New Acute Care Hospital and Skilled Nursing Facility

San Francisco, CA

Technical Report 3

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Executive Summary

The purpose of this report was to complete an in-depth analysis on the lateral system of the New Acute Care Hospital and Skilled Nursing Facility in San Francisco, CA. This was accomplished through a combination of methods including hand calculations, a 2D computer model, and a 3D computer model.

Before this analysis began in earnest, the seismic loads, which were found to be critical in *Technical Report I*, were reevaluated. The revised seismic analysis resulted in loads that, while still appeared to control, were lower than those originally calculated in *Technical Report I*.

This study found that lateral loads are transmitted through the structural primarily through a set of special steel moment frames. Torsional effects were analyzed, and it was found that each frame takes a percentage of load that is a function of both its stiffness, as well as its length. Shorter frames were shown to carry a lower percentage of load, whereas long frames take a greater percentage.

The report also included a study of the lateral loads and the combinations of loads that might control design in the structure. It was found that Wind Case II from ASCE7-05 would be the controlling wind load on the structure. In addition, it was confirmed that seismic loads would be the controlling lateral load. Load combinations including seismic loads were found to control over those without them.

Lastly, several checks were undertaken to insure that drift met industry standards, critical members were appropriately sized, and that overturning would not occur. It was found that while drift was properly controlled and the members were adequately sized, overturning would be an issue.

Table of Contents

Table of Figures

Introduction

The New Acute Care Hospital and Skilled Nursing Facility will serve as an addition to the existing Chinese Hospital located in the historic Chinatown district of San Francisco (See Fig. 1). The site lies on the north flank of Nob Hill, at an elevation of approximately 110’ above sea level. Due to the slope of the site, the ground floor of the site is located partially below grade.

Figure 1: Site View of New Acute Care Hospital (blue) located adjacent to existing Chinese Hospital. Photo Courtesy of Google Maps.

This new addition will be connected directly to the existing Chinese Hospital, located at 845 Jackson Street. As part of the construction of this addition, the original portion of the hospital built in 1925 will be demolished. Then the new facility, which has seven stories above ground and one below will be constructed with a hard connection to a previous addition built in 1975. Therefore, the precast concrete panel exterior façade has been designed in a way that respects the 1975 design while providing a more modern look.

At approximately 92,000 SF, this new facility will provide additional patient rooms as well as well several new medical departments to serve the local community. Construction is expected to begin in 2010 and reach completion by Chinese New Year 2013.

Figure 2: Exterior view of New Acute Care Hospital and surrounding buildings

Structure Overview

The structure of the New Acute Care hospital rests on a mat foundation and consists primarily of composite steel decking with steel framing. A perimeter moment frame system is used to resist lateral loading.

Foundation System

According to the geotechnical report provided by Treadwell & Rollo, the soil conditions on the site can be described as “very stiff to hard sandy clay and clay with gravel,” which rests on “intensely fractured, low hardness, weak, deeply weathered shale.” Because of this, the New Acute Care Facility has been designed to bear on a 36” mat foundation. Columns rest on concrete pedestals, typically sized at 3’-0” x 3’-0”. Since the base of the structure will lie below the water table, the foundation was also designed for hydrostatic uplift.

The close proximity to nearby structures, particularly the 1975 addition to the Chinese Hospital, provided a challenge to the designers. Underpinning was used to maintain the foundations of existing structures on either side of the building (see Figure 2).

Framing System

The New Acute Care Hospital uses steel columns (See Figure 3) to support the buildings gravity loads. These columns range in size from W14x445 near the base of the structure to W8x40’s near the roof level. As the columns rise vertically through the structure they are spliced together, usually at a distance of 22’-0”. Aside from those used in the lateral system, most of the columns are connected to beams and girders using pinned connections.

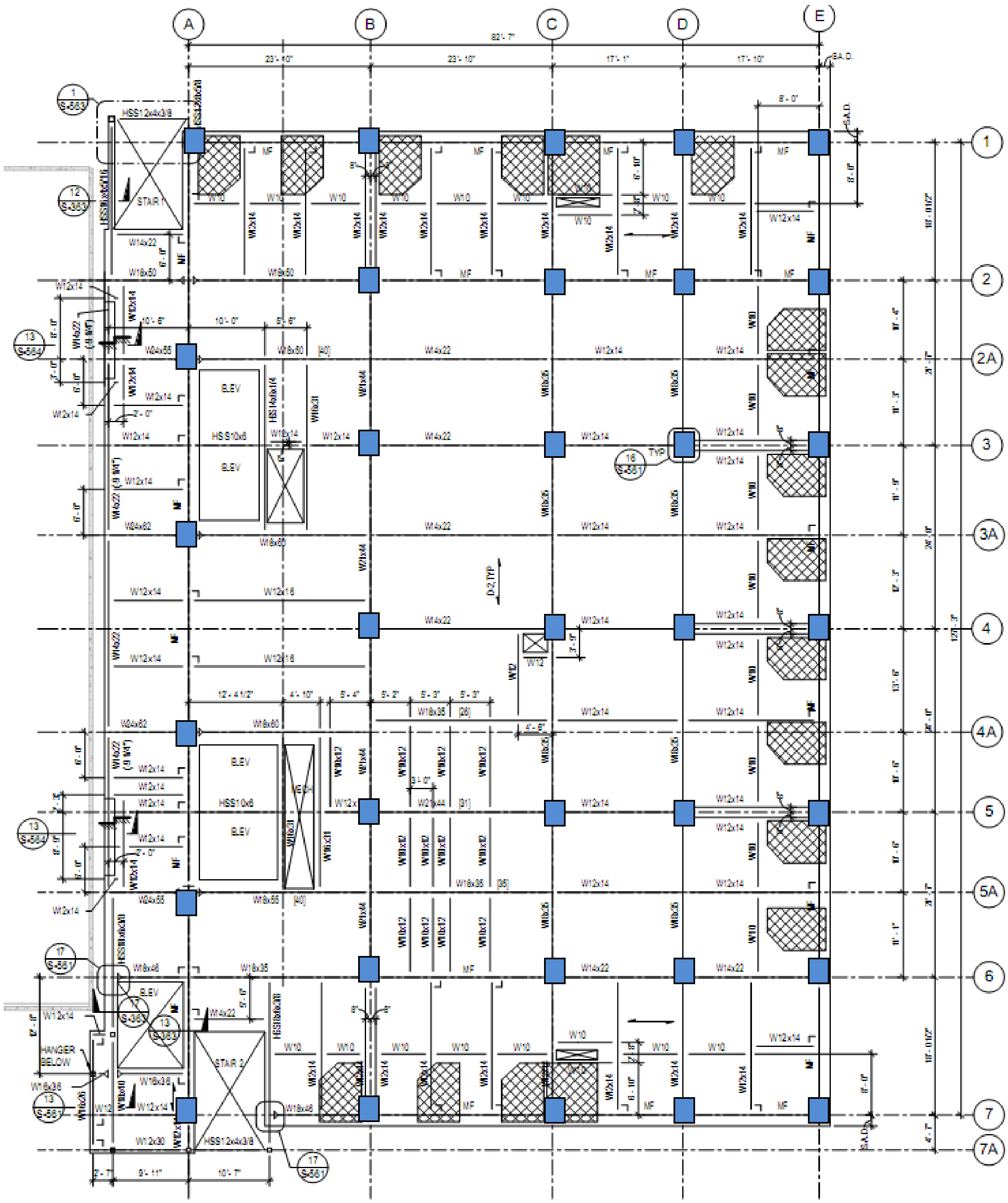


Figure 3: Typical Framing Plan with columns highlighted

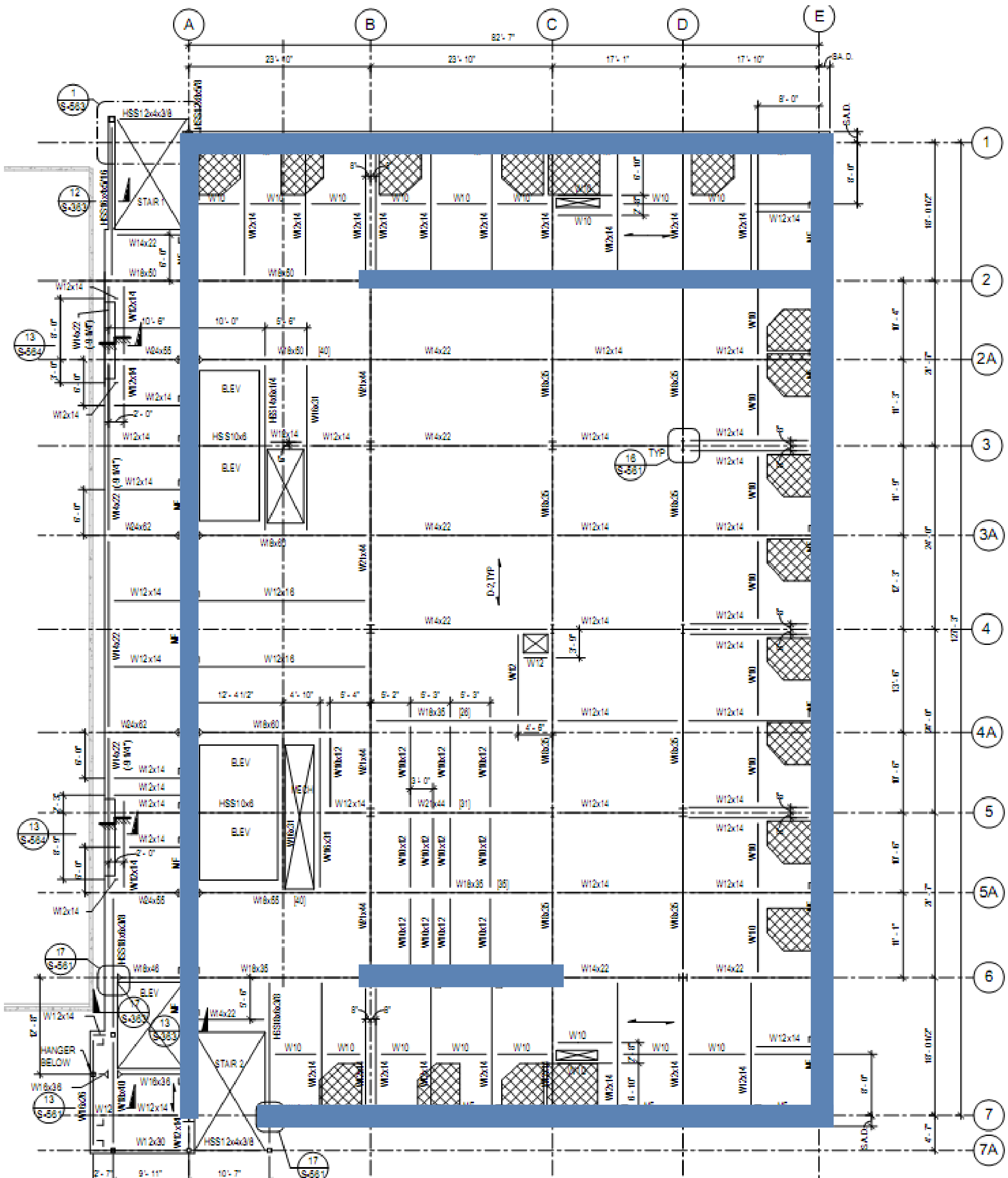


Figure 4: Typical Framing Plans with lateral system highlighted in blue

Lateral System

Lateral loads are transmitted through the structure primarily through the use of a series of special moment frames. There are 4 special moment frames running east-west, and 2 running north-south. One of the EW frames, located along gridline 2, terminates at the third floor level.

Since brittle failure of connections in moment frames tends to be a problem in regions of high seismic activity, the moment frame beams have been designed using Reduced Beam Sections (RBS). These RBS sections help to insure that yielding occurs in the reduced section of the beam rather than in the connection itself. See Figure 5 below.

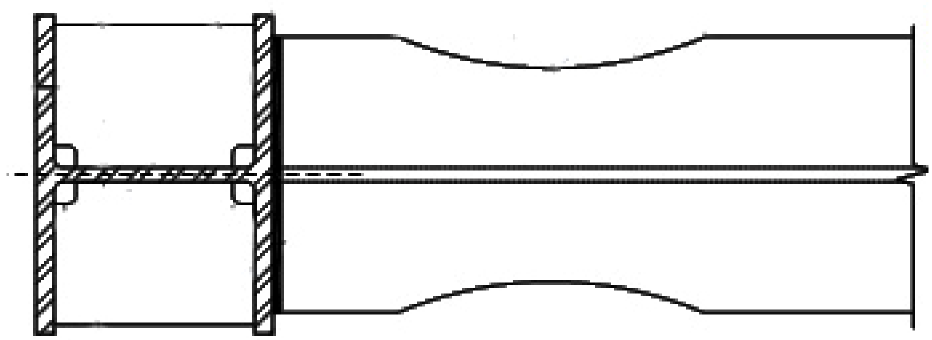


Figure 5: Reduced Beam Section

In addition to the steel moment frames, the basement walls also serve as shear walls for the basement level. These walls are constructed are 18" thick and composed of 4ksi concrete.

Roof System

The roof system is supported in a similar manner to the floors below, with a concrete filled metal deck supported by beams and girders. However, beams at this level are typically spaced much closer together, at a distance of approximately 10-12 feet. The sizes of these roof beams generally vary from W10x12's to W24x104's.

.Other Features

One of the unique structural features of the New Acute Care Hospital is its connection to the existing Chinese Hospital. The structures are connected with a seismic gap that allows the two structures to act independently. This size of this gap varies with story height so that a greater amount of movement is allowed at the upper floors.

A second unique feature of the New Acute Care Hospital is a result of the tight floor plan. There are several areas in which partition walls lie directly on beams. Since plumbing would normally be routed through these partition walls, a system of two, parallel beams spaced at 16” were used to create a gap for the plumbing system. See Figure 6 below.

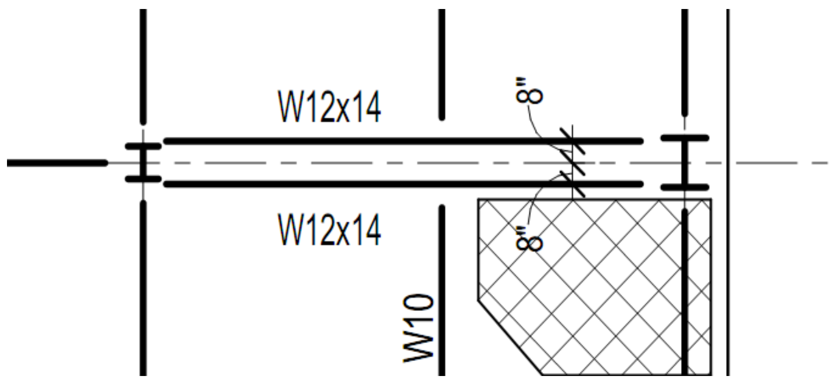


Figure 6: Parallel beams used for plumbing

Materials Used

Concrete

Location	Weight	Strength f'c (ksi)
Foundation	Normal	4000
Drilled Piers	Normal	4000
Slab-on-Grade Walls, Columns, and Piers	Normal	4000
Fill in Metal Deck and Curbs at Ground Floor	Normal	4500
Fill in Metal Deck at First Floor and Above, Topping Slab, Curbs, and Pads	Light	4000
Fill in Stair Pans	Normal	2500
Fill in Over-Excavated Areas and Conduit Encasement	Normal	1500

Structural Steel

Type	Standard	Grade
W-Shapes	ASTM A992	Grade 50
Other Shapes	ASTM A992	Grade 50
Plates for Built-Up Members	ASTM A572	Grade 50
Steel Channels, Angles, Base Plates, Shear Tabs	ASTM A36	Grade 36
Structural Steel Plates	ASTM A572	Grade 50
Steel Bars	ASTM A529	Grade 50
Square or Rectangular Steel Tubes	ASTM A500	Grade B
Round Steel Tubes	ASTM A500	Grade C
Pipe Sections	ASTM A53	Grade B

Reinforcing Steel

	ASTM A615	Grade 60

Applicable Codes

Original Design Codes Used

In addition to the following codes, the California State Government requires that all new government and hospital buildings are approved by the Office of Statewide Health Planning and Development (OSHPD).

- 2007 California Administration Code
    - Part 1, Title 24, CCR
- 2001 California Building Code
    - Part 2, Title 24, CCR
    - (1997 UBC and 2001 CA Amendments)
- 2004 California Electrical Code
    - Part 3, Title 24, CCR
    - (2002 NEC and 2004 CA Amendments)
- 2001 California Fire Code
    - Part 4, Title 24, CCR
    - (2000 UMC and 2001 Amendments)

Design Codes Used in Thesis Analysis

- American Society of Civil Engineers (ASCE)
    - ASCE7-05, Minimum Design Loads for Buildings and Other Structures
- International Building Code, 2006 Edition
- American Institute of Steel Construction (AISC)
    - Steel Construction Manual, Thirteenth Edition (LRFD)
- American Concrete Institute
    - ACI 318-08, Building Code Requirements for Structural Concrete

Design Loads

Gravity Loads

Live Load (psf)		
Live Load	As Designed	Per ASCE 7
Treatment Rooms	80*+20(partitions)	60
Patient Room	80*+20(partitions)	40
Other Rooms (offices)	80*+20(partitions)	50
Storage Areas		
Fixed Racks	125	125
Mobile Racks	250	250
Corridors	100	80
Mechanical Rooms	125	-
Roof (Mech)	125	100
Roof (Other)	20*	20

The designed live loads were found to be larger than the minimum live loads specified by ASCE7-05. It is likely that these values were higher based on the more stringent requirements of OSHPD as well as the experience of the designers.

Floor Dead Loads	
Material	(psf)
6 1/4" Concrete Deck	50
Finishes	1
MEP and Misc.	20
Total	71

Partition Wall Dead Loads (psf)	
Per ASCE7-05 12.7.2	10

Exterior Wall Dead Loads	
Material	(psf)
5" Concrete Panels	50
6" Metals Studs and Wallboard	0.38
6" Batt Insulation	0.9
Total	51.28

Roof Dead Loads	
Material	(psf)
80 Mil. TPO Roof Membrane	5.5
5/8" Dens Deck	2.5
6 1/4" Concrete Deck	60.4
Total	68.4

Dead load values were determined from a combination of sources including but not limited to ASCE7-05, design aids, and manufacturer specifications

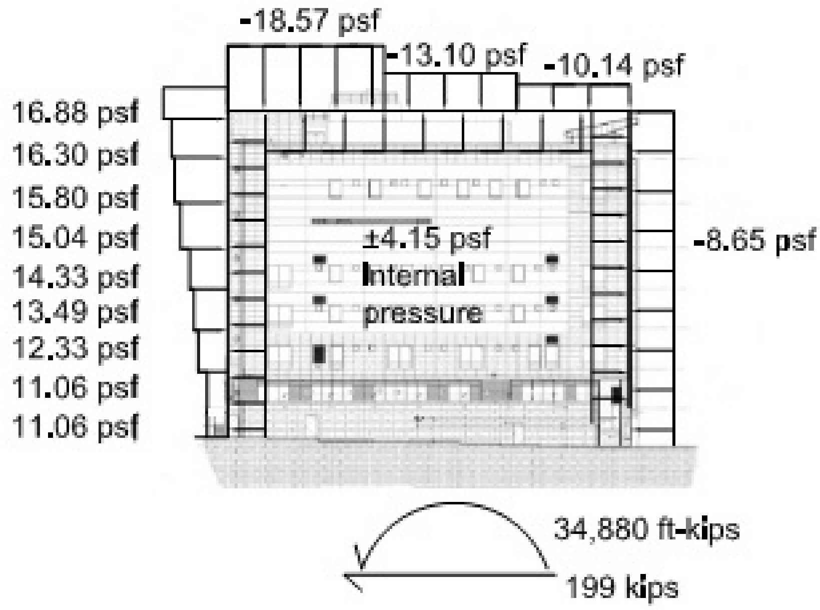
According to ASCE7-05 Figure 7-1, the ground snow load for San Francisco CA is 0 lb/ft^2. Therefore, the structure experiences no snow load.

Lateral Loads

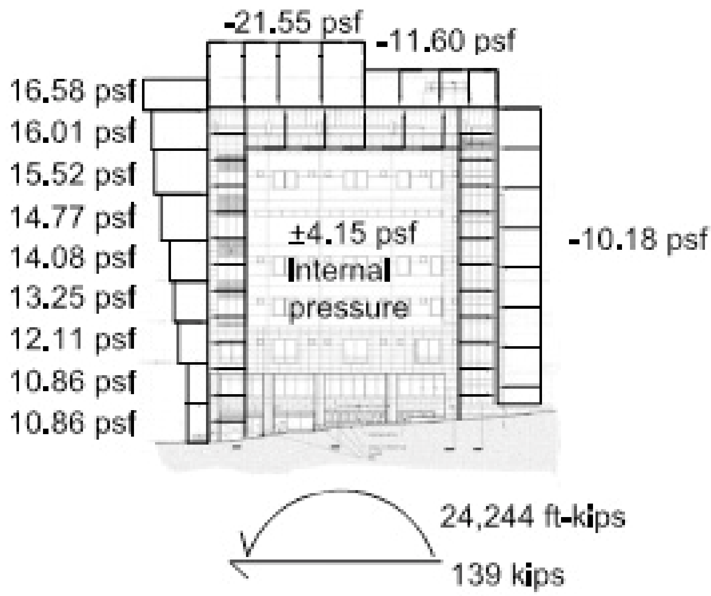
Wind Loads

Wind loads were calculated as prescribed by ASCE7-05 Chapter 6. Although the New Acute Care Facility is an addition to an existing structure, it was modeled as an independent structure for the purpose of this analysis. This simplification was appropriate in that it allows for the possibility of the existing Chinese Hospital structure being demolished at a later date.

Microsoft Excel was used extensively in both the analysis and determination of net wind pressures, story forces, and overturning moments. The net wind pressures comprised of pressure of the windward, leeward, side, and internal area of the building. A detailed summary of the analysis can be found in Appendix A. Once the net wind pressures were determined, the net wind loads were found. Wind loads were the largest in the NS direction resulting in a base shear of 199 kips and an overturning moment of 34,880 ft-kips (See Figure 4).



Wind Loads - NS Direction					
Floor Level	Floor Height (ft)	Elevation (ft)	Story Force (kips)	Total Story Shear (kips)	Overturning Moment (ft-k)
Ground	6.25	0	9.32	199.43	0
1	13	12.5	19.38	190.11	2376.43
2	13.5	26	22.44	170.74	4439.11
3	13.5	39.5	24.55	148.29	5857.46
4	13.5	53	26.09	123.74	6558.12
5	14.25	66.5	28.89	97.65	6493.54
6	15	81.5	31.95	68.76	5603.72
PH	16.75	96.5	36.81	36.81	3551.96
		Total Overturning Moment (ft-lbs)			34880.34
		Total Shear (lbs)			199.43



Wind Loads - EW Direction					
Floor Level	Floor Height (ft)	Elevation (ft)	Story Force (lbs)	Total Story Shear (lbs)	Overturning Moment (ft-lbs)
Ground	6.25	0	6.48	138.62	0
1	13	12.5	13.47	132.14	1651.76
2	13.5	26	15.60	118.67	3085.45
3	13.5	39.5	17.07	103.07	4071.29
4	13.5	53	18.13	86.01	4558.29
5	14.25	66.5	20.08	67.87	4513.40
6	15	81.5	22.21	47.79	3894.92
PH	16.75	96.5	25.58	25.58	2468.82
		Total Overturning Moment (ft-lbs)			24243.92
		Total Shear (lbs)			138.62

ASCE 7-05

Seismic Loads

The seismic loads evaluated in *Technical Report I* were reevaluated as a means of confirming the loads determined in the initial investigation. As before, the loads were calculated using the Equivalent Lateral Force method outlined in ASCE7-05 Chapter 12. Since a computer model was available at the time of this analysis, the fundamental period of the structure was compared with that calculated using the code ($T_a=C_t h_n^x$) which resulted in a period of 1.75sec. However, since the period determined using ETABS, 2.04 sec., was greater than the code specified period, the code specified value was still used.

Since the New Acute Care Hospital uses special moment frames in both directions, the code specified period, T_a is independent of direction for this structure. Therefore, a single analysis holds for both directions. For a detailed set of calculation procedures, see Appendix B: Seismic Calculations.

This revised analysis resulted in a both a lower base shear (897.6lbs vs. 1521.7lbs) and overturning moment (99.9 ft-k vs. 118.6 ft-k) in respect to those calculated in *Technical Report I*. This is due mainly to the presence of errors in the original calculations.

Seismic Loads

Level	Story Weight (lbs)	Story Height h_x (ft)	Modified h_x^k	$w_x h_x^k$	C_{vx}	Story Force (lbs) $F_x=C_{vx}V$	Story Shear (lbs) $V_x=\Sigma F_i$	Moment Contribution (ft-lbs) M_x	
7	2401.39	111.5	181.19	435102.59	0.27	341.14	0.00	38037.02	
6	1839.94	96.5	156.81	288525.42	0.18	226.22	341.14	21829.88	
5	1850.11	81.5	132.44	245024.45	0.15	192.11	567.36	15656.94	
4	1850.60	68	110.50	204491.25	0.13	160.33	759.47	10902.44	
3	1865.87	54.5	88.56	165246.07	0.10	129.56	919.80	7061.02	
2	1907.14	41	66.63	127062.98	0.08	99.62	1049.36	4084.54	
1	1881.67	27.5	44.69	84086.98	0.05	65.93	1148.98	1813.02	
Ground	1879.64	15	24.38	45816.22	0.03	35.92	1214.91	538.83	
Basement	0.00	0	0.00	0.00	0.00	0.00	1250.83	0	
				Overturning Moment $M=\Sigma M_x$ (ft-lbs)				99923.68	99.924 ft-k
				Effective Seismic Weight W (lbs)				15476.36	15.476 k
				Base Shear $V=C_sW$ (lbs)				897.63	0.898 k

Figure 7: Seismic Loads

Computer Models

Two independent computer models were used in this analysis. A 2D model was created using SAP for the purposes of determining drift based on unit loads on individual frames, while a 3D model was created using ETABs to determine effects of loads on the complete lateral system.

While these models had several differences, they were created using a number of similar attributes. In addition to the geometric and material based constraints of the structure, there were several aspects of the special moment frames that were incorporated into both models.

There are 3 major attributes of special moment frames that were modeled using each software package. First, panel zones were explicitly modeled at beam-column connections to account for the yielding and deformations that occur at these areas due to buildup of shear forces due to moment transfer. This is required by ASCE 7 §12.7.3b. Secondly, the reduced properties of the beam sections due to the RBS's had to be taken into account. This was accomplished by modeling the beams using the RBS connection type in ETABs and 90% of the section properties in SAP. Lastly, the columns were modeled as "pinned" connections in order to achieve a conservative approximation of the column base fixity.

In addition to these requirements, the concrete shear walls at the basement level were assigned a modification of 70% of the moment of inertia as specified by ACI 318.08 §10.10.4.1 and ASCE7-05 §12.7.3a. This effectively "cracks" the section giving a reduced strength.

A detailed account of other modeling assumptions can be found in Appendix C: Computer Modeling.

2D SAP model

The main purpose of the SAP model was to determine drifts in order to determine the relative stiffness of each frame or wall element. In order to accomplish this, a 1k load was applied to each frame in three iterations; first at the top of the basement level, then at the 3rd floor level and finally at the roof level. This was necessary due to the presence or lack thereof of each frame at different floor levels. The deflections were then measured at the level which the unit load was applied.

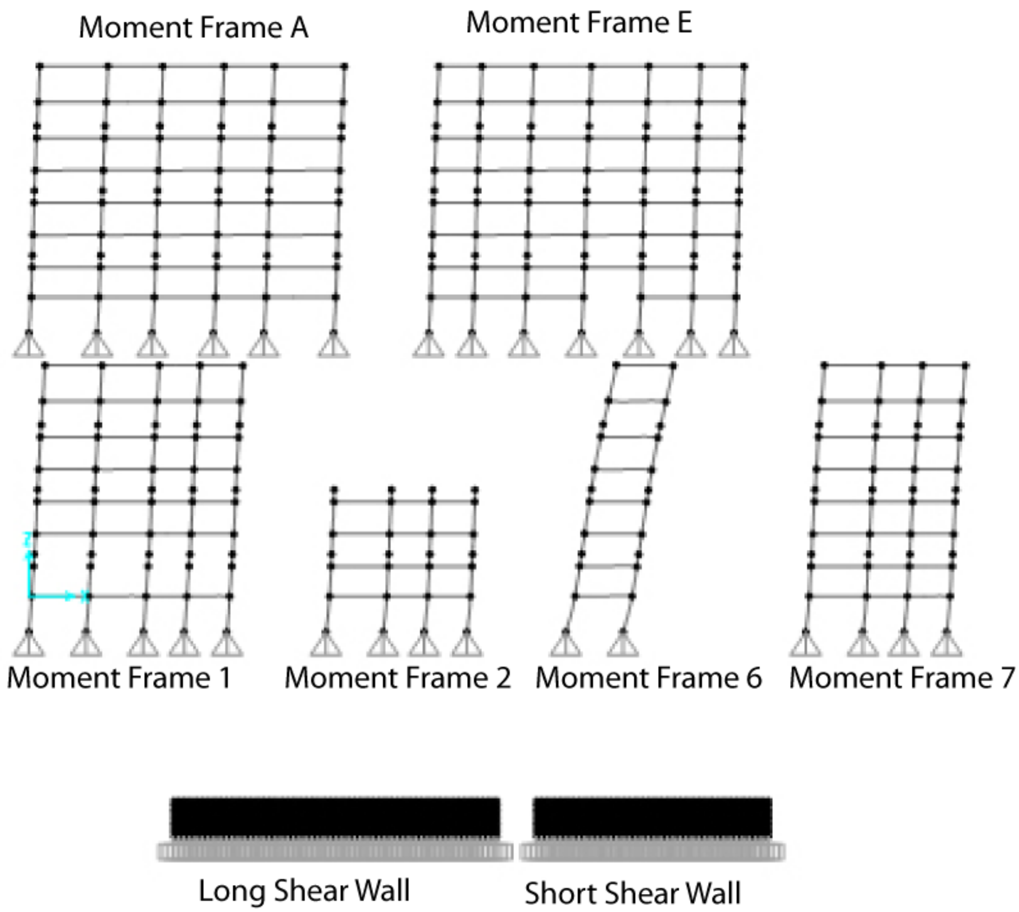


Figure 8: SAP frames under lateral loading

The deflections found using SAP were then compared with a set of hand calculations performed for the basement shear walls. These shear walls were treated as a cantilever section, and the total deflection was taken as the sum of the deflection due to flexure in addition to that due to shear.

$$\Delta_{TOTAL} = \Delta_{FLEXURE} + \Delta_{SHEAR}$$

This comparison showed that the deflections found by hand calculations were 11% higher than those found using SAP. See Appendix C: Computer Modeling for the deflection comparison calculations. This was deemed to be an acceptable difference, therefore the model deflections were used for the duration of the stiffness calculations.

3D ETABs Model

The main purpose of the ETABs model was to determine the effect of applied lateral loads on the complete lateral system. Each lateral element was modeled, and then connected as appropriate by rigid diaphragms at each floor level. Loads were then applied to the center of mass of each rigid diaphragm. Due to the simple rectangular plan of the structure as well as the uniform structural layout, the center of mass was taken to be the geometric center of each floor. The accuracy of this model was verified through the determination of the center of rigidity through hand calculations as well as a through the fundamental period of the structure.

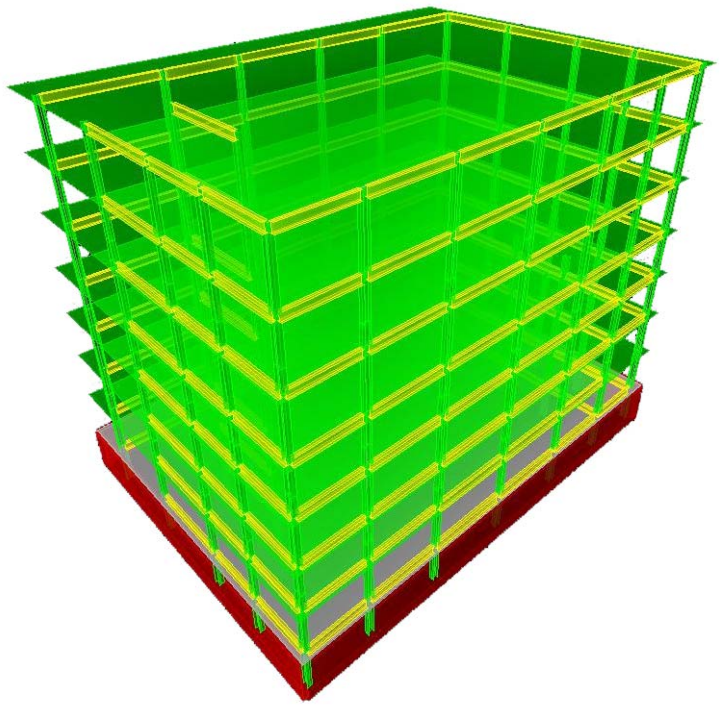


Figure 9: 3D Lateral System Model

The center of rigidity of each floor was determined using the relative stiff of each frame element. This stiffness was taken as the ratio of the applied load to horizontal displacement it causes.

$$k_i = \frac{p}{\Delta_p}$$

Once the stiffness of each element was found, the center of rigidity was found by dividing the sum of each elements stiffness times its location by the total stiffness in that direction.

$$\overline{X} = \frac{\sum k_{iy} x_i}{\sum k_{iy}} \qquad \overline{Y} = \frac{\sum k_{ix}\, y_i}{\sum k_{ix}}$$

This center of rigidity was then compared to that given by ETABs, which shows that both points lie relatively close to one another (See Figure 10, Figure 11 and Figure 12).

The other method with which the accuracy of the model was determined was the fundamental period of the structure. ETAB's modal analysis determined the 1st mode fundamental period of the structure in the x, y, and z directions.

1st Mode Period of Vibrations (secs)	
x	2.04
y	1.90
z	1.29

These values can be compared to the approximate fundamental period specified by ASCE7-05 §12.8.2.1.

$$T_a = C_t h_n^x$$

This frequency, which was found to be 1.75 sec, is a conservative approximation of the structures behavior given *only* the type of system used and its height. A more sophisticated analysis should generally give a smaller, more "accurate" frequency. Since the ETABs determined frequency is about 16% higher than the code determined frequency, it can be concluded that while the ETABs model will deliver results in the ballpark of the actual structures behavior, its results will likely be overly conservative.

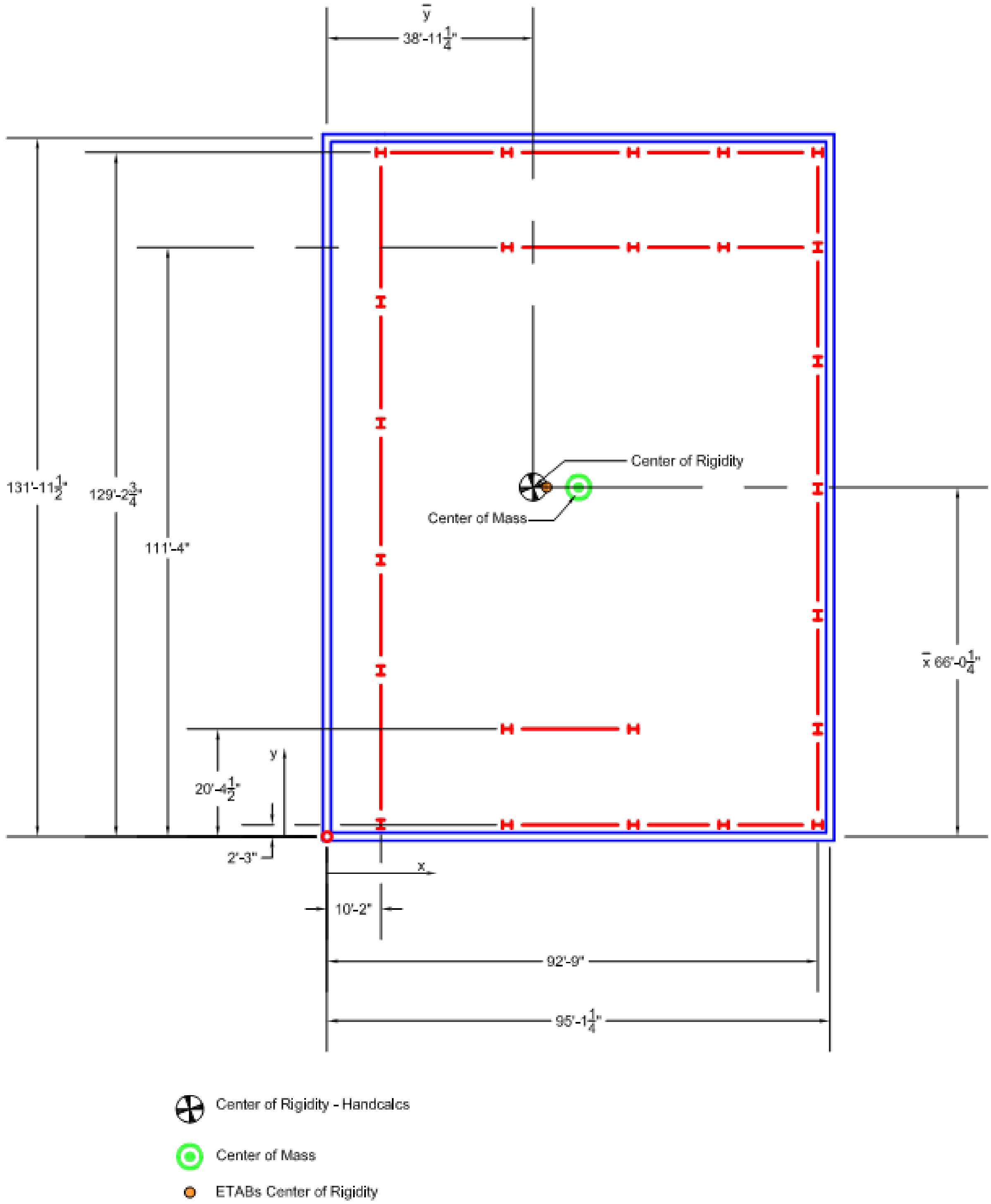


Figure 10: Center of Rigidity for Basement Level

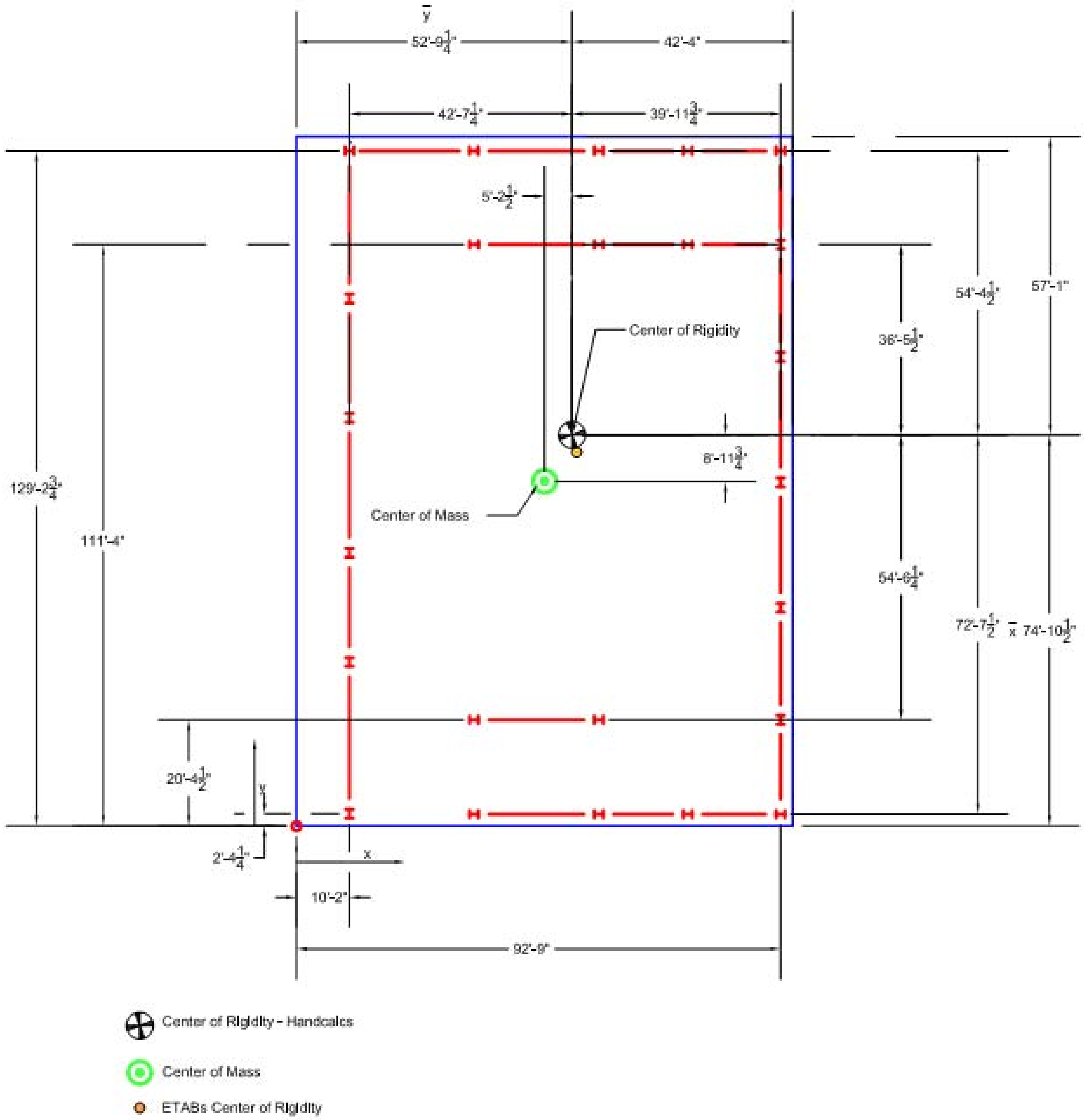


Figure 11: Center of Rigidity for Ground Floor-3rd Floor

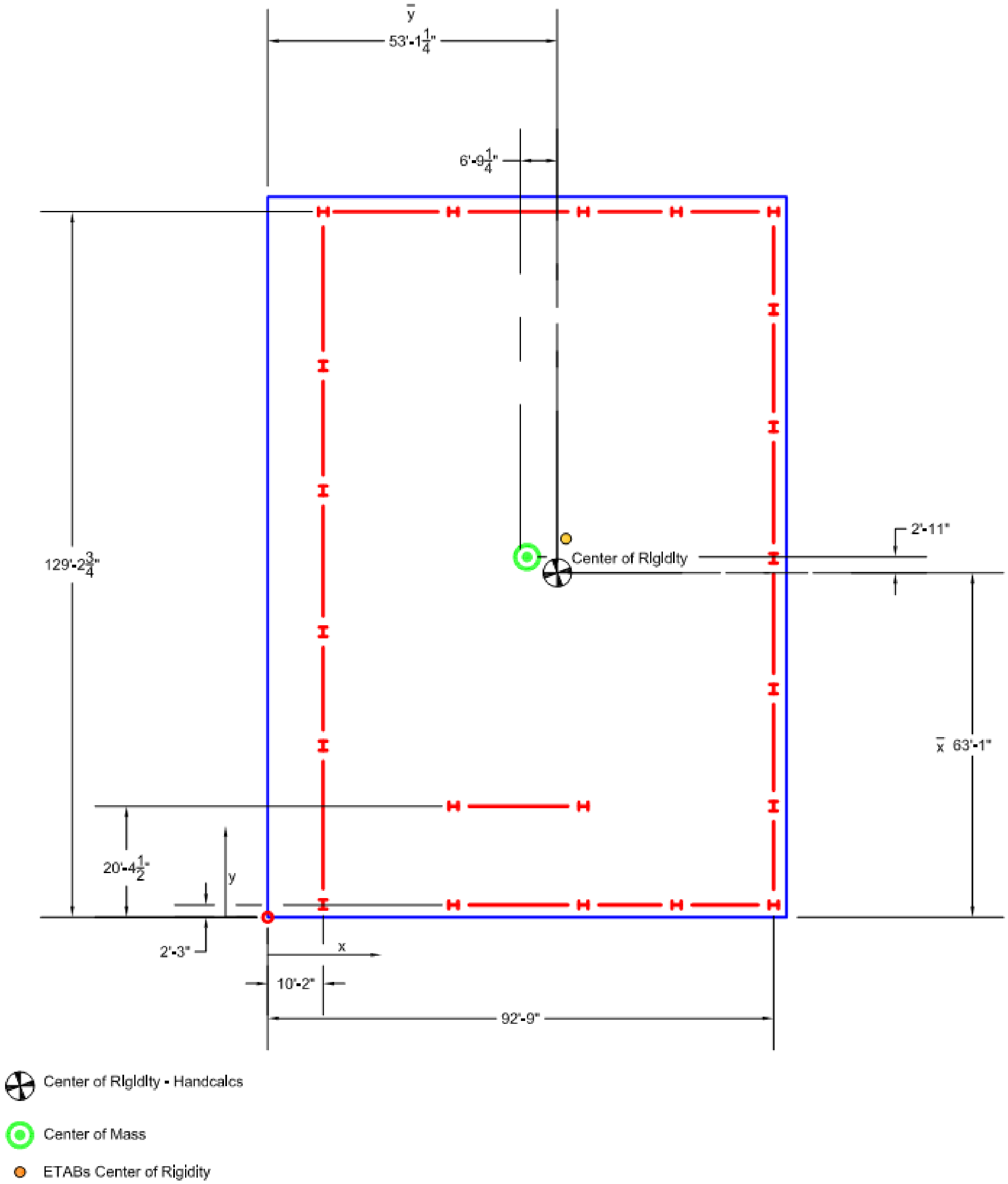


Figure 12: Center of Rigidity for Floor 4-Roof

Structure Behavior

Once the center of rigidity was found, a thorough analysis was undertaken to determine the behavior of the structure under lateral loading. This was accomplished by applying a unit load to the center of mass of each floor. The load path was determined by adding the force in each frame developed due to direct forces to the torsional forces developed due to eccentricity.

$$F_i = F_{i\,direct} + F_{i\,torsion}$$

Since these forces were determined using a unit load, they can easily be used to express the percentage of the lateral load that each frame element carries. As with stiffness, this analysis was performed in separate iterations for the basement, the ground floor through the third floor, and the fourth floor through the roof level. There are several interesting conclusions that can be drawn from this analysis. (See Appendix D: Structural Behavior for calculations).

$F_i = F_{idirect} \pm F_{itorsion}$ Basement

Moment Frame	$F_{idirect}$	$F_{itorsion}$	F_i	% Load	Check
Grid Line 1	0.001005	-0.02633	-0.02532	-2.53217	100.35
Grid Line 2	0.000292	-0.01153	-0.01124	-1.12354	
Grid Line 6	0.000699	0.007679	0.008378	0.83779	
Grid Line 7	0	0.024461	0.024461	2.446065	
Basement - short high	0.499002	-0.02948	0.469522	46.95222	
Basement - short low	0.499002	0.038668	0.537671	53.76705	
Grid Line A	0.000743	0	0.000743	0.074269	100.00
Grid Line E	0.000796	0	0.000796	0.079574	
Basement - long west	0.499231	0	0.499231	49.92308	
Basement - long east	0.499231	0	0.499231	49.92308	

At the basement level, the shear walls, which were also the stiffest elements by a large margin, absorbs the majority of the lateral load (nearly 50% per wall). It is interesting to note that there seems to be some shear reversal at this level in the frames along gridline 1 and gridline 2.

$F_i = F_{idirect} \pm F_{itorsion}$ Ground Floor - 3rd Floor					
Moment Frame	$F_{idirect}$	$F_{itorsion}$	F_i	% Load	Check
Grid Line 1	0.303399	0.04693	0.350329	35.03289	100.00
Grid Line 6	0.110249	-0.0171	0.09315	9.314996	
Grid Line 7	0.292078	-0.06034	0.231736	23.17355	
Grid Line 2	0.294274	0.03052	0.324794	32.47942	
Grid Line A	0.484076	0.063064	0.54714	54.71403	100.00
Grid Line E	0.515924	-0.06307	0.452852	45.2852	

Once the basement walls terminate at the ground floor level, the forces begin to distribute themselves differently. For the levels in between the ground floor and the 3rd floor, forces are absorbed nearly equally by the pairs of perimeter moment frames (1 and 7, A and E), while the interior moment frames (6 and 2) carry a smaller percentage of the load.

$F_i = F_{idirect} \pm F_{itorsion}$ 4th Floor - Roof					
Moment Frame	$F_{idirect}$	$F_{itorsion}$	F_i	% Load	Check
Grid Line 1	0.458388	0.018751	0.477139	47.71393	99.11
Grid Line 6	0.145529	-0.00597	0.13956	13.95597	
Grid Line 7	0.396083	-0.02164	0.374443	37.44426	
Grid Line A	0.48	0.081335	0.561335	56.13349	99.87
Grid Line E	0.52	-0.08268	0.437316	43.73162	

The final iteration of this analysis was performed for the 4th floors through the roof level, where the frame along grid 2 no longer exists. Like the lower portion of the structure, the perimeter frames again take the majority of the load.

Load Combinations

According to ASCE 7-05, there are four cases that must be considered when wind loads are being analyzed. Loading conditions were developed in ETABS to analyze each of these cases. The results of these conditions were compared, and Case II proved to be controlling wind condition for the structure, as it produced the highest drift.

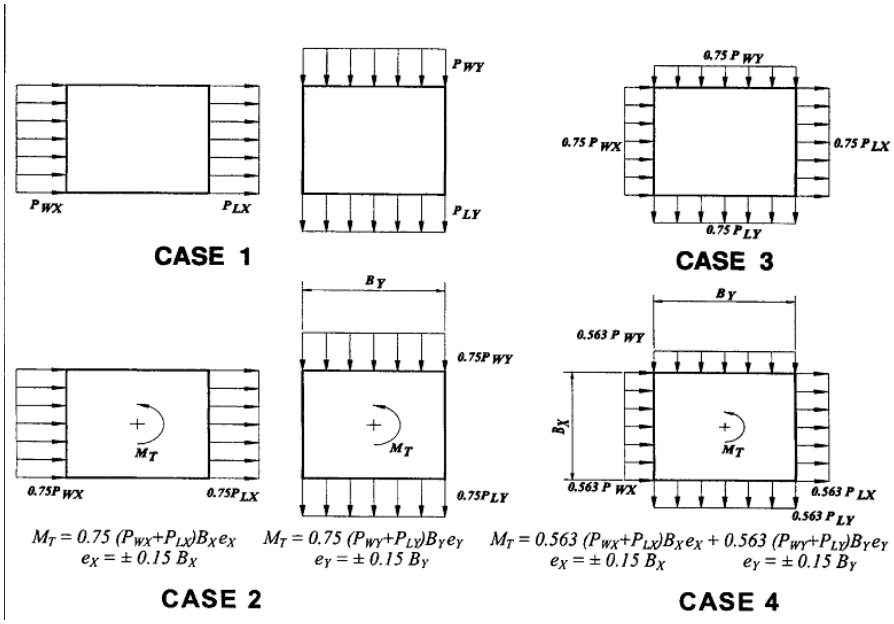


Figure 13: Wind Load Cases from ASCE7-05 Figure 6-9

Case 2 is described by ASCE7-05 as being "Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment…, considered separately for each principal axis. (See Figure 13 above). This condition was broken up into four individual conditions. The first condition had wind pressure in the EW direction with a positive eccentricity. The second condition was EW wind pressure with a negative eccentricity. Conditions three and four corresponded to conditions one and two but in the NS direction.

Once the controlling wind condition was determined, it was necessary to determine the controlling combination of loads. ASCE7-05 specifies that the following 7 load combinations that must be considered in the strength design of structures.

1. $1.4(D + F)$
2. $1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$
4. $1.2D + 1.6W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $\boxed{1.2D + 1.0E + L + 0.2S}$
6. $0.9D + 1.6W + 1.6H$
7. $0.9D + 1.0E + 1.6H$

Figure 14: ASCE 7-05 Load Combinations

For the analysis of the lateral system, the key load combinations are 4 and 5 for general loading, and 6 and 7 for uplift. It can be seen determination of the governing load case can be simplified to whether 1.6W+L is greater than 1.0E for the general loading conditions, and whether 1.6W is greater than 1.0E for uplift. Since seismic loads are greater than the wind loads by a large margin, cases 5 and 7 can be said to control strength design for general loading and uplift respectively. In addition, it is evident that the general loading combination for seismic will control strength design.

ETABs was used to confirm this assertion by comparing the story shears at the seventh floor for each load combination.

Story	Load	Loc	P	VX	VY	T	MX	MY
STORY7	COMB401	Bottom	163.05	-45.41	-62.08	-201736	130307.6	-93249.3
STORY7	COMB402	Bottom	163.05	-45.41	-62.08	215429.6	130307.6	-93249.3
STORY7	COMB501	Bottom	163.05	-341.14	-341.14	107629.7	180538.4	-146481
STORY7	COMB601	Bottom	122.28	-45.41	-62.08	-201736	100524.3	-71980.3
STORY7	COMB701	Bottom	122.28	-341.14	-341.14	107629.7	150755.1	-125212

Drift

The ETABs model was used to determine the maximum drifts for both wind and seismic forces. These forces were then compared with industry accepted values as well as the maximum allowable drift to prevent collision with the existing hospital.

Since deflections due to wind loads are a serviceability issue, they were analyzed using unfactored service loads. Only the four Case 2 conditions previously described were investigated since they were already shown to control. These values were then compared with the industry standards of H/400 and the more conservative H/600. In addition, these drifts were also checked against the constraints of the seismic joint as specified in the structural drawings. As Figure 15 below shows, the structure met all necessary criteria.

Maximum Drift - EW Wind - positive eccentricity (in)					
	height (h_x)	Δ_{actual} (ETABS)	$\Delta_{allowable}$ = L/400	$\Delta_{allowable}$ = H/600	Δ_{max} (Pounding)
Roof	1338	1.2288	3.345	2.23	-
Story 6	1158	1.1279	2.895	1.93	19.2
Story 5	978	0.9665	2.445	1.63	15.96
Story 4	798	0.7669	1.995	1.33	12.72
Story 3	636	0.5808	1.59	1.06	9.48
Story 2	474	0.3878	1.185	0.79	6.24
Story 1	312	0.174	0.78	0.52	3
Ground	150	0.0068	0.375	0.25	0

Maximum Drift - EW Wind - negative eccentricity (in)					
	height (h_x)	Δ_{actual} (ETABS)	$\Delta_{allowable}$ = 0.01h_x	$\Delta_{allowable}$ = H/600	Δ_{max} (Pounding)
Roof	1338	2.4731	13.38	2.23	-
Story 6	1158	2.242	11.58	1.93	19.2
Story 5	978	1.8814	9.78	1.63	15.96
Story 4	798	1.4456	7.98	1.33	12.72
Story 3	636	1.0718	6.36	1.06	9.48
Story 2	474	0.7404	4.74	0.79	6.24
Story 1	312	0.3411	3.12	0.52	3
Ground	150	0.0149	1.5	0.25	0

Maximum Drift - NS Wind - positive eccentricity (in)					
	height (h_x)	Δ_{actual} (ETABS)	$\Delta_{allowable}$ = 0.01h_x	$\Delta_{allowable}$ = H/600	Δ_{max} (Pounding)
Roof	1338	1.5948	13.38	2.23	
Story 6	1158	1.4607	11.58	1.93	
Story 5	978	1.2476	9.78	1.63	
Story 4	798	0.9868	7.98	1.33	
Story 3	636	0.7445	6.36	1.06	
Story 2	474	0.4952	4.74	0.79	
Story 1	312	0.2234	3.12	0.52	
Ground	150	0.0115	1.5	0.25	

Maximum Drift - NS Wind - negative eccentricity (in)					
	height (h_x)	Δ_{actual} (ETABS)	$\Delta_{allowable}$ = 0.01h_x	$\Delta_{allowable}$ = H/600	Δ_{max} (Pounding)
Roof	1338	1.9394	13.38	2.23	
Story 6	1158	1.7674	11.58	1.93	
Story 5	978	1.5021	9.78	1.63	
Story 4	798	1.1842	7.98	1.33	
Story 3	636	0.8891	6.36	1.06	
Story 2	474	0.5846	4.74	0.79	
Story 1	312	0.2613	3.12	0.52	
Ground	150	0.0119	1.5	0.25	

Figure 15: Drift Values for Wind Loads

While wind loads were primarily a serviceability issue, seismic loads are classified as a strength issue. Therefore, factored loads were used in this drift check (1.0). In addition, the drifts were compared to a maximum drift of $0.01h_x$, which is specified in ASCE7-05 Table 12.12-1. Like the wind drifts, the seismic drifts met all necessary deflection criteria (See Figure 16).

Maximum Drift - EW Seismic (in)				
	height (h_x)	Δ_{actual} (ETABS)	$\Delta_{allowable} = 0.01h_x$	Δ_{max} (Pounding)
Roof	1338	6.8097	13.38	-
Story 6	1158	6.0003	11.58	19.2
Story 5	978	4.9122	9.78	15.96
Story 4	798	3.6956	7.98	12.72
Story 3	636	2.6469	6.36	9.48
Story 2	474	1.6586	4.74	6.24
Story 1	312	0.7203	3.12	3
Ground	150	0.0436	1.5	0

Maximum Drift - NS Seismic (in)				
	height (h_x)	Δ_{actual} (ETABS)	$\Delta_{allowable} = 0.01h_x$	Δ_{max} (Pounding)
Roof	1338	5.7867	13.38	
Story 6	1158	5.168	11.58	
Story 5	978	4.2836	9.78	
Story 4	798	3.2913	7.98	
Story 3	636	2.4161	6.36	
Story 2	474	1.5444	4.74	
Story 1	312	0.6851	3.12	
Ground	150	0.0412	1.5	

Figure 16: Drifts values for Seismic Loads

Spot Checks

Several spot checks were completed in order to check the validity of member sizes as well as the implications of this analysis. The first two spot checks were on a typical girder and a typical column on moment frame 7. In order to determine the loads on these members, a portal analysis was undertaken using the fraction of the seismic loads taken by that moment frame. Only seismic loads were considered in the portal analysis since those loads were found to be the controlling lateral force for strength. Once the moments present in the members were determined, it was confirmed that the members would be able to carry them as well as any gravity loads that might be present on the members. Both the column and girder were found to be more than adequate to carry the required loads.

The next spot check performed was the effect of the lateral loads on the foundation system through overturning. For this analysis, moment frame 6 was selected since it has the shortest length, and will therefore prove to be critical for overturning forces. The uplift forces were determined in the frame, and it was found that the dead loads in the structure would not be sufficient to counteract it. This indicates that overturning will be an issue for this structure, which is a potential subject to look into in the upcoming proposal. See Appendix F for a complete set of calculations for these spot checks.

Conclusions

The three major conclusions that can be drawn from this analysis regard to controlling lateral loads, torsional effects, and modeling procedures. It was shown that seismic loads are critical for both strength and deflection. In addition, load combinations containing seismic will control the design of the lateral elements. Torsional effects were examined and it was found that torsional effects generally only made a small contribution (usually less than 5%) to the overall force in each frame element. Lastly, modeling practices were investigated using the ETABs and SAP software packages. The results of these models were compared to hand calculations and were shown to produce an adequate, although overly conservative level of accuracy.

APPENDIX

Appendix A: Wind Calculations

JACOBS

Subject: Wind Analysis | Project:
Sheet No. 1 Of 3
Authored by ______ Date ______ Checked by ______ Date ______

Use Method 2 since building is not a low-rise and meets req of 6.5.1 & 6.5.2

- Basic Wind Speed. ⇒ Using Table 6-1, V = 85 mph
- Wind Directionality Factor ⇒ $K_d = 0.85$ ⇒ Using Table 6-4 (Buildings)
- Occupancy Category = III (Health Care Facility) IBC 1604
  ↳ Importance Factor I = 1.15 (ASCE Table 6-1)
- Use Exposure C (Confirmed in Plans)
- Topographic Factor

  Nob Hill Elev = 315' ← Assume bottom of hill is sea level

  Building Elev. = 110'

  $\frac{110}{315} = 0.349$ ∴ Not top ½ of hill

  ∴ Use $K_{zt} = 1.0$

- Interpolate Table 6-3 to find K_h & K_z at building elevations: see Excel spreadsheet
- Velocity Pressures

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

  $q_h = 0.00256\, K_h K_{zt} K_d V^2 I$

  > See Excel for graphs

Gust Effect Factors

$n_1 = \frac{75}{H} = \frac{75}{115} = 0.652$ (Lower Bound ASCE 7-05 C6-17)

$n_1 = \frac{100}{H} = \frac{100}{115} = 0.869$ (Avg for n_1 ASCE 7-05 C6-18) ✱ Use for calcs

In either case $n_1 \leq 1$ Hz ∴ Structure is flexible

JACOBS

Subject: Wind Analysis | Project: | Sheet No. 2 Of 3

Authored by ______ Date ______ Checked by ______ Date ______

- $g_Q = g_v = 3.4$
- $g_R = \sqrt{2\ln(3600 n_1)} + \dfrac{0.577}{\sqrt{2\ln(3600 n_1)}}$
- $\bar{z} = 0.6h \geq z_{min}$
- $I_{\bar{z}} = c\left(\dfrac{33}{\bar{z}}\right)^{1/6}$
- $L_{\bar{z}} = \ell\left(\dfrac{\bar{z}}{33}\right)^{\bar{\epsilon}}$
- $Q = \sqrt{\dfrac{1}{1+0.63\left(\dfrac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$
- $\bar{V}_{\bar{z}} = \bar{b}\left(\dfrac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\dfrac{88}{60}\right)$
- $N_1 = \dfrac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}}$
- $R_n = \dfrac{7.47 N_1}{(1+10.3N_1)^{5/3}}$
- $R_h = \dfrac{1}{\eta} - \dfrac{1}{2\eta^2}\left(1-e^{-2\eta}\right)$
- $\eta = 4.6 n_1 h/\bar{V}_{\bar{z}}$
- $R_B = \dfrac{1}{\eta} - \dfrac{1}{2\eta^2}\left(1-e^{-2\eta}\right)$
- $\eta = 4.6 n_1 B/\bar{V}_{\bar{z}}$
- $R_L = \dfrac{1}{\eta} - \dfrac{1}{2\eta^2}\left(1-e^{-2\eta}\right)$
- $\eta = 15.4 n_1 L/\bar{V}_{\bar{z}}$
- $R = \sqrt{\dfrac{1}{\beta} R_n R_h R_B (0.53+0.47R_L)}$

$$G_f = 0.925\left(\frac{1+1.7 I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1+1.7 g_v I_{\bar{z}}}\right)$$

- SEE EXCEL SPREADSHEET FOR CALCS
- SELECT VALUES CAN BE FOUND IN ASCE7-05 TABLE 6-2.

APPROPRIATE VALUES WERE CALCULATED IN NS & EW DIRECTIONS.

JACOBS

Subject: Wind Analysis | Project:
Sheet No. 3 of 3
Authored by ___ Date ___ Checked by ___ Date ___

- Building is Fully Enclosed.
- Building Has a Parapet
  - Parapet Pressures

$P_p = q_p G C_{pn}$ → Determined using K_z & q_z spreadsheet

- Find External Pressure Coefficients C_p ⇒ See Excel Spreadsheet
  - Walls (use ASCE Fig. 6-6)
    - Interpolate L/B values to obtain C_p
  - Roof

0 → 0
h/2 → 52.875'
h = 105.75'
2h = 211.50'
Mean Roof Height

- Use Fig. 6-6
  - Ridge = 0°
  - Interpolate between 1st C_p values for each distance using h/L so for 0 → h/2

$$\frac{h}{L} \quad \frac{(0.78-0.5)}{(1-.5)} = \frac{(x - -.9)}{(-1.3 - -.9)}$$

- Use appropriate values for NS & EW directions.
- Use reduction factors as appropriate values interpolate

- Internal Pressure
  ±0.18 for enclosed structures. (Fig 6-5)

Designed Wind Pressures

Windward Walls $P_z = q_z G_f C_p - q_h (GC_{pi})$

Leeward Walls, Side Walls, Roofs: $P_h = q_h G_f C_p - q_h (GC_{pi})$

Velocity Pressure Coefficents K_z and Velocity Pressure q_z			
Floor Level	Height	K_z	q_z
Ground	0	0.850	15.368
1	12.5	0.850	15.368
2	26	0.948	17.140
3	39.5	1.037	18.749
4	53	1.102	19.924
5	66.5	1.156	20.900
6	81.5	1.215	21.958
Roof	96.5	1.253	22.654
Parapet	101.5	1.264	22.848
Penthouse	115	1.298	23.459

Gust Effect Factors G and G_f		
Term	NS Wind	EW Wind
n_1	0.86	
g_Q	3.40	
g_v	3.40	
g_R	4.15	
z_{MEAN}	63.45	
c	0.2	
I_{zMEAN}	0.179	
L_{zMEAN}	569.841	
Q	0.858	0.844
V_{zMEAN}	89.607	
N_1	5.469	
R_n	0.048	
η_h	4.669	
R_h	0.191	
η_B	4.212	5.953
R_B	0.209	0.154
η_L	19.928	14.099
R_L	0.049	0.068
β	0.010	
R	0.326	0.282
G_f	0.899	0.883

Wind Load Design Criteria	
Design Wind Speed	85 mph
Directionality Factor K_d	0.85
Importance Factor (I_w)	1.15
Exposure	C
Topographic Factor (k_{zt})	1
Mean Roof Height (h)	105.75 ft
K_h	1.27
q_h	23.04

Internal Pressure Coefficent GC_{pi}	
For Enclosed Buildings	0.18
	-0.18

External Pressure Coefficents		
Wind Direction	NS	EW
L/B	1.413	0.708
C_p (walls) windward	0.800	
C_p (walls) leeward	-0.417	-0.500
C_p (walls) sidewall	-0.700	
h/L	0.784	1.109
C_p (roof)		
0-h/2	-1.120	-1.300
h/2-h	-0.790	-0.700
h-2h	-0.612	-
>2h	-	-
Reduction Factor	0.800	0.800

Wind Loads - NS Direction							
Floor	Height Above Ground (ft)	Story Height (ft)	Wind Pressure (psf)	Internal Pressure (psf)		Net Pressure (psf)	
				(+)(Gc_{pi})	(-)(Gc_{pi})	(+)(Gc_{pi})	(-)(Gc_{pi})
Ground	0	12.5	6.91	4.15	-4.15	2.76	11.06
1	12.5	13.5	6.91	4.15	-4.15	2.76	11.06
2	26	13.5	8.18	4.15	-4.15	4.04	12.33
3	39.5	13.5	9.34	4.15	-4.15	5.19	13.49
4	53	13.5	10.19	4.15	-4.15	6.04	14.33
5	66.5	15	10.89	4.15	-4.15	6.74	15.04
6	81.5	15	11.65	4.15	-4.15	7.50	15.80
PH	96.5	18.5	12.15	4.15	-4.15	8.00	16.30
Parapet	101.5	5	12.29	4.15	-4.15	8.14	16.44
PH Roof	115	-	12.73	4.15	-4.15	8.58	16.88
Leeward	All	-	-12.79	4.15	-4.15	-16.94	-8.65
Side	All	-	-18.65	4.15	-4.15	-22.80	-14.50
Roof	0 to 52.875'	-	-22.71	4.15	-4.15	-26.86	-18.57
	52.875' to 105.75'	-	-17.24	4.15	-4.15	-21.39	-13.10
	105.75' to 134.83'	-	-14.29	4.15	-4.15	-18.44	-10.14

Wind Loads - EW Direction							
Floor	Height Above Ground (ft)	Story Height (ft)	Wind Pressure (psf)	Internal Pressure (psf)		Net Pressure (psf)	
				(+)(Gc_{pi})	(-)(Gc_{pi})	(+)(Gc_{pi})	(-)(Gc_{pi})
Ground	0	12.5	6.71	4.15	-4.15	2.57	10.86
1	12.5	13.5	6.71	4.15	-4.15	2.57	10.86
2	26	13.5	7.97	4.15	-4.15	3.82	12.11
3	39.5	13.5	9.10	4.15	-4.15	4.96	13.25
4	53	13.5	9.93	4.15	-4.15	5.79	14.08
5	66.5	15	10.62	4.15	-4.15	6.48	14.77
6	81.5	15	11.37	4.15	-4.15	7.22	15.52
PH	96.5	18.5	11.86	4.15	-4.15	7.72	16.01
Parapet	101.5	5	12.00	4.15	-4.15	7.85	16.15
PH Roof	115	-	12.43	4.15	-4.15	8.29	16.58
Leeward	All	-	-14.32	4.15	-4.15	-18.47	-10.18
Side	All	-	-4.15	4.15	-4.15	-8.29	0.00
Roof	0 to 52.875'	-	-25.696353	4.15	-4.15	-29.84	-21.55
	52.875' to 95.395'	-	-15.750632	4.15	-4.15	-19.90	-11.60
		-	-	4.15	-4.15	-	-

Appendix B: Seismic Calculations

EQUIVALENT LATERAL FORCE PROCEDURE

$S_S = 1.5 > 0.15$
$S_1 = 0.62 > 0.04$
→ DETERMINED USING USGS WEBSITE

PENTHOUSE FLOOR SF = 2265SF WHICH IS 18% OF A TYPICAL FLOOR. FLOORS < 20% TYPICAL FLOOR SF'S CAN BE CONSIDERED A NON SUBSTANTIAL FLOOR & DISREGARDED
∴ h = 111.5' = (FROM BASEMENT TO ROOF)

- A MAT FOUNDATION IS USED FOR SEISMIC DAMPING
- SITE CLASSIFIED AS SITE CLASS C ACCORDING TO GEOTEC REPORT.

USING TABLE 11.4-1
$F_a = 1.0$ FOR $S_S > 1.25$ | SITE CLASS C
∴ $S_{MS} = F_a S_S = \underline{1.5}$

USING TABLE 11.4-2
$F_v = 1.3$ FOR $S_1 \geq 0.5$ w/ SITE CLASS C
∴ $S_{M1} = F_v S_1 = \underline{0.806}$

$S_{DS} = 2S_{MS}/3 = 1$

$S_{D1} = 2S_{M1}/3 = \underline{0.537}$

SEISMIC DESIGN CATAGORY OCCUPANCY IS IV BASED ON IBC TABLE 1604.5 (HOSPITAL / HEALTH CARE FACILITY)

CONDITIONS FOR USE OF SIMPLIFIED DESIGN PROCEDURE

1) T_a (12.8.2.1) IN EACH DIRECTION $< 0.8\,T_s$ (11.4.5)

$T_a = C_t h_n^x$ (12.8-7)
$= (0.028)(111.5')^{0.8}$
$= 1.22$ SECS
OR
$T_a = 0.1N$ (12.8-8)
$= 0.1(7) = 0.7$ SECS
SINCE ALL STEEL MOMENT FRAMES & < 12 STORIES

$T_s = \dfrac{S_{D1}}{S_{DS}} = \dfrac{0.537}{1} = 0.537$

$T_a > 0.8T_s$ ∴ SIMPLIFIED DESIGN PROCEDURE MAY NOT BE USED.

T_a IS THE SAME IN EACH DIRECTION

THE SDC IS DETERMINED AS THE MORE SEVERE OF SDC'S GIVEN IN TABLES 11.6-1 & 11.6-2

11.6-1 ⇒ $S_{DS} = 1$ SO SDC D
11.6-2 ⇒ $S_{D1} = 0.537$ SO SDC D

$\boxed{\text{USE SEISMIC DESIGN CATAGORY D}}$

T WAS DETERMINED USING ETABS MODEL, GIVING

$T_x = 2.04$ SECS
$y = 1.89$ SECS
$z = 1.29$ SECS

HOWEVER THE UPPER LIMIT THAT MAY BE USED FOR T IS $C_u T_a = 1.75$
WHERE $C_u = 1.4$ (TABLE 12.8-1)

$\therefore$ USE $T_a = 1.75 = T$

THE HIGHER T_a VALUE WAS USED SINCE IT IS CLOSER TO THAT CALCULATED USING ETABS $\therefore$ CONSERVATIVE

$3.5T_s = 1.879$ SECS $> T = 1.75$ SECS

ACCORDING TO THE CRITERIA LISTED IN TABLE 12.3-1 (HORIZONTAL) & TABLE 12.3-2 (VERTICAL IRREGULARITY), THE STRUCTURE CAN BE CLASSIFIED AS REGULAR.

$\therefore$ $\boxed{\text{EQUIVALENT LATERAL FORCE METHOD CAN BE USED}}$

EQUIVALENT LATERAL FORCE METHOD

USING TABLE 12.2-1 FOR SPECIAL STEEL MOMENT FRAMES
$R = 8$ (RESPONSE MODIFICATION COEFFICIENT)
$\Omega_o = 3$ (SYSTEM OVERSTRENGTH FACTOR)
$C_d = 5\frac{1}{2}$ (DEFLECTION AMPLIFICATION FACTOR)

$\boxed{I = 1.5 \text{ USING TABLE 11.5.1}}$

$T_L = 12$ SEC FROM TABLE 22-16 (SAN FRANCISCO)

$T = 1.75 < T_L$

ACCORDING TO 12.8.1.1 C_s = SEISMIC RESPONSE COEFFICIENT

$$= \frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{1}{\left(\frac{8}{1.5}\right)} = 0.1875$$ WHICH MUST BE LESS THAN

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.537}{1.75\left(\frac{8}{1.5}\right)} = 0.058 \quad \text{(CONTROLS)}$$

ALSO, $S_1 = 0.62g > 0.6g$ SO

$$C_s \geq \frac{0.5S_1}{\left(\frac{R}{I}\right)} = \frac{0.5(0.62)}{\left(\frac{8}{1.5}\right)} = 0.058 \quad \underline{\text{GOOD}}$$

THE TOTAL SEISMIC WEIGHT, CALCULATED FOR THE ENTIRE STRUCTURE ACCORDING TO §12.7.2 IS

$W = 15,476.36^{\text{lbs}} \Rightarrow$ SAY $\underline{15.5^{\text{KIPS}}}$ INCLUDES PENTHOUSE WEIGHT

$\therefore$ SEISMIC BASE SHEAR $V = C_s W = 0.058(15.5) = \underline{0.897^{\text{KIPS}}}$

THE LATERAL SEISMIC FORCE INDUCED AT ANY LEVEL SHALL BE DETERMINED BY $F_x = C_{vx} V$ WHERE

$$C_{vx} = \frac{w_x h_x^k}{\sum w_i h_i^k}$$

$h_{i/x}$ = height of story from base

$$k \Rightarrow \frac{(1.75 - .5)}{(2.5 - .5)} = \frac{(k-1)}{(2-1)} \quad \text{SEE §12.8.3}$$

$k = 1.625$

SINCE T_a; WHICH IS INDEPENDENT OF FRAME DIRECTION WAS USED RATHER THAN T; THIS ANALYSIS HOLDS FOR BOTH THE NS & EW DIRECTIONS.

Appendix C: Computer Modeling

Relative Stiffnesses of Lateral Elements - Basement		
Moment Frame	Drift per 1k load (Δp) (in)	$k=P/\Delta p$ (k/in.)
Grid Line 1	0.0032	312.50
Grid Line 2	0.0049	204.08
Grid Line 6	0.0110	90.91
Grid Line 7	0.0046	217.39
Grid Line A	0.0030	333.33
Grid Line E	0.0028	357.14
Basement - long	0.0000	224064.53
Basement - short	0.0000	155231.29

Center of Rigidity - Basement				
X Direction	k_{iy}	x_i	$k_{iy}x_i$	
MF A	333.33	122	40666.667	
MF E	357.14	1113	397500	
W_A	224064.53	0	0	
W_B	155231.29	1141.25	177157715	
SUM	379986.30		177595882	
$x = \Sigma k_{iy}x_i/\Sigma k_{iy}$				467.3744
Y Direction	k_{iy}	x_i	$k_{iy}x_i$	
MF 1	312.50	1550.75	484609.38	
MF 2	204.08	1336	272653.06	
MF6	90.91	244.5	22227.273	
MF7	217.39	27	5869.5652	
W_1	0.00	1583.5	0	
W_2	0.00	0	0	
SUM	824.88		785359.27	
$y = \Sigma k_{iy}x_i/\Sigma k_{iy}$				952.0868

Relative Stiffnesses of Lateral Elements -Ground - 1st		
Moment Frame	Drift per 1^k load (Δ_p) (in)	$k=P/\Delta_p$ (k/in.)
Grid Line 1	0.0129	77.52
Grid Line 2	0.0133	75.19
Grid Line 6	0.0355	28.17
Grid Line 7	0.0134	74.63
Grid Line A	0.0081	123.46
Grid Line E	0.0076	131.58
Basement - long		
Basement - short		

Center of Rigidity - Floors Ground -3				
X Direction	k_{iy}	x_i	$k_{iy}x_i$	
MF A	123.46	122	15061.73	
MF E	131.58	1113	146447.4	
SUM	255.04		161509.1	
$x = \Sigma k_{ix}x_i/\Sigma k_{ix}$				633.2802548
Y Direction	k_{iy}	x_i	$k_{iy}x_i$	
MF 1	77.52	1550.75	120213.2	
MF 2	75.19	1336	100451.1	
MF6	28.17	244.5	6887.324	
MF7	74.63	27	2014.925	
SUM	255.50		229566.6	
$y = \Sigma k_{iy}x_i/\Sigma k_{iy}$				898.4878816

Relative Stiffnesses of Lateral Elements - Floors 4-7		
Moment Frame	Drift per 1k load (Δp) (in)	k=P/Δp (k/in.)
Grid Line 1	0.0267	37.45
Grid Line 6	0.0841	11.89
Grid Line 7	0.0309	32.36
Grid Line A	0.0182	54.95
Grid Line E	0.0168	59.52

Center of Rigidity - Floors 4 - Roof				
X Direction	k_{iy}	x_i	$k_{iy}x_i$	
MF A	54.95	122	6703.297	
MF E	59.52	1113	66250	
SUM	114.47		72953.3	
$x = \Sigma k_{ix}x_i/\Sigma k_{ix}$				637.32
Y Direction	k_{iy}	x_i	$k_{iy}x_i$	
MF 1	37.45	1550.75	58080.52	
MF6	11.89	244.5	2907.253	
MF7	32.36	27	873.7864	
SUM	81.71		61861.56	
$y = \Sigma k_{iy}x_i/\Sigma k_{iy}$				757.1216

LATERAL SYSTEM MODELING

- MATERIAL PROPERTIES (MEMBERS MODELED W/ 0 MASS TO CONTROL MASS LOCATION)
  - CONCRETE $f'_c = 4^{ksi}$ $E = 3,600,000 psi$ $\nu = 0.2$
  - STEEL $F_y = 50^{ksi}$ $E = 29,000,000 psi$ $\nu = 0.3$
- FRAME SECTIONS
  - SHAPE SECTIONS TAKEN FROM ETABS DATABASE & AISC13 PROPERTIES
- BASEMENT WALL SECTIONS
  - "MEMBRANE" ELEMENT → CONCRETE MATERIAL → MEMBRANE 18"
    BENDING 18"
    → WALL I_g "CRACKED" TO $0.7 I_g$ ACI 318.08 §10.10.4.1
- MODEL GEOMETRY.
  - FLOORPLAN SIMPLIFIED TO A RECTANGULAR SHAPE OF DIMENSIONS 82'-7" × 131'-11.5"
  - FLOORS MODELED
    BASEMENT (-15'-0")
    ROOF (96'-6")

    TOTAL HEIGHT (111'-6")

    PENTHOUSE SF = 2265 SF => 18% SF FOR TYPICAL FLOOR
    ∴ CAN BE IGNORED
  - COLUMN SPLICES LOCATED 5' ABOVE FLOORS 1, 3, & 5
  - COLUMN ORIENTATIONS ADJUSTED APPROPRIATELY.
  - INSERTION POINTS WERE DISREGARDED AT THIS STAGE OF ANALYSIS
  - JOINT RESTRAINTS @ BASE MODELED AS PINNED. THIS IS A CONSERVATIVE APPROXIMATION OF COLUMN/BASE FIXITY. SEE NEHRP pg 18.
  - BASEMENT WALLS MESHED W/ MAX SIZE = 24" & RESTRAINTS WERE MODELED ON EDGES.
- FLOOR DIAPHRAGMS
  - GROUND FLOOR DIAPHRAGM.
    - 6.25" CONCRETE MEMBRANE, MESHED @ 48"
    - HELPS TO ACCOUNT FOR SHEAR REVERSAL EFFECT.
  - OTHER FLOORS
    - RIGID DIAPHRAGM.
- EXPLICIT PANEL ZONE MODELING USED @ ALL BEAM-COLUMN CONNECTIONS TO ACCOUNT FOR "SOFTENING" OF THESE AREAS DUE TO LOCALIZED STRESSES (REQ'D BY ASCE 7, §12.7.3b)
  - MODELED USING ELASTIC PROPERTIES FROM COLUMN.
- BEAMS WERE MODELED AS REDUCED BEAM SECTION USING PROGRAM DEFAULTS FOR CUTOUTS.

- ADDITIONAL MASS PER FLOOR
  - $\frac{1879.64^{\text{kips}}}{12{,}325\ \text{SF}} = 0.1525^{\text{ksf}} \left(\frac{1}{32.2^{\text{ft/sec}^2}}\right)\left(\frac{1'}{(12''/1')^3}\right) = 2.74 \times 10^{-6}$
  - ADDITIONAL MASS ADDED TO DIAPHRAGMS.

- MOMENT FRAME ELEVATION 1
  - RBS MOMENT CONNECTIONS USED.
  - NO MOMENT CONNECTIONS @ 1ST FLOOR LEVEL

  PORTIONS OF THE BEAM FLANGE ARE SELECTIVELY TRIMMED IN REGION ADJACENT TO COLUMN BEAM CONNECTION TO ENSURE YIELDING & HINGE FORMATION w/in REDUCED AREA

- DETERMINATION OF Δ_p USING 2D MODELING
  - ~~IGNORE BASEMENT~~
  - USE EXPLICIT PANEL ZONE MODELING TO ACCOUNT FOR PANEL ZONE DEFORMATIONS (REQ. BY $\boxed{\text{ASCE 7 §12.7.3b}}$)
  - USE CONSTRAINTS TYPE "EQUAL" AT EACH LEVEL FOR X TRANS
  - MAT STRENGTHS
    - STEEL
    - CONCRETE
  - BASEMENT WALLS EXIST @ BASEMENT LEVEL ONLY
    - PORTIONS OF CONCRETE WALLS ON 1ST GROUND FLOOR IGNORED
    - MODELED AS "THICK SHELL" w/ MEMBRANE t = 18" BENDING t = 1.8"
    - "JOGS" IN WALL WERE SIMPLIFIED TO CREATE A RECTANGLE.

- ASSUME CENTER OF MASS LOCATED @ GEOMETRIC CENTER OF STRUCTURE

Appendix D: Structural Behavior

Direct Force - Basement			
Moment Frame	$k=P/\Delta_p$ (k/in.)	Σki (Basement)	$F_{iy}=(k_{iy}/\Sigma k_{jy})P_y$
Grid Line 1	312.50	311083	0.001004554
Grid Line 6	90.91		0.000292234
Grid Line 7	217.39		0.00069882
Grid Line 2	0.00		0
Basement - short (2)	155231.29		0.499002196
Grid Line A	333.33	448820	0.000742689
Grid Line E	357.14		0.000795738
Basement - long (2)	224064.53		0.499230786

Force due to eccentricity - Basement						
Moment Frame	$k=P/\Delta_p$ (k/in.)	d_i	$k_id_i^2$ (Basement)	e_i	$\Sigma k_jd_i^2$ (Basement)	$F_{it}=(k_id_iP_ye_x)/\Sigma k_jd_j^2$
Grid Line 1	312.50	652.50	133048828.13	104	801644776	0.026326245
Grid Line 2	204.08	437.50	39062500.00			0.011527639
Grid Line 6	90.91	654.25	38913005.68			0.007679084
Grid Line 7	217.39	871.50	165111358.70			0.024460647
Basement - short high	333.33	685.00	156408333.33			0.029480015
Basement - short low	333.33	898.5	269100750.00			0.038668312
Grid Line A	333.33	511.25	87125520.83	0	102477546981	0
Grid Line E	357.14	479.75	82200022.3			0
Basement - long west	155231.29	633.25	62248612620.3			0
Basement - long east	155231.29	508.00	40059608817.1			0

Direct Force Floors 1-3			
Moment Frame	$k=P/\Delta_p$ (k/in.)	Σki (Floors 1-3))	$F_{iy}=(k_{iy}/\Sigma k_{jy})P_y$
Grid Line 1	77.52	255.50	0.303398826
Grid Line 6	28.17		0.110249151
Grid Line 7	74.63		0.292077974
Grid Line 2	75.19		0.294274049
Grid Line A	123.46	255.04	0.484076433
Grid Line E	131.58		0.515923567

Force due to eccentricity - Floors 1-3

Moment Frame	$k=P/\Delta_p$ (k/in.)	d_i	$k_i d_i^2$ (Floors 1-3))	e_i	$\Sigma k_i d_i^2$ (Floors 1-3))	$F_{it}=(k_i d_i P_y e_x)/\Sigma k_j d_j^2$
Grid Line 1	77.52	652.50	33004360.47	107.75	116133377.5	0.046930051
Grid Line 6	28.17	654.25	12057551.06			0.017099192
Grid Line 7	74.63	871.50	56680018.66			0.060342433
Grid Line 2	75.19	437.50	14391447.37			0.030520148
Grid Line A	123.46	511.25	32268711.42	62.50	62552930.2	0.063063876
Grid Line E	131.58	479.75	30284218.75			0.063071586

Direct Force Floors 4-Roof

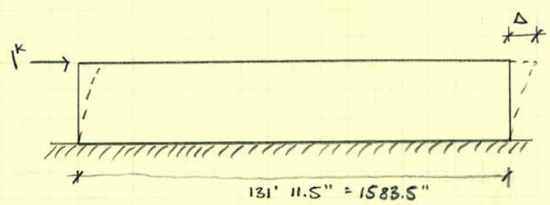
Moment Frame	$k=P/\Delta_p$ (k/in.)	Σki (4-Roof)	$F_{iy}=(k_{iy}/\Sigma k_{jy})P_y$
Grid Line 1	37.45	81.71	0.458388235
Grid Line 6	11.89		0.145528726
Grid Line 7	32.36		0.396083038
Grid Line A	54.95	114.47	0.48
Grid Line E	59.52		0.52

Force due to eccentricity - Floors 4-Roof

Moment Frame	$k=P/\Delta_p$ (k/in.)	d_i	$k_i d_i^2$ (Floors 4-7))	e_i	$\Sigma k_i d_i^2$ (Floors 4-7))	$F_{it}=(k_i d_i P_y e_x)/\Sigma k_j d_j^2$
Grid Line 1	37.45	652.50	15945926.97	35.00	45615303.0	0.018751099
Grid Line 6	11.89	654.25	5089691.59			0.00596905
Grid Line 7	32.36	871.50	24579684.47			0.021640455
Grid Line A	54.95	511.25	14361349.59	81.25	28061353.3	0.081334854
Grid Line E	59.52	479.75	13700003.72			0.082683806

TECH III

SHEAR WALL DEFLECTION CHECK.



$\Delta = \Delta_{FLEXURE} + \Delta_{SHEAR}$

$= \frac{Ph^3}{3EI} + \frac{2.78Ph}{AE}$

$= \frac{(1^k)(15' \times 12''/')^3}{3(3,600)(5.96\times10^9)} + \frac{2.78(1^k)(15\times12)}{(28503)(3,600)}$

$= 9.06\times10^{-8}\,in + 4.87\times10^{-6}\,in$

$= 4.97\times10^{-6}\,in$

$I = \frac{tl^3}{12} = \frac{(18'')(1583.5'')^3}{12}$

$= 5.96\times10^9\,in^4$

$A = lt$

$= (1583.5)(18)$

$= 28503\,in^2$

$E = 3,600^{ksi}$

COMPARE w/ 4.46×10^{-6} in USING SAP.

$\frac{[(4.97\times10^{-6})-(4.46\times10^{-6})]}{(4.46\times10^{-6})}(100\%) = 11\%$ ACCEPTABLE.

SAMPLE CALCS FOR STRUCTURAL BEHAVIOR.

FLOORS 4 - ROOF ⇒ NS DIRECTION.

• DIRECT FORCE $\Sigma k_x = 114.47\ ^{k/in}$ $\Sigma k_y = 448{,}820\ ^{k}/in$

$F_{AD} = \dfrac{k_A}{\Sigma k} P$

$= \dfrac{54.95}{114.47}(1^k)$

$= 0.48^k$ OR 48% OF THE DIRECT FORCE

$F_{ED} = \dfrac{59.52}{114.47}(1^k)$

$= 0.52^k$ OR 52% OF THE DIRECT FORCE

• FORCE DUE TO ECCENTRICITY $e = 81.25''$

$F_{AT} = \dfrac{k_A d_i P_y e_x}{\Sigma k_j d_j^2}$

$= \dfrac{(54.95)(511.25)(1^k)(81.25)}{28061353.3}$

$= 0.0813^k\ (+)$

$F_{ET} = \dfrac{59.52(479.75)(1^k)(81.25)}{28061353.3}$

$= 0.0827^k\ (-)$

TOTAL FORCE

$F_A = F_{AD} + F_{AT}$

$= 0.48^k + 0.0813^k$

$= 0.5613^k$

OR

56% OF THE LATERAL LOAD

$F_E = F_{ED} + F_{ET}$

$= 0.52 - 0.827$

$= 0.437^k$

OR

44% OF THE LATERAL LOAD.

Appendix E: Load Combinations

LOAD COMBINATIONS 1 OF

D = SELF-WEIGHT & SUPERIMPOSED DEAD LOAD, PER FLOOR LEVEL

AVG FLOOR SF

$= 71\text{PSF}(12325\text{ SF}) = 875{,}075\text{ lbs}$ (FLOOR)

$+ 51.28\text{PSF}(460.45\text{ FT})(1\text{ FT}) = 23612\text{ lbs}$ (EXTERIOR WALLS)

PERIMETER THICKNESS

$+ 10\text{ PSF}(12325\text{ SF}) = 123{,}250\text{ lbs}$ (PARTITION WALLS)

$+ 105{,}627\text{ lbs}$ (BEAM WEIGHT)

$+ 143{,}012.5$ (COLUMN WEIGHT)

$1.27\times10^{-6}\text{ lbs}/12325\text{ SF} = \boxed{103\text{ PSF DEAD LOAD}}$

L = LIVE LOAD = L_R

= SAY $\boxed{125\text{ PSF}}$ CRITICAL VALUE USED IN STORAGE AREAS & ROOF

W = WIND LOAD REF ASCE 7-05 FIGURE 6-9; SEE EXCEL FOR VALUES

NET PRESSURE = (WINDWARD/LEEWARD) + INTERNAL PRESSURE

CASE I => COMPLETE LOAD IN EACH DIRECTION, CONSIDERED SEPERATELY

$\boxed{\text{CASE II} \Rightarrow}$ $P_{wx\,eff} = 0.75P_{wx}$ $\quad P_{Lx\,eff} = 0.75P_{Lx}$ EACH DIRECTION ACTS SEPERATELY

$P_{wy\,eff} = 0.75P_{wy}$ $\quad P_{Ly\,eff} = 0.75P_{Ly}$

PRODUCES MAX DISPLACEMENT OF $U_x = 2.71$, $U_y = 1.71$ @ 7TH FLOOR

$M_T = 0.75(P_w + P_L)B_x e_x$

CASE III => $P_{wx\,eff} = 0.75P_{wx}$

$P_{wy\,eff} = 0.75P_{wy}$ EACH DIRECTION ACTS SIMULTANEOUSLY

CASE IV => $P_{wx\,eff} = 0.75(0.75)P_{wx}$ $\quad P_{ex\,eff} = 0.75(0.75)P_{ex}$

$P_{wy\,eff} = 0.75(0.75)P_{wx}$ $\quad P_{ey\,eff} = 0.75(0.75)P_{ey}$

$M_T = 0.75(0.75)(P_{wx}+P_{Lx})B_y e_y + 0.75(0.75)(P_{wy}+P_{ey})B_y e_y$

S = SEISMIC LOAD → SEE SEISMIC CALC; LOADS ARE THE SAME FOR BOTH NS & EW DIRECTIONS.

CRITICAL LOAD COMBINATIONS ⇒ FROM ASCE 7-05

4) $1.2D + 1.6W + 0.5L_R$

P: $1.2(103) + 1.6(38.8) + 0.5(125) = 248.18\text{ lbs}$

M: $1.6(52{,}963.8) = 84{,}742'\text{ lbs}$

5) $1.2D + 1.0E + L$

$P = 1.2(103) + 1.0(341.14) + 125 = 589.7\text{ lbs}$

6) $0.9D + 1.6W$

$0.9(103) + 1.6(38.8) = 154.78\text{ lbs}$

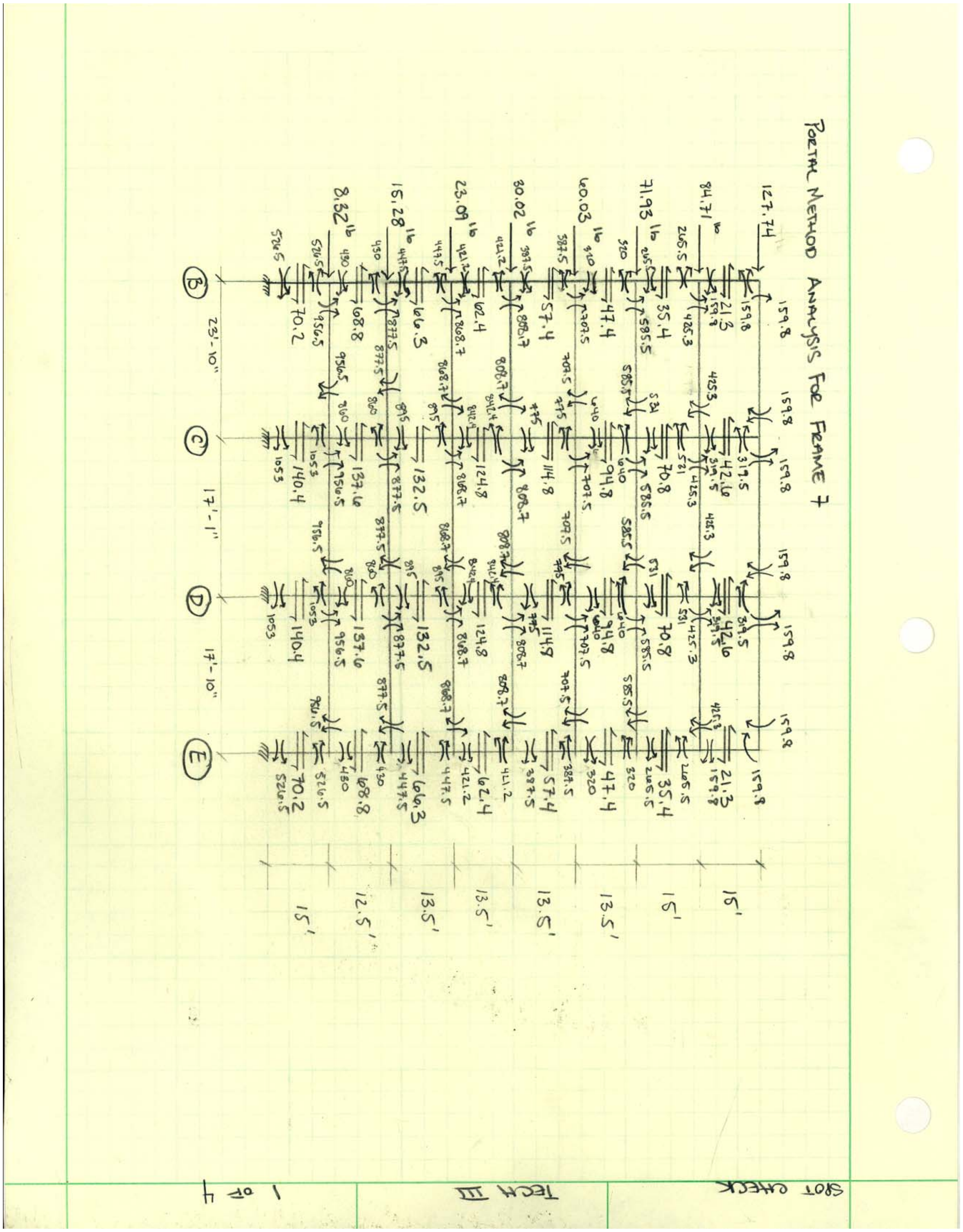
7) $0.9D + 1.0E$

$0.9(103) + 1.6(341.14) = 433.84\text{ lbs}$

Appendix F: Spot Checks

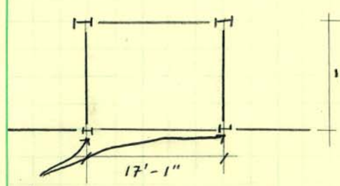
Forces on Frame 7			
	Seismic Loa	Factor	F (K)
7	341.14	0.37	127.74
6	226.22	0.37	84.71
5	192.11	0.37	71.93
4	160.33	0.37	60.03
3	129.56	0.23	30.02
2	99.62	0.23	23.09
1	65.93	0.23	15.28
Ground	35.92	0.23	8.32
Basement	0.00	0.02	0.00

Forces on Frame 6			
	Seismic Load	Factor	F (k)
7	341.14	0.14	47.61
6	226.22	0.14	31.57
5	192.11	0.14	26.81
4	160.33	0.14	22.38
3	129.56	0.09	12.07
2	99.62	0.09	9.28
1	65.93	0.09	6.14
Ground	35.92	0.09	3.35
Basement	0.00	0.01	0.00


PORTAL METHOD ANALYSIS FOR FRAME 7
SPOT CHECK
TECH III
1 OF 4

CHECK GROUND FLOOR GIRDER BETWEEN C & D.

W24 × 207



W14×426

CONTROLLING LOAD COMBINATION

$1.2(D) + 1.0(E) + 1.0(L) = 1.2(103) + 1.0(40)$
$= 127.6 \text{ PSF}$

TRIB WIDTH = $\frac{18.0417}{2} = 9.02'$

$\omega_u = 127.6(9.02) = 1151 \text{ PLF} = 1.15 \text{ KLF}$

FROM PORTAL ANALYSIS

$M_E = 956.5^{'K}$

MOMENT FROM DL + LL.

$\omega_{LL} = 40 \text{ PSF}$ (PATIENT ROOMS)

$\omega_{DL} = 103 \text{ PSF}$ (SEE LOAD COMBO CALCS)

$$l_n = 17.083' - \left(\frac{14''}{2(12''/1')}\right) = 16.5'$$

USING MOMENT COEFFICENT METHOD OF ACI 318.08 8.3.3

- NEG @ INTERIOR FACE OF SUPPORTS

$$\frac{\omega_u l_n^2}{11} = \frac{(1.15)(16.5)^2}{11} = 28.5^{'K}$$

$$\therefore M_{TOTAL} = M_{D+L} + M_E = 28.5 + 956.5 = 985^{'K}$$

USING STEEL MANUAL TABLE 3-2

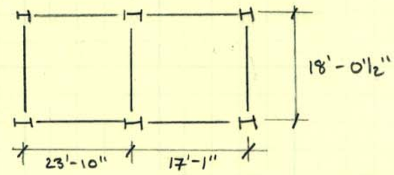
$\phi M_p = 2270^{'K} >> 985^{'K}$

CHECK SERVICEABILITY

$$\Delta_{TOTAL\ LOAD} = \frac{5 \omega l^4}{384 EI} = \frac{5(1.15)(17.083)^4\, 1728}{384(29{,}000)(6820)} = 0.011''$$

GOOD BY INSPECTION

CHECK COLUMN C @ BASEMENT LEVEL W14 x 426



- FROM PORTAL ANALYSIS $M_E = 1053'^k$
- FIND P_U FROM DL & LL ON FLOOR

$$A_T = \left(\frac{18.0417'}{2}\right)\left(\frac{23.83 + 17.083}{2}\right)(7 \text{ FLOORS ABOVE})$$

$$= 1291.75 \text{ FT}^2$$

$$A_I = 4(1291.75) = 5166.98 \text{ FT}^2$$

$$L_{REDUCED} = 40\left[0.25 + \frac{15}{\sqrt{5166.98}}\right] = 18.4$$

$$0.4L = 0.4(40) = 16 < 18.4 \quad \therefore \text{ USE } 18.4 \text{ PSF}$$

$$P_{LL} = 5166.98 \text{ FT}^2 (18.4 \text{ PSF}) = 94.8^k$$

$$P_{DL} = 103 \text{ PSF}(1291.75) = 133.1^k$$

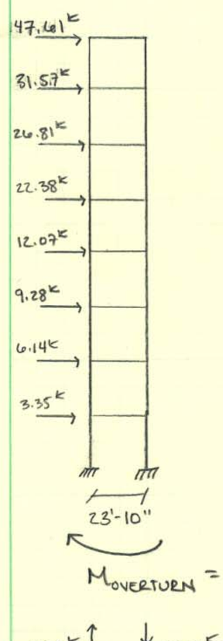
$$\therefore P_U = 1.2D + 1.0L = 1.2(133.1) + 1(94.8) = 254.52^k$$

KL = 15' USING TABLE 6-1 FOR W14x426

$$p = 0.202 \times 10^{-3} \qquad b_x = 0.273 \times 10^{-3}$$

$$pP_U + b_x M_U = [0.202 \times 10^{-3}]254.52 + [0.273 \times 10^{-3}]1053'^k$$

$$= 0.34 < 1 \text{ } \underline{\text{OK}}$$



CHECK OVERTURNING ON FRAME 6 SINCE IT HAS THE SHORTEST FRAME WIDTH AND WILL CONTROL

$M_{OVERTURNING} = 47.61(111.5') + 31.57(96.5) + 26.81(81.5)$
$+ 22.38(68) + 12.07(54.5) + 9.28(41) + 6.14(27.5)$
$+ 3.35(15) = 13318.9'^{k}$

$$\text{UPLIFT FORCE} = \frac{M_{OVERTURN}}{\text{FRAME LENGTH}} = \frac{13318.9}{23.833'} = 558.8^{k}$$

IF DL ACTING ON COL G-1 IS GREATER THAN UPLIFT, THE FOUNDATION WILL NOT OVERTURN.

$W_{DL} = 103\text{PSF}$ (SEE LOAD COMBINATION CALCS)

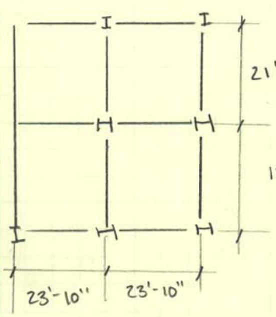
$$\text{TRIB AREA} = (23.833')\left(\frac{21.583 + 18.083}{2}\right)'(8\text{ FLOORS})$$

$= 3781.49\text{ SF}$

$P_{DL} = 103\text{ PSF}(3781.49\text{ SF})$

$= 389,493\text{ lbs} = 389.5^{k} < 558^{k}$

∴ OVERTURNING WILL BE AN ISSUE.



Appendix G: Plans

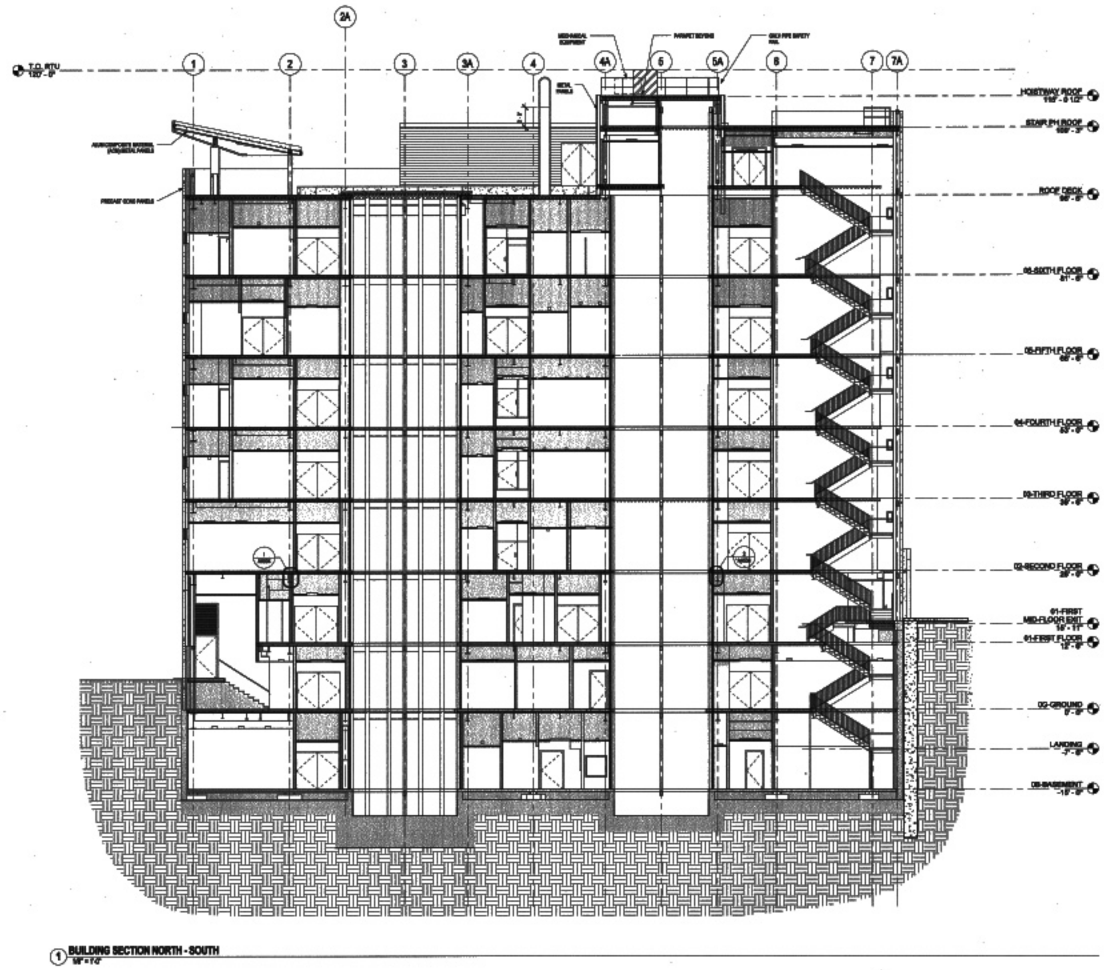


Figure 17: NS Buiding Section

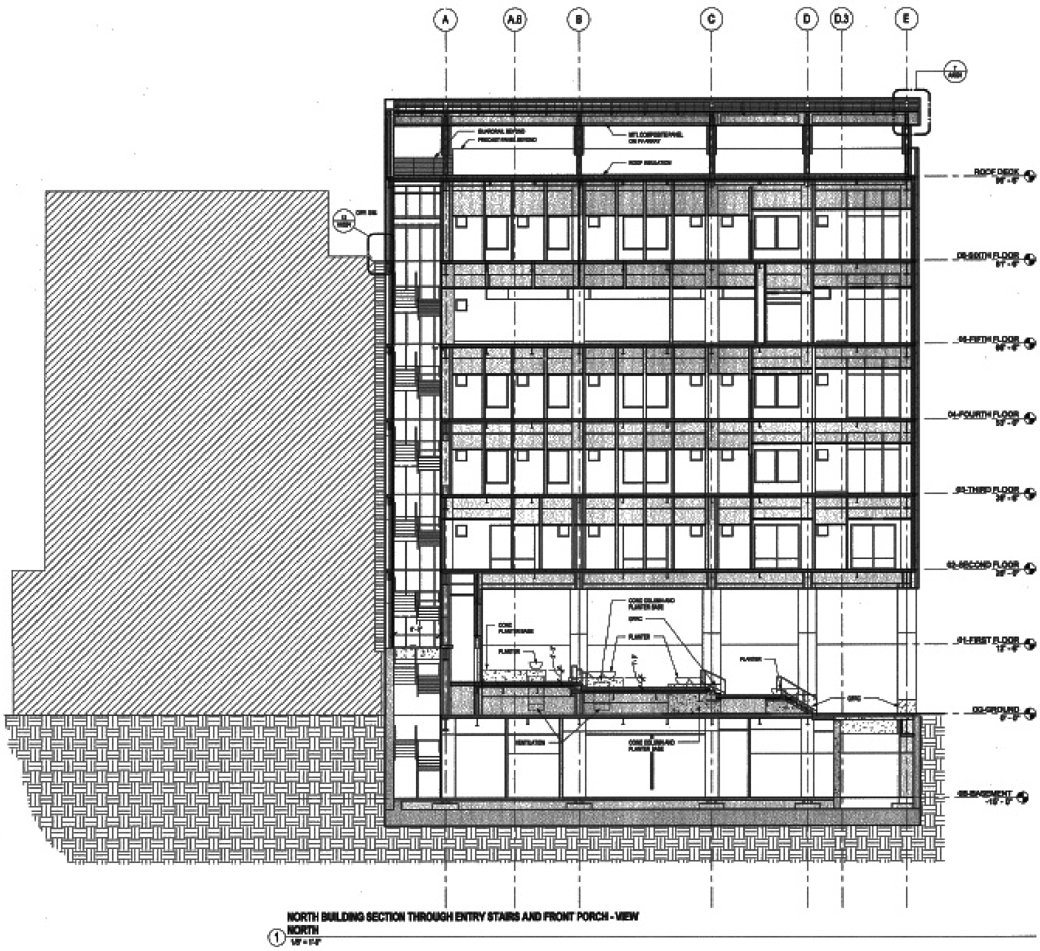


Figure 18: EW Building Section

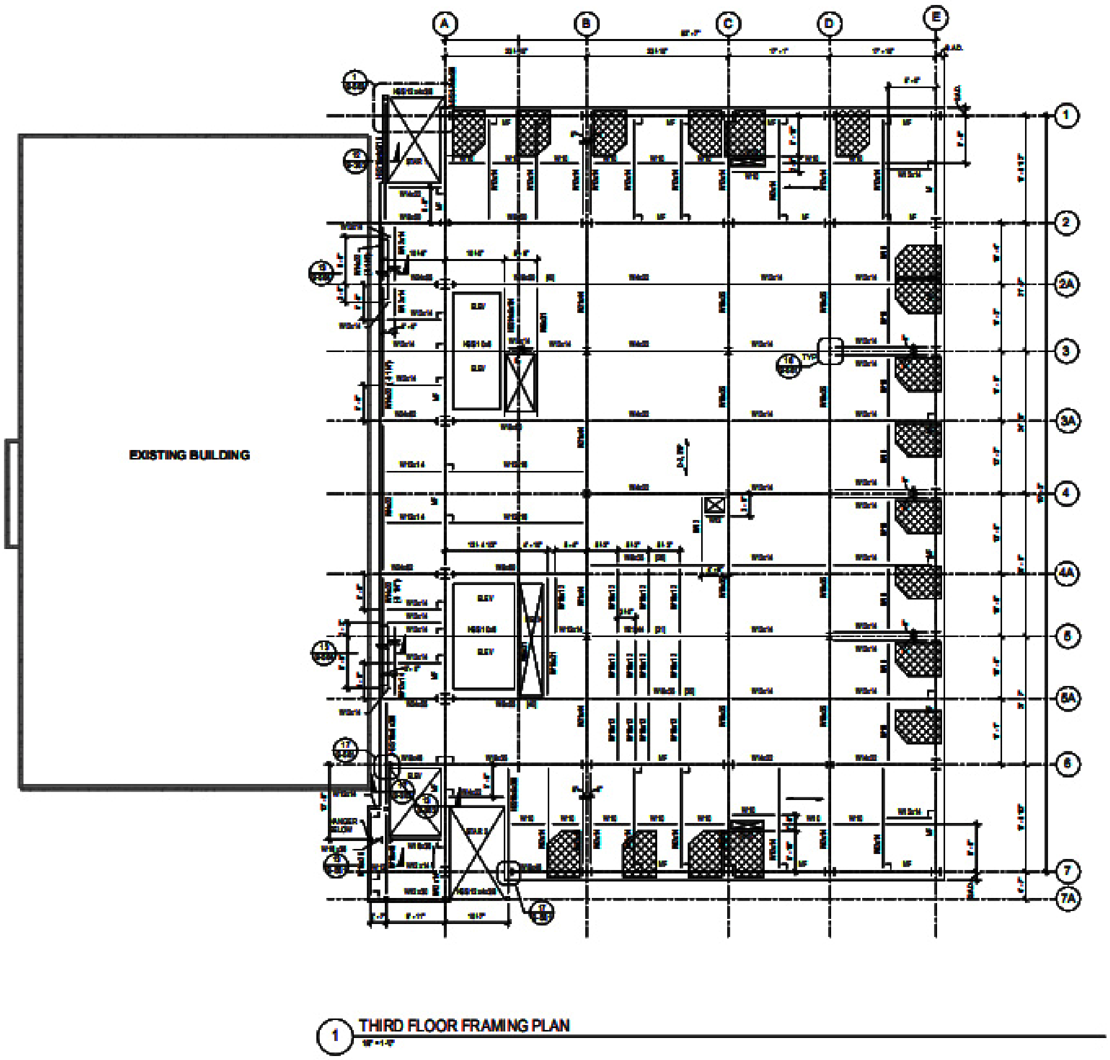


Figure 19: Typical Framing Plan

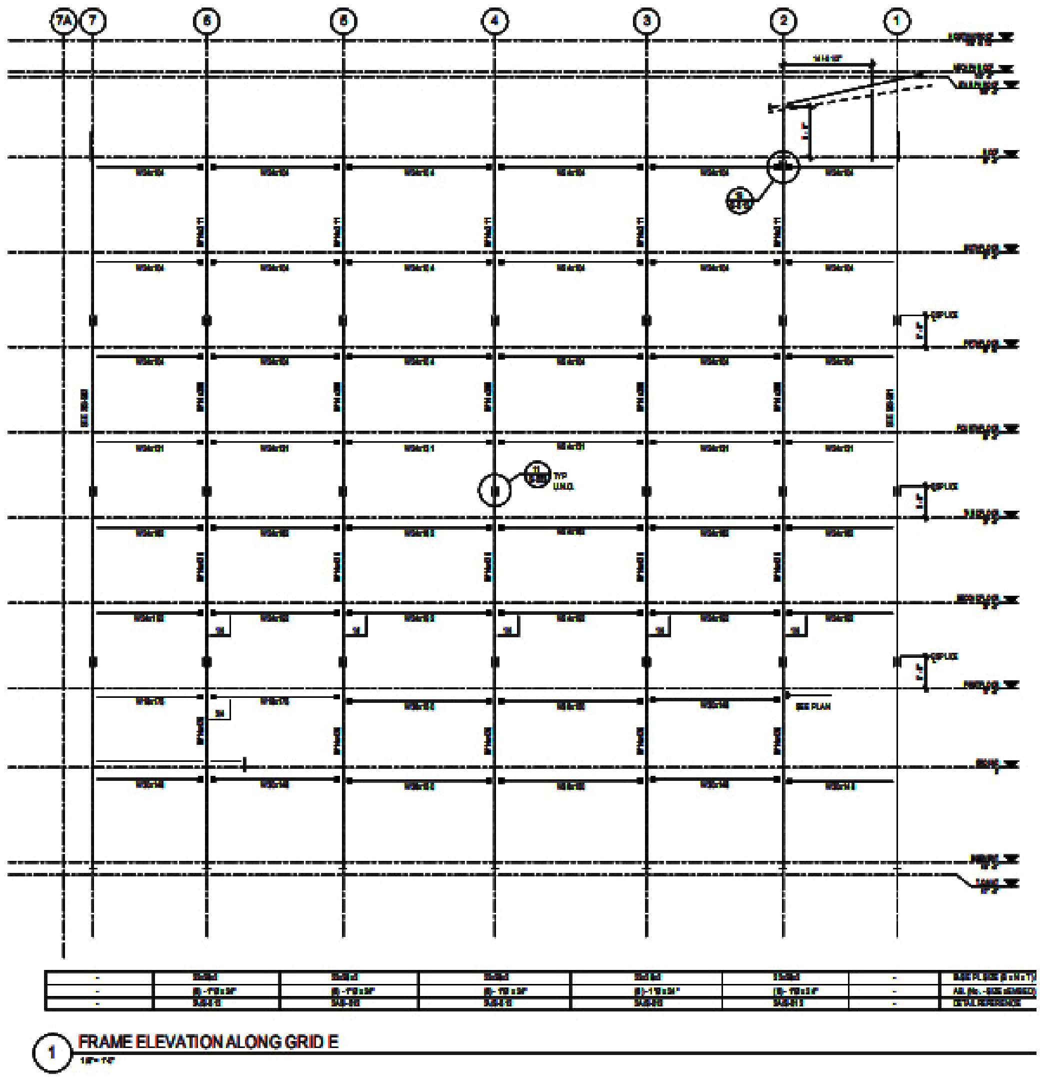


Figure 20: Typical Moment Frame Elevation