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Option: Structural
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Technical Report III

North-East Corner, Source: Oliver, Glidden, Spina

Largo Medical Office Building

Largo, Florida

Table of Contents

Executive Summary

The existing lateral force resisting system and lateral load distribution were studied in Technical Report III. Lateral system of the Largo Medical Office Building (LMOB) was evaluated for wind load irregularity effects, horizontal and vertical seismic irregularities. Also spot check/design was implemented to determine whether the current shear wall dimensions were adequate.

LMOB only experiences soft story irregularity, with the possibility for torsional irregularity. The soft story irregularity occurs on the first story. Occurrence of soft story in this location is caused by the higher floor-to-floor height, 16 ft. for the first story, while other stories only have a 14 ft. floor-to-floor height. Torsional irregularity is only a possibility because only a structural computer model was used. Hand calculations in torsional irregularity wasn't implemented because of the need to design all lateral force resisting members and time to finish the hand calculations. Another reason that torsional irregularity is a possibility is that the center of rigidity is different between ETABS output and the one determined by hand. Not only that, but the fundamental period determined by the hand calculations and computer modeling is significantly different. Thus the computer model can't be trusted.

As determined in hand calculations in Technical Report I, the fundamental period of LMOB is 0.66 seconds. There were changes to the lateral loads when the lateral system was downgraded to an ordinary reinforced concrete shear wall and revising gust factor. The reason for downgrading the lateral force resisting system is the realization that it is unlikely for a seismically inactive region to use seismic detailing. These changes modified the lateral loads, but the wind loads still control over the seismic loads.

Spot check/design was only done for the member with the highest base shear and overturning moment. All lateral force resisting members have stiffness based on their respective lengths. In the building, the member with the second longest length has the highest loads. Reason that the longest length member didn't have the highest load is the smaller torsion induced shear. Hand calculations indicate that the current shear wall dimensions are sufficient to resist the controlling wind load in the North/South direction.

Building Overview

Largo Medical Office Building (LMOB) is an expansion of the Largo Medical Center complex. Designed in 2007 and completed in 2009; LMOB is managed and constructed by The Greenfield Group. Located in Largo, Florida; the six story facility was designed to house improved and centralized patient check-in area. The facility also houses office space for future tenants, as well as screening and diagnostic equipment.

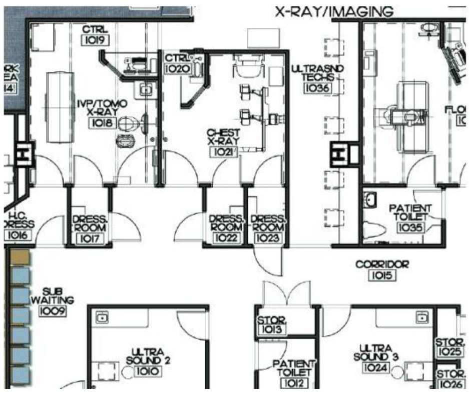

Figure 1.1, Illustrated Floorplans
Source: Oliver, Glidden, Spina & Partners

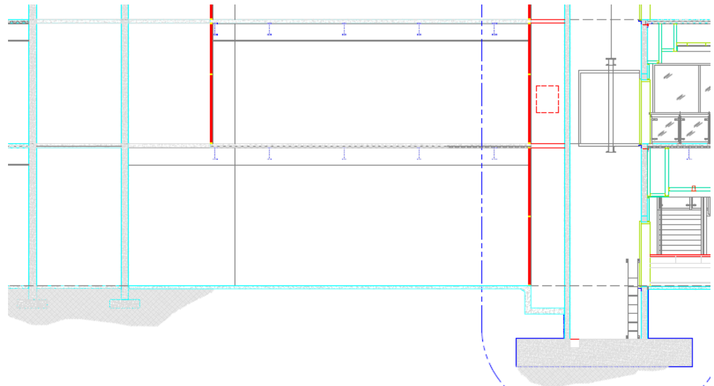

Figure 1.2, Building Section
Source: Oliver, Glidden, Spina & Partners

Patient privacy is a major concern for facilities housing medical related activities. Oliver, Glidden, Spina & Partners answered this by clustering the screening and diagnostic spaces close to the dressing areas (Figure 1.1). The architect went a step further, to preserve privacy by compartmentalizing the building's interior.

LMOB is a steel framed facility with ordinary reinforced concrete shear walls to resist lateral loads. The shear walls and structural columns rest on top of spread footings which are at least 27" below grade (Figure 1.2). LMOB's envelope consists of 3-ply bituminous waterproofing with insulating concrete for the roof; impact resistant glazing and reinforced CMU for the façade.

Structural System

Largo Medical Office Building is a 105' tall and 155,000 ft^2 facility which utilizes ordinary reinforced concrete shear walls and a steel frame.

Unique building components and site conditions not considered in this report includes:

1. Effects of drain placement on the rain load
2. Wind loading on the overhang (Figure 2.1)
3. Soil profile

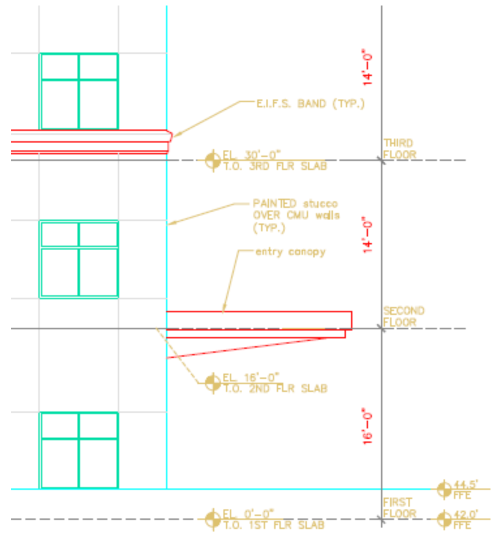

Figure 2.1, Overhang
Source: Oliver, Glidden, Spina & Partners

Framing & Lateral System

The steel frame is organized in the usual rectilinear pattern. There are only slight variations to the bay sizes, but the most typical is 33'-0" x 33'-0". Please refer to Appendix A for typical plans and elevations. Girders primarily span in the East/West (longitudinal) direction. Only the overhang above the lobby entrance and loading area are girders are orientated. It is assumed that the columns, girders, and beams are fastened together by bearing bolts. As a result, the steel frame only carries gravity loads.

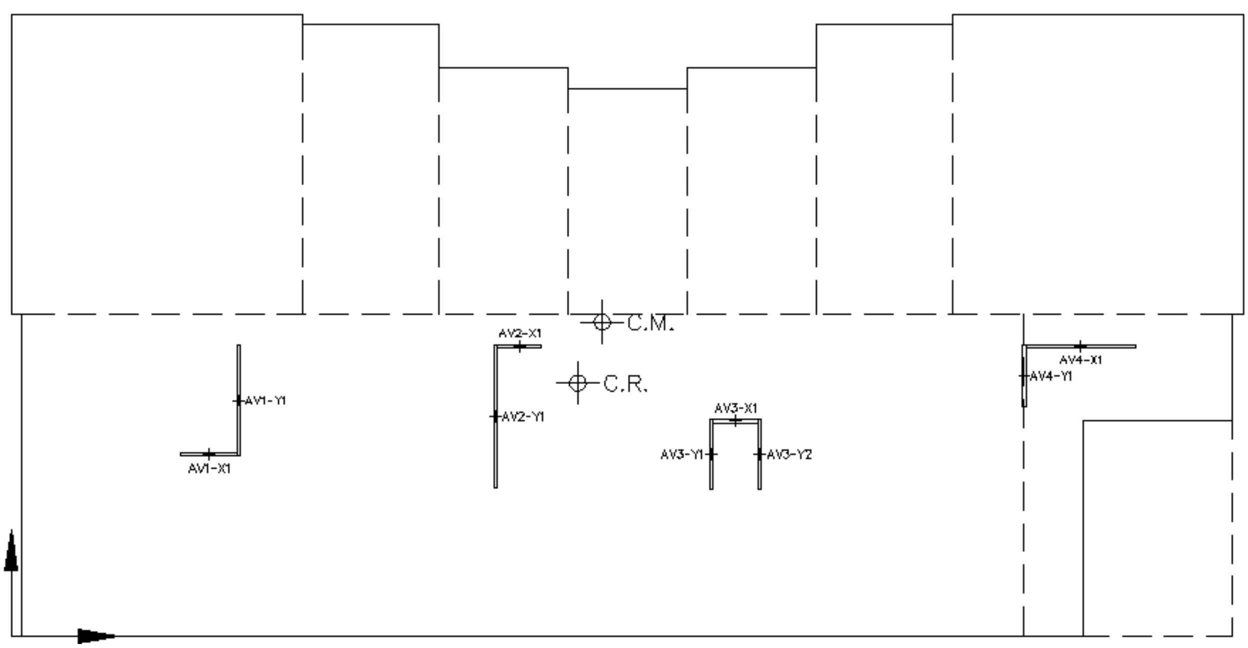

Figure 2.2, Shear Wall Locations

To deal with the lateral load, ordinary reinforced shear walls are used. The shear walls help the facility resist wind from the North/South and East/West direction. All shear walls are continuous and span from the ground floor level to the primary roof (86' above ground floor level). See Figure 2.2 for shear wall locations.

Flooring System

In general, the structural flooring system is primarily a 5" thick composite slab (Figure 2.4). On all floor levels, except for the ground, the composite slab spans 8'-3". To satisfy the 2-hour fire rating defined by the FBC, it is likely that the floor assembly received a sprayed cementitous fireproofing. Exposed 2" composite deck with 3" of normal weight (NW) topping only has a 1.5-hour rating, per 2008 Vulcraft Decking Manual.

Roof System

LMOB has three roof levels: main roof, east emergency stairwell roof, and the overhang over the main entrance. There is only one roof type for all three roof levels, consisting of a 3-ply bituminous waterproofing applied over the insulated cast-in-place concrete (Figure 2.3). To ensure adequate rainwater drainage, the insulated cast-in-place concrete is sloped ¼" for every 12" horizontal.

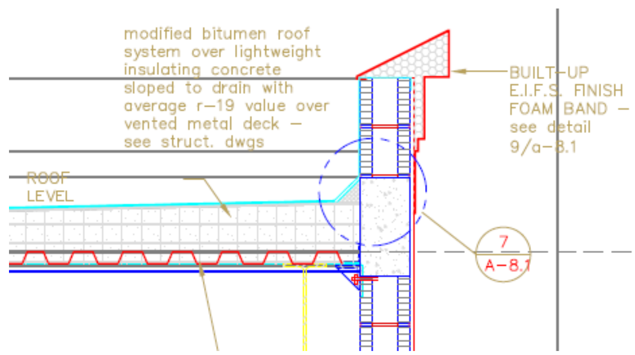

Figure 2.3, Roof Detail
Source: Oliver, Glidden, Spina & Partners

The insulated cast-in-place concrete was used in-lieu of rigid insulation with stone ballast. One reason is that the facility is in a hurricane zone. This means that loose material can potentially become airborne projectiles and cause damage when there is a hurricane. The insulated concrete has sufficient mass to resist becoming airborne in a hurricane. In addition, the added mass counters the uplift wind force.

Lateral Force Resisting System

Wind Loads

Method 2 in Chapter 6 of ASCE 7-05 was used to determine the Main Wind Force Resisting System (MWFRS) and wind load on the Components & Cladding (CCL). Story forces and overturning moments were determined by calculating the wind pressures and loads. Assumptions were made to simplify method 2, as follows:

1. Ignore the overhang
2. Connection between floor diaphragm and façade allows thermal induced movement
3. Due to multiple roof levels, that average roof elevation 95'-6" was utilized
4. 0.85 Gust factor was used, since diaphragm is rigid
5. Internal pressurization is unlikely due to use of impact resistant glazing
6. Type III for importance category

From the wind analysis, the MWFRS loads due to wind in the North/South direction controls over the East/West direction. Higher story shears, in the North/South directions, can be attributed to greater façade area. All wind calculations are available for reference in Appendix D.

LMOB is located in a suburban area, where most neighboring buildings are less than 30 ft. Only to the west are there tall buildings, namely the Largo Medical Center (highlighted blue in Figure 4.1). Though the parking garage is the other tall structure in the immediate vicinity of LMOB, the effects are neglected. The parking garage was built after LMOB was completed. As a result of the surrounding buildings, the site is classified as having wind Exposure Category B.

Figure 4.1, Neighboring Buildings
Source: Google Maps

Seismic Loads

Equivalent Lateral Force method was used to determine the seismic loads on LMOB. Seismic load transfers from the floor diaphragms to the shear walls. The shear wall locations can be referenced in Figure 2.2. No seismic loads were transferred to the top roof, at 105', due to the lack seismically designed masonry structure supporting the diaphragms (Figure 4.2, on the following page).

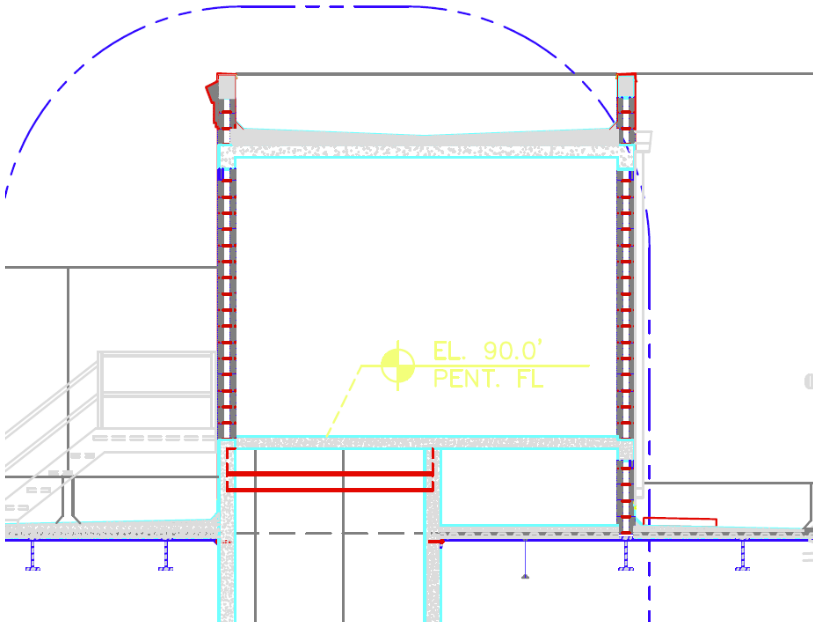

Figure 4.2, Non Seismic Design Top Roof
Source: Oliver, Glidden, Spina & Partners

Table 4.1, Maximum Base Shear (V_n) and Overturning Moment (M_n)		
	Seismic	Wind
Base Shear (Kip)	376.4	916.2
Overturning Moment (Kip-ft)	23340.1	47192.8

Using ASCE 7-05 it was discovered that the facility doesn't have to resist significant seismic forces, approximately 376.4 kip. This translates to ~ 1.7% of the effective building weight. Live load due to storage, and dead loads determined previously in were used to calculate the effective building weight. Refer to Appendix E for more details. After analyzing both wind and seismic loads, it was found that the wind loading in the North/South direction is the controlling lateral scenario. See Table 4.1 for wind and seismic base shear and overturning moment. Due to Florida's low seismic activity but high hurricane risk it is logical that the facility experiences high wind loads when compared to seismic loads.

Irregularity Analysis

Wind Irregularity

Eccentricity between the center of mass (CM) and the center of rigidity (CR) affects the loads experienced by the shear walls. Torsion is present whenever there is an eccentricity between the CM and CR. LMOB has three types of floors, each with a distinct CM; see Table 4.2 (on the following page).

Table 4.2, CM of Floor Types			
Floor Type	Floor Levels	X_{cm} (ft)	Y_{cm} (ft)
A	0	110.07	59.34
B	1	114.69	58.72
C	2, 3, 4, 5	114.79	58.90

Assumptions were made to simplify and expedite the hand calculation process, and are as follows:

1. No mechanical or other large openings in shear walls
2. All shear walls have stiffness' proportional to their respective length
3. Shear walls resisting lateral load in the North/South direction are not connected to shear walls resisting lateral load in the East/West direction
4. Floor diaphragm concrete will crack before shear walls, due to exposure to both gravity and wind loads

Wind loads, determined in Technical Report I, were distributed to each lateral resisting element based on stiffness. Deep members had the greatest share of shear, primarily due to high stiffness. It was initially expected that the deepest member, AV2-Y1, would have the greatest shear. The hand analysis indicated that AV1-Y1 had greater shear, due to the torsion shear component. Go to Appendix F for more details on calculations.

Table 4.3, Maximum Base Shear			
Lateral Force Resisting Member	Controlling Wind Case	Maximum Base Shear (Kip)	Maximum Base Shear per Length (Kip/ft)
AV1-X1	I	76.49	7.40
AV1-X1	II	325.00	15.42
AV2-Y1	I	304.42	11.27
AV2-X1	I	63.85	7.82
AV3-Y1	I	126.60	9.62
AV3-X1	I	63.35	7.53
AV3-Y2	I	121.65	9.24
AV4-Y1	I	84.03	7.20
AV4-X1	I	159.59	7.82

Each wind case was calculated, to determine the case and member with the highest base shear. Accidental torsion in Case II and Case IV was applied to maximize member base shear. Determined in Case I and Case III, the torsion shear component at max was only 25.5% of the direct shear, which is small. There is no possibility that a low stiffness lateral member will experience greater base shear, when compared to a higher stiffness lateral member. As a result, the accidental torsion was applied clockwise to increase base shear experienced by high stiffness members (AV1-Y1 and AV2-Y1). The maximum base shears and load case for each member can be referenced in Table 4.3.

Seismic Irregularity and Building Period

LMOB was evaluated for horizontal and vertical irregularity, though not required for seismic category A. A reason is the potential to move the facility to a more seismically active region, in the spring 2013 semester. By visual inspection facility's regular shape, continuous lateral system, and parallel lateral force resisting system eliminated the need to check the facility for horizontal irregularity (4) and (5). Vertical irregularities checks eliminated; due to the visual inspection; are vertical irregularity (3), (4), (5a), and (5b). Other horizontal and vertical irregularities were analyzed by both hand calculations and through the use of ETABS.

When analyzing the facility assumptions were made, and are listed below:

1. Floor diaphragm openings due to MEP are not significant and not included in diaphragm discontinuity irregularity analysis
2. Stiffness in soft story irregularity is inversely proportionate to the story height
3. Construction effects on stiffness was not considered

The rational behind assumption (2), is based on the equation: $K = 12EI / L^3$ (fixed-fixed member). Continuity of all lateral force resisting members translates to constant moment of inertia at all stories. As a result the stiffness equation's numerator is a constant and only the height (L) of the story has an impact.

Table 4.4, Re-Entrant Corner Analysis

Floor Level	Building Dimension w/o Re-Entrant Corners (ft)		Re-Entrant Corner Dimensions (ft)		Externsion Percentage	
	Long Side	Short Side	Long Side	Short Side	Long Side	Short Side
0	197.51	73.59	28	40.83	14.2%	55.5%
1	225.51	115.43	2	2	0.9%	1.7%
2	225.51	115.43	2	2	0.9%	1.7%
3	225.51	115.43	2	2	0.9%	1.7%
4	225.51	115.43	2	2	0.9%	1.7%
5	225.51	115.43	2	2	0.9%	1.7%
Roof 1	225.51	115.43	2	2	0.9%	1.7%

Re-entrant corner, floor diaphragm discontinuity, mass, soft story, and torsional irregularity were analyzed according guidelines established in ASCE 7-05 Tables 12.3-1 and 12.3-2. At a quick glance of Table 4.4, LMOB appears to have re-entrant corner irregularity, but this is not so, because both re-entrant corner extension percentage in the long and short sides must be greater than 15%. The max floor diaphragm discontinuity occurs at floor level 1 and is only 7.8%, primarily due to the two story lobby. This is nowhere close to the 50% threshold, which ASCE 7-05 would classify that floor diaphragm discontinuity exist. After comparing the values on Table 4.5 (located below) to ASCE 7-05 Table 12.3-1 and 12.3-2, there is soft story irregularity but no mass irregularity. The facility doesn't have extreme soft story irregularity because the K_i / K_{i+1} is greater than 60%. All hand calculation, pertaining to the seismic irregularity analysis, is in Appendix F.

Table 4.5, Soft Story and Mass Irregularity Analysis					
Story	Story Height (ft)	$K \sim 1 / L^3$	K_i / K_{i+1}	K_i / K_{avg}	$W_{eff,j} / W_{eff,i}$
1	16	0.00024	67.0%	75.3%	101.7%
2	14	0.00036	100.0%	100.0%	101.4%
3	14	0.00036	100.0%	100.0%	101.8%
4	14	0.00036	100.0%	100.0%	100.2%
5	14	0.00036	100.0%		
6	14	0.00036			

Instead of using hand calculations to determine torsional irregularity, *ETABS* was used. The need to determine the effective moment of inertia of each member at each story will require the design of all lateral force resisting members. Long duration of the hand analysis is the main reason for not implementing hand calculations. To ensure that the *ETABS* result are accurate; the center of mass, center of rigidity, as well as the case I wind induced force on member AV2-Y1; will be compared with the hand calculations. For more details about the structural computer modeling and assumptions, see Appendix H.

Table 4.6, Typical Floor Diaphragm Center of Mass and Rigidity							
Hand Analysis				Computer Analysis			
Center of Mass		Center of Rigidity		Center of Mass		Center of Rigidity	
x	y	x	y	x	y	x	y
114.79	58.90	105.51	47.79	114.78	58.80	89.90	47.79

Table 4.7, Wind Case I Base Shear of Member AV2-Y1	
Hand Analysis	Computer Analysis
304.42 Kip	327.44 Kip

Evident in Table 4.6 and Table 4.7, the structural computer model is not entirely accurate. The structural computer model has a different center of rigidity from the hand calculation. An impact of the center of rigidity difference is change in torsion induced shear and extreme torsional irregularity. Unlike the hand calculation, it was assumed that the shear walls are monolithically cast; meaning that the shear wall will act more like an angle/L-section. This is the reason for the change in center of rigidity.

Though the change in center of rigidity was expected, the significant difference between the building's fundamental period wasn't. When using ASCE 7-05 equation 12.8-9, the fundamental period is 0.66 seconds. ETABS determined the fundamental period to be 2.38 seconds, due to torsion. It was verified that the building mass and dimensions in ETABS is the same as the hand calculations. Since the period $T = 2\pi * (\text{mass/stiffness})^{1/2}$, it is likely that the lateral force resisting element's stiffness is the culprit for the error.

It was decided that the ETABS model is not accurate and additional debugging of the structural computer model is required. Unfortunately, at this time it can't be determine whether or not the building has torsional irregularity.

Story Drift

Story drift, was evaluated to prevent damage of building components. Wind induced story drift controls over seismic story drift. There are two reasons for this; one is the higher wind loads. The other reason is that greater drift of the lateral force resisting system are permissible in seismic design, to facilitate energy dissipation.

Instead of determining the story drift by first designing each shear wall, it was assumed that the effective moment of inertia is 25% of the uncracked moment of inertia. Shear wall drifts was determine by subtracting the deflections at top and bottom of each story. The formula used to determine the top and bottom deflection is $\Delta_{dfl} = PL^3 / (12EI_{effective})$. Refer to Appendix F, for more details about the story drift calculations. The maximum story drift occurs at the first story (least stiff story) and is approximately 0.01. ASCE 7-05 Section CC1.2 dictates that the maximum allowable story drift shall be $H_{story}/400$, in our case the maximum allowable story drift shall be 0.48. From the comparison, between the maximum allowable story drift and actual maximum story drift, the building doesn't violate the serviceability criteria.

Lateral Spot Check/Design

The shear wall experiencing the largest base shear was selected to be designed and lateral system spot check. In addition, the design was checked with a computer model, *RAM*. Member AV1-Y1 was evaluated for flexure and shear due to wind loads, the controlling lateral load. Load combination $1.2D + L + 0.5L_r + 1.6W$ was used in designing the lateral force resisting member.

Shear wall AV1-Y1 was designed similar to a long flexural member as opposed to a deep beam, because the height-to-length ratio is greater than 4.

To reduce the number of design iterations assumptions were made during the design process and are as follows:

1. Shear walls take no axial loads
2. Reinforcement responsible for controlling thermal induced cracks don't contribute to strength
3. All vertical reinforcements are the same size
4. Two layers of flexural rebar
5. $\varepsilon_t = 0.005$ for flexural reinforcement furthest from the neutral axis

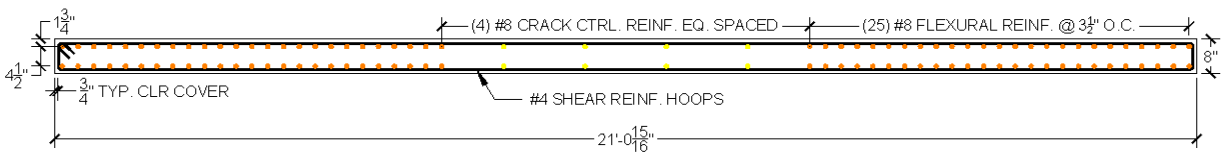

Figure 4.3, Flexural Reinforcement Design

Current shear wall, AV1-Y1, dimensions are sufficient to resist base shear and maximum moment. Top reinforcement is required, due to the likely hood that the wind load will reverse. The other reason is to strain the flexural reinforcement to 0.005, in order to use a $\Phi = 0.9$. Refer to Figure 4.3 for the flexural and crack control reinforcement. As for shear reinforcement hoops, these are not necessary at distances less than *d* from the face of support and where the shear is less than 183.3 Kips. However, a decision was made to place hoops at locations where shear reinforcement hoops are not required, to confine the concrete core and avoid possible rebar buckling during the construction process. All design calculations, pertaining to shear wall AV1-Y1's design is in Appendix G.

Table 4.8, Wall Design		
Design Method	Hand	Computer
Flexural Reinforcement	Tension Zone: (50) #8 @ 3.5" O.C. Compression Zone: (50) #8	Tension Zone: (64) #8 @ 4" O.C. Compression Zone: (0) #8

RAM's design of wall AV1-Y1 is logical, when comparing values in Table 4.8. Greater spacing between rebars and no compression rebar, in the computer design, necessitates additional reinforcement; as evident in the greater quantities of flexural rebar. Without top reinforcement the rebar furthest from the neutral axis will not reach a strain of 0.005, thus preventing the use of $\Phi = 0.9$.

The design procedure used for AV1-Y1 can be used most lateral resisting members except for AV2-Y1. With a height-to-length ration of 3.19, member AV2-Y1 must be designed as a deep beam (per ACI 318-11 Section 11.7.1), based on the strut-and-tie model.

Conclusion

Technical Report III studies the wind and seismic effects on the individual lateral force resisting members. Only the member with the largest base shear was designed, AV1-Y1. The building's story drift satisfies the maximum allowable drift limit H_{story} / 400. Both horizontal and vertical seismic irregularities were analyzed. LMOB has soft story irregularity and potentially torsional irregularity.

It is not well known whether or not LMOB has torsional irregularity, there are a number of reason for this. Hand calculations were not done for torsion irregularity, primarily due to the need to design all the lateral force resisting members and duration of the hand analysis. Though an ETABS was used to evaluate the building for torsional irregularity, the result of the ETABS model should not be used. The ETABS model has a greater eccentricity between center or rigidity and center of mass when compared to the hand calculations done previously. This caused a 2.38 second fundamental period and greater base shear in member AV2-Y1. Hand calculations yielded 0.66 second fundamental period and 304.42 kip base shear in member AV2-Y1. Additional debugging of the structural computer model is necessary to achieve an accurate analysis and determine whether LMOB has torsional irregularity.

Using the hand calculations in this Technical Report and previous ones, member AV1-Y1 was designed to the controlling lateral load (wind). Due to a height-to-width ratio greater than 4, member AV1-Y1 was designed as a flexural member instead of a deep beam with strut-and-tie. Lateral member AV1-Y1 experiences a base shear of 325 kip of base and an overturning moment of 16608.2 kip-ft. According to hand calculations (25) #8 rebar in each of the two layers of flexural reinforcement is required along with compression reinforcement, to resist the loads mentioned above. The purpose of the compression reinforcement is required to yield the reinforcement in tension. Unlike the torsional irregularity analysis, *RAM* generated a design AV1-Y1 similar to the hand calculation.

Appendix A: Floor Plans & Elevation

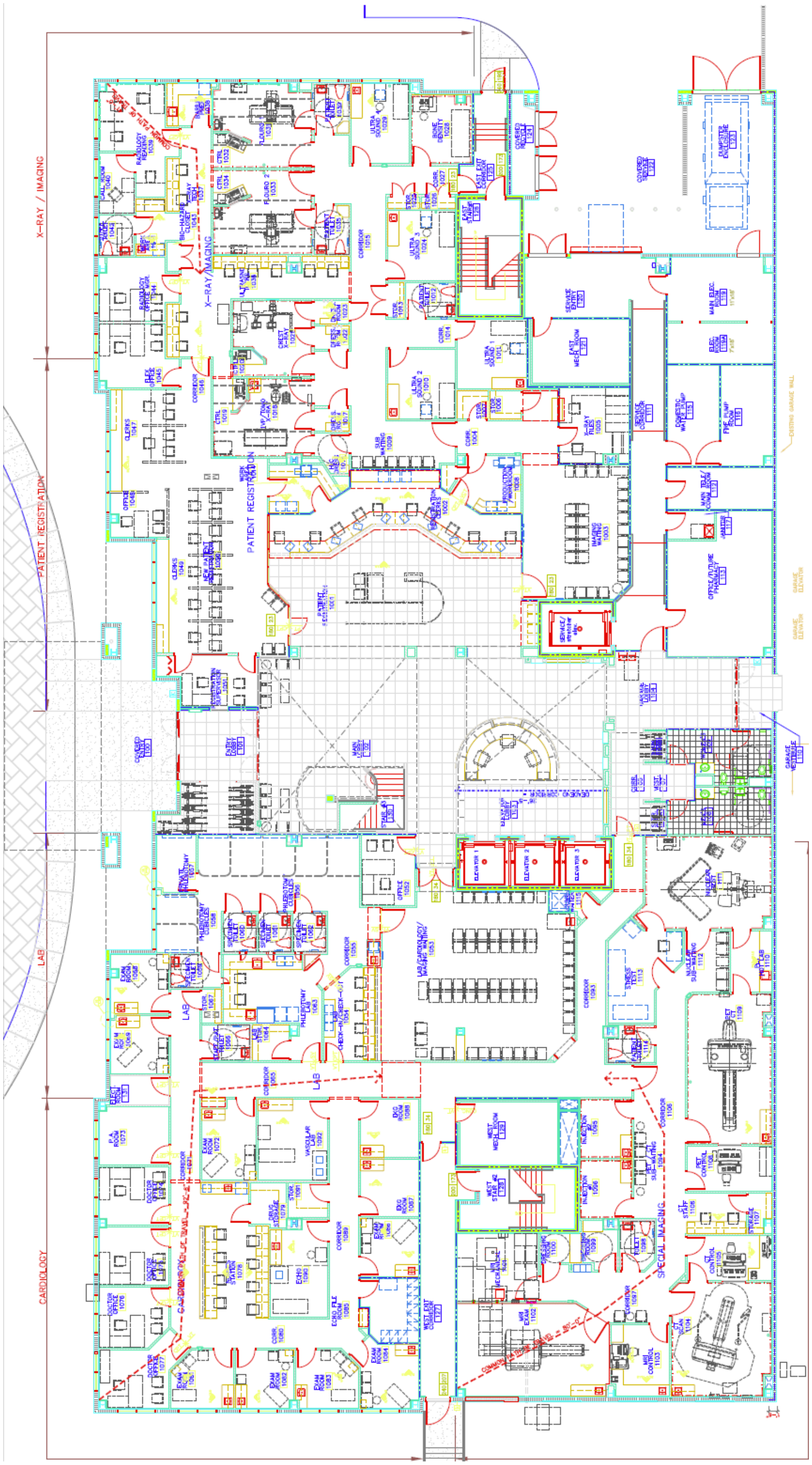

Figure AA.1, First Floor Plan w/ Tenant Build-Out

Source: Oliver, Glidden, Spina & Partners

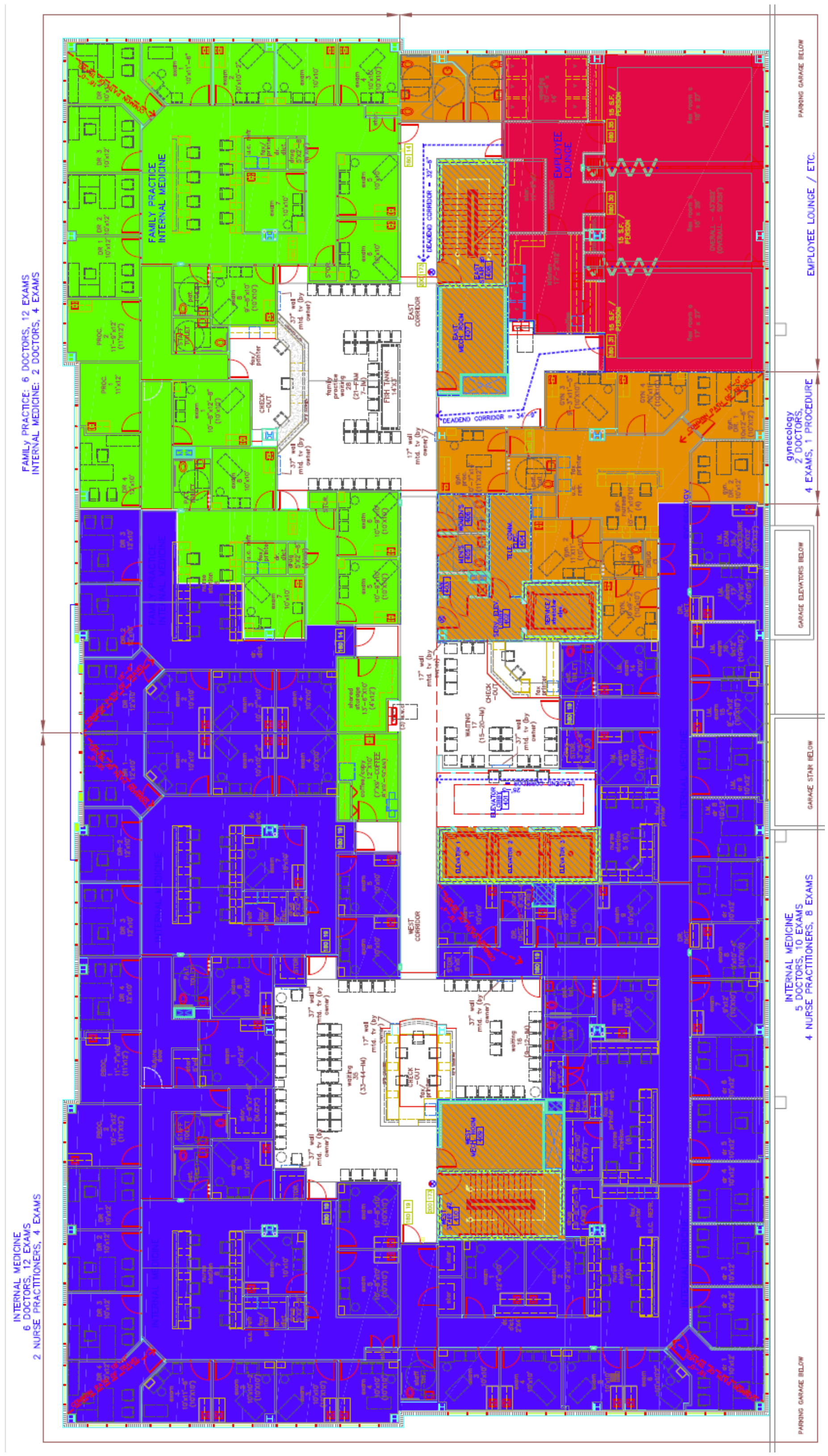

Figure AA.2, Typical Upper Floors
Source: Oliver, Glidden, Spina & Partners

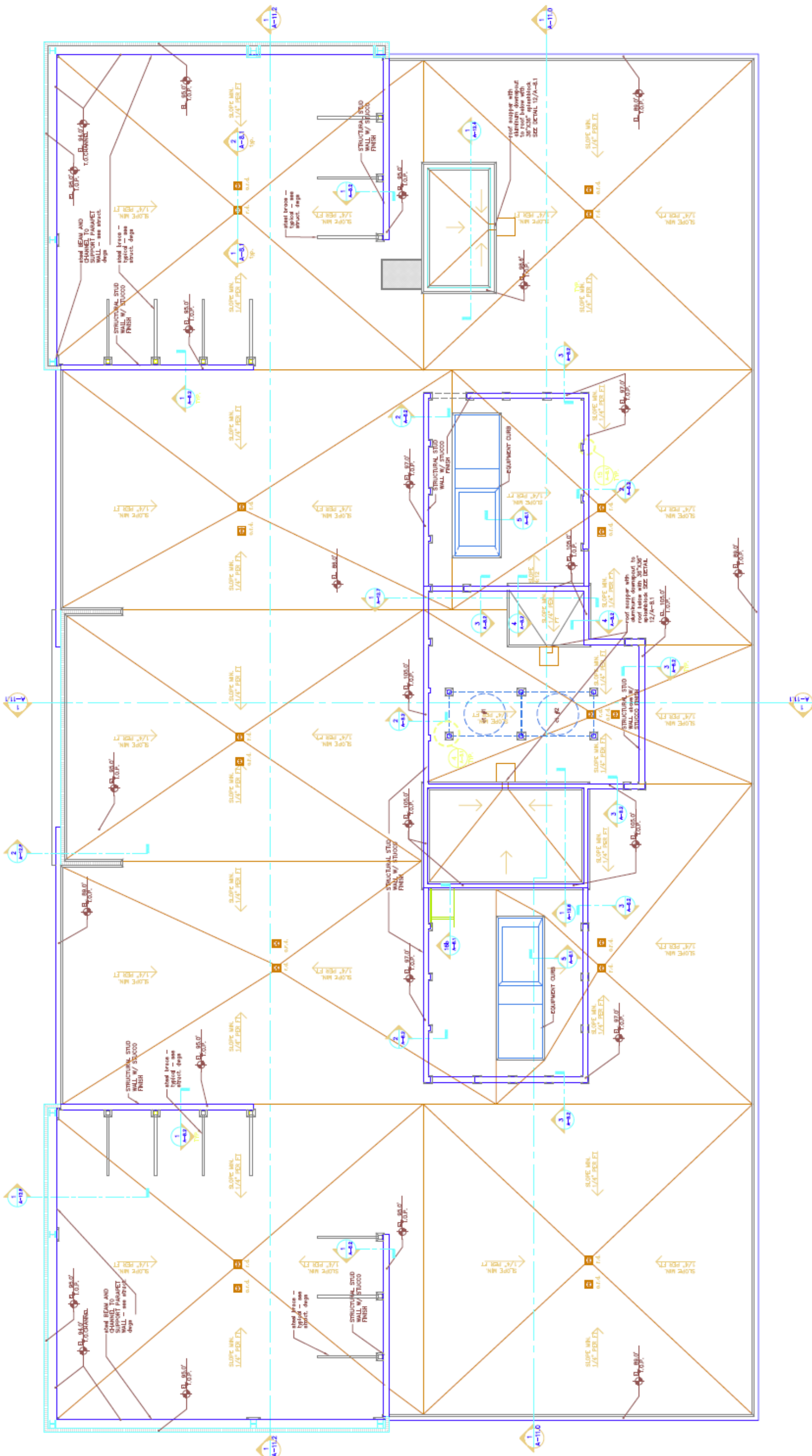

Figure AA.3, Roof Plan
Source: Oliver, Glidden, Spina & Partners

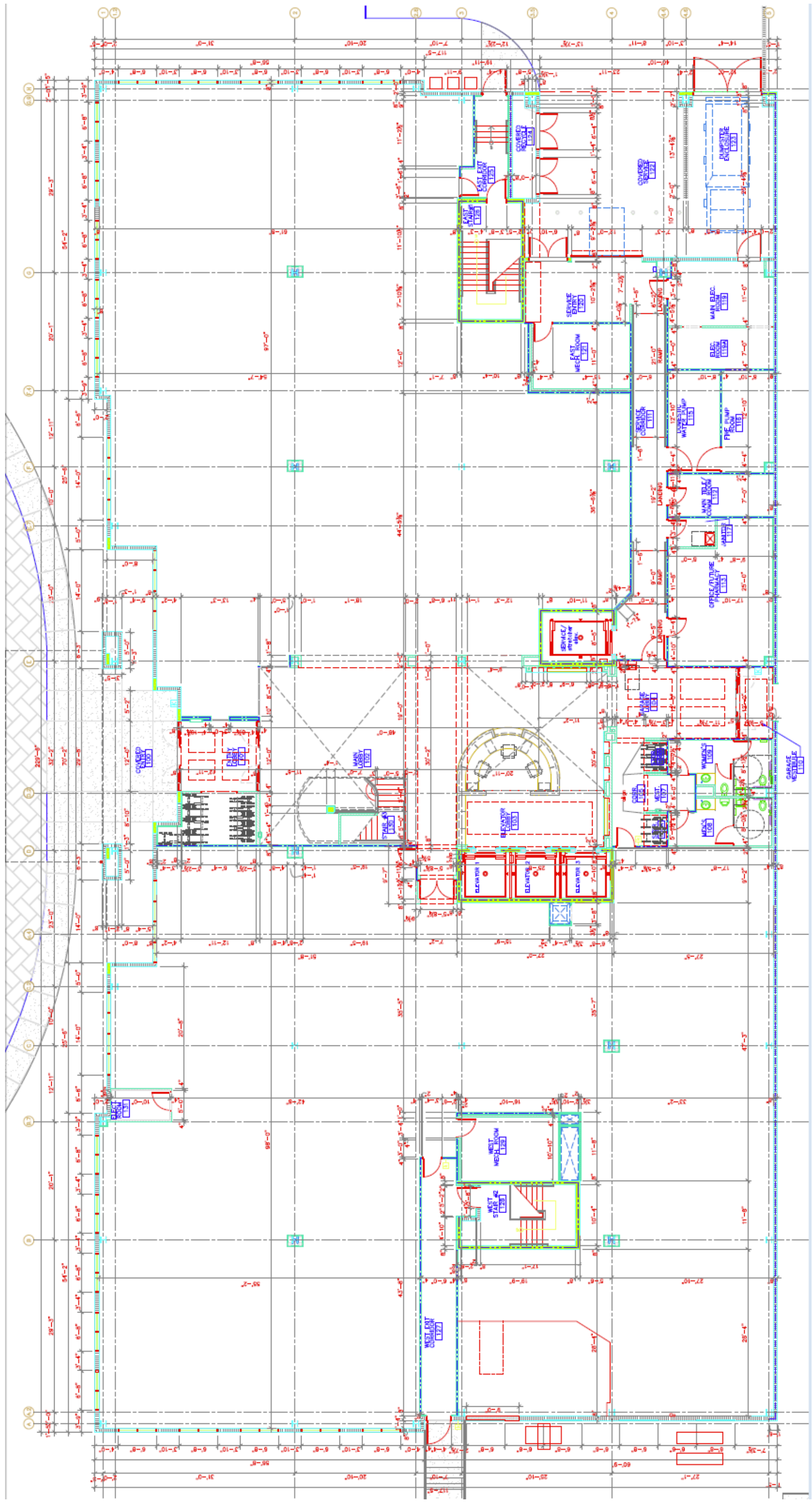

Figure AA.4, Typical Column Layout
Source: Oliver. Glidden. Spina & Partners

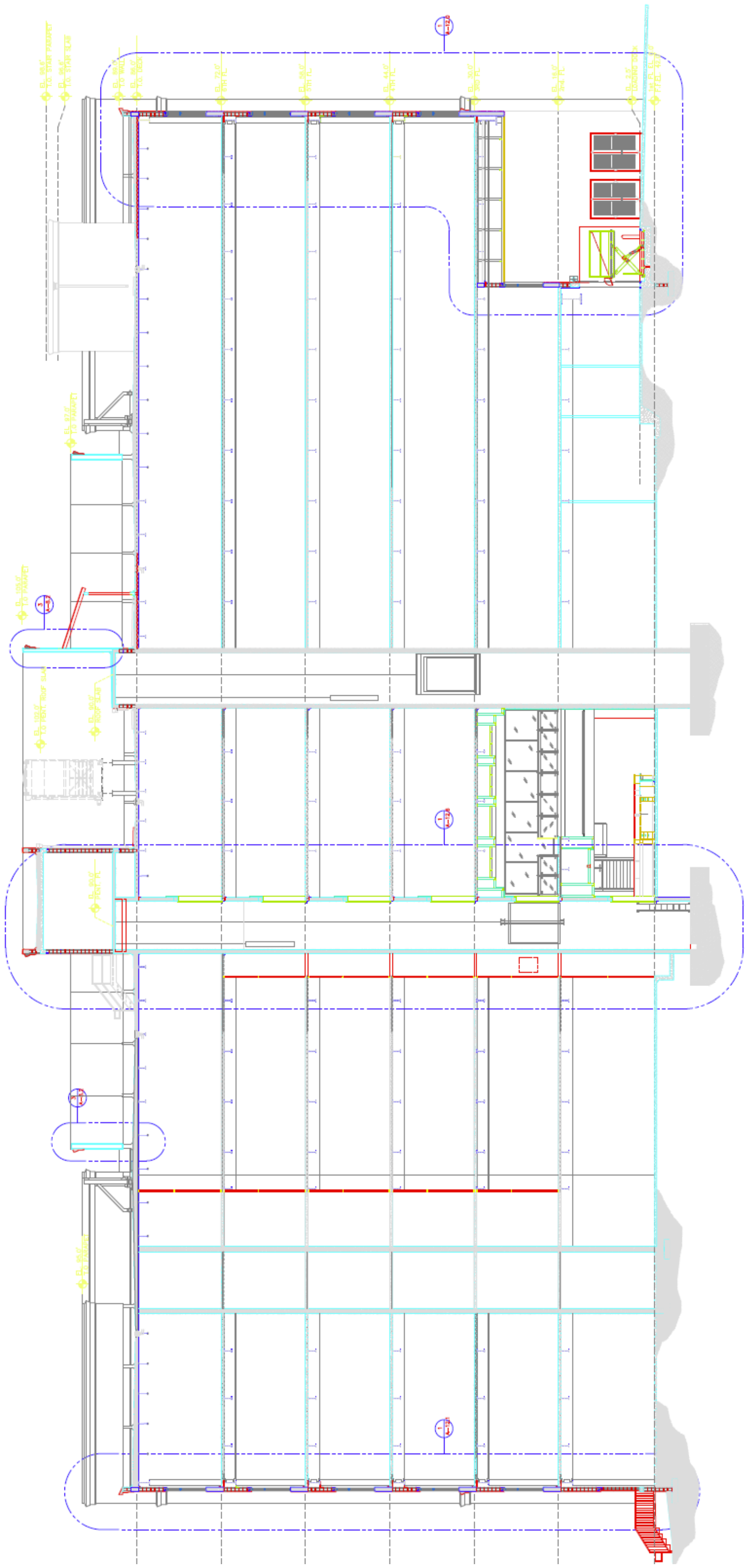

Figure AA.5, Longitudinal Building Section
Source: Oliver, Glidden, Spina & Partners

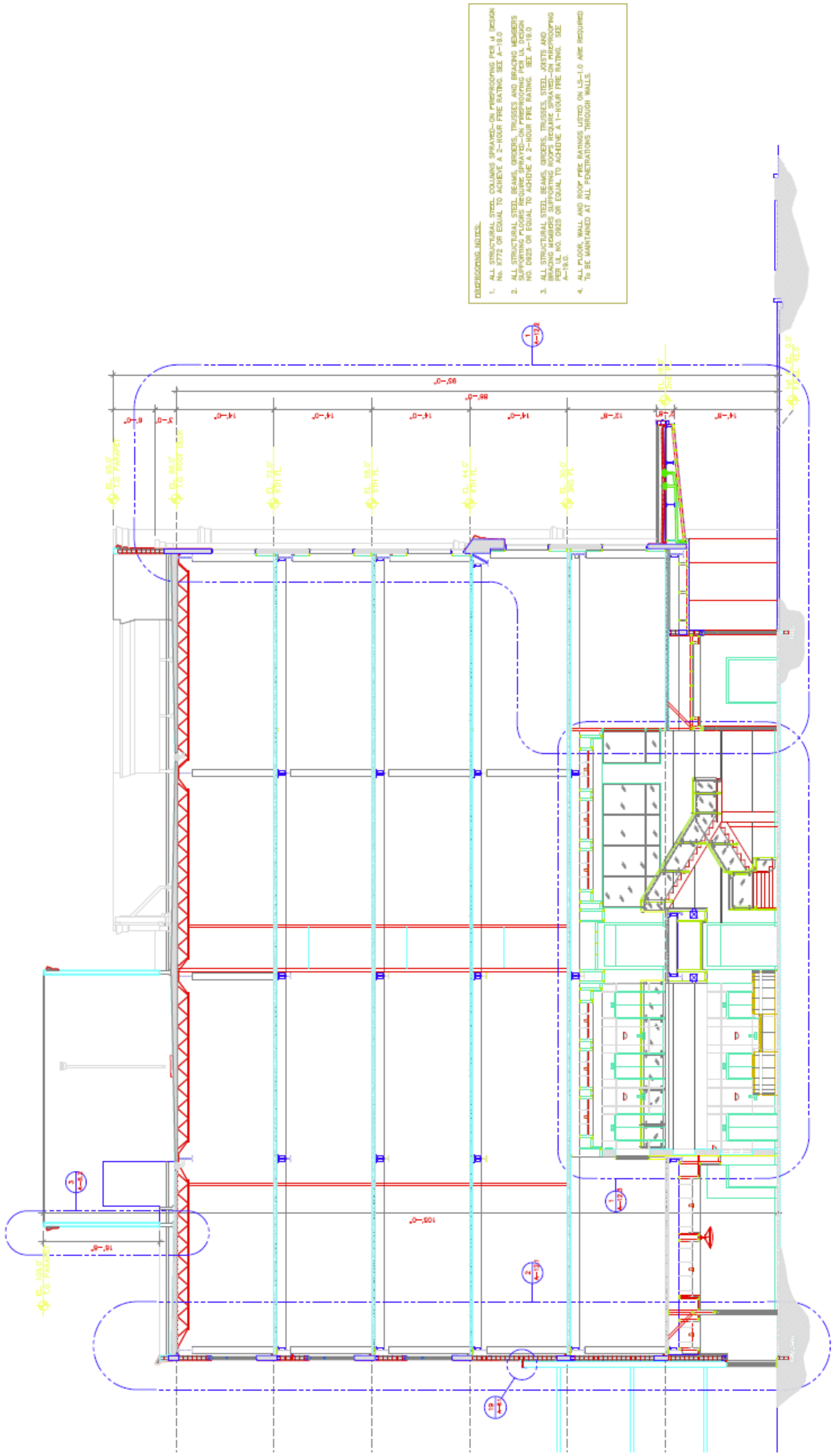

Figure AA.6, Building Section
Source: Oliver. Glidden. Spina & Partners

Appendix B: Load Determination Dead, Live, Rain

Thaison Nguyen | Load Determination – DEAD, LIVE, RAIN | 1/5

Floor Level	A_{gross} (ft^2)	$A_{fl\ opening}$[1] (ft^2)	A_{stairs} (ft^2)
0	24153.00	293.00	724.00
1	26440.00	1571.00	609.00
2	26440.00	293.00	609.00
3	26440.00	293.00	609.00
4	26440.00	293.00	609.00
5	26440.00	293.00	609.00
Roof[2]	26440.00	N/A	204.00

[1] Does not include stairwell openings
[2] Stairs extending to roof top has a roof

Story	A_{facade} (ft^2)	$A_{glazing}$ (ft^2)
1	11093.33	1588.00
2	9706.67	1920.20
3	9706.67	1846.20
4	9706.67	2681.60
5	9706.67	2780.40
6	9706.67	2780.40
Roof[3]	5079.00	N/A

[3] Roof has partitions enclosing mechanical equipment and stairwell.
** 5 lb/ft^2 dead load collateral.

Material	Weight	Notes
NW. CONC	150 lb/ft^3	AISC 14 Ed. Table 17-13
LW. CONC	113 lb/ft^3	Arch. Graphics Standards 11 Ed.
VCT	1.33 lb/ft^3	Arch Graphics Standards 11 Ed.
Ceramic / Porcelain Tile	10 lb/ft^2	AISC 14 Ed. Table 17-13
3 Ply Roofing	1 lb/ft^2	AISC 14 Ed. Table 17-13
Laminated Glass – 0.8"	8.2 lb/ft^2	
MEP	15 lb/ft^2	
Partitions	15 lb/ft^2	ASCE 7-05 4.2.2

a) Floor / Deck Thickness

1) Level: 0

$t_{floor} = 4''$, solid reinf. conc.

2) Level: 1 → 5

$d_{deck} = 2''$, assume metal deck has equal size corrogations

$t_{floor} = 5''$

$t_{floor,eq} = t_{floor} - d_{deck}/2 = 4''$, use to determine conc. weight

Thaison Nguyen | Load Determination – DEAD, LIVE, RAIN | 2/5

3) Level: Roof

$d_{deck} = 1.5''$, assume metal deck has equal size corrugations

$t_{floor} = 10\ 1/8'' \rightarrow 3\ 11/16''$
$t_{floor,avg} = (10\ 1/8 + 3\ 11/16)/2$
$t_{floor,avg} \approx 7''$

$t_{floor,eq} = t_{floor,avg} - d_{deck}/2 \approx 6.25''$, use to determine conc. weight

b) Floor Level Dead Weight w/o structural steel, Metal Deck, Flooring, Facade

1) Level: 0

$DL = 0.150(t_{floor})(A_{gross}) + 0.015(A_{gross} - A_{fl\,opening} - A_{stairs}) + 0.005(A_{gross})$
$DL = 0.150(4/12)(24153) + 0.015(24153 - 293 - 724) + 0.005(24153)$
$DL = 1675.5$ Kip

2) Level: 1

$DL = 0.150(t_{floor,eq})(A_{gross} - A_{fl\,opening}) + 0.015(A_{gross} - A_{fl\,opening} - A_{stairs}) + 0.005(A_{gross})$
$DL = 0.150(4/12)(26440 - 1571) + 0.015(26440 - 1571 - 609) + 0.005(26440)$
$DL = 1739.6$ Kip

3) Level: 2→5

$DL = 0.150(t_{floor,eq})(A_{gross} - A_{fl\,opening}) + 0.015(A_{gross} - A_{fl\,opening} - A_{stairs}) + 0.005(A_{gross})$
$DL = 0.150(4/12)(26440 - 293) + 0.015(26440 - 293 - 609) + 0.005(26440)$
$DL = 1822.6$ Kip/floor level

4) Level: Roof

$DL = 0.113(t_{floor,eq})(A_{gross}) + 0.015(A_{gross} \times 0.20) + 0.001(A_{gross}) + 0.005(A_{gross})$
$DL = 0.113(6.25/12)(26440) + 0.015(26440)(0.20) + 0.001(26440) + 0.005(26440)$
$DL = 1794.1$ Kip

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c) Dead Weight of Flooring

Floor Level	0		1		2 or 3 or 4 or 5	
Flooring	VCT	Ceramic	VCT	Ceramic	VCT	Ceramic
Area (ft^2)	1410	2841	531	653	531	339

*Other areas have exposed conc.

1) Level: 0

$DL = 1.33(1410) + 10(2841) = 30.3$ Kip

2) Level: 1

$DL = 1.33(531) + 10(653) = 7.2$ Kip

3) Level: 2 → 5

$DL = 1.3(531) + 10(339) = 4.1$ Kip / floor level

d) Dead Weight of Facade Envelope (by story)

1) Story: 1

$DL = 0.150(A_{facade} - A_{glazing}) + 0.0082(A_{glazing})$
$DL = 0.150(11093.33 - 1588.00) + 0.0082(1588.00)$
$DL = 1438.8$ Kip

2) Story: 2

$DL = 0.150(9706.67 - 1920.20) + 0.0082(1920.20)$
$DL = 1183.7$ Kip

3) Story: 3

$DL = 0.150(9706.67 - 1846.20) + 0.0082(1846.20)$
$DL = 1194.2$ kip

4) Story: 4

$DL = 0.150(9706.67 - 2681.60) + 0.0082(2681.60)$
$DL = 1073.7$ Kip

→

Thaison Nguyen		Load Determination – DEAD, LIVE RAIN	4/5

5) Story : 5

$DL = 0.150(9706.67 - 2780.40) + 0.0082(2780.40)$
$DL = 1061.7$ Kip

6) Story : 6

$DL = 0.150(9706.67 - 2783.40) + 0.0082(2783.40)$
$DL = 1061.3$ Kip

7) Story : Roof

$DL = 0.150(5079.00)$
$DL = 761.85$ Kip

e) Live Load w/o Live Load Reduction

Room Type	Load (lb/ft²)	Notes
Stairs	100	ASCE 7-05 Table 4-1
Lobby & First Floor Corridor	100	↓
Corridor Above First Floor	80	
Ordinary Flat Roofs	20	

* Partitions : 15 lb/ft², per ASCE 7-05 4.2.2

1) Level : 0

$LL = 0.100(A_{gross} - A_{el\ opening} - A_{stairs}) + 0.100(A_{stairs})$
$LL = 0.100(24153 - 293 - 724) + 0.100(724)$
$LL = 2313.6$ Kip

2) Level : 1

$LL = 0.080(26440 - 1571.00 - 609.00) + 0.100(609.00)$
$LL = 2001.7$ Kip

3) Level : 2 → 5

$LL = 0.080(26440 - 293.00 - 609.00) + 0.100(609.00)$
$LL = 2103.9$ Kip

→

f) Rain Load

Rainfall Rate(I): 4.5" per hour (100 year return period); per International Plumbing Code 2009 Appendix B, ASCE 7-05 C8.5

(A) = 52 * 60.17 = 3128.7, per ASCE 7-05 C8.5
(Q) = 0.0104(A)(I) = 146.42, per ASCE 7-05 C8.3

$d_s = 2\frac{5}{8} + 4(\frac{1}{4}) = 3.63''$

$d_h = 1 + \left[\frac{(Q-80)}{(170-80)}\right] = 1.738''$, interpolation of ASCE 7-05 Table C8-1

$R = 5.2(d_s + d_h)$
$R = 5.2(3.63 + 1.738)$
$R = 27.89\ lb/ft^2 > (\text{Roof live load} = 20\ lb/ft^2)$

→

Appendix C: Gravity Spot Check

Thaison Nguyen		Gravity Spot Check	1/5

Member Type	Typical Span (ft)	Typical Spacing (ft)	Location
Beam	33	8.25	B1 → B2
Girder	33	33	B2 → C2
Joist	28.67	5.5	B1 → B2

a) Roof and Floor Deck, Joists

Load Combination: $1.2D + 1.6L + 0.5(L_r \text{ or } R \text{ or } S)$

	Roof Deck[1]	Floor Deck[1]	Joist
l_{span} (ft)	5.5	8.25	28.67
Spacing (ft)	N/A	N/A	5.5

[1] Assume 3 span decks

1) Roof Deck

* Assume 2 hr. fire rating.

Total Load (TL) = DL + LL + R

$TL = 79.9 + w_{DECK} + 27.98$

$TL = 107.9 \text{ lb/ft}^2 + w_{DECK}$

$DL = 0.113(6.25/12) + 0.015 + 0.001 + 0.005 + w_{DECK}$

$DL = 0.0799 \text{ kip/ft}^2 + w_{DECK}$

$DL = 79.9 \text{ lb/ft}^2 + w_{DECK}$

Check 1.5B24 (using Vulcraft 2008 Manual)

* Since roof live load = 20 lb/ft² is smaller than Rain load (27.98 lb/ft²) and unlikelines of work performed on roof during rain → Use Rain load

Max SDI Span = 5'-10" > 5'-6" ✓, Good.

Max Allowable Load = 128 lb/ft²

$TL = 107.9 + 1.46$

$TL = 109.4 \text{ lb/ft}^2 < 128 \text{ lb/ft}^2$ ✓, Good

Load Causing $l/180 = 4/3 * 90$

Load Causing $l/180 = 120 \text{ lb/ft}^2 > 109.4 \text{ lb/ft}^2$ ✓, Good.

* Servicability Criteria

$\Delta \le l/180$, Supporting Non-Plaster Ceiling

* Un-protected deck is rated up to 2 hrs ✓, Good.

May use 1.5B24

2) Floor Deck

* Assume 2hr fire rating
* Assume floor deck is composite type

LL = 100 lb/ft², areas close to stairs

Check 2VLI22 using Vulcraft 2008 Manual

Weight of deck = 1.62 lb/ft²

Max SDI span = 8'-11" > 8'-3" ✓, Good

Max Superimposed Live Load = 153 lb/ft² > 100 lb/ft² ✓, Good

→

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Thaison Nguyen | Gravity Spot Check | 2/5

• Use Cementitious or sprayed fiber fire protection to achieve 2 hr rating

May use 2VLI22 w/ either cementitous or spray fiber protection.

3) Joists

$W_u = 1.2DL + 0.5R$
$W_u = [1.2(71.5 + W_{Joist}) + 0.5(27.89)]\ 5.5$
$W_u = [99.8\ lb/ft^2 + 1.2(W_{Joist})]\ 5.5$
$W_u = 548.9\ lb/ft + 6.6\,W_{Joist}$

$DL = 0.150(4/12) + 0.015 + 0.005 + W_{deck} + W_{Joist}$
$DL = 70\ lb/ft + 1.46 + W_{Joist}$
$DL = 71.5\ lb/ft + W_{Joist}$

* Since roof live load = 20 lb/ft² is smaller than Rain load (27.89 lb/ft²) and unlikeliness of work performed on roof during rain → use Rain load

Check 22K6 using SJI Economy Table
* Assume 2 hr. fire rating
$W_u = 548.9 + 6.6(9.2)$, $W_{Joist} = 9.2\ lb/ft$
$W_u = 609.6\ lb/ft$

$W_{u,capacity} = (29-28.67)(540-597) + 597$
$W_{u,capacity} = 611.2\ lb/ft > 609.6\ lb/ft$ ✓, Good

$LL_{capacity} = [(29-28.67)(328-295) + 295]\ ^{360}/_{180}$
$LL_{capacity} = 611.8\ lb/ft > 27.89(5.5)$

$611.8\ lb/ft > 153.4\ lb/ft$ ✓, Good

* Serviceability Criteria
$\Delta \le l/180$, supporting Non Plaster ceiling

* Use spray applied fire resistive materials (ex. Cementitious or fiber) to achieve 2hr. rating, per SJI

May use 22K6 w/ spray applied fire resistive materials

b) Beam, Girders

Load Combination: $1.2D + 1.6L + 0.5(L_r$ or R or $S)$
* Assume beams and girders are pinned connected, A992 Gr 50

1) Beam
$W_u = [1.2(DL) + 1.6(LL)] *$ Spacing of bm
$W_u = [1.2(71.6) + 1.6(80)] * 8.25 + 1.2(W_{bm})$
$W_u = 1765\ lb/ft + 1.2\,W_{bm}$

$DL = 0.150(4/12) + 0.015 + 0.005 + W_{bm} + W_{Deck}$
$DL = 71.6\ lb/ft^2 + W_{bm}$
$LL = 80\ lb/ft^2$

$M_u = W_u l^2/8$
$M_u = (1765 + 1.2\,W_{bm})(33^2)/8$
$M_u = 240251 + 163.4\,W_{bm}$

$V_u = W_u l/2$
$V_u = (1765 + 1.2\,W_{bm})(33/2)$

→

Thaison Nguyen | Gravity Spot Check | 3/5

$V_u = 29123 + 19.8\ W_{bm}$

Check W14x74 using AISC 14Ed. Table 3-10, 3-6

$M_u = 240261 + 163.4(74)$
$M_u = 252.4$ kip·ft

$\phi M_n = 272.0$ kip·ft > 252.4 kip·ft ✓, Good.

$V_u = 29123 + 19.8(74)$
$V_u = 30.6$ kip

$\phi V_n = 192$ kip > 30.6 kip ✓, Good.
May use W14x74

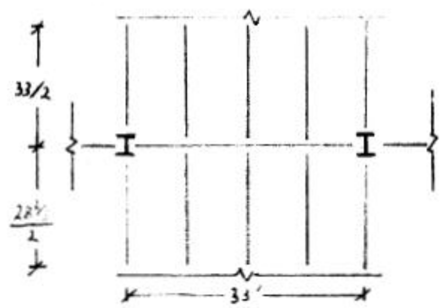
$\Delta_{TL} \le \ell/240$
$\Delta_{TL} \le 33(12)/240$
$\Delta_{TL} \le 1.65''$

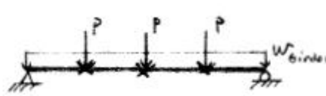
$W_T = (DL + LL)$
$W_T = 8.25(71.6 + 80) + W_{bm}$
$W_T = 1250.7 + W_{bm}$
$\Delta_{TL} = \dfrac{5(1250.7 + 74)(33^4)}{384(29 \times 10^6)(795)}$
$\Delta_{TL} = 1.53'' < 1.65''$ ✓, Good

2) Girder
* Assume girders use shear studs to have composite action.
* For ease in constructability assume all beams are W16x89

$\ell_{brace} = 0$

$M_u = \frac{33}{4}(P_u)(1.5) + 1.2 W_{Girder}\left(\frac{33^2}{8}\right)$
$M_u = \frac{33}{4}(98.4)(1.5) + 1.2 W_{Girder}\left(\frac{33^2}{8}\right)$
$M_u = 1217.7 + 204.2\ W_{Girder}$ ← in kip





$P_D = [0.150(4/12) + 0.015 + 0.005 + 1 + 2](8.25)(33 + 28\frac{2}{3})/2 + 0.089(33 + 28\frac{2}{3})/2$
$P_D = 52.1 + 2.7$
$P_D = 54.8$ kip, unfactored Dead Load

$P_L = 0.080(8.25)(33 + 28\frac{2}{3})/2$
$P_L = 20.4$ kip, unfactored Live Load

$P_u = 1.2 P_D + 1.6 P_L$
$P_u = 1.2(54.8) + 1.6(20.4)$
$P_u = 98.4$ kip

$W_u = 1.2\ W_{Girder}$

$b_{eff} = \min \begin{cases} 2 \cdot \ell/8 \\ (\text{Spacing 1} + \text{Spacing 2})\ 1/2 \end{cases}$

$b_{eff} = \min \begin{cases} 8.25' \\ 30.8' \end{cases} = 8.25'$

$A_s f_y = 0.85 f'_c b_{eff} a$, assume $f'_c = 4000$ psi
$a = \dfrac{50 A_s}{0.85(4)(8.25)(12)}$
$a = 0.149 A_s$, if neutral axis (Plastic) is in conc.

→

Thaison Nguyen | Gravity Spot Check | 4/5

Check W24x76 using AISC 14Ed. Table 3-19, Table 1-1

* Assume perfect shear transfer

$A_s = 22.4\ in^2$

$a = 0.149(22.4)$

$a = 3.34'' > 3''$ (Solid part of floor slab), PNA is in flange of STL. member

$A_s f_y - 0.85 f'_c b_{eff} T_{solid} + 2 F_y b_f x$

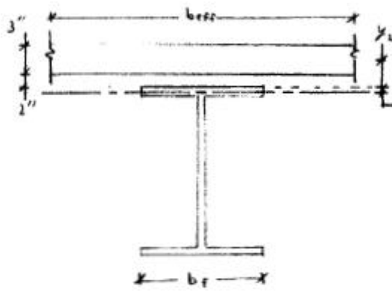
$\frac{A_s f_y - 0.85 f'_c b_{eff} T_{solid}}{2 f_y b_f} = x$

$x = \frac{22.4(50) - 0.85(4)(8.25 \cdot 12)(3)}{2(50)(8.99)}$

$x = 110.2 / 899$

$x = 0.123''$

$y_1 = x$



$M_u = 1217.7 + 204.2(76/1000)$

$M_u = 1233.2$ kip-ft

$I_{LB} = \frac{(0.17 - 0.123)(4770 - 4580)}{0.17} + 4580$

$I_{LB} = 4632.5\ in^4$

$\phi M_n = \frac{(0.17 - 0.123)}{0.17} \cdot (1300 - 1260) + 1260$, interpolation of Table 3-19

$\phi M_n = 1271.1$ kip-ft > 1233.2 kip-ft ✓, Good

$\Delta_{LL} \le \ell/360$, final live Load

$\Delta_{LL} = \frac{5(P_L/33)(33^4)(1728)}{384(29000)(4632.5)}$

$\Delta_{LL} = 0.123'' < 1.1''$ ✓, Good.

May use W24x76 w/ Shear Studs (Composite Action, Partial)

$\Delta_{LLD} \le \ell/360$

$\Delta_{LLD} \le 33(12)/360$

$\Delta_{LLD} \le 1.1''$

$P_{constr} = [0.150(4/12) + 0.005 + 1.52](8.25)(33 + 28^{1/3})/2 + 0.089(33 + 28^{1/3})/2$

$P_{constr} = 48.3 + 2.7 = 51.0$ Kip

$W_{Girder} = 0.076$ Kip/ft

$\Delta_{LLD} = \frac{5(0.076)(33^4)(1728)}{384(29000)(2100)} + \frac{5(51/33)(33^4)(1728)}{384(29000)(2100)}$

$\Delta_{LLD} = 0.033 + 0.677$

$\Delta_{LLD} = 0.71''$, during construction

$0.71'' < 1.1''$ ✓, no shoring req.

→

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C) Column

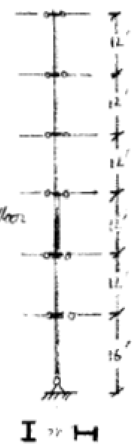
Location: B-2
*Assume pinned base

K = 1

$P_L = 0.080(A_{trib})(5\ floors)$
$P_L = 0.080(990.5)(5)$
$P_L = 396.2$ Kip , live load w/o reduction

$A_{trib} = \frac{(31.25+33)}{2} * \frac{(33+28^{2/3})}{2}$
$A_{trib} = 990.5\ ft^2/floor$

$P_R = 0.02789(990.5)(1)$
$P_R = 27.6$ Kip , rain load



Dead Load Components [2]	Weight	Notes
LW CONC.	113 lb/ft^3	Arch Graphics Standard 11 Ed.
Roof Deck	1.46 lb/ft^2	Vulcraft 2008 Deck Manual, 1.5B24
Joist	9.2 lb/ft	Vulcraft 2008 Joist Manual, 22K6
3 PLy - Roofing	1 lb/ft^2	AISC 14 Ed Table 17-13
NW CONC.	150 lb/ft^3	AISC 14 Ed Table 17-13
Floor Deck	1.62 lb/ft^2	Vulcraft 2008 Deck Manual, 2VLI 22
Beam	74 lb/ft	AISC 14 Ed , W14 x 74
Girder	76 lb/ft	AISC 14 Ed , W24x76
MEP	15 lb/ft^2	

[2] 5 lb/ft^2 per floor level , dead load collateral to be included

$P_D = 113(7/12)(990.5) + 1.46(990.5) + 9.2(33+28^{2/3})(0.5)(5.5) + 1(990.5)$
$+ [150(4/12)(990.5) + 1.62(990.5) + 74(33+28^{2/3})(0.5)(3.5) + 76(33+31.25)(0.5)]5$
$+ 15(990.5)(6) + 5(990.5)(6)$
$P_D = 69.3 + 61.6(5) + 14.9(6) + 5.0(6)$
$P_D = 496.7$ Kip , dead load

$P_{TL} = 1.2P_D + 1.6P_L + 0.5P_R$
$P_{TL} = 1243.8$ Kip

$Kl_x = 1(16) = 16'$
$Kl_y = 1(16) = 16'$, weak axis bending controls.

Check W14x120 using Table 4-1 in AISC 14Ed

$\phi P_n = 1310$ kip > 1243.8 kip ✓ , Good

May use W14x120 for Column B-2

Appendix D: Wind Load Calculations

Thaison Nguyen | | Wind Load | 1/2

Importance Category : III , ASCE 7-05 Table 1-1
Importance Factor (I) : 1.15 , ASCE 7-05 Table 6-1
Exposure Category : B , ASCE 7-05 §6.5.6.3
Mean Height : 95.5'

Building Face	North	South	East	West	Roof
A_{facade} (ft^2)	21412.9	21412.9	10957.9	10957.9	26440

*** When comparing lateral force resisting shear wall and floor diaphragm rigidity, it is assume that the floor diaphram will crack first

V = 130 mi/hr , ASCE 7-05 Figure 6-1

Component	MWFRS	CCL[1]	Notes
K_d	0.85	0.85	ASCE 7-05 Table 6-4

[1] Components and Cladding

Height (ft)	K_z		Notes
	Case I: CCL	Case II: MWFRS	
≤15	0.7	0.57	ASCE 7-05 Table 6-3
20	0.7	0.62	
25	0.7	0.66	
30	0.7	0.7	
40	0.76	0.76	
50	0.81	0.81	
60	0.85	0.85	
70	0.89	0.89	
80	0.93	0.93	
90	0.96	0.96	
100	0.99	0.99	
120	1.04	1.04	

$K_{zt} = 1$, no ridges or escarpments at site

$GC_{pi} = \pm 0.18$, ASCE 7-05 Figure 6-5

$a = \max\begin{cases} 0.1 * \text{Least Horizontal Dimension} \\ 3' \end{cases}$, ASCE 7-05 Figure 6-17

$a = \max\begin{cases} 0.1(117.42) \\ 3' \end{cases}$

$a = 11.74'$

Wind Perpendicular To:	North/South Wall	East/West Wall
3(ft)	229.5	117.42

→

Thaison Nguyen | | Wind Loads | 2/2

	Windward	Leeward		Side	Roof			Notes
L/B		1.95	0.51					ASCE7-05 Fig.6-6
Distance from Windward Edge					[0,95'-6"]	[95'-6", 191'-0")	[191'-0", ∞)	ASCE7-05 Fig.6-6, §6.5.11.3
C_p, MWFRS	0.8	0.3	0.5	0.7	0.9, 0.18	0.5, 0.18	0.3, 0.18	↓

Wall Perpendicular to: North and South Facing Walls										Notes
Wall	Windward		Leeward		Side		Roof			
Zone	4	5	4	5	4	5	1	2	3	
Area (ft^2)	19170	2243	11190	2243	8715	2243	18845	5941	1654.4	
GC_p, CCL	0.6	0.6	0.7	1	0.7	1	0.9	1.6	2.3	ASCE 7-05 Fig 6-17

Wall Perpendicular to: East and West Facing Walls										Notes
Wall	Windward		Leeward		Side		Roof			
Zone	4	5	4	5	4	5	1	2	3	
Area (ft^2)	8715	2243	8715	2243	19170	2243	15845	5941	1654.4	
GC_p, CCL	0.6	0.6	0.7	1	0.7	1	0.9	1.6	2.3	ASCE 7-05 Fig 6-17

Building Natural Frequency (n_1) ~ 100 / Mean Height, ASCE C6.5.8, Eq C6-17
Building Natural Frequency (n_1) ~ 1.047
$T_{n,\,lower bound} = 75/(height = 86) = 0.87$ sec, per ASCE7-05 Eq C6-18
$T_{n,avg} = 100/(height = 86) = 1.16$ sec, per ASCE 7-05 Eq C6-17

$G_f = 0.85$, conservative rigid diaphragms

$q_z = 0.00256\, K_z K_{zt} K_d V^2 I$, see excel table following this page

$q_h = 0.00256\,(0.99)(1)(0.85)(130^2)(1.15)$; leeward and side walls
$q_h = 41.9\ lb/ft^2$

$P_{MWFRS} = q G_f C_p - q_i (GC_{pi})$, $q_i = q_h$ for conservative internal pressurization.

$P_{CCL} = q(GC_p) - q_i (GC_{pi})$, $q_i = q_h$ for conservative internal pressurization

* See excel table following this page for MWFRS and CCL Wind loads.

$V_{base} = \Sigma$ Wind Load at floor diaphragm
$V_{base,\ wind \perp North/South\ Wall} = 916.2$ Kip
$V_{base,\ wind \perp East/West\ Wall} = 363.3$ Kip

$M_{Tot\ overturn,\ wind \perp North/South\ Wall} = 47192.8$ Kip-ft
$M_{Tot\ overturn,\ wind \perp East/West\ Wall} = 18152.9$ Kip-ft

* Load distribution is on excel sheet following this page

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Design Wind Pressures (lb/ft^2)											
Height (ft)	Velocity Pressure q_z (lb/ft^2)		MWFRS								
			qG_fC_p								$q_i(GC_{pi})$, Conservative
	CCL	MWFRS	Windward	Leeward		Side	Roof				
				B/L = 1.95	B/L = 0.51		Cp = 0.3	Cp = 0.5	Cp =0.9	Cp = 0.18	
≤ 15	29.6	24.1	16.39	10.68	17.79	24.91	10.68	17.79	32.03	6.41	7.54
20	29.6	26.2	17.83								
25	29.6	27.9	18.98								
30	29.6	29.6	20.13								
40	32.1	32.1	21.86								
50	34.3	34.3	23.29								
60	35.9	35.9	24.44								
70	37.6	37.6	25.59								
80	39.3	39.3	26.74								
90	40.6	40.6	27.61								
100	41.9	41.9	28.47								
120	44.0	44.0	29.91								

External Wind Forces								
Floor Level	Elevation (ft)	Mid Elevation (ft)	Wind Load on Floor Diaphram (kip)		Story Shear (kip)		Story Overturning Moment (kip-ft)	
			Wind Perpendicular to North/ South Wall	Wind Perpendicular to East/ West Wall	Wind Perpendicular to North/ South Wall	Wind Perpendicular to East/ West Wall	Wind Perpendicular to North/ South Wall	Wind Perpendicular to East/ West Wall
0	0.0	8	62.76	25.43	916.18	363.28	0.00	0.00
1	16.0	23	121.12	49.43	853.41	337.85	1937.85	790.87
2	30.0	37	124.09	51.79	732.30	288.42	3722.82	1553.67
3	44.0	51	131.29	55.47	608.20	236.63	5776.66	2440.65
4	58.0	65	137.03	58.41	476.92	181.16	7947.72	3387.61
5	72.0	79	141.78	60.84	339.89	122.76	10208.27	4380.35
Roof 1	86.0	95.5	168.50	47.45	198.10	61.92	14490.78	4080.71
Top	105.0		29.61	14.47	29.61	14.47	3108.66	1519.07

Design Wind Pressures (lb/ft^2)													
Height (ft)	Velocity Pressure q_z (lb/ft^2)		CCL										
			q(GC$_p$), Wind Perpendicular to North/South Wall										q$_i$(GC$_{pi}$), Conservative
	CCL	MWFRS	Windward		Leeward		Side		Roof				
			Zone 4	Zone 5	Zone 4	Zone 5	Zone 4	Zone 5	Zone 1	Zone 2	Zone 3		
$\leq$ 15	29.6	24.1	17.76	17.76	29.31	41.87	29.31	41.87	37.68	66.99	96.30	7.54
20	29.6	26.2	17.76	17.76								
25	29.6	27.9	17.76	17.76								
30	29.6	29.6	17.76	17.76								
40	32.1	32.1	19.28	19.28								
50	34.3	34.3	20.55	20.55								
60	35.9	35.9	21.57	21.57								
70	37.6	37.6	22.58	22.58								
80	39.3	39.3	23.60	23.60								
90	40.6	40.6	24.36	24.36								
100	41.9	41.9	25.12	25.12								
120	44.0	44.0	26.39	26.39								

Design Wind Pressures (lb/ft^2)													
Height (ft)	Velocity Pressure q_z (lb/ft^2)		CCL										
			q(GC$_p$), Wind Perpendicular to East/West Wall										q$_i$(GC$_{pi}$), Conservative
	CCL	MWFRS	Windward		Leeward		Side		Roof				
			Zone 4	Zone 5	Zone 4	Zone 5	Zone 4	Zone 5	Zone 1	Zone 2	Zone 3		
$\leq$ 15	29.6	24.1	17.76	17.76	29.31	41.87	29.31	41.87	37.68	66.99	96.30	7.54
20	29.6	26.2	17.76	17.76								
25	29.6	27.9	17.76	17.76								
30	29.6	29.6	17.76	17.76								
40	32.1	32.1	19.28	19.28								
50	34.3	34.3	20.55	20.55								
60	35.9	35.9	21.57	21.57								
70	37.6	37.6	22.58	22.58								
80	39.3	39.3	23.60	23.60								
90	40.6	40.6	24.36	24.36								
100	41.9	41.9	25.12	25.12								
120	44.0	44.0	26.39	26.39								

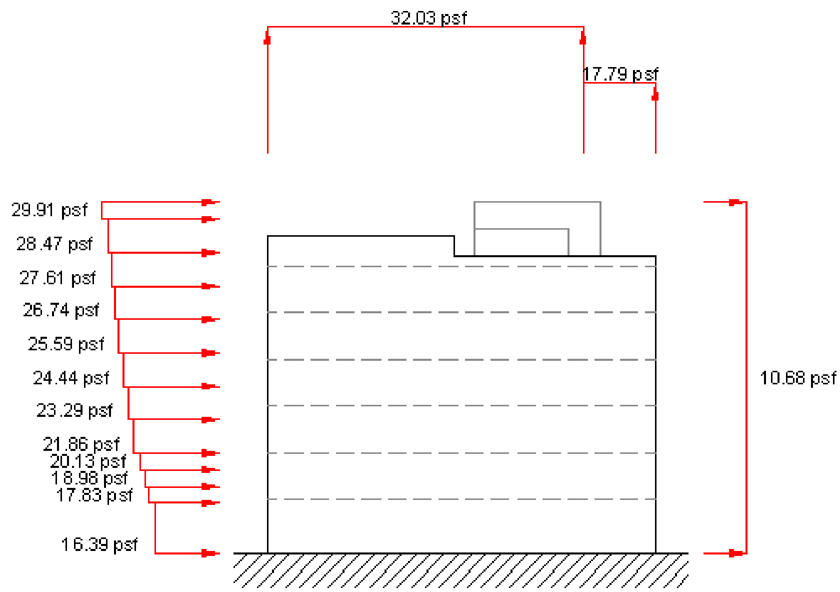

Figure AD.1, MWFRS North/South Wind Load Distribution

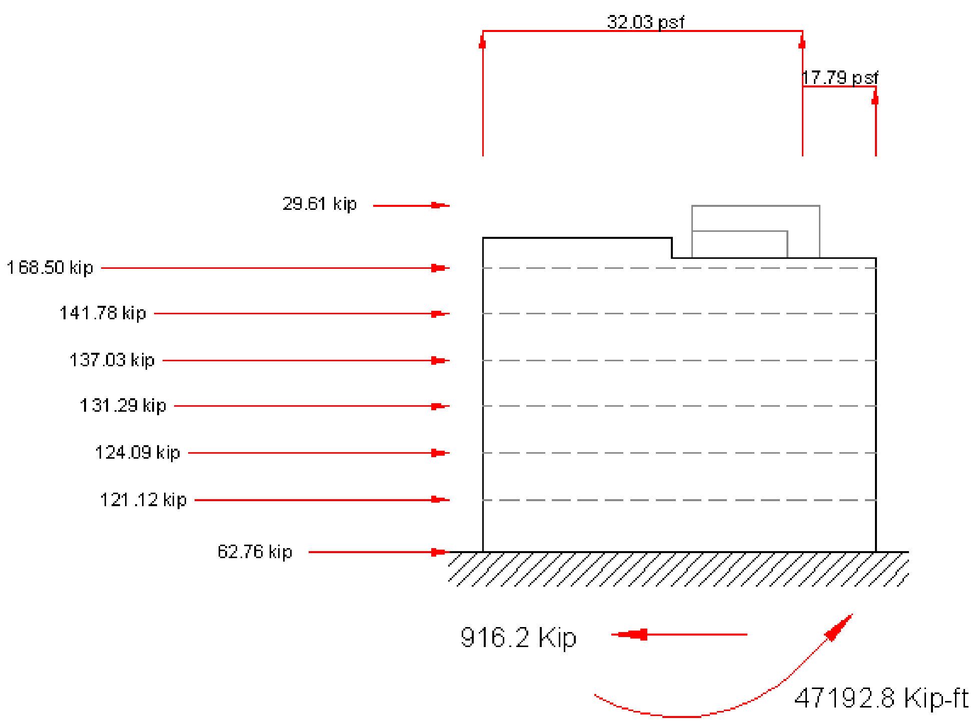

Figure AD.2, MWFRS Loads – North/South

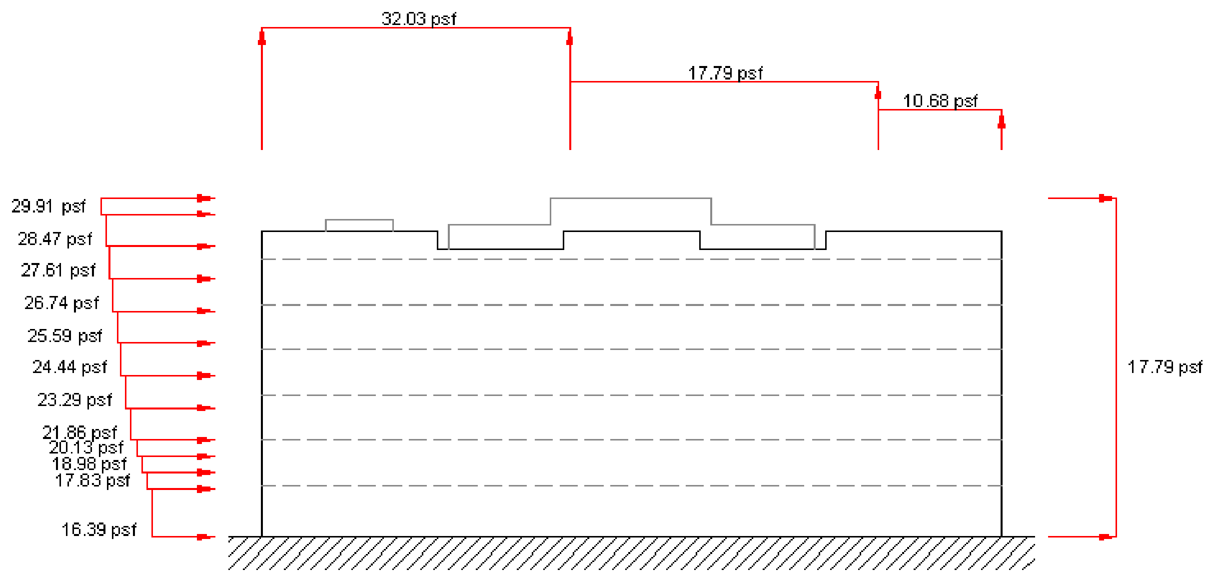

Figure AD.3, MWFRS East/West Wind Load Distribution

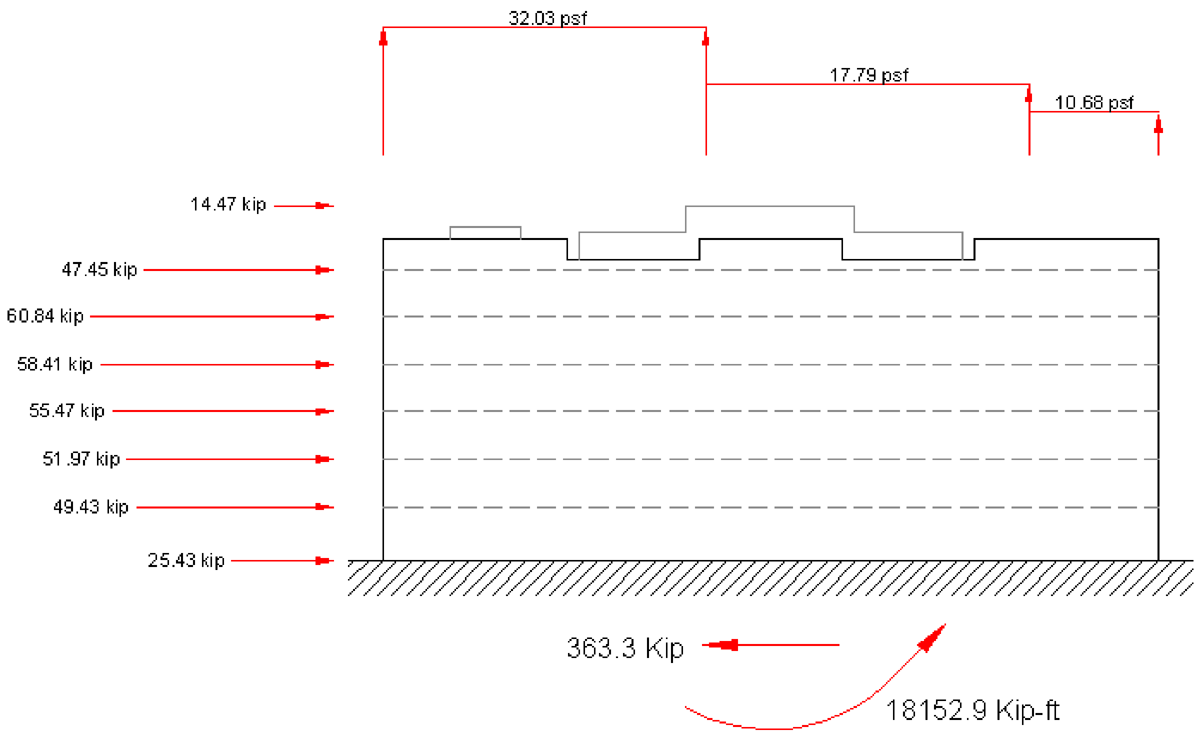

Figure AD.4, MWFRS Loads – East/West

Appendix E: Seismic Load Calculations

Thaison Nguyen | Seismic Loads | 1/4

Importance Category : III , ASCE 7-05 Table 1-1
Importance Factor: 1.25 , ASCE 7-05 Table 11.5-1
Site Class : D , ASCE 7-05 §11.4.2, 20.3.3 ; Table 20.3.1

*** Assume ordinary reinforced concrete shear walls → Lateral System

a) Effective Building Weight ($w_x = DL + 0.25LL$)

1) Level: 1

$DL = DL_{slab} + DL_{deck} + DL_{bm} + DL_{girder} + DL_{flooring} + DL_{envelope}$

$DL = 1675.5 + \frac{1.62}{1000}(A_{gross} - A_{flopening} - A_{stair}) + 217.6 + 74.8 + 7.2 + \left(\frac{1338.8 + 1183.7}{2}\right)$

$DL = 1675.5 + 39.3 + 217.6 + 74.8 + 7.2 + 1261.25$

$DL = 3325.7$ kip

$LL = 2001.7$ kip , value for Load Determination - DEAD, LIVE, RAIN section in the Appendix.

$w_x = 3325.7 + 0.25(2001.7)$

$w_x = 3826.1$ kip

$DL_{bm} = \frac{w_{bm}}{Spacing} * (A_{gross} - A_{flopening} - A_{stair})$

$w_{bm} = 74$ lb/ft , W14×74 from spot check

$DL_{bm} = \frac{74}{8.25}(26440 - 1571 - 609)$

$DL_{bm} = 8.97(24260)$

$DL_{bm} = 217.6$ kip

$DL_{girder} = l_{girder} w_{girder}$

$w_{girder} = 76$ lb/ft , W24×76 from spot check

$DL_{girder} = \{[31.25(2) + 33(4) + 32\tfrac{1}{6}]3 + [29.25(2) + 33 + (33-9) + 32\tfrac{1}{6} + (33-8.5) + 33(4)]\} * 76$

$DL_{girder} = [680 + 304.2] * 76$

$DL_{girder} = 74.8$ kip

2) Level: 2→5

$DL = \{1822.6 + \frac{1.62}{1000}(A_{gross} - A_{flopening} - A_{stair}) + 8.97(A_{gross} - A_{flopening} - A_{stair}) + \frac{76}{1000}(680 + 304.2 + 29.25 + 32\tfrac{1}{6}) + 4.1\}4 + \left[\frac{1}{2}(1183.7) + 1194.2 + 1073.7 + 1061.7 + \frac{1}{2}(1761.3)\right]$

$A_{gross} - A_{flopening} - A_{stair} = 26440 - 293 - 609 = 25538$ ft²

$DL = 8706.5 + 4452.3$

$DL = 13158.8$ kip

$LL = 2103.9(4)$, value from Load Determination - DEAD, LIVE, RAIN section in the Appendix.

$LL = 8415.6$ kip

→

Floor Level	$DL_{envelope}$ (Kip)	DL (Kip)	W_x
2	1188.95	3365.6	3891.6
3	1133.95	3310.6	3836.6
4	1067.70	3244.4	3770.4
5	1061.50	3238.2	3764.2

3) Level : Roof

$$DL = 1794.1 + \frac{1.46(26440)}{1000} + \frac{9.2(26440)}{5.5(1000)} + \frac{76}{1000}(680 + 304.2 + 29.25 + 32\tfrac{1}{6}) + \left[\frac{1}{2}(1061.3) + 761.85\right]$$

$$LL = \frac{20}{1000} * 26440$$

$LL = 528.8$ Kip

$DL = 1956.4 + 1292.5$

$DL = 3248.9$ kip

$W_x = 3381.1$ Kip

4) Total Effective Weight

$W_{x,tot} = 3826.1 + 3891.6 + 3836.6 + 3770.4 + 3764.2 + 3381.1$

$W_{x,tot} = 22470$ Kip

b) Equivalent Lateral Load

1) V_{base}

$S_s = \frac{6.3}{100}$, ASCE 7-05 Fig. 22-1

$S_s = 0.063$

$S_1 = \frac{2.2}{1000}$, ASCE 7-05 Fig. 22.2

$S_1 = 0.022$

$F_a = 1.6$, ASCE 7-05 Table 11.4-1

$F_v = 2.4$, ASCE 7-05 Table 11.4-2

$S_{MS} = S_s F_a$

$S_{MS} = 0.063(1.6)$

$S_{MS} = 0.101$

$S_{M1} = S_1 F_v$

$S_{M1} = 0.022(2.4)$

$S_{M1} = 0.053$

→

Thaison Nguyen | Seismic Load | 3/4

$S_{DS} = 2/3\, S_{MS}$
$S_{DS} = 0.067$

$S_{D1} = 2/3\, S_{M1}$
$S_{D1} = 0.035$

Seismic Design Category (Short Period): A, ASCE 7-05 Table 11.6-1
Seismic Design Category (Long Period): A, ASCE 7-05 Table 11.6-2

$h_n = 105'$
$C_T : 0.2$, ASCE 7-05 Table 12.8-2
$x = 0.75$, ASCE 7-05 Table 12.8-2

$T_L = 8$ sec, ASCE 7-05 Fig. 22-15 $T < T_L$
$T \sim C_T h_n^{\,x}$, ASCE 7-05 Eq 12.8-a
$T \sim 0.66$ sec

$R = 5$, ASCE 7-05 Table 12.8-2
$K = 2.5$
$C_s = \dfrac{S_{DS}}{\frac{R}{I}}$, ASCE 7-05 Eq. 12.8-3

$V_{base} = W_{x,tot}\, C_s$
$V_{base} = 22470(0.017)$
$V_{base} = 376.4$ kip

2) Story Shear (V_x) and overturning moment

$C_x = \dfrac{W_x h_x^{\bar{k}}}{\Sigma W_x h_x^{\bar{k}}}$

$\bar{k} = 1 + \dfrac{(T-0.5)}{(2.5-0.5)}(2-1)$, ASCE 7-05 Eq. 12.8-12
$\bar{k} = 1 + \dfrac{(0.66-0.5)}{(2.5-0.5)}$
$\bar{k} = 1.078$

$F_x = C_x V_{base}$, equiv. lateral load at floor level

$\Sigma(W_x h_x^{\bar{k}}) = 1545062.1$

Floor Level	h_x (ft)	W_x	$W_x h_x^{k}$	F_x (kip)	Story Shear (kip)
0	0	0	0	0	376.4
1	16	3826.1	75997.2	18.51	376.4
2	30	3891.6	152217.6	37.08	357.89
3	44	3836.6	226771.5	55.24	320.81
4	58	3770.4	300166.9	73.13	265.57
5	72	3764.2	378335.5	92.17	192.44
Roof 1	86	3381.1	411573.4	100.27	100.27

Thaison Nguyen | Seismic Load | 4/4

Floor Level	h_x (ft)	Overturning Moment = $F_x h_x$ (kip*ft)
0	0	0
1	16	296.16
2	30	1112.40
3	44	2430.56
4	58	4241.54
5	72	6636.24
Roof 1	86	8623.22

$M_{tot,overturning} = 296.16 + 1112.40 + 2430.56 + 4241.54 + 6636.24 + 8623.22$
$M_{tot,overturning} = 23340.1$ kip*ft

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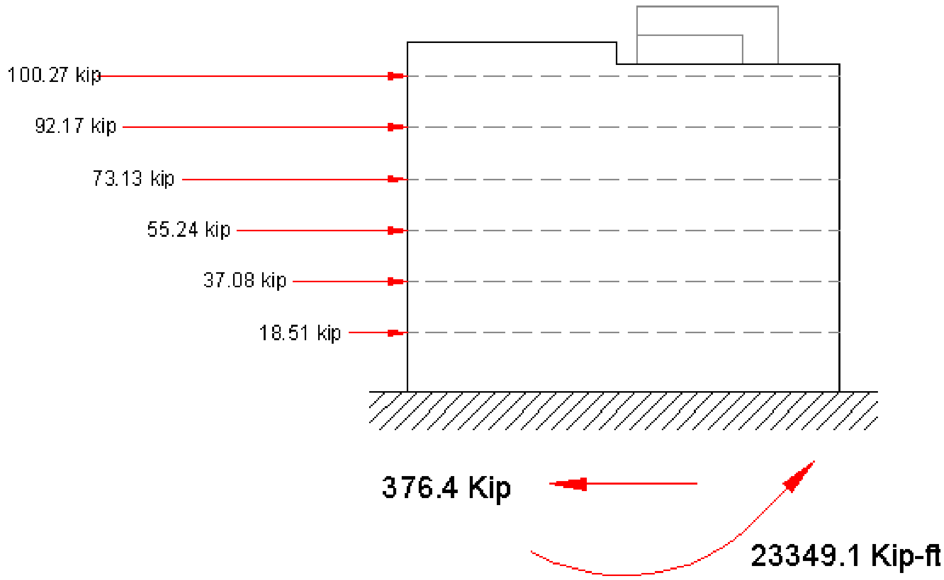

Figure AE.1, Seismic Loads

Appendix F: Irregularity Analysis

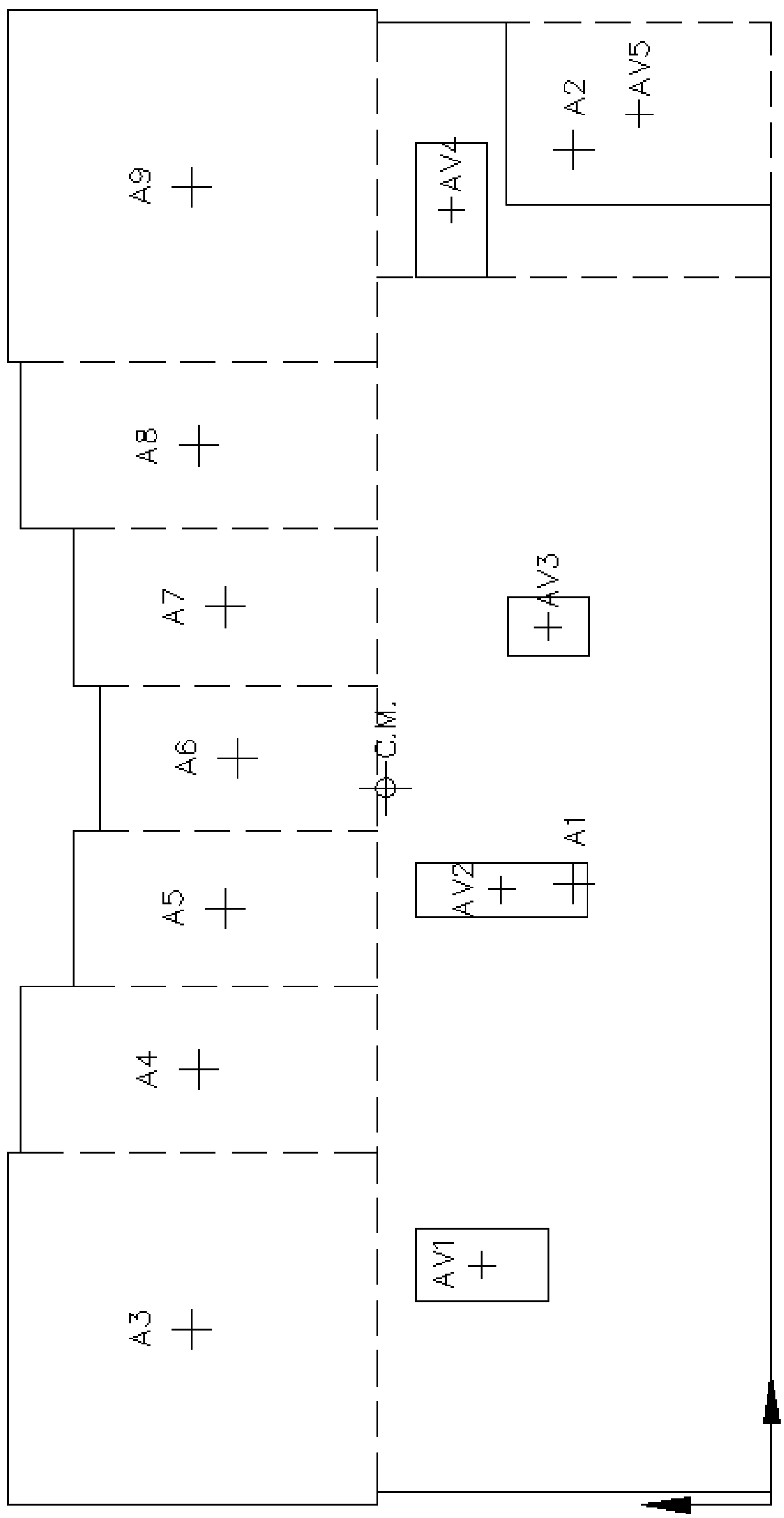

Figure AF.1, Floor Type A Area Divisions and Designations

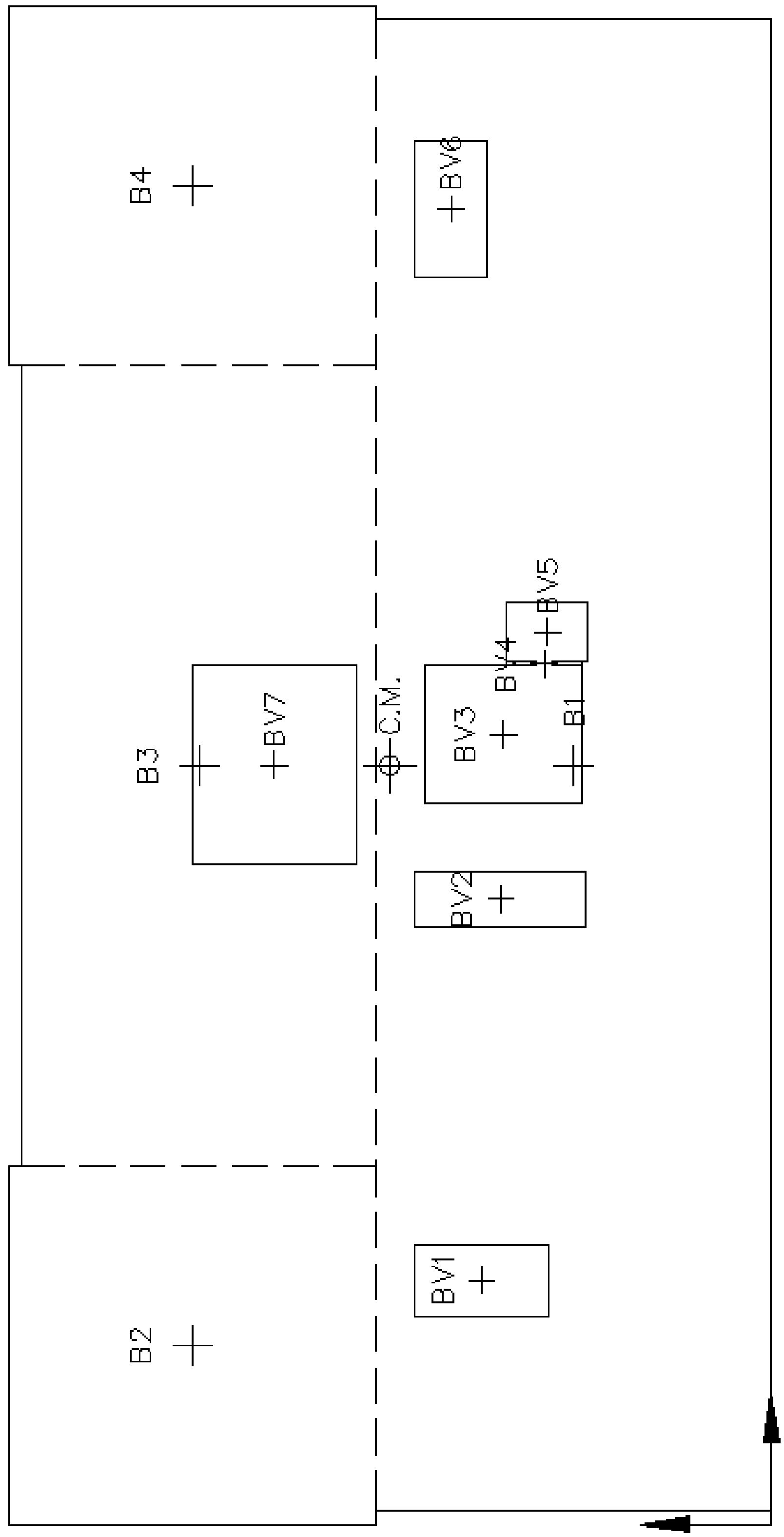

Figure AF.2, Floor Type B Area Divisions and Designations

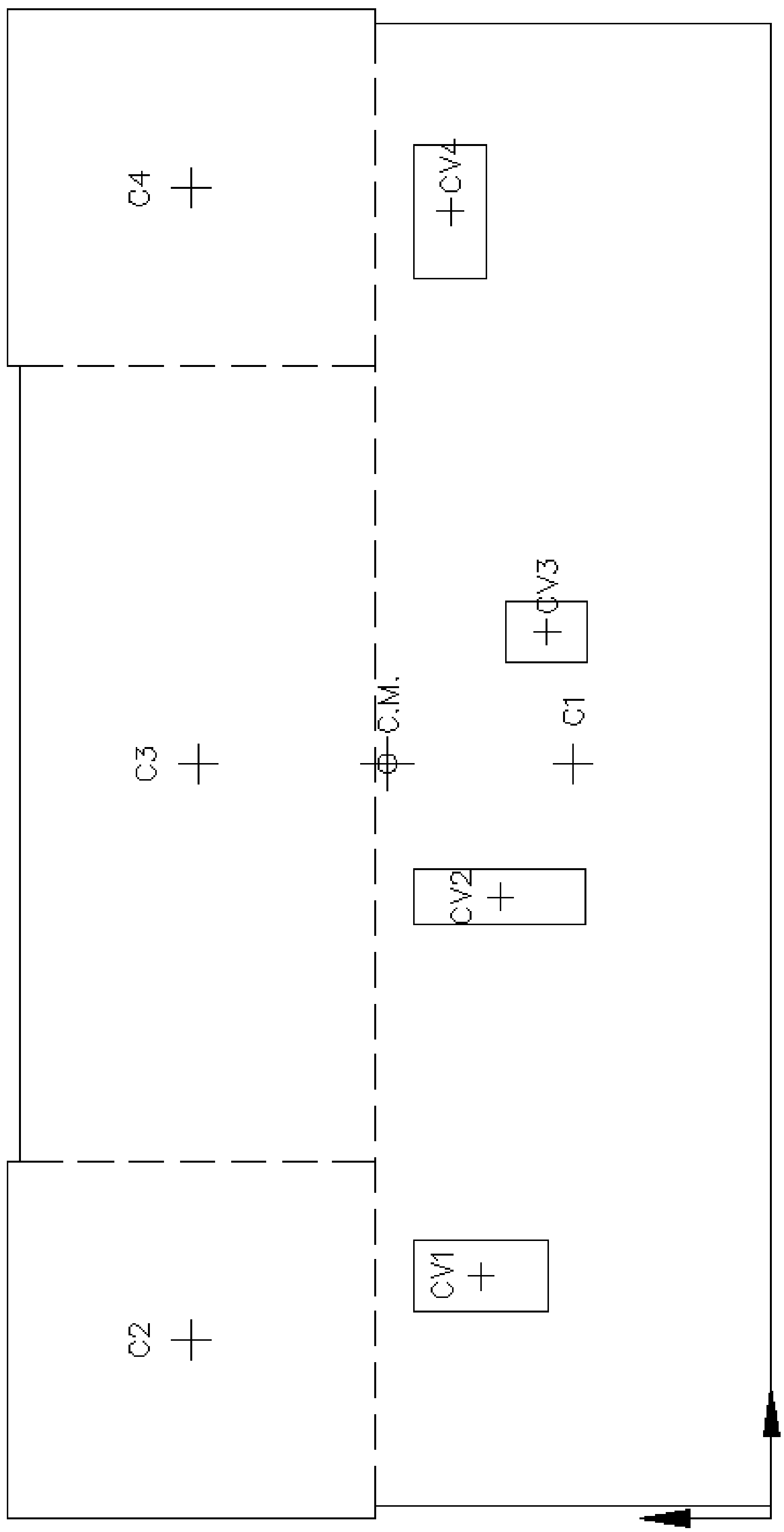

Figure AF.3, Floor Type C Area Divisions and Designations

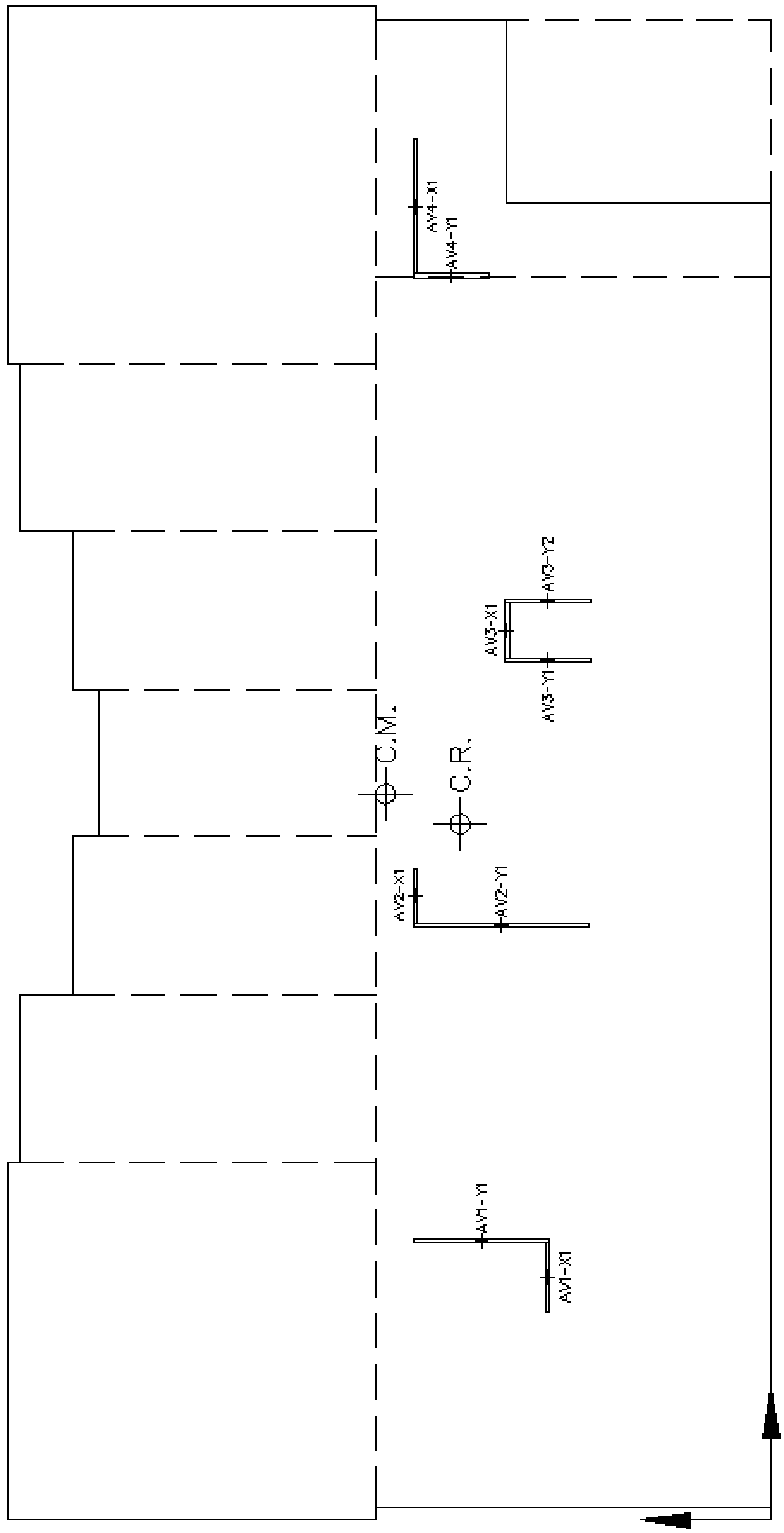

Figure AF.4, Shear Wall Designations

Thaison Nguyen | Irregularity Analysis | 1/7

*** Lateral Load Resisting Structure considered to be rigid (Concrete Shear Walls)

A. Center of Mass and Rigidity

*** Assume all lateral resisting elements have a stiffness proportional to respective length.

Floor Type	Floor Level
A	0
B	1
C	2,3,4,5, Roof 1

Floor Type	Component	Area (ft^2)	Center of Mass	
			X (ft)	Y (ft)
A				
	A1	11324.15	95.31	30.38
	AV1	-224.55	36.84	44.54
	AV2	-223.83	94.51	41.58
	AV3	-113.50	134.88	34.42
	AV4	-224.55	198.83	49.26
	A2	2362.09	206.07	30.38
	AV5	-1143.33	213.51	20.42
	A3	3069.82	27.09	89.09
	A4	1394.00	66.92	88.09
	A5	1115.96	91.63	84.09
	A6	949.17	114.76	82.01
	A7	1115.96	137.88	84.09
	A8	1394.00	162.58	88.09
	A9	3069.82	202.42	89.09
B				
	B1	13701.04	114.76	30.38
	BV1	-224.55	36.84	44.54
	BV2	-223.83	94.51	41.58
	BV3	-503.60	119.39	41.21
	BV4	-5.75	128.09	34.92
	BV5	-113.50	134.88	34.42
	BV6	-224.55	198.83	49.26
	B2	3069.82	27.09	89.09
	B3	6623.78	114.76	82.09
	BV7	-757.99	114.76	76.48
	B4	3069.52	202.42	89.09
C				
	C1	13701.04	114.76	30.38
	CV1	-224.55	36.84	44.54
	CV2	-223.83	94.51	41.58
	CV3	-113.50	134.88	34.42
	CV4	-224.55	198.83	49.26

→

	C2	3069.82	27.09	89.09
	C3	6623.78	114.76	88.09
	C4	3069.82	202.42	89.09

Floor Type	Global Center of Mass	
	x (ft) [1]	y (ft) [2]
A	110.07	59.34
B	114.69	58.72
C	114.79	58.90

[1] $x_{cm} = \dfrac{\Sigma(x_i A_i)}{\Sigma(A_i)}$

[2] $y_{cm} = \dfrac{\Sigma(y_i A_i)}{\Sigma(A_i)}$

Lateral Resisting Elements		Length	Element Center of Rigidity		Global Center of Rigidity	
Designation	Resisting Direction	(ft)	x (ft)	y (ft)	x_r (ft) [3]	y_r (ft) [4]
AV1-X1	x	10.33	36.84	34.33		
AV1-Y1	y	21.08	42.34	44.54		
AV2-Y1	y	27.00	90.26	41.59		
AV2-X1	x	8.17	94.68	54.76		
AV3-Y1	y	13.17	130.34	34.42	105.51	47.79
AV3-X1	x	8.41	134.88	[illegible]		
AV3-Y2	y	13.17	139.42	[illegible]		
AV4-Y1	y	11.67	188.63	49.26		
AV4-X1	x	20.41	[illegible]	54.76		

[3] $x_r = \dfrac{\Sigma(x_i L_y)}{\Sigma(L_y)}$

[4] $y_r = \dfrac{\Sigma(y_i L_x)}{\Sigma(L_x)}$

Floor Type	Eccentricity	
	$\lvert x \rvert$ [5]	$\lvert y \rvert$ [6]
A	4.56	11.55
B	9.18	10.93
C	9.28	11.10

[5] $\lvert x \rvert = \lvert x_{cm} - x_r \rvert$

[6] $\lvert y \rvert = \lvert y_{cm} - y_r \rvert$

→

Lateral Resisting Element	d_i [7] x	d_i [7] y
AV1-X1	-68.67	-13.46
AV1-Y1	-63.17	-3.25
AV2-Y1	-15.25	-6.20
AV2-X1	-10.83	6.96
AV3-Y1	24.83	-13.38
AV3-X1	29.37	-7.13
AV3-Y2	33.91	-13.38
AV4-Y1	83.12	1.46
AV4-X1	93.66	6.96

[7] d_i = Element Center of Rigidity − Global Center of Rigidity

$$\sum(K_{x,i}\, d_{x,i}^2 + K_{y,i}\, d_{y,i}^2) = 197931 \quad , \; K = L$$

Lateral Resisting Elements	$K_i \lvert d_i \rvert d_x / [\sum(K_{x,i} d_{x,i}^2 + K_{y,i} d_{y,i}^2)]$					
	Floor Type: A		Floor Type: B		Floor Type: C	
	x	y	x	y	x	y
AV1-X1	0.0032	0.0081	0.0065	0.0077	0.0065	0.0078
AV1-Y1	0.0031	0.078	0.062	0.074	0.062	0.075
AV2-Y1	0.0095	0.024	0.019	0.023	0.019	0.023
AV2-X1	0.0013	0.0033	0.0026	0.0031	0.0027	0.0032
AV3-Y1	0.0075	0.0191	0.0152	0.0181	0.0153	0.0183
AV3-X1	0.0014	0.0035	0.0028	0.0033	0.0028	0.0034
AV3-Y2	0.0010	0.026	0.021	0.025	0.021	0.025
AV4-Y1	0.022	0.057	0.045	0.054	0.045	0.054
AV4-X1	0.0033	0.0083	0.0066	0.0078	0.0067	0.0080

d_x = eccentricity between Center of Mass and Center of Rigidity.

B. Irregularity and Wind

$e_{acc,long} = 0.15(229.51) = 34.43'$, per ASCE7-05; Figure 6-9 Case II
$e_{acc,short} = 0.15(117.42) = 17.61'$

Floor Level	Wind ⊥ to Long side			Wind ⊥ to Short side		
	Case I	Case II, III	Case IV	Case I	Case II, III	Case IV
0	62.76	47.07	35.30	25.43	19.07	14.30
1	121.12	90.84	68.13	49.43	37.07	27.80
2	124.09	93.07	69.80	51.79	38.84	29.13
3	131.29	98.47	73.85	55.47	41.60	31.20
4	137.03	102.77	77.08	58.41	43.81	32.86
5	141.78	106.34	79.75	60.84	45.63	34.22
Roof 1	168.50	126.58	94.78	47.45	35.59	26.69
Top	29.61	22.21	16.66	14.47	10.85	8.14

→

Thaison Nguyen | Irregularity Analysis | 4/7

Floor Level	$M_{acc,torsion}$, Kip-ft [8]			
	Wind ⊥ to Long Side		Wind ⊥ to Short Side	
	Case II	Case IV	Case II	Case IV
0	1215.32	911.49	251.95	188.96
1	2345.43	1759.07	489.73	367.29
2	2402.94	1802.21	513.11	384.83
3	2542.37	1906.77	549.57	412.18
4	2653.52	1990.14	578.70	434.02
5	2745.50	2059.12	602.77	452.08
Roof 1	3262.92	2447.19	470.11	352.58
Top	573.38	430.04	143.36	107.52

[8] $|M_{acc,torsion}| = |\text{Wind Load} * e_{acc}|$

*See excel on following page for the load on each lateral resisting element per case.

Maximum Wind Base Shear is in element AV1-Y1
Maximum Wind Base Shear = 325 Kips, element AV1-Y1
Maximum Overturning moment on Element = 16608 kip*ft

1) Story Drift

*** Assume concrete remains elastic, for drift calculations.
*** Deflection calculations don't consider creep or other long term effects
*** Assume only 25% of I_g is effective

Lateral Resisting Member	$P_{max-w,level}$ (Kip)							
	0 [9]	1	2	3	4	5	Roof 1	Top
AV1-X1	5.35	10.41	10.90	11.68	12.30	12.81	9.99	3.05
AV1-Y1	21.15	43.63	44.76	47.36	49.43	51.15	60.79	9.10
AV2-Y1	20.28	40.31	41.32	43.72	45.63	47.21	56.11	9.86
AV2-X1	4.47	8.69	9.10	9.75	10.27	10.69	8.34	2.54
AV3-Y1	9.13	16.69	17.08	18.07	18.86	19.51	23.19	4.08
AV3-X1	4.43	8.62	9.03	9.67	10.19	10.61	8.27	2.52
AV3-Y2	8.95	16.02	16.38	17.33	18.09	18.72	22.25	3.91
AV4-Y1	7.10	10.92	11.18	11.82	12.34	12.77	15.18	2.67
AV4-X1	11.18	21.71	22.75	24.37	25.66	26.73	20.54	6.36

[9] Lateral load goes to ground.

$\Delta_{drift} = \Delta_{i+1} - \Delta_i$
$\Delta_{drift} = (P_{i+1}\ell_{i+1}^3 - P_i\ell_i^3)/C$
$\Delta_{drift} = \ell^3(P_{i+1} - P_i)/C$, when $\ell_{i+1} = \ell_i$

$\Delta = P\ell^3/C$
C = Constant = $12EI$, fixed ends.
$E = 57000\sqrt{f'_c}$

→

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Wind Perpendicular to Long Side									
Floor Level	Wind Load Direct Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	0.00	15.37	19.69	0.00	9.60	0.00	9.60	8.51	0.00
1	0.00	29.66	37.99	0.00	18.53	0.00	18.53	16.42	0.00
2	0.00	30.39	38.92	0.00	18.98	0.00	18.98	16.82	0.00
3	0.00	32.15	41.18	0.00	20.08	0.00	20.08	17.79	0.00
4	0.00	33.55	42.98	0.00	20.96	0.00	20.96	18.57	0.00
5	0.00	34.72	44.47	0.00	21.69	0.00	21.69	19.22	0.00
Roof 1	0.00	41.26	52.85	0.00	25.77	0.00	25.77	22.84	0.00
Top	0.00	7.25	9.29	0.00	4.53	0.00	4.53	4.01	0.00

Wind Perpendicular to Short Side									
Floor Level	Wind Load Direct Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	5.55	0.00	0.00	4.39	0.00	4.52	0.00	0.00	10.97
1	10.79	0.00	0.00	8.53	0.00	8.79	0.00	0.00	21.32
2	11.31	0.00	0.00	8.94	0.00	9.21	0.00	0.00	22.34
3	12.11	0.00	0.00	9.57	0.00	9.86	0.00	0.00	23.93
4	12.75	0.00	0.00	10.08	0.00	10.38	0.00	0.00	25.19
5	13.28	0.00	0.00	10.50	0.00	10.81	0.00	0.00	26.24
Roof 1	10.36	0.00	0.00	8.19	0.00	8.43	0.00	0.00	20.47
Top	3.16	0.00	0.00	2.50	0.00	2.57	0.00	0.00	6.24

Wind Perpendicular to Long Side									
Floor Level	Wind Load Torsion Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	-0.20	1.93	0.60	0.08	-0.47	-0.09	-0.65	-1.40	0.21
1	-0.78	7.48	2.31	0.32	-1.84	-0.34	-2.51	-5.45	0.80
2	-0.81	7.75	2.40	0.33	-1.90	-0.35	-2.60	-5.64	0.83
3	-0.86	8.20	2.53	0.35	-2.01	-0.37	-2.75	-5.97	0.88
4	-0.89	8.56	2.65	0.37	-2.10	-0.39	-2.87	-6.23	0.91
5	-0.92	8.85	2.74	0.38	-2.17	-0.40	-2.97	-6.45	0.94
Roof 1	-1.10	10.52	3.25	0.45	-2.58	-0.47	-3.53	-7.66	1.12
Top	-0.19	1.85	0.57	0.08	-0.45	-0.08	-0.62	-1.35	0.20

Wind Perpendicular to Short Side									
Floor Level	Wind Load Torsion Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	-0.21	1.98	0.61	0.08	-0.49	-0.09	-0.66	-1.44	0.21
1	-0.38	3.63	1.12	0.16	-0.89	-0.16	-1.22	-2.65	0.39
2	-0.40	3.87	1.20	0.17	-0.95	-0.17	-1.30	-2.82	0.41
3	-0.43	4.14	1.28	0.18	-1.02	-0.19	-1.39	-3.02	0.44
4	-0.46	4.36	1.35	0.19	-1.07	-0.20	-1.46	-3.18	0.47
5	-0.47	4.54	1.41	0.19	-1.12	-0.20	-1.52	-3.31	0.49
Roof 1	-0.37	3.54	1.10	0.15	-0.87	-0.16	-1.19	-2.58	0.38
Top	-0.11	1.08	0.33	0.05	-0.27	-0.05	-0.36	-0.79	0.12

Wind Perpendicular to Long Side									
Floor Level	Total Wind Load in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	-0.20	17.29	20.28	0.08	9.13	-0.09	8.95	7.10	0.21
1	-0.78	37.14	40.31	0.32	16.69	-0.34	16.02	10.97	0.80
2	-0.81	38.13	41.32	0.33	17.08	-0.35	16.38	11.18	0.83
3	-0.86	40.35	43.72	0.35	18.07	-0.37	17.33	11.82	0.88
4	-0.89	42.11	45.63	0.37	18.86	-0.39	18.09	12.34	0.91
5	-0.92	43.57	47.21	0.38	19.51	-0.40	18.72	12.77	0.94
Roof 1	-1.10	51.78	56.11	0.45	23.19	-0.47	22.25	15.18	1.12
Top	-0.19	9.10	9.86	0.08	4.08	-0.08	3.91	2.67	0.20

Wind Perpendicular to Short Side									
Floor Level	Total Wind Load in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	5.35	1.98	0.61	4.47	-0.49	4.43	-0.66	-1.44	11.18
1	10.41	3.63	1.12	8.69	-0.89	8.62	-1.22	-2.65	21.71
2	10.90	3.87	1.20	9.10	-0.95	9.03	-1.30	-2.82	22.75
3	11.68	4.14	1.28	9.75	-1.02	9.67	-1.39	-3.02	24.37
4	12.30	4.36	1.35	10.27	-1.07	10.19	-1.46	-3.18	25.66
5	12.81	4.54	1.41	10.69	-1.12	10.61	-1.52	-3.31	26.73
Roof 1	9.99	3.54	1.10	8.34	-0.87	8.27	-1.19	-2.58	20.84
Top	3.05	1.08	0.33	2.54	-0.27	2.52	-0.36	-0.79	6.36

Table AF.1, Wind Case I

Wind Perpendicular to Long Side									
Floor Level	Wind Load Direct Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	0.00	11.53	14.76	0.00	7.20	0.00	7.20	6.38	0.00
1	0.00	22.24	28.49	0.00	13.90	0.00	13.90	12.31	0.00
2	0.00	22.79	29.19	0.00	14.24	0.00	14.24	12.61	0.00
3	0.00	24.11	30.89	0.00	15.06	0.00	15.06	13.35	0.00
4	0.00	25.17	32.24	0.00	15.72	0.00	15.72	13.93	0.00
5	0.00	26.04	33.35	0.00	16.27	0.00	16.27	14.41	0.00
Roof 1	0.00	30.95	39.64	0.00	19.33	0.00	19.33	17.13	0.00
Top	0.00	5.44	6.97	0.00	3.40	0.00	3.40	3.01	0.00

Wind Perpendicular to Long Side									
Floor Level	Wind Load Torsion Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	-0.15	1.44	0.45	0.06	-0.35	-0.07	-0.48	-1.05	0.15
1	-0.59	5.61	1.74	0.24	-1.38	-0.25	-1.88	-4.09	0.60
2	-0.61	5.81	1.80	0.25	-1.43	-0.26	-1.95	-4.23	0.62
3	-0.64	6.15	1.90	0.26	-1.51	-0.28	-2.06	-4.48	0.66
4	-0.67	6.42	1.98	0.27	-1.58	-0.29	-2.15	-4.67	0.68
5	-0.69	6.64	2.05	0.28	-1.63	-0.30	-2.23	-4.83	0.71
Roof 1	-0.82	7.89	2.44	0.34	-1.94	-0.36	-2.65	-5.75	0.84
Top	-0.14	1.39	0.43	0.06	-0.34	-0.06	-0.46	-1.01	0.15

Wind Perpendicular to Long and Short Side									
Floor Level	Total Wind Load in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	3.86	14.45	15.67	3.42	6.48	3.26	6.22	4.25	8.54
1	7.22	30.58	31.07	6.75	11.85	6.21	11.10	6.24	16.88
2	7.57	31.50	31.89	7.08	12.10	6.51	11.31	6.27	17.68
3	8.12	33.37	33.75	7.57	12.79	6.98	11.96	6.61	18.93
4	8.55	34.86	35.23	7.97	13.34	7.35	12.47	6.87	19.93
5	8.91	36.09	36.46	8.30	13.80	7.66	12.90	7.09	20.75
Roof 1	6.67	41.49	42.90	6.59	16.74	5.85	15.79	9.45	16.48
Top	2.14	7.64	7.65	1.97	2.86	1.83	2.66	1.41	4.92

Wind Perpendicular to Short Side									
Floor Level	Wind Load Direct Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	4.16	0.00	0.00	3.29	0.00	3.39	0.00	0.00	8.23
1	8.10	0.00	0.00	6.40	0.00	6.59	0.00	0.00	15.99
2	8.48	0.00	0.00	6.70	0.00	6.90	0.00	0.00	16.75
3	9.08	0.00	0.00	7.18	0.00	7.39	0.00	0.00	17.94
4	9.57	0.00	0.00	7.56	0.00	7.79	0.00	0.00	18.90
5	9.96	0.00	0.00	7.87	0.00	8.11	0.00	0.00	19.68
Roof 1	7.77	0.00	0.00	6.14	0.00	6.33	0.00	0.00	15.35
Top	2.37	0.00	0.00	1.87	0.00	1.93	0.00	0.00	4.68

Wind Perpendicular to Short Side									
Floor Level	Wind Load Torsion Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	-0.15	1.48	0.46	0.06	-0.36	-0.07	-0.50	-1.08	0.16
1	-0.28	2.73	0.84	0.12	-0.67	-0.12	-0.91	-1.99	0.29
2	-0.30	2.90	0.90	0.12	-0.71	-0.13	-0.97	-2.11	0.31
3	-0.32	3.11	0.96	0.13	-0.76	-0.14	-1.04	-2.26	0.33
4	-0.34	3.27	1.01	0.14	-0.80	-0.15	-1.10	-2.38	0.35
5	-0.36	3.41	1.05	0.15	-0.84	-0.15	-1.14	-2.48	0.36
Roof 1	-0.28	2.66	0.82	0.11	-0.65	-0.12	-0.89	-1.94	0.28
Top	-0.08	0.81	0.25	0.03	-0.20	-0.04	-0.27	-0.59	0.09

Table AF.2, Wind Case III

Wind Perpendicular to Long Side									
Floor Level	Wind Load Direct Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	0.00	11.53	14.76	0.00	7.20	0.00	7.20	6.38	0.00
1	0.00	22.24	28.49	0.00	13.90	0.00	13.90	12.31	0.00
2	0.00	22.79	29.19	0.00	14.24	0.00	14.24	12.61	0.00
3	0.00	24.11	30.89	0.00	15.06	0.00	15.06	13.35	0.00
4	0.00	25.17	32.24	0.00	15.72	0.00	15.72	13.93	0.00
5	0.00	26.04	33.35	0.00	16.27	0.00	16.27	14.41	0.00
Roof 1	0.00	30.95	39.64	0.00	19.33	0.00	19.33	17.13	0.00
Top	0.00	5.44	6.97	0.00	3.40	0.00	3.40	3.01	0.00

Wind Perpendicular to Long Side									
Floor Level	Wind Load Torsion Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	-1.00	9.62	2.97	0.41	-2.36	-0.43	-3.23	-7.01	1.03
1	-2.23	21.39	6.61	0.91	-5.25	-0.96	-7.17	-15.58	2.28
2	-2.30	21.98	6.79	0.94	-5.40	-0.99	-7.37	-16.00	2.35
3	-2.43	23.25	7.19	0.99	-5.71	-1.05	-7.80	-16.93	2.48
4	-2.53	24.27	7.50	1.04	-5.96	-1.09	-8.14	-17.67	2.59
5	-2.62	25.11	7.76	1.07	-6.17	-1.13	-8.42	-18.29	2.68
Roof 1	-3.12	29.84	9.23	1.27	-7.33	-1.34	-10.01	-21.73	3.19
Top	-0.55	1.29	0.43	0.06	-0.34	-0.06	-0.46	-1.01	0.15

Wind Perpendicular to Long Side									
Floor Level	Total Wind Load in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	-1.00	21.15	17.74	0.41	4.84	-0.43	3.97	-0.63	1.03
1	-2.23	43.63	35.11	0.91	8.64	-0.96	6.72	-3.27	2.28
2	-2.30	44.76	35.99	0.94	8.84	-0.99	6.87	-3.39	2.35
3	-2.43	47.36	38.07	0.99	9.35	-1.05	7.26	-3.59	2.48
4	-2.53	49.43	39.74	1.04	9.76	-1.09	7.58	-3.74	2.59
5	-2.62	51.15	41.12	1.07	10.10	-1.13	7.84	-3.87	2.68
Roof 1	-3.12	60.79	48.87	1.27	12.00	-1.34	9.32	-4.60	3.19
Top	-0.55	6.72	7.39	0.06	3.06	-0.06	2.93	2.00	0.15

Wind Perpendicular to Short Side									
Floor Level	Wind Load Direct Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	4.16	0.00	0.00	3.29	0.00	3.39	0.00	0.00	8.23
1	8.10	0.00	0.00	6.40	0.00	6.59	0.00	0.00	15.99
2	8.48	0.00	0.00	6.70	0.00	6.90	0.00	0.00	16.75
3	9.08	0.00	0.00	7.18	0.00	7.39	0.00	0.00	17.94
4	9.57	0.00	0.00	7.56	0.00	7.79	0.00	0.00	18.90
5	9.96	0.00	0.00	7.87	0.00	8.11	0.00	0.00	19.68
Roof 1	7.77	0.00	0.00	6.14	0.00	6.33	0.00	0.00	15.35
Top	2.37	0.00	0.00	1.87	0.00	1.93	0.00	0.00	4.68

Wind Perpendicular to Short Side									
Floor Level	Wind Load Torsion Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	-0.33	3.18	0.98	0.14	-0.78	-0.14	-1.07	-2.31	0.34
1	-0.63	6.02	1.86	0.26	-1.48	-0.27	-2.02	-4.38	0.64
2	-0.66	6.35	1.96	0.27	-1.56	-0.29	-2.13	-4.63	0.68
3	-0.71	6.80	2.10	0.29	-1.67	-0.31	-2.28	-4.96	0.73
4	-0.75	7.17	2.22	0.31	-1.76	-0.32	-2.40	-5.22	0.76
5	-0.78	7.46	2.31	0.32	-1.83	-0.34	-2.50	-5.44	0.80
Roof 1	-0.61	5.82	1.80	0.25	-1.43	-0.26	-1.95	-4.24	0.62
Top	-0.19	0.81	0.25	0.03	-0.20	-0.04	-0.27	-0.59	0.09

Wind Perpendicular to Short Side									
Floor Level	Total Wind Load in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	3.83	3.18	0.98	3.43	-0.78	3.25	-1.07	-2.31	8.57
1	7.47	6.02	1.86	6.65	-1.48	6.32	-2.02	-4.38	16.63
2	7.82	6.35	1.96	6.97	-1.56	6.62	-2.13	-4.63	17.43
3	8.37	6.80	2.10	7.47	-1.67	7.09	-2.28	-4.96	18.67
4	8.82	7.17	2.22	7.87	-1.76	7.46	-2.40	-5.22	19.66
5	9.18	7.46	2.31	8.19	-1.83	7.77	-2.50	-5.44	20.48
Roof 1	7.16	5.82	1.80	6.39	-1.43	6.06	-1.95	-4.24	15.97
Top	2.18	0.81	0.25	1.91	-0.20	1.89	-0.27	-0.59	4.77

Table AF.3, Wind Case II

Wind Perpendicular to Long Side									
Floor Level	Wind Load Direct Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	0.00	8.64	11.07	0.00	5.40	0.00	5.40	4.78	0.00
1	0.00	16.68	21.37	0.00	10.42	0.00	10.42	9.23	0.00
2	0.00	17.09	21.89	0.00	10.68	0.00	10.68	9.46	0.00
3	0.00	18.08	23.16	0.00	11.30	0.00	11.30	10.01	0.00
4	0.00	18.87	24.18	0.00	11.79	0.00	11.79	10.45	0.00
5	0.00	19.53	25.02	0.00	12.20	0.00	12.20	10.81	0.00
Roof 1	0.00	23.21	29.73	0.00	14.50	0.00	14.50	12.85	0.00
Top	0.00	4.08	5.22	0.00	2.55	0.00	2.55	2.26	0.00

Wind Perpendicular to Long Side									
Floor Level	Wind Load Torsion Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	-0.75	7.22	2.23	0.31	-1.77	-0.32	-2.42	-5.25	0.77
1	-1.68	16.04	4.96	0.69	-3.94	-0.72	-5.38	-11.68	1.71
2	-1.72	16.48	5.10	0.70	-4.05	-0.74	-5.53	-12.00	1.76
3	-1.82	17.44	5.39	0.74	-4.28	-0.78	-5.85	-12.70	1.86
4	-1.90	18.20	5.63	0.78	-4.47	-0.82	-6.10	-13.25	1.94
5	-1.97	18.83	5.82	0.80	-4.62	-0.85	-6.32	-13.71	2.01
Roof 1	-2.34	22.38	6.92	0.96	-5.50	-1.01	-7.51	-16.30	2.39
Top	-0.41	0.96	0.32	0.04	-0.26	-0.05	-0.35	-0.76	0.11

Wind Perpendicular to Long Side and Short									
Floor Level	Total Wind Load in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	2.12	18.24	14.04	2.88	3.04	2.11	2.18	-2.20	7.19
1	3.92	37.24	27.73	5.68	5.37	4.02	3.53	-5.74	14.19
2	4.14	38.34	28.46	5.93	5.46	4.22	3.55	-6.01	14.83
3	4.46	40.63	30.13	6.35	5.76	4.53	3.74	-6.41	15.86
4	4.71	42.45	31.47	6.68	6.00	4.78	3.88	-6.72	16.69
5	4.92	43.96	32.57	6.95	6.20	4.98	4.01	-6.98	17.37
Roof 1	3.03	49.95	38.00	5.75	7.93	3.54	5.53	-6.63	14.37
Top	1.23	5.65	5.73	1.48	2.14	1.37	2.00	1.06	3.69

Wind Perpendicular to Short Side									
Floor Level	Wind Load Direct Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	3.12	0.00	0.00	2.47	0.00	2.54	0.00	0.00	6.17
1	6.07	0.00	0.00	4.80	0.00	4.94	0.00	0.00	11.99
2	6.36	0.00	0.00	5.03	0.00	5.18	0.00	0.00	12.57
3	6.81	0.00	0.00	5.38	0.00	5.55	0.00	0.00	13.46
4	7.17	0.00	0.00	5.67	0.00	5.84	0.00	0.00	14.17
5	7.47	0.00	0.00	5.91	0.00	6.08	0.00	0.00	14.76
Roof 1	5.83	0.00	0.00	4.61	0.00	4.74	0.00	0.00	11.51
Top	1.78	0.00	0.00	1.40	0.00	1.45	0.00	0.00	3.51

Wind Perpendicular to Short Side									
Floor Level	Wind Load Torsion Component in Lateral Resisting Elements (Kip)								
	AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
0	-0.25	2.38	0.74	0.10	-0.58	-0.11	-0.80	-1.74	0.25
1	-0.47	4.52	1.40	0.19	-1.11	-0.20	-1.51	-3.29	0.48
2	-0.50	4.76	1.47	0.20	-1.17	-0.21	-1.60	-3.47	0.51
3	-0.53	5.10	1.58	0.22	-1.26	-0.23	-1.71	-3.72	0.54
4	-0.56	5.37	1.66	0.23	-1.32	-0.24	-1.80	-3.91	0.57
5	-0.58	5.60	1.73	0.24	-1.37	-0.25	-1.88	-4.08	0.60
Roof 1	-0.46	4.37	1.35	0.19	-1.07	-0.20	-1.46	-3.18	0.47
Top	-0.14	0.61	0.19	0.03	-0.15	-0.03	-0.20	-0.44	0.06

Table AF.4, Wind Case IV

Maximum Wind Base Shear in Lateral Resisting Elements (Kip)								
AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
76.49	325.00	304.42	63.85	126.60	63.35	121.65	84.03	159.59

Maximum Wind Base Shear in Lateral Resisting Elements (Kip/ft Length)								
AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
7.40	15.42	11.27	7.82	9.62	7.53	9.24	7.20	7.82

Maximum Overturning Moment Shear in Lateral Resisting Elements (Kip)								
AV1-X1	AV1-Y1	AV2-Y1	AV2-X1	AV3-Y1	AV3-X1	AV3-Y2	AV4-Y1	AV4-X1
3822.4	16608.2	15713.7	3190.7	6495.6	3165.7	6231.1	4251.5	7974.7

Table AF.5, Maximum Element Base Shear and Overturning Moment

Thaison Nguyen | | Irregularity Analysis | 5/7

Lateral Resisting Member	$\Delta P_{avg\text{-}w,\,level}$ (Kip)						
	0,1	1,2	2,3	3,4	4,5	5, Roof 1	Roof 1, Top
AV1-X1	10.41	0.49	0.77	0.62	0.51	2.52	6.94
AV1-Y1	43.63	1.13	2.60	2.07	1.71	9.64	51.69
AV2-Y1	40.31	1.01	2.40	1.91	1.58	8.90	46.25
AV2-X1	8.69	0.42	0.65	0.52	0.43	2.35	5.80
AV3-Y1	16.69	0.34	0.99	0.79	0.65	3.68	19.12
AV3-X1	8.62	0.41	0.64	0.51	0.42	2.33	5.75
AV3-Y2	16.02	0.37	0.95	0.76	0.63	3.53	18.34
AV4-Y1	10.99	0.21	0.65	0.52	0.43	2.41	12.51
AV4-X1	21.71	1.04	1.62	1.29	1.07	5.88	14.49

$I_{cr,xx} = x_{eff}\, d_{eff}^{3}/12$

$x_{eff} = 8 - (0.75 + 0.5 + 1/2)2$

$x_{eff} = 4.5''$

$d_{eff} = d_{tot} - 2(0.75 + 0.5 + 1/2)$

$d_{eff} = d_{tot} - 3.5''$

* See excel calculations (Story drift) on following page

Drift Limit = H/400, per AISC 7-05 Section CC.1.2

Drift Limit(h=14') = 14(12)/400 = 0.42''

Drift Limit(h=16') = 16(12)/400 = 0.48''

Lateral Resisting Member	Thickness (in)	Depth (in)	$I_{conc,\ xx}$ (in^4) [2]	I_{cr} / I_{gross}	$\Delta P_{max\text{-}w,\ level}$ (Kip)		$\Delta_{drift,\ story}$ (in) [2] [3]	
					0, 1	1, . . ., Top	0, 1	1, . . ., Top
AV1-X1	8	124.00	317771	0.25	10.41	6.94	0.0062	0.0028
AV1-Y1	8	252.94	2697046	0.25	43.63	51.69	0.0031	0.0024
AV2-Y1	8	324.00	5668704	0.25	40.31	46.25	0.0013	0.0010
AV2-X1	8	98.00	156865	0.25	8.69	5.80	0.010	0.0047
AV3-Y1	8	158.00	657385	0.25	16.69	19.12	0.0048	0.0037
AV3-X1	8	100.94	171398	0.25	8.62	5.75	0.0095	0.0042
AV3-Y2	8	158.00	657385	0.25	16.02	18.34	0.0046	0.0035
AV4-Y1	8	140.00	457333	0.25	10.97	12.51	0.0045	0.0035
AV4-X1	8	244.94	2449146	0.25	21.71	14.49	0.0017	0.00075

C. Irregularity and Seismic

1) Check for Re-entrant Corner, diaphragm discontinuity irreg.

Floor Type	Dimensions w/o Re-entrant Corners		Re-entrant Corner Dimensions		Gross Area (ft^2)	Void Area (ft^2)
	x	y	x	y		
A	197.51	73.59	28.00	40.83	25785.62	1929.76
B	225.51	115.43	2	2	26440	2053.77
C	225.51	115.43	2	2	26440	786.43

Floor Type	Re-entrant Corner Extension Percentage		Void Percentage
	x	y	
A	28/197.51 * 100 = 14.2%	40.83/73.59 * 100 = 55.5%	1929.76/25785.62 * 100 = 7.5%
B	2/225.51 * 100 = 0.9%	2/115.43 * 100 = 1.7%	2053.77/26440 * 100 = 7.8%
C	2/225.51 * 100 = 0.9%	2/115.43 * 100 = 1.7%	786.43/26440 * 100 = 3.0%

Floor Type A doesn't have re-entrant corner irregularity because only the y-direction has an extension ≥ 15%

All floor types don't have diaphragm discontinuity irregularity since the voids are ≤ 50% of the gross floor diaphragm area.

*** Due to continuity of lateral force resisting elements there are no out-of-plane offset irreg.

*** Non parallel systems irreg doesn't exist

Increase in forces due to horizontal forces is not required for SDC A, B, C per ASCE 7-05 §12.3.3.4

2) Check Soft Story and Weight Irregularity.

$K \sim 1/L^3$

Story	Height (ft)	K
1	16	0.00024
2	14	0.00036
3	14	0.00036
4	14	0.00036
5	14	0.00036
6	14	0.00036

→

Thaison Nguyen | Irregularity Analysis | 7/7

Storys	K_{avg}
1,2,3	$[0.00024+0.00036(2)]/3 = 0.00032$
2,3,4	0.00036
3,4,5	0.00036
4,5,6	0.00036

Story	K_i/K_{i+1} (%)	$W_{eff,j}/W_{eff,i}$ (%) [10]
1	$0.00024/0.00036 \times 100 = 67\%$	101.7%
2	100%	101.4%
3	100%	101.8%
4	100%	100.2%
5	100%	
6		

[10] Using effective floor weight determined in seismic analysis.

Only the first story exper. soft story irreg. due to $K_i/K_{i+1} \leq 80\%$.

Extreme soft story irreg. doesn't exist in any story, since $K_i/K_{i+1} > 70\%$

Weight irreg. doesn't exist because $W_{eff,j}/W_{eff,i} \leq 150\%$

In force increase or modifications are required, per ASCE7-05 Table 12.3-2, § 12.3.3.4

Drift Limit Factor = 0.015 L, per ASCE 7-05 Table 12.12-1

→

Appendix G: Lateral Spot Check/Design

Thaison Nguyen | Lateral spot check / Design | 1/4

Controlling Lateral Load : Wind
Load Combination: $1.2D + L + 0.5L_r + 1.6W$

*** Design lateral force resisting member w/ maximum overturning and shear

Member w/ Max Overturning and Shear : AV1-Y1

$M_{u,max} = 1.6(16608.2)$, using value calculated in Irregularity Analysis – Irregularity and Wind
$M_{u,max} = 26573.1$ Kip*ft
$V_{u,max} = 1.6(325)$, using value calculated in Irregularity analysis – Irregularity and Wind
$V_{u,max} = 520$ Kip

A. Classify Shear Wall design

$L_b/W_b = 86/21.08$
$L_b/W_b = 4.08 > 3$; Slender /Flexural behavior per Reinforced Conc. Mech. & Design by James K. Wright, James G. MacGregor §18-5 pp.937
$L_b/T_b = 86/(8/12)$
$L_b/T_b = 129$

B. Design (Flexural)

*** Assume #8 flexural reinforcement, #4 shear reinforcement

$f'_c = 3000$ psi
$f_y = 60$ ksi
$b = 8''$
$A_s = 0.79(2) = 1.58$ in^2, (2) #8 per row

$a = \dfrac{A_{s,tot} f_y}{0.85 f'_c b}$
$a = \dfrac{nA_s(60)}{0.85(3)(8)}$
$a = 2.94\,nA_s$

*** Assume $\varnothing = 0.9$, where $\varepsilon_s \geq 0.005$
*** Initially assume no top reinf.

$M_u = \varnothing A_{s,tot} f_y (d - a/2)$
$\dfrac{M_u}{\varnothing f_y} = nA_s(d - a/2)$
$5905.1 = nA_s\{[251.21 - 0.5nS_c + 0.5S_c] - 1.47nA_s\}$
$5905.1 = nA_s\{[251.21 + 0.5S_c] - [0.5S_c + 1.47A_s]n\}$
$5905.1 = 1.58n\{[251.21 + 0.5S_c] - [0.5S_c + 1.47(1.58)]n\}$
$5905.1 = (396.9 + 0.79S_c)n - (0.79S_c + 3.67)n^2$

$d = 21.08(12) - 0.75 - 0.5 - 0.5 - \dfrac{(n-1)}{2}S_c$
S_c = Space btw bars (o.c.)
n = # of Rows
$d = 251.21 - \dfrac{(n-1)}{2}S_c$

$a_1x^2 + b_1x + c_1 = 0$
$a_1 = -0.79S_c - 3.67$
$b_1 = 396.9 + 0.79S_c$
$c_1 = -5905.1$

$n_{max} = \dfrac{21.08(6) - 1.75}{S_c} + 1$
$n_{max} = \dfrac{124.73}{S_c} + 1$

Thaison Nguyen | Lateral Spot Check / Design | 2/4

$s_c = 2''$

$a_1 = -0.79(6) - 3.67$
$a_1 = -8.41$
$b_1 = 396.9 + 0.79(6)$
$b_1 = 401.64$
$n = 20.2$
$n \approx 21$ rows $\leq n_{max}$ ✓

$n_{max} = 63$ rows

$a = 2.94(21)(1.58)$
$a = 97.5''$

$d = 251.21 - \frac{(21-1)(2)}{2}$
$d = 231.21''$

$s_c = 3.5''$

$a_1 = -6.44$
$b_1 = 399.25$
$n = 24.3$
$n \approx 25$ rows $\leq n_{max}$

$n_{max} = 35$ rows

$a = 2.94(25)(1.58)$
$a = 116.1''$

$\rho_{min,\ thermal/cracks\ control} = 0.0018$

$\rho_{s,min} = \max\begin{cases} 3\sqrt{f'_c}/f_y \\ 200/f_y \end{cases} = 0.0033$, ACI 318-11 §10.5.1 flexure members.

Max rebar spacing shall be 18'', per ACI 318-11 §7.12.2.2
$\rightarrow \rho_{3min} = 2(0.79)/18 = 0.087$, controlling min. reinf.

n_{add} = additional rows of (2) #8 to satisfy controlling min. reinf.
$n_{add} = [251.21 - 2(25-1)(3.5) - 2(1.75)]/18 - 1$
$n_{add} = 79.71/18 - 1$
$n_{add} = 4$, assumed to not contribute to strength.

$d = 251.21 - \frac{(24)(3.5)}{2}$
$d = 209.21''$
$c = a/0.85$
$\varepsilon_{s,extreme} = \frac{0.003}{136.59}(251.21 - 136.59)$
$\varepsilon_{s,extreme} = 0.00252 < 0.005$, can't use $\phi = 0.9$

Determining $A'_{s,min}$ to ach. $\phi = 0.9$

$0.85 f'_c a b + A'_{s,min} \varepsilon'_s E_s = A_s f_y$
$0.85 f'_c \beta_1 b c^2 + n' A'_s (0.003)(c - d') E_s = A_s f_y$
$0.85^2(3)(8)(94.2)^2 + n'(1.58)(0.003)(94.2 - 1.75n')E_s = 39.5(60)$
$153868.9 + 12948.7n' - 240.6n'^2 = 2370$
$-240.6n'^2 + 12948.7n' + 151498.9 = 0$
$n' \approx 25$ rows

$A'_{s,min} = 25(1.58)$
$A'_{s,min} = 39.5\ in^2$

$\varepsilon_s = \frac{0.003}{c}(d - c)$
$0.005c = 0.003(d_{max} - c)$
$c = \frac{3}{8} d_{max}$
$c = 0.375(251.21)$
$c = 94.2''$

$d' = \frac{(n'-1)s_c}{2} + 1.75$
$d' = 0.5 s_c n' - 0.5 s_c + 1.75$
$d' = 1.75n'$

$\rightarrow$

Use 25 rows of (2)#8 at each end of the flexural wall, 3.5" O.C.

*See drawing on following page to see arrangement (Rebar)

$\varepsilon_s' = \frac{0.003}{94.2}(94.2 - 1.75 \times 25)$

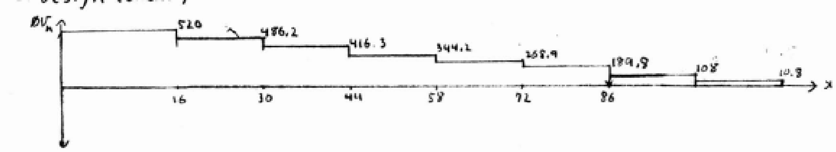
$\varepsilon_s' : 0.0016 < 0.002$, top reinf doesn't yield.

$\varepsilon_s = \frac{0.003}{94.2}(209.21 - 94.2)$

$\varepsilon_s = 0.0037 > 0.0021$, yields.

$\varepsilon_{s,extreme} = 0.005$

C. Design (Shear)



*** Assume shear reinforcements are (2)#4

$d = 209.21" = 17.43'$

$b_w = 8"$

$M_{u,ed} = 486.2$ kip

$V_c = 2\sqrt{3000}\,(8)(209.21)$

$V_c = 183.3$ kip

$V_s = \frac{486.2}{0.75} - 183.3$

$V_s = 465$ kip

$V_s \le 8\sqrt{3000}\,(8)(209.21)$

$V_s \le 733.3$ kip

$V_{s,spacing} = 733.3/2$

$V_{s,spacing} = 366.7 < 465$

$S_{max} = \min\begin{cases} d/4 \\ 12" \end{cases} = 12"$

$A_v = 0.2(2) = 0.4\ in^2$

$S = A_v(f_y)\,d / V_s$

$S = 0.4(60)(209.21)/(465)$

$S \approx 10"$

→

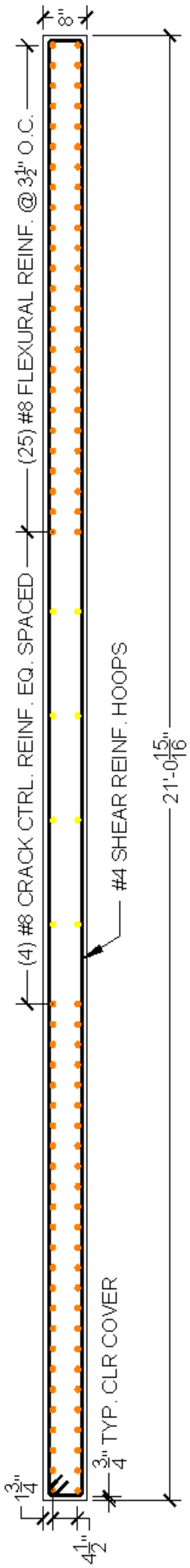


Figure AG.1, Reinforcement

Thaison Nguyen		Lateral Spot Check / Design	4/4

1) Determine when $s = 12''$

$V_s = A_v(f_y)d/s$

$V_{tot,u} = (V_s + V_c)0.75$
$V_{tot,u} = [0.4(60)(209.21)/12 + 183.3]0.75$
$V_{tot,u} = 451.29$ kip, where s can equal 12"

*** Use $s = 12''$ when $x < d$, for thermal and crack control.

* See drawings on following page see shear reinf. arrangement.

AMPAD

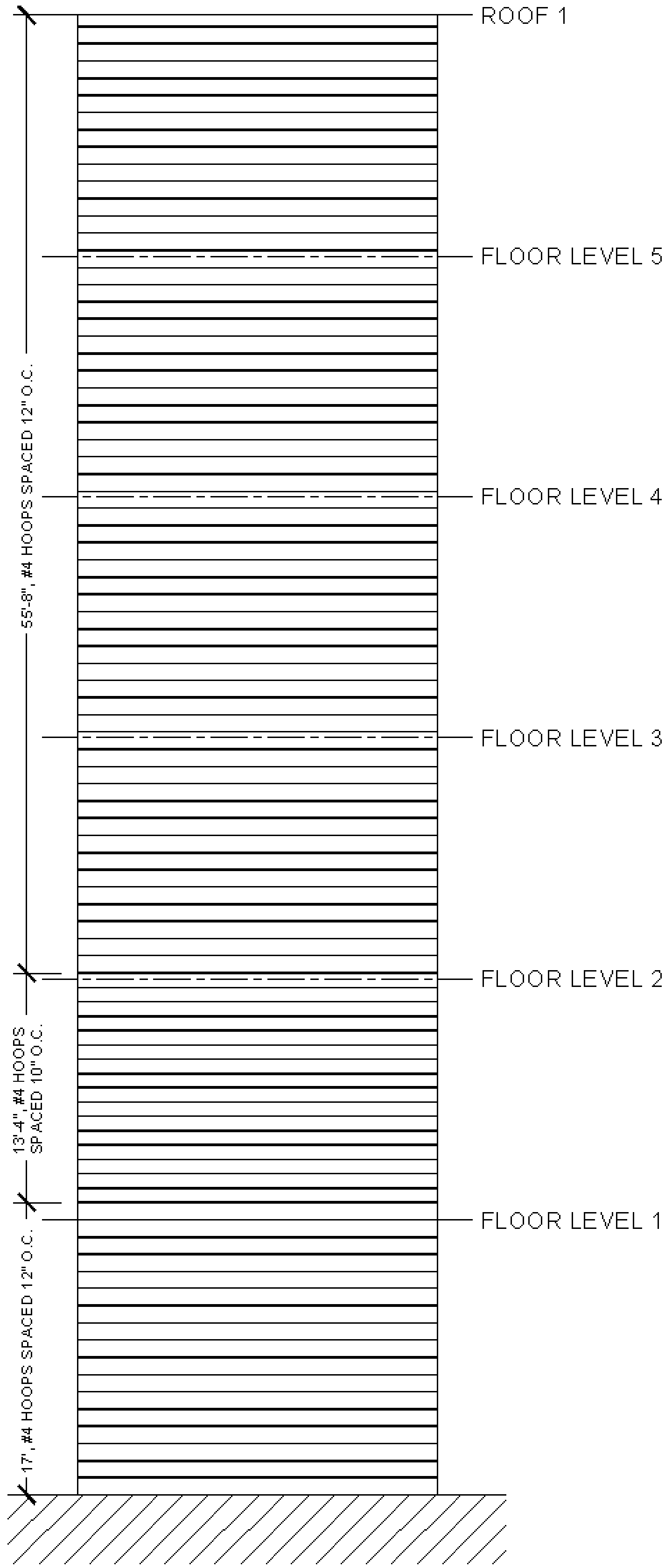


Figure AG.1, Shear Reinforcement Spacing

Appendix H: Structural Computer Modeling

Modeling Assumptions

1. All shear walls are monolithically cast
2. Model all shear walls as frame elements in-lieu of 2-D elements w/ mesh
3. Fixed base connection
4. Rigid floor diaphragm
5. No MEP openings in floor slab or shear walls

Monolithically cast concrete shear wall are modeled by modifying the moment of inertia in the strong direction. The modifying factor was determined by dividing the monolithic shear wall's moment of inertia by the individual/non-monolithic shear wall's moment of inertia. Moment of inertia in the weak direction was left to be zero. See the excel spread sheet below for the modification factors to the shear wall's moment of inertia in the strong direction.

Lateral Resisting Element		Length (ft)	Thk (in)	Area (in^2)	Local		Global	
Designation	Resisting Direction				X_{cm} (in)	Y_{cm} (in)	X_{cm} (in)	Y_{cm} (in)
AV1-X1	X	10.333	8	992.00	62.00	4.00	106.29	86.18
AV1-Y1	Y	21.078		2023.50	128.00	126.47		
AV2-Y1	Y	27.000		2592.00	4.00	162.00	16.31	198.69
AV2-X1	X	8.167		784.00	57.00	320.00		
AV3-Y1	Y	13.167		1264.00	4.00	79.00	58.47	97.16
AV3-X1	X	8.411		807.50	58.47	154.00		
AV3-Y2	Y	13.167		1264.00	112.94	79.00		
AV4-Y1	Y	11.667		1120.00	4.00	70.00	84.47	112.00
AV4-X1	X	20.411		1959.50	130.47	136.00		

Lateral Resisting Element		I_{indiv}	I_{flange}	Ad^2		I_{syst}	Stiffness Factor
Designation	Resisting Direction			Indiv	Flange		
AV1-X1	X	1271083	10792	1945751	953884	4181510	3.29
AV1-Y1	Y	10788186	5291	3284418	6699617	20777512	1.93
AV2-Y1	Y	22674816	4181	3489606	11537065	37705669	1.66
AV2-X1	X	627461	13824	1298174	392658	2332117	3.72
AV3-Y1	Y	2629541	2153	416709	1304570	4352973	1.66
AV3-X1	X	685593	2247	0	7500183	8188024	11.94
AV3-Y2	Y	2629541	2153	416709	1304570	4352973	1.66
AV4-Y1	Y	1829333	10451	1975313	1129039	4944136	2.70
AV4-X1	X	9796582	5973	4145600	90129	14038284	1.43

Center Mass Rigidity

Edit View

Center Mass Rigidity

Story	Diaphragm	MassX	MassY	XCM	YCM	CumMassX	CumMassY	XCCM	YCCM	XCR	YCR
STORY6	D1	101.0603	101.0603	114.753	58.442	101.0603	101.0603	114.753	58.442	89.191	47.780
STORY5	D1	97.6140	97.6140	114.787	58.897	198.6743	198.6743	114.770	58.665	89.469	47.784
STORY4	D1	98.0577	98.0577	114.787	58.897	296.7320	296.7320	114.776	58.742	89.936	47.791
STORY3	D1	99.8325	99.8325	114.787	58.897	396.5645	396.5645	114.779	58.781	90.846	47.800
STORY2	D1	101.6073	101.6073	114.787	58.897	498.1719	498.1719	114.780	58.804	92.885	47.811
STORY1	D1	95.3270	95.3270	114.690	58.721	593.4988	593.4988	114.766	58.791	97.909	47.809

OK

Figure AH.1, Center of Mass and Rigidity

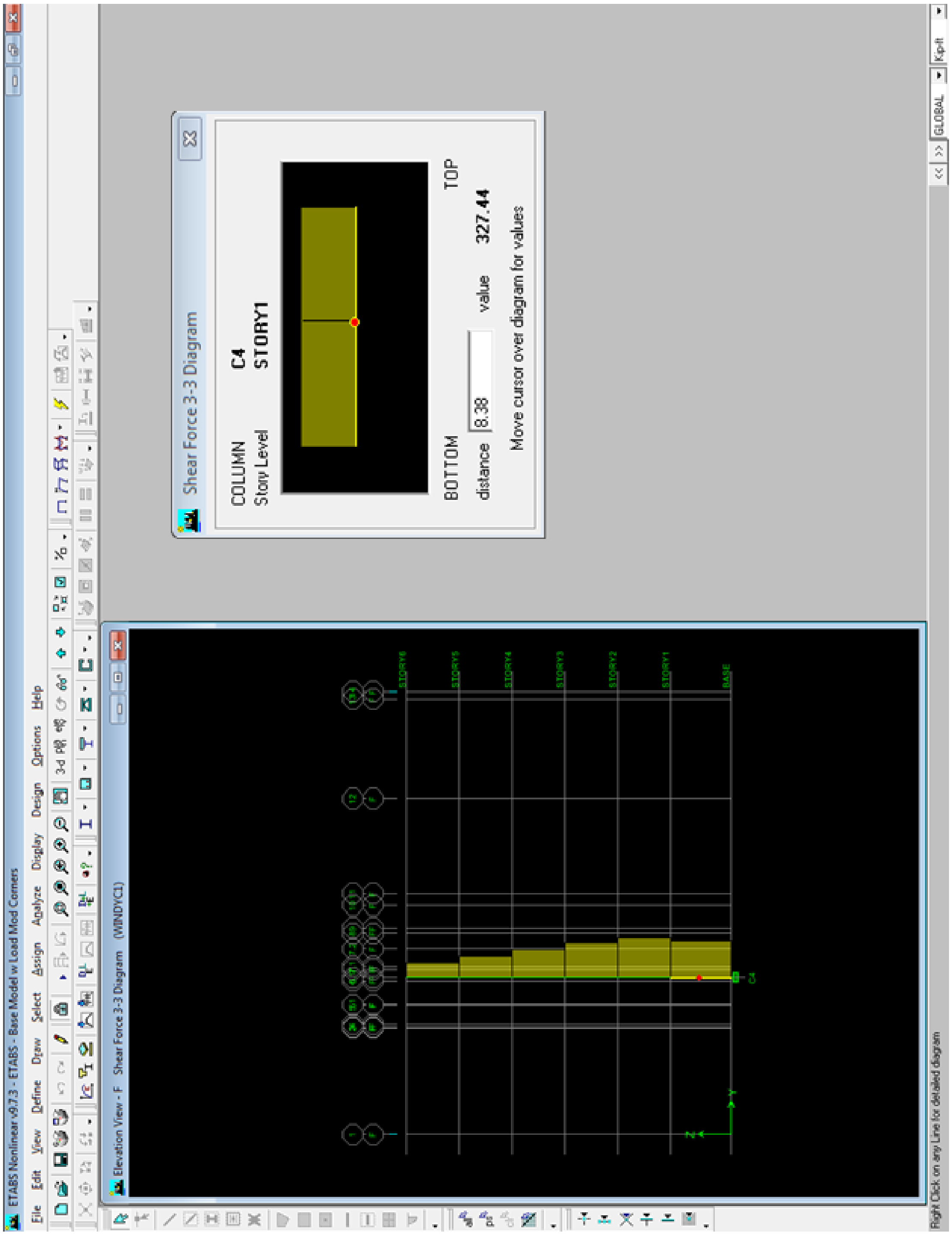
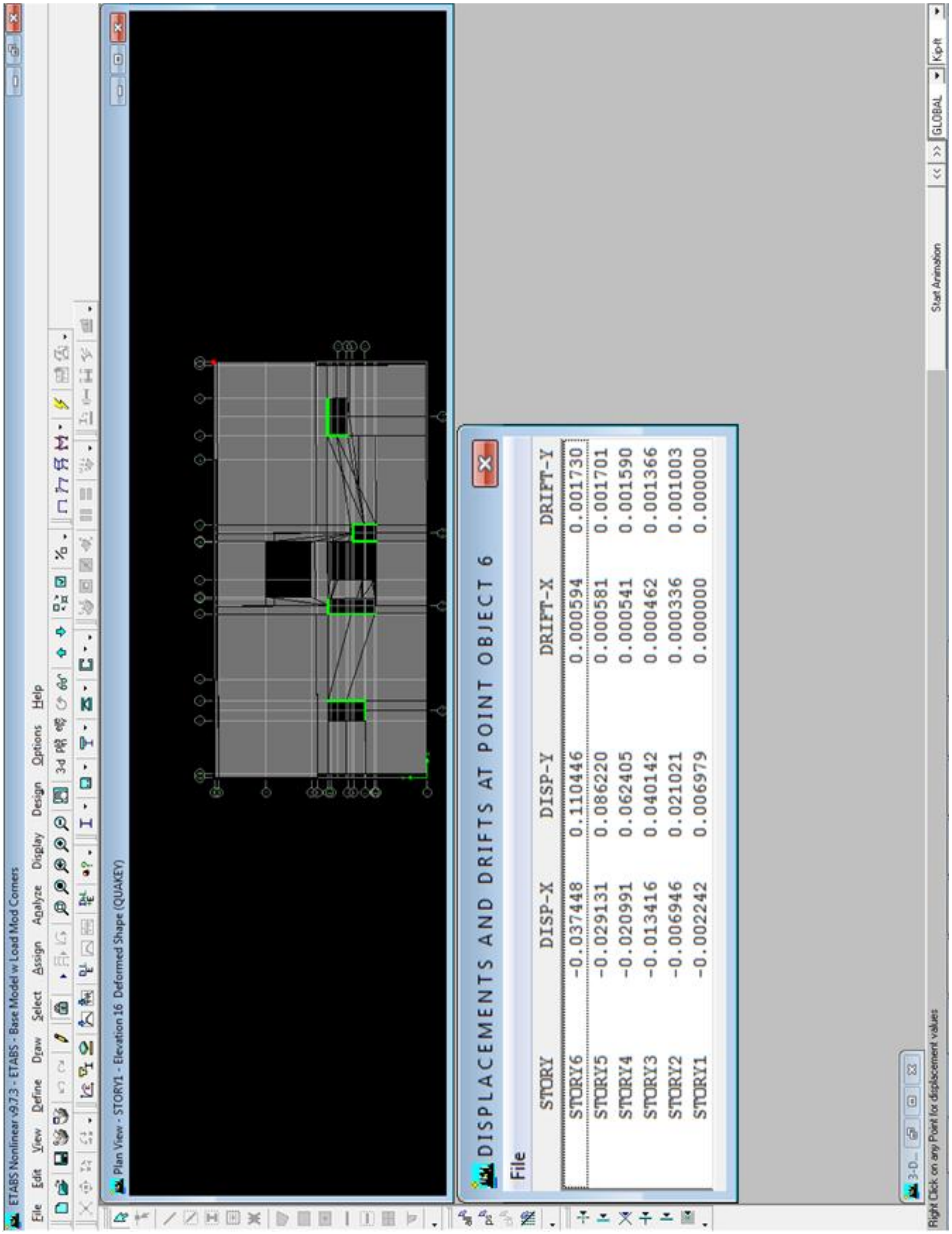
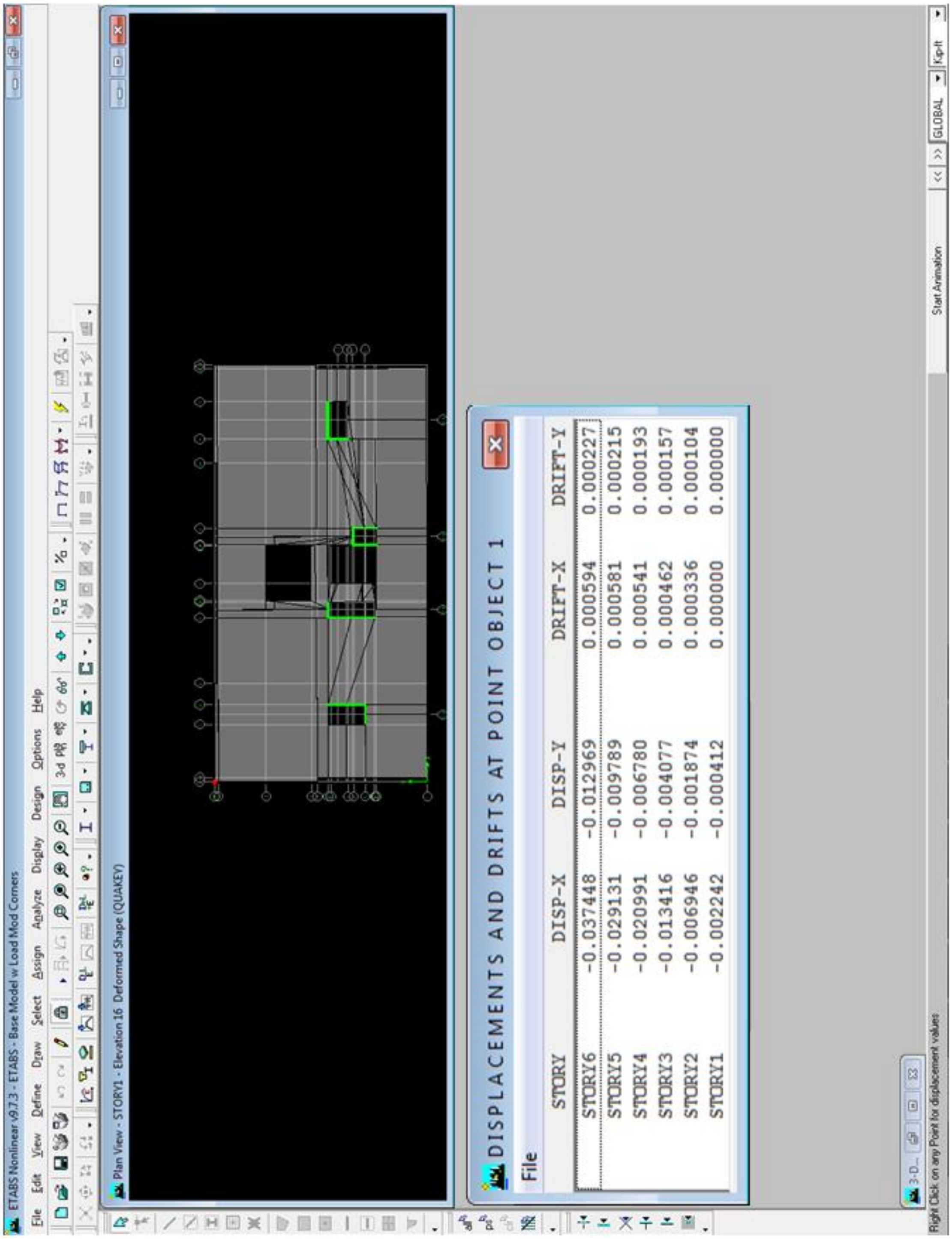


Figure AH.2, Shear in Member AV2-Y1



STORY	DISP-X	DISP-Y	DRIFT-X	DRIFT-Y
STORY6	-0.037448	0.110446	0.000594	0.001730
STORY5	-0.029131	0.086220	0.000581	0.001701
STORY4	-0.020991	0.062405	0.000541	0.001590
STORY3	-0.013416	0.040142	0.000462	0.001366
STORY2	-0.006946	0.021021	0.000336	0.001003
STORY1	-0.002242	0.006979	0.000000	0.000000

Figure AH.3, Displacement and Story Drift at Right Corner (in red)



STORY	DISP-X	DISP-Y	DRIFT-X	DRIFT-Y
STORY6	-0.037448	-0.012969	0.000594	0.000227
STORY5	-0.029131	-0.009789	0.000581	0.000215
STORY4	-0.020991	-0.006780	0.000541	0.000193
STORY3	-0.013416	-0.004077	0.000462	0.000157
STORY2	-0.006946	-0.001874	0.000336	0.000104
STORY1	-0.002242	-0.000412	0.000000	0.000000

Figure AH.4, Displacement and Story Drift at Left Corner (in red)