Kyle MacDonald
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October 16, 2015

Dr. Thomas Boothby
The Pennsylvania State University
209 Engineering Unit A
University Park, PA 16802

Dear Dr. Boothby,

The following document, Technical Report III – Member Spot Check & Alternate Systems, is a detailed analysis of the gravity load resisting system of The Medical Center by means of applicable building codes and reference design standards. Through presentation of hand calculations as well as diagrammatic sketches, inclusive of material submitted in Notebook A, this report documents a typical bay representative of the existing gravity framing system of the building.

Accompanying the analysis of the existing gravity load resisting system is the assertion of three additional gravity load resisting system design alternatives. Each alternate system will be presented as an individual solution, analyzed and designed in terms of applicable strength and serviceability criteria. Ultimately, each proposed system, including the existing gravity framing system, will be presented, by means of a comparative study, to determine the most appropriate system design for The Medical Center.

Additionally, this report includes a building abstract which illustrates general building information, outlines the primary project team, and provides a brief description of the architectural schemes as well as the essential engineering system criteria used to design The Medical Center.

Thank you for your consideration and evaluation of this report.

Sincerely,

Kyle MacDonald

The Pennsylvania State University

Schreyer Honors College | Class of 2016

The Medical Center | Southeast, USA

Member Spot Check & Alternate Systems

Structural Notebook Submission B

Submitted to: Dr. Thomas Boothby, Advisor

Prepared by: Kyle M. MacDonald [Structural Option]

Prepared on: October 16th, 2015

Executive Summary

The Medical Center is a 570,000 square foot hospital located at the cornerstone of an expanding medical district. The building site is woven into the urban fabric of Southeast, USA. The urban context of the site, totaling 37 acres in size, influences the boundaries of design of this building project. Programmatically, The Medical Center houses 446 hospital beds as well as inpatient facilities such as medical offices, intensive care units, and dietary facilities. A high degree of programmatic intuition is demonstrated through the relationship between the environmental concerns of the region and the location of all mission-critical components within the building. The Medical Center was budgeted at $190 million.

Comprised of three identical, structurally isolated, L-shaped inpatient towers, The Medical Center is designed utilizing a reinforced concrete (RC) structural system. The structure features concrete slabs with pan joists, RC beams, RC girders, and vertical RC columns. These structural elements frame into composite timber piles and pre-cast, prestressed concrete piles, by means of a varied pile cap system, which are driven into the earth until a depth, below the original grade, of 62 ft. Concrete moment frames and concrete walls serve as the lateral force resisting system.

NBBJ Architects and Blitch Knevel Architects served as the joint-venture architects on this building project. Structural, MEP, and Fire Protection engineering services were provided by URS Corporation (recently AECOM), and IBA Consultants served as the exterior envelope design experts. The project was delivered by means of design-bid-build contract, and Skanska served as the Construction Manager at Risk on the project. The Medical Center began construction in December 2012 and is scheduled to be completed in November 2015.

The Medical Center was designed based on the Southeast, USA Building Code, associated with the International Building Code (IBC), 2009 edition. The American Society of Civil Engineers (ASCE) 7-05 was utilized as a reference standard. The building is scheduled to be completed in August 2015.

THE MEDICAL CENTER | SOUTHEAST, USA

GENERAL INFORMATION

Full Height . . . . 113 ft.
Number of Stories . . . . 7 above grade
Size of Building . . . . 570,000 sq. ft.
Cost of Building $190,000,000
Date of Construction . . . . Dec. 2012-Nov. 2015
Project Delivery Method . . . . Design-Bid-Build

IMAGES COURTESY OF NBBJ ARCHITECTS

PROJECT TEAM

Owner . . . . State of Louisiana
Construction Manager (at risk) . . . . Skanska
Architect . . . . NBBJ Architects
Architect (Joint Venture) . . . . Blitch Knevel Architects
Structural Engineers . . . . URS Corporation (AECOM)
MEP Engineers . . . . URS Corporation (AECOM)
Fire Protection . . . . URS Corporation (AECOM)
Exterior Envelope . . . . IBA Consultants

ARCHITECTURE

The inpatient towers feature a polished and refined design that influences the form and shape of the medical district of Southeast, USA. The L-shaped orthogonal design scheme introduces order and logic into the overall program of the building. A high degree of programmatic intuition is demonstrated through the relationship between the environmental concerns of the region and the location of the mission-critical components within the building.

MECHANICAL SYSTEMS

The indoor design conditions are defined as seen below:
Summer: 75°F db/50% RH | Winter: 70°F db/30% RH

LIGHTING AND ELECTRICAL SYSTEMS

The hospital is equipped with emergency power capable of sustaining mission-critical operation after a category 3 hurricane for up to a week with no outside support.

CONSTRUCTION

Due to the unfavorable and unpredictable conditions of the site soil, a 7.5% structural foundation allowance was allotted to account for any variability in pile length.

STRUCTURAL SYSTEMS

Foundation: Timber Composite Piles, Precast/Prestressed Concrete Piles, Pile Cap, Grade Beams

Framing: Concrete Frame Horizontal – Joist, Beam, Girder | Concrete Frame Vertical – Column | Concrete Slab

Lateral: Concrete Moment Frame – Detailed Lateral Connection at Column and Beam Interface

Kyle M. MacDonald | Structural Option | http://www.kym5182.wix.com/thesis

Table of Contents

[1] Introduction

1.1 Purpose

This report functions as a detailed analysis of typical bay framing of The Medical Center. Spot checks of typical members of the existing gravity framing system of The Medical Center will be performed, and evaluations of additional gravity framing design alternatives will be addressed. The Medical Center utilizes an existing pan joist, beam, girder, and slab reinforced concrete framing system. The alternate framing schemes are the following: non-composite steel framing system, composite steel framing system, and two-way flat plate concrete slab system.

1.2 Scope

The content of this report is divided into three major sections: gravity loads, typical member spot checks for gravity loads, and alternate framing systems for gravity loads. All gravity load information being presented was extracted from the previous documentation, Technical Report II – Building Codes, Specifications, and Loads (Notebook A). Additionally, this document discusses pertinent information with respect to site location and resource documentation. This information is framed by the context of the member analysis and alternate design of a typical bay of The Medical Center. Appendices are included at the end of the document in order to display original load calculations executed by URS Corporation (AECOM).

1.3 Site Location and Plan

The Medical Center sits at the cornerstone of an expanding medical district, contributing to an expansive network of hospitals in Southeast, USA. Nestled in between pockets of urban residential construction, The Medical Center briefly interrupts the major urban grid of the existing environment. Existing as a mission-critical facility, the building's proximity to a major network of highways enhances its public accessibility. The urban context of the site, totaling 37 acres in size, influences the boundaries of design of this building project (as seen in Figure 1 and Figure 2).

Figure 1 - Site Context (Macro)

Figure 2 - Site Context (Aerial)

1.4 Document List

- IBC 2009 (for existing analysis)
- IBC 2012 (for alternate design study)
- ASCE 7-05 (for existing analysis)
- ASCE 7-10 (for alternate design study)
- AISC Steel Manual, 14th Edition
- ACI 301, ACI 315, ACI 318
- USGS Seismic Design Maps
- Vulcraft Steel Deck Catalogue, 2008 Edition

[2] Gravity Loads

This section investigates the gravity loading of the structural system, inclusive of dead, live, and snow loads. Each load case is investigated separately but applied to the building structure in combination. The gravity loading information being presented was extracted directly from Technical Report II – Building Codes, Specifications, and Loads (Notebook A).

2.1 Dead Loads

Table 1 - Dead Loads

Dead Load	Load Value (psf)
Exterior Glazed Framing System	20
Exterior Precast Concrete Panel	50
Exterior Composite Metal Panel	15
Hospital Floor	60
Hung Load Allowance (Typical Floors)	8
Hung Load Allowance (Main Roof)	13
Roofing Allowance (W/O Pavers)	12
Roofing Allowance (W/ Pavers)	37

2.2 Live Loads

Table 2 - Live Loads

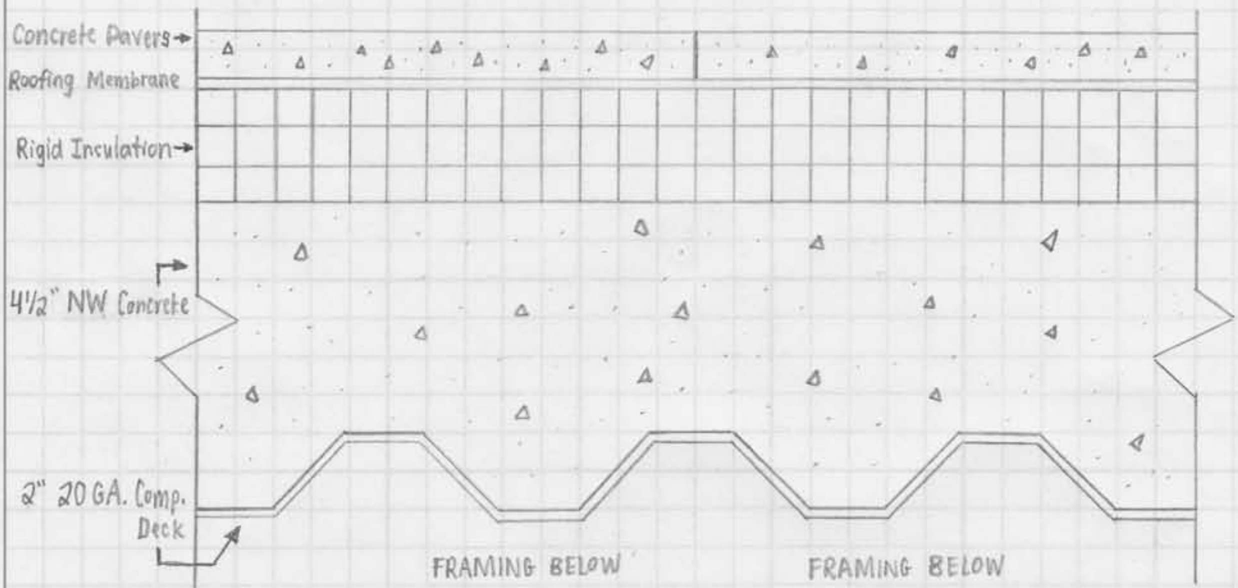
Live Load	Load Value (psf)
Offices	50
Corridors (1st Floor)	100
Corridors (Other)	80
Operating Rooms	60
Patient Rooms	40
Lobbies, Stair and Exit Ways	100
Mechanical Rooms	125 (or equipment weight)

2.3 Snow Loads

Due to the climate region of the building site, the applicable reference standard dictates a ground snow load equal to zero pounds per square foot; therefore, snow conditions will not imposed any load on the building structure (and can be rightly omitted from design load considerations).

GRAVITY LOADS

- Roof Construction (Diagram & Loading)



- Roof Loading (Dead)

Concrete Pavers	:	13 psf
Roofing Membrane	:	1 psf
Rigid Insulation	:	1½ psf
4½" NW Concrete	:	57 psf
2" 20GA Comp. Deck	:	2 psf
Framing	:	6½ psf
Miscellaneous	:	9 psf

Total Assembly Dead Load = 90psf

Framing per Typical Bay

Area : approx. 30' x 32' = 960 sf → 6½psf

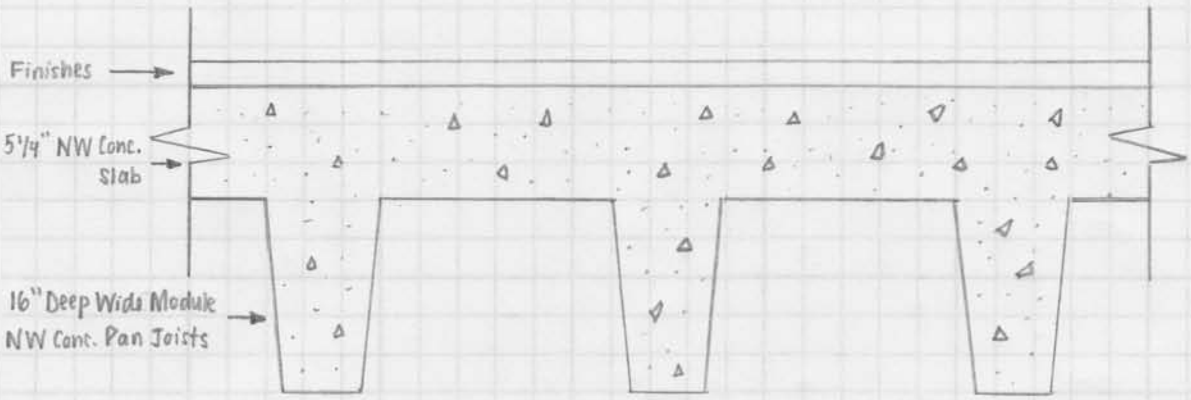
- Roof Loading (Live)

Roof Live Load : 20 psf

(according to ASCE 7-05 Table 4-1 → Minimum Uniformly Distributed Live Loads

GRAVITY LOADS (cont.)

• Floor Construction (Pan Joist System)



• Floor Loading (Pan Joist System)

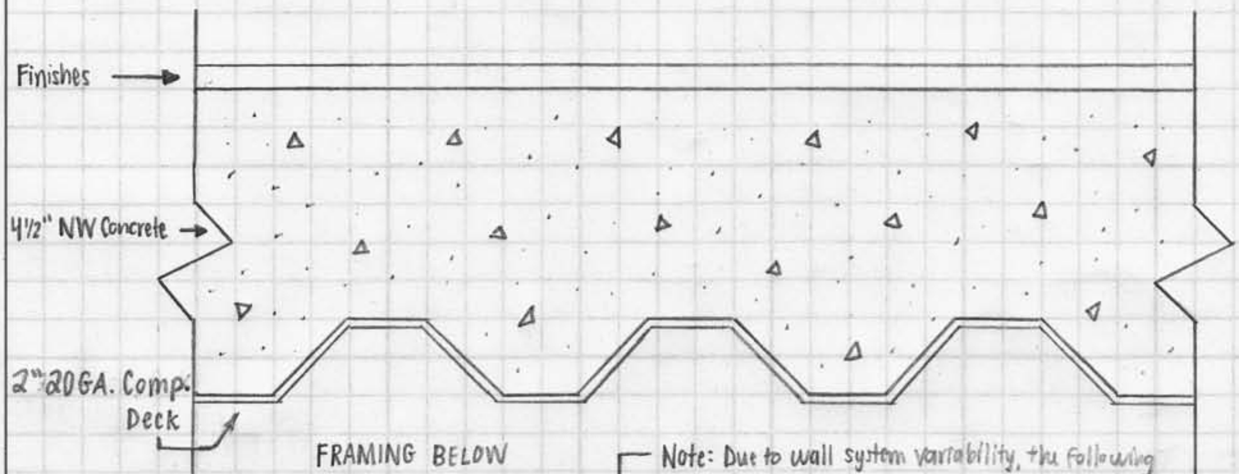
Finishes	:	2 psf
Conc. Slab	:	67 psf
Pan Joists	:	14 1/2 psf
Framing	:	6 1/2 psf
Ceiling	:	5 psf
Miscellaneous	:	15 psf

Total Assembly Dead Load = 110 psf

The pan joist floor assembly system relies on 16 inch deep pan joists, spaced at 66 inches apart, to provide sections of increased stiffness. The framed floor (structural) follows a defined module that dictates the framing pattern of the building.

GRAVITY LOADS (cont.)

- Floor Construction (Slab and Deck System)

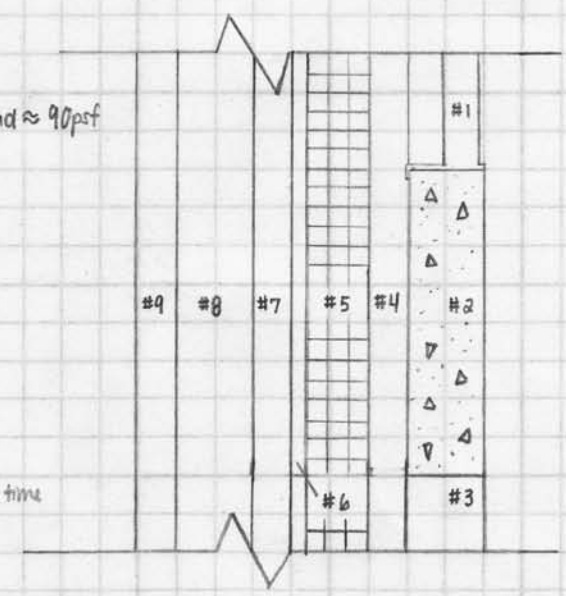


Note: Due to wall system variability, the following diagram is representative of the types of material included in exterior enclosure wall assemblies

- Floor Loading (Slab and Deck System)

Finishes	:	2 psf	Total Dead Load ≈ 90 psf
Conc. Slab	:	57 psf	
Steel Deck	:	2 psf	
Framing	:	6½ psf	
Ceiling	:	5 psf	
Miscellaneous	:	15 psf	

- Exterior Enclosure Construction



#1 : Curtain Wall / Glazed Framing System (GFS)
#2 : Precast Concrete Wall Panel
#3 : Composite Metal Wall Panel
#4 : Air Gap
#5 : Rigid Insulation
#6: Vapor Barrier
#7: Gypsum Board
#8: Metal Stud
#9: Gypsum Board

- Exterior Enclosure Loading

Curtain Wall / GFS	:	20 psf	only consider one at a time
Precast Conc. Panel	:	50 psf	
Comp. Metal Panel	:	15 psf	
Air Gap	:	0 psf	
Insulation	:	1½ psf	
Vapor Barrier	:	½ psf	
Gypsum Board	:	5½ psf (2¾ psf each)	
Metal Stud Framing	:	5	

Total Dead Load = 62½ psf

SNOW LOADS

- Flat Roof Snow Loads → $p_f = 0.7 C_e C_t I_s p_g = 0.7(1.0)(1.0)(1.2)(0) = 0$ psf

$p_g = 0$ psf = ZERO (ASCE 7-05 Figure 7-1)

Terrain Category: C (ASCE 7-05 Table 7-2)

$C_e \longrightarrow 1.0$ (ASCE 7-05 Table 7-2)

$C_t \longrightarrow 1.0$ (ASCE 7-05 Table 7-3)

Occupancy Category: IV (ASCE 7-05 Table 1-1)

$I_s \longrightarrow 1.2$ (ASCE 7-05 Table 7-4)

Note: SNOW LOAD WILL NOT CONTROL IN ANY LOAD COMBINATION CASE

[3] Wind Loads

This section investigates the lateral loading, due to wind pressures, of the structural system. Figure 6 illustrates the various lateral force resisting systems used in the building. The original design wind load calculations are recorded in Appendix B.

3.1 Calculations

The following calculations follow the simplified procedure, as outlined in ASCE 7-05. The calculations that follow exist as a representative set of seismic design load calculations.

Table 3 - Wind Design Parameters

Design Parameter	Applicable Information
Occupancy Category	IV
Exposure Category	C
Basic Wind Speed (v)	150mph
Importance Factor (I)	1.15
Directionality Factor (K_d)	0.85
Topographic Factor (K_{zt})	1.0
Enclosure Classification	Enclosed Building

Gust Effect Factor

h =	113.0 ft
B =	62.5 ft
$\bar{z}$ (0.6h) =	67.8 ft

Flexible structure if natural frequency < 1 Hz (T > 1 second).
However, if building h/B < 4 then probably rigid structure (rule of thumb).
h/B = 1.81 Therefore, probably rigid structure

Figure 3 - Building Dimensions vs. Rigidity

Rigid Structure

$\bar{\varepsilon}$ =	0.20
ℓ =	500 ft
z_{min} =	15 ft
c =	0.20
g_Q, g_v =	3.4
L_z =	577.4 ft
Q =	0.88
I_z =	0.18
G =	0.87 use G = 0.85

Figure 4 - Gust Effect Factor Parameters

Surface Pressures (psf)	Wind Normal to Ridge (psf)				Wind Parallel to Ridge (psf)				
	B/L = 0.28		h/L = 1.81			L/B = 3.63		h/L = 0.50	
Surface	Cp	q_hGC_p	w/+q_iGC_{pi}	w/-q_hGC_{pi}	Dist.*	Cp	q_hGC_p	w/ +q_iGC_{pi}	w/ -q_hGC_{pi}
Windward Wall (WW)	0.80	49.7	see table below			0.80	49.7	see table below	
Leeward Wall (LW)	-0.50	-31.1	-44.2	-17.9		-0.22	-13.6	-26.7	-0.4
Side Wall (SW)	-0.70	-43.5	-56.7	-30.3		-0.70	-43.5	-56.7	-30.3
Leeward Roof (LR)		**				Included in windward roof			
Windward Roof: 0 to h/2*	-1.04	-64.6	-77.8	-51.5	0 to h/2*	-0.90	-55.9	-69.1	-42.8
> h/2*	-0.70	-43.5	-56.7	-30.3	h/2 to h*	-0.90	-55.9	-69.1	-42.8
					h to 2h*	-0.50	-31.1	-44.2	-17.9
					> 2h*	-0.30	-18.6	-31.8	-5.5

Figure 5 - Wind Surface Pressure Parameters

Windward Wall Pressures at "z" (psf)						Combined WW + LW	
			Windward Wall			Normal	Parallel
z	Kz	Kzt	q_zGC_p	w/+q_iGC_{pi}	w/-q_hGC_{pi}	to Ridge	to Ridge
0 to 15'	0.85	1.00	32.5 psf	19.3 psf	45.7 psf	63.6 psf	46.1 psf
20.0 ft	0.90	1.00	34.5	21.4	47.7	65.6	48.1
25.0 ft	0.95	1.00	36.2	23.0	49.4	67.3	49.8
30.0 ft	0.98	1.00	37.6	24.4	50.8	68.7	51.2
40.0 ft	1.04	1.00	40.0	26.8	53.1	71.0	53.5
50.0 ft	1.09	1.00	41.9	28.7	55.0	73.0	55.5
60.0 ft	1.14	1.00	43.5	30.4	56.7	74.6	57.1
70.0 ft	1.17	1.00	45.0	31.8	58.1	76.0	58.5
80.0 ft	1.21	1.00	46.2	33.1	59.4	77.3	59.8
90.0 ft	1.24	1.00	47.4	34.2	60.6	78.5	61.0
100.0 ft	1.27	1.00	48.5	35.3	61.6	79.5	62.0
h= 113.0 ft	1.30	1.00	49.7	36.6	62.9	80.8	63.3

Figure 6 - Wind Pressures

<u>Note</u>: The above Figure 6 includes the windward wall pressures (in both the positive and negative interior pressure cases) as well as the combined windward wall and leeward wall pressures in both the (N-S) and (E-W) directions.

WIND LOADS

- Wind Design Criteria

Occupancy Category : IV (ASCE 7-05 Table 1-1)

Basic Wind Speed (3 sec. gust) : 150 mph (ASCE 7-05 Figure 6-1)

Exposure Category : C (ASCE 7-05 Chapter 6)

Importance Factor (I) : 1.15 (ASCE 7-05 Table 6-1)

Wind Directionality Factor (k_d) : 0.85 (ASCE 7-05 Table 6-4)

Topographic Factor (k_{zt}) : 1.0 (ASCE 7-05 6.5.7.2)

- Wind Parameter Investigation (calculate G)

Rigid or Flexible → Note: Rule of Thumb - if $h/B < 4$, then building probably rigid

$h/B = 113\text{ft.} / 62.5\text{ft} = 1.808 < 4 \therefore$ rigid

Due to rigidity, take $G = 0.85$ or Calculate $G = 0.925\left((1 + 1.7 g_Q I_{\bar{z}} Q) / 1 + 1.7 g_v I_{\bar{z}}\right)$

$g_Q = g_v = 3.4$

$I_{\bar{z}} = c\left(33/\bar{z}\right)^{1/6} = 0.2\left(33/0.6(113)\right)^{1/6} = 0.177$

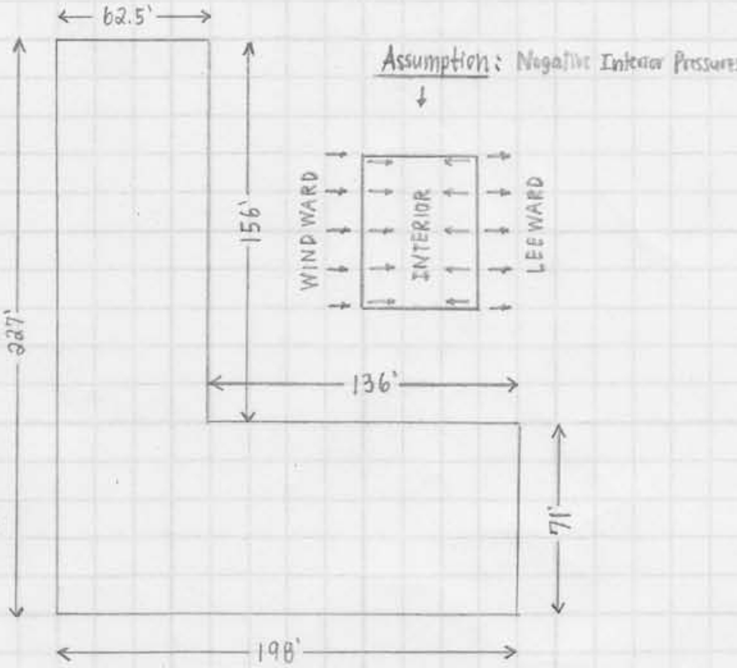
$Q = \sqrt{1/(1 + 0.63((B+h)/L_{\bar{z}})^{0.63})} = \sqrt{1/1 + 0.63((62.5 + 113)/577.45)^{0.63}} = 0.8779$

$L_{\bar{z}} = \ell\left(\bar{z}/33\right)^{\bar{\epsilon}} = 500\left(0.6(113)/33\right)^{(1/5.0)} = 577.45$

$G = 0.925\left((1 + 1.7(3.4)(0.177)(0.8779)) / 1 + 1.7(3.4)(0.177)\right) = 0.867$ → use $G = 0.85$

WIND LOADS (cont.)

Building Plan - Inpatient Tower



- Example Calculation of "P"

at $z = 113'$

$q_h = 0.00256\, k_z k_{zt} k_d V^2$ where $k_z = 1.2925$

interpolate between

$= 0.00256(1.2925)(1)(0.85)(150)^2$ $z=100$, $k_z = 1.26$

$z=120$, $k_z = 1.31$

$= 63.3$

at $z = 93'$

$q_z = 0.00256\, k_z k_{zt} k_d V^2$ where $k_z = 1.246$

interpolate between

$= 0.00256(1.246)(1)(0.85)(150)^2$ $z=90$, $k_z = 1.24$

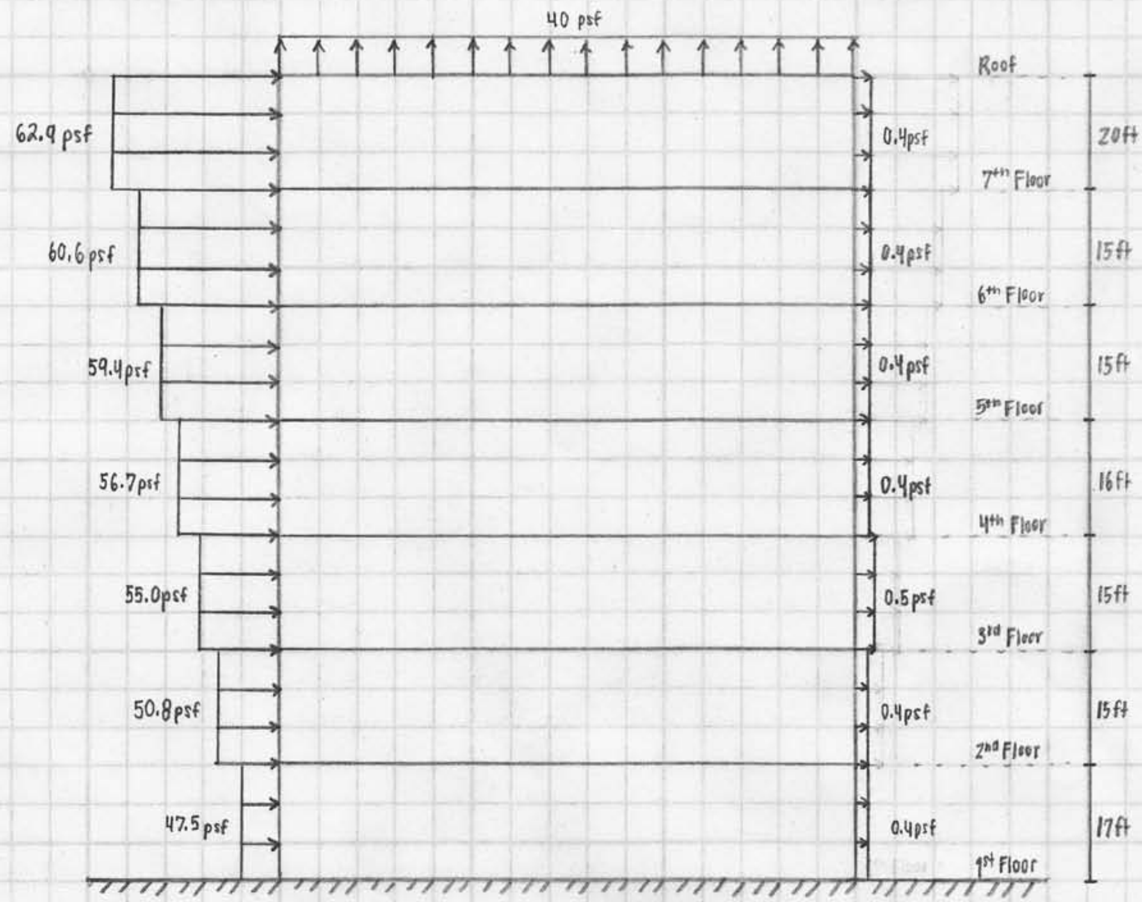
$z=100$, $k_z = 1.26$

$= 61.0$

$$P = qGC_p - q_i(GC_{pi}) = 61(0.85)(\underbrace{0.8}_{C_p,\ \text{windward}}) - 63.3(\underbrace{-0.18}_{\text{enclosed building}}) = 52.9\text{ k}$$

Note: See Wind Loading Diagram and Base Shear Calculations on next page.

WIND LOAD DIAGRAM



Wind Base Shear (E-W) → Representative Calculation

(47.9 psf)(227')(17') + (51.2psf)(227')(15') + (55.5psf)(227')(15') + (57.1psf)(227')(16')

+ (59.8psf)(227')(15') + (61.0psf)(227')(15') + (63.4 psf)(227')(20')

≃ 1468.55 k → 1470 k (Wind Loads control the lateral design of the building)

[4] Seismic Loads

This section investigates the lateral loading, due to seismic ground accelerations, of the structural system. Table 4 can be referenced for a list of lateral force resisting systems used in the building. The original design seismic load parameters and considerations are recorded in Appendix C.

Table 4 - Lateral Force Resisting Systems

LFRS	Direction of Resistance	R-Value
Intermediate Concrete Moment Frames	North-South	5
Intermediate Concrete Moment Frames	East-West	5

4.1 Calculations

The following calculations follow the Equivalent Lateral Force procedure, as outlined in ASCE 7-05. All ground motion parameters were determined referencing the USGS seismic design maps. The calculations that follow exist as a representative set of seismic design load calculations.

SEISMIC LOADS (ELF according to ASCE 7-05 Table 12.6-1)

- Design Criteria

Occupancy Category : IV (ASCE 7-05 Table 1-1)

Importance Factor (I_e) : 1.5 (ASCE 7-05 Table 11.5-1)

Soil Site Class : E (ASCE 7-05 11.4.2)

Seismic Design Category : C (ASCE 7-05 Table 11.6-2)

- Ground Motion Parameters and Calculations

S_s = 11.0% (USGS Ground Motion Parameter Application - verified by ASCE 7-05 EQ Ground Motion Maps)

S_1 = 4.8% (USGS Ground Motion Parameter Application - verified by ASCE 7-05 EQ Ground Motion Maps)

F_a = 2.5 (ASCE 7-05 Table 11.4-1)

F_v = 3.5 (ASCE 7-05 Table 11.4-2)

$S_{MS} = F_a S_s = (2.5)(0.11) = 0.275$

$S_{M1} = F_v S_1 = (3.5)(0.048) = 0.168$

$S_{DS} = (2/3) S_{MS} = (2/3)(0.275) = 0.183 \longrightarrow 18.3\%$

$S_{D1} = (2/3) S_{M1} = (2/3)(0.168) = 0.112 \longrightarrow 11.2\%$

- Lateral System - Intermediate Reinforced Concrete Moment Frames

Response Modification Factor (R) = 5 (ASCE 7-05 Table 12.2-1)

Overstrength Factor (Ω) = 3 (ASCE 7-05 Table 12.2-1)

Deflection Amplification Factor (C_d) = 4 1/2 (ASCE 7-05 Table 12.2-1)

- Building Period Calculation

$T = C_t h_n^x = (0.016)(113)^{0.9} = 1.127s$ (ASCE 7-05 Table 12.8-2)

$T_L = 12s$ (ASCE 7-05 Figure 22-15)

$T < T_L$ $\therefore$ use $C_s = S_{DS}/(R/I_e) \leq S_{D1}/(R/I_e)(T)$

SEISMIC LOADS (continued)

- Seismic Response Coefficient

$C_s = S_{DS}/(R/I_e) = 0.183/(5/1.5) = 0.0549 \leq C_s = S_{D1}/(R/I_e)(T) = 0.112/(5/1.5)(1.127) = 0.0298$

Use $C_s = 0.0298$

- Total Dead Load (W) = Dead Load + 20% snow load (on roof) for $P_s \geq 30psf$ (in this case, snow load not relevant)

Roof Load: $W_{RF} = [(226.92' \times 62.41') + (136' \times 71.17')](90psf) + (226.92' + 62.41' + 155.75' + 136' + 71.17' + 198.41')(20'/2)(62.5psf)$
$= 2677353 lb = 2677.353k$

Floor load: $W_{FL} = [(226.92' \times 62.41') + (136' \times 71.17')](110psf)(7) + (226.92' + 62.41' + 155.75' + 136' + 71.17' + 198.41')(103')(62.5psf)$
$= 23833693 lb = 23833.693k$

Total Load: $W_{RF} + W_{FL} = 2677.353k + 23833.693k = 26511k$

- Seismic Base Shear

$V = C_s W = (0.0298)(26511k) = 790k$ (Seismic Loads do not control the lateral design of the building)

For C_{vx} calculations:

$W_2 = [(226.92' \times 62.41') + (136' \times 71.17')](110psf) + (850.66')(17/2 + 15/2)(62.5psf) = 3473.170$

$W_3 = (23841 ft^2)(110psf) + (850.66')(15/2 + 15/2)(62.5psf) = 3420.004k$

$W_4 = (23841 ft^2)(110psf) + (850.66')(15/2 + 16/2)(62.5psf) = 3446.587k$

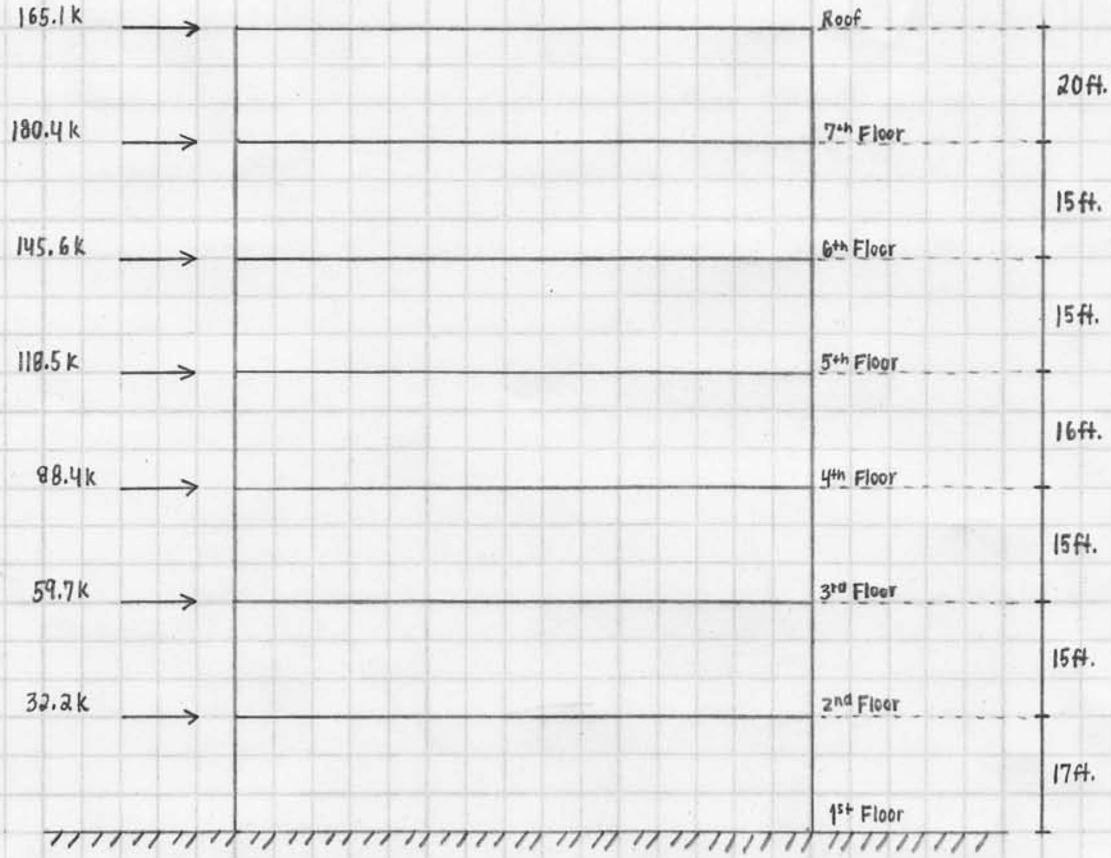
$W_5 = (23841 ft^2)(110psf) + (850.66')(16/2 + 15/2)(62.5psf) = 3446.587k$

$W_6 = (23841 ft^2)(110psf) + (850.66')(15/2 + 15/2)(62.5psf) = 3420.004k$

$W_7 = (23841 ft^2)(110psf) + (850.66')(15/2 + 20/2)(62.5psf) = 3552.919k$

$W_{Roof} = (23841 ft^2)(90psf) + (850.66')(20/2)(62.5psf) = 2677.353k$

SEISMIC LOAD DIAGRAM



Find Story Weights

Level	$C_{vx} = W_x h_x^k / \sum_{i=1}^{n} w_i h_i^k$ where $\sum_{i=1}^{n} w_i h_i^k = 1447231$ (in this case)
2	$C_{vx} = (3473)(17)/1447231 = 0.0408 \times 790k = 32.2k$
3	$C_{vx} = (3420)(32)/1447231 = 0.0756 \times 790k = 59.7k$
4	$C_{vx} = (3446)(47)/1447231 = 0.1119 \times 790k = 88.4k$
5	$C_{vx} = (3446)(63)/1447231 = 0.1500 \times 790k = 118.5k$
6	$C_{vx} = (3420)(78)/1447231 = 0.1843 \times 790k = 145.6k$
7	$C_{vx} = (3553)(93)/1447231 = 0.2283 \times 790k = 180.4k$
Roof	$C_{vx} = (2677)(113)/1447231 = 0.2090 \times 790k = 165.1k$

[5] Flood Loads

Due to the region's environmental conditions, the building is required to withstand flood design loads. The building was designed for hydrodynamic flow per ASCE 24-05. An assumed flood elevation was established at 15 ft. above mean sea level, and the flow velocity was considered to be 10 ft./sec. The advisory base flood elevation map as well as the flood insurance rate map were utilized in the consideration of imposed flood loading on the building.

Due to the scope of the report, no further discussion or consideration of flood loading will take place. The omission of flood loading is in direction response to the scope of the report and does not exist as a commentary on the importance of flood loading consideration within the design conditions of this building.

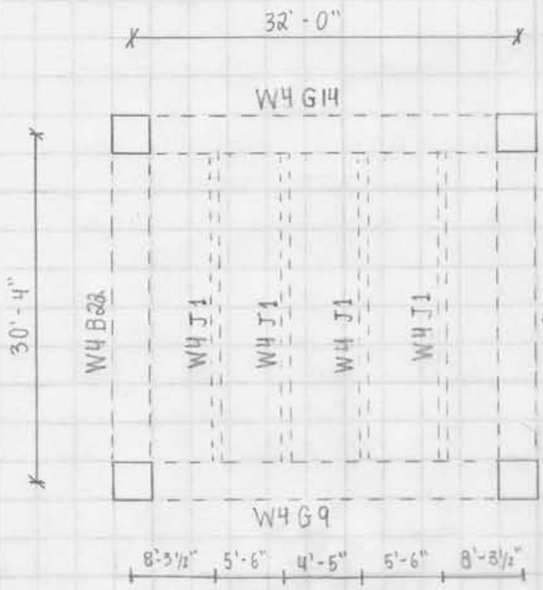
[6] Typical Member Spot Checks for Gravity Loads

This section investigates typical member design for gravity loads applied to the The Medical Center's existing gravity load resisting system. Spot checks will be performed for both strength and serviceability requirements. The spot checks include investigation of the following conditions: slab investigation (in conjunction with pan-joist investigation), joist moment capacity, beam moment capacity, girder moment capacity and deflection limits, and column axial capacity (for an interior and exterior column).

6.1 Calculations

The following calculations exist as spot checks for gravity loads applied to typical members in the building's existing gravity framing system. The calculations that follow exist as a representative set of spot check calculations.

GRAVITY LOAD SPOT CHECKS - typical members for existing gravity framing system



TYPICAL BAY FRAMING

W4 = west tower, 4th floor
J1 = joist one
B22 = beam twenty-two
G9 = girder nine
G14 = girder fourteen

Mark	Width	Depth	Bot. Bars	Top Bars	Stirrups
W4 J1	8"	21.25"	(2) #8	(2) #6	-
W4 B22	35"	21.25"	(4) #9	(4) #9	(19)@4"
W4 G9	42"	21.25"	(7) #7	(6) #9	(19)@4"
W4 G14	32"	30"	(6) #8	(6) #9	(15)@6"

*Note: all stirrups are #4 bars in size

Dead Loads

Slab = ... : 65 psf

Beams = ... : 10 psf (5 psf beam / 5 psf girder)

Ceiling = ... : 2 psf

Finishes = ... : 2 psf

Misc / Equip / Mech / Collateral = ... : 5 psf

Total : 84 psf

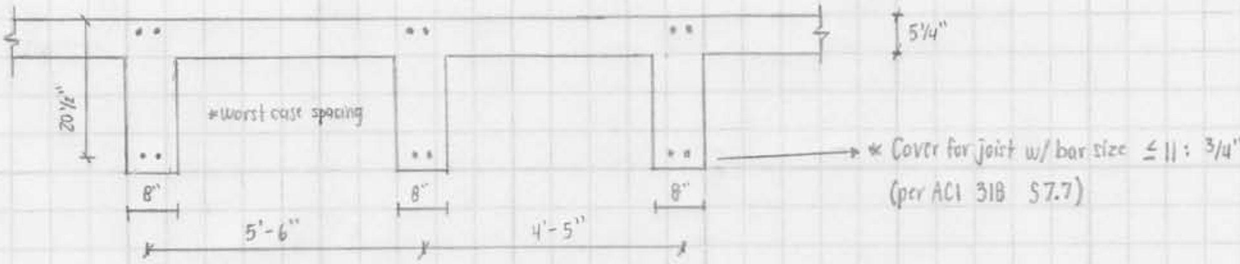
Live Loads

Cooridors (above 1st floor) ...... : 80 psf ← Controlling LL case

Patient Rooms .... : 40 psf

Joist Spot Check (W4J1)

Note: due to pan-joist configuration, treat section as a T-beam and analyze appropriately



Note: pan joist configuration is tapered (w/ 8" b_w at bottom - tapered end of section) ∴ for simplification and conservatively, a rectangular joist section will be analyzed, w/ a continuous b_w = 8" through the height of the section

b_w = 8 in
d = 20.5 in
$t_s = h_f$ = 5.25 in

f'_c: 4000 psi = 4 ksi
f_y = 60,000 psi = 60 ksi

Effective Flange Width (b_e)

b_e = min
- $2(8h_f) + b_w = 2(8 \times 5.25") + 8" = 92"$
- span/4 = 364"/4 = 91"
- b_w + clear spacing = 8" + 58" = 66" ← controlling b_e

Existing Gravity Framing System:

joist details

dimension: depth = 21.25", width = 8"

reinforcement: top: (2) #6, bot: (2) #8, stirrups/transverse: none

spacing: typ. 5½' o.c. (special spacing conditions noted on plans)

Joist Reactions

$1.2D + 1.6L = 1.2(74) + 1.6(80) = 217$ psf

217 psf (5.5 ft) = 1194 plf = 1.194 klf

↑
Controlling Case

OR

$1.4D = 1.4(74) = 104$ psf

104 psf (5.5 ft) = 574 plf = 0.574 klf

Reaction = 1.194 klf × (30.333) = 36 k

* for sake of simplicity, assume all spacing to be worst case spacing of 5.5 ft between pan joists

Live Load Reductions

joists: do not qualify for live load reduction because $K_{LL} A_T$ equates to $2(5.5)(30.333) = 333.67\,ft^2 < 400\,ft^2$

beams: $L_i = \begin{cases} 0.5(80) = 40\,psf \\ 80\left(0.25 + 15/\sqrt{30.333 \times 64}\right) = 49\,psf \end{cases}$ where $30.333 \times 64 = 1941 > 400$

girders: $L_i = \begin{cases} 0.5(80) = 40\,psf \\ 80\left(0.25 + 15/\sqrt{32 \times 40.833}\right) = 54\,psf \end{cases}$ where $32 \times 40.833 = 1307 > 400$

Joist Moment Calculations - use $D_{Total} - D_{Beams} = 84 - 10 = 74\,psf$

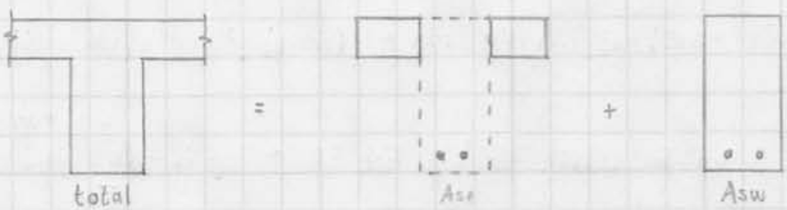
* $1.2D + 1.6L = 1.2(74) + 1.6(80) = 217\,psf$

$217\,psf\,(5.5\,ft) = 1194\,plf = 1.194\,klf$ ← controlling case

* $1.4D = 1.4(74) = 104\,psf$

$104\,psf\,(5.5\,ft) = 574\,plf = 0.574\,klf$

* $M_u = w\ell^2/8 = 1.194(30.333)^2/8 = 137\,ft \cdot k \times 12\,in/ft = 1644\,k\text{-}in$



*Analyze pan joist configuration like a T-section beam (RC)

Assuming $\phi = 0.9$ (tension) - check regular RC section first

$M_n = M_u/\phi = A_s f_y (d - a/2) \Rightarrow A_s = 1644/0.9 \times 60 \times (20.5 - (5.25/2)) = 1.70\,in^2$

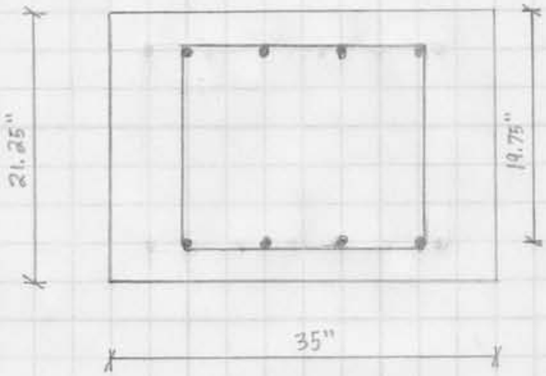
$0.85 f'_c ab = T = A_s f_y \Rightarrow a = A_s f_y / 0.85ab = (1.70)(60)/0.85(5.25)(8) = 2.86'' < h_f = 5.25''$ ∴ compression zone remains in the flange (slab)

Bot. Bars = (2) #8 : $A_s = 2(0.79) = 1.58\,in^2 < 1.70\,in^2$ (but loads were calculated very conservatively) → (2) #9

Top Bars: (2) #6 = $A_s = 2(0.44) = 0.88\,in^2 > A_{s,min} = \left[3\sqrt{4000}/60000\right](8)(20.5) = 0.52\,in^2$

Beam Spot Check (W4B22)

Note: due to regularity of beam section, the beam will be analyzed as a rectangular RC section



Top Bars: : (4) #9 = 4in²

Bottom Bars: : (4) #9 = 4in²

Stirrups: : (19) #4 @ 4" o.c.

*Note: concrete reinforcement cover = 1½" per ACI 318 §7.7

$0.85 f'_c a b + A_s' f_s' = A_s f_y$

$\downarrow$

$0.85 f'_c \beta_1 c b + A_s' \left(0.003(c-d')/c\right)(E) = A_s F_y$

$0.85(4)(0.85)c(35) + 4\left(0.003(c-1.5)/c\right)29000 = 4(60)$

$101.15c + (0.012 - 0.018/c)29000 = 240$

$101.15c^2 + 348c - 762 = 0 \longrightarrow c = 1.52\text{in} \rightarrow a = \beta_1 c = 0.85(1.52) = 1.30\text{in}$

$\varepsilon_s = 0.003(d-c)/c = 0.003(19.75-1.52)/1.52 = 0.035 > 0.005$ ✓

$\varepsilon_s' = 0.003(c-d')/c = 0.003(1.52-1.5)/1.52 = 3.947\times10^{-5} < 0.005$ ✗ does not yield

$M_{n1} = A_{s1} f_y (d - a/2) = 4(60)(19.75 - 1.3/2) = 4584\ \text{k·in} = 382\ \text{k·ft}$

$M_{n2} = A_{s2} f_y (d-d') = A_s' f_s' (d-d') = 4(60)(19.75-1.5) = 4380\ \text{k·in} = 365\ \text{k·ft}$

$M_n = 382\ \text{k·ft} + 365\ \text{k·ft} = 747\ \text{k·ft}$

$\phi M_n = 0.9(747) = 672\ \text{k·ft}$

Beam Moment Calculations - use $D_{Total} - D_{Girder} = 84 - 5 = 79 psf$

* $1.2D + 1.6L = 1.2(79) + 1.6(48) = 172$ psf

172 psf $(8.29167) = 1426$ plf $= 1.426$ klf ← controlling case

* $1.4D = 1.4(79) = 111$ psf

111 psf $(8.29167) = 920$ plf $= 0.920$ klf

* $M_u = w l^2/8 = (1.426)(30.333)^2/8 = 164$ ft·k $\times 12$ in/ft $= 1968$ k·in

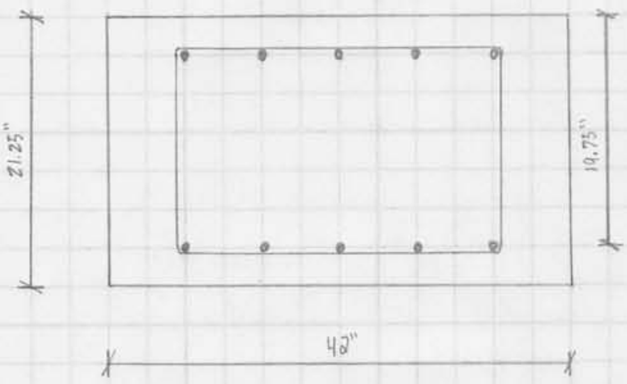
$\phi M_n = 672$ k·ft $> M_u = 164$ k·ft ✓

Reinforcement Check

$A_{s,min} = 3\sqrt{4000}\,/60000\,(35)(19.75) = 2.19 in^2 < 4 in^2$ ✓

Girder Spot Check - spot check only one girder case (W469)

Note: due to regularity of girder section, the girder will be analyzed as a rectangular RC section



Top Bars: (6) #9 = $6 in^2$

Bottom Bars: (7) #7 = $4.2 in^2$

Stirrups: (19) #4 @ 4" o.c.

* Note: concrete reinforcement cover = 1 1/2" per ACI 318 S7.7

$0.85 f'_c ab + A_s' f_s' = A_s f_y$

$0.85 f'_c \beta_1 cb + A_s' (0.003 (c-d')/c) E = A_s f_y$

$0.85 (4)(0.85) c (42) + 6 (0.003 (c-1.5)/c) 29000 = 4.2(60)$

$121.38c + 522 - 783/c = 252$

$121.38c^2 + 270c - 783 = 0 \rightarrow c = 1.66 in \rightarrow a = \beta_1 c = (0.85)(1.66) = 1.41 in$

$\varepsilon_s = 0.003(d-c)/c = 0.003(19.75 - 1.66)/1.66 = 0.0327 > 0.005$ ✓

$\varepsilon_s' = 0.003(c-d')/c = 0.003(1.66-1.5)/1.66 = 2.892 \times 10^{-4} < 0.005$ × does not yield

$M_{n1} = A_{s1} f_y (d - a/2) = 4.2(60)(19.75 - 1.41/2) = 4800\ k\cdot in /12 = 400\ k\cdot ft$

$M_{n2} = A_{s2} f_y (d-d') = A_s' f_s' (d-d') = 6(60)(19.75-1.5) = 6570\ k\cdot in /12 = 547.5\ k\cdot ft$

$M_n = 400 + 547.5 = 947.5\ k\cdot ft$

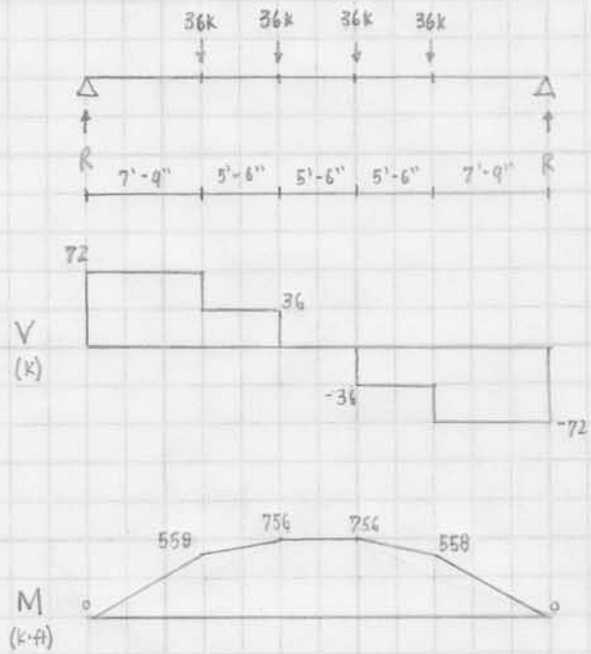
$\phi M_n = 0.9(947.5) = 853\ k\cdot ft$

$M_u = 756\ k\cdot ft < \phi M_n = 853\ k\cdot ft$ ✓

$A_{s,min} = 3\sqrt{4000}\ (42)(19.75) / 60000 = 2.62 in^2 < 4.2 in^2$ ✓

Girder Moment Calculations

Assumption: joist collect gravity load and distribute to girder by applying a concentrated point load
(standard assumption throughout all design alternatives involving girder framing)



$M_u = 756 \text{ k·ft} < \phi M_n = 853 \text{ k·ft}$ ✓

Dead Load

74 psf (from previous joist calculations)

Live Load

80 psf (from previous joist calculations)

Point Loads

$P = [1.2(74) + 1.6(80)](5.5 \text{ ft})(30.333 \text{ ft}) / 1000 \text{ lb/k} = 36 \text{ k}$

Reactions

$R = 4(P)/2 = 4(36)/2 = 72 \text{ k}$

Girder Deflection Calculations

(from previous)

$\Delta_{limit} = L/360 = 32(12in/ft)/360 = 1.067in$ $\quad$ $(74psf + 80psf)5.5ft \times 38.333ft = 25.7k / 32ft = 0.802 klf$

$\Delta_{actual} = 5w\ell^4/384EI$ where $w = 0.802 klf$, $\ell = 32ft$ (384in), $E = 29000ksi$, $I = bh^3/12 = 42(21.25)^3/12 = 33585 in^4$

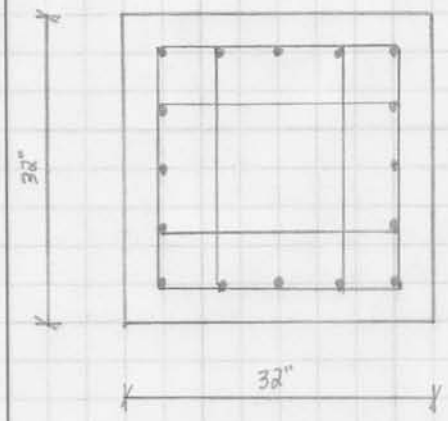
$= 5(0.802)(384)^4/384(29000)(33585)$

$= 0.233in < 1.067in$ ✓

Note: due to member section properties, Flexural bending will be the controlling failure mode ∴
shear is not investigated due to efficiency of report

Column Spot Check - interior

Typical Column (interior) at base of building : Type V



Longitudinal: (16) #10 = 20.32 in² } reinforcement 5 ksi = concrete

Transvers: #4 @ 8"

Height of Column : 17 ft ← longest unbraced length
(story)

Per ACI 318 S.10.3.6.2:

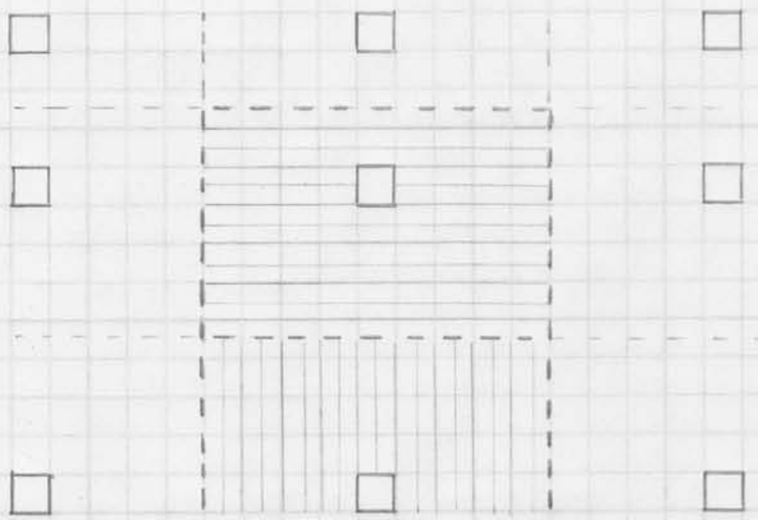
$\phi P_{n,max} = 0.8\,\phi\,[0.85 f'_c (A_g - A_{st}) + A_{st} f_y]$ (eq. 10-2)

$= 0.8(0.65)\,[0.85(5)(32\times32 - 20.32) + 20.32(60)]$

$= 2852\,k$

Column Axial Calculations - interior

Typical Column (interior) at base of building : Type V



Interior Column Trib. Area

32'-0" x 20'-5"

Exterior Column Trib. Area

32'x0" x 15'-2"

Story	Dead (psf)	Live (psf)	Area (sf)
roof	90	50	653
7	85	80	653
6	85	80	653
5	85	80	653
4	85	80	653
3	85	80	653
2	85	80	653

LL Reduction: $80(0.5) = 40$ ← use 40 psf (can only reduce 50%)

max $80(0.25 + 15/\sqrt{653(6)}) = 39.17$

min roof live load per structural notes in CD's

$P_u = [1.2(85) + 1.6(40)](6)(653) + [1.2(90) + 1.6(50)](653) =$

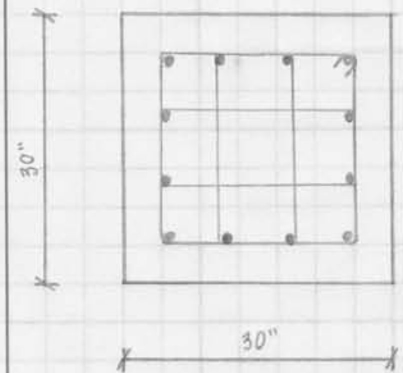
$= 650388 + 122764$

$= 773152/1000 = 773\text{ k}$

$\phi P_n = 2852\text{k} > P_u = 773\text{k}$ ✓

Column Spot Check - exterior

Typical Column (exterior) at base of building: Type III



Longitudinal: (12) #9 = 9 in^2 } reinforcement

Transverse: #4 @ 8"

5 ksi = concrete

Height of Column (story) : 17 ft ← longest unbraced length

Per ACI 318 S 10.3.6.2 :

$\phi P_{n,max} = 0.8\phi [0.85 f'_c (A_g - A_{st}) + A_{st} f_y]$ (eq. 10-2)

$= 0.8(0.65)[0.85(5)(30 \times 30 - 9) + 9(60)]$

$= 2250$ k

Column Axial Calculations - exterior

Typical Column (exterior) at base of building: Type III

(see previous page - interior column - for diagram and information on tributary area)

Story	Dead (psf)	Live (psf)	Area (sf)	Facade (psf)	Facade Area (sf)
roof	90	50	485	43.4	640
7	85	80	485	43.4	480
6	85	80	485	43.4	480
5	85	80	485	43.4	512
4	85	80	485	43.4	480
3	85	80	485	43.4	480
2	85	80	485	43.4	544

Exterior Glazed Framing System (GFS) = 20 psf

Average % of wall area covered by fenestration = 45% $\therefore$ $0.45(20) + 0.55(67.5) = 43.38$ psf ≈ 43.4 psf

LL Reduction: $80(0.5) = 40$

max $80\left(0.25 + 15/\sqrt{485(6)}\right) = 42.75$ psf

$P_u = [1.2(85) + 1.6(42.25)](6)(485) + [1.2(90) + 1.6(50)](485) + (544 + 512 + 640 + (480 \times 4))(43.4)(7)(1.2)$

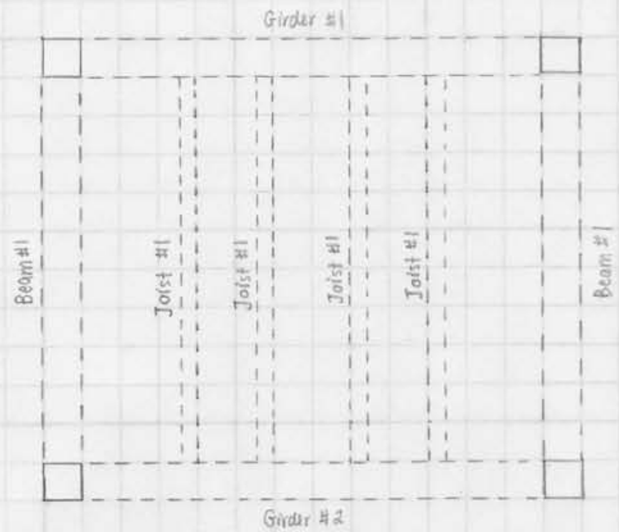
$= 403536 + 91180 + 1318249$

$= 1902965 / 1000 = 1903k$

$= 1903k$

$\phi P_n = 2250k > P_u = 1903k$ ✓

Summary: Concrete Slab w/ Pan Joists



Framing Assembly: 5 1/4" NWC slab w/ 16" deep wide module NWC pan joists @66" o.c. (typ) unless otherwise noted

Joist #1: 8" wide x 21.25" deep

(2) #8 bottom reinforcement
(2) #6 top reinforcement

Beam #1: 35" wide x 21.25" deep

(4) #9 bottom reinforcement
(4) #9 top reinforcement
(19) #4 @ 4" o.c. stirrups

Girder #1: 32" wide x 30" deep

(6) #8 bottom reinforcement
(6) #9 top reinforcement
(15) #4 @ 6" o.c. stirrups

Girder #2: 42" wide x 21.25" deep

(7) #7 bottom reinforcement
(6) #9 top reinforcement
(19) #4 @ 4" o.c. stirrups

[7] Alternate Framing Systems for Gravity Loads

This section investigates three alternate framing systems for gravity loads within The Medical Center. This study is performed in order to determine the best approach to further consider an alternative system redesign. All assumptions and evaluation criteria may be seen directly below or within the body of the calculation package.

7.1 Assumptions

The following calculations are representative of a typical bay design for three alternate gravity framing systems. Using ultimate strength design, the following alternate systems were designed under pure gravity loading (vertical dead and live loads). The typical bay utilized for design of alternate systems reflects a typical bay within the existing design of The Medical Center. Column design was not considered during the design of alternate gravity framing systems.

7.2 System Comparison

An evaluation of each gravity framing system considered within the scope of this report was performed in order to determine the most appropriate system. The systems involved in comparison are the following: concrete slab with pan joist framing system, non-composite steel framing system, composite steel framing system, and two-way flat plate concrete slab system. The criteria considered for use in the system comparison are as follows: weight, depth, cost, fire protection, and fire rating.

Table 5 - System Comparison

Criteria	Concrete Slab w/ Pan Joists	Non-Composite Steel Framing	Composite Steel Framing	2-Way Flat Plate Concrete Slab
Weight (psf)	75	63	64	150
Depth	30”	24”	21”	12”
Cost	$11.32/SF	$9.69/SF	$12.34/SF	$17.12/SF
Fire Protection	None	None	None	None
Fire Rating	4 Hr	2 Hr	1 Hr	4 Hr

Note: Cost comparison data extracted from RS Means Building Construction Cost Data (2015)

7.3 Decision Matrix

The decision matrix is organized to compute a weighted-value corresponding to the efficiency and appropriateness of each gravity framing system design. The decision matrix assigns values, on a scale of 1 to 4, depending on the performance of the system for the given criteria. A value of 1 represents poor performance, and a value of 4 represents exceptional performance. Each value can only be used once per each criterion (exception: durability).

Table 6 - Decision Matrix

Criteria	Importance Factor	Concrete Slab w/ Pan Joists	Non-Composite Steel Framing	Composite Steel Framing	2-Way Flat Plate Concrete Slab
Weight - Site	1.25	2	4	3	1
Weight - Uplift	1.25	3	1	2	4
Constructability	1.00	1	4	2	3
Cost	1.50	3	4	2	1
Fire Resistivity	1.00	3	2	1	4
Durability	1.00	2	2	2	2
		16.75	20.25	14.25	16.75

7.4 Calculations

The following design calculations are representative of the design of a typical bay for three alternate gravity framing systems. Each system-specific calculation follows the same sequence of calculation:

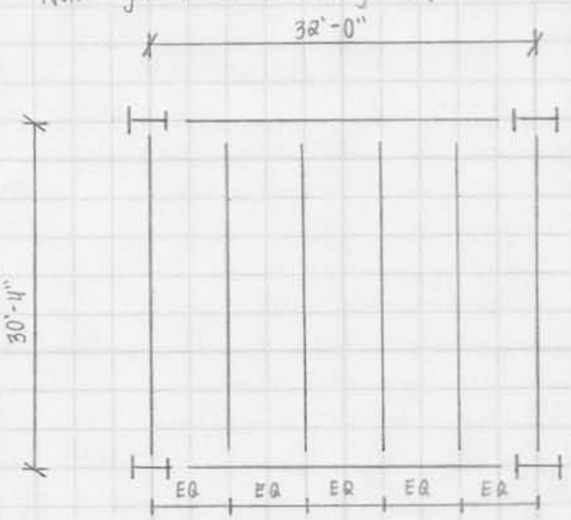
- Layout/Sketch of Typical Bay
- Determination of Loading Conditions
- Slab/Deck Design or Check
- Joist/Beam Design or Check
- Girder Design or Check
- Design Summary

The calculation packages for each alternate system can be seen below. The order of presentation of these calculations follows the order of presentation of these systems throughout the body of the report thus far.

ALTERNATE SYSTEM STUDY - non-composite steel framing system

Purpose: analyze steel framing option to improve economy (consideration of poor soil conditions - heavy vs. light structure)

Note: girder and beam framing configuration will remain the same, except beam spacing will be equal across the bay (typ.)



TYPICAL BAY FRAMING - non-composite

Selection of Deck Assembly: acheive at least a 2hr - fire rating (mission critical space - ICU) - life safety

Original Slab Assembly: 5 1/4" concrete slab

* Use Vulcraft 2 C Conform → 2C22 non-composite deck w/ 6×6 - W2.9 × W2.9 WWF
w/ 3.5 in NWC topping (total slab depth: 5.5in > 5.25in)

Dead Load

Deck Assembly : .... : 56 psf

Beams : .... : 5 psf

Girders : .... : 2 psf

Finishes : .... : 2 psf

Misc/Equip/Mech/Collateral : .... : 5 psf

Total: 70 psf (or 68 psf w/o girders → beam condition)

Live Load

Corridors (above 1st floor) ... : 80 psf ← Controlling LL case

Patient Rooms : .... : 40 psf

LL Reductions (see previous pages for calculations) → conservative due to original spacing configuration

Beam Condition : 48 psf
Girder Condition : 54 psf

Beam Design

Find M_u : $1.2(D) + 1.6(L) = 1.2(68) + 1.6(48) = 158.4 psf \approx 160 psf \rightarrow 160(6.4) = 1024 = 1.024 klf$

$1.4(D) = 1.4(68) = 95.2$ psf

$M_u = w\ell^2/8 = (1.024)(30.333)^2/8 = 117.78 \approx 118 k\cdot ft$

$M_u = \phi M_n \rightarrow M_n = M_u/\phi = 118/0.9 = 131 k\cdot ft$

Try W12x26 $\rightarrow \phi M_{px} = 140 k\cdot ft$ (Table 3-2)

Check LL Deflections $\rightarrow$ distributed Load

$\downarrow$

$w_{LL} = 48(6.4) = 307 plf$

$\Delta_{LL} = 5w\ell^4/384EI = 5(0.307)(30.333)^4(1728)/384(29000)(204) = 0.988 in$

$\Delta_{allow} = \ell/360 = 30.333 \times 12/360 = 1.01 in > 0.988 in$ ✓

Check TL Deflections $\rightarrow$ distributed Load

$\downarrow$

$w_{TL} = (68+49)(6.4) = 743 plf$

$\Delta_{TL} = 5w\ell^4/384EI = 5(0.743)(30.333)^4(1728)/384(29000)(204) = 2.39 in$

$\Delta_{allow} = \ell/240 = (30.333 \times 12)/240 = 1.52 in < 2.39 in$ ✗

Try W16x31

Check LL Deflections $\rightarrow$ distributed Load

$\downarrow$

$w_{LL} = 307 plf$

$\Delta_{LL} = 5w\ell^4/384EI = 5(0.307)(30.333)^4(1728)/384(29000)(375) = 0.53 in$

$\Delta_{allow} = \ell/360 = 30.333 \times 12/360 = 1.01 in > 0.53 in$ ✓

Check TL Deflections $\rightarrow$ distributed Load

$\downarrow$

$w_{TL} = 743 plf$

$\Delta_{TL} = 5w\ell^4/384EI = 5(0.743)(30.333)^4(1728)/384(29000)(375) = 1.31 in$

$\Delta_{allow} = \ell/240 = 30.333 \times 12/240 = 1.52 in > 1.31 in$ ✓

Weight Check $\rightarrow$ $31/6.4 = 4.84 psf < 5 psf$ ✓

Summary : Use W16x31 sections for beams

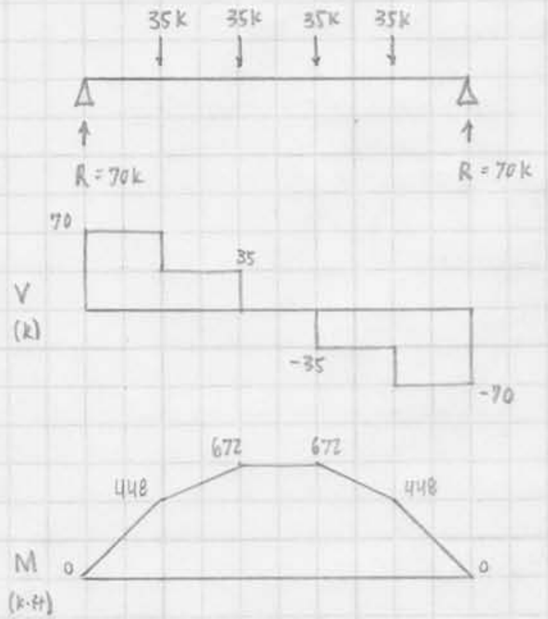
Girder Design

Point Loads:

$P_{LL} = 54(6.4)(32) = 11.1k$

$P_{DL} = 70(6.4)(32) = 14.3k$

$P_u = 1.2(14.3) + 1.6(11.1) = 34.92k = 35k$



where $R = 4(35)/2 = 70k$

Try W24×76 → $\phi M_{px} = 750 k{\cdot}ft > M_u = 672 k{\cdot}ft$

Check Wet Concrete Deflections → $w_{wc} = 56(6.4) + 31 = 390 plf = 0.390 klf$

$\Delta_{wc} = 5wl^4/384EI = 5(0.390)(30.333)^4(1728)/384(29000)(375) = 0.68in$

$\Delta_{allow} = l/360 = 32 \times 12/360 = 1.067in > 0.68in$ ✓

Check LL Deflections → $w_{LL} = 11.1/32 = 0.35 klf$

$\Delta_{LL} = 5wl^4/384EI = 5(0.35)(32)^4(1728)/384(29000)(2100) = 0.14in$

$\Delta_{allow} = l/360 = 32 \times 12/360 = 1.067in > 0.14in$ ✓

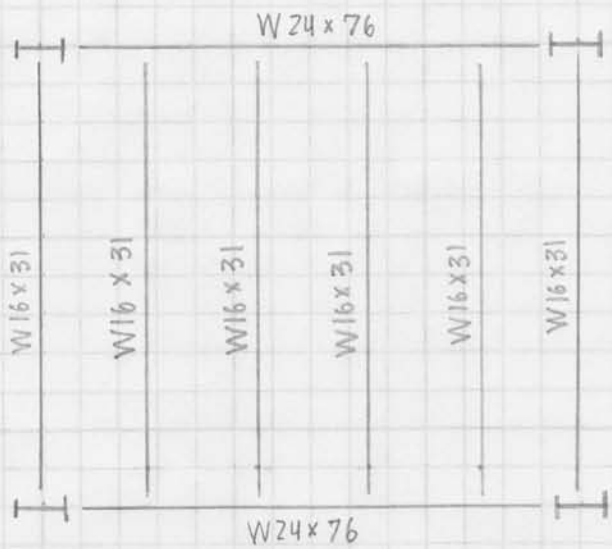
Check TL Deflections → $w_{TL} = (11.1 + 14.3)/32 = 0.73 klf$

$\Delta_{TL} = 5wl^4/384EI = 5(0.73)(32)^4(1728)/384(29000)(2100) = 0.28in$

$\Delta_{allow} = l/240 = 32 \times 12/240 = 1.6in > 0.28in$ ✓

Summary → use W24×76

Summary: Non-Composite System Design



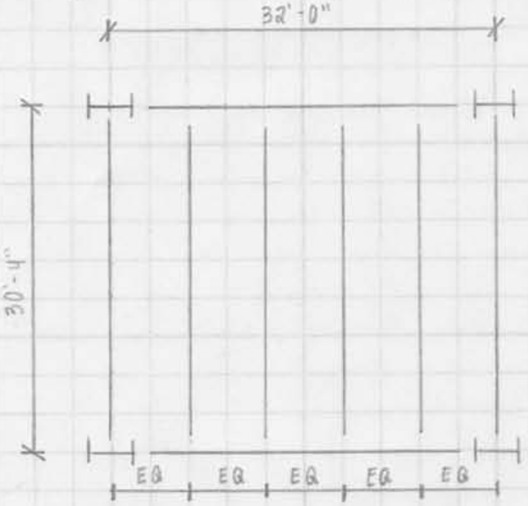
Deck Assembly:

2C22 Non-Composite Deck w/ 3 1/2" NWC topping (total slab depth 5 1/2 in > 5 1/4 in)
w/ 6x6 - W2.9 x W2.9 WWF

ALTERNATE SYSTEM STUDY - composite steel framing system

Purpose: analyze steel framing option to improve economy - investigate intermediate economic option due to consideration of environmental factors (loads) and potential uplift

Note: girder and beam framing configuration will remain the same, except beam spacing will be equal across the bay (typ)



TYPICAL BAY FRAMING - composite

Selection of Deck Assembly: acheive at least a 2hr - fire rating (mission-critical spaces - ICU) - life safety

Original slab Assembly: 5 1/4" concrete slab

* Use 2VLI20 w/ 3.5" NWC topping w/ 6x6 - W2.1x W2.1 WWF (recommended by SDI Vulcraft)

Dead Load

Deck Assembly : .... : 57 psf

Beams : .... : 5 psf

Girders : .... : 2 psf

Finishes : .... : 2 psf

Misc/Equip/Mech/Collateral : .... : 5 psf

Total : 71 psf (or 69 psf w/o girders → beam condition)

Live Load

Coorldors (above 1st floor) ...... : 80 psf ← Controlling LL case

Patient Rooms .... : 40 psf

Deck Consideration

From Vulcraft 2008 Steel Deck Catalogue

2VLI20 w/ 3.5" NWC topping

Span	SDI Max Unshored Span
1	7'-5"
2	9'-5"
3	9'-9"

span: 6'-4 4/5" < 7'-5" ∴ any span condition may be used with this deck

Note: 2VLI22 w/ 3.5" NWC topping could be used in the 2 and 3 span conditions, but design elects to use 2VLI20 w/ 3.5" NWC topping to allow for all span conditions to be available for consideration

2VLI22 w/ 3.5" NWC topping → SDI Max Unshored Span - 1span condition = 6'-4" < 6'-4 4/5" X

Beam Design

$1.2D + 1.6L = 1.2(71) + 1.6(48) = 162 psf$

$w = 162 psf (6.4 ft) = 1036.8/1000 = 1.04 klf$

$M_u = wl^2/8 = (1.04)(30.333)^2/8 = 120\ k \cdot ft$ ← controlling case

$1.4D = 1.4(71) = 99.4 psf \approx 100 psf$

$w = 100 psf (6.4) = 640/1000 = 0.640 klf$

$M_u = wl^2/8 = (0.640)(30.333)^2/8 = 73.6\ k \cdot ft$

Assuming Composite Action:

assume $a = 1"$

$y_2 = 5 \frac{1}{2} - \frac{1}{2} = 5"$

Beam Design (cont.)

Possible Selections

					Composite ↓
X	W10×19	$\Sigma Q_n = 70.3k$	$70.3/17.2 = 4.08$	$2(5\times10) + 19(30.333) = 676$ lbs	$\phi M_n = 127$ k·ft
X	W12×19	$\Sigma Q_n = 69.6k$	$69.6/17.2 = 4.04$	$2(5\times10) + 19(30.333) = 676$ lbs	$\phi M_n = 143$ k·ft
✓	W16×26	$\Sigma Q_n = 96.0k$	$96/17.2 = 5.58$	$2(6\times10) + 26(30.333) = 909$ lbs	$\phi M_n = 244$ k·ft

↳ extra weight on structure will help to counter uplift forces from environmental conditions, therefore I did not elect to go shallower or lighter and camber (do not camber when possible)

Unshored Strength:

$1.4D = 1.4(57 \times 6.4) + 1.4(26) = 547$ plf

$1.2D + 1.6L = 1.2(57\times 6.4 + 26) + 1.6(20)(6.4) = 674$ plf

↑ construction LL

$w\ell^2/8 = (0.674)(30.333)^2/8 = 77.5$ k·ft

166 k·ft > 77.5 k·ft ✓ (capacity check)

Wet Concrete Deflection

$w_{wc} = (57)(6.4) + 26 = 391/1000 = 0.391$ klf

$\Delta_{wc} = 5w\ell^4/384EI_x = 5(0.391)(30.333)^4(1728)/384(29000)(301) = 0.85$ in

$\Delta_{allow} = \ell/360 = 30.333\times12/360 = 1.01$ in > 0.85 in ✓

Live Load Deflection

$w_{LL} = (48)(6.4) = 307.2$ plf $= 0.307$ klf

$I_{LB} = 555$ in^4 (Table 3-20)

$\Delta_{LL} = 5w\ell^4/384EI_{LB} = 5(0.307)(30.333)^4(1728)/384(29000)(555) = 0.36$ in

$\Delta_{allow} = \ell/360 = 30.333\times12/360 = 1.01$ in > 0.36 in ✓

Stud Calculations

Max Spacing: $32(3/4") = 24"$, $8(5.5") = 44"$

# of studs required: $6(2) = 12$ studs → $30.333\times12/12 = 30.333"$ → max spacing = 24" ∴ use 24"

Beam Design (cont.)

Check "a" Assumption

$$b_{eff} = \min \left| \begin{array}{l} (6.4/2)(12) = 38.4\text{in} \\ 30.333(12)/8 = 45.5\text{in} \end{array} \right\} \times 2 \left\{ \begin{array}{l} = 76.8\text{in} \leftarrow \text{use} \\ = 91.0\text{in} \end{array} \right.$$

$V_c = (3.5")(76.8")(4\text{ksi})(0.85) = 914\text{k}$

$V_s = (7.68\text{in}^2)(50\text{ksi}) = 384\text{k}$

12 studs $(17.2\text{k/stud}) = 206.4\text{k}$

$a = 206.4 / 0.85(3.5)(76.8) = 0.903 < 1.0$ ✓

$\Sigma Q_n < V_s \rightarrow 206.4\text{k} < 384\text{k}$ ∴ this beam is partially composite

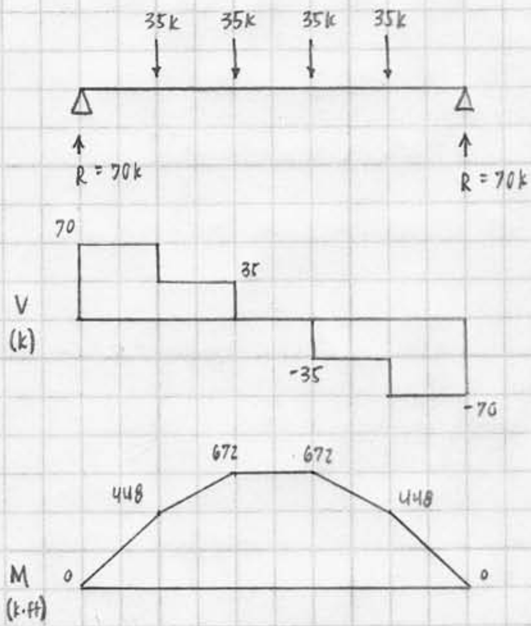
Girder Design

Point Loads:

$P_{LL} = 54(6.4)(32) = 11.1\,k$

$P_{DL} = 71(6.4)(32) = 14.5\,k$

$P_u = 1.2(14.5) + 1.6(11.1) = 35.2\,k$ → enough safety factors are built into process to consider this load as 35 k



where $R = 4(35)/2 = 70k$

Assuming Composite Action:

assume $a = 1''$

$y_2 = 5\frac{1}{2} - \frac{1}{2} = 5''$

Possible Selections

✓ W 21 x 57 $\Sigma Q_n = 209k$ $209/17.2 = 12.15$ $2(13 \times 10) + 57(32) = 2084$ lbs composite ↓ $\phi M_n = 684\,k \cdot ft$

Girder Design (cont.)

Unshored Strength → use superposition

Distributed Load

$1.4(57) = 80\ plf$

$1.2(57) + 1.6(0) = 69\ plf$ ← controlling case

Point Load

$1.4\,[(57(6.4) + 26)(30.333)] = 16.6k$

$1.2[(57(6.4) + 26)(30.333)] + 1.6[20(6.4)(30.333)] = 20.4k$ ← controlling case

Moment Superposition

$(0.069)(32)^2/8 = 8.83\ k\cdot ft$ ← from distributed

$(40.8 \times 6.4) + (20.4 \times 6.4) = 391.7\ k\cdot ft$

$M_u = 8.83 + 391.7 = 400.5\ k\cdot ft$

$484\ k\cdot ft > 400.5\ k\cdot ft$ ✓

Check Wet Concrete Deflection

$W_{wc} = 57\underbrace{(20.333)}_{30.333/2 + 10.333/2} + 26 = 1185\ plf = 1.185\ klf$

$\Delta_{wc} = 5w\ell^4/384EI_x = 5(1.184)(32)^4(1728)/384(29000)\underbrace{(1170)}_{I_x} = 0.82in$

$\Delta_{allow} = \ell/360 = 32 \times 12/360 = 1.067in > 0.82in$ ✓

Check Live Load Deflection

$w_{LL} = (54)(20.333) = 1098\ plf = 1.098\ klf$

$\Delta_{LL} = 5w\ell^4/384EI_{LB} = 5(1.098)(32)^4(1728)/384(29000)\underbrace{(1980)}_{I_{LB}} = 0.45in$

$\Delta_{allow} = \ell/360 = 32 \times 12/360 = 1.067\ in > 0.45in$ ✓

Check "a" assumption

$b_{eff} = \begin{cases} (20.333/2) \times 12 = 122'' \times 2 = 244'' \\ \text{min } 32(12)/8 = 48 \times 2 = 96'' \leftarrow \text{controls} \end{cases}$

$V_c = 0.85(3.5)(96)(4) = 1142k$

$V_s = 16.7(50) = 835\ k$

$a = 209/0.85(96)(3.5)$

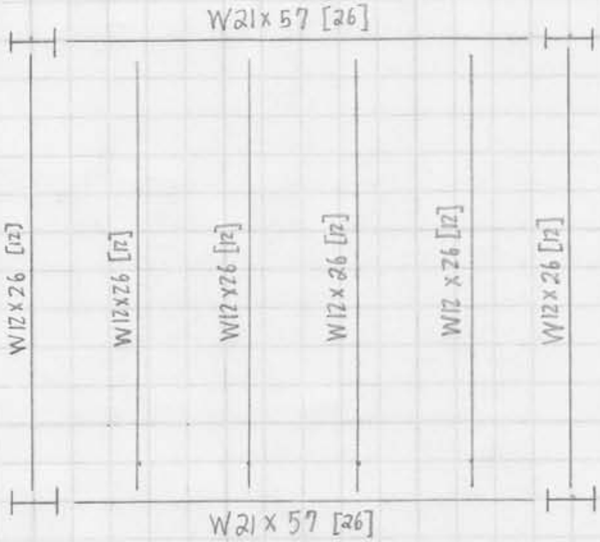
$a = 0.73in < 1in$ ✓

Stud Calculations

Actual Spacing: $(32ft)(12in/ft)/26\ studs$
$= 14.76'' < 24''$ ✓ ok

Stud Spacing: $\begin{cases} 32(3/4'') = 24'' \\ \text{min } 8(5.5'') = 44'' \end{cases}$

Summary: Composite System Design



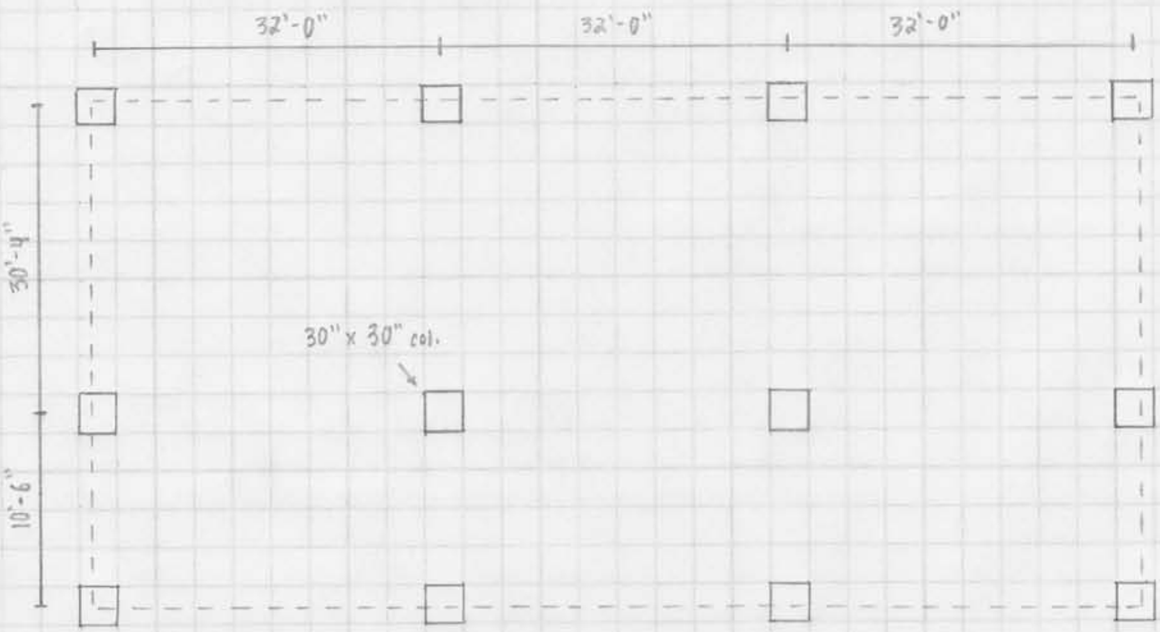
Deck Assembly:

2VLI20 Composite Deck w/ 3 1/2" NWC topping (total slab depth 5 1/2 in > 5 1/4 in)
w/ 6x6 - W2.1 x W2.1 WWF (recommended by SDI Vulcraft)

ALTERNATE SYSTEM STUDY - two-way flat plate concrete slab

Purpose: analyze an additional concrete system, reducing depth of section to increase floor to floor heights (add appeal as necessary)

Note: girder and beam framing (as well as joist members) will be absent from gravity framing assembly



Dead Loads: 150 pcf (12 in / 12 in / ft) = 150 psf

Live Loads: 80 psf (LL reduction need not be applied - deflection is not an issue)

Superimposed Dead Load: 2 psf + 5 psf + 2 psf = 9 psf

Finishes, MEP, Misc/Collateral

Factored Load: 1.2(159) + 1.6(80) = 319 psf

Min Col. = 30"

Factored Superimposed Load: 1.2(9) + 1.6(80) = 139 psf

fy = 60 ksi (reinforcement)

Minimum Panel Thickness (ACI Table 9.5(c))

f'c = 4 ksi (slab)

- w/o drop panels, interior panels = $\ell n/33$
- w/o drop panels, exterior w/ edge beams = $\ell n/33$

→ $\ell n/33 = 354/33 = 10.7$ in → 12 in slab

Span col-col: 30.333', 32'

$\ell n = (32 \times 12) - 30 = 354$

Shear Strength

One Way:

$V_u = 0.319\left(32/2 - 30/2(12) - 10.75/12\right)(32) = 141\ k$

$d = (12 - 3/4'' - 1/2'') = 10.75$

$V_c = 2\lambda\sqrt{f'c}\ b_w d = 2\sqrt{4000}\ (32 \times 12)(10.75) = 522155/1000 = 522\ K$

$b_w = 32/2 + 32/2 = 32$

$\phi V_c = 0.75(522) = 392\ k > V_u = 141\ k$ ✓

Two Way:

$V_u = 0.319\left((32 \times 30.333) - (30 + 10.75/12)\right) = 308\ k$

$V_c = 4\lambda\sqrt{f'c}\ b_w d = 4\sqrt{4000}\ (4(30+8))(10.75) = 413\ k$

$\phi V_c = 0.75(413) = 310\ k > V_u = 308\ k$ ✓

Total Factored Moment

$M_o = q_u\, l_2\, l_n^2/8 = 0.319(30.333)(32 - 30/12)^2/8 = 1053\ k \cdot ft$ (long span - 32 ft)
$M_o = q_u\, l_2\, l_n^2/8 = 0.319(32 \cdot)(30.333 - 30/12)^2/8 = 988\ k \cdot ft$ (short span - 30.333

Distribute Moments: Direct Design Method for Two-Way Slab - Exterior Edge Fully Restrained

Long Span:

Column Strip: $+M = 0.6(0.35 M_o) = 221\ k \cdot ft$
$-M = 0.75(0.65 M_o) = -513\ k \cdot ft$

Middle Strip: $+M = 0.4(0.35 M_o) = 147\ k \cdot ft$
$-M = 0.25(0.65 M_o) = -171\ k \cdot ft$

Short Span:

Column Strip: $+M = 0.6(0.35 M_o) = 207\ k \cdot ft$
$-M = 0.75(0.65 M_o) = -482\ k \cdot ft$

Middle Strip: $+M = 0.4(0.35 M_o) = 138\ k \cdot ft$
$-M = 0.25(0.65 M_o) = -161\ k \cdot ft$

Area of Steel

$A_s = M_u / \phi f_y jd \rightarrow jd = 0.95(10.3) = 9.79$

Calculate d

top : $d \approx 12 - 1.7 = 10.3$ ← worst case
bottom : $d \approx 12 - 1.1 = 10.9$

$A_{s,min} = 0.0018\,bh = 0.0018\,(16 \times 12)(12) = 4.14\,in^2$

$S_{max} = \min \begin{cases} 2h = 2(12) = 24" \\ 18" \rightarrow \text{use } 18" \text{ as } S_{max} \end{cases}$

Long Span:

sample calculations for reference

Column Strip : + = $221(12)/0.9(60)(9.79) = 5.02\,in^2 \rightarrow (12)\#6 = 5.28\,in^2$ ✓
− = $513(12)/0.9(60)(9.79) = 11.64\,in^2 \rightarrow (15)\#8 = 11.85\,in^2$ ✓

Middle Strip : + : $3.34\,in^2 \rightarrow (12)\#5 = 3.72\,in^2$ ✓
− : $3.88\,in^2 \rightarrow (5)\#8 = 3.95\,in^2$ ✓

Short Span :

Column Strip : + = $4.70\,in^2 \rightarrow (6)\#8 = 4.74\,in^2$ ✓
− : $10.94\,in^2 \rightarrow (14)\#8 = 11.06\,in^2$ ✓

Middle Strip : + = $3.13\,in^2 \rightarrow (4)\#8 = 3.16\,in^2$ ✓
− = $3.65\,in^2 \rightarrow (12)\#5 = 3.72\,in^2$ ✓

Strain Check

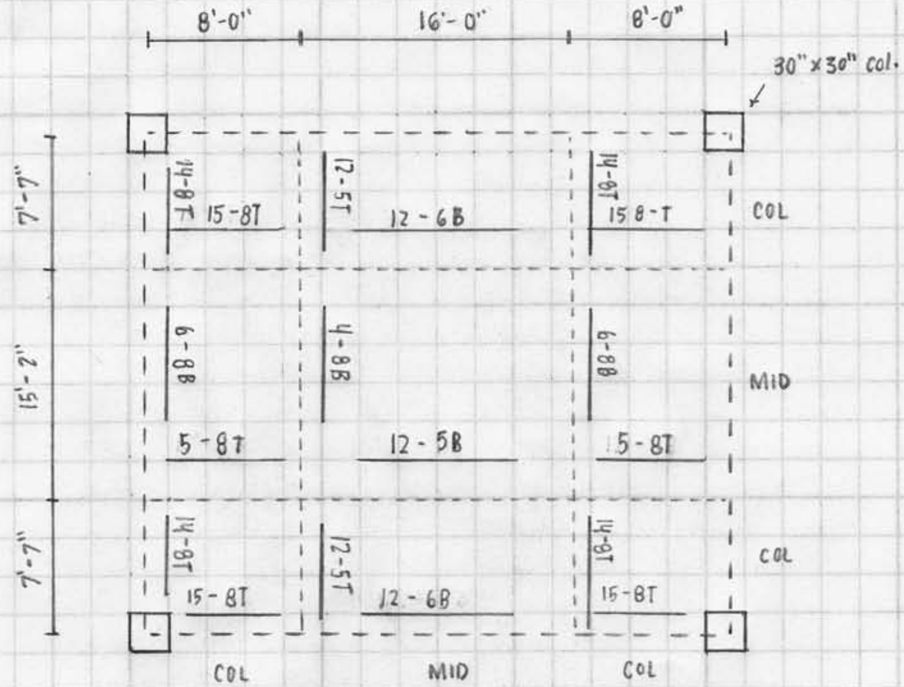
$\varepsilon_s = (d - c/c)\,\varepsilon_{cu} = ((10.3 - 1.28)/1.28)\,0.003 = 0.021 > 0.00207$ and $0.021 > 0.005$ ✓

where $a = 11.85(60)/0.85(4)(16 \times 12) = 1.089" \rightarrow c = a/\beta = 1.089/0.85 = 1.28"$

$\phi = 0.9$ (assumed)

− col	0.021	0.9	✓
+ col	0.051	0.9	✓
− mid	0.069	0.9	✓
+ mid	0.074	0.9	✓
	↑ ε_s	↑ ϕ	

Summary: two-way flat plate concrete slab system



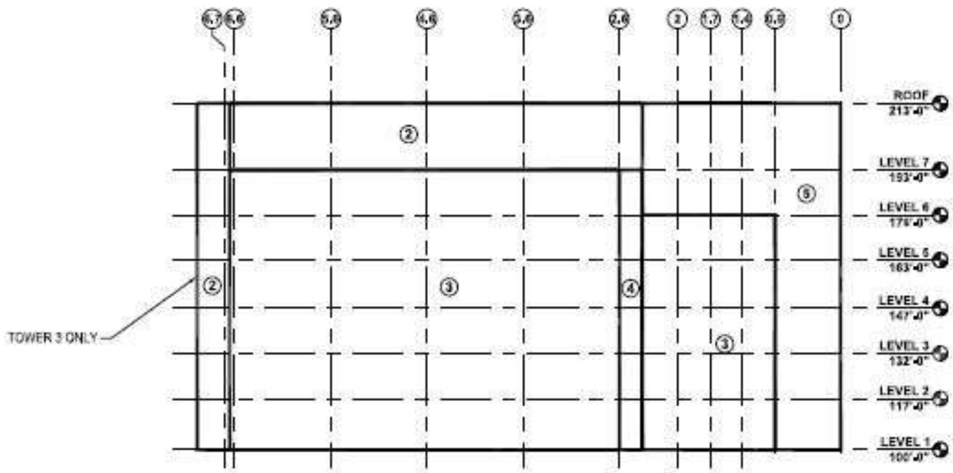
[8] Conclusion

After extensive consideration of the multiple gravity framing systems, an educated and purpose-driven comparison concludes that a non-composite steel framing system is the most appropriate and efficient system in performance for use in The Medical Center. The system yielded the best results in regards to weight of the system with respect to soil and site conditions, overall ease of constructability, durability, and cost.

The composite steel framing system yielded the worst results of all of the considered gravity framing systems. Out of the six criteria, the composite steel framing system ranked in the bottom half of all systems considered in five out of the six criteria. Although the composite nature of the framing would add dead weight to the structure, making it less susceptible to uplift forces, other system options exist as more viable approaches to gravity framing system design.

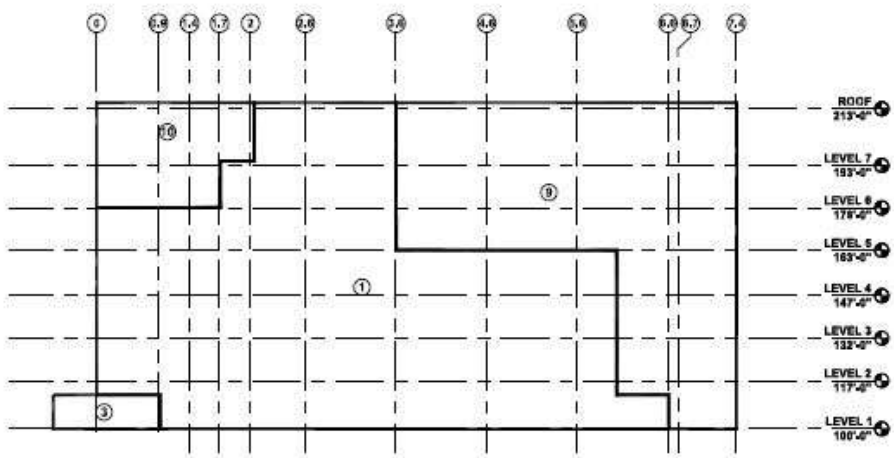
In order to gain full appreciation of this matrixed comparison, consideration of lateral loads must be realized. Due to environmental factors causing extreme wind loads as well as poor site soil causing foundation and settlement issues, axial compressive and tensile loads in columns due to lateral wind and seismic loads must be considered in order to properly select the most appropriate gravity framing system. Technical Report IV will allow for the opportunity to more closely investigate lateral loads and design of lateral force resisting systems. After analyzing the lateral conditions of the structure, a proper and more accurate proposal for redesign can be executed.

[9] Appendix A: Wind Loads



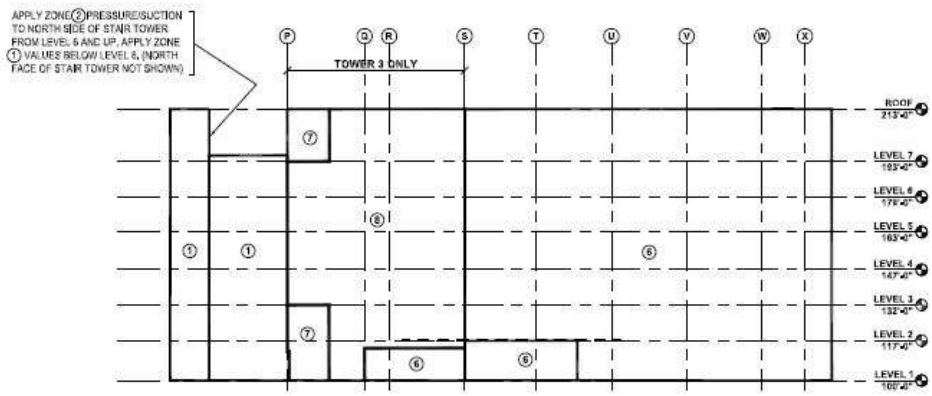
INPATIENT TOWER NORTH ELEVATION PRESSURE ZONES

NOTE:
COLUMN BUBBLES BASED ON
TOWER 1, PRESSURE ZONES ARE
IDENTICAL AT TOWERS 2 AND 3

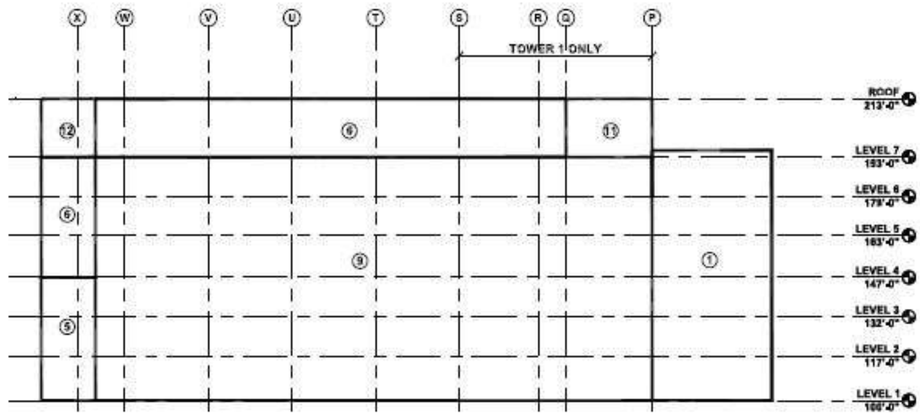


INPATIENT TOWER SOUTH ELEVATION PRESSURE ZONES

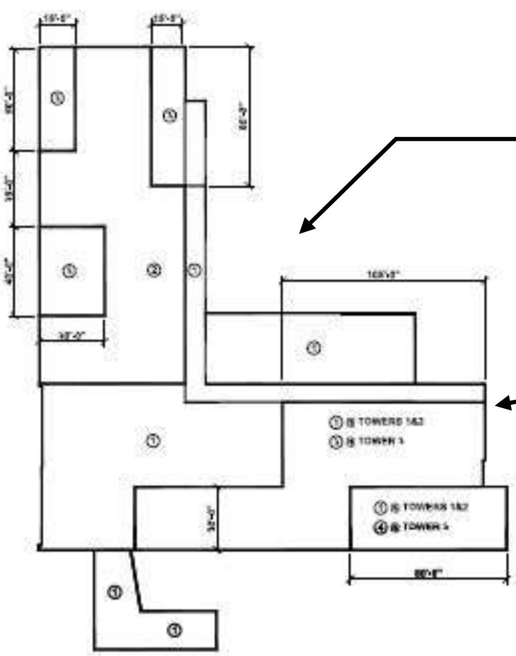
NOTE:
COLUMN BUBBLES BASED ON
TOWER 1, PRESSURE ZONES ARE
IDENTICAL AT TOWERS 2 AND 3



INPATIENT TOWER EAST ELEVATION PRESSURE ZONES



INPATIENT TOWER WEST ELEVATION PRESSURE ZONES



Inpatient Towers

Components and Cladding

Roof Pressure Zones

Wind Load Tables for Inpatient Tower

INPATIENT TOWER COMPONENTS AND CLADDING ROOF LOAD CRITERA (PSF)								
COMPONENT	ROOF ZONE 1		ROOF ZONE 2		ROOF ZONE 3		ROOF ZONE 4	
AREA (SQ FT)	PRESSURE	SUCTION	PRESSURE	SUCTION	PRESSURE	SUCTION	PRESSURE	SUCTION
10	38	-82	38	-92	38	-104	38	-138
20	38	-77	38	-86	38	-97	38	-129
50	38	-71	38	-79	38	-90	38	-119
100	38	-65	38	-73	38	-82	38	-109
200	38	-62	38	-69	38	-78	38	-104
500	38	-53	38	-60	38	-67	38	-89
700	38	-53	38	-60	38	-67	38	-89

Wind Load Tables for Inpatient Tower

INPATIENT TOWER COMPONENTS AND CLADDING WALL LOAD CRITERIA (PSF)												
COMPONENT	WALL ZONE 1		WAL. ZONE 2		WALL ZONE 3		WALL ZONE 4		WALL ZONE 5		WALL ZONE 6	
AREA (SQ FT)	PRESSURE	SUCTION	PRESSURE	SUCTION	PRESSURE	SUCTION	PRESSURE	SUCTION	PRESSURE	SUCTION	PRESSURE	SUCTION
10	64	-63	64	-82	64	-71	71	-63	71	-94	71	-82
20	64	-63	64	-82	64	-71	71	-63	71	-94	71	-82
50	59	-61	59	-79	59	-68	65	-61	65	-90	65	-79
100	56	-58	56	-75	56	-65	62	-58	62	-85	62	-75
200	53	-55	53	-71	53	-62	59	-55	59	-81	59	-71
500	47	-52	47	-67	47	-59	52	-52	52	-76	52	-67
700	47	-52	47	-67	47	-59	52	-52	52	-76	52	-67

COMPONENT	WALL ZONE 7		WALL ZONE 8		WALL ZONE 9		WALL ZONE 10		WALL ZONE 11		WALL ZONE 12	
AREA (SQ FT)	PRESSURE	SUCTION	PRESSURE	SUCTION	PRESSURE	SUCTION	PRESSURE	SUCTION	PRESSURE	SUCTION	PRESSURE	SUCTION
10	71	-151	71	-117	71	-71	82	-71	71	-105	82	-82
20	71	-151	71	-117	71	-71	82	-71	71	-105	82	-82
50	65	-144	65	-112	65	-68	75	-68	65	-100	75	-79
100	62	-136	62	-106	62	-65	71	-65	62	-95	71	-75
200	59	-128	59	-100	59	-62	67	-62	59	-90	67	-71
500	52	-121	52	-94	52	-59	59	-59	52	-85	59	-67
700	52	-121	52	-94	52	-59	59	-59	52	-85	59	-67

[10] Appendix B: Seismic Loads

USGS Design Maps Summary Report

View Detailed Report   Print

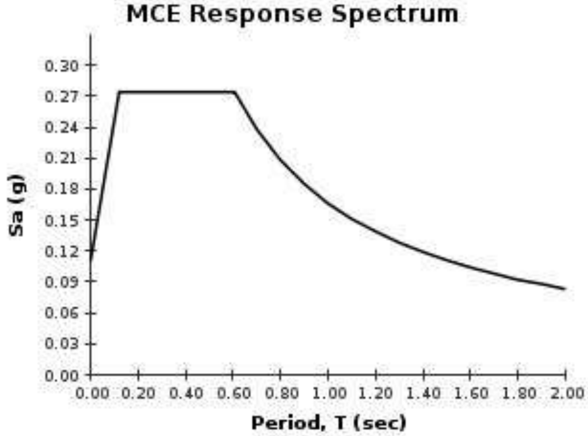
User–Specified Input

Building Code Reference Document	ASCE 7-05 Standard (which utilizes USGS hazard data available in 2002)
Site Coordinates	
Site Soil Classification	Site Class E – "Soft Clay Soil"
Occupancy Category	IV

USGS–Provided Output

S_S = 0.110 g	S_{MS} = 0.274 g	S_{DS} = 0.183 g
S_1 = 0.048 g	S_{M1} = 0.166 g	S_{D1} = 0.111 g

MCE Response Spectrum



Design Response Spectrum