Size of Girders
8th floor

Earth Quake BM = 106000 x 12 = 1,270000
D L " = 559000
B M = = 1,829000
R M = 133.8 x 12 x 24 x 1.1 = 925
21,6 = 1904000
41800

a'_s = 00.97 x 12 x 24 = 2.79

a''_s = 41800 / 16000 = 2.61

a'''_s = 5.40 = T

f_s = 7700

C_{as} = 41800 / 7700 = 5.45 C

7th

M_E = 137 x 12 = 1645000
B M = 559000
BM 2204000
R M = 133.8 x 12 x 26 x 1.1 = 1085
234) 1119000
47800

a'_s = 00.97 x 12 x 26 = 3.13

a''_s = 47800 / 16000 = 2.99

a'''_s = 6.12 T

O = 47800 / 7700 = 6.22 C

6th floor

$M_E = 176500 \times 12 = 2120000$

$B_M = 554$

$B_M = 2679.00$

$R_M = 133.8 \times 13 \times 28 \times 28 = 1365000$

$25.2 \overline{)1314000}$

52200

$a'_s = 0.09 \times 13 \times 28 = 3.53$

$a''_s = \frac{52200}{16000} = 3.26$

$a'''_s = 6.89$ T

$C = \frac{52200}{7700} = 6.78$ C

5th floor

$M_E = 215000 \times 12 = 2580000$

$B_M =$

$B_M = 313.9006$

$R_M = 133.8 \times 14 \times 29 \times 29 = 1575$

$26.1 \overline{)1564000}$

59800

$a'_s = 0.09 \times 14 \times 29 = 3.94$

$a''_s = \frac{59800}{16000} = 3.73$

$a'''_s = 7.67$ T

$C = \frac{59800}{7700} = 7.76$ C

4th floor

$M_E = 254500 \times 12 = 3050000$

$B_M = = 559000$

3609000

$R_M = 133.8 \times 15 \times 32 \times 30 = 1810$

$27 \overline{)1799080}$

66600

$a's = 0091 \times 15 \times 30 = 4.36$

$a'' = \frac{66600}{16000} = 4.16$

$a''' = 8.52$ T

$C = \frac{66600}{7700} = 8.65$ C

3rd floor

$M_E = 294000 \times 12 = 3530000$

$B_M = = 559$

4089000

$R_M = 133.8 \times 15 \times 32 \times 32 = 205$

$28.8 \overline{)2038.000}$

7700

$a's = 0091 \times 15 \times 32 = 4.65$

$a'' = \frac{}{16000} = 448$

$a''' = 9.13$ T

$C = \frac{7700}{7700} = 9.31$ C

2nd floor

$M_{\frac{L}{BM}} = 347200 \times 12 = 4170000$

559

$RM = 133.8 \times 16 \times 33 \times 33 \overset{BM}{=} 4729000$

233

$29.7 \quad 2394000$

80500

$a' = .0097 \times 16 \times 33 = 5.28$

$a'' = \frac{80500}{16000} = 5.03$

$a''' = 10.31$ T

$C = \frac{80500}{7700} = 10.45$ C

1st floor

$M_{\frac{L}{BM}} = 403000 \times 12 = 4840000$

559

$RM = 133.8 \times 16 \times 35 \times 35 \overset{BM}{=} 5399000$

262

$34.5 \quad 2779000$

88200

$a' = .0097 \times 16 \times 35 = 5.43$

$a'' = \frac{88200}{16000} = 5.52$

$a''' = 10.95$ T

$C = \frac{88200}{7700} = 11.45$ C

Girder Schedule

Floor	Concrete (outside)	Steel T	Steel C
8	12 x 26	5.40	5.45
7	12 x 28	6.12	6.22
6	13 x 30	6.84	6.78
5	14 x 31	7.67	7.76
4	15 x 32	8.52	8.65
3	15 x 34	9.13	9.31
2	16 x 35	10.31	10.45
1	16 x 37	10.95	11.45

The stirrups required would be approximately the same as before, that is 80 @ 9 - 3/8"
The increase in shear due to the increase in size of girder would be neutralized by the increased shear that the large beam would be able to carry

Columns

The shear at the center of the columns is the greatest and the shear at the interior columns is twice as much as that on the wall columns.

as the columns are (with hooping) they would be strong enough to stand the shear.

There is a possibility that the present longitudinal steel will also stand the bending moment caused by the earthquake stresses.

The condition that must be met ~~that~~ the column would be less likely to with stand is the direct stress ~~that~~ comes on the column.

The results of these direct stresses will be investigated first.

The method followed in finding the direct stresses gave values for all columns but the direct stresses on the interior columns were neutralized because of the differnce in direction of stress.

Direct Stresses (Col 7)

8th Floor

D.L. = 39200

Direct stress = $\frac{39}{43100}$ assume 3.0% steel.

$$P = f_c A [1 + (n-1)p]$$

$$A = \frac{P}{f_c [1 + (n-1)p]}$$

$$f_c [1 + (n-1)p] = 800 [1 + (15-1).030] = 1136$$

$$A = \frac{43100}{1136}$$

$$A = 37.9 \text{ sq in}$$

Present Col = 12 x 12 = 144 **OK**

7th Floor

DL = 1211

DS = $\frac{127}{133800}$

$$A = \frac{133800}{1136} = 117.8 \text{ sq in}$$

Present Col = 14x14 = 196 OK

6th Floor

DL = 2015

DS = $\frac{240}{2255}$

$$A = \frac{225500}{1136} = 198 \text{ sq in}$$

Present Col = $\frac{\pi \times 18 \times 18}{4}$ = 254 **OK**

5th Floor

DL = 2801
DS = 3840
3185000

$A = \frac{318000}{1136} = 280$ sq in

Present Col = $\frac{\pi \times 20 \times 20}{4} = 314$ O/K

4th Floor

DL = 3610
DS = 56.2
4172

$A = \frac{\quad}{1136} = 367$ sq in

Present Col = $\frac{\pi \times 22 \times 22}{4} = 380$

3rd floor

DL = 4402
DS = 77.2
5174

$A = \frac{517400}{1136} = 455$ sq in

Present Col = $\frac{\pi \times 24 \times 24}{4} = 453$ O/K

2nd floor

DL = 5181
DS = 10600
61910

$A = \frac{61910}{1136} = 536$ sq in

present Col = $\frac{\pi \times 26 \times 26}{1} = 531$

1st Floor

DL = 5944
DS = 1396
7342

$A = \frac{7342}{1136} = 646$ sq in

present Col = $\frac{\pi \times 27 \times 27}{4} = 573$ No.
increase to 29 = $\frac{\pi \times 29 \times 29}{4} = 661$ OK

Basement

DL = 6754
DS = 1627
8381

$A = \frac{838100}{1136} = 739$ sq in

Present Col = $\frac{\pi \times 29 \times 29}{4} = 661$ NG
increase to 31 = $\frac{\pi \times 31 \times 31}{4} = 755$ sq in OK

Bend Moments.

All columns are square, that is, the core is circular with two inches of concrete outside of each end of a diameter this makes the side of the square column four inches greater than the core diameter.

In calculating for the necessary steel "d" will be taken as equal to the core diameter plus two inches, "b" equal to the side of the column.

Taking into account a reversal of direction of force, it is necessary to have the same amount of steel on opposite side of the column.

The bending moment is twice as much on the interior columns as it is on the wall columns, so these will be checked first.

Bending Moments (Col 8)

8th floor

$M_s = p f_s j b d^2$

$p = \frac{.3906}{16 \times 14} = .00174$

$k = \sqrt{2 \times .00174 \times 15 + (.00174 \times 15)^2} - .00174 \times 15$

$k = 0.202$

$j = .923$

$b = 16$

$d = 14$

$M_s = .00174 \times 16.000 \times .923 \times 16 \times 14 \times 14$

$= 80,600$

BM on Col = $95,600 \times 12 = 1,148,000$

∴ steel must be added.

BM = 1148.000

RM = $133.8 \times 16 \times 14 \times 14$

420

12.6 | 72x000

57800

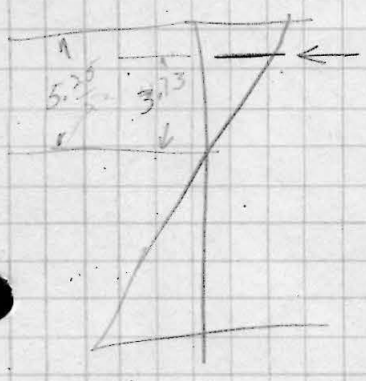
337,000

228

$a_s = 16 \times 14 \times .0077 = 2.17$

$a'_s = \frac{57800}{16000} = 3.61$

$a''_s = 5.78$ T



$k = \frac{3}{8}$

$\frac{4.73}{5.25} \times 750 = 550$

$f_s = 550 \times (15 - 1) = 7700$

Comp St. $\frac{57800}{7700} = 7.50$ in C

use 7.50 on each side to take care of reversal of stress.

7th Floor

$BM = 12 \times 117.5 = 1,410,000$

$RM = 133.8 \times 20 \times 18 \times 18 = 866$

$16.2 \overline{)\,544000}$

340000

$a_s' = .18 \times 20 \times .0097 = 3.49$

$a'' = \frac{340000}{16000} = 21.2$

$a_s'' = 5.61$ T

$\frac{340000}{7700} = 4.41$ C

6th floor

$BM = 12 \times 1566 = 1880000$

$RM = 133.8 - 24 \times 22 \times 22 = 1552$

$19.8 \overline{)\,328000}$

$16,600$

$a_s' = 22 \times 24 \times .0097 = 5.12$

$a_s'' = \frac{16660}{16000} = 1.04$

$a_s'' = 6.16$ T

$C = \frac{16600}{7700} = 2.16$

5th floor

$BM = 12 \times 196.8 = 2350000$

$RM = 1338 \times 27 \times 25 \times 25 = 226$

$22.5 \overline{)\,190,000}$

4000

$a_s' = 27 \times 25 \times .97 = 6.55$

$a_s'' = \frac{4000}{16000} = .25$

6.80 T

C is negligible.

4th floor

BM = 12 x 230 = 2820000

RM = 133.8 x 28x26x26 = 253

23.4 | 2900000

12400

a'_b = 28x26x.009 = 7.28

a''_b = 12400/16000 = .78

a'''_b = 8.06

3rd floor

BM = 12 x 274 = 3,290,000

RM = 133.8 x 30x28x28 = 3140

25.2 | 1500000

5950

a'_b = 30x28x.009 = 8.15

a''_b = 5950/10000 = .37

a'''_b = 8.52

2nd floor

BM = 12 x 313 = 3760000

RM = 133.8 x 32x30x30 = 3850000

90000 over

a''_b = 30x32x.009 = 9.30

1st floor

BM = 12x 382.8 = 4590000

RM = 133.8x34x32x32 = 4520000

37

28.8 | 70100

2430

a'_b = 34x32x.009 = 10.52

a''_b = 2430/16000 = .15

a'''_b = 10.67

Basement

BM = 12 × 425 = 5,100,000

RM = 133.8 × 34 × 32 × 32 = 454

28.8 | 590,000
20500

a_o = 34 × 32 × 0097 = 10.52

a'_b = 20500 / 16000 = 1.28

a''_b = 12.80

Wall Column # 1

8th floor

BM = 12 × 47800 = 575000

RM = 133.8 × 16 × 14 × 14 = 420000

12.6 | 155000
12300

a'_b = 0097 × 14 × 16 = 2.17

a''_b = 12300 / 16000 = 1.78

a'''_b = 2.94 T

Conf. = 28200 / 7700 = 3.66 C

7th floor

BM = 12 × 58750 = 705000

RM = 133.8 × 18 × 16 × 16 = 616000

144 | 89000
6170

a_b = 0097 × 18 × 16 = 2.77

a'_b = 6170 / 16000 = .39

a''_b = 3.68

6th floor

BM = 78300 x 12 = 940000
RM = 10.70 x 22 x 20 x 20 = 940000

a_s = 0077 x 20 x 22 = 3.39

5th floor

BM = 98400 x 12 = 1180000
RM = 10.2 x 24 x 22 x 22 = 1180000

a_s = 0073 x 24 x 22 = 3.85

4th floor

BM = 117500 x 12 = 1410000
RM = 94.5 x 26 x 24 x 24 = 141

a_s = 0067 x 24 x 26 = 4.18

3rd floor

BM = 137000 x 12 = 1645000
RM = 88 x 28 x 26 x 26 = 1645

a_s = 0063 x 26 x 28 = 4.58

2nd floor

BM = 156500 x 12 = 1880000
RM = 80.0 x 30 x 28 x 28 = 1880000

a_s = 0056 x 30 x 28 = 4.76

1st floor

BM = 191400 x 12 = 23000000
RM = 88.1 x 31 x 29 x 29 = 23000000

a_s = 0063 x 29 x 31 = 5.65

Basement

BM = 212,500 x 12 = 2,550,000

RM = 80.5 x 33 x 31 x 31 = 255

as = .0057 x 31 x 33 = 5,84

The ratio of the steel in the N+S sides of col #7 to the steel in the E+W sides will be approximately the same ratio as in Col #8 See pages 40-43 + 57-58

The ratio of N+S steel to E+W steel in Col #8 is about 1:2, the may be applied to col 7

Force North & South

Considering Columns 11 to 21

Dead Load

Roof

$LL = \left(\frac{24.5+24}{2}\right) 170 \times 30 = 123800$

$DL = 36.0 + 35.2 + 90.0 + 92.2 + 197.7 + 263.6 + 265.7 + 61.2 + 92.2 + 73.6 - 123.8 = 1023.6$

8th floor

$LL = \left(\frac{24.5+24}{2}\right) \times 170 \times 125 = 515000 + 123.8 = 6388$

$DL = 94.7 + 89.5 + 203.2 + 208.7 + 308.2 + 374.1 + 321.2 + 176.7 + 207.7 + 115.5 + 157.2 - 6388 = 1597.9$

7th floor

$LL = 515000 \times 2 = 330000 + 123.8 = 11538$

$DL = 152.8 + 143.1 + 312.0 + 318.7 + 414.9 + 480.3 + 432.2 + 289.7 + 320.7 + 226.5 + 239.2 - 11538 = 22558$

6th floor

$LL = 515 \times 3 + 1238 = 1668.8$

$DL = 210.3 + 196.3 + 416.0 + 425.2 + 516.4 + 582.3 + 538.7 + 394.2 + 425.2 + 340.9 + 319.9 - 1668.8 = 2696.6$

5th floor

$LL = 515 \times 4 + 123.8 = 2183.8$

$DL = 266.6 + 248.4 + 520.2 + 531.8 + 648.5 + 684.8 + 645.3 + 564.7 + 531.9 + 451.2 + 397.4 - 2183.8 = 3217.0$

4th floor

$LL = 515 \times 5 + 123.8 = 2698.8$

$DL = 324.6 + 305.6 + 625.9 + 639.0 + 721.8 + 787.4 + 732.8 + 611.9 + 646.9 + 559.4 + 480.0 - 2698.8 = 3949.6$

3rd floor

$LL = 515 \times 6 + 123.8 = 3213.8$

$DL = 381.0 + 359.3 + 731.9 + 741.7 + 826.5 + 892.1 + 861.5 + 718.0 + 741.52 + 664.8 + 560.0 - 3213.8 = 4274.2$

2nd floor

LL = 515 x 7 + 123.8 = 3728.8

DL = 4921.0 + 4154 + 8169 + 8050 + 8815 + 9470 + 9411.5 + 8253 + 8515 + 7649.6 + 641.3 − 3728.8 = 47022

1st floor

LL = 515 x 8 + 123.8 = 4243.8

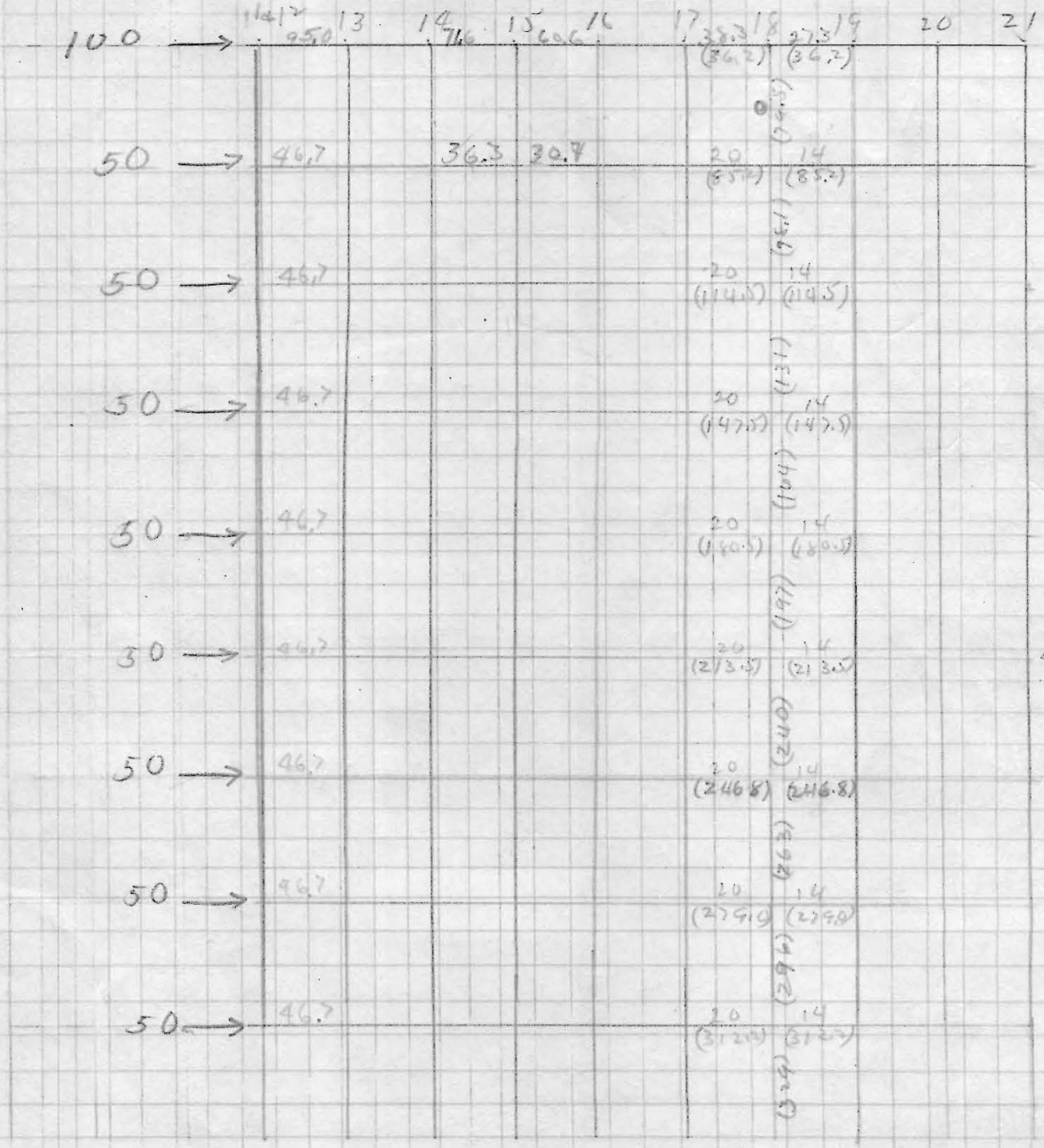
DL = 5715 + 4857.8 + 9149.9 + 9157 + 9885 + 10531 + 10501.5 + 9353 + 9614.5 + 8278.5 + 7211.4 − 4243.8 = 52016

Weight of individual floor in 1000#

Roof	1023.6 − 0	=	1023.6
8th	1597.7 − 1023.6	=	474.3
7th	22558 − 1597.9	=	657.9
6th	26966 − 22558	=	440.8
5th	32170 − 26966	=	520.4
4th	37496 − 32170	=	532.6
3rd	42742 − 37496	=	524.6
2nd	47022 − 42742	=	428.0
1st	52016 − 47022	=	504.4
		8	4078.0
			509.7

This means 50000# at each floor and 100000 at the roof ($\frac{1}{10}$ of Dead Weight)
Our attention will be taken mainly to the effects on Column #18; important results that affect the other members will be noted, however.

West Elevation



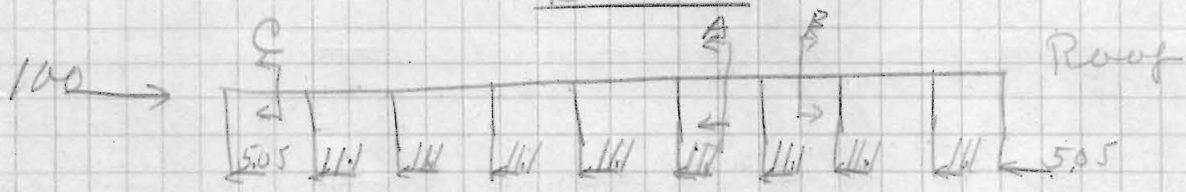
direct stress –

BM : ( )

Direct Stresses

It was shown in the other solution that there were two stresses of an equal an opposite nature being applied to all interior columns so the resultant would be zero

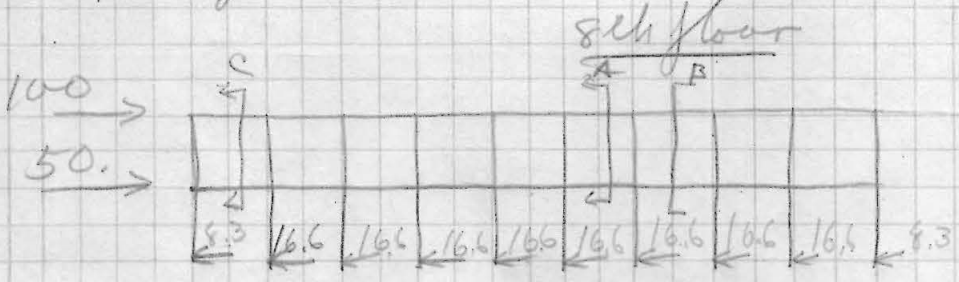
Beams



F at A = 38.35

F at B = 27.25

F greatest at C = 95



F at A = 58.3 − 38.3 = 20

F at B = 41.6 − 27.2 = 14.4

F greatest at C = 46.7

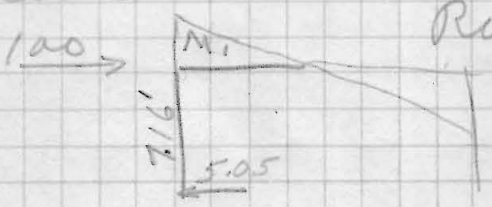
These stresses will be the same for all the remaining floors

The direct stresses vary inversely as the distance from the load.

B.M. in Beams

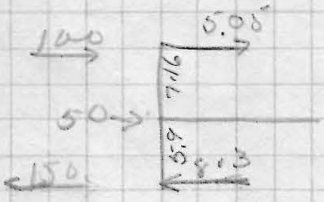
It was found that the B.M. in each beam on the individual floors was the same.
This being the case we will take the simplest

Roof



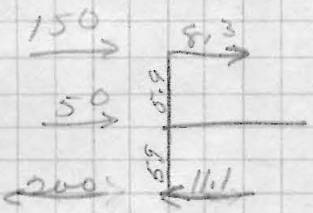
$M = 5.05 \times 7.16 = 36.2^{\#}$

8th floor



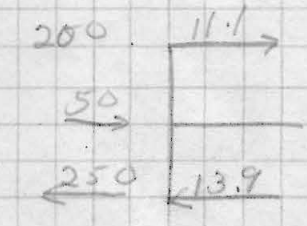
$M = 5.05 \times 7.16 + 8.13 \times 5.9 = 85.2$

7th floor



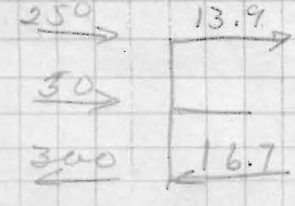
$M = (8.13 + 11.1)\ 5.9 = 114.5$

6th floor



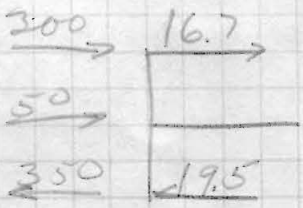
$M = (11.1 + 13.9)\ 5.9 = 147.5$

5th floor



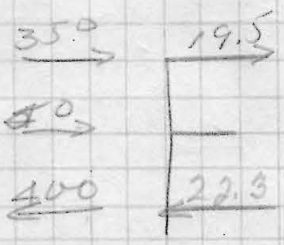
$M = (16.7 + 13.9)\ 5.9 = 180.5$

4th floor



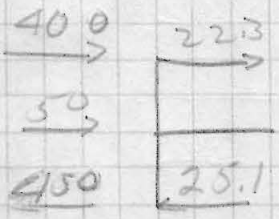
$M = (16.7 + 19.5)\ 5.9 = 213.5$

3rd floor



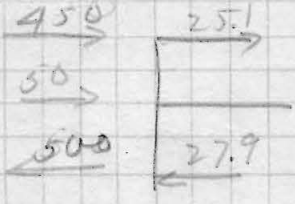
$M = (19.5 + 22.3)\ 5.9 = 246.8$

2nd floor



$M = (22.3 + 25.1)\ 5.9 = 279.0$

1st floor



$M = (25.1 + 27.9)\ 5.7 = 312.2$

B.M. in Columns.

The B.M. on interior columns is taken as same as on the wall columns and will be alone considered.

Floor	expression	B M
Roof	11.1 x 7.16	79.5
8	16.6 x 5.9	98.1
7	22.2 x 5.9	131.0
6	27.8 x 5.9	164.0
5	33.4 x 5.9	197.0
4	39.0 x 5.9	240.0
3	44.6 x 5.9	263.0
2	50.2 x 5.9	296.0
1	55.8 x 5.9	329.0

Beams Design.

Roof

BM = 36.2 × 12 = 432000

RM = 133.8 × 12 × 20 × 20 = 642000

D.L BM is negligible

$a' = .0097 \times 12 \times 20 = 2.33$

present steel = 1.56

∴ increase to 2.33

8th floor

BM = 85.2 × 12 = 1,020,000

RM = 133.8 × 11 × 19 × 19 = 528000

17.1 | 492000

2880

$a_s = .0097 \times 11 \times 19 = 2.03$

$a''_s = \frac{2880}{16000} = \frac{180}{3.83}$ T

$\frac{28800}{7700} = 375$ C

7th floor

BM = 12 × 1145 = 1375000

RM = 133.8 × 12 × 21 × 21 = 709

19.9 | 666000

33200

$a_s = 12 \times 21 \times .0097 = 2.44$

$\therefore a_s = \frac{33200}{16000} = 2.07$

$a'''_s = 451$ T

$C = \frac{33200}{7700} = 4.31$

6th floor

BM = 12 x 147.5 = 1770000

RM = 133.8 x 13 x 22.5 x 22.5 = 880000 / 490000 / 43900
(20,25)

a'_b = 13 x 22.5 x 0.097 = 2,83

a''_b = 43900 / 16000 = 2.74

a''_b = 5.57 T

C = 43900 / 7700 = 5.70 C

5th floor

BM = 12 x 180.5 = 2165000

RM = 133.8 x 13 x 25 x 25 = 1085 / 1080000 / 43200
(22.5)

a'_b = 13 x 25 x 0.097 = 3.13

a''_b = 43200 / 16000 = 2.70

a''_b = 5,83 T

C = 43200 / 7700 = 5,61 C

4th floor

BM = 12 x 213.5 = 2560000

RM = 133.8 x 14 x 26.5 x 26.5 = 1314 / 1246 / 52200
(23.85)

a'_b = 14 x 26.5 x 0.097 = 3.60

a''_b = 52200 / 16000 = 3.26

a''_b = 6.86 T

C = 52200 / 7700 = 6.77 C

3rd floor

BM = 12 × 246.8 = 2850000

RM = 133.8 × 14 × 28 × 28 = 1470000

25.2 | 1380000

54800

a'_s = 14 × 28 × 0097 = 3.80

a''_s = 54800 / 16000 = 3.42

a'''_s = 7.20 ✓

C = 54800 / 7700 = 7.13 C

2nd floor

BM = 12 × 279. = 3350000

RM = 133.8 × 14 × 30 × 30 = 1685000

27 | 1665000

616000

a'_s = 14 × 30 × .0097 = 4.08

a''_s = 616000 / 16000 = 385

a'''_s = 7.93 ✓

C = 61600 / 7700 = 8.00 C

1st floor

BM = 12 × 312.2 = 3750000

RM = 133.8 × 15 × 31 × 31 = 1930000

27.9 | 1820000

65300

a'_s = 15 × 31 × 0097 = 4.52

a''_s = 65300 / 16000 = 4.18

a'''_s = 8.70 ✓

C = 65300 / 7700 = 8.47 C

No change will be made in the columns to stand the direct stress, for the force can only act either north and south or east and west.

It will be necessary however to add steel on the north and south sides of the columns however to stand the Bending moment.

8th floor

BM = 12 × 79.5 = 955000

RM = 133.8 × 16 × 14 × 14 = 420000

12.6 | 535000
42500

a_s = 16 × 14 × .0097 = 217

a_s = 42500 / 16000 = 265

a_{tb} = 4.82 T

C = 42500 / 7700 = 5.52 C

7th floor

BM = 12 × 98.1 = 1178000

RM = 133.8 × 20 × 18 × 18 = 867000

16.2 | 311000
19200

a_s = 20 × 18 × .0097 = 3.49

a_s = 19200 / 16000 = 120

a_{tb} = 4.69 T

C = 19200 / 7700 = 2.49 C

6th floor

BM = 12 × 131 = 1570000

RM = 133.8 × 24 × 22 × 22 = 155

19.8 | 20000
1010

a_s = 24 × 22 × .0097 = 5.12

a_s = 1010 / 16000 = .06

a_{tb} = 5.18

5th floor

BM = 12 x 164 = 1970000
RM = 116.5 x 27 x 25 x 25 = 197

a's = 27 x 25 x .0081 = 5.46

4th floor

BM = 12 x 197 = 237
RM = 125 x 28 x 26 x 26 = 237

as = 28 x 26 x .0090 = 6.54

3rd floor

BM = 12 x 240 = 2880000
RM = 122.5 x 30 x 28 x 28 = 288

as = 28 x 30 x .0088 = 7.80

2nd floor

BM = 12 x 263 = 3160000
RM = 109.6 x 32 x 30 x 30 = 3160000

as = 32 x 30 x .0079 = 7.58

1st floor

BM = 12 x 296 = 3570000
RM = 102.5 x 34 x 32 x 32 = 3570000

as = 32 x 34 x .0073 = 7.94

Basement

BM = 12 x 329 = 3950000
RM = 113.5 x 34 x 32 x 32 = 3950000

as = 34 x 32 x .0081 = 8.80

Amount of steel steel that is necessary to be added to Col. 18 (E & W BM)

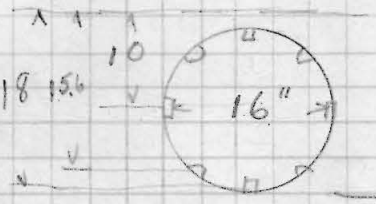
8th floor.

Col. 18 has a square core on 8th floor with 4-$\frac{5}{8}$" bars. one of these bars on each side which can be included in the steel needed.

Steel needed = 7.50 area $\frac{5}{8}$" Rod = .39

∴ add 7.09 sq in.

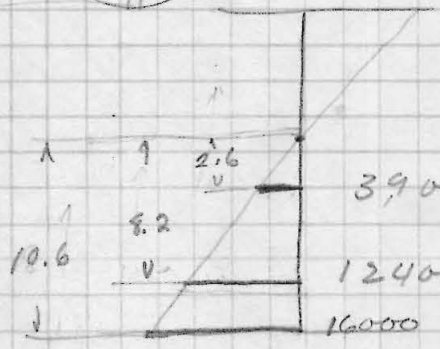
7th floor



8 - 3/4" bars.

$p = .0097$ $k = .410$

$.41 \times 18 = 7.4$



$3900 \times 2 \times .56 = 4360$

$12400 \times 2 \times .56 = 13900$

$16000 \times 1 \times .56 = 895$

$27210\#$

amt steel required at 18" for BM = 5.61

$5.61 \times 16000 = 89600$

amt supplied $= 27210$

62390

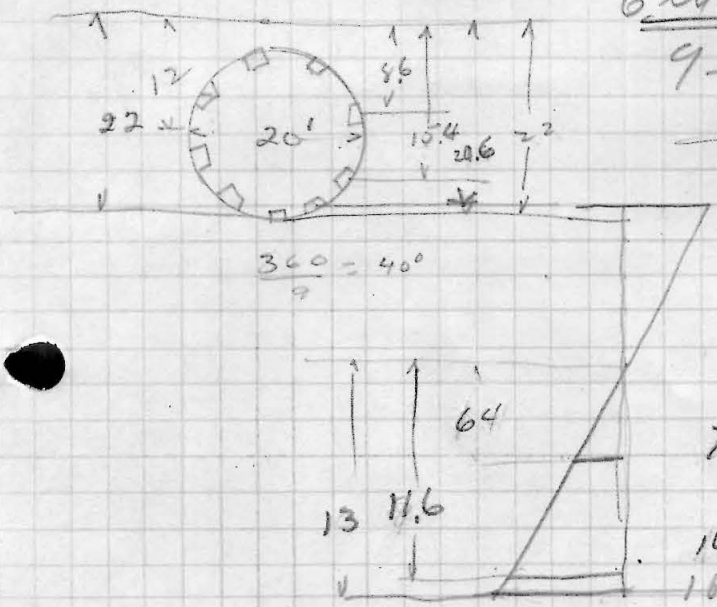
$\frac{62390}{16000} = 3.90$ amt to be added.

6th floor

9 - 7/8" bars.

$p = .004$ > $k = .41$

$.41 \times 22 = 9"$



$7880 \times 2 \times .765 =$	12000
$14300 \times 2 \times .765 =$	21900
$16000 \times 1 \times .765 =$	12100
	46000

amt steel needed at 16000# = 6.16

$6.16 \times 16000 = 98600$

46000

$52600 = 3.29$

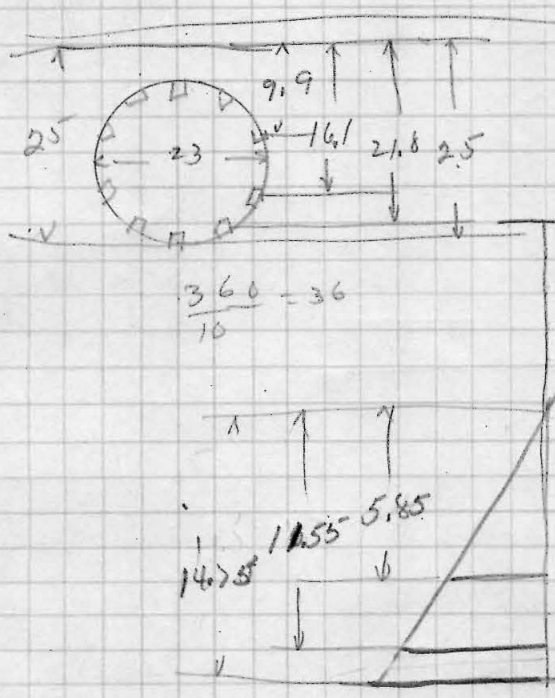
16000

5th floor

16 - 1" bars.

$p = .004$ > $k = .41$

$25 \times .41 = 10.25$



$6350 \times 2 \times 1 =$	12700
$12500 \times 2 \times 1 =$	25000
$16000 \times 1 \times 1 =$	16000
	53700

amt steel need at 16000# = 6.80

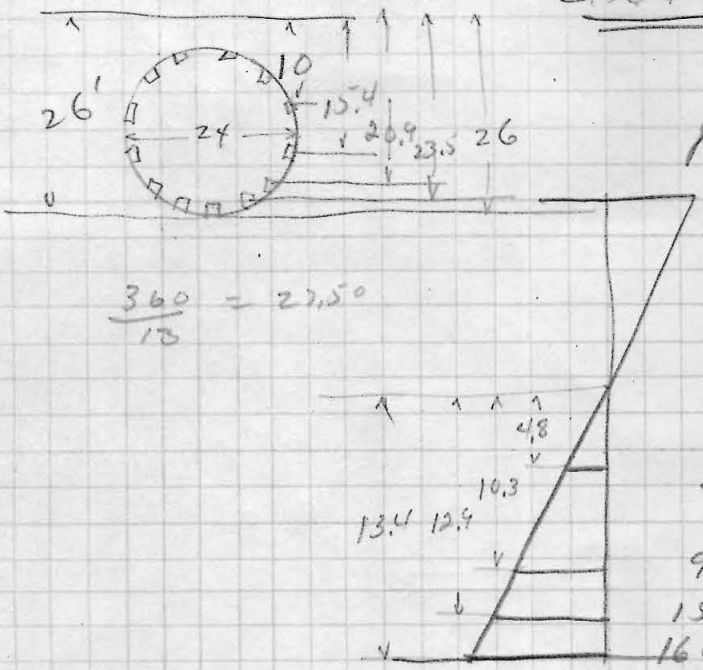
$6.80 \times 16000 = 108700$

53700

$55000 = 3.44$

16000

2nd floor



13-1" bars.

$\rho = 0.04 \Rightarrow k = .41$

$.41 \times 26 = 10.65$

$\frac{360}{13} = 27.5°$

$4300 \times 2 \times 1$	=	8600
$9240 \times 2 \times 1$	=	18480
$15400 \times 2 \times 1$	=	30800
$16000 \times 1 \times 1$	=	16000
		73880

aut steel @ 16000 = 8.06

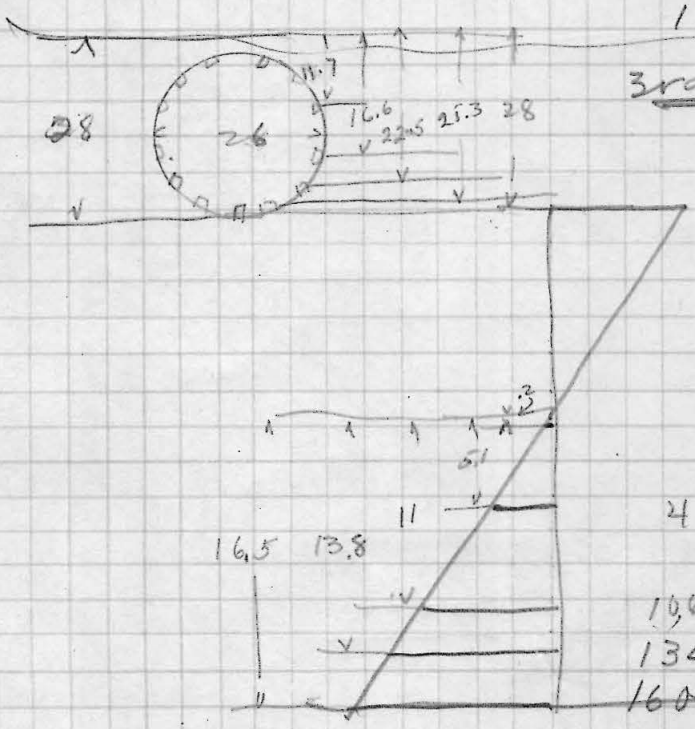
$8.06 \times 16000 = 129$

73880

55120

16000 = 3.45

3rd floor



$\rho = 0.04 \Rightarrow k = .41$

$.41 \times 28 = 11.5$

$194 \times 2 \times 1.125$		400
$4780 \times 2 \times 1.125$		10750
$10650 \times 2 \times 1.125$		24000
$13400 \times 2 \times 1.125$		30200
$16000 \times 1 \times 1.125$	=	18000
		83150

aut steel @ 16000 = 8.52

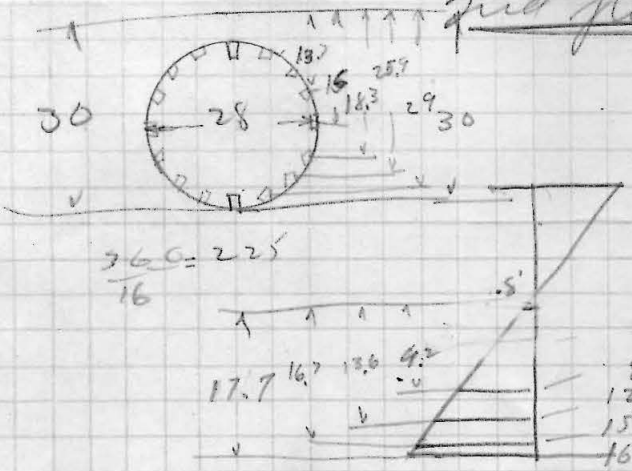
$8.52 \times 16000 = 136000$

83150

52850

16000 = 3.30

2nd floor 16-1⅛"



p = .0097 K = .41

.41 × 30 = 12.30

7201 x 2 x 1.125	=	16000
8300 x 2 x 1.125	=	18700
12300 x 2 x 1.125	=	25400
15100 x 2 x 1.125	=	34000
16000 x 1 x 1.125	=	18000
		97700

ant steel needed @ 16000 = 9.30

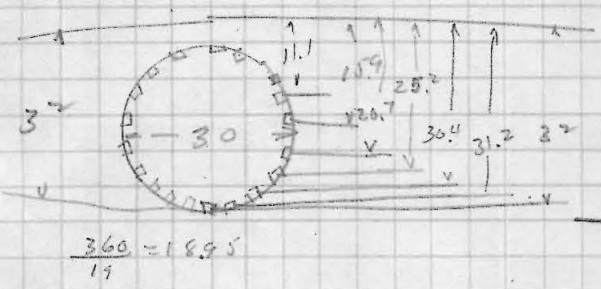
9.30 × 16000 = 149000

97700

51300

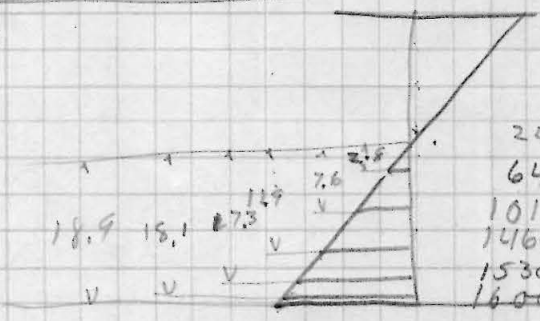
16000 = 3.20"

1st floor 19-1⅛"



p = .0097 K = .0641

.0641 × 32 = 13.1



2400 x 2 x 1.125	=	5400
6450 x 2 x 1.125	=	14500
10100 x 2 x 1.125	=	22700
14600 x 2 x 1.125	=	32800
15300 x 2 x 1.125	=	34400
16000 x 1 x 1.125	=	18000
		127800

steel needed @ 16000 = 10.67

10.67 × 16000 = 170500

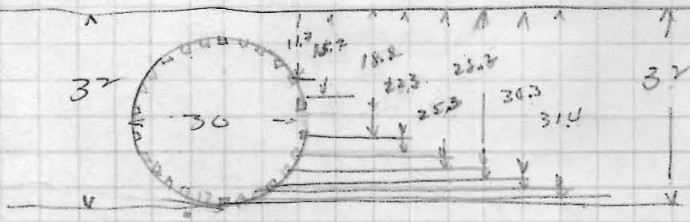
127800

43700

16000 = 2.73

Basement

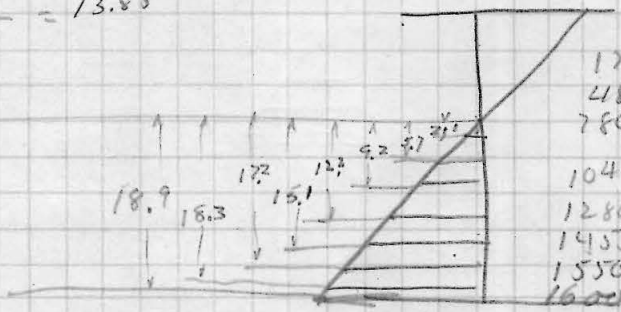
26 - 1" bars



p = 0047 k = .41

0.41 x 32 = 13.1

$\frac{360}{26} = 13.85$



1280 x 2 x 1.125	4000
4800 x 2 x 1.125	10800
7800 x 2 x 1.125	17500
10400 x 2 x 1.125	23400
12800 x 2 x 1.125	28800
14500 x 2 x 1.125	32600
15500 x 2 x 1.125	34900
16000 x 1 x 1.125 =	18000
	170000

steel needed for 16000 = 12.80

12.80 x 16000 = 205000

170000

35000

16000 = 2.19

Steel that must added to stand N+S B.M.

8th floor

Col. 11 has a square core on the 8th floor with 4-5/8" sq rods, one on each side which can be included in the steel needed.

Steel needed = 5.52 "
area of 1 sq 5/8" = .34
amt to be added = 5.13 "

7th floor.

amt steel needed at 16000 #/sq in = 4.69

16000 x 4.69 = 75100
Stress taken by original Col = 27210
47890
16000 = 2.99 sq" to be added

6th floor

steel @ 16000 = 5.18
16000 x 5.18 = 83000
stress taken by original col = 46000
37000
16000 = 2.31 to be added.

5th floor

steel @ 16000 = 5.46
16000 x 5.46 = 90300
stress taken by original Col = 53700
36600
16000 = 2.29 to be added

4th floor

steel @ 16000 = 6.54
16000 x 6.54 = 104500
stress taken by original col = 73880
30620
16000 = 1.92 to be added

3rd floor

steel @ 16000 = 7.80
16000 × 7.80 = 124800
stress taken by orig. col = 83150
41650
16000
= 2.60 to be added

2nd floor

steel @ 16000 = 7.58
16000 × 7.58 = 121500
stress taken by orig. col = 97700
23800
16000
= 1.49 to be added

1st floor

steel @ 16000 = 7.94
16000 × 7.94 = 127000
stress taken by orig col = 127800

none needed.

basement.

steel @ 16000 = 8.80
16000 × 8.80 = 158000
stress taken by orig col = 170000

none needed.

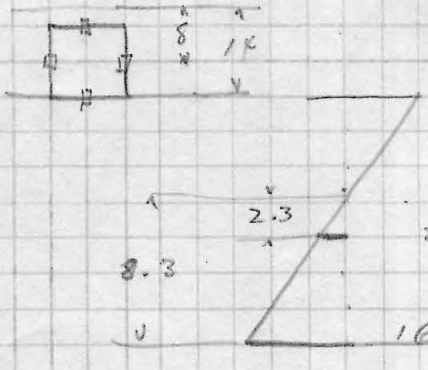
Amount of steel to be added to Col 7 (= SW BM)

8th floor.

4 - 5/8 bars

p = .0097 k = .41

.41 x 14 = 5.7



2 280 x 2 x .39 = 1780

16000 x 1 x .39 = 6240

8020

steel @ 16000 = 294

16000 x 2.94 = 47000

steel taken by orig. col = 8020

38990

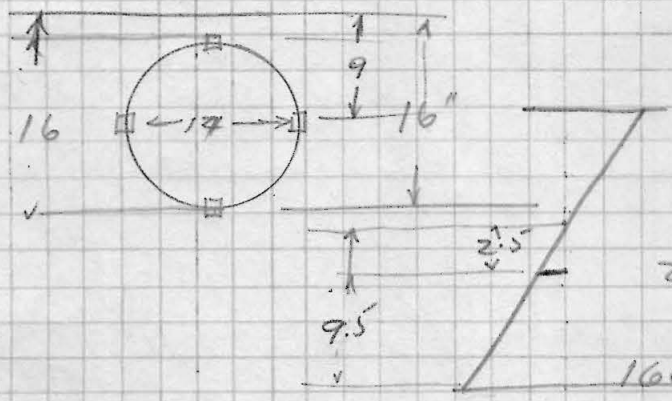
16000 = 2.44 to be added

7th floor

4 - 3/4 bars.

p = .0097 k = .41

.41 x 16 = 6.5



2500 x 2 x .56 = 2800

16000 x 1 x .56 = 9000

11800

steel @ 16000 = 3.08

16000 x 3.08 = 48300

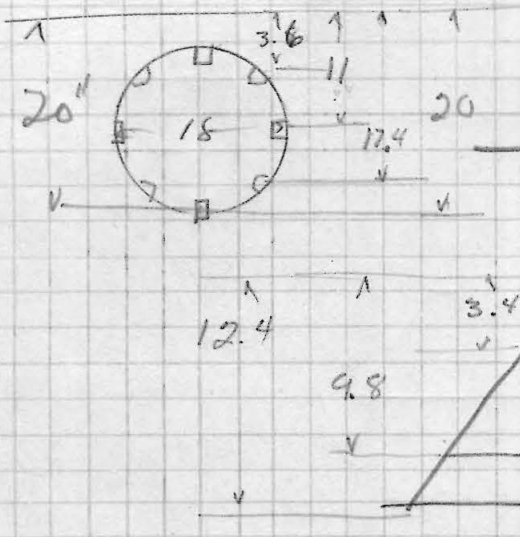
steel taken by orig col = 16800

36500

16000 = 2.28 to be added

6th floor.

8 - $\frac{5}{8}$ bars



$p = .0077$ $k = .38$

$20 \times .38 = 7.6$

4400 x 2 x .39 = 3400

12600 x 2 x .39 = 9800

16000 x 1 x .39 = 6240

19400

steel @ 16000 = 3.39

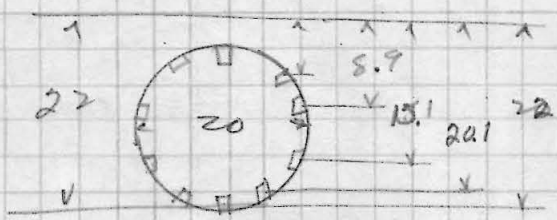
16000 x 3.39 = 54200

shear taken by conc = 19400

34800 = 2.17 to be added

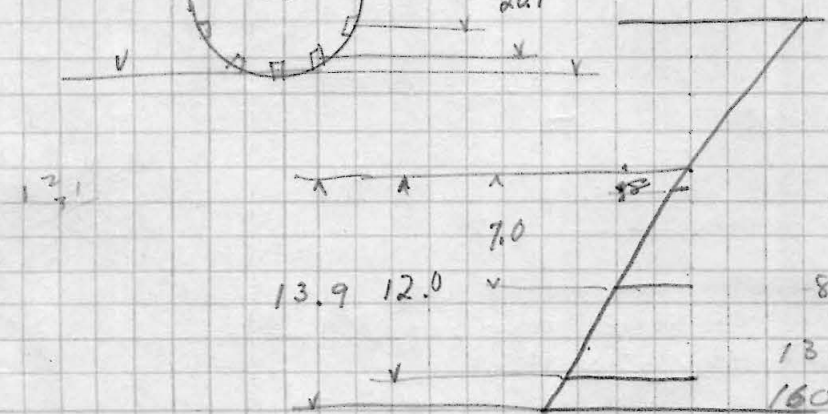
16000

5th floor



$p = .0073$ $k = .37$

$.37 \times 22 = 8.1$



900 x 2 x .56 = 1000

8050 x 2 x .56 = 9000

13800 x 2 x .56 = 15500

16000 x 1 x .56 = 8945

34450

steel @ 16000 = 3.85

16000 x 3.85 = 61600

shear taken by conc = 34450

27150 = 1.70 to be added

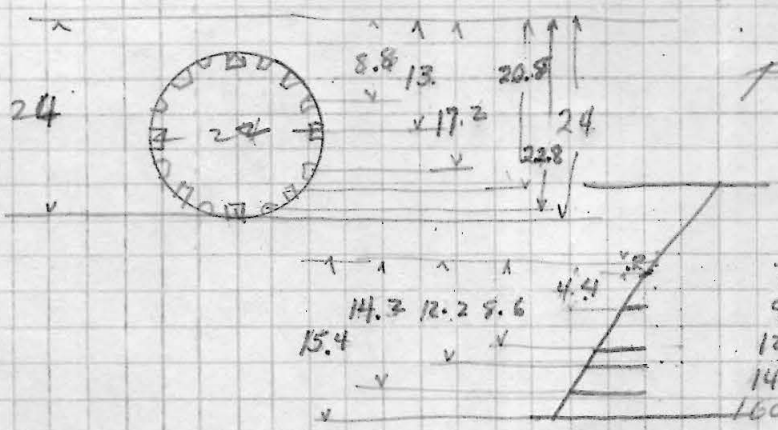
16000

4th floor

12 - $\frac{7}{8}$ bars

$p = .0067$ $k = .36$

$.36 \times 24 = 8.6$



200 x 2 x .76	=	300
4500 x 2 x .76	=	6800
9000 x 2 x .76	=	13700
12700 x 2 x .76	=	19000
14700 x 2 x .76	=	22000
16000 x 1 x .76	=	12000
		73800

steel @ 16000 = 4.18

16000 x 4.18 = 67000

stress taken by conc col = 73800

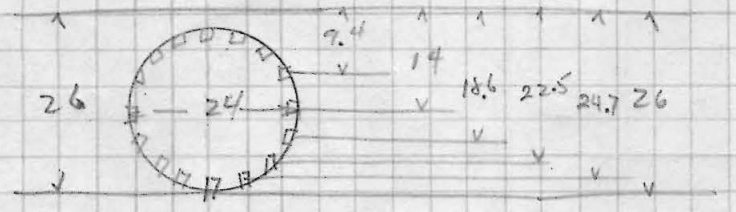
none needed.

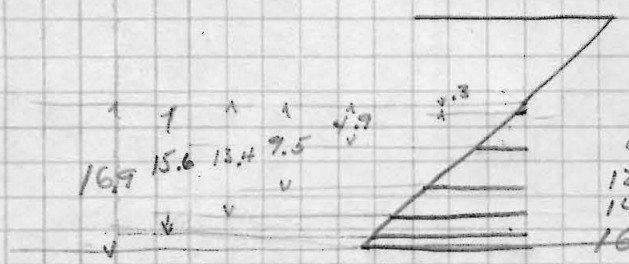
3rd floor

12 - 1" bars

$p = .063$ $k = .35$

$.35 \times 26 = 9.1$





300 x 2 x 1	=	600
4600 x 2 x 1	=	9200
9000 x 2 x 1	=	18000
12700 x 2 x 1	=	25400
14700 x 2 x 1	=	29400
16000 x 1 x 1	=	16000
		98600

steel @ 16000 = 4.58

16000 x 4.58 = 73500

stress taken by conc col = 98600

none needed.

Amount of steel to be added to col 7 (N&S Bldg)

As the Bldg. in col 7 were not calculated for N & S forces, the amount of steel to be added will be taken in the same proportion that steel was needed for E & W forces in col. 18 and 7.

8th floor

$\frac{5.13}{7.09} \times 2.44 = 1.77$ to be added.

7th floor

$\frac{2.99}{5.61} \times 2.28 = 1.21$ to be added.

6th floor

$\frac{2.31}{3.29} \times 2.17 = 1.57$ to be added.

5th floor

$\frac{2.29}{3.44} \times 1.70 = 1.13$ to be added.

none needed from 4th floor to basement

Stationary Columns.

	Col 7		Col 18	
8th	1.56 x 16.75 x 12 =	312.	1.56 x 14 x 12 =	262
7th	2.25 x 13.33 x 12 =	360	4.15 x 13.25 x 12 =	715
6th	3.12 x 13.33 x 12 =	503	6.89 x 13.33 x 12 =	1103
5th	5.62 x 13.33 x 12 =	900	10. x 13.58 x 12 =	1628
4th	9.19 x 13.33 x 12 =	1470	13 x 13.62 x 12 =	2124
3rd	12 x 13.62 x 12 =	1960	16.45 x 14.08 x 12 =	2775
2nd	14 x 1 x 13.62 x 12 =	2290	20.25 x 14.08 x 12 =	3415
1st	17. x 1 x 14.62 x 12 =	3980	24.05 x 15.08 x 12 =	4350
base	21.57 x 15.15 x 12 =	4000	32.90 x 15.75 x 12 =	6220
		15773 cu in		22592 cu in

steel in new columns.

col 7

8th	312 + 2(2.44 + 1.17) 16.75 x 12 =	2002
7th	360 + 2(2.28 + 1.21) 13.33 x 12 =	1470
6th	503 + 2(2.17 + 1.57) 6.67 x 12 =	1030
5th	900 + 2(1.70 + 1.13) 6.67 x 12 =	1353
4th		1470
3rd		1960
2nd		2290
1st		3980
base		4000
		19655 cu in

Col 18

8th	262 + 2(7.09 + 5.13) 14 X 12 =	4375
7th	715 + 2(3.90 + 2.99) 13.25 X 12 =	2905
6th	1103 + 2(3.29 + 2.31) 6.67 X 12 =	1998
5th	1628 + 2(3.44 + 2.29) 6.79 X 12 =	2623
4th	2124 + 2(3.45 + 1.92) 6.81 X 12 =	3060
3rd	2775 + 2(3.30 + 2.60) 7.04 X 12 =	3771
2nd	3415 + 2(3.20 + 1.49) 7.04 X 12 =	4208
1st	4350 + 2(2.73 + 0) 7.54 X 12 =	4843
Base	6220 + 2(2.19 + 0) 15.75 X 1.2 X 1.25 =	7255
		34978 cuin

Comparison of Old + New Design:

Slabs

Roof.

The roof slab is the same for both cases

4" slab steel - 3/8 ϕ @ 7" + 3/8" @ 18"

Vol con. = .33 x 18.87 x 36.48 = 223 cu ft.

Vol steel = .3 x 36.48 x 18.87 x 12 = 2475 cu in

8th floor - 1st floor

Old.

Vol. conc. = .66 x 18.87 x 36.48 + 6.33 x 8 x .375 + 6.33 x 4 x .375

= 443 + 29

= 472 cu ft.

Vol steel (N+S) = (2.73 x G 19 x 12) + (4.30 x H 14 x 12) 2.5 + (2.75 x 5 14 x 12) 2.75 +

(1.87 x T 14 x 12) = 3658 cu. in

E+W = (2.75 x 30.6 x 12) 2 + (2.25 x 30 x 12) 1.5

= 3234 cu. in.

Vol. conc = 472 cu ft.

Vol steel = 6892 cu in.

New.

Vol. conc. = .33 x 18.87 x 36.48 = 223 cu ft.

Vol steel = .32 x 19 x 36.5 x 12 = 3660 cu in

Outside Beams (8th - 1st)

No beams on old design.

New:

inside beams.

Vol concrete = $\frac{[10 \times (18-4) 19]}{12} 3 = 554$ cu ft.

Vol steel = $3.72 \times 19 \times 15 \times 12 \times 3 = 3820$ cu in

Roof.

Roof beams in both cases.

Vol con. = $\frac{[12 \times (20-4) 19]}{144} \times 3 = 76.0$ cu ft.

Vol steel = $2[(78+76) 19 \times 1.5 \times 12] + 1[(78+1) 19 \times 1.5 \times 12]$

= $1054 + 609 = 1663$ cu in.

Col. Beams.

Roof

Old

Vol con. = $\frac{[12 \times (20-4) 19]}{144} 1 = 25.3$ cu ft

Vol steel = $(78+76) 19 \times 1.5 \times 12 = 527$ cu in.

New

Vol con = $\frac{12 \times (20-4) 19}{144} = 25.3$ cu ft

Vol. steel = $2.33 \times (19 \times 1.5 \times 12) = 797$ cu in

8 th.

Old no beams.

New

Vol con = $\frac{11 \times 17 \times 19}{144} = 24.7$ cu ft

Vol steel = $3.83 \times 392 \times 1.75 = 2290$ cu in

7th floor

new

Vol con = $\frac{12 \times 19 \times 19}{144}$ = 30.1 cuft

Vol steel = 342 x 4.5 / x 1.75 = 2700 cuin.

6th floor

new

Vol con = $13 \times 20.5 \times \frac{19}{144}$ = 35.2 cuft

Vol steel = 5.20 x 342 x 1.75 = 3420 cuin

5th floor

new

Vol con = $13 \times 23 \times \frac{19}{144}$ = 39.5 cuft

Vol steel = 5.83 x 342 x 1.75 = 3490 cuin

4th floor

new

Vol con = $14 \times 24.5 \times \frac{19}{144}$ = 45.2 cuft

Vol steel = 6.86 x 342 x 1.75 = 4120 cuin

3rd floor

new

Vol con = $14 \times 26 \times \frac{19}{144}$ = 48.1 cuft

Vol steel = 7.20 x 342 x 1.75 = 4310 cuin

2nd floor

new

Vol con = $14 \times 28 \times \frac{19}{144}$ = 51.8 cuft

Vol steel = 8.00 x 342 x 1.75 = 2736 cuin

new 1st floor

Volcon = $15 \times 29 \times \frac{19}{144}$ = 57.4 cuft

Vol steel = $870 \times 342 \times 1.25$ = 5200 cuin

Girders

Roof

old.

Volcon = $12 \times 26 \times \frac{365}{144} - \frac{12 \times 16 \times 12 \times 3}{1728}$ = 75.1 cuft

Vol steel = $5.06 \times 42 \times 12$ = 2550 cuin

new

Volcon = 75.1 cu ft

Vol steel = 2550×2 = 5100 cuin

8th floor

No old

new

Vol con = $12 \times 26 \times \frac{365}{144} - \frac{16 \times 14 \times 14 \times 3}{1728} - \frac{11 \times 17 \times 14}{1728}$ = 75.2 cuft

Vol steel = $8.49 \times 504 \times 1.25$ = 7500 cuin

new 7th floor

Vol con $12 \times 28 \times \frac{365}{144} - \frac{10 \times 14 \times 15 \times 3}{1728} - \frac{11 \times 17 \times 15}{1728}$ = 82.9 cuft

Vol steel = $9.18 \times 504 \times 1.25$ = 8100 cuin

new 6th floor

Vol con = $15 \times 30 \times 1.253 - 15 \times 2.43 - 15 \times 1.08$ = 93.2 cuft

Vol steel = $9.85 \times 504 \times 1.25$ = 8800 cuin

new 5th floor

Vol con = $14 \times 31 \times 1.253 - 16 \times 2.43 - 16 \times 1.08$ = 104.3 cuft

Vol steel = $10.64 \times 5.04 \times 1.25$ = 9400 cuin

new. 4th floor

Vol con = 15 x 34 x.253 - 16x.243 - 16x108 = 116.8 cuft.

Vol steel = 1095 x504 x1.05 = 9660 cuin

new 3rd floor

Vol con = 15x34x.253 - 16x.243 - 16x108 = 123.3 cuft.

Vol steel = 1150 x504 x1.05 = 10160 cuin

2nd floor

new

Vol con = 16x35x.253 - 17x.243 - 17x108 = 132.0 cuft

Vol steel = 1228 x504 x1.05 = 10800 cuin

1st floor

new

Vol con = 16x37.5x.253 - 17x.243 - 17x108 = 196 cuft

Vol steel = 1290x504 x1.05 = 11380 cuin

Vol of old & New Concrete in Interior Col. 18

8th floor = $\frac{16 \times 16}{144} \times 11.75' =$ 29.4

7th = $\frac{20 \times 20}{144} \times 11.75 =$ 32.7

6th = $\frac{24 \times 24 \times 11.75'}{144} =$ 46.9

5th = $\frac{27 \times 27}{144} \times 11.75 =$ 58.5

4th = $\frac{28 \times 28 \times 11.75}{144} =$ 63.9

3rd = $\frac{30 \times 30 \times 11.75}{144} =$ 73.4

2nd = $\frac{32 \times 32 \times 11.75}{144} =$ 83.5

1st = $\frac{34 \times 34 \times 11.75'}{144} =$ 94.0

basement = $\frac{34 \times 34 \times 11.75'}{144} =$ 94.0

Vol of Old Concrete col 7

8th = $16 \times 16 \times \frac{11.25}{144}$ = 20.9

7th = $18 \times 18 \times \frac{11.25}{144}$ = 26.4

6th = $22 \times 22 \times \frac{11.25}{144}$ = 39.4

5th = $24 \times 24 \times \frac{11.25}{144}$ = 46.8

4th = $26 \times 26 \times \frac{11.25}{144}$ = 55.0

3rd = $28 \times 28 \times \frac{11.25}{144}$ = 63.8

2nd = $30 \times 30 \times \frac{11.25}{144}$ = 73.4

1st = $31 \times 31 \times \frac{11.25}{144}$ = 78.2

basement = $33 \times 33 \times \frac{11.25}{144}$ = 89.0

Total Concrete

		Old		New	
Floors	roof	223	= 223	roof-1st = 223 × 9 =	2007
	8th-1st	472 × 8	= 3776		
			3999 cuft		2007 cu
Interior Beams	Roof		76. "	roof	76.
	none			8th-1st = 8 × 55.4	443.2
					519
Columns	Roof		25.3		25.3
	8th	none			24.7
	7th	"			30.1
	6th	"			35.2
Beams.	5th	"			39.5
	4th	"			45.2
	3rd	"			48.1
	2nd	"			51.6.
	1st	"			57.4
			257.3		357.3
Girders	Roof		75.1		75.1
	8th	none			75.7
	7th	"			82.9
	6th	"			93.2
	5th	"			104.3
	4th	"			116.8
	3rd	"			123.8
	2nd	"			137.0
	1st	"			146.0
			75.1		934.3

		Old	New
Col. 18	8 th	20.9	20.9
	7 th	32.7	32.7
	6 th	46.9	46.9
	5 th	58.5	58.5
	4 th	63.9	63.9
	3rd	73.4	73.4
	2nd	83.5	83.5
	1st	94.0	94.0
	bas	94.0	94.0
		567.8	567.8

Col 7	8 th	20.9	20.9
	7 th	26.4	26.4
	6 th	34.6	34.6
	5 th	46.8	46.8
	4 th	55.0	55.0
	3rd	63.8	63.8
	2nd	73.4	73.4
	1st	78.2	84.4
	bas	89.0	101.5
		502.9	535.6

	Old	New
Floor	39.490.0	20070
Intern Beams	760	5197.2
Cal Beams	25.3	357.3
Girders	755.1	954.3
Col 18	567.8	567.8
Col 7	502.9	535.6
	52461 cuft	49412 cuft

Total Steel

		Old		New
Floors	Roof	2475 =	2475	2475
	8th–1st	8× 6892 =	55136	8×3860 = 29280
			57611	31755
Interior Beams	roof		1663	1663
	8th–1st	none		8×3820 = 30560
			1663	32223
Col Beams	roof		527	797
	8th	none		2290
	7th	"		2700
	6th	"		3420
	5th	"		3490
	4th	"		4120
	3rd	"		4310
	2nd	"		4780
	1st	"		5200
			527	28107
Girders	roof		2550	5110
	8th	none		7500
	7th	"		8100
	6th	"		8800
	5th	"		9400
	4th	"		9660
	3rd	"		10160
	2nd	"		10800
	1st	"		11380
			2550	80910

		Old	New
Col 18	8th	262	4375
	7th	715	2905
	6th	1103	1998
	5th	1628	2623
	4th	2124	3000
	3rd	2775	3771
	2nd	3415	4208
	1st	4350	4843
	base	6220	2255
		22592	34978

Col 7	8th	312	2002
	7th	360	1470
	6th	503	1030
	5th	400	1353
	4th	1470	1470
	3rd	1960	1960
	2nd	2240	2240
	1st	3980	3980
	base	4000	4000
		15773	19655

Floors	57611	31755
Interior Beams	1663	32223
Col Beams	527	28107
guides	2550	80910
Col 18	22592	34978
Col 7	15773	19655
	100716	227628

Comparison of Designs

	Old	New
Volume Concrete	52461 cu.ft	49412 Cu.ft
" Steel	100,716 cu. in	227,628 cu. in.

New Design has 94.3 % of Concrete of Old
" " " 226.0 % of Steel of Old.