Michael Bologna

665 Meadow Ln
Rochester PA, 15074

State College, PA 16801

November 16, 2015

Linda M. Hanagan, PhD, P.E.
The Pennsylvania State University

Dear Dr. Hanagan,

The attached technical reports covers the assigned topics for Structural Notebook Submission C along with the previously submitted gravity and lateral loads analysis as part of Submission A and Submission B.

Submission C is an analysis of how the lateral forces are distributed to resisting members and the drift the forces cause on the members. This is accomplished through the use of a 3D analysis model and hand calculations.

In addition, I revised selected pages from Submission B to correct my wind load calculations. I also added pages that include a consideration for the weight of the exterior walls to add to the seismic weight.

I appreciate your effort in reviewing my submission and I look forward to receiving feedback from you.

Sincerely,

Michael Bologna

Jackson Crossing

Alexandria, Virginia

Notebook Submission C

Michael Bologna
Structural Option

November 16, 2015

Advisor: Dr. Linda Hanagan

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Abstract

Jackson Crossing - Alexandria, VA

Michael Bologna
Structural Option

Photos courtesy of Bonstra | Haresign Architects, LLP

Building Statistics

Building Height: 54' 7 1/4"
Number of Floors: 5
Gross Square Foot: 107,740 sq. ft.
Type of Building: Multi-Family Residential
Total Project Cost: $16 Million
Construction Dates: 4/4/2014-12/17/2015

Project Team

Owner: AHC, Inc.
Construction Management: Harkins Builders, Inc.
Architect: Bonstra | Haresign Architects, LLP
Civil Engineer: VIKA, Virginia, LLC
Structural Engineer: Rathgeber Goss Associates
MEP Engineer: Metropolitan Engineering, Inc.
Landscape Architect: Landscape Architectural Bureau
Specifications Cons.: Bethel Specifications Consulting

Mechanical

-All aparment units have operable windows
-Typical floor houses a mounted vertical heat pump (DX Split System) and is provided with vibration isolation
-Roof houses condensing units
-Upper garage exhausts 12,000 CFM of air and supplies 17,250 CFM of air
-Lower Garage exhauts 5250 CFM of air

Photos courtesy of Bonstra | Haresign Architects, LLP

Electrical

-Dominion Virgina Power Service supplies power into one pad mounted transformer
-2 1600A, 208/120V Feeders run from the transformer
-All units are individually metered

Structural System

Gravity System
-18" deep wood trusses spaced at 24" o.c.
-Wood bearing walls
-12" Reinforced two-way concrete slab
-24"x16" Concrete columns typical
Lateral System
-Reinforced Concrete Shear Wall
-Reinforced Masonry Shear Wall
-Wood Shear Wall

Thesis Advisor: Linda M. Hanagan, PhD, P.E.
Website: http://www.engr.psu.edu/ae/thesis/portfolios/2016/mab6150/index.htm

Executive Summary

Jackson Crossing is a development in Alexandria, Virginia by AHC, Inc. Offering one, two, and three-bedroom apartments, it is targeted at low-income residents with families. The structure is five floors and 107,740 square feet. Included in the building is an underground parking garage. The project will be completed by December 2015 and will come to a total project cost of sixteen million dollars.

The gravity system consists of four floors of wood floors with wood trusses and bearing walls. The wood members sit on two floors of concrete, one of which is below grade. The slab on the second floor is a reinforced two-way slab while the ground floor is a reinforced one-way slab with concrete beams.

The lateral system for the top four floors include masonry shear walls and wood shear walls with OSB sheathing. The wood shear walls are anchored into the second floor slab while the masonry shear walls are integrated into reinforced concrete shear walls that extend down into the foundation.

Location Plan

Figure 1 (Courtesy of Google Maps)

Site Plan

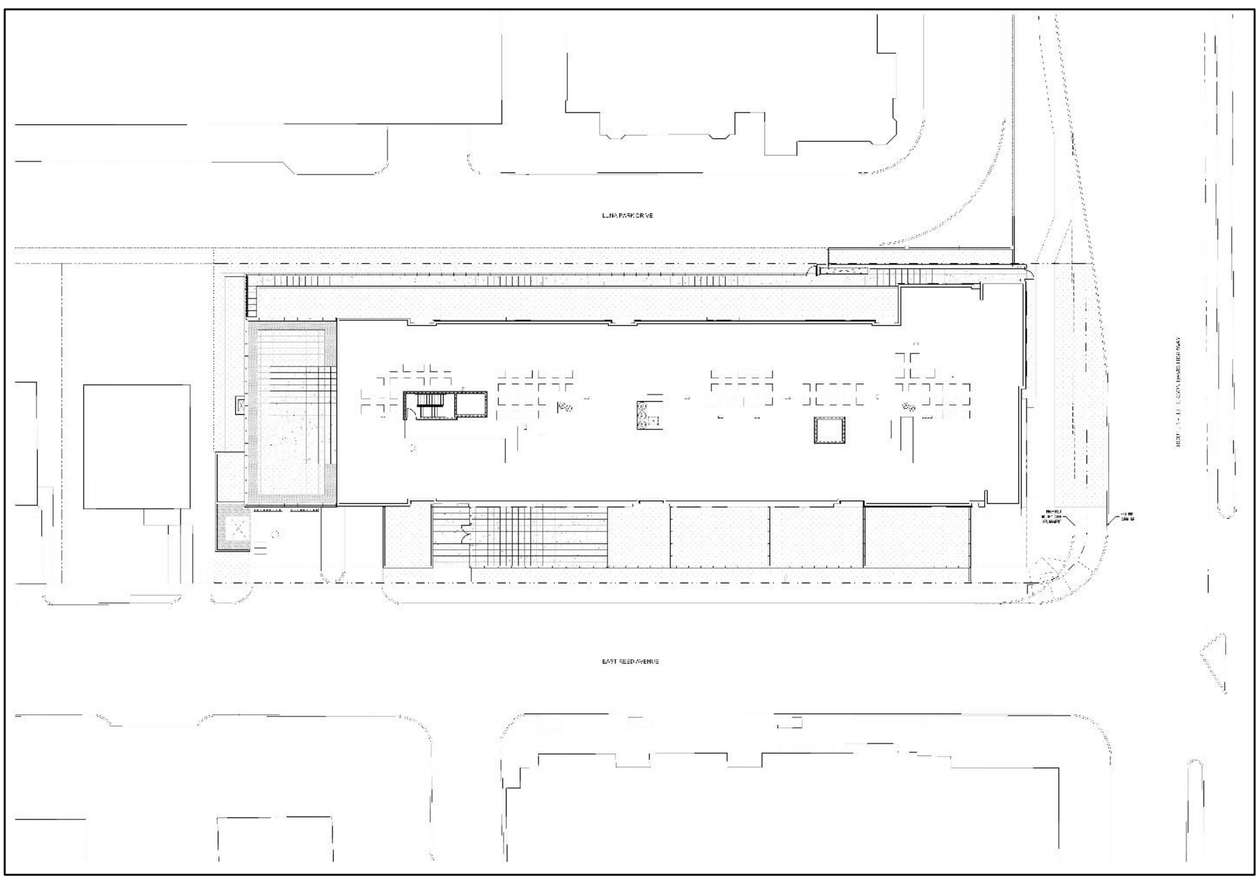


Figure 2

List of Documents Used in Report

AISC, *Steel Construction Manual*, Fourteenth Edition

Breyer, Donald, Kelly Cobeen, Kenneth Fridley, and David Pollock, *Design of Wood Structures ASD/LRFD*, 7th Edition

Usg.com, DUROCK Cement Board

Minimum Design Loads for Buildings and Other Structures (ASCE 7-10)

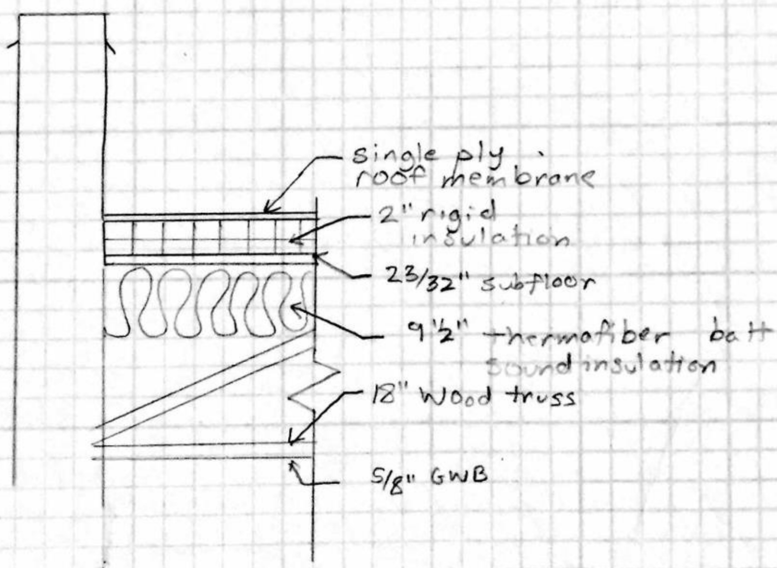
Minimum Design Loads for Buildings and Other Structures (ASCE 7-05)

RS Means Assemblies Cost Data, 2014

GRAVITY LOADS

- ROOF
  - Existing Design Loads (from General notes)
    *SUPERIMPOSED INCL. STRUCTURE
    - DEAD LOAD
      ⇒ 20psf (15psf top chord / 5psf bottom chord)
    - ROOF LIVE LOAD
      ⇒ 30psf min (unless snow load greater)
    - ROOF SNOW LOAD
      ⇒ P_f = 17.5psf
      - values to find P_f
        - P_g = 25psf
        - C_e = 1.0
        - C_t = 1.0
        - I = 1.0

TYPICAL PARAPET DETAIL



WL

WEIGHT OF DETAILS

- SINGLE PLY ROOF MEMBRANE ⇒ 1 psf
- 2" RIGID INSULATION ⇒ $1\frac{1}{2}$ psf / 1" · 2" = 3 psf
- 23/32" WOOD SUBFLOOR ⇒ 3 psf / (3/4") · 4/3" · 23/32" = 2.875 psf (wood sheathing)
- $9\frac{1}{2}$" BATT INSULATION ⇒ ½ psf / 1" · 9.5" = 4.75 psf (loose insulation)
- 5/8" GWB ⇒ $2\frac{1}{2}$ psf
- Truss self weight ⇒ 5.5 plf / 2ft spacing = 2.75 psf

TOTAL = 16.875 psf ~ 17 psf

☆ WEIGHTS FROM TABLE 17-13 OF STEEL MANUAL

- TYPICAL WOOD FLOOR

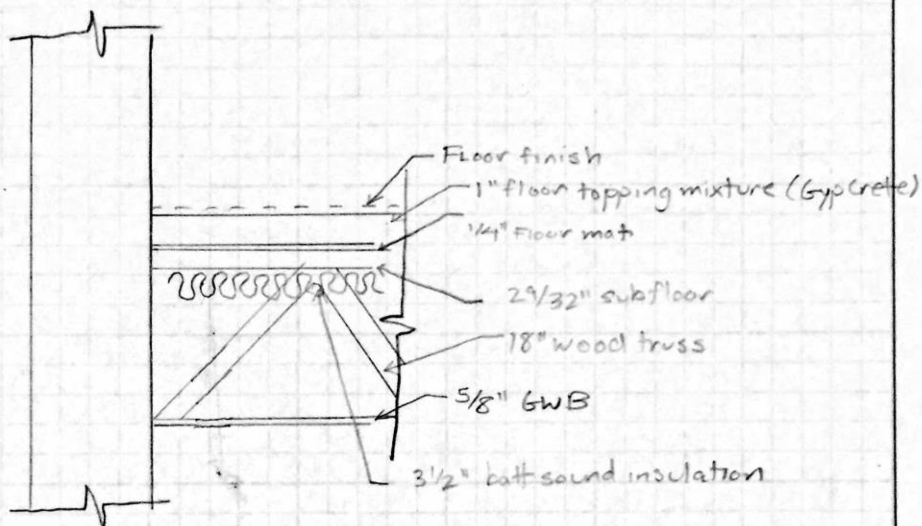
  - EXISTING DESIGN LOADS (from General Notes)

    & SUPERIMPOSED INCL. STRUCTURE

  - DEAD LOAD

    ⇒ 25 psf (20 psf top chord/5 psf bottom chord)

TYPICAL WOOD LEVEL DETAIL



WEIGHT OF DETAILS

- GYPCRETE ⇒ 6.9 psf/3/4" · 4/3" = 9.2 psf

- 1/4" FLOOR MAT ⇒ 3 psf/1" · 1/4" = 0.75 psf (plywood)

- 29/32" SUBFLOOR ⇒ 3 psf/3/4" · 4/3" · 23/32" = 2.875 (wood sheathing)

- 5/8" GWB ⇒ 2 1/2 psf (5/8" drywall)

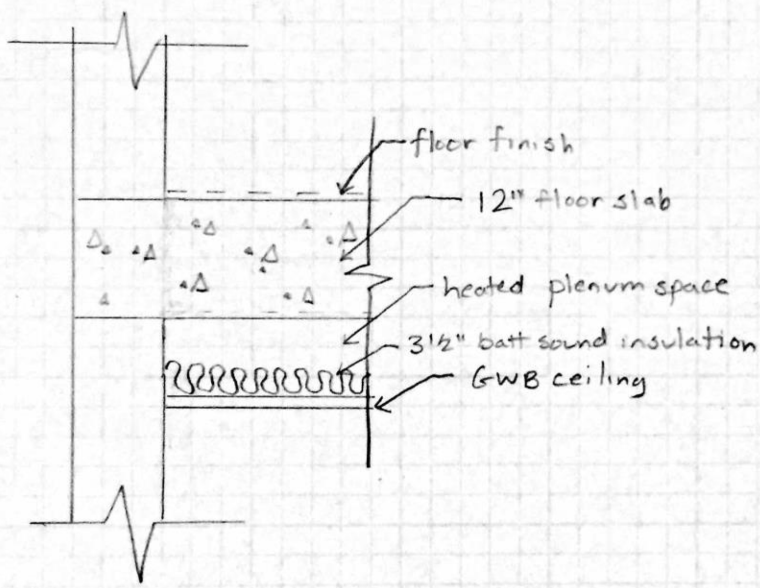
- 3 1/2" BATT INSULATION ⇒ 1/2 psf · 3 1/2" = 1.75 psf (loose insulation)
- MECHANICAL ALLOWANCE ⇒ 4 psf
- FLOOR FINISH ⇒ 1 psf
- TRUSS SELF WEIGHT ⇒ 5.5 plf/2 ft = 2.75 psf

TOTAL = 24.825 psf ~ 25 psf ✓ weight

★ WEIGHTS FROM - TABLE 17-13 OF STEEL MANUAL
- APPENDIX B OF DESIGN OF WOOD STRUCTURES

- TYPICAL CONCRETE LEVEL
  - EXISTING DESIGN LOADS (from General Notes)
    *SUPERIMPOSED INCL. STRUCTURE
    - DEAD LOAD
      ⇒ 15 psf

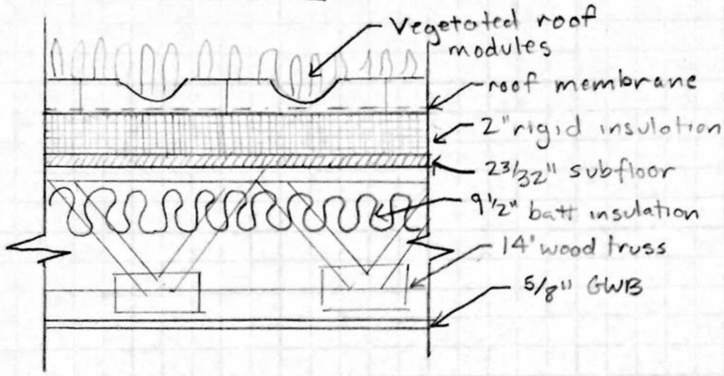
- TYPICAL CONCRETE LEVEL DETAIL



- WEIGHT OF DETAILS
  - FLOOR FINISH = 1 psf
  - 3½" BATT INSULATION = $\frac{1/2\text{ psf}}{1"} \cdot 3\frac{1}{2}" = 1.75$ psf (loose insulation)
  - GWB ceiling = 2½ psf (5/8 drywall)
  - NORMAL WEIGHT CONCRETE = 150 pcf · 12" = 150 psf
  - MECHANICAL ALLOWANCE = 4 psf

    TOTAL = 159.25 psf

ROOF TERRACE DETAIL

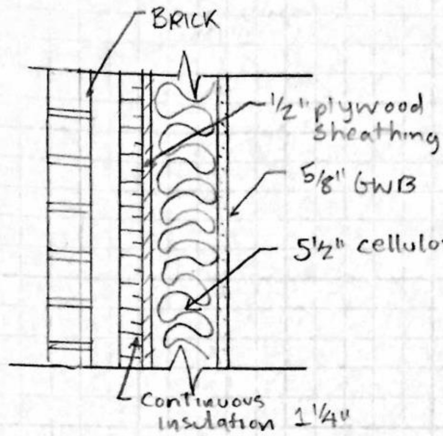


WEIGHT OF DETAILS

- ROOF MEMBRANE => 1psf
- 2" RIGID INSULATION => 3psf
- 23/32" SUBFLOOR => 3psf
- 9½" BATT INSULATION => ½psf/1" · 9.5" = 4.75psf
- 5/8" GWB => 2½psf
- 14" TRUSS @ 16"oc => 5plf/16/12 = 3.75psf
- MISC => 5psf
- VEGETATED MODULES => 20 psf

TOTAL = 43psf

TYPICAL EXTERIOR WALL DETAIL (BRICK)



WEIGHT OF DETAILS

4 IN BRICK => 40 psf

1/2" PLYWOOD => 1.5 psf

1 1/4" CONTINUOUS INSULATION => 1.88 psf

5/8" GWB => 2 1/2 psf

5 1/2" CELLULOSE INSULATION => 10 psf

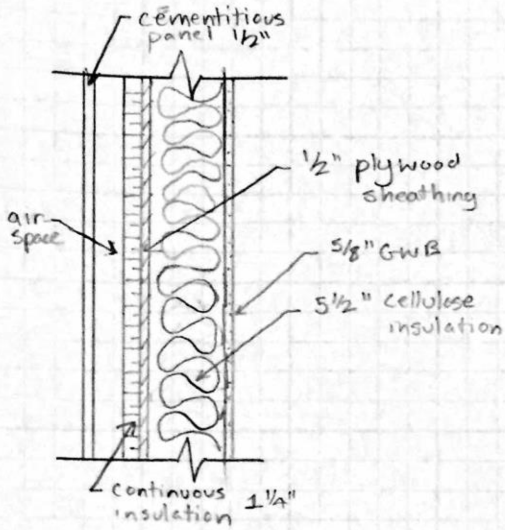
TOTAL = 55.88 psf

*WEIGHTS FROM TABLE 17-13 OF STEEL MANUAL

LOAD PATH DESCRIPTION

AT THE TOP FOUR LEVELS, GRAVITY LOADS ARE APPLIED TO THE WOOD TRUSSES. THE WOOD TRUSSES TRANSFER THEIR LOADS TO THE EXTERIOR AND INTERIOR BEARING WALLS. THEN THE BEARING WALLS REST THEIR LOADS ON A TRANSFER SLAB AT THE 2ND FLOOR.

TYPICAL EXTERIOR WALL DETAIL (CEMENTITOUS PANEL)



WEIGHT OF DETAILS

5 1/2" CELLULOSE INSULATION => $\frac{2\,psf}{1''} \cdot 5\frac{1}{2}'' = 10\,psf$ (poured insulation)

1 1/4" CONTINUOUS INSULATION => $\frac{1\frac{1}{2}\,psf}{1''} \cdot 1\frac{1}{4}'' = 1.88\,psf$ (rigid insulation)

1/2" PLYWOOD => $\frac{3\,psf}{1''} \cdot \frac{1}{2}'' = 1.5\,psf$

5/8" GWB => 2 1/2 psf

1/2" CEMENTITOUS PANEL => 2.4 psf

TOTAL = 18.28 psf

* WEIGHTS FROM TABLE 17-13 OF STEEL MANUAL & USG.COM FOR DUROCK CEMENT BOARD AS AN ESTIMATE FOR CEMENTITIOUS PANEL

EXTERIOR WALL LOAD

GROUND FLOOR

- WINDOWS ⇒ 648 sq ft + 240 sq ft + 673 sq ft + 240 sq ft = 1,801 sq ft
- PANELS ⇒ 126 sq ft + 126 sq ft = 252 sq ft
- BRICK ⇒ 1,901 sq ft + 478 sq ft + 1876 sq ft + 478 sq ft = 4,733 sq ft

SECOND FLOOR

- WINDOWS ⇒ 673 + 240 + 673 + 240 = 1,826 sq ft.
- PANELS ⇒ 126 + 126 = 252 sq ft.
- BRICK ⇒ 1,876 + 478 + 1,876 + 478 = 4,708 sq ft.

THIRD FLOOR

- WINDOWS ⇒ 1,826 sq ft
- PANELS ⇒ 252 sq ft
- BRICK ⇒ 4,708 sq ft

FOURTH FLOOR

- WINDOWS ⇒ 606 + 180 + 606 + 137 = 1,529 sq. ft
- PANELS ⇒ 390 + 30 + 1333 + 483 = 2,236 sq. ft
- BRICK ⇒ 1,192 + 507 + 250 + 94 = 2,043 sq. ft

FIFTH FLOOR

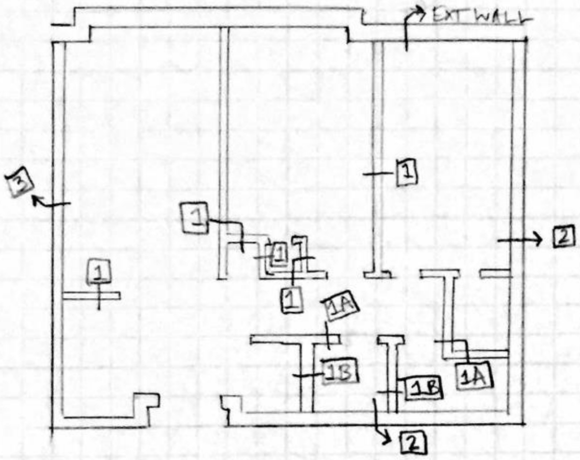
- WINDOWS ⇒ 606 + 180 + 606 + 105 = 1,497 sq ft
- PANELS ⇒ 2069 + 467 + 2107 + 779 = 5,422 sq. ft
- BRICK ⇒ 44 + 290 + 0 + 0 = 334 sq. ft

TOTAL LOAD AT EACH FLOOR

GROUND = 1,801 sq ft · 8 psf + 252 sq. ft. · 11 psf + 4,733 sq ft · 48 psf = 244.4K
SECOND = 1,826 sq ft · 8 psf + 252 sq. ft · 11 psf + 4,708 sq. ft · 48 psf = 243.4K
THIRD = 243.4K
FOURTH = 1,529 sq ft · 8 psf + 2,236 sq ft · 11 psf + 2,043 sq. ft · 48 psf = 134.9k
FIFTH = 1,497 sq. ft · 8 psf + 5,422 sq ft · 11 psf + 334 sq. ft · 48 psf = 87.7K

953.8K

TYPICAL 2BD UNIT (INTERIOR WALL LOAD)



LENGTH OF WALL

WALL 1 (49' 4")

2×4 stud@16"oc => 0.9 psf

(2) 5/8 GWB => 5 psf

6 psf · 49' 4" = 296 plf

WALL 1A (17' 8")

2×4 stud@16"oc => 0.9psf

(2) 5/8 GWB => 5 psf

BATT INSULATION => 0.5 lb

6.5psf · 17'8" = 115plf

WALL 1B (12')

2×6 stud @16" o.c. => 1.4 psf

(2) 5/8 GWB => 5 psf

6.5psf · 12' = 78plf

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER

COMET

WALL 2 (55' 10")

2×6 studs @ 16 in. o.c. ⇒ 1.4 psf

(2) 5/8" GWB ⇒ 5 psf

BATT INSULATION ⇒ 0.5 psf

7 psf · (55' 10") = 391 plf

WALL 3 (24' 5")

2×6 studs @ 16 in o.c ⇒ 1.4 psf

(2) 5/8" GWB ⇒ 5 psf

BATT INSULATION ⇒ 0.5 psf

1/2" OSB ⇒ 2 psf

9 psf · (24' 5") = 220 plf

TOTAL = 1100 PLF · (HEIGHT OF WALL)

= 1100 PLF · (9' 2") = 10,633 lbs

INTERIOR PARTITION WEIGHT:

$= \frac{10,633 \text{ lbs}}{\text{SQ. FT}}$

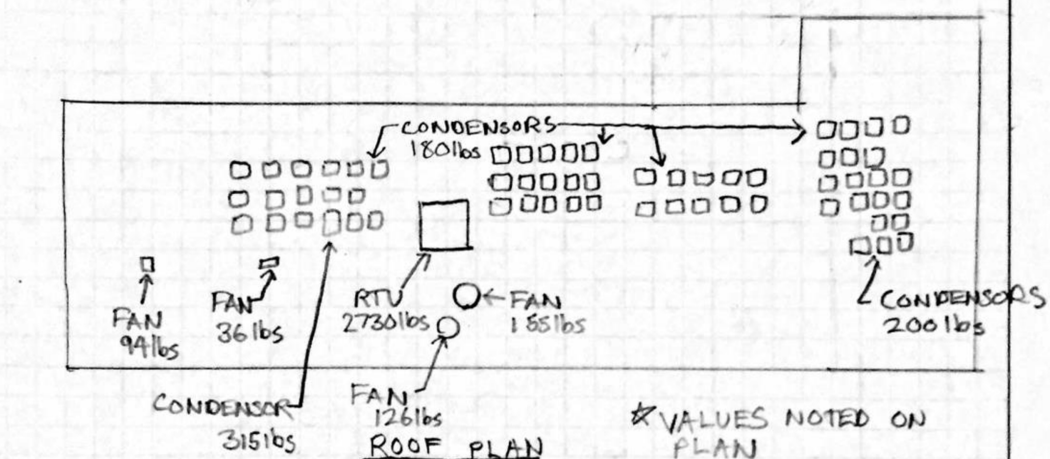
$= \frac{10,633 \text{ lbs}}{850 \text{ sqft}}$

= 12.5 psf

DESIGN LIVE LOADS

AREA	PSF	ASCE 7-10 MIN. (PSF)
LIVING UNITS	40	40
LOBBIES/STAIRS/EXITS	100	100
MECHANICAL	AS NOTED	
CORRIDORS ABOVE 1ST FLOOR	80	40
PARKING DECKS	40	40
PARKING DECKS (TOP LEVEL)	70 (40LL + 30 SNOW)	
ROOF TERRACE	100	100
LOADING DOCK	250	

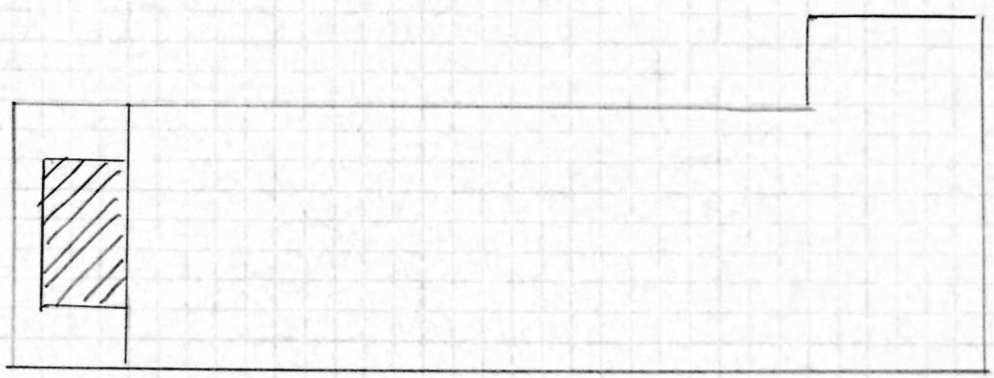
NON TYPICAL LOADS ON ROOF



ROOF PLAN

☆VALUES NOTED ON PLAN

NON TYPICAL LOADS ON 4TH FLOOR



4TH FLOOR

▨ ROOF TERRACE

- DEAD LOAD: 40 PSF TOP CHORD
  5 PSF BOTTOM CHORD
- LIVE LOAD: 100 PSF

☆VALUES NOTED IN GENERAL NOTES

FLAT ROOF SNOW LOAD + DRIFT

FROM GENERAL NOTES

$P_g = 25\,psf$
$C_e = 1.0$
$I = 1.0$
$C_t = 1.0$

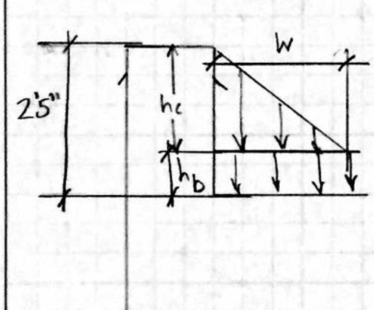
$\Big\}\; P_f = 0.7\, C_e\, C_t\, I\, p_g$

$p_f = 0.7 \cdot 1 \cdot 1 \cdot 1 \cdot 25\ psf$

$p_f = 17.5\,psf$

$p_{f\,min} = 20 \cdot I = 20 \cdot 1 = 20\,psf \Rightarrow$ controls

DRIFT AT PARAPET



$\gamma = 0.13\,p_g + 14 = 0.13 \cdot 25 + 14 = 17.25\,pcf < 30\,pcf$ ✓

$h_b = 20\,psf / 17.25\,pcf = 1.16\,ft \qquad h_c = 2.42 - 1.16 = 1.26\,ft$

Using $l_u = 0'$ for upper roof $\Rightarrow h_d = 1.5\,ft$

Using $l_u = 60'$ for lower roof $\Rightarrow h_d = 2.5\,ft$

$3/4 \cdot 1.5\,ft = 1.125\,ft$ } USE 2.5ft AS DRIFT HEIGHT
$2.5\,ft$ }

★ EXCEEDS $h_c \Rightarrow$ <u>$h_d = h_c$</u>

DRIFT WIDTH, $w = 4\,\dfrac{h_d^2}{h_c} = 4 \cdot \dfrac{1.26\,ft^2}{1.26\,ft} = 5.04\,ft < 8\,h_c$

WIND LOADS

VARIABLES FROM GENERAL NOTES

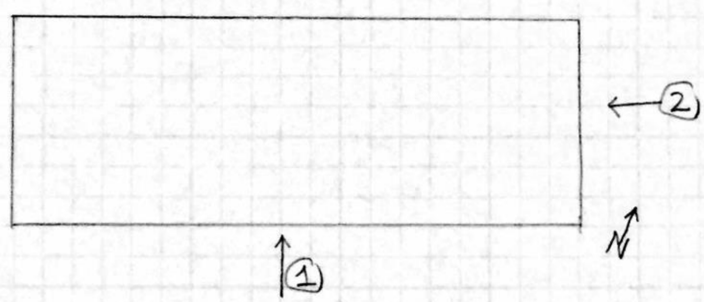
BASIC WIND SPEED: 90 MPH
IMPORTANCE FACTOR: 1.0
EXPOSURE CATEGORY: B

BUILDING HEIGHT: 54' 7 1/4"
★ I WILL USE THE ANALYTICAL PROCEDURE EVEN THOUGH THE SIMPLIFIED PROCUDERE, 6.4.2, MAY BE USED.
★ I WILL USE ASCE-7-05 AS GENERAL NOTES INDICATES

6.5.1 SCOPE

1. REGULAR SHAPED ✓
2. ✓

6.5.3
1. BASIC WIND SPEED: 90MPH
2. IMPORTANCE FACTOR I: 1.0
3.
WIND DIRECTIONS CONSIDERED:



SURFACE ROUGHNESS CATEGORY: B
EXPOSURE CATEGORY: B

TABLE 6-3

	Z	K_z★
2^{ND} FLOOR:	14.5ft	0.57
3^{RD} FLOOR:	24.17ft	0.62
4^{TH} FLOOR:	33.83ft	0.72
5^{TH} FLOOR:	43.5ft	0.78
ROOF:	54.6ft	0.83

★CASE 2, EXPOSURE B, I WILL NOT DESIGN USING FIG 6-10

4.
6.5.7.1: NO APPLICABLE HILL OR ESCARPMENT
$\Rightarrow K_{ZT} = 1.$

5. ASSUME LOW-RISE BUILDING IS RIGID
$\Rightarrow G = 0.85$

6. BUILDING IS ENCLOSED

7. $GC_{pi} = \pm 0.18$, FIG 6-5

8. FIG 6-6 ★NEGLECTING FIG 6-10 FOR LOW-RISE
WINDWARD: $C_p = 0.8$
LEEWARD: FOR DIRECTION ① $L/B = 74'3^{5/8}" / 256'11^{5/8}" = 0.29$
$\Rightarrow C_p = -0.5$
② $L/B = 1/0.29 = 3.46$
$\Rightarrow C_p = -0.23$
SIDE WALLS: $C_p = -0.7$
ROOF: FOR DIRECTION ① $h/L = 54.6'/74'3^{5/8}" = 0.73$
AREA $\geq$ 1000sqft
$\Rightarrow$ USE REDUCTION OF 0.8 WHEN APPLICABLE

FOR $0 - h/2 \Rightarrow C_p = -0.97$ $>h \Rightarrow C_p = -0.60$
$h/2 - h \Rightarrow C_p = -0.80$

② $h/L = 54.6'/256'11^{5/8}" = 0.21$
$0 - h \Rightarrow C_p = -0.9$
$h - 2h \Rightarrow C_p = -0.5$
$>2h \Rightarrow C_p = -0.3$

9 $q_z = 0.00256 K_z K_{zt} K_d V^2 I$

$K_d = 0.85$

	q_z
2^{ND} FLOOR:	10.0
3^{RD} FLOOR:	11.0
4^{TH} FLOOR:	12.7
5^{TH} FLOOR:	13.8
ROOF:	14.7

10. $p = q_x GC_p - q_h(GC_{pi})$

WIND FORCE AT EACH LEVEL:

$$p_{story} = q_x GC_p - q_h(GC_{pi}) - [q_h GC_p - q_h(GC_{pi})]$$

DIRECTION (1)

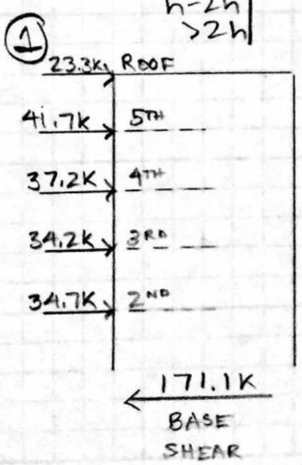
LEVELS	p_{story} (psf)	Floor Ht.(ft)	F_{story} (k) ($B = 256' 11 5/8''$)
2ND	13.1	10.3	34.7
3RD	13.7	9.7	34.2
4TH	14.9	9.7	37.2
5TH	15.6	10.4	41.7
ROOF	16.2	5.6	23.3

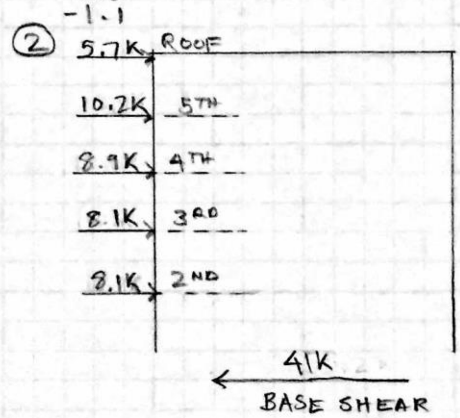
	p (+GCpi) (psf)	p (−GCpi) (psf)
SIDE WALLS	−11.4	−6.1
0−h/2	−14.8	−9.5
h/2−h	−12.6	−7.4
>h	−10.1	−4.9

DIRECTION (2)

LEVELS	p_{story} (psf)	Floor Ht(ft)	F_{story} (k) ($B = 74' 3 5/8''$)
2ND	10.6	10.3	8.1
3RD	11.3	9.7	8.1
4TH	12.4	9.7	8.9
5TH	13.2	10.4	10.2
ROOF	13.8	5.6	5.7

	p (+GCpi) (psf)	p (−GCpi) (psf)
SIDE WALLS	−8.9	−3.6
0−h	−13.9	−8.6
h−2h	−8.9	−3.6
>2h	−6.4	−1.1





3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

SEISMIC LOADS

- EFFECTIVE SEISMIC WEIGHT

	SQFT.	DL (psf)	INT. WALL (psf)	W(k)
ROOF	14,000	17	12.5	238
5^{TH}	14,000	25	12.5	525
4^{TH}	16,460	25	12.5	617.25
3^{RD}	16,460	25	12.5	617.25
2^{ND}	16,460	160	12.5	2,840
ROOF TERRACE (ON 4TH)	704	43		30.3
			EXT. WALLS	953.8
			CONDENSORS	14.5
			FANS	0.3
			RTU	2.73
			Σ =	5,840 K

(ASCE 7-05)

EQUIVALENT LATERAL FORCE DESIGN

- OCCUPANCY CATEGORY: II
- SEISMIC IMPORTANCE FACTOR: $I_E = 1.0$
  - $S_S = 0.153$
  - $S_1 = 0.050$
- SITE CLASS: D
  - $S_{DS} = 0.163$
  - $S_{D1} = 0.081$
- SDC: B
- R (NOT SEISMICALLY DETAILED) = 3

APPROXIMATE FUNDAMENTAL PERIOD

$T_a = C_t h_n^x$

TABLE 12.8-2 ⇒ $C_t = 0.02$, $x = 0.75$

$T_a = 0.02 \cdot 54.6ft^{0.75} = 0.4s$

$T_L = 8s$ (FIG 22-15)

$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{0.163}{\left(\frac{3}{1}\right)} = 0.0543 > 0.01$ ✓

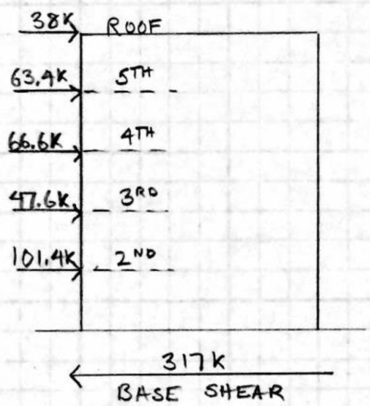
$T \le T_L \Rightarrow C_s \le \frac{0.081}{0.4\left(\frac{3}{1}\right)} \le 0.0675$

USE $C_s = 0.0543$

V = 5,840 K · 0.0543 = 317 K

STORY	w_x	h_x	$w_x h_x^{K}$ *	C_{Vx}	F_x(K)
ROOF	300.0	54'7 1/4"	16,381.3	0.12	38.0
5TH	636.3	43'6"	27,679.0	0.20	63.4
4TH	836.7	33'10"	28,308.4	0.21	66.6
3RD	860.6	24'2"	20,797.8	0.15	47.6
2ND	3,083.4	14'6"	44,709.3	0.32	101.4
			Σ = 137,875.8		

* K = 1



* SAME FORCES IN BOTH DIRECTIONS

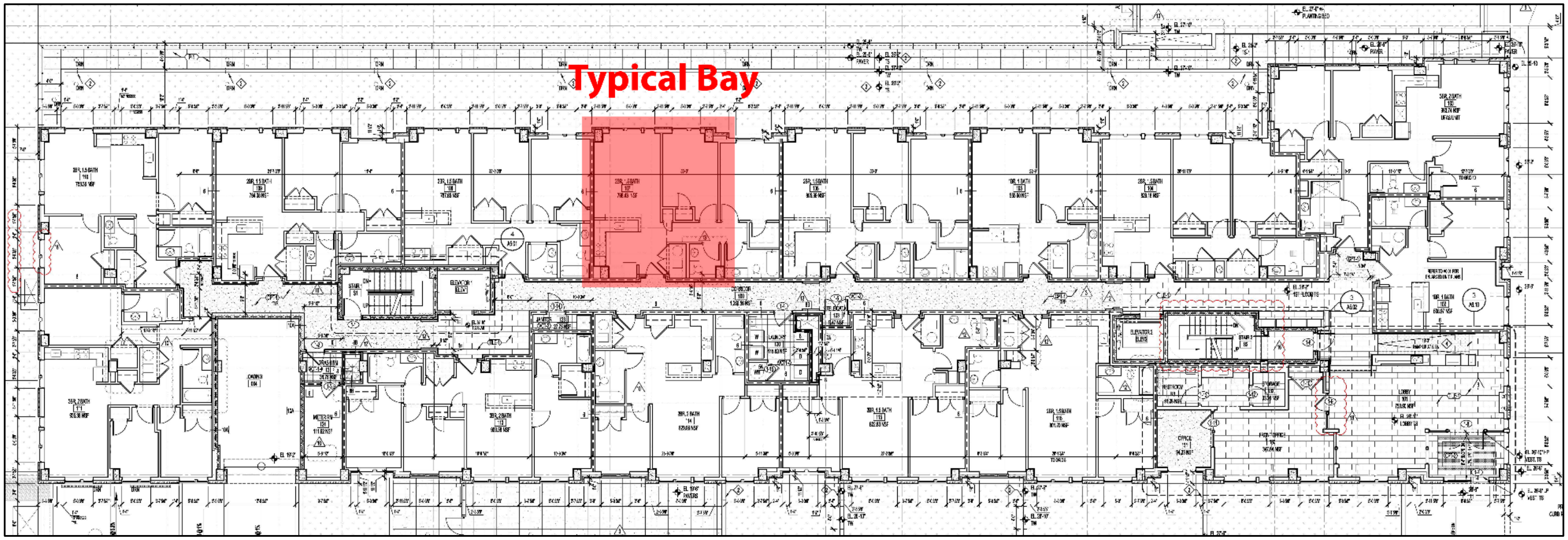
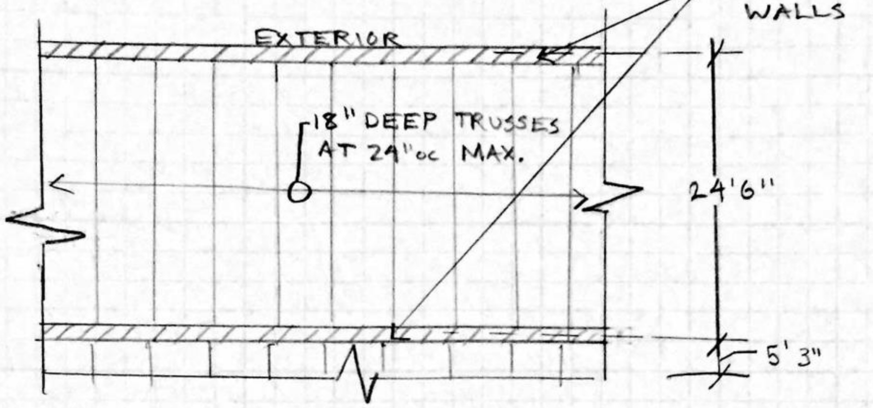


Figure 3- Area in red highlights location of typical bay

Typical Bay Location

Figure 3 shades in red the location of the typical bay I will use to analyze the existing structural system and the three alternative systems.

GRAVITY LOADS SPOT CHECK



TYPICAL FLOOR PLAN

- LOADING ON WOOD TRUSS

DEAD LOAD

25 psf · 24"oc max = 50 plf

LIVE LOAD

40 psf · 24" oc max = 80 plf

- LOADING ON EXTERIOR BEARING WALL FROM FLOOR

DEAD LOAD

(25 psf · 24.5')/2 = 307 plf

LIVE LOAD

(40 psf · 24.5')/2 = 490 plf

- LOADING ON INTERIOR WALL FROM FLOOR

DEAD LOAD

[25 psf · (24.5' + 5.25')]/2 = 335 plf

LIVE LOAD

[40 psf · (24.5' + 5.25')]/2 = 595 plf

- LOADING ON EXTERIOR BEARING WALL FROM ROOF

DEAD LOAD

(17 psf · 24.5')/2 = 208.25 plf

LIVE LOAD

(30 psf · 24.5')/2 = 367.5 plf

- LOADING ON INTERIOR BEARING WALL FROM ROOF

DEAD LOAD

[17 psf · (24.5' + 5.25')] /2 = 253 plf

LIVE LOAD

[30 psf · (24.5' + 5.25')] /2 = 447 plf

- EXTERIOR WALL DEAD LOAD

=> 48 psf · 29 ft + 11 psf · 10.6 ft = 1,508.6 plf

- INTERIOR BEARING WALL DEAD LOAD

WEIGHT OF WALL ESTIMATE:

5/8 GWB BOTH SIDES: 2½ psf · 2 =	5 psf
2×6 STUDS:	1.4 psf
INSULATION:	1 psf
	8 psf

=> 8 psf · (39.6 ft) = 317 plf

TOTAL LOAD ON 2ND FLOOR BEARING WALL

EXTERIOR - (307 + 490) · 3 + (209 + 368) + 1,509

=> 4,477 plf

INTERIOR - (335 + 598) · 3 + (253 + 447) + 317

=> 3,807 plf

USING ASD

- FOR SAWN LUMBER

$$F'_c = C_D \times C_M \times C_t \times C_F \times C_i \times C_p \times F_c$$

- STUD INFORMATION
  - HEIGHT = 9.7ft
  - (1) 2x6 @ 16" oc
  - 19% MAXIMUM MOISTURE CONTENT
  - SPF/HF No.1/No.2
  - F_c (PARA) = 1,150 psi
  - E'_{min} = 0.51E6 psi

- FOR D+L ⇒ $C_D = 1.0$

- MAX M.C. = 19% ⇒ $C_M = 1$

- ASSUMING NO ELEVATED TEMPERATURES ⇒ $C_t = 1$

- FOR 2x6 ⇒ $C_F = 1.1$

- ASSUME $C_i = 1$

  - SLENDERNESS ASSUMING SHEATHING BRACES WEAK AXIS:

$$\frac{K l_1}{d_1} = \frac{9.7ft \cdot 12}{5.5in} = 21.2 < 50 \checkmark$$ * CONTROLS

$$\frac{K l_2}{d_2} = 0$$

$$F_c^* = 1{,}150 \times 1 \times 1 \times 1 \times 1.1 \times 1 = 1{,}265 \text{ psi}$$

$$F_{cE} = \frac{0.822 \cdot 0.51E6}{(21.2)^2} = 936 \text{ psi}$$

$$F_{cE}/F_c^* = 0.74$$

$$C_p = \frac{1+0.74}{2\cdot 0.8} - \sqrt{\left[\frac{1+0.74}{2\cdot 0.8}\right]^2 - \frac{0.74}{0.8}}$$

$$C_p = 0.58$$

$$F_c' = 1{,}265\,\text{psi}\cdot 0.58 = 733.5\,\text{psi}$$

$$733.5\,\text{psi}\cdot 1.5\,\text{in}\cdot 5.5\,\text{in} = \frac{6{,}051\,\text{lbs}}{(16''/12'')} = 4{,}538\,\text{plf}$$

CAPACITY OF (1) 2×6@16"oc SPF No1/No2:

⇒ 4,538plf

LOAD CHECK

EXTERIOR WALL:

D+L = 4,477plf < 4,538plf

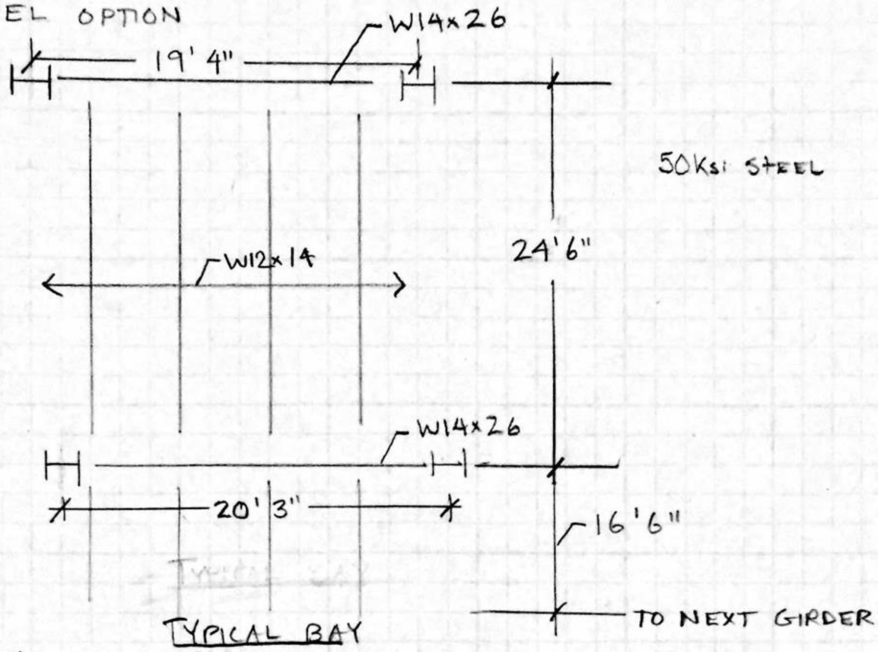
∴ GOOD

INTERIOR WALL:

D+L = 3,807plf < 4,538plf

∴ GOOD

NONCOMPOSITE
STEEL OPTION



LOADING

- DEAD LOAD
  - ALLOW 35 PSF FOR DECK & CONCRETE TOPPING PLUS FINISH
  - 2 PSF FOR SOUND INSULATION
  - 4 PSF FOR MECHANICAL DUCTS
  - 5 PSF FOR SELF WEIGHT
  - 2½ PSF FOR CEILING DRYWALL
  - TOTAL 48.5 PSF
    ⇒ USE 49 PSF

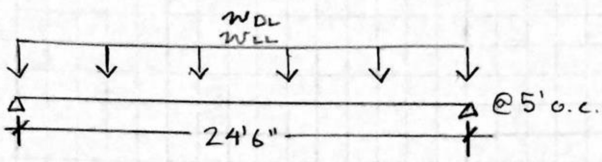
- LIVE LOAD
  - 40 PSF

INFILL BEAMS

- 24' 6" LENGTH
- TRY 5' SPACING

LRFD

-JOISTS



STRENGTH

$w_{DL} = 49\ PSF \cdot 5' = 245\ PLF$

$w_{LL} = 40\ PSF \cdot 5' = 200\ PLF \Rightarrow K_{LL}A_T = 2 \cdot 24.5 \cdot 5 = 245 ft^2 < 400 ft^2$

LL UNREDUCIBLE

1.2D+1.6L

$w_{D+L} = 1.2(245) + 1.6(200) = 614\ PLF$

$$M_U = \frac{614\ PLF \cdot (24.5ft)^2}{8} = 46.1\ K\text{-}ft$$

ASSUMING DECK BRACES COMPRESSION FLANGE

FOR W10×12

$\phi M_n = 46.9\ K\text{-}ft > M_U = 46.1\ K\text{-}ft \quad \therefore$ GOOD

SERVICABILITY

FOR W10×12, $I_x = 53.8\ in^4$

$L/360 = 24.5ft/360 = 0.817"$

$$\Delta_{LL} = \frac{5 \cdot 0.200\ KLF \cdot (24.5ft)^4 \cdot (1728\ in^3/ft^3)}{384 \cdot 29{,}000\ Ksi \cdot 53.8\ in^4}$$

$\Delta_{LL} = 1.04\ in > 0.817\ in \quad \therefore$ NO GOOD

REQ'D I_x

$$I_x = \frac{5 \cdot 0.200\ KLF \cdot (24.5ft)^4 (1728\ in^3/ft^3)}{384 \cdot 29{,}000\ Ksi \cdot 0.817"}$$

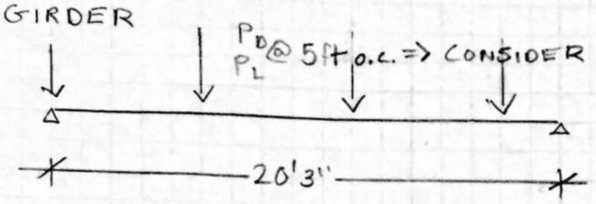
$I_x = 68.5\ in^4$

FOR W12×14

$I_x = 88.6\ in^4 > 68.5\ in^4 \quad \therefore$ GOOD

WEIGHT CHECK ⇒ $14\ PLF / 5ft = 2.8\ psf < 5\ psf$ ✓

INTERIOR
GIRDER



STRENGTH

$w_{DL} = 49\ \text{PSF} \cdot (24.5+16.5)/2 = 1{,}004.5\ \text{PLF}$

$w_{LL} = 40\ \text{PSF} \cdot (24.5+16.5)/2 = 820\ \text{PLF}$

1.2D+1.6L

$w_{D+L} = 1.2(1{,}004.5) + 1.6(820) = 2{,}517.4\ \text{PLF}$

$M_U = \frac{2{,}517.4\ \text{PLF}\ (20.25\ \text{ft})^2}{8} = 129.1\ \text{K-ft}$

FOR W14x26: L_b of 5 ft, $C_b = 1$, $F_y = 50\ \text{Ksi}$

$M_U = 141\ \text{K-ft} > 129.1\ \text{K-ft}$ ∴ GOOD

SERVICABILITY

FOR W14x26, $I_x = 245\ \text{in}^4$

$L/360 = 20.25'/360 = 0.675\ \text{in}$

$\Delta_{LL} = \frac{5 \cdot 0.82\ \text{KLF} \cdot (20.25\ \text{ft})^4 \cdot (1728\ \text{in}^3/\text{ft}^3)}{384 \cdot 29{,}000\ \text{Ksi} \cdot 245\ \text{in}^4}$

$\Delta_{LL} = 0.437" < 0.675"$ ∴ GOOD

WEIGHT ASSUMPTION => 26 PLF / [(24'6" + 16'6")/2] = 1.27 PSF < 5 ✓

NON COMPOSITE DECK

TOTAL LOAD = 49 PSF + 40 PSF = 89 PSF

TRY 1.0C24

5'0" SPAN
CONTINUOUS OVER 3 SPANS

FOR 1.0C24
- ALLOWABLE TOTAL UNIFORM LOAD
⇒ 123 psf > 89 psf

∴ GOOD

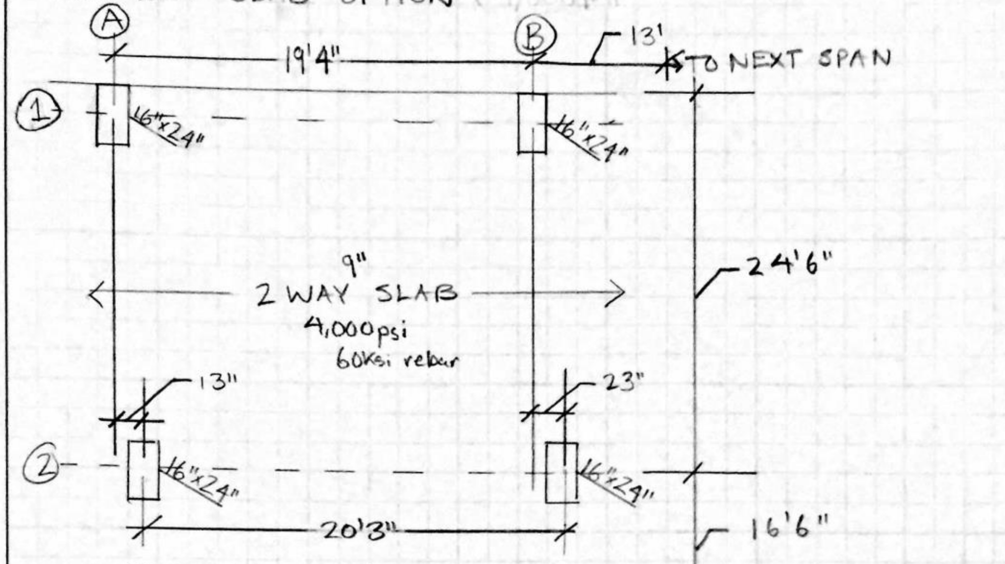
- L/240
⇒ 57 psf > 40 psf

∴ GOOD

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARE
3-0137 — 200 SHEETS — FILLER

COMET

TWO WAY SLAB OPTION (4,000 psi)



TYPICAL LAYOUT (PRELIMINARY COLUMN SIZES SHOWN)

DIRECT DESIGN METHOD

-REQUIRMENTS

1) 3 CONTINUOUS SPANS EACH DIRECTION ✓

2) LONG/SHORT SPAN = 24'6"/19'4" = 1.3 < 2 ✓

3) 19'4" − 13' = 6'4" < 1/3(19'4") = 6'5 1/3" ✓

24'6" − 16'6" = 8' < 1/3(24'6") = 8'2" ✓

4) 0.1 × 19'4" = 23.2" > 23" ✓

5) GRAVITY LOADS ONLY ✓

6) DEAD LOAD (ASSUMING 10" SLAB)

FLOOR FINISH = 1 psf
BATT INSULATION = 1.75 psf
GWB CEILING = 2.5 psf
SELF WEIGHT = (10/12) 150 = 125 psf
MECHANICAL = 4 psf

TOTAL = 135 psf

LIVE LOAD = 40psf < 2(135) = 270psf ✓

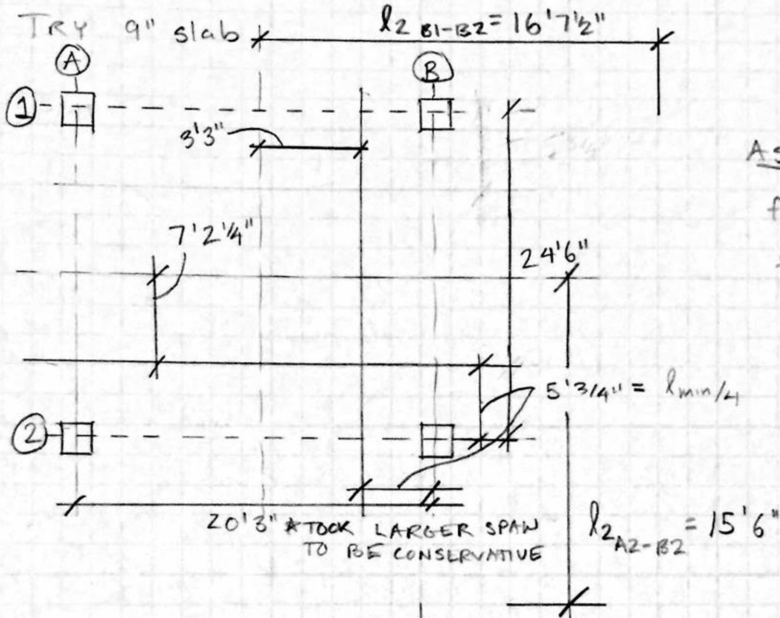
7) NO BEAMS ✓

TRIAL THICKNESS FROM ACI TABLE 9.5(c)

EXTERIOR PANEL ⇒ $\ell_n/30$

$(24'6" - 2')/30 = 9$ in

TRY 9" slab



ASSUMPTIONS

f'c = 4000 psi

fy = 60,000

SPAN: A2–B2	SPAN: B1–B2
$\ell_1 = 20'3"$	$\ell_1 = 24'6"$
$\ell_n = 18'11"$	$\ell_n = 22'6"$
$\ell_2 = 15'6"$	$\ell_2 = 16'7\frac{1}{2}"$
$A_T = 415.1\ ft^2$ ($K_{LL} = 1$), L = 39.5 psf *JUST USE 40psf	$A_T = 396 < 400$, L = 40psf
q_u(Ksf) = 0.226	q_u(Ksf) = 0.226
M_0(k-ft) = 156.7	M_0(K-ft) = 237.8
COEFFICIENTS −0.65 0.35 −0.65	COEFFICIENTS −0.26 0.52 −0.70
MOMENTS(K-ft) −102 55 −102	MOMENTS(K-ft) −62 124 −166.5

SPAN A2-B2

-POSITIVE MOMENT

- COLUMN STRIP

$\alpha_f = 0 \Rightarrow +M_c = 0.6(55) = 33\ \text{K-ft}$

- ESTIMATE d as 9in − 1.6in = 7.4in & j as 0.95

$- As_{req} = \dfrac{33\ \text{K-ft} \cdot 12''/_1}{0.9 \times 60 \times 0.95 \times 7.4} = 1.04\ \text{in}^2$

$a = \dfrac{1.04\ \text{in}^2 \times 60}{0.85 \times 4 \times (10'\ 1\tfrac{1}{2}'')} = 0.15\text{in}$

$c = 0.15\text{in} / 0.85 = 0.178\ \text{in}$

$3/8\,(7.4) = 2.8\ \text{in} > c \Rightarrow \phi = 0.9$

$jd = 7.4\text{in} - \dfrac{0.15}{2} = 7.32\ \text{in}$

$As_{req} = \dfrac{33\ \text{K-ft} \times 12''/_1}{0.9 \times 60 \times 7.32\text{in}} = 1.00\ \text{in}^2$

$As_{min} = 0.0018 \times 10'1\tfrac{1}{2}'' \times 9'' = 1.97\ \text{in}^2$ ☆ CONTROLS

- MIDDLE STRIP (HALF OF STRIP)

$+M_M = 22\ \text{K-ft}/2 = 11\ \text{K-ft}$

d = 7.4in.

$As_{min} = 0.0018 \times 7'\,2\tfrac{1}{4}'' \times 9'' = 1.4\ \text{in}^2$

$As_{req} = \dfrac{11\ \text{K-ft} \times 12''/_1}{0.9 \times 60 \times 0.95 \times 7.4} = 0.3\ \text{in}^2$

☆USE As min

- NEGATIVE MOMENT
  - COLUMN STRIP

$\alpha_f = 0 \Rightarrow -M_c = 0.75(102) = 76.5\,K\text{-}ft$

$-A_{s\,req} = \dfrac{76.5K\text{-}ft \cdot 12''/_1}{0.9 \times 60 \times 0.95 \times 7.4} = 2.42\,in^2$

$a = \dfrac{2.42\,in^2 \times 60}{0.85 \times 4 \times (10'1\tfrac{1}{2}'')} = 0.35$

$c = 0.41 < \tfrac{3}{8}d \Rightarrow \phi = 0.9$

$jd = 7.4 - 0.35/2 = 7.22\,in$

$A_{s\,req} = \dfrac{76.5K\text{-}ft \times 12''/_1}{0.9 \times 60 \times 7.22} = 2.35\,in^2$

  - MIDDLE STRIP (HALF OF STRIP)

$-M_M = (102 - 76.5)/2 = 12.75\,K\text{-}ft$

$-A_{s\,req} = \dfrac{12.75\,K\text{-}ft \cdot 12''/_1}{0.9 \times 60 \times 0.95 \times 7.4} = 0.40\,in^2$

★ USE $A_{s\,min}$ of $1.4\,in^2$

SPAN B1-B2

- POSITIVE MOMENT
  - COLUMN STRIP

$\alpha_f = 0 \Rightarrow +M_c = 0.6(124) = 74.4\,k\text{-}ft$

$-A_{s\,req} = \dfrac{74.4\,K\text{-}ft \cdot 12''/_1}{0.9 \times 60 \times 0.95\,(7.4 + 0.5)} = 2.20\,in^2$

$a = \dfrac{2.20\,in^2 \times 60}{0.85 \times 4 \times (10'1\tfrac{1}{2}'')} = 0.32$

$c = 0.32/0.85 = 0.38 < \tfrac{3}{8}(7.9) = 2.96\,in \Rightarrow \phi = 0.9$

$jd = 7.9\,in - 0.32/2 = 7.74\,in$

- $A_{s\,req} = \dfrac{74.4\text{K-ft} \cdot 12''/1}{0.9 \times 60 \times 7.74\text{in}} = 2.14\text{in}^2$ * CONTROLS

- $A_{s\,min} = 1.97\text{in}^2$

- MIDDLE STRIP (HALF OF STRIP)

$+M_M = (124 - 74.4)/2 = 24.8\text{K-ft}$

- $A_{s\,req} = \dfrac{24.8\text{K-ft} \cdot 12''/1}{0.9 \times 60 \times 0.95 \times 7.9} = 0.73\text{in}^2$

- $A_{s\,min} = 0.0018 \times 3'3'' \times 9'' = 0.63\text{in}^2$

$a = \dfrac{0.73\text{in}^2 \times 60}{0.85 \times 4 \times (3'3'')} = 0.33\text{in}$

$c = 0.33/0.85 = 0.39\text{in} < 2.96\text{in} \Rightarrow \phi = 0.9$

$jd = 7.9\text{in} - 0.33/2 = 7.73\text{in}$

$A_{s\,req} = \dfrac{24.8\text{K-ft} \cdot 12''/1}{0.9 \times 60 \times 7.73\text{in}} = 0.71\text{in}^2$

- NEGATIVE MOMENT

- COLUMN STRIP

$\alpha_f = 0 \Rightarrow -M_c = 0.75(166.5) = 124.9\text{K-ft}$

- $A_{s\,req} = \dfrac{124.9\text{K-ft} \cdot 12''/1}{0.9 \times 60 \times 0.95 \times 7.9\text{in}} = 3.70\text{in}$

- $A_{s\,min} = 1.97\text{in}^2$

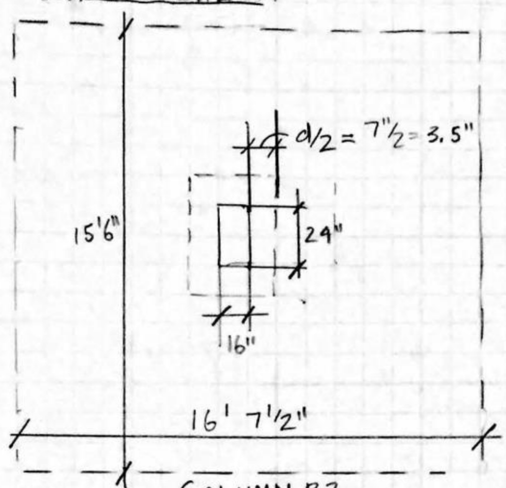
$a = \dfrac{3.70\text{in}^2 \times 60}{0.85 \times 4 \times (10'1\tfrac{1}{2}'')} = 0.537\text{in}$

$c = 0.537/0.85 = 0.63\text{in} < 2.96\text{in} \Rightarrow \phi = 0.9$

$jd = 7.9\text{in} - 0.537\text{in}/2 = 7.63\text{in}$

$A_{s\,req} = \dfrac{124.9\text{K-ft} \cdot 12''/1}{0.9 \times 60 \times 7.63\text{in}} = 3.64\text{in}^2$

TWO-WAY
SHEAR CHECK



COLUMN B2

$b_o = 2(16" + 7" + 24" + 7") = 108\ in$

$V_u = 0.226\ Ksf \cdot [(16'7\tfrac{1}{2}")(15'6") - (23" \times 31")] = 58\ K$

$\phi V_c = 4\lambda \sqrt{f'c}\ b_o \cdot d \cdot 0.75$

$\phi V_c = 4 \cdot 1 \cdot \sqrt{4000} \cdot 108\ in \cdot 7\ in \cdot 0.75$

$\phi V_c = 143\ K > V_u$

∴ CONCRETE STRENGTH ADEQUATE

USE MINIMUM SHEAR REINFORCEMENT

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

- MIDDLE STRIP (HALF OF STRIP)

$(-M_{u}) = (166.5 - 124.4)/2 = 29.14\ \text{K-ft}$

$-A_{s\,req} = \dfrac{29.14\text{K-ft} \cdot 12''/_{1}}{0.9 \times 60 \times 0.95 \times 7.9\text{in}} = 0.863\ \text{in}^2$

$-A_{s\,min} = 0.63\ \text{in}^2$

$a = \dfrac{0.863\text{in}^2 \times 60}{0.85 \times 4 \times (3'3'')} = 0.39\text{in}$

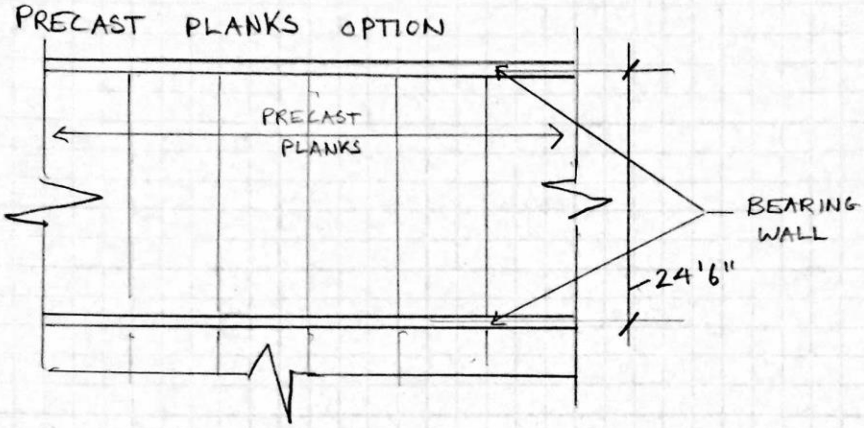
$c = a/0.85 = 0.46\text{in} < 2.96\text{in} \Rightarrow \phi = 0.9$

$jd = 7.9\text{in} - 0.39\text{in}/2 = 7.70\text{in}$

$-A_{s\,req} = \dfrac{29.14\text{k-ft} \cdot 12''/_{1}}{0.9 \times 60 \times 7.70\text{in}} = 0.84\text{in}^2$

POSITIVE INTERIOR MOMENT	$A_{s\,req}/A_{s\,min}$ (in²)	(No.) BAR SIZE
COLUMN STRIP		
A2-B2	1.97	(5)#6
B1-B2	2.14	(5)#6
MIDDLE STRIP		
A2-B2	1.40	(4)#6
B1-B2	0.71	(4)#4
NEGATIVE INTERIOR MOMENT		
COLUMN STRIP		
A2-B2	2.35	(3)#9
B1-B2	3.64	(4)#9
MIDDLE STRIP		
A2-B2	1.40	(4)#6
B1-B2	0.84	(5)#4

PRECAST PLANKS OPTION



MANUFACTERER: OLD CASTLE PRECAST

PRODUCT: ELEMATIC HOLLOW-CORE PLANK

TRY E8"x48" w/ NO TOPPING (ADD 1/2" GYPCRETE FOR FIRE RATING)

UNIFORMELY DISTRIBUTED SUPER IMPOSED LOAN
- LIVE LOAD PLUS DEAD LOAD ADDITIONAL TO S.W.

1/2" GYPCRETE => 6.9 psf/3/4" · 4/3" · 1/2" = 6.13 psf

1/4" FLOOR MAT => 0.75 psf

29/32" SUBFLOOR => 3 psf

5/8 GWB => 2 1/2 psf

3 1/2" BATT INSULATION => 1.75 psf

MECHANICAL => 4 psf

FLOOR FINISH => 1 psf

DEAD TOTAL = 20 psf

DEAD + LIVE = 20 psf + 40 psf
= 60 psf

ULTIMATE BENDING MOMENT (PER UNIT) ⇒ $\frac{(60\,psf)\cdot 4ft\cdot (24'6")^2}{8} = M_u$

$M_u = 18$ K-ft

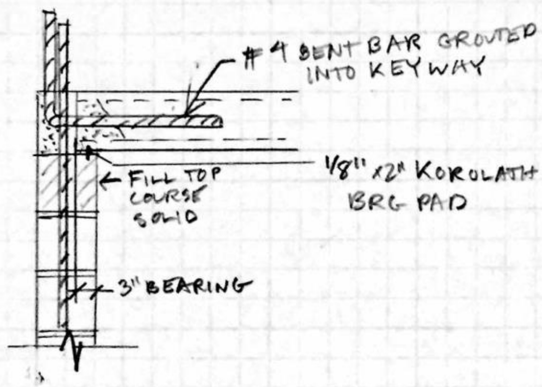
FOR 20_08704

MAX TOTAL LOAD = 877 psf > 60 psf

$\phi M_n = 58.88$ K-ft > 18. K-ft

∴ GOOD

<u>CONNECTION TO CMU BEARING WALL</u>

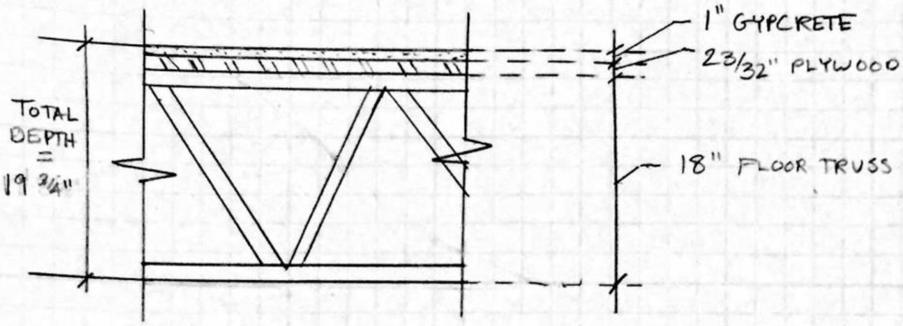


System Comparison

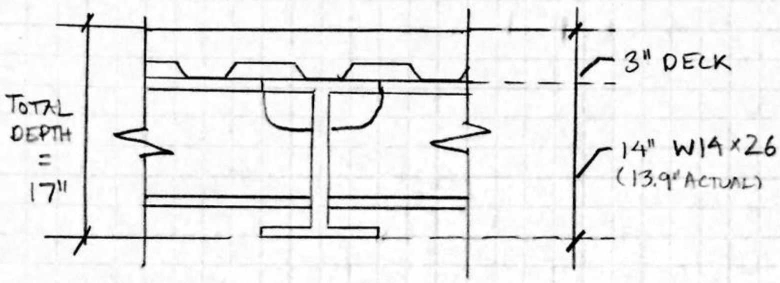
	Systems			
Parameters	Existing Wood Truss Joists	Non-Composite Joists and Girder	Flat Plate Two-Way Slab	Precast Planks
Thickness (in)	19	17	9	8
Weight (psf)	13	40	113	61
Material Cost ($/sq.ft.)	6.24	8.80	5.95	7.80
Installation Cost ($/sq.ft.)	4.01	3.19	9.20	2.57
Total Cost ($/sq.ft.)	10.25	11.99	15.15	10.37
Advantages	-Lowest Cost -Lightest -Voids for Mechanical -Stable during construction	-Light weight system -Relatively Low Cost	-Small slab thickness -Durable	-Thinnest thickness -Low Cost -Efficient with prestressed strands -Easy construction
Disadvantages	-Largest Structural depth	-Vibrations could cause uncomfortablility	-Heaviest -Most Expensive	-Heavy Structure -Transportation can cause trouble as planks are fragile and large

STRUCTURE THICKNESSES

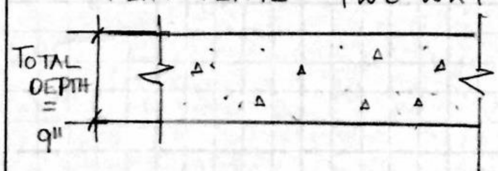
- EXISTING SYSTEM (WOOD TRUSSES)



- NON COMPOSITE STEEL



- FLAT PLATE TWO WAY SLAB



- PRECAST PLANKS

WEIGHT OF SYSTEMS

EXISTING WOOD SYSTEM

- 18" FLOOR JOIST = 3 PSF
- GYPCRETE = 10 PSF

TOTAL = 13 PSF

NON COMPOSITE STEEL

- JOISTS = 3 PSF
- GIRDERS = 1.5 PSF
- DECKING - 35 PSF

TOTAL = 40 PSF

FLAT PLAT TWO-WAY

- 9 IN SLAB = 113 PSF

TOTAL = 113 PSF

PRE CAST PLANKS

- GYPCRETE = 7 PSF
- PLANKS = 54 PSF

TOTAL = 61 PSF

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER

COMET

ASSEMBLY COSTS

(RSMEANS 2014)

- NON COMPOSITE STEEL OPTION

20x25 BAY, TOTAL LOAD = 90 PSF, 4 JOISTS

COST PER SQ FT.

MAT = 8.80 INST. = 3.19 TOTAL = 11.99

- FLAT PLATE TWO-WAY SLAB

20x25 BAY, 75 PSF SUPERIMP. LOAD, 20 MIN COLUMN SIZE
9in SLAB, 188 PSF TOTAL LOAD

COST PER SQ FT.

MAT = 5.95 INST. = 9.20 TOTAL = 15.15

- PRECAST PLANKS

25 FT SPAN, 75 PSF SUPERIMPOSED LOAD, 8 in DEPTH
55 PSF DL, 130 PSF TOTAL LOAD.

COST PER SQ FT.

MAT = 7.80 INST. = 2.57 TOTAL = 10.37

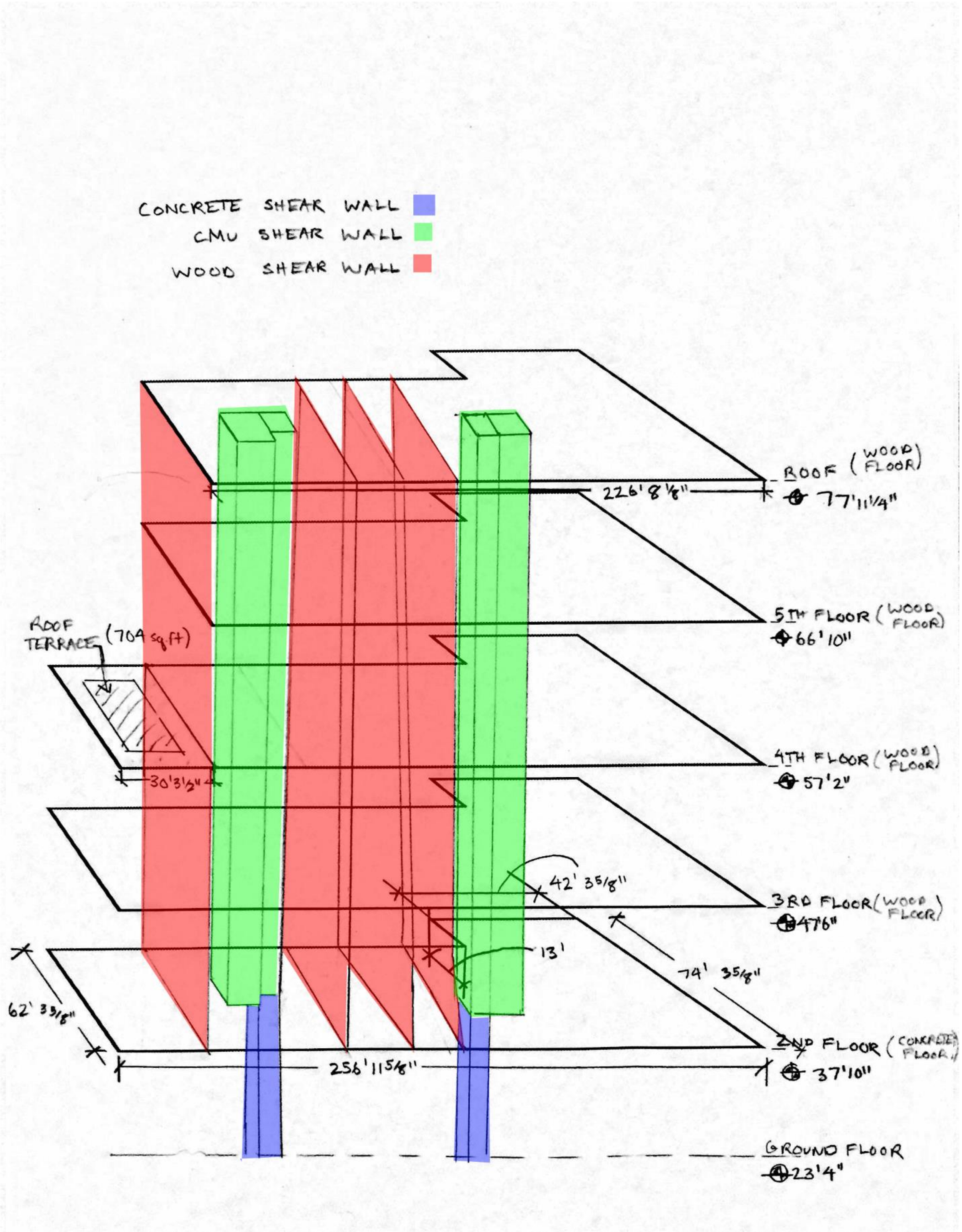
- EXISTING SYSTEM

16" OPEN WEB JOISTS, 16" O.C.
☆ COST ESTIMATE COULD BE INFLATED DUE TO ACTUAL SPACING BEING 24" O.C NOT 16" O.C

COST PER SQ FT.

MAT = 6.24 INST. 4.01 TOTAL = 10.25

Lateral System Overview



Typical Lateral Floor Plan

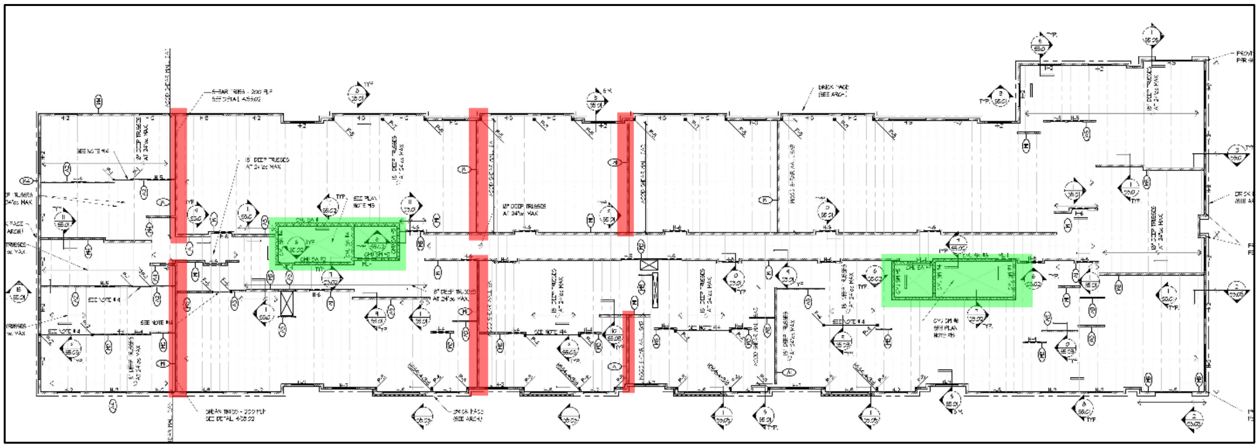


Figure 3 – Typical Floor Plan, Floor 2 through Floor 5

Wood Shear Walls   CMU Shear Walls

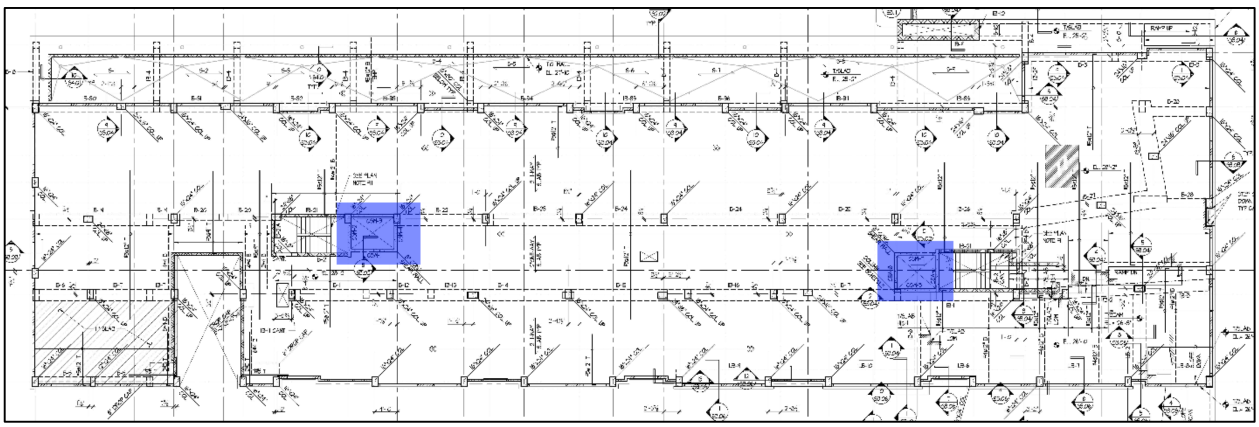


Figure 4 – Ground Floor Plan

Concrete Shear Walls

Modeling

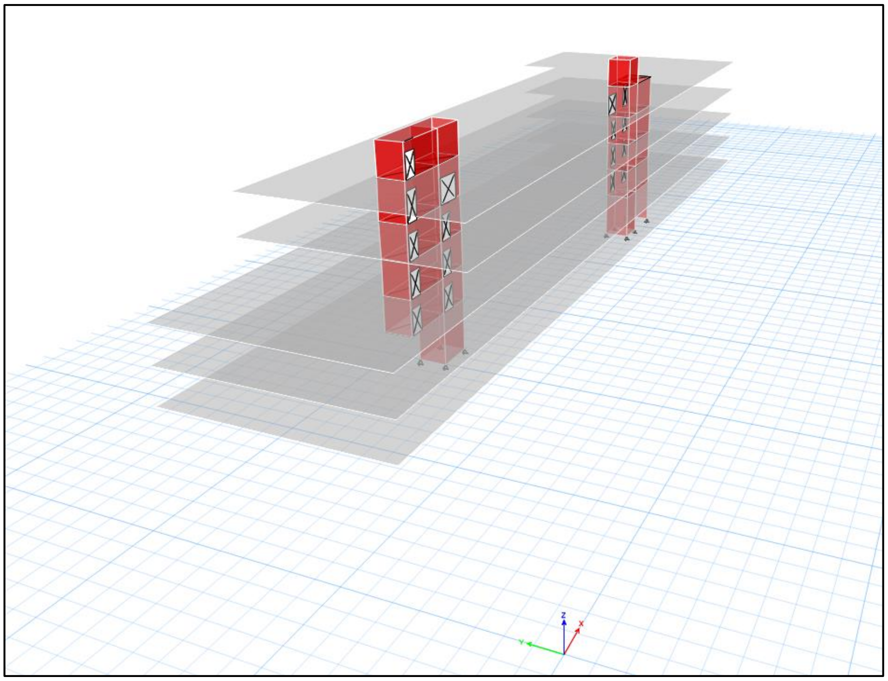


Figure 5 – User View of ETabs Model

The lateral system of Jackson Crossing was analyzed using ETABS. Because the concrete and masonry shear walls are considerably more rigid than the wood shear walls, only the concrete and masonry shear walls were modeled with a rigid diaphragm to allow them to take the full lateral force. The wood shear walls were analyzed separately by hand calculations while considering the diaphragm as flexible to account for the wood shear wall taking its full tributary force.

To calculate seismic forces through ETABS, self-weights and applicable dead loads were applied to the lateral model. The self-weight of the floor systems were accounted for as a weight per unit volume in a material assigned to a slab section. Additional weight including superimposed dead loads and rooftop units were applied as area dead loads over the whole floor at each level.

Wall openings were modeled in the shear walls to account for openings for thresholds in the elevator core and stairwell. The shear walls were fixed at their base by pin connections at each end of the wall.

Model Reactions

Base Shear

Base Shear (k)

Wind Y Direction		Wind X Direction		Seismic Y Direction		Seimic X Direction	
ETABS	Hand Calc.	ETABS	Hand Calc.	ETABS	Hand Calc.	ETABS	Hand Calc.
165.9	171.1	39.1	41.0	177.4	317	118.2	317

Story Drift

Story Drift (in)

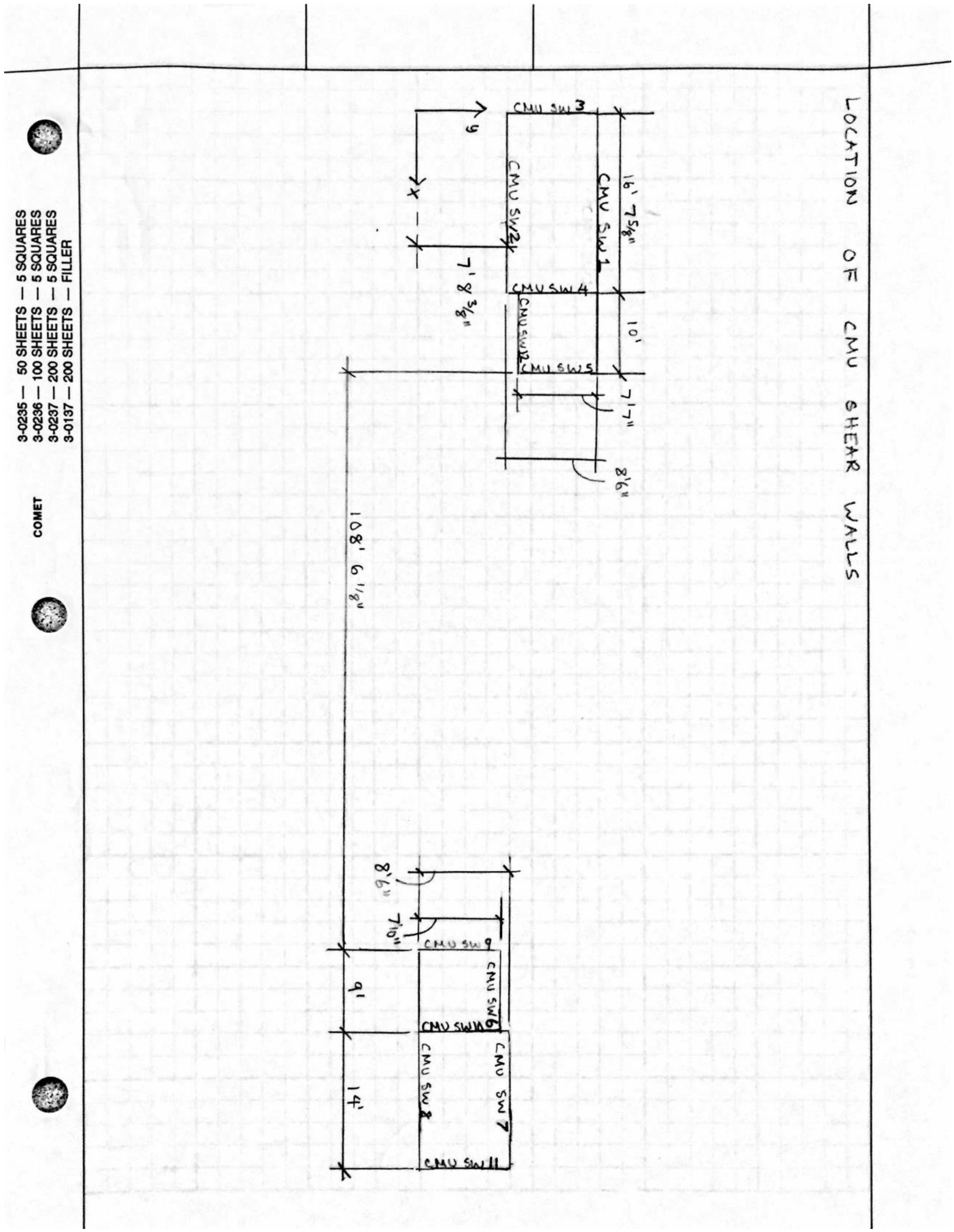
Floor	Wind Y Direction					
	Case 1	Case 2	Case 3	Case 4	h/400	Acceptable?
Roof	0.062	0.078	0.045	0.059	0.323	Yes
Fifth	0.056	0.072	0.040	0.054	0.290	Yes
Fourth	0.053	0.069	0.040	0.052	0.290	Yes
Third	0.043	0.060	0.034	0.045	0.290	Yes
Second	0.028	0.040	0.020	0.030	0.435	Yes

Floor	Seismic Y Direction		
	Actual	Code Min.*	Acceptable?
Roof	0.066	2.585	Yes
Fifth	0.049	2.320	Yes
Fourth	0.060	2.320	Yes
Third	0.050	2.320	Yes
Second	0.033	3.480	Yes

The wind loads calculated by ETABS were reasonably similar to the hand calculations with a difference no greater than 5% for both directions.

The hand calculated seismic loads differed greatly from the ETABS output. This could be due to errors in the hand calculations the punished the top levels for the weight of the concrete slab at the second floor.

As it caused the greatest story drift, wind in the Y direction for Case 2 will be considered for the following member spot checks.


LOCATION OF CMU SHEAR WALLS
COMET
3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
CMU SW 1
CMU SW 2
CMU SW 3
CMU SW 4
CMU SW 5
CMU SW 6
CMU SW 7
CMU SW 8
CMU SW 9
CMU SW 10
CMU SW 11
CMU SW 12
16' 7 5/8"
10'
7'-7"
8'6"
7' 8 3/8"
108' 6 1/8"
14'
16'
7'10"
X
Y

RELATIVE RIGIDITY OF CMU SHEAR WALL (ONE STORY)

$$R_F = 0.1\left(\frac{h}{d}\right)^3 + 0.3\left(\frac{h}{d}\right)$$

<u>CMU SW 3/CMU SW4/CMU SW 10/CMU SW11</u>

$$R_F = 1/\left[0.1\left(\frac{9.66}{8'6''}\right)^3 + 0.3\left(\frac{9.66}{8'6''}\right)\right]$$

$$R_F = 2.05$$

<u>CMU SW 5/CMU SW 9</u>

$$R_F = \frac{1}{0.1\left(\frac{9.66}{7'10''}\right)^3 + 0.3\left(\frac{9.66}{7'10''}\right)}$$

$$R_F = 1.79$$

<u>CMU SW 1</u>

$$R_F = \frac{1}{0.1\left(\frac{9.66}{26'7\,5/8''}\right)^3 + 0.3\left(\frac{9.66}{26'7\,5/8''}\right)}$$

$$R_F = 8.80$$

<u>CMU SW 2</u>

$$R_F = \frac{1}{0.1\left(\frac{9.66'}{16'7\,5/8''}\right)^3 + 0.3\left(\frac{9.66'}{16'7\,5/8''}\right)}$$

$$R_F = 5.16$$

<u>CMU SW 6</u>

$$R_F = \frac{1}{0.1\left(\frac{9.66'}{9'}\right)^3 + 0.3\left(\frac{9.66'}{9'}\right)}$$

$$R_F = 2.24$$

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER

COMET

CMU SW 7

$$R_F = \frac{1}{0.1\left(\frac{9.66}{14'}\right)^3 + 0.3\left(\frac{9.66}{14'}\right)}$$

$R_F = 4.17$

CMU SW 8

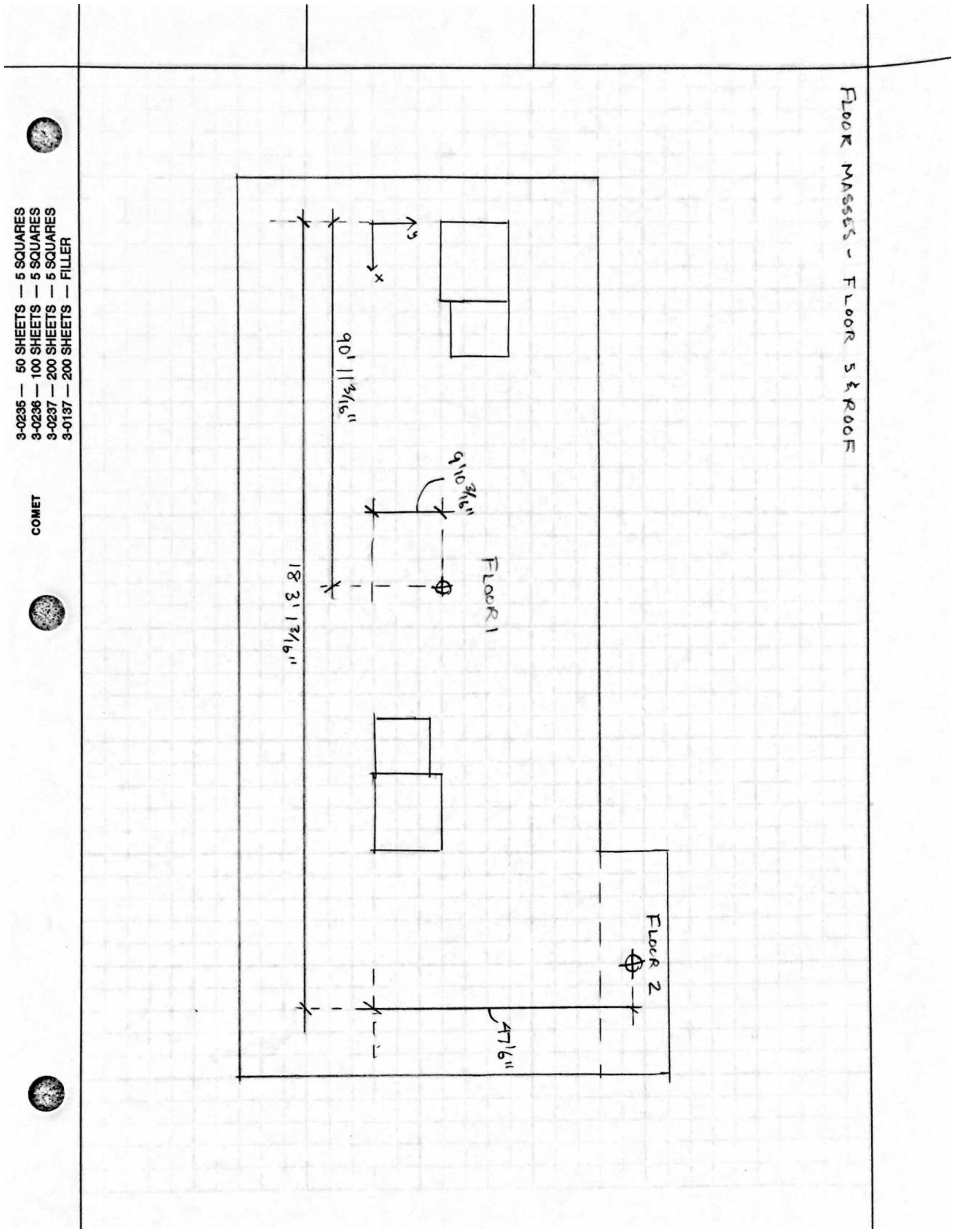
$$R_F = \frac{1}{0.1\left(\frac{9.66}{23'}\right)^3 + 0.3\left(\frac{9.66}{23'}\right)}$$

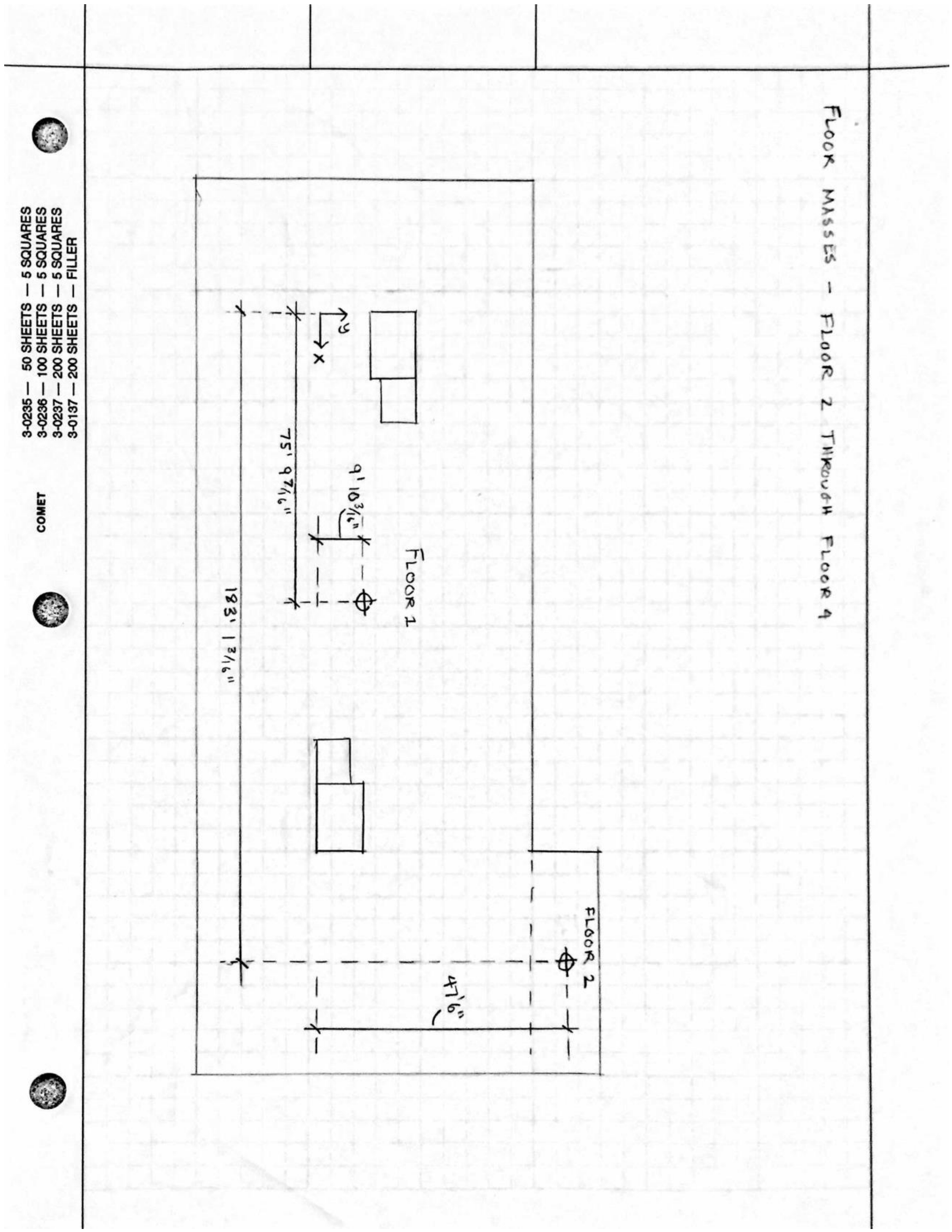
$R_F = 7.49$

CMU SW 12

$$R_F = \frac{1}{0.1\left(\frac{9.66}{10}\right)^3 + 0.3\left(\frac{9.66}{10}\right)}$$

$R_F = 2.61$


COMET
3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
FLOOR MASSES - FLOOR 5 & ROOF
FLOOR 1
FLOOR 2


FLOOR MASSES - FLOOR 2 THROUGH FLOOR 4
FLOOR 1
FLOOR 2
75' 9 7/16"
9' 10 3/16"
183' 1 3/16"
47'6"
y
x
COMET
3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER

Lateral Forces

Wind Y Direction
Case 1- Figure 6-9, ASCE 7-05

e (ft)	4.32	
Level	V (k)	Mt (k-ft)
Floor 3	136.40	589.02
Floor 4	102.20	441.33
Floor 5	65.00	280.69
Roof	23.30	100.62

Wind Y Direction – Case 2
Case 2- Figure 6-9, ASCE 7-05

e (ft)	38.55[1]	
Level	V (k)[2]	Mt (k-ft)
Floor 3	102.30	3943.15
Floor 4	76.65	2954.47
Floor 5	48.75	1879.07
Roof	17.48	673.57

(1)15% of B dimension
(2)75% of story force

Center of Rigidity

COR

Shear Wall	Direction	Distance from Axis		Rx	Ry	Rx*Y	Ry*X
		x (ft)	y (ft)				
1	x	-	16.20	8.80	-	142.54	-
2	x	-	7.70	5.16	-	39.72	-
3	y	0	-	-	2.05	-	0.00
4	y	16.64	-	-	2.05	-	34.10
5	y	26.64	-	-	1.79	-	47.68
6	x	-	7.83	2.24	-	17.55	-
7	x	-	8.50	4.17	-	35.45	-
8	x	-	0.00	7.49	-	0.00	-
9	y	135.15	-	-	1.79	-	241.91
10	y	144.15	-	-	2.05	-	295.50
11	y	158.15	-	-	2.05	-	324.20
12	x	-	8.61	2.61	-	22.48	-
			Σ=	30.47	11.78	257.74	943.39

		ETABS
Xcor (ft)	80.08	80.65
Ycor (ft)	8.46	7.03

Center of Mass

Floor 2 through Floor 4

COM

Element	Length of Wall (ft)	W (k)	Distance from Axis		W*X	W*Y
			x (ft)	y (ft)		
Floor 1		400.24	75.79	9.85	30332.92	3941.97
Floor 2		13.75	183.10	47.50	2517.24	653.03
Wall 1	26.64	21.61	13.32	16.20	287.84	350.09
Wall 2	16.64	13.50	8.32	7.70	112.28	103.91
Wall 3	8.50	6.90	0.00	11.95	0.00	82.41
Wall 4	8.50	6.90	16.64	11.95	114.74	82.41
Wall 5	7.58	6.15	26.64	12.41	163.90	76.34
Wall 6	9.00	7.30	139.65	7.83	1019.83	57.21
Wall 7	14.00	11.36	151.15	8.50	1717.04	96.56
Wall 8	23.00	18.66	146.65	0.00	2736.87	0.00
Wall 9	7.83	6.36	135.15	3.92	859.02	24.90
Wall 10	8.50	6.90	144.15	4.25	994.21	29.31
Wall 11	8.50	6.90	158.15	4.25	1090.77	29.31
Wall 12	10.00	8.11	21.64	8.61	175.56	69.90
	Σ=	534.64		**Σ=**	42122.22	5597.34

		ETABS
Xcom (ft)	78.79	76.58
Ycom (ft)	10.47	9.00

Floor 5 and Roof

COM

Element	Length of Wall (ft)	W (k)	Distance from Axis x (ft)	Distance from Axis y (ft)	W*X	W*Y
Floor 1		353.06	90.93	9.85	32104.68	3477.29
Floor 2		13.75	183.10	47.50	2517.24	653.03
Wall 1	26.64	21.61	13.32	16.20	287.84	350.09
Wall 2	16.64	13.50	8.32	7.70	112.28	103.91
Wall 3	8.50	6.90	0.00	11.95	0.00	82.41
Wall 4	8.50	6.90	16.64	11.95	114.74	82.41
Wall 5	7.58	6.15	26.64	12.41	163.90	76.34
Wall 6	9.00	7.30	139.65	7.83	1019.83	57.21
Wall 7	14.00	11.36	151.15	8.50	1717.04	96.56
Wall 8	23.00	18.66	146.65	0.00	2736.87	0.00
Wall 9	7.83	6.36	135.15	3.92	859.02	24.90
Wall 10	8.50	6.90	144.15	4.25	994.21	29.31
Wall 11	8.50	6.90	158.15	4.25	1090.77	29.31
Wall 12	10.00	8.11	21.64	8.61	175.56	69.90
	Σ=	487.46		**Σ=**	43893.97	5132.66

Xcom (ft)	90.05
Ycom (ft)	10.53

Direct Shear

Wall	Ry	Vd (k) Level: Floor 3	Floor 4	Floor 5	Roof
3	2.05	17.80	13.34	8.48	3.04
4	2.05	17.80	13.34	8.48	3.04
5	1.79	15.54	11.65	7.41	2.66
9	1.79	15.54	11.65	7.41	2.66
10	2.05	17.80	13.34	8.48	3.04
11	2.05	17.80	13.34	8.48	3.04
Σry=	11.78				

Torsional Rigidity

Shear Wall	dx	dy	Rx*dy^2	Ry*dx^2
1	-	7.74	527.07	-
2	-	0.76	2.99	-
3	80.08	-	-	13147.55
4	63.45	-	-	8252.72
5	53.45	-	-	5113.58
6	-	0.63	0.88	-
7	-	0.04	0.01	-
8	-	8.46	535.92	-
9	55.06	-	-	5426.94
10	64.06	-	-	8413.05
11	78.06	-	-	12492.00
12	-	0.16	0.06	-
			J=	53912.75

Torsional Shear – Case 1

Torsional Shear

Shear Wall	Ridi	Vt (k)				Direction
		Level				
		Floor 3	Floor 4	Floor 5	Roof	
1	68.10	0.74	0.56	0.35	0.13	-X
2	3.93	0.04	0.03	0.02	0.01	+X
3	164.17	1.79	1.34	0.85	0.31	-Y
4	130.07	1.42	1.06	0.68	0.24	-Y
5	95.67	1.05	0.78	0.50	0.18	-Y
6	1.40	0.02	0.01	0.01	0.00	+X
7	0.17	0.00	0.00	0.00	0.00	-X
8	63.36	0.69	0.52	0.33	0.12	+X
9	98.56	1.08	0.81	0.51	0.18	+Y
10	131.33	1.43	1.08	0.68	0.25	+Y
11	160.03	1.75	1.31	0.83	0.30	+Y
12	0.41	0.00	0.00	0.00	0.00	-X

Torsional Shear – Case 2

Torsional Shear

Shear Wall	Ridi	Vt (k)				Direction
		Level				
		Floor 3	Floor 4	Floor 5	Roof	
1	68.10	4.98	3.73	2.37	0.85	-X
2	3.93	0.29	0.22	0.14	0.05	+X
3	164.17	12.01	9.00	5.72	2.05	-Y
4	130.07	9.51	7.13	4.53	1.63	-Y
5	95.67	7.00	5.24	3.33	1.20	-Y
6	1.40	0.10	0.08	0.05	0.02	+X
7	0.17	0.01	0.01	0.01	0.00	-X
8	63.36	4.63	3.47	2.21	0.79	+X
9	98.56	7.21	5.40	3.44	1.23	+Y
10	131.33	9.61	7.20	4.58	1.64	+Y
11	160.03	11.70	8.77	5.58	2.00	+Y
12	0.41	0.03	0.02	0.01	0.01	-X

Total Shear

Case 1

Total Shear

Shear Wall	Vtotal (k) Level				Direction
	Floor 3	Floor 4	Floor 5	Roof	
1	0.74	0.56	0.35	0.13	-X
2	0.04	0.03	0.02	0.01	+X
3	21.94	16.44	10.46	3.75	+Y
4	22.32	16.72	10.63	3.81	+Y
5	19.68	14.75	9.38	3.36	+Y
6	0.02	0.01	0.01	0.00	+X
7	0.00	0.00	0.00	0.00	-X
8	0.69	0.52	0.33	0.12	+X
9	21.80	16.34	10.39	3.72	+Y
10	25.17	18.86	12.00	4.30	+Y
11	25.49	19.10	12.14	4.35	+Y
12	0.00	0.00	0.00	0.00	-X

Case 2

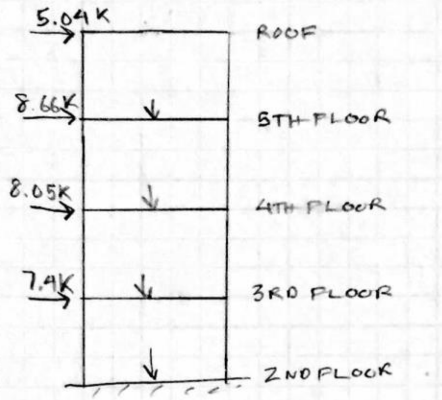
Total Shear

Shear Wall	Vtotal (k) Level				Direction
	Floor 3	Floor 4	Floor 5	Roof	
1	4.98	3.73	2.37	0.85	-X
2	0.29	0.22	0.14	0.05	+X
3	5.80	4.34	2.76	0.99	+Y
4	8.29	6.21	3.95	1.42	+Y
5	8.55	6.40	4.07	1.46	+Y
6	0.10	0.08	0.05	0.02	+X
7	0.01	0.01	0.01	0.00	-X
8	4.63	3.47	2.21	0.79	+X
9	22.75	17.05	10.84	3.89	+Y
10	27.41	20.54	13.06	4.68	+Y
11	29.51	22.11	14.06	5.04	+Y
12	0.03	0.02	0.01	0.01	-X

Masonry Shear Wall Spot Check

LOAD ON SHEAR WALL 11 @ 2ND FLOOR

★ CHOSE CASE 2 AS WORST CASE



8" FULLY GROUTED CMU

⇒ 84 psf

★ ONLY CONSIDERING WIND AND SELF WEIGHT AS JOISTS DUE NOT FRAME INTO SW 11

SELF WEIGHT

$W_{S_{5TH}} = 84\,psf \cdot (8'6") \cdot (77'11\tfrac{1}{4}" - 66'10") = 7.93K$

$W_{S_{4TH}} = 84\,psf \cdot (8'6") \cdot (66'10" - 57'2") = 6.91K$

$W_{S_{3RD}} = 84\,psf \cdot (8'6") \cdot (57'2" - 47'6") = 6.91K$

$W_{S_{2ND}} = 84\,psf \cdot (8'6") \cdot (47'6" - 37'10") = 6.91K$

$W_{TOTAL} = 28.66K$

WIND LOAD (FROM CASE 2)

SHEAR AT 2ND FLOOR = 29.51K

LOAD COMBONATION: D+W

P

P = 28.66K

V

V = 29.51K

M

M = 29.51K · 9'8" = 3,423.16 K-in

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

SHEAR CHECK

$A_{nv} = (8.5' \cdot 12 - 3.91'') \cdot 7.625'' = 748 in^2$

$f_v = \frac{V}{A_n} = \frac{29.51K}{748 in^2} = 39.45 psi$

MAXIMUM ALLOWABLE SHEAR STRESS

$\frac{M}{Vd} = \frac{3,423.16}{29.51 \cdot 98} = 1.18 > 1.0$

$\Rightarrow F_v = (2\sqrt{f'm})\gamma_g$

$F_v = 2 \cdot \sqrt{1500} = 77.5 psi$

$F_{vm} = \frac{1}{2}\left[\left(4 - 1.75\left(\frac{M}{Vd}\right)\right)\sqrt{f'm}\right] + \frac{1}{4}\frac{P}{A_N}$

$F_{vm} = \frac{1}{2}\left[\left(4 - 1.75(1.18)\right) \cdot \sqrt{1500}\right] + \frac{1}{4} \cdot \frac{28.66 \cdot 1000}{7.625 \cdot 102}$

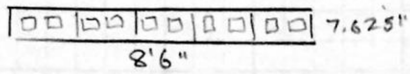
$F_{vm} = 46.55 psi$

$F_v > \Big|_{min} \begin{matrix} 77.5 \\ 46.55 \end{matrix} = 46.55 psi$

$f_v = 39.45 psi < F_v = 46.55 psi$

∴ GOOD

FLEXURAL CHECK



$A_n = (8'6'') \cdot (7.625'') = 777.75 \text{ in}^2$

$r_1 = \frac{d}{\sqrt{12}} = \frac{102''}{\sqrt{12}} = 29.4 \text{ in}$

$r_2 = \frac{7.625}{\sqrt{12}} = 2.20 \text{ in}$

SLENDERNESS

$h/r_1 = 9.66' \cdot 12 / 29.4 = 3.9 < 99$ ✓

$h/r_2 = 9.66' \cdot 12 / 2.20 = 52.7 < 99$ ✓

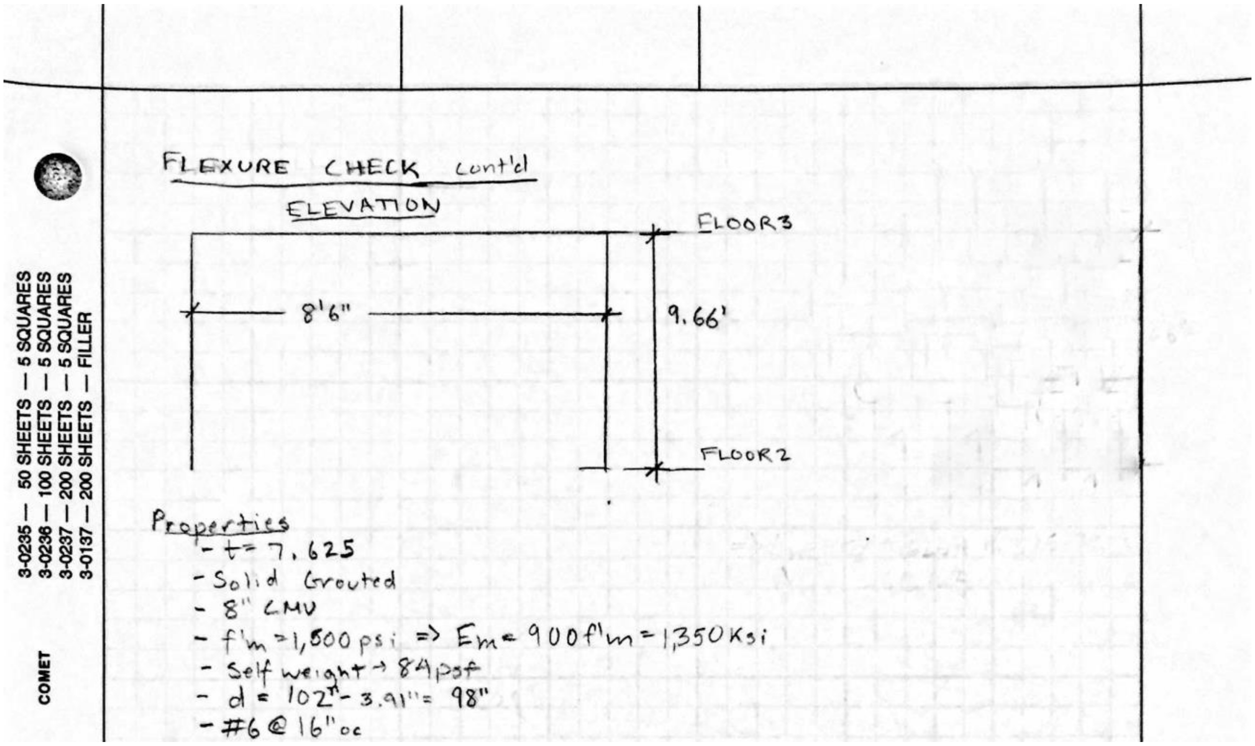
$R_2 = \left[1 - \left(\frac{52.7}{140}\right)^2\right] = 0.86$

$P_a = \frac{1}{4} f'_m A_n [R]$

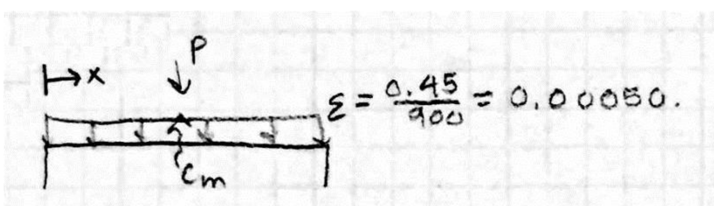
$P_a = \frac{1}{4} \cdot 1{,}500 \text{psi} \cdot 777.75 \text{ in}^2 \cdot 0.86$

$P_a = 250 \text{K}$

Interaction Diagram



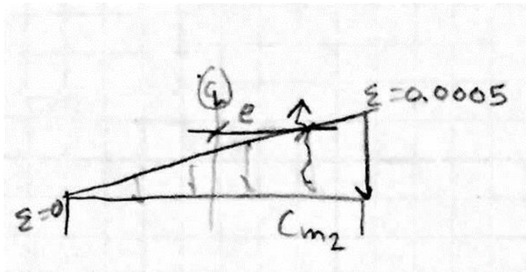
Point 1



Layer	Area (in2)	ε	F (k)	x (in)	e (in)	M (k-in)
CMU	777.75	0.00050	524.98	51	0.00	0.00

P1 (k) 524.98 **M1(k-in)** 0.00

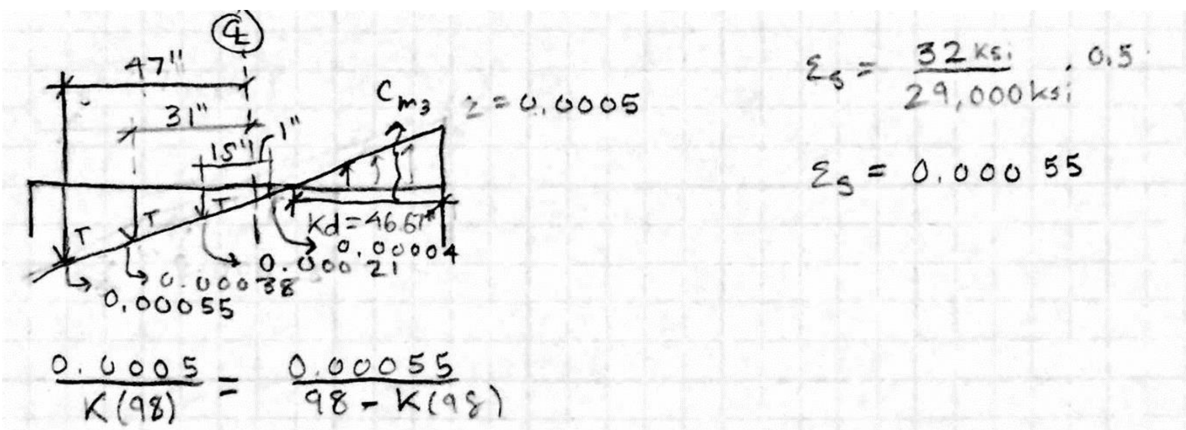
Point 2



Layer	Area (in2)	ε	F (k)	x (in)	e (in)	M (k-in)
CMU	388.88	0.00050	262.49	68	17.00	4462.34

P2 (k) 262.49 **M2(k-in)** 4462.34

Point 3

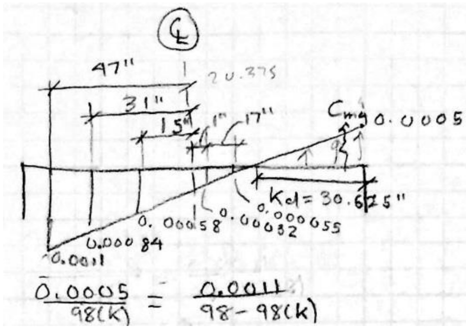


k= 0.48
kd= 46.67

Layer	Area (in2)	ε	F (k)	x (in)	e (in)	M (k-in)
CMU	177.92	0.00050	120.09	82.44	31.44	3776.28
Steel 1	0.44	0.00055	-7.02	4.00	-47.00	329.85
Steel 2	0.44	0.000379	-4.83	20.00	-31.00	149.75
Steel 3	0.44	0.000207	-2.64	36.00	-15.00	39.65
Steel 4	0.44	3.57E-05	-0.46	52.00	1.00	-0.46

P3 (k) 105.15 **M3(k-in)** 4295.07

Point 4

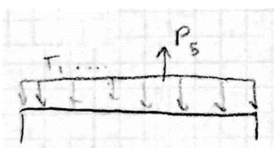


k=	0.31
kd=	30.63

Layer	Area (in2)	ε	F (k)	x (in)	e (in)	M (k-in)
CMU	116.76	0.00050	78.81	87.79	36.79	2899.61
Steel 1	0.44	0.0011	-14.04	4.00	-47.00	659.69
Steel 2	0.44	0.000839	-10.70	20.00	-31.00	331.79
Steel 3	0.44	0.000578	-7.37	36.00	-15.00	110.54
Steel 4	0.44	0.000316	-4.04	52.00	1.00	-4.04
Steel 5	0.44	5.51E-05	-0.70	68.00	17.00	-11.95

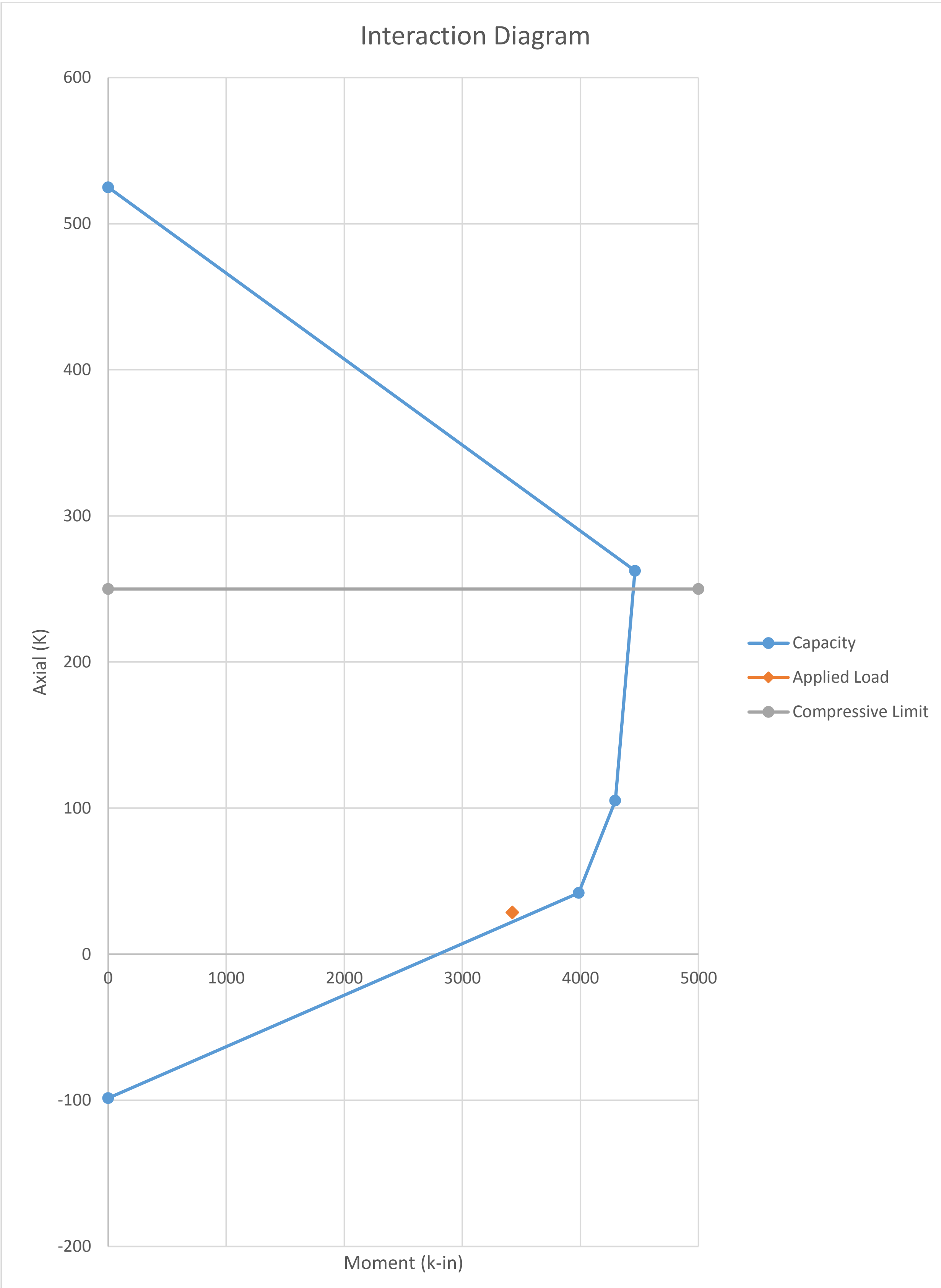
P4 (k) 41.96 **M4(k-in)** 3985.64

Point 5



Layer	Area (in2)	F (k)
Steel 1	0.44	-14.08
Steel 2	0.44	-14.08
Steel 3	0.44	-14.08
Steel 4	0.44	-14.08
Steel 5	0.44	-14.08
Steel 6	0.44	-14.08
Steel 7	0.44	-14.08

P5 (k) -98.56 **M5(k-in)** 0.00

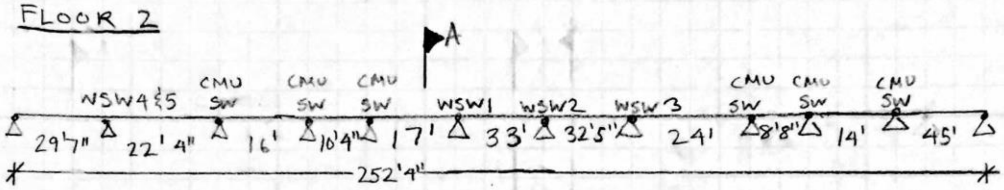

Interaction Diagram
600
500
400
300
200
100
0
-100
-200
0
1000
2000
3000
4000
5000
Axial (K)
Moment (k-in)
Capacity
Applied Load
Compressive Limit

Wood Shear Wall Spot Check

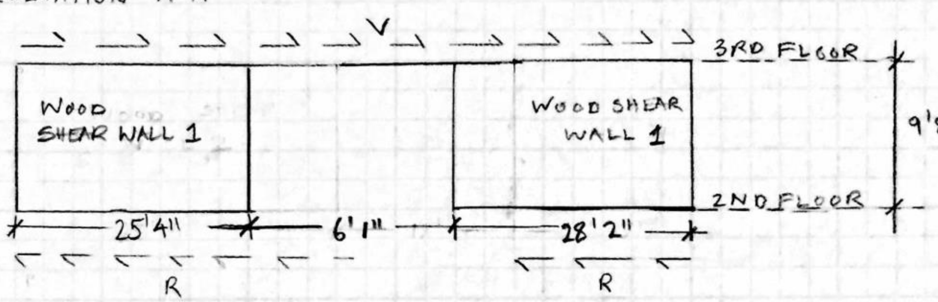
WOOD SHEAR WALL

CONSIDERING DIAPHRAGM AS FLEXIBLE

FLOOR 2



ELEVATION A-A



IF F = STORY SHEAR FORCE (lbs)

$$V = \frac{(17' + 33')}{252'4''} F$$

$$V = 0.20F \text{ (lbs)}$$

$$R = \frac{0.20F}{(25'4'' + 28'2'')}$$

$$R = 0.00370F \text{ (lbs/ft)}$$

FROM WIND LOAD CALCULATIONS FOR DIRECTION (1)
@ 3RD LEVEL

$$F = 23.3K + 41.7K + 37.2K + 34.2K = 136.4K$$

$$V = 136.4K(0.20) = 27.3K$$

$$R = 0.00370(136.4K) = 505 \text{ lb/ft}$$

$$V_{W_{ACTUAL}} = \frac{505 \text{ (ADJ. FOR ASD)}}{\text{(SPECIFIC GRAVITY ADJ)}}$$

$$= \frac{505(2)}{1-(0.5-0.43)}$$

$$V_{W_{ACTUAL}} = 1{,}086 \text{ lb/ft}$$

SHEAR WALL 1 SPECIFICATIONS

- STRUCTURAL I PANELING
- 7/16" OSB SHEATHING (ONE SIDE)
- 8d NAILS
- BLOCKED ON ALL EDGES
- 4" EDGE FASTNER SPACING

FROM TABLE 4.3A, NDS 2015, SDPWS

$$V_W = 1105 \text{ lb/ft}$$

$$V_W = 1{,}105 \text{ lb/ft} > V_{W_{ACTUAL}} = 1{,}086 \text{ lb/ft}$$

$\therefore$ GOOD