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Structural Option
Building: Vickroy Hall
Location: Duquesne University
Pittsburgh, PA 15282
Date: November 21, 2006
Title of Report: Technical Report 3
Faculty Consultant: Dr. Boothby

Executive Summary

The purpose of this report is to analyze and confirm the aspects of the lateral resisting system of Vickroy Hall. The loads for both wind and seismic have been recalculated and are up to the standard code (IBC 2003 and ASCE-07).

In this report, the lateral system is described in great detail including explanations on the façade, the support, and floor system. From there, hand calculations were performed for a basis from which to compare later values from a computer model on ETABS. The calculations performed in this section were that for the Main Wind Force Resisting System and the Seismic Resisting System. The Foundation Impact was analyzed as well, using the ETABS information.

Next, you will find the distribution of the lateral loading through the building. There is a load path and a distribution example. After the distribution section, the ETABS model was analyzed noting area concerns, including notes on the animation of the model and torsion.

Lastly, there are spot checks of drift and strength. Allowable code and ETABS results are present, along with sample calculations.

In summary, I believe that the lateral system does its job very well. Due to the moment frames and possibly even the aerodynamic effects of the wall surfaces, the wind does not seem to affect the building, except on a very small scale. The seismic is also controlled well by the moment connections. I do believe that there may be some unforeseen problems with the ETABS model due to inexperience. However, I think it did a passing job for allowing the analysis of Vickroy Hall.

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1. Introduction

Standing at approximately 105' above the grounds below, Vickroy Hall shows its beautiful façade to the passerby. This Hall is a Living/Learning center for upper class students at Duquesne University. The Hall serves as a residence hall with meeting rooms, multipurpose rooms and laundry facilities, and offices on the ground floor. The suites that the students use as their residency consist of two double rooms with an adjoining bathroom. Each room is approximately 150 square feet and contains at least one window. Some rooms have two or three because of their location within the building. The building can accommodate up to 280 students. Please see Figure 1 in the Appendix for the typical floor plan.

The façade has not only won an award at the 1999 Western PA Golden Trowel Masonry Awards, but it, along with its components behind the scenes, also works efficiently as a structural system. This 77,000 square foot, eight story building, battles wind forces from the three surrounding rivers in Pittsburgh, and has the potential to withstand the seismic forces of its region.

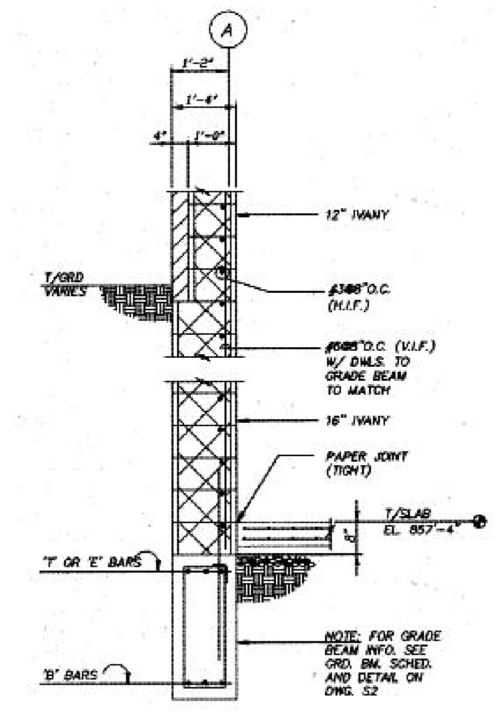
To complicate the ability to withstand the forces of nature, Vickroy Hall has multiple protrusions and two story columns around the base. These could force the building to submit to nature, but it stands strong, with no visible problems to the façade. The lateral system has withstood the sands of time.

This report is to describe the lateral system in great detail, and use IBC 2003 and ASCE-7 to analyze the lateral system. The analysis will include calculations based on wind loading, seismic loading, drift, foundation impact, and torsion. Strength checks of critical members will also be included. An ETABS computer model was used in the generation of some of the values for the comparison to hand calculations.

2. The Lateral Resisting System

2.1 The Façade

The facade is primarily made up of brickwork, accented with bands of concrete. Behind the façade, there are 6" – 16 gage structural metal studs with batt insulation between the framing components. Relief angles are positioned at every floor to prevent the cracking of the façade. The windows are composed of aluminum with plastic laminate sills (see photos below).

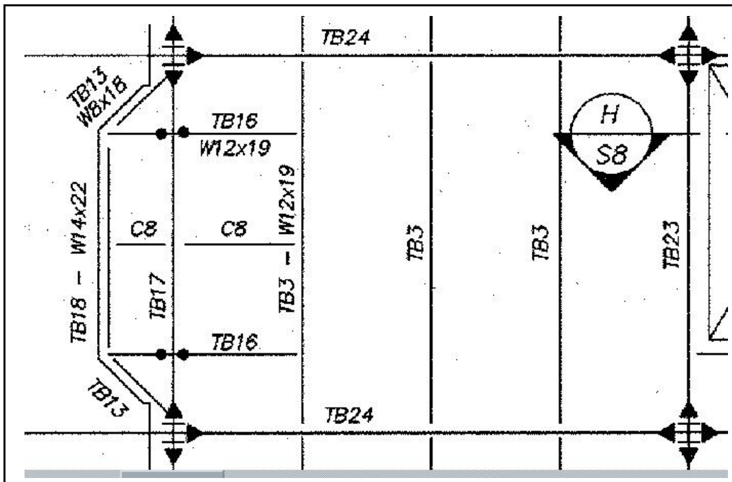


(Above) Reinforced masonry wall

(Right) Reinforced masonry wall detail showing transition from 16" Ivany block to 12" Ivany block

2.2 The Support

The façade is supported by a structural steel frame consisting of C-channels and W-shapes The W-shapes are the framing for typical members and the C-channels provide support for the cantilevers and other protrusions. They are usually oriented perpendicular to the other framing members. The main members extending from column to column are detailed as moment connections. These moment connections are either classified as a wind moment connections or a moment resisting connections. The typical floor plan generally calls for W12 to W16's. (See partial framing plan below or Figure 2 in the Appendix which illustrates the typical full framing plan.).



Typical Framing Plan showing Partial Framing of cantilevered protrusion

2.3 The Floor System

The floor system is a composite metal and concrete deck. On a typical floor, the deck is 2" – 20 gage corrugation with 3-1/4" light weight concrete and 6x6 – W2.9 x W 2.9 welded wire fabric. The deck was to be welded to the supporting structural member. (See photo below)

Typical Floor System: Shows corrugated metal deck supported by steel framing

3. Lateral Load Analysis (Hand Calculations)

The lateral system was evaluated using wind forces and seismic forces according to IBC 2003 and ASCE-7. The values in Technical Assignment 1 were from BOCA 1999. Therefore, I re-calculated the values to conform to the accepted current code standards.

3.1 Main Wind Force Resisting System

Assumptions:

1. Category II
2. I = 1.0
3. Exposure B
4. Hip roof due to geometry
5. Neglecting inside open space on roof that frames create
6. Rigid Structure

Long Side of Building Windward:

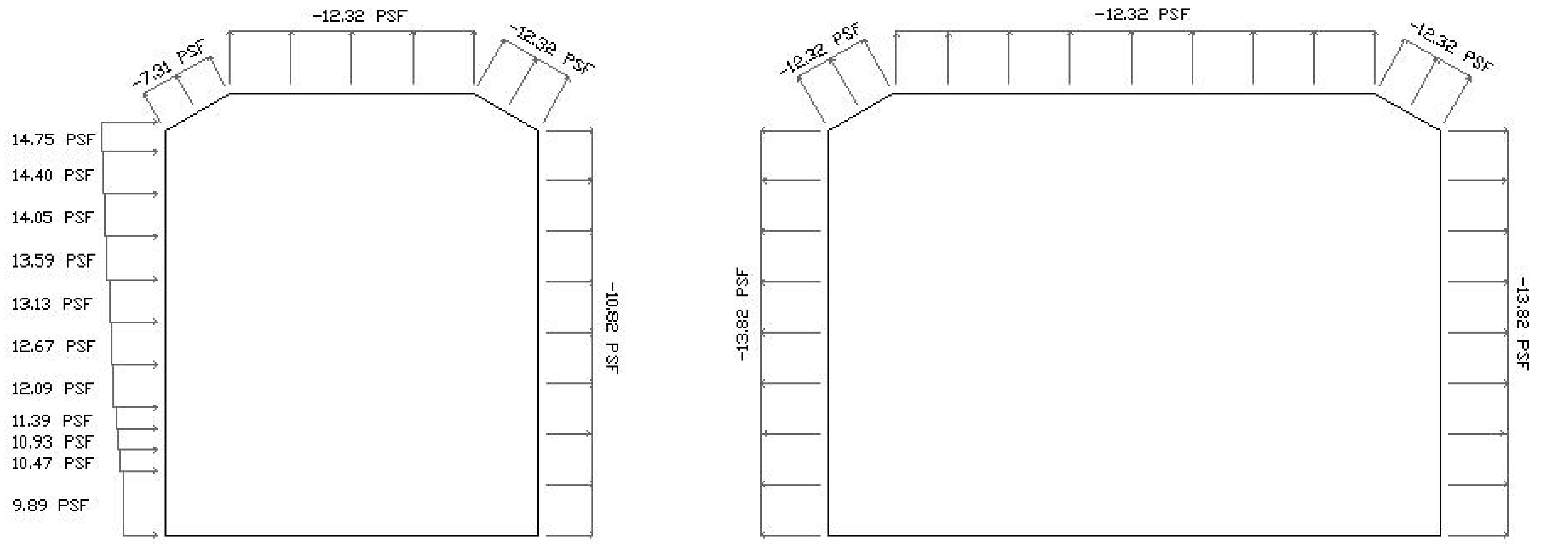
Long Side Windward: MWFRS								
			Windward			Leeward		
			P=qGCp - q(Gcpi)			P=qhGCp - qh(Gcpi)		
				Roof				
z	Kz&Kh	qz	Wall	Negative	Positive	Roof	Leeward Wall	Side Walls
0-15	0.57	10.05	9.89	-7.81	6.31	-12.32	-10.82	-13.82
20	0.62	10.93	10.47	-7.81	6.31	-12.32	-10.82	-13.82
25	0.66	11.63	10.93	-7.81	6.31	-12.32	-10.82	-13.82
30	0.70	12.34	11.39	-7.81	6.31	-12.32	-10.82	-13.82
40	0.76	13.40	12.09	-7.81	6.31	-12.32	-10.82	-13.82
50	0.81	14.28	12.67	-7.81	6.31	-12.32	-10.82	-13.82
60	0.85	14.98	13.13	-7.81	6.31	-12.32	-10.82	-13.82
70	0.89	15.69	13.59	-7.81	6.31	-12.32	-10.82	-13.82
80	0.93	16.39	14.05	-7.81	6.31	-12.32	-10.82	-13.82
90	0.96	16.92	14.40	-7.81	6.31	-12.32	-10.82	-13.82
100	0.99	17.45	14.75	-7.81	6.31	-12.32	-10.82	-13.82
120	1.04	18.33	15.32	-7.81	6.31	-12.32	-10.82	-13.82

V (mph) =	90				
I =	1				
Kd =	0.85				
Kzt =	1				
qh =	18.33				
G =	0.82				
h = 105	L = 88'	B = 144'	L/B = 0.61	h/L = 1.19	
Cp =	Windward	Leeward	Side	Roof (leeward)	Roof (windward)
	0.8	-0.5	-0.7	-0.6	-0.3
					0.2
GCpi = (+-)	0.18	0.18			

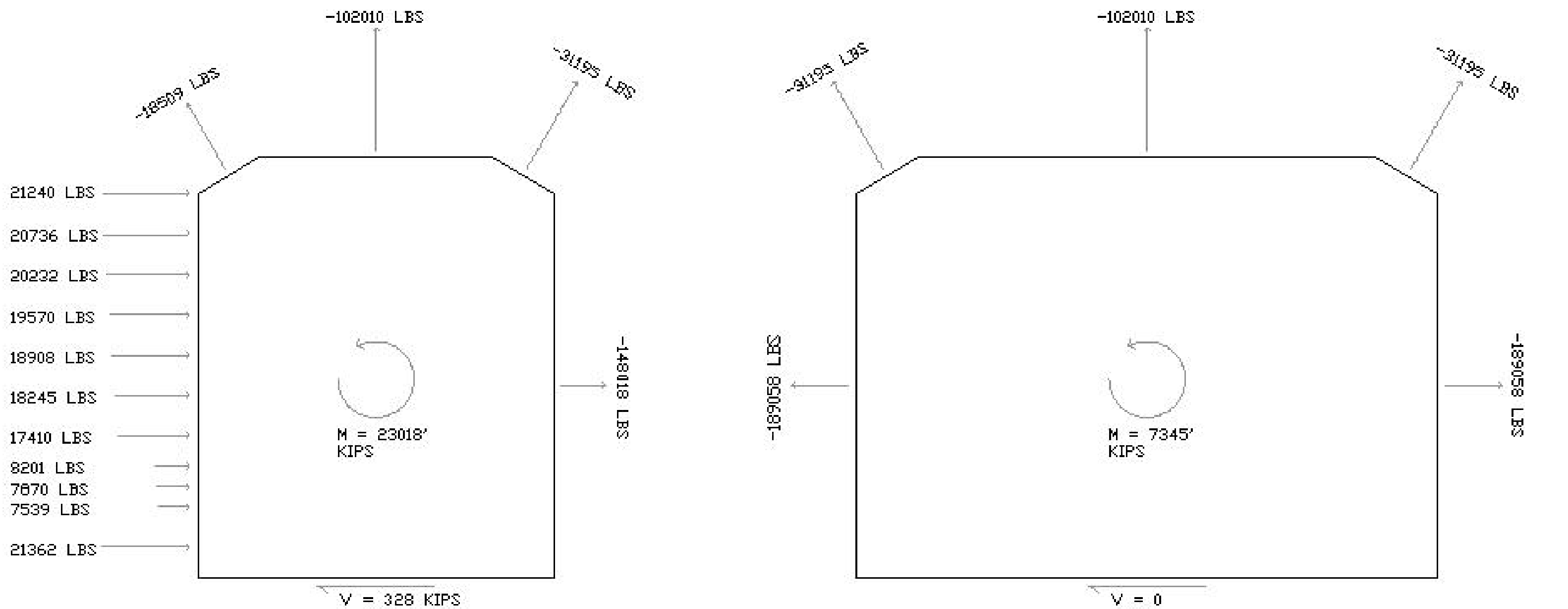
Short Side of Building Windward

Short Side Windward: MWFRS								
			Windward			Leeward		
			P=qGCp - q(Gcpi)			P=qhGCp - qh(Gcpi)		
				Roof				
z	Kz&Kh	qz	Wall	Negative	Positive	Roof	Side Walls	Leeward Wall
0-15	0.57	10.05	10.05	-7.92	6.38	-12.54	-14.08	-7.92
20	0.62	10.93	10.64	-7.92	6.38	-12.54	-14.08	-7.92
25	0.66	11.63	11.12	-7.92	6.38	-12.54	-14.08	-7.92
30	0.70	12.34	11.59	-7.92	6.38	-12.54	-14.08	-7.92
40	0.76	13.40	12.30	-7.92	6.38	-12.54	-14.08	-7.92
50	0.81	14.28	12.89	-7.92	6.38	-12.54	-14.08	-7.92
60	0.85	14.98	13.37	-7.92	6.38	-12.54	-14.08	-7.92
70	0.89	15.69	13.84	-7.92	6.38	-12.54	-14.08	-7.92
80	0.93	16.39	14.31	-7.92	6.38	-12.54	-14.08	-7.92
90	0.96	16.92	14.67	-7.92	6.38	-12.54	-14.08	-7.92
100	0.99	17.45	15.03	-7.92	6.38	-12.54	-14.08	-7.92
120	1.04	18.33	15.62	-7.92	6.38	-12.54	-14.08	-7.92

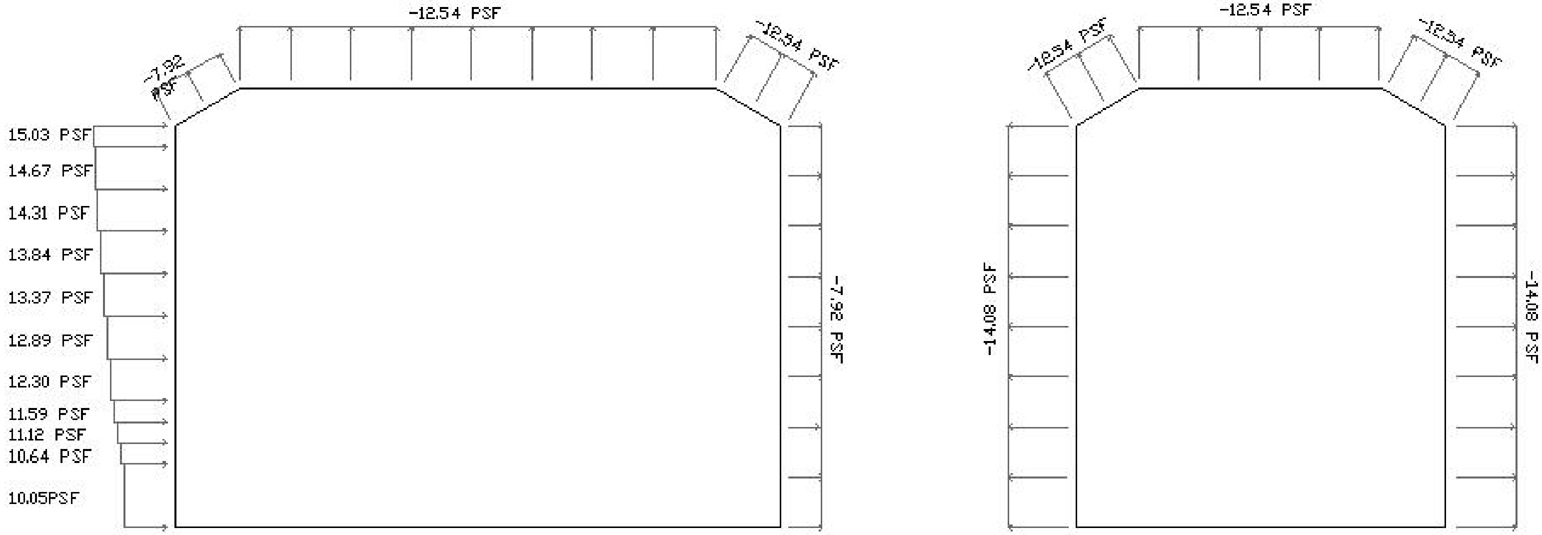
V (mph) =	90				
I =	1				
Kd =	0.85				
Kzt =	1				
qh =	18.33				
G =	0.84				
h = 105	L = 144'	B = 88'	L/B = 1.64	h/L = 0.73	
Cp =	Windward	Leeward	Side	Roof (leeward)	Roof (windward)
	0.8	-0.3	-0.7	-0.6	-0.3
					0.2
GCpi = (+-)	0.18	0.18			



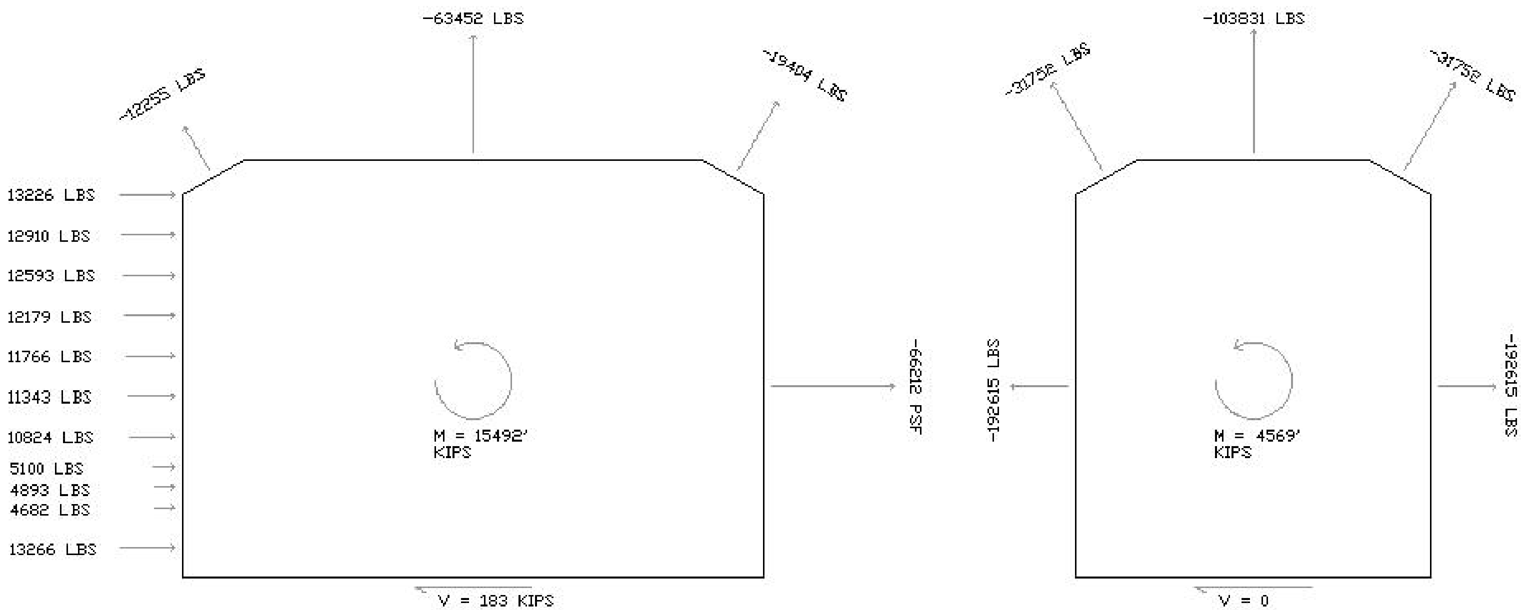
LONG SIDE WINDWARD
WIND LOADING



LONG SIDE WINDWARD
WIND FORCES



SHORT SIDE WINDWARD
WIND LOADING



SHORT SIDE WINDWARD
WIND FORCES

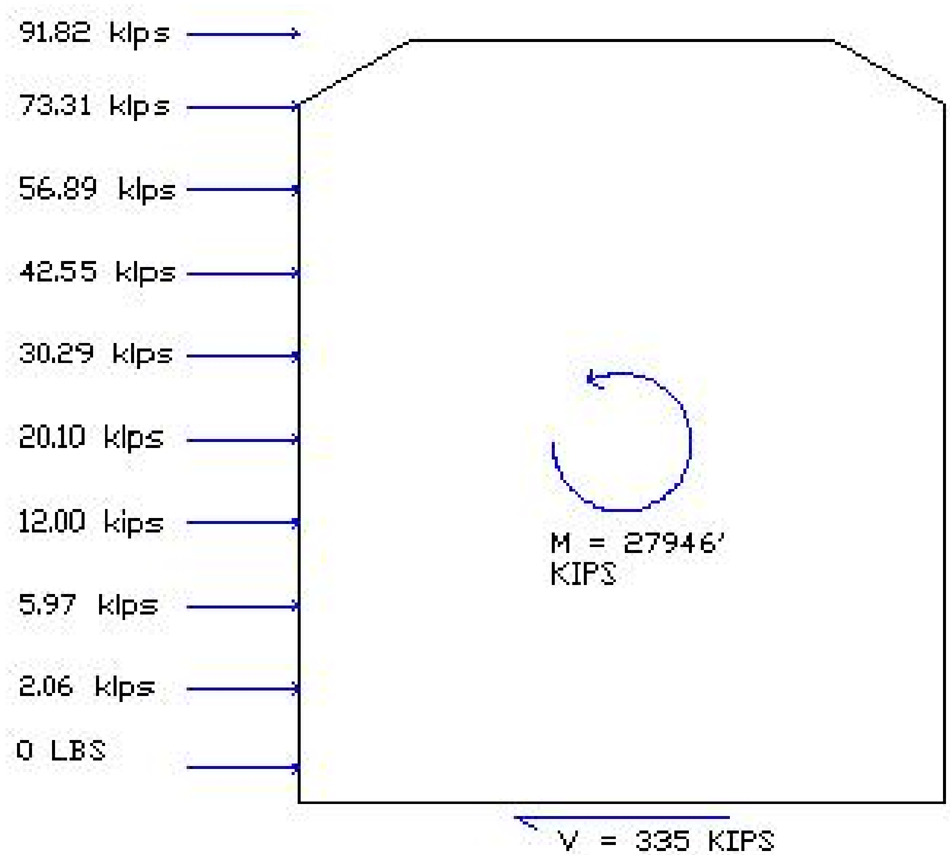
3.2 Seismic Resisting System

The Chart and Diagram below show the assumptions and results of the seismic analysis. For the excel spreadsheet detailing the calculations, please refer to Figure 3 in the Appendix.

Assumptions:

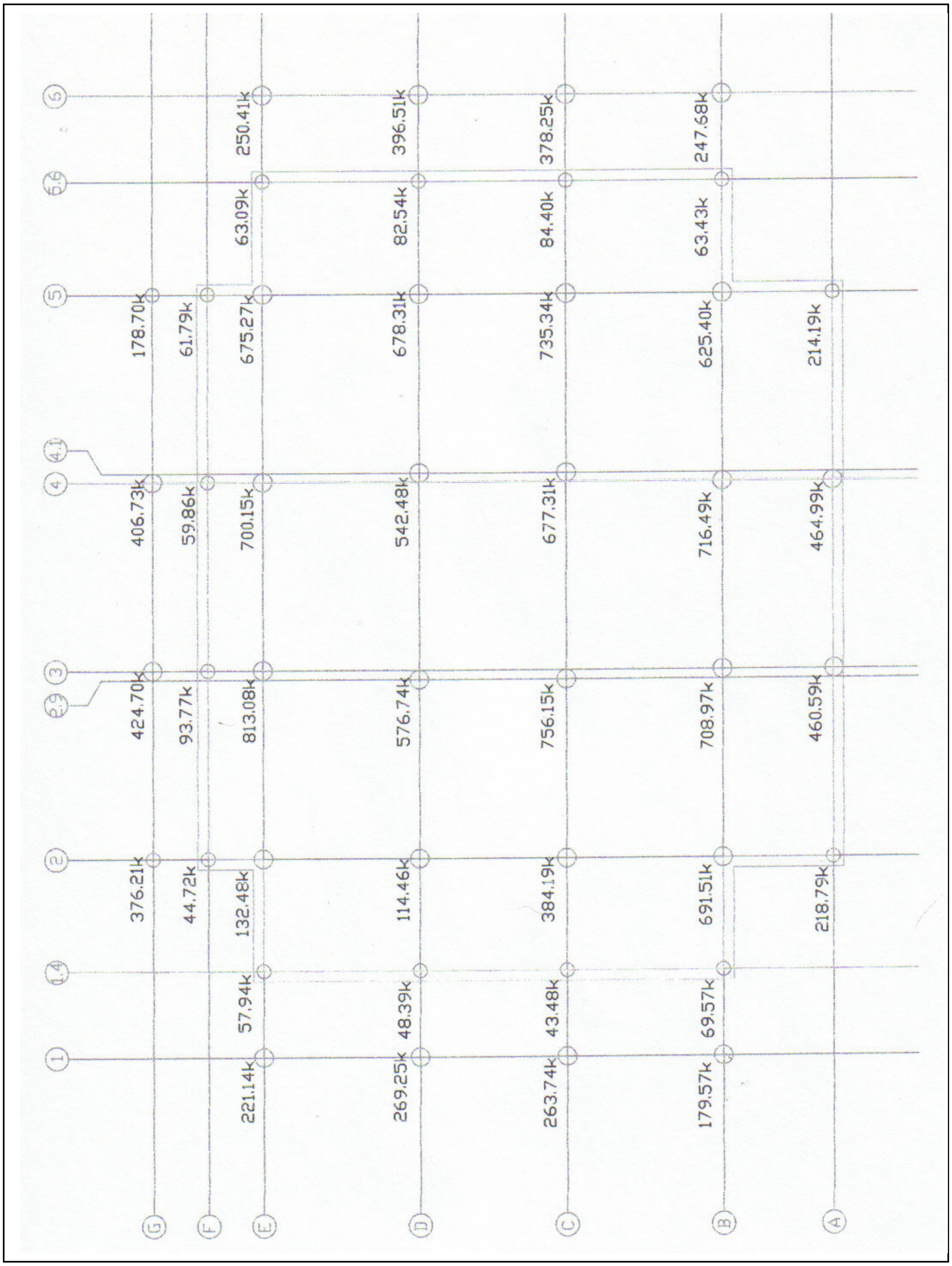
1. Occupancy Category II
2. Seismic Use Group I
3. I = 1.0
4. Ordinary Moment Frame: R = 3, Cd = 3
5. Site Class D
6. Ss = 0.127 => S_{MS} = 0.2032
7. S_1 = 0.054 => S_{M1} = 01296
8. Fa = 1.6
9. Fv = 2.4
10. Ta = 0.8 Conservatively
11. K = 2 Conservatively
12. Seismic Design Category B
13. Allowable Story Drift = $0.02h_{sx}$
14. Story Heights
    a. Mechanical Mezzanine: 4.5'
    b. Story 1: 15.33'
    c. Story 2-8: 11.33'
    d. To Top of Roof: 10'

14. Floor Areas
    a. Total: 77,000 sf
    b. Mezzanine: 200 sf
    c. 8 floors at 9,600 sf
15. Loads
    a. Floor:
       i. W_D = 61 psf
       ii. W_L = 40 psf
       iii. W_U = 138 psf
    b. Roof:
       i. Snow = 25 psf
       ii. W_D = 61 psf
       iii. W_U = 113 psf
    c. Walls:
       i. 15 psf for brick façade
       ii. Perimeter: 371'



3.3 Foundation Impact of Lateral Loading

The following diagram details how the caissons are impacted due to the wind loading. The allowable loading on the caissons is designed for a maximum of twenty-five tons per square foot. All of the values below are within the maximum loading of the caissons. The caisson values range from 30” to 54” in diameter.



3.4 Summary of Lateral Load Analysis

According to the calculations, having the long side of the building facing windward is more critical than having the short side of the building facing windward. However, the seismic analysis shows that it is more critical than the long side of the building facing windward by a moment of 5,000'-kips and a shear of 5 kips. Therefore, the seismic will control the design, but the wind cannot be ignored either because of its significant forces as well.

4. Distribution of Lateral Loading

4.1 Load Path

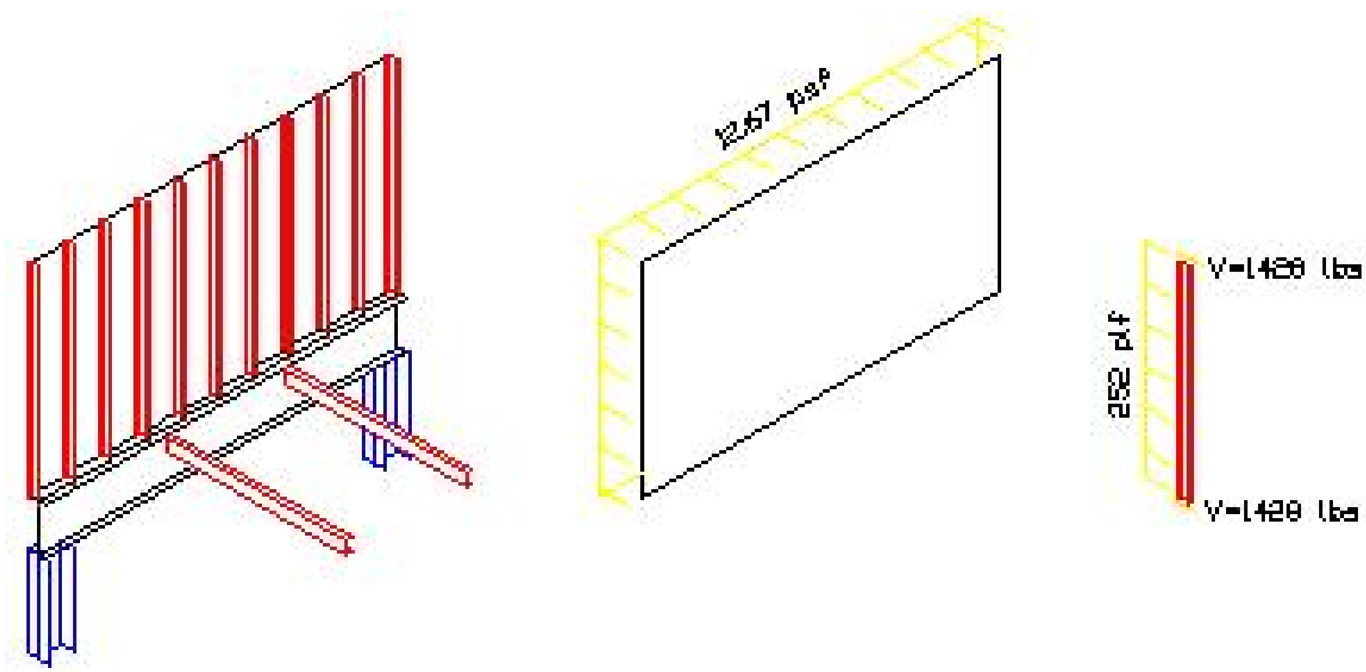
The lateral loads should be distributed to the system through the following order:

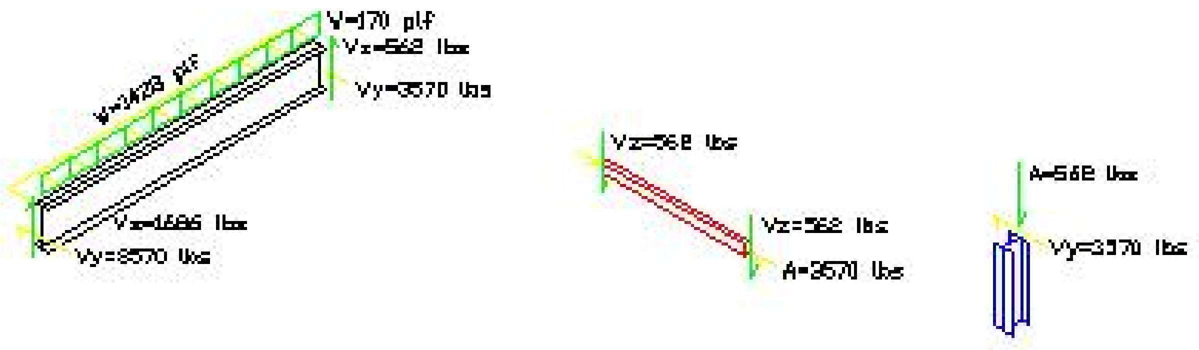
1. Façade
2. Light Gage steel framing
3. Perimeter (Secondary) Structural Steel Framing
4. Interior (Primary) Structural Steel Framing
5. Moment Connections
6. Columns
7. Piers
8. Grade Beams
9. Caissons

4.2 Distribution of Lateral Loading Example

Assumptions:

1. Floor 3, Long side windward: 12.67 psf loading
2. H = 11.33', Width of frame = 19'-10"
3. Metal Studs at 24"o.c.
4. Wall Load = 15 psf





Additional Loads Not on Drawings:

$M_{mombeam}$ = 2696' lbs
M_{zIbeam} = 562' lbs
M_{yIbeam} = 5200' lbs
M_{Column} = 5762' lbs
C channel sees additional floor loading that is not part of the lateral system

5. Lateral Load Analysis (ETABS Program)

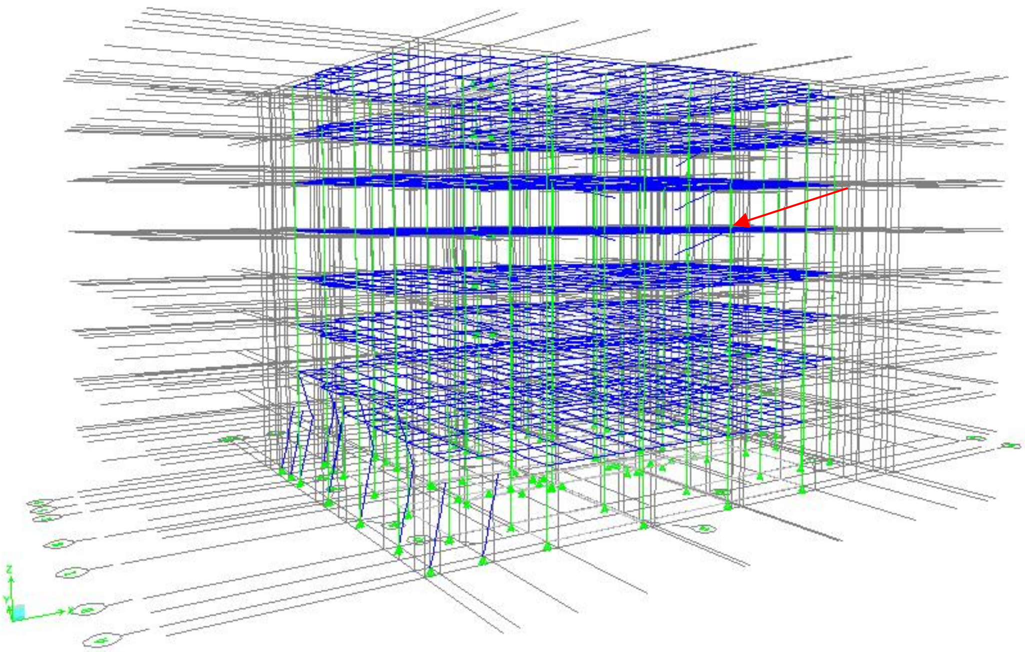
According to the loading of the columns, the above distribution is indeed logical. An example of the loading under its most extreme loading (1.2D + 1.6L + 0.5S) is shown below. A complete set of column forces may be furnished upon request.

Column C2								
Level	Roof	8	7	6	5	4	3	2
Load (k)	62.97	122.99	182.97	242.32	300.83	359.33	412.05	460.59

5.1 Areas of Concern

The model of the deflected shape shows broken beams on the frames surrounding the mechanical ductwork openings on every level. These frames are specifically between column lines B and C and D and E and 4 and 5. The most severe rupture happens with a loading of (1.2D + 1.6W +1.0L +0.5S). Note the red arrow in the Figure on the next page.

In addition, the animation does not show the building deflecting at all, under any of the loading conditions. This evaluation is supported by the deflection values in the output from ETABS. I believe that this is due to the moment connections at every point between the primary steel members and columns.



5.2 Torsion

Due to the symmetry of Vickroy Hall on every level, coupled with the fact that there are no shear walls and only moment frames, I do not believe that there is torsion on the building. The incidental torsion that could be placed into calculations (5%) was not taken into effect in this case. The Center of Rigidity and Center of Mass would be in the same place due to the symmetry of the building geometry, floor symmetry, and lack of shear walls.

6. Spot Checks

6.1 Drift Checks

Allowable

The allowable drift value for wind loading is h/400. This is from the Gaylord and Gaylord Engineering Handbook from 1963. Though it is not a code, the value has basically fixed itself into the practical design methodologies and is accepted as a rule of thumb industry standard.

The allowable story drift for seismic is $0.02h_{sx}$. This is from the IBC 2003, governed by building type and framing.

Modeled

The model from ETABS showed very little drift (i.e. 1 e -4 values). This may be due to a user error in the program, or it may be due to the use of moment connections for every primary member to column. Therefore, in reference to the code, the model is up to code.

Calculated

The allowable total drift is h/400 or 3.15 inches. The allowable story drift for seismic can be found in the Figure 3 in the Appendix.

6.2 Strength Checks

Beam 1

This beam is the beam that takes a substantial load from wind and gravity. This beam is the support for the typical floor (floors 3-8). It is between column lines B and C or D and E, and is on column line 1 or 6. See Figure below. Note the orange arrow. In this instance, the beam on level 3 was analyzed.

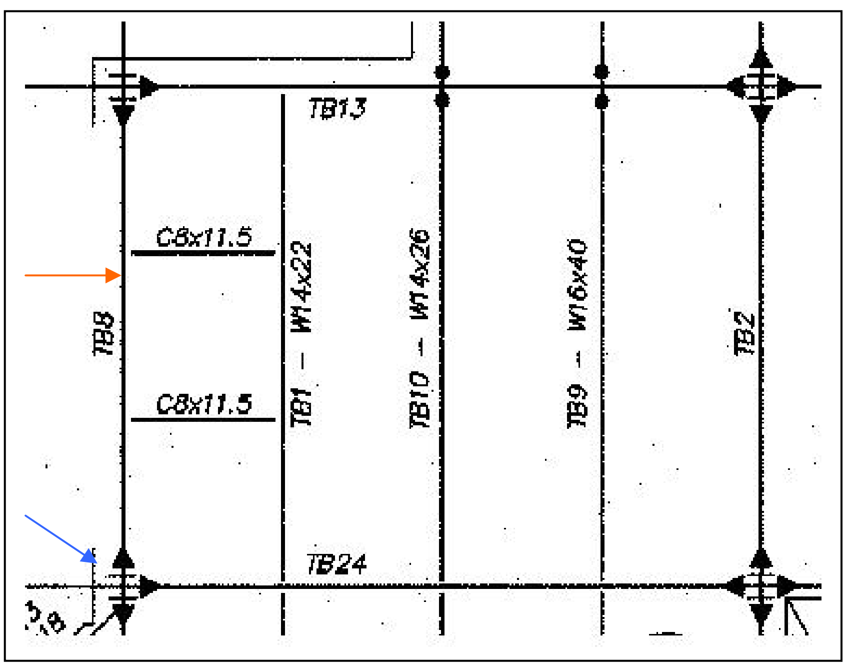
From the lateral distribution above, the beam must take these loads (To see the calculations, please review Figure 4 in the Appendix):

Vy = 3570 lbs
Vz = 562 lbs
Mz = 562’ lbs
My = 5200’ lbs
Total M = 5762’ lbs

Required I = 259 in4
Beam: W12x96 with I of 270 in4 and ΦMp = 551’kips, and 189 kips

The actual beam is a W30x99. From the ETABS Analysis, the typical beam holds 160’k and a shear of approximately 9 kips. From the steel manual, the beam can hold up to 1170’ kips with a shear of 417 kips.

I am not sure why the beam is so large, unless there are things that I did not take into account. Some things not taken into account could be forces from above floors, or a higher wind load than what I calculated.



Column 1

The column that is analyzed below is one that supports one side of the beam from the previous page (See Figure on previous page: Note the blue arrow). To see the calculations for this, you may refer to Figure 4 in the Appendix.

F = 562 lb
V = 3570 lb
M = 5762’ lb

Due to just the axial force, I chose a W12x58 which will hold 568 kips. However, the loads from the above floors were not taken into account. Therefore, a check from the model is necessary. The actual column is a W14x176. This column can hold a force up to 2000 kips. However, the actual force due to the ETABS model is approximately 260 kips with its worse moment case.

Once again, there is a discrepancy in my calculations to the computer model. This could be user error, or an error in my calculations.

7. Summary

From the analysis, I believe that the lateral system does its job very well. Due to the moment frames and possibly even the aerodynamic effects of the wall surfaces, the wind does not seem to affect the building, except on a very small scale. The seismic is also controlled well by the moment connections. I do believe that there may be some unforeseen problems with the ETABS model due to inexperience. However, I think it did a passing job for allowing the analysis of Vickroy Hall.

Appendix

Figure 1: Typical Architectural Floor Plan

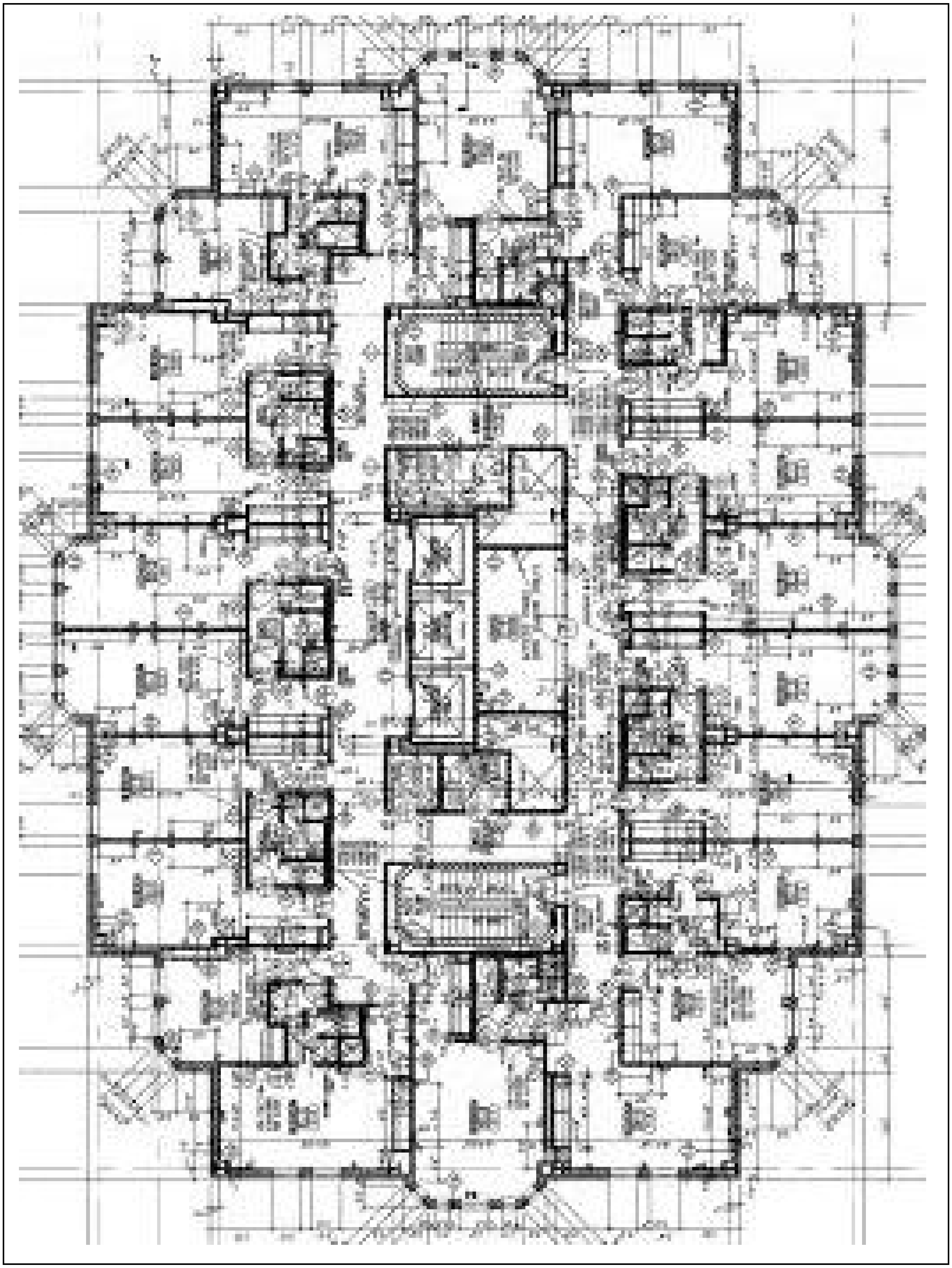


Figure 2: Typical Framing Plan

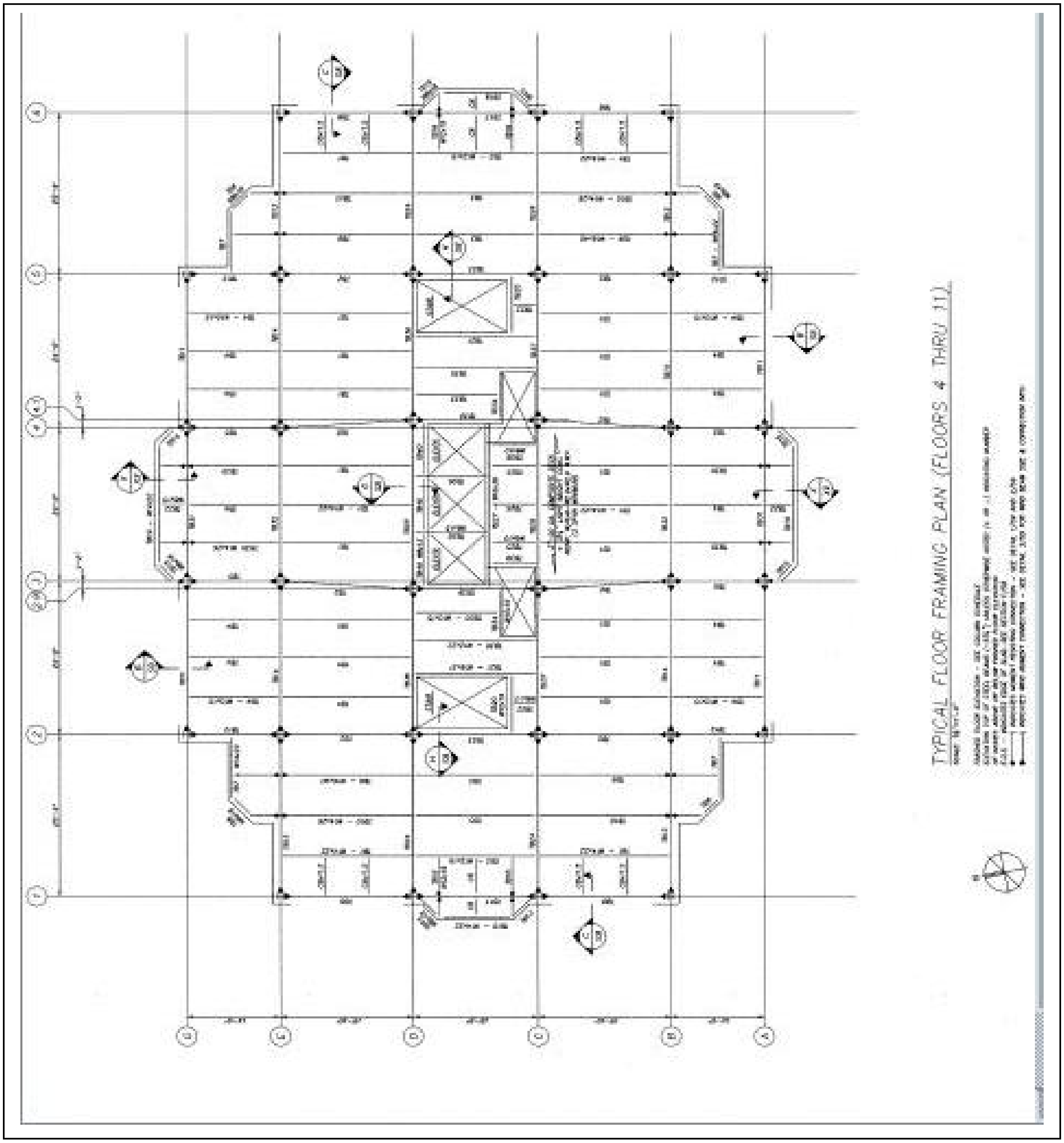


Figure 3: Seismic Calculations

Seismic Forces					
Story	hx^k*Wx	Cvx	Fx	Mx	Drift
Ground	0	0	0	0	0
Mezz	567	0.0000	0.00	0.30	0.09
1	353729.9	0.0062	2.06	163.33	0.31
2	1024383	0.0178	5.97	405.30	0.53
3	2057363	0.0358	12.00	678.05	0.76
4	3446905	0.0600	20.10	908.25	0.99
5	5193009	0.0904	30.29	1025.19	1.21
6	7295675	0.1270	42.55	958.21	1.44
7	9754903	0.1698	56.89	636.62	1.67
8	12570692	0.2188	73.31	-10.26	1.89
Roof	15743043	0.2741	91.82	-12.85	2.12
Total	57440269		335.00		

Wx (k) =	1388
Wx floor1 =	1411
Wx mezz =	28
k =	2.00
hfloor 1 (ft) =	15.33
h (ft) =	11.33
Hmezz (ft)	4.5
V (k) =	335

Figure 4: Lateral Distribution/Strength Calculations