THE PENNSYLVANIA STATE UNIVERSITY

SCHREYER HONORS COLLEGE

DEPARTMENT OF ARCHITECTURAL ENGINEERING

# THE HEALTH CENTRE: STRUCTURAL REDESIGN WITH REINFORCED CONCRETE FLAT SLAB AND SHEAR WALL SYSTEMS

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A thesis submitted in partial fulfillment of the requirements for a baccalaureate degree in Architectural Engineering with honors in Architectural Engineering

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## ABSTRACT

The Health Centre is a 750,000 square foot university hospital expansion located in the southeastern United States. Built adjacent to existing hospital facility 'Clinic B,' this ten-story L-shaped offers state-of-the-art medical technology, additional research space, and hundreds of new hospital beds. At a height of 166 feet, the Health Centre will be the tallest building in the surrounding area when its construction is complete in 2016.

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. Structural design considerations by the engineer of record included flexibility of interior spaces and the possibility of future vertical expansion.

This structural redesign of the Health Centre aims to reduce slab thickness for a more efficient use of materials and potential reduction in floor-to-floor heights. After an initial study of the existing cast-in-place skip joist gravity system, a flat slab was selected as an alternative design option. Precast reinforced concrete shear walls replaced the existing concrete moment frame system to resist wind and seismic building loads on the building. Three-dimensional modeling and hand calculation methods determined the modal response of the redesigned gravity system and its feasibility under the current criteria for vibration sensitive research equipment. Relocation of mechanical equipment Impact of the redesign and vibration criteria on construction cost and schedule.

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# Chapter 1

# Existing Building and Structural Systems

### 1.1 General Building Overview

The Health Centre is a university hospital expansion project consisting of a nine-story hospital bed tower with a mechanical penthouse and four-story underground parking garage. Located in the southeastern United States in the middle of a university campus, this L-shaped concrete building is adjacent to existing hospital facilities referred to as 'Clinic B.' Figure 1 shows the site plan of the Health Centre and the surrounding campus facilities. The hospital features a lower green roof and new state-of-the-art technical facilities – including an ICU, emergency department, clinical facilities, and med-surg patient rooms. Upon completion in 2016, the building will be certified LEED Silver.

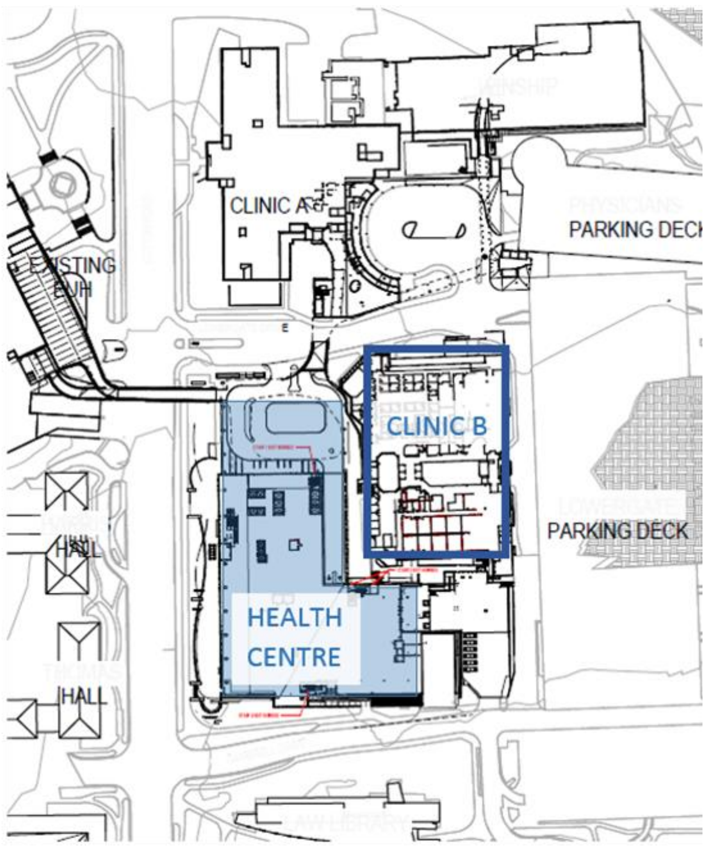



Figure 1: Site Plan (Courtesy of SmithGroupJJR)

The new building will add approximately 450,000 square feet and 210 new hospital beds to the existing hospital complex. It a "core-and-shell" building with adaptable spaces and the possibility for future expansion on the lower fourth floor roof. SmithGroupJJR was the party responsible for the architectural design and programming of the building's interior spaces. Walter P. Moore was responsible for the building's existing structural design.

**Figure 2: East Elevation (left) and South Facade (right) (SmithGroupJJR)**

The tallest building in the immediate area, the Health Centre stands out architecturally with a glass and metal façade system as depicted in Figure 2. In addition to the new technical facilities, the building site features a new entry drive for easier patient access and sustainable initiatives such as bioswales. As the building increases in height, its floor plan becomes restricted to the rectangular bed tower area. Elevators are located in the northern end of the tower, and extend the full height of the building. Three main floor plans exist throughout the building, depicted in Figure 3. As the building increases in height, it becomes narrower and loses its L-shape.

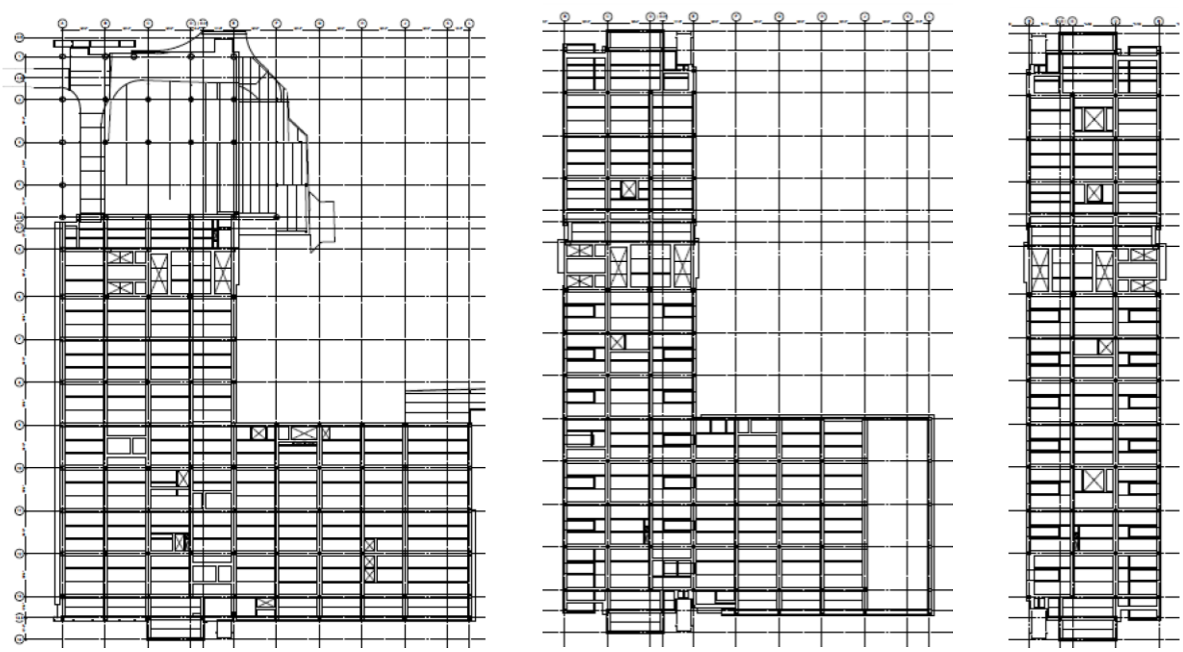


**Figure 3: Typical Building Floor Plans (SmithGroupJJR)**

## 1.2 Existing Structural Systems

### 1.2.1 Overview

Above ground, The Health Centre is mainly a cast-in-place concrete frame structure, as shown in Figure 4. The choice in structural material was driven by the contractor McCarthy Building Construction, who noted the availability of concrete over steel in the building's southeastern location. Cast-in-place floors are one-way slabs that connect the building's diaphragm. Bays are 30'x30' squares wherever possible.

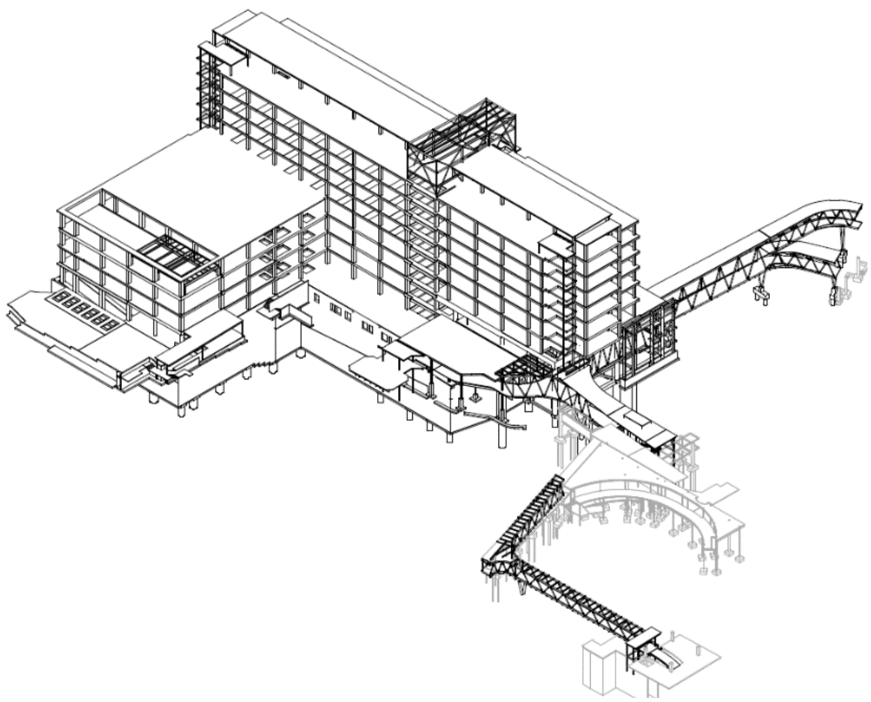


**Figure 4: Overall Building Structure (Walter P. Moore)**

The below-grade parking garage consists of two-way post-tensioned flat slabs. Foundations are typically cast-in-place spread footings, with some deep drilled piers on competent rock. CMU walls in the building are non-load bearing. A steel bridge connects the structure to other campus buildings. Due to the life-safety concerns for hospital structures, The Health Centre is Building Category IV under ASCE 7-05. The building lateral loads per ASCE 7-05 used basic wind speeds of 90 MPH and Seismic Design Category C

### 1.2.2 Foundations

The two primary foundation types used in this building are drilled piers and spread footings. All foundations were designed to be 4,000 psi normal weight concrete and are spaced the width of a typical 30'x30' bay in most locations.

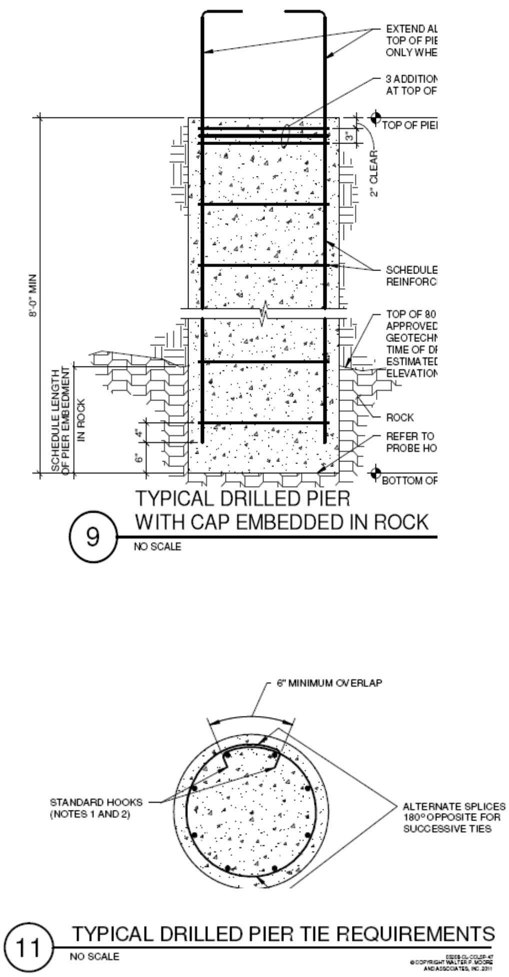



**Figure 5: Typical Drilled Pier Details (Walter P. Moore)**

Drilled piers are located mainly underneath the parking structure and the western side of the building. They were designed for 80 ksf net end bearing pressure on competent rock. There are 14 types of drilled piers with pier diameters ranging from 36" to 96". The piers use both #3 and #4 ties spaced at 12" and typically have #11 bars for vertical reinforcement. Typical details for drilled piers are provided in Figure 5. In some locations, drilled piers are embedded into the rock a depth of 5'-0".

Rectangular spread footings range in size from 4'x6'x18" with #6 rebar to 12'x12'x82" with #9 rebar. They were designed for 30 ksf net pressure on competent rock or 2 ksf net pressure on compacted soil depending on their location in the building. Some spread footings have an equivalent drilled pier that the contractor may choose to use instead. Typical details for spread footings are shown in Figure 6, while a partial foundation layout is provided in Figure 7.

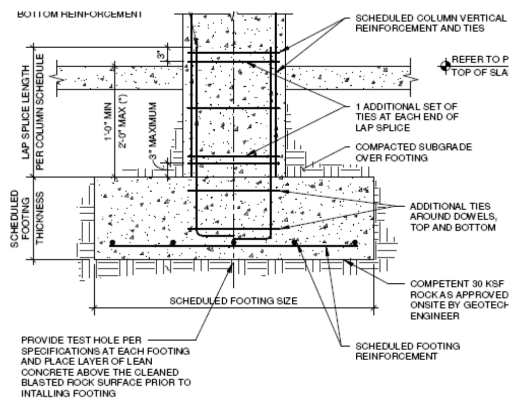



**Figure 6: Typical Foundation Detail (Walter P. Moore)**

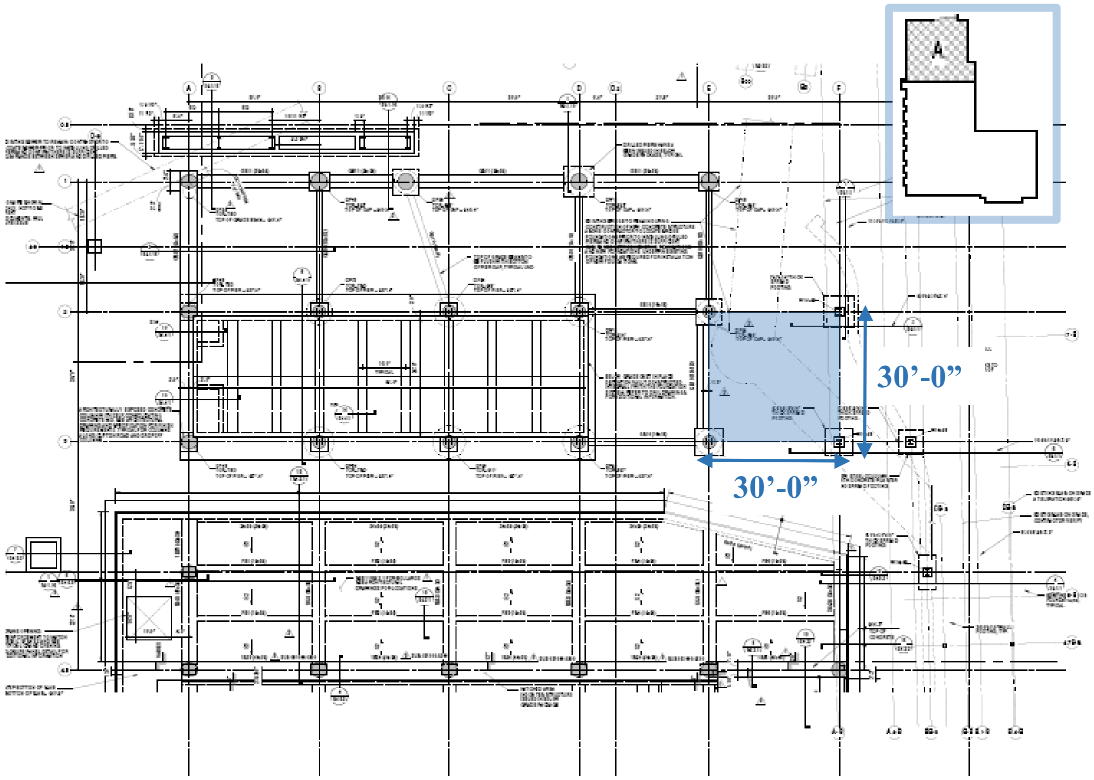
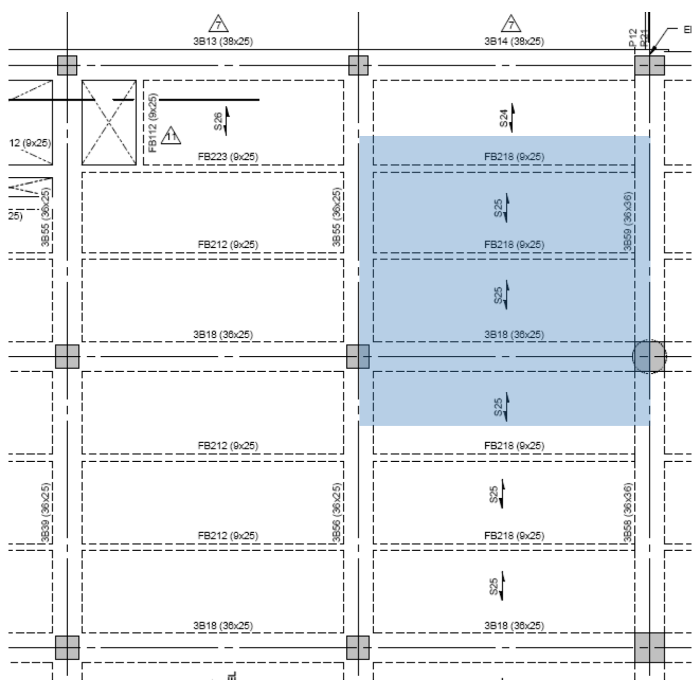



Figure 7: Partial Foundation Plan (Walter P. Moore)

### 1.2.4 Gravity System

A typical bay size of 30’x30’ is used throughout the building, as depicted in Figure 8. The floor system consists of a cast-in-place one-way skip-pan joists. Typical girders are 36”x25” and typical purlins are 9”x25” with #8 top and bottom bars. A typical section detail for the floor system is provided in Figure 9. In some locations – usually near floor openings or slab depressions – there is variation in beam width and depth.



The flat floor slab depth ranges from 5" to 14" thick, with a typical depth of 5" and 7". Top and bottom bars vary in size and

**Figure 8: Typical Bay from Third Floor Area D Floor Plan (WPM)**

spacing depending on the slab location, but are typically #3 or #4 spaced at 12". Some depressions in the slab occur for medical equipment and other hospital technology. All floor system components for the hospital bed tower and parking garage are 5,000 psi normal weight concrete.

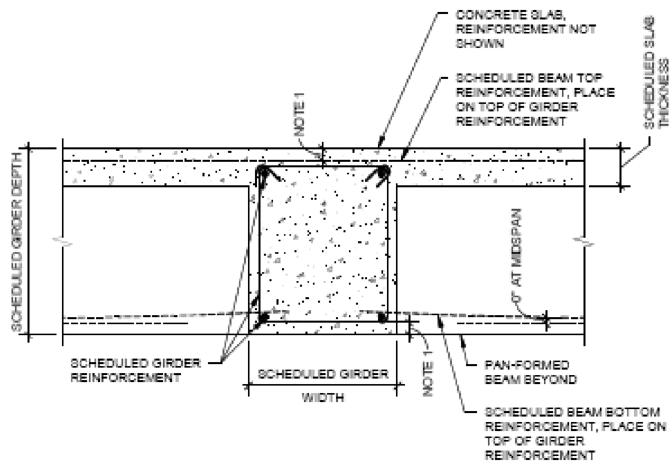



**Figure 9: Typical Floor Section (Walter P. Moore)**

For the bed tower, column size decreases as floor level increases. Parking garage columns are typically 28"x44" with 22 #9 rebar for vertical reinforcement. From the ground floor of the bed tower to the bottom of the fourth level, most columns are 28"x32" with vertical reinforcement consisting of 12 #8 rebar. Floors above the fourth floor, including the penthouse, typically have 24"x24" square columns with 8 #8 vertical reinforcement. A column splice is used when these size changes occur (Figure 10). Some columns have also been sized for future steel expansion. 7000 psi normal weight concrete is used for all cast-in-place columns.

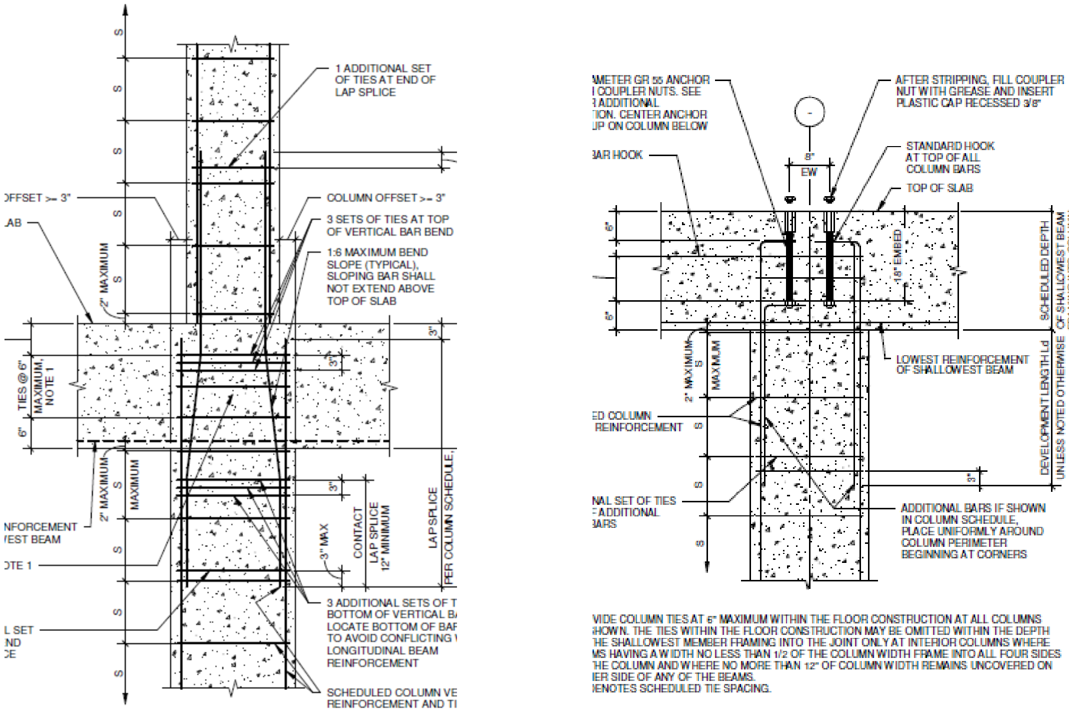



**Figure 10: Typical Column Splice and Steel Expansion Details {Walter P. Moore)**

Ellipse concrete columns are used in some locations, including underneath the bridge structure, and are detailed in Figure 11. Additionally, W14 structural steel encased in concrete is used in some locations between the ground and fourth levels when columns do not continue below grade to the underground parking garage.

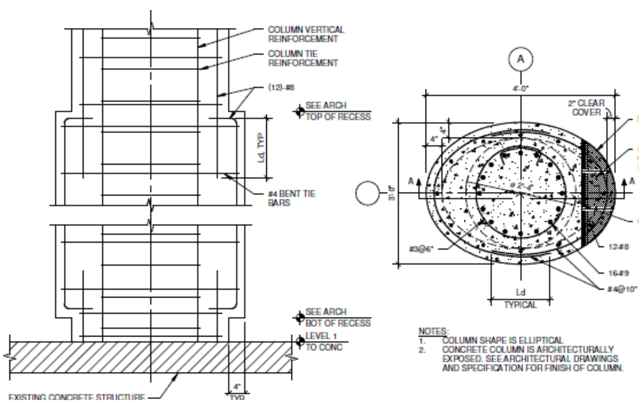



**Figure 11: Ellipse Column Detail (Walter P. Moore)**

### 1.2.5 Lateral System

Above grade, concrete moment frames resist lateral loads in both the north-south and east-west directions (Figure 12). All beams and columns above grade are detailed to resist gravity and lateral loads. Below grade, exterior shear walls resist soil and potential seismic forces. 7000 psi normal weight concrete is used for all lateral system vertical elements. Lateral loads are transferred into the moment frames and shear walls through the floor diaphragm. Large girder sizes on the exterior are typically 50”x48”. A concrete floor diaphragm consisting of a 5” one-way flat slab and 25” deep beams (in most locations) transfers the lateral load throughout the building (Figure 13). Due to the nature of this lateral system, concrete moment frames extend the full height of the building with some stopping at the fourth floor green roof.

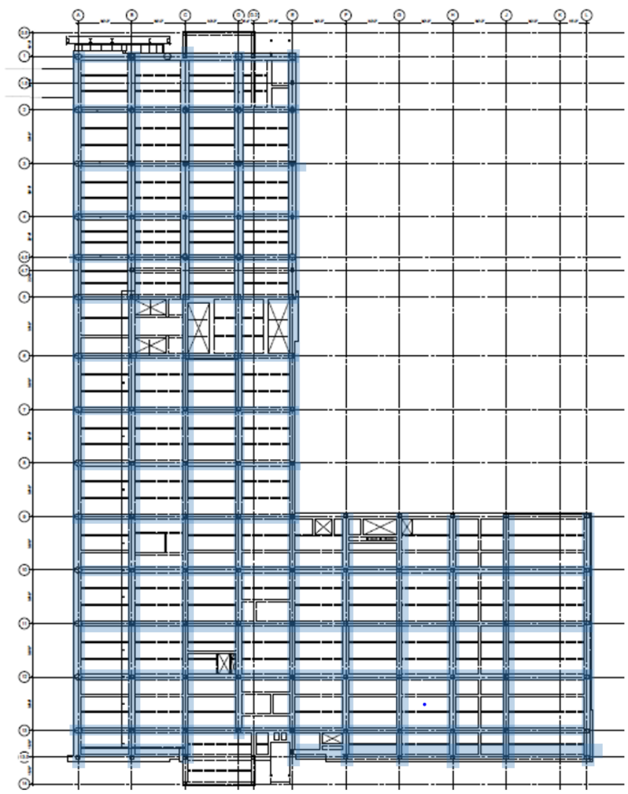


**Figure 12: Fourth Floor Concrete Moment Frames (Walter P. Moore)**

Basement shear walls in the parking garage begin at the bottommost level and end at grade. Typically 2’ thick and approximately 50’ high, they extend around the exterior of the parking structure. A 1.5”x3” continuous shear key and standard hooks connect the walls to the post-tensioned floor diaphragm. Design for service requirements take into consideration the lateral deflection of wind. Wind loads with a 10-year mean recurrence interval were considered for a lateral deflection of the typical floor height/400.

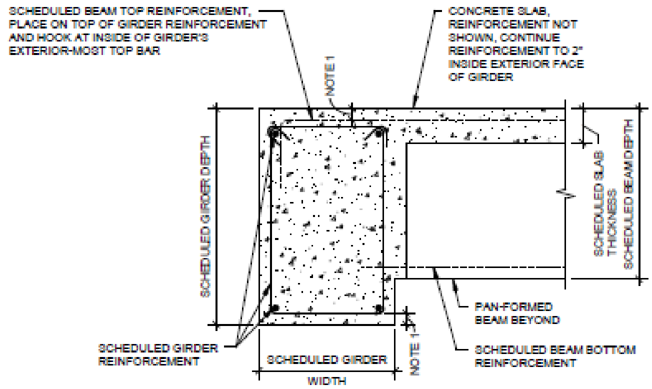


Figure 13: Typical Deep Beam Cross-Section (Walter P. Moore)

### 1.3 Load Path

The loads under consideration for this report are gravity, wind, and seismic loads. All three load cases follow a different path throughout the building.

For gravity loads, the load path begins at the penthouse roof. Roof live and dead loads are carried by columns to the floor below. On the penthouse level, mechanical equipment and cooling tower loads are carried by the concrete floor diaphragm and distributed to the columns. This process of load distribution continues throughout the remaining levels, noting the additional green roof loads on the fourth level. At grade, dead and live loads on the floor are distributed onto slab on grade and cast-in-place grade beams. Eventually, all building gravity loads are carried by the columns to the drilled pier and spread footing foundations. Depending on the foundation location, these loads are either distributed into surrounding compact soil or competent rock.

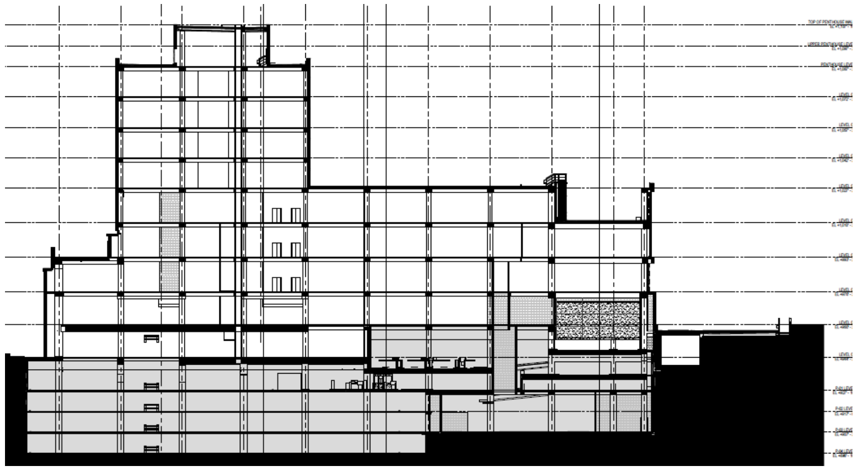


Figure 14: Building Section Looking North (SmithGroupJJR)

Wind loads place a lateral pressure on the bed tower façade. Pressure generates a force on the exterior façade that is distributed along its surface. This force is distributed based on stiffness to building elements. Exterior moment frames will take some of the lateral load, and the floor diaphragm will distribute the remaining lateral force to the other concrete moment frames. The columns of the moment frames will carry the lateral force and its foundation will resist overturning moment created by the lateral force, as depicted in Figure 14.

Seismic loads are due to ground acceleration during a seismic event. Ground acceleration causes building acceleration, which is based on the building mass and is quantified during design as story forces. The forces caused by acceleration are distributed on each floor according to stiffness of building framing elements. Above grade, seismic story forces are distributed by the diaphragm to concrete moment frames and carried by the columns down to the foundations. Below grade, seismic story forces are taken to the foundations by the exterior shear walls. The diaphragm will not distribute seismic forces as much due to the exterior location of the walls and their similarity in stiffness in each direction.

## 1.4 Structural Details

An architecturally exposed structural steel (AESS) canopy is a major architectural feature on the western side of the building. The canopy consists of double steel plate fins welded to AESS 4"x2"x ½" HSS. The canopy is connected to and supported by a 2x3 steel plates and 4x4x5/16 double angle kicker. These steel supports are welded to embed plates to connect the canopy to the concrete framing as seen in Figure 15.

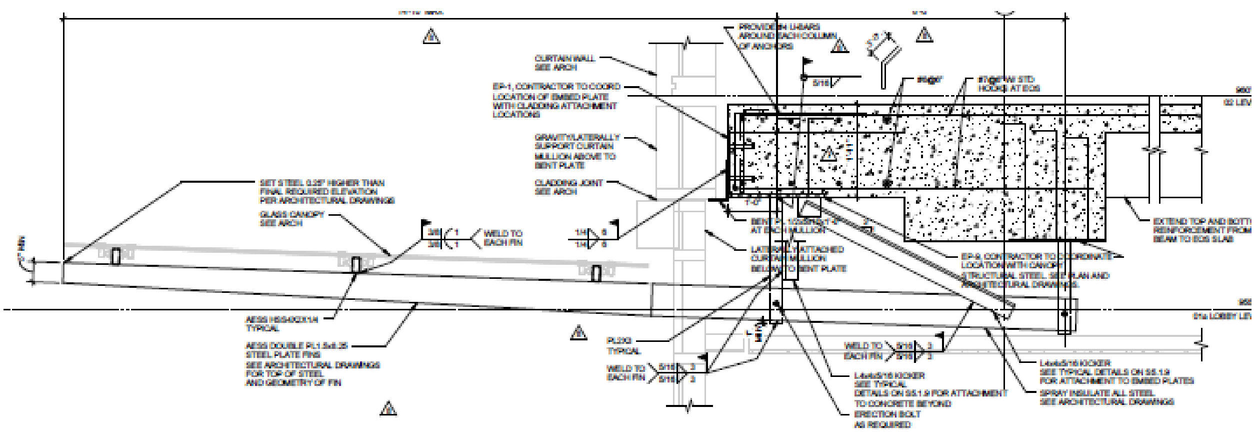


**Figure 15: West HSS Canopy Connection to Concrete Framing (Walter P. Moore)**

Post-tensioning is utilized in the four-story underground post-tensioned parking garage. Structures such as parking garages are often post-tensioned to prevent deflection and sagging due to car weight, and allow longer bay spans. At the exterior of the structure, the post-tension slab connects to a cast-in-place columns. 1" of clear cover for top and bottom of the slab is typical above and below anchors. #4 back-up bars are located between the tendon anchors. At the stressing end are pocket formers. Once the tendons are cut, the pocket is capped with non-shrink, non-staining grout.

# Chapter 2

# Codes and Loading Conditions

## 2.1 Applicable Codes

All known applicable building codes and standards are listed in Table 1. An exemption for the structural design permitted the use of 2006 ICC (International Code Council) codes as amended by the state. The rest of the building systems use 2012 ICC codes. A Special Land Use Permit was obtained from the county to build on land zoned for office-institutional use.

Table 1: Applicable Codes

| Category | Applicable Code |
|---|---|
| **Building Code** | 2006 IBC \| Structural Documentation Only<br>2012 International Building Code<br>2012 International Existing Building Code |
| **Energy Code** | 2012 International Energy Conservation Code |
| **Fire Code** | 2012 International Fire Code |
| **Mechanical Code** | 2012 International Mechanical Code |
| **Plumbing Code** | 2012 International Plumbing Code |
| **Fire Protection** | NFPA 10 \| Standard for Portable Fire Extinguishers 2010<br>NFPA 13 \| Installation of Sprinkler Systems 2010<br>NFPA 14 \| Standard for the Installation of Standpipe and Hose Systems 2010<br>NFPA 70 \| National Electrical Code 2011<br>NFPA 72 \| National Fire Alarm and Signaling Code 2010<br>NFPA 101 \| Life Safety Code 2000 |
| **Accessibility** | ADA 2010<br>ICC/ANSI 117.1 Accessible and Usable Buildings and Facilities 2003 |
| **Elevators** | ASME A17.1 Safety Code for Elevators and Escalators 2010 |
| **American Society of Civil Engineers** | ASCE 7-05 \| Minimum Design Loads for Buildings and Other Structures<br>ASCE 7-10 \| Non-Structural Requirements |
| **American Concrete Institute** | ACI 318 \| Building Code Requirements for Structural Concrete |
| **American Institute of Steel Construction** | Steel Construction Manual, Edition Unknown |

## 2.2 Gravity Loads

### 2.2.1 Original Design Loads

Design dead and live loads used for The Health Centre are tabulated in Table 2 below. Live loads for each occupancy/use category were taken from ASCE 7-05. Dead loads for different room types were taken from construction documentation. Unless otherwise noted as NR (non-reducible), live loads are reducible. In particular, horizontal framing floor members in parking garages did not have reduced live loads.

**Table 2: Original Design Loads**

| Occupancy/Use | Superimposed Dead Load (psf) | Live Load - Uniform (psf) | Live Load – Concentrated (lbs) |
|---|---|---|---|
| **Green Roof/Outdoor Area (RF2)** | 100 | 100 NR | - |
| **Typical Hospital Areas (HOS3)** | 15 | 100 | 2000 |
| **Hospital Diagnostic Areas (HOS2)** | 75 | 350 NR | 106,000 |
| **Mech./Elec. Rooms (MEC)** | 75 | 150 NR | 2000 |
| **Penthouse Roof (RFPH)** | 50 | 20 | - |
| **Mixed Use Areas (MU1)** | 55 | 100 | 2000 |
| **Restroom (RR)** | 40 | 100 | - |
| **Patient Rooms (PAT)** | 15 | 80 + 15 psf (partitions) | 1000 |
| **Lobbies and Corridors (PUB1)** | 15 | 100 | 2000 |
| **Parking Garage (PK1)** | 5 | 40 NR | 3000 |
| **Storage (STO)** | 15 | 125 NR | 2000 |
| **Kitchen (KIT)** | 95 | 150 NR | 2000 |
| **Typical Roof (RF3)** | 25 | 20 NR | - |
| **Insulated Roof (RF1)** | 50 | 20 NR | - |

### 2.2.2 Roof Loads

This section summarizes the roof gravity loads due to dead and live loads. Values provided in Table 3 are based upon standard values in engineering practice, ASCE 7-05, manufacturer data, and previous design experience. Due to the southern location of the building, snow loads were low and did not control design values.

**Table 3: Roof Gravity Loads Summary**

| Load Type | Dead | Live |
|---|---|---|
| **Typical Roof** | 83 psf | 20 psf<br>Not reduced |
| **Penthouse Roof** | 40 psf<br>(50 psf SDL from structural drawings) | 20 psf<br>Not reduced |
| **Green Roof** | 103 psf | 100 psf<br>Not reduced |

### 2.2.3 Floor Loads

Floor dead and live loads were determined for both the bed tower and parking garage floor systems in Table 4. Table values include slab self-weight for the two typical 5 and 7 inch slabs. All dead load values are for the typical 30'x30' bay and are based upon standard values in engineering practice and design experience.

**Table 4: Floor Gravity Loads Summary**

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

#### 2.2.4 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. Perimeter load values for these systems are listed in Table 5.

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 the steel bolts.

**Table 5: Perimeter Load Summary**

| Wall Type | Dead Load |
|---|---|
| **Curtain Wall** | 16 psf |
| **Panel Systems** | 18 psf |

### 2.3 Lateral Loads

#### 2.3.1 Wind Loads

Wind loads were calculated for the existing building and structural system according to ASCE 7-05 provisions. Table 6 shows the design criteria used for the perpendicular and parallel wind directions and Table 7 lists the resulting wind loads. The building frame is flexible, and the gust effect factor changes with the redesigned structural system. New wind pressures are presented later on for the redesigned structural system.

**Table 6: Design Criteria**

| | | | | |
|---|---|---|---|---|
| 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) |

**Table 7: Wind Load Summary**

| 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** |

### 2.3.2 Seismic Loads

Seismic loads for The Health Centre were calculated using the Equivalent Lateral Force (ELF) method provisions from ASCE 7-05 chapters 11 and 12. Seismic loads account for the weight of the existing structure, including the floors, mechanical rooms, and green roof. Below in Table 8 are values for Seismic Story Shear $V_x$. The corresponding story and floor forces and building overturning moment are depicted in the diagram in Figure 16. For the structural redesign, the shear wall lateral system and flat slab gravity system will have a different seismic response and weight. New seismic loads will be introduced in later sections.

**Table 8: Seismic Load Summary**

| 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 |

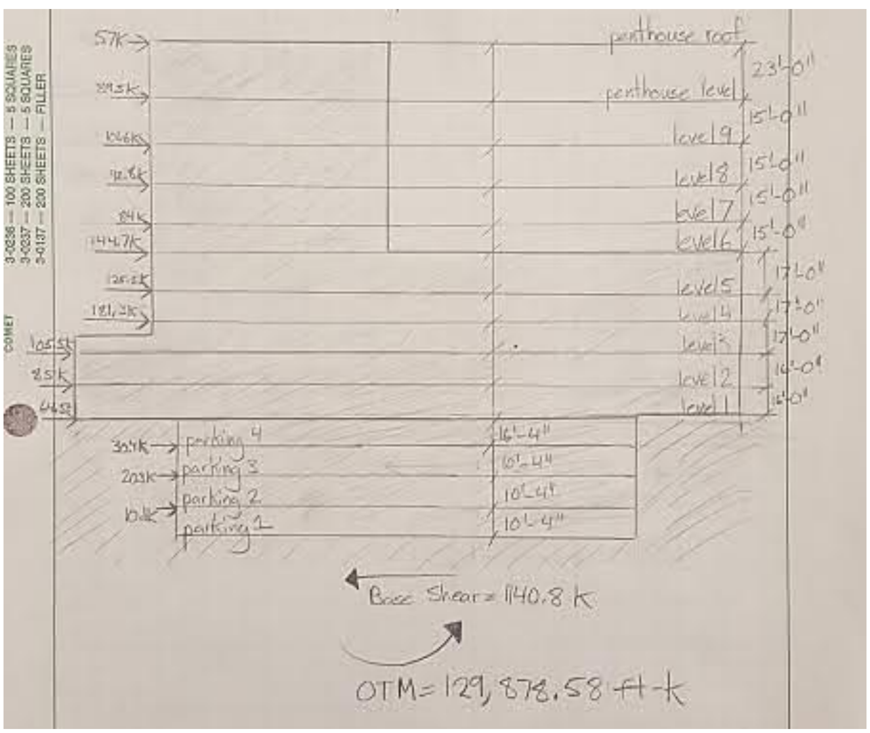



**Figure 16: Seismic Story Forces Diagram**

# Chapter 3
# Flat Slab and Shear Wall Design

### 3.1 Design Proposal

#### 3.1.1 Problem Statement

The existing structure of the Health Centre bed tower meets all necessary strength, code, and serviceability requirements. Additionally, the building meets the contractor's request for a concrete structural system. To continue to fulfill this request and pursue a deeper knowledge of concrete design, further consideration will be given for alternative concrete gravity and lateral systems for the bed tower. The alternative systems will be selected to satisfy the client's desire to fit in more with the surrounding university campus buildings and decrease the Health Centre's overall height. Solutions will explore the feasibility of a thinner gravity system to decrease story heights above grade. The below grade parking garage gravity system will not be included in the scope of this redesign.

A scenario in which the client wishes to replace some patient beds with additional research areas that use vibration sensitive equipment will be introduced for the alternative structural system. Such areas will be designed for the appropriate vibration criteria.

#### 3.1.2 Construction Breadth

The impact of the alternative structural system on construction cost and schedule will be analyzed in a construction breadth. In general, changes to the structural system will alter the critical path of construction. The new critical path – in addition to the new cost of materials - will affect the overall project cost. Cost and schedule analysis will be used to determine the feasibility of the proposed structural system.

### 3.1.3 Mechanical Breadth

The warmer southeastern US climate identifies cooling loads as a design driver when selecting HVAC systems for the building. Hospital and research areas in particular require proper mechanical conditioning for a building to function properly. The addition of large shear walls to the exterior and interior of the building will act as insulation and potentially retain cold air throughout the course of the day. However, the addition of thick concrete walls can also keep heat in the building, and is a potential coordination issue between mechanical and structural systems outside of shear wall and duct locations. This breadth will study the change in building envelope R values caused by the addition of shear walls. The Cooling Load Temperature Difference method, along with Appendix E of Mechanical and Electrical Equipment for Buildings, 10$^{th}$ edition and 1997 ASHRAE Fundamentals tables will be consulted for a preliminary analysis of the effects of shear walls on the building cooling loads.

### 3.1.4 MAE and Schreyer Requirements

Work for this thesis will meet requirements set by both the Schreyer Honors College and the Department of Architectural Engineering. To satisfy Honors College requirements, an investigation of current requirements, research, and design approaches for vibration sensitive equipment will be carried out. This investigation will focus on information relevant for equipment typically found in hospital and research laboratories, including microscopes and MRI equipment. Both steel and concrete structures will be considered, with a focus on the redesigned concrete structure of the Health Centre. Appropriate finite element analysis software, such as SAP2000, will be used to model a three-bay span of the Health Centre for vibration analysis. The results of this modeling and research will provide a better understanding of building stiffness and behavior under walking excitation. Overall, this investigation will provide experience for situations that may occur in the structural engineering industry and encourage professional development.

Additionally, the proposed concrete redesign will fulfil requirements for the Graduate School of the Pennsylvania State University. Coursework from AE 530: Advanced Computer

Modeling of Building Structures will be used to construct and verify a three-dimensional computer model of the redesigned building in Etabs. SAP2000 will also be utilized to verify Etabs output and analyze the fundamental period of several bays for vibration analysis. Modeling the building in three dimensions will promote a greater understanding of building behavior, stiffness, and various end and joint conditions. Coursework from AE 538: Earthquake Resistant Design of Buildings will be used to provide seismic reinforcing detailing for the shear wall design that increases ductility and strength in a seismic event.

### 3.2 Weighing Alternative Design Solutions

Three alternative gravity systems were explored for the Health Centre structural redesign. The systems under consideration include a composite wide-flanged steel system, a two-way flat slab, and non-composite steel joists. All gravity framing systems maintained the 30'-0" x 30'-0" bay size of the existing structure for better coordination with the below-grade parking garage. The parking garage gravity system will remain the same post-tension flat plate system as the original structure. Potential structural systems were evaluated based on strength and serviceability. Additional consideration was given to architectural and construction concerns in the final decision matrix show in Table 9. The flat slab system was chosen for the redesign proposal due to the ability to maintain a both thinner slab and a stiffer structure at drop panel locations.

**Table 9: Gravity System Decision Matrix**

| | 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 |

## 3.3 Two-Way Flat Slab with Drop Panels

### 3.3.1 Preliminary Design and Codes

The two-way flat slab system was an economical choice for the building's regular 30'x30' bays, spanning large distances with less concrete than the existing one-way skip joist system. Due to the large 30 ft span, issues with punching shear were expected from the beginning of design. The addition of drop panels would eliminate problems with punching shear and stiffen the system for later vibration considerations for the 5$^{th}$ and 6$^{th}$ floors. This system would also reduce floor thickness, one of the main goals of the redesigned system. Overall, the floor system is regular and a typical mesh of reinforcing is possible for the bottom and top bars. However, regular slab depressions for beds on floors 7-9 and openings for elevators and mechanical shafts will require special detailing. Although edge beams were considered during the preliminary alternative solution analysis, they were not implemented in the final design. 5000 psi concrete was chosen to match the existing design requirements, with the intent of increasing concrete strength if necessary

Code considerations for design used ACI 318-11. The use of ASCE 7-05 for the redesigned system was chosen for a more consistent comparison to the gravity and lateral loads of the existing structure. Design of the two-way flat slab system followed ACI Chapter 13: Two-Way Slab Systems of the ACI 318-11 Building Code Requirements for Reinforced Concrete. The direct design method detailed in ACI Chapter 13 was used to obtain preliminary slab and initial drop panel thickness. Typical floors considered for design include the 1$^{st}$–3$^{rd}$ floors, 4$^{th}$ floor, 5$^{th}$–6$^{th}$ floors, and 7$^{th}$–9$^{th}$ floors. Dead, live, roof live, and snow loads were all determined according to engineering experience and ASCE 7-05. All ASCE 7-05 load combinations were considered during design. The penthouse floor and its atypical loads were also included in the design scope.

### 3.3.2 Modeling and Results

Design of the two-way flat slab system had several iterations. Preliminary sizes obtained from the direct design method were used for the first design iteration in RAM Concept. A 10 in slab was initially used per minimum requirements from ACI 318-11 Table 9.5c to simplify calculation of slab deflections. Drop panels were kept to the minimum width of the span/6. Most drop panels are 10'x10' squares, with the exception of panels along grid lines J and 13. Initial drop panel thicknesses assumed the depth of a typical 2x4 dimensional lumber and a horizontal ¾" for formwork, totaling 14 ¼ inches from the top of the slab. The initial sizes required larger rebar sizes than expected, and several iterations of design followed. Design spans were laid out in both the N-S and E-W Directions, as shown in Figure 17 and Figure 18. Column and middle strip widths are typically 14'-2" in both directions for typical bays due to the square bay size. Square bays also allowed top and bottom reinforcing to be uniform in both directions.

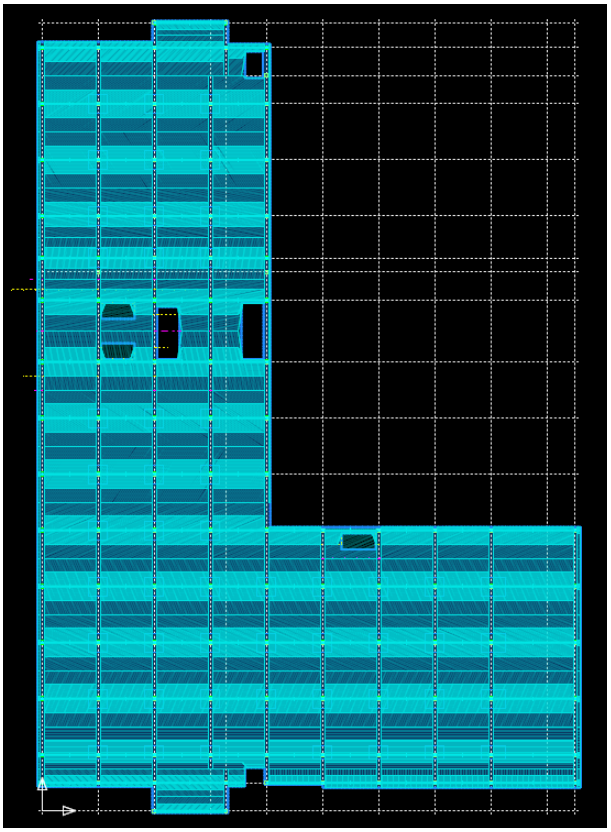


**Figure 17: 4th Floor Longitudinal Design Strip Plan**

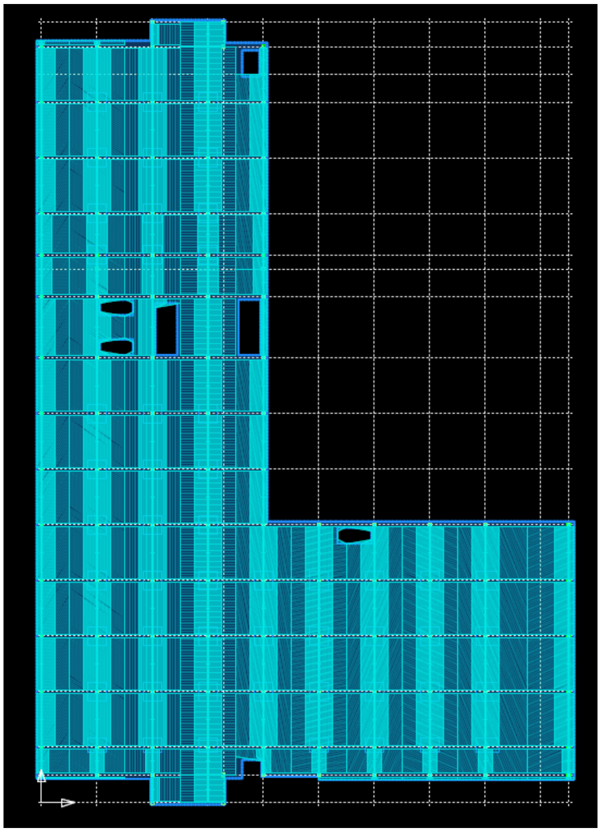


**Figure 18: 4th Floor Latitudinal Design Strip Plan**

Design iterations for slab and drop panel thicknesses considered during design are listed on the following page. The final thicknesses chosen for design were an 11 in slab with 19 ¼ drop panels, matching dimensions suggested by the CSRI Manual of Standard of Practice. This choice maintained the thinnest slab possible for punching shear limitations without requiring larger rebar sizes or excessively tight rebar spacing. Note that all drop panel thicknesses listed below begin at the top of the slab.

- 10 in slab with 14 ¼ in drop panels
- 11 in slab with 14 ¼ in drop panels
- 13 in slab with 17 ¼ in drop panels
- 11 in slab with 19 ¼ in drop panels

A standard mesh of #5 rebar for bottom reinforcement and #7 rebar for top reinforcement in column and middle strips is maintained throughout the typical conditions in both directions. Top and bottom reinforcement in the longitudinal direction was considered more critical due to the longer spans, and assigned a minimum clear cover of 0.75 in. In the latitudinal direction, top and bottom reinforcement was assigned a minimum clear cover of 1.5 in. The exception for this is at slab depressions on the 7$^{th}$ – 9$^{th}$ floor. Top reinforcement in these locations is 2 in lower than the typical standard mesh. Slab thickness also extends 2 in below the 11 in slab due to issues with excessive deflections shown in RAM Concept. A 3-D perspective view of floors 7-9 is shown in Figure 19. When modeling the slab, slab depressions, and slab openings in RAM Concept, a priority of 1, 2, and 3 were assigned to each element, respectively.

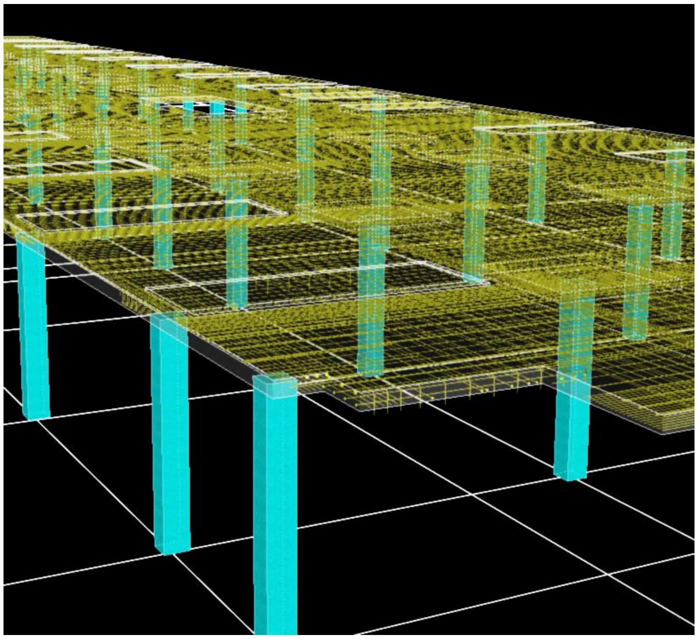


Figure 19: Floors 7-9 Depression and Drop Panel Perspective

Punching shear was checked at all columns in RAM Concept and via hand calculations located in Appendix A.1. The punching shear radius used in RAM Concept was increased to 11 ft on interior columns to consider critical sections both a distance d/2 away from the column and a distance d/2 away from the drop panel. “Max Shear Core” was selected to include the thickness of the drop panels in RAM Concept punching shear checks.

Deflection limits of l/240 were also checked during the design process in RAM Concept. The choice of an 11 in. slab based on Table 9.5(c) from ACI 318-11 prevented most issues with deflection, as the minimum thicknesses listed in the table are based on previous experience with concrete design and performance over the years. However, some locations required further study on all floors, as highlighted in Figure 20.

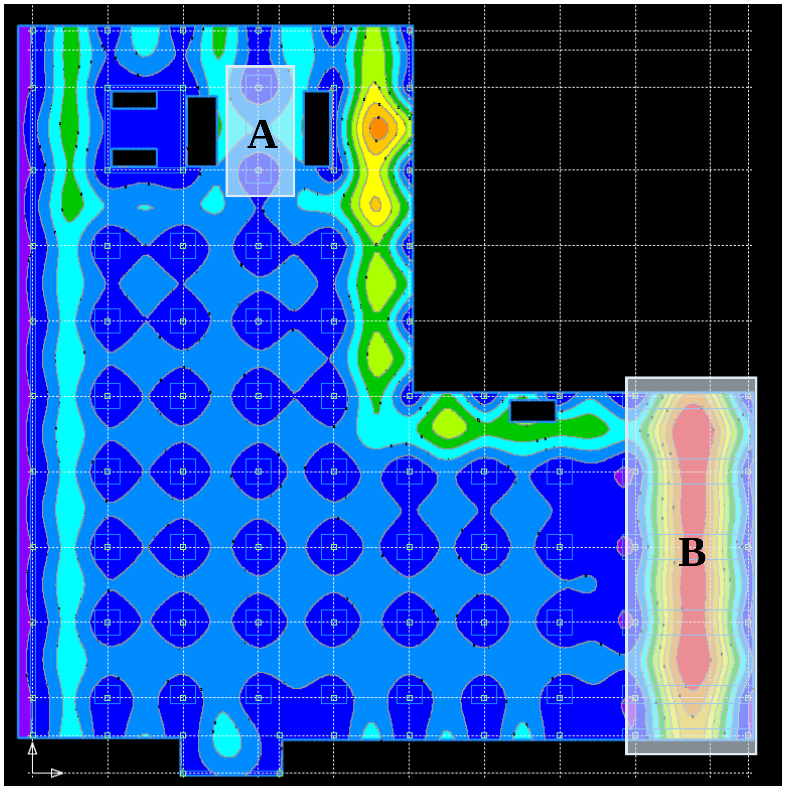



Figure 20: Floors 1-3 Short Term Deflections

In location A, multiple openings in the slab required an increase in floor stiffness. A drop panel was run between columns across the 30’ bay in this location. Additionally, beams were added to frame the bay in this location. The width of the beams was kept the width of the column for constructability, and a preliminary depth of 36 in was assigned. These adjustments solved the short and long term deflections issues in this location.

In location B, a 45 ft bay span from the original architectural layout created problems with excessive deflections. Drop panels were run between columns in this location, but short term deflections were still 7 inches in some locations. Framing solutions with beams were considered and a preliminary study of the effects of this framing system was run in RAM Concept. Ultimately, these solutions proved to add more weight and create deflections in other areas of the slab, and were not pursued further. To best solve deflection issues in these locations, the bay size should be adjusted and shortened. Due to this change necessitating the relocation of mechanical and hospital equipment, the project architect would need to be consulted.

A typical bay from the 4$^{th}$ floor is shown in Figure 21. Typical top bars for all floors are (10) #7 at 15”. Typical bottom bars for all floors are (18) #5 at 11”. All RAM Concept floor plans are located in the Appendix, along with direct design hand calculation verification. Additional vibration analysis of the two-way flat slab system will be discussed in Chapter 4. A typical interior bay on the 5$^{th}$ and 6$^{th}$ floors will be assessed for AISC Design Guide 11 provisions for MRI equipment. Precedent for using Design Guide 11 criteria for concrete gravity systems is discussed in the attached literature review, and includes buildings such as Cornell’s Nanotechnology Laboratory constructed in 2013 and Duke’s Science Center constructed in 2006.

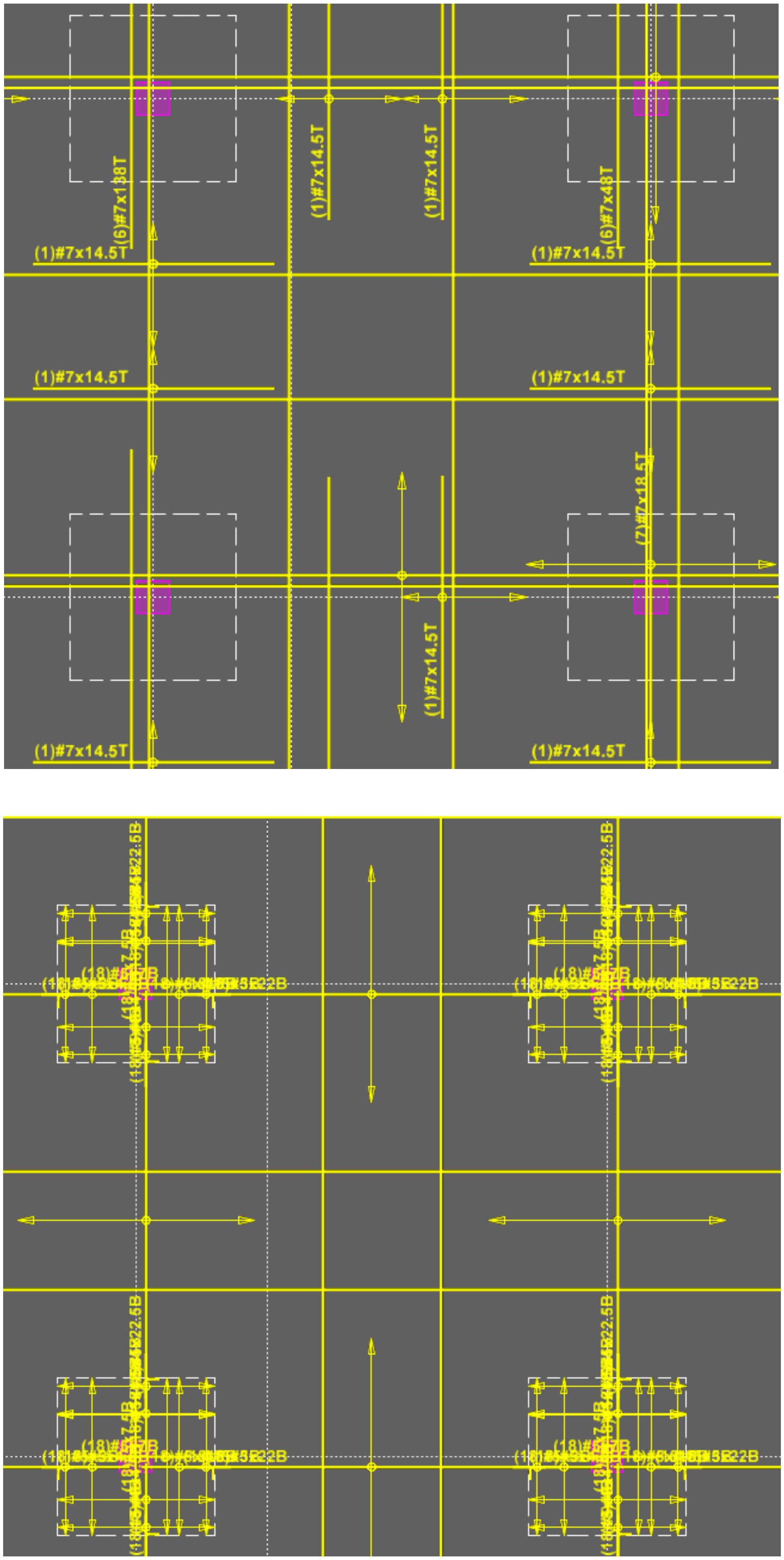



Figure 21: Typical Bay Top (Above) and Bottom (Bellow) Bars

## 3.4 Gravity Columns

### 3.4.1 Preliminary Design and Codes

Gravity columns were designed to support all building gravity loads. As a result, axial load on shear walls could essentially be neglected with the exception of self-weight later in the design process. An exception to this is exterior columns, which may still have moments due to lateral loads. The preliminary column sizes were chosen based upon the existing column dimensions of the Health Centre in combination with the CSRI manual. These sizes were 28"x28", 20"x20", and 18"x18" and checked with initial strength calculations for typical preliminary building loads. Column splices will occur at changes in column size as the building height increases. Locations of columns were kept the same as the original layout. The change in lateral system from intermediate concrete moment frames to concrete shear walls meant that column orientation was

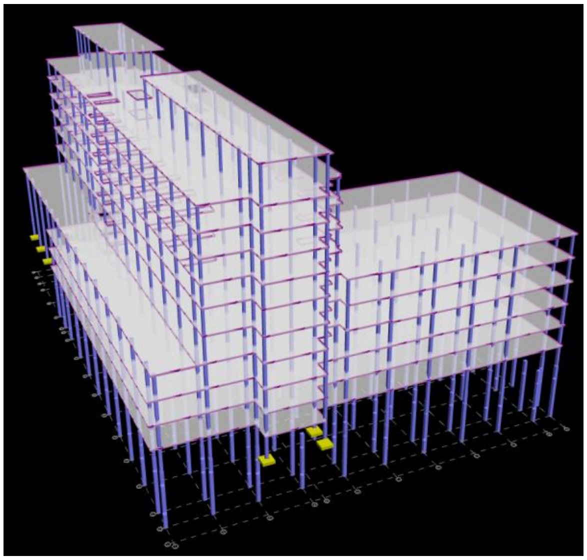


Figure 22: Gravity Model for RAM Structural System

no longer integral to resisting wind and seismic forces, and square columns could be used.

Loads and load combinations used for gravity column design came from ASCE 7-05. Loads for design were checked with these preliminary loads, and then taken from the RAM Structural System model shown in Figure 22. ACI 318-11 Chapters referenced during design include: Chapter 9: Strength and Serviceability Requirements, Chapter 10: Flexure and Axial Loads, and Chapter 21: Earthquake Resistant Structures.

### 3.4.2 Modeling and Results

Initial column sizes were placed into RAM Structural System to obtain building loads for RAM Concept and SP Column analysis. Columns were fixed-fixed between floors, and pinned-fixed at the lowest parking level below grade. The largest unbraced length for column design was 17 ft for building floors, and 23 ft for the penthouse roof. Columns were then modeled with dead, live, and roof live loads obtained from RAM in StructurePoint (SP) Column. Cracked property modifiers of 0.35 and 0.7 were used for beams and columns, respectively, during design in SP Column and RAM. Columns were placed into four design groups based on building loads, accounting for differences in loading such as the green roof between column lines A and B. Column groups also considered location to account for the floor plan decreasing area as the building height increases. Figure 23 shows the column groups used for design.

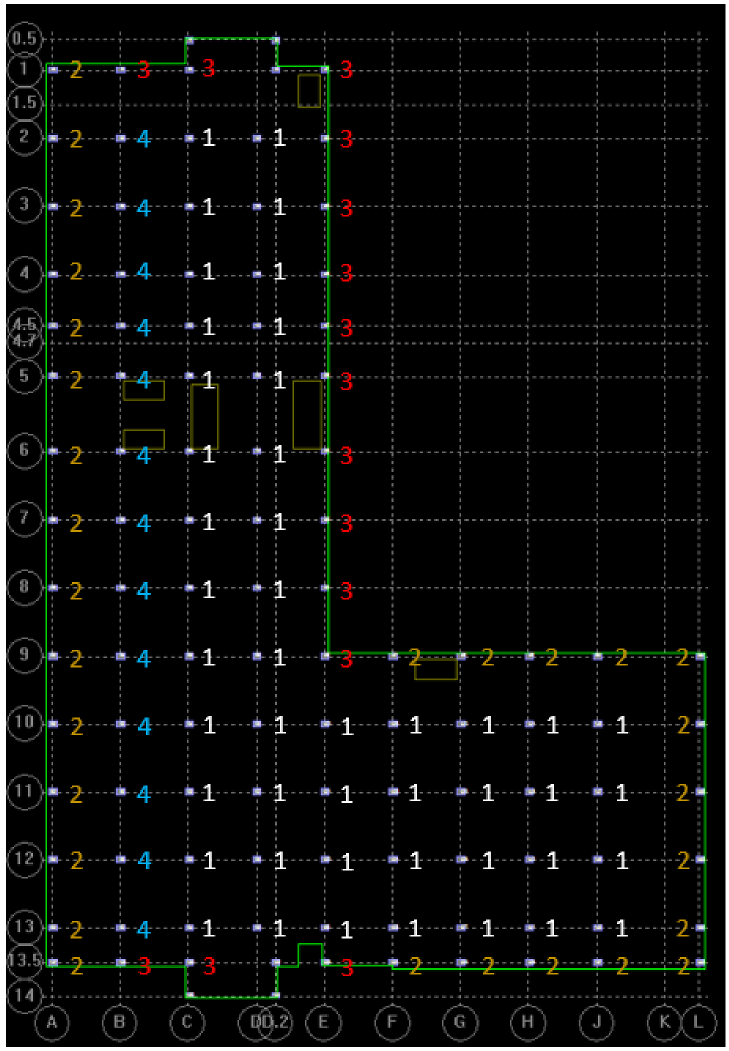



Figure 23: Column Design Groups

Inputs for SP Column considered slenderness due to columns above and below. An f'c value of 5000 psi was used for columns. 6000 psi concrete was considered, but ultimately not implemented because column sizes were acceptable to obtain the required strength. Service and factored load combinationswere run for all iterations of design in SP Column. Once initial column sizes were run and resized in SP Column, the new column sizes were updated RAM Structural System and RAM Concept. The new building loads from RAM were then input in SP Column. This iterative process occurred until the column capacity in SP Column could carry the updated building loads. Typical column loads in Table 10 were first determined for the controlling 1.2D + 1.6L + 0.5$L_r$ load case to corroborate building loads.

Table 10: Axial Load Verification

| Level | Dead (psf) | Live (psf) | Red. Live (psf) | Total Axial Load (K) | | |
|---|---|---|---|---|---|---|
| | | | | Dead | L or $L_r$ | 1.2D+1.6L+.5$L_r$ |
| **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.

Table 11 shows the final dimensions and reinforcing for all four column groups. All columns were designed to have equal bars on all sides, with a minimum clear cover of 0.75 in. A clear cover of 1 ¼ in is typical for most columns. Reinforcing ratios were kept below 4% to avoid mechanical column splicing. Group 4 columns required a larger column size to have enough capacity to extend from the lowest parking deck to the penthouse roof and carry increased loads from the occupied green roof. Final column sizes considered coordination with maintaining proper circulation on the parking garage levels.

Table 11: Column Group Final Design

| Column Group | Floor | Sq. Dim. (in) | Bars | As ($in^2$) | Rein. Ratio |
|---|---|---|---|---|---|
| 1 | P04-Level 1 | 30 | (16) #8 | 12.64 | 0.014044 |
| | Level 1-4 | 24 | (18) #8 | 14.22 | 0.024688 |
| | Level 4-9 | 20 | (16) #8 | 12.64 | 0.031600 |
| | Penthouse | 20 | (8) #8 | 6.32 | 0.015800 |
| 2 | P04-Level 1 | 28 | (12) #9 | 12 | 0.015306 |
| | Level 1-4 | 22 | (16) #10 | 20.32 | 0.041983 |
| | Level 4-6 | 20 | (16) #8 | 12.64 | 0.031600 |
| 3 | P04-Level 1 | 28 | (8) #9 | 8 | 0.010204 |
| | Level 1-4 | 22 | (8) #8 | 6.32 | 0.013058 |
| | Level 4-9 | 20 | (8) #7 | 4.8 | 0.012000 |
| | Penthouse | 20 | (8) #7 | 4.8 | 0.012000 |
| 4 | P04-Level 1 | 36 | (16) #11 | 24.96 | 0.019259 |
| | Level 1-4 | 32 | (24) #11 | 37.44 | 0.036563 |
| | Level 4-9 | 24 | (8) #8 | 6.32 | 0.010972 |
| | Penthouse | 20 | (8) #7 | 4.8 | 0.012000 |

## 3.5 Shear Wall Design

### 3.5.1 Preliminary Design and Codes

Shear walls resist lateral building loads in both the North-South and East-West directions around the elevator core, mechanical shafts, and exterior walls. Due to the changes in floor plan on each level, designing a completely symmetric shear wall layout was not possible. Exterior walls shift backwards as the building height increases, and floors 7-9 are much smaller in area than the lower levels. Architectural coordination also complicated the placement of shear walls. As a core and shell building, the architectural floor plan has no permanent walls to ensure future flexibility of the spaces. Windows surround the building exterior, as seen in Figure 24.

Ultimately, an architectural design decision was made to prioritize keeping open interior spaces over coordination with the exterior window façade. This decision created 15 ft architectural spaces on the west exterior façade that could be used for potential offices, conference rooms, or waiting areas in the hospital. The final design layout is shown in Figure 25.

**Figure 24: Exterior Facade of the Health Centery (SmithGroupJJR)**

Design of the shear walls follows the provisions of ACI 318-11 Chapters as follows: Chapter 9: Strength and Serviceability Requirements, Chapter 10: Flexure and Axial Loads, Chapter 11: Shear and Torsion, and Chapter 21: Earthquake Resistant Structures. Cracked section modifiers for out-of-plane bending will be used when modeling both gravity and lateral concrete elements per ACI requirements. The 2006 International Building Code and minimum design loads from ASCE 7-05 were referenced for consistency with the gravity structural system.

### 3.5.2 Lateral Loads

Wind and seismic loads were both recalculated for the new structural system. A new gust effect factor was found for wind loads, and seismic loads were updated for the new structural system and building masses. Overall, seismic forces controlled for strength design. Equivalent Lateral Force procedure was permitted for use to determine the new building loads. Both seismic and wind loads were considered for building deflection limits. New lateral loads for the redesigned structure are listed in Tables 12 and 13.

Table 12: Revised Wind Story Forces in Perpendicular and Parallel Directions

| 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** |

Table 13: Revised Seismic Story Shear and Overturning Moment

| 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 | | 1.31 | 0 | 0 | 0 | 0 | |
| **Penthouse Level** | 188.2 | 3987 | 1.31 | 3805379.17 | 0.126135 | 270.5592 | 50919.232 | |
| **Level 9** | 173.2 | 4487.5 | 1.31 | 3841512 | 0.127332 | 273.1282 | 47305.798 | |
| **Level 8** | 158.2 | 4487.5 | 1.31 | 3411653.76 | 0.113084 | 242.5656 | 38373.881 | |
| **Level 7** | 143.2 | 4487.5 | 1.31 | 2994262.11 | 0.099249 | 212.8894 | 30485.767 | |
| **Level 6** | 128.2 | 6492.4 | 1.31 | 3747473.6 | 0.124215 | 266.4421 | 34157.879 | |
| **Level 5** | 111.2 | 6492.4 | 1.31 | 3110301.86 | 0.103095 | 221.1398 | 24590.741 | |
| **Level 4** | 94.2 | 8774.7 | 1.31 | 3382508.99 | 0.112118 | 240.4934 | 22654.483 | |
| **Level 3** | 77.2 | 7232.2 | 1.31 | 2148073.27 | 0.071201 | 152.7261 | 11790.458 | |
| **Level 2** | 62.2 | 7232.2 | 1.31 | 1618586.19 | 0.05365 | 115.0801 | 7157.9805 | |
| **Level 1** | 47.2 | 7232.2 | 1.31 | 1127546.46 | 0.037374 | 80.16757 | 3783.9095 | |
| **P4** | 30.9 | 6010.7 | 1.31 | 537985.611 | 0.017832 | 38.25031 | 1181.9346 | |
| **P3** | 20.6 | 6010.7 | 1.31 | 316294.115 | 0.010484 | 22.48824 | 463.25767 | |
| **P2** | 10.3 | 6010.7 | 1.31 | 127568.021 | 0.004228 | 9.069975 | 93.420746 | |
| **P1** | 0 | 6010.7 | 1.31 | 0 | 0 | 0 | 0 | |
| | | | | | | **2797** | **272958.74** | **ft-k** |

### 3.5.3 Modeling and Verification

Etabs was chosen to model the new shear wall system. The model included floor diaphragms and shear walls, with the assumption that gravity loads would be carried by the gravity columns. To determine the best layout, walls were placed in ETABS in prospective locations on the exterior and around elevator and mechanical shafts with the same initial 16" wall section. Etabs analysis showed this layout and wall thickness meant that many shear walls had inadequate flexural strength to resist seismic loads at the lowest building level. An iterative process of increasing shear wall thicknesses to a typical thickness of 18" and identifying appropriate interior locations on the open floor plans for additional shear walls led to the final shear wall layouts shown in Figure 25. Assumptions for the Etabs model are listed in Table 14.

**Table 14: Etabs Property Summary**

| Element | Type | Properties | Modifiers | Assumptions |
|---|---|---|---|---|
| **Shear Wall** | shell-thin | f'c = 6000 psi<br>thickness = 18",20",22" | $I = 0.35I_g$<br>Self-Wt. = 1.0 | Cracked concrete<br>Fixed at base |
| **Diaphragm** | shell-thin | f'c = 5000 psi<br>thickness = 13" | $I = 0.25I_g$ | Cracked concrete<br>Rigid |

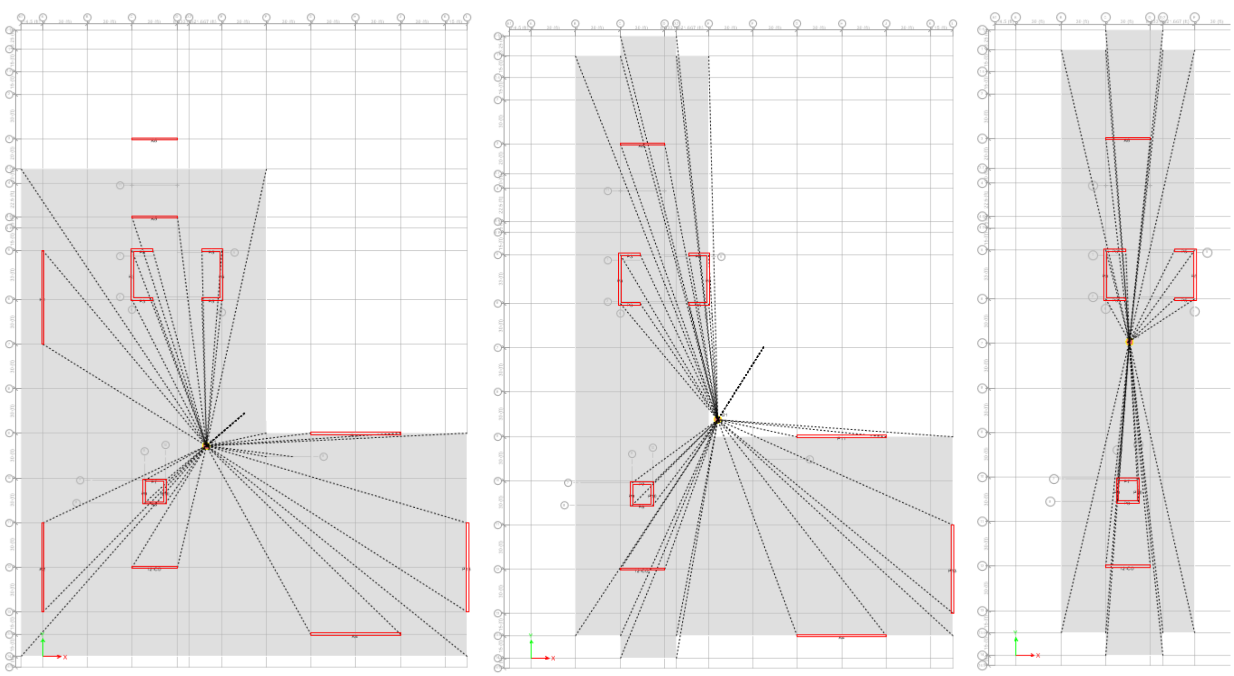


**Figure 25: Shear Wall Layouts of 1st, 5th, and 7th floors, respectively**

During the iterative design process, building eccentricities were checked on each level to limit the level of torsional shear on each floor. After the final layout was determined, centers of mass and rigidity were calculated for the 5$^{th}$ floor of the model to verify results. Table 15 and

Figure 26 depict the results of both Etabs and hand calculations. Overall, results verify the model for center of mass. Differences in center of rigidity between the calculated and Etabs values may be attributed to the manner in which Etabs calculates element stiffness. A unit load is applied in the x and y direction, causing rotations $R_{zx}$ and $R_{zy}$, respectively. A unit moment is applied to the z-axis for rotation $R_{zz}$. The building's center of rigidity is calculated by determining $-R_{zy}/R_{zz}$ and $R_{zx}/R_{zz}$. The spreadsheets used for the hand calculation analysis of center of rigidity are located in Appendix A.3.

**Table 15: Center of Mass and Rigidity Comparison**

| | $COM_x$ (ft) | $COM_y$ (ft) | $COR_x$ (ft) | $COR_y$ (ft) | $e_x$ (ft) | $e_y$ (ft) |
|---|---|---|---|---|---|---|
| **Calculated** | 129.2158 | 160.9153 | 62.16482 | 89.01671 | 67.05097 | 71.89855 |
| **Etabs** | 126.0024 | 161.4631 | 94.4072 | 116.1521 | 31.5952 | 45.311 |
| **% error** | -2.48684 | 0.340456 | | | | |

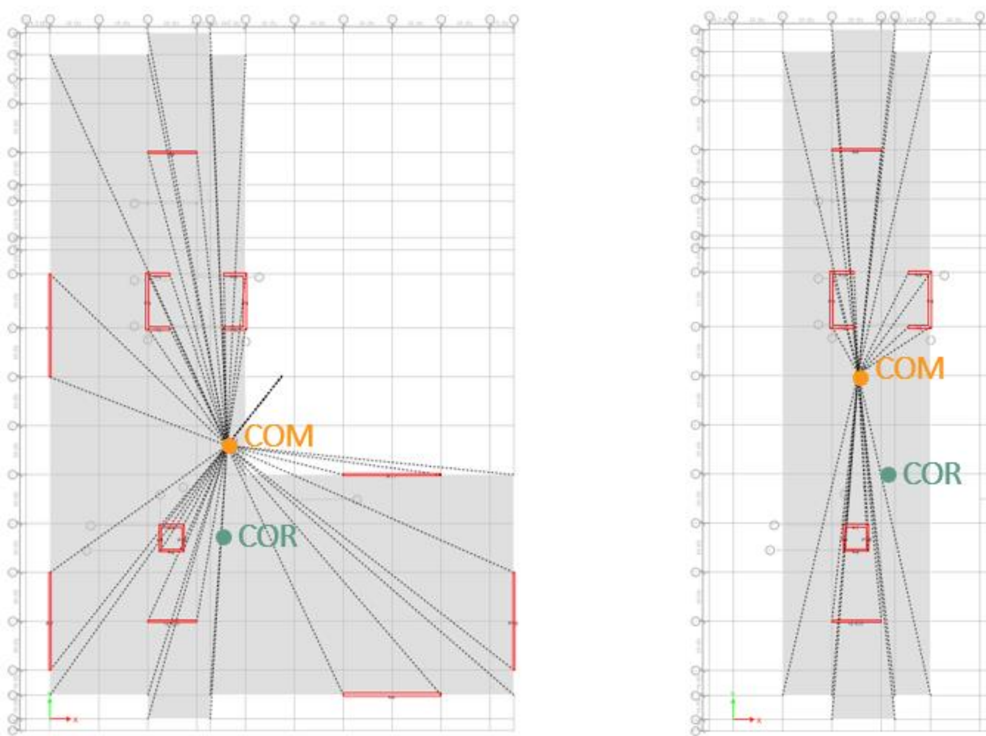



**Figure 26: Etabs COM and COR results**

For strength design, seismic load cases controlled design. Etabs generated seismic loads used for analysis, after verifying that the loads were similar to base shear and story forces determined from the previous hand calculations. Etabs load combinations consider ASCE 7 requirements that the building design accounts for 100 percent of seismic loading in one direction, and 30 percent of seismic loading in the other. Table 17 shows the base shear comparison between Etabs and hand calculated loads. For Etabs seismic loading, all seismic coefficients and factors were user defined from the inputs for ordinary reinforced concrete shear walls. listed in Table 16. The bottom story considered in seismic as the lowest parking level, and the penthouse roof as the top story.

**Table 16: Seismic Coefficients**

| User-Defined Inputs Factors | Seismic Coeff. | Program Defined Inputs |
|---|---|---|
| R = 4 | Ss =.228 | Fa = 1.2 |
| Omega = 2.5 | S1 = 0.086 | Fv = 1.7 |
| $C_d$ = 4 | Long Period = 12s | SDS = .1824 |
| Importance = 1.5 | Site Class C | SD1 = 0.0975 |

**Table 17: Base Shear Comparison**

| Base Shear Comparison | | F (K) |
|---|---|---|
| Seismic | Calculated | 2145 |
| | Etabs | 2681.408 |

Final verification of the Etabs model analyzed lateral load distribution on the 5$^{th}$ floor level. Calculations verified that the amount of shear in each wall approximately matched distribution based on the relative stiffness of each wall and torsional effects, as listed in Table 18. Shear values in Table 3.10 checked walls for forces in the perpendicular direction, shown in white. Some differences in load distribution between Etabs and hand calculations may be attributed to the difference in methods for calculating relative stiffness. Additionally, the use of three-dimensional computer model allows for calculation of a more accurate building period than can be obtained from the approximation equations in ASCE 7 commentary.

**Table 18: Shear Distribution Check**

| Object | $R_x$ (lb-in) | Rel. Stiffness | Direct Shear (K) | | Torsional Shear (K) | | | | Total Shear (K) |
|---|---|---|---|---|---|---|---|---|---|
| | | | Seismic | $d_i$ (ft) | $R_x$*$d_i$ (k-ft$^2$) | $R_x$*$d_i^2$(k-ft$^3$) | $R_id_i/J$ (1/ft) | $V_t$ - Seismic | Seismic |
| A6 | 50756464 | 0.112483 | 24.173 | 258.9832924 | 1095423.019 | 4633316615 | 4.72761E-05 | 0.730462206 | 24.903 |
| P1 | 21238612 | 0.047068 | 10.115 | 183.9832924 | 325629.1518 | 576325943.2 | 1.7566E-05 | 0.271412295 | 10.386 |
| P4 | 21238612 | 0.047068 | 10.115 | 183.9832924 | 325629.1518 | 576325943.2 | 1.81297E-05 | 0.280121206 | 10.395 |
| P3 | 62035679 | 0.226551 | 48.686 | 167.4832924 | 865828.308 | 4476020552 | 4.83597E-05 | 0.747205228 | 49.433 |
| P5 | 62035679 | 0.226551 | - | - | - | - | - | - | - |
| A1 | 21238612 | 0.047068 | - | - | - | - | - | - | - |
| P6 | 21238612 | 0.047068 | 10.115 | 150.4832924 | 266338.0258 | 471387506.4 | 1.4876E-05 | 0.229848301 | 10.345 |
| P11 | 1.08E+08 | 0.239293 | 51.424 | 60.98329241 | 548737.9582 | 4937636767 | 3.0649E-05 | 0.473557942 | 51.898 |
| A4 | 1.2E+08 | 0.265882 | 57.138 | 74.01670759 | 740016.4754 | 7398659056 | 4.13326E-05 | 0.638630285 | 57.777 |
| P7 | 18407968 | 0.040794 | 8.767 | 28.98329241 | 44460.29302 | 68201970.55 | 2.48327E-06 | 0.038368997 | 8.805 |
| P8 | 18407968 | 0.040794 | 8.767 | 13.98329241 | 21450.33314 | 32904753.66 | 1.19808E-06 | 0.018511524 | 8.785 |
| P9 | 20888305 | 0.076283 | - | - | - | - | - | - | - |
| P10 | 20888305 | 0.076283 | - | - | - | - | - | - | - |
| 12-CD | 50756464 | 0.112483 | 24.17258789 | 29.01670759 | 122732.1236 | 519120720.7 | 6.85504E-06 | 0.10591717 | 24.279 |
| P13 | 1.08E+08 | 0.394331 | - | - | - | - | - | - | - |

### 3.5.4 Results

Story drifts for wind and seismic loading conditions were obtained from Etabs output. Table 19 evaluates this output against allowable inter-story drifts from ASCE 7-05 seismic limitations of $0.010h_{sx}$. Wind drifts are evaluated against a limitation of H/400, where H is the total height of the story. All stories met the allowable drift limitations in both the x and y directions.

**Table 19: Seismic and Wind Drift Limits**

| Story | h (ft) | Wind | | | Seismic | | |
|---|---|---|---|---|---|---|---|
| | | H/400 (in) | Story Deflect. (in) | Acceptable? | $0.010h_{sx}$ (in) | Drift (in) | Acceptable? |
| **Penthouse** | 23.5 | 5.706 | 0.786335 | Yes | 1.8 | 0.2785 | Yes |
| **9** | 15 | 5.256 | 0.697929 | Yes | 1.8 | 0.2784 | Yes |
| **8** | 15 | 4.806 | 0.609323 | Yes | 1.8 | 0.2774 | Yes |
| **7** | 15 | 4.356 | 0.521461 | Yes | 1.8 | 0.2753 | Yes |
| **6** | 15 | 3.906 | 0.436133 | Yes | 2.04 | 0.3177 | Yes |
| **5** | 17 | 3.396 | 0.345983 | Yes | 2.04 | 0.287 | Yes |
| **4** | 17 | 2.886 | 0.262567 | Yes | 2.04 | 0.2983 | Yes |
| **3** | 17 | 2.376 | 0.165761 | Yes | 1.92 | 0.1649 | Yes |
| **2** | 16 | 1.896 | 0.111967 | Yes | 1.92 | 0.122 | Yes |
| **1** | 16 | 1.416 | 0.068157 | Yes | 1.956 | 0.0966 | Yes |
| **P1** | 16.3 | 0.927 | 0.032648 | Yes | 1.236 | 0.0451 | Yes |
| **P2** | 10.3 | 0.618 | 0.016079 | Yes | 1.236 | 0.0302 | Yes |
| **P3** | 10.3 | 0.309 | 0.001666 | Yes | 1.236 | 0.0138 | Yes |
| **P4** | 10.3 | 0 | 0 | Yes | 0 | 0 | Yes |

An example shear wall design based on the loads from the worst case Etabs load combination is show in Figure 27. Hand calculations for this design are provided in the Appendix. Design of shear walls was run in Etabs to determine approximate areas of steel required for a building cost estimate and verify walls had adequate strength for all seismic load cases. However, the program was not used as a black box to design the walls themselves.

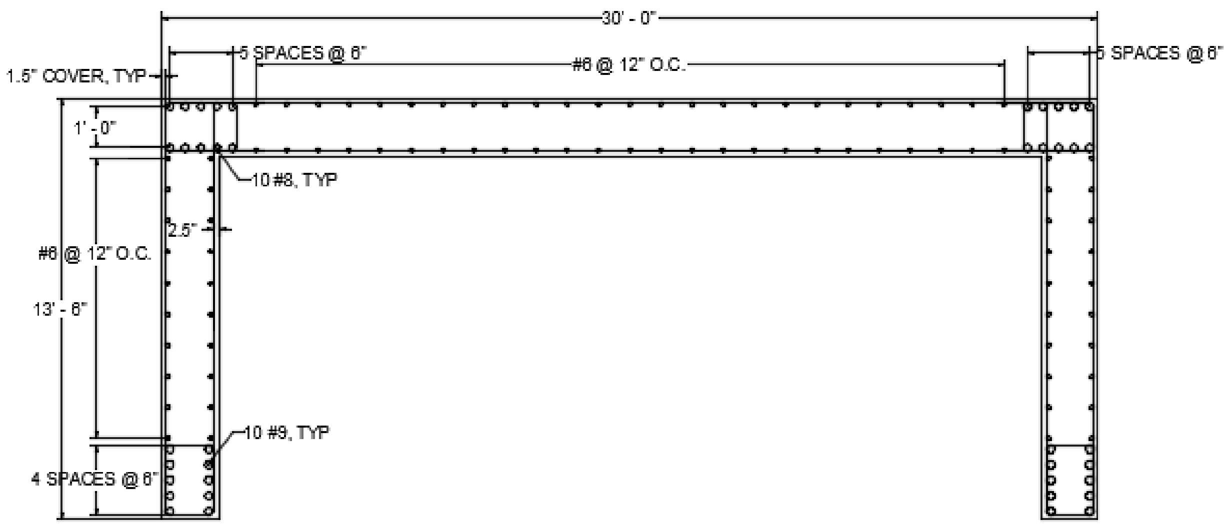



Figure 27: Elevator Core Shear Wall Cross Section at Base

### 3.5.5 Conclusions

The redesigned shear wall lateral system introduced architectural coordination issues that were not present with the existing moment frame system. A core-and-shell building aims to maintain a flexible floor plan such that interior spaces may shift with the building occupant's needs. Coordination of such spaces with a constantly changing floor plan along the building height and a curtain wall exterior is difficult with a shear wall lateral system. Overall, the redesigned system is a less efficient use of space and would require further architectural coordination in the design process.

# Chapter 4

## Vibration Considerations and Analysis

### 4.1 Literature Review

The following is a review of existing literature for structural engineers who wish to design for vibration sensitive equipment. Current industry practice is to follow standards set by Design Guide 11, released by the American Institute of Steel Construction (AISC). This literature review will also explore other design criteria and engineering practices, as well as the possible sources of structure-borne vibrations.

The most recent edition of AISC Design Guide 11 was published in 1997. It defines basic floor vibration terminology and principles for vibration design of structural steel. Additionally, the document provides acceptance criteria for floor vibration design, discussion of various vibration sources, and example calculations.

#### 4.1.1 Design Guide 11 Walking Excitation

Current AISC criterion for floor vibrations due to walking excitation were developed based upon North American floor systems constructed over 20 years ago. Floors of this age typically have a natural frequency between 5 and 8 hz. Floors with a natural frequency above 9 hz will not usually have significant resonance under walking excitation. Recent changes in the ways in which floors are designed and constructed - including the introduction of limit states design, longer span lengths, and lightweight concrete - have resulted in floors with frequencies lower than 5 hz than cannot always be properly predicted by the criterion provided by AISC data.

A floor system is properly designed for walking excitation if the peak acceleration does not exceed the acceleration limit ao / g. For typical offices, residences, and churches, ao / g is equal to 0.5%. Acceleration is evaluated for both girder and joist panel modes, and is based upon effective weight, member properties, and beam spacing. Continuity effects allow for a 50 percent increase in the effective weight considered, with the exception of beams framing into columns and joists connected solely at their top chord.

The damping of walking excitation due to the effects of non-structural elements, such as partitions, furnishings, and occupants, is considered by the modal damping ratio. Recommended modal damping ratios range from 0.01-0.05, with 0.05 typically used for buildings with partitions that extend the entire height of the floor. Additionally, floor stiffness is considered for floor systems with a natural frequency greater than 9-10 hz. The criterion for walking excitation vibrations should be evaluated with consideration for the location of walkways in a building. Exterior panels are typically stiffer due to exterior cladding and will not have the steady traffic flow of an interior corridor. Panels with interior floor edges may require stiffening due to a reduction in mass.

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**4,1.2 Design Guide 11 Vibration Sensitive Equipment**

General requirements for vibration sensitive equipment are to be used when product vibration criteria is not available. Chapter 6 of AISC Design Guide 11 provides one such set of general requirements for floors with frequencies between 1 Hz and 80 Hz. The greatest vibrational velocity of a floor may be determined from its frequency and the type of equipment under consideration. For example, equipment used for eye or neuro surgery is designed for a vibrational velocity of 2,000 micro inch/second.

Sensitive equipment used for viewing enlarged images, such as microscopes, may be designed for a more specific vibrational velocity based upon the magnification level of the equipment. It should be noted that 40x magnification is typical for surgical and workshop equipment. Peak vibrations are considered for this design method, while the decay rate of vibration and its effect on the human eye's perception is ignored.

Peak vibration of floors is usually caused by walking excitation. Walking creates a force pulse whose maximum force and decay rate is dependent upon the person's walking speed and weight. The maximum floor displacement due to a force pulse is dependent on the aforementioned factors and the fundamental natural frequency of the floor. The value of the maximum floor displacement is dependent on the beams and girders with consideration for composite action, while ignoring effects due to the slab and deck. If floor mass stays fairly constant, redesign of floors for vibration sensitive equipment will depend on increasing the floor's stiffness.

Typically, it is sufficient to stiffen the bays in which the sensitive equipment is located. A stiffer bay may be achieved with minor changes to the floor framing. Shortened spans will create a less flexible framing system. Stiffening the greater contributor to the floor's deflection - beams or girders - will also decrease floor flexibility.

Other redesign possibilities include moving the equipment further from areas with high walking traffic and closer to column lines, and placing expansion joints between corridors and the rooms with sensitive equipment. A separate structure may also be designed to support the vibration sensitive equipment if it is rigidly connected to the structural system to prevent amplification of vibrations. If sensitive equipment is not located in the middle of the bay or only slow walking may occur in the laboratory space, Design Guide 11 suggests computer analysis to determine floor vibration in the scenarios required for design.

#### 4.1.3 Vibration Criteria

*Generic Vibration Criteria for Vibration Sensitive Equipment*

The use of vibration criterion curves as the basis for the generic requirements of vibration-sensitive equipment is widely accepted in the engineering community. Since their establishment in the 1980's, VC curves have been published by organizations such as the American Institute of Steel Construction in Design Guide 11 and the Institute of Environmental Sciences and Technology. These curves are based upon root-means-square velocity to account for the variation in maximum sensitivity of people and equipment, and are designed to be used for the most sensitive equipment in each equipment category.

Alternatives to the VC curves exist but have not been widely accepted for various reasons. The Medearis Time Domain Method uses the time between peak displacements instead of frequency to assess sensitive equipment (Gordon). However, the Medearis Method is impractical when equipment manufacturers do not provide a time criterion for vibration evaluation. In development at the time Gordon's writing was the Ahlin Response Spectrum Method. The Ahlin Method is based upon the idea of a vibration response spectrum similar to the ones used in seismic design. Further review of this method was required to determine its practicality for a wider use.

VC curves common in today's vibration assessment may vary in accuracy of predicting actual behavior. Base conditions not accounting for the effects of HVAC equipment that may be installed nearby in the building may have been used to determine the manufacturer criteria. As vibration sensitive equipment ages, its performance will naturally decay over time. If a specific bay is designed for the equipment location, it decreases the flexibility of the space. Equipment must stay in the correct location to perform properly. With the technological advances of sensitive equipment, it may be necessary to revise VC curves for lower frequencies (Gordon). Overall, there is a clear need for further review of the accepted method of vibration criteria.

*Vibration Criteria for Sensitive Equipment*

Equipment may be considered sensitive to vibrations if internal relative displacement of components will affect equipment performance (Ungar, 1992). Additionally, equipment sensitivity to vibrations may be due to movement relative to an observer, such as in the case of a microscope. Amplitude criteria curves for equipment are based upon the most severe vibrations considered acceptable from experimental tests run at multiple frequencies and directions. Most vibrational velocity curves are developed from data from multiple manufactures. These curves, as previously mentioned, show that low frequencies may tolerate higher velocity vibrations. Ungar also discusses the development of the one-third octave bands and the decision to assume 10% critical damping that is currently used in Design Guide 11.

Studies by House and Randell evaluated the acceptability of various vibration environments for a range of microscope magnification (Ungar, 1992). As frequency increases, the eyes cannot follow the movement of object being magnified and views a blurry image. Depending on the type of observation being made, this blurring may still be considered acceptable.

*Evolving Criteria for Research Facilities: I - Vibration*

In 2005, generic criteria for vibration equipment reflects the evolution and progress made since the development of the VC curves in the 1980's. The VC curves remain a standard for generic vibration evaluation. Two important changes to these curves include the addition of curves VC-E, VC-F, and VC-G (Amick, 2005). These curves were introduced after the growth of the semiconductor industry, and are used for such equipment. Additionally, some curves were flattened since the original VC curves were introduced to account for additional sensitivity of equipment to lower frequencies.

The nano-science industry has also impacted the development of criteria for vibration sensitive equipment with its need for additional consideration for noise and low frequency sensitivity. Unlike in 1980, most sensitive equipment is now designed with internal isolation (Amick, 2005). The NIST-A criterion was introduced to account for pneumonic isolation increasing performance of sensitive equipment at frequencies less than 4 Hz. Ambient site survey data from existing nanotechnology facilities is still being collected and researched to determine if existing facilities meet the theoretical NIST-A criterion.

New technology is making it possible to adjust floor stiffness to meet the needs of sensitive equipment. Step-and-scan systems can now adjust the rectile and stage of equipment as needed, with forces from such adjustments being supported by the structure. Such systems are creating a new consideration for dynamic stiffness requirements that will need to be considered in addition to vibration in future structural design (Amick, 2005).

### 4.1.4 Design Approach

*Vibration Control Design of High Technology Facilities*

The sources of vibration for sensitive equipment can be broken down into three categories: external, internal, and service machinery. External sources include construction activities, highways and railroads, and nearby outdoor machinery. The most common internal source of vibration is pedestrian traffic in corridors (Ungar, 1990). In general, service machinery refers to HVAC equipment. Each vibration source has different design solutions. In his 1990 paper, Ungar, suggests solutions that are applicable for all vibration sources, such as stiffening the overall structure, which were covered in previous discussion of Design Guide 11.

Ungar's solutions including addressing external vibrations at the foundation level. Isolation of foundations through the use of air springs or bearing pads help mitigate soil vibration effects on the building structure. Changing the footprint and shape of the foundation can also be used to “tune” the building to attenuate a desired range of frequencies. In Japan, “tuned absorbers” have been successful with reducing soil vibrations. These tuned absorbers are large masses, typically concrete slabs, mounted to foundations to create a spring-mass system that dampens vibration. Although not always possible in practice, proper site selection away from sources of external vibration will eliminate the issue as well.

Internal vibrations may be mitigated by a structural solution suggested by Ungar in his 1990 paper. Corridors are the largest source of internal vibrations, and locating these closer to column lines and away from midspan is one possible way to reduce vibration. Pedestrian “bridges” that frame corridors directly into columns help reduce the transmittance of vibrations throughout the structure. A physical joint between structure supporting sensitive equipment and corridor structure may also achieve outcome. However, the authors also suggests non-structural solutions for internal vibration problems. They include placing signs requesting occupants to walk more slowly or eliminating long corridors as possible solutions for decreasing internal vibrations. These solutions seem impractical to enforce, particularly in a hospital setting that requires rolling of hospital beds and corridors for patient movement.

Mechanical vibration sources may be partially controlled with careful selection and placement of equipment. If the mechanical equipment is placed on grade, it may be isolated with sleeved columns on bedrock, a tuned bed of fill, or an air spring system (Ungar, 1990). Important to note for mechanical vibration is its tendency to transmit through other building components, such as piping and conduits.

*Floor System Vibration Control*

LeMessurier Consultants reviewed current practices for floor system vibration control in existing concrete and composite steel buildings using criteria from AISC Design Guide 11 (Hines). Concrete systems were evaluated using Design Guide 11 requirements and uncracked concrete sections during the design process. Experimental methods were used to estimate excited natural frequencies during a simulated vibration of the floor system and assess the its natural frequencies under resonance testing. Damping of each floor system was estimated after each resonance test.

Situations not covered in detail in Design Guide 11, specifically irregular floor systems and concrete floor systems, are also discussed by LeMessurier and Hines. Irregular floor systems, such as cantilevers or bridge walkways, may have special boundary conditions that require more attention than the typical floor system. Coupling between bays and proximity to columns have a greater impact on the performance of such systems. Depending on the situation, LeMessurier subscribes to the view that a finite element analysis model of the whole or subsection of the structural system may be necessary.

Concrete floor systems have much higher damping than structural steel floor systems, but in practice have been designed for AISC Design Guide 11 requirements. Some challenges in designing concrete floor systems for vibration control include a difficulty predicting fundamental frequencies and the depending of the natural frequency on the bay's end connections. The appropriate times to use approximations versus a finite element model are dependent upon the situation and the structure.

*Limiting Effects of Construction Vibrations on Sensitive Equipment*

Construction-related ground vibrations may impact nearby laboratory and medical vibration sensitive equipment. Because experimental data can be discarded or timed around construction activities, the main concern for these vibrations stems from damage to the equipment itself. Operating rooms and MRI equipment may also experience vibrations that render the equipment and space unusable during construction activities. Sources of construction vibration include driving piles, blasing, and the use of vibratory compactors (Ungar, 2011).

Ungar discusses developments in the assessment of construction vibration effects on vibration sensitive equipment, particularly in a research and hospital setting. Because vibration criteria provided by manufacturers for equipment may be conservative or for operation at the most sensitive setting, users of sensitive equipment are the best course for evaluating construction effects. Methods of user evaluation range from dropping a large concrete block to simulate construction vibrations to having users occupy the space during construction activities.

If construction vibrations are determined to be a problem, there are several courses of action one can take. Scheduling construction and sensitive equipment activities to occur at different times of day may work for hospitals with operating rooms. Alternative methods of construction may also be used, such as using vibratory pile drivers instead of impact pile drivers. These solutions are likely to add cost to the project (Ungar, 2011). However, they may be the best approach if significant vibration reduction is required. Structural measures, including slurry walls and trenches, are limited in their effectiveness and are only able to reduce construction vibration by a factor of 5 to 10.

*Case Studies of Structures with Man-Induced Vibrations*

These case studies review five types of structures that may experience vibrations due to human activity: footbridges, gymnasiums, dance and concert halls without fixed seats, concert halls and theaters with fixed seats, and swimming pool diving boards. The types of human activities that can cause vibration in these settings include walking, running, jumping, dancing, clapping, and swaying (Bachmann). To address man-induced vibrations, a designer may choose to frequency tune the structure, calculate and compare the vibration response spectra to acceptance criteria such as Design Guide 11, or stiffen the structure with damping or vibration absorbers.

According to Hines, frequency tuning designs a structure such that the frequencies of its modes of vibration avoid the range of frequencies typical for the human activities under consideration. The five types of structures previously discussed all have a different set of criteria typically used for frequency tuning. These natural frequency requirements may also be used to install a tuned-vibration absorber.

For example, reinforced concrete gymnasiums and sports halls recommend a natural frequency greater than 7.5 Hz (Bachmann). Such recommendations are based upon experiments in a two-story cast-in-place concrete gymnasium with a ribbed slab. Dynamic behavior was induced by two methods: dropping sandbags and having schoolchildren perform a standardized jumping test. Data from this experiment identified the damping ratio of the floor and the peak amplitudes for velocity and acceleration. It was determined in this case study that floor damping was 3% of g, less than the 5% or 10% typically required for serviceability, and that the structure needed fixed. The course of action chosen in this instance was keep the existing floor and install a steel girder grid underneath. This caused the fundamental frequency to change to 7.47 Hz, close to the 7.5 Hz recommended by the article's authors.

*Vibration Design of Concrete Floors for Serviceability*

ADAPT provides guidance on determining the vibration response of a reinforced concrete floor system. Recommendations are given for the appropriate load cases and property modifications for reinforced concrete vibration analysis. These recommendations include damping factors, extent of cracking to consider, and modulus of elasticity adjustments. The P-delta effects of post-tensioning on vibration response are also discussed by Aalami in this technical note. Two examples are provided for approximation hand-calculation verification of natural frequencies of a reinforced concrete floor system.

### 4.1.5 Equipment Specific Requirements

*Vibration Sensitivity of Laboratory Bench Microscopes*

In 2002, experiments were performed to confirm current standards for the vibration sensitivity of microscopes mounted on lab benches. Microscopes of 40x, 100x, 400x, and 1000x were tested to determine when motion was perceptible during use and establish a "threshold of perception." The threshold of perception determined the conditions at which vibratory motion was first perceptible during microscope use. Taken into account during the experimental process were the effects of both horizontal and vertical motion, and the movement and properties of the lab bench. Also studied were the dynamic effects of frequency and amplitude on 1000x microscopes. They determined that the sensitivity of the microscopes tested was highest at resonant frequencies between 10 and 50 Hz. Overall, these experiments confirmed that current standards are appropriate for microscope equipment (Amick, 2007).

*Meeting the Vibration Challenges of Next-Generation Photolithography Tools*

One of the challenges in meeting the vibration requirements of sensitive equipment is the lack of an industry standard for the manner in which the requirements are reported (ESTECH).

The new photolithography tools includes step-and-scan systems, which provide a way to assess the dynamic resistance properties of structural floors and pedestals in terms of receptance spectra. The receptance spectra is the inverse of the response spectra, which is typically measured via displacement, velocity, or acceleration as discussed in Design Guide 11. In order to use step-and-scan systems, floors must meet the receptance, stiffness, and frequency spectrums of these scanner systems.

The authors of the ESTECH report discuss and assess design philosophy of floors and pedestals. Floors designed to stricter VC curves are most likely to meet the receptance criteria requirements of step-and-scan systems. A combination of receptance criteria and floor stiffness controls the design of a fabricated floor for vibration requirements. VC curves will adequately address the ambient vibration criteria of step-and-scan systems, but require further review for meeting receptance criteria. Pedestals fabricated from steel or concrete may be combined with the floor structure to satisfy a scanner's resistance requirements.

## 4.2 Design Objectives

The Health Centre is a university hospital with a range of medical and research and diagnostic equipment from microscopes to MRI machines. Excessive floor vibrations can disrupt the performance of such equipment, and therefore should be considered during the design process of hospitals and research buildings. Typical sources of vibration in the Health Centre are from walking, including both the faster walking speeds induced when transporting a patient and the slower walking speeds of hospital residents. While steel is more prone to issues with vibration, particularly due to walking excitation, concrete may also have such issues. The longer 30 ft bay spans of the Health Centre and decrease in stiffness of the redesigned gravity system suggested the need for further investigation into the floor system's dynamic behavior.

After reviewing the available literature for vibration sensitive equipment and design for vibration control, AISC Design Guide 11 was selected as the basis for evaluation of the new gravity system. Design Guide 11 VC curves and requirements for vibration sensitive equipment are the most established in industry practice, and have been applied to concrete buildings such as Cornell's

Nanotechnology Library in 2003. The Design Guide 11 chapters considered in this investigation are Chapter 4: "Design for Walking Excitation" and Chapter 6: "Design for Sensitive Equipment." ADAPT Technical Note: "Vibration Design of Concrete Floors for Serviceability" was also referenced for guidance on the level of damping, extent of cracking, and other unique concrete conditions to consider.

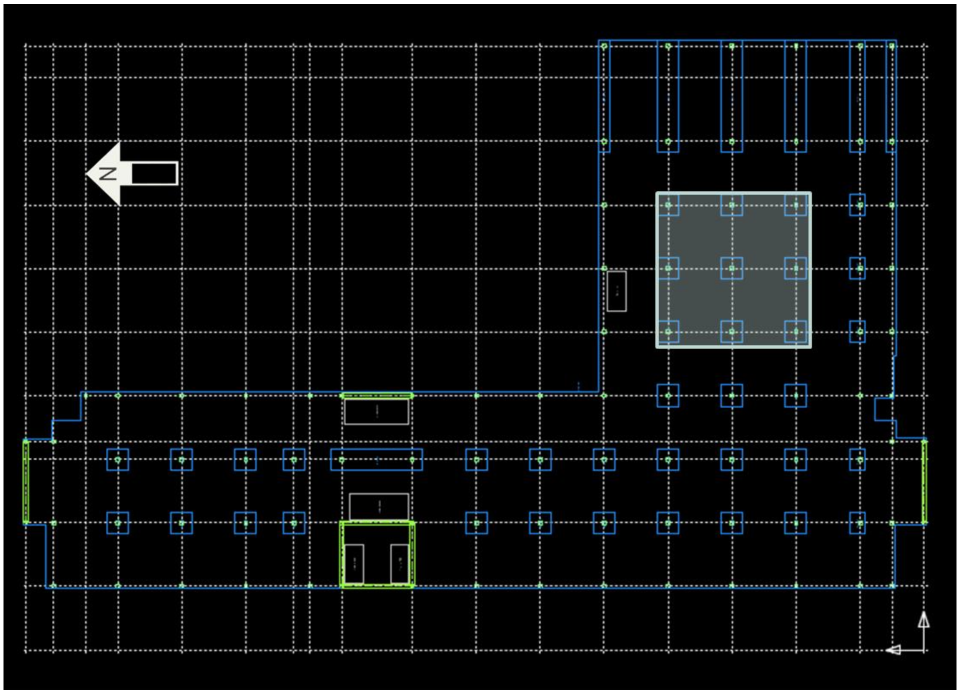
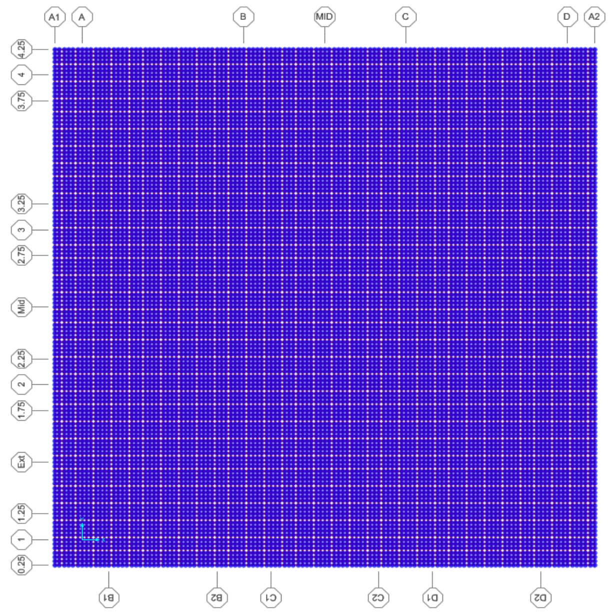



**Figure 28: Selected Bay for Vibration Analysis**

An interior bay on fifth and sixth floors, as shown in Figure 28 was selected for this vibration evaluation. These floors were large enough that it would be feasible to place operating rooms, research microscopes, and MRI equipment in the area. Design Guide 11 provides vibrational velocity limitations as the acceptance criterion for various types of sensitive equipment. Velocity limitations are often used for vibration design because they are applicable over a range of given frequencies. Table 6.1 in Design Guide 11 provides vibrational velocity criterion for different types of vibration sensitive equipment and sensitive environments, from operating rooms to MRI machines. Ultimately, two velocity limitations of 500 micro-in/sec for MRI equipment and 8,000 micro-in/sec for operating rooms were identified as possible vibration criteria for this study. Walking paces considered from Design Guide 11 Table 6.2 ranged from 100 steps/minute to 50 steps/minute to account for the range in walking speed of hospital visitors and patients.

## 4.3 Modeling Approach

After designing the gravity system in RAM Concept, a typical span of three 30'x30' interior bays on the fifth floor was modeled in SAP2000. Modeling three bays instead of one allowed for the consideration of the monolithic nature of concrete when assessing its dynamic and static behavior. Creating a separate model for vibration analysis ensured that the appropriate load cases and material properties to be considered during analysis. Additionally, the three-bay model gives a better general overview of the floor's behavior, rather than its behavior in a specific section of the building. The analysis is not location dependent if the MRI equipment location changes.



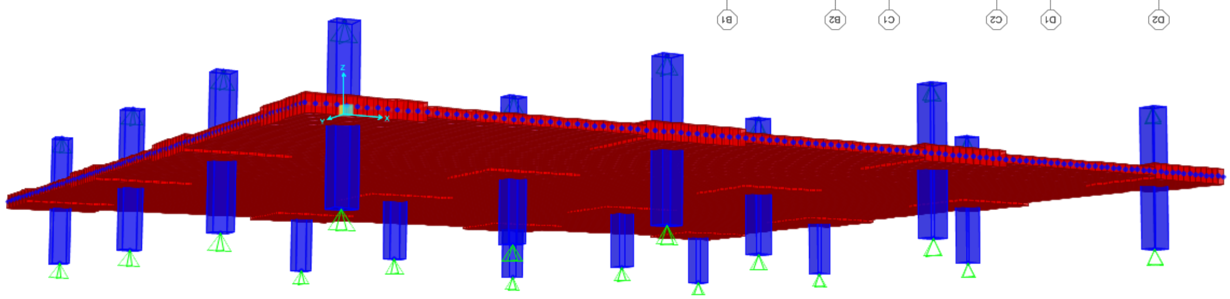



Figure 29: SAP2000 3-Bay Model

The 11" concrete slab and 19.75" drop panels were modeled as thin shell elements. Drop panels are 10 ft wide, and were extended past the columns 5 ft on the exterior edges. Columns are 24"x24", and were modeled to half of the floor height both above and below the slab. Columns at this point were pinned under the assumption that there would be zero moment at this point on the column. Figure 29 shows a two-dimensional and three-dimensional view of the SAP2000 model used for vibration analysis. Several property modifiers were used based on recommendations from ADAPT Technical Note: "Vibration Design of Concrete Floors for Serviceability." Cracked sections were considered by modifying the flexural moment of intertia to 0.25$I_g$ for slabs and 0.7$I_g$

for columns. 5000 psi concrete properties were modified to allow for 0.05 modal damping and $1.2E_c$ to better reflect dynamic loading conditions. This modified 5000 psi concrete was used for slabs, drop panels, and columns. Areas were divided into 10”x10” discrete elements to manually create the desired fine mesh, and ensure that this mesh aligned properly between area elements of varying thickness and columns.

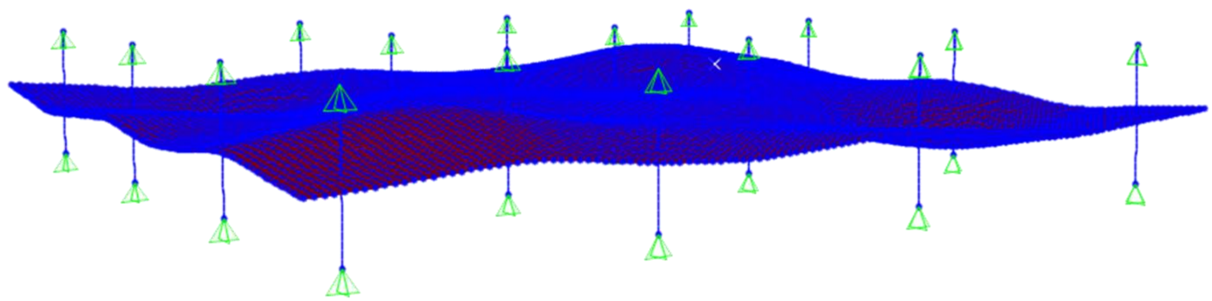


Figure 30: Excited Mode Shape from Midspan Analysis

Two static load cases were modeled: self-weight plus a 1 kip point load applied to the center of the slab, and self-weight plus a 1-kip load applied to the center of the exterior bay. This allowed for a comparison of behavior between the two load locations. Two modal load cases were created with the same respective loads. Table 20 shows the results obtained from both load cases, while Figure 29 shows the excited mode shape of the bays from the application of the 1 kip load at the center of the three bays.

Table 20: SAP2000 Analysis Output

| Load Case | Δp (in) | $F_n$ (Hz) | T (s) |
|---|---|---|---|
| Interior Bay | -0.001359 | 5.1715 | 0.19337 |
| Exterior Bay | -0.016204 | 5.1715 | 0.19337 |

### 4.4 Vibration Analysis

Using the results from the analysis model, Design Guide 11 equation 6.4b was used to determine the vibrational velocity from the two SAP2000 load cases. Values of Uv for walking

$$V = \frac{U_v \Delta_p}{f_n}$$

speeds of 100, 75, and 50 steps per minute were considered for the purposes of this study. The graphs in Figure 31 summarize the results of the vibrational velocities determined for each walking speed and loading condition considered. Hand calculations for all graph results may be found in Appendix A.1. All walking speeds for the interior 1 kip load passed vibrational velocity requirements for operating rooms, while only a walking speed of 50 steps/min satisfied any velocity requirements for the exterior 1 kip load case.

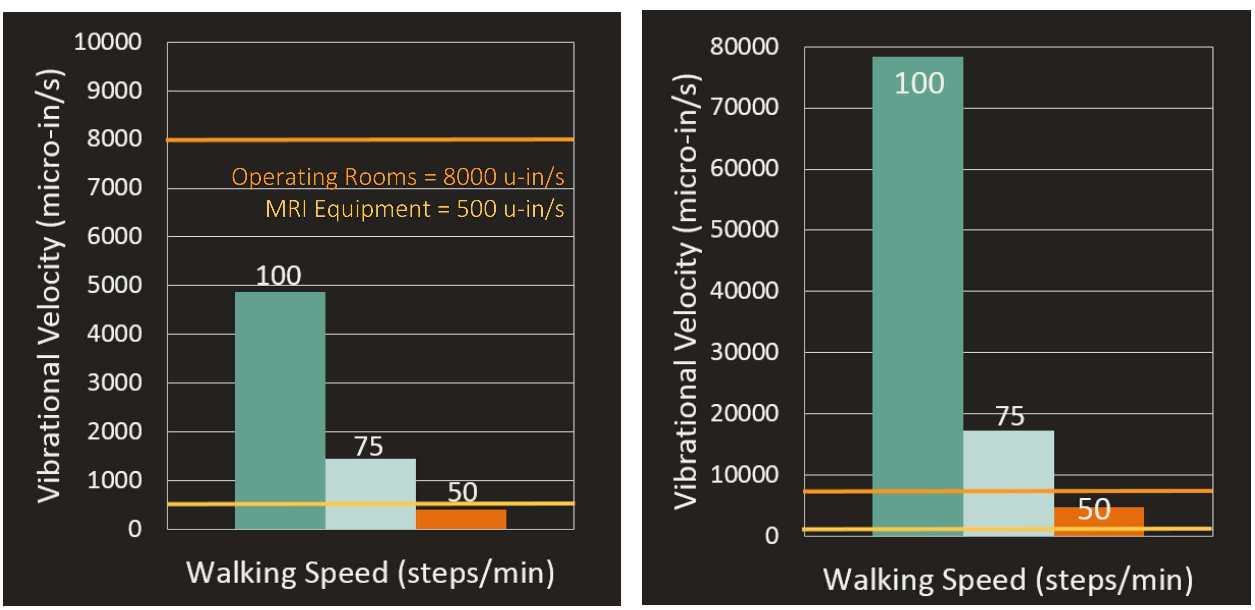



Figure 31: Interior (left) and Exterior (right) Bay Vibrational Velocities

The vibrational velocities determined from the exterior 1 kip model are significantly larger than expected for a concrete slab with deep drop panels. Therefore, the exterior bay may not be the best 1 kip load location for determining accurate and meaningful modal frequencies and deflections. Further analysis and conclusions will consider only the interior bay results.

### 4.5 Conclusions

Review of the vibration analysis results indicates that the flat slabs on floors 5 and 6 are adequately designed for operating room conditions at all walking speeds. The current architectural narrative indicates these floors are mainly for hospital beds, which may occasionally function as an operating room environment. Therefore, these floors perform well to their current function, and no changes are required for the gravity system design.

However, if the hospital wished to move MRI equipment to this floor, there may be vibration issues to consider further. Typically, performing to vibrational velocity criteria for walking speed of 75 steps/min is adequate for design in a hospital or laboratory environment. To decrease a bay’s vibrational velocity to 500 u-in/min at 75 steps/min, several changes could be made to the design. These changes include increasing the concrete strength, increasing the slab and/or drop panel thickness, and decreasing the span length. Additionally, a waffle slab – which is typically stiffer than a flat slab – could be used in areas that would house MRI equipment.

# Chapter 5

## Construction Schedule and Cost Comparison

### 5.1 Detailed Cost Analysis

Both existing and redesigned structural systems utilize cast-in-place concrete systems. These systems have similar contributors to overall cost, including formwork, concrete, shoring, and finishing. However, the removal of joists and decrease in some column sizes was expected to decrease the overall amount of concrete used for the Health Centre. The following section compares and contrasts the costs of both systems.

To estimate the cost of the redesigned flat slab and shear wall system, a detailed estimate was performed using RS Means Building Construction Cost Data 2014. Takeoffs for structural elements were tabulated from measurements taken from the RAM and Etabs models, including the square footage of formwork for walls and slabs, the linear footage of slab openings, and weight of rebar. Some takeoffs were simplified by dividing the building into lower-left (LL), lower-right (LR), upper-left (UL), and upper-right (UR) sections. The spreadsheet used for these takeoffs is located in the Appendix. The numbers obtained from building takeoffs were multiplied by the total cost multiplier to determine the overall building cost. A location factor decreased the overall building cost. The total cost of the redesigned structural system was $12,825,404.

**Table 21: Detailed Cost Estimate Comparison**

| | Existing | Redesign |
|---|---|---|
| Slab + Beams | $10,404,390 | $9,065,737 |
| Columns | $2,676,476 | $1,847,487 |
| Shear Walls | - | $1,912,180 |
| Misc. Steel | $1,273,915 | - |
| Total | **$14,354,781** | **$12,825,404** |

The cost of the existing double-skip joist and intermediate concrete moment frame structure was obtained from a detailed estimate for the building provided by McCarthy Construction. To obtain an accurate comparison between building systems, only structural elements that were changed in the redesign were considered in the existing structural cost. Elements such as the structural steel framing of the exterior bridge were not included in the existing structural system cost because they did not change with the redesigned structure. A total cost of $14,354,781 was determined from the information provided. Table 21 and Figure 32 provide a numerical and visual breakdown of the differences in cost between the redesigned and existing.

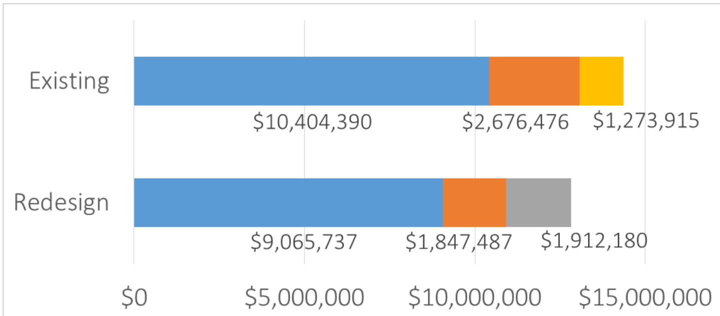



**Figure 32: Structural System Cost Comparison Graph**

A decrease in cost of $1,529,400 for the redesigned structural system was determined from the detailed cost estimates. Overall, this is a 10.6% difference in total structural system cost. However, this is a preliminary comparison between structural systems. The redesigned structural system is less refined than the original design. The cost of resolving excessive deflections across the 45 ft bays – whether an architectural or increase in concrete strength – would likely increase the redesigned system cost. If the client wishes to meet vibration requirements in some building locations, an increase in concrete strength or waffle slab may also affect the building cost.

## 5.2 Schedule Analysis

While two cast-in-place structural systems should have similar construction schedules, a schedule was determined for the redesigned structural system for a base comparison with the existing system. Only the superstructure was considered in the scope of this schedule, as the below-grade parking garage was unchanged in the redesigned system. Construction above grade for the Health Centre began in September 2014. September 1, 2014 was used as the start date for the redesigned structural system. Scheduling information was input into Microsoft Project, as shown in Figure 33, and an overall timeline was developed. To determine the duration of each task, each floor was divided into zones. Zones were typically around 7,000 total sq ft. Zone sizes were based upon the amount of square-footage that could typically be covered per day. An expanded view of the schedule with zones is in the Appendix.

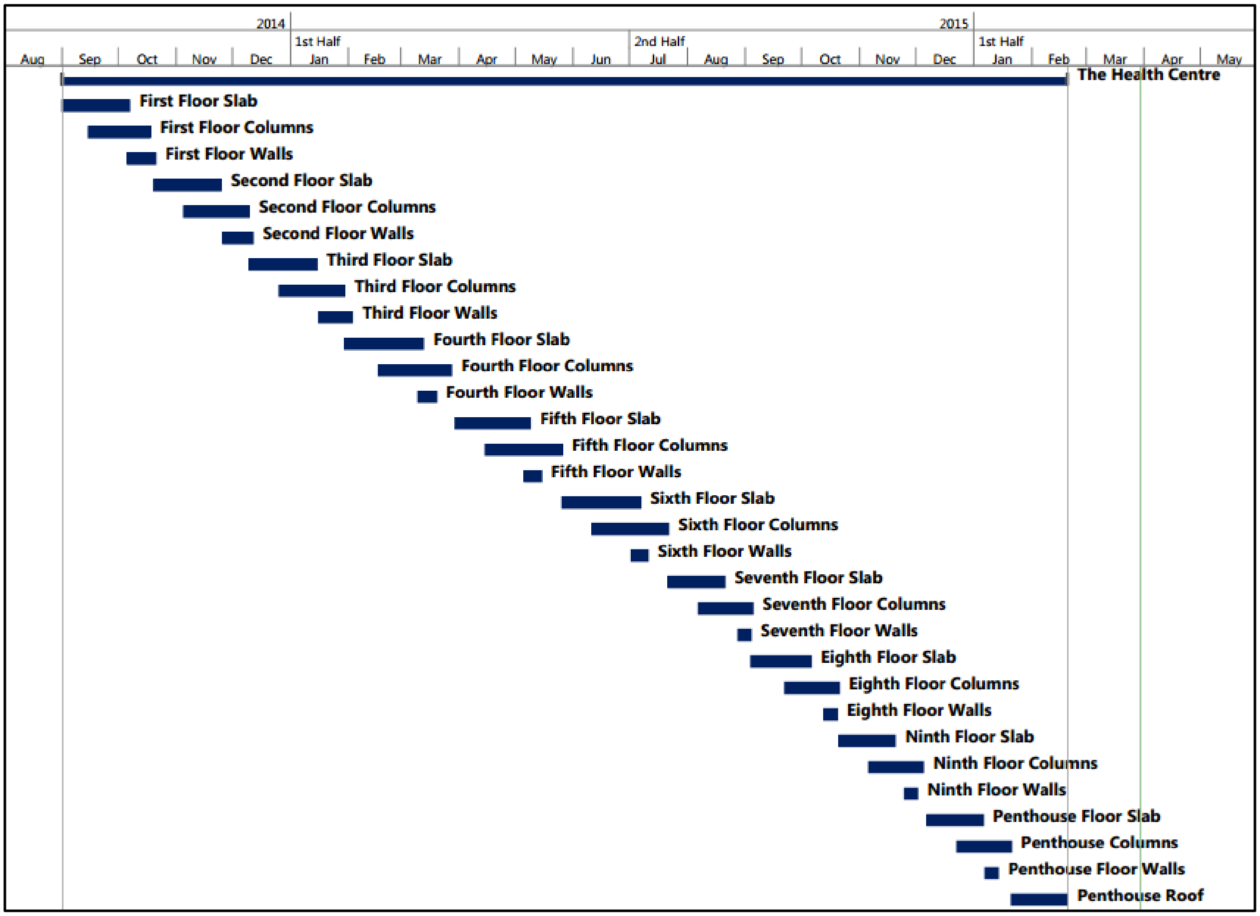



**Figure 33: Microsoft Project Construction Schedule**

For the redesigned flat slab and shear wall structural system, the total duration was expected to last 385 days, or 55 weeks. Construction spans from September 1, 2014 to February 19, 2016. This projected schedule assumes that no other tasks will occur during construction of the concrete superstructure, which is unlikely to occur during actual construction. The estimated schedule for the redesigned structural system is approximately two months longer than the actual schedule of the existing building. The double-skip joist and concrete moment frame system topped out in December, 2015. One of the reasons the existing system takes less time to construct is the lack of shear walls. Shear walls are an additional element to frame and place rebar and concrete for during the construction of the building.

# Chapter 6

## Mechanical Breadth

### 6.1 Cooling Load Study

The addition of shear walls to the Health Centre introduced large concrete masses to portions of the building's exterior. These walls will provide additional insulation for the building during the summer, fall, and spring in the southeastern US climate. As temperatures in this part of the United States do not stay below freezing for long, heating loads will likely remain unaffected by the addition of shear walls. This breadth will study the change in R values caused by the addition of shear walls to the building exterior using the Cooling Load Temperature Difference (CLTD) method. The window and aluminum panel system will be the exterior wall section considered in this analysis, as shown in Figure 34. Solar heat gain with and without the shear wall and the effects of the cooling load on the current HVAC system capacity for this zone will be considered in this study.

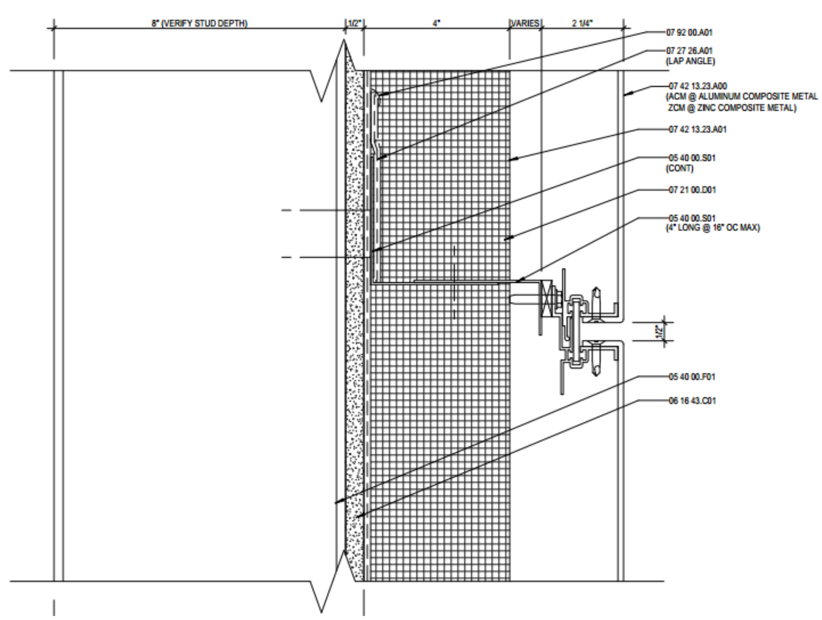



**Figure 34: Metal Panel Wall Section**

The CLTD method uses the equation q= U x A x CLTD, where q is solar heat gain, U is the heat transfer coefficient, A is the area, and CLTD is the cooling load temperature difference. The heat transfer coefficient U is determined for each material by taking the inverse of its R value. R values in Table 22 are from Table E.1 of Mechanical and Electrical Equipment for Buildings (12th edition) for the per-inch-thickness specified in the table. The aluminum curtain wall assembly R value and U factor were obtained from specifications provided by SmithGroupJJR.

**Table 22: Building Material U-Values**

| Material | Thickness (in) | R- Value |
|---|---|---|
| Aluminum Curtain Wall | 2.25 | 2.22 |
| Rigid Insulation | 4 | 18.2 |
| Gypsum Wall Sheathing | 0.25 | 0.32 |
| | Total R-Value | 20.74 |
| | U-Value ($1/R_{total}$) | 0.048216 |

| Material | Thickness (in) | R- Value |
|---|---|---|
| Aluminum Curtain Wall | 2.25 | 2.22 |
| Rigid Insulation | 4 | 18.2 |
| Gypsum Wall Sheathing | 0.25 | 0.32 |
| Shear Wall | 20 | 2 |
| | Total R-Value | 22.74 |
| | U-Value ($1/R_{total}$) | 0.043975 |

For a building assembly without shear walls, a total U value of 0.043975 was obtained. With the addition of shear walls, the total U value for the building envelope is 0.048216. The square-footage considered for calculations will be 60 ft, the width of one exterior wall, multiplied by 15 ft the height of one of the upper floors.

CLTD values were determined from 1997 ASHRAE Fundamentals Table 32. The principle wall material for shear walls was considered to be 200 mm high density filled concrete block (C18) for best comparison to the thickness and properties of shear walls. The principle wall material for the non-shear wall system was considered concrete columns for this study. This used 100 mm high density concrete (C5). While the systems taken from ASHRAE Fundamentals tables are not direct comparisons to the systems in the building, they will be used as a preliminary study to determine if a further, more detailed, study would be required for the building. The aluminum panel system is steel, or light-weight, siding with an R value between 0.44 and 0.53. Therefore, the wall types from Table 33C were determined to be 4 and 3 for the shear wall and non-shear wall systems, respectively. The south facing shear wall located at 40 degrees north latitude was considered for this study.

ASHRAE Fundamentals Table 23 shows the results of the cooling load analysis for a 24 hour period during the month of July. Each hour has a different CLTD value that varies with the wall type. Overall, the daily total cooling load is 9,301 BTU/hr for a building envelope with shear walls and 10,111 BTU/hr for a building envelope without shear walls. The addition of shear walls decreases the overall cooling load by 810 BTU/hr for the zone on this floor.

**Table 23: Cooling Load Calculations**

| Hour | Area ($ft^2$) | U | | CLTD | | q (BTU/hr) | |
|---|---|---|---|---|---|---|---|
| | | No SW | SW | No SW | SW | No SW | SW |
| 1 | 900 | 0.048216 | 0.043975 | 4 | 6 | 173.5776 | 237.467 |
| 2 | 900 | 0.048216 | 0.043975 | 3 | 4 | 130.1832 | 158.3113 |
| 3 | 900 | 0.048216 | 0.043975 | 2 | 3 | 86.78881 | 118.7335 |
| 4 | 900 | 0.048216 | 0.043975 | 1 | 2 | 43.39441 | 79.15567 |
| 5 | 900 | 0.048216 | 0.043975 | 1 | 1 | 43.39441 | 39.57784 |
| 6 | 900 | 0.048216 | 0.043975 | 0 | 1 | 0 | 39.57784 |
| 7 | 900 | 0.048216 | 0.043975 | 0 | 0 | 0 | 0 |
| 8 | 900 | 0.048216 | 0.043975 | 1 | 1 | 43.39441 | 39.57784 |
| 9 | 900 | 0.048216 | 0.043975 | 2 | 0 | 86.78881 | 0 |
| 10 | 900 | 0.048216 | 0.043975 | 5 | 1 | 216.972 | 39.57784 |
| 11 | 900 | 0.048216 | 0.043975 | 9 | 3 | 390.5497 | 118.7335 |
| 12 | 900 | 0.048216 | 0.043975 | 13 | 7 | 564.1273 | 277.0449 |
| 13 | 900 | 0.048216 | 0.043975 | 17 | 11 | 737.7049 | 435.3562 |
| 14 | 900 | 0.048216 | 0.043975 | 21 | 16 | 911.2825 | 633.2454 |
| 15 | 900 | 0.048216 | 0.043975 | 23 | 19 | 998.0714 | 751.9789 |
| 16 | 900 | 0.048216 | 0.043975 | 23 | 23 | 998.0714 | 910.2902 |
| 17 | 900 | 0.048216 | 0.043975 | 22 | 24 | 954.677 | 949.8681 |
| 18 | 900 | 0.048216 | 0.043975 | 20 | 23 | 867.8881 | 910.2902 |
| 19 | 900 | 0.048216 | 0.043975 | 17 | 22 | 737.7049 | 870.7124 |
| 20 | 900 | 0.048216 | 0.043975 | 14 | 19 | 607.5217 | 751.9789 |
| 21 | 900 | 0.048216 | 0.043975 | 12 | 17 | 520.7329 | 672.8232 |
| 22 | 900 | 0.048216 | 0.043975 | 9 | 13 | 390.5497 | 514.5119 |
| 23 | 900 | 0.048216 | 0.043975 | 8 | 11 | 347.1553 | 435.3562 |
| 24 | 900 | 0.048216 | 0.043975 | 6 | 8 | 260.3664 | 316.6227 |
| | | | | | Total | 10110.9 | 9300.792 |

## 6.2 Conclusions

The CLTD analysis indicates that the addition of shear walls will decrease the overall cooling load on the mechanical system in the region of the building by the 60 ft south-facing shear wall by 810 BTU/hr over the course of a day. Over the course of a year, this is approximately 295,650 BTU/hr total. In this instance, the shear walls are not completely exposed by the aluminum panel system and provide additional insulation for the building. This change is not large enough to significantly impact building systems, as the client requested that cooling systems provide additional capacity for potential expansion. Three 1250 ton centrifugal chillers, two 1250 ton cooling tower cells, and a 400 ton heat pump chiller serve the building. Dedicated air-handling units for individual patient and operating areas will also remain unaffected.

# Appendix A

# Structural References

# A.1 Slab References

Flat Slab w/ drop panels + w/o spandrel beams

1) Min thickness of slab (table 8.3.1.1)

Exterior Panels: $\ell_n/33 = \frac{(30)(12)-20}{33} = 10.3''$

Interior Panels: $\ell_n/36 = \frac{(30)(12)-20}{36} = 9.44''$

20" column, $f_y = 60{,}000$ psi

2) Direct Design Method (ACI 318-14, 8.10.2)

- 3 continuous spans ✓
- Successive spans not different by > 1/3 ✓
- $\ell_1/\ell_2 = 1 < 2$ ✓
- Max offset = 8'-4"

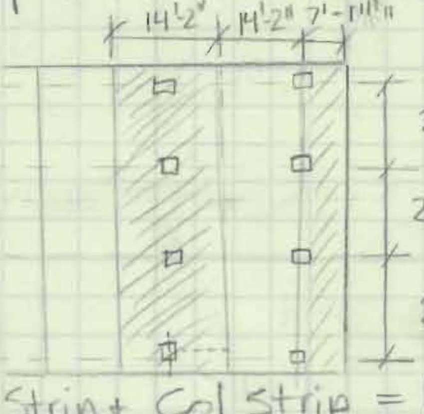
$0.2(30\text{ ft}) = 6' < 8'-4''$

∴ Can't use direct design

3) Equivalent Frame Method

⇒ Col strip + Middle Strip dim. for 20" col floors

$\ell_2 = \ell_1 = (30)(12) - 20 = 340'' = 28.3'$




$\ell_2/4 = 28.3'/4 = 7'-11''$

$\ell_2/2 = 28.3'/2 = 14'-2''$

Middle Strip + Col Strip = 14'-2"

$f'_c = 5000$ psi for slabs + columns

Dead: 23 psf

Self Wt: $150 \times (11/12) = 137.5$ psf

Live: 100 psf

↳ Reduction: Exterior: $L_0 \times \max\begin{cases} 0.4 \\ 0.25 + \frac{15}{\sqrt{1(30\times15)}} = 0.957 \end{cases}$ $L = 95.7$ psf

Interior: $L_0 \times \max\begin{cases} 0.4 \\ 0.25 + \frac{15}{\sqrt{1(30\times30)}} = 0.75 \end{cases}$ $L = 75$ psf

$W_{ext} = 1.2(23 + 137.5\text{ psf}) + 1.6(95.7) = 346$ psf

$W_{int} = 1.2(23 + 137.5\text{ psf}) + 1.6(75) = 313$ psf

4) One-Way Shear

$V_u = (346\ psf)(14.167\ ft)(30\ ft) = 147\ K$

$\phi V_c = \phi 2 \lambda \sqrt{f'_c}\, b_w d = 0.75(2)\sqrt{5000}(30' \times 12^{in/ft})(9.5\ in)$
$= 363\ K$

5) Two-Way (Punching) Shear

- Critical location @ $d/2 = 19.25 - 1.5/2 = 8.875$ in for drop panels
- Critical perim $b_o = 4(20 + 2(8.875)) = 151$ in

$\phi V_c = \phi 4 \lambda \sqrt{f'_c}\, b_o d = 0.75(4)\sqrt{5000}(151)(19.25 - 1.5)$ 17.25
$= 569\ K$

$V_u = (0.376\ ksf)\left[30 \times 30 - \left(\frac{20 + 2(8.875)}{12^{in/ft}}\right)^2\right] = 334\ K < 569\ k$

$\therefore$ 19.25" drop panels Ok

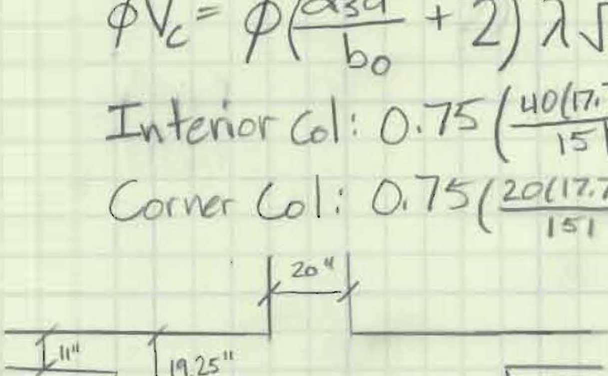
Direct Design Prelim Sizing

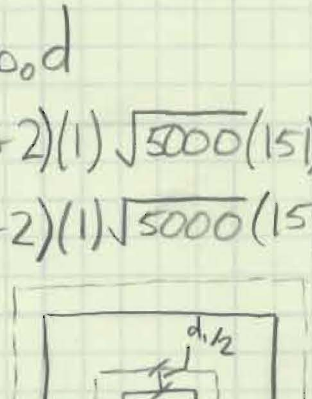
$\frac{b_o}{d} = \frac{151}{17.75} = 8.51 > 2$ $\therefore$ ACI eqn 11-32 applies

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

Interior Col: $0.75\left(\frac{40(17.75)}{151} + 2\right)(1)\sqrt{5000}(151)(17.75) = 953\ k$

Corner Col: $0.75\left(\frac{20(17.75)}{151} + 2\right)(1)\sqrt{5000}(151)(17.75) = 618\ k$ Ok




6) Slab Moments + Prelim Reinforcing

$M_o = \frac{w_u l_2 l_n^2}{8} = \frac{(346\ psf)(14.167\ ft)(28.3\ ft)^2}{8} = 491\ k\text{-}ft$

ACI 13.6.3.3 Slab w/o Interior + Edge Beams

Interior Span: $-M_u = -0.70M_o = -0.70(491\text{ k-ft}) = -344\text{ k-ft}$
$+M_u = +0.52M_o = 0.52(491\text{ k-ft}) = 255\text{ k-ft}$

Exterior Span: $-M_u = -0.26M_o = -0.26(491\text{ k-ft}) = -128\text{ k-ft}$

Interior Neg. Moments:
Col. Strip: $0.75(-344\text{ k-ft}) = -258\text{ k-ft}$
Mid. Strip: $0.25(-344\text{ k-ft}) = -86\text{ k-ft}$

Interior Pos. Moments:
Col. Strip: $0.6(255\text{ k-ft}) = 153\text{ k-ft}$
Mid. Strip: $0.4(255\text{ k-ft}) = 102\text{ k-ft}$

$A_{s,min}$ Requirements

$A_{s,min} = 0.0018bh = 0.0018(12'')(11'') = 0.2376\text{ in}^2/\text{ft}$

$A_{s,req} = \dfrac{M_u \times (12\text{ in/ft})}{\phi f_y' jd}$ $\phi = 0.9$
$f_y' = 60\text{ ksi}$
$jd = 0.95d = 0.95(9.5) = 9.025$

| Location | Mu (ft-k) | As,req | Use | Spacing | As,actual |
|---|---|---|---|---|---|
| -col | 258k-ft | 6.35 in² | 12 #7's | 14'' | |
| +col | 153k-ft | 3.77 in² | 14 # 5's | 11.5' | |
| -mid | -86k-ft | 2.12 in² | 8 #7's | 21'' & 18'' min | |
| +mid | 102k-ft | 2.51 in² | 10#5's | 16.5'' ~18'' | |

Verify steel in tension... Pos col strip check

$\varepsilon_s = \dfrac{d-c}{c}\varepsilon_{cu} = (0.003)\dfrac{10.25-0.7975}{0.7975} = \boxed{0.00355}$  $d = 11-0.75 = 10.25$

$a = \dfrac{14(0.31)(60)}{0.85(5)(96)} = 0.638$  let b = column transfer zone
$1.5(24)(2) + 24 = 96''$
↑ w. column

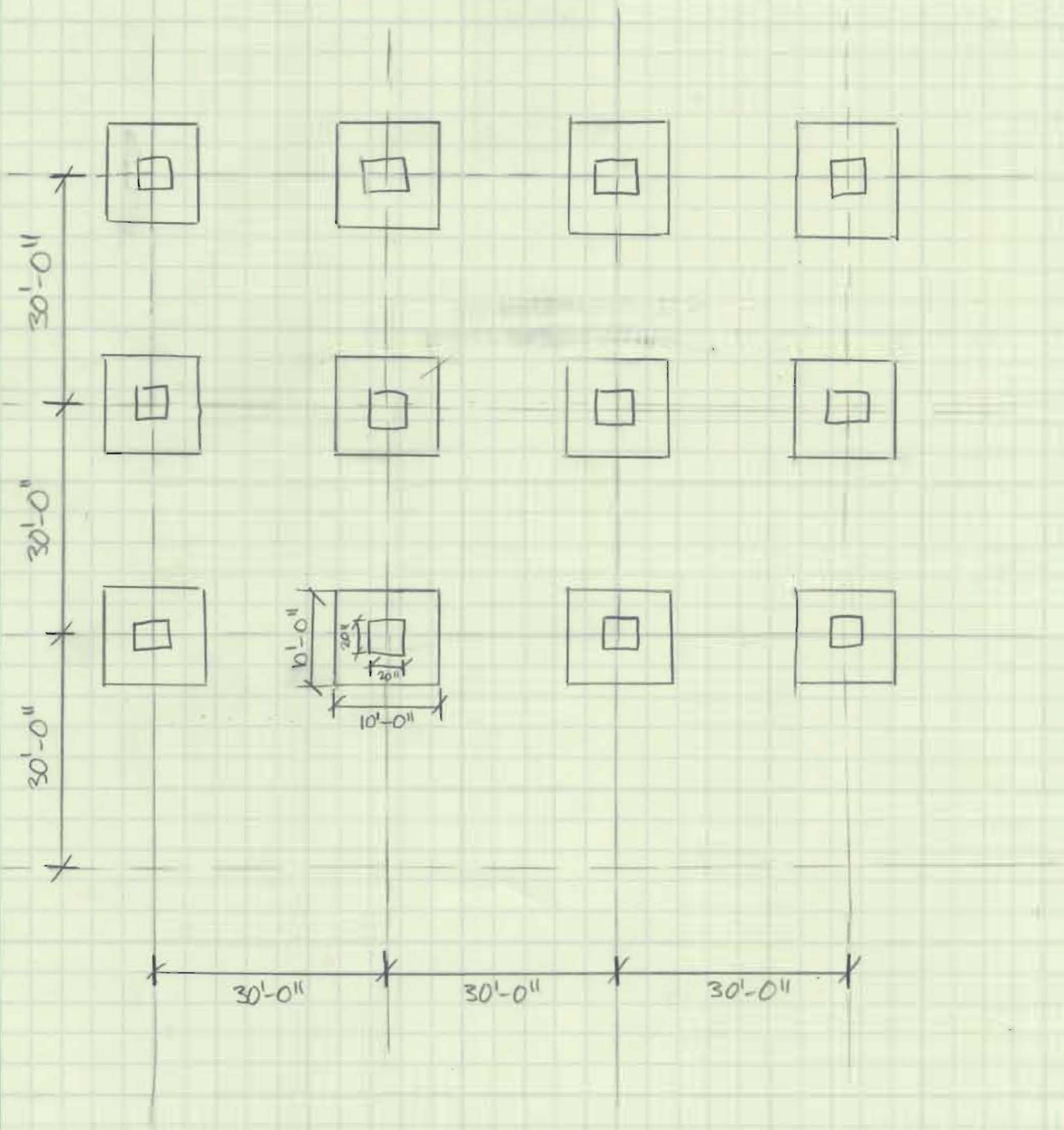
$c = \dfrac{a}{B_1} = \dfrac{0.638}{0.8} = 0.7975'$

$\varepsilon_s = 0.00355 > 0.00205$ OK

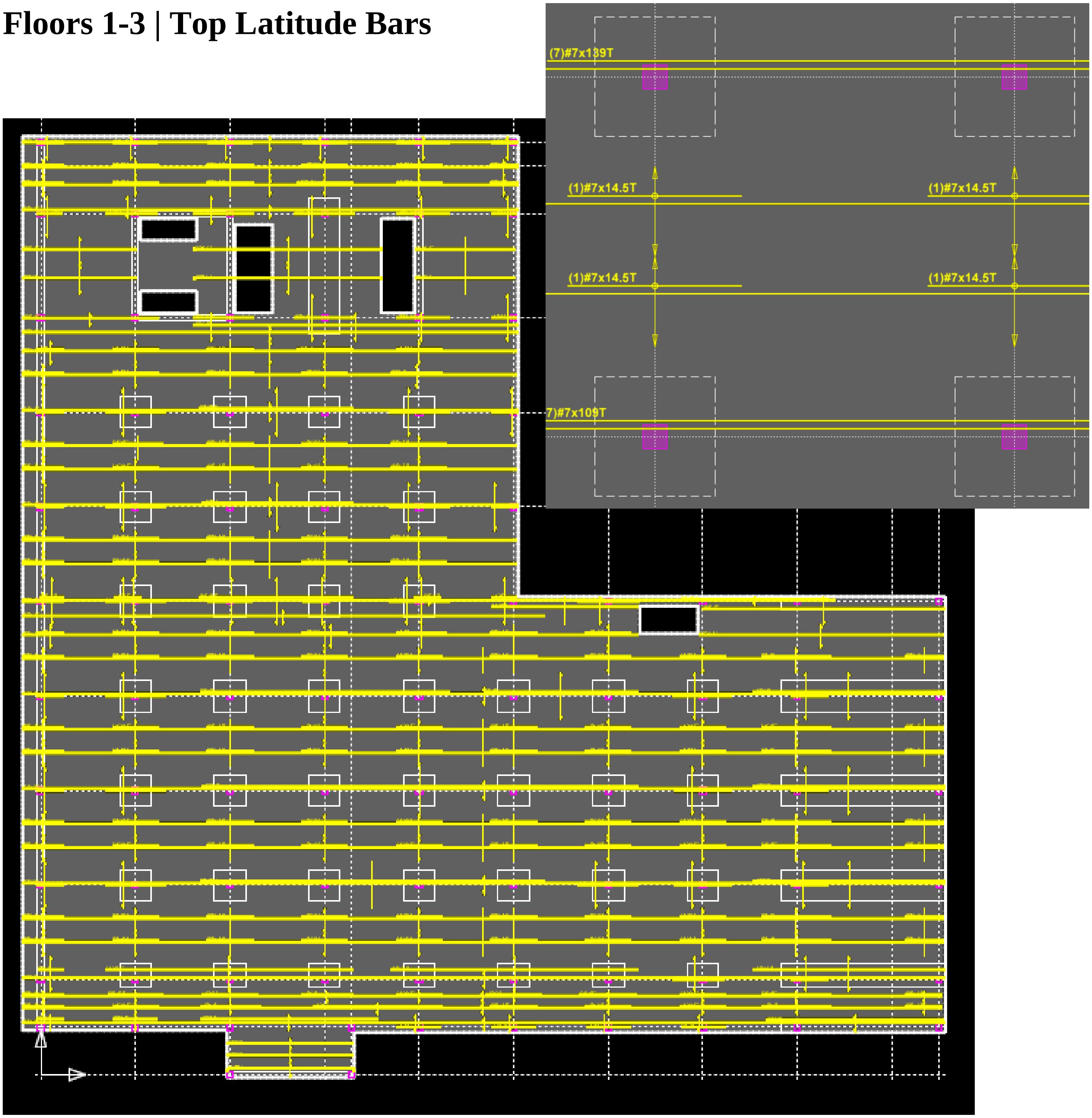
# Gravity System Results

Slab Depth = 11"

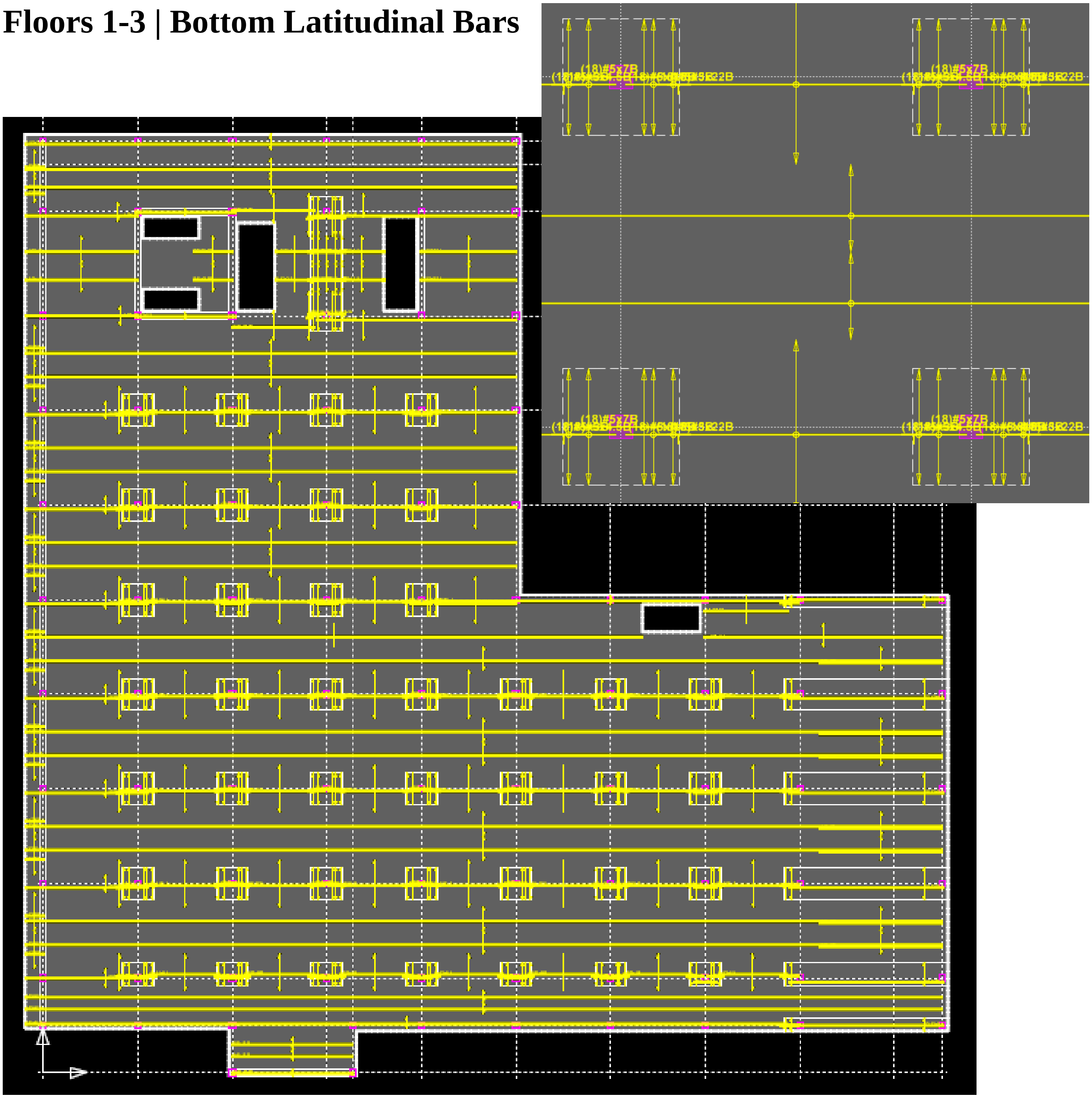
Drop Panel Depth = 19 1/4" (6 1/2" + 8 1/4")



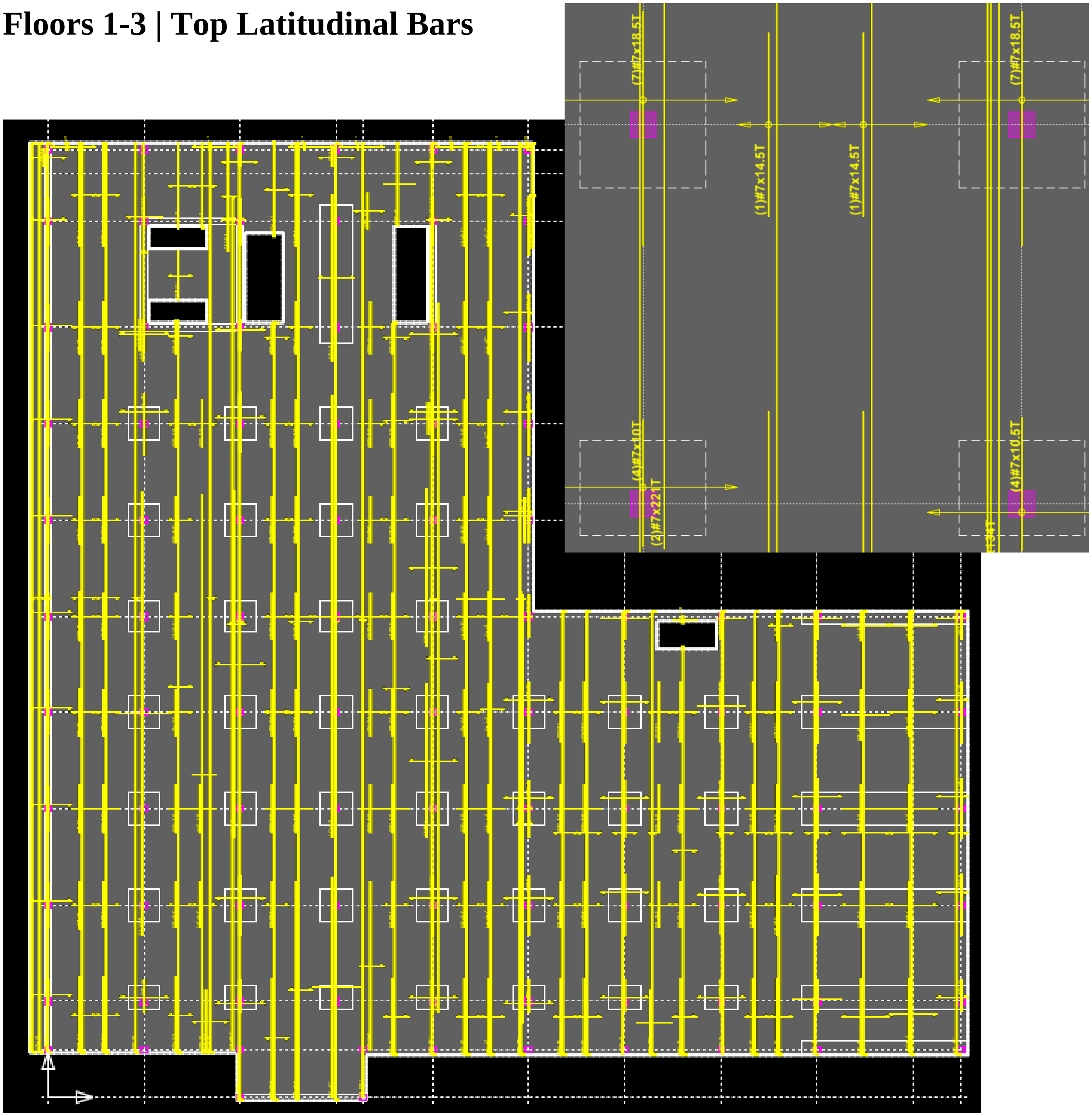
# Floors 1-3 | Top Latitude Bars



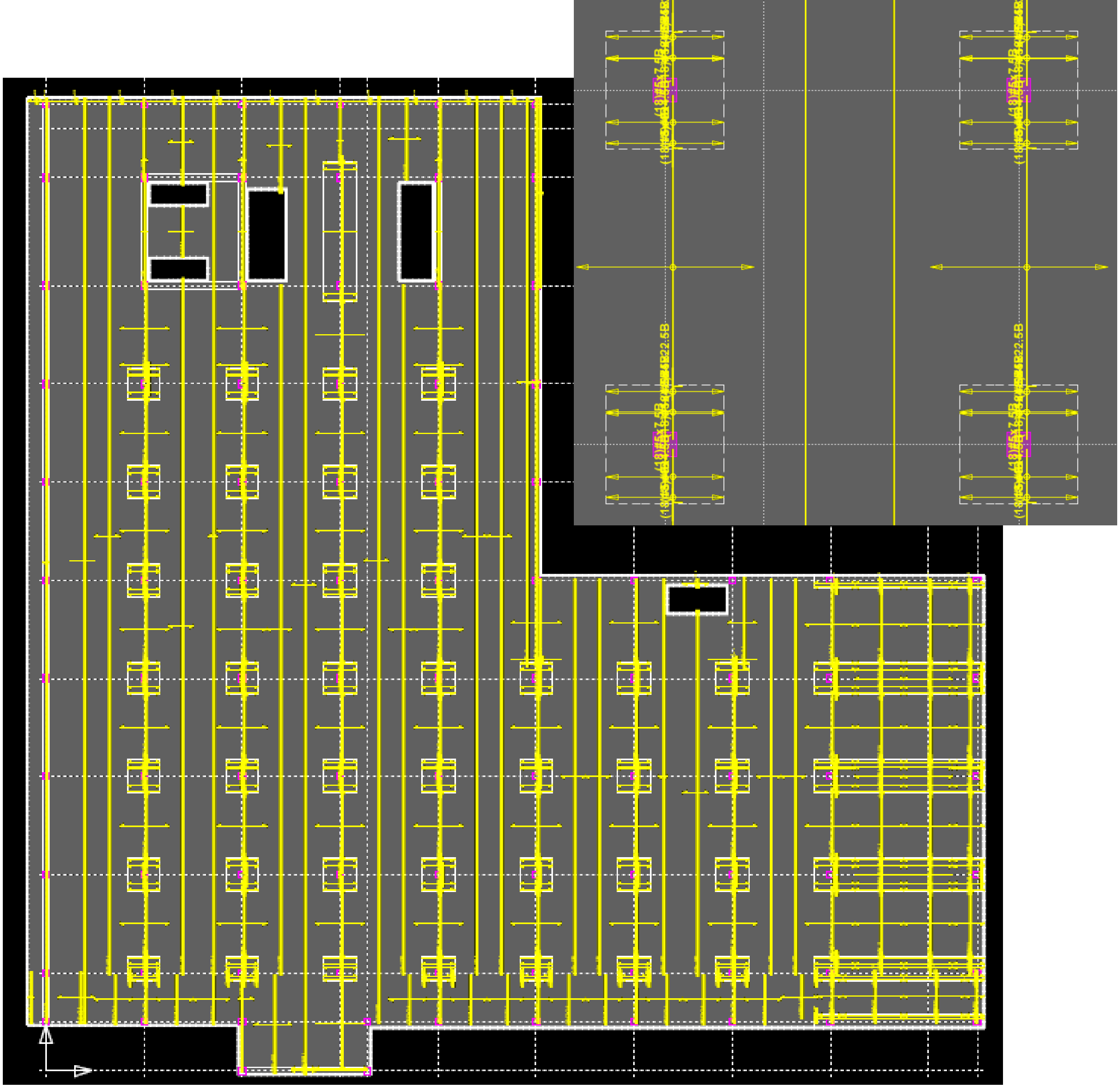
# Floors 1-3 | Bottom Latitudinal Bars



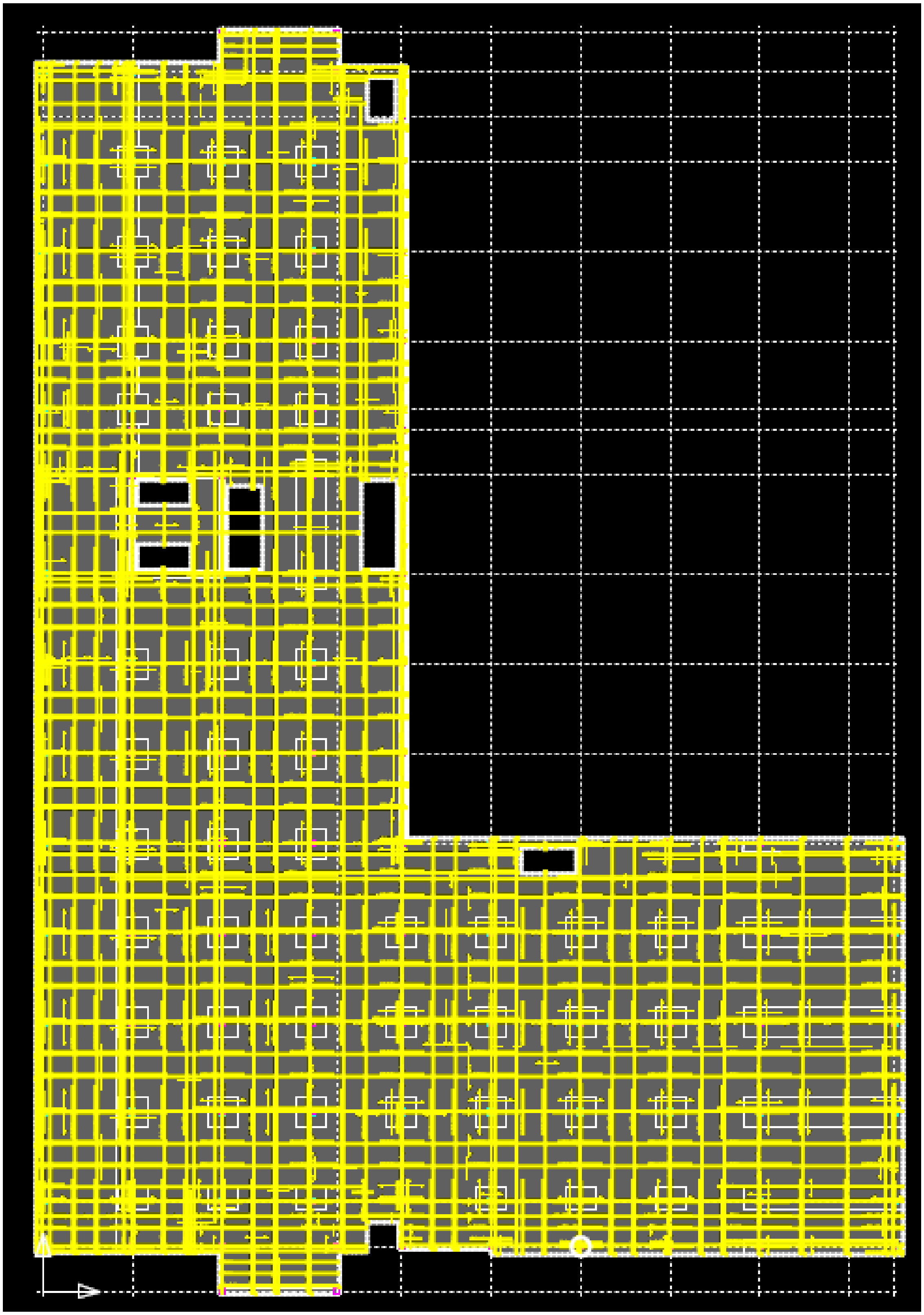
# Floors 1-3 | Top Latitudinal Bars



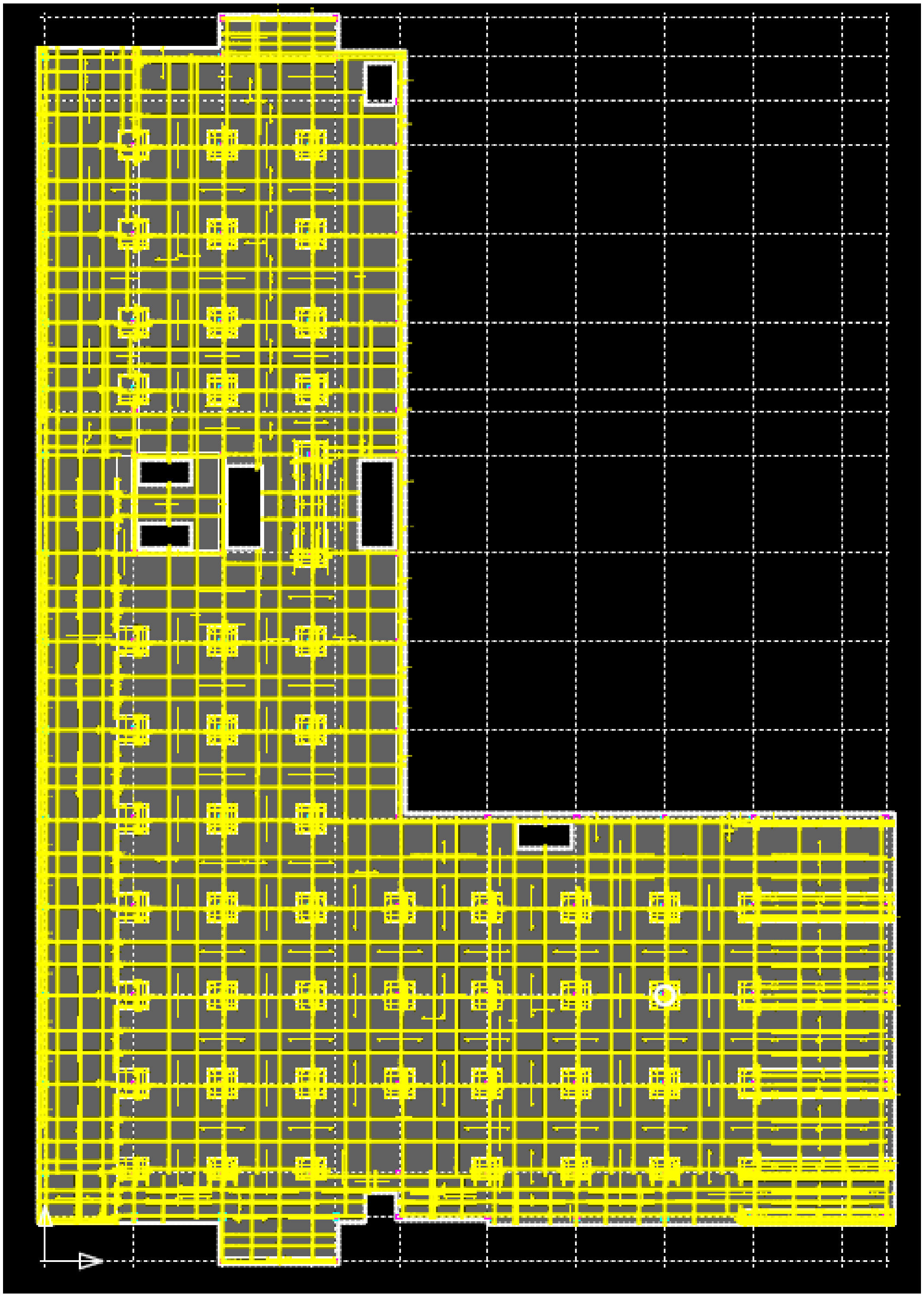
# Floors 1-3 | Bottom Latitudinal Bars



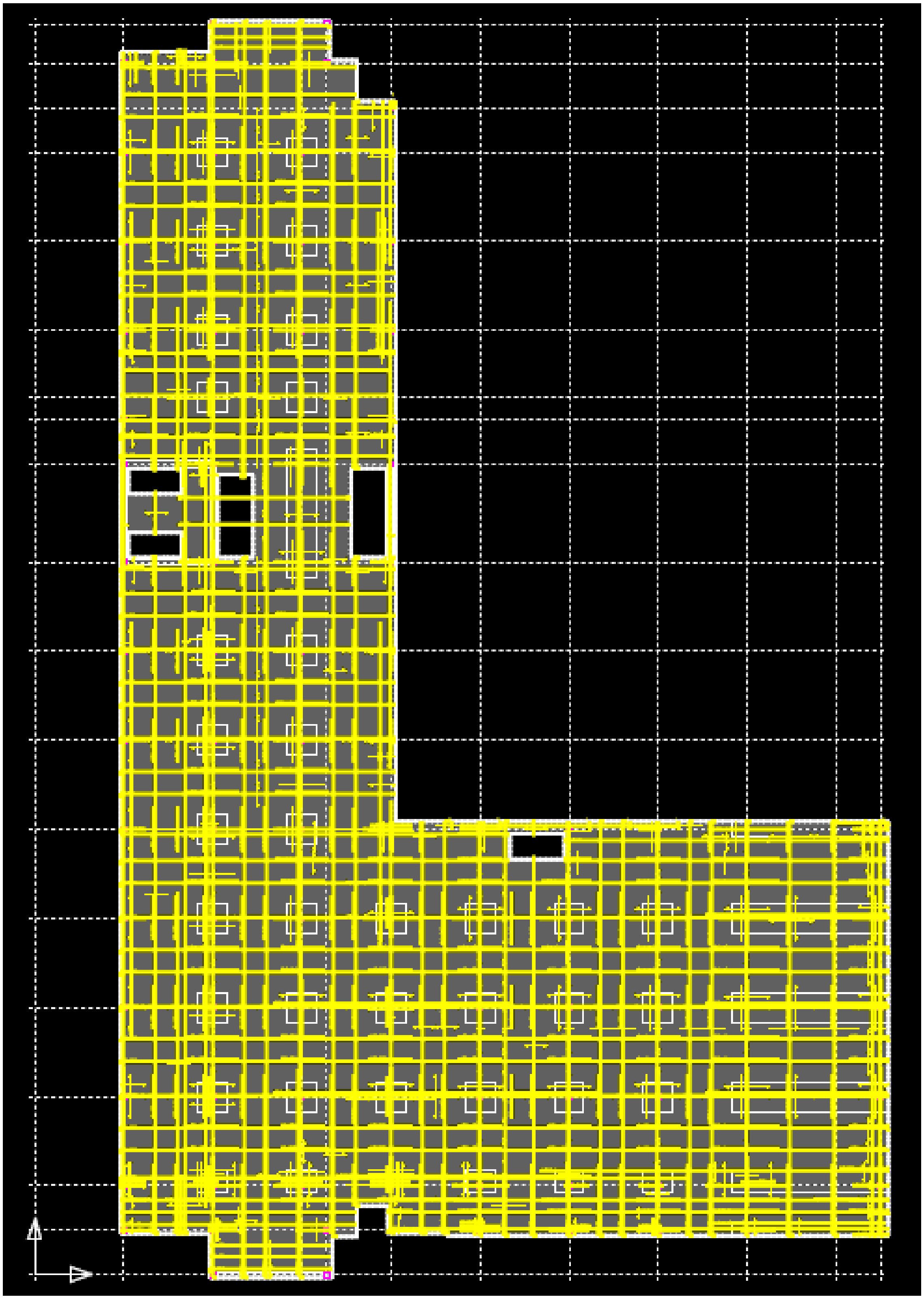
## Floor 4 | Top Bars Plan



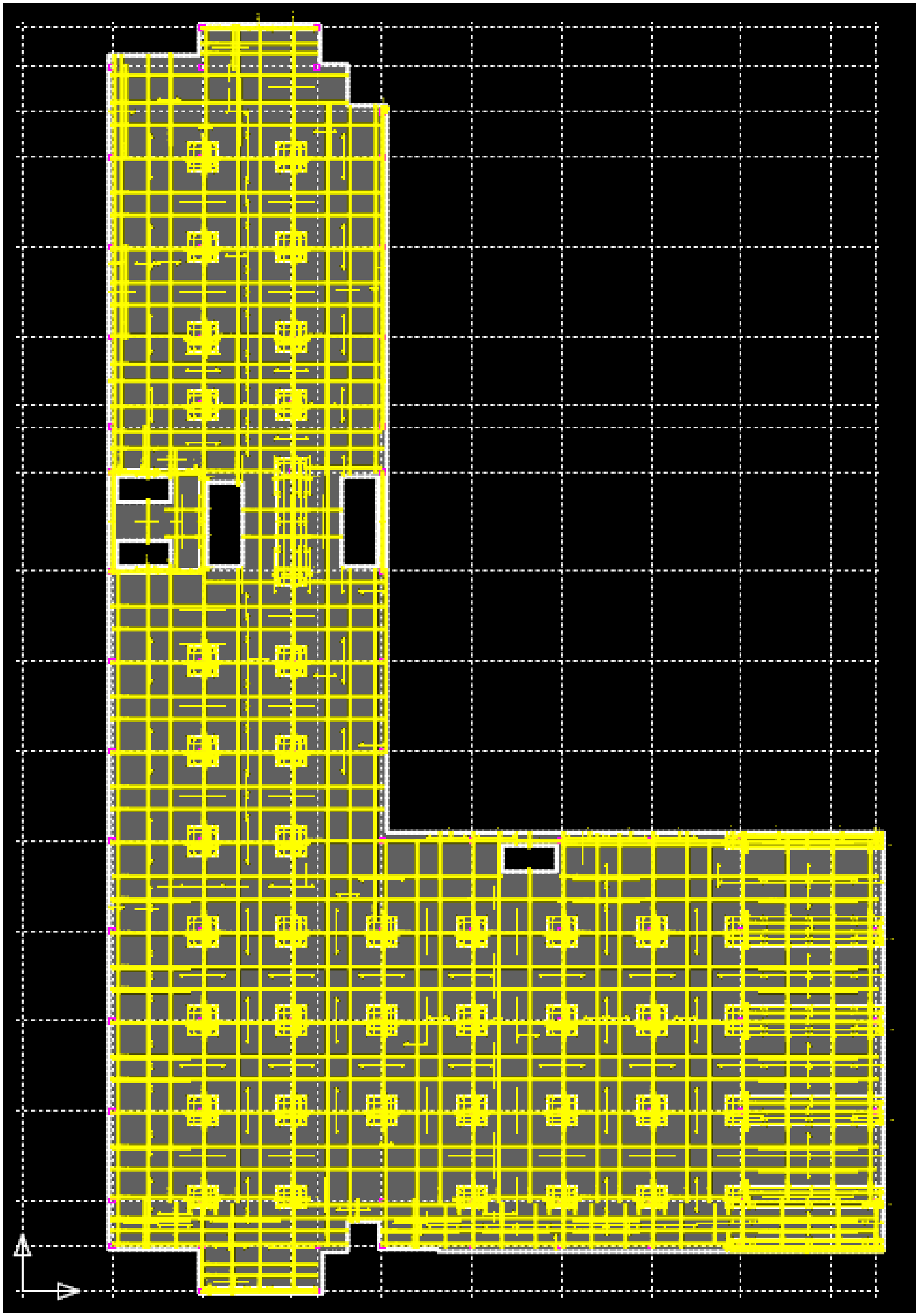
# Floor 4 | Bottom Bars Plan



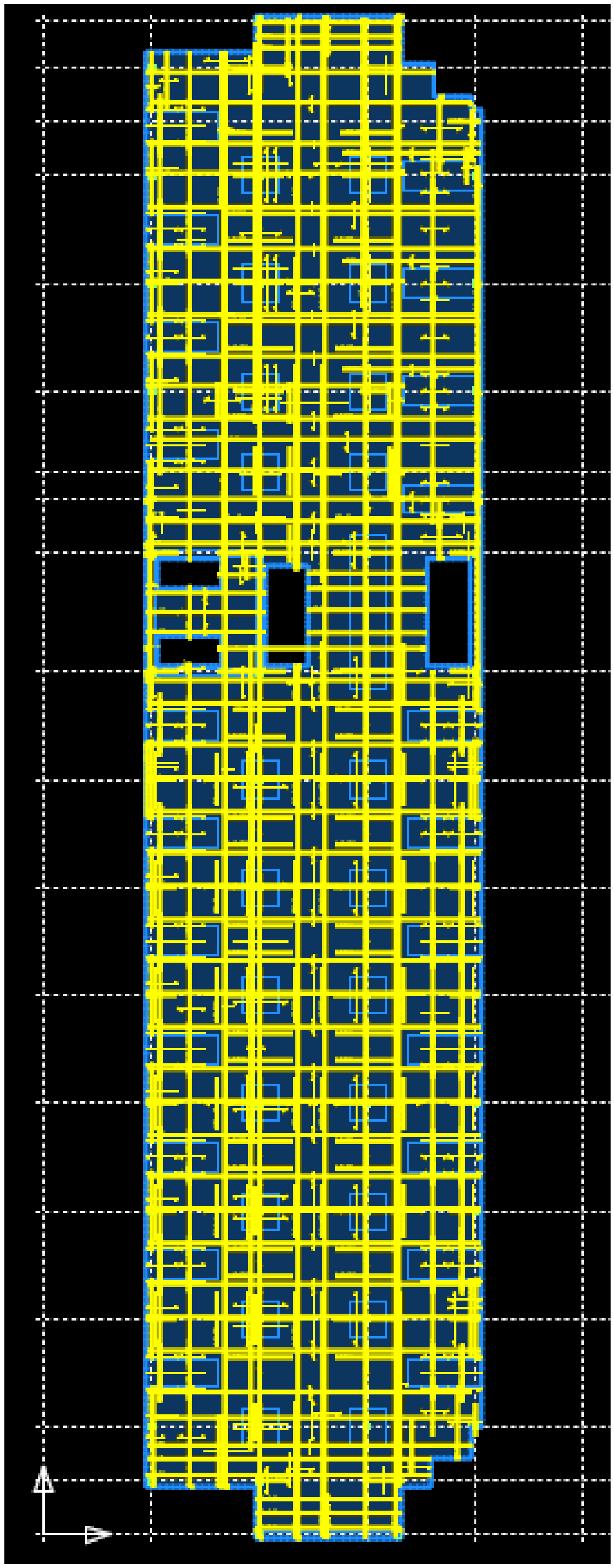
## Floors 5-6 | Top Bars Plan



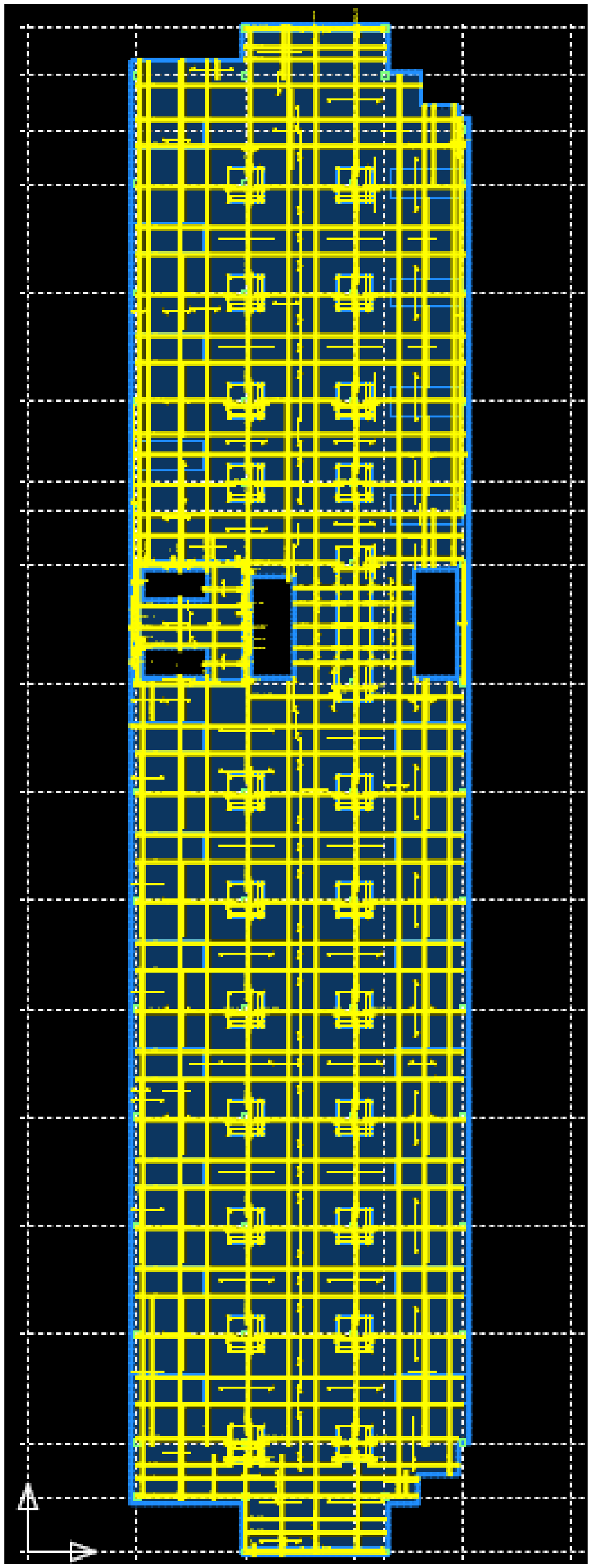
## Floors 5-6 | Bottom Bars Plan



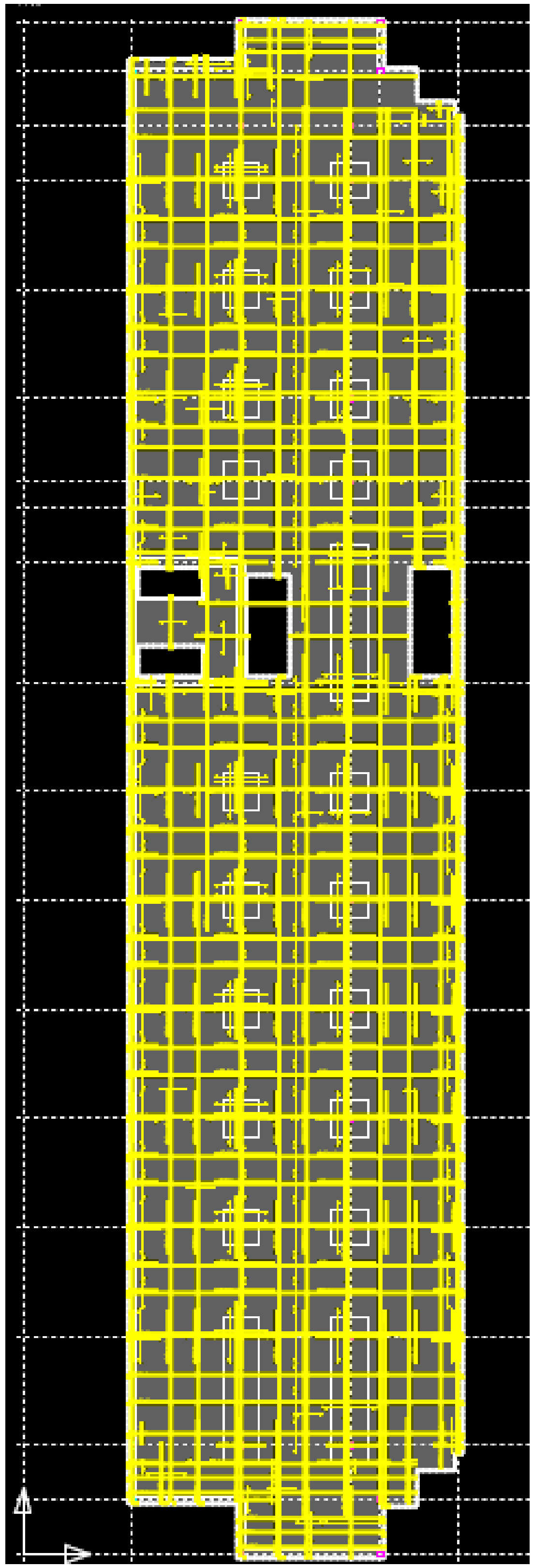
# Floor 7-9 | Top Bars Plan



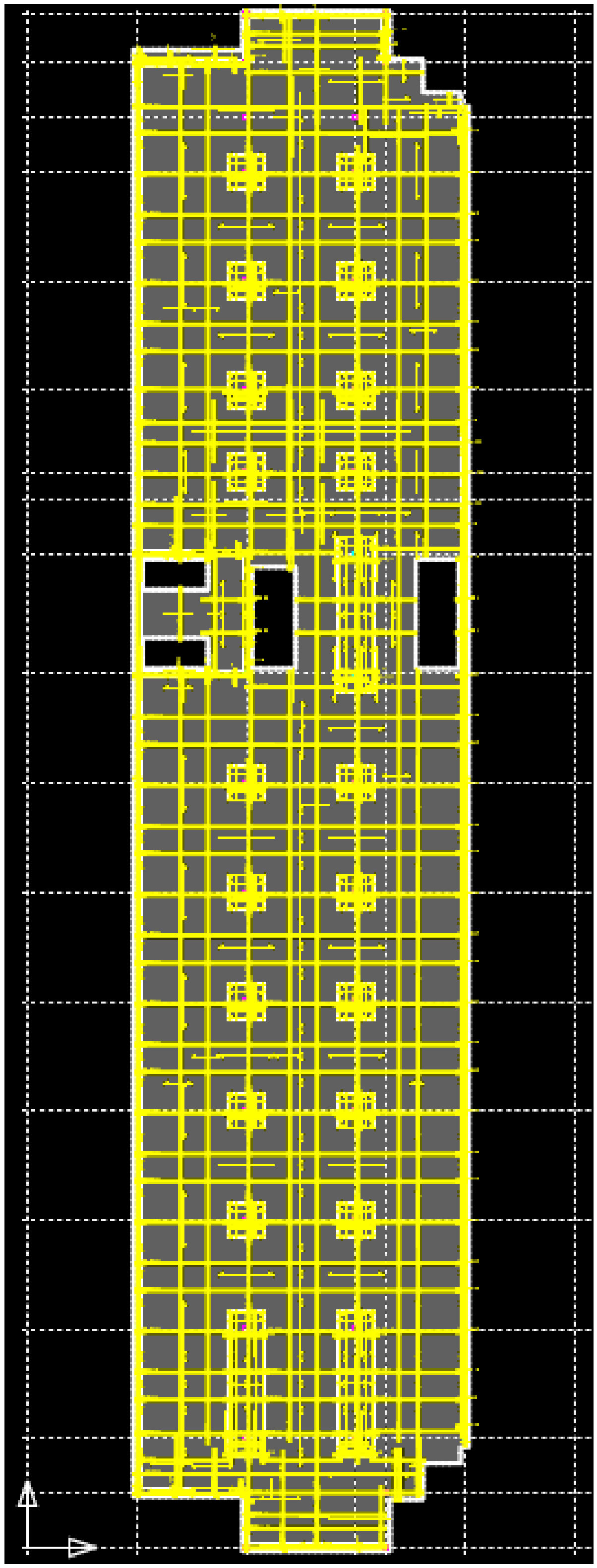
## Floor 7-9 | Bottom Bars Plan



## Penthouse | Top Bars Plan

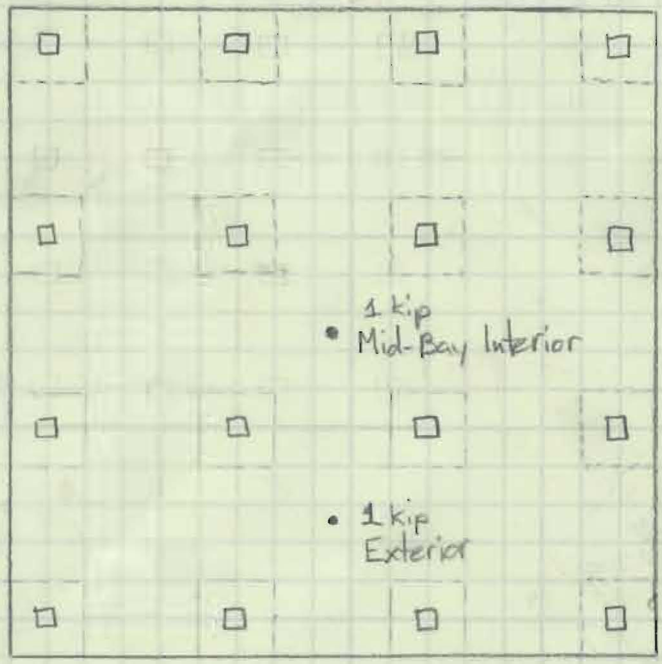


## Penthouse | Bottom Bars Plan



# Vibrational Velocity

Design Guide 11: $f_n > 5.0$ $\quad V = \frac{U_v \Delta_p}{f_n}$ (6.4b)




① Mid-Bay Interior: (joint 62

$\Delta_p = 0.001359$ in/1000 lb $= 1.359\ \mu in/lb$

$F_n = 5.1715$ Hz

$T = 0.193367 s$

Considering all walking paces... (Table 6.2)

| Steps/min | $U_v$ (lb-Hz) |
|---|---|
| 100 | 25,000 |
| 75 | 5,500 |
| 50 | 1,500 |

$V_{100\ step/min} = \frac{(25{,}000)(1.359\ \mu in/lb)}{5.1715} = 4879.5\ \mu in/s$

$V_{75\ step/min} = \frac{(5{,}500)(1.359\ \mu in/lb)}{5.1715} = 1445\ \mu in/s$

$V_{50\ step/min} = \frac{(1{,}500)(1.359\ \mu in/lb)}{5.1715} = 396.5\ \mu in/s$

② Exterior: $\Delta_p = 0.016204$ in/1000 lb $= 16.204\ \mu in/lb$

$F_n = 5.1715$ Hz

$T = 0.19337$ s

$V_{100\ step/min} = \frac{(25000)(16.204\ \mu in/lb)}{5.1715} = 78{,}333.2\ \mu in/s$

$V_{75\ step/min} = \frac{(5{,}500)(16.204\ \mu in/lb)}{5.1715} = 17{,}233.3\ \mu in/s$

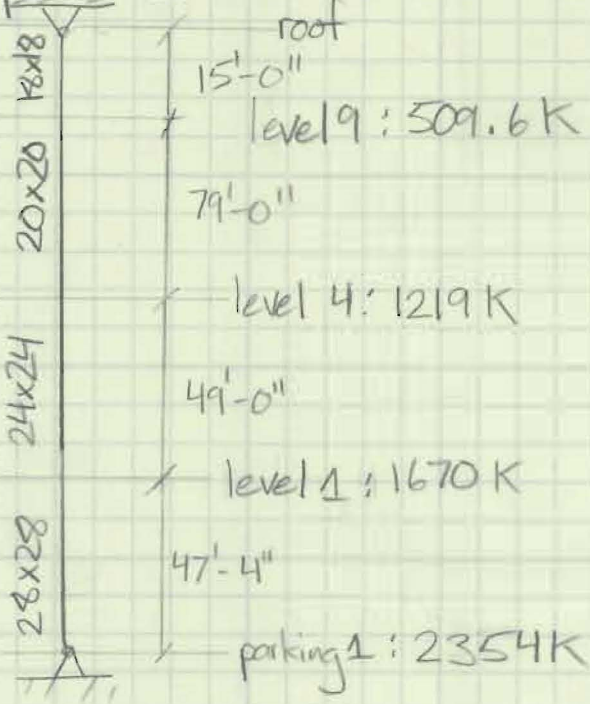
$V_{50\ step/min} = \frac{(1{,}500)(16.204\ \mu in/lb)}{5.1715} = 4700.0\ \mu in/s$

# A.2 Column References

# Initial Column Sizes

Typical Column: D12 (see excel)

$f'_c = 5{,}000$




Parking - Level 1  Try 28×28

Axial Str. Check: $\phi P_n = 0.8\,\phi\,[0.85 f'_c (A_g - A_s) + f_y A_s]$

$A_g = (28)^2 = 784\ in^2$

$2{,}354{,}000 = 0.8(0.65)[0.85(5000)(784 - A_s) + (60{,}000)A_s]$

$4526923 = 3332000 - 4250A_s + 60000A_s$

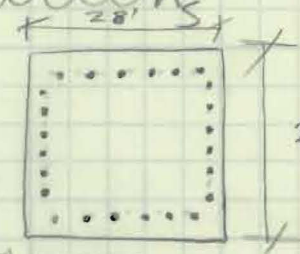
$A_s = 21.4\ in^2$

Try #9s ... $A_s = 1.00\ in^2$

$21.4/1.00 = 21.4$

24 bars, 6 on each side

w/1.5 cover ... $28 - 3 = 25''/5 = 5''$ spaces




spColumn trial: 28 x 28 with 24 #9s

Level 1 - Level 4   Try 24x24

$A_g = (24\,in)^2 = 576\ in^2$

Axial Str. Check

$167000 = 0.8(0.65)[0.85(5000)(576 - A_s) + (60{,}000)A_s]$

$3211538.5 = [2448000 - 4250A_s + 60{,}000A_s]$

$A_s = 13.7\ in^2$

Try 20 #8s $= 0.79 \times 20 = 15.8\,in^2$

spColumn Trial: 24x24 with 20 #8's

Level 4 - Level 9   Try 18x18, $A_g = 324$

Axial Str. Check

$1219{,}000 = 0.8(0.65)[0.85(5000)(324 - A_s) + 60000A_s]$

$2344231 = 1377000 - 4250A_s + 60000A_s$

$A_s = 17.3\ in^2$

Try 20x20, $A_g = 400$

$A_s = 11.55\ in^2$, 16 #8s

spColumn Trial: 20x20 with 16 #8's

Level 9 - Penthouse: Try 18x18, $A_g = 324$

$509600 = 0.8(0.65)[0.85(5000)(324 - A_s) + 60{,}000A_s]$

$980000 = 1377000 + 55750A_s$

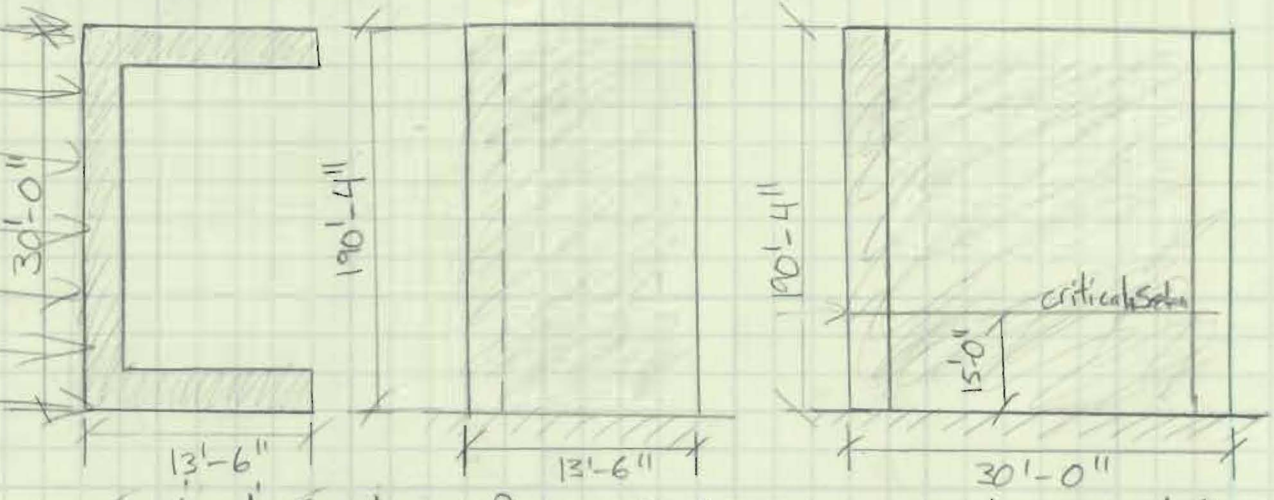
$A_s = 10.8\ in^2$

spColumn trial: 18x18 with 16 #8s

# A.3 Lateral References

# Shear Wall Check | Along Line C5-C6 | (1)

$t = 24"$ from COM/COR iterations




Critical Section for 30'-0" span     * See excel for Etabs story forces

$$a = \text{Min of} \begin{cases} h_w/2 = 190.3/2 \\ \ell_w/2 = 30/2 = \underline{15'} \\ \text{story ht} = \underline{15'} \end{cases}$$

$\therefore$ Critical Section at 15'-0'

Slenderness...

Max unsupported wall ht = 17'-0'

$h_w/\ell_w = 190.3/30 = 6.34 > 2$ $\therefore$ Wall is slender

Design Forces for 30' direction

$M_u = 23{,}436$ ft-k $= 281232000$ lb-in

$V_u = 515.874$ K

$P_u$ = self wt = $150\,pcf(190.33\,ft)(30\,ft)(18/12)$

$= 1285$ K

Load Combo: $1.3D + 1.0E$ (controls)

$d = 0.8\ell_w = 0.8(30)(12) = 288$ in

$N_u = 1.3(1285\,k) = 1670.5$ K

$A_g = (30 \times 12)(22) = 7920\ in^2$

$$V_c = \min \begin{cases} 2\left(1 + \frac{N_u}{500 A_g}\right)\lambda\sqrt{f'_c}\,hd = \\ 3.3\lambda\sqrt{f'_c}\,hd + \frac{N_u d}{4\ell_w} \\ \left[0.6\lambda\sqrt{f'_c} + \frac{\ell_w\left(1.25\lambda\sqrt{f'_c} + 0.2\frac{N_u}{\ell_w h}\right)}{\frac{M_u}{V_u} - \frac{\ell_w}{2}}\right] hd \end{cases}$$

$$V_c = \min \begin{cases} 2\left(1+\frac{1{,}670{,}500}{500(6480)}\right)\sqrt{6000}\,(22)(288) = 1487.7\text{ K} \\ 3.3\sqrt{6000}\,(22)(288)+\frac{1670500\,(288)}{4(30\times12)} = 1953.7\text{K} \\ \left[0.6\sqrt{6000}+\frac{360\left(1.25\sqrt{6000}+0.2\left(\frac{1670500}{360\times22}\right)\right)}{\frac{281232000}{515{,}874}-\frac{360}{2}}\right](22)(288) = \boxed{1162.8\text{K}} \end{cases}$$

$\phi V_c = 0.75\,(1162.8) = 872.1\text{K} > 515.9\text{ K}$

$\therefore$ No $V_s$ req'd for str.

ACI 318-11 requires...

$P_t = \frac{A_{v,horiz}}{h s_2} \geq 0.0020$ for #5 or less (ACI 14.3.2)

$s_2 = \min \begin{cases} 3h = 3(22) = 66'' \\ 18 \end{cases}$

$P_t = \frac{A_{v,horiz}}{(18)(22)} = 0.0020,\ A_v = \frac{0.792\text{ in}^2}{2\text{ curtains}} = 0.396\text{ in}^2$ Need #6's Use diff. reinf. $P_t$

$P_t = \frac{A_v}{(18)(22)} \geq 0.0025$ (ACI 14.3.3b)

$A_v = 0.99 / 2\text{ rows} = 0.495\text{ in}^2$

$\therefore$ Use 2 rows of #7's @ 18" OC,

$P_t = \frac{(0.60\text{in}^2)(2)}{(18)(22)} = 0.00303$

Flexural:

$$P_{\ell,min} = \max \begin{cases} \underline{0.0025}\text{ controls} \\ 0.0025+0.5\left(2.5-\frac{h_w}{l_w}\right)(P_t-0.0025) \\ 0.0025+0.5\left(2.5-\frac{190.33}{30}\right)(0.002716-0.0025) \\ = 0.002085 \end{cases}$$

W/ 12" OC. Spacing: $\frac{A_s}{22(12)} = 0.0025$

$A_s = 0.66\text{ in}^2 / 2\text{ rows} = 0.33\text{in}^2$

Try 2 rows of #6 @ 12" O.C.

$$\rho_\ell = \frac{2(0.44\ in^2)}{(22)(12)} = 0.0033$$

MacGregor Ch. 18: $T = A_s f_y = 14(1.00\ in^2)(60\ ksi) = 840\ k$

$$d = \ell_w - (3 + 3(6)) = 288 - (3 + 3(6)) = 267\ in$$

$$a = \frac{T + N_u}{0.85 f'_c b} = \frac{840 + 1670.5}{0.85(6)(22)}$$


estimate #9s

$a = 22.38\ in$

$$\phi M_n = \phi\left[T\left(d - \frac{a}{2}\right) + N_u\left(\frac{\ell_w - a}{2}\right)\right]$$
$$= 0.9\left[840\left(267 - \frac{22.38}{2}\right) + 1670.5\left(\frac{288 - 22.38}{2}\right)\right]$$
$$= 393065\ k\text{-}in = 32755\ k\text{-}ft > 23436\ k\text{-}ft$$

$\Rightarrow$ Try 4 @ 6 in spacing + 10 #9s

$T = 10(1.00\ in^2)(60\ ksi) = 600\ k$

$d = 288 - (3 + 2(6)) = 273\ in$

$$a = \frac{600 + 1670.5}{0.85(6)(22)} = 20.24\ in$$

$$\phi M_n = 0.9\left[600\left(273 - \frac{20.24}{2}\right) + 1670.5\left(\frac{288 - 20.24}{2}\right)\right]$$
$$= 343237\ k\text{-}in = 28603\ ft\text{-}k > 23436\ k\text{-}ft$$

$\Rightarrow$ Try 4 @ 6 in spacing + 10 #8s

$A_s = 10(0.79\ in^2) = 7.9\ in^2$

$T = (7.9)(60\ ksi) = 474\ k$

$$a = \frac{474 + 1670.5}{0.85(6)(22)} = 19.11\ in$$

$$\phi M_n = 0.9\left[474\left(273 - \frac{19.11}{2}\right) + 1670.5\left(\frac{288 - 19.11}{2}\right)\right]$$
$$= 314456\ k\text{-}in = 26{,}204\ ft\text{-}k > 23436\ ft\text{-}k$$

$\therefore$ Use

Shear Capacity Check:

$$M_{pr} = T\left(d - \frac{a}{2}\right) + N_{pr}\left(\frac{\ell_w - a}{2}\right) = 474\left(273 - \frac{15.68}{2}\right) + 1357\left(\frac{288 - 15.68}{2}\right)$$

$$N_{pr} = \underbrace{\frac{1670.5}{1.3}}_{N_D} + \underbrace{\frac{(30\ ft)(30\ ft)(80\ psf)}{1000}}_{N_L} = 1357 \qquad = 25{,}871\ ft\text{-}k$$

$$a = \frac{474 + 1245}{0.85(6)(22)} = 15.68\ in$$

# Shear Wall Check

Based on MacGregor Recommendation...

$$V_u\ (\text{cap-based}) = \frac{M_{pr}}{0.5h_w} = \frac{2587\ (\text{ft-k})}{0.5(190.33\ \text{ft})} = 271.9\text{k}$$

$271.9\text{k} < V_c$, OK

Shear Str.

$A_{cv} = h\ell_w = 288\text{in}(22\ \text{in}) = 6336\ \text{in}^2$

$V_n = A_{cv}(\alpha_c \lambda \sqrt{f'_c} + \rho_t f_y)$ , $\alpha_c = 2.0$ for slender wall

$= (6336)(2\sqrt{6000} + 0.00303(60000))$

$= 2133.5\ \text{K}$

Check $V_n \leq 8A_{cv}\sqrt{f'_c}$ (ACI 318-11 9.3.2.3)

$= 8(6336)\sqrt{6000}$

$= 3926\text{K}$

$\phi V_n = 0.75(2133.5\ \text{K}) = 1600.1\text{k} > 515.874\text{k}$ OK

# Wind Load Pressures – Perpendicular

**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 | (Table 6-3) |
| 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** | **= $q_h$** |

*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 | $n_1$ = | 0.364 | Hz | (C6-15) |
| Resonant Response Factor | $g_R$ = | 3.7889544 | | (6-9) |
| Background & Wind Factor | $g_v$, $g_Q$ = | 3.4 | | (6-9) |
| Mean Hourly Wind | $V_{z,bar}$ = | 75.4275119 | mph | (6-14) |
| Turbulence Length | $L_{z,bar}$ = | 440.022037 | | (6-7) |
| Reduced Frequency | $N_1$ = | 2.12346951 | | (6-12) |
| Resonant Responcse Factor | R = | 0.25258702 | | (6-10) |
| Turbulence Intensity | $I_z$ = | 0.25583445 | | (6-5) |
| Background Response Factor | Q = | 0.75878531 | | (6-6) |
| **Flexible Gust Effect Factor** | **$G_f$ =** | **0.82602947** | | (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 | | (Figure 6-6) |
| 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 | |

**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 | 0.826029 | 9.3762 | 0.18 | 5.2096 | 13.5428 |
| | 16 - Case 2 | 11.7563 | 0.8 | 0.826029 | 7.7688 | 0.18 | 3.6022 | 11.9354 |
| | 32 | 14.4318 | 0.8 | 0.826029 | 9.5369 | 0.18 | 5.3703 | 13.7035 |
| | 49 | 16.3169 | 0.8 | 0.826029 | 10.7826 | 0.18 | 6.6160 | 14.9492 |
| | 66 | 17.7155 | 0.8 | 0.826029 | 11.7068 | 0.18 | 7.5402 | 15.8734 |
| | 83 | 19.0330 | 0.8 | 0.826029 | 12.5775 | 0.18 | 8.4109 | 16.7440 |
| | 98 | 19.9451 | 0.8 | 0.826029 | 13.1802 | 0.18 | 9.0136 | 17.3468 |
| | 113 | 20.7255 | 0.8 | 0.826029 | 13.6959 | 0.18 | 9.5293 | 17.8625 |
| | 128 | 21.4856 | 0.8 | 0.826029 | 14.1982 | 0.18 | 10.0316 | 18.3648 |
| | 143 | 22.2153 | 0.8 | 0.826029 | 14.6804 | 0.18 | 10.5138 | 18.8470 |
| | 166 | 23.1477 | 0.8 | 0.826029 | 15.2965 | 0.18 | 11.1300 | 19.4631 |
| Leeward | All | 23.1477 | -0.5 | 0.826029 | -9.5603 | 0.18 | -13.7269 | -5.3938 |
| Side | All | 23.1477 | -0.7 | 0.826029 | -13.3845 | 0.18 | -17.5511 | -9.2179 |
| Roof (0'-83') | 166 | 23.1477 | -0.9 | 0.826029 | -17.2086 | 0.18 | -21.3752 | -13.0420 |
| Roof (83'-166') | 166 | 23.1477 | -0.9 | 0.826029 | -17.2086 | 0.18 | -21.3752 | -13.0420 |
| Roof (166'-332') | 166 | 23.1477 | -0.5 | 0.826029 | -9.5603 | 0.18 | -13.7269 | -5.3938 |
| Roof (> 332') | 166 | 23.1477 | -0.3 | 0.826029 | -5.7362 | 0.18 | -9.9028 | -1.5696 |

# Wind Load Pressures – Parallel

**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 $q_z$**

$q_z = 0.00256K_zK_{zt}K_dV^2I$ (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 | (Table 6-3) |
| 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** | **= $q_h$** |

*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 | $n_1$ = | 0.364 | Hz | (C6-15) |
| Resonant Response Factor | $g_R$ = | 3.7889544 | | (6-9) |
| Background & Wind Factor | $g_v$, $g_Q$ = | 3.4 | | (6-9) |
| Mean Hourly Wind | $V_{z,bar}$ = | 75.4275119 | mph | (6-14) |
| Turbulence Length | $L_{z,bar}$ = | 440.022037 | | (6-7) |
| Reduced Frequency | $N_1$ = | 2.12346951 | | (6-12) |
| Resonant Responcse Factor | R = | 0.25258702 | | (6-10) |
| Turbulence Intensity | $I_z$ = | 0.25583445 | | (6-5) |
| Background Response Factor | Q = | 0.75878531 | | (6-6) |
| **Flexible Gust Effect Factor** | **$G_f$ =** | **0.82602947** | | (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) $q_zG_fC_p - q_h(+GC_{pi})$ | Net Pressure (psf) $q_zG_fC_p - q_h(-GC_{pi})$ |
|---|---|---|---|---|---|---|---|---|
| Windward | 16 - Case 1 | 14.1886 | 0.8 | 0.826029 | 9.3762 | 0.18 | 5.2096 | 13.5428 |
| | 16 - Case 2 | 11.7563 | 0.8 | 0.826029 | 7.7688 | 0.18 | 3.6022 | 11.9354 |
| | 32 | 14.4318 | 0.8 | 0.826029 | 9.5369 | 0.18 | 5.3703 | 13.7035 |
| | 49 | 16.3169 | 0.8 | 0.826029 | 10.7826 | 0.18 | 6.6160 | 14.9492 |
| | 66 | 17.7155 | 0.8 | 0.826029 | 11.7068 | 0.18 | 7.5402 | 15.8734 |
| | 83 | 19.0330 | 0.8 | 0.826029 | 12.5775 | 0.18 | 8.4109 | 16.7440 |
| | 98 | 19.9451 | 0.8 | 0.826029 | 13.1802 | 0.18 | 9.0136 | 17.3468 |
| | 113 | 20.7255 | 0.8 | 0.826029 | 13.6959 | 0.18 | 9.5293 | 17.8625 |
| | 128 | 21.4856 | 0.8 | 0.826029 | 14.1982 | 0.18 | 10.0316 | 18.3648 |
| | 143 | 22.2153 | 0.8 | 0.826029 | 14.6804 | 0.18 | 10.5138 | 18.8470 |
| | 166 | 23.1477 | 0.8 | 0.826029 | 15.2965 | 0.18 | 11.1300 | 19.4631 |
| Leeward | All | 23.1477 | -0.4044 | 0.826029 | -7.7324 | 0.18 | -11.8990 | -3.5658 |
| Side | All | 23.1477 | -0.7 | 0.826029 | -13.3845 | 0.18 | -17.5511 | -9.2179 |
| Roof (0'-83') | 166 | 23.1477 | -0.9 | 0.826029 | -17.2086 | 0.18 | -21.3752 | -13.0420 |
| Roof (83'-166') | 166 | 23.1477 | -0.9 | 0.826029 | -17.2086 | 0.18 | -21.3752 | -13.0420 |
| Roof (166'-332') | 166 | 23.1477 | -0.5 | 0.826029 | -9.5603 | 0.18 | -13.7269 | -5.3938 |
| Roof (> 332') | 166 | 23.1477 | -0.3 | 0.826029 | -5.7362 | 0.18 | -9.9028 | -1.5696 |

## Gust Effect Calculations

Inputs

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Natural Freq (n) | 0.364 s | $g_R$ = | 3.788954 | R | n | value |
| C = | 0.3 | $z_{bar}$ = | 85.8 | $R_h$ | 3.174428 | 0.265486 |
| $alpha_{bar}$ = | 0.25 | $V_{barz}$ = | 75.42751 | $R_B$ | 9.351243 | 0.10122 |
| $b_{bar}$ = | 0.45 | $L_{zbar}$ = | 440.022 | $R_L$ | 21.18055 | 0.046099 |
| h = | 143 ft | $N_1$ = | 2.12347 | $R_n$ | N/A | 0.086073 |
| V = | 90 mph | R = | 0.252587 | | | |
| epsilon = | 0.333333 | $I_z$ = | 0.255834 | | | |
| fancy l = | 320 | Q = | 0.758785 | | | |
| beta = | 0.02 | **$G_f$ =** | **0.826029** | | | |

## Wind Pressures

| Level | Floor Height (ft) | Windward (psf) Perp | Windward (psf) Parallel | Leeward Pressure Perp | Leeward Pressure Parallel | Length (ft) Perpendicular | Length (ft) Parallel | Shear (K) Perpendicular | Shear (K) 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** |

# Modified Wind Loads

Estimate building freq: 0.364 s (ETABs)

∴ Building not rigid

Table 6-2: $c = 0.30$

$\bar{\alpha} = 1/4.0$

$\bar{b} = 0.45$

Taking structural ht from bottom of penthouse: $h =$

$190.32 - 47.32 = 143\text{ft}$

$\bar{z} = \max\left\{ \begin{array}{l} 0.6(143) \\ 30 \end{array} \right.$

$\bar{V}_z = 75.4$

$L_{\bar{z}} = 440$

$R_n: \ n = \dfrac{4.6 n_1 h}{\bar{V}_z}$

---

Updated $G_f$: <u>0.826</u>

Updated Bed Tower

Ordinary Reinforced Conc. Shear Walls

$R = 4$

$\Omega = 2.5$

$C_d = 4$

$I = 1.5$

Permitted for Site Class C

$$C_s = \frac{S_{DS}}{(R/I)} = \frac{0.1824}{4/1.5} = 0.0684$$

Need not exceed...

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.09747}{(1.12)(4/1.5)} = \boxed{0.03263}$$

Fundamental Period

$$T = C_t h_n^x = (0.02)(214.1)^{0.75} = 1.12\,s$$

$C_t = 0.02 \quad x = 0.75 \quad$ (Table 12.8-2)

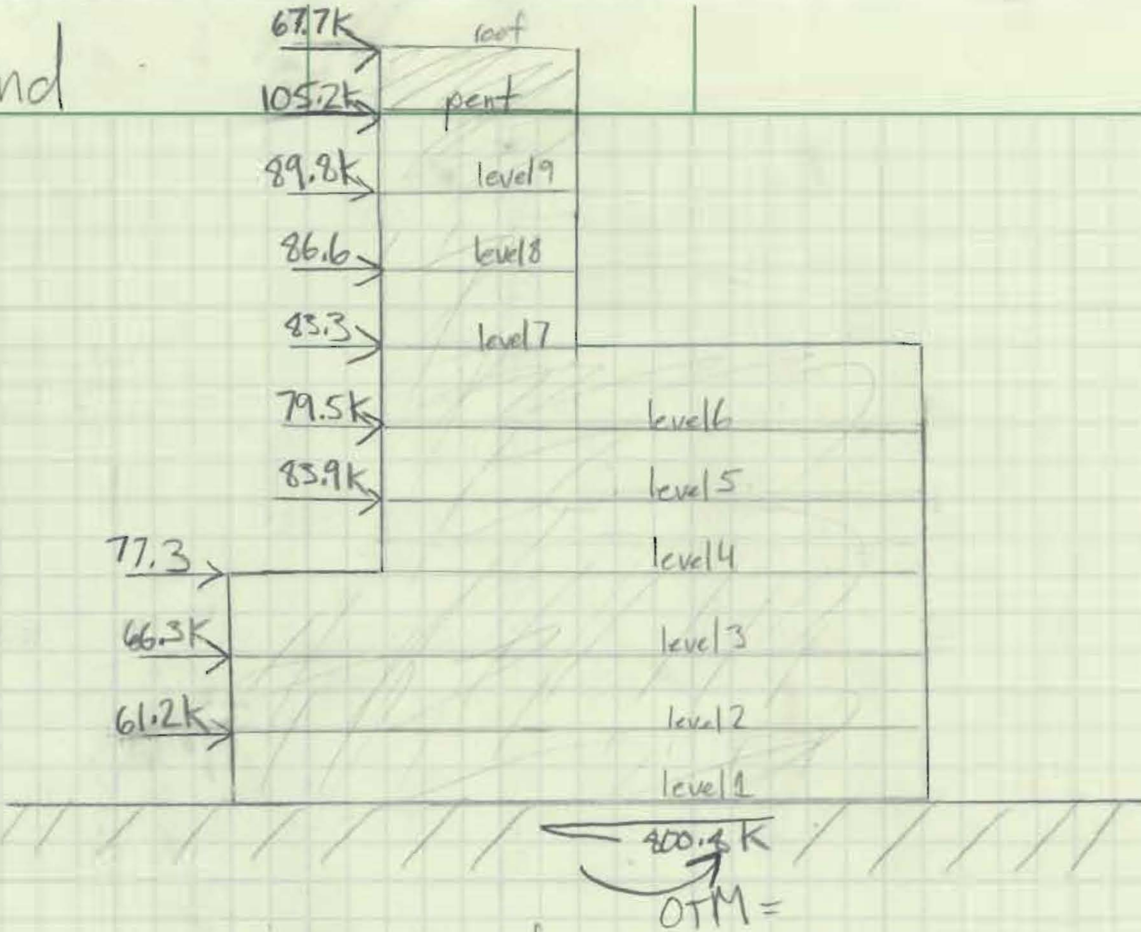
$h_n = 214.1$ ft

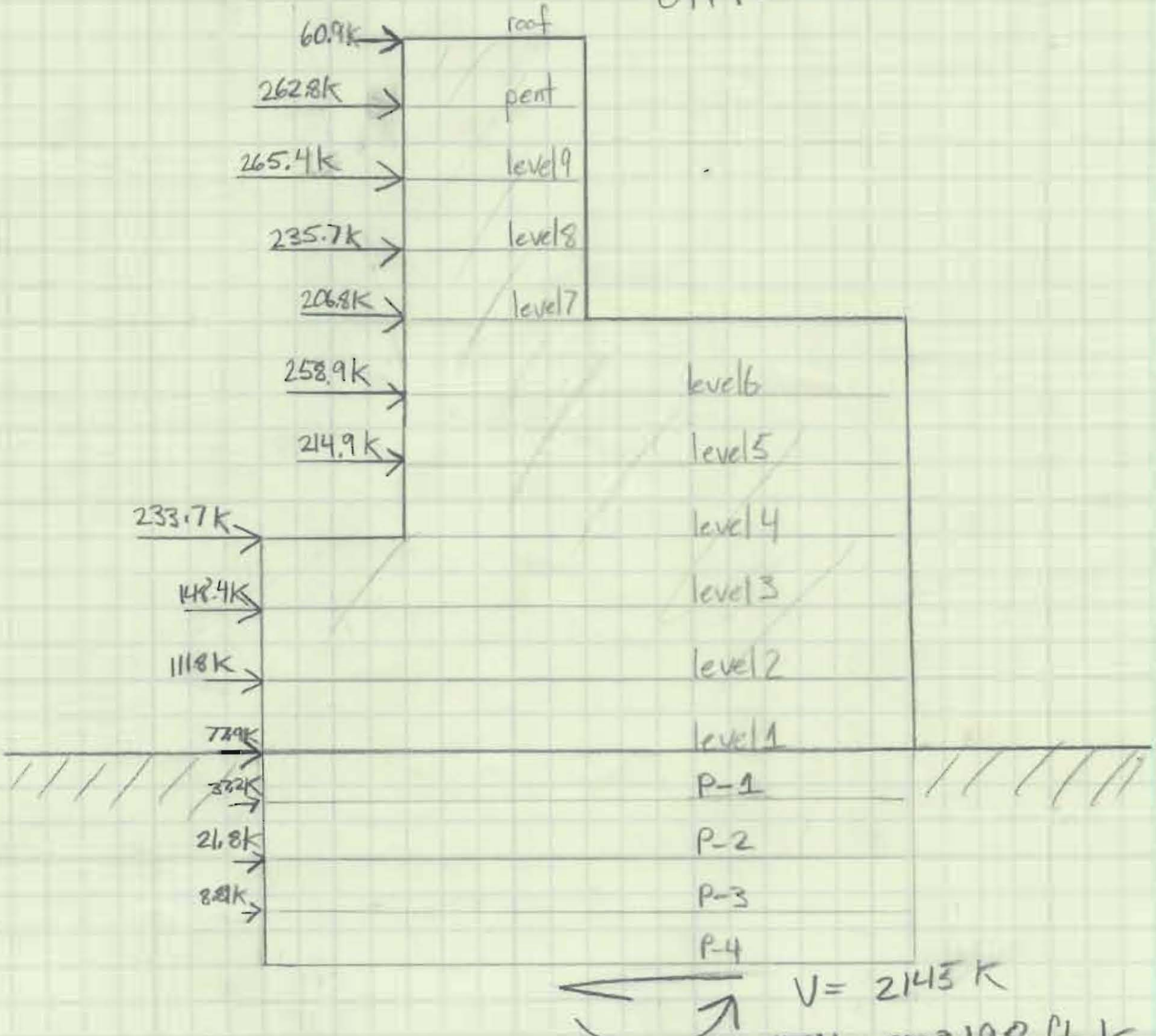
New k value

| | |
|---|---|
| $T = 0.5\,s$ | 1 |
| $1.12\,s$ | 1.31 |
| $T = 2.5\,s$ | 2 |

$$V = C_s W = (0.03263)(85732) = 2797\,k$$

Wind




V = 2145 K

OTM = 278,198 ft-K

## Center of Mass – 5th Floor

| Object | Weight (pcf) | Area ($ft^2$) | Thickness (ft) | Point Load (k) | x (ft) | y (ft) | w*x | w*y |
|---|---|---|---|---|---|---|---|---|
| **Slab** | 150 | 60200 | 0.916666667 | 8277.5 | 126 | 163 | 1042965 | 1349233 |
| **Drop Panels** | 150 | 4100 | 0.6875 | 422.8125 | 131 | 148 | 55388.44 | 62576.25 |
| **A6** | 150 | 450 | 1.5 | 101.25 | 75 | 348 | 7593.75 | 35235 |
| **P1** | 150 | 202.5 | 1.833333333 | 55.6875 | 65.5 | 273 | 3647.531 | 15202.69 |
| **P4** | 150 | 202.5 | 1.833333333 | 55.6875 | 113.5 | 273 | 6320.531 | 15202.69 |
| **P3** | 150 | 450 | 1.833333333 | 123.75 | 60 | 256.5 | 7425 | 31741.88 |
| **P5** | 150 | 450 | 1.833333333 | 123.75 | 120 | 256.5 | 14850 | 31741.88 |
| **A1** | 150 | 202.5 | 1.833333333 | 55.6875 | 65.5 | 239.5 | 3647.531 | 13337.16 |
| **P6** | 150 | 202.5 | 1.833333333 | 55.6875 | 113.5 | 239.5 | 6320.531 | 13337.16 |
| **P11** | 150 | 900 | 1.5 | 202.5 | 210 | 150 | 42525 | 30375 |
| **A4** | 150 | 900 | 1.666666667 | 225 | 210 | 15 | 47250 | 3375 |
| **P7** | 150 | 210 | 1.5 | 47.25 | 75 | 118 | 3543.75 | 5575.5 |
| **P8** | 150 | 210 | 1.5 | 47.25 | 75 | 103 | 3543.75 | 4866.75 |
| **P9** | 150 | 228 | 1.5 | 51.3 | 68 | 110.5 | 3488.4 | 5668.65 |
| **P10** | 150 | 228 | 1.5 | 51.3 | 82 | 110.5 | 4206.6 | 5668.65 |
| **12-CD** | 150 | 450 | 1.5 | 101.25 | 75 | 60 | 7593.75 | 6075 |
| **P13** | 150 | 900 | 1.5 | 202.5 | 285 | 60 | 57712.5 | 12150 |

$COM_x$ 129.2158 ft

$COM_y$ 160.9153 ft

## Center of Rigidity – 5th Floor

| Object | Dir. | h (ft) | b (ft) | Thickness (ft) | $h^3/(Eb^3t)$ | 1.2h/Gbt | k (lb-in) | Rel Stiffness | x (ft) | y (ft) | k*x | k*y |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **A6** | y | 15 | 30 | 1.5 | 1.57285E-09 | 1.8129E-08 | 50756464.3 | 0.112482959 | 75 | 348 | - | 17663249574 |
| **P1** | Y | 15 | 13.5 | 1.833333333 | 1.41221E-08 | 3.2962E-08 | 21238612.3 | 0.047067541 | 65.5 | 273 | - | 5798141165 |
| **P4** | Y | 15 | 13.5 | 1.833333333 | 1.41221E-08 | 3.2962E-08 | 21238612.3 | 0.047067541 | 113.5 | 273 | - | 5798141165 |
| **P3** | X | 15 | 30 | 1.833333333 | 1.28688E-09 | 1.4833E-08 | 62035678.6 | 0.226551461 | 60 | 256.5 | 3722140715 | - |
| **P5** | X | 15 | 30 | 1.833333333 | 1.28688E-09 | 1.4833E-08 | 62035678.6 | 0.226551461 | 120 | 256.5 | 7444281430 | - |
| **A1** | Y | 15 | 13.5 | 1.833333333 | 1.41221E-08 | 3.2962E-08 | 21238612.3 | 0.047067541 | 65.5 | 239.5 | - | 5086647652 |
| **P6** | Y | 15 | 13.5 | 1.833333333 | 1.41221E-08 | 3.2962E-08 | 21238612.3 | 0.047067541 | 113.5 | 239.5 | - | 5086647652 |
| **P11** | Y | 15 | 60 | 1.5 | 1.96606E-10 | 9.0645E-09 | 107978025 | 0.239293418 | 210 | 150 | - | 16196703815 |
| **A4** | Y | 15 | 60 | 1.666666667 | 1.76946E-10 | 8.1581E-09 | 119975584 | 0.265881575 | 210 | 15 | - | 1799633757 |
| **P7** | Y | 15 | 14 | 1.5 | 1.54763E-08 | 3.8848E-08 | 18407967.9 | 0.040794463 | 75 | 118 | - | 2172140212 |
| **P8** | Y | 15 | 14 | 1.5 | 1.54763E-08 | 3.8848E-08 | 18407967.9 | 0.040794463 | 75 | 103 | - | 1896020693 |
| **P9** | X | 15 | 15.2 | 1.5 | 1.20926E-08 | 3.5781E-08 | 20888305 | 0.076283135 | 68 | 110.5 | 1420404742 | - |
| **P10** | X | 15 | 15.2 | 1.5 | 1.20926E-08 | 3.5781E-08 | 20888305 | 0.076283135 | 82 | 110.5 | 1712841013 | - |
| **12-CD** | Y | 15 | 30 | 1.5 | 1.57285E-09 | 1.8129E-08 | 50756464.3 | 0.112482959 | 75 | 60 | - | 3045387858 |
| **P13** | X | 15 | 60 | 1.5 | 1.96606E-10 | 9.0645E-09 | 107978025 | 0.39433081 | 285 | 60 | 3.0774E+10 | - |

**E =** 4415201 $lb/in^2$

**G =** 1838667.08 $lb/in^2$

**$COR_x$** 62.1648193 ft

**$COR_y$** 89.0167076 ft

**$e_x$** -67.050971 ft

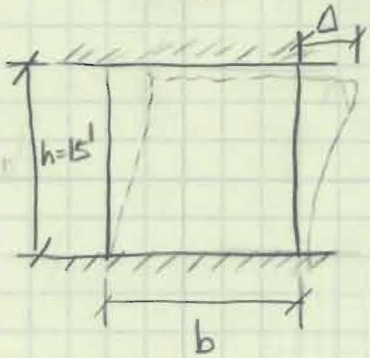
**$e_y$** -71.898546 ft

# Load Distribution – 5th Floor Seismic Loads

| Object | $R_x$ (lb-in) | Rel. Stiffness | Direct Shear (K) | | Torsional Shear (K) | | | | Total Shear (K) |
|---|---|---|---|---|---|---|---|---|---|
| | | | Seismic | $d_i$ (ft) | $R_x$*$d_i$ (k-ft$^2$) | $R_x$*$d_i^2$(k-ft$^3$) | $R_id_i$/J (1/ft) | $V_t$ - Seismic | Seismic |
| A6 | 50756464 | 0.112483 | 24.173 | 258.9832924 | 1095423.019 | 4633316615 | 4.72761E-05 | 0.730462206 | 24.903 |
| P1 | 21238612 | 0.047068 | 10.115 | 183.9832924 | 325629.1518 | 576325943.2 | 1.7566E-05 | 0.271412295 | 10.386 |
| P4 | 21238612 | 0.047068 | 10.115 | 183.9832924 | 325629.1518 | 576325943.2 | 1.81297E-05 | 0.280121206 | 10.395 |
| P3 | 62035679 | 0.226551 | 48.686 | 167.4832924 | 865828.308 | 4476020552 | 4.83597E-05 | 0.747205228 | 49.433 |
| P5 | 62035679 | 0.226551 | - | - | - | - | - | - | - |
| A1 | 21238612 | 0.047068 | - | - | - | - | - | - | - |
| P6 | 21238612 | 0.047068 | 10.115 | 150.4832924 | 266338.0258 | 471387506.4 | 1.4876E-05 | 0.229848301 | 10.345 |
| P11 | 1.08E+08 | 0.239293 | 51.424 | 60.98329241 | 548737.9582 | 4937636767 | 3.0649E-05 | 0.473557942 | 51.898 |
| A4 | 1.2E+08 | 0.265882 | 57.138 | 74.01670759 | 740016.4754 | 7398659056 | 4.13326E-05 | 0.638630285 | 57.777 |
| P7 | 18407968 | 0.040794 | 8.767 | 28.98329241 | 44460.29302 | 68201970.55 | 2.48327E-06 | 0.038368997 | 8.805 |
| P8 | 18407968 | 0.040794 | 8.767 | 13.98329241 | 21450.33314 | 32904753.66 | 1.19808E-06 | 0.018511524 | 8.785 |
| P9 | 20888305 | 0.076283 | - | - | - | - | - | - | - |
| P10 | 20888305 | 0.076283 | - | - | - | - | - | - | - |
| 12-CD | 50756464 | 0.112483 | 24.17258789 | 29.01670759 | 122732.1236 | 519120720.7 | 6.85504E-06 | 0.10591717 | 24.279 |
| P13 | 1.08E+08 | 0.394331 | - | - | - | - | - | - | - |

| Object | $R_x$ (lb-in) | Rel. Stiffness | Direct Shear (K) | Torsional Shear (K) | | | | | Total Shear (K) |
|---|---|---|---|---|---|---|---|---|---|
| | | | Seismic | $d_i$ (ft) | $R_x$*$d_i$ (k-ft$^2$ | $R_x$*$d_i^2$(k-ft$^3$) | $R_id_i$/J (1/ft) | $V_t$ - Seismic | Seismic |
| A6 | 50756464 | 0.112483 | - | - | - | - | - | - | |
| P1 | 21238612 | 0.047068 | - | - | - | - | - | - | |
| P4 | 21238612 | 0.047068 | - | - | - | - | - | - | |
| P3 | 62035679 | 0.226551 | 48.686 | 194.3351807 | 1004643 | 5193642005 | 9.4066E-05 | 1.355420292 | 50.041 |
| P5 | 62035679 | 0.226551 | 48.686 | 194.3351807 | 1004643 | 5193642005 | 9.25471E-05 | 1.333534862 | 50.019 |
| A1 | 21238612 | 0.047068 | - | - | - | - | - | - | |
| P6 | 21238612 | 0.047068 | - | - | - | - | - | - | |
| P11 | 1.08E+08 | 0.239293 | - | - | - | - | - | - | |
| A4 | 1.2E+08 | 0.265882 | - | - | - | - | - | - | |
| P7 | 18407968 | 0.040794 | - | - | - | - | - | - | |
| P8 | 18407968 | 0.040794 | - | - | - | - | - | - | |
| P9 | 20888305 | 0.076283 | 16.39324561 | 48.33518067 | 84136.67 | 146456029.5 | 7.87782E-06 | 0.113513513 | 16.507 |
| P10 | 20888305 | 0.076283 | 16.39324561 | 48.33518067 | 84136.67 | 146456029.5 | 0.000261509 | 0.113513513 | 16.507 |
| 12-CD | 50756464 | 0.112483 | - | - | - | - | - | - | |
| P13 | 1.08E+08 | 0.394331 | 84.74169103 | 2.164819326 | 19479.41 | 175279016.2 | 0.000111134 | 0 | 84.742 |

- Shear Wall Stiffness
- Fixed-fixed between floors




$$k = \frac{1}{\left(\frac{h^3}{Eb^3t} + \frac{1.2h}{Gbt}\right)}$$

See excel...

COR: (104 ft, 161.9 ft)

Eccentricity: x-dir: 104 - 96.5 = 7.5 ft
y-dir: 161.9 - 149 = 12.9 ft

COM calcs

① Center of Slab

$36{,}130\ ft^2$ @ $(130\ ft,\ 75\ ft)$

$24{,}070\ ft^2$ @ $(45\ ft,\ 275\ ft)$

$$x:\ \frac{(130\ ft)(36{,}130\ ft^2) + (45\ ft)(24{,}070\ ft)^2}{36130 + 24070} = 96\ ft$$

$$y:\ \frac{(75)(36{,}130) + (275)(24070)}{36130 + 24070} = 148\ ft$$

Center of Drop Panels

$14(100\ ft^2)$ @ $(45,\ 245)$

$27(100\ ft^2)$ @ $(130,\ 75)$

$$x:\ \frac{14(100)(45) + 27(100)(130)}{41(100)} = 101\ ft$$

$$y:\ \frac{14(100)(245) + 27(100)(75)}{41(100)} = 133\ ft$$

See Attached Excel, COM at (96.5, 149. ft)

COR + COM Calcs | Floor 5 

Excel Spreadsheet Verification (see Wall Layout for Corresponding Labels)

Wall A6

Wt: 150 pcf
Thickness: 18/12 = 1.5 ft
Area = 30 ft length × 15 ft ht = 450 ft$^2$

Pt load: (150 pcf)(1.5 ft)(450 ft$^2$) = 101.25 k

$x = 75$ ft
$y = 348$ ft

$w(x) = 101.25(75) = 7593.75$
$w(y) = 101.25(348) = 35235$

*See excel for other values

$COM_x = 129.2158$ ft
$COM_y = 160.9153$ ft

Note: Dimensions changed, put directly into excel after calcs verified spreadsheet output

Stiffness of Wall A6

$$\frac{h^3}{Eb^3t} = \frac{(15\text{ ft})^3}{(4415201\frac{\text{lb}}{\text{in}^2})(30\text{ ft})^3(18\text{ in})} = 1.5728 \times 10^{-9}\text{ in/lb}$$

$$\frac{1.2h}{Gbt} = \frac{1.2(15\text{ ft})}{(1838667.08\frac{\text{lb}}{\text{in}^2})(30\text{ ft})(18)} = 1.8129 \times 10^{-8}\text{ in/lb}$$

$$K = \frac{1}{1.5728\times10^{-9} + 1.8129\times10^{-8}} = 50756783.64$$

See excel for relative stiffness values

# Appendix B

# Construction References

# B.1 Estimate References

| Forms in Place | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Section | | | Activity | Floor | Total | Crew | Daily Output | LH | Unit | Material | Labor | Equip | Total | Crews | Duration | Total $ | Location | Factored Cost |
| 031113 | 20 | 1600 | Exterior spandrel, job-built plywood, 24" wide, 3 use | TOTAL | 9078.42 | C-2 | 315 | 0.152 | SFCA | 0.99 | 6.8 | | 7.79 | | 28.82038 | 70720.89 | 0.875 | 61880.78033 |
| | | | | LL | 2019.93 | | | | | | | | | | | | | |
| | | | | LR | 0 | | | | | | | | | | | | | |
| | | | | UL | 7058.49 | | | | | | | | | | | | | |
| | | | | UR | 0 | | | | | | | | | | | | | |
| 031113 | 20 | 2600 | Interior beam, job-built plywood, 24" wide, 3 use | TOTAL | 3060 | C-2 | 385 | 0.125 | SFCA | 1 | 5.55 | | 6.55 | | 7.948052 | 20043 | 0.875 | 17537.625 |
| | | | | LL | 0 | | | | | | | | | | | | | |
| | | | | LR | 0 | | | | | | | | | | | | | |
| | | | | UL | 3060 | | | | | | | | | | | | | |
| | | | | UR | 0 | | | | | | | | | | | | | |
| 031113 | 25 | 7700 | 20"X20" column | TOTAL | 11700 | C-2 | 420 | 0.076 | SFCA | 1.08 | 3.32 | | 4.4 | | 27.85714 | 51480 | 0.875 | 45045 |
| | | | | LL | | | | | | | | | | | | | | |
| | | | | LR | | | | | | | | | | | | | | |
| | | | | UL | | | | | | | | | | | | | | |
| | | | | UR | | | | | | | | | | | | | | |
| 031113 | 25 | 7700 | 24"X24" column | TOTAL | 16593.3 | C-2 | 440 | 0.073 | SFCA | 0.78 | 3.17 | | 3.95 | | 37.71205 | 65543.54 | 0.875 | 57350.59313 |
| | | | | LL | | | | | | | | | | | | | | |
| | | | | LR | | | | | | | | | | | | | | |
| | | | | UL | | | | | | | | | | | | | | |
| | | | | UR | | | | | | | | | | | | | | |
| 031113 | 25 | 7755 | 30"X30" column | TOTAL | 4648.5 | C-2 | 440 | 0.073 | SFCA | 0.99 | 3.17 | | 4.16 | | 10.56477 | 19337.76 | 0.875 | 16920.54 |
| | | | | LL | | | | | | | | | | | | | | |
| | | | | LR | | | | | | | | | | | | | | |
| | | | | UL | | | | | | | | | | | | | | |
| | | | | UR | | | | | | | | | | | | | | |
| 031113 | 25 | 7755 | 36"X36" column | TOTAL | 671.45 | C-2 | 460 | 0.07 | SFCA | 0.87 | 3.03 | | 3.9 | | 1.459674 | 2618.655 | 0.875 | 2291.323125 |
| | | | | LL | | | | | | | | | | | | | | |
| | | | | LR | | | | | | | | | | | | | | |
| | | | | UL | | | | | | | | | | | | | | |
| | | | | UR | | | | | | | | | | | | | | |
| 031113 | 35 | 2050 | Flat slab, drop panels, job-built plywood, to 15' high, | TOTAL | 145811.6 | C-2 | 509 | 0.094 | SFCA | 2.31 | 4.21 | | 6.52 | | 286.4669 | 950691.9 | 0.875 | 831855.4062 |
| | | | 2 use | LL | | | | | | | | | | | | | | |
| | | | | LR | | | | | | | | | | | | | | |
| | | | | UL | | | | | | | | | | | | | | |
| | | | | UR | | | | | | | | | | | | | | |
| 031113 | 35 | 2250 | Flat slab, drop panels, job-built plywood, 15'-20' high | TOTAL | 410591.7 | C-2 | 480 | 0.1 | SFCA | 2.43 | 4.47 | | 6.9 | | 855.3994 | 2833083 | 0.875 | 2478947.57 |
| | | | 4 use | LL | | | | | | | | | | | | | | |
| | | | | LR | | | | | | | | | | | | | | |
| | | | | UL | | | | | | | | | | | | | | |
| | | | | UR | | | | | | | | | | | | | | |
| 031113 | 35 | 7101 | Edge forms, 7" to 12" high, 4 use | TOTAL | 11972.5 | C-1 | 350 | 0.091 | L.F. | 0.18 | 3.98 | | 4.16 | | 34.20714 | 49805.6 | 0.875 | 43579.9 |
| | | | | LL | | | | | | | | | | | | | | |
| | | | | LR | | | | | | | | | | | | | | |
| | | | | UL | | | | | | | | | | | | | | |
| | | | | UR | | | | | | | | | | | | | | |
| 031113 | 35 | 7500 | Depressed area forms to 12" high, 4 use | TOTAL | 3696 | C-1 | 300 | 0.107 | L.F. | 0.92 | 4.65 | | 5.57 | | 12.32 | 20586.72 | 0.875 | 18013.38 |
| | | | | LL | | | | | | | | | | | | | | |
| | | | | LR | | | | | | | | | | | | | | |
| | | | | UL | | | | | | | | | | | | | | |
| | | | | UR | | | | | | | | | | | | | | |
| 031113 | 85 | 2500 | Wall, job-built plywood, 8' to 16' high, 3 use | TOTAL | 64448.79 | C-2 | 375 | 0.128 | SFCA | 0.89 | 5.7 | | 6.59 | | 171.8634 | 424717.5 | 0.875 | 371627.8561 |
| | | | | LL | 18205.53 | | | | | | | | | | | | | |
| | | | | LR | 11156.4 | | | | | | | | | | | | | |
| | | | | UL | 18800.46 | | | | | | | | | | | | | |

| | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | UR | 16286.4 | | | | | | | | | | | | | |
| 031113 | 85 | 2850 | Wall, job-built plywood, over 16' high, 4 use | TOTAL | 114790.3 | C-2 | 330 | 0.145 | SFCA | 0.82 | 6.5 | | 7.32 | | 347.8494 | 840264.9 | 0.875 | 735231.8151 |
| | | | | LL | 26775.87 | | | | | | | | | | | | | |
| | | | | LR | 35758.8 | | | | | | | | | | | | | |
| | | | | UL | 29042.82 | | | | | | | | | | | | | |
| | | | | UR | 23212.8 | | | | | | | | | | | | | |
| Shores | | | | | | | | | | | | | | | | | | |
| Section | | | Activity | Floor | Total | Crew | Daily Output | LH | Unit | Material | Labor | Equip | Total | Crews | Duration | Total $ | Location | Factored Cost |
| 031505 | 70 | 0500 | Aluminum joists and stringers, spaced @ 2' O.C., per mo. | TOTAL | 6387 | 2 carp | 60 | 0.267 | EA. | | 12.25 | | 12.25 | | | 78240.75 | 0.875 | 68460.65625 |
| | | | | LL | | | | | | | | | | | | | | |
| | | | | LR | | | | | | | | | | | | | | |
| | | | | UL | | | | | | | | | | | | | | |
| | | | | UR | | | | | | | | | | | | | | |
| 031505 | 70 | 3600 | #3 post shore, 8'-10" to 16'-1" high, 3800# capacity | TOTAL | 6387 | | | | EA. | 187 | | | 187 | | | 1194369 | 0.875 | 1045072.875 |
| | | | | LL | | | | | | | | | | | | | | |
| | | | | LR | | | | | | | | | | | | | | |
| | | | | UL | | | | | | | | | | | | | | |
| | | | | UR | | | | | | | | | | | | | | |
| 031505 | 70 | 1500 | Reshoring | TOTAL | 556403.4 | 2 Carp | 1400 | 0.011 | S.F. | 0.58 | 0.52 | | 1.1 | | 397.431 | 612043.7 | 0.875 | 535538.2436 |
| | | | | LL | | | | | | | | | | | | | | |
| | | | | LR | | | | | | | | | | | | | | |
| | | | | UL | | | | | | | | | | | | | | |
| | | | | UR | | | | | | | | | | | | | | |
| Rebar and Accessories | | | | | | | | | | | | | | | | | | |
| Section | | | Activity | Floor | Total | Crew | Daily Output | LH | Unit | Material | Labor | Equip | Total | Crews | Duration | Total $ | Location | Factored Cost |
| 032111 | 60 | 0252 | Columns, #8-#18 | TOTAL | 700491.8 | 4 Rodm | 4600 | 0.007 | Lb. | 0.5 | 0.35 | | 0.85 | | 152.2808 | 595418 | 0.875 | 520990.7763 |
| | | | | LL | | | | | | | | | | | | | | |
| | | | | LR | | | | | | | | | | | | | | |
| | | | | UL | | | | | | | | | | | | | | |
| | | | | UR | | | | | | | | | | | | | | |
| 032111 | 60 | 0402 | Elevated slabs, #4-#7 | TOTAL | 3311400 | 4 Rodm | 5800 | 0.006 | Lb. | 0.5 | 0.28 | | 0.78 | | 570.931 | 2582892 | 0.875 | 2260030.5 |
| | | | | LL | | | | | | | | | | | | | | |
| | | | | LR | | | | | | | | | | | | | | |
| | | | | UL | | | | | | | | | | | | | | |
| | | | | UR | | | | | | | | | | | | | | |
| 032111 | 60 | 0702 | Walls, #3-#7 | TOTAL | 267553 | 4 Rodm | 6000 | 0.005 | Lb. | 0.5 | 0.27 | | 0.77 | | 44.59217 | 206015.8 | 0.875 | 180263.8338 |
| | | | | LL | 145661 | | | | | | | | | | | | | |
| | | | | LR | 18114.5 | | | | | | | | | | | | | |
| | | | | UL | 77323 | | | | | | | | | | | | | |
| | | | | UR | 26454.5 | | | | | | | | | | | | | |
| 032111 | 60 | 0752 | Walls, #8-#18 | TOTAL | 116704.7 | 4 Rodm | 8000 | 0.004 | Lb. | 0.5 | 0.2 | | 0.7 | | 14.58809 | 81693.29 | 0.875 | 71481.62875 |
| | | | | LL | 55959 | | | | | | | | | | | | | |
| | | | | LR | 20877.3 | | | | | | | | | | | | | |
| | | | | UL | 29705.4 | | | | | | | | | | | | | |
| | | | | UR | 10163 | | | | | | | | | | | | | |
| Placing Concrete | | | | | | | | | | | | | | | | | | |
| Section | | | Activity | Floor | Total | Crew | Daily Output | LH | Unit | Material | Labor | Equip | Total | Crews | Duration | Total $ | Location | Factored Cost |
| 033113 | 35 | 0400 | Heavyweight concrete, ready mix, 5000 psi | TOTAL | 23405.5 | | | | CY | 110 | | | 110 | | | 2574605 | 0.875 | 2252779.375 |
| | | | | LL | | | | | | | | | | | | | | |
| | | | | LR | | | | | | | | | | | | | | |
| | | | | UL | | | | | | | | | | | | | | |
| | | | | UR | | | | | | | | | | | | | | |
| 033113 | 35 | 0411 | Heavyweight concrete, ready mix, 6000 psi | TOTAL | 4504.5 | | | | CY | 113 | | | 113 | | | 509008.5 | 0.875 | 445382.4375 |
| | | | | LL | 909.3 | | | | | | | | | | | | | |
| | | | | LR | 1351.5 | | | | | | | | | | | | | |
| | | | | UL | 1507.1 | | | | | | | | | | | | | |

| | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | UR | 736.6 | | | | | | | | | | | | | |
| 033113 | 70 | 0800 | Columns, 24" thick, pumped | TOTAL | 2495.5 | C-20 | 92 | 0.696 | CY | | 27.5 | 8.45 | 35.95 | | 27.125 | 89713.23 | 0.875 | 78499.07188 |
| | | | | LL | | | | | | | | | | | | | | |
| | | | | LR | | | | | | | | | | | | | | |
| | | | | UL | | | | | | | | | | | | | | |
| | | | | UR | | | | | | | | | | | | | | |
| 033113 | 70 | 1600 | Slabs over 10" thick, pumped | TOTAL | 20910 | C-20 | 180 | 0.356 | CY | | 14 | 4.31 | 18.31 | | 116.1667 | 382862.1 | 0.875 | 335004.3375 |
| | | | | LL | | | | | | | | | | | | | | |
| | | | | LR | | | | | | | | | | | | | | |
| | | | | UL | | | | | | | | | | | | | | |
| | | | | UR | | | | | | | | | | | | | | |
| 033113 | 70 | 5350 | Walls, 15" thick, pumped | TOTAL | 4504.5 | C-20 | 120 | 0.533 | CY | | 21 | 6.45 | 27.45 | | 37.5375 | 123648.5 | 0.875 | 108192.4594 |
| | | | | LL | 909.3 | | | | | | | | | | | | | |
| | | | | LR | 1351.5 | | | | | | | | | | | | | |
| | | | | UL | 1507.1 | | | | | | | | | | | | | |
| | | | | UR | 736.6 | | | | | | | | | | | | | |
| 033513 | 30 | 0125 | Bull float and manual float | TOTAL | 556403.4 | C-10 | 2000 | 0.012 | SF | | 0.5 | | 0.5 | | 278.2017 | 278201.7 | 0.875 | 243426.4744 |
| | | | | LL | | | | | | | | | | | | | | |
| | | | | LR | | | | | | | | | | | | | | |
| | | | | UL | | | | | | | | | | | | | | |
| | | | | UR | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | 3461.322 | | **TOTAL** | **12,825,404.46** |

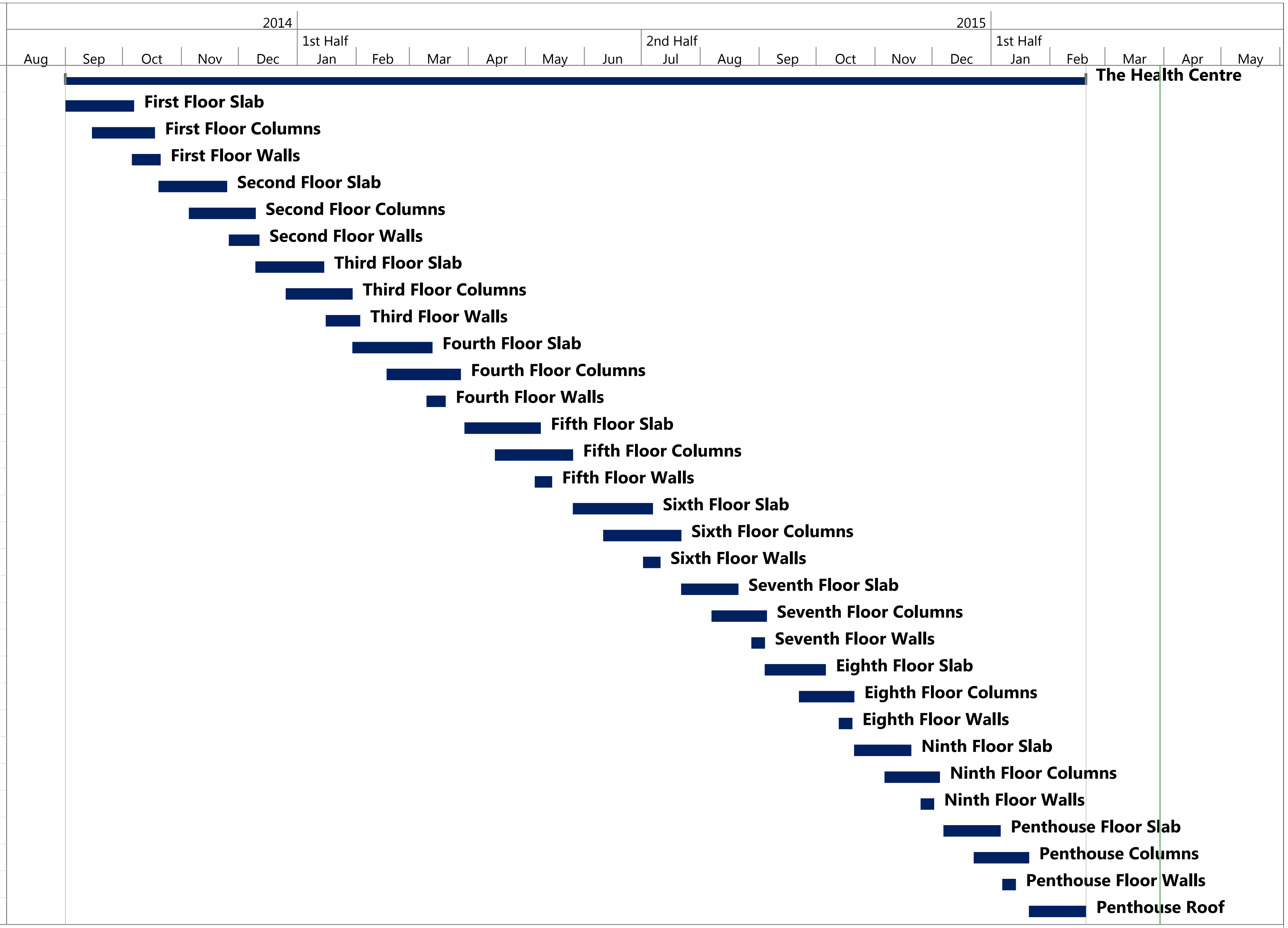
| Section | | | Activity | Unit | | Floor 1-3 | Floor 4 | Floor 5-6 | Floor 7-Pent | Total |
|---|---|---|---|---|---|---|---|---|---|---|
| 031113 | 20 | 1600 | Exterior spandrel, job-built plywood, 24" wide, 3 use | SFCA | TOTAL | 2524.5 | 1003.98 | 1849.98 | 3699.96 | 9078.42 |
| | | | | | LL | 1215 | 114.99 | 229.98 | 459.96 | 2019.93 |
| | | | | | LR | 0 | 0 | 0 | 0 | 0 |
| | | | | | UL | 1309.5 | 888.99 | 1620 | 3240 | 7058.49 |
| | | | | | UR | 0 | 0 | 0 | 0 | 0 |
| 031113 | 20 | 2600 | Interior beam, job-built plywood, 24" wide, 3 use | SFCA | TOTAL | 1080 | 360 | 540 | 1080 | 3060 |
| | | | | | LL | 0 | 0 | 0 | 0 | 0 |
| | | | | | LR | 0 | 0 | 0 | 0 | 0 |
| | | | | | UL | 1080 | 360 | 540 | 1080 | 3060 |
| | | | | | UR | 0 | 0 | 0 | 0 | 0 |
| 031113 | 25 | 7700 | 20"X20" column | SFCA | TOTAL | 0 | 2550 | 5100 | 4050 | 11700 |
| | | | | | | 0 | | | | 0 |
| | | | | | | 0 | | | | 0 |
| | | | | | | 0 | | | | 0 |
| | | | | | | 0 | | | | 0 |
| 031113 | 25 | 7700 | 24"X24" column | SFCA | TOTAL | 8296.65 | 8296.65 | 0 | 0 | 16593.3 |
| | | | | | LL | | | | | 0 |
| | | | | | LR | | | | | 0 |
| | | | | | UL | | | | | 0 |
| | | | | | UR | | | | | 0 |
| 031113 | 25 | 7755 | 30"X30" column | SFCA | TOTAL | 4648.5 | 0 | 0 | 0 | 4648.5 |
| | | | | | LL | | | | | 0 |
| | | | | | LR | | | | | 0 |
| | | | | | UL | | | | | 0 |
| | | | | | UR | | | | | 0 |
| 031113 | 25 | 7755 | 36"X36" column | SFCA | TOTAL | 671.45 | 0 | 0 | 0 | 671.45 |
| | | | | | LL | | | | | 0 |
| | | | | | LR | | | | | 0 |
| | | | | | UL | | | | | 0 |
| | | | | | UR | | | | | 0 |
| 031113 | 35 | 2050 | Flat slab, drop panels, job-built plywood, to 15' high, | SFCA | TOTAL | 0 | 0 | 0 | 145811.64 | 145811.64 |
| | | | 2 use | | LL | | | | | 0 |
| | | | | | LR | | | | | 0 |
| | | | | | UL | | | | | 0 |
| | | | | | UR | | | | | 0 |
| 031113 | 35 | 2250 | Flat slab, drop panels, job-built plywood, 15'-20' high | SFCA | TOTAL | 222618 | 70517.91 | 117455.8 | 0 | 410591.73 |
| | | | 4 use | | LL | | | | | 0 |
| | | | | | LR | | | | | 0 |
| | | | | | UL | | | | | 0 |
| | | | | | UR | | | | | 0 |
| 031113 | 35 | 5500 | Slab box outs, over 10 S.F. | L.F. | TOTAL | 946.38 | 472.1 | 685.38 | 1044 | 3147.86 |
| | | | | | LL | 0 | 0 | 0 | 0 | 0 |
| | | | | | LR | 163.38 | 163.38 | 163.38 | 0 | 490.14 |
| | | | | | UL | 783 | 308.72 | 522 | 1044 | 2657.72 |
| | | | | | UR | 0 | 0 | 0 | 0 | 0 |
| 031113 | 35 | 7101 | Edge forms, 7" to 12" high, 4 use | L.F. | TOTAL | 3765 | 1412.5 | 2705 | 4090 | 11972.5 |
| | | | | | | | | | | 0 |
| | | | | | | | | | | 0 |
| | | | | | | | | | | 0 |

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | 0 |
| 031113 | 35 | 7500 | Depressed area forms to 12" high, 4 use | L.F. | TOTAL | 0 | 0 | 0 | 3696 | 3696 |
| | | | | | | | | | | 0 |
| | | | | | | | | | | 0 |
| | | | | | | | | | | 0 |
| | | | | | | | | | | 0 |
| 031113 | 35 | 8000 | Perimeter deck and rail for elevated slabs, straight | L.F. | TOTAL | 3765 | 1412.5 | 2705 | 4090 | 11972.5 |
| | | | | | LL | | | | | 0 |
| | | | | | LR | | | | | 0 |
| | | | | | UL | | | | | 0 |
| | | | | | UR | | | | | 0 |
| 031113 | 85 | 2500 | Wall, job-built plywood, 8' to 16' high, 3 use | SFCA | TOTAL | 42909.99 | 0 | 0 | 21538.8 | 64448.7936 |
| | | | | | LL | 9626.734 | 0 | 0 | 8578.8 | 18205.5336 |
| | | | | | LR | 11156.4 | 0 | 0 | 0 | 11156.4 |
| | | | | | UL | 10970.46 | 0 | 0 | 7830 | 18800.46 |
| | | | | | UR | 11156.4 | 0 | 0 | 5130 | 16286.4 |
| 031113 | 85 | 2850 | Wall, job-built plywood, over 16' high, 4 use | SFCA | TOTAL | 66919.65 | 19356.88 | 28513.76 | 0 | 114790.2912 |
| | | | | | LL | 15013.23 | 5280.88 | 6481.76 | 0 | 26775.8712 |
| | | | | | LR | 17398.8 | 6120 | 12240 | 0 | 35758.8 |
| | | | | | UL | 17108.82 | 6018 | 5916 | 0 | 29042.82 |
| | | | | | UR | 17398.8 | 1938 | 3876 | 0 | 23212.8 |
| Shores | | | | | | | | | | |
| Section | | | Activity | Unit | Floor | Floor 1-3 | Floor 4 | Floor 5-6 | Floor 7-Pent | Total |
| 031505 | 70 | 0500 | Aluminum joists and stringers, spaced @ 2' O.C., per mo. | EA. | TOTAL | 943.5 | 1123.5 | 1890 | 2430 | 6387 |
| | | | | | LL | | | | | |
| | | | | | LR | | | | | |
| | | | | | UL | | | | | |
| | | | | | UR | | | | | |
| 031505 | 70 | 3600 | #3 post shore, 8'-10" to 16'-1" high, 3800# capacity | EA. | TOTAL | 943.5 | 1123.5 | 1890 | 2430 | 6387 |
| | | | | | LL | | | | | |
| | | | | | LR | | | | | |
| | | | | | UL | | | | | |
| | | | | | UR | | | | | |
| 031505 | 70 | 1500 | Reshoring | SF | TOTAL | 222618 | 70517.91 | 117455.8 | 145811.64 | 556403.37 |
| | | | | | LL | | | | | |
| | | | | | LR | | | | | |
| | | | | | UL | | | | | |
| | | | | | UR | | | | | |
| Rebar and Accessories | | | | | | | | | | |
| Section | | | Activity | Unit | Floor | Floor 1-3 | Floor 4 | Floor 5-6 | Floor 7-Pent | Total |
| 032111 | 60 | 0252 | Columns, #8-#18 | Lb. | TOTAL | 448988 | - | 128681.7 | 122822.1 | 700491.8 |
| | | | | | LL | | | | | |
| | | | | | LR | | | | | |
| | | | | | UL | | | | | |
| | | | | | UR | | | | | |
| 032111 | 60 | 0402 | Elevated slabs, #4-#7 | Lb. | TOTAL | 1152000 | 450200 | 734000 | 975200 | 3311400 |
| | | | | | LL | | | | | |
| | | | | | LR | | | | | |
| | | | | | UL | | | | | |
| | | | | | UR | | | | | |

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 032111 | 60 | 0702 | Walls, #3-#7 | Lb. | TOTAL | 267553 | | | | 267553 |
| | | | *Floor 1-3 includes below grade and level 4 | | LL | 145661 | | | | 145661 |
| | | | | | LR | 18114.5 | | | | 18114.5 |
| | | | | | UL | 77323 | | | | 77323 |
| | | | | | UR | 26454.5 | | | | 26454.5 |
| 032111 | 60 | 0752 | Walls, #8-#18 | Lb. | TOTAL | 116704.7 | | | | 116704.7 |
| | | | | | LL | 55959 | | | | 55959 |
| | | | | | LR | 20877.3 | | | | 20877.3 |
| | | | | | UL | 29705.4 | | | | 29705.4 |
| | | | | | UR | 10163 | | | | 10163 |
| Placing Concrete | | | | | | | | | | |
| Section | | | Activity | Unit | Floor | Floor 1-3 | Floor 4 | Floor 5-6 | Floor 7-Pent | Total |
| 033113 | 35 | 0400 | Heavyweight concrete, ready mix, 5000 psi | CY | TOTAL | 8764.4 | 3081 | 4968.5 | 6591.6 | 23405.5 |
| | | | *Flor 1-3 includes below grade | | LL | | | | | |
| | | | | | LR | | | | | |
| | | | | | UL | | | | | |
| | | | | | UR | | | | | |
| 033113 | 35 | 0411 | Heavyweight conrete, ready mix, 6000 psi | CY | TOTAL | 4504.5 | | | | 4504.5 |
| | | | | | LL | 909.3 | | | | 909.3 |
| | | | | | LR | 1351.5 | | | | 1351.5 |
| | | | | | UL | 1507.1 | | | | 1507.1 |
| | | | | | UR | 736.6 | | | | 736.6 |
| 033113 | 70 | 0800 | Columns, 24" thick, pumped | CY | TOTAL | 1210.4 | 259 | 518.5 | 507.6 | 2495.5 |
| | | | | | LL | | | | | |
| | | | | | LR | | | | | |
| | | | | | UL | | | | | |
| | | | | | UR | | | | | |
| 033113 | 70 | 1600 | Slabs over 10" thick, pumped | CY | TOTAL | 7554 | 2822 | 4450 | 6084 | 20910 |
| | | | | | LL | | | | | |
| | | | | | LR | | | | | |
| | | | | | UL | | | | | |
| | | | | | UR | | | | | |
| 033113 | 70 | 5350 | Walls, 15" thick, pumped | CY | TOTAL | 4504.5 | | | | 4504.5 |
| | | | | | LL | 909.3 | | | | 909.3 |
| | | | | | LR | 1351.5 | | | | 1351.5 |
| | | | | | UL | 1507.1 | | | | 1507.1 |
| | | | | | UR | 736.6 | | | | 736.6 |
| 033513 | 30 | 0125 | Bull float and manual float | SF | TOTAL | 222618 | 70517.91 | 117455.8 | 145811.64 | 556403.37 |
| | | | | | LL | | | | | |
| | | | | | LR | | | | | |
| | | | | | UL | | | | | |
| | | | | | UR | | | | | |

# B.2 Schedule References

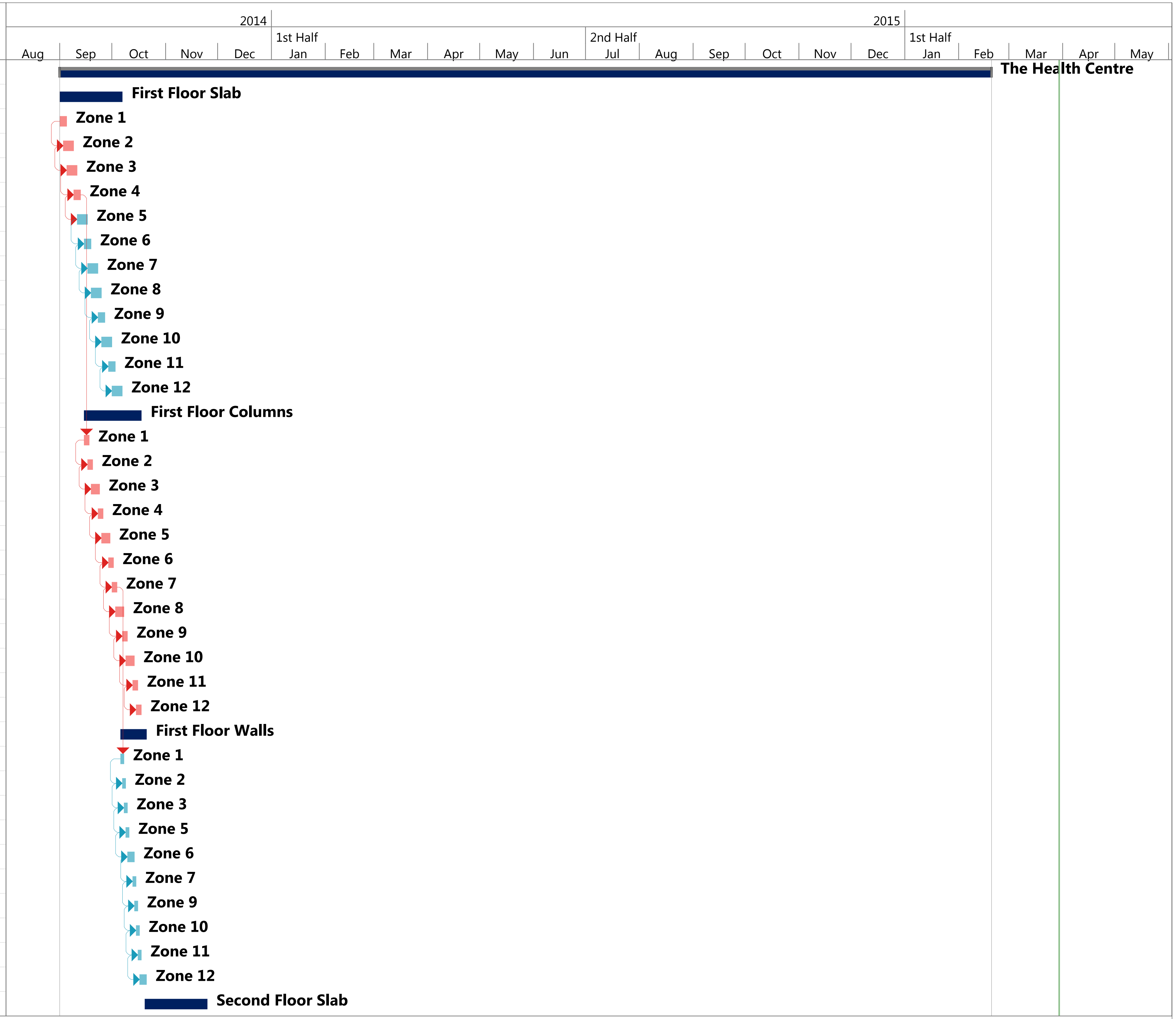
| ID | Task Name | Duration | Start | Finish |
|---|---|---|---|---|
| 0 | **The Health Centre** | **385 days** | **Mon 9/1/14** | **Fri 2/19/16** |
| 1 | **First Floor Slab** | **26 days** | **Mon 9/1/14** | **Mon 10/6/14** |
| 14 | **First Floor Columns** | **25 days** | **Mon 9/15/14** | **Fri 10/17/14** |
| 27 | **First Floor Walls** | **11 days** | **Mon 10/6/14** | **Mon 10/20/14** |
| 38 | **Second Floor Slab** | **26 days** | **Mon 10/20/14** | **Mon 11/24/14** |
| 51 | **Second Floor Columns** | **25 days** | **Wed 11/5/14** | **Tue 12/9/14** |
| 64 | **Second Floor Walls** | **12 days** | **Wed 11/26/14** | **Thu 12/11/14** |
| 76 | **Third Floor Slab** | **26 days** | **Wed 12/10/14** | **Wed 1/14/15** |
| 89 | **Third Floor Columns** | **25 days** | **Fri 12/26/14** | **Thu 1/29/15** |
| 102 | **Third Floor Walls** | **12 days** | **Fri 1/16/15** | **Mon 2/2/15** |
| 114 | **Fourth Floor Slab** | **30 days** | **Fri 1/30/15** | **Thu 3/12/15** |
| 129 | **Fourth Floor Columns** | **29 days** | **Tue 2/17/15** | **Fri 3/27/15** |
| 144 | **Fourth Floor Walls** | **8 days** | **Tue 3/10/15** | **Thu 3/19/15** |
| 152 | **Fifth Floor Slab** | **30 days** | **Mon 3/30/15** | **Fri 5/8/15** |
| 167 | **Fifth Floor Columns** | **29 days** | **Wed 4/15/15** | **Mon 5/25/15** |
| 182 | **Fifth Floor Walls** | **7 days** | **Wed 5/6/15** | **Thu 5/14/15** |
| 189 | **Sixth Floor Slab** | **30 days** | **Tue 5/26/15** | **Mon 7/6/15** |
| 204 | **Sixth Floor Columns** | **29 days** | **Thu 6/11/15** | **Tue 7/21/15** |
| 219 | **Sixth Floor Walls** | **7 days** | **Thu 7/2/15** | **Fri 7/10/15** |
| 226 | **Seventh Floor Slab** | **22 days** | **Wed 7/22/15** | **Thu 8/20/15** |
| 237 | **Seventh Floor Columns** | **21 days** | **Fri 8/7/15** | **Fri 9/4/15** |
| 248 | **Seventh Floor Walls** | **5 days** | **Fri 8/28/15** | **Thu 9/3/15** |
| 253 | **Eighth Floor Slab** | **22 days** | **Fri 9/4/15** | **Mon 10/5/15** |
| 264 | **Eighth Floor Columns** | **21 days** | **Tue 9/22/15** | **Tue 10/20/15** |
| 275 | **Eighth Floor Walls** | **5 days** | **Tue 10/13/15** | **Mon 10/19/15** |
| 280 | **Ninth Floor Slab** | **22 days** | **Wed 10/21/15** | **Thu 11/19/15** |
| 291 | **Ninth Floor Columns** | **21 days** | **Fri 11/6/15** | **Fri 12/4/15** |
| 302 | **Ninth Floor Walls** | **5 days** | **Wed 11/25/15** | **Tue 12/1/15** |
| 307 | **Penthouse Floor Slab** | **22 days** | **Mon 12/7/15** | **Tue 1/5/16** |
| 318 | **Penthouse Columns** | **21 days** | **Wed 12/23/15** | **Wed 1/20/16** |
| 329 | **Penthouse Floor Walls** | **5 days** | **Thu 1/7/16** | **Wed 1/13/16** |
| 334 | **Penthouse Roof** | **22 days** | **Thu 1/21/16** | **Fri 2/19/16** |




Project: The Health Centre

| | | | | | |
|---|---|---|---|---|---|
| Task | Inactive Task | Manual Summary Rollup | External Milestone | Manual Progress |
| Split | Inactive Milestone | Manual Summary | Deadline | |
| Milestone | Inactive Summary | Start-only | Critical | |
| Summary | Manual Task | Finish-only | Critical Split | |
| Project Summary | Duration-only | External Tasks | Progress | |

| ID | Task Name | Duration | Start | Finish |
|---|---|---|---|---|
| 0 | **The Health Centre** | **385 days** | **Mon 9/1/14** | **Fri 2/19/16** |
| 1 | **First Floor Slab** | **26 days** | **Mon 9/1/14** | **Mon 10/6/14** |
| 2 | Zone 1 | 4 days | Mon 9/1/14 | Thu 9/4/14 |
| 3 | Zone 2 | 4 days | Wed 9/3/14 | Mon 9/8/14 |
| 4 | Zone 3 | 4 days | Fri 9/5/14 | Wed 9/10/14 |
| 5 | Zone 4 | 4 days | Tue 9/9/14 | Fri 9/12/14 |
| 6 | Zone 5 | 4 days | Thu 9/11/14 | Tue 9/16/14 |
| 7 | Zone 6 | 4 days | Mon 9/15/14 | Thu 9/18/14 |
| 8 | Zone 7 | 4 days | Wed 9/17/14 | Mon 9/22/14 |
| 9 | Zone 8 | 4 days | Fri 9/19/14 | Wed 9/24/14 |
| 10 | Zone 9 | 4 days | Tue 9/23/14 | Fri 9/26/14 |
| 11 | Zone 10 | 4 days | Thu 9/25/14 | Tue 9/30/14 |
| 12 | Zone 11 | 4 days | Mon 9/29/14 | Thu 10/2/14 |
| 13 | Zone 12 | 4 days | Wed 10/1/14 | Mon 10/6/14 |
| 14 | **First Floor Columns** | **25 days** | **Mon 9/15/14** | **Fri 10/17/14** |
| 15 | Zone 1 | 3 days | Mon 9/15/14 | Wed 9/17/14 |
| 16 | Zone 2 | 3 days | Wed 9/17/14 | Fri 9/19/14 |
| 17 | Zone 3 | 3 days | Fri 9/19/14 | Tue 9/23/14 |
| 18 | Zone 4 | 3 days | Tue 9/23/14 | Thu 9/25/14 |
| 19 | Zone 5 | 3 days | Thu 9/25/14 | Mon 9/29/14 |
| 20 | Zone 6 | 3 days | Mon 9/29/14 | Wed 10/1/14 |
| 21 | Zone 7 | 3 days | Wed 10/1/14 | Fri 10/3/14 |
| 22 | Zone 8 | 3 days | Fri 10/3/14 | Tue 10/7/14 |
| 23 | Zone 9 | 3 days | Tue 10/7/14 | Thu 10/9/14 |
| 24 | Zone 10 | 3 days | Thu 10/9/14 | Mon 10/13/14 |
| 25 | Zone 11 | 3 days | Mon 10/13/14 | Wed 10/15/14 |
| 26 | Zone 12 | 3 days | Wed 10/15/14 | Fri 10/17/14 |
| 27 | **First Floor Walls** | **11 days** | **Mon 10/6/14** | **Mon 10/20/14** |
| 28 | Zone 1 | 2 days | Mon 10/6/14 | Tue 10/7/14 |
| 29 | Zone 2 | 2 days | Tue 10/7/14 | Wed 10/8/14 |
| 30 | Zone 3 | 2 days | Wed 10/8/14 | Thu 10/9/14 |
| 31 | Zone 5 | 2 days | Thu 10/9/14 | Fri 10/10/14 |
| 32 | Zone 6 | 2 days | Fri 10/10/14 | Mon 10/13/14 |
| 33 | Zone 7 | 2 days | Mon 10/13/14 | Tue 10/14/14 |
| 34 | Zone 9 | 2 days | Tue 10/14/14 | Wed 10/15/14 |
| 35 | Zone 10 | 2 days | Wed 10/15/14 | Thu 10/16/14 |
| 36 | Zone 11 | 2 days | Thu 10/16/14 | Fri 10/17/14 |
| 37 | Zone 12 | 2 days | Fri 10/17/14 | Mon 10/20/14 |
| 38 | **Second Floor Slab** | **26 days** | **Mon 10/20/14** | **Mon 11/24/14** |




Project: The Health Centre

| | | | | | |
|---|---|---|---|---|---|
| Task | Inactive Task | Manual Summary Rollup | External Milestone | Manual Progress | |
| Split | Inactive Milestone | Manual Summary | Deadline | | |
| Milestone | Inactive Summary | Start-only | Critical | | |
| Summary | Manual Task | Finish-only | Critical Split | | |
| Project Summary | Duration-only | External Tasks | Progress | | |

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# ACADEMIC VITAE

## Hannah N. Valentine

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**Objective**

Pursue a full-time structural engineering position for the summer of 2016 and obtain a Professional Engineer license after working in practice.

**Education**

The Pennsylvania State University, University Park, PA
Schreyer Honors College
B.A.E./M.A.E. Architectural Engineering | ABET Accredited
Structural Option | EIT Upon Graduation
Architectural Studies Minor

The Pantheon Institute, Rome, Italy — Summer 2014

- Studied 12 credits of Penn State courses including daylighting, cartography, and architecture

**Work Experience**

SmithGroupJJR | Structural Intern, Detroit MI — Summer 2015

- Modeled canopy, mechanical support framing, and other structures in RISA-3D
- Reviewed steel shop drawings for 4 sequences of steel beams, columns & connections
- Prepared hand calculations for wind, seismic, and façade loading analysis
- Worked with senior through graduate engineers on interdisciplinary coordination in Revit

The Pike Company | Project Engineer Intern, Cornell University, NY — Summer 2013

- Managed on-site project drawing updates, daily RFIs, and the project punchlist for the Cornell Food Science building
- Coordinated orders of supplies and delivered bids for local projects

Delta Engineers | Transportation Intern, Endwell, NY — Summer 2012

- Completed road surveys for the Road Protection Program
- Checked calculations, completed pick-up work in Microstation, and collected project bids for Transportation department projects

**Involvement**

Structural Engineer's Association (SEA), President — 2015-2016

- Planned meetings, guest speakers, and annual trip to the American Institute of Steel Construction conference

Global Architecture Brigades, Secretary — 2011-2013

- Assisted with planning trip to rural Honduras to work on the construction of a medical center and a secondary school with the local communities
- Constructed compost latrines in an indigenous Kuna community in Darien, Panama

**Awards**

- Schreyer Honors College Academic Excellence Scholarship
- William and Wyllis Leonhard University Scholarship
- Recipient of the Thomas J. Watson Memorial Scholarship

**Organizations**

- Student Society of Architectural Engineers (SSAE)
- American Institute of Steel Construction (AISC)
- Engineering Orientation Network Mentor
- National Association of Home Builders 2012 Construction Competition

**SofVaware**

RAM | SAP2000 | RISA 3D | ETABS | STADD | Revit | Newforma | AutoCAD | Sketchup

**Relevant Courses**

Advanced Steel and Concrete Design | Interdisciplinary BIM Studio | Steel Connections
Computer Modeling of Buildings | Earthquake Resistant Design | Building Failures