# OFFICE BUILDING-G

Eastern United States

# Technical Report I


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Structural Option

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October 4, 2010

# Table of Contents

## Executive Summary

Technical Report I is a detailed description of the structural concepts and existing conditions of Office Building-G. It provides insight into the structural system design and functionality as well as the loading conditions. Due to owner restrictions certain information (i.e. the owner, building name, occupant) cannot be disclosed.

This 14-story high rise office building, located in the eastern United States, is built on top of a 4-story parking garage, both of which are supported by a concrete bearing wall core with concrete edge columns. The floor system is a one way flat plate slab which is supported by post tensioned beams. Lateral forces on the building are resisted by the concrete core that also functions as a shear wall core.

Wind and seismic design forces were determined using ASCE 7-10. When compared, it was concluded that seismic forces govern the design except in the North-South direction in which the wind base shear controls. Base shear in the North-South is 1370 kips and in the East-West it totals to 1198 kips. Overturning moments are controlled by seismic forces in all directions with a value of 154,790 ft-kips. These loads are from the shear wall core into enlarged spread footings below the bottom level of the parking garage's slab on grade.

Gravity load spot checks were performed on a typical slab, post-tensioned beam and column. Any discrepancies found between member sizes or strengths were analyzed and possible explanations were explored. These members will have to be checked again when the lateral forces are included in the analysis.

# Introduction

## Building Description

As noted above, Office Building-G is a 14-story office building with a 4-story parking garage below the superstructure. The roof of the mechanical penthouse is 195 ft above grade, categorizing this building as a high rise structure for its location. A typical office floor has a gross square footage of 25,376 sf and when the total superstructure and garage are added, Office Building-G is 649,461 sf.

The southern façade of the building is a curved glass curtain wall, breaking the mold of precast concrete panels the other three sides of the building follow. On the first and second floor there is a restaurant which has a glass façade with concrete pilasters between the panes of glass. Figures 1 is a view of the South-West corner of the building. The red lines outline the restaurant while the blue show the extents of the parking garage.

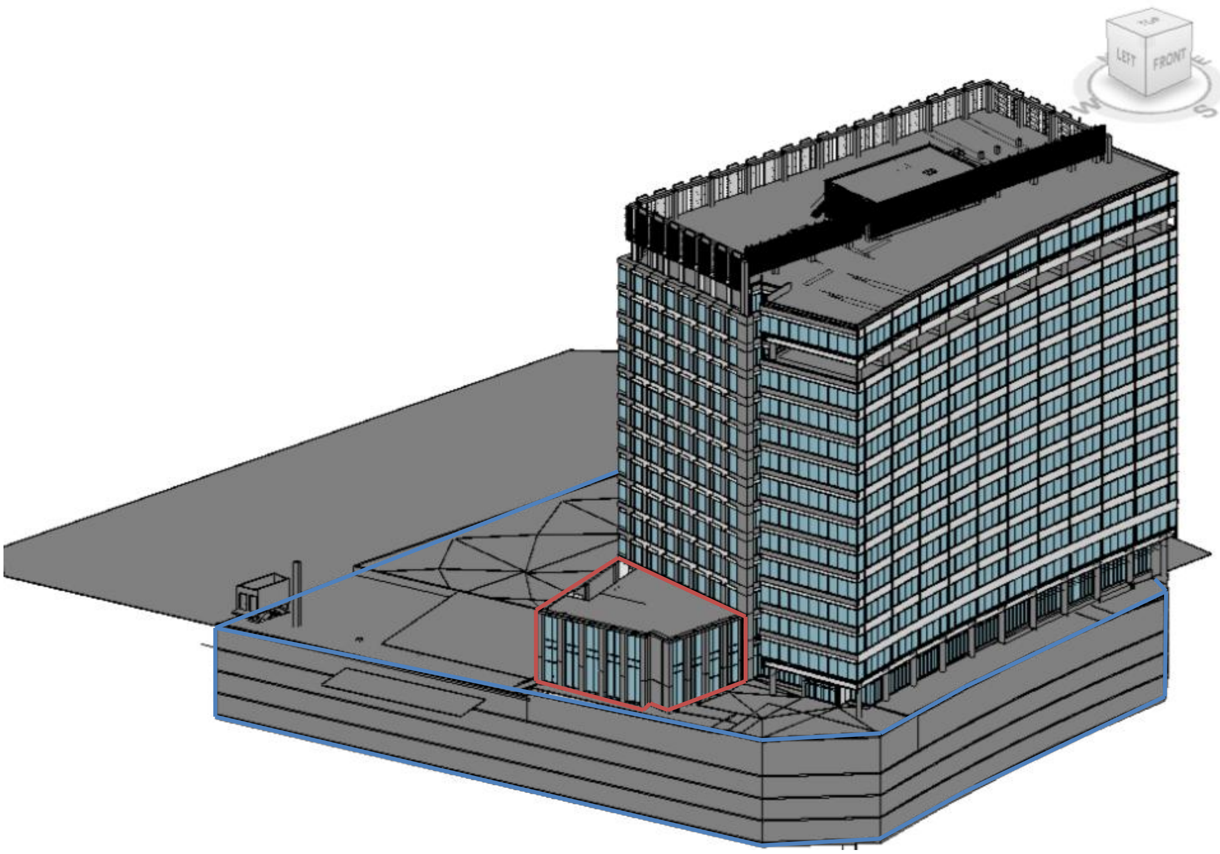


Figure 1

Directly to the right of the restaurant Figure 1 is the main entrance of the building. The upscale lobby, along with the entire first floor, has a 17 ft floor to floor height, compared to the typical height of 12 ft 3 in. Figure 1 also shows how the perimeter columns supporting the glass façade are on the exterior of the building on the first floor due to a setback on the first floor.

Figure 2 is a view of the North-East corner of Office Building-G. Again, the extents of the below grade parking garage are outlined in blue. Other building aspects displayed in this figure are the bank in green, the loading dock in red, architectural screen wall in purple and the mechanical penthouse in orange.

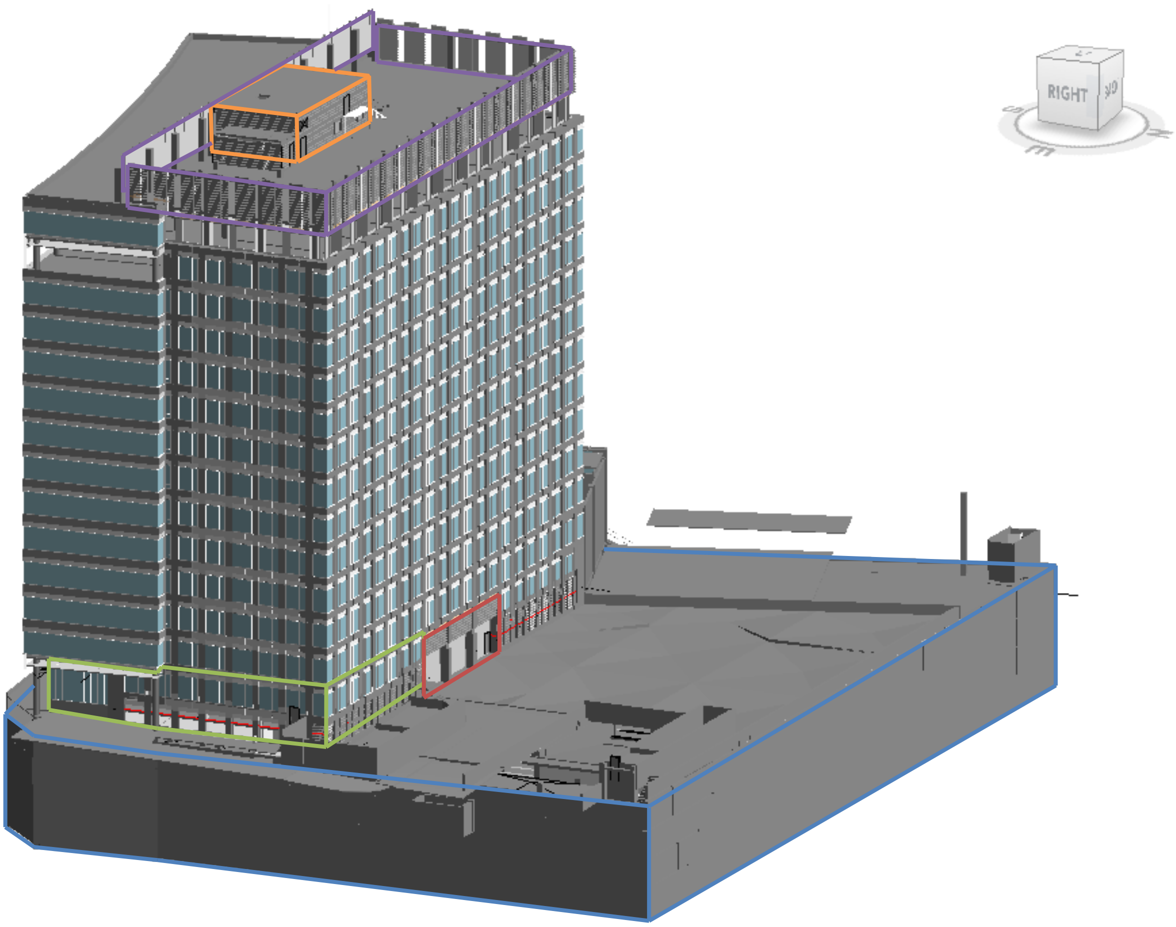



**Figure 2**

Figure 4 is a typical floor plan which is followed for the majority of the structure. On the 12th and 13th floors of the building the exterior columns slope in creating a slightly smaller square footage for the 13th and 14th floors. Other building features are described throughout the report as needed.

## Structural Introduction

The curved glass curtain wall on the southern façade of Office Building-G disguises the framing but a reinforced concrete structure is suspected due to the scale of the precast concrete used to frame the windows on the other elevations. Structural plans confirm the suspicions as they reveal circular and rectangular concrete columns with typical 20' x 45' bays. Post-tensioned beams run north-south between columns and rely on one-way slab construction to span from beam to beam. There is an internal concrete shear wall core extending the entire height of Office Building-G. This core houses the majority of the vertical transportation within the building while providing lateral stiffness and rotational resistance.

Spread footings ranging in sizes of 4' x 4' to 15' x 15' act as the primary foundation for Office Building-G. The concrete exterior wall of the parking garage is supported by a wall footing around the entire perimeter of the building. Two large footings (approximately 40' x 40' and 40' x 10') are located below the internal shear walls. All foundations are required to be a minimum of 1' below the low point of the 5" normal weight slab on grade.

The structural engineer's design of the structure uses concrete strengths between 3000 psi and 10,000 psi. A small amount of structural steel is used to support the elevators but the rest of the building is concrete. 5000 psi concrete is primarily used in the slab construction but 8000psi concrete is utilized below columns and shear walls to account for any punching shear requirements. Concrete strength of the columns and shear walls vary with the number of floors above them. Cast in place beams use 5000 psi normal weight concrete and are reinforced with Grade 60 steel as per ACI 318 requirements. The post-tensioned beams use 5000 psi concrete with seven wire stress relieved strand with a minimum ultimate tensile strength of 270 ksi.

The following report contains an in-depth evaluation of the structural concepts and the existing conditions of Office Building-G. Included within this report is an overview of the design codes and requirements as well as determinations of the loads used by the project engineer. This information is used to perform an analysis of the lateral wind and seismic loads as well as spot checks of typical column gravity loads.

# Structural Systems

## Gravity System:

Gravity loads are carried down the building through a combination of interior and exterior concrete columns and a shear wall core. The typical floor system is a cast in place concrete flat slab. Thickness changes based on loading conditions but the typical floor is a one-way, 7”, 5000 psi normal weight concrete flat slab. On the first floor there is a 12” concrete slab designed for fire separation between the parking garage and superstructure. The slab system carries the loads to post-tensioned concrete beams with spans between 41’-5” and 45’-1 1/4”.

The post tensioned beams range in width from 18” to 48” and have a maximum depth of 24”. Forces in the beams are between 162 kips to 675 kips. These beams collect the floor loads from the slab and distribute their reactions to the columns supporting them.

Rectangular and round concrete columns then transfer the loads down the strictly followed grid. Typical floors have columns sizes of 24” x 24”, 24” x 30”, and 30” diameter. Smaller columns are used in the mechanical penthouse due to the much lower loads they are carrying. On above grade floors, higher strength concrete is placed below columns and shear walls to accommodate for any possibility of punching shear. In the parking garage, 8” drop panels are used instead of the different concrete strengths. Figure 3 and 4 below highlights the post-tensioned beams in yellow, the reinforced beams in purple, and the columns in red.

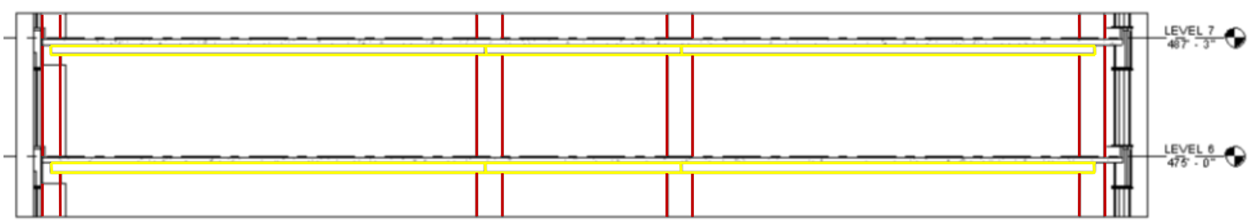



Figure 3

## Lateral System:

Wind and seismic forces are resisted by an internal shear wall core. The core is made of reinforced concrete walls which have a consistent floor plan from the bottom floor of the parking garage up to the slab of the roof. Basement shear walls were designed with f’c = 10,000 psi, levels 1-4 use f’c = 8,000, and levels 5-14 use f’c = 5,000. Precast concrete beams attached to concrete columns using precast lateral connections provide the required resistance for the mechanical penthouse and elevator machine room.

The magnitude of the lateral force due to wind is proportional to the length of the building normal to the direction of the force. Due to the larger width of the North-South elevations, the lateral force experienced in this direction is going to be larger than that of the East-West direction. To compensate for the larger forces there is 85’ of 12” thick wall in the N/S direction compared to the 65’ of 12” wall in the E/W.

Lateral forces are engaged by the shear walls through the use of floor diaphragms. The building façade collects wind forces that are then transferred to the respective floor diaphragm. Forces then travel through the diaphragm until the shear walls are engaged, at which point the forces are distributed based on the relative stiffness of the walls. An estimate of amount of force in a shear wall can be performed using the tributary area of the façade for each shear wall. Figure 4 below is a typical floor plan with the lateral system highlighted in both directions. Blue lines represent concrete shear walls in the North-South Direction while green lines represent the walls in the East-West direction.

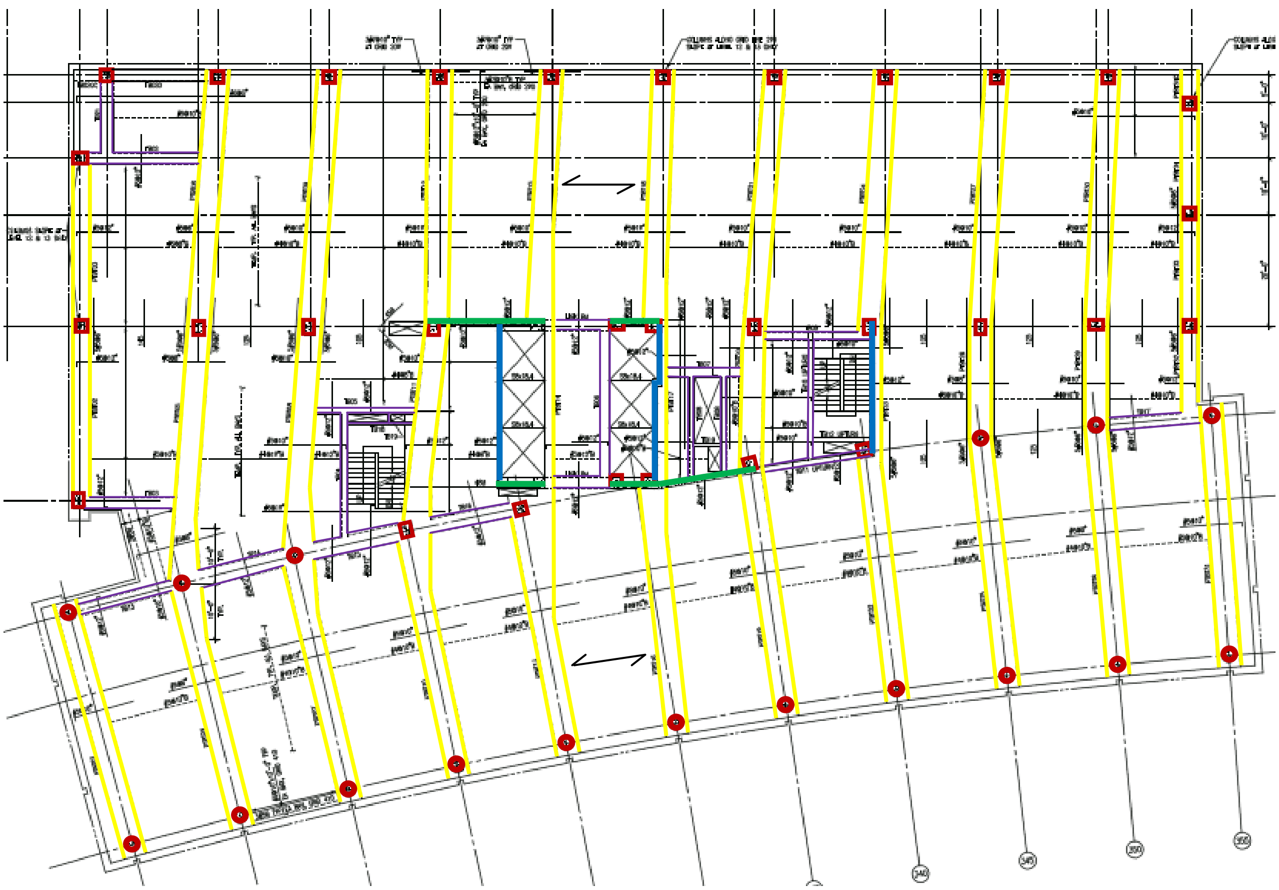


Figure 4

Figure 5 is a detailed plan of the shear wall core that extents the entire height of the building. Link beams responsible for the East-West shear walls to act together when loaded are highlighted in this figure. The boundary elements are also more visible in this graphic. These end conditions provide lateral confinement and allow a vertical couple to form, resisting the moment induced by lateral forces.

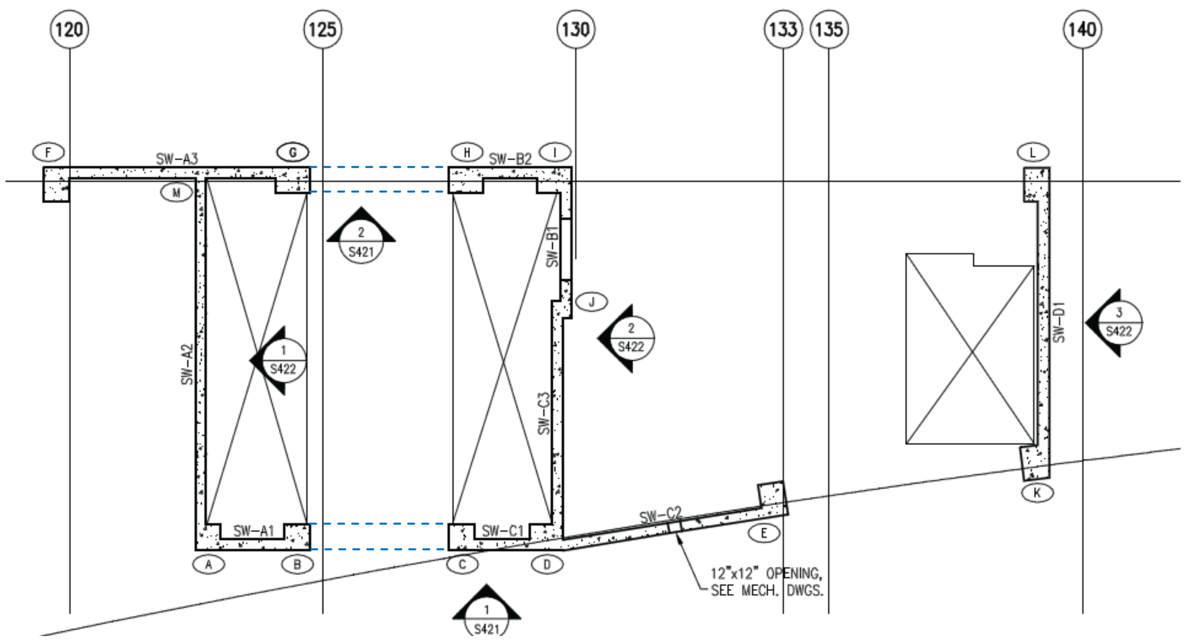



Figure 5

## Foundation System:

The geotechnical engineer performed a geotechnical study for the location of Office Building-G which determined the possible foundation systems as spread footings, caissons or geopiers. The structural engineer decided to use a system of spread footings under the columns, shear walls and along the perimeter concrete bearing wall. Square footage and depth of the footings are based on the load carrying capability of the soil and the vertical load on the column.

Service loads on the columns ranged greatly depending on whether or not the column extended up into the super structure of the building. Based on the structure above the foundation, the load capacity of soil was engineered to support a range of 3,000 psf to 10,000 psf. Loads on the footings varied between 60 kips to 3075 kips, once again depending on which part of Office Building-G they are supporting. Figure 6 outlines major foundations in relation with the size of the load they carry and whether or not they carry super structure loads. Blue foundations carry the load of the entire structure while green foundations only carry loads associated with the parking garage.

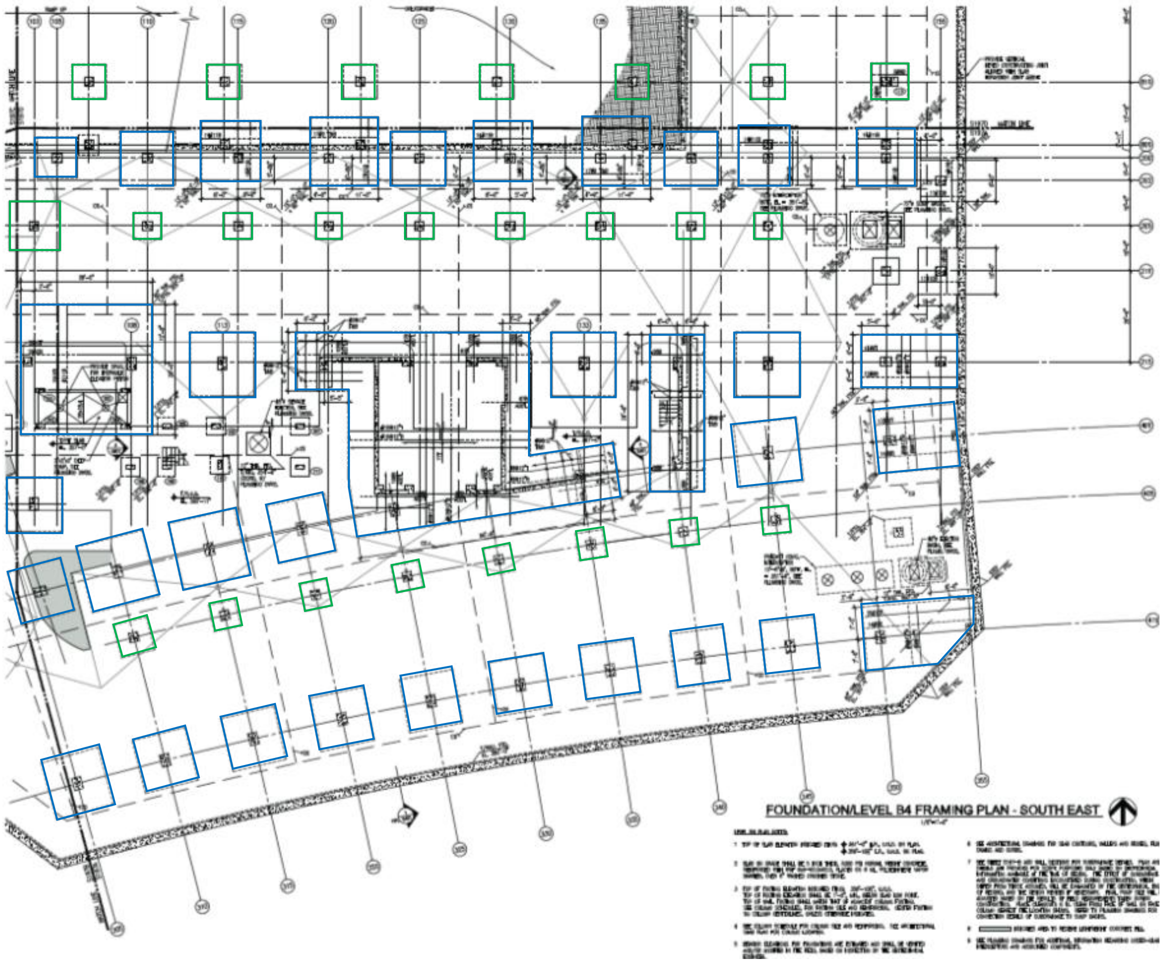



Figure 6

Note on foundations: Shear walls have been identified as the lateral force resistance system for Office Building-G. After the walls collect the lateral forces, an overturning moment is formed at the structures foundation which must be designed for. It is clear that the structural engineer took overturning into consideration with the large spread footings under the shear wall core.

## Structural Materials

| Material | Element | Level | Strength |
|---|---|---|---|
| **Cast-in-Place Concrete** | Spread Footings | Foundation | $f'_c$ = 10,000 psi |
| | | | $f'_c$ = 6,000 psi |
| | | | $f'_c$ = 3,000 psi |
| | Foundation Walls | B4 | $f'_c$ = 5,000 psi |
| | | B3-B1 | $f'_c$ = 4,000 psi |
| | Shear Walls | B4-B1 | $f'_c$ = 10,000 psi |
| | | L1-L4 | $f'_c$ = 8,000 psi |
| | | L5-L7 | $f'_c$ = 6,000 psi |
| | | L8-L14 | $f'_c$ = 5,000 psi |
| | Columns | B4-B1 | $f'_c$ = 10,000 psi |
| | | | $f'_c$ = 6,000 psi |
| | | L1-L4 | $f'_c$ = 10,000 psi |
| | | | $f'_c$ = 8,000 psi |
| | | | $f'_c$ = 6,000 psi |
| | | L5-L7 | $f'_c$ = 6,000 psi |
| | | L8-Roof | $f'_c$ = 5,000 psi |
| | Reinforced Beams | ALL | $f'_c$ = 5,000 psi |
| | Post-Tensioned Beams | ALL | $f'_c$ = 5,000 psi |
| **Tendons** | Post-Tensioned Beams | ALL | $F_u$= 270 ksi |
| **Reinforcing Steel** | Concrete | ALL | $F_y$ = 60 ksi |
| **Structural Steel** | Elevator Framing - A36 | ALL | $F_y$ = 36 ksi |
| | Bolts - A325 | ALL | $F_u$= 120ksi |

# Codes and Reference

## Design Codes:

National Model Code:

Local building code based on the 2006 International Building Code

Sections: 1603.1.1-1603.1.7, 1603.2, 1607.11, 1608.1, 1608.7, 1608.8, 1609.1

Design Codes:

American Concrete Institute (ACI) 318-08, Building Code Requirements for Structural Concrete and Commentary

ACI 301, Specifications for Structural Concrete for Buildings

ACI 347, Standard Recommended Practice for Concrete Formwork

American Institute for Steel Construction (AISC), Specification for the design, fabrication and erection of structural steel for buildings

## Thesis Codes:

National Model Code:

International Building Code, 2006

Design Codes:

ACI 318-08, Building Code Requirements for Structural Concrete and Commentary

Structural Standards:

American Standards of Civil Engineers (ASCE), ASCE 7-10 Minimum Design Loads for Buildings and Other Structures

# Gravity Loads

## Live Loads

A comparison between the live loads used by the designer, ASCE 7-10 minimums, and the thesis loads can be found below. ASCE 7-10 does not cover all of the possible loading conditions that present themselves in the design of a building so reasonable assumptions must be made to ensure a safe design. The loads used by the structural engineer were taken from the local building code which was based on the 2006 International Building Code. Most of the loads found in ASCE 7-10 were found to coincide with the designer loads but there were a few instances where ASCE 7-10 permitted a reduction. An office live load of 100 psf was used to account for a 20 psf partition load allowing for a universal use of the floor plan.

**Floor Live Loads**

| Description | Designer | ASCE 7-10 | Thesis |
|---|---|---|---|
| Parking Areas | 50 | 40 | 40 |
| Slab on Grade | 100 | 100 | 100 |
| Office Areas | 100 | 80 | 100 |
| Ground Floor Retail Areas | 100 | 100 | 100 |
| Vestibules, Corridors, Lobbies, Stairs | 100 | 100 | 100 |
| Terraces | 100 | 100 | 100 |
| Mechanical Room | 150 | - | 150 |
| Pump Room | 150 | - | 150 |
| Penthouse Floor | 150 | - | 150 |
| Elevator Machine Room | 125 | - | 125 |
| Truck Bays | 350 | 250 | 250 |
| Areas Accessible by Fire Fighting Equipment | 350 | - | 350 |

## Dead Loads

The dead loads used throughout design as well as a comparison to the thesis loads used can be found below. A more detailed description of how the dead loads were calculated can be found in Appendix A. Although partition loads were accounted for in the live loads and dead loads they were not double counted. The 20 psf partition load was included in the determination of seismic loads due to the assumption that in the event of an earthquake the partitions would respond with the building, thus not acting like a live load.

| Dead Loads | | | |
|---|---|---|---|
| Description | Designer | Superimposed | Thesis |
| Concrete | 150 | - | 150 |
| Partitions | 20 | - | 20 |
| MEP | - | 15 | 15 |
| Precast Panels | - | 30 | 25 |
| Curtain Glass | - | 15 | |

## Snow Loads

Chapter 7 of ASCE 7-10 was referenced for the calculation of the snow loads for Office Building-G. Due to different importance factors and other load reducing coefficients, a significant difference is noticeable in the designer snow loads and the thesis snow loads. These can be viewed in the table below.

| Snow Loads | | |
|---|---|---|
| **Factor** | **Designer** | **Thesis** |
| Exposure | B | B |
| $P_g$ | 30 | 25 |
| $C_e$ | 0.9 | 0.9 |
| $C_t$ | 1 | 1 |
| $I_s$ | 1 | 0.8 |
| $p_f$ | **27** | **12.6** |

An additional cause for the thesis snow load to lower than the designer load is a 0.7 multiplier applied to the thesis calculation per ASCE equation 7.3-1. The snow drift calculations can be viewed in the below tables and the additional hand calculations for all of the snow loads determined can be viewed in the Appendix A.

| Roof Snow Drift | | |
|---|---|---|
| **Factor** | **Coefficient** | **Reference** |
| $l_u$ | 50 | F 7-8 |
| $p_g$ | 25 | 7.3-1 |
| $\gamma$ | 17.25 | 7.7-1 |
| $h_b$ | 0.73 | F 7-8 |
| $h_d$ | 2.2 | F 7-9 |
| $h_c$ | 16.02 | F 7-9 |
| w | 8.8 | 7.7.1 |
| $p_d$ | 37.95 | 7.7.1 |

| Level 2 Roof Drift | | |
|---|---|---|
| **Factor** | **Coefficient** | **Reference** |
| $l_u$ | 204 | F 7-8 |
| $p_g$ | 25 | 7.3-1 |
| $\gamma$ | 17.25 | 7.7-1 |
| $h_b$ | 0.73 | F 7-8 |
| $h_d$ | 4 | F 7-9 |
| $h_c$ | 143 | F 7-9 |
| w | 16 | 7.7.1 |
| $p_d$ | 69 | 7.7.1 |

When combining the snow drift with the flat roof snow load, there is no instance when the snow load is greater than 100 psf. With this in mind, the 100 psf live loads from the previous load determinations should continue to be used in all locations on the roofs.

# Lateral Loads

## Wind:

ASCE 7-10 was used for the determination of the wind loads for the Main Wind-Force Resisting System (MWRFS) of Office Building-G. Loads were calculated in the North-South and in the East-West direction due to the roughly rectangular shape of the building. The forces were determined using the Chapter 27 guidelines for Enclosed and Partially Enclosed Rigid Buildings.

The first step in calculating wind loads is determining if the building is flexible or rigid. This classification is based on the natural frequency of the structure. ASCE 7-10 allows for an estimation of a buildings frequency based on relationships between the building height and characteristics of the lateral force resisting system. Through this estimation it was determined that the natural frequency of Office Building-G > 1 defining the building as rigid.

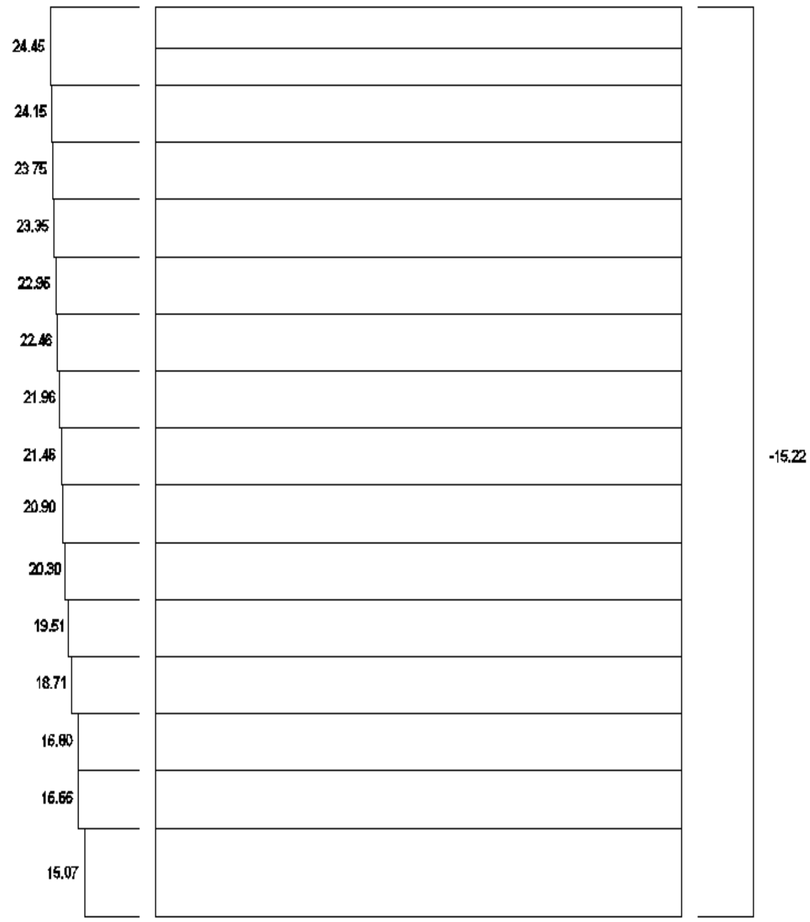
The building is fairly square on three sides but the curved southern façade creates a scenario where the West wall has a greater length than the East wall. If the curvature had been so severe that the West wall was wider than the North wall is deep, an additional wind load would have had to be calculated. Since this is not the case and L/B < 1 a single wind load calculation can be used for both the East and West loads. Using the similar rationale, the North-South wind loads were calculated using the worst case for the different geometries of the building. The building receives the largest wind force in the North-South directions, as these are the longer façades of the building normal to the wind.

The final step in analyzing the wind loads is to compare them to the seismic calculations. This is accomplished by calculating the base shear and overturning moments acting on the building. A comparison between these two forces is performed to determine the governing lateral force for design. Reference Appendix A tor a complete set of values, tables, references and equations used to calculate the design wind pressures and forces.

Note: There is an architectural screen panel that extends the entire length of the building. For this reason, the effect that the roof parapet would have on the wind load on the top of the structure was not calculated.

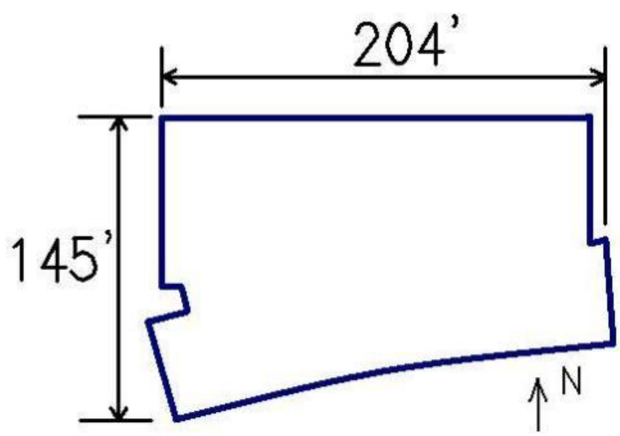
**<u>North-South Wind Distribution</u>**

| North-South | | | |
|---|---|---|---|
| Story Level | Story Height (ft) | Windward pz (psf) | Leeward ph (psf) |
| Roof | 195 | 24.45 | -15.22 |
| Elevator | 186.0 | 24.45 | -15.22 |
| Penthouse | 178.3 | 24.15 | -15.22 |
| 14 | 166.0 | 23.75 | -15.22 |
| 13 | 153.8 | 23.35 | -15.22 |
| 12 | 141.5 | 22.95 | -15.22 |
| 11 | 129.3 | 22.46 | -15.22 |
| 10 | 117.0 | 21.96 | -15.22 |
| 9 | 104.8 | 21.46 | -15.22 |
| 8 | 92.5 | 20.90 | -15.22 |
| 7 | 80.3 | 20.30 | -15.22 |
| 6 | 68.0 | 19.51 | -15.22 |
| 5 | 55.8 | 18.71 | -15.22 |
| 4 | 43.5 | 16.80 | -15.22 |
| 3 | 31.3 | 16.66 | -15.22 |
| 2 | 19.0 | 15.07 | -15.22 |
| 1 | 0 | 15.07 | -15.22 |




**L=204’** **B=145’**

The larger B value of Office Building-G’s dimensions was used in order to decrease the L/B ratio. The $C_p$ multiplier used when calculating Leeward Wind forces is inversely related to this ratio so when L/B decrease, $C_p$ and thus the wind force, increase.

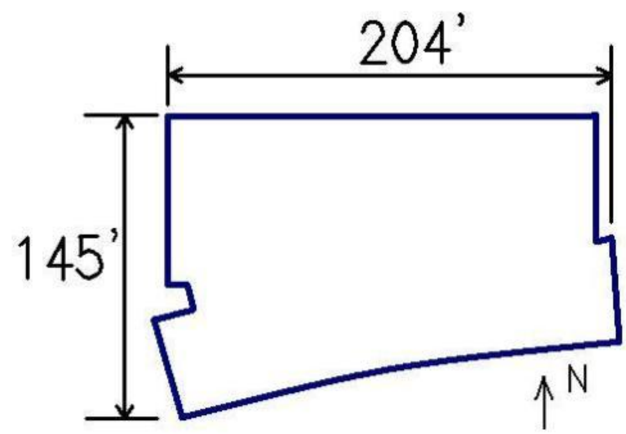


## East-West Wind Distribution

| East-West | | | |
|---|---|---|---|
| Story Level | Story Height (ft) | Windward pz (psf) | Leeward ph (psf) |
| Roof | 195 | 24.45 | -17.20 |
| Elevator | 186.0 | 24.45 | -17.20 |
| Penthouse | 178.3 | 24.15 | -17.20 |
| 14 | 166.0 | 23.75 | -17.20 |
| 13 | 153.8 | 23.35 | -17.20 |
| 12 | 141.5 | 22.95 | -17.20 |
| 11 | 129.3 | 22.46 | -17.20 |
| 10 | 117.0 | 21.96 | -17.20 |
| 9 | 104.8 | 21.46 | -17.20 |
| 8 | 92.5 | 20.90 | -17.20 |
| 7 | 80.3 | 20.30 | -17.20 |
| 6 | 68.0 | 19.51 | -17.20 |
| 5 | 55.8 | 18.71 | -17.20 |
| 4 | 43.5 | 16.80 | -17.20 |
| 3 | 31.3 | 16.66 | -17.20 |
| 2 | 19.0 | 15.07 | -17.20 |
| 1 | 0 | 15.07 | -17.20 |

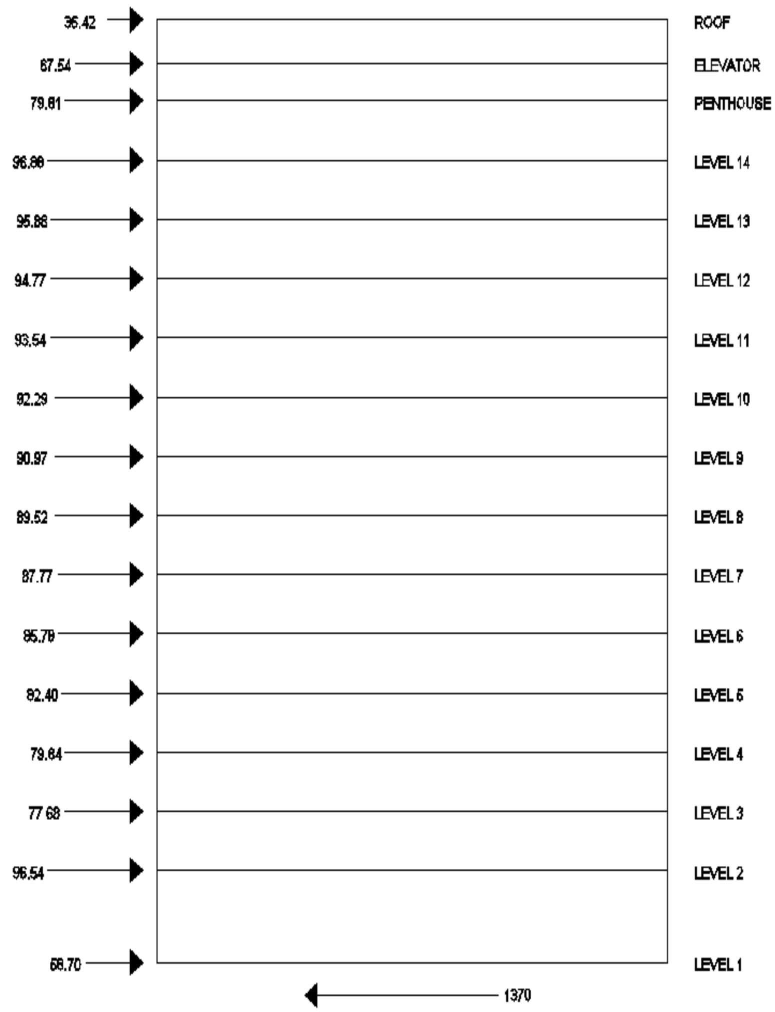
**L=145’** **B=204**

As noted above, a conservative value for the L/B ratio should be used to ensure the greatest value of $C_p$. In this particular instance the L value used does not matter because both ratios (either 114’/204’ or 145’/204’) result in a value of less than one, resulting in a $C_p$ value of -0.5.



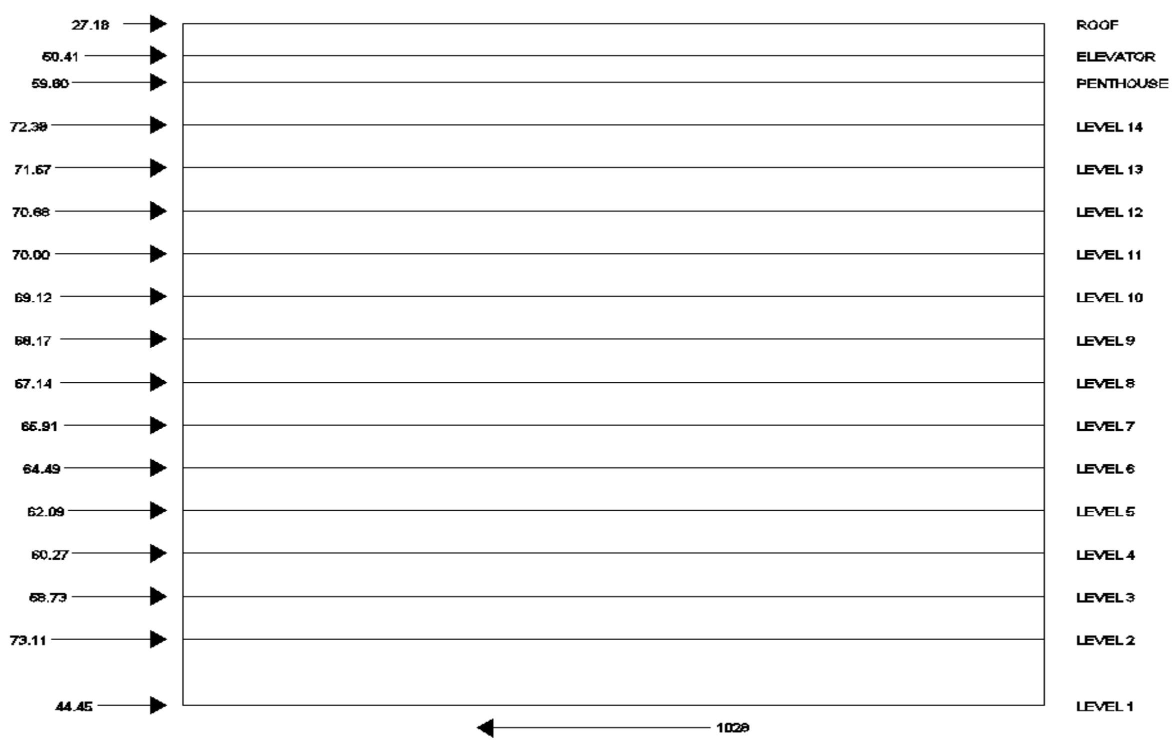

## North-South Wind Story Forces

| North-South | | | |
|---|---|---|---|
| Story Level | Story Height (ft) | Story Shear (Kips) | Moment (k-ft) |
| Roof | 195 | 36.42 | 7101 |
| Elevator | 186.0 | 67.54 | 12562 |
| Penthouse | 178.3 | 79.81 | 14226 |
| 14 | 166.0 | 96.88 | 16082 |
| 13 | 153.8 | 95.88 | 14742 |
| 12 | 141.5 | 94.77 | 13410 |
| 11 | 129.3 | 93.54 | 12090 |
| 10 | 117.0 | 92.29 | 10798 |
| 9 | 104.8 | 90.97 | 9529 |
| 8 | 92.5 | 89.52 | 8280 |
| 7 | 80.3 | 87.77 | 7044 |
| 6 | 68.0 | 85.78 | 5833 |
| 5 | 55.8 | 82.40 | 4594 |
| 4 | 43.5 | 79.84 | 3473 |
| 3 | 31.3 | 77.68 | 2427 |
| 2 | 19.0 | 96.54 | 1834 |
| 1 | 0 | 58.70 | 0 |
| Total Base Shear = | | 1369.92 | |
| Total Overturning Moment (k-ft) = | | | 136924.9 |



## East-West Wind Story Forces

| East-West | | | |
|---|---|---|---|
| Story Level | Story Hight (ft) | Story Shear (Kips) | Moment (k-ft) |
| Roof | 195 | 27.18 | 5299 |
| Elevator | 186.0 | 50.41 | 9376 |
| Penthouse | 178.3 | 59.60 | 10623 |
| 14 | 166.0 | 72.38 | 12015 |
| 13 | 153.8 | 71.67 | 11019 |
| 12 | 141.5 | 70.88 | 10030 |
| 11 | 129.3 | 70.00 | 9048 |
| 10 | 117.0 | 69.12 | 8087 |
| 9 | 104.8 | 68.17 | 7141 |
| 8 | 92.5 | 67.14 | 6211 |
| 7 | 80.3 | 65.91 | 5289 |
| 6 | 68.0 | 64.49 | 4385 |
| 5 | 55.8 | 62.09 | 3461 |
| 4 | 43.5 | 60.27 | 2622 |
| 3 | 31.3 | 58.73 | 1835 |
| 2 | 19.0 | 73.11 | 1389 |
| 1 | 0 | 44.45 | 0 |
| Total Base Shear = | | 1028.41 | |
| Total Overturning Moment (k-ft) = | | | 107830.34 |



## Seismic:

The Equivalent Lateral Force Procedure is in chapters 11 and 12 of ASCE 7-10 and these were referenced during the calculation of the seismic loads for Office Building-G. General design parameters of the building are a site classification of type D, a seismic design category of B, and a seismic importance factor of 1.0.

The first step taken in determining the seismic forces of the building was to determine the seismic response coefficient; $C_s$. $C_s$ is based on a variety of factors that take into account the lateral system of the building as well as its geographical. The lateral system of the building is classified as ordinary reinforced concrete shear walls, corresponding to a response modification factor of R=5. When determined, Cs can then be multiplied by the total dead load weight of the building to yield the seismic base shear.

The next step was to consider all of the possible areas that could contribute to the dead weight of the building. The building elements considered were: slabs, beams, columns, shear walls, exterior walls, partitions, and imposed MEP loads. These loads were either a pound per square foot or a total per floor, depending on the nature of the element. It should be noted that partitions included in a 100 psf live load for office space but since they are secured to the floor of the structure it was assumed that they will not move freely in the instance of an earthquake.

A typical floor plan was used to determine many of the weights calculated. This yielded a very reasonable estimate because Office Building-G follows a typical floor plan design and variations are uncommon and minor.

Shear forces and the corresponding overturning moments at each floor were calculated and the total of these forces can be compared to the wind forces. The table below displays the story force, the total shear at that level, as well as the overturning moment caused by seismic forces. An additional table provides values of a rotational moment that could result due to a slight eccentricity of the seismic forces. These seismic loads are for both North-South and East-West directions. There would have been different seismic loads had one direction had a lateral system consisting of shear walls while the other utilized a different system like moment frames for example. Reference the Appendix A for a complete list of values used and calculations.

| Seismic Forces | | | | | | |
|---|---|---|---|---|---|---|
| Level | Height (ft) | Wx (k) | $w_i*h_i^k$ | fx (k) | $V_x$ (k) | Overturning Moment (k-ft) |
| Roof | 195 | 126 | 97,684 | 5 | 5 | 881 |
| Elevator | 186 | 421 | 307,496 | 14 | 19 | 2644 |
| Penthouse | 178 | 5457 | 3,777,032 | 175 | 193 | 31127 |
| 14 | 166 | 5300 | 3,353,492 | 155 | 348 | 25737 |
| 13 | 154 | 5300 | 3,044,317 | 141 | 489 | 21640 |
| 12 | 142 | 5300 | 2,741,529 | 127 | 616 | 17935 |
| 11 | 129 | 5300 | 2,445,532 | 113 | 729 | 14614 |
| 10 | 117 | 5300 | 2,156,798 | 100 | 829 | 11667 |
| 9 | 105 | 5300 | 1,875,881 | 87 | 915 | 9085 |
| 8 | 93 | 5300 | 1,603,449 | 74 | 990 | 6857 |
| 7 | 80 | 5300 | 1,340,323 | 62 | 1052 | 4973 |
| 6 | 68 | 5300 | 1,087,537 | 50 | 1102 | 3419 |
| 5 | 56 | 5300 | 846,448 | 39 | 1141 | 2182 |
| 4 | 44 | 5300 | 618,927 | 29 | 1170 | 1245 |
| 3 | 31 | 5300 | 407,758 | 19 | 1188 | 589 |
| 2 | 19 | 5300 | 217,642 | 10 | 1198 | 191 |
| Total Base Shear (k)= | | | | | 1198 | |
| Total Over Turning Moment (k-ft)= | | | | | | 154786 |

Note: The weight and story shear force on the penthouse level is higher than any of the other levels. This is due to the additional weight of the mechanical equipment on this floor. Also, an estimated base shear of 1198 kips is very reasonable due to the fact that the design documents list the seismic base shear at 1200 kips.

| Twisting Moment | | | |
|---|---|---|---|
| Level | Height (ft) | N/S (k-ft) | E/W (k-ft) |
| Roof | 195 | 46 | 33 |
| Elevator | 186 | 145 | 103 |
| Penthouse | 178 | 1781 | 1266 |
| 14 | 166 | 1581 | 1124 |
| 13 | 154 | 1436 | 1020 |
| 12 | 142 | 1293 | 919 |
| 11 | 129 | 1153 | 820 |
| 10 | 117 | 1017 | 723 |
| 9 | 105 | 885 | 629 |
| 8 | 93 | 756 | 537 |
| 7 | 80 | 632 | 449 |
| 6 | 68 | 513 | 365 |
| 5 | 56 | 399 | 284 |
| 4 | 44 | 292 | 207 |
| 3 | 31 | 192 | 137 |
| 2 | 19 | 103 | 73 |
| Total Moment N/S (k-ft) = | | 12224 | |
| Total Moment E/W (k-ft) = | | | 8744 |

The above moments were calculated based on a 5% eccentricity that may occur from the Office Building-G's seismic forces being applied at a location not in direct alignment with the center of rigidity.

## Lateral Load Comparison:

Below is a chart that compares the base shear and overturning moments of the wind and seismic forces. Seismic is the controlling lateral force for overturning moment in both directions and base shear in the East-West Direction. However, the wind force in the North-South direction has a larger base shear. This can be attributed to the larger width of the building causing greater wind loads to accumulate.

| Lateral Load Analysis | | | | | | |
|---|---|---|---|---|---|---|
| | **Wind** | | **Seismic** | | **Controlling** | |
| Type | N-S | E-W | N-S | E-W | N-S | E-W |
| Total Base Shear (k) | 1370 | 1028 | 1198 | 1198 | 1370 | 1198 |
| Overturning Moment (k-ft) | 136924 | 107830 | 154786 | 154786 | 154786 | 154786 |

## Lateral Soil Loads:

Beneath the superstructure of Office Building-G is a four story parking garage with floor plans of almost double the square footage of a typical office floor. This parking garage extends 47 ft below grade before reaching the slab on grade and spread footings. Based on the geotechnical report provided by the geotechnical engineer, significant soil pressure accumulates on the exterior concrete walls of the garage. In the report, the engineer recommends a fluid pressure of 45H psf, with H = 47 ft, be resisted by the basement walls. Figure 7 was provided by the geotechnical engineer and graphically explains the loads that the basement walls must be designed to resist.

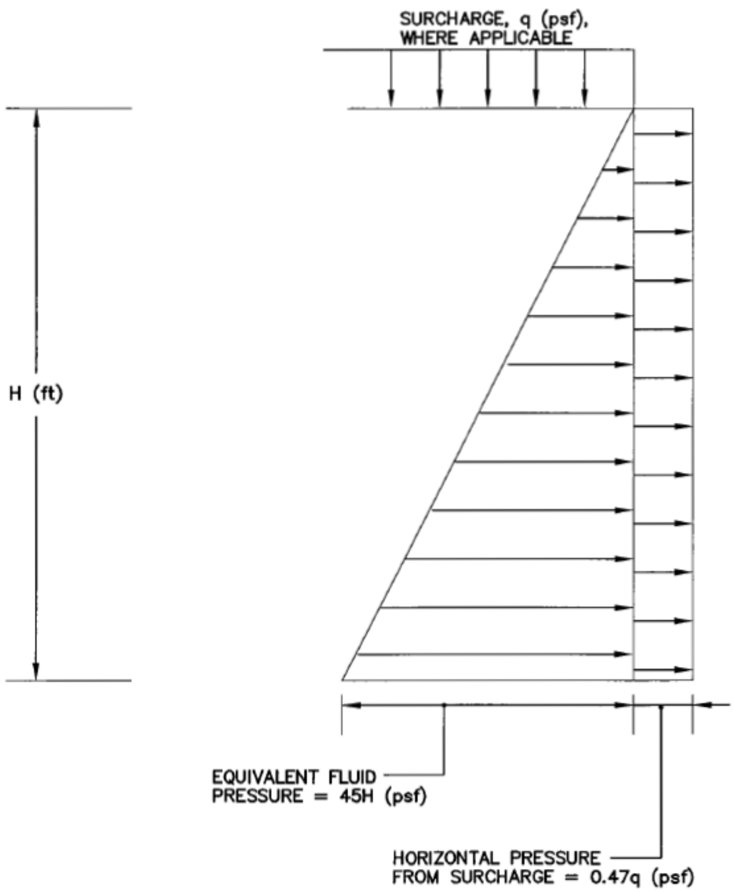



Figure 7

# Gravity Member Spot Checks

## Concrete Slab

A typical floor in Office Building-G is a one way 7" thick slab reinforced with #4 bars at 10" OC. This slab experiences an office live load of 100 psf and must be designed to support its own self-weight and any superimposed loads. Live load reduction would typically be used with such a large load but slab design does not allow for any reduction. Using a width of one foot, the integrity of a floor slab spanning between two post-tensioned beams was checked and the calculations can be found in Appendix B. Figure 8 displays the slab checked.

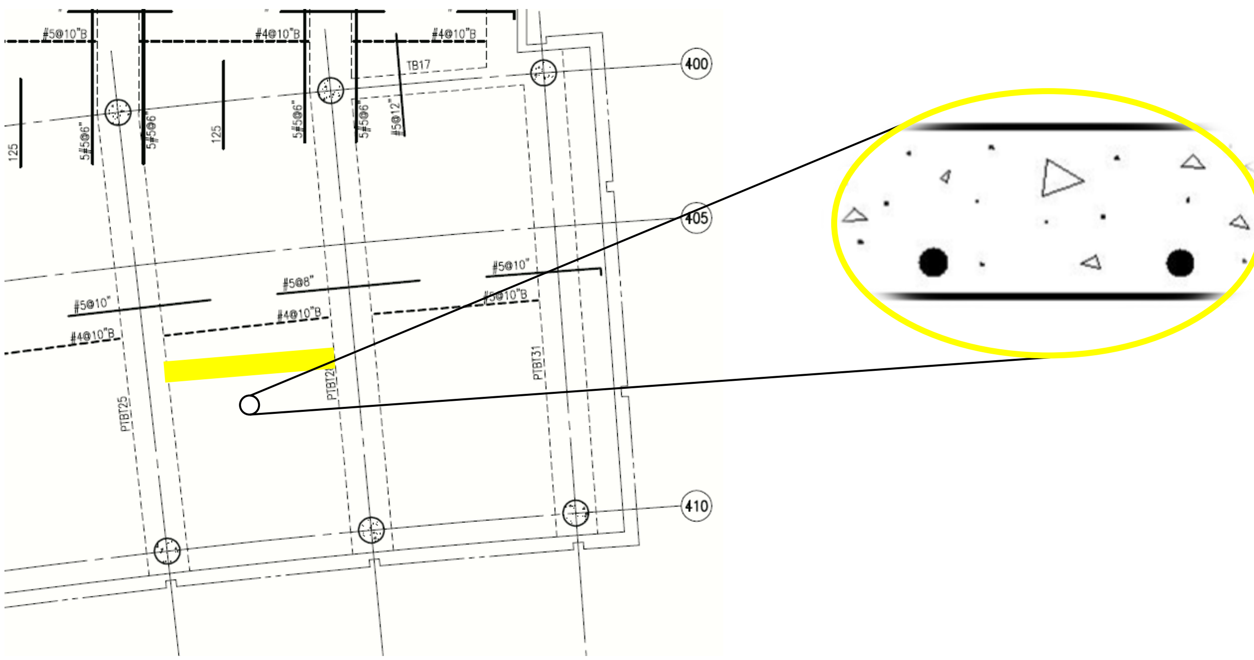



Figure 8

When the as designed slab was analyzed with the loads determined to be supported by the slab, the slab failed in its moment carrying capacity. Spot check results determined that the applied moment is 0.256 ft-k larger than the allowed moment. A possible explanation for this failure is a difference in the assumed superimposed loads because the applied moment was calculated using a relatively large imposed load of 15 psf. This theory was tested at the end of the spot check calculation and using a superimposed load of 5 psf results in an acceptable design.

## Post-Tensioned Beam

48" x 18" post-tensioned beam spanning 43' between columns support 20' of tributary slab width. Unlike the slabs, live load reduction may be used in the beam design. Target Load Balancing analysis was used in the confirmation of the number of tension members. Since an allowable strength design method was implemented in the hand calculations, service loads on the beam were used. Figure 9 highlights the post-tensioned beam design that was checked.

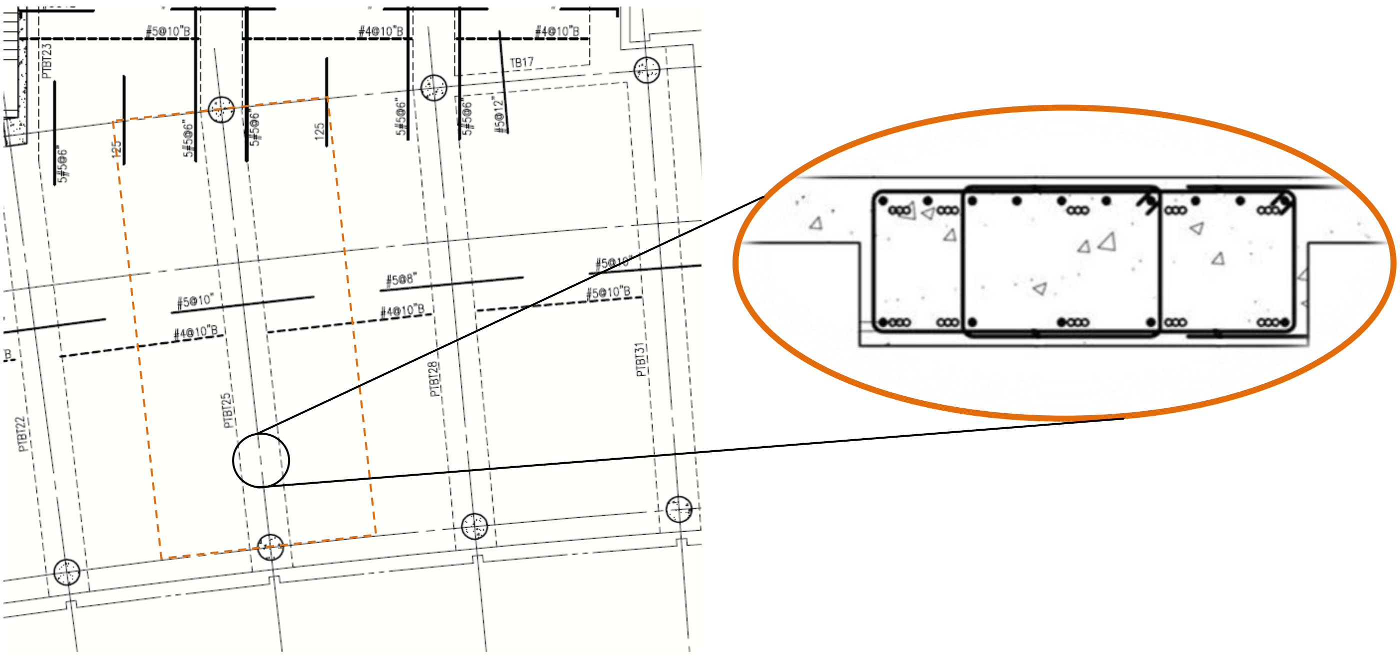



Figure 9

Spot check calculations confirmed the beam dimensions and the tension in the member based on the number of tendons. The load balancing method used provided fairly accurate results for the simplified single span assumption but an analysis of the entire frame with the aid of a computer program will be provided in the future. Assumptions made in the analysis are included in hand calculations found in Appendix B.

## Column Check

A typical exterior column's axial strength was checked using a tributary area and accumulation of loads from above floors. Live load reduction on columns is allowed and was used in the analysis of the typical column. The influence area of columns is much greater than that of beams because the areas of all the floors above are included in this value. A maximum reduction of 40% of the original live load is permissible for columns supporting more than one floor.

Spot check calculations confirmed the strength of a 30" diameter exterior column on the fourth floor, reinforced by 10 #10 grade 60 bars. It was discovered that the column strength is much greater than the required strength. This is attributed to columns on levels 1-4 having the same detail and material strength. With this in mind, the column on the fourth floor is expected to have a much lower axial stress than the column was designed to carry because the same column on the first floor must support three additional floors of load.

## Conclusion

Based on the loads identified and determined in Technical Report I, Office Building-G's structural concepts and existing conditions have been confirmed as adequate. Gravity load spot checks provide the most measurable results of this confirmation. Discrepancies in the as designed members to the values determined in Technical Report I can be attributed to different design loads and references.

Loads used by the structural engineer were determined using the local building code, which was based off of the 2006 International Building Code. In Technical Report I, ASCE 7-10 was referenced for load determination.

Following the design procedures described in ASCE 7-10, lateral load calculations were performed to determine the controlling forces experienced by Office Building-G. Despite being located in a low earthquake activity area, seismic forces control the design of the lateral members. Due to the rectangular shape of the building, wind base shear controls in the North-South direction.

Technical Report I is the first of three technical documents to be completed for Office Building-G. The following papers will reference this report for the existing structural systems and loads experienced for an exploration of alternate floor systems and a detailed analysis of the lateral system.

# Appendix A: Load Calculation

## Wind Loads

| Factors and Coefficients | | | |
|---|---|---|---|
| | North-South | East-West | Reference |
| Exposure | B | B | 26.7.3 |
| Importance | I | I | T 1.5-1 |
| $L_{eff}$ | 114 | 204 | 26.9-1 |
| $C_w$ | 0.92 | 0.45 | 26.9-5 |
| $n_a$ | 1.94 | 1.35 | 26.9-5 |
| $G_f$ | 0.85 | 0.85 | 26.9.4 |
| V | 105 | 105 | 26.5-1C |
| $K_{zt}$ | 1.0 | 1.0 | 26.8.2 |
| Kh | 1.185 | 1.185 | 27.3-1 |
| $GC_{pi}$ | +/- 0.18 | +/- 0.18 | T 26.11-1 |
| L | 204 | 145 | Bldg Plan |
| B | 145 | 204 | Bldg Plan |
| Windward $C_p$ | 0.8 | 0.8 | F 27.4-1 |
| Leeward $C_p$ | -0.418 | -0.5 | F 27.4-1 |

| Kz and qz Calculations | | |
|---|---|---|
| Height | $K_z$ | $q_z$ |
| 19.0 | 0.610 | 14.63 |
| 31.3 | 0.708 | 16.97 |
| 43.5 | 0.716 | 17.18 |
| 55.8 | 0.833 | 19.98 |
| 68.0 | 0.882 | 21.16 |
| 80.3 | 0.931 | 22.33 |
| 92.5 | 0.968 | 23.21 |
| 104.8 | 1.002 | 24.04 |
| 117.0 | 1.033 | 24.77 |
| 129.3 | 1.063 | 25.50 |
| 141.5 | 1.093 | 26.22 |
| 153.8 | 1.118 | 26.81 |
| 166.0 | 1.142 | 27.40 |
| 178.3 | 1.167 | 27.98 |
| 186.0 | 1.185 | 28.43 |
| 195.0 | 1.185 | 28.43 |

# Wind Calculations

| | | | North / South | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Story Level | Floor to Floor Height (ft) | Story Height (ft) | Windward (psf) | Leeward (psf) | Windward (plf) | Leeward (plf) | Windward (kips) | Leeward (kips) | Story Shear (Kips) | Moment (k-ft) |
| 1 | 19 | 0 | 15.07 | -15.22 | 3074 | -3104.9 | 29.20 | -29.50 | 58.70 | 0.0 |
| 2 | 12.25 | 19 | 15.07 | -15.22 | 3074 | -3104.9 | 48.03 | -48.51 | 96.54 | 1834.4 |
| 3 | 12.25 | 31.25 | 16.66 | -15.22 | 3398 | -3104.9 | 39.64 | -38.03 | 77.68 | 2427.5 |
| 4 | 12.25 | 43.5 | 16.80 | -15.22 | 3428 | -3104.9 | 41.81 | -38.03 | 79.84 | 3473.2 |
| 5 | 12.25 | 55.75 | 18.71 | -15.22 | 3816 | -3104.9 | 44.37 | -38.03 | 82.40 | 4593.9 |
| 6 | 12.25 | 68 | 19.51 | -15.22 | 3979 | -3104.9 | 47.75 | -38.03 | 85.78 | 5833.1 |
| 7 | 12.25 | 80.25 | 20.30 | -15.22 | 4141 | -3104.9 | 49.74 | -38.03 | 87.77 | 7043.8 |
| 8 | 12.25 | 92.5 | 20.90 | -15.22 | 4264 | -3104.9 | 51.48 | -38.03 | 89.52 | 8280.3 |
| 9 | 12.25 | 104.75 | 21.46 | -15.22 | 4378 | -3104.9 | 52.93 | -38.03 | 90.97 | 9528.7 |
| 10 | 12.25 | 117 | 21.96 | -15.22 | 4480 | -3104.9 | 54.26 | -38.03 | 92.29 | 10798.1 |
| 11 | 12.25 | 129.25 | 22.46 | -15.22 | 4582 | -3104.9 | 55.50 | -38.03 | 93.54 | 12090.0 |
| 12 | 12.25 | 141.5 | 22.95 | -15.22 | 4681 | -3104.9 | 56.74 | -38.03 | 94.77 | 13410.4 |
| 13 | 12.25 | 153.75 | 23.35 | -15.22 | 4763 | -3104.9 | 57.85 | -38.03 | 95.88 | 14741.7 |
| 14 | 12.25 | 166 | 23.75 | -15.22 | 4844 | -3104.9 | 58.85 | -38.03 | 96.88 | 16082.1 |
| Penthouse | 7.75 | 178.25 | 24.15 | -15.22 | 4926 | -3104.9 | 48.76 | -31.05 | 79.81 | 14226.0 |
| Elevator | 9 | 186 | 24.45 | -15.22 | 4988 | -3104.9 | 41.53 | -26.00 | 67.54 | 12561.6 |
| Roof | 0 | 195 | 24.45 | -15.22 | 4988 | -3104.9 | 22.44 | -13.97 | 36.42 | 7101.1 |

| | | | East/West | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Story Level | Floor to Floor Height (ft) | Story Height (ft) | Windward (psf) | Leeward (psf) | Windward (plf) | Leeward (plf) | Windward (kips) | Leeward (kips) | Story Shear (Kips) | Moment (k-ft) |
| 1 | 19 | 0 | 15.07 | -17.20 | 2185 | -2494 | 20.76 | -23.69 | 44.45 | 0 |
| 2 | 12.25 | 19 | 15.07 | -17.20 | 2185 | -2494 | 34.14 | -38.97 | 73.11 | 1389.1 |
| 3 | 12.25 | 31.25 | 16.66 | -17.20 | 2416 | -2494 | 28.18 | -30.55 | 58.73 | 1835.3 |
| 4 | 12.25 | 43.5 | 16.80 | -17.20 | 2436 | -2494 | 29.72 | -30.55 | 60.27 | 2621.7 |
| 5 | 12.25 | 55.75 | 18.71 | -17.20 | 2712 | -2494 | 31.54 | -30.55 | 62.09 | 3461.4 |
| 6 | 12.25 | 68 | 19.51 | -17.20 | 2828 | -2494 | 33.94 | -30.55 | 64.49 | 4385.3 |
| 7 | 12.25 | 80.25 | 20.30 | -17.20 | 2944 | -2494 | 35.35 | -30.55 | 65.91 | 5288.9 |
| 8 | 12.25 | 92.5 | 20.90 | -17.20 | 3031 | -2494 | 36.59 | -30.55 | 67.14 | 6210.8 |
| 9 | 12.25 | 104.75 | 21.46 | -17.20 | 3112 | -2494 | 37.62 | -30.55 | 68.17 | 7141.3 |
| 10 | 12.25 | 117 | 21.96 | -17.20 | 3184 | -2494 | 38.56 | -30.55 | 69.12 | 8086.6 |
| 11 | 12.25 | 129.25 | 22.46 | -17.20 | 3257 | -2494 | 39.45 | -30.55 | 70.00 | 9048.0 |
| 12 | 12.25 | 141.5 | 22.95 | -17.20 | 3327 | -2494 | 40.33 | -30.55 | 70.88 | 10029.6 |
| 13 | 12.25 | 153.75 | 23.35 | -17.20 | 3385 | -2494 | 41.12 | -30.55 | 71.67 | 11019.0 |
| 14 | 12.25 | 166 | 23.75 | -17.20 | 3443 | -2494 | 41.83 | -30.55 | 72.38 | 12014.8 |
| Penthouse | 7.75 | 178.25 | 24.15 | -17.20 | 3501 | -2494 | 34.66 | -24.94 | 59.60 | 10623.4 |
| Elevator | 9 | 186 | 24.45 | -17.20 | 3545 | -2494 | 29.52 | -20.89 | 50.41 | 9375.9 |
| Roof | 0 | 195 | 24.45 | -17.20 | 3545 | -2494 | 15.95 | -11.22 | 27.18 | 5299.3 |

Hand Calculations

Wind loads : Flexible or Rigid

$$L_{eff} = \frac{\sum_{i=1}^{n} h_i L_i}{\sum_{i=1}^{n} h_i} = \frac{200839.5}{1761.75} = 114'$$

$4L_{eff} \geq 195' \Rightarrow 456 > 195'$ ∴ Proceed

$$n_a = 385(C_w)^{0.5}/h$$

$$C_w = \frac{100}{A_B} \sum_{i=1}^{n} \left(\frac{h}{h_i}\right)^2 \frac{A_i}{\left[1 + 0.83\left(\frac{h_i}{D_i}\right)^2\right]}$$

NORTH – SOUTH

$A_B = 210 \times 120 = 25,200$ sf

$$\left.\begin{array}{l} \left(\frac{h}{h_i}\right)^2 = 1.0 \\ A_i = 85 \text{ sf} \\ D_i = 85 \text{ ft} \\ h_i = 195 \text{ ft} \end{array}\right\} \sum_{i=1}^{n} \left(\frac{h}{h_i}\right)^2 \frac{A_i}{\left[1 + 0.83\left(\frac{h_i}{D_i}\right)^2\right]} \Rightarrow 231$$

$$C_w = \frac{100(231)}{25200} = 0.92$$

$$n_a = \frac{385\sqrt{0.92}}{195} = 1.94$$

$1.94 > 1.0$ ∴ Rigid

EAST - WEST

$$\left.\begin{aligned} \left(\frac{h}{h_i}\right)^2 &= 1.0 \\ A_i &= 65\text{ sf} \\ D_i &= 65\text{ ft} \\ h_i &= 195\text{ ft} \end{aligned}\right\} \sum_{i=1}^{n} \left(\frac{h}{h_i}\right)^2 \frac{A_i}{\left[1 + 0.83\left(\frac{h_i}{D_i}\right)^2\right]} = 112.5$$

$$C_W = \frac{100(112.5)}{25200} = 0.45$$

$$n_a = \frac{385\sqrt{0.45}}{195} = 1.35$$

$1.35 > 1.0$ $\therefore$ Rigid

TYPICAL WIND CALC.
EXPOSURE B

N-S WINDWARD : $p_z = q_z G_f C_p - q_h(GC_{pi})$

$q_z = 0.00256 K_z K_{zt} K_d V^2$

$\left.\begin{array}{l} K_z = 0.61 \\ K_z = 1.0 \\ K_d = 0.85 \\ V = 105 \end{array}\right\} q_z = 0.00256(0.61)(1.0)(.85)(105^2) = 14.6 \text{ psf}$

$G_f = 0.85$
$C_p = 0.8$ $\quad L/B = 1.61$
$GC_{pi} = \pm 0.18$
$q_i = q_h$

$q_h = 0.00256 K_h K_{ht} K_d V^2 = 0.00256(1.185)(1.0)(0.85)(105^2) = 28.43$
$h = 195'$
$K_h = 1.185$

$p_z = q_z G_f C_p - q_h(GC_{pi}) = 14.6(0.85)(0.8) - 28.43(\pm 0.18)$

$p_z = 9.93 \pm 5.12$ @ $h = 19'$

$\boxed{p_z = 15.05 \text{ psf @ } h = 19'}$

N-S LEEWARD: $p_h = q_h G_f C_p - q_h (GC_{pi})$

$q_h = 0.00256 K_h K_{zt} K_d V^2 = 28.43$
$G_f = 0.85$
$C_p = -0.418 \quad (L/B = 1.41)$
$GC_{pi} = \pm 0.18$

$p_h = 28.43(0.85)(-.418) - 28.43(\pm 0.18)$
$= -10.10 \pm 5.12$

$\boxed{p_h = -15.22 \text{ psf}}$

E-W WINDWARD
$q_z = 14.6$ psf
$q_h = 28.43$ psf

$p_z = 14.6(0.85)(0.8) - 28.43(\pm 0.18)$

$\boxed{p_z = 15.05 \text{ psf @ } h = 19'}$

E-W LEEWARD
$q_h = 28.43$
$C_p = -0.5 \quad (L/B = 0.623)$

$p_h = 28.43(0.85)(-0.5) - 28.43(\pm 0.18)$

$\boxed{p_h = -17.22 \text{ psf}}$

## Seismic Loads

| Coefficients and References | | |
|---|---|---|
| Factor | Coefficient | Reference |
| Site Class | D | Geo. Report |
| Design Category | B | T 11.6-1 |
| Importance | 1 | T 1.5-2 |
| $S_s$ | 16 | USGS Website |
| $S_1$ | 5.1 | USGS Website |
| $F_a$ | 1.6 | T 11.4-1 |
| $F_v$ | 2.4 | T 11.4-2 |
| $S_{ms}$ | 0.256 | 11.4-1 |
| $S_{m1}$ | 0.0816 | 11.4-2 |
| $S_{ds}$ | 0.171 | 11.4-3 |
| $S_{d1}$ | 0.0816 | 11.4-4 |
| $C_t$ | 0.02 | T 12.8-2 |
| x | 0.75 | T 12.8-2 |
| $h_n$ | 195 | Bldg Drawings |
| $T_a$ | 1.0235 | 12..8-7 |
| $T_L$ | 8 | F 22-12 |
| R | 5 | T 12.2-1 |
| Cs | 0.0159 | 12.8-3 |
| W | 74903 | 12.7.2 |
| $V_b$ | 1198 | 12.8-1 |

# N/S-Direction Loading

| | |
|---|---|
| k= | 1.262 |
| $V_b$= | 1198 |

| i | $h_i$ | h | w | $w*h^k$ | $C_{VX}$ | $f_i$ | $V_i$ | N/S L | 5%L | Ax | $M_z$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | ft | ft | kips | | | kips | kips | ft | ft | | k-ft |
| | | | | | | | | | | | |
| Roof | 0 | 195 | 126 | 97684 | 0.004 | 5 | 5 | 204 | 10 | 1.0 | 46 |
| Elevator | 9 | 186 | 421 | 307496 | 0.012 | 14 | 19 | 204 | 10 | 1.0 | 145 |
| Penthouse | 7.75 | 178.25 | 5457 | 3777032 | 0.146 | 175 | 193 | 204 | 10 | 1.0 | 1781 |
| 14 | 12.25 | 166 | 5300 | 3353492 | 0.129 | 155 | 348 | 204 | 10 | 1.0 | 1581 |
| 13 | 12.25 | 153.75 | 5300 | 3044317 | 0.117 | 141 | 489 | 204 | 10 | 1.0 | 1436 |
| 12 | 12.25 | 141.5 | 5300 | 2741529 | 0.106 | 127 | 616 | 204 | 10 | 1.0 | 1293 |
| 11 | 12.25 | 129.25 | 5300 | 2445532 | 0.094 | 113 | 729 | 204 | 10 | 1.0 | 1153 |
| 10 | 12.25 | 117 | 5300 | 2156798 | 0.083 | 100 | 829 | 204 | 10 | 1.0 | 1017 |
| 9 | 12.25 | 104.75 | 5300 | 1875881 | 0.072 | 87 | 915 | 204 | 10 | 1.0 | 885 |
| 8 | 12.25 | 92.5 | 5300 | 1603449 | 0.062 | 74 | 990 | 204 | 10 | 1.0 | 756 |
| 7 | 12.25 | 80.25 | 5300 | 1340323 | 0.052 | 62 | 1052 | 204 | 10 | 1.0 | 632 |
| 6 | 12.25 | 68 | 5300 | 1087537 | 0.042 | 50 | 1102 | 204 | 10 | 1.0 | 513 |
| 5 | 12.25 | 55.75 | 5300 | 846448 | 0.033 | 39 | 1141 | 204 | 10 | 1.0 | 399 |
| 4 | 12.25 | 43.5 | 5300 | 618927 | 0.024 | 29 | 1170 | 204 | 10 | 1.0 | 292 |
| 3 | 12.25 | 31.25 | 5300 | 407758 | 0.016 | 19 | 1188 | 204 | 10 | 1.0 | 192 |
| 2 | 12.25 | 19 | 5300 | 217642 | 0.008 | 10 | 1198 | 204 | 10 | 1.0 | 103 |
| 1 | 19 | 0 | | | | | | | | | |
| | | $\Sigma$ | 74903.5 | 25921845 | 1.0 | 1198 | | | | | 12224 |

# E/W-Direction Loading

| | |
|---|---|
| k= | 1.262 |
| $V_b$= | 1198 |

| i | $h_i$ | h | w | $w*h^k$ | $C_{VX}$ | $f_i$ | $V_i$ | E/S L | 5%L | Ax | $M_z$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | ft | ft | kips | | | kips | kips | ft | ft | | k-ft |
| | | | | | | | | | | | |
| Roof | 0 | 195 | 126 | 97684 | 0.004 | 5 | 5 | 145 | 7 | 1.0 | 33 |
| Elevator | 9 | 186 | 421 | 307496 | 0.012 | 14 | 19 | 145 | 7 | 1.0 | 103 |
| Penthouse | 7.75 | 178.25 | 5457 | 3777032 | 0.146 | 175 | 193 | 145 | 7 | 1.0 | 1266 |
| 14 | 12.25 | 166 | 5300 | 3353492 | 0.129 | 155 | 348 | 145 | 7 | 1.0 | 1124 |
| 13 | 12.25 | 153.75 | 5300 | 3044317 | 0.117 | 141 | 489 | 145 | 7 | 1.0 | 1020 |
| 12 | 12.25 | 141.5 | 5300 | 2741529 | 0.106 | 127 | 616 | 145 | 7 | 1.0 | 919 |
| 11 | 12.25 | 129.25 | 5300 | 2445532 | 0.094 | 113 | 729 | 145 | 7 | 1.0 | 820 |
| 10 | 12.25 | 117 | 5300 | 2156798 | 0.083 | 100 | 829 | 145 | 7 | 1.0 | 723 |
| 9 | 12.25 | 104.75 | 5300 | 1875881 | 0.072 | 87 | 915 | 145 | 7 | 1.0 | 629 |
| 8 | 12.25 | 92.5 | 5300 | 1603449 | 0.062 | 74 | 990 | 145 | 7 | 1.0 | 537 |
| 7 | 12.25 | 80.25 | 5300 | 1340323 | 0.052 | 62 | 1052 | 145 | 7 | 1.0 | 449 |
| 6 | 12.25 | 68 | 5300 | 1087537 | 0.042 | 50 | 1102 | 145 | 7 | 1.0 | 365 |
| 5 | 12.25 | 55.75 | 5300 | 846448 | 0.033 | 39 | 1141 | 145 | 7 | 1.0 | 284 |
| 4 | 12.25 | 43.5 | 5300 | 618927 | 0.024 | 29 | 1170 | 145 | 7 | 1.0 | 207 |
| 3 | 12.25 | 31.25 | 5300 | 407758 | 0.016 | 19 | 1188 | 145 | 7 | 1.0 | 137 |
| 2 | 12.25 | 19 | 5300 | 217642 | 0.008 | 10 | 1198 | 145 | 7 | 1.0 | 73 |
| 1 | 19 | 0 | | | | | | | | | |
| | | ∑ | 74903.5 | 25921845 | 1 | 1198 | | | | | 8689 |

## Dead Load Determination

| Element | Width (in) | Depth (in) | Length (ft) | Volume ($ft^3$) | Number | pcf | psf | SF/Floor | Weight/Floor (lbs) | Weight/Floor (kips) | Concentrated (kips) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Slab | - | 7 | - | - | 1 | 150 | 87.5 | 25376 | 2220400 | 2220 | - |
| | | | | | | | | | | | |
| Columns | 24 | 24 | 12.25 | 49.0 | 15 | 150 | - | - | 110250 | 110 | - |
| | 24 | 32 | 12.25 | 65.3 | 7 | 150 | - | - | 68600 | 69 | - |
| | 30 | - | 12.25 | 60.1 | 18 | 150 | | - | 162356 | 162 | - |
| | | | | | | | | | | | |
| Post-Tensioned Beam # | | | | | | | | | | | |
| 31 | 48 | 18 | 45 | 270.0 | 1 | 150 | - | - | 40500 | 41 | - |
| 28 | 48 | 18 | 45 | 270.0 | 1 | 150 | - | - | 40500 | 41 | - |
| 25 | 48 | 18 | 45 | 270.0 | 1 | 150 | - | - | 40500 | 41 | - |
| 22 | 48 | 18 | 45 | 270.0 | 1 | 150 | - | - | 40500 | 41 | - |
| 19 | 48 | 18 | 45 | 270.0 | 1 | 150 | - | - | 40500 | 41 | - |
| 16 | 48 | 18 | 45 | 270.0 | 1 | 150 | - | - | 40500 | 41 | - |
| 13 | 48 | 18 | 45 | 270.0 | 1 | 150 | - | - | 40500 | 41 | - |
| 10 | 48 | 18 | 45 | 270.0 | 1 | 150 | - | - | 40500 | 41 | - |
| 7 | 48 | 18 | 45 | 270.0 | 1 | 150 | - | - | 40500 | 41 | - |
| 4 | 48 | 18 | 45 | 270.0 | 1 | 150 | - | - | 40500 | 41 | - |
| 1 | 48 | 18 | 45 | 270.0 | 1 | 150 | - | - | 40500 | 41 | - |
| 6 | 48 | 18 | 45 | 270.0 | 1 | 150 | - | - | 40500 | 41 | - |
| 9 | 48 | 18 | 45 | 270.0 | 1 | 150 | - | - | 40500 | 41 | - |
| 12 | 48 | 18 | 45 | 270.0 | 1 | 150 | - | - | 40500 | 41 | - |
| 15 | 48 | 18 | 45 | 270.0 | 1 | 150 | - | - | 40500 | 41 | - |
| 18 | 48 | 18 | 45 | 270.0 | 1 | 150 | - | - | 40500 | 41 | - |
| 21 | 48 | 18 | 45 | 270.0 | 1 | 150 | - | - | 40500 | 41 | - |
| 24 | 48 | 18 | 45 | 270.0 | 1 | 150 | - | - | 40500 | 41 | - |
| 27 | 48 | 18 | 45 | 270.0 | 1 | 150 | - | - | 40500 | 41 | - |
| 30 | 48 | 18 | 45 | 270.0 | 1 | 150 | - | - | 40500 | 41 | - |
| 34 | 36 | 18 | 45 | 202.5 | 1 | 150 | - | - | 30375 | 30 | - |
| 14 | 18 | 24 | 30 | 90.0 | 1 | 150 | - | - | 13500 | 14 | - |
| 2 | 36 | 18 | 46 | 207.0 | 1 | 150 | - | - | 31050 | 31 | - |
| 5 | 48 | 18 | 41 | 246.0 | 1 | 150 | - | - | 36900 | 37 | - |
| 8 | 48 | 18 | 37 | 222.0 | 1 | 150 | - | - | 33300 | 33 | - |
| 11 | 48 | 18 | 37 | 222.0 | 1 | 150 | - | - | 33300 | 33 | - |
| 17 | 24 | 18 | 29 | 87.0 | 1 | 150 | - | - | 13050 | 13 | - |
| 20 | 48 | 18 | 25.5 | 153.0 | 1 | 150 | - | - | 22950 | 23 | - |
| 23 | 24 | 18 | 23 | 69.0 | 1 | 150 | - | - | 10350 | 10 | - |
| 26 | 48 | 18 | 20 | 120.0 | 1 | 150 | - | - | 18000 | 18 | - |
| 29 | 48 | 18 | 18 | 108.0 | 1 | 150 | - | - | 16200 | 16 | - |

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 32 | 36 | 18 | 16 | 72.0 | 1 | 150 | - | - | 10800 | 11 | - |
| 33 | 36 | 18 | 20 | 90.0 | 1 | 150 | - | - | 13500 | 14 | - |
| | | | | | | | | | | | |
| Beam # | | | | | | | - | - | 0 | 0 | - |
| 1 | 24 | 18 | 15 | 45 | 1 | 150 | - | - | 6750 | 7 | - |
| 20C | 24 | 18 | 6 | 18 | 1 | 150 | - | - | 2700 | 3 | - |
| 20 | 24 | 18 | 20 | 60 | 1 | 150 | - | - | 9000 | 9 | - |
| 2 | 24 | 18 | 17 | 51 | 1 | 150 | - | - | 7650 | 8 | - |
| 3 | 24 | 18 | 17 | 51 | 1 | 150 | - | - | 7650 | 8 | - |
| 4 | 12 | 18 | 22 | 33 | 1 | 150 | - | - | 4950 | 5 | - |
| 5 | 18 | 18 | 18 | 40.5 | 1 | 150 | - | - | 6075 | 6 | - |
| 6 | 18 | 24 | 26 | 78 | 1 | 150 | - | - | 11700 | 12 | - |
| 7 | 18 | 18 | 13 | 29.25 | 1 | 150 | - | - | 4388 | 4 | - |
| 8 | 12 | 18 | 12 | 18 | 1 | 150 | - | - | 2700 | 3 | - |
| 8 | 12 | 18 | 12 | 18 | 1 | 150 | - | - | 2700 | 3 | - |
| 9 | 18 | 18 | 17 | 38.25 | 1 | 150 | - | - | 5738 | 6 | - |
| 10 | 10 | 20 | 21 | 29.167 | 1 | 150 | - | - | 4375 | 4 | - |
| 11 | 12 | 24 | 18 | 36 | 1 | 150 | - | - | 5400 | 5 | - |
| 12 | 8 | 18 | 10 | 10 | 1 | 150 | - | - | 1500 | 2 | - |
| 13 | 24 | 18 | 20 | 60 | 1 | 150 | - | - | 9000 | 9 | - |
| 14 | 36 | 18 | 20 | 90 | 1 | 150 | - | - | 13500 | 14 | - |
| 15 | 36 | 18 | 20 | 90 | 1 | 150 | - | - | 13500 | 14 | - |
| 16 | 36 | 18 | 20 | 90 | 1 | 150 | - | - | 13500 | 14 | - |
| 17 | 24 | 18 | 20 | 60 | 1 | 150 | - | - | 9000 | 9 | - |
| 18 | 8 | 18 | 12 | 12 | 1 | 150 | - | - | 1800 | 2 | - |
| 19 | 6 | 12 | 5 | 2.5 | 1 | 150 | - | - | 375 | 0 | - |
| | | | | | | | | | | | |
| Shear Walls | 1020 | 12 | 12.25 | 1041.3 | 1 | 150 | - | - | 156188 | 156 | - |
| | 780 | 12 | 12.25 | 796.25 | 1 | 150 | - | - | 119438 | 119 | - |
| | | | | | | | | | | | |
| Partitions | - | - | - | - | - | - | 20 | 25376 | 507520 | 508 | - |
| | | | | | | | | | | | |
| Exterior Walls | 706 | - | 195 | - | - | - | 30 | 137670 | 337151 | 337 | - |
| | | | | | | | | | | | |
| Superimposed | - | - | - | - | - | - | 15 | 25376 | 380640 | 381 | - |
| | | | | | | | | | | | |
| Mechanical Equipment | - | - | - | - | 1 | - | - | - | - | - | |
| Cooling tower | - | - | - | - | 2 | - | - | - | - | - | 66.4 |
| RTU | - | - | - | - | 1 | - | - | - | - | - | 33 |
| Chiller 1 | - | - | - | - | 1 | - | - | - | - | - | 21.4 |

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Chiller 2 | - | - | - | - | 1 | - | - | - | - | - | 21.4 |
| Chiller 3 | - | - | - | - | 1 | - | - | - | - | - | 14.3 |
| Total | | | | | | | | | 5299767 | 5300 | 156.5 |

Hand Calculations:

Seismic Base Shear:

$V = C_s W$

$T_a = C_t h_n^x = 0.02(195)^{0.75} = 1.0235$

$C_t = 0.02$

$x = 0.75$

$h_n = 195'$

$T_L = 8$ (Figure 22-12)

$$C_s = \min \begin{cases} \dfrac{S_{D1}}{T(R/I_e)} = \dfrac{0.0816}{1.0235(5/1)} = 0.0159 \leftarrow \text{Controls} \\ \dfrac{S_{DS}}{(R/I_e)} = \dfrac{0.171}{(5/1)} = 0.0342 \end{cases}$$

$S_{DS} = \frac{2}{3} S_{MS} = 0.171$

• $S_{MS} = F_a S_s = 0.256$

$S_s = 16.0\% = .16$, $F_a = 1.6$

$S_{D1} = \frac{2}{3} S_{M1} = 0.0816$

• $S_{M1} = F_v S_1 = 2.4(0.051) = 0.1224$

$S_1 = 5.1\% = 0.051$, $F_v = 2.4$

$R = 5$

$I_e = 1.0$

## Snow Loads

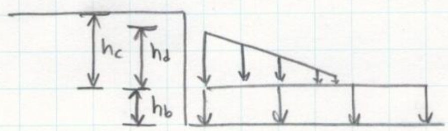
Ground Snow Load : $p_g = 25$ psf

Flat roof Snow load :

$$p_f = 0.7 C_e C_t I_s p_g = 0.7(0.9)(1.0)(0.8)25 = 12.6 \text{ psf}$$

$C_e = 0.9$
$C_t = 1.0$
$I_s = 0.8$
$p_g = 25$

Drift Snow load on Roof : E/W




$h_c/h_b < 0.2$

$16.02/0.73 = 21.9 > 0.2$

∴ Drift must be calculated.

$h_d$ leeward use $l_u = 50'$
windward use $0.75\, h_d$
} Use larger in design

$h_d \le h_c \Rightarrow w = 4h_d$

$h_d > h_c \Rightarrow w = \dfrac{4h_d^2}{h_c}$ & $h_d = h_c$

$w \le 8h_c$

$p_d = h_d \gamma$ $\quad \gamma = 0.13 p_g + 14 < 30$ pcf

$\gamma = 0.13(25) + 14 = 17.25$ pcf

$$h_b = 12.6 \frac{lb}{ft^2} \times \frac{1}{17.25} \frac{ft^3}{lb} = 0.73 ft$$

$h_c = 16.75 - 0.73 = 16.02$

Figure 7-9

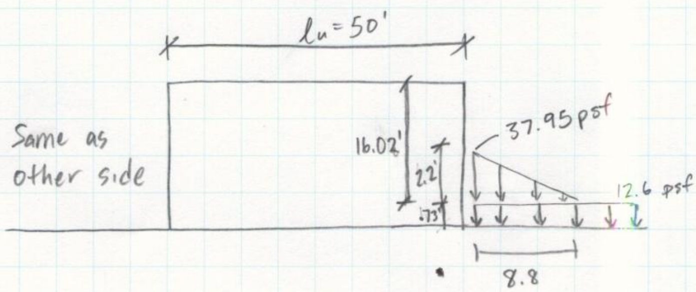
$p_g = 25$, $l_u = 50$, $h_d = 2.2\text{ft}$

$h_c > h_d \Rightarrow w = 4h_d$

$w = 4(2.2) = 8.8\text{ft}$

$p_d = h_d \gamma$
$= 2.2(17.25) = 37.95$

LOADING DIAGRAM: E-W



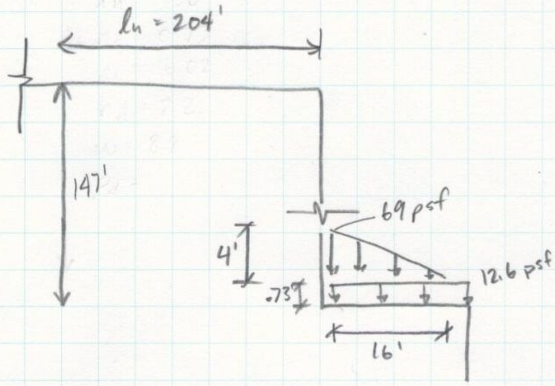

Maximum snow load = 12.6 psf + 37.95 psf = 50.55 psf

50.55 psf $<$ 100 psf live load

↑ Controls

Note: NORTH SOUTH HAS THE SAME RESULT

Drift on Roof of Level 2




$p_g = 25\,psf$, $l_u = 204'$, $\Rightarrow h_d = 4'$ $\quad w = 4h_d = 16$

$p_d = \gamma h_d = 17.25(4) = 69\,psf$

MAX snow @ Level 2 Roof $= 69 + 12.6 = 81.6\,psf$

$81.6\,psf < 100\,psf$ Live Load

↑

Controls

# Appendix B: Spot Checks

## Slab

SLAB SPOT CHECK:

LOADS:

– LIVE : OFFICE SPACE – 100 psf

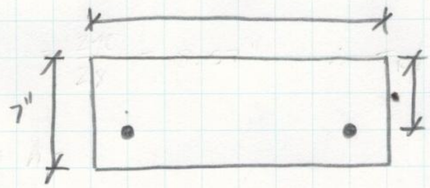
– DEAD : CONCRETE S.W.

$(7/12)(150) = 87.5$ psf

Superimposed = 15.0 psf

SLAB DESIGN : 7" w/ #4 @ 10" o.c. BOTTOM BARS

$f_y = 60^{ksi}$ $f'_c = 5000$ psi




$d = 7 - 3/4 - .5/2 = 6"$

$A_s/ft = \frac{0.20}{10} \times 12 = 0.24 \ in^2/ft$

$a = \beta_1 c$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.24(60)}{0.85(5)(12)} = 0.2824$

$0.375 d = .375(6) = 2.25$

$0.2824 < 2.25 \therefore$ Tension Controls

$\phi = 0.9$

$M_n = A_s f_y (d - a/2)/12$

$= 0.24(60)(6 - .28/2)/12 = 7.032^{k\text{-}ft/ft}$

$\phi M_n = 0.9(7.032) = 6.33^{k\text{-}ft}$

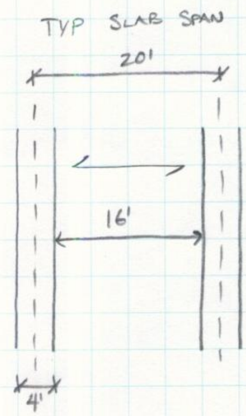
Determine $M_u$: 8.3.3 ACI

Positive Moment (interior Span):

$$M_u = \frac{W_u l^2}{16}$$

Negative Moment (interior Span):

$$M_u = \frac{W_u l^2}{11}$$




$$W_u = 1.2D_L + 1.6L_L$$
$$= 1.2(87.5+15) \times 1ft + 1.6(100) \times 1ft$$
$$= 283 \ lb/ft$$

$$283/1000 = .283 \ ^k/ft/ft$$

$l$ = Distance between faces of support

$$l = 20' - 4/2 - 4/2$$
$$l = 16'$$

$$M_u = \frac{.283(16')^2}{11} = 6.586 \ k\text{-}ft/ft$$

$$\phi M_n < M_u$$

$6.33^{k\text{-}ft} < 6.586^{k\text{-}ft}$ ∴ SLAB DOES NOT WORK

-Try Superimposed = 5 psf

$$W_u = 1.2(87.5+5) + 1.6(100) = .271 \ ^{lb}/ft$$

$$M_u = \frac{.271(16^2)}{11} = 6.31^{k\text{-}ft}$$ ∴ WORKS

## Post-Tensioned Beam

Post-Tensioned Beam Check : PTBT 22

DEAD LOADS : SLAB $= 87.5(20)/1000 = 1.75\ ^{k}/ft$

S. IMPOSED $= 15(20)/1000 = 0.3$

S.W $= (48/12)(18/12)(150)/1000 = 0.9$

$2.95\ ^{k}/ft$

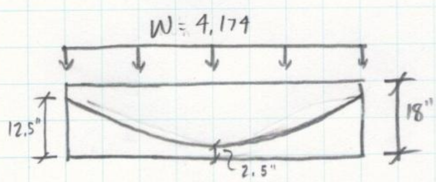
LIVE LOAD REDUCTION : $A_T = 20 \times 43 = 860$

$K_{LL} = 2$

$$L_R = L\left[0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right] = 100\left[0.25 + \frac{15}{\sqrt{2 \cdot 860}}\right] = 61.2\ psf$$

$L_L$ ON BEAM $= 61.2(20)/1000 = 1.224$

SERVICE LOAD $= 2.95 + 1.224 = 4.174$




$h = L/30 = \frac{43(12)}{30} = 17.2" \leqq 18"$ ∴ OK

$A = 48" \times 18" = 876\ in^2$

$1/2"\ \phi \quad A = .153\ in^2 \quad 270^{KSI}$ STRANDS

TARGET LOAD BALANCING : 75% OF LOAD ON BEAM

$0.75(4.174) = 3.131\ ^{k}/ft$

$$P = \frac{W_b a L^2}{8} = \frac{3.131(10/12)(43^2)}{8} = 603^k$$

ASSUME 15 ksi LOSES IN STRAND

$f_{se} = 0.75 f_{pu} = 0.75(270{,}000) - 15{,}000 = 187.5 \text{ ksi}$

$P_{eff} = f_{se} A = (187.5)(.153) = 28.69$

\# OF TENDONS

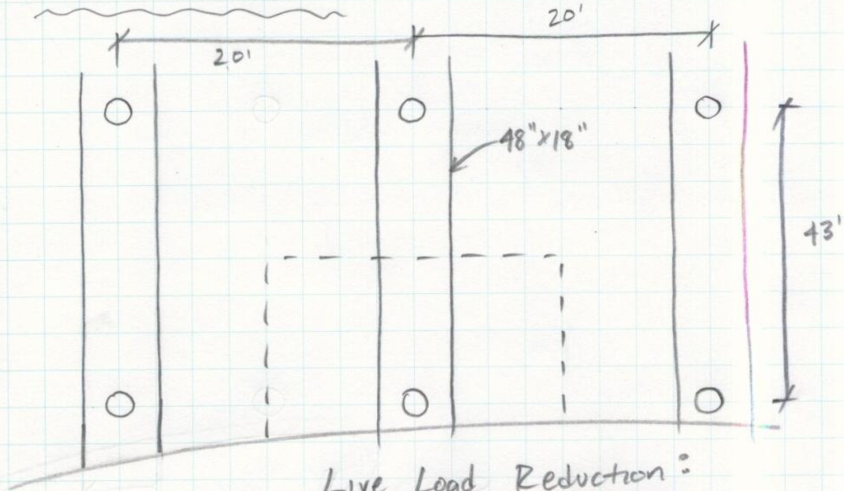
$\frac{603}{28.69} = 21.02$

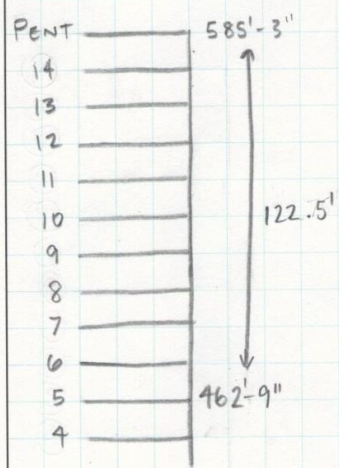
$P_{actual}$ :

$21(28.69) = 603^k \cong 594^k$ AS DESIGNED $\therefore$ OK

## Column

COLUMN SPOT CHECK: $4^{th}$ FLOOR EXTERIOR COLUMN




Live Load Reduction:

$A_T = 43/2 \times 20/2 = 215\ ft^2$

# of floors = 10 (cannot reduce roof)

$K_{LL} = 4.0$

$A_{influence} = 4.0 \times 11 \times 215 = 9,460\ ft^2$

$L_R = L\left[0.25 + \frac{15}{\sqrt{A_I}}\right]$

$= 100\left[0.25 + \frac{15}{\sqrt{9460}}\right]$

$= 40.4\ psf$

COLUMN is supporting more than 1 floor so reduction up to $0.4\ L_L$ is allowed

LOADS ON COLUMN:

LIVE LOADS:

ROOF/TERRACE = 100 psf × 1 floor × 215 = 21,500 lbs
OFFICE (R) = 40.4 psf × 10 floors × 215 = 86,860 lbs

TOTAL AXIAL LIVE: 108,360 lbs

DEAD LOADS:

SUPERIMPOSED = 15 psf × 11 floors × 215 = 35,475 lb
SLAB = 87.5 psf × 11 floors × 215 = 206,940 lb
BEAMS:
$(48/12)(18/12)(150)(43/2) = 19,350$ lb
$19,350 \times 11$ floors $= 212,850$ lb

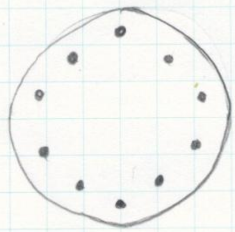
EXTERIOR WALL:
$(30\ psf)(12.25')(20') = 7,350$ lb
$7,350 \times 10$ floors $= 73,500$ lb

COLUMNS: $(\pi(15^2)/144)(12.25)(150) \times 10$ floors $= 90,200$ lb

TOTAL AXIAL DEAD: 618,965 lb

STRENGTH OF DESIGNED COLUMN: 410-345

30" $\phi$ w/ 10 #10



PURE AXIAL STRENGTH, $P_o$

$A_s = 10(1.27) = 12.7 \text{ in}^2$

$A_c = \pi r^2 - A_s$
$= \pi(15^2) - 12.7 \text{ in}^2$
$= 694.2 \text{ in}^2$


$P_o = 0.85 f'_c A_c + f_y A_s$
$P_o = 0.85(8)(694.2) + 12.7(60)$

$P_o = 5483^K$

$\phi_c = 0.65$

$0.8\phi_c P_o = 0.8(0.65)(5483) = 2851^K$

$2851^K > 619^K$ ∴ design good

NOTE: PURE AXIAL Assumption