Point Pleasant Apartments
Point Pleasant, NJ

Ryan P. Flynn
Structural Option
Faculty Consultant: Dr. Hanagan

Technical Report #3:
Lateral System Analysis and Confirmation Design

Table of Contents

Introduction / Executive Summary

Point Pleasant is a 5-building apartment complex located at the New Jersey Shore. This report will focus on building 1, which is 64,000 square feet and has four stories over a partially exposed parking garage. There are sixteen luxury apartments in the building, four on each floor. The apartments are approximately 2,500 square feet and each has a front balcony facing the central courtyard and a rear balcony overlooking the Manasquan River. The exterior of the building is a combination of stone, stucco, and hardshingle siding. This change in material along with the bump out balconies creates an interesting façade and effectively masks its basic box shape. The roof is a simple hip accented with multiple dormers, a dome feature on one side, and steeple at the center.

The purpose of this report is to perform an in depth analysis of the lateral forces due to wind and seismic loading and the resulting force distribution to the braced frames. A three dimensional model of the lateral system for Point Pleasant Apartments was created using ETABS. As was expected for the hurricane prone region of Point Pleasant, wind was the controlling force for the design.

After the lateral system and rigid diaphragms were entered into ETABS, the program produced a period in both the (N-S) and (E-W) directions. These periods were then entered into the design equations given in ASCE-7 ’05 to provide a base shear of 148 kips in the (N-S) direction and 140 kips in the (E-W) direction. These numbers are significantly lower than the base shear of 224 kips that is listed on the structural plans. This could be due to the superimposed dead loads used as well as incorrect data input in ETABS.

Story forces due to wind calculated in previous reports were entered into ETABS to produce a distribution of load to the braced frames in the building. The ETABS model confirmed the assumption that the forces were equally distributed among the braced frames. Because wind was the controlling design load, spot checks were performed for a typical frame to compare to allowable forces as well as ETABS output. The spot checks confirmed an accurate ETABS model and were consistent with the number of frames and the of size the straps used in the original design.

ETABS effectively calculated the story drift due to wind and seismic loading in both the (N-S) and (E-W) directions. These values were compared to the allowable displacement of H/400 or 1.6 in. For simplification purposes, the complicated roof system was excluded from the model. Thus the wind load at the roof peak, which was insignificant in comparison to the overall load, was ignored and the seismic story force at the roof peak was simply added to the attic level. Because of this simplification, the effective H at the attic level was taken to be 642 in. The maximum drift calculated by ETABS was 1.08 in. due to wind in the (N-S) direction. This value is approximately 70% of the allowable displacement.

Because of the basic box shape of the building and the fact that it is only five stories high, torsion will not have much impact on the design. The animation provided by ETABS confirmed these insignificant torsional effects. The effects of torsion will be looked at in greater depth in future reports.

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

Design Loads

Dead Loads

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

Superimposed Dead Loads

Mechanical, Electrical, Plumbing…………….5 psf
Ceiling Finishes……………………………… 3 psf
Floor Finishes……………………………….. 5 psf

Live Loads

Residential (private rooms and corridors)…... 40 psf
Residential Balconies…………………………60 psf
First Floor Corridors and Lobbies.………….. 100 psf
Roof (Ground Snow)……………………….... 30 psf
Partition Wall Allowance................................ 20 psf

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 basically 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 at 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

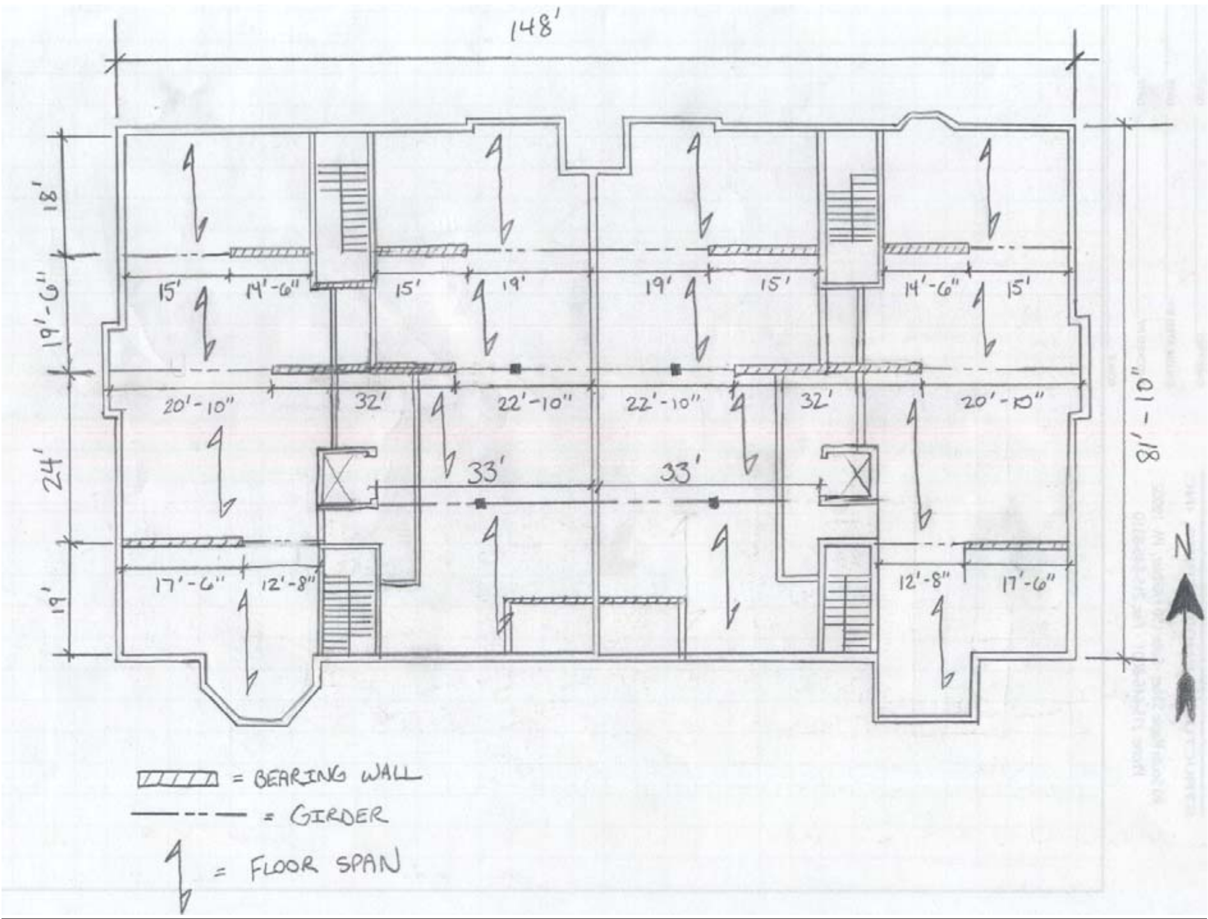
The roof system is a simple hip with two large dormers in the rear and two smaller dormers, a tower, and a dome feature in the front. The roof is made up of light gage metal roof trusses spaced at 48" o.c.

Lateral Framing

The walls of the building are comprised of metal studs, therefore, light gage shearpanels and are utilized to resist lateral load. The shearwalls, which actually act as braced frames, typically consist of 4"x14 gage flat strap bracing with 3 1/2"x3 1/2"x1/2" HSS shapes. The flat straps can either be screwed or welded to the HSS's. All of the panels are 9' 6" in length.

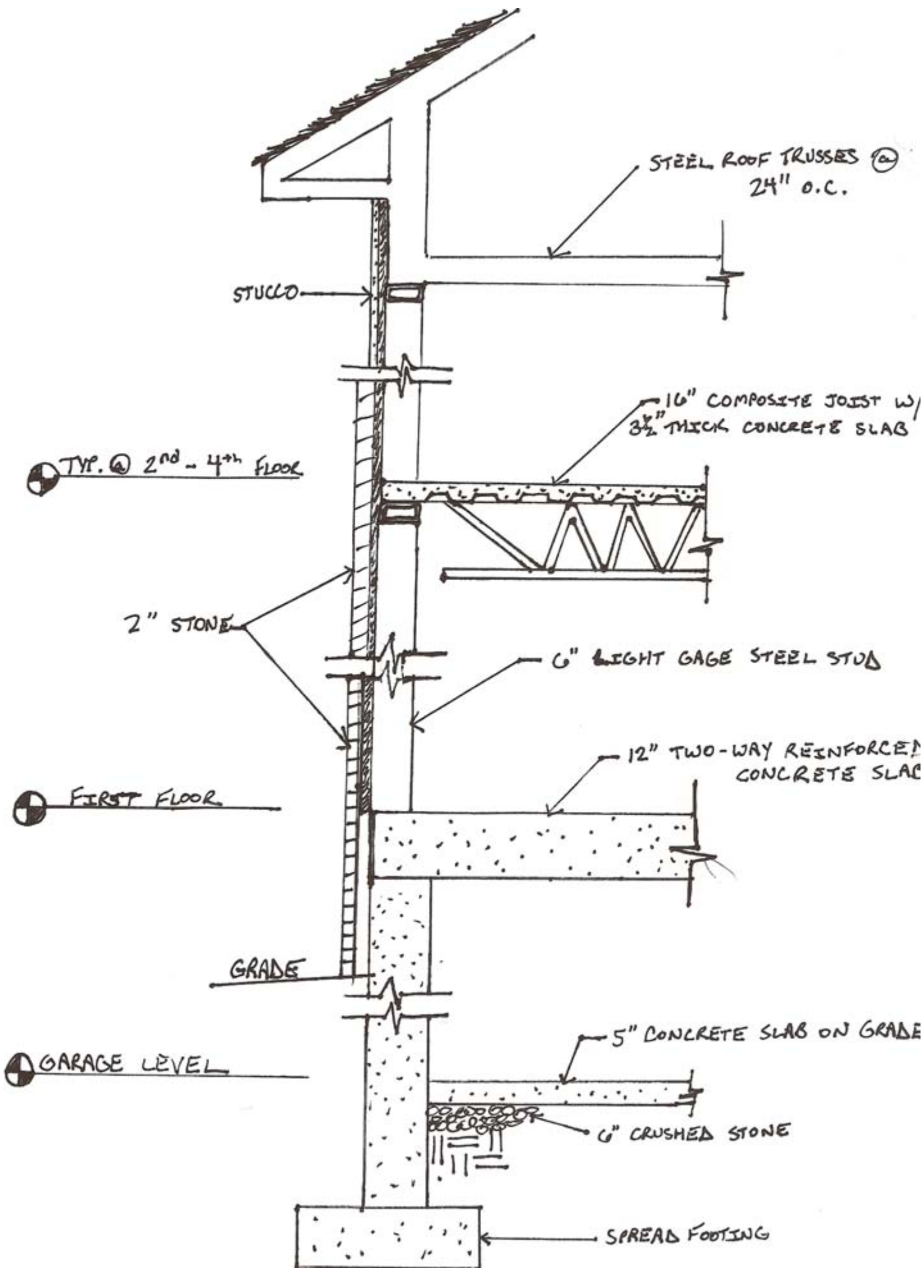
Typical Floor Plan (Structural Layout)

The floor plan below illustrates the typical framing for floors 2-4. The span arrows represent the composite joist system used for these floors. The outline of the building is the same for first floor and parking garage level as well.



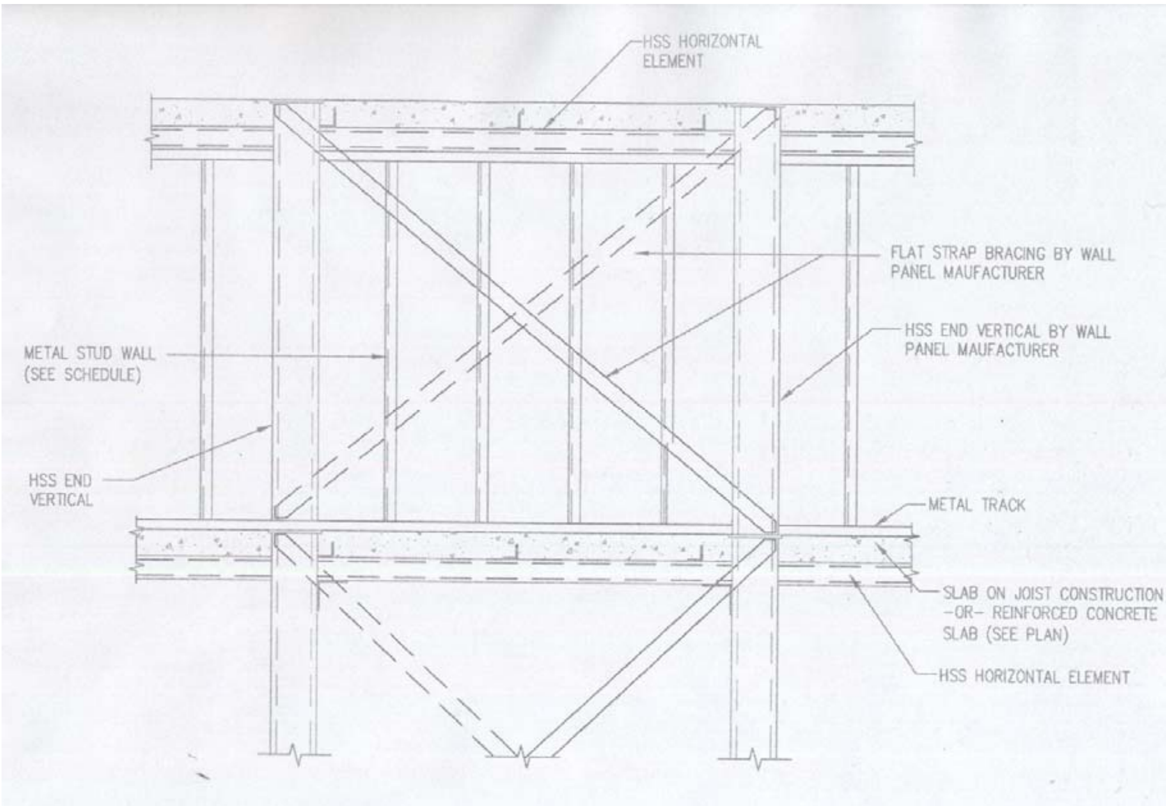
Typical Exterior Wall Section

The section below shows the basic structural framing from the foundation up to the roof. Floors 2-4 were generalized with one section because they use the same composite joist system. At different areas of the building the façade material may change to include hardshingle siding but this image gives a typical snapshot of the framing. How much of the garage that is above grade also changes around the building. For example, at the rear of the building, the full height of the garage is exposed so that cars can enter and exit.



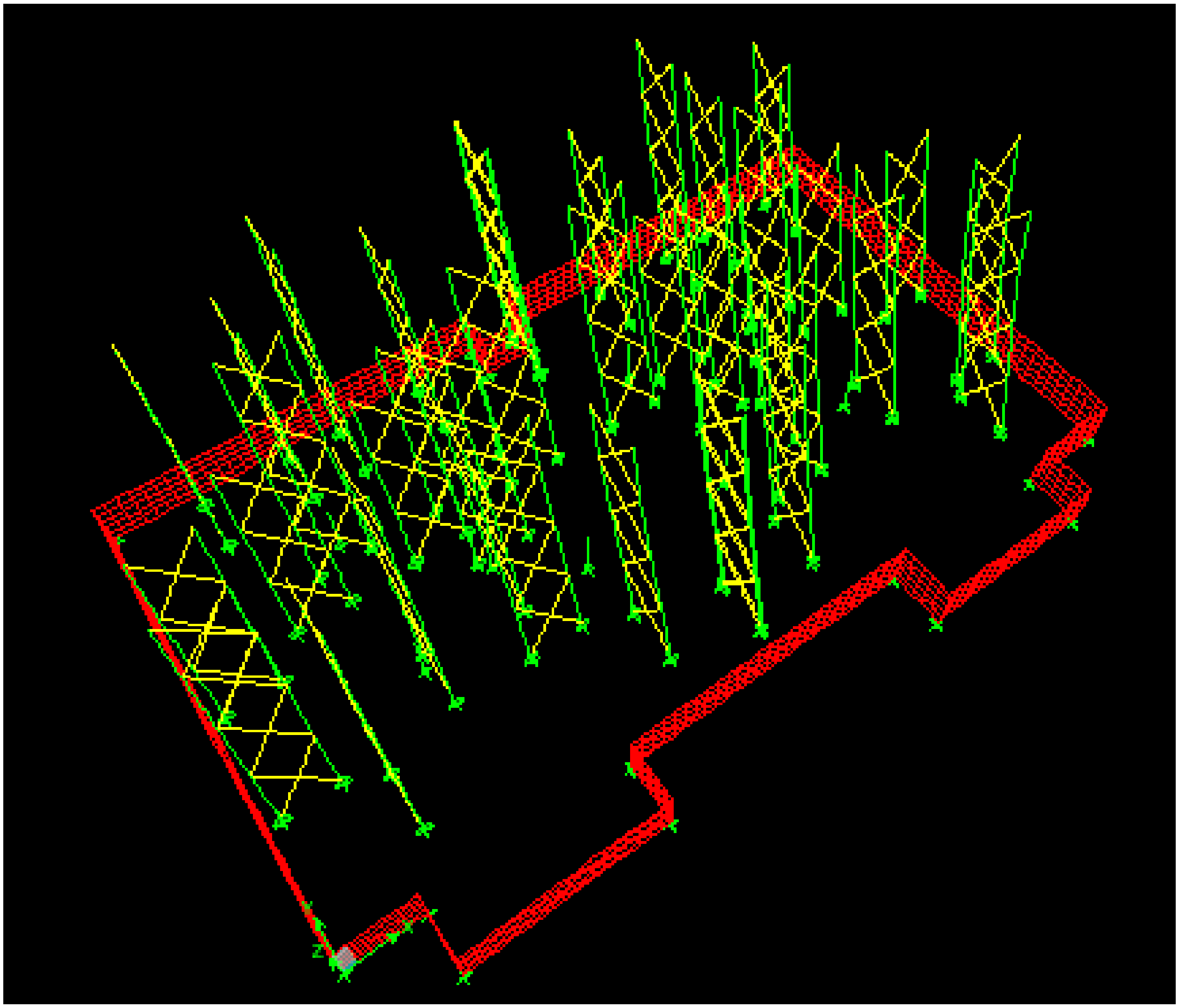
Braced Frame Details

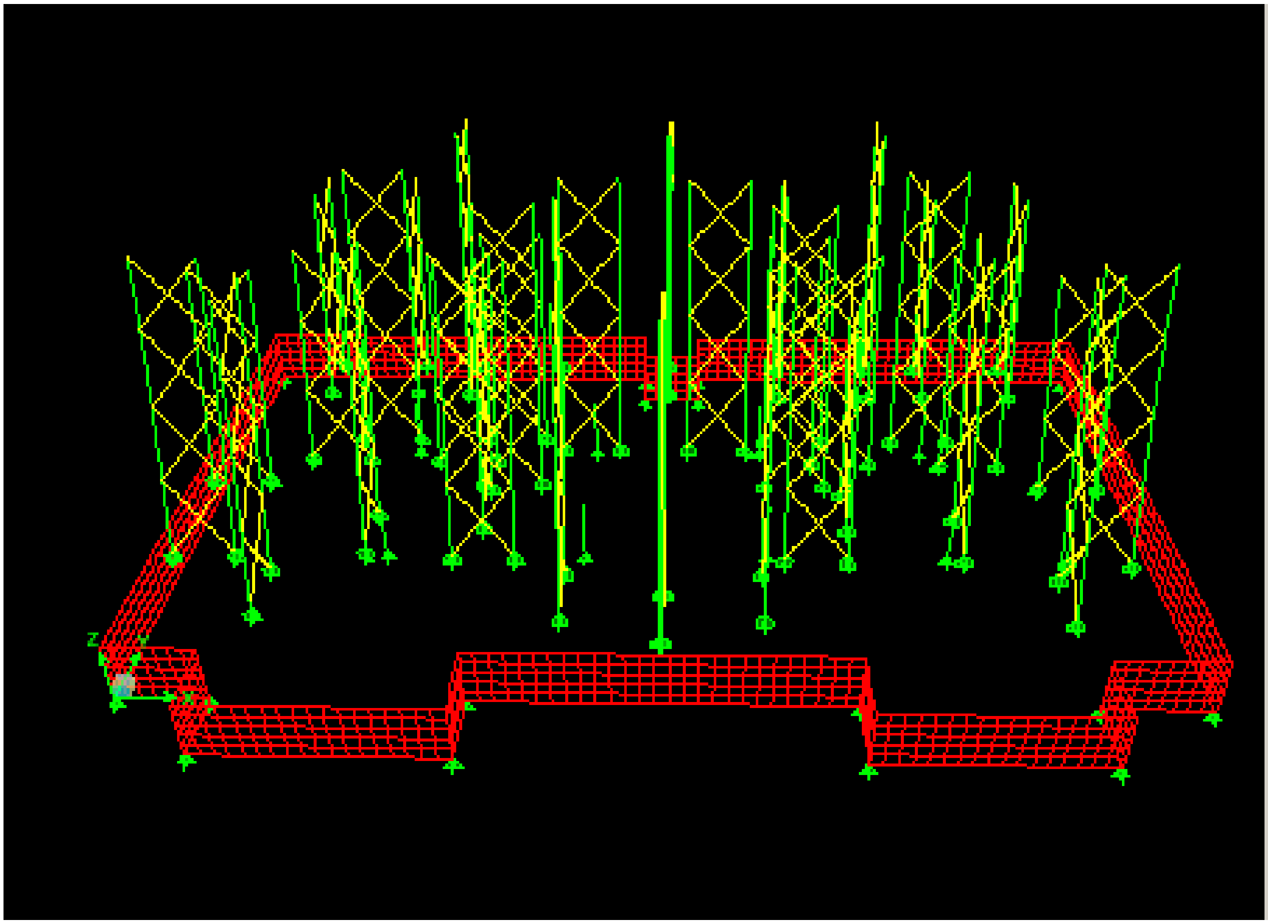
The image below illustrates the braced frames used for lateral resistance in the building. The HSS shapes at each end of the panel act as restraining points for the 4"x14 gage metal cross-braced straps. The story force is distributed among the braced frames, with the forces being transferred into tension in the straps. The manufacturer of the straps is Marinoware. Their design manual was consulted during the spot checks of the straps.



ETABS 3-D View of Lateral System

The images below show the ETABS layout of the lateral system. At the first floor, the lateral resisting element is the 12" thick concrete wall. The concrete columns of the first floor that support the two-way slab above are also modeled. The main lateral resistance is provided by the braced frames throughout the building. Floors 2 thru 4 have the same frame layout while only some of the frames are carried up to the 5th floor. For the purpose of this analysis, the slab on grade for the parking garage is considered to be at ground level. The level of grade actually varies around the perimeter of the building, but for simplification the walls of the parking garage are considered to be completely above grade.





Seismic Analysis

The table below shows the results from Technical Report #1. At this point in the analysis, the simplified method described in ASCE 7-05 was used for the 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, the website http://earthquake.usgs.gov/research/hazmaps/design/ was consulted, 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 its 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	0.05983	9.812	9.812	711.395
Attic	53.5	195	1.1614	12116.3	0.06726	11.031	20.843	590.157
4	43.5	1120	1.1614	56583.4	0.31411	51.515	72.358	2240.895
3	32.67	1120	1.1614	42496.1	0.23591	38.689	111.048	1263.983
2	21.83	1120	1.1614	28395.8	0.15764	25.852	136.900	564.353
1	11	2330	1.1614	29766.7	0.16525	27.100	164.000	298.103
Total:		**6013**		**180136**		**V=164 k**		**M=5670 ft-k**

After entering a model of the building into ETABS, a new period was calculated for the seismic forces in each direction. The two tables below show the new base shear results using the periods calculated by ETABS.

Seismic Floor to Floor Force Distribution (E-W) Direction

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	0.05983	8.376	8.376	607.289
Attic	53.5	195	1.1614	12116.3	0.06726	9.417	17.793	503.792
4	43.5	1120	1.1614	56583.4	0.31411	43.976	61.769	1912.959
3	32.67	1120	1.1614	42496.1	0.23591	33.028	94.797	1079.010
2	21.83	1120	1.1614	28395.8	0.15764	22.069	116.866	481.764
1	11	2330	1.1614	29766.7	0.16525	23.134	140.000	254.478
Total:		**6013**		**180136**		**V=140k**		**M=4840 ft-k**

Seismic Floor to Floor Force Distribution (N-S) Direction

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	0.05983	8.855	8.855	641.991
Attic	53.5	195	1.1614	12116.3	0.06726	9.955	18.810	532.580
4	43.5	1120	1.1614	56583.4	0.31411	46.489	65.299	2022.271
3	32.67	1120	1.1614	42496.1	0.23591	34.915	100.214	1140.668
2	21.83	1120	1.1614	28395.8	0.15764	23.330	123.544	509.294
1	11	2330	1.1614	29766.7	0.16525	24.456	148.000	269.020
Total:		**6013**		**180136**		**V=148k**		**M=5116 ft-k**

The base shear results from ETABS are both lower than the initial calculation of 164k because the ETABS periods of 0.9656 in the (E-W) direction and 0.9158 in the (N-S) direction are higher than the period of 0.8228 that was calculated in Technical Report #1. The base shear of 148k is only about 70% of the base shear of 224k listed on the plans. As in Technical Report #1, one reason for this could be the assumptions for superimposed dead load. Another reason for the difference could be incorrect data input or defining of materials in the ETABS model.

The tables below show the displacement and drift at the top floor under seismic loading in both directions. All of these values are safely under the limitation of H/400. For this analysis, the seismic forces calculated at the roof peak were added to the force at the attic level and the roof height was ignored. This will result in a lower overturning moment but the weight of the building and base shear remain unaffected. This was done to simplify the ETABS model. Thus, the H in the H/400 limitation will be the height at the attic level (642 in) resulting in a displacement restriction of 1.6 in.

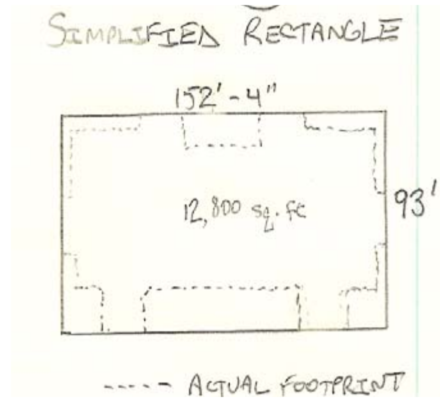
Summary of Story Drift Results From ETABS Due to Seismic Loading (E-W)

	Disp-X	**Disp-Y**	**Drift-X**	**Drift-Y**
Top Story	0.580616	0.05636	0.000829	0.000112

Summary of Story Drift Results From ETABS Due to Seismic Loading (N-S)

	Disp-X	**Disp-Y**	**Drift-X**	**Drift-Y**
Top Story	0.110421	0.58131	0.000157	0.000817

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. For the purposes of calculating story forces and pressures, the building was simplified into a rectangle as shown in the image to the right. Below are tables showing the wind pressures for the building using Method 2 for wind analysis found in ASCE-7-05 as well as the images showing the distribution of load to each story in both directions. The building is considered rigid because the period is less than 1, as was mentioned above in the Seismic Analysis section. The shorter dimension of the building runs in the North-South direction and the longer East-West. The calculations for the wind pressures and resultant forces can be found in the Appendix.

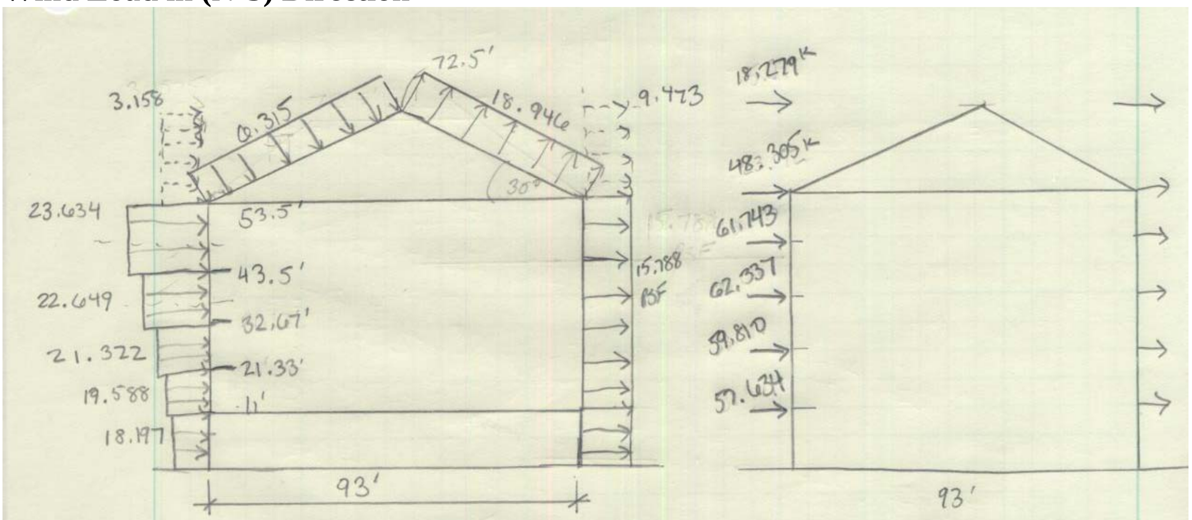
Wind Pressures, PSF (from N-S dir.)

z(ft)	K_z	q_z	Windward P	Leeward P	Total
0-11	0.850	26.634	18.197	-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

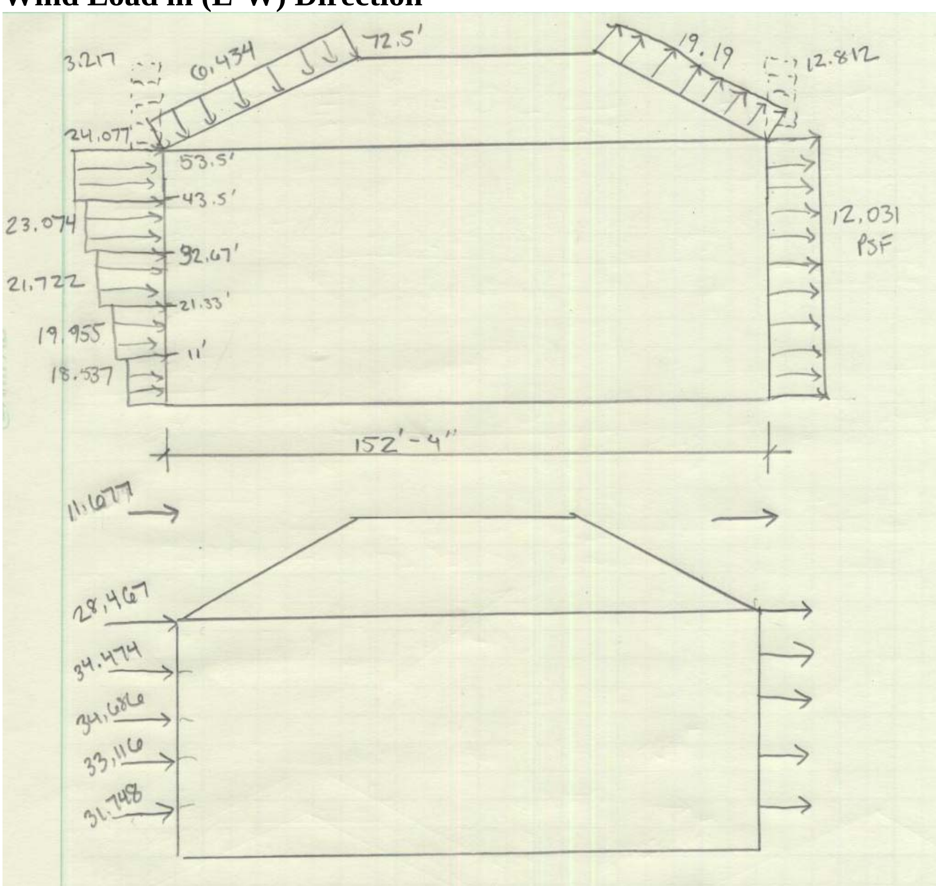
Wind Pressures, PSF (from E-W dir.)

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

Wind Load in (N-S) Direction



Wind Load in (E-W) Direction



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

The tables below show the displacement and drift at the top floor due to wind loading in both directions. All of these values are safely under the limitation of H/400. For this analysis, the wind loads at the roof peak were left out of the ETABS model to simplify it because these forces only accounted for approximately 5% of the overall wind load and the complicated roof structure is very difficult to model. This will only affect the story drift results as they were taken at the attic level. The roof peak forces were still included in the calculation of base shear and overturning moment. Thus, the H in the H/400 limitation will be the height at the attic level (642 in) resulting in a displacement restriction of 1.6 in.

Summary of Story Drift Results From ETABS Due to Wind Load (E-W)

	Disp-X	**Disp-Y**	**Drift-X**	**Drift-Y**
Top Story	0.703749	0.06734	0.001173	0.000017

Summary of Story Drift Results From ETABS Due to Wind Load (N-S)

	Disp-X	**Disp-Y**	**Drift-X**	**Drift-Y**
Top Story	0.003777	1.0814	0.000006	0.001745

After completing both seismic and wind analysis, the report shows that wind is the controlling factor for base shear and overturning moment. The largest story drift of 1.08 in. also results from wind loading. 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. The resisting moments to overturning are much larger than the overturning moments themselves which is expected since the building is relatively long and wide and only five stories high. The resisting moments are found by multiplying the overall weight of the building by the distance from the outside wall to the center of mass in each direction (roughly half of the length and width of the building).

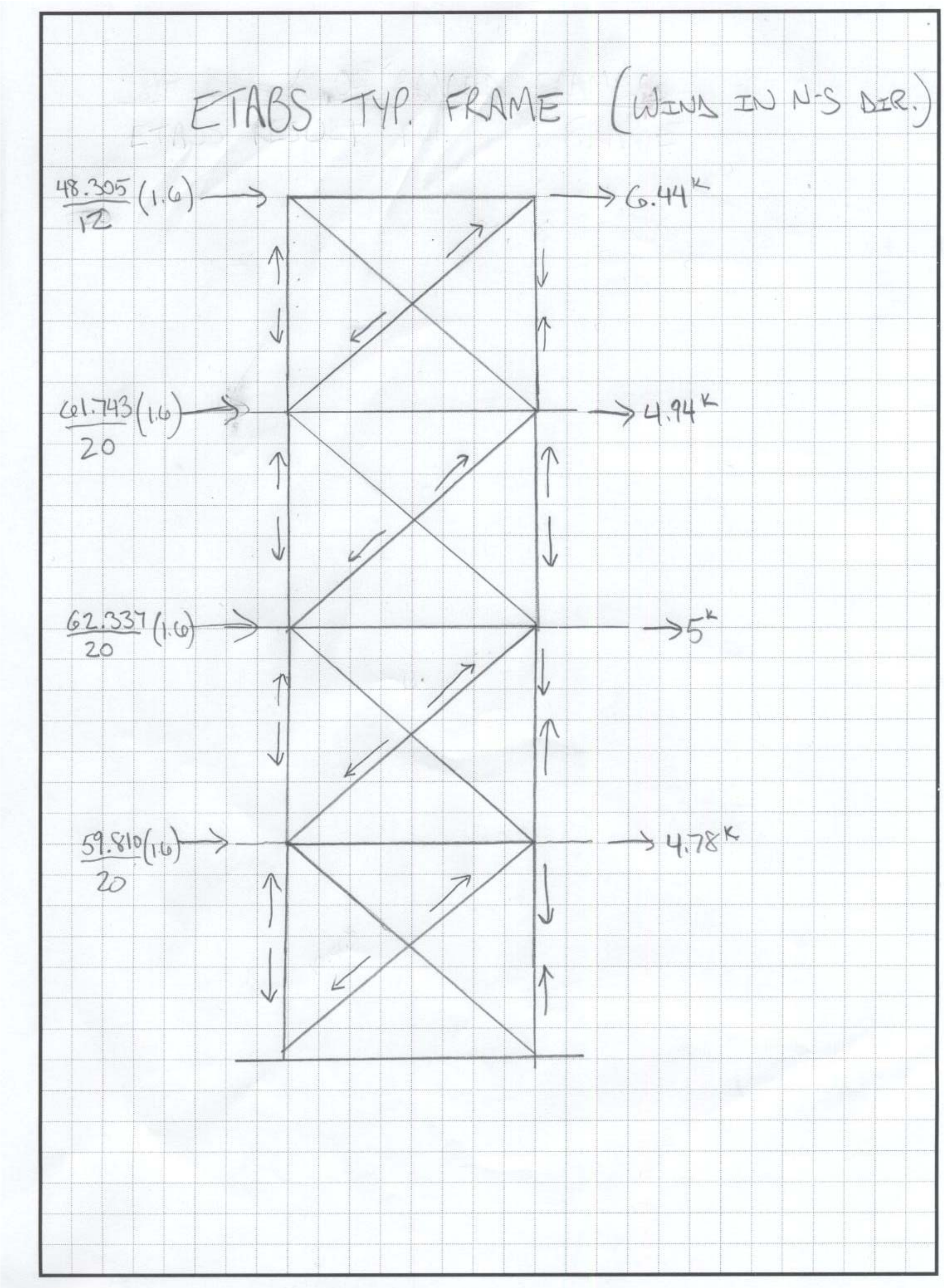
Lateral Load Distribution

For the distribution of the lateral forces to the braced frames, the calculated story forces were applied at the center of mass at each floor. For simplicity, only the braced frames were modeled and connected to rigid diaphragms at the floors. The load cases from ASCE-7 ’05 listed below were used in the analysis of the braced frames in Point Pleasant Apartments. Separate combinations were used for wind and seismic for each direction that the loads were applied.

1.) 1.4D
2.) 1.2D + 1.6L
3.) 1.2D + 1.6W + 1.0L
4.) 1.2D + 1.0E + 1.0L
5.) 0.9D + 1.6W
6.) 0.9D + 1.0E
7.) 1.2D + 1.6S + 1.0L

By inspection within the ETABS model, the story forces are evenly distributed among the braced frames of the building. This is expected since the frames are the same length and are placed symmetrically about the center of the building. The image below shows a typical frame with the story force loading that was entered into the ETABS model. For the purposes of spot checking, the frames were viewed with a factor of 1.6 on the wind load since the braces only take lateral load and because wind controls over seismic in the design. An HSS column within a frame that was acting as a bearing wall was also spot checked. The results for the forces in the strap and HSS calculated by hand were within a couple hundred pounds of the forces calculated by ETABS. This shows that the building was modeled successfully and accurately in the program.

Point Pleasant Apartments has a fairly basic, boxlike shape and the braced frames are placed relatively symmetric about the center of mass of the building. Because of these characteristics, wind and seismic loading create little torsion on the building. Torsional effects will be analyzed more closely in future reports.


ETABS TYP. FRAME (WIND IN N-S DIR.)
48.305/12 (1.6)
6.44k
61.743/20 (1.6)
4.94k
62.337/20 (1.6)
5k
59.810/20 (1.6)
4.78k

Conclusions

By utilizing ETABS, a thorough analysis of the lateral system of Point Pleasant Apartments under wind and seismic loading was completed. Modeling the building in ETABS provided a more accurate period for the building than the one calculated in Technical Assignment #1. The period from each direction was then applied to the equations from ASCE-7 '05 to calculate new base shear values. The new base shear values were lower than those of Technical Assignment #1 by approximately 10% and lower than the shear on the plans of 224k by approximately 30%. Some reasons for this difference could be the superimposed dead load used as well incorrect input of material properties in ETABS.

As was expected, wind proved to be the controlling force for base shear, overturning moment and story drift. The story forces from Technical Assignment #1 were entered into the ETABS model for distribution among the lateral resisting elements. ETABS confirmed the assumption that each braced frame takes the same percentage of the story force. The values produced by the ETABS model were also similar to those of the spot checks. The resulting values of the spot checks for the straps and the HSS shapes in the frames were compared to the capacities found in the Marinoware design manual and steel manual, respectively, and both loads were less than the maximum allowed.

ETABS was also used to calculate the story drift due to wind and seismic loads in both the (N-S) and (E-W) directions. These values were compared to the allowable displacement of H/400. For simplification, the roof system was not entered into ETABS. The load due to wind at the roof peak was insignificant compared to the overall shear and the load at the peak for seismic was simply added to the load at the attic level. This could explain why my spot check values were slightly higher than the forces produced by ETABS. The largest displacement of 1.08 in. was due to wind in the (N-S) direction. This value is approximately 70% of the allowable story drift of 1.6 in. This shows that the ETABS model is accurate because the drift is less than the allowable but is close enough to it to validate the number of panels used.

APPENDIX

Seismic Analysis Calculations

BASE SHEAR CALCULATION

$V = C_s W$

BUILDING WEIGHT

3 FLOORS COMPOSITE JOIST @ 65 PSF

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

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

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

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

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

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

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

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, $32 \times 14 \text{ lb/ft} \times 10' = \underline{4.5^K}$

FLOOR 4, $8 \times 14 \text{ lb/ft} \times 10' = \underline{1.12^K}$

SUPERIMPOSED DEAD LOAD

5 PSF MEP
5 PSF FLOOR FIN.
3 PSF CEILING

$13 \text{ PSF} \times 4 \times 12{,}800 = \underline{665.6^K}$

ROOF 10 PSF TOP CHORD
10 PSF BOTTOM CHORD

$20 \text{ PSF} \times 12{,}800 = 256^{k}$

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

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
$S_{D1} = \frac{2}{3} S_{m1} = \frac{2}{3}(0.1344) = 0.0896$

$T = C_u \cdot T_a$ ($C_u = 1.7$)

$T_a = C_t \cdot h_n^{x} \rightarrow C_t = 0.02,\ x = 0.75$ 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

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

OPTION 1

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

OPTION 2

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

OPTION 3

PER FIG 22-15

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

$C_S = 0.0272$ $\Sigma W = 6020^k$

$V = 0.0272\,(6020)$

$V = 164^k$

NEW PERIOD IN (E-W) DIRECTION FROM ETABS

$T = 0.9656$

$$C_s = \text{Min} \begin{cases} S_{DS}/(R/I) = 0.05625 \\ S_{D1}/(T \times R/I) = 0.0232 \checkmark \text{ CONTROLS} \\ S_{D1} \times T_L/(T^2 \times R/I) = 0.14415 \end{cases}$$

$C_s = 0.0232 \quad \Sigma W = 6020^k$

$V = 0.0232(6020^k)$

$\boxed{V = 140^k}$ (E-W) Direction

NEW PERIOD IN (N-S) DIRECTION FROM ETABS

$T = 0.9158$

$$C_s = \text{Min} \begin{cases} S_{DS}/(R/I) = 0.05625 \\ S_{D1}/(T \times R/I) = 0.0245 \checkmark \text{ CONTROLS} \\ S_{D1} \times T_L/(T^2 \times R/I) = 0.1602 \end{cases}$$

$C_s = 0.0245 \quad \Sigma W = 6020^k$

$V = 0.0245(6020^k)$

$\boxed{V = 148^k}$ (N-S) Direction

Wind Analysis Calculations

WIND SPEED, $V = 120$ MPH

$I = 1.0$

EXPOSURE C

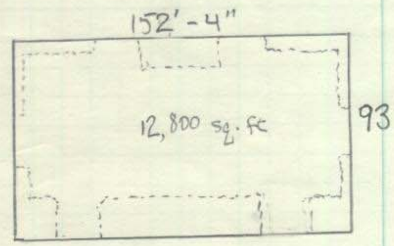
$K_d = 0.85$ FROM TABLE 6-4

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

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

PLAN N

SIMPLIFIED RECTANGLE



---- ACTUAL FOOTPRINT

LONG DIR. (N-S WIND)

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

$\perp$ LONG

$B = 152' - 4"$

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

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

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

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

SHORT DIR. (E-W WIND)

$B = 93'$

$h = 63'$

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

$\perp$ SHORT

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

LONG

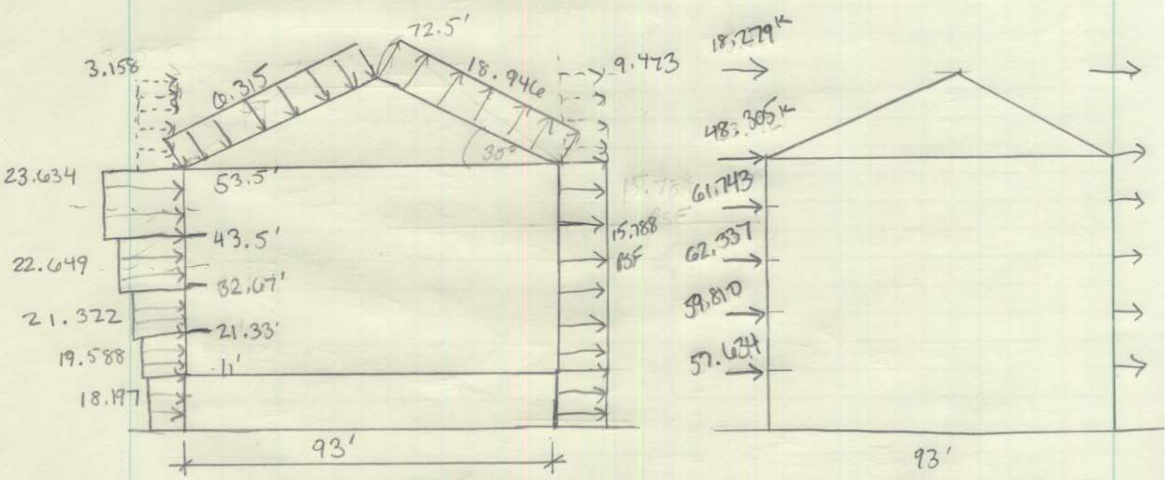
$$G = 0.925\left(\frac{(1+1.7(3.4)(0.1955)(0.856))}{1+1.7(3.4)(0.1955)}\right) = \underline{0.854}$$

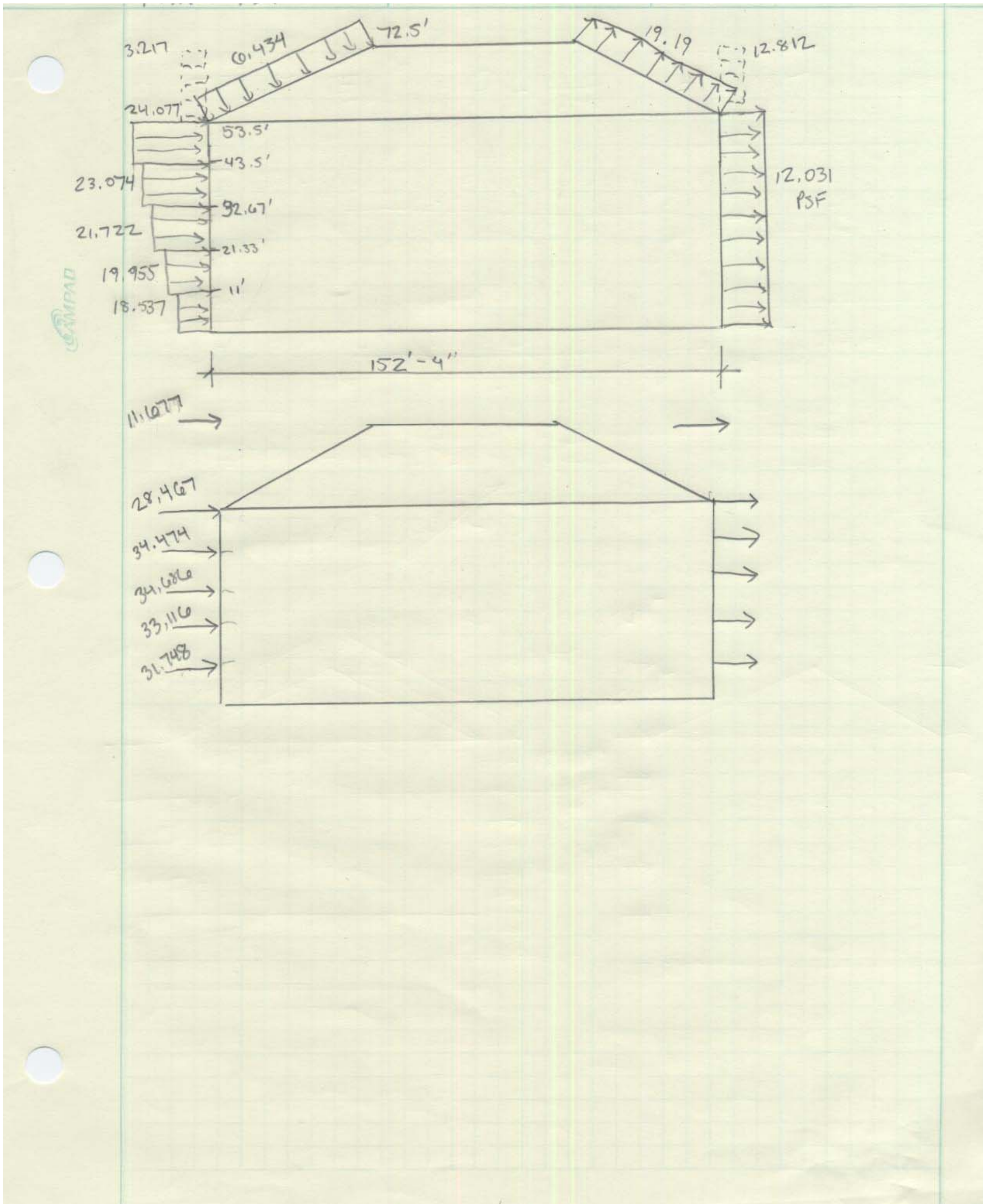
SHORT

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

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

$$\boxed{p = qGC_p} - q_i(\cancelto{0}{GC_{pi}})$$




3.217
6.434
72.5'
19.19
12.812
24.077
53.5'
43.5'
23.074
12.031
PSF
32.67'
21.722
21.33'
19.955
11'
18.537
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

Story Drift Results From ETABS

Summary of Story Drift Results From ETABS Due to Seismic Loading (E-W)

	Disp-X	Disp-Y	Drift-X	Drift-Y
Top Story	0.580616	0.05636	0.000829	0.000112
4th Floor	0.481117	0.04293	0.001082	0.000096
3rd Floor	0.340403	0.03043	0.001305	0.000116
2nd Floor	0.170779	0.01531	0.001305	0.000117
1st Floor	0.001115	0.00015	0.000000	0.000000

Summary of Story Drift Results From ETABS Due to Seismic Loading (N-S)

	Disp-X	Disp-Y	Drift-X	Drift-Y
Top Story	0.110421	0.58131	0.000157	0.000817
4th Floor	0.091562	0.48332	0.000206	0.001086
3rd Floor	0.064736	0.34214	0.000249	0.001309
2nd Floor	0.032396	0.17192	0.000249	0.001310
1st Floor	0.000047	0.00165	0.000000	0.000000

Summary of Story Drift Results From ETABS Due to Wind Load (E-W)

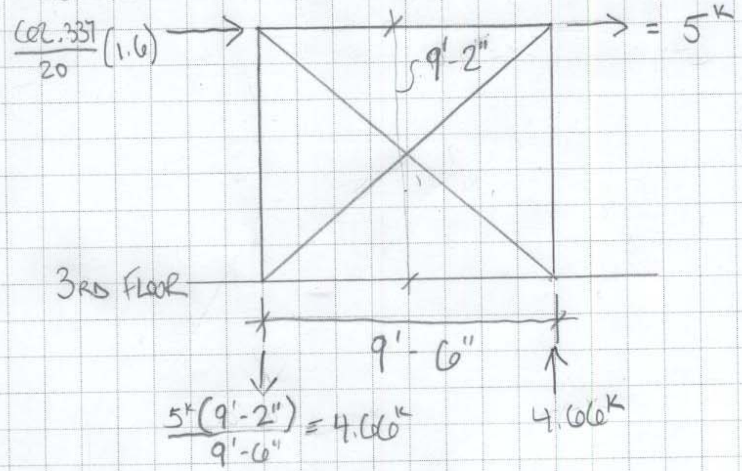
	Disp-X	Disp-Y	Drift-X	Drift-Y
Top Story	0.703749	0.0290	0.001173	0.000017
4th Floor	0.563001	0.0281	0.001244	0.000063
3rd Floor	0.401282	0.0200	0.001489	0.000075
2nd Floor	0.207671	0.0103	0.001587	0.000080
1st Floor	0.001305	0.0001	0.000000	0.000000

Summary of Story Drift Results From ETABS Due to Wind Load (N-S)

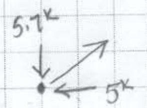
	Disp-X	Disp-Y	Drift-X	Drift-Y
Top Story	0.003777	1.0814	0.000006	0.001745
4th Floor	0.003041	0.8718	0.000007	0.001909
3rd Floor	0.002174	0.6236	0.000008	0.002302
2nd Floor	0.001128	0.3243	0.000009	0.002470
1st Floor	0.000006	0.0032	0.000000	0.000000

<u>Spot Check: Braced Frame Strap</u>

SPOT CHECK OF BRACED FRAME



(WIND FROM N-S DIR.)



force in strap

$F_s = \sqrt{4.66^2 + 5^2}$

$\boxed{F_s = 6.8^k}$

Strap Capacity From MarinoWare (Manuf.) website

4" x 14 gage $P_{allow} = 11.41^k > 6.8^k$ ✓

OK

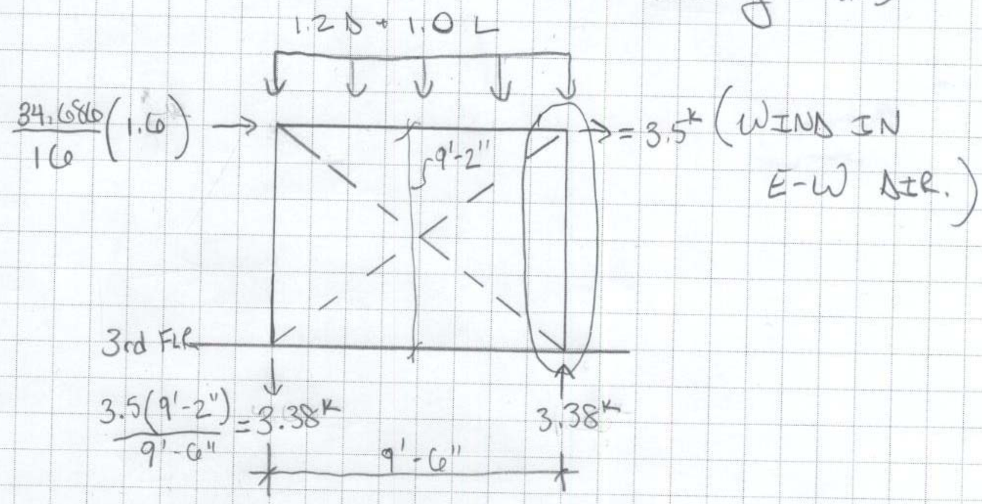
ETABS VALUE FOR TENSION IN BRACE $\Rightarrow$ $P_{ETABS} = 6.5^k$

$6.5^k \approx 6.8^k$

INPUT FOR ETABS SEEMS ACCURATE!

Spot Check: Braced Frame HSS

SPOT CHECK OF HSS COLUMN IN BRACED FRAME (using frame w/in bearing wall)



$1.2(65 + 13) + 1.0(60) = 154$ psf

TRIB AREA $= \underbrace{(19'-6" + 20'-4")}_{\text{span of floor supported}} \underbrace{\left(\frac{16"}{12"}\right)}_{\text{width supported}} = 53.1\ ft^2$

$154\ psf \times 53.1\ ft^2 = 8.2^k$

Total Load in HSS $= 8.2 + 3.38$
$= 11.58^k$

$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$ HSS $P_{max\ comp} = 91.9^k >> 11.58^k$ ✓

ETABS VALUE FOR COMPR. IN HSS $\Rightarrow P_{ETABS} = 12.2^k \approx 11.58^k$