Point Pleasant Apartments
Point Pleasant, NJ

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Structural Option
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Technical Report #1

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Executive Summary

The purpose of this report is to analyze the structural system in the original design of the Point Pleasant Apartment complex located in Ocean County, NJ. The site contains five separate buildings but this report will just focus on building one. Building one consists of four stories of approximately 2,500 square foot residential units over a one level parking garage. The building sits right along the coast of the Manasquan River and each apartment features two balconies, one of which faces the river.

The basic structure of Point Pleasant Apartments is composite steel joists with steel stud walls and roof trusses. This report will describe in detail the components of the structural system, the loading and materials used and the codes that governed the design.

Both a seismic and wind analysis were carried out for comparison to the original design. First was seismic since the period needed to be calculated before starting lateral analysis to ensure a rigid structure. This report used the simplified seismic analysis found in ASCE 7-05 to calculate the base shear, followed by a floor to floor distribution of forces. .

The wind resistance calculation that was used in this report is also found in ASCE 7-05, Section 6. The main wind-force resisting system was analyzed using Method 2 and then compared to the result of the seismic calculations to determine which was in control. Along with the wind analysis is a calculation for the unit shear and moment of a typical shearwall used throughout the building.

Finally, both a column and beam were spot checked to verify which code or design approach was originally used. One possible reason for variation of results between the original design and this report is the different publications of code used. Point Pleasant Apartments began design before the new IBC and ASCE, so in order to be as accurate as possible, this report used the most current editions of the code books. Also, many simplifications and assumptions were made that could have thrown the final numbers off a touch.

At the back of the report is an appendix containing all of the calculations for the seismic and wind analyses, the beam and column spot checks, as well as a full exterior building section and typical floor framing plan.

After completing both a seismic and wind analysis, I have found that wind controls the lateral design with a base shear of about 310 kips and an overturning moment of about 10,540 ft-k. Point Pleasant is located in a hurricane prone region and has a design wind speed of 120 MPH so this result is not surprising. In the spot check of the wide flange beam, I found the beam to fail in deflection but the column was more than adequate.

Structural System

Foundation

For Point Pleasant Apartments, a traditional shallow foundation with spread footings was used. The building was designed based on a 3,000 PSF soil bearing capacity. The exterior foundation walls are 12" thick concrete over either a 2'-6"x12" thick footing with #5 @ 24" o.c. S.W.B. and (3) #4 L.W.B. or a 3'-0"x12" thick footing with #5 @ 16" o.c. S.W.B. and (3) # 5 L.W.B. There is a 5" concrete slab on grade with 6.0x6.0 – W2.0x2.0 welded wire fabric over 4" of crushed stone and a 6 Mil vapor barrier. The main columns at this level are 16"x24", 18"x26", or 24"x24" reinforced concrete columns. Beneath these columns are 11'-0"x11'-0"x26" deep concrete spread footings which are reinforced with (12) #7 bars each way.

Floor System

The framing for floors 2, 3, and 4 is all pretty much the same. These stories are supported by 16" deep Vescom composite joists with a 3 1/2"reinforced concrete slab. The slab is supported by a 1 5/16", 22 gage UFX 36 metal form deck. The joists are spaced and 48" o.c. and are designed to carry a total load of about 380 plf. The typical span for these joists is approximately 20', with a maximum span of about 24'. Spans run front to back. This composite system is supported by a series of steel girder trusses, wide flange beams, and HSS columns.

Each of the apartments throughout the building features front and rear balconies. The balconies are supported by a shallower composite joist of 12". HSS shapes are used as both edge beams and columns for the balconies.

The first floor is framed very differently from the floors above. Instead of a composite joist system, the first floor is a 12" thick, reinforced two-way slab. In addition to the 12" thick slab, there are slab beams in the outer apartments for additional support. Above the concrete columns below, are 12'-0"x12'-0"x20" deep (20"-12"=8" below slab depth) drop panels.

Roof Sytem

The roof system is a simple hip with two large dormers in the rear and two smaller dormers, a tower, and a dome feature in the front. The roof is made up of light gage metal roof trusses spaced at 48" o.c.

Lateral Framing

The walls of the building are comprised of metal studs, therefore, light gage shearpanels and special reinforced shearwalls are utilized to resist lateral load. The shearwalls typically consist of 4"x14 or 16 gage flat strap bracing with 3 1/2"x3 1/2"x1/2" or 6"x3 1/2"x1/2" HSS shapes. The flat straps can either be screwed or welded to the HSS's.

Codes

Because the Point Pleasant apartment complex was designed a few years ago, the most recent code books had not yet been published. In order to make my project a more practical and beneficial learning experience, I will be using the most up to date design codes available.

Design Codes used in original design:

- International Building Code (IBC), 2000 Edition
- American Society of Civil Engineers (ASCE-7), 2002 Edition
- American Concrete Institute (ACI 318), 2000 Edition
- American Institute of Steel Construction ASD (AISC), 9th Edition

Design Codes used in my analysis:

- International Building Code (IBC), 2006 Edition
- American Society of Civil Engineers (ASCE-7), 2005 Edition
- American Concrete Institute (ACI 318), 2005 Edition
- American Institute of Steel Construction (AISC), 13th Edition

Material Strength Properties

Concrete (minimum compressive strengths in 28 days)

Precast concrete lintels……………………….4000 psi
Footings……………………………………4000 psi
Foundation wall.…………………...............4000 psi
Slab on grade….…………………...............4000 psi
Cast in place slabs, beams, and columns…..5000 psi
Concrete fills on metal deck……………….4000 psi
All exposed concrete ………………………4000 psi (with entrained air)

Structural Steel

Wide flange beams: ASTM – A992, 50 ksi
Angles, channels, and plates: ASTM – A36, 36 ksi
Pipe shapes: ASTM – A53, 35 ksi or ASTM – A501, 36 ksi
HSS shapes: ASTM – A500, Grade B, 46 ksi
Anchor bolts: ASTM – F1554, Grade 36

Masonry

CMU: ASTM – C90, minimum net compressive strength of 1900 psi
Mortar Type: Portland Cement/Lime, Type S

Design Loads

Dead Loads

Composite Floor System…………………….. 65 psf (including non-bearing partitions)
5” Concrete Slab…………………………….. 63 psf
4” Concrete Slab…………………………….. 50 psf
Roof Trusses…………………………………. 10 psf (top and bottom chord)

Superimposed Dead Loads

Mechanical, Electrical, Plumbing…………….10 psf
Ceiling Finishes……………………………… 5 psf
Floor Finishes……………………………….. 10 psf
Interior Walls………………………………… 15 psf

Live Loads

Residential (private rooms and corridors)…... 40 psf
Residential Balconies…………………………60 psf
First Floor Corridors and Lobbies.………….. 100 psf
Roof (Snow)…………………………………. 21 psf

Seismic Analysis

At this point in my analysis, I used the simplified method described in ASCE 7-05 for my seismic calculations for base shear (V). According to the Geo-Tech report, the plot of land where the apartments are being constructed is in Site Class D. In order to find the latitude and longitude for the site, I went to http://earthquake.usgs.gov/research/hazmaps/design/ which provided an Ss value of 0.239 and a S1 value of 0.056. The seismic design category for the building is Category B, the importance factor is 1.0 and the R value is 4.0 for light framed wall systems using flat strap bracing as it's means of lateral resistance.

Seismic Floor to Floor Force Distribution

Level	Height (ft.)	Weight (k)	Exp. K	sum $w_i h_i^k$	C_{vx}	f_x	V_x (k)	M_x (ft-k)
Roof Peak	72.5	128	1.1614	10777.8	0.04831	9.758	9.758	707.446
Attic	53.5	195	1.1614	12116.3	0.05431	10.970	20.728	586.880
4	43.5	1460	1.1614	73760.5	0.3306	66.780	87.508	2904.948
3	32.67	1460	1.1614	55396.7	0.24829	50.154	137.662	1638.544
2	21.83	1460	1.1614	37015.9	0.16591	33.513	171.175	731.589
1	11	2665	1.1614	34046.4	0.1526	30.825	202.000	339.070
Total:		**7368**		**223114**		**V=202 ^{k}**		**M=6910 'k**

The chart above shows a total calculated base shear of 202 kips. The actual design value was 224 kips which is approximately 110% of the value I calculated. Because this is a simplified method and many assumptions were made, this is a reasonable and fairly accurate result. I think one area that could have made the difference was the superimposed dead load.

Wind Analysis

Point Pleasant is located right along the coast of New Jersey; therefore, the design wind speed is 120 MPH and the wind exposure category is C. This wind speed is increased from 115 MPH, which was used in the older code and used in the original design. Below are tables showing the wind pressures for the building using Method 2 for wind analysis found in ASCE-7-05. The building is considered rigid because the period is less than 1, as was mentioned above in the Seismic Analysis section. The longer dimension of the building runs in the North-South direction and the shorter East-West. The calculations for the wind pressures and resultant forces can be found in the Appendix.

Wind Pressures, PSF (from E-W dir.)

z(ft)	K_z	q_z	Windward P	Leeward P	Total
0-11	0.850	26.634	18.197	-15.788	33.985
21.83	0.915	28.671	19.588	-15.788	35.376
32.67	0.996	31.209	21.322	-15.788	37.110
43.50	1.058	33.152	22.649	-15.788	38.438
53.50	1.104	34.593	23.634	-15.788	39.422
72.50	1.180	36.975	3.158	-9.473	12.631

Wind Pressures, PSF (from N-S dir.)

z(ft)	K_z	q_z	Windward P	Leeward P	Total
0-11	0.850	26.634	18.537	-12.031	30.568
21.83	0.915	28.671	19.955	-12.031	31.986
32.67	0.996	31.209	21.722	-12.031	33.752
43.50	1.058	33.152	23.074	-12.031	35.105
53.50	1.104	34.593	24.077	-12.031	36.108
72.50	1.180	36.975	3.217	-9.595	12.812

Total Base Shear:
308.108 kips (E-W Direction, Perpendicular to Long Dimension)
174.168 kips (N-S Direction, Perpendicular to Short Dimension)
Overturning Moment:
10,542 ft-k (E-W Direction, Perpendicular to Long Dimension)
6058 ft-k (N-S Direction, Perpendicular to Short Dimension)

After completing both seismic and wind analysis, the report shows that wind is the controlling factor for base shear and overturning moment. This is not surprising since the building is in a low risk seismic design category and is in a hurricane prone region. The maximum base shear was found to be about 310 kips and the maximum moment was 10,540 ft-k which is each approximately 150% of the seismic design base shear and overturning moment.

Beam and Column Spot Check

The beam I chose to spot check was a wide flange girder sized as W12x58, which is typical for floors 2, 3 and 4. The beam was originally designed using the 9th Edition AISC Steel Manual, ASD. For my analysis, I used the 13th Edition AISC Steel Manual, LRFD. I found the beam to be fine in both shear and moment. The moment was 180 ft-k, which is approximately 55% of the beam's capacity. However the beam failed in total load deflection (L/220). This could be a result of using the new manual, LRFD instead of ASD, or a difference in total load calculation. The next size W12 (W12x65) works with a deflection of L/247.

For the column, I analyzed a 16x24 concrete column reinforced with (18) #6 bars and #3 ties @ 12"o.c. The column supports the first floor concrete slab as well as the load carried down by the HSS columns on each of the three floors above. I found a total load of 400 kips, and, assuming pure axial, the capacity far exceeded this with a factored strength of 1300 kips. I used live load reduction on all of the floors above and also may have differed from the original design in dead load calculation. Both of these factors could have contributed to the large difference between the load and the capacity.

APPENDIX

Seismic Analysis Calculations

RYAN FLYNN | TECH 1 | SEISMIC

BASE SHEAR CALCULATION

$V = C_s W$

BUILDING WEIGHT

3 FLOORS COMPOSITE JOIST @ 65 PSF

$3 \times 12,800\ ft^2 \times 65\ PSF = \underline{2500^k}$

1ST FLOOR REINF. CONC. SLAB (12")

$12''\left(\frac{1'}{12''}\right)(150\ PCF) \times 12,800 = \underline{1920^k}$

8, 20" DROP PANELS (20" − SLAB DEPTH) = 8"

$8 \times 8''\left(\frac{1'}{12''}\right)(12 \times 12) \times 150\ PCF = \underline{115.2^k}$

EXT WALLS – 6" STEEL STUD, 5/8" GYPSUM, INSULATION, HARDSHIGLE } 15 PSF

ABOUT ½ SHINGLE ½ STONE OR STUCCO

STONE/STUCCO – 30 PSF

LENGTH, 2 WALLS, HEIGHT, AMOUNT OF FACADE

$152'\text{-}4'' \times 2 \times 50' \times 30 \times 0.5 = \underline{230^k}$ STONE

$\frac{15}{30}(230) = \underline{115^k}$ SHINGLE

$93' \times 50' \times 2 \times 30 \times 0.5 = \underline{140^k}$ STONE

$\frac{15}{30}(140) = \underline{70^k}$

HSS COLUMNS

FLOORS 1–3, $32 \times 14\ \text{lb}/\text{ft} \times 10' = \underline{4.5^k}$

FLOOR 4, $8 \times 14\ \text{lb}/\text{ft} \times 10' = \underline{1.2^k}$

SUPERIMPOSED DEAD LOAD

10 PSF MEP
10 PSF FLOOR FIN.
5 PSF CEILING
15 PSF INT. WALLS

$40\ PSF \times 4 \times 12,800 = \underline{2048^k}$

RYAN FLYNN | TECH 1 | SEISMIC CONT.

ROOF 10 PSF TOP CHORD
10 PSF BOTTOM CHORD

20 PSF × 12,800 = 256K

$$C_s = \text{Min}\begin{Bmatrix} S_{DS}/(R/I) \\ S_{D1}/(T \times R/I) \\ S_{D1} \times T_L/(T^2 \times R/I) \end{Bmatrix} \geq 0.01$$

LAT. & LONG. → 40° 4' 59", −74° 4' 7"

FROM WEBSITE: http://earthquake.usgs.gov/research/hazmaps/design/

$S_s = 0.239$
$S_1 = 0.056$

SITE CLASS D, $F_a = 1.6$ $F_v = 2.4$

$S_{ms} = F_a \cdot S_s = 1.6(0.239) = 0.3824$
$S_{m1} = F_v \cdot S_1 = 2.4(0.056) = 0.1344$

$S_{DS} = \frac{2}{3} S_{ms} = \frac{2}{3}(0.3824) = 0.255$ ∴ SEISMIC DC: B
$S_{D1} = \frac{2}{3} S_{m1} = \frac{2}{3}(0.1344) = 0.0896$

$T = C_u \cdot T_a$ [1.7]

$T_a = C_t \cdot h_n^x \rightarrow C_t = 0.02,\ x = 0.75$ PER TABLE 12.8-2
$T_a = 0.02 \cdot 70^{(0.75)}$
$T_a = 0.484$
$T = 1.7(0.484) = \boxed{0.8228}$ ∴ RIGID

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$I = 1.0$, $R = 4$ (LIGHT FRAMED WALL SYSTEMS USING FLAT STRAPPED BRACING)

OPTION 1

$$S_{DS}/(R/I) = 0.225/4 = \underline{0.05625}$$

OPTION 2

$$S_{D1}/(T \times R/I) = 0.0896/(0.8228 \times 4) = \underline{0.0272}\ \checkmark$$

OPTION 3

PER FIG 22-15

$$S_{D1} \times T_L/(T^2 \times R/I) = 0.0896 \times 6/(0.8228^2 \times 4) = \underline{0.1985}$$

$C_S = 0.0272$ $\quad \Sigma W = 7400^K$

$V = 0.0272\,(7400)$

$V = 202^K$

THIS BASE SHEAR VALUE OF 202^K IS SLIGHTLY OVER 90% OF THE DESIGN BASE SHEAR OF 224^K. THIS IS REASONABLE BECAUSE ALOT OF ASSUMPTIONS AND SIMPLIFICATIONS WERE MADE IN THIS ANALYSIS, PARTICULARLY IN CALCULATING THE SUPER-IMPOSED LOADS.

Wind Analysis Calculations

RYAN FLYNN | TECH 1 | WIND

WIND SPEED, $V = 120$ MPH

$I = 1.0$

EXPOSURE C

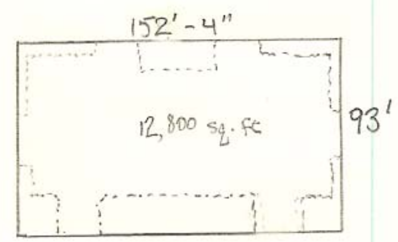
$K_d = 0.85$ FROM TABLE 6-4

$K_{zt} = (1 + K_1 K_2 K_3)^2 = 1.0$

$G = 0.925\left(\dfrac{1 + 1.7 g_Q I_z Q}{1 + 1.7 g_v I_z}\right)$

N

SIMPLIFIED RECTANGLE



---- ACTUAL FOOTPRINT

LONG DIR. (E-W WIND)

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1 + 0.63\left(\frac{152'\text{-}4'' + 63'}{513.77}\right)^{0.63}}} = \underline{0.856}$$

$\perp$ LONG

$B = 152' - 4''$

$h = \dfrac{53.5 + 72.5}{2} = 63'$

$L_{\bar{z}} = \ell\left(\dfrac{\bar{z}}{33}\right)^{\bar{\epsilon}}$, $\bar{z} = 0.6(63) = 37.8'$, $\bar{\epsilon} = 1/5.0$

$L_{\bar{z}} = 500\left(\dfrac{37.8}{33}\right)^{1/5.0}$

$L_{\bar{z}} = 513.77$

SHORT DIR. (N-S WIND)

$B = 93'$

$h = 63'$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{93' + 63'}{513.77}\right)^{0.63}}} = \underline{0.8880}$$

$\perp$ SHORT

RYAN FLYNN | TECH 1 | WIND CONT.

$$I_z = c\left(\frac{33}{z}\right)^{1/6} = 0.2\left(\frac{33}{37.8}\right)^{1/6} = \underline{0.1955}$$

LONG

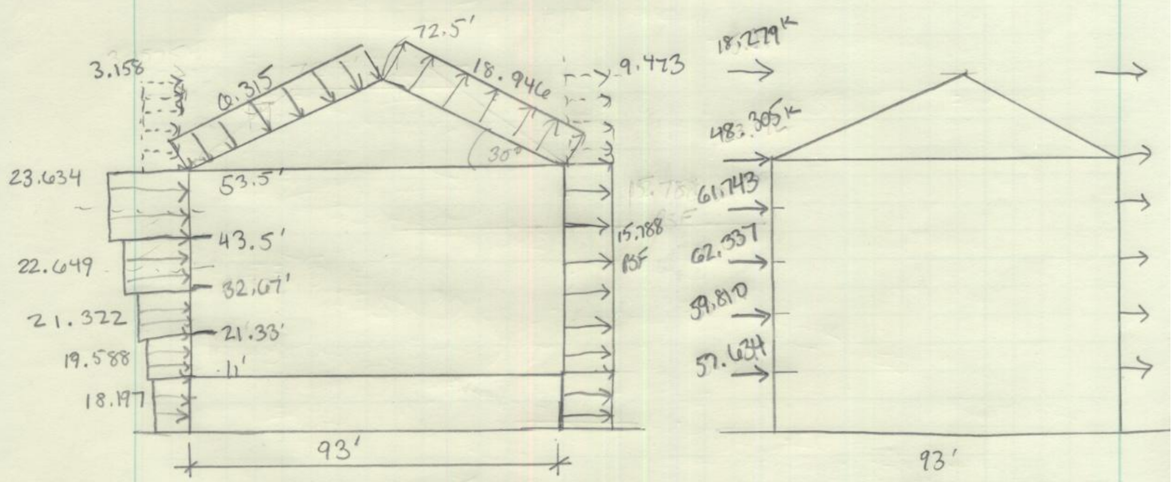
$$G = 0.925\left(\frac{(1+1.7(3.4)(0.1955)(0.856))}{1+1.7(3.4)(0.1955)}\right) = \underline{0.854}$$

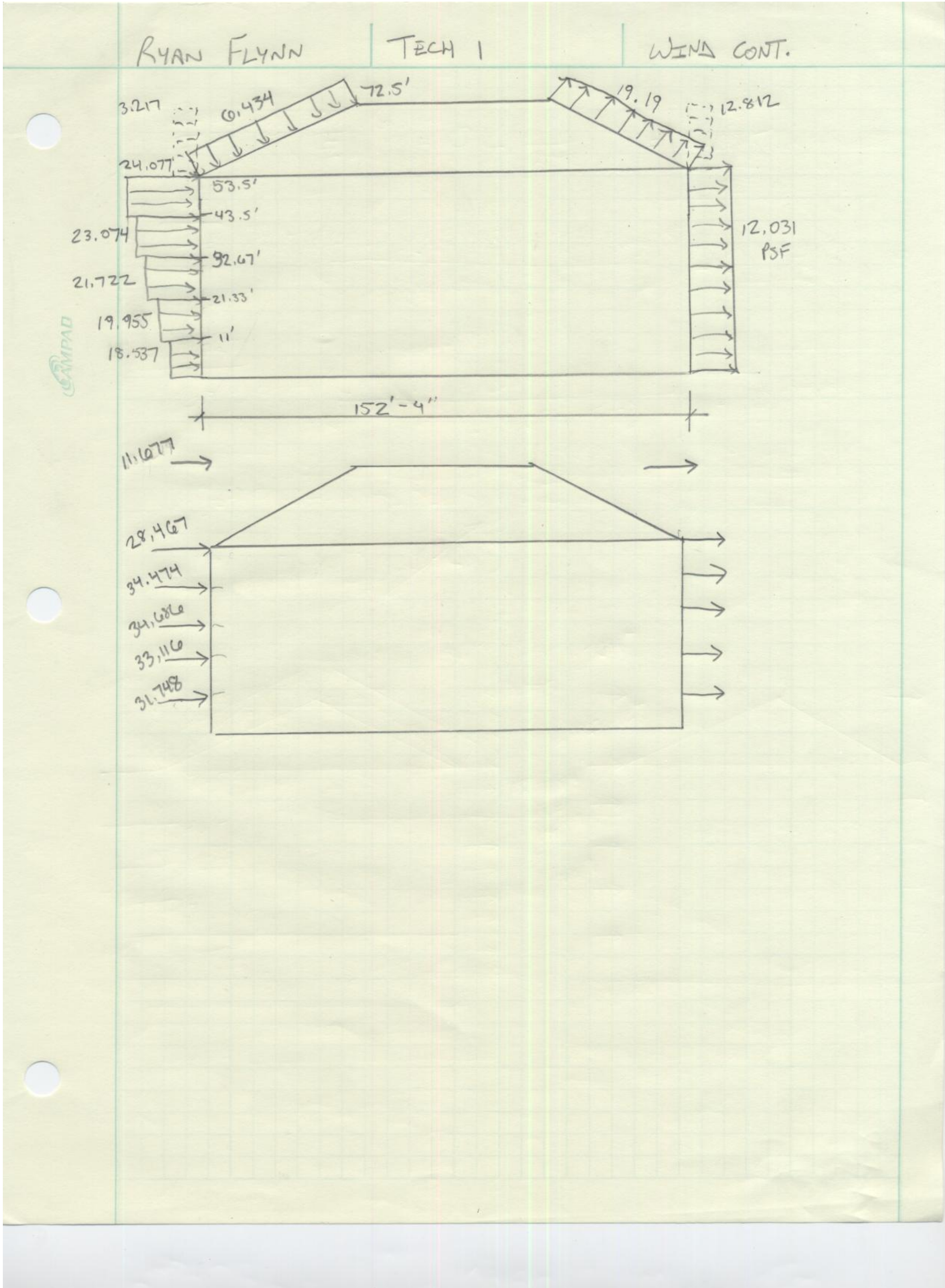
SHORT

$$G = 0.925\left(\frac{(1+1.7(3.4)(0.1955)(0.8880))}{1+1.7(3.4)(0.1955)}\right) = 0.870$$

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

$$\boxed{P = q\, G C_p} - q_i(\cancel{G C_{pi}}) \quad 0$$




RYAN FLYNN
TECH 1
WIND CONT.
3.217
6.434
72.5'
19.19
12.812
24.077
53.5'
43.5'
23.074
32.67'
21.722
21.33'
19.955
11'
18.537
12.031
PSF
152'-4"
11.677
28.467
34.474
34.686
33.116
31.748

Wind from E-W

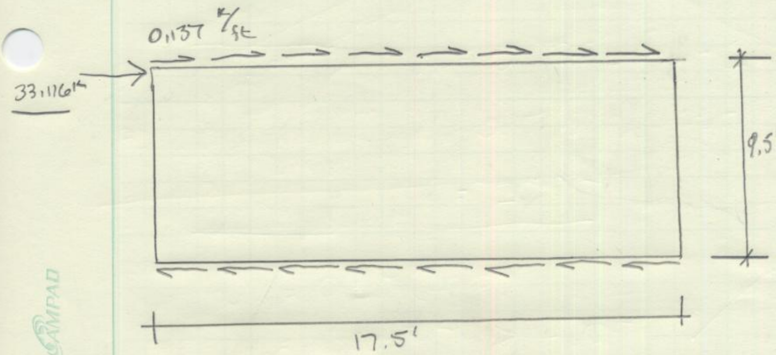
Level	Height (ft.)	Total PSF	Story Force (k)	**Total Shear (k)**	**Moment (ft-k)**
Parking	0	0.000	0.000	**308.108**	**10541.637**
1	11	33.985	57.634	308.108	633.974
2	21.33	35.376	59.810	250.474	1275.747
3	32.67	37.110	62.337	190.664	2036.550
4	43.5	38.438	61.743	128.327	2685.821
Attic	53.5	39.422	48.305	66.584	2584.318
Roof	72.5	12.631	18.279	18.279	1325.228

Wind from N-S

Level	Height (ft.)	Total PSF	Story Force (k)	**Total Shear (k)**	**Moment (ft-k)**
Parking	0	0.000	0.000	**174.168**	**6057.970**
1	11	30.568	31.748	174.168	349.228
2	21.33	31.986	33.116	142.420	706.364
3	32.67	33.752	34.686	109.304	1133.192
4	43.5	35.105	34.474	74.618	1499.619
Attic	53.5	36.108	28.467	40.144	1522.985
Roof	72.5	12.812	11.677	11.677	846.583

Shearwall Calculation

RYAN FLYNN | TECH 1 | SHEARWALL ANALYSIS



SHEARWALL @ FRONT LEFT CORNER ON GIVEN PLAN.
TAKEN @ 2^{nd} FLR.

STORY FORCE IS 33.116^k

14 SIMILAR SHEARWALL SEGMENTS

$33.116^k / 14 = 2.4^k$

UNIT SHEAR = $2.4^k / 17.5' = \boxed{0.137 \ k/ft}$

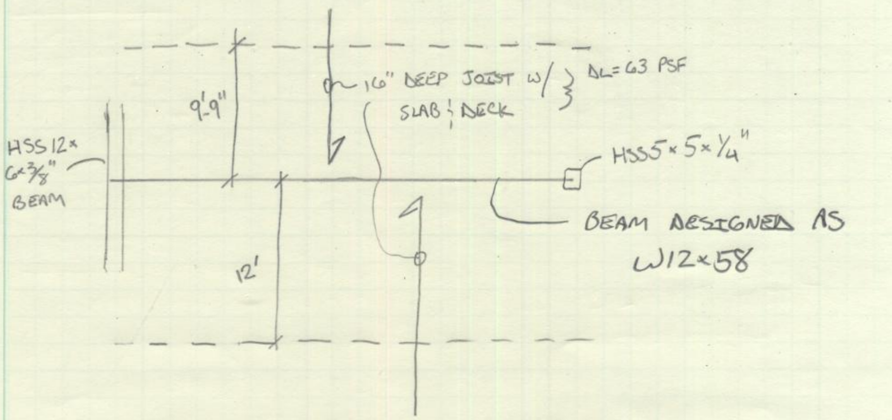
$M = 33.116^k (9.5')$

$\boxed{M = 314^{k\text{-}ft}}$

THE SHEARWALL USES 4" x 14 GA. FLAT STRAP BRACING, W/ A $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$" HSS. (12) 14 GA. NO. 1/4-14 SCREWS OR 1/8" x 6" FIELD WELDS USED TO CONNECT STRAPS TO HSS.

Beam Spot Check

RYAN FLYNN | TECH 1 | BEAM SPOT CHECK



DEAD LOAD FOR COMP. SYSTEM = 65 PSF

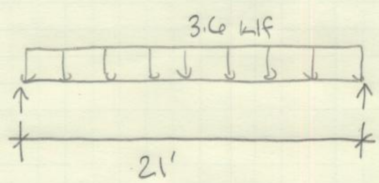
SUPERIMPOSED DL = 20 PSF

LIVE LOAD = 40 PSF

$w_u = 1.2D + 1.6L$

$w_u = 1.2(85(9'\text{-}9" + 12')) + 1.6(40(9'\text{-}9" + 12'))$

$w_u = 3.6\ \text{klf}$



$M_u = \dfrac{wl^2}{8}$

$M_u = \dfrac{3.6(21')^2}{8} = 200\ \text{k-ft}$

$= 2380\ \text{k-in}$

$V_{max} = \dfrac{wl}{2} = \dfrac{3.6(21)}{2} = 37.8^k < 132^k$ ✓

$\phi M_n = 0.9\,(200\ \text{k-ft})$

$\phi M_n = 180\ \text{k-ft} < 324\ \text{k-ft}$

L/220

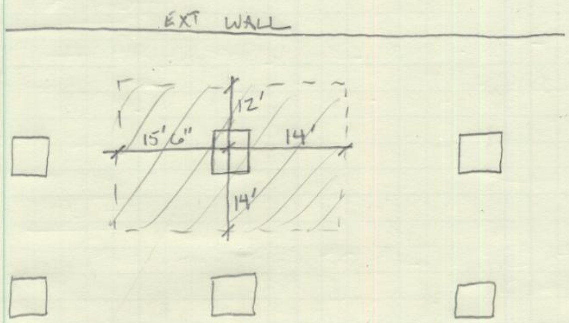
$\Delta_T = \dfrac{L}{240} = \dfrac{21 \times 12}{240} \geq \dfrac{5(3.6)(21)^4(1728)}{384(29000)I} \Rightarrow 517 > 475$ X

$\boxed{\text{NEXT W12× THAT WORKS IS W12×65}}$

Column Spot Check

RYAN FLYNN | TECH 1 | COL. SPOT CHECK

COLS UNDER 12" REINF 1ST FLOOR SLAB



COL - 16×24 CONC.
(18) #6 VERT
#3 @ 12" o.c. TIES

SUPPORTS HSS FROM FLOORS 1, 2 & 3

$$L = 40\left(0.25 + \frac{15}{\sqrt{4(152)}}\right) = 35 \text{ PSF}$$

LOAD ON EA HSS

DL = 65 PSF + 40 PSF (SI) = 105 PSF
LL = 35 PSF

$$1.2(105) + 1.6(35) = 182 \text{ PSF}$$

$$\text{AREA PER COL} = (6' \times 9.5') + (10' \times 9.5') = 152 \text{ ft}^2$$

$$\text{LOAD PER COL} = \underline{28^k} \times 3 = \underline{84^k}$$

$$\text{FIRST FLOOR LOAD} = 1.2(150 \text{ PSF} + 40 \text{ PSF (SI)}) + 1.6(21 \text{ PSF}) = 262 \text{ PSF}$$

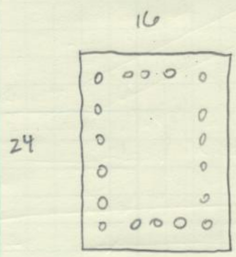
FIRST FLOOR REDUCTION

$$L = 40\left(0.25 + \frac{15}{\sqrt{4(26 \times 29.5)}}\right) = 21 \text{ PSF}$$

TOTAL LOAD

$$\frac{(26 \times 29.5)(262)}{1000} + 84^k + \underset{\text{(DROP PANEL)}}{115.2^k}(1.2) = \boxed{400^k}$$

RYAN FLYNN | TECH 1 | COL SPOT CHECK CONT.



$$P_o = 0.85 f'_c \left(bh - \Sigma A_s\right) + \Sigma A_s f_y$$

$$P_o = 0.85(5000)\left(16(24) - (18)(0.44)\right) + (18)(0.44)(60{,}000)$$

$$\underline{P_o = 2010^K}$$

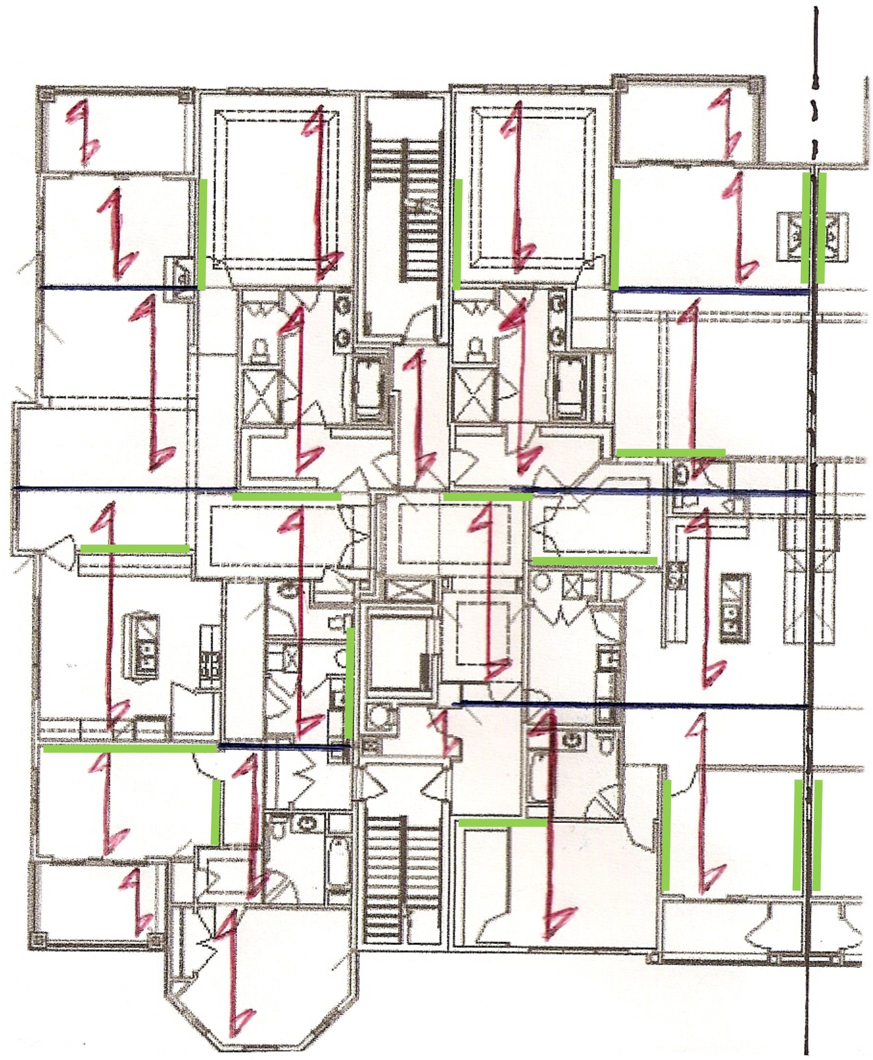
$$\phi P_o = 0.65(2010)$$

$$\phi P_o = 1306.5^K > 400^K \quad \checkmark$$

COLUMN IS MORE THAN ADEQUATE

Floor Framing Plan

The floor plan below shows the typical floor framing for the 2nd thru 4th floors. The red span arrows in the plan indicate the composite joist system while the blue lines represent larger beams and joist girders and the green lines are to mark the shearwalls. This is only one side of the building. For the most part, the apartment complex is symmetric about its centerline.



Typical Exterior Wall Section