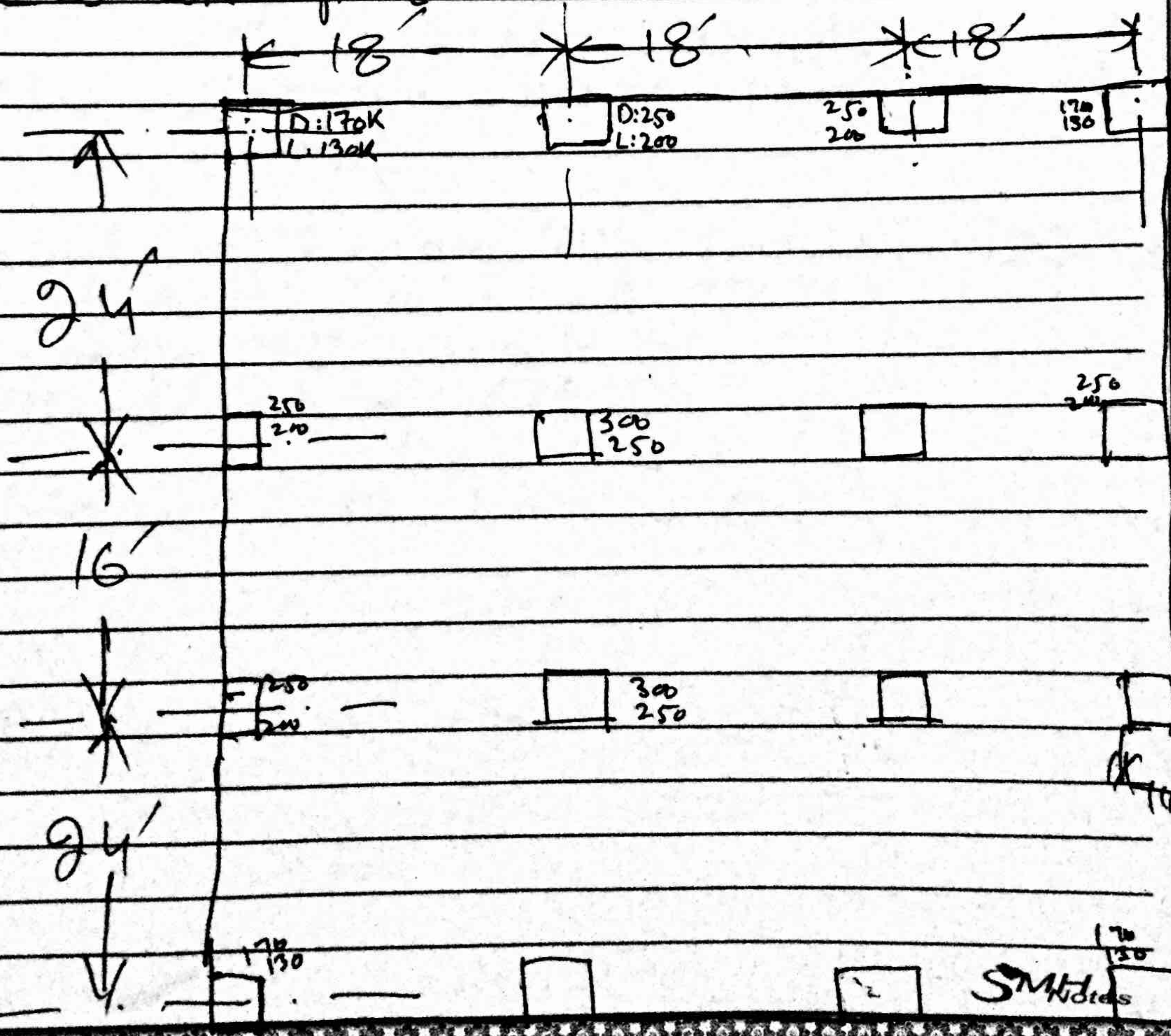
Date

I.D : 15404 ①

Assignment (Sessional):

Design Foundation For given Column layout.
Each Column 24" x 24".
All Four exterior Column sides are fixed property lines. Allowable bearing capacity of soil is 2000 lb/ft². Depth of bottom of foundation is about 6 ft below Ground level. Take $f_c' = 4000$ Psi and $f_y = 60,000$ Psi. Column D.Ls and L.Ls are given.



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Total Service D.L = 4×170 + 8×250 + 4×300

= 3880 K

Total Service L.L = 4×130 + 8×200 + 4×250

= 3120 K

Total Service load on Footing

= 3880 + 3120 = 7000 K.

Now, allowable bearing pressure of Soil is 2000 lb/ft^2.

Unit weight of over fill considering 100 lb/ft^3 and R.C.C unit wt 150 lb/ft^3, average fill weight is 125 lb/ft^3.

At a depth of 6 ft,

Effective Earth bearing pressure will be

q_e = 2000 − 125×6 = 1250 lb/ft^2.

Now, Footing Area Required is

$= \frac{7000000}{1250} = 5600\ ft^2$

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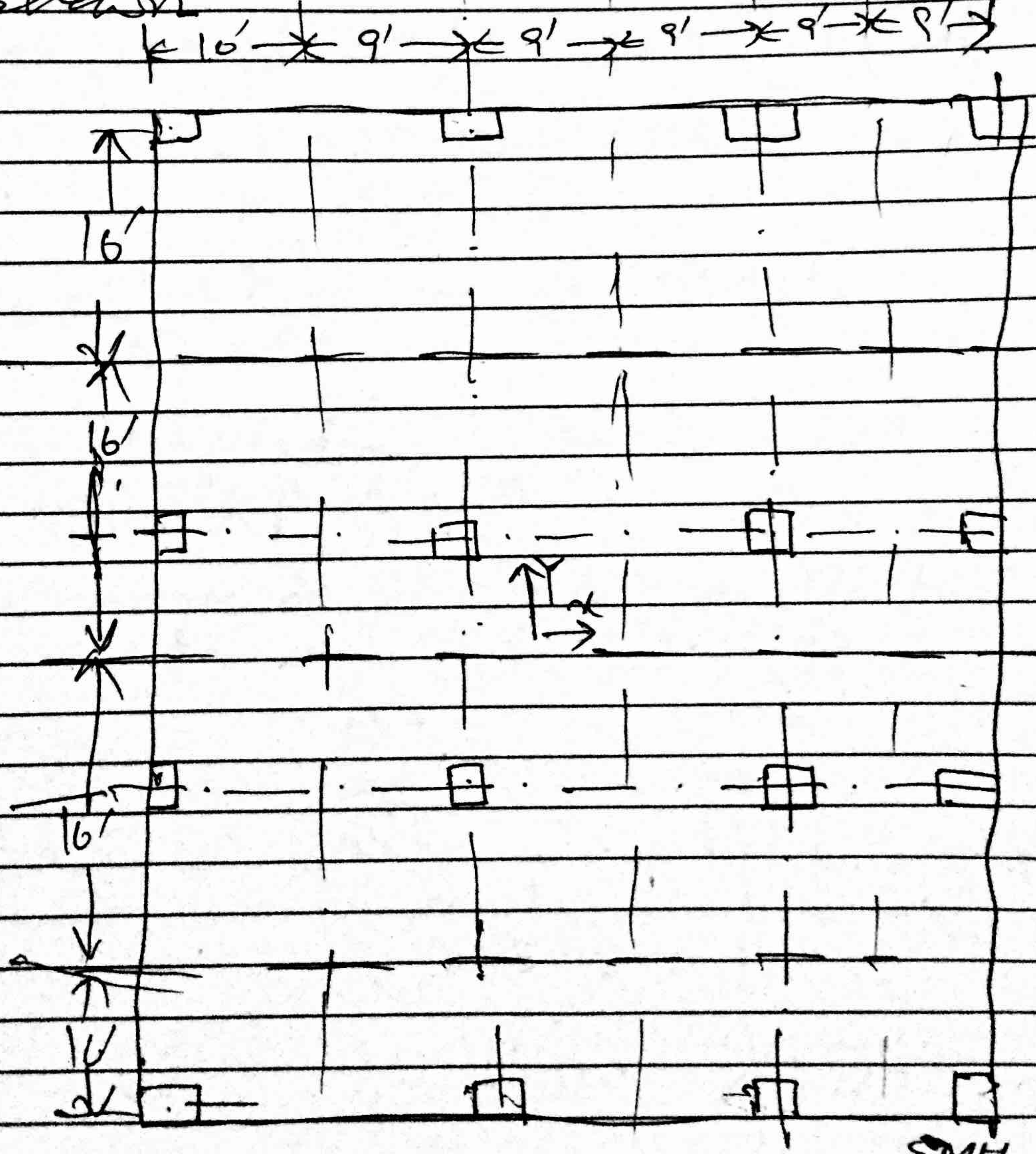
Area available = 56 * 66 = 3696 ft² (Re

So, opting for Mat/Raft foundation

Dividing given layout in Longitudinal and Transverse Beams, as shown



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Now, Calculating ultimate bearing of Soil on Raft foundation will be

$$q_u = \frac{\text{Total Factored Load}}{\text{Area of Footing}}$$

Total factored load = 1.2 * DL + 1.6 LL

= 1.2 * 3880 + 1.6 * 3120

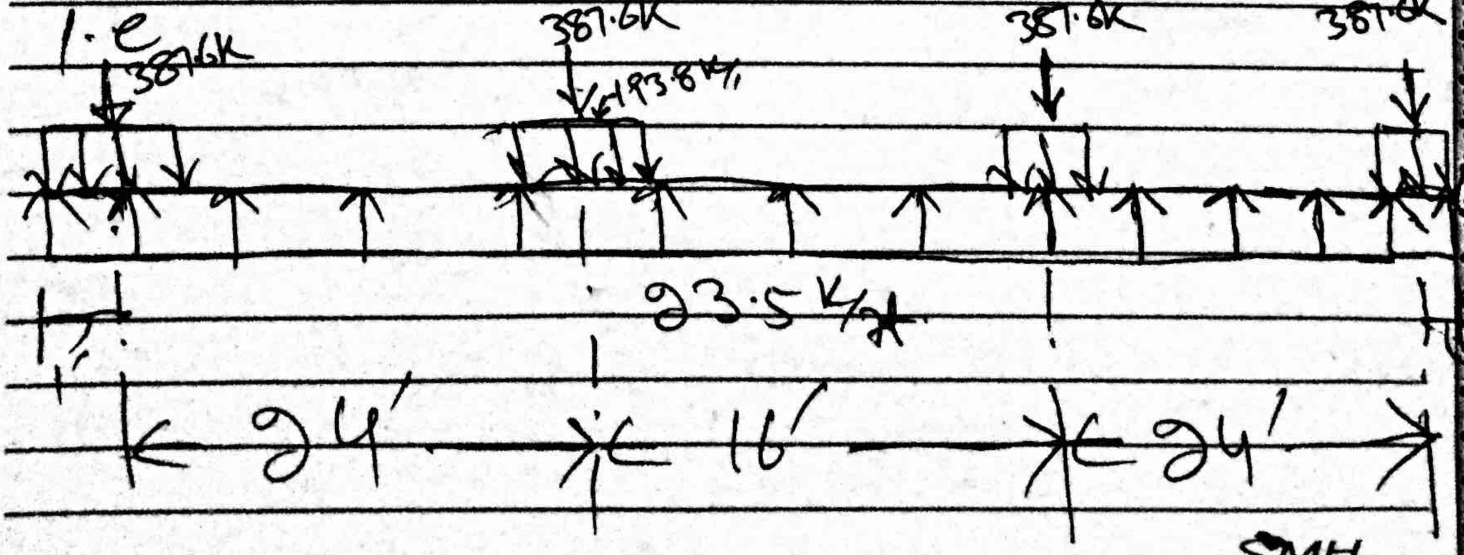
= 9648 K.

$$q_u = \frac{9648}{56 * 66} = 2.61 \, K/ft^2.$$

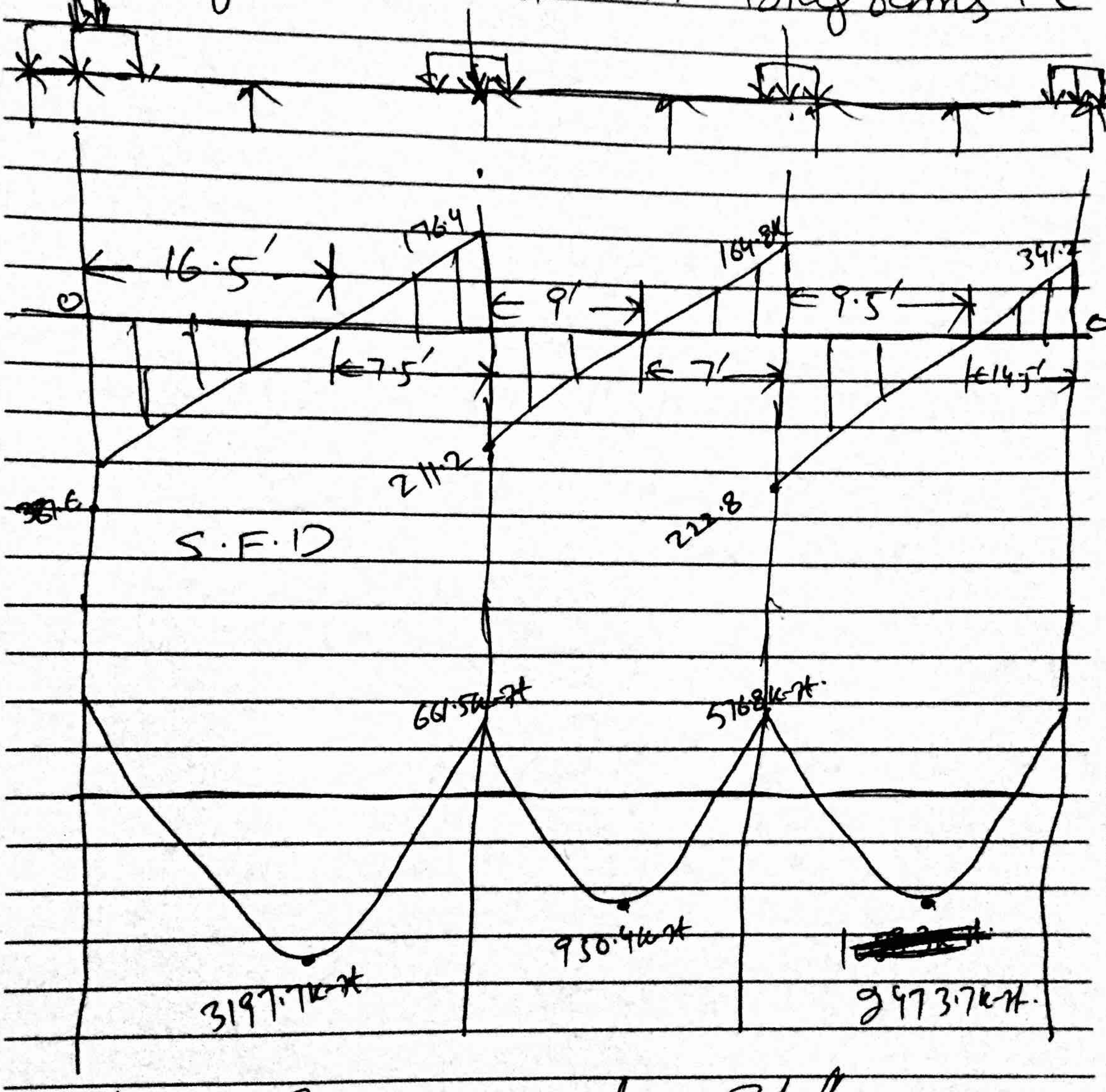
Now, Longitudinal Strength design Exterior Strip of width 9ft.

Earth pressure on Strip per linear foot will be = 2.61 * 9 = 23.5 K/ft.

i.e



Drawing S.F and B.M Diagrams i.e



Now, Considering $d = 34''$,

$\rho_{min} = 0.018 \Rightarrow A_{s\,min} = 0.018 \times 9 \times 12 \times 34$

$= 6.61\ in^2$

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Now

$$a = \frac{6.61 * 60}{0.85 * 4 * 9 * 12} = 1.08''$$

$$\Rightarrow \phi M_n = \phi A_s f_y (d - a/2)$$

$$= 0.9 * 6.61 * 60 (34 - a/2) / 12$$

$$\phi M_n = 995.34 \text{ k-ft}$$

Now Calculating A_s from M_u i.e take $a = 0.1d$ i.e $3.4''$

$$A_s = \frac{3197.7 * 12}{0.9 * 60 * (34 - 3.4/2)}$$

$$A_s = 22 \text{ in}^2$$

Recalculate 'a' i.e $a = \frac{A_s f_y}{0.85 f'_c b}$

$$\Rightarrow a = \frac{22 * 60}{0.85 * 4 * 9 * 12} = 3.6'' \Rightarrow$$

$A_s = 22.1 \text{ in}^2$ hence OK.

Now $A_{smax} = \rho_{max} * b * d = 0.0206 * 9 * 12 * 34$
$= 75 \text{ in}^2$

Selecting #10 bars with $A_b = 1.27 \text{ in}^2$

No of bars $= 18 \Rightarrow A_s = 22.86 \text{ in}^2$

$\Rightarrow$ #10 @ 6" c/c (Top reinforcement for -ve moment).

SMNotes

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(7)

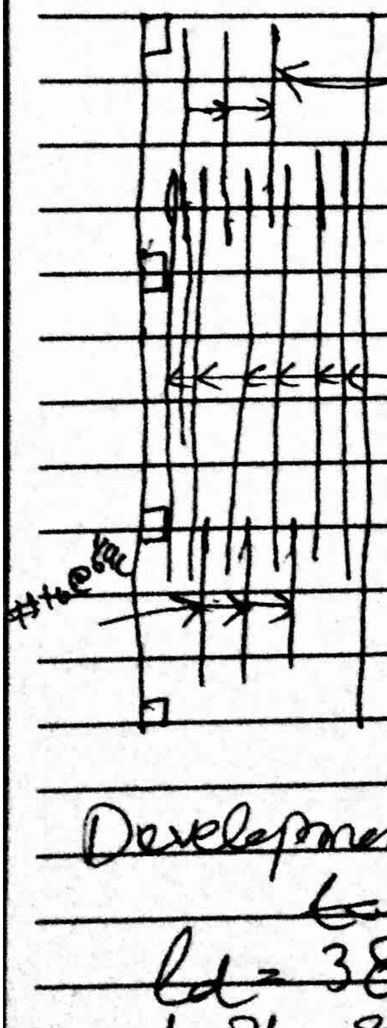
Now Bottom Reinforcement for +ve moments @ Column intersections (Supports)

$A_{s\,min} = 6.61\ in^2$

$\Rightarrow \phi M_n = 985.3$ k-ft.

Using #6 i.e $A_b = 0.44\ in^2$
15 Nos of bars

$\Rightarrow$ #6 @ 7" c/c.



Development length for top bars from table A-10

$l_d = 49 * 10/8 = 5.1$ ft.

(Available length upto Support = 7.5 ft hence OK).

Development length bottom bars from table A-10

$l_d = 38 * 6/8 = 2.4$ ft. (So upto $l_n/3$ on both sides of column).

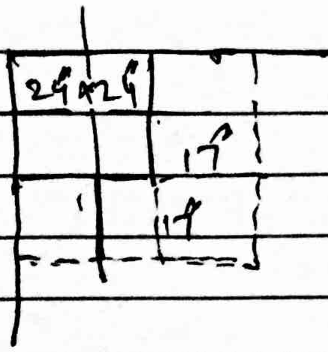
SMNotes

Now Checking depth of footing for most critical Column i.e Outer Exterior Edge Column having 2-fixed property lines i.e (8)

Perimeter for Punching Shear is

$= (24+17)*2 / 12 = 6.83ft$

or $82''$.



Now $\phi V_{cp} = \phi * 4 * \sqrt{f'_c}\, b_o * d$

$= 0.75 * 4 * \sqrt{4000} * 82 * 34 / 1000$

$= 528.9K > V_u$

'd' is ok for Punching Shear.

Check for beam Shear at a distance 'd' from the face of

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Column.

Beam Shear Capacity of Strap is

$\Phi V_{cb} = \Phi 2 \sqrt{f'c} B * d$

$= 0.75 * 2 * \sqrt{4000} * 9 * 12 * 34 / 1000$

$\Phi V_{cb} = 348.4 K$.

Now ultimate Shear at distance 'd' from face of exterior column from shear force diagram is

$V_u = 387.6 - 23.5 \frac{(12+34)}{12}$

$= 297.5 Ki$

which show that

$\Phi V_{cb} > V_u$, hence depth is also ok for Beam Shear at Critical Column of the building.

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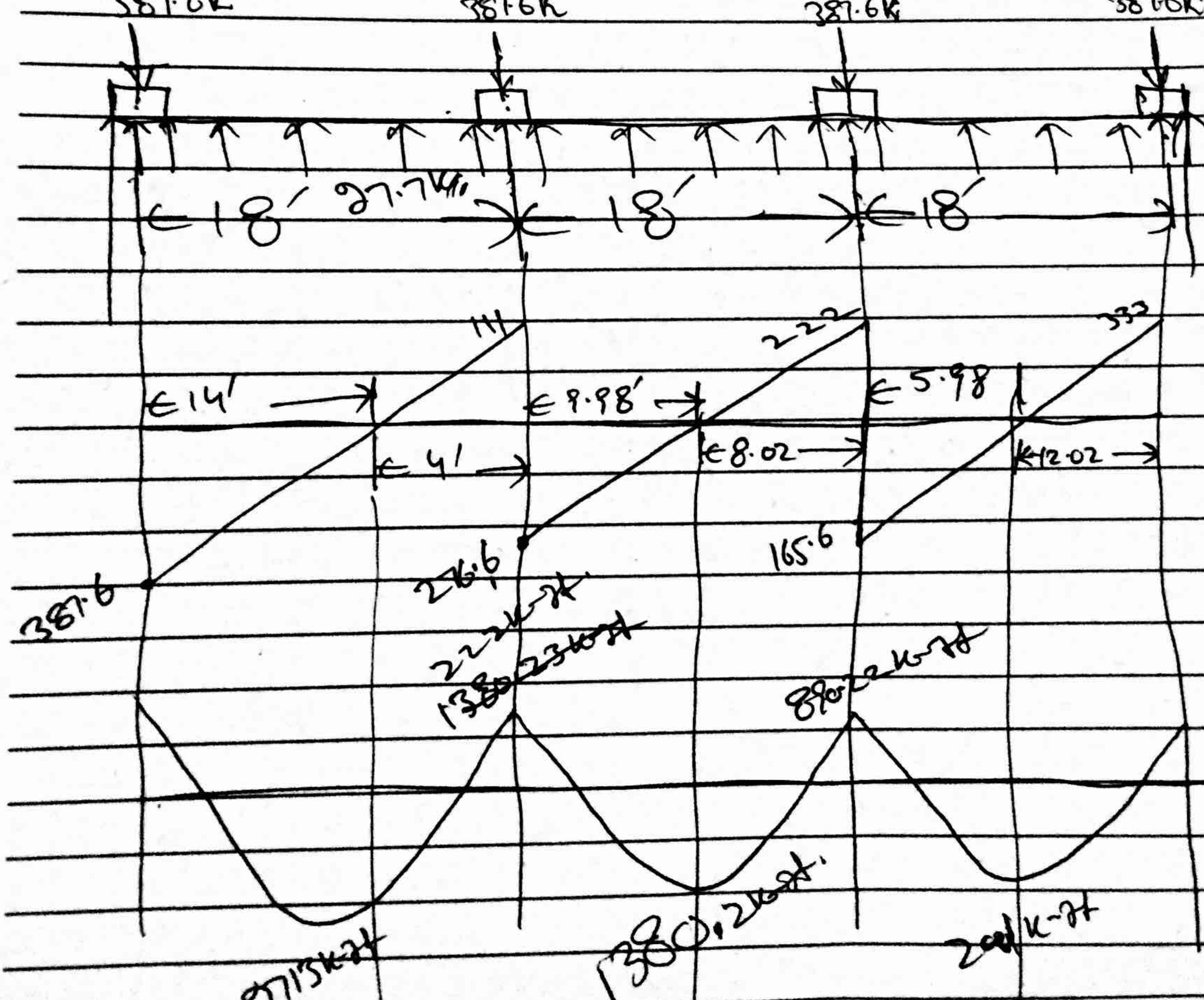
Now checking Strength design of ~~exterior~~ transverse beam i.e



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Exterior Spans Strength design for -ve moments i.e

$M_u = 2713$ k-ft.

$A_{s\,min} = 0.0018 * 16 * 12 * 34$

$= 11.75\ in^2$

~~Try~~ $\phi M_n = 0.9 * 11.75 * 60 * (34 - a/2) / 12$

$a = \dfrac{11.75 * 60}{0.85 * 4 * 16 * 12} = 1.08''$

⇒ $\phi M_n = 1769$ k-ft.

⇒ #6 @ 7" c/c (Mid Span Top & Bottom)

Exterior Spans i.e

$a = 0.1 * d = 3.4''$

$A_s = \dfrac{2713 * 12}{0.9 * 60 * (34 - 3.4/2)} = 18.66\ in^2$

Re-calculate 'a' i.e $A_s = 18.66\ in^2$

⇒ $a = 1.7''$ ⇒ $A_s = 18.2\ in^2$

Taking #10, bars, $A_b = 1.27\ in^2$

⇒ 15, #10 bars, with $A_s = 19.05\ in^2$

#10 @ 12.5" c/c. (Top Reinforcement).

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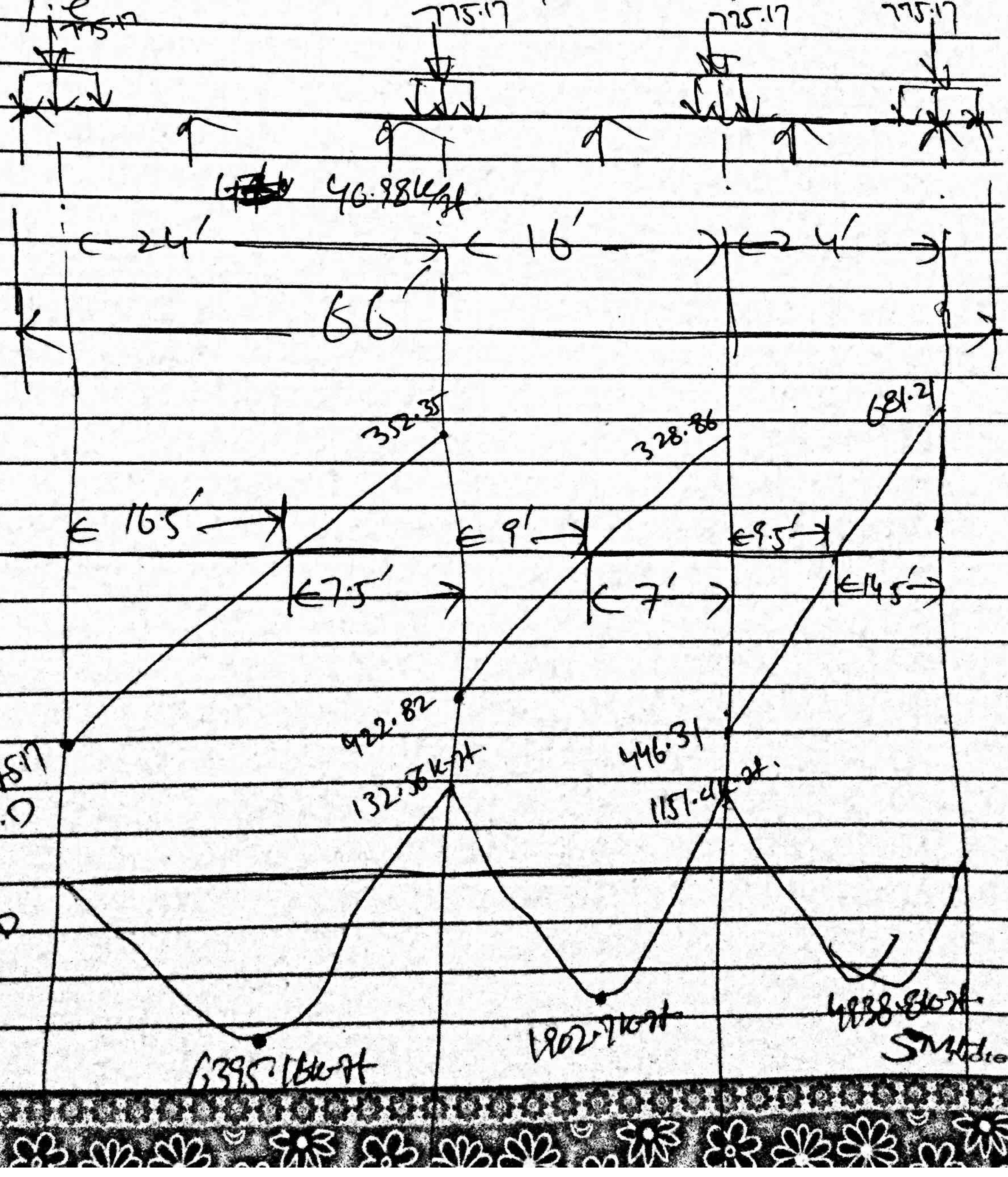
Analysis of internal longitudinal Strap 1-1

Ultimate bearing pressure on the strap (18 ft width) per linear foot will be

$q = 2.61 \times 18 = 46.98\ k/ft$

i.e



Check 'd' for beam Shear Capacity at distance 'd' from face of column i.e.

$V_u = 775.17 - 46.98 \frac{(12 + 34)}{12}$

$= 595.62$ Kips.

Beam Shear Capacity

$\phi V_{cb} = \phi 2 \sqrt{f'_c} \cdot B * d$

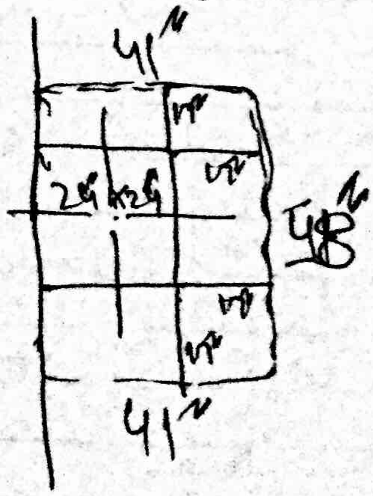
$= 0.75 * 2 * \sqrt{4000} * 18 * 12 * 34 / 1000$

$= 696.7$ Kips $> V_u$ OK.

Check Punching Shear Capacity i.e

$\phi V_{cp} = \phi 4 \sqrt{f'_c}\, b_o \cdot d$

$b_o = 41 + 41 + 58 = 140''$.



$\Rightarrow \phi V_{cp} = 0.75 * 4 * \sqrt{4000} * 140 * 34 / 1000$

$= 903.15$ Kips $> V_u$

'd' OK for Shear.

Exterior Span Top Reinforcement (-ve Moment)

flexure design i.e

$M_u = 6395.16$ k-ft.

⇒ Let $a = 0.1*d = 3.4''$

⇒ $A_s = \frac{6395.16*12}{0.9*60*(34-3.4/2)} = 44 \text{ in}^2$.

Recalculate 'a' i.e

$a = \frac{44*60}{0.85*4*18*12} = 0.~~36~~'' \; 3.6 \text{ in}$

~~⇒ As = 42 in² ⇒ a =~~

⇒ $A_s = 44 \text{ in}^2$ hence OK.

Using #10 @ $A_b = 1.27 \text{ in}^2$ ⇒ 35 No of bars

⇒ #10 @ 6" c/c (Top Longitudinal in exterior Span).

Now

$A_{s,min} = 0.0018*18*12*34 = 13.2 \text{ in}^2$.

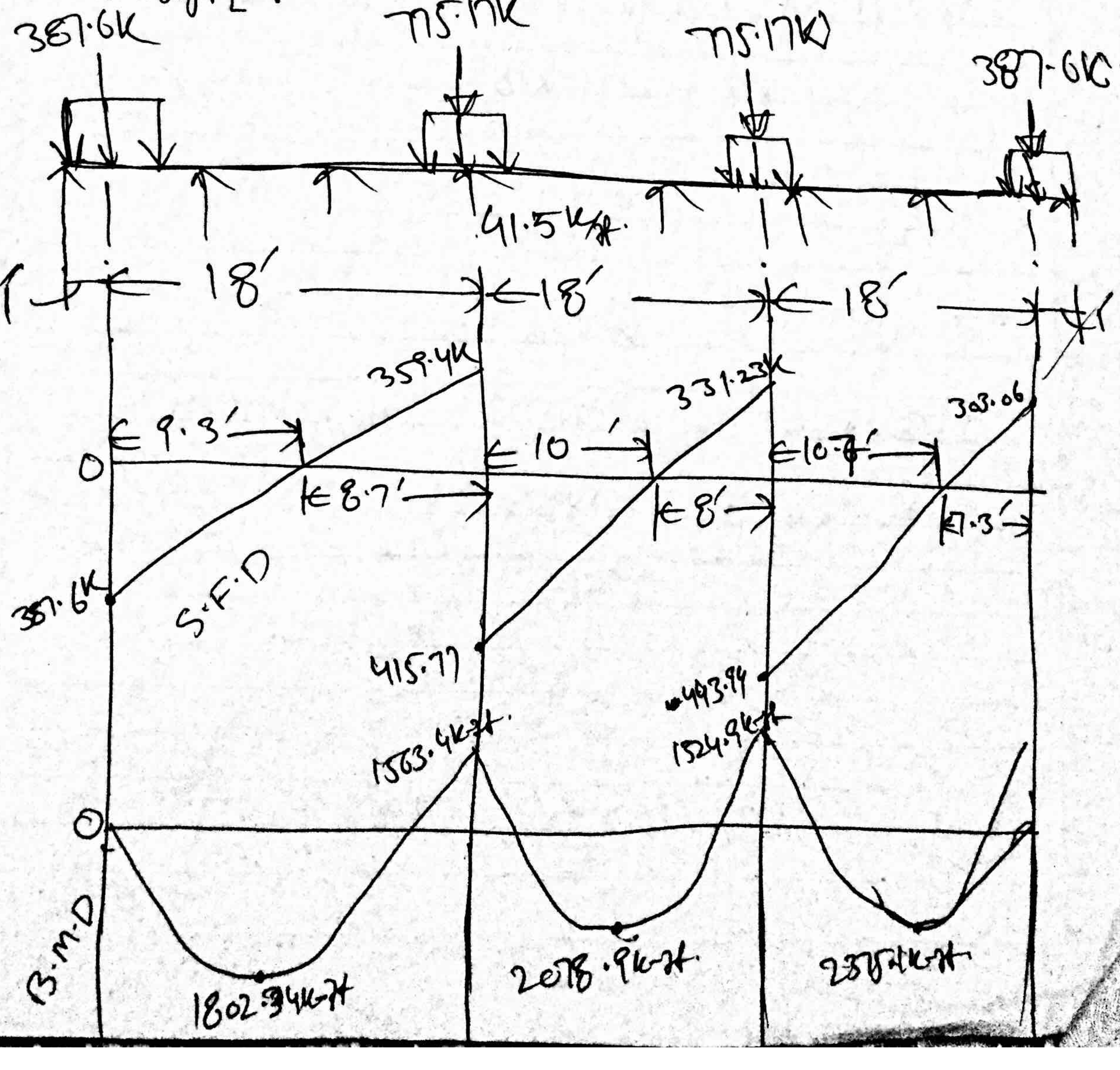
⇒ $a = 1.08''$ ⇒ $\phi M_n = 0.9*13.2*60*(34-1.08/2)/12$

$\phi M_n = 1987.5$ k-ft $> M_{us}$ (Calculated)

Trying #6, @ $0.44 in^2$ ⇒

⇒ #6@7" c/c (Top and Bottom Reinforcement @ interior supports and mid span).

Internal Transverse Beam analysis and Design:



Design of Top Reinforcement (-ve Moment) in transverse direction i.e.

$M_u = 2375.1$ k-ft.

~~Assume $a = 3.4''$ ⇒ $A_s = 2375.1$*~~

First of All, $A_{s\,min} = 0.0018 * 16 * 12 * 34$

$= 11.75\ in^2$

⇒ $a = \frac{11.75 * 60}{0.85 * 4 * 16 * 12} = 4.3''$

⇒ $\phi M_n = 0.9 * 11.75 * 60 (34 - 4.3/2) / 12 = 1683.5$ k-ft.

Use #6 @ 7" c/c (Bottom Reinforcement). (+ve Moment).

Top Reinforcement (-ve Moment)

$M_u = 2375.1$ k-ft.

Assume $a = 3.4''$ ⇒ $A_s = \frac{2375.1 \times 12}{0.9 * 60 * (34 - 3.4/2)} = 16.4\ in^2$

Recalculate 'a' i.e

⇒ $a = 1.5$ in ⇒ $A_s = 15.9\ in^2$

⇒ $a = 1.45$ in ⇒ $A_s = 15.9\ in^2$ hence OK.

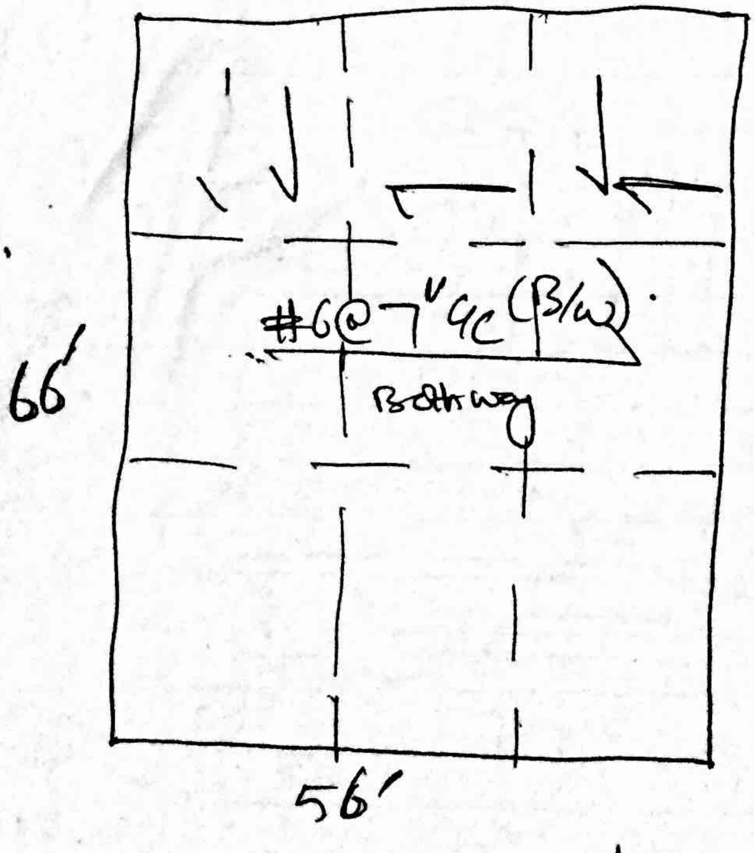
Try #10 @ $1.27\ in^2$ ⇒ 13 No bars

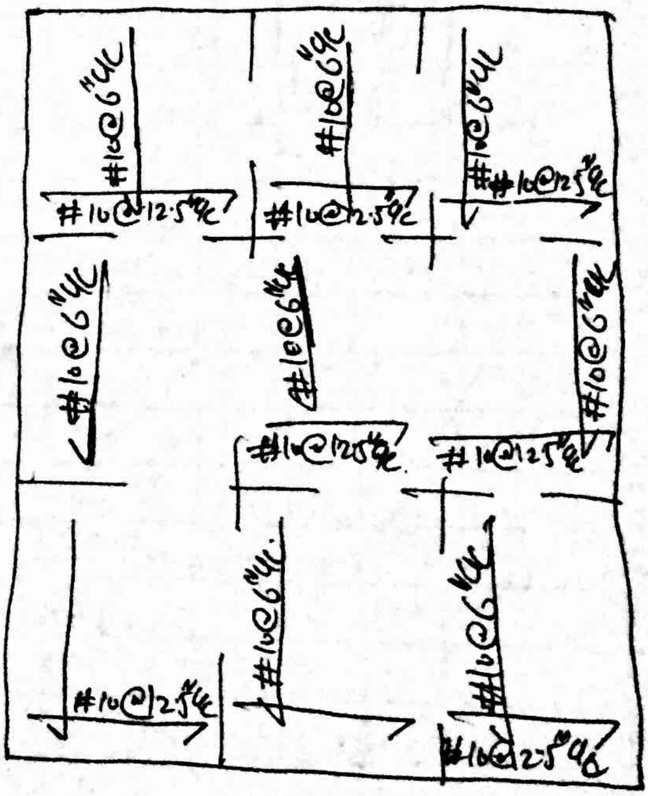
⇒ #10 @ 14" c/c (Top Reinforcement)

Taking 4.5" bottom cover and 3.5" top cover, total height/depth of footing will be

$$34 + 4.5 + 3.5 = 42" = 3'-6"$$

And Reinforcement will be (on the basis of Calculation and Ease of Physical work) as shown below;





Bottom Reinforcement

Top Reinforcement.