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Technical Report I

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

Largo Medical Office Building

Largo, Florida

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Executive Summary

Design concepts, site conditions, and building characteristics are explored in the following pages of Technical Report I. Technical Report I encompasses analysis of the Largo Medical Office Building's (LMOD) structure and comparisons between the original design and thesis spot checks. Studying the facility's use and design intent allowed assumptions concerning the loads and possible changes as the facility ages.

Systems included in the total gravity loads are: floor and flooring system, framing system, and building envelope. The typical bay utilized is 33'-0" x 33'-0", where the beams are typically spaced 8'-3" and joists are spaced 5'-6". Criteria for determining gravity system adequacy are bending capacity and deflection adherence to serviceability. Results showed that decks and girders are adequate but there slight discrepancies with the original joist, beams, and columns. Joists, beams, and columns have a max discrepancy of 14 percent; which can stem from vibration requirements, live load reductions, or use of predominant sections. Due to the lack of available information member weight comparison was no achieved, but member depths were compared.

Method 2 and Equivalent Lateral Load procedures were used to determine the wind and seismic loads respectively. The building's shape, roof heights, and gust factors were simplified. The lack of access to the original wind and seismic loads is responsible no comparisons with the spot check. It was determined that the base shear is only 1.4 percent of the effective building weight. As a result, the wind load in the North/South direction is the controlling lateral load case. Base shear and total overturning moment, for wind loading in the North/South direction, are 1077.9 kips and 555209 kip-ft respectively. Seismic loading produced a base shear of 314.6 kips and an overturning moment of 19507.5 kip-ft.

Included in the Appendix are all the gravity, wind, and seismic load calculations; as well as plans of typical building features.

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 155,000 ft^2 facility also houses office space for future tenants, as well as screening and diagnostic equipment.

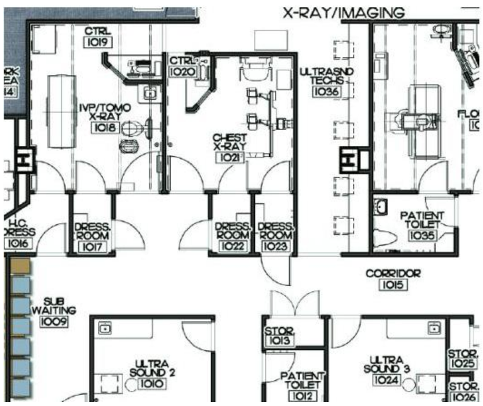

Figure 1.1, Illustrated Floorplans
Source: Oliver, Glidden, Spina & Partners

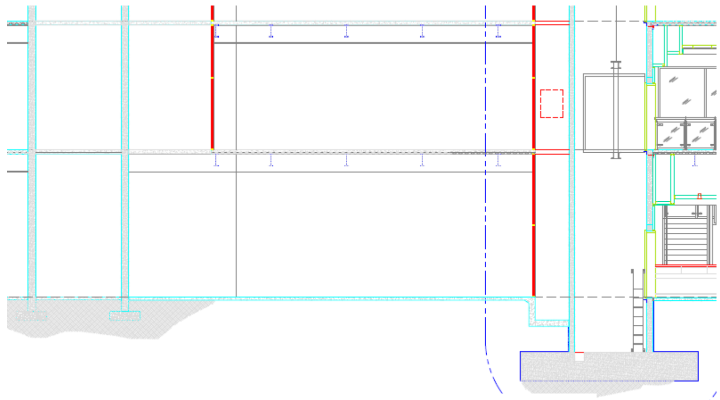

Figure 1.2, Building Section
Source: Oliver, Glidden, Spina & Partners

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 105' tall, steel framed facility with specially reinforced concrete shear walls to resist lateral loads. The shear walls rest on top of strip 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 specially reinforced concrete shear walls and a steel frame.

Concerns about the structural system arose, after looking at the available plans. These concerns include:

1. Effects of drain placement on the rain load
2. Wind loading on the overhang (Figure 2.1)
3. Lack of information due to incomplete drawing set
   - Soil profile
   - Structural member sizes
   - Actual design assumptions and loads

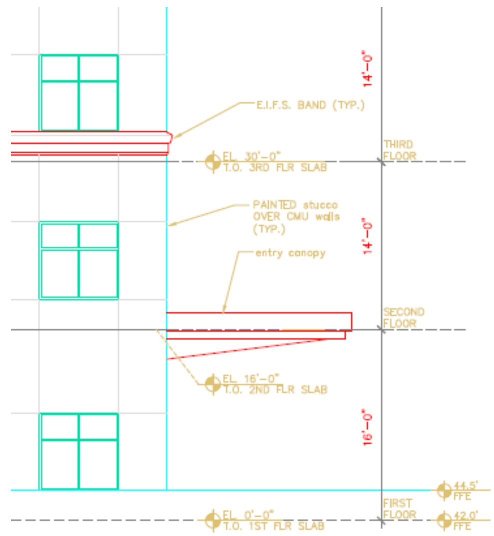

Figure 2.1, Overhang
Source: Oliver, Glidden, Spina & Partners

Due to the lack of information the list of design codes, structural material, and some system details are incomplete. The uncertainty also generated numerous assumptions were made. Assumptions are highlighted in **red** lettering.

Design Codes

Structural engineer consulting firm, McCarthy and Associates, designed the building to comply with the following codes and standards:

1. 2004 Florida Building Code (FBC)
   – Adoption of the 2003 International Building Code (IBC)
2. 13th Edition AISC Steel Manual
3. Design Manual for Floor and Roof Decks by Steel Deck Institute (SDI)
4. ACI 318-05

Codes and standards used for thesis are as follows:

1. 2009 International Building Code (IBC)
2. ASCE 7-05
3. 14th Edition AISC Steel Manual
4. 2008 Vulcraft Decking Manual
5. 2007 Vulcraft Steel Joists and Joist Girders Manual
6. ACI 318-08

Structural Materials Used

Table 2.1, List of Structural Materials	
Steel	
W-Shapes	ASTM A992 Gr. 50
Angles	ASTM A36
Plates	ASTM A36
Reinforcing Bars	ASTM A615
Concrete	
Footings	3000 psi
Slab-on-Grade	3000 psi
Floor Slab	3000 psi

Framing & Lateral System

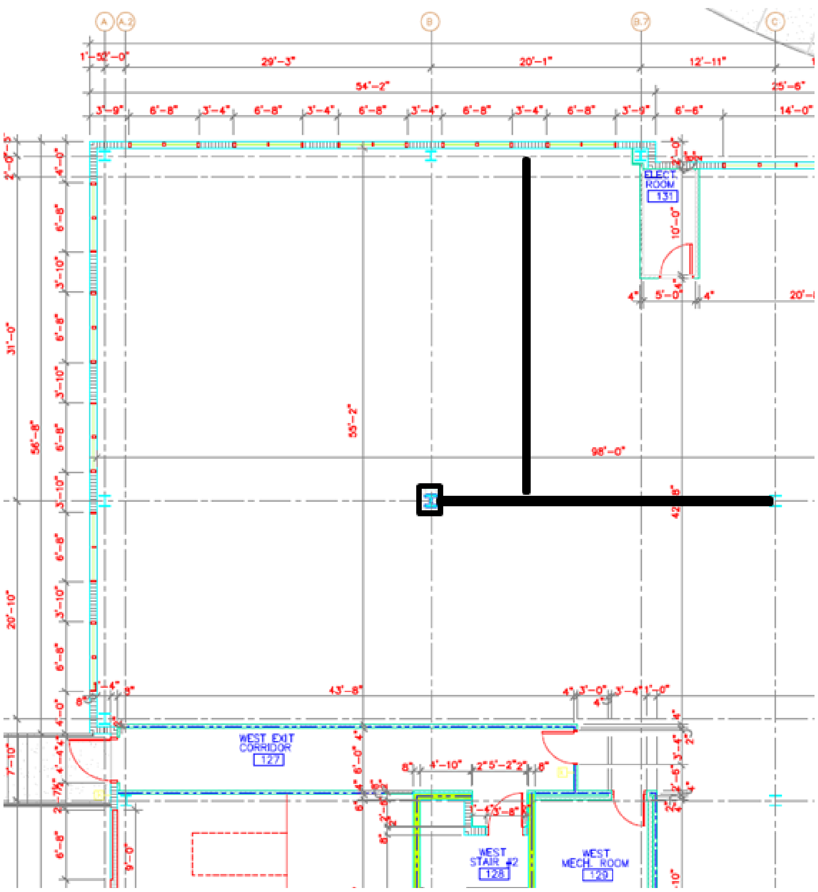

Figure 2.2, Typical Structural Bay
Source: Oliver, Glidden, Spina & Partners

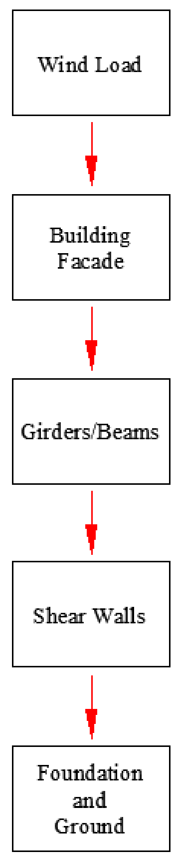

Figure 2.3, Lateral Load Path

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" (Figure 2.2). Please refer to Appendix A for typical plans and elevations. Girders primarily span in the East/West (longitudinal) direction. The only locations where girders are orientated differently include: the overhang above the lobby entrance and the loading dock area. 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.

To deal with the lateral load, specially reinforced shear walls are used. The shear walls help the facility resist wind from the North/South and East/West direction. From the drawings it appears that the shear walls are positioned around the emergency stairwells and the two elevator cores. Typical shear walls span from the ground floor level to the primary roof (86' above ground floor level), highlighted black in Figure 2.2. Only the east emergency stairwell has a greater span due to the need for a direct access to roof level from the interior. Lateral load distribution path is demonstrated in Figure 2.3.

In lieu of using shear walls for the lateral system, brace frames and moment frames could be utilized. There are advantages and drawbacks to each lateral system, see Table 2.2 for a comparison of the systems.

Table 2.2, Comparison of Lateral Systems

System	Shear Walls	Brace Frames	Moment Frames
Lateral Resistance Mechanism	Wall Mass and Solidity	Elongation of Brace	Rigid Connection
Member Size	Large	Small	Large
Footprint and Space Flexibility	Mid	Mid	Small
Weight	Heavy	Light	Mid
Vibration Dampening	High	Low	Low
Cost	High - due to labor	Low	High - due to connection quality control and fastening system

From comparing the various lateral systems with the building's primary function, it appears that the original decision to use shear walls is logical. Throughout the lifetime of the facility will house various tenants with different interior preferences, space flexibility is a significant concern. Both the shear walls and moment frames satisfy the space flexibility criteria. Drift is another concern when evaluating for the optimum lateral system. Greater amounts of drift increases the complexity of joining and fastening the building façade; which in turn leaves room for inadequate construction and rainwater leakage. Shear walls and brace frames are fairly stiff systems which results in reduced story drift when compared to moment frames. In addition the fire rating and safe emergency egress is an equally important criteria. Steel structures require significantly greater fire proofing, in concrete the cover is usually increased and is less labor intensive.

Regional preference also plays a role in choosing a lateral system. In the southern U.S. concrete is the predominant building material, due to the lack of vital ingredients for steel production and steel labor base. As a result, lateral systems requiring special connection methods must be ruled out, such as moment frames.

Flooring System

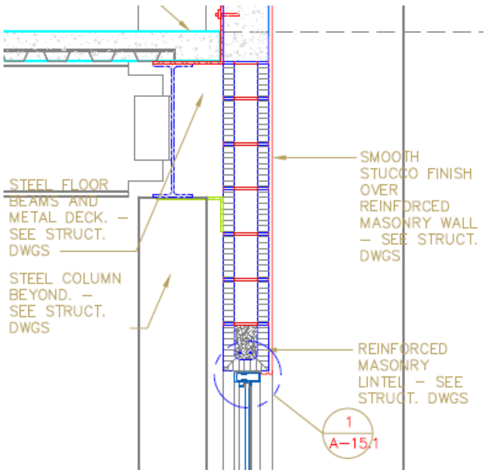

Figure 2.4, Typical Composite Slab
Source: Oliver, Glidden, Spina & Partners

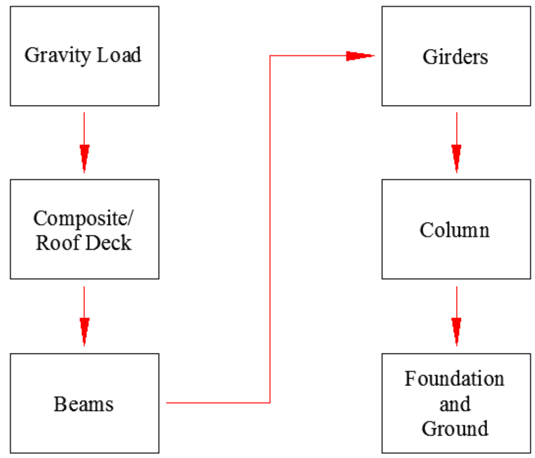

Figure 2.5, Gravity Load Distribution

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". Gravity load distribution path can be followed in Figure 2.5. 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.

Hollow core planks and post-tension (pt) slabs are alternatives to the composite slab. PT-slabs do have an advantage in having a thin structural floor, thus allowing greater number of floors when compared to an equally high steel structure. Echoing the frame and lateral system, structural systems for office facilities should allow flexibility in partition and opening placement. Tensioned cables in pt-slabs prevent modification of the slab, like putting an opening into the floor, without first de-stressing the cables and temporary support the floor strip. On the other hand, hollow core planks don't hinder future floor openings. Though pt-slabs aren't easily modified once formed, the system has the advantage in having the thinnest structural floor system. This is advantageous for cities with height limitations since pt-slabs allow greater numbers of floors when compared to an equally high steel structure. In terms of quality control, both pt-slabs and composite slab concrete is typically cast in the field. The results of concrete cast in the field are mix inconsistency and weather induced strength variations. Hollow core planks doesn't have strength inconsistency problems, other than the typical 2" toping.

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 are the same, consisting of a 3-ply bituminous waterproofing applied over the insulated cast-in-place concrete (Figure 2.6). To ensure adequate rainwater drainage, the insulated cast-in-place concrete is sloped ¼" for every 12" horizontal.

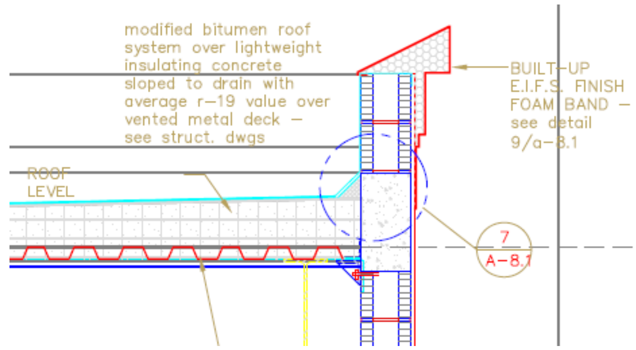

Figure 2.6, Roof Detail
Source: Oliver, Glidden, Spina & Partners

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. What it means is, 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 addition, the added mass counters the uplift wind force.

Gravity Loads

Dead, live, rain, and snow loads were calculated for verification of the gravity system. ASCE 7-05 was utilized to factor the loads, using the LRFD method, to determine the size gravity members and check adequacy of actual system. Figure 2.2 shows the typical members, highlighted, which were checked.

Due to the lack of sufficient information, stemming from incomplete drawing set and specifications, a direct comparison of member sizes and design loads was not achieved. Instead actual member sizes were taken by measuring the member depth on the CAD architectural files.

Gravity load and member size calculations can be referenced in Appendix A and Appendix C, respectively.

Dead Loads

Before any dead load calculations were performed, quantity takeoffs and research in material weight were implemented. Take-offs was organized by floor level, which allowed ease of future analysis and design of alternate structural systems. The division by floor level has flexibility built in, where changes in materials can be easily tracked without having to decipher the entire building load equation. Items included in the take offs are: slab concrete volume, floor finish areas, areas of roofing layers/components, volume and area of façade components. See Table 3.1 and Table 3.2 for the material weights and total un-factored dead load by floor level.

Table 3.1, Weight of Building Materials		
Material	Weight	Reference
Normal-Weight (NW) Concrete	150 lb/ft^3	AISC 14th Edition – Table 17-13
Light-Weight (LW) Concrete	113 lb/ft^3	Arch. Graphics Standards 11 Edition
Vinyl Composition Tile (VCT)	1.33 lb/ft^2	Arch. Graphics Standards 11 Edition
Ceramic/Porcelain Tile	10 lb/ft^2	AISC 14th Edition – Table 17-13
3-Ply Roofing	1 lb/ft^2	AISC 14th Edition – Table 17-13
0.8” Laminated Glass	8.2 lb/ft^2	
MEP	15 lb/ft^2	

Table 3.2, Unfactored Dead Load	
Floor Level	Load (kip)
Ground	2425.2
1	3325.7
2	3289.7
3	3289.7
4	3289.7
5	3289.7
Roof	3248.9

Once material quantities and material weight were determined, floor weight was determined. Items not included in the floor weight are the metal decking, joists, and structural steel members. Only after sizing the metal decking, joists, and structural steel members were the items included in the floor weight. A collateral load, of 5 lb/ft^2, was included in the dead load to account for unforeseen items.

Assumptions were made to accelerate and simplify the take-offs and load determination. The assumptions are as follows:

1. Metal deck has equal rib volume
2. All beams are identical to the beam in the typical bay
3. All girders identical to the girder in the typical bay
4. Glazing and concrete are the only façade materials
5. All floors except for the roof use the same type of concrete

Live Loads

LMOB is classified as a type B occupancy, by the 2009 IBC. The outcome of the classification is the use of office live loads. The other live load used to analyze the gravity system is associated with emergency egress. Due to the lack of access to the actual live loads used by the structural consultant, the 2003 IBC live loads were compared to the ASCE 7-05 live loads. Comparison of the live loads is on Table 3.3.

Table 3.3, Live Load Comparison		
Description	2003 IBC	ASCE 7-05
Stairs	100 lb/ft^2	100 lb/ft^2
Lobby & First Floor Corridor	100 lb/ft^2	100 lb/ft^2
Corridors Above First Floor	80 lb/ft^2	80 lb/ft^2
Ordinary Flat Roofs	To Be Calculated	20 lb/ft^2
Partitions	20 lb/ft^2	15 lb/ft^2

The option to use live load reductions was not taken up. Primary reason is that there is a likelihood that the busy hospital will expand its use of facility. Already the hospital occupies 39700 ft^2 of LMOB and has added a parking garage to accommodate additional patients. Another reason, it is likely that the facility will see new equipment, un-foreseen by the designers, in the future.

Table 3.4, Unfactored Live Load	
Floor Level	Load (kip)
Ground	2313.6
1	2001.7
2	2103.9
3	2103.9

4	2103.9
5	2103.9
Roof	528.8

Like the dead load calculations, live loads are broken down by floor level (Table 3.4).

Rain & Snow Loads

Location of LMOB was the deciding factor in whether rain or snow loads controlled. Being that the facility is in Largo, Florida; Figure 7-1 in ASCE 7-05 indicates that the ground snow load is zero. The result is no snow roof loads. Rain load was determined through the use of ASCE 7-05 and the International Plumbing Code (IPC). A ponding instability investigation was not required by ASCE 7-05, because the roof slope is a 1/4" rise for every 12" horizontal. Thus there was no study of ponding potential on the roof.

The hourly rain rate for Largo, Florida wasn't in the standards; the closest city's hourly rain rate was used. Tampa, Florida is the closest city to Largo, Florida. It was determined that the rain load is greater than the live roof load. In many calculations, the rain load (27.89 lb/ft^2) substituted the live roof load (20 lb/ft^2).

Gravity Spot Checks

Deck & Joist

Determining the building weight was the primary reason to size the deck and joist. All decks and joist shall use of cementitious fire protection, to achieve a 2-hour fire rating required by the FBC. There were only two assumptions made concerning decks; as follows: the deck has equal rib sizes, and all decks are 3 spans. Figure 3.1 and 3.2 shows the deck and joist placement.

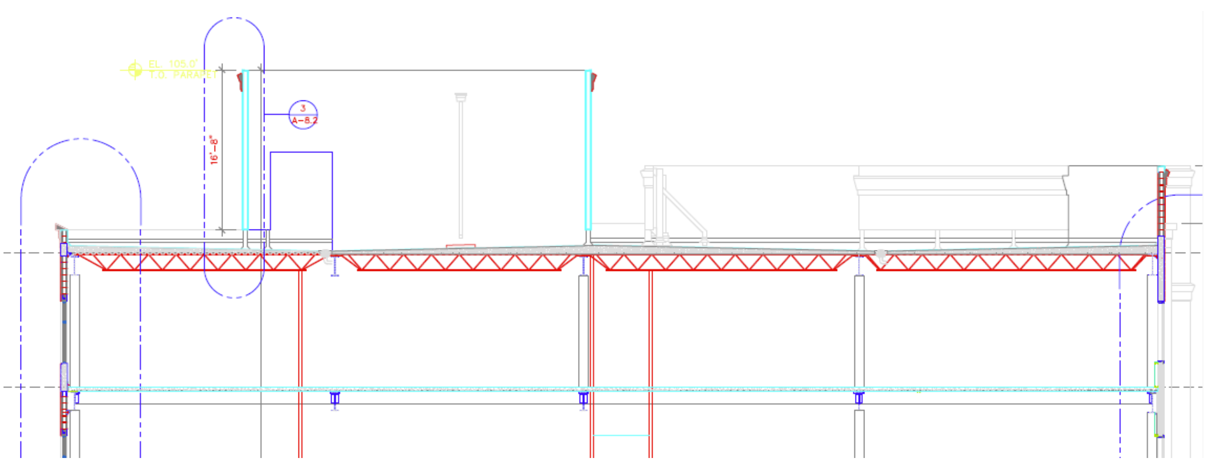

Figure 3.1, Roof Structure
Source: Oliver, Glidden, Spina & Partners

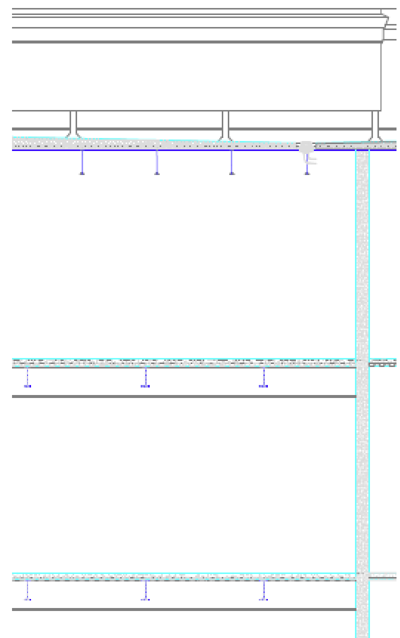

Figure 3.3, Joist and Beam Offsets
Source: Oliver, Glidden, Spina & Partners

Rain and dead load was used to size the metal roof deck instead of recommended the roof live and dead load. The 27.89 lb/ft^2 rain load is greater than 20 lb/ft^2 live roof load. From the spot check, the original 1.5" thick metal roof deck spanning 5'-6" is sufficient to resist the superimposed rain and dead load.

The only deviation with the original deck and joist design, appears to be the joist. The spot check showed that a 22K6 joist, also the lightest, is required to support the rain and dead load. Depth of the designed joist is 20" deep, this is a 10 percent difference with the spot check. The difference can be due to a number of factors:

1. Actual rainfall rate could be smaller than the substitute (Tampa, Florida)
2. Use of the prescribed live roof load instead of the rain load
3. Selection of heavier member but with less depth

See Table 3.5 for comparison of the decks and joists used in the original design and spot check.

Table 3.5, Comparison of Original Decks and Joist with Spot Check		
Component	Original	Spot Check
Roof Deck	1.5B	1.5B24
Floor Deck	2VLI	2VLI22
Roof Joist	20" Depth	22K6

Beam & Girder

Beams and girders spanning the largest typical bay, 33'-0"x33'-0", were used for the floor system spot check. In addition to spot checking, the calculated size of the beams and girders were factored into the weight of the building. The members were evaluated for flexural capacity and deflection. It was assumed that the girders use shear studs to have composite action and that shear is completely transferred from the composite slab to the girder. Comparison of the typical beams and girders can be referenced in Table 3.6.

Table 3.6, Comparison of Original Beams and Girders with Spot Check		
Component	Original	Spot Check
Beam	W16	W14x74
Girder	W24	W24x76

There are slight differences between the original beam sizes. The difference is approximately 14 percent, some possible explanations for the difference are:

1. Vibration criteria not evaluated in the spot check
2. Use of economical and predominate sections
3. Greater gravity load due to additional mechanical equipment

Column

Spot check calculations of the typical column, at the intersection of lines B and 2, were implemented once the other structural steel members were sized according to the ASCE 7-05 loads. Column, B-2, was selected because it is an interior column not part of the lateral system. As a result it does not experience lateral loads, as the exterior columns. In terms of bracing, beams and girders prevent the column from having an un-braced length greater than 16'.

Due to the existence of the specially reinforced shear walls, it was assumed that the typical column is pin base. Also, it was assumed that the column did not change size to suit the changing gravity loads. Instead all columns are the same size, to ensure ease of construction and reduce complex column splice connections.

Neither the live load nor live roof load were reduced. All floor levels, other than the roof, were loaded with 80 lb/ft^2 live load. The spot check resulted in W14x120 as the lightest column size to resist gravity loads. McCarthy Associates used a W12 column, the difference is 14%. Reason for a slightly smaller original column can be attributed to:

1. Smaller live load assumption due to either different load criteria or use of live load reduction
2. Use of predominant sections

Lateral Loads

Wind Load

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 canopy
2. Due to multiple roof levels, that average roof elevation 95'-6" was utilized
3. Gust factor of one was used, since the calculated gust factor is 0.89 and 0.91 (depending on wind orientation)
4. Internal pressurization is unlikely due to use of impact resistant glazing
5. Type III for importance category
6. Flexible building

A majority of the wind calculations were done by hand. Excel was only used to determine the total overturning moment and the wind load distribution at each story. All wind calculations and site characteristics are available for reference in Appendix D. Shown below in Figure 4.1, 4.2, 4.3, and 4.4 is the MWFRS wind distribution and story shear.

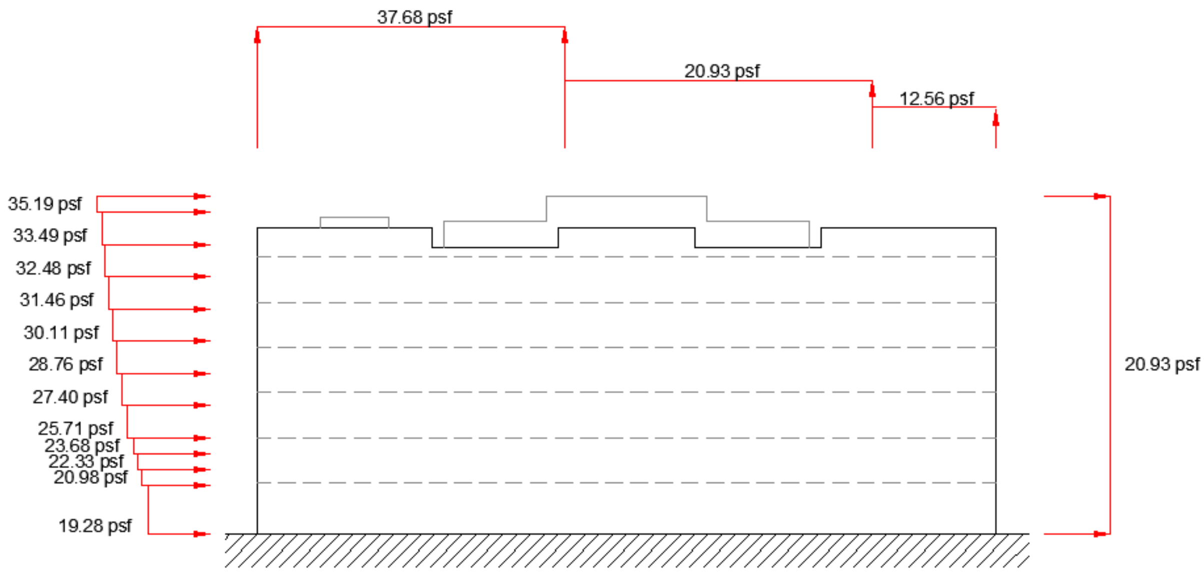

Figure 4.1, MWFRS East/West Wind Load Distribution

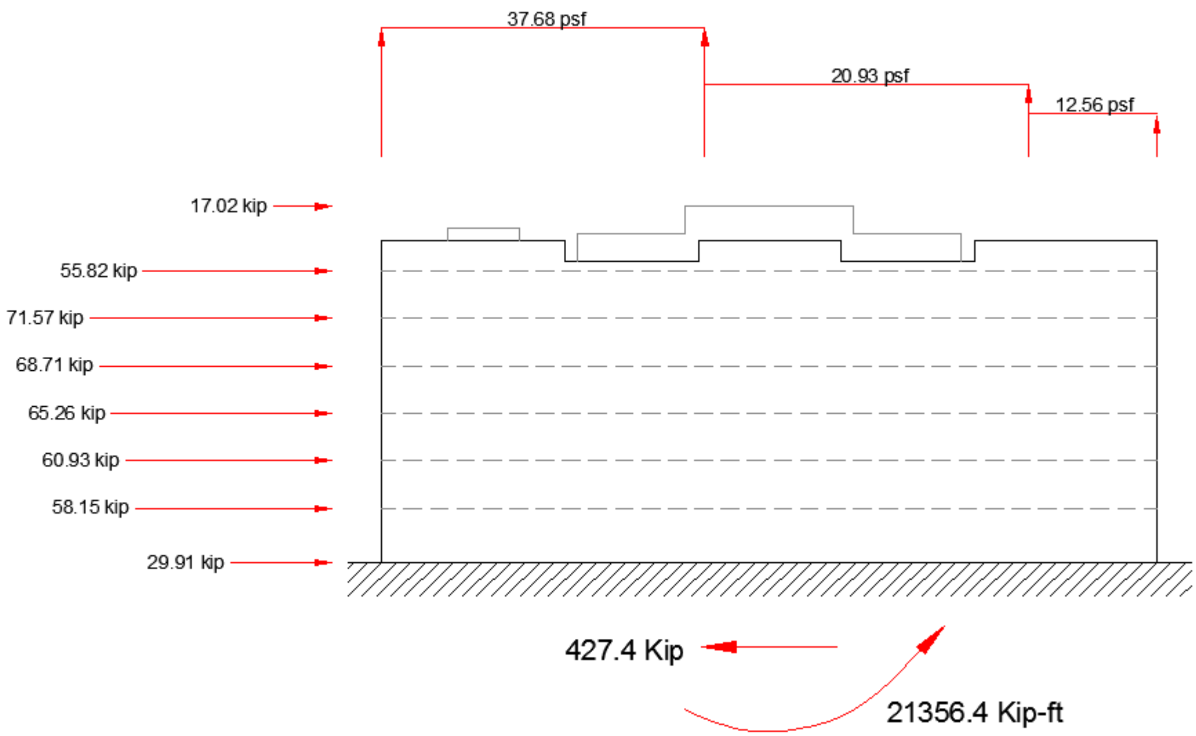

Figure 4.2, MWFRS Loads - East/West

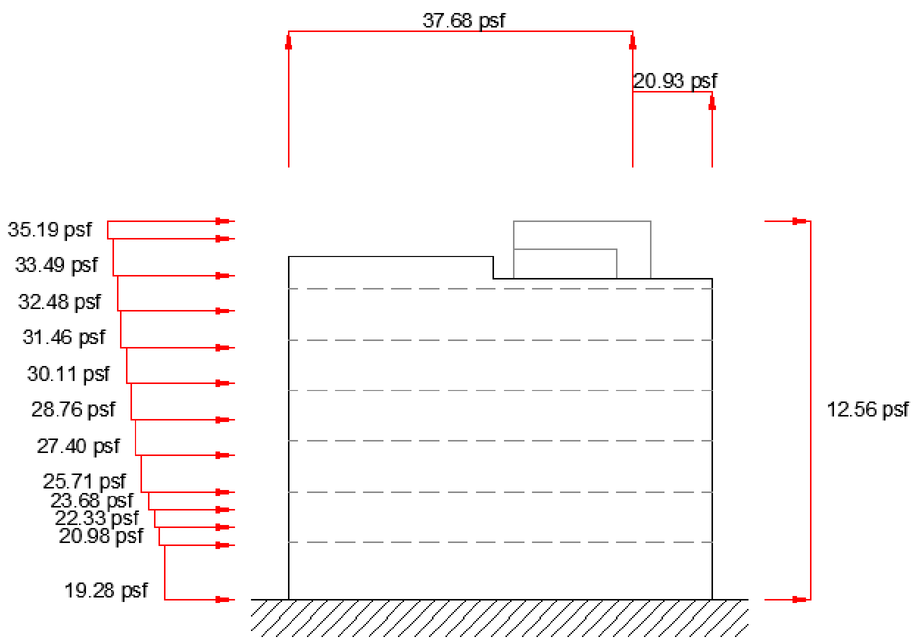

Figure 4.3, MWFRS North/South Wind Load Distribution

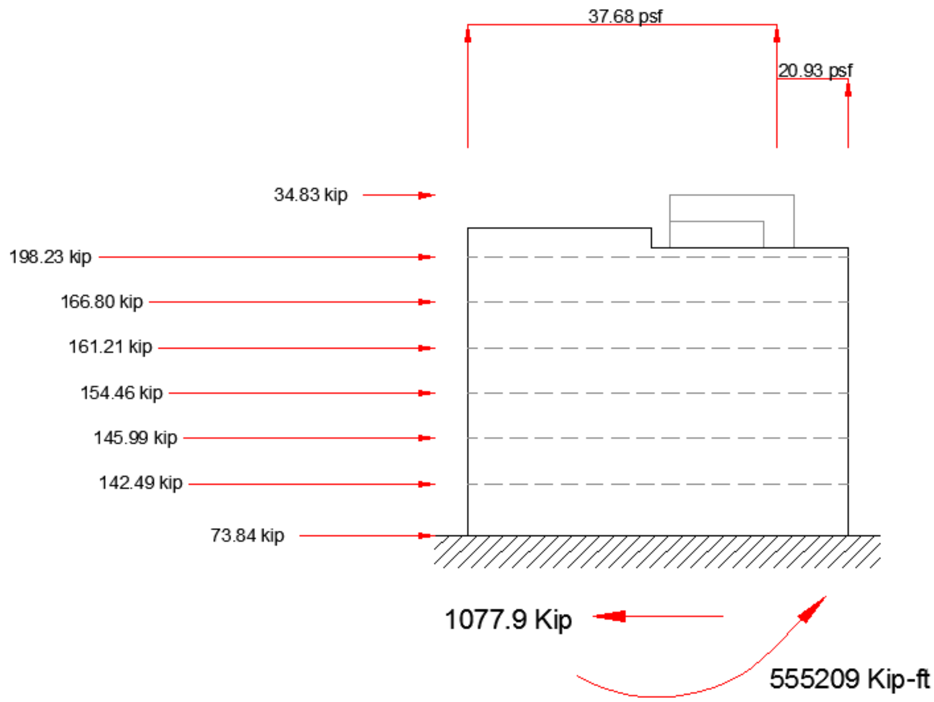

Figure 4.4, MWFRS Loads – North/South

From the wind analysis, the MWFRS loads due to wind in the North/South direction controls over the East/West direction. MWFRS loads on the North/South sides are more than two times that of the East/West sides. The higher wind loads can be attributed to greater façade area of the North/South building sides.

Seismic Load

Equivalent Lateral Force method was used to determine the seismic loads on LMOB. The seismic load, an inertia load, is caused by ground acceleration. Seismic load transfers from the floor diaphragms to the shear walls. The shear walls enclose the emergency stairwells and elevator core, an illustration of the shear wall locations are highlighted black in Figure 4.5. No seismic loads were transferred to the top roof, at 105', due to the lack seismically designed masonry structure supporting the diaphragms (Figure 4.6).

The fundamental period of the facility is 0.66 seconds, per ASCE 7-05 equation 12.8-9. Using ASCE 7-05 it was discovered that the facility doesn't have to resist significant seismic forces, approximately 314.6 kip. This translates to 1.4% of the effective building weight. Live, dead, and rain loads determined previously in were used to calculate the effective building weight. Table 4.1, describes the effective building weight by floor level. Torsion irregularity of the facility was ignored in the seismic analysis. For the seismic load diagram, please see Figure 4.7.

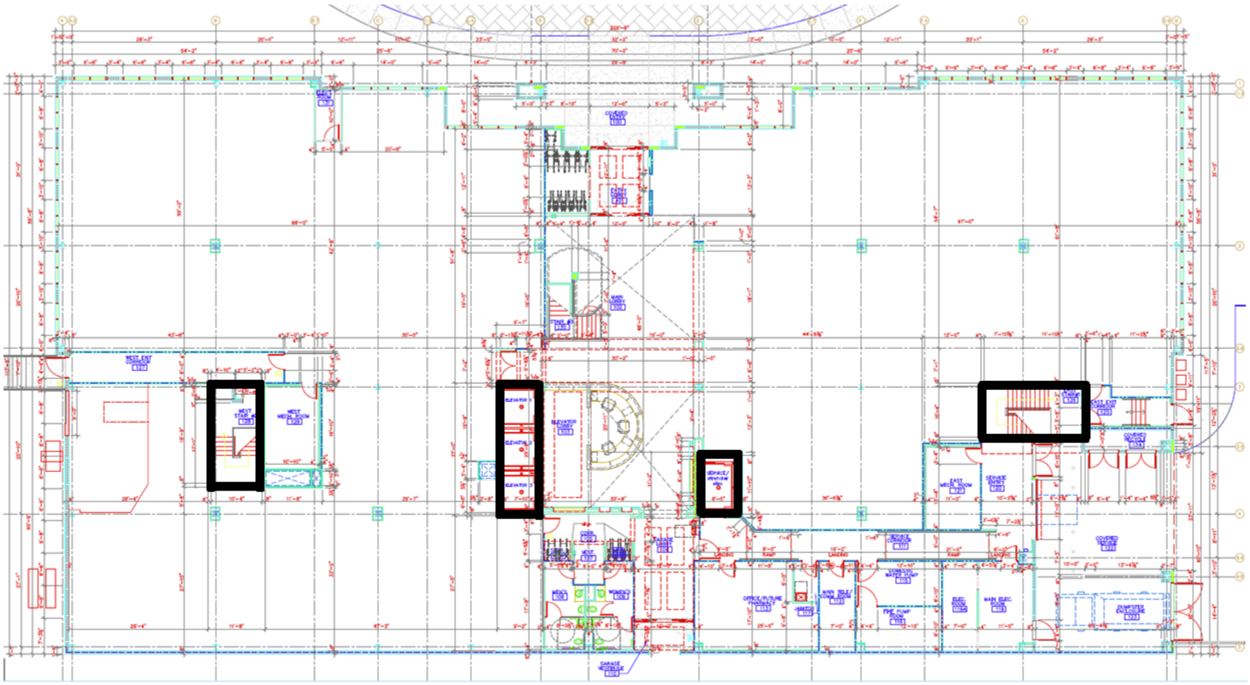

Figure 4.5, Locations of Shear Walls
Source: Oliver, Glidden, Spina & Partners

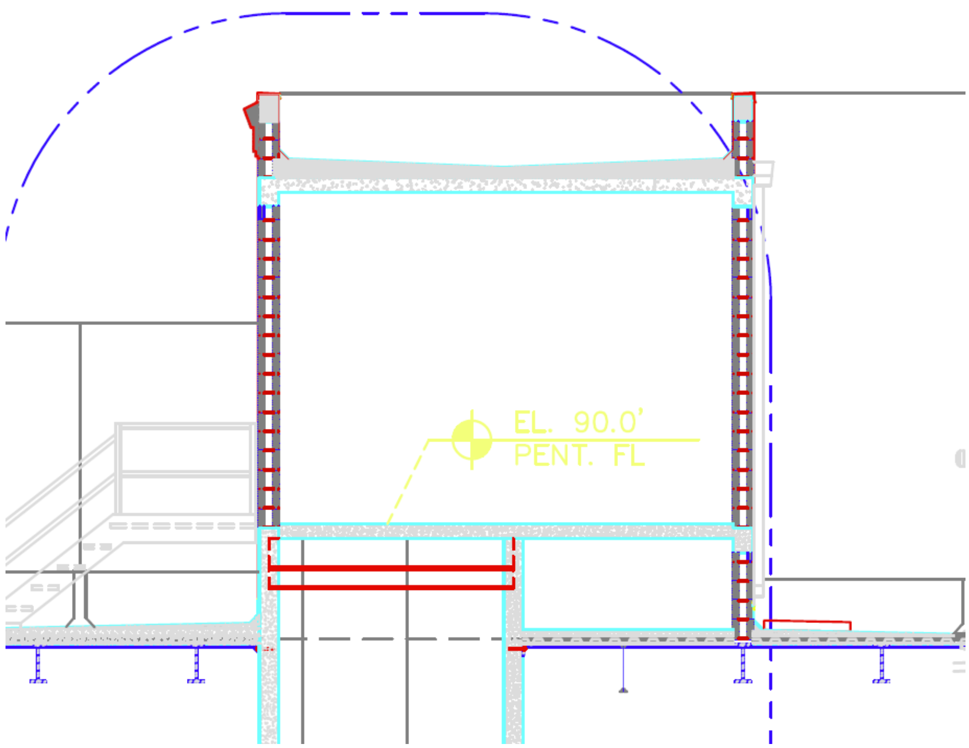

Figure 4.6, Non Seismic Design Top Roof
Source: Oliver, Glidden, Spina & Partners

Table 4.1, Effective Weight	
Floor Level	Level Effected Weight (kip)
Ground	0
1	3826.1
2	3891.6
3	3836.6
4	3770.4
5	3764.2
Roof	3381.1

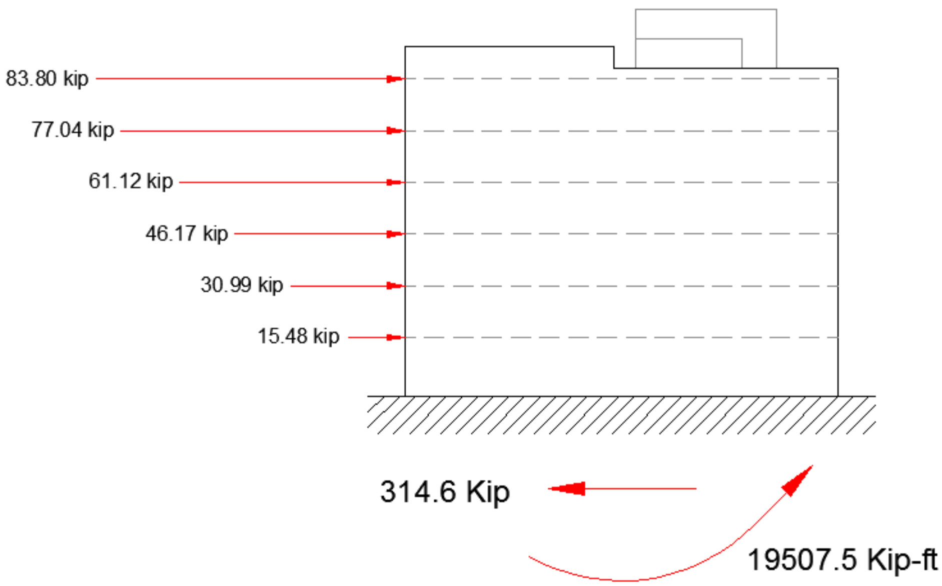

Figure 4.7, Seismic Loads
Source: Oliver, Glidden, Spina & Partners

Conclusion

Technical Report I studies the structural system of the Largo Medical Office Building (LMOB) through analysis of the available plans, and design load discussions. Many simplifying assumptions were made; primarily concerning the facility's shape and structural components, as well as to satisfy the ASCE 7-05 criteria.

Spot check for the current gravity system, using the AISC 14th Edition Steel Manual, showed that decks and girders are adequate. It was discovered that there are slight discrepancies with the original joist, beams, and columns. The maximum difference is 14%, when comparing the depth of the members. It is likely that the small difference is caused by or a combination of vibration requirement, live load reductions, or use of predominant sections. Member weight comparison was not implemented, due to the lack of information in the available drawings

Much of the dead, live, rain and snow loads were determined through the use of ASCE 7-05. These gravity loads along with the gravity member weights were used to analyze LMOB for seismic loads. As it turned out the equivalent lateral system is only 1.4 percent of the effective building weight. Unfortunately, comparison with the original seismic load was not possible, due to lack of information on the available drawings. This was the same case for the wind analysis.

After analysis of both the wind and seismic loads, it was found that the wind loading in the North/South direction is the controlling lateral scenario. Wind loading in the North/South direction dominates in base shear and overturning component. Due to the Florida's low seismic activity but high hurricane risk it is logical that the facility experience high wind loads when compared to the seismic load.

Appendix A: Floor Plans & Elevation

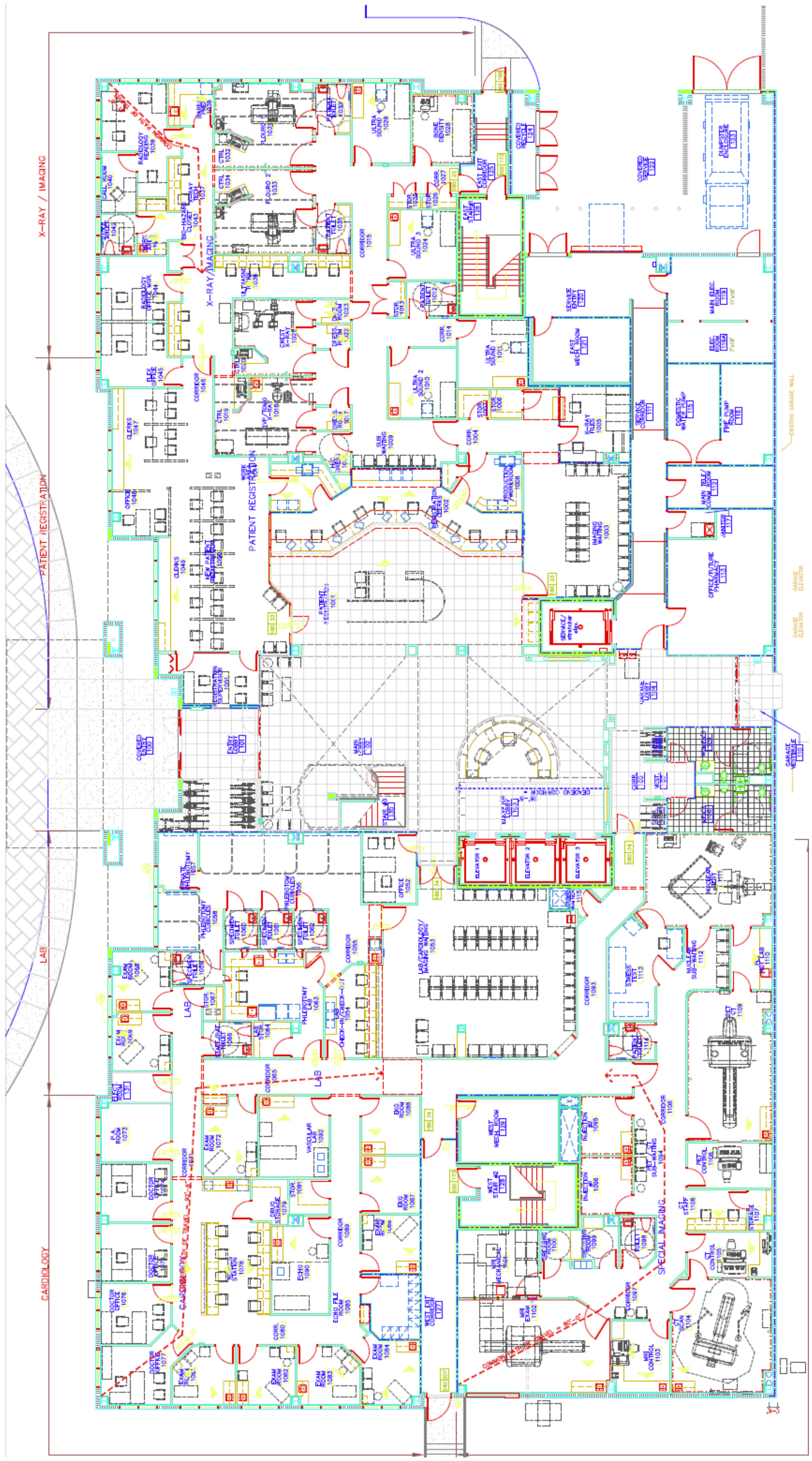

Figure AA.1, First Floor Plan w/ Tenant Build-Out
Source: Oliver, Glidden, Spina & Partners

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

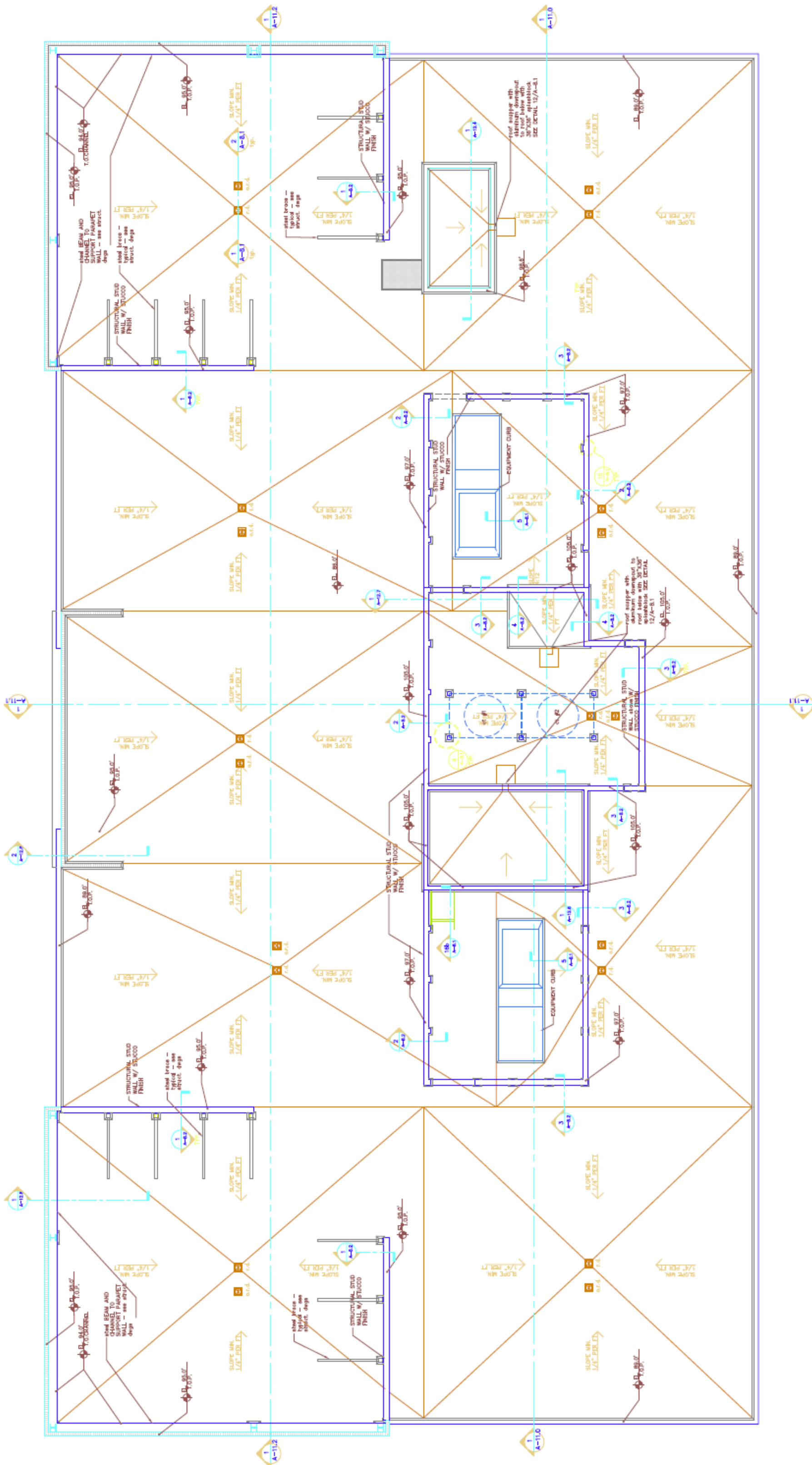

Figure AA.3, Roof Plan
Source: Oliver, Glidden, Spina & Partners

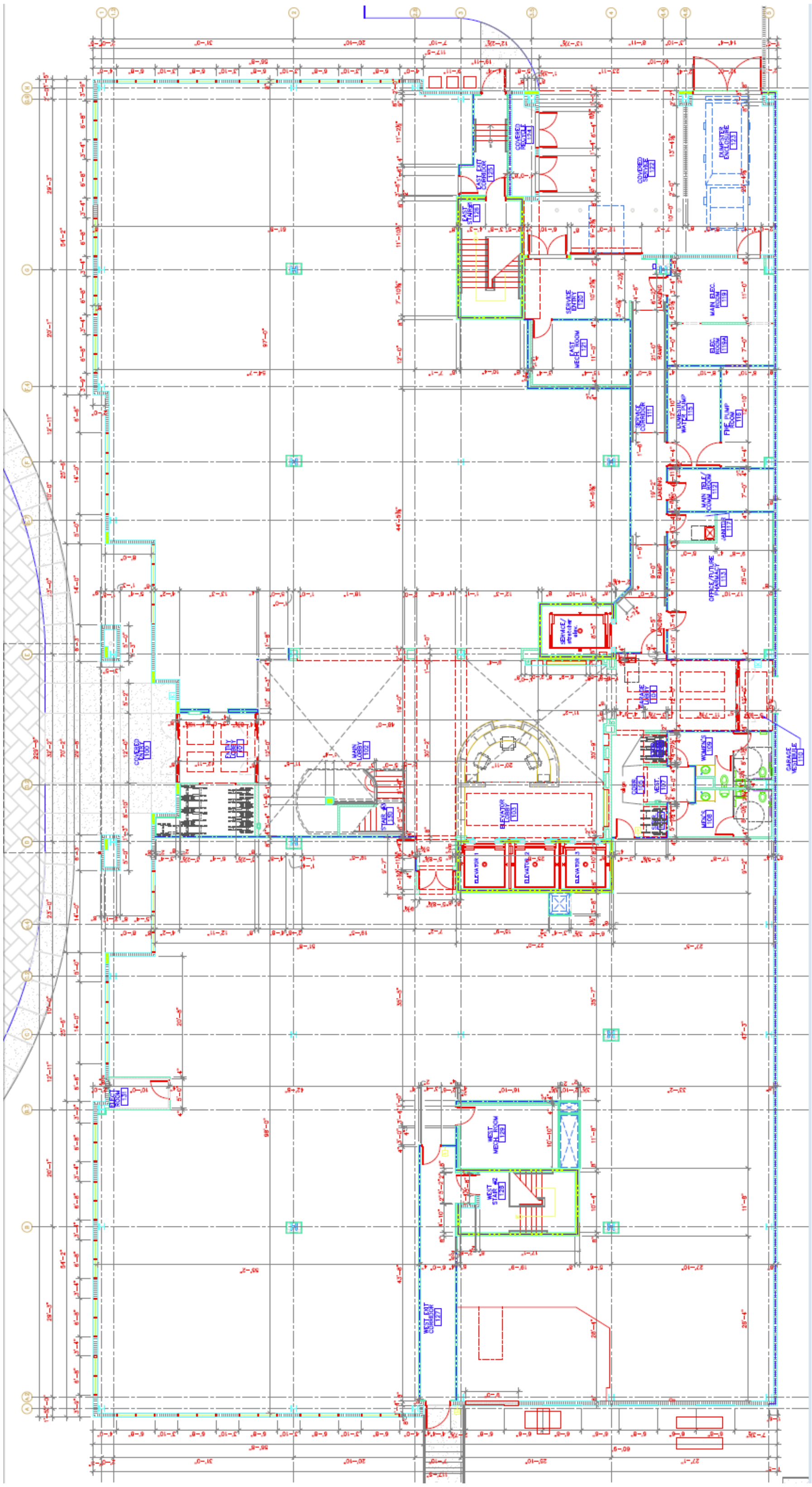

Figure AA.4, Typical Column Layout
Source: Oliver. Glidden. Spina & Partners

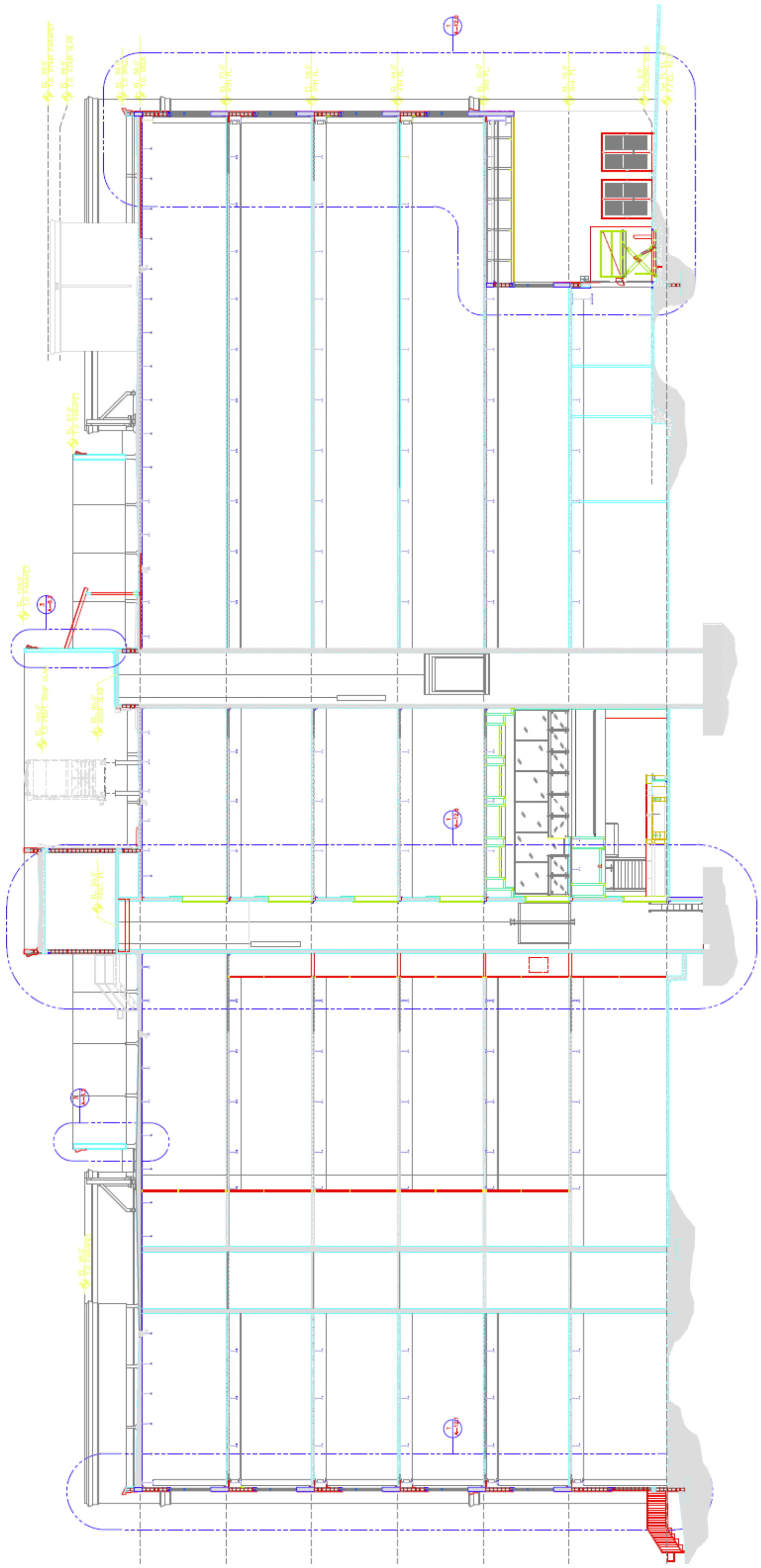

Figure AA.5, Longitudinal Building Section
Source: Oliver, Glidden, Spina & Partners

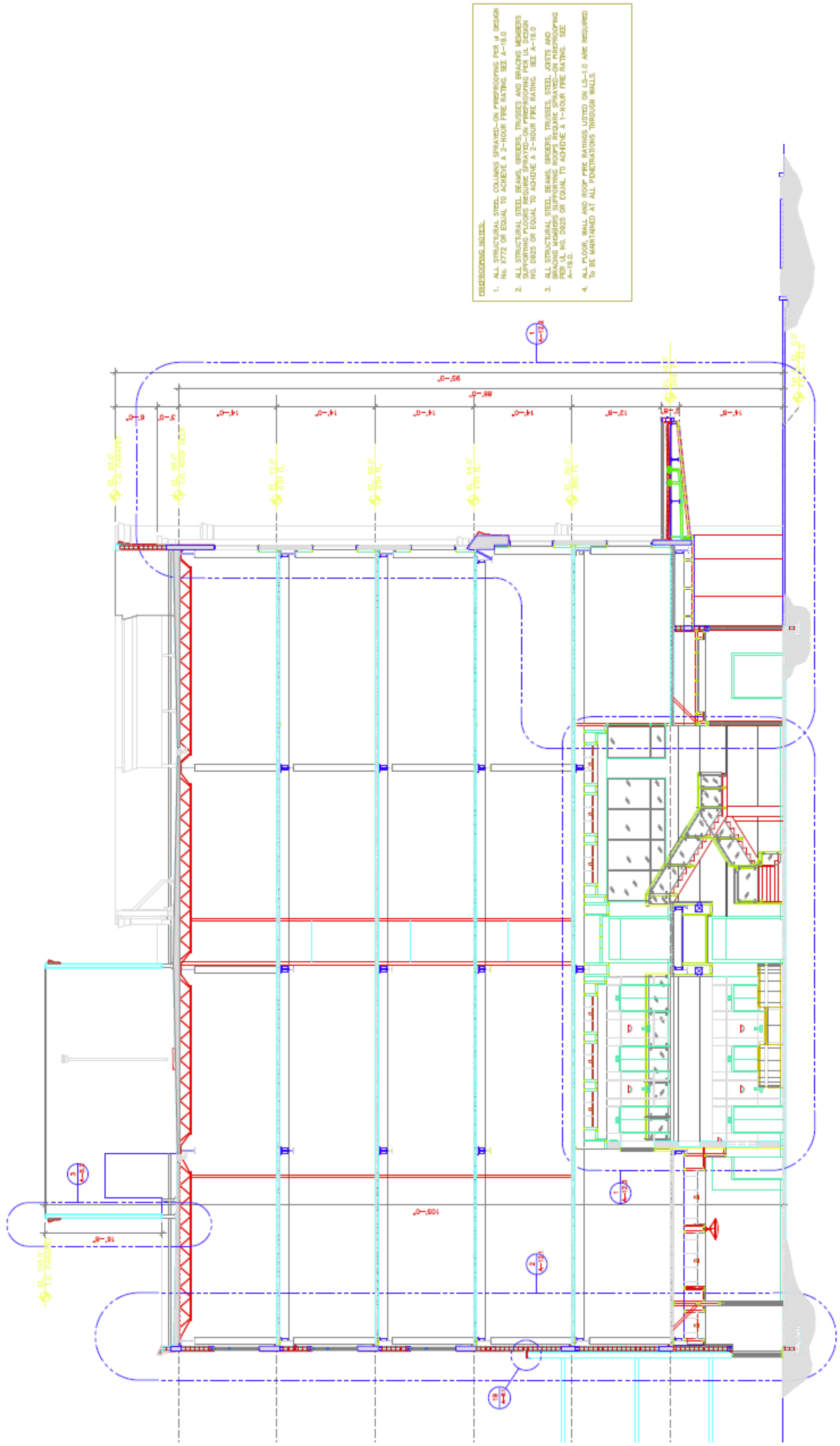

Figure AA.6, Building Section
Source: Oliver. Glidden. Spina & Partners

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 corrugations

$t_{floor} = 5''$
$t_{floor,eq} = t_{floor} - d_{deck}/2 = 4''$, 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 corregations

$\mathcal{T}_{floor} = 10\ 1/8'' \rightarrow 3\ 11/16''$
$\mathcal{T}_{floor,avg} = (10\ 1/8 + 3\ 11/16)/2$
$\mathcal{T}_{floor,avg} \approx 7''$

$\mathcal{T}_{floor,eq} = \mathcal{T}_{floor,avg} - d_{deck}/2 \approx 6.25''$, use to determine conc. weight

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

1) Level: 0

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

2) Level: 1

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

3) Level: 2→5

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

4) Level: Roof

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

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c) Dead Weight of Flooring

Floor Level	0				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

→

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

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

Rain fall 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

$R = 5.2(d_s + d_h)$

$R = 5.2(3.63 + 1.738)$

$R = 27.89 \text{ lb/ft}^2 > (\text{Roof live load} = 20 \text{ lb/ft}^2)$

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Appendix C: Gravity Load Calculations

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)

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 $\ell/180 = 4/3 * 90$

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

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

May use 1.5B24

* 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

* Serviceability Criteria $\Delta \le \ell/180$, Supporting Non-Plaster Ceiling

2) Floor Deck

* Assume 2 hr 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
$W_u = 548.9 + 6.6(9.2)$, $W_{Joist} = 9.2\ lb/ft$
$W_u = 609.6\ lb/ft$

* Servicability Criteria
$\Delta \le \ell/180$, supporting Non Plaster ceiling

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

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

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

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

$M_u = W_u \ell^2/8$
$M_u = (1765 + 1.2\,W_{bm})(33^2)/8$
$M_u = 240261 + 163.4\,W_{bm}$

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

→

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

Check W14×74 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

$\Delta_{TL} \le \ell/240$

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

$\Delta_{TL} \le 1.65''$

$W_T = (DL + LL)$

$W_T = 8.25(71.6 + 80) + W_{bm}$

$W_T = 1250.7 + W_{bm}$

$\Delta_{TL} = \dfrac{5(1250.7 + 74)(33^4)}{384(29\times10^6)(795)}$

$\Delta_{TL} = 1.53'' < 1.65''$ ✓, Good

2) Girder

- Assume girders use shear studs to have composite action.
- For ease in constructability assume all beams are W16×89

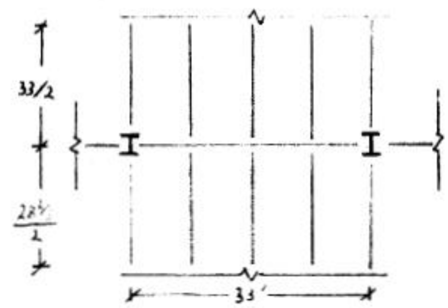
$\ell_{brace} = 0$

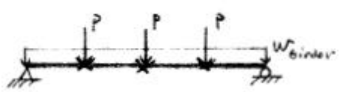
$M_u = \frac{33}{4}(P_u)(1.5) + 1.2 W_{Girder}\left(\frac{33^2}{8}\right)$

$M_u = \frac{33}{4}(98.4)(1.5) + 1.2 W_{Girder}\left(\frac{33^2}{8}\right)$

$M_u = 1217.7 + 204.2\, W_{Girder}$

in Kip





$P_D = [0.150(4/12) + 0.015 + 0.005 + 1.62](8.25)(33 + 28\tfrac{2}{3})/2 + 0.089(33 + 28\tfrac{2}{3})/2$

$P_D = 52.1 + 2.7$

$P_D = 54.8$ kip, unfactored Dead Load

$P_L = 0.080(8.25)(33 + 28\tfrac{2}{3})/2$

$P_L = 20.4$ 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

$W_u = 1.2\, W_{Girder}$

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

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

$A_s f_y = 0.85 f'_c b_{eff}\, a$, 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.

→

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Thaison Nguyen | Gravity Spot Check | 4/5

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

- Assume perfect shear transfer

$A_s = 22.4 \text{ in}^2$

$a = 0.149(22.4)$

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

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

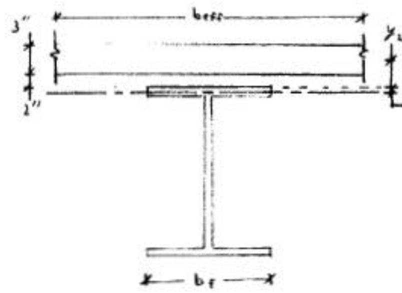
$\frac{A_s F_y - 0.85 f'_c b_{eff} T_{solid}}{2 F_y b_f} = x$

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

$x = 110.2 / 899$

$x = 0.123''$

$y_1 = x$



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

$M_u = 1233.2 \text{ kip} \cdot \text{ft}$

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

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

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

$\phi M_n = 1271.1 \text{ kip} \cdot \text{ft} > 1233.2 \text{ kip} \cdot \text{ft}$ ✓, Good.

$\Delta_{LL} \leq \ell/360$, 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)

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

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

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

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

$P_{constr} = 48.3 + 2.7 = 51.0 \text{ kip}$

$W_{Girder} = 0.076 \text{ kip/ft}$

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

$\Delta_{LLD} = 0.033 + 0.677$

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

$0.71'' < 1.1''$ ✓, no shoring req.

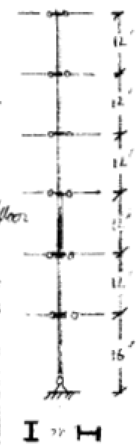
→

Thaison Nguyen		Gravity Spot Check	5/5

C) Column

Location: B-2
*Assume pinned base

K = 1



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

$A_{trib} = \frac{(31.25+33)}{2} * \frac{(33+28^2/_3)}{2}$
$A_{trib} = 990.5 \text{ ft}^2/\text{floor}$

$P_R = 0.02780(990.5)(1)$
$P_R = 27.6$ kip , rain load

Dead Load Components [2]	Weight	Notes
LW CONC.	113 lb/ft³	Arch Graphics Standard 11 Ed.
Roof Deck	1.46 lb/ft²	Vulcraft 2008 Deck Manual, 1.5B24
Joist	9.2 lb/ft	Vulcraft 2008 Joist Manual, 22K6
3 Ply - Roofing	1 lb/ft²	AISC 14 Ed Table 17-13
NW CONC	150 lb/ft³	AISC 14 Ed Table 17-13
Floor Deck	1.62 lb/ft²	Vulcraft 2008 Deck Manual, 2VLI 22
Beam	74 lb/ft	AISC 14 Ed , W14x74
Girder	76 lb/ft	AISC 14 Ed , W24x76
MEP	15 lb/ft²	

[2] 5 lb/ft² 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

$K\ell_x = 1(16) = 16'$
$K\ell_y = 1(16) = 16'$, 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

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Appendix D: Wind Load Calculations

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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 Section 6.5.6.3
Mean Height : 95.5'

Building Face	North	South	East	West	Roof
$A_{\text{tributary}}$ (ft^2)	21412.9	21412.9	10957.9	10957.9	26440

* Flexible building
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 Case I : CCL	K_z Case II : MWFRS	Notes
≤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

$$a = \max \begin{cases} 0.1 \cdot \text{Least Horizontal Dimension} \\ 3' \end{cases}$$, 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
B (ft)	229.5	117.42

	Windward	Leeward		Side	Roof			Notes
L/B		1.95	0.51					ASCE 7-05 Fig. 6-6
Distance from Windward Edge					[0, 95'-6")	[95'-6", 191'-0")	[191'-0", ∞)	ASCE 7-05 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	↓

→

Wind 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	19170	2243	8715	2243	18845	5941	1654	
GCp, 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	18845	5941	1654.4	
GCp, CCL	0.6	0.6	0.7	1	0.7	1	0.9	1.6	2.3	ASCE 7-05 Fig 6-17

Turbulence Intensity Factor (c) = 0.3 , ASCE 7-05 Table 6-2

$z_{min} = 30$, ASCE 7-05 Table 6-2

$\bar{z} = 0.6 \cdot$ Mean Height , ASCE 7-05 Table 6-2, 6.5.8.1

$\bar{z} = 0.6(95.5)$

$\bar{z} = 57.3$

Turbulence Intensity ($I_{\bar{z}}$) = $c\left(\frac{33}{\bar{z}}\right)^{1/6}$

$I_{\bar{z}} = 0.3\left(\frac{33}{57.3}\right)^{1/6}$

$I_{\bar{z}} = 0.274$

$n_1 = 100/$Mean Height , ASCE 7-05 C6.5.8, Eq C6-17

$n_1 = 1.047$

$$g_R = \sqrt{2\ln(3600 n_1)} + \frac{0.577}{\sqrt{2\ln(3600 n_1)}}$$

$g_R = 4.06 + 0.577/4.06$

$g_R = 4.2$

$\ell = 320$, ASCE 7-05 Table 6-2

$\bar{\epsilon} = 0.33$

$\bar{b} = 0.45$

$\bar{\alpha} = 7$

$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}$

$L_{\bar{z}} = 320(57.3/33)^{0.33}$

$L_{\bar{z}} = 384.62$

$$Q_{\perp North/South} = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$$

$$Q_{\perp North/South} = \sqrt{\frac{1}{1 + 0.63\left(\frac{229.5 + 95.5}{384.62}\right)^{0.63}}}$$

$Q_{\perp North/South} = 0.799$

$$Q_{\perp East/West} = \sqrt{\frac{1}{1 + 0.63\left(\frac{117.42 + 95.5}{384.62}\right)}}$$

$Q_{\perp East/West} = 0.835$

→

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$$V_z = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right)$$
$$V_z = 0.45\left(\frac{57.3}{33}\right)^{?}(130)\left(\frac{88}{60}\right)$$
$$V_z = 4082.9 \text{ ft/s}$$

$$N_1 = n_1 L_z / V_z$$
$$N_1 = 1.047(384.62)/4082.9$$
$$N_1 = 0.10$$

$$n_{1,h} = 4.6 n_1 h / V_z \text{ , for } R_h$$
$$n_{1,h} = 4.6(1.047)(95.5)/(4082.9)$$
$$n_{1,h} = 0.11$$

$$n_{1,B=229.5} = 4.6 n_1 B / V_z \text{ , for } R_B$$
$$n_{1,B=229.5} = 4.6(1.047)(229.5)/4082.9$$
$$n_{1,B=229.5} = 0.27$$

$$n_{1,B=117.42} = 4.6(1.047)(117.42)/4082.9 \text{ , for } R_B$$
$$n_{1,B=117.42} = 0.14$$

$$n_{1,L} = 15.4 n_1 L / V_z \text{ , for } R_L$$
$$n_{1,L=117.42} = 15.4(1.047)(117.42)/4082.9$$
$$n_{1,L=117.42} = 0.46$$

$$n_{1,L=229.5} = 1.5(1.047)(229.5)/4082.9$$
$$n_{1,L=229.5} = 0.91$$

$$R_n = \frac{7.47 N_1}{(1+10.3N_1)^{5/3}}$$
$$R_n = \frac{7.47(0.1)}{[1+10.3(0.1)]^{5/3}}$$
$$R_n = 0.23$$

$$R_h = \frac{1}{n_{1,h}} - \frac{1}{2n_{1,h}}(1-e^{-2n_{1,h}})$$
$$R_h = \frac{1}{0.11} - \frac{1}{2(0.11)}(1-e^{-2\times0.11})$$
$$R_h = 0.93$$

$$R_B = \frac{1}{n_{1,B}} - \frac{1}{2n_{1,B}}(1-e^{-2n_{1,B}})$$
$$R_{B,B=229.5} = 0.84 \qquad R_{B,B=117.42} = 0.91$$

$$R_L = \frac{1}{n_{1,L}} - \frac{1}{2n_{1,L}}(1-e^{-2n_{1,L}})$$
$$R_{L,L=117.42} = 0.75 \qquad R_{L,L=229.5} = 0.59$$

$\beta = 1$; ASCE 7-05 C6.5.8 ,
assume conservative dampening ratio

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$$

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$$R_{B=229.5} = \sqrt{\frac{1}{1}(0.23)(0.93)(0.84)(0.53+0.47*0.75)}$$

$$R_{B=229.5} = 0.40$$

$$R_{B=117.42} = \sqrt{\frac{1}{1}(0.23)(0.93)(0.91)(0.53+0.47*0.59)}$$

$$R_{B=117.42} = 0.40$$

$$G_f = 0.925\left(\frac{1+1.7I_z\sqrt{g_Q^2Q^2+g_R^2R^2}}{1+1.7g_vI_z}\right)$$

$$G_{f,B=229.5} = 0.925\left(\frac{1+1.7*0.274\sqrt{3.4^2*0.799^2+4.2^2*0.4^2}}{1+1.7*3.4*0.274}\right)$$

$$G_{f,B=229.5} = 0.89$$

$$G_{f,B=117.42} = 0.91$$

* Assume $G_f = 1$, conservative and proximity of G_f to 1.

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

$q_h = 0.00256(0.99)(1)(0.85)(130^2)(1.15)$; for leeward, 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 design.

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

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

$V_{base} = \Sigma$ Wind load at floor diaphram
$V_{base,\ wind \perp North/South\ wall} = 1077.9$ kip
$V_{base,\ wind \perp East/West\ wall} = 427.4$ kip

$M_{tot\ overturn,\ wind \perp North/South\ wall} = 55520.9$ kip-ft
$M_{tot\ overturn,\ wind \perp East/West\ wall} = 21356.4$ kip-ft

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	19.28	12.56	20.93	29.31	12.56	20.93	37.68	7.54	7.54
20	29.6	26.2	20.98								
25	29.6	27.9	22.33								
30	29.6	29.6	23.68								
40	32.1	32.1	25.71								
50	34.3	34.3	27.40								
60	35.9	35.9	28.76								
70	37.6	37.6	30.11								
80	39.3	39.3	31.46								
90	40.6	40.6	32.48								
100	41.9	41.9	33.49								
120	44.0	44.0	35.19								

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	73.84	29.91	1077.86	427.38	0.00	0.00
1	16.0	23	142.49	58.15	1004.01	397.47	2279.83	930.44
2	30.0	37	145.99	60.93	861.53	339.32	4379.79	1827.84
3	44.0	51	154.46	65.26	715.53	278.39	6796.07	2871.36
4	58.0	65	161.21	68.71	561.08	213.13	9350.26	3985.42
5	72.0	79	166.80	71.57	399.87	144.42	12009.74	5153.36
Roof 1	86.0	95.5	198.23	55.82	233.06	72.84	17047.98	4800.84
Top	105.0		34.83	17.02	34.83	17.02	3657.24	1787.14

Design Wind Pressures (lb/ft²)													
Height (ft)	Velocity Pressure q_z (lb/ft²)		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		
≤ 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/ft²)													
Height (ft)	Velocity Pressure q_z (lb/ft²)		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		
≤ 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									

Appendix C: Seismic Load Calculations

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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 special reinforced concrete shear walls → Lateral system

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

1) Level : 1

$DL = DL_{slab} + DL_{deck} + DL_{BM} + DL_{Girder} - DL_{Floor\ op} + DL_{envelope}$

$DL = 1675.5 + \frac{1.62}{1000}(A_{gross} - A_{fl\ opening} - A_{stair}) + 217.6 + 74.8 + 7.2 + \left(\frac{1438.8 + 1183.7}{2}\right)$

$DL = 1675.5 + 39.3 + 217.6 + 74.8 + 7.2 + 1311.25$

$DL = 3325.7$ kip

$LL = 2001.7$ kip , value from Load Determination - DEAD, LIVE, RAIN of Appendix

$W_x = 3325.7 + 0.25(2001.7)$

$W_x = 3826.1$ kip

$DL_{BM} = \frac{W_{BM}}{spacing} * (A_{gross} - A_{fl\ opening} - A_{stair})$

$W_{BM} = 74$ 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

$DL_{Girder} = \ell_{Girder} W_{Girder}$

$W_{Girder} = 76$ 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

$DL = \{1822.6 + \frac{1.62}{1000}(A_{gross} - A_{fl\ opening} - A_{stair}) + 8.97(A_{gross} - A_{fl\ opening} - A_{stair}) + \frac{76}{1000}[680 + 304.2 + 29.25 + 32\tfrac{1}{6}] + 4.1\}4 + \left[\frac{1}{2}(1183.7) + 1194.2 + 1073.7 + 1051.7 + \frac{1}{2}(1061.3)\right]$

$A_{gross} - A_{fl\ opening} - A_{stair} = 26440 - 293 - 609 = 25538$

$DL = 8706.5 + 4452.1$

$DL = 13158.8$ kip

$LL = 2103.9 * 4$, value from Load Determination - DEAD, LIVE, RAIN of Appendix

$LL = 8415.6$ kip

→

Floor Level	DL envelope (k)	DL (kip)	W_x
2	1188.95	3365.6	3891.6
3	1133.95	3310.6	3836.6
4	1067.7	3244.4	3770.4
5	1061.5	3238.2	3764.2

3) Level: Roof

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

$DL = 1956.4 + 1292.5$

$DL = 3248.9$ kip

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

$LL = 528.8$ kip

$W_x = 3248.9 + 0.25(528.8)$

$W_x = 3381.1$ 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 Figure 22-1

$S_s = 0.063$

$S_1 = \frac{2.2}{100}$, ASCE 7-05 Figure 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

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$S_{ms} = S_s F_a$
$S_{ms} = 0.063(1.6)$
$S_{ms} = 0.101$

$S_{m1} = S_1 F_v$
$S_{m1} = 0.022(2.4)$
$S_{m1} = 0.053$

$S_{DS} = \frac{2}{3} S_{ms}$
$S_{DS} = 0.067$

$S_{D1} = \frac{2}{3} S_{m1}$
$S_{D1} = 0.035$

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 Figure 22-15
$T \sim C_t h_n^{x}$, ASCE 7-05 Equation 12.8-7
$T \sim 0.66$

$R = 6$, ASCE 7-05 Table 12.2-1
$k = 2.5$
$C_s = \frac{S_{DS}}{\frac{R}{I}}$, ASCE 7-05 Equation 12.8-3

$V_{base} = W_{x,tot} C_s$
$V_{base} = 22470(0.014)$
$V_{base} = 314.6$ kip

$\bar{k} = 1 + \left(\frac{T - 0.5}{2.5 - 0.5}\right)(2-1)$, ASCE 7-05 Equation 12.8-12
$\bar{k} = 1 + \left(\frac{0.66 - 0.5}{2.5 - 0.5}\right)$
$\bar{k} = 1.078$

2) Story Shear (V_x) and Overturning Moment

$C_x = \frac{W_x h_x^{\bar{k}}}{\sum W_i h_i^{\bar{k}}}$
$F_x = C_x V_{base}$, equivalent lateral load at floor level

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AMPAD

Floor Level	h_x (ft)	W_x	$W_x h_x^k$	F_x (Kip)	Story Shear (Kip)
0	0	0	0	0	314.60
1	16	3826.1	75997.2	15.48	314.60
2	30	3891.6	152217.6	30.99	299.12
3	44	3836.6	226771.5	46.17	268.13
4	58	3770.4	300166.9	61.12	221.96
5	72	3764.2	378335.5	77.04	160.84
Roof	86	3381.1	411573.4	83.80	83.80

Floor Level	h_x (ft)	Overturning Moment = $F_x h_x$ (Kip·ft)
0	0	0
1	16	247.68
2	30	929.70
3	44	2031.48
4	58	3544.96
5	72	5546.88
Roof	86	7206.80

$M_{tot,overturning} = 247.68 + 929.7 + 2031.48 + 3544.96 + 5546.88 + 7206.8$

$M_{tot,overturning} = 19507.5$ kip·ft