Letter of Transmittal

November 16, 2015

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Dear Dr. Hanagan,

The attached document Notebook C contains a comprehensive lateral analysis of the wind and seismic loading for The Health Centre located in the southeastern United States. The information provided in Notebook C fulfills requirements assigned by the structural faculty for the architectural engineering senior thesis.

Enclosed in Notebook C is analysis of the concrete moment frame system, 3D modeling documentation, and hand calculations used to verify results. Lateral loading determined in previous notebook submissions have been updated. The appendix contains typical floor plans to further describe the loading conditions in The Health Centre.

Thank you for taking the time to review my analysis and calculations. I look forward to your comments and discussing my work with you in the future.

Regards,

Hannah N. Valentine

NOTEBOOK SUBMISSION C

THE HEALTH CENTRE

LOCATION | SOUTHEASTERN US

Hannah N Valentine

STRUCTURAL OPTION | ADVISOR: DR. HANAGAN

THE HEALTH CENTRE

GENERAL INFORMATION

Location	Southeastern US
Occupancy	Healthcare
Height	166 ft
Total Levels	14 (above + below grade)
Size	450,000 SF of program space
Cost	$168-203 million
Construction	January 2012-2016 (projected)
Project Delivery	CM At-Risk

PROJECT TEAM

Architecture	SmithGroupJJR
Structural	Walter P. Moore
Lighting/Mechanical	ccrd
Construction	McCarthy Building Construction
Civil/Site	Kimley-Horn and Associates, Inc
Wind Consultant	RWDI Consulting Engineers

ARCHITECTURE

- 14 story, core-and-shell university hospital expansion
- 10 story bed tower and 4 story below-grade parking garage
- Operating rooms, intensive care unit, emergency department, clinical facilities, and med-surg patient rooms
- Inspired by the concept of lifelines and classical campus buildings

STRUCTURAL SYSTEMS

Framing........... Cast-in place concrete with one-way floor slabs are used for framing above grade. Post-tensioned two-way concrete slabs used in the parking garage.

Foundations.....Slab on grade is connected by grade beams. Below grade are cast-in-place spread footings & drilled piers.

Lateral .... Concrete moment frames resist wind lateral loads. Parking garage shear walls resist seismic/soil loads.

MECHANICAL SYSTEMS

- Central Energy Plant (CEP) located on level 2 of parking garage
- Chilled water cooling and high pressure steam heating systems
- 3 large cooling towers extend to the roof
- Fan coil units and custom central-station air-handling units with split system air conditioners
- Dedicated air handling units for operating and surgical rooms

LIGHTING/ELECTRICAL SYSTEMS

- Interior lighting uses linear T8 and T5 LEDs fixtures
- Surge protective devices for low-voltage equipment
- Photoelectric switches and daylight-harvesting switching controls contribute to energy savings

HANNAH VALENTINE | STRUCTURAL
ADVISOR | DR. LINDA HANAGAN

CONSTRUCTION

- Building occupancy in 2017
- Sustainable construction process emphasized
- Dirt and filling material from foundations used to build a new soccer field
- All trees removed during the building process to be replanted

RENDERINGS AND INFORMATION COURTESY OF:

SMITHGROUPJJR

WALTER P MOORE

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2 | General Information

2.1 Executive Summary

The Health Centre is a 450,000 square foot university hospital expansion project located in the southeastern United States. Located adjacent to existing hospital facility 'Clinic B,' this nine story L-shaped building is connected by two bridges to the surrounding campus. Demand for new, state-of-the-art medical technology, additional research space, and extra hospital beds prompted the design and construction the Health Centre. At a height of 163 feet, the Health Centre will be by far the tallest building in the surrounding area when its construction is complete in 2016.

As a nod to the heritage and character of the surrounding university campus, The Health Centre takes its architectural cues from classical Italian and contemporary sources. Façade materials used on the building include stucco, metal panels, and a glass curtain wall. A green roof and four story underground parking garage contribute towards its goal of LEED silver certification. This building was designed as a "core-and shell," necessitating a structural consideration for flexibility of spaces and future expansion.

The structure of the Health Centre is mainly cast-in-place concrete on drilled piers and spread footings. Its floor system in the hospital bed tower consists of cast-in-place one-way concrete slabs and beams. Concrete moment frames spread throughout the structure resist the building's lateral loads. Below grade, parking garage floor slabs consist of two-way post-tensioned concrete slabs. The parking garage has its own lateral system of concrete shear walls. Some structural steel components exist in the building, including roofing and bridges connecting to other buildings on campus.

Governing codes for the design of the Health Centre required the use of IBC 2012. However, an exemption was obtained to allow the structural design to use IBC 2006 requirements. ASCE 7-05 provides the minimum design loads for live, snow, wind, and seismic considerations. Due to the life safety importance associated with hospital structures, a conservative approach was used to determine building loads.

2.2 Site Plan

The Health Centre is located on a university campus in the southeastern United States. Adjacent to the site is 'Clinic B,' the existing hospital building. Bridges connect the hospital facilities to the surrounding campus. A new entry drive allows patients and emergency vehicles direct access to the new Health Centre. Figure 1 shows the site plans from SmithGroupJJR documents.

Terrain around the site is extremely flat. As the tallest building in the immediate area, The Health Centre will be fully impacted by wind loads.

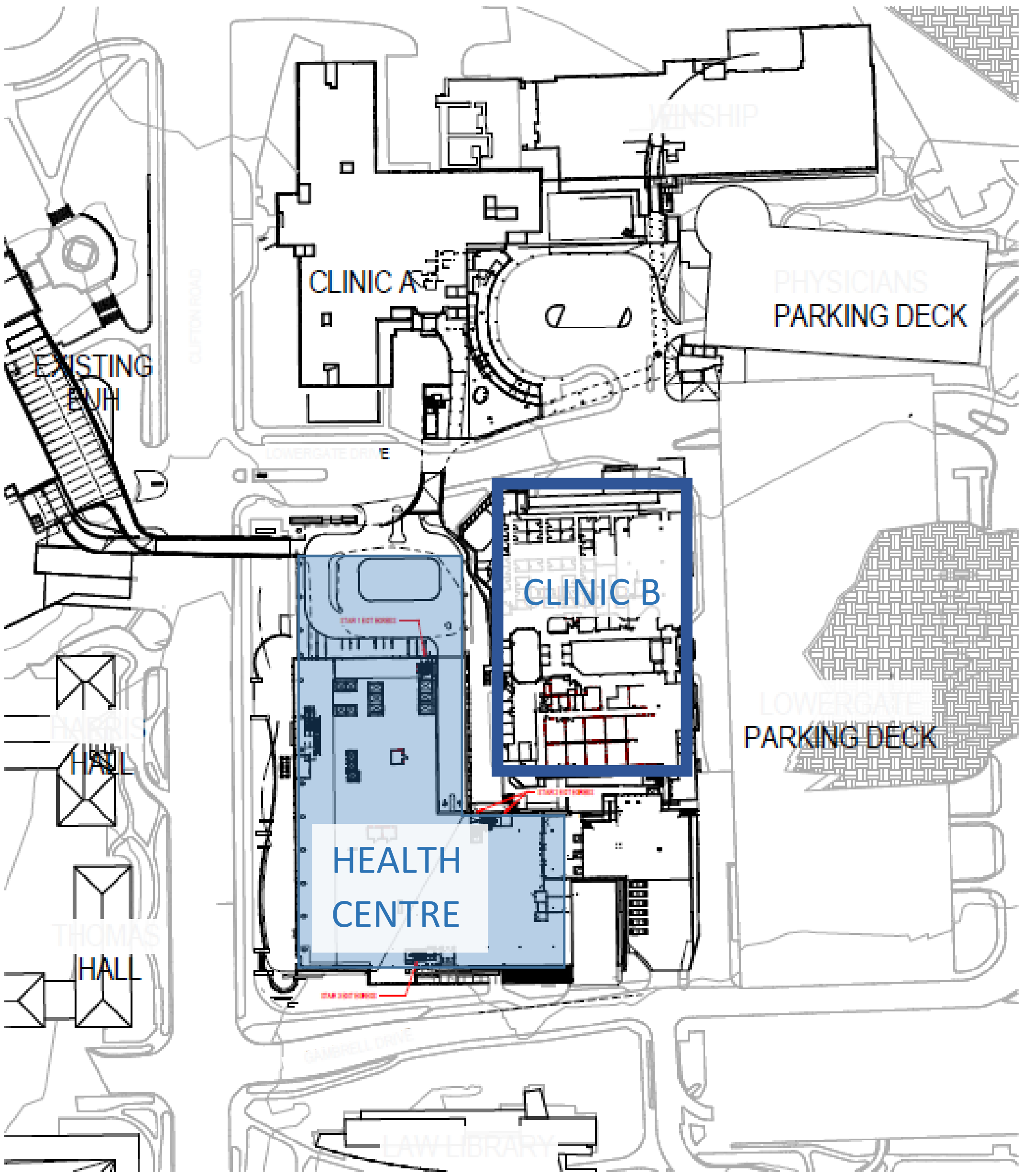


Figure 1 | Site Plan of Surrounding Area

2.3 References

The following table is a list of documents referenced during the preparation of Notebook C to determine building loads.

Organization	Reference
International Building Code	2006 International Building Code
American Society of Civil Engineers	ASCE 7-05 | Minimum Design Loads for Buildings and Other Structures
American Concrete Institute	ACI 318-11 | Building Code Requirements for Structural Concrete
American Institute of Steel Construction	Steel Construction Manual, 14th Edition
United States Geological Survey	Seismic Design Maps
Computers & Structures, Inc	CSI Technical Support for Etabs
Penn State	Architectural Engineering Course Notes
Vulcraft	Deck Catalog
Walter P. Moore	Health Centre General Notes Sheet

Table 1 | Notebook C References

3 | Gravity Loads

This section summarizes the building gravity loads due to dead, live, and snow, and perimeter loads. Loads have been updated from previous Notebook A submission. A full list of design gravity loads used by the original structural engineer may be found in Appendix A.

3.1 Roof Loads

Three roof gravity load cases exist for this building: typical concrete roof, penthouse roof, and green roof. Figures 2-4 depict the roof sections that correspond with each load case.

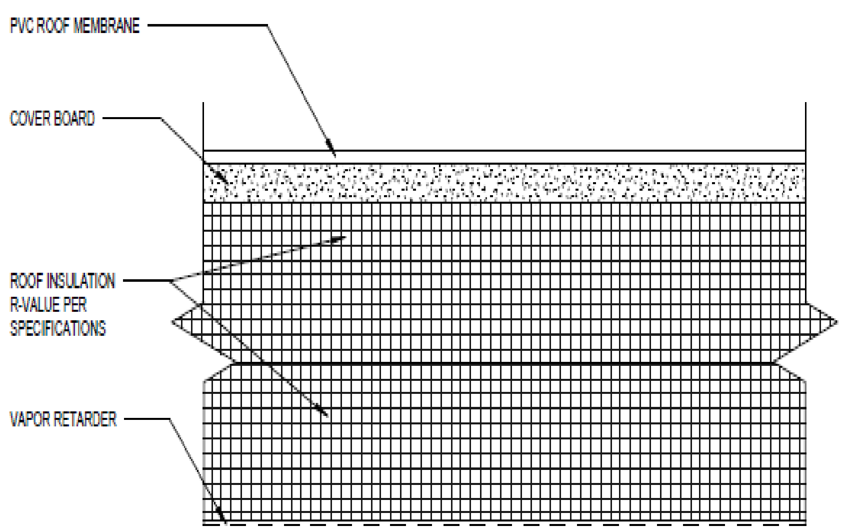


Figure 2 | Typical Concrete Roof Section (SmithGroupJJR)

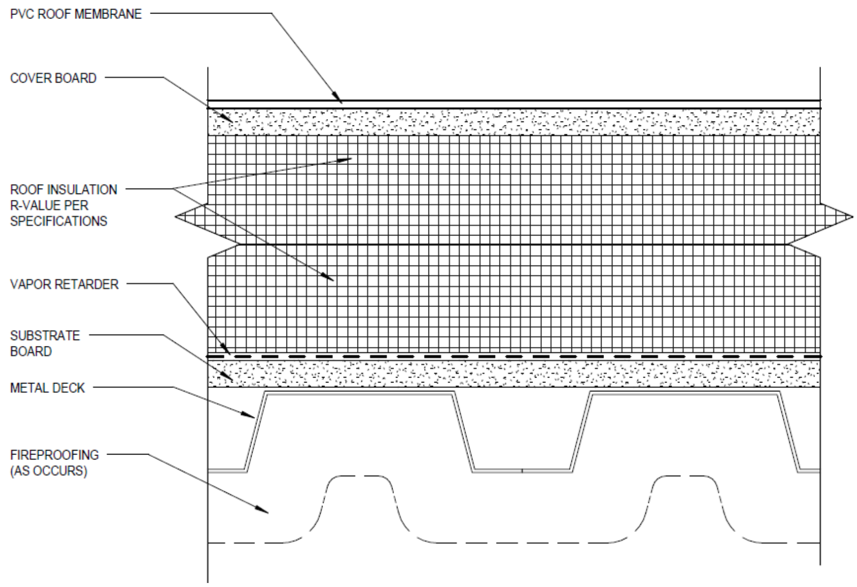


Figure 3 | Penthouse Roof Section (SmithGroupJJR)

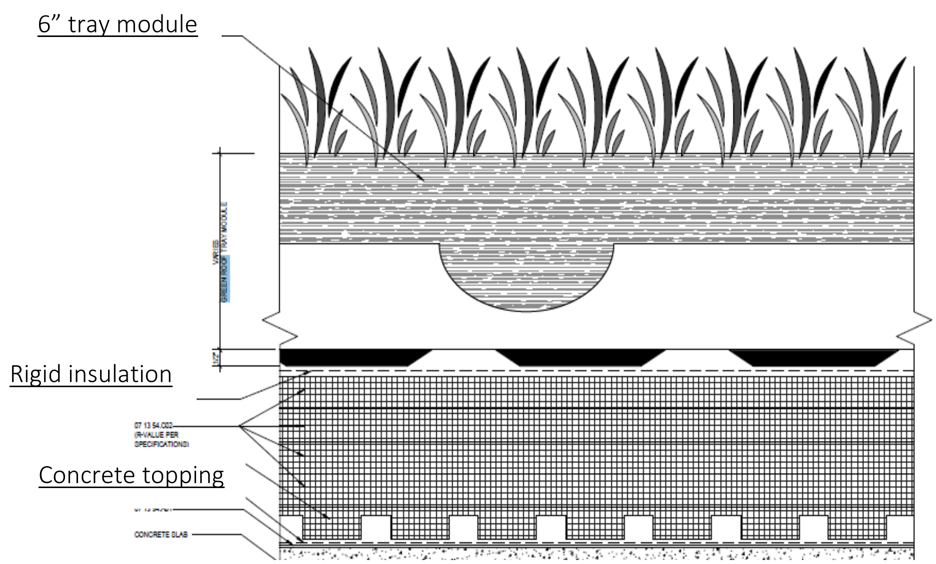


Figure 4 | Green Roof Section (SmithGroupJJR)

Below is a summary of the roof gravity dead and live load values determined in this section.

Load Type	Dead	Live
Typical Roof	83 psf	20 psf Not reduced
Penthouse Roof	40 psf (50 psf SDL from structural drawings)	20 psf Not reduced
Green Roof	103 psf	100 psf Not reduced

Table 2 | Roof Gravity Load Summary

A flat roof snow load for the building is calculated below, but will not control design. Snow drift will be considered for the green roof and lower 6th level roof due to the large height difference between these levels and the penthouse roof. The 6th level is designed for future expansion and may become an enclosed floor in the future. Floor live loads for the 6th level roof will likely control.

3.2 Floor Loads

Floor dead and live loads will be determined for both the bed tower and parking garage floor systems in this report. On the following page, Figures 6 shows typical details for the floor slabs under consideration for this report. Concrete floor slabs in the bed tower are typically 5 or 7 inches. All dead load values are based on the typical bay pictured in Figure 5.

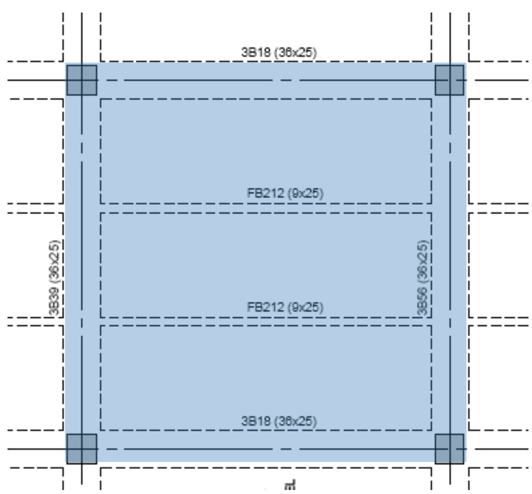


Figure 5 | Typical Bay from Third Floor Area D Floor Plan (Walter P. Moore)

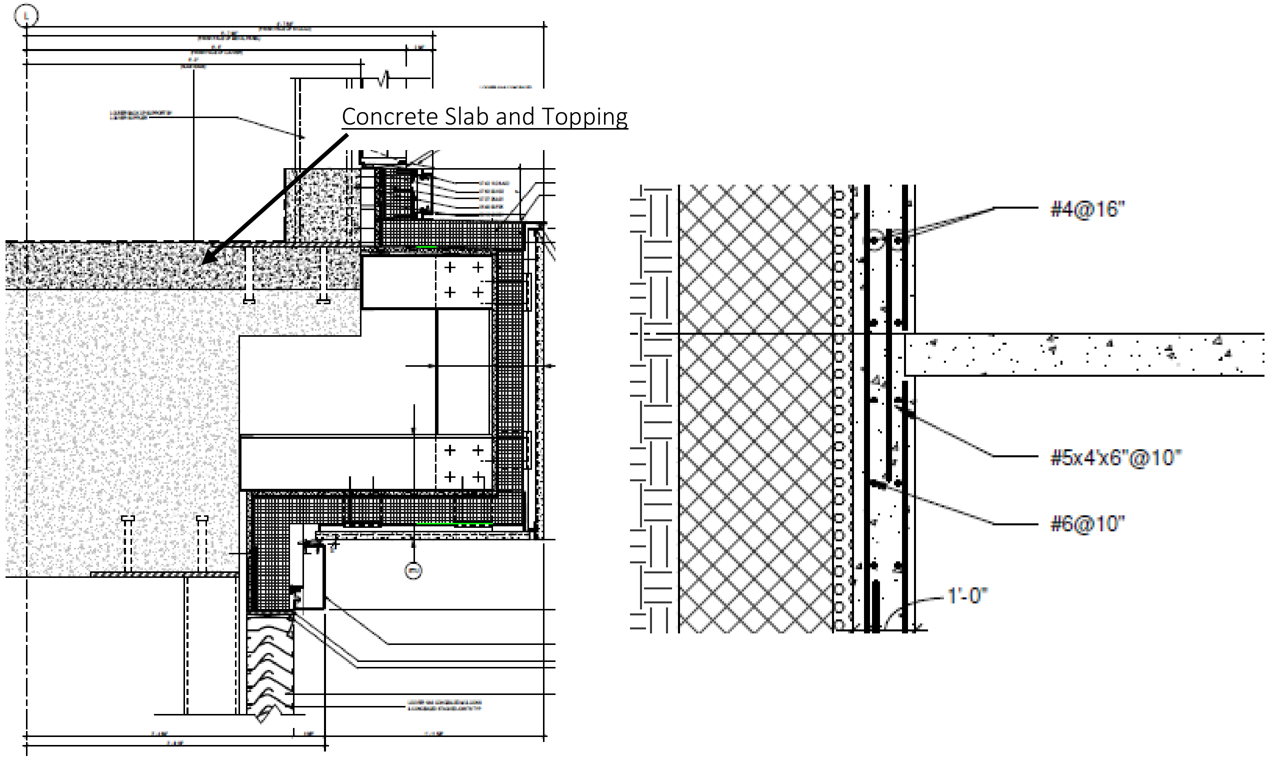


Figure 6 | Floor Section in Bed Tower (left, SmithGroupJJR) and Parking Garage (right, Walter P. Moore)

Floor Use	Dead	Live
Typical Hospital Areas	5" slab – 86 psf 7" slab – 111 psf	100 psf – reduced (design value)
Corridors + Lobbies	5" slab – 86 psf 7" slab – 111 psf	100 psf
Stairs	5" slab – 86 psf 7" slab – 111 psf	100 psf
Mechanical Rooms	5" slab – 86 psf + 200 K mech. equip 7" slab – 111 psf	150 psf
Diagnostics + Imaging	5" slab – 86 psf + 80 K diagnostic equip. 7" slab – 111 psf	350 psf – not reduced (design value)
Patient Rooms (Designed as Hospital – Corridor)	5" slab – 86 psf 7" slab – 111 psf	80 psf
Parking Garage	5" slab – 86 psf 7" slab – 111 psf	40 psf

Table 3 | Floor Gravity Loads Summary

3.3 Perimeter Loads

The building perimeter enclosure produces a linear dead load through its attachment to the main building structure. The Health Centre has three main enclosure systems: curtain wall, stucco panels, and metal panels. Figures 7-8 depict the methods of attachment for each system.

Each system has a different load path that is dependent on its connection to the structure. The curtain wall's framing system is connected to the main structure by a structural steel plate and embedded metal stud.

Loads transfer from the stucco wall via continuous light gauge angles attached to continuous light gauge zees. The light gauge zees are connected by a fiberglass thermal spacer clip to gypsum sheathing, which takes the load to the main structure via another light gauge zee.

A light gauge zee connects the metal wall panels to the main structure, and load is transferred through steel bolts.

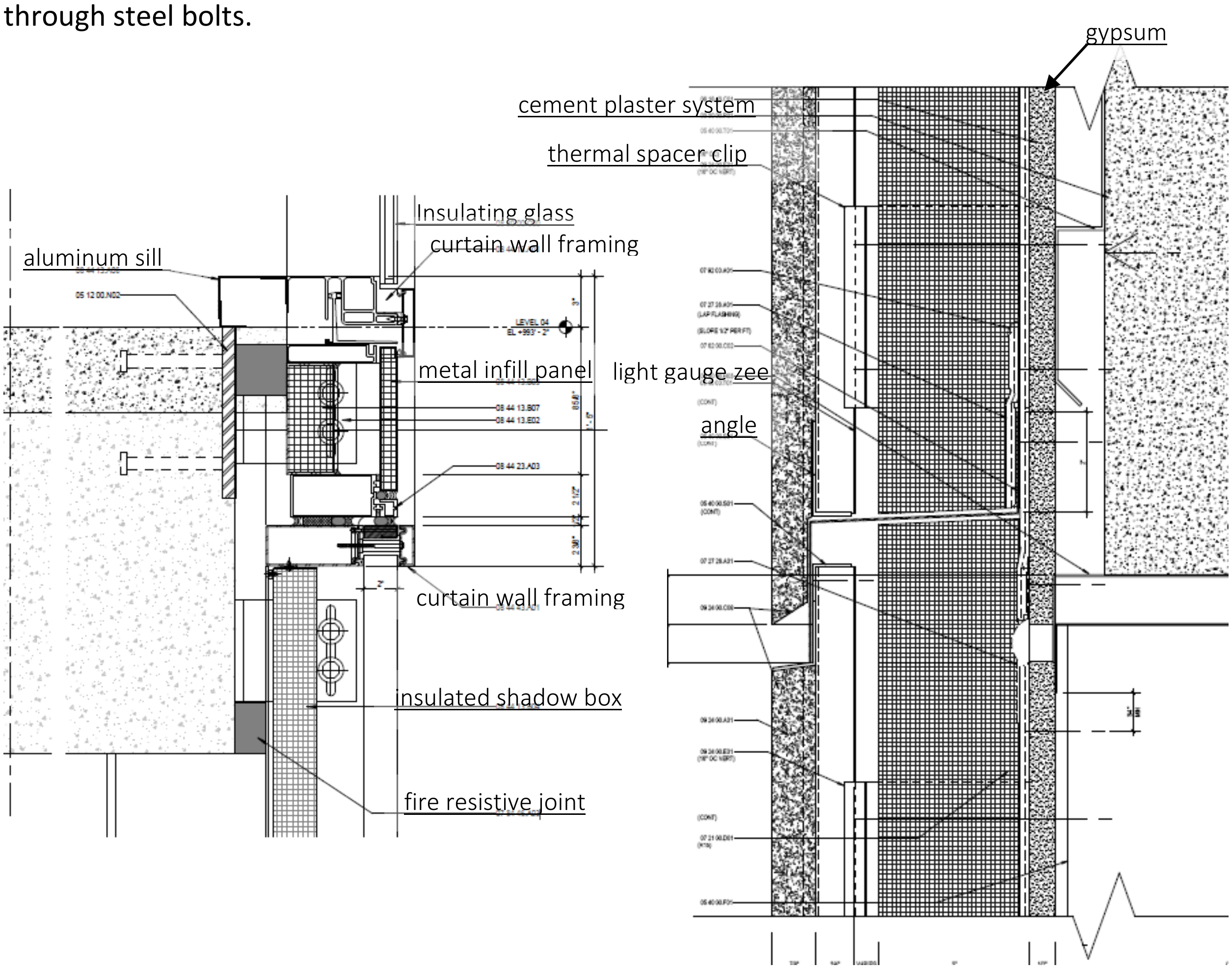


Figure 7 | Curtain Wall Connection Detail (left) and Stucco Panel Wall Envelope (right) from SmithGroupJJR

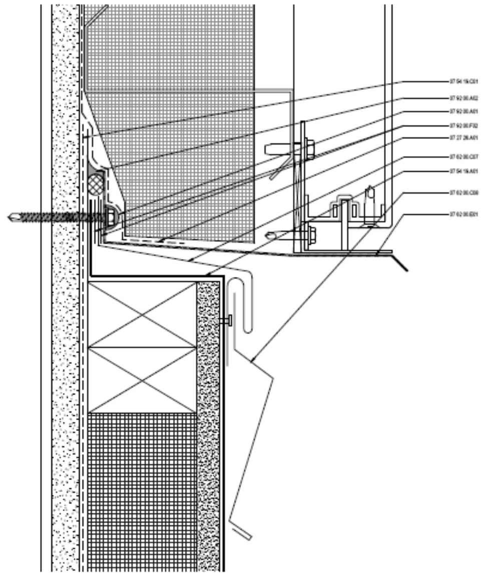


Figure 8 | Metal Panel Connection Detail (SmithGroupJJR)

Wall Type	Dead Load
Curtain Wall	16 psf
Panel Systems	18 psf

Table 4 | Perimeter Loads Summary

The following pages are hand calculations used to determine the gravity loads listed in this section. Some loads have been updated from the previous Notebook A submission.

Gravity Loads - Roof

Dead Loads:

1) Green Roof System

6" Green Roof Tray Module	30 psf
Landscaping Stone	5 psf
Insulation	5 psf
Concrete Slab *(see note below)	63 psf
	103 psf

* See floor loads for 5" conc slab ca

2) Typical Concrete Roof

Roof Finishes + Insulation	5 psf
Concrete Slab + Framing	63 psf
MEP	15 psf
	83 psf

3) Penthouse Roof

Roof Finishes + Insulation	5 psf
Steel Framing	15 psf
MEP	15 psf
Deck - 1.5" 22GA Type B	1.78 psf
Misc	3 psf
	40 psf

Live Loads:

1) Green Roof System — 100 psf
Yards + Terraces, pedestrian - Not Reduced

2) Typical Roof — 20 psf
Ordinary Flat Roof - Not Reduced

3) Penthouse Roof — 20 psf
Ordinary Flat Roof - Not Reduced

Live Loads from ASCE 7-05 Table 4-1

Snow Loads:

By inspection, snow loads will not control.

$p_f = 0.7 C_e C_t I p_g$ (7-1)

$p_g = 5$ psf (Fig. 7-1)

$C_e = 0.9$ (Table 7-2)

$C_t = 1.0$ (Table 7-3)

$I = 1.2$ (Table 7-4)

$p_f = 0.7(0.9)(1.0)(1.2)(5) = 4$ psf

1) Snow Drift on 4th Floor Green Roof From Penthouse Level (117 ft)

$h_d = \max \begin{vmatrix} 2\text{ ft (leeward) (fig 7-9)} \\ <1\text{ ft (windward)} \end{vmatrix} = 2\text{ ft}$ (Section 7-7)

$\gamma = 0.13p_g + 14 = 0.13(5) + 14 = 14.65$ (7-2)

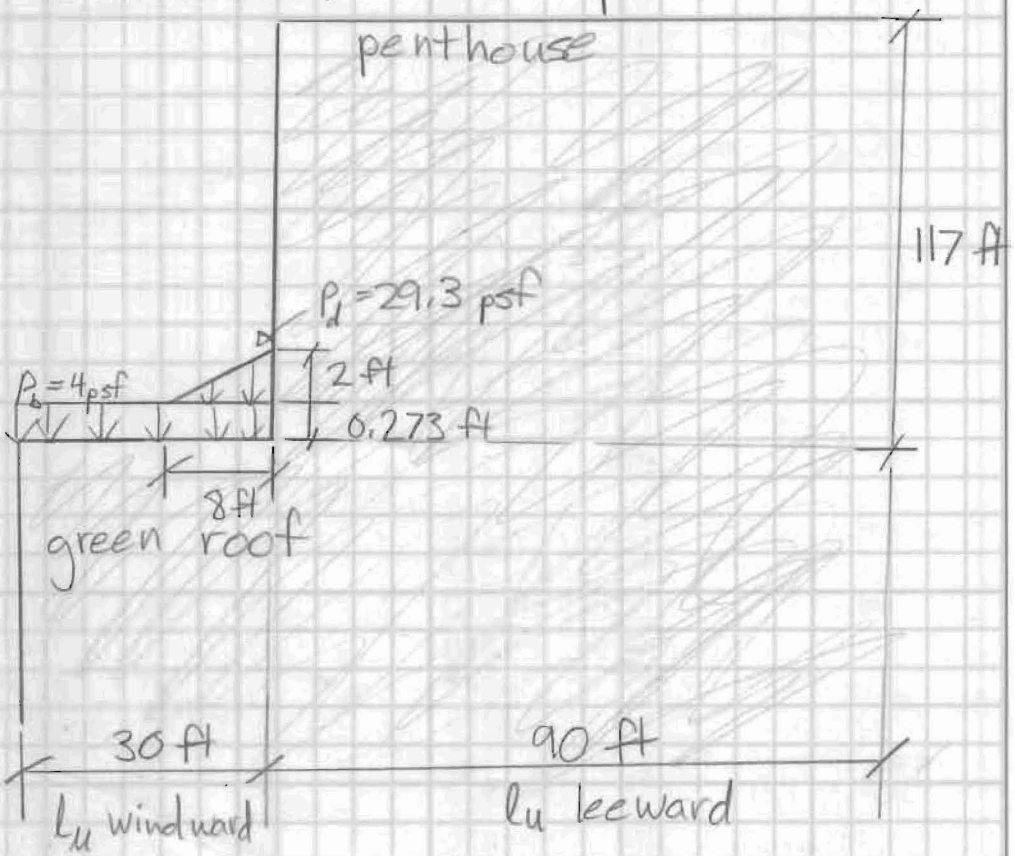
$h_b = P_o/14.65 = 4/14.65 = 0.273$ ft

$h_c \approx 117$ ft

$h_c/h_b > 0.2$ ∴ Must calculate drift

$h_d < h_c$ ∴ $W = 4h_d = 4(2) = 8$ ft

$P_d = h_d\gamma = 2(14.65) = 29.3$ psf



2) Snow Drift on 6^{th} Floor From Penthouse Roof Level (68 ft)

$\gamma = 14.65$

$h_b = 0.273$

$h_c \approx 68$ ft

$h_c/h_b > 0.2$ ∴ Must calculate drift

$h_d = \max \begin{cases} 2 \\ 3/4(3) = \underline{2.25 \text{ ft}} \end{cases}$

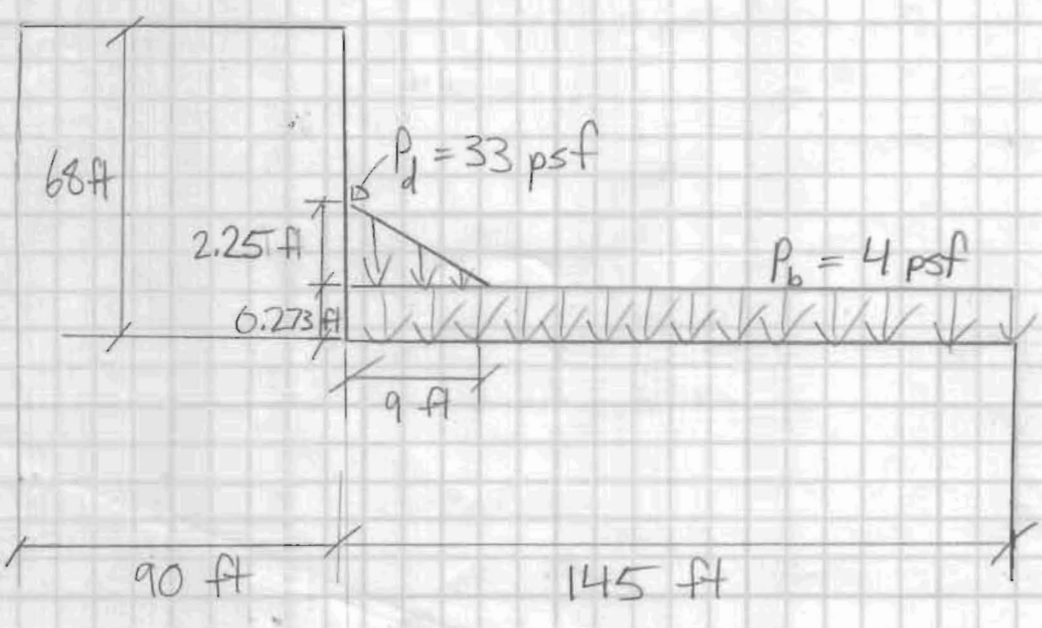
145 ft

$l_u = 90$ ft leeward

$l_u = 145$ ft windward

$h_d < h_c$ ∴ $w = 4h_d = 4(2.25) = 9$ ft

$P_d = h_d \gamma = (2.25)(14.65) = 33$ psf



Bed Tower Dead Loads – Concrete

Thickness	Area + Floors	Slab Wt	Slab + Floor Loads Total Wt.	Seismic Wt (total + framing)
5"	2B, C, D 3B, C, D 4A, B, C, D 5A, B, C, D 6A, B, C, D 7(all), 8(all) 9(all), penthouse	$5'' \cdot \frac{1 ft}{12''} \times 150$ $= 62.5$ psf	86 psf	117.5 psf ≈ 120 psf
7"	2B, 3B, 4B 5A/B, 6A/B 7A/B, 8(all) 9(all)	$7'' \cdot \frac{1 ft}{12''} \times 150$ $= 87.5$ psf	111 psf	142.5 psf ≈ 145 psf

Occurances of all other slab thicknesses are infrequent and in small areas

5000 psi NWC → 150 pcf

Typical Bay: $4 \times \left((36'' \times (25.5)'' \times \frac{1 ft^2}{144 in^2} \times 30'\right) = 150 ft^3$

$2 \times \left(9'' \times (25.5)'' \times \frac{1 ft^2}{144 in^2} \times 30'\right) = 37.5 ft^3$

Beam Wt $(150 + 37.5 ft^3)(150 pcf) = 28125$ lbs

$$\frac{28125 \text{ lbs}}{30' \times 30' \text{ bay}} = 32 \text{ psf}$$

Floor Loads

Finishes + Ceiling	5 psf
Concrete Slab	See above
MEP	15 psf
Misc.	3 psf
	23 psf + slab

Bed Tower Dead Loads - Elevator Machine Room

3 ½" LWC on 2" Deep 20GA Composite Deck

Vulcraft Value: 57 psf

Steel Framing	15 psf
Finishes + Ceiling	5 psf
Metal Deck	57 psf
MEP	15 psf
Elev. Equip	20 psf
Misc.	3 psf
	$\boxed{115 \text{ psf}}$

Parking Garage Dead Loads

Typical Depth = 8" NWC (structural plan notes)

Slab Wt = $8'' \cdot \frac{1}{12} \cdot 150$ pcf = 100 psf

Concrete Slab	100 psf
Misc. (lighting, etc)	5 psf
	105 psf

Floor Live Loads - ASCE 7-05 Table 4-1

Bed Tower

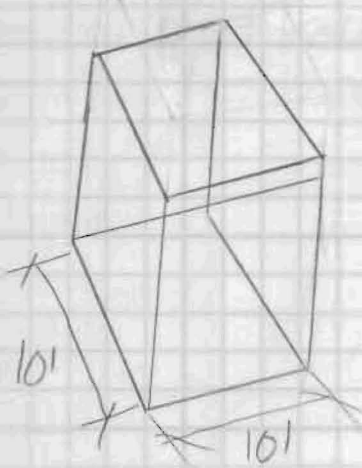
Typical Hospital Areas (restroom, etc)	100 RED (design value)
Corridors + Lobbies	100 psf
Stairs	100 psf
Mechanical Rooms	150 psf (design value)
Diagnostics + Imaging	350 psf NR (design value)
Patient Rooms (Hospital-Corridor)	80 psf
Light Storage	125 psf NR

Parking Garage - 40 psf

Non-typical Loads Considered:

- Green roof (see roof loads)
- Diagnostic Equipment
  - 350 psf design live load in some diagnostic areas.
  - Equip. weight unknown from specs
  - Assume 10'x10' machine...

$$350\ \text{psf} \times (10\ \text{ft})^2 = \underline{35\ \text{k}}$$



3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

Perimeter Enclosure Dead Loads

Curtain Wall

2" Insulating Glass	1 psf
Curtain Wall Framing Insulating Shadow Box	4 psf
3" Rigid Insulation	5 psf
Interior Sheathing 1/2"	2 psf
Misc. (sealants for joints, etc)	4 psf
	16 psf

Panel Systems

Stucco/Metal Panels (7/8")	10 psf
3" Rigid Insulation	4 psf
1/2" Gypsum Interior	2 psf
Metal Framing	2 psf
Air Gap	0 psf
	18 psf

*Building material weights via AISC Steel Manual, 14th edition

3 | Wind Loads

The following section calculates wind loads perpendicular and parallel to The Health Centre using criteria from chapter 6 of ASCE 7-05 for a flexible building. Excel and hand calculations were used to determine load values and gust effect factors. Both building and parapet loads are included in this section.

4.1 Perpendicular Loads

Building Geometry

B =	421.25	ft
L =	285	ft
h =	166	ft
z_{bar} =	99.6	ft

Variables Used

Basic Wind Speed	V =	90	mph	(Figure 6-1)
Directionality Factor	K_d =	0.85		(Table 6-4)
Occupancy Category		IV		(Table 1-1)
Importance Factor	I =	1.15		(Table 6-1)
Topographic Factor	K_{zt} =	1		(Walter P. Moore)
Exposure Category		B		(Walter P. Moore)

Calculation of K_z and q_z

$q_z = 0.00256K_zK_{zt}K_dV^2I$ (6-15)

Story	Height (ft)	K_z - Case 1	K_z - Case 2	q_z - Case 1 (psf)	q_z - Case 2 (psf)
2	16	0.7	0.58	14.1886	11.7563
3	32	0.712	0.712	14.4318	14.4318
4	49	0.805	0.805	16.3169	16.3169
5	66	0.874	0.874	17.7155	17.7155
6	83	0.939	0.939	19.0330	19.0330
7	98	0.984	0.984	19.9451	19.9451
8	113	1.0225	1.0225	20.7255	20.7255
9	128	1.06	1.06	21.4856	21.4856
penthouse	143	1.096	1.096	22.2153	22.2153
roof/q_h	166	1.142	1.142	**23.1477**	**23.1477**

*Note: Only discrepency between Case 1 and 2 values occurs at 16 ft

Gust Effect Factor G_f

See pages 1-3 of wind calcs for detailed calculations and code references.

Natural Frequency	n_1 =	0.437	Hz	(C6-15)
Resonant Response Factor	g_R =	3.987		(6-9)
Background & Wind Factor	g_v, g_Q =	3.4		(6-9)
Mean Hourly Wind	$V_{z,bar}$ =	78.3	mph	(6-14)
Turbulence Length	$L_{z,bar}$ =	462.45		(6-7)
Reduced Frequency	N_1 =	2.581		(6-12)
Resonant Responcse Factor	R =	0.1958		(6-10)
Turbulence Intensity	I_z =	0.25		(6-5)
Background Response Factor	Q =	0.76		(6-6)
Flexible Gust Effect Factor	G_f =	**0.8123**		(6-8)

External Pressure Coefficient C_p

See pages 3 of wind calcs for detailed calculations.

	L/B =	0.6766	
	h/L =	0.5825	
	Θ =	< 10	degrees
Windward Wall	C_p =	0.8	
Leeward Wall	C_p =	-0.5	
Side Wall	C_p =	-0.7	
Roof - 0 to h/2	C_p =	-0.9	-0.18
Roof - h/2 to h	C_p =	-0.9	-0.18
Roof - h to 2h	C_p =	-0.5	-0.18
Roof - >2h	C_p =	-0.3	-0.18

(Figure 6-6)

Design Wind Pressure P

$p = qG_fC_p - q_i(Gc_{pi})$ (6-19)

Location	z (ft)	q_z / q_h (psf)	C_p	G_f	q_zG_fCp (psf)	GC_{pi}	Net Pressure (psf) $q_zG_fC_p - q_h(+GC_{pi})$	Net Pressure (psf) $q_zG_fC_p - q_h(-GC_{pi})$
Windward	16 - Case 1	14.1886	0.8	0.8123	11.3509	0.18	7.1843	15.5175
	16 - Case 2	11.7563	0.8	0.8123	9.4050	0.18	5.2384	13.5716
	32	14.4318	0.8	0.8123	11.5455	0.18	7.3789	15.7121
	49	16.3169	0.8	0.8123	13.0535	0.18	8.8869	17.2201
	66	17.7155	0.8	0.8123	14.1724	0.18	10.0058	18.3390
	83	19.0330	0.8	0.8123	15.2264	0.18	11.0598	19.3930
	98	19.9451	0.8	0.8123	15.9561	0.18	11.7895	20.1227
	113	20.7255	0.8	0.8123	16.5804	0.18	12.4138	20.7470
	128	21.4856	0.8	0.8123	17.1885	0.18	13.0219	21.3551
	143	22.2153	0.8	0.8123	17.7722	0.18	13.6057	21.9388
	166	23.1477	0.8	0.8123	18.5182	0.18	14.3516	22.6847
Leeward	All	23.1477	-0.5	0.8123	-11.5739	0.18	-15.7404	-7.4073
Side	All	23.1477	-0.7	0.8123	-16.2034	0.18	-20.3700	-12.0368
Roof (0'-83')	166	23.1477	-0.9	0.8123	-20.8329	0.18	-24.9995	-16.6663
Roof (83'-166')	166	23.1477	-0.9	0.8123	-20.8329	0.18	-24.9995	-16.6663
Roof (166'-332')	166	23.1477	-0.5	0.8123	-11.5739	0.18	-15.7404	-7.4073
Roof (> 332')	166	23.1477	-0.3	0.8123	-6.9443	0.18	-11.1109	-2.7777

4.2 Parallel Loads

Building Geometry

B =	285	ft
L =	421.25	ft
h =	166	ft
z_{bar} =	99.6	ft

Variables Used

Basic Wind Speed	V =	90	mph	(Figure 6-1)
Directionality Factor	K_d =	0.85		(Table 6-4)
Occupancy Category		IV		(Table 1-1)
Importance Factor	I =	1.15		(Table 6-1)
Topographic Factor	K_{zt} =	1		(Walter P. Moore)
Exposure Category		B		(Walter P. Moore)

Calculation of K_z and q_z

$q_z = 0.00256 K_z K_{zt} K_d V^2 I$ (6-15)

Story	Height (ft)	K_z - Case 1	K_z - Case 2	q_z - Case 1 (psf)	q_z - Case 2 (psf)
2	16	0.7	0.58	14.1886	11.7563
3	32	0.712	0.712	14.4318	14.4318
4	49	0.805	0.805	16.3169	16.3169
5	66	0.874	0.874	17.7155	17.7155
6	83	0.939	0.939	19.0330	19.0330
7	98	0.984	0.984	19.9451	19.9451
8	113	1.0225	1.0225	20.7255	20.7255
9	128	1.06	1.06	21.4856	21.4856
penthouse	143	1.096	1.096	22.2153	22.2153
roof/q_h	166	1.142	1.142	**23.1477**	**23.1477**

*Note: Only discrepency between Case 1 and 2 values occurs at 16 ft

Gust Effect Factor G_f

See pages 6-7 of wind calcs for detailed calculations and code references.

Natural Frequency	n_1 =	0.437	Hz	(C6-15)
Resonant Response Factor	g_R =	3.987		(6-9)
Background & Wind Factor	g_v, g_Q =	3.4		(6-9)
Mean Hourly Wind	$V_{z,bar}$ =	78.3	mph	(6-14)
Turbulence Length	$L_{z,bar}$ =	462.45		(6-7)
Reduced Frequency	N_1 =	2.581		(6-12)
Resonant Responcse Factor	R =	0.1958		(6-10)
Turbulence Intensity	I_z =	0.25		(6-5)
Background Response Factor	Q =	0.76		(6-6)
Flexible Gust Effect Factor	**G_f =**	**0.8123**		(6-8)

External Pressure Coefficient C_p

See pages 7 of wind calcs for detailed calculations.

	L/B =	1.4781		
	h/L =	0.3941		
	Θ =	< 10	degrees	
Windward Wall	C_p =	0.8		(Figure 6-6)
Leeward Wall	C_p =	-0.4044		
Side Wall	C_p =	-0.7		
Roof - 0 to h/2	C_p =	-0.9	-0.18	
Roof - h/2 to h	C_p =	-0.9	-0.18	
Roof - h to 2h	C_p =	-0.5	-0.18	
Roof - >2h	C_p =	-0.3	-0.18	

Design Wind Pressure P

$p = qG_fC_p - q_i(Gc_{pi})$ (6-19)

Location	z (ft)	q_z / q_h (psf)	C_p	G_f	q_zG_fCp (psf)	GC_{pi}	Net Pressure (psf) $q_zG_fC_p - q_h(+GC_{pi})$	$q_zG_fC_p - q_h(-GC_{pi})$
Windward	16 - Case 1	14.1886	0.8	0.8123	11.3509	0.18	7.1843	15.5175
	16 - Case 2	11.7563	0.8	0.8123	9.4050	0.18	5.2384	13.5716
	32	14.4318	0.8	0.8123	11.5455	0.18	7.3789	15.7121
	49	16.3169	0.8	0.8123	13.0535	0.18	8.8869	17.2201
	66	17.7155	0.8	0.8123	14.1724	0.18	10.0058	18.3390
	83	19.0330	0.8	0.8123	15.2264	0.18	11.0598	19.3930
	98	19.9451	0.8	0.8123	15.9561	0.18	11.7895	20.1227
	113	20.7255	0.8	0.8123	16.5804	0.18	12.4138	20.7470
	128	21.4856	0.8	0.8123	17.1885	0.18	13.0219	21.3551
	143	22.2153	0.8	0.8123	17.7722	0.18	13.6057	21.9388
	166	23.1477	0.8	0.8123	18.5182	0.18	14.3516	22.6847
Leeward	All	23.1477	-0.4044	0.8123	-9.3609	0.18	-13.5275	-5.1943
Side	All	23.1477	-0.7	0.8123	-16.2034	0.18	-20.3700	-12.0368
Roof (0'-83')	166	23.1477	-0.9	0.8123	-20.8329	0.18	-24.9995	-16.6663
Roof (83'-166')	166	23.1477	-0.9	0.8123	-20.8329	0.18	-24.9995	-16.6663
Roof (166'-332')	166	23.1477	-0.5	0.8123	-11.5739	0.18	-15.7404	-7.4073
Roof (> 332')	166	23.1477	-0.3	0.8123	-6.9443	0.18	-11.1109	-2.7777

4.3 Parapet Loads

Building Geometry

B =	285	ft
L =	421.25	ft
h =	166	ft
z_{bar} =	99.6	ft

Variables Used

Basic Wind Speed	V =	90	mph	(Figure 6-1)
Directionality Factor	K_d =	0.85		(Table 6-4)
Occupancy Category		IV		(Table 1-1)
Importance Factor	I =	1.15		(Table 6-1)
Topographic Factor	K_{zt} =	1		(Walter P. Moore)
Exposure Category		B		(Walter P. Moore)

Calculation of K_z and q_z

$q_p = 0.00256 K_z K_{zt} K_d V^2 I$ (6-15)

Parapet	Height (ft)	K_z - Case 1	K_z - Case 2	q_p - Case 1 (psf)	q_p - Case 2 (psf)
Mech Roof	170	1.15	1.15	23.3099	23.3099
Green Roof	54	0.826	0.826	16.7426	16.7426
Penthouse	147	1.104	1.104	22.3775	22.3775
q_h	166	1.142	1.142	**23.1477**	**23.1477**

*Note Case 1 and 2 values are the same for all parapet types.

Design Wind Pressure P

$p_p = q_p GC_{pu}$ (6-20)

				Net Pressure (psf)	
Parapet	q_p (psf)	GC_{pu} - windward	Gc_{pu} - leeward	p_p - windward	p_p - leeward
Mech Roof	23.3099	1.5	-1	34.9648	-23.309856
Green Roof	16.7426	1.5	-1	25.1138	-16.74255744
Penthouse	22.3775	1.5	-1	33.5662	-22.37746176

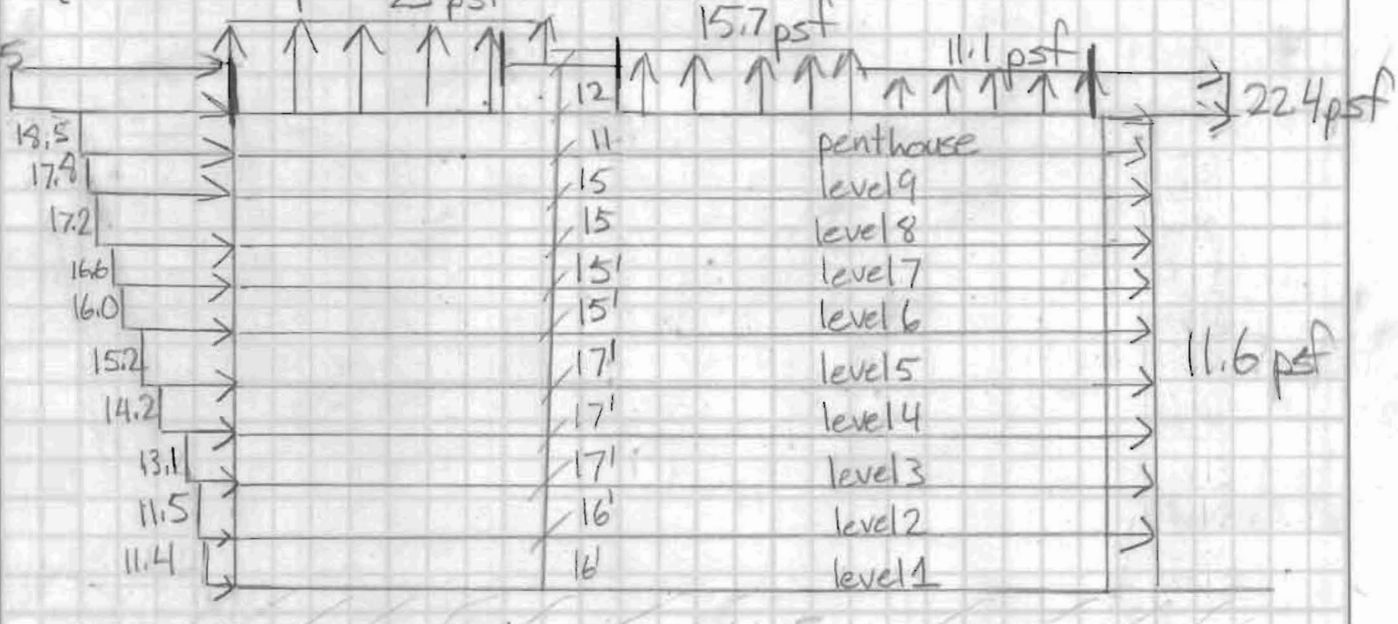
See pages 4 and 5 of hand calculations.

4.4 Summary and Hand Calculations

Below is a summary of base shear values and story forces for the perpendicular and parallel wind directions. On the following page, wind load diagrams summarize the loads on the building.

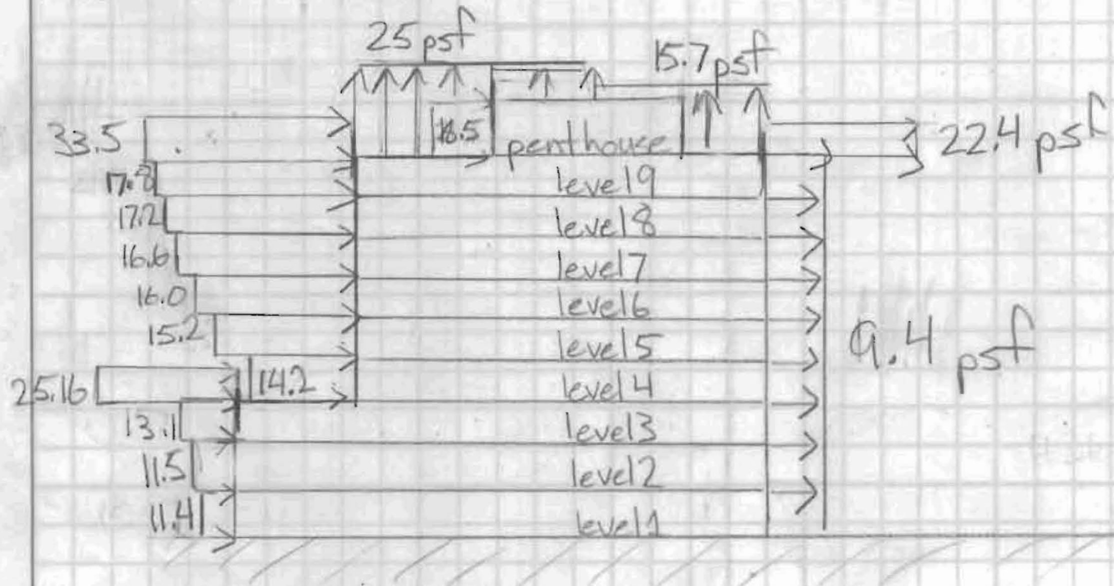
Level	Floor Height (ft)	Windward (psf)		Leeward Pressure		Length (ft)		Shear (K)	
		Perp	Parallel	Perp	Parallel	Perpendicular	Parallel	Perpendicular	Parallel
2	16	11.3508864	11.3508864	11.5738502	9.36093007	280.5	285	102.886218	94.4458831
3	17	11.545473	11.54547302	11.5738502	9.36093007	280.5	285	110.244493	101.291523
4	17	13.0535194	13.05351936	11.5738502	9.36093007	421.25	255	176.3627505	97.1666383
5	17	14.1723924	14.17239245	11.5738502	9.36093007	421.25	255	184.3752804	102.016953
6	15	15.2264033	15.22640333	11.5738502	9.36093007	421.25	285	169.3441022	105.11085
7	15	15.9561032	15.95610317	11.5738502	9.36093007	421.25	90	173.9548931	34.1779949
8	15	16.5804019	16.58040192	11.5738502	9.36093007	421.25	90	177.8996808	35.0207982
9	15	17.1884851	17.18848512	11.5738502	9.36093007	421.25	90	181.7420066	35.8417105
penthouse	19	17.772245	17.77224499	11.5738502	9.36093007	421.25	90	234.8788097	46.3977294
						Base Shear (k)		**1511.688234**	**651.470081**

Perpendicular Wind Loads - Looking East
(Values in psf)



Base shear =

Parallel Wind Loads - Looking North
(Values in psf)



52.49
77.51

Base shear =

Wind Loads - 6.5.3 Design Procedure

1. $V = 90$ mph (figure 6-1)
   $K_d = 0.85$ (table 6-4)

2. $I = 1.15$ (table 6-1)
   (table 1-1)

3. $K_z = ?$ (table 6-3)

   Wind Exposure Category B
   See Table - Wind Pressures.

4. $K_{zt} = 1.0$ (figure 6-4)

5. $G = ?$

   Estimating rigidity for concrete moment frames

$$n_1 = \frac{43.5}{H^{0.9}} = \frac{43.5}{(166\text{ ft})^{0.9}} = 0.437\text{ Hz} \quad (C6-15)$$

∴ Building cannot be considered rigid

$$G_f = 0.925\left(\frac{1 + 1.7 I_z \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_z}\right) \quad (6-8)$$

$$g_R = \sqrt{2\ln(3600 n_1)} + \frac{0.577}{\sqrt{2\ln(3600 n_1)}} \quad (6-9)$$

$$= \sqrt{2\ln(3600(0.437))} + \frac{0.577}{\sqrt{2\ln(3600(0.437))}}$$

$$= 3.83689 + 0.15038$$

$$g_R = \boxed{3.987}$$

$$\bar{V}_z = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) \quad (6-14)$$

$c = 0.30$ $\bar{\alpha} = \frac{1}{4.0}$ $\bar{b} = 0.45$ (table 6-2)

$$\bar{z} = \max\left| \begin{array}{l} 0.6h = 0.6(166) = \boxed{99.6\text{ ft}} \\ z_{min} = 30\text{ ft} \end{array} \right. \quad (6.5.8.1)$$

$$\bar{V}_z = 0.45\left(\frac{99.6}{33}\right)^{1/4}(90)\left(\frac{88}{60}\right) = 78.3$$

$$L_{\bar{z}} = l\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} \quad (6\text{-}7)$$

$$\left.\begin{aligned}\bar{\epsilon} &= 1/3.0 \\ l &= 320 \text{ ft}\end{aligned}\right\} \text{Table 6-2}$$

$$L_{\bar{z}} = 320\left(\frac{99.6}{33}\right)^{1/3} = 462.45$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_z} = \frac{(0.437)(462.45)}{78.3} = 2.581 \quad (6\text{-}12)$$

$$R_l = \frac{1}{n} - \frac{1}{2n^2}\left(1 - e^{-2n}\right) \quad (6\text{-}13a)$$

for $n > 0$

$$R_l = 0 \text{ for } n = 0 \quad (6\text{-}13b)$$

$\underline{R_h:}$ $$n = \frac{4.6 n_1 h}{\bar{V}_{\bar{z}}} = \frac{4.6(0.437)(166)}{78.3} = 4.262$$

$$R_h = \frac{1}{4.262} - \frac{1}{2(4.262)^2}\left(1 - e^{-2(4.262)}\right) = 0.207$$

$\underline{R_B:}$ $$n = \frac{4.6 n_1 B}{\bar{V}_z} = \frac{4.6(0.437)(421.25)}{78.3} = 10.81$$

$$R_B = \frac{1}{10.81} - \frac{1}{2(10.81)^2}\left(1 - e^{-2(10.81)}\right) = 0.0882$$

$\underline{R_L:}$ $$n = \frac{15.4 n_1 L}{\bar{V}_z} = \frac{15.4(0.437)(285)}{78.3} = 24.5$$

$$R_L = \frac{1}{24.5} - \frac{1}{2(24.5)^2}\left(1 - e^{-2(24.5)}\right) = 0.04$$

$\underline{R_n:}$ $$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = \frac{7.47(2.581)}{(1 + 10.3(2.581))^{5/3}} \quad (6\text{-}11)$$

$$= 0.07656$$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)} \quad (6\text{-}10)$$

$$R = \sqrt{\frac{1}{0.02}(0.07656)(0.267)(0.0882)(0.53+0.47(0.04))} = 0.1958$$

$\beta = 2\%$ Ch 6 Commentary, pg 294

$$g_Q = g_v = \boxed{3.4}$$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.3\left(\frac{33}{99.6}\right)^{1/6} = 0.25 \quad (6\text{-}5)$$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{421.25+166}{462.45}\right)^{0.63}}} \quad (6\text{-}6)$$

$$= 0.76$$

$$G_f = 0.925\left(\frac{1+1.7(0.25)\sqrt{3.4^2(0.76)^2+(3.987)^2(0.1958)^2}}{1+1.7(3.4)(0.25)}\right)$$

$$G_f = 0.8123$$

6. Enclosure Classification: Enclosed (6.5.11.1)

7. $GC_{pi} = \pm 0.18$ (Fig. 6-5)

8. Wall Pressure Coeff. C_p

$L/B = 285/421.25 = 0.676$

$h/L = 166/285 = 0.582$

$\theta < 10°$

See excel table for all C_p values. (Figure 6-6)

9. Velocity Pressure q_{v_z}

See excel table for all values.

10. $q_{v_z} = 0.00256 K_z K_{zt} K_d V^2 I$ (6-15)

For 16 ft, Case 1: $= 0.00256(0.7)(1)(0.85)(90)^2(1.15) = 14.1886$ psf

Hand calculations for q_z verify excel value.

10. Design Wind Pressure p

For flexible buildings...

$$p = qG_fC_p - q_i GC_{pi} \quad (6\text{-}19)$$

See excel table for all values

For Windward direction at height h, 166 ft

$$p = qG_fC_p - q_i(GC_{pi})$$

Parapets

$$P_p = q_p GC_{pu} \quad (6\text{-}20)$$

$GC_{pu} = +1.5$ for windward
-1.0 for leeward

Parapet 1: Detail 30, A5.1.6
Height = 4 ft from mech roof

Parapet 4C: Detail 27, A5.1.44
Height = $2' - 2\,^{3/32}$" from mech roof

Parapet Green Roof: Detail 72, A5.2.4
Height = 5' from green roof

Parapet Type 8: Detail 69, A5.3.19
Height = 4', L Block from penthouse

Parapets
Mech Roof 1
Mech Roof 2
Green Roof

Parapets – MWFRC

$P_p = q_{v_p} G C_{pu}$ (6-20)

Parapet Types:

K_z values almost identical, combine to Mech Roof type

Mech Roof 1: 4 ft above mech roof
height = 170 ft
Detail 30 sheet A5.1.6

Mech Roof 2: 2'-2 3/32" above mech roof
height = 168'-2"
Detail 27 sheet A5.1.44

Green Roof: 5 ft above 4th floor
height = 54 ft
Detail 72/A5.2.4

Penthouse: 4 ft above mech penthouse
height = 147 ft
Detail 69/A5.3.19

1. Find K_z
   - Mech 1: $K_z = 1.15$
   - Mech 2: $K_z = 1.146$ ⇒ Use 1.15 for both
   - Penthouse: $K_z = 1.104$ (Figure 6-6)
   - Green Roof:

2. Calculation of q_{v_p}
   Mech Roof:
   $q_{v_p} = (0.00256)(1.15)(1)(0.85)(90)^2(1.15)$
   $= 23.3099$ psf ⇒ Matches excel
   See Excel for all q_{v_p} values.

3. Calculation of Design Wind Pressure
   Mech Roof – Windward
   $P_p = (23.3099)(1.5) = 34.96$ psf
   See excel for all q_{v_p} values

Wind Loads - Direction 2

New G_f Value - see pages 1-3 for additional calcs

$g_R = 3.987$

$\bar{z} = 99.6\text{ ft}$

$\bar{V}_{\bar{z}} = 0.45\left(\frac{99.6}{33}\right)^{1/4}(90)\left(\frac{88}{60}\right) = 78.3$

$L_{\bar{z}} = 462.45$

$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = 2.581$

$\underline{R_h}$: 0.207 → dependent on height

$\underline{R_B}$: $\eta = \frac{4.6 n_1 B}{\bar{V}_{\bar{z}}} = \frac{4.6(0.437)(285)}{78.3} = 7.317$

$R_B = \frac{1}{7.317} - \frac{1}{2(7.317)^2}\left(1 - e^{-2(7.317)}\right) = 0.1273$

$\underline{R_L}$: $\eta = \frac{15.4 n_1 L}{\bar{V}_{\bar{z}}} = \frac{15.4(421.25)(0.437)}{78.3} = 36.206$

$R_L = \frac{1}{36.206} - \frac{1}{2(36.206)^2}\left(1 - e^{-2(36.206)}\right) = 0.0272$

$\underline{R_n}$: $R_n = 0.07656$ → dependent on N_1

$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$ (6-10)

$R = \sqrt{\frac{1}{0.02}(0.07656)(0.207)(0.1273)(0.53 + 0.47(0.0272))}$

$R = 0.2340$

$g_Q = g_v = 3.4$

$I_{\bar{z}} = 0.25$

$$Q=\sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_z}\right)^{0.63}}}=\sqrt{\frac{1}{1+0.63\left(\frac{285+166}{462.45}\right)^{0.63}}}=0.7856$$

$$G_f=0.925\left(\frac{1+1.7(0.25)\sqrt{(3.4)^2(0.7856)^2+(3.987)^2(0.2340)^2}}{1+1.7(3.4)(0.25)}\right)$$

$$\boxed{G_f=0.8332}$$

New Wall Pressure Coeff. C_p

$L/B = 421.25/285 = 1.4781$

$h/L = 166/421.25 = 0.3941$

- See excel table for all C_p values. (Figure 6-6)
- Leeward Wall interpolation between 0-1 and 2 values of C_p.

Velocity Pressure q_z and Design Wind Pressure
↳ See excel

4 | Seismic Loads

The following section calculates seismic loads for The Health Centre using the Equivalent Lateral Force (ELF) method provisions from ASCE 7-05 chapters 11 and 12.

Design Criteria from Structural Dwgs

$S_s = 0.228g$
$S_1 = 0.086g$
$F_a = 1.2$ * Structure not exempt under 11.1.2
$F_v = 1.7$
$S_{DS} = 0.18g$
$S_{D1} = 0.10g$
Occupancy Category IV
Importance Factor = 1.5
SDC = C
Site Class = C

1) Bed Tower Seismic Loads

Lateral System - Intermediate Reinforced Concrete Moment Frames

$R = 5$ (table 12-2-1)
$\Omega = 3$
$C_d = 4\frac{1}{2}$
Perimitted in SDC C

$S_{MS} = F_a S_s$ (11.4-1)
$= (1.2)(0.228g) = 0.2736g$

$S_{M1} = F_v S_1$ (11.4-2)
$= (1.7)(0.086g) = 0.1462g$

$S_{DS} = \frac{2}{3} S_{MS}$ (11.4-3)
$= \frac{2}{3}(0.2736g) = 0.1824g$

$S_{D1} = \frac{2}{3} S_{M1}$ (11.4-4)
$= \frac{2}{3}(0.1462g) = 0.09747g$

Note: h will be considered the full height of the building above and below grade for seismic calculations, h = 213.3 ft

Seismic Design Response Spectrum

Fundamental Period $T = C_t h_n^x$ (12.8-7)

$C_t = 0.028$, $x = 0.8$ (Table 12.8-2)

$T = 0.028(213.3\text{ ft})^{0.8} = 2.043\text{ s}$

$T_o = 0.2\frac{S_{D1}}{S_{DS}} = 0.2\frac{0.09747}{0.1824} = 0.1069\text{ s}$ (11.4-7)

$T_S = \frac{S_{D1}}{S_{DS}} = \frac{0.09747}{0.1824} = 0.5344\text{ s}$ (11.4-7)

Long transition period $T_L = 12\text{ s}$ (Fig 22-15)

Equivalent Lateral Force Procedure (12.8)

* permitted under table 12.6-1

Seismic Response Coefficeint

$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{0.1824}{\left(\frac{5}{1.5}\right)} = 0.05472$ (12.8-2)

⇒ Need not exceed...

$C_s = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.09747}{2.043\left(\frac{5}{1.5}\right)} = \boxed{0.01431}$ (12.8-3)

Vertical Distribution of forces - 12.8.3

$F_x = C_{vx}V$ (12.8-11)

$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$ (12.8-12), where k = linear interpolat between $T = 0.5\text{ s}$, 1; $T = 2.5\text{ s}$, 2; for $T = 2.0$...

$V = C_s W$ $k = 1.7715$

Total Effective Seismic Weight

1) Penthouse Roof – 18,522 ft^2

Area Loads = (40 psf)(18522 ft^2) = 740.9 k
Perim Loads = (16 psf)(6.4 ft)(420 ft) = 43.0 k
W = 740.9 k + 43.0 k = 783.9 k

2) Penthouse Level – 35,430 ft^2 + 983' perim

Area Loads = $(120\ psf_{penthouse})(18522\ ft^2) + (83\ psf_{roof})(35{,}430-18522)$ = 3626.0
Perim Loads = 16 psf(6.4)(420) + 16(7.5)(983) = 161.0 k
Mechanical Equip. = 200 k
W = 3626 + 161 + 200 = 3987 k

3) Levels 7-9 – 35,430 ft^2 + 983' perim

Area Loads = (120 psf)(35,430 ft^2) = 4251.6 k
Perim Loads = (16 psf)(15 ft)(983') = 235.9 k
W = 4251.6 k + 235.9 k = 4487.5 k

4) Levels 5-6 – 51,455 ft^2 + 1324' perim

Area Loads = (120 psf)(51,455 ft^2) = 6174.6 k
Perim Loads = (16 psf)(15 ft)(1324 ft) = 317.8 k
W = 6174.6 + 317.8 k = 6492.4 k
* Roof designed as floor for expansion

5) Level 4 + Green Roof – (GR = 12,177 ft^2), 59,228 ft^2 + 1388' perim

Area Loads = (120 psf)(59228 ft^2) + (103 psf)(12177 ft^2) = 8361.6
Perim Loads = (16 psf)(15 ft)(1388') = 333.1 k
Hospital Equip Allowance** = 80 k
W = 8361.6 k + 333.1 k + 80 k = 8774.7 k

* 5" slab labeled typical for all floors. Used value for seismic hand calcs.

** Levels 1-4 contain diagnostic equipment, allowance

6) Levels 1-3 – 57,245 ft^2 + 1,178.5 ft perim

Area Loads = (120 psf)(57,245 ft^2) = 6869.4 k

Perim Loads = (16 psf)(15 ft)(1178.5 ft) = 282.8 k

Hospital Equip. Allowance = 80 k

$w = 6869.4\text{ k} + 282.8\text{ k} + 80\text{ k} = \underline{7232.2\text{ k}}$

7) Parking Garage Levels – 57245 ft^2

(105 psf)(57245 ft^2) = $\underline{6010.7\text{ k}}$

$W = 783.9\text{ k} + 3987\text{ k} + 3(4487.5\text{ k}) + 2(6492.4\text{ k})$
$+ 8774.7\text{ k} + 3(7232.2\text{ k}) + 3(6010.7\text{ k})$

$W = 79721.6\text{ k}$

$V = C_s W = 0.01431\,(79721.6) = \boxed{1140.8\text{ k}}$

See excel for spreadsheet of story shears

Below in Table 4 are values for Seismic Story Shear V_x (12.8.4). The corresponding story and floor forces are depicted in the diagram in Figure 8.

Level	h_x (ft)	w_x (k)	k	$w_x h_x^k$	C_{vx}	F_x (k)	h_x*F_x (ft-k)
Penthouse Roof	213.3	783.9	1.7715	28587331.8	0.004986	5.688352	1213.3256
Penthouse Level	188.2	3987	1.7715	449950176	0.078482	89.5318	16849.884
Level 9	173.2	4487.5	1.7715	510591748	0.089059	101.5984	17596.835
Level 8	158.2	4487.5	1.7715	466371908	0.081346	92.79942	14680.868
Level 7	143.2	4487.5	1.7715	422152069	0.073633	84.00048	12028.869
Level 6	128.2	6492.4	1.7715	727049465	0.126814	144.6694	18546.623
Level 5	111.2	6492.4	1.7715	630638850	0.109998	125.4855	13953.989
Level 4	94.2	8774.7	1.7715	910933937	0.158888	181.2591	17074.605
Level 3	77.2	7232.2	1.7715	530047588	0.092452	105.4697	8142.2613
Level 2	62.2	7232.2	1.7715	427059067	0.074489	84.97689	5285.5624
Level 1	47.2	7232.2	1.7715	324070546	0.056525	64.48407	3043.648
P4	30.9	6010.7	1.7715	152870684	0.026664	30.41845	939.93011
P3	20.6	6010.7	1.7715	101913790	0.017776	20.27897	417.74671
P2	10.3	6010.7	1.7715	50956894.8	0.008888	10.13948	104.43668
P1	0	6010.7	1.7715	0	0	0	0
						1140.8	**129878.58** ft-k

Table 4 | Seismic Story Forces

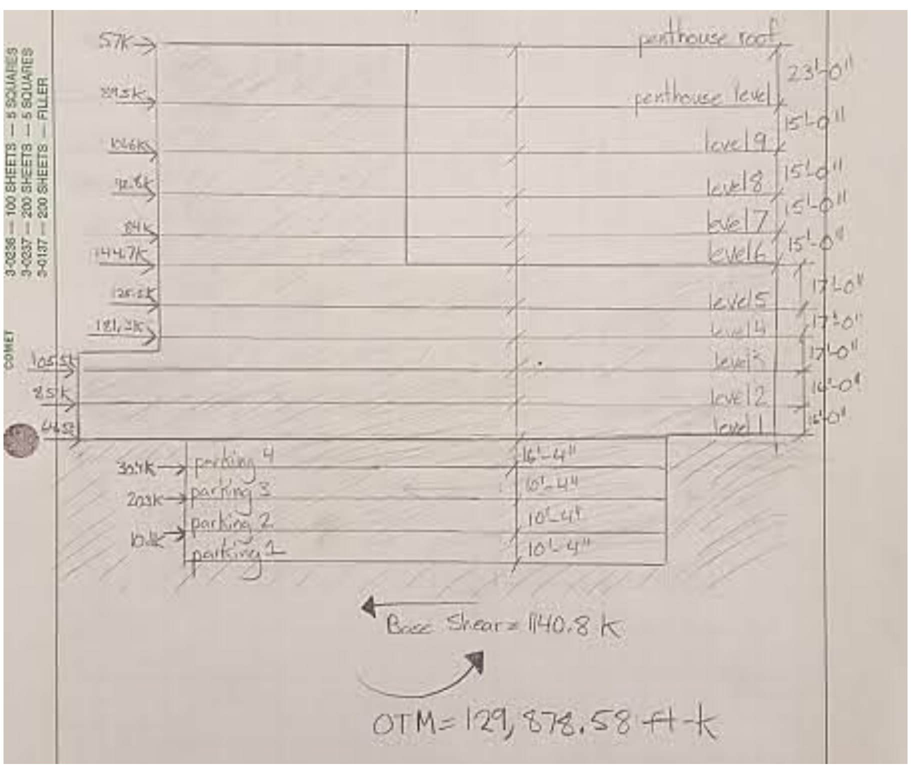
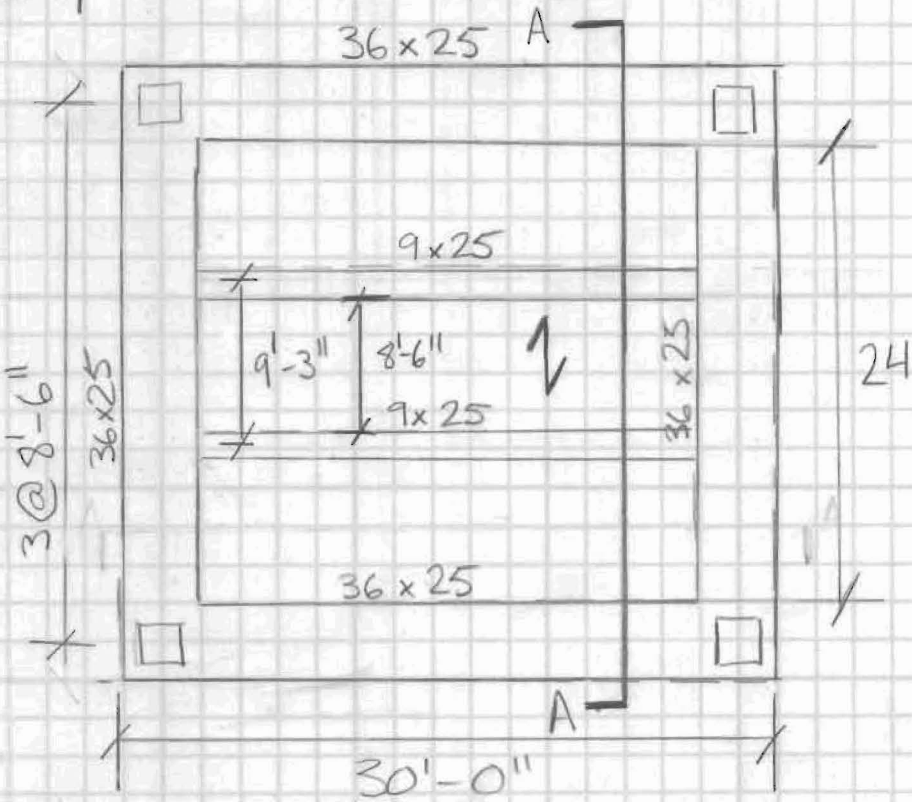


Figure 8 | Seismic Story Forces Diagram

6 | Typical Spot Checks for Gravity Loads

The following section analyzes the existing gravity framing system of the Health Centre. The existing system is a one-way cast-in-place concrete slab with intermediate concrete beams. Framing members and slab were analyzed for flexural and shear capacity.

One-Way Slab w/ Intermediate Beams



One-way Slab
- 5" thick
- Top Bars: #3 @ 12"
- Bottom Bars: #3 @ 12"

] 3/4" cover

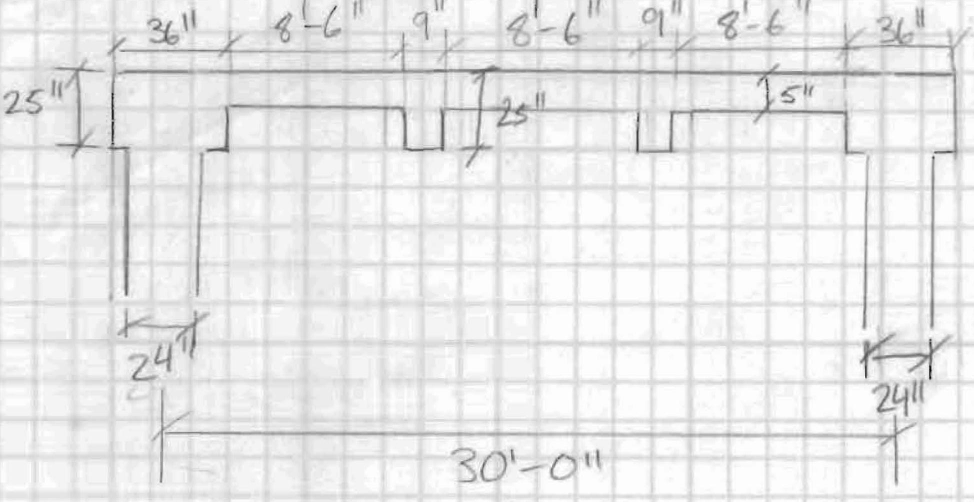
- Temp Shrinkage: #4 @ 18"

Floor Dead: 86 psf (includes slab self-weight)
Floor Live: 80 psf

Spot Check: 1-Way Slab

- Min thickness $= \ell/28 = \frac{30' \times 12 \text{ in/ft}}{28} = 12.85" > 5"$

$\therefore$ Must calculate deflections (Table 9.5(a) ACI)



One bay Cross Section

Load Combinations

$1.4D = 1.4(86\text{ psf}) = 120.6\text{ psf}$

$1.2D + 1.6L = 1.2(86\text{ psf}) + 1.6(80\text{ psf}) = \boxed{231\text{ psf}}$

For 1 ft of slab section: $w = 231\text{ plf}$

Max Moment will be @ exterior face of first interior support: $M_u = \frac{w_u l_n^2}{10}$ (ACI 8.3.3)

$$M_u = \frac{(231\text{ plf})(8.5\text{ ft})^2}{10} = \underline{1.67\text{ ft-k/ft}}$$

$$d = 5" - 0.75" - \frac{0.375"}{2} = 4.06"$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.11\text{ in}^2)(60)}{0.85(5)(12\text{ in})} = 0.13\text{ in}$$

Moment Capacity

$$\phi M_n = \phi f_y A_s \left(d - \frac{a}{2}\right) = 0.9(60)(0.11\text{ in}^2)(4.06 - 0.13)$$

$$= 23.3\text{ in-k} = \underline{1.94\text{ ft-k}}$$

$1.94\text{ ft-k/ft} > 1.67\text{ ft-k/ft}$ <u>OK</u>

Shear Capacity:

$$V_u = \frac{1.15 w_u l_n}{2} = \frac{1.15(231\text{ lb/ft})(8.5\text{ ft})}{2} = 1.13\text{k}$$

(ACI 8.3.3)

$V_c = 2\lambda\sqrt{f'_c}\, b_w d = 2(1)\sqrt{5000}(12\text{ in})(4.06'') = 6.89\text{ k}$

$\phi = 0.75,\ \phi V_c = 5.17\text{ k} > 1.13\text{ k}$ OK

Min Steel Check:

$A_{s,min} = 0.0018bh = 0.0018(12'')(5'') = 0.108\text{ in}^2 < 0.11\text{ in}^2$
OK

Max Spacing Check:

$$S_{max} = \min \begin{cases} 15\left(\frac{40{,}000}{40{,}000}\right) - 2.5(0.75) = 13.1\text{ in} \\ 12\left(\frac{40{,}000}{40{,}000}\right) = \boxed{12\text{ in}} \\ 18'' \end{cases}$$

(ACI 10.6.4)

$f_s = 2/3 f_y = 40{,}000\text{ psi}$

$\boxed{\therefore \#3 @ 12'' \text{ top + bottom flexural reinf. checks}}$

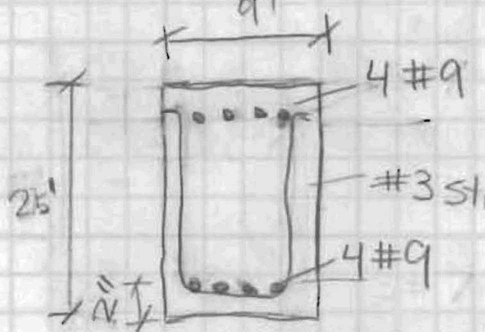
Shrinkage Reinf:

$A_s = 0.0018bh = 0.108\text{ in}^2$

$$S_{max} = \min \begin{cases} 5h = 25'' \\ \boxed{18''} \end{cases}$$

$\boxed{\therefore \#4 @ 18'' \text{ temp shrinkage reinf. checks}}$

Spot Check - 9x25 Interior Beam FB231



Live = 80 psf

Dead = 86 psf + bm wt.

$w_u = 1.2\left[8.5(86) + 150 \cdot \frac{(20.9)}{144}\right] + 1.6(72.8)(8.5)$

$w_u = 2.09$ klf

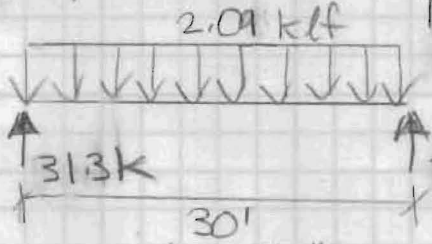
$M_u = \frac{(2.09)(30)^2}{8} = 235$ ft-k

$d = 25'' - 2 - \left(\frac{1.128}{2}\right) = 22.5''$

Can reduce LL?

$K_{LL} A_T = 2(8.5)(30) = 510 \text{ ft}^2 > 400 \text{ ft}^2$ ∴ Yes

$L_{red} = \max \begin{cases} 0.5 L_o \\ 0.25 + \frac{15}{\sqrt{K_{LL} A_T}} = 0.91(80) = 72.8 \end{cases}$



$l_n = 8'\text{-}6''$, $l = 9.3$ ft

Flexural Strength:

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(60)(4 \text{ in}^2)}{0.85(5)(9'')} = 6.27''$ $\quad c = \frac{6.27}{0.8} = 7.84''$

$\beta_1 = 0.8$

$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right) / 12''/\text{ft}$

$= 0.9(4 \text{ in}^2)(60)\left(22.5 - \frac{6.27}{2}\right) / 12 = 348 \text{ ft-k} > 235 \text{ ft-k}$ OK

steel yielding?

$\varepsilon_s = \varepsilon_c \frac{d-c}{c} = \frac{0.003}{7.84}(22.5 - 7.84) = 0.0056 > 0.00207$ ✓

∴ Beam ok for flexure

Steel Area Req.

$A_{s,min} = \frac{3\sqrt{f'_c}\, b_w d}{f_y} \geq \frac{200 b_w d}{f_y} = \frac{200(9)(22.5)}{60{,}000} = 0.675 \text{ in}^2$

$= \frac{3(\sqrt{5000})(9)(22.5)}{60{,}000} = 0.0795 \text{ in}^2$

$4 \text{ in}^2 > 0.675 \text{ in}^2$ OK

$A_{s,max}$: $\rho = \frac{A_s}{A_{conc}} = \frac{4 in^2}{9 \times 25} = 0.01778$

$\rho_{max} = 0.0213$ for $\beta_1 = 0.80$, $f'_c = 5000$, $f_y = 60$ ksi

$0.0213 > 0.01778$ <u>OK</u>

<u>Min Spacing</u>

$$S_{min} = \max \begin{cases} d_b = 1.128'' \\ 1'' = 1'' \\ 4/3\, S_a = \underline{4/3''} \text{ controls} \end{cases}$$

$$\text{Spacing} = \frac{9 - 2(1.5) + 2(0.5) - 4(1.128)}{3} = 0.163''$$

∴ Does not meet min spacing req.

<u>Shear Strength:</u>

<u>Note:</u> All beams sized for lateral + gravity loads by Walter P. Moore. This will affect design values matching calculated values.

V_u @ distance d from support

$$V_u = 31.3k \left[-\left(\frac{22.5}{2} \,/\, 12 in/ft\right)\right] = 29.3 \ k$$

$$V_c = 2\lambda\sqrt{f'_c}\, b_w d = 2(1)\sqrt{5000}\,(9)(22.5) = 28.6k$$

$$\frac{V_u}{\phi} = \frac{29.3}{0.75} = 39.1\ k > 28.6k$$

∴ Shear stirrups needed

$$V_{s,req} = 39.1 - 28.6 = 10.5\ k$$

$$V_s = 8\sqrt{f'_c}\, b_w d = 8\sqrt{5000}\,(9)(22.5) = 114.5k \geq 10.5k$$

<u>Ok</u>

$V_s \le 4\sqrt{f'_c}\, b_w d$

$10.5 \le 4\sqrt{5000}\,(9)(22.5) = 57.3\text{k}$ ✓

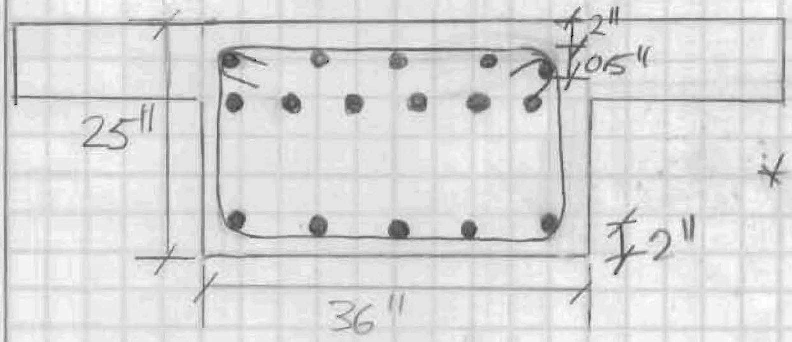
$S_{max} = \Big|_{min} \begin{array}{l} d/2 = 22.5/2 = 11.25'' > 11'' \ \underline{\text{OK}} \\ 24'' \end{array}$

Minimum Shear Reinforcing

$A_{v,min} = \Big|_{max} \begin{array}{l} 0.75\sqrt{5000}\,(9)(11)/60000 = \underline{0.087\ in^2} \\ 50(9)(11)/60000 = 0.082 \end{array}$

#3 stirrups at 11'' Checks ✓

Spot Check - 36 x 25 Interior Girder 3B39

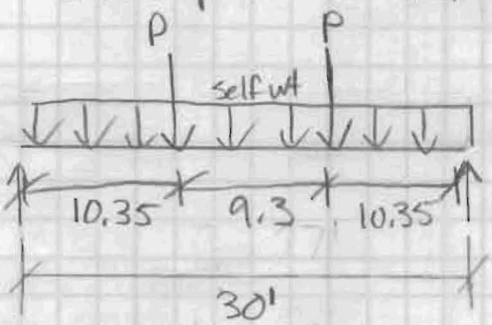


* Note: All concrete beams have extra reinf. because designed for lateral and gravity.

Top Bars: 5-#6, 6-#8 $\quad A_s = 6(0.79\,in^2) + 5(0.44\,in^2) = 6.94\,in^2$

Bottom Bars: 5-#8 $\quad A_s = 5(0.79\,in^2) = 3.95\,in^2$

Stirrups: #3 S27 $\quad$ 1@2/10@5/R@10



$$P_D = \left(8.5(86) + \frac{150}{\frac{(20.9)}{144}}\right)30 = 25.5\,k$$

$$P_L = (8.5)(30)(48) = 12.2\,k$$

$$l_n = (30 \times 12) - 36'' = 324'' = 27'$$

$$L_{red}\ \begin{cases} 0.5 \\ 0.25 + \dfrac{15}{\sqrt{2(30)^2}} = 0.6(80) = 48\,k \end{cases} \quad max$$

Flexural Strength:

$$P_u = 1.2(25.5\,k) + 1.6(12.2\,k) \qquad W_D = 150\,\frac{lb}{ft^3} \times \left(\frac{36 \cdot 20}{144}\right) = 0.750\,klf$$

$$P_u = 50.1\,k$$

$$w_u = 1.2(0.750\,klf) = 0.9\,klf$$

See following pages for M_u and V_u value determination

$$M_{u,max} = 619.8\,ft\text{-}k$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(3.95\,in^2)(60)}{0.85(5)(36)} = 1.55'' \qquad c = \frac{1.55}{0.8} = 1.94''$$

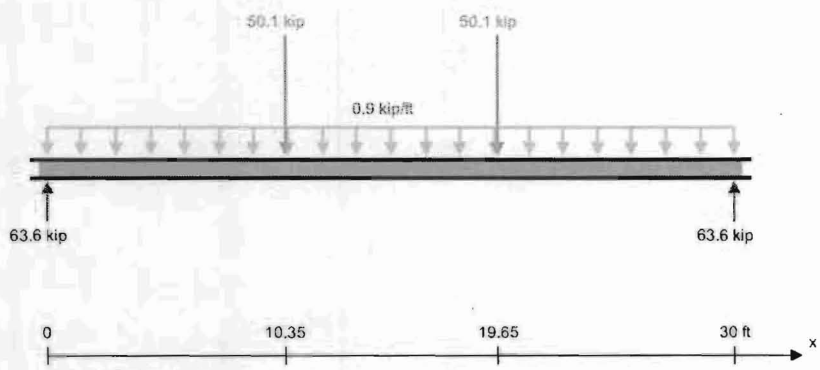
$$\phi M_{n_1} = 0.9(3.95)\left(22.5 - \frac{1.55''}{2}\right)(60) = 386\,ft\text{-}k$$

$\therefore$ Look at doubly reinf. section...

8

The FREE Shear & Moment Calculator has solved 748,407 bending moment diagrams and counting...

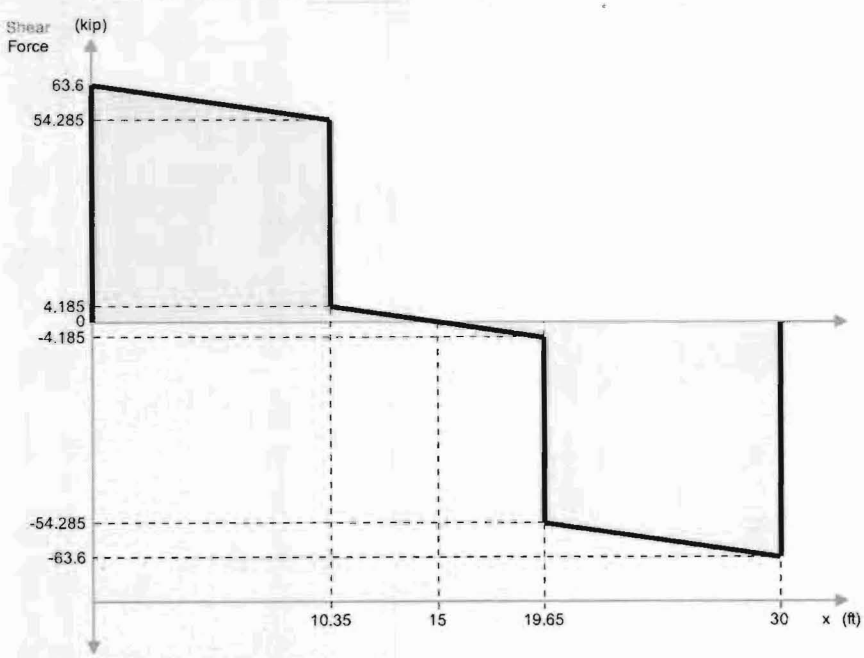
Free Body Diagram (FBD)



Full Working/Hand Solution for Reaction Forces

Shear Force Diagram (SFD)

The Shear Force (V) at ft along the beam is: (No Location Entered)

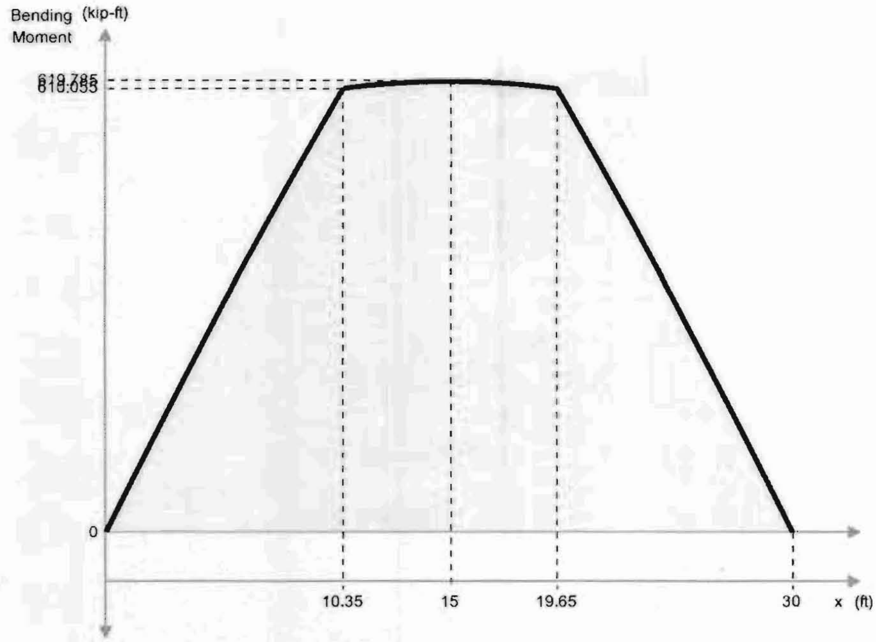


Full Working/Hand Solution for Shear Force Diagram

Bending Moment Diagram (BMD) Reverse BMD Sign Convention

The Bending Moment (M) at ft along the beam is: (No Location Entered)

9



Full Working/Hand Solution for Bending Moment Diagram

RETURN TO THE CALCULATOR

Think these answers are wrong? REPORT A BUG!

SAVE DATA

SAVE RESULT TO IMAGE FIL

Do you have any suggestions to make the calculator better? Please let us know on our feedback pages by clicking here.

$d' = 2'' + 0.75'' + \frac{0.5''}{2} = 3''$

$A_{s2} = 6.94 \text{ in}^2$

$M_{n2} = (6.94 \text{ in}^2)(60)(22.5 - 3 \cdot \frac{1}{12}) = 676.7 \text{ ft-k}$

$M_n = 386 \text{ ft-k} + 676.7 \text{ ft-k} = 1063 \text{ ft-k}$

$\phi M_n = 0.9(1063) = 956 \text{ ft-k}$
$\quad\quad 0.75(1063) = 797 \text{ ft-k}$ $> 619.8 \text{ ft-k}$

Check Strains...

$\varepsilon_s = \frac{0.003(22.5 - 1.94)}{1.94} = 0.0318$

$\varepsilon_s' = \frac{0.003(3 - 1.94)}{1.94} = 0.002$

Steel close to
not yielding, some reinf. not flexural

<u>Steel Area Req'</u>

$A_{s,min} = \frac{3\sqrt{f'_c}\, b_w d}{f_y} \geq \frac{200\, b_w d}{f_y}$

$= \frac{3\sqrt{5000}(36)(25)}{60,000} \geq \frac{200(36)(25)}{60,000}$

$= 3.18 \text{ in}^2 \geq 3 \text{ in}^2$

<u>OK</u>

$A_{s,max}$: $\rho = \frac{A_s}{A_c} = \frac{6.94 + 3.95}{36 \times 25} = 0.0121$

$\rho_{max} = 0.0213 > 0.0121$ <u>Ok</u>

Min Spacing

$$S_{min} = \begin{cases} d_b = 1'' \\ 1'' = 1'' \\ 4/3\, S_a = 4/3'' \end{cases}$$

$$\text{Smallest Spacing} = \frac{36 - 2(1.5) - 2(0.5) - 6(1'')}{5} = 5.2''$$

OK

Shear Strength:

V_u @ distance d from support

Slope of shear from previous diag...

$$\frac{63.6 - 54.285}{10.35} = 0.89,\quad d = 22.5\text{ in} = 1.875\text{ ft}$$

$$V_u = 63.6 - (0.89)(1.875\text{ ft}) = 61.9\text{ k}$$

$$V_s = 8\sqrt{f'_c}\, b_w d = 8\sqrt{5000}(36)(22.5) = 458.2\text{k} > 61.9\text{k}$$

∴ Shear stirrups not required, will check spacing of #3 stirrups

$V_s \leq 4\sqrt{f'_c}\, b_w d$?

$$\leq 4\sqrt{5000}(36)(22.5) = 229\text{ k} ✓$$

$$S_{max} = \begin{cases} d/2 = 22.5/2 = 11.25'' > 10'' \text{ OK} \\ 24 \end{cases}$$

#3 stirrups w/ max spacing @ 10'' checks ✓

Typical Column D12

2 lower parking levels: 28 x 38 w/ 22 #9
2 upper parking levels: 28 x 38 w/ 14 #8
levels 1-4: 28 x 32 w/ 12 #8
levels 4-9: 24 x 24 w/ 8 #8
level 9 - pent roof: 24 x 24 w/ 12 #9

Column Loads

➡ See following excel. Note that roof, penthouse, and parking live loads are not reduced.

Live load reduction calc: 9th floor

$$L = 80 \times \left| \begin{matrix} 0.4 \\ 0.25 + \frac{15}{\sqrt{4(900)}} \end{matrix} \right._{max} = 40 \text{ psf}$$

8th floor

$$L = 80 \times \left| \begin{matrix} 0.4 \\ 0.25 + \frac{15}{\sqrt{4(\underbrace{2 \cdot 900}_{2 \text{ floors}})}} \end{matrix} \right._{max} = 32 \text{ psf}$$

See excel for all values. No reducible LL floor has column supporting 1 floor, $0.5L_o$ not a limit.

Snow Loads do not control for typical column.

Axial loads $= (1.2D + 1.6L)(900 \text{ ft}^2)$ per floor

Roof trib area $= (15 + 8.333)(30) = 700 \text{ ft}^2$

See excel for all axial values

$$\text{Self-Weight} = \frac{150 \text{ pcf}}{1000}\left(\frac{28 \times 38}{144}(47.3) + \frac{28 \times 32}{144}(49) + \frac{24 \times 24}{144}(117.8)\right) = 169 \text{ k}$$

Controlling Case $= 1.2D + 1.6L + 0.5L_r$

$$P_u = 1.2(1188 + 169) + 1.6(576.13) + 0.5(14) = \boxed{2557.2 \text{ k}}$$

TOTAL VALUES FROM EXCEL

Column Strength Check

$$\phi P_n = 0.8\phi[0.85 f'_c(A_g - A_s) + f_y A_s)]$$

$f'_c = 5000$ psi
$\phi = 0.65$
$A_g = 28'' \times 38'' = 1064 \text{ in}^2$
$A_s = 22(1 \text{in}^2) = 22 \text{ in}^2$

$$\phi P_n = 0.8(0.65)[0.85(5000)(1064-22) + 60{,}000(22)]$$
$$\phi P_n = 2989 \text{ K} > 2557.2 \text{ K} \quad \underline{\text{OK}}$$

* Note: All columns were designed for lateral loads and gravity loads, and should be over capacity.

Interior Column D12

Level	Dead (psf)	Live (psf)	Red. Live (psf)	Total Axial Load (K)		
				Dead	L or L_r	1.2D+1.6L+.5L_r
Penthouse Roof	40	20	20	36	13.9998	50.200
Penthouse	86	150	150	113.4	135.000	359.080
Level 9	86	80	40	190.8	171.000	509.560
Level 8	86	80	34.142	268.200	201.728	651.605
Level 7	86	80	32	345.6	230.528	790.565
Level 6	86	80	32	423	259.328	929.525
Level 5	86	80	32	500.4	288.128	1068.485
Level 4	86	100	40	577.8	324.128	1218.965
Level 3	86	100	40	655.2	360.128	1369.445
Level 2	86	100	40	732.6	396.128	1519.925
Level 1	86	100	40	810	432.128	1670.405
Parking 1	105	40	40	904.5	468.128	1841.405
Parking 2	105	40	40	999	504.128	2012.405
Parking 3	105	40	40	1093.5	540.128	2183.405
Parking 4	105	40	40	1188	576.128	2354.405
					Axial + 1.2 Self Wt.	**2557.205**

Trib Area =	900	ft^2
K_{LL} =	4	
Roof Trib=	700	ft^2
Self-Weight=	169	K

Note: L_r excluded from total axial live load total and added as .5 L_r to third column.

Exterior Column C13.5

4 lower parking levels: 28 x 38 w/ 14 #8
level 1-2: 24 x 24 w/ 16 #9
levels 2-9: 24 x 24 w/ 8 #8
levels 9 - pent. roof: 24 x 24 w/ 12 #9

$f'_c = 5000$ psi

Column Loads:

Trib Area varies by level

Pent roof: $15' \times 15' = 225\ ft^2$
Penthouse: $(15' \times 15') + \left(\frac{7.5' \times 15'}{2}\right) = 281.25\ ft^2$
Level 9-5: $281.25\ ft^2$
Level 4-1: $(15 \times 15) + (7.5 \times 15) = 337.5\ ft^2$

Parking 4: $(5' + 7.5') \times 30' = 375\ ft^2$
Parking 3-1: $375 - (15' \times 15') = 150\ ft^2$

Live Load Reduction:

$K_{LL} = 3$ due to cantilevered slabs

Can reduce LL for all levels $K_{LL} A_T = 3(150) > 400\ ft^2$

↳ Will not reduce roof, penthouse, parking

At level 9

$$L = 80 \times \left| \begin{matrix} 0.4 \\ 0.25 + \frac{15}{\sqrt{3(281.25)}} \end{matrix} \right._{max} = 0.766(80) = 61.3\ psf$$

See excel for all values

Perimeter Dead Load:

Panel System = 16 psf

Perim. Levels PR-5: $15 + 7 + 6 + 7.5 = 35.5$ ft
4-1: 22.5 ft

panel nominal thickness

$$DL = 16\left(6(35.5) + 3(22.5)\right)\left(\frac{36''}{12''/ft}\right) = 13.46\ k$$

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

Self-Weight

$$\frac{150\text{ pcf}}{1000}\left(\frac{28\times38}{144}(47.3)+\frac{24\times24}{144}(49+117.8)\right)=152.5\text{k}$$

Controlling Case = $1.2D + 1.6L + 0.5L_r$

$$P_u = 1.2(356.85+13.46+152.5)+1.6(198)+0.5(45)$$
$$=\boxed{946.4\text{k}}$$

Column Strength Check

$$\phi P_n = 0.8\,\phi\left[0.85 f'_c(A_g - A_s) + f_y A_s\right]$$

$A_g = 28\times38 = 1064 \text{ in}^2$
$A_s = 14\times0.79 = 11.06 \text{ in}^2$

$$\phi P_n = 0.8(0.65)\left[0.85(5000)(1064-11.06)+(60)(11.06)\right]$$
$$\phi P_n = 2327\text{ k} > 946.4\text{k} \quad \underline{\text{ok}}$$

Exterior Column C13.5

Level	Dead	Live	Reduced Live	Trib Area	Axial Load (K)		
					Dead - Area	L or L_r	1.2D+1.6L+.5L_r
Penthouse Roof	40	20	20	225	9	4.500	13.050
Penthouse	86	150	150	281.25	33.1875	42.188	109.575
Level 9	86	80	61.312	281.25	57.375	59.431	166.190
Level 8	86	80	49.212	281.25	81.5625	73.272	217.361
Level 7	86	80	43.851	281.25	105.75	85.605	266.119
Level 6	86	80	40.656	281.25	129.9375	97.040	313.439
Level 5	86	80	38.475	281.25	154.125	107.861	359.778
Level 4	86	100	44.245	337.5	183.15	122.794	418.500
Level 3	86	100	42.817	337.5	212.175	137.245	476.452
Level 2	86	100	41.667	337.5	241.2	151.307	533.782
Level 1	86	100	40.713	337.5	270.225	165.048	590.597
Parking 1	105	40	40	375	309.6	180.048	661.847
Parking 2	105	40	40	150	325.35	186.048	690.347
Parking 3	105	40	40	150	341.1	192.048	718.847
Parking 4	105	40	40	150	356.85	198.048	747.347
						Axial + 1.2(Perim + SW)	**747.347**

Perimeter DL=	13.46	K
K_{LL} =	3	
Self-Weight=	153	K

Note: L_r excluded from total axial live load total and added as .5 L_r to third column.

7 | Alternative Framing Systems for Gravity Loads

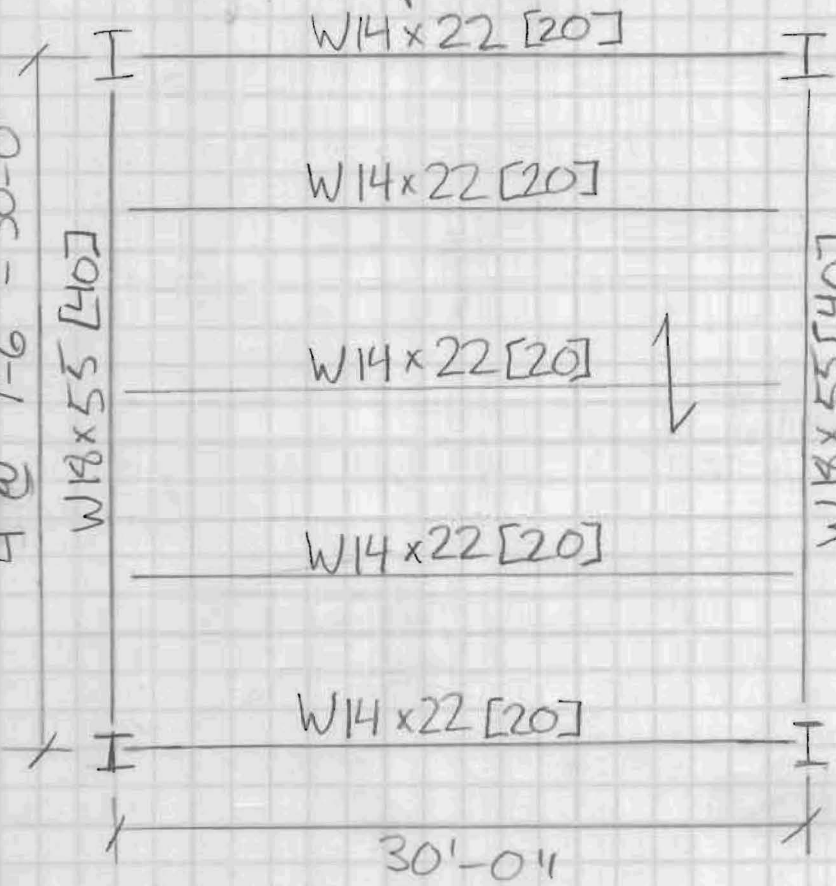
Three alternative gravity systems were explored for the existing building loads. The systems under consideration in this report are:

- Alternative 1: Composite Wide-Flange Steel
- Alternative 2: Two-way Flat Slab
- Alternative 3: Non-Composite Steel Joists

All gravity framing systems maintained the 30'-0" x 30'-0" bay size of the existing structure. Structural systems were evaluated based on strength and serviceability.

7.1 Alternative 1: Composite Wide-Flange Steel

Alternative 1: Composite Steel



4" Deck: 1.5VLI20 with 2½ NWC topping

1) Composite Decking

- 2 hr fire-rating req'd
- Super Imposed Dead:

Finishes + Ceiling	5 psf
MEP	15 psf
Misc	3 psf
	23 psf

- Live Load: 100 psf

W_{Total} = 123 psf

$\boxed{\text{Try 1.5VLI20 deck w/ 2½ NWC topping}}$

- Max3 span unshored: 7'-11" > 7'-6" ∴ OK
- SDL = 217 psf < 123 psf ∴ OK
- Slab weight = 39 psf

2) Infill Beams

Dead: 23 + 39 + 5 (self-weight) = 67 psf
Live: 100 psf

- Unshored Strength

$1.4D = 1.4(67 \text{ psf}) = 93.8 \text{ psf}$

$1.2D + 1.6L = 1.2(67) + 1.6(20) = 112.4 \text{ psf}$

$w = 7.5 \text{ ft}(112.4 \text{ psf}) = 843 \text{ plf}$

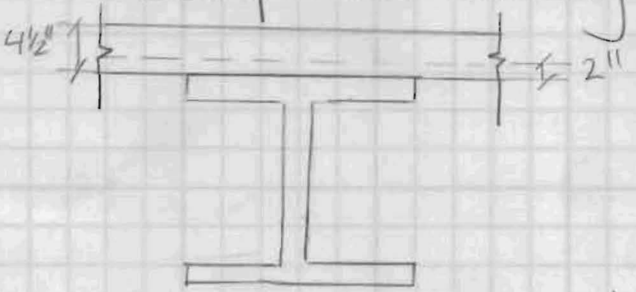
$$M = \frac{wl^2}{8} = \frac{(0.843)(30)^2}{8} = 94.8 \text{ k-ft}$$

W14x22: $\phi M_p = 125$ ft-k

W12x30: $\phi M_p = 162$ ft-k > 94.8 ft-k

W16x26: $\phi M_p = 166$ ft-k

- Composite Strength



Live Load Reduction

$K_{LL} = 2$

$A_T = (7.5)(30) = 225 \text{ ft}^2$

(eq 4-1) $L = 100 \times \left| \begin{matrix} 0.5 \\ 0.25 + \frac{15}{\sqrt{2(225)}} \end{matrix} \right._{max} = 0.9571$

$L = 95.7 \text{ psf}$

Load Combos

$1.4D = 93.8 \text{ psf}$

$1.2D + 1.6L = 1.2(67 \text{ psf}) + 1.6(95.7 \text{ psf}) = \underline{233.5 \text{ psf}}$

$w = (233.5 \text{ psf})(7.5 \text{ ft}) = 1.751 \text{ klf}$

$$M = \frac{(1.751)(30)^2}{8} = \underline{197 \text{ k-ft}}$$

$$b_{eff} = \min \left| \begin{array}{l} 3.75\text{ ft} \times 12\text{ in/ft} \\ 30\text{ ft} \times 12\text{ in/ft}/8 \end{array} \right. \times 2 = 90''$$

Assume $a = 1''$

$Y2 = 4'' - \frac{1}{2}'' = 3.5''$

From Table 3-19:

W14x22: $\Sigma Q_n = 157k$

W12x30: $\Sigma Q_n = 136k$

W14x26: $\Sigma Q_n = 96k$

Economy

$\frac{157k}{17.2} = 9.13$ $2(10 \times 10) + 22(30) = 860$ lbs

$\frac{136}{17.2} = 7.91$ $2(8 \times 10) + 30(30) = 1060$ lbs

$\frac{96}{17.2} = 5.58$ $2(6 \times 10) + 26(30) = 900$ lbs

$\boxed{\therefore \text{ Use W14x22 with 20 studs/beam}}$

- Check a Assumption

$V_c = (3.5'')(90'')(4\text{ ksi})(0.85) = 1071\text{ k}$

$V_s = (50\text{ ksi})(6.49\text{ in}^2) = 324.5\text{ k}$

20 studs $\times$ 17.2 k/stud = 344 k

$$a = \frac{344}{0.85(3.5)(90)}$$

$a = 1.28'' > 1''$ $\therefore$ a assumption wrong

- Assume $a = 2''$, $Y2 = 3''$ $\phi M_n = 200$ ft-k

200 ft-k > 197 ft-k

$\Sigma Q_n = 157$ k still

a assumption <u>OK</u>

- Wet Conc Deflection

$W = (39 \text{ psf})(7.5 \text{ ft}) = 292.5 \text{ plf}$

$$\Delta = \frac{5(0.2925)(30)^4(1728)}{384(29000)(199 \text{ in}^4)} = 0.924$$

- Live Load Deflection

$W_{LL} = (95.7 \text{ psf})(7.5 \text{ ft}) = 717.8 \text{ plf}$

$I_{LB} = 426 \text{ in}^4$

$$\Delta_{LL} = \frac{5(0.7178)(30)^4(1728)}{384(29000)(426)} = 1''$$

$$\frac{l}{360} = \frac{30 \times 12}{360} = 1'' = 1'' \underline{\text{OK}}$$

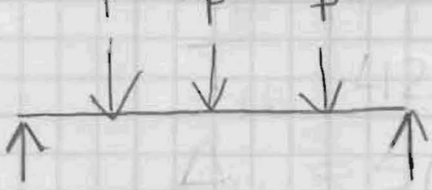
3) Girders

$P_D = (23 + 39 + 7\ psf)(7.5\ ft)(30\ ft) = 15.5\ k$

$P_L = (60.4\ psf)(7.5\ ft)(30\ ft) = 13.6\ k$

LL Reduction

$L_{red} = 100 \times \left| \begin{array}{l} 0.5 \\ 0.25 + \frac{15}{\sqrt{2(30)^2}} = 0.604 \end{array} \right. = 60.4\ psf$

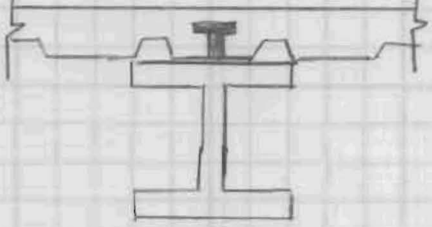


Load Combos

$1.4 P_D = 1.4(15.5\ K) = 21.7\ k$

$1.2 P_D + 1.6 P_L = 1.2(15.5) + 1.6(13.6\ k) = 40.4\ k$

$M_{max} = \frac{Pl}{4} + Pa = \frac{40.4(30)}{4} + 40.4(7.5) = 606\ ft\text{-}k$



If had assumed distributed load...

$w = 30(1.2(23 + 39 + 7) + 1.6(60.4))$

$w = 5.38\ klf$

$M = \frac{wl^2}{8} = \frac{(5.38)(30)^2}{8}$

$M = 605.3\ ft\text{-}k$

∴ Can assume a distrib. load for further calc.

Flexural Design

$b_{eff} = \left| \begin{array}{l} 15\ ft \times 12\ in/ft \\ \frac{30\ ft \times 12\ in/ft}{8} \end{array} \right._{min} \times 2 = \underline{90''}$

$Y1 = 0.285$ Assume $a = 1''$, $Y2 = 3.5''$

		Economy	
W18x50	$\Sigma Q_n = 521\ k$	$\frac{521}{17.2} = 30.3$	$2(31 \times 10) + 50(30) = 2120$ lbs
BFL $\boxed{W18x55}$	$\Sigma Q_n = 336\ k$	$\frac{336}{17.2} = 19.5$	$2(20)(10) + 55(30) = 2050$ lbs

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

- Unshored Strength

$1.4D = 1.4(69\ psf) = 96.6\ psf$

$1.2D + 1.6L = 1.2(69) + 1.6(20) = 114.8\ psf$

$w = (114.8\ psf)(30\ ft) = 3.44\ klf$ & $L_b = 7.5\ ft$ (Table 3-10 AISC)

$M = \frac{wl^2}{8} = \frac{(3.44\ klf)(30\ ft)^2}{8} = 387\ k\text{-}ft$

W18x50: $\phi M_n = 379\ k\text{-}ft < 387\ k\text{-}ft$ NG

$\boxed{W18x35}$: $\phi M_n = 420\ k\text{-}ft > 387\ k\text{-}ft$ OK

- Wet Concrete Deflection

$w = (39\ psf)(30\ ft) = 1.17\ klf$

$\Delta = \frac{5(1.17)(30)^4(1728)}{384(29000)(890\ in^4)} = 0.826\ in$

$\boxed{\therefore \text{Camber } 0.5'' - 0.75''}$

- Live Load Deflection

$w_{LL} = (60.4\ psf)(30\ ft) = 1.812\ klf$

$I_{LB} = 2140\ in^4$

$\Delta_{LL} = \frac{5(1.812)(30)^4(1728)}{384(2140\ in^4)(29000\ ksi)} = 0.532\ in$

- Check a assumption

$V_c = 1071\ k$ (see previous)

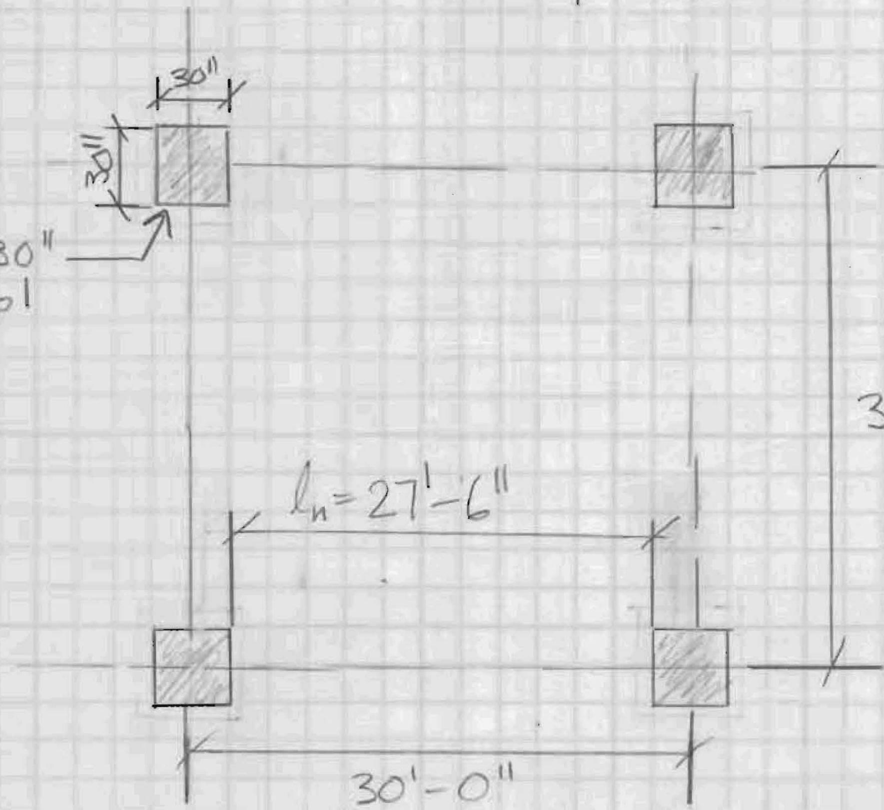
$V_s = 50\ ksi(16.2\ in^2) = 810\ k$

15 studs x $17.2\ k/stud = 258\ k$

$a = \frac{258}{0.85(3.5)(90)} = 0.9636'' < 1''$ OK

7.2 Alternative 2: Two-Way Flat Slab

Alternative 2: Two-Way Flat Slab



Deflections

Table 9.5(c) in ACI 318-11

- f_y = 60,000 psi

Design - f_y=60,000	Interior t_{min}	Exterior t_{min}
W/O drop panels + w/ edge beams	$l_n/33 = \frac{27.5ft}{33} = 10''$	$l_n/33 = 10''$
W/ drop panels + W/ edge beams	$l_n/36 = \frac{27.5ft}{36} = 9.2$ in	$l_n/36 = \frac{27.5'}{36} = 9.2$ in

∴ Start w/ 10" thick slab w/o drop panels

$d_{(avg)} = 10'' - 0.75'' - 0.5'' = 8.75''$

Assuming #4 bars

One-Way Shear:

$LL = 100\ psf$

$DL = 10''/12^{in}/ft \times 150\ pcf = 125\ psf$

$W = 1.2(125) + 1.6(100) = 310\ psf$

$V_u = 0.310\ ksf \times 22.5\ ft \times 30\ ft = 209\ k$

middle strip width, see moment design

$\phi V_c = 0.75\, 2\lambda\sqrt{f'_c}\, bd$

$= 0.75(2)(1)\sqrt{5000_{psi}}(30 \times \frac{12\ in}{ft})(8.75\ in)$

$\phi V_c = 334.1\ k > 209\ k$ OK

Two-Way Shear/Punching Shear:

-Critical location @ $d/2 = 8.75''/2 = 4.38''$

$V_u = (0.310\ ksf)\left[30' \times 30' - \left(\frac{30'' + 2(4.38'')}{12^{in}/ft}\right)^2\right]$

$V_u\ 275\ K$

Critical perim. $b_o = 4(34.38'') = 137.5''$

$\phi V_c\ \phi 4\lambda\sqrt{f'_c}\, b_o d = 4(1)\sqrt{5000}(137.5'')(8.75'')(0.75) = \boxed{255K}$

* ACI eq. 11-31 for highly rect. column not applicable w/ square columns

$\frac{b_o}{d} = \frac{137.5''}{4.38''} = 31.4$ $\therefore$ ACI eq 11-32 applies

Interior Col: $\phi V_c = \phi\left(\frac{\alpha_s d}{b_o} + 2\right)\lambda\sqrt{f'_c}\, b_o d$

$= 0.75\left(\frac{40 \cdot 8.75}{137.5} + 2\right)(1)\sqrt{5000}(137.5)(8.75)$

$= 290\ K$

Corner Col: $\phi V_c = 0.75\left(\frac{20 \cdot 8.75}{137.5} + 2\right)(1)\sqrt{5000}(137.5)(8.75)$

$= \boxed{209K}$

Edge Col: $\phi V_c = 0.75\left(\frac{30 \cdot 8.75}{137.5} + 2\right)(1)\sqrt{5000}(137.5)(8.75)$

$\phi V_c = \boxed{249\text{ K}}$

Eqn 11-32 ϕV_c values control $< V_u = 275\text{ K}$

$\therefore$ Use drop panels and $t = 9.5''$

One-Way Shear w/ $t = 9.5''$

$d_{avg} = 9.5'' - 0.75'' - 0.5'' = 8.25''$

$LL = 100$ psf

$DL = 9.5/12 \times 150 = 119$ psf

$w = 1.2(119) + 1.6(100) = 303$ psf

$V_u = 0.303\text{ ksf} \times 22.5' \times 30' = 204.5\text{ k}$

$\phi V_c = 0.75(2)(1)\sqrt{5000}\,(30 \times 12\text{ in/ft})(8.25)$

$= 315\text{ k} > 204.5\text{ k}$ OK

Two-Way Shear / Punching Shear

$d/2 = 8.25/2 = 4.125''$, $b_o = 4(34.125) = 136.5''$

$V_u = (0.303)\left[30^2 - \left(\frac{30 + 2(4.125)}{12}\right)^2\right] = 270\text{ k}$

$\phi V_c = \phi\left(\frac{\alpha_s d}{b_o} + 2\right)\lambda\sqrt{f'_c}\,b_o d$

Interior: $\left[\left(\frac{40 \cdot 8.25''}{136.5''} + 2\right)(1)\sqrt{5000}(136.5)(8.25)\right] \times 0.75$

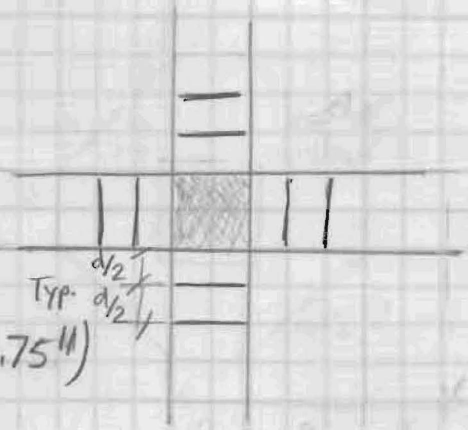
$\phi V_c = 264\text{ K}$

Edge: $\alpha_s = 30$, $\phi V_c = 228\text{ K}$

Corner: $\alpha_s = 20$, $\phi V_c = 191\text{ K}$

$\therefore$ Use Shear Reinforcement

Stirrup Cage Design



$V_n = V_c + V_s$

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

$V_c = 2(1)\sqrt{5000}\,(137.5'')(8.75'')$

$V_c = 170\text{k}$

$V_u = \phi(V_c + V_s)$

<u>Stirrup Req'd:</u>

$275\text{k} = 0.75(170 + V_s)$

$V_{s,req} = 197\text{ K}$

$V_s = A_v f_{yt} \dfrac{d}{s}$

$d = 8.75''$ $\quad f_{yt} = 60\text{ ksi}$

$s = \dfrac{d}{2} = 4.38''$

$197 = A_v(60)\dfrac{8.75}{4.38}$

$A_v = 1.64\text{ in}^2$

Try #4 bars

$\dfrac{1.64\text{ in}^2}{0.5\text{ in}^2} = 4$ bars on a shear plane

<u>Check Max Shear</u>

$\phi V_n = \phi 6\lambda\sqrt{f'_c}\, b_o d = 0.75(6)\sqrt{5000}\,(137.5)(8.75'')$

$= 383\text{ K}$

$V_s = (0.5\text{ in}^2)(4)(60)\left(\frac{8.75}{4.38}\right) = 240\text{K}$

$\phi V_n = 0.75(170 + 240) = 307\text{ K} < 383\text{K}$ <u>OK</u>

Slab Moments

Square Bays = same in both dir.

Can use direct design? (13.6.1 ACI)

- Min of 3 cont. spans ✓
- longer : shorter = 1:1 < 2 ✓
- Successive span lengths ok ✓
- Max col. offset = 10% span
- Will only consider gravity loads
- LL vs. DL

$LL = 100$ psf

$DL = 10/12 \times 150 = 125$ psf

$100 \text{ psf} \not> 2(125 \text{ psf})$ OK

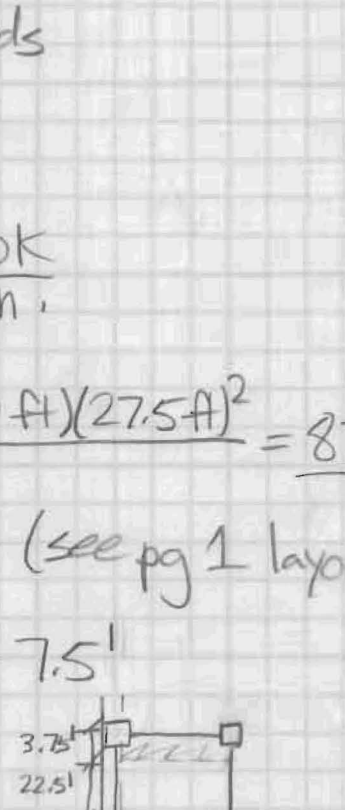
∴ Yes, can use direct design.

$$M_o = \frac{w_u \ell_2 \ell_n^2}{8} = \frac{(0.310 \text{ ksf})(30 \text{ ft})(27.5 \text{ ft})^2}{8} = \underline{879 \text{ ft-k}}$$

$\ell_1 = \ell_2 = 30$ ft + $\ell_n = 27.5$ ft (see pg 1 layout)

Col strip Dimension: $\frac{30 \text{ ft}}{4} = 7.5'$

Interior $\alpha_f = 0$ (no beams)



Exterior Span w/ Edge Beam

ACI 13.6.3.3 Slab w/o Interior Beams but w/ Edge Beams

Interior $-M_u = -0.70\,M_o = -0.7(879) = -615$ ft-k

$+M_u = 0.50\,M_o = 0.50(879) = 440$ ft-k

Exterior $-M_u = -0.30\,M_o = -0.3(879) = -264$ ft-k

Interior Neg. Moments (ACI 13.6.4.1)

⇒ Col Strip: $0.75(-615) = -461$ ft-k

$-461 \text{ ft-k} / 7.5 \text{ ft} = 61.5$ ft-k/ft of mid. strip

Middle Strip: $(-615 \text{ ft-k})(0.25) = -154$ ft-k

Positive Moments - ACI 13.6.4.4

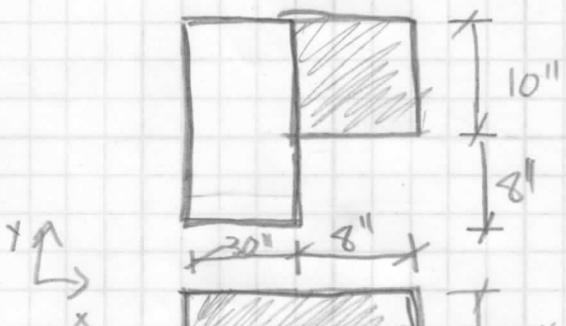
$\Rightarrow$ Col Strip: $0.60(440 \text{ ft-k}) = 264 \text{ ft-k}$

Middle Strip: $0.40(440 \text{ ft-k}) = 176 \text{ ft-k}$

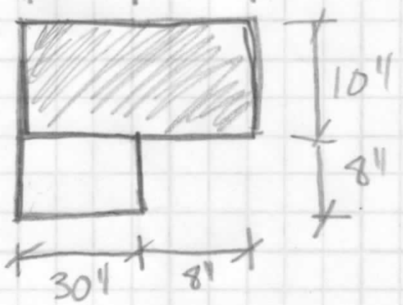
Exterior Neg. Moment - ACI 13.6.4.2

$$\beta_t = \frac{\cancel{E_{cb}}\, C}{2\cancel{E_{cs}}\, I_s}$$

$E_{cb} = E_{cs}$ $\qquad C = \sum\left(1 - 0.63\frac{x}{y}\right)\frac{x^3 y}{3}$



$$C = \left[\left(1 - 0.63\left(\frac{18}{30}\right)\right)\frac{(18)^3(30)}{3}\right] + \left[\left(1 - 0.63\left(\frac{8}{10}\right)\right)\frac{(8)^3(10)}{3}\right] = \underline{37122 \text{ in}^4}$$



$$C = \left[\left(1 - 0.63\left(\frac{10}{38}\right)\right)\frac{(10)^3(38)}{3}\right] + \left[\left(1 - 0.63\left(\frac{8}{30}\right)\right)\frac{(8)^3(30)}{3}\right]$$

$$C = 14827 \text{ in}^4$$

* 18" trial beam depth

$$I_s = \frac{(30 \text{ ft} \times 12 \text{ in/ft})(10 \text{ in})^3}{12} = 30{,}000 \text{ in}^4$$

$$\beta_t = \frac{37122}{2(30000)} = 0.62$$

Interpolating between $\beta_t = 0$ and $\beta_t = 2.5$ in table,

93.8% of Ext. Neg. M_u to Column Strip

$\Rightarrow$ Col strip $= 0.938(-264) = -247.6$ ft-k

Middle Strip $= -16.4$ ft-k

Interior Span

$-M_u = 0.65 M_o = 0.65(879) = -571$ ft-k
$+M_u = 0.35 M_o = 0.35(879) = 308$ ft-k

Negative Moments (ACI 13.6.4.1)

Col Strip: $0.75(-571$ ft-k$) = \underline{-428 \text{ ft-k}}$
Middle Strip: $0.25(-571$ ft-k$) = \underline{-143 \text{ ft-k}}$

Positive Moments (ACI 13.6.4.4)

Col Strip: $0.60(306$ ft-k$) = 185$ ft-k
Middle Strip: $0.40(308$ ft-k$) = 123$ ft-k

Typical Interior Bay Flexural Reinf:

-428 ft-k   185 ft-k   -428 ft-k   Column Strip

-143 ft-k   123 ft-k   -143 ft-k   Middle Strip

-428 ft-k   185 ft-k   -428 ft-k   Column Strip

Approximate d:
long bars: $d \approx h - 1.1 = 10 - 1.1 = 8.9$ in
short bars: $d \approx h - 1.7 = 10 - 1.7 = \underline{8.3 \text{ in}}$ use

$A_{s,min} = 0.0018bh$
$= 0.0018(12'')(10'') = 0.216$ in²/ft

$$A_{s,req} = \frac{M_u (\times 12^{in}/_{ft})}{\phi f'_y jd} \quad \text{where} \quad \phi = 0.9$$

$f'_y = 60$ ksi

$jd = 0.95d = 0.95(8.3in) = 7.89$ in

Location	M_u (ft-k)	$A_{s,req}$ (in^2)	Use...	Spacing	$A_{s,actual}$ (in^2)	$A_{s,min}$/ft	ε_s
-col	-428	12.05	16 #8	12" o.c.	12.64	12.64/12 = 1.05	
+col	185	5.21	12 #6	12" o.c.	5.28	0.44	
-mid	-143	4.03	13 #5	12"	4.03	0.336	
+mid	123	3.46	12 #5	12"	3.72	0.31	
						ok	

Check that steel is in tension...

$$\varepsilon_s = \frac{d-c}{c}\varepsilon_{cu}$$

$$A_s f'_y = 0.85 f'_c ba$$

$$\varepsilon_s = \frac{d-c}{c}\varepsilon_{cu}, \text{ where}$$

$$a = \frac{(12.64)(60)}{0.85(5)(15\times12)} = 0.991$$

$$\varepsilon_s = \frac{7.89 - 1.24}{1.24}(0.003)$$

$$c = \frac{a}{0.80} = \frac{0.991}{0.80} = 1.24$$

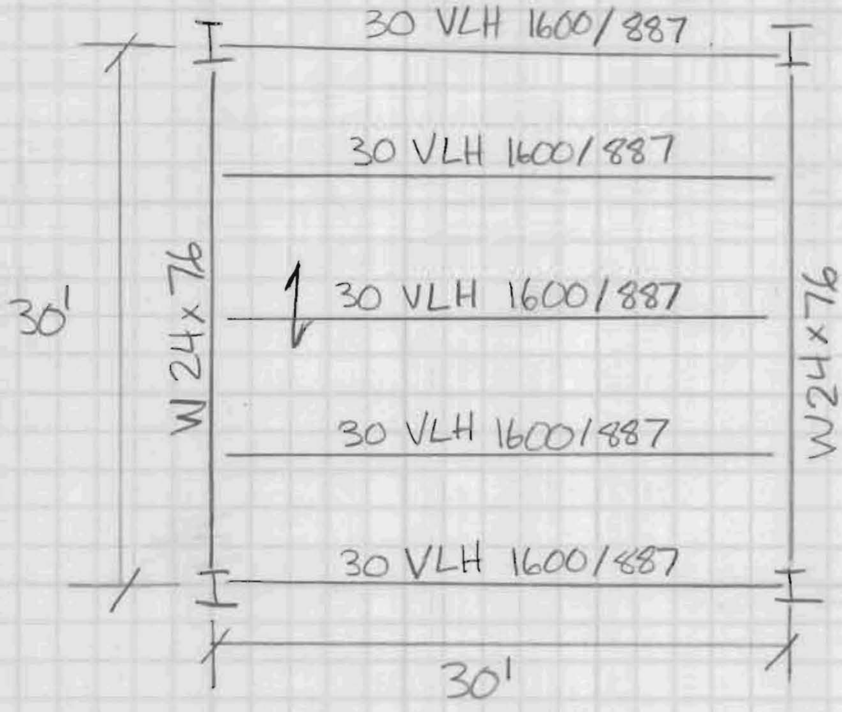
$$\varepsilon_s = 0.0161 > 0.00208$$

$\beta_1 = 0.8$ for 5000 psi conc

OK

5.3 Alternative 3: Non-Composite Steel Joists

Alternative 3: Non-composite Steel Joists



Deck: 5" deck
2C18 with
3" LW topping

1) Non-Composite Steel Deck
- LW conc to decrease weight of deck on joists
- 2 hr fire-rating required
- SDL = 23 psf
  LL = 80 psf
  W_{total} = 103 psf

Try 2C18 with 3" LW conc. topping

Fire-Rating Check – Will need Cementitious or Sprayed Fiber

Max Construction Span:

for LW conc, 3-span = 11'-11"
∴ Try 10' or 7'-6" joist spacing

Allowable Uniform Load:

10'-0" clear span: 123 psf
7'-6" clear span: 215 psf
> 103 psf OK

* for 3-span construction

W_{TL} = L/180 for Roof

- 20 + 23 = 43 psf for design

10'-0" clear span: 92 psf
7'-6" clear span: 218 psf
> 43 psf OK

$W_{LL} = L/240$ for floor live:

10'-0" clear span: 69 psf < 80 psf
7'-6" clear span: 164 psf > 80 psf
∴ Can't Use 10'-0" clear span. Decrease gage of deck for 7'-6" span...

Try 2C22 w/ 3" LW conc. topping

Max Construction Span: 8'-11" > 7'-6" OK
Uniform Load: 113 psf > 103 psf OK
$W_{TL} = L/180$: 126 psf > 43 psf OK
$W_{LL} = L/240$: 95 psf > 80 psf OK
→ Some areas have 100 psf LL. Use 2C20 with W_{LL} of 119 psf

∴ Use 2C20 deck with 3" LW Conc. topping

2) Steel Joists

Live: 80 psf
Dead: 23 + 39 psf = 62 psf
$W_u = 1.2(62\text{ psf}) + 1.6(80\text{ psf}) = 202\text{ psf}$
$W = D + L = 62 + 80 = 142\text{ psf}$

Try VLH Series Non-Composite Floor Joists

Span = 30 ft
$W_u = (202\text{ psf})(7.5\text{ ft}) = 1515\text{ plf}$
$W_L = (80\text{ psf})(7.5\text{ ft}) = 600\text{ plf}$ (for LL Δ)

Try 30VLH 1600/887 with 18" depth
- 5" deep bearings
- 1 bridging

Checks:
W_{TL}: 1600 > 1515 + 1.2(38) = 1561 plf ✓
Δ_{LL}: 887 > 600 plf ✓

Use 18" floor joists: 30 VLH 1600/887 from Vulcraft Composite + Noncomposite Floor Joists Catalog checks.

3) Wide-Flange Girders – Non-Composite

Live: 80 psf

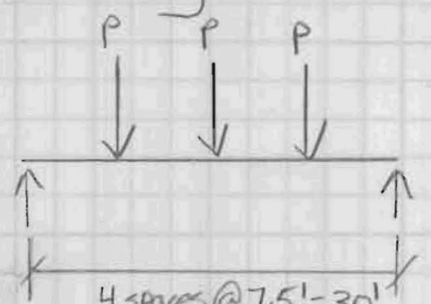
Dead: $62\text{ psf} + \left(\frac{38\text{ plf}}{30\text{ ft}}\right) = 63.3\text{ psf}$

Live Load Reduction

$$L = L_o \left|\begin{matrix} 0.4 \\ 0.25 + \dfrac{15}{\sqrt{K_{LL} A_T}} \end{matrix}\right._{max} = 0.25 + \frac{15}{\sqrt{2(30\times 50)}} = 0.6$$

$= 80 \times 0.6 = 40$ psf

Loading:



$P_{u,TL} = (1561\text{ plf})(30\text{ ft}) = 46.8\text{ K}$

$P_{u,LL} = (600\text{ plf})(30\text{ ft}) = 18\text{ K}$

$$M_{max} = \frac{Pl}{4} + Pa = \frac{46.8(30)}{4} + 46.8(7.5) = 702\text{ ft-k}$$

(superposition from AISC tables)

W24x76: $\phi M_p = 750\text{ ft-k} > 702\text{ ft-k}$

LL Deflection

$$\Delta_{LL} = \frac{Pl^3}{48EI} + \frac{Pa}{24EI}(3l^2 - 4a^2)$$

$$= \frac{(18\text{ K})(30' \times 12)^3}{48(29000)(2100\text{ in}^4)} + \frac{(18)(7.5)(12\text{ in/ft})}{24(29000)(2100)}\left(3(30\times 12)^2 - 4(7.5\times 12)^2\right)$$

$= 0.286" + 0.395" = 0.681" < 1"$ OK

$$\frac{l}{360} = \frac{30 \times 12}{360} = 1"$$

Camber Check

Weight of wet conc: $\frac{39 \times 7.5' \times 30'}{1000} = 8.78k$

$$\Delta_{DL(wet\ conc)} = \frac{(8.78)(30 \times 12)^3}{48(29000)(2100)} + \frac{(8.78)(7.5)(12)}{24(29000)(2100)}\left(3(30 \times 12)^2 - 4(7.5 \times 12)^2\right)$$

$= 0.14 + 0.19 = 0.33''$

Don't camber.....

Use W24 x 76 for girders

6 | System Comparisons

	Existing: One-Way Slab	Composite Steel	Two-Way Slab	Non-Composite Joists
Architectural Coordination				
Depth	25"	22"	10"	29"
Fire Rating	> 2 hr	2 hr	> 2 hr	2 hr
Fire Protection Type	None	Cementitious/Sprayed	None	Cementitious/Sprayed
Construction Statistics				
Cost	$21.45 / SF	$28.41 / SF	$16.70 / SF	$23.90 / SF
Durability	High	Acceptable	High	Acceptable
Structural Considerations				
Weight	175.1 psf	48.3 psf	125 psf	50.4 psf
Servicability	N/A	Vibrations	N/A	Vibrations
Lateral Systems				
Concrete Shear Wall	Yes	Yes	Yes	No
Steel Moment Frame	No	Yes	No	Yes
Steel Braced Frame	No	Yes	No	Yes
Moving Forward?	N/A	YES	YES	NO

Further consideration will be given to the composite steel and two-way slab gravity systems for a more in-depth investigation. Both systems offer the fire-rating and durability required for a healthcare facility. The two-way slab system is a cheaper and thinner gravity system, but the composite steel system is much lighter. Additional investigation will also be required to determine if the vibration requirements of hospital equipment are better suited to concrete construction. Non-composite joists were not as light-weight as expected, and are likely to have vibration issues. Therefore, this system has been ruled out for future consideration.

1) Weight per bay

Existing - One Way Slab w/ Intermediate Beams

- Slab: $(150\,pcf)(5/12)(30)(30) = 56.3\,k$
- 9x25 bms $2(150\,pcf)\left(\frac{9\times 20}{144}\right)(30') = 11.25\,k$
- 36x25 bms $4(150\,pcf)\left(\frac{36\times 20}{144}\right)(30') = 90.00\,k$

$157.6\,k$

- $\frac{157.6\,k \times 1000}{30' \times 30'} = \boxed{175.1\ psf}$

Alternative 1 - Composite Steel

- Deck: $(39\,psf)(30')(30') = 35.1\,k$
- Beams: $5(22\,plf)(30') = 3.3\,k$
- Girders: $2(55\,plf)(30') = 3.3\,k$
- Studs: $5(20\times 10) + 2(40\times 10) = 1.8\,k$

$43.5\,k$

- $\frac{43.5\,k \times 1000}{30' \times 30'} = \boxed{48.3\ psf}$

Alternative 2 - Flat Plate Two-Way Slab

- Slab: $(150\,pcf)(10/12)(30)(30) = 112.5\,k$
- $\frac{112.5\,k \times 1000}{30' \times 30'} = \boxed{125\ psf}$

Alternative 3 - Non-Composite Steel Joists

- Deck: $(39\,psf)(30')(30') = 35.1\,k$
- Joists: $5(38\,plf)(30') = 5.7\,k$
- Girders: $2(76\,plf)(30') = 4.56\,k$

$45.36\,k$

- $\frac{45.36 \times 1000}{30' \times 30'} = \boxed{50.4\ psf}$

2) <u>Cost per bay</u>

— Existing - One Way Slab w/ Intermediate Beams

⇒ RSMeans B10B10 (2015 RSMeans Assemblies)

⇒ Location Factor:

Values for table: - B1010 219 7300 Solid Conc one way slab monolithically cast w/ reinf. concrete support, beams + girders

- Bay Size 30 x 30
- SDL ≈ 125 psf
- Min col size 20"

∴ Use Assembly 7300

Total Base Cost / SF = 21.45 \$/SF

— Composite Steel Slab

Use B1010 254 2300 - composite gage steel deck

- Bay Size 30x30
- SDL ≈ 125 psf (100 psf LL)
- Depth: 4" + 18" = 22"

Total Base Cost / SF = \$28.40 / SF

— Two-Way Flat Slab

Use B1010 223 7600 - Flat Plate 10" thick w/ 125 SDL

Total Base Cost / SF = \$16.70 / SF

— Non-Composite Steel Joists

Use B1010 250 ~~7000~~ 6900 - Steel joists, beams, + slab on column w/ column add

Total Base Cost / SF = 22.10 + 1.80 = \$23.9 / SF

8 | Three Dimensional Modeling

8.1 Model Development and Assumptions

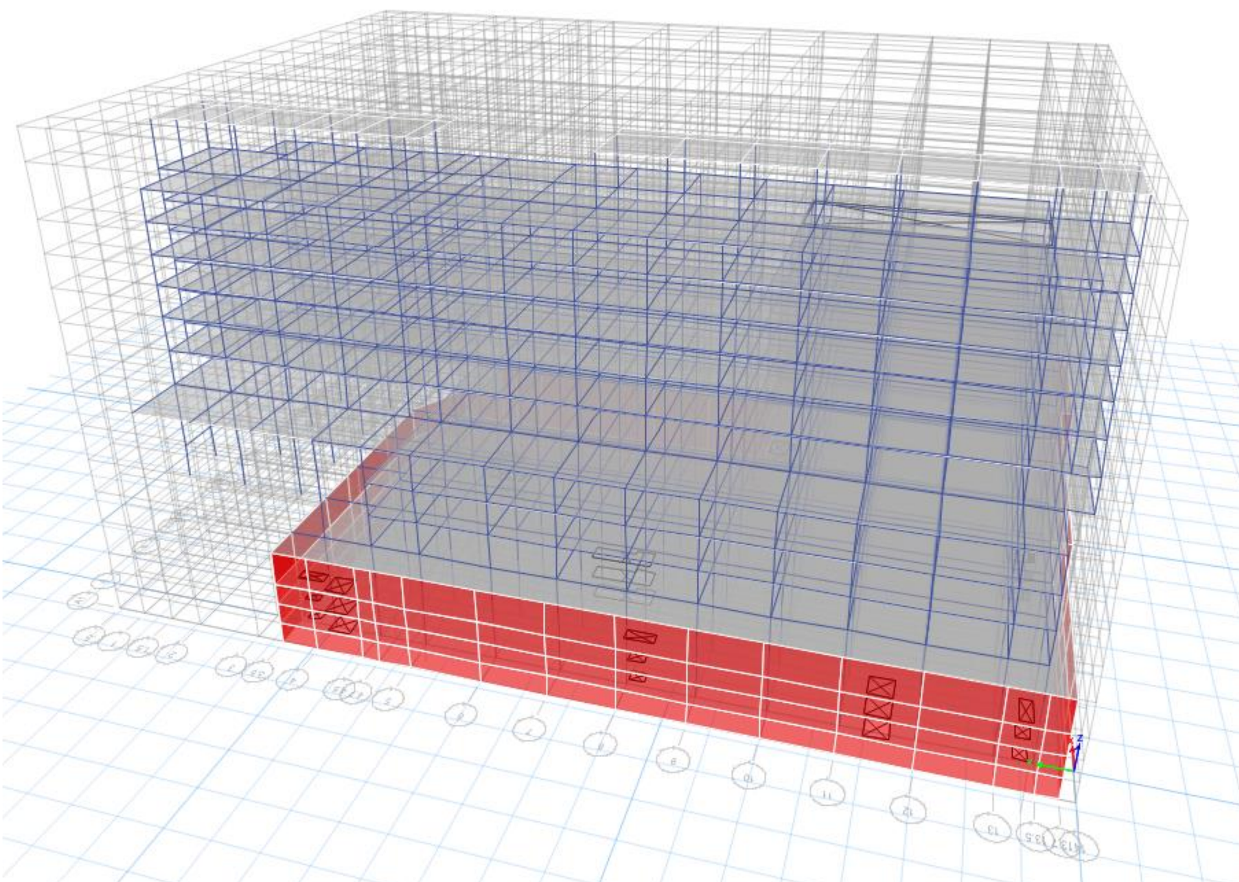


Figure 9 | ETABS Model

All concrete framing members were designed for lateral loads and modelled in three dimensions in ETABS. Sizes of concrete moment frame members vary by frame and by floor, in addition to a variation in below-grade retaining wall thickness between floors and bays. The exact sizes of concrete moment frame members and shear walls were modeled in ETABS. Gravity loads were neglected for the purpose of focusing on the effects of lateral forces on the framing system. Rigid diaphragms were used due to the behavior of a concrete slab and to account for the additional rigidity of intermediate beams between moment frames that were not modeled.

All frame and wall elements were modeled with appropriate f’c values (7000 psi and 5000 psi) as specified by drawing documents. Wall elements were modeled as thin shells with piers, and connections between concrete moment frame members were fixed. Base fixities at the 4th parking level below grade were considered pins for both wall and frame elements after examining reinforcing detailing for columns and the effects of using a fixed instead of pinned connection for column members.

Lateral members from gridlines 1-4 become part of bridge structures outside the scope of this thesis project at lower building levels, and include some composite columns. These column members were modeled to reflect the building load path, but were not connected to the building diaphragm. Some irregular framing geometries were approximated as regular to simplify the modeling process.

Lateral loads were generated automatically according to ASCE 7-05 code requirements and compared to hand calculated values. Seismic loads were applied beginning at the 4th parking deck level, while wind loads were applied beginning at the ground level. The following section evaluates output given by the three dimensional ETABS model for accurate values and correct modeling assumptions.

8.2 Model Validation

Results of the ETABS modeled were validated with comparisons between center of rigidity and mass, wind and seismic load values, and torsional behavior. Drift values for seismic and wind loads generated by ETABS were evaluated for code compliance.

8.2.1 Center of Rigidity and Mass

The center of rigidity of each floor above grade was approximated with hand calculation methods and compared to ETABS values. To simplify the hand calculation process for relative stiffness, all concrete moment frames were assumed to have the same column and beam sizes. Initially, rigidities for all frames on the third floor were calculated to determine its stiffness and the relative stiffness of each frame. An approximate method using portal frame analysis and relative bay widths to determine relative frame stiffness values was found to have similar results, and used for subsequent floors to find the center of rigidity due to the large number of floors and frames to consider.

Hand calculated and ETABS COR values matched within a few feet for most floors. A closer match with ETABS may have been achieved if frames were approximated as the same sizes instead of modeled with their exact. Beam size may vary from 56"x 48" and 45" x 36" to 36" x 25" on the same floor, which contributed to this difference in COR values. The larger differences between calculated and ETABS values occurred on floors of the building with less symmetrical floor plans geometry, which may be referenced in Appendix A. Hand calculations for COR and COM are included at the end of Section 8.2.1.

Story	COR_x (ft)				COR_y (ft)			
	Calculated	Etabs	e_x	% error	Calculated	Etabs	e_y	% error
Penthouse	74	74.5	0.5	0.671141	193.6	197	1.375	1.725888
9	74	74.5	0.5	0.671141	193.6	196	0.375	1.22449
8	74	74.5	0.5	0.671141	193.6	195.5	-0.125	0.971867
7	74	74.5	0.5	0.671141	193.6	196.5	0.875	1.475827
6	114.7	113	30.99953	-1.50442	139.5	154	31.37874	9.415584
5	114.7	126	43.99953	8.968254	139.5	146.5	23.87874	4.778157
4	114.7	103	22.96118	-11.3592	163.5	176.5	54.5553	7.365439
3	114.7	115.5	31.27489	0.692641	120.4	112.5	28.5408	-7.02222
2	114.7	135.5	51.27489	15.35055	120.4	110	26.0408	-9.45455
1	114.7	109	24.77489	-5.22936	120.4	136	36.4408	11.47059

Table 5 | Center of Rigidity and Eccentricity Values

Center of mass was determined using the same assumption that all columns and beams in concrete moment frames on the same floor were the same size. Therefore, frames on the same floor with the same number of bays have the same mass. Due to similar floor geometries, stories 1-3 and stories 5-6 were assumed to have the same center of mass, respectively. Story 4 was calculated separately, and floors 7-9 were assumed to have a center of mass in the center of the floor due to symmetry.

Values on lower floors did not match as closely as desired to verify model results. However, closer results on upper floors do verify model results. Similar differences between levels 1-3 COR and COM results suggest that the variety in beam and column sizes on these floors may have impacted COM results. Larger beams are typically located on the north end of the building, and COMy results from ETABS. Walkway columns on floors 1-3 are not part of the lateral system but were included in the ETABS model. These columns were removed from the model to calculate more accurate COM results. Although this report was unable to match values as closely as desired, all COMx and COMy values from ETABS skew the COM in the direction from the geometric center that would be expected for the building.

Story	COMx (ft)			COMy (ft)		
	Calculated	Etabs	% error	Calculated	Etabs	% error
Penthouse	74	74.723	0.967574	193.6	197.6369	-2.08518
9	74	74.8708	1.16307	193.6	197.2187	-1.86916
8	74	74.7987	1.067799	193.6	196.9988	-1.75558
7	74	74.8708	1.16307	193.6	197.2187	-1.86916
6	91.543562	112.8105	18.85191	136.5028254	154.5055	-13.1885
5	91.543562	126.2774	27.50598	136.5028254	147.0923	-7.7577
4	90.190614	102.7133	12.19188	136.3578738	176.8276	-29.6791
3	93.910688	115.7295	18.85328	93.31501002	113.0906	-21.1923
2	93.910688	124.2351	24.40889	93.31501002	114.6037	-22.8138
1	93.910688	108.1246	13.14586	93.31501002	127.0763	-36.1799

Table 6 | Center of Rigidity and Eccentricity Values

The following pages show the excel spreadsheets and hand calculations used to determine the COR and COM values in the previous tables.

Frame	Relative Stiffness				Frame Mass (kips)			Distance from Datum		W*x or W*y		
	Levels 1-3	Level 4	Level 5	Levels 7 -Pent	Levels 1-3	Level 4	Levels 5-6	X	Y	Levels 1-3	Level 4	Levels 5-6
A	0.1438	0.1438	-	-	455.8	546.7	-	0	-	0	0	0
B	0.1438	0.1438	0.1561	0.2552	455.8	546.7	546.7	30	-	13674	16401	16401
C	0.1438	0.1438	0.1561	0.2552	455.8	546.7	546.7	60	-	27348	32802	32802
D	0.1438	0.1438	0.1561	0.2552	455.8	546.7	546.7	90	-	41022	49203	49203
E	0.1438	0.1438	0.1561	0.2343	455.8	546.7	546.7	120	-	54696	65604	65604
F	0.06923	0.06923	0.0751	-	221.7	187.7	187.7	150	-	33255	28155	28155
G	0.06923	0.06923	0.0751	-	221.7	187.7	187.7	180	-	39906	33786	33786
H	0.06923	0.06923	0.0751	-	221.7	187.7	187.7	210	-	46557	39417	39417
J	0.06923	0.06923	0.0751	-	221.7	187.7	187.7	240	-	53208	45048	45048
L	0.06923	0.06923	0.0751	-	221.7	187.7	187.7	285	-	63184.5	53494.5	53494.5
1	-	0.10622	0.06	0.04651	-	202.6	163.5	-	393	-	79621.8	64255.5
2	-	0.04039	0.06	0.06977	-	163.5	163.5	-	363	-	59350.5	59350.5
3	-	0.04039	0.06	0.06977	-	163.5	163.5	-	333	-	54445.5	54445.5
4	-	-	0.06	0.06977	-	163.5	163.5	-	303	-	49540.5	49540.5
4.5	0.1375	0.10622	0.06	0.06977	191.8	202.6	163.5	-	280.5	53799.9	56829.3	45861.75
5	0.0523	0.04039	0.06	0.06977	191.8	163.5	163.5	-	273	52361.4	44635.5	44635.5
6	0.0523	0.04039	0.06	0.06977	191.8	163.5	163.5	-	225	43155	36787.5	36787.5
7	0.0523	0.04039	0.06	0.06977	191.8	163.5	163.5	-	195	37401	31882.5	31882.5
8	0.0523	0.04039	0.06	0.06977	191.8	163.5	163.5	-	165	31647	26977.5	26977.5
9	0.1176	0.09087	0.1067	0.06977	426	369	369	-	135	57510	49815	49815
10	0.1176	0.09087	0.1067	0.06977	426	369	369	-	105	44730	38745	38745
11	0.1176	0.09087	0.1067	0.06977	426	369	369	-	75	31950	27675	27675
12	0.1176	0.09087	0.1067	0.06977	426	369	369	-	45	19170	16605	16605
13	0.1176	0.09087	0.1067	0.06977	426	369	369	-	15	6390	5535	5535
13.5	0.1176	0.09087	0.1067	0.04651	426	369	369	-	0	0	0	0
				W_{floor}=	5667.255	5863.572	5094.045					
				ΣW=	12569.755	13299.27	11904.845					

	Levels 1-3	Level 4	Level 5-6
ΣW*x	1180434.3	1199470	1089811.913
ΣW*y	1172946.8	1813460	1625044.978
COMx (ft)	93.910688	90.19061	91.54356168
COMy (ft)	93.31501	136.3579	136.5028254

Center of Rigidity Approximation

Levels 1-3: COR_x Coordinate

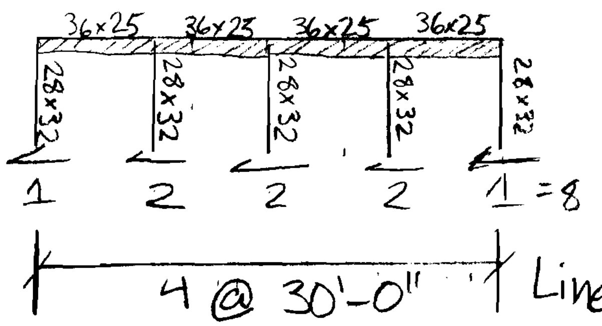
Column Stiffness in Moment Frame = $\frac{12EI}{h^3}$

28x32 col: $I = \frac{bh^3}{12} = \frac{(28")(32")^3}{12} = 76458 \text{ in}^4$

45" dia col: $I = \frac{\pi}{4} r^4 = \frac{\pi}{4}(22.5)^4 = 201289 \text{ in}^4$

24x36 col: $I = \frac{(24)(36)^3}{12} = 93312 \text{ in}^4$

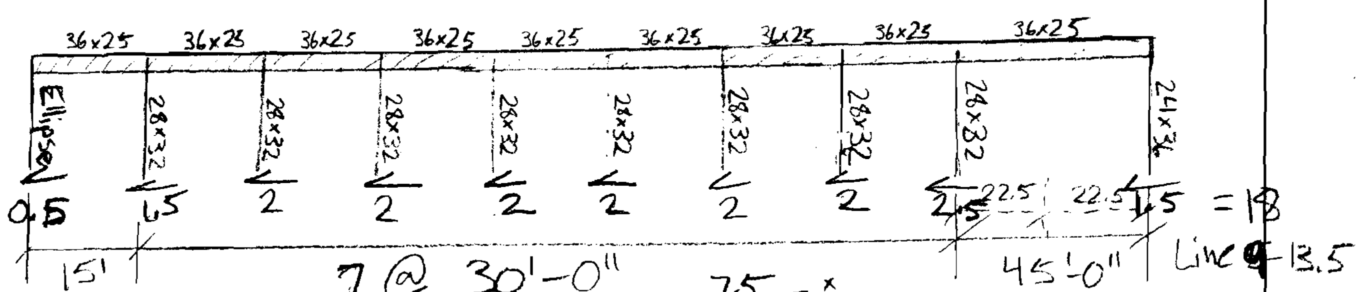
Ellipse col: $I = \frac{1}{4}\pi a b^3 = \frac{1}{4}\pi(24)(36)^3 = 879445 \text{ in}^4$



Line 5-8

Note: See page 3 for COR_x and COR_y process used.

1 frame w/ circular columns
4 frames w/ rect. columns



$\frac{7.5}{30} = \frac{x}{2}$

Line 9-13.5
(6 frames)

K values

4@30' w/ circular columns: $\frac{12(4 \times 201289)(4768.962 \text{ ksi})}{(17 \text{ ft} \times 12 \text{ in/ft})^3} = 5427.4 \text{ k/in}$

4@30' w/ 28x32 columns: $\frac{12(4 \times 76458)(4768.962 \text{ ksi})}{(17 \text{ ft} \times 12 \text{ in/ft})^3} = 2061.57 \text{ k/in}$

8@30' frame: $\frac{12(879445 + 8(76458) + 93312)(4768.962)}{(17' \times 12 \text{ in/ft})^3} = 10680$

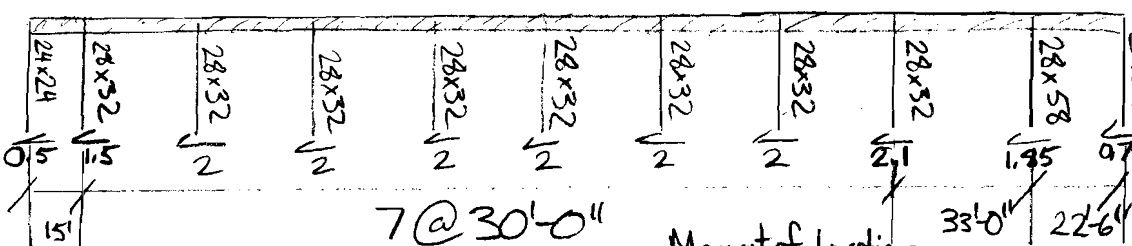
COR Y-Coord:

Total Stiffness $= 2.2587\times10^8 + 3(8.5794\times10^7) + 6(4.4447\times10^8) = 3150072000$

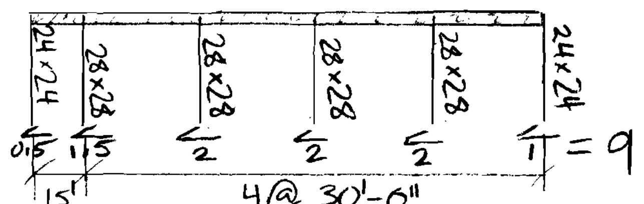
$$COR_y = \frac{\Sigma R_x y}{\Sigma R_x} = \frac{(15)(4.44\times10^8)+45(4.44\times10^8)+75(4.44\times10^8)+105(4.44\times10^8)+135(4.44\times10^8)+165(8.58\times10^7)+195(8.58\times10^7)+228(8.58\times10^7)+250.5(8.58\times10^7)}{3150072000}$$

$COR_y = 75.7$ ft

COR$_y$ Coordinate



4 frames
Line A: 1 24x24, 10 ellipses
Line B-E: see frame



Line L: All 24x36 columns (1 frame)
Line F-J: See frame

Moment of Inertias

$24\times24\ I = \frac{(24)^4}{12} = 27648\ in^4$

$28\times32\ I = \frac{(32)(28)^3}{12} = 58538.7\ in^4$

$28\times58 = \frac{(58)(28)^3}{12} = 106101\ in^4$

$24\times36 = \frac{(36)(24)^3}{12} = 41472\ in^4$

$28\times28 = \frac{(28)^4}{12} = 51221\ in^4$

Stiffness K:

Line A: $\frac{12(27648 + 10(879445))(4768.962)}{(17\times12^{in}/_{ft})^3} = 59468.$ k-in

Line B-E: $\frac{12(27648 + 9(58538.7) + 201289)(4768.962)}{(17\times12)^3} = 5695$ k-in

Line L: $\frac{12(6(41472))(4768.962)}{(17\times12)^3} = 1677.3$ k-in

Line F-J: $\frac{12(2(27648) + 4(51221))(4768.962)}{(17\times12)^3} = 1754$ k-in

~~COR ... ΣR ...~~

~~COR$_y$ = ...~~ ∴ Try a different approach for more accurate results!

COR Coordinates - Levels 1-3

Portal Analysis Approximation of stiffness

* Assume all frames with same bay number/size have approx. same stiffness

4 bay frame = 8

9 bay frame = 18

$\Sigma K = 5(8) + 6(18) = 148$

$$COR_y = \frac{(15+45+75+105+135)(18) + (165+195+225+255+285)(8)}{148}$$

$COR_y = 106.3$ ft vs. 112.5 ft

---

5 bay frame = 9 (5 frames)

10 bay frame = 18.7 (5 frames)

$\Sigma K = 5(8) + 5(18.7) = 132.5$

$$COR_x = \frac{(30+60+90+120)(18.7) + (156+180+210+240+285)(9)}{132.5}$$

$COR_x = 114.7$ ft $= 116$ ft from Etabs

Fix COR_y Approx:

4 bay frame = 8

4 bay frame with circular col. = 21.04

9 bay frame = 18

Adj. for circular col. $\frac{45'' \text{ dia}}{28 \times 32 \text{ col}} = \frac{201289}{76458} = 2.63$

$2.63(8) = 21.04$

$\Sigma K = 161.04$

$COR_y = 120.4$ ft vs 112.5 ft OK

Relative Stiffness Level 3 - x direction

4 bay frames: $8/153.04 = 0.0523$

4 bay circular: $21.04/153.04 = 0.1375$

9 bay frames: $18/153.04 = 0.1176$

Relative Stiffness Level 3 - y-direction

5 bay : 9/132.5 = 0.0679

10 bay : 18.7/132.5 = 0.1411

COR_y - Level 4:

Line 9-13.5 : 6 frames @ 18 k_{rel} = 0.09087

Line 8-5,3,2 : 6 frames @ 8 k_{rel} = 0.04039

Line 4,5 + 1 : 2 frames @ 21.04 k_{rel} = 0.10622

$\Sigma K = 6(18) + 6(8) + 2(21.04) = 198.08$

$$COR_y = \frac{(15+45+75+105+135)(18) + (165+195+225+258+310.5+340.5)(8) + (370.5+280.5)(21.04)}{198.08}$$

$COR_y = 163.5$ ft vs 177 ft

COR_x - Level 4:

Line A-E: 5 frames @ 18.7

Line F-J: 5 frames @ 9

Same as levels 1-3

COR_y - Level 5:

Lose 1 frame → Line 9-13.5: 6 frames @ (0.5 + 1.5 + 5(2) + 4) = 16

Line 1-8 : 9 frames @ 6

$\Sigma k = 150$

$k_{rel} = 16/150 = 0.1067$

$k_{rel} = 9/150 = 0.06$

$$COR_y = \frac{(15+45+75+105+135)(16) + (165+195+225+258+273+280.5+303+333+363+393)(6)}{150}$$

$COR_y = 139.5$ ft vs 147 ft

COR_x – Level 5:

Line B–E : 4 frames @ 18.7 $K_{rel} = 0.1561$

Line F–J : 5 frames @ 9 $k_{rel} = 0.0751$

$\Sigma k = 4(18.7) + 5(9) = 119.8$

$COR_x = 114$ ft in same coord. system as previous levels

COR_y – Level 7-Penthouse

Line 1 + 13.5 : 2 frames @ 4 $K_{rel} = 0.04651$

Line 2 – 13 : 13 frames @ 6 $K_{rel} = 0.06977$

$\Sigma k : 2(4) + 13(6) = 86$

$$COR_y = \frac{(15+45+75+105+135+165+195+225+273+280.5+303+333+363)(6) + 393(4)}{86}$$

$COR_y = 193.6$ ft

COR_x – Level 7–Penthouse

Line B, C, D.2 : 3 frames @ $(1+1.5+6(2)+1.85(2)+2(2)+1) = 23.2$

Line E : 1 frame @ $(1+6(2)+2.1+1.85(2)+2+0.5) = 21.3$

$\Sigma k = 90.9$

$K_{rel} = 0.2552$

$K_{rel} = 0.2343$

$$COR_x = \frac{(30+60+90)(23.2) + 120(21.3)}{90.9}$$

$COR_x = 74$ ft

Frame Weight Approx. by floor

Levels 1-3:

A-E: 11 28x32 col. @ 17 ft
10 25x36 bms @ 30 ft

$150 \text{ pcf} \times \left(17 \times \frac{28 \times 32}{144} \times 11 + 30 \times \frac{25 \times 36}{144} \times 10\right) = 455.8k$

F-L: 6 28x32 col @ 17 ft
4 25x36 bms @ 30 ft
1 " @ 15 ft

$150 \times \left(17 \times \frac{28 \times 32}{144} \times 6 + 30 \times \frac{25 \times 36}{144} \times 4 + 15 \times \frac{25 \times 36}{144}\right) = 221.7k$

13.5-9: 10 28x32 col @ 17 ft
8 25x36 bms @ 30 ft
1 " @ 45 ft

$150 \times \left(17 \times \frac{28 \times 32}{144} \times 10 + 30 \times \frac{25 \times 36}{144} \times 8 + 45 \times \frac{25 \times 36}{144}\right) = 426\text{ K}$

8-4.5: 5 28x32 col @ 17 ft
4 25x36 bms @ 30 ft
W = 191.8 k

Level 4:

A-E: 15 col - 24x24 @ 17 ft
14 bms - 25x36 bms @ 30 ft
W = 546.7 k

F-L: 6 24x24 col @ 17 ft
W = 187.7 K

13.5-9: Same as levels 1-3 w/ 24x24 cols
W = 369 K

2,3,8-5: Same as levels 1-3 w/ 24x24 cols
W = 163.5 K

4.5,1: Same w/ 36" dia Circular Columns
W = 202.6 k

8.2.2 Lateral Load Comparison

Wind loads were applied using ETABS auto-generated values along the edge of the diaphragm. Adjustments were made manually for project location and other building-specific coefficients such as occupancy category and importance factor. ETABS calculates values based on the projected area of each floor diaphragm. Base shear values were not used for comparison due to some frames ending before the base level. Instead, story forces were determined from ETABS output and compared to calculated values. Typically, the calculated values were larger than the ETABS values, although they are in a similar range. This is likely due to different projected wall areas used in the manual and ETABS calculations.

Wind Story Force Comparison			
		Fx (k)	Fy (k)
Penthouse	Calculated	234.9	46.4
	ETABS	212.3	45.3
9	Calculated	181.7	35.8
	ETABS	161.6	34.5
8	Calculated	177.9	35
	ETABS	159.2	33.6
7	Calculated	173.9	34.2
	ETABS	156.6	34
6	Calculated	169.3	105.1
	ETABS	163.8	99.16
5	Calculated	184.4	102
	ETABS	274.349	58.013
4	Calculated	176.4	97.2
	ETABS	240.228	49.828
3	Calculated	110.2	101.3
	ETABS	66.205	30.04
2	Calculated	102.9	94.4
	ETABS	102.6	100.282

*ETABS Wind Load Cases 1 and 2

Table 7 | Wind Story Force Comparison

Seismic loads were also applied using auto-generated ETABS loads with adjustments. The full height of the building was considered for both seismic and wind loads. Adjustments were made manually for project location and other building-specific coefficients such as short and long term periods. The period calculated by the program gave a much higher base shear, and so an approximated period based up calculated values was used. Variation between calculated and ETABS values were greater for seismic loading. A difference in seismic weights used for each floor may account for some difference in seismic load values.

Seismic Load Comparison		
		F (k)
Base Shear	Calculated	1140.8
	ETABS	1802.522

Table 8 | Seismic Load Comparison

8.2.3 Torsional Shear Check

Direct and torsional shear for floor 3 were calculated to verify the proper distribution of story forces into frame elements based on frame stiffness. One frame was checked in both the x and y direction for correct shear values. Shear in moment frames on a single level were determined by adding shear diagram values for columns in the frame on that level. Overall, values were similar but not as close as desired. This may be due to approximations made for the center of rigidity and mass in previous sections. The excel tables created for the calculation of direct shear and torsional shear in the x and y directions are also included in this section.

Frame	Direct V (k)	Torsional Shear (k)	Total Shear (k)	ETABS Value (k)	% diff.
A	**16.88967**	**15.5036317**	**32.39330171**	**28.6**	**-11.71014**
B	16.88967	0.98178272	17.87145272		
C	16.88967	0.63527117	17.52494117		
D	16.88967	0.28875962	17.17842962		
E	16.88967	0.05775192	16.94742192		
F	8.130024	0.13917137	8.26919537		
G	8.130024	0.25846112	8.388485116		
H	8.130024	0.37775086	8.507774862		
J	8.130024	0.49704061	8.627064608		
L	8.130024	0.64641576	8.776439763		
4.5	14.74	1.9747819	16.7147819		
5	5.60656	0.68000695	6.286566946		
6	5.60656	0.52577857	6.132338567		
7	5.60656	0.38557095	5.992130949		
8	5.60656	0.24536333	5.851923331		
9	12.60672	0.54476104	13.15148104		
10	12.60672	0.18158701	12.78830701		
11	12.60672	0.90793507	13.51465507		
12	**12.60672**	**1.63428312**	**14.24100312**	**17.53**	**18.7621**
13	12.60672	2.36063118	14.96735118		
13.5	12.60672	2.7238052	15.3305252		

Table 9 | Total Shear in Frames A and 12

Direct Story Shear Calculations for Level 3:

Level	Height (ft)	Story Force Wind Parallel (k)	Total Shear Wind Parallel (k)	Story Shear in Element - X direction (kips)														
				1	2	3	4	4.5	5	6	7	8	9	10	11	12	13	13.5
Penthouse	23	270.2	270.2	12.57	18.85	18.85	18.85	18.85	18.85	18.85	18.85	18.85	18.85	18.85	18.85	18.85	18.85	12.57
9	15	171.9	442.1	7.995	11.99	11.99	11.99	11.99	11.99	11.99	11.99	11.99	11.99	11.99	11.99	11.99	11.99	7.995
8	15	168.1	610.2	7.818	11.73	11.73	11.73	11.73	11.73	11.73	11.73	11.73	11.73	11.73	11.73	11.73	11.73	7.818
7	15	164.3	774.5	7.642	11.46	11.46	11.46	11.46	11.46	11.46	11.46	11.46	11.46	11.46	11.46	11.46	11.46	7.642
6	15	160.5	935	9.63	9.63	9.63	9.63	9.63	9.63	9.63	9.63	9.63	17.13	17.13	17.13	17.13	17.13	17.13
5	17	176.1	1111.1	10.57	10.57	10.57	10.57	10.57	10.57	10.57	10.57	10.57	18.79	18.79	18.79	18.79	18.79	18.79
4	17	169	1280.1	17.95	6.826	6.826	-	17.95	6.826	6.826	6.826	6.826	15.36	15.36	15.36	15.36	15.36	15.36
3	17	107.2	1387.3	-	-	-	-	14.74	5.607	5.607	5.607	5.607	12.61	12.61	12.61	12.61	12.61	12.61
2	16	93.8	1481.1	-	-	-	-	12.9	4.906	4.906	4.906	4.906	11.03	11.03	11.03	11.03	11.03	11.03
1	16	93.36	1574.46	-	-	-	-	12.84	4.883	4.883	4.883	4.883	10.98	10.98	10.98	10.98	10.98	10.98
P1	16.3	-	1574.46	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
P2	10.3	-	1574.46	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
P3	10.3	-	1574.46	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
P4	10.3	-	1574.46	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
			First Story Shear (k)	74.17	81.06	81.06	74.23	132.7	96.45	96.45	96.45	96.45	139.9	139.9	139.9	139.9	139.9	121.9

Level	Height (ft)	Story Force Wind Perp (k)	Total Shear Wind Perp (kips)	Story Shear in Element - Y direction (kips)									
				A	B	C	D	E	F	G	H	J	L
Penthouse	23	62.3	62.3	-	15.9	15.9	15.9	14.6	-	-	-	-	-
9	15	39.6	101.9	-	10.11	10.11	10.11	9.278	-	-	-	-	-
8	15	38.8	140.7	-	9.902	9.902	9.902	9.091	-	-	-	-	-
7	15	38	178.7	-	9.698	9.698	9.698	8.903	-	-	-	-	-
6	15	110.8	289.5	-	17.3	17.3	17.3	17.3	8.321	8.321	8.321	8.321	8.321
5	17	122.1	411.6	-	19.06	19.06	19.06	19.06	9.17	9.17	9.17	9.17	9.17
4	17	125	536.6	17.64	17.64	17.64	17.64	17.64	8.49	8.49	8.49	8.49	8.49
3	17	119.7	656.3	16.89	16.89	16.89	16.89	16.89	8.13	8.13	8.13	8.13	8.13
2	16	105.3	761.6	14.86	14.86	14.86	14.86	14.86	7.152	7.152	7.152	7.152	7.152
1	16	104.8	866.4	14.79	14.79	14.79	14.79	14.79	7.118	7.118	7.118	7.118	7.118
P1	16.3	-	866.4	-	-	-	-	-	-	-	-	-	-
P2	10.3	-	866.4	-	-	-	-	-	-	-	-	-	-
P3	10.3	-	866.4	-	-	-	-	-	-	-	-	-	-
P4	10.3	-	866.4	-	-	-	-	-	-	-	-	-	-
			First Story Shear (k)	64.17	146.1	146.1	146.1	142.4	48.38	48.38	48.38	48.38	48.38

Torsional Shear for Parallel Wind Direction:

Frame	Stiffness (k/in)	d_i (ft)	$R_id_i^2$	R_id_i	M_t (ft-k)	V_t (k)	Equilibrium Check
A	59468	115	786464300	6838820	3352.7	15.5036317	1782.9176
B	5095	85	36811375	433075	3352.7	0.98178272	83.451531
C	5095	55	15412375	280225	3352.7	0.63527117	34.939914
D	5095	25	3184375	127375	3352.7	0.28875962	7.2189906
E	5095	5	127375	25475	3352.7	0.05775192	0.2887596
F	1754	35	2148650	61390	3352.7	0.13917137	4.870998
G	1754	65	7410650	114010	3352.7	0.25846112	16.799973
H	1754	95	15829850	166630	3352.7	0.37775086	35.886332
J	1754	125	27406250	219250	3352.7	0.49704061	62.130076
L	1677.3	170	48473970	285141	3352.7	0.64641576	109.89068
4.5	5427.4	160.5	139811180.9	871097.7	3352.7	1.9747819	316.95249
5	2061.57	145.5	43643952.29	299958.435	3352.7	0.68000695	98.941011
6	2061.57	112.5	26091745.31	231926.625	3352.7	0.52577857	59.150089
7	2061.57	82.5	14031560.81	170079.525	3352.7	0.38557095	31.809603
8	2061.57	52.5	5682202.313	108232.425	3352.7	0.24536333	12.881575
9	10680	22.5	5406750	240300	3352.7	0.54476104	12.257123
10	10680	7.5	600750	80100	3352.7	0.18158701	1.3619026
11	10680	37.5	15018750	400500	3352.7	0.90793507	34.047565
12	10680	67.5	48660750	720900	3352.7	1.63428312	110.31411
13	10680	97.5	101526750	1041300	3352.7	2.36063118	230.16154
13.5	10680	112.5	135168750	1201500	3352.7	2.7238052	306.42809
		J (k/in)ft^2 =	1478912312			SUM	3352.7

Torsional Shear for Perpendicular Wind Direction:

Frame	Stiffness (k/in)	d_i (ft)	$R_id_i^2$	R_id_i	M_t (ft-k)	V_t (k)	Equilibrium Check
A	59468	115	786464300	6838820	3416.36	15.7980097	1816.7711
B	5095	85	36811375	433075	3416.36	1.0004245	85.036082
C	5095	55	15412375	280225	3416.36	0.6473335	35.603342
D	5095	25	3184375	127375	3416.36	0.2942425	7.3560625
E	5095	5	127375	25475	3416.36	0.0588485	0.2942425
F	1754	35	2148650	61390	3416.36	0.14181391	4.9634869
G	1754	65	7410650	114010	3416.36	0.26336869	17.118965
H	1754	95	15829850	166630	3416.36	0.38492348	36.56773
J	1754	125	27406250	219250	3416.36	0.50647826	63.309782
L	1677.3	170	48473970	285141	3416.36	0.6586897	111.97725
4.5	5427.4	160.5	139811180.9	871097.7	3416.36	2.01227843	322.97069
5	2061.57	145.5	43643952.29	299958.435	3416.36	0.6929187	100.81967
6	2061.57	112.5	26091745.31	231926.625	3416.36	0.53576188	60.273212
7	2061.57	82.5	14031560.81	170079.525	3416.36	0.39289205	32.413594
8	2061.57	52.5	5682202.313	108232.425	3416.36	0.25002221	13.126166
9	10680	22.5	5406750	240300	3416.36	0.55510479	12.489858
10	10680	7.5	600750	80100	3416.36	0.18503493	1.387762
11	10680	37.5	15018750	400500	3416.36	0.92517465	34.694049
12	10680	67.5	48660750	720900	3416.36	1.66531437	112.40872
13	10680	97.5	101526750	1041300	3416.36	2.40545409	234.53177
13.5	10680	112.5	135168750	1201500	3416.36	2.77552395	312.24644
		J (k/in)ft^2 =	1478912312			SUM	3416.36

8.3 Code and Member Checks

8.3.1 Drift Limitations

Drifts due to automatically generated wind and seismic load cases were checked against ASCE 7-05 limitations. All were found to be within the acceptable limit. Seismic requirements come from Table 12.12-1. Drift values were lower than expected for all floors, particularly floors 5 and below. Review of the ETABS model could not determine the cause of this irregularity. All beams, columns, and diaphragms were modeled the same for all floors with the same fixities. Further model review will be required to determine the accuracy of story drifts given.

Story	h (ft)	Wind			Seismic		
		h/400 (in)	Drift (in)	Acceptable?	$0.010h_{sx}$ (in)	Drift (in)	Acceptable?
Penthouse	23	0.69	0.056972	Yes	1.8	0.231197	Yes
9	15	0.45	0.056284	Yes	1.8	0.086756	Yes
8	15	0.45	0.079487	Yes	1.8	0.119367	Yes
7	15	0.45	0.106105	Yes	1.8	0.149634	Yes
6	15	0.45	0.150269	Yes	2.04	0.197121	Yes
5	17	0.51	0.125244	Yes	2.04	0.157321	Yes
4	17	0.51	0.017368	Yes	2.04	0.020494	Yes
3	17	0.51	0.00384	Yes	1.92	0.004936	Yes
2	16	0.48	0.01286	Yes	1.92	0.015523	Yes
1	16	0.48	0.000584	Yes	1.956	0.000286	Yes
P1	16.3	0.489	0.000084	Yes	1.236	0.00027	Yes
P2	10.3	0.309	0.0000036	Yes	1.236	0.00031	Yes
P3	10.3	0.309	0	Yes	1.236	0.00031	Yes
P4	10.3	0.309	0	Yes	0	0	Yes

Table 10 | Lateral Drift Code Check

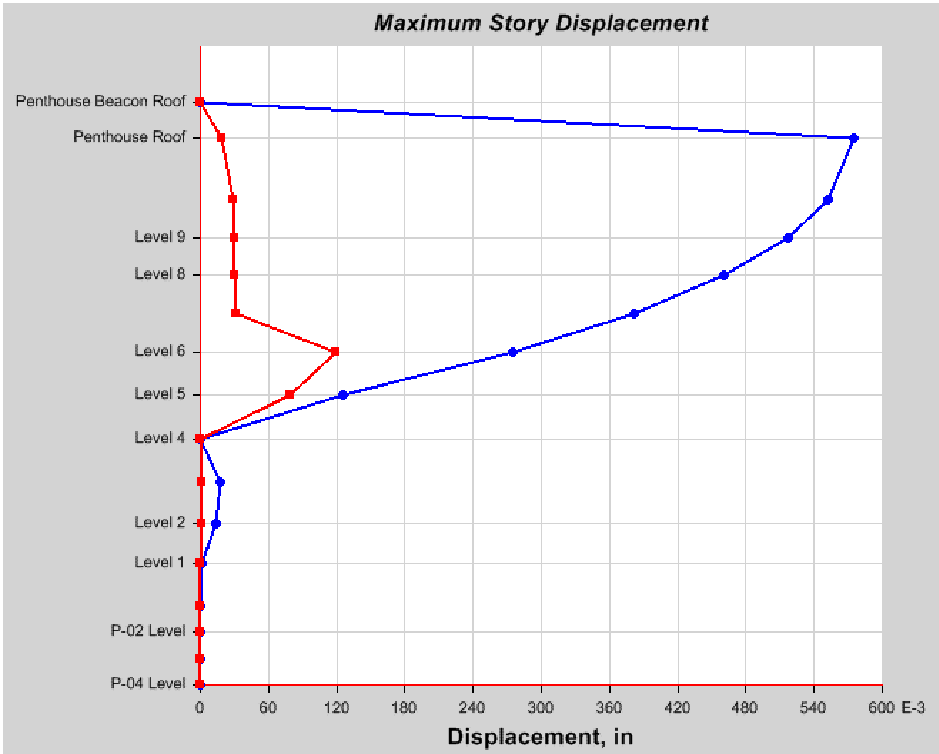


Figure 10 | Maximum Story Displacement Due to Wind Loads

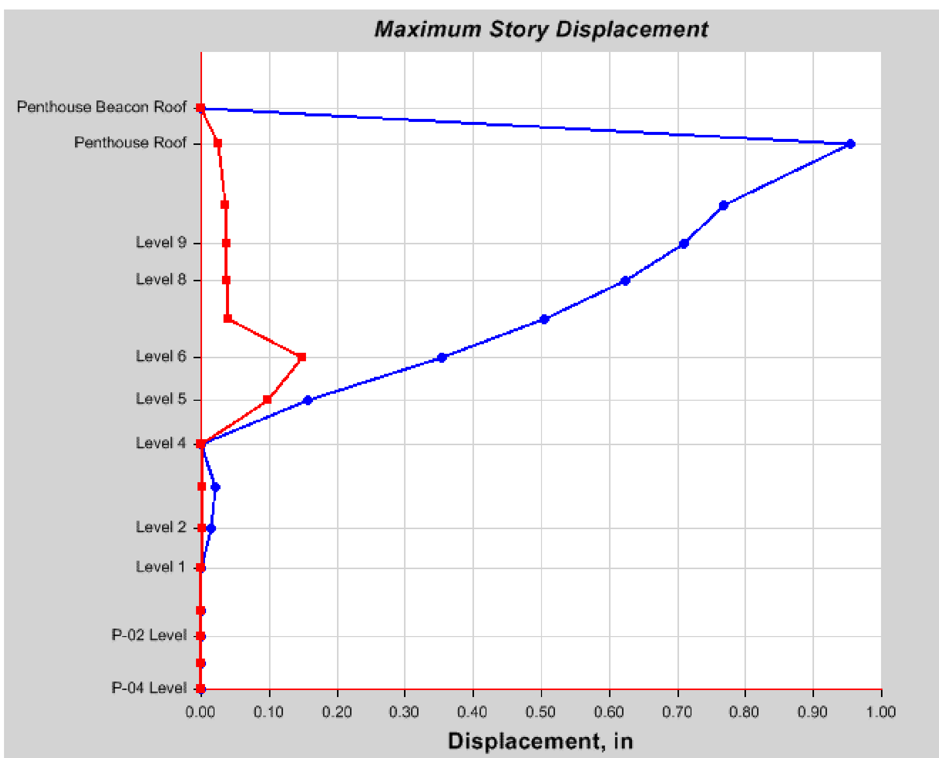


Figure 11 | Maximum Story Displacement Due to Seismic Loads

8.3.2 Member Checks

Strength checks were completed for a critical column and beam in the bed tower moment frames at level 7. This level was chosen due to deflection, shear, and moment behavior closer to expected results and values. Additionally, these frames may be considered more critical because they extend the entire height of the building core. Column C11 and beam BC along line 11 will be checked for this report, and are highlighted in elevation and plan view in Figure 12 and 13. ETABS generated seismic loads were used for this check due to slightly larger shear and moment values. All members passed strength checks.

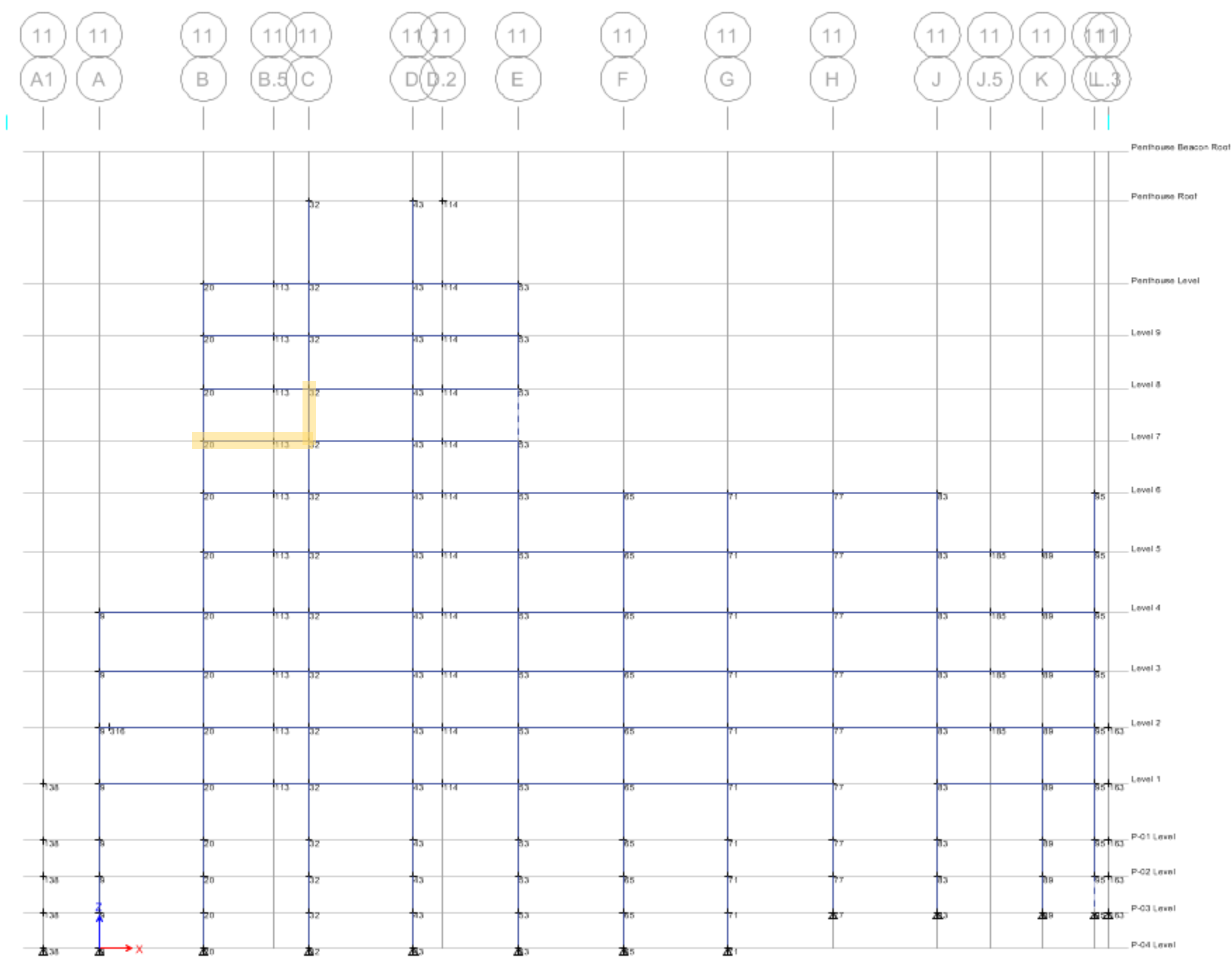


Figure 12 | Elevation along Line 11

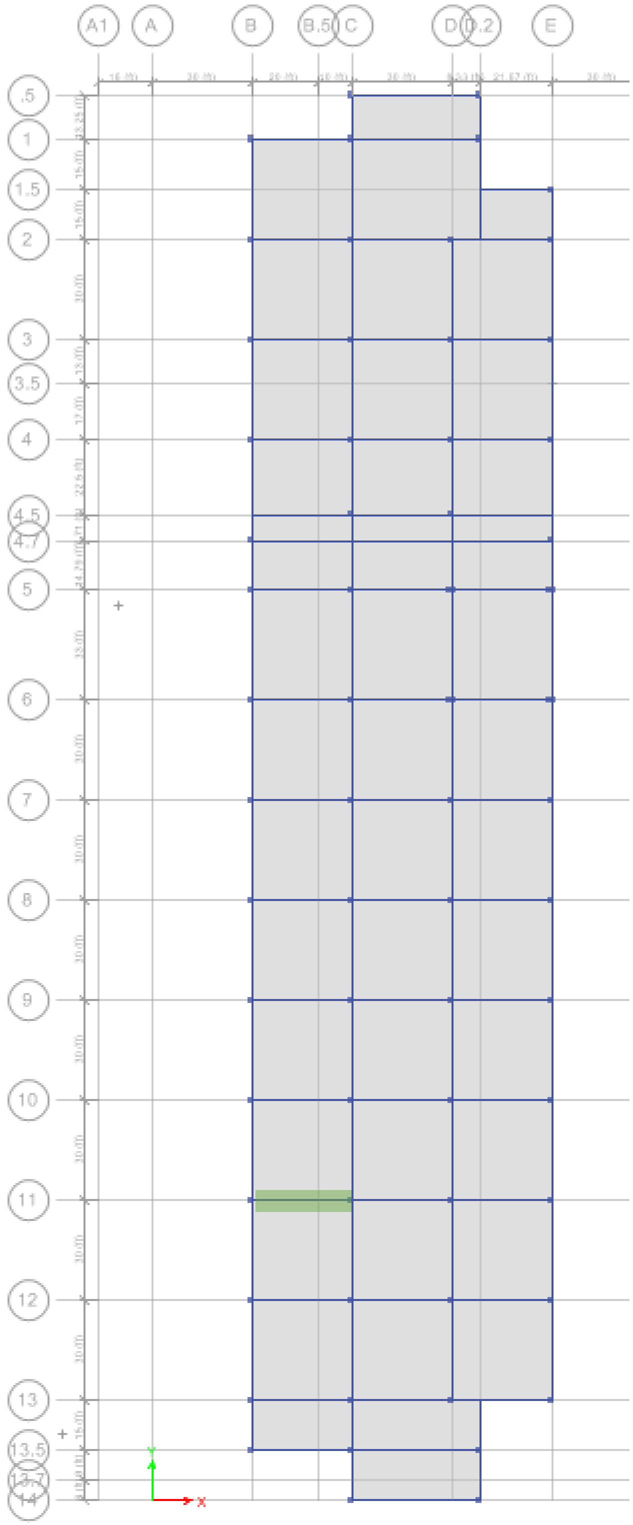
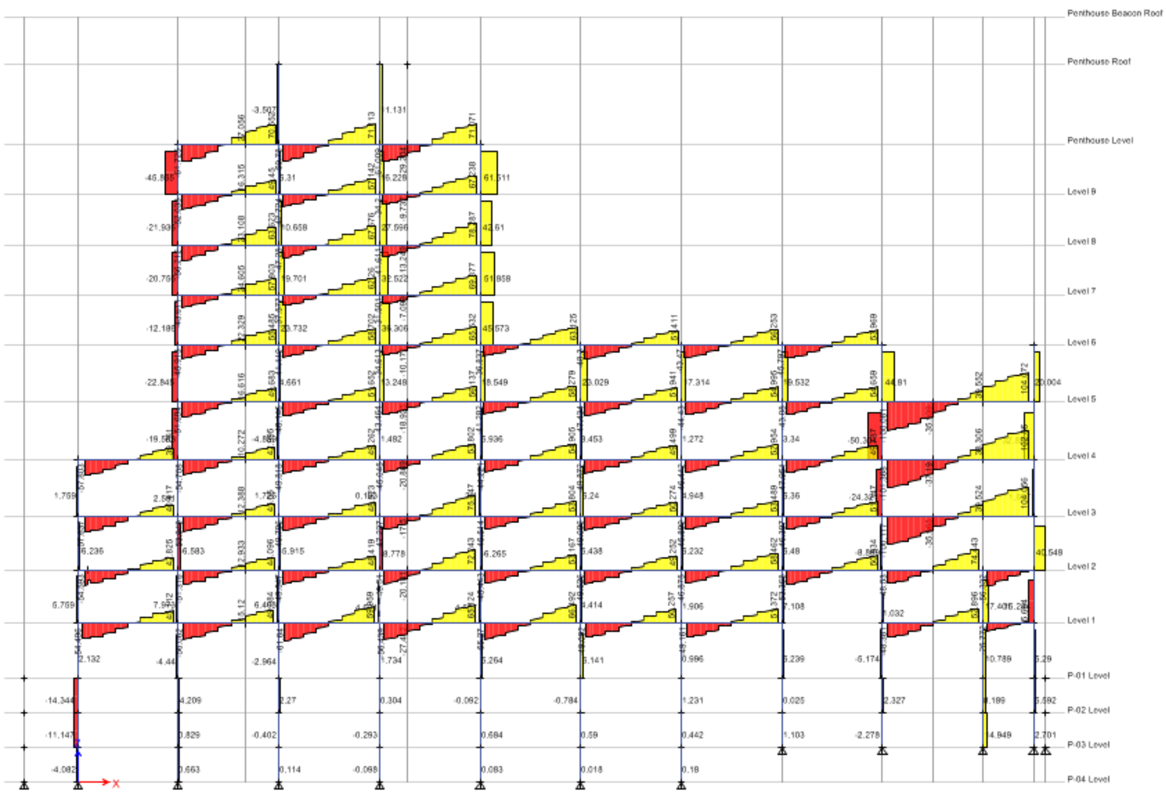
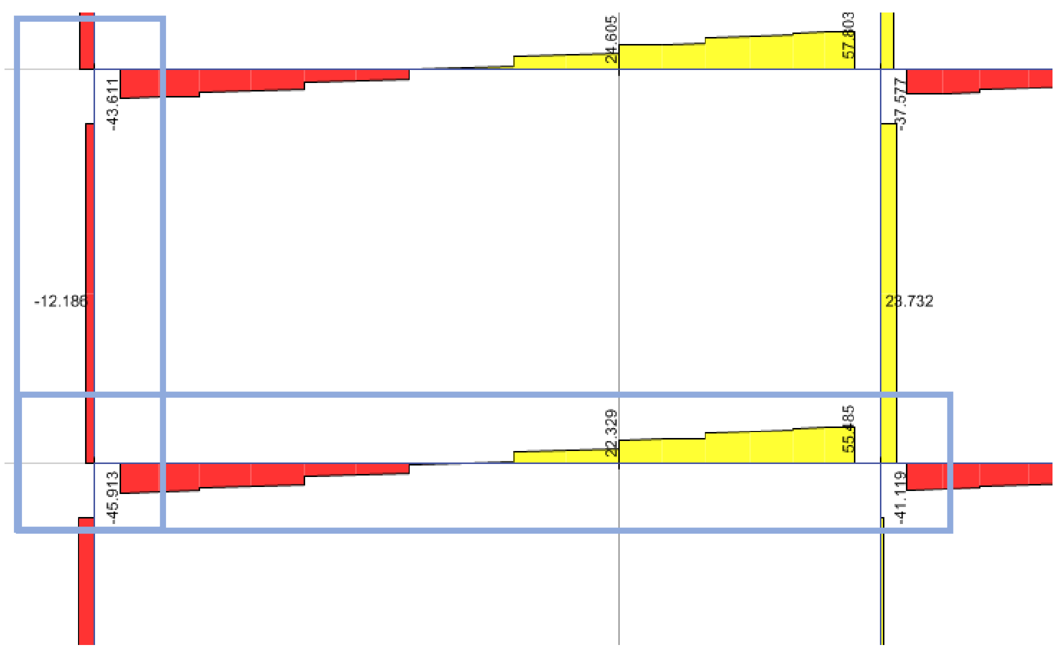
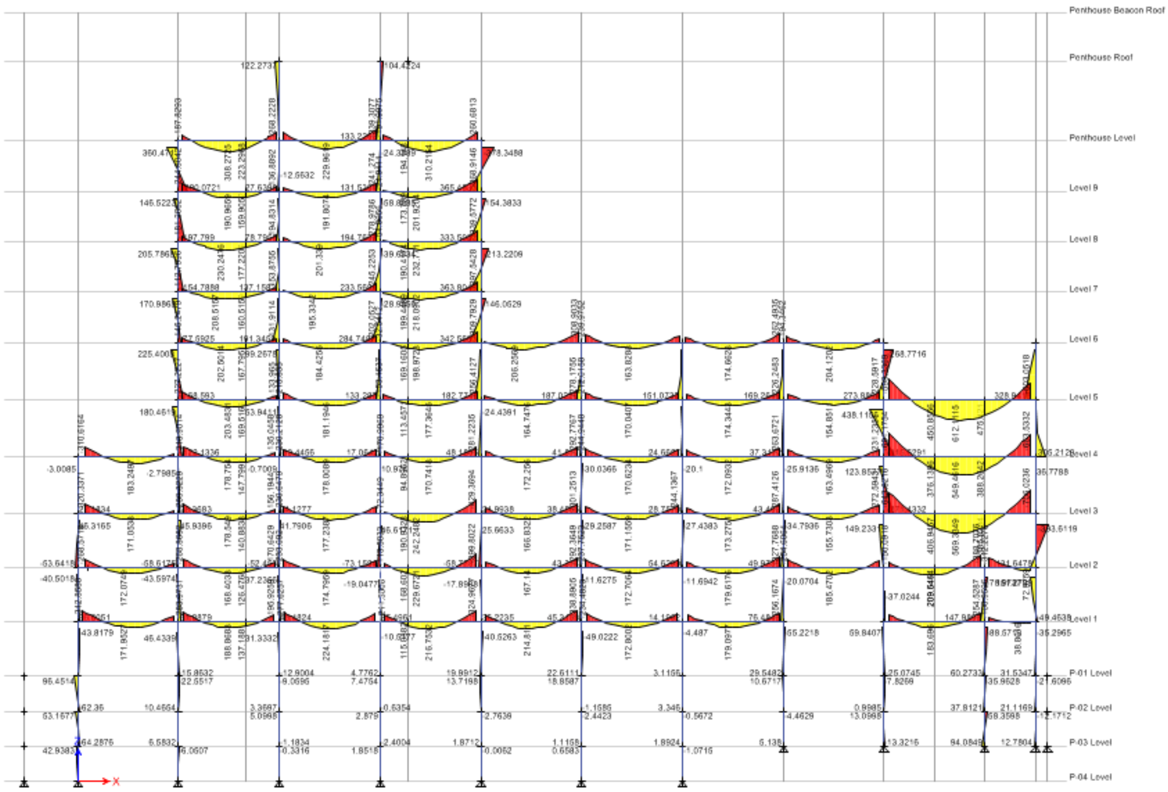


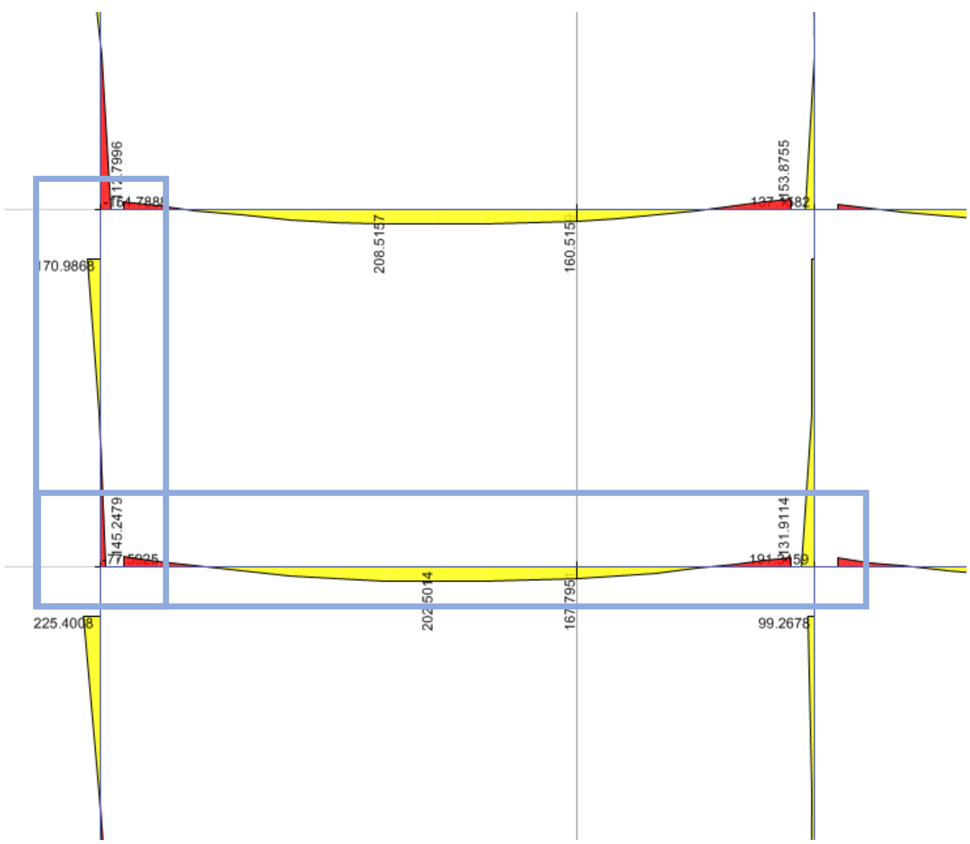
Figure 13 | Level 7 Floor Plan

On the following pages are shear and moment diagrams for the 1.2D+1.0E+1.0L load case used in this member strength check. Values are in kips and ft-kips.

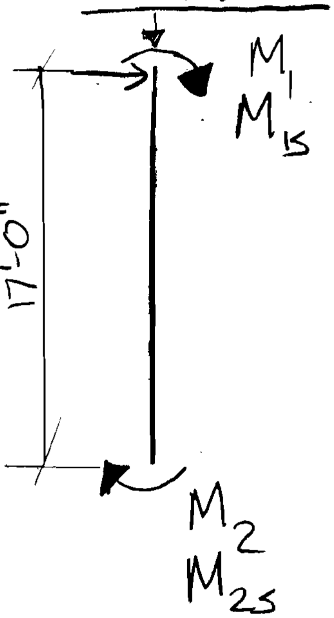





Penthouse Beacon Roof
Penthouse Roof
Penthouse Level
Level 9
Level 8
Level 7
Level 6
Level 5
Level 4
Level 3
Level 2
Level 1
P-01 Level
P-02 Level
P-03 Level
P-04 Level


153.8755
208.5157
160.515
170.9868
131.9114
202.5014
167.795
225.4008
99.2678

Column: 24 x 24, $f'_c = 7000$ psi



Consider Load Cases of
A) Gravity only: $1.2D + 1.6L$
B) Gravity + Wind: $1.2D + 1.0W + 1.0L$
for sway frame.

① Determine cracked moment of inertia

$$I_c = 0.7 I_g = 0.7 \frac{24^4}{12} = 19354 \text{ in}^4$$

② Are columns slender?

$$\frac{k l_u}{r} = \frac{1.2(17 \times 12)}{0.3(24)} = 34 \geq 22$$

∴ Yes, column is slender

Gravity Only:

③ $P_u = 658.5$k
$M_2 = 264.9$ ft-k
$M_1 = 205.1$ ft-k

$$P_c = \frac{\pi^2 EI}{(k l_u)^2}$$

$$\delta_{ns} = \frac{C_m}{1 - P_u / 0.75 P_c}$$

$$C_m = 0.6 + 0.4\left(\frac{M_1}{M_2}\right) = 0.6 + 0.4\left(\frac{205.1}{264.9}\right) = 0.91 \geq 1.0$$

∴ Use $C_m = 1.0$

$$\beta_{dns} = \frac{\text{max axial}}{\text{story axial}} = \frac{658.5\text{k}}{52249.259\text{k}} = 0.0126$$

$$EI = \frac{0.2 E_c I_g + E_s I_s}{1 + \beta_{dns}} = \frac{0.2(4768.9)\left(\frac{24^4}{12}\right) + 29000(750)}{1 + 0.0126} = 47521340$$

$$I_s = 2.2(0.015)\left(\frac{24 - 2(2.5)}{24}\right)^2 \left(\frac{24^4}{12}\right) = 750 \text{ in}^4$$

$$P_c = \frac{\pi^2 (47521340)}{(1.2 \times 17 \times 12)^2} = 7826.5$$

$$\delta_{ns} = \frac{1}{1 - \frac{658.5}{0.75(7826.5)}} = 1.126$$

COMET

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER

$M_c = 1.126(264.9 \text{ ft-k}) = 298.3 \text{ ft-k}$

④ Check Gravity Only Load Capacity:

$e = \frac{M_c}{P_u} = \frac{298.3}{658.5} = 0.45''$

$e/h = 0.45/24 = 0.01875$

$\gamma = \frac{24 - 2(2.5)}{24} = 0.8$

Approximating behavior with $f'_c = 6000$ psi design aid in Reinforced Concrete and Mechanics Design by MacGregor, figures A-11b and A-11c, interaction diagrams...

$\gamma = 0.75$: $\frac{\phi P_n}{bh} = 4.0$ $\frac{\phi M_n}{bh^2} = 0.3$

$\gamma = 0.9$:

$\boxed{\therefore 6000 \text{ psi column ok, therefore } 7000 \text{ psi should as well}}$ $\frac{658.5}{(24)^2} = 1.14 < 4$, $\frac{298.3}{24^3} = 0.021 < 0.3$

Minimum moment does not govern by inspection

⑤ Check Gravity + Seismic Loads

$P_u = 515.5\text{k}$
$M_{1s} = 77.59 \text{ k-ft}$
$M_{2s} = 171 \text{ k-ft}$

ETABs values

$M_n = 1.2M_D + 0.5M_L$

$M_{1ns} = 1.2(102.5 \text{ k-ft}) + 0.5(88.6727) = 167.3 \text{ k-ft}$

$M_{2ns} = 1.2(80.16 \text{ k-ft}) + 0.5(68.0746) = 130.2 \text{ k-ft}$

$\delta_{ns} = \frac{1}{1 - \frac{515.5}{0.75(7826.5)}} = 1.096$

$$M_1 = M_{1s} + \delta_s M_{1ns} = 77.59 + 1.096(167.3)$$
$$= 261 \text{ ft-k}$$

$$M_2 = M_{2s} + \delta_s M_{2ns} = 171 + 1.096(130.2)$$
$$= 313.7 \text{ ft-k}$$

Using Same Interaction Diagrams as previous...

$$e = \frac{M_c}{P_u} = \frac{313.7 \text{ ft-k}}{515.5} = 0.6$$

$$\gamma = 0.8$$

$$\gamma = 0.75: \frac{\phi P_n}{bh} = 1.5 \qquad \frac{\phi M_n}{bh^2} = 0.85$$

$$\gamma = 0.9: \frac{\phi P_n}{bh} = 1.7 \qquad \frac{\phi M_n}{bh^2} = 0.9$$

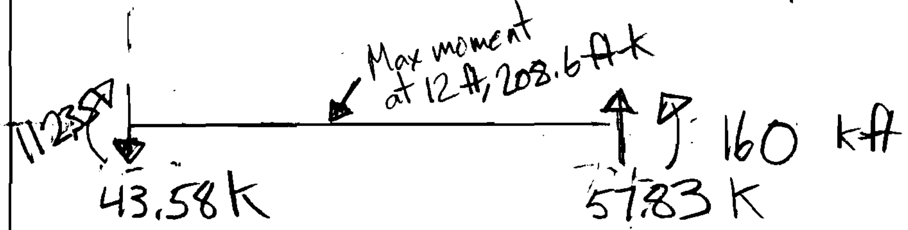
For $\gamma = 0.8: \frac{\phi P_n}{bh} = 1.57, \frac{\phi M_n}{bh^2} = 0.867$

$$\frac{P_u}{bh} = \frac{515.5}{24^2} = 0.89 < 1.57 \text{ } \underline{\text{OK}}$$

$$\frac{\phi M_n}{bh^2} = \frac{313.7}{24^3} = 0.0227 \text{ } \underline{\text{OK}}$$

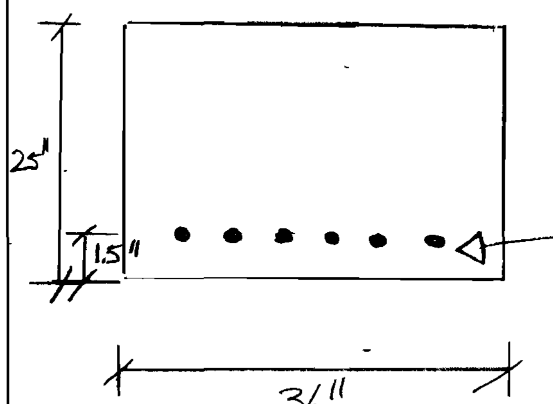
$\boxed{\therefore \text{ 7000 psi 24x24 column passes member check}}$

Beam - 36x25, 5000 psi = f'_c



* No axial force transfers into beam in ETABs

Controlling lateral load case: 1.2D + 1.0E + 1.0L
⇒ Check beam capacity for 57.83K shear
208.6 ft-k flexural



6#6
$A_s = 2.64 in^2$

Flexural Strength:

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(2.64 in^2)(60)}{0.85(5)(36)} = 1.035''$$

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right) / 12 \, ^{in}/_{ft}$$

$$= 0.9(2.64 in^2)(50)\left(23.5 - \frac{1.035}{2}\right) / 12$$

$$= 227.5 \text{ ft-k} > 208.63 \text{ ft-k}$$

∴ Beam passes flexual strength

Shear Strength:

$$V_c = 2\lambda\sqrt{f'_c}\, b_w d = 2(1)\sqrt{5000}(36)(23.5) = 119.6k > 57.83k$$

∴ Beam passes shear strength

Beam passes strength requirements for lateral load combination w/ seismic

9 | Appendix A

Design building loads from the load key plan on structural documentation are listed in Table 5. The table includes superimposed dead loads, live loads, and concentrated live loads. Superimposed dead loads do not account for the total dead load of the structure.

LOAD KEY TABLE SUPERIMPOSED DEAD AND LIVE LOAD				SENSITIVE EQUIPMENT VIBRATION CRITERIA
OCCUPANCY	SUPER IMPOSED DEAD LOAD (PSF)	UNIFORM LIVE LOAD (PSF)	CONCENTRATED LIVE LOAD (LB)	
AHU 1 = AIR HANDLING UNIT	15CMEP + 60TOPPING = 75	150 NR	15,000 LBS	-
CEPR1 = CENTRAL ENERGY PLANT ROOF	80CMEP	40 NR	3,000	-
CEPR2 = CENTRAL ENERGY PLANT ROOF	80CMEP + 80RAMP = 160	40 NR	3,000	-
CT = COOLING TOWER	50	150 NR	52,800 LBS (EACH CELL) THREE CELL LOCATIONS	-
DROP = DROP OFF AREA	300	100 NR	2,000	-
EXT1 = EXTERIOR PLAZA ALONG CLIFTON RD	230	100 NR	2,000	-
EXT2 = EXTERIOR SOIL ALONG GRID 13.5	1,180	200 NR	2,000	-
HOS1 = TYPICAL HOSPITAL AREAS	15	100 RED	2,000	8,000 MIPS @ 85 PPM
HOS2 = HOSPITAL DIAGNOSTICS AND IMAGING	15CMEP + 60TOP = 75	350 NR	106 KIPS OR EQUIP. WGT	1000 MIPS @ 100 PPM
HOS 3 = HOSPITAL DIAGNOSTICS AND SURGICAL SUITES	15	100 RED	2,000 OR EQUIP. WGT	4,000 MIPS @ 85 PPM
KIT = KITCHEN	15CMEP + 40TOP + 40CMU = 95	150 NR	2,000	-
LD = LOADING DOCK	15	250 NR	-	-
MEC = MECHANICAL/ELECTRICAL ROOMS	15CMEP + 60TOPPING = 75	150 NR	2,000 OR EQUIP. WGT	60 TOPPING NOT APPLIED AT LEVEL 3 PNEUMATIC TUBE ROOM
MRI ACC	TYPICAL SDL FLOOR LOADS PER FINAL USE	150 NR	-	-
MU1 = MIXED USE 1	15CMEP + 40CMU = 55	100 RED	2,000	-
MU2 = MIXED USE 2	15CMEP + 5 FIN = 20	100 RED	2,000	-
MU3 = MIXED USE 3	15CMEP + 40 TOP + 40CMU = 95	100 RED	2,000	-
PAT = TYPICAL PATIENT ROOMS	15	80 RED (+)	1,000	-
PK1 = TYPICAL PARKING	5	40 NR	3,000	-
PK2 = PARKING WITH CURB ALLOWANCE	5CMEP + 40TOP = 45	40 NR	3,000	-
PUB1 = PUBLIC AREAS, LOBBIES, AND CORRIDORS	15	100 NR	2,000	-
PUB2 = PUBLIC AREAS, LOBBIES, AND CORRIDORS w/ THICK SET TILE/TOPPING	15CMEP + 25TILE = 40	100 NR	2,000	-
RF1 = ROOF WITH INSULATED CONCRETE TOPPING	15CMEP + 25TOP + 10ROOF = 50	20 NR	-	-
RF2 = GREEN ROOF/OUTDOOR PUBLIC AREA	15CMEP + 25TOP + 10ROOF + 50GREEN ROOF/PAVERS = 100	100 NR	-	-
RF3 = TYPICAL ROOF	25	20 NR	-	-
RFPH = PENTHOUSE ROOF	25CMEP + 25 ROOF = 50	20 RED	-	-
RR = RESTROOM	15CMEP + 25FIN = 40	100 RED	-	-
STA = METAL STAIR	60	100 NR	-	-
STO = LIGHT STORAGE	15	125 NR	2,000	-
TVROOF = TRANSFORMER VAULT ROOF	-	-	-	-

Table 5 | Load Key Plan Values

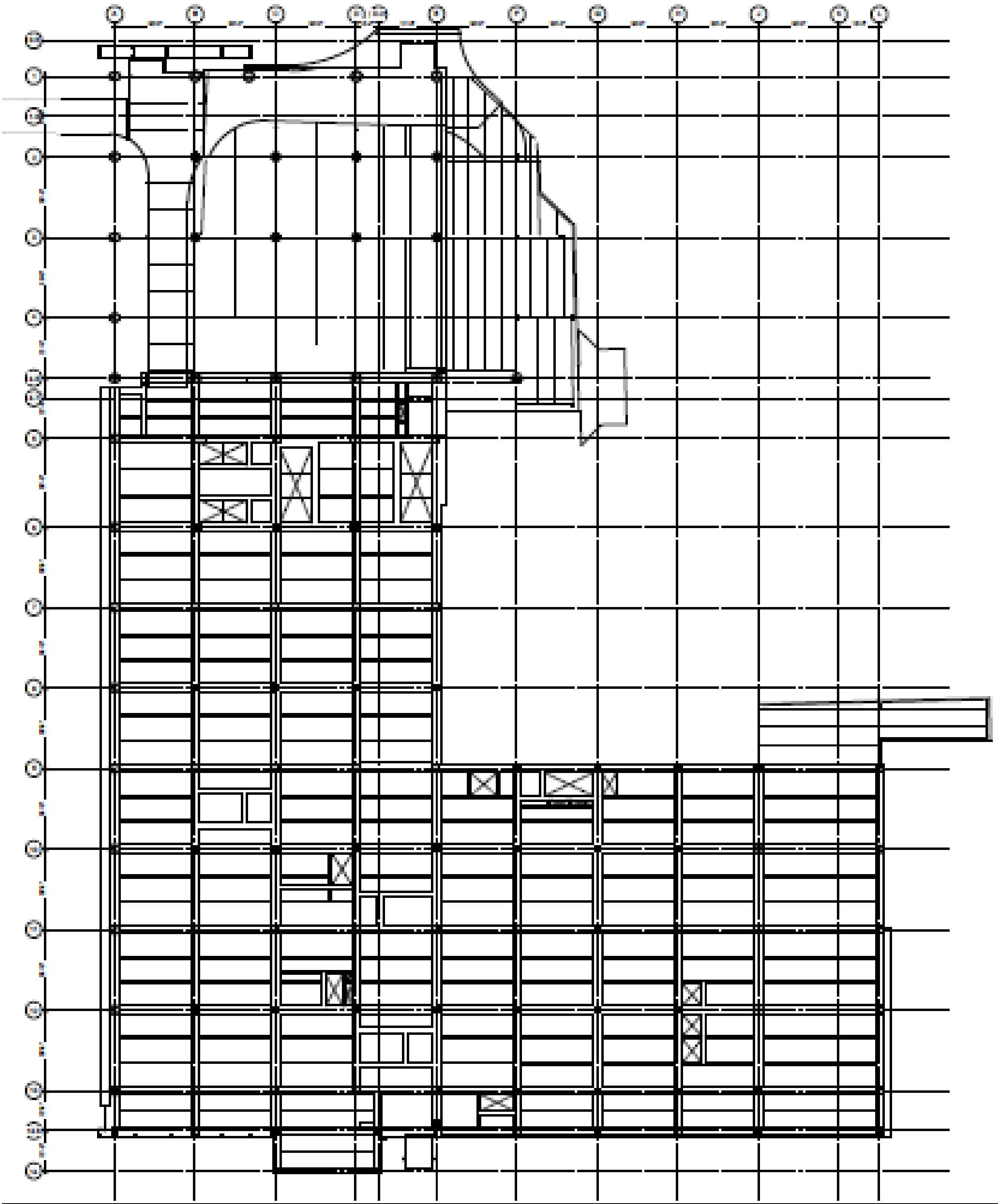


Figure 9 | Typical Structural Floor Plan for Floors 1-3 (Walter P. Moore)

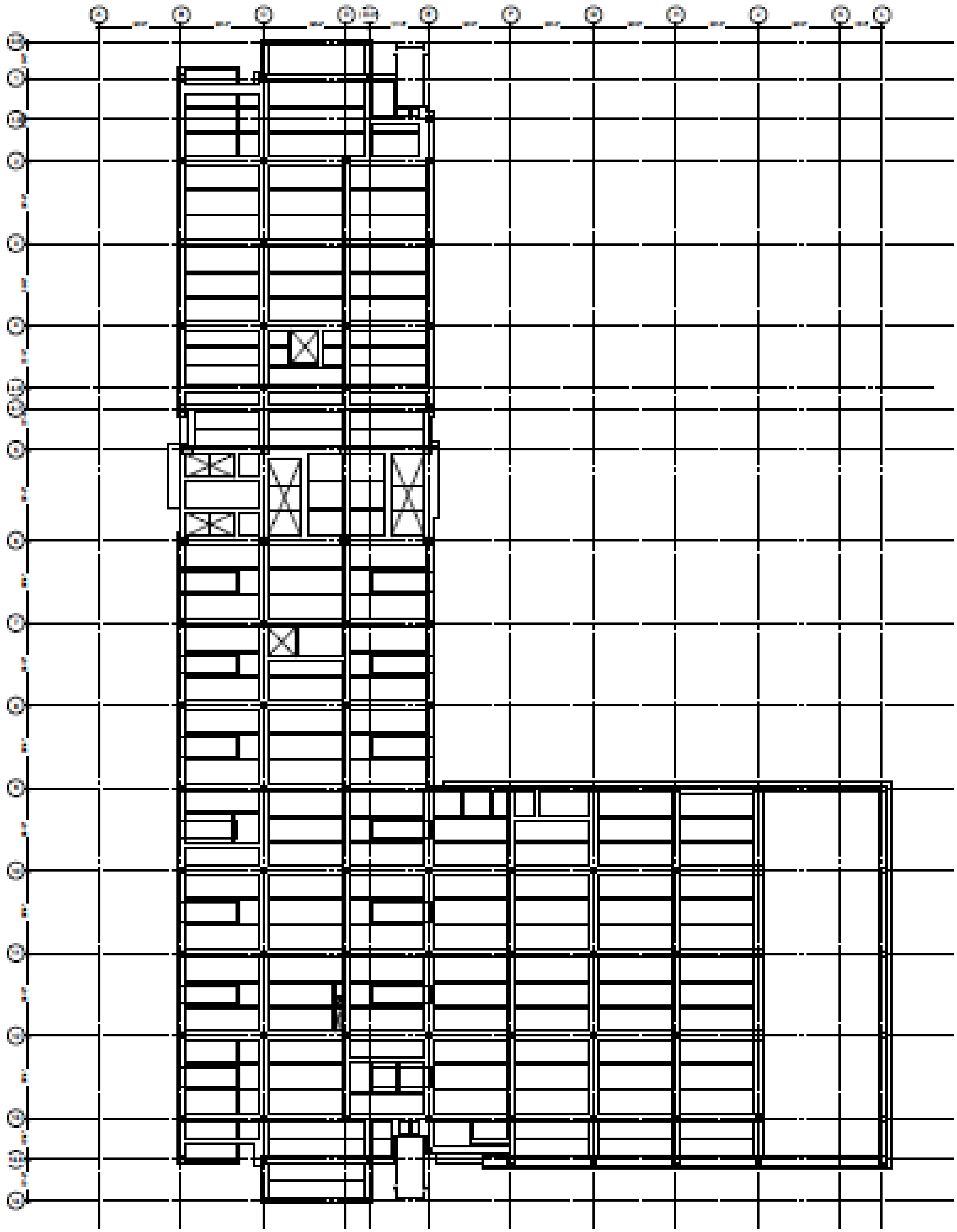


Figure 10 | Typical Structural Floor Plan for Floors 5-6 (Walter P. Moore)

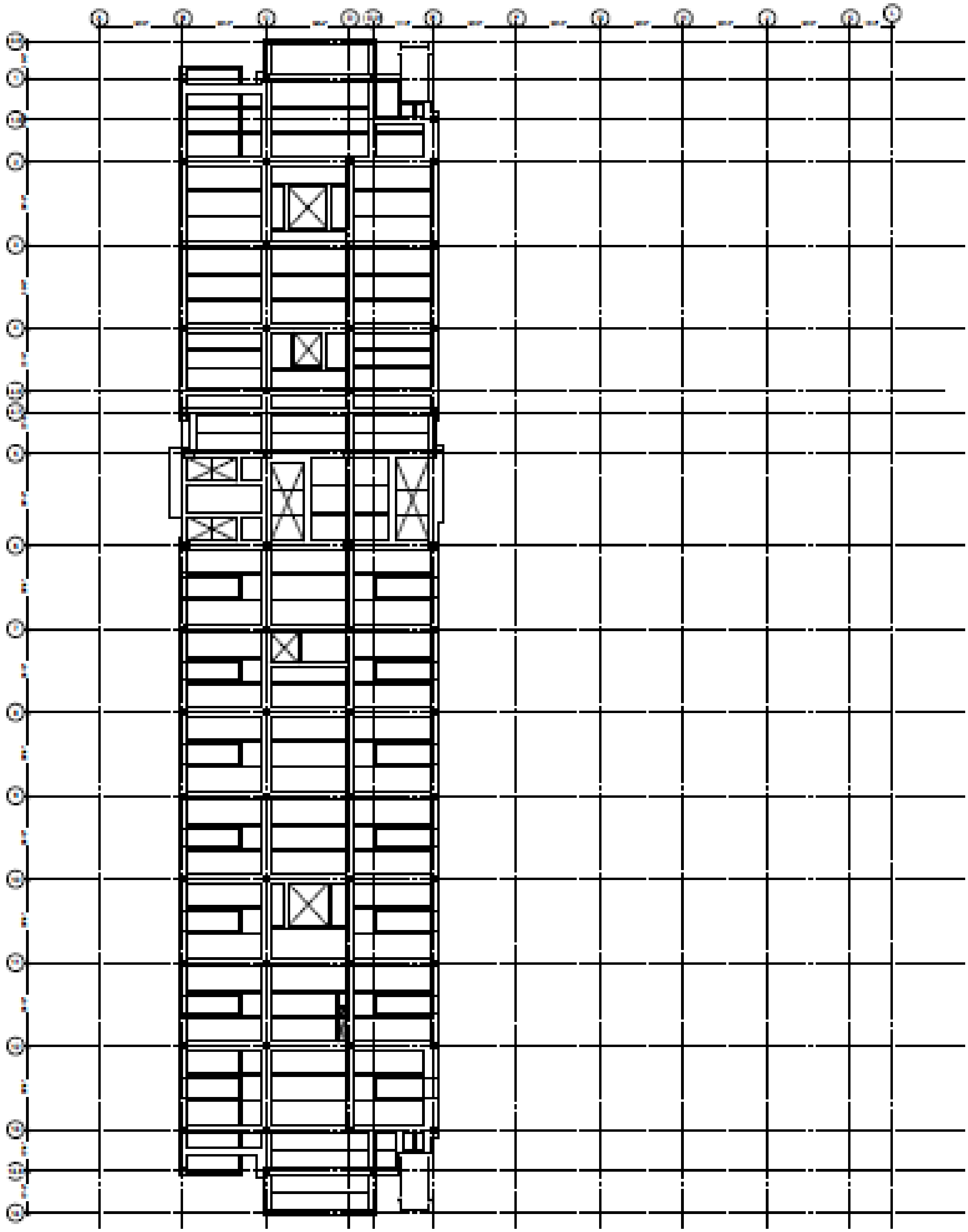


Figure 11 | Typical Structural Floor Plan for Floors 7-9 (Walter P. Moore)