Letter of Transmittal

Jeremy Swartz

November 13, 2017

Dr. Aly Said
The Pennsylvania State University
209 Engineering Unit A

Dear Dr. Said,

The following report, Structural Notebook Submission C, is the third of a three part evaluation of One City Center in Washington D.C. The report has been appended to my previous Submissions and consists of an analysis of the Lateral system in the building. The analysis is based off of both hand and model results. The model has been verified by comparing the center of mas and rigidity as well as the story forces that were calculated by hand. Overall and story displacements have been determined and compared to the allowable code value. Furthermore flexural and shear capacity were determined for each shear wall and compared to their calculated flexural and shear forces.

Thank you for your evaluation of this report. Please let me know if you have any questions regarding the material. I look forward to improving this report based on your feedback.

Sincerely,

Jeremy Swartz


COURTESY OF CLARKE CONSTRUCTION

One and Two City Center Washington D.C.

Notebook Submission C

Lateral System Analysis Study

Report 4
By: Jeremy Swartz
Option: Structural
Advisor: Dr. Aly Said

Executive Summary

One and Two City Center are commercial buildings that are a part of a multi-use development located in Washington D.C. Being approximately 312,000 square feet the building is part of a four lot project. Planning and design began as early as April 2007 but due to the recession, construction was delayed until April of 2011 and was finished later in 2014.

The twin office buildings now stand 12 stories tall with a floor to floor height of 12'. The shell of the structures is a glazed aluminum curtain wall with movable louvers. Like many roofs in D.C., there is a rooftop mezzanine on both One and Two City Center with several areas used as a green roof. Connecting the two buildings on every floor are glass coated walkways which span the alleyway separating the One and Two City Center. The building has achieved LEED Gold certification and the development has been one of the first to achieve LEED-ND (Neighborhood Development) certification.

The structural floor systems are two way post tensioned concrete slabs supported by typical 24" x 24" concrete columns. These columns run down through the building into the below grade parking and come to rest on shallow concrete foundations. Lateral loads are resisted by a series of shear walls which surround the elevators and stairwells. The glazed aluminum curtain wall is fastened to the structure at the concrete slab and supported by HSS sections. The penthouse roof and floor are supported by a series of W10's.

The additional lots feature commercial, residential, parking and public areas. To the north of One and Two City Center (Lot46) is an outside plaza with a captivating reflecting pool. To the east of the site is a four structure commercial and residential development (Lot 47). The two main lots are connected by an alleyway lined with retail stores. At the center of Lot 47 is a small courtyard offering relief from the city. Underneath Lot 46 and 47 is a four story parking garage for public access and the use of delivery trucks.

Table of Contents

Site Plan

One and Two City Center are located in the downtown area of Washington D.C. The site is a part of a larger development shown in figure one below. The entire development sits on four stories of below grade parking. The two office buildings are connected by a series of bridges which span the alleyway separating them.

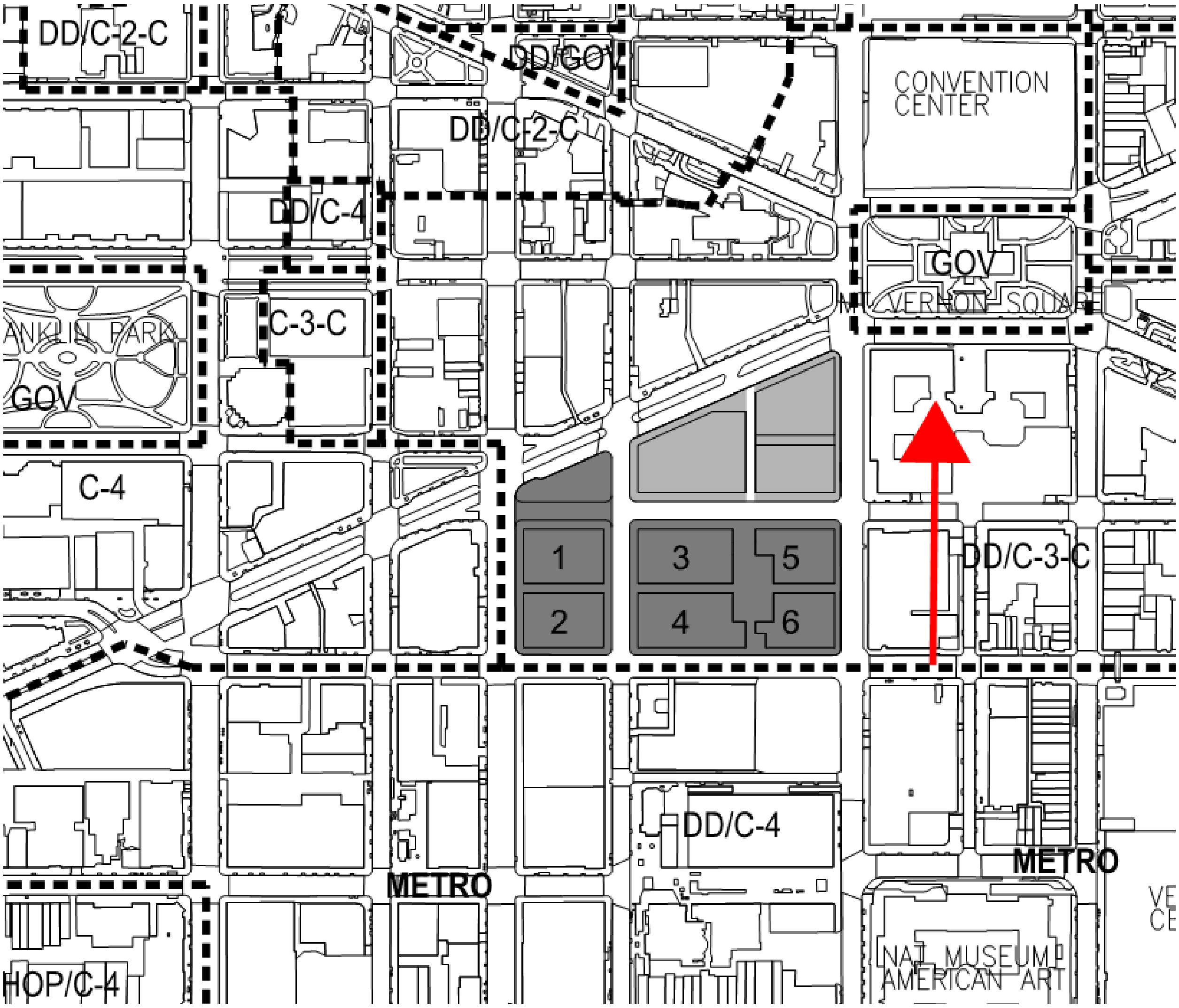


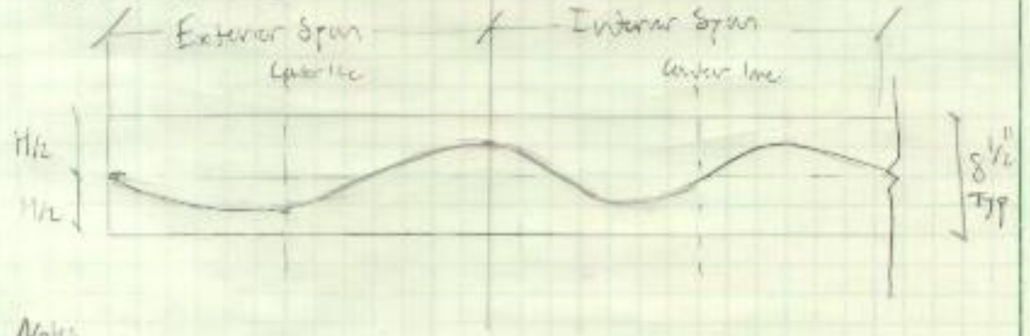
Figure 1: A plan view of the buildings inside the development shaded grey.

1. Gravity Loads

1.1 Floor Loads

Floor Dead Loads

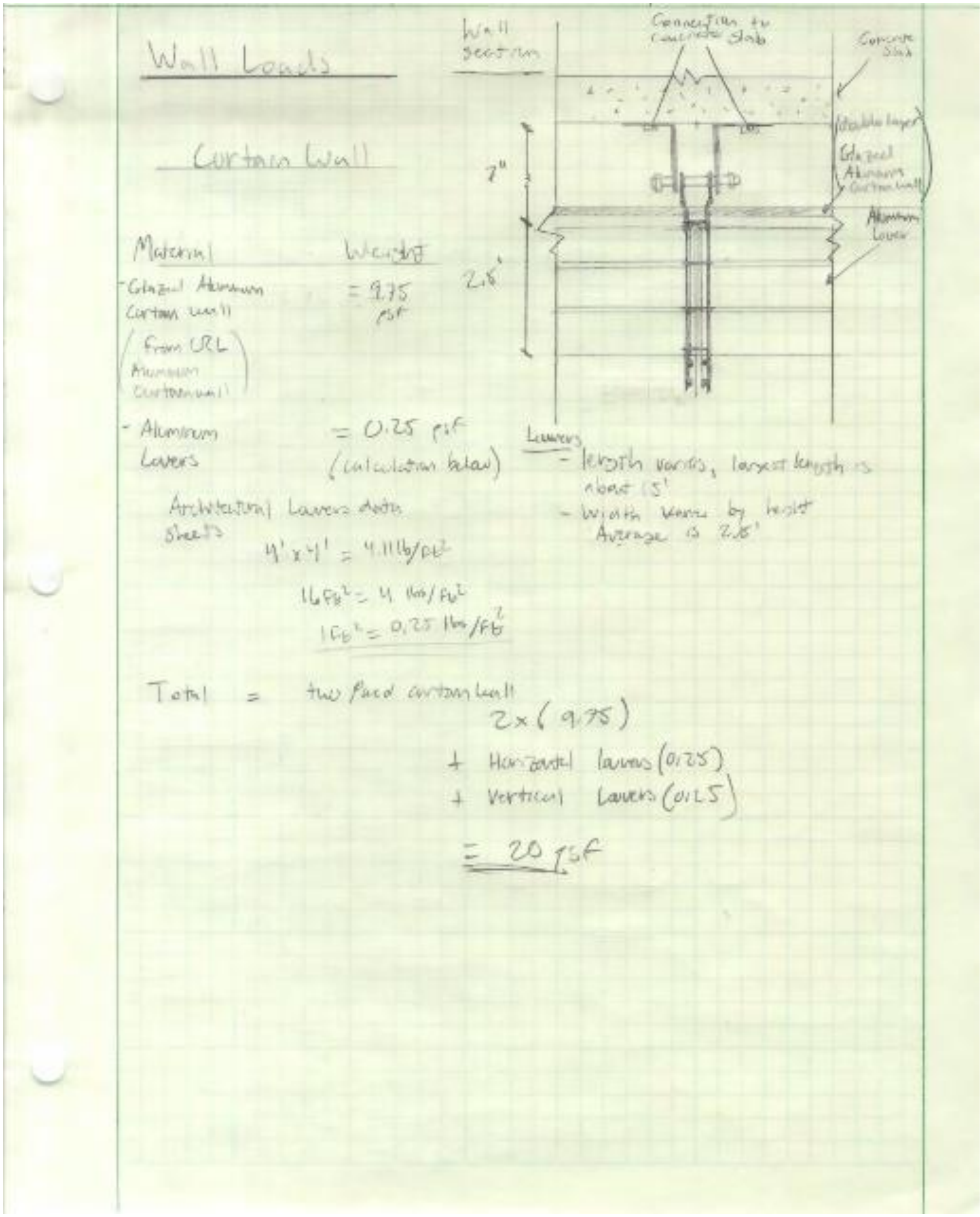
Typical Tendon Profile for Slab



Notes
- weight of tendon is neglected
- uplift due to cable under tension is neglected

	Component	Weight PSF
	8 1/2" Concrete	150 lbs/ft³ · 8.5/12 = 106.25 lbs
SDL	MEP allowance	= 10 psf
	Office Partitions	= 20 psf
	Total	= 136.25 psf ≈ 137 psf

1.2 Wall Loads



Wall Loads

Curtain Wall

Material	Weight
- Glazed Aluminum Curtain wall (from URL Aluminum Curtainwall)	= 9.75 psf
- Aluminum Louvers	= 0.25 psf (calculation below)

Louvers
- length varies, largest length is about 15'
- Width varies by height Average is 2.8'

Architectural Louvers data sheets

4' x 4' = 4.1 lb/ft²

16 ft² = 4 lbs/ft²

1 ft² = 0.25 lbs/ft²

Total = two faced curtain wall

2 x (9.75)

+ Horizontal louvers (0.25)

+ Vertical Louvers (0.25)

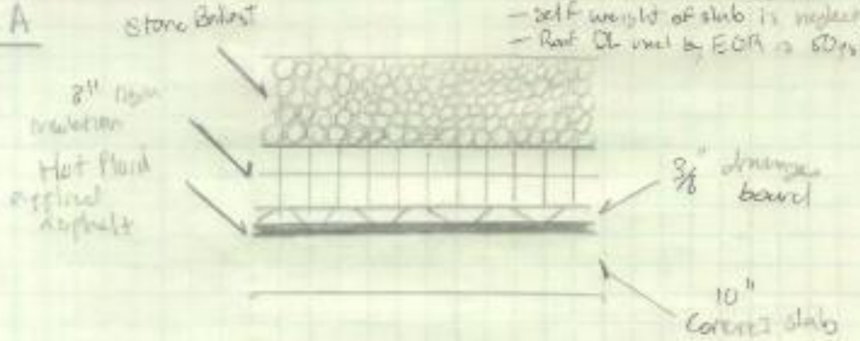
= 20 psf

1.3 Roof Loads

General Notes
- Uplift due to Post tensioned system not accounted for
- material weights from technical data sheets
- self weight of slab is neglected
- Roof DL used by EOR is 50 psf

Roof Dead Loads

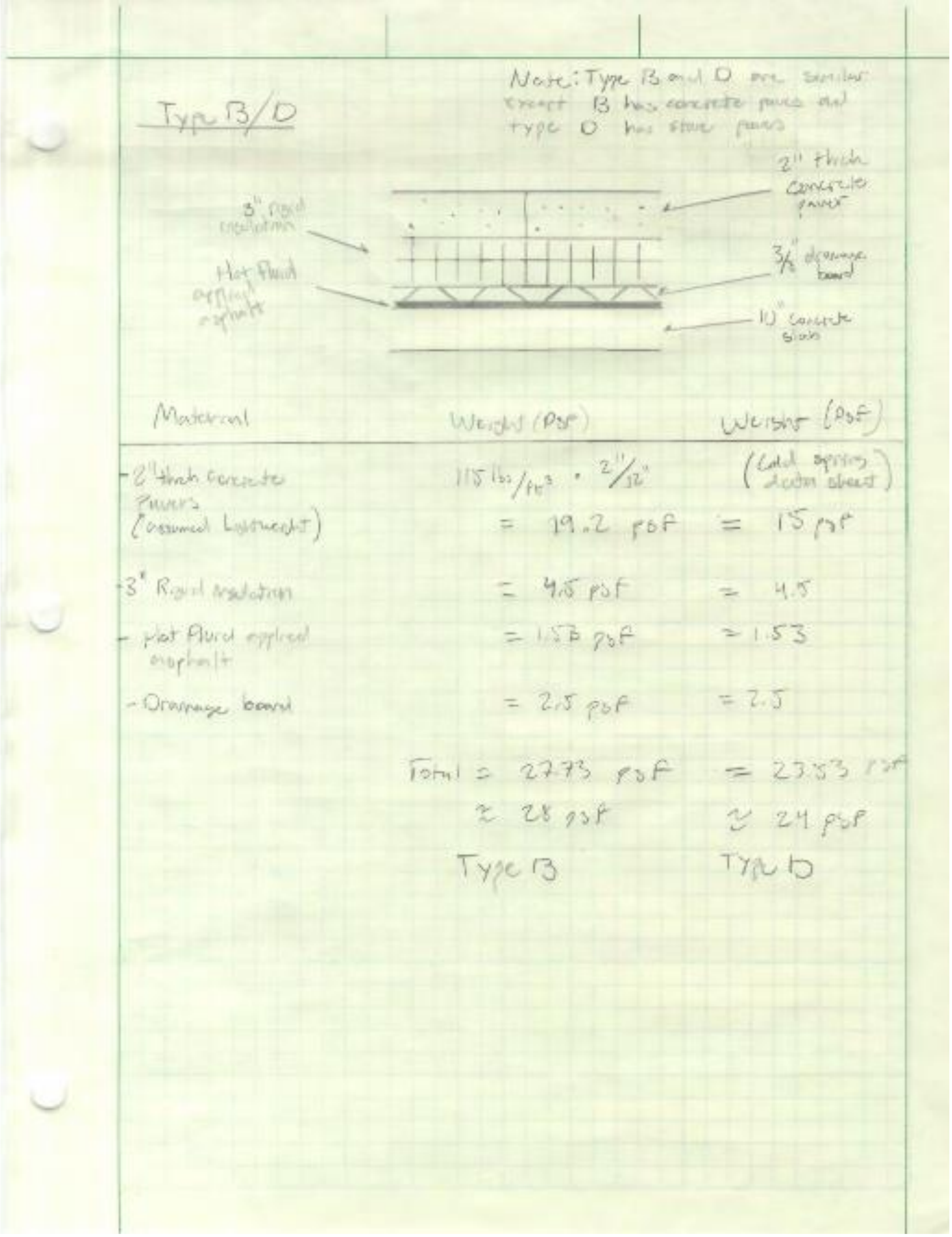
Type A



Material	Weight psf
- 8" rigid insulation	1" = 0.5 psf = 4.5 psf (Base Cascade data)
- Stone Ballast	= 8.7 psf (Base Cascade data)
- Hot fluid applied asphalt	= 1.53 psf (215 mils, from Carlisle technical data)
- Drainage board	= 2.5 psf (Carlisle datasheet)

☆ Ignoring ☆
Self weight of Slab
= 150 × 10/12
= 125 psf

Total = 17.23 psf
≈ 17.5 psf

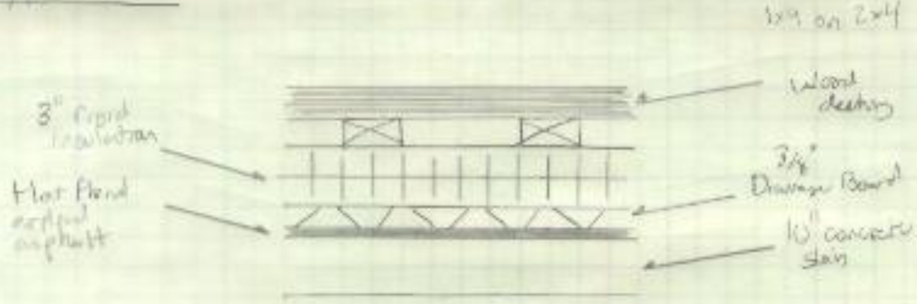


Type B/D

Note: Type B and D are similar except B has concrete pavers and type D has stone pavers

Material	Weight (psf)	Weight (psf)
- 2" thick concrete Pavers (assumed Lightweight)	$115\ \mathrm{lbs/ft^3} \cdot 2''/12''$ = 19.2 psf	(Cold spring data sheet) = 15 psf
- 3" Rigid insulation	= 4.5 psf	= 4.5
- Hot fluid applied asphalt	= 1.53 psf	= 1.53
- Drainage board	= 2.5 psf	= 2.5
	Total = 27.73 psf ≈ 28 psf	= 23.53 psf ≈ 24 psf
	Type B	Type D

Type C

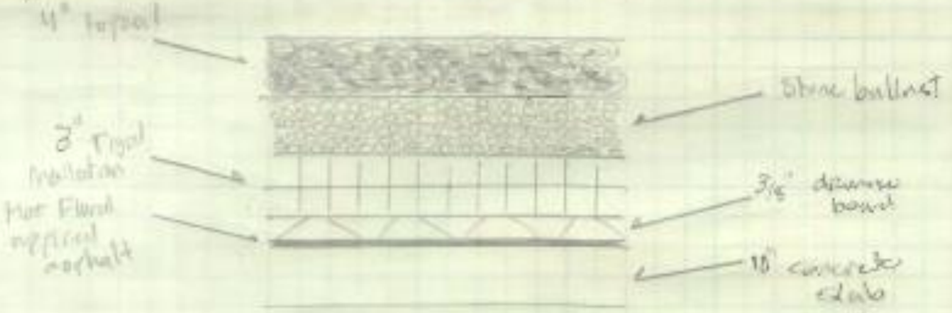


Material	Weight (PSF)	
- 1" Hardwood Decking	= 4 psf	(Boise Cascade)
- 2"x4" Sleepers	= 1.1 psf	(Boise Cascade
- 3" rigid insulation	= 4.5 psf	
- 3/8" Drainage Board	= 2.5 psf	
- Hot Fluid applied asphalt	= 1.53 psf	

Total = 14.13 psf

≈ 15 psf

Type E



Material	Weight (PSF)
- 4" Topsoil (0-4 blows per ft)	70-100 lbs/ft³ (Geotechnicalinfo.com) 85 lbs/ft³ · 4"/12 = 28.3 psf
- Stone ballast	= 8.7 psf
- 3" Rigid Insulation	= 4.5 psf
- Drainage board	= 2.5 psf
- Hot Fluid applied Asphalt	= 1.53 psf

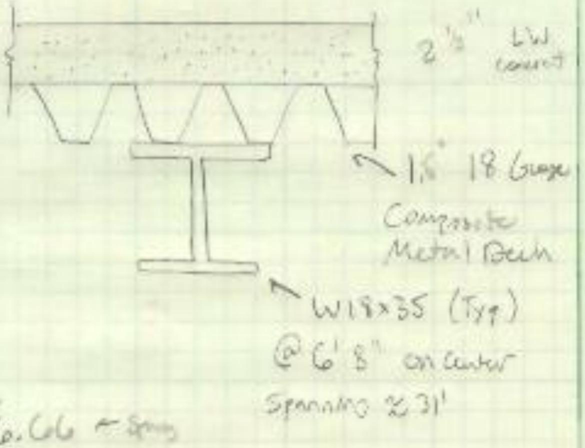
Total = 45.53 psf

≈ 46 psf

Note: EOR used 50 psf for Green Roof

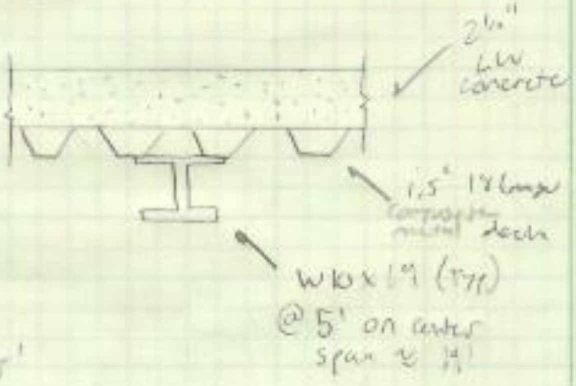
Pent House Roof Framing

Material	Weight
2½" concrete	$= 115\ \text{p/ft}^3 \cdot 2\tfrac{1}{2}/12$ = 24 psf
18 GA Metal Deck (From Vulcraft Catalog)	= 2.82 psf
W18x35	$= 35\ \text{lb/ft} \div 1/6.66$ ← Spacing = 5.83 psf
MEP	= 10 psf
Total	= 42.65 psf ≈ 43 psf

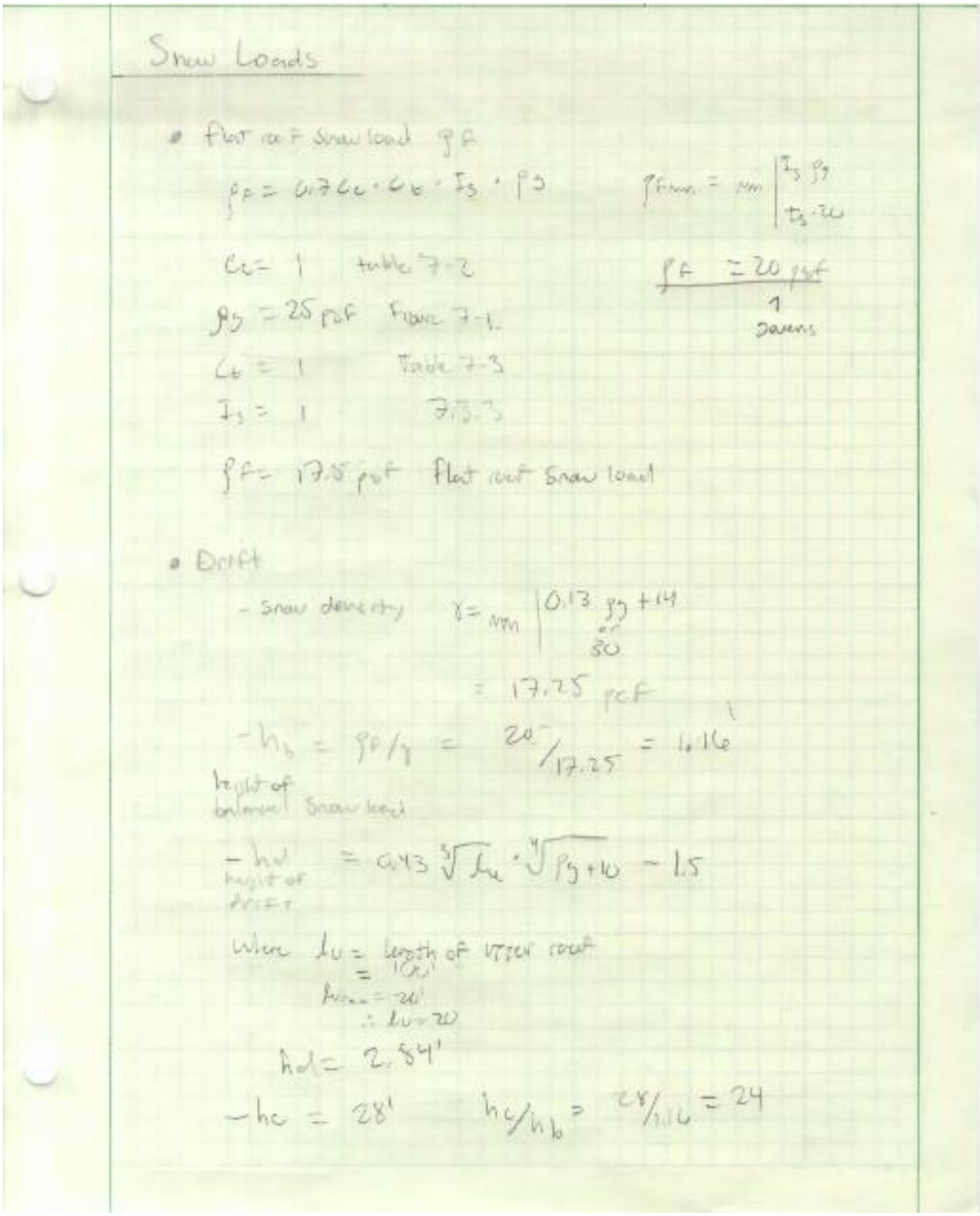


Pent House Mezzanine Framing

Material	Weight
2½" concrete	= 24 psf
MEP	= 10 psf
18 GA Metal Deck	= 2.82 psf
W10x19	$= 19\ \text{lb/ft} \div 1/5'$ = 3.8 psf
Total	= 40.62 psf ≈ 41 psf



1.4 Snow Loads



Snow Loads

- flat roof snow load p_f

$p_f = 0.7 C_e \cdot C_t \cdot I_s \cdot p_g$

$p_{f\,min} = \min \left| \begin{matrix} I_s\, p_g \\ I_s \cdot 20 \end{matrix} \right.$

$C_e = 1$ table 7-2

$\underline{p_f = 20 \text{ psf}}$

↑ governs

$p_g = 25$ psf figure 7-1

$C_t = 1$ Table 7-3

$I_s = 1$ 7.3.3

$p_f = 17.5$ psf flat roof snow load

- Drift

– snow density $\gamma = \min \left| \begin{matrix} 0.13\, p_g + 14 \\ \text{or} \\ 30 \end{matrix} \right.$

$= 17.25$ pcf

$- h_b = p_f / \gamma = 20 / 17.25 = 1.16'$

height of balanced snow load

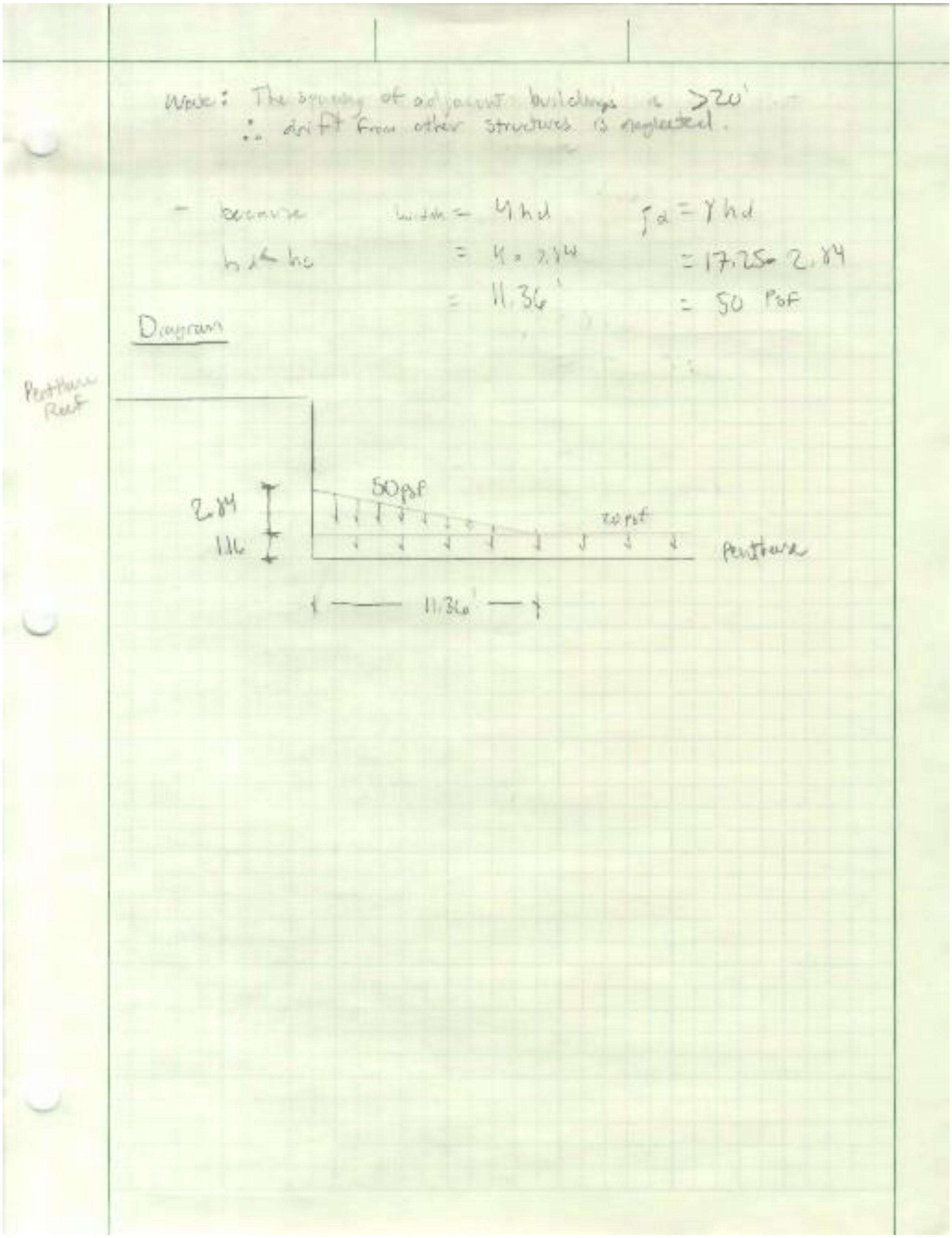
$- h_d = 0.43 \sqrt[3]{l_u} \cdot \sqrt[4]{p_g + 10} - 1.5$

height of drift

where l_u = length of upper roof
= 100'
$l_{min} = 20'$
$\therefore l_u = 20$

$h_d = 2.84'$

$- h_c = 28'$ $h_c / h_b = 28 / 1.16 = 24$



Note: The spacing of adjacent buildings is >20'
∴ drift from other structures is neglected.

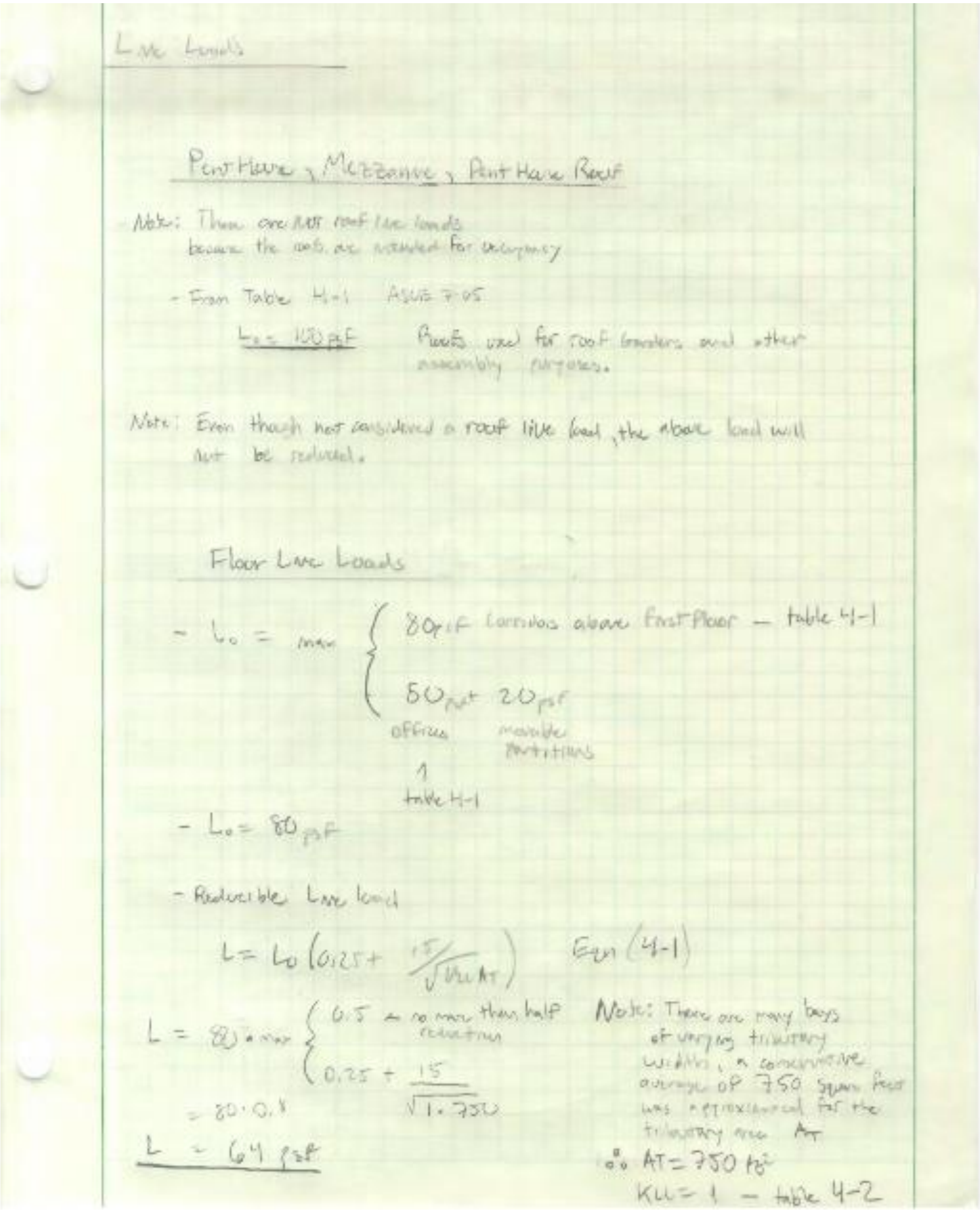
- because $h_d < h_c$

$w_{dth} = 4h_d$
$= 4 \times 2.84$
$= 11.36'$

$p_d = \gamma h_d$
$= 17.25 \times 2.84$
$= 50$ psf

Diagram

1.5 Live Loads



Live Loads

Pent House, Mezzanine, Pent House Roof

Note: There are not roof live loads because the roofs are intended for occupancy

- From Table 4-1 ASCE 7-05

$L_o = 100$ psf — Roofs used for roof gardens and other assembly purposes.

Note: Even though not considered a roof live load, the above load will not be reduced.

Floor Live Loads

$$- L_o = \max \begin{cases} 80\,\text{psf corridors above first floor — table 4-1} \\ \underset{\text{offices}}{50_{psf}} + \underset{\text{movable partitions}}{20_{psf}} \end{cases}$$

↑ table 4-1

$- L_o = 80_{psf}$

- Reducible Live load

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right) \quad \text{Eqn } (4\text{-}1)$$

$$L = 80 \cdot \max \begin{cases} 0.5 \leftarrow \text{no more than half reduction} \\ 0.25 + \dfrac{15}{\sqrt{1 \cdot 750}} \end{cases}$$

$= 80 \cdot 0.8$

$\underline{L = 64\ \text{psf}}$

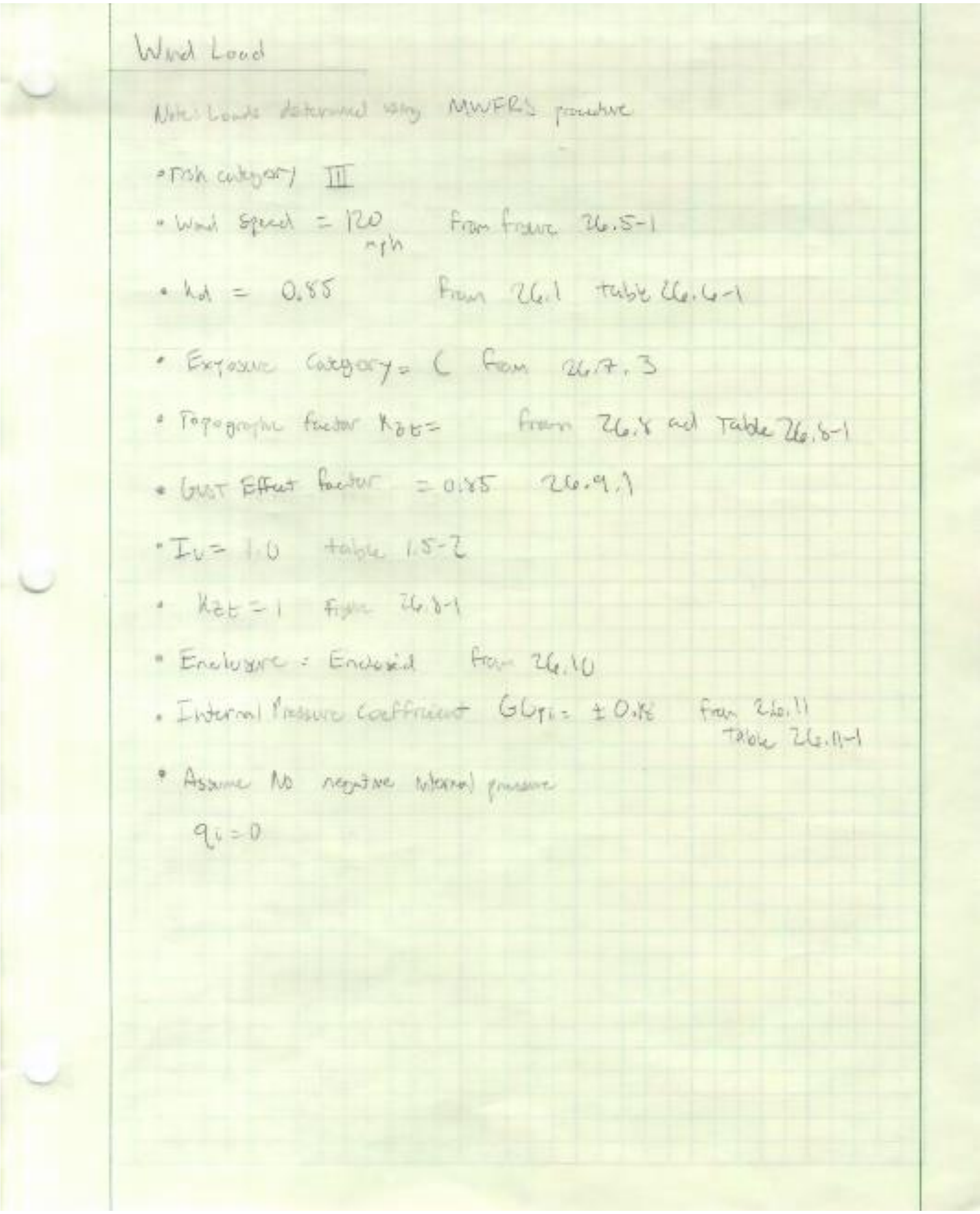
Note: There are many bays of varying tributary widths, a conservative average of 750 square feet was approximated for the tributary area A_T

$\therefore A_T = 750\ \text{ft}^2$

$K_{LL} = 1$ — table 4-2

2. Lateral Loads

2.1 Wind Loads



Wind Load

Note: Loads determined using MWFRS procedure

- Risk category III
- Wind Speed = 120 mph from figure 26.5-1
- K_d = 0.85 from 26.1 table 26.6-1
- Exposure Category = C from 26.7.3
- Topographic factor K_{zt} = from 26.8 and Table 26.8-1
- Gust Effect factor = 0.85 26.9.1
- I_w = 1.0 table 1.5-2
- K_{zt} = 1 figure 26.8-1
- Enclosure = Enclosed from 26.10
- Internal Pressure Coefficient GC_{pi} = ± 0.18 from 26.11 Table 26.11-1
- Assume No negative internal pressure

  $q_i = 0$

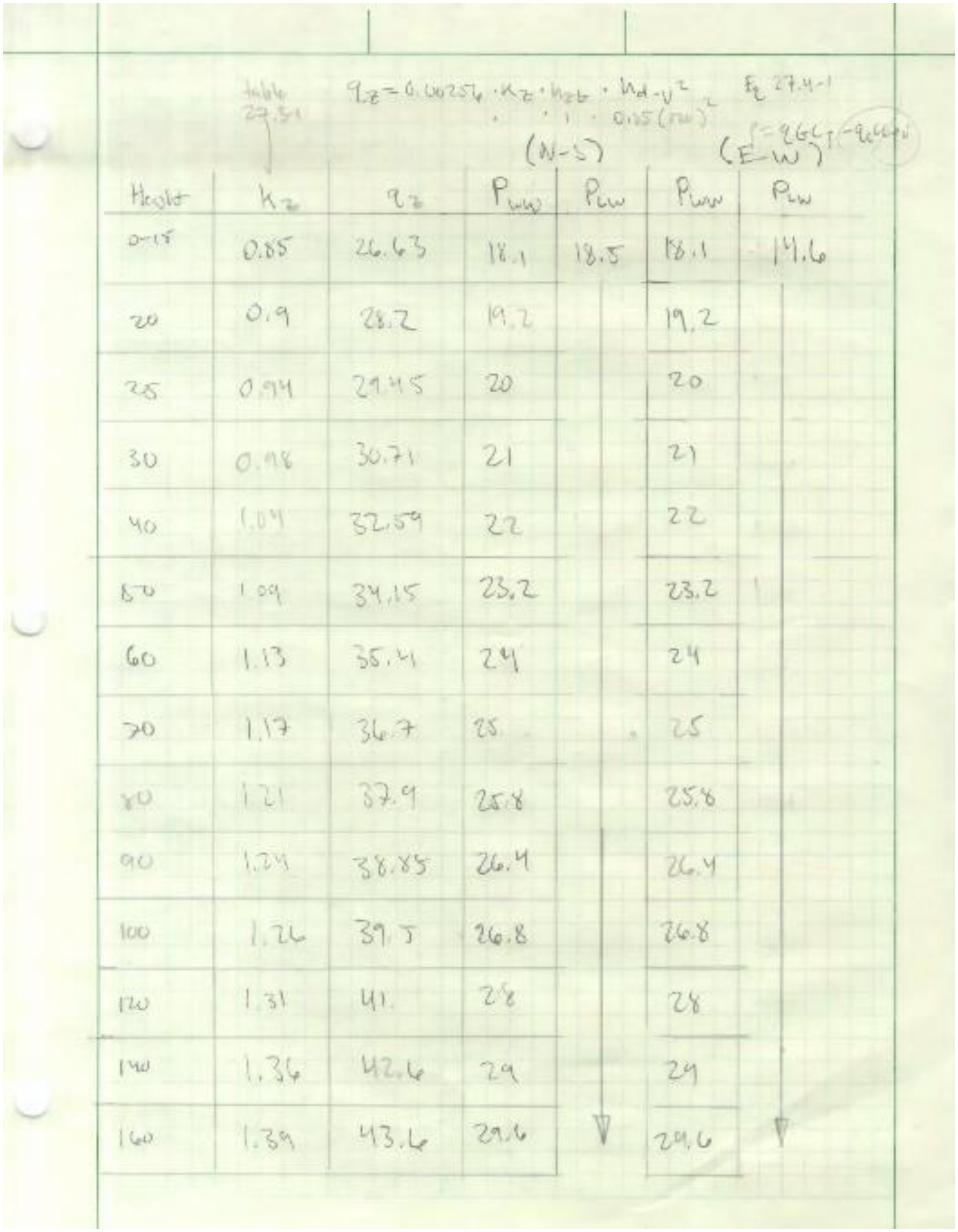


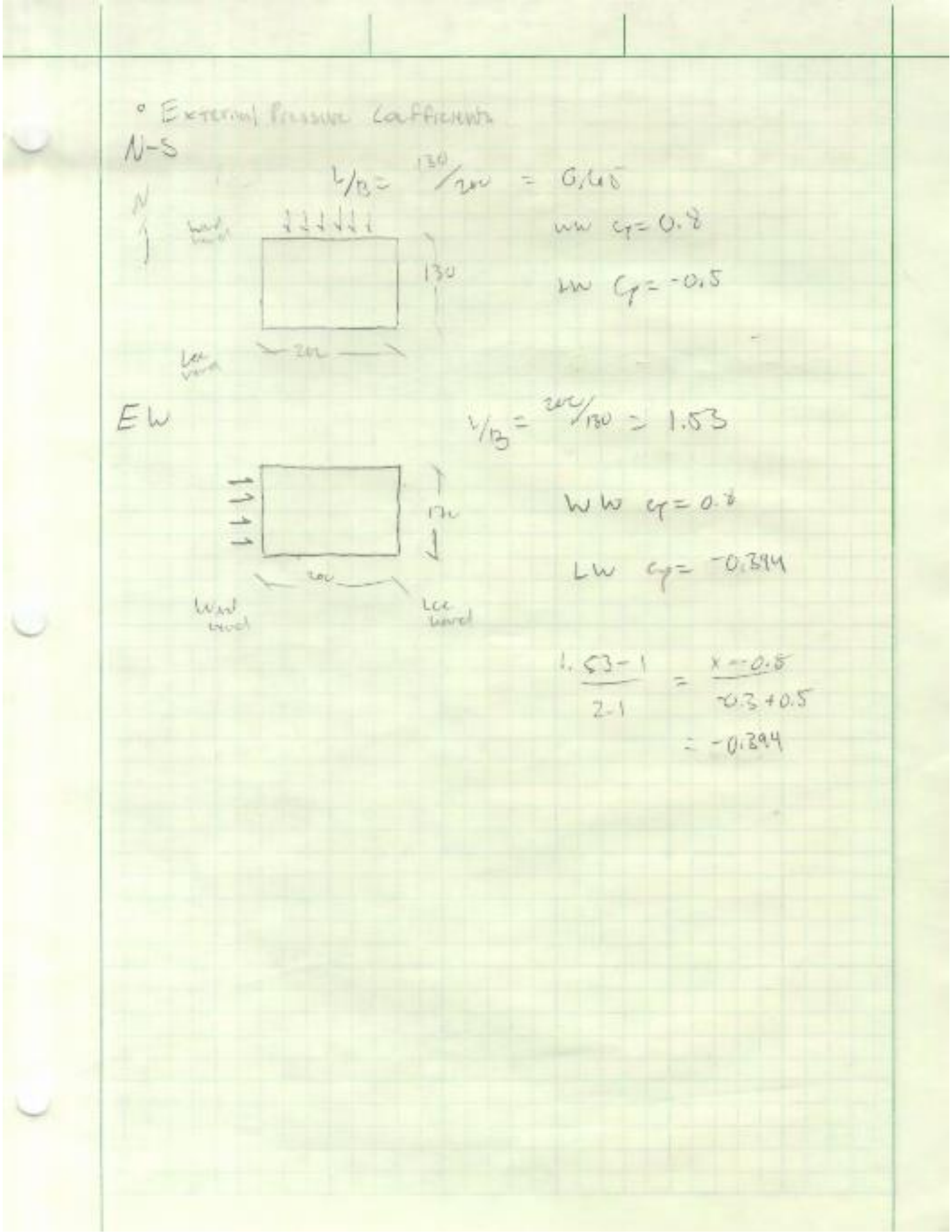
table 27.3.1

$q_z = 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2$ Eq 27.4-1

$\cdot \quad \cdot 1 \cdot 0.85 (\ldots)^2$

$p = qGC_p - q_i(GC_{pi})$

			(N-S)		(E-W)	
Height	K_z	q_z	P_{WW}	P_{LW}	P_{WW}	P_{LW}
0-15	0.85	26.63	18.1	18.5	18.1	14.6
20	0.9	28.2	19.2		19.2	
25	0.94	29.45	20		20	
30	0.98	30.71	21		21	
40	1.04	32.59	22		22	
50	1.09	34.15	23.2		23.2	
60	1.13	35.4	24		24	
70	1.17	36.7	25		25	
80	1.21	37.9	25.8		25.8	
90	1.24	38.85	26.4		26.4	
100	1.26	39.5	26.8		26.8	
120	1.31	41.	28		28	
140	1.36	42.6	29		29	
160	1.39	43.6	29.6		29.6	



- External Pressure Coefficients

N-S

$L/B = 130/200 = 0.65$

WW $C_p = 0.8$

LW $C_p = -0.5$

EW

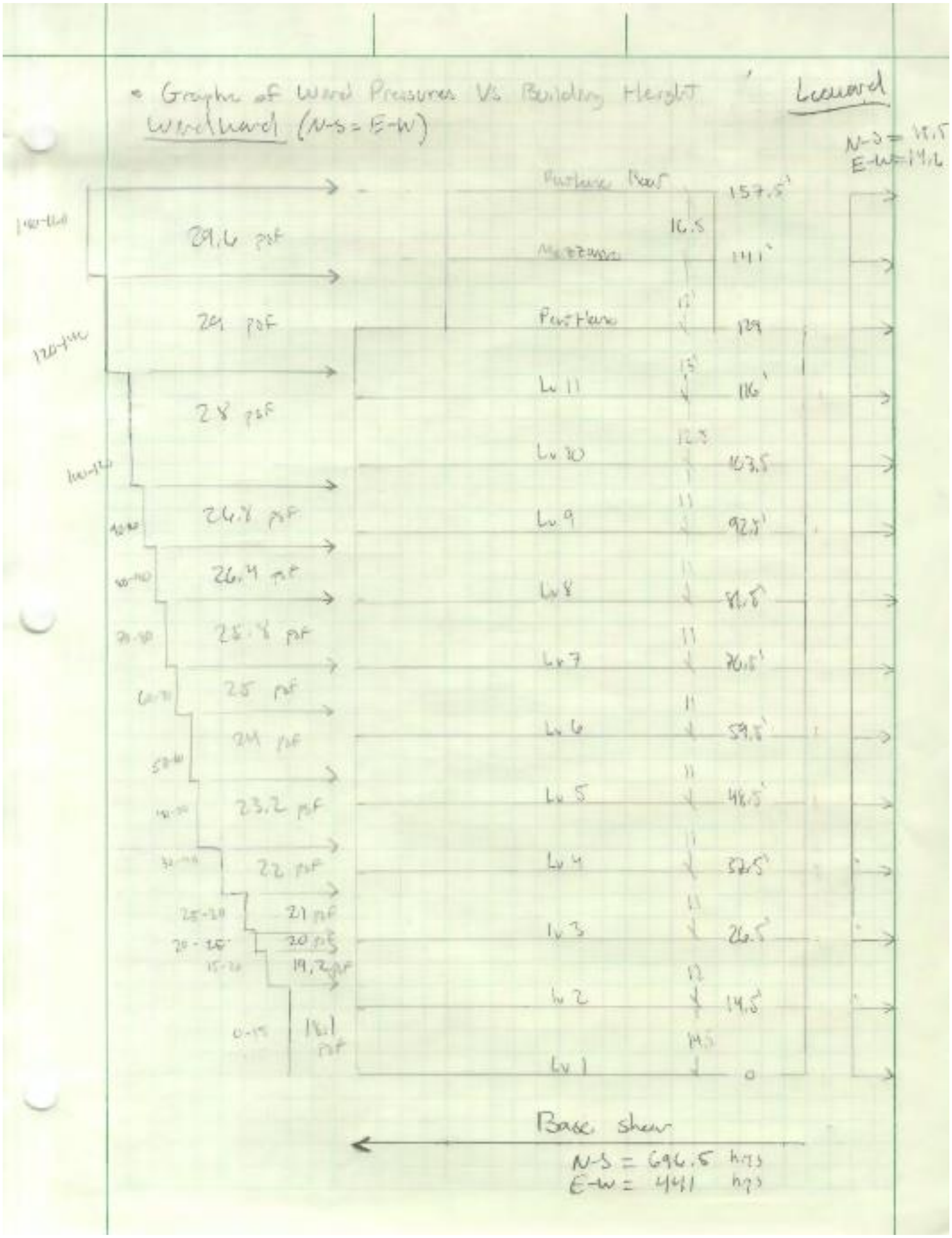
$L/B = 200/130 = 1.53$

WW $C_p = 0.8$

LW $C_p = -0.394$

$$\frac{1.53-1}{2-1} = \frac{x - -0.5}{-0.3+0.5}$$

$$= -0.394$$



- Graphs of Wind Pressures Vs Building Height

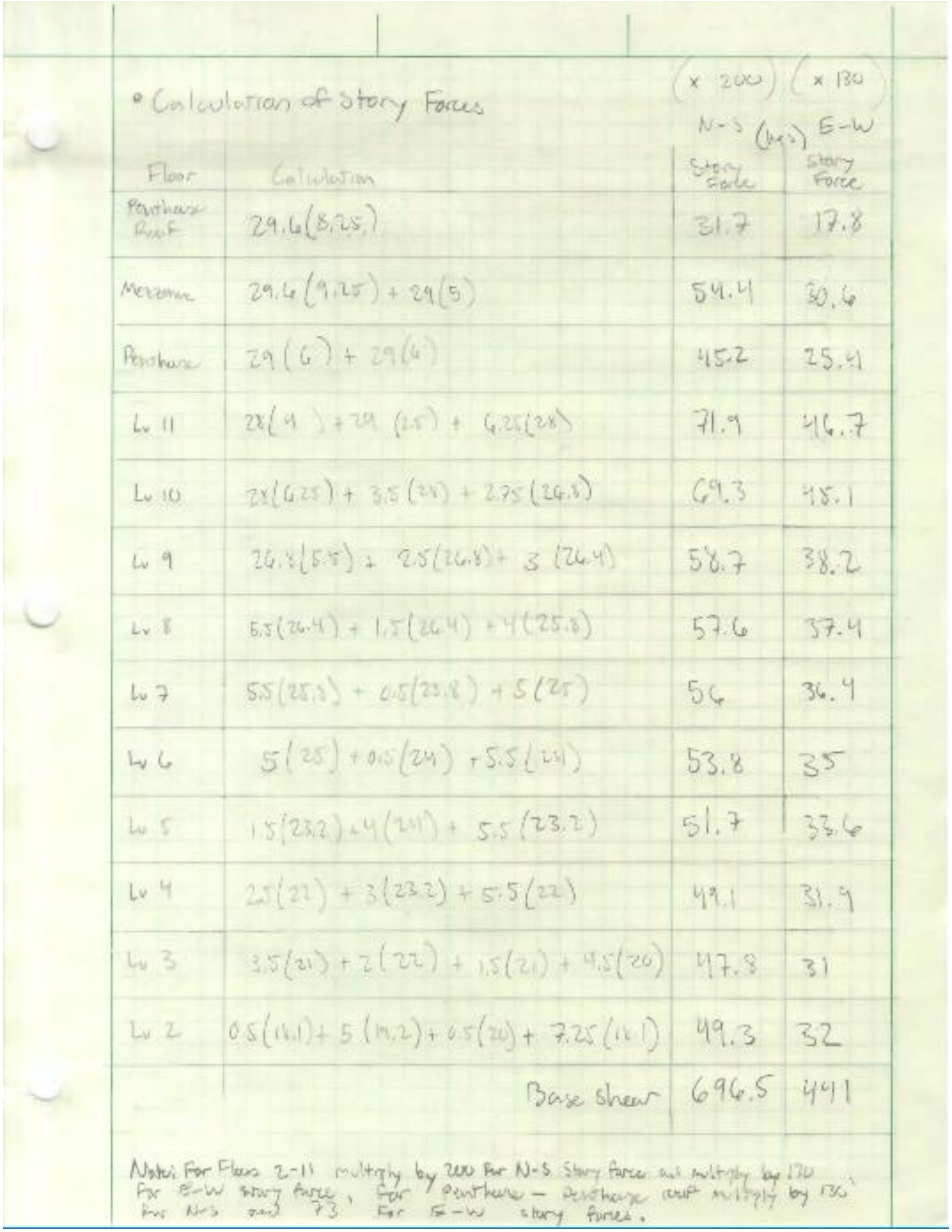
Windward (N-S = E-W)

Leeward

N-S = 15.5
E-W = 14.6

Base shear

N-S = 696.5 kips
E-W = 441 kips

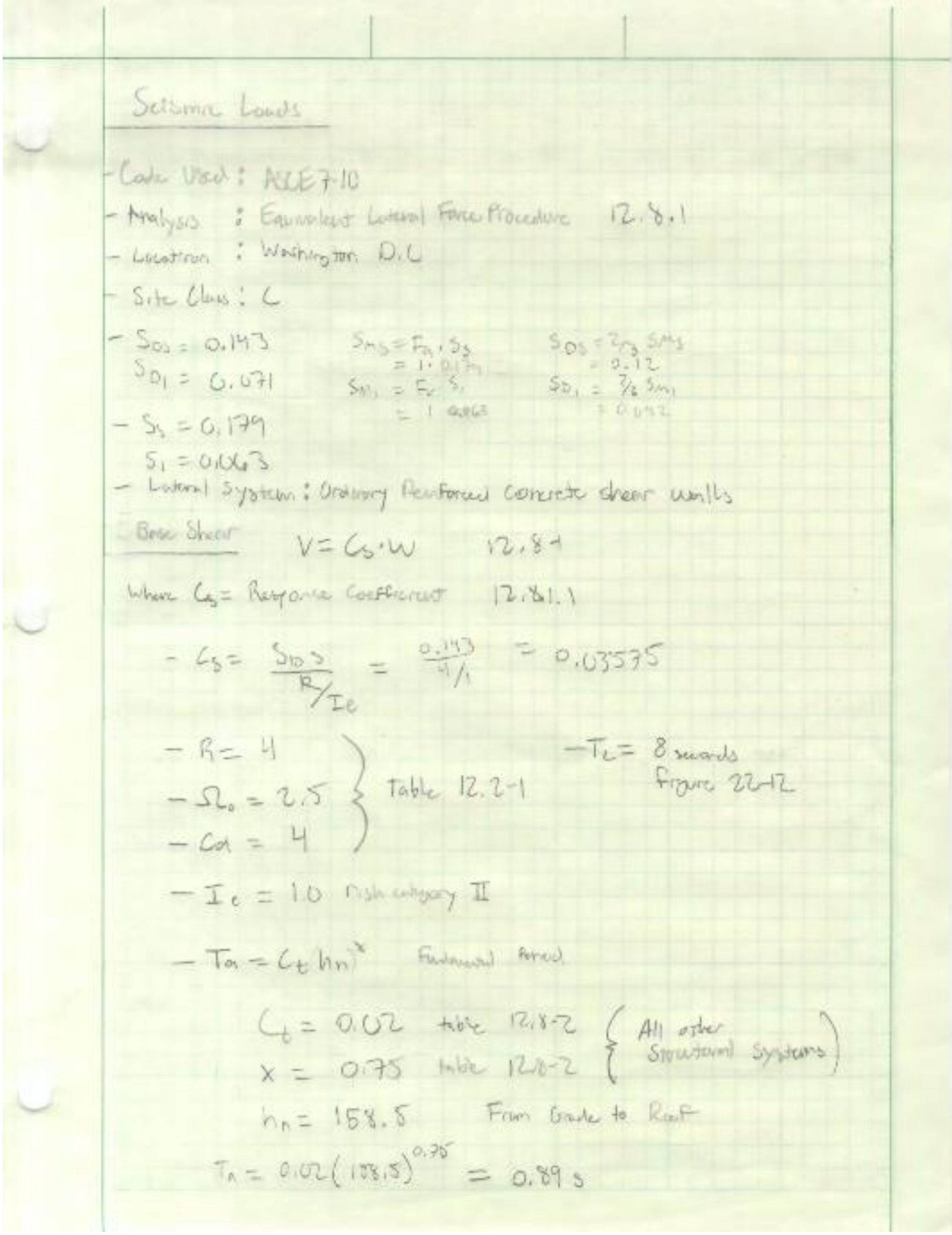


• Calculation of Story Forces

Floor	Calculation	(× 200) N-S Story Force	(kips) (× 130) E-W Story Force
Penthouse Roof	29.6(8.25)	31.7	17.8
Mezzanine	29.6(9.25) + 29(5)	54.4	30.6
Penthouse	29(6) + 29(4)	45.2	25.4
Lv 11	28(4) + 29(2.5) + 6.25(28)	71.9	46.7
Lv 10	28(6.25) + 3.5(28) + 2.75(26.8)	69.3	48.1
Lv 9	26.8(5.5) + 2.5(26.8) + 3(26.4)	58.7	38.2
Lv 8	5.5(26.4) + 1.5(26.4) + 4(25.8)	57.6	37.4
Lv 7	5.5(25.8) + 0.5(23.8) + 5(25)	56	36.4
Lv 6	5(25) + 0.5(24) + 5.5(24)	53.8	35
Lv 5	1.5(23.2) + 4(24) + 5.5(23.2)	51.7	33.6
Lv 4	2.5(22) + 3(23.2) + 5.5(22)	49.1	31.9
Lv 3	3.5(21) + 2(22) + 1.5(21) + 4.5(20)	47.8	31
Lv 2	0.5(16.1) + 5(19.2) + 0.5(20) + 7.25(16.1)	49.3	32
	Base shear	696.5	441

Note: For Floors 2-11 multiply by 200 for N-S story force and multiply by 130 for E-W story force, for Penthouse – Penthouse roof multiply by 130' for N-S and 73' for E-W story forces.

2.2 Seismic Loads



Seismic Loads

- Code Used : ASCE 7-10
- Analysis : Equivalent Lateral Force Procedure 12.8.1
- Location : Washington D.C
- Site Class : C
- $S_{DS} = 0.143$
  $S_{D1} = 0.071$

$S_{MS} = F_a \cdot S_s = 1 \cdot 0.179$

$S_{M1} = F_v S_1 = 1 \; 0.063$

$S_{DS} = \frac{2}{3} S_{MS} = 0.12$

$S_{D1} = \frac{2}{3} S_{M1} = 0.042$

- $S_s = 0.179$
  $S_1 = 0.063$
- Lateral System : Ordinary Reinforced Concrete shear walls

Base Shear $V = C_s \cdot W$ 12.8-1

Where C_s = Response Coefficient 12.8.1.1

- $C_s = \frac{S_{DS}}{R/I_e} = \frac{0.143}{4/1} = 0.03575$

- $R = 4$
- $\Omega_0 = 2.5$
- $C_d = 4$

Table 12.2-1

- $T_L = 8$ seconds figure 22-12

- $I_e = 1.0$ risk category II

- $T_a = C_t h_n^x$ Fundamental Period

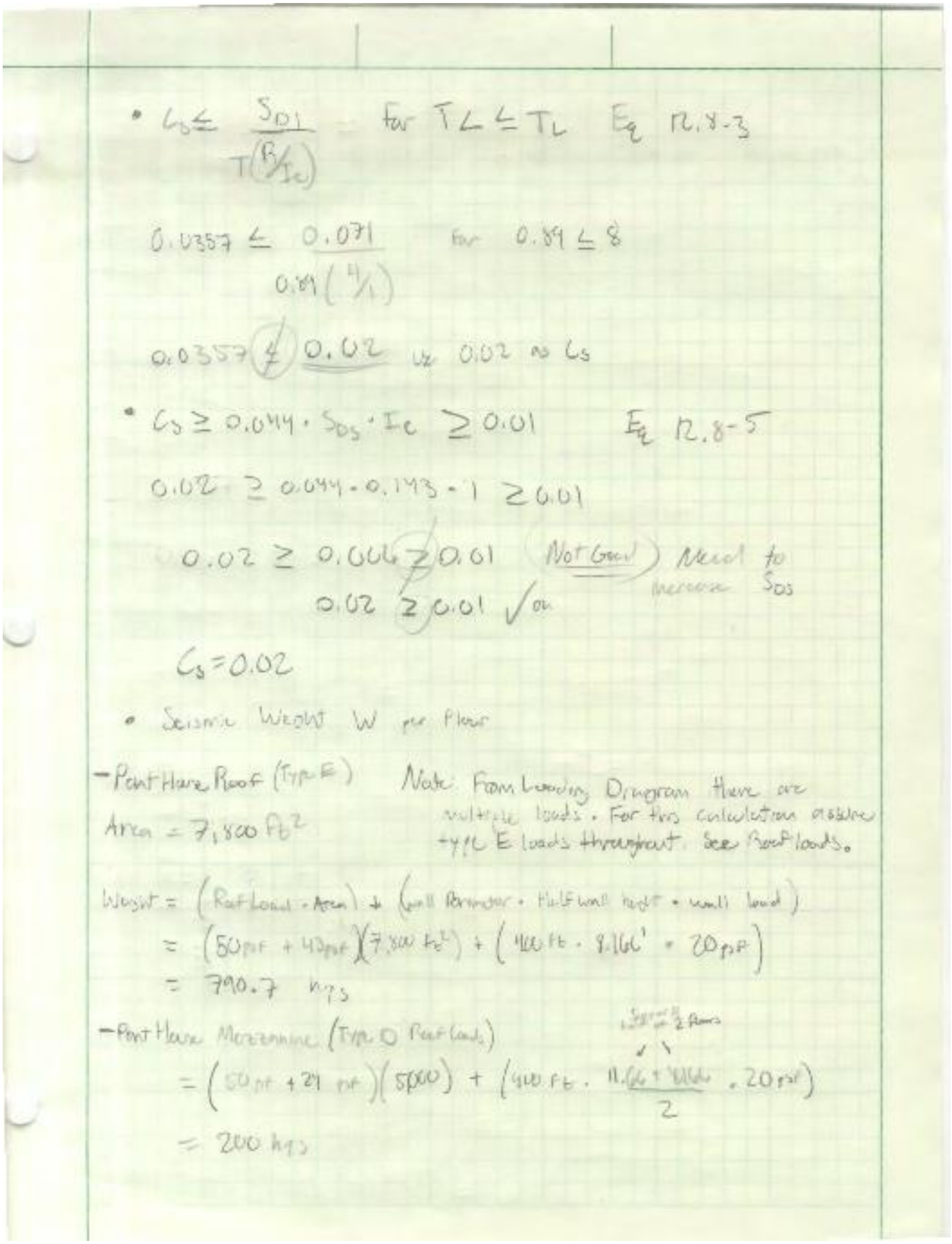
$C_t = 0.02$ table 12.8-2

$x = 0.75$ table 12.8-2

(All other Structural Systems)

$h_n = 158.5$ From Grade to Roof

$T_a = 0.02(158.5)^{0.75} = 0.89\text{ s}$



- $C_s \leq \dfrac{S_{D1}}{T\left(\frac{R}{I_e}\right)}$ for $T < \leq T_L$ Eq 12.8-3

$0.0357 \leq \dfrac{0.071}{0.89\left(\frac{4}{1}\right)}$ for $0.89 \leq 8$

$0.0357 \not\leq \underline{0.02}$ use 0.02 as C_s

- $C_s \geq 0.044 \cdot S_{DS} \cdot I_e \geq 0.01$ Eq 12.8-5

$0.02 \geq 0.044 \cdot 0.143 \cdot 1 \geq 0.01$

$0.02 \geq 0.006 \not\geq 0.01$ (Not Good) Need to increase S_{DS}

$0.02 \geq 0.01$ ✓ ok

$C_s = 0.02$

- Seismic Weight W per Floor

–Pent House Roof (Type E)

Area = 7,800 ft²

Note: From Loading Diagram there are multiple loads. For this calculation assume type E loads throughout. See Roof loads.

Weight = (Roof Load · Area) + (wall Perimeter · Half wall height · wall load)

= (50 psf + 43 psf)(7,800 ft²) + (400 ft · 8.166' · 20 psf)

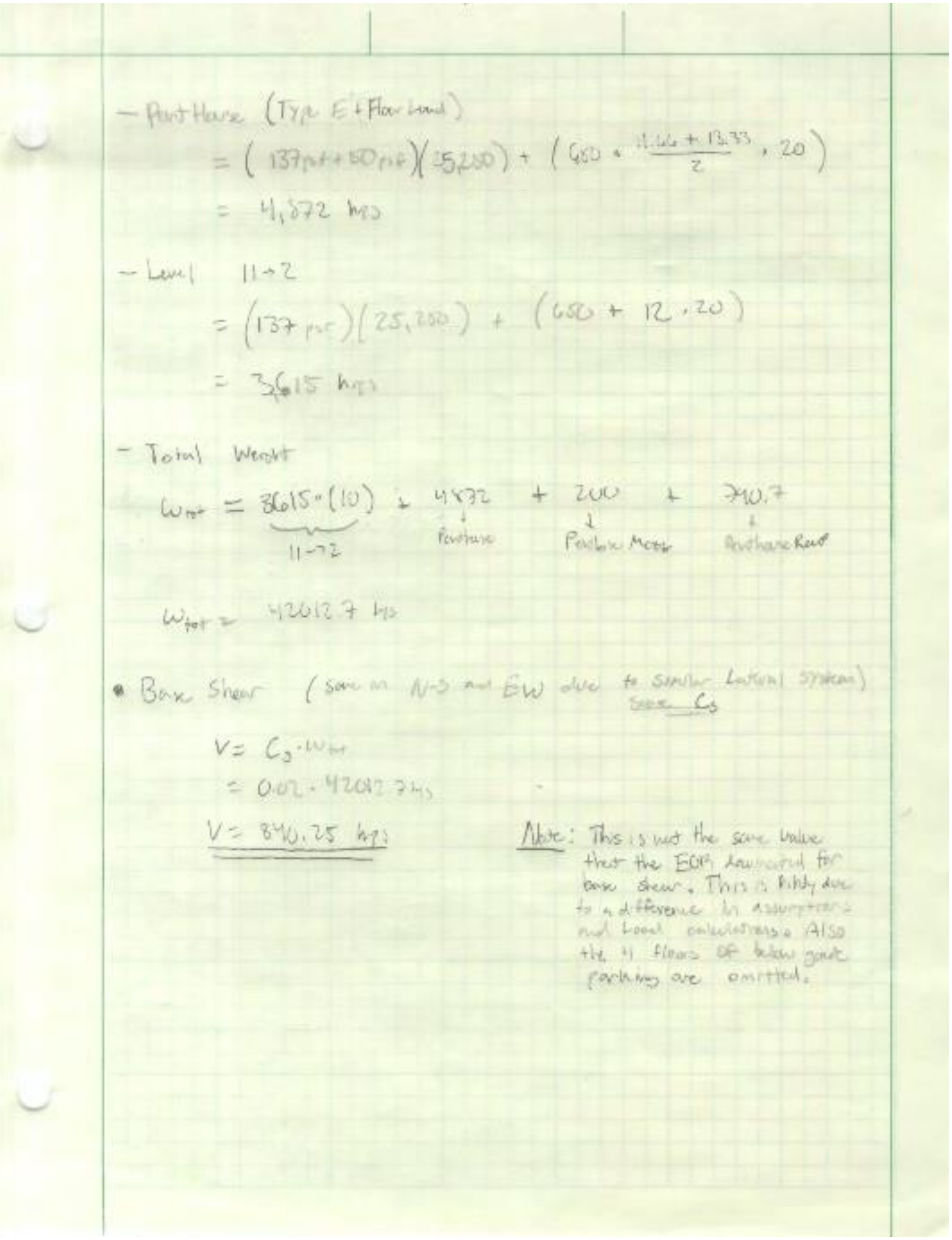
= 790.7 kips

–Pent House Mezzanine (Type D Roof Loads)

[illegible] 2 floors

$= (50\text{ psf} + 27\text{ psf})(5000) + \left(400\text{ ft} \cdot \dfrac{11.66 + 8.166}{2} \cdot 20\text{ psf}\right)$

= 200 kips



– Pent House (Type E + Floor Load)

$$= (137\,psf + 50\,psf)(25{,}200) + \left(650 \times \frac{11.66 + 13.33}{2} \times 20\right)$$

$$= 4{,}872 \text{ kips}$$

– Level 11 → 2

$$= (137\,psf)(25{,}200) + (650 \times 12 \times 20)$$

$$= 3{,}615 \text{ kips}$$

– Total Weight

$$W_{tot} = \underbrace{3615 \cdot (10)}_{11 \to 2} + \underset{\text{Penthouse}}{4872} + \underset{\text{Penthouse Mech}}{200} + \underset{\text{Penthouse Roof}}{740.7}$$

$$W_{tot} = 42012.7 \text{ kips}$$

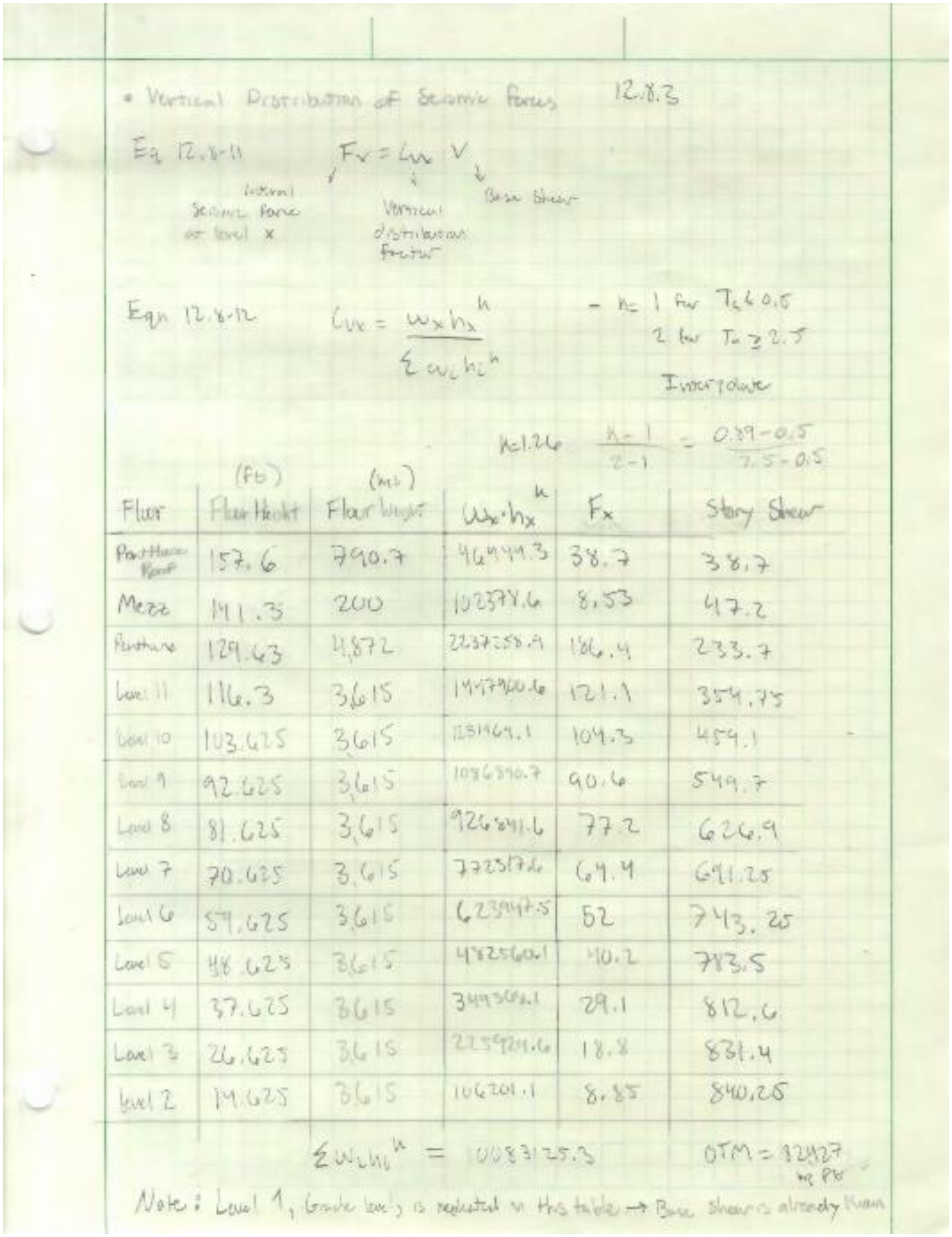
- Base Shear (same in N-S and EW due to similar Lateral System) Same C_s

$$V = C_s \cdot W_{tot}$$

$$= 0.02 \cdot 42012.7 \text{ kips}$$

$$\underline{\underline{V = 840.25 \text{ kips}}}$$

Note: This is not the same value that the EOR accounted for base shear. This is likely due to a difference in assumptions and load calculations. Also the 4 floors of below grade parking are omitted.



- Vertical Distribution of Seismic Forces 12.8.3

Eq 12.8-11 $F_x = C_{vx} V$

lateral seismic force at level x

Vertical distribution factor

Base Shear

Eqn 12.8-12 $C_{vx} = \frac{w_x h_x^k}{\Sigma w_i h_i^k}$

- k = 1 for $T_a \leq 0.5$

2 for $T_a \geq 2.5$

Interpolate

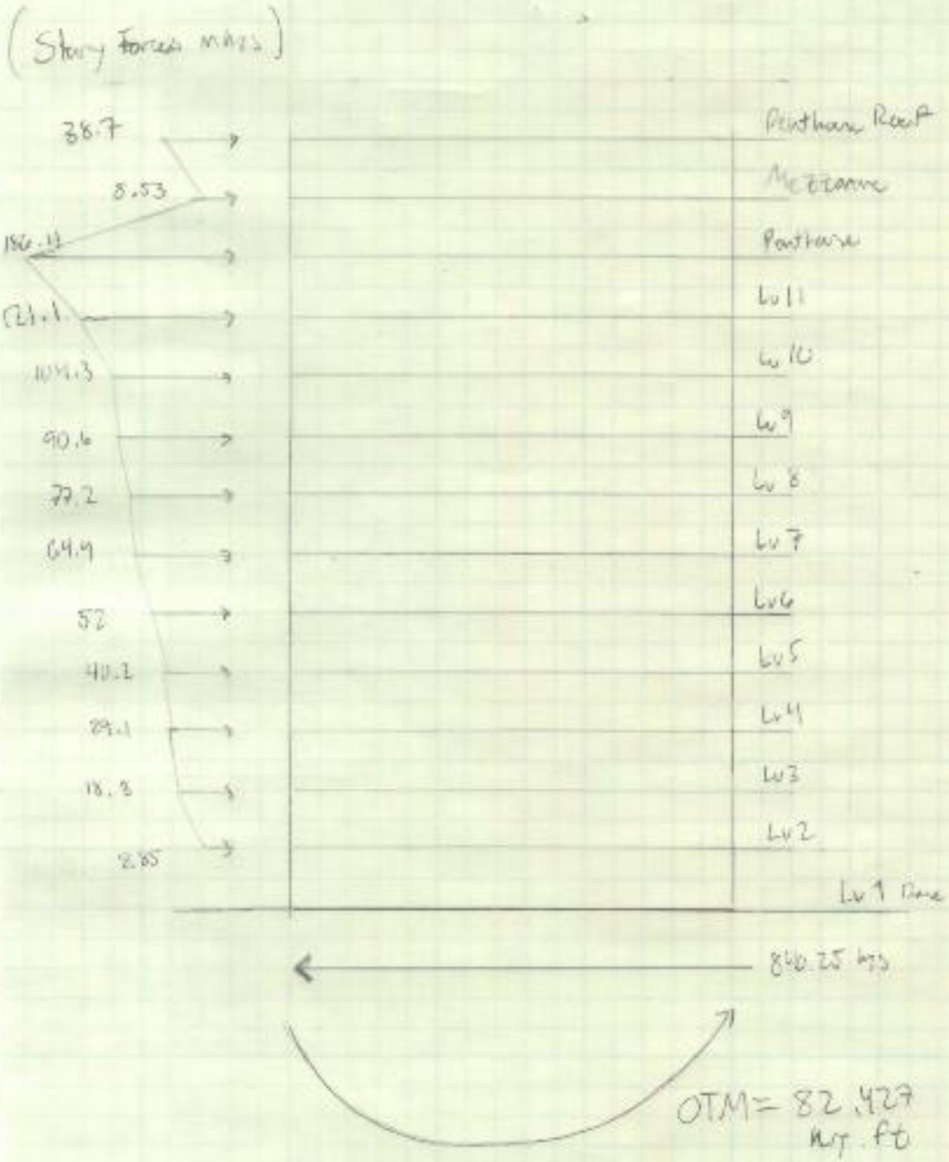
k=1.26 $\frac{k-1}{2-1} = \frac{0.79-0.5}{2.5-0.5}$

Floor	(ft) Floor Height	(kips) Floor Weight	$w_x \cdot h_x^k$	F_x	Story Shear
PentHouse Roof	157.6	790.7	46944.3	38.7	38.7
Mezz	141.3	200	102378.6	8.53	47.2
Penthouse	129.63	11,872	2237258.9	186.4	233.7
Level 11	116.3	3,615	1447960.6	121.1	354.75
Level 10	103.625	3,615	1251464.1	104.3	459.1
Level 9	92.625	3,615	1086890.7	90.6	549.7
Level 8	81.625	3,615	926841.6	77.2	626.9
Level 7	70.625	3,615	772317.6	64.4	691.25
Level 6	59.625	3,615	623947.5	52	743.25
Level 5	48.625	3,615	482560.1	40.2	783.5
Level 4	37.625	3,615	349360.1	29.1	812.6
Level 3	26.625	3,615	225924.6	18.8	831.4
Level 2	14.625	3,615	106201.1	8.85	840.25

$\Sigma w_i h_i^k = 10083125.3$ OTM = 82427 k-ft

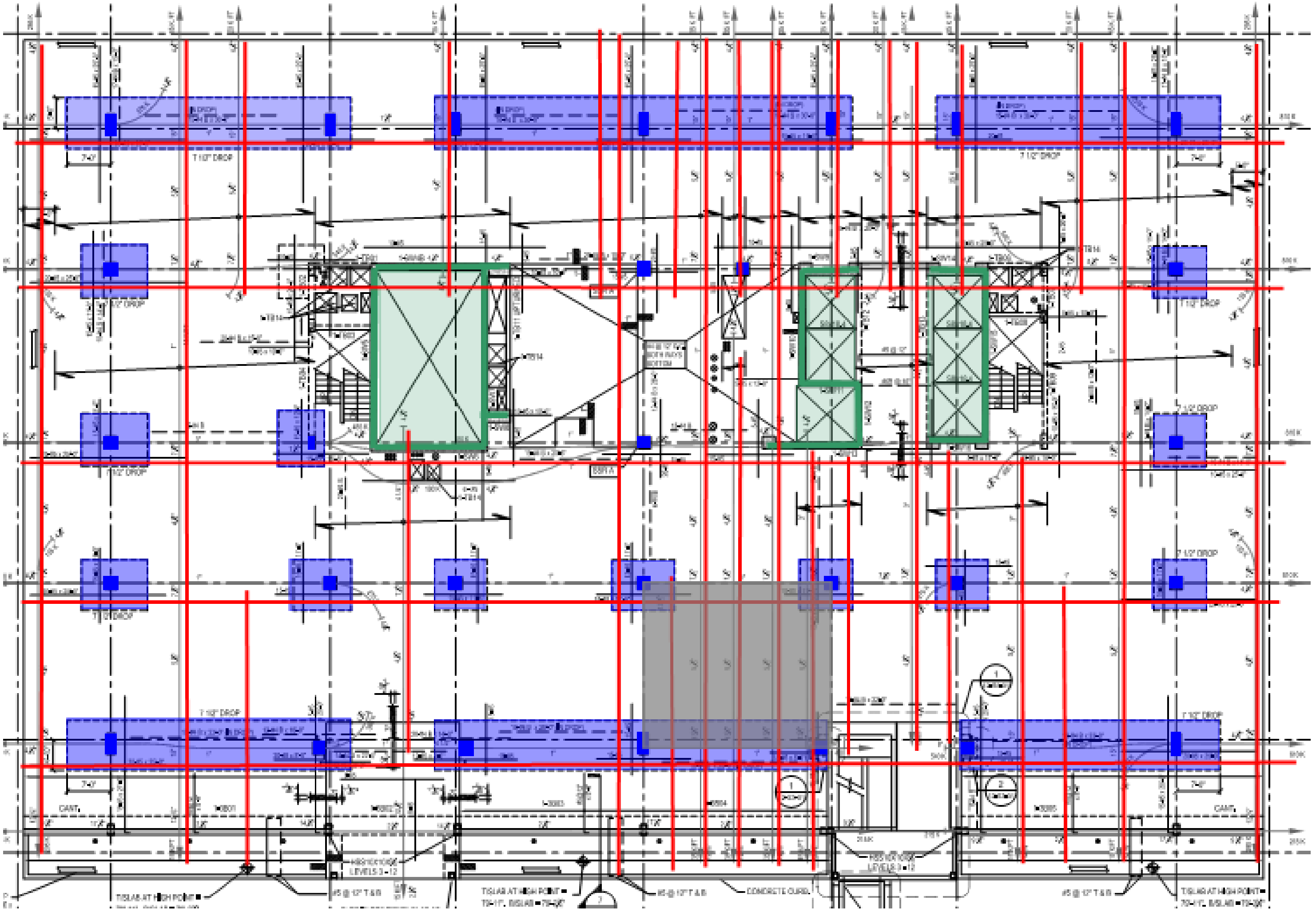
Note: Level 1, Grade level, is neglected in this table → Base shear is already known

N-S / E-W Profile of Story Shears with OTM



3.0 Existing System, gravity spot check

Many of the bays inside One City Center are not typical as far as reinforcing steel and post tensioned steel. The sizes of bays are typically 20'-30' in one direction to 20'-25' in the other direction. Thus it was decided to choose an interior bay that had a decent amount of post tensioned steel to be analyzed and to choose a bay that was within the typical dimensions. Figure 2 below shows a floor plan with the important structural details highlighted in various colors. More importantly Figure 2 depicts the bay that shall be analyzed and redesigned.



Key

-Drop Panels 🟦

-Columns 🟦

-Shear Walls 🟩

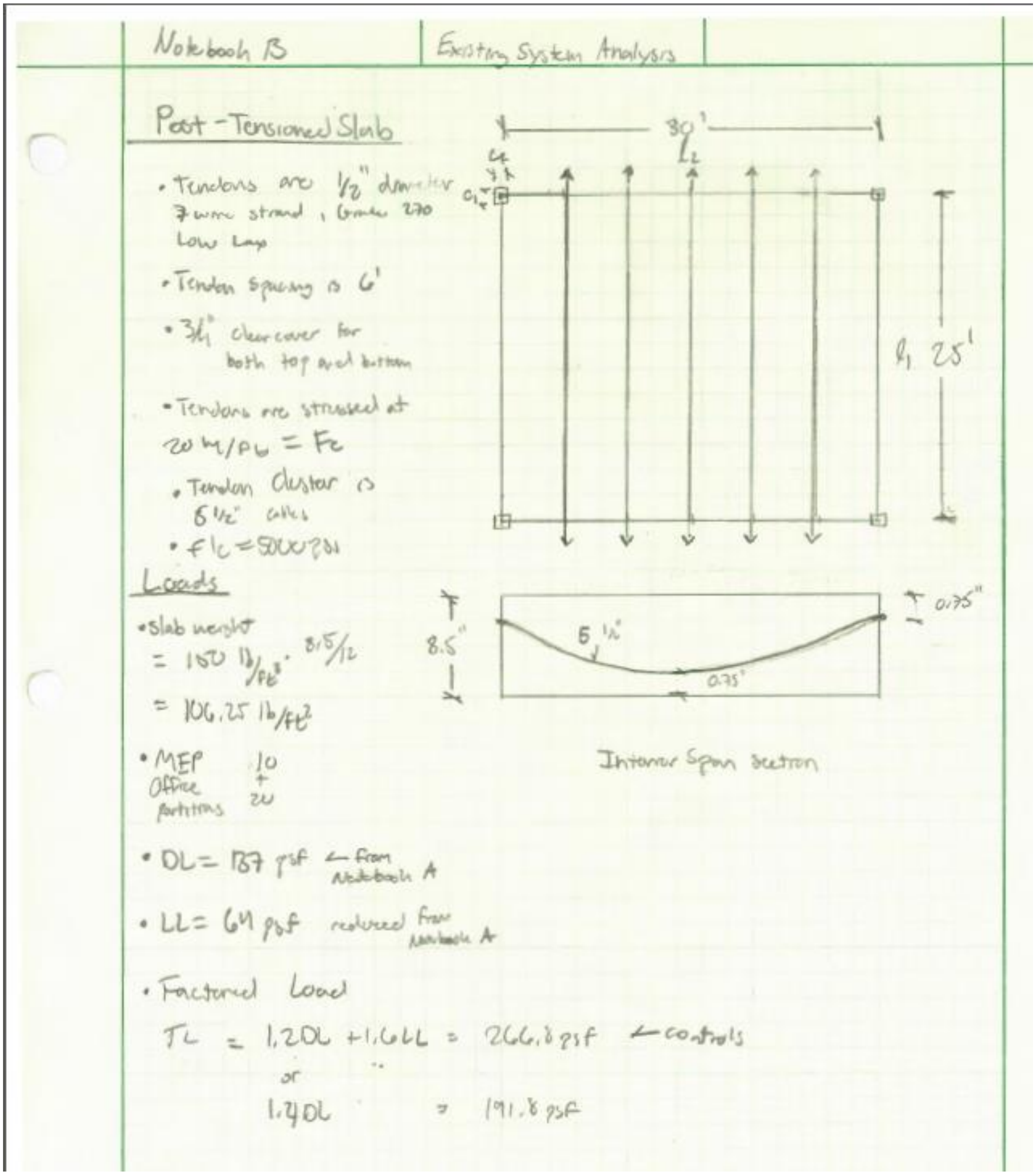
-Elevator Shaft 🟩

-Post-Tensioned Cables 🟥

Figure 2: Plan view of the structural components in a typical floor.

3.1 Post Tensioned Slab

The analysis method for the existing post tensioned slab was the equivalent frame method. This method takes the stiffness properties into account when computing the moments throughout the slab. The moments were then determined using moment distribution. Stresses caused by these moments were then checked against the minimum compressive and tensile stresses from ACI 318-14. Shear stresses along with punching shear forces were then calculated and compared to the slabs shear capacity.



Notebook B | Existing System Analysis

Post-Tensioned Slab

- Tendons are 1/2" diameter 7 wire strand, Grade 270 Low Lax
- Tendon spacing is 6'
- 3/4" clear cover for both top and bottom
- Tendons are stressed at 20 k/PL = Fe
- Tendon cluster is 6 1/2" cables
- f'c = 5000 psi

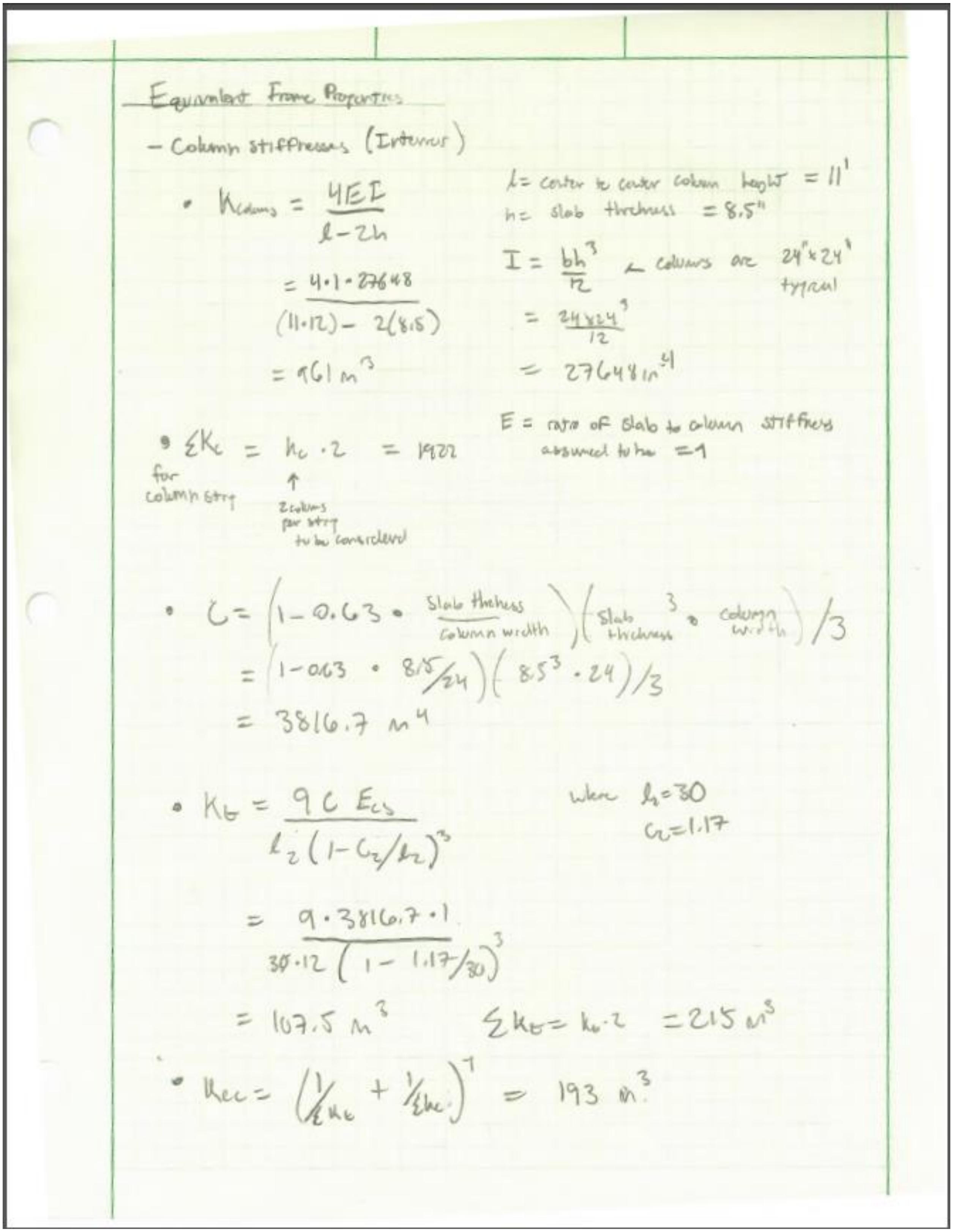
Loads

- Slab weight = 150 lb/ft³ · 8.5/12 = 106.25 lb/ft²
- MEP 10, Office partitions + 20
- DL = 137 psf ← from Notebook A
- LL = 64 psf reduced from Notebook A
- Factored Load

TL = 1.2DL + 1.6LL = 266.8 psf ← controls

or

1.4DL = 191.8 psf



Equivalent Frame Properties

- Column stiffnesses (Interior)

l = center to center column height $= 11'$

h = slab thickness $= 8.5''$

- $$K_{\text{cols}} = \frac{4EI}{l - 2h} = \frac{4 \cdot 1 \cdot 27648}{(11 \cdot 12) - 2(8.5)} = 961 \text{ in}^3$$

$$I = \frac{bh^3}{12} \leftarrow \text{columns are } 24'' \times 24'' \text{ typical}$$

$$= \frac{24 \times 24^3}{12}$$

$$= 27648 \text{ in}^4$$

E = ratio of slab to column stiffness assumed to be $= 1$

- $$\Sigma K_c = \underset{\substack{\uparrow \\ \text{2 columns per strip to be considered}}}{K_c \cdot 2} = 1922$$

for column strip

- $$C = \left(1 - 0.63 \cdot \frac{\text{slab thickness}}{\text{column width}}\right)\left(\text{slab thickness}^3 \cdot \text{column width}\right)/3$$

$$= \left(1 - 0.63 \cdot 8.5/24\right)\left(8.5^3 \cdot 24\right)/3$$

$$= 3816.7 \text{ in}^4$$

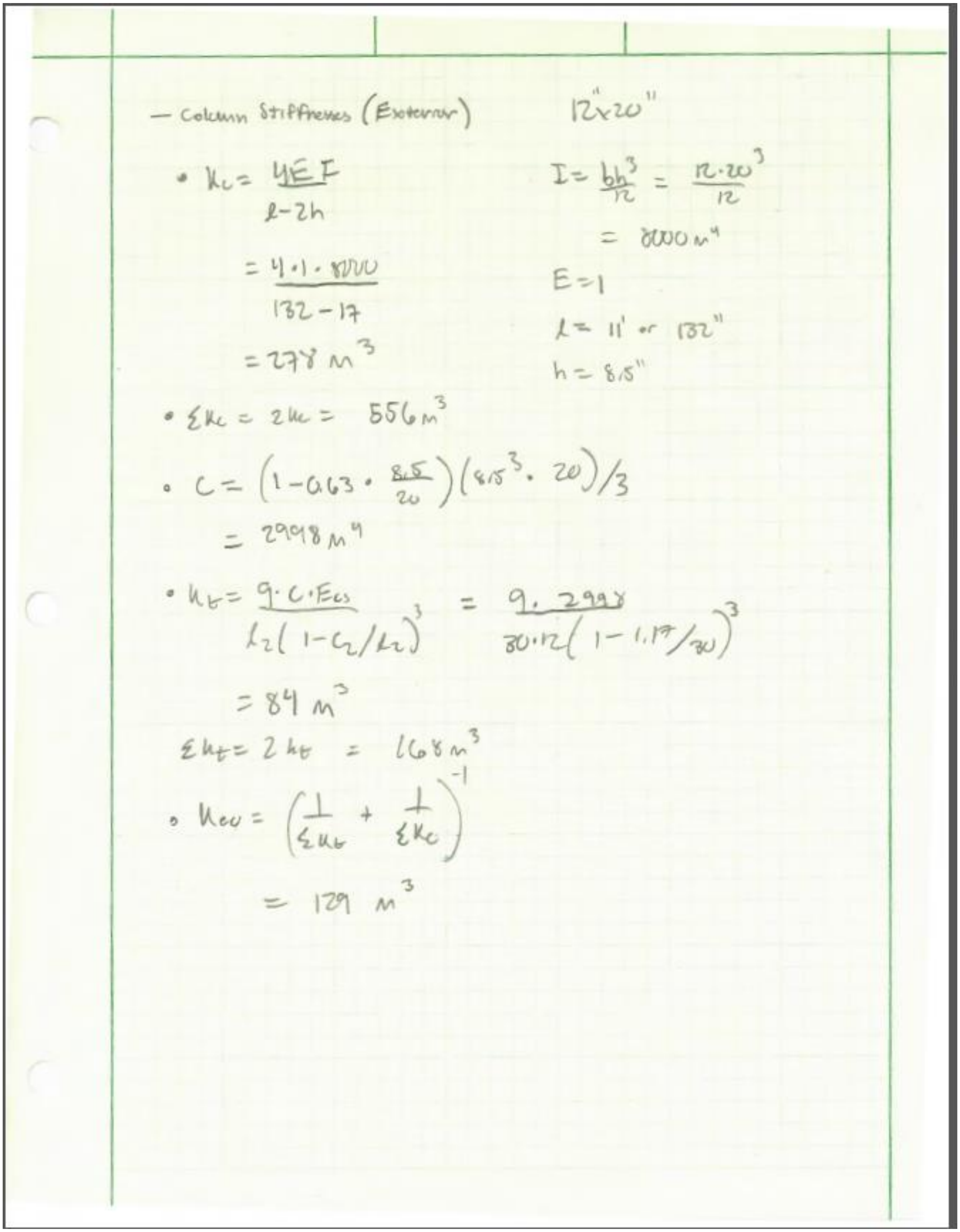
- $$K_b = \frac{9 C E_{cs}}{l_2 (1 - c_2/l_2)^3}$$

where $l_2 = 30$, $c_2 = 1.17$

$$= \frac{9 \cdot 3816.7 \cdot 1}{30 \cdot 12 \left(1 - 1.17/30\right)^3}$$

$$= 107.5 \text{ in}^3 \qquad \Sigma K_b = K_b \cdot 2 = 215 \text{ in}^3$$

- $$K_{ec} = \left(\frac{1}{\Sigma K_c} + \frac{1}{\Sigma K_b}\right)^{-1} = 193 \text{ in}^3$$



— Column Stiffnesses (Exterior) $12'' \times 20''$

- $K_c = \frac{4EI}{\ell - 2h}$

  $= \frac{4 \cdot 1 \cdot 8000}{132 - 17}$

  $= 278\ in^3$

$I = \frac{bh^3}{12} = \frac{12 \cdot 20^3}{12}$

$= 8000\ in^4$

$E = 1$

$\ell = 11'$ or $132''$

$h = 8.5''$

- $\Sigma K_c = 2K_c = 556\ in^3$

- $C = \left(1 - 0.63 \cdot \frac{8.5}{20}\right)\left(8.5^3 \cdot 20\right)/3$

  $= 2998\ in^4$

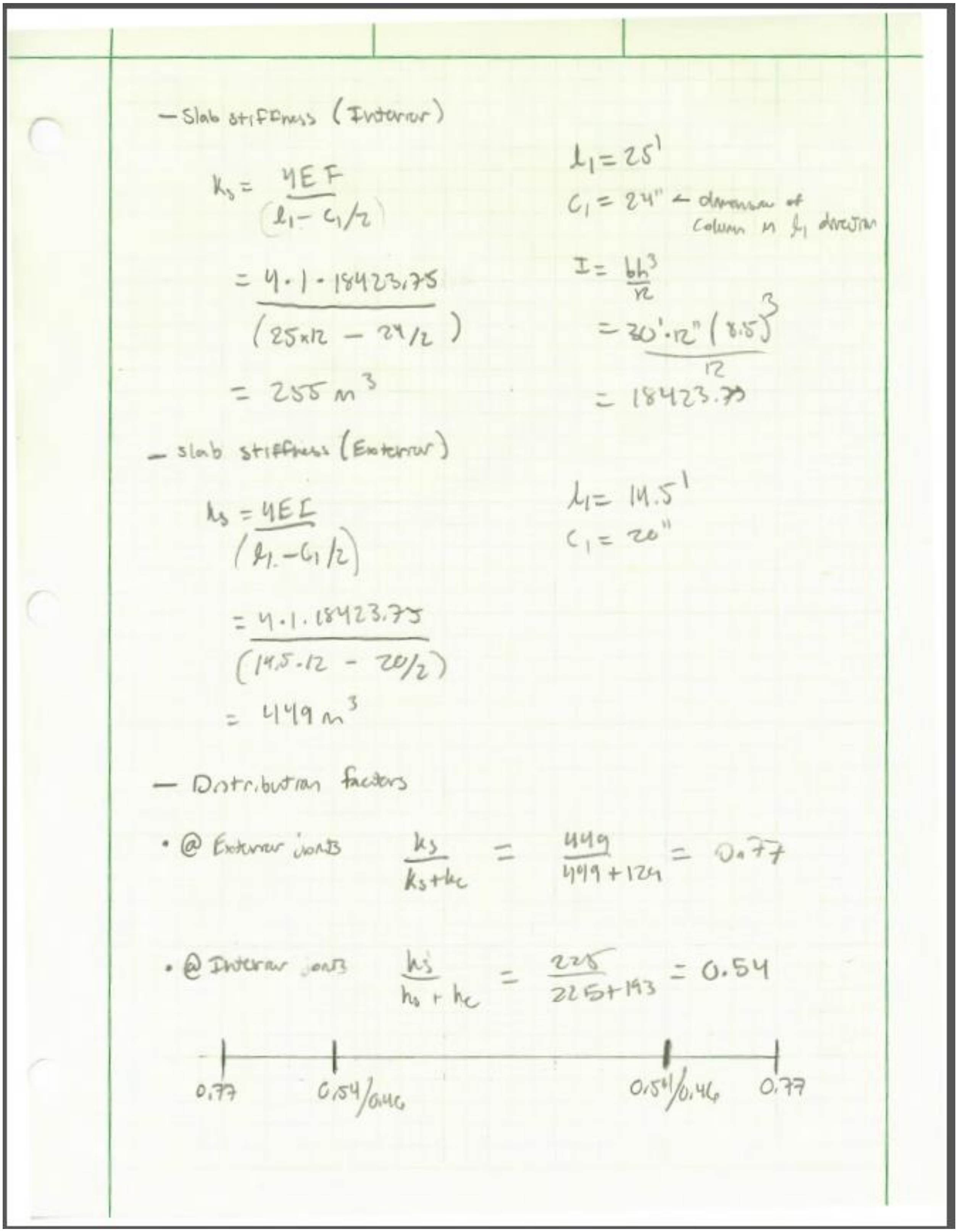
- $K_t = \frac{9 \cdot C \cdot E_{cs}}{\ell_2 \left(1 - c_2/\ell_2\right)^3} = \frac{9 \cdot 2998}{30 \cdot 12 \left(1 - 1.17/30\right)^3}$

  $= 84\ in^3$

$\Sigma K_t = 2K_t = 168\ in^3$

- $K_{ec} = \left(\frac{1}{\Sigma K_t} + \frac{1}{\Sigma K_c}\right)^{-1}$

  $= 129\ in^3$



– Slab stiffness (Interior)

$k_s = \dfrac{4EI}{(l_1 - c_1/2)}$

$= \dfrac{4 \cdot 1 \cdot 18423.75}{(25 \times 12 - 24/2)}$

$= 255 \text{ in}^3$

$l_1 = 25'$

$c_1 = 24''$ ← dimension of column in l_1 direction

$I = \dfrac{bh^3}{12}$

$= \dfrac{30' \cdot 12'' (8.5)^3}{12}$

$= 18423.75$

– slab stiffness (Exterior)

$k_s = \dfrac{4EI}{(l_1 - c_1/2)}$

$= \dfrac{4 \cdot 1 \cdot 18423.75}{(14.5 \cdot 12 - 20/2)}$

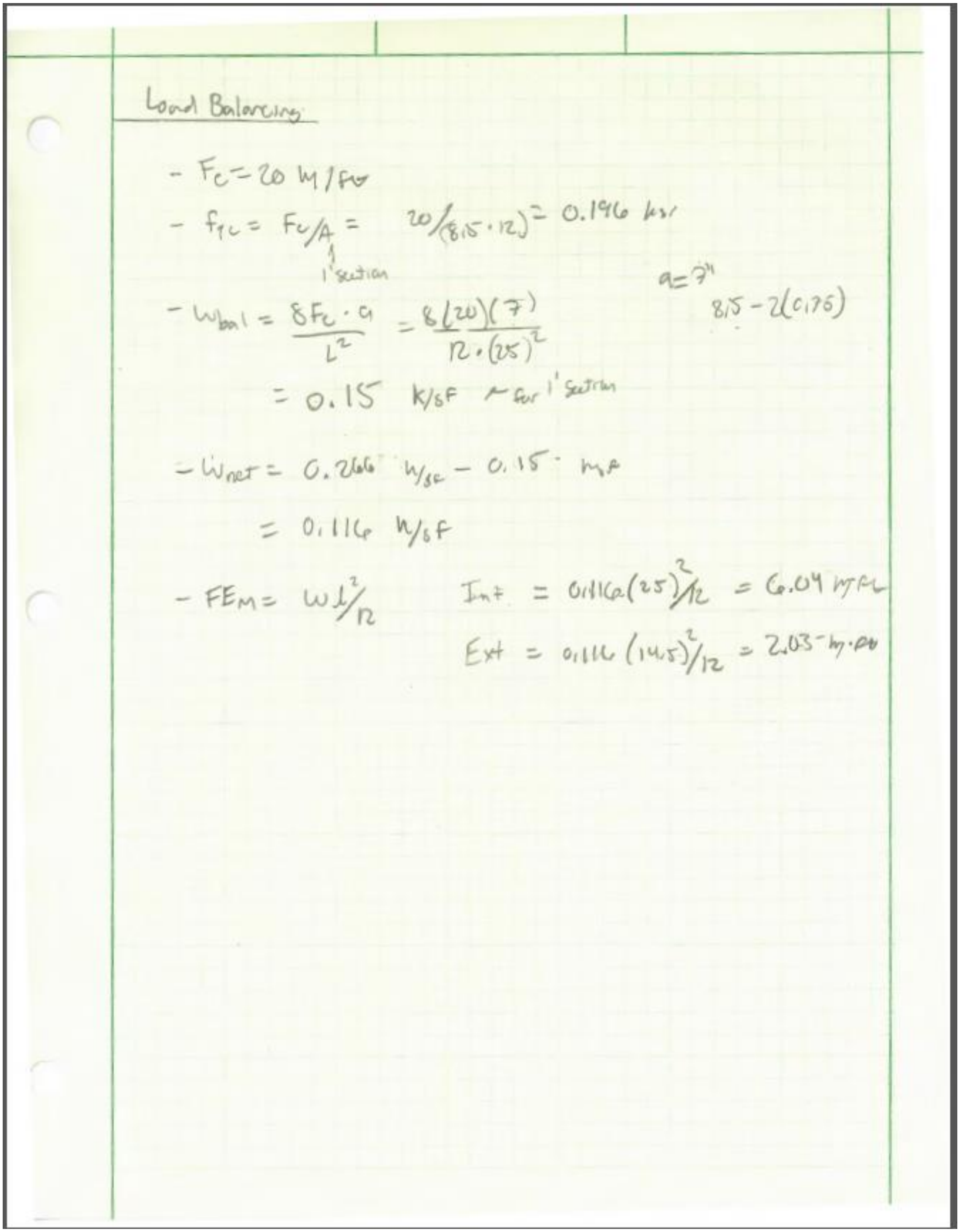
$= 449 \text{ in}^3$

$l_1 = 14.5'$

$c_1 = 20''$

– Distribution factors

- @ Exterior joints $\dfrac{k_s}{k_s + k_c} = \dfrac{449}{449 + 124} = 0.77$
- @ Interior joints $\dfrac{k_s}{k_s + k_c} = \dfrac{225}{225 + 193} = 0.54$



Load Balancing:

- $F_c = 20$ k/ft
- $f_{pc} = F_c/A = 20/(8.5 \cdot 12) = 0.196$ ksi

1' section

$a = 7"$

$8.5 - 2(0.75)$

- $w_{bal} = \frac{8 F_c \cdot a}{L^2} = \frac{8(20)(7)}{12 \cdot (25)^2}$

$= 0.15$ k/sf ← for 1' section

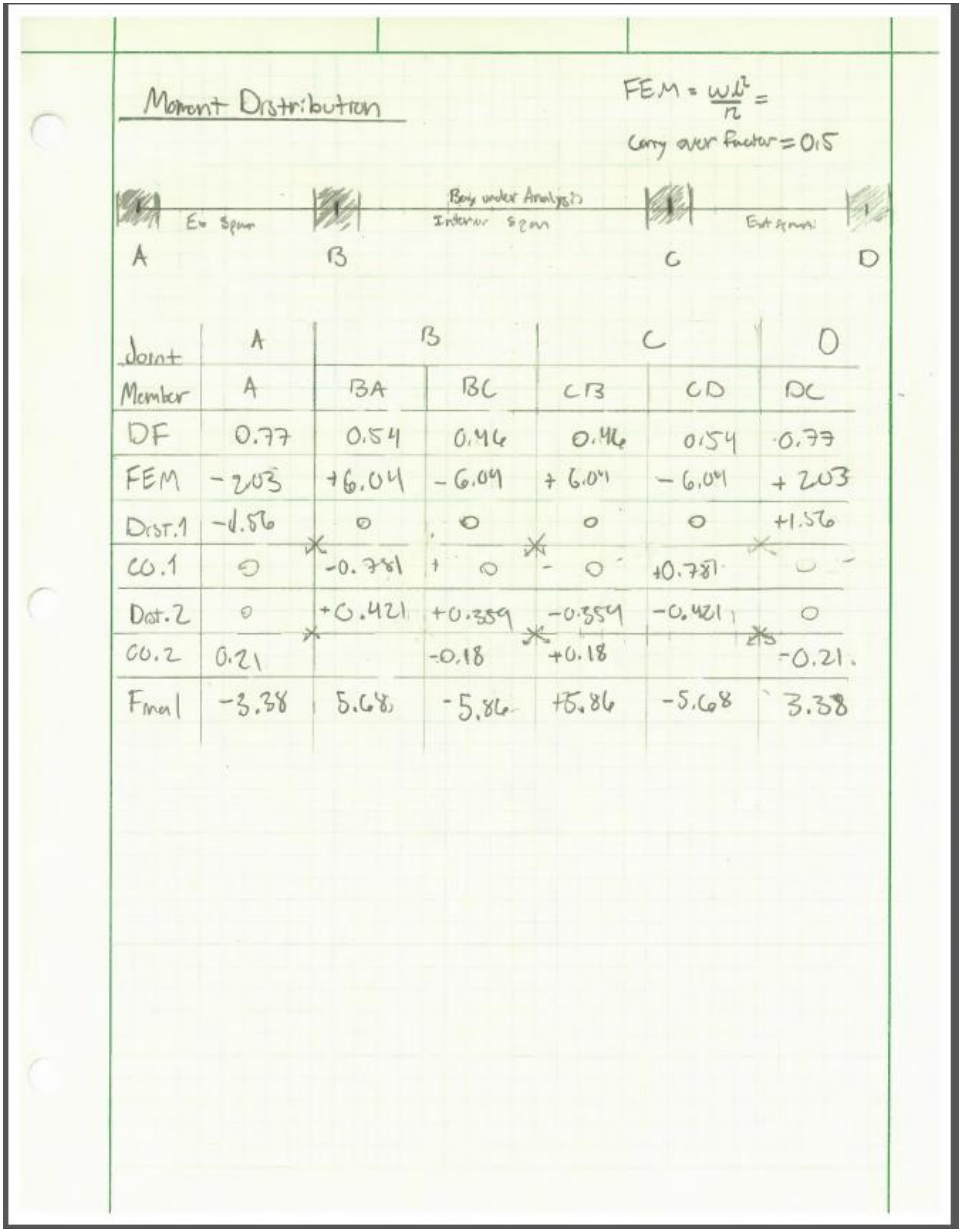
- $W_{net} = 0.266$ k/sf $- 0.15$ k/sf

$= 0.116$ k/sf

- $FE_M = wl^2/12$

Int $= 0.116(25)^2/12 = 6.04$ k-ft

Ext $= 0.116(14.5)^2/12 = 2.03$ k-ft

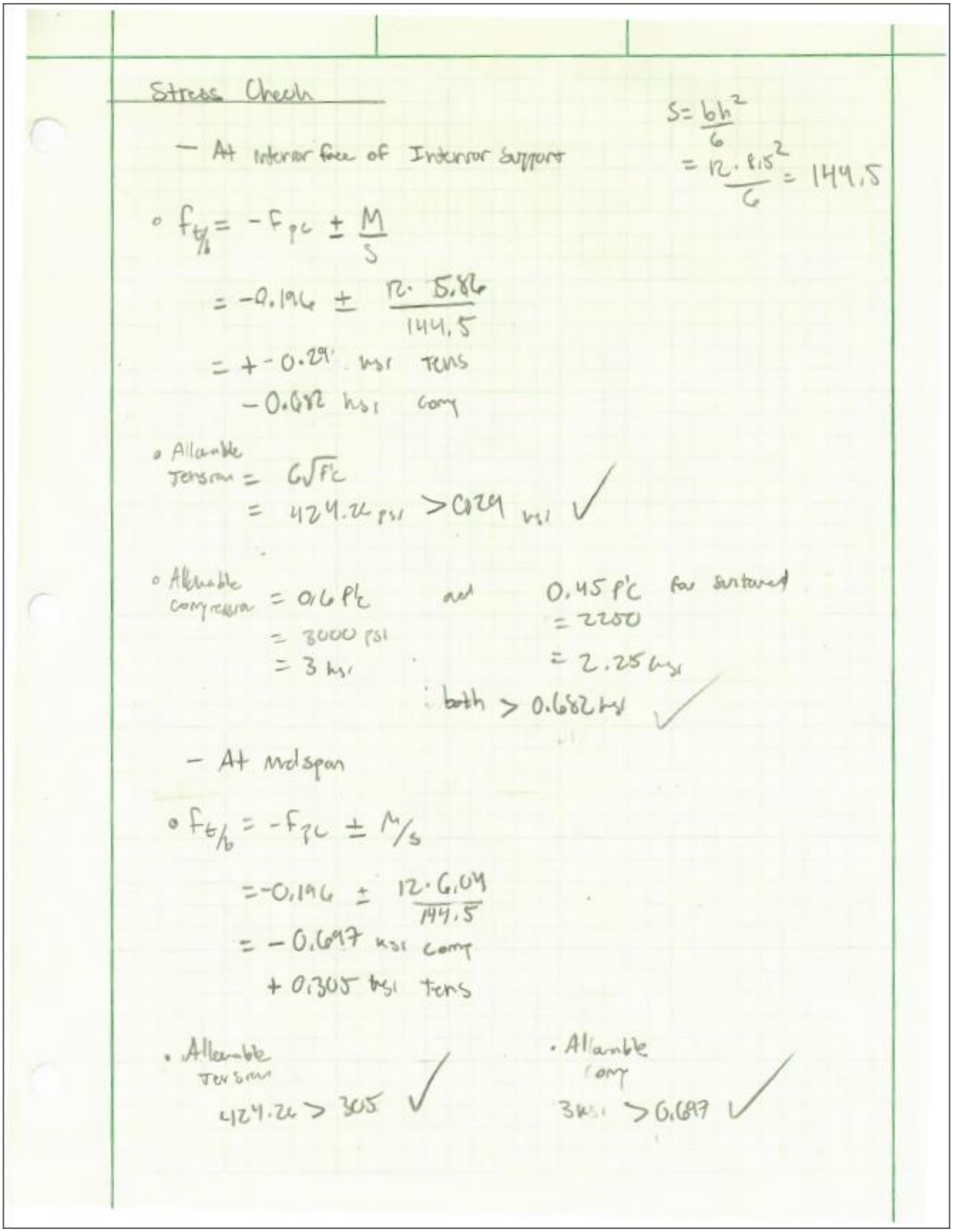


Moment Distribution

$FEM = \frac{wl^2}{12} =$

Carry over Factor = 0.5

Joint	A	B		C		D
Member	A	BA	BC	CB	CD	DC
DF	0.77	0.54	0.46	0.46	0.54	0.77
FEM	-2.03	+6.04	-6.04	+6.04	-6.04	+2.03
Dist.1	-1.56	0	0	0	0	+1.56
CO.1	0	-0.781	+ 0	- 0	+0.781	0
Dst.2	0	+0.421	+0.359	-0.359	-0.421	0
CO.2	0.21		-0.18	+0.18		-0.21
Final	-3.38	5.68	-5.86	+5.86	-5.68	3.38



Stress Check

$S = \frac{bh^2}{6}$

$= \frac{12 \cdot 8.5^2}{6} = 144.5$

— At interior face of Interior Support

• $f_{t/b} = -F_{pc} \pm \frac{M}{S}$

$= -0.196 \pm \frac{12 \cdot 5.86}{144.5}$

$= + -0.29$ ksi Tens

-0.682 ksi comp

• Allowable Tension $= 6\sqrt{f'c}$

$= 424.26$ psi > 0.29 ksi ✓

• Allowable compression $= 0.6 f'c$ and $0.45 f'c$ for sustained

$= 3000$ psi $= 2250$

$= 3$ ksi $= 2.25$ ksi

∴ both > 0.682 ksi ✓

— At midspan

• $f_{t/b} = -F_{pc} \pm M/s$

$= -0.196 \pm \frac{12 \cdot 6.04}{144.5}$

$= -0.697$ ksi comp

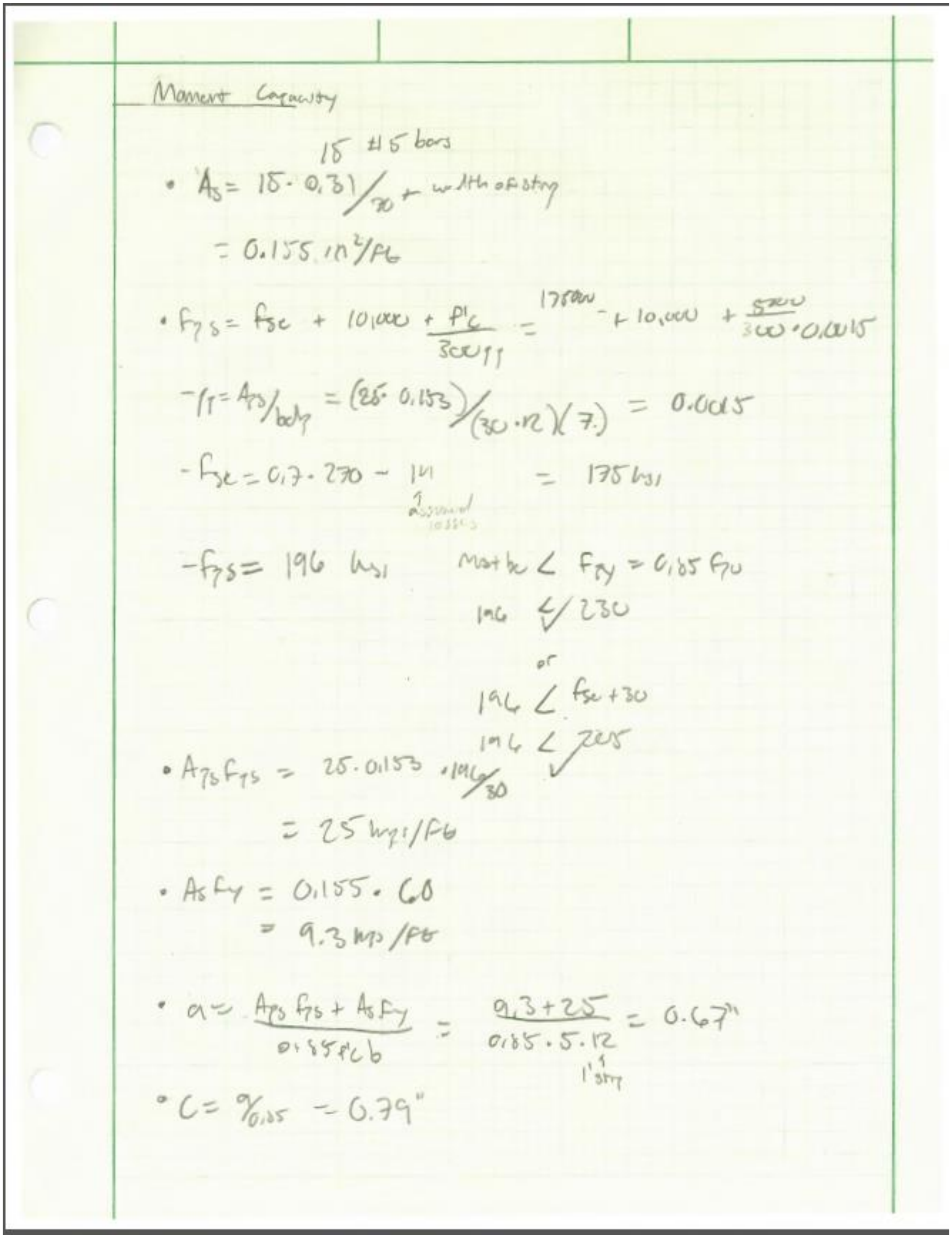
$+ 0.305$ ksi tens

• Allowable Tension

$424.26 > 305$ ✓

• Allowable comp

3ksi > 0.697 ✓



Moment Capacity

18 #5 bars

- $A_s = 15 \cdot 0.31 / 30$ ← width of strip

  $= 0.155 \; in^2/ft$

- $f_{ps} = f_{se} + 10{,}000 + \frac{f'_c}{300\rho_p} = 175000 + 10{,}000 + \frac{5000}{300 \cdot 0.0015}$

  $-\rho_p = A_{ps}/bd_p = (25 \cdot 0.153) / (30 \cdot 12)(7.) = 0.0015$

  $-f_{se} = 0.7 \cdot 270 - 14 = 175 \; ksi$

  $\uparrow$ assumed losses

  $-f_{ps} = 196 \; ksi$ Must be $< f_{py} = 0.85 f_{pu}$

  $196 \leq 230$ ✓

  or

  $196 < f_{se} + 30$

  $196 < 205$ ✓

- $A_{ps} f_{ps} = 25 \cdot 0.153 \cdot 196 / 30$

  $= 25 \; kips/ft$

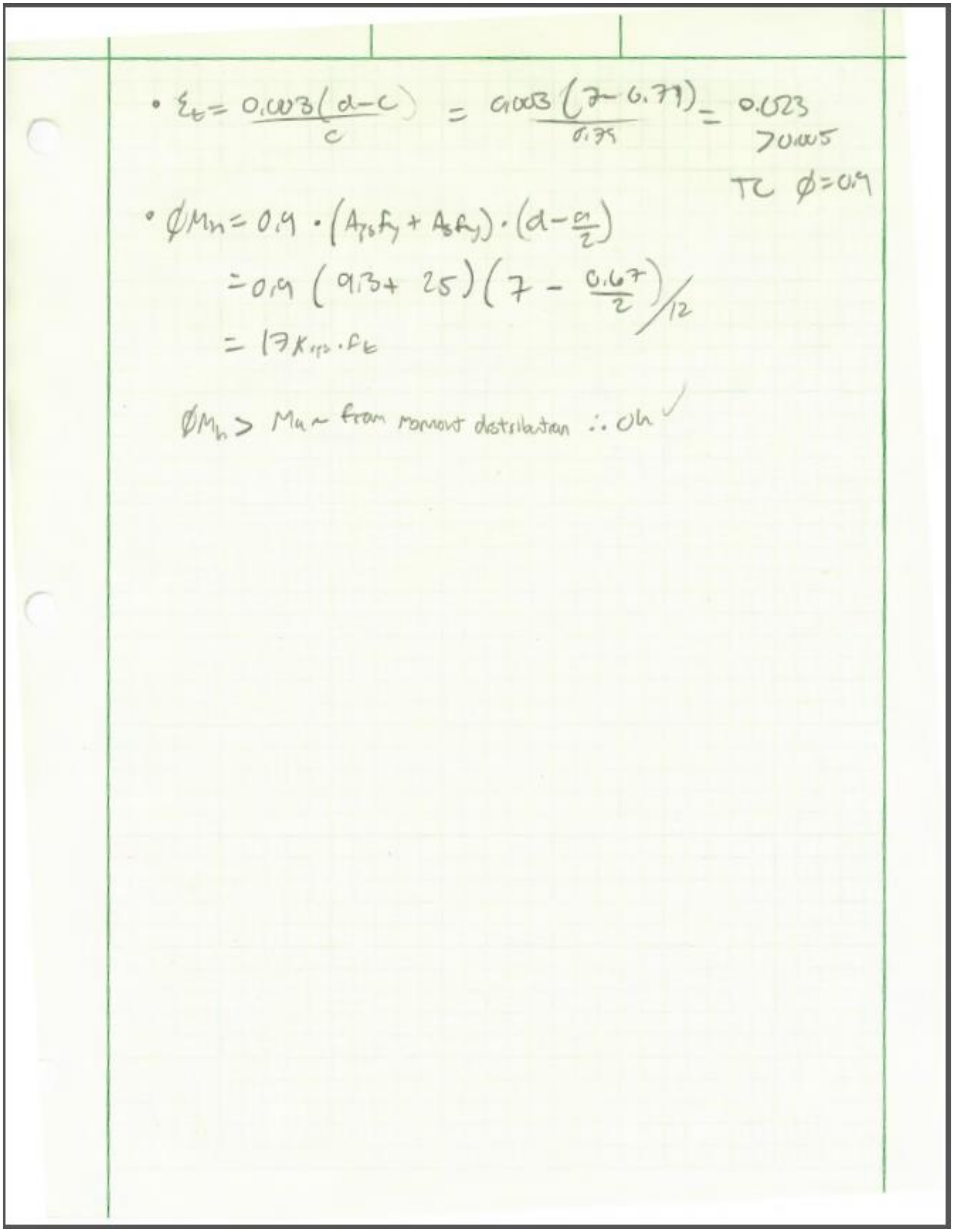
- $A_s f_y = 0.155 \cdot 60$

  $= 9.3 \; kips/ft$

- $a = \frac{A_{ps} f_{ps} + A_s f_y}{0.85 f'_c b} = \frac{9.3 + 25}{0.85 \cdot 5 \cdot 12} = 0.67"$

  $\uparrow$ 1' strip

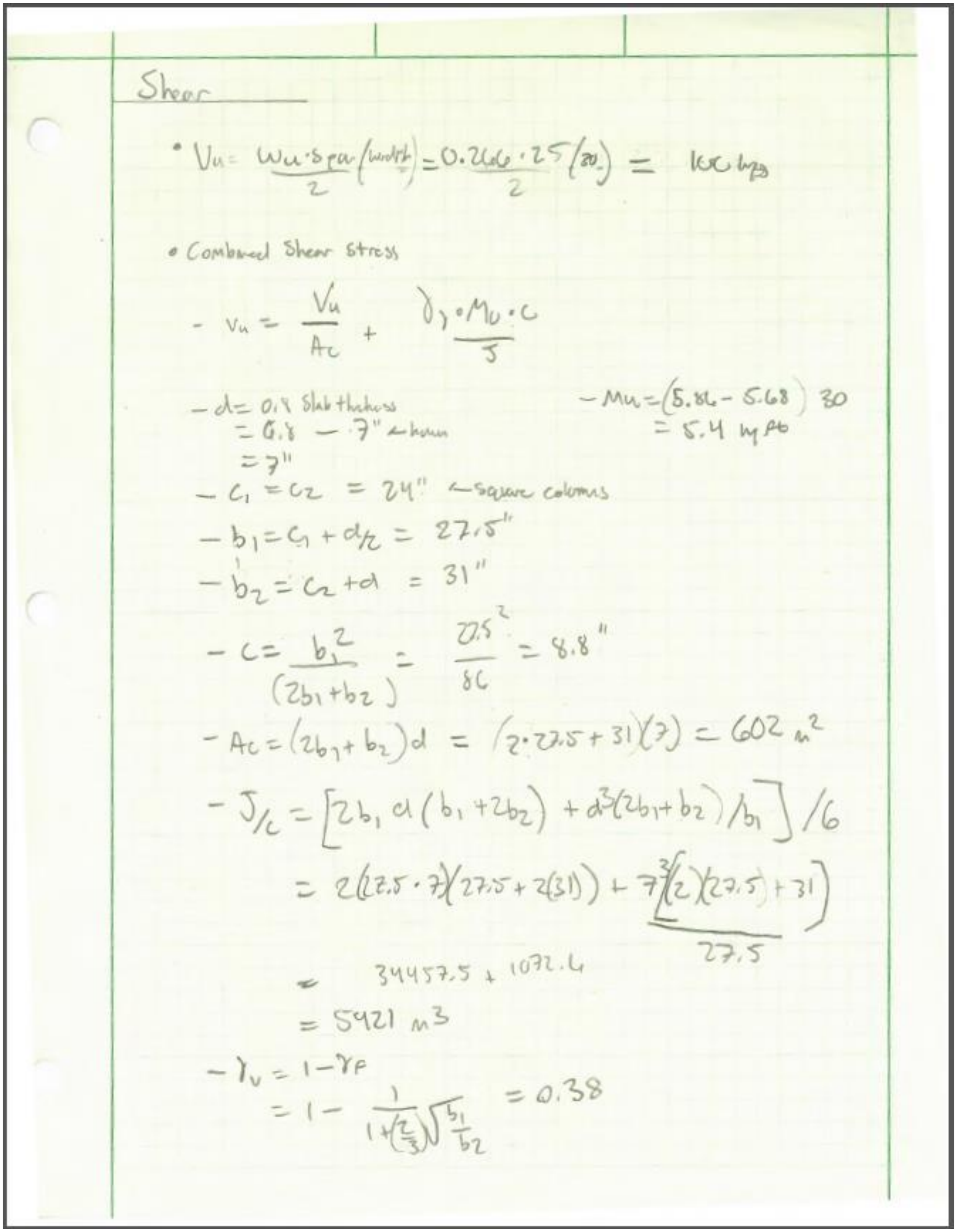
- $c = a/0.85 = 0.79"$



- $\varepsilon_t = \frac{0.003(d-c)}{c} = \frac{0.003(7-0.79)}{0.79} = 0.023$
  > 0.005
  TC $\phi = 0.9$

- $\phi M_n = 0.9 \cdot (A_{ps}f_y + A_s f_y) \cdot (d - \frac{a}{2})$
  $= 0.9\,(9.3 + 25)(7 - \frac{0.67}{2}) / 12$
  $= 17\,\text{kips} \cdot \text{ft}$

$\phi M_n > M_u$ ~ from moment distribution ∴ ok ✓



Shear

- $V_u = \frac{w_u \cdot span}{2}(width) = \frac{0.266 \cdot 25}{2}(30) = 100$ kips

- Combined Shear stress

  - $v_u = \frac{V_u}{A_c} + \frac{\gamma_v \cdot M_u \cdot c}{J}$

  - $d = 0.8$ slab thickness
    $= 6.8 - 7"$ ← hmm
    $= 7"$

  - $M_u = (5.86 - 5.68)\ 30$
    $= 5.4$ k ft

  - $c_1 = c_2 = 24"$ ← square columns

  - $b_1 = c_1 + d/2 = 27.5"$

  - $b_2 = c_2 + d = 31"$

  - $c = \frac{b_1^2}{(2b_1 + b_2)} = \frac{27.5^2}{86} = 8.8"$

  - $A_c = (2b_1 + b_2)d = (2 \cdot 27.5 + 31)(7) = 602\ in^2$

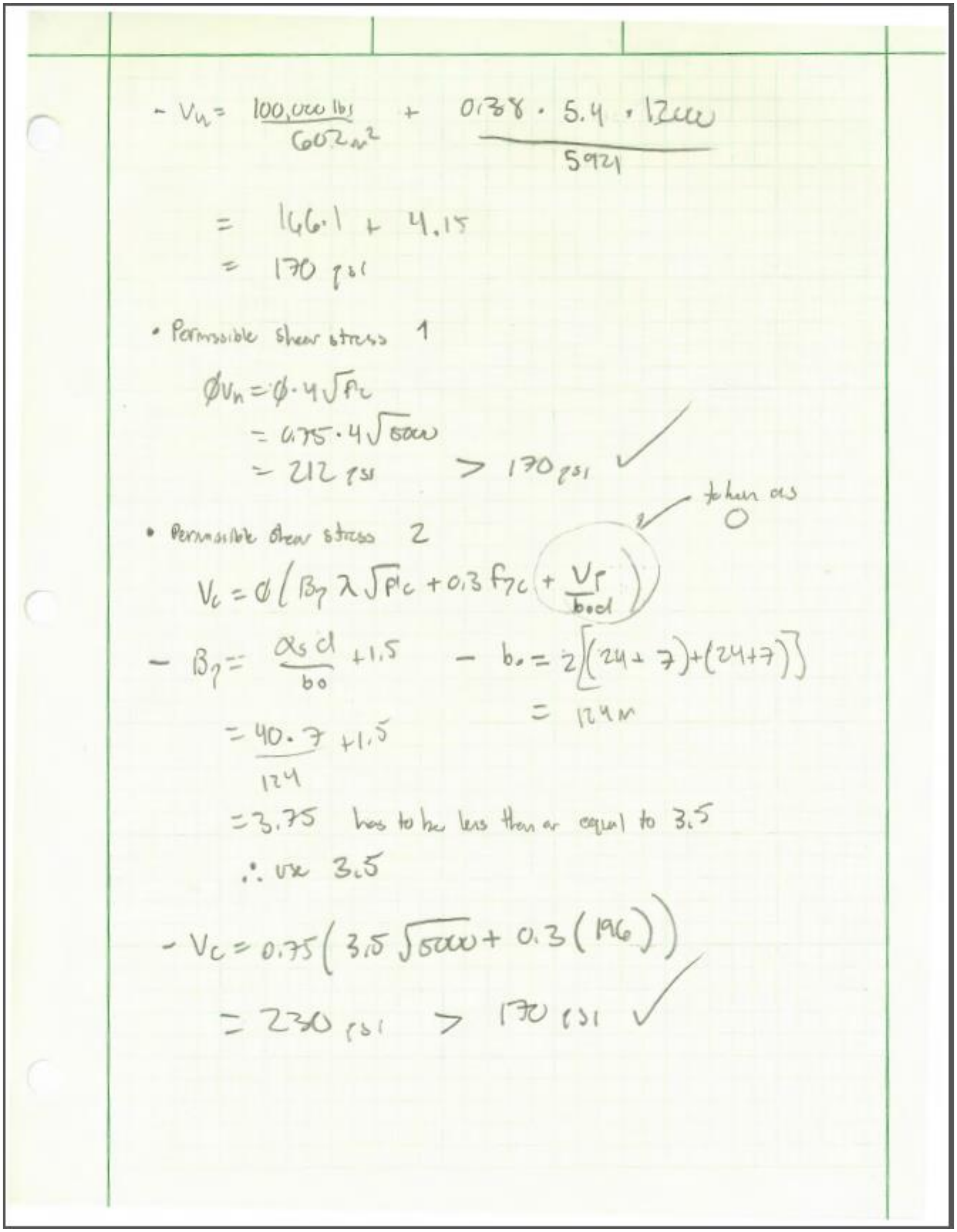
  - $J/c = \left[2b_1 d(b_1 + 2b_2) + d^3(2b_1 + b_2)/b_1\right]/6$

    $= 2(27.5 \cdot 7)(27.5 + 2(31)) + \frac{7^3[(2)(27.5) + 31)}{27.5}$

    $= 34457.5 + 1072.4$

    $= 5421\ in^3$

  - $\gamma_v = 1 - \gamma_f$
    $= 1 - \frac{1}{1 + \left(\frac{2}{3}\right)\sqrt{\frac{b_1}{b_2}}} = 0.38$



- $V_u = \frac{100{,}000 \text{ lbs}}{602 \text{ in}^2} + \frac{0.38 \cdot 5.4 \cdot 12000}{5921}$

$= 166.1 + 4.15$

$= 170$ psi

- Permissible shear stress 1

$\phi V_n = \phi \cdot 4\sqrt{f'_c}$

$= 0.75 \cdot 4\sqrt{5000}$

$= 212$ psi > 170 psi ✓

- Permissible shear stress 2

taken as 0

$V_c = \phi\left(\beta_p \lambda \sqrt{f'_c} + 0.3 f_{pc} \boxed{+ \frac{V_p}{b_o d}}\right)$

- $\beta_p = \frac{\alpha_s d}{b_o} + 1.5$ — $b_o = 2[(24+7)+(24+7)]$

$= 124$ in

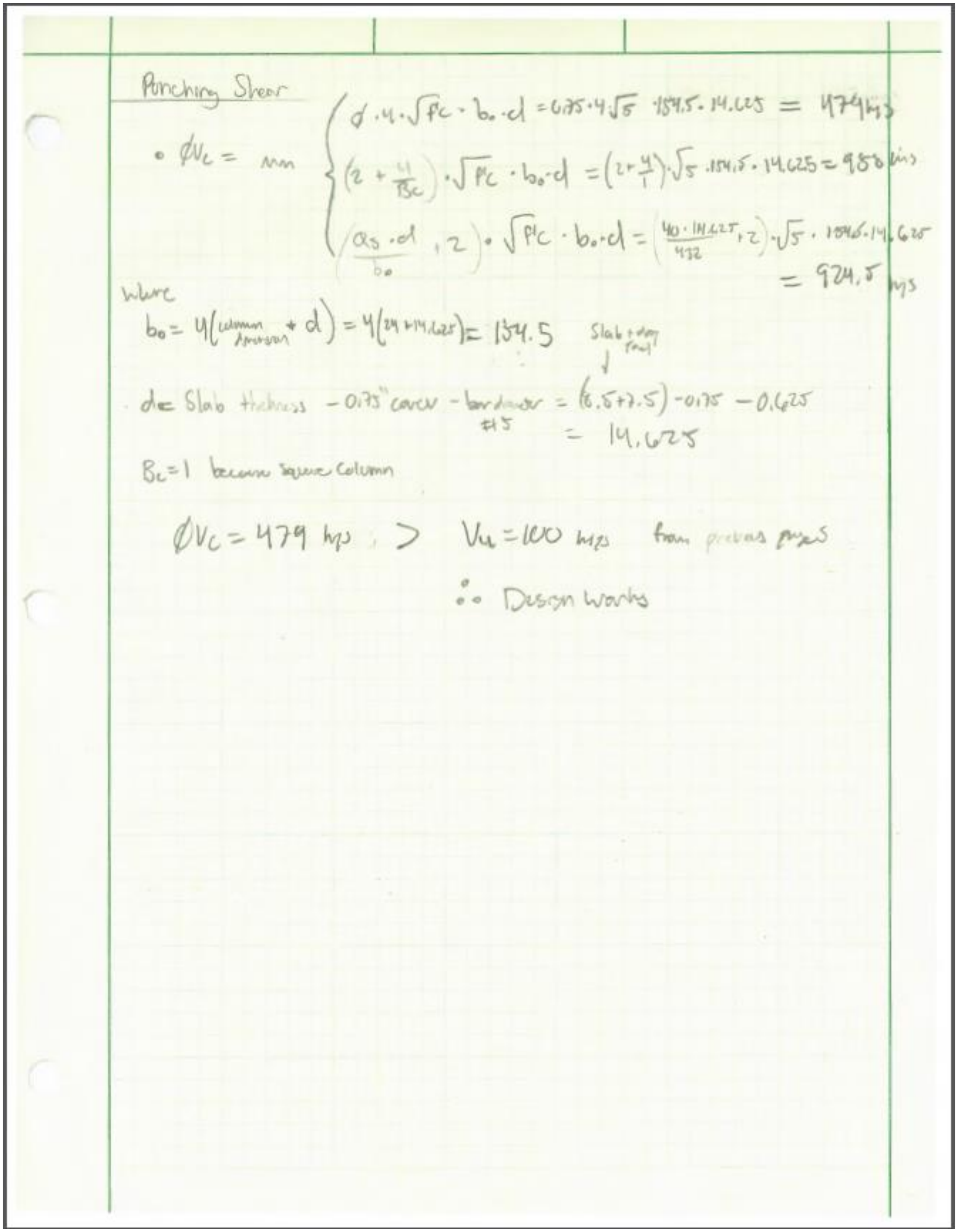
$= \frac{40 \cdot 7}{124} + 1.5$

$= 3.75$ has to be less than or equal to 3.5

$\therefore$ use 3.5

- $V_c = 0.75\left(3.5\sqrt{5000} + 0.3(196)\right)$

$= 230$ psi > 170 psi ✓



Punching Shear

• $\phi V_c = \min \begin{cases} \phi \cdot 4 \cdot \sqrt{f'c} \cdot b_o \cdot d = 0.75 \cdot 4 \sqrt{5} \cdot 154.5 \cdot 14.625 = 479 \text{ kips} \\ \left(2 + \frac{4}{\beta c}\right) \cdot \sqrt{f'c} \cdot b_o \cdot d = \left(2 + \frac{4}{1}\right) \cdot \sqrt{5} \cdot 154.5 \cdot 14.625 = 958 \text{ kips} \\ \left(\frac{\alpha_s \cdot d}{b_o}, 2\right) \cdot \sqrt{f'c} \cdot b_o \cdot d = \left(\frac{40 \cdot 14.625}{432} + 2\right) \cdot \sqrt{5} \cdot 154.5 \cdot 14.625 = 924.5 \text{ kips} \end{cases}$

where

$b_o = 4(\text{column dimension} + d) = 4(24 + 14.625) = 154.5$

Slab + drop panel ↓

$d = \text{Slab thickness} - 0.75'' \text{ cover} - \underset{\#5}{\text{bar diameter}} = (6.5 + 7.5) - 0.75 - 0.625 = 14.625$

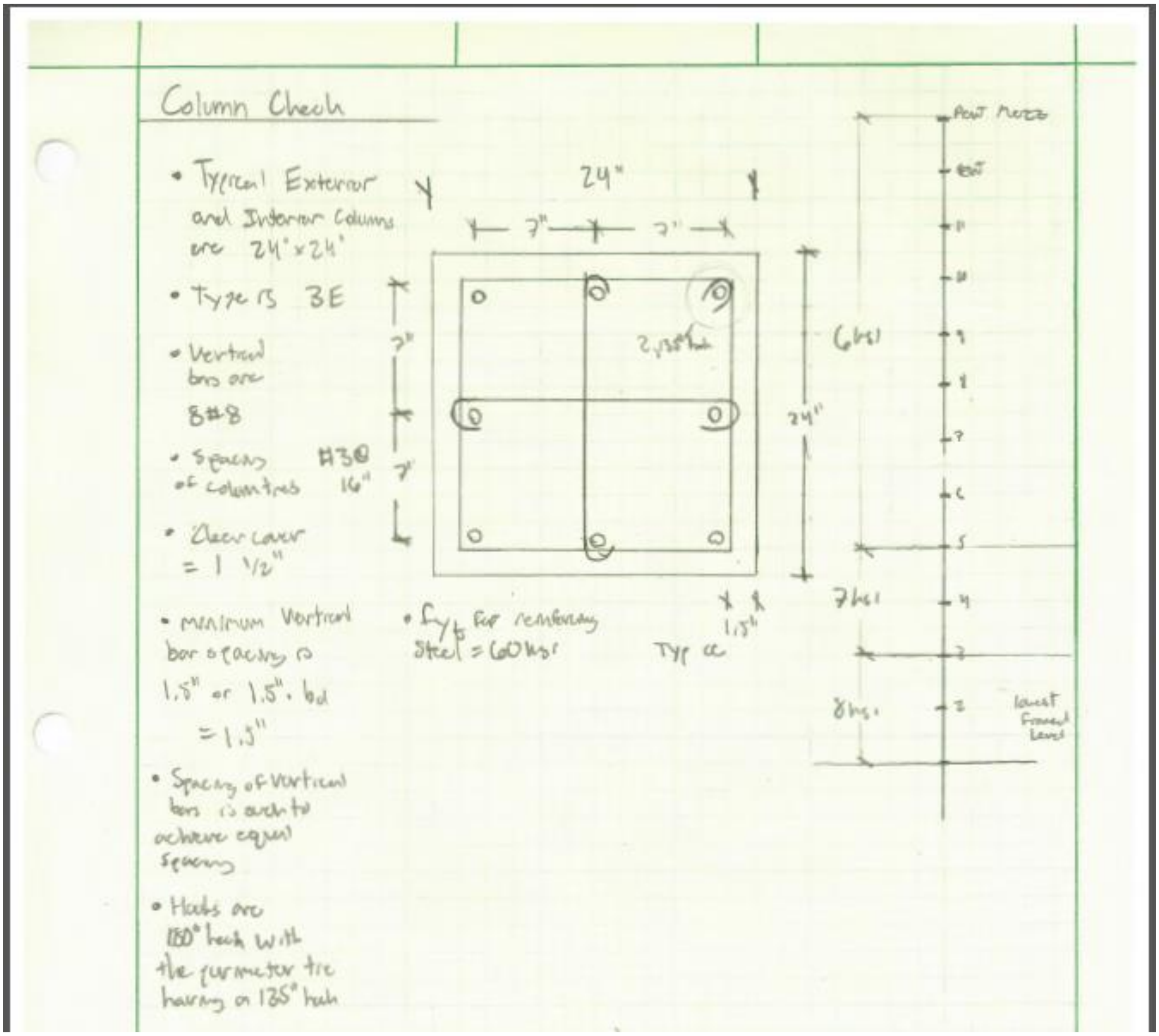
$B_c = 1$ because square column

$\phi V_c = 479 \text{ kips} > V_u = 100 \text{ kips}$ from previous pages

∴ Design Works

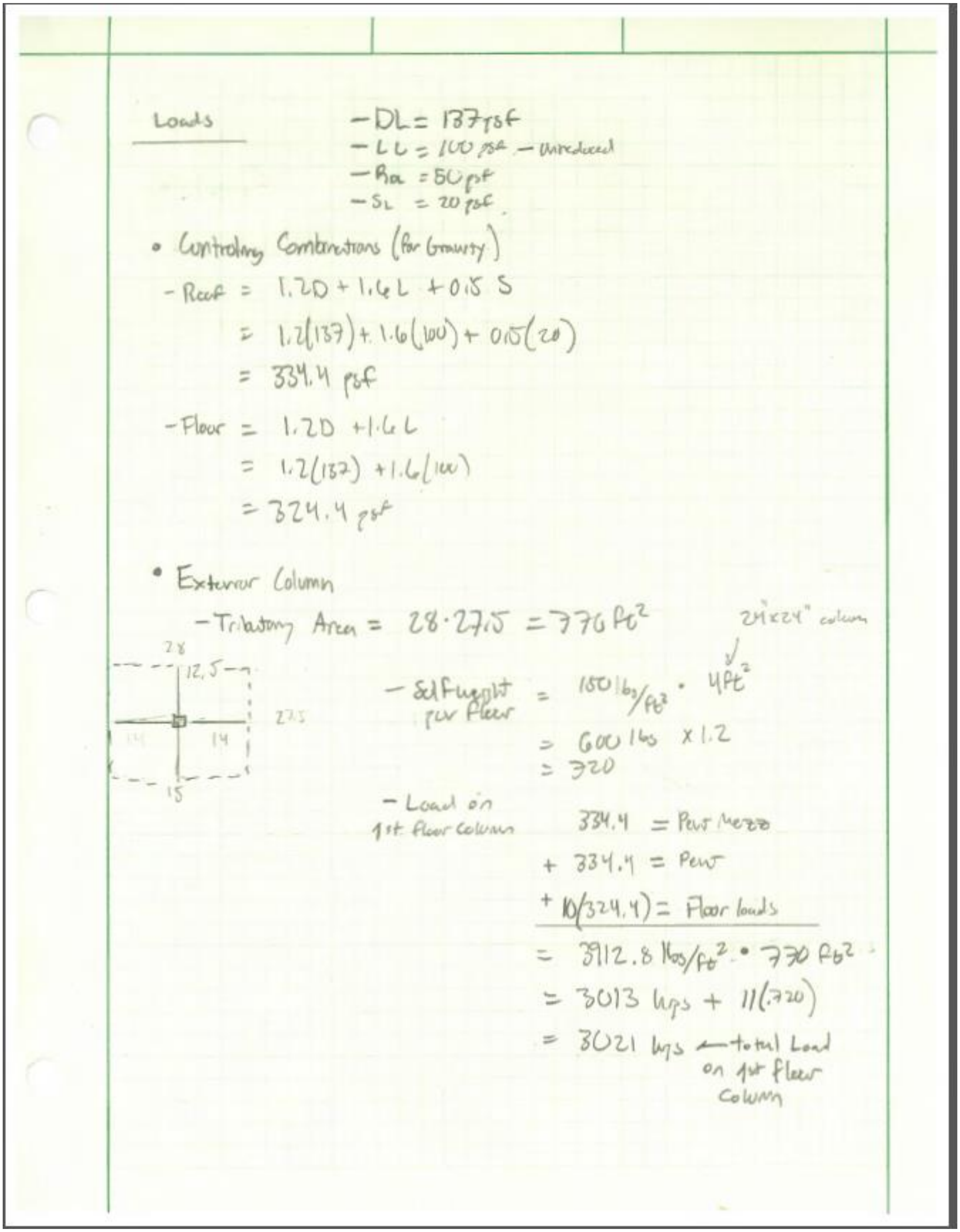
3.2 Exterior and Interior Columns

Columns were checked based on their axial loading capacity. Typical columns were 24” x 24” with 8 #8 bars as detailed below. The columns that were analyzed were below the lowest framed level and thus saw the most axial load. It is important to note that the columns axial capacity was severely controlled by its strength reduction factor which was determined from ACI 318-14. If this factor was slightly smaller the columns would not have passed.



Column Check

- Typical Exterior and Interior Columns are 24"x24"
- Type is 3E
- Vertical bars are 8#8
- Spacing of column ties #3@ 16"
- Clear cover = 1 1/2"
- minimum Vertical bar spacing is 1.5" or 1.5"· bd = 1.5"
- Spacing of vertical bars is such to achieve equal spacing
- Hooks are 180° hook with the perimeter tie having a 135° hook
- fy for reinforcing steel = 60ksi



Loads

- DL = 137 psf
- LL = 100 psf – unreduced
- Ra = 50 psf
- S_L = 20 psf

- Controlling Combinations (for Gravity)

- Roof = 1.2D + 1.6L + 0.5 S
  = 1.2(137) + 1.6(100) + 0.5(20)
  = 334.4 psf

- Floor = 1.2D + 1.6L
  = 1.2(137) + 1.6(100)
  = 324.4 psf

- Exterior Column

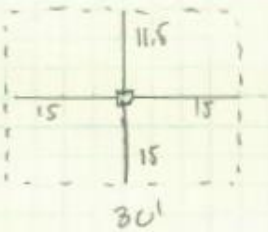
- Tributary Area = 28 · 27.5 = 770 ft^2

24x24" column

- Selfweight per floor = 150 lbs/ft^3 · 4 ft^2
  = 600 lbs x 1.2
  = 720

- Load on 1st floor column
  334.4 = Pent Mezz
  + 334.4 = Pent
  + 10(324.4) = Floor loads
  = 3912.8 lbs/ft^2 • 770 ft^2
  = 3013 kips + 11(.720)
  = 3021 kips ← total Load on 1st floor column

- Interior column



— Tributary Area = $30 \cdot 26.5$
$\geq 795 \text{ ft}^2$

— Load on 1st floor column $= 3912.8 \text{ lb/ft}^2 \cdot 795 \text{ ft}^2$

$= 3111 \text{ kips} + 11(.720)$

$= 3119 \text{ kips}$ ← total Load on 1st floor Column

Exterior Column Analysis

- Slenderness effects as per ACI 6.2.5

— $\frac{k l_u}{r} \leq 22$ where $r = \sqrt{I/A} = \sqrt{\frac{bh^3/12}{bh}} = 6.9$

$k = 1.0$

$l_u = 11'$

$\frac{1 \cdot 11 \cdot 12}{6.9} \leq 22$

$19 \leq 22$ ✓ ∴ Slenderness effects can be neglected

— $\frac{k l_u}{r} \leq 34 + 12 \frac{M_1}{M_2}$ where M_1 is assumed to be the same as M_2

$19 \leq 46$ ✓

Note: Ignoring the moments caused by the load and analyze the columns based on axial capacity

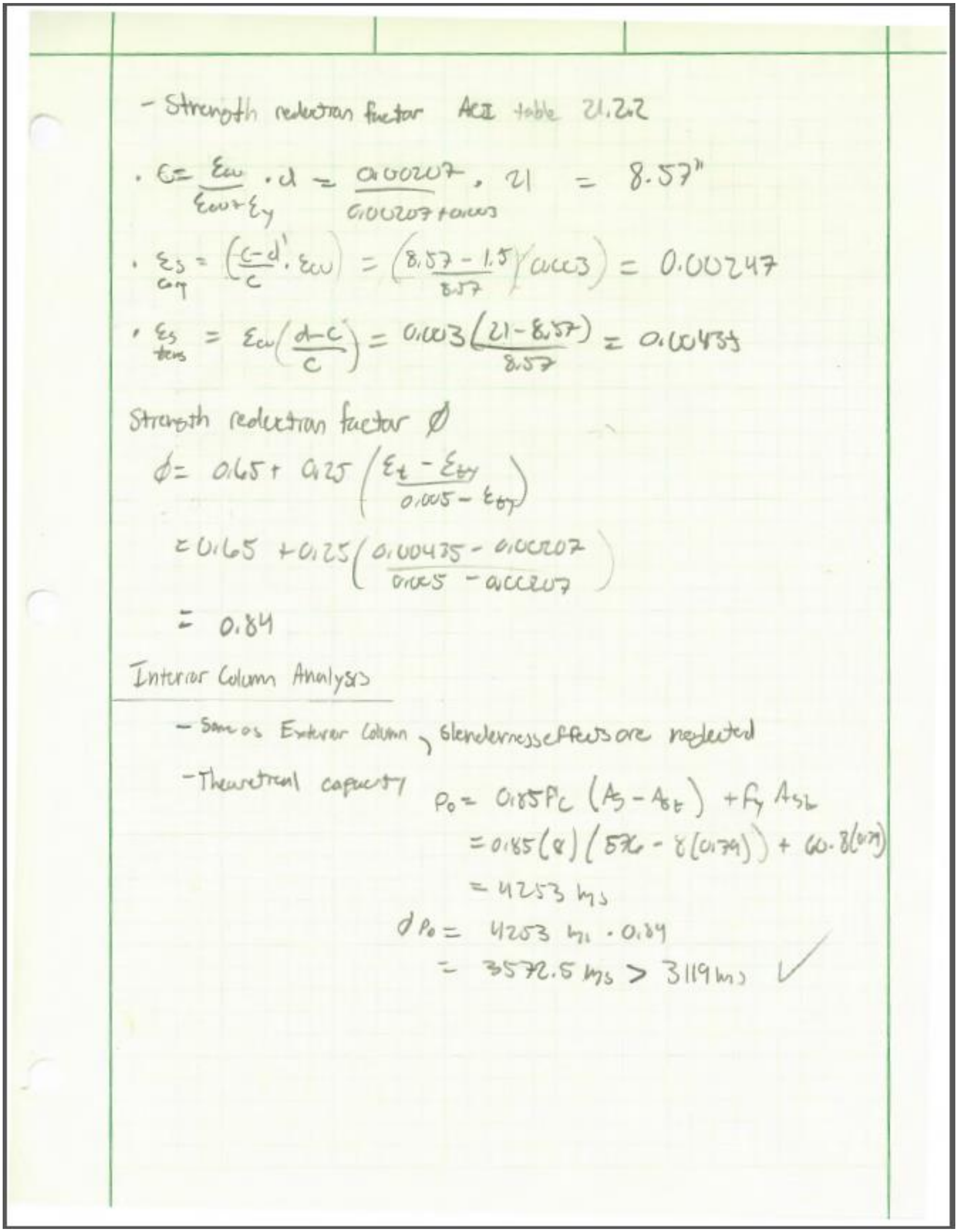
— Theoretical Capacity $P_o = 0.85 f'_c (A_g - A_{st}) + f_y A_{st}$

$= 0.85(8)(576 - 8(0.79)) + 60 \cdot 8(0.79)$

$= 4253 \text{ kips} >$ Actual Load

$\phi P_o = 4253 \cdot 0.84$

$= 3572.5 \text{ kips} > 3021 \text{ kips}$ ✓



- Strength reduction factor ACI table 21.2.2

$\cdot\ c = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_y} \cdot d = \frac{0.00207}{0.00207 + 0.003} \cdot 21 = 8.57''$

$\cdot\ \varepsilon_{s\,comp} = \left(\frac{c - d'}{c} \cdot \varepsilon_{cu}\right) = \left(\frac{8.57 - 1.5}{8.57}\right)(0.003) = 0.00247$

$\cdot\ \varepsilon_{s\,tens} = \varepsilon_{cu}\left(\frac{d - c}{c}\right) = 0.003\left(\frac{21 - 8.57}{8.57}\right) = 0.00435$

Strength reduction factor ϕ

$\phi = 0.65 + 0.25\left(\frac{\varepsilon_t - \varepsilon_{ty}}{0.005 - \varepsilon_{ty}}\right)$

$= 0.65 + 0.25\left(\frac{0.00435 - 0.00207}{0.005 - 0.00207}\right)$

$= 0.84$

Interior Column Analysis

- Same as Exterior Column, slenderness effects are neglected
- Theoretical capacity $P_o = 0.85 f'_c\,(A_g - A_{st}) + f_y A_{st}$

$= 0.85(8)\left(576 - 8(0.79)\right) + 60 \cdot 8(0.79)$

$= 4253$ kips

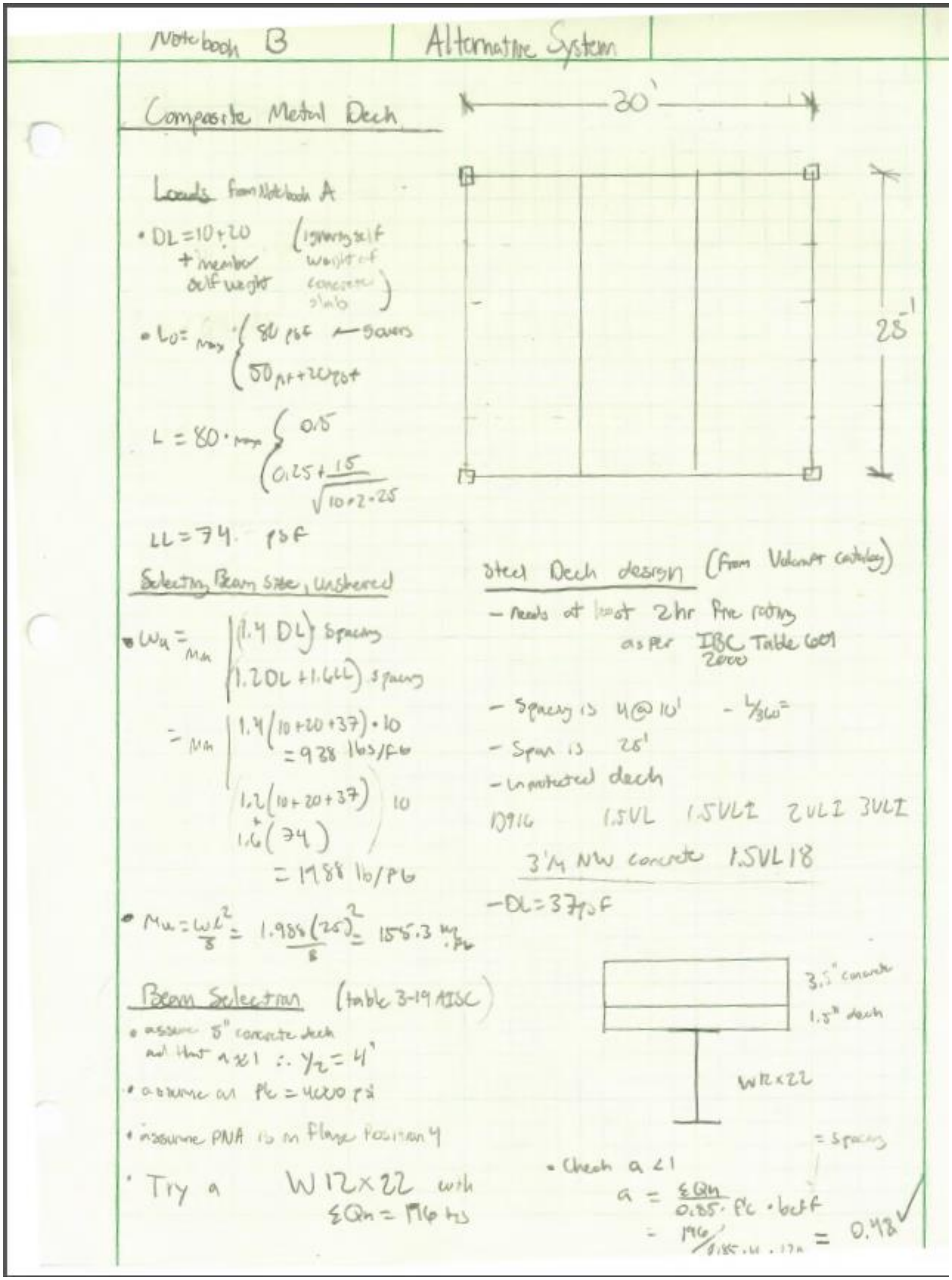
$\phi P_o = 4253$ kips $\cdot\ 0.84$

$= 3572.5$ kips > 3119 kips ✓

4. Alternative Systems

4.1 Composite Metal Deck

A composite system was chosen over a non-composite for its higher level of strength and performance. The metal decking was chosen from the Vulcraft catalog. This deck is then supported by steel wide flange members which were checked against moment capacity for unshored strength, live load and wet concrete deflections.



Notebook B | Alternative System

Composite Metal Deck

Loads from Notebook A

- DL = 10 + 20 + member self weight (ignoring self weight of concrete slab)
- $L_0 = \max \begin{cases} 80 \text{ psf} \leftarrow \text{Savers} \\ 50_{psf} + 20_{psf} \end{cases}$

$L = 80 \cdot \max \begin{cases} 0.5 \\ 0.25 + \frac{15}{\sqrt{10 \cdot 2 \cdot 25}} \end{cases}$

LL = 74. psf

Selecting Beam Size, Unshored

- $w_u = \min \begin{vmatrix} (1.4\ DL)\ \text{spacing} \\ (1.2DL + 1.6LL)\ \text{spacing} \end{vmatrix}$

$= \min \begin{vmatrix} 1.4(10+20+37) \cdot 10 = 938 \text{ lbs/ft} \\ 1.2(10+20+37) + 1.6(74)\ 10 = 1788 \text{ lb/ft} \end{vmatrix}$

- $M_u = \frac{wl^2}{8} = \frac{1.988(25)^2}{8} = 155.3 \text{ k}\cdot\text{ft}$

Beam Selection (table 3-19 AISC)

- assume 5" concrete deck and that a ≈ 1 ∴ $Y_2 = 4"$
- assume an $f'c = 4000$ psi
- assume PNA is in flange Position 4
- Try a W12×22 with $\Sigma Q_n = 196$ kips

Steel Deck design (From Vulcraft catalog)

- needs at least 2hr fire rating as per IBC 2000 Table 601
- Spacing is 4@10' - L/360 =
- Span is 25'
- Unprotected deck

1.5VL 1.5VLI 2VLI 3VLI

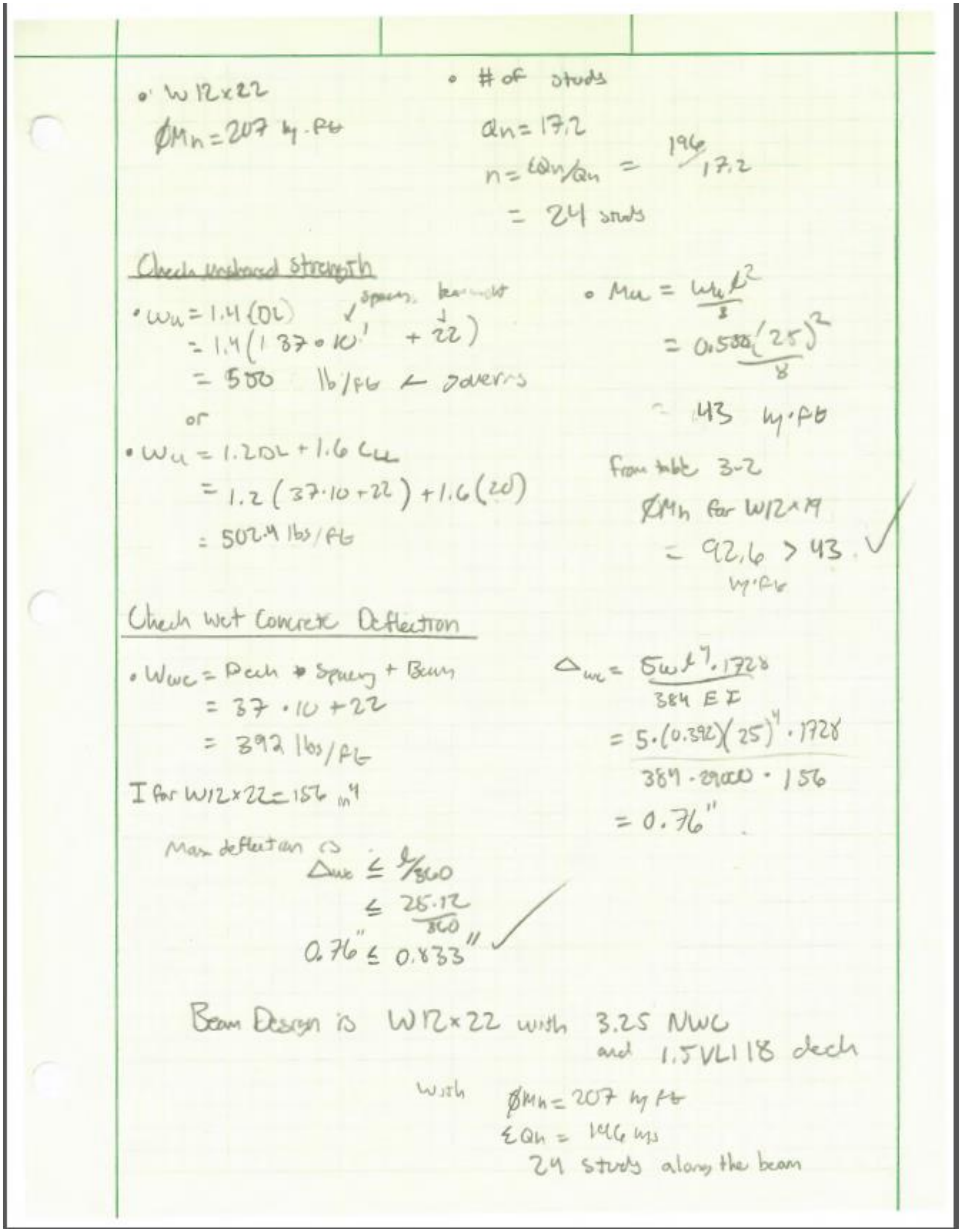
3 1/4 NW concrete 1.5VL18

- DL = 37psf

- Check a < 1

$a = \frac{\Sigma Q_n}{0.85 \cdot f'c \cdot b_{eff}}$ ← = spacing

$= \frac{196}{0.85 \cdot 4 \cdot 120} = 0.48$ ✓



- W12×22

$\phi M_n = 207$ k·ft

- # of studs

$Q_n = 17.2$

$n = \Sigma Q_n / Q_n = 196 / 17.2$

$= 24$ studs

Check unshored strength

- $w_u = 1.4(DL)$

$= 1.4(\overset{\text{spacing}}{37 \cdot 10'} + \overset{\text{beam wt}}{22})$

$= 500$ lb/ft ← governs

or

- $w_u = 1.2DL + 1.6LL$

$= 1.2(37 \cdot 10 + 22) + 1.6(20)$

$= 502.4$ lbs/ft

- $M_u = \frac{w_u l^2}{8}$

$= \frac{0.500(25)^2}{8}$

$= 43$ k·ft

From table 3-2

ϕM_n for W12×19

$= 92.6 > 43$ k·ft ✓

Check Wet Concrete Deflection

- W_{wc} = Deck * Spacing + Beam

$= 37 \cdot 10 + 22$

$= 392$ lbs/ft

I for W12×22 = 156 in^4

Max deflection is

$\Delta_{wc} \le l/360$

$\le \frac{25 \cdot 12}{360}$

$0.76'' \le 0.833''$ ✓

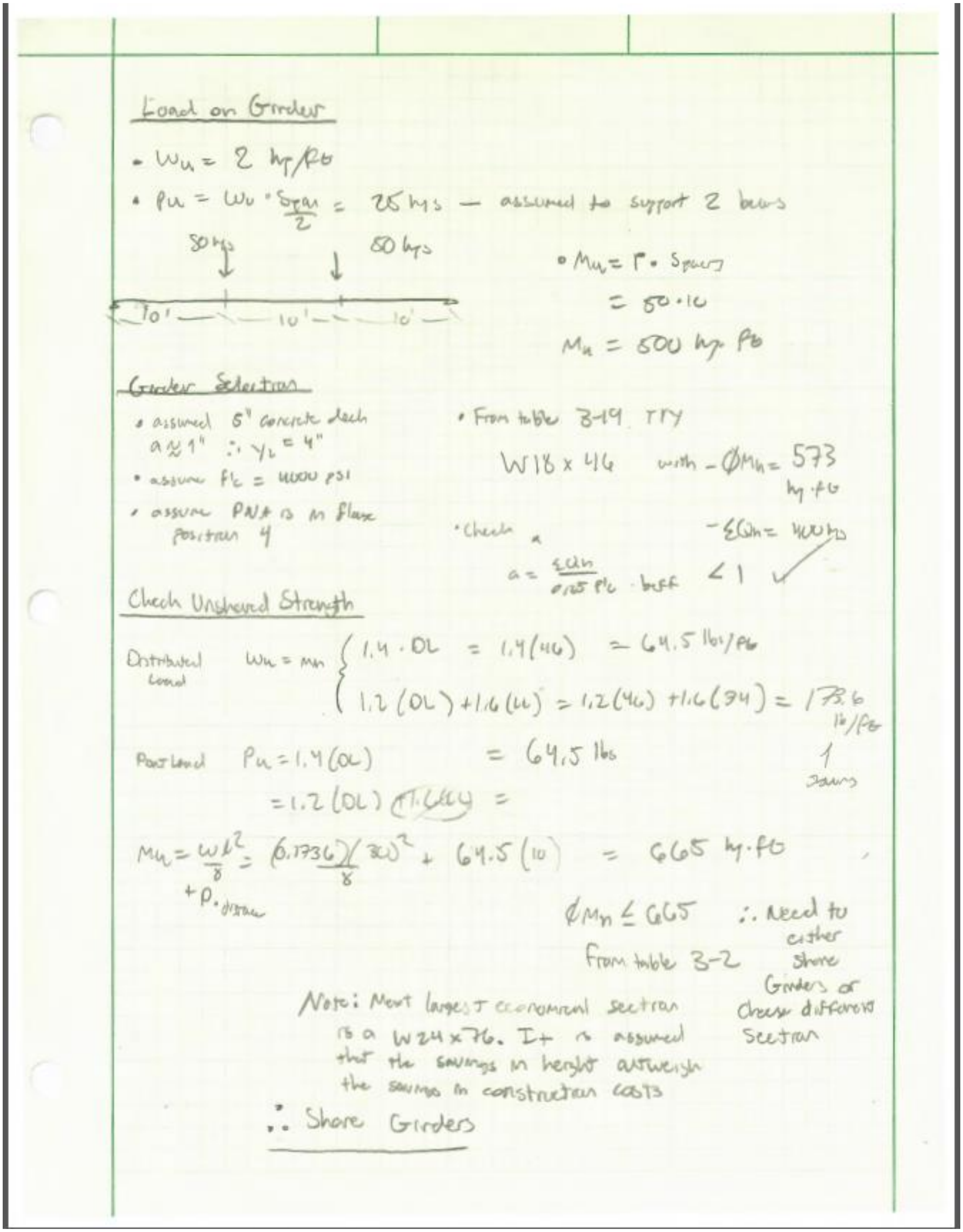
$\Delta_{wc} = \frac{5 w l^4 \cdot 1728}{384 EI}$

$= \frac{5 \cdot (0.392)(25)^4 \cdot 1728}{384 \cdot 29000 \cdot 156}$

$= 0.76''$

Beam Design is W12×22 with 3.25 NWC
and 1.5VL118 deck

with $\phi M_n = 207$ k·ft
$\Sigma Q_n = 196$ kips
24 studs along the beam



Load on Girder

- $W_u = 2$ kips/ft
- $P_u = W_u \cdot \frac{Span}{2} = 25$ kips — assumed to support 2 beams

- $M_u = P \cdot Spacing$

$= 50 \cdot 10$

$M_u = 500$ kip·ft

Girder Selection

- assumed 5" concrete deck $a \approx 1"$ $\therefore Y_2 = 4"$
- assume $f'_c = 4000$ psi
- assume PNA is in flange position 4

- From table 3-19 TRY

W18 x 46 with $-\phi M_n = 573$ kip·ft

$-\Sigma Q_n = 400$ kips

- Check a

$a = \frac{\Sigma Q_n}{0.85 f'_c \cdot b_{eff}} < 1$ ✓

Check Unshored Strength

Distributed Load $W_u = \max \begin{cases} 1.4 \cdot DL = 1.4(46) = 64.5 \text{ lbs/ft} \\ 1.2(DL) + 1.6(LL) = 1.2(46) + 1.6(74) = 173.6 \text{ lb/ft} \end{cases}$

Point Load $P_u = 1.4(DL) = 64.5$ lbs

$= 1.2(DL)$ ~~+1.6LL~~ $=$

1 kips

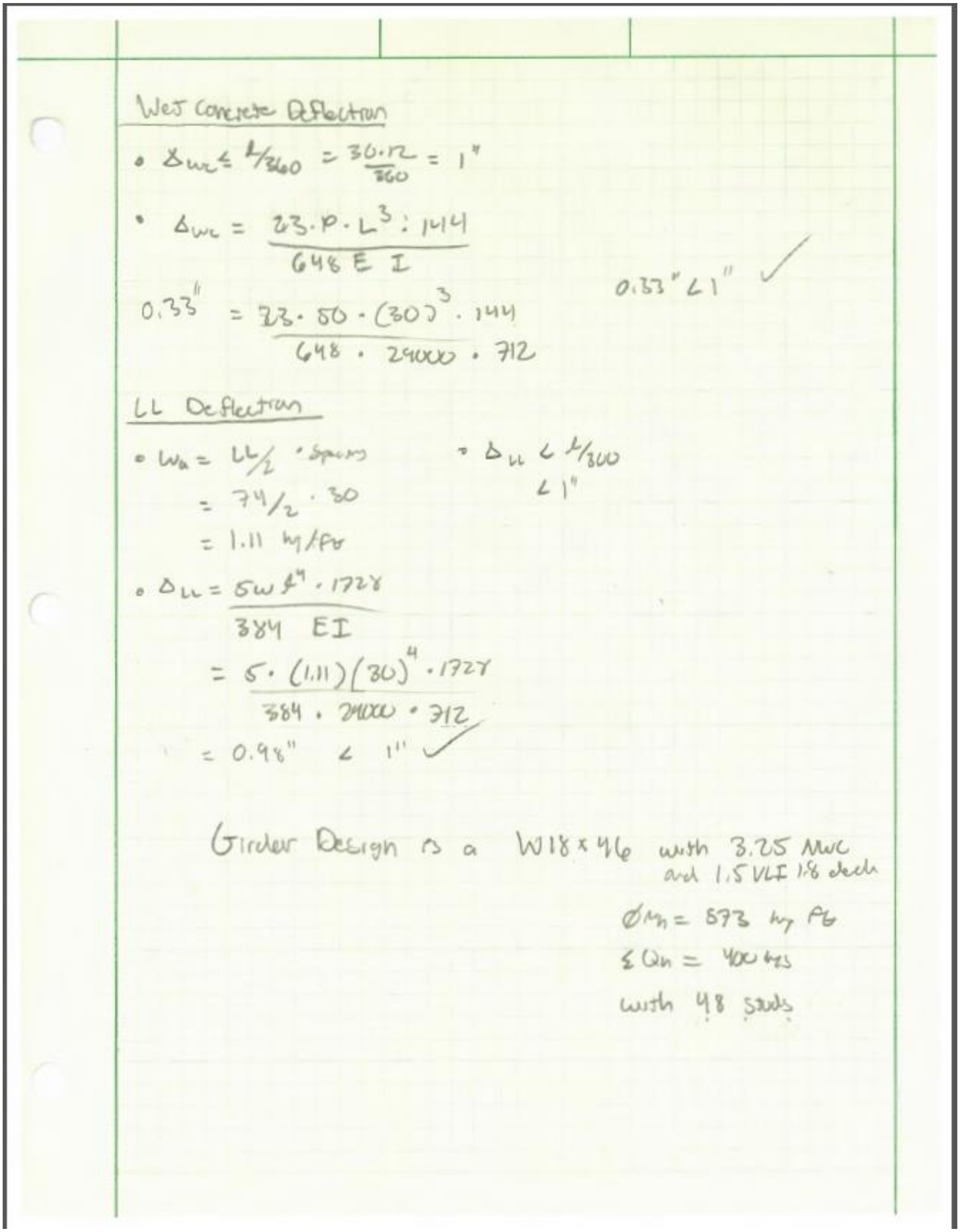
$M_u = \frac{wl^2}{8} + P \cdot distance = \frac{(0.1736)(30)^2}{8} + 64.5(10) = 665$ kip·ft

$\phi M_n \le 665$ ∴ Need to either shore Girders or choose different section

From table 3-2

Note: Most largest economical section is a W24x76. It is assumed that the savings in height outweigh the savings in construction costs

∴ Shore Girders



Wet Concrete Deflection

- $\Delta_{wc} \leq l/360 = \frac{30 \cdot 12}{360} = 1''$

- $\Delta_{wc} = \frac{23 \cdot P \cdot L^3 \cdot 144}{648 E I}$

$0.33'' = \frac{23 \cdot 50 \cdot (30)^3 \cdot 144}{648 \cdot 29000 \cdot 712}$ $0.33'' < 1''$ ✓

LL Deflection

- $w_{LL} = LL/2 \cdot \text{spans}$
  $= 74/2 \cdot 30$
  $= 1.11 \text{ k/ft}$

- $\Delta_{LL} < l/360$
  $< 1''$

- $\Delta_{LL} = \frac{5 w l^4 \cdot 1728}{384 EI}$
  $= \frac{5 \cdot (1.11)(30)^4 \cdot 1728}{384 \cdot 29000 \cdot 712}$
  $= 0.98'' < 1''$ ✓

Girder Design is a W18×46 with 3.25 NWC and 1.5 VLI 18 deck

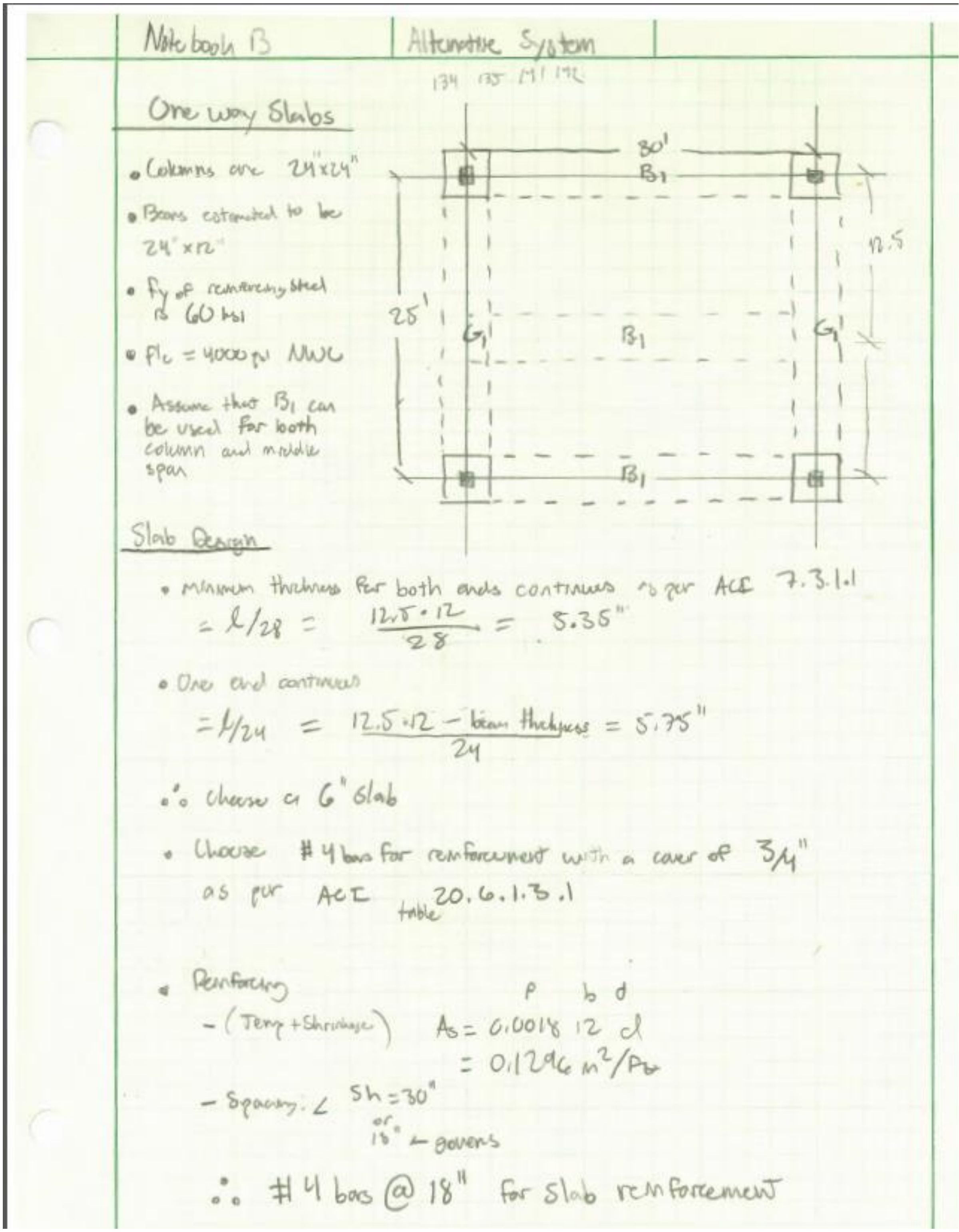
$\phi M_n = 573 \text{ k-ft}$

$\Sigma Q_n = 400 \text{ kips}$

with 48 studs

4.2 One Way Slab

A one way slab system was initially chosen out of interest for feasibility and system requirements. It was known initially that a two way system is more practical given the square dimension of the bay. This slab design could be used in the future if the dimensions of the bay become more rectangular in nature. The system features a concrete beam spanning the middle of the bay and supported by a concrete girder. Slab and member design were based on ACI 318-14 for reinforcement, moment capacity, shear capacity and deflection.



Notebook B | Alternative System

One way Slabs

- Columns are 24"x24"
- Beams estimated to be 24"x12"
- fy of reinforcing steel is 60 ksi
- f'c = 4000 psi NWC
- Assume that B1 can be used for both column and middle span

Slab Design

- Minimum thickness for both ends continuous as per ACI 7.3.1.1

$$= l/28 = \frac{12.5 \cdot 12}{28} = 5.35''$$

- One end continuous

$$= l/24 = \frac{12.5 \cdot 12 - \text{beam thickness}}{24} = 5.75''$$

∴ Choose a 6" Slab

- Choose #4 bars for reinforcement with a cover of 3/4" as per ACI table 20.6.1.3.1

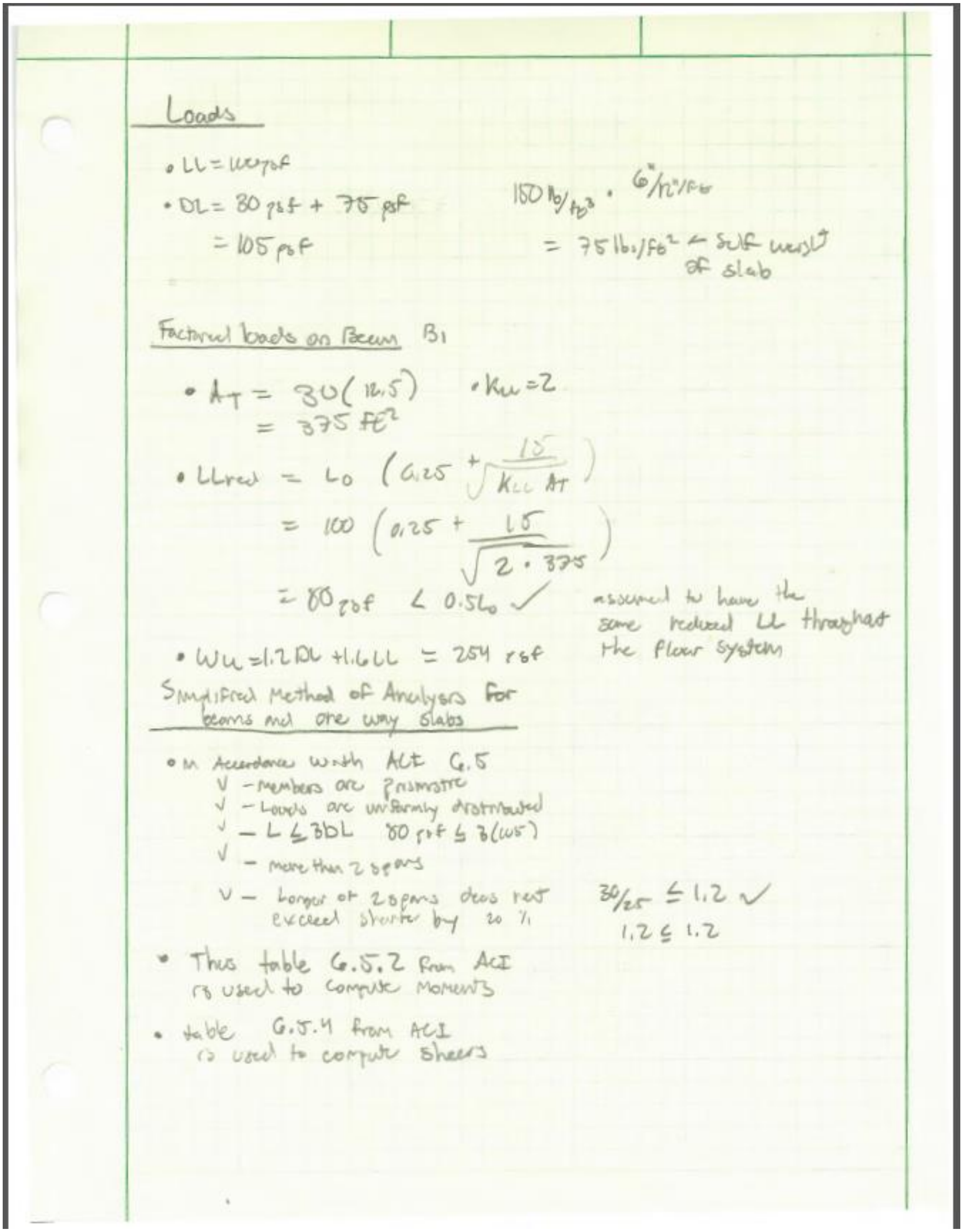
- Reinforcing

– (Temp + Shrinkage) $A_s = \overset{\rho}{0.0018}\ \overset{b}{12}\ \overset{d}{d}$

$= 0.1296\ in^2/ft$

– Spacing: $<$ 5h = 30" or 18" ← governs

∴ #4 bars @ 18" for slab reinforcement



Loads

- $LL = 100 \text{ psf}$
- $DL = 30 \text{ psf} + 75 \text{ psf}$
  $= 105 \text{ psf}$

$150 \text{ lb/ft}^3 \cdot 6''/12''/\text{ft}$

$= 75 \text{ lb/ft}^2$ ← self weight of slab

Factored loads on Beam B1

- $A_T = 30(12.5)$
  $= 375 \text{ ft}^2$
- $K_{LL} = 2$
- $$LL_{red} = L_0\left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right)$$
  $$= 100\left(0.25 + \frac{15}{\sqrt{2 \cdot 375}}\right)$$
  $$= 80 \text{ psf} < 0.5L_0 \checkmark$$

assumed to have the same reduced LL throughout the floor system

- $W_u = 1.2DL + 1.6LL = 254 \text{ psf}$

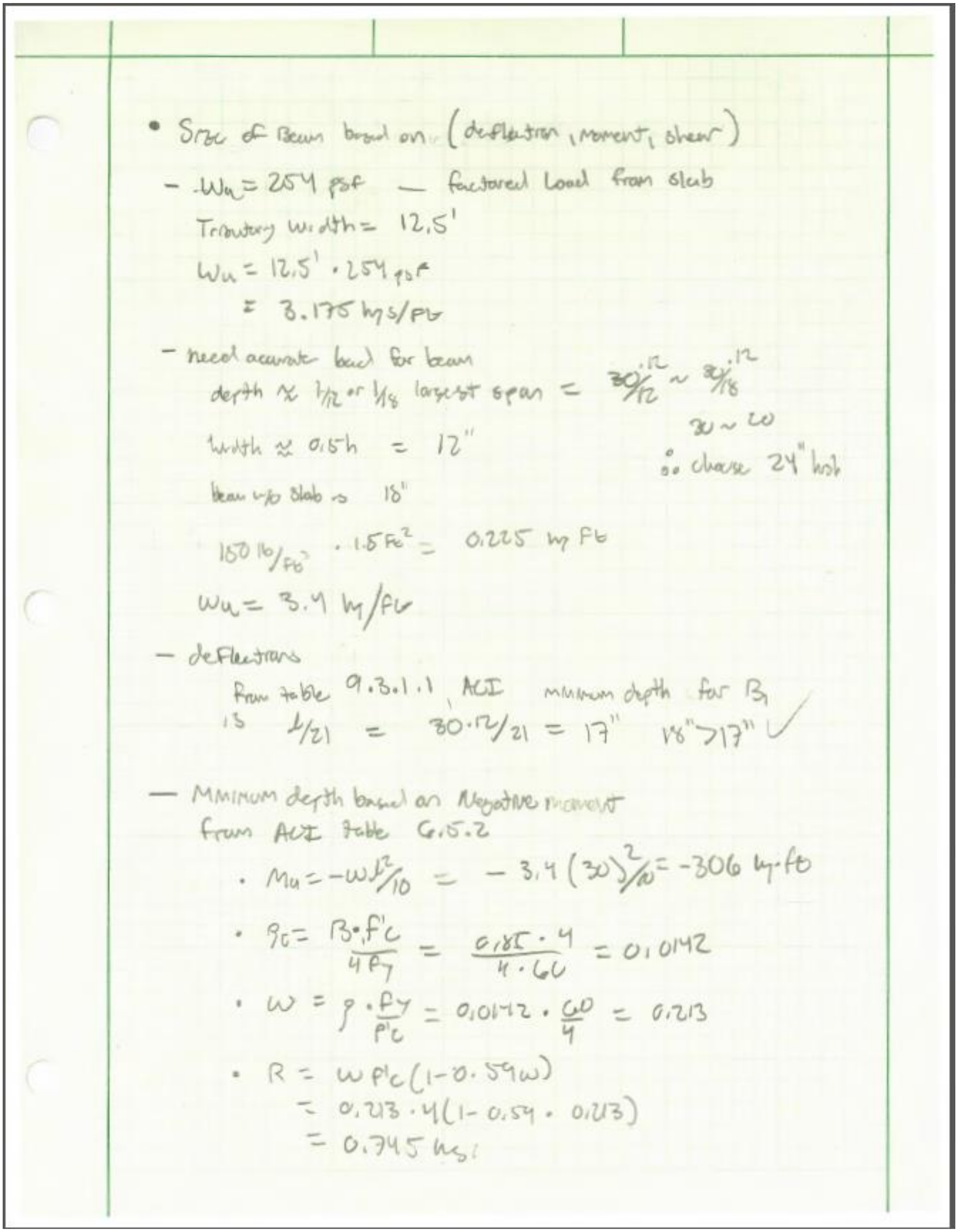
Simplified Method of Analysis for beams and one way slabs

- In Accordance with ACI 6.5
  - ✓ – members are prismatic
  - ✓ – Loads are uniformly distributed
  - ✓ – $L \leq 3DL$ $\quad 80 \text{ psf} \leq 3(105)$
  - ✓ – more than 2 spans
  - ✓ – Longer of 2 spans does not exceed shorter by 20 %

$30/25 \leq 1.2 \checkmark$

$1.2 \leq 1.2$

- Thus table 6.5.2 from ACI is used to compute moments
- table 6.5.4 from ACI is used to compute shears



- Size of Beam based on (deflection, moment, shear)

- $W_u = 254$ psf — factored Load from slab

  Tributary width = 12.5'

  $W_u = 12.5' \cdot 254$ psf

  $= 3.175$ kips/ft

- need accurate load for beam

  depth ≈ 1/12 or 1/18 largest span = $\frac{30}{12}^{\cdot 12} \sim \frac{30}{18}^{\cdot 12}$

  $30 \sim 20$

  ∴ chose 24" high

  width ≈ 0.5h = 12"

  beam w/o slab is 18"

  $150 \text{ lb}/\text{ft}^3 \cdot 1.5 \text{ ft}^2 = 0.225$ kip ft

  $W_u = 3.4$ kip/ft

- deflections

  From table 9.3.1.1 ACI minimum depth for B_1

  is $\ell/21 = 30 \cdot 12/21 = 17"$ 18" > 17" ✓

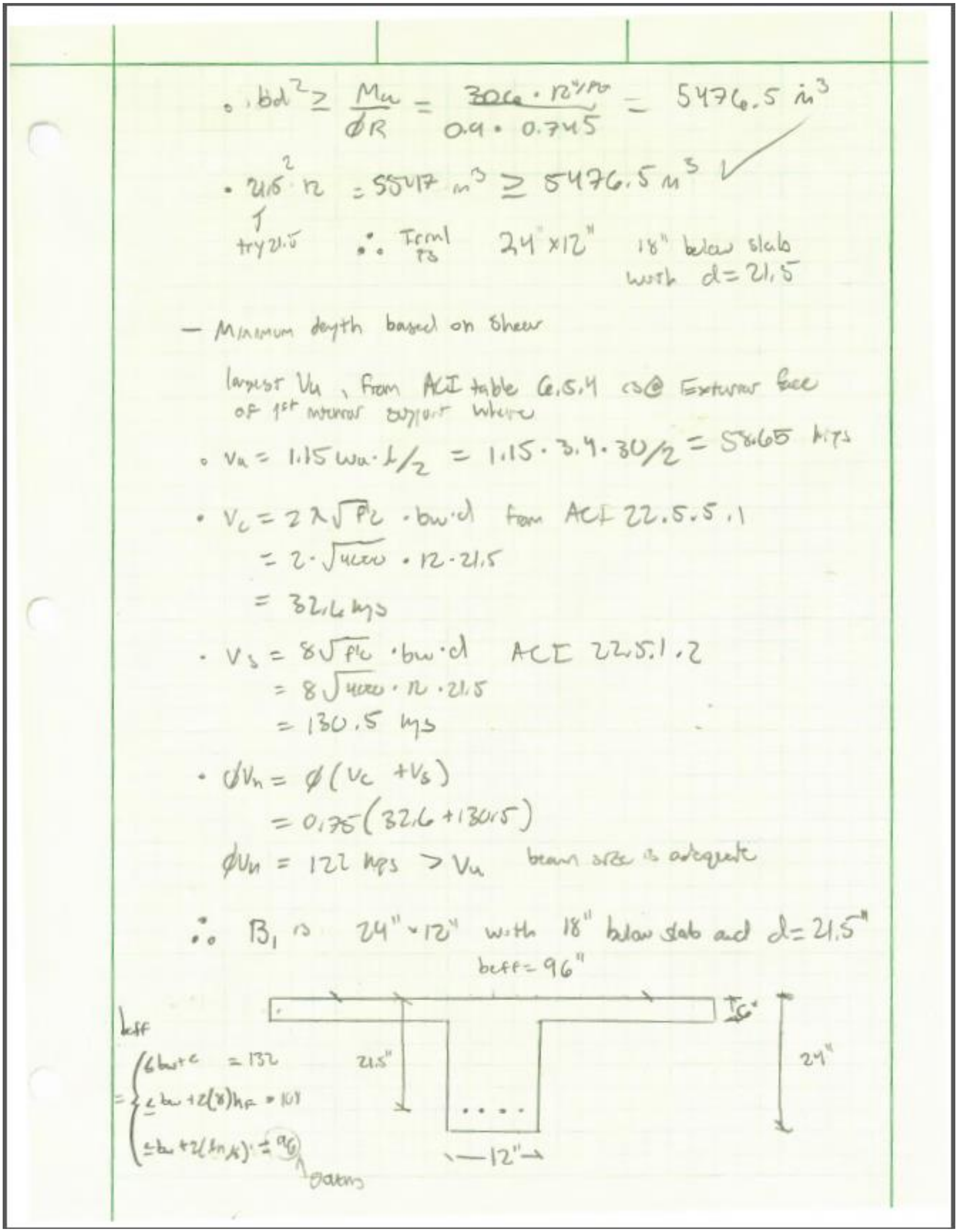
- Minimum depth based on Negative moment

  from ACI table 6.5.2

  - $M_u = -w\ell^2/10 = -3.4(30)^2/10 = -306$ kip-ft
  - $\rho_t = \frac{\beta_1 f'_c}{4 f_y} = \frac{0.85 \cdot 4}{4 \cdot 60} = 0.0142$
  - $\omega = \rho \cdot \frac{f_y}{f'_c} = 0.0142 \cdot \frac{60}{4} = 0.213$
  - $R = \omega f'_c (1 - 0.59\omega)$

    $= 0.213 \cdot 4(1 - 0.59 \cdot 0.213)$

    $= 0.745$ ksi



- $bd^2 \geq \frac{M_u}{\phi R} = \frac{306 \cdot 12 \text{ in/ft}}{0.9 \cdot 0.745} = 5476.5 \text{ in}^3$
- $21.5^2 \cdot 12 = 5547 \text{ in}^3 \geq 5476.5 \text{ in}^3$ ✓

try 21.5 ∴ Trial is 24"×12" 18" below slab with $d = 21.5$

— Minimum depth based on Shear

largest V_u, from ACI table 6.5.4 is @ Exterior face of 1st interior support where

- $V_u = 1.15 w_u \cdot l/2 = 1.15 \cdot 3.4 \cdot 30/2 = 58.65$ kips
- $V_c = 2\lambda\sqrt{f'_c} \cdot b_w \cdot d$ from ACI 22.5.5.1
  $= 2 \cdot \sqrt{4000} \cdot 12 \cdot 21.5$
  $= 32.6$ kips
- $V_s = 8\sqrt{f'_c} \cdot b_w \cdot d$ ACI 22.5.1.2
  $= 8\sqrt{4000} \cdot 12 \cdot 21.5$
  $= 130.5$ kips
- $\phi V_n = \phi(V_c + V_s)$
  $= 0.75(32.6 + 130.5)$
  $\phi V_n = 122 \text{ kips} > V_u$ beam size is adequate

∴ B_1 is 24"×12" with 18" below slab and $d = 21.5$"

beff = 96"

$$b_{eff} = \begin{cases} 6b_w + c = 132 \\ \leq b_w + 2(8)h_f = 108 \\ \leq b_w + 2(l_n/8) = \boxed{96} \end{cases}$$

governs

6"

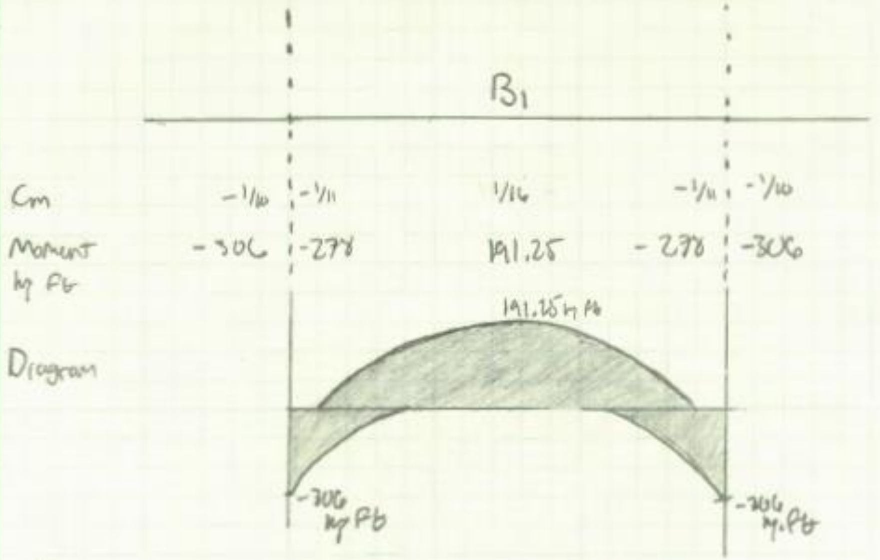
21.5"

24"

12"

Beam Moments

- $w_u = 3.4$ k/ft



Cm	$-1/10$	$-1/11$	$1/16$	$-1/11$	$-1/10$
Moment k ft	-306	-278	191.25	-278	-306

Area of Steel Required for Negative moments

Assume $j = 0.9$
$\phi = 0.9$

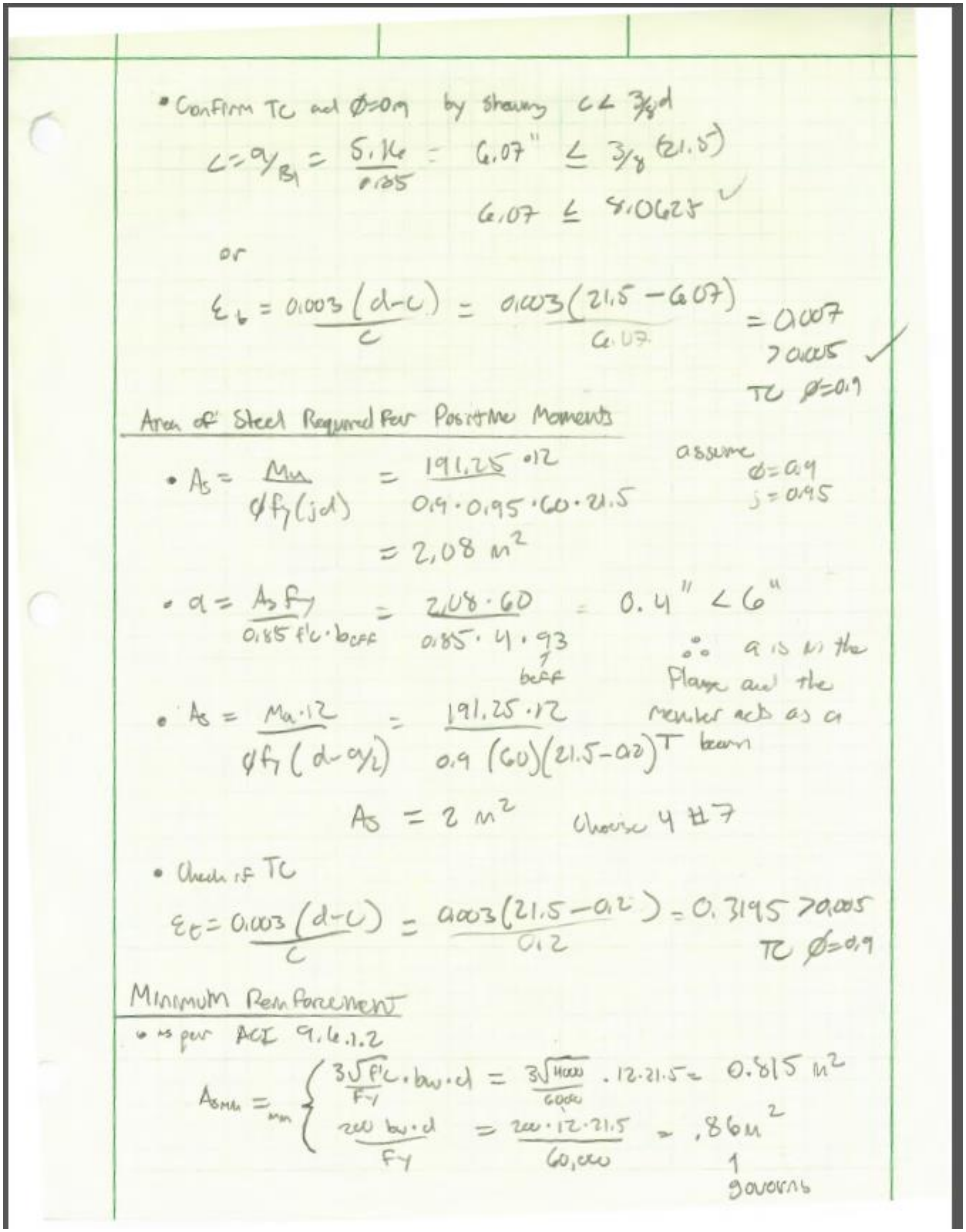
- $$A_s \geq \frac{M_u}{\phi F_y (d - a/2)} = \frac{M_u}{\phi F_y (jd)}$$
$$= \frac{306 \cdot 12}{(0.9)^2 \cdot 60 \cdot 21.5}$$
$$= 3.51 \text{ in}^2$$

- $$a = \frac{A_s F_y}{0.85 f'_c \cdot b} = \frac{3.51 \cdot 60}{0.85 \cdot 4 \cdot 12} = 5.16''$$

- Solve for more accurate A_s $$A_s \geq 306 \cdot 12 / 0.9 \cdot 60 (21.5 - 5.16/2)$$

$$A_s \geq 3.6 \text{ in}^2$$

Choose 6 #7 bars



- Confirm TC and $\phi = 0.9$ by showing $c < 3/8 d$

$$c = a/\beta_1 = \frac{5.16}{0.85} = 6.07'' \leq 3/8\,(21.5)$$

$$6.07 \leq 8.0625 \checkmark$$

or

$$\varepsilon_t = \frac{0.003\,(d-c)}{c} = \frac{0.003\,(21.5 - 6.07)}{6.07} = 0.007$$

> 0.005 ✓

TC $\phi = 0.9$

Area of Steel Required For Positive Moments

- $A_s = \frac{M_u}{\phi f_y (jd)} = \frac{191.25 \cdot 12}{0.9 \cdot 0.95 \cdot 60 \cdot 21.5}$ assume $\phi = 0.9$, $j = 0.95$

$$= 2.08\ in^2$$

- $a = \frac{A_s f_y}{0.85 f'c \cdot b_{eff}} = \frac{2.08 \cdot 60}{0.85 \cdot 4 \cdot \underset{b_{eff}}{93}} = 0.4'' < 6''$

$\therefore$ a is in the Flange and the member acts as a T beam

- $A_s = \frac{M_u \cdot 12}{\phi f_y (d - a/2)} = \frac{191.25 \cdot 12}{0.9\,(60)(21.5 - 0.2)}$

$$A_s = 2\ in^2$$ Choose 4 #7

- Check if TC

$$\varepsilon_t = \frac{0.003\,(d-c)}{c} = \frac{0.003\,(21.5 - 0.2)}{0.2} = 0.3195 > 0.005$$

TC $\phi = 0.9$

Minimum Reinforcement

- as per ACI 9.6.1.2

$$A_{s,min} = \min \begin{cases} \frac{3\sqrt{f'c}}{F_y} \cdot b_w \cdot d = \frac{3\sqrt{4000}}{60000} \cdot 12 \cdot 21.5 = 0.815\ in^2 \\ \frac{200\ b_w \cdot d}{F_y} = \frac{200 \cdot 12 \cdot 21.5}{60{,}000} = \underset{\text{governs}}{.86\ in^2} \end{cases}$$

Distribution of Reinforcement

- $C_c = 1.5'' + \underset{\text{stirrup}}{\underset{\uparrow}{0.375}} = 1.875''$

ACI 24.3.2

- $\text{Spacing} = 15\left(\frac{40{,}000}{f_s}\right) - 2.5c_c \leq 12\left(\frac{40{,}000}{f_s}\right)$

where $f_s = 0.67 f_y$

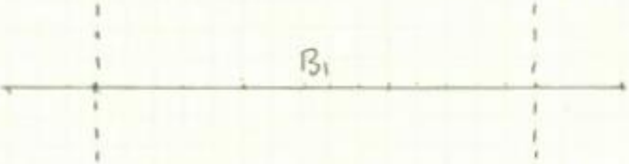
$$\text{Spacing} = 14.925 - 2.5(1.875) \leq 11.94$$

$$10.23 \leq 11.94 \checkmark$$

- Negative moment region

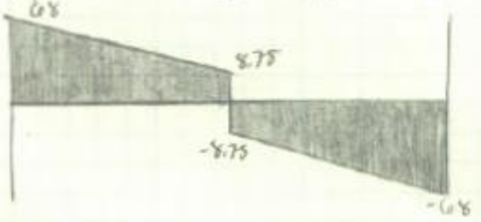
ACF 24.3.4 spaced at over$_{min}$ $\begin{cases} \text{effective width} = 93'' \\ l_n/_{10} = 36'' \end{cases}$

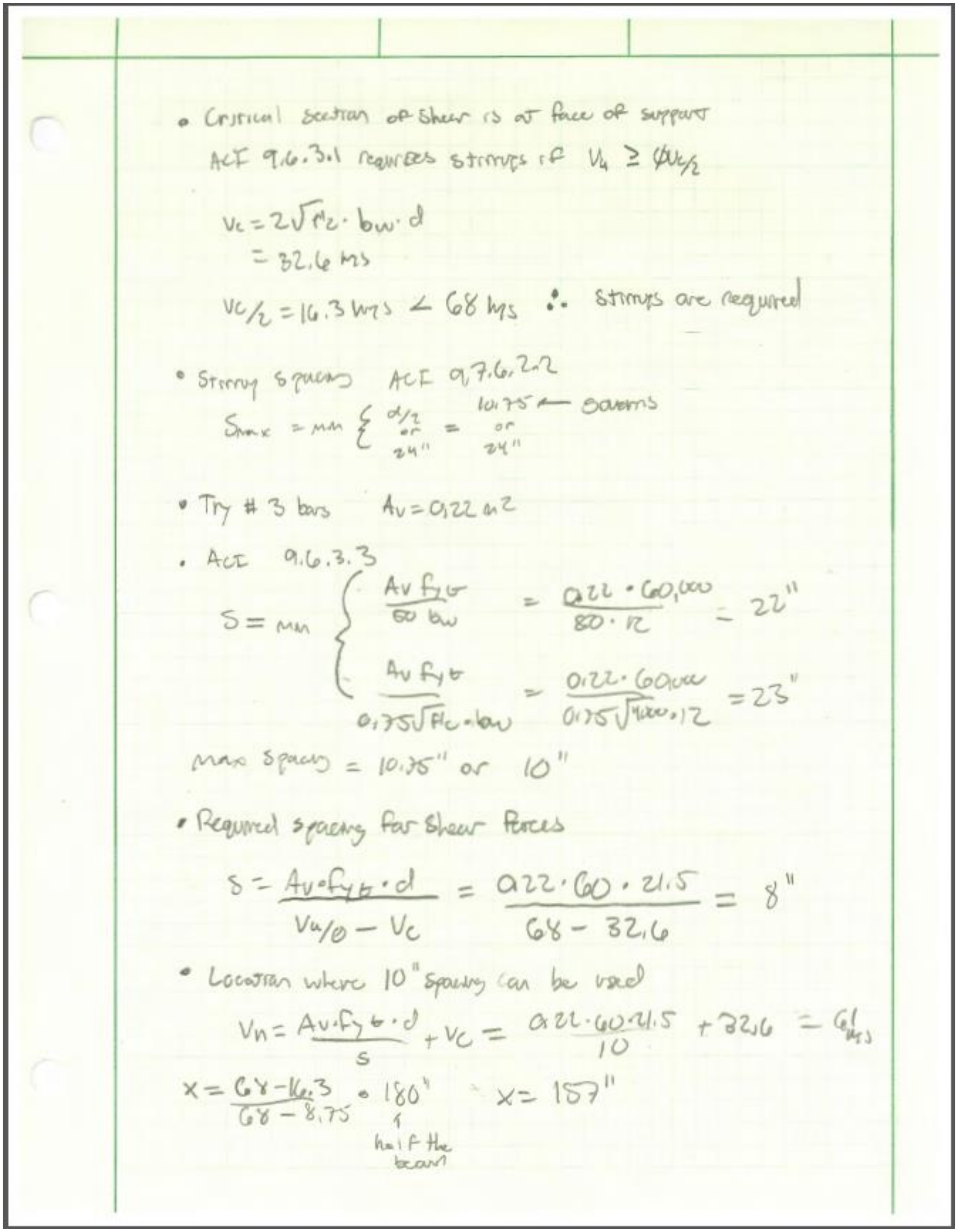
Shear Reinforcement



Shear at supports is $w_u l/2$ $= \frac{3.4 \cdot 30}{2}$	51	51	51
C_v	1	0.125	1
V_u	51 kips	6.5	51 kips
V_u/ϕ	68 kips	8.75 kips	68 kips

Shear Diagram





- Critical section of shear is at face of support

  ACI 9.6.3.1 requires stirrups if $V_u \geq \phi V_c/2$

$$V_c = 2\sqrt{f'_c} \cdot b_w \cdot d$$
$$= 32.6 \text{ kips}$$
$$V_c/2 = 16.3 \text{ kips} < 68 \text{ kips} \quad \therefore \text{ stirrups are required}$$

- Stirrup spacing ACI 9.7.6.2.2

$$S_{max} = \min \begin{cases} d/2 \\ \text{or} \\ 24'' \end{cases} = \begin{matrix} 10.75 \leftarrow \text{governs} \\ \text{or} \\ 24'' \end{matrix}$$

- Try # 3 bars $A_v = 0.22 \text{ in}^2$

- ACI 9.6.3.3

$$S = \min \begin{cases} \dfrac{A_v f_{yt}}{50\, b_w} = \dfrac{0.22 \cdot 60{,}000}{80 \cdot 12} = 22'' \\[2ex] \dfrac{A_v f_{yt}}{0.75\sqrt{f'_c} \cdot b_w} = \dfrac{0.22 \cdot 60{,}000}{0.75\sqrt{4000} \cdot 12} = 23'' \end{cases}$$

Max spacing = 10.75" or 10"

- Required spacing for shear forces

$$s = \frac{A_v \cdot f_{yt} \cdot d}{V_u/\phi - V_c} = \frac{0.22 \cdot 60 \cdot 21.5}{68 - 32.6} = 8''$$

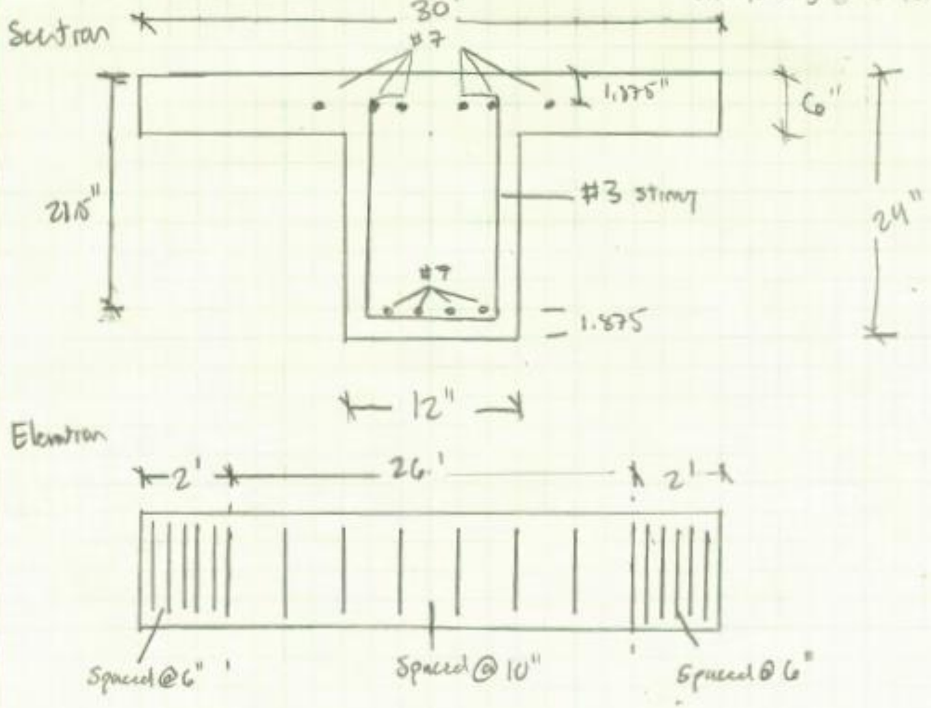
- Location where 10" spacing can be used

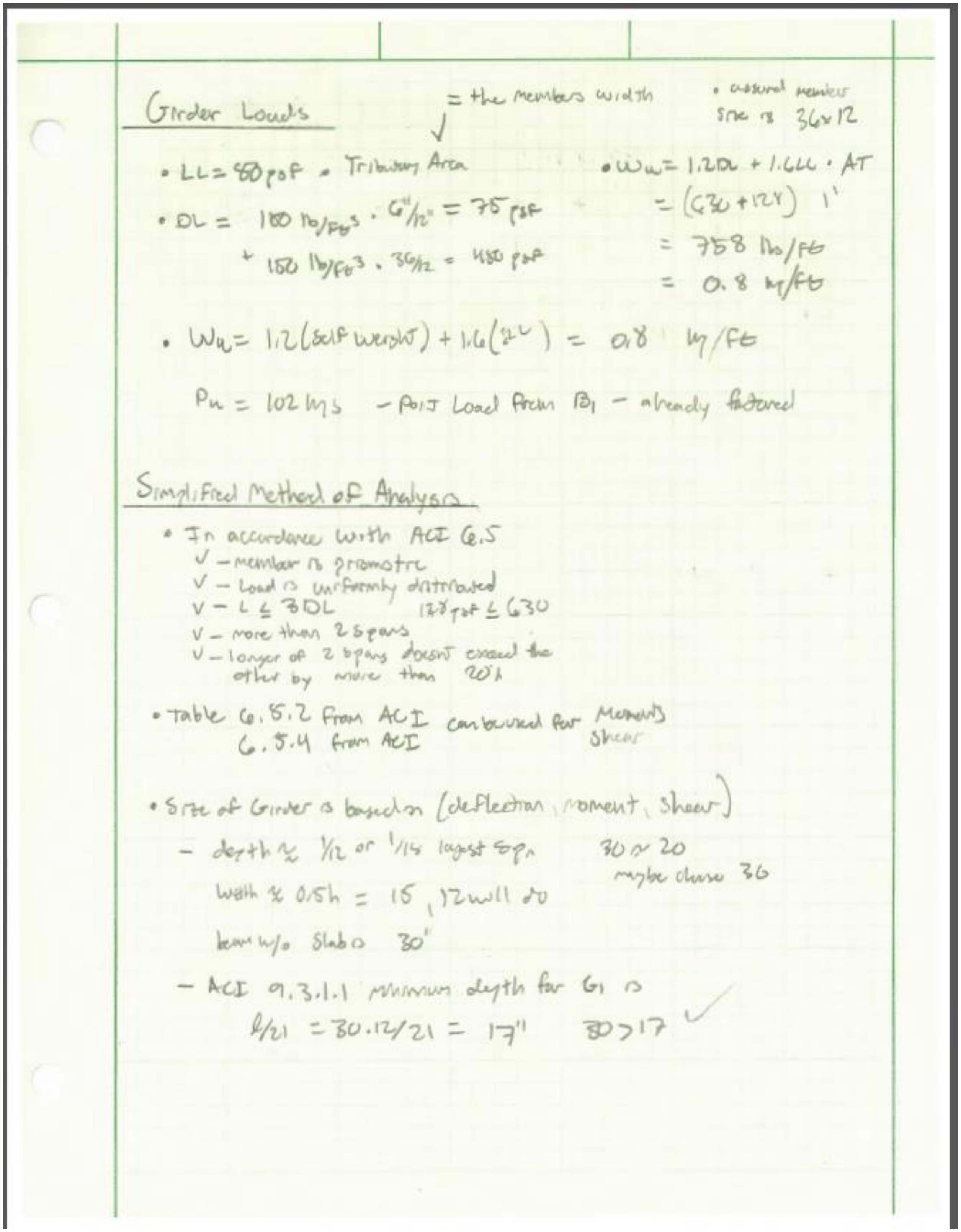
$$V_n = \frac{A_v \cdot f_{yt} \cdot d}{s} + V_c = \frac{0.22 \cdot 60 \cdot 21.5}{10} + 32.6 = 61 \text{ kips}$$

$$x = \frac{68 - 16.3}{68 - 8.75} \cdot \underset{\substack{\uparrow \\ \text{half the} \\ \text{beam}}}{180''} \qquad x = 157''$$

Details for B_1

Notes: top 6 #7
bot 4 #7
Stirrups #3 @ 6' for first 2'





Girder Loads

= the members width ✓

- assumed member size is 36x12

- LL = 80 psf · Tributary Area
- DL = 180 lb/ft³ · 6"/12" = 75 psf
  + 150 lb/ft³ · 36/12 = 450 psf

- $W_u = 1.2DL + 1.6LL \cdot AT$
  $= (630 + 128)\ 1'$
  $= 758$ lb/ft
  $= 0.8$ k/ft

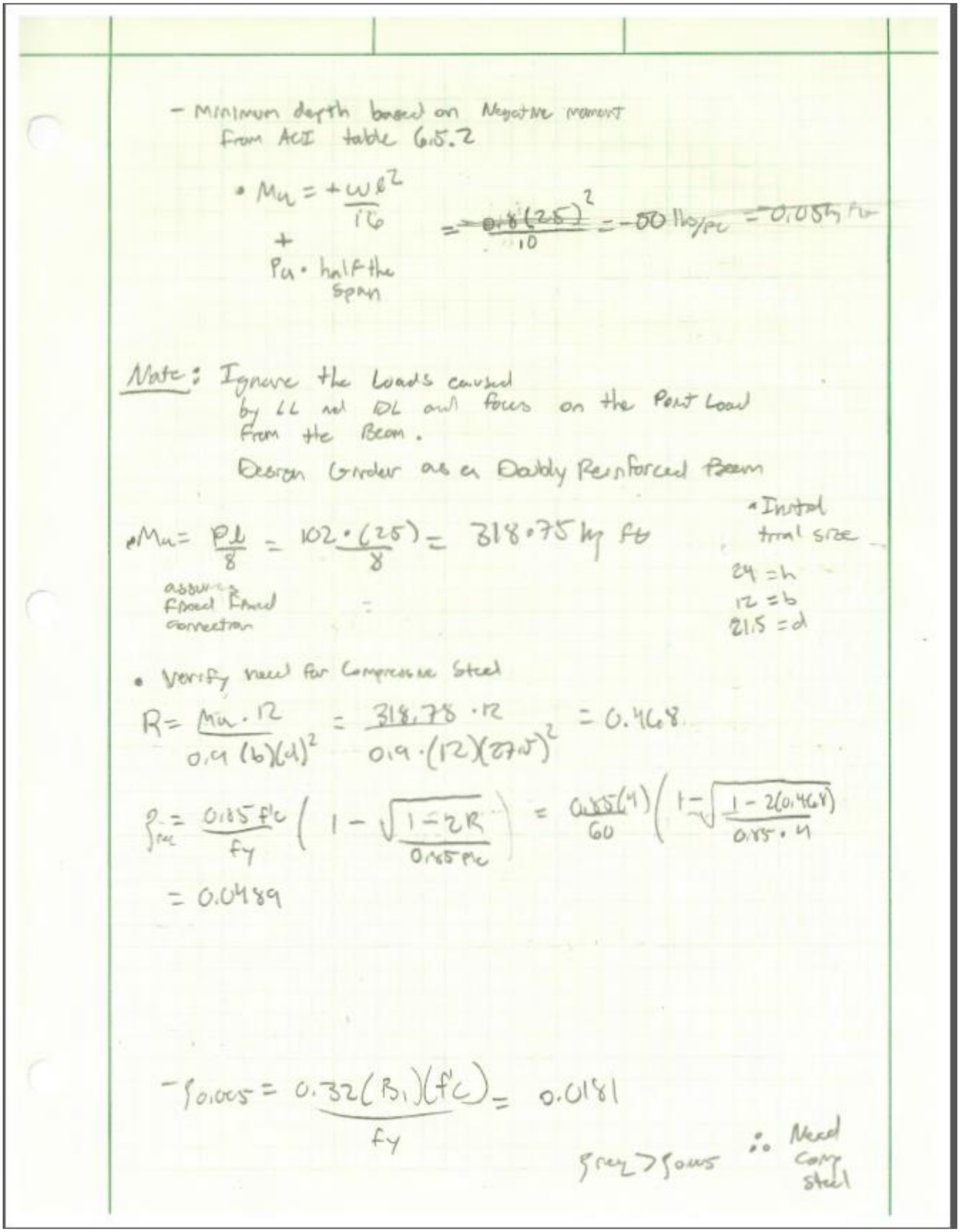
- $W_u = 1.2(\text{self weight}) + 1.6(LL) = 0.8$ k/ft

$P_u = 102$ kips – Point Load from B_1 – already factored

Simplified Method of Analysis

- In accordance with ACI 6.5
  - ✓ – member is prismatic
  - ✓ – Load is uniformly distributed
  - ✓ – $L \leq 3DL$ 128 psf ≤ 630
  - ✓ – more than 2 spans
  - ✓ – longer of 2 spans doesn't exceed the other by more than 20%
- Table 6.5.2 from ACI can be used for Moments
  6.5.4 from ACI Shear

- Size of Girder is based on (deflection, moment, shear)
  - depth ≈ 1/12 or 1/16 largest span 30 ~ 20
    maybe choose 36
    width ≈ 0.5h = 15, 12 will do
    beam w/o slab is 30"
  - ACI 9.3.1.1 minimum depth for G_1 is
    $\ell/21 = 30 \cdot 12/21 = 17"$ $30 > 17$ ✓



- Minimum depth based on Negative moment
from ACI table 6.5.2

$$\bullet\ M_u = +\frac{w\ell^2}{16} \quad = \cancel{\frac{0.8(25)^2}{10} = -50\ \text{lbs/ft} = -0.05k\ \text{ft}}$$

$+$

$P_u \cdot$ half the span

Note: Ignore the Loads caused by LL and DL and focus on the Point Load from the Beam.

Design Girder as a Doubly Reinforced Beam

$$\bullet M_u = \frac{P\ell}{8} = \frac{102\cdot(25)}{8} = 318.75\ \text{k ft}$$

assuming fixed fixed connection

• Initial trial size

$24 = h$

$12 = b$

$21.5 = d$

• Verify need for Compressive Steel

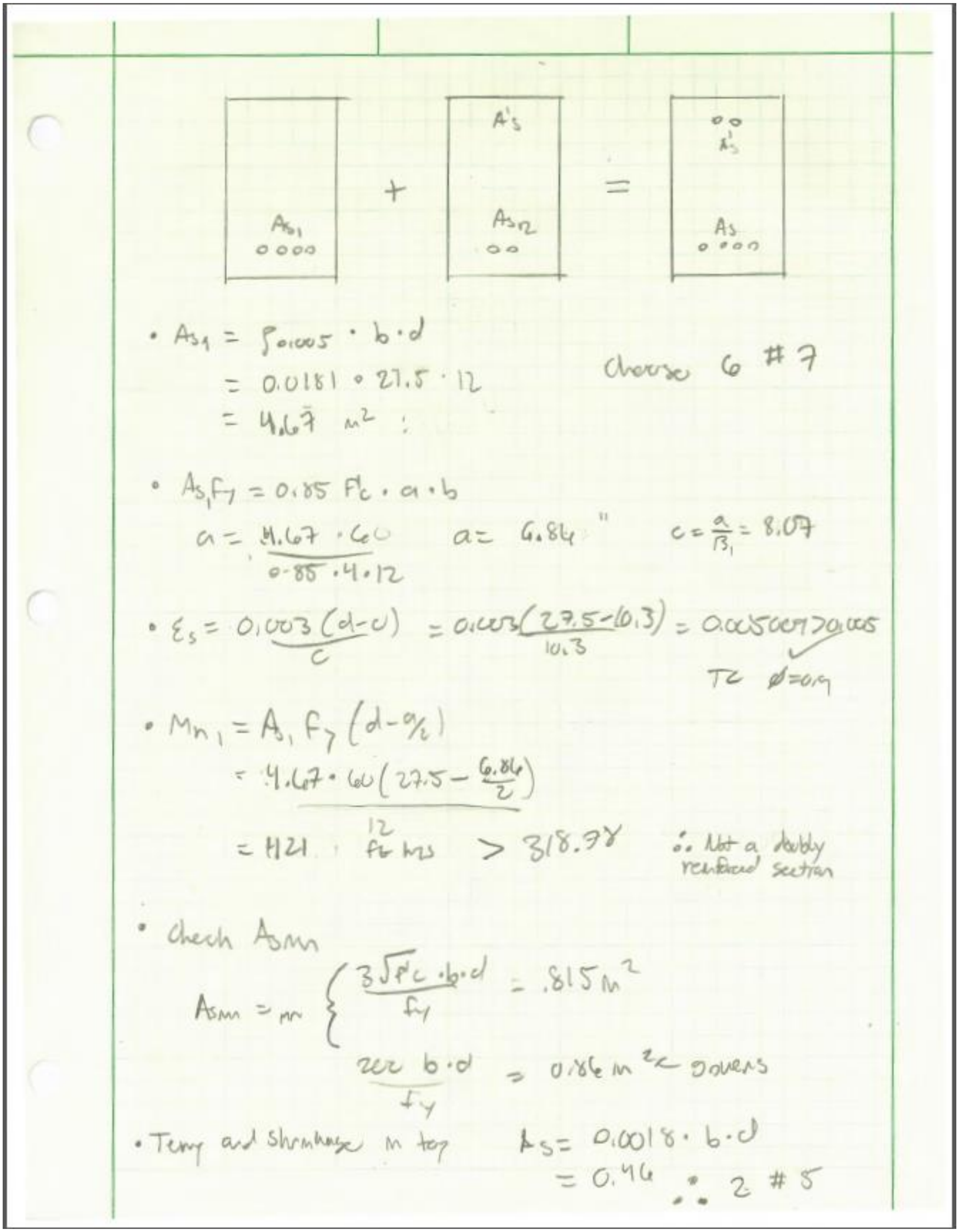
$$R = \frac{M_u \cdot 12}{0.9\,(b)(d)^2} = \frac{318.78\cdot 12}{0.9\cdot(12)(27.5)^2} = 0.468$$

$$\rho_{req} = \frac{0.85 f'c}{fy}\left(1 - \sqrt{\frac{1-2R}{0.85 f'c}}\right) = \frac{0.85(4)}{60}\left(1 - \sqrt{\frac{1-2(0.468)}{0.85\cdot 4}}\right)$$

$= 0.0489$

$$-\rho_{0.005} = \frac{0.32(B_1)(f'c)}{fy} = 0.0181$$

$\rho_{req} > \rho_{0.005}$ ∴ Need Comp steel



- $A_{s1} = \rho_{0.005} \cdot b \cdot d$

  $= 0.0181 \cdot 27.5 \cdot 12$

  $= 4.67\ in^2$

  Choose 6 #7

- $A_{s1} f_y = 0.85\ f'_c \cdot a \cdot b$

  $a = \frac{4.67 \cdot 60}{0.85 \cdot 4 \cdot 12}$ $a = 6.84''$ $c = \frac{a}{\beta_1} = 8.07$

- $\varepsilon_s = 0.003 \frac{(d-c)}{c} = 0.003 \frac{(27.5-10.3)}{10.3} = 0.005009 > 0.005$ ✓

  TC $\phi = 0.9$

- $M_{n1} = A_{s1} f_y (d - a/2)$

  $= \frac{4.67 \cdot 60 \left(27.5 - \frac{6.84}{2}\right)}{12}$

  $= 1121$ ft kips > 318.98 ∴ Not a doubly reinforced section

- Check $A_{s,min}$

  $A_{s,min} = min \begin{cases} \frac{3\sqrt{f'_c} \cdot b \cdot d}{f_y} = .815\ in^2 \\ \frac{200\ b \cdot d}{f_y} = 0.86\ in^2 \leftarrow \text{governs} \end{cases}$

- Temp and shrinkage in top $A_s = 0.0018 \cdot b \cdot d$

  $= 0.46$ ∴ 2 #5

- Stirrup Reinforcement
  - $V_u = \underset{\uparrow}{1.15} \cdot P/2 = 1.15 \cdot 102/2 = 58.65$ kips

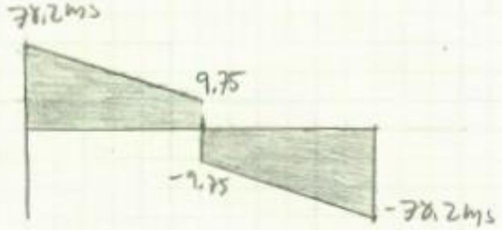
    15% increase
  - $V_c = 2\sqrt{f'c} \cdot b_w \cdot d$

    $= 32.6$ kips
  - $V_s = 8\sqrt{f'c} \cdot b_w \cdot d$

    $= 130.5$ kips
  - $\phi V_n = \phi(V_c + V_s)$

    $= 0.75(32.6 + 130.5)$

    $= 122$ kips > 58.65 beam size is adequate for shear
  - Shear Diagram



  - stirrup spacing

$$S_{max} = \min\begin{cases} d/2 \\ \text{or} \\ 24'' \end{cases} = \begin{cases} 21.5/2 = 10.75 \leftarrow \text{governs} \\ 24'' \end{cases} \quad \therefore 10''$$

assume #3 bars

$$S = \min\begin{cases} \dfrac{A_v f_{yt}}{50\, b_w} = 22'' \\ \dfrac{A_v f_{yt}}{0.75\sqrt{f'c} \cdot b_w} = 23'' \end{cases} \quad \therefore \max = 10''$$

- Required Spacing for Shear Force

$$s = \frac{A_v \cdot f_{yt} \cdot d}{V_u/\phi - V_c} = \frac{0.22 \cdot 60 \cdot 21.5}{78.2 - 32.6} = 6.22''$$

or

6"

- Location where 10" can be used

$$V_n = \frac{A_v \cdot f_{yt} \cdot d}{s} + V_c = \frac{0.22 \cdot 60 \cdot 21.5}{10} + 32.6$$

$$= 61 \text{ ks}$$

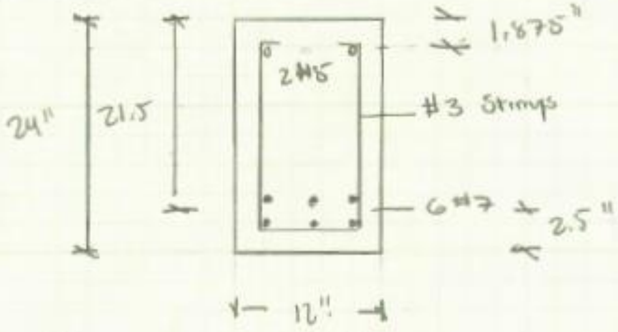
$$x = \frac{78.2 - \overset{V_c/2}{16.3}}{78.2 - 4.75} \cdot 180 \qquad x = 135.6$$

or 136"

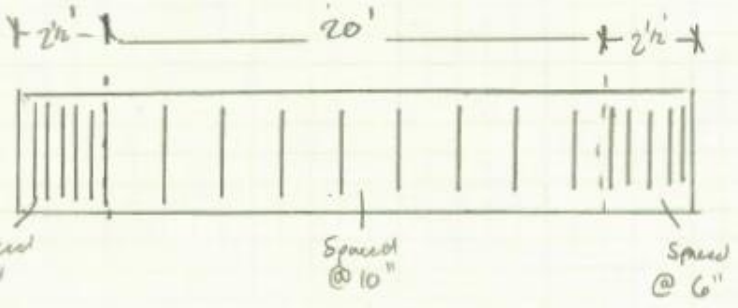
after 2½'!

Details for G1

Section

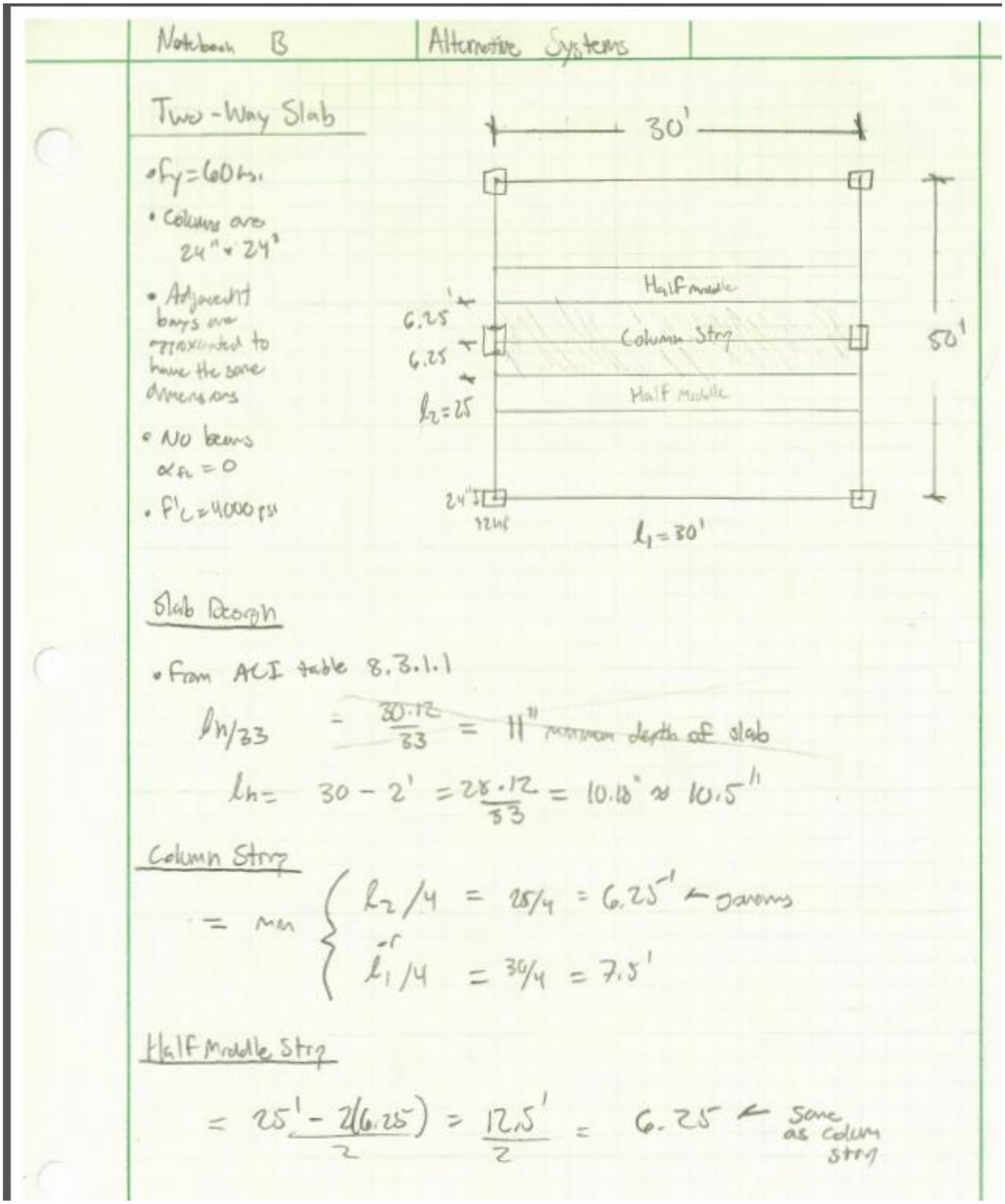


Elevation



4.3 Two Way Slab

A two way slab system is more appropriate than a one way slab system given the geometry of the bay. This system was designed by determining the column and half middle strip for each direction then designing the reinforcing steel to support the negative and positive moment at different points along each strip. One way and two way shear was also determined along with the shear due to the transfer of the moments in the slab. The design was a 10.5" thick slab with an f'c of 4000 psi. Reinforcing steel was #9 at 12" top and bottom at a location of 7' away from supports, everywhere else had #5 at 12" top and bottom.



Notebook B | Alternative Systems

Two-Way Slab

- $f_y = 60$ ksi
- Columns are 24" × 24"
- Adjacent bays are approximated to have the same dimensions
- No beams $\alpha_{fL} = 0$
- $f'_c = 4000$ psi

Slab Design

- From ACI table 8.3.1.1

~~$\ell_n/33 = \frac{30 \cdot 12}{33} = 11''$ minimum depth of slab~~

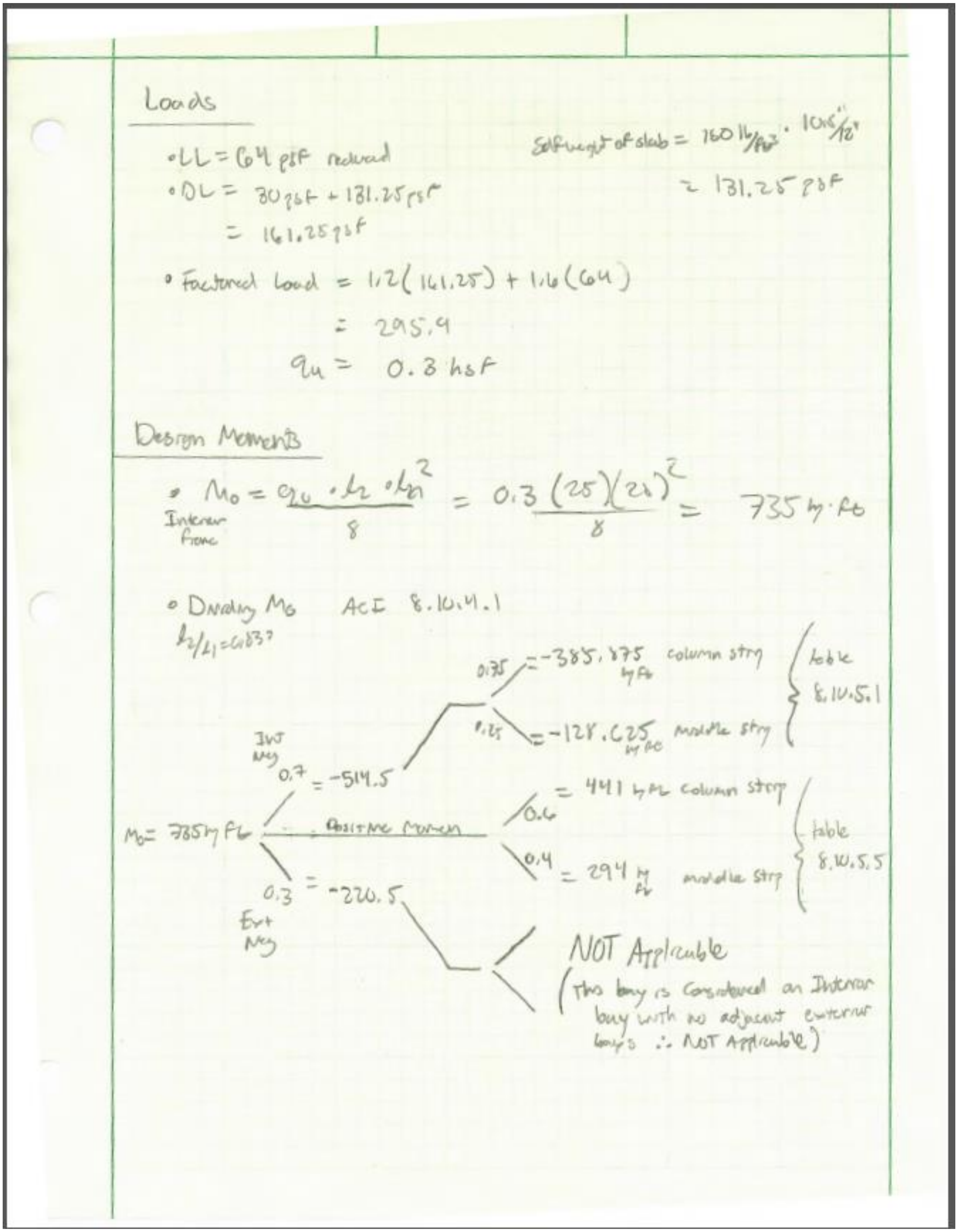
$\ell_n = 30 - 2' = \frac{28 \cdot 12}{33} = 10.18'' \approx 10.5''$

Column Strip

$= \min \begin{cases} \ell_2/4 = 25/4 = 6.25' \leftarrow \text{governs} \\ \text{or} \\ \ell_1/4 = 30/4 = 7.5' \end{cases}$

Half Middle Strip

$= \frac{25' - 2(6.25)}{2} = \frac{12.5'}{2} = 6.25 \leftarrow$ same as column strip



Loads

- LL = 64 psf reduced
- DL = 30 psf + 131.25 psf
  = 161.25 psf

Selfweight of slab = $160 \text{ lb/ft}^3 \cdot 10.5''/12''$
≈ 131.25 psf

- Factored Load = 1.2(161.25) + 1.6(64)
  = 295.4

$q_u = 0.3$ ksf

Design Moments

- $$M_o = \frac{q_u \cdot l_2 \cdot l_n^2}{8} = 0.3\frac{(25)(21)^2}{8} = 735 \text{ k}\cdot\text{ft}$$

Interior frame

- Dividing M_o ACI 8.10.4.1

$l_2/l_1 = 0.833$

$M_o = 735$ k ft

Int Neg 0.7 = −514.5

0.75 = −385.875 k ft column strip

0.25 = −128.625 k ft middle strip

table 8.10.5.1

Positive Moment

0.6 = 441 k ft column strip

0.4 = 294 k ft middle strip

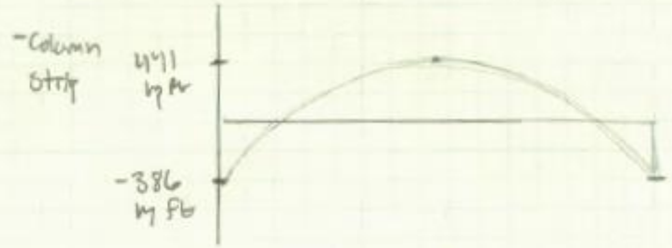
table 8.10.5.5

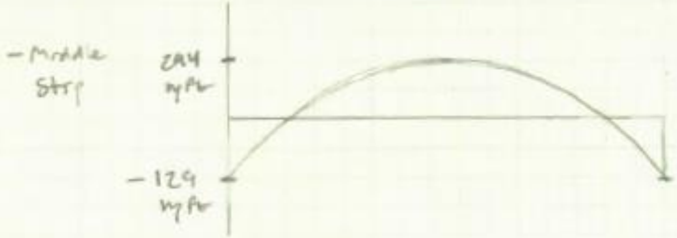
Ext Neg 0.3 = −220.5

NOT Applicable

(This bay is considered an Interior bay with no adjacent exterior bays ∴ NOT Applicable)

- Moment Diagrams





Reinforcement

- ACI 8.6.1.1

$A_{s,min} = 0.0018\,b \cdot h$ ∴ for 1' section

$A_{s,min} = 0.0018(12)(10.5)$

$= 0.2268 \text{ in}^2/\text{ft}$

- ACI 8.7.2.2

Spacing max = 2·slab thickness
= 21"

Spacing max = 18" ← governs

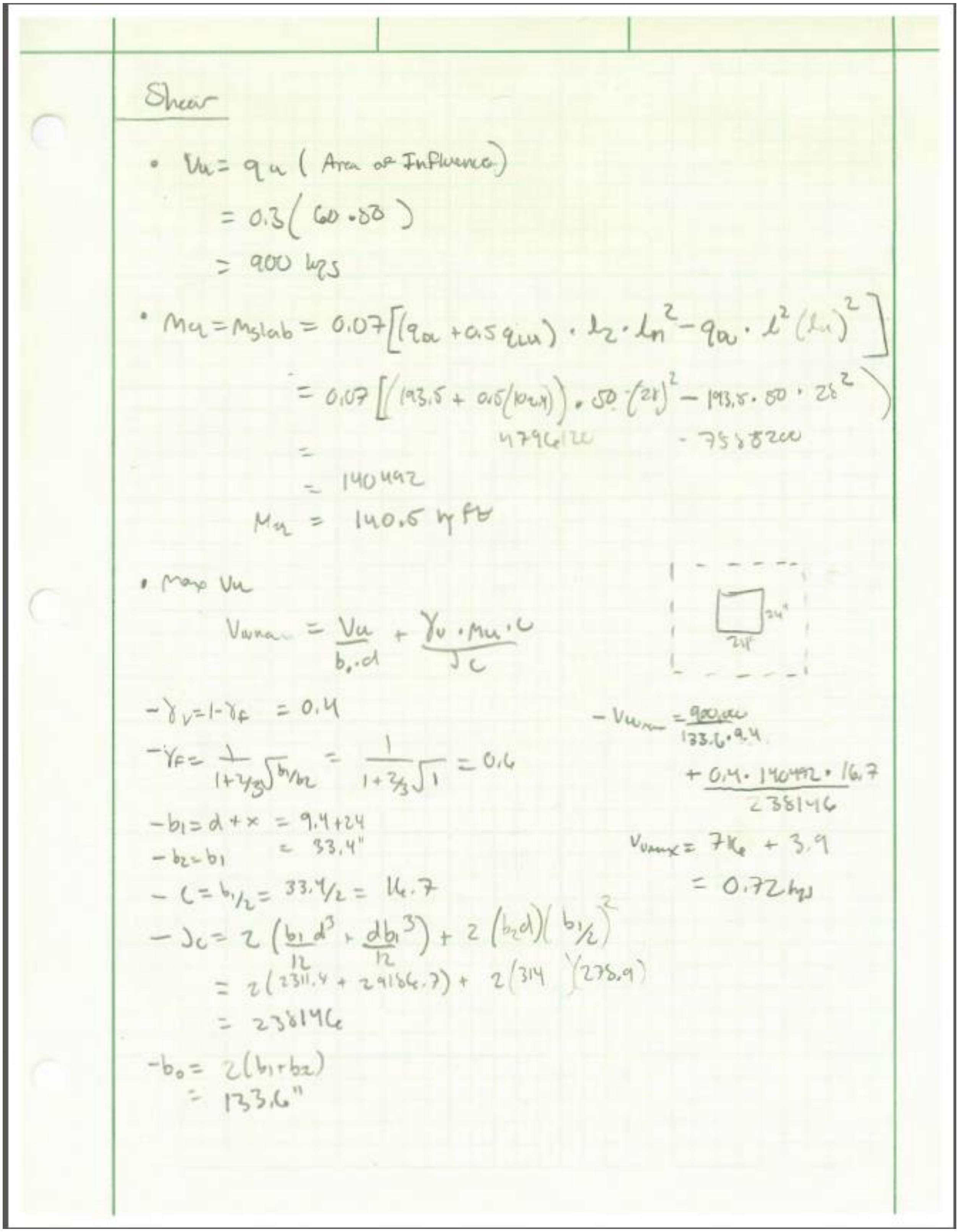
∴ choose #5 bars @ 12"

- $A_{s,req} = \dfrac{M_u}{\phi f_y \cdot jd} = \dfrac{441 \cdot 12''/\text{ft}}{0.95(9.4)(60) \cdot 12''/\text{ft strip}}$ where $jd = 0.95d$

$= 0.82 \text{ in}^2 > A_{s,min}$

∴ use #9 bars @ 12"

d = slab thickness − 0.75" − 0.5 d_b
= 10.5 − 0.75 − 0.5 (0.625)
= 9.4



Shear

- $V_u = q_u$ (Area of Influence)

$= 0.3(60 \cdot 50)$

$= 900$ kips

- $M_u = M_{slab} = 0.07\left[(q_{Du} + 0.5\,q_{Lu}) \cdot l_2 \cdot l_n^2 - q_{Du} \cdot l^2 (l_n)^2\right]$

$= 0.07\left[\left(193.5 + 0.5(102.4)\right) \cdot 50 \cdot (28)^2 - 193.5 \cdot 50 \cdot 28^2\right)$

4796120 -7588200

$=$

$= 140492$

$M_u = 140.5$ k ft

- Max V_u

$$V_{max} = \frac{V_u}{b_o \cdot d} + \frac{\gamma_v \cdot M_u \cdot c}{J_c}$$

24"

24"

$-\gamma_v = 1 - \gamma_f = 0.4$

$-\gamma_f = \frac{1}{1 + 2/3\sqrt{b_1/b_2}} = \frac{1}{1 + 2/3\sqrt{1}} = 0.6$

$-b_1 = d + x = 9.4 + 24$

$-b_2 = b_1 \quad = 33.4"$

$-c = b_1/2 = 33.4/2 = 16.7$

$-J_c = 2\left(\frac{b_1 d^3}{12} + \frac{d b_1^3}{12}\right) + 2(b_2 d)\left(b_1/2\right)^2$

$= 2(2311.4 + 29186.7) + 2(314)(278.9)$

$= 238146$

$-b_o = 2(b_1 + b_2)$

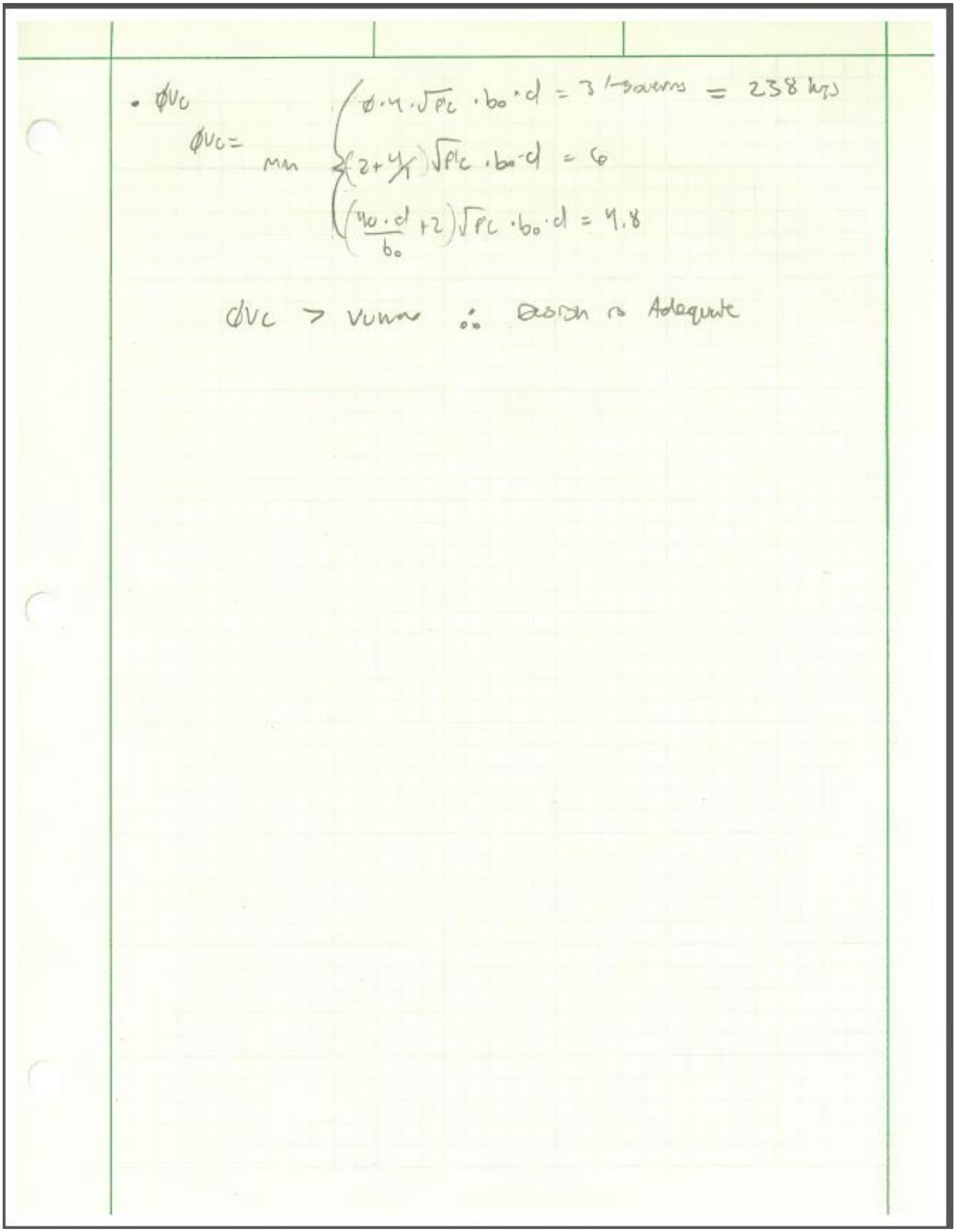
$= 133.6"$

$- V_{umax} = \frac{900{,}000}{133.6 \cdot 9.4}$

$+ \frac{0.4 \cdot 140492 \cdot 16.7}{238146}$

$V_{umax} = 716 + 3.9$

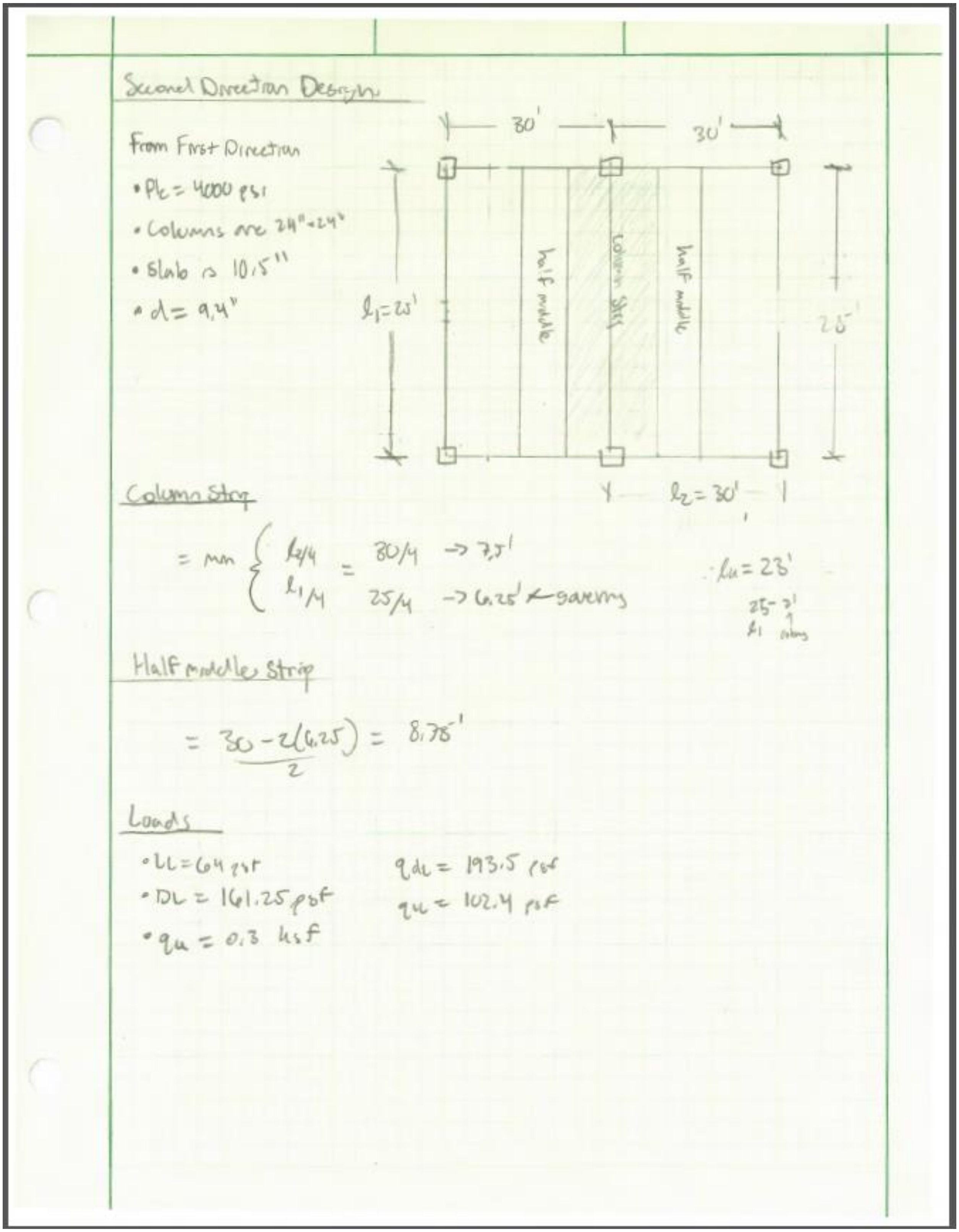
$= 0.72$ kips



- ϕV_c

$$\phi V_c = \min \begin{cases} \phi \cdot 4 \cdot \sqrt{f'c} \cdot b_o \cdot d = 3 \leftarrow \text{governs} = 238 \text{ kips} \\ \left(2 + \cancel{4/1}\right)\sqrt{f'c} \cdot b_o \cdot d = 6 \\ \left(\frac{40 \cdot d}{b_o} + 2\right)\sqrt{f'c} \cdot b_o \cdot d = 4.8 \end{cases}$$

$\phi V_c > V_{u\,max}$ ∴ Design is Adequate



Second Direction Design

From First Direction

- $f'_c = 4000$ psi
- Columns are 24"×24"
- Slab is 10.5"
- $d = 9.4"$

Column Strip

$$= \min \begin{cases} \ell_2/4 = 30/4 \rightarrow 7.5' \\ \ell_1/4 \quad 25/4 \rightarrow 6.25' \leftarrow \text{governs} \end{cases}$$

$\ell_u = 23'$

$\underset{\ell_1}{25} - \underset{\uparrow \text{colms}}{2'}$

Half middle strip

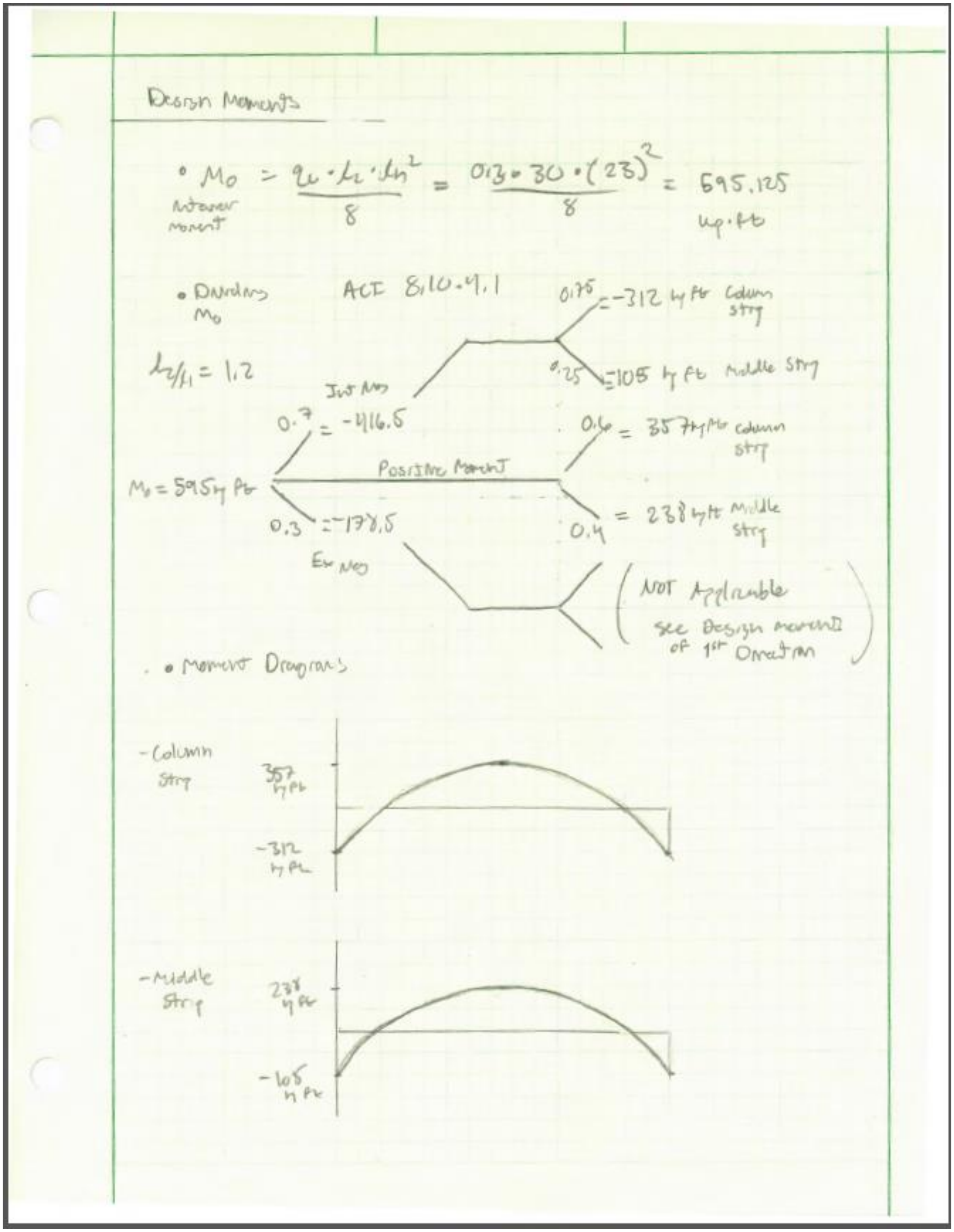
$$= \frac{30 - 2(6.25)}{2} = 8.75'$$

Loads

- LL = 64 psf
- DL = 161.25 psf
- $q_u = 0.3$ ksf

$q_{dl} = 193.5$ psf

$q_u = 102.4$ psf



Design Moments

• $M_0 = \frac{q_u \cdot l_2 \cdot l_n^2}{8} = \frac{0.3 \cdot 30 \cdot (23)^2}{8} = 595.125$ kip·ft

Interior moment

• Dividing M_0 ACI 8.10.4.1

$l_2/l_1 = 1.2$

$M_0 = 595$ kip ft

Int Neg 0.7 = −416.5

0.75 = −312 kip ft Column strip

0.25 = 105 kip ft Middle strip

Positive Moment

0.6 = 357 kip ft Column strip

0.4 = 238 kip ft Middle strip

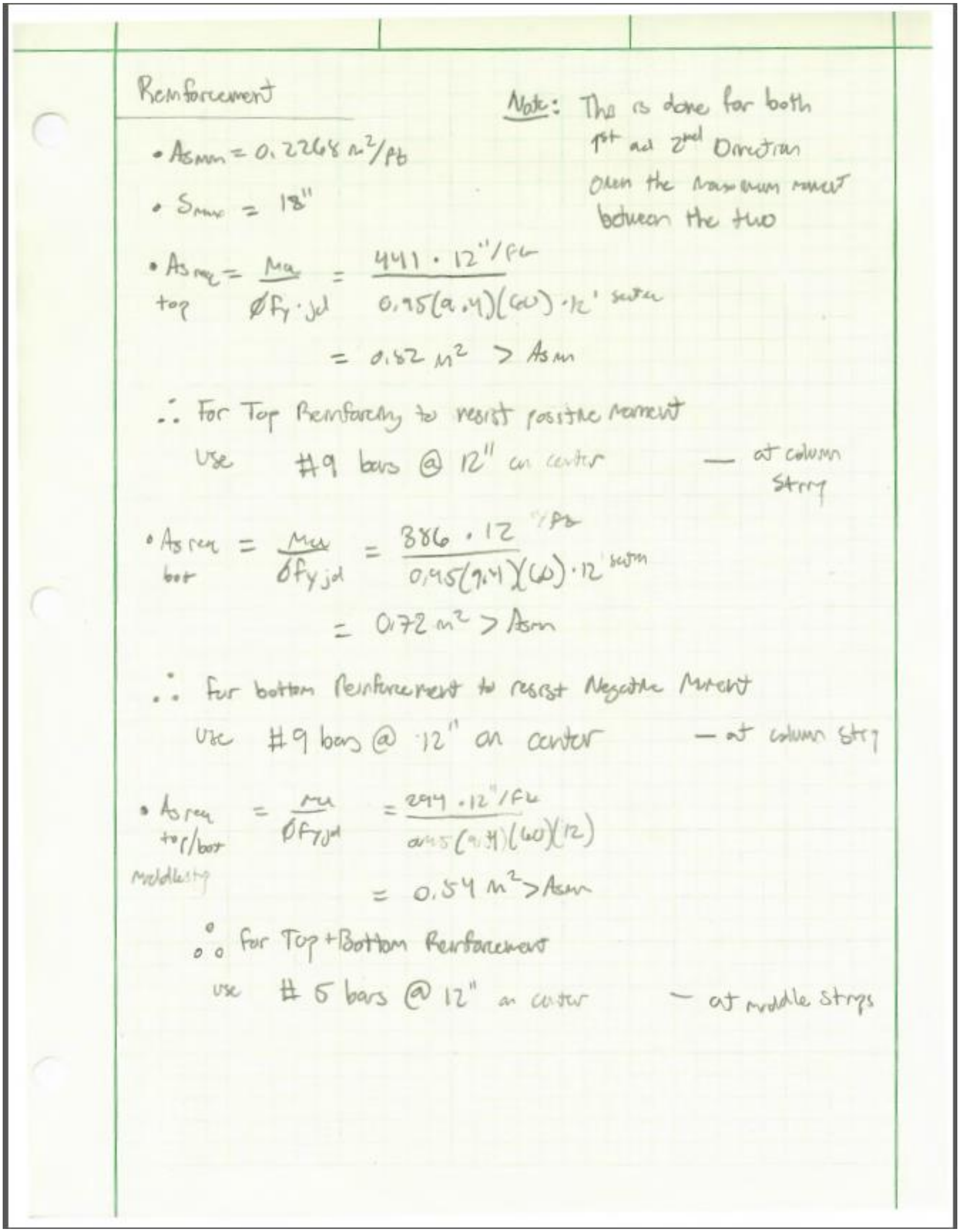
Ext Neg 0.3 = −178.5

(Not Applicable see Design moments of 1st Orientation)

• Moment Diagrams

- Column strip

- Middle strip



Reinforcement

Note: This is done for both 1st and 2nd Direction given the maximum moment between the two

- $A_{s\,min} = 0.2268\ in^2/ft$
- $S_{max} = 18''$
- $A_{s\,req\ top} = \frac{M_u}{\phi f_y \cdot jd} = \frac{441 \cdot 12''/ft}{0.95(9.4)(60) \cdot 12'\ \text{section}}$

$$= 0.82\ in^2 > A_{s\,min}$$

∴ For Top Reinforcing to resist positive moment

use #9 bars @ 12" on center — at column strip

- $A_{s\,req\ bot} = \frac{M_u}{\phi f_y jd} = \frac{386 \cdot 12\ ''/ft}{0.95(9.4)(60) \cdot 12'\ \text{section}}$

$$= 0.72\ in^2 > A_{s\,min}$$

∴ for bottom Reinforcement to resist Negative Moment

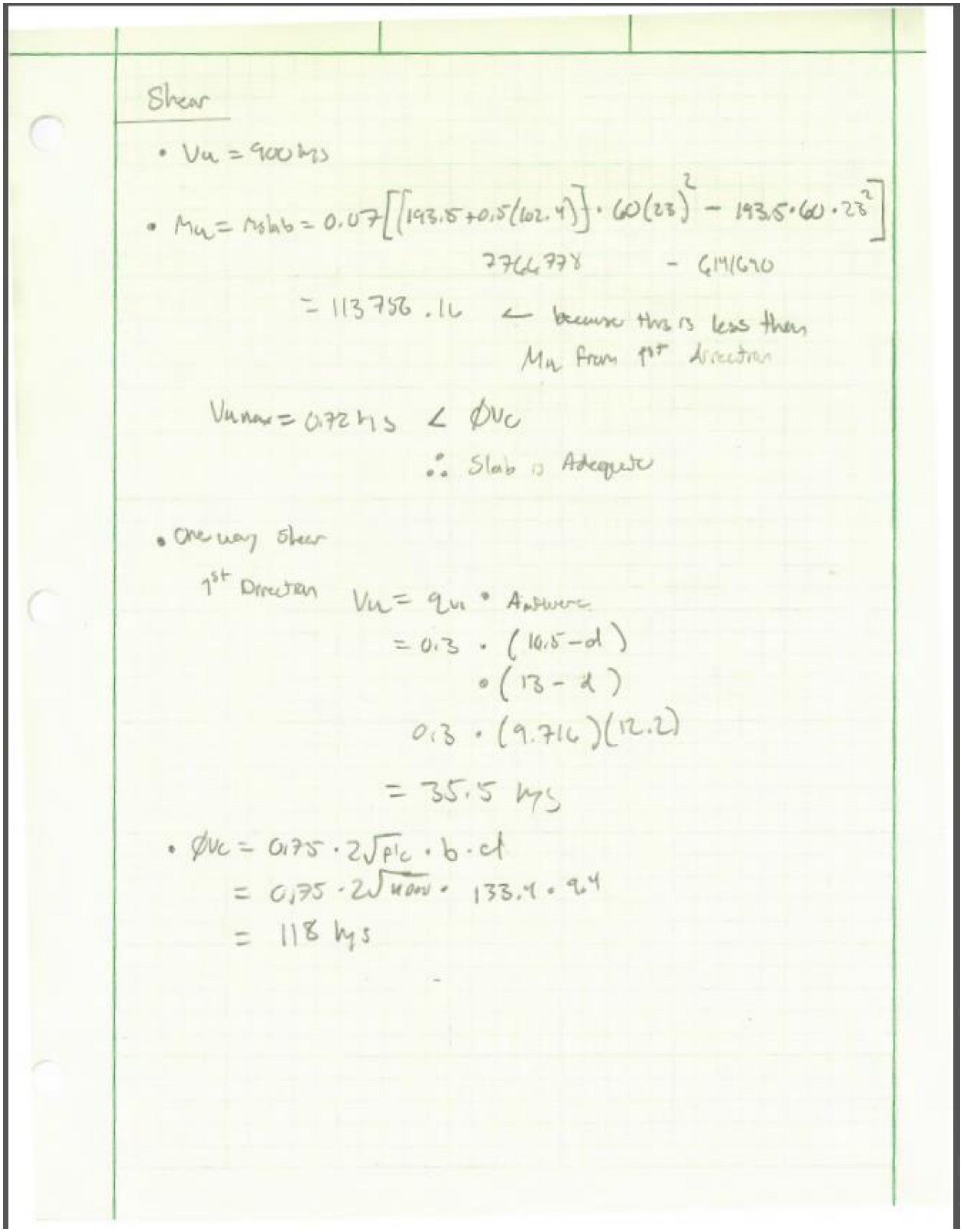
use #9 bars @ 12" on center — at column strip

- $A_{s\,req\ top/bot\ \text{middle strip}} = \frac{M_u}{\phi f_y jd} = \frac{294 \cdot 12''/ft}{0.95(9.4)(60)(12)}$

$$= 0.54\ in^2 > A_{s\,min}$$

∴ for Top + Bottom Reinforcement

use # 5 bars @ 12" on center — at middle strips



Shear

- $V_u = 900$ kips
- $M_u = M_{slab} = 0.07\left[\left[193.5 + 0.5(62.4)\right] \cdot 60(23)^2 - 193.5 \cdot 60 \cdot 23^2\right]$

$7766778 \quad - 6141690$

$= 113756.16$ ← because this is less than M_u from 1st direction

$V_{u\,max} = 0.72$ kips $< \phi V_c$

∴ Slab is Adequate

- One way shear

1st Direction $V_u = q_u \cdot A_{tributary}$

$= 0.3 \cdot (10.5 - d) \cdot (13 - d)$

$0.3 \cdot (9.716)(12.2)$

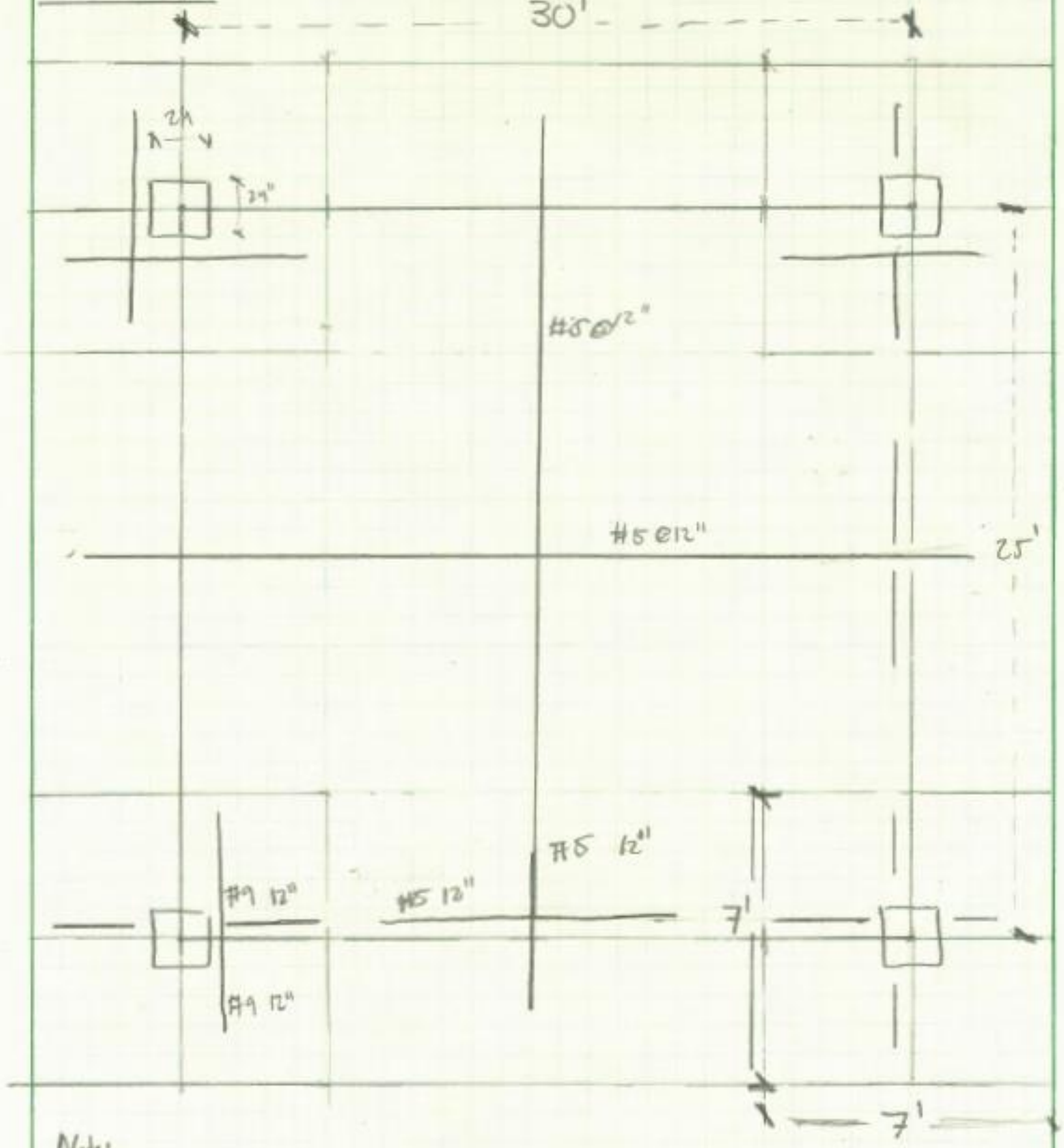
$= 35.5$ kips

- $\phi V_c = 0.75 \cdot 2\sqrt{f'c} \cdot b \cdot d$

$= 0.75 \cdot 2\sqrt{4000} \cdot 133.4 \cdot 9.4$

$= 118$ kips

Detail

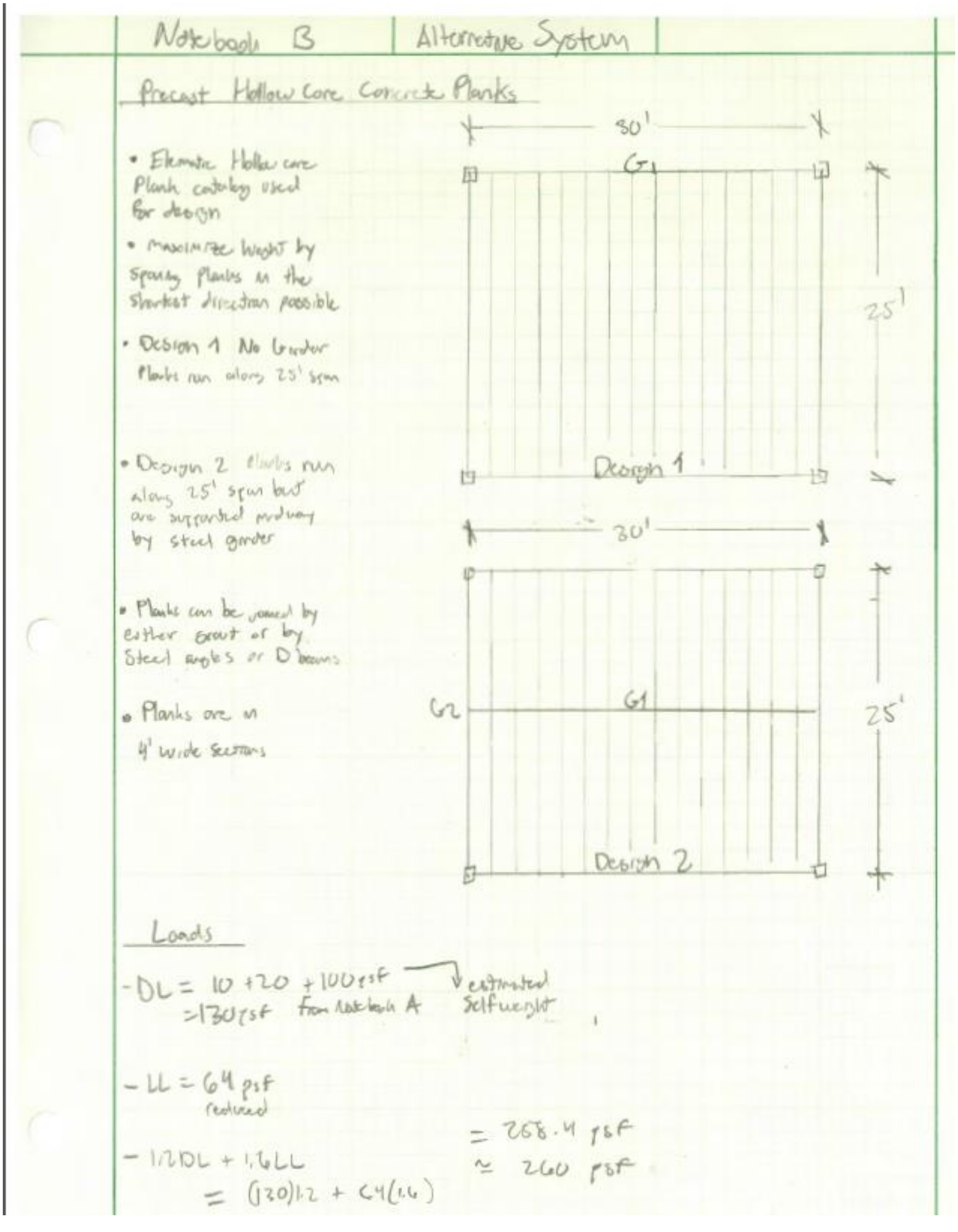


Notes

- Columns are 24"x24"
- Top Reinforcing is #9 @12" from 7' away from column, #5@12" EE
- Bottom Reinforcing is #9@12" from 7' away from column, #5@12" EE
- f'_c = 4000 psi

4.4 Precast hollow Core Concrete Plank

A system of precast prestressed hollow core concrete planks was chosen for the last alternative system. Two different layouts were chosen for design. The main difference between the layouts is the span of the planks. In the first layout the planks span the entire width of the bay. In the second the planks span half the width and are supported by a steel wide flange beam. The Elematic Hollow Core Plank Catalog was used to determine the moment capacity along with the live load deflection limit per plank. D- Beams were not designed in this system, the planks can be interlocked together through their own geometry and through grout.



Notebook B | Alternative System

Precast Hollow Core Concrete Planks

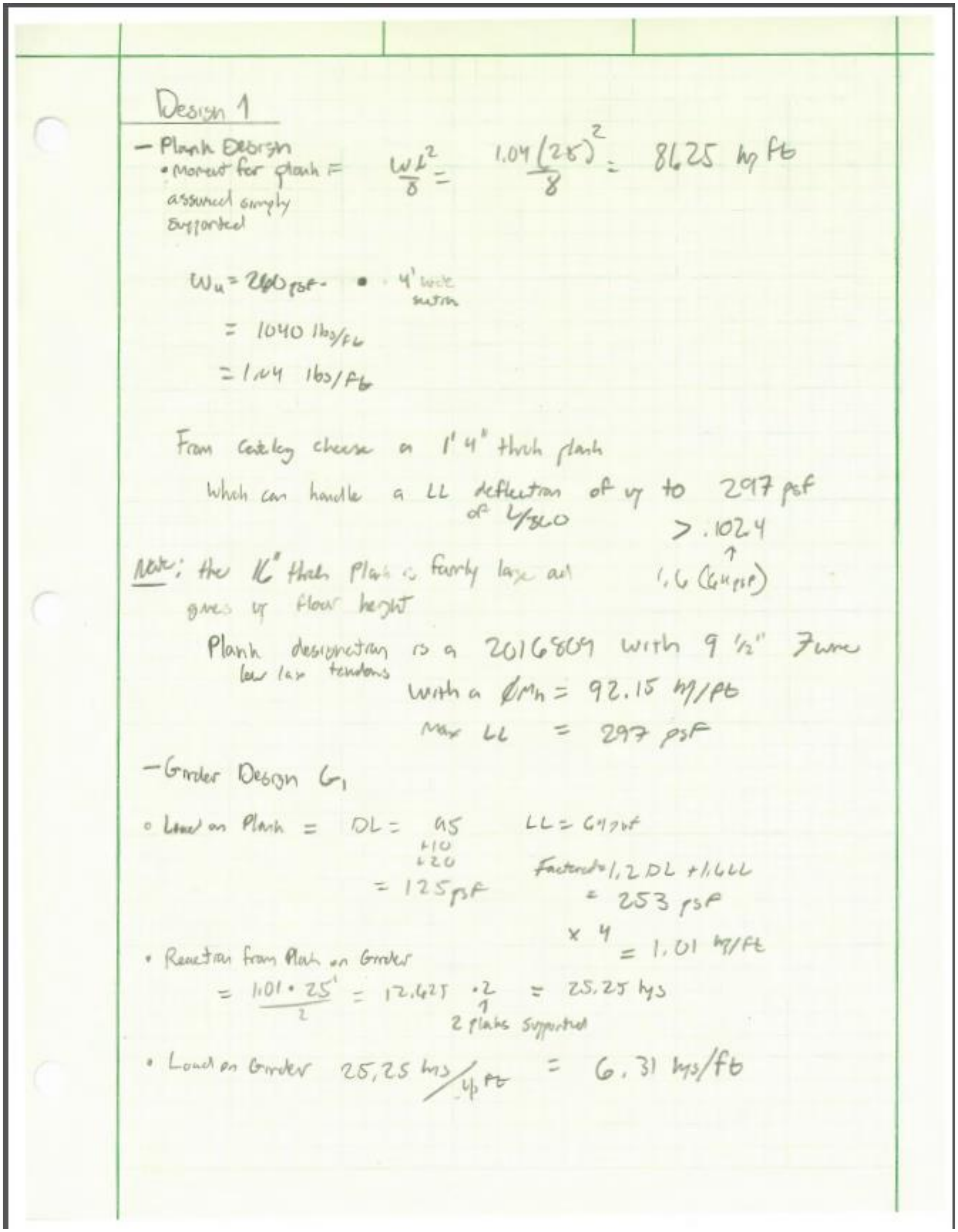
- Elematic Hollow core Plank catalog used for design
- maximize weight by spanning planks in the shortest direction possible
- Design 1 No Girder
  Planks run along 25' span
- Design 2 Planks run along 25' span but are supported midway by steel girder
- Planks can be joined by either grout or by Steel angles or D beams
- Planks are in 4' wide sections

Loads

- DL = 10 + 20 + 100 psf → estimated selfweight
  = 130 psf from Notebook A

- LL = 64 psf
  reduced

- 1.2DL + 1.6LL
  = (130)1.2 + 64(1.6)
  = 258.4 psf
  ≃ 260 psf



Design 1

– Plank Design

• Moment for plank = $\frac{wL^2}{8} = \frac{1.04(28)^2}{8} = 86.25$ k ft

assumed simply supported

W_u = 260 psf · 4' wide section

= 1040 lbs/ft

= 1.04 lbs/ft

From Catalog choose a 1' 4" thick plank

Which can handle a LL deflection of L/360 of up to 297 psf

> .1024

↑

1.6 (64 psf)

Note: the 16" thick Plank is fairly large and gives up floor height

Plank designation is a 2016809 with 9 ½" 7 wire low lax tendons

with a ϕM_n = 92.15 k/ft

Max LL = 297 psf

– Girder Design G_1

∘ Load on Plank = DL = 95 +10 +20 = 125 psf

LL = 64 psf

Factored = 1.2 DL + 1.6 LL = 253 psf

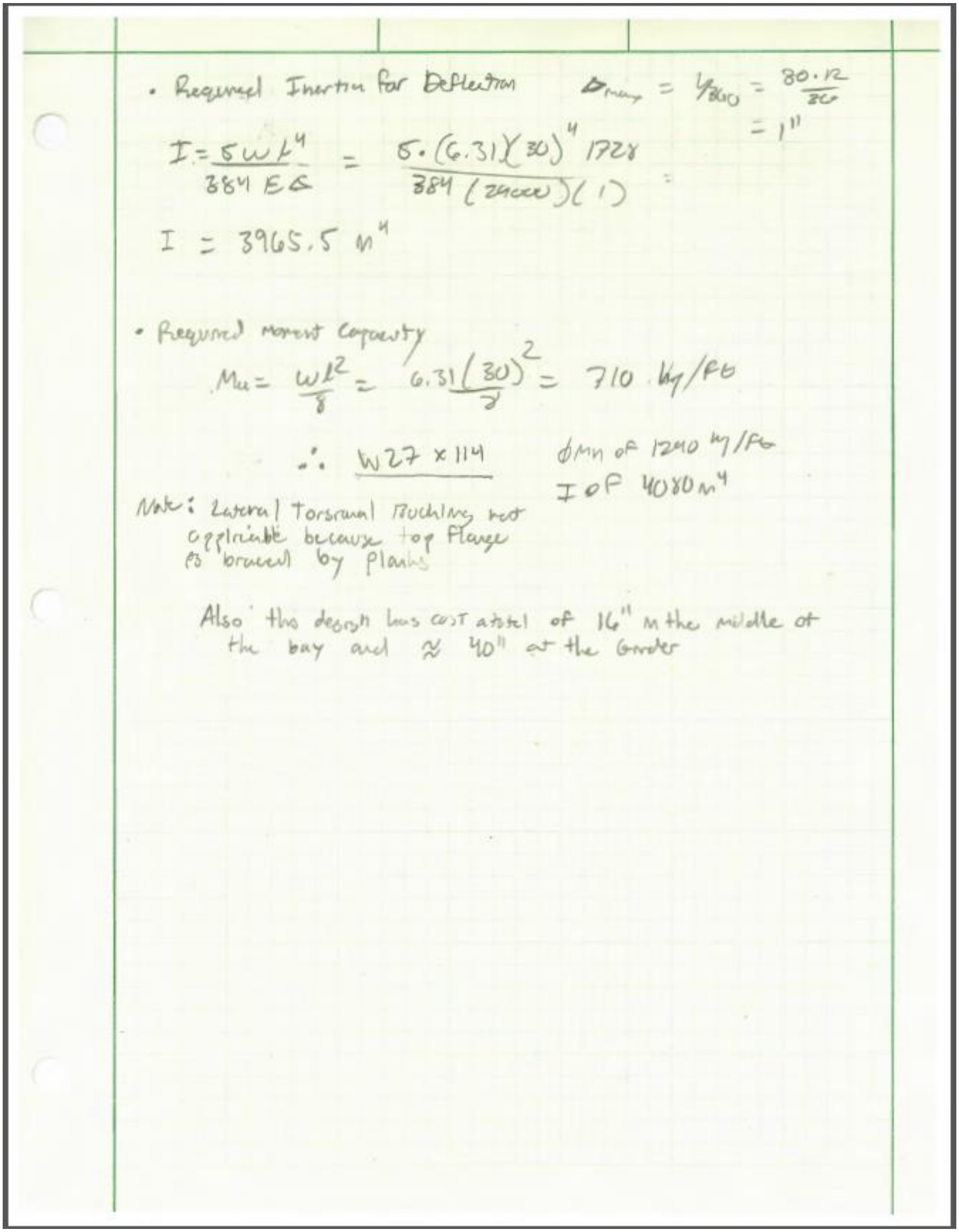
× 4 = 1.01 k/ft

• Reaction from Plank on Girder

$= \frac{1.01 \cdot 25'}{2} = 12.625 \cdot 2 = 25.25$ kips

2 planks supported

• Load on Girder $25.25 \text{ kips} / 4 \text{ ft} = 6.31$ kips/ft



- Required Inertia for Deflection

$\Delta_{max} = l/360 = \frac{30 \cdot 12}{360}$

$= 1''$

$$I = \frac{5 w l^4}{384 E \Delta} = \frac{5 \cdot (6.31)(30)^4 \, 1728}{384 (29000)(1)} =$$

$$I = 3965.5 \text{ in}^4$$

- Required moment capacity

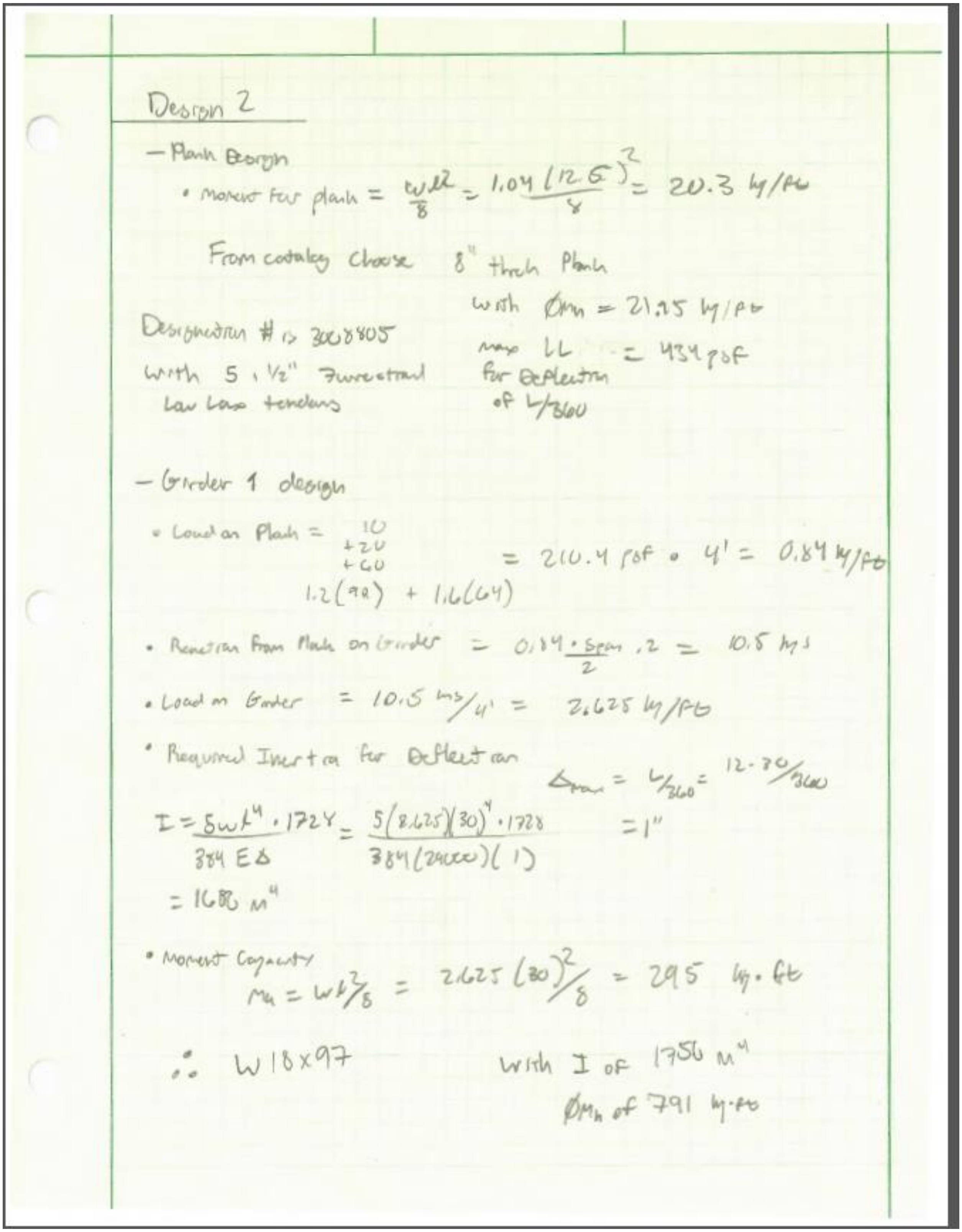
$$M_u = \frac{w l^2}{8} = \frac{6.31 (30)^2}{8} = 710 \text{ k/ft}$$

$\therefore$ W27 x 114 ϕM_n of 1240 k/ft

I of 4080 in^4

Note: Lateral Torsional Buckling not applicable because top flange is braced by planks

Also this design has cost a total of 16" in the middle of the bay and ≈ 40" at the Girder



Design 2

— Plank Design

- Moment for plank = $\frac{w\ell^2}{8} = \frac{1.04(12.5)^2}{8} = 20.3$ k/ft

From catalog choose 8" thick Plank

with $\phi M_n = 21.75$ k/ft

max LL = 434 psf

for Deflection

of L/360

Designation # is 3008805

with 5, 1/2" 7wire strand

Low Loss tendons

— Girder 1 design

- Load on Plank = 10 +20 +60 $= 210.4$ psf · 4' = 0.84 k/ft

$1.2(90) + 1.6(64)$

- Reaction from Plank on Girder $= 0.84 \cdot \frac{\text{Span}}{2} , 2 = 10.5$ kips
- Load on Girder = 10.5 kips/4' = 2.625 k/ft
- Required Inertia for Deflection

$\Delta_{max} = L/360 = 12 \cdot 30/360 = 1''$

$$I = \frac{5wL^4 \cdot 1724}{384 E \Delta} = \frac{5(2.625)(30)^4 \cdot 1728}{384(29000)(1)}$$

$= 1688$ in^4

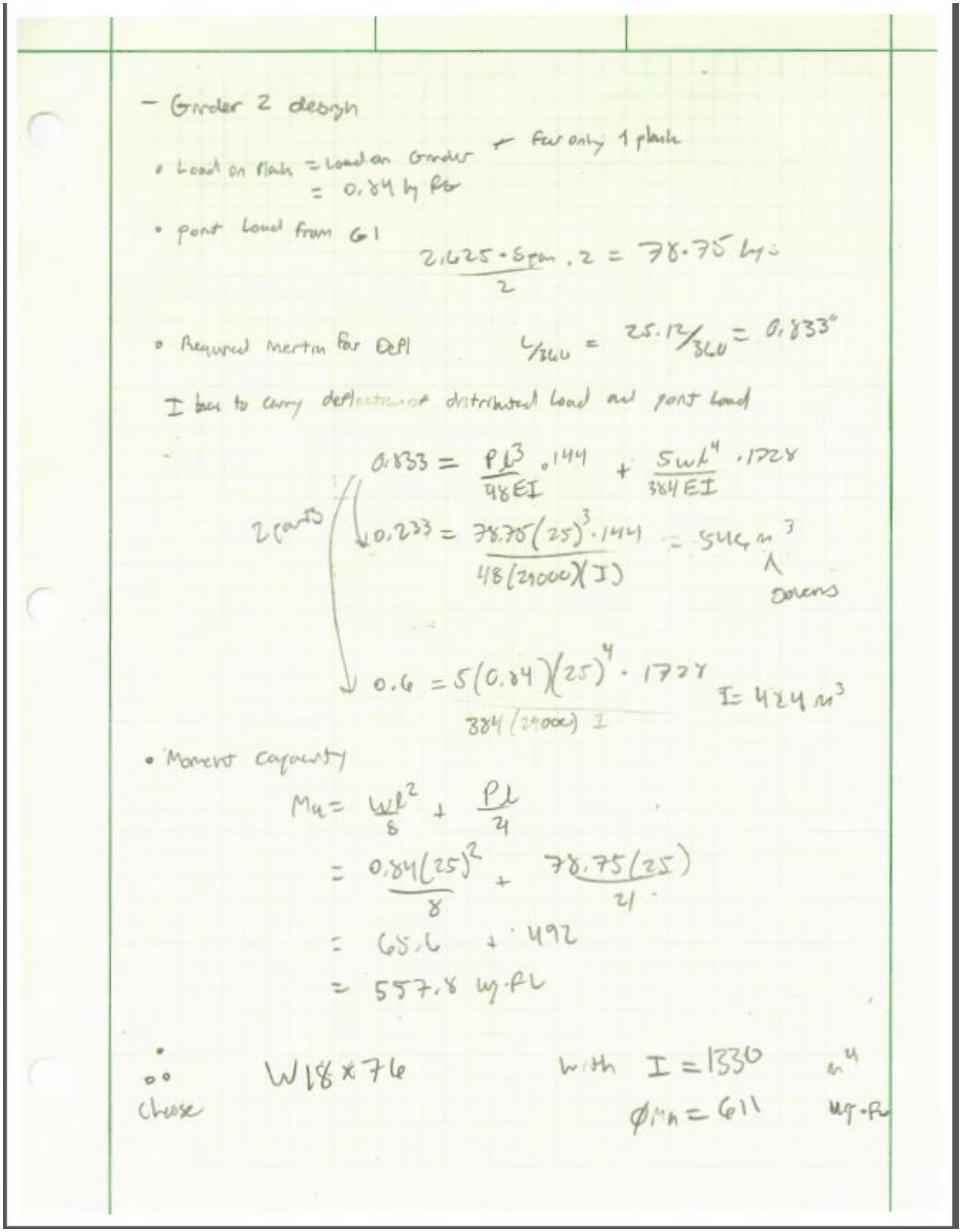
- Moment Capacity

$M_u = wL^2/8 = 2.625(30)^2/8 = 295$ k·ft

∴ W18x97

with I of 1750 in^4

ϕM_n of 791 k·ft



– Girder 2 design

- Load on Plank = Load on Girder ← For only 1 plank
  = 0.84 k/ft
- point Load from G1

$$\frac{2.625 \cdot \text{Span}}{2} \cdot 2 = 78.75 \text{ kips}$$

- Required Inertia for Defl $\quad L/360 = 25 \cdot 12/360 = 0.833''$

I has to carry deflection of distributed load and point load

$$0.833 = \frac{Pl^3}{48EI} \cdot 144 + \frac{5wl^4}{384EI} \cdot 1728$$

2 parts

$$0.233 = \frac{78.75(25)^3 \cdot 144}{48(29000)(I)} = 544 \text{ in}^3$$

^ Dozens

$$0.6 = \frac{5(0.84)(25)^4 \cdot 1728}{384(29000)\,I} \qquad I = 424 \text{ in}^3$$

- Moment capacity

$$M_u = \frac{wl^2}{8} + \frac{Pl}{4}$$

$$= \frac{0.84(25)^2}{8} + \frac{78.75(25)}{4}$$

$$= 65.6 + 492$$

$$= 557.8 \text{ kip}\cdot\text{ft}$$

∴ choose W18×76 with $I = 1330$ in⁴

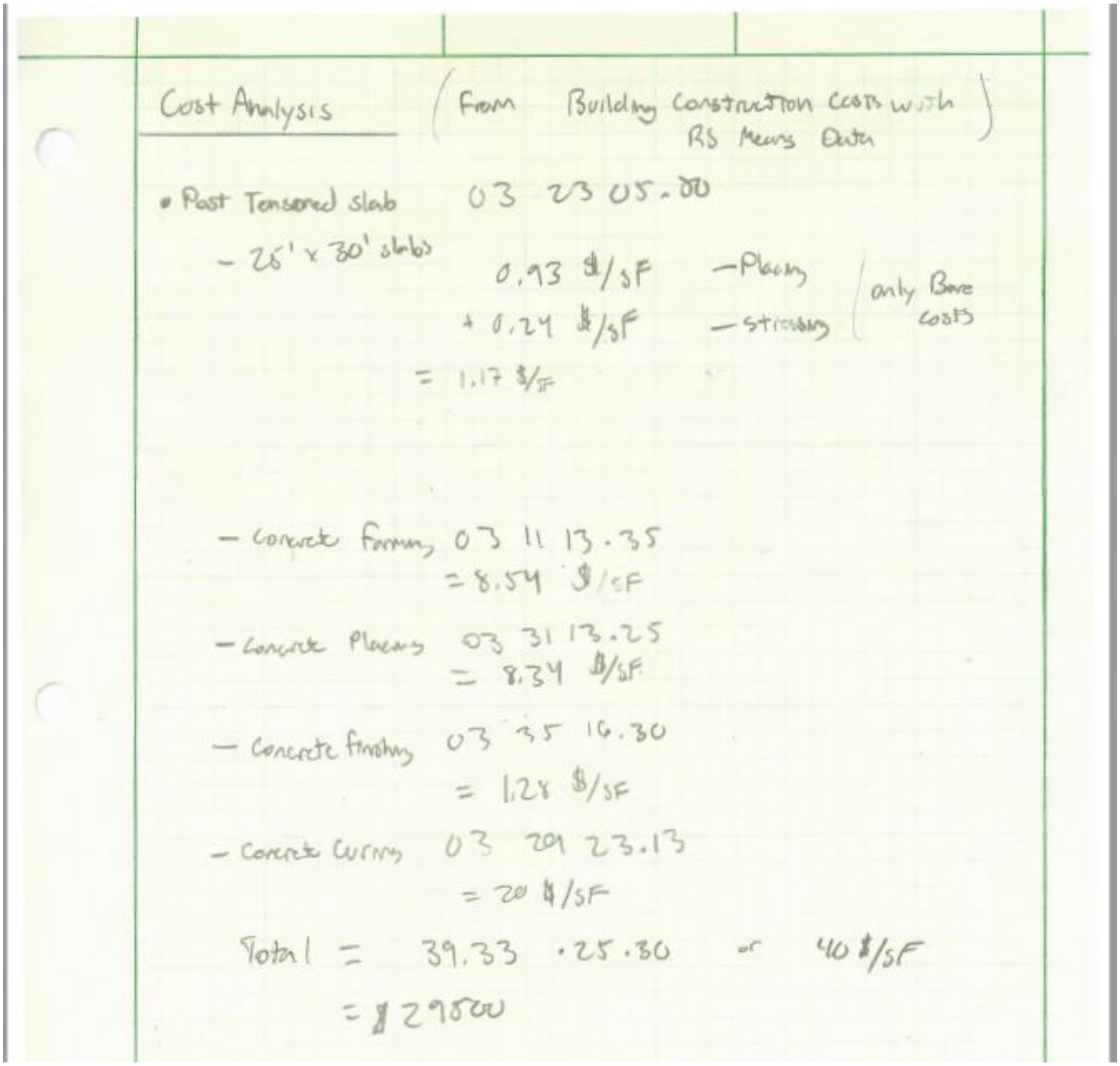
$\phi M_n = 611$ kip·ft

5. System Comparison

System	Height	Cost(per bay)	Notes
Post Tensioned Slab	-8.5” slab - 7.5” drop panels Total Height = 16”	$29,000	-complex analysis -involves only concrete subcontractors
Composite Metal Deck	-3.5” concrete slab -1.5” metal deck -12” beam -18” girder Total Height = 24” max	$33,000	-moderate analysis -high level of capacity -best for vibration control
One Way Slab	-6” slab -18” beam -24” girder Total Height = 30”	$34,250	-most expensive system -lowest floor to floor height
Two Way Slab	-10.5” reinforced slab -#9, #5 bars both ways top and bottom Total Height = 10.5”	$30,750	-low level of capacity - heavily dependent on reinforcing steel -best overall height
Hollow Core Planks (Design 2)	-8” plank -18” girder Total Height = 26”	$28,100	-simple analysis -involves multiple contractors of various trades

5.1 Cost Analysis

The following calculations are a simplified version of a detailed estimate. The quatities for each line item are roughly approximated and then multiplied by the base cost from the Building Construction Costs wit RS Means Data.



Cost Analysis (From Building Construction Costs with RS Means Data)

- Past Tensored slab 03 23 05.00
  - 25' x 30' slabs

0.93 \$/SF —Placing
\+ 0.24 \$/SF —stressing
= 1.17 \$/SF

(only Bare costs)

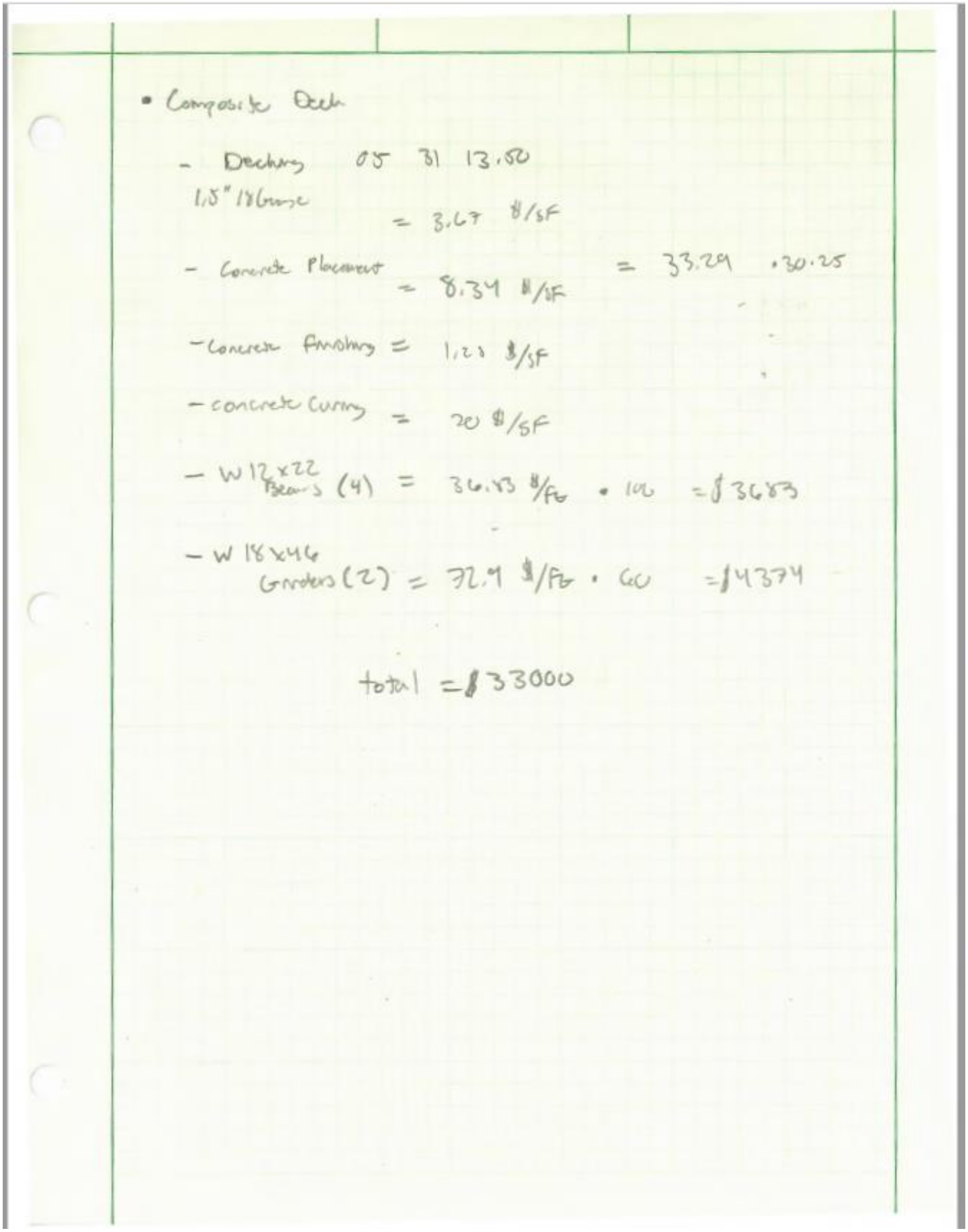
- Concrete forming 03 11 13.35
  = 8.54 \$/SF

- Concrete Placing 03 31 13.25
  = 8.34 \$/SF

- Concrete finishing 03 35 16.30
  = 1.28 \$/SF

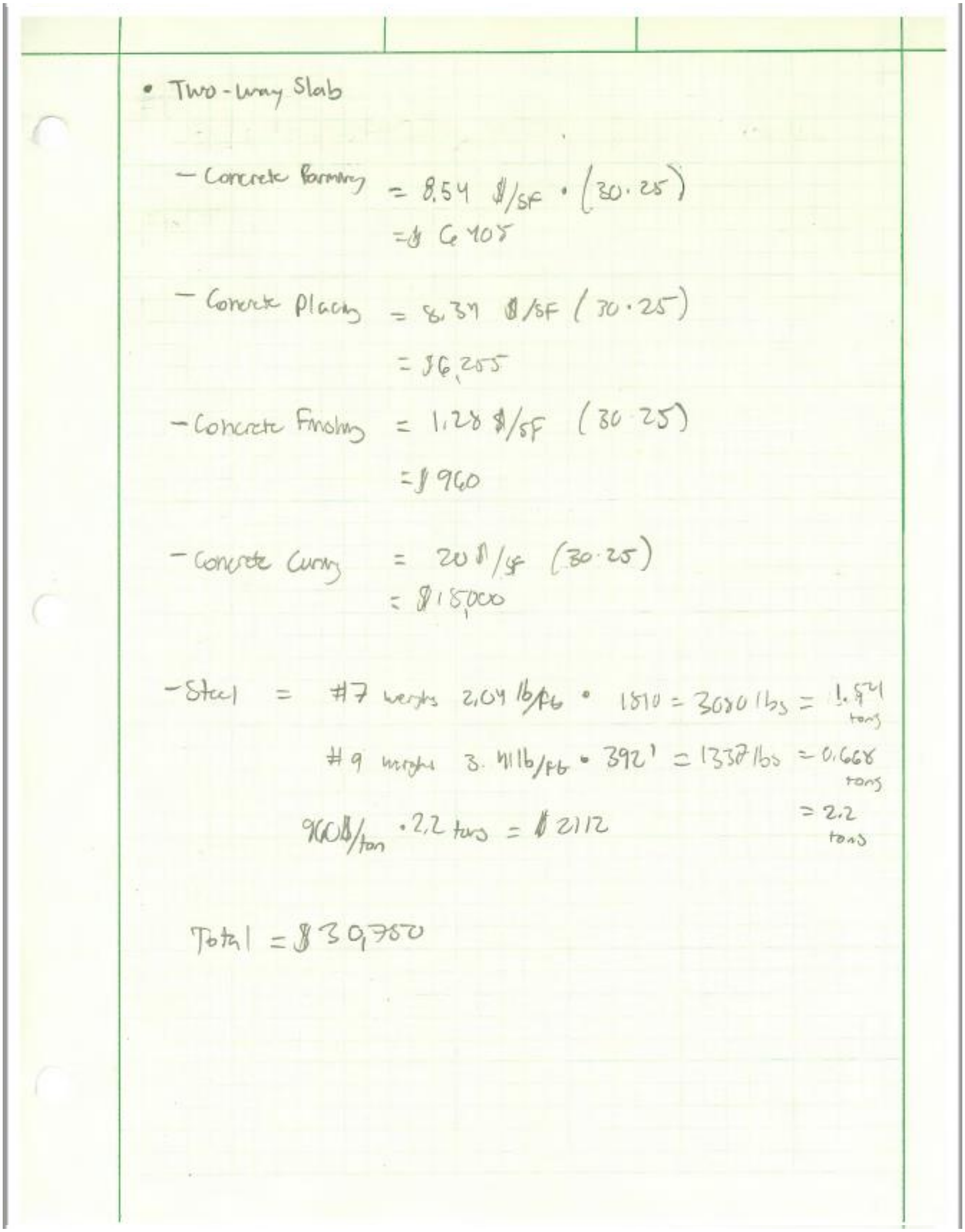
- Concrete Curing 03 39 23.13
  = 20 \$/SF

Total = 39.33 · 25 · 30 or 40 \$/SF
= \$ 29500



- Composite Deck
  - Decking 05 31 13.50
    1.5" 18 Gauge = 3.67 $/SF
  - Concrete Placement = 8.34 $/SF = 33.29 · 30.25
  - Concrete Finishing = 1.23 $/SF
  - Concrete Curing = 20 $/SF
  - W12x22 Beams (4) = 36.83 $/Ft · 100 = $3683
  - W18x46 Girders (2) = 72.9 $/Ft · 60 = $4374

total = $33000



- Two-way Slab

— Concrete Forming = 8.54 \$/SF · (30·25)
= \$ 6408

— Concrete Placing = 8.34 \$/SF (30·25)
= \$6,255

— Concrete Finishing = 1.28 \$/SF (30·25)
= \$ 960

— Concrete Curing = 20 \$/sf (30·25)
= \$15,000

— Steel = #7 weighs 2.04 lb/ft · 1810 = 3080 lbs = 1.54 tons
#9 weighs 3.41 lb/ft · 392' = 1337 lbs = 0.668 tons
= 2.2 tons
960 \$/ton · 2.2 tons = \$ 2112

Total = \$ 30,780

6. Lateral System Analysis

6.1 Modeling Information and Assumptions

The Modeling software used for the lateral analysis was ETABS 2016. A plan view of the lateral system for One City Center is shown below in Figure 3. There are a total of twelve shear walls that are the full height of the building. The compressive strength of these shear walls change throughout their height. Thus each separate compressive strength region was modeled as a different shell element with its respective compressive strength. Overall analysis did not include any of the below grade parking that the shear walls go into. This was dealt with by assuming a completely fixed support at the base of each shear wall due to how the building would behave in real life. The diaphragm is post tensioned concrete that was modeled as a rigid diaphragm that transfers all lateral load to the columns. It was not determined in this analysis weather or not the columns take any lateral load and thus they were not included in the model. Holes were put in each level of the diaphragm where there would be service elevators or stairwells. A 3D image of this model is shown in Figure 4.

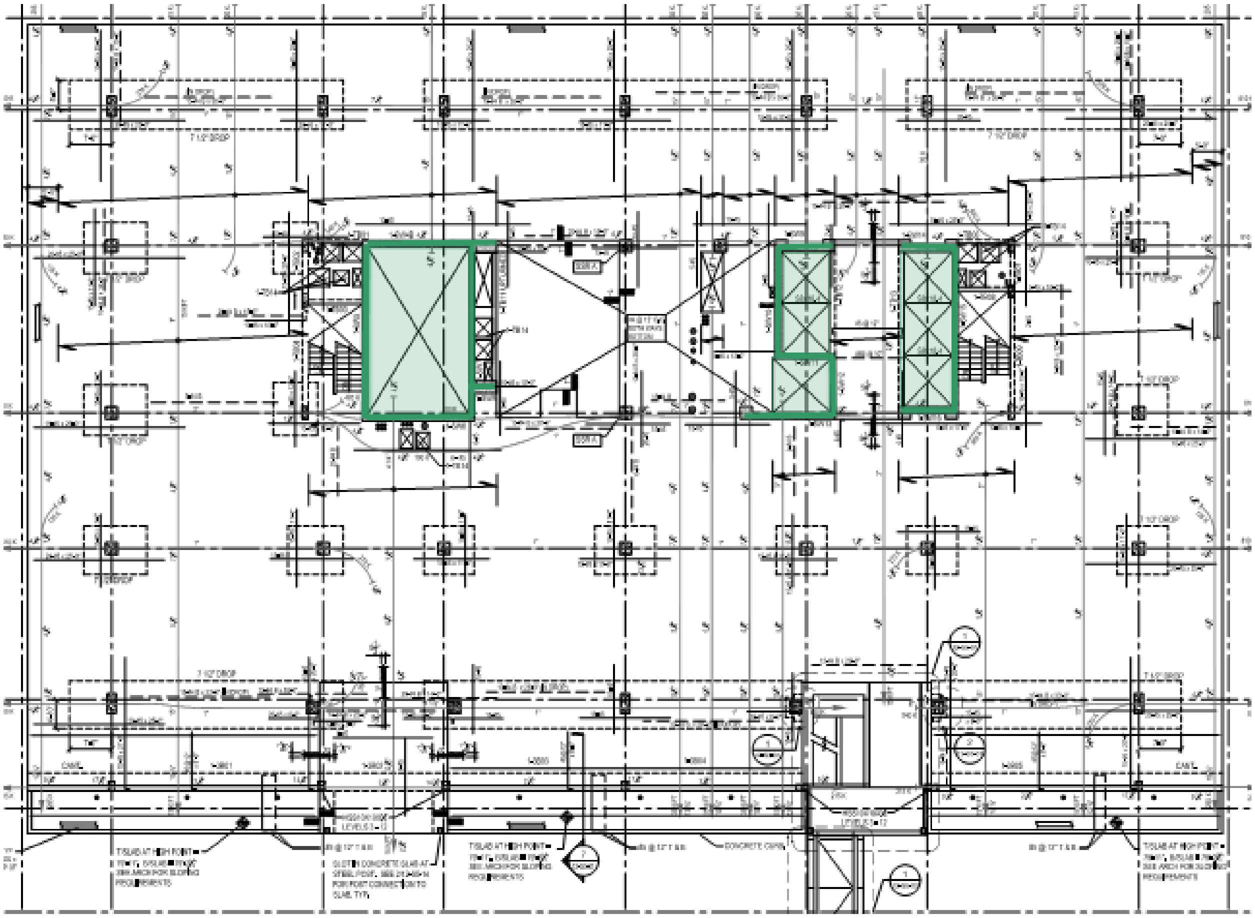


Figure 3: Plan view of the lateral system with shear walls shown in dark green.

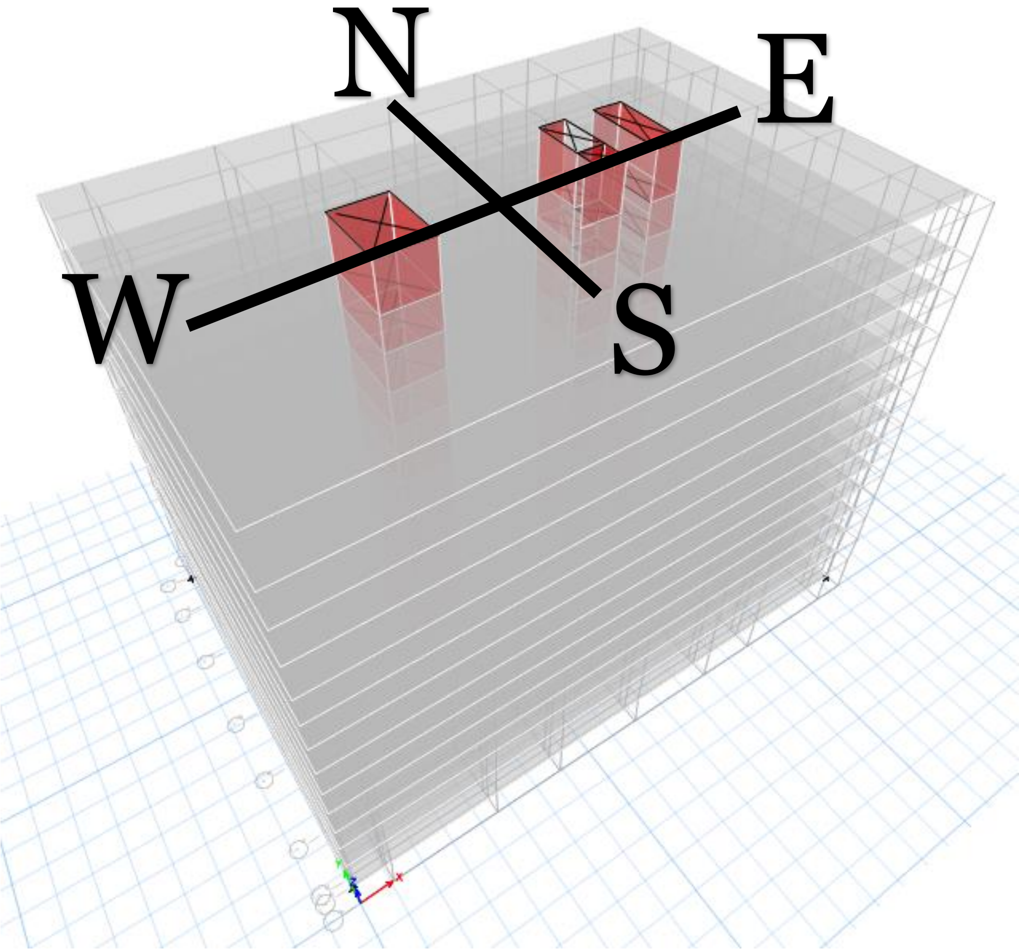


Figure 4: A 3D view of the lateral model from ETABS 2016

The Loading parameters used for this analysis was wind and seismic loads. In order to compare the loads obtained for wind and seismic, shown in section 2, parameter were entered into ETABS that would generate an additional set of lateral loads. These loads and their forces on each shear wall will later be compared in this analysis. Thus the loads on this model are not user defined but come from ASCE 7-10. The Seismic and Wind loads for the model were broken up into their X and Y components. This will better show the forces that go to the shear walls resisting N-S (Y) versus E-W (X).

6.2 Model Validation

In order to validate the lateral model three results were compared to hand calculations. These results were center of mass (COM), center of rigidity (COR) and story forces. It is important to note that the values for the model generated story forces were obtained by subtracting the story shears. As a result from this method the story forces at the top of each shear wall could not be determined and were therefore approximated to zero. It is known that this is not the actual case but was only used for comparison purposes. For the manual calculations both paper and excel sheets were used. For a better look at excel calculations see the separate excel sheet posted to the CPEP page.

The COM and COR found from both hand calculations and the ETABS model were fairly close to one another. As shown in Figure 5, the center of masses and center of rigidities are fairly close to one another. Their eccentricities are also a foot off from one another. Due to this similarity it can be assumed that the diaphragms and shear walls have the correct stiffness's and masses.

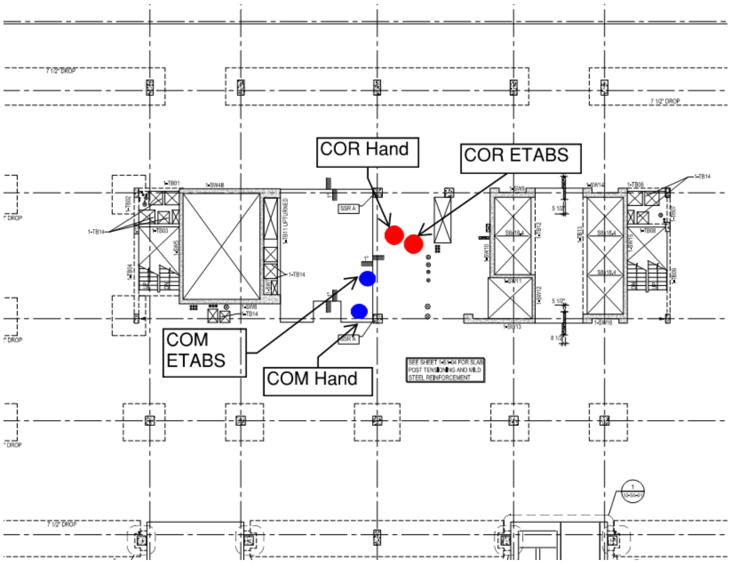
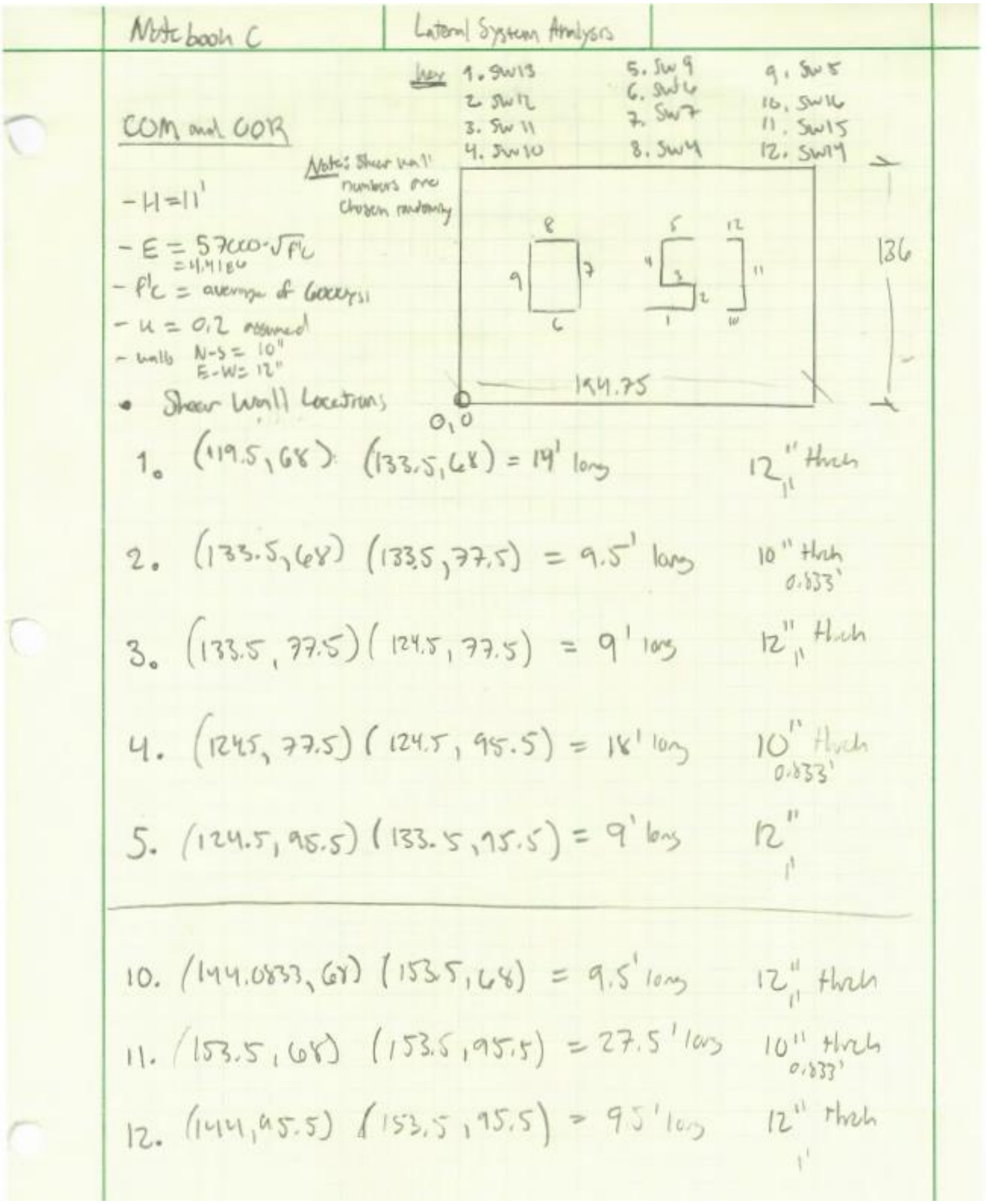


Figure 5: A plan view of the center of mass and center of rigidity generated from both hand calculations and ETABS model.

The other system of checks that was done to validate the ETABS model was a comparison of the story forces generated by hand to the story forces found in the ETABS model. For each instance the story forces were found for both wind and seismic in both N-S and E-W directions. It is important to note that the controlling wind case was case one from ASCE 7-10 27.4-8. The results were, for most stories, close to the hand calculations. However at the base of each shear wall the ETABS model story forces were notably different. This could be a result of a number of things such as fixity at the base, difference in dead load (Nu) approximation or material properties. The story force comparison excel sheet is shown in the Appendix and is posted on the CPEP website.

6.2.1 Hand Calculations for COM, COR and relative stiffness



Notebook C | Lateral System Analysis

Key
1. SW13
2. SW12
3. SW 11
4. SW10
5. SW 9
6. SW6
7. SW7
8. SW4
9. SW5
10. SW16
11. SW15
12. SW14

COM and COR

Note: Shear wall numbers are chosen randomly

- H = 11'
- $E = 57000 \cdot \sqrt{f'c}$ = 4.41E6
- f'c = average of 6000psi
- u = 0.2 assumed
- walls N-S = 10"
  E-W = 12"

- Shear Wall Locations

1. (119.5, 68) (133.5, 68) = 14' long 12" thick 1'

2. (133.5, 68) (133.5, 77.5) = 9.5' long 10" thick 0.833'

3. (133.5, 77.5) (124.5, 77.5) = 9' long 12" thick 1'

4. (124.5, 77.5) (124.5, 95.5) = 18' long 10" thick 0.833'

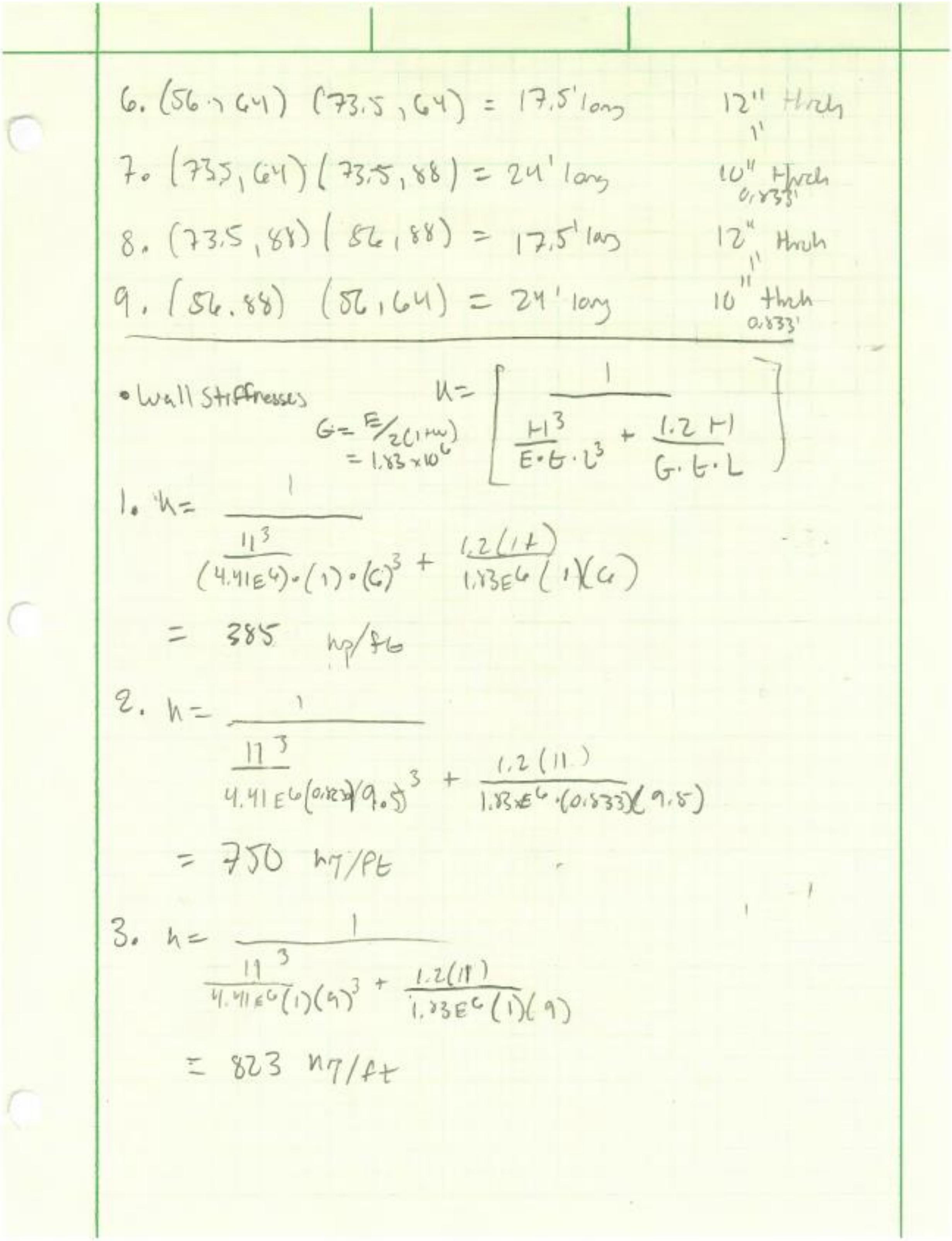
5. (124.5, 95.5) (133.5, 95.5) = 9' long 12" 1'

---

10. (144.0833, 68) (153.5, 68) = 9.5' long 12" thick 1'

11. (153.5, 68) (153.5, 95.5) = 27.5' long 10" thick 0.833'

12. (144, 95.5) (153.5, 95.5) = 9.5' long 12" thick 1'



6. (56, 64) (73.5, 64) = 17.5' long   12" thick
1'

7. (73.5, 64) (73.5, 88) = 24' long   10" thick
0.833'

8. (73.5, 88) (56, 88) = 17.5' long   12" thick
1'

9. (56, 88) (56, 64) = 24' long   10" thick
0.833'

- Wall Stiffnesses

$$G = E/2(1+\nu) = 1.83 \times 10^6 \qquad k = \left[\frac{1}{\frac{H^3}{E \cdot t \cdot L^3} + \frac{1.2\,H}{G \cdot t \cdot L}} \right]$$

1. $$k = \frac{1}{\frac{11^3}{(4.41E4) \cdot (1) \cdot (6)^3} + \frac{1.2(11)}{1.83E6(1)(6)}}$$

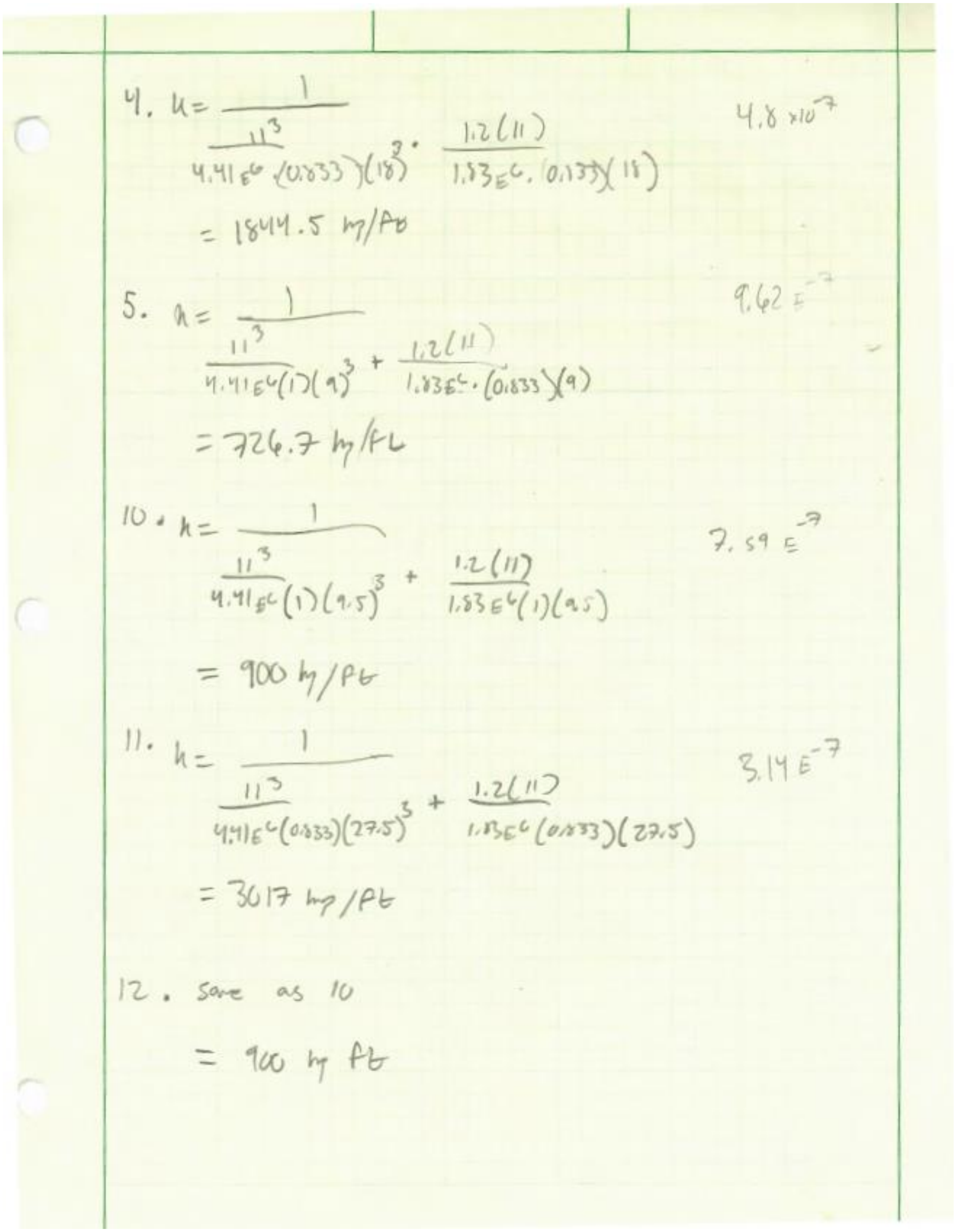
$= 385$ k/ft

2. $$k = \frac{1}{\frac{11^3}{4.41E6(0.833)(9.5)^3} + \frac{1.2(11)}{1.83E6 \cdot (0.833)(9.5)}}$$

$= 750$ k/ft

3. $$k = \frac{1}{\frac{11^3}{4.41E6(1)(9)^3} + \frac{1.2(11)}{1.83E6(1)(9)}}$$

$= 823$ k/ft



4. $k = \dfrac{1}{\dfrac{11^3}{4.41E^6(0.833)(18)^3} \cdot \dfrac{1.2(11)}{1.83E^6 \cdot (0.133)(18)}}$ 4.8×10^{-7}

$= 1844.5$ lb/ft

5. $k = \dfrac{1}{\dfrac{11^3}{4.41E^6(1)(9)^3} + \dfrac{1.2(11)}{1.83E^6 \cdot (0.833)(9)}}$ $9.62\,E^{-7}$

$= 726.7$ lb/ft

10. $k = \dfrac{1}{\dfrac{11^3}{4.41E^6(1)(9.5)^3} + \dfrac{1.2(11)}{1.83E^6(1)(9.5)}}$ $7.59\,E^{-7}$

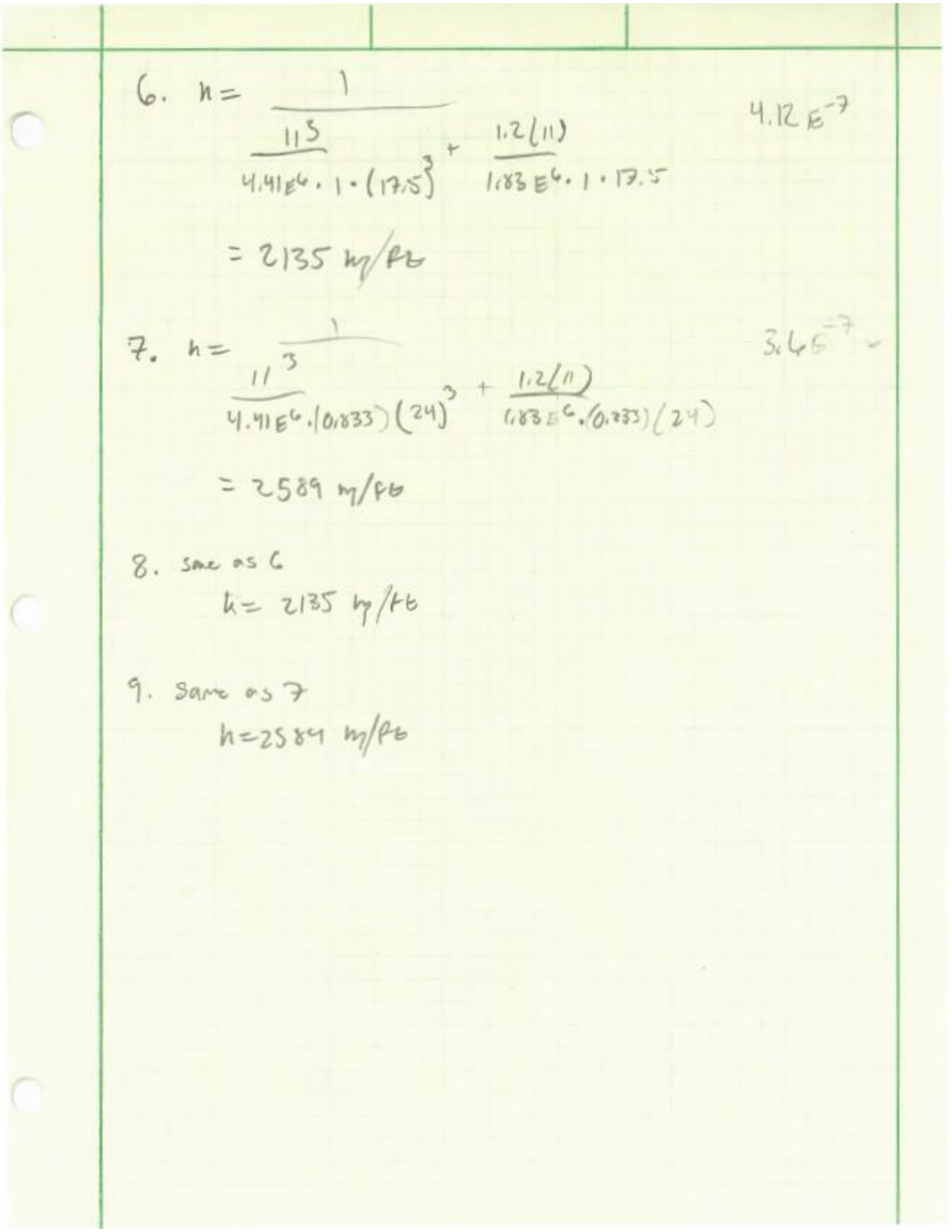
$= 900$ lb/ft

11. $k = \dfrac{1}{\dfrac{11^3}{4.41E^6(0.833)(27.5)^3} + \dfrac{1.2(11)}{1.83E^6(0.833)(27.5)}}$ $3.14\,E^{-7}$

$= 3017$ lb/ft

12. Same as 10

$= 900$ lb ft



6. $h = \dfrac{1}{\dfrac{11^3}{4.41E^6 \cdot 1 \cdot (17.5)^3} + \dfrac{1.2(11)}{1.83E^6 \cdot 1 \cdot 17.5}}$ $4.12E^{-7}$

$= 2135$ m/ft

7. $h = \dfrac{1}{\dfrac{11^3}{4.41E^6 \cdot (0.833)(24)^3} + \dfrac{1.2(11)}{1.83E^6 \cdot (0.833)(24)}}$ $3.6E^{-7}$

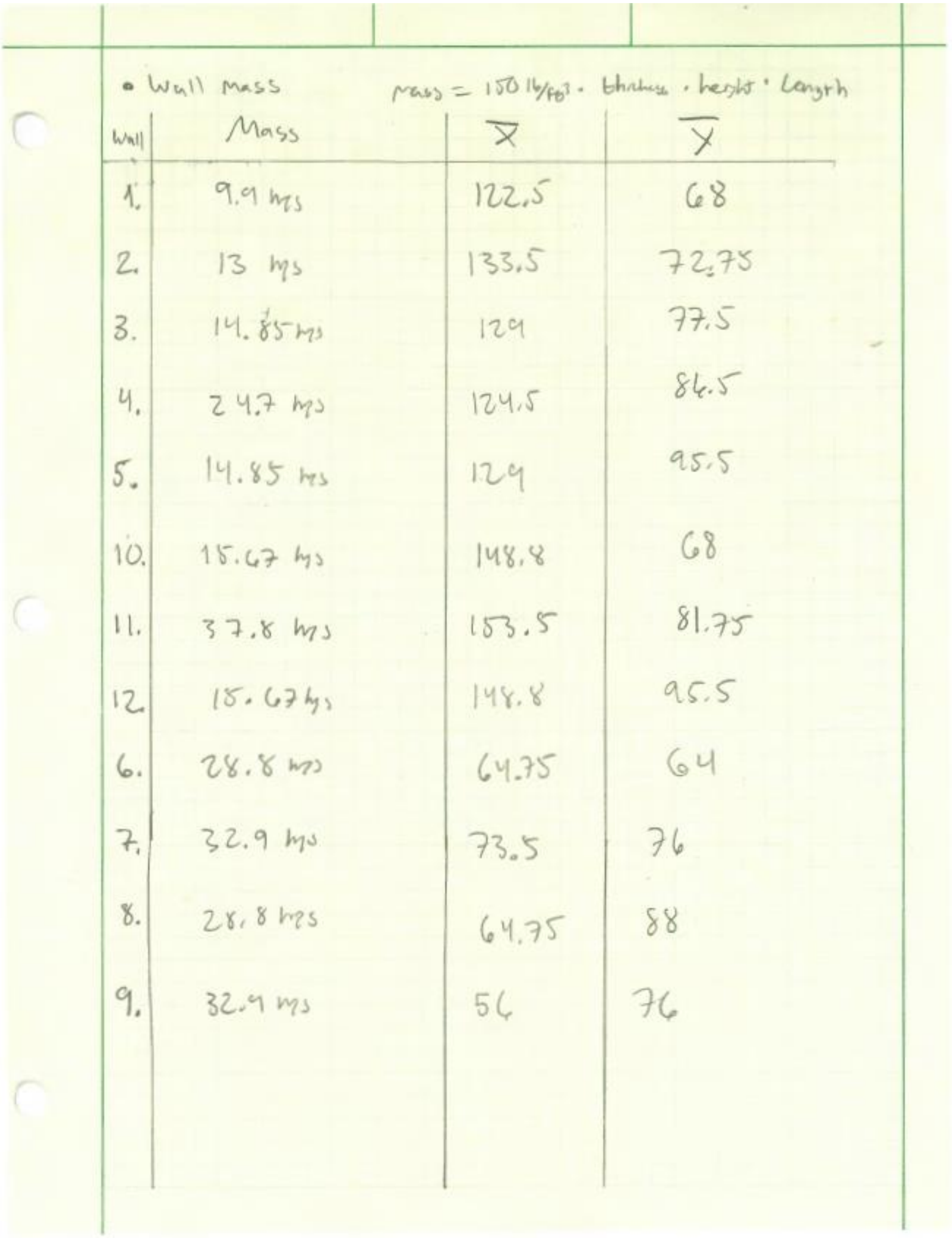
$= 2589$ m/ft

8. same as 6

$h = 2135$ m/ft

9. same as 7

$h = 2584$ m/ft



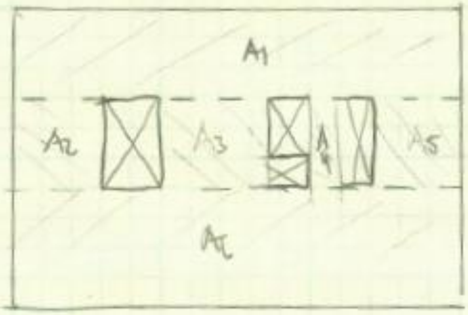
- Wall Mass mass = 150 lb/ft³ · thickness · height · Length

Wall	Mass	$\bar{X}$	$\bar{Y}$
1.	9.9 kps	122.5	68
2.	13 kps	133.5	72.75
3.	14.85 kps	129	77.5
4.	24.7 kps	124.5	86.5
5.	14.85 kps	129	95.5
10.	15.67 kps	148.8	68
11.	37.8 kps	153.5	81.75
12.	15.67 kps	148.8	95.5
6.	28.8 kps	64.75	64
7.	32.9 kps	73.5	76
8.	28.8 kps	64.75	88
9.	32.9 kps	56	76

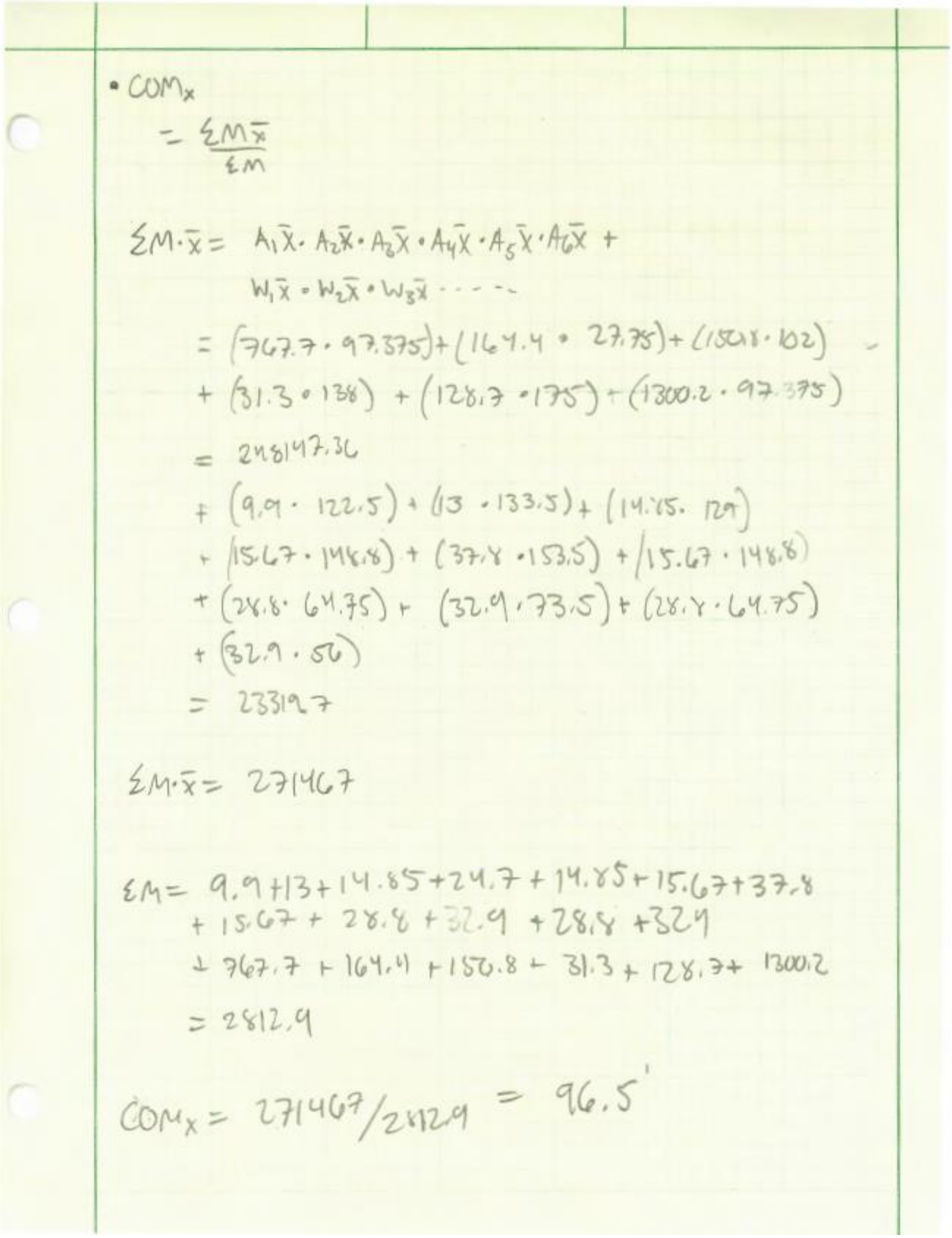
- Floor Mass

Slab is 8.5" thick (not accounting for Drop Panels)
= 0.708'

$A_1 = 7226\ ft^2$
$A_2 = 1548\ ft^2$
$A_3 = 1420\ ft^2$
$A_4 = 295\ ft^2$
$A_5 = 1212\ ft^2$
$A_6 = 12,243\ ft^2$



Section	Mass = A·t·150	$\bar{x}$	$\bar{y}$
A_1	767.7 kips	97.375	115.5
A_2	164.4 kips	27.75	83.8
A_3	150.8 kips	102	83.8
A_4	31.3 kips	138	83.8
A_5	128.7 kips	175	83.8
A_6	1300.2 kips	97.375	31



- COM_x

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

$$\Sigma M \cdot \bar{x} = A_1\bar{x} \cdot A_2\bar{x} \cdot A_3\bar{x} \cdot A_4\bar{x} \cdot A_5\bar{x} \cdot A_6\bar{x} +$$
$$W_1\bar{x} \cdot W_2\bar{x} \cdot W_3\bar{x} \cdots\cdots$$

$$= (767.7 \cdot 97.375) + (164.4 \cdot 27.75) + (150.8 \cdot 102)$$
$$+ (31.3 \cdot 138) + (128.7 \cdot 175) + (1300.2 \cdot 97.375)$$

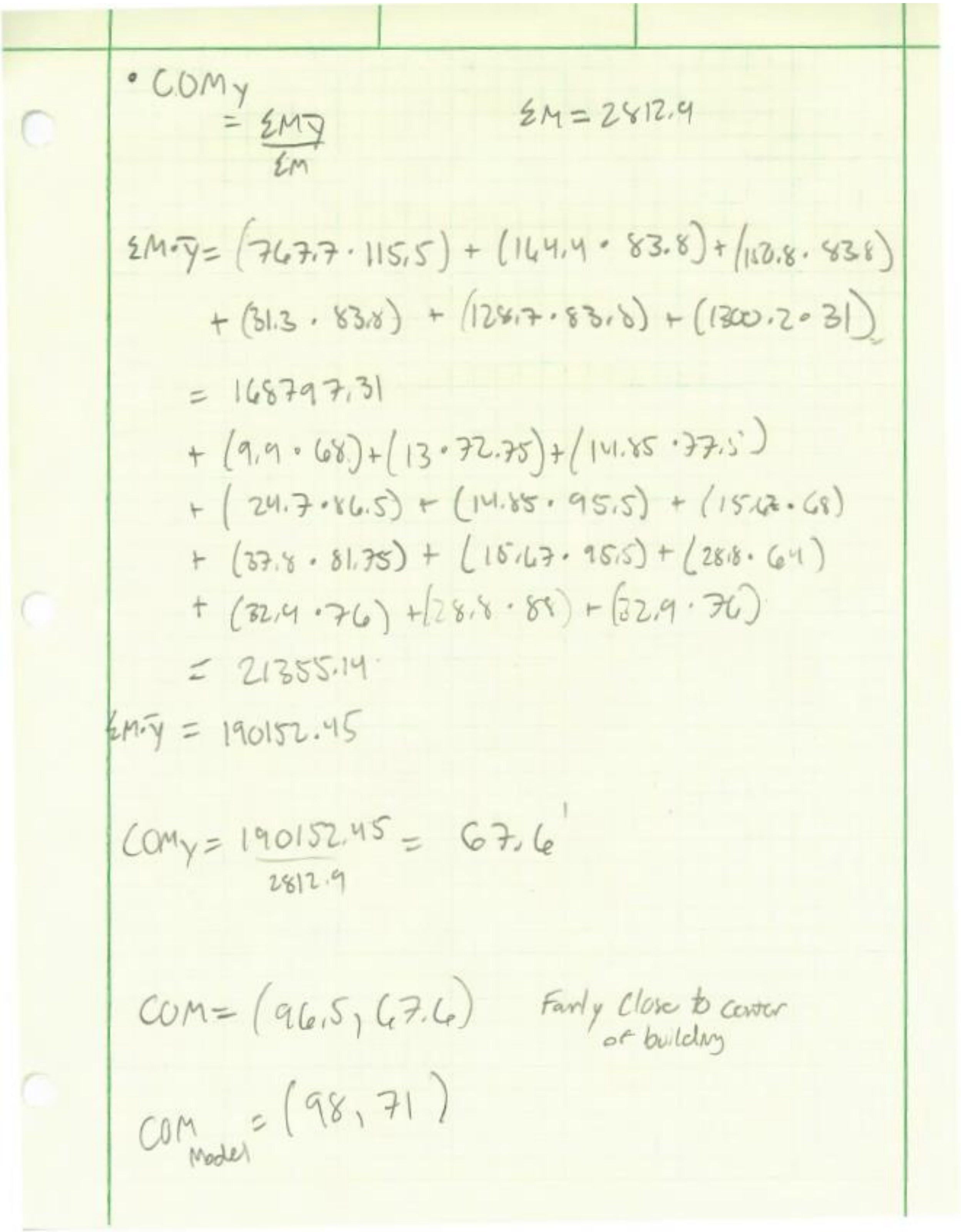
$$= 248147.36$$

$$+ (9.9 \cdot 122.5) + (13 \cdot 133.5) + (14.85 \cdot 129)$$
$$+ (15.67 \cdot 148.8) + (37.8 \cdot 153.5) + (15.67 \cdot 148.8)$$
$$+ (28.8 \cdot 64.75) + (32.9 \cdot 73.5) + (28.8 \cdot 64.75)$$
$$+ (32.9 \cdot 56)$$
$$= 23319.7$$

$$\Sigma M \cdot \bar{x} = 271467$$

$$\Sigma M = 9.9 + 13 + 14.85 + 24.7 + 14.85 + 15.67 + 37.8$$
$$+ 15.67 + 28.8 + 32.9 + 28.8 + 329$$
$$+ 767.7 + 164.4 + 150.8 + 31.3 + 128.7 + 1300.2$$
$$= 2812.9$$

$$COM_x = 271467 / 2812.9 = 96.5'$$



- COM_y

$$= \frac{\Sigma M \bar{y}}{\Sigma M} \qquad \Sigma M = 2812.9$$

$$\Sigma M \cdot \bar{y} = (767.7 \cdot 115.5) + (164.4 \cdot 83.8) + (150.8 \cdot 83.8)$$
$$+ (31.3 \cdot 83.8) + (128.7 \cdot 83.8) + (1300.2 \cdot 31)$$
$$= 168797.31$$
$$+ (9.9 \cdot 68) + (13 \cdot 72.75) + (14.85 \cdot 77.5)$$
$$+ (24.7 \cdot 86.5) + (14.85 \cdot 95.5) + (15.67 \cdot 68)$$
$$+ (37.8 \cdot 81.75) + (15.67 \cdot 95.5) + (28.8 \cdot 64)$$
$$+ (32.4 \cdot 76) + (28.8 \cdot 88) + (32.9 \cdot 76)$$
$$= 21355.14$$
$$\Sigma M \cdot \bar{y} = 190152.45$$

$$COM_y = \frac{190152.45}{2812.9} = 67.6'$$

$$COM = (96.5, 67.6)$$ Fairly close to center of building

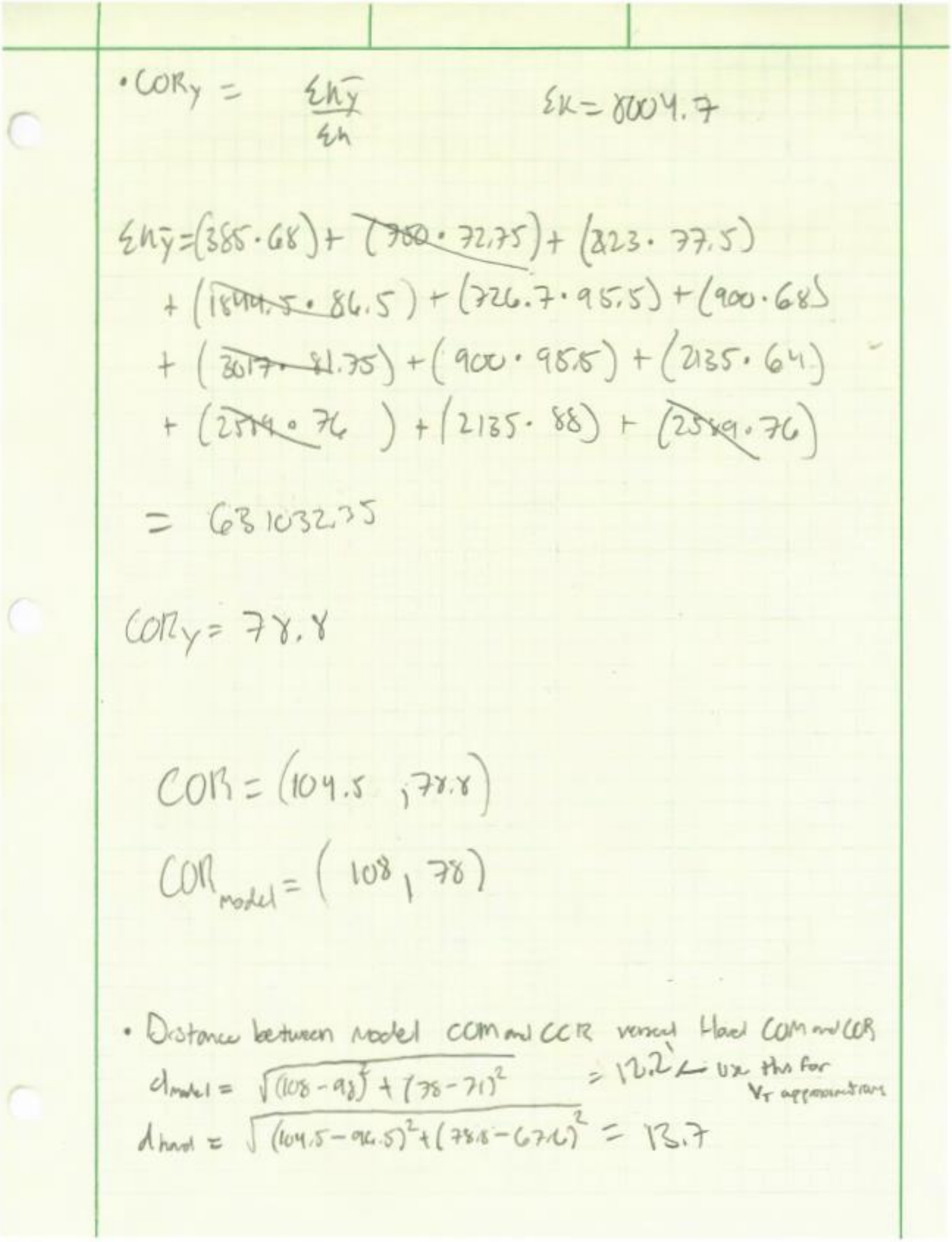
$$COM_{Model} = (98, 71)$$

- $COR_x = \frac{\Sigma K \bar{x}}{\Sigma K}$ walls in y direction

$$\Sigma K\bar{x} = \cancel{(385 \cdot 122.5)} + (750 \cdot 133.5) + \cancel{(883 \cdot 124)}$$
$$+ (1844.5 \cdot 124.5) + \cancel{(726.7 \cdot 124)} + \cancel{(900 \cdot 148.8)}$$
$$+ (3017 \cdot 153.5) + \cancel{(900 \cdot 148.8)} + \cancel{(2135 \cdot 64.75)}$$
$$+ (2589 \cdot 73.5) + \cancel{(2135 \cdot 64.75)} + (2589 \cdot 56)$$
$$= 1128150.25$$

$$\Sigma K = \cancel{385} + 750 + \cancel{883} + 1844.5 + \cancel{726.7} + \cancel{900}$$
$$+ 3017 + \cancel{900} + \cancel{2135} + 2589 + \cancel{2135} + 2589$$
$$= 10789.5$$

$$COR_x = 104.56'$$



- $COR_y = \frac{\Sigma h\bar{y}}{\Sigma h}$ $\quad \Sigma h = 8004.7$

$$\Sigma h\bar{y} = (385 \cdot 68) + \cancel{(700 \cdot 72.75)} + (823 \cdot 77.5) + \cancel{(1844.5 \cdot 86.5)} + (726.7 \cdot 95.5) + (900 \cdot 68) + \cancel{(3017 \cdot 81.75)} + (900 \cdot 95.5) + (2135 \cdot 64) + \cancel{(2514 \cdot 76)} + (2135 \cdot 88) + \cancel{(2549 \cdot 76)}$$

$$= 631032.75$$

$$COR_y = 78.8$$

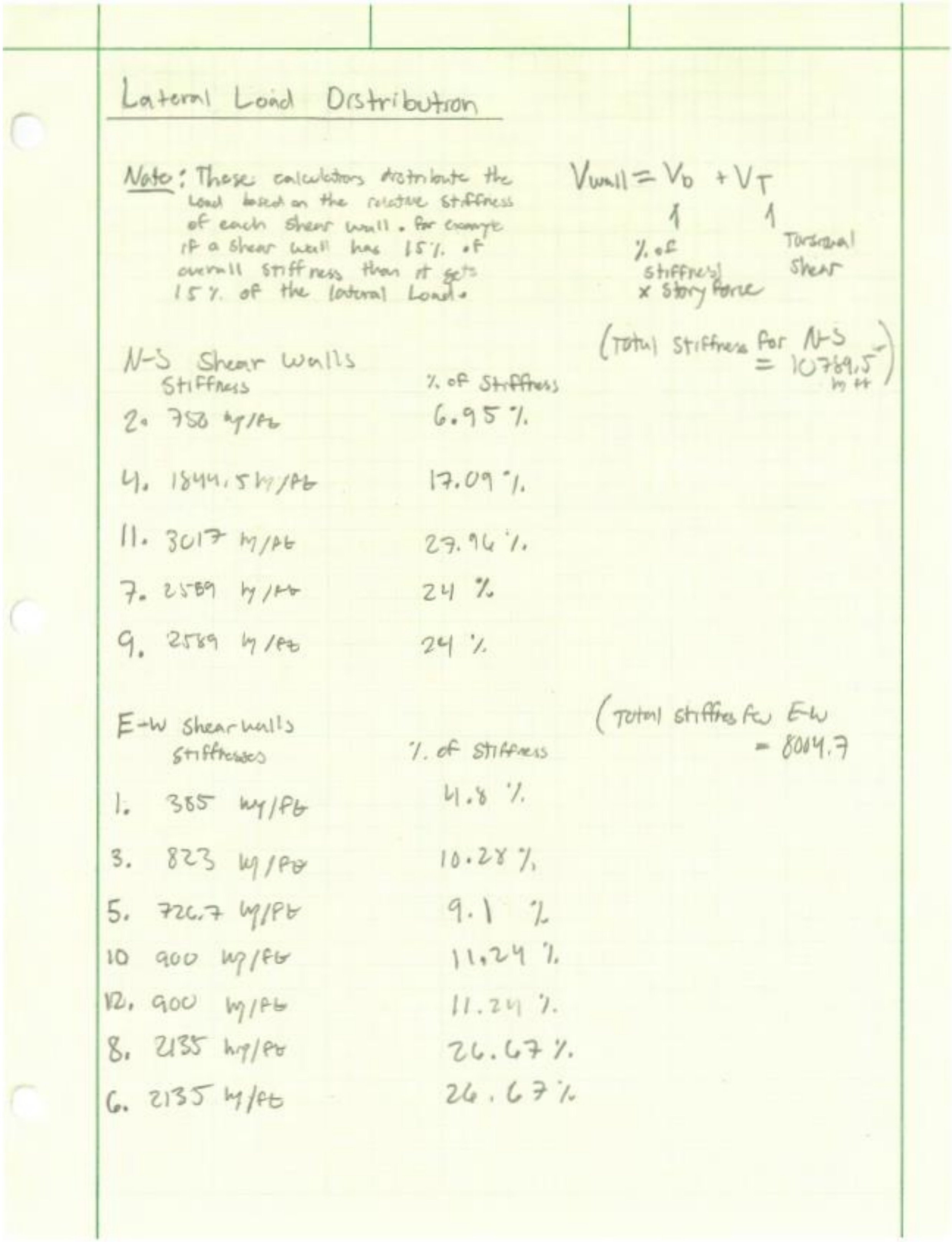
$$COR = (104.5, 78.8)$$

$$COR_{model} = (108, 78)$$

- Distance between model COM and COR versus Hand COM and COR

$$d_{model} = \sqrt{(108-98)^2 + (78-71)^2} = 12.2 \leftarrow \text{use this for } V_T \text{ approximations}$$

$$d_{hand} = \sqrt{(104.5-96.5)^2 + (78.8-67.6)^2} = 13.7$$



Lateral Load Distribution

Note: These calculators distribute the load based on the relative stiffness of each shear wall. For example if a shear wall has 15% of overall stiffness than it gets 15% of the lateral Load.

$$V_{wall} = V_D + V_T$$

V_D = % of stiffness × Story Force

V_T = Torsional Shear

(Total stiffness for N-S = 10789.5 k/ft)

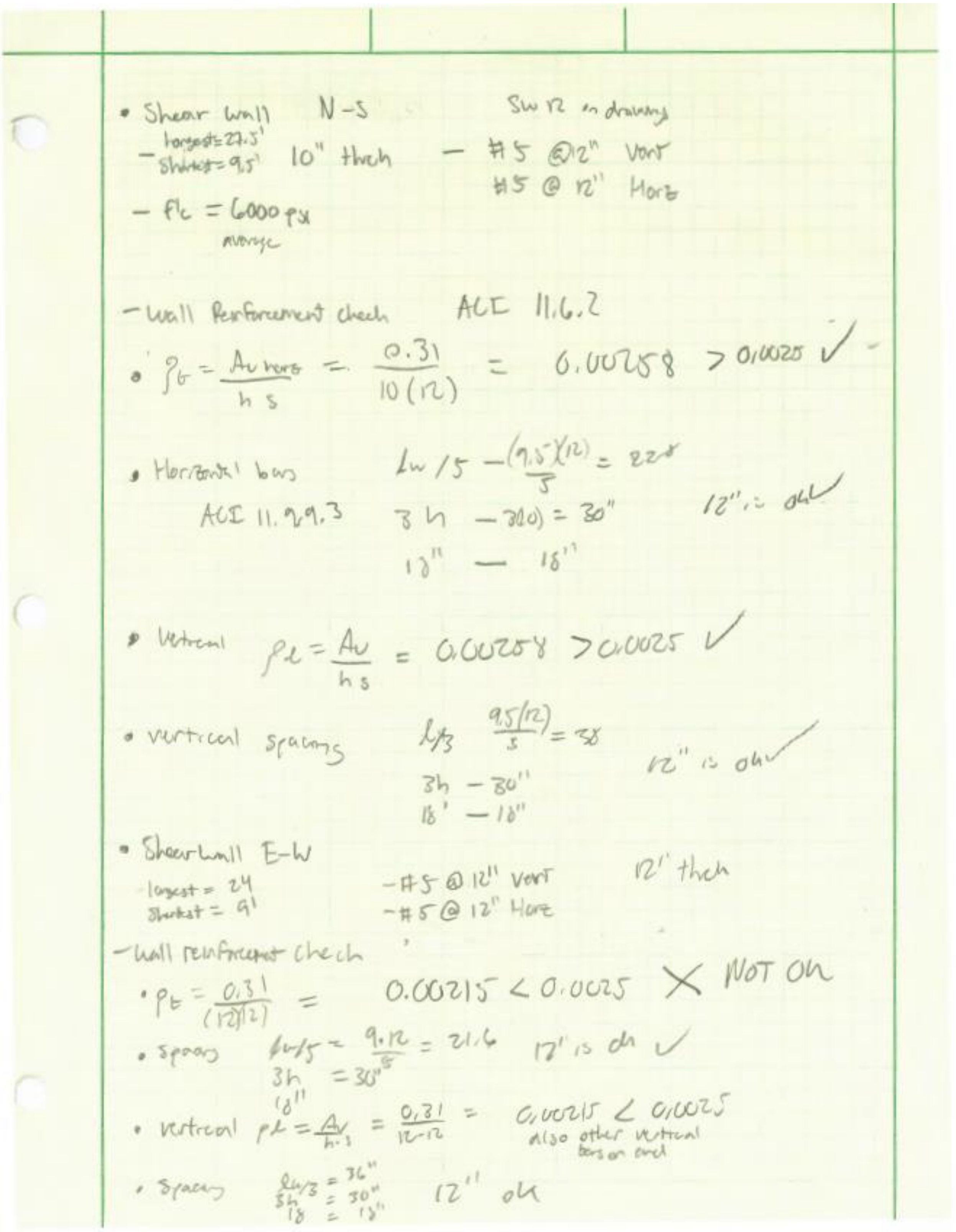
N-S Shear Walls

Stiffness	% of Stiffness
2. 750 k/ft	6.95 %
4. 1844.5 k/ft	17.09 %
11. 3017 k/ft	27.96 %
7. 2589 k/ft	24 %
9. 2589 k/ft	24 %

(Total stiffness for E-W = 8004.7

E-W Shear walls

Stiffnesses	% of Stiffness
1. 385 k/ft	4.8 %
3. 823 k/ft	10.28 %
5. 726.7 k/ft	9.1 %
10 900 k/ft	11.24 %
12. 900 k/ft	11.24 %
8. 2135 k/ft	26.67 %
6. 2135 k/ft	26.67 %



- Shear wall N-S Sw 12 in drawing
  - largest = 27.5'
shortest = 9.5' 10" thick — #5 @12" Vert
    #5 @ 12" Horz
  - f'c = 6000 psi
    average

- Wall Reinforcement check ACI 11.6.2

- $\rho_t = \frac{A_{v\,horz}}{h\,s} = \frac{0.31}{10(12)} = 0.00258 > 0.0025$ ✓

- Horizontal bars $l_w/5$ — $\frac{(9.5)(12)}{5} = 22.8$
  ACI 11.99.3 3h — 3(10) = 30" 12" is ok ✓
  18" — 18"

- Vertical $\rho_l = \frac{A_v}{h\,s} = 0.00258 > 0.0025$ ✓

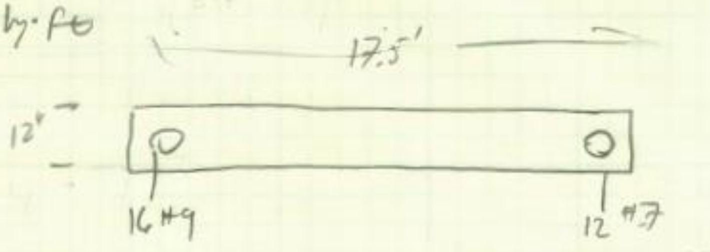
- vertical spacing $l_w/3$ $\frac{9.5(12)}{3} = 38$
  3h — 30" 12" is ok ✓
  18" — 18"

- Shearwall E-W
  largest = 24 —#5 @ 12" Vert 12" thick
  shortest = 9' —#5 @ 12" Horz

- wall reinforcement check
- $\rho_t = \frac{0.31}{(12)(12)} = 0.00215 < 0.0025$ × NOT OK
- Spacing $l_w/5 = \frac{9\cdot 12}{5} = 21.6$ 12" is ok ✓
  3h = 30"
  18"
- vertical $\rho_l = \frac{A_v}{h\cdot s} = \frac{0.31}{12\cdot 12} = 0.00215 < 0.0025$
  also other vertical bars on end
- Spacing $l_w/3 = 36"$ 12" ok
  3h = 30"
  18 = 18"

- Shear wall #8 analysis for Wind E-W (Excell Verification)

$M_u = 15800$ kip·ft



- $T = A_s f_y = 12 \cdot 0.6 = 7.2 = 11\,in^2$ avg

  Avg $16 \cdot 1 = 16\,in^2$

  $= 660$ kips

- $d = l_w - \underset{cover}{3''} - 8''$

  $= 17.5 \cdot 12 - 3'' - 8''$

  $= 200''$

- $N_u = \text{Selfweight} + 13 \cdot (\overset{lb/ft^2}{137}) \cdot (506) / 1000$

  $413.5 + 901$

  $= 1314.5$

- $a = \frac{T + N_u}{0.85 f'_c\, b} = \frac{660 + 1314.5}{0.85 \cdot 6 \cdot 12} = 32.26''$

- $c = a/\beta_1 = \frac{32.6}{0.75} = 43''$

  $c/d = 0.2 < 0.375$

  $\phi = 0.9$

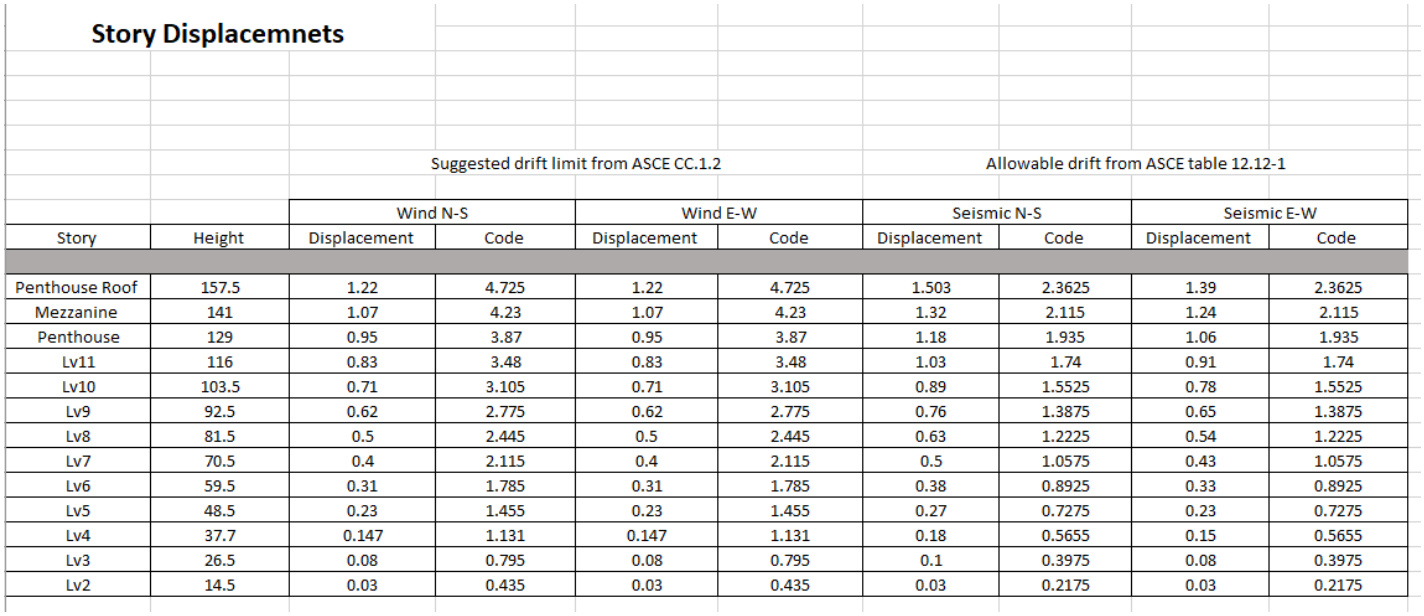
- $\phi M_n = 0.9 \left[660 \left(200 - \frac{32.24}{2} \right) + 1314.5 \left(210 - \frac{32.24}{2} \right) \right]$

  $= 121354.2 + 116819.6$

  $= 17{,}863$ kip·ft $> M_u$

6.3 Member Spot Checks

Once the ETABS model was validated against the hand calculations it was deemed correct for generating results such as story displacements. Figure 6 depicts a graphic representation of height vs displacement for every load case and code.



Story Displacemnets

		Suggested drift limit from ASCE CC.1.2				Allowable drift from ASCE table 12.12-1			
		Wind N-S		Wind E-W		Seismic N-S		Seismic E-W	
Story	Height	Displacement	Code	Displacement	Code	Displacement	Code	Displacement	Code
Penthouse Roof	157.5	1.22	4.725	1.22	4.725	1.503	2.3625	1.39	2.3625
Mezzanine	141	1.07	4.23	1.07	4.23	1.32	2.115	1.24	2.115
Penthouse	129	0.95	3.87	0.95	3.87	1.18	1.935	1.06	1.935
Lv11	116	0.83	3.48	0.83	3.48	1.03	1.74	0.91	1.74
Lv10	103.5	0.71	3.105	0.71	3.105	0.89	1.5525	0.78	1.5525
Lv9	92.5	0.62	2.775	0.62	2.775	0.76	1.3875	0.65	1.3875
Lv8	81.5	0.5	2.445	0.5	2.445	0.63	1.2225	0.54	1.2225
Lv7	70.5	0.4	2.115	0.4	2.115	0.5	1.0575	0.43	1.0575
Lv6	59.5	0.31	1.785	0.31	1.785	0.38	0.8925	0.33	0.8925
Lv5	48.5	0.23	1.455	0.23	1.455	0.27	0.7275	0.23	0.7275
Lv4	37.7	0.147	1.131	0.147	1.131	0.18	0.5655	0.15	0.5655
Lv3	26.5	0.08	0.795	0.08	0.795	0.1	0.3975	0.08	0.3975
Lv2	14.5	0.03	0.435	0.03	0.435	0.03	0.2175	0.03	0.2175

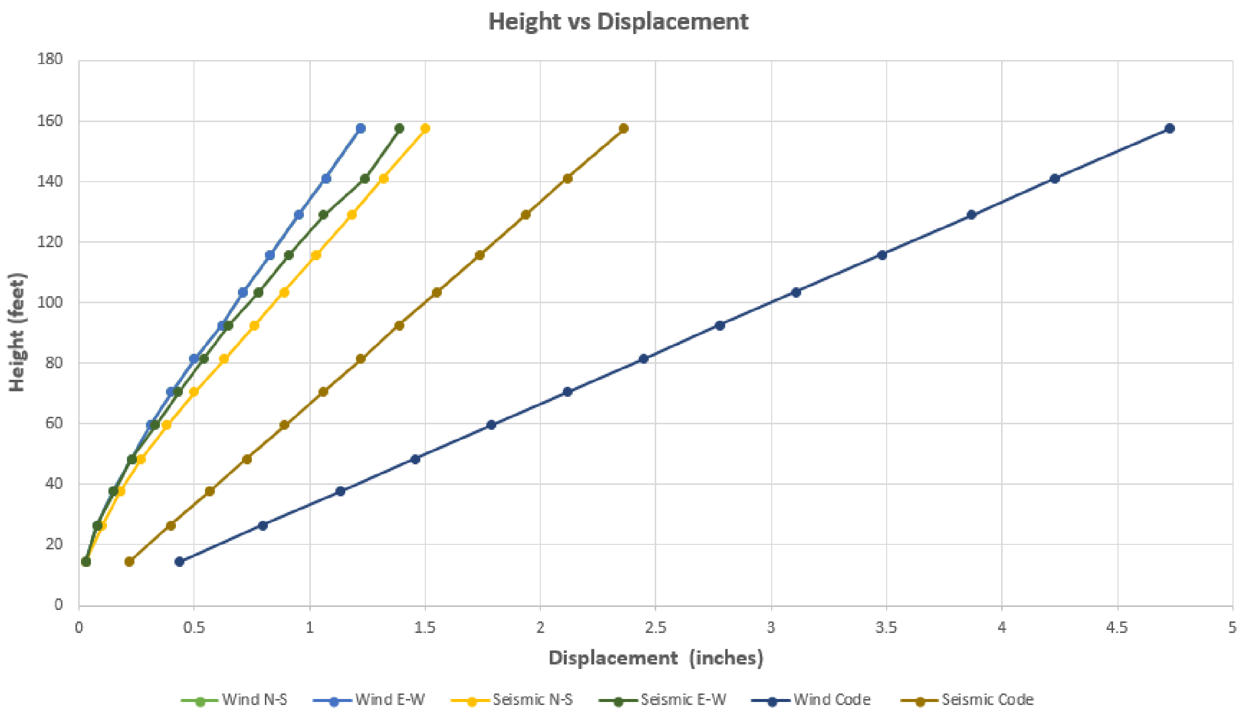


Figure 6: Graph of height vs displacement for the lateral load cases compared to the allowable code dispolacement.

Individual shear walls were also analyzed for both shear and moment capacity and compared to their base shear (max) and base moment (max) respectively. The critical factor for each shear wall passing in flexure was the dead load approximation Nu. This value was determined from taking the dead load previously determined in report A and multiplying it by the shear walls tributary area. Many of the shear walls passed for the wind loads but not the seismic loads. Due to the nature of how the analysis method was fairly simple and approximate it can be said that the shear walls might have passed if a more detailed analysis was done. Furthermore the shear walls that didn't pass flexure might have if the dead load was reanalyzed.