FINAL REPORT

THE REVIVE APARTMENTS

SUBURBAN DELAWARE

SHUBHAM K KAPADIYA

STRUCTURAL OPTION
ADVISOR: DR. LINDA HANAGAN

EXECUTIVE SUMMARY

The Revive Apartments is a fictitious name used to maintain confidentiality of this project. It was designed to act as a catalyst to redevelop the neighborhood. This project was issued for construction in June of 2017 and is currently under construction. This six-story high/68-foot-tall mixed use/residential Revive Apartments is located in suburban Delaware. The Revive Apartments is mainly divided into 5 stories of multi-family residential space, 1 story of retail space, and 2 stories of underground parking. Approximately 330 vehicle parking, 10 retail spaces, 165 residential units, and amenities are housed in this 376,000 square feet of low-rise building. The building takes its shape from the property line and streets surrounding it. This shape gives raise to three wings connecting at acute angles. This three-wing design gives birth to a courtyard area at the center.

The structure of this building is complex due to the shape and size of this building. The foundation of this building consists of continuous strip footing for the perimeter walls, slab-on-grade for the lower level parking garage, and concrete spread footings for all the W-shape columns. The parking garages below grade and first two levels above grade are framed using steel beams and columns with composite decking. The top four levels consist of wood floor trusses and shear walls.

In this report, an alternate concrete structural system has been proposed aimed at creating a possibility for multiple apartment purchases and potentially increasing floor-to-ceiling height for apartments. After an initial study of the existing floor plan, a column grid was created with minimal architectural change in mind. Two-way slab with drop panel was selected for the redesigned floor system and concrete shear walls for lateral system. The alternate system was designed and analyzes using code specifications for concrete, design aids, and computer modeling software. Eliminating a podium level affected the proposed architectural plans for garage and retail levels.

The primary codes adopted by the local council with amendments at the time of design are the International Building Code (IBC) 2012, the International Mechanical Code (IMC) 2012, the International Plumbing Code (IPC) 2012, and Ordinance No. 13-034. The ICC A117.1-2009 (Accessible and Useable Building and Facilities), the Delaware State Fire Prevention Regulations 2015, the 2015 Edition of Life Safety (NFPA 101), and Standards for Accessible Design (ADA) 2010 are also used in the design of this building.

ACKNOWLEDGEMENTS

I would like to thank the following friends and family for their support:

- **Mr. Bradley Kirkham**, P.E., Green LEED Assoc., Senior Structural Engineer at Baker Ingram & Associates and **Mr. Lawrence Baker, Jr.**, P.E., President of Bake Ingram & Associates for providing me with all the help necessary for this project.
- My Advisor, **Dr. Linda Hanagan** for helping me throughout the structural analysis and design.
- The AE faculty for their advice and knowledge.
- My parents for supporting me from the beginning and without whom attending Penn State would not have been possible.
- My friends and classmates for their willingness to help.

THE APARTMENT BUILDING

PROJECT TEAM

Architect: Bernardon
Structural Engr: Baker Ingram & Associates
MEP Engineer: Advanced Engineering Inc.
Civil Engineer: CDA Engineering Inc.

ARCHITECTURE

The Apartment Building is a 6-story mixed-use building housing approximately 165 residential units, amenities and 10 retail spaces. The building is designed to revitalize the site from a strip mall and act as a catalyst for redevelopment in the neighborhood. Two below grade levels allow space for a 330 vehicle parking. First floor consists of retail spaces and five stories above it consist apartments. From second floor above, the building splits into three apartment wings formation, which creates a courtyard area with swimming pool in the middle. The shape and façade of the building creates space for private balconies. The six-story tall acts as a visual anchor of the building.

Structural

- Concrete spread footing and wall footing used for foundation system
- 4-stories of Wood structure on 2 composite steel podium levels
- Moment frames and wood shear walls used for lateral support

MEP

- Mechanical equipment located on the roof
- 120/208 volt system
- 125KW/156KVA Diesel Generator

Construction:

- Design-Bid-Build
- New Construction with multi-phase
- Construction started in June of 2017

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* Stiffness of Shear Wall:

$$K = \frac{t}{\left[\frac{H^3}{EL^3} + \frac{1.2H}{GL}\right]}$$

H = Height of shear wall (in)

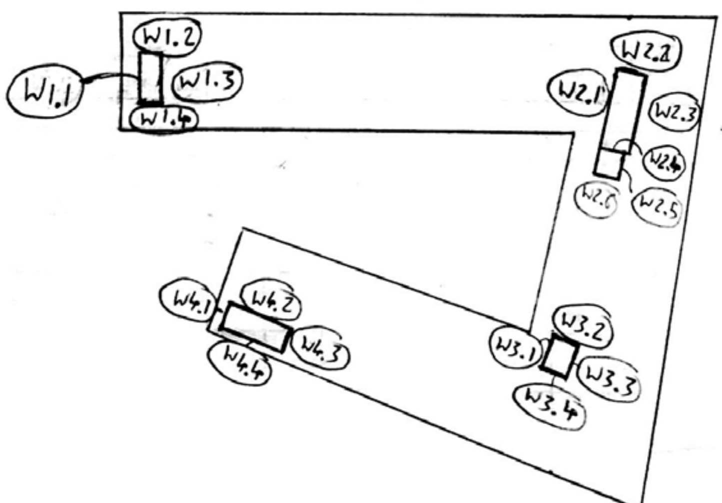
L = Length of shear wall (in)

$E = 57{,}000\sqrt{f'_c}$

$= 57{,}000\sqrt{4000} = 3{,}604{,}996.5 \ lb/in^2$

$G = \frac{E}{2(1+v)} = \frac{3{,}604{,}996.5}{2(1+0.2)} = 1{,}502{,}081.9 \ lbs/in^2$

t = Thickness of wall (in) = 8 in



→ Wall 1.1 : $K = \frac{8}{\left[\frac{(132)^3}{3605 \cdot (235)^3} + \frac{1.2(132)}{1502 \cdot (235)}\right]} = 16{,}067.5 \ kip/in$

1 INTRODUCTION

1.1 PURPOSE & SCOPE

The purpose of this report is to provide a detailed description of the existing structural system and the process of its redesigned. This report contains detailed calculations and process which were used to make engineering decisions to propose an alternate structural system. The first half of the report includes a description of the building floor layout and existing structural systems. This half also includes components from the codes it used for its design to better understand the structure. The second half of the report describes the process through which an alternate structural system was designed also with effects on construction and acoustics of the Revive Apartments.

1.2 GENERAL BUILDING OVERVIEW

The Revive Apartments is a six-story and 68 feet tall building with an accessible green roof and total square feet of approximately 376,000. This project is aimed at re-developing the site from a strip mall. The building footprint occupies the entire site area for the first two levels and then splits up into three wings creating a courtyard on the second level. Approximately 330 vehicles will be able to park in the lower, mid, and partial first flood levels. The first level consists of about 10 retail spaces and levels two and above houses about 165 residential units. This building creates a lot of community spaces including, but not limited to a courtyard on the second level and a roof top garden. These spaces compensate the green area lost due to the footprint of this building. The courtyard in the middle of three wings provides more balcony space for apartments facing it. Only one wing extends to the sixth-floor level, which gives a perception of a tower anchoring the building.

Figure 1: Rendered view of building's North-East Corner (Bernardon)

The Revive Apartments is located on a heavy traffic street. An avenue runs parallel to the building's North-West side and a railway track on its South-East side, as shown in figure 2. The site grades down from its South-East corner to South-West corner.

Figure 2: Site Plan

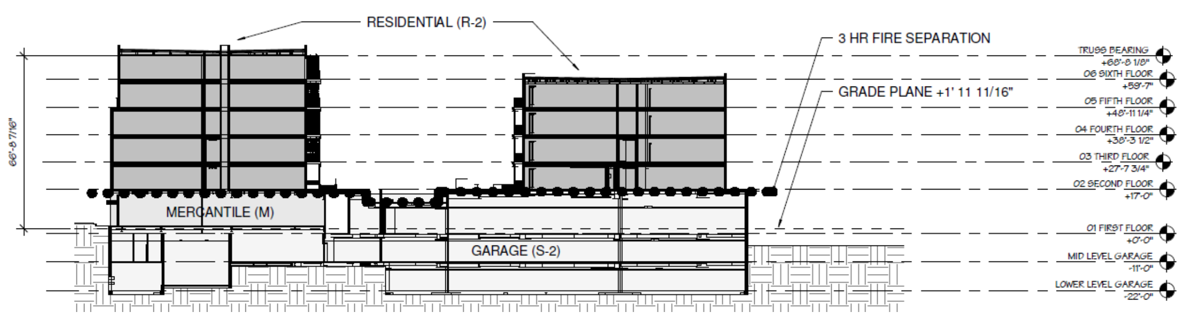


Figure 3: East-West Building Section

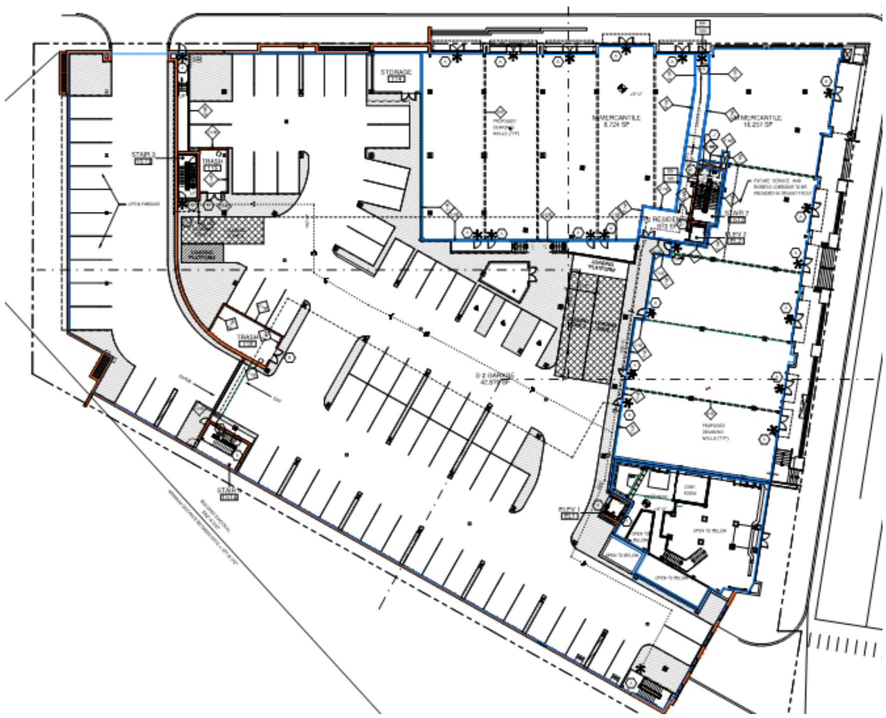


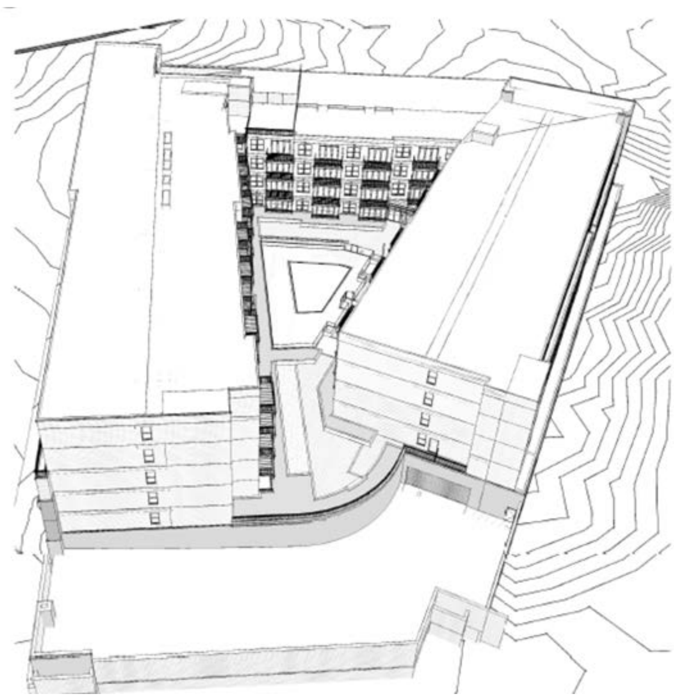
Figure 4: First floor plan

Figure 5: Typical Residential Floor Plan

2 EXISTING STRUCTURAL SYSTEM

2.1 STRUCTURAL SYSTEM OVERVIEW

Starting from ground up, the lower-level parking sits on slab on grade, with concrete strip footing supporting the surrounding retaining walls. Floors above are supported by a steel framed structure with composite steel beams and steel columns come down to square concrete spread footings. These floors are supported laterally by moment frames. Floors including and above third floor are supported by a wood framed structure consisting of floor joists and plywood decking tied to wood shear walls.



Due to the shape of the building, shown in figure 6 on the left, the framing system was designed using three different grid system intersecting at two corners. This meant having complex and expensive steel connections at the first and second levels. The stair towers and elevator shafts have a self-supporting CMU structures and are not connected to the main structure.

Figure 6: Perspective view looking East

2.2 FOUNDATION SYSTEM

The Revive Apartments utilizes a combination of concrete strip and spread footings with the lower parking deck sitting on slab-on-grade as shown in the figure below. The foundation system is designed in accordance with an allowable soil bearing capacity of 600 PSF. Geo-technology Associates were responsible for providing the geotechnical report for this site. Concrete strip footings, with compressive strength of 4000 PSI, are supporting the perimeter walls and stairs and elevator shafts. Exterior columns rest on concrete piers with compressive strength of 4000 PSI and spread footings with compressive strength of 3000 PSI. Interior columns are supported directly by spread footings using 3000 PSI concrete. Interior slab-on-grade uses 3500 PSI concrete and exterior used 4500 PSI concrete. All concrete is normal weight and requires at least 28 days to assume its design strength.

A wide range of footing sizes are used for this project as shown in the footing schedule on the right (Figure 7). In the footing plan shown below (Figure 8), the footings highlighted in red, support moment frames.

FOOTING SCHEDULE		
MARK	SIZE	REINFORCING (EACH WAY - BOT)
F4.0	4'-0 x 4'-0 x 1'-0	4#5
F5.0	5'-0 x 5'-0 x 1'-4	6#5
F6.0	6'-0 x 6'-0 x 1'-6	6#6
F7.0	7'-0 x 7'-0 x 1'-10	6#7
F8.0	8'-0 x 8'-0 x 2'-0	8#7
F9.0	9'-0 x 9'-0 x 2'-4	9#7
F10.0	10'-0 x 10'-0 x 2'-6	11#7
F11.0	11'-0 x 11'-0 x 2'-8	11#8
F12.0	12'-0 x 12'-0 x 2'-10	10#9
F13.0	13'-0 x 13'-0 x 3'-2	12#9
F14.0	14'-0 x 14'-0 x 3'-3	11#10
F15.0	15'-0 x 15'-0 x 3'-6	13#10

Figure 7: Footing Schedule

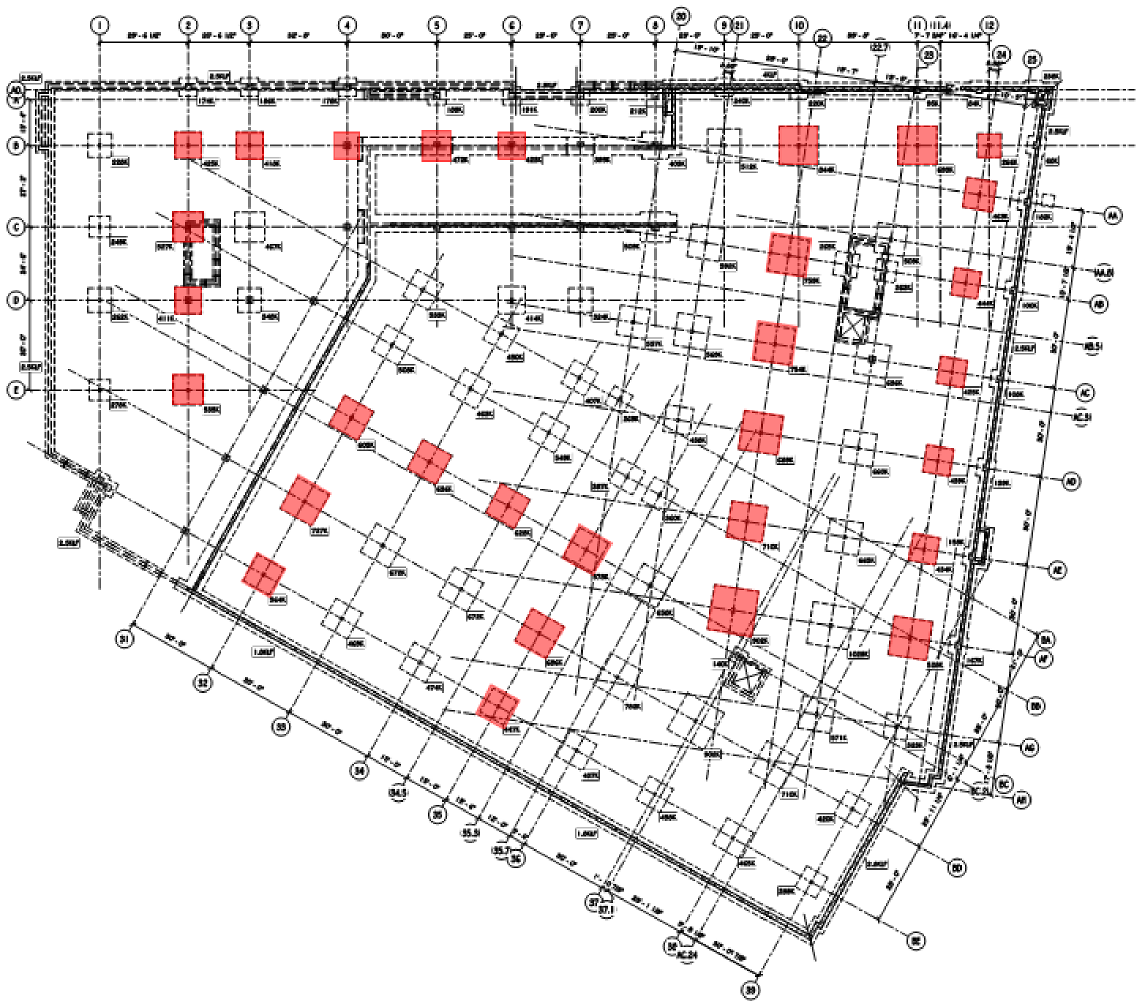


Figure 8: Foundation Loading Plan

2.3 GRAVITY SYSTEM

The Revive Apartments mainly uses two different gravity systems. Wood framing system is used for the apartments on the top four floors and steel framing system for the parking garages and first two floors. Due to the architectural layout of this building, the first two floors including underground parking garages have no similar bays. A relatively typical bay for steel and wood framed floors are shown in figures 9 and 10 below. Beam sizes range from W12 to W40 with studs to form composite steel floor system with a 1.5” deck and 3.5” of lightweight concrete topping. Wood framed floors layout changes with apartment layouts as their separating walls are used as load bear walls. 18” deep 4x2 Floor Trusses @ 19.2” o.c. with variable lengths and 5/4” plywood decking with poured gypsum are used thoughtout the wood frames floors, as shown in figures 11 and 12 below.

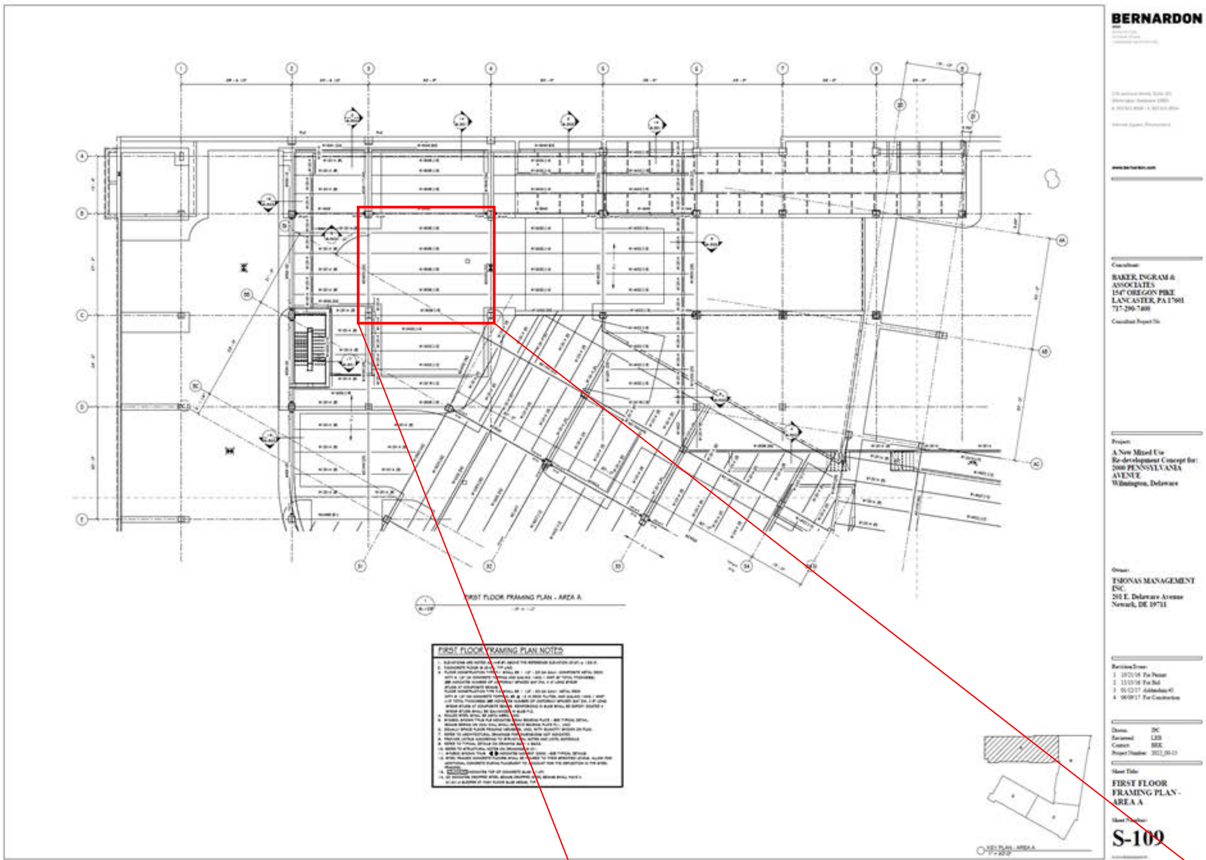


Figure 9: Second Floor Structural Drawing

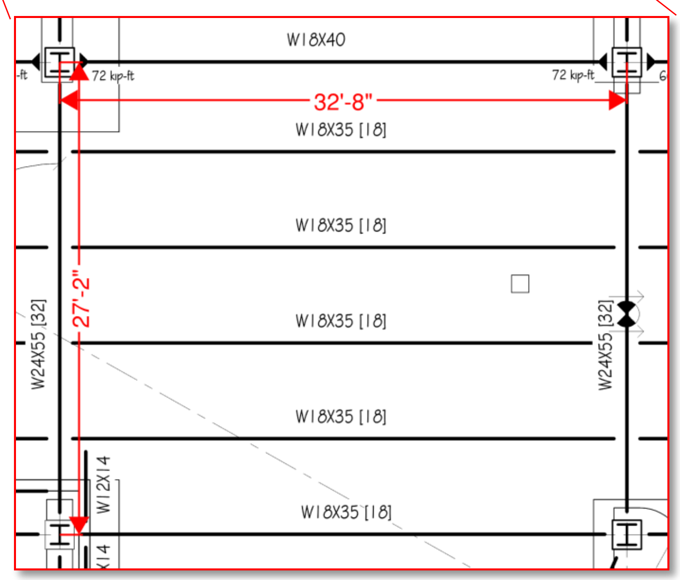


Figure 10: Steel Framed Bay

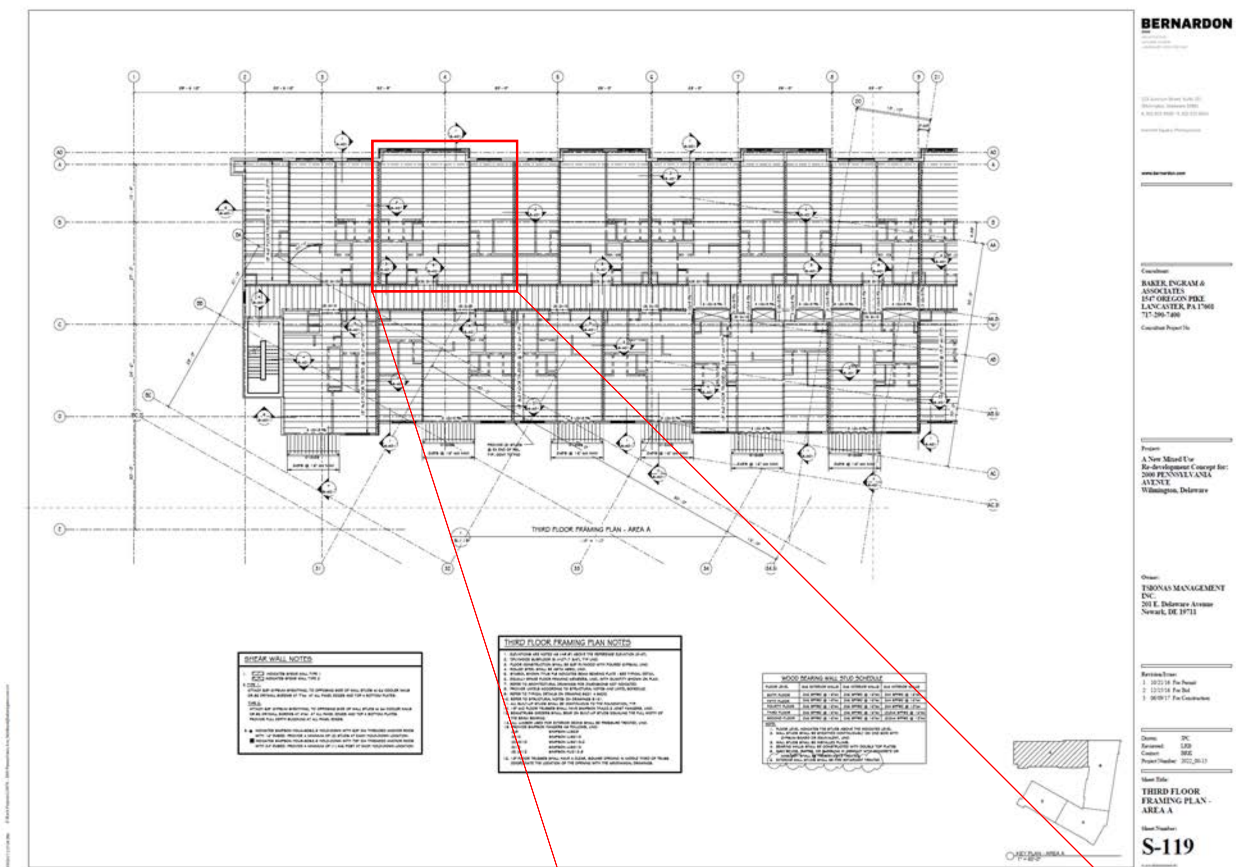


Figure 11: Third Floor Structural Drawing

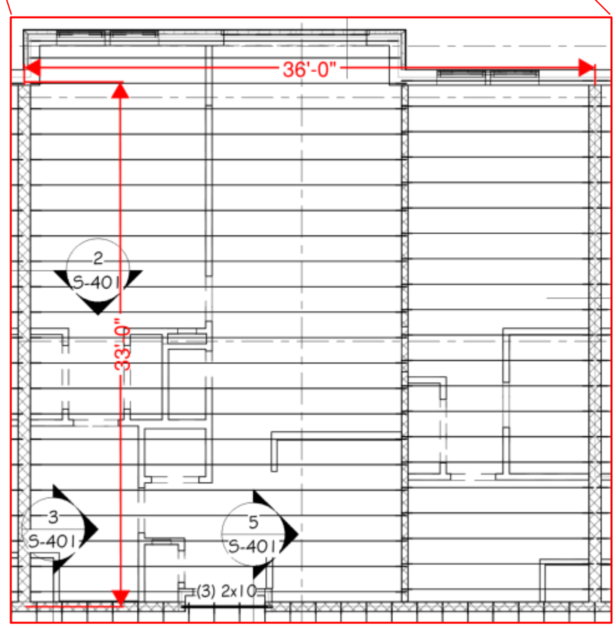


Figure 12: Wood Framed Typical Bay

2.4 LATERAL SYSTEM

The lateral system of this building is divided into two systems. Steel moment frames resist lateral loads on the first and second floors in both the East-West and North-South directions. Wood Shear walls resist lateral loads on third to sixth floors. The location of moment frames are highlighted in figure 13 and location of wood shear walls are highlighted in figure 14. Beams and columns in frames highlighted, in figure 15 below, are detailed to resist gravity and lateral loads. In these moment frames, beams on the first floor range from 18" to 24" deep and on the second floor from 21" to 44" deep. Due to the nature of these lateral systems, wood shear walls sit on the second floor, podium level, and transfer its lateral loads from third to sixth floors to the moment frames on that level. Wood shear walls are designed with 5/8" gypsum sheathing on both sides of wall studs with 6d or #6 drywall screws at 7" o.c. for levels four to six, and 4" o.c. screws for level three. As seen in figure 16, these shear walls are located between apartments and hallway walls.

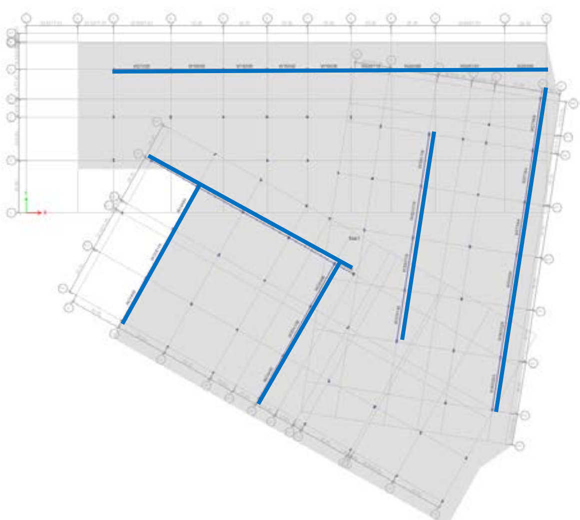


Figure 13: Steel Moment Frame location

Figure 14: Wood Shear Wall location

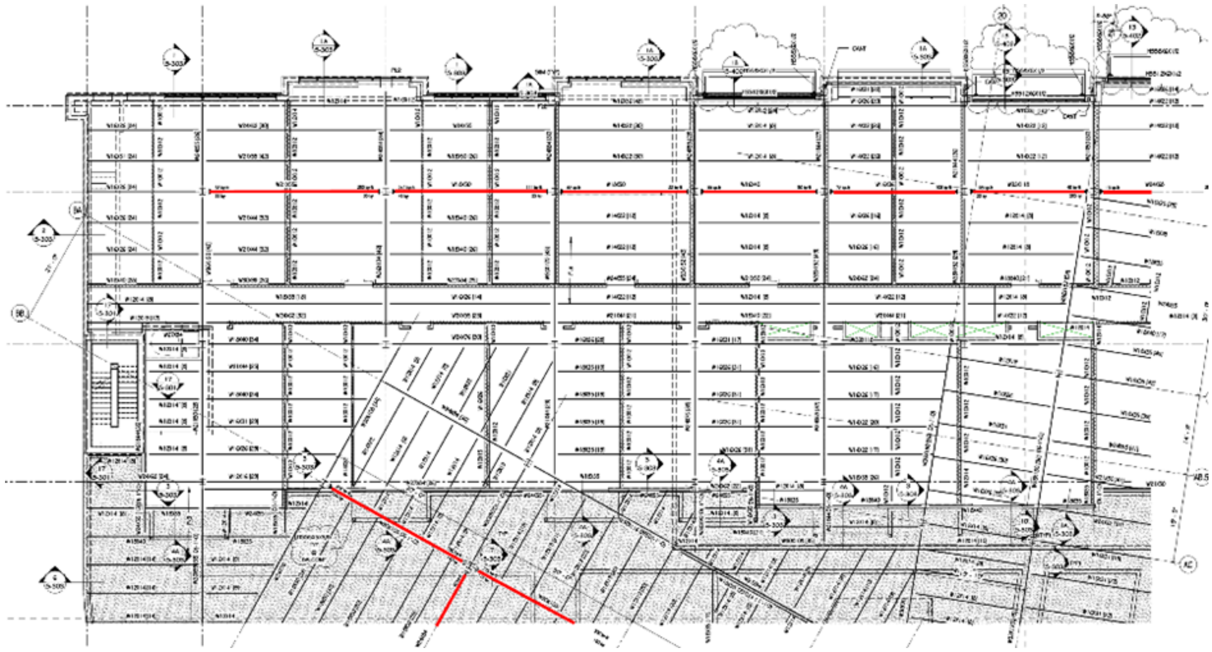


Figure 15: Second Floor Framing plan (Moment Frames highlighted)

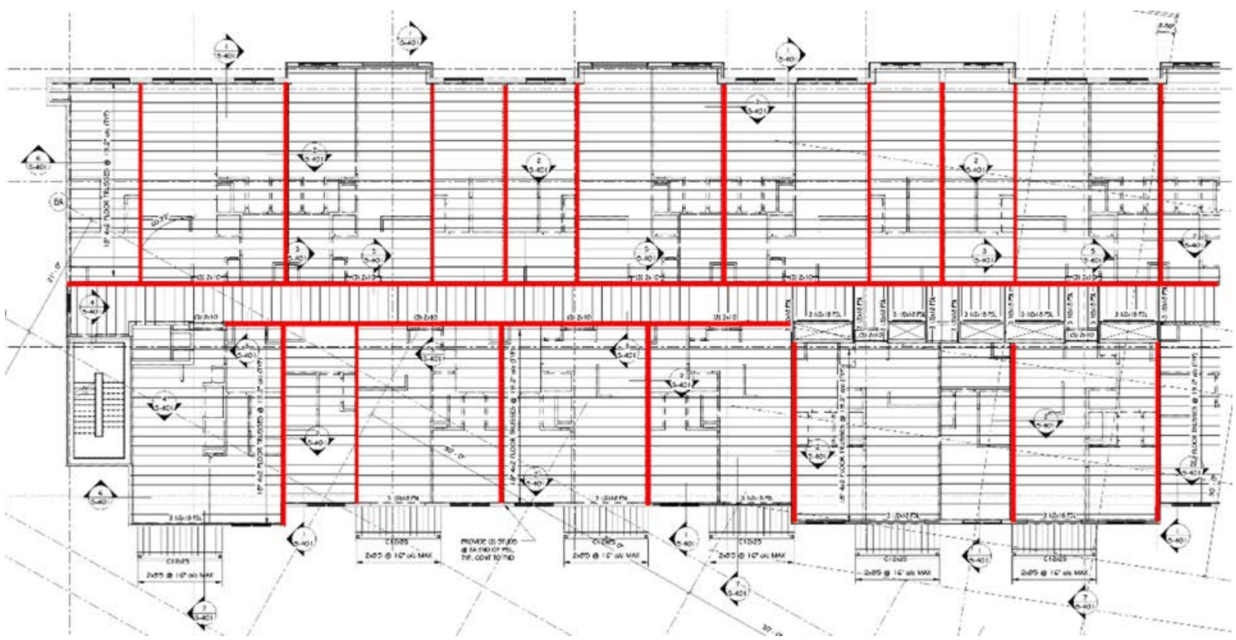


Figure 16: Third Floor Framing Plan (Wood Shear Walls highlighted)

3 LOADS

3.1 APPLICABLE CODES

The Revive Apartments are located in Suburban Delaware. The 2012 Edition of the International Building Code (IBC) with local ordinances is the code adopted by the local council were used along with this project's CDs and notes from previous structural courses during the design and analysis process described in this report. The parent code refers to the standard references listed in the Table 1 to determine the structural design criteria.

ACI 318 – 11	Building Code Requirements for Structural Concrete
ACI 530 – 11	Building Code Requirements for Masonry Structures
ACI 530.1 – 11	Specifications for Masonry Structures
AF&PA NDS – 12	National Design Specifications for Wood Construction
AISC 360 – 10	Specification for Structural Steel Buildings
ASCE 7 – 10	Minimum Design Loads for Buildings and Other Structures

Table 1: IBC 2012 Standard Refrences

3.2 DEAD LOADS

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* Typical Roof Dead Loads:

Categories	Assumed Load
Structure	5 psf
Insulation	4 psf
Plywood Deck	2 psf
Ceiling	3 psf
HVAC	2 psf
Misc.	4 psf
Total	20 psf

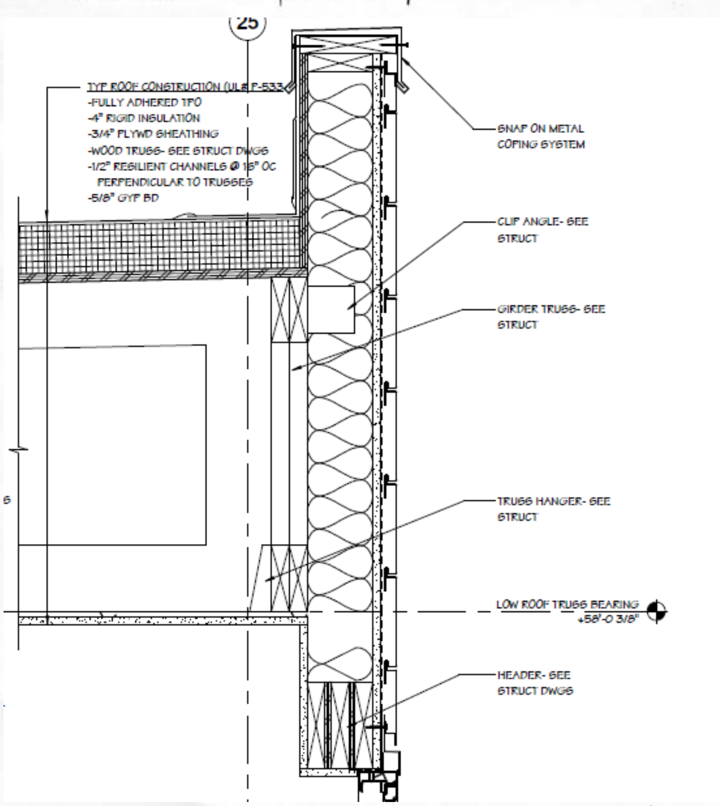

Figure 17: Wood Framed Roof Detailed Section

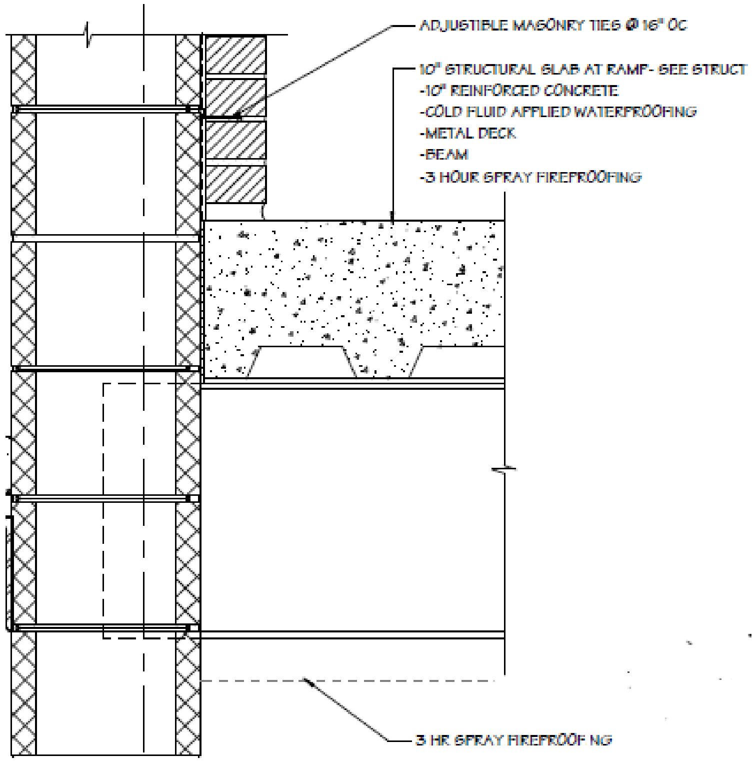

Figure 18: Steel Framed Floor Detailed Section

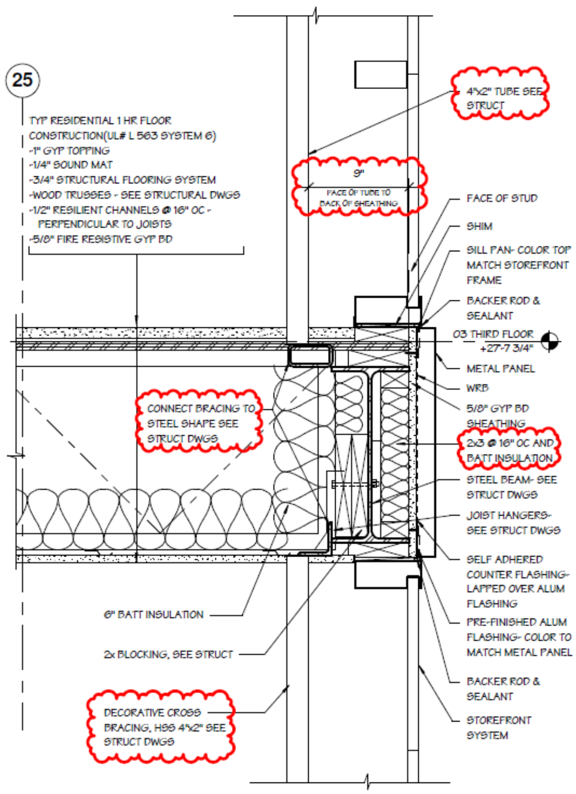

Figure 19: Wood Framed Floor Detailed Section

Shubham Kapadiya | Gravity Loads | Notebook Submission A

* Typical Floor Dead Loads:

→ Steel Framed Structure (Dead Loads):

Categories	Assumed Loads
Structure	5 psf
HVAC	4 psf
5" LW Conc Slab w/ Deck	39 psf
Misc.	11 psf
Total	65 psf

→ Wood Framed Structure (Dead Loads):

Categories	Assumed Loads
3/4" gyp topping	5 psf
3/4" plywood	3 psf
4X2 Floor trusses	5 psf
5/8" drywall	3 psf
Misc.	4 psf
Total	20 psf

→ Other Structure Dead Loads:

Pools = 250 psf

24" planters = 200 psf

3.3 LIVE LOADS

Shubham Kapadiya Gravity Loads Notebook Submission A

* Typical Floor Live Loads:

→ Following table lists live loads according to occupancy of use from Table 4-1 in ASCE 7

Occupancy of Use	Uniform Load	Code Minimum
Assembly Areas	100 psf	100 psf
First Floor Corridors	100 psf	100 psf
Mechanical Rooms	150 psf	N/A
Lobbies	100 psf	100 psf
Stairs	100 psf	100 psf
Parking	40 psf	40 psf
Residential Rooms	40 psf	40 psf
Residential Balconies	40 psf	1.5(40) = 60 psf
Residential Corridors	40 psf	40 psf
Roof Gardens	100 psf	100 psf
Residential Public Rooms	100 psf	100 psf
Roof Live	30 psf	20 psf

3.4 EXTERIOR WALL LOADS

Façade of the Revive Apartments is different for each face. Metal siding and brick are two typical façade types used in the design of this building. Shown in figure 20 are typical exterior wall sections. The exterior wall loads are carried by slabs which transfer them to steel or wood structural members.

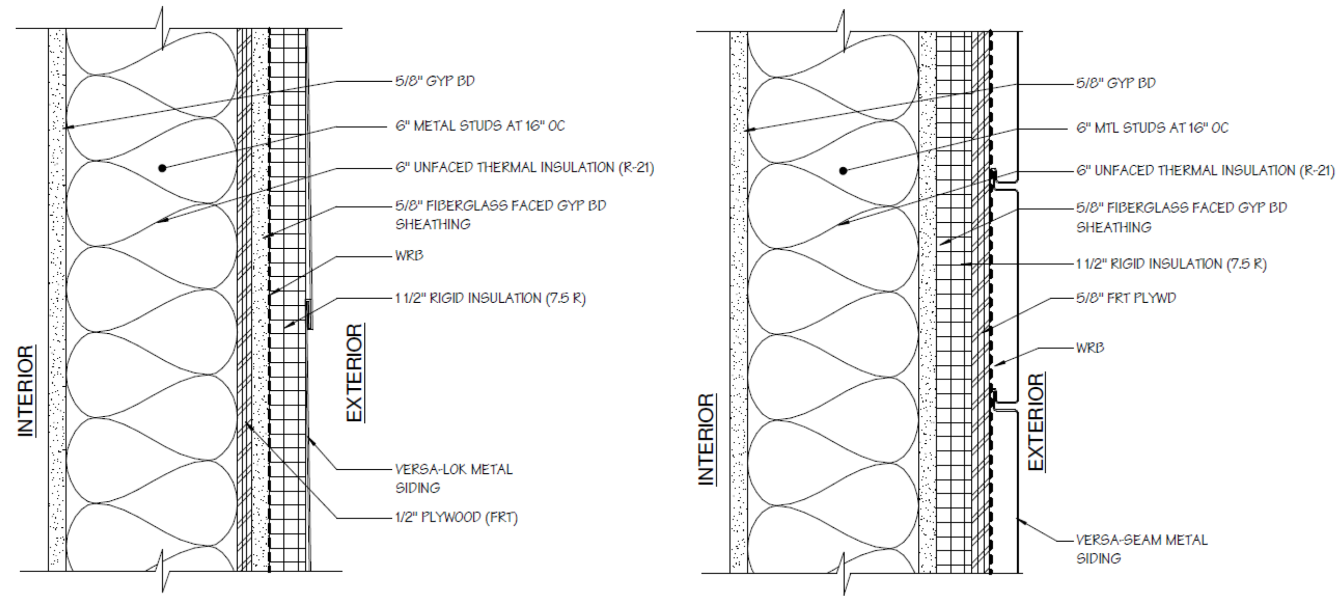

Figure 20: Typical Exterior Wall Sections

Exterior Wall Weight (Table C3-1)	
5/8" Gypsum Board	2 psf
6" Metal Studs at 16" oc	8 psf
6" Unfaced Thermal Insulation	6 psf
5/8" Fiberglass	1 psf
1 ½" Rigid Insulation	2 psf
½" Plywood	2 psf
Brick (Parts of Exterior Wall)	40 psf
Misc.	4 psf
Total	**65 psf**

Table 2: Exterior Wall Weight Summary

3.5 SNOW LOAD

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* Typical Snow Loads:

→ Roof Live Load (Table 4.1):

$$L_r = 30 \text{ psf}$$

→ Snow Loads (Chapter 7):

Ground Snow Load, $P_g = 30$ psf (Figure 7-1)
Snow Exposure Factor, $C_e = 1.0$ (Table 7-2)
Snow Thermal Factor, $C_t = 1.0$ (Table 7-3)
Importance Factor, $I = 1.0$ (Table 1.5-2)

Flat Roof Snow Loads;

$$P_f = 0.7\, C_e C_t I_s P_g \quad \text{(Equation 7.3-1)}$$
$$P_f = 0.7(1)(1)(1)(30)$$
$$P_f = 21 \text{ psf} + \text{Drift}$$

⇒ Snow Drift or sliding snow loads have been considered where appropriate.

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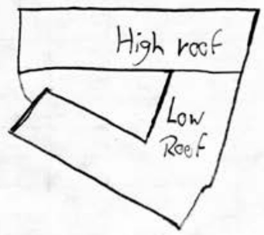
→ Canopy Snow drift:



⇒ Width of high roof ; $l_{u_H} = 20$ ft

⇒ Width of low roof ; $l_{u_L} = 295$ ft

⇒ Drift Density ; $D = 0.13\,p_g + 14$

$= 0.13(30) + 14 = 17.9$ pcf

⇒ Elevation change ; $h_r = 15$ ft

⇒ Height of base snow ; $h_b = p_f / D = 1.2$ ft

$h_c = 15 - 1.2 = 13.8$ ft

⇒ Height of snow drift ; $h_d = 0.75\left[0.43\sqrt[3]{295}\sqrt[4]{30+10} - 1.5\right]$

$h_d = 4.3$ ft

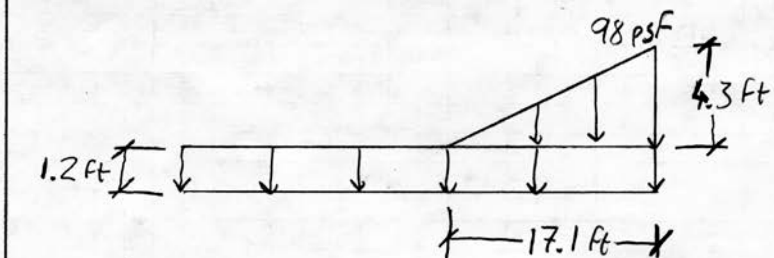
$h_c / h_d = 13.8 / 4.3 = 3.2 > 0.2$ ⇒ Design for Drift

⇒ Width of snow drift, $W_d = 4(4.3) = 17.2$ ft

⇒ Max weight of snow drift ; $P_m = D h_d + P_f$

$= 17.9(4.3) + 21$

$P_m = 98$ psf



3.6 WIND LOADS

Only one scenario of wind loading, ignoring the hole in the middle, calculated in this section. More scenarios of wind loads are calculated in Revised Wind Loads section.

* Wind Loads Summary:

→ Basic Wind Speed ; $V = 115$ mph (Figure 26.5-1A)
→ Exposure Category = B
→ Importance Factor ; $I_w = 1.0$ (Table 1.5-2)
→ Risk Category = II

→ Wind Load Parameters:
(Table 26.6-1) $K_d = 0.85$ $\quad \alpha = 7.0$ Table 26.9-1
(Figure 26.8-1) $K_{zt} = 1.00$ $\quad z_g = 1200$ ft Table 26.9-1

Rigid Building ⇒ $G = 0.85$ (Section 26.9.1)
Enclosed Building ⇒ $GC_{pi} = \pm 0.18$ (Table 26.11-1)
K_z = varies according to height

→ Velocity Pressure, q_z (Section 27.3.2)

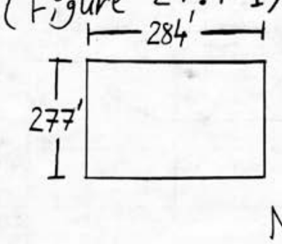
$$q_z = 0.00256 K_z K_{zt} K_d V^2$$
$$= 0.00256 (K_z)(1)(0.85)(115)^2$$
$$= 28.78 K_z \text{ lb/ft}^2$$

$$K_z = 2.01 (z/z_g)^{2/\alpha} \quad \text{For } 15 \text{ ft} \le z \le z_g$$
$$K_z = 2.01 (15/z_g)^{2/\alpha} \quad \text{For } z < 15 \text{ ft}$$
where z = height of each floor with respect to ground

* q_z values shown in tables 3 & 5 *

Shubham Kapadiya | Wind Loads | Notebook Submission A

→ External pressure coefficient, C_p (Figure 27.4-1)



North-South : $L/B = \frac{277}{284} = 0.98$

East-West : $L/B = \frac{284}{277} = 1.03$

=> Walls : Windward $C_{P_w} = 0.8$

Leaward $C_{P_L} = -0.5$ (N-S) & -0.3(E-W)

Side Wall $C_{P_s} = -0.7$

→ Wind Pressure:

$P_{Windward} = q_z G C_{P_w}$

$P_{Leeward} = q_h G C_{P_L}$

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

$= 0.00256(0.9)(1)(0.85)(115)^2(1)$

$q_h = 25.9$ psf

$P_{Total} = P_{windward} - P_{Leeward}$

* P_{Total} values shown in tables 2 & 4 *

Wind Pressure Determination (North-South)						
Level	Height "z" (ft)	K_z	q_z (psf)	P_W (psf)	P_L (psf)	P_{total} (psf)
Level 2	17	0.60	17.14	11.66	-11.01	22.66
Level 3	27.65	0.68	19.70	13.39	-11.01	24.40
Level 4	38.29	0.75	21.62	14.70	-11.01	25.71
Level 5	48.94	0.81	23.19	15.77	-11.01	26.77
Level 6	59.58	0.85	24.53	16.68	-11.01	27.69
Roof	68.71	0.89	25.55	17.37	-11.01	28.38

Table 2: N-S Wind Pressure Calculations Summary

Base Shear Determination (North-South)					
Level	Height "z" (ft)	Tributary Height (ft)	Tributary Width (ft)	Total Pressure (psf)	Total Story Force (kip)
Level 2	17	13.83	284	22.66	88.98
Level 3	27.65	10.65	284	24.40	73.77
Level 4	38.29	10.65	284	25.71	77.72
Level 5	48.94	10.65	284	26.77	80.94
Level 6	59.58	9.89	284	27.69	77.72
Roof	68.71	4.57	284	28.38	36.79
				Base Shear, V =	435.92

Table 3: N-S Base Shear Calculations Summary

Wind Pressure Determination (East-West)						
Levels	Height "z" (ft)	K_z	q_z (psf)	P_w (psf)	P_L (psf)	P_{total} (psf)
Level 2	17	0.60	17.14	11.66	-6.60	18.26
Level 3	27.65	0.68	19.70	13.39	-6.60	20.00
Level 4	38.29	0.75	21.62	14.70	-6.60	21.30
Level 5	48.94	0.81	23.19	15.77	-6.60	22.37
Level 6	59.58	0.85	24.53	16.68	-6.60	23.28
Roof	68.71	0.89	25.55	17.37	-6.60	23.98

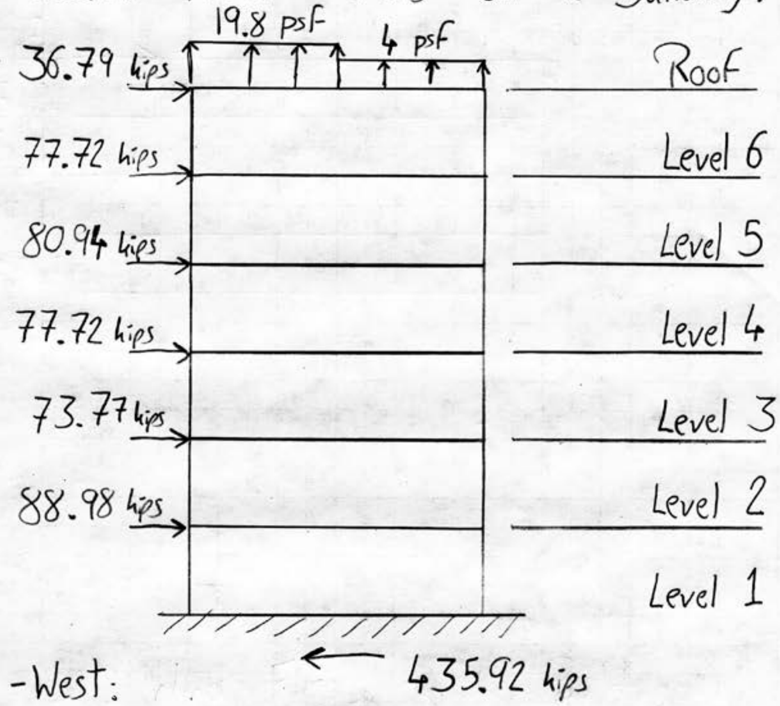
Table 4: E-W Wind Pressure Calculations Summary

Base Shear Determination (East-West)					
Levels	Height "z" (ft)	Tributary Height (ft)	Tributary Width (ft)	Total Pressure (psf)	Total Story Force (kip)
Level 2	17	13.83	277	18.26	69.93
Level 3	27.65	10.65	277	20.00	58.97
Level 4	38.29	10.65	277	21.30	62.82
Level 5	48.94	10.65	277	22.37	65.97
Level 6	59.58	9.89	277	23.28	63.75
Roof	68.71	4.57	277	23.98	30.32

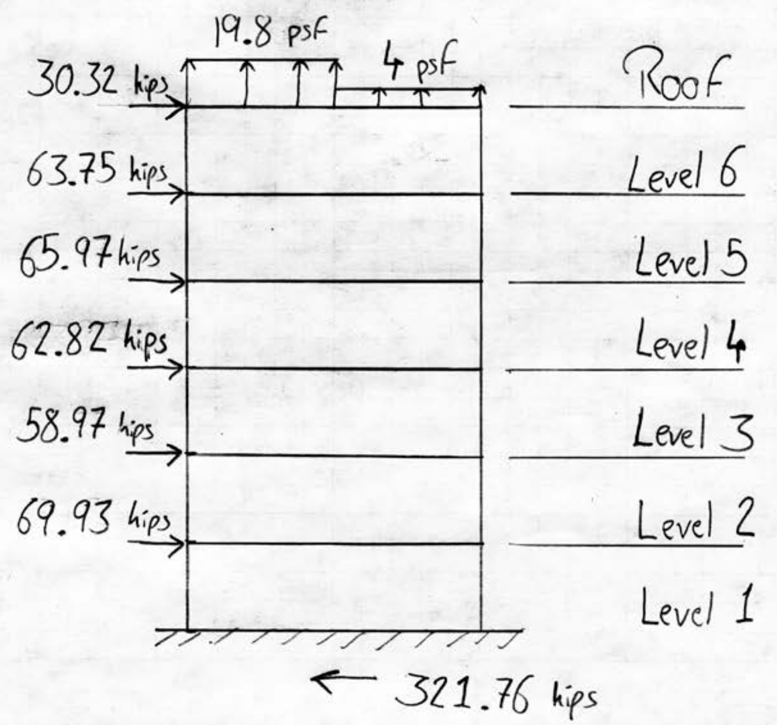
Table 5: E-W Base Shear Calculations Summary

Shubham Kapadiya | Wind Loads | Notebook Submission A

→ North - South Wind Forces on the Building:



→ East - West:



3.7 SEISMIC LOADS

Shubham Kapadiya | Seismic Loads | Notebook Submission A

* Seismic Loads Summary:

→ Seismic Importance Factor ; $I_s = 1.0$
→ Seismic Site Class = C
→ Seismic Design Category = B
→ Analysis Procedure = Equivalent Lateral Force
→ Basic Structural System = Bearing Wall & Building Frame

→ Site Data:

$S_s = 0.196\ g$ $S_{M1} = 0.141\ g$
$S_1 = 0.059\ g$ $S_{DS} = 0.209\ g$
$S_{MS} = 0.313\ g$ $S_{D1} = 0.094\ g$

→ Response Modification Coefficient, $R = 3$ (Table 12.2-1)
→ Overstrength Factor, $\Omega_o = 3$ (Table 12.2-1)
→ Deflection Amplification Factor, $C_d = 2\frac{1}{2}$ (Table 12.2-1)
→ Long Period Transition, $T_L = 6$ sec (USGS website)

→ Seismic Base Shear (Section 12.8.1):

$$V = C_s W$$

where ; Seismic response coefficient, $C_s = \dfrac{S_{DS}}{\left(\frac{R}{I_e}\right)}$ (12.8-2)

W = Effective seismic weight

Excel Spreadsheet was used to calculate W

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→ Seismic Base Shear (Cont.):

⇒ Calculating T:

For steel moment-resisting frames, (Table 12.8-2)

$C_t = 0.028$; $x = 0.8$

$T_a = C_t h_n^x$

$= 0.028(68.71)^{0.8}$

$T_a = 0.826$ s

$T_{max} = C_u T_a$ where $C_u = 1.7$ as $S_{D1} \leq 0.1$ (Table 12.8-1)

$= 1.7(0.826)$

$T_{max} = 1.4$ s $> T_a$ $\therefore T = 0.826$ s $< T_L$

⇒ Calculating seismic response coefficient

$$C_s = \begin{cases} \dfrac{S_{DS}}{R/I_e} = \dfrac{0.209}{3/1} = 0.0697 \\ \dfrac{S_{D1}}{T(R/I_e)} = \dfrac{0.094}{0.826(3/1)} = 0.0379 \\ \dfrac{S_{D1}T_L}{T^2(R/I_e)} = \dfrac{0.094(6)}{(0.826)^2(3/1)} = 0.276 \end{cases}$$

$C_s = 0.0379 > \underset{= 0.01}{0.044 S_{DS} I_e} \geq 0.01$

$\therefore C_s = 0.0379$

Levels	Height (ft)	Dead Load (psf)	Partition (psf)	Exterior Wall (psf)	Snow Load (psf)	Floor Surface Area	Open Surface Area	Wall Surface Area	Total Weight
Level 2	17	20	15	65	30	45,144	12,700	5,698	2,331
Level 3	27.65	20	15	65	0	44,096	-	5,692	1,913
Level 4	38.29	20	15	65	0	44,096	-	5,698	1,914
Level 5	48.94	20	15	65	0	44,096	-	5,692	1,913
Level 6	59.58	20	15	65	30	17,552	26,300	4,885	1,721
Roof	68.71	20	0	65	30	17,552	20,600	4,885	1,287
								Total Weight =	11,079

Table 6: Story Weight Calculations Summary

Level	C_v	Total Weight (kips)	Story Shear (kips)
Level 2	0.30	2,331	127
Level 3	0.16	1,913	65
Level 4	0.16	1,914	65
Level 5	0.16	1,913	65
Level 6	0.14	1,721	59
Roof	0.09	1,287	38

Table 7: Story Shear Calculations Summary

4 STRUCTURAL DEPTH

Based on the analysis performed last semester, the existing structure of the Revive Apartments meets all necessary strength, code, and serviceability requirements. In addition, the structure is seamlessly integrated with the architecture of the building. Therefore, an alternate structural system design with minimal impact on the architecture was not apparent. The existing structure comes with complicated integration of the steel and wood systems. The variation in architectural plans for the first retail space and apartments above, forces the podium level structure to be extremely heavy and costly. The wood structure also doesn't provide a good acoustical barrier between apartment units. An alternate structural system is proposed while keeping the above discussed topics in mind.

The proposed alternative system for the Apartment Building is reinforced concrete two way slab with reinforced concrete columns and shear walls. A new structural grid layout will be created for the entire building while minimally affecting the architecture of the building. Additionally, the new column grid will create possibility of multiple apartment purchases as owners would be able combine two apartments. The concrete shear walls will be place around stairs towers and elevator shafts. ASCE 7-10 along with local provisions will be utilized to determine structural loads.

ACI 318-14 and ACI 318R-14 (commentary) were used for the redesign of this building. ASCE 07-10 was also used to recalculate gravity and lateral loads. Chapter 8 of ACI 318-14 was used to design two-way flab slab system with drop panels. Equivalent lateral frame method was used to obtain preliminary slab thickness and drop panel thickness. First floor, third floor and six floor typical plans were considered during the design.

4.1 GRAVITY SYSTEM

4.1.1 GRAVITY LOADS

As determined in this report, gravity loads considered in the redesign of this building include dead, live and snow, as shown in table 8 below. Table 4-1 in ASCE 07-10 was used to determine live loads. Snow load on the second floor are taken into account as the building forms a courtyard area by splitting in three wings.

Floors	Superimposed Dead Load	Live Load	Snow Load
Roof	15	30	21
6th Floor	21	40	0
5th Floor	21	40	0
4th Floor	21	40	0
3rd Floor	21	40	0
2nd Floor	21	40	21
1st Floor	21	100	0
Mid-level garage	21	50	0

Table 8: Revised Load summary

4.1.2 COLUMN GRID

Before initiating the design of two-way flat slab floor system, it was important to create an effective column grid layout with minimal change on the architecture. The column grid layout was influenced mostly from the residential floor plans. The floors with residential apartments were investigated first for laying out a column grid. Columns were placed at the existing design's shear wall locations. Afterwards, they were moved within those walls to accommodate for ramps and parking spots on the lower levels. Ultimately, a grid layout with relatively symmetric bays was formed, which worked with all residential, retail and parking plans with some changes.

As shown in figure 21 below, columns were organized around ramps on the parking levels. Columns fall on about 25 parking spots and will need to be rearranged. Most columns can also be seen hidden in partition walls between apartments or interior partition walls in figure 22 of residential floor plan. Typical spans between columns range from 12' to 36'. Larger spans were used in order to not interrupt ramps on parking levels.

Figure 21: Mid-level garage plan with columns and drop panels

Figure 22: Third Floor plan with columns and drop panels

Initial columns were designed using the calculations shown below:

Initial Column Sizes

* Typical C2 column (See Two-way slab calcs):

Roof
$W_u = 1.2(125+15) + 1.6(30) = 216$ psf
$W_{u_{Drop\ panels}} = 1.2(150)(4.25)(\frac{1}{12}) = 63.75$ plf (Same for all floors)
$P_u = 0.216(661) + 0.06375(73.44) = 147.5$ kips

9.13' — 147.5k

6^{th}
$W_u = 1.2(125+21) + 1.6(40) = 239$ psf
$P_u = 0.239(661) + 0.06375(73.44) = 162.7$ kips

10.65' — 310.2k

5^{th}
$P_u = 162.7$ kips

10.64' — 472.9k

4^{th}
$P_u = 162.7$ kips

10.65' — 635.6k

3^{rd}
$P_u = 162.7$ kips

10.64' — 798.3k

2^{nd}
$W_u = 1.2(125+21) + 1.6(100) = 335$ psf
$P_u = 0.335(661) + 0.06375(73.44) = 226.1$ kips

17' — 1024.4k

1^{st}
$P_u = 226.1$ kips

11' — 1187.1k

Mid-level
$W_u = 1.2(125+21) + 1.6(50) = 255$ psf
$P_u = 0.255(661) + 0.06375(73.44) = 173.2$ kips

11' — 1360.3k

Lower-level

→ Lower-level column: Try 24" × 24" with 16 #9 vertical bars

$\phi P_{n,max} = 0.8\phi[0.85 f_c'(A_g - A_{st}) + f_y A_{st}]$

$= 0.8(0.65)[0.85(6000)(24^2 - 16(0.79)) + 60{,}000(16)(0.79)]$

$= 1888.4$ kips > 1360.3 k $\therefore$ OK

→ Mid-Level column: Try 24"x24" with 12 #9 vertical bars

$$\phi P_{n,max} = 0.8(0.65)[0.85(6000)(24^2 - 12(0.79)) + 60,000(12)(0.79)]$$
$$= 1798.2 \text{ kips} > 1187.1^k \quad \therefore OK$$

→ 1^{st} Level: Try 24"x24" with 4 #9 vertical bars

$$\phi P_{n,max} = 0.8(0.65)[0.85(6000)(24^2 - 4(0.79)) + 60,000(4)(0.79)]$$
$$= 1618 \text{ kips} > 1024.4^k \quad \therefore OK$$

→ 2^{nd} Level: Try 20"x20" with 4 #9 vertical bars

$$\phi P_{n,max} = 0.8(0.65)[0.85(6000)(20^2 - 4(0.79)) + 60,000(4)(0.79)]$$
$$= 1173.6 \text{ kips} > 798.3^k \quad \therefore OK$$

→ 3^{rd} & 4^{th} Levels use the same column as 2^{nd} Level

→ 5^{th} & 6^{th} Levels: Try 16"x16" with 4 #9 vertical bars

$$\phi P_{n,max} = 0.8(0.65)[0.85(6000)(16^2 - 4(0.79)) + 60,000(4)(0.79)]$$
$$= 769.1 \text{ kips} > 310.2 \quad \therefore OK$$

A RAM Structural Systems model with the initial column sizes of 24”x24”, 20”x20”, and 16”x16” and reinforcement detailing above was used to design columns.

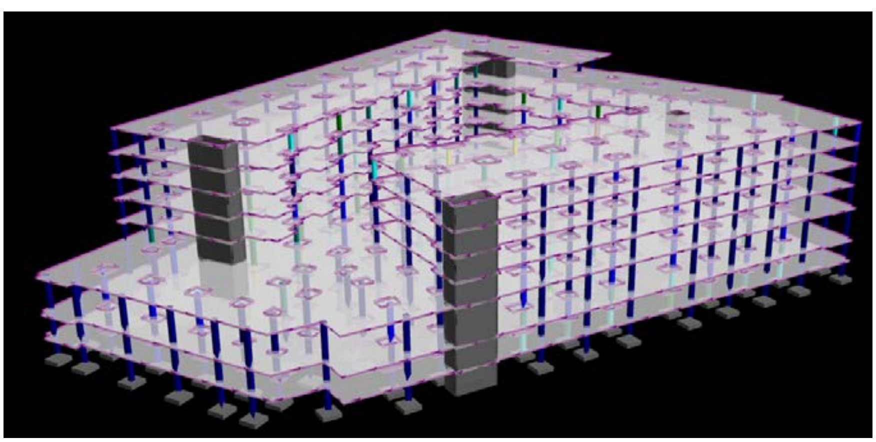


Figure 23: RAM Structural System 3D Model

4.1.3 TWO-WAY FLAT SLAB WITH DROP PANELS

Calculations described in this section can be found in Appendix: Gravity System Hand Calcs. Preliminary sizes for columns and slab thickness for two-way construction without interior beams were calculated using Table 8.3.1.1 in the ACI. After multiple iterations, a 10-inch thick two-way slab with 4.25 inch-thick drop panels was chosen. A 24 feet design strip, shown in Figure 24, was analyzed using equivalent frame method. While dimensioning the thickness of the drop panels, formwork considerations were taken into account. Drop panels were dimensioned to extend at least sixth the span length from the centerline of support in each direction in accordance to ACI section 8.2.4(b). One-way shear calculation for critical section at distance d from the edge of the column was performed for a 12-in wide strip. Afterwards, two-way shear at distance d/2 from the edge of the column was performed to check for adequacy.

In the process of using equivalent frame analysis, negative and positive moments were calculated using the moment distribution method. Appendix 20A of PCA Notes from ACI was used to calculate moment distributing factors and fixed-end moments. Section 8.11.6.6 of the ACI was used to distribute factored moments to Column and Middle Strips. Required reinforcement was calculated using the distributed moments and section 24.4.3.2 was used to calculate minimum area of reinforcement and max spacing requirements.

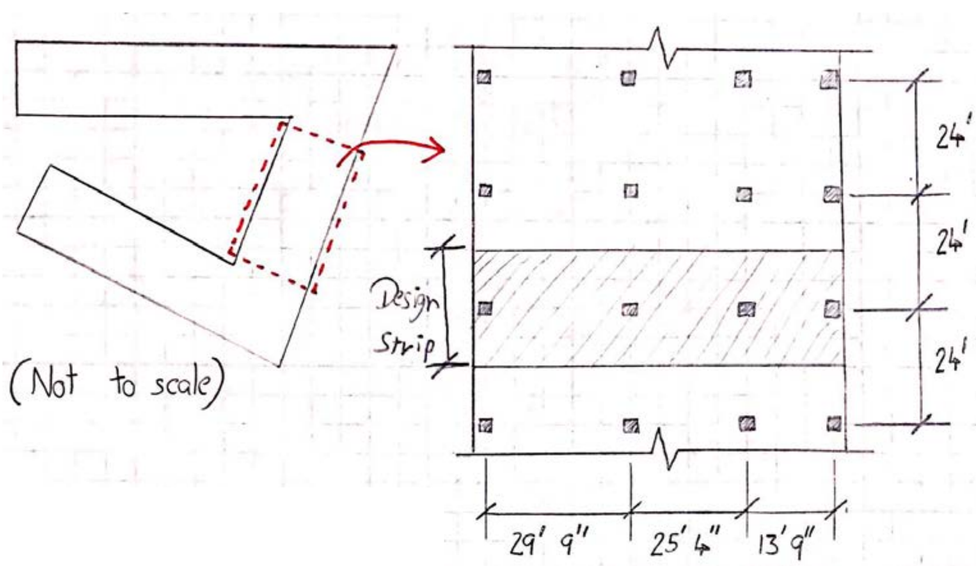
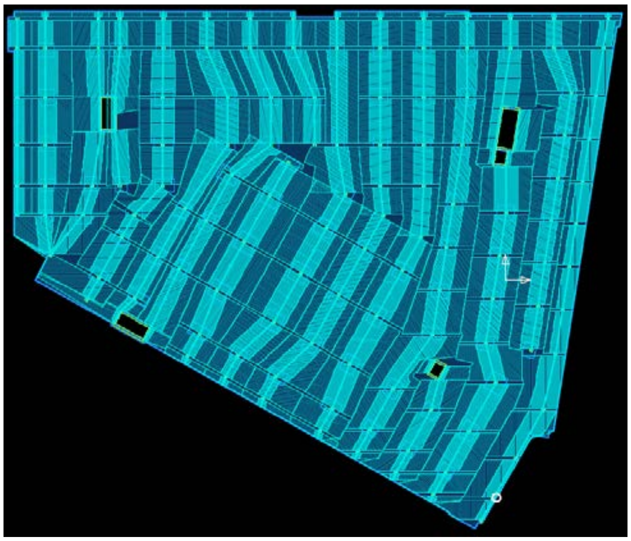


Figure 24: Design Strip using hand calcs

4.1.4 COMPUTER MODELING

Preliminary sizes obtained from Equivalent Frame Method, described in the section above, were used while modeling in RAM Concept. A 10 in slab was used per minimum requirements from ACI. Drop panels were dimensioned to extend at least sixth the span length from the centerline of support in each direction in accordance to ACI section 8.2.4(b) with a 4.25" of additional thickness on top of slab thickness. Three different typical floors were modelled in RAM Concept and design strips were laid out N-S and E-W direction, as shown in figures 25, 26, and 27. Design iterations for design strips considering evenly lay out the reinforcement and seamlessly intersecting three different directionally oriented rebar.



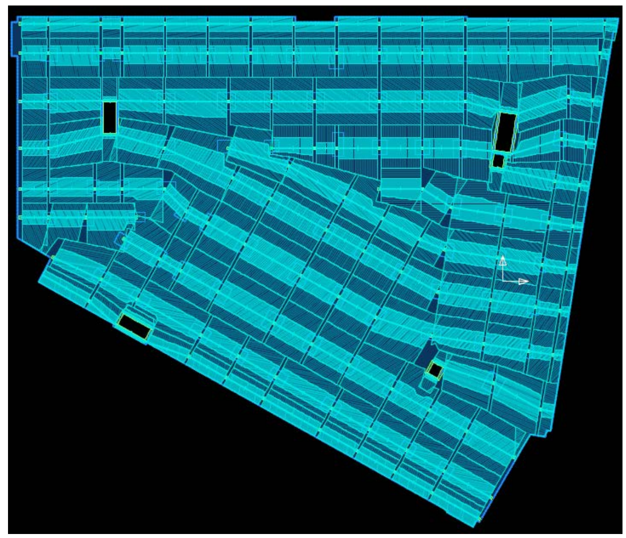
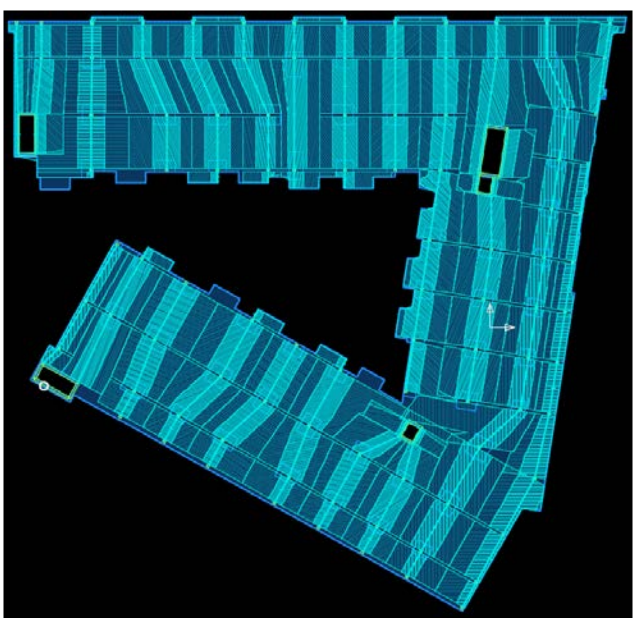


Figure 25: Mid-level to Second floor latitudinal and longitudinal design strips accordingly



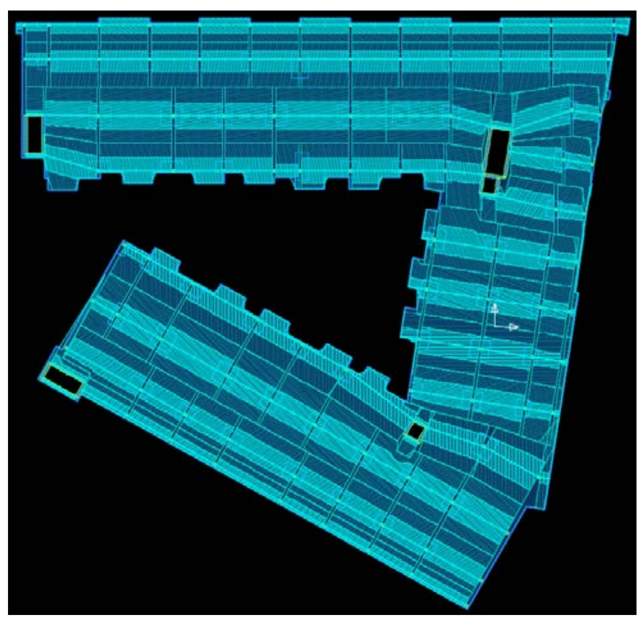


Figure 26: Third to Fifth floor latitudinal and longitudinal design strips accordingly

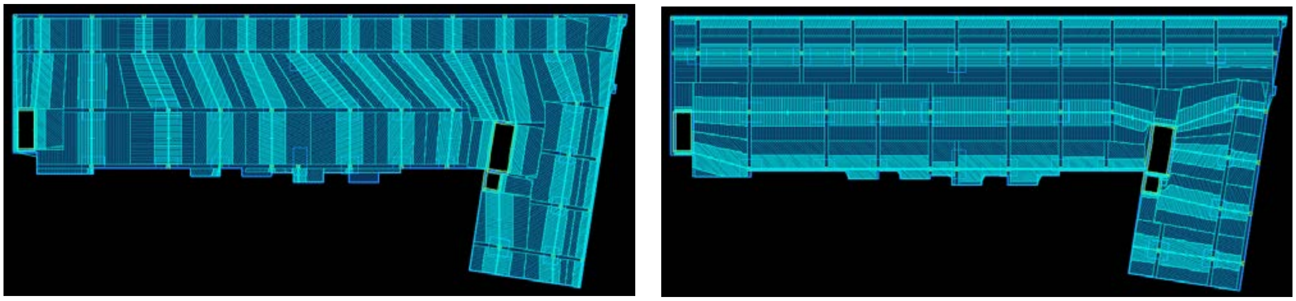


Figure 27: Sixth floor latitudinal and longitudinal design strips accordingly

In the column and middle strips, standard #6 rebar are designed for top and bottom reinforcement throughout the slab. Latitudinal bars are assigned a 0.75 inch minimum cover due to longer spans and longitudinal bars are assigned a 1.5 inch minimum cover. RAM Concept and hand calculations checked for punching shear at all typical column locations located in Appendix: Gravity System.

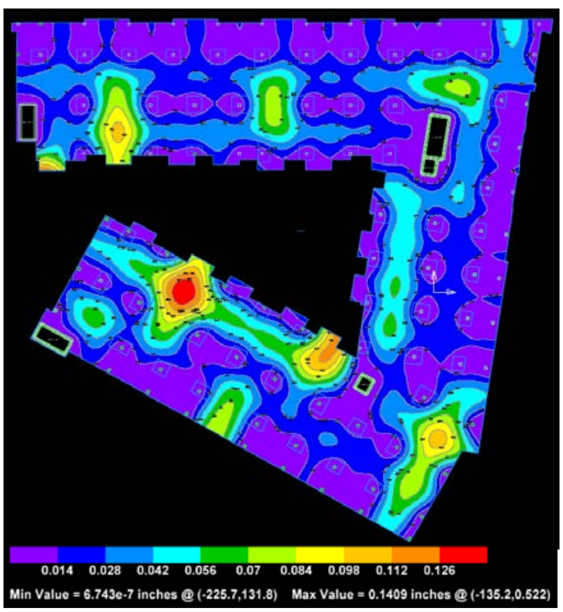


Figure 28: Typical slab deflection

Deflection limit for this building is L/360. This deflection limit was checked in RAM Concept. The maximum deflection, as shown in figure 28, is 0.1409 inches which is smaller than 1.02 inches, 30.5 feet divided by 360.

4.2 LATERAL SYSTEM

4.2.1 REVISED WIND LOADS

Wind loads were recalculated in accordance to ASCE 7-05 provisions. The wind load summary below shows design criteria used to calculate velocity pressures and wind pressures for each level. The assumed building shapes shown on the next page were used to simplify wind load calculations. Later in this section, 4 different wind loading cases are used to calculate a more accurate scenario for a U-shaped building.

* Wind Loads Summary:

→ Basic Wind Speed ; $V = 115$ mph (Figure 26.5-1 A)

→ Exposure Category = B

→ Importance Factor ; $I_w = 1.0$ (Table 1.5-2)

→ Risk Category = II

→ Wind Load Parameters:

(Table 26.6-1) : $K_d = 0.85$

(Figure 26.8-1) : $K_{zt} = 1.00$

Table 26.9-1 $\begin{cases} \alpha = 7.0 \\ z_g = 1200 \text{ ft} \end{cases}$

Rigid Building ⇒ $G = 0.85$ (Section 26.9.1)

Enclosed Building ⇒ $GC_{pi} = \pm 0.18$ (Table 26.11-1)

K_z = varies according to height

→ Velocity Pressure, q_z (Section 27.3.2)

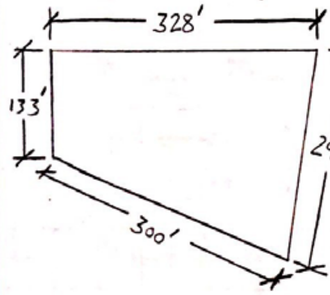
$$q_z = 0.00256 K_z K_{zt} K_d V^2$$
$$= 0.00256 (K_z)(1)(0.85)(115)^2$$
$$= 28.78 K_z \text{ lb/ft}^2$$

$K_z = 2.01 (z/z_g)^{2/\alpha}$ for $15 \text{ ft} \leq z \leq z_g$

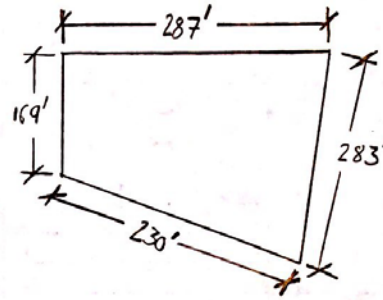
$K_z = 2.01 (15/z_g)^{2/\alpha}$ for $z < 15 \text{ ft}$

where z = height of each floor with respect to the ground

→ Assumed building shape:

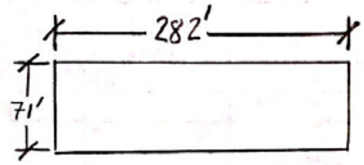


First Floor



$2^{nd} - 5^{th}$ Floor

↑ N



6^{th} Floor

⇒ First Floor:

North - South: $L/B = 292/328 = 0.89$

East-West: $L/B = 328/292 = 1.123$

→ Walls: Leeward $C_{PL} = -0.5$ (N-S) (Figure 27.4-1)

-0.475 (E-W)

⇒ $2^{nd} - 5^{th}$ Floor:

North-South: $L/B = 283/287 = 0.986$

East-West: $L/B = 287/283 = 1.014$

→ Walls: Leeward $C_{PL} = -0.5$ (N-S) (Figure 27.4-1)

-0.497 (E-W)

⇒ 6^{th} Floor:

North - South: $L/B = 71/282 = 0.25$

East - West: $L/B = 282/71 = 3.97$

→ Walls: Leeward $C_{P_L} = -0.5$ (N-S) (Figure 27.4-1)

-0.2 (E-W)

⇒ Windward: $C_{P_W} = 0.8$

⇒ Side Wall: $C_{P_S} = -0.7$

→ Wind Pressures:

$$P = q_z G C_p - q_h (G C_{pi})$$

Sample • Windward @ Level 5 (Height = 59.58) N-S

$$P_W = q_z G C_p - q_h (G C_{pi})$$

$$= 24.53(0.85)(0.8) - 25.55(-0.18)$$

$$= 21.28 \text{ psf}$$

• Leeward @ Level 5

$$P_L = q_h [G C_p - G C_{pi}]$$

$$= 25.55[0.85(-0.5) - (0.18)]$$

$$= -15.46 \text{ psf}$$

$$P_{Total} = 21.28 - (-15.46) = 36.73 \text{ psf}$$

• Total Story shear

$H_5 = 59.58 - 48.94 = 10.64'$; $H_6 = 68.71 - 59.58 = 9.13$

$$\text{Story Shear} = \frac{10.64'(287')(36.73)}{2} + \frac{9.13'(282')(37.43)}{2}$$

$$= 104.3 \text{ kips}$$

	Windward (North-South)							
Level	Height "z" (ft)	K_z	q_z (psf)	C_p	q_zGC_p	q_zGC_p-$q_h(-GC_{pi})$	q_zGC_p-$q_h(+GC_{pi})$	P_w (psf)
Level 6	68.71	0.89	25.55	0.8	17.37	21.97	12.77	21.97
Level 5	59.58	0.85	24.53	0.8	16.68	21.28	12.08	21.28
Level 4	48.94	0.81	23.19	0.8	15.77	20.37	11.17	20.37
Level 3	38.29	0.75	21.62	0.8	14.70	19.30	10.10	19.30
Level 2	27.65	0.68	19.70	0.8	13.39	17.99	8.80	17.99
Level 1	17	0.60	17.14	0.8	11.66	16.25	7.06	16.25

Table 9: Windward Wind Pressure Determination (North-South)

	Leeward (North-South)							
Level	Height "z" (ft)	K_z	q_z (psf)	C_p	q_hGC_p	q_zGC_p-$q_h(-GC_{pi})$	q_zGC_p-$q_h(+GC_{pi})$	P_L (psf)
Level 6	68.71	0.89	25.55	-0.5	-10.86	-4.60	4.60	-15.46
Level 5	59.58	0.85	24.53	-0.5	-10.86	-4.60	4.60	-15.46
Level 4	48.94	0.81	23.19	-0.5	-10.86	-4.60	4.60	-15.46
Level 3	38.29	0.75	21.62	-0.5	-10.86	-4.60	4.60	-15.46
Level 2	27.65	0.68	19.70	-0.5	-10.86	-4.60	4.60	-15.46
Level 1	17	0.60	17.14	-0.5	-10.86	-4.60	4.60	-15.46

Table 10: Leeward Wind Pressure Determination (North-South)

Base Shear Determination (North-South)					
Level	Height "z" (ft)	Tributary Width (ft)	Tributary Area (ft^2)	Total Pressure (psf)	Total Story Force (kip)
Roof	68.71	282.00	2,574.66	37.43	48.18
Level 6	59.58	287.00	3,053.68	36.73	104.27
Level 5	48.94	287.00	3,056.55	35.82	110.83
Level 4	38.29	287.00	3,053.68	34.75	107.81
Level 3	27.65	287.00	3,056.55	33.45	104.18
Level 2	17.00	328.00	5,576.00	31.71	139.53
				Base Shear	**614.80**

Table 11: Base Shear Determination (North-South)

Windward (East-West)								
Level	Height "z" (ft)	K_z	q_z (psf)	C_p	q_zGCp	q_zGC_p - $q_h(-GC_{pi})$	q_zGC_p- $q_h(+GC_{pi})$	P_w (psf)
Level 6	68.71	0.89	25.55	0.8	17.37	21.97	12.77	21.97
Level 5	59.58	0.85	24.53	0.8	16.68	21.28	12.08	21.28
Level 4	48.94	0.81	23.19	0.8	15.77	20.37	11.17	20.37
Level 3	38.29	0.75	21.62	0.8	14.70	19.30	10.10	19.30
Level 2	27.65	0.68	19.70	0.8	13.39	17.99	8.80	17.99
Level 1	17	0.60	17.14	0.8	11.66	16.25	7.06	16.25

Table 12: Windward Wind Pressure Determination (East-West)

Leeward (East-West)								
Level	Height "z" (ft)	K_z	q_z (psf)	C_p	q_hGC_p	q_zGC_p - $q_h(-GC_{pi})$	q_zGC_p- $q_h(+GC_{pi})$	P_L (psf)
Level 6	68.71	0.89	25.55	-0.475	-10.31	-4.60	4.60	-14.91
Level 5	59.58	0.85	24.53	-0.497	-10.79	-4.60	4.60	-15.39
Level 4	48.94	0.81	23.19	-0.497	-10.79	-4.60	4.60	-15.39
Level 3	38.29	0.75	21.62	-0.497	-10.79	-4.60	4.60	-15.39
Level 2	27.65	0.68	19.70	-0.497	-10.79	-4.60	4.60	-15.39
Level 1	17	0.60	17.14	-0.2	-4.34	-4.60	4.60	-8.94

Table 13: Leeward Wind Pressure Determination (East-West)

Base Shear Determination (East-West)					
Level	Height "z" (ft)	Tributary Width (ft)	Tributary Area (ft^2)	Total Pressure (psf)	Total Story Force (kip)
Roof	68.71	282.00	2,574.66	36.88	47.48
Level 6	59.58	287.00	3,053.68	36.67	103.47
Level 5	48.94	287.00	3,056.55	35.76	110.63
Level 4	38.29	287.00	3,053.68	34.69	107.61
Level 3	27.65	287.00	3,056.55	33.38	103.98
Level 2	17.00	328.00	5,576.00	25.20	121.27
				Base Shear	595.72

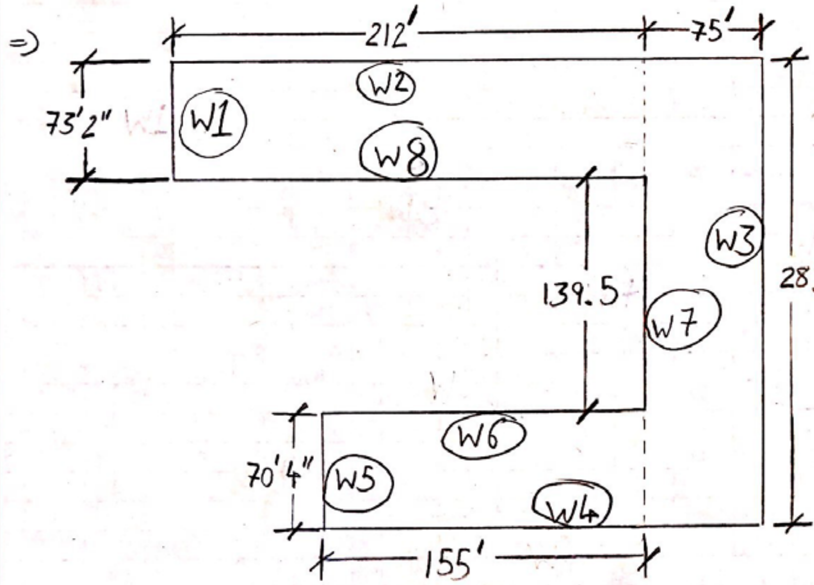
Table 14: Base Shear Determination (East-West)

This section of the wind load calculations assumes the building shape as an orthogonal U, with three wings intersecting at right angles. Chapter 9 from ASCE 7-10 Guide to the Wind Load Provisions for U-shaped apartment building was used as a reference to calculate 4 cases of wind loading on the assumed building shape.

→ Assumed building Shape:

⇒ First Floor – same as previous assumed shape

⇒



→ For wind Normal to:

⇒ W2 : W2, W6 → Windward → $C_{P_W} = 0.8$

W8 → $L/B = 283/287 = 0.986 \rightarrow C_{P_L} = -0.5$

W4 → $L/B = 283/230 = 1.23 \rightarrow C_{P_L} = -0.454$

W1, W3, W5, W7 → Side → $C_{P_S} = -0.7$

⇒ W4 : W4, W8 → Windward → $C_{P_W} = 0.8$

W6 → $L/B = 283/230 = 1.23 \rightarrow C_{P_L} = -0.454$

W2 → $L/B = 283/287 = 0.986 \rightarrow C_{P_L} = -0.5$

W1, W3, W5, W7 → Side → $C_{P_S} = -0.7$

⇒ W3 : W3 → Windward → $C_{P_W} = 0.8$

W1, W7, W5 → $L/B = 287/283 = 1.014$ → $C_{P_L} = -0.497$

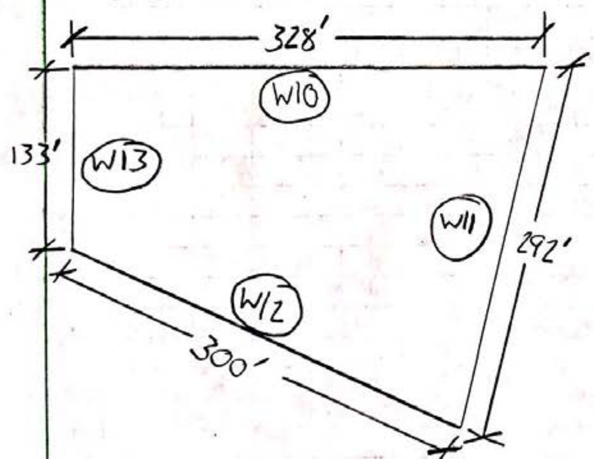
W2, W4, W6, W8 → Side → $C_{P_S} = -0.7$

⇒ W1 - W7 - W5 : W1, W7, W5 → Windward → $C_{P_W} = 0.8$

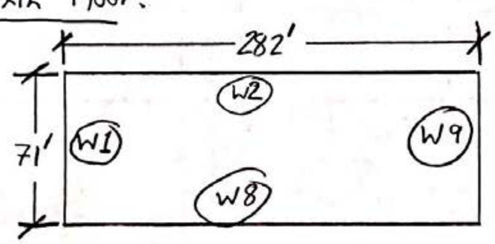
W3 → $L/B = 287/283 = 1.014$ → $C_{PL} = -0.497$

W2, W4, W6, W8 → Side → $C_{P_S} = -0.7$

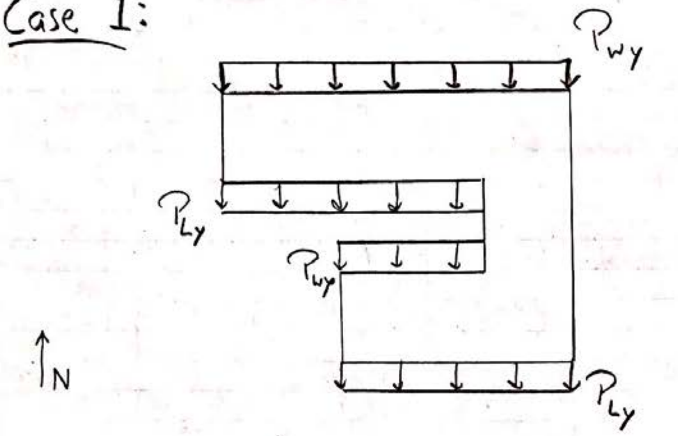
⇒ First Floor :

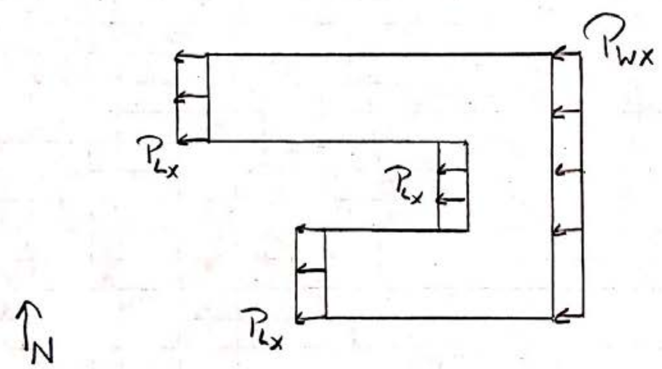


⇒ Sixth Floor :



→ Case 1:





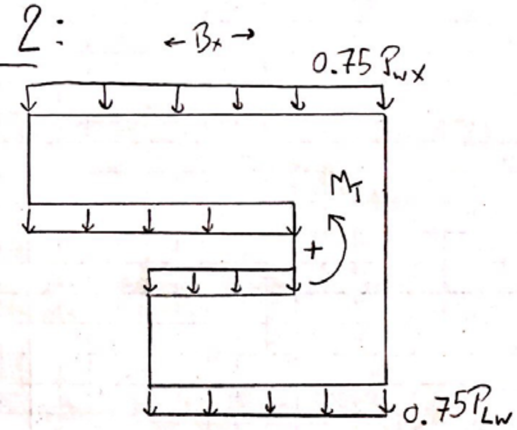
Base Shear Determination (Normal to W2)							
Level	**Height "z" (ft)**	**Wall**	**P (psf)**	**Trib Width (ft)**	**Shear (kips)**	**North - South (kips)**	**East - West (kips)**
Level 6	68.71	W2	21.97	282.00	56.57	39.79	-
		W8	-8.94	282.00	-23.02		
Level 5	59.58	W2	21.28	287.00	64.97	131.12	-
		W6	21.28	155.00	35.09		
		W8	-15.46	287.00	-47.20		
		W4	-14.46	230.00	-35.38		
Level 4	48.94	W2	20.37	287.00	62.25	180.58	-
		W6	20.37	155.00	33.62		
		W8	-15.46	287.00	-47.24		
		W4	-14.46	230.00	-35.41		
Level 3	38.29	W2	19.30	287.00	58.93	175.93	-
		W6	19.30	155.00	31.83		
		W8	-15.46	287.00	-47.20		
		W4	-14.46	230.00	-35.38		
Level 2	27.65	W2	17.99	287.00	54.99	170.34	-
		W6	17.99	155.00	29.70		
		W8	-15.46	287.00	-47.24		
		W4	-14.46	230.00	-35.41		
Level 1	17	W10	16.25	328.00	90.64	168.41	-
		W12	-15.46	300.00	-78.83		
					Base Shear	**866.17**	**-**

Table 15: Base Shear Determination (Normal to W2)

Wind Pressure Determination tables for wind forces Normal to walls W2, W4, W3, W1, W7, and W5 can be found in the Appendix A2: Wind Loads.

Base Shear Determination tables for case 1 where wind forces are Normal to walls W4, W3, W1, W7, and W5 can be found in the Appendix A2: Wind Loads.

Case 2:



$$M_T = 0.75(P_{wx} + P_{Lx}) B_x e_x$$

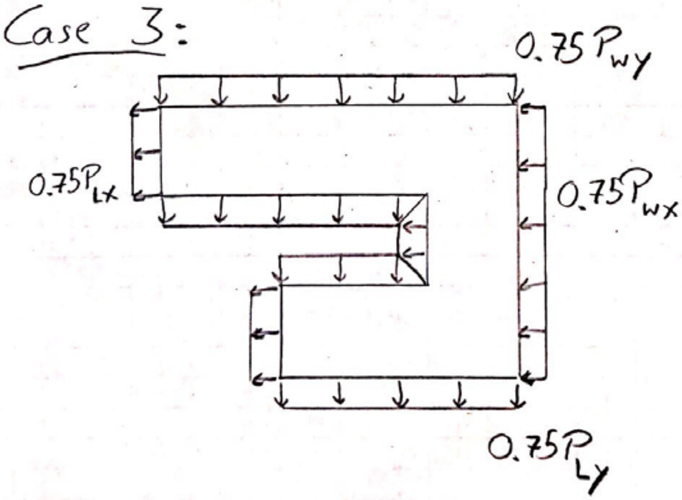
$$; e_x = 0.15(B_x)$$

$$= 0.15(287) = 43.1$$

Level 5: $M_T = 0.75[64.97 + 35.09 + 47.2 + 30.3](43.1)$

$= 5904$ kip-ft

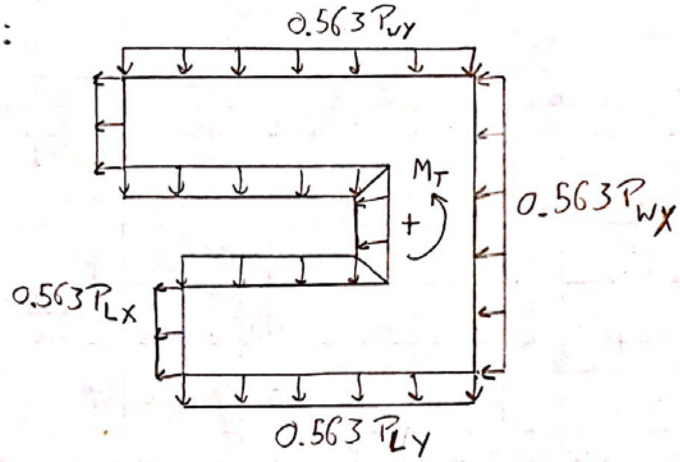
Case 3:



Base Shear Determination tables for case 2 where wind forces are Normal to walls W2, W4, W3, W1, W7, and W5 can be found in the Appendix A2: Wind Loads.

Base Shear Determination tables for case 3 where wind forces are Normal to walls W2 and W3 can be found in the Appendix A2: Wind Loads.

Case 4:



$$M_T = 0.563(P_{wx}+P_{Lx})\,B_x\,e_x + 0.563(P_{wy}+P_{WY})\,B_y\,e_y$$

where $e_x = 0.15(287) = 43.1$

$e_y = 0.15(283) = 42.5$

Level 5:

$$M_T = [36.58+19.76+26.57+19.92](43.1) + [6.75+36.07+6.48+12.86](42.5)$$

$$= 7074 \text{ kip-ft}$$

Base Shear Determination tables for case 4 can be found in the Appendix A2: Wind Loads.

4.2.2 REVISED SEISMIC LOADS

Seismic loads for the Revive Apartments were recalculated using the Equivalent Lateral Force (ELF) method from ASCE 7-10. This section includes a summary of seismic load calculation for the new proposed structure. Design base shear was determined to be 513.8 kips. Geo-technology Associates were responsible for providing geotechnical report for the site.

* Seismic Loads Summary:

→ Seismic Importance Factor ; $I_s = 1.0$

→ Seismic Site Class = C

→ Seismic Design Category = B

→ Analysis Procedure = Equivalent Lateral Force

→ Basic Structural System = Ordinary reinforced concrete shear walls

→ Site Data:

$S_s = 0.196g$ $S_{M1} = 0.141g$

$S_1 = 0.059g$ $S_{DS} = 0.209g$

$S_{MS} = 0.313g$ $S_{D1} = 0.094g$

→ Response Modification Coefficient, $R = 4$ (Table 12.2-1)

→ Overstrength Factor, $\Omega_o = 2\frac{1}{2}$ (Table 12.2-1)

→ Deflection Amplification Factor, $C_d = 4$ (Table 12.2-1)

→ Long Period Transition, $T_L = 6$ sec (USGS website)

→ Seismic Base Shear (Section 12.8.1):

$$V = C_s W$$

where ; Seismic response coefficient, $C_s = \dfrac{S_{DS}}{(R/I_e)}$ (12.8-2)

W = Effective Seismic Weight

⇒ Calculating T:

For all other structural systems, (Table 12.8-2)

$C_t = 0.02$; $x = 0.75$

$T_a = C_t h_n^x$

$= 0.02(68.71)^{0.75}$

$= 0.477$

$T_{max} = C_u T_a$, where $C_u = 1.7$ as $S_{D1} \leq 0.1$ (Table 12.8-1)

$= 1.7(0.477)$

$= 0.811\ s > T_a$ $\therefore T = 0.477\ s < T_L$

⇒ Calculating seismic response coefficient:

$$C_s = \begin{cases} \dfrac{S_{DS}}{R/I_e} = \dfrac{0.209}{4/1} = 0.052 \\ \dfrac{S_{D1}}{T(R/I_e)} = \dfrac{0.094}{0.477(4/1)} = 0.049 \\ \dfrac{S_{D1}T_L}{T^2(R/I_e)} = \dfrac{0.094(6)}{(0.477)^2(4/1)} = 0.620 \end{cases}$$

$C_s = 0.049 > 0.044 S_{DS} I_e \geq 0.01$

$= 0.01$

$\therefore C_s = 0.049$

⇒ Building Weight:

$W = 11{,}328$ kips

Includes : - Structure Self-weight
- Super Imposed Dead Load
- Partition
- Exterior Wall

→ Seismic Base Shear:

$$V = C_s W$$
$$= 0.049(10{,}486)$$
$$= 513.8 \text{kips}$$

→ Vertical Distribution:

$$C_{vx} = \frac{W_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \quad \text{; where} \quad k = 1$$

⇒ Sample Calc For Level 6:

$$C_{vx} = \frac{1115(59.58)}{2236(17) + 2123(27.65 + 38.29 + 48.94) + 1115(59.58) + 765(68.71)}$$
$$= 0.1657$$

$$V_6 = 0.1657\,(513.83) = 85.14 \text{ k}$$

Level	Height (ft)	Concrete Slab (ft^3)	Slab Dead Load (kips)	# of columns	Column Dead Load (kips)	Linear Feet of Shear Wall	Shear Wall Dead Load (kips)	Structure Self-Weight
Roof	79.71	19458.90	78808.545	49	186.40	153	279.378	79274.33
Level 6	70.58	40797.00	165227.85	97	430.03	248	527.744	166185.63
Level 5	59.94	40797.00	165227.85	97	430.44	248	528.24	166186.53
Level 4	49.29	40797.00	165227.85	97	430.03	248	527.744	166185.63
Level 3	38.65	40797.00	165227.85	97	430.44	248	528.24	166186.53
Level 2	28	59265.00	240023.25	129	1505.00	248	843.2	242371.45
Level 1	11	59265.00	240023.25	130	595.83	249	547.8	241166.88

Table 16: Structural Self-Weight Determination

Level	Height (ft)	Structure Self-Weight (kips)	SDL (psf)	Partition (psf)	Exterior Wall (psf)	Floor Surface Area	Wall Surface Area	Total Weight (kips)
Roof	79.71	79274.33	21	0	65	17,552	4,885	765
Level 6	70.58	166185.63	21	15	65	17,552	4,885	1,116
Level 5	59.94	166186.53	21	15	65	44,096	5,692	2,124
Level 4	49.29	166185.63	21	15	65	44,096	5,698	2,124
Level 3	38.65	166186.53	21	15	65	44,096	5,692	2,124
Level 2	28	242371.45	21	15	65	45,144	5,698	2,238
Level 1	11	241166.88	21	0	65	45,144	9,095	1,780
							Total Weight =	10,490

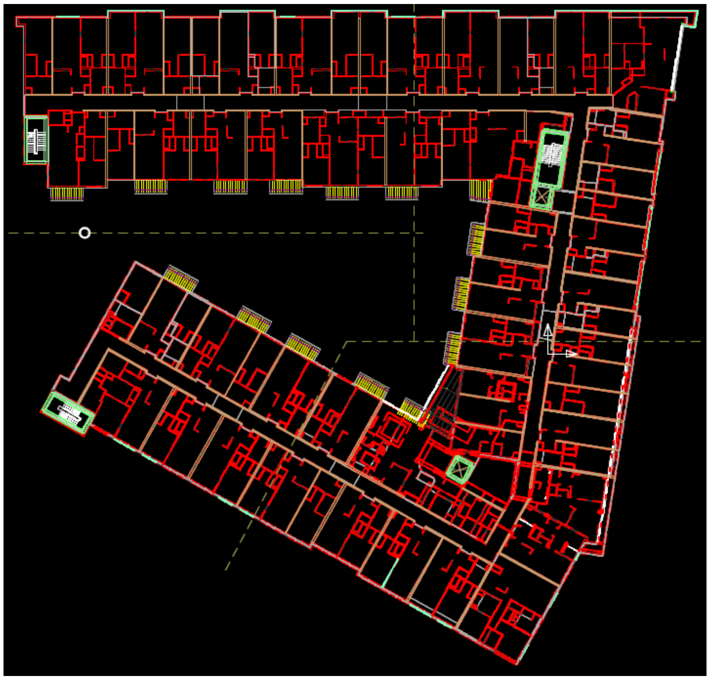
Table 17: Seismic Load Determination

Level	Height (ft)	Total Weight (kips)	C_{vx}	Story Shear (kips)
Roof	79.71	765	0.1138	58.50
Level 6	70.58	1,116	0.1469	75.50
Level 5	59.94	2,124	0.2375	122.06
Level 4	49.29	2,124	0.1953	100.39
Level 3	38.65	2,124	0.1531	78.71
Level 2	28	2,238	0.1169	60.09
Level 1	11	1,780	0.0365	18.78
Total			1.0000	514.02

Table 18: Base Shear Distribution

4.2.3 COMPUTER MODELING

Shear walls are located around elevator shafts and stair towers. They resist lateral loads in both North-South and East-West direction. As seen in figures 29 and 30, the shear walls are spread out evenly throughout the building. Shear walls around two stair towers and an elevator shaft on the Northern part of the building goes all the way up to the sixth floor.



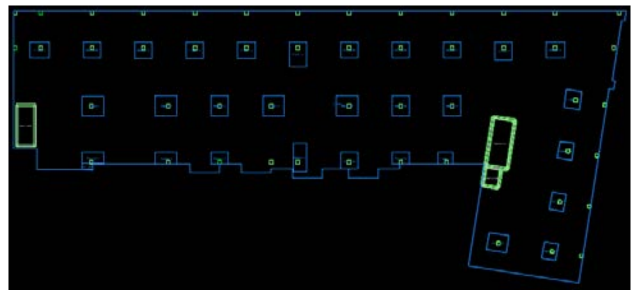


Figure 30: Sixth Floor Plan

Figure 29: Third Floor Typical Plan

This new shear wall system was designed using ETABS. In ETABS, shear walls are modeled as shell-thin type with 4000 psi concrete compressive strength and 8-inch thickness with fixed base (Figure 31). Floor diaphragms are modeled as shell-thin type with 4000 psi concrete compressive strength with a rigid diaphragm.

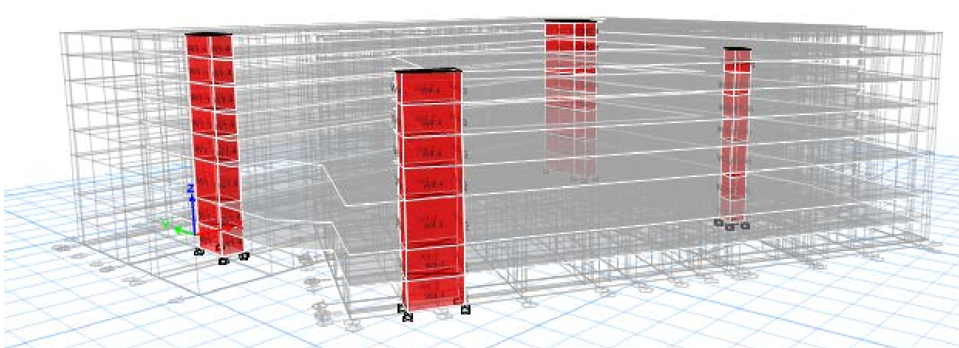


Figure 31: ETABS model 3D view

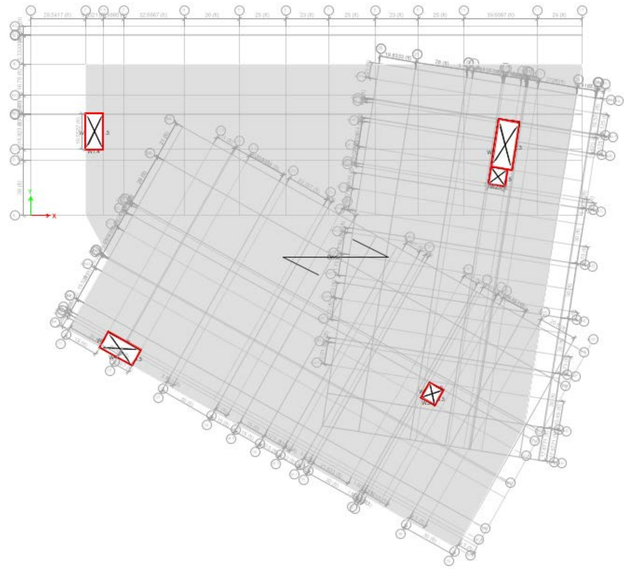


Figure 32: Typical Mid to 2nd Floor

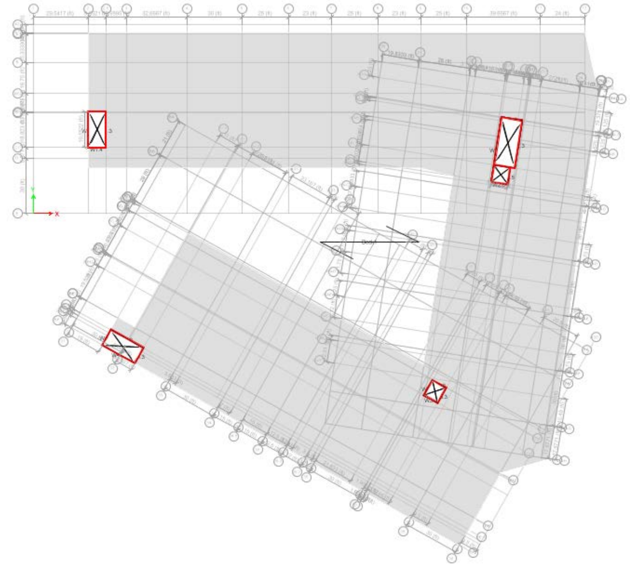


Figure 33: Typical 3rd to 5th Floor

Wind load cases controlled the lateral design for this building. In order to determine which load cases controlled, all of the load cases were applied on ETABS, and deflection, moment and shear diagrams of all shear walls were examined. Wind load case calculated assuming the building as a box controls in the x-direction with a base shear of 595.72 kips and wind load case 1, assuming a U-shaped building, controls in the y-direction with a base shear of 866.17 kips. More details on wind load calculations can be found in the revised wind load section above and in the Appendix: Wind Load calculations.

Table 19 below shows a comparison between hand calculated COR values and the ones ETABS calculated. Hand calculating COR values involved a complicated process of splitting stiffness values for shear walls not orthogonal to the loads applied. Stiffness for each shear wall was calculated assuming a fixed-fixed at the top and bottom. More information on these calculations can be found in the Appendix: Lateral System. Comparing hand calculated COR values and ETABS calculated, percent error in the x-direction is between 2% to 14% and y-direction ranges from 110% to 216% error. This error could have occurred as ETABS takes into account floor diaphragm above and below the level it calculates center of rigidity and hand calculations don't account for slab stiffness.

	Hand Calculated		Computer Model	
Floor Level	COR, X_r (ft)	COR, Y_r (ft)	COR, X_r (ft)	COR, Y_r (ft)
Roof	190.68	43.84	187.61	-51.05
Sixth Floor	162.39	-21.57	186.24	-52.47
Fifth Floor	162.39	-21.58	185.06	-52.30
Fourth Floor	162.39	-21.57	183.39	-51.85
Third Floor	162.39	-21.58	180.97	-51.11
Second Floor	161.70	-23.81	177.33	-50.06

Table 19: COR using hand calcs and computer modeling

	Wind		Seismic	
	Max Story Displacement	Max Allowable (H/400)	Max Story Displacement	Max Allowable ($0.01h_x$)
Roof	0.496229	2.06	0.501861	1.0956
Sixth	0.433309	2.06	0.43746	1.2768
Firth	0.359337	2.06	0.362167	1.278
Fourth	0.285708	2.06	0.286959	1.2768
Third	0.214078	2.06	0.213856	1.278
Second	0.146914	2.06	0.145783	2.04

Table 20: Displacement Summary

5 CONSTRUCTION BREADTH

This section of the report summarizes the change in construction cost and schedule for the redesigned structural system. RSMeans Building Construction Cost Data – 67th Annual Edition – 2009 was used to calculate estimated cost and schedule.

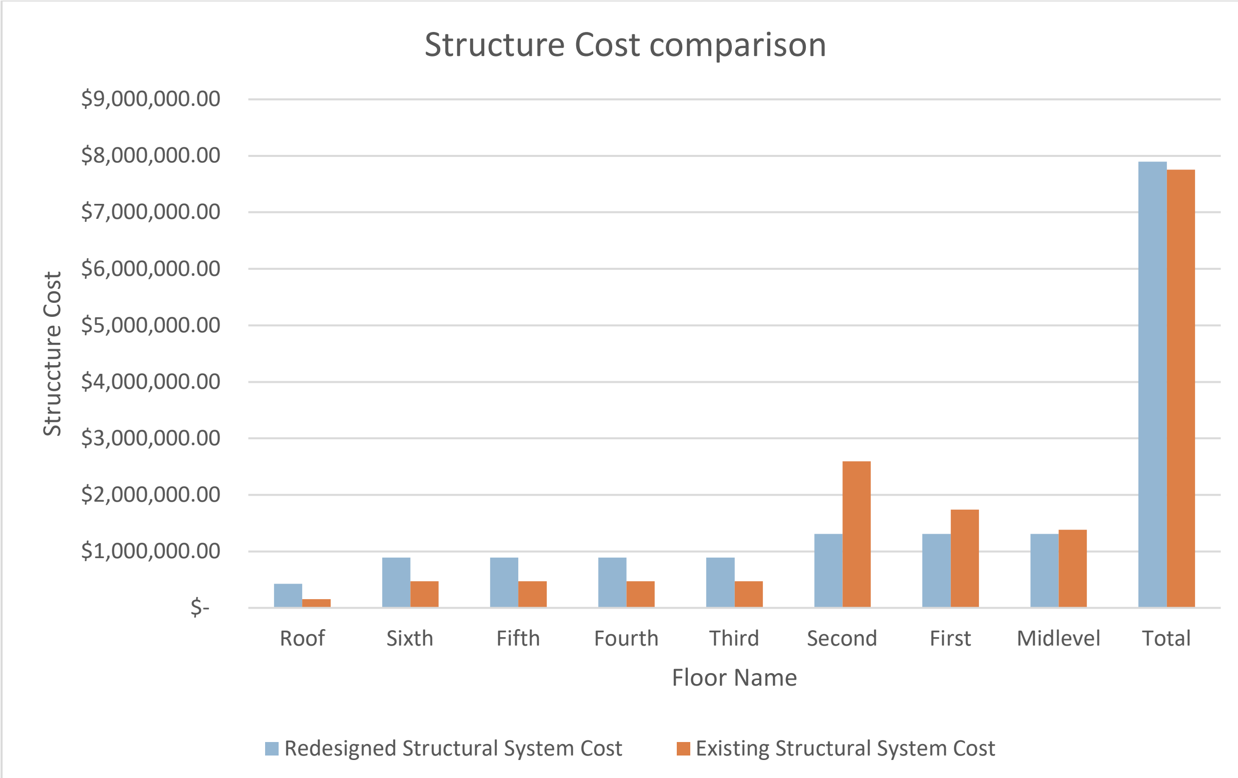
5.1 COST ESTIMATE

The existing structural system consists of three stories of steel framed floors and 6 stories, including roof, of wood framed structure. This combination of systems is used to minimize the cost of a building as wood is the most cost-efficient construction material. Takeoffs for the steel framed floors were tabulated from the ETABS model, created during notebook submissions last semester. These takeoffs include tons of steel used in the existing structure and the square-footage of the slabs. Takeoffs for the wood framed floors were tabulated using structural CD's, which included linear feet of shear walls, thousands of feet of wood joists and gross floor area. More details about these takeoffs can be found in the Appendix: Construction Breadth.

The redesigned structural system consists of two-way flat slab with drop panels with columns and shear walls. RAM Concept was used to tabulate the takeoffs for the two-way slab with drop panels. These included cubic yards of concrete, square feet of formwork, and tons of reinforcing. Takeoffs for concrete columns and shear wall were tabulated using designed sizes and reinforcing detail. More details about these takeoffs can be found in the Appendix: Construction Breadth.

After calculating the estimated structural cost of the above mentioned structural systems in accordance to 2009 RS Means, a time and location adjustment factors were used to make it more relatable to this project.

The graph below is a comparison of the existing structural system estimated cost and the redesigned system. Going from roof down, the redesigned system costs more than the existing until the third floor as it uses wood construction which is more economical than concrete. Existing structure on the second floor costs around $2 million versus $1.2 million for the redesigned system. In addition to second floor, first and Midlevel cost more than the redesigned structure, thus balancing out the levels where concrete structure is less economical. The redesigned concrete two-way slab with shear walls costs about $140,000 more than the existing.

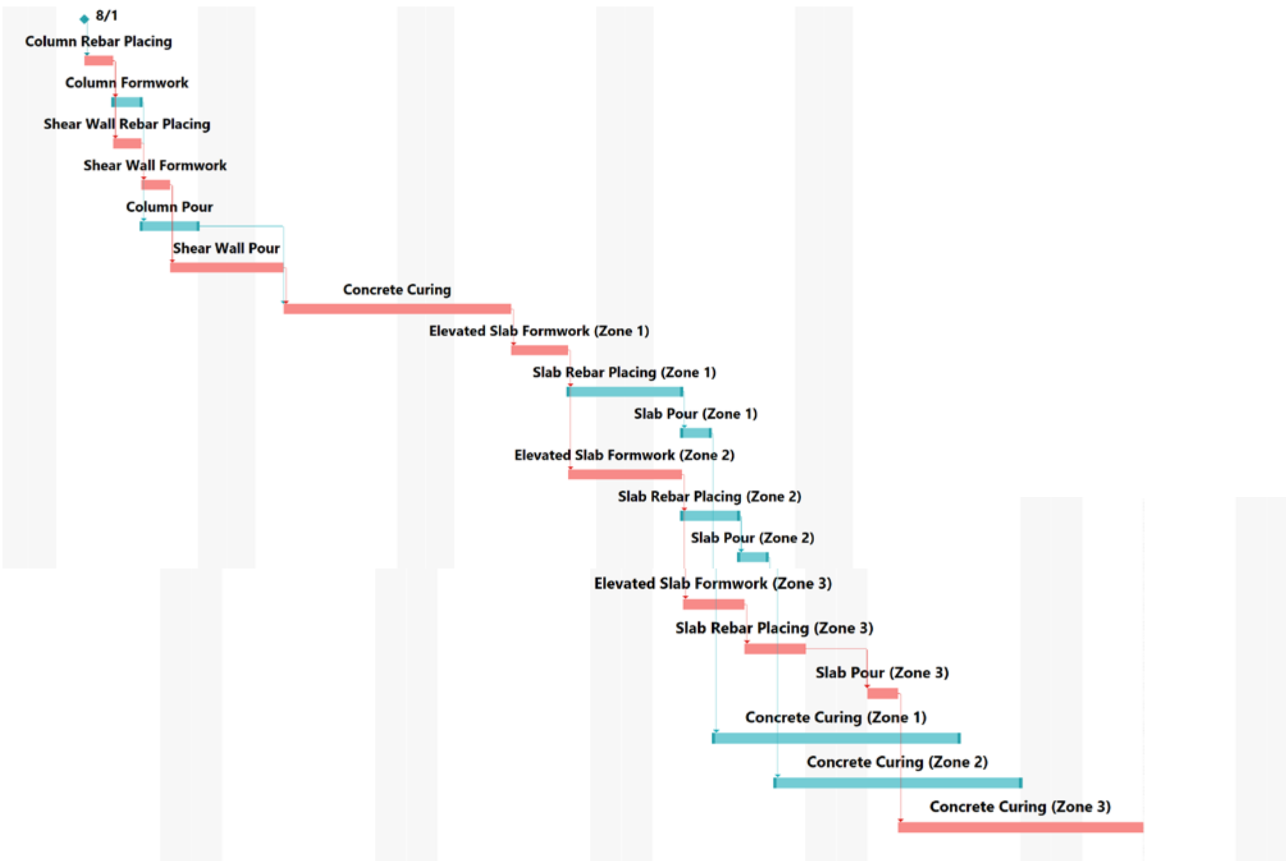


Level	Redesigned Total Cost	Existing Total Cost	Difference
Roof	$ 425,363.73	$ 152,634.55	$ 272,729.19
Sixth	$ 888,149.22	$ 472,167.62	$ 415,981.60
Fifth	$ 888,149.22	$ 472,167.62	$ 415,981.60
Fourth	$ 888,149.22	$ 472,167.62	$ 415,981.60
Third	$ 888,149.22	$ 472,167.62	$ 415,981.60
Second	$ 1,306,750.82	$ 2,592,831.79	$ (1,286,080.98)
First	$ 1,306,750.82	$ 1,737,930.81	$ (431,180.00)
Midlevel	$ 1,306,750.82	$ 1,382,427.14	$ (75,676.33)

Table 21: Existing and Redesign Cost Comparison

5.2 CONSTRUCTION SCHEDULE

Construction schedule was created using Microsoft Project and RS Means 2009. Due to the size of this building, it needed three zone. A floor would be completed with three pours.



6 ACOUSTICS BREADTH

This section of the report will focus on the redesigned structure's impact on sound transmissibility though partitions and floor system. The existing floor and partition wall assemblies will be analyzed and if necessary, it will be improved to at least meet the STC and IIC code minimum. Various floor assemblies and their STC and IIC ratings were investigated in order to find the best floor and partition wall assembly for the new structural system.

Minimum STC requirements according to floor-ceiling construction can be found in figure 34 below. Apartments fall under minimum quality classification. This means that while choosing a new assembly a target STC of 55 shall be achieved.

Table 15.4 Sound Transmission Class vs Level of Quality for Party Wall and Floor-Ceiling Construction

Classification	STC	FSTC
Minimum Code	50	45
Minimum Quality	55	50
Medium Quality	60	55
High Quality	65	60

Figure 34: STC versus level of construction for floor-ceiling construction (Marshall Long "Architectural Acoustics")

6.1 EXISTING FLOOR ASSEMBLY

Figure 35 on the right is a detailed section of typical partition assembly used between apartments. This particular type of partition assemblies are assigned a STC rating of 33. More details about the partition's STC requirements can be found in Appendix A5: Acoustics Breadth.

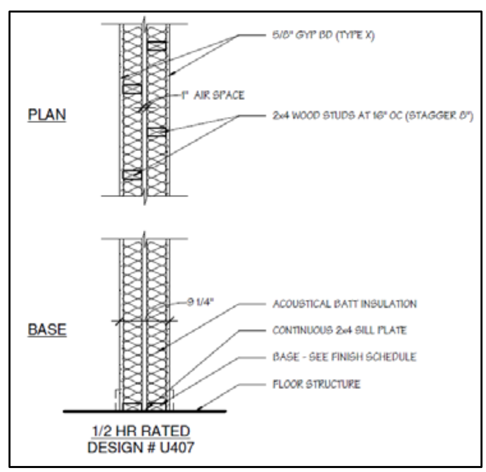


Figure 35: Detailed section of typical partition

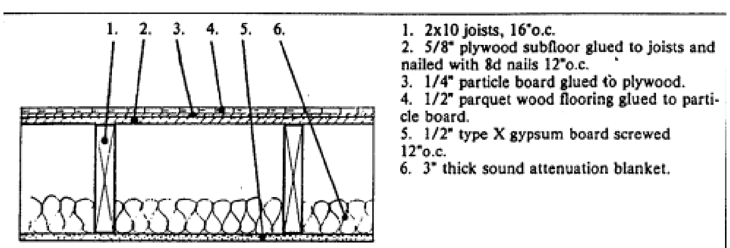


Figure 36: Existing Floor System (CDHS)

The floor assembly in figure 36 has STC of 43 was found in California Office of Noise Control's published Catalog of STC and IIC Ratings for Wall and Floor/Ceiling Assemblies.

6.2 REDESIGNED FLOOR ASSEMBLY

* Concrete slab STC calculation:

$STC = 16.5 \log(m) + 25$; where m = 125 psf weight of concrete slab

(6.2 From Architectural Acoustics Principles & Design)

$STC = 16.5 \log(125) + 25$

$= 59.6 > 55$ ∴ New Floor system meets target STC

7 CONCLUSION

The Revive Apartments was designed to act as a catalyst to redevelop the neighborhood. This six-story high/68-foot-tall mixed use/residential Revive Apartments is located in suburban Delaware. The Revive Apartments is mainly divided into 5 stories of multi-family residential space, 1 story of retail space, and 2 stories of underground parking. Approximately 330 vehicle parking, 10 retail spaces, 165 residential units, and amenities are housed in this 376,000 square feet of low-rise building. The existing superstructure is steel composite framing with moment frames on the bottom two levels and wood shear walls with wood joists on the remaining 5 stories.

A redesign of the structural system composed of concrete two-way flat plate slab with drop panels and columns as gravity system, and concrete shear walls as lateral system. Design and analysis of the redesigned system was performed using knowledge from AE 530 – Computer Modeling of Building Structures by utilizing software like RAM concept, RAM Structural System, and ETABS. Minimal impact was ensures while this overall redesign of the structural systems.

Additionally, Construction Cost Estimate and Schedule analysis and Acoustics were performed as breadth studies. The redesigned concrete structural system cost was found to be within a hundred thousand dollars of the original system's cost. The building enhanced its Sound Transmission Class and Impact Insulation Class ratings to meet its target.

The main purpose of redesigning the structural system to concrete was achieved by creating a more open floor plan and providing the owners an opportunity for multiple apartment sales. Additionally, the building will not require fireproofing its structure as 10" normal weight concrete slab is 4-hour fire rated and also provides better STC ratings.

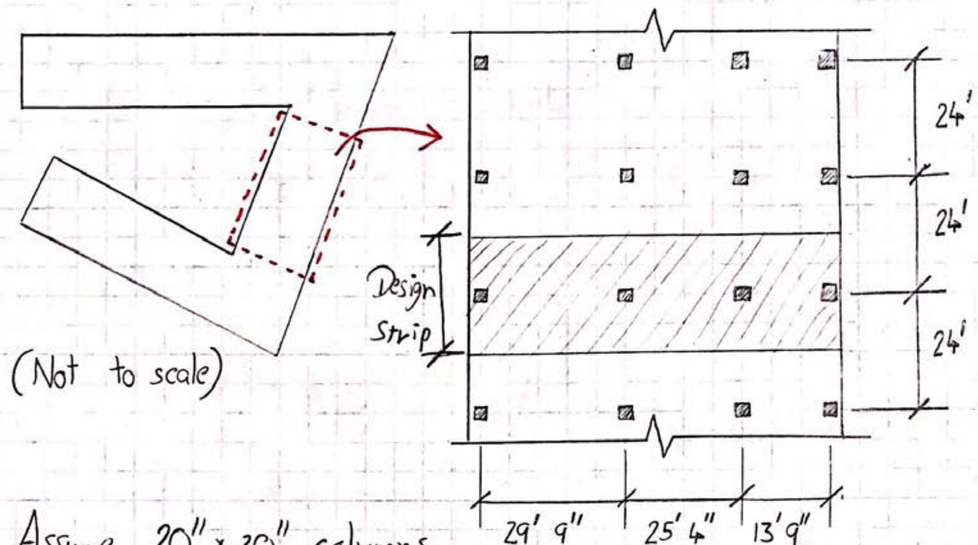
APPENDIX

A1 GRAVITY SYSTEM

A1.1 HAND CALCULATION

1

* Two Way Concrete Slab with Drop Panels:



Assume 20" x 20" columns
Story Height = 10.65 ft
Live Load = 100 psf (for retail space)
SDL = 21 psf
$f_y = 60$ ksi ; $f'_c = 4000$ psi (for slab) ; $f'_c = 6000$ psi (for columns)
Wall load = 65 psf (10.65 ft) = 692 plf

2

* Preliminary member sizing:

→ For Flat Plate (without Drop Panels):

⇒ Slab minimum thickness - Deflection

Exterior Panels: $h_s = \frac{l_n}{30} = \frac{337}{30} = 11.23$ in

Interior Panels: $h_s = \frac{l_n}{33} = \frac{337}{33} = 10.21$ in

$l_n = 29.75(12) - 20 = 337$ in

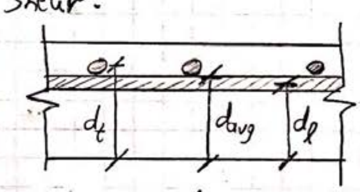
Try 11 in slab for all panels

⇒ Self-weight $= 150\text{ pcf}\left(\frac{11}{12}\right) = 137.5$ psf

⇒ Slab shear strength - one way shear:

Assume #6 bars &

$C_{clear} = 0.75$ in

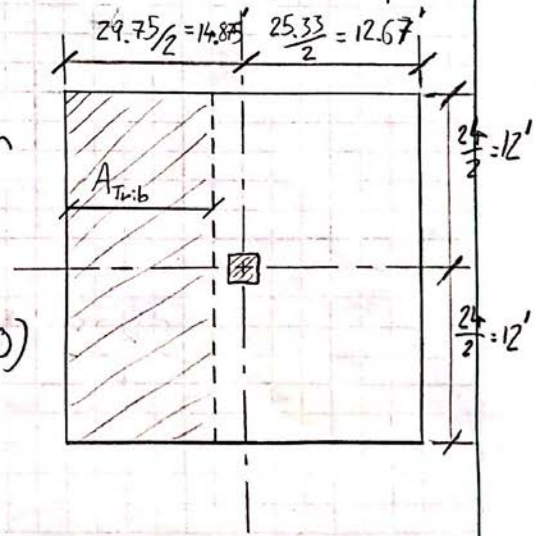


$d_l = 11 - 0.75 - 0.75 - \frac{0.75}{2}$

$= 9.13$ in

$d_t = 11 - 0.75 - \frac{0.75}{2} = 9.88$ in

$d_{avg} = \frac{9.13 + 9.88}{2} = 9.51$ in



$W_u = 1.2[137.5 + 21] + 1.6(100)$

$= 350.2$ psf

3

→ Interior column:

12-in wide strip

Trib area for one-way shear: $A_{Trib} = \left[\frac{29.75}{2} - \frac{20}{2(12)} - \frac{9.51}{12}\right]\frac{12}{12}$

$= 13.37 \ ft^2$

$V_u = W_u A_{Trib}$

$= 350.2(13.37) = 4.68$ kips

$\phi V_c = \phi 2 \lambda \sqrt{f_c'} b_w d$

$= 0.75(2)(1)\sqrt{4000}(12)(9.51)/1000$

$= 10.83$ kips $> V_u$

∴ 11-in is adequate for one-way shear

⇒ Slab shear strength – two way shear

$A_{Trib} = (14.875 + 12.67)(24) - \left(\frac{20+9.51}{12}\right)^2$

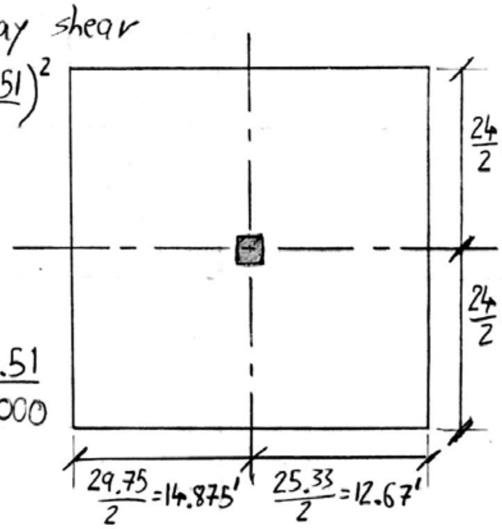
$= 655 \ ft^2$

$V_u = 0.350(655) = 229.3$ kips

$\phi V_c = \phi 4 \lambda \sqrt{f_c'} b_w d$

$= 0.75(4)(1)\sqrt{4000}\left[4(20+9.51)\right]\frac{9.51}{1000}$

$= 213$ kips $< V_u$



∴ Slab thickness of 11 in is not adequate for Two-way shear

4

→ For Flat Plate (with Drop Panels):

⇒ Slab min thickness - Deflection:

Exterior Panels: $h_s = \frac{l_n}{33} = \frac{337}{33} = 10.2$ in

Interior Panels: $h_s = \frac{l_n}{36} = \frac{337}{36} = 9.36$ in

∴ Try 10 in slab for all panels

Self weight without drop panel $= 150\left(\frac{10}{12}\right) = 125$ psf

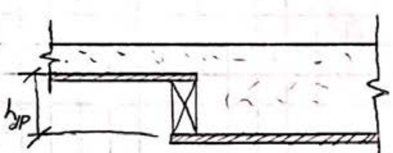
⇒ Drop panel size:

→ Project at least one-fourth of slab thickness

$h_{dp,min} = 0.25(10) = 2.5$ in

→ Formwork consideration

$h_{dp} = \underbrace{3\tfrac{1}{2}}_{(4x)} + \underbrace{\tfrac{3}{4}}_{\text{plywood}} = 4.25'' > h_{dp,min}$



Self weight with drop panel $= 150\left(\frac{14.25}{12}\right) = 178.1$ psf

→ Drop panel shall not extend less than 1/6th the span length

$L_{1,dp} = \frac{1}{6}(24) + \frac{1}{6}(24) = 8$ ft

$L_{2,dp} = \frac{1}{6}(29.75) + \frac{1}{6}(25.33) = 9.18$ ft

$L_{3,dp} = \frac{1}{6}(25.33) + \frac{1}{6}(13.75) = 6.51$ ft

⇒ Slab shear strength - one way shear:

Assume #6 bars & $c_{clear} = 0.75$ in

$d_\ell = 14.25 - 0.75 - 0.75 - \frac{0.75}{2} = 12.375$ in

$d_t = 14.25 - 0.75 - \frac{0.75}{2} = 13.125$ in

$d_{avg} = \frac{12.375 + 13.125}{2} = 12.75$ in

$W_u = 1.2[178.1 + 21] + 1.6(100) = 399$ psf

→ Consider 12-in wide strip

Trib area for one-way shear; $A_{Trib} = \left[\frac{29.75}{2} - \frac{20}{2(12)} - \frac{12.75}{12}\right]\frac{12}{12}$

$= 12.98\ ft^2$

$V_u = 399(12.98) = 5179$ lbs

$\phi V_c = \phi 2 \lambda \sqrt{f'_c}\, b_w d$

$= 0.75(2)(1)\sqrt{4000}(12)(12.75)$

$= 14{,}515 > V_u$ ∴ OK for one-way shear

⇒ Critical section at edge of drop panel (slab section without drop panel):

Assume #6 bars & $c_{clear} = 0.75$ in

$d_\ell = 10 - 0.75 - 0.75 - \frac{0.75}{2} = 8.13$ in

$d_t = 10 - 0.75 - 0.75/2 = 8.88$ in

$d_{avg} = \frac{8.13 + 8.88}{2} = 8.51$ in

$W_u = 1.2[178.1 + 21] + 1.6(100) = 399$ psf

6

→ Consider 12-in wide strip

$$A_{trib} = \left[\frac{29.75}{2} - \frac{9.18}{2}\right]\frac{12}{12} = 10.3\ ft^2$$

$$V_u = 0.4(10.3) = 4.12\ kips$$

$$\phi V_c = 0.75(2)(1)\sqrt{4000}\ (12)(8.51)$$

$$= 9.7\ kips > V_u$$

⇒ Slab shear strength – two-way shear:

→ For critical section at distance $d/2$ from edge of column (slab with drop panel):

⇒ Interior column:

$$A_{Trib} = (14.875 + 12.67)(24) - \left(\frac{20+12.75}{12}\right)^2 = 654\ ft^2$$

$$V_u = 0.4(654) = 261.5\ kips$$

$$\phi V_c = 0.75(4)(1)\sqrt{4000}\left[4(20+12.75)\right]\frac{12.75}{1000}$$

$$= 317\ kips > V_u$$

∴ Ok for two-way shear with drop panels

→ For critical section at the edge of the drop panel. (slab without drop panel):

$$A_{Trib} = (14.875 + 12.67)(24) - 8(9.18) = 587.6\ ft^2$$

$$V_u = 0.4(587.6) = 235\ kips$$

$$\phi V_c = 0.75(4)(1)\sqrt{4000}\ \left[4(8)(9.18)\right]\frac{8.51}{1000}$$

$$= 474.5\ kips > V_u$$

∴ Ok for two-way shear for second critical section

7

⇒ Column Dimensions - axial load:

$$A_{Trib} = \frac{(29.75+25.33)}{2}\frac{(24+24)}{2} = 661 \ ft^2$$

For additional drop panel weight : $w_{uDP} = 150(4.25)(\frac{1}{12}) = 0.053$ klf

$$A_{Trib} = 8(9.18) = 73.44 \ ft^2$$

For retail level: $w_u = 1.2(125+21) + 1.6(100) = 0.335$ klf

$$P_u = 0.335(661) + 0.053(73.44)$$

$$= 225.3 \text{ kips}$$

For parking & residential levels: $w_u = 1.2(125+21) + 1.6(40) = 0.239$ klf

$$P_u = 6[0.239(661) + 0.053(73.44)]$$

$$= 971.2 \text{ kips}$$

Total: $P_u = 225.3 + 971.2$

$$= 1196.5 \text{ kips}$$

→ Assume 20 in square column with 4 - No. 14 vertical bars

$$\phi P_{n,max} = 0.8\phi[0.85 f_c'(A_g - A_{st}) + f_y A_{st}]$$

$$= 0.8(0.65)[0.85(6000)(20^2 - 4(2.25)) + 60{,}000(4)(2.25)]$$

$$= 1318 \text{ kips} > P_u$$

∴ Column dimensions of 20 in × 20 in are adequate
For axial load with 4 #14
with $f_c' = 6000$ psi

8

* Frame members of equivalent frame:

→ Flexural stiffness of slab-beams, K_{sb} (29'9" span):

$$\frac{C_{N1}}{\ell_1} = \frac{20}{29.75(12)} = 0.056 \quad ; \quad \frac{C_{N2}}{\ell_2} = \frac{20}{24(12)} = 0.069$$

Table A1: $C_{F1} = C_{N1}$; $C_{F2} = C_{N2}$

$$k_{NF} = k_{FN} = 4.1$$

$$K_{sb} = \frac{k_{NF} E_{cs} I_s}{\ell_1}$$, where $E_{cs} = 57{,}000\sqrt{4000} = 3.6 \times 10^6$ psi

$$I_s = \frac{\ell_2 h^3}{12} = \frac{24(12)(10)^3}{12} = 24000 \text{ in}^4$$

$$= \frac{4.1(3.6)(10)^6(24{,}000)}{29.75(12)}$$

$$= 992 \times 10^6 \text{ in-lbs}$$

Table A1: Carry-over factor, COF = 0.506

Fixed-end moment, FEM: $m_{NF} = 0.0151$ @ a=0

Uniform Load: $m_{NF} = 0.0838$ $\qquad$ $m_{NF} = 0.0023$ @ a=0.8

→ Flexural stiffness of column members at both ends, K_c

Bottom Column:

$$t_a = \frac{10}{2} + 4.25 = 9.25 \text{ in} \quad ; \quad t_b = \frac{10}{2} = 5 \text{ in}$$

$$\frac{t_a}{t_b} = \frac{9.25}{5} = 1.85$$

$$H_c = [11(12) - 9.25 - 5]/12 = 9.813 \text{ ft}$$

$$\frac{H}{H_c} = \frac{11}{9.813} = 1.12$$

Thus, from Table A7:

$k_{AB} = 5.616$

$C_{AB} = 0.5465$

9

$K_{c,bottom} = \frac{5.616 E_{cc} I_c}{l_c}$; where $I_c = \frac{(20)^4}{12} = 13,333 \text{ in}^4$

$E_{cc} = 57,000\sqrt{5000} = 4.03 \times 10^6$ psi

$l_c = 11 \text{ ft} = 132 \text{ in}$

$= \frac{5.616(4.03 \times 10^6)(13,333)}{132}$

$= 2286 \times 10^6$ in-lbs

$\Rightarrow$ Top Column:

$\frac{t_a}{t_b} = \frac{5}{9.25} = 0.54$

$H_c = 17(12) - 9.25 - 5 = 15.813$ ft

$\frac{H}{H_c} = \frac{17}{15.813} = 1.08$

Thus, from Table A7: $k_{BA} = 4.693$

$C_{BA} = 0.576$

$K_{c,top} = \frac{4.693 E_{cc} I_c}{l_c}$

$= \frac{4.693(4.03)(10)^6(13,333)}{17(12)} = 1236 \times 10^6$ in-lbs

10

→ Torsional stiffness of torsional members, K_t

$$K_t = \frac{9E_{cs}C}{\left[l_2\left(1-\frac{c_2}{l_2}\right)^3\right]} \quad ; \text{ where } \quad C = \sum\left(1-0.63\frac{x}{y}\right)\left(\frac{x^3y}{3}\right)$$

$$= \left(1-0.63\left(\frac{14.25}{20}\right)\right)\left[\frac{(14.25)^3(20)}{3}\right]$$

$$= 10632 \text{ in}^4$$

$c_2 = 20$ in & $l_2 = 29.75$ ft $= 357$ in

$$= \frac{9(3.6)(10)^6(10632)}{357\left(1-\frac{20}{357}\right)^3}$$

$$= 1147 \times 10^6 \text{ in-lb}$$

→ Equivalent column stiffness:

$$K_{ec} = \frac{\sum K_c \times \sum K_t}{\sum K_c + \sum K_t}$$

$$= \frac{(2286+1236)(2)(1147)}{2286+1236+2(1147)} \times 10^6 = \underline{1389 \times 10^6 \text{ in-lbs}}$$

→ Flexural stiffness of slab-beams, K_{sb} (25'4" span):

$$\frac{C_{N1}}{\ell_1} = \frac{20}{25.33(12)} = 0.066 \quad ; \quad \frac{C_{N2}}{\ell_2} = 0.069$$

Table A1 : $C_{F1} = C_{N1}$; $C_{F2} = C_{N2}$

$$k_{NF} = k_{FN} = 4.12$$

$$K_{sb} = \frac{4.12(3.6)(10)^6(24{,}000)}{25.33(12)}$$; where E_{cs} & I_s same as 29'9" span

$$= 1171 \times 10^6 \text{ in-lbs}$$

Table A1 : COF = 0.506

FEM : same as 29'9" span calcs

→ Flexural stiffness of slab-beams, K_{sb} (13'9" span):

$$\frac{C_{N1}}{\ell_1} = \frac{20}{13.75(12)} = 0.12 \quad ; \quad \frac{C_{N2}}{\ell_2} = 0.069$$

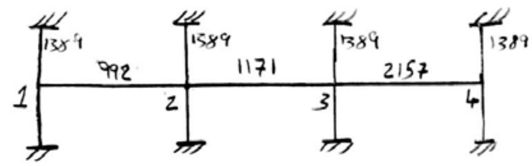
Table A1 : $C_{F1} = C_{N1}$; $C_{F2} = C_{N2}$

$$k_{NF} = k_{FN} = 4.12$$

$$k_{sb} = \frac{4.12(3.6)(10)^6(24{,}000)}{13.75(12)}$$

$$= 2157 \times 10^6 \text{ in-lbs}$$

→ Slab-beam joint DF :



Exterior : $DF_{1-2} = \dfrac{992}{992+1389} = 0.417$

Interior: $DF_{2-1} = \dfrac{992}{992+1171+1389} = 0.279$

$DF_{2-3} = \dfrac{1171}{992+1171+1389} = 0.33$

$DF_{3-2} = \dfrac{1171}{1171+2157+1389} = 0.248$

$DF_{3-4} = \dfrac{2157}{1171+2157+1389} = 0.457$

Exterior: $DF_{4-3} = \dfrac{2157}{2157+1389} = 0.608$

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* Equivalent frame analysis:

$$\frac{L}{D} = \frac{100}{(125+21)} = 0.68 < \frac{3}{4}$$

→ Factored load and Fixed-end Moments:

⇒ For slab:

$$q_{Du} = 1.2\,(125+21) = 175.2 \text{ psf}$$

$$q_{Lu} = 1.6\,(100) = 160 \text{ psf}$$

$$q_u = 175.2 + 160 = 335.2 \text{ psf}$$

⇒ For drop panels:

$$q_u = 1.2\,(150)\left(\tfrac{4.25}{12}\right) + 1.6\,(0) = 63.75 \text{ psf}$$

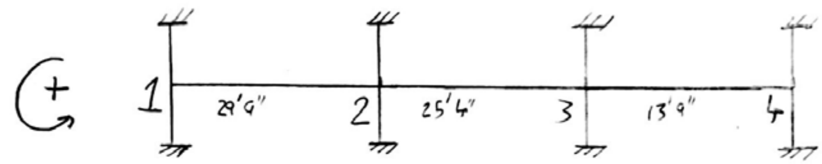
$$\Rightarrow FEM_{1-2} = \sum_{i=1}^{n} m_{NF_i} W_i l_i^2$$

$$= 0.0838\,(0.335)(24)(29.75)^2 + 0.0151\,(0.064)\frac{(24)}{3}(29.75)^2 + 0.0023\,(0.064)\frac{(24)}{3}(29.75)^2$$

$FEM_{1-2} = 604.2$ ft-kips ; $FEM_{2-3} = 438$ ft-kips ; $FEM_{3-4} = 129$ ft-kips

→ Moment distribution:

$$M_{u,\text{midspan}} = M_o - \frac{(M_{uL} + M_{uR})}{2}$$



Joint	1	2		3		4
Member	1-2	2-1	2-3	3-2	3-4	4-3
DF	0.417	0.279	0.33	0.248	0.457	0.608
COF	0.506	0.506	0.506	0.506	0.506	0.506
FEM	604.2	-604.2	438	-438	129	-129
Dist	-252	46.4	$^{166.2}$ 54.8	76.6	$^{33.9}$ 141.2	78.4
CO	23.5	-127.5	38.8	27.7	39.7	71.4
Dist	-9.8	24.7	$^{88.7}$ 29.3	-16.7	$^{-67.4}$ -30.8	-43.4
CO	12.5	-5	-8.5	14.8	-22	-15.6
Dist	-5.2	3.8	$^{13.5}$ 4.5	1.8	$^{7.2}$ 3.3	9.5
CO	1.9	-2.6	0.9	2.3	4.8	1.7
Dist	-0.8	0.5	$^{1.7}$ 0.6	-1.8	$^{-7.1}$ -3.2	-1
CO	0.3	-0.4	-0.91	0.3	-0.51	-1.6
Dist	-0.13	0.37	$^{1.31}$ 0.43	0.05	$^{0.21}$ 0.1	0.97
CO	0.19	-0.07	0.03	0.22	0.5	0.05
M(k-ft)	375	-664	558	-333	262.1	-28.6
M @ midspan	370.5		199.5		44.7	

$$M_u = 0.335(24)\frac{(29.75)^2}{8} + 2\left[\frac{0.064(29.75)/6\,(24)/6}{(29.75)(24)}(29.75)/6\,(29.75 - 24/2)\right]$$

$$- \left(\frac{M_{uL} + M_{uR}}{2}\right)$$

Span 1-2: $M_u = 890 - \frac{(375+664)}{2} = 370.5$

Span 2-3: $M_u = 645 - \frac{(558+333)}{2}$

$= 199.5$

Span 3-4: $M_u = 190 - \frac{(262.1+28.6)}{2}$

$= 44.7$

$\rightarrow$ Design moments:

$\frac{20''}{2(12)} = 0.833\ ft < 0.175(24) = 4.2\ ft$

$\therefore$ Use face of support location

→ Total factored moment per span:

$$M_o = \frac{q_u l_2 l_n^2}{8} = \frac{0.335(24)(29.75 - 20/12)^2}{8}$$

$$= 574.4 \text{ ft-kips}$$

End span: $370.5 + \frac{375 + 664}{2} = 890$ ft-kips

Interior span: $199.5 + \frac{558 + 333}{2} = 645$ ft-kips

Permissible reduction $= \frac{574.4}{890} = 0.645$

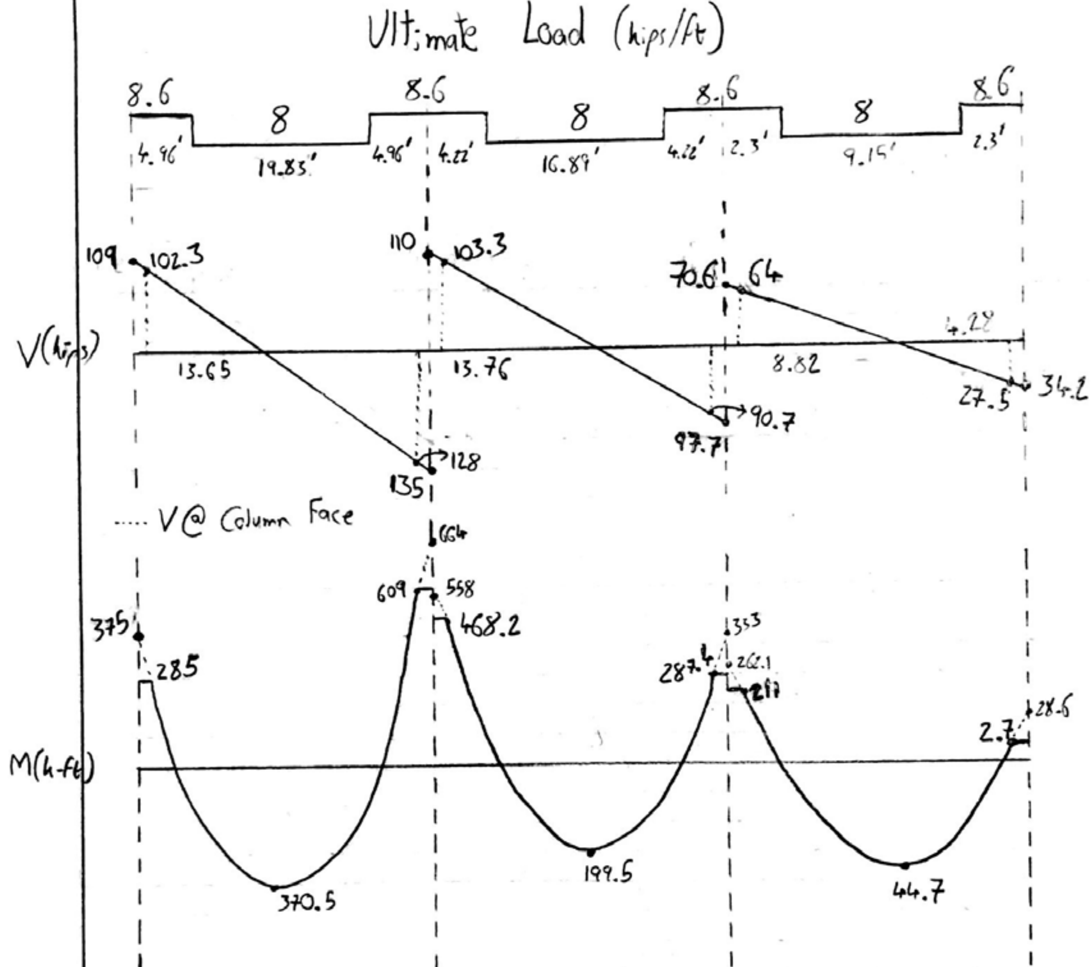
Adj. −ve design moment = 664 (0.645) = 428.3

Adj. +ve design moment = 375 (0.645) = 241.9

$$M_o = 241.9 + \frac{428.3 + 428.3}{2} = 670.2 \text{ ft-kips}$$

⇒ Distribution of factored moments:

		Slab-beam Strip	Column Strip		Middle Strip	
		Moment (kip-ft)	Percent	Moment (kip-ft)	Percent	Moment (kip-ft)
End Span	Exterior −ve	285	100	285	0	0
	+ve	370.5	60	222.3	40	148.2
	Interior −ve	609	75	456.8	25	152.2
Interior Span	−ve	468.2	75	351.2	25	117
	+ve	199.5	60	119.7	40	79.8
	−ve	287.4	75	215.6	25	71.8
End Span	Interior −ve	211	75	158.3	25	52.7
	+ve	44.7	60	26.8	40	17.9
	Exterior −ve	2.7	100	2.7	0	0



$$M = \frac{102.3\left(13.65 - \frac{10}{12}\right)}{2}$$

$= 285$ k-ft

* Slab flextural and shear strength at exterior column:

→ Reinforcement required for column strip moment $M_u = 285$ k-ft

⇒ Assume tension controlled section: $d_{avg} = 12.75$ in

Column strip width, $b = \frac{24(12)}{2} = 144$ in

$$R_u = \frac{M_u}{\phi b d^2} = \frac{285(12)}{0.9(144)(12.75)^2} = 162 \text{ psi}$$

$$\rho = \frac{0.85 f'_c}{f_y}\left(1 - \sqrt{1 - \frac{2R_u}{0.85 f'_c}}\right)$$

$$= \frac{0.85(4)}{60}\left[1 - \sqrt{1 - \frac{2(162)}{0.85(4000)}}\right] = 0.00277$$

$$A_s = \rho b d = 0.00277(144)(12.75) = 5.08 \text{ in}^2$$

Weighted slab thickness, $h_w = \frac{14.25(24/3) + 10(24/2 - 24/3)}{24/3 + (24/2 - 24/3)}$

$= 12.83$ in

$$A_{s,min} = 0.0018(144)(12.83) = 3.33 \text{ in}^2 < 5.08 \quad \therefore OK$$

$$S_{max} = 2h_w = 2(12.83) = 25.66 \text{ in} > 18$$

$$\therefore S_{max} = 18 \text{ in}$$

Provide 8 #8 with $A_s = 6.32$ in² and

$$s = \frac{144}{8} = 18 \text{ in} \leq s_{max}$$

Based on the procedure outlined above, values for all span locations are given in the table below:

Required Slab Reinforcement for Flexure

Span	Location	M_u (k-ft)	b (in)	d (in)	A_s Req'd (in^2)	$A_{s,min}$ (in^2)	Reinforcement Provided	A_s Prov. (in^2)
			End	Span				
Column Strip	Exterior -ve	285	144	12.75	5.08	3.33	15 #6	6.6
	+ve	222.3	144	8.5	6.08	2.2	15 #6	6.6
	Interior -ve	456.8	144	12.75	8.29	3.33	20 #6	8.8
Middle Strip	Exterior -ve	0	144	8.5	–	2.2	8#6	3.52
	+ve	148.2	144	8.5	3.99	2.2	10#6	4.4
	Interior -ve	152.2	144	8.5	4.1	2.2	10#6	4.4
			Interior	Span				
Column Strip	+ve	119.7	144	8.5	3.2	2.2	8#6	3.52
Middle strip	+ve	79.8	144	8.5	2.12	2.2	8#6	3.52
			End	Span				
Column Strip	Interior -ve	158.3	144	12.75	2.8	3.33	8#6	3.52
	+ve	26.8	144	8.5	0.7	2.2	8#6	3.52
	Exterior -ve	2.7	144	12.75	–	3.33	8#6	3.52
Middle Strip	Interior -ve	52.7	144	8.5	1.4	2.2	8#6	3.52
	+ve	17.9	144	8.5	0.5	2.2	8#6	3.52
	Exterior -ve	0	144	8.5	–	2.2	8#6	3.52

→ Check slab reinforcement at exterior column for moment transfer between slab & column:

⇒ Portion of unbalanced moment transferred by flexure is $\gamma_f \times M_u$

$\gamma_f = \dfrac{1}{1+\frac{2}{3}\sqrt{b_1/b_2}}$; where $d = h - \text{cover} - d/2$

$= 14.25 - 0.75 - 0.75/2 = 13.13$ in

$b_1 = c_1 + d/2$

$= 20 + 13.13/2 = 26.56$ in

$b_2 = c_2 + d$

$= 20 + 13.13 = 33.13$ in

$b_s = 20 + 3(14.25) = 62.75$ in

$$\gamma_f = \frac{1}{1+\frac{2}{3}\sqrt{26.56/33.13}} = 0.626$$

$\gamma_f M_u = 0.626(285) = 178.4$ ft-kip

$$R_u = \frac{M_u}{\phi b d^2} = \frac{178.4(12)}{0.9(62.75)(12.75)^2} = 233 \text{ psi}$$

$$\rho = \frac{0.85(4)}{60}\left(1-\sqrt{1-\frac{2(233)}{0.85(4000)}}\right) = 0.004$$

$A_s = 0.004(62.75)(12.75) = 3.22$ in^2

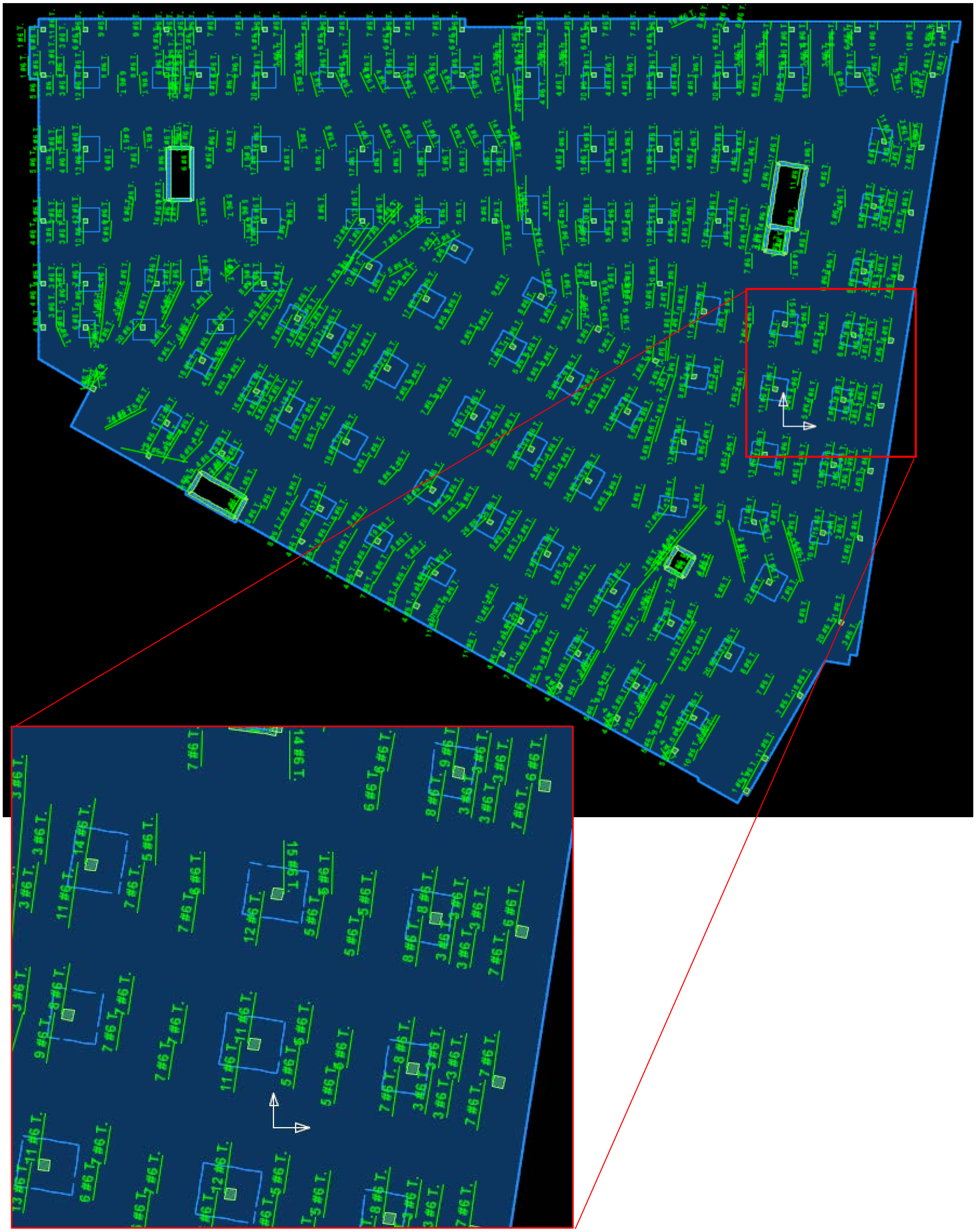
$A_{s,provided} = 6.32\dfrac{(62.75)}{144} = 2.75$ in^2

$A_{s,additional} = 3.22 - 2.75 = 0.47$ in^2

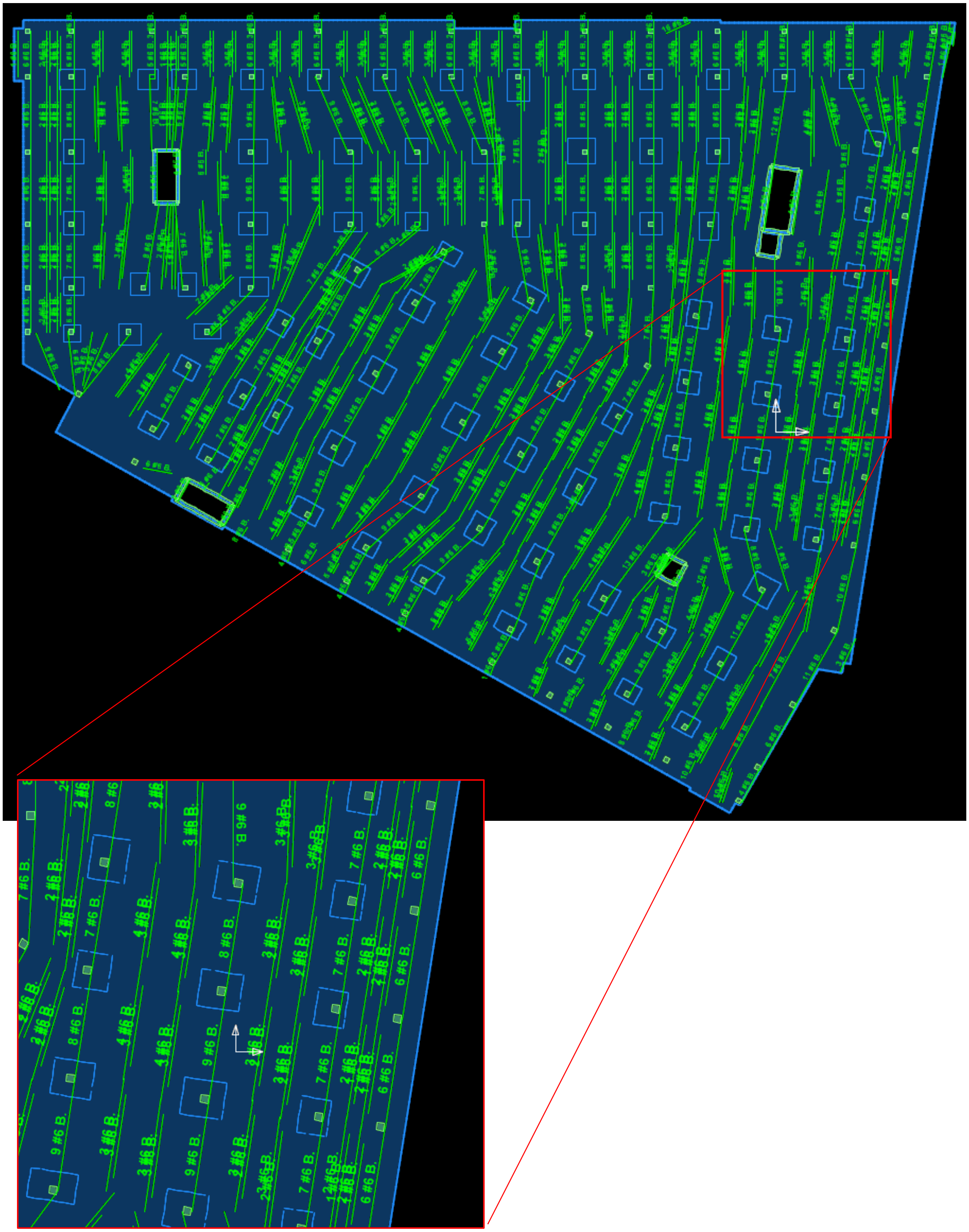
Provide 1-#8 additional bar with $A_s = 0.79$ in^2

A1.2 RAM CONCEPT RESULTS

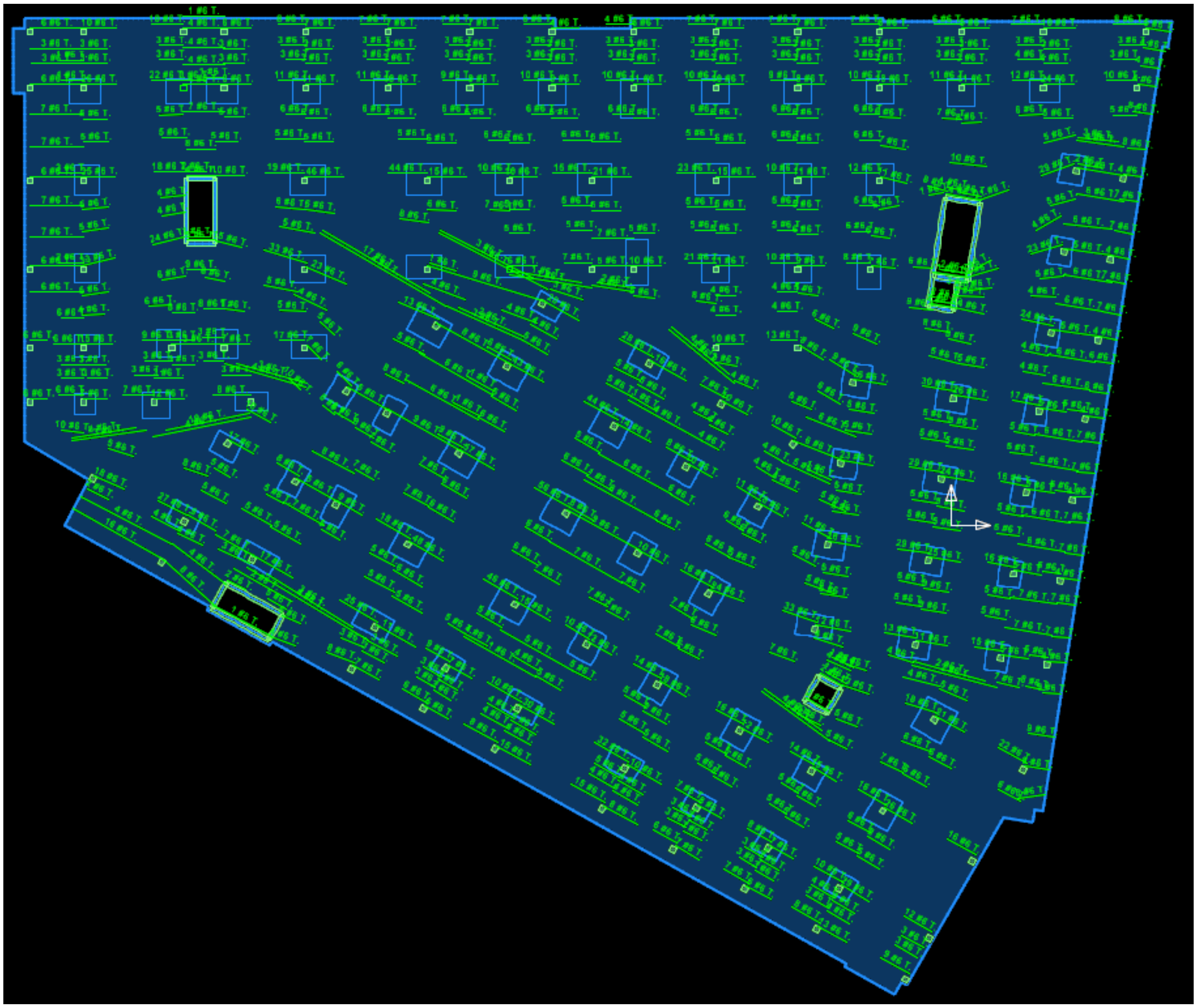
Top Latitude Bars (Typical Floors Midlevel to Second)



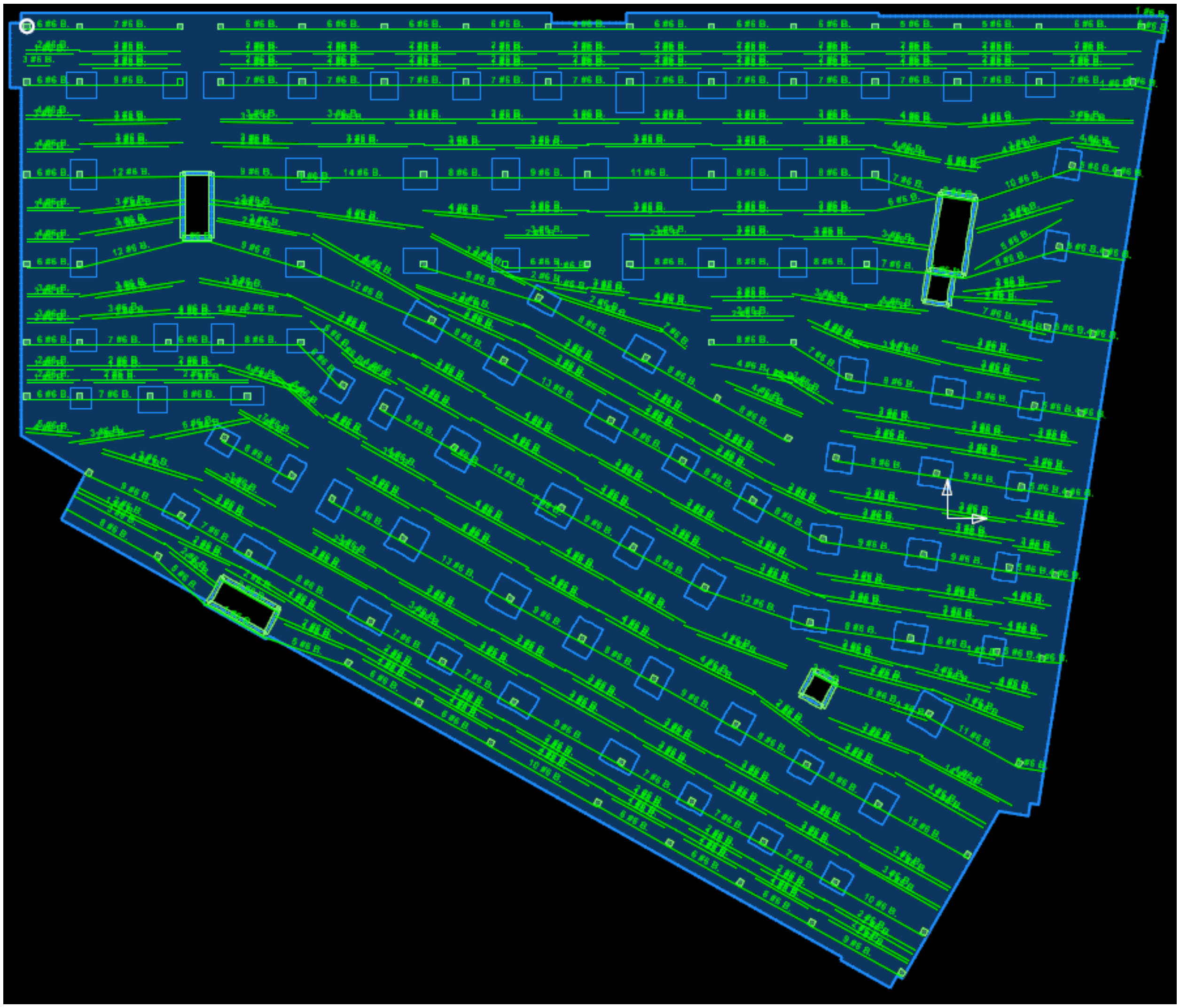
Bottom Latitude Bars (Typical Floors Midlevel to Second)



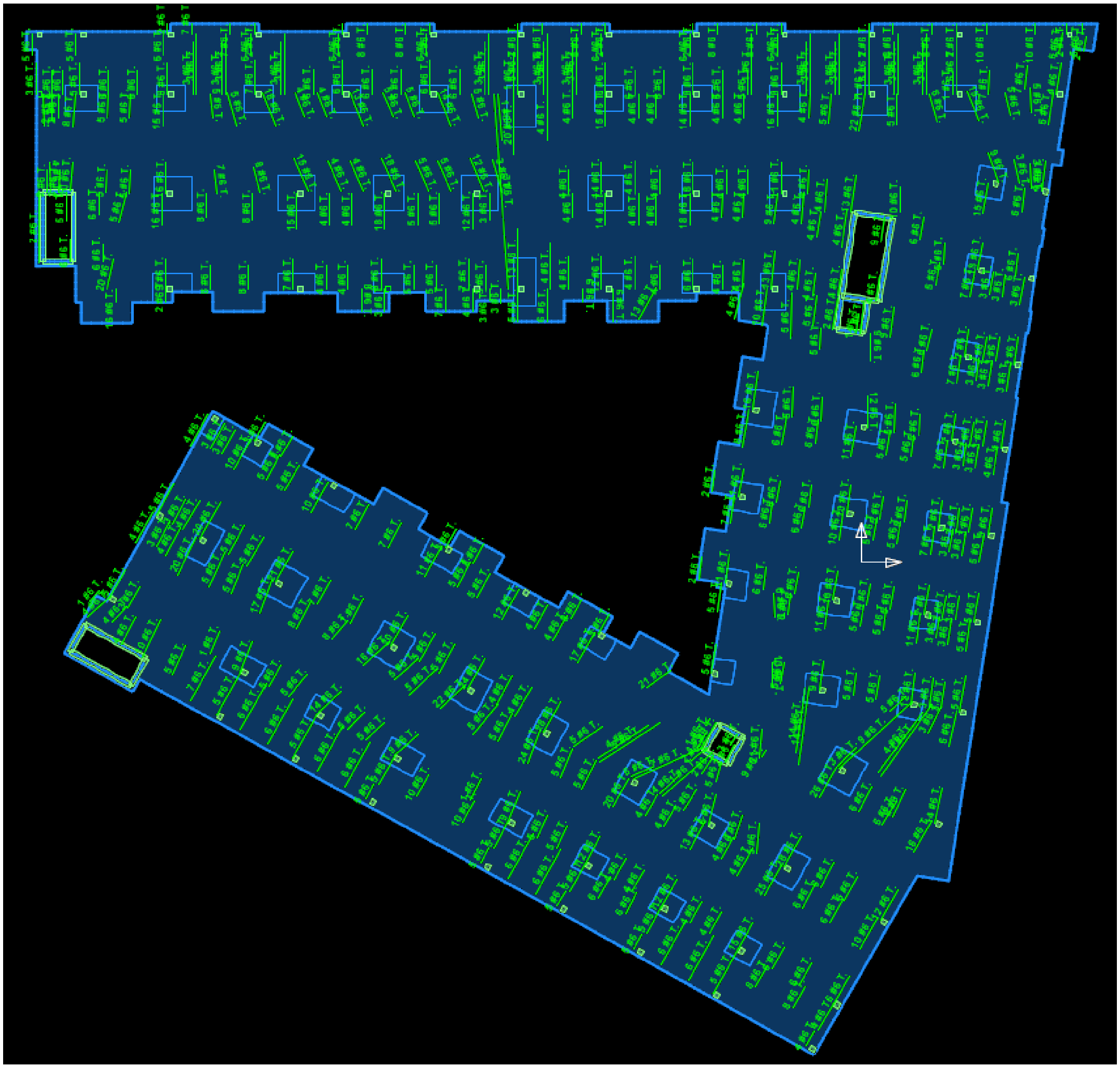
Top Longitude Bars (Typical Floors Midlevel to Second)



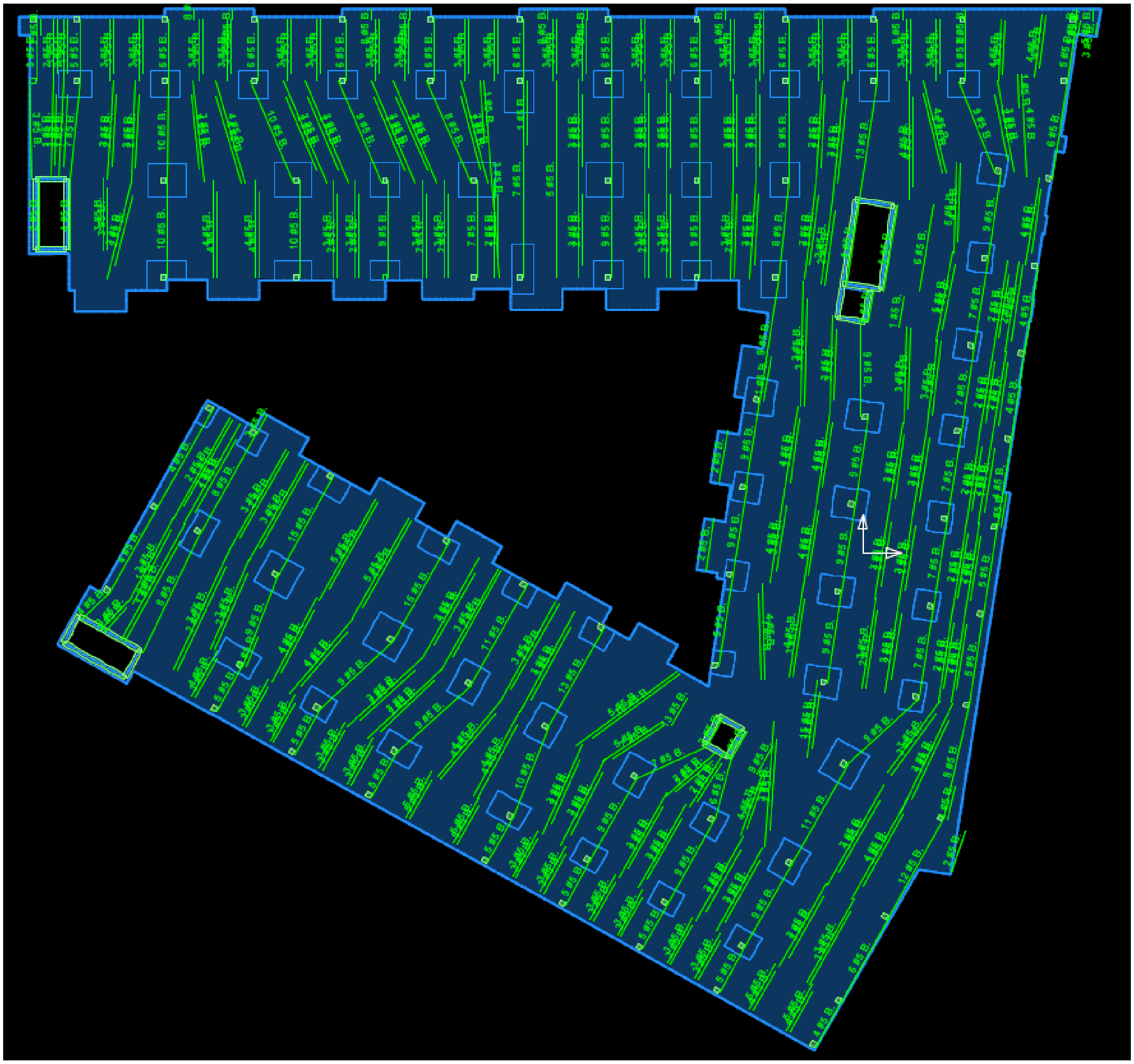
Bottom Longitude Bars (Typical Floors Midlevel to Second)



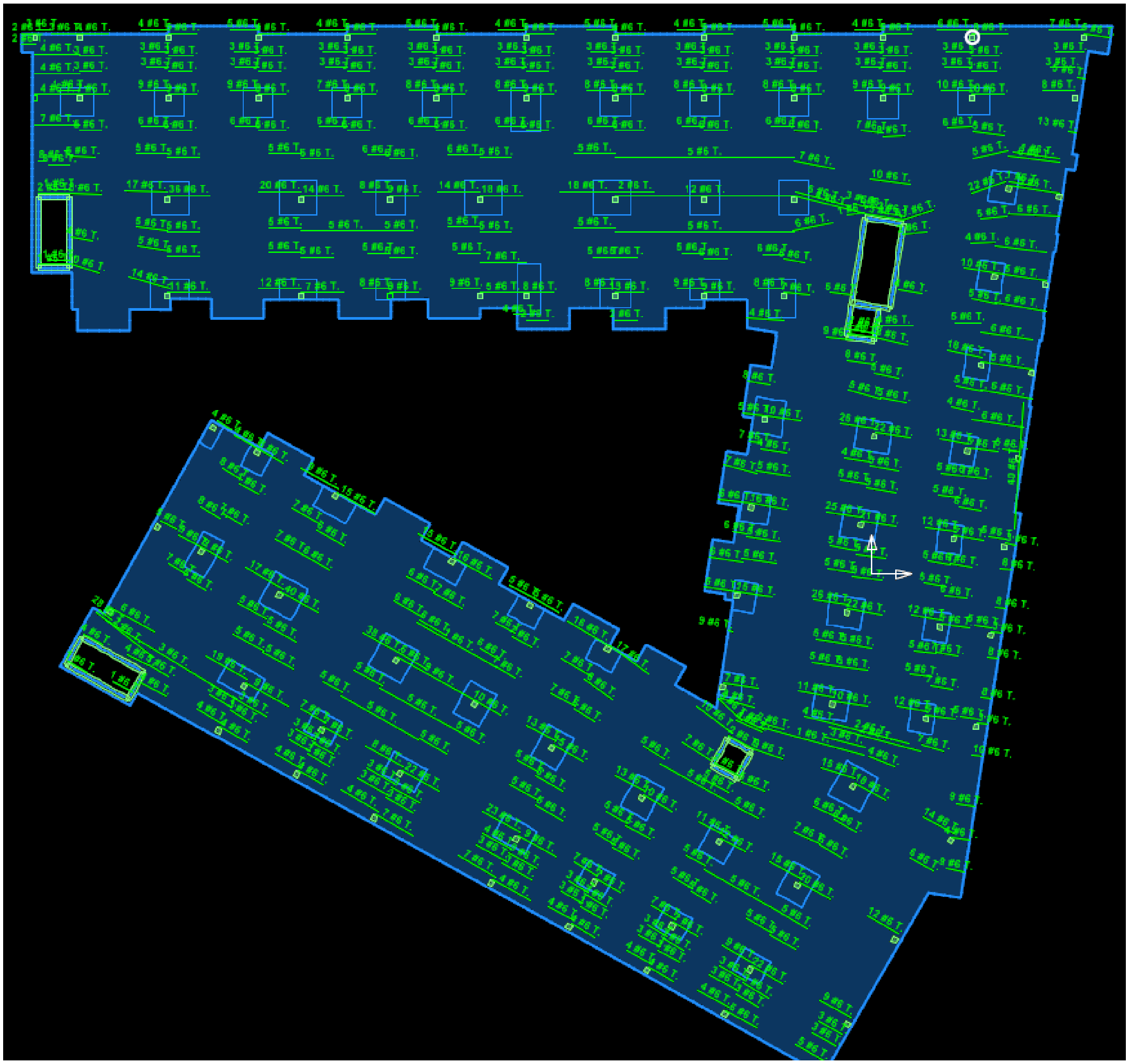
Top Latitude Bars (Typical Floors Third to Fifth)



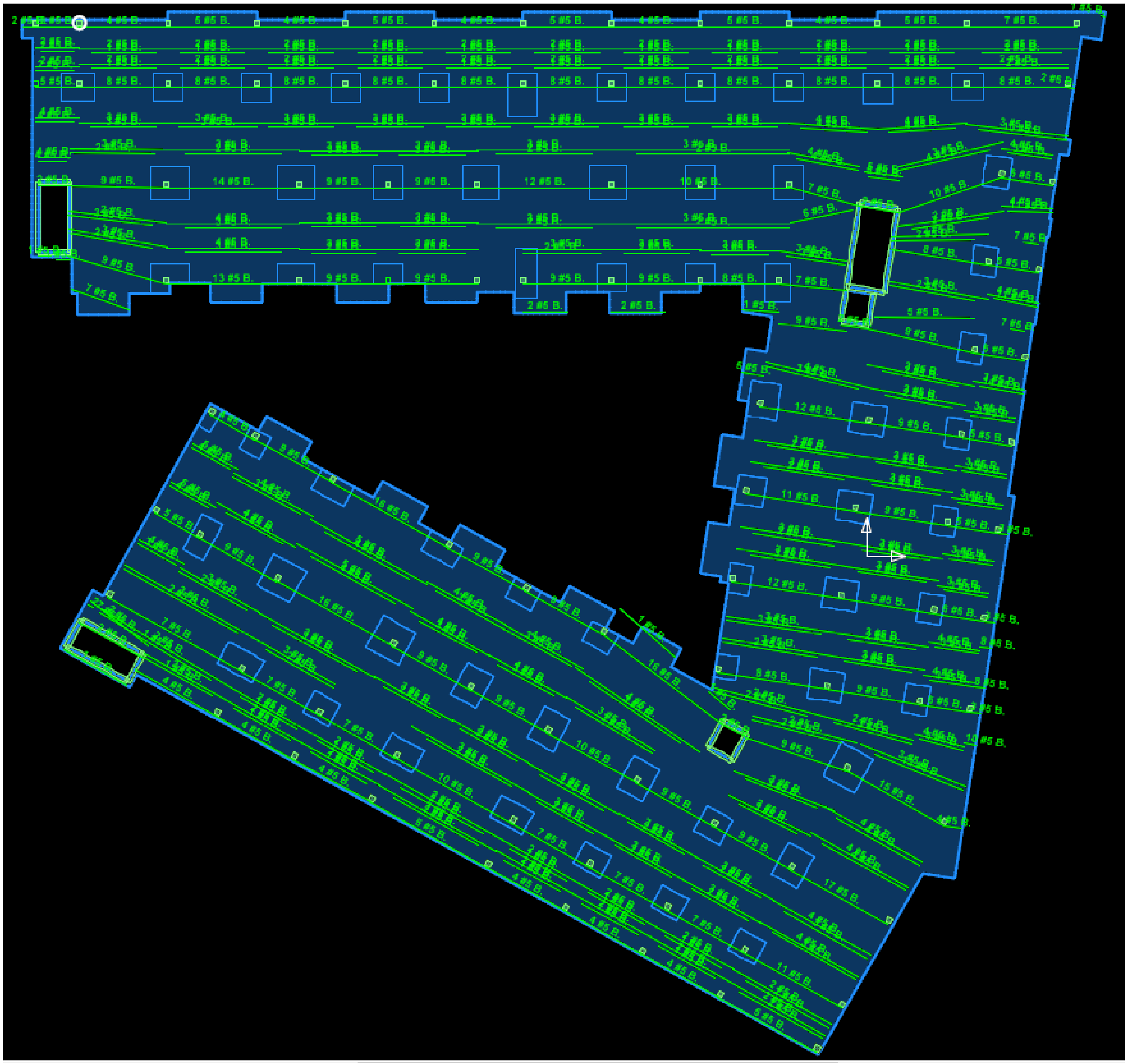
Bottom Latitude Bars (Typical Floors Third to Fifth)



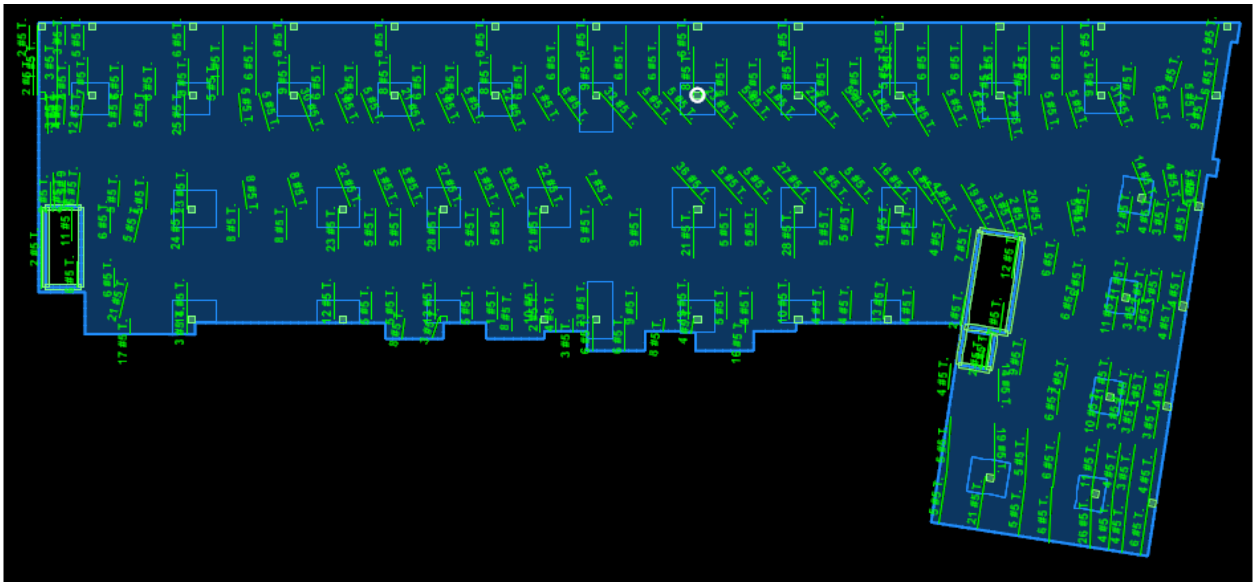
Top Longitude Bars (Typical Floors Third to Fifth)



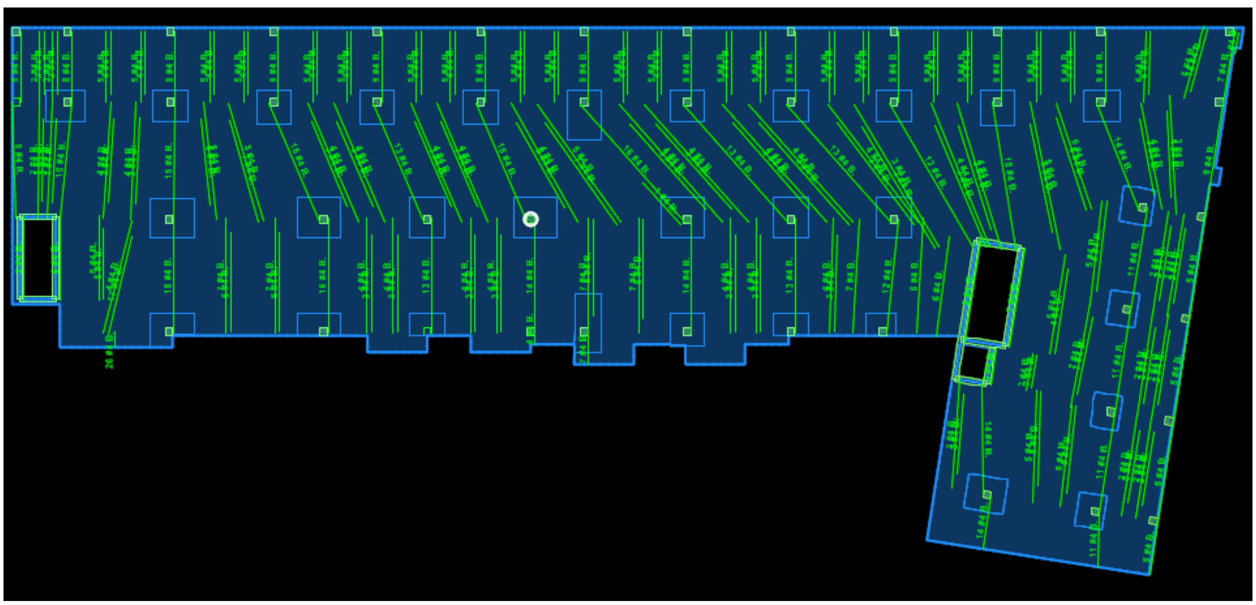
Bottom Longitude Bars (Typical Floors Third to Fifth)



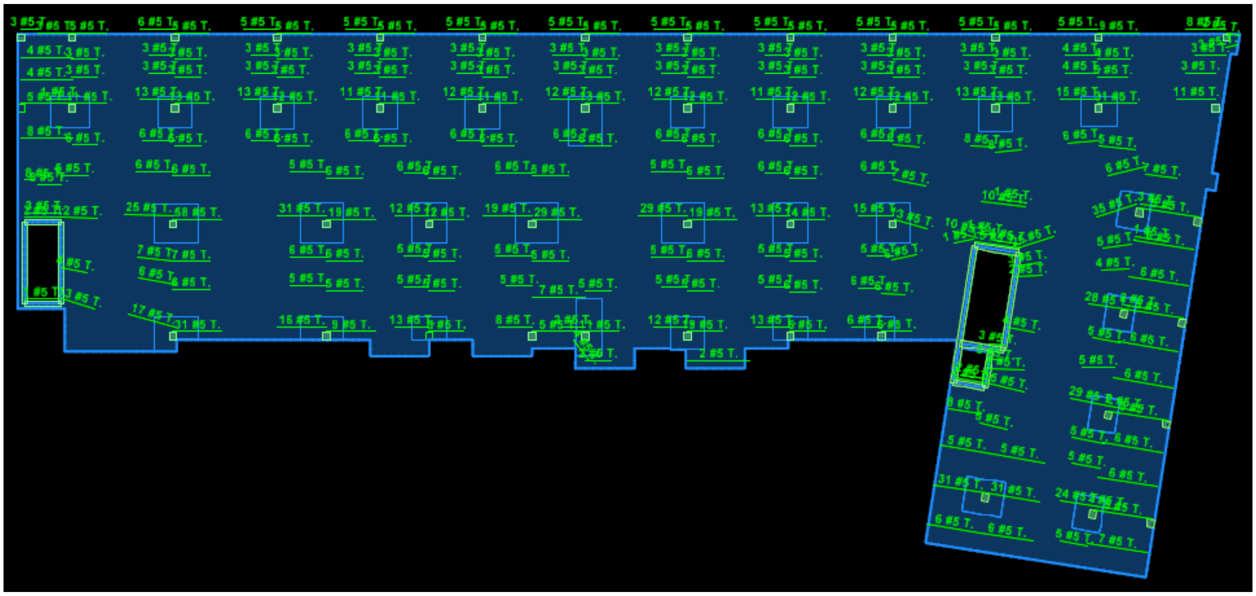
Top Latitude Bars (Sixth Floor)



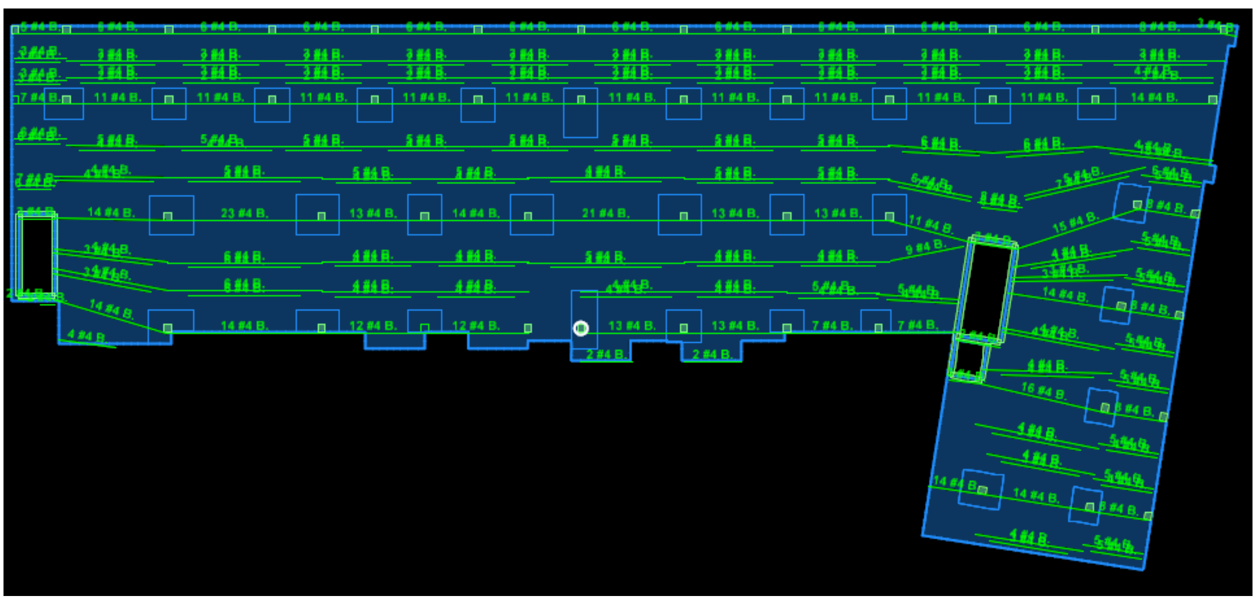
Bottom Latitude Bars (Sixth Floor)



Top Longitude Bars (Sixth Floor)



Bottom Longitude Bars (Sixth Floor)



A2 WIND LOADS

Wind Pressure Determination (Normal to W2)									
Level	**Height "z" (ft)**	**K_z**	**q_z (psf)**	**C_p**	**q_zGC_p**	**q_zGC_p - $q_h(-GC_{pi})$**	**q_zGC_p- $q_h(+GC_{pi})$**	**P (psf)**	**Notes**
Level 6	68.71	0.89	25.55	0.8	17.37	21.97	12.77	21.97	W2
				-0.2	-4.34	0.26	-8.94	-8.94	W8
				-0.7	-15.20	-10.60	-19.80	-19.80	W1, W9
Level 5	59.58	0.85	24.53	0.8	16.68	21.28	12.08	21.28	W2, W6
				-0.5	-10.86	-6.26	-15.46	-15.46	W8
				-0.454	-9.86	-5.26	-14.46	-14.46	W4
				-0.7	-15.20	-10.60	-19.80	-19.80	W1, W3, W5, W7
Level 4	48.94	0.81	23.19	0.8	15.77	20.37	11.17	20.37	W2, W6
				-0.5	-10.86	-6.26	-15.46	-15.46	W8
				-0.454	-9.86	-5.26	-14.46	-14.46	W4
				-0.7	-15.20	-10.60	-19.80	-19.80	W1, W3, W5, W7
Level 3	38.29	0.75	21.62	0.8	14.70	19.30	10.10	19.30	W2, W6
				-0.5	-10.86	-6.26	-15.46	-15.46	W8
				-0.454	-9.86	-5.26	-14.46	-14.46	W4
				-0.7	-15.20	-10.60	-19.80	-19.80	W1, W3, W5, W7
Level 2	27.65	0.68	19.70	0.8	13.39	17.99	8.80	17.99	W2, W6
				-0.5	-10.86	-6.26	-15.46	-15.46	W8
				-0.454	-9.86	-5.26	-14.46	-14.46	W4
				-0.7	-15.20	-10.60	-19.80	-19.80	W1, W3, W5, W7
Level 1	17	0.60	17.14	0.8	11.66	16.25	7.06	16.25	W10
				-0.5	-10.86	-6.26	-15.46	-15.46	W12
				-0.7	-15.20	-10.60	-19.80	-19.80	W11, W13

Table X: Wind Pressure Determination (Normal to W2)

Wind Pressure Determination (Normal to W4)									
Level	**Height "z" (ft)**	**K_z**	**q_z (psf)**	**C_p**	**q_zGC_p**	**q_zGC_p - $q_h(-GC_{pi})$**	**q_zGC_p- $q_h(+GC_{pi})$**	**P (psf)**	**Notes**
Level 6	68.71	0.89	25.55	0.8	17.37	21.97	12.77	21.97	W8
				-0.2	-4.34	0.26	-8.94	-8.94	W2
				-0.7	-15.20	-10.60	-19.80	-19.80	W1, W9
Level 5	59.58	0.85	24.53	0.8	16.68	21.28	12.08	21.28	W4, W8
				-0.454	-9.86	-5.26	-14.46	-14.46	W6
				-0.5	-10.86	-6.26	-15.46	-15.46	W2
				-0.7	-15.20	-10.60	-19.80	-19.80	W1, W3, W5, W7
Level 4	48.94	0.81	23.19	0.8	15.77	20.37	11.17	20.37	W4, W8
				-0.454	-9.86	-5.26	-14.46	-14.46	W6
				-0.5	-10.86	-6.26	-15.46	-15.46	W2
				-0.7	-15.20	-10.60	-19.80	-19.80	W1, W3, W5, W7
Level 3	38.29	0.75	21.62	0.8	14.70	19.30	10.10	19.30	W4, W8
				-0.454	-9.86	-5.26	-14.46	-14.46	W6
				-0.5	-10.86	-6.26	-15.46	-15.46	W2
				-0.7	-15.20	-10.60	-19.80	-19.80	W1, W3, W5, W7
Level 2	27.65	0.68	19.70	0.8	13.39	17.99	8.80	17.99	W4, W8
				-0.454	-9.86	-5.26	-14.46	-14.46	W6
				-0.5	-10.86	-6.26	-15.46	-15.46	W2
				-0.7	-15.20	-10.60	-19.80	-19.80	W1, W3, W5, W7
Level 1	17	0.60	17.14	0.8	11.66	16.25	7.06	16.25	W12
				-0.5	-10.86	-6.26	-15.46	-15.46	W10
				-0.7	-15.20	-10.60	-19.80	-19.80	W11, W13

Table X: Wind Pressure Determination (Normal to W4)

Wind Pressure Determination (Normal to W3)									
Level	**Height "z" (ft)**	**K_z**	**q_z (psf)**	**C_p**	**q_zGC_p**	**q_zGC_p - $q_h(-GC_{pi})$**	**q_zGC_p- $q_h(+GC_{pi})$**	**P (psf)**	**Notes**
Level 6	68.71	0.89	25.55	0.8	17.37	21.97	12.77	21.97	W9
				-0.2	-4.34	0.26	-8.94	-8.94	W1
				-0.7	-15.20	-10.60	-19.80	-19.80	W2, W8
Level 5	59.58	0.85	24.53	0.8	16.68	21.28	12.08	21.28	W3
				-0.497	-10.79	-6.19	-15.39	-15.39	W1
				-0.497	-10.79	-6.19	-15.39	-15.39	W5, W7
				-0.7	-15.20	-10.60	-19.80	-19.80	W2, W4, W6, W8
Level 4	48.94	0.81	23.19	0.8	15.77	20.37	11.17	20.37	W3
				-0.497	-10.79	-6.19	-15.39	-15.39	W1
				-0.497	-10.79	-6.19	-15.39	-15.39	W5, W7
				-0.7	-15.20	-10.60	-19.80	-19.80	W2, W4, W6, W8
Level 3	38.29	0.75	21.62	0.8	14.70	19.30	10.10	19.30	W3
				-0.497	-10.79	-6.19	-15.39	-15.39	W1
				-0.497	-10.79	-6.19	-15.39	-15.39	W5, W7
				-0.7	-15.20	-10.60	-19.80	-19.80	W2, W4, W6, W8
Level 2	27.65	0.68	19.70	0.8	13.39	17.99	8.80	17.99	W3
				-0.497	-10.79	-6.19	-15.39	-15.39	W1
				-0.497	-10.79	-6.19	-15.39	-15.39	W5, W7
				-0.7	-15.20	-10.60	-19.80	-19.80	W2, W4, W6, W8
Level 1	17	0.60	17.14	0.8	11.66	16.25	7.06	16.25	W11
				-0.5	-10.86	-6.26	-15.46	-15.46	W13
				-0.7	-15.20	-10.60	-19.80	-19.80	W10, W12

Table X: Wind Pressure Determination (Normal to W3)

Wind Pressure Determination (Normal to W1 - W7 - W5)									
Level	Height "z" (ft)	K_z	q_z (psf)	C_p	q_zGC_p	q_zGC_p - $q_h(-GC_{pi})$	q_zGC_p- $q_h(+GC_{pi})$	P (psf)	Notes
Level 6	68.71	0.89	25.55	0.8	17.37	21.97	12.77	21.97	W1
				-0.2	-4.34	0.26	-8.94	-8.94	W9
				-0.7	-15.20	-10.60	-19.80	-19.80	W2, W8
Level 5	59.58	0.85	24.53	0.8	16.68	21.28	12.08	21.28	W1, W7, W5
				-0.497	-10.79	-6.19	-15.39	-15.39	W3
				-	-	-	-	-	None
				-0.7	-15.20	-10.60	-19.80	-19.80	W2, W4, W6, W8
Level 4	48.94	0.81	23.19	0.8	15.77	20.37	11.17	20.37	W1, W7, W5
				-0.497	-10.79	-6.19	-15.39	-15.39	W3
				-	-	-	-	-	None
				-0.7	-15.20	-10.60	-19.80	-19.80	W2, W4, W6, W8
Level 3	38.29	0.75	21.62	0.8	14.70	19.30	10.10	19.30	W1, W7, W5
				-0.497	-10.79	-6.19	-15.39	-15.39	W3
				-	-	-	-	-	None
				-0.7	-15.20	-10.60	-19.80	-19.80	W2, W4, W6, W8
Level 2	27.65	0.68	19.70	0.8	13.39	17.99	8.80	17.99	W1, W7, W5
				-0.497	-10.79	-6.19	-15.39	-15.39	W3
				-	-	-	-	-	None
				-0.7	-15.20	-10.60	-19.80	-19.80	W2, W4, W6, W8
Level 1	17	0.60	17.14	0.8	11.66	16.25	7.06	16.25	W13
				-0.5	-10.86	-6.26	-15.46	-15.46	W11
				-0.7	-15.20	-10.60	-19.80	-19.80	W10, W12

Table X: Wind Pressure Determination (Normal to W1-W7-W5)

Case 1 - Base Shear Determination (Normal to W4)							
Level	**Height "z" (ft)**	**Wall**	**P (psf)**	**Trib Width (ft)**	**Shear (kips)**	**North - South (kips)**	**East - West (kips)**
Level 6	68.71	W2	-8.94	282.00	-23.02	(39.79)	-
		W8	21.97	282.00	56.57		
Level 5	59.58	W2	-15.46	287.00	-47.20	(133.84)	-
		W6	-14.46	155.00	-23.84		
		W8	21.28	287.00	64.97		
		W4	21.28	230.00	52.07		
Level 4	48.94	W2	-15.46	287.00	-47.24	(185.66)	-
		W6	-14.46	155.00	-23.87		
		W8	20.37	287.00	62.25		
		W4	20.37	230.00	49.89		
Level 3	38.29	W2	-15.46	287.00	-47.20	(180.22)	-
		W6	-14.46	155.00	-23.84		
		W8	19.30	287.00	58.93		
		W4	19.30	230.00	47.23		
Level 2	27.65	W2	-15.46	287.00	-47.24	(173.69)	-
		W6	-14.46	155.00	-23.87		
		W8	17.99	287.00	54.99		
		W4	17.99	230.00	44.07		
Level 1	17	W10	-15.46	328.00	-86.18	(169.63)	-
		W12	16.25	300.00	82.90		
					Base Shear	**(882.83)**	**-**

Table X: Case 1 - Base Shear Determination (Normal to W4)

Base Shear Determination (Normal to W3)							
Level	Height "z" (ft)	Wall	P (psf)	Trib Width (ft)	Shear (kips)	North - South (kips)	East - West (kips)
Level 6	68.71	W1	21.97	71.00	14.24	-	10.02
		W9	-8.94	71.00	-5.80		
Level 5	59.58	W1	-15.39	73.17	-11.98	-	65.23
		W3	21.28	283.00	64.07		
		W5	-15.39	70.33	-11.52		
		W7	-15.39	139.50	-22.84		
Level 4	48.94	W1	-15.39	73.17	-11.99	-	109.09
		W3	20.37	283.00	61.38		
		W5	-15.39	70.33	-11.53		
		W7	-15.39	139.50	-22.87		
Level 3	38.29	W1	-15.39	73.17	-11.98	-	106.11
		W3	19.30	283.00	58.11		
		W5	-15.39	70.33	-11.52		
		W7	-15.39	139.50	-22.84		
Level 2	27.65	W1	-15.39	73.17	-11.99	-	102.53
		W3	17.99	283.00	54.23		
		W5	-15.39	70.33	-11.53		
		W7	-15.39	139.50	-22.87		
Level 1	17	W11	16.25	292.00	80.69	-	108.12
		W13	-15.46	133.00	-34.95		
					Base Shear	**-**	**501.10**

Table X: Case 1 - Base Shear Determination (Normal to W3)

Base Shear Determination (Normal to W1 - W7 - W5)							
Level	**Height "z" (ft)**	**Wall**	**P (psf)**	**Trib Width (ft)**	**Shear (kips)**	**North - South (kips)**	**East - West (kips)**
Level 6	68.71	W1	21.97	71.00	14.24	-	(10.02)
		W9	-8.94	71.00	-5.80		
Level 5	59.58	W1	21.28	73.17	16.56	-	(45.19)
		W3	-15.39	283.00	-46.34		
		W5	21.28	70.33	15.92		
		W7	21.28	139.50	31.58		
Level 4	48.94	W1	20.37	73.17	15.87	-	(109.09)
		W3	-15.39	283.00	-46.39		
		W5	20.37	70.33	15.25		
		W7	20.37	139.50	30.26		
Level 3	38.29	W1	19.30	73.17	15.02	-	(106.11)
		W3	-15.39	283.00	-46.34		
		W5	19.30	70.33	14.44		
		W7	19.30	139.50	28.64		
Level 2	27.65	W1	17.99	73.17	14.02	-	(102.53)
		W3	-15.39	283.00	-46.39		
		W5	17.99	70.33	13.48		
		W7	17.99	139.50	26.73		
Level 1	17	W11	-15.46	292.00	-76.72	-	(107.05)
		W13	16.25	133.00	36.75		
					Base Shear	**-**	**(479.99)**

Table X: Case 1 - Base Shear Determination (Normal to W1-W7-W5)

Case 2 - Base Shear Determination (Normal to W2)									
Level	**Height "z" (ft)**	**Wall**	**P (psf)**	**Trib Width (ft)**	**Shear (kips)**	**North - South (kips)**	**East - West (kips)**	**Eccentricity, $e_x = 0.15B$**	**Moment (kips - ft)**
Level 6	68.71	W2	21.97	282.00	56.57	39.79	-	-	-
		W8	-8.94	282.00	-23.02				
Level 5	59.58	W2	21.28	287.00	48.73	108.29	-	43.05	5,897.09
		W6	21.28	155.00	26.32				
		W8	-15.46	287.00	-35.40				
		W4	-14.46	230.00	-26.53				
Level 4	48.94	W2	20.37	287.00	46.69	135.44	-	43.05	5,764.08
		W6	20.37	155.00	25.21				
		W8	-15.46	287.00	-35.43				
		W4	-14.46	230.00	-26.56				
Level 3	38.29	W2	19.30	287.00	44.20	131.95	-	43.05	5,596.54
		W6	19.30	155.00	23.87				
		W8	-15.46	287.00	-35.40				
		W4	-14.46	230.00	-26.53				
Level 2	27.65	W2	17.99	287.00	41.25	127.76	-	43.05	5,403.36
		W6	17.99	155.00	22.28				
		W8	-15.46	287.00	-35.43				
		W4	-14.46	230.00	-26.56				
Level 1	17	W10	16.25	328.00	90.64	147.49	-	-	-
		W12	-15.46	300.00	-78.83				
					Base Shear	**690.71**	**-**	**-**	**-**

Table X: Case 2 - Base Shear Determination (Normal to W2)

Case 2 - Base Shear Determination (Normal to W4)									
Level	**Height "z" (ft)**	**Wall**	**P (psf)**	**Trib Width (ft)**	**Shear (kips)**	**North - South (kips)**	**East - West (kips)**	**Eccentricity, $e_x = 0.15B$**	**Moment (kips - ft)**
Level 6	68.71	W2	-8.94	282.00	-23.02	39.79	-	-	-
		W8	21.97	282.00	56.57				
Level 5	59.58	W2	-15.46	287.00	-47.20	133.84	-	43.05	8,097.08
		W6	-14.46	155.00	-23.84				
		W8	21.28	287.00	64.97				
		W4	21.28	230.00	52.07				
Level 4	48.94	W2	-15.46	287.00	-47.24	185.66	-	43.05	7,888.60
		W6	-14.46	155.00	-23.87				
		W8	20.37	287.00	62.25				
		W4	20.37	230.00	49.89				
Level 3	38.29	W2	-15.46	287.00	-47.20	180.22	-	43.05	7,628.36
		W6	-14.46	155.00	-23.84				
		W8	19.30	287.00	58.93				
		W4	19.30	230.00	47.23				
Level 2	27.65	W2	-15.46	287.00	-47.24	173.69	-	43.05	7,326.04
		W6	-14.46	155.00	-23.87				
		W8	17.99	287.00	54.99				
		W4	17.99	230.00	44.07				
Level 1	17	W10	-15.46	328.00	-86.18	169.63	-	-	-
		W12	16.25	300.00	82.90				
					Base Shear	**882.83**	**-**	**-**	**-**

Table X: Case 2 - Base Shear Determination (Normal to W4)

Case 2 - Base Shear Determination (Normal to W3)									
Level	**Height "z" (ft)**	**Wall**	**P (psf)**	**Trib Width (ft)**	**Shear (kips)**	**North - South (kips)**	**East - West (kips)**	**Eccentricity, e_x = 0.15B**	**Moment (kips - ft)**
Level 6	68.71	W1	21.97	71.00	14.24	-	10.02	-	-
		W9	-8.94	71.00	-5.80				
Level 5	59.58	W1	-15.39	73.17	-8.99	-	51.42	42.45	2,752.31
		W3	21.28	283.00	48.05				
		W5	-15.39	70.33	-8.64				
		W7	-15.39	139.50	-17.13				
Level 4	48.94	W1	-15.39	73.17	-8.99	-	81.82	42.45	2,667.42
		W3	20.37	283.00	46.04				
		W5	-15.39	70.33	-8.65				
		W7	-15.39	139.50	-17.15				
Level 3	38.29	W1	-15.39	73.17	-8.99	-	79.58	42.45	2,562.56
		W3	19.30	283.00	43.58				
		W5	-15.39	70.33	-8.64				
		W7	-15.39	139.50	-17.13				
Level 2	27.65	W1	-15.39	73.17	-8.99	-	76.90	42.45	2,439.69
		W3	17.99	283.00	40.67				
		W5	-15.39	70.33	-8.65				
		W7	-15.39	139.50	-17.15				
Level 1	17	W11	16.25	292.00	80.69	-	95.55	-	-
		W13	-15.46	133.00	-34.95				
					Base Shear	**-**	**395.29**	**-**	**-**

Table X: Case 2 - Base Shear Determination (Normal to W3)

Case 2 - Base Shear Determination (Normal to W1 - W7 - W5)									
Level	**Height "z" (ft)**	**Wall**	**P (psf)**	**Trib Width (ft)**	**Shear (kips)**	**North - South (kips)**	**East - West (kips)**	**Eccentricity, e_x = 0.15B**	**Moment (kips - ft)**
Level 6	68.71	W1	21.97	71.00	14.24	-	(10.02)	-	-
		W9	-8.94	71.00	-5.80				
Level 5	59.58	W1	21.28	73.17	16.56	-	(45.19)	42.45	(3,280.69)
		W3	-15.39	283.00	-46.34				
		W5	21.28	70.33	15.92				
		W7	21.28	139.50	31.58				
Level 4	48.94	W1	20.37	73.17	15.87	-	(109.09)	42.45	(3,227.45)
		W3	-15.39	283.00	-46.39				
		W5	20.37	70.33	15.25				
		W7	20.37	139.50	30.26				
Level 3	38.29	W1	19.30	73.17	15.02	-	(106.11)	42.45	(3,158.51)
		W3	-15.39	283.00	-46.34				
		W5	19.30	70.33	14.44				
		W7	19.30	139.50	28.64				
Level 2	27.65	W1	17.99	73.17	14.02	-	(102.53)	42.45	(3,080.81)
		W3	-15.39	283.00	-46.39				
		W5	17.99	70.33	13.48				
		W7	17.99	139.50	26.73				
Level 1	17	W11	-15.46	292.00	-76.72	-	(107.05)	-	-
		W13	16.25	133.00	36.75				
					Base Shear	**-**	**(479.99)**	**-**	**-**

Table X: Case 2 - Base Shear Determination (Normal to W1-W7-W5)

Case 3 - Base Shear Determination (Normal to W2)							
Level	**Height "z" (ft)**	**Wall**	**P (psf)**	**Trib Width (ft)**	**Shear (kips)**	**North - South (kips)**	**East - West (kips)**
Level 6	68.71	W2	21.97	282.00	56.57	39.79	-
		W8	-8.94	282.00	-23.02		
Level 5	59.58	W2	21.28	287.00	48.73	108.29	-
		W6	21.28	155.00	26.32		
		W8	-15.46	287.00	-35.40		
		W4	-14.46	230.00	-26.53		
Level 4	48.94	W2	20.37	287.00	46.69	135.44	-
		W6	20.37	155.00	25.21		
		W8	-15.46	287.00	-35.43		
		W4	-14.46	230.00	-26.56		
Level 3	38.29	W2	19.30	287.00	44.20	131.95	-
		W6	19.30	155.00	23.87		
		W8	-15.46	287.00	-35.40		
		W4	-14.46	230.00	-26.53		
Level 2	27.65	W2	17.99	287.00	41.25	127.76	-
		W6	17.99	155.00	22.28		
		W8	-15.46	287.00	-35.43		
		W4	-14.46	230.00	-26.56		
Level 1	17	W10	16.25	328.00	90.64	147.49	-
		W12	-15.46	300.00	-78.83		
					Base Shear	**690.71**	**-**

Table X: Case 3 - Base Shear Determination (Normal to W2)

Case 3 - Base Shear Determination (Normal to W3)							
Level	**Height "z" (ft)**	**Wall**	**P (psf)**	**Trib Width (ft)**	**Shear (kips)**	**North - South (kips)**	**East - West (kips)**
Level 6	68.71	W1	21.97	71.00	14.24	-	10.02
		W9	-8.94	71.00	-5.80		
Level 5	59.58	W1	-15.39	73.17	-8.99	-	51.42
		W3	21.28	283.00	48.05		
		W5	-15.39	70.33	-8.64		
		W7	-15.39	139.50	-17.13		
Level 4	48.94	W1	-15.39	73.17	-8.99	-	81.82
		W3	20.37	283.00	46.04		
		W5	-15.39	70.33	-8.65		
		W7	-15.39	139.50	-17.15		
Level 3	38.29	W1	-15.39	73.17	-8.99	-	79.58
		W3	19.30	283.00	43.58		
		W5	-15.39	70.33	-8.64		
		W7	-15.39	139.50	-17.13		
Level 2	27.65	W1	-15.39	73.17	-8.99	-	76.90
		W3	17.99	283.00	40.67		
		W5	-15.39	70.33	-8.65		
		W7	-15.39	139.50	-17.15		
Level 1	17	W11	16.25	292.00	80.69	-	95.55
		W13	-15.46	133.00	-34.95		
					Base Shear	**-**	**395.29**

Table X: Case 3 - Base Shear Determination (Normal to W3)

Level	Height "z" (ft)	Wall	0.563*Shear (kips)	North - South (kips)	Eccentricity, e_x = 0.15B	0.563*Shear (kips)	East - West (kips)	Eccentricity, e_x = 0.15B	Moment (kips - ft)
Level 6	68.71	W2	56.57	39.79	43.05	-50.98	-	-	-
		W8	-23.02			-50.98			
		W1	-12.83	-	-	14.24	10.02	42.45	-
		W9	-12.83			-5.80			
Level 5	59.58	W2	36.58	91.21	43.05	-60.46	-	-	7,065.53
		W6	19.76			-32.65			
		W8	-26.57			-60.46			
		W4	-19.92			-48.45			
		W1	-15.41	-	-	-6.75	41.10	42.45	
		W3	-59.62			36.07			
		W5	-14.82			-6.48			
		W7	-29.39			-12.86			
Level 4	48.94	W2	35.05	101.67	43.05	-60.52	-	-	6,902.50
		W6	18.93			-32.68			
		W8	-26.60			-60.52			
		W4	-19.94			-48.50			
		W1	-15.43	-	-	-6.75	61.42	42.45	
		W3	-59.67			34.56			
		W5	-14.83			-6.49			
		W7	-29.42			-12.87			
Level 3	38.29	W2	33.18	99.05	43.05	-60.46	-	-	6,697.49
		W6	17.92			-32.65			
		W8	-26.57			-60.46			
		W4	-19.92			-48.45			
		W1	-15.41	-	-	-6.75			

		W3	-59.62			32.72	59.74	42.45	
		W5	-14.82			-6.48			
		W7	-29.39			-12.86			
Level 2	27.65	W2	30.96	95.90	43.05	-60.52	-	-	6,460.77
		W6	16.72			-32.68			
		W8	-26.60			-60.52			
		W4	-19.94			-48.50			
		W1	-15.43	-	-	-6.75	57.73	42.45	
		W3	-59.67			30.53			
		W5	-14.83			-6.49			
		W7	-29.42			-12.87			
Level 1	17	W10	90.64	131.84	43.05	-110.40	-	-	-
		W12	-78.83			-100.98			
		W11	-98.28	-	-	80.69	86.14	42.45	-
		W13	- 44.76614181			-34.95			
			Base Shear	**559.46**		**Base Shear**	**316.14**		

Table X: Case 4 - Base Shear Determination

A3 LATERAL SYSTEM

Shubham Kapadiya | Lateral Force Distribution

* Stiffness of Shear Wall:

$$K = \frac{t}{\left[\frac{H^3}{EL^3} + \frac{1.2H}{GL}\right]}$$

H = Height of shear wall (in)

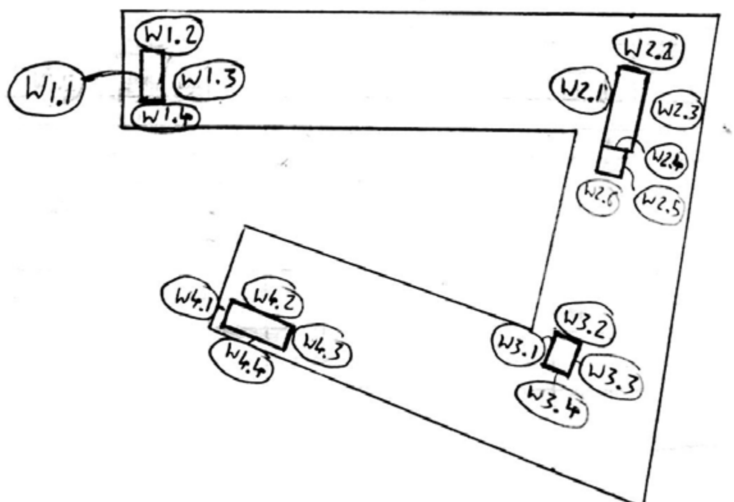
L = Length of shear wall (in)

$E = 57{,}000\sqrt{f'_c}$

$= 57{,}000\sqrt{4000} = 3604{,}996.5 \text{ lb/in}^2$

$G = \frac{E}{2(1+\nu)} = \frac{3604{,}996.5}{2(1+0.2)} = 1502{,}081.9 \text{ lbs/in}^2$

t = Thickness of wall (in) = 8 in



→ Wall 1.1 : $K = \frac{8}{\left[\frac{(132)^3}{3605\cdot(235)^3} + \frac{1.2(132)}{1502\cdot(235)}\right]} = 16{,}067.5 \text{ kip/in}$

→ Calculating K_x & K_y for diagonal shear walls:

$$K_x = K\left(\frac{L_x}{L}\right)^2$$

$$K_y = K\left(\frac{L_y}{L}\right)^2$$

where, K = stiffness of shear wall (kip/in)

L = length of shear wall (in)

$L_x = L\sin(\theta)$

$L_y = L\cos(\theta)$

where, $\theta = 8.88°$ for W2.1 – W2.6

$\theta = 29.283°$ for W3.1 – W4.4

⇒ Wall 2.1: $K = 31196.73$ kip/in

$L = 425$ in

$\theta = 8.88°$

$L_x = 425\sin(8.88°)$
$= 65.61$ in

$L_y = 425\cos(8.88°)$
$= 419.91$ in

$K_x = 31196.73\left(\frac{65.61}{425}\right)^2$
$= 743.5$ kip/in

$K_y = 31196.73\left(\frac{419.91}{425}\right)^2$
$= 30{,}453.95$ kip/in

* COR:

$$X_r = \frac{\Sigma k_{iy} x_i}{\Sigma k_{iy}}$$

$$Y_r = \frac{\Sigma k_{ix} y_i}{\Sigma K_{ix}}$$

where, K_i = Stiffness of shear wall (kip/in)
X_i = Perpendicular x-direction distance from origin to SW (ft)
y_i = " y-direction " " " " "

⇒ Mid-level : $X_r = 162.31$ ft
$Y_r = -21.689$ ft

* Direct Shear:

$$V_{D_i} = \frac{K_i}{\Sigma K_i} V_{story}$$

* Torsional Shear:

$$V_{T_i} = \frac{K_i d_i}{J} V_{story}\, e$$

where ; K_i = Stiffness of shear wall (kip/ft)
d_i = Perpendicular distance to the origin (ft)
V_{story} = Shear at story (kips)
$e_x = |COM_x - COR_x|$ (ft)
$e_y = |COM_y - COR_y|$ (ft)
$J = \Sigma K_i d_i^2$ (kip-ft)

or center of Facade for wind loads

Wall name	Story	Height (in)	Length (in)	K (kip/in)	X - direction Length	Y - direction Length	K_x	K_y	X_i (ft)	Y_i (ft)	V_{Dx}	V_{Dy}	$K_i d_i^2$ (kip-ft)	V_{Tx}	V_{Ty}
W1.1	Roof	109.56	235	19972	0	235	0	19972	30	0	0	6	499269652	24	0
	Sixth	127.68	235	16717	0	235	0	16717	30	0	0	13	410721898	31	0
	Firth	127.8	235	16699	0	235	0	16699	30	0	0	19	404092972	31	0
	Fourth	127.68	235	16717	0	235	0	16717	30	0	0	18	395909880	27	0
	Third	127.8	235	16699	0	235	0	16699	30	0	0	17	383128459	22	0
	Second	204	235	9143	0	235	0	9143	30	0	0	18	199810392	29	0
W1.2	Roof	109.56	115	7992	115	0	7992	0	0	55	21	0	90311493	0	28
	Sixth	127.68	115	6316	115	0	6316	0	0	55	21	0	73291394	0	6
	Firth	127.8	115	6307	115	0	6307	0	0	55	22	0	72952834	0	9
	Fourth	127.68	115	6316	115	0	6316	0	0	55	22	0	72445962	0	8
	Third	127.8	115	6307	115	0	6307	0	0	55	21	0	71339379	0	7
	Second	204	115	2698	115	0	2698	0	0	55	23	0	29917517	0	5
W1.3	Roof	109.56	235	19972	0	235	0	19972	39	0	0	6	441074905	22	0
	Sixth	127.68	235	16717	0	235	0	16717	39	0	0	13	362444076	29	0
	Firth	127.8	235	16699	0	235	0	16699	39	0	0	19	356244379	29	0
	Fourth	127.68	235	16717	0	235	0	16717	39	0	0	18	348538038	25	0
	Third	127.8	235	16699	0	235	0	16699	39	0	0	17	336577214	21	0
	Second	204	235	9143	0	235	0	9143	39	0	0	18	174954561	27	0
W1.4	Roof	109.56	115	7992	115	0	7992	0	0	36	21	0	60101556	0	23
	Sixth	127.68	115	6316	115	0	6316	0	0	36	21	0	49065951	0	5
	Firth	127.8	115	6307	115	0	6307	0	0	36	22	0	48805344	0	7
	Fourth	127.68	115	6316	115	0	6316	0	0	36	22	0	48374658	0	7
	Third	127.8	115	6307	115	0	6307	0	0	36	21	0	47487303	0	6
	Second	204	115	2698	115	0	2698	0	0	36	23	0	19825468	0	4

A4 CONSTRUCTION BREADTH

Existing Steel & Wood structure Cost Estimate:

Steel						
Level	Beam (lbs)	Column (lbs)	Brace (lbs)	Total tons	Cost Per Ton	Total Cost
Second	759457	51800	2590	406.9235	$ 4,100.00	$ 1,668,386.35
First	409047	51800	2590	231.7185	$ 4,100.00	$ 950,045.85
Midlevel	261215	51800	2590	157.8025	$ 4,100.00	$ 646,990.25
					(05 12 23.77.0700)	$ 3,265,422.45

Wood					
Level	Wood Shear Walls (ft)	Cost of Wood Walls	Wood Joists (thousand ft)	Cost of Wood Joists	Total Cost
Roof	710.75	$ 26.50	15	$ 5,200.00	$ 96,834.88
Sixth	5125	$ 26.50	35	$ 5,200.00	$ 317,812.50
Fifth	5125	$ 26.50	35	$ 5,200.00	$ 317,812.50
Fourth	5125	$ 26.50	35	$ 5,200.00	$ 317,812.50
Third	5125	$ 26.50	35	$ 5,200.00	$ 317,812.50
		(06 11 10.26.1180)		(06 11 10.18.4030)	$ 1,368,084.88

Level	Gross Floor Area	Cost Per SF	Total Cost
Roof	17552	$ 1.79	$ 31,418.08
Sixth	44096	$ 1.79	$ 78,931.84
Fifth	44096	$ 1.79	$ 78,931.84
Fourth	44096	$ 1.79	$ 78,931.84
Third	44096	$ 1.79	$ 78,931.84
			$ 347,145.44

Level	Total Cost	Adjusted Cost
Roof	$ 128,252.96	$ 152,634.55
Sixth	$ 396,744.34	$ 472,167.62
Fifth	$ 396,744.34	$ 472,167.62
Fourth	$ 396,744.34	$ 472,167.62
Third	$ 396,744.34	$ 472,167.62
Second	$ 2,178,657.10	$ 2,592,831.79
First	$ 1,460,316.60	$ 1,737,930.81
Midlevel	$ 1,161,600.50	$ 1,382,427.14
	$ 6,515,804.52	$ 7,754,494.77

Redesigned Two-Way flat plate slab Cost Estimate:

				Concrete Slab Costs		
	Concrete Cost			Formwork Cost		
Level	Concrete (yd^3)	Material Cost ($ per yd^3)	Labor ($ per yd^3)	Formwork (ft^2)	Material Cost ($ per ft^2)	Labor ($ per ft^2)
Roof	720.70	$ 116.00	$ 23.50	22,500.00	$ 1.55	$ 3.43
Sixth	1,511.00	$ 116.00	$ 23.50	47,230.00	$ 1.55	$ 3.43
Fifth	1,511.00	$ 116.00	$ 23.50	47,230.00	$ 1.55	$ 3.43
Fourth	1,511.00	$ 116.00	$ 23.50	47,230.00	$ 1.55	$ 3.43
Third	1,511.00	$ 116.00	$ 23.50	47,230.00	$ 1.55	$ 3.43
Second	2,195.00	$ 116.00	$ 23.50	68,650.00	$ 1.55	$ 3.43
First	2,195.00	$ 116.00	$ 23.50	68,650.00	$ 1.55	$ 3.43
Midlevel	2,195.00	$ 116.00	$ 23.50	68,650.00	$ 1.55	$ 3.43
		$ 1,548,565.20	$ 313,717.95		$ 646,923.50	$ 1,431,579.10
		(03 31 05.35.0300)	(03 31 05.70.1500)		(03 11 13.35.2150)	(03 11 13.35.2050)

Reinforcing Cost			
Reinforcing (tons)	Material Cost ($ per tons)	Labor ($ per tons)	Total
43.16	$ 1,650.00	$ 490.00	$ 304,950.05
95.16	$ 1,650.00	$ 490.00	$ 649,632.30
95.16	$ 1,650.00	$ 490.00	$ 649,632.30
95.16	$ 1,650.00	$ 490.00	$ 649,632.30
95.16	$ 1,650.00	$ 490.00	$ 649,632.30
150.50	$ 1,650.00	$ 490.00	$ 970,149.50
150.50	$ 1,650.00	$ 490.00	$ 970,149.50
150.50	$ 1,650.00	$ 490.00	$ 970,149.50
	$ 1,444,245.00	$ 428,897.00	$ 5,813,927.75
	(03 21 10.60.0400)	(03 21 10.60.0400)	

Redesigned Two-Way flat plate slab Cost Estimate (Continued):

Concrete Column Costs						
	Concrete Cost			Formwork Cost		
Level	Concrete (yd^3)	Material ($ per yd^3)	Labor ($ per yd^3)	Formwork (ft^2)	Material ($ per ft^2)	Labor ($ per ft^2)
Roof	46.00	$ 139.00	$ 46.50	136.00	$ 1.67	$ 2.75
Sixth	106.20	$ 139.00	$ 46.50	270.00	$ 1.67	$ 2.75
Fifth	106.20	$ 139.00	$ 46.50	270.00	$ 1.67	$ 2.75
Fourth	106.20	$ 139.00	$ 46.50	270.00	$ 1.67	$ 2.75
Third	106.20	$ 139.00	$ 46.50	270.00	$ 1.67	$ 2.75
Second	225.60	$ 139.00	$ 46.50	358.00	$ 1.67	$ 2.75
First	225.60	$ 139.00	$ 46.50	358.00	$ 1.67	$ 2.75
Midlevel	225.60	$ 139.00	$ 46.50	358.00	$ 1.67	$ 2.75
		$ 159,516.40	$ 53,363.40		$ 3,824.30	$ 6,297.50
		(03 31 05.35.0300)	(03 31 05.70.0800)		(03 11 13.25.7700)	(03 11 13.25.7700)

Reinforcing Cost			Total
Formwork (tons)	Material ($ per tons)	Labor ($ per tons)	
5.50	$ 1,550.00	$ 620.00	$ 21,069.12
10.50	$ 1,550.00	$ 620.00	$ 43,678.50
10.50	$ 1,550.00	$ 620.00	$ 43,678.50
10.50	$ 1,550.00	$ 620.00	$ 43,678.50
10.50	$ 1,550.00	$ 620.00	$ 43,678.50
14.50	$ 1,550.00	$ 620.00	$ 74,896.16
14.50	$ 1,550.00	$ 620.00	$ 74,896.16
14.50	$ 1,550.00	$ 620.00	$ 74,896.16
	$ 141,050.00	$ 56,420.00	$ 420,471.60
	(03 21 10.60.0250)	(03 21 10.60.0250)	

Redesigned Two-Way flat plate slab Cost Estimate (Continued):

				Concrete Shear Wall Costs		
	Concrete Cost			Formwork Cost		
Level	Concrete (ft)	Material ($ per ft)	Labor ($ per ft)	Formwork (ft^2)	Material ($ per ft^2)	Labor ($ per ft^2)
Roof	153.00	$ 116.00	$ 23.50	204.00	$ 0.50	$ 4.79
Sixth	248.00	$ 116.00	$ 23.50	330.67	$ 0.50	$ 4.79
Fifth	248.00	$ 116.00	$ 23.50	330.67	$ 0.50	$ 4.79
Fourth	248.00	$ 116.00	$ 23.50	330.67	$ 0.50	$ 4.79
Third	248.00	$ 116.00	$ 23.50	330.67	$ 0.50	$ 4.79
Second	248.00	$ 116.00	$ 23.50	330.67	$ 0.50	$ 4.79
First	248.00	$ 116.00	$ 23.50	330.67	$ 0.50	$ 4.79
Midlevel	248.00	$ 116.00	$ 23.50	330.67	$ 0.50	$ 4.79
		$ 219,124.00	$ 44,391.50		$ 1,259.33	$ 12,064.41
		(03 31 05.35.0300)	(03 31 05.70.5300)		(03 11 13.85.4750)	(03 11 13.85.4750)

Reinforcing Cost			
Formwork (tons)	Material ($ per tons)	Labor ($ per tons)	Total
5.00	$ 1,400.00	$ 395.00	$ 31,397.66
9.26	$ 1,400.00	$ 395.00	$ 52,966.93
9.26	$ 1,400.00	$ 395.00	$ 52,966.93
9.26	$ 1,400.00	$ 395.00	$ 52,966.93
9.26	$ 1,400.00	$ 395.00	$ 52,966.93
9.26	$ 1,400.00	$ 395.00	$ 52,966.93
9.26	$ 1,400.00	$ 395.00	$ 52,966.93
9.26	$ 1,400.00	$ 395.00	$ 52,966.93
	$ 97,748.00	$ 27,578.90	$ 402,166.15
	(03 21 10.60.0550)	(03 21 10.60.0550)	

Redesigned Two-Way flat plate slab Cost Estimate (Continued):

Level	Slab Cost	Column Cost	Shear Wall Cost	Total	Adjusted Total
Roof	$ 304,950.05	$ 21,069.12	$ 31,397.66	$ 357,416.83	$ 425,363.73
Sixth	$ 649,632.30	$ 43,678.50	$ 52,966.93	$ 746,277.73	$ 888,149.22
Fifth	$ 649,632.30	$ 43,678.50	$ 52,966.93	$ 746,277.73	$ 888,149.22
Fourth	$ 649,632.30	$ 43,678.50	$ 52,966.93	$ 746,277.73	$ 888,149.22
Third	$ 649,632.30	$ 43,678.50	$ 52,966.93	$ 746,277.73	$ 888,149.22
Second	$ 970,149.50	$ 74,896.16	$ 52,966.93	$ 1,098,012.59	$ 1,306,750.82
First	$ 970,149.50	$ 74,896.16	$ 52,966.93	$ 1,098,012.59	$ 1,306,750.82
Midlevel	$ 970,149.50	$ 74,896.16	$ 52,966.93	$ 1,098,012.59	$ 1,306,750.82

	Total Cost	Total Ajdusted Cost
Slab	$ 5,813,927.75	$ 6,919,187.37
Column	$ 420,471.60	$ 500,405.56
Wall	$ 402,166.15	$ 478,620.14
	$ 6,636,565.50	$ 7,898,213.08

A5 ACOUSTICS BREADTH