Gea Johnson | Structural Option

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1000 CONNECTICUT AVENUE

Washington DC

FINAL THESIS REPORT:

STEEL DESIGN AND ANALYSIS

1000 CONNECTICUT AVENUE, NW OFFICE BUILDING

WASHINGTON DC

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GENERAL INFORMATION

Function:	Office/Retail/Parking Garage
Size:	555,000 SF (370,000 SF above grade and 185,000 below grade)
Height:	130 feet
Construction:	September 2009-February 2011
Construction Cost:	$60 million
Delivery Method:	Design-Bid-Build

Project Team

General Contractor:	Clark Construction Group
Architect-of-Record:	WDG Architecture
Design Architect:	Pei Cobb Freed and Partners
MEP:	Girard Engineering
Structural:	SK&A Structural Engineers
Civil:	VIKA, Inc
Owner Representative:	MJ Tyler and Associates

Structural

Foundation:

- Shallow foundation consisting of spread footings, strap beams, and 5" thick slab on grade

Superstructure:

- Typical floor consists of an 8" thick two-way flat slab with drop panels
- Framing system is composed of reinforced concrete columns with an average 30'x30' column spacing
- Reinforced concrete moment frame lateral system comprised of two-way flat slab and concrete columns

ARCHITECTURE

- Retail located on first level
- Two story intricate lobby space with Carrera marble and Chelmsford granite flooring and European white oak wood screens
- Office space located on levels 2-12
- Roof-top terrace with a green roof
- Curtain wall exterior façade comprised of stainless steel clad aluminum, floor to ceiling glass windows, and stone curtain panel wall units
- Combination of tall story heights and continuous floor to ceiling glass façade enables natural daylight to enter the building space
- four levels of underground parking

Perspective of lobby

Perspective of roof-top terrace with green roof

Perspective of typical office with floor to ceiling windows

MEP Systems

HVAC:

- Direct digital automatic temperature control
- Separate cooling loop on each floor
- Min. one VAV per 450 SF for perimeter zones and one per 1,000 SF for interior zones supplemental cooling

Lighting/Electrical:

- 460/265 volt high voltage; 208/120 volt low voltage
- 7 watts per rentable sf
- Uses Fluorescent, Metal Halide and LED lighting

CPEP WEBSITE: http://www.engr.psu.edu/ae/thesis/portfolios/2012/GJJ5014/index.html

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Executive Summary

1000 Connecticut Avenue is a 12 story, 565, 000 GSF commercial office building located at the corner of K Street and Connecticut Avenue in Washington D.C. The building is used primarily for office space, but also contains retail space on the first level, commercial office space on levels 3-12, a roof-top terrace with a green roof, and four levels of underground parking.

For this thesis report, 1000 Connecticut Avenue was re-located to Arlington, Virginia and the existing two-way flat slab floor system with lateral resisting concrete moment frames was re-designed as a composite steel floor gravity floor system with lateral resisting moment and braced frames. Before re-locating the building to Arlington, VA it was found that Washington D.C. has a zoning height limit of 130 ft. With the existing structure having a height of 130 ft., it was found that to use the new steel system the building would either need to be designed for a reduced number of stories or relocated to a region that does not have a height limit since the new steel system will increase the floor structural depth. To use the new steel structural system in Washington D.C., the structure would need to be re-designed for a reduced number of stories to maintain a minimum floor-to-ceiling height of 8'-6" and to remain within the restricted 130 ft. height limit. Reducing the number of stories from 12 to 11 was undesirable, therefore to create a fair comparison between the two systems the building was relocated to Arlington, VA, which does not have a height limit. The goal of the re-design was to

- increase the bay sizes to open the floor plan layout;
- increase floor-to-floor height to increase the openness of the space;
- Reduce the construction schedule;
- Reduce the structural system cost;
- Increase the annual revenue by increasing the rental value of the space and increasing the amount of rentable space

When designing the steel framing layout, a uniform layout was created to reduce the number of required skewed members and wider bays were created by removing certain existing column lines and relocating columns. Wider bays were created to open the floor plan and to increase the rental value of the space with reduced column obstructions and more rentable space. Maintaining an open floor layout was an important aspect of the re-design, therefore for the lateral system moment frames were used to avoid obstructions in the in the floor plan layout and braced frames were located around the elevator shafts and stairwell cores. The gravity system was designed as a composite steel system to achieve long spans while maintaining minimal structural depth. AISC 14th edition was used to design the gravity frame members. ETABS was used to analyze and design the lateral system. The lateral system design and analysis was based on the wind and seismic lateral loads calculated according to ASCE 7-10. The wind loads were determined by using Analytical Procedure (method 2) outlined in ASCE 7-10 and the seismic loads were determined by using the Equivalent Lateral Force Procedure outlined in ASCE 7-10. After designing the gravity and lateral systems, typical member connections were designed. The typical connections designed were orthogonal and skewed shear connections and a moment frame connection.

After designing the gravity and lateral systems, two breadth studies were conducted to determine how the new structural system will affect other aspects of the building. The first breath study was construction management impact. This breadth analyzed the impact of the structural system redesign on the superstructure cost; construction sequence of the existing system to the proposed construction sequence of the new structural system; site logistics of steel versus concrete; building LEED certification; and the anticipated revenue increase from the use of the new structural system. First the cost of the current structural system was compared to the cost estimate of the new structural system. In this portion of the analysis it was found that the new structural system will cost $5,994,630 more than the existing structural system. Second, the new structural system construction schedule was compared to the existing system construction schedule. It was found that the new structural system was erected 18 days earlier than the existing structural system, thus representing the use of the new system reduced the construction schedule. Third, how the construction site will have to be managed differently for steel compared to concrete was be evaluated. Using the existing 1000 Connecticut Avenue existing site for analysis, it was found that the site will be managed similarly for both materials. Fourth, the building LEED certification with the use of the new structural system was be compared to the existing building LEED certification and it after the analysis it was found that the building will maintain LEED Gold Certification. Last, the revenue obtained from the new structural system with wider bays and higher floor-to-ceiling heights was compared to the existing structural system's revenue. Wider bays and higher floor-to-ceiling heights increased the rental value of the floor space and therefore the building owner will be able charge higher rent which increased the revenue. The additional revenue obtained from using the new structural system is $3,705,450. This shows that even though the structural system costs more than the existing system, the amount of additional revenue obtained from using the new system is far more beneficial than using the existing system. Therefore the re-designed structural system with wider bays and floor-to-ceiling heights results in an overall very successful design with a reduced construction schedule and increased rental value. The proposed steel structural system is a viable alternative system to use in Arlington, VA since the new system has many additional benefits compared to the existing concrete structural system.

The second breadth studied was acoustics and lighting impact. This breadth involved determining the sound treatments required for a typical office space located in the new structural system. The analysis began by determining the level of speech privacy the common wall barrier between offices provided. It was shown that a 54 STC rated 8" partition wall with 2-layers of ½" thick gypsum wall board on both sides, staggered electrical boxes isolated with insulation, and 2 ½" metal studs spaced 24" o.c. and is very adequate for providing speech privacy for the offices housed in the new steel structural system. In addition, since the new structural system was designed for higher floor-to-ceiling heights, lighting illuminance applied to the work plane surfaces were affected. As a result, a lighting breadth was conducted by designing the lighting system for a typical office space using the existing floor-to-ceiling height of 8'-6" and checking to determine if the same lighting system can be used for the new floor-to-ceiling height of 10'-6". AGI was be used to design the lighting system for the space and the average illuminance in the space was compared to the target illuminance of the space. The IESNA Handbook 10th edition was used to determine the target illuminance and maximum power density for a private office space. It was found that the lighting system designed for the space with a floor-to-ceiling height of 8’-6” also achieved the target lighting illuminance for the space with a floor-to-ceiling height of 10’-6”.

The appendices in this report include hand calculations for wind, seismic, snow and gravity loads; gravity system design; construction management breadth calculations; floor plans and a building section.

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Building Introduction

1000 Connecticut Avenue, NW Office Building is a new 12 story office building located at the northwest intersection of K Street and Connecticut Avenue in Washington DC, as can be seen in Figure 1. The 1000 Connecticut Avenue Office building is designed to achieve LEED Gold certification upon completion. Despite being used primarily for office space, the building is comprised of mix occupancies, which include: office space, a gymnasium, retail, and parking garages. The structure has 4 levels of underground parking. The building's total square footage is 555,000 SF with 370,000 SF above grade and 185,000 SF below grade.

Figure 1 Building Site

To create a new Washington landmark, the building is designed to complement surrounding institutions by blending both traditional and modern materials. The facade consists of a glass, stainless steel and stone panel curtain wall system. Exterior and interior aluminum and glass storefront windows and doors are on the ground level. The lobby and retail space are located on the 1st level, which has a 12'-6 1/2" floor-to-floor story height. A canopy facing K Street brings attention to the main lobby entrance, as can be seen in Figure 2.

Figure 2 Main Lobby Entrance facing K Street (left) and perspective of curtain wall system (right)

Beyond the main entrance is a two story intricate lobby space with carrera marble and Chelmsford granite flooring, aluminum spline panels integrated with glass fiber reinforced gypsum (GFRG) ceiling tiles and European white oak wood screens, as can be seen in Figure 3.

Figure 3 Perspective of lobby

The retail space is broken down into several retail stores facing K Street and Connecticut Avenue. These retail stores are housed behind storefront glass to enable display of merchandise to potential customers. The 2nd-12th levels have 10’-7 ½” floor-to-floor story heights. Housed on the typical levels (3rd-12th) is the office space. A combination of tall story heights and a continuous floor to ceiling glass façade enables natural daylight to enter the building space as well as provides scenery to the Washington monuments, Farragut Park , and the White House, as can be seen in Figure 4.

Figure 4 Perspective of typical office with floor-to-ceiling windows that supply views to the city

In addition, located on the penthouse level is a roof-top terrace with a green roof and a mechanical penthouse, as can be seen in Figure 5.

Figure 5 Perspective of green roof on roof-top terrace and mechanical penthouse

Housed on the basement levels (B1-B4) are underground parking and a fitness center. A total of 253 parking spaces are provided; level B1 has 19 parking spaces; level B2 has 74 parking spaces; level B3 has 78 parking spaces; level B4 has 82 parking spaces. In addition, the fitness center is located on level B1.

Existing Structural Overview

1000 Connecticut Avenue Office Building's structural system is comprised of a reinforced concrete flat slab floor system with drop panels and a bay spacing of approximately 30 feet by 30 feet. The slab and columns combined perform as a reinforced concrete moment frame. The substructure and superstructure floor systems are both comprised of an 8" thick two-way system with #5 reinforcing bars spaced 12" on center in both the column and middle strips and 8" thick drop panels. The below grade parking garage ramp is comprised of a 14" thick slab with #5 reinforcing bars provided both top and bottom with a spacing of 12" on center.

Foundation

ECS Mid-Atlantic, LLC performed a geotechnical analysis of the building's site soil conditions as well as provided recommendations for the foundation. A total of five borings were observed in the geotechnical analysis. It was determined that a majority of the site's existing fill consists of a mixture of silt, sand, gravel, and wood. The natural soils consisted of sandy silt, sand with silt, clayey gravel, silty gravel, and silty sand. The soil varies from loose to extremely dense in relative density. Based on the samples recovered from the rock coring operations, the rock is classified as completely to moderately weathered, thinly bedded, and hard to very hard gneiss.

At the time of the study, the groundwater was recorded at a boring depth of 7.5 feet below the existing ground surface. The shallow water table is located at an elevation of 35 to 38 feet in the vicinity of the site.

1000 Connecticut Avenue, NW Office Building is supported by a shallow foundation consisting of column footings and strap beams, as can be seen in Figure 6. The typical column footing sizes are 4'-0" x 4'-0", 5'-0" x 5'-0", and 4'-0" x 8'-0".

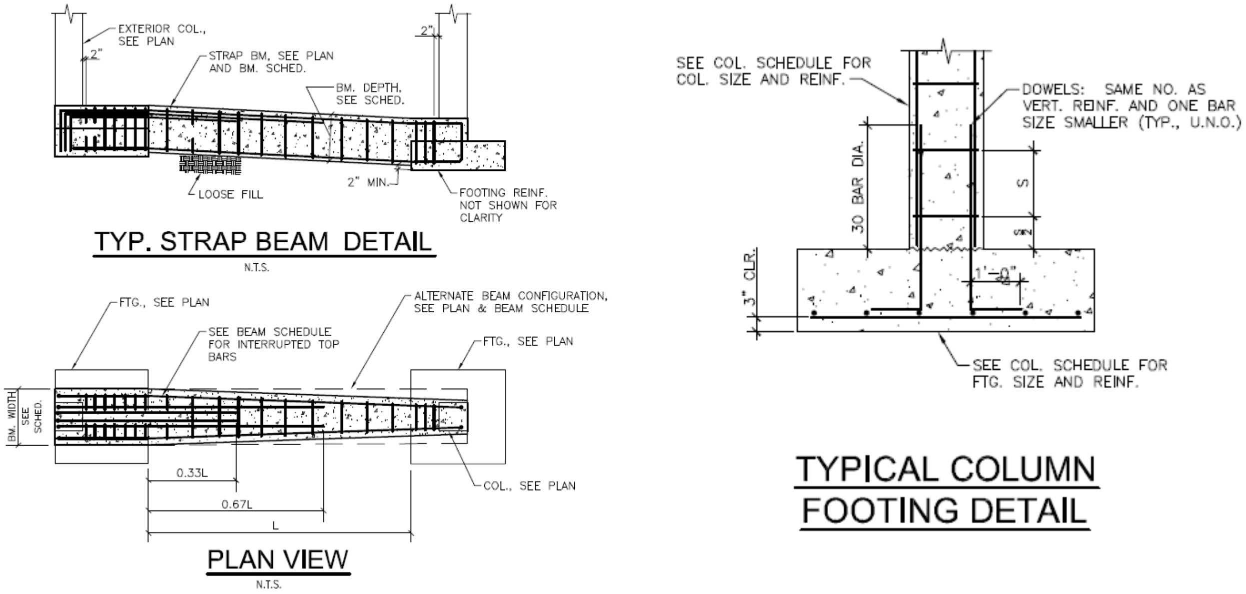


Figure 6 Details of typical strap beam and column footing

The footings bear on 50 KSF competent rock. The Strap beams (cantilever footings) are used to prevent the exterior footings from overturning by connecting the strap beam to both the exterior footing and to an adjacent interior footing. A simplified foundation plan can be seen in Figure 7.

The slab on grade is 5” thick, 5000 psi concrete with 6x6-W2.9xW2.9 wire welded fabric on a minimum 15 mil Polyethylene sheet over 6” washed crushed stone. The foundation walls consists of concrete masonry units vertically reinforced with #5 bars at 16” on center and horizontally reinforced with #4 bars at 12” on center and are subjected to a lateral load (earth pressure) of 45 PSF per foot of wall depth.

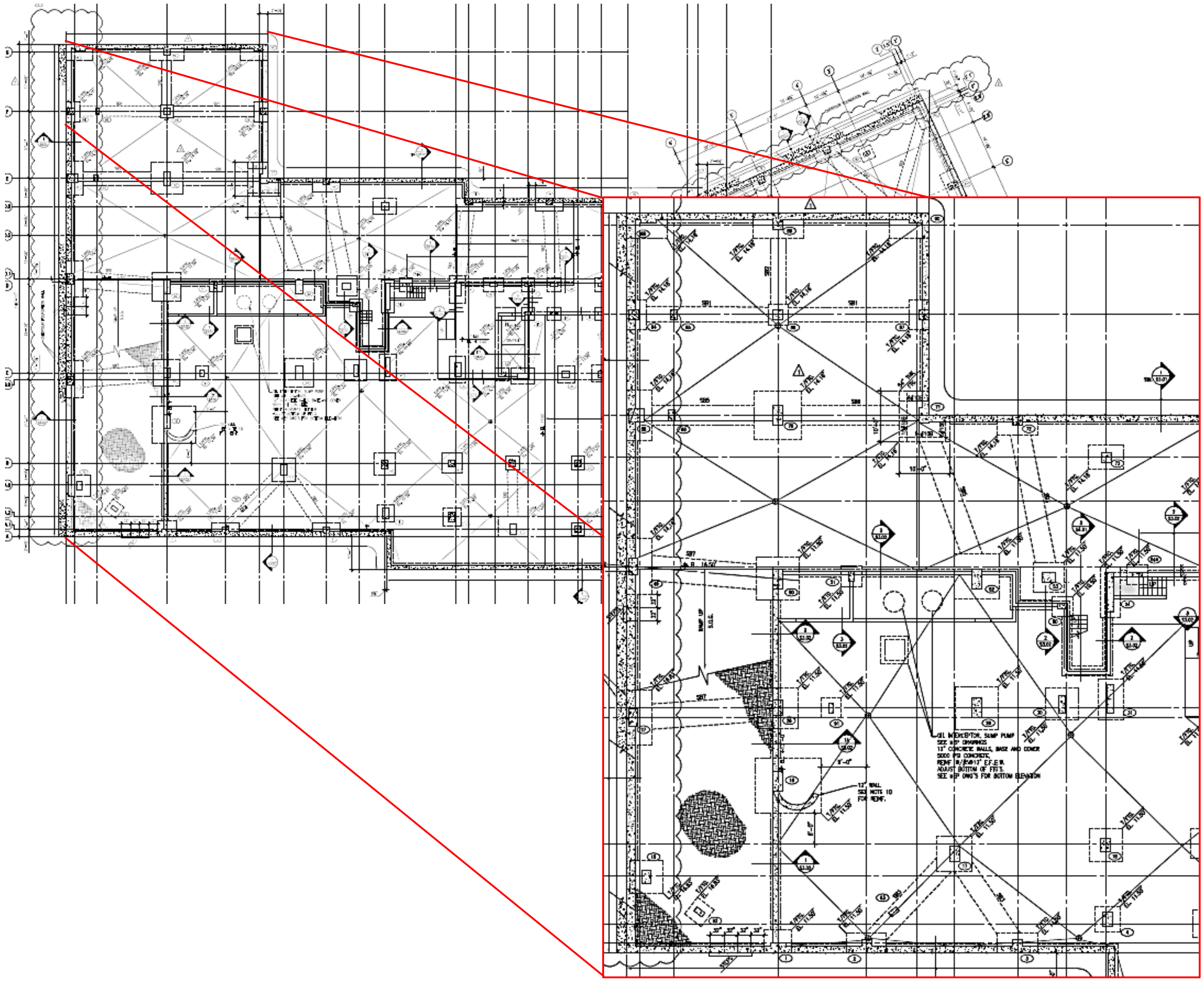


Figure 7 Foundation plan

Framing and Floor System

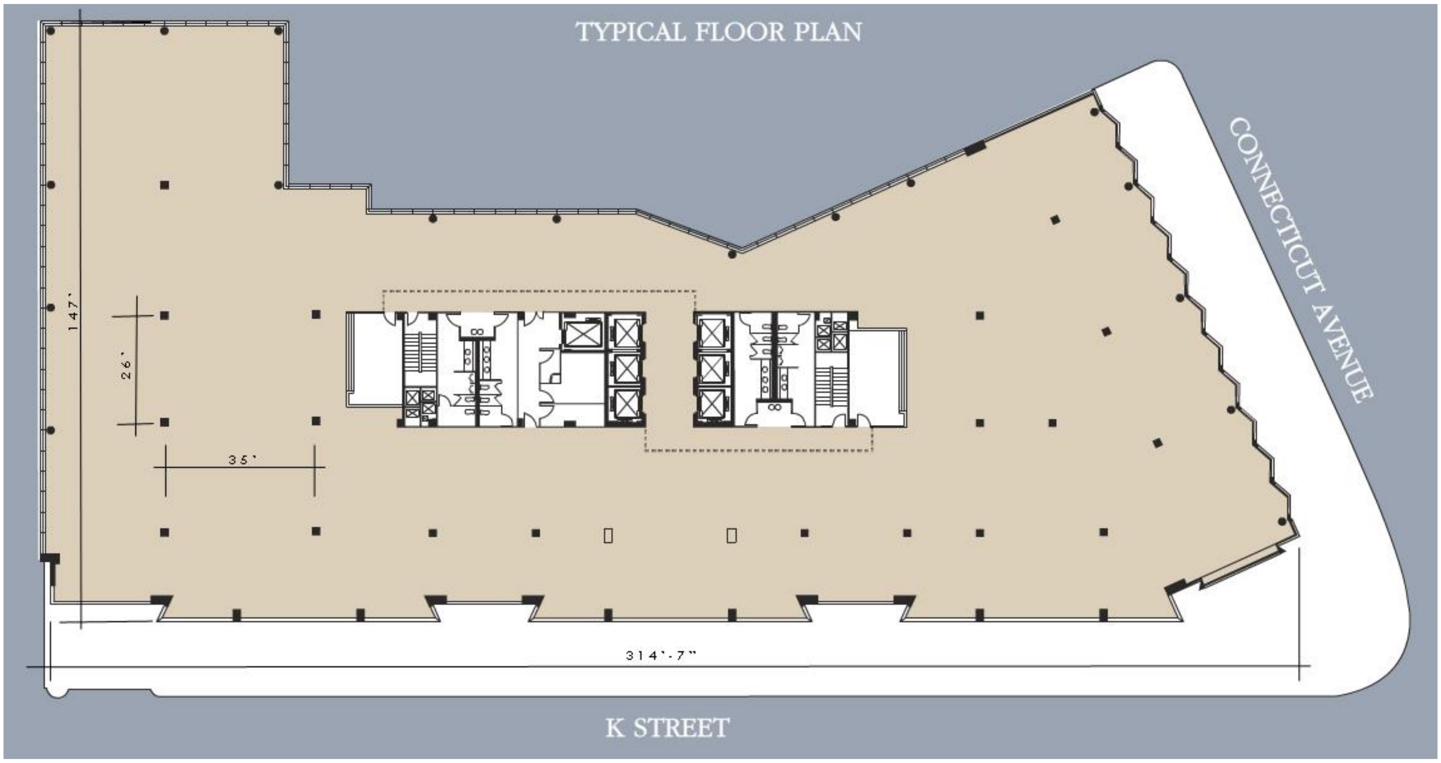
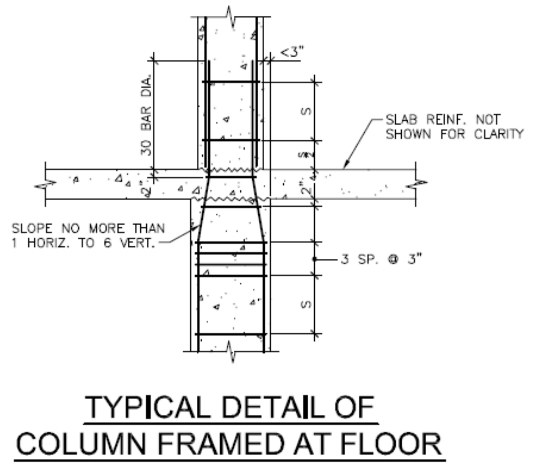


Figure 8 Floor plan displaying column locations and bays

The framing system is composed of reinforced concrete columns with an average column-to-column spacing of 30'x30', as can be seen in Figure 8. The columns have a specified concrete strength of f'c=8000 psi for columns on levels B4 to level 3, f'c=6000 psi for columns on levels 4-7, and f'c=5000 psi for columns on levels 8-mechanical penthouse. The columns are framed at the concrete floor, as can be seen in Figure 9, and the columns vary in size. The most common column sizes are 24"x24", 16"x48", and 24"x30". The column capitals are 6" thick, measured from the bottom of the drop panel, extending 6" all around the face of the column, as can be seen in Figure 10.



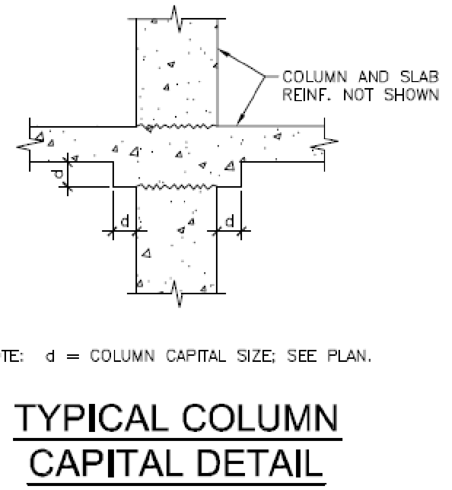


Figure 9 Typical Detail of column framed at the floor

Figure 10 Typical column capital detail

The typical floor system is comprised of an 8” thick two-way flat slab with drop panels reinforced with #5 bottom bars spaced 12” on center in both the column and middle strips, as can be seen in Figure 11.

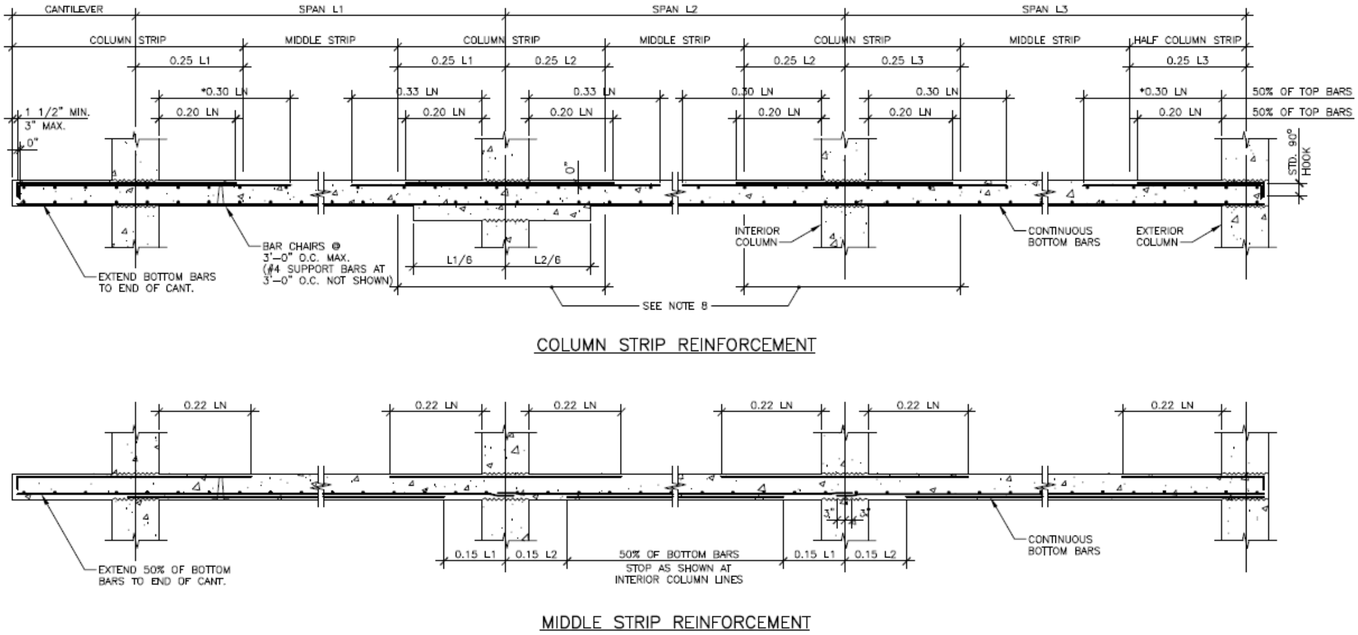


Figure 11 Typical two-way slab reinforcing detail

The individual drop panels are 8” thick, extending a distance d/6 from the centerline of the column, as can be seen in Figure 12.

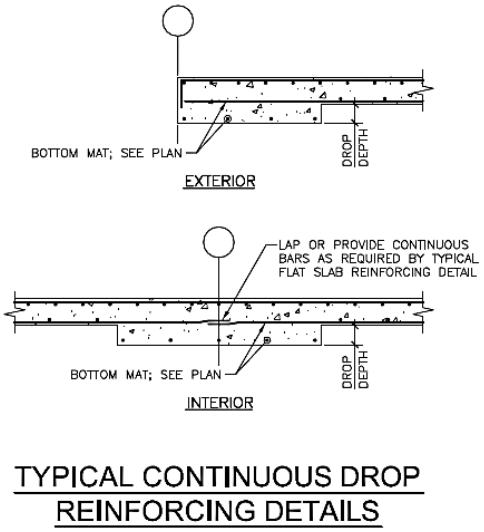


Figure 12 Typical Continuous drop panel

A 36” wide by 3 ½” deep continuous drop panel is located around the perimeter on all floor levels. Levels 3-12 are supported by four post-tension beams above the lobby area. Due to the two story lobby, there’s a large column-to-column spacing. As a result, post tension beams are used to support the slab on levels 3-12 located above the lobby. In addition, four post-tension beams support the slab on levels 3-12 that are located above the two-story parking deck, which also has a large column-to-column spacing, as can be seen in Figure 13.

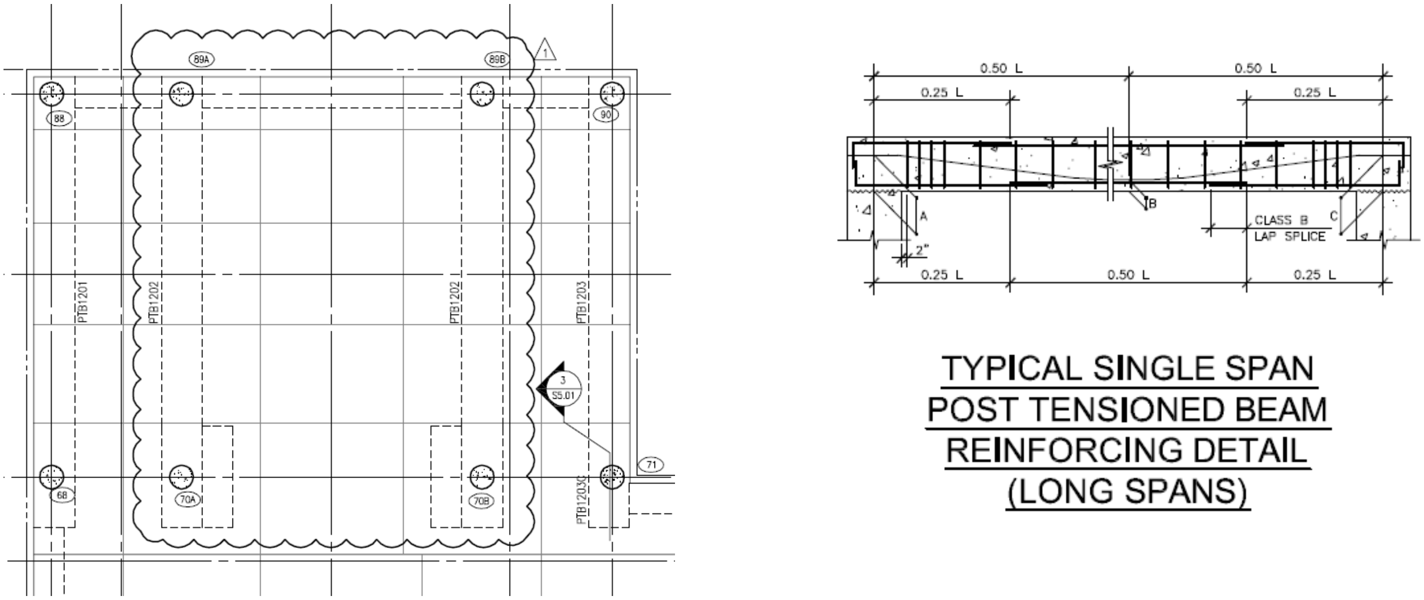
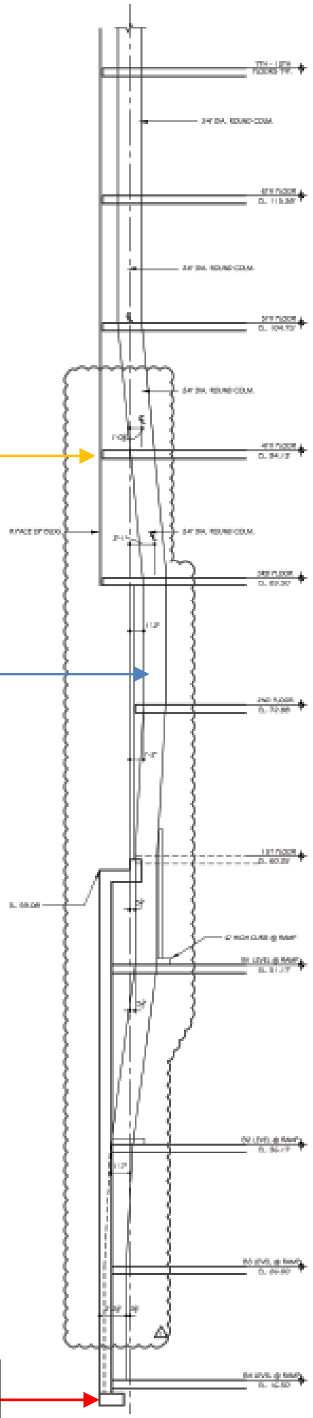


Figure 13 Plan view and typical detail of Post-tension beams supporting slab on levels above two-story loading dock

Lateral System

The lateral system is comprised of a reinforced concrete moment frame. The columns and slab are poured monolithically, thus creating a rigid connection between the elements. The curtain wall is attached to the concrete slab, which puts the slab in bending. The curtain wall transfers the lateral load to the slab. The slab then transfers the lateral load to the columns and in turn the columns transfer the load to the foundation. Transfer girders on the lower level are used to transfer the loads from the columns that do not align with the basement columns in order to transfer the load to the foundation. A depiction of how the lateral load is transferred through the system can be seen in Figure 14.



Curtain wall collects the lateral load and directly transfers the load to the concrete slab

The slab transfers the lateral load to the columns

The columns transfer the lateral load to the foundation

Figure 14 Lateral load path depiction

Roof System

The main roof framing system is supported by an 8”thick concrete slab with #5 bars spaced 12” on center at the bottom in the east-west direction. The slab also has 8” thick drop panels. The penthouse framing system is separated into two roofs: Elevator Machine Room roof and the high roof. The elevator machine room roof framing system is supported by 14” and 8” thick slab with #7 bars with 6” spacing on center top and bottom in the east-west direction.

Design Codes

According to sheet S601, the original building was designed to comply with the following:

- 2000 International Building Code (IBC 2000)
- Building Code Requirements for Structural Concrete (ACI 318)
- Specifications for Structural Concrete (ACI 301)
- Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315)
- Specification for the Design, Fabrication and Erection of Structural Steel for Buildings (AISC manual), Allowable Strength Design (ASD) method

The codes that were used to complete the analyses within this report are the following:

- Minimum Design Loads for Building and Other Structures (ASCE 7-10)
- AISC Steel Construction Manual, 14th Edition, Load and Resistance Factor Design (LRFD) method
- Vulcraft Steel Roof and Floor Deck Manual, 2008

Structural Materials

Table 1 below shows the several types of materials that were used for this project according to the general notes page of the structural drawings on sheet S601.

Concrete (Cast-in-Place)		
Usage	**Weight**	**Strength (psi)**
Spread Footings	Normal	4000
Strap Beams	Normal	4000
Foundation Walls	Normal	4000
Formed Slabs and Beams	Normal	5000
Columns	Normal	Varies (based on column schedule)
Concrete Toppings	Normal	5000
Slabs on Grade	Normal	5000
Pea-gravel concrete (or grout)	Normal	2500 (for filling CMU units)
All other concrete	Normal	3000
Reinforcing Steel		
Type	**Standard**	**Grade**
Deformed Reinforcing Bars	ASTM A615	60
	ASTM A775	N/A
Welded Wire Fabric	ASTM A185	N/A
Reinforcing Bar Mats	ASTM A184	N/A
Post-Tensioning (Unbonded)		
Type	**Standard**	**Strength (ksi)**
Prestressed Steel (seven wire low-relaxation or stressed relieved strand)	ASTM A416	270
Miscellaneous Steel		
Type	**Standard**	**Grade**
Structural Steel	ASTM A36	N/A
Bolts	ASTM A325	N/A
Welds	AWS	N/A

Table 1 Design materials

Gravity Loads

For this technical report, live loads and snow loads were compared to the loads listed on the structural drawings. In addition, dead loads were calculated and assumed in order to spot check gravity members and typical columns. The system evaluations were then compared to the original design. The hand calculations for the gravity member checks can be found in Appendix A.

Dead and Live Loads

Table 2 below is a list of the live loads in which the project was designed for compared to the minimum design live loads outlined in ASCE 7-10.

Floor Live Loads		
Occupancy	Design Load (psf)	ASCE 7-10
Parking Levels	50	40
Retail	100	100
Vestibules & Lobbies	100	100
Office Floors	100=(80 psf+ 20 psf partitions)	70= (50 psf + 20 psf partitions)
Corridors	100	100 on ground level 80 above 1st level
Stairs	100	100
Balconies & Terraces	100	100
Mechanical Room	150	-
Pump Room, Generator Room	150	-
Light Storage	125	125
Loading Dock, Truck Bays	350	250
Slab On Grade	100	-
Green Roof Areas	30	-
Terrace	100	100

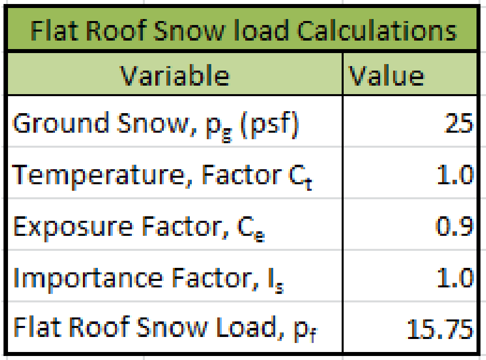
Table 2 Summary of design live loads compared to minimum design live loads on ASCE 7-10

Note: - Means the load for the specified occupancy was not provided

Based on the above design live loads, certain spaces were designed for higher loads to create a more conservative design and to allow for design flexibility. For this technical report, the design live loads were used for the gravity member analyses.

Snow Load

The snow load was determined in conformance to chapter 7 in ASCE 7-10. A summary of the snow drift parameters are shown in table 3.



Flat Roof Snow load Calculations	
Variable	Value
Ground Snow, p_g (psf)	25
Temperature, Factor C_t	1.0
Exposure Factor, C_e	0.9
Importance Factor, I_s	1.0
Flat Roof Snow Load, p_f	15.75

Table 3 Summary of roof snow calculations

According to structural drawing sheet S601, the flat roof snow load was 22.5 psf whereas 15.75 psf was calculated in this technical report. The 15.75 psf value was used to determine the snow load and snow drifts. These subsequent calculations can be found in Appendix A.

Table 4 below is a list of the dead loads that were used for the gravity spot checks. The superimposed dead loads for the floor levels and roofs were assumed.

Dead Loads	
Normal Weight Concrete	150 pcf
Curtain Wall	250 plf
Precast Panels	450 plf
Floor Superimposed Dead Load (ceiling, lights, MEP, miscellaneous)	10 psf
Main Roof Superimposed Dead Load (ceiling, lights, MEP, miscellaneous)	10 psf
Penthouse Roof Superimposed Dead Loads	5 psf

Table 4 Summary of dead loads

Lateral Loads

In this report, wind and seismic lateral loads were calculated to determine the loads acting on the structure’s lateral system. To perform manual calculations for determining the lateral loads, simplifying assumptions were made. In addition, it was determined how much of the story force was distributed to each moment frame, which will be discussed later in this report. The hand calculations associated with the wind and seismic loads determination can be found in Appendices B and C.

Wind Loads

Wind loads were determined using the Main Wind Force Resisting System (MWFRS) procedure (method 2) in conformance to Chapters 26 and 27 outlined in ASCE 7-10. Due to the building’s complex geometry, a rectangular building shape was assumed to simplify the wind load analysis, as can be seen in Figure 17.

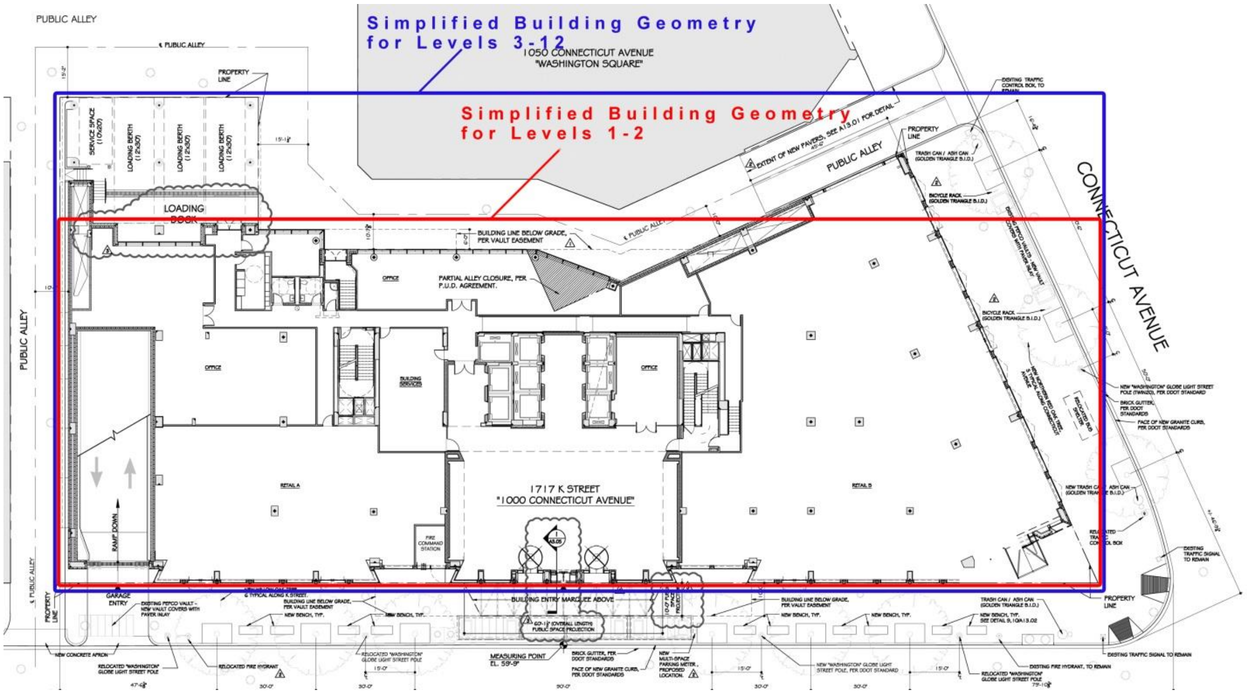
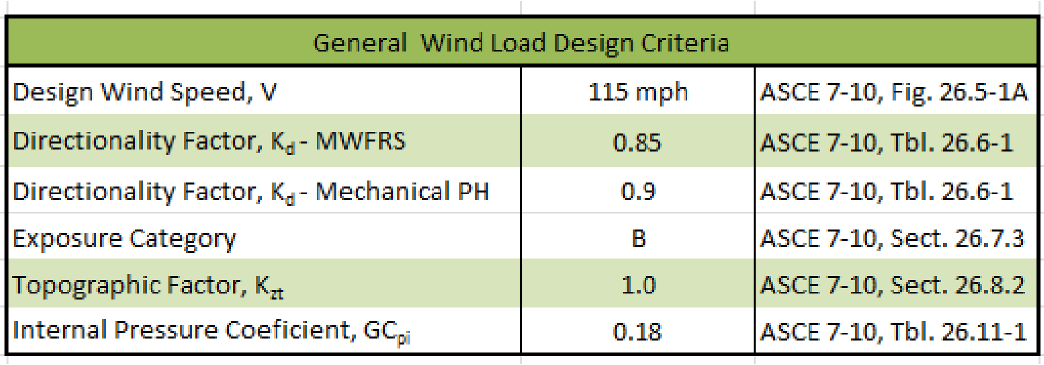


Figure 17 Simplified building shape for wind load analysis

Most of the calculations for determining the wind pressures and story forces were performed in Microsoft Excel. In the analysis, windward, leeward, sidewall, and roof suction pressures were determined. Internal pressures were neglected in calculating the design wind pressure because internal pressures do not contribute towards the external wind pressures acting on the building.

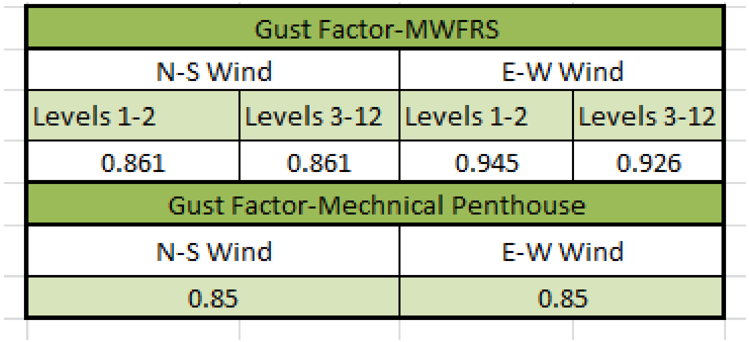
The general wind load design criteria and guest effect factors can be found in Tables 5 and 6. The calculated approximate lower- bound natural frequency for the building was 0.544 Hz, which is less than 1 Hz, therefore the gust factors were calculated in the event the building is flexible.

Further, wind pressures in the N-S and E-W directions can be seen in Tables 7 and 8 with the corresponding vertical profile sketch of the wind pressures shown in Figures 18 and 19. The story forces were then determined based on the wind pressures. The resulting base shears were 1401 k for the N-S direction and 553 k in the E-W direction. The story forces and overturning moments for both the N-S and E-W directions can be found in Tables 9 and 10 along with the vertical profile of the story forces in Figures 20 and 21.



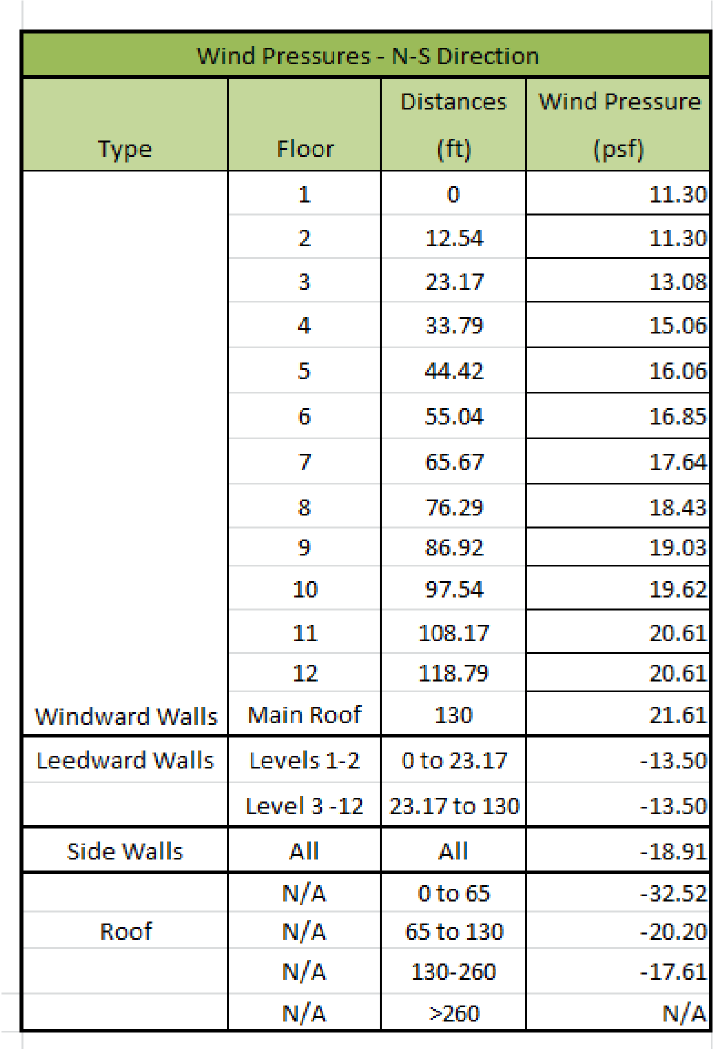
General Wind Load Design Criteria		
Design Wind Speed, V	115 mph	ASCE 7-10, Fig. 26.5-1A
Directionality Factor, K_d - MWFRS	0.85	ASCE 7-10, Tbl. 26.6-1
Directionality Factor, K_d - Mechanical PH	0.9	ASCE 7-10, Tbl. 26.6-1
Exposure Category	B	ASCE 7-10, Sect. 26.7.3
Topographic Factor, K_{zt}	1.0	ASCE 7-10, Sect. 26.8.2
Internal Pressure Coeficient, GC_{pi}	0.18	ASCE 7-10, Tbl. 26.11-1

Table 5 General wind design criteria



Gust Factor-MWFRS			
N-S Wind		E-W Wind	
Levels 1-2	Levels 3-12	Levels 1-2	Levels 3-12
0.861	0.861	0.945	0.926
Gust Factor-Mechnical Penthouse			
N-S Wind		E-W Wind	
0.85		0.85	

Table 6 Guest Factors



Wind Pressures - N-S Direction			
Type	Floor	Distances (ft)	Wind Pressure (psf)
Windward Walls	1	0	11.30
	2	12.54	11.30
	3	23.17	13.08
	4	33.79	15.06
	5	44.42	16.06
	6	55.04	16.85
	7	65.67	17.64
	8	76.29	18.43
	9	86.92	19.03
	10	97.54	19.62
	11	108.17	20.61
	12	118.79	20.61
	Main Roof	130	21.61
Leedward Walls	Levels 1-2	0 to 23.17	-13.50
	Level 3 -12	23.17 to 130	-13.50
Side Walls	All	All	-18.91
Roof	N/A	0 to 65	-32.52
	N/A	65 to 130	-20.20
	N/A	130-260	-17.61
	N/A	>260	N/A

Table 7 N-S Wind Pressures

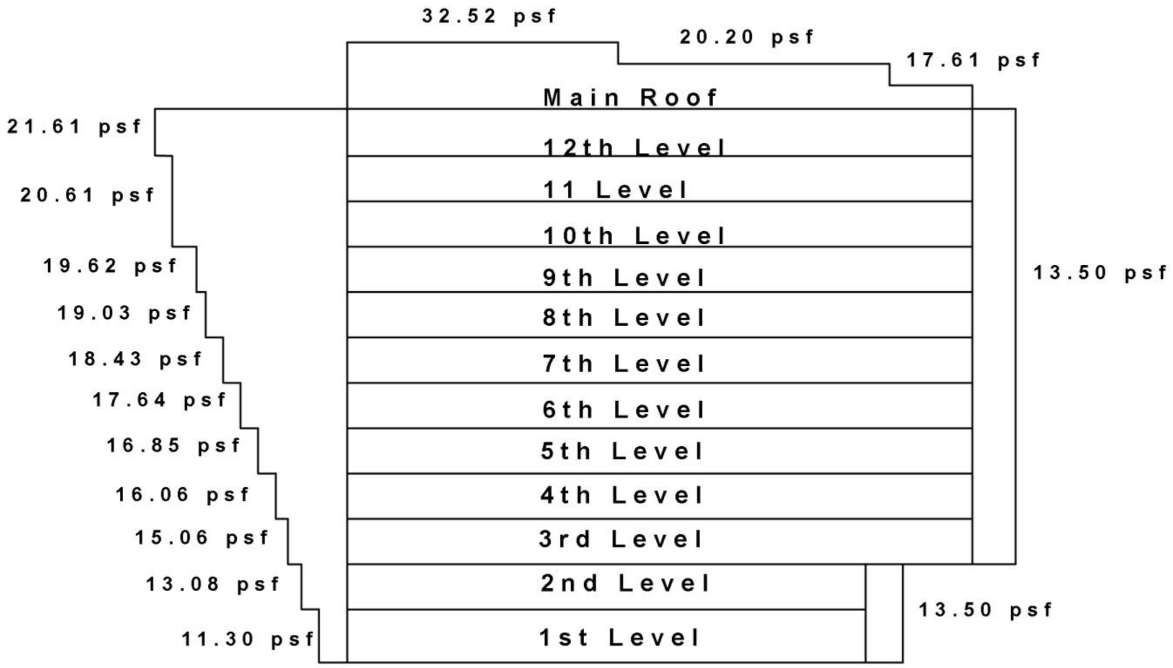
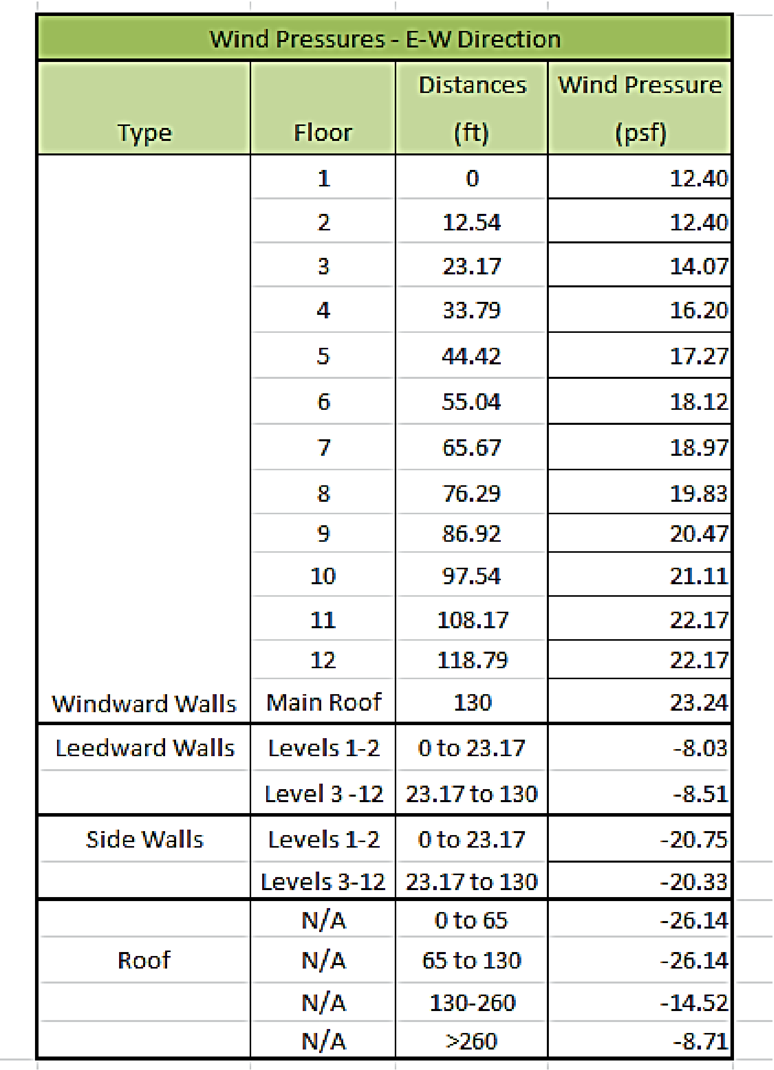


Figure 18 N-S wind pressure vertical pressure sketch



Wind Pressures - E-W Direction			
Type	Floor	Distances (ft)	Wind Pressure (psf)
Windward Walls	1	0	12.40
	2	12.54	12.40
	3	23.17	14.07
	4	33.79	16.20
	5	44.42	17.27
	6	55.04	18.12
	7	65.67	18.97
	8	76.29	19.83
	9	86.92	20.47
	10	97.54	21.11
	11	108.17	22.17
	12	118.79	22.17
	Main Roof	130	23.24
Leedward Walls	Levels 1-2	0 to 23.17	-8.03
	Level 3 -12	23.17 to 130	-8.51
Side Walls	Levels 1-2	0 to 23.17	-20.75
	Levels 3-12	23.17 to 130	-20.33
Roof	N/A	0 to 65	-26.14
	N/A	65 to 130	-26.14
	N/A	130-260	-14.52
	N/A	>260	-8.71

Table 8 E-W wind pressures

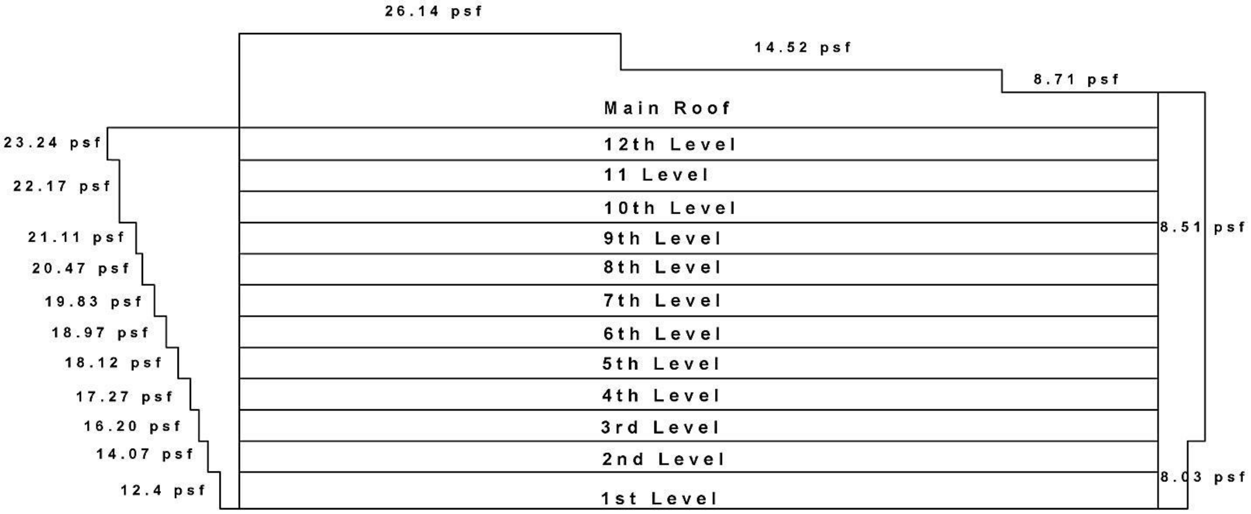


Figure 19 E-W vertical wind pressure profile

Wind Forces - N-S Direction										
Floor	Elevation (ft)	Tributary Below			Tributary Above			Story Force (Kips)	Story Shear (Kips)	Overturning Moment (K-ft)
		Height (ft)	Length (ft)	Area (ft^2)	Height (ft)	Length (ft)	Area (ft^2)			
PH Roof	148.5	18.5	199.83	3696.86	0	199.83	0	142.82	142.82	21208.42
Main Roof	130	5.31	314.58	1671.21	0	314.58	0	58.68	201.49	7627.83
12	118.79	5.31	314.58	1671.21	5.31	314.58	1671.21	115.69	317.19	13743.40
11	108.17	5.31	314.58	1671.21	5.31	314.58	1671.21	114.04	431.23	12335.55
10	97.54	5.31	314.58	1671.21	5.31	314.58	1671.21	112.38	543.61	10961.76
9	86.92	5.31	314.58	1671.21	5.31	314.58	1671.21	109.73	653.34	9537.91
8	76.29	5.31	314.58	1671.21	5.31	314.58	1671.21	107.74	761.09	8219.83
7	65.67	5.31	314.58	1671.21	5.31	314.58	1671.21	105.43	866.51	6923.30
6	55.04	5.31	314.58	1671.21	5.31	314.58	1671.21	102.78	969.29	5656.76
5	44.42	5.31	314.58	1671.21	5.31	314.58	1671.21	100.13	1069.41	4447.57
4	33.79	5.31	314.58	1671.21	5.31	314.58	1671.21	97.14	1166.56	3282.49
3	23.17	5.31	314.58	1671.21	5.31	314.58	1671.21	92.17	1258.73	2135.69
2	12.54	6.27	314.58	1972.42	5.31	314.58	1671.21	93.35	1352.08	1170.63
1	0	0	314.58	0.00	6.27	314.58	1972.42	48.92	1401.00	0.00
									Total Base Shear =	1401 K
									Total Overturning Moment =	107,251 K-ft

Table 9 N-S Story forces, base shear, and overturning moment

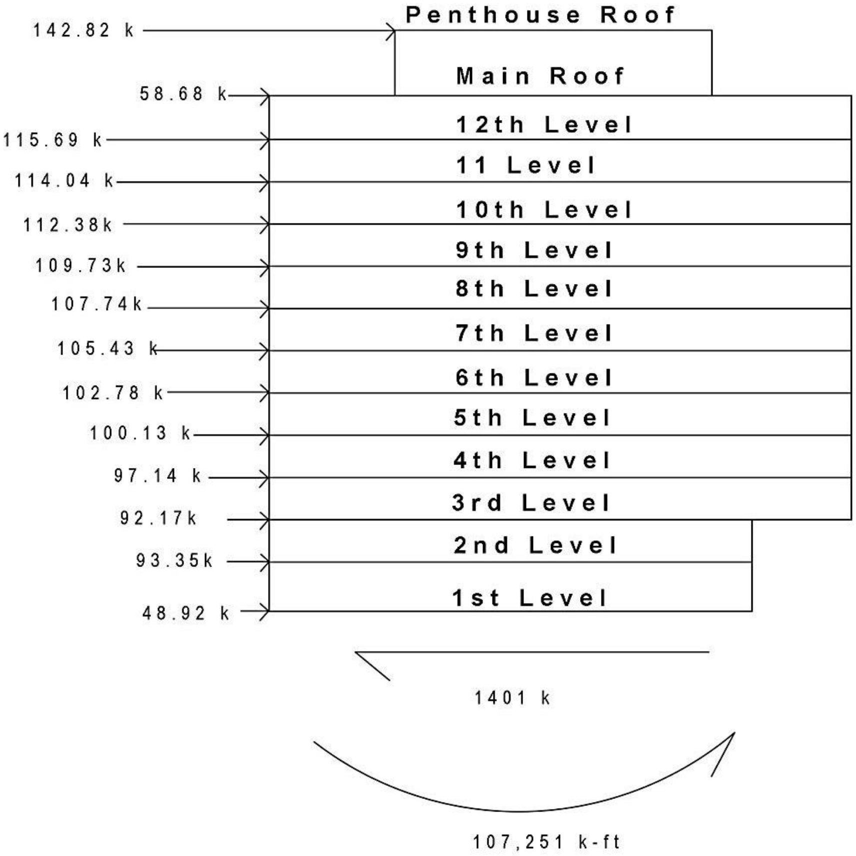
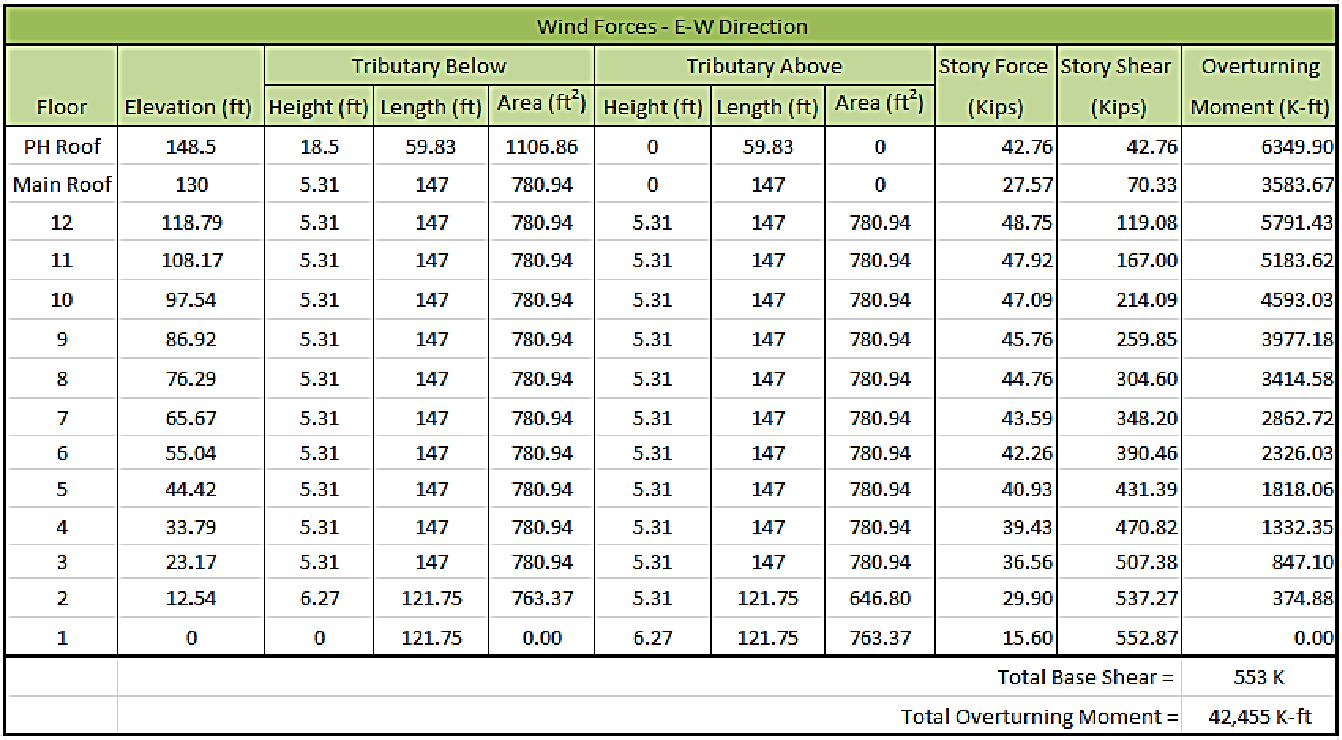


Figure 20 Vertical profile of story forces in N-S direction



Wind Forces - E-W Direction

		Tributary Below			Tributary Above			Story Force	Story Shear	Overturning
Floor	Elevation (ft)	Height (ft)	Length (ft)	Area (ft^2)	Height (ft)	Length (ft)	Area (ft^2)	(Kips)	(Kips)	Moment (K-ft)
PH Roof	148.5	18.5	59.83	1106.86	0	59.83	0	42.76	42.76	6349.90
Main Roof	130	5.31	147	780.94	0	147	0	27.57	70.33	3583.67
12	118.79	5.31	147	780.94	5.31	147	780.94	48.75	119.08	5791.43
11	108.17	5.31	147	780.94	5.31	147	780.94	47.92	167.00	5183.62
10	97.54	5.31	147	780.94	5.31	147	780.94	47.09	214.09	4593.03
9	86.92	5.31	147	780.94	5.31	147	780.94	45.76	259.85	3977.18
8	76.29	5.31	147	780.94	5.31	147	780.94	44.76	304.60	3414.58
7	65.67	5.31	147	780.94	5.31	147	780.94	43.59	348.20	2862.72
6	55.04	5.31	147	780.94	5.31	147	780.94	42.26	390.46	2326.03
5	44.42	5.31	147	780.94	5.31	147	780.94	40.93	431.39	1818.06
4	33.79	5.31	147	780.94	5.31	147	780.94	39.43	470.82	1332.35
3	23.17	5.31	147	780.94	5.31	147	780.94	36.56	507.38	847.10
2	12.54	6.27	121.75	763.37	5.31	121.75	646.80	29.90	537.27	374.88
1	0	0	121.75	0.00	6.27	121.75	763.37	15.60	552.87	0.00
									Total Base Shear =	553 K
									Total Overturning Moment =	42,455 K-ft

Table 10 E-W Story forces, base shear, and overturning moment

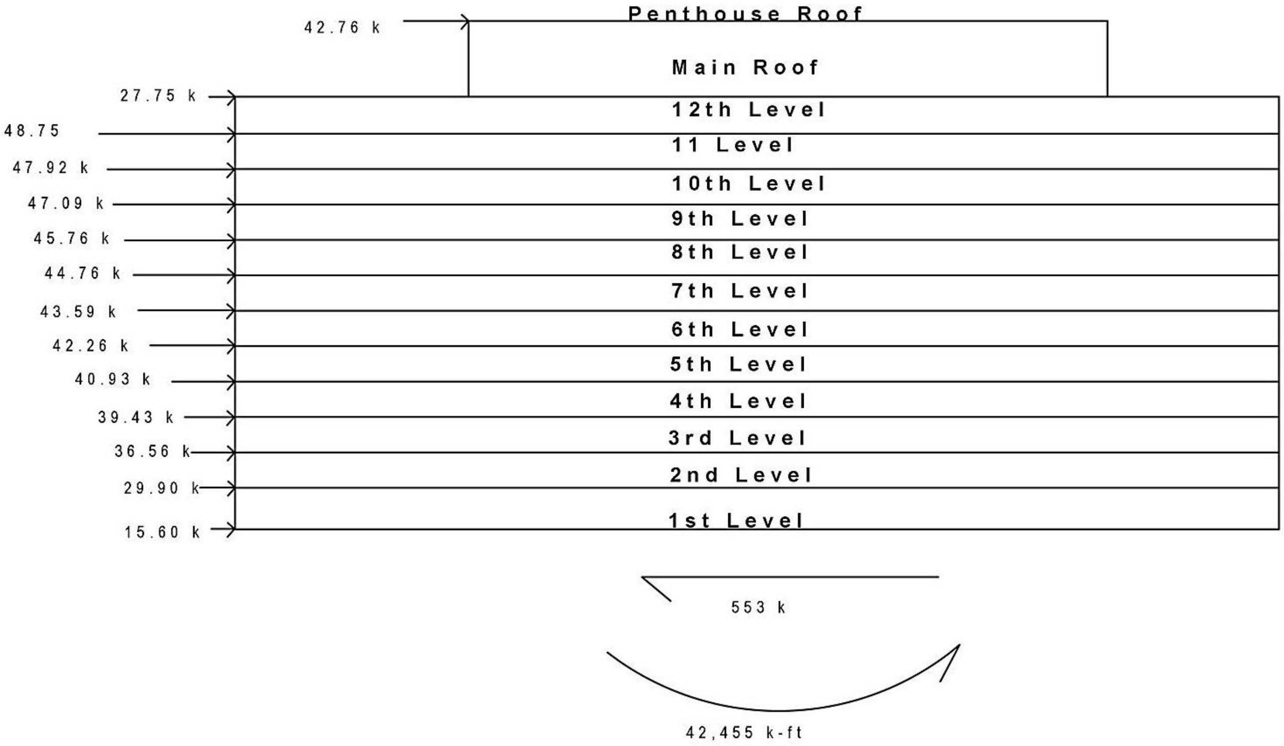


Figure 21 Vertical profile of story forces in E-W direction

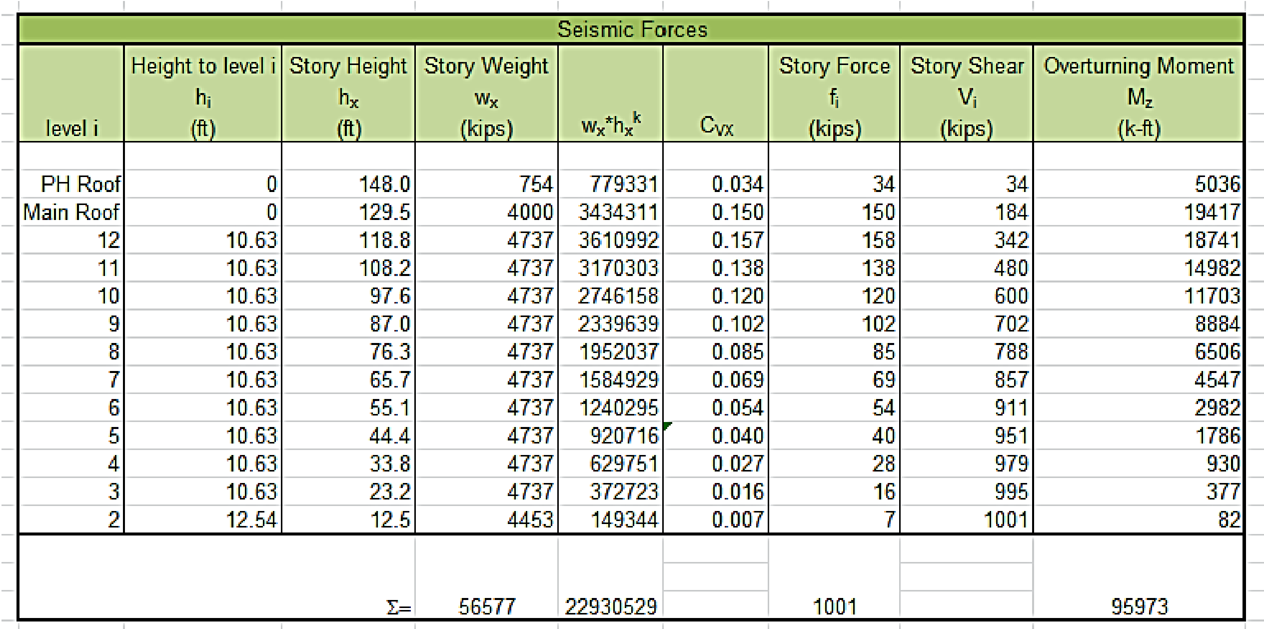
Seismic Loads

Seismic loads were determined using the Equivalent Lateral Force Procedure outlined in Chapters 11 and 12 in ASCE 7-10. For analysis, the 1st level weight was neglected and thus the 2nd-12th levels, main roof, and penthouse were considered for building weight calculations. The typical floor level slab thickness is 8” with small areas consisting of 12” slabs. For calculation simplification, a uniform slab thickness of 8” was used.

Since the lateral resisting system consists of a reinforced concrete moment frame in both the N-S and E-W directions, one analysis was performed to determine the seismic story forces and base shear for both directions.

Since this building has several stories above grade, building weight was determined by calculating the dead weight for the typical floor level and applying that story weight to the other floor levels (levels 2-12). The weight on the main roof and penthouse roof were calculated separately. The weight included for summing the total building weight were the weight of the slabs, columns, drop panels, and superimposed dead loads.

After the analysis, the determined base shear was 1001 kips and an overturning moment of 95,973 K-ft. Refer to Table 11 for seismic force analysis results.



Seismic Forces								
level i	Height to level i h_i (ft)	Story Height h_x (ft)	Story Weight w_x (kips)	$w_x*h_x^k$	Cvx	Story Force f_i (kips)	Story Shear V_i (kips)	Overturning Moment M_z (k-ft)
PH Roof	0	148.0	754	779331	0.034	34	34	5036
Main Roof	0	129.5	4000	3434311	0.150	150	184	19417
12	10.63	118.8	4737	3610992	0.157	158	342	18741
11	10.63	108.2	4737	3170303	0.138	138	480	14982
10	10.63	97.6	4737	2746158	0.120	120	600	11703
9	10.63	87.0	4737	2339639	0.102	102	702	8884
8	10.63	76.3	4737	1952037	0.085	85	788	6506
7	10.63	65.7	4737	1584929	0.069	69	857	4547
6	10.63	55.1	4737	1240295	0.054	54	911	2982
5	10.63	44.4	4737	920716	0.040	40	951	1786
4	10.63	33.8	4737	629751	0.027	28	979	930
3	10.63	23.2	4737	372723	0.016	16	995	377
2	12.54	12.5	4453	149344	0.007	7	1001	82
		Σ=	56577	22930529		1001		95973

Table 11 Story forces, base shear, and overturning moment due to seismic loads

Problem Statement

1000 Connecticut Avenue's structural system currently consists of a two-way flat slab floor system supported by concrete columns with an average spacing of 30ft x 30 ft. The current lateral system consists of concrete moment frames comprised of the concrete columns and the two-way flat slab system. The in-depth analyses performed in technical reports 1-3 showed that the existing structural system is adequate to support the combined lateral and gravity loads and meets serviceability requirements.

The author of this report was extremely interested in steel design. Therefore a scenario was created in which 1000 Connecticut Avenue NW Office Building was re-located to Arlington, VA and re-designed as a steel frame system consisting of two lateral systems: moment frames and braced frames. The new structural system will be analyzed to determine whether:

- the overall building cost can be reduced;
- the construction schedule can be reduced;
- LEED certification will remain unchanged;
- the bay sizes and floor-to-ceiling heights can be increased;
- the annual revenue can be increased

Since the existing 12 story structure is located in Washington DC, which has a zoning building height restriction of 130 ft., in order to use the new steel system the structural system will have to be designed as 11 stories to stay within the height limit or re-located to an area that does not have a height restriction. To make a fair comparison between the two systems, the building will be re-located to Arlington, VA so that the new structural system can be designed as 12 stories.

The major design differences between the existing structural system and the proposed structural system can be seen below.

- The steel structural system will increase the structural depth and therefore to maintain a minimum floor-to-ceiling height of 8'-6" the overall building height will need to be increased. Since the building height is currently 130 ft., the building height cannot be increased with the existing 12 stories. As a result, the number of stories will have to reduce to 11 to stay within the height limitation or the building will have to be re-located.
- The current column layout is non-uniform and therefore to reduce the number of skewed connections with using the new steel structural system, a uniform framing layout will need to be created by removing and re-locating columns to create a uniform layout.
- The alternative lateral systems will be subjected to different seismic loads; therefore the seismic loads will need to be re-calculated for the new system.
- To maintain a minimum floor-to-ceiling height of 8'-6"with the use of the new structural system, the floor-to-floor height will need to increase. As a result of increasing the floor-to-floor height, the wind loads for the new system will need to be recalculated.
- The steel system will be subjected to more vibration.

- The structural steel system is more flexible and therefore braced frames will be needed to resist lateral loads.

Proposed Solution

1000 Connecticut Avenue's structural system will be re-designed as a steel framing system. The lateral force resisting system will consist of moment frames around the perimeter and in the core of the building and concentric braced frames will be located around the elevator shafts and stairwell cores. The lateral force resisting beams that connect the columns in the moment frame will be designed as non-composite beams. After calculating the wind and seismic loads for the new structural system, the new lateral system will be modeled and analyzed in ETABS for both seismic and wind loads.

A composite beam/girder system with composite deck will be used for the gravity system. To use this gravity floor system, the building height will need to increase since the structural depth for each level will increase. 1000 Connecticut Avenue is currently 130 feet and the zoning height restriction in Washington DC is 130 ft. Therefore to use the composite steel beam/girder floor system the number of stories will need to be reduced from 12 to 11 to maintain high floor-to-ceiling heights and to remain within the restricted height limit or the building will have to be re-located. Therefore, the structural system will be designed as 12 stories by re-locating the building to Arlington, VA, which does not have a zoning height restriction. In addition, to decrease the number of skewed connections, columns will be re-located to create a more uniform framing layout, certain column lines will be removed to create wider bays, and the new structural system will be designed for higher floor-to-floor heights.

MAE Material Incorporation

For re-designing 1000 Connecticut Avenue's new structural system, material learned in two MAE courses were used. The lateral system was modeled, analyzed, and designed in ETABS using material learned in AE 597A (Computer Modeling). In addition, material learned in AE 534 (Steel Connections Design) was used to design the typical orthogonal and skewed shear connections and a typical moment connection. Each connection was designed and checked based on each connection's limit states. Both the lateral system and connection designs can be seen in the "Structural Depth: Steel Re-designs" section.

Breadth Studies

The integrated studies taught in the Architectural Engineering Program were incorporated in the report by conducting two breadth studies. The first breath studied was construction management Impact. This breadth will analyze the impact of the structural system redesign on the total building cost; construction schedule; site logistics of steel versus concrete; building LEED certification; and the anticipated revenue increase from the use of the new structural system. First, the current cost estimate will be compared to the cost estimate of the new structural system. Second, the new structural system construction schedule will be compared to the existing system construction schedule. Third, how the construction site will have to be managed differently for steel compared to concrete will be evaluated. Fourth, the building LEED certification with the use of the new structural system will be compared to the existing building LEED certification. Last, the revenue obtained from the new structural system with wider bays and higher floor-to-ceiling heights will be compared to the existing structural system's revenue. Wider bays and higher floor-to-ceiling heights will increase the rental value of the floor space and therefore the building owner will be able charge higher rent, which will potentially increase revenue.

The second breadth studied was acoustics and lighting impact. This breadth will involve determining the sound treatments required for a typical office space housed in the new structural system. Based on the sound treatments in the space, the sound transmission class (STC) and noise reduction (NR) values will be determined for the typical office space. In addition, since the new structural system will be designed for higher floor-to-ceiling heights, lighting illuminance applied to the work plane surfaces will be affected. As a result, a lighting breadth will be conducted by designing the lighting system for a typical office space using the existing floor-to-ceiling height of 8'-6" and checking to determine if the same lighting system can be used for the space with a new floor-to-ceiling height of 10'-6". AGI will be used to design the lighting system for the space and the average illuminance in the space will be compared to the target illuminance. The IESNA Handbook 10th edition was used to determine the target illuminance and maximum power density for a private office space. Both spaces with the lighting system layout will be represented through renderings.

Structural Depth: Steel Re-Designs

Gravity System Design

To begin the structural system re-design, the framing layout and lateral system locations were determined. The goal of the re-design was to increase the rental value of the building space by creating wider bays and higher floor-to-ceiling heights. As a result, certain column lines that were in the existing structural layout were removed to increase the bay sizes and columns were re-located to create a uniform framing layout to reduce the number of required skewed connections. After designing the framing layout, a 3VLI20 composite deck was chosen for the design. The new framing system layout can be seen in Figure22.

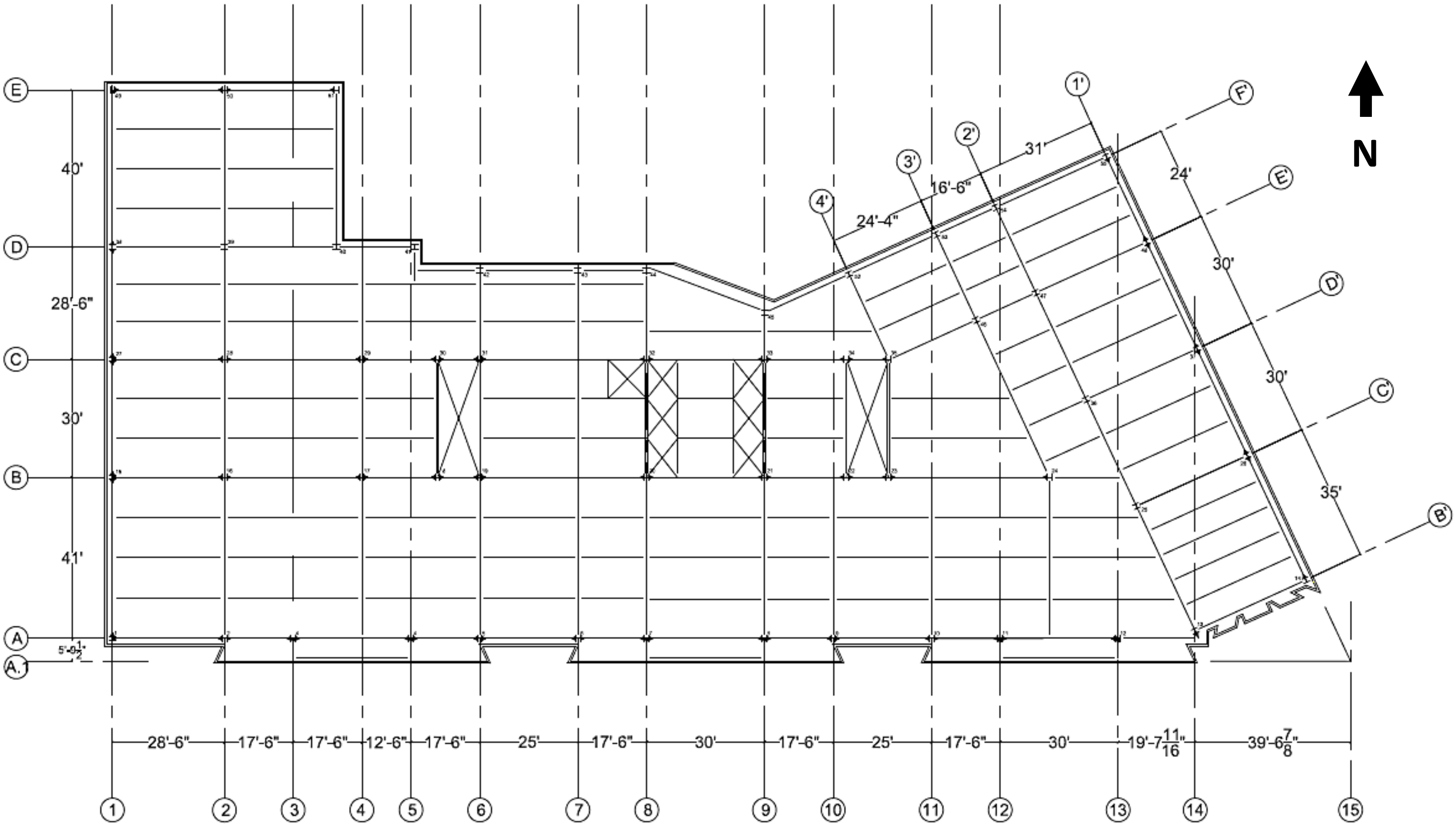


Figure 22 Typical framing plan layout

When designing the framing layout, it was found that skewed members will be needed to transition the framing layout to the portion of the building that is tilted 25 degrees counterclockwise from North axis. When initially designing the framing layout, the section located between column lines 2’ and 4’ was designed as can be seen in Figure 23. This design was then changed to the final design to avoid spanning the members at sharp, acute angles. As a result, column line 3 was added to increase the skewed angle and to decrease the span length of the beam members spanning into the girders at skewed angles. As a result of adding the additional column line, the beams were designed using smaller beam sections to support the loads. In addition, according to “Orthogonal and Skewed Shear Connections Design and Detailing Requirements” article most skewed members carry less tributary area, therefore when creating the framing layout the system was designed so that only beam members will be connected at skewed angles where necessary. The girders throughout the framing layout are all connected at orthogonal angles with the exception of two girders.

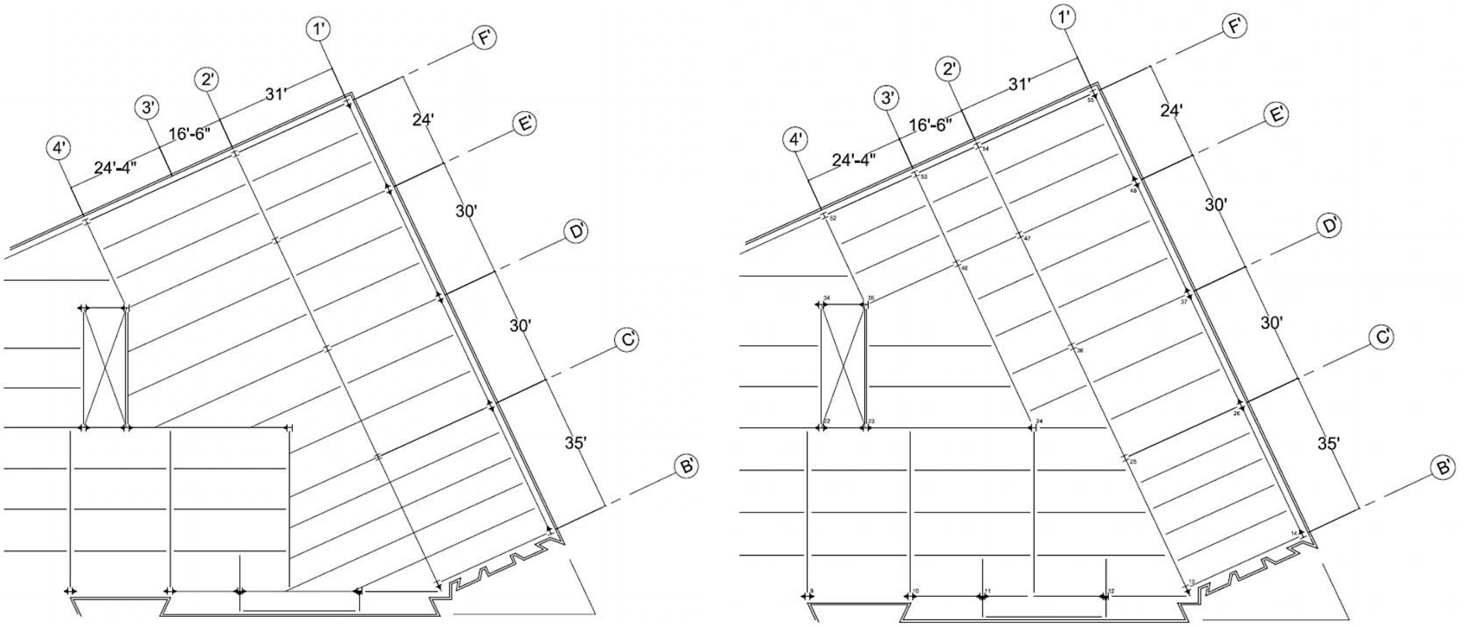


Figure 23 Original (left) and final (right) design of framing layout in tilted building region

After creating the framing layout, the moment frame and brace frame locations were determined. Five moment frames were chosen to resist the lateral loads in the East-West direction. Three of the moment frames are located around the perimeter of the building and two of the moment frames are located in the core of the building. To resist the lateral loads in the North-South direction, two moment frames located around the perimeter of the building and four braced frames located around the elevator shafts and stairwell cores were used. Moment frames were used to maintain an open floor plan without any obstructions. To avoid obstructions in the floor plan, the braced frames were located around the elevator shafts and elevator cores, where there are no openings and to keep the floor layout open. The moment frame and brace frame locations can be seen in Figure 24.

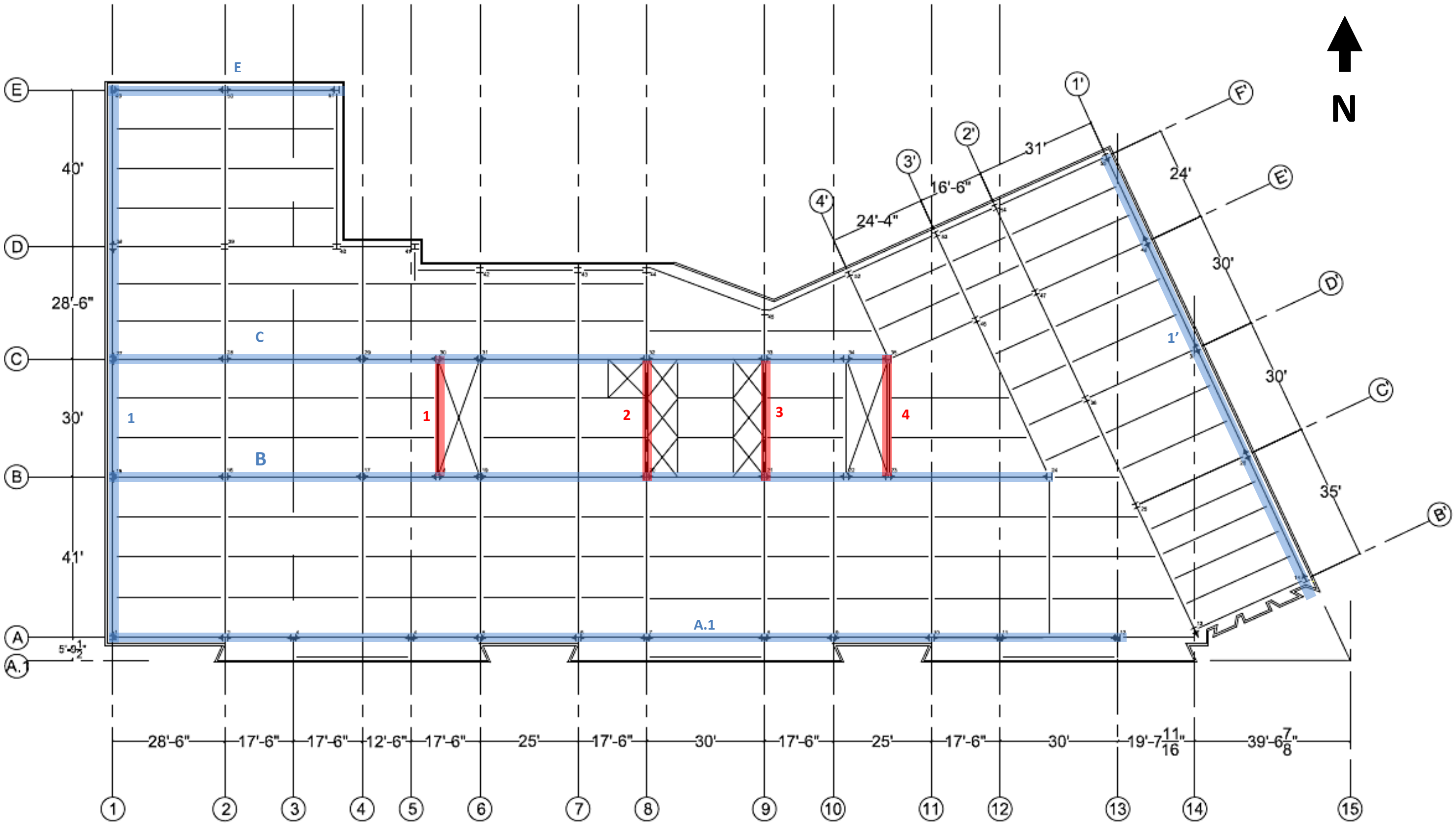


Figure 24 Floor plan with moment frames indicated in blue and braced frames indicated in red

After creating the framing layout, the composite beam and girder gravity system was designed manually using AISC 14th edition. Since the framing layout consists of varying bay sizes, the members were designed for each bay. The framing layout with member sizes can be seen in Figures 25 and 26. The calculations for the gravity system design can be found in Appendix A.

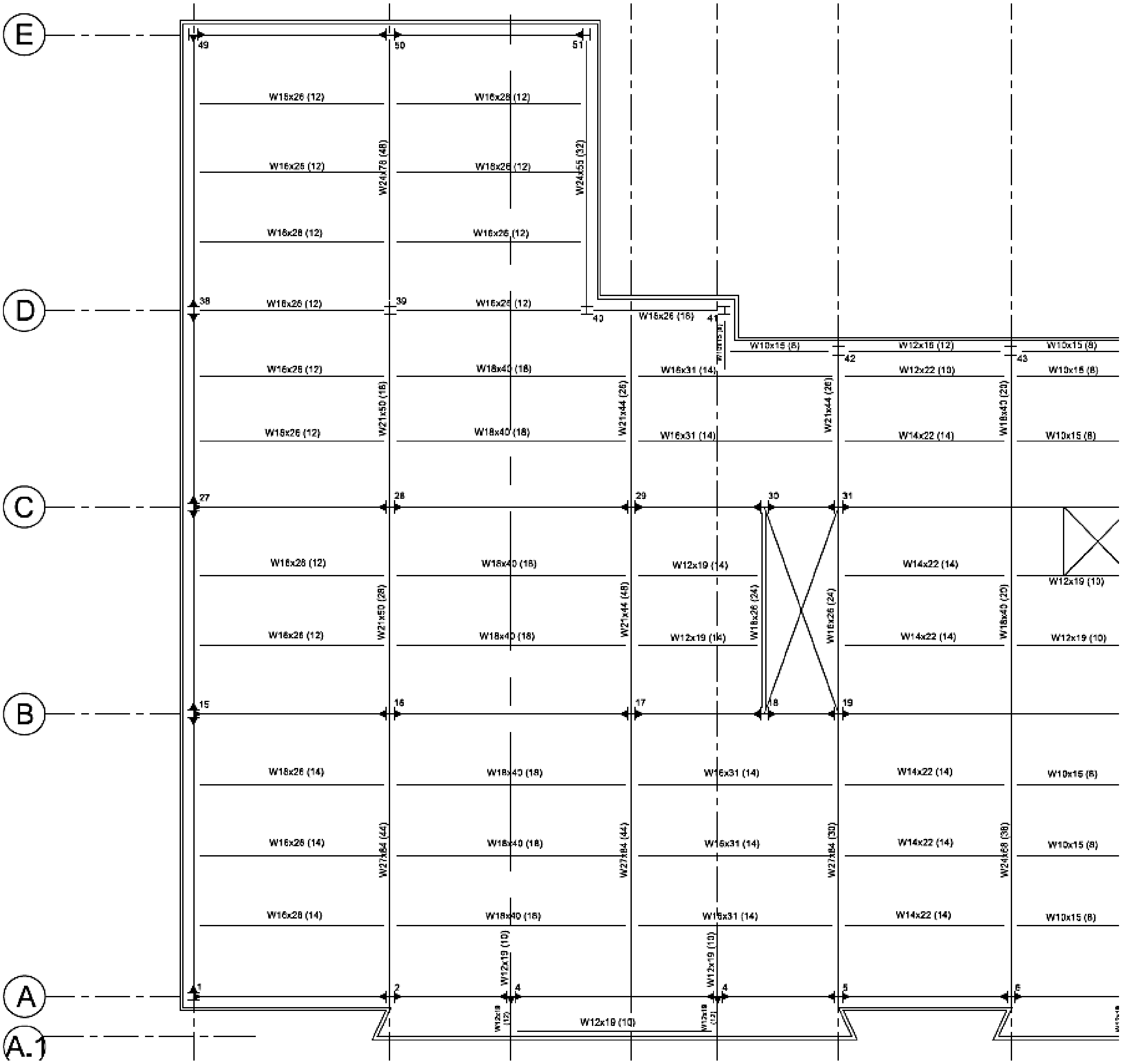


Figure 25 Typical framing plan A with frame sections

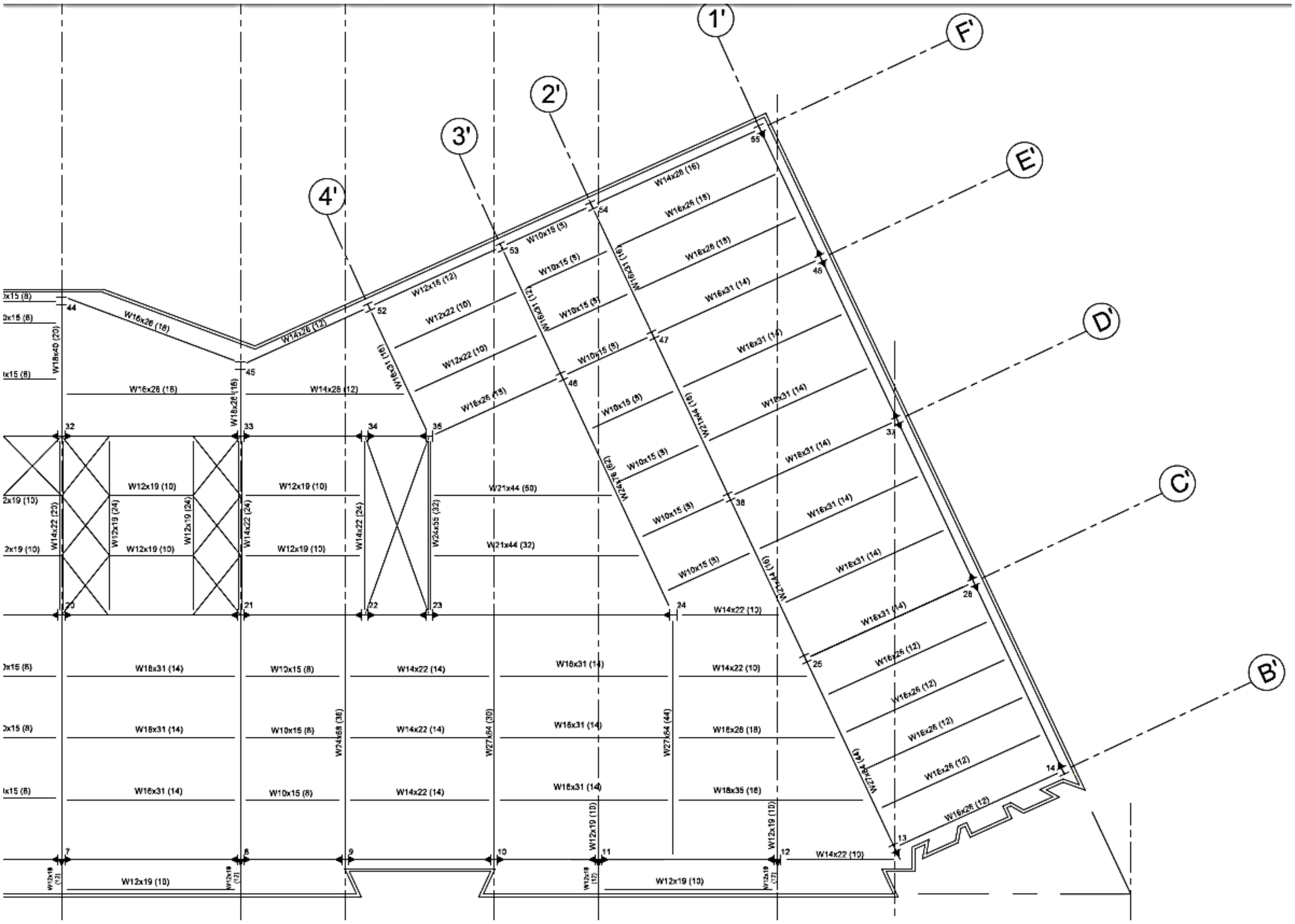
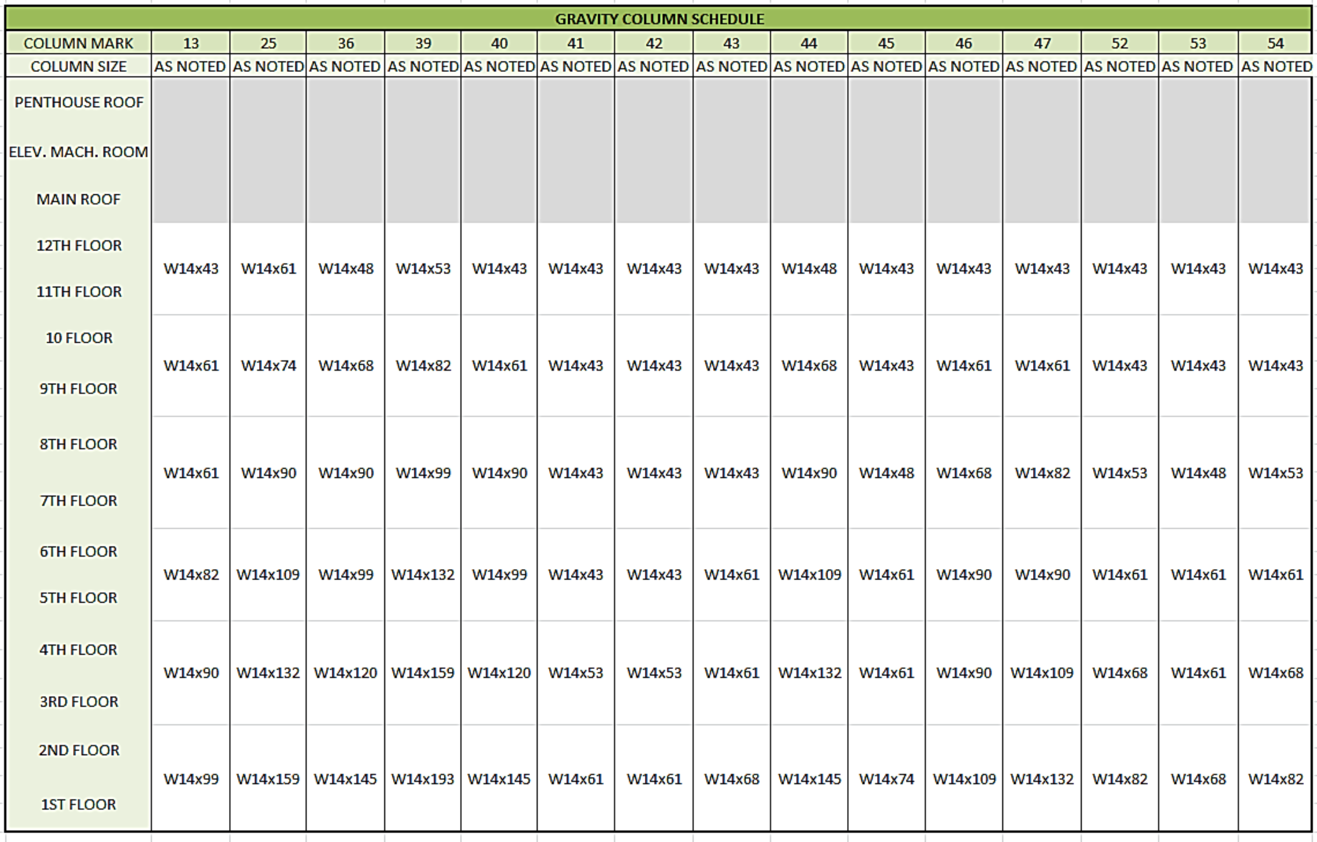


Figure 26 Typical framing plan B with frame sections

After designing the gravity system, the floor-to-floor height was chosen to be increased from the existing 10'-7" to 15'-0". The increased floor-to-floor height will increase the building height from 130ft to 180 ft. The purpose of this height increase was to maintain high floor-to-ceiling heights while taking into account the increase in structural depth due to the gravity members. The existing system has a floor-to-ceiling height of 8'-6", but after increasing the floor-to-floor height to 15'-0" in the new structural system a floor-to-ceiling height of 10'-6" will be achieved. The higher ceiling height will increase both the openness and rentable value of the space.

After increasing the floor-to-floor height, the columns were designed as two tiers. This represents the columns will be spliced every two stories. Designing the columns as 4 tiers would result in a shipment on site of 60ft long columns, which is undesirable. Therefore, the columns were designed as 2 tiers to decrease the length of the columns shipped to the construction site and to decrease cost by using smaller columns sections throughout the height of the building. The gravity columns were designed using AISC 14th edition and using the assistance of Microsoft Excel. The gravity column calculations can be seen in Appendix A. The gravity column schedule can be seen in Table 12.



GRAVITY COLUMN SCHEDULE															
COLUMN MARK	13	25	36	39	40	41	42	43	44	45	46	47	52	53	54
COLUMN SIZE	AS NOTED	AS NOTED	AS NOTED	AS NOTED	AS NOTED	AS NOTED	AS NOTED	AS NOTED	AS NOTED	AS NOTED	AS NOTED	AS NOTED	AS NOTED	AS NOTED	AS NOTED
PENTHOUSE ROOF															
ELEV. MACH. ROOM															
MAIN ROOF															
12TH FLOOR	W14x43	W14x61	W14x48	W14x53	W14x43	W14x43	W14x43	W14x43	W14x48	W14x43	W14x43	W14x43	W14x43	W14x43	W14x43
11TH FLOOR															
10 FLOOR	W14x61	W14x74	W14x68	W14x82	W14x61	W14x43	W14x43	W14x43	W14x68	W14x43	W14x61	W14x61	W14x43	W14x43	W14x43
9TH FLOOR															
8TH FLOOR	W14x61	W14x90	W14x90	W14x99	W14x90	W14x43	W14x43	W14x43	W14x90	W14x48	W14x68	W14x82	W14x53	W14x48	W14x53
7TH FLOOR															
6TH FLOOR	W14x82	W14x109	W14x99	W14x132	W14x99	W14x43	W14x43	W14x61	W14x109	W14x61	W14x90	W14x90	W14x61	W14x61	W14x61
5TH FLOOR															
4TH FLOOR	W14x90	W14x132	W14x120	W14x159	W14x120	W14x53	W14x53	W14x61	W14x132	W14x61	W14x90	W14x109	W14x68	W14x61	W14x68
3RD FLOOR															
2ND FLOOR	W14x99	W14x159	W14x145	W14x193	W14x145	W14x61	W14x61	W14x68	W14x145	W14x74	W14x109	W14x132	W14x82	W14x68	W14x82
1ST FLOOR															

Table 12 Gravity column schedule

After designing the gravity floor system and gravity columns, a typical orthogonal connection and a typical skewed shear connection was designed. A double angel was used for the orthogonal shear connection. According to the “Orthogonal and Skewed Shear Connections Design and Detailing Requirements” article, the preferred skewed connections for economy and safety are single plates and end plates. As a result, an end plate skewed shear connection was designed in accordance to AISC 14th edition and the “Orthogonal and Skewed Shear Connections Design and Detailing Requirements” article. The typical shear connections can be seen in Figure 27. The design of the typical shear connections can be seen in Appendix D.

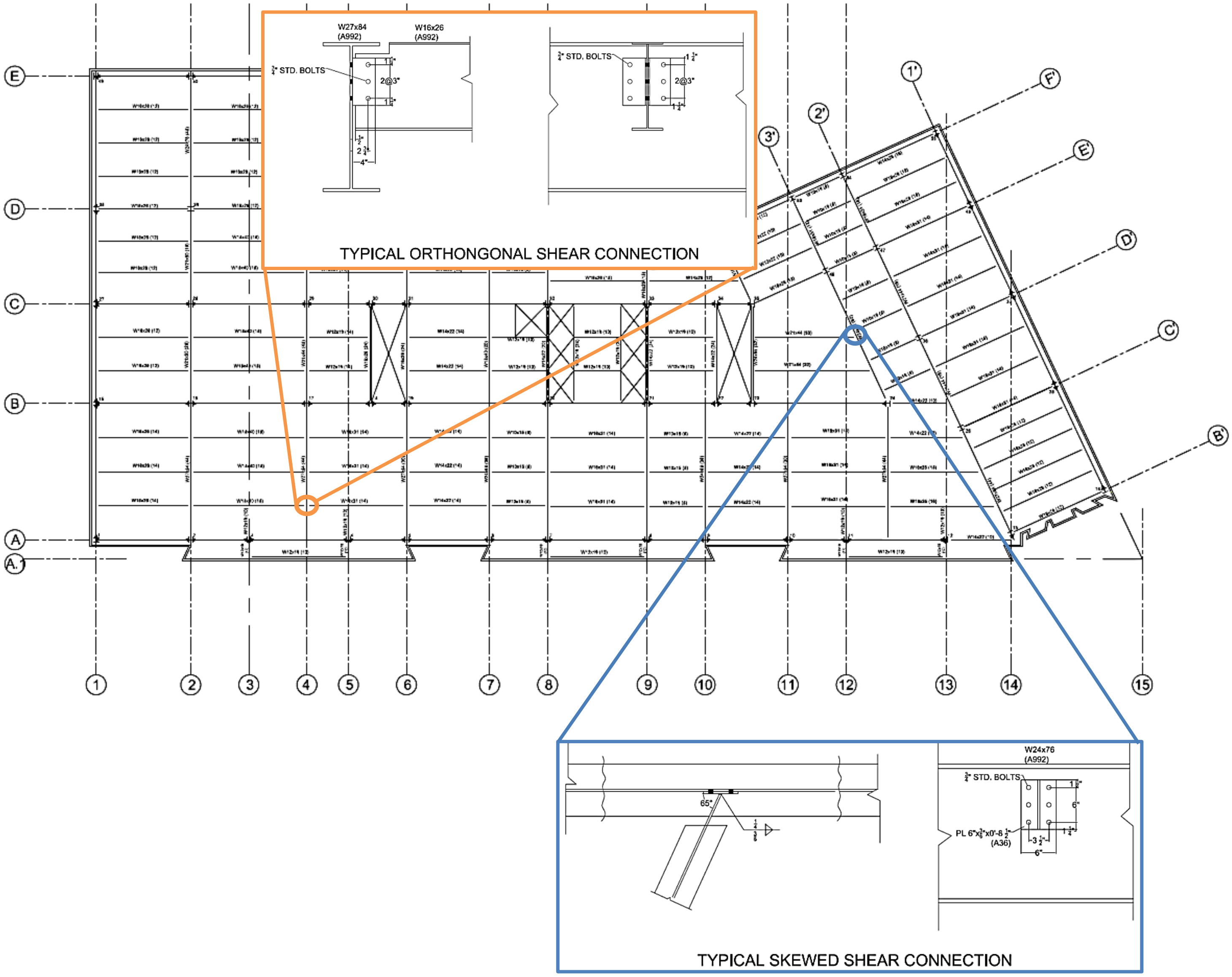


Figure 27 Typical shear connections

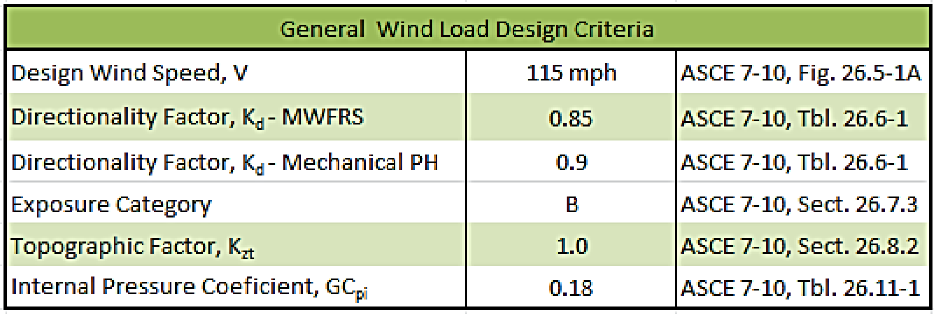
Lateral System Design

The lateral- force resisting beams that connect the columns were designed as non-composite. To begin the design of the lateral system, the member sizes were estimated by designing the beams, girders, and columns for gravity loads only and using AISC 14th edition. The estimated moment frame member sizes can be seen in Appendix A. After estimating the member sizes, the wind loads and seismic loads were calculated for the new structural system. The wind and seismic load calculations can be found in Appendices B and C.

Wind loads were determined using the Main Wind Force Resisting System (MWFRS) procedure (method 2) in conformance to Chapters 26 and 27 outlined in ASCE 7-10. Due to the building's complex geometry, a rectangular building shape was assumed to simplify the wind load analysis, as can be seen in Figure 17.

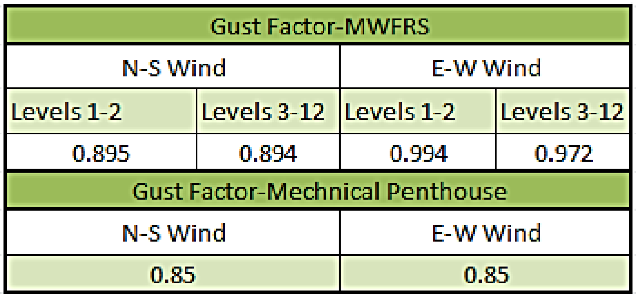
Most of the calculations for determining the wind pressures and story forces were performed in Microsoft Excel. In the analysis, windward, leeward, sidewall, and roof suction pressures were determined. Internal pressures were neglected in calculating the design wind pressure because internal pressures do not contribute towards the external wind pressures acting on the building.

The general wind load design criteria and guest effect factors can be found in Tables 13 and 14. The calculated approximate lower- bound natural frequency for the building was 0.417 Hz, which is less than 1 Hz, therefore the gust factors were calculated in the event the building is flexible.



General Wind Load Design Criteria		
Design Wind Speed, V	115 mph	ASCE 7-10, Fig. 26.5-1A
Directionality Factor, K_d - MWFRS	0.85	ASCE 7-10, Tbl. 26.6-1
Directionality Factor, K_d - Mechanical PH	0.9	ASCE 7-10, Tbl. 26.6-1
Exposure Category	B	ASCE 7-10, Sect. 26.7.3
Topographic Factor, K_{zt}	1.0	ASCE 7-10, Sect. 26.8.2
Internal Pressure Coeficient, GC_{pi}	0.18	ASCE 7-10, Tbl. 26.11-1

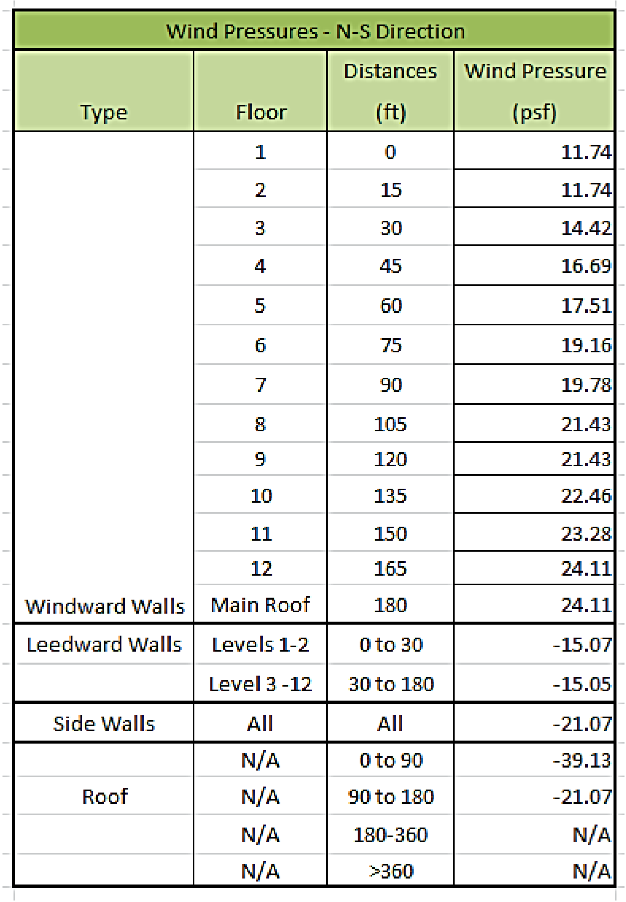
Table 13 General wind design criteria



Gust Factor-MWFRS			
N-S Wind		E-W Wind	
Levels 1-2	Levels 3-12	Levels 1-2	Levels 3-12
0.895	0.894	0.994	0.972
Gust Factor-Mechnical Penthouse			
N-S Wind		E-W Wind	
0.85		0.85	

Table 14 Gust factors for the Main Wind Force Resisting System

Further, wind pressures in the N-S and E-W directions can be seen in Tables 15 and 16 with the corresponding vertical profile sketch of the wind pressures shown in Figures 28 and 29. The story forces were then determined based on the wind pressures. The resulting base shears were 2119 kips in the N-S direction with an overturning moment of 218,031 kip-ft and 850 kips in the E-W direction with an overturning moment of 88,086 kip-ft. The story forces and overturning moments for both the N-S and E-W directions can be found in Tables 17 and 18 along with the vertical profile of the story forces shown Figures 30 and 31.



Wind Pressures - N-S Direction			
Type	Floor	Distances (ft)	Wind Pressure (psf)
Windward Walls	1	0	11.74
	2	15	11.74
	3	30	14.42
	4	45	16.69
	5	60	17.51
	6	75	19.16
	7	90	19.78
	8	105	21.43
	9	120	21.43
	10	135	22.46
	11	150	23.28
	12	165	24.11
	Main Roof	180	24.11
Leedward Walls	Levels 1-2	0 to 30	-15.07
	Level 3 -12	30 to 180	-15.05
Side Walls	All	All	-21.07
Roof	N/A	0 to 90	-39.13
	N/A	90 to 180	-21.07
	N/A	180-360	N/A
	N/A	>360	N/A

Table 15 Wind pressures in North-South direction

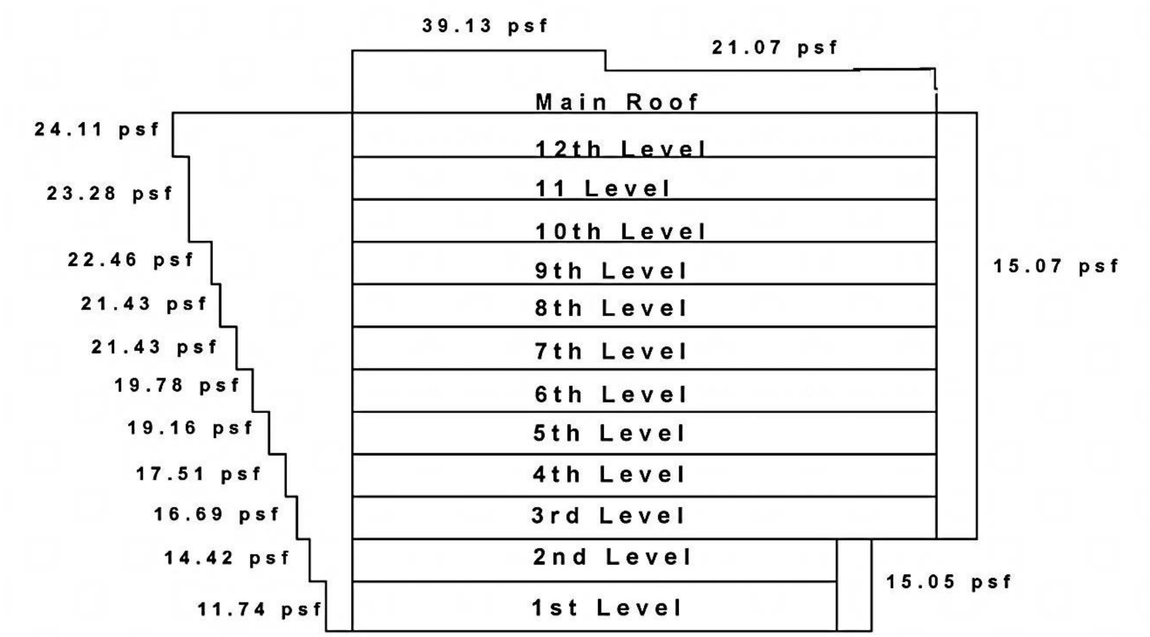


Figure 28 Vertical profile of wind pressure distribution in North-South direction

Wind Pressures - E-W Direction			
Type	Floor	Distances (ft)	Wind Pressure (psf)
Windward Walls	1	0	13.04
	2	15	13.04
	3	30	15.66
	4	45	18.13
	5	60	19.02
	6	75	20.81
	7	90	21.48
	8	105	23.27
	9	120	23.27
	10	135	24.39
	11	150	25.29
	12	165	26.18
	Main Roof	180	26.18
Leedward Walls	Levels 1-2	0 to 30	-9.07
	Level 3 -12	30 to 180	-9.59
Side Walls	Levels 1-2	0 to 30	-23.43
	Levels 3-12	30 to 180	-22.91
Roof	N/A	0 to 90	-31.29
	N/A	90 to 180	-28.54
	N/A	180-360	-17.28
	N/A	>360	N/A

Table 16 Wind pressures in East-West direction

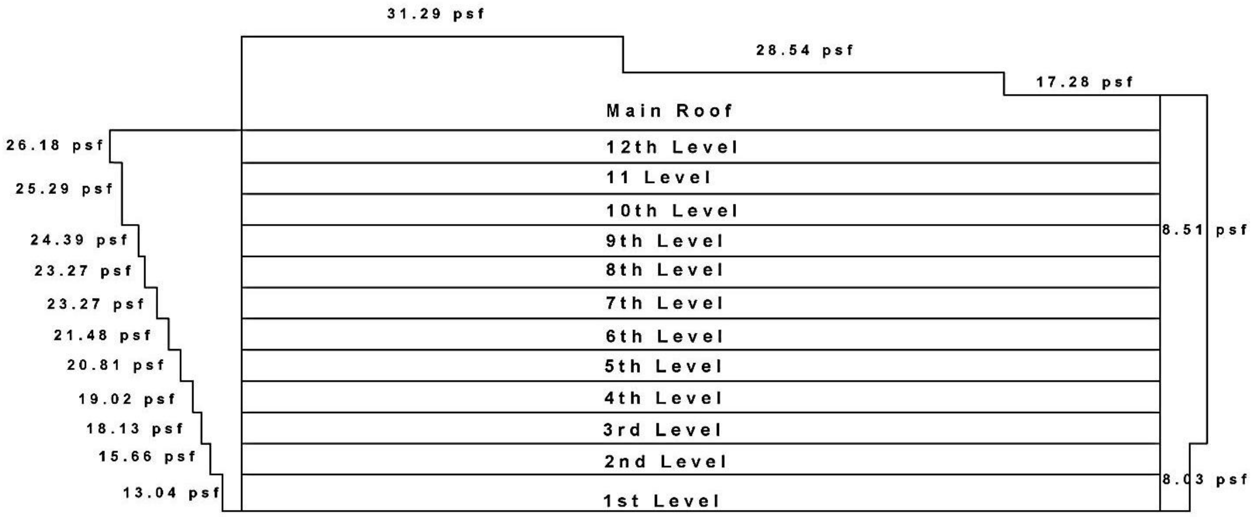


Figure 29 Vertical profile wind pressure distribution in East-West direction

Wind Forces - N-S Direction										
		Tributary Below			Tributary Above			Story Force	Story Shear	Overturning
Floor	Elevation (ft)	Height (ft)	Length (ft)	Area (ft^2)	Height (ft)	Length (ft)	Area (ft^2)	(Kips)	(Kips)	Moment (K-ft)
PH Roof	198.5	18.5	199.83	3696.86	0	199.83	0	152.81	152.81	30333.53
Main Roof	180	7.50	314.58	2359.35	0	314.58	0	92.39	245.20	16629.72
12	165	7.50	314.58	2359.35	7.50	314.58	2359.35	184.77	429.98	30487.83
11	150	7.50	314.58	2359.35	7.50	314.58	2359.35	182.83	612.81	27424.52
10	135	7.50	314.58	2359.35	7.50	314.58	2359.35	179.02	791.83	24167.77
9	120	7.50	314.58	2359.35	7.50	314.58	2359.35	174.57	966.39	20947.90
8	105	7.50	314.58	2359.35	7.50	314.58	2359.35	172.14	1138.53	18074.19
7	90	7.50	314.58	2359.35	7.50	314.58	2359.35	168.25	1306.77	15142.14
6	75	7.50	314.58	2359.35	7.50	314.58	2359.35	162.90	1469.67	12217.39
5	60	7.50	314.58	2359.35	7.50	314.58	2359.35	157.55	1627.22	9453.06
4	45	7.50	314.58	2359.35	7.50	314.58	2359.35	151.72	1778.94	6827.28
3	30	7.50	314.58	2359.35	7.50	314.58	2359.35	144.43	1923.37	4332.76
2	15	7.50	314.58	2359.35	7.50	314.58	2359.35	132.84	2056.20	1992.56
1	0	0	314.58	0.00	7.50	314.58	2359.35	63.26	2119.46	0.00
									Total Base Shear =	2119 K
									Total Overturning Moment =	218,031 K-ft

Table 17 N-S Story forces, base shear, and overturning moment

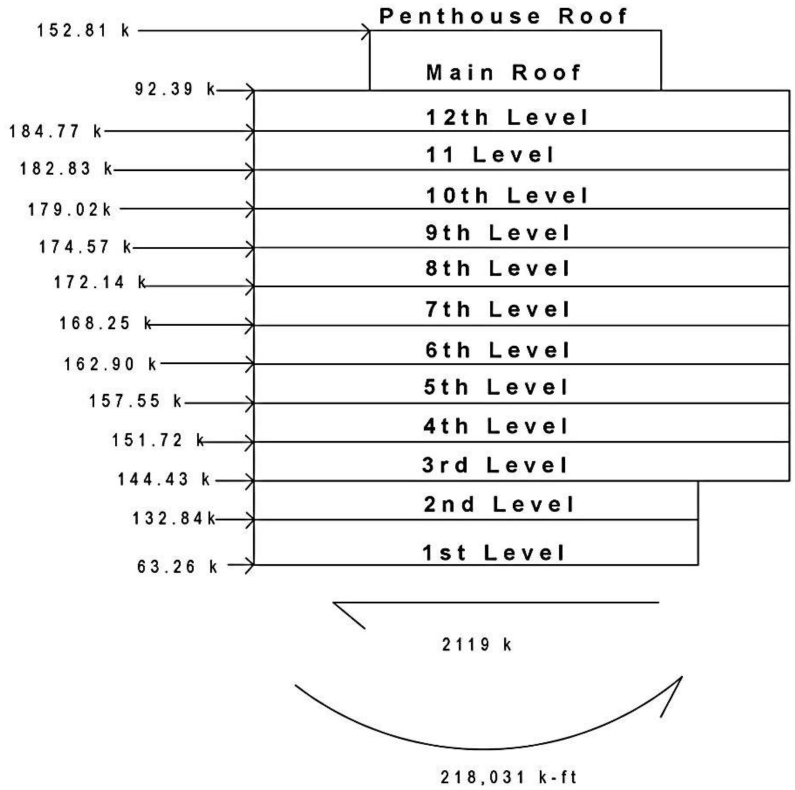


Figure 30 Vertical profile of story forces in N-S direction

Wind Forces - E-W Direction

		Tributary Below			Tributary Above			Story Force	Story Shear	Overturning
Floor	Elevation (ft)	Height (ft)	Length (ft)	Area (ft^2)	Height (ft)	Length (ft)	Area (ft^2)	(Kips)	(Kips)	Moment (K-ft)
PH Roof	198.5	18.5	59.83	1106.86	0	59.83	0	45.75	45.75	9081.99
Main Roof	180	7.50	147	1102.50	0	147	0	39.48	85.23	7106.20
12	165	7.50	147	1102.50	7.50	147	1102.50	78.87	164.11	13014.26
11	150	7.50	147	1102.50	7.50	147	1102.50	77.89	241.99	11683.12
10	135	7.50	147	1102.50	7.50	147	1102.50	75.91	317.91	10248.36
9	120	7.50	147	1102.50	7.50	147	1102.50	73.69	391.60	8843.20
8	105	7.50	147	1102.50	7.50	147	1102.50	72.46	464.06	7608.28
7	90	7.50	147	1102.50	7.50	147	1102.50	70.49	534.55	6343.75
6	75	7.50	147	1102.50	7.50	147	1102.50	67.77	602.32	5082.92
5	60	7.50	147	1102.50	7.50	147	1102.50	65.06	667.38	3903.51
4	45	7.50	147	1102.50	7.50	147	1102.50	62.10	729.48	2794.41
3	30	7.50	147	1102.50	7.50	147	1102.50	57.82	787.30	1734.74
2	15	7.50	121.75	913.13	7.50	121.75	913.13	42.78	830.08	641.67
1	0	0	121.75	0.00	7.50	121.75	913.13	20.19	850.27	0.00
									Total Base Shear =	850 K
									Total Overturning Moment =	88,086 K-ft

Table 18 E-W Story forces, base shear, and overturning moment

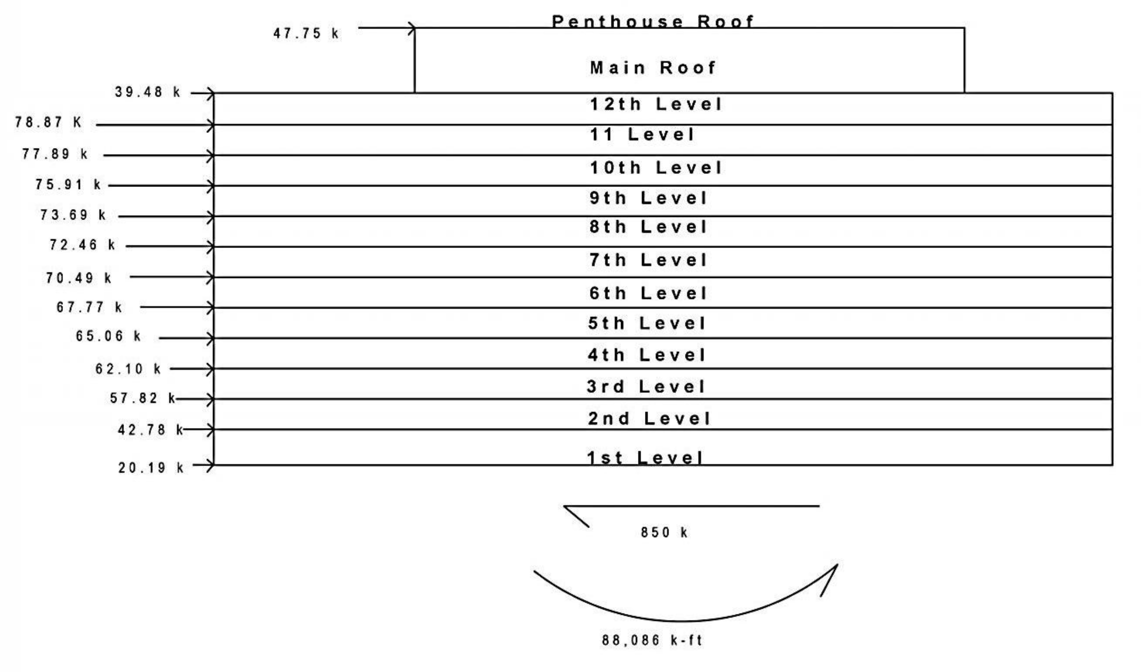
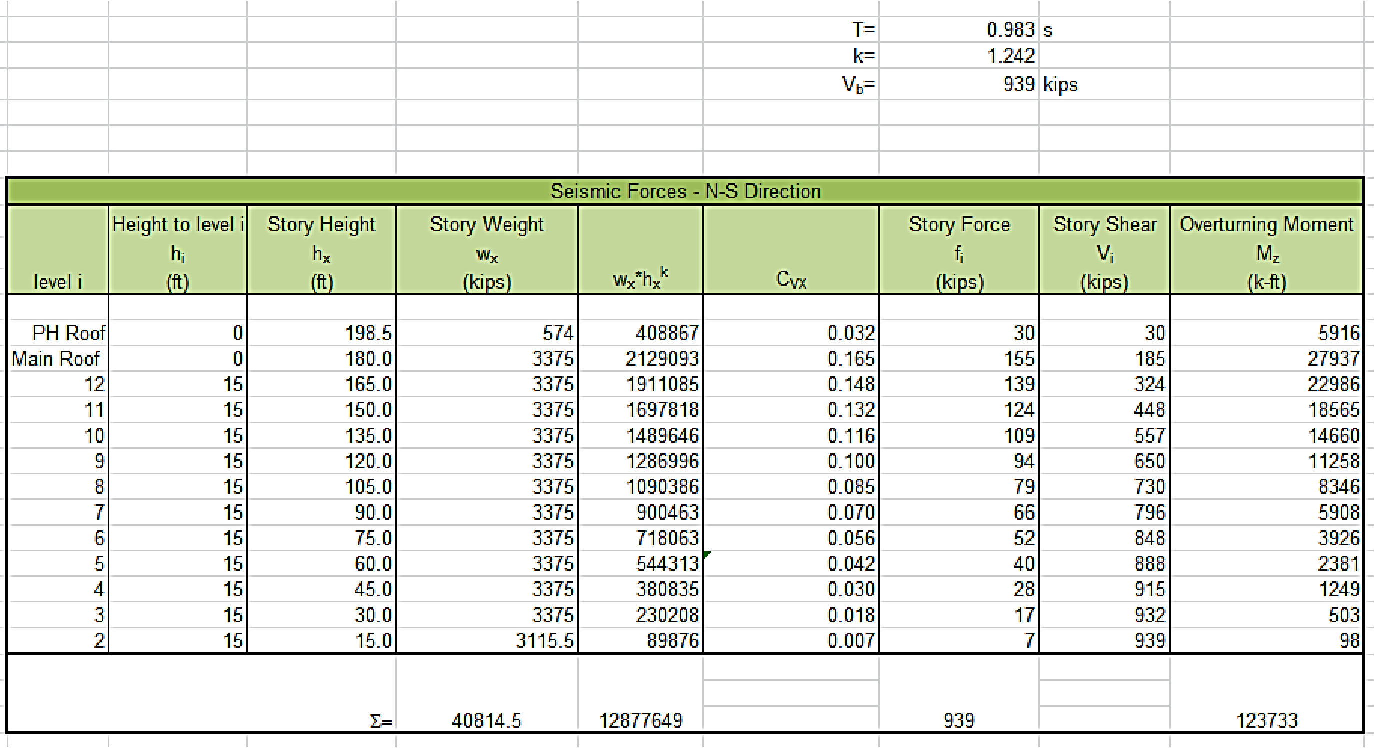


Figure 31 Vertical profile of story forces in E-W direction

Seismic loads were determined using the Equivalent Lateral Force Procedure outlined in Chapters 11 and 12 in ASCE 7-10. For analysis, the 1st level weight was neglected and only the 2nd-12th levels, main roof, and penthouse were considered for building weight calculations. For determining the seismic loads, the member self-weights (including the beams, girders and columns) were assumed to be 15 psf. Since the lateral system consists of a dual system with the combined use of moment frames and braced frames, seismic loads were calculated separately for the North-South and East-West directions. The seismic story forces and overturning moments for the N-S and E-W directions can be seen in Tables 19 and 20 and the story force distributions can be seen in Figures 32 for the N-S direction and Figure 33 for the E-W direction.

Since this building has several stories above grade, building weight was determined by calculating the dead weight for the typical floor level and applying that story weight to the other floor levels (levels 2-12). The weight on the main roof and penthouse roof were calculated separately. The weight included for summing the total building weight were the weight of the slab on deck, member self-weight allowance, super-imposed dead loads, and curtain wall self-weight.

After the analysis, the determined base shear in the North-South direction was 939 kips with an overturning moment of 123,733 K-ft. The baser shear in East-West direction was 518 kips with an overturning moment of 71,659 k-ft.



T= 0.983 s

k= 1.242

V_b= 939 kips

Seismic Forces - N-S Direction								
level i	Height to level i h_i (ft)	Story Height h_x (ft)	Story Weight w_x (kips)	$w_x{}^*h_x{}^k$	Cvx	Story Force f_i (kips)	Story Shear V_i (kips)	Overturning Moment M_z (k-ft)
PH Roof	0	198.5	574	408867	0.032	30	30	5916
Main Roof	0	180.0	3375	2129093	0.165	155	185	27937
12	15	165.0	3375	1911085	0.148	139	324	22986
11	15	150.0	3375	1697818	0.132	124	448	18565
10	15	135.0	3375	1489646	0.116	109	557	14660
9	15	120.0	3375	1286996	0.100	94	650	11258
8	15	105.0	3375	1090386	0.085	79	730	8346
7	15	90.0	3375	900463	0.070	66	796	5908
6	15	75.0	3375	718063	0.056	52	848	3926
5	15	60.0	3375	544313	0.042	40	888	2381
4	15	45.0	3375	380835	0.030	28	915	1249
3	15	30.0	3375	230208	0.018	17	932	503
2	15	15.0	3115.5	89876	0.007	7	939	98
		Σ=	40814.5	12877649		939		123733

Table 19 N-S Story forces, base shear, and overturning moment

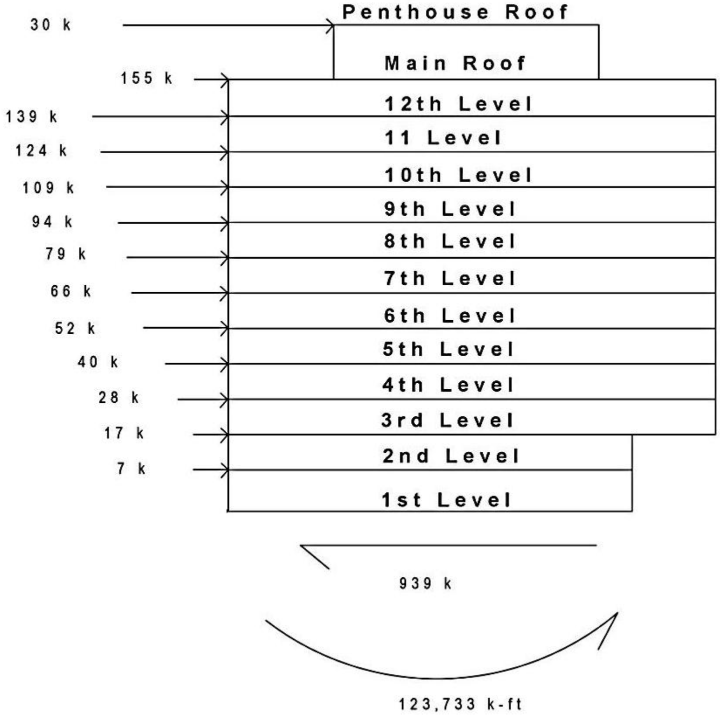
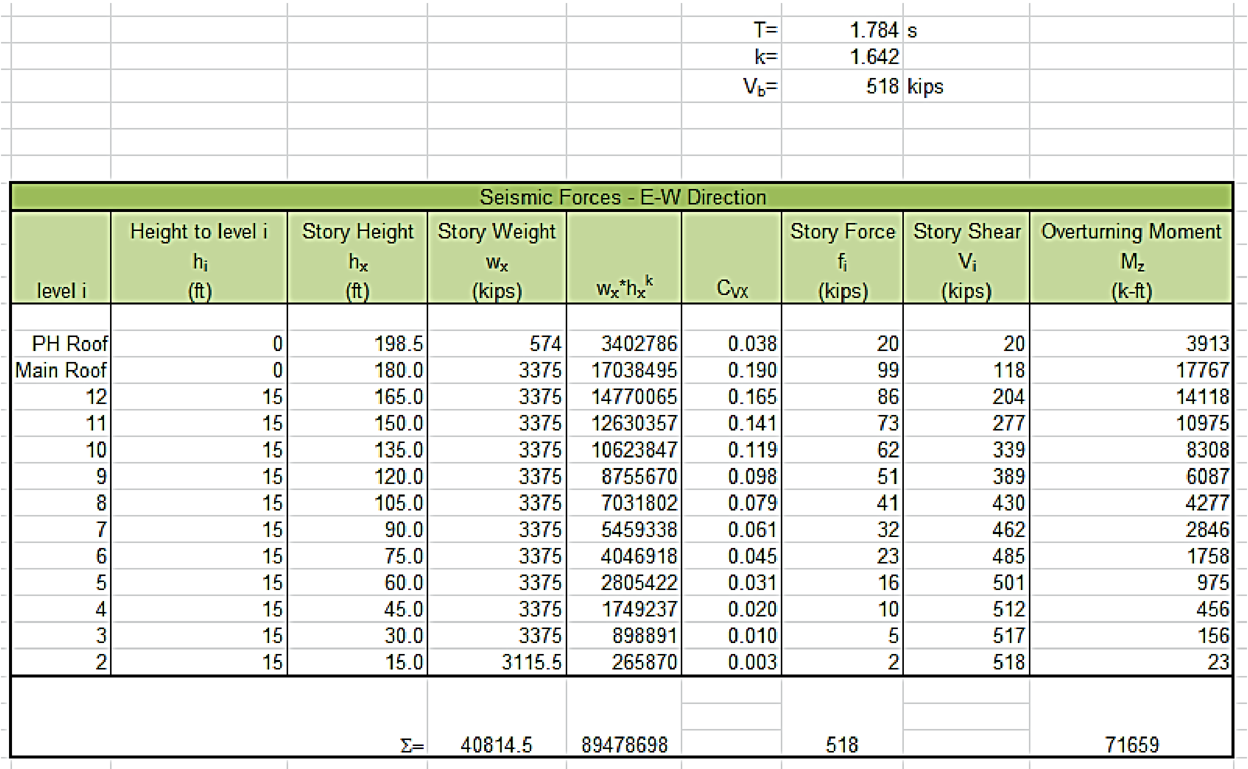


Figure 32 Vertical profile of story forces in N-S direction



T= 1.784 s

k= 1.642

V_b= 518 kips

Seismic Forces - E-W Direction								
level i	Height to level i h_i (ft)	Story Height h_x (ft)	Story Weight w_x (kips)	$w_x{*}h_x^k$	C_{vx}	Story Force f_i (kips)	Story Shear V_i (kips)	Overturning Moment M_z (k-ft)
PH Roof	0	198.5	574	3402786	0.038	20	20	3913
Main Roof	0	180.0	3375	17038495	0.190	99	118	17767
12	15	165.0	3375	14770065	0.165	86	204	14118
11	15	150.0	3375	12630357	0.141	73	277	10975
10	15	135.0	3375	10623847	0.119	62	339	8308
9	15	120.0	3375	8755670	0.098	51	389	6087
8	15	105.0	3375	7031802	0.079	41	430	4277
7	15	90.0	3375	5459338	0.061	32	462	2846
6	15	75.0	3375	4046918	0.045	23	485	1758
5	15	60.0	3375	2805422	0.031	16	501	975
4	15	45.0	3375	1749237	0.020	10	512	456
3	15	30.0	3375	898891	0.010	5	517	156
2	15	15.0	3115.5	265870	0.003	2	518	23
		Σ=	40814.5	89478698		518		71659

Table 20 E-W Story forces, base shear, and overturning moment

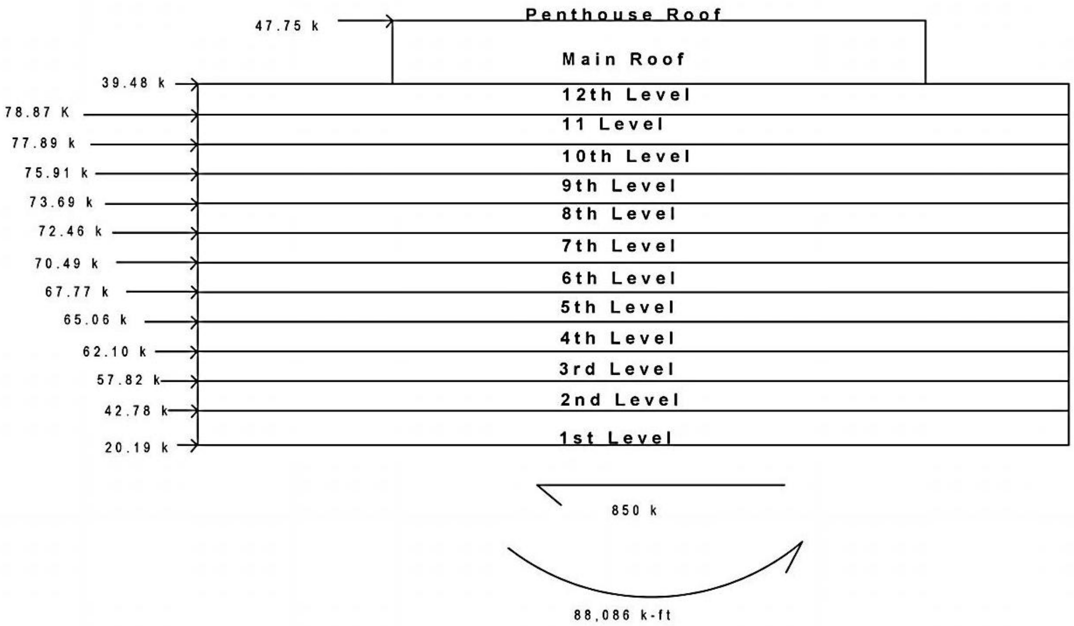


Figure 33 Vertical profile of story forces in E-W direction

Computer Model

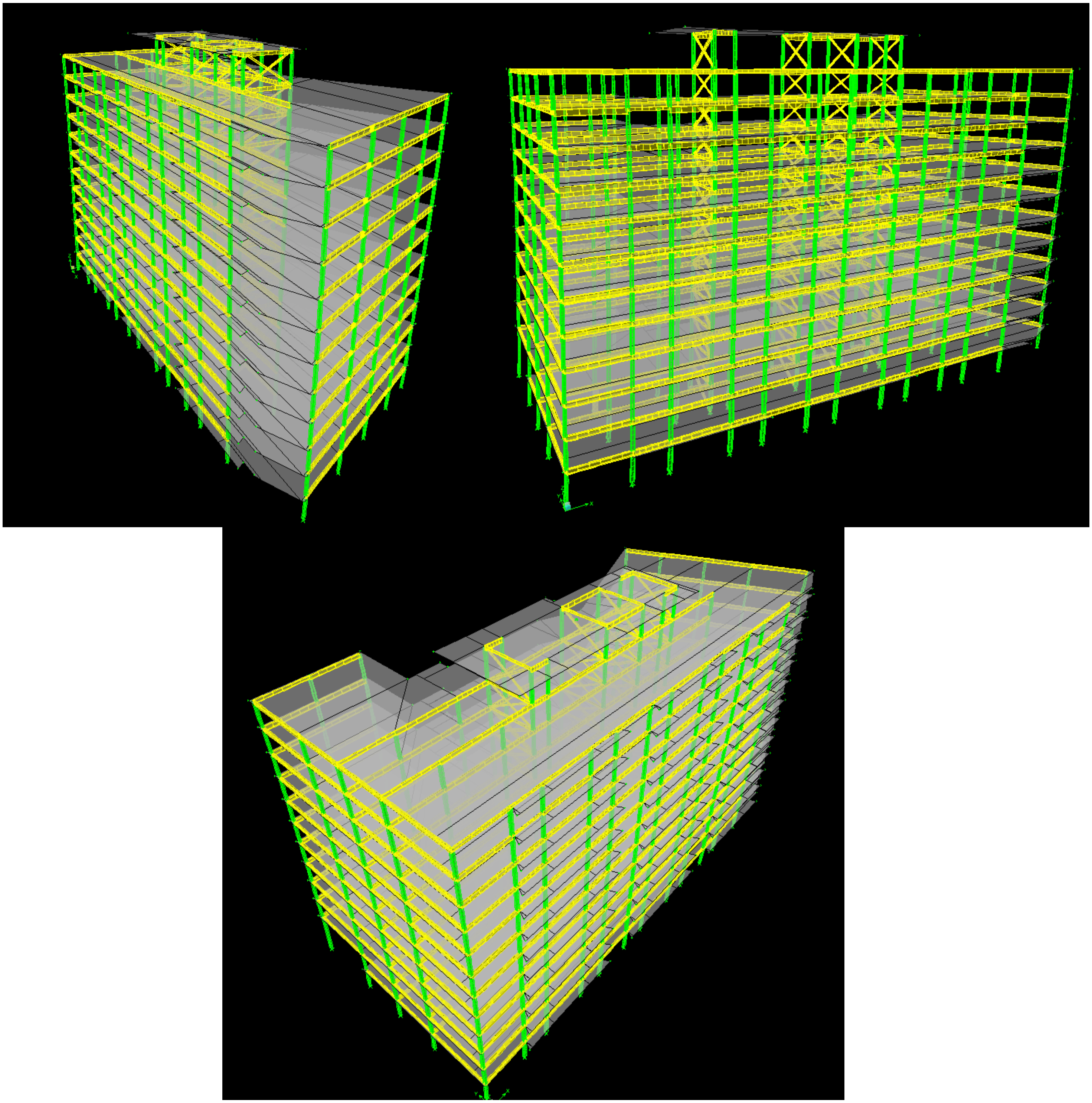
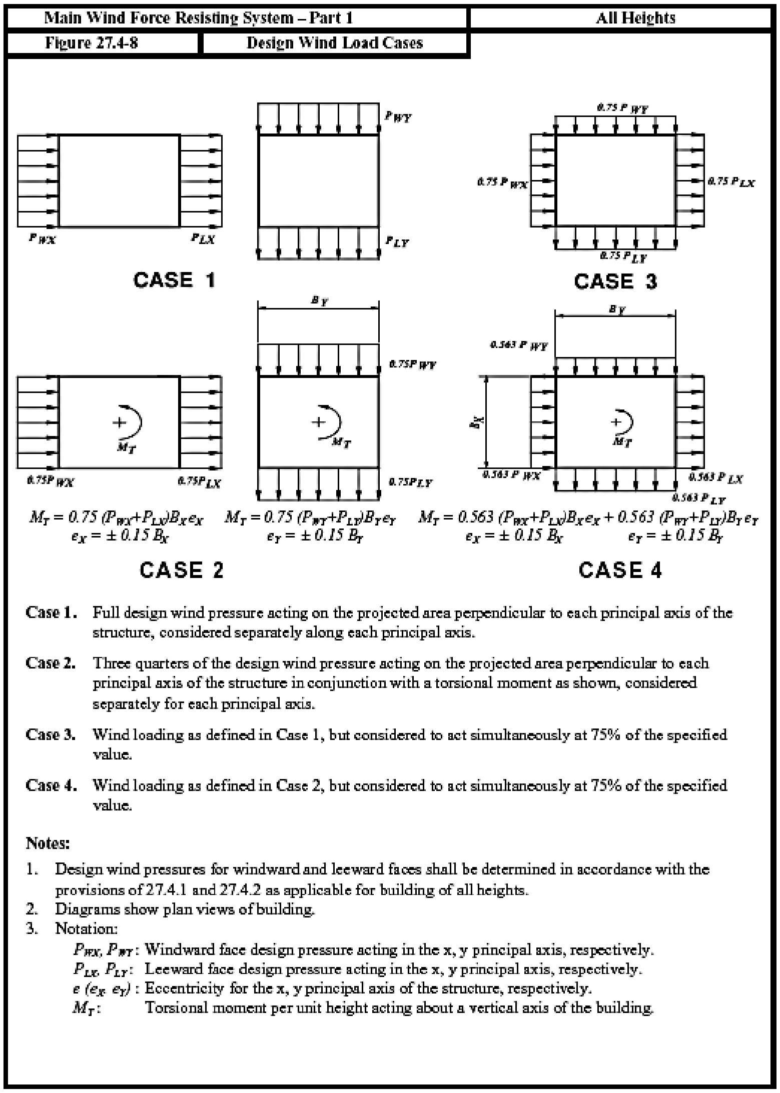


Figure 34 3D perspectives of the new lateral system modeled in ETABS

After estimating the moment frame members and determining the lateral loads, the new structural system was modeled in ETABS. Several assumptions were made when creating the lateral model. The columns were modeled as line elements and were then assigned section properties based on the gravity analysis performed to estimate the member sizes. The base supports were modeled as pin supports since the foundation consists of spread footings, which are not very rigid and thus do not carry much moment. Each floor level was modeled as an area element and assigned a rigid diaphragm since the floor system consists of a 3VLI20 composite deck with 7 ½" slab thickness. In addition, material properties were modified by eliminating the self-mass from the material definitions and applying the floor mass calculated in the seismic analysis to the diaphragm by using the Additional Area Mass function.

The ETABS model was then used to determine the controlling wind load case. The four possible wind load cases from ASCE 7-10, as can be seen in Figure 35, were considered to determine which wind case controlled the design.



Main Wind Force Resisting System – Part 1 | **All Heights**

Figure 27.4-8 | **Design Wind Load Cases**

$M_T = 0.75\ (P_{WX}+P_{LX})B_X e_X$
$e_X = \pm\ 0.15\ B_X$

$M_T = 0.75\ (P_{WY}+P_{LY})B_Y e_Y$
$e_Y = \pm\ 0.15\ B_Y$

$M_T = 0.563\ (P_{WX}+P_{LX})B_X e_X + 0.563\ (P_{WY}+P_{LY})B_Y e_Y$
$e_X = \pm\ 0.15\ B_X \qquad e_Y = \pm\ 0.15\ B_Y$

Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

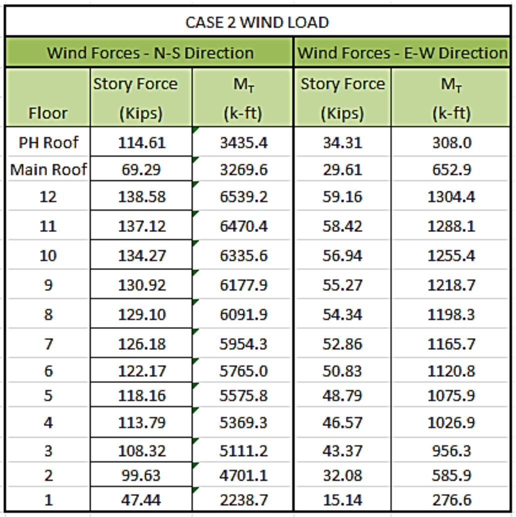
Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

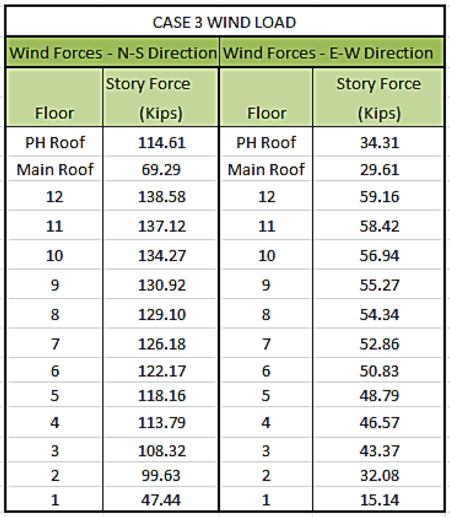
1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 27.4.1 and 27.4.2 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:
   P_{WX}, P_{WY} : Windward face design pressure acting in the x, y principal axis, respectively.
   P_{LX}, P_{LY} : Leeward face design pressure acting in the x, y principal axis, respectively.
   e (e_X, e_Y) : Eccentricity for the x, y principal axis of the structure, respectively.
   M_T : Torsional moment per unit height acting about a vertical axis of the building.

Figure 35 Design wind load cases from ASCE 7-10

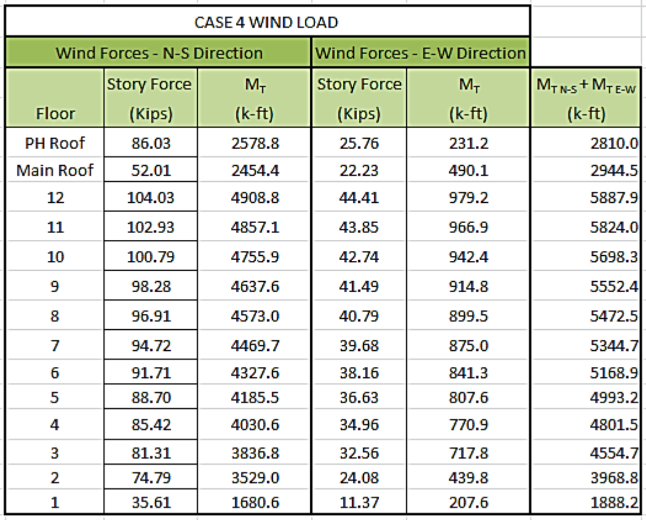
The wind loads were calculated for wind load cases 2 through 4, as can be seen in Tables 21 through 23.



CASE 2 WIND LOAD				
Wind Forces - N-S Direction			Wind Forces - E-W Direction	
Floor	Story Force (Kips)	M_T (k-ft)	Story Force (Kips)	M_T (k-ft)
PH Roof	114.61	3435.4	34.31	308.0
Main Roof	69.29	3269.6	29.61	652.9
12	138.58	6539.2	59.16	1304.4
11	137.12	6470.4	58.42	1288.1
10	134.27	6335.6	56.94	1255.4
9	130.92	6177.9	55.27	1218.7
8	129.10	6091.9	54.34	1198.3
7	126.18	5954.3	52.86	1165.7
6	122.17	5765.0	50.83	1120.8
5	118.16	5575.8	48.79	1075.9
4	113.79	5369.3	46.57	1026.9
3	108.32	5111.2	43.37	956.3
2	99.63	4701.1	32.08	585.9
1	47.44	2238.7	15.14	276.6



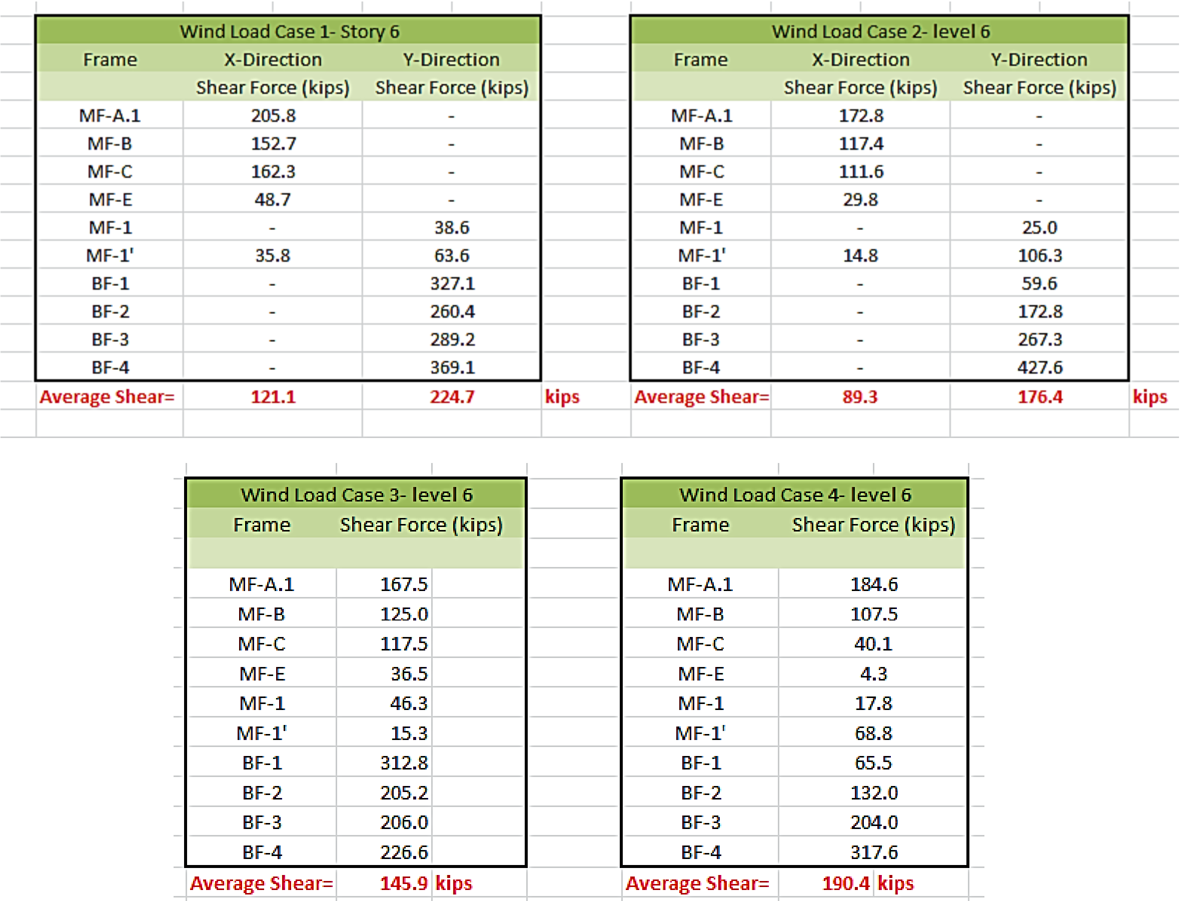
CASE 3 WIND LOAD			
Wind Forces - N-S Direction		Wind Forces - E-W Direction	
Floor	Story Force (Kips)	Floor	Story Force (Kips)
PH Roof	114.61	PH Roof	34.31
Main Roof	69.29	Main Roof	29.61
12	138.58	12	59.16
11	137.12	11	58.42
10	134.27	10	56.94
9	130.92	9	55.27
8	129.10	8	54.34
7	126.18	7	52.86
6	122.17	6	50.83
5	118.16	5	48.79
4	113.79	4	46.57
3	108.32	3	43.37
2	99.63	2	32.08
1	47.44	1	15.14



CASE 4 WIND LOAD					
Wind Forces - N-S Direction			Wind Forces - E-W Direction		
Floor	Story Force (Kips)	M_T (k-ft)	Story Force (Kips)	M_T (k-ft)	$M_{T\,N\text{-}S} + M_{T\,E\text{-}W}$ (k-ft)
PH Roof	86.03	2578.8	25.76	231.2	2810.0
Main Roof	52.01	2454.4	22.23	490.1	2944.5
12	104.03	4908.8	44.41	979.2	5887.9
11	102.93	4857.1	43.85	966.9	5824.0
10	100.79	4755.9	42.74	942.4	5698.3
9	98.28	4637.6	41.49	914.8	5552.4
8	96.91	4573.0	40.79	899.5	5472.5
7	94.72	4469.7	39.68	875.0	5344.7
6	91.71	4327.6	38.16	841.3	5168.9
5	88.70	4185.5	36.63	807.6	4993.2
4	85.42	4030.6	34.96	770.9	4801.5
3	81.31	3836.8	32.56	717.8	4554.7
2	74.79	3529.0	24.08	439.8	3968.8
1	35.61	1680.6	11.37	207.6	1888.2

Tables 21-23 Calculated wind load cases 2-4 from ASCE 7-10

It was found that wind load case 1 controlled in both the North-South and East-West directions. To determine the controlling wind load case, shear forces acting in each frame on story 6 were used. The wind load case that resulted in the highest shear forces in the frames was concluded to control the design. Tables 24 through 27 show the analysis results of the shear forces acting in each frame due to each wind load case.



Wind Load Case 1- Story 6		
Frame	X-Direction	Y-Direction
	Shear Force (kips)	Shear Force (kips)
MF-A.1	205.8	-
MF-B	152.7	-
MF-C	162.3	-
MF-E	48.7	-
MF-1	-	38.6
MF-1'	35.8	63.6
BF-1	-	327.1
BF-2	-	260.4
BF-3	-	289.2
BF-4	-	369.1
Average Shear=	**121.1**	**224.7** **kips**

Wind Load Case 2- level 6		
Frame	X-Direction	Y-Direction
	Shear Force (kips)	Shear Force (kips)
MF-A.1	172.8	-
MF-B	117.4	-
MF-C	111.6	-
MF-E	29.8	-
MF-1	-	25.0
MF-1'	14.8	106.3
BF-1	-	59.6
BF-2	-	172.8
BF-3	-	267.3
BF-4	-	427.6
Average Shear=	**89.3**	**176.4** **kips**

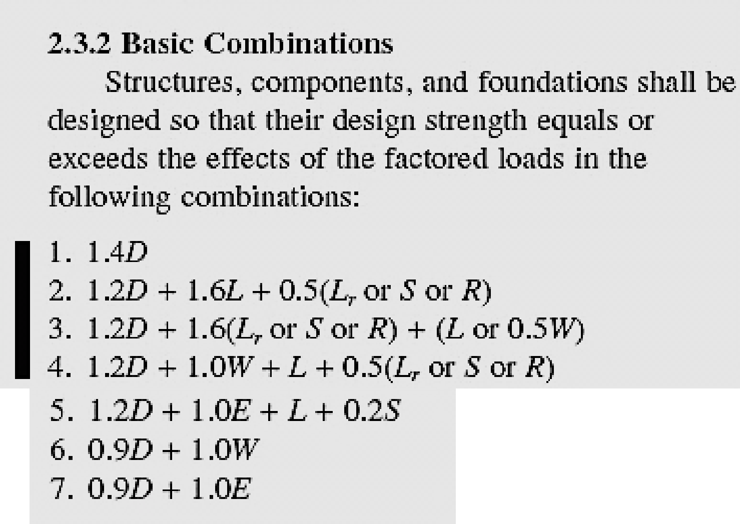
Wind Load Case 3- level 6	
Frame	Shear Force (kips)
MF-A.1	167.5
MF-B	125.0
MF-C	117.5
MF-E	36.5
MF-1	46.3
MF-1'	15.3
BF-1	312.8
BF-2	205.2
BF-3	206.0
BF-4	226.6
Average Shear=	**145.9** **kips**

Wind Load Case 4- level 6	
Frame	Shear Force (kips)
MF-A.1	184.6
MF-B	107.5
MF-C	40.1
MF-E	4.3
MF-1	17.8
MF-1'	68.8
BF-1	65.5
BF-2	132.0
BF-3	204.0
BF-4	317.6
Average Shear=	**190.4** **kips**

Tables 24-27 Shears acting in each frame due to the four wind load cases

As can be seen in Tables 24-27, the shear forces are greatest in the frames subjected to lateral wind load case 1, except in the case for brace frame B-4 which is subjected to a larger shear in load case 2. Overall, wind load case 1 is the controlling wind load case.

After determining the controlling wind load case, the load combination that would control the strength of the design was checked. Figure 36 shows a list of possible load combinations in ASCE 7-10.



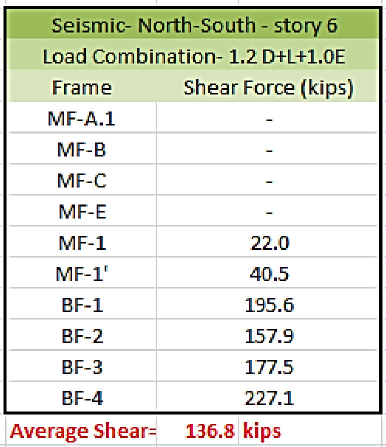
2.3.2 Basic Combinations

Structures, components, and foundations shall be designed so that their design strength equals or exceeds the effects of the factored loads in the following combinations:

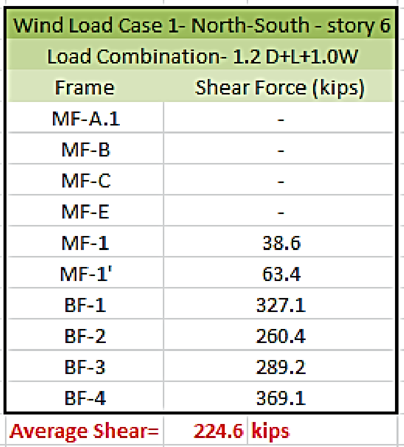
1. $1.4D$
2. $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$
4. $1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + L + 0.2S$
6. $0.9D + 1.0W$
7. $0.9D + 1.0E$

Figure 36 Load and Resistance Factor Design (LRFD) load combinations from Chapter 2 of ASCE 7-10

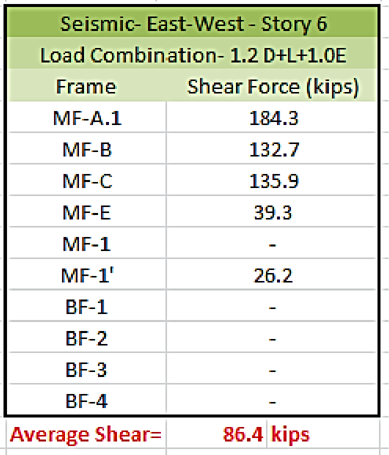
The controlling load combination for strength was found to be combinations 4 and 5. The two combinations were then checked in both the N-S and E-W directions to determine which one controlled the strength of the design. After analysis, it was found that load combination 4 controlled the strength of the design for both the N-S and E-W directions. According to section 12.8.4.2 of ASCE 7-10 for seismic design, for a rigid diaphragm the design must include the accidental torsional moments caused by assumed displacement of the center of mass each away from its actual location by a distance equal to 5% of the dimension of the structure perpendicular to the direction of the applied forces. As a result, this accidental torsional moment was taken into account by applying seismic loads in ETABS at a 5% eccentricity from the center of mass. For analysis, story 6 was used as a sample story to determine which load combination controlled the strength of the design. The analysis results can be seen in Tables 28 through 31.



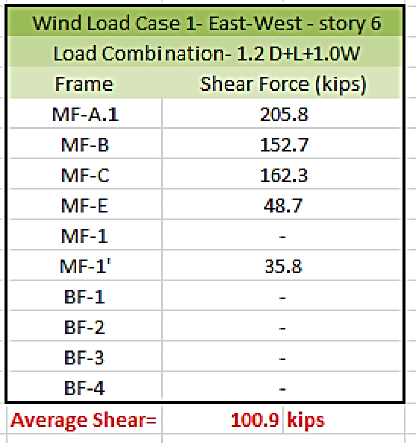
Seismic- North-South - story 6	
Load Combination- 1.2 D+L+1.0E	
Frame	Shear Force (kips)
MF-A.1	-
MF-B	-
MF-C	-
MF-E	-
MF-1	22.0
MF-1'	40.5
BF-1	195.6
BF-2	157.9
BF-3	177.5
BF-4	227.1
Average Shear=	136.8 kips



Wind Load Case 1- North-South - story 6	
Load Combination- 1.2 D+L+1.0W	
Frame	Shear Force (kips)
MF-A.1	-
MF-B	-
MF-C	-
MF-E	-
MF-1	38.6
MF-1'	63.4
BF-1	327.1
BF-2	260.4
BF-3	289.2
BF-4	369.1
Average Shear=	224.6 kips



Seismic- East-West - Story 6	
Load Combination- 1.2 D+L+1.0E	
Frame	Shear Force (kips)
MF-A.1	184.3
MF-B	132.7
MF-C	135.9
MF-E	39.3
MF-1	-
MF-1'	26.2
BF-1	-
BF-2	-
BF-3	-
BF-4	-
Average Shear=	86.4 kips



Wind Load Case 1- East-West - story 6	
Load Combination- 1.2 D+L+1.0W	
Frame	Shear Force (kips)
MF-A.1	205.8
MF-B	152.7
MF-C	162.3
MF-E	48.7
MF-1	-
MF-1'	35.8
BF-1	-
BF-2	-
BF-3	-
BF-4	-
Average Shear=	100.9 kips

Tables 28-31 Controlling load combinations that control strength of design

As can be seen in Tables 28-31 load combination 4 controls the strength of the design in both the N-S and E-W directions. The controlling load combination in both directions was used to check if the estimated designed members determined from the gravity only analysis was adequate to support the combined lateral and gravity loads and if the structural system was within the allowable drift limits. After using the steel frame design check in ETABS, it was shown that the estimated member sizes were not adequate to support the combined gravity and lateral loads and the structure displaced as much as 10 inches in the E-W direction and 12" in the N-S direction under the unfactored wind loads. With the building having a total height of 180 ft., using a drift limit of L/400 due to unfactored wind loads the structure can displace up to 5.4 inches to remain within the allowable drift limit. Using a drift limit of 0.02H due to unfactored seismic loads, the structure can displace up to 43.2 inches to remain within the allowable drift limit. To design the lateral system to meet both strength and drift requirements, the members were then assigned AUTO sections, which is an automatic select list of members chosen as prospective design members. The lateral displacement target for the system was also set to 4 inches to keep lateral drift to a minimum.

Initially it was assumed that diagonal bracing would be used as the brace frame configuration to resist the lateral loads in the N-S direction, but after running the steel design it was shown that the displacement in the N-S direction was beyond the allowable limit. Therefore the brace frame configuration was changed to X-bracing. After running the design with the braced frames with X bracing, the lateral drift in the N-S direction was within the allowable limits. Figure 37 shows the strength adequacy of the members chosen for the design.

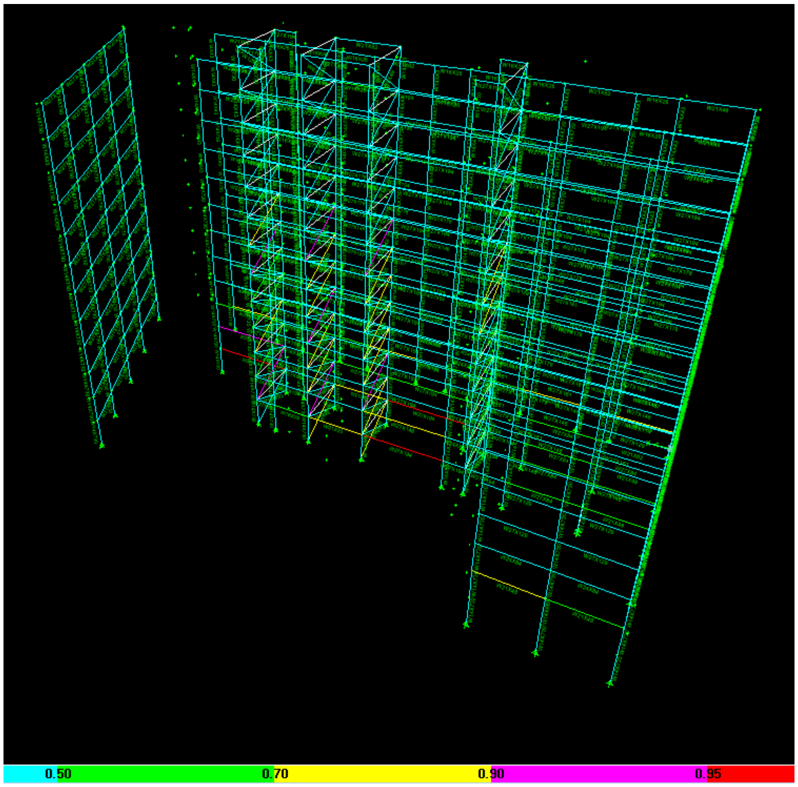


Figure 37 Strength of the design members with the use of the interaction diagram

The color at the bottom represent the interaction diagram in which red means the member is inadequate in strength to support the load and blue means the member is very adequate to support the load. The members in Figure 37 are all adequate to support the load except 3 members highlighted in red. This represents the members highlighted in red need to be increased in size to support the load. After re-rerunning the design, ETABS selected all members that were adequate to support the combined gravity and lateral loads and an overall system that was within the allowable drift limits. The final member selection can be seen in Figure 38.

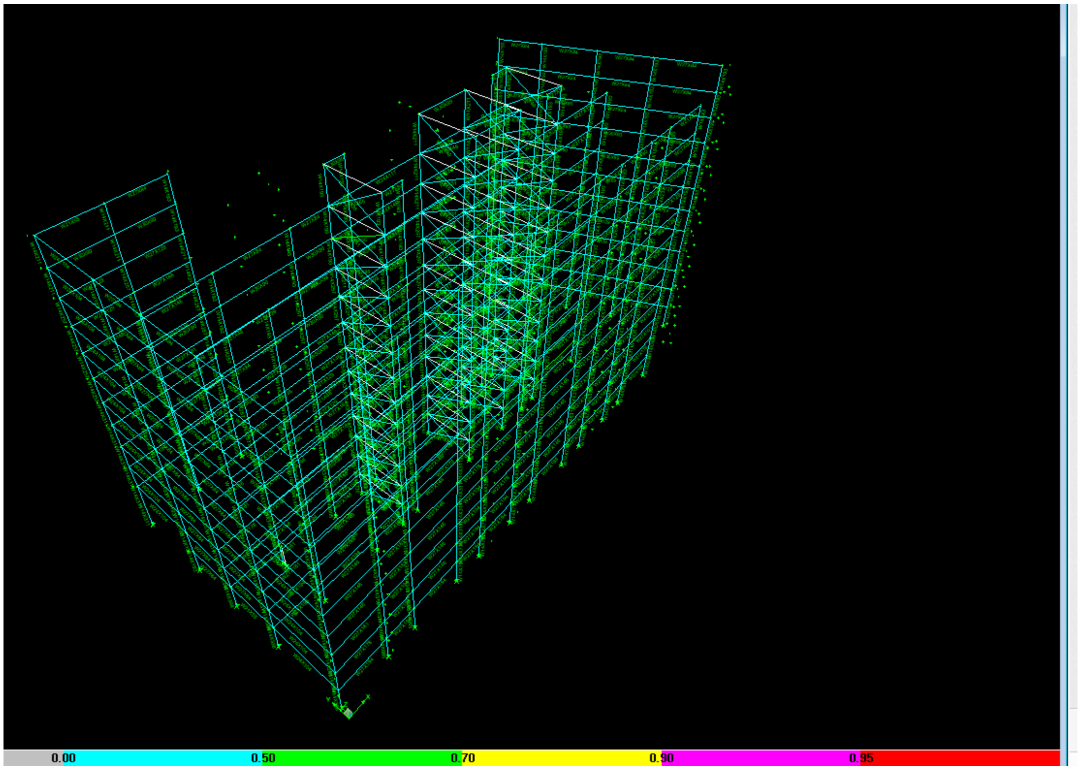


Figure 38 Final lateral system member selection

The final moment frame and braced frame design sections can be seen In Figures 39 through 46.

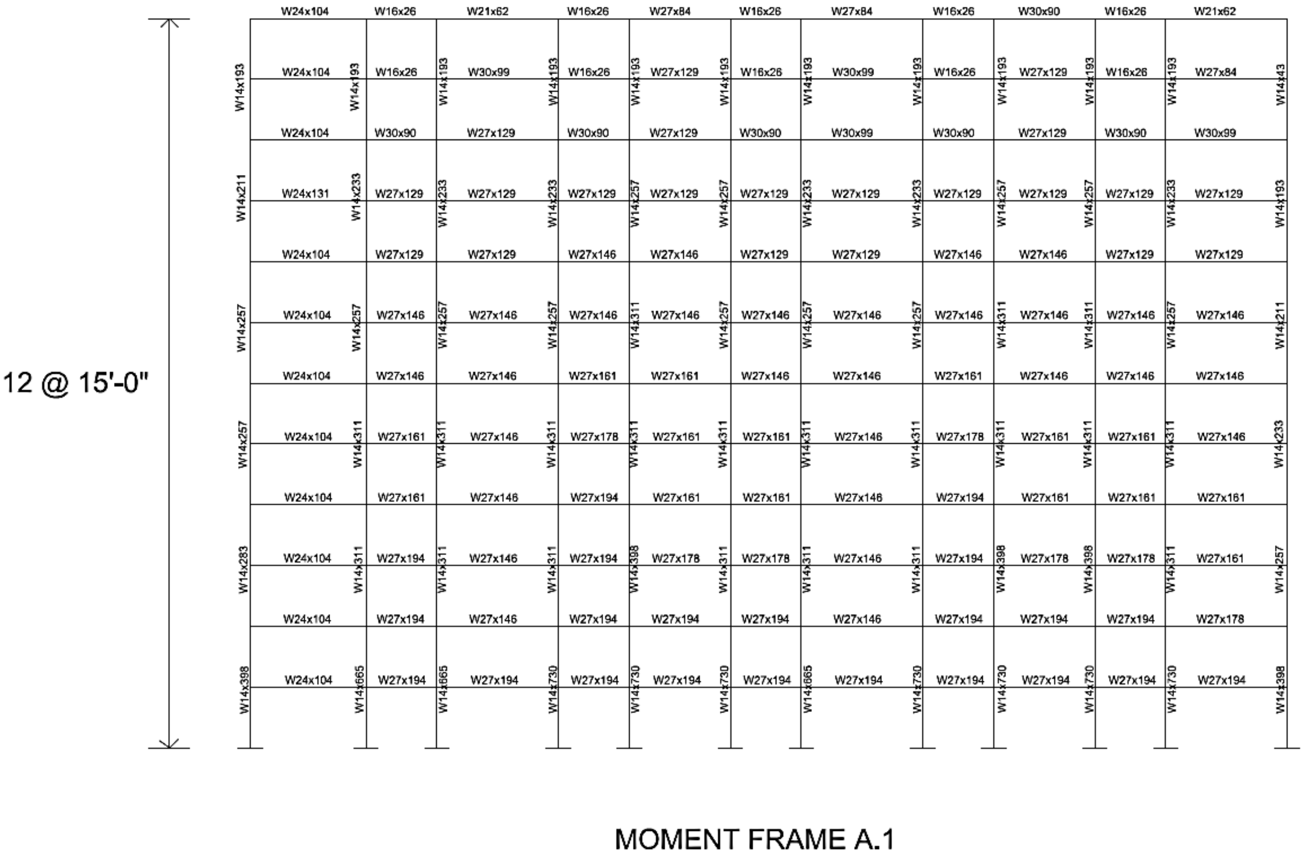


Figure 39 Moment Frame A.1

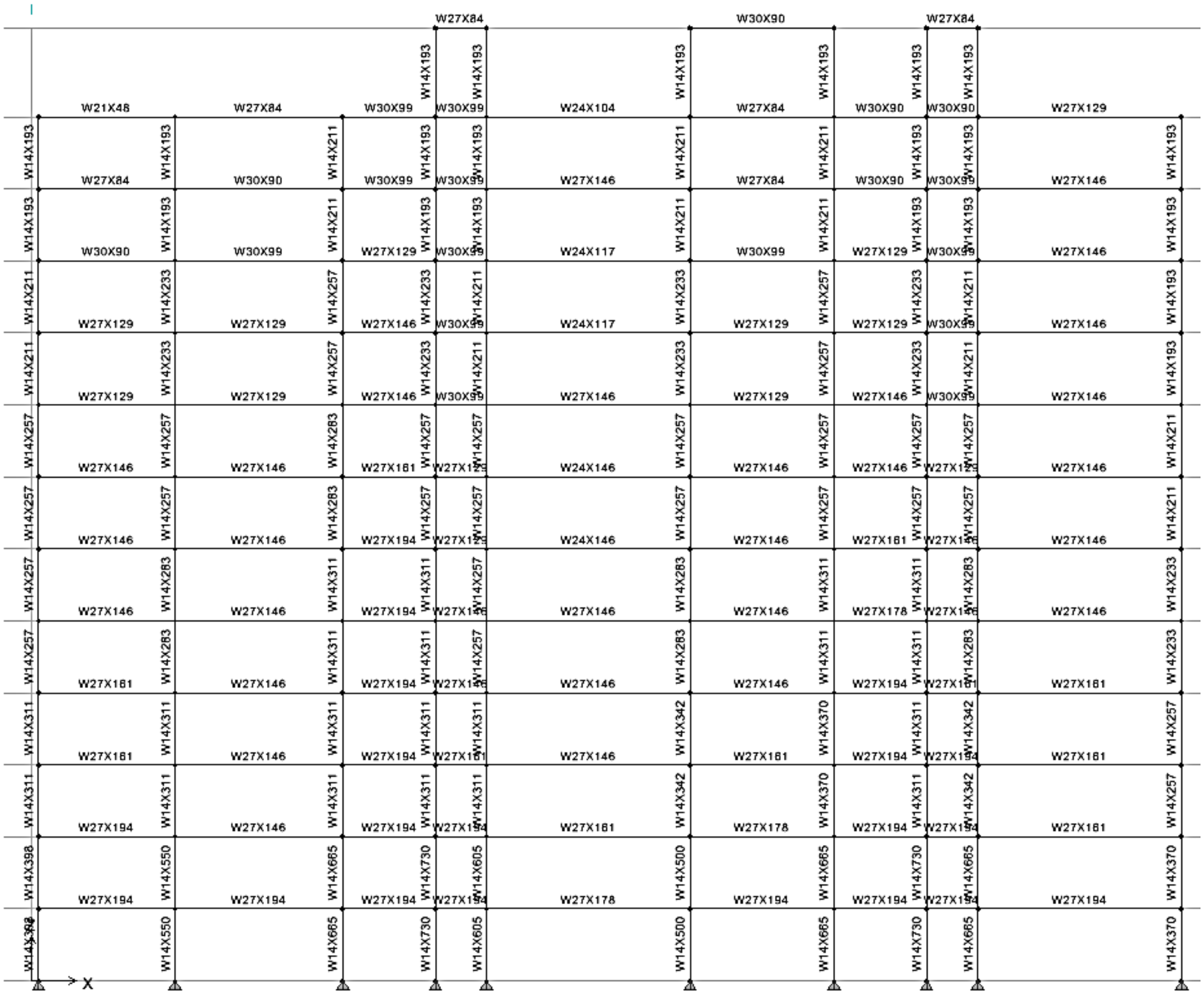


Figure 40 Moment Frame B

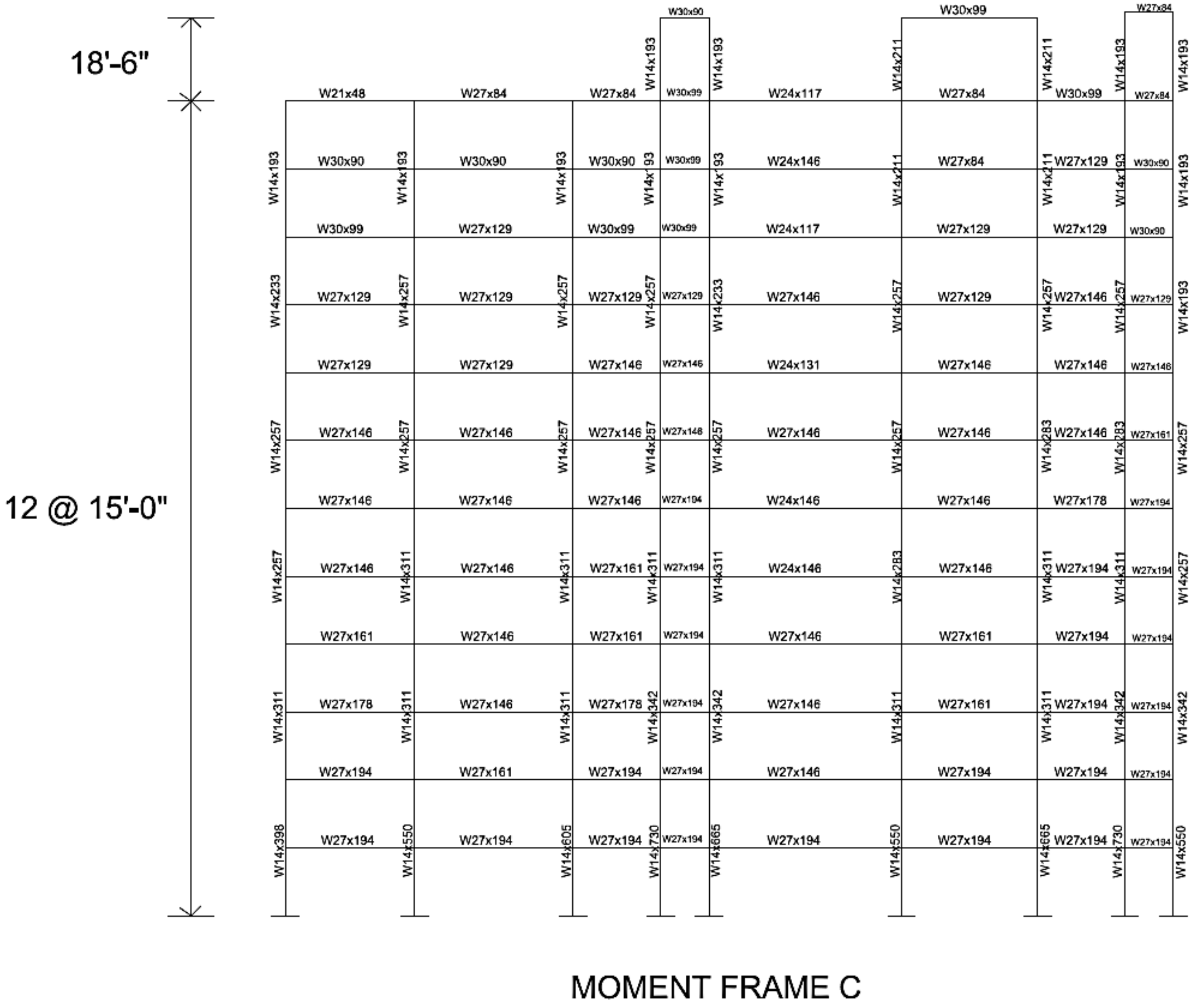


Figure 41 Moment Frame C

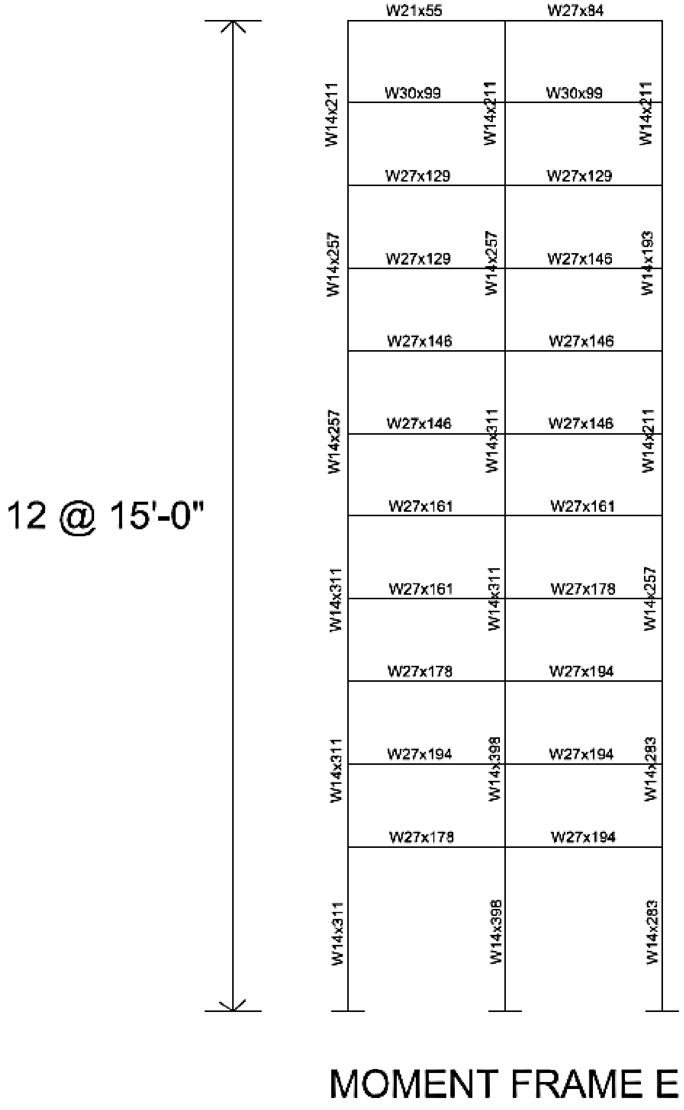


Figure 42 Moment Frame E

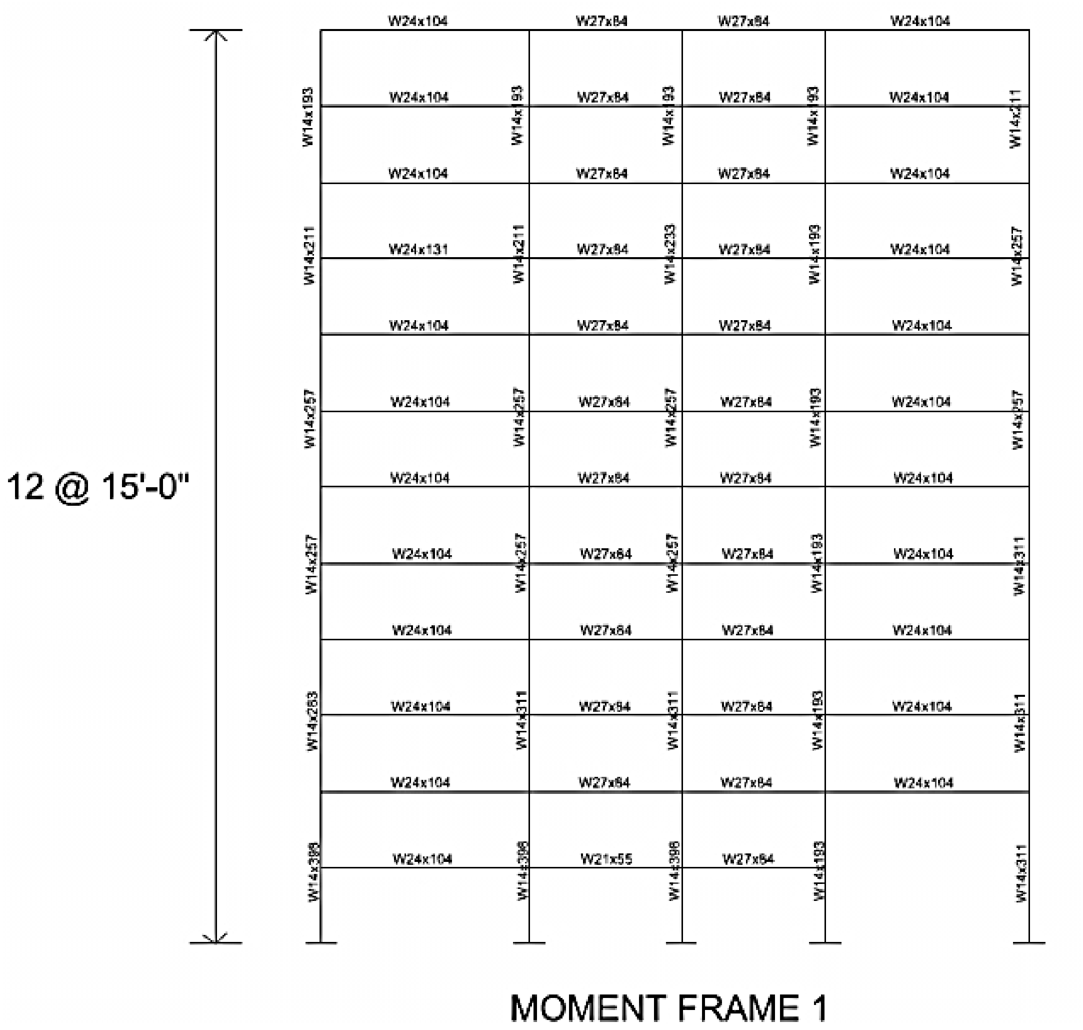


Figure 43 Moment Frame 1

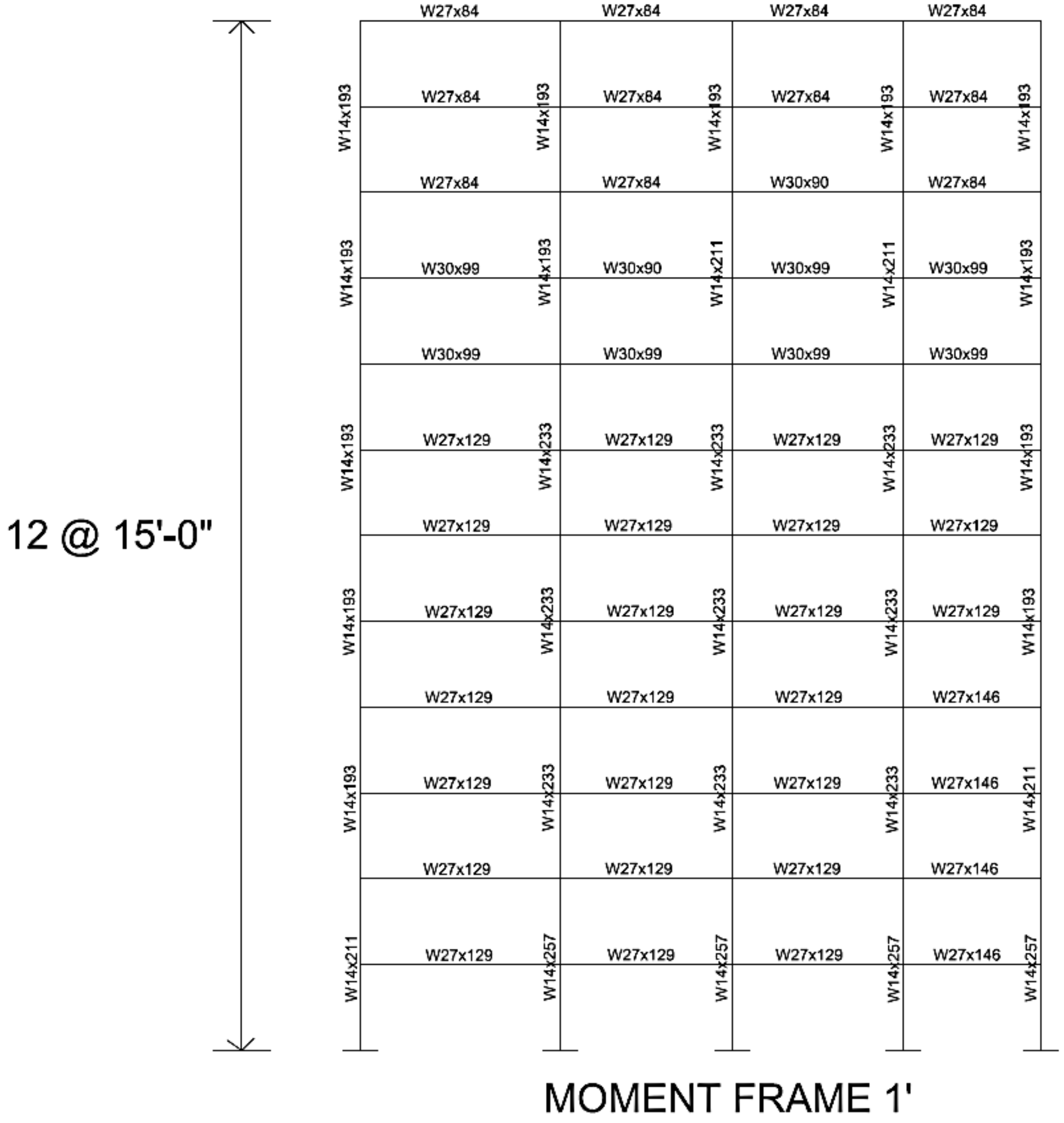
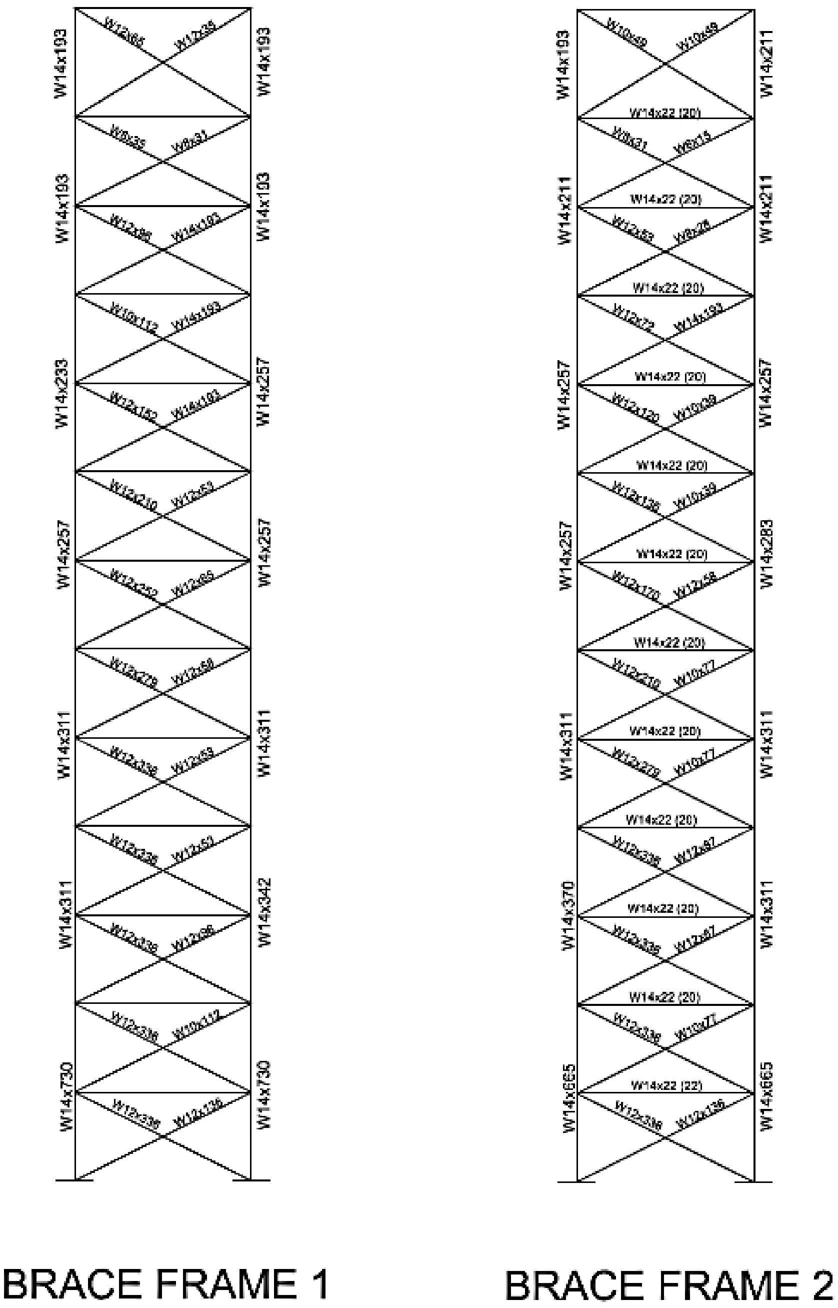


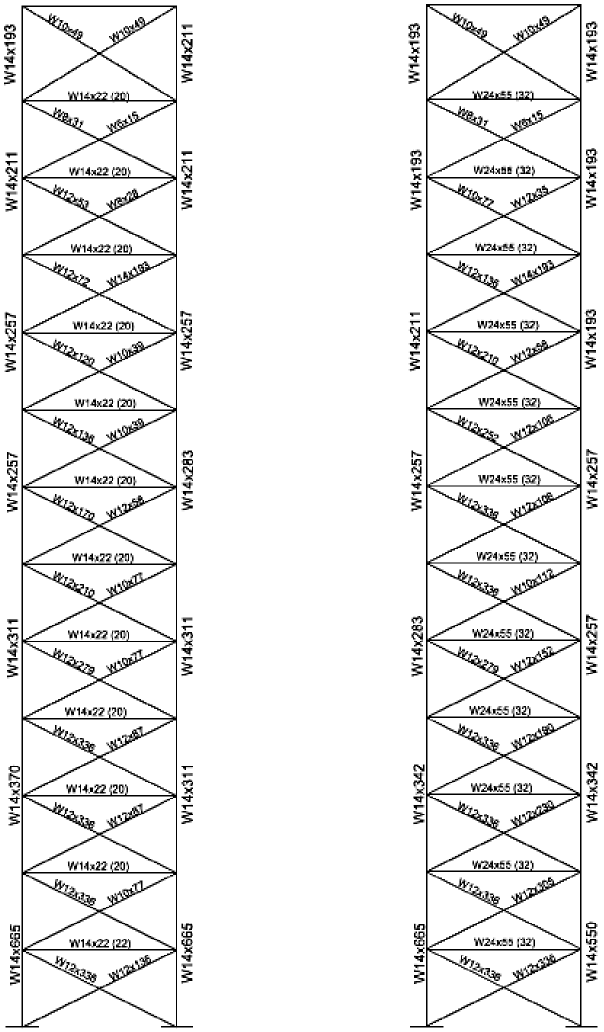
Figure 44 Moment Frame 1’



BRACE FRAME 1

BRACE FRAME 2

Figure 45 Braced frames 1 and 2



BRACE FRAME 3

BRACE FRAME 4

Figure 46 Braced frames 3 and 4

After designing the lateral system members, a typical moment frame connection was designed. The connection can be seen in Figure 47. The calculations for the typical moment connection design can be seen in Appendix D.

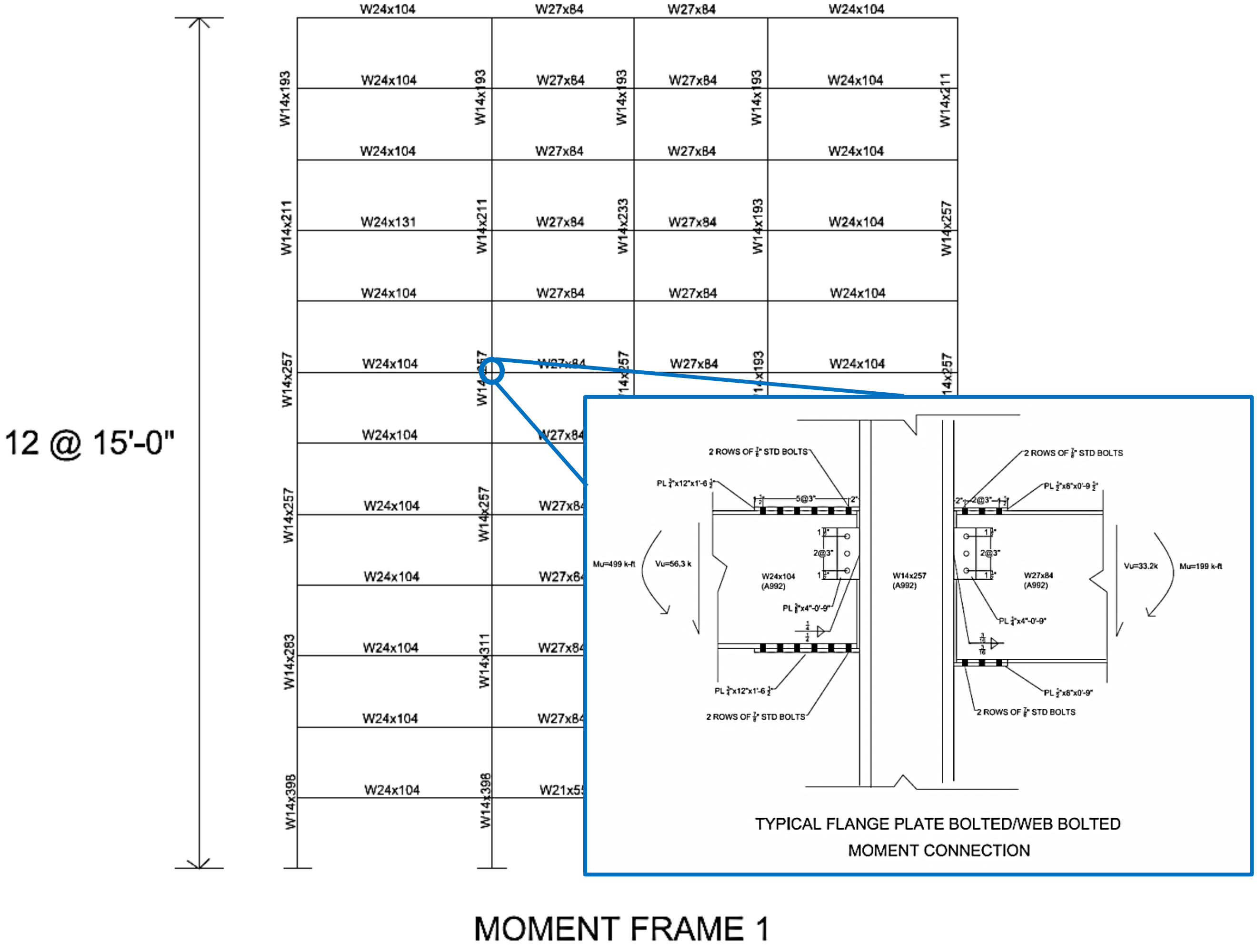


Figure 47 Typical moment frame connection

After designing the lateral system, the system was checked for relative stiffness, building torsion, lateral drift and displacement, and overturning moment.

Building Torsion

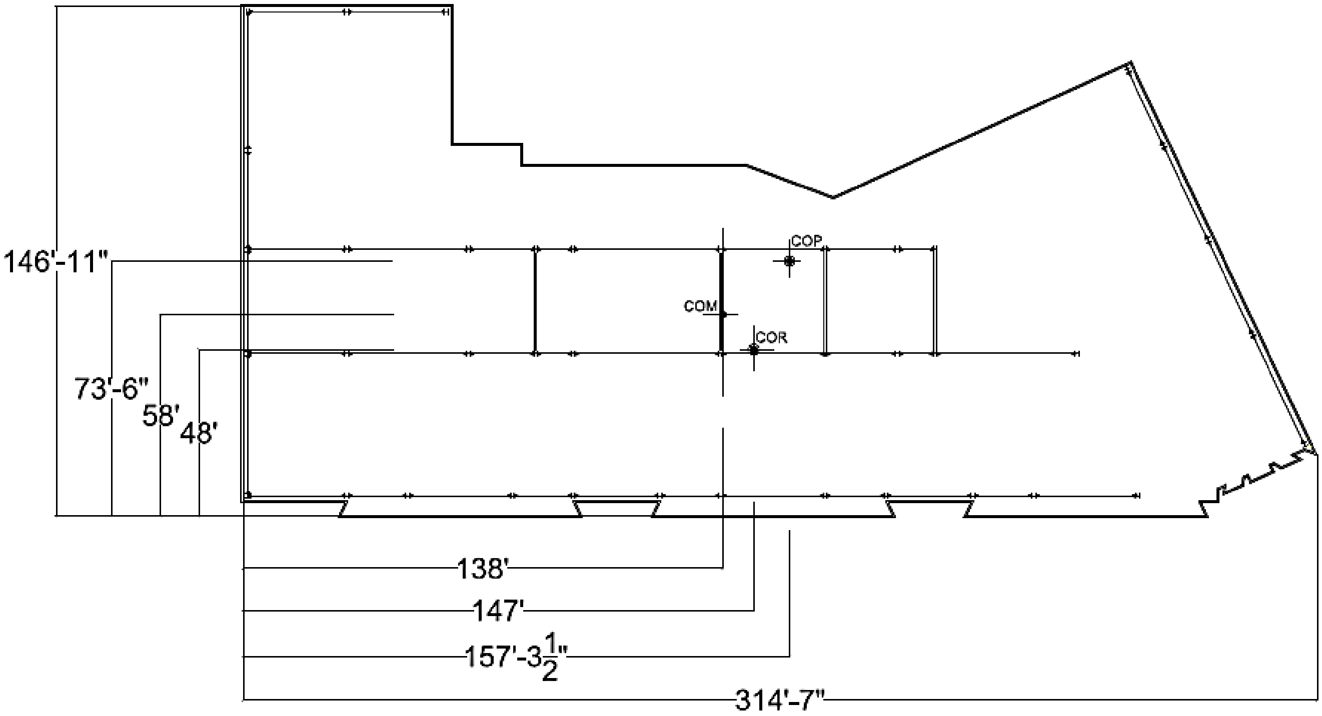
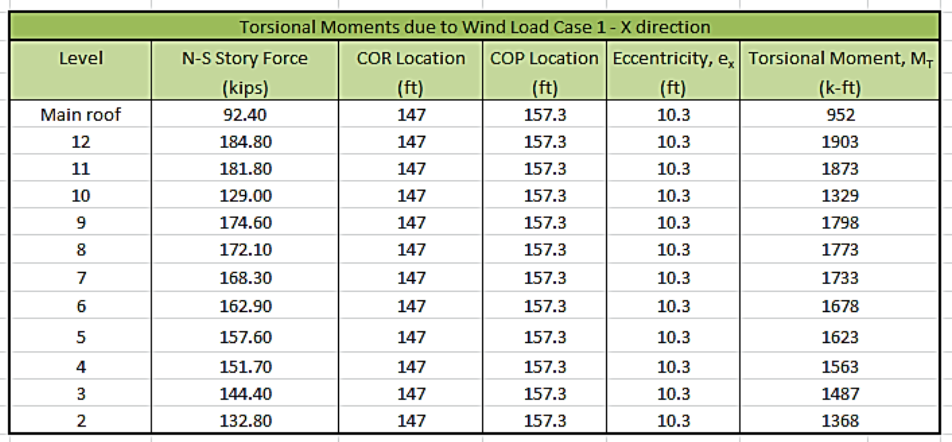


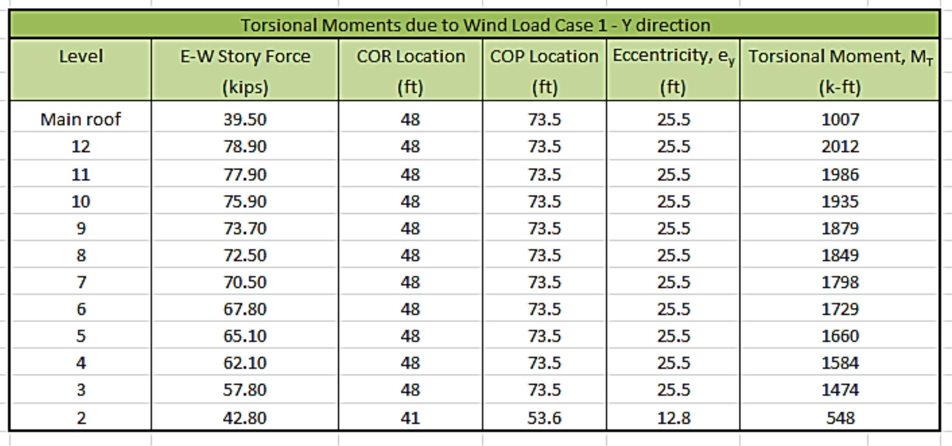
Figure 48 Plan view showing the location of the Center of Mass (COM), Center of Rigidity (COR), and Center of Pressure (COP)

When the Center of Mass (COM) and Center of Rigidity (COR) do not coincide, the building will be subjected to torsional effects caused by the seismic loads. In addition, wind loads act at the Center of Pressure (COP) and are resisted at the COR and if the COM and COP do not coincide, the building will be subjected to torsional effects caused by the wind loads. These torsional effects must be accounted for in design. To determine the total building torsion, one must consider the torsion due to the location difference between the COR and COM (or COR and COP). Torsional moments were calculated for the controlling wind load case 1, as can be seen in Table 32.



Torsional Moments due to Wind Load Case 1 - X direction

Level	N-S Story Force (kips)	COR Location (ft)	COP Location (ft)	Eccentricity, e_x (ft)	Torsional Moment, M_T (k-ft)
Main roof	92.40	147	157.3	10.3	952
12	184.80	147	157.3	10.3	1903
11	181.80	147	157.3	10.3	1873
10	129.00	147	157.3	10.3	1329
9	174.60	147	157.3	10.3	1798
8	172.10	147	157.3	10.3	1773
7	168.30	147	157.3	10.3	1733
6	162.90	147	157.3	10.3	1678
5	157.60	147	157.3	10.3	1623
4	151.70	147	157.3	10.3	1563
3	144.40	147	157.3	10.3	1487
2	132.80	147	157.3	10.3	1368



Torsional Moments due to Wind Load Case 1 - Y direction

Level	E-W Story Force (kips)	COR Location (ft)	COP Location (ft)	Eccentricity, e_y (ft)	Torsional Moment, M_T (k-ft)
Main roof	39.50	48	73.5	25.5	1007
12	78.90	48	73.5	25.5	2012
11	77.90	48	73.5	25.5	1986
10	75.90	48	73.5	25.5	1935
9	73.70	48	73.5	25.5	1879
8	72.50	48	73.5	25.5	1849
7	70.50	48	73.5	25.5	1798
6	67.80	48	73.5	25.5	1729
5	65.10	48	73.5	25.5	1660
4	62.10	48	73.5	25.5	1584
3	57.80	48	73.5	25.5	1474
2	42.80	41	53.6	12.8	548

Table 32 Torsional moments due to eccentric wind load case 1 in both the N-S and E-W directions

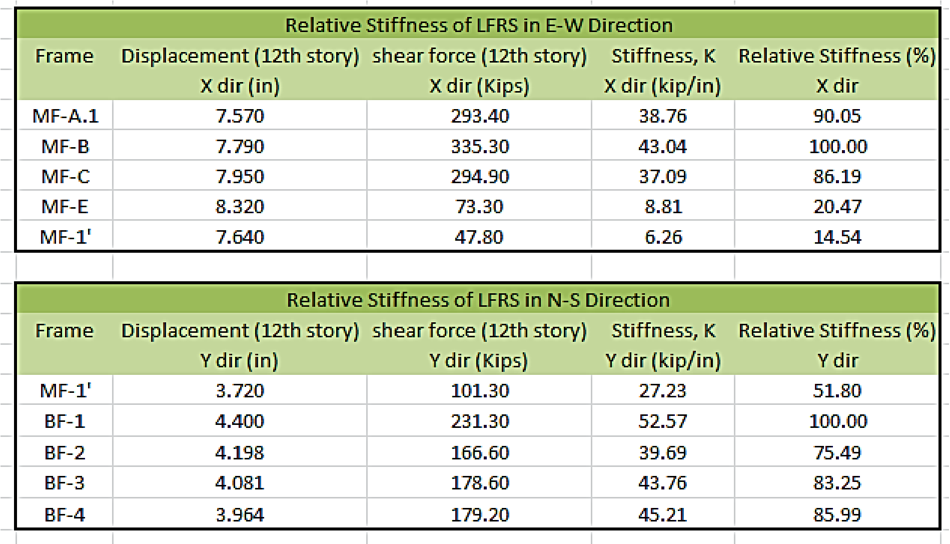
Relative Stiffness

The distribution of lateral story forces at a given story level to the lateral force resisting systems at that story is done according to the relative stiffness of each lateral system. The stiffness of each system is determined by applying a unit load at the top story of each lateral force resisting system element. The stiffer the system, the more lateral load it will resist. The location and orientation of each moment frame and braced frame can be seen in Figure 24. The stiffness of each frame was found in order to complete an analysis of both the direct and torsional shears, which will be discussed later in this report.

Each frame's stiffness was determined by applying a 1000 kip story load in the X –direction at the main roof level, which is the top level of the lateral force resisting system, and using ETABS to find the shear and displacement of each frame at the main roof level due to the 1000 kip story load. This same procedure was also applied to the Y-direction. The shear force and displacement in each frame at the main roof level were used to determine each frame's stiffness, K, where:

$K_i = P/\delta$, where P is the shear force in the frame at the main roof level and δ is the frame's displacement due to the 1000 k story load.

After determining each frame's stiffness, the relative stiffness was calculated by comparing the stiffness of each frame to the frame with the greatest stiffness. Firstly, the frame with the largest stiffness was set to have a relative stiffness of 1 (or 100 percent). The remaining frames' relative rigidity was determined by dividing each frame's stiffness, K, by the highest stiffness. This procedure was also applied to the Y-direction. Each frame's relative stiffness can be seen in Table 33.



Relative Stiffness of LFRS in E-W Direction

Frame	Displacement (12th story) X dir (in)	shear force (12th story) X dir (Kips)	Stiffness, K X dir (kip/in)	Relative Stiffness (%) X dir
MF-A.1	7.570	293.40	38.76	90.05
MF-B	7.790	335.30	43.04	100.00
MF-C	7.950	294.90	37.09	86.19
MF-E	8.320	73.30	8.81	20.47
MF-1'	7.640	47.80	6.26	14.54

Relative Stiffness of LFRS in N-S Direction

Frame	Displacement (12th story) Y dir (in)	shear force (12th story) Y dir (Kips)	Stiffness, K Y dir (kip/in)	Relative Stiffness (%) Y dir
MF-1'	3.720	101.30	27.23	51.80
BF-1	4.400	231.30	52.57	100.00
BF-2	4.198	166.60	39.69	75.49
BF-3	4.081	178.60	43.76	83.25
BF-4	3.964	179.20	45.21	85.99

Table 33 Relative stiffness of the Lateral Force Resisting Systems (LFRS)

As can be seen in Table 33, Moment Frame B (MF-B) resists the largest portion of the 1000 kip lateral load applied in the in the E-W direction because it's the stiffest frame in the E-W direction and thus resists a larger portion of the lateral loads acting in the E-W direction. Its location and span length relative to the other moment frame can be seen in Figure 24. Also Table33 shows that Brace Frame 1(BF-1) resists the largest portion of the 1000 kip lateral load applied in the N-S direction. This represents that brace frame 1 is the stiffest lateral force resisting frame in the N-S direction. Load follows stiffness and therefore the stiffer frames resist the largest portion of the lateral loads.

Lateral Load Distribution

Lateral force resisting systems resist lateral loads through direct shear and torsional shear. For 1000 Connecticut Avenue, to determine the portion of the story lateral force resisted by each frame, sample calculations were completed by solving for both the direct and torsional shears in each frame. The total shear in each frame was determined by adding the direct shear to the torsional shear. A plan view of the direction of the direct shear (DS) and torsional shear (TS) forces acting on the frames subjected to a 155 kip seismic lateral load acting on the main roof level in the N-S direction can be seen in Figure 49. The sample calculations for the direct shear and torsional shear acting on the North-South resisting frames due to the 155 kip seismic load can be seen in Tables 34 through 36.

Direct Shear

The frames that are parallel to the direct shear will participate in resistance. For example, the lateral loads acting in the North-South direction will be resisted directly by braced frames 1-4 and moment frames 1 and 1'. The lateral loads acting in the East-West direction will be resisted directly by moment frames A.1, B, C, E, and 1'. Since moment frame 1' is oriented at an angle, it will participate in resisting the lateral loads in both the N-S and E-W directions.

The direct shear of each frame was calculated by multiplying the relative stiffness of each frame by the lateral load. The relative stiffness represents the portion of the story lateral load resisted by the frame.

Relative stiffness= $= \frac{\mathrm{Ki}}{\sum \mathrm{ki}}$

Where,

K_i is the stiffness of the frame parallel to the lateral load

A sample distribution of the 155 kip seismic lateral load acting on the main roof level can be found in Table 34.

Torsional Shear

If the Center of Mass (COM) and Center of Rigidity (COR) do not coincide, then the seismic loads will cause torsional effects; seismic loads act through the COM, but are resisted through the COR. In addition, the wind loads act at the Center of Pressure (COP) and are resisted at the COR. Contrast to direct shear, all of the frames will participate in resisting these torsional effects. The torsional shear in each frame was first determined by finding the eccentricity between the COM and COR. Next, the distance between the frame and COR was determined where the distance is the moment arm between the COR and the frame. The torsional Shear equation with corresponding variable definitions can be seen below.

Torsional Shear, $V_i = \frac{\mathrm{Ved_iK_i}}{\sum \mathrm{K_id_i^2}}$

Where,

V- story lateral load
e- eccentricity (distance between the COM and COR or COM and COP)
Ki- stiffness of the lateral force resisting system element
di- moment arm between COR to the lateral force resisting system element

The sample calculations for torsional shears and total shears acting on the North-South resisting frames due to the 155 kip seismic load can be seen in Tables 35 and 36.

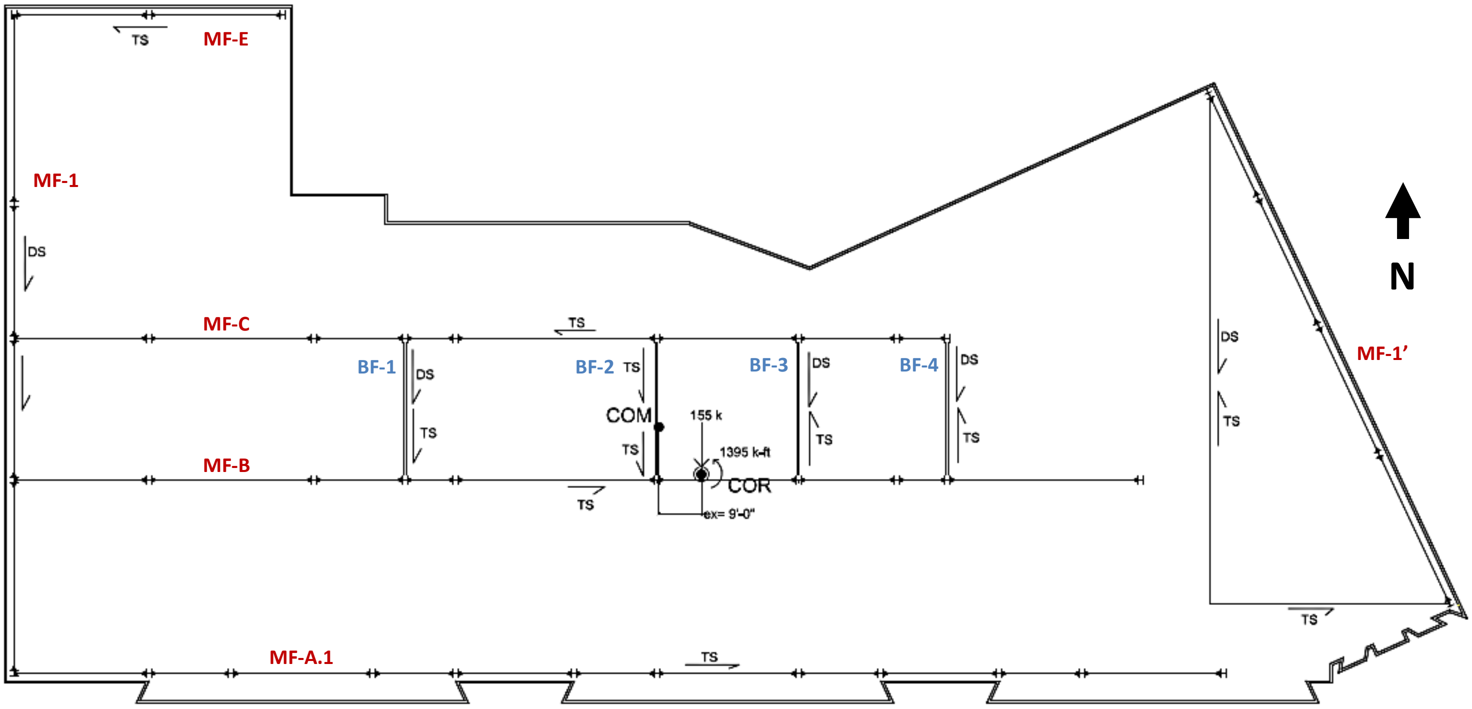
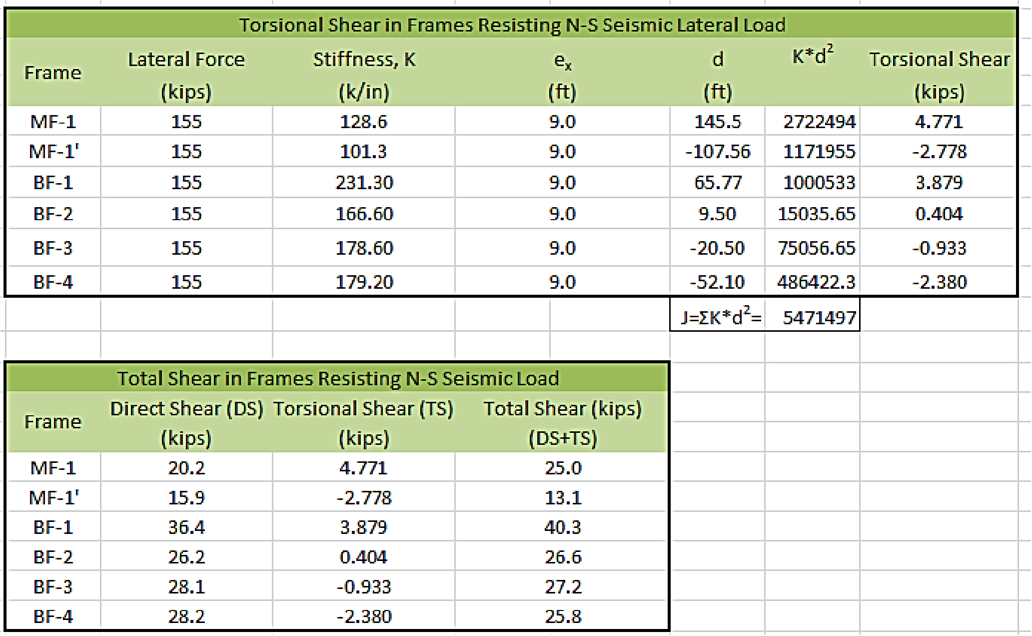


Figure 49 Plan view of the direction of the direct shear (DS) and torsional shear (TS) forces acting on the frames subjected to a 155 kip seismic lateral load acting on the main roof level in the N-S direction

Direct Shear in Frames Resisitng N-S Seismic Lateral Load			
Frame	Lateral Force (kips)	Stiffness, K (k/in)	Direct shear (kips)
MF-1	155	128.6	20.2
MF-1'	155	101.3	15.9
BF-1	155	231.30	36.4
BF-2	155	166.60	26.2
BF-3	155	178.60	28.1
BF-4	155	179.20	28.2

Table 34 Direct shear calculation for frames resisting 155 kip seismic load

As can be seen in Table 34, brace frame 1 resists the largest portion of the seismic load applied in the N-S direction. This was also shown in table 33 under the “relative stiffness” section in which it was shown that brace frame 1 would resist most of the lateral load because its stiffer than the other frames participating in resisting the direct shear. The stiffer the frame the more load it will resist because load follows stiffness. In addition, the torsional shears acting on the N-S frames can be seen in table 35 and the total shear acting on the N-S frames can be seen in Table 36.



Torsional Shear in Frames Resisting N-S Seismic Lateral Load						
Frame	Lateral Force (kips)	Stiffness, K (k/in)	e_x (ft)	d (ft)	$K*d^2$	Torsional Shear (kips)
MF-1	155	128.6	9.0	145.5	2722494	4.771
MF-1'	155	101.3	9.0	-107.56	1171955	-2.778
BF-1	155	231.30	9.0	65.77	1000533	3.879
BF-2	155	166.60	9.0	9.50	15035.65	0.404
BF-3	155	178.60	9.0	-20.50	75056.65	-0.933
BF-4	155	179.20	9.0	-52.10	486422.3	-2.380
				$J=\Sigma K*d^2=$	5471497	

Total Shear in Frames Resisting N-S Seismic Load			
Frame	Direct Shear (DS) (kips)	Torsional Shear (TS) (kips)	Total Shear (kips) (DS+TS)
MF-1	20.2	4.771	25.0
MF-1'	15.9	-2.778	13.1
BF-1	36.4	3.879	40.3
BF-2	26.2	0.404	26.6
BF-3	28.1	-0.933	27.2
BF-4	28.2	-2.380	25.8

Tables 35 and 36 Torsional shear and total shear acting on the N-S resisting frames

Story Drift and Lateral Displacement

The lateral displacements and story drifts were obtained from ETABS. This was done by using only un-factored wind and seismic loads. The inter-story drifts due to the un-factored wind load case 1 were compared to the H/400 allowable displacement, from ASCE 7-10, where H is the story-to-story- height. For the un-factored seismic loads, the inter-story drifts were compared to 0.020H from table 12.12-1 of ASCE 7-10, as can be seen in Figure 50. 1000 Connecticut Avenue has a risk category of II and has a combined moment frame and brace frame dual lateral system; therefore the allowable drift will be 0.02H, where H is the story-to-story height.

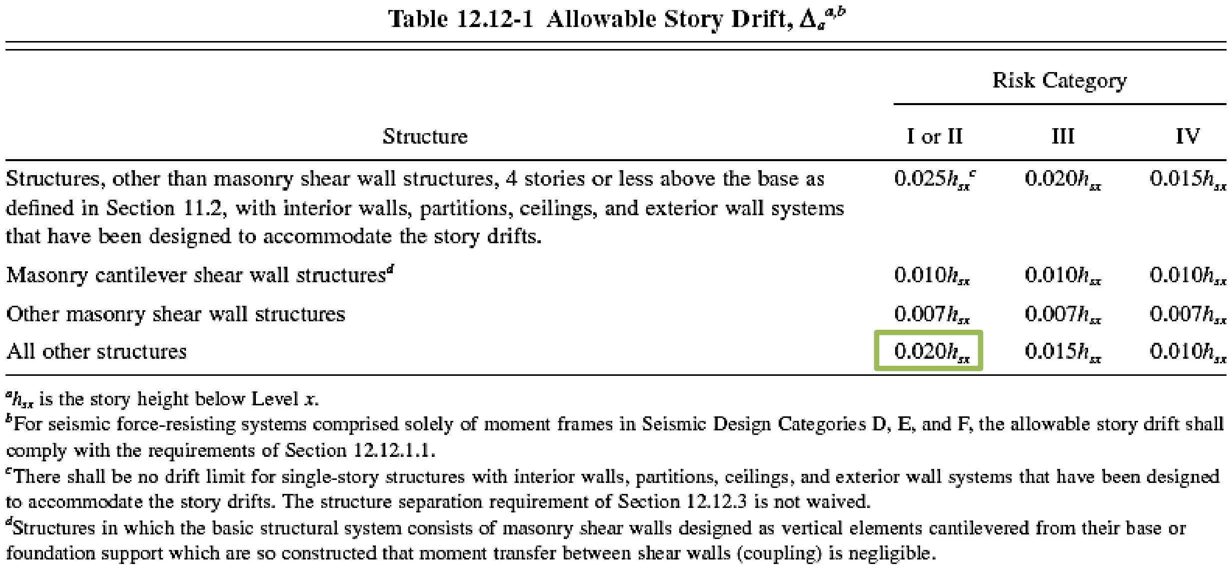


Table 12.12-1 Allowable Story Drift, $\Delta_a^{a,b}$

	Risk Category		
Structure	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less above the base as defined in Section 11.2, with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}^{c}$	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures[d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

[a] h_{sx} is the story height below Level x.

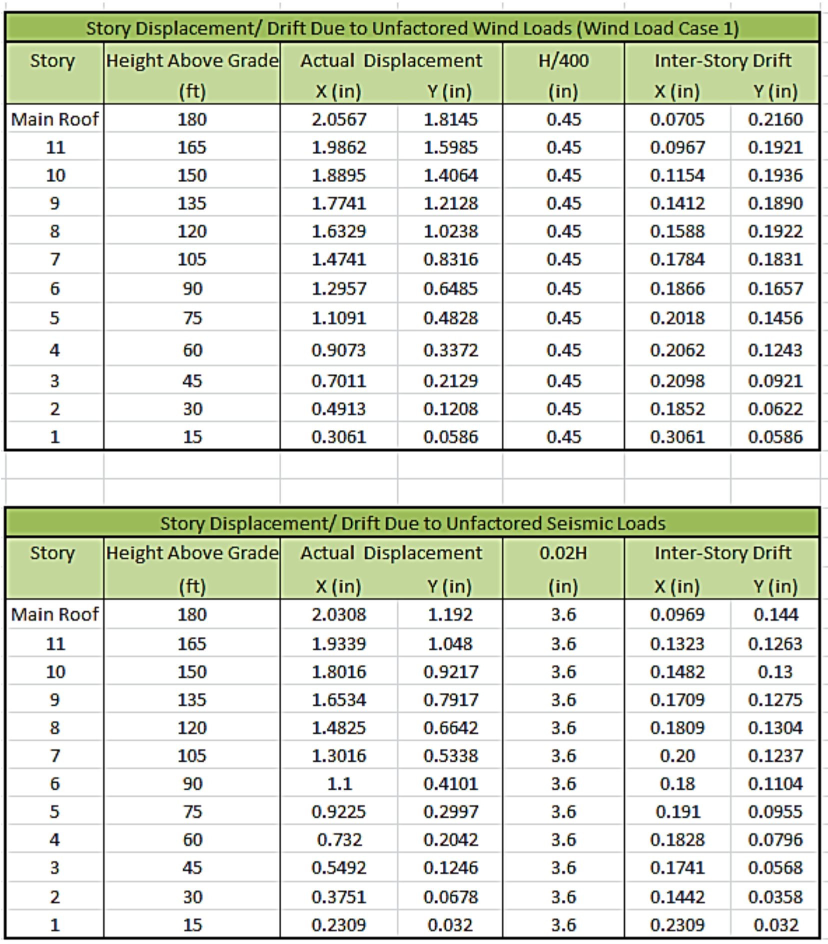
[b] For seismic force-resisting systems comprised solely of moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.

[c] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived.

[d] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

Figure 50 Table of allowable story drift for seismic loads

The serviceability for both the wind and seismic loads were found to be within the allowable limits. The story displacements and story drifts in the N-S and E-W directions can be found in Tables 37 and 38.

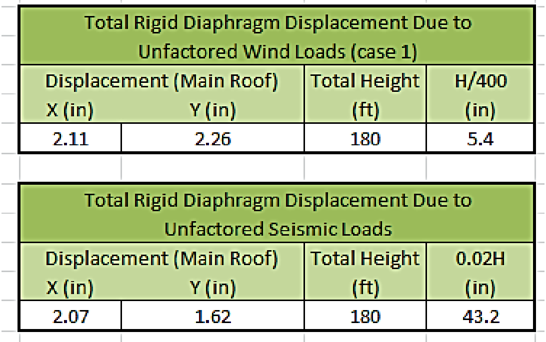


Story Displacement/ Drift Due to Unfactored Wind Loads (Wind Load Case 1)

Story	Height Above Grade	Actual Displacement		H/400	Inter-Story Drift	
	(ft)	X (in)	Y (in)	(in)	X (in)	Y (in)
Main Roof	180	2.0567	1.8145	0.45	0.0705	0.2160
11	165	1.9862	1.5985	0.45	0.0967	0.1921
10	150	1.8895	1.4064	0.45	0.1154	0.1936
9	135	1.7741	1.2128	0.45	0.1412	0.1890
8	120	1.6329	1.0238	0.45	0.1588	0.1922
7	105	1.4741	0.8316	0.45	0.1784	0.1831
6	90	1.2957	0.6485	0.45	0.1866	0.1657
5	75	1.1091	0.4828	0.45	0.2018	0.1456
4	60	0.9073	0.3372	0.45	0.2062	0.1243
3	45	0.7011	0.2129	0.45	0.2098	0.0921
2	30	0.4913	0.1208	0.45	0.1852	0.0622
1	15	0.3061	0.0586	0.45	0.3061	0.0586

Story Displacement/ Drift Due to Unfactored Seismic Loads

Story	Height Above Grade	Actual Displacement		0.02H	Inter-Story Drift	
	(ft)	X (in)	Y (in)	(in)	X (in)	Y (in)
Main Roof	180	2.0308	1.192	3.6	0.0969	0.144
11	165	1.9339	1.048	3.6	0.1323	0.1263
10	150	1.8016	0.9217	3.6	0.1482	0.13
9	135	1.6534	0.7917	3.6	0.1709	0.1275
8	120	1.4825	0.6642	3.6	0.1809	0.1304
7	105	1.3016	0.5338	3.6	0.20	0.1237
6	90	1.1	0.4101	3.6	0.18	0.1104
5	75	0.9225	0.2997	3.6	0.191	0.0955
4	60	0.732	0.2042	3.6	0.1828	0.0796
3	45	0.5492	0.1246	3.6	0.1741	0.0568
2	30	0.3751	0.0678	3.6	0.1442	0.0358
1	15	0.2309	0.032	3.6	0.2309	0.032



Total Rigid Diaphragm Displacement Due to Unfactored Wind Loads (case 1)

Displacement (Main Roof)		Total Height	H/400
X (in)	Y (in)	(ft)	(in)
2.11	2.26	180	5.4

Total Rigid Diaphragm Displacement Due to Unfactored Seismic Loads

Displacement (Main Roof)		Total Height	0.02H
X (in)	Y (in)	(ft)	(in)
2.07	1.62	180	43.2

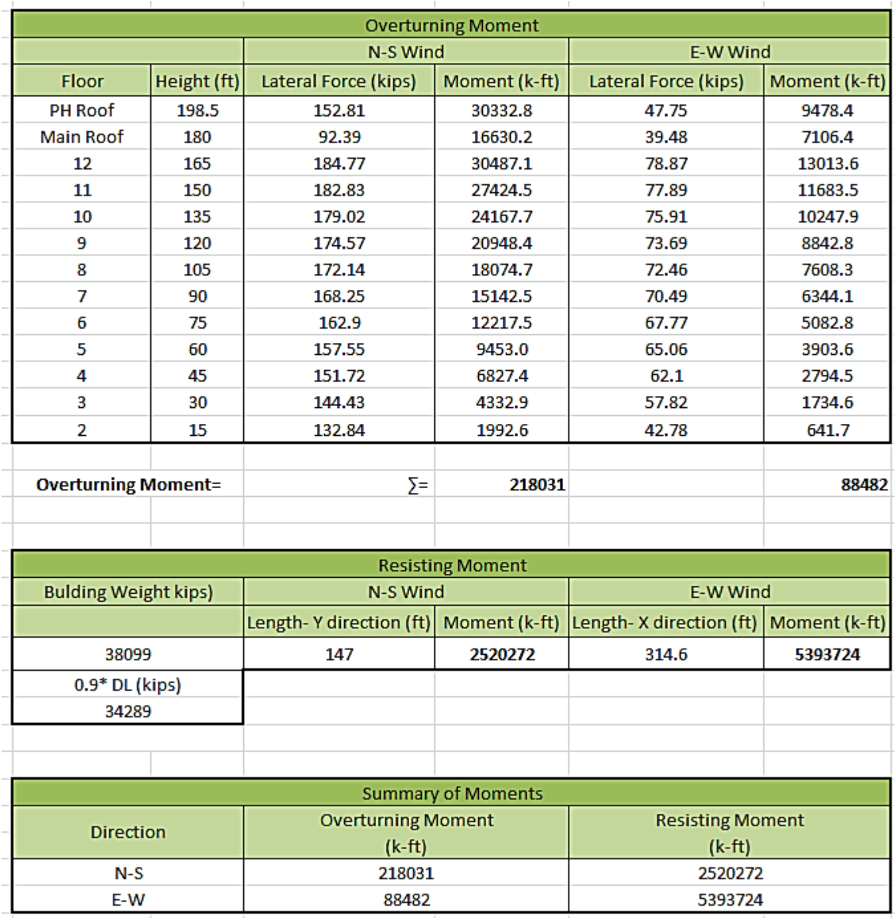
Tables 37 and 38 Story displacements/drifts due to un-factored wind and seismic loads

As can be seen in Tables 37 and 38, the inter-story drift of the lateral system is within the permissible limits for both the wind and seismic cases.

Overturning and Stability Analysis

A building's foundation must be designed to support both axial loads and bending moments caused by the lateral loads. The support base of lateral force resisting columns is subjected to uplift forces caused by the lateral loads. As a result, these uplift forces subject the building to overturning moments.

1000 Connecticut Avenue's foundation is comprised of spread footings, which behave as pinned connections due to their low rigidity. As a result, the foundation does not participate in resisting moments caused by the lateral loads. Through the analysis of the lateral system, the foundation was checked to determine if it is adequate to carry the moment due to the lateral forces on the slab, which transfers the load to the columns. The overturning moments were found by using the controlling lateral loads in each direction. It was determined in preceding sections of this thesis report that wind load case 1 was the controlling lateral load for both the North-South and East-West directions. Wind load case 1 was used to calculate the overturning moments by multiplying the lateral loads by the story height. The resisting moments were calculated by multiplying the total building weight by half of the building length, where the building length is in the direction in which the resisting moment is acting. Load combination 0.9D + 1.0W was found to control for checking the overturning moments. As can be seen in Table 39, the resisting moment is much greater than the overturning moment in both the N-S and E-W directions. Therefore, it was found that the slab-to-column moment frame systems below grade are adequate to carry the moments due to the lateral loads. Since the spread footings will behave as pinned connections, the columns will not transfer any moment to the foundation. Therefore the rigid connection between the slab and columns will carry the overturning moment.



Overturning Moment					
		N-S Wind		E-W Wind	
Floor	Height (ft)	Lateral Force (kips)	Moment (k-ft)	Lateral Force (kips)	Moment (k-ft)
PH Roof	198.5	152.81	30332.8	47.75	9478.4
Main Roof	180	92.39	16630.2	39.48	7106.4
12	165	184.77	30487.1	78.87	13013.6
11	150	182.83	27424.5	77.89	11683.5
10	135	179.02	24167.7	75.91	10247.9
9	120	174.57	20948.4	73.69	8842.8
8	105	172.14	18074.7	72.46	7608.3
7	90	168.25	15142.5	70.49	6344.1
6	75	162.9	12217.5	67.77	5082.8
5	60	157.55	9453.0	65.06	3903.6
4	45	151.72	6827.4	62.1	2794.5
3	30	144.43	4332.9	57.82	1734.6
2	15	132.84	1992.6	42.78	641.7
Overturning Moment=		∑=	**218031**		**88482**

Resisting Moment				
Bulding Weight kips)	N-S Wind		E-W Wind	
	Length- Y direction (ft)	Moment (k-ft)	Length- X direction (ft)	Moment (k-ft)
38099	**147**	**2520272**	**314.6**	**5393724**
0.9* DL (kips)				
34289				

Summary of Moments		
Direction	Overturning Moment (k-ft)	Resisting Moment (k-ft)
N-S	218031	2520272
E-W	88482	5393724

Table 39 Overturning and resisting moments in the N-S and E-W directions

In addition, with the lateral system consisting of braced frames, the braced frames will subject the foundation to uplift. As a result the foundation must be checked to determine if it is stable enough to resist the uplift forces. To check for uplift forces, brace frame 3 was used. The controlling load combination for checking uplift is 0.9D+1.0W. As can be seen in Figure 51, the braced frame is subjected to a factored tensile uplift force of 6123 kips.

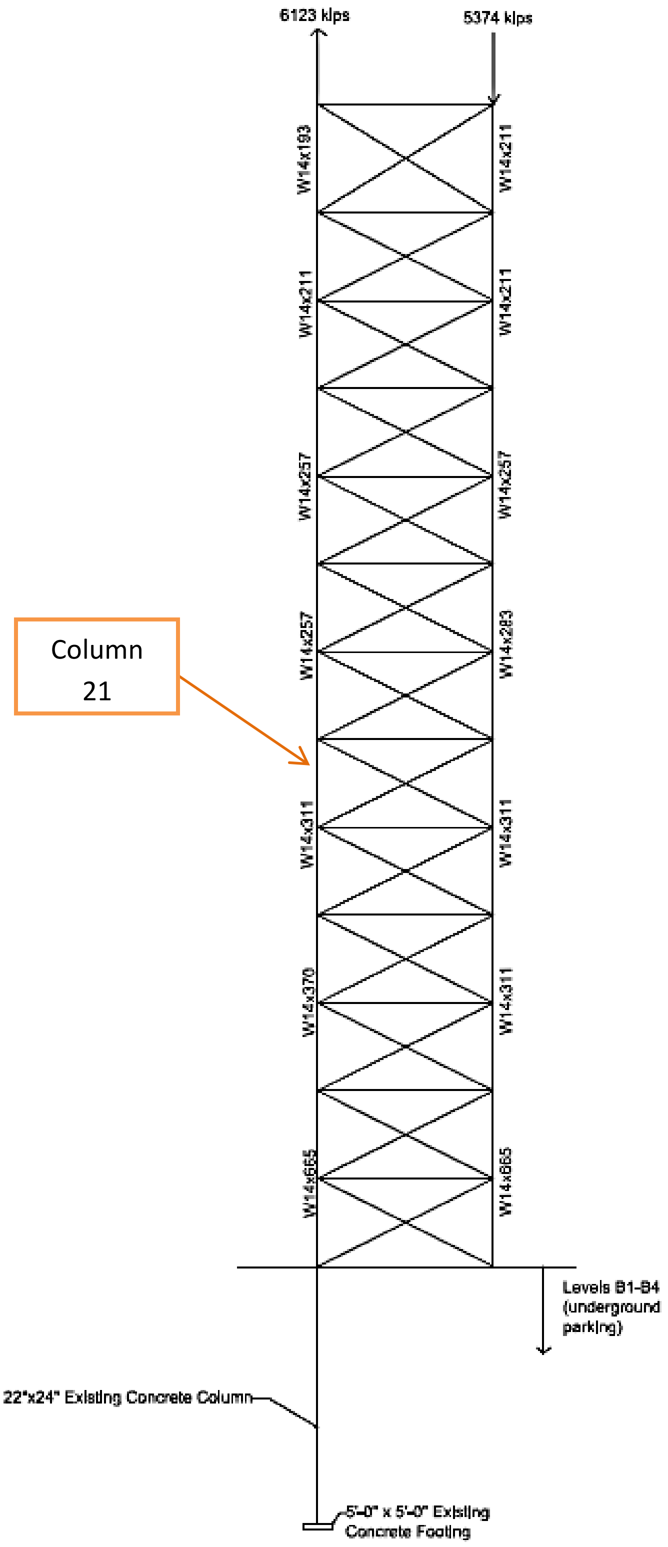
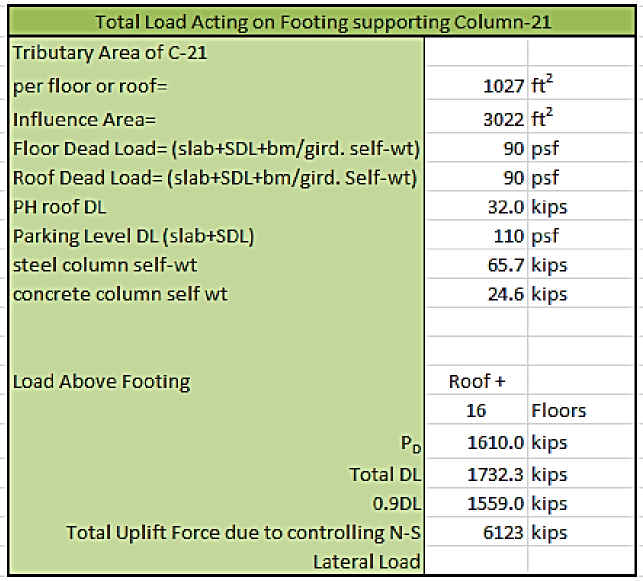


Figure 51 Uplift force braced frame is subjected to due to wind load case 1 acting in the N-S direction

The concrete footing subjected to the uplift force carries a resistive dead load of 1559 kips, which is smaller than the uplift force of 6123 kips acting on the footing. As a result, the foundation will need to be designed to resist this uplift force. A summary of the loads acting on the footing supporting column 21 can be seen in Table 40.



Total Load Acting on Footing supporting Column-21		
Tributary Area of C-21		
per floor or roof=	1027	ft^2
Influence Area=	3022	ft^2
Floor Dead Load= (slab+SDL+bm/gird. self-wt)	90	psf
Roof Dead Load= (slab+SDL+bm/gird. Self-wt)	90	psf
PH roof DL	32.0	kips
Parking Level DL (slab+SDL)	110	psf
steel column self-wt	65.7	kips
concrete column self wt	24.6	kips
Load Above Footing	Roof +	
	16	Floors
P_D	1610.0	kips
Total DL	1732.3	kips
0.9DL	1559.0	kips
Total Uplift Force due to controlling N-S Lateral Load	6123	kips

Table 40 Total load acting on footing supporting column 21

The existing foundation consists of spread footings, but in order to resist the uplift on the foundation caused by the braced frames there are three options that can be used to resist the uplift forces. One option is to use a grade beam that connects two spread footings to resist the uplift forces. The additional rigidity provided by the beam enables the foundation to resist the lateral loads. The grade beam configuration can be seen in Figure 52.

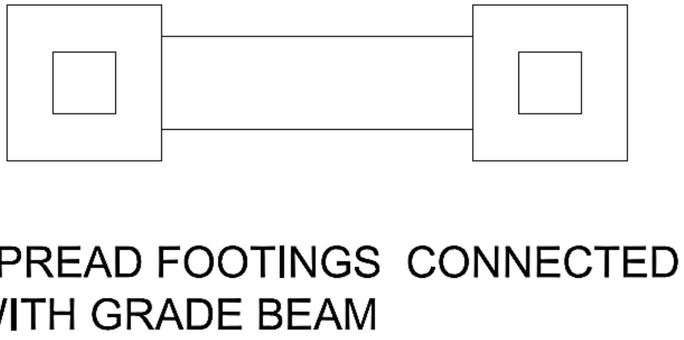


Figure 52 Spread footings connected with a grade beam

Another alternative foundation is to use a combined spread footing. The combined footings will have additional rigidity needed to resist the uplift forces subjected on it by the braced frames. Figure 53 displays a typical layout of a combined spread footing.

Figure 53 Combined spread footing

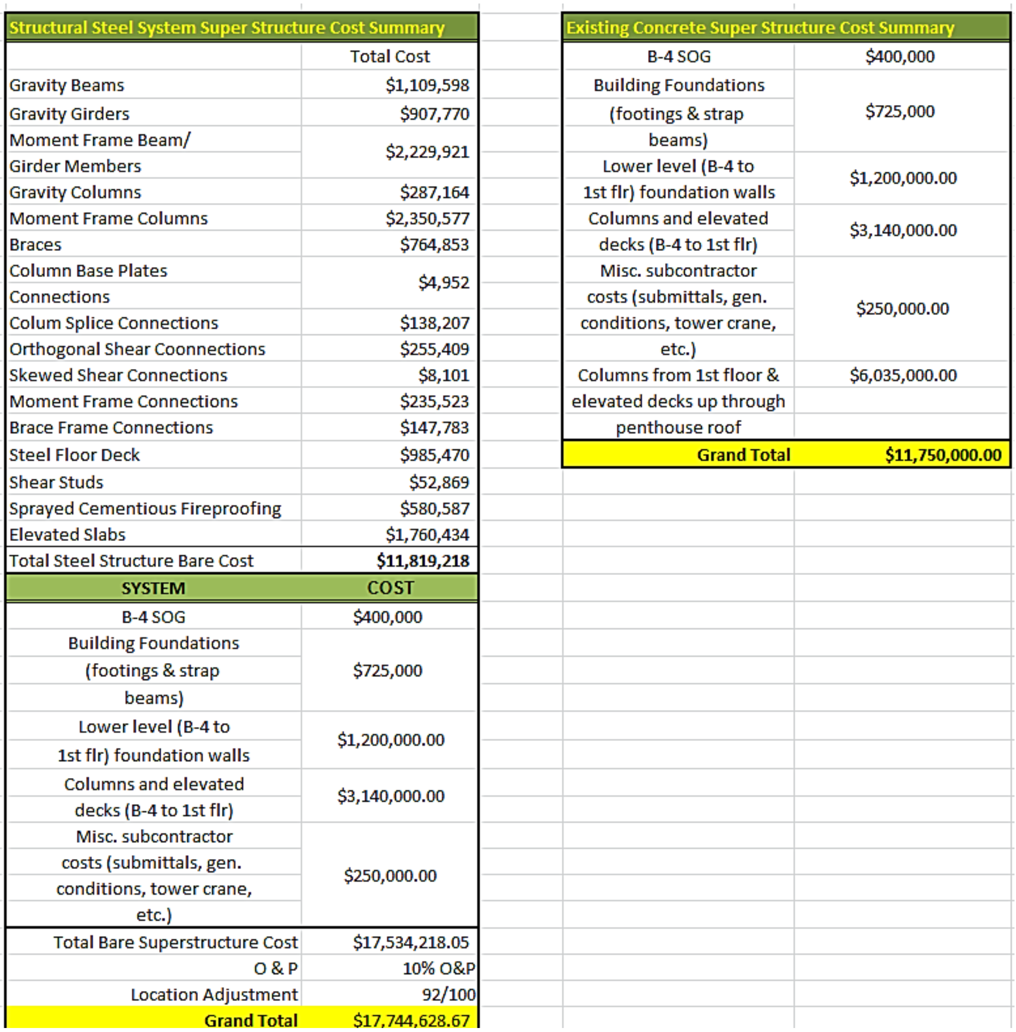
The last alternative is to use a mat foundation, which acts as a fixed base connection and thus will resist uplift forces.

Construction Management Breadth

The construction management breadth was analyzed to determine the impact the structural system redesign would have on the total building cost; construction schedule; site logistics of steel versus concrete construction; building LEED certification; and the anticipated revenue increase from the use of the new structural system. First, the current concrete construction cost estimate was compared to the cost estimate of the new structural system. Second, the new structural system construction schedule was compared to the existing system construction schedule. Third, how the existing construction site had to be managed differently for steel construction compared to concrete construction was evaluated. Fourth, the building LEED certification with the use of the new structural system was compared to the building LEED certification with the use of the existing concrete structural system. Last, the revenue obtained from the new structural system with wider bays and higher floor-to-ceiling heights was compared to the existing structural system's revenue. Wider bays and higher floor-to-ceiling heights increases the rental value of the floor space and therefore the building owner will be able to charge higher rent, which will potentially increase revenue.

New System Cost

After changing the structural system to steel, a cost analysis was completed to determine if the new system would cost less than the existing structural system. The cost was determined for the superstructure and the cost of the new superstructure was compared to the existing superstructure cost. A summary of each system's cost can be seen in Table 41. The analysis showed that the new structural system will cost $5,994,630 more than the cost of the existing superstructure. RS Means 2012 was used to determine the cost of the new structural system. The detailed superstructure cost calculations can be found in Appendix E.



Structural Steel System Super Structure Cost Summary	
	Total Cost
Gravity Beams	$1,109,598
Gravity Girders	$907,770
Moment Frame Beam/ Girder Members	$2,229,921
Gravity Columns	$287,164
Moment Frame Columns	$2,350,577
Braces	$764,853
Column Base Plates Connections	$4,952
Colum Splice Connections	$138,207
Orthogonal Shear Coonnections	$255,409
Skewed Shear Connections	$8,101
Moment Frame Connections	$235,523
Brace Frame Connections	$147,783
Steel Floor Deck	$985,470
Shear Studs	$52,869
Sprayed Cementious Fireproofing	$580,587
Elevated Slabs	$1,760,434
Total Steel Structure Bare Cost	**$11,819,218**
SYSTEM	**COST**
B-4 SOG	$400,000
Building Foundations (footings & strap beams)	$725,000
Lower level (B-4 to 1st flr) foundation walls	$1,200,000.00
Columns and elevated decks (B-4 to 1st flr)	$3,140,000.00
Misc. subcontractor costs (submittals, gen. conditions, tower crane, etc.)	$250,000.00
Total Bare Superstructure Cost	$17,534,218.05
O & P	10% O&P
Location Adjustment	92/100
Grand Total	$17,744,628.67

Existing Concrete Super Structure Cost Summary	
B-4 SOG	$400,000
Building Foundations (footings & strap beams)	$725,000
Lower level (B-4 to 1st flr) foundation walls	$1,200,000.00
Columns and elevated decks (B-4 to 1st flr)	$3,140,000.00
Misc. subcontractor costs (submittals, gen. conditions, tower crane, etc.)	$250,000.00
Columns from 1st floor & elevated decks up through penthouse roof	$6,035,000.00
Grand Total	**$11,750,000.00**

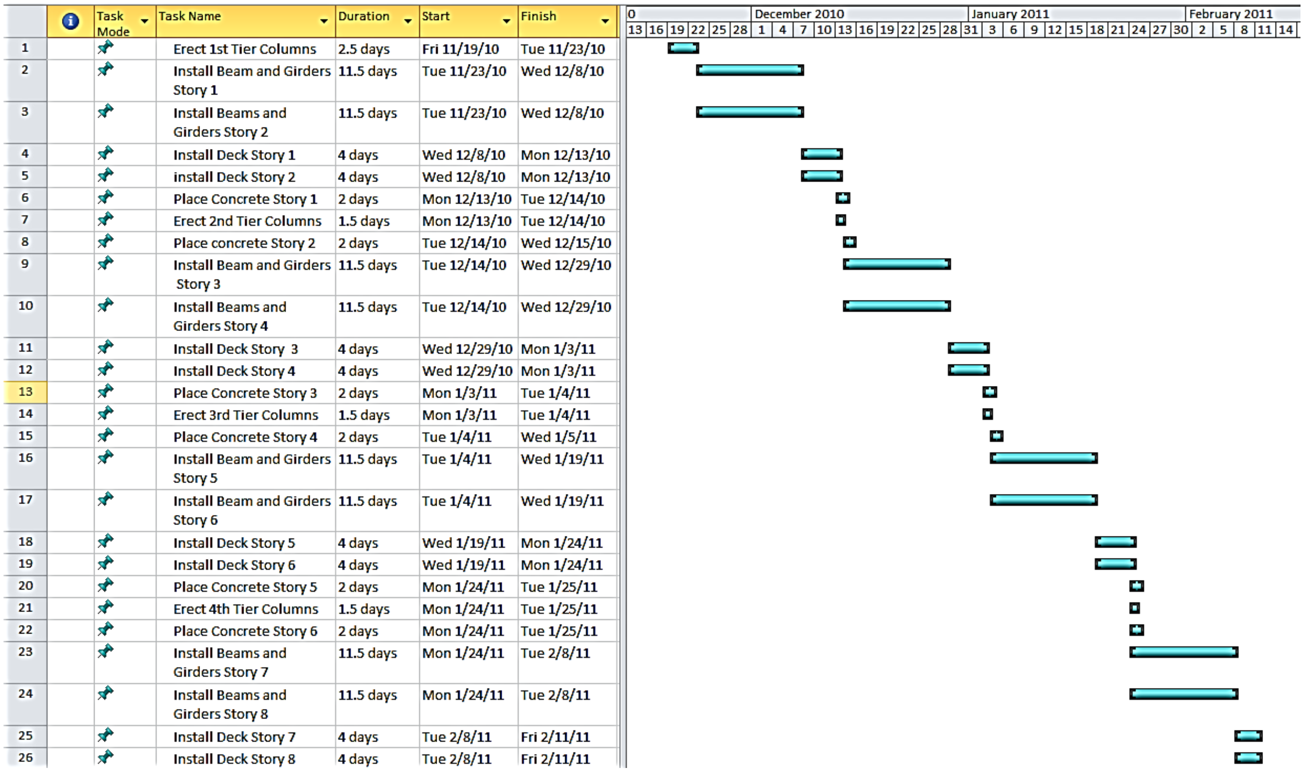
Table 41 New system cost versus existing system cost

Construction Schedule

After changing the structural system from steel to concrete, a construction schedule study was conducted to determine if the schedule of the new structural system can be shorten. The schedule path chosen to decrease the construction of the steel framing system can be seen listed below.

1. erect the first set 2 tier columns
2. erect the steel beams and girders at stories one and two
3. Erect the metal decks at stories 1 and 2
4. Pour the slab on deck at story 1 while the second set of 2 tier columns are being erected
5. Pour the slab on deck at story 2 while the beams and girders at stories 3 and 4 are being erected

The steel construction schedule will follow the above sequence until its completion. The steel system's proposed construction schedule can be seen in Figure 54. The schedule date starts on November 19, 2010 because that is the same day in which the existing concrete system reached grade level.



	ℹ	Task Mode	Task Name	Duration	Start	Finish
1		📌	Erect 1st Tier Columns	2.5 days	Fri 11/19/10	Tue 11/23/10
2		📌	Install Beam and Girders Story 1	11.5 days	Tue 11/23/10	Wed 12/8/10
3		📌	Install Beams and Girders Story 2	11.5 days	Tue 11/23/10	Wed 12/8/10
4		📌	Install Deck Story 1	4 days	Wed 12/8/10	Mon 12/13/10
5		📌	install Deck Story 2	4 days	Wed 12/8/10	Mon 12/13/10
6		📌	Place Concrete Story 1	2 days	Mon 12/13/10	Tue 12/14/10
7		📌	Erect 2nd Tier Columns	1.5 days	Mon 12/13/10	Tue 12/14/10
8		📌	Place concrete Story 2	2 days	Tue 12/14/10	Wed 12/15/10
9		📌	Install Beam and Girders Story 3	11.5 days	Tue 12/14/10	Wed 12/29/10
10		📌	Install Beams and Girders Story 4	11.5 days	Tue 12/14/10	Wed 12/29/10
11		📌	Install Deck Story 3	4 days	Wed 12/29/10	Mon 1/3/11
12		📌	Install Deck Story 4	4 days	Wed 12/29/10	Mon 1/3/11
13		📌	Place Concrete Story 3	2 days	Mon 1/3/11	Tue 1/4/11
14		📌	Erect 3rd Tier Columns	1.5 days	Mon 1/3/11	Tue 1/4/11
15		📌	Place Concrete Story 4	2 days	Tue 1/4/11	Wed 1/5/11
16		📌	Install Beam and Girders Story 5	11.5 days	Tue 1/4/11	Wed 1/19/11
17		📌	Install Beam and Girders Story 6	11.5 days	Tue 1/4/11	Wed 1/19/11
18		📌	Install Deck Story 5	4 days	Wed 1/19/11	Mon 1/24/11
19		📌	Install Deck Story 6	4 days	Wed 1/19/11	Mon 1/24/11
20		📌	Place Concrete Story 5	2 days	Mon 1/24/11	Tue 1/25/11
21		📌	Erect 4th Tier Columns	1.5 days	Mon 1/24/11	Tue 1/25/11
22		📌	Place Concrete Story 6	2 days	Mon 1/24/11	Tue 1/25/11
23		📌	Install Beams and Girders Story 7	11.5 days	Mon 1/24/11	Tue 2/8/11
24		📌	Install Beams and Girders Story 8	11.5 days	Mon 1/24/11	Tue 2/8/11
25		📌	Install Deck Story 7	4 days	Tue 2/8/11	Fri 2/11/11
26		📌	Install Deck Story 8	4 days	Tue 2/8/11	Fri 2/11/11

27		📌	Place Concrete Story 7	2 days	Fri 2/11/11	Mon 2/14/11
28		📌	Erect 5th Tier Columns	1.5 days	Fri 2/11/11	Mon 2/14/11
29		📌	Place Concrete Story 8	2 days	Mon 2/14/11	Tue 2/15/11
30		📌	Install Beams and Girders Story 9	11.5 days	Mon 2/14/11	Tue 3/1/11
31		📌	Install Beams and Girders Story 10	11.5 days	Mon 2/14/11	Tue 3/1/11
32		📌	Install Deck Story 9	4 days	Tue 3/1/11	Fri 3/4/11
33		📌	Install Deck Story 10	4 days	Tue 3/1/11	Fri 3/4/11
34		📌	Place Concrete Story 9	2 days	Fri 3/4/11	Mon 3/7/11
35		📌	Erect 6th Tier Columns	1.5 days	Fri 3/4/11	Mon 3/7/11
36		📌	Place Concrete Story 10	2 days	Mon 3/7/11	Tue 3/8/11
37		📌	Install Beams and Girders Story 11	11.5 days	Mon 3/7/11	Tue 3/22/11
38		📌	Install Beams and Girders story 12	11.5 days	Mon 3/7/11	Tue 3/22/11
39		📌	Install Deck Story 11	4 days	Tue 3/22/11	Fri 3/25/11
40		📌	Install Deck Story 12	4 days	Tue 3/22/11	Fri 3/25/11
41		📌	Place Concrete Story 11	2 days	Fri 3/25/11	Mon 3/28/11
42	✓	📌	Place Concrete Story 12	2 days	Fri 3/25/11	Mon 3/28/11

Figure 54 Proposed construction sequence for the steel framing system

The existing system's first level through main roof concrete construction schedule sequence can be seen in Figure 55.

First Floor

Concrete Operations

Typical Activities

AGS100000	FRP Elevated Deck - 01	20	0	13OCT10A	19NOV10A
AGS100020	FRP Columns - 01	16	0	01NOV10A	02DEC10A
AGS100120	Strip/Reshore - 01 to 02	10	0	02DEC10A	15DEC10A
BGS100220	Remove Reshores - B1 to 01	10	0	29DEC10A	03JAN11A
CN121700	FRP Concrete Pads/Curbs - 01	5	0	07MAR11A	10MAY11A

Second Floor

Concrete Operations

Typical Activities

AGS100080	FRP Elevated Deck - 02	14	0	05NOV10A	15DEC10A
AGS100100	FRP Columns - 02	11	0	29NOV10A	16DEC10A
AGS100200	Strip/Reshore - 02 to 03	10	0	29NOV10A	13DEC10A
AGS100060	Remove Reshores - 01 to 02	10	0	03JAN11A	26JAN11A
CN121720	FRP Concrete Pads/Curbs - 02	5	0	23FEB11A	25FEB11A

Third Floor

Concrete Operations

Typical Activities

AGS100160	FRP Elevated Deck - 03	10	0	08DEC10A	28DEC10A
AGS100180	FRP Columns - 03	6	0	10DEC10A	29DEC10A
AGS100280	Strip/Reshore - 03 to 04	10	0	31DEC10A	14JAN11A
AGS100140	Remove Reshores - 02 to 03	10	0	17JAN11A	25JAN11A
CN121740	FRP Concrete Pads/Curbs - 03	5	0	23FEB11A	25FEB11A

Fourth Floor

Concrete Operations

Typical Activities

AGS100240	FRP Elevated Deck - 04	9	0	22DEC10A	06JAN11A
AGS100260	FRP Columns - 04	5	0	27DEC10A	07JAN11A
AGS100360	Strip/Reshore - 04 to 05	10	0	14JAN11A	28JAN11A

Fifth Floor

Concrete Operations

Typical Activities

AGS100320	FRP Elevated Deck - 05	9	0	03JAN11A	14JAN11A
AGS100340	FRP Columns - 05	5	0	10JAN11A	17JAN11A
AGS100440	Strip/Reshore - 05 to 06	10	0	12JAN11A	28JAN11A
AGS100300	Remove Reshores - 04 to 05	10	0	15FEB11A	18FEB11A
CN121780	FRP Concrete Pads/Curbs - 05	5	0	15MAR11A	25APR11A

Sixth Floor

Concrete Operations

Typical Activities

AGS100400	FRP Elevated Deck - 06	9	0	12JAN11A	25JAN11A
AGS100420	FRP Columns - 06	5	0	17JAN11A	26JAN11A
AGS100520	Strip/Reshore - 06 to 07	10	0	04FEB11A	14FEB11A
AGS100380	Remove Reshores - 05 to 06	10	0	16FEB11A	22FEB11A
CN121800	FRP Concrete Pads/Curbs - 06	5	0	15MAR11A	25APR11A

Seventh Floor

Concrete Operations

Typical Activities

AGS100480	FRP Elevated Deck - 07	9	0	20JAN11A	07FEB11A
AGS100500	FRP Columns - 07	6	0	01FEB11A	08FEB11A
AGS100600	Strip/Reshore - 07 to 08	10	0	17FEB11A	22FEB11A
AGS100460	Remove Reshores - 06 to 07	10	0	28FEB11A	09MAR11A
CN121820	FRP Concrete Pads/Curbs - 07	5	0	27APR11A	02MAY11A

Eighth Floor

Concrete Operations

Typical Activities

AGS100560	FRP Elevated Deck - 08	9	0	03FEB11A	16FEB11A
AGS100580	FRP Columns - 08	5	0	09FEB11A	17FEB11A
AGS100680	Strip/Reshore - 08 to 09	10	0	23FEB11A	03MAR11A
AGS100540	Remove Reshores - 07 to 08	10	0	16MAR11A	21MAR11A
CN121840	FRP Concrete Pads/Curbs - 08	5	0	27APR11A	02MAY11A

Ninth Floor

Concrete Operations

Typical Activities

AGS100640	FRP Elevated Deck - 09	9	0	11FEB11A	28FEB11A
AGS100660	FRP Columns - 09	5	0	18FEB11A	02MAR11A
AGS100760	Strip/Reshore - 09 to 10	10	0	08MAR11A	17MAR11A
AGS100620	Remove Reshores - 08 to 09	10	0	18MAR11A	29MAR11A
CN121860	FRP Concrete Pads/Curbs - 09	5	0	27APR11A	02MAY11A

Tenth Floor					
Concrete Operations					
Typical Activities					
AGS100720	FRP Elevated Deck - 10	9	0	21FEB11A	11MAR11A
AGS100740	FRP Columns - 10	0	0	03MAR11A	15MAR11A
AGS100840	Strip/Reshore - 10 to 11	10	0	18MAR11A	29MAR11A
AGS100700	Remove Reshores - 09 to 10	10	0	30MAR11A	20APR11A
CN121880	FRP Concrete Pads/Curbs - 10	5	0	27APR11A	02MAY11A
Eleventh Floor					
Concrete Operations					
Typical Activities					
AGS100800	FRP Elevated Deck - 11	9	0	08MAR11A	24MAR11A
AGS100820	FRP Columns - 11	5	0	16MAR11A	25MAR11A
AGS100920	Strip/Reshore - 11 to 12	10	0	30MAR11A	15APR11A
AGS100780	Remove Reshores - 10 to 11	5	0	20APR11A	26APR11A
CN121920	FRP Concrete Pads/Curbs - 11	5	0	27APR11A	02MAY11A
Twelfth Floor					
Concrete Operations					
Typical Activities					
AGS100880	FRP Elevated Deck - 12	9	0	17MAR11A	08APR11A
AGS100900	FRP Columns - 12	5	0	28MAR11A	09APR11A
AGS101000	Strip/Reshore - 12 to R	10	0	15APR11A	25APR11A
AGS100860	Remove Reshores - 11 to 12	5	0	20APR11A	26MAY11A
CN121940	FRP Concrete Pads/Curbs - 12	5	0	27APR11A	02MAY11A
Roof					
Concrete Operations					
Typical Activities					
AGS100960	FRP Elevated Deck - R	12	0	29MAR11A	15APR11A
AGS100980	FRP Columns - R	8	0	13APR11A	14APR11A
AGS101020	FRP Elevated Deck - EMR	5	0	22APR11A	28APR11A
AGS101080	Strip/Reshore - R to PHR	10	0	26APR11A	03MAY11A
CN121960	FRP Concrete Pads/Curbs - PH	5	0	30APR11A	04MAY11A

Figure 55 Existing construction sequence for levels 1 through Main roof

As can be seen in Figure 55, the elevated slab for the roof was completed by April 15, 2011 where as for the steel system the slab on deck on the main roof level would be completed by March 28, 2011. As a result, the use of the steel system shortens the construction schedule by 18 days. RS Means 2012 was used to determine the duration for each activity required to complete the steel system construction. The detailed calculations for durations of the steel system schedule can be seen in Appendix E.

Site Logistics

The site logistics of concrete versus steel construction will vary, therefore a site logistics study was conducted to determine how the two materials will have to be managed differently on the same site. The 1000 Connecticut Avenue project incorporated the use of Ox Blue to track the progress of the project. Ox Blue is a web camera used to keep track and view the progress of the project on site. The use of the web camera was executed on the first day construction began, which was on October 19, 2009. For the site logistics study, images taken by the camera system were used to determine the site logistics of the existing system. Select images taken during the course of construction were used to help with the site logistics study. Select images used for the study can be seen in Figures 56 through 61.

Figure 56 construction site before excavation (October 2009)

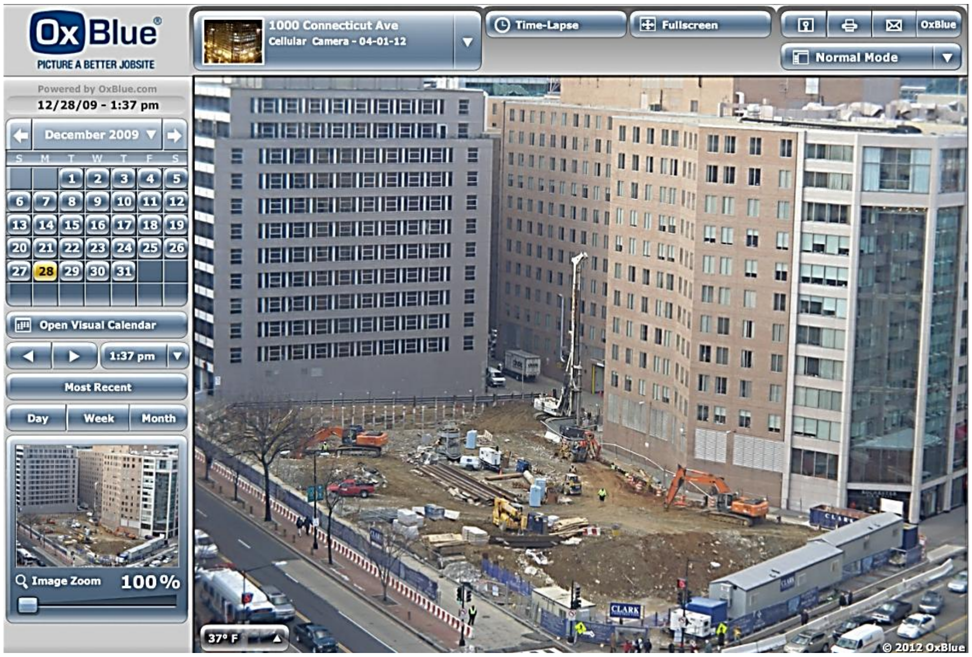


Figure 57 Beginning stages of excavation (December 2009)

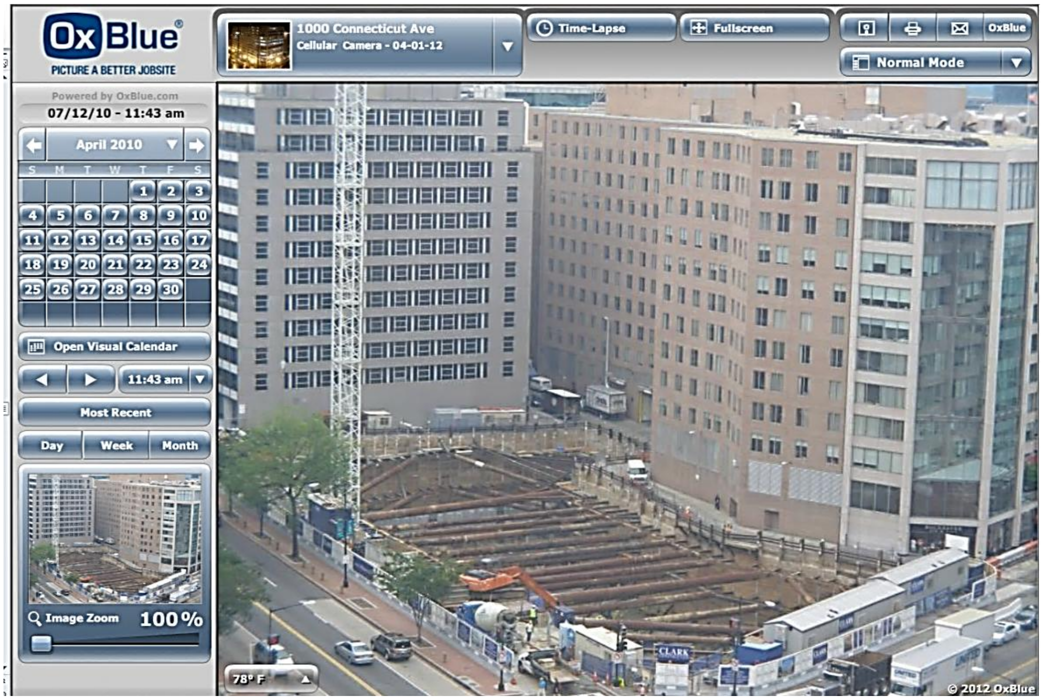


Figure 58 Construction site after excavation (April 2010)

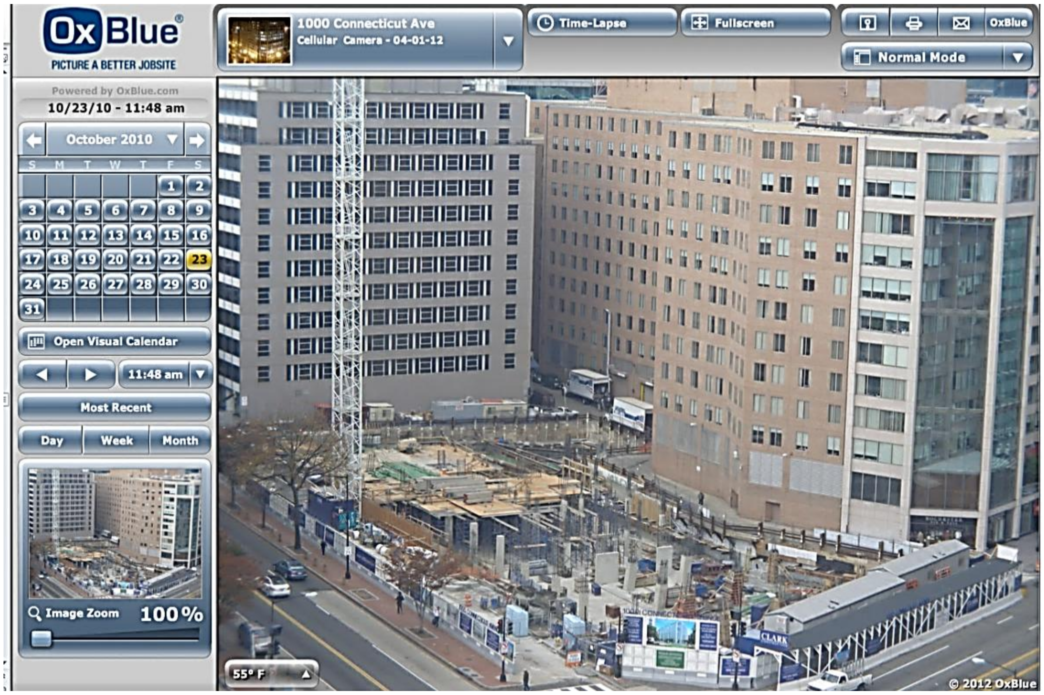


Figure 59 erection of the subgrade four - level underground parking garage (October 2010)

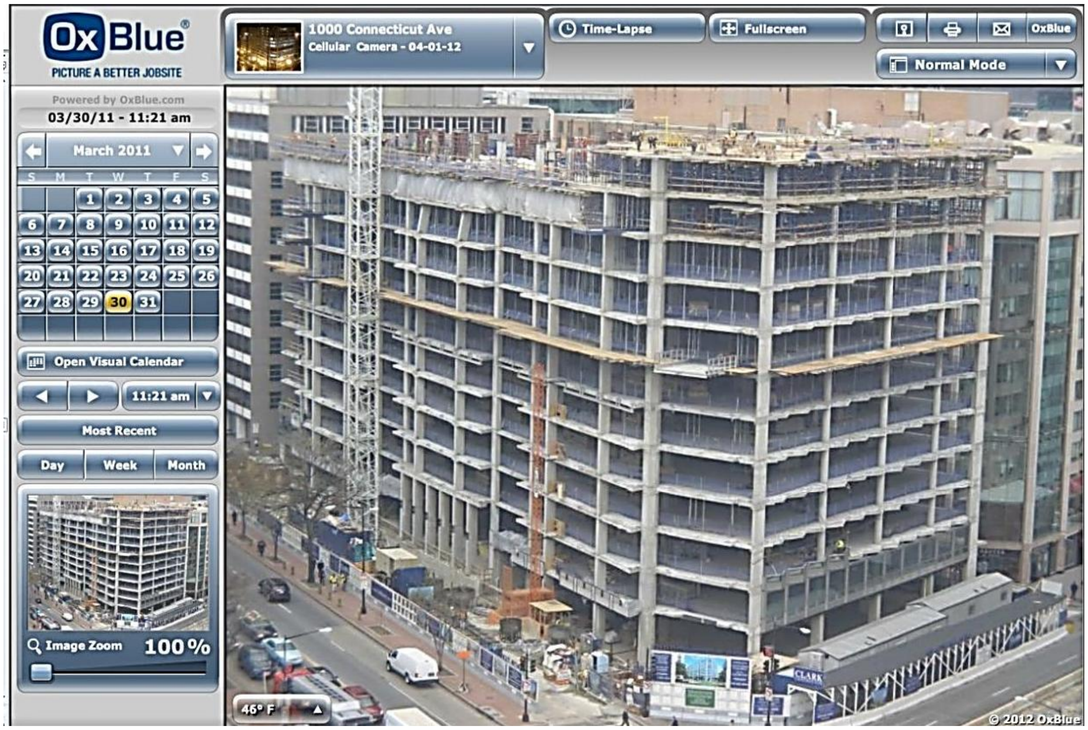


Figure 60 Erection of the twelfth story (main roof) (March 2011)

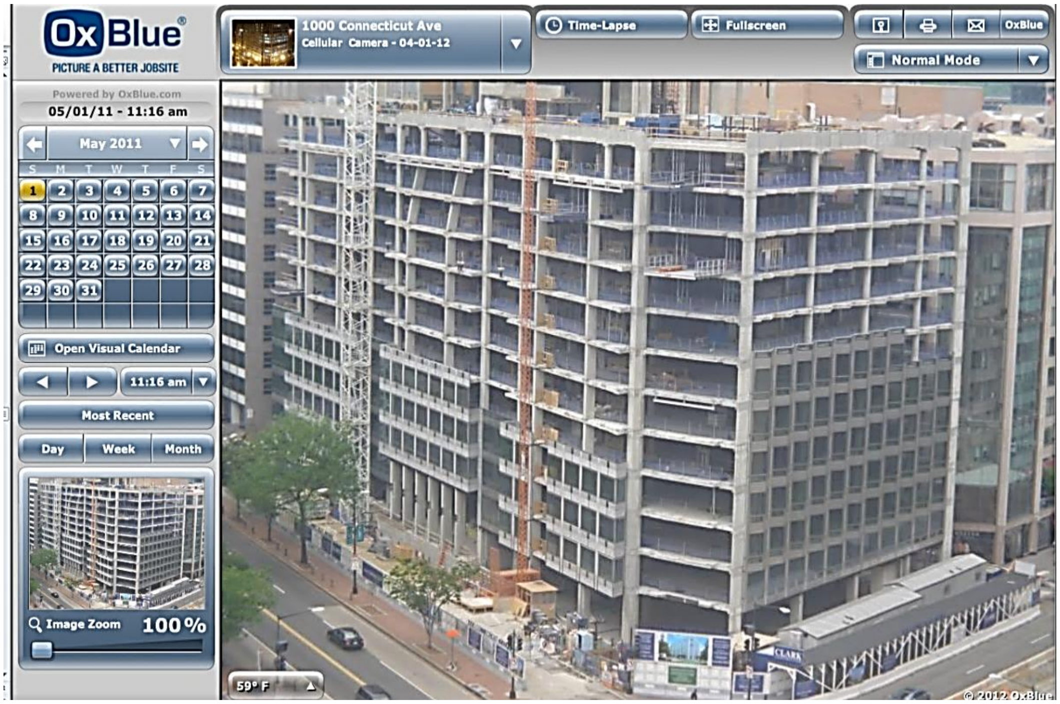


Figure 61 Concrete tops out in May 2011 and the early stages of glass curtain wall installation

Based on Figures 56 through 61, the site appears to have been managed the same throughout the structural system's construction. An animated depiction of the site logistics can be seen in Figure 62.

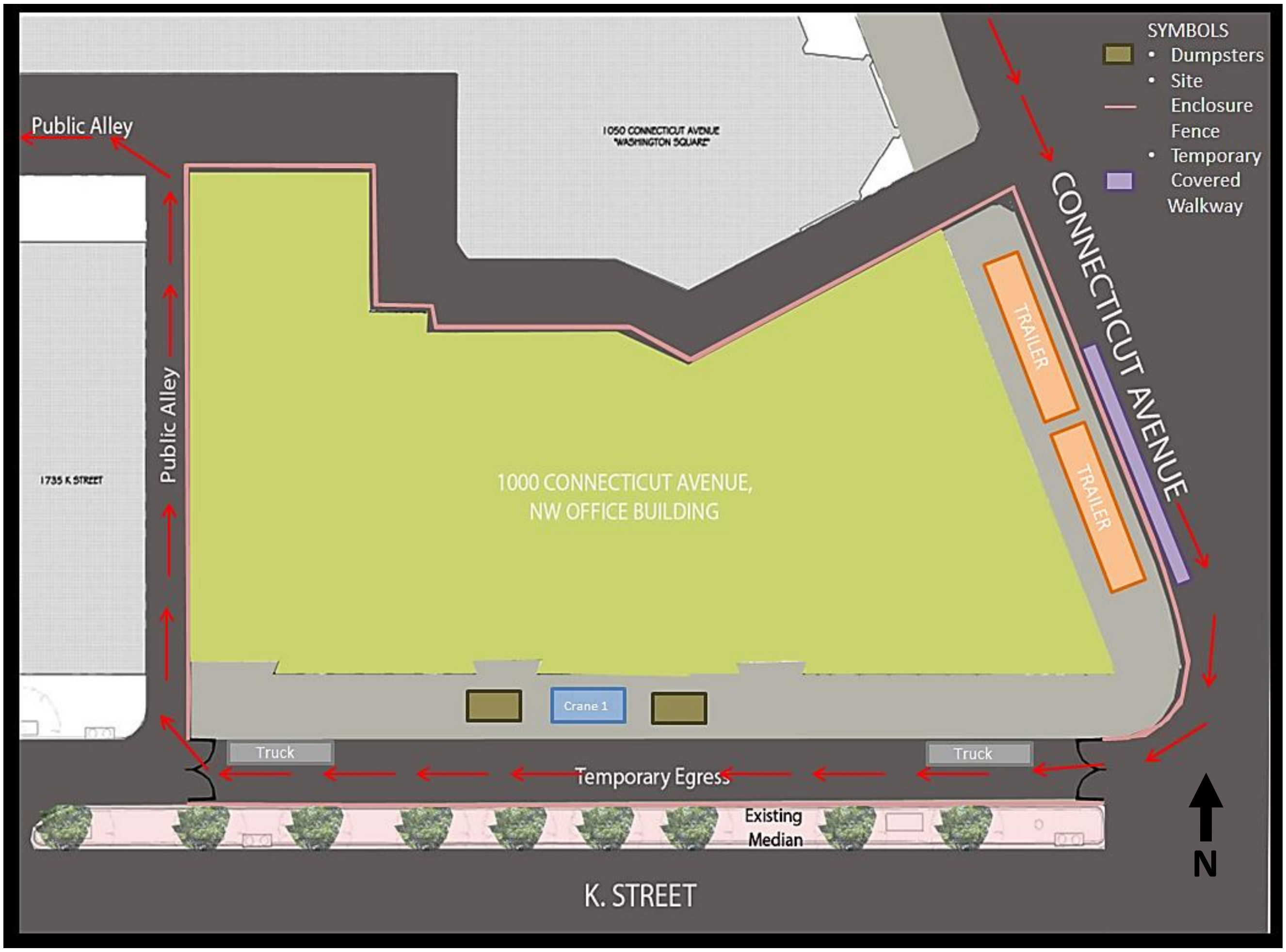


Figure 62 Existing concrete system's site logistics

As can be seen in Figures 56 through 61, the management of the site appeared to stay the same throughout the different phases of the project in terms of equipment location and vehicular egress. The crane shown in Figures 56 through 61 is used to lift the form work and is used to place the concrete. The existing site appears to have used the crane and bucket placement method to pour and place the concrete. The private alleys are shut down during construction and are used as egress for the trucks. As can be seen in Figure 58 the trucks enter the site by traveling South on Connecticut Avenue and using the service road along K Street and the alleys as egress to gain access to the sight. The trailers are located along Connecticut Avenue which is a good viewing location for the project managers and engineers to track the progress of the project.

After analyzing the site logistics for the existing concrete structure, a study was completed to determine how the site will have to be managed if steel were used. The proposed site logistics for the steel construction can be seen in Figure 63.

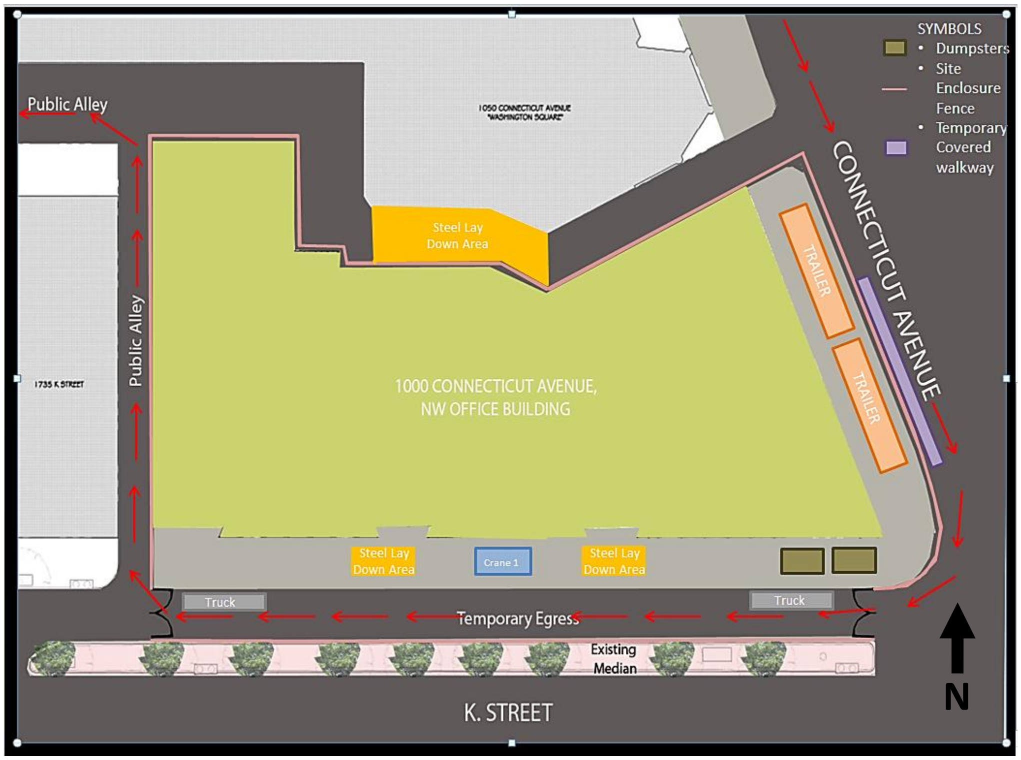


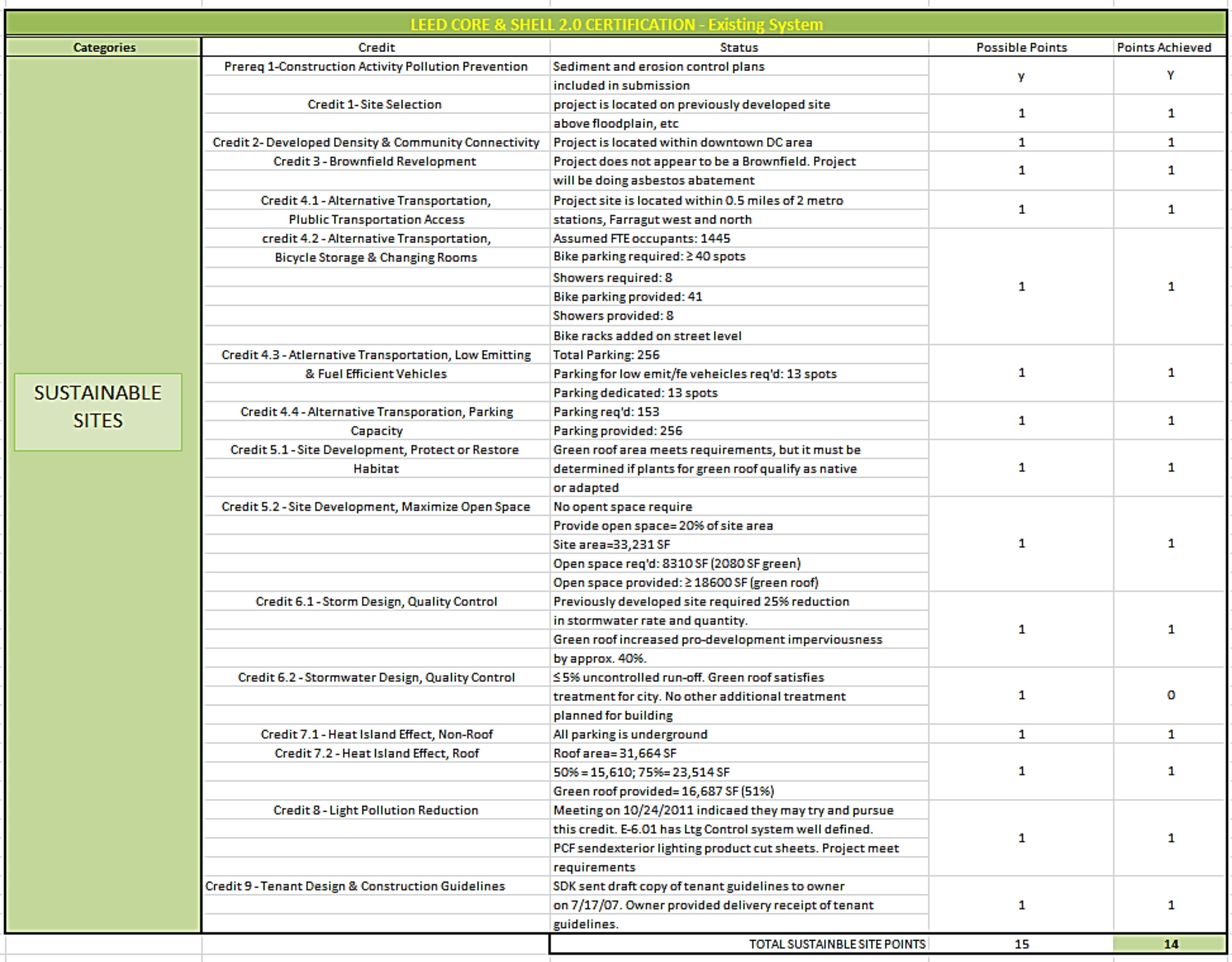
Figure 63 proposed site logistics plan for steel construction

It's assumed the steel members will be labeled before arriving to the construction site. As a result, the members can be placed directly in the steel lay down areas upon arrival to the site. The same tall crane used for the concrete construction will also be used for erecting the steel members. Concrete will be placed by using the crane and bucket method, the same method used in the concrete construction. The crane and bucket method takes longer to execute than using a concrete pump, but it's still effective and less expensive. In addition, with the use of the same crane and concrete placement method there will be no additional cost accumulated when erecting the steel system. The lay down areas for the steel will be located adjacent to the crane and near the south facing wall for easy access. The same egress paths used for the concrete construction will also be used for the steel system construction.

After the analysis, it was shown that the site will be management very similarly to that of the concrete construction site, with the difference being the requirement of lay down areas for the steel members. The same equipment will be used which will avoid any additional cost. The Crane and bucket method will be used to pour the concrete. The same crane used for the construction of the existing system can be used for the erection of the steel system.

LEED Certification

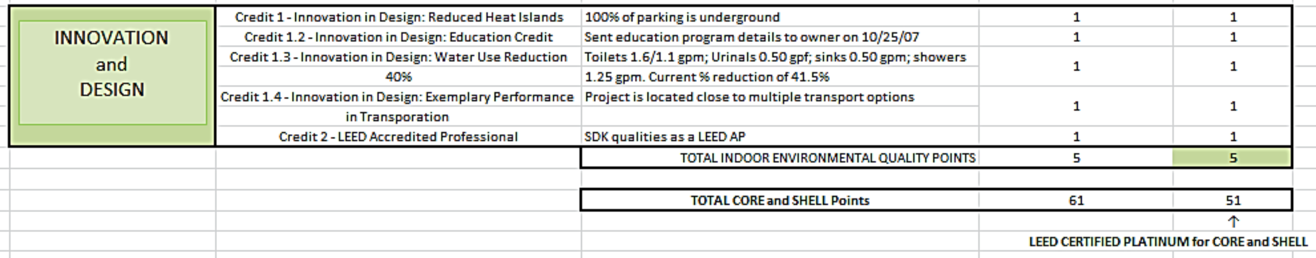
After changing the structural system to steel, it was shown that the certification for the shell and core will remain platinum certified. The LEED analysis of the new and existing systems was based on LEED 2.0 for New Construction and Major Renovations. The use of the new system will increase the rating from 51 points to 52 points. Under the Material and Resources category, with the use of steel the building will be able to use at least 1% of reused steel for the structural members and metal decks, in which the new system will be able to obtain 1 point for credit 3 (Materials and Reuse, 1%). In addition it is assumed that the building re-located to Arlington, VA will be located in a previously developed site (Credit 1 under "Sustainable Sites") and the building will be located in a developed community. Since the building will be re-located to downtown Arlington, the point for credit 2 under "Sustainable Sites" will be achieved. The analysis of the existing system's LEED certification can be seen in Table 42.



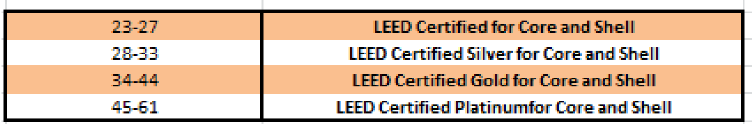
LEED CORE & SHELL 2.0 CERTIFICATION - Existing System

Categories	Credit	Status	Possible Points	Points Achieved
SUSTAINABLE SITES	Prereq 1-Construction Activity Pollution Prevention	Sediment and erosion control plans included in submission	y	Y
	Credit 1- Site Selection	project is located on previously developed site above floodplain, etc	1	1
	Credit 2- Developed Density & Community Connectivity	Project is located within downtown DC area	1	1
	Credit 3 - Brownfield Revelopment	Project does not appear to be a Brownfield. Project will be doing asbestos abatement	1	1
	Credit 4.1 - Alternative Transportation, Plublic Transportation Access	Project site is located within 0.5 miles of 2 metro stations, Farragut west and north	1	1
	credit 4.2 - Alternative Transportation, Bicycle Storage & Changing Rooms	Assumed FTE occupants: 1445 Bike parking required: ≥ 40 spots Showers required: 8 Bike parking provided: 41 Showers provided: 8 Bike racks added on street level	1	1
	Credit 4.3 - Atlernative Transportation, Low Emitting & Fuel Efficient Vehicles	Total Parking: 256 Parking for low emit/fe veheicles req'd: 13 spots Parking dedicated: 13 spots	1	1
	Credit 4.4 - Alternative Transporation, Parking Capacity	Parking req'd: 153 Parking provided: 256	1	1
	Credit 5.1 - Site Development, Protect or Restore Habitat	Green roof area meets requirements, but it must be determined if plants for green roof qualify as native or adapted	1	1
	Credit 5.2 - Site Development, Maximize Open Space	No opent space require Provide open space= 20% of site area Site area=33,231 SF Open space req'd: 8310 SF (2080 SF green) Open space provided: ≥ 18600 SF (green roof)	1	1
	Credit 6.1 - Storm Design, Quality Control	Previously developed site required 25% reduction in stormwater rate and quantity. Green roof increased pro-development imperviousness by approx. 40%.	1	1
	Credit 6.2 - Stormwater Design, Quality Control	≤ 5% uncontrolled run-off. Green roof satisfies treatment for city. No other additional treatment planned for building	1	0
	Credit 7.1 - Heat Island Effect, Non-Roof	All parking is underground	1	1
	Credit 7.2 - Heat Island Effect, Roof	Roof area= 31,664 SF 50% = 15,610; 75%= 23,514 SF Green roof provided= 16,687 SF (51%)	1	1
	Credit 8 - Light Pollution Reduction	Meeting on 10/24/2011 indicaed they may try and pursue this credit. E-6.01 has Ltg Control system well defined. PCF sendexterior lighting product cut sheets. Project meet requirements	1	1
	Credit 9 - Tenant Design & Construction Guidelines	SDK sent draft copy of tenant guidelines to owner on 7/17/07. Owner provided delivery receipt of tenant guidelines.	1	1
		TOTAL SUSTAINBLE SITE POINTS	15	14

WATER EFFICIENCY	Credit 1.1 - Water Efficient Landscaping, Reduce 50%	-	1	1
	Credit 1.2 - Water Efficent Landscaping, No Potable Water Use or No Irrigation	6/21/07 Team confirmed no permanent irrigation will be included in the project	1	1
	Credit 2 - Innovation Wastewater Technologies	No fixture performance in DD Set. MEP spec refers to the use of water saver type fixtures	1	1
	Credit 3.1 - Water Use Reduction, 20% Reduction	Toilets 1.6/1.1 gpm: Urinals 0.50 gpf; sinks 0.50 gpm; showers 1.25 gpm. Current % reduction at 41.5 %	1	1
	Credit 3.2 - Water Use Reduction, 30% Reduction	Toilets 1.6/1.1 gpm: Urinals 0.50 gpf; sinks 0.50 gpm; showers 1.25 gpm. Current % reduction at 41.5 %	1	1
		TOTAL WATER EFFICIENCY POINTS	5	5
ENERGY & ATMOSPHERE	Prereq - Fundamental Commissioning of the Building Energy Systems	SDK engaged as Cx agent. Addendum 1 has 230800 with full checklists; OK. No reference in 239000 or 239250 (BAS Sections) or 260100n to 230800 or 019100 in Add 1, Add 2, or Amd 1. (WDG indicated that references are not allowed and all DIV 1 spcs will be applicable).	Y	Y
	Prereq 2 - Minumun Energy Performance	EMO report confirms compliance.	Y	Y
	Prereq 3 - Fundamental Refrigerant Management	Drawings show use chillers to use R-134a	Y	Y
	Credit 1 - Optimize Energy Performance	CDC report shows 21.1% or 4 points. 6/3/10 - EMO indicates project will earn maxmimum 8 points.	8	8
	Credit 2 - On-Site Renewable Energy, 1%	No use of renewable energy shown in drawings	1	0
	Credit 3 - Enhanced Commissioning	Enhanced Cx not selected for implementation by owner. SDK Engineers downgraded point, no acceptance of enhanced cx and project is to late to include.	1	0
	Credit 4 - Enhanced Refrigerant Management	Calc made 10/9/07 with Chillers and Packaged ACUs.	1	1
	Credit 5.1 - Measurement & Verificsation, Base Building	Amd 1 provided additional requirements needed to meet M&V	1	1
	Credit 5.2 - Measurement & Verification, Tenant Sub-metering	Amd 1 provided additional requirements needed to meet tenant M&V	1	1
	Credit 6 - Green Power, 35%	SDK sent green power options/cost estimate to ownership 10/25/07	1	1
		TOTAL ENERGY & ATMOSPHERE POINTS	14	12
MATERIALS & RESOURCES	Prereq 1 - Storage & Collection of Recyclables	90 sf of recycling shown in loading dock area. Distributed recycling space is shown throughout the building and enforced in tenant guidelines.	Y	Y
	Credit 1.1 - Building Reuse, Maintain 25%, 50%, 75% of Existing Walls, Floors & Roof	Building will not be re-using existing shell	3	0
	Credit 2.1 - Construction Waste Management, Divert 50%, 75% From Disposal	CWM spefication included in permit set. SDK received demo waste management plan 12/31/07	2	2
	Credit 3 - Materials Reuse, 1%	-	1	0
	Credit 4.1 - Recycled Content, Speify 10%, 20% (post-consumer + pre-consumer)	Construction document specification sections support credit	2	2
	Credit 5.1 - Regional Materials, 10%, 20% Extracted and Manufactured Regionally	Construction document specification sections support credit	2	2
	Cresdit 6 - Certified Wood	Wood is to be used for finishes and wood doors	1	1
		TOTAL MATERIALS & RESOURCES POINTS	11	7
INDOOR ENVORONMENTAL QUALITY	Prereq 1 - Minumum IAQ Performance	Ventilation calcs indicate all araea exceed 62.1-04. Addendum 1 brough Fitness OA cfm up to 30% above 62.1-04	Y	Y
	Prereq 2 - Environ. Tobacco Smoke Control	No smoking allowed within the building according to DC code	Y	Y
	Credit 1 - Outdoor Air Delivery Monitoring	Add. 1 now has OA Flow monitoring AI points for OA Valves for all AHUs and all ACUs serving occupied spaces.	1	1
	Credit 2 - Increased Ventilation	Ventilation calcs and Addendum 1 for Fitness OA now show all mechanically ventilated paces are at least 30% higher than 62.1-04	1	1
	Credit 3 - Constuction IAQ Management, During Construction	Clark submitted Construction IAQ Management Plan.	1	1
	Credit 4.1 - Low-Emitting Materials, Adhesives & Sealants	No mention of Low VOC adhesive and sealants in permit spec	3	2
	Credit 4.2 - Low-Emitting Materials, Paints	Low VOC paints enforced in bid and addendum set		
	Credit 4.3 - Low-Emitting Materials, Carpet	Carpet specification included. CRI green label plus enforced in Addendum 1		
	Credit 4.4 - Low-Emitting Materials, Composite Wood & Agrifiber Products	Composite wood requirements included in specifications		
	Credit 5 - Indoor Chemical & Pollutant Source Control	10/24/11 meeting w/ Owner indicated credit is not being pursued. Addendum 1 shows final filters revied to MERV-11	1	0
	Credit 6 - Controllability of Systems, Thermal Comfort	Based on HVAC design, thermostats controlling VAV boxes will be included 1 per 1.8 people on perimeter and 1 per 4 FTE on interior	1	1
	Credit 7 - Thermal Comfort, Design	MEP outline spec gives design conditions for HVAC system	1	1
	Credit 8.1 - Daylight & Views, Daylight 75%	Tvis for glass is 0.61. Intials daylight calculation does not meet 75% area for 2% day lighting	1	0
	Credit 8.2 - Daylight & Views, Views for 90% of spaces	Documentation is complete and ready for submission in LEED outline.	1	1
		TOTAL INDOOR ENVIRONMENTAL QUALITY POINTS	11	8



INNOVATION and DESIGN	Credit 1 - Innovation in Design: Reduced Heat Islands	100% of parking is underground	1	1
	Credit 1.2 - Innovation in Design: Education Credit	Sent education program details to owner on 10/25/07	1	1
	Credit 1.3 - Innovation in Design: Water Use Reduction 40%	Toilets 1.6/1.1 gpm; Urinals 0.50 gpf; sinks 0.50 gpm; showers 1.25 gpm. Current % reduction of 41.5%	1	1
	Credit 1.4 - Innovation in Design: Exemplary Performance in Transporation	Project is located close to multiple transport options	1	1
	Credit 2 - LEED Accredited Professional	SDK qualities as a LEED AP	1	1
		TOTAL INDOOR ENVIRONMENTAL QUALITY POINTS	5	5
		TOTAL CORE and SHELL Points	61	51
				↑
			LEED CERTIFIED PLATINUM for CORE and SHELL	



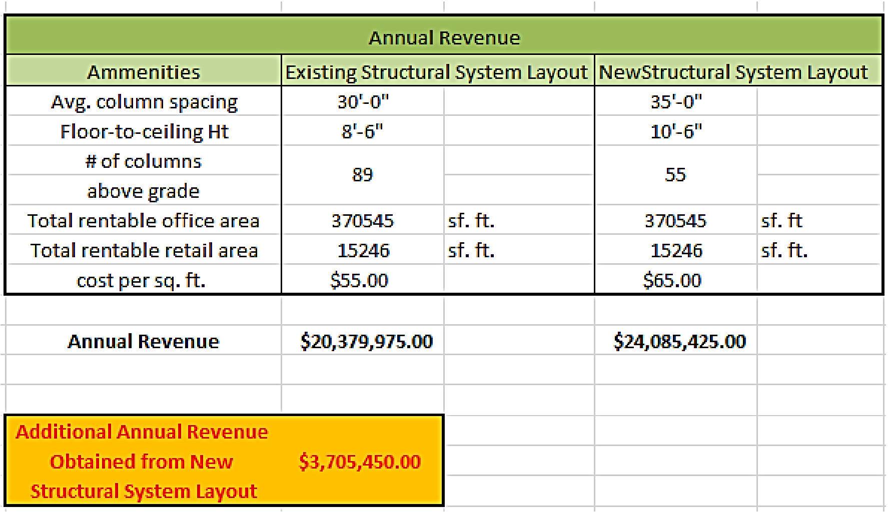
23-27	LEED Certified for Core and Shell
28-33	LEED Certified Silver for Core and Shell
34-44	LEED Certified Gold for Core and Shell
45-61	LEED Certified Platinumfor Core and Shell

Table 42 Existing LEED certification check

Annual Revenue

After increasing the floor-to-floor height to 15’-0” and creating wider bays increased the rental value of the space. The floor layout is more open and due to fewer obstructions due to columns and with an increase floor-to-ceiling height of 10’-6” increases the openness of the space. A combination of wider bays and higher-floor-to-ceiling heights increases the rental value of the space, therefore a revenue study was performed to determine the amount of annual revenue that can be obtained with the use of the new structural system. The analysis was conducted by contacting a realtor representative in Washington D.C. to obtain information on the current asking rental price per square footage for the space. A realtor representative at Summit Commercial Real Estate Agency located in Washington, D.C. disclosed that the asking price for 1000 Connecticut Avenue is $55.00 per square foot. After asking the representative how much more rent can be charged with the additional amenities of wider bays and higher floor-to-ceiling heights, the representative disclosed that an additional $10-$20 can be charged per square foot. Therefore the asking price can increase up to $65-$75 per square foot.

For the analysis, it was assumed that the new building system will be located in a business district in Arlington, VA and that the asking price for the existing building re-located to Arlington, VA will be $55 per square foot. It was also assumed that the rent would increase to $65 per square foot if the new steel system were used in place of the concrete structure. The results of the annual revenue obtained with the use of the new structural system versus the revenue obtained from the use of the existing system can be found in Table 43.



Annual Revenue				
Ammenities	Existing Structural System Layout		NewStructural System Layout	
Avg. column spacing	30'-0"		35'-0"	
Floor-to-ceiling Ht	8'-6"		10'-6"	
# of columns above grade	89		55	
Total rentable office area	370545	sf. ft.	370545	sf. ft
Total rentable retail area	15246	sf. ft.	15246	sf. ft.
cost per sq. ft.	$55.00		$65.00	
Annual Revenue	**$20,379,975.00**		**$24,085,425.00**	
Additional Annual Revenue Obtained from New Structural System Layout	**$3,705,450.00**			

Table 43 Annual revenue comparison between new steel system and existing concrete system

As can be seen in Table 43, the annual revenue obtained with the use of the steel structural system layout will increase the annual revenue an additional $3,705,450 per year.

Acoustics and Lighting Breadths

After designing the new steel structural system, acoustics and lighting breadths were conducted for the office spaces supported by the new system. The acoustics breadth involved determining the sound treatments required for a typical office space housed in the new structural system. Based on the sound treatments in the space, the sound transmission class (STC) and noise reduction (NR) values were determined for a typical office space. In addition, since the new structural system was designed for higher floor-to-ceiling heights, lighting illuminance applied to the work plane surfaces were affected. As a result, a lighting breadth will be conducted by designing the lighting system for a typical office space using the existing floor-to-ceiling height of 8'-6" and checking to determine if the same lighting system can be used for the space with a new floor-to-ceiling height of 10'-6".

Acoustics Breadth

After changing the structural system from concrete to steel, the amount of sound transmitted between the space increases. As a result, an acoustical study was performed to determine the type of wall partitions, finish floor materials, and ceiling materials will be needed to attenuate the sound transmitted between the office spaces. As a can be seen in Figure 64 1000 Connecticut Avenue will be comprised of a series of office spaces located around the perimeter of the building. The private offices will be occupied by attorneys.

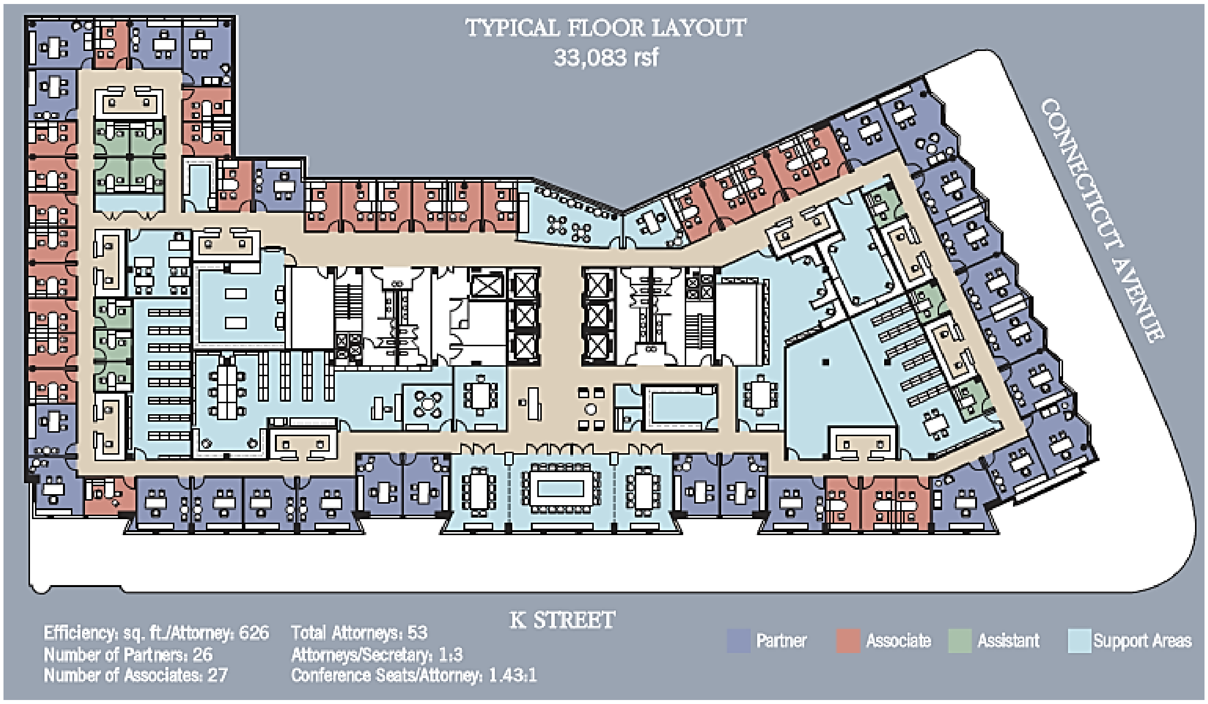
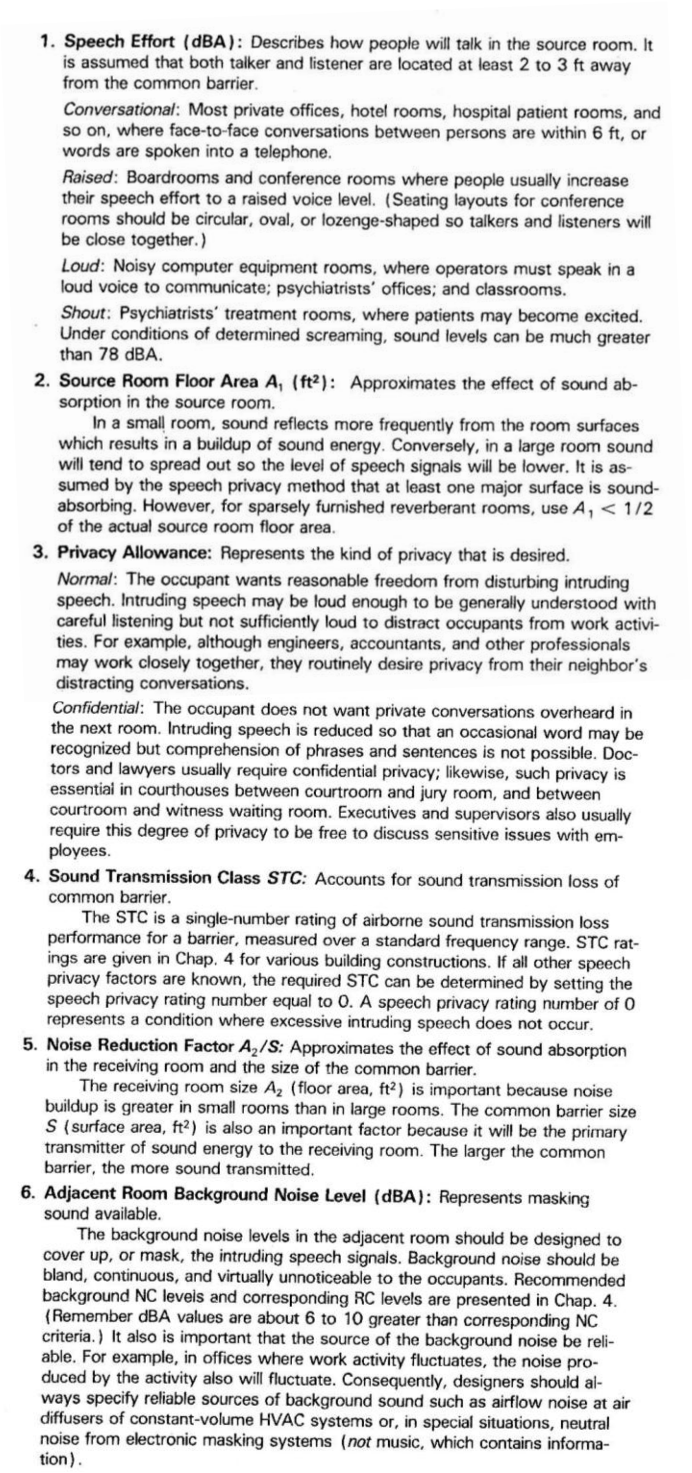


Figure 64 Typical floor plan layout

With the private office spaces being occupied by attorneys, speech privacy will be very important and must be considered when designing the office spaces. For analyzing the office space, the speech privacy analysis method outlined in Chapter 6 of "Architectural Acoustics" by David M. Egan will be used. The speech analysis method is a step-by-step procedure broken down into 6 steps that are used to determine the minimum STC rating for common barriers between adjacent spaces in order to achieve satisfactory privacy. The speech privacy analysis procedure can be seen listed and described in Figure 65.



1. **Speech Effort (dBA):** Describes how people will talk in the source room. It is assumed that both talker and listener are located at least 2 to 3 ft away from the common barrier.

   Conversational: Most private offices, hotel rooms, hospital patient rooms, and so on, where face-to-face conversations between persons are within 6 ft, or words are spoken into a telephone.

   Raised: Boardrooms and conference rooms where people usually increase their speech effort to a raised voice level. (Seating layouts for conference rooms should be circular, oval, or lozenge-shaped so talkers and listeners will be close together.)

   Loud: Noisy computer equipment rooms, where operators must speak in a loud voice to communicate; psychiatrists' offices; and classrooms.

   Shout: Psychiatrists' treatment rooms, where patients may become excited. Under conditions of determined screaming, sound levels can be much greater than 78 dBA.

2. **Source Room Floor Area A_1 (ft^2):** Approximates the effect of sound absorption in the source room.

   In a small room, sound reflects more frequently from the room surfaces which results in a buildup of sound energy. Conversely, in a large room sound will tend to spread out so the level of speech signals will be lower. It is assumed by the speech privacy method that at least one major surface is sound-absorbing. However, for sparsely furnished reverberant rooms, use $A_1 < 1/2$ of the actual source room floor area.

3. **Privacy Allowance:** Represents the kind of privacy that is desired.

   Normal: The occupant wants reasonable freedom from disturbing intruding speech. Intruding speech may be loud enough to be generally understood with careful listening but not sufficiently loud to distract occupants from work activities. For example, although engineers, accountants, and other professionals may work closely together, they routinely desire privacy from their neighbor's distracting conversations.

   Confidential: The occupant does not want private conversations overheard in the next room. Intruding speech is reduced so that an occasional word may be recognized but comprehension of phrases and sentences is not possible. Doctors and lawyers usually require confidential privacy; likewise, such privacy is essential in courthouses between courtroom and jury room, and between courtroom and witness waiting room. Executives and supervisors also usually require this degree of privacy to be free to discuss sensitive issues with employees.

4. **Sound Transmission Class *STC*:** Accounts for sound transmission loss of common barrier.

   The STC is a single-number rating of airborne sound transmission loss performance for a barrier, measured over a standard frequency range. STC ratings are given in Chap. 4 for various building constructions. If all other speech privacy factors are known, the required STC can be determined by setting the speech privacy rating number equal to 0. A speech privacy rating number of 0 represents a condition where excessive intruding speech does not occur.

5. **Noise Reduction Factor A_2/S:** Approximates the effect of sound absorption in the receiving room and the size of the common barrier.

   The receiving room size A_2 (floor area, ft^2) is important because noise buildup is greater in small rooms than in large rooms. The common barrier size S (surface area, ft^2) is also an important factor because it will be the primary transmitter of sound energy to the receiving room. The larger the common barrier, the more sound transmitted.

6. **Adjacent Room Background Noise Level (dBA):** Represents masking sound available.

   The background noise levels in the adjacent room should be designed to cover up, or mask, the intruding speech signals. Background noise should be bland, continuous, and virtually unnoticeable to the occupants. Recommended background NC levels and corresponding RC levels are presented in Chap. 4. (Remember dBA values are about 6 to 10 greater than corresponding NC criteria.) It also is important that the source of the background noise be reliable. For example, in offices where work activity fluctuates, the noise produced by the activity also will fluctuate. Consequently, designers should always specify reliable sources of background sound such as airflow noise at air diffusers of constant-volume HVAC systems or, in special situations, neutral noise from electronic masking systems (*not* music, which contains information).

Figure 65 Speech privacy analysis step-by-step procedure from "Architectural Acoustics" by M. David Egan

For the acoustical study, the common wall barrier between a conference room and private office was evaluated.

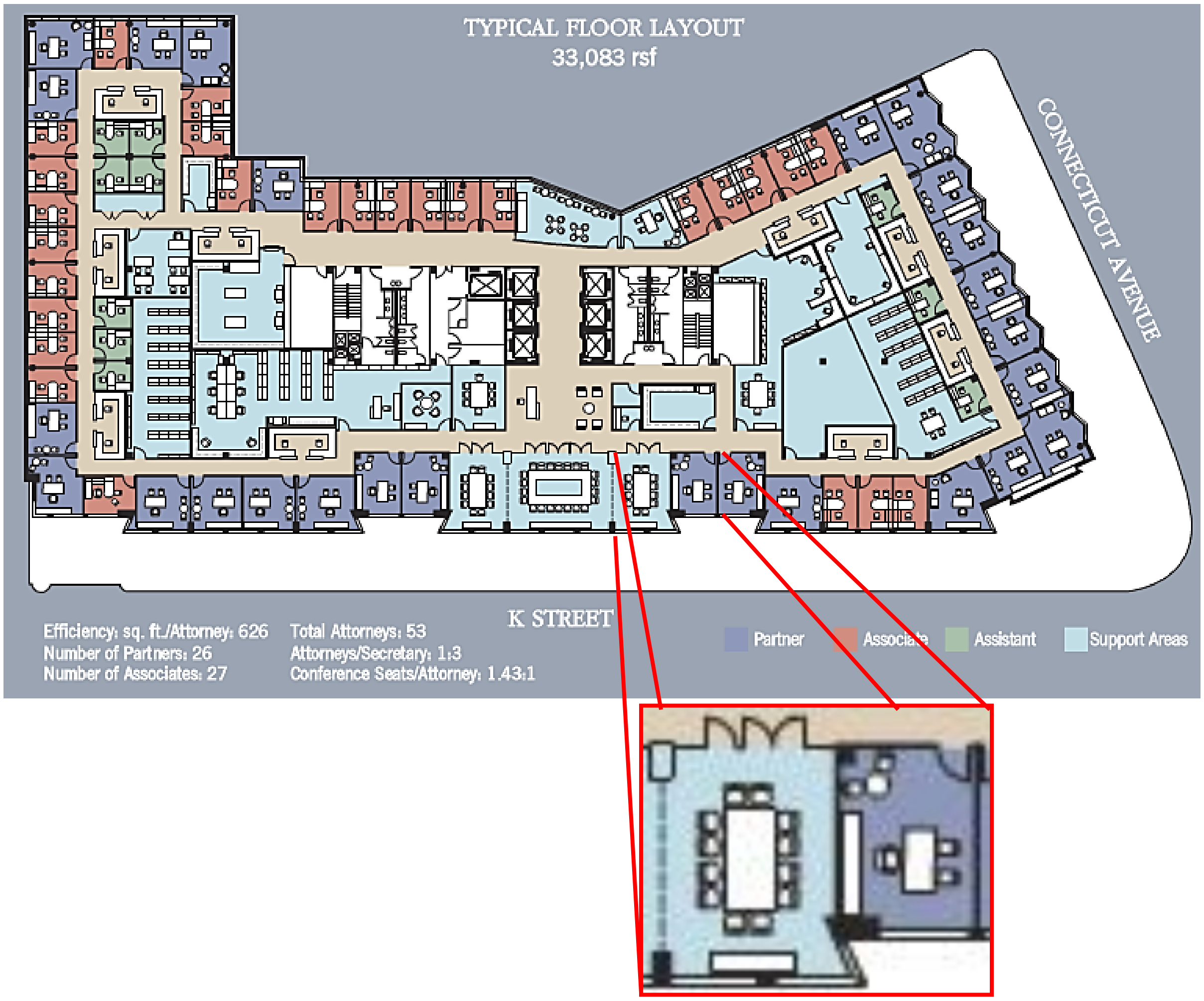


Figure 66 Plan of an attorney's private office (right) and adjacent conference room (left)

The dimensions for the two spaces used for analysis can be seen in Figure 67.

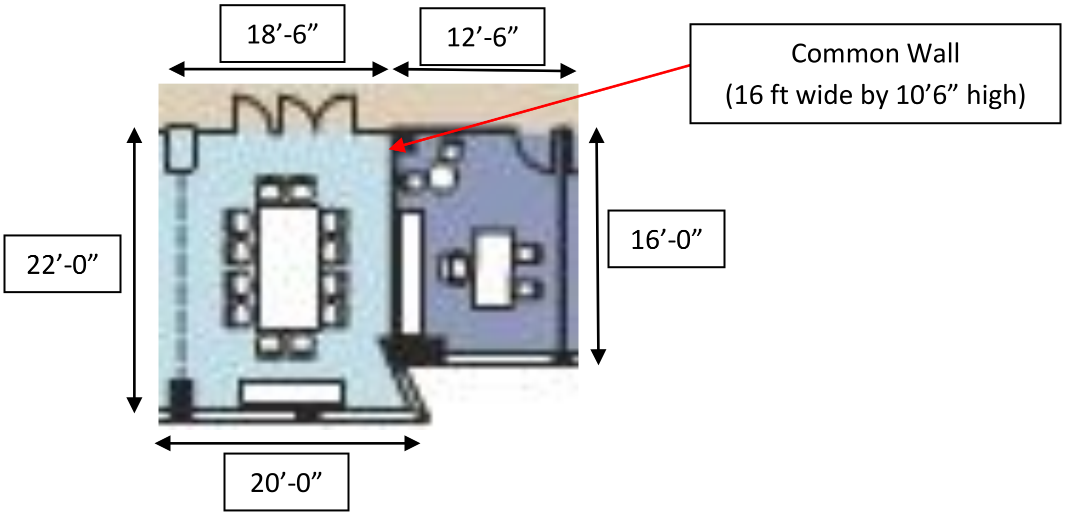


Figure 67 Private office room and conference room dimensions

According to the existing partition schedule, one of the partitions used as a common barrier between the enclosed spaces can been seen in Figure 68. For analysis, this partition type will be used as a common wall barrier between the office spaces housed in the new structural system.

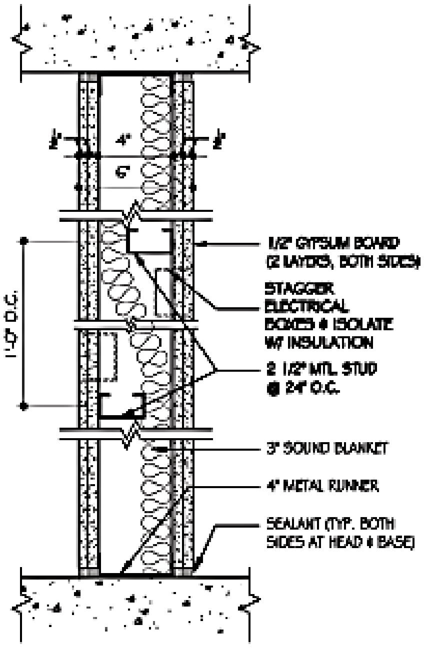
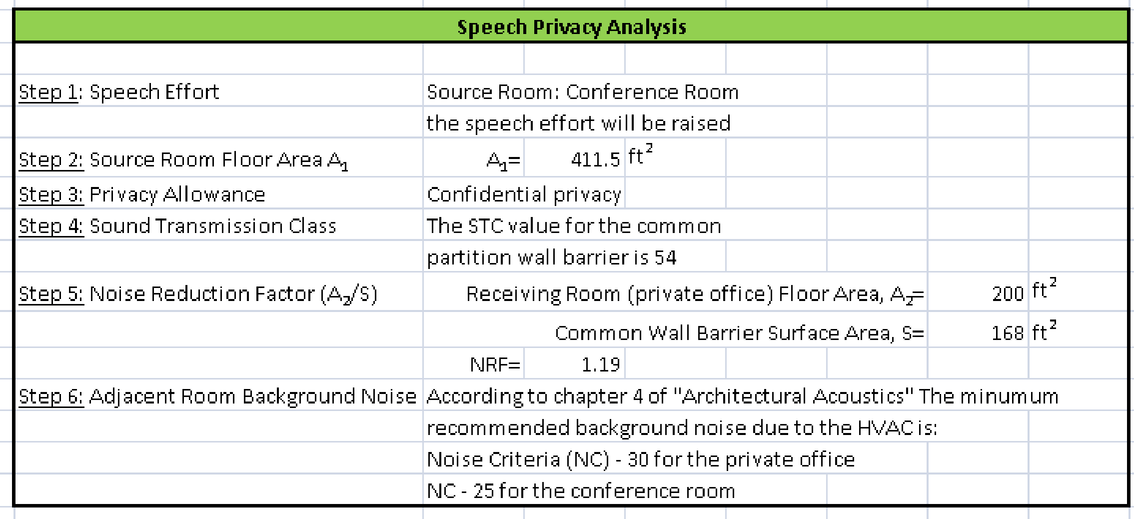


Figure 68 Common partition wall barrier between the private offices and conference rooms with an STC rating of 54

Image obtained from the existing partition wall schedule sheet A1.50

The above partition wall barrier was used to determine if it provides satisfactory privacy between the two spaces chosen for analysis.

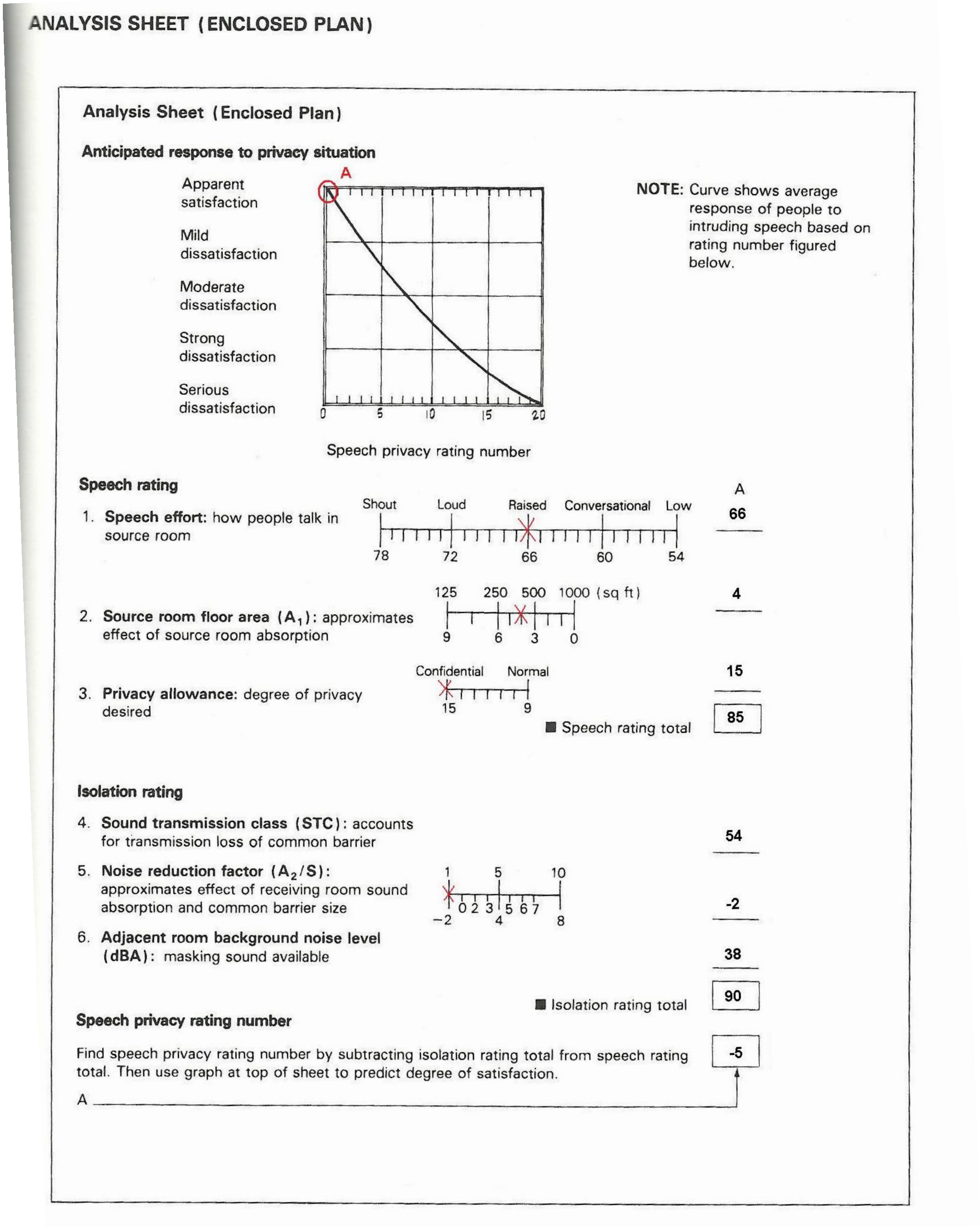
To begin the analysis, it was decided that both enclosed spaces will have carpeted floors and sound-absorbing acoustical ceilings. With the spaces being occupied by attorneys, it was assumed that both spaces will be used for confidential work. The step-by-step speech privacy analysis can be seen in Table 44 and Figure 70.



Speech Privacy Analysis

Step 1: Speech Effort	Source Room: Conference Room		
	the speech effort will be raised		
Step 2: Source Room Floor Area A_1	A_1=	411.5	ft^2
Step 3: Privacy Allowance	Confidential privacy		
Step 4: Sound Transmission Class	The STC value for the common		
	partition wall barrier is 54		
Step 5: Noise Reduction Factor (A_2/S)	Receiving Room (private office) Floor Area, A_2=	200	ft^2
	Common Wall Barrier Surface Area, S=	168	ft^2
	NRF=	1.19	
Step 6: Adjacent Room Background Noise	According to chapter 4 of "Architectural Acoustics" The minumum		
	recommended background noise due to the HVAC is:		
	Noise Criteria (NC) - 30 for the private office		
	NC - 25 for the conference room		

Table 44 Summary of speech privacy analysis results



ANALYSIS SHEET (ENCLOSED PLAN)

Analysis Sheet (Enclosed Plan)

Anticipated response to privacy situation

Apparent satisfaction
Mild dissatisfaction
Moderate dissatisfaction
Strong dissatisfaction
Serious dissatisfaction

Speech privacy rating number

NOTE: Curve shows average response of people to intruding speech based on rating number figured below.

Speech rating

1. **Speech effort:** how people talk in source room — Shout 78, Loud 72, Raised 66, Conversational 60, Low 54 — A 66
2. **Source room floor area (A$_1$):** approximates effect of source room absorption — 125 9, 250 6, 500 3, 1000 (sq ft) 0 — 4
3. **Privacy allowance:** degree of privacy desired — Confidential 15, Normal 9 — 15

■ Speech rating total 85

Isolation rating

4. **Sound transmission class (STC):** accounts for transmission loss of common barrier — 54
5. **Noise reduction factor (A$_2$/S):** approximates effect of receiving room sound absorption and common barrier size — 1 5 10; −2 0 2 3 4 5 6 7 8 — -2
6. **Adjacent room background noise level (dBA):** masking sound available — 38

■ Isolation rating total 90

Speech privacy rating number

Find speech privacy rating number by subtracting isolation rating total from speech rating total. Then use graph at top of sheet to predict degree of satisfaction.

A -5

Figure 70 Analysis sheet showing minimum required STC for the wall

As shown in Figure 70, the speech privacy analysis resulted in a speech privacy rating of -5. This shows that the STC-54 rated 8” partition wall with 2-layers of ½” thick gypsum wall board on both sides, staggered electrical boxes isolated with insulation, and 2 ½” metal studs spaced 24” o.c. and is very adequate for providing speech privacy for the offices housed in the new steel structural system.

Lighting Breadth

Increasing the floor-to-ceiling height from 8’-6” in the existing structure to 10’-6” in the new structural system caused the distance to the work plane to increase. Assuming the light fixtures are suspended 1.5 ft. from the ceiling and the work plane is 2.5 ft from the floor, the work plane distance will increase from 4.5 ft. to 6.5 ft in the new system. As a result, the lighting system used in the existing system may not work in the new system with higher floor-to-ceiling heights. For the lighting breadth, the lighting system was designed for a typical office space using the original floor to ceiling height of 8’-6”. After changing the floor-to-ceiling height to 10’-6”, the same lighting system was checked to determine if it could be used with the new work plane distance. The space chosen for analysis can be seen in Figure 71.

Figure 71 Typical office with existing lighting system

To begin the design, the important tasks that occur in the space had to be determined. It was found that the tasks that will occur in the private office space consist of reading, writing, and computer work. Next, based on the tasks that occur in the space, the target illuminance for the office was found to be 30 foot-candles, which was obtained from the IESNA Handbook. The light distribution must be within ± 10 percent of the target illuminance. Therefore, the illuminance of the light distribution must range between 27-33 foot-candles to be acceptable.

The lighting fixture was selected using Delta Light and a (2) 28 W T5 lamp was chosen using Sylvania's lamp and ballast catalog, which can be found in Appendix F. The light fixture chosen has 87.3% efficiency, which consists of 27.4% of up light and 60% of down light. The light fixture can be seen in Figure72.

Figure 72 Lighting fixture chosen for the typical office space

After assuming the surface reflectances and determining the total light loss factors, AGI was used to determine both the layout and number of luminaires needed to meet the 30 foot-candle target illuminance for the given space, which can be seen in Figures 73. For design simplicity, the triangular shape of the curtain wall was neglected and was assumed to be straight.

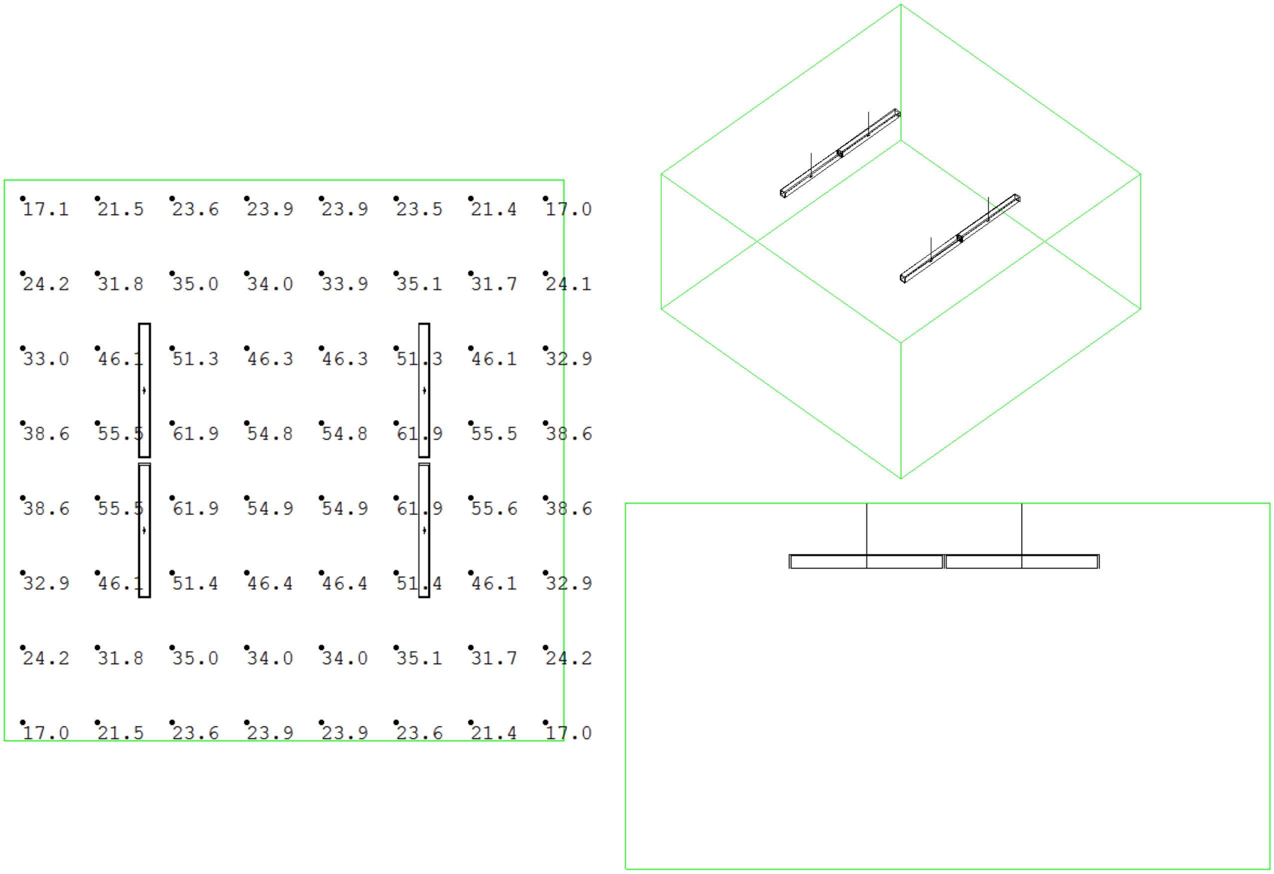


Figure 73 office plan with luminaire layout and illuminance values

The above layout results in a 37.4 foot-candle illuminance which meets the space's target illuminance. A rendering of the space with the new layout can be seen in Figure 74 and the office space's thermograph with the new lighting system can be seen in Figure 75.

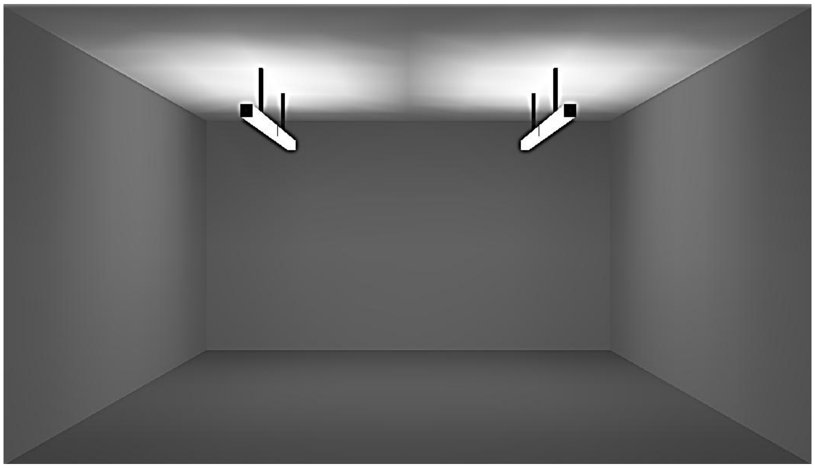


Figure 74 Rendering of the office space with 8’-6” floor-to-ceiling height

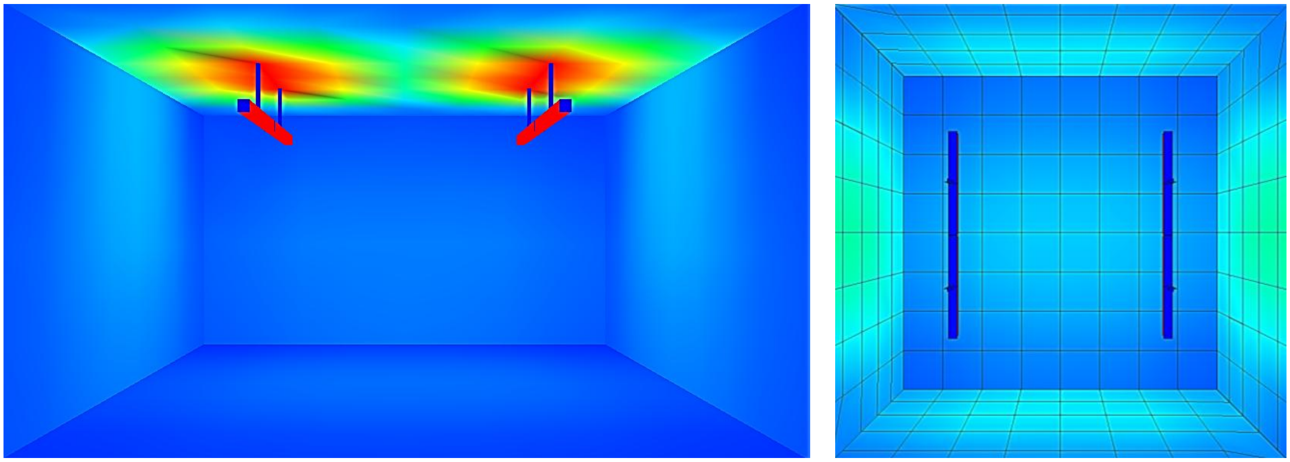
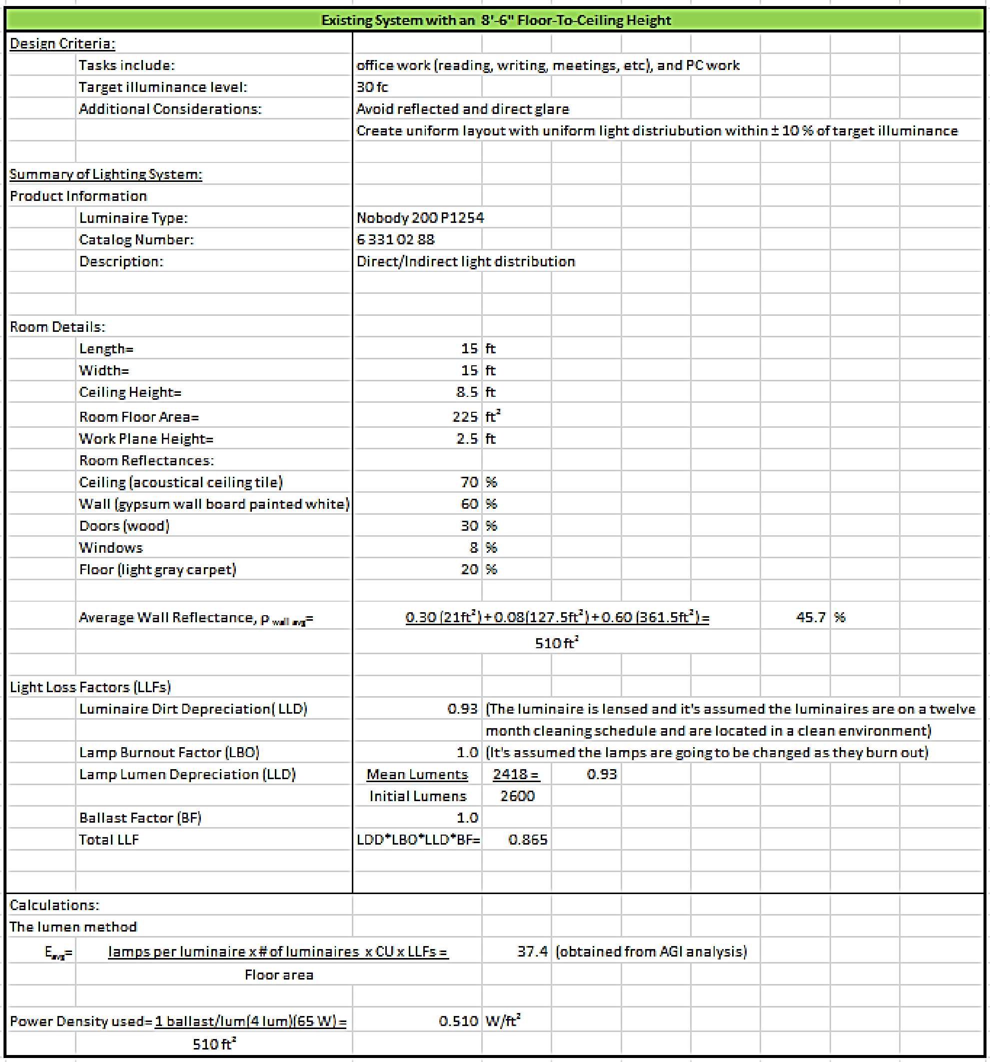


Figure 75 A thermograph of the of the office space with 8’-6” floor-to-ceiling height

The consistent blue color on the floor and walls in Figure 75 represents the designed lighting layout uniformly distributes the light through the space, thus preventing any hot spots from forming on the vertical and horizontal work planes.

In addition, after determining the number of luminaires needed to meet the target illuminance, the power density was calculated to determine the amount of energy the new lighting system uses. A summary of the power density calculations can be seen in Table 45.

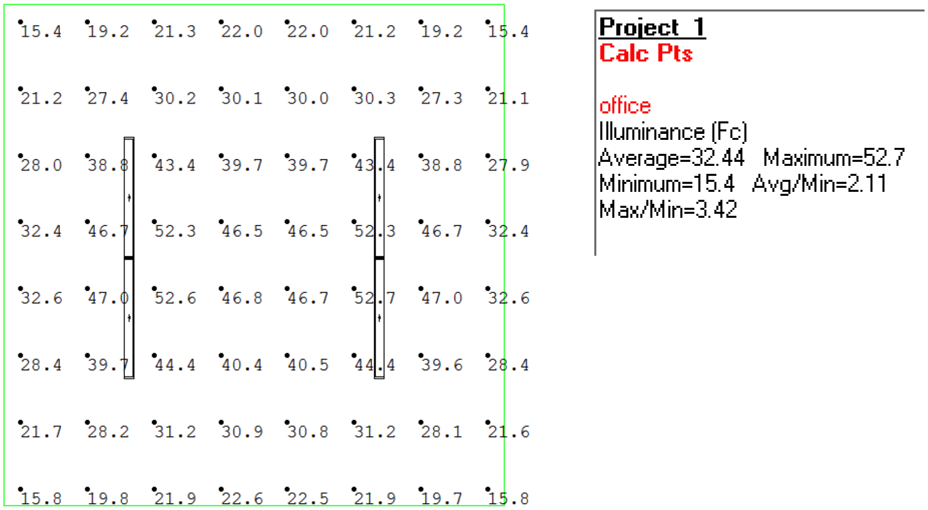


Existing System with an 8'-6" Floor-To-Ceiling Height				
Design Criteria:				
	Tasks include:	office work (reading, writing, meetings, etc), and PC work		
	Target illuminance level:	30 fc		
	Additional Considerations:	Avoid reflected and direct glare		
		Create uniform layout with uniform light distriubution within ± 10 % of target illuminance		
Summary of Lighting System:				
Product Information				
	Luminaire Type:	Nobody 200 P1254		
	Catalog Number:	6 331 02 88		
	Description:	Direct/Indirect light distribution		
Room Details:				
	Length=	15	ft	
	Width=	15	ft	
	Ceiling Height=	8.5	ft	
	Room Floor Area=	225	ft^2	
	Work Plane Height=	2.5	ft	
	Room Reflectances:			
	Ceiling (acoustical ceiling tile)	70	%	
	Wall (gypsum wall board painted white)	60	%	
	Doors (wood)	30	%	
	Windows	8	%	
	Floor (light gray carpet)	20	%	
	Average Wall Reflectance, $\rho_{wall\ avg}$=	$\frac{0.30\,(21ft^2)+0.08(127.5ft^2)+0.60\,(361.5ft^2)=}{510\,ft^2}$	45.7	%
Light Loss Factors (LLFs)				
	Luminaire Dirt Depreciation(LLD)	0.93	(The luminaire is lensed and it's assumed the luminaires are on a twelve	
			month cleaning schedule and are located in a clean environment)	
	Lamp Burnout Factor (LBO)	1.0	(It's assumed the lamps are going to be changed as they burn out)	
	Lamp Lumen Depreciation (LLD)	Mean Luments	2418 =	0.93
		Initial Lumens	2600	
	Ballast Factor (BF)	1.0		
	Total LLF	LDD*LBO*LLD*BF=	0.865	
Calculations:				
The lumen method				
E_{avg}=	$\frac{\text{lamps per luminaire x \# of luminaires x CU x LLFs =}}{\text{Floor area}}$	37.4	(obtained from AGI analysis)	
Power Density used= $\frac{\text{1 ballast/lum(4 lum)(65 W) =}}{510\ ft^2}$		0.510	W/ft^2	

Table 45 Power density calculations

It was found that the power density of the lighting system was 0.510 W/ft^2. According to IESNA 2010, the maximum power density for a closed office space is 1.11 W/ft^2. This represents that the new lighting system conserves energy and thus results in energy savings.

After designing the lighting system for the office space in the existing structural system, the same lighting system was checked to determine if it will meet the office space target illuminance in the new structural system with higher floor-to-floor heights. Using AGI to check the design, the floor-to-ceiling height was increased to 10-6”. Keeping the work plane height at 2’-6” and the suspended lighting fixture distance at 1’-6”, the distance to the work plane increased to 6’-6” in the new system. The illuminance values of the new space can be seen in Figure 76.



15.4	19.2	21.3	22.0	22.0	21.2	19.2	15.4
21.2	27.4	30.2	30.1	30.0	30.3	27.3	21.1
28.0	38.8	43.4	39.7	39.7	43.4	38.8	27.9
32.4	46.7	52.3	46.5	46.5	52.3	46.7	32.4
32.6	47.0	52.6	46.8	46.7	52.7	47.0	32.6
28.4	39.7	44.4	40.4	40.5	44.4	39.6	28.4
21.7	28.2	31.2	30.9	30.8	31.2	28.1	21.6
15.8	19.8	21.9	22.6	22.5	21.9	19.7	15.8

Project 1
Calc Pts

office
Illuminance (Fc)
Average=32.44 Maximum=52.7
Minimum=15.4 Avg/Min=2.11
Max/Min=3.42

Figure 76 Illuminance values of the office space with a 10'-6" floor-to-ceiling height

After the analysis, it was found that the lighting layout used in the existing office space can also be used in the new space with an increased work plane distance of 2'-0". As can be seen in Figure 76, the design resulted in an average illuminance of 32.44 foot-candles, which meets the target illuminance within ±10%. A rendering of the space with the new layout can be seen in Figure 77 and the office space's thermograph with the lighting system can be seen in Figure 78.

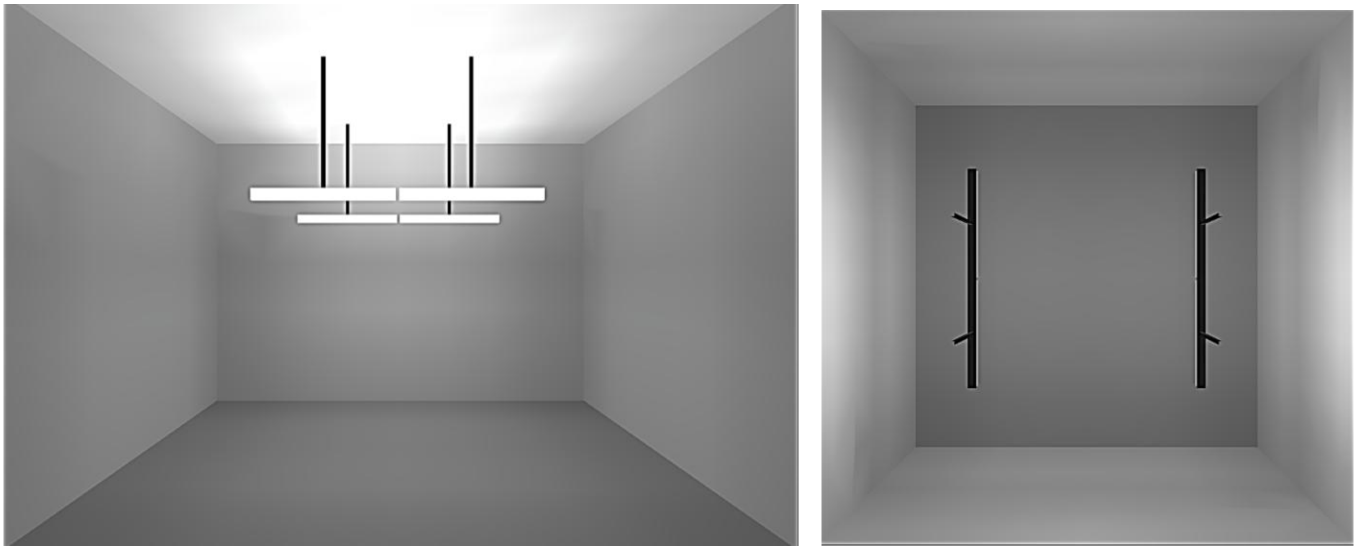


Figure 77 Rendering of the office space with 10'-6" floor-to-ceiling height

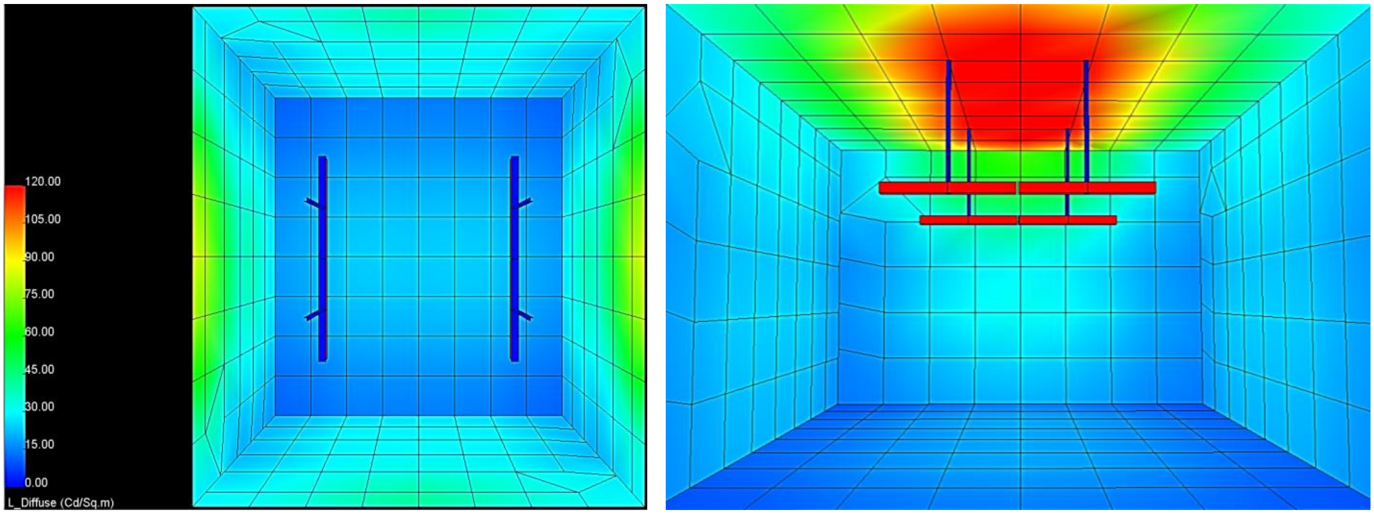
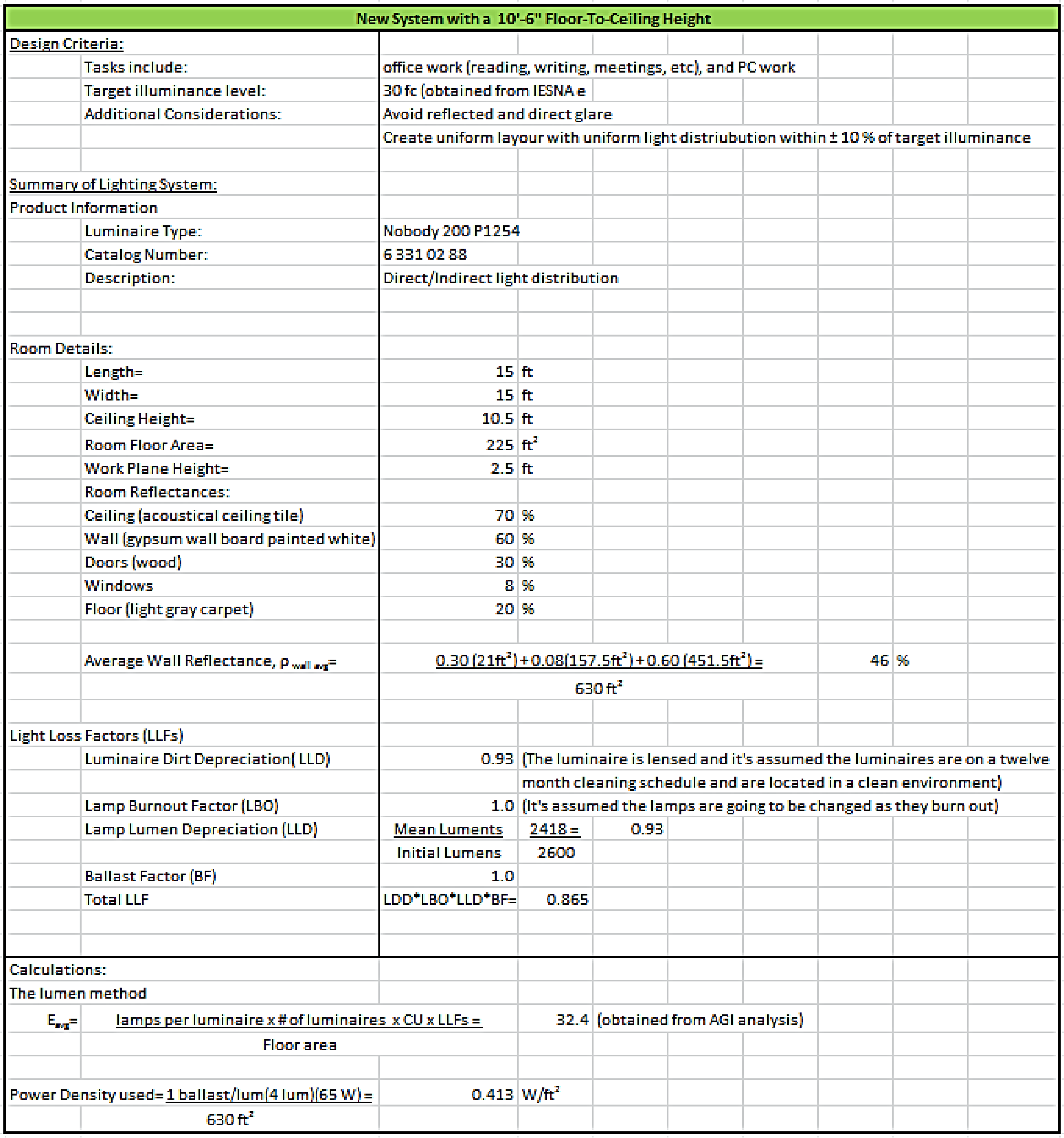


Figure 78 A thermograph of the office space with 10’-6” floor-to-ceiling height

The consistent blue color on the floor and walls represents that the designed lighting layout uniformly distributes the light through the space therefore preventing any hot spot from forming on the vertical and horizontal work planes.

In addition, after determining the number of luminaires needed to meet the target illuminance, the power density was calculated to determine the amount of energy the new lighting system uses. A summary of the power density calculations can be seen in Table 46.



New System with a 10'-6" Floor-To-Ceiling Height					
Design Criteria:					
	Tasks include:	office work (reading, writing, meetings, etc), and PC work			
	Target illuminance level:	30 fc (obtained from IESNA e			
	Additional Considerations:	Avoid reflected and direct glare			
		Create uniform layour with uniform light distriubution within ± 10 % of target illuminance			
Summary of Lighting System:					
Product Information					
	Luminaire Type:	Nobody 200 P1254			
	Catalog Number:	6 331 02 88			
	Description:	Direct/Indirect light distribution			
Room Details:					
	Length=	15	ft		
	Width=	15	ft		
	Ceiling Height=	10.5	ft		
	Room Floor Area=	225	ft^2		
	Work Plane Height=	2.5	ft		
	Room Reflectances:				
	Ceiling (acoustical ceiling tile)	70	%		
	Wall (gypsum wall board painted white)	60	%		
	Doors (wood)	30	%		
	Windows	8	%		
	Floor (light gray carpet)	20	%		
	Average Wall Reflectance, $\rho_{wall\ avg}$=	$\frac{0.30\,(21ft^2)+0.08(157.5ft^2)+0.60\,(451.5ft^2)}{630\ ft^2}=$		46	%
Light Loss Factors (LLFs)					
	Luminaire Dirt Depreciation(LLD)	0.93	(The luminaire is lensed and it's assumed the luminaires are on a twelve		
			month cleaning schedule and are located in a clean environment)		
	Lamp Burnout Factor (LBO)	1.0	(It's assumed the lamps are going to be changed as they burn out)		
	Lamp Lumen Depreciation (LLD)	Mean Luments	2418 =	0.93	
		Initial Lumens	2600		
	Ballast Factor (BF)	1.0			
	Total LLF	LDD*LBO*LLD*BF=	0.865		
Calculations:					
The lumen method					
E_{avg}=	lamps per luminaire x # of luminaires x CU x LLFs =		32.4	(obtained from AGI analysis)	
	Floor area				
Power Density used= 1 ballast/lum(4 lum)(65 W)=		0.413	W/ft^2		
	630 ft^2				

Table 46 Power density calculations

It was found that the power density of the lighting system was 0.413 W/ft^2. According to IESNA 2010, the maximum power density for a closed office space is 1.11 W/ft^2, which represents that the new lighting system conserves energy and thus results in energy savings for the new space.

In addition to designing the lighting system for the typical office space supported by the existing and new structural systems, the control of reflected glare was investigated. According to "Mechanical and Electrical Equipment for Buildings," there are a number of techniques that can be used to minimize contrast loss due to veiling reflections while maintaining adequate illumination. One of the techniques investigated was physical arrangement of system elements. In a space that uses multiple sources, particularly continuous rows as the design layout chosen for the office space, placing the work between rows with the line of sight parallel to the long axis of the units is an effective technique. Figure 79 shows both the preferred and non-preferred arrangement of work.

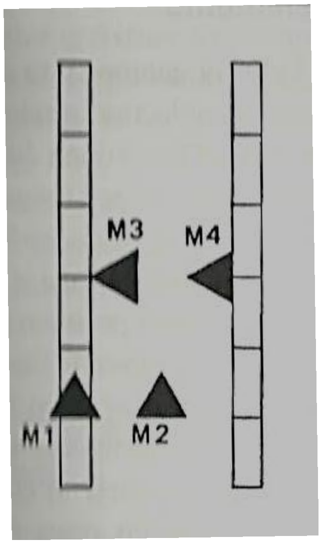


Figure79 Preferred and non-preferred arrangements of four possible work planes

Image obtained from "Mechanical and Electrical Equipment for Buildings"

According to *"Mechanical and Electrical Equipment for Buildings,"* M2 is the best location in that the work plane receives light from both rows of luminaires. Positions M1 and M3 are undesirable because they have bright sources in the offending zone, which can be seen depicted in Figure 80. Position M4 is also an ideal location because there are no glare sources in the offending zone.

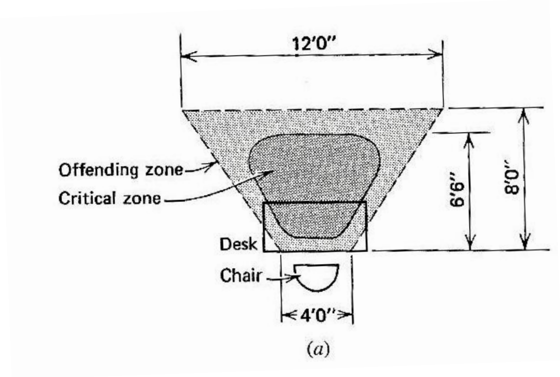


Figure 80 Offending and critical zones for the work plane

Image obtained from "Mechanical and Electrical Equipment for Buildings"

Based on the above information, if the work plane (desk) in the office space were located between the two rows of luminaires, where the occupants line of sight were parallel to the long axis of the luminaire units (similar to location M2 in Figure 79), direct and reflective glare would be avoided because the light contributed by the two rows of luminaires would bounce off of the desk away from the occupant. This desk configuration would also prevent shadows. If the desk were placed in front or directly beneath the row of luminaires (similar to locations M3 and M1 in Figure 79) the occupant would be subjected to direct and reflected glare and shadows, which are undesirable.

Conclusion

Before re-locating 1000 Connecticut Avenue to Arlington, VA it was found that to stay within Washington D.C.'s zoning height limit of 130 ft. when using the new steel structural system the system would have to be designed for a reduced number of stories. Reducing the number of stories from 12 to 11 was undesirable, therefore to create a fair comparison between the existing concrete system and new steel system the building was relocated to Arlington, VA, which does not have a height limit. The goal of the re-design was to

- increase the bay sizes to open the floor plan layout;
- increase floor-to-floor height to increase the openness of the space;
- Reduce the construction schedule;
- Reduce the structural system cost;
- Increase the annual revenue by increasing the rental value of the space and increasing the amount of rentable space

When designing the steel framing layout, a uniform layout was created to reduce number of required skewed members and wider bays were created by removing certain existing column lines and relocating columns. Wider bays were created to open the floor plan and to increase the rental value of the space with reduced column obstructions and more rentable space. Maintaining an open floor layout was an importance aspect of the re-design, therefore for the lateral system moment frames were used to avoid obstructions in the in the floor plan layout and braced frames were located around the elevator shafts and stairwell cores. The gravity system was designed as a composite steel system to achieve long spans while maintaining minimal structural depth. AISC 14th edition was used to design the gravity frame members. ETABS was used to analyze and design the lateral system. The lateral system design and analysis was based on the wind and seismic lateral loads calculated according to ASCE 7-10. The wind loads were determined by using Analytical Procedure (method 2) outlined in ASCE 7-10 and the seismic loads were determined by using the Equivalent Lateral Force Procedure outlined in ASCE 7-10. After designing the gravity and lateral systems, typical member connections were designed. The typical connections designed were orthogonal and skewed shear connections and a moment frame connection.

After designing the gravity and lateral systems, two breadth studies were conducted to determine how the new structural system affected other aspects of the building. The first breath study was construction management impact. In this breadth, it was found that the new structural system will cost $5,994,630 more than the existing structural system. Second, the proposed construction sequence for the new structural was erected 18 days sooner than the existing structural system, thus representing the use of the new system reduced the construction schedule. Third, using the existing 1000 Connecticut Avenue site for the site logistics analysis, it was found that the site will be managed similarly for both concrete and steel construction. Fourth, the building will maintain LEED Gold Certification with the use of the new steel structural system. Last, the revenue obtained from the new structural system with wider bays and higher floor-to-ceiling heights resulted in additional revenue of $3,705,450 annually since the rental value of the space increased with the new framing layout. Therefore based on the construction management breadth, it is concluded that the new structural system with wider bays and higher floor-to-ceiling heights results in an overall very successful design with a reduced construction schedule and increased rental value. It is concluded that the proposed steel structural system is a viable alternative system to use in Arlington, VA since the new system has many additional benefits compared to the existing concrete structural system.

The second breadth studied was acoustics and lighting impact. This breadth involved determining the sound treatments required for a typical office space located in the new structural system. The analysis began by determining the level of speech privacy the common wall barrier between offices provided. It was shown that a 54 STC rated 8” partition wall with 2-layers of ½” thick gypsum wall board on both sides, staggered electrical boxes isolated with insulation, and 2 ½” metal studs spaced 24” o.c. and is very adequate for providing speech privacy for the offices housed in the new steel structural system. In addition, since the new structural system was designed for higher floor-to-ceiling heights, lighting illuminance applied to the work plane surfaces were affected. As a result, a lighting breadth was conducted by designing the lighting system for a typical office space using the existing floor-to-ceiling height of 8’-6” and checking to determine if the same lighting system can be used for the new floor-to-ceiling height of 10’-6”. AGI was used to design the lighting system for the space and the average illuminance in the space was compared to the target illuminance of the space. The IESNA Handbook 10th edition was used to determine the target illuminance and maximum power density for a private office space. It was found that the lighting system designed for the space with a floor-to-ceiling height of 8’-6” also achieved the target lighting illuminance for the space with a floor-to-ceiling height of 10’-6”.

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Appendix A: Gravity System Design

Snow Load Calculation

Snow Drift | tech 1 | page 1 of 2

step 1: ground snow load, p_g → from fig. 7-1, $p_g = 25$ psf

step 2: Exposure factor, C_e, → from table 7-2, Terrain category B
roof fully exposed
⇒ $C_e = 0.90$

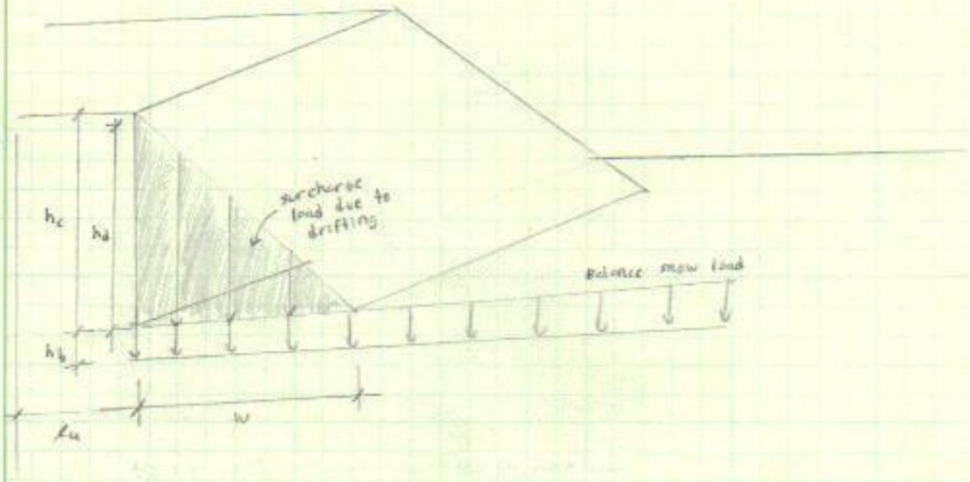
step 3: Thermal factor, C_t → from table 7-3, $C_t = 1.0$

step 4: importance factor, I → from table 1.5-1, $I = 1.0$ (occ. II)

step 5: flat roof snow load, p_f → sect 7.3,

$$p_f = 0.7\, C_e C_t I_s p_g$$
$$= 0.7(0.90)(1.0)(1.0)(25\text{ psf})$$
$$= 15.75\text{ psf}$$

Snow drift : Penthouse level



step 6: maximum intensity of the drift surcharge load, p_d → sect 7.7.1

$p_d = h_d \gamma$ ⇒ snow density $\gamma = 0.13 p_g + 14$
$= 0.13(25\text{ psf}) + 14$
$= 17.25\text{ pcf} < \gamma_{max} = 30\text{ pcf}$

leeward drift
E-W wind : $\ell_u = 135'$ ⇒ snow drift height, h_d → from fig. 7-9,

$$h_d = 0.43 \sqrt[3]{135} \sqrt[4]{25+10} - 1.5 = 3.7\text{ ft}$$

leeward drift
N-S wind : $\ell_u = 55.9'$ ⇒ $h_d = 0.43 \sqrt[3]{55.9} \sqrt[4]{25+10} - 1.5 = 2.5$ ft

snow drift | Tech 1 | page 2 of 2

leeward drift

E-W wind: $p_d = 3.9\ ft(17.25\ pcf) = 67.3\ psf$; width of snow drift, $w = 4h_d = 4(3.9\ ft) = 15.6\ ft$

leeward drift

N-S wind: $p_d = 2.5\ ft(17.25\ pcf) = 43.1\ psf$; $w = 4(2.5) = 10\ ft$

step 7: balanced snow load height, h_b → from sect 2.1

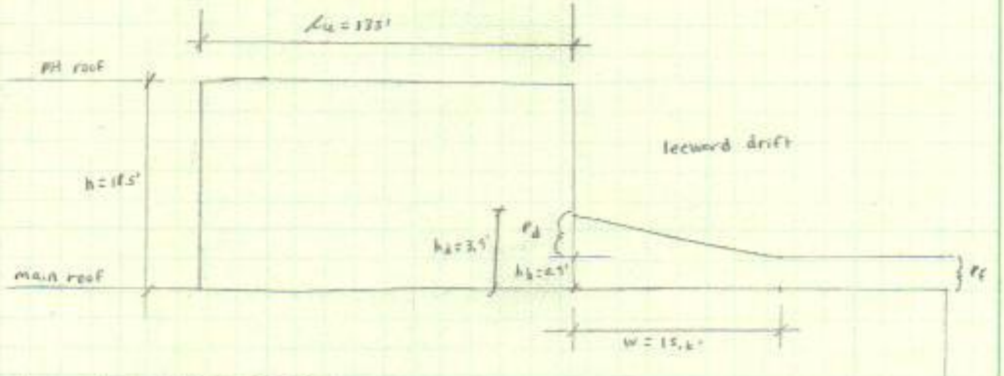
$$h_b = \frac{p_f}{\gamma} = \frac{15.75\ psf}{17.25\ pcf} = 0.9\ ft$$

step 8: $h = \underset{\text{PH roof}}{148.5'} - \underset{\text{main roof}}{130'} = 18.5\ ft$

step 9: $h_c = h - h_b = 18.5' - 0.9' = 17.6\ ft$

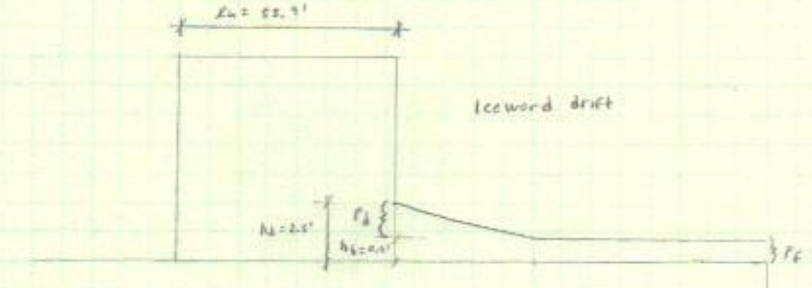
step 10: total snow load, $p = p_d + p_f$

E-W wind



$p_{total} = 67.3\ psf + 15.75\ psf = 83\ psf$

N-S wind

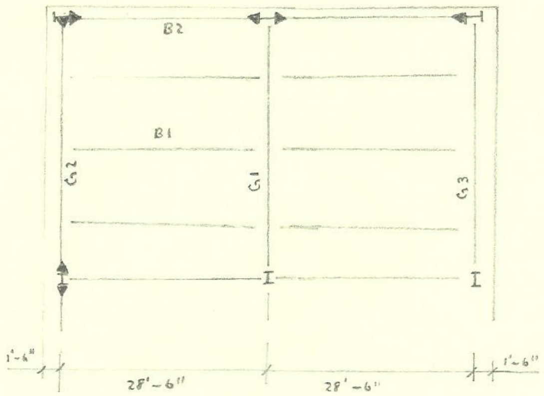


$p_{total} = 43.1 + 15.75 = 59\ psf$

since $p_{tot,E\text{-}W} > p_{tot,N\text{-}S}$, use $p_{tot} = 83\ psf$ for a conservative design

Gravity Floor System Design

composite Flr System Design | 28'-6" x 40'-0" Bay | page 1 of 9



interior bm B1

① total load

dead load:

SDL - 10 psf
slab - 75 psf

$w_D = (10\text{ psf} + 75\text{ psf})(10\text{ft}) = 850\text{ plf}$

live load:

$L_o = 100\text{ psf}$

influence area $= k_{LL} A_T = 2(28.5\text{ ft})(10\text{ft}) = 570\text{ ft}^2 > 400\text{ ft}^2 \therefore$ reduce LL

$$L = 100 \times \max \left| \begin{array}{l} 0.5 \\ 0.25 + \frac{15}{\sqrt{570}} = 0.878 \end{array} \right. = 87.8\text{ psf}$$

$w_L = 87.8\text{ psf}(10\text{ft}) = 878\text{ plf}$

② required moment, M_u → assumptions: - bm simply supported
- add 5 psf for bm self-wt

$w_u = 1.2(5\text{psf})(10\text{ft}) + 1.2(850) + 1.6(878) = 2485\text{ plf}$

$$M_u = \frac{w_u L^2}{8} = \frac{(2485)(28.5)^2}{8} = 252{,}305\text{ lb-ft} = 252\text{ Kft} \; ; \; V_u = \frac{w_u L}{2} = \frac{2.485(28.5)}{2} = 35.4\text{ K}$$

③ starting moment arm for the concrete, Y2

assume $a = 1"$

$Y2 = t - \frac{a}{2} = 7.5 - \frac{1}{2} = 7"$

④ determine I_{LB}

$\Delta_{LL} \leq \Delta_{LL,max} = L/360$

$$\frac{5(0.878)(28.5)^4(1728)}{384 E I_{LB}} \leq \frac{28.5(12)}{360}$$

$$I_{LB} \geq \frac{5(0.878)(28.5)^4(1728)}{384(29000)(0.95)} = 473 \text{ in}^4$$

$\Delta_{TL} \leq \Delta_{TL,max} = L/240$

$$I_{LB} \geq \frac{5(0.878+0.850)(28.5)^4(1728)}{384(29000)(1.425)} = 621 \text{ in}^4$$

⑤ select potential W-shapes from table 3-19 and 3-20 in manual

$(\phi M_n \geq 252 \text{ Kft},\ I_{LB} \geq 621 \text{ in}^4)$

W10x26, $I_{LB} = 655 \text{ in}^4$, $\phi M_n = 313$ Kft, $\Sigma Q_n = 317$ K (Y1=2)

W12x22, $I_{LB} = 631 \text{ in}^4$, $\phi M_n = 274$ Kft, $\Sigma Q_n = 238$ K (Y1=3)

W14x22, $I_{LB} = 672 \text{ in}^4$, $\phi M_n = 270$ Kft, $\Sigma Q_n = 199$K (Y1=4)

W14x26, $I_{LB} = 634 \text{ in}^4$, $\phi M_n = 265$ Kft, $\Sigma Q_n = 135$K (Y1=6)

⑥ # of studs/bm

W10x26: $\frac{\Sigma Q_n}{Q_n} \times 2 = \frac{317}{\underset{\substack{\uparrow \\ 1\text{ stud/rib}}}{17.2}} \times 2 = 38 > 28.5'$ ∴ use 2 studs/rib

$\frac{317}{14.6} \times 2 = 44$ studs $< 28.5'(2) = 57'$ ∴ ok

W14x26: $\frac{135}{17.2} \times 2 = 16$ studs $< 28.5'$

W12x22: $\frac{238}{17.2} \times 2 = 28$ studs $< 28.5'$

W14x22: $\frac{199}{17.2} \times 2 = 24$ studs $< 28.5'$

⑦ check economy

W10x26 w/ 44 studs : 26# x 28.5' + 44 studs x 10#/stud = 1181 #

W12x22 w/ 28 studs : 22# x 28.5' + 28 x 10#/stud = 907 #

W14x22 w/ 24 studs : 22# x 28.5' + 24 x 10#/stud = 867 #

W14x26 w/ 16 studs : 26# x 28.5' + 16 x 10#/stud = 901 #

28'-6" x 40'-0" Bay | page 3 of 9

⑧ check a

$$a = \frac{\Sigma Q_n}{0.85 f'_c \, b_{eff}}$$

$$b_{eff} = \min \begin{cases} \frac{28.5(12)}{8} \times 2 = 85.5'' \leftarrow \\ \frac{1}{2}(10)(12) \times 2 = 120'' \\ \frac{28.5(12)}{8} + \frac{10(12)}{2} = 102.75'' \end{cases}$$

W14X22: $a = \frac{199}{0.85(4)(85.5)} = 0.68'' < \text{assumed } 1''$ ∴ ok

⑨ check unshored strength

W14X22, $\phi_b M_p = 125$ Kft

$w_u = 1.2(75 \text{ psf})(10 \text{ft}) + 1.2(22 \text{ plf}) + 1.6(20 \text{ psf})(10 \text{ft}) = 1246 \text{ plf}$

$M_u = \frac{1.246 \text{ Klf } (28.5)^2}{8} = 126.5 \text{ Kft} > \phi_p M_p$ ∴ increase section

try W14X26

$w_u = 900 + 1.2(26) + 320 = 1251 \text{ plf}$

$M_u = 127 \text{ Kft} < \phi_b M_p = 151 \text{ Kft}$ ∴ ok for no shoring

⑩ check Δ_{wc}

$w_{wc} = 75 \text{ psf } (10 \text{ft}) + 26 \text{ plf} = 776 \text{ plf}$

$$\Delta_{wc} = \frac{5(0.776 \text{ Klf})(28.5)^4(1728)}{384(29000)(245)} = 1.62''$$

$$\Delta_{wc,max} = \frac{28.5(12)}{240} = 1.43'' < \Delta_{wc} \quad \underline{\text{not ok}}$$

try W16X26 : $\Delta_{wc} = \frac{5(0.776)(28.5)^4(1728)}{384(29000)(301)} = 1.32'' < \Delta_{wc,max}$ ok

⑪ check LL and TL deflection

W16X26;

$Y2 = 7$, point $Y1 = 7$ $(\Sigma Q_n = 96 \text{ K})$, $I_{LB} = 640 \text{ in}^4$

$$\Delta_{LL} = \frac{5(0.878)(28.5)^4(1728)}{384(29000)(640)} = 0.70'' < \Delta_{LL,max} = 0.95''$$

$$\Delta_{TL} = \frac{5(1.728)(28.5)^4(1728)}{384(29000)(640)} = 1.38'' < \Delta_{TL,max} = 1.43''$$

28'-6" x 40'-0" Bay page 4 of 9

⑫ check M_u, V_u and bm self-wt assumption

$M_u = 252 \text{ kft} < \phi M_n = 296 \text{ Kft} \therefore \text{OK}$

$V_u = 35.4 \text{ K} < \phi V_n = 106 \text{ K} \therefore \text{OK}$

$$\text{self wt} = \frac{26 \text{ plf}}{\underset{\overset{\uparrow}{w_{trib.}}}{10 \text{ ft}}} = 2.6 \text{ psf} < 5 \text{ psf} \therefore \text{OK}$$

⑬ check economy: $W16 \times 26: \frac{\Sigma Q_n}{Q} \times 2 = \frac{96 \text{ K}}{17.2 \text{ K}} \times 2 \Rightarrow 12 \text{ studs} < 28.5' \text{ ok}$

<u>use W16x26 w/ 12 studs</u>

28'-6" x 40'-0" Bay

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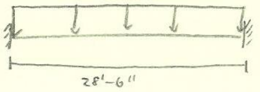
exterior bm B2



① total load

dead load :

slab - 75 psf

SDL - 10 psf

self-wt allow. - 5 psf

curtain wall - 250 plf

$w_D = (75 + 10 + 5)$ psf $(6.5$ ft$) + 250$ plf $= 835$ plf

live load:

$L_0 = 100$ psf

influence area $= 28.5$ ft $(11.5$ ft$) = 327.75$ ft$^2 < 400$ ft^2, LL reduction not applicable

$w_L = 100$ psf $(6.5$ft$) = 650$ plf

$w_u = 1.2(835) + 1.6(650) = 2042$ plf

② $M_u = \dfrac{2.042 \text{ klf } (28.5 \text{ft})^2}{12} = 138.2$ Kft $\leq \phi M_n = \phi M_p \Rightarrow Z_x \geq \dfrac{138.2 \text{ kft } (12)}{0.9 \text{ } (50 \text{ ksi})} = 36.9$ in^3

$V_u = \dfrac{2.042 \text{ klf } (28.5 \text{ft})}{2} = 29$ K $\leq \phi V_n$

③ $\Delta_{TL} \leq \Delta_{TL,max} = L/240 = 1.425"$

$\Delta_{TL} = \dfrac{wL^4}{384EI} \Rightarrow I_x \geq \dfrac{2.042 \, (28.5)^4 (1728)}{384 (29000)(1.425)} = 146.7$ in^4

④ select non-composite W-shape

try W12x26 beam

$Z_x = 37.2$ in$^3 > 36.9$ in^3 ok

$I_x = 204$ in$^4 > 146.7$ in^4 ok

$\phi M_n = 140$ kft > 138.2 kft ok

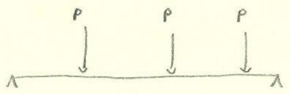
$\phi V_n = 84.2$ k > 29 k ok

note: required bm size to support gravity load.

For bm size due to combined lateral and gravity analysis,

refer to moment frame & elevation in report page 6 of 9

Interior girder G1



① total load

dead load : slab - 75 psf
SDL - 10 psf
bm self wt allowance - 5 psf
girder self wt allowance - 5 psf

live load : L_o = 100 psf

I.A. = $(28.5 + 28.5)(40) = 2280\ ft^2 > 400\ ft^2 \therefore$ reduce LL

$$L = 100 \times \max \left| \begin{array}{l} 0.50 \\ 0.25 + \frac{15}{\sqrt{2280}} = 0.56 \end{array} \right. = 56\ psf$$

28'-6" x 40'-0" Bay

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$w_u = 1.2(75 + 10 + 5 + 5) + 1.6(56) = 204 \text{ psf}$

$P_u = 204 \text{ psf} \left(\frac{28.5 + 28.5}{2}\right)(10) = 58.1 \text{ K}$

② $M_u = 0.50PL = 0.50(58.1\text{ K})(40\text{ft}) = 1162 \text{ Kft}$

$V_u = 1.5P = 1.5(58.1\text{ K}) = 87.2 \text{ K}$

③ $\Delta_{LL} \leq \Delta_{LL,max}$

$$\frac{0.050PL^3}{EI} \leq \frac{L}{360} = \frac{40(12)}{360} = 1.33''$$

$$I_{LB} \geq \frac{0.050(16\text{k})(40\text{ft})^3(1728)}{29000(1.33\text{in})} = 2294 \text{ in}^4$$

$$\Delta_{TL} \leq \Delta_{TL,max} = \frac{L}{240} = \frac{40(12)}{240} = 2''$$

$$I_{LB} \geq \frac{0.05(43\text{k})(40\text{ft})^3(1728)}{29000(2)} = 4100 \text{ in}^4$$

④ required unshored strength

$$P_u = 1.2[(\underset{\substack{\uparrow \\ \text{slab}}}{75} + \overset{\substack{\text{girder self-wt allowance} \\ \downarrow}}{5})\text{psf}(10\text{ft})](28.5\text{ ft}) + 1.6(20\text{ psf})(10\text{ft})(28.5\text{ft}) = 36.5\text{K}$$

$M_u = 0.50PL = 0.50(36.5\text{ K})(40\text{ft}) = 730 \text{ Kft} \leq \phi_b M_p$

⑤ assume $a = 2''$, $Y2 = 7.5 - \frac{2}{2} = 6.5''$

⑥ select potential w-shapes

$(\phi M_n \geq 1162 \text{ Kft},\ I_{LB} \geq 4100 \text{ in}^4,\ \phi_b M_p \geq 730 \text{ kft})$

W24x76: $Y1 = \text{BFL}$, $I_{LB} = 4480 \text{ in}^4$, $\phi M_n = 1240 \text{ Kft}$, $\phi_b M_p = 750 \text{ kft}$, $\Sigma Q_n = 509 \text{ K}$

W24x84: $Y1 = 6$, $I_{LB} = 4550 \text{ in}^4$, $\phi M_n = 1290 \text{ Kft}$, $\phi_b M_p = 840 \text{ Kft}$, $\Sigma Q_n = 425 \text{ K}$

⑦ # of studs/girder and economy

from table 3-21 in manual, $Q_n = 21.5 \text{ K}$

W24x76: $\frac{\Sigma Q_n}{Q_n} \times 2 = \frac{509}{21.5} \times 2 = 48$ studs ; 76# x 40' + 48 studs x 10#/stud = 3520 #

W24x84: $\frac{425}{21.5} \times 2 = 40$ studs ; 84# x 40' + 40 studs x 10#/stud = 3760 #

try W24x76 since it's more economical

28'-6" x 40'-0" Bay   page 8 of 9

⑧ check a

$$a = \frac{\Sigma Q_n}{0.85 f'_c\, b_{eff}} = \frac{509}{0.85(4)(120)} = 1.25'' < 2'' \quad ok$$

for interior girder, $b_{eff} = \min \left| \begin{array}{l} 2 \times \frac{span}{8} = 2 \times \frac{40(12)}{8} = 120'' \leftarrow \\ 2 \times \frac{1}{2} trib_{width} = 2 \times \frac{28.5(12)}{2} = 342'' \end{array} \right.$

⑨ check Δ_{LL}, Δ_{TL}, unshored strength, M_u, V_u, girder self-wt assumpt.

$$\Delta_{LL} = \frac{0.050(16k)(40')^3(1728)}{29000(4480\ in^4)} = 0.68'' < 1.33'' \quad ok$$

$$\Delta_{TL} = \frac{0.050(43k)(40')^3(1728)}{29000(4480)} = 1.83'' < 2'' \quad ok$$

unshored strength: $P_u = 1.2\,[(75psf)(10ft) + 76\ plf\,](28.5ft) + 1.6(20)(10ft)(28.5ft) = 37.4\ k$

$M_u = 0.50(37.4k)(40ft) = 748\ kft < \phi_b M_p = 750\ k \quad ok$

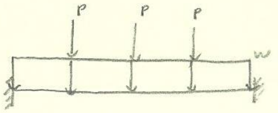
$M_u = 1162\ kft < \phi M_n = 1240\ kft \quad ok$

$V_u = 87.2\ k < \phi V_n = 315\ K \quad ok$

$$\text{self wt} = \frac{76\ plf}{28.5\ ft} = 2.67\ psf < 5psf \quad ok$$

use W24x76 girder with 48 studs

Exterior Girder G2



① load :

dead: slab - 75 psf
SDL - 10 psf
bm - wt allow. - 5 psf
girder self-wt allow. - 5 psf
wall - 250 plf

live : $100\ psf \times \max \left| \begin{array}{l} 0.50 \\ 0.25 + \frac{15}{\sqrt{1200}} = 0.68 \end{array} \right. = 68\ plf$

28'-6" X 40'-0" Bay   page 9 of 9

$$P_u = [1.2(75 + 10 + \overset{\text{bm-wt allow.}}{5})\,psf + 1.6(68\,psf)](15.75\,ft)(10\,ft) = 34.2\,K$$

$$w_u = 1.2(\underset{\text{girder self-wt allow.}}{5}\,psf)(15.75\,ft) + 1.2(250\,plf) = 395\,plf$$

② $M_u = 0.313PL + \frac{wL^2}{12} = 0.313(34.2\,K)(40\,ft) + \frac{0.395\,klf(40\,ft)^2}{12} = 481\,Kft \le \phi M_n = \phi M_p$

$V_u = 1.5P + \frac{wL}{2} = 1.5(34.2\,K) + \frac{0.395\,klf(40\,ft)}{2} = 59.2\,K \le \phi V_n$

$\phi M_p = \frac{\phi F_y Z_x}{12} \Rightarrow Z_x = \frac{481\,Kft(12)}{0.9(50\,ksi)} = 128.3\,in^3$

③ $\Delta_{TL} \le \Delta_{TL,max} = L/240 = 2.0''$

$\Delta_{TL} = \frac{0.010PL^3}{EI} + \frac{wL^4}{384EI} = \frac{0.010(34.2)(40)^3(1728)}{29000\,I_x} + \frac{0.395(40)^4(1728)}{384(29000)\,I_x} = \frac{1304}{I_x} + \frac{157}{I_x}$

$2.0'' = \frac{1461}{I_x} \Rightarrow I_x \ge 731\,in^4$

④ select non-composite W-section

<u>W24x55</u> girder required
to support gravity load

w/ $Z_x = 134\,in^3 > 128.3\,in^3$ ok
$I_x = 1350\,in^4 > 731\,in^4$ ok

$\phi M_n = 503\,Kft > 481\,Kft$ ok
$\phi V_n = 252\,K > 59.2\,K$ ok

note: Etabs used for
lateral analysis

self wt $= \frac{55\,plf}{15.75\,ft} = 3.5\,psf < 5\,psf$ ok

- refer to moment frame 1
elevation in report for
member sizes due to combined lateral
and gravity analysis

<u>Exterior Girder G3</u>
<u>use W24x55 w/ 32 studs</u>

- refer to excel spreadsheet for design

28'-6" x 40'-0" Bay

Exterior Girder G3- simply supported with 3 point loads and a distributed dead load

Step 1 Dead Load:

slab	75	psf
SDL	10	psf
bm self-wt allow.	5	psf
girder self-wt allow.	5	psf
wall	250	plf

Live Load: office space 100 psf

Trib. Width:	15.75	ft
Span, L:	40	ft
Influence Area:	1200	ft^2
LL_{red}	0.68	

P_u= 34.2 kips

W_u= 0.395 klf

step 2

M_u= 763 k-ft ≤ ØMn

V_u= 59.2 kips ≤ ØVn

step 3 assume a=2" Y2=7-a/2=6.5"

step 4

$\Delta_{TL} \leq \Delta_{TL,max}$= L/240= 2 in

$P_{unfactored}$= 24.9 kips

$W_{unfactored}$= 0.329 klf

$I_{LB} \geq$ 2703 in^4

step 5 unshored strength $Ø_bM_p$

Dead Load:

slab	75	psf
girder self-wt allow.	5	psf

Live Load: Constr. LL 20 psf

Trib. Width:	15.75	ft
Span, L:	40	ft

P_u= 20.2 kips

M_u= 403.2 k-ft ≤ $Ø_bM_p$

step 6 Select potential w-shapes

($ØM_n \geq$763 kft, $I_{LB} \geq$ 2703 in4, ØbMp ≥ 403 kft

W21x57				W24x55			
	Y1=	BFL			Y1=	6	
	∑Qn	409	kips		∑Qn	329	kips
	ØMn=	850	kft		ØMn=	853	kft
	ØbMp=	484	kft		ØbMp=	503	kft
	I_{LB}=	2770			I_{LB}=	2920	

step 7 # of studs per girder and economy

stud strength 21.5 kips

W21x57	∑Qn/Q x2	38.05	→	40	studs
	economy:	2680 #			
W24x55	∑Qn/Q x2	30.6	→	32	studs
	economy:	2520 #			

try W24x55 w/ 32 studs since it's more economical

step 8 check a

b_{eff}=

120	in
231	in
78	in
360	in

min

∑Qn= 329 kips

f'c= 4 ksi

beff= 78 in

a= ∑Qn/(0.85x f'c xbeff) 1.24 ≤ 2.0 in ok

step 9 check ΔLL, ΔTL, unshored strengh, Mu, Vu, and girder self-wt assumption

$P_{L, unfactored}$= 10.8 kips

I_{LB}= 2920.0 in^4

Δ_{LL}= 0.70 in ≤ $\Delta_{LL,max}$= L/360 = 1.33 in ok

Δ_{TL}= 1.85 in ≤ $\Delta_{TL,max}$= L/240 = 2.0 in ok

Unshored strength ØbMp= 503 kft ≥ 403 kft ok

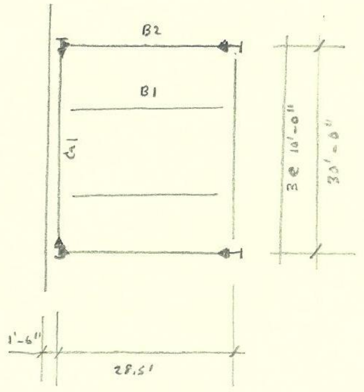
Mu= 763 kft ≤ ØMn = 853 kft ok

Vu= 59.2 kips ≤ ØVn=252 kips ok

girder-self wt assumpt= girder wt/trib. Width= 55 plf/15.75 ft= 3.5 psf ≤ 5 psf ok

Girder Design: Use W24x55 with 32 studs

composite Flr System Design | 30'-0" x 28'-6" Bay | page 1 of 4



Interior Beam B1

① $w_D = (75 + 10 + 5)\ \text{psf}\ (10\text{ft}) = 900\ \text{plf}$

$LL = 100\ \text{psf} \times \max\left|\begin{array}{l} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{570}} = 0.878 \end{array}\right. = 87.8\ \text{psf}$

$w_L = 87.8\ \text{psf}\ (10\text{ft}) = 878\ \text{plf}$

$w_u = 1.2(900) + 1.6(878) = 2485\ \text{plf}$

② $M_u = \dfrac{w_u L^2}{8} = \dfrac{2.485\,(28.5)^2}{8} = 252\ \text{Kft}$

$V_u = \dfrac{w_u L}{2} = \dfrac{2.485\,(28.5)}{2} = 35.4\ \text{k}$

③ assume $a = 1"$, $Y2 = 7"$

④ $\Delta_{TL} \le \Delta_{TL,max} = \dfrac{L}{240}$

$I_{LB} \ge \dfrac{5(0.90 + 0.878)(28.5)^4 (1728)}{384\,(29000)(1.425)} = 639\ \text{in}^4$

$\Delta_{wc} \le \Delta_{wc,max} = L/240$

$w_{wc} = (75 + 5)\ \text{psf}\ (10\text{ft}) = 0.800\ \text{klf}$

$I_{\text{non-comp}} \ge \dfrac{5(0.80)(28.5)^4 (1728)}{384\,(29000)(1.425)} = 287\ \text{in}^4$

⑤ required unshore strength, $\phi_b M_p$

$w_u = 1.2(75 + \overset{\substack{\text{bm self-wt allowance} \\ \uparrow}}{5})\ \text{psf}\ (10\text{ft}) + 1.6(20\ \text{psf})(10\text{ft}) = 1280\ \text{plf}$

$M_u = \dfrac{1.28\,(28.5)^2}{8} = 130\ \text{Kft} \le \phi_b M_p$

30'-0" x 28'-6" Bay		page 2 of 4

⑥ select potential w-shapes

($\phi M_n \geq 252$ kft, $I_{LB} \geq 639$ in^4, $\phi_b M_p \geq 130$ kft, $I \geq 287$ in^4)

W14x30: $Y1 = 6$, $\Sigma Q_n = 147$k, $I_{LB} = 717$ in^4, $I = 291$ in^4, $\phi M_n = 300$ kft, $\phi_b M_p = 177$ kft

W16x26: $Y1 = 7$, $\Sigma Q_n = 96$k, $I_{LB} = 640$ in^4, $I = 301$ in^4, $\phi M_n = 259$ kft, $\phi_b M_p = 166$ kft

⑦ # of studs/bm and economy

W14x30: $\frac{147k}{17.2k} \times 2 \Rightarrow$ 18 studs ; $30\# \times 28.5' + 18$ studs $\times$ 10#/stud = 1035#

W16x26: $\frac{96k}{17.2} \times 2 \Rightarrow$ 12 studs ; $26\# \times 28.5' + 12$ studs $\times$ 16#/stud = 861#

try W16x26 w/ 12 studs

⑧ check a

beff = 85.5"

$$a = \frac{96}{0.85(4)(85.5)} = 0.33" < 1" \text{ ok}$$

⑨ check Δ_{LL}, Δ_{TL}, Δ_{WC}, unshored strength, Mu, Vu, bm self-wt assumption

$$\Delta_{LL} = \frac{5(0.878 \text{ klf})(28.5 \text{ ft})^4(1728)}{384(29000)(640)} = 0.70" < \ell/360 = 0.95" \quad \text{ok}$$

$$\Delta_{TL} = \frac{5(0.900 + 0.878)(28.5)^4(1728)}{384(29000)(640)} = 1.42" < \ell/240 = 1.43" \quad \text{ok}$$

$$\Delta_{WC} = \frac{5(0.800)(28.5)^4(1728)}{384(29000)(301)} = 1.36" < \ell/240 \quad \text{ok}$$

$M_u = 252$ kft $< \phi M_n = 259$ kft

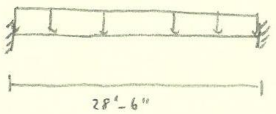
$V_u = 35.4$ k $< \phi V_n =$

bm self-wt = $\frac{26 \text{ plf}}{10 \text{ ft}} = 2.6$ psf < 5 psf ok

<u>use W16x26 bm w/ 12 studs</u>

30'-0" x 28'-6" Bay page 3 of 4

interior bm B2



① $w_D = (75 + \overset{\text{SDL}}{10} + 5)\text{ psf}\,(9.75\text{ ft}) = 877.5\text{ plf}$

slab ↑ bm self-wt allowance ↑

$$LL = 100\text{ psf} \times \max\left|\begin{array}{l} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{555.75}} = 0.886 \end{array}\right. = 88.6\text{ psf}$$

$w_L = 88.6\text{ psf}\,(5 + 4.75)\text{ ft} = 864\text{ plf}$

$w_u = 1.2(877.5\text{ plf}) + 1.6(864\text{ plf}) = 2435\text{ plf}$

② $$M_u = \frac{w_u L^2}{12} = \frac{2.435(28.5)^2}{12} = 165\text{ kft} \le \phi M_n = \phi M_p \Rightarrow Z_x \ge \frac{165(12)}{0.9(50)} = 44\text{ in}^3$$

$$V_u = \frac{2.435(28.5)}{2} = 34.7\text{ K} \le \phi V_n$$

④ $\Delta_{TL} \le \Delta_{TL,max} = L/240$

$$I_x \ge \frac{(0.878 + 0.864)(28.5)^4(1728)}{384(29000)(1.425)} = 125\text{ in}^4$$

⑤ select non-composite w-shape

try $\underline{\text{W W16x26}}$

$Z_x = 44.2\text{ in}^3 > 44\text{ in}^3$

$I_x = 301\text{ in}^4 > 125\text{ in}^4$ ok

$\phi M_n = 166\text{ kft} > 165\text{ kft}$ ok

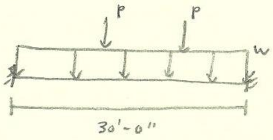
$\phi V_n = 106\text{ K} > 34.7\text{ K}$ ok

note: W16x26 required to support gravity load.

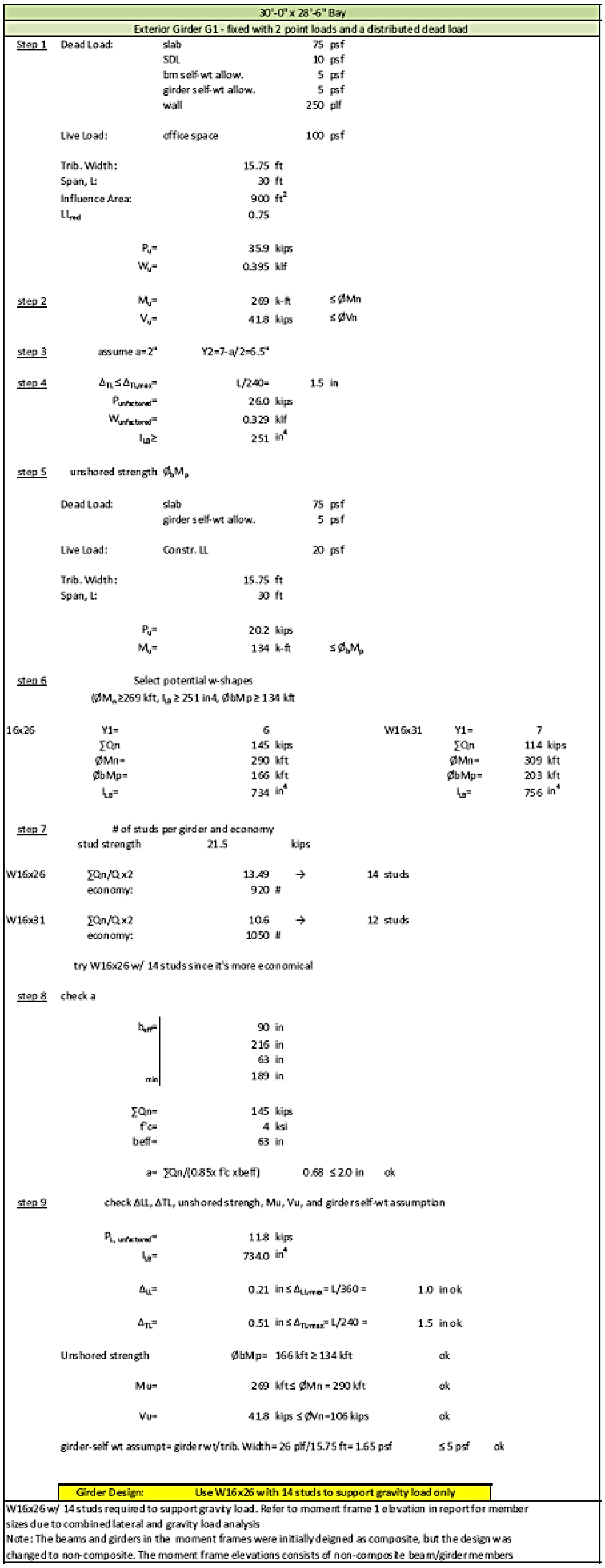
Refer to moment frames B and C elevations in report for member sizes due to combined lateral and gravity analysis

30'-0" x 28'-6" Bay page 4 of 4

Exterior Girder Cal



- refer to excel spreadsheet for design



30'-0" x 28'-6" Bay

Exterior Girder G1 - fixed with 2 point loads and a distributed dead load

Step 1	Dead Load:	slab	75	psf			
		SDL	10	psf			
		bm self-wt allow.	5	psf			
		girder self-wt allow.	5	psf			
		wall	250	plf			
	Live Load:	office space	100	psf			
	Trib. Width:		15.75	ft			
	Span, L:		30	ft			
	Influence Area:		900	ft^2			
	LL_{red}		0.75				
	P_u=		35.9	kips			
	W_u=		0.395	klf			
step 2	M_u=		269	k-ft	≤ ØMn		
	V_u=		41.8	kips	≤ ØVn		
step 3	assume a=2"		Y2=7-a/2=6.5"				
step 4	$\Delta_{TL} \leq \Delta_{TL,max}$=		L/240=		1.5	in	
	$P_{unfactored}$=		26.0	kips			
	$W_{unfactored}$=		0.329	klf			
	$I_{LB} \geq$		251	in^4			
step 5	unshored strength	$Ø_bM_p$					
	Dead Load:	slab	75	psf			
		girder self-wt allow.	5	psf			
	Live Load:	Constr. LL	20	psf			
	Trib. Width:		15.75	ft			
	Span, L:		30	ft			
	P_u=		20.2	kips			
	M_u=		134	k-ft	$\leq Ø_bM_p$		
step 6	Select potential w-shapes						
	($ØM_n$≥269 kft, I_{LB} ≥ 251 in4, ØbMp ≥ 134 kft						
16x26	Y1=		6		W16x31	Y1=	7
	∑Qn		145	kips		∑Qn	114 kips
	ØMn=		290	kft		ØMn=	309 kft
	ØbMp=		166	kft		ØbMp=	203 kft
	I_{LB}=		734	in^4		I_{LB}=	756 in^4
step 7	# of studs per girder and economy						
	stud strength	21.5		kips			
W16x26	∑Qn/Q.x2		13.49	→	14	studs	
	economy:		920	#			
W16x31	∑Qn/Q.x2		10.6	→	12	studs	
	economy:		1050	#			
	try W16x26 w/ 14 studs since it's more economical						
step 8	check a						
	b_{eff}=		90	in			
			216	in			
			63	in			
	min		189	in			
	∑Qn=		145	kips			
	f'c=		4	ksi			
	beff=		63	in			
	a=	∑Qn/(0.85x f'c xbeff)			0.68 ≤ 2.0 in	ok	
step 9	check ΔLL, ΔTL, unshored strengh, Mu, Vu, and girder self-wt assumption						
	$P_{L,\ unfactored}$=		11.8	kips			
	I_{LB}=		734.0	in^4			
	Δ_{LL}=		0.21	in ≤ $\Delta_{LL,max}$= L/360 =	1.0	in ok	
	Δ_{TL}=		0.51	in ≤ $\Delta_{TL,max}$= L/240 =	1.5	in ok	
	Unshored strength		ØbMp=	166 kft ≥ 134 kft		ok	
	Mu=		269	kft≤ ØMn = 290 kft		ok	
	Vu=		41.8	kips ≤ ØVn=106 kips		ok	
	girder-self wt assumpt= girder wt/trib. Width= 26 plf/15.75 ft= 1.65 psf					≤ 5 psf	ok

Girder Design: **Use W16x26 with 14 studs to support gravity load only**

W16x26 w/ 14 studs required to support gravity load. Refer to moment frame 1 elevation in report for member sizes due to combined lateral and gravity load analysis

Note: The beams and girders in the moment frames were initially deigned as composite, but the design was changed to non-composite. The moment frame elevations consists of non-composite beam/girder members

41'-0" X 28'-6" Bay

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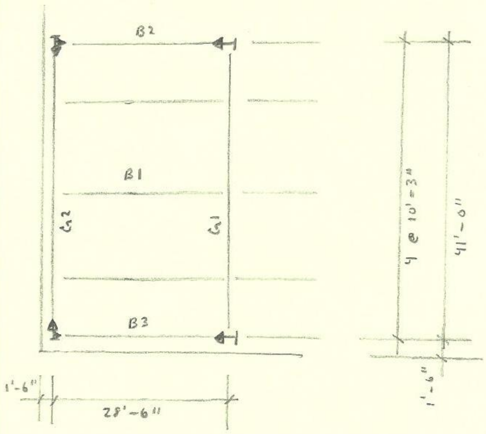
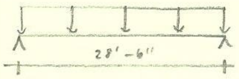


interior bm B1



① $w_D = (75 + 10 + 5)\text{ psf }(10.25\text{ ft}) = 923\text{ plf}$

$$LL = 100\text{ psf} \times \left| \begin{array}{l} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{584.25}} = 0.871 \end{array} \right. = 87.1\text{ psf}$$

$w_L = 87.1\text{ psf }(10.25\text{ ft}) = 893\text{ plf}$

$w_u = 1.2(923) + 1.6(893) = 2536\text{ plf}$

② $M_u = \dfrac{2.536(28.5)^2}{8} = 257\text{ kft}$

$V_u = \dfrac{2.536(28.5)}{2} = 36.1\text{ K}$

③ assume $a = 1"$, $Y2 = 7"$

④ $\Delta_{TL} \leq \Delta_{TL,\max} = L/240$

$I_{LB} \geq \dfrac{5(0.923 + 0.893)(28.5)^4(1728)}{384(29000)(1.425)} = 652\text{ in}^4$

$\Delta_{wc} \leq \Delta_{wc,\max} = L/240$

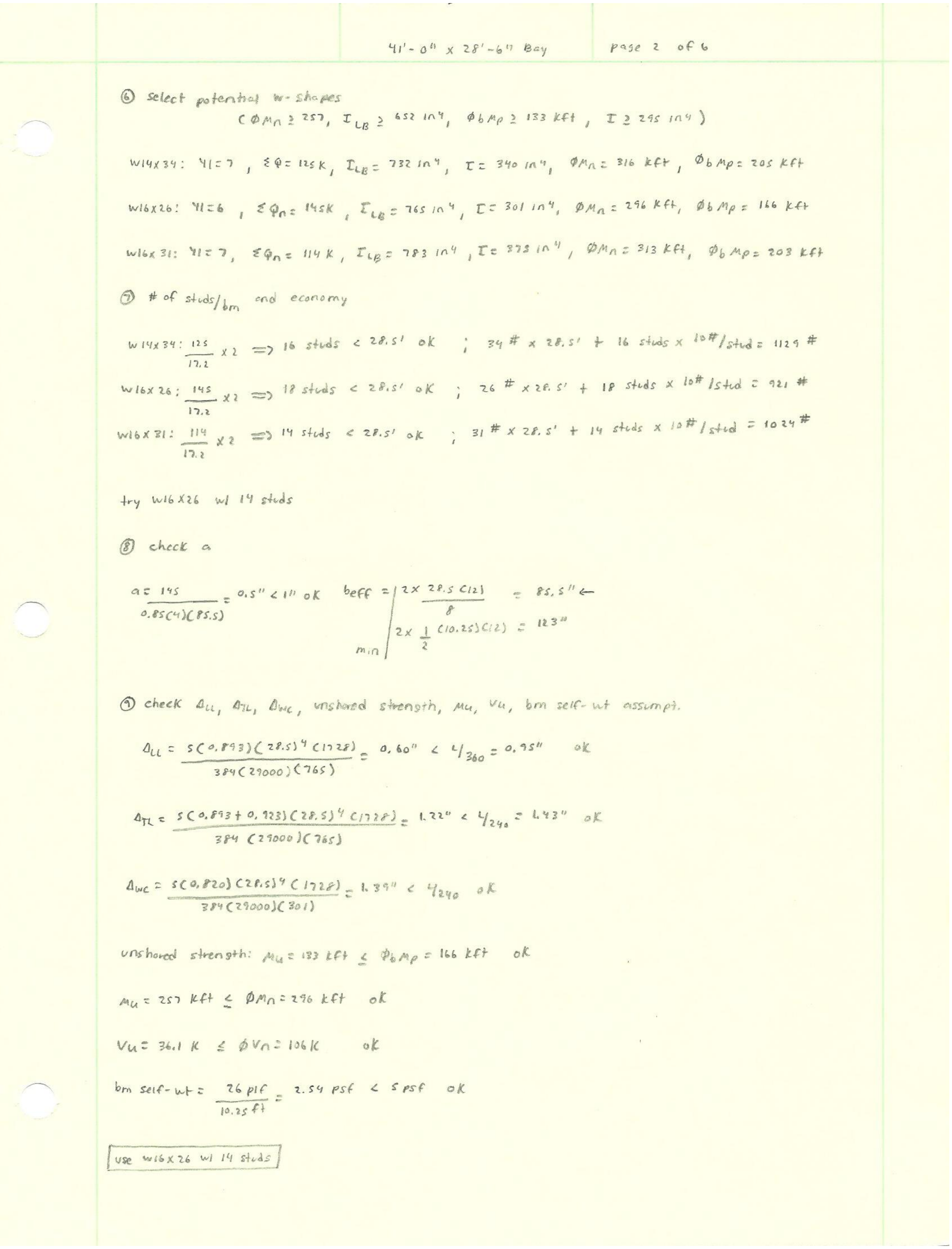
$w_{wc} = (75 + 5)\text{ psf }(10.25\text{ ft}) = 0.820\text{ klf}$

$I_{\text{non-comp.}} = \dfrac{5(0.820)(28.5)^4(1728)}{384(29000)(1.425)} = 295\text{ in}^4$

⑤ required unshore strength, $\phi_b M_p$

$w_u = 1.2(75+5)(10.25) + 1.6(20)(10.25) = 1312\text{ plf}$

$M_u = \dfrac{1.312(28.5)^2}{8} = 133\text{ kft} \leq \phi_b M_p$



41'-0" x 28'-6" Bay page 2 of 6

⑥ select potential w-shapes

$(\phi M_n \geq 257,\ I_{LB} \geq 652\ in^4,\ \phi_b M_p \geq 133\ kft,\ I \geq 295\ in^4)$

W14x34: $Y1 = 7$, $\Sigma Q = 125K$, $I_{LB} = 732\ in^4$, $I = 340\ in^4$, $\phi M_n = 316\ kft$, $\phi_b M_p = 205\ kft$

W16x26: $Y1 = 6$, $\Sigma Q_n = 145K$, $I_{LB} = 765\ in^4$, $I = 301\ in^4$, $\phi M_n = 296\ kft$, $\phi_b M_p = 166\ kft$

W16x31: $Y1 = 7$, $\Sigma Q_n = 114\ K$, $I_{LB} = 783\ in^4$, $I = 375\ in^4$, $\phi M_n = 313\ kft$, $\phi_b M_p = 203\ kft$

⑦ # of studs/bm and economy

W14x34: $\frac{125}{17.2} \times 2 \Rightarrow$ 16 studs < 28.5' ok ; 34# x 28.5' + 16 studs x 10#/stud = 1129 #

W16x26: $\frac{145}{17.2} \times 2 \Rightarrow$ 18 studs < 28.5' ok ; 26# x 28.5' + 18 studs x 10#/stud = 921 #

W16x31: $\frac{114}{17.2} \times 2 \Rightarrow$ 14 studs < 28.5' ok ; 31# x 28.5' + 14 studs x 10#/stud = 1024 #

try W16X26 w/ 14 studs

⑧ check a

$a = \frac{145}{0.85(4)(85.5)} = 0.5'' < 1''$ ok

$b_{eff} = \min \begin{cases} 2 \times \frac{28.5(12)}{8} = 85.5'' \leftarrow \\ 2 \times \frac{1}{2}(10.25)(12) = 123'' \end{cases}$

⑨ check Δ_{LL}, Δ_{TL}, Δ_{WC}, unshored strength, M_u, V_u, bm self-wt assumpt.

$$\Delta_{LL} = \frac{5(0.893)(28.5)^4(1728)}{384(29000)(765)} = 0.60'' < L/360 = 0.95'' \quad ok$$

$$\Delta_{TL} = \frac{5(0.893+0.923)(28.5)^4(1728)}{384(29000)(765)} = 1.22'' < L/240 = 1.43'' \quad ok$$

$$\Delta_{WC} = \frac{5(0.820)(28.5)^4(1728)}{384(29000)(301)} = 1.39'' < L/240 \quad ok$$

unshored strength: $M_u = 133\ kft \leq \phi_b M_p = 166\ kft$ ok

$M_u = 257\ kft \leq \phi M_n = 296\ kft$ ok

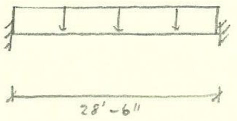
$V_u = 36.1\ K \leq \phi V_n = 106\ K$ ok

bm self-wt $= \frac{26\ plf}{10.25\ ft} = 2.54\ psf < 5\ psf$ ok

$\boxed{\text{use W16x26 w/ 14 studs}}$

41'-0" x 28'-6" Bay page 3 of 6

interior bm B2



① $w_D = (75 + 10 + 5)\,psf\,(10.125\,ft) = 911\,plf$

$$LL = 100\,psf \times \max\left|\begin{array}{l} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{577.13}} = 0.874 \end{array}\right. = 87.4\,psf$$

$w_L = 87.4\,psf\,(10.125\,ft) = 885\,plf$

$w_u = 1.2(911) + 1.6(885) = 2509\,plf$

② $$M_u = \frac{w_u L^2}{12} = \frac{2.509\,klf\,(28.5\,ft)^2}{12} = 170\,kft \le \phi M_n = \phi M_p \Rightarrow Z_x \ge \frac{170(12)}{0.9(50)} = 45.3\,in^3$$

$$V_u = \frac{2.509\,klf\,(28.5)}{2} = 35.8\,K \le \phi V_n$$

③ $\Delta_{TL} \le \Delta_{TL,max} = L/240$

$$I_x \ge \frac{(0.911 + 0.885)(28.5)^4(1728)}{384(29000)(1.425)} = 129\,in^4$$

④ Select non-composite W-shape

W14X30 bm required to support gravity load

w/ $Z_x = 47.3\,in^3 > 45.3\,in^3$ ok

$I_x = 291\,in^4 > 129\,in^4$ ok

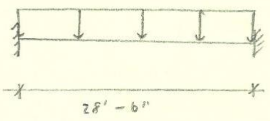
$\phi_b M_{px} = 177\,kft > 170\,kft$ ok

$\phi V_n = 112\,K > 35.8\,K$ ok

- refer to moment frame B elevation in report for member sizes due to combined lateral and gravity analysis

41'-0" x 28'-6" Bay page 4 of 6

exterior bm B3



① $w_D = (75 + 10 + 5)$ psf $(6.625$ ft$) + 250$ plf $= 846.25$ plf

$LL = 100$ psf $A_T = (10.25 + 1.5)$ ft $(28.5$ ft$) = 335$ ft$^2 < 400$ ft^2 ∴ LL reduction not required

$w_L = 100$ psf $(6.625$ ft$) = 662.5$ plf

$w_u = 1.2(846.25) + 1.6(662.5) = 2076$ plf

② $M_u = \frac{2.076(28.5)^2}{12} = 141$ Kft $\le \phi M_n = \phi M_p \Rightarrow Z_x \ge \frac{141(12)}{0.9(50)} = 37.6$ in^3

$V_u = \frac{2.076(28.5)}{2} = 29.6$ K $\le \phi V_n$

③ $\Delta_{TL} \le \Delta_{TL,max} = \ell/240 = \frac{28.5(12)}{240} = 1.425"$

$I_x \ge \frac{(0.846 + 0.663)(28.5)^4(1728)}{384(29000)(1.425)} = 108$ in^4

④ select non-composite w-shape

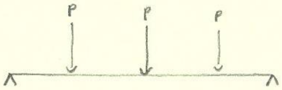
W14X26 bm required to support gravity load w/ $Z_x = 40.2$ in$^3 > 37.6$ in^3 oK
$I_x = 245$ in$^4 > 108$ in^4 oK

$\phi_b M_{px} = 151$ Kft > 141 Kft oK
$\phi V_n = 106$ K > 29.6 K oK

- refer to moment frame A.1 elevation in report for member sizes due to combined lat. and gravity analysis

41'-0" X 28'-6" Bay page 5 of 6

interior girder G1



① $w_D = (75 + 10 + \underset{\substack{\uparrow \\ \text{bm} \\ \text{wt} \\ \text{allow.}}}{5} + \overset{\substack{\text{girder self-wt allow.} \\ \downarrow}}{5})\ \text{psf} = 95\ \text{psf}$

$$LL = 100\ \text{psf} \times \left| \begin{array}{l} 0.50 \\ \\ 0.25 + \dfrac{15}{\sqrt{2603.5}} = 0.544 \end{array} \right. = 54.4\ \text{psf}$$

$w_u = 1.2(95\ \text{psf}) + 1.6(54.4\ \text{psf}) = 201\ \text{psf}$

$P_u = 201\ \text{psf} \left(\dfrac{28.5 + 35}{2}\right) \text{ft}\ (10.25\text{ft}) = 65.4\ \text{K}$

② $M_u = 0.50 PL = 0.50(65.4\ k)(41\ ft) = 1341\ \text{Kft}$ ③ assume $a = 2"$, $Y2 = 6.5"$

$V_u = 1.5P = 1.5(65.4) = 98.1\ k$

④ $\Delta_{TL} \leq \Delta_{TL,max} = L/240 = 41(12)/240 = 2.05"$

$$I_{LB} \geq \frac{0.050\ (48.6\ K)(41\ ft)^3 (1728)}{29000\ (2.05)} = 4868.\ \text{in}^4$$

⑤ required unshored strength, $\phi_b M_p$

$P_u = 1.2\left[(75 + 5)\ \text{psf}\ (10.25\ ft)\right](31.75\ ft) + 1.6(20)(10.25)(31.75) = 41.7K$

$M_u = 0.50(41.7k)(41\ ft) = 855\ \text{Kft} \doteq \phi_b M_p$

⑥ select potential w-shapes
$(\phi M_n \geq 1341\ \text{Kft},\ I_{LB} \geq 4868\ \text{in}^4,\ \phi_b M_p \geq 855\ \text{Kft})$

W24X94: $Y1 = 6$ $\Sigma Q_n = 469$, $\phi M_n = 1450\ \text{Kft}$, $\phi_b M_p = 953\ \text{Kft}$, $I_{LB} = 5140\ \text{in}^4$

W27X84: $Y1 = 6$ $\Sigma Q_n = 452\ K$, $\phi M_n = 1420\ \text{Kft}$, $\phi_b M_p = 915\ \text{kft}$, $I_{LB} = 5460\ \text{in}^4$

⑦ # of studs / girder and economy

W24X94: $\dfrac{469}{21.5K} \times 2 = 44$ studs ; 94# x 41' + 44 studs x 10# / stud = 4294 #

W27X84: $\dfrac{452}{21.5K} \times 2 = 44$ studs ; 84# x 41' + 44 studs x 10# / stud = 3884 #

try W27X84 w/ 44 studs since it's more economical

41'-0" x 28'-6" Bay

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⑧ check a

$$a = \frac{452}{0.85(4)(123)} = 1.08" < 2" \text{ oK}$$

$$b_{eff} = \min \begin{vmatrix} 2 \times \frac{41(12)}{8} = 123" \leftarrow \text{controls} \\ \frac{1}{2}(28.5)(12) + \frac{1}{2}(35)(12) = 381" \\ \frac{41(12)}{8} + \frac{1}{2}(28.5)(12) = 61.5 + 171 = 232.5" \\ \frac{41(12)}{8} + \frac{1}{2}(35)(12) = 61.5 + 210 = 271.5" \end{vmatrix}$$

⑨ check Δ_{LL}, Δ_{TL}, unshored strength, M_u, V_u, girder self-wt assumption

$$\Delta_{LL} = \frac{0.050(17.7K)(41ft)^3(1728)}{29000(5460 in^4)} = 0.67" < L/360 = 1.37" \text{ oK}$$

$$\Delta_{TL} = \frac{0.050(48.6K)(41ft)^3(1728)}{29000(5460 in^4)} = 1.83" < L/240 = 2.05" \text{ oK}$$

unshored strength: $P_u = 1.2[(75psf)(10.25ft) + 84 plf](31.75ft) + 1.6(20psf)(10.25)(31.75) = 42.9 K$

$$M_u = 0.50(42.9K)(41ft) = 879 Kft \le \phi_b M_p = 915 Kft \text{ oK}$$

$M_u = 1341 Kft \le \phi M_n = 1420 Kft$ oK

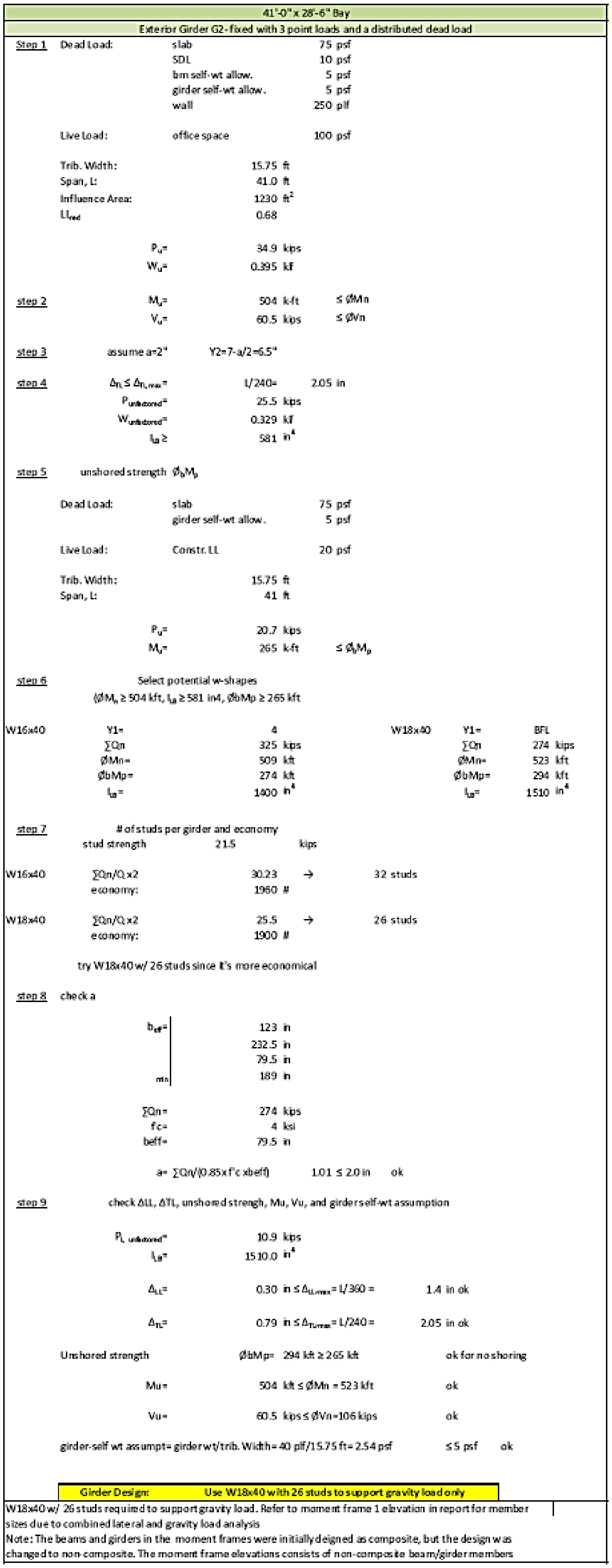
$V_u = 98.1 K \le \phi V_n = 368 K$ oK

$$\text{girder self-wt} = \frac{84 plf}{31.75 ft} = 2.65 psf < 5psf \text{ assumed oK}$$

$\boxed{\text{use W27 x 84 w/ 44 studs}}$

Exterior Girder G2

- refer to excel spread sheet for design



41'-0" x 28'-6" Bay

Exterior Girder G2-fixed with 3 point loads and a distributed dead load

Step 1 Dead Load:
slab 75 psf
SDL 10 psf
bm self-wt allow. 5 psf
girder self-wt allow. 5 psf
wall 250 plf

Live Load: office space 100 psf

Trib. Width: 15.75 ft
Span, L: 41.0 ft
Influence Area: 1230 ft^2
LL_{red} 0.68

P_u= 34.9 kips
W_u= 0.395 klf

step 2
M_u= 504 k-ft ≤ ØMn
V_u= 60.5 kips ≤ ØVn

step 3 assume a=2" Y2=7-a/2=6.5"

step 4
$\Delta_{TL} \le \Delta_{TL,max}$= L/240= 2.05 in
$P_{unfactored}$= 25.5 kips
$W_{unfactored}$= 0.329 klf
$I_{LB} \ge$ 581 in^4

step 5 unshored strength $Ø_bM_p$

Dead Load:
slab 75 psf
girder self-wt allow. 5 psf

Live Load: Constr. LL 20 psf

Trib. Width: 15.75 ft
Span, L: 41 ft

P_u= 20.7 kips
M_u= 265 k-ft ≤ $Ø_bM_p$

step 6 Select potential w-shapes
($ØM_n \ge$ 504 kft, $I_{LB} \ge$ 581 in4, ØbMp ≥ 265 kft

W16x40	Y1=	4		W18x40	Y1=	BFL
	∑Qn	325 kips			∑Qn	274 kips
	ØMn=	509 kft			ØMn=	523 kft
	ØbMp=	274 kft			ØbMp=	294 kft
	I_{LB}=	1400 in^4			I_{LB}=	1510 in^4

step 7 # of studs per girder and economy
stud strength 21.5 kips

W16x40 ∑Qn/Q x2 30.23 → 32 studs
economy: 1960 #

W18x40 ∑Qn/Q x2 25.5 → 26 studs
economy: 1900 #

try W18x40 w/ 26 studs since it's more economical

step 8 check a

b_{eff}=
123 in
232.5 in
79.5 in
min 189 in

∑Qn= 274 kips
f'c= 4 ksi
beff= 79.5 in

a= ∑Qn/(0.85x f'c xbeff) 1.01 ≤ 2.0 in ok

step 9 check ΔLL, ΔTL, unshored strengh, Mu, Vu, and girder self-wt assumption

$P_{L\ unfactored}$= 10.9 kips
I_{LB}= 1510.0 in^4

Δ_{LL}= 0.30 in ≤ $\Delta_{LL,max}$= L/360 = 1.4 in ok

Δ_{TL}= 0.79 in ≤ $\Delta_{TL,max}$= L/240 = 2.05 in ok

Unshored strength ØbMp= 294 kft ≥ 265 kft ok for no shoring

Mu= 504 kft ≤ ØMn = 523 kft ok

Vu= 60.5 kips ≤ ØVn=106 kips ok

girder-self wt assumpt= girder wt/trib. Width= 40 plf/15.75 ft= 2.54 psf ≤5 psf ok

Girder Design: Use W18x40 with 26 studs to support gravity load only

W18x40 w/ 26 studs required to support gravity load. Refer to moment frame 1 elevation in report for member sizes due to combined lateral and gravity load analysis
Note: The beams and girders in the moment frames were initially deigned as composite, but the design was changed to non-composite. The moment frame elevations consists of non-composite beam/girder members

41'-0" x 35'-0" Bay

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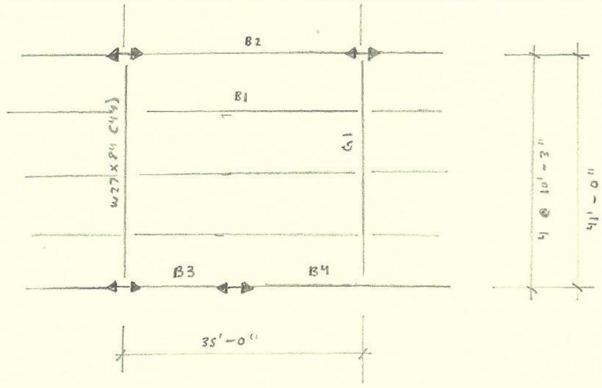
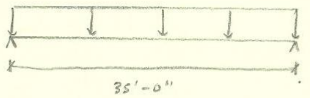


interior bm B1

① $w_D = (75 + 10 + 5)\,psf\,(10.25\,ft) = 922.5\,plf$

$LL_o = 100\,psf$

$LL = 100\,psf \times \left| \begin{array}{l} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{717.5}} = 0.81 \end{array} \right. = 81\,psf$



$w_L = 81\,psf\,(10.25\,ft) = 830.25\,plf$

$w_u = 1.2(922.5) + 1.6(830.25) = 2435\,plf$

② $M_u = \dfrac{2.435\,klf\,(35\,ft)^2}{8} = 373\,kft \leq \phi M_n$

$V_u = \dfrac{2.435\,(35)}{2} = 42.6\,k \leq \phi V_n$

③ assume $a = 1''$, $Y2 = 7''$

④ $\Delta_{TL} \leq \Delta_{TL,max} = {}^{L}/_{240} = 1.75''$

$I_{LB} \geq \dfrac{5(0.923 + 0.830)(35)^4(1728)}{384(29000)(1.75)} = 1166\,in^4$

$\Delta_{wc} \leq \Delta_{wc,max} = {}^{L}/_{240}$

$w_{wc} = (75 + 5)\,psf\,(10.25\,ft) = 0.820\,klf$

$I_{non\text{-}comp.} \geq \dfrac{5(0.820)(35)^4(1728)}{384(29000)(1.75)} = 546\,in^4$

⑤ required unshored strength, $\phi_b M_p$

$W_u = 1.2(75 + 5)(10.25) + 1.6(20)(10.25) = 1312\,plf$

$M_u = \dfrac{1.312\,(35)^2}{8} = 201\,kft \leq \phi_b M_p$

41'-0" x 35'-0" Bay page 2 of 5

⑥ select potential W- shapes

($\phi M_n \geq 373$, $I_{LB} \geq 1166 \text{ in}^4$, $\phi_b M_p \geq 201 \text{ kft}$, $I \geq 546 \text{ in}^4$)

W16x45: $Y1 = 7$, $\Sigma Q_n = 166 \text{ k}$, $I_{LB} = 1190 \text{ in}^4$, $I = 586 \text{ in}^4$, $\phi M_n = 467 \text{ kft}$, $\phi_b M_p = 309 \text{ kft}$

W18x40: $Y1 = 7$, $\Sigma Q_n = 148 \text{K}$, $I_{LB} = 1210 \text{ in}^4$, $I = 612 \text{ in}^4$, $\phi M_n = 446 \text{ kft}$, $\phi_b M_p = 294 \text{ kft}$

⑦ # of studs/bm and economy

W16x45: $\dfrac{166 \text{K}}{17.2 \text{K}} \times 2 \Rightarrow 20$ studs < 35' ok ; $45\# \times 35' + 20 \text{ studs} \times 10\#/\text{stud} = 1775\#$

W18x40: $\dfrac{148 \text{K}}{17.2 \text{K}} \times 2 \Rightarrow 18$ studs < 35' ok ; $40\# \times 35' + 18 \text{ studs} \times 10\#/\text{stud} = 1580\#$

try W18x40 w/ 18 studs since it's more economical

⑧ check a

$$a = \frac{148}{0.85(4)(105)} = 0.41" < 1"$$

$$b_{eff} = \min \left| \begin{array}{l} 2 \times \dfrac{35(12)}{8} = 105" \leftarrow \\ 2 \times \dfrac{1}{2}(10.25)(12) = 123" \\ \dfrac{35(12)}{8} + \dfrac{1}{2}(10.25)(12) = 114" \end{array} \right.$$

⑨ check Δ_{LL}, Δ_{TL}, Δ_{WC}, unshored strength, M_u, V_u, bm self-wt assumption

$$\Delta_{LL} = \frac{5(0.830)(35)^4(1728)}{384(29000)(1210)} = 0.80" < L/360 = 1.17" \text{ ok}$$

$$\Delta_{TL} = \frac{5(1.753)(35)^4(1728)}{384(29000)(1210)} = 1.69" < L/240 = 1.75" \text{ ok}$$

$$\Delta_{WC} = \frac{5(0.820)(35)^4(1728)}{384(29000)(612)} = 1.56" < L/240 = 1.75" \text{ ok}$$

unshored strength: $M_u = 201 \text{ kft} \leq \phi_b M_p = 294 \text{ kft}$ ok

$M_u = 373 \text{ kft} \leq \phi M_n = 446 \text{ kft}$ ok

$V_u = 42.6 \text{ K} \leq \phi V_n = 169 \text{ K}$ ok

bm self-wt $= \dfrac{40 \text{ plf}}{10.25 \text{ ft}} = 3.9$ psf < 5 psf assumed ok

$\boxed{\text{use W18X40 w/ 18 studs}}$

41'-0" X 35'-0" Bay

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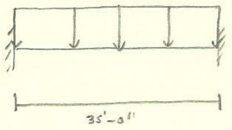
interior bm B2



① $w_D = (75 + 10 + 5)\ \text{psf}\ (10.125\ \text{ft}) = 911.25\ \text{plf}$

$L_o = 100\ \text{psf}$

$$LL = 100\ \text{psf} \times \left| \begin{array}{l} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{708.75}} = 0.813 \end{array} \right. = 81.3\ \text{psf}$$

$w_L = 81.3\ \text{psf}\ (10.125\text{ft}) = 823\ \text{plf}$

$w_u = 1.2(911.25) + 1.6(823) = 2410\ \text{plf}$

② $M_u = \dfrac{2.410\ \text{klf}\ (35\ \text{ft})^2}{12} = 246\ \text{Kft} \le \phi M_n = \phi M_p \Rightarrow Z_x \ge \dfrac{246(12)}{0.9(50)} = 65.6\ in^3$

$V_u = \dfrac{2.410\ (35)}{2} = 42.2\ K \le \phi V_n$

③ $\Delta_{TL} \le \Delta_{TL,\,max} = l/240 = 1.75''$

$$I_x \ge \frac{(0.911 + 0.823)(35)^4(1728)}{384(29000)(1.75)} = 231\ in^4$$

④ select non-composite W-shape

W18X35 bm required to support gravity load

w/ $Z_x = 66.5\ in^3 > 65.6\ in^3$ ok

$I_x = 510\ in^4 > 231\ in^4$ ok

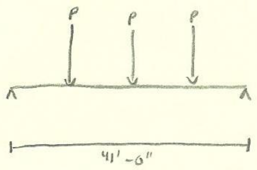
$\phi_b M_{px} = 249\ \text{Kft} > 246\ \text{Kft}$ ok

$\phi V_n = 159\ K > 42.2\ K$ ok

- refer to moment frame B elevation in report for member sizes due to combined lat. and gravity analysis

41'-0" x 35'-0" Bay page 4 of 5

interior girder Cal



① $w_D = (75 + 10 + 5 + 5) = 95$ psf

girder self-wt allow.

bm wt allow.

$$LL = 100 \text{ psf} \times \left| \begin{matrix} 0.50 \\ 0.25 + \frac{15}{\sqrt{2665}} = 0.54 \end{matrix} \right. = 54.1 \text{ psf}$$

$w_u = 1.2(95) + 1.6(54.1) = 200.6$ psf

$P_u = 200.6 \text{ psf} \left(\frac{30 + 35}{2}\right) \text{ft} \, (10.25 \text{ ft}) = 66.8 \text{ K}$

② $M_u = 0.5PL = 0.5(66.8 \text{ K})(41 \text{ ft}) = 1369 \text{ Kft}$

$V_u = 1.5P = 1.5(66.8 \text{ k}) = 100 \text{ k}$

③ assume $a = 2$, $Y2 = 6.5''$

④ $\Delta_{TL} \leq \Delta_{TL,max} = {}^{\ell}/_{240} = 2.05''$

$$I_{LB} \geq \frac{0.050 (49.7k)(41ft)^3 (1728)}{29000(2.05)} = 4978 \text{ in}^4$$

⑤ unshored strength, $\phi_b M_p$

$P_u = 1.2[(75+5) \text{psf} (10.25 \text{ft})](32.5 \text{ ft}) + 1.6(20)(10.25)(32.5) = 42.6 \text{ K}$

$M_u = 0.50(42.6 \text{k})(41 \text{ft}) = 873 \text{ kft} \leq \phi_b M_p$

⑥ select potential w-shapes

$(\phi M_n \geq 1369 \text{ kft}, \; I_{LB} \geq 4978 \text{ in}^4, \; \phi_b M_p \geq 873 \text{ kft})$

W24x94: $Y1 = 6$, $I_{LB} = 5140 \text{ in}^4$, $\phi M_n = 1450$ kft, $\phi_b M_p = 953$ kft, $\Sigma Q = 469$ K

W27x84: $Y1 = 6$, $\Sigma Q = 452$ K, $I_{LB} = 5460 \text{ in}^4$, $\phi M_n = 1420$ kft, $\phi_b M_p = 915$ kft

⑦ # of studs / girder and economy

W24x94: $\frac{469 K}{21.5 k} \times 2 \Rightarrow 44$ studs ; 94# x 41' + 44 studs x 10 #/stud = 4294 #

W27x84: $\frac{452 K}{21.5 K} \times 2 \Rightarrow 44$ studs ; 84 # x 41' + 44 studs x 10 # / stud = 3884 #

try W27x84 w/ 44 studs

41'-0" x 35'-0" Bay

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⑧ check a

$$a = \frac{452}{0.85(4)(123)} = 1.08'' < 2'' \text{ ok}$$

$$b_{eff} = \min \begin{cases} 2 \times \frac{41(12)}{8} = 123'' \leftarrow \\ \frac{1}{2}(35)(12) + \frac{1}{2}(30)(12) = 390'' \end{cases}$$

⑨ check Δ_{LL}, Δ_{TL}, unshored strength, M_u, V_u, girder self-wt assumption

$$\Delta_{LL} = \frac{0.050(18K)(41\text{ ft})^3(1728)}{29000(5460)} = 0.68'' < L/360 = 1.37''$$

$$\Delta_{TL} = \frac{0.050(49.7K)(41\text{ft})^3(1728)}{29000(5460)} = 1.87'' < L/240 = 2.05''$$

unshored strength: $P_u = 1.2[75\text{psf}(10.25\text{ft}) + 84\text{ plf}](32.5\text{ft}) + 1.6(20\text{ psf})(10.25)(32.5) = 43.9\text{ K}$

$M_u = 0.50(43.9K)(41\text{ft}) = 900\text{ Kft} \leq \phi_b M_p = 915\text{ Kft}$ ok

$M_u = 1369\text{ Kft} \leq \phi M_n = 1420\text{ Kft}$ ok

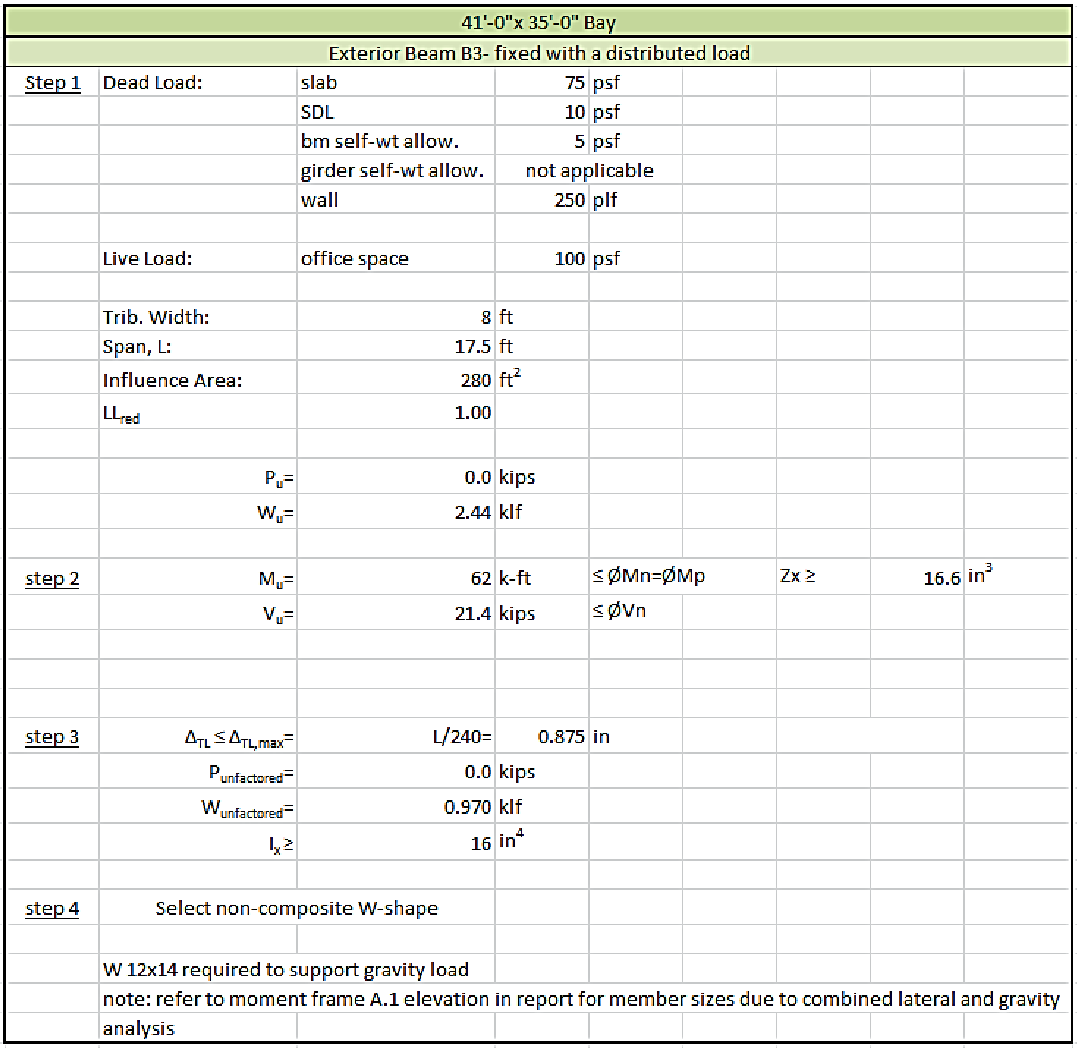
$V_u = 100K \leq \phi V_n = 368K$ ok

$$\text{girder self-wt} = \frac{84\text{plf}}{32.5\text{ft}} = 2.58\text{ psf} < 5\text{psf assumed ok}$$

$\boxed{\text{use W27x84 w/ 44 studs}}$

Exterior bm B3

- refer to excel spreadsheet for design



41'-0"x 35'-0" Bay								
Exterior Beam B3- fixed with a distributed load								
Step 1	Dead Load:	slab	75	psf				
		SDL	10	psf				
		bm self-wt allow.	5	psf				
		girder self-wt allow.	not applicable					
		wall	250	plf				
	Live Load:	office space	100	psf				
	Trib. Width:	8	ft					
	Span, L:	17.5	ft					
	Influence Area:	280	ft^2					
	LL_{red}	1.00						
	P_u=	0.0	kips					
	W_u=	2.44	klf					
step 2	M_u=	62	k-ft	≤ØMn=ØMp		Zx ≥	16.6	in^3
	V_u=	21.4	kips	≤ØVn				
step 3	$\Delta_{TL} \leq \Delta_{TL,max}$=	L/240=	0.875	in				
	$P_{unfactored}$=	0.0	kips					
	$W_{unfactored}$=	0.970	klf					
	$I_x \geq$	16	in^4					
step 4	Select non-composite W-shape							
	W 12x14 required to support gravity load							
	note: refer to moment frame A.1 elevation in report for member sizes due to combined lateral and gravity							
	analysis							

41'-0" x 30'-0" Bay

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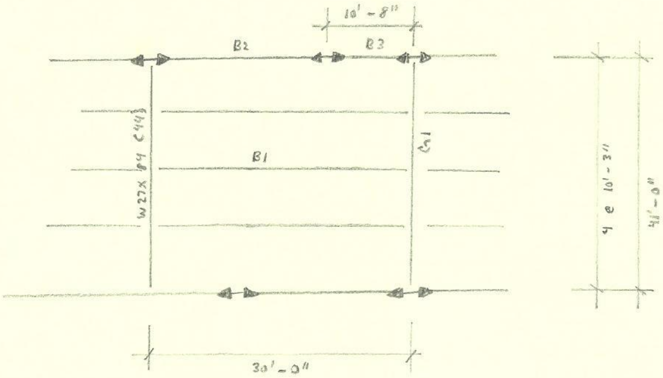
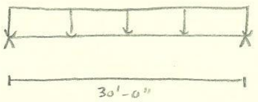


interior bm B1



① $w_D = (75 + 10 + 5)\ psf\ (10.25\ ft) = 922.5\ plf$

$L_0 = 100\ psf$

$LL = 100\ psf \times \left| \begin{array}{l} 0.50 \\ 0.25 + \frac{15}{\sqrt{615}} = 0.855 \end{array} \right. = 85.5\ psf$

$w_L = 85.5\ psf\ (10.25\ ft) = 876\ plf$

$w_u = 1.2(922.5) + 1.6(876) = 2509\ plf$

② $M_u = \frac{2.509(30)^2}{8} = 282\ kft \leq \phi M_n$

$V_u = \frac{2.509(30)}{8} = 37.6\ K \leq \phi V_n$

③ assume $a = 1"$, $Y2 = 7"$

④ $\Delta_{TL} \leq \Delta_{TL,max} = l/240 = \frac{30(12)}{240} = 1.5"$

$I_{LB} \geq \frac{5(0.923 + 0.876)(30)^4(1728)}{384(29000)(1.5)} = 754\ in^4$

$\Delta_{wc} \leq \Delta_{wc,max} = l/240$

$w_{wc} = (75 + 5)\ psf\ (10.25\ ft) = 0.820\ klf$

$I_{non\text{-}comp} \geq \frac{5(0.820)(30)^4(1728)}{384(29000)(1.5)} = 344\ in^4$

⑤ unshored strength, $\phi_b M_p$

$w_u = 1.2(75 + 5)\ psf\ (10.25)\ ft + 1.6(20\ psf)(10.25\ ft) = 1312\ plf$

$M_u = \frac{1.312(30)^2}{8} = 148 \leq \phi_b M_p$

41'-0" X 30'-0" Bay

page 2 of 6

⑥ select potential w-shapes

$(\phi M_n \geq 282 \text{ kft},\ I_{LB} \geq 754 \text{ in}^4,\ \phi_b M_p \geq 148 \text{ kft},\ I_{NC} \geq 344 \text{ in}^4)$

W16X31; $Y1 = 7$ $\Sigma Q_n = 114\text{k},\ I_{LB} = 783 \text{ in}^4,\ I = 375 \text{ in}^4,\ \phi M_n = 313 \text{ kft},\ \phi_b M_p = 203 \text{ kft}$

⑦ # of studs/bm and economy

W16X31: $\frac{114\text{k}}{17.2\text{k}} \times 2 \Rightarrow$ 14 studs < 30' ok ; 31# x 30' + 14 studs x 10#/stud = 1070 #

⑧ check a

$a = \frac{114}{0.85(4)(90)} = 0.37" < 1"$ ok

$$b_{eff} = \min \left| \begin{array}{l} 2 \times \frac{30(12)}{8} = 90" \leftarrow \\ 2 \times \frac{1}{2}(10.25)(12) = 123" \\ \frac{30(12)}{8} + \frac{1}{2}(10.25)(12) = 106.5" \end{array} \right.$$

⑨ check Δ_{LL}, Δ_{TL}, Δ_{WC}, unshored strength, M_u, V_u, bm self-wt assumption

$$\Delta_{LL} = \frac{5(0.876)(30)^4(1728)}{384(29000)(783)} = 0.70" < L/360 = 1" \text{ OK}$$

$$\Delta_{TL} = \frac{5(1.80)(30)^4(1728)}{384(29000)(783)} = 1.44" < L/240 = 1.5" \text{ ok}$$

$$\Delta_{WC} = \frac{5(0.820)(30)^4(1728)}{384(29000)(375)} = 1.37" < L/240 = 1.5" \text{ ok}$$

unshored strength: $M_u = 148 \text{ Kft} \leq \phi_b M_p = 203 \text{ Kft}$ ok

$M_u = 282 \text{ Kft} \leq \phi M_n = 313 \text{ Kft}$ ok

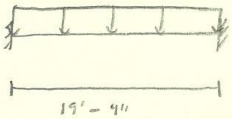
$V_u = 37.6 \text{ Kft} \leq \phi V_n = 131\text{K}$ ok

bm self-wt $= \frac{31 \text{ plf}}{10.25 \text{ ft}} = 3.02$ psf < 5psf assumed ok

$\boxed{\text{use W16X31 w/ 14 studs}}$

41'-0" X 30'-0" Bay page 3 of 6

interior bm B2



① $w_D = (75 + 10 + 5)$ psf $(10.125$ ft$) = 911.25$ plf

$L_o = 100$ psf , I.A. $= (10.25 + 10)$ ft $(19.33$ ft$) = 391.4$ ft^2 < 400 ft^2 ∴ LL reduction not required

$w_L = 100(10.125) = 1012.5$ plf

$w_u = 1.2(911.25) + 1.6(1012.5) = 2714$ plf

② $M_u = \frac{2.714 \text{ klf} (19.33)^2}{12} = 84.5 \text{ kft} \le \phi M_n = \phi M_p \Rightarrow Z_x \ge \frac{84.5(12)}{0.9(50)} = 22.5 \text{ in}^3$

$V_u = \frac{2.714 \text{ klf} (19.33 \text{ ft})}{2} = 26.2 \text{ K} \le \phi V_n$

③ $\Delta_{TL} \le \Delta_{TL,max} = L/240 = 0.97''$

$I_x \ge \frac{(0.911 + 1.013)(19.33)^4 (1728)}{384(29000)(0.97)} = 43 \text{ in}^4$

④ select non-composite W-shape

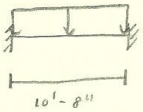
W12X19 bm required to support w/ $Z_x = 24.7 \text{ in}^3 > 22.5 \text{ in}^3$ ok
gravity load $I_x = 130 \text{ in}^4 > 43 \text{ in}^4$ ok

$\phi_b M_{px} = 92.6 \text{ Kft} > 84.5 \text{ Kft}$ ok
$\phi V_n = 86 \text{ K} > 26.2 \text{ K}$ ok

- refer to moment frame B elevation in report for member sizes due to combined lat. and gravity load analysis

41'-0" x 30'x 0" Bay

interior bm B3



① $w_D = (75 + 10 + 5)\text{ psf }(10.125\text{ ft}) = 911.25\text{ plf}$

$L_o = 100\text{ psf}$, $T.A. = (10.25 + 10)\text{ft }(10.67\text{ ft}) = 216\text{ ft}^2 < 400\text{ ft}^2$ ∴ LL reduction not required

$w_L = 100\text{ psf }(10.125\text{ ft}) = 1012.5\text{ plf}$

$w_u = 1.2(911.25) + 1.6(1012.5) = 2714\text{ plf}$

② $M_u = \frac{2.714\text{ klf }(10.67\text{ ft})^2}{12} = 25.7\text{ kft} \leq \phi M_n = \phi M_p \Rightarrow Z_x \geq \frac{25.7(12)}{0.9(50)} = 6.9\text{ in}^3$

$V_u = \frac{2.714(10.67)}{2} = 14.5\text{ k} \leq \phi V_n$

③ $\Delta_{TL} \leq \Delta_{TL,max} = l/240 = 0.53"$

$I_{LB} \geq \frac{(0.912 + 1.013)(10.67)^4(1728)}{384(29000)(0.53)} = 7.3\text{ in}^4$

④ select non-composite W-shape

W8x10 bm required to support gravity load

w/ $Z_x = 8.87\text{ in}^3 > 6.9\text{ in}^3$ ok

$I_x = 30.8\text{ in}^4 > 7.3\text{ in}^4$ ok

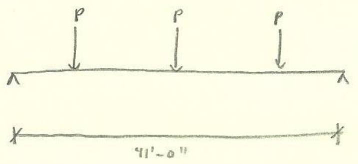
$\phi_b M_{px} = 32.9\text{ kft} > 25.7\text{ kft}$ ok

$\phi V_n = 40.2\text{ k} > 14.5\text{k}$ ok

- refer to moment frame B elevation in report for member sizes due to combined lat. and gravity load analysis

41'-0" x 30'-0" Bay page 5 of 6

interior girder Cal



① $w_D = (75 + 10 + 5 + 5) = 95$ psf

girder self-wt allow.

bm wt. allow.

$$LL = 100 \text{ psf} \times \max \left| \begin{array}{l} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{2255}} = 0.566 \end{array} \right. = 56.6 \text{ psf}$$

$w_u = 1.2(95) + 1.6(56.6) = 204.6$ psf

$P_u = 204.6 \text{ psf} \left(\dfrac{30 + 25}{2}\right) \text{ft} \, (10.25 \text{ft}) = 57.7 \text{ k}$

② $M_u = 0.5(57.7 \text{ k})(41 \text{ft}) = 1183 \text{ kft} \le \phi M_n$

$V_u = 1.5(57.7 \text{ k}) = 86.6 \text{ k} \le \phi V_n$

③ assume $a = 2''$ $Y2 = 6.5''$

④ $\Delta_{TL} \le \Delta_{TL,max} = L/240 = 2.05''$

$$I_{LB} \ge \frac{0.050(42.7 \text{ k})(41 \text{ft})^3(1728)}{29000(2.05)} = 4277 \text{ in}^4$$

⑤ unshored strength, $\phi_b M_p$

$P_u = 1.2[(75+5) \text{ psf } (10.25 \text{ ft})](27.5 \text{ ft}) + 1.6(20 \text{ psf})(10.25 \text{ft})(27.5 \text{ ft}) = 36.1 \text{ K}$

$M_u = 0.50(36.1 \text{ k})(41 \text{ft}) = 740 \le \phi_b M_p$

⑥ select potential w-shapes

$(\phi M_n \ge 1183 \text{ kft}, \; I_{LB} \ge 4277 \text{ in}^4, \; \phi_b M_p \ge 740 \text{ kft})$

W24x84: $Y1 = 6$, $\Sigma Q_n = 425 \text{k}$, $I_{LB} = 4550 \text{ in}^4$, $\phi M_n = 1290$ kft, $\phi_b M_p = 840$ kft

W27x84: $Y1 = 7$, $\Sigma Q_n = 309 \text{k}$, $I_{LB} = 4800 \text{ in}^4$, $\phi M_n = 1300$ kft, $\phi_b M_p = 915$ kft

⑦ # of studs/girder and economy

W24x84: $\dfrac{425 \text{k}}{21.5 \text{k}} \times 2 \Rightarrow 40$ studs ; $84^{\#} \times 41' + 40 \text{ studs} \times 10^{\#}/\text{stud} = 3844^{\#}$

W27x84: $\dfrac{309}{21.5} \times 2 \Rightarrow 30$ studs ; $84^{\#} \times 41' + 30 \text{ studs} \times 10^{\#}/\text{stud} = 3744^{\#}$

try W27x84 w/ 30 studs

41'-0" x 30'-0" Bay

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⑧ check a

$$a = \frac{309}{0.85(4)(123)} = 0.74'' < 2'' \quad ok$$

$$b_{eff} = \min \left| \begin{array}{l} 2 \times \frac{41(12)}{8} = 123'' \leftarrow \\ \frac{1}{2}(30)(12) + \frac{1}{2}(25)(12) = 330'' \end{array} \right.$$

⑨ check Δ_{LL}, Δ_{TL}, unshored strength, M_u, V_u, girder self-wt assumption

$$\Delta_{LL} = \frac{0.050(16K)(41ft)^3(1728)}{29000(4800\ in^4)} = 0.68'' < L/360 = 1.37'' \quad ok$$

$$\Delta_{TL} = \frac{0.050(42.7\ K)(41ft)^3(1728)}{29000(4800\ in^4)} = 1.83'' < L/240 = 2.05'' \quad ok$$

unshored strength: $P_u = 1.2\,[(75\ psf)(10.25ft) + 84\ plf\,](27.5\ ft) + 1.6(20\ psf)(10.25ft)(27.5\ ft) = 37.2\ K$

$M_u = 0.50(37.2\ K)(41\ ft) = 763\ Kft < \phi_b M_p = 915\ Kft$ ok

$M_u = 1183 < \phi M_n = 1300\ Kft$ ok

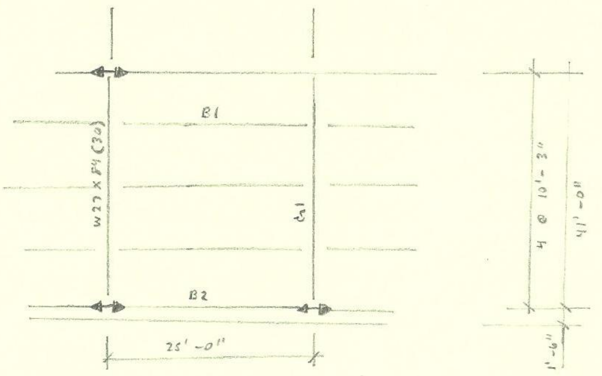
$V_u = 86.6\ K < \phi V_n = 368\ K$ ok

girder self-wt $= \frac{84\ plf}{27.5\ ft} = 3.05\ psf < 5\ psf$ assumed ok

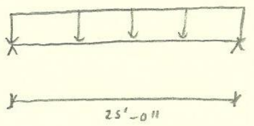
$\boxed{\text{use W27x84 w/ 30 studs}}$

41'-0" x 25'-0" Bay

page 1 of 5



interior bm B1



① $w_D = (75 + 10 + 5)\ psf\ (10.25\ ft) = 922.5\ plf$

$L_0 = 100\ psf$

$LL = 100\ psf \times \left| \begin{array}{l} 0.50 \\ 0.25 + \frac{15}{\sqrt{512.5}} = 0.913 \end{array} \right. = 91.3\ psf$

$w_L = 91.3\ psf\ (10.25\ ft) = 936\ plf$

$w_u = 1.2(922.5) + 1.6(936) = 2605\ plf$

② $M_u = \frac{2.605\ klf\ (25 ft)^2}{8} = 204\ kft \leq \phi M_n$

$V_u = \frac{2.605\ (25)}{2} = 32.7\ K \leq \phi V_n$

③ assume $a = 1"$, $Y2 = 7"$

④ $\Delta_{TL} \leq \Delta_{TL,max} = L/240 = 1.25"$

$I_{LB} \geq \frac{5(0.923 + 0.936)(25)^4(1728)}{384(29000)(1.25)} = 451\ in^4$

$\Delta_{wc} \leq \Delta_{wc,max} = L/240$

$w_{wc} = (75 + 5)\ psf\ (10.25\ ft) = 0.820\ klf$

$I_{non\text{-}comp} \geq \frac{5(0.820)(25)^4(1728)}{384(29000)(1.25)} = 199\ in^4$

⑤ unshored strength, $\phi_b M_p$

$w_u = 1.2(75 + 5)\ psf\ (10.25 ft) + 1.6(20\ psf)(10.25 ft) = 1312\ plf$

$M_u = \frac{1.312\ (25)^2}{8} = 103\ kft \leq \phi_b M_p$

41'-0" x 25'-0" Bay

pase 2 of 5

⑥ select potential w-shape

$(\phi M_n \geq 204 \text{ kft}, I_{LB} \geq 451 \text{ in}^4, \phi_b M_p \geq 103 \text{ kft}, I_{NC} \geq 199 \text{ in}^4)$

w12x26: $Y1 = 7$, $\Sigma Q_n = 95.6$ K, $I_{LB} = 467 \text{ in}^4$, $I = 204 \text{ in}^4$, $\phi M_n = 218$ kft, $\phi_b M_p = 140$ kft

w14x22: $Y1 = 6$, $\Sigma Q_n = 119$k, $I_{LB} = 533 \text{ in}^4$, $I = 199 \text{ in}^4$, $\phi M_n = 225$ kft, $\phi_b M_p = 125$ kft

⑦ # of studs/bm and economy

w12x26: $\frac{95.6 \text{ k}}{17.2 \text{ k}} \times 2 \Rightarrow$ 12 studs < 25' ok ; 26# x 25' + 12 studs x 10 #/stud = 770#

w14x22: $\frac{119\text{k}}{17.2\text{k}} \times 2 \Rightarrow$ 14 studs < 25' ok ; 22# x 25' + 14 studs x 10# /stud = 690 #

try w14x22 w/ 14 studs since it's more economical

⑧ check a

$$a = \frac{119}{0.85(4)(75)} = 0.47'' < 1'' \text{ ok}$$

$$b_{eff} = \min \left| \begin{array}{l} 2 \times \frac{25(12)}{8} = 75'' \leftarrow \\ 2 \times \frac{1}{2}(10.25)(12) = 123'' \end{array} \right.$$

⑨ check Δ_{LL}, Δ_{TL}, Δ_{wc}, unshored strength, M_u, V_u, bm-self-wt

$$\Delta_{LL} = \frac{5(0.936)(25)^4(1728)}{384(29000)(533)} = 0.53'' < L/360 = 0.83'' \text{ ok}$$

$$\Delta_{TL} = \frac{5(1.86)(25)^4(1728)}{384(29000)(533)} = 1.06'' < L/240 = 1.25'' \text{ OK}$$

$$\Delta_{wc} = \frac{5(0.820)(25)^4(1728)}{384(29000)(199)} = 1.25'' \leq L/240 = 1.25'' \text{ ok}$$

unshored strength : $M_u = 103 \text{kft} \leq \phi_b M_p = 125$ kft ok

$M_u = 204$ kft $\leq \phi M_n = 225$ kft ok

$V_u = 32.7$ K $\leq \phi V_n = 94.5$ K ok

bm self-wt $= \frac{22 \text{ plf}}{10.25 \text{ ft}} = 2.15$ psf < 5 psf assumed ok

$\boxed{\text{use W14 x 22 w/ 14 studs}}$

41'-0" x 25'-0" Bay

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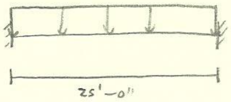
exterior bm B2



curtain wall wt

① $w_D = (10 + 75 + 5)\text{ psf }(6.625\text{ ft}) + 250\text{ plf} = 846.25\text{ plf}$

$LL = 100\text{ psf}$, $I.A. = (10.25 + 1.5)\text{ ft }(25\text{ ft}) = 293.75\text{ ft}^2 < 400\text{ ft}^2$ ∴ LL reduction not required

$w_L = 100(6.625) = 662.5\text{ plf}$

$w_u = 1.2(846.25) + 1.6(662.5) = 2076\text{ plf}$

② $M_u = \dfrac{2.076\text{ klf }(25\text{ ft})^2}{12} = 108\text{ kft} \le \phi M_n = \phi M_p \Rightarrow Z_x \ge \dfrac{108(12)}{0.9(50)} = 28.8\text{ in}^2$

$V_u = \dfrac{2.076(25)}{2} = 26\text{ K} \le \phi V_n$

③ $\Delta_{TL} \le \Delta_{TL,max} = L/240 = 1.25''$

$I_{LB} \ge \dfrac{(0.846 + 0.663)(25)^4(1728)}{384(29000)(1.25)} = 73\text{ in}^4$

④ select non-composite W-shape

W12x22 required to support gravity load

w/ $Z_x = 29.3\text{ in}^3 > 28.8\text{ in}^3$ ok

$I_x = 156\text{ in}^4 > 73\text{ in}^4$ ok

$\phi_b M_{p_x} = 110\text{ Kft} > 108\text{ kft}$ ok

$\phi V_n = 95.9\text{ K} > 26\text{ K}$ ok

- refer to moment frame A.1 elevation in report for member sizes due to combined lat. and gravity load analysis

41'-0" x 25'-0" Bay

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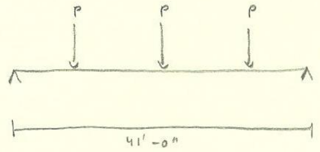
interior girder Cal



girder self-wt allow.

① $W_D = (75 + 10 + 5 + 5) = 95$ psf

bm wt allow.

$$LL = 100 \text{ psf} \times \max \left| \begin{array}{l} 0.50 \\ 0.25 + \frac{15}{\sqrt{1742.5}} = 0.61 \end{array} \right. = 61 \text{ psf}$$

$w_u = 1.2(95) + 1.6(61) = 211.6$ psf

$$P_u = 211.6 \text{ psf} \left(\frac{25 + 17.5}{2}\right) \text{ft} \; (10.25 \text{ ft}) = 46.1 \text{ K}$$

② $M_u = 0.5(46.1)(41) = 945$ Kft $\leq \phi M_n$

$V_u = 1.5(46.1 \text{ K}) = 69.2$ K

③ assume $a = 1''$, $Y2 = 6.5''$

④ $\Delta_{TL} \leq \Delta_{TL,max} = L/240 = 2.05''$

$$I_{LB} \geq \frac{0.050(34K)(41ft)^3(1728)}{29000(2.05)} = 3406 \text{ in}^4$$

⑤ unshored strength, $\phi_b M_p$

$P_u = 1.2[(75 + 5) \text{ psf } (10.25 \text{ ft})](21.25 \text{ ft}) + 1.6(20 \text{ psf})(10.25 \text{ ft})(21.25 \text{ ft}) = 27.9$ K

$M_u = 0.50(27.9 \text{ K})(41\text{ft}) = 572$ Kft $\leq \phi_b M_p$

⑥ select potential w-shapes

$(\phi M_n \geq 945 \text{ kft}, \; I_{LB} \geq 3406 \text{ in}^4, \; \phi_b M_p \geq 572 \text{ kft})$

W24 x 68: $Y1 = 6$, $\Sigma Q_n = 366$K, $I_{LB} = 3640$ in^4, $\phi M_n = 1050$ kft, $\phi_b M_p = 664$ kft

W24x76: $Y1 = 7$, $\Sigma Q_n = 280$ K, $I_{LB} = 3630$ in^4, $\phi M_n = 1070$ kft, $\phi_b M_p = 750$ kft

41'-0" x 25'-0" Bay

page 5 of 5

⑦ # of studs/girder and economy

$$W24x68: \frac{366\ K}{21.5\ K} \times 2 \Rightarrow 36 \text{ studs} \; ; \; 68^{\#} \times 41' + 36 \text{ studs} \times 10^{\#}/\text{stud} = 3148^{\#}$$

$$W24x76: \frac{280\ K}{21.5\ K} \times 2 \Rightarrow 28 \text{ studs} \; ; \; 76^{\#} \times 41' \times 28 \text{ studs} \times 10^{\#}/\text{stud} = 3396^{\#}$$

try W24x68 w/ 36 studs since it's more economical

⑧ check a

$$a = \frac{366}{0.85(4)(123)} = 0.88" < 2" \text{ OK}$$

$$b_{eff} = \min \left| \begin{array}{l} 2 \times \frac{41(12)}{8} = 123" \leftarrow \\ \frac{1}{2}(25)(12) + \frac{1}{2}(17.5)(12) = 255" \end{array} \right.$$

⑨ check Δ_{LL}, Δ_{TL}, unshored strength, M_u, V_u, girder self-wt assumpt.

$$\Delta_{LL} = \frac{0.050(13.3K)(41\ ft)^3(1728)}{29000(3640\ in^4)} = 0.75" \leq L/360 = 1.37" \text{ OK}$$

$$\Delta_{TL} = \frac{0.050(34K)(41\ ft)^3(1728)}{29000(3640\ in^4)} = 1.92" \leq L/240 = 2.05" \text{ OK}$$

unshored strength: $P_u = 1.2\,[(75\ psf)(10.25\ ft) + 68\ plf\,](21.25\ ft) + 1.6\,(20\ psf)(10.25\ ft)(21.25\ ft) = 28.3K$

$M_u = 0.50(28.3K)(41 ft) = 580.2\ kft \leq \phi_b M_p = 664\ kft$ OK

$M_u = 945\ kft \leq \phi M_n = 1050\ kft$ OK

$V_u = 69.2\ K \leq \phi V_n = 295\ K$ OK

$$\text{self-wt} = \frac{68\ plf}{21.25\ ft} = 3.2\ psf \leq 5\ psf \text{ assumed OK}$$

$$\boxed{\text{use W24 X 68 w/ 36 studs}}$$

35'-0" X 31'-0" Bay

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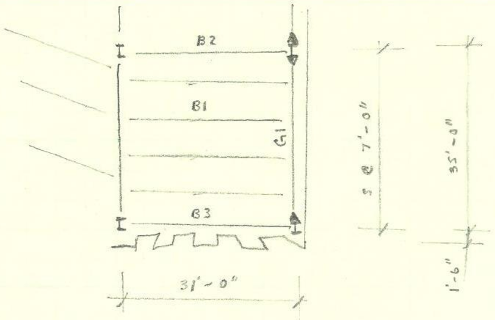
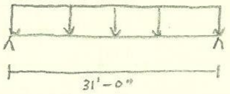


interior bm B1



① $w_D = (75 + 10 + 5)\ psf\ (7 ft) = 630\ plf$

$$LL = 100\ psf \times \left| \begin{array}{l} 0.50 \\ 0.25 + \frac{15}{\sqrt{434}} = 0.97 \end{array} \right. = 97\ psf$$

$w_L = 97\ (7 ft) = 679\ plf$

$w_u = 1.2(630) + 1.6(679) = 1842\ plf$

② $M_u = \frac{1.842\ klf\ (31 ft)^2}{8} = 221\ kft \leq \phi M_n$

$V_u = \frac{1.842\ (31)}{2} = 28.6\ k \leq \phi V_n$

③ assume $a = 1"$, $Y2 = 7"$

④ $\Delta_{TL} \leq \Delta_{TL,max} = {}^{L}/_{240} = 1.55"$

$$I_{LB} \geq \frac{5(0.630 + 0.679)(31)^4(1728)}{384(29000)(1.55)} = 605\ in^4$$

$\Delta_{wc} \leq \Delta_{wc,max} = {}^{L}/_{240}$

$w_{wc} = (75 + 5)\ psf\ (7 ft) = 560\ plf$

$$I_{\substack{non-\\comp.}} \geq \frac{5(0.560)(31)^4(1728)}{384(29000)(1.55)} = 259\ in^4$$

⑤ unshored strength, $\phi_b M_p$

$w_u = 1.2\ (75 + 5)\ psf\ (7 ft) + 1.6\ (20\ psf)(7 ft) = 896\ plf$

$M_u = \frac{0.896\ (31)^2}{8} = 108 \leq \phi_b M_p$

35'-0" x 31'-0" Bay

page 2 of 6

⑥ select potential w-shapes

$(\phi M_n \geq 221 \text{ kft},\ I_{LB} \geq 605 \text{ in}^4,\ \phi_b M_p \geq 108 \text{ kft},\ I_{NC} \geq 259 \text{ in}^4)$

W14X30: $Y1 = 7$, $\Sigma Q_n = 111\text{K}$, $I_{LB} = 634 \text{ in}^4$, $I = 291 \text{ in}^4$, $\phi M_n = 275 \text{ kft}$, $\phi_b M_p = 177 \text{ kft}$

W16x26: $Y1 = 7$, $\Sigma Q_n = 96\text{K}$, $I_{LB} = 640 \text{ in}^4$, $I = 301 \text{ in}^4$, $\phi M_n = 259 \text{ kft}$, $\phi_b M_p = 166 \text{ kft}$

⑦ # of studs / bm and economy

W14X30: $\frac{111\text{K}}{17.2\text{K}} \times 2 \Rightarrow 14$ studs $< 31'$ ok ; $30^{\#} \times 31' + 14$ studs $\times 10^{\#}/\text{stud} = 1070^{\#}$

W16X26: $\frac{96\text{K}}{17.2\text{K}} \times 2 \Rightarrow 12$ studs $< 31'$ ok ; $26^{\#} \times 31' + 12$ studs $\times 10^{\#}/\text{stud} = 926^{\#}$

try W16X26 w/ 12 studs since it's more economical

⑧ check a

$$a = \frac{96}{0.85(4)(84)} = 0.34" < 1" \text{ OK}$$

$$b_{eff} = \min \left| \begin{array}{l} 2 \times \frac{31(12)}{8} = 93" \\ 2 \times \frac{1}{2}(7)(12) = 84" \leftarrow \\ \frac{31(12)}{8} + \frac{1}{2}(7)(12) = 88.5" \end{array} \right.$$

⑨ check Δ_{LL}, Δ_{TL}, Δ_{WC}, unshored strength, M_u, V_u, bm self-wt

$$\Delta_{LL} = \frac{5(0.675)(31)^4(1728)}{384(29000)(640)} = 0.76" \leq L/360 = 1.03" \text{ ok}$$

$$\Delta_{TL} = \frac{5(1.31)(31)^4(1728)}{384(29000)(640)} = 1.47" \leq L/240 = 1.55" \text{ ok}$$

$$\Delta_{WC} = \frac{5(0.560)(31)^4(1728)}{384(29000)(301)} = 1.33" \leq L/240 = 1.55" \text{ ok}$$

unshored strength: $M_u = 108 \text{ kft} \leq \phi_b M_p = 166 \text{ kft}$ ok

$M_u = 221 \text{ kft} \leq \phi M_n = 259 \text{ kft}$ ok
$V_u = 28.6 \text{ K} \leq \phi V_n = 106 \text{ K}$ ok

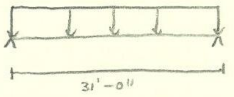
$$\text{self-wt} = \frac{26 \text{ plf}}{7 \text{ ft}} = 3.7 \text{ psf} \leq 5 \text{ psf assumed ok}$$

$\boxed{\text{use W16X26 w/ 12 studs}}$

35'-0" x 31'-0" Bay

page 3 of 6

interior bm B2



① $w_D = (75 + 10 + 5)\text{ psf }(8.5\text{ ft}) = 765\text{ plf}$

$$LL = 100\text{ psf} \times \left| \begin{array}{l} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{527}} = 0.903 \end{array} \right. = 90.3\text{ plf}$$

$w_L = 90.3(8.5) = 768\text{ plf}$

$w_u = 1.2(765) + 1.6(768) = 2147\text{ plf}$

② $M_u = \dfrac{2.147\text{ klf }(31\text{ft})^2}{8} = 258\text{ kft} \leq \phi M_n$

$V_u = \dfrac{2.147\ (31)}{2} = 33.3\text{ k} \leq \phi V_n$

③ assume $a = 1"$, $Y2 = 7"$

④ $\Delta_{TL} \leq \Delta_{TL,max} = {}^{L}/_{240} = 1.55"$

$I_{LB} \geq \dfrac{5(0.765 + 0.768)(31)^4 (1728)}{384(29000)(1.55)} = 709\text{ in}^4$

$\Delta_{wc} \leq \Delta_{wc,max} = {}^{L}/_{240}$

$w_{wc} = (75 + 5)\text{ psf }(8.5\text{ ft}) = 680\text{ plf}$

$I_{\text{non-comp}} \geq \dfrac{5(0.680)(31)^4 (1728)}{384(29000)(1.55)} = 314\text{ in}^4$

⑤ unshored strength, $\phi_b M_p$

$w_u = 1.2(75 + 5)\text{ psf }(8.5\text{ft}) + 1.6(20\text{ psf})(8.5\text{ ft}) = 1088\text{ plf}$

$M_u = \dfrac{1.088\text{ klf }(31\text{ft})^2}{8} = 131\text{ kft} \leq \phi_b M_p$

⑥ select potential W-shapes

$(\phi M_n \geq 258\text{ kft},\ I_{LB} \geq 709\text{ in}^4,\ \phi_b M_p \geq 131\text{ kft},\ I_{NC} \geq 314\text{ in}^4)$

W14X34: $Y2 = 7$, $\Sigma Q_n = 125\text{ k}$, $I_{LB} = 732\text{ in}^4$, $I = 340\text{ in}^4$, $\phi M_n = 316\text{ kft}$, $\phi_b M_p = 205\text{ kft}$

W16X31: $Y2 = 7$, $\Sigma Q_n = 114\text{ k}$, $I_{LB} = 783\text{ in}^4$, $I = 375\text{ in}^4$, $\phi M_n = 313\text{ kft}$, $\phi_b M_p = 203\text{ kft}$

⑦ # of studs/bm and economy

W14X34: $\dfrac{125\text{k}}{17.2\text{k}} \times 2 \Rightarrow 16$ studs < 31' ; $34\ \# \times 31' + 16$ studs $\times$ 10 #/stud = 1214 #

W16X31: $\dfrac{114\text{k}}{17.2\text{k}} \times 2 \Rightarrow 14$ studs < 31' ; $31\ \# \times 31' + 14$ studs $\times$ 10 #/stud = 1101 #

try W16X31 w/ 14 studs since it's more economical

35'-0" X 31'-0" Bay

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⑧ check a

$a = \dfrac{114}{0.85(4)(88.5)} = 0.38" < 1"$ ok

$b_{eff} = $
$2 \times \dfrac{31(12)}{8} = 93"$
$\dfrac{1}{2}(7)(12) + \dfrac{1}{2}(10)(12) = 102"$
$\dfrac{31(12)}{8} + \dfrac{1}{2}(7)(12) = 88.5" \leftarrow$
min $\dfrac{31(12)}{8} + \dfrac{1}{2}(10)(12) = 106.5"$

⑨ Summary

$\Delta_{LL} = \dfrac{5(0.768)(31)^4(1728)}{384(29000)(783)} = 0.70" \leq L/360 = 1.03"$ ok

$\Delta_{TL} = \dfrac{5(1.53)(31)^4(1728)}{384(29000)(783)} = 1.40" \leq L/240 = 1.55"$ ok

$\Delta_{wc} = \dfrac{5(0.680)(31)^4(1728)}{384(29000)(375)} = 1.30" \leq L/240 = 1.55"$ ok

unshored strength: $M_u = 131$ kft $\leq \phi_b M_p = 203$ kft ok

$M_u = 258$ kft $\leq \phi M_n = 313$ kft ok

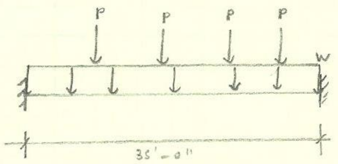
$V_u = 33.3$ K $\leq \phi V_n = 131$ K ok

self-wt $= \dfrac{31 \text{ plf}}{8.5 \text{ ft}} = 3.64$ psf < 5 psf assumed ok

$\boxed{\text{use W16X31 w/ 14 studs}}$

35'-0" x 31'-0" Bay   page 5 of 6

exterior girder G1



① load:

Dead: slab - 75 psf
SDL - 10 psf
bm wt allow. - 5 psf
girder self wt allow - 5 psf
wall - 250 plf

Live: $100\text{ psf} \times \begin{vmatrix} 0.50 \\ 0.25 + \frac{15}{\sqrt{1085}} = 0.705 \end{vmatrix} = 70.5\text{ psf}$

$$P_u = \left[1.2\,(75 + 10 + \overset{\text{bm self-wt allow.}}{5})\text{psf} + 1.6\,(70.5\text{ psf})\right](17\text{ft})(7\text{ft}) = 26.3\text{K}$$

$$w_u = 1.2(5\text{ psf})(17\text{ ft}) + 1.2\,(250\text{ plf}) = 402\text{ plf}$$

② $M_u = 0.40PL + \frac{wL^2}{12} = 0.40\,(26.3\text{ K})(35\text{ft}) + \frac{0.462\text{ klf}\,(35\text{ft})^2}{12} = 409\text{ Kft} \le \phi M_n = \phi M_p \Rightarrow Z_x \ge \frac{409\,(12)}{0.9(50)} = 109\text{ in}^3$

$V_u = 2.0P + \frac{wL}{2} = 2.0(26.3\text{ K}) + \frac{0.402\text{ klf}\,(35\text{ft})}{2} = 59.6\text{ K} \le \phi V_n$

③ $\Delta_{TL} \le \Delta_{TL,max} = L/240 = 1.75"$

$$\Delta_{TL} = \frac{0.013PL^3}{EI_x} + \frac{wL^4}{384EI_x} = \frac{0.013(19.1\text{K})(35\text{ft})^3(1728)}{29000\,I_x} + \frac{0.335\text{ klf}\,(35\text{ft})^4(1728)}{384\,(29000)\,I_x} = \frac{634.3}{I_x} + \frac{78}{I_x}$$

$1.75" = \frac{712.3}{I_x} \Rightarrow I_x \ge 407\text{ in}^4$

④ Select non-composite W-shape

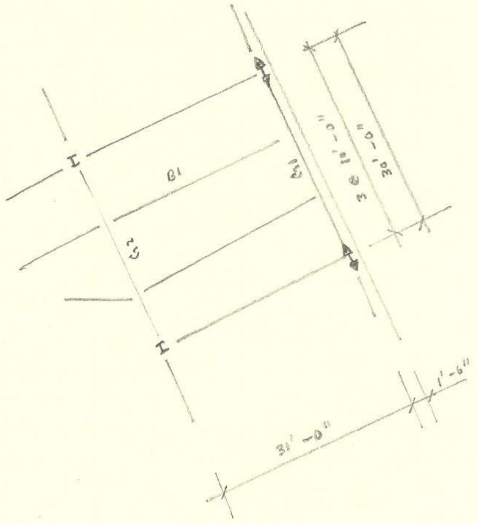
W18x55 required to support gravity load

w/ $Z_x = 112\text{ in}^3 > 109\text{ in}^3$ ok
$I_x = 890\text{ in}^4 > 407\text{ in}^4$ ok

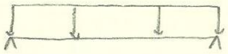
$\phi_b M_{px} = 420\text{ Kft} > 409\text{ Kft}$ ok
$\phi V_n = 212\text{ K} > 59.6\text{ K}$ ok

- refer to moment frame 1' elevation in report for member sizes due to combined lat. and gravity load analysis

30'-0" x 31'-0"   page 1 of 4



interior bm B1



① $w_D = (75 + 10 + 5)\ psf\ (10\ ft) = 900\ plf$

$$LL = 100\ psf \times \left| \begin{array}{l} 0.50 \\ 0.25 + \frac{15}{\sqrt{620}} = 0.852 \end{array} \right. = 85.2\ psf$$

$w_L = 85.2\ psf\ (10\ ft) = 852\ plf$

$w_u = 1.2(900) + 1.6(852) = 2443\ plf$

② $M_u = \frac{2.443\ klf\ (31\ ft)^2}{8} = 293\ kft \leq \phi M_n$

$V_u = \frac{2.443(31)}{2} = 38\ K \leq \phi V_n$

③ assume $a = 1"$, $Y2 = 7"$

④ $\Delta_{TL} \leq \Delta_{TL,max} = L/240 = 1.55"$

$$I_{LB} \geq \frac{5(0.90 + 0.852)(31)^4(1728)}{384(29000)(1.55)} = 810\ in^4$$

$\Delta_{wc} \leq \Delta_{wc,max} = L/240$

$w_{wc} = (75 + 5)\ psf\ (10\ ft) = 800\ plf$

$$I_{non\text{-}comp} = \frac{5(0.80\ klf)(31\ ft)^4(1728)}{384(29000)(1.55)} = 370\ in^4$$

30'-0" X 31'-6" Bay | page 2 of 4

⑤ unshored strength, $\phi_b M_p$

$w_u = 1.2(75+5)\text{ psf}(10\text{ft}) + 1.6(20\text{ psf})(10\text{ft}) = 1280\text{ plf}$

$$M_u = \frac{1.28\text{ klf}(31\text{ ft})^2}{8} = 154\text{ kft} \le \phi_b M_p$$

⑥ select potential w-shapes

$(\phi M_n \ge 293\text{ kft},\ I_{LB} \ge 810\text{ in}^4,\ \phi_b M_p \ge 154\text{ kft},\ I_{NC} \ge 370\text{ in}^4)$

$W16X31:\ Y1 = 6,\ \Sigma Q_n = 114K,\ I_{LB} = 914\text{ in}^4,\ I = 375\text{ in}^4,\ \phi M_n = 350\text{ kft},\ \phi_b M_p = 203\text{ kft}$

⑦ # of studs/bm and economy

$$W16X31:\ \frac{114\text{ K}}{17.2\text{ K}} \times 2 \Rightarrow 14\text{ studs} < 31'\text{ ok};\ 31^{\#} \times 31' + 14\text{ studs} \times 10^{\#}/\text{stud} = 1101^{\#}$$

⑧ check a

$$a = \frac{114\text{ K}}{0.85(4\text{ ksi})(93\text{ in})} = 0.36" < 1"\text{ ok}$$

$$b_{eff} = \min\left|\begin{array}{l} 2 \times \frac{31(12)}{8} = 93" \leftarrow \\ 2 \times \frac{1}{2}(10)(12) = 120" \\ \frac{31(12)}{8} + \frac{1}{2}(10)(12) = 106.5" \end{array}\right.$$

⑨ summary

$$\Delta_{LL} = \frac{5(0.852)(31)^4(1728)}{384(29000)(914)} = 0.67" \le l/360 = 1.03"\text{ ok}$$

$$\Delta_{TL} = \frac{5(1.75)(31)^4(1728)}{384(29000)(914)} = 1.37" \le l/240 = 1.55"\text{ ok}$$

$$\Delta_{WC} = \frac{5(0.80)(31)^4(1728)}{384(29000)(375)} = 1.53" \le l/240\text{ ok}$$

unshored strength: $M_u = 154\text{ kft} \le \phi_b M_p = 203\text{ kft}$ ok

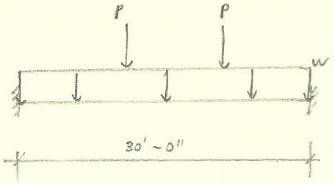
$M_u = 293\text{ kft} \le \phi M_n = 350\text{ kft}$ ok
$V_u = 38\text{ K} \le \phi V_n = 131\text{ K}$ ok

$$\text{Self-wt} = \frac{31\text{ plf}}{10\text{ ft}} = 3.1\text{ psf} < 5\text{ psf assumed ok}$$

$\boxed{\text{use W16X31 w/ 14 studs}}$

30'-0" X 31'-0" Bay | page 3 of 4

exterior girder G1



① Dead loads: slab - 75 psf
SDL - 10 psf
bm wt allow. - 5 psf
girder self-wt allow. - 5 psf
curtain wall - 250 plf

Live load; $100 \text{ psf} \times \left| \begin{array}{l} 0.50 \\ 0.25 + \frac{15}{\sqrt{975}} = 0.73 \end{array} \right. = 73 \text{ psf}$

$P_u = [1.2 (75 + 10 + \overset{\text{bm wt. allow.}}{\downarrow 5}) \text{ psf} + 1.6 (73 \text{ psf})] (17 \text{ ft}) (10 \text{ ft}) = 38.2 \text{ K}$

$w_u = 1.2 (5 \text{ psf})(17 \text{ ft}) + 1.2 (250 \text{ plf}) = 402 \text{ plf}$

② $M_u = 0.222 PL + \frac{wL^2}{12} = 0.222 (38.2 \text{ k})(30 \text{ ft}) + \frac{0.402 \text{ klf} (30 \text{ ft})^2}{12} = 285 \text{ kft} \leq \phi M_n$

$V_u = 1.0 P = 38.2 \text{ k} \leq \phi V_n$

③ assume $a = 2"$ $Y2 = 6.5"$

④ $\Delta_{TL} \leq \Delta_{TL,max} = {}^{L}/_{240} = 1.5"$

$\Delta_{TL} = \frac{0.008 PL^3}{EI} + \frac{wL^4}{384 EI} = \frac{0.008 (27.7 \text{ k}) (30 \text{ ft})^3 (1728)}{29000 I_{LB}} + \frac{0.335 \text{ klf} (30 \text{ ft})^4 (1728)}{384 (29000) I_{LB}} = \frac{356.5}{I_{LB}} + \frac{42.1}{I_{LB}}$

$1.5" = \frac{398.6}{I_{LB}} \Rightarrow I_{LB} \geq 266 \text{ in}^4$

⑤ unshored strength, $\phi_b M_p$

$P_u = 1.2 [(75 + \overset{\text{girder self-wt allow.}}{\downarrow 5}) \text{ psf} (10 \text{ ft})] (17 \text{ ft}) + 1.6 (20 \text{ psf})(10 \text{ ft})(17 \text{ ft}) = 21.8 \text{ K}$

$M_u = 0.222 (21.8 \text{ K})(30 \text{ ft}) = 145 \text{ kft} \leq \phi_b M_p$

30'-0" x 31'-0" Bay

page 4 of 4

⑥ select potential W-shapes

$(\phi M_n \geq 285 \text{ kft}, \ I_{LB} \geq 266 \text{ in}^4, \ \phi_b M_p \geq 145 \text{ kft})$

W16x31: $Y1 = 7$ $\Sigma Q_n = 114$ k, $I_{LB} = 783$ in^4, $\phi M_n = 313$ kft, $\phi_b M_p = 203$ kft

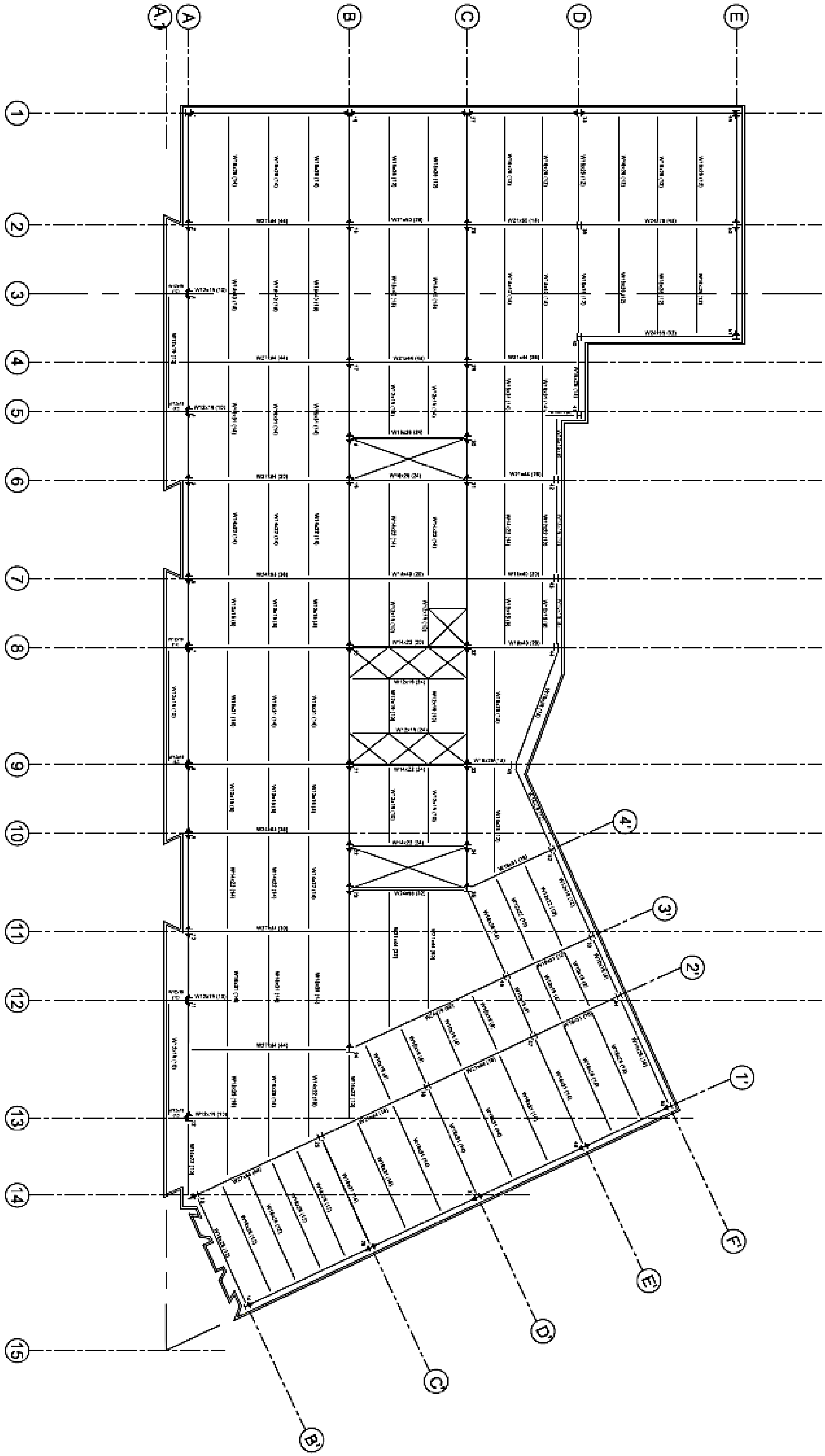
⑦ # of studs / girder and economy

W16x31: $\frac{114}{21.5} \times 2 \Rightarrow 12$ studs ; $31^{\#} \times 30' + 12$ studs $\times 10^{\#}/\text{stud} = 1050^{\#}$

note: beams and girders in moment frames were initially designed as composite, but the design of the lateral beam and girder members were changed to non-composite

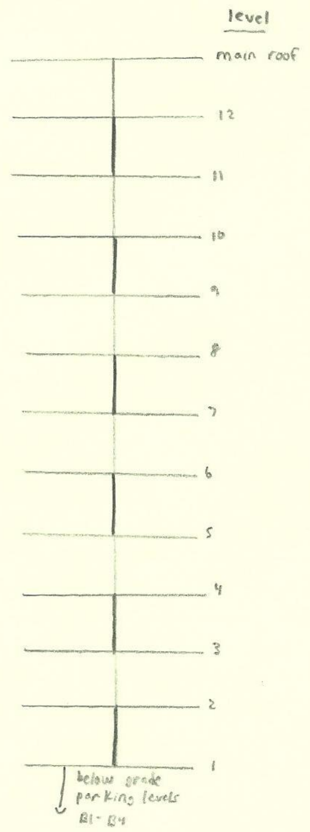
- refer to moment frame 1' elevation in report for member sizes due to combined lateral and gravity load analysis

TYPICAL FLOOR FRAMING PLAN



Column Design

Column Design | General info | page 1 of 1



notes:

- floor-to-floor height is 15 feet
- the columns will be spliced every 2 tiers (the darkly highlighted columns in the figure to the left will be designed throughout the ht. of each column)

Loads

main roof:

dead loads: slab - 75 psf
SDL - 10 psf
member self-wt. allowance - 10 psf

live loads: balconies and terraces - 100 psf
green roof areas - 30 psf
mech. room - 150 psf
pump rm / generator rm - 150 psf

snow loads: 15.75 psf (flat roof snow load)

levels 2-12:

dead loads: slab - 75 psf
SDL - 10 psf
member self-wt allowance - 10 psf

live loads: office floors - 100 psf

penthouse roof:

dead loads: slab - 75 psf
SDL - 5 psf
member self-wt allowance - 10 psf

live loads: green roof areas - 30 psf

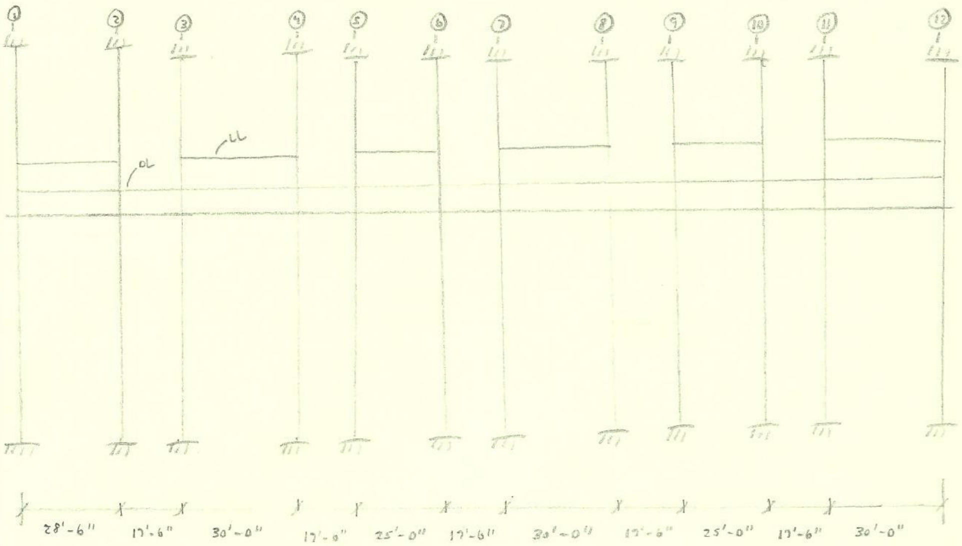
snow loads: 15.75 psf (flat roof snow load)

roof constr. LL: 20 psf

- refer to excel spreadsheets for the gravity columns analyses and sizes

Estimate column sizes | moment frame A.1 | page 1 of 13

Estimate unbalanced moment in interior and exterior columns



dead loads : slab - 75 psf
SDL - 10 psf
bm self-wt allow. - 5 psf
curtain wall - 250 plf

live loads: office space - 100 psf

note: - the slab transfers the curtain wall self-wt to the exterior bms
- the 30'-0" span bm a part of moment frame A.1 do not support the curtain wall self-wt

28'-6" span

$w_D = (75 + 10 + 5)\text{psf}\ (6.63\text{ft}) + 250\text{ plf} = 847\text{ plf}$

influence area = $(10.25 + 1.5)\text{ft}\ (28.5\text{ ft}) = 335\text{ ft}^2 < 400\text{ ft}^2$, LL reduction not applicable

$w_L = 100\text{ psf}\ (6.63\text{ ft}) = 663\text{ plf}$

$w_u = 1.2(847) + 1.6(663) = 2077\text{ plf}$

$$FEM_u = \frac{wL^2}{12} = \frac{2.077\text{ klf}\ (28.5\text{ ft})^2}{12} = 141\text{ kft}$$

17'-6" span

$w_D = (90\text{ psf})(10.88\text{ ft}) + 250 = 1229\text{ plf}$

$w_u = 1.2(1229) = 1475\text{ plf}$

$$FEM_u = \frac{1.475\text{ klf}\ (17.5\text{ ft})^2}{12} = 37.6\text{ kft}$$

Estimate Column sizes	moment frame A.1	page 2 of 13

30'- 0" span

$w_D = 90 \text{ psf} (7.38 \text{ ft}) = 664 \text{ plf}$

influence area = $(4.5 + 10.25) \text{ ft} (30 \text{ ft}) = 442.5 \text{ ft}^2 > 400 \text{ ft}^2$, LL reduction required

$$LL_{red} = \max \begin{cases} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{442.5}} = 0.96 \end{cases}$$

$w_L = 0.96 (100 \text{ psf})(7.38 \text{ ft}) = 708.5 \text{ plf}$

$w_u = 1.2 (664) + 1.6 (708.5) = 1930 \text{ plf}$

$$FEM_u = \frac{1.930 (30)^2}{12} = 145 \text{ Kft}$$

25'-0" span

$w_D = 90 \text{ psf} (6.63 \text{ ft}) + 250 = 847 \text{ plf}$

influence area = $(10.25 + 1.5) \text{ ft} (25 \text{ ft}) = 294 \text{ ft}^2 < 400 \text{ ft}^2$, LL reduction not applicable

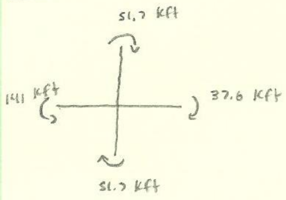
$w_L = 100 \text{ psf} (6.63 \text{ ft}) = 663 \text{ plf}$

$w_u = 1.2 (847) + 1.6 (663) = 2077 \text{ plf}$

$$FEM_u = \frac{2.077 (25)^2}{12} = 108 \text{ Kft}$$

Estimate column sizes | moment frame A.1 | page 3 of 13

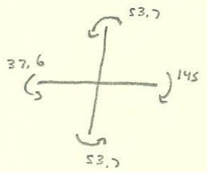
col 2



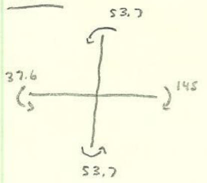
UBM = 141 - 37.6 = 103.4 kft

assume: half of UBM is distributed to col above and half to col. below

col 7

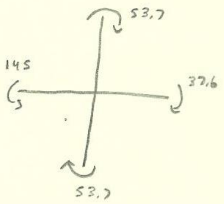


col 3

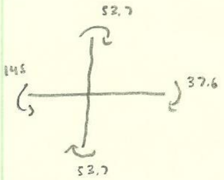


UBM = 145 - 37.6 = 107.4

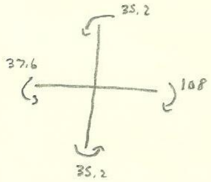
col 8



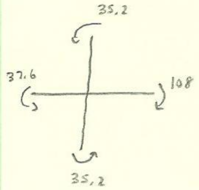
col 4



col 9

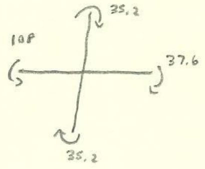


col 5

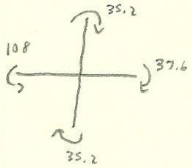


UBM = 108 - 37.6 = 70.4

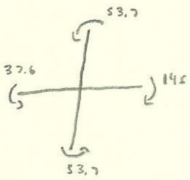
col 10



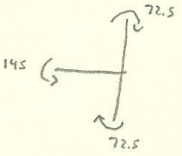
col 6



col 11



col 12



note: K = 1.0, L = 15'-0", d = 14" — W14's

$$P_{eq} = P_u + \frac{24 M_u}{d}$$

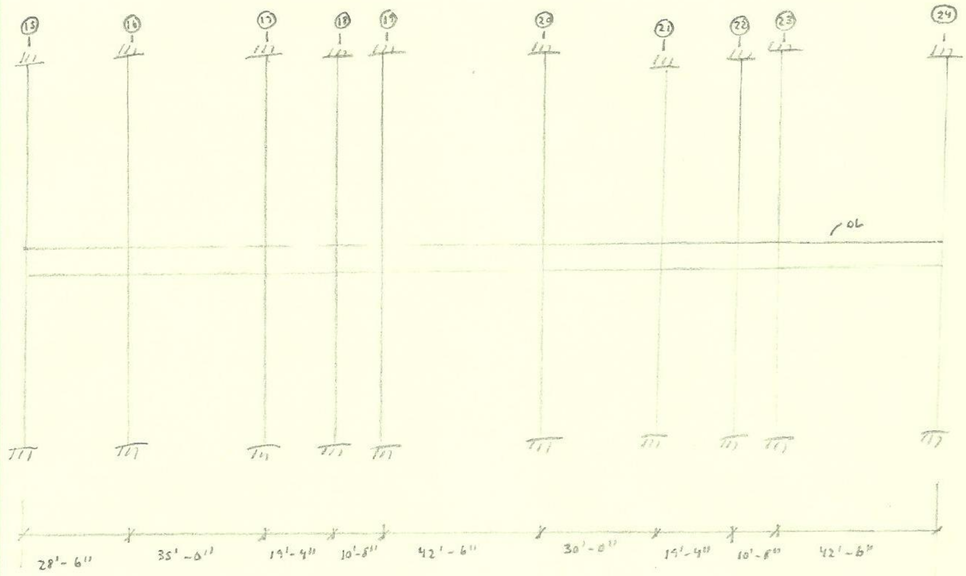
← unbalanced col mom.

axial load

depth of col.

refer to excel sheets for Peq and estimated col. size for each column

Estimate column sizes | moment frame B | page 4 of 13



dead loads : slab - 75 psf

SDL - 10 psf

bm self-wt allow - 5 psf

girder self-wt allow - 5 psf

live loads: office space - 100 psf

mech. rm - 150 psf

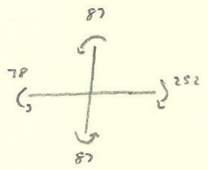
<u>max VBM in Col 16</u>

- load the 35'-0" span with DL and LL
- load 28'-6" span with DL only

28'-6" span; $w_D = 95$ psf (10.13 ft) = 962 plf

$w_u = 1.2(962) = 1154$ plf

$$FEM = \frac{1.154\ (28.5)^2}{12} = 78\ \text{kft}$$



VBM = 252 - 78 = 174

35'-0" span: $w_D = 962$ plf

influence area = (10.25 + 10) ft (35 ft) = 709 ft² > 400 ft² ∴ reduce LL

$$LL_{red} = \max \left| \begin{array}{l} 0.50 \\ 0.25 + \frac{15}{\sqrt{709}} = 0.81 \end{array} \right.$$

$w_L = 0.81\ (100\ \text{psf})(10.13\ \text{ft}) = 821$ plf

$w_u = 1.2(962) + 1.6(821) = 2468$ plf

$$FEM = \frac{2.468\ (35)^2}{12} = 252\ \text{kft}$$

Estimate column size | moment frame B | page 5 of 13

max UBM in Col 17

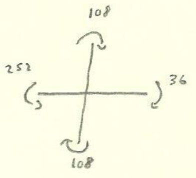
- load 35'-0" span with DL and LL
- load 19'-4" span with DL only

35'-0" span: FEM = 252 Kft

19'-4" span: $w_D = 95\text{ psf}(10.13\text{ ft}) = 962\text{ plf}$

$w_u = 1.2(962) = 1154\text{ plf}$

$FEM = \frac{1.154(19.33)^2}{12} = 36\text{ Kft}$



UBM = 252 - 36 = 216

max UBM in col 18

- load 19'-4" span with DL and LL
- load 10'-8" span with DL only

19'-4" span: $w_D = 962\text{ plf}$

$w_L = 150\text{ psf}(10.13\text{ ft}) = 1520\text{ plf}$

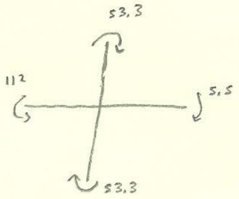
$w_u = 1.2(962) + 1.6(1520) = 3586$

$FEM = \frac{3.586(19.33)^2}{12} = 112\text{ Kft}$

10'-8" span: $w_D = 95\text{ psf}(5.13\text{ ft}) = 487\text{ plf}$

$w_u = 1.2(487) = 584\text{ plf}$

$FEM = \frac{0.584(10.67)^2}{12} = 5.5\text{ Kft}$



UBM = 112 - 5.5 = 106.5

max UBM in Col 19

- load 42'-6" span with DL and LL
- load 10'-8" span with DL only

42'-6" span: $w_D = 962\text{ plf}$

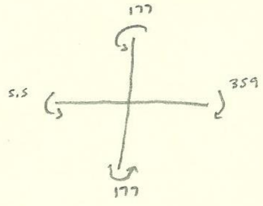
influence area: 20.25 ft (42.5 ft) = 861 ft² > 400 ft², LL reduction required

$$LL_{red} = \max\left|\begin{array}{l} 0.50 \\ 0.25 + \frac{15}{\sqrt{861}} = 0.76 \end{array}\right.$$

$w_L = 0.76(100\text{ psf})(10.13\text{ ft}) = 770\text{ plf}$

$w_u = 1.2(962) + 1.6(770) = 2386\text{ plf}$

$FEM = \frac{2.386(42.5)^2}{12} = 359\text{ Kft}$

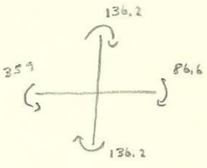


10'-8" span: FEM = 5.5 Kft

max UBM for col 20

- load 42'-6" span with DL and LL
- load 30'-0" span with DL only

42'-6" span; FEM = 359 kft



30'-0" span: $w_D = 962$ plf

$w_u = 1.2(962) = 1154$ plf

$FEM = \frac{1.154(30)^2}{12} = 86.6$ kft

UBM = 359 - 86.6 = 272.4 kft

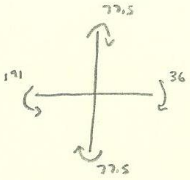
max UBM for col 21

- load 30'-0" span with DL and LL
- load 19'-4" span with DL only

30'-0" span: $w_D = 962$ plf

influence area = 20.25 ft (30ft) = 608 ft² > 400 ft², LL reduction required

$$LL_{red} = \max \begin{cases} 0.50 \\ 0.25 + \frac{15}{\sqrt{608}} = 0.86 \end{cases}$$



$w_L = 0.86(100 \text{ psf})(10.13 \text{ ft}) = 871$ plf

$w_u = 1.2(962) + 1.6(871) = 2548$ plf

$FEM = \frac{2.548(30)^2}{12} = 191$ kft

UBM = 191 - 36 = 155 kft

19'-4" span: FEM = 36 kft

max UBM col 22

- load 19'-4" span with DL and LL
- load 10'-8" span with DL only

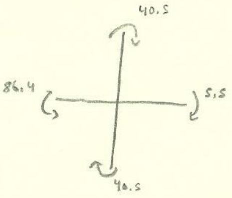
19'-4" span; $w_D = 962$ plf

influence area = 20.25 ft (19.33 ft) = 391 < 400 ft², LL reduction not applicable

$w_L = 100 \text{ psf}(10.13 \text{ ft}) = 1013$ plf

$w_u = 1.2(962) + 1.6(1013) = 2775$ plf

$FEM = \frac{2.775(19.33)^2}{12} = 86.4$ kft



10'-8" span: FEM = 5.5 kft

UBM = 86.4 - 5.5 = 81 kft

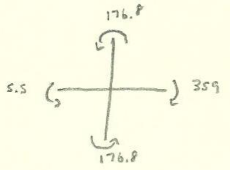
Estimate column sizes | moment frame B | page 7 of 13

max UBM on col 23

- load 42'-6" span with DL and LL
- load 10'-8" span with DL only

42'-6" span: FEM = 359 Kft

10'-8" span: FEM = 5.5 Kft

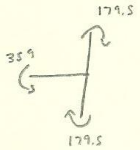


UBM = 359 − 5.5 = 353.5 Kft

max UBM on col 24

- load 42'-6" span with DL and LL

42'-6" span: FEM = 359 Kft



max UBM in col 28 - refer to col 16

max UBM in col 29 - refer to col 17

max UBM in col 30 - refer to col 18

max VBM in col 31 - refer to col 19

max UBM in col 32 - refer to col 20

max UBM in col 33 - refer to col 21

max UBM in col 34 - refer to col 22

max UBM in col 35

- load 10'-8" span with DL and LL

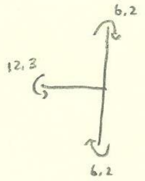
10'-8" span: $w_D = 95 \text{ psf} (4.75 \text{ ft}) = 451 \text{ plf}$

influence area = 9.5 ft (10.67 ft) = 101 ft² < 400 ft², LL reduction not applicable

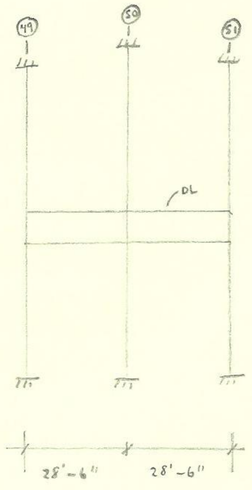
$w_L = 100 \text{ psf} (4.75 \text{ ft}) = 475 \text{ plf}$

$w_u = 1.2(451) + 1.6(475) = 1301 \text{ plf}$

$$FEM = \frac{1.301 (10.67)^2}{12} = 12.3 \text{ Kft}$$



Estimate column sizes | moment frame E | page 9 of 13



max UBM in col 49 and 51

- load 28'-6" span with DL and LL

28'-6" span: w_D = 95 psf (6.5 ft) = 618 plf

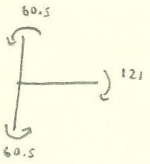
influence area = 11.5 ft (28.5 ft) = 328 ft^2 < 400 ft^2

LL reduction not applicable

w_L = 100 psf (6.5 ft) = 650 plf

w_u = 1.2 (618) + 1.6 (650) = 1782 plf

$$FEM = \frac{1.782\,(28.5)^2}{12} = 121 \text{ Kft}$$



max UBM in col 50

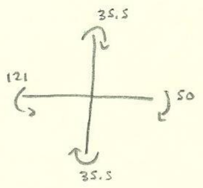
- load are of the 28'-6" spans with DL and LL
- load the other 28'-6" span with DL only

28'-6" span (DL only): w_D = 618 plf

w_u = 1.2 (618) = 742 plf

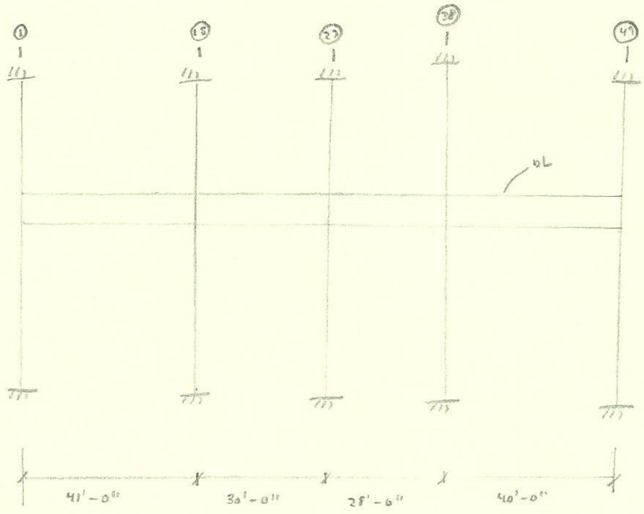
$$FEM = \frac{0.742\,(28.5)^2}{12} = 50 \text{ Kft}$$

28'-6" span (DL and LL): FEM = 121 Kft



UBM = 121 - 50 = 71 Kft

Estimate column sizes | moment frame 1 | page 10 of 13



max UBM on Col 1

- load 41'-0" span with DL and LL

41'-0" span : $W_D = 95 psf (15.75 ft) = 1496 plf$

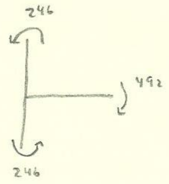
$I.A. = (28.5 + 1.5) ft (41 ft) = 1230 ft^2 > 400 ft^2$, LL reduction required

$$LL_{req} = \max \begin{cases} 0.50 \\ 0.25 + \frac{15}{\sqrt{1230}} = 0.68 \end{cases}$$

$W_L = 0.68 (100 psf)(15.75 ft) = 1071 plf$

$W_u = 1.2 (1496) + 1.6 (1071) = 3509 plf$

$$FEM = \frac{3.509 (41)^2}{12} = 492 Kft$$



max UBM in Col 15

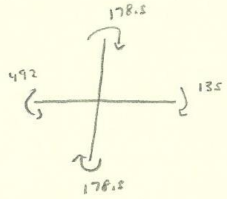
- load 41'-0" span with DL and LL
- load 30'-0" span with DL only

41'-0" span: $FEM = 492 kft$

30'-0" span: $W_D = 1496 plf$

$W_u = 1.2 (1496) = 1795 plf$

$$FEM = \frac{1.795 (30)^2}{12} = 135 Kft$$



$UBM = 492 - 135 = 357 Kft$

Estimate column sizes | moment frame 1 | page 11 of 13

max VBM in col 27

- load 30'-0" span with DL and LL
- load 28'-6" span with DL only

30'-0" span: $w_D = 1496$ plf

I.A. = 30ft (30ft) = 900 ft² > 400 ft², LL reduction required

$$LL_{red} = \max\begin{cases} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{900}} = 0.75 \end{cases}$$

$w_L = 0.75 (100)(15.75) = 1181$ plf

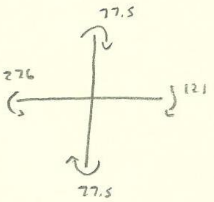
$w_u = 1.2 (1496) + 1.6 (1181) = 3685$ plf

$$FEM = \frac{3.685 (30)^2}{12} = 276 \text{ Kft}$$

28'-6" span: $w_D = 1496$ plf

$w_u = 1.2 (1496) = 1795$ plf

$$FEM = \frac{1.795 (28.5)^2}{12} = 121 \text{ Kft}$$



UBM = 276 - 121 = 155 Kft

max VBM in col 38

- load 40'-0" span with DL and LL
- load 28'-6" span with DL only

40'-6" span: $w_D = 1496$ plf

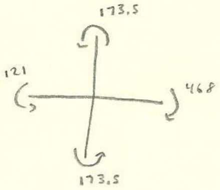
I.A. = 30ft (40ft) = 1200 ft² > 400 ft², LL reduct. req'd

$$LL_{red} = \max\begin{cases} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{1200}} = 0.68 \end{cases}$$

$w_L = 0.68 (100)(15.75) = 1071$ plf

$w_u = 1.2 (1496) + 1.6 (1071) = 3509$

$$FEM = \frac{3.509 (40)^2}{12} = 468 \text{ Kft}$$



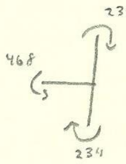
UBM = 468 - 121 = 347 Kft

28'-6" span: FEM = 121 Kft

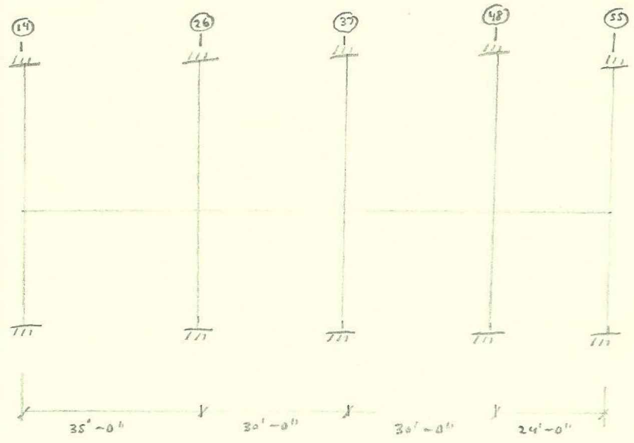
max VBM in col 49

- load 40'-0" span with DL and LL

span 40'-0": FEM = 468 Kft



Estimate column size | moment frame 1' | page 12 of 13



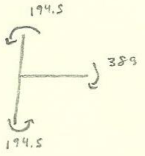
max UBM in col 14

- load 35'-0" span with DL and LL

35'-0" span: $w_D = 95\text{ psf}\,(17\text{ ft}) = 1615\text{ plf}$

$I.A. = 32.5\text{ ft}\,(35\text{ft}) = 1138\text{ ft}^2 > 400\text{ft}^2$, LL reduction req'd

$$LL_{red} = \max \left| \begin{array}{l} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{1138}} = 0.69 \end{array} \right.$$



$w_L = 0.69\,(100)(17) = 1173\text{ plf}$

$w_u = 1.2\,(1615) + 1.6\,(1173) = 3815\text{ plf}$

$$FEM = \frac{3.815\,(35)^2}{12} = 389\text{ Kft}$$

max UBM in col 26

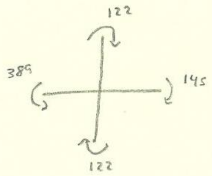
- load 35'-0" span with DL and LL
- load 30'-0" span with DL only

35'-0" span: $FEM = 389\text{ Kft}$

30'-0" span: $w_D = 1615\text{ plf}$

$w_u = 1.2\,(1615) = 1938\text{ plf}$

$$FEM = \frac{1.938\,(30)^2}{12} = 145\text{ Kft}$$



$UBM = 389 - 145 = 144\text{ Kft}$

Estimate column sizes | moment frame 1' | page 13 of 13

max UBM in col 37

- load one of the 30'-0" spans with DL and LL
- load the other 30'-0" span with DL only

30'-0" span (DL only): FEM = 145 kft

30'-0" span (DL and LL): w_D = 1615 plf

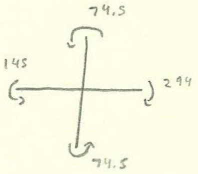
I.A. = 32.5 ft (30ft) = 975 ft² > 400 ft², LL reduct. req'd

$$LL_{red} = \max \begin{vmatrix} 0.50 \\ 0.25 + \frac{15}{\sqrt{975}} \end{vmatrix} = 0.73$$

w_L = 0.73 (100)(17) = 1241 plf

w_u = 1.2 (1615) + 1.6 (1241) = 3924 plf

$$FEM = \frac{3.924\ (30)^2}{12} = 294 \text{ kft}$$



UBM = 294 - 145 = 149 kft

max UBM in col 48

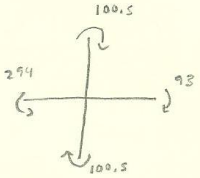
- load 30'-0" span with DL and LL
- load 24'-0" span with DL only

30'-0" span: FEM = 294

24'-0" span: w_D = 1615 plf

w_u = 1938 plf

$$FEM = \frac{1.938\ (24)^2}{12} = 93 \text{ kft}$$



UBM = 294 - 93 = 201 kft

max UBM in col 55

- load 24'-0" span with DL and LL

24'-0" span: w_D = 1615 plf

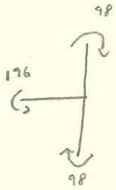
I.A. = 32.5 ft (24 ft) = 780 ft² > 400 ft², LL reduct. req'd

$$LL_{red} = \max \begin{vmatrix} 0.50 \\ 0.25 + \frac{15}{\sqrt{780}} \end{vmatrix} = 0.79$$

w_L = 0.79 (100)(17) = 1343

w_u = 1.2 (1615) + 1.6 (1343) = 4087 plf

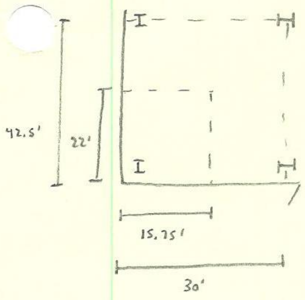
$$FEM = \frac{4.087\ (24)^2}{12} = 196 \text{ kft}$$



Estimate Column sizes

Moment Frame Column Design | column line A.1 | page 1 of 2

column 1



tributory area per floor or roof = 22 ft (15.75 ft) = 346.5 ft²

influence area per floor = 42.5 ft (30ft) = 1275 ft²

- Load above level 12: roof + 1 floor

$$LL_{red} = \max\left|\begin{array}{l} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{1275}} = 0.67 \end{array}\right.$$

$$P_L = \overbrace{0.67\ (100\ psf)}^{\text{floor level}}(346.5\ ft^2) + \overbrace{30\,psf}^{\text{main roof}}\ (346.5\ ft^2) = 33.6\ K$$

$$P_S = 15.75\ psf\ (346.5\ ft^2) = 5.5\ K$$

$$P_D = \underbrace{(75+10)}_{\text{main roof}}psf\ (346.5\ ft^2) + \underbrace{(75+10)}_{\text{floor level}}psf\ (346.5\ ft^2) = 58.9K$$

$$P_u = 1.2\ (58.9\ K) + 1.6\ (33.6\ K) + 0.5\ (5.5K) = 127.2\ K$$

- Load above level 10: roof + 3 floors

$$LL_{red} = \max\left|\begin{array}{l} 0.40 \\ 0.25 + \dfrac{15}{\sqrt{3\times1275}} = 0.49 \end{array}\right.$$

$$P_L = (0.49\,(100\ psf)(3\,flrs) + 30\ psf)\ 346.5\ ft^2 = 61.3K$$

$$P_S = 15.75\ (346.5) = 5.5K$$

$$P_D = (85\,psf\ (3\,flrs) + 85\ psf)\ 346.5\ ft^2 = 117.8K$$

$$P_u = 1.2\ (117.8) + 1.6\ (61.3) + 0.5\ (5.5) = 242.2\ K$$

- Load above level 8: roof + 5 flrs

$$LL_{red} = \max\left|\begin{array}{l} 0.40 \\ 0.25 + \dfrac{15}{\sqrt{5\times1275}} = 0.44 \end{array}\right.$$

$$P_L = (0.44\,(100)(5\,flrs) + 30)\ 346.5 = 86.6K$$

$$P_S = 5.5\ K$$

$$P_D = (85\,(5\,flrs) + 85)\ 346.5 = 176.7K$$

$$P_u = 1.2\,(176.7) + 1.6\ (86.6) + 0.5\,(5.5) = 353.4K$$

- Load above level 6: roof + 7 flrs

$$LL_{red} = \max\left|\begin{array}{l} 0.40 \\ 0.25 + \dfrac{15}{\sqrt{7\times1275}} = 0.41 \end{array}\right.$$

$$P_L = (0.41\,(100)(7) + 30)\ 346.5 = 109.8\ K$$

$$P_S = 5.5\ K$$

$$P_D = (85\,(7) + 85)\ 346.5 = 235.6\ K$$

$$P_u = 1.2\,(235.6) + 1.6\,(109.8) + 0.5\,(5.5) = 461.2\ K$$

- Load above level 4: roof + 9 flrs

$$P_L = (0.40\,(100)(9) + 30)\ 346.5 = 135.1\ K$$

$$P_S = 5.5K$$

$$P_D = (85\,(9) + 85)\ 346.5 = 294.5\ K$$

$$P_u = 1.2\,(294.5) + 1.6\,(135.1) + 0.5\,(5.5) = 572.3K$$

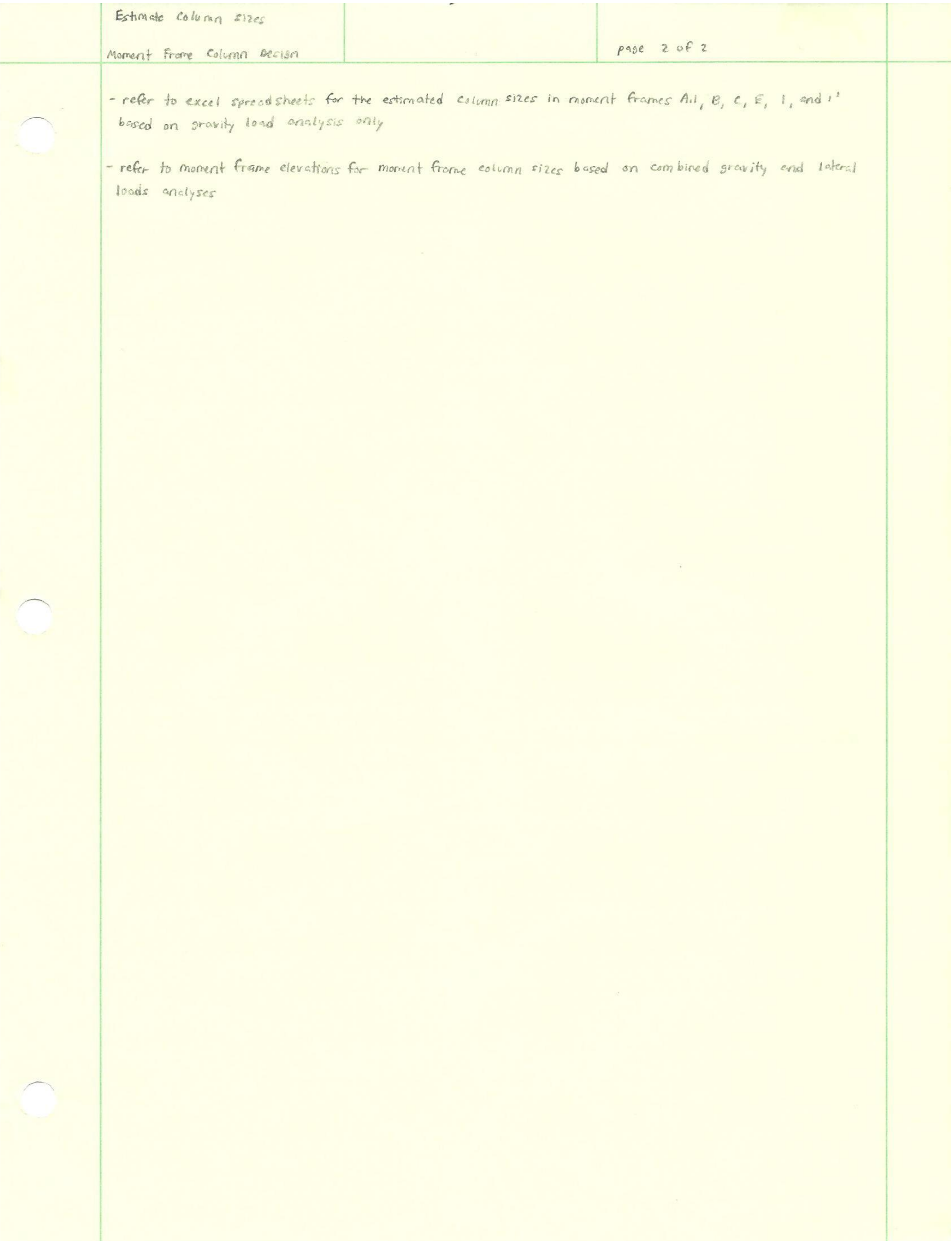
- Load above level 2: roof + 11 flrs

$$P_L = (0.4\,(100)(11) + 30)\ 346.5 = 162.9\ K$$

$$P_S = 5.5K$$

$$P_D = (85\,(11) + 85)\ 346.5 = 353.4\ K$$

$$P_u = 1.2\,(353.4) + 1.6\,(162.9) + 0.5\,(5.5) = 687.5\ K$$



Estimate column sizes

Moment Frame Column Design

page 2 of 2

- refer to excel spreadsheets for the estimated column sizes in moment frames A.1, B, C, E, 1, and 1' based on gravity load analysis only

- refer to moment frame elevations for moment frame column sizes based on combined gravity and lateral loads analyses

Moment Frame A.1: Estimated column sizes based on gravity load analysis only

Column 5		
Tributary Area		
per floor or roof=	505 ft^2	
Influence Area=	1881 ft^2	
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf	
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf	
Floor Live Load= (Office Space)	100 psf	
Roof Live Load= (Green roof area)	30 psf	
Roof Constr. Live Load	20 psf	
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.60	
P_L=	45.2 kips	
P_D=	96.0 kips	
P_{Lr}=	10.1 kips	
P_U=	192.6 kips	
Unbalanced Moment, M_u=	35.2 kft	
Column Shape=	W14's	
Column Depth=	14.0 in	
Effective Length, K=	1.0	
Column Length, L=	15.0 ft	
Equivalent Axial Load, P_{eq}=	253	
Estimated Column Size:	W14x43	ΦP_n=292 k
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.45	
P_L=	83.3 kips	
P_D=	191.9 kips	
P_{Lr}=	10.1 kips	
P_U=	368.6 kips	
Unbalanced Moment, M_u=	35.2 kft	
Column Shape=	W14's	
Column Depth=	14.0 in	
Effective Length, K=	1.0	
Column Length, L=	15.0 ft	
Equivalent Axial Load, P_{eq}=	429	
Estimated Column Size:	W14x61	ΦP_n=543 k
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.40	
P_L=	117.3 kips	
P_D=	287.9 kips	
P_{Lr}=	10.1 kips	
P_U=	538.2 kips	
Unbalanced Moment, M_u=	35.2 kft	
Column Shape=	W14's	
Column Depth=	14.0 in	
Effective Length, K=	1.0	
Column Length, L=	15.0 ft	
Equivalent Axial Load, P_{eq}=	599	
Estimated Column Size:	W14x68	ΦP_n=608 k
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	156.6 kips	
P_D=	383.8 kips	
P_{Lr}=	10.1 kips	
P_U=	716.1 kips	
Unbalanced Moment, M_u=	35.2 kft	
Column Shape=	W14's	
Column Depth=	14.0 in	
Effective Length, K=	1.0	
Column Length, L=	15.0 ft	
Equivalent Axial Load, P_{eq}=	776	
Estimated Column Size:	W14x90	ΦP_n=1000 k
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	197.0 kips	
P_D=	479.8 kips	
P_{Lr}=	10.1 kips	
P_U=	895.9 kips	
Unbalanced Moment, M_u=	35.2 kft	
Column Shape=	W14's	
Column Depth=	14.0 in	
Effective Length, K=	1.0	
Column Length, L=	15.0 ft	
Equivalent Axial Load, P_{eq}=	956	
Estimated Column Size:	W14x90	ΦP_n=1000 k
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	237.4 kips	
P_D=	575.7 kips	
P_{Lr}=	10.1 kips	
P_U=	1075.7 kips	
Unbalanced Moment, M_u=	35.2 kft	
Column Shape=	W14's	
Column Depth=	14.0 in	
Effective Length, K=	1.0	
Column Length, L=	15.0 ft	
Equivalent Axial Load, P_{eq}=	1136	
Estimated Column Size:	W14x109	ΦP_n=1210 k

Column 6		
Tributary Area		
per floor or roof=	505 ft^2	
Influence Area=	1881 ft^2	
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf	
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf	
Floor Live Load= (Office Space)	100 psf	
Roof Live Load= (Terrace)	100 psf	
Roof Constr. Live Load, Lr	20 psf	
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.60	
P_L=	80.6 kips	
P_D=	96.0 kips	
P_{Lr}=	10.1 kips	
P_U=	249.1 kips	
Unbalanced Moment, M_u=	35.2 kft	
Column Shape=	W14's	
Column Depth=	14.0 in	
Effective Length, K=	1.0	
Column Length, L=	15.0 ft	
Equivalent Axial Load, P_{eq}=	309	
Estimated Column Size:	W14x48	ΦP_n=331 k
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.45	
P_L=	118.6 kips	
P_D=	191.9 kips	
P_{Lr}=	10.1 kips	
P_U=	425.1 kips	
Unbalanced Moment, M_u=	35.2 kft	
Column Shape=	W14's	
Column Depth=	14.0 in	
Effective Length, K=	1.0	
Column Length, L=	15.0 ft	
Equivalent Axial Load, P_{eq}=	485	
Estimated Column Size:	W14x61	ΦP_n=543 k
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.40	
P_L=	152.7 kips	
P_D=	287.9 kips	
P_{Lr}=	10.1 kips	
P_U=	594.8 kips	
Unbalanced Moment, M_u=	35.2 kft	
Column Shape=	W14's	
Column Depth=	14.0 in	
Effective Length, K=	1.0	
Column Length, L=	15.0 ft	
Equivalent Axial Load, P_{eq}=	655	
Estimated Column Size:	W14x74	ΦP_n=667 k
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	191.9 kips	
P_D=	383.8 kips	
P_{Lr}=	10.1 kips	
P_U=	772.7 kips	
Unbalanced Moment, M_u=	35.2 kft	
Column Shape=	W14's	
Column Depth=	14.0 in	
Effective Length, K=	1.0	
Column Length, L=	15.0 ft	
Equivalent Axial Load, P_{eq}=	833	
Estimated Column Size:	W14x90	ΦP_n=1000 k
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	232.3 kips	
P_D=	479.8 kips	
P_{Lr}=	10.1 kips	
P_U=	952.4 kips	
Unbalanced Moment, M_u=	35.2 kft	
Column Shape=	W14's	
Column Depth=	14.0 in	
Effective Length, K=	1.0	
Column Length, L=	15.0 ft	
Equivalent Axial Load, P_{eq}=	1013	
Estimated Column Size:	W14x99	ΦP_n=1100 k
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	272.7 kips	
P_D=	575.7 kips	
P_{Lr}=	10.1 kips	
P_U=	1132.2 kips	
Unbalanced Moment, M_u=	35.2 kft	
Column Shape=	W14's	
Column Depth=	14.0 in	
Effective Length, K=	1.0	
Column Length, L=	15.0 ft	
Equivalent Axial Load, P_{eq}=	1193	
Estimated Column Size:	W14x109	ΦP_n=1210 k

Column 7		
Tributary Area		
per floor or roof=	623.4 ft^2	
Influence Area=	2221 ft^2	
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf	
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf	
Floor Live Load= (Office Space)	100 psf	
Roof Live Load= (Terrace)	100 psf	
Roof Constr. Live Load	20 psf	
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.57	
P_L=	97.8 kips	
P_D=	118.4 kips	
P_{Lr}=	12.5 kips	
P_U=	304.8 kips	
Unbalanced Moment, M_u=	53.7 kft	
Column Shape=	W14's	
Column Depth=	14.0 in	
Effective Length, K=	1.0	
Column Length, L=	15.0 ft	
Equivalent Axial Load, P_{eq}=	397	
Estimated Column Size:	W14x61	ΦP_n=543 k
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.43	
P_L=	143.5 kips	
P_D=	236.9 kips	
P_{Lr}=	12.5 kips	
P_U=	520.0 kips	
Unbalanced Moment, M_u=	53.7 kft	
Column Shape=	W14's	
Column Depth=	14.0 in	
Effective Length, K=	1.0	
Column Length, L=	15.0 ft	
Equivalent Axial Load, P_{eq}=	612	
Estimated Column Size:	W14x68	ΦP_n=667 k
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.40	
P_L=	187.0 kips	
P_D=	355.3 kips	
P_{Lr}=	12.5 kips	
P_U=	731.9 kips	
Unbalanced Moment, M_u=	53.7 kft	
Column Shape=	W14's	
Column Depth=	14.0 in	
Effective Length, K=	1.0	
Column Length, L=	15.0 ft	
Equivalent Axial Load, P_{eq}=	824	
Estimated Column Size:	W14x90	ΦP_n=1000 k
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	236.9 kips	
P_D=	473.8 kips	
P_{Lr}=	12.5 kips	
P_U=	953.8 kips	
Unbalanced Moment, M_u=	53.7 kft	
Column Shape=	W14's	
Column Depth=	14.0 in	
Effective Length, K=	1.0	
Column Length, L=	15.0 ft	
Equivalent Axial Load, P_{eq}=	1046	
Estimated Column Size:	W14x99	ΦP_n=1100 k
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	286.8 kips	
P_D=	592.2 kips	
P_{Lr}=	12.5 kips	
P_U=	1175.7 kips	
Unbalanced Moment, M_u=	53.7 kft	
Column Shape=	W14's	
Column Depth=	14.0 in	
Effective Length, K=	1.0	
Column Length, L=	15.0 ft	
Equivalent Axial Load, P_{eq}=	1268	
Estimated Column Size:	W14x120	ΦP_n=1340 k
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	336.6 kips	
P_D=	710.7 kips	
P_{Lr}=	12.5 kips	
P_U=	1397.7 kips	
Unbalanced Moment, M_u=	53.7 kft	
Column Shape=	W14's	
Column Depth=	14.0 in	
Effective Length, K=	1.0	
Column Length, L=	15.0 ft	
Equivalent Axial Load, P_{eq}=	1490	
Estimated Column Size:	W14x145	ΦP_n=1650 k

Column 8		
Tributary Area		
per floor or roof=	623.4	ft^2
Influence Area=	2221	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Terrace)	100	psf
Roof Constr. Live Load	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.57	
P_L=	97.8	kips
P_D=	118.4	kips
P_{Lr}=	12.5	kips
P_U=	304.8	kips
Unbalanced Moment, M_u=	53.7	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	397	
Estimated Column Size:	**W14x61**	**ΦP_n=543 k**
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.43	
P_L=	143.5	kips
P_D=	236.9	kips
P_{Lr}=	12.5	kips
P_U=	520.0	kips
Unbalanced Moment, M_u=	53.7	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	612	
Estimated Column Size:	**W14x68**	**ΦP_n=667 k**
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.40	
P_L=	187.0	kips
P_D=	355.3	kips
P_{Lr}=	12.5	kips
P_U=	731.9	kips
Unbalanced Moment, M_u=	53.7	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	824	
Estimated Column Size:	**W14x90**	**ΦP_n=1000 k**
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	236.9	kips
P_D=	473.8	kips
P_{Lr}=	12.5	kips
P_U=	953.8	kips
Unbalanced Moment, M_u=	53.7	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1046	
Estimated Column Size:	**W14x99**	**ΦP_n=1100 k**
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	286.8	kips
P_D=	592.2	kips
P_{Lr}=	12.5	kips
P_U=	1175.7	kips
Unbalanced Moment, M_u=	53.7	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1268	
Estimated Column Size:	**W14x120**	**ΦP_n=1340 k**
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	336.6	kips
P_D=	710.7	kips
P_{Lr}=	12.5	kips
P_U=	1397.7	kips
Unbalanced Moment, M_u=	53.7	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1490	
Estimated Column Size:	**W14x145**	**ΦP_n=1650 k**

Column 9		
Tributary Area		
per floor or roof=	505	ft^2
Influence Area=	1881	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Terrace)	100	psf
Roof Constr. Live Load	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.60	
P_L=	80.6	kips
P_D=	96.0	kips
P_{Lr}=	10.1	kips
P_U=	249.1	kips
Unbalanced Moment, M_u=	35.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	309	
Estimated Column Size:	**W14x48**	**ΦP_n=331 k**
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.45	
P_L=	118.6	kips
P_D=	191.9	kips
P_{Lr}=	10.1	kips
P_U=	425.1	kips
Unbalanced Moment, M_u=	35.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	485	
Estimated Column Size:	**W14x61**	**ΦP_n=543 k**
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.40	
P_L=	152.7	kips
P_D=	287.9	kips
P_{Lr}=	10.1	kips
P_U=	594.8	kips
Unbalanced Moment, M_u=	35.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	655	
Estimated Column Size:	**W14x74**	**ΦP_n=667 k**
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	191.9	kips
P_D=	383.8	kips
P_{Lr}=	10.1	kips
P_U=	772.7	kips
Unbalanced Moment, M_u=	35.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	833	
Estimated Column Size:	**W14x90**	**ΦP_n=1000 k**
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	232.3	kips
P_D=	479.8	kips
P_{Lr}=	10.1	kips
P_U=	952.4	kips
Unbalanced Moment, M_u=	35.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1013	
Estimated Column Size:	**W14x99**	**ΦP_n=1100 k**
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	272.7	kips
P_D=	575.7	kips
P_{Lr}=	10.1	kips
P_U=	1132.2	kips
Unbalanced Moment, M_u=	35.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1193	
Estimated Column Size:	**W14x109**	**ΦP_n=1210 k**

Column 10		
Tributary Area		
per floor or roof=	505	ft^2
Influence Area=	1881	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Terrace)	100	psf
Roof Constr. Live Load	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.60	
P_L=	80.6	kips
P_D=	96.0	kips
P_{Lr}=	10.1	kips
P_U=	249.1	kips
Unbalanced Moment, M_u=	35.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	309	
Estimated Column Size:	**W14x48**	**ΦP_n=331 k**
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.45	
P_L=	118.6	kips
P_D=	191.9	kips
P_{Lr}=	10.1	kips
P_U=	425.1	kips
Unbalanced Moment, M_u=	35.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	485	
Estimated Column Size:	**W14x61**	**ΦP_n=543 k**
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.40	
P_L=	152.7	kips
P_D=	287.9	kips
P_{Lr}=	10.1	kips
P_U=	594.8	kips
Unbalanced Moment, M_u=	35.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	655	
Estimated Column Size:	**W14x74**	**ΦP_n=667 k**
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	191.9	kips
P_D=	383.8	kips
P_{Lr}=	10.1	kips
P_U=	772.7	kips
Unbalanced Moment, M_u=	35.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	833	
Estimated Column Size:	**W14x90**	**ΦP_n=1000 k**
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	232.3	kips
P_D=	479.8	kips
P_{Lr}=	10.1	kips
P_U=	952.4	kips
Unbalanced Moment, M_u=	35.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1013	
Estimated Column Size:	**W14x99**	**ΦP_n=1100 k**
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	272.7	kips
P_D=	575.7	kips
P_{Lr}=	10.1	kips
P_U=	1132.2	kips
Unbalanced Moment, M_u=	35.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1193	
Estimated Column Size:	**W14x109**	**ΦP_n=1210 k**

Column 11		
Tributary Area		
per floor or roof=	623.4	ft^2
Influence Area=	2221	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Terrace)	100	psf
Roof Constr. Live Load	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.57	
P_L=	97.8	kips
P_D=	118.4	kips
P_{Lr}=	12.5	kips
P_U=	304.8	kips
Unbalanced Moment, M_u=	53.7	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	397	
Estimated Column Size:	W14x61	ΦP_n=543 k
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.43	
P_L=	143.5	kips
P_D=	236.9	kips
P_{Lr}=	12.5	kips
P_U=	520.0	kips
Unbalanced Moment, M_u=	53.7	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	612	
Estimated Column Size:	W14x74	ΦP_n=667 k
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.39	
P_L=	184.6	kips
P_D=	355.3	kips
P_{Lr}=	12.5	kips
P_U=	728.1	kips
Unbalanced Moment, M_u=	53.7	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	820	
Estimated Column Size:	W14x90	ΦP_n=1000 k
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	236.9	kips
P_D=	473.8	kips
P_{Lr}=	12.5	kips
P_U=	953.8	kips
Unbalanced Moment, M_u=	53.7	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1046	
Estimated Column Size:	W14x99	ΦP_n=1100 k
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	286.8	kips
P_D=	592.2	kips
P_{Lr}=	12.5	kips
P_U=	1175.7	kips
Unbalanced Moment, M_u=	53.7	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1268	
Estimated Column Size:	W14x120	ΦP_n=1340 k
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	336.6	kips
P_D=	710.7	kips
P_{Lr}=	12.5	kips
P_U=	1397.7	kips
Unbalanced Moment, M_u=	53.7	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1490	
Estimated Column Size:	W14x145	ΦP_n=1650 k

Column 12		
Tributary Area		
per floor or roof=	663	ft^2
Influence Area=	2361	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Terrace)	100	psf
Roof Constr. Live Load	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.56	
P_L=	103.3	kips
P_D=	126.0	kips
P_{Lr}=	13.3	kips
P_U=	323.1	kips
Unbalanced Moment, M_u=	72.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	447	
Estimated Column Size:	W14x61	ΦP_n=543 k
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.43	
P_L=	151.5	kips
P_D=	251.9	kips
P_{Lr}=	13.3	kips
P_U=	551.3	kips
Unbalanced Moment, M_u=	72.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	676	
Estimated Column Size:	W14x82	ΦP_n=735 k
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.40	
P_L=	198.9	kips
P_D=	377.9	kips
P_{Lr}=	13.3	kips
P_U=	778.4	kips
Unbalanced Moment, M_u=	72.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	903	
Estimated Column Size:	W14x90	ΦP_n=1000 k
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	251.9	kips
P_D=	503.9	kips
P_{Lr}=	13.3	kips
P_U=	1014.4	kips
Unbalanced Moment, M_u=	72.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1139	
Estimated Column Size:	W14x109	ΦP_n=1210 k
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	305.0	kips
P_D=	629.9	kips
P_{Lr}=	13.3	kips
P_U=	1250.4	kips
Unbalanced Moment, M_u=	72.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1375	
Estimated Column Size:	W14x132	ΦP_n=1480 k
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	358.0	kips
P_D=	755.8	kips
P_{Lr}=	13.3	kips
P_U=	1486.4	kips
Unbalanced Moment, M_u=	72.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1611	
Estimated Column Size:	W14x145	ΦP_n=1650 k

Note: Refer to Moment Frame A.1 elevation in final report for column sizes that support combined gravity and lateral loads

Moment Frame B: Estimated column sizes based on gravity load analysis only

Column 15	
Tributary Area	
per floor or roof=	559 ft^2
Influence Area=	2130 ft^2
Floor Dead Load= (slab+SDL)	95 psf
Roof Dead Load= (slab+SDL)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Green roof area)	30 psf
Roof Constr. Live Load	20 psf

Load above level 12:	Roof +
	1 Floor
LL_{red}	0.58
P_L=	48.9 kips
P_D=	106.2 kips
P_{Lr}=	11.2 kips
P_U=	211.3 kips
Unbalanced Moment, M_u=	178.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	517
Estimated Column Size:	W14x61 ΦP_n=543 k

Load above level 10:	Roof +
	3 Floors
LL_{red}	0.44
P_L=	90.2 kips
P_D=	212.4 kips
P_{Lr}=	11.2 kips
P_U=	404.8 kips
Unbalanced Moment, M_u=	178.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	711
Estimated Column Size:	W14x82 ΦP_n=735 k

Load above level 8:	Roof +
	5 Floors
LL_{red}	0.40
P_L=	128.6 kips
P_D=	318.6 kips
P_{Lr}=	11.2 kips
P_U=	593.7 kips
Unbalanced Moment, M_u=	178.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	900
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 6:	Roof +
	7 Floors
LL_{red}	0.40
P_L=	173.3 kips
P_D=	424.8 kips
P_{Lr}=	11.2 kips
P_U=	792.7 kips
Unbalanced Moment, M_u=	178.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1099
Estimated Column Size:	W14x99 ΦP_n=1100 k

Load above level 4:	Roof +
	9 Floors
LL_{red}	0.40
P_L=	218.0 kips
P_D=	531.1 kips
P_{Lr}=	11.2 kips
P_U=	991.7 kips
Unbalanced Moment, M_u=	178.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1298
Estimated Column Size:	W14x120 ΦP_n=1340 k

Load above level 2:	Roof +
	11 Floors
LL_{red}	0.40
P_L=	262.7 kips
P_D=	637.3 kips
P_{Lr}=	11.2 kips
P_U=	1190.7 kips
Unbalanced Moment, M_u=	178.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1497
Estimated Column Size:	W14x145 ΦP_n=1650 k

Column 16	
Tributary Area	
per floor or roof=	1127 ft^2
Influence Area=	4508.5 ft^2
Floor Dead Load= (slab+SDL)	95 psf
Roof Dead Load= (slab+SDL)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Cooling Towers)	150 psf
Roof Constr. Live Load	20 psf

Load above level 12:	Roof +
	1 Floor
LL_{red}	0.50
P_L=	225.4 kips
P_D=	214.1 kips
P_{Lr}=	22.5 kips
P_U=	628.9 kips
Unbalanced Moment, M_u=	87.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	778
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 10:	Roof +
	3 Floors
LL_{red}	0.40
P_L=	304.3 kips
P_D=	428.3 kips
P_{Lr}=	22.5 kips
P_U=	1012.0 kips
Unbalanced Moment, M_u=	87.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1161
Estimated Column Size:	W14x109 ΦP_n=1210 k

Load above level 8:	Roof +
	5 Floors
LL_{red}	0.40
P_L=	394.5 kips
P_D=	642.4 kips
P_{Lr}=	22.5 kips
P_U=	1413.3 kips
Unbalanced Moment, M_u=	87.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1562
Estimated Column Size:	W14x145 ΦP_n=1650 k

Load above level 6:	Roof +
	7 Floors
LL_{red}	0.40
P_L=	484.6 kips
P_D=	856.5 kips
P_{Lr}=	22.5 kips
P_U=	1814.5 kips
Unbalanced Moment, M_u=	87.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1964
Estimated Column Size:	W14x176 ΦP_n=2010 k

Load above level 4:	Roof +
	9 Floors
LL_{red}	0.40
P_L=	574.8 kips
P_D=	1070.7 kips
P_{Lr}=	22.5 kips
P_U=	2215.7 kips
Unbalanced Moment, M_u=	87.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	2365 kips
Estimated Column Size:	W14x211 ΦP_n=2420 k

Load above level 2:	Roof +
	11 Floors
LL_{red}	0.40
P_L=	664.9 kips
P_D=	1284.8 kips
P_{Lr}=	22.5 kips
P_U=	2616.9 kips
Unbalanced Moment, M_u=	87.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	2766 kips
Estimated Column Size:	W14x257 ΦP_n=2960 k

Column 17	
Tributary Area	
per floor or roof=	964.5 ft^2
Influence Area=	3857.4 ft^2
Floor Dead Load= (slab+SDL)	95 psf
Roof Dead Load= (slab+SDL)	95 psf
Floor Live Load= (Mech. Room)	150 psf
Roof Live Load= (Cooling Towers)	150 psf
Roof Constr. Live Load	20 psf

Load above level 12:	Roof +
	1 Floor
LL_{red}	1.00
P_L=	289.4 kips
P_D=	183.3 kips
P_{Lr}=	19.3 kips
P_U=	692.5 kips
Unbalanced Moment, M_u=	108.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	878 kips
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 10:	Roof +
	3 Floors
LL_{red}	0.80
P_L=	491.9 kips
P_D=	366.5 kips
P_{Lr}=	19.3 kips
P_U=	1236.5 kips
Unbalanced Moment, M_u=	108.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1422 kips
Estimated Column Size:	W14x132 ΦP_n=1480 k

Load above level 8:	Roof +
	5 Floors
LL_{red}	0.80
P_L=	723.4 kips
P_D=	549.8 kips
P_{Lr}=	19.3 kips
P_U=	1826.8 kips
Unbalanced Moment, M_u=	108.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	2012 kips
Estimated Column Size:	W14x193 ΦP_n=2210 k

Load above level 6:	Roof +
	7 Floors
LL_{red}	0.80
P_L=	954.9 kips
P_D=	733.0 kips
P_{Lr}=	19.3 kips
P_U=	2417.0 kips
Unbalanced Moment, M_u=	108.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	2602 kips
Estimated Column Size:	W14x233 ΦP_n=2680 k

Load above level 4:	Roof +
	9 Floors
LL_{red}	0.80
P_L=	1186.3 kips
P_D=	916.3 kips
P_{Lr}=	19.3 kips
P_U=	3007.3 kips
Unbalanced Moment, M_u=	108.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	3192 kips
Estimated Column Size:	W14x283 ΦP_n=3270 k

Load above level 2:	Roof +
	11 Floors
LL_{red}	0.80
P_L=	1417.8 kips
P_D=	1099.5 kips
P_{Lr}=	19.3 kips
P_U=	3597.6 kips
Unbalanced Moment, M_u=	108.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	3783 kips
Estimated Column Size:	W14x342 ΦP_n=3980 k

Column 18	
Tributary Area	
per floor or roof=	452.5 ft^2
Influence Area=	1810 ft^2
Floor Dead Load= (slab+SDL)	95 psf
Roof Dead Load= (slab+SDL)	95 psf
Floor Live Load= (Mech. Room)	150 psf
Roof Live Load= (Mech. Room)	150 psf
Roof Constr. Live Load	20 psf

Load above level 12:	Roof +
	1 Floor
LL_{red}	1.00
P_L=	135.8 kips
P_D=	86.0 kips
P_{Lr}=	9.1 kips
P_U=	359.6 kips
Unbalanced Moment, M_u=	53.3 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	451 kips
Estimated Column Size:	W14x61 ΦP_n=543 k

Load above level 10:	Roof +
	3 Floors
LL_{red}	0.80
P_L=	230.8 kips
P_D=	172.0 kips
P_{Lr}=	9.1 kips
P_U=	614.8 kips
Unbalanced Moment, M_u=	53.3 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	706 kips
Estimated Column Size:	W14x82 ΦP_n=735 k

Load above level 8:	Roof +
	5 Floors
LL_{red}	0.80
P_L=	339.4 kips
P_D=	257.9 kips
P_{Lr}=	9.1 kips
P_U=	891.7 kips
Unbalanced Moment, M_u=	53.3 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	983 kips
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 6:	Roof +
	7 Floors
LL_{red}	0.80
P_L=	448.0 kips
P_D=	343.9 kips
P_{Lr}=	9.1 kips
P_U=	1168.7 kips
Unbalanced Moment, M_u=	53.3 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1260 kips
Estimated Column Size:	W14x120 ΦP_n=1340 k

Load above level 4:	Roof +
	9 Floors
LL_{red}	0.80
P_L=	556.6 kips
P_D=	429.9 kips
P_{Lr}=	9.1 kips
P_U=	1445.6 kips
Unbalanced Moment, M_u=	53.3 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1537 kips
Estimated Column Size:	W14x145 ΦP_n=1650 k

Load above level 2:	Roof +
	11 Floors
LL_{red}	0.80
P_L=	665.2 kips
P_D=	515.9 kips
P_{Lr}=	9.1 kips
P_U=	1722.5 kips
Unbalanced Moment, M_u=	108.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1908 kips
Estimated Column Size:	W14x176 ΦP_n=2010 k

Tributary area per PH roof=	209 ft^2
Influence area on PH roof=	835 ft^2
Roof Dead Load= (slab+SDL)	90 psf
Roof Live Load= (green roof area)	30 psf
Snow Load=	15.75 psf
Roof constr Live Load=	20 psf

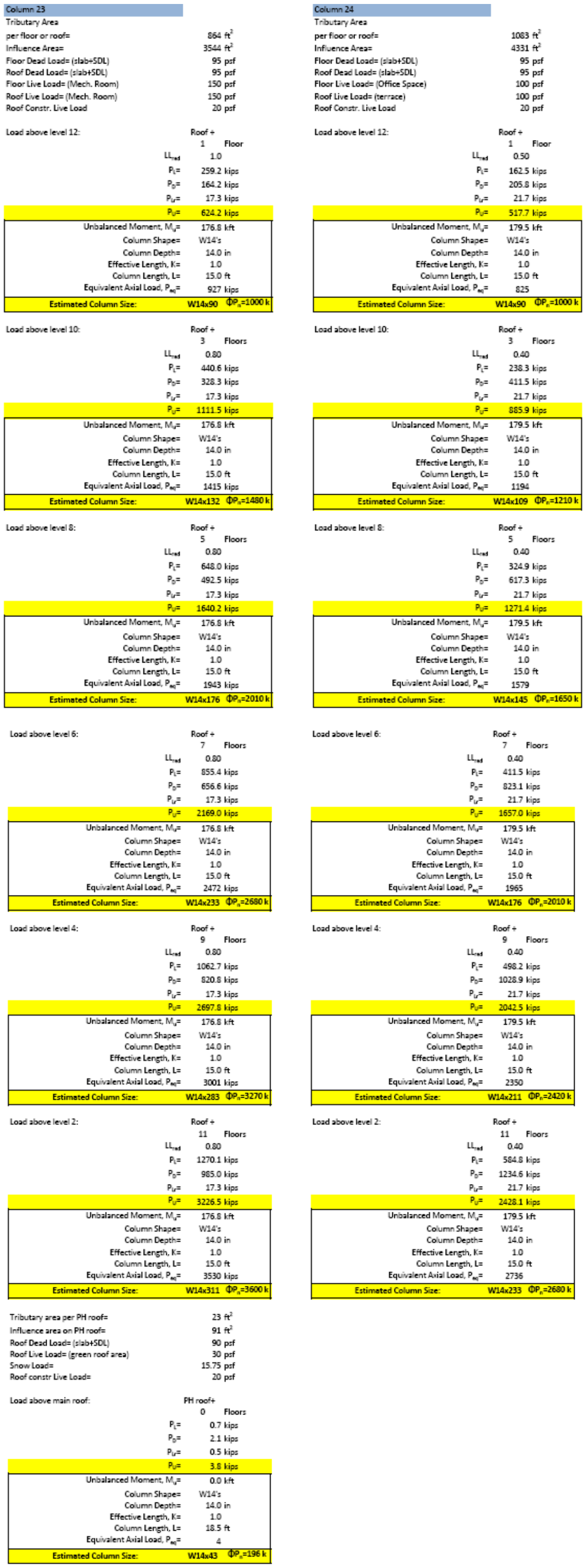
Load above main roof:	PH roof+
	0 Floors
P_L=	6.3 kips
P_D=	18.8 kips
P_{Lr}=	4.2 kips
P_U=	34.7 kips
Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	18.5 ft
Equivalent Axial Load, P_{eq}=	35 kips
Estimated Column Size:	W14x43 ΦP_n=196 k

Column 19		
Tributary Area		
per floor or roof=	864	ft^2
Influence Area=	3455	ft^2
Floor Dead Load= (slab+SDL)	95	psf
Roof Dead Load= (slab+SDL)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Mech. Room)	150	psf
Roof Constr. Live Load	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.51	
P_L=	173.2	kips
P_D=	164.2	kips
P_{Lr}=	17.3	kips
P_U=	554.5	kips
Unbalanced Moment, M_u=	177.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	858	kips
Estimated Column Size:	**W14x90**	**ΦP_n=1000 k**
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.40	
P_L=	233.3	kips
P_D=	328.3	kips
P_{Lr}=	17.3	kips
P_U=	847.6	kips
Unbalanced Moment, M_u=	177.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1151	kips
Estimated Column Size:	**W14x109**	**ΦP_n=1210 k**
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.40	
P_L=	302.4	kips
P_D=	492.5	kips
P_{Lr}=	17.3	kips
P_U=	1155.2	kips
Unbalanced Moment, M_u=	177.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1459	kips
Estimated Column Size:	**W14x132**	**ΦP_n=1480 k**
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	371.5	kips
P_D=	656.6	kips
P_{Lr}=	17.3	kips
P_U=	1462.8	kips
Unbalanced Moment, M_u=	177.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1766	kips
Estimated Column Size:	**W14x159**	**ΦP_n=1810 k**
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	440.6	kips
P_D=	820.8	kips
P_{Lr}=	17.3	kips
P_U=	1770.3	kips
Unbalanced Moment, M_u=	177.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	2074	kips
Estimated Column Size:	**W14x193**	**ΦP_n=2210 k**
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	509.8	kips
P_D=	985.0	kips
P_{Lr}=	17.3	kips
P_U=	2077.9	kips
Unbalanced Moment, M_u=	177.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	2381	kips
Estimated Column Size:	**W14x211**	**ΦP_n=2420 k**
Tributary area per PH roof=	432	ft^2
Influence area on PH roof=	1727	ft^2
Roof Dead Load= (slab+SDL)	90	psf
Roof Live Load= (green roof area)	30	psf
Snow Load=	15.75	psf
Roof constr Live Load=	20	psf
Load above main roof:	PH roof+	
	0	Floors
P_L=	13.0	kips
P_D=	38.9	kips
P_{Lr}=	8.6	kips
P_U=	71.7	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	18.5	ft
Equivalent Axial Load, P_{eq}=	72	kips
Estimated Column Size:	**W14x43**	**ΦP_n=196 k**

Column 20		
Tributary Area		
per floor or roof=	1062	ft^2
Influence Area=	4168	ft^2
Floor Dead Load= (slab+SDL)	95	psf
Roof Dead Load= (slab+SDL)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Mech. Room)	150	psf
Roof Constr. Live Load	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.50	
P_L=	212.4	kips
P_D=	201.8	kips
P_{Lr}=	21.2	kips
P_U=	688.5	kips
Unbalanced Moment, M_u=	136.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	922	kips
Estimated Column Size:	**W14x90**	**ΦP_n=1000 k**
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.40	
P_L=	286.7	kips
P_D=	403.6	kips
P_{Lr}=	21.2	kips
P_U=	1049.6	kips
Unbalanced Moment, M_u=	136.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1283	kips
Estimated Column Size:	**W14x120**	**ΦP_n=1340 k**
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.40	
P_L=	339.8	kips
P_D=	605.3	kips
P_{Lr}=	21.2	kips
P_U=	1376.7	kips
Unbalanced Moment, M_u=	136.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1610	kips
Estimated Column Size:	**W14x145**	**ΦP_n=1650 k**
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	456.7	kips
P_D=	807.1	kips
P_{Lr}=	21.2	kips
P_U=	1805.8	kips
Unbalanced Moment, M_u=	136.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	2039	
Estimated Column Size:	**W14x193**	**ΦP_n=2210 k**
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	541.6	kips
P_D=	1008.9	kips
P_{Lr}=	21.2	kips
P_U=	2183.8	kips
Unbalanced Moment, M_u=	136.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	2417	kips
Estimated Column Size:	**W14x211**	**ΦP_n=2420 k**
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	626.6	kips
P_D=	1210.7	kips
P_{Lr}=	21.2	kips
P_U=	2561.9	kips
Unbalanced Moment, M_u=	136.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	2795	kips
Estimated Column Size:	**W14x257**	**ΦP_n=2960 k**
Tributary area per PH roof=	578	ft^2
Influence area on PH roof=	2231	ft^2
Roof Dead Load= (slab+SDL)	90	psf
Roof Live Load= (green roof area)	30	psf
Snow Load=	15.75	psf
Roof constr Live Load=	20	psf
Load above main roof:	PH roof+	
	0	Floors
P_L=	17.3	kips
P_D=	52.0	kips
P_{Lr}=	11.6	kips
P_U=	95.9	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	18.5	ft
Equivalent Axial Load, P_{eq}=	96	
Estimated Column Size:	**W14x43**	**ΦP_n=196 k**

Column 21		
Tributary Area		
per floor or roof=	1027	ft^2
Influence Area=	3022	ft^2
Floor Dead Load= (slab+SDL)	95	psf
Roof Dead Load= (slab+SDL)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Mech. Room)	150	psf
Roof Constr. Live Load	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.52	
P_L=	207.7	kips
P_D=	195.1	kips
P_{Lr}=	20.5	kips
P_U=	635.8	kips
Unbalanced Moment, M_u=	77.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	769	
Estimated Column Size:	**W14x90**	**ΦP_n=1000 k**
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.41	
P_L=	279.6	kips
P_D=	390.3	kips
P_{Lr}=	20.5	kips
P_U=	984.9	kips
Unbalanced Moment, M_u=	77.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1118	
Estimated Column Size:	**W14x99**	**ΦP_n=1100 k**
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.40	
P_L=	359.5	kips
P_D=	585.4	kips
P_{Lr}=	20.5	kips
P_U=	1346.8	kips
Unbalanced Moment, M_u=	77.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1480	
Estimated Column Size:	**W14x132**	**ΦP_n=1480 k**
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	441.6	kips
P_D=	780.5	kips
P_{Lr}=	20.5	kips
P_U=	1712.4	kips
Unbalanced Moment, M_u=	77.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1845	
Estimated Column Size:	**W14x159**	**ΦP_n=1810 k**
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	523.8	kips
P_D=	975.7	kips
P_{Lr}=	20.5	kips
P_U=	2078.0	kips
Unbalanced Moment, M_u=	77.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	2211	
Estimated Column Size:	**W14x193**	**ΦP_n=2210 k**
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	605.9	kips
P_D=	1170.8	kips
P_{Lr}=	20.5	kips
P_U=	2404.7	kips
Unbalanced Moment, M_u=	77.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	2538	
Estimated Column Size:	**W14x233**	**ΦP_n=2680 k**
Tributary area per PH roof=	355	ft^2
Influence area on PH roof=	1419	ft^2
Roof Dead Load= (slab+SDL)	90	psf
Roof Live Load= (green roof area)	30	psf
Snow Load=	15.75	psf
Roof constr Live Load=	20	psf
Load above main roof:	PH roof+	
	0	Floors
P_L=	10.7	kips
P_D=	32.0	kips
P_{Lr}=	7.1	kips
P_U=	58.9	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	18.5	ft
Equivalent Axial Load, P_{eq}=	59	
Estimated Column Size:	**W14x43**	**ΦP_n=196 k**

Column 22		
Tributary Area		
per floor or roof=	452.5	ft^2
Influence Area=	1810	ft^2
Floor Dead Load= (slab+SDL)	95	psf
Roof Dead Load= (slab+SDL)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Mech. Room)	150	psf
Roof Constr. Live Load	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.60	
P_L=	95.1	kips
P_D=	86.0	kips
P_{Lr}=	9.1	kips
P_U=	294.6	kips
Unbalanced Moment, M_u=	40.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	364	kips
Estimated Column Size:	**W14x53**	**ΦP_n=369 k**
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.45	
P_L=	129.4	kips
P_D=	172.0	kips
P_{Lr}=	9.1	kips
P_U=	452.7	kips
Unbalanced Moment, M_u=	40.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	522	
Estimated Column Size:	**W14x61**	**ΦP_n=543 k**
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.41	
P_L=	160.1	kips
P_D=	257.9	kips
P_{Lr}=	9.1	kips
P_U=	604.9	kips
Unbalanced Moment, M_u=	40.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	674	
Estimated Column Size:	**W14x82**	**ΦP_n=735 k**
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	194.6	kips
P_D=	343.9	kips
P_{Lr}=	9.1	kips
P_U=	763.2	kips
Unbalanced Moment, M_u=	40.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	833	
Estimated Column Size:	**W14x90**	**ΦP_n=1000 k**
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	230.8	kips
P_D=	429.9	kips
P_{Lr}=	9.1	kips
P_U=	924.3	kips
Unbalanced Moment, M_u=	40.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	994	
Estimated Column Size:	**W14x90**	**ΦP_n=1000 k**
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	267.0	kips
P_D=	515.9	kips
P_{Lr}=	9.1	kips
P_U=	1085.4	kips
Unbalanced Moment, M_u=	40.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1155	
Estimated Column Size:	**W14x109**	**ΦP_n=1210 k**
Tributary area per PH roof=	209	ft^2
Influence area on PH roof=	707	ft^2
Roof Dead Load= (slab+SDL)	90	psf
Roof Live Load= (green roof area)	30	psf
Snow Load=	15.75	psf
Roof constr Live Load=	20	psf
Load above main roof:	PH roof+	
	0	Floors
P_L=	6.3	kips
P_D=	18.8	kips
P_{Lr}=	4.2	kips
P_U=	34.7	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	18.5	ft
Equivalent Axial Load, P_{eq}=	35	
Estimated Column Size:	**W14x43**	**ΦP_n=196 k**



Column 23		
Tributary Area		
per floor or roof=	864	ft^2
Influence Area=	3544	ft^2
Floor Dead Load= (slab+SDL)	95	psf
Roof Dead Load= (slab+SDL)	95	psf
Floor Live Load= (Mech. Room)	150	psf
Roof Live Load= (Mech. Room)	150	psf
Roof Constr. Live Load	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	1.0	
P_L=	259.2	kips
P_D=	164.2	kips
P_{Lr}=	17.3	kips
P_U=	624.2	kips
Unbalanced Moment, M_u=	176.8	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	927	kips
Estimated Column Size:	**W14x90**	**ΦP_n=1000 k**
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.80	
P_L=	440.6	kips
P_D=	328.3	kips
P_{Lr}=	17.3	kips
P_U=	1111.5	kips
Unbalanced Moment, M_u=	176.8	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1415	kips
Estimated Column Size:	**W14x132**	**ΦP_n=1480 k**
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.80	
P_L=	648.0	kips
P_D=	492.5	kips
P_{Lr}=	17.3	kips
P_U=	1640.2	kips
Unbalanced Moment, M_u=	176.8	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1943	kips
Estimated Column Size:	**W14x176**	**ΦP_n=2010 k**
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.80	
P_L=	855.4	kips
P_D=	656.6	kips
P_{Lr}=	17.3	kips
P_U=	2169.0	kips
Unbalanced Moment, M_u=	176.8	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	2472	kips
Estimated Column Size:	**W14x233**	**ΦP_n=2680 k**
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.80	
P_L=	1062.7	kips
P_D=	820.8	kips
P_{Lr}=	17.3	kips
P_U=	2697.8	kips
Unbalanced Moment, M_u=	176.8	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	3001	kips
Estimated Column Size:	**W14x283**	**ΦP_n=3270 k**
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.80	
P_L=	1270.1	kips
P_D=	985.0	kips
P_{Lr}=	17.3	kips
P_U=	3226.5	kips
Unbalanced Moment, M_u=	176.8	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	3530	kips
Estimated Column Size:	**W14x311**	**ΦP_n=3600 k**
Tributary area per PH roof=	23	ft^2
Influence area on PH roof=	91	ft^2
Roof Dead Load= (slab+SDL)	90	psf
Roof Live Load= (green roof area)	30	psf
Snow Load=	15.75	psf
Roof constr Live Load=	20	psf
Load above main roof:	PH roof+	
	0	Floors
P_L=	0.7	kips
P_D=	2.1	kips
P_{Lr}=	0.5	kips
P_U=	3.8	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	18.5	ft
Equivalent Axial Load, P_{eq}=	4	
Estimated Column Size:	**W14x43**	**ΦP_n=196 k**

Column 24		
Tributary Area		
per floor or roof=	1083	ft^2
Influence Area=	4331	ft^2
Floor Dead Load= (slab+SDL)	95	psf
Roof Dead Load= (slab+SDL)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (terrace)	100	psf
Roof Constr. Live Load	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.50	
P_L=	162.5	kips
P_D=	205.8	kips
P_{Lr}=	21.7	kips
P_U=	517.7	kips
Unbalanced Moment, M_u=	179.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	825	
Estimated Column Size:	**W14x90**	**ΦP_n=1000 k**
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.40	
P_L=	238.3	kips
P_D=	411.5	kips
P_{Lr}=	21.7	kips
P_U=	885.9	kips
Unbalanced Moment, M_u=	179.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1194	
Estimated Column Size:	**W14x109**	**ΦP_n=1210 k**
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.40	
P_L=	324.9	kips
P_D=	617.3	kips
P_{Lr}=	21.7	kips
P_U=	1271.4	kips
Unbalanced Moment, M_u=	179.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1579	
Estimated Column Size:	**W14x145**	**ΦP_n=1650 k**
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	411.5	kips
P_D=	823.1	kips
P_{Lr}=	21.7	kips
P_U=	1657.0	kips
Unbalanced Moment, M_u=	179.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1965	
Estimated Column Size:	**W14x176**	**ΦP_n=2010 k**
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	498.2	kips
P_D=	1028.9	kips
P_{Lr}=	21.7	kips
P_U=	2042.5	kips
Unbalanced Moment, M_u=	179.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	2350	
Estimated Column Size:	**W14x211**	**ΦP_n=2420 k**
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	584.8	kips
P_D=	1234.6	kips
P_{Lr}=	21.7	kips
P_U=	2428.1	kips
Unbalanced Moment, M_u=	179.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	2736	
Estimated Column Size:	**W14x233**	**ΦP_n=2680 k**

Note: Refer to Moment Frame B elevation in final report for column sizes that support combined gravity and lateral loads

Moment Frame C: Estimated column sizes based on gravity load analysis only

Column 27		
Tributary Area		
per floor or roof=	439	ft^2
Influence Area=	1755	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Green roof area)	30	psf
Roof Constr. Live Load	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.61	
P_L=	39.9	kips
P_D=	83.4	kips
P_{Lr}=	8.8	kips
P_U=	168.3	kips
Unbalanced Moment, M_u=	77.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	301	
Estimated Column Size:	W14x48	ΦP_n=331 k
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.46	
P_L=	73.3	kips
P_D=	166.8	kips
P_{Lr}=	8.8	kips
P_U=	321.9	kips
Unbalanced Moment, M_u=	77.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	455	
Estimated Column Size:	W14x61	ΦP_n=543 k
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.41	
P_L=	103.2	kips
P_D=	250.2	kips
P_{Lr}=	8.8	kips
P_U=	469.8	kips
Unbalanced Moment, M_u=	77.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	603	
Estimated Column Size:	W14x68	ΦP_n=608 k
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	136.1	kips
P_D=	333.6	kips
P_{Lr}=	8.8	kips
P_U=	622.5	kips
Unbalanced Moment, M_u=	77.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	755	
Estimated Column Size:	W14x90	ΦP_n=1000 k
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	171.2	kips
P_D=	417.1	kips
P_{Lr}=	8.8	kips
P_U=	778.8	kips
Unbalanced Moment, M_u=	77.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	912	
Estimated Column Size:	W14x90	ΦP_n=1000 k
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	206.3	kips
P_D=	500.5	kips
P_{Lr}=	8.8	kips
P_U=	935.1	kips
Unbalanced Moment, M_u=	77.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1068	
Estimated Column Size:	W14x99	ΦP_n=1100 k

Column 28		
Tributary Area		
per floor or roof=	929	ft^2
Influence Area=	3715	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Cooling Towers)	150	psf
Roof Constr. Live Load	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.50	
P_L=	185.8	kips
P_D=	176.5	kips
P_{Lr}=	18.6	kips
P_U=	518.4	kips
Unbalanced Moment, M_u=	87.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	668	kips
Estimated Column Size:	W14x82	ΦP_n=735 k
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.40	
P_L=	250.8	kips
P_D=	353.0	kips
P_{Lr}=	18.6	kips
P_U=	834.2	kips
Unbalanced Moment, M_u=	87.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	983	kips
Estimated Column Size:	W14x90	ΦP_n=1000 k
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.40	
P_L=	325.2	kips
P_D=	529.5	kips
P_{Lr}=	18.6	kips
P_U=	1165.0	kips
Unbalanced Moment, M_u=	87.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1314	kips
Estimated Column Size:	W14x120	ΦP_n=1340 k
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	399.5	kips
P_D=	706.0	kips
P_{Lr}=	18.6	kips
P_U=	1495.7	kips
Unbalanced Moment, M_u=	87.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1645	kips
Estimated Column Size:	W14x145	ΦP_n=1650 k
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	473.8	kips
P_D=	882.6	kips
P_{Lr}=	18.6	kips
P_U=	1826.4	kips
Unbalanced Moment, M_u=	87.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1976	kips
Estimated Column Size:	W14x176	ΦP_n=2010 k
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	548.1	kips
P_D=	1059.1	kips
P_{Lr}=	18.6	kips
P_U=	2157.1	kips
Unbalanced Moment, M_u=	87.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	2306	kips
Estimated Column Size:	W14x211	ΦP_n=2420 k

Column 29		
Tributary Area		
per floor or roof=	795	ft^2
Influence Area=	3178	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Mech. Room)	150	psf
Roof Live Load= (Mech. Room)	150	psf
Roof Constr. Live Load	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	1.00	
P_L=	238.5	kips
P_D=	151.1	kips
P_{Lr}=	15.9	kips
P_U=	570.8	kips
Unbalanced Moment, M_u=	108.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	756	kips
Estimated Column Size:	W14x90	ΦP_n=1000 k
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.80	
P_L=	405.5	kips
P_D=	302.1	kips
P_{Lr}=	15.9	kips
P_U=	1019.2	kips
Unbalanced Moment, M_u=	108.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1204	kips
Estimated Column Size:	W14x109	ΦP_n=1210 k
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.80	
P_L=	596.3	kips
P_D=	453.2	kips
P_{Lr}=	15.9	kips
P_U=	1505.7	kips
Unbalanced Moment, M_u=	108.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1691	kips
Estimated Column Size:	W14x159	ΦP_n=1810 k
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.80	
P_L=	787.1	kips
P_D=	604.2	kips
P_{Lr}=	15.9	kips
P_U=	1992.3	kips
Unbalanced Moment, M_u=	108.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	2177	kips
Estimated Column Size:	W14x193	ΦP_n=2210 k
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.80	
P_L=	977.9	kips
P_D=	755.3	kips
P_{Lr}=	15.9	kips
P_U=	2478.8	kips
Unbalanced Moment, M_u=	108.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	2664	kips
Estimated Column Size:	W14x257	ΦP_n=2960 k
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.80	
P_L=	1168.7	kips
P_D=	906.3	kips
P_{Lr}=	15.9	kips
P_U=	2965.4	kips
Unbalanced Moment, M_u=	108.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	3150	kips
Estimated Column Size:	W14x283	ΦP_n=3270 k

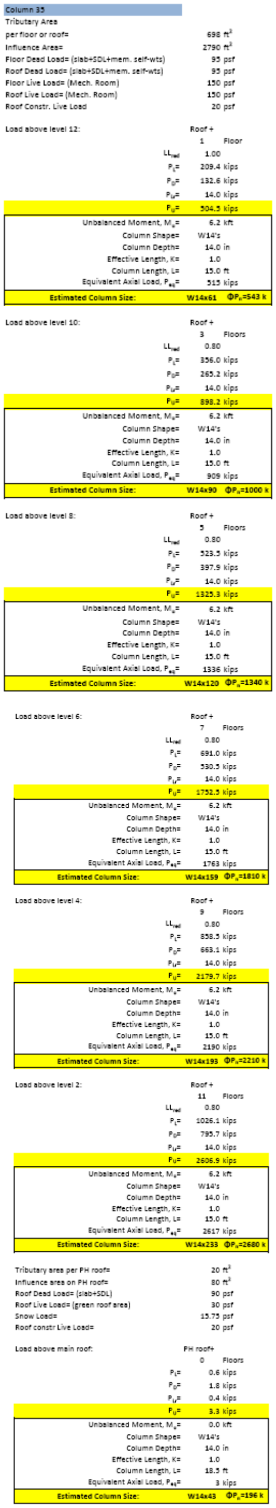
Column 30		
Tributary Area		
per floor or roof=	359	ft^2
Influence Area=	1435	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Mech. Room)	150	psf
Roof Live Load= (Mech. Room)	150	psf
Roof Constr. Live Load	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	1.00	
P_L=	107.7	kips
P_D=	68.2	kips
P_{Lr}=	7.2	kips
P_U=	292.5	kips
Unbalanced Moment, M_u=	53.3	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	384	kips
Estimated Column Size:	W14x61	ΦP_n=543 k
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.80	
P_L=	183.1	kips
P_D=	136.4	kips
P_{Lr}=	7.2	kips
P_U=	494.9	kips
Unbalanced Moment, M_u=	53.3	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	586	kips
Estimated Column Size:	W14x68	ΦP_n=608 k
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.80	
P_L=	269.3	kips
P_D=	204.6	kips
P_{Lr}=	7.2	kips
P_U=	714.6	kips
Unbalanced Moment, M_u=	53.3	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	806	kips
Estimated Column Size:	W14x90	ΦP_n=1000 k
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.80	
P_L=	355.4	kips
P_D=	272.8	kips
P_{Lr}=	7.2	kips
P_U=	934.3	kips
Unbalanced Moment, M_u=	53.3	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1026	kips
Estimated Column Size:	W14x99	ΦP_n=1100 k
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.80	
P_L=	441.6	kips
P_D=	341.1	kips
P_{Lr}=	7.2	kips
P_U=	1154.1	kips
Unbalanced Moment, M_u=	53.3	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1245	kips
Estimated Column Size:	W14x120	ΦP_n=1340 k
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.80	
P_L=	527.7	kips
P_D=	409.3	kips
P_{Lr}=	7.2	kips
P_U=	1373.8	kips
Unbalanced Moment, M_u=	53.3	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1465	kips
Estimated Column Size:	W14x132	ΦP_n=1480 k
Tributary area per PH roof=	209	ft^2
Influence area on PH roof=	835	ft^2
Roof Dead Load= (slab+SDL)	90	psf
Roof Live Load= (green roof area)	30	psf
Snow Load=	15.75	psf
Roof constr Live Load=	20	psf
Load above main roof:	PH roof+	
	0	Floors
P_L=	6.3	kips
P_D=	18.8	kips
P_{Lr}=	4.2	kips
P_U=	34.7	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	18.5	ft
Equivalent Axial Load, P_{eq}=	35	kips
Estimated Column Size:	W14x43	ΦP_n=196 k

Column 31		
Tributary Area		
per floor or roof=	442	ft^2
Influence Area=	1767	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Mech. Room)	150	psf
Roof Constr. Live Load	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.61	
P_L=	93.1	kips
P_D=	84.0	kips
P_{Lr}=	8.8	kips
P_U=	336.9	kips
Unbalanced Moment, M_u=	177.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	640	kips
Estimated Column Size:	**W14x74**	**ΦP_n=667 k**
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.46	
P_L=	126.8	kips
P_D=	168.0	kips
P_{Lr}=	8.8	kips
P_U=	491.5	kips
Unbalanced Moment, M_u=	177.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	795	kips
Estimated Column Size:	**W14x90**	**ΦP_n=1000 k**
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.41	
P_L=	156.8	kips
P_D=	251.9	kips
P_{Lr}=	8.8	kips
P_U=	640.3	kips
Unbalanced Moment, M_u=	177.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	944	kips
Estimated Column Size:	**W14x90**	**ΦP_n=1000 k**
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	190.1	kips
P_D=	335.9	kips
P_{Lr}=	8.8	kips
P_U=	794.3	kips
Unbalanced Moment, M_u=	177.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1098	kips
Estimated Column Size:	**W14x99**	**ΦP_n=1100 k**
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	225.4	kips
P_D=	419.9	kips
P_{Lr}=	8.8	kips
P_U=	951.6	kips
Unbalanced Moment, M_u=	177.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1255	kips
Estimated Column Size:	**W14x120**	**ΦP_n=1340 k**
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	260.8	kips
P_D=	503.9	kips
P_{Lr}=	8.8	kips
P_U=	1109.0	kips
Unbalanced Moment, M_u=	177.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1412	kips
Estimated Column Size:	**W14x132**	**ΦP_n=1480 k**
Tributary area per PH roof=	498	ft^2
Influence area on PH roof=	1913	ft^2
Roof Dead Load= (slab+SDL)	90	psf
Roof Live Load= (green roof area)	30	psf
Snow Load=	15.75	psf
Roof constr Live Load=	20	psf
Load above main roof:	PH roof+	
	0	Floors
P_L=	14.9	kips
P_D=	44.8	kips
P_{Lr}=	10.0	kips
P_U=	82.7	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	18.5	ft
Equivalent Axial Load, P_{eq}=	83	kips
Estimated Column Size:	**W14x43**	**ΦP_n=196 k**

Column 32		
Tributary Area		
per floor or roof=	779	ft^2
Influence Area=	3355	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Mech. Room)	150	psf
Roof Constr. Live Load	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.51	
P_L=	156.5	kips
P_D=	148.0	kips
P_{Lr}=	15.6	kips
P_U=	533.4	kips
Unbalanced Moment, M_u=	136.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	767	kips
Estimated Column Size:	**W14x90**	**ΦP_n=1000 k**
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.40	
P_L=	210.2	kips
P_D=	296.0	kips
P_{Lr}=	15.6	kips
P_U=	797.0	kips
Unbalanced Moment, M_u=	136.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1030	kips
Estimated Column Size:	**W14x99**	**ΦP_n=1100 k**
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.37	
P_L=	259.3	kips
P_D=	444.0	kips
P_{Lr}=	15.6	kips
P_U=	1053.2	kips
Unbalanced Moment, M_u=	136.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1287	kips
Estimated Column Size:	**W14x120**	**ΦP_n=1340 k**
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	335.0	kips
P_D=	592.0	kips
P_{Lr}=	15.6	kips
P_U=	1351.8	kips
Unbalanced Moment, M_u=	136.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1585	kips
Estimated Column Size:	**W14x145**	**ΦP_n=1650 k**
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	397.3	kips
P_D=	740.1	kips
P_{Lr}=	15.6	kips
P_U=	1629.1	kips
Unbalanced Moment, M_u=	136.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1863	kips
Estimated Column Size:	**W14x176**	**ΦP_n=2010 k**
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	459.6	kips
P_D=	888.1	kips
P_{Lr}=	15.6	kips
P_U=	1906.4	kips
Unbalanced Moment, M_u=	136.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	2140	kips
Estimated Column Size:	**W14x193**	**ΦP_n=2210 k**
Tributary area per PH roof=	588	ft^2
Influence area on PH roof=	2594	ft^2
Roof Dead Load= (slab+SDL)	90	psf
Roof Live Load= (green roof area)	30	psf
Snow Load=	15.75	psf
Roof constr Live Load=	20	psf
Load above main roof:	PH roof+	
	0	Floors
P_L=	17.6	kips
P_D=	52.9	kips
P_{Lr}=	11.8	kips
P_U=	97.6	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	18.5	ft
Equivalent Axial Load, P_{eq}=	98	kips
Estimated Column Size:	**W14x43**	**ΦP_n=196 k**

Column 33		
Tributary Area		
per floor or roof=	484	ft^2
Influence Area=	2177	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Mech. Room)	150	psf
Roof Constr. Live Load	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.57	
P_L=	100.3	kips
P_D=	92.0	kips
P_{Lr}=	9.7	kips
P_U=	332.5	kips
Unbalanced Moment, M_u=	77.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	465	kips
Estimated Column Size:	**W14x61**	**ΦP_n=543 k**
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.44	
P_L=	135.9	kips
P_D=	183.9	kips
P_{Lr}=	9.7	kips
P_U=	499.8	kips
Unbalanced Moment, M_u=	77.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	633	kips
Estimated Column Size:	**W14x74**	**ΦP_n=667 k**
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.40	
P_L=	169.4	kips
P_D=	275.9	kips
P_{Lr}=	9.7	kips
P_U=	663.9	kips
Unbalanced Moment, M_u=	77.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	797	kips
Estimated Column Size:	**W14x90**	**ΦP_n=1000 k**
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	208.1	kips
P_D=	367.8	kips
P_{Lr}=	9.7	kips
P_U=	836.2	kips
Unbalanced Moment, M_u=	77.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	969	kips
Estimated Column Size:	**W14x90**	**ΦP_n=1000 k**
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	246.8	kips
P_D=	459.8	kips
P_{Lr}=	9.7	kips
P_U=	1008.5	kips
Unbalanced Moment, M_u=	77.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1141	kips
Estimated Column Size:	**W14x109**	**ΦP_n=1210 k**
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	285.6	kips
P_D=	551.8	kips
P_{Lr}=	9.7	kips
P_U=	1180.8	kips
Unbalanced Moment, M_u=	77.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1314	kips
Estimated Column Size:	**W14x120**	**ΦP_n=1340 k**
Tributary area per PH roof=	343	ft^2
Influence area on PH roof=	1370	ft^2
Roof Dead Load= (slab+SDL)	90	psf
Roof Live Load= (green roof area)	30	psf
Snow Load=	15.75	psf
Roof constr Live Load=	20	psf
Load above main roof:	PH roof+	
	0	Floors
P_L=	10.3	kips
P_D=	30.9	kips
P_{Lr}=	6.9	kips
P_U=	56.9	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	18.5	ft
Equivalent Axial Load, P_{eq}=	57	kips
Estimated Column Size:	**W14x43**	**ΦP_n=196 k**

Column 34		
Tributary Area		
per floor or roof=	359	ft^2
Influence Area=	1435	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Mech. Room)	150	psf
Roof Constr. Live Load	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.65	
P_L=	77.0	kips
P_D=	68.2	kips
P_{Lr}=	7.2	kips
P_U=	242.2	kips
Unbalanced Moment, M_u=	40.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	312	kips
Estimated Column Size:	**W14x48**	**ΦP_n=331 k**
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.48	
P_L=	105.4	kips
P_D=	136.4	kips
P_{Lr}=	7.2	kips
P_U=	369.5	kips
Unbalanced Moment, M_u=	40.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	439	kips
Estimated Column Size:	**W14x61**	**ΦP_n=543 k**
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.43	
P_L=	130.5	kips
P_D=	204.6	kips
P_{Lr}=	7.2	kips
P_U=	491.5	kips
Unbalanced Moment, M_u=	40.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	561	kips
Estimated Column Size:	**W14x68**	**ΦP_n=608 k**
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	154.4	kips
P_D=	272.8	kips
P_{Lr}=	7.2	kips
P_U=	611.5	kips
Unbalanced Moment, M_u=	40.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	681	kips
Estimated Column Size:	**W14x82**	**ΦP_n=735 k**
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	183.1	kips
P_D=	341.1	kips
P_{Lr}=	7.2	kips
P_U=	739.3	kips
Unbalanced Moment, M_u=	40.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	809	kips
Estimated Column Size:	**W14x90**	**ΦP_n=1000 k**
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	211.8	kips
P_D=	409.3	kips
P_{Lr}=	7.2	kips
P_U=	867.1	kips
Unbalanced Moment, M_u=	40.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	937	kips
Estimated Column Size:	**W14x90**	**ΦP_n=1000 k**
Tributary area per PH roof=	202	ft^2
Influence area on PH roof=	805	ft^2
Roof Dead Load= (slab+SDL)	90	psf
Roof Live Load= (green roof area)	30	psf
Snow Load=	15.75	psf
Roof constr Live Load=	20	psf
Load above main roof:	PH roof+	
	0	Floors
P_L=	6.1	kips
P_D=	18.2	kips
P_{Lr}=	4.0	kips
P_U=	33.5	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	18.5	ft
Equivalent Axial Load, P_{eq}=	34	kips
Estimated Column Size:	**W14x43**	**ΦP_n=196 k**



Column 35		
Tributary Area		
per floor or roof=	698	ft^2
Influence Area=	2790	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Mech. Room)	150	psf
Roof Live Load= (Mech. Room)	150	psf
Roof Constr. Live Load	20	psf

Load above level 12:	Roof +	
	1	Floor
LL_{red}	1.00	
P_L=	209.4	kips
P_D=	132.6	kips
P_{Lr}=	14.0	kips
P_U=	504.5	kips
Unbalanced Moment, M_u=	6.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	515	kips
Estimated Column Size:	W14x61	ΦP_n=543 k

Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.80	
P_L=	356.0	kips
P_D=	265.2	kips
P_{Lr}=	14.0	kips
P_U=	898.2	kips
Unbalanced Moment, M_u=	6.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	909	kips
Estimated Column Size:	W14x90	ΦP_n=1000 k

Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.80	
P_L=	523.5	kips
P_D=	397.9	kips
P_{Lr}=	14.0	kips
P_U=	1325.3	kips
Unbalanced Moment, M_u=	6.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1336	kips
Estimated Column Size:	W14x120	ΦP_n=1340 k

Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.80	
P_L=	691.0	kips
P_D=	530.5	kips
P_{Lr}=	14.0	kips
P_U=	1752.5	kips
Unbalanced Moment, M_u=	6.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1763	kips
Estimated Column Size:	W14x159	ΦP_n=1810 k

Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.80	
P_L=	858.5	kips
P_D=	663.1	kips
P_{Lr}=	14.0	kips
P_U=	2179.7	kips
Unbalanced Moment, M_u=	6.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	2190	kips
Estimated Column Size:	W14x193	ΦP_n=2210 k

Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.80	
P_L=	1026.1	kips
P_D=	795.7	kips
P_{Lr}=	14.0	kips
P_U=	2606.9	kips
Unbalanced Moment, M_u=	6.2	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	2617	kips
Estimated Column Size:	W14x233	ΦP_n=2680 k

Tributary area per PH roof=	20	ft^2
Influence area on PH roof=	80	ft^2
Roof Dead Load= (slab+SDL)	90	psf
Roof Live Load= (green roof area)	30	psf
Snow Load=	15.75	psf
Roof constr Live Load=	20	psf

Load above main roof:	PH roof+	
	0	Floors
P_L=	0.6	kips
P_D=	1.8	kips
P_{Lr}=	0.4	kips
P_U=	3.3	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	18.5	ft
Equivalent Axial Load, P_{eq}=	3	kips
Estimated Column Size:	W14x43	ΦP_n=196 k

Note: Refer to Moment Frame C elevation in final report for column sizes that support combined gravity and lateral loads

Moment Frame E: Estimated column sizes based on gravity load analysis only

Column 49		
Tributary Area		
per floor or roof=	339	ft^2
Influence Area=	1245	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Green roof area)	30	psf
Roof Constr. Live Load	20	psf

Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.68	
P_L=	33.1	kips
P_D=	64.4	kips
P_{Lr}=	6.8	kips
P_U=	133.6	kips
Unbalanced Moment, M_u=	234.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	535	
Estimated Column Size:	**W14x61**	**ΦP_n=543 k**

Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.50	
P_L=	60.6	kips
P_D=	128.8	kips
P_{Lr}=	6.8	kips
P_U=	254.9	kips
Unbalanced Moment, M_u=	234.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	656	
Estimated Column Size:	**W14x74**	**ΦP_n=667 k**

Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.44	
P_L=	84.8	kips
P_D=	193.2	kips
P_{Lr}=	6.8	kips
P_U=	370.9	kips
Unbalanced Moment, M_u=	234.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	772	
Estimated Column Size:	**W14x90**	**ΦP_n=1000 k**

Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.41	
P_L=	107.6	kips
P_D=	257.6	kips
P_{Lr}=	6.8	kips
P_U=	484.8	kips
Unbalanced Moment, M_u=	234.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	886	
Estimated Column Size:	**W14x90**	**ΦP_n=1000 k**

Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	132.2	kips
P_D=	322.1	kips
P_{Lr}=	6.8	kips
P_U=	601.4	kips
Unbalanced Moment, M_u=	234.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1003	
Estimated Column Size:	**W14x99**	**ΦP_n=1100 k**

Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	159.3	kips
P_D=	386.5	kips
P_{Lr}=	6.8	kips
P_U=	722.1	kips
Unbalanced Moment, M_u=	234.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1123	
Estimated Column Size:	**W14x109**	**ΦP_n=1210 k**

Column 50		
Tributary Area		
per floor or roof=	613	ft^2
Influence Area=	2366	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Green roof area)	30	psf
Roof Constr. Live Load	20	psf

Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.56	
P_L=	52.6	kips
P_D=	116.5	kips
P_{Lr}=	12.3	kips
P_U=	230.1	kips
Unbalanced Moment, M_u=	35.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	291	kips
Estimated Column Size:	**W14x43**	**ΦP_n=292 k**

Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.43	
P_L=	97.1	kips
P_D=	232.9	kips
P_{Lr}=	12.3	kips
P_U=	441.0	kips
Unbalanced Moment, M_u=	35.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	502	kips
Estimated Column Size:	**W14x61**	**ΦP_n=543 k**

Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.40	
P_L=	141.0	kips
P_D=	349.4	kips
P_{Lr}=	12.3	kips
P_U=	651.0	kips
Unbalanced Moment, M_u=	35.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	712	kips
Estimated Column Size:	**W14x74**	**ΦP_n=667 k**

Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	190.0	kips
P_D=	465.9	kips
P_{Lr}=	12.3	kips
P_U=	869.2	kips
Unbalanced Moment, M_u=	35.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	930	kips
Estimated Column Size:	**W14x90**	**ΦP_n=1000 k**

Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	239.1	kips
P_D=	582.4	kips
P_{Lr}=	12.3	kips
P_U=	1087.5	kips
Unbalanced Moment, M_u=	35.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1148	kips
Estimated Column Size:	**W14x109**	**ΦP_n=1210 k**

Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	288.1	kips
P_D=	698.8	kips
P_{Lr}=	12.3	kips
P_U=	1305.7	kips
Unbalanced Moment, M_u=	35.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1367	kips
Estimated Column Size:	**W14x132**	**ΦP_n=1480 k**

Column 51		
Tributary Area		
per floor or roof=	339	ft^2
Influence Area=	1245	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Green roof area)	30	psf
Roof Constr. Live Load	20	psf

Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.68	
P_L=	33.1	kips
P_D=	64.4	kips
P_{Lr}=	6.8	kips
P_U=	133.6	kips
Unbalanced Moment, M_u=	60.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	237	kips
Estimated Column Size:	**W14x43**	**ΦP_n=292 k**

Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.50	
P_L=	60.6	kips
P_D=	128.8	kips
P_{Lr}=	6.8	kips
P_U=	254.9	kips
Unbalanced Moment, M_u=	60.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	359	kips
Estimated Column Size:	**W14x61**	**ΦP_n=543 k**

Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.44	
P_L=	84.8	kips
P_D=	193.2	kips
P_{Lr}=	6.8	kips
P_U=	370.9	kips
Unbalanced Moment, M_u=	60.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	475	kips
Estimated Column Size:	**W14x61**	**ΦP_n=543 k**

Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.41	
P_L=	107.6	kips
P_D=	257.6	kips
P_{Lr}=	6.8	kips
P_U=	484.8	kips
Unbalanced Moment, M_u=	60.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	588	kips
Estimated Column Size:	**W14x68**	**ΦP_n=608 k**

Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	132.2	kips
P_D=	322.1	kips
P_{Lr}=	6.8	kips
P_U=	601.4	kips
Unbalanced Moment, M_u=	60.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	705	kips
Estimated Column Size:	**W14x82**	**ΦP_n=735 k**

Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	159.3	kips
P_D=	386.5	kips
P_{Lr}=	6.8	kips
P_U=	722.1	kips
Unbalanced Moment, M_u=	60.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	826	kips
Estimated Column Size:	**W14x90**	**ΦP_n=1000 k**

Note: Refer to Moment Frame E elevation in final report for column sizes that support combined gravity and lateral loads

Moment Frame 1: Estimated column sizes based on gravity load analysis only

Column 38		
Tributary Area		
per floor or roof=	540	ft^2
Influence Area=	2055	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Green roof area)	30	psf
Roof Constr. Live Load	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.58	
P_L=	47.6	kips
P_D=	102.6	kips
P_{Lr}=	10.8	kips
P_U=	204.6	kips
Unbalanced Moment, M_u=	173.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	502	
Estimated Column Size:	**W14x61**	**ΦP_n=543 k**
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.44	
P_L=	87.6	kips
P_D=	205.2	kips
P_{Lr}=	10.8	kips
P_U=	391.9	kips
Unbalanced Moment, M_u=	173.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	689	
Estimated Column Size:	**W14x82**	**ΦP_n=735 k**
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.40	
P_L=	124.2	kips
P_D=	307.8	kips
P_{Lr}=	10.8	kips
P_U=	573.5	kips
Unbalanced Moment, M_u=	173.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	871	
Estimated Column Size:	**W14x90**	**ΦP_n=1000 k**
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	167.4	kips
P_D=	410.4	kips
P_{Lr}=	10.8	kips
P_U=	765.7	kips
Unbalanced Moment, M_u=	173.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1063	
Estimated Column Size:	**W14x99**	**ΦP_n=1100 k**
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	210.6	kips
P_D=	513.0	kips
P_{Lr}=	10.8	kips
P_U=	958.0	kips
Unbalanced Moment, M_u=	173.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1255	
Estimated Column Size:	**W14x120**	**ΦP_n=1340 k**
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	253.8	kips
P_D=	615.6	kips
P_{Lr}=	10.8	kips
P_U=	1150.2	kips
Unbalanced Moment, M_u=	173.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1448	
Estimated Column Size:	**W14x132**	**ΦP_n=1480 k**

Column 1
Refer to Moment Frame 1
Column 15
Refer to Moment Frame 2
Column 27
Refer to Moment Frame 3
Column 49
Refer to Moment Frame 4

Note: Refer to Moment Frame 1 elevation in final report for column sizes that support combined gravity and lateral loads

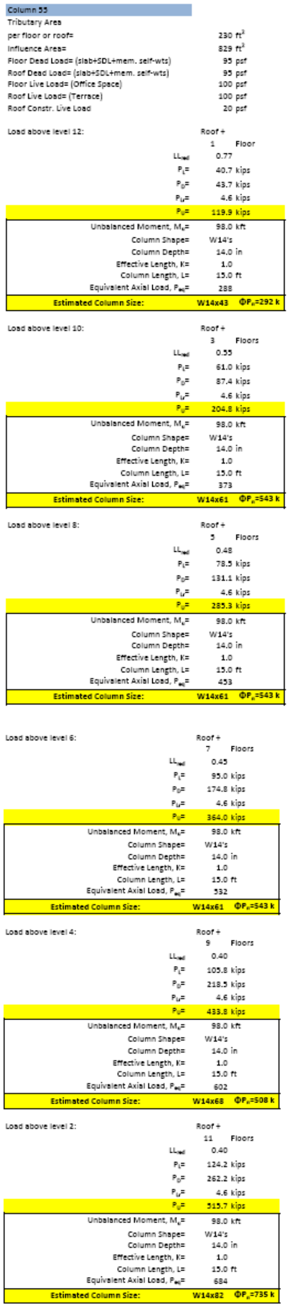
Moment Frame 1’: Estimated column sizes based on gravity load analysis only

Column 14		
Tributary Area		
per floor or roof=	323	ft^2
Influence Area=	1186	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Terrace)	100	psf
Roof Constr. Live Load	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.69	
P_L=	54.4	kips
P_D=	61.4	kips
P_{Lr}=	6.5	kips
P_U=	164.0	kips
Unbalanced Moment, M_u=	194.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	497	
Estimated Column Size:	**W14x61**	**ΦP_n=543 k**
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.50	
P_L=	80.9	kips
P_D=	122.7	kips
P_{Lr}=	6.5	kips
P_U=	279.9	kips
Unbalanced Moment, M_u=	194.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	613	
Estimated Column Size:	**W14x74**	**ΦP_n=667 k**
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.44	
P_L=	104.1	kips
P_D=	184.1	kips
P_{Lr}=	6.5	kips
P_U=	390.8	kips
Unbalanced Moment, M_u=	194.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	724	
Estimated Column Size:	**W14x82**	**ΦP_n=735 k**
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.41	
P_L=	126.0	kips
P_D=	245.5	kips
P_{Lr}=	6.5	kips
P_U=	499.5	kips
Unbalanced Moment, M_u=	194.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	833	
Estimated Column Size:	**W14x90**	**ΦP_n=1000 k**
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	148.6	kips
P_D=	306.9	kips
P_{Lr}=	6.5	kips
P_U=	609.2	kips
Unbalanced Moment, M_u=	194.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	943	
Estimated Column Size:	**W14x90**	**ΦP_n=1000 k**
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	174.4	kips
P_D=	368.2	kips
P_{Lr}=	6.5	kips
P_U=	724.2	kips
Unbalanced Moment, M_u=	194.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1058	
Estimated Column Size:	**W14x99**	**ΦP_n=1100 k**

Column 26		
Tributary Area		
per floor or roof=	553	ft^2
Influence Area=	2113	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Terrace)	100	psf
Roof Constr. Live Load	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.58	
P_L=	87.2	kips
P_D=	105.1	kips
P_{Lr}=	11.1	kips
P_U=	271.1	kips
Unbalanced Moment, M_u=	122.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	480	
Estimated Column Size:	**W14x61**	**ΦP_n=543 k**
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.44	
P_L=	128.0	kips
P_D=	210.1	kips
P_{Lr}=	11.1	kips
P_U=	462.5	kips
Unbalanced Moment, M_u=	122.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	672	
Estimated Column Size:	**W14x82**	**ΦP_n=735 k**
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.40	
P_L=	165.9	kips
P_D=	315.2	kips
P_{Lr}=	11.1	kips
P_U=	649.2	kips
Unbalanced Moment, M_u=	122.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	858	
Estimated Column Size:	**W14x90**	**ΦP_n=1000 k**
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	210.1	kips
P_D=	420.3	kips
P_{Lr}=	11.1	kips
P_U=	846.1	kips
Unbalanced Moment, M_u=	122.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1055	
Estimated Column Size:	**W14x99**	**ΦP_n=1100 k**
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	254.4	kips
P_D=	525.4	kips
P_{Lr}=	11.1	kips
P_U=	1043.0	kips
Unbalanced Moment, M_u=	122.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1252	
Estimated Column Size:	**W14x120**	**ΦP_n=1340 k**
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	298.6	kips
P_D=	630.4	kips
P_{Lr}=	11.1	kips
P_U=	1239.8	kips
Unbalanced Moment, M_u=	122.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1449	
Estimated Column Size:	**W14x132**	**ΦP_n=1480 k**

Column 37		
Tributary Area		
per floor or roof=	510	ft^2
Influence Area=	1950	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Terrace/Balcony)	100	psf
Roof Constr. Live Load	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.59	
P_L=	81.1	kips
P_D=	96.9	kips
P_{Lr}=	10.2	kips
P_U=	251.1	kips
Unbalanced Moment, M_u=	74.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	379	
Estimated Column Size:	**W14x61**	**ΦP_n=543 k**
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.45	
P_L=	119.3	kips
P_D=	193.8	kips
P_{Lr}=	10.2	kips
P_U=	428.5	kips
Unbalanced Moment, M_u=	74.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	556	
Estimated Column Size:	**W14x68**	**ΦP_n=608 k**
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.40	
P_L=	153.5	kips
P_D=	290.7	kips
P_{Lr}=	10.2	kips
P_U=	599.5	kips
Unbalanced Moment, M_u=	74.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	727	
Estimated Column Size:	**W14x82**	**ΦP_n=735 k**
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	193.8	kips
P_D=	387.6	kips
P_{Lr}=	10.2	kips
P_U=	780.3	kips
Unbalanced Moment, M_u=	74.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	908	
Estimated Column Size:	**W14x90**	**ΦP_n=1000 k**
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	234.6	kips
P_D=	484.5	kips
P_{Lr}=	10.2	kips
P_U=	961.9	kips
Unbalanced Moment, M_u=	74.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1090	
Estimated Column Size:	**W14x99**	**ΦP_n=1100 k**
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	275.4	kips
P_D=	581.4	kips
P_{Lr}=	10.2	kips
P_U=	1143.4	kips
Unbalanced Moment, M_u=	74.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1271	
Estimated Column Size:	**W14x120**	**ΦP_n=1340 k**

Column 48		
Tributary Area		
per floor or roof=	459	ft^2
Influence Area=	1755	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Terrace/Balcony)	100	psf
Roof Constr. Live Load	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.61	
P_L=	73.8	kips
P_D=	87.2	kips
P_{Lr}=	9.2	kips
P_U=	227.3	kips
Unbalanced Moment, M_u=	100.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	400	
Estimated Column Size:	**W14x61**	**ΦP_n=543 k**
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.46	
P_L=	108.8	kips
P_D=	174.4	kips
P_{Lr}=	9.2	kips
P_U=	388.0	kips
Unbalanced Moment, M_u=	100.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	560	
Estimated Column Size:	**W14x68**	**ΦP_n=608 k**
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.41	
P_L=	140.0	kips
P_D=	261.6	kips
P_{Lr}=	9.2	kips
P_U=	542.6	kips
Unbalanced Moment, M_u=	100.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	715	
Estimated Column Size:	**W14x90**	**ΦP_n=1000 k**
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	174.4	kips
P_D=	348.8	kips
P_{Lr}=	9.2	kips
P_U=	702.3	kips
Unbalanced Moment, M_u=	100.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	875	
Estimated Column Size:	**W14x90**	**ΦP_n=1000 k**
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	211.1	kips
P_D=	436.1	kips
P_{Lr}=	9.2	kips
P_U=	865.7	kips
Unbalanced Moment, M_u=	100.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1038	
Estimated Column Size:	**W14x99**	**ΦP_n=1100 k**
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	247.9	kips
P_D=	523.3	kips
P_{Lr}=	9.2	kips
P_U=	1029.1	kips
Unbalanced Moment, M_u=	100.5	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1201	
Estimated Column Size:	**W14x109**	**ΦP_n=1210 k**



Column 55		
Tributary Area		
per floor or roof=	230	ft^2
Influence Area=	829	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Terrace)	100	psf
Roof Constr. Live Load	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.77	
P_L=	40.7	kips
P_D=	43.7	kips
P_{Lr}=	4.6	kips
P_U=	119.9	kips
Unbalanced Moment, M_u=	98.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	288	
Estimated Column Size:	**W14x43**	**ΦP_n=292 k**
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.55	
P_L=	61.0	kips
P_D=	87.4	kips
P_{Lr}=	4.6	kips
P_U=	204.8	kips
Unbalanced Moment, M_u=	98.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	373	
Estimated Column Size:	**W14x61**	**ΦP_n=543 k**
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.48	
P_L=	78.5	kips
P_D=	131.1	kips
P_{Lr}=	4.6	kips
P_U=	285.3	kips
Unbalanced Moment, M_u=	98.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	453	
Estimated Column Size:	**W14x61**	**ΦP_n=543 k**
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.45	
P_L=	95.0	kips
P_D=	174.8	kips
P_{Lr}=	4.6	kips
P_U=	364.0	kips
Unbalanced Moment, M_u=	98.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	532	
Estimated Column Size:	**W14x61**	**ΦP_n=543 k**
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	105.8	kips
P_D=	218.5	kips
P_{Lr}=	4.6	kips
P_U=	433.8	kips
Unbalanced Moment, M_u=	98.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	602	
Estimated Column Size:	**W14x68**	**ΦP_n=508 k**
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	124.2	kips
P_D=	262.2	kips
P_{Lr}=	4.6	kips
P_U=	515.7	kips
Unbalanced Moment, M_u=	98.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	684	
Estimated Column Size:	**W14x82**	**ΦP_n=735 k**

Note: Refer to Moment Frame 1’ elevation in final report for column sizes that support combined gravity and lateral loads

Gravity Column Design

Column 13 - Gravity Column		
Tributary Area		
per floor or roof=	456	ft^2
Influence Area=	1608	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Terrace)	100	psf
Construct. Roof LL=	20	psf

Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.62	
P_L=	74.1	kips
P_D=	86.6	kips
P_{Lr}=	9.1	kips
P_U=	227.0	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	227	
Column Size:	**W14x43**	**ΦP_n=292 k**

Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.47	
P_L=	109.3	kips
P_D=	173.3	kips
P_{Lr}=	9.1	kips
P_U=	387.4	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	387	
Column Size:	**W14x61**	**ΦP_n=542 k**

Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.42	
P_L=	140.7	kips
P_D=	259.9	kips
P_{Lr}=	9.1	kips
P_U=	541.7	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	542	
Column Size:	**W14x61**	**ΦP_n=543 k**

Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	173.3	kips
P_D=	346.6	kips
P_{Lr}=	9.1	kips
P_U=	697.7	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	698	
Column Size:	**W14x82**	**ΦP_n=735 k**

Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	209.8	kips
P_D=	433.2	kips
P_{Lr}=	9.1	kips
P_U=	860.0	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	860	
Column Size:	**W14x90**	**ΦP_n=1000 k**

Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	246.2	kips
P_D=	519.8	kips
P_{Lr}=	9.1	kips
P_U=	1022.4	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1022	
Column Size:	**W14x99**	**ΦP_n=1100 k**

Column 25 - Gravity Column		
Tributary Area		
per floor or roof=	772	ft^2
Influence Area=	3482	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Terrace)	100	psf
Construct. Roof LL=	20	psf

Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.50	
P_L=	116.1	kips
P_D=	146.7	kips
P_{Lr}=	15.4	kips
P_U=	369.5	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	370	
Column Size:	**W14x61**	**ΦP_n=543 k**

Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.40	
P_L=	169.8	kips
P_D=	293.4	kips
P_{Lr}=	15.4	kips
P_U=	631.5	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	631	
Column Size:	**W14x74**	**ΦP_n=667 k**

Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.40	
P_L=	231.6	kips
P_D=	440.0	kips
P_{Lr}=	15.4	kips
P_U=	906.3	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	906	
Column Size:	**W14x90**	**ΦP_n=1000 k**

Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	293.4	kips
P_D=	586.7	kips
P_{Lr}=	15.4	kips
P_U=	1181.2	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1181	
Column Size:	**W14x109**	**ΦP_n=1210 k**

Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	355.1	kips
P_D=	733.4	kips
P_{Lr}=	15.4	kips
P_U=	1456.0	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1456	
Column Size:	**W14x132**	**ΦP_n=1480 k**

Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	416.9	kips
P_D=	880.1	kips
P_{Lr}=	15.4	kips
P_U=	1730.8	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1731	
Column Size:	**W14x159**	**ΦP_n=1810 k**

Column 36 - Gravity Column		
Tributary Area		
per floor or roof=	712.5	ft^2
Influence Area=	2850	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Terrace)	100	psf
Roof Live Load= (Green Roof)	30	psf
Construct. Roof LL=	20	psf

Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.53	
P_L=	84.1	kips
P_D=	135.4	kips
P_{Lr}=	14.3	kips
P_U=	304.2	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	304	
Column Size:	**W14x48**	**ΦP_n=331 k**

Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.41	
P_L=	134.4	kips
P_D=	270.8	kips
P_{Lr}=	14.3	kips
P_U=	547.1	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	547	
Column Size:	**W14x68**	**ΦP_n=608 k**

Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.40	
P_L=	188.8	kips
P_D=	406.1	kips
P_{Lr}=	14.3	kips
P_U=	796.6	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	797	
Column Size:	**W14x90**	**ΦP_n=1000 k**

Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	245.8	kips
P_D=	541.5	kips
P_{Lr}=	14.3	kips
P_U=	1050.2	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1050	
Column Size:	**W14x99**	**ΦP_n=1100 k**

Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	302.8	kips
P_D=	676.9	kips
P_{Lr}=	14.3	kips
P_U=	1303.9	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1304	
Column Size:	**W14x120**	**ΦP_n=1340 k**

Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	359.8	kips
P_D=	812.3	kips
P_{Lr}=	14.3	kips
P_U=	1557.5	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1558	
Column Size:	**W14x145**	**ΦP_n=1650 k**

Column 39 - Gravity Column		
Tributary Area		
per floor or roof=	976	ft^2
Influence Area=	3904.5	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Green Roof)	30	psf
Construct. Roof LL=	20	psf

Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.50	
P_L=	78.1	kips
P_D=	185.4	kips
P_{Lr}=	19.5	kips
P_U=	357.2	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	357	
Column Size:	**W14x53**	**ΦP_n=369 k**

Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.40	
P_L=	146.4	kips
P_D=	370.9	kips
P_{Lr}=	19.5	kips
P_U=	689.1	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	689	
Column Size:	**W14x82**	**ΦP_n=735 k**

Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.40	
P_L=	224.5	kips
P_D=	556.3	kips
P_{Lr}=	19.5	kips
P_U=	1036.5	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1037	
Column Size:	**W14x99**	**ΦP_n=1100 k**

Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	302.6	kips
P_D=	741.8	kips
P_{Lr}=	19.5	kips
P_U=	1384.0	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1384	
Column Size:	**W14x132**	**ΦP_n=1480 k**

Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	380.6	kips
P_D=	927.2	kips
P_{Lr}=	19.5	kips
P_U=	1731.4	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1731	
Column Size:	**W14x159**	**ΦP_n=1810 k**

Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	458.7	kips
P_D=	1112.6	kips
P_{Lr}=	19.5	kips
P_U=	2078.9	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	2079	
Column Size:	**W14x193**	**ΦP_n=2210 k**

Column 40 - Gravity Column		
Tributary Area		
per floor or roof=	745	ft^2
Influence Area=	2808	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Green Roof)	30	psf
Construct. Roof LL=	20	psf

Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.53	
P_L=	62.1	kips
P_D=	141.6	kips
P_{Lr}=	14.9	kips
P_U=	276.6	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	277	
Column Size:	**W14x43**	**ΦP_n=292 k**

Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.41	
P_L=	114.8	kips
P_D=	283.1	kips
P_{Lr}=	14.9	kips
P_U=	530.8	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	531	
Column Size:	**W14x61**	**ΦP_n=543 k**

Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.40	
P_L=	171.4	kips
P_D=	424.7	kips
P_{Lr}=	14.9	kips
P_U=	791.2	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	791	
Column Size:	**W14x90**	**ΦP_n=1000 k**

Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	231.0	kips
P_D=	566.2	kips
P_{Lr}=	14.9	kips
P_U=	1056.4	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1056	
Column Size:	**W14x99**	**ΦP_n=1100 k**

Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	290.6	kips
P_D=	707.8	kips
P_{Lr}=	14.9	kips
P_U=	1321.6	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1322	
Column Size:	**W14x120**	**ΦP_n=1340 k**

Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	350.2	kips
P_D=	849.3	kips
P_{Lr}=	14.9	kips
P_U=	1586.9	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1587	
Column Size:	**W14x145**	**ΦP_n=1650 k**

Column 41 - Gravity Column		
Tributary Area		
per floor or roof=	200	ft^2
Influence Area=	1010	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Green Roof)	30	psf
Construct. Roof LL=	20	psf

Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.72	
P_L=	20.4	kips
P_D=	38.0	kips
P_{Lr}=	4.0	kips
P_U=	80.3	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	80	
Column Size:	**W14x43**	**ΦP_n=292 k**

Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.52	
P_L=	37.4	kips
P_D=	76.0	kips
P_{Lr}=	4.0	kips
P_U=	153.0	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	153	
Column Size:	**W14x43**	**ΦP_n=292 k**

Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.46	
P_L=	52.1	kips
P_D=	114.0	kips
P_{Lr}=	4.0	kips
P_U=	222.2	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	222	
Estimated Column Size:	**W14x43**	**ΦP_n=292 k**

Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.43	
P_L=	66.0	kips
P_D=	152.0	kips
P_{Lr}=	4.0	kips
P_U=	290.0	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	290	
Column Size:	**W14x43**	**ΦP_n=292 k**

Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.41	
P_L=	79.3	kips
P_D=	190.0	kips
P_{Lr}=	4.0	kips
P_U=	356.9	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	357	
Column Size:	**W14x53**	**ΦP_n=369 k**

Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	94.0	kips
P_D=	228.0	kips
P_{Lr}=	4.0	kips
P_U=	426.0	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	426	
Column Size:	**W14x61**	**ΦP_n=543 k**

Column 42 - Gravity Column		
Tributary Area		
per floor or roof=	203	ft^2
Influence Area=	1417	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Green Roof)	30	psf
Construct. Roof LL=	20	psf

Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.65	
P_L=	19.3	kips
P_D=	38.6	kips
P_{Lr}=	4.1	kips
P_U=	79.1	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	79	
Column Size:	**W14x43**	**ΦP_n=292 k**

Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.48	
P_L=	35.3	kips
P_D=	77.1	kips
P_{Lr}=	4.1	kips
P_U=	151.1	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	151	
Column Size:	**W14x43**	**ΦP_n=292 k**

Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.43	
P_L=	49.6	kips
P_D=	115.7	kips
P_{Lr}=	4.1	kips
P_U=	220.2	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	220	
Estimated Column Size:	**W14x43**	**ΦP_n=292 k**

Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	63.0	kips
P_D=	154.3	kips
P_{Lr}=	4.1	kips
P_U=	288.0	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	288	
Column Size:	**W14x43**	**ΦP_n=292 k**

Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	79.2	kips
P_D=	192.9	kips
P_{Lr}=	4.1	kips
P_U=	360.1	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	360	
Column Size:	**W14x53**	**ΦP_n=369 k**

Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	95.4	kips
P_D=	231.4	kips
P_{Lr}=	4.1	kips
P_U=	432.4	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	432	
Column Size:	**W14x61**	**ΦP_n=543 k**

Column 43 - Gravity Column		
Tributary Area		
per floor or roof=	256	ft^2
Influence Area=	1025	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Green Roof)	30	psf
Construct. Roof LL=	20	psf

Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.72	
P_L=	26.1	kips
P_D=	48.6	kips
P_{Lr}=	5.1	kips
P_U=	102.6	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	103	
Column Size:	W14x43	ΦP_n=292 k

Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.52	
P_L=	47.7	kips
P_D=	97.3	kips
P_{Lr}=	5.1	kips
P_U=	195.5	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	196	
Column Size:	W14x43	ΦP_n=292 k

Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.46	
P_L=	66.5	kips
P_D=	145.9	kips
P_{Lr}=	5.1	kips
P_U=	284.1	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	284	
Estimated Column Size:	W14x43	ΦP_n=292 k

Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.43	
P_L=	84.2	kips
P_D=	194.6	kips
P_{Lr}=	5.1	kips
P_U=	370.8	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	371	
Column Size:	W14x61	ΦP_n=543 k

Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.41	
P_L=	101.3	kips
P_D=	243.2	kips
P_{Lr}=	5.1	kips
P_U=	456.4	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	456	
Column Size:	W14x61	ΦP_n=543 k

Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	120.3	kips
P_D=	291.8	kips
P_{Lr}=	5.1	kips
P_U=	545.3	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	545	
Column Size:	W14x68	ΦP_n=608 k

Column 44 - Gravity Column		
Tributary Area		
per floor or roof=	772	ft^2
Influence Area=	1082	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Green Roof)	30	psf
Construct. Roof LL=	20	psf

Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.71	
P_L=	77.7	kips
P_D=	146.7	kips
P_{Lr}=	15.4	kips
P_U=	308.0	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	308	
Column Size:	W14x48	ΦP_n=331 k

Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.51	
P_L=	142.0	kips
P_D=	293.4	kips
P_{Lr}=	15.4	kips
P_U=	587.0	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	587	
Column Size:	W14x68	ΦP_n=608 k

Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.45	
P_L=	198.4	kips
P_D=	440.0	kips
P_{Lr}=	15.4	kips
P_U=	853.2	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	853	
Estimated Column Size:	W14x90	ΦP_n=1000 k

Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.42	
P_L=	251.4	kips
P_D=	586.7	kips
P_{Lr}=	15.4	kips
P_U=	1114.0	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1114	
Column Size:	W14x109	ΦP_n=1210 k

Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	302.5	kips
P_D=	733.4	kips
P_{Lr}=	15.4	kips
P_U=	1371.8	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1372	
Column Size:	W14x132	ΦP_n=1480 k

Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	362.8	kips
P_D=	880.1	kips
P_{Lr}=	15.4	kips
P_U=	1644.4	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1644	
Column Size:	W14x145	ΦP_n=1650 k

Column 45 - Gravity Column		
Tributary Area		
per floor or roof=	290	ft^2
Influence Area=	1130	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Green Roof)	30	psf
Construct. Roof LL=	20	psf

Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.70	
P_L=	28.9	kips
P_D=	55.1	kips
P_{Lr}=	5.8	kips
P_U=	115.2	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	115	
Column Size:	W14x43	ΦP_n=292 k

Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.51	
P_L=	52.9	kips
P_D=	110.2	kips
P_{Lr}=	5.8	kips
P_U=	219.7	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	220	
Column Size:	W14x43	ΦP_n=292 k

Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.45	
P_L=	73.9	kips
P_D=	165.3	kips
P_{Lr}=	5.8	kips
P_U=	319.5	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	319	
Estimated Column Size:	W14x48	ΦP_n=331 k

Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.42	
P_L=	93.7	kips
P_D=	220.4	kips
P_{Lr}=	5.8	kips
P_U=	417.3	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	417	
Column Size:	W14x61	ΦP_n=543 k

Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	113.1	kips
P_D=	275.5	kips
P_{Lr}=	5.8	kips
P_U=	514.5	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	514	
Column Size:	W14x61	ΦP_n=543 k

Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	136.3	kips
P_D=	330.6	kips
P_{Lr}=	5.8	kips
P_U=	617.7	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	618	
Column Size:	W14x74	ΦP_n=667 k

Column 46 - Gravity Column		
Tributary Area		
per floor or roof=	551	ft^2
Influence Area=	2944	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Green Roof)	30	psf
Construct. Roof LL=	20	psf

Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.53	
P_L=	45.5	kips
P_D=	104.7	kips
P_{Lr}=	11.0	kips
P_U=	204.0	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	204	
Column Size:	W14x43	ΦP_n=292 k

Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.41	
P_L=	84.2	kips
P_D=	209.4	kips
P_{Lr}=	11.0	kips
P_U=	391.5	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	392	
Column Size:	W14x61	ΦP_n=543 k

Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.40	
P_L=	126.7	kips
P_D=	314.1	kips
P_{Lr}=	11.0	kips
P_U=	585.2	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	585	
Estimated Column Size:	W14x68	ΦP_n=608 k

Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	170.8	kips
P_D=	418.8	kips
P_{Lr}=	11.0	kips
P_U=	781.3	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	781	
Column Size:	W14x90	ΦP_n=1000 k

Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	214.9	kips
P_D=	523.5	kips
P_{Lr}=	11.0	kips
P_U=	977.5	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	977	
Column Size:	W14x90	ΦP_n=1000 k

Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	259.0	kips
P_D=	628.1	kips
P_{Lr}=	11.0	kips
P_U=	1173.6	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1174	
Column Size:	W14x109	ΦP_n=1210 k

Column 47 - Gravity Column

Tributary Area		
per floor or roof=	641.3	ft^2
Influence Area=	2883	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Green Roof)	30	psf
Construct. Roof LL=	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.53	
P_L=	53.2	kips
P_D=	121.8	kips
P_{Lr}=	12.8	kips
P_U=	237.7	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	238	
Column Size:	**W14x43**	**ΦP_n=292 k**
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.41	
P_L=	98.4	kips
P_D=	243.7	kips
P_{Lr}=	12.8	kips
P_U=	456.2	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	456	
Column Size:	**W14x61**	**ΦP_n=543 k**
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.40	
P_L=	147.5	kips
P_D=	365.5	kips
P_{Lr}=	12.8	kips
P_U=	681.1	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	681	
Estimated Column Size:	**W14x82**	**ΦP_n=735 k**
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.40	
P_L=	198.8	kips
P_D=	487.4	kips
P_{Lr}=	12.8	kips
P_U=	909.4	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	909	
Column Size:	**W14x90**	**ΦP_n=1000 k**
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	250.1	kips
P_D=	609.2	kips
P_{Lr}=	12.8	kips
P_U=	1137.7	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1138	
Column Size:	**W14x109**	**ΦP_n=1210 k**
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	301.4	kips
P_D=	731.1	kips
P_{Lr}=	12.8	kips
P_U=	1366.0	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	1366	
Column Size:	**W14x132**	**ΦP_n=1480 k**

Column 52 - Gravity Column

Tributary Area		
per floor or roof=	324	ft^2
Influence Area=	1123	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Green Roof)	30	psf
Construct. Roof LL=	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.70	
P_L=	32.3	kips
P_D=	61.6	kips
P_{Lr}=	6.5	kips
P_U=	128.8	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	129	
Column Size:	**W14x43**	**ΦP_n=292 k**
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.51	
P_L=	59.1	kips
P_D=	123.1	kips
P_{Lr}=	6.5	kips
P_U=	245.6	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	246	
Column Size:	**W14x43**	**ΦP_n=292 k**
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.45	
P_L=	82.6	kips
P_D=	184.7	kips
P_{Lr}=	6.5	kips
P_U=	357.1	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	357	
Column Size:	**W14x53**	**ΦP_n=369 k**
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.42	
P_L=	104.8	kips
P_D=	246.2	kips
P_{Lr}=	6.5	kips
P_U=	466.4	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	466	
Column Size:	**W14x61**	**ΦP_n=543 k**
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	126.4	kips
P_D=	307.8	kips
P_{Lr}=	6.5	kips
P_U=	574.8	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	575	
Column Size:	**W14x68**	**ΦP_n=608 k**
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	152.3	kips
P_D=	369.4	kips
P_{Lr}=	6.5	kips
P_U=	690.1	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	690	
Column Size:	**W14x82**	**ΦP_n=735 k**

Column 53 - Gravity Column

Tributary Area		
per floor or roof=	275.6	ft^2
Influence Area=	1041.2	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Green Roof)	30	psf
Construct. Roof LL=	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.71	
P_L=	28.0	kips
P_D=	52.4	kips
P_{Lr}=	5.5	kips
P_U=	110.3	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	110	
Column Size:	**W14x43**	**ΦP_n=292 k**
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.52	
P_L=	51.1	kips
P_D=	104.7	kips
P_{Lr}=	5.5	kips
P_U=	210.2	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	210	
Column Size:	**W14x43**	**ΦP_n=292 k**
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.46	
P_L=	71.4	kips
P_D=	157.1	kips
P_{Lr}=	5.5	kips
P_U=	305.5	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	305	
Column Size:	**W14x48**	**ΦP_n=331 k**
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.43	
P_L=	90.4	kips
P_D=	209.5	kips
P_{Lr}=	5.5	kips
P_U=	398.7	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	399	
Column Size:	**W14x61**	**ΦP_n=543 k**
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	108.7	kips
P_D=	261.8	kips
P_{Lr}=	5.5	kips
P_U=	490.9	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	491	
Column Size:	**W14x61**	**ΦP_n=543 k**
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	129.5	kips
P_D=	314.2	kips
P_{Lr}=	5.5	kips
P_U=	587.0	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	587	
Column Size:	**W14x68**	**ΦP_n=608 k**

Column 54 - Gravity Column

Tributary Area		
per floor or roof=	320.6	ft^2
Influence Area=	1211	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Green Roof)	30	psf
Construct. Roof LL=	20	psf
Load above level 12:	Roof +	
	1	Floor
LL_{red}	0.68	
P_L=	31.5	kips
P_D=	60.9	kips
P_{Lr}=	6.4	kips
P_U=	126.6	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	127	
Column Size:	**W14x43**	**ΦP_n=292 k**
Load above level 10:	Roof +	
	3	Floors
LL_{red}	0.50	
P_L=	57.6	kips
P_D=	121.8	kips
P_{Lr}=	6.4	kips
P_U=	241.6	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	242	
Column Size:	**W14x43**	**ΦP_n=292 k**
Load above level 8:	Roof +	
	5	Floors
LL_{red}	0.44	
P_L=	80.6	kips
P_D=	182.7	kips
P_{Lr}=	6.4	kips
P_U=	351.4	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	351	
Column Size:	**W14x53**	**ΦP_n=369 k**
Load above level 6:	Roof +	
	7	Floors
LL_{red}	0.41	
P_L=	102.3	kips
P_D=	243.7	kips
P_{Lr}=	6.4	kips
P_U=	459.2	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	459	
Column Size:	**W14x61**	**ΦP_n=543 k**
Load above level 4:	Roof +	
	9	Floors
LL_{red}	0.40	
P_L=	125.0	kips
P_D=	304.6	kips
P_{Lr}=	6.4	kips
P_U=	568.7	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	569	
Column Size:	**W14x68**	**ΦP_n=608 k**
Load above level 2:	Roof +	
	11	Floors
LL_{red}	0.40	
P_L=	150.7	kips
P_D=	365.5	kips
P_{Lr}=	6.4	kips
P_U=	682.9	kips
Unbalanced Moment, M_u=	0.0	kft
Column Shape=	W14's	
Column Depth=	14.0	in
Effective Length, K=	1.0	
Column Length, L=	15.0	ft
Equivalent Axial Load, P_{eq}=	683	
Column Size:	**W14x82**	**ΦP_n=735 k**

Note: Refer to final report for gravity column schedule

Appendix B: Wind Load Calculations

Existing Concrete System Wind Analysis

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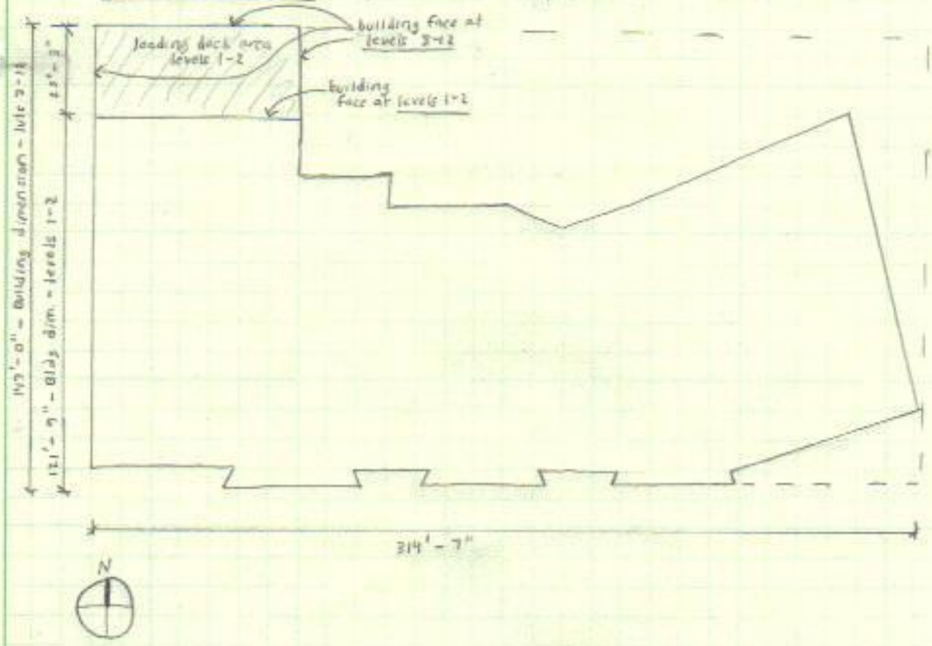
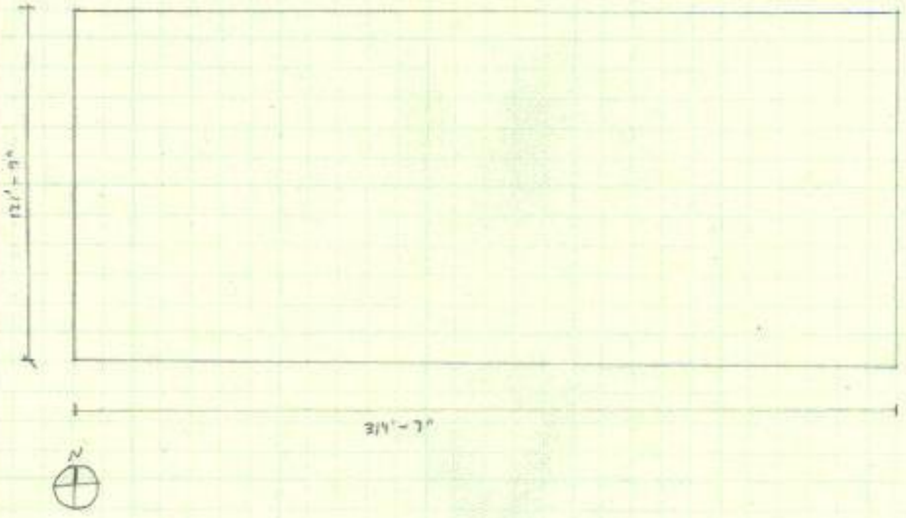
simplifying assumptions: change building geometry to rectangular for calculations



Levels 1-2 simplified building footprint



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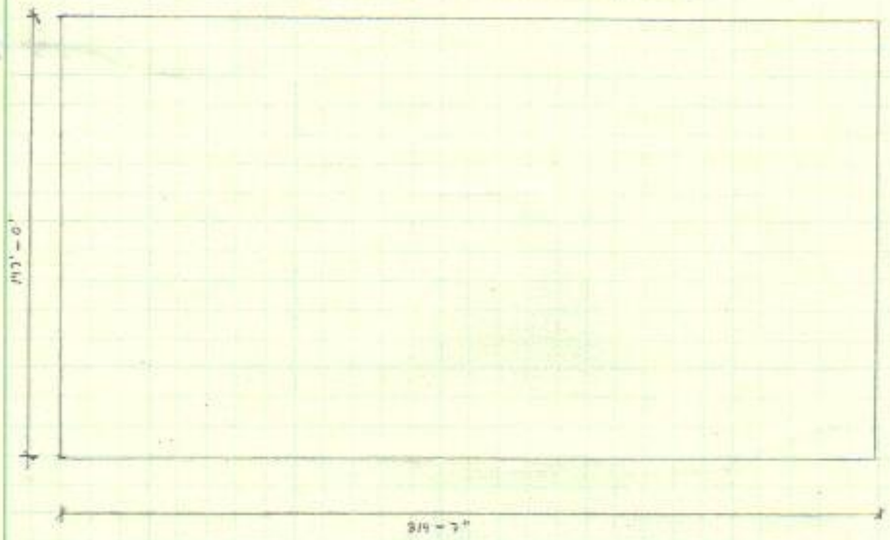
Levels 3-12 simplified building footprint



For analysis, use method 2

step 1: wind speed (V) → using Fig. 26.5-1A, V = 115 mph

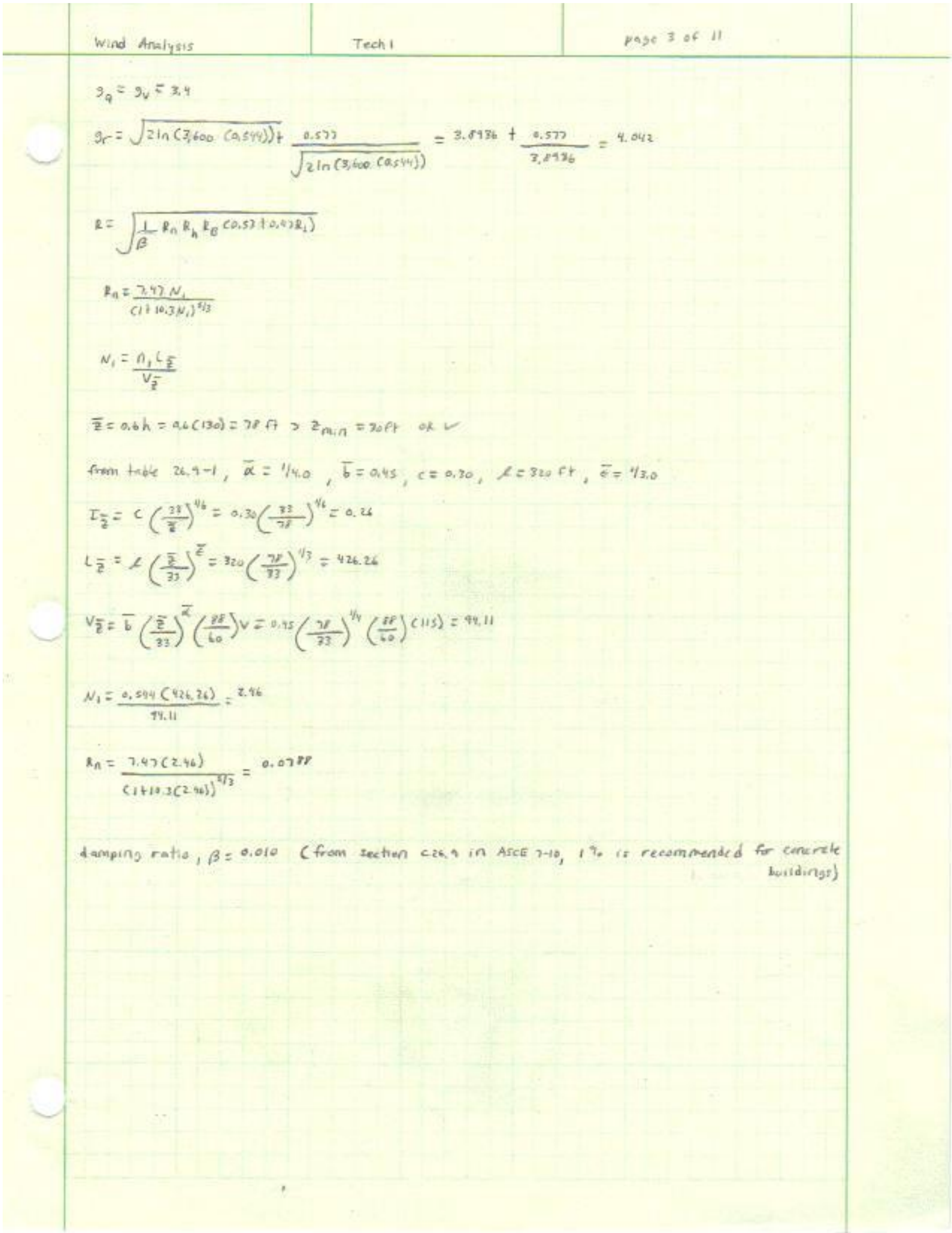
step 2: directionality factor, K_d → using Table 26.6-1, $K_d = 0.85$
step 3: exposure category - B (urban, suburban)
step 4: topographic factor, K_{zt} → from sect 26.8.2, $K_{zt} = 1.0$ for homogeneous topography

step 5: gust effect factor, G_f → from sect 26.9
- since bldg ht < 300' and bldg ht < 4Leff in both N-S and E-W directions, use na to determine the natural frequency.
  the approximate lower-band natural frequency, na, for a concrete moment-resiting frame (sect 26.9.3)

$$n_1 = n_a = \frac{43.5}{h^{0.9}}$$

$$= \frac{43.5}{130^{0.9}} = 0.544 \text{ hz} < 1 \text{ Hz} \therefore$$ calculate G_f in the event the building is flexible

$$G_f = 0.925\left(\frac{1+1.7 I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1+1.7 g_v I_{\bar{z}}}\right)$$



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$g_Q = g_v = 3.4$

$$g_R = \sqrt{2\ln(3{,}600\,(0.544))} + \frac{0.577}{\sqrt{2\ln(3{,}600\,(0.544))}} = 3.8936 + \frac{0.577}{3.8936} = 4.042$$

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

$$R_n = \frac{7.47\,N_1}{(1+10.3N_1)^{5/3}}$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{V_{\bar{z}}}$$

$\bar{z} = 0.6h = 0.6(130) = 78$ ft $> z_{min} = 30$ ft ok ✓

from table 26.9-1, $\bar{\alpha} = 1/4.0$, $\bar{b} = 0.45$, $c = 0.30$, $\ell = 320$ ft, $\bar{\epsilon} = 1/3.0$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.30\left(\frac{33}{78}\right)^{1/6} = 0.26$$

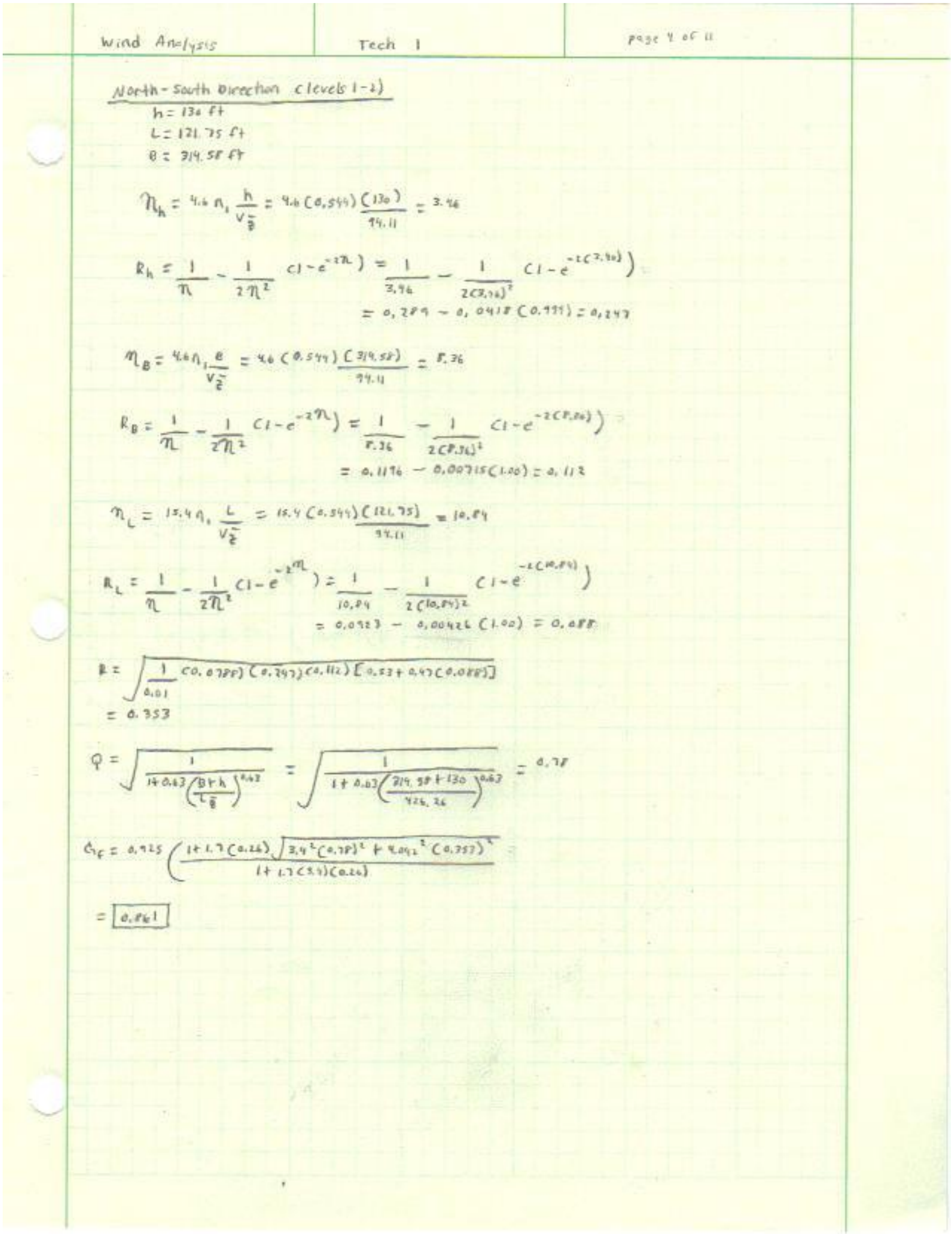
$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{78}{33}\right)^{1/3} = 426.26$$

$$V_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}}\left(\frac{88}{60}\right)V = 0.45\left(\frac{78}{33}\right)^{1/4}\left(\frac{88}{60}\right)(115) = 94.11$$

$$N_1 = \frac{0.594\,(426.26)}{94.11} = 2.46$$

$$R_n = \frac{7.47(2.46)}{(1+10.3(2.46))^{5/3}} = 0.0788$$

damping ratio, $\beta = 0.010$ (from section C26.9 in ASCE 7-10, 1% is recommended for concrete buildings)



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North-South Direction (levels 1-2)

h = 130 ft
L = 121.75 ft
B = 314.58 ft

$$\eta_h = 4.6\, n_1 \frac{h}{\bar{V}_{\bar{z}}} = 4.6(0.544)\frac{(130)}{94.11} = 3.46$$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}(1 - e^{-2\eta}) = \frac{1}{3,46} - \frac{1}{2(3,46)^2}(1 - e^{-2(3.46)})$$

$$= 0,289 - 0,0418(0.999) = 0,247$$

$$\eta_B = 4.6\, n_1 \frac{B}{\bar{V}_{\bar{z}}} = 4.6(0.544)\frac{(314.58)}{94.11} = 8.36$$

$$R_B = \frac{1}{\eta} - \frac{1}{2\eta^2}(1 - e^{-2\eta}) = \frac{1}{8.36} - \frac{1}{2(8.36)^2}(1 - e^{-2(8.36)})$$

$$= 0.1196 - 0.00715(1.00) = 0.112$$

$$\eta_L = 15.4\, n_1 \frac{L}{\bar{V}_{\bar{z}}} = 15.4(0.544)\frac{(121.75)}{94.11} = 10.84$$

$$R_L = \frac{1}{\eta} - \frac{1}{2\eta^2}(1 - e^{-2\eta}) = \frac{1}{10.84} - \frac{1}{2(10.84)^2}(1 - e^{-2(10.84)})$$

$$= 0.0923 - 0.00426(1.00) = 0.088$$

$$R = \sqrt{\frac{1}{0.01}(0.788)(0.247)(0.112)[0.53 + 0.47(0.088)]}$$

$$= 0.353$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1 + 0.63\left(\frac{314.58+130}{426.26}\right)^{0.63}}} = 0.78$$

$$G_f = 0.925\left(\frac{1 + 1.7(0.26)\sqrt{3.4^2(0.78)^2 + 4.092^2(0.353)^2}}{1 + 1.7(3.4)(0.26)}\right.$$

$$= \boxed{0.861}$$

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East-West Direction (levels 1-2)

$h = 130 \text{ ft}$

$L = 314.58 \text{ ft}$

$B = 121.75 \text{ ft}$

$n_h = 3.46$ (see N-S direction)

$R_h = 0.247$ (see N-S direction)

$$n_B = 4.6(0.544)\frac{(121.75)}{94.11} = 3.24$$

$$R_B = \frac{1}{3.24} - \frac{1}{2(3.24)^2}\left(1 - e^{-2(3.24)}\right)$$

$$= 0.309 - 0.0476(0.998) = 0.261$$

$$n_L = 15.4(0.544)\frac{(314.58)}{94.11} = 28.00$$

$$R_L = \frac{1}{28.00} - \frac{1}{2(28)^2}\left(1 - e^{-2(28)}\right)$$

$$= 0.0357 - 0.000638 = 0.035$$

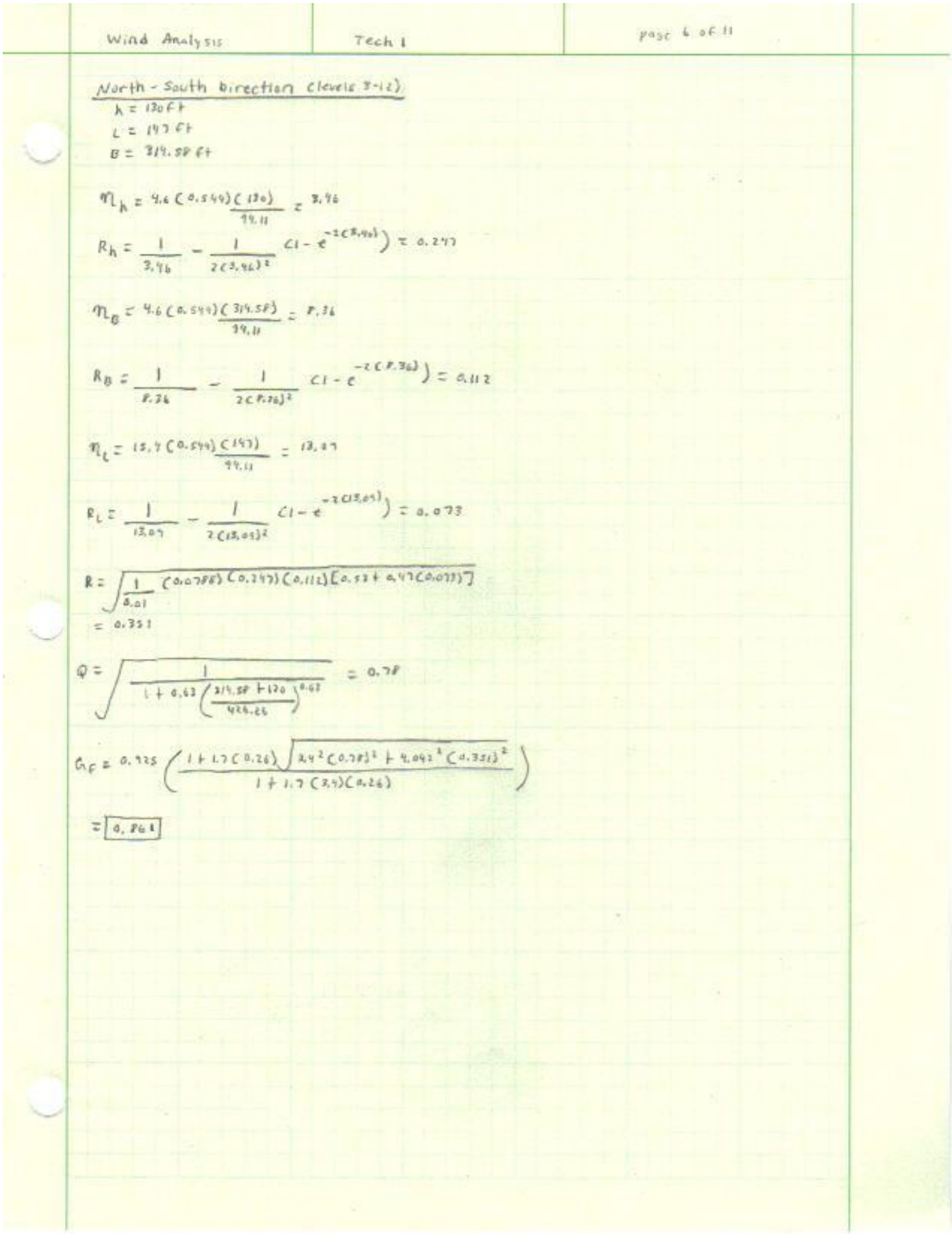
$$R = \sqrt{\frac{1}{0.01}(0.0788)(0.247)(0.261)[0.53 + 0.47(0.035)]}$$

$$= 0.527$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{121.75 + 130}{426.26}\right)^{0.63}}} = 0.83$$

$$G_f = 0.925\left(\frac{1 + 1.7(0.26)\sqrt{3.4^2(0.83)^2 + 4.042^2(0.527)^2}}{1 + 1.7(3.4)(0.26)}\right)$$

$$= \boxed{0.945}$$



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North - South Direction (levels 3-12)

$h = 130 ft$

$L = 147 ft$

$B = 314.58 ft$

$$\eta_h = 4.6(0.544)\frac{(130)}{94.11} = 3.46$$

$$R_h = \frac{1}{3.46} - \frac{1}{2(3.46)^2}(1 - e^{-2(3.46)}) = 0.247$$

$$\eta_B = 4.6(0.544)\frac{(314.58)}{94.11} = 8.36$$

$$R_B = \frac{1}{8.36} - \frac{1}{2(8.36)^2}(1 - e^{-2(8.36)}) = 0.112$$

$$\eta_L = 15.4(0.544)\frac{(147)}{94.11} = 13.09$$

$$R_L = \frac{1}{13.09} - \frac{1}{2(13.09)^2}(1 - e^{-2(13.09)}) = 0.073$$

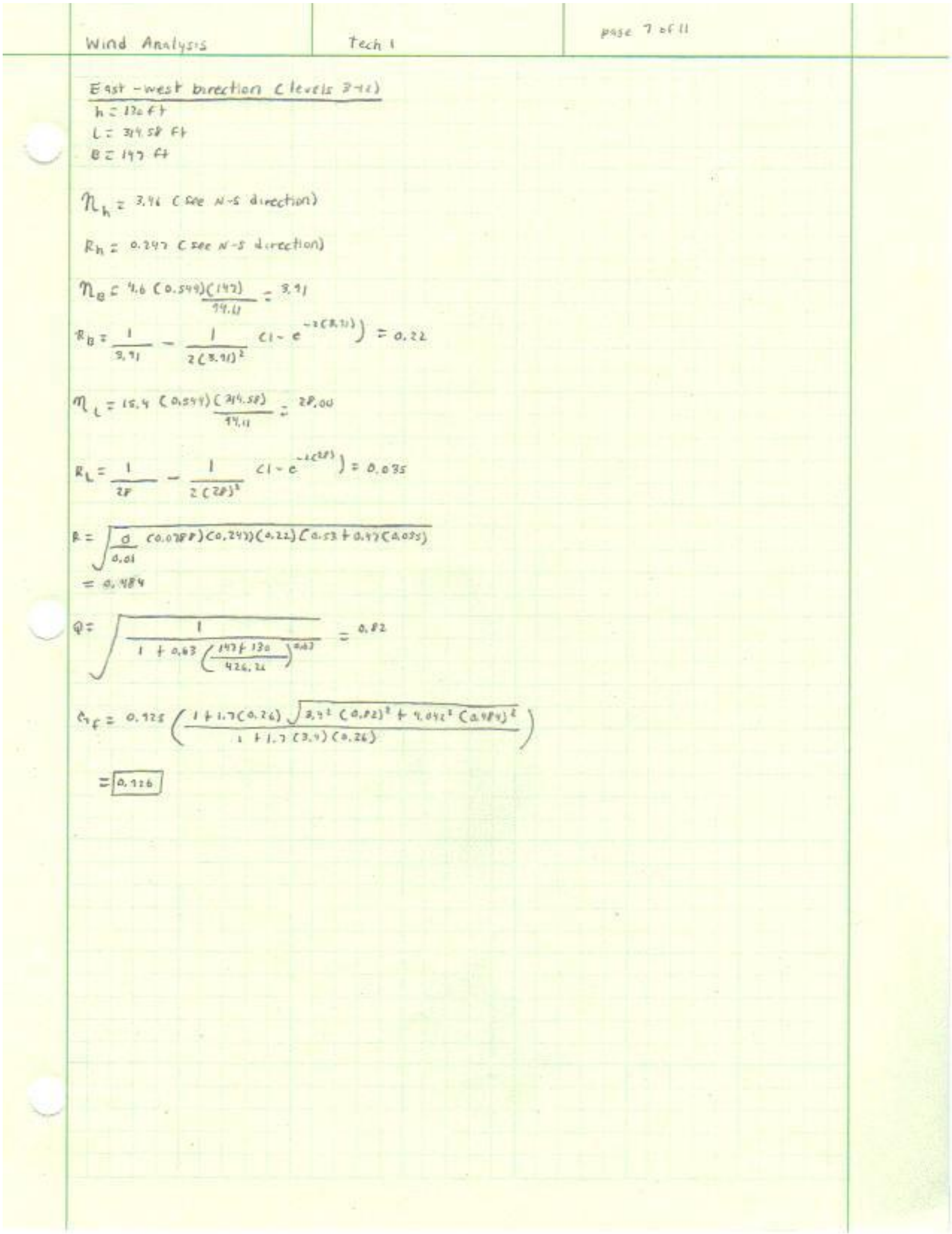
$$R = \sqrt{\frac{1}{0.01}(0.0786)(0.247)(0.112)[0.53 + 0.47(0.073)]}$$

$$= 0.351$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{314.58 + 130}{426.26}\right)^{0.63}}} = 0.78$$

$$G_F = 0.925\left(\frac{1 + 1.7(0.26)\sqrt{3.4^2(0.78)^2 + 4.042^2(0.351)^2}}{1 + 1.7(3.4)(0.26)}\right)$$

$$= \boxed{0.861}$$



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East-west Direction (levels 3-12)

h = 130 ft
L = 314.58 ft
B = 147 ft

$\eta_h = 3.96$ (see N-S direction)

$R_h = 0.247$ (see N-S direction)

$$\eta_B = 4.6\,(0.544)\frac{(147)}{44.11} = 8.91$$

$$R_B = \frac{1}{8.91} - \frac{1}{2(8.91)^2}\left(1 - e^{-2(8.91)}\right) = 0.22$$

$$\eta_L = 15.4\,(0.544)\frac{(314.58)}{44.11} = 28.04$$

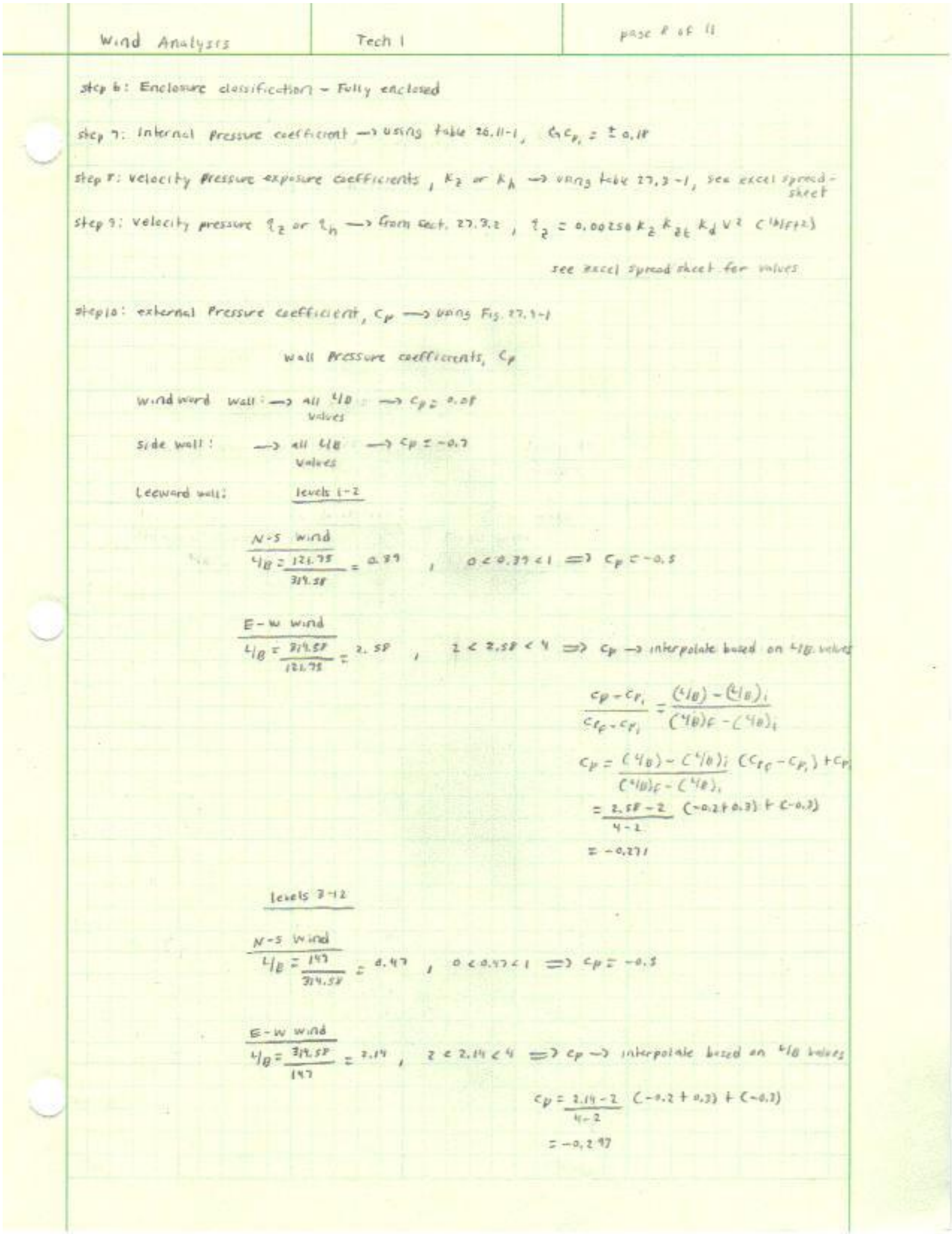
$$R_L = \frac{1}{28} - \frac{1}{2(28)^2}\left(1 - e^{-2(28)}\right) = 0.035$$

$$R = \sqrt{\frac{1}{0.01}(0.0788)(0.247)(0.22)(0.53 + 0.47(0.035))}$$
$$= 0.484$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{147 + 130}{426.26}\right)^{0.63}}} = 0.82$$

$$G_f = 0.925\left(\frac{1 + 1.7(0.26)\sqrt{3.4^2(0.82)^2 + 4.042^2(0.484)^2}}{1 + 1.7(3.4)(0.26)}\right)$$

$$= \boxed{0.926}$$



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step 6: Enclosure classification - Fully enclosed

step 7: Internal pressure coefficient → using table 26.11-1, $GC_{p_i} = \pm 0.18$

step 8: velocity pressure exposure coefficients, K_z or K_h → using table 27.3-1, see excel spreadsheet

step 9: velocity pressure q_z or q_h → from sect. 27.3.2, $q_z = 0.00256 K_z K_{zt} K_d V^2$ (lb/ft2)

see excel spreadsheet for values

step 10: external pressure coefficient, C_p → using Fig. 27.4-1

wall pressure coefficients, C_p

windward wall: → all L/B values → $C_p = 0.8$

side wall: → all L/B values → $C_p = -0.7$

Leeward wall: levels 1-2

N-S wind

$$L/B = \frac{121.75}{319.58} = 0.39 \quad , \quad 0 < 0.39 < 1 \Rightarrow C_p = -0.5$$

E-W wind

$$L/B = \frac{319.58}{121.75} = 2.58 \quad , \quad 2 < 2.58 < 4 \Rightarrow C_p \rightarrow \text{interpolate based on L/B values}$$

$$\frac{C_p - C_{p_i}}{C_{p_f} - C_{p_i}} = \frac{(L/B) - (L/B)_i}{(L/B)_f - (L/B)_i}$$

$$C_p = \frac{(L/B) - (L/B)_i}{(L/B)_f - (L/B)_i}(C_{p_f} - C_{p_i}) + C_{p_i}$$

$$= \frac{2.58 - 2}{4 - 2}(-0.2 + 0.3) + (-0.3)$$

$$= -0.271$$

levels 3-12

N-S wind

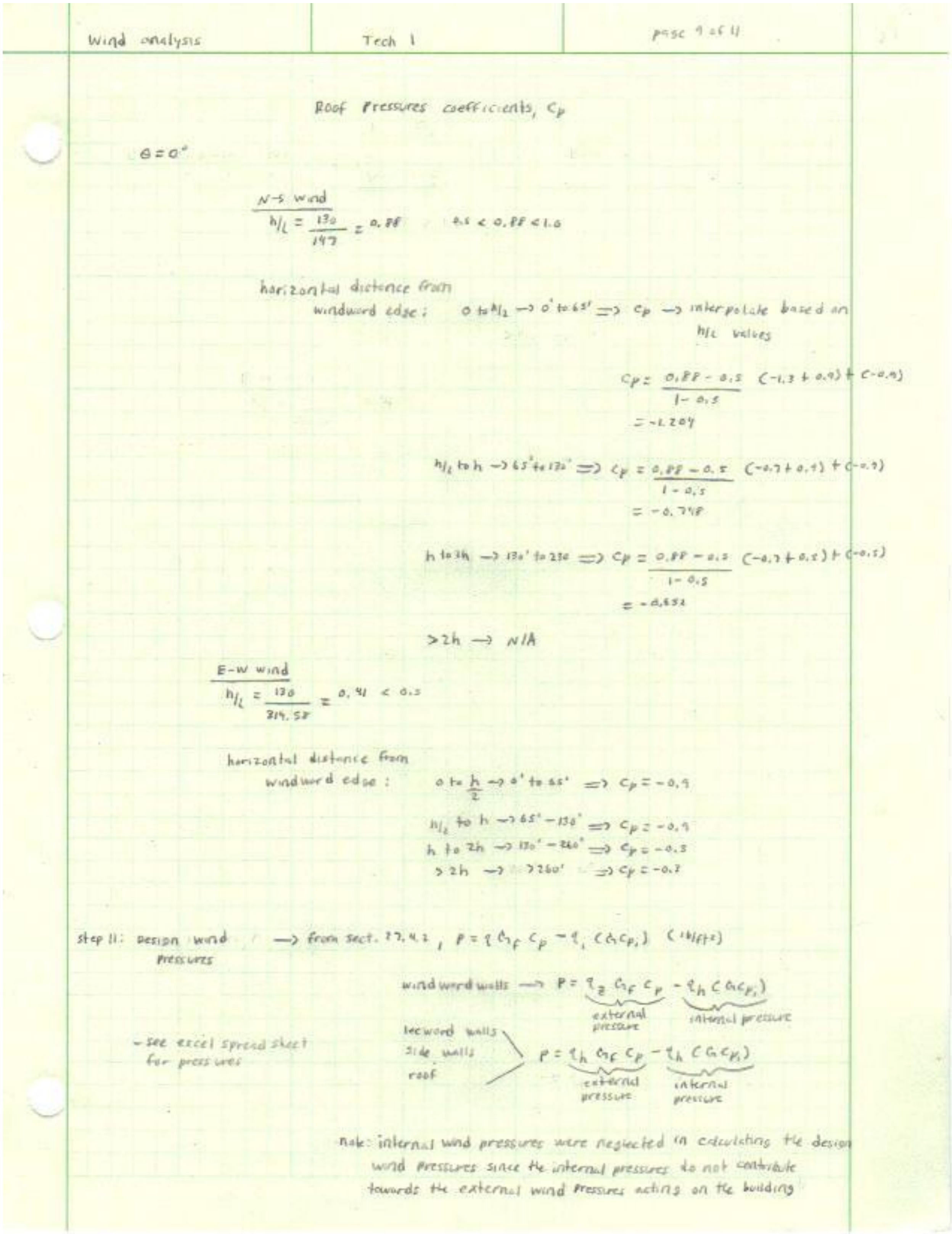
$$L/B = \frac{147}{314.58} = 0.47 \quad , \quad 0 < 0.47 < 1 \Rightarrow C_p = -0.5$$

E-W wind

$$L/B = \frac{314.58}{147} = 2.14 \quad , \quad 2 < 2.14 < 4 \Rightarrow C_p \rightarrow \text{interpolate based on L/B values}$$

$$C_p = \frac{2.14 - 2}{4 - 2}(-0.2 + 0.3) + (-0.3)$$

$$= -0.297$$



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Roof Pressures coefficients, C_p

$\theta = 0°$

N-S wind

$h/L = \frac{130}{147} = 0.88 \qquad 0.5 < 0.88 < 1.0$

horizontal distance from windward edge: 0 to $h/2 \rightarrow 0'$ to $65' \Rightarrow C_p \rightarrow$ interpolate based on h/L values

$$C_p = \frac{0.88 - 0.5}{1 - 0.5}(-1.3 + 0.9) + (-0.9)$$
$$= -1.204$$

$h/2$ to $h \rightarrow 65'$ to $130' \Rightarrow C_p = \frac{0.88 - 0.5}{1 - 0.5}(-0.7 + 0.9) + (-0.9)$

$$= -0.748$$

h to $2h \rightarrow 130'$ to $230 \Rightarrow C_p = \frac{0.88 - 0.5}{1 - 0.5}(-0.7 + 0.5) + (-0.5)$

$$= -0.652$$

$> 2h \rightarrow$ N/A

E-W wind

$h/L = \frac{130}{314.58} = 0.41 < 0.5$

horizontal distance from windward edge:

0 to $\frac{h}{2} \rightarrow 0'$ to $65' \Rightarrow C_p = -0.9$

$h/2$ to $h \rightarrow 65' - 130' \Rightarrow C_p = -0.9$

h to $2h \rightarrow 130' - 260' \Rightarrow C_p = -0.5$

$> 2h \rightarrow > 260' \Rightarrow C_p = -0.3$

step II: Design wind pressures $\rightarrow$ from sect. 27.4.2, $p = qG_f C_p - q_i(GC_{p_i})$ (lb/ft²)

windward walls $\rightarrow p = \underbrace{q_z G_f C_p}_{\text{external pressure}} - \underbrace{q_h(GC_{p_i})}_{\text{internal pressure}}$

- see excel spread sheet for pressures

leeward walls
side walls
roof

$p = \underbrace{q_h G_f C_p}_{\text{external pressure}} - \underbrace{q_h(GC_{p_i})}_{\text{internal pressure}}$

note: internal wind pressures were neglected in calculating the design wind pressures since the internal pressures do not contribute towards the external wind pressures acting on the building

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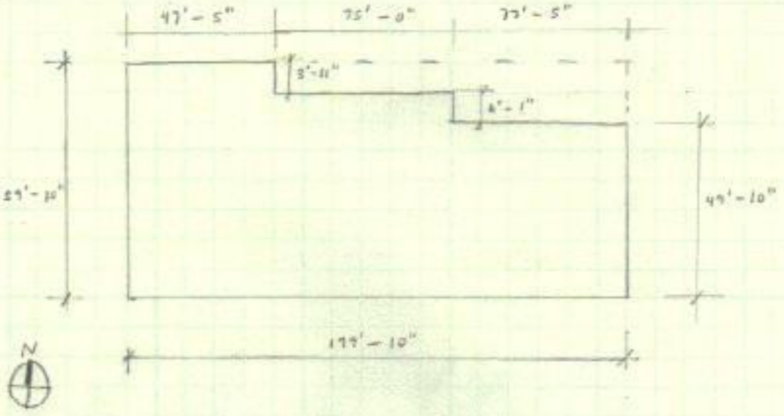
Mechanical Penthouse

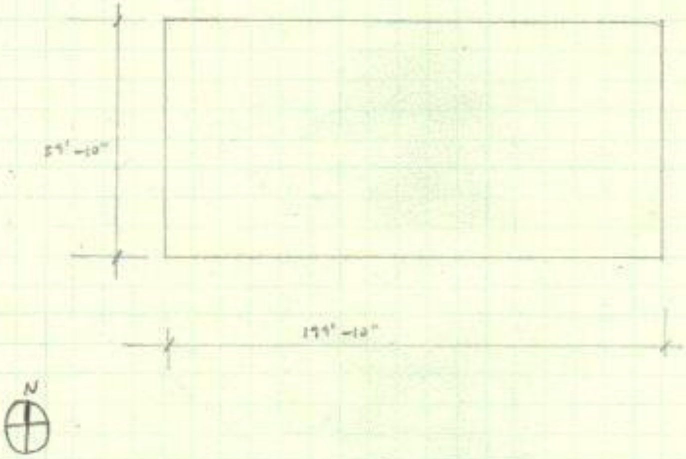
Assumption: mech. penthouse is not a part of the MWFRS

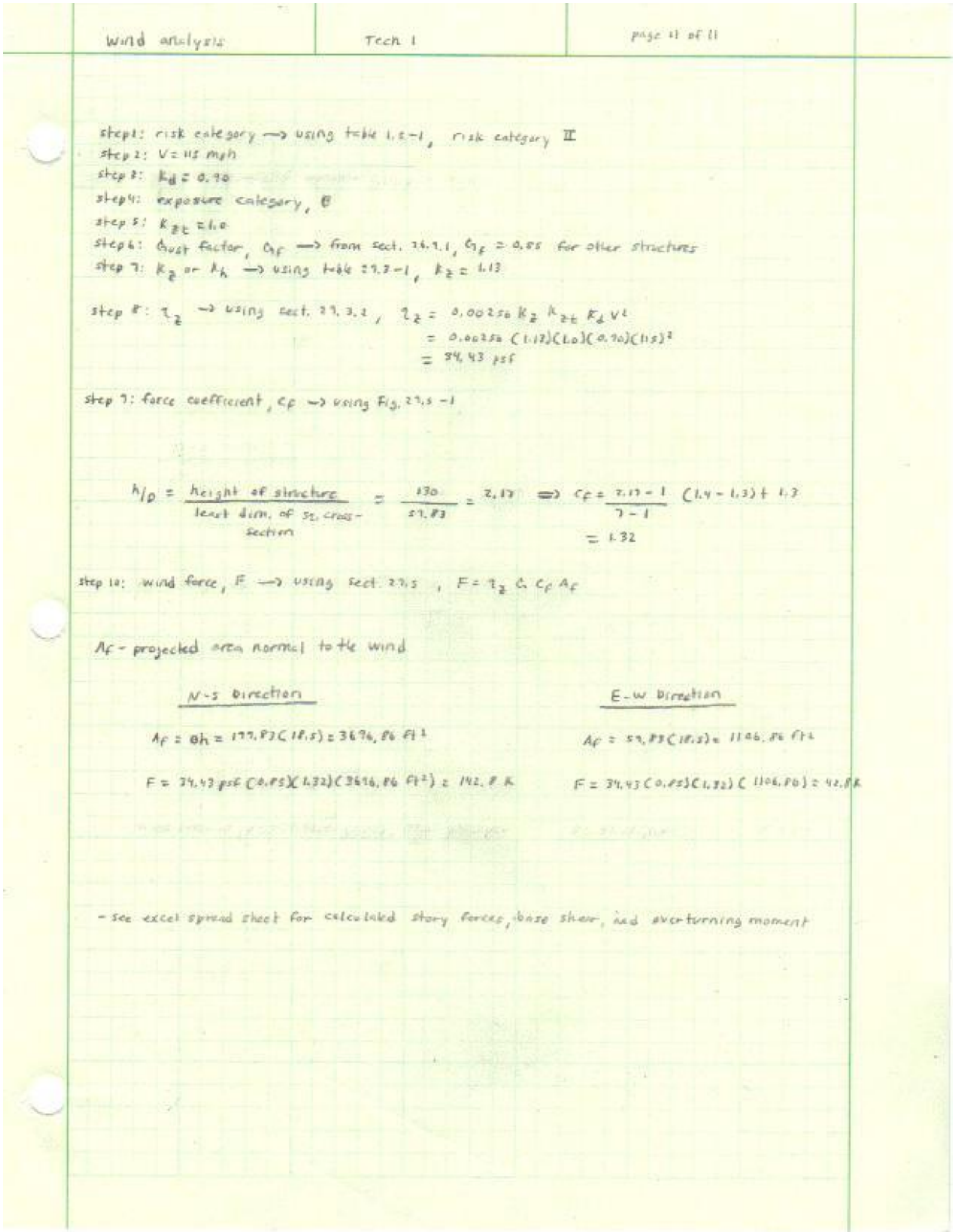
- the penthouse will be analyzed as a separate structure and therefore the guidelines listed in chapter 29 (wind loads on other structures and building appurtenances - MWFRS) will be followed for analysis

h = 18'-6"



simplifying assumptions: change penthouse geometry to rectangular for calculations





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step1: risk category → using table 1.5-1, risk category II
step 2: V = 115 mph
step 3: $K_d = 0.90$
step4: exposure category, B
step 5: $K_{zt} = 1.0$
step6: Gust factor, G_f → from sect. 26.9.1, $G_f = 0.85$ for other structures
step 7: K_z or K_h → using table 29.3-1, $K_z = 1.13$

step 8: q_z → using sect. 29.3.2, $q_z = 0.00256\, K_z K_{zt} K_d V^2$

$$= 0.00256\,(1.13)(1.0)(0.90)(115)^2$$

$$= 34.43 \text{ psf}$$

step 9: force coefficient, C_f → using Fig. 29.5-1

$$h/D = \frac{\text{height of structure}}{\text{least dim. of st. cross-section}} = \frac{130}{59.83} = 2.17 \Rightarrow C_f = \frac{2.17-1}{7-1}(1.4-1.3)+1.3$$

$$= 1.32$$

step 10: wind force, F → using sect. 29.5, $F = q_z G C_f A_f$

A_f - projected area normal to the wind

N-S Direction

$A_f = Bh = 199.83(18.5) = 3696.86 \text{ ft}^2$

$F = 34.43 \text{ psf}\,(0.85)(1.32)(3696.86 \text{ ft}^2) = 142.8 \text{ K}$

E-W Direction

$A_f = 59.83(18.5) = 1106.86 \text{ ft}^2$

$F = 34.43(0.85)(1.32)(1106.86) = 42.8 \text{ K}$

- see excel spread sheet for calculated story forces, base shear, and overturning moment

N-S Direction				E-W Direction			
Level	h_i (ft) (Height above grade of level i)	L_i (ft) (Building Length at level i)	h_i*L_i	Level	h_i (ft) (Height above grade of level i)	L_i (ft) (Building Length at level i)	h_i*L_i
1	0	121.75	0	1	0	314.58	0
2	12.54	121.75	1526.75	2	12.54	314.58	3944.833
3	23.17	147	3405.99	3	23.17	314.58	7288.819
4	33.79	147	4967.13	4	33.79	314.58	10629.66
5	44.42	147	6529.74	5	44.42	314.58	13973.64
6	55.04	147	8090.88	6	55.04	314.58	17314.48
7	65.67	147	9653.49	7	65.67	314.58	20658.47
8	76.29	147	11214.63	8	76.29	314.58	23999.31
9	86.92	147	12777.24	9	86.92	314.58	27343.29
10	97.54	147	14338.38	10	97.54	314.58	30684.13
11	108.17	147	15900.99	11	108.17	314.58	34028.12
12	118.79	147	17462.13	12	118.79	314.58	37368.96
Main roof	130	147	19110	Main roof	130	314.58	40895.4
∑=	852.34		124977.35	∑=	852.34		268129.1
L_{eff}=	146.63			L_{eff}=	314.58		

Building Effective Length, L_{eff}, used to determine the natural frequency of the building according to Sect. 26.9 of ASCE 7-10

Velocity Pressure Coefficients, K_z, and Velocity Pressures, q_z

Level	Elevation (ft)	K_z	q_z
1	0	0.57	16.40
2	12.54	0.57	16.40
3	23.17	0.66	18.99
4	33.79	0.76	21.87
5	44.42	0.81	23.31
6	55.04	0.85	24.46
7	65.67	0.89	25.61
8	76.29	0.93	26.76
9	86.92	0.96	27.63
10	97.54	0.99	28.49
11	108.17	1.04	29.93
12	118.79	1.04	29.93
Main Roof	130	1.09	31.37
PH Roof	148.5	1.13	34.43

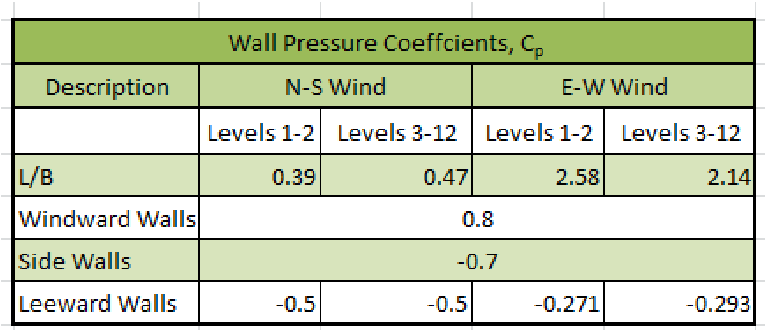
Gust Factor-MWFRS

Variable	N-S Wind		E-W Wind	
$n_1=n_a$	0.544			
$g_Q=g_v$	3.4			
g_R	4.042			
Z_{mean}	78			
$I_{z, mean}$	0.26			
$L_{z, mean}$	426.26			
$V_{z,mean}$	94.11			
N_1	2.46			
R_n	0.0788			
β	0.01			
η_h	3.46			
R_h	0.247			
	Levels 1-2	Levels 3-12	Levels 1-2	Levels 3-12
η_B	8.36	8.36	3.24	3.91
R_B	0.112	0.112	0.261	0.22
η_L	10.84	13.09	28	28
R_L	0.088	0.073	0.035	0.035
R	0.353	0.351	0.527	0.484
Q	0.78	0.78	0.83	0.82
G_f	0.861	0.861	0.945	0.926

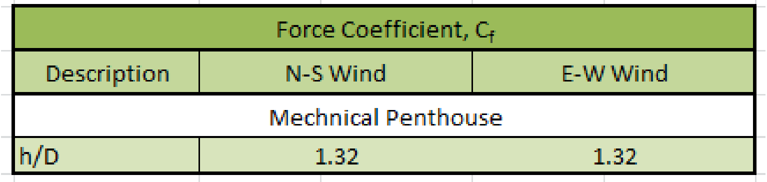
Gust Factor-Mechnical Penthouse

Variable	N-S Wind	E-W Wind
G_f	0.85	0.85

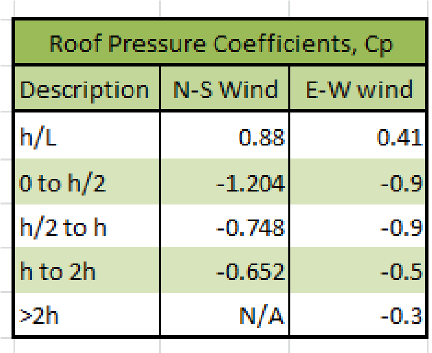
Velocity pressures (above) and gust factors (below) for the MWFRS



Wall Pressure Coefficients, C_p				
Description	N-S Wind		E-W Wind	
	Levels 1-2	Levels 3-12	Levels 1-2	Levels 3-12
L/B	0.39	0.47	2.58	2.14
Windward Walls	0.8			
Side Walls	-0.7			
Leeward Walls	-0.5	-0.5	-0.271	-0.293

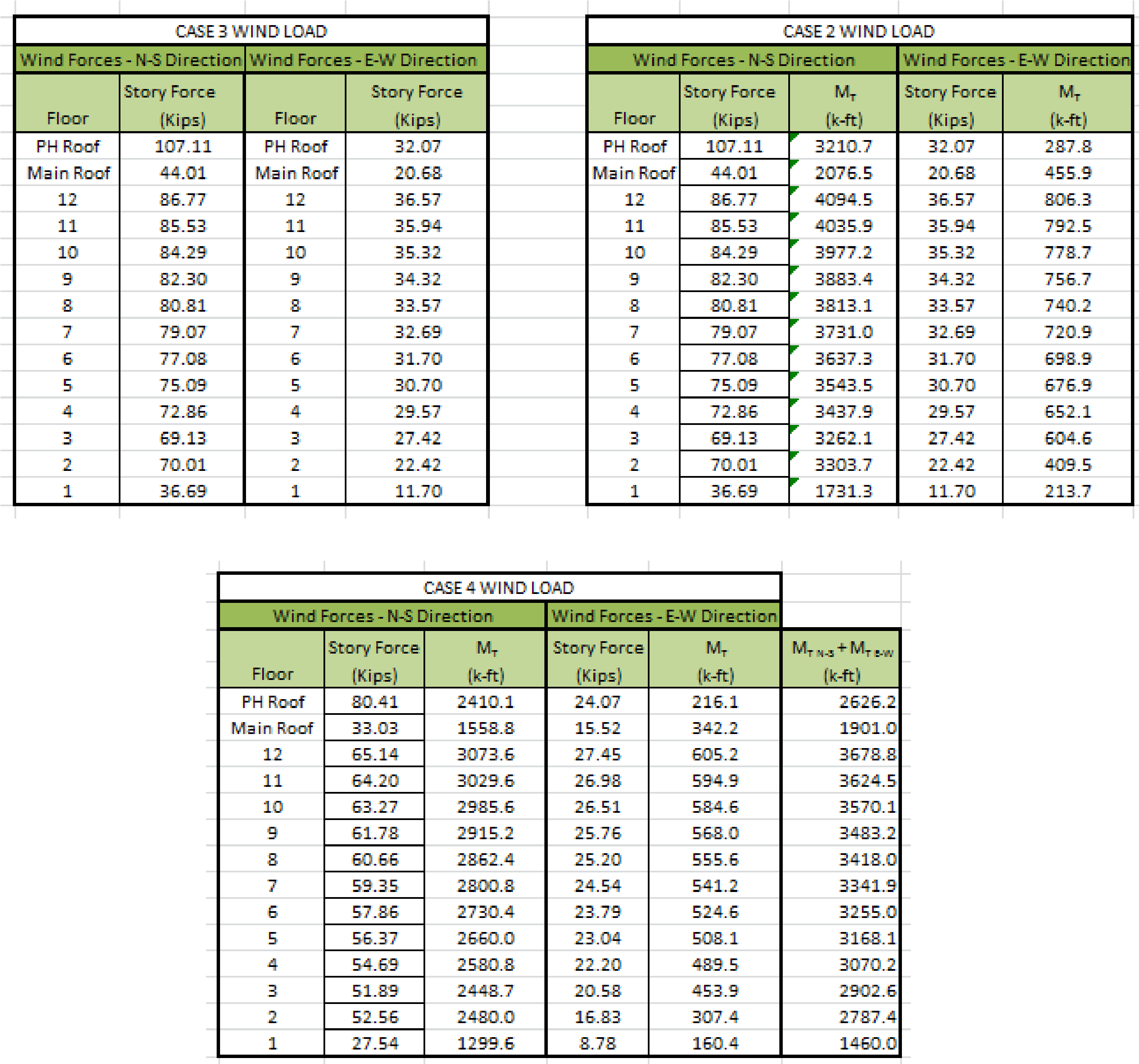


Force Coefficient, C_f		
Description	N-S Wind	E-W Wind
Mechnical Penthouse		
h/D	1.32	1.32



Roof Pressure Coefficients, Cp		
Description	N-S Wind	E-W wind
h/L	0.88	0.41
0 to h/2	-1.204	-0.9
h/2 to h	-0.748	-0.9
h to 2h	-0.652	-0.5
>2h	N/A	-0.3

Pressure coefficients



CASE 3 WIND LOAD			
Wind Forces - N-S Direction		Wind Forces - E-W Direction	
Floor	Story Force (Kips)	Floor	Story Force (Kips)
PH Roof	107.11	PH Roof	32.07
Main Roof	44.01	Main Roof	20.68
12	86.77	12	36.57
11	85.53	11	35.94
10	84.29	10	35.32
9	82.30	9	34.32
8	80.81	8	33.57
7	79.07	7	32.69
6	77.08	6	31.70
5	75.09	5	30.70
4	72.86	4	29.57
3	69.13	3	27.42
2	70.01	2	22.42
1	36.69	1	11.70

CASE 2 WIND LOAD				
Wind Forces - N-S Direction			Wind Forces - E-W Direction	
Floor	Story Force (Kips)	M_T (k-ft)	Story Force (Kips)	M_T (k-ft)
PH Roof	107.11	3210.7	32.07	287.8
Main Roof	44.01	2076.5	20.68	455.9
12	86.77	4094.5	36.57	806.3
11	85.53	4035.9	35.94	792.5
10	84.29	3977.2	35.32	778.7
9	82.30	3883.4	34.32	756.7
8	80.81	3813.1	33.57	740.2
7	79.07	3731.0	32.69	720.9
6	77.08	3637.3	31.70	698.9
5	75.09	3543.5	30.70	676.9
4	72.86	3437.9	29.57	652.1
3	69.13	3262.1	27.42	604.6
2	70.01	3303.7	22.42	409.5
1	36.69	1731.3	11.70	213.7

CASE 4 WIND LOAD					
Wind Forces - N-S Direction			Wind Forces - E-W Direction		
Floor	Story Force (Kips)	M_T (k-ft)	Story Force (Kips)	M_T (k-ft)	$M_{T\ N\text{-}S} + M_{T\ E\text{-}W}$ (k-ft)
PH Roof	80.41	2410.1	24.07	216.1	2626.2
Main Roof	33.03	1558.8	15.52	342.2	1901.0
12	65.14	3073.6	27.45	605.2	3678.8
11	64.20	3029.6	26.98	594.9	3624.5
10	63.27	2985.6	26.51	584.6	3570.1
9	61.78	2915.2	25.76	568.0	3483.2
8	60.66	2862.4	25.20	555.6	3418.0
7	59.35	2800.8	24.54	541.2	3341.9
6	57.86	2730.4	23.79	524.6	3255.0
5	56.37	2660.0	23.04	508.1	3168.1
4	54.69	2580.8	22.20	489.5	3070.2
3	51.89	2448.7	20.58	453.9	2902.6
2	52.56	2480.0	16.83	307.4	2787.4
1	27.54	1299.6	8.78	160.4	1460.0

Story forces for wind loads cases 2 through 4

New Steel System Wind Analysis

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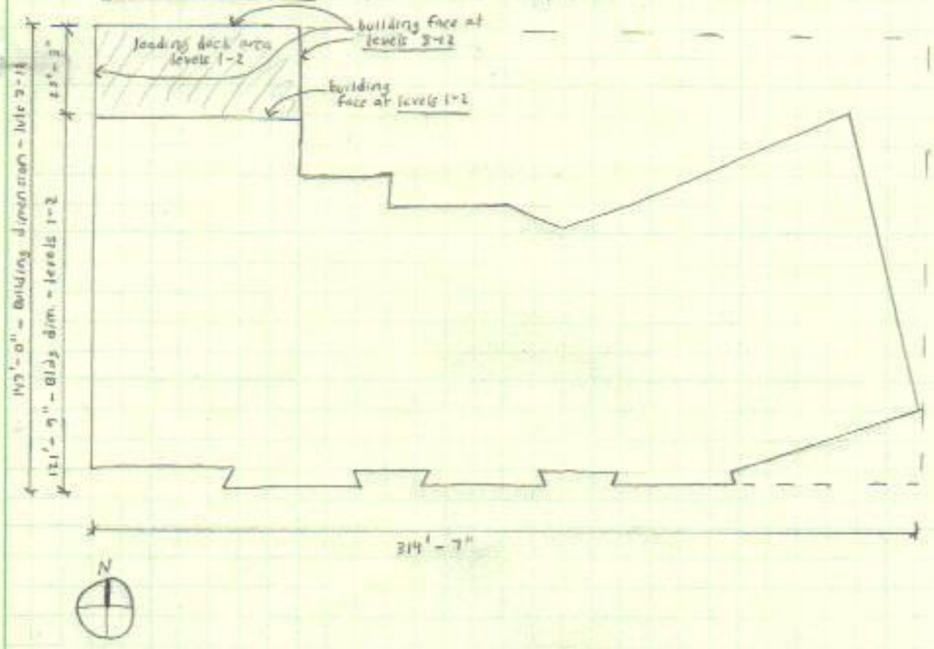
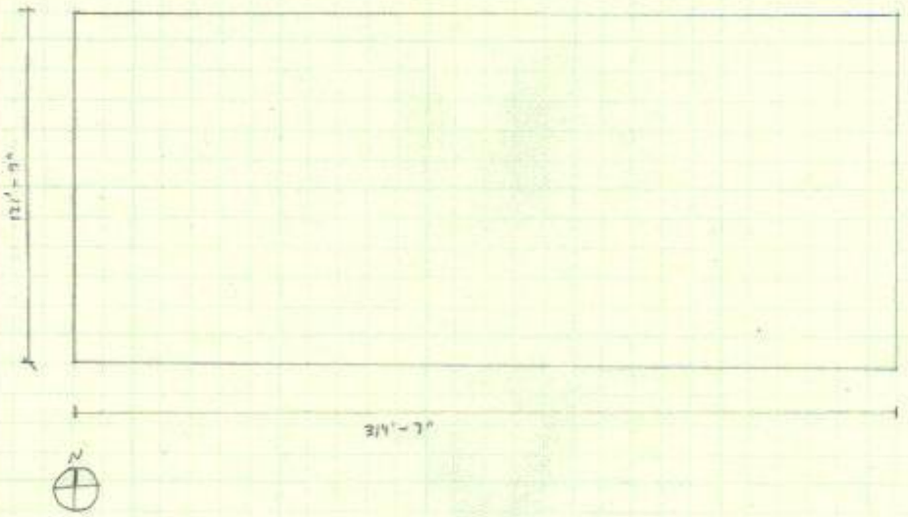
simplifying assumptions: change building geometry to rectangular for calculations



Levels 1-2 simplified building footprint



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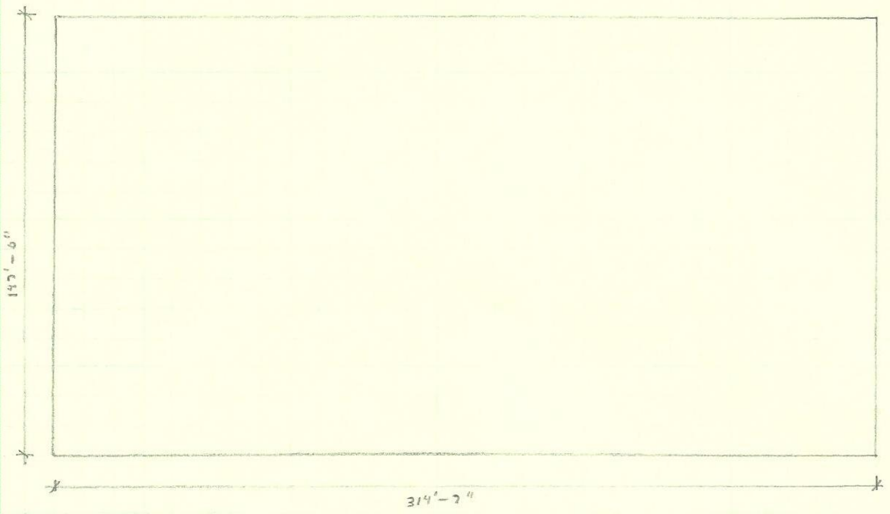
Levels 3-12 simplified building foot print



For analysis, use method 2

step 1: wind speed (V) → using Fig. 26.5-1A, V = 115 mph (location Arlington, VA)
step 2: directionality factor, K_d → using Table 26.6-1, $K_d = 0.85$
step 3: exposure category - B (urban, suburban)
step 4: topographic factor, K_{zt} → from sect. 26.8.2, $K_{zt} = 1.0$ for homogeneous topography

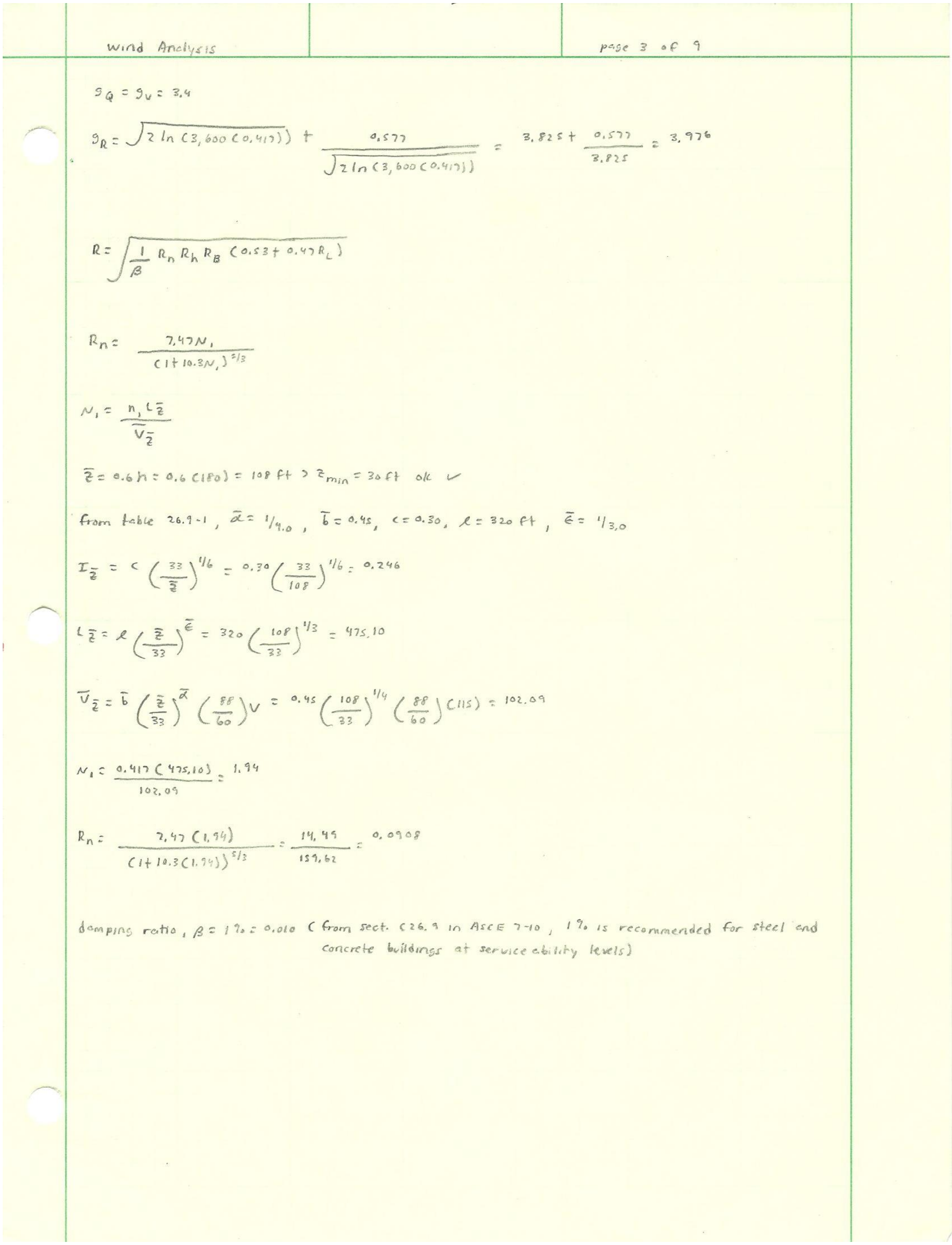
step 5: gust effect factor, G_f → from sect. 26.9

- since bldg ht < 300 ft and bldg ht < $4L_{eff}$ in both N-S and E-W direction, use n_a to determine the natural frequency.

the approximate lower-bound natural frequency, n_a, for structural steel and concrete buildings with other lateral-force resisting systems:

$n_1 = n_a = \frac{75}{h} = \frac{75}{180} = 0.417\ Hz < 1\ Hz$ ∴ calculate G_f in the event the building is flexible

$$G_f = 0.925\left(\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right)$$



wind Analysis

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$g_Q = g_v = 3.4$

$$g_R = \sqrt{2\ln(3{,}600(0.417))} + \frac{0.577}{\sqrt{2\ln(3{,}600(0.417))}} = 3.825 + \frac{0.577}{3.825} = 3.976$$

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

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}}$$

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

$\bar{z} = 0.6h = 0.6(180) = 108\text{ ft} > z_{min} = 30\text{ ft}$ ok ✓

from table 26.9-1, $\bar{\alpha} = 1/4.0$, $\bar{b} = 0.45$, $c = 0.30$, $\ell = 320\text{ ft}$, $\bar{\epsilon} = 1/3.0$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.30\left(\frac{33}{108}\right)^{1/6} = 0.246$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{108}{33}\right)^{1/3} = 475.10$$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}}\left(\frac{88}{60}\right)V = 0.45\left(\frac{108}{33}\right)^{1/4}\left(\frac{88}{60}\right)(115) = 102.09$$

$$N_1 = \frac{0.417(475.10)}{102.09} = 1.94$$

$$R_n = \frac{7.47(1.94)}{(1 + 10.3(1.94))^{5/3}} = \frac{14.49}{159.62} = 0.0908$$

damping ratio, $\beta = 1\% = 0.010$ (from sect. C26.9 in ASCE 7-10, 1% is recommended for steel and concrete buildings at serviceability levels)

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North-South Direction (levels 1-2)

$h = 180 \text{ ft}$
$L = 121.75 \text{ ft}$
$B = 314.58 \text{ ft}$

$$\eta_h = 4.6 n_1 \frac{h}{\bar{V}_{\bar{z}}} = \frac{4.6(0.417)(180)}{102.09} = 3.38$$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta}) = \frac{1}{3.38} - \frac{1}{2(3.38)^2}(1-e^{-2(3.38)}) = 0.296 - 0.0438(0.999) = 0.252$$

$$\eta_B = 4.6 n_1 \frac{B}{\bar{V}_{\bar{z}}} = \frac{4.6(0.417)(314.58)}{102.09} = 5.91$$

$$R_B = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta}) = \frac{1}{5.91} - \frac{1}{2(5.91)^2}(1-e^{-2(5.91)}) = 0.1692 - 0.0143(1.00) = 0.155$$

$$\eta_L = 15.4 n_1 \frac{L}{\bar{V}_{\bar{z}}} = \frac{15.4(0.417)(121.75)}{102.09} = 7.66$$

$$R_L = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta}) = \frac{1}{7.66} - \frac{1}{2(7.66)^2}(1-e^{-2(7.66)}) = 0.1305 - 0.00852(1.00) = 0.122$$

$$R = \sqrt{\frac{1}{0.01}(0.0908)(0.252)(0.155)(0.53 + 0.47(0.122))} = 0.456$$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{314.58+180}{475.10}\right)^{0.63}}} = 0.78$$

$$G_f = 0.925\left(\frac{1+1.7(0.246)\sqrt{3.4^2(0.78)^2 + 3.976^2(0.456)^2}}{1+1.7(3.4)(0.246)}\right)$$

$$= \boxed{0.895}$$

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East - west Direction (levels 1-2)

$h = 180 \text{ ft}$

$L = 314.58 \text{ ft}$

$B = 121.75 \text{ ft}$

$\eta_h = 3.38$ (see N-S direction)

$R_h = 0.252$ (see N-S direction)

$$\eta_B = 4.6 (0.417) \frac{(121.75)}{102.09} = 2.29$$

$$R_B = \frac{1}{2.29} - \frac{1}{2(2.29)^2} (1 - e^{-2(2.29)}) = 0.437 - 0.0953(0.9897) = 0.343$$

$$\eta_L = 15.4 (0.417) \frac{(314.58)}{102.09} = 19.79$$

$$R_L = \frac{1}{19.79} - \frac{1}{2(19.79)^2} (1 - e^{-2(19.79)}) = 0.0505 - 0.00128 = 0.0492$$

$$R = \sqrt{\frac{1}{0.01} (0.0908)(0.252)(0.343)(0.53 + 0.47 (0.0492))} = 0.659$$

$$\varphi = \sqrt{\frac{1}{1 + 0.63 \left(\frac{121.75 + 180}{475.10}\right)^{0.63}}} = 0.82$$

$$G_f = 0.925 \left(\frac{1 + 1.7(0.246)\sqrt{3.4^2 (0.82)^2 + 3.976^2 (0.659)^2}}{1 + 1.7 (3.4)(0.246)} \right. = 0.994$$

North - south Direction (levels 3-12) - for G_f, refer to excel spreadsheet

East - west Direction (levels 3-12) - for G_f, refer to excel spreadsheet

Wind Analysis

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step 6: enclosure classification - fully enclosed

step 7: internal pressure coefficient → using Table 26.11-1, $GC_{p_i} = \pm 0.18$

step 8: velocity pressure exposure coefficients, K_z or K_h → using Table 27.3-1, see excel spreadsheet

step 9: velocity pressure q_z or q_h → from sect. 27.3.2, $q_z = 0.00256\, K_z K_{zt} K_d V^2$ (lb/ft²)

see excel spreadsheet for values

step 10: external pressure coefficient, C_p → using Fig. 27.4-1

wall pressure coefficients, C_p

windward wall: → all L/B values → $C_p = 0.80$

sidewall: → all L/B values → $C_p = -0.70$

Leeward wall: <u>levels 1-2</u>

<u>N-S wind</u>

$L/B = \frac{121.75}{314.58} = 0.39$, $0 < 0.39 < 1 \Rightarrow C_p = -0.50$

<u>E-W wind</u>

$L/B = \frac{314.58}{121.75} = 2.58$, $2 < 2.58 < 4 \Rightarrow C_p \rightarrow$ interpolate based on L/B values

$$C_p = \frac{2.58-2}{4-2}(-0.2+0.3) + (-0.3)$$

$$= -0.271$$

<u>levels 3-12</u>

<u>N-S wind</u>

$L/B = \frac{147}{314.58} = 0.47$, $0 < 0.47 < 1 \Rightarrow C_p = -0.50$

<u>E-W wind</u>

$L/B = \frac{314.58}{147} = 2.14$, $2 < 2.14 < 4 \rightarrow$ interpolate based on L/B values

$$C_p = \frac{2.14-2}{4-2}(-0.2+0.3) + (-0.3)$$

$$= -0.293$$

Wind Analysis

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Roof Pressures coefficients, C_p

$\theta = 0°$

N-S wind

$h/L = \frac{186}{147} = 1.22$, $1.22 \geq 1.0$

horizontal distance from windward edge : 0 to $h/2$ → 0' to 90' ⇒ $C_p = -1.3$

$> h/2$ → $> 90'$ ⇒ $C_p = -0.70$

E-W wind

$h/L = \frac{180}{314.58} = 0.57$, $0.5 < 0.57 < 1.0$

horizontal distance from windward edge : 0 to $h/2$ → 0' to 90' ⇒ C_p → interpolate based on h/L values

$$C_p = \frac{0.57 - 0.50}{1 - 0.5}(-1.3 + 0.9) + (-0.9)$$

$$= -0.956$$

$h/2$ to h → 90' to 180' ⇒ $C_p = \frac{0.57 - 0.50}{1 - 0.50}(-0.7 + 0.9) + (-0.9)$

$$= -0.872$$

h to $2h$ → 180' to 360' ⇒ $C_p = \frac{0.57 - 0.50}{1 - 0.50}(-0.7 + 0.5) + (-0.5)$

$$= -0.528$$

$> 2h$ → N/A

step 11: design wind pressures → from sect. 27.4.2 , $p = q G_f C_p - q_i (G C_{p_i})$ (lb/ft²)

windward walls → $p = \underbrace{q_z G_f C_p}_{\text{external pressure}} - \underbrace{q_h (G C_{p_i})}_{\text{internal pressure}}$

- see excel spreadsheet for pressures

leeward walls
side walls
roof

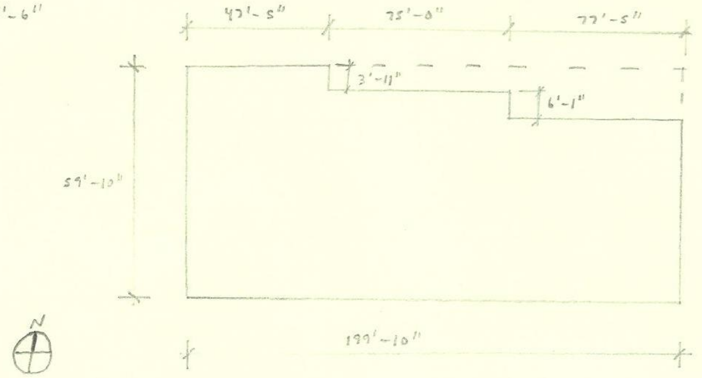
$\left.\right\} p = \underbrace{q_h G_f C_p}_{\text{external pressure}} - \underbrace{q_h (G C_{p_i})}_{\text{internal pressure}}$

note: internal wind pressures were neglected in calculating the design wind pressures because the internal pressures of the same magnitude will cancel and thus do not contribute towards the external wind pressures acting on the building

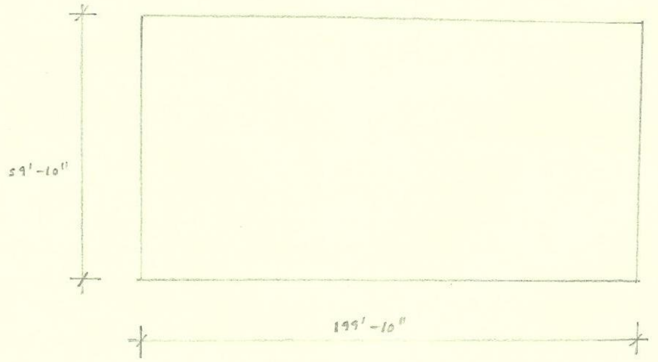
Mechanical Penthouse

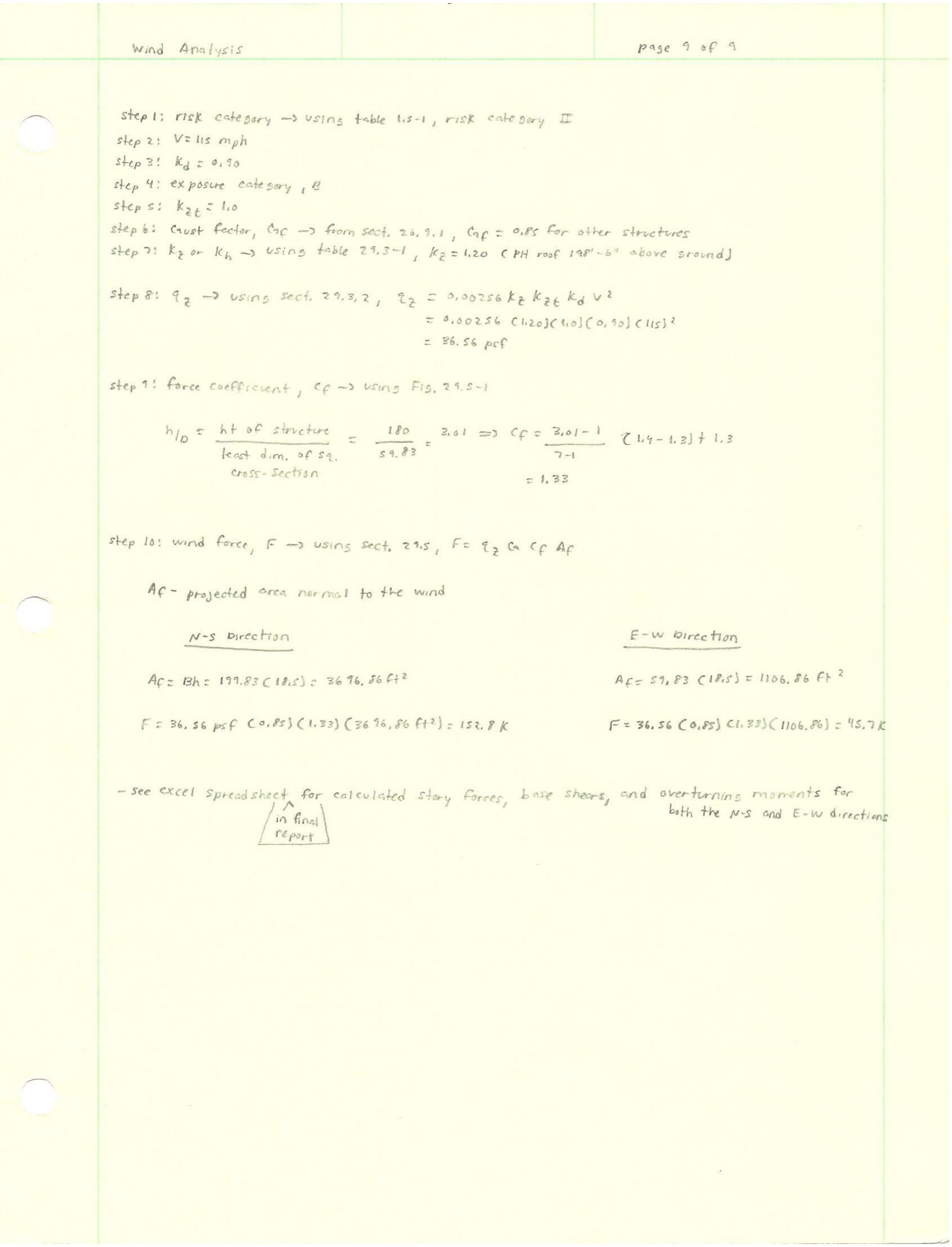
Assumption: mech. Penthouse is not a part of the MWFRS
- the penthouse will be analyzed as a separate structure and therefore the guidelines listed in chapter 29 (wind loads on other structures and building appurtenances - MWFRS) will be followed for analysis

h = 18'-6"



simplified assumptions: Change penthouse geometry to rectangular for calculations





Wind Analysis

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step 1: risk category → using table 1.5-1, risk category II

step 2: $V = 115$ mph

step 3: $K_d = 0.90$

step 4: exposure category, B

step 5: $K_{zt} = 1.0$

step 6: Gust factor, G_f → from sect. 26.9.1, $G_f = 0.85$ for other structures

step 7: K_z or K_h → using table 29.3-1, $K_z = 1.20$ (PH roof 198'-6" above ground)

step 8: q_z → using sect. 29.3.2,

$$q_z = 0.00256\, K_z K_{zt} K_d V^2$$
$$= 0.00256\,(1.20)(1.0)(0.90)(115)^2$$
$$= 36.56 \text{ psf}$$

step 9: force coefficient, C_f → using Fig. 29.5-1

$$h/D = \frac{\text{ht of structure}}{\text{least dim. of sq. cross-section}} = \frac{180}{59.83} = 3.01 \Rightarrow C_f = \frac{3.01-1}{7-1}(1.4-1.3)+1.3$$
$$= 1.33$$

step 10: wind force, F → using sect. 29.5, $F = q_z G C_f A_f$

A_f - projected area normal to the wind

N-S Direction

$A_f = Bh = 199.83\,(18.5) = 3696.86 \text{ ft}^2$

$F = 36.56 \text{ psf}\,(0.85)(1.33)(3696.86 \text{ ft}^2) = 152.8 \text{ K}$

E-W Direction

$A_f = 59.83\,(18.5) = 1106.86 \text{ ft}^2$

$F = 36.56\,(0.85)(1.33)(1106.86) = 45.7 \text{ K}$

- see excel spreadsheet ^in final report for calculated story forces, base shears, and overturning moments for both the N-S and E-W directions

Building Effective Length, L_{eff}							
N-S Direction				E-W Direction			
Level	h_i (ft) (Height above grade of level i)	L_i (ft) (Building Length at level i)	h_i*L_i	Level	h_i (ft) (Height above grade of level i)	L_i (ft) (Building Length at level i)	h_i*L_i
1	0	121.75	0	1	0	314.58	0
2	15	121.75	1826.25	2	15	314.58	4718.7
3	30	147	4410	3	30	314.58	9437.4
4	45	147	6615	4	45	314.58	14156.1
5	60	147	8820	5	60	314.58	18874.8
6	75	147	11025	6	75	314.58	23593.5
7	90	147	13230	7	90	314.58	28312.2
8	105	147	15435	8	105	314.58	33030.9
9	120	147	17640	9	120	314.58	37749.6
10	135	147	19845	10	135	314.58	42468.3
11	150	147	22050	11	150	314.58	47187
12	165	147	24255	12	165	314.58	51905.7
Main roof	180	147	26460	Main roof	180	314.58	56624.4
∑=	1170		171611.25	∑=	1170		368058.6
L_{eff}=	146.68			L_{eff}=	314.58		

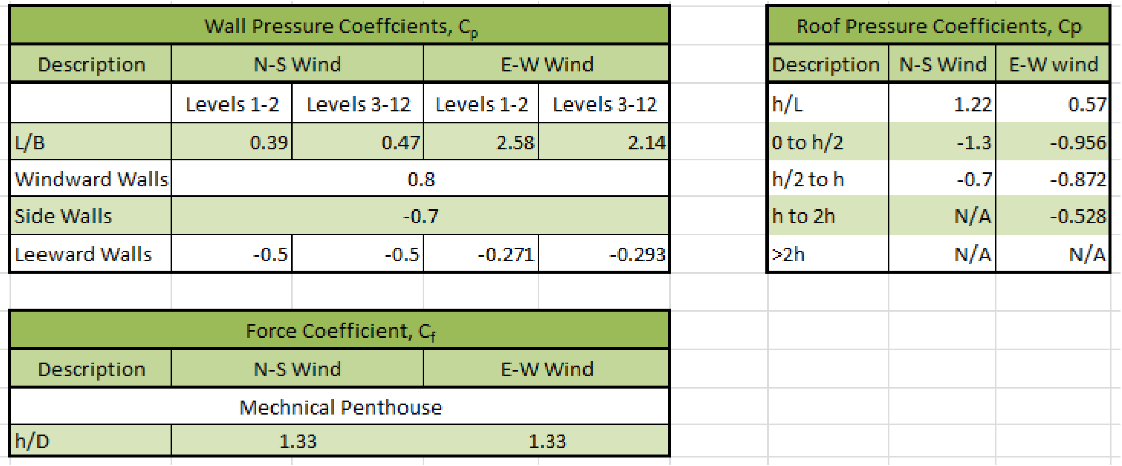
Building Effective Length, L_{eff}, used to determine the natural frequency of the building according to Sect. 26.9 of ASCE 7-10

Velocity Pressure Coefficients, K_z, and Velocity Pressures, q_z			
Level	Elevation (ft)	K_z	q_z
1	0	0.57	16.40
2	15	0.57	16.40
3	30	0.70	20.14
4	45	0.81	23.31
5	60	0.85	24.46
6	75	0.93	26.76
7	90	0.96	27.63
8	105	1.04	29.93
9	120	1.04	29.93
10	135	1.09	31.37
11	150	1.13	32.52
12	165	1.17	33.67
Main Roof	180	1.17	33.67
PH Roof	198.5	1.2	36.56

Gust Factor-MWFRS				
Variable	N-S Wind		E-W Wind	
$n_1=n_a$	0.417			
$g_Q=g_v$	3.4			
g_R	3.976			
Z_{mean}	108			
$I_{z, mean}$	0.246			
$L_{z, mean}$	475.1			
$v_{z,mean}$	102.1			
N_1	1.94			
R_n	0.0908			
β	0.01			
η_h	3.38			
R_h	0.252			
	Levels 1-2	Levels 3-12	Levels 1-2	Levels 3-12
η_B	5.91	5.91	2.29	2.76
R_B	0.155	0.155	0.343	0.297
η_L	7.66	9.25	19.79	19.79
R_L	0.122	0.102	0.0492	0.0493
R	0.456	0.453	0.659	0.613
Q	0.78	0.78	0.82	0.817
G_f	0.895	0.894	0.994	0.972

Gust Factor-Mechnical Penthouse		
Variable	N-S Wind	E-W Wind
G_f	0.85	0.85

Velocity pressure and gust factors for the MWFRS



Wall Pressure Coeffcients, C_p

Description	N-S Wind		E-W Wind	
	Levels 1-2	Levels 3-12	Levels 1-2	Levels 3-12
L/B	0.39	0.47	2.58	2.14
Windward Walls	0.8			
Side Walls	-0.7			
Leeward Walls	-0.5	-0.5	-0.271	-0.293

Roof Pressure Coefficients, Cp

Description	N-S Wind	E-W wind
h/L	1.22	0.57
0 to h/2	-1.3	-0.956
h/2 to h	-0.7	-0.872
h to 2h	N/A	-0.528
>2h	N/A	N/A

Force Coefficient, C_f

Description	N-S Wind	E-W Wind
Mechnical Penthouse		
h/D	1.33	1.33

Pressure coefficients

Appendix C: Seismic Load Calculations

Existing Concrete System Seismic Load Analysis

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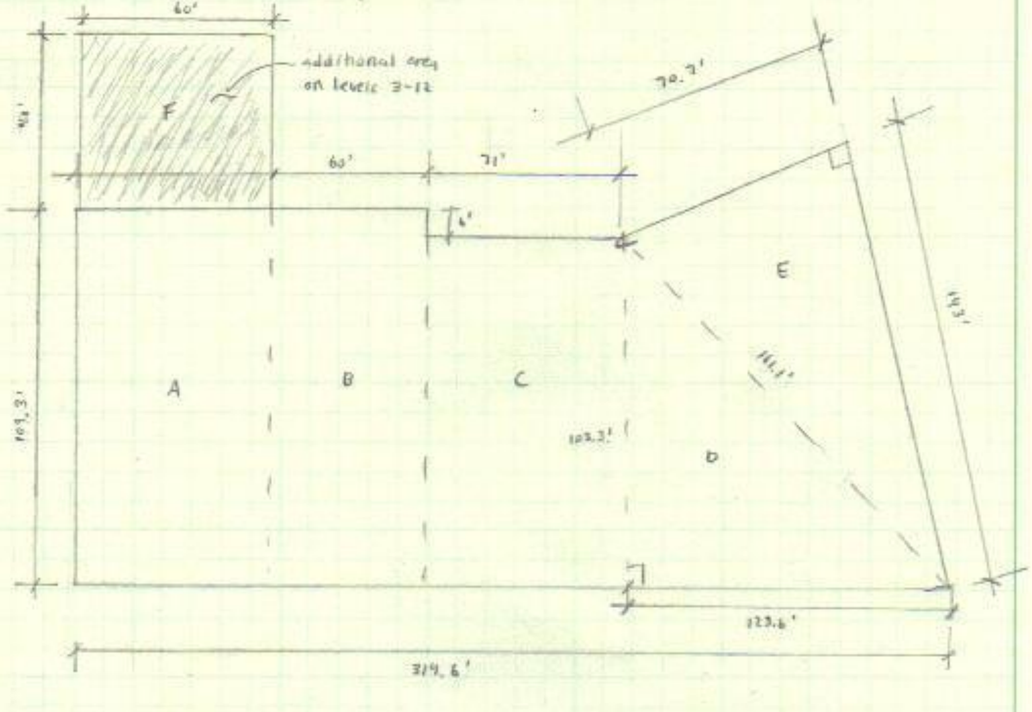
simplified building geometry for seismic load analysis



Area A = 60' (109.3') = 6558 ft²

Area B = 60' (109.3') = 6558 ft²

Area C = 71' (103.3') = 7334.3 ft²

Area D = $\frac{1}{2}$ (123.6') (103.3) = 6383.9 ft²

Area E = $\frac{1}{2}$ (143') (70.7') = 5055.1 ft²

Area F = 60 (40) = 2400 ft²

building perimeter (levels 1-2) = 834.6 ft

building perimeter (levels 3-12, main roof) = 919.6 ft

floor areas for levels 1-2 = $A_A + A_B + A_C + A_D + A_E$ = 6558 (2) + 7334.3 + 6383.9 + 5055.1
= 31, 889 ft²

floor area for levels 3-12 = 31, 889 + A_F = 31, 889 + 2400 = 34, 289 ft²

main roof area = 34, 289 ft²

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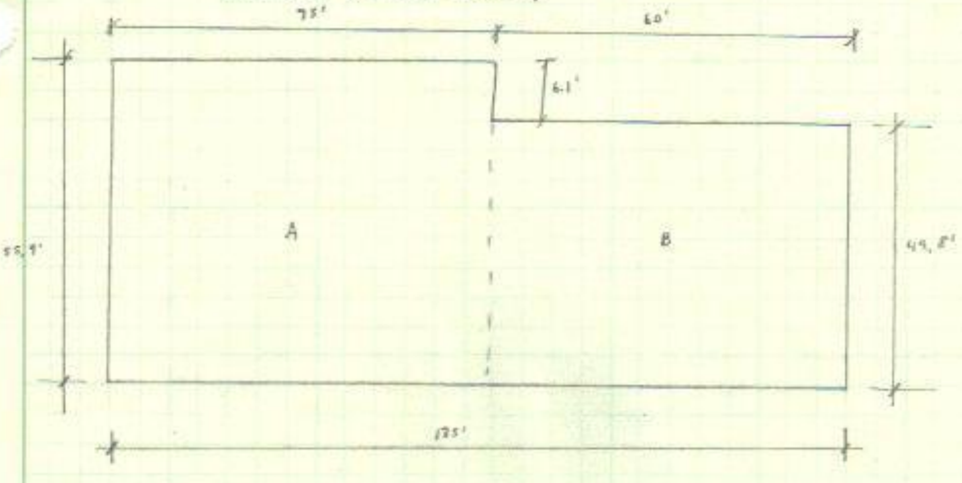
Mechanical Penthouse geometry



Area A = 75'(55.9') = 4192.5 ft²

Area B = 49.8'(60') = 2988 ft²

total Mech. PH area = $A_A + A_B$ = 4192.5+2988 = 7180.5

Mech. PH perimeter = 381.8 ft

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Building weight:

- neglect 1st level weight because level 1 will not contribute towards resisting the seismic loads
- for calculation simplicity, slab openings due to stairways and elevator shafts were neglected, therefore resulting in a more conservative calculation

2nd level

Dead loads:

8" thick normal wt concrete = 150 pcf × $\frac{8}{12}$ ft = 100 psf

SDL = 10 psf

curtain wall = 250 plf

column wt = 488 K

drop panel wt = 248 K

W_{2nd} level = (100 + 10) psf (31, 289 ft²) + 250 plf (834.6 ft) + 488 K + 248 K = 4452 K

levels 3 - 12

- dead loads are the same as on level 2

W = (110 psf)(34, 289 ft²) + 250 plf (714.6 ft) + 488 K + 248 K = 4736 K/flr (10 flrs) = 47364 K

- main roof

Dead loads;

8" thick normal wt concrete slab = 100 psf

SDL = 10 psf

curtain wall = 250 plf

$W_{main\ roof}$ = (110 psf)(34, 289 ft²) + 250 plf (714.6 ft) = 4000 K

Mechanical Penthouse roof

Dead loads:

8" slab = 100 psf

SDL = 5 psf

$W_{PH\ roof}$ = (105) psf (7181 ft²) = 754 K

Total building dead load = 4452 + 47364 + 4000 + 754 = 56, 570 K

Seismic analysis | Tech 1 | page 2 of 5

step1: site class → given in geotechnical report, "C" (very dense soil and hard rock, from table 20.3-1)

step2: spectral response acceleration

at short periods, S_S → from Fig. 22-1, $S_S = 0.20$

at 1-second period, S_1 → from Fig. 22-2, $S_1 = 0.06$

step3: site coefficients and adjusted maximum considered E.Q. spectral response acceleration parameters

from table 11.4-1

$S_{MS} = F_a S_S$

with $S_S < 0.25$ and site class C

$F_a = 1.2 \Rightarrow S_{MS} = 1.2(0.20) = 0.24$

from table 11.4-2

$S_{M1} = F_v S_1$

with $S_1 < 0.1$ and site class C

$F_v = 1.7 \Rightarrow S_{M1} = 1.7(0.06) = 0.102$

step4: Design spectral response acceleration parameters at short periods, S_{DS}, and at 1-sec. period, S_{D1}, for 5% damp ⇒ from sect. 11.4.4

$S_{DS} = 2/3\ S_{MS} = 2/3\ (0.24) = 0.16$

$S_{D1} = 2/3\ S_{M1} = 2/3\ (0.102) = 0.068$

step5: occupancy category and importance factors

occupancy category II → from table 1.5-1

importance factor, I → from table 1.5.2, $I = 1.0$

step6: seismic design category, SDC

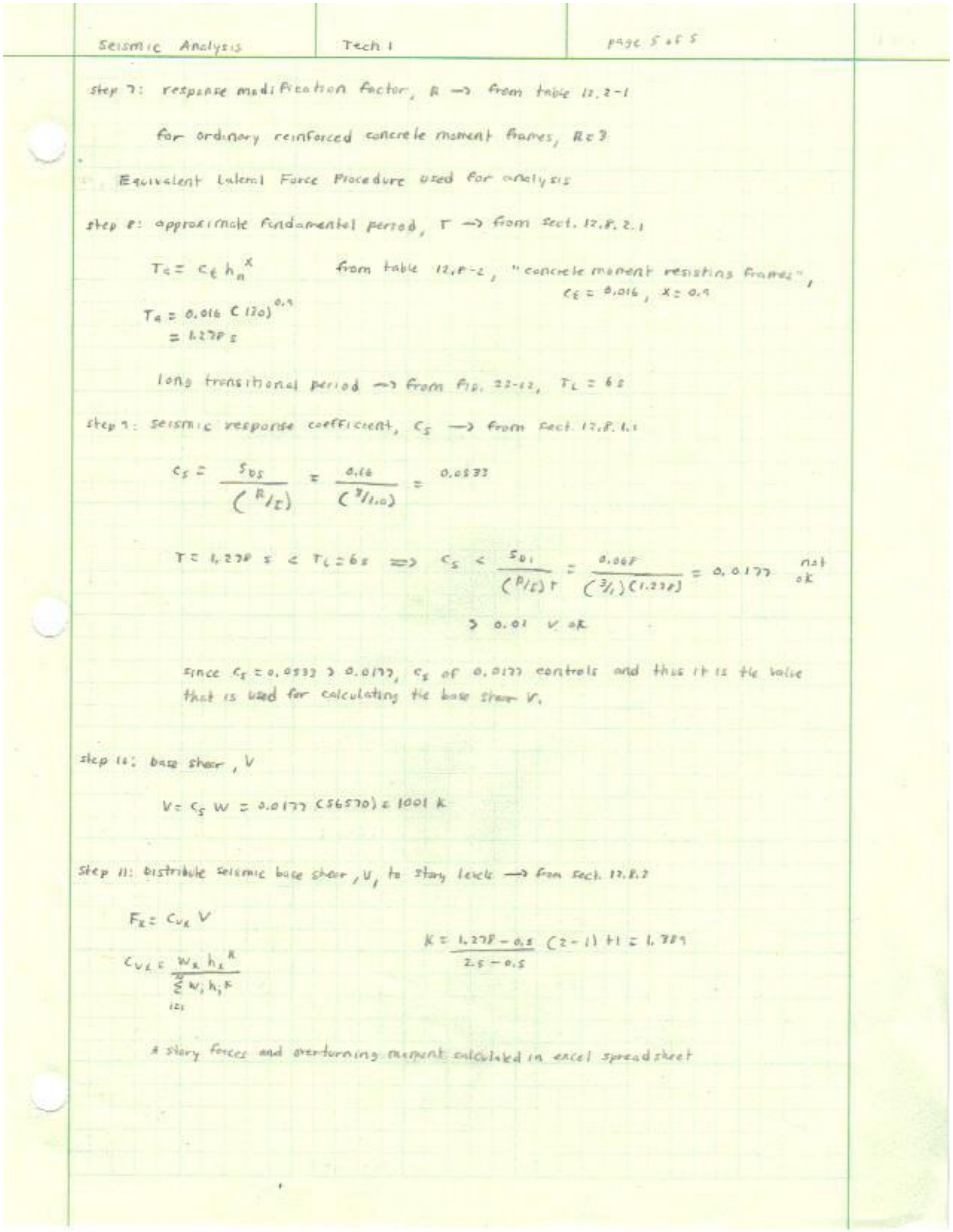
SDC based on short period response acceleration parameter → from table 11.6-1

for $S_{DS} = 0.16$ and occ. II → SDC = "A"

SDC based on 1-sec. response acceleration parameter

for $S_{D1} = 0.068$ and occ. II → SDC = "B"

since risk category B is more severe than risk category A,
use SDC = "B"



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step 7: response modification factor, R → from table 12.2-1

for ordinary reinforced concrete moment frames, $R = 3$

Equivalent Lateral Force Procedure used for analysis

step 8: approximate fundamental period, T → from sect. 12.8.2.1

$T_a = C_t h_n^{x}$ from table 12.8-2, "concrete moment resisting frames", $C_t = 0.016$, $x = 0.9$

$T_a = 0.016\ (130)^{0.9}$
$= 1.278\ s$

long transitional period → from fig. 22-12, $T_L = 6s$

step 9: seismic response coefficient, C_s → from sect. 12.8.1.1

$$C_s = \frac{S_{DS}}{(R/I)} = \frac{0.16}{(3/1.0)} = 0.0533$$

$$T = 1.278\ s < T_L = 6s \implies C_s < \frac{S_{D1}}{(R/I)T} = \frac{0.068}{(3/1)(1.278)} = 0.0177 \quad \text{not ok}$$

> 0.01 ✓ ok

since $C_s = 0.0533 > 0.0177$, C_s of 0.0177 controls and thus it is the value that is used for calculating the base shear V.

step 10: base shear, V

$V = C_s W = 0.0177\ (56570) = 1001\ k$

step 11: distribute seismic base shear, V, to story levels → from sect. 12.8.3

$F_x = C_{vx} V$

$$C_{vx} = \frac{w_x h_x^{k}}{\sum_{i=1}^{n} w_i h_i^{k}}$$

$$k = \frac{1.278 - 0.5}{2.5 - 0.5}(2-1) + 1 = 1.389$$

* story forces and overturning moment calculated in excel spreadsheet

Existing Superstructure Weight

Floor Weight Calculations						
Floor	Area (ft^2)	Perimeter (ft)	8" slab weight (psf)	Superimposed DL (psf)	Curtain Wall Weight (plf)	Total Weight (Kips)
2	31889.00	834.6	100	10	250.00	3716
3	34289.00	914.6	100	10	250.00	4000
4	34289.00	914.6	100	10	250.00	4000
5	34289.00	914.6	100	10	250.00	4000
6	34289.00	914.6	100	10	250.00	4000
7	34289.00	914.6	100	10	250.00	4000
8	34289.00	914.6	100	10	250.00	4000
9	34289.00	914.6	100	10	250.00	4000
10	34289.00	914.6	100	10	250.00	4000
11	34289.00	914.6	100	10	250.00	4000
12	34289.00	914.6	100	10	250.00	4000
Main Roof	34289.00	914.6	100	10	250.00	4000
PH Roof	7181.00	381.8	100	5	N/A	754
					Total Floor Weight=	48475

Typical Floor Column Weight (4th Level)					
Size (in x in)	Quantity	Length Clear Span(ft)	Unit Weight (lbs/ft^3)	Volume (ft^3)	Weight (Kips)
18X36	4	9.96	150	179.25	26.89
16X32	7	9.96	150	247.85	37.18
18X28	1	9.96	150	34.85	5.23
18X42	3	9.96	150	156.84	23.53
24X30	6	9.96	150	298.75	44.81
24X24	10	9.96	150	398.33	59.75
24 dia	12	9.96	150	375.2	56.28
16X48	2	9.96	150	106.22	15.93
16X66	1	9.96	150	73.03	10.95
14X48	2	9.96	150	92.94	13.94
12X24	5	9.96	150	99.58	14.94
22X26	1	9.96	150	39.56	5.93
20.5X24	1	9.96	150	34.02	5.10
22X24	1	9.96	150	36.51	5.48
14X96	1	9.96	150	92.94	13.94
24X36	2	9.96	150	119.50	17.93
28X28	3	9.96	150	162.65	24.40
12X48	1	9.96	150	39.83	5.98
11X24	4	9.96	150	73.03	10.95
16X96	1	9.96	150	106.22	15.93
14X66	1	9.96	150	63.90	9.58
28 dia	8	9.96	150	340.5	51.07
18X64	1	9.96	150	79.67	11.95
		Column Weight per floor (11 total flrs)=			488
		Total Column Weight=			5365
Typical Floor Drop Panel Weight (4th Level)					
Size (in x in)	Quantity	Thickness (in)	Unit Weight (lbs/ft^3)	Volume (ft^3)	Weight (Kips)
36 wide	Continuous Drop (around perimeter of all floors)	3.50	150	800.28	120
min. 68X68	40	8.00	150	856.3	128
		Drop Panel Weight per floor (11 total flrs)=			248
		Total Drop Panel Weight=			2733

New Steel System Seismic Load Analysis

Seismic Analysis

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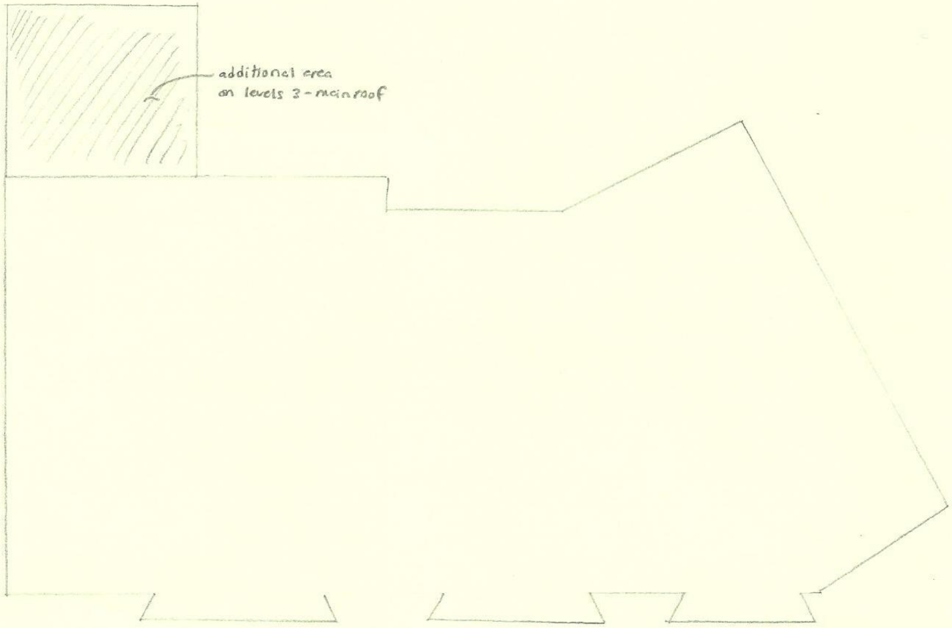


stairwell opening area = 2(10.67 ft)(30 ft) = 640 ft^2

↑ # of stairwells

elevator opening area = 7(8 ft)(10 ft) = 560 ft^2

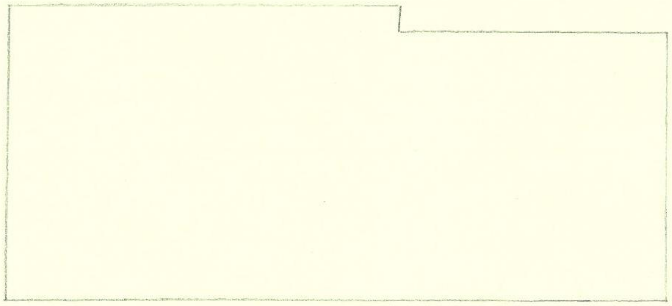
↑ # of elevators

floor area for levels 3-mainroof = 32,632 ft^2 - 640 ft^2 - 560 ft^2 = 31,432 ft^2

building perimeter (levels 3-main roof) = 929.33 ft

floor area for levels 1-2 = 30232 ft^2 - 640 ft^2 - 560 ft^2 = 29,032 ft^2

building perimeter = 849.3 ft



Mech. PH. area = 7180.5 ft^2

Mech PH perimeter = 381.8 ft

MECH. PH. Geometry

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Estimated
Building weight:

- neglect 1st level weight because level will not contribute towards resisting the seismic loads
- for design purposes, assume member weight (including beams, girders, and columns) will be 15 psf allowance

2nd level

dead loads:
- 3VL120 composite deck w/ 7½" slab depth = 75 psf
- member wt allow. = 15 psf
- curtain wall = 250 plf
- SDL = 10 psf

$W_{2nd\ level}$ = (75 + 15 + 10) psf (29,032 ft²) + 250 plf (849.3 ft) = 3116 K

levels 3-12

- dead loads are the same as on level 2

w = (100 psf)(31, 432 ft²) + 250 plf (929.33 ft) = 3376 K

w = 3376 K/flr (10 flrs) = 33760 K

main roof

- dead loads are the same as on level 2

$W_{main\ roof}$ = 3376 K

mechanical Penthouse roof

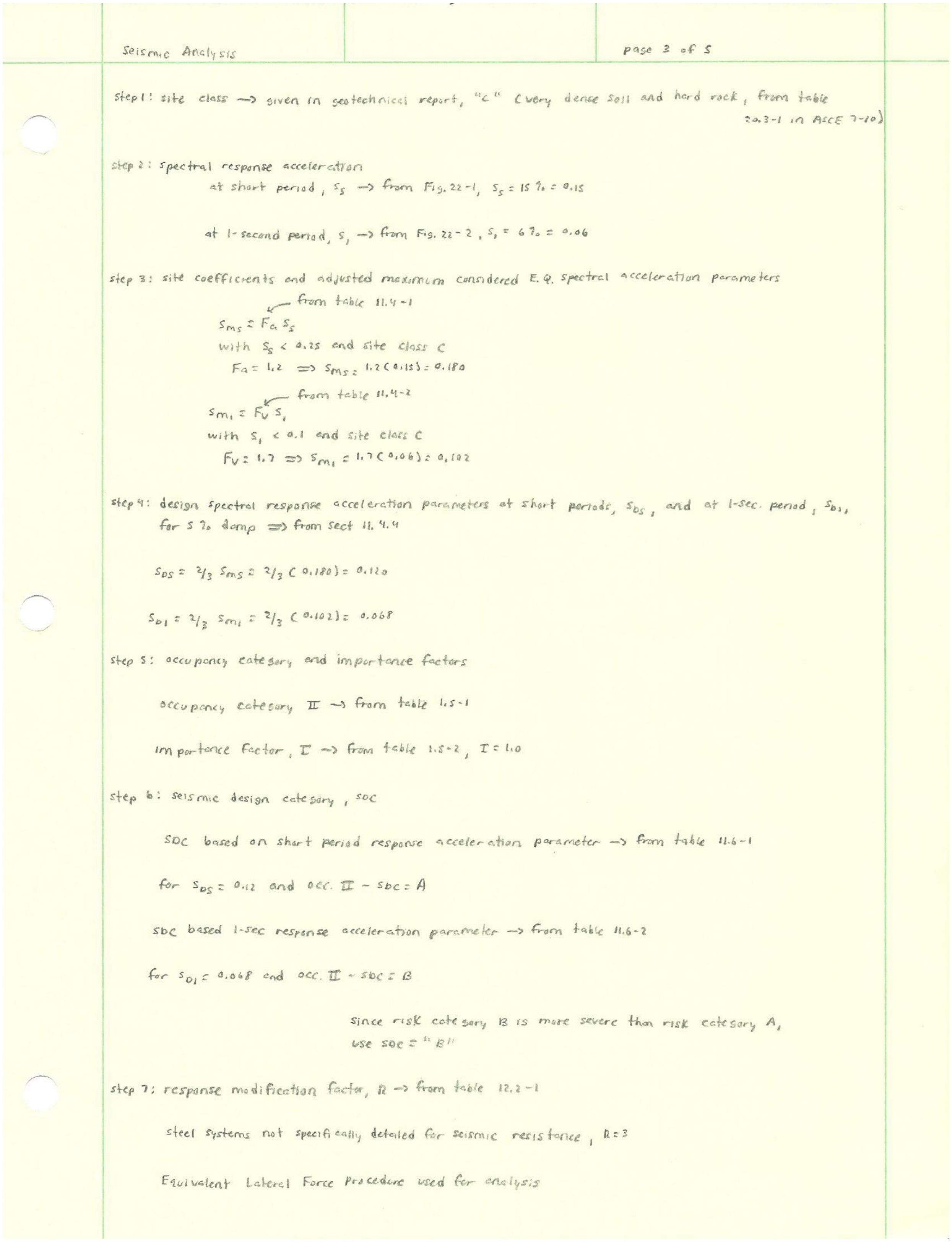
dead loads:
- 3VL120 composite deck w/ 7½" slab depth = 75 psf
- SDL = 5 psf

$W_{PH\ roof}$ = (75 + 5) psf (7180.5 ft²) = 574 K

estimated total building dead load = 3116 + 33760 + 3376 + 574 = 40,826 K

- refer to excel spreadsheets for actual building weight

(after the lateral analysis, it was found that the wind load case 1 controlled the lateral system design and after designing the lateral system the actual total building wt. was calculated)



Seismic Analysis		page 3 of 5

step 1: site class → given in geotechnical report, "C" (very dense soil and hard rock, from table 20.3-1 in ASCE 7-10)

step 2: spectral response acceleration

at short period, S_S → from Fig. 22-1, $S_S = 15\% = 0.15$

at 1-second period, S_1 → from Fig. 22-2, $S_1 = 6\% = 0.06$

step 3: site coefficients and adjusted maximum considered E.Q. spectral acceleration parameters

from table 11.4-1

$S_{MS} = F_a S_S$

with $S_S < 0.25$ and site class C

$F_a = 1.2 \Rightarrow S_{MS} = 1.2(0.15) = 0.180$

from table 11.4-2

$S_{M1} = F_v S_1$

with $S_1 < 0.1$ and site class C

$F_v = 1.7 \Rightarrow S_{M1} = 1.7(0.06) = 0,102$

step 4: design spectral response acceleration parameters at short periods, S_{DS}, and at 1-sec. period, S_{D1}, for 5% damp ⇒ from sect 11.4.4

$S_{DS} = 2/3\, S_{MS} = 2/3\,(0.180) = 0.120$

$S_{D1} = 2/3\, S_{M1} = 2/3\,(0.102) = 0.068$

step 5: occupancy category and importance factors

occupancy category II → from table 1.5-1

importance factor, I → from table 1.5-2, $I = 1.0$

step 6: seismic design category, SDC

SDC based on short period response acceleration parameter → from table 11.6-1

for $S_{DS} = 0.12$ and occ. II – SDC = A

SDC based 1-sec response acceleration parameter → from table 11.6-2

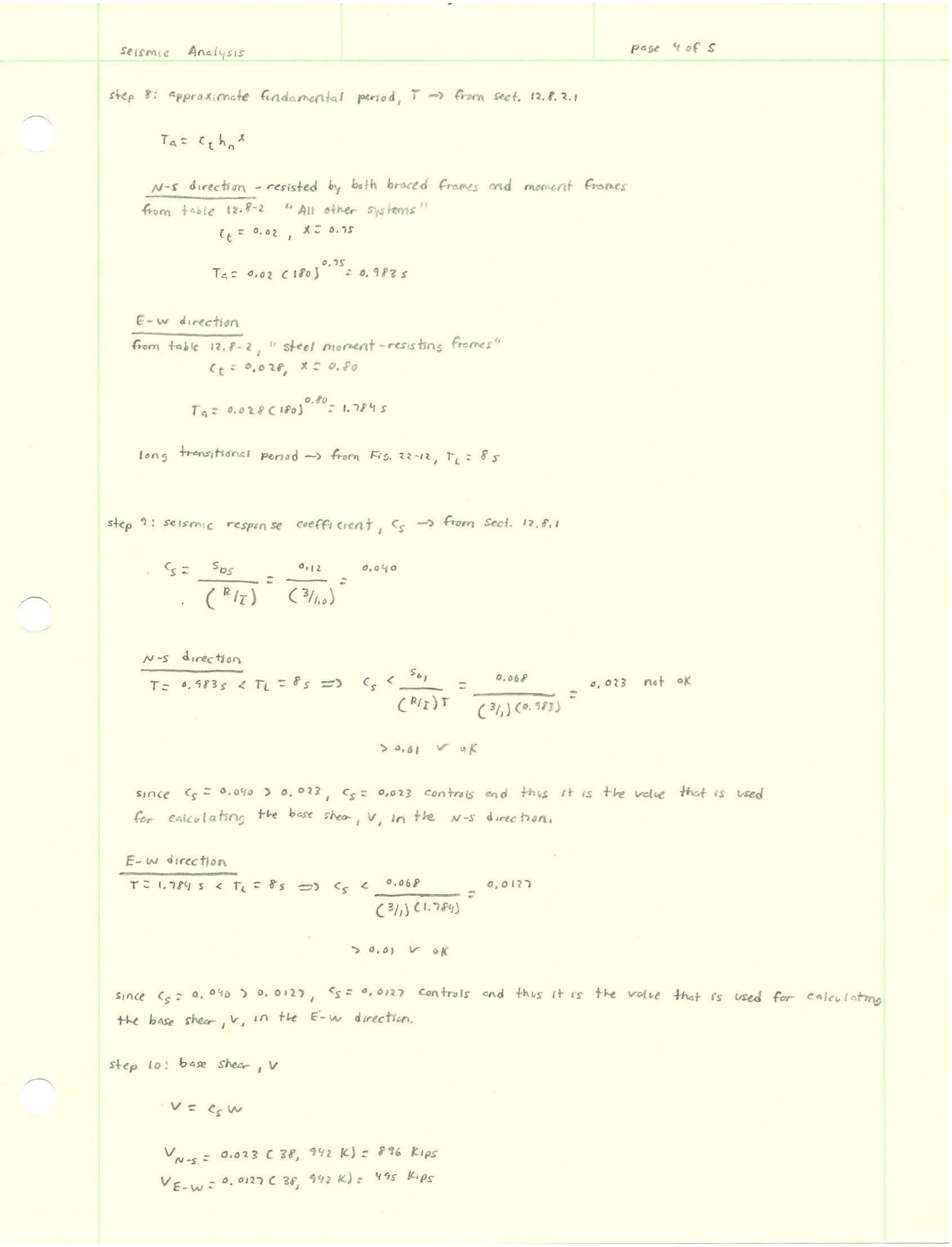
for $S_{D1} = 0.068$ and occ. II – SDC = B

since risk category B is more severe than risk category A,
use SDC = "B"

step 7: response modification factor, R → from table 12.2-1

steel systems not specifically detailed for seismic resistance, $R = 3$

Equivalent Lateral Force Procedure used for analysis



Seismic Analysis

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step 8: approximate fundamental period, T → from sect. 12.8.2.1

$T_a = C_t h_n^x$

N-S direction – resisted by both braced frames and moment frames

from table 12.8-2 "All other systems"

$C_t = 0.02$, $x = 0.75$

$T_a = 0.02\,(180)^{0.75} = 0.983\,s$

E-W direction

from table 12.8-2, "steel moment-resisting frames"

$C_t = 0.028$, $x = 0.80$

$T_a = 0.028\,(180)^{0.80} = 1.784\,s$

long transitional period → from Fig. 22-12, $T_L = 8\,s$

step 9: seismic response coefficient, C_s → from Sect. 12.8.1

$$C_s = \frac{S_{DS}}{(R/I)} = \frac{0.12}{(3/1.0)} = 0.040$$

N-S direction

$$T = 0.983\,s < T_L = 8\,s \Rightarrow C_s < \frac{S_{D1}}{(R/I)T} = \frac{0.068}{(3/1)(0.983)} = 0.023 \text{ not ok}$$

> 0.01 ✓ ok

since $C_s = 0.040 > 0.023$, $C_s = 0.023$ controls and thus it is the value that is used for calculating the base shear, V, in the N-S direction.

E-W direction

$$T = 1.784\,s < T_L = 8\,s \Rightarrow C_s < \frac{0.068}{(3/1)(1.784)} = 0.0127$$

> 0.01 ✓ ok

since $C_s = 0.040 > 0.0127$, $C_s = 0.0127$ controls and thus it is the value that is used for calculating the base shear, V, in the E-W direction.

step 10: base shear, V

$V = C_s W$

$V_{N-S} = 0.023\,(38,942\,K) = 896$ Kips

$V_{E-W} = 0.0127\,(38,942\,K) = 495$ Kips

Seismic Analysis

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step 11: distribute seismic base shear, V, to story levels -> sect. 12.8.3

$F_x = C_{vx} V$

$$C_{vx} = \frac{w_x h_x^{k}}{\sum_{i=1}^{N} w_i h_i^{k}}$$

N-S direction

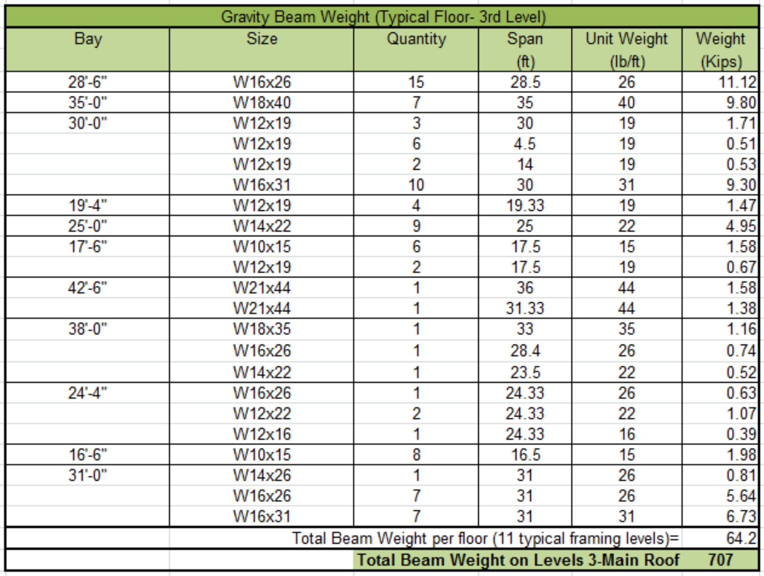
$$k = \frac{0.983 - 0.5}{2.5 - 0.5}(2-1) + 1 = 1.246$$

E-W direction

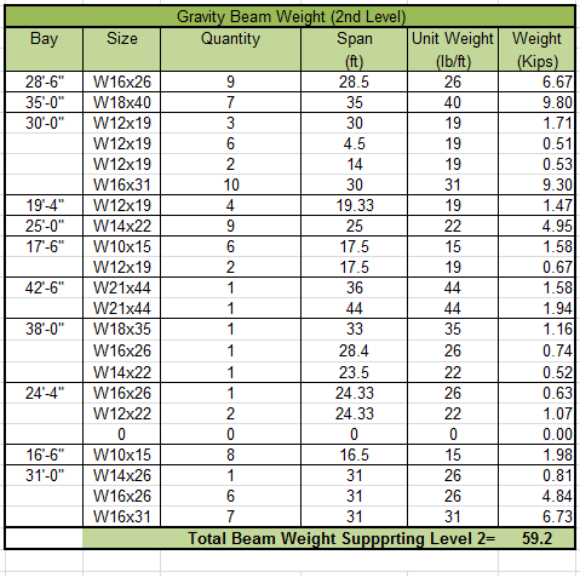
$$K = \frac{1.784 - 0.5}{2.5 - 0.5}(2-1) + 1 = 1.642$$

- refer to the excel spreadsheets in the report for the base shears, story forces, end overturning moments in the N-S and E-W directions

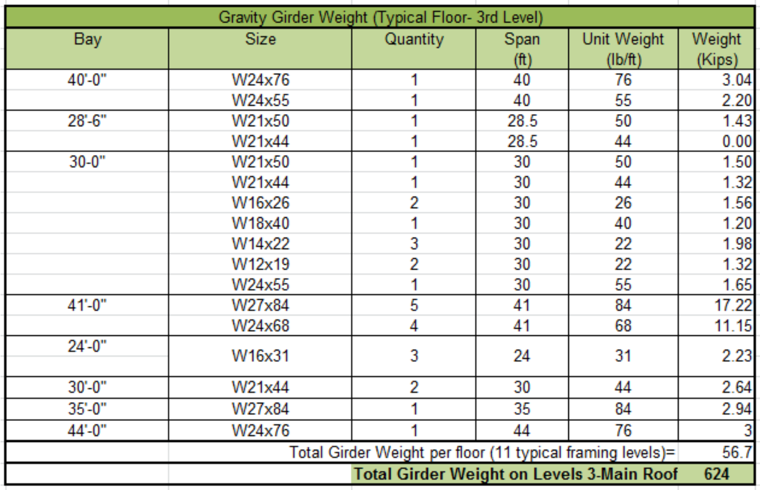
New Steel Superstructure Weight



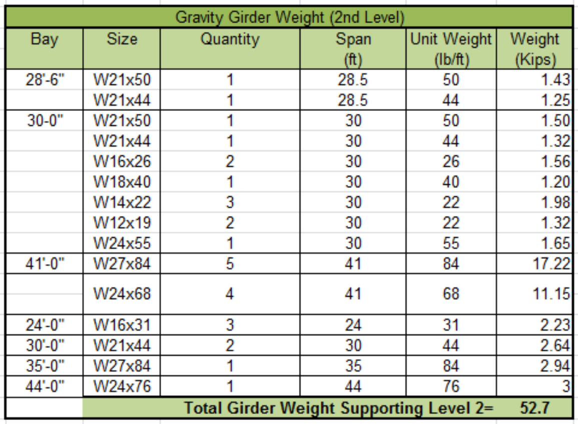
Gravity Beam Weight (Typical Floor- 3rd Level)					
Bay	Size	Quantity	Span (ft)	Unit Weight (lb/ft)	Weight (Kips)
28'-6"	W16x26	15	28.5	26	11.12
35'-0"	W18x40	7	35	40	9.80
30'-0"	W12x19	3	30	19	1.71
	W12x19	6	4.5	19	0.51
	W12x19	2	14	19	0.53
	W16x31	10	30	31	9.30
19'-4"	W12x19	4	19.33	19	1.47
25'-0"	W14x22	9	25	22	4.95
17'-6"	W10x15	6	17.5	15	1.58
	W12x19	2	17.5	19	0.67
42'-6"	W21x44	1	36	44	1.58
	W21x44	1	31.33	44	1.38
38'-0"	W18x35	1	33	35	1.16
	W16x26	1	28.4	26	0.74
	W14x22	1	23.5	22	0.52
24'-4"	W16x26	1	24.33	26	0.63
	W12x22	2	24.33	22	1.07
	W12x16	1	24.33	16	0.39
16'-6"	W10x15	8	16.5	15	1.98
31'-0"	W14x26	1	31	26	0.81
	W16x26	7	31	26	5.64
	W16x31	7	31	31	6.73
	Total Beam Weight per floor (11 typical framing levels)=				64.2
		Total Beam Weight on Levels 3-Main Roof			**707**



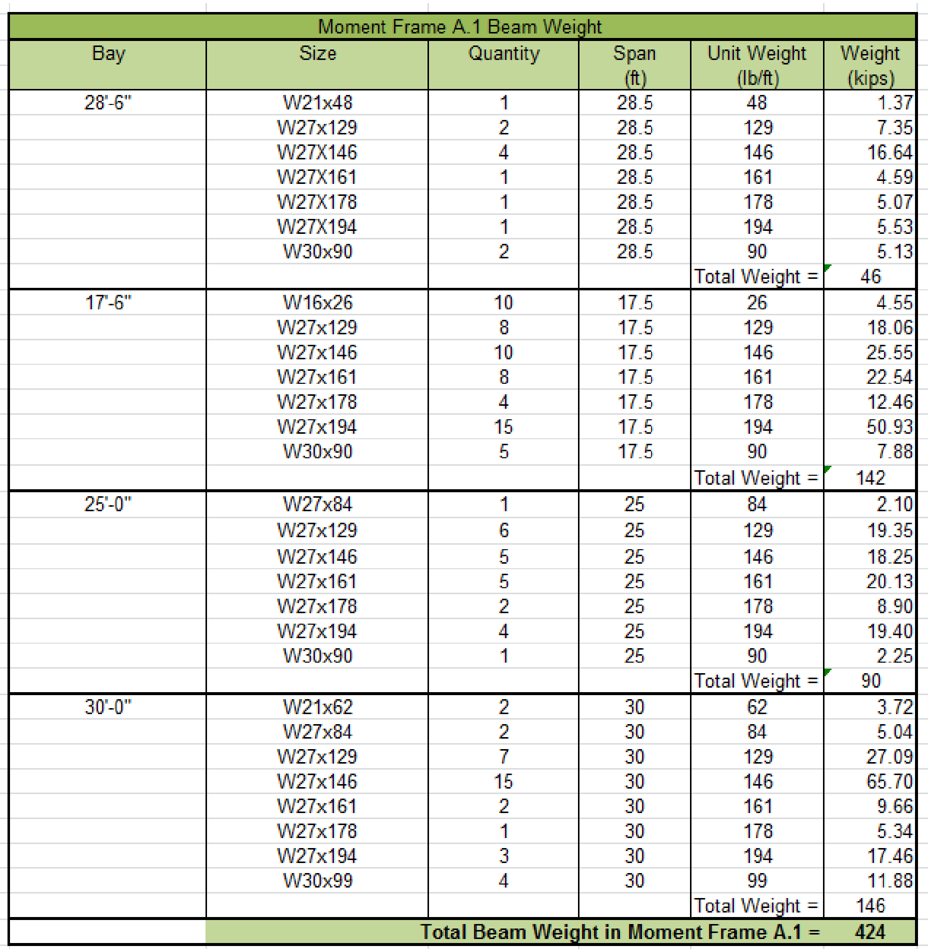
Gravity Beam Weight (2nd Level)					
Bay	Size	Quantity	Span (ft)	Unit Weight (lb/ft)	Weight (Kips)
28'-6"	W16x26	9	28.5	26	6.67
35'-0"	W18x40	7	35	40	9.80
30'-0"	W12x19	3	30	19	1.71
	W12x19	6	4.5	19	0.51
	W12x19	2	14	19	0.53
	W16x31	10	30	31	9.30
19'-4"	W12x19	4	19.33	19	1.47
25'-0"	W14x22	9	25	22	4.95
17'-6"	W10x15	6	17.5	15	1.58
	W12x19	2	17.5	19	0.67
42'-6"	W21x44	1	36	44	1.58
	W21x44	1	44	44	1.94
38'-0"	W18x35	1	33	35	1.16
	W16x26	1	28.4	26	0.74
	W14x22	1	23.5	22	0.52
24'-4"	W16x26	1	24.33	26	0.63
	W12x22	2	24.33	22	1.07
	0	0	0	0	0.00
16'-6"	W10x15	8	16.5	15	1.98
31'-0"	W14x26	1	31	26	0.81
	W16x26	6	31	26	4.84
	W16x31	7	31	31	6.73
	Total Beam Weight Suppprting Level 2=				**59.2**



Gravity Girder Weight (Typical Floor- 3rd Level)					
Bay	Size	Quantity	Span (ft)	Unit Weight (lb/ft)	Weight (Kips)
40'-0"	W24x76	1	40	76	3.04
	W24x55	1	40	55	2.20
28'-6"	W21x50	1	28.5	50	1.43
	W21x44	1	28.5	44	0.00
30-0"	W21x50	1	30	50	1.50
	W21x44	1	30	44	1.32
	W16x26	2	30	26	1.56
	W18x40	1	30	40	1.20
	W14x22	3	30	22	1.98
	W12x19	2	30	22	1.32
	W24x55	1	30	55	1.65
41'-0"	W27x84	5	41	84	17.22
	W24x68	4	41	68	11.15
24'-0"	W16x31	3	24	31	2.23
30'-0"	W21x44	2	30	44	2.64
35'-0"	W27x84	1	35	84	2.94
44'-0"	W24x76	1	44	76	3
	Total Girder Weight per floor (11 typical framing levels)=				56.7
		Total Girder Weight on Levels 3-Main Roof			**624**

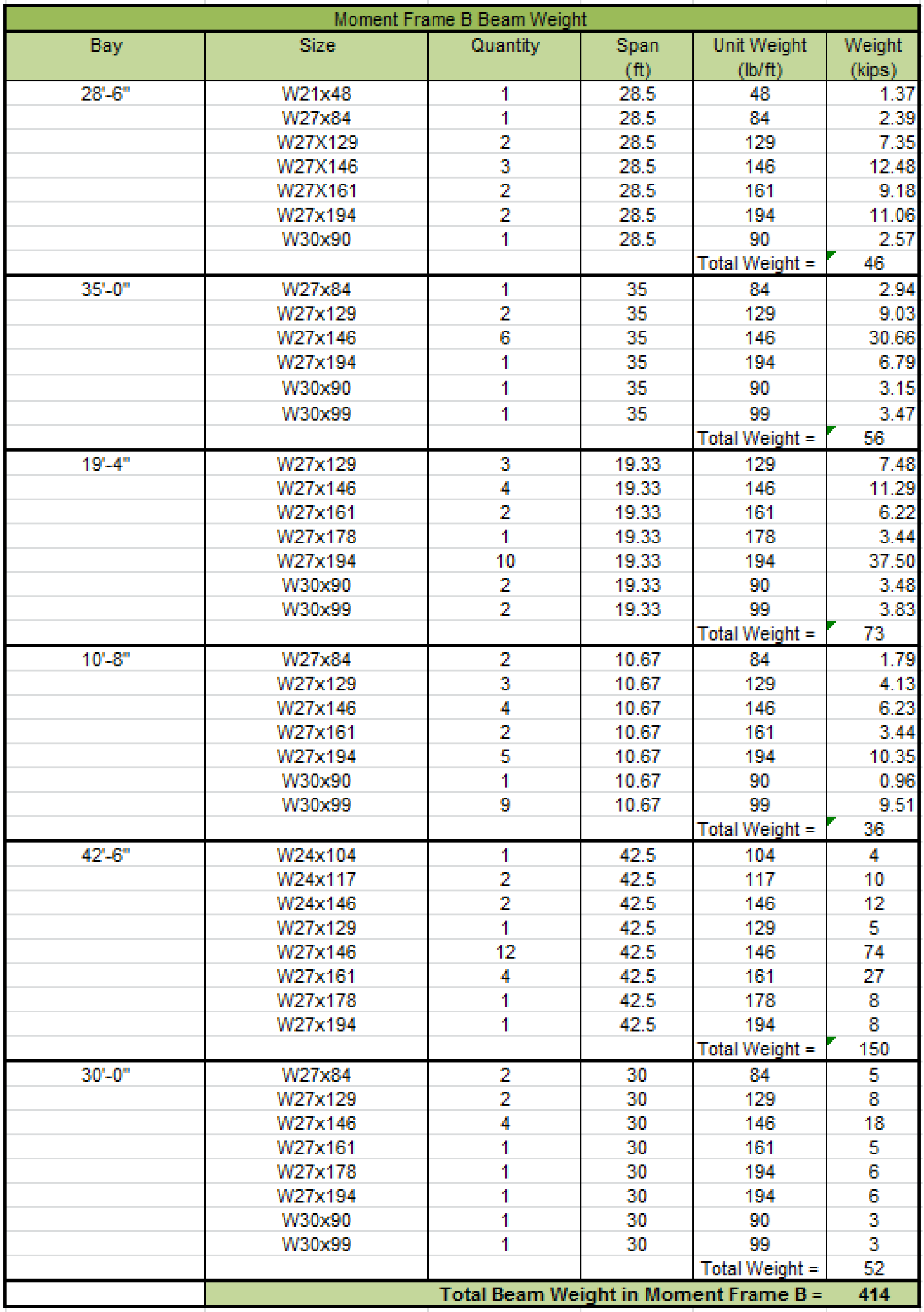


Gravity Girder Weight (2nd Level)					
Bay	Size	Quantity	Span (ft)	Unit Weight (lb/ft)	Weight (Kips)
28'-6"	W21x50	1	28.5	50	1.43
	W21x44	1	28.5	44	1.25
30-0"	W21x50	1	30	50	1.50
	W21x44	1	30	44	1.32
	W16x26	2	30	26	1.56
	W18x40	1	30	40	1.20
	W14x22	3	30	22	1.98
	W12x19	2	30	22	1.32
	W24x55	1	30	55	1.65
41'-0"	W27x84	5	41	84	17.22
	W24x68	4	41	68	11.15
24'-0"	W16x31	3	24	31	2.23
30'-0"	W21x44	2	30	44	2.64
35'-0"	W27x84	1	35	84	2.94
44'-0"	W24x76	1	44	76	3
	Total Girder Weight Supporting Level 2=				**52.7**



Moment Frame A.1 Beam Weight

Bay	Size	Quantity	Span (ft)	Unit Weight (lb/ft)	Weight (kips)
28'-6"	W21x48	1	28.5	48	1.37
	W27x129	2	28.5	129	7.35
	W27X146	4	28.5	146	16.64
	W27X161	1	28.5	161	4.59
	W27X178	1	28.5	178	5.07
	W27X194	1	28.5	194	5.53
	W30x90	2	28.5	90	5.13
				Total Weight =	46
17'-6"	W16x26	10	17.5	26	4.55
	W27x129	8	17.5	129	18.06
	W27x146	10	17.5	146	25.55
	W27x161	8	17.5	161	22.54
	W27x178	4	17.5	178	12.46
	W27x194	15	17.5	194	50.93
	W30x90	5	17.5	90	7.88
				Total Weight =	142
25'-0"	W27x84	1	25	84	2.10
	W27x129	6	25	129	19.35
	W27x146	5	25	146	18.25
	W27x161	5	25	161	20.13
	W27x178	2	25	178	8.90
	W27x194	4	25	194	19.40
	W30x90	1	25	90	2.25
				Total Weight =	90
30'-0"	W21x62	2	30	62	3.72
	W27x84	2	30	84	5.04
	W27x129	7	30	129	27.09
	W27x146	15	30	146	65.70
	W27x161	2	30	161	9.66
	W27x178	1	30	178	5.34
	W27x194	3	30	194	17.46
	W30x99	4	30	99	11.88
				Total Weight =	146
	Total Beam Weight in Moment Frame A.1 =				**424**



Moment Frame B Beam Weight					
Bay	Size	Quantity	Span (ft)	Unit Weight (lb/ft)	Weight (kips)
28'-6"	W21x48	1	28.5	48	1.37
	W27x84	1	28.5	84	2.39
	W27X129	2	28.5	129	7.35
	W27X146	3	28.5	146	12.48
	W27X161	2	28.5	161	9.18
	W27x194	2	28.5	194	11.06
	W30x90	1	28.5	90	2.57
				Total Weight =	46
35'-0"	W27x84	1	35	84	2.94
	W27x129	2	35	129	9.03
	W27x146	6	35	146	30.66
	W27x194	1	35	194	6.79
	W30x90	1	35	90	3.15
	W30x99	1	35	99	3.47
				Total Weight =	56
19'-4"	W27x129	3	19.33	129	7.48
	W27x146	4	19.33	146	11.29
	W27x161	2	19.33	161	6.22
	W27x178	1	19.33	178	3.44
	W27x194	10	19.33	194	37.50
	W30x90	2	19.33	90	3.48
	W30x99	2	19.33	99	3.83
				Total Weight =	73
10'-8"	W27x84	2	10.67	84	1.79
	W27x129	3	10.67	129	4.13
	W27x146	4	10.67	146	6.23
	W27x161	2	10.67	161	3.44
	W27x194	5	10.67	194	10.35
	W30x90	1	10.67	90	0.96
	W30x99	9	10.67	99	9.51
				Total Weight =	36
42'-6"	W24x104	1	42.5	104	4
	W24x117	2	42.5	117	10
	W24x146	2	42.5	146	12
	W27x129	1	42.5	129	5
	W27x146	12	42.5	146	74
	W27x161	4	42.5	161	27
	W27x178	1	42.5	178	8
	W27x194	1	42.5	194	8
				Total Weight =	150
30'-0"	W27x84	2	30	84	5
	W27x129	2	30	129	8
	W27x146	4	30	146	18
	W27x161	1	30	161	5
	W27x178	1	30	194	6
	W27x194	1	30	194	6
	W30x90	1	30	90	3
	W30x99	1	30	99	3
				Total Weight =	52
	Total Beam Weight in Moment Frame B =				**414**

Moment Frame E Beam Weight					
Bay	Size	Quantity	Span (ft)	Unit Weight (lb/ft)	Weight (kips)
28'-6"	W21x55	1	28.5	55	1.57
	W27X84	1	28.5	84	2.39
	W27X129	3	28.5	129	11.03
	W27X146	5	28.5	146	20.81
	W27x161	3	28.5	161	13.77
	W27x178	3	28.5	178	15.22
	W27x194	4	28.5	194	22.12
	W30x99	2	28.5	99	5.64
	Total Beam Weight in Moment Frame E =				**93**

Moment Frame 1 Beam Weight					
Bay	Size	Quantity	Span (ft)	Unit Weight (lb/ft)	Weight (kips)
41'-0"	W24x104	11	41	104	46.90
	W24x131	1	131	131	17.16
				Total Weight =	64
30'-0"	W21x55	1	30	55	1.65
	W27x84	11	30	84	27.72
				Total Weight =	29
28'-6"	W27x84	12	28.5	84	28.73
				Total Weight =	29
40'-0"	W24x104	11	40	104	45.76
				Total Weight =	46
	Total Beam Weight in Moment Frame 1 =				**168**

Moment Frame 1' Beam Weight					
Bay	Size	Quantity	Span (ft)	Unit Weight (lb/ft)	Weight (kips)
35'-0"	W27X84	3	35	84	8.82
	W27x129	7	35	129	31.61
	W30x99	2	35	99	6.93
				Total Weight =	47
30'-0"	W27X84	5	30	84	12.60
	W27x129	14	30	129	54.18
	W30x90	2	30	90	5.40
	W30x99	3	30	99	8.91
				Total Weight =	81
24'-0"	W27x84	3	24	84	6.05
	W27x129	3	24	129	9.29
	W27x146	4	24	146	14.02
	W30x99	2	24	99	4.75
				Total Weight =	34
	Total Beam Weight in Moment Frame 1' =				**163**

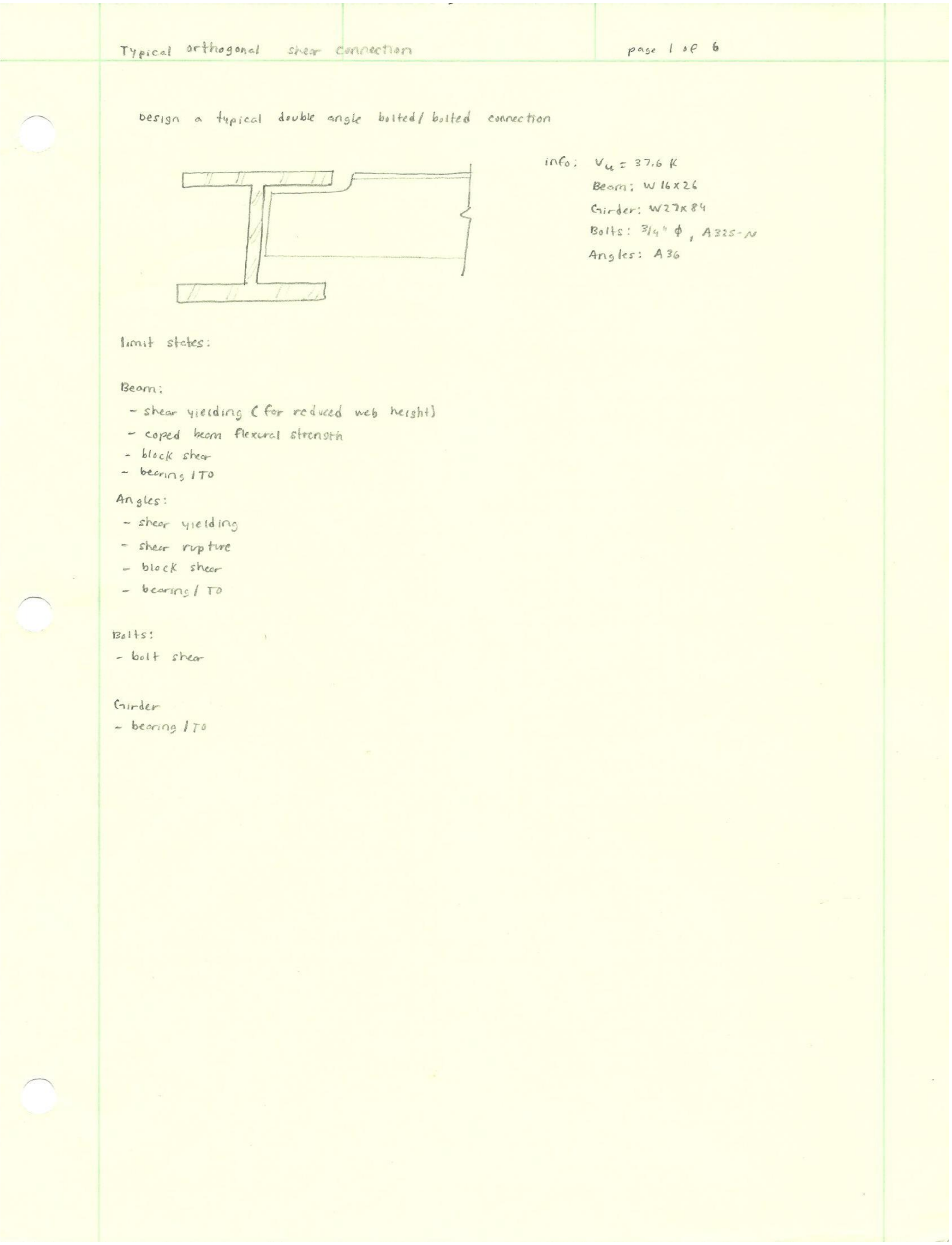
Moment Frame Column Weight				
Size	Quantity	Span - 2 tier (ft)	Unit Weight (lbs/ft)	Weight (Kips)
W14x43	1	30.00	43	1.29
W14x193	53	30.00	193	306.87
W14x211	19	30.00	211	120.27
W14x233	23	30.00	233	160.77
W14x257	54	30.00	257	416.34
W14x283	10	30.00	283	84.90
W14x311	46	30.00	311	429.18
W14x342	6	30.00	342	61.56
W14x370	2	30.00	370	22.20
W14x500	1	30.00	500	15.00
W14x550	4	30.00	550	66.00
W14x605	2	30.00	605	36.30
W14x665	8	30.00	665	159.60
W14x730	11	30.00	730	240.90
Total Column Weight in Moment Frames =				**2121**

Bracing Weight				
Size	Quantity	Span (ft)	Unit Weight (lbs/ft)	Weight (Kips)
W6x15	5	33.54	15	2.52
W8x28	1	33.54	28	0.94
W8x31	6	33.54	31	6.24
W8x35	1	33.54	35	1.17
W10x39	2	33.54	39	2.62
W10x45	3	33.54	45	4.53
W10x49	6	35.25	49	10.36
W10x77	4	33.54	77	10.33
W12x35	1	33.54	35	1.17
W12x35	1	35.25	35	1.23
W12x53	3	33.54	53	5.33
W12x58	5	33.54	58	9.73
W12x65	3	33.54	65	6.54
W12x65	1	35.25	65	2.29
W12x72	1	33.54	72	2.41
W12x87	2	33.54	87	5.84
W12x96	2	33.54	96	6.44
W12x106	2	33.54	106	7.11
W12x120	4	33.54	120	16.10
W12x136	4	33.54	136	18.25
W12x152	2	33.54	152	10.20
W12x170	2	33.54	170	11.40
W12x190	1	33.54	190	6.37
W12x210	4	33.54	210	28.17
W12x230	1	33.54	230	7.71
W12x252	3	33.54	252	25.36
W12x279	2	33.54	279	18.72
W12x305	1	33.54	305	10.23
W12x336	23	33.54	336	259.20
W14x193	7	33.54	193	45.31
		Total Bracing Weight =		**544**

Total Superstructure Weight= 5713

Appendix D: Typical Connections Design and Analysis

Typical Orthogonal Shear Connection



Typical orthogonal shear connection page 1 of 6

Design a typical double angle bolted/bolted connection

info: V_u = 37.6 K
Beam: W 16x26
Girder: W27x84
Bolts: 3/4" ϕ, A325-N
Angles: A36

limit states:

Beam:
- shear yielding (for reduced web height)
- coped beam flexural strength
- block shear
- bearing / TO

Angles:
- shear yielding
- shear rupture
- block shear
- bearing / TO

Bolts:
- bolt shear

Girder
- bearing / TO

Typical orthogonal shear connection

page 2 of 6

step 1: Design bolts

- bolt shear

using table 7-1 in steel construction manual,
$\phi r_n = 35.8 \text{ K/bolt}$

$V_u = \phi R_n$ (# of bolts)

$\# \text{ of bolts} = \frac{37.6 \text{ K}}{35.8 \text{ K/bolt}} = 1.05 \implies$ need to use at least 2 bolts

according to table 10-1 in steel construction manual, for a double angle connection supporting a w16 using 3/4" bolts, <u>use at least 3 rows of bolts</u>

step 2: Design Angles

- shear yielding

$\phi R_n = \phi (0.6 F_y) A_{gv} \geq V_u$

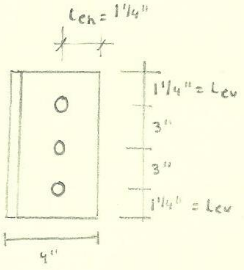
$A_{gv} \geq \frac{37.6 \text{ k}}{1.0 (0.6)(36 \text{ ksi})} = 1.74 \text{ in}^2 \implies$ try 2L4X4X 1/4 – 0'-8 1/2" w/ $A_{gv} = 3.64 \text{ in}^2$

- check shear rupture

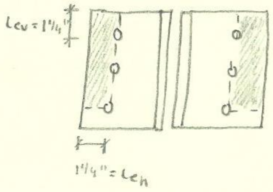
$\phi R_n = \phi (0.6 F_u) A_{nv} \geq V_u$, $A_{nv} = 3.64 \text{ in}^2 - A_{holes} = 3.64 \text{ in}^2 - \underbrace{(3/4" + 1/16" + 1/16")}_{d_{hole}} \underset{\substack{\uparrow \\ \text{angle thickness}}}{(1/4")} \underset{\substack{\uparrow \\ \text{\# of angles}}}{(2)} \overset{\substack{\text{\# of holes} \\ \downarrow}}{(3)}$

$= 2.33 \text{ in}^2$

$= 0.75 (0.6)(58 \text{ ksi})(2.33 \text{ in}^2)$
$= 60.8 \text{ K} > 37.6 \text{ K}$ oK



- check block shear



from table 9.3a ; $\frac{\phi F_u A_{nt}}{t} = 35.3$ K/in thickness ←

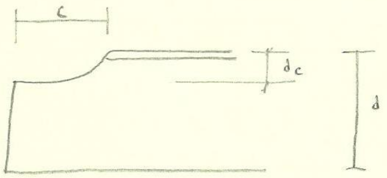
from table 9.3b: $\frac{\phi 0.6 F_y A_{gv}}{t} = 117$ K/in thickness ←

from table 9.3c : $\frac{\phi 0.6 F_u A_{nv}}{t} = 132$ K/in thickness

$$\phi R_n = (35.3 + 117) K/_{in} \underset{\substack{\uparrow \\ \text{angle} \\ \text{thickness}}}{(1/4 \text{ in})} \underset{\substack{\uparrow \\ \text{\# of angles}}}{(2)} = 76.2 K \geq V_u \quad \text{OK}$$

<u>step 3</u> : check beam limit states

- design single cope



$c \leq 2d = 2(15.7 in) = 31.4" \Rightarrow$ use $c = 6"$

$d_c \leq d/_2 = \frac{15.7}{2} = 7.85" \Rightarrow$ use $d_c = 2"$

- check shear yielding

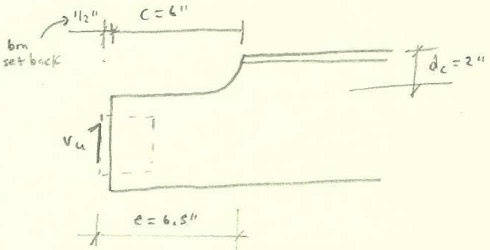
w16x26 : $F_y = 50$ ksi $F_u = 65$ ksi
$d = 15.7$ in , $t_w = 0.25$ in

$$\phi R_n = \phi (0.6 F_y) t_w \underset{\substack{\uparrow \\ d - d_c}}{h_o} = 1.0 (0.6)(50)(0.25)(15.7 - 2) = 102.8 K > V_u \quad \text{OK}$$

- check coped beam flexural strength

single cope :

$$F_{BC} = 26,210 \left(t_w / h_o \right)^2 f K \leq F_y$$



Typical orthogonal shear connection

page 4 of 6

$\frac{c}{d} = \frac{6}{15.7} = 0.382 < 1.0 \Rightarrow f = 2\left(c/d\right) = 2(0.382) = 0.764$

$\frac{c}{h_o} = \frac{6}{13.7} = 0.438 < 1.0 \Rightarrow K = 2.2\left(h_o/c\right)^{1.65} = 8.59$

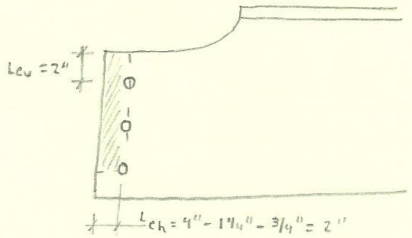
$F_{BC} = 26210\left(\frac{0.250}{13.7}\right)^2 (0.764)(8.59) = 57.3 \text{ ksi} \leq F_y = 50 \text{ ksi}$

↑ controls

$\phi M_n = \phi F_y S_{net} = 0.90(50)(11.6 \text{ in}^3) = 522 \text{ K-in}$

$\phi V_n = \frac{\phi M_n}{e} = \frac{522 \text{ K-in}}{6.5 \text{ in}} = 80.3 \text{ K} > V_u$ ok

- check block shear



↑

½" set back + additional ¼" for design

from table 9.3a: $\frac{\phi F_u A_{nt}}{t} = 76.2 \text{ K/in}$ ←

From table 9.3b: $\frac{\phi 0.6 F_y A_{gv}}{t} = 180 \text{ K/in}$

from table 9.3c: $\frac{\phi 0.6 F_u A_{nv}}{t} = 170 \text{ K/in}$ ←

$\phi R_n = (76.2 + 170) \text{ K/in } (0.25 \text{ in}) = 61.55 \text{ K} > V_u$ ok

Typical orthogonal shear connection

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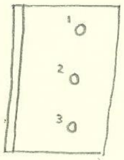
bolt shear and

step 4: check^bearing /TO on angle, beam, and girder

bolt shear: $\phi r_n = 35.8$ K ← all bolts



bearing on angles: using table 7-4

$s = 3''$

$F_u = 58$ ksi

$d_b = 3/4''$ std

$\phi r_n = 78.3\ k/_{in}\ (1/4\ in)(2)$

↑ # of angles

$= 39.15$ K ← all bolts

tear-out on angles: using table 7-5

edge dist, $L_e = 1.25''$

$F_u = 58$ ksi

$d_b = 3/4''$

$\phi r_n = 44\ k/_{in}\ (1/4\ in)(2) = 22$ K ← bolt 1

bearing on beam: using table 7-4

$s = 3''$

$F_u = 65$ ksi

$d_b = 3/4''$ std

$\phi r_n = 87.8\ k/_{in}\ (0.25) = 22$ K ← all bolts

tear-out on beam: using table 7-5

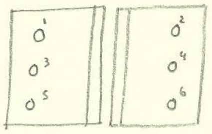
edge dist, $L_e = 2''$

$F_u = 65$ ksi

$d_b = 3/4''$

$\phi r_n = 87.8\ k/_{in}\ (0.25) = 22$ K ← bolt 1

bearing on girder: using table 7-4



$s = 3''$

$F_u = 65$ ksi

$d_b = 3/4''$

$\phi r_n = 87.8\ k/_{in}\ (0.460\ in) = 40.4$ K ← all bolts

tear-out on girder: will not control by inspection

$\phi R_n = \Sigma r_{n_{i,min}} = 22K + 22K + 22K = 66K \geq V_u$ oK

Typical orthogonal shear connection

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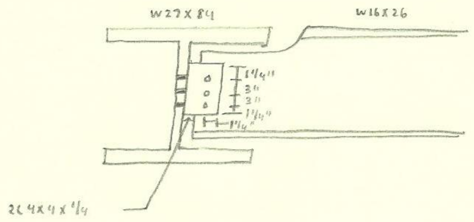
step 5: check the connection's spacing and edge distance limitations

using pg. 16.1-122 of AISC steel construction manual and table J3.4 (sect J3)

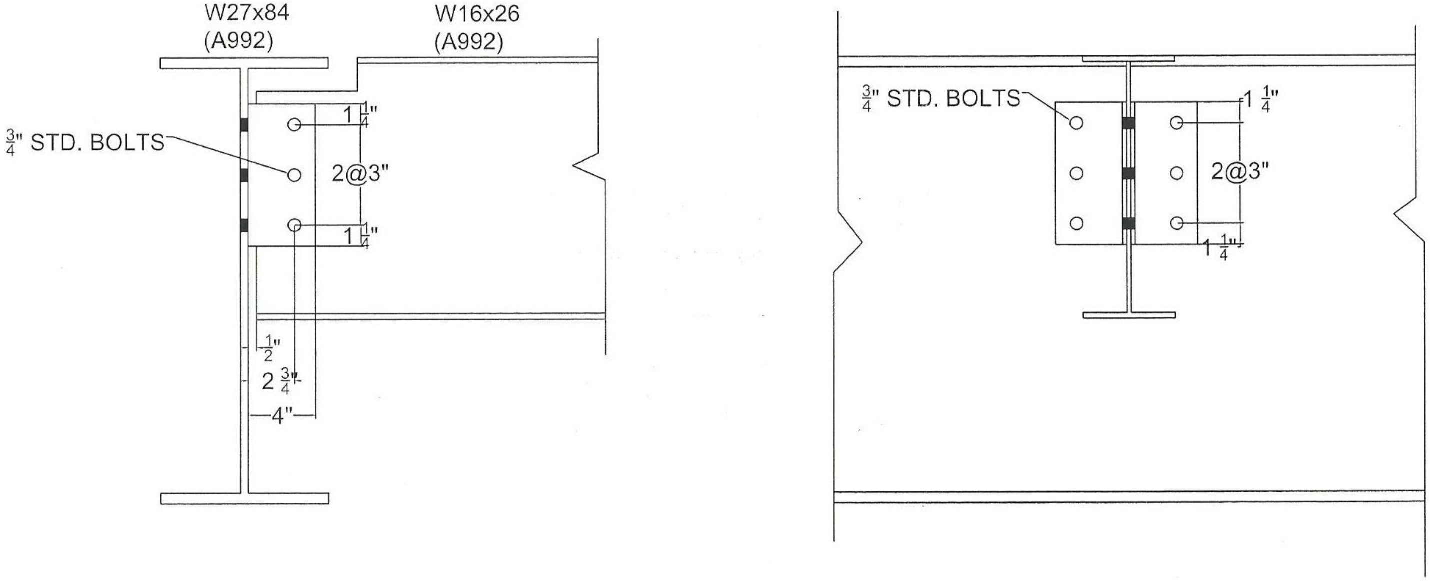
connection spacing and edge dist.		connection spacing and edge distance limitations	
center-to-center spacing between bolts	: 3"	min. ctr-to-ctr spacing between bolts	: 3d = 3(3/4) = 2.25"
edge dist. from ctr of std hole to edge of angles	: 1.25"	min. edge dist. from std. hole to connected part	: 1" for 3/4" ϕ bolts
edge dist. from ctr. of std. hole to edge of beam	: 2"	max. edge dist. from ctr of std. hole to connected part	: 12(0.25) = 3" ↑ smaller of the thicknesses of the connected parts Angle t = 1/4 (2) = 1/2" beam t_w = 1/4" ← controls

The designed connection meets all spacing and edge distance limitations

Final Design:



- refer to AutoCAD drawing for final detailed design



TYPICAL ORTHOGONAL SHEAR CONNECTION

Typical Skewed Shear Connection

Typical skewed shear connection

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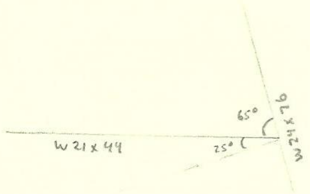
Design a typical skewed shear connection



info: $V_u = 81.8$ K
Beam: W21x44
Girder: W24x76
plate: A36

For design, use AISC journal "orthogonal and skewed shear connections design and detailing requirements" and chapter 10 of AISC steel manual

limit states:

Beam:
- beam gross shear yield (depth of coped beams)
- coped beam flexural strength

weld:
- check min. weld
- weld rupture
- base metal

plate:
- shear yield
- shear rupture
- block shear
- bearing / TO

bolts:
- bolt shear

girder:
- bearing / TO

Typical skewed shear connection

page 2 of 6

step 1: Design bolts

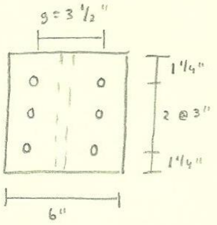
- bolt shear

using table 7-1 in steel manual

$\phi r_n = 17.9$ k/bolt

$V_u = \phi r_n$ (# of bolts)

$\# \text{ of bolts} = \frac{81.8k}{17.9 \text{ k/bolt}} = 4.57 \Rightarrow$ use 6 bolts



step 2: Design plate

- plate gross shear

$\phi V_n = \phi(0.6 F_y)(2 l_p t_p) \geq V_u$

$t_p \geq \frac{81.8k}{1.0(0.6)(36 ksi)(2)(8.5 in)} = 0.22" \Rightarrow$ use 1/4" thick plate

- shear rupture

$d_h = 3/4" + 1/16" + 1/16" = 7/8"$

$A_{net} = (8.5 - 3 \times 7/8)(t)(2) = 11.75 t_p$

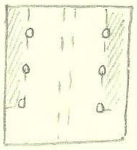
$\phi V_n = 0.75(0.6 F_u) A_n \geq V_u$

$t_p \geq \frac{81.8k}{0.75(0.6)(58 ksi)(11.75)} = \underset{\substack{\uparrow \\ \text{controls}}}{0.27"} \Rightarrow$ use 3/8" thick plate

- check block shear

PL 6" x 3/8" x 0'-8 1/2"

$l_{eh} = 1 1/4"$, $l_{ev} = 1 1/4"$



from table 9.3a: $\frac{\phi F_u A_{nt}}{t} = 35.3$ k/in ←

from table 9.3b: $\frac{\phi 0.6 F_y A_{gv}}{t} = 117$ k/in ←

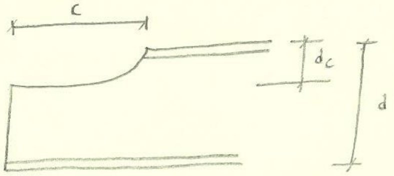
from table 9.3c: $\frac{\phi 0.6 F_u A_{nv}}{t} = 132$ k/in

$\phi R_n = (35.3 + 117) \text{ k/in} (3/8 \text{ in}) \underset{\substack{\uparrow \\ \text{\# of block shear planes}}}{(2)} = 114.2K > V_u$ OK

Typical skewed shear connection

step 3: check beam limit states

- design single cope



$c \leq 2d = 2(20.7) = 41.4 \Longrightarrow$ use $c = 6''$

$d_c \leq d/_2 = \frac{20.7}{2} = 10.35'' \Longrightarrow$ use $d_c = 2''$

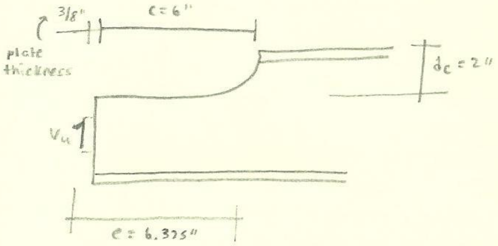
- check beam gross shear yield

W21x44; $F_y = 50$ ksi $\quad F_u = 0.65$ ksi

$d = 20.7$ in , $\quad t_w = 0.350$ in

$\phi R_n = \phi(0.6 F_y) t_w h_o = 1.0(0.6)(50)(0.350)(20.7-2) = 196.4$ K $> V_u$ OK

- check coped beam flexural strength



single cope:

$F_{BC} = 26,210 \left(t_w/h_o\right)^2 fk \leq F_y$

$\frac{c}{d} = \frac{6}{20.7} = 0.29 < 1.0 \Longrightarrow f = 2\left(c/_d\right) = 2(0.29) = 0.58''$

$\frac{c}{h_o} = \frac{6}{18.7} = 0.32 < 1.0 \Longrightarrow k = 2.2\left(h_o/_c\right)^{1.65} = 14.36$

$F_{BC} = 26210\left(\frac{0.350}{18.7}\right)^2(0.58)(14.36) = 76.5 \text{ ksi} \leq F_y = 50 \text{ ksi}$

↑ controls

$\phi M_n = \phi F_y S_{net} = 0.90(50 \text{ ksi})(29.1 \text{ in}^3) = 1310$ k-in

$\phi V_n = \frac{\phi M_n}{e} = \frac{1310 \text{ K-in}}{6.375 \text{ in}} = 205.5$ K $> V_u$ OK

Typical skewed shear connection

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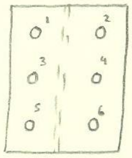
Step 4: check bolt shear, bearing/TO on plate, bearing/TO on girder

bolt shear : $\phi r_n = 17.9$ K/bolt ← all bolts



bearing on plate: using table 7-4

$s = 3''$

$F_u = 58$ ksi

$d_b = 3/4''$ std

$\phi r_n = 78.3$ k/in (3/8 in)

$= 29.4$ k /bolt ← all bolts

tear-out on plate: using table 7-5

edge dist, $L_e = 1.25''$

$F_u = 58$ ksi

$d_b = 3/4''$

$\phi r_n = 44$ K/in (3/8 in) = 16.5 K/bolt ← bolts 1 and 2

bearing on girder; using table 7-4

$s = 3''$

$F_u = 65$ ksi

$d_b = 3/4''$

$\phi r_n = 87.8$ k/in (0.440 in) = 38.6 K/bolt ← all bolts

tear-out on girder; will not control by inspection

$\phi R_n = \Sigma r_{n_{i},min} = 16.5 + 16.5 + 17.9 + 17.9 + 17.9 + 17.9 = 104.6$ k > V_u OK

Step 5:

- check min. weld

using table J2.4 in steel manual,
the mat. thickness of thinner part joined is the beam with $t_w = 0.35''$ ∴ min weld size = 3/16"

- beam web strength at weld

$$\phi V_n = 0.75\,(0.6\,F_u)(L_p - 2\,t_{weld})\,t \geq V_u$$

$$8.5 - 2\,t_{weld} \geq \frac{81.8}{0.75(0.6)(65)(0.35)} = 8.0$$

$$t_{weld} \leq \frac{8.0 - 8.5}{-2} = 1/4'' \Rightarrow \text{use } t_{weld} = 3/16''$$

Typical skewed shear connection

page 5 of 6

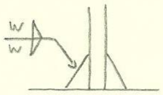
- check weld rupture

$$\phi V_n = \overset{\#\text{ of } 1/16\text{ ths}}{D} \times 1.392 \, (L_p - 2\, t_{weld})$$

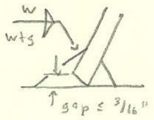
$$= 6 \times 1.392 \, (8.5 - 2 \, (3/16)) = 67.86 \text{ k} < V_u$$ not good ∴ increase $t_{weld} = 1/4"$

$$\phi V_n = 8 \times 1.392 \, (8.5 - 2 \, (1/4)) = 89.1 \text{ K} > V_u \text{ oK}$$

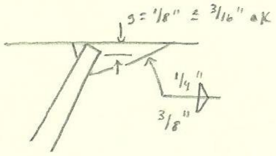
according to "orthogonal and skewed shear connections" journal, the AISC method increases the strength on the acute side by maintaining a constant fillet size w while the increased size w+g on the obtuse side actually losses strength because of the gap g and the geometrical reduction in effective throat, due to the dihedral angle being greater than 90°.



Required orthogonal weld



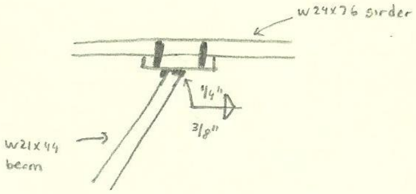
for the designed connection

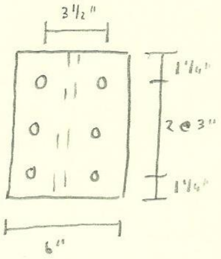


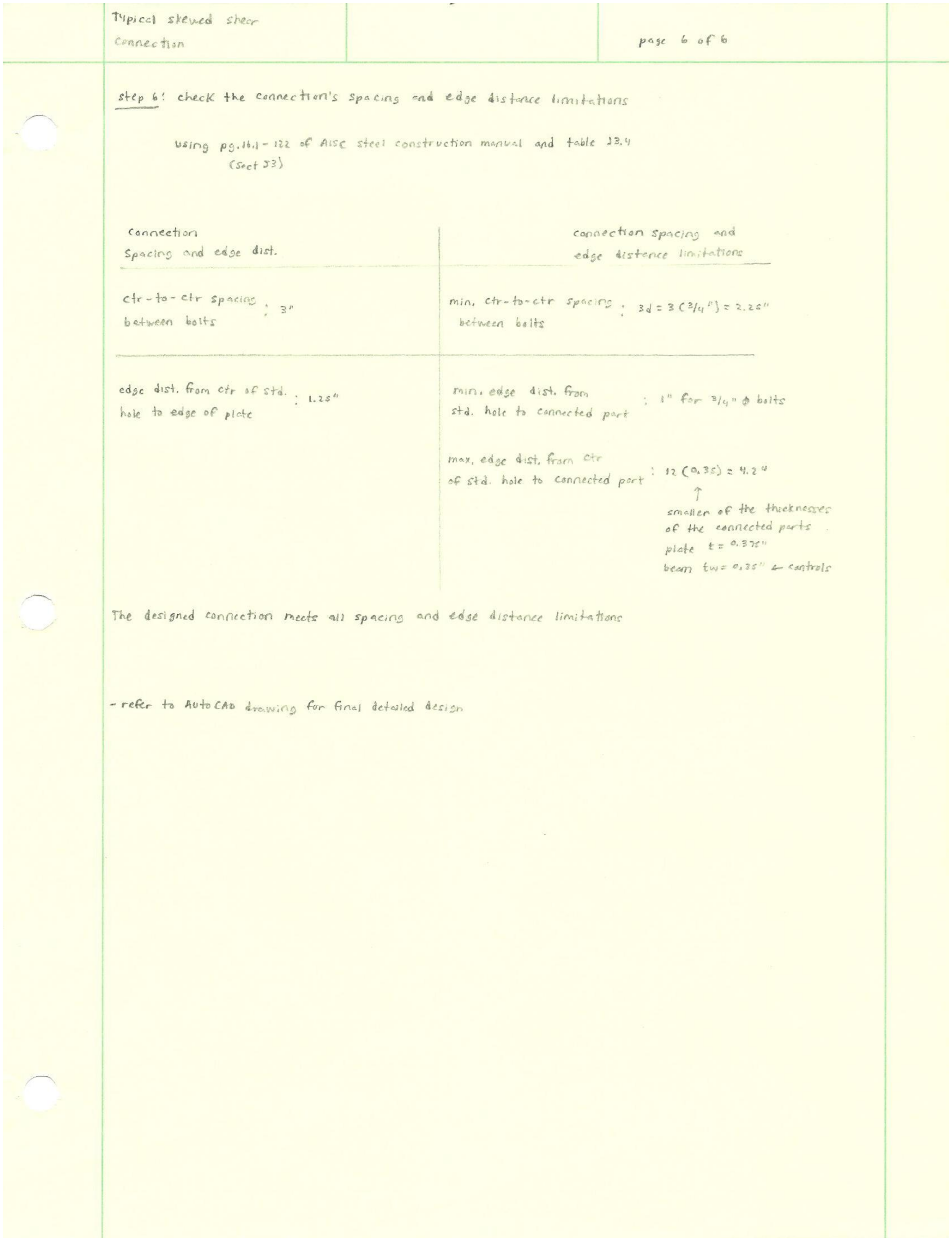
$w = 1/4"$

$w + g = 1/4" + 1/8" = 3/8"$

Final Design:







Typical skewed shear connection

page 6 of 6

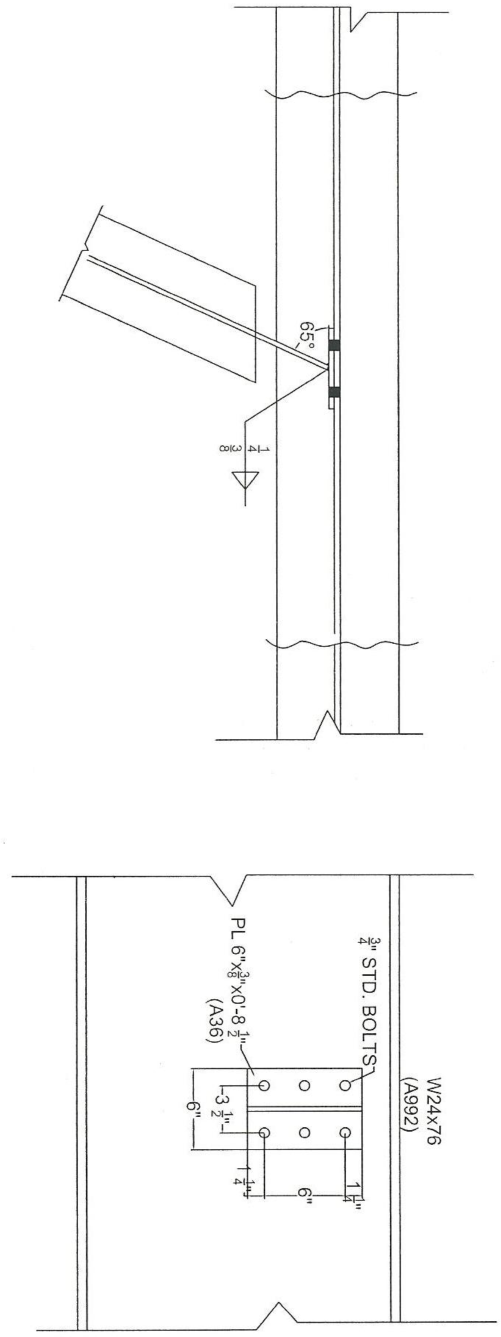
step 6: check the connection's spacing and edge distance limitations

using pg. 16.1 - 122 of AISC steel construction manual and table J3.4 (Sect J3)

Connection spacing and edge dist.	connection spacing and edge distence limitations
ctr-to-ctr spacing between bolts : 3"	min. ctr-to-ctr spacing between bolts : 3d = 3 (3/4") = 2.25"
edge dist. from ctr of std. hole to edge of plate : 1.25"	min. edge dist. from std. hole to connected part : 1" for 3/4" ϕ bolts max. edge dist. from ctr of std. hole to connected part : 12 (0.35) = 4.2" ↑ smaller of the thicknesses of the connected parts plate t = 0.375" beam tw = 0.35" ← controls

The designed connection meets all spacing and edge distance limitations

- refer to AutoCAD drawing for final detailed design



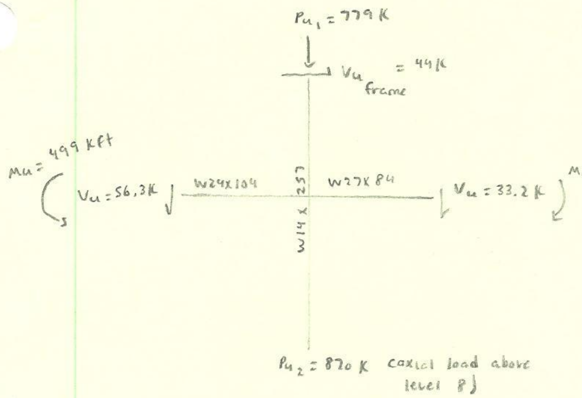
TYPICAL SKEWED SHEAR CONNECTION

Typical Moment Connection

Typical Moment connection

page 1 of 11

Design a flange plate bolted / web bolted moment connection



controlling load combination: 1.2D + 1.0W + L

W24 X 104: $P_{u_{Dead}} = 1.2\,(\underset{\text{slab}}{75} + \underset{\text{SDL}}{10} + \underset{\text{bm self wt allow.}}{5})\text{ psf }(15.75)(10)/1000$

$= 17.01$ K

$P_{u_L} = 68\text{ psf }(15.75\text{ft})(10\text{ft})/1000$

$= 10.71$

$P_{u_{total}} = 27.7$ K

$w_D = 1.2\,(\underset{\text{curtain wall self-wt}}{250} + \underset{\text{self-wt}}{104})/1000 = 0.425$ KIf

W27x84: $P_{u_{Dead}} = 1.2\,(75 + 10 + 5)\text{psf }(15.75)(10)/1000$

$= 17.01$K

$P_{u_L} = 75\text{psf }(15.75\text{ ft})(10\text{ft})/1000$

$= 11.8$ K

$P_{u_{total}} = 17.0 + 11.8 = 28.8$K

$w_D = 1.2\,(250 + 84)/1000 = 0.401$ kIf

Limit states:

- Tension Flange plate limit states
  1. tension yielding
  2. tension rupture
  3. block shear
  4. bearing / TO
- compression plate limit states
  1. buckling
  2. bolt shear / bearing / tear out
- Girder limit states
  1. reduced flexural strength
  2. beam flange block shear
- web Plate
  1. shear yielding
  2. shear rupture
  3. bolt shear / bearing / Tear out
  4. block shear
- weld strength
- web bolts
  1. bolt shear

Typical Moment connection

page 2 of 11

All wide flange shapes are A992 steel.
All plates are A36

Design moment connection for the W27x84

geometric properties: girder W27x84 ; $t_w = 0.460$ in , $d = 26.7$ in , $t_f = 0.640$ in , $b_f = 10.0$ in , $S_x = 213$ in^3

- check beam available flexural strength
  assume 2 rows of bolts in standard holes ; use 7/8" bolts

$A_{fg} = b_f t_f = 10 \text{ in} (0.640 \text{ in}) = 6.40 \text{ in}^2$

$A_{fn} = A_{fg} - 2(d_b + 1/8) t_f = 6.40 - 2(7/8 + 1/8)(0.640) = 5.12 \text{ in}^2$

$\frac{F_y}{F_u} = \frac{50}{65} = 0.769 < 0.80 \therefore Y_t = 1.0$

$F_u A_{fn} = 65 \text{ ksi} (5.12 \text{ in}^2) = 332.8 \text{ K}$

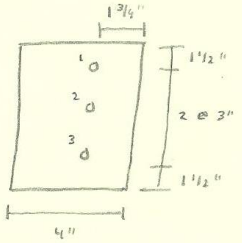
$Y_t F_y A_{fg} = (1.0)(50 \text{ ksi})(6.40) = 320 \text{ K} < 332.8 \text{ K}$ ∴ flange rupture / web yield does not control

- Design single plate web connection

from table 10-10a : Try a PL 1/4" x 4" x 0'- 9 1/2", with (3) 7/8" Φ A325-N bolts and 3/16" Fillet weld

- design shear strength of bolts
  single shear ; from table 7-1 , $\phi r_n = 24.3 \text{ K/bolt}$ ← all bolts

  $\# \text{ of bolts} = \frac{33.2 \text{ K}}{24.3 \text{ K/bolt}} = 1.36 \text{ bolts} \leq 3 \text{ bolts} \therefore \text{ok}$



from table J3.4, for 7/8" bolts, min. edge dist = 1 1/8"
use 1 3/4" and 2 1/4" ∴ ok

- bearing strength of bolts
  bolt spacing = 3", from table 7-4 : $\phi r_n = 91.4 \text{ K/in}$

  $\phi r_n = 91.4 \text{ K/in/bolt} (1/4) = 22.9 \text{ K/bolt}$ ← all bolts

  $= \frac{33.2 \text{ K}}{22.9 \text{ K/bolt}} = 1.45 \text{ bolts} < 4 \text{ bolts} \therefore \text{ok}$

- check tear-out
  $\phi r_n = 0.75(1.2)[1.5 - 0.5(7/8 + 1/8)](58)(1/4) = 13.05 \text{ K/bolt}$ ← bolt 1

  $\phi R_n = 13.05 + 22.9 + 22.9 = 58.9 \text{ K} > V_u = 33.2 \text{ K}$

- plate shear yielding
  $\phi R_n = \phi 0.6 F_y A_g = 1.0(0.6)(36)(9)(1/4) = 48.6 \text{ K} > 33.2 \text{ K} \therefore \text{ok}$

Typical moment connection

page 3 of 11

- plate shear rupture

$\phi R_n = \phi\, 0.6\, F_u A_{nv}$

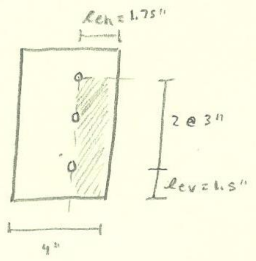
$= 0.75(0.6)(58)(1.5)$

$= 39.2\text{ K} > 33.2\text{ K}$ ∴ ok

$A_{hole} = (3\text{ bolts})(7/8 + 1/8)(1/4) = 3/4\text{ in}^2$

$A_{nv} = 9(1/4) - 3/4 = 1.5\text{ in}^2$

- block shear rupture strength of the plate



$$\text{T9.3a}: \frac{\phi F_u A_{nt}}{t} = 54.4\text{ K/in}$$

$$\text{T9.3b}: \frac{\phi\, 0.6\, F_y A_{gv}}{t} = 121\text{ K/in} \leftarrow$$

$$\text{T9.3c}: \frac{\phi\, 0.6\, F_u A_{nv}}{t} = 131\text{ K/in}$$

$$\phi R_n = \frac{1}{4}(54.4 + 121) = 43.9\text{ K} > 33.2\text{ K} \therefore \text{ok}$$

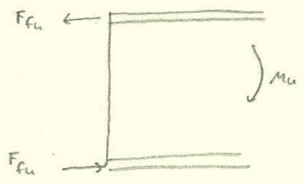
- weld strength

$\phi R_n = 1.392\, D\ell\, (2)$

$= 1.392\,(3\text{ sixteenths})(9\text{ in})(2) = 75.2\text{ K} > 33.2\text{ K}$ ∴ ok

– Design tension flange plate and connection

$$F_{fu} = \frac{M_u}{d} = \frac{(199\text{ k ft})(12)}{26.7\text{ in}} = 89.4\text{ K}$$



try a PL 1/2'' x 8'' with 7/8'' ϕ A-325N bolts, $\phi r_n = 24.3$ K/bolt

- bearing on flange

$\phi r_n = (102\text{ K/in})(0.640\text{ in}) = 65.3\text{ K/bolt} > 24.3\text{ K/bolt}$

↑
from
table 7-4

- bearing on plate

$\phi r_n = (91.4\text{ K/in})(0.50\text{ in}) = 45.7\text{ K/bolt} > 24.3\text{ K/bolt}$

↑
from
table
7-4

shear controls ∴ the number of bolts required is : $n_{min} = \frac{F_{fu}}{\phi r_n} = \frac{89.4\text{ K}}{24.3\text{ K/bolt}} = 3.67 \Rightarrow$ use 6 bolts

- check flange plate tension yielding

$P_n = F_y A_g = (36)(8)(1/2) = 144\ K$

$F_{fu} = \dfrac{M_u}{d + t_p} = \dfrac{(199\ kft)(12)}{(26.7 + 1/2)\,in} = 87.8K$

$\phi P_n = 0.9(144) = 129.6K > F_{fu} = 87.8K$ ∴ ok to use 1/2" x 8" plate

- check flange plate tension rupture

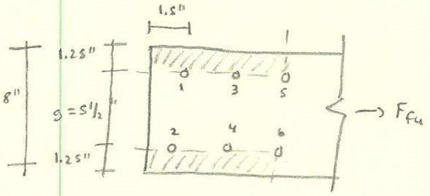
$A_n \le 0.85 A_g = 0.85(8\,in)(1/2\,in) = 3.4\,in^2$

$A_n = [b_p - 2(d_b + 1/8)]\,t_p = [8 - 2(7/8 + 1/8)](1/2) = 3\,in^2$

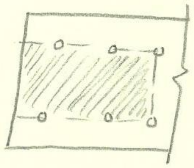
$A_e = U A_n = 1.0(3\,in^2) = 3\,in^2$

$\phi P_n = 0.75 F_u A_e = 0.75(58\,ksi)(3\,in^2) = 130.5\ K > 87.8K$ ∴ ok

- check flange plate block shear



case 1



case 2

by inspection, case 1 is more critical since it results in a smaller block shear area

$L_{eh} = 1.25"$, $L_{ev} = 1.5"$

T 9.3a ; 32.6 k/in
T 9.3b ; 121 k/in
T 9.3c : 131 k/in

$\phi R_n = 1/2\,in\,(32.6 + 121)\,k/in\,(2) = 153.6\,K > F_{fu} = 87.8\,K$ ∴ ok

↑ # of block shear planes

- check flange plate bolt shear / bearing / Tear-out

tear-out $\phi r_n = 0.75(1.2)[1.5 - 0.5(7/8 + 1/16)](58)(1/2) : 26.9\ k/bolt$ ← bolts 1 and 2

bolt shear, $\phi r_n = 24.3\ k/bolt$ ← all bolts

bearing, $\phi r_n = 91.4\ k/in/bolt\,(1/2) = 45.7\ k/bolt$ ← all bolts

$\phi R_n = 24.3(6) = 145.8\ K > 87.8\ K$ ∴ ok

Typical moment connection

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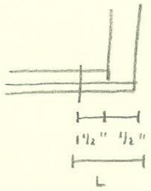
- design compression flange plate and connection

try PL ½" x 8"

- flexural buckling

assume $k = 0.65$ and $L = 2"$ (1.5" edge dist. + ½" set back



$$\frac{kL}{r} = \frac{0{,}65(2)}{0{,}289(\frac{1}{2}")} = 9 < 25 \quad \therefore F_{cr} = F_y$$

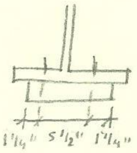
$\uparrow$ t_p

$A_g = (\frac{1}{2}")(8") = 4\,in^2$

$$\phi P_n = 0.9(36)(4) = 129.6\ K > F_{fu} = 87.8K \quad \therefore ok$$

$\phi \cdot F_{cr} \cdot A_g$

- check local buckling



$b_f = 5\frac{1}{2}"$
$b = 1\frac{1}{4}"$
$t_p = \frac{1}{2}"$

stiffened element

$$\frac{b_f}{t_p} \le \frac{253}{\sqrt{F_y}}$$

$$\frac{5.5}{\frac{1}{2}} \le \frac{253}{\sqrt{36}}$$

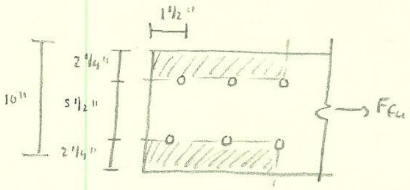
$$11 \le 42.2 \quad \therefore ok$$

unstiffened element

$$\frac{b}{t_p} \le \frac{95}{\sqrt{F_y}}$$

$$\frac{1.25}{1} = 1.25 \le \frac{95}{\sqrt{36}} = 15.8 \quad \therefore ok$$

- check beam flange block shear



$L_{eh} = 2\frac{1}{4}"$, $L_{ev} = 1\frac{1}{2}"$

T9.3a : 85.3 K/in
T9.3b : 186 K/in
T9.3c : 168 K/in

$$\phi R_n = (85.3 + 168)\ K/in\ (0.640\,in) = 162\ K > F_{fu} = 87.8K \quad \therefore ok$$

Typical Moment connection

page 6 of 11

Design moment connection for W24x104

geometric properties: girder W24x104 ; $t_w = 0.500$ in, $d = 24.1$ in, $t_f = 0.750$ in, $b_f = 12.8$ in, $S_x = 258$ in^3

- check beam available flexural strength

  assume 2 rows of bolts in standard holes; use 7/8" bolts

$A_{fg} = b_f t_f = 12.8(0.750) = 9.6\ in^2$

$A_{fn} = A_{fg} - 2(d_b + 1/8)t_f = 9.6 - 2(7/8 + 1/8)(0.750) = 5.7\ in^2$

$\frac{F_y}{F_u} = \frac{50}{65} = 0.769 < 0.80 \quad \therefore Y_t = 1.0$

$F_u A_{fn} = 65\ ksi(5.7\ in^2) = 370.5\ K$

$Y_t F_y A_{fg} = 1.0(50\ ksi)(9.6\ in^2) = 480\ K > 370.5\ K$ ∴ flange rupture / web yield will control

$M_n = \frac{F_u A_n S_x}{A_{fg}} = \frac{65(5.7)(258)}{9.6} = 9957.2\ K\text{-}in = 830\ Kft$

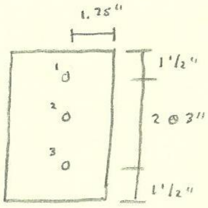
$\phi_b M_n = 0.9(830) = 747\ Kft > M_u = 499\ Kft \quad \therefore$ ok

- Design single plate web connection

  from table 10-10a : Try PL 3/8" X 4" x 0'- 9" , with (3) 7/8" ϕ A325-N bolts and 1/4" fillet weld

  - design shear strength of bolts

    single shear : from table 7-1, $\phi r_n = 24.3\ k/bolt$ ← all bolts



$\# \text{ of bolts} = \frac{56.3\ K}{24.3\ K/bolt} = 2.32$ bolts ≤ 3 bolts ∴ ok

  - bearing strength of bolts

    bolt spacing = 3", from table 7-4 : $\phi r_n = 91.4\ K/in$

$\phi r_n = 91.4\ K/in/bolt\ (3/8\ in) = 34.3\ K/bolt$ ← all bolts

$= \frac{56.3\ K}{34.3\ K/bolt} = 1.64$ bolts < 3 bolts ∴ ok

  - check tear-out

$\phi r_n = 0.75(1.2)[1.5 - 0.5(7/8 + 1/8)](58)(3/8) = 19.6\ K/bolt$ ← bolt 1

$\phi R_n = 19.6 + 24.3 + 24.3 = 68.2\ K > V_u = 56.3\ K \quad \therefore$ ok

  - plate shear yielding

$\phi R_n = 1.0\ (0.60)(36)(9)(3/8) = 72.9\ K > 56.3\ K \quad \therefore$ ok

Typical Moment connection

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$F_{fu} = \frac{499(12)}{(24.1+3/4)} = 241\text{ K}$

$\phi P_n = 0.9(36)(12)(3/4) = 291.6\text{ K} > 241\text{ K}$ ∴ ok to use 3/4" x 12" plate

- check flange plate tension rupture

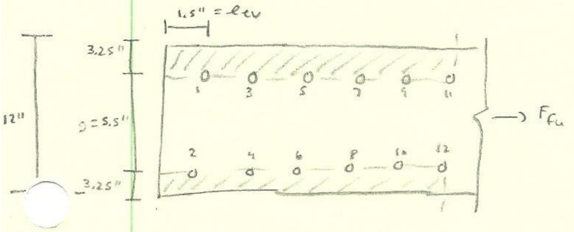
$A_n \leq 0.85 A_g = 0.85(12)(3/4) = 7.65\text{ in}^2$ ← controls

$A_n = [12.8 - 2(7/8+1/8)](3/4) = 8.1\text{ in}^2$

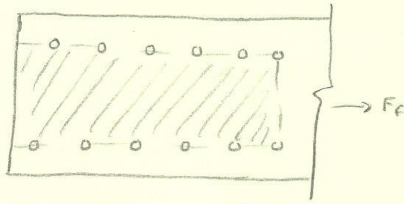
$A_e = 1.0 A_n = 1.0(7.65\text{ in}^2) = 7.65\text{ in}^2$

$\phi P_n = 0.75(58\text{ ksi})(7.65\text{ in}^2) = 332.8\text{ K} > 241\text{ K}$ ∴ ok

- check flange plate block shear



case 1



case 2

by inspection, case 2 is more critical since it results in a smaller block shear area

use $\ell_{eh} = 2.75"$ and $\ell_{ev} = 1.5"$

^ in tables

T 9.3a: 97.9 K/in

T 9.3b: 267 K/in

T 9.3c: 287 K/in

$\phi R_n = 3/4\text{ in}(97.9+267)(2) = 547.4\text{ K} > 241\text{ K}$ ∴ oK

↑ # of failure planes

$A_{gv} = 2(5(3\text{in})+1.5)(3/4) = 24.75\text{ in}^2$

$A_{nv} = 2[16.5 - 5.5(7/8+1/8)](3/4) = 16.5\text{ in}^2$

$A_{nt} = (5.5 - (7/8+1/8)](3/4) = 3.375\text{ in}^2$

$\phi U_{BS} F_u A_{nt} = 146.9\text{ K}$

$\phi\, 0.6 F_u A_{nv} = 430.65\text{ K}$

$\phi\, 0.6 F_y A_{gv} = 401\text{ K}$

$\phi R_n = 146.9 + 401 = 547.9\text{ K}$

- check flange plate bolt shear / bearing / tearout

tear-out: $\phi r_n = 0.75(1.2)[1.5 - 0.5(7/8+1/16)](58)(3/4) = 40.4\text{ K/bolt}$ ← bolts 1 and 2

bolt shear, $\phi r_n = 24.3\text{ K/bolt}$ (from table 7-1)

↑ all bolts

bearing on plate, $\phi r_n = 91.4\text{ K/bolt}(3/4) = 68.6\text{ K/bolt}$ ← all bolts

$\phi R_n = 12(24.3\text{ K/bolt}) = 291.6\text{ K} > 241\text{ K}$ ∴ oK

Typical Moment Frame

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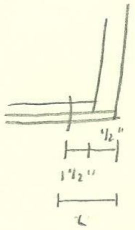
- design compression flange plate and connection

  try PL $3/4" \times 12"$

- flexural buckling

  assume $K = 0.65$ and $L = 2"$ (1.5" edge dist. + 1/2" set back)



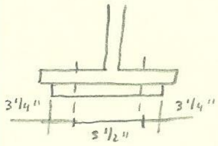
$$\frac{KL}{r} = \frac{0.65(2)}{0.289(3/4")} = 6 < 25 \therefore F_{cr} = F_y$$

$\uparrow$ t_p

$A_g = (3/4")(12") = 9 \text{ in}^2$

$\phi P_n = 0.9(36)(9) = 291.6 \text{ K} > F_{fu} = 241 \text{ K} \therefore \text{ok}$

- check local buckling



$b_f = 5\,1/2"$

$b = 3\,1/4"$

$t_p = 3/4"$

stiffened element

$$\frac{b_f}{t_p} \le \frac{253}{\sqrt{F_y}}$$

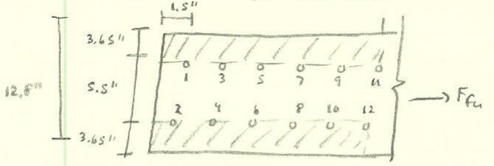
$$\frac{5.5}{3/4} = 7.33 \le \frac{253}{\sqrt{36}} = 42.2 \therefore \text{ok}$$

unstiffened element

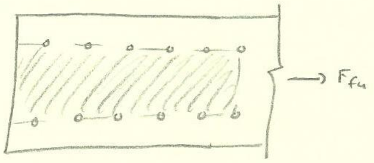
$$\frac{b}{t_p} \le \frac{95}{\sqrt{F_y}}$$

$$\frac{3\,1/4}{3/4} = 4.33 \le \frac{95}{\sqrt{36}} = 15.8 \therefore \text{ok}$$

- check beam flange block shear



case 1



case 2

by inspection, case 2 is more critical since it results in a smaller block shear area

use $l_{eh} = 2.75"$ and $l_{ev} = 1.5"$

T 9.3a : 110 K/in

T 9.3b : 371 K/in

T 9.3c : 322 K/in

$$\phi R_n = 0.75(110 + 322) \text{ K/in } (2) = 648 \text{ K} > F_{fu} = 241 \text{ K} \therefore \text{ok}$$

$\uparrow$ # of shear planes

- check beam flange bolt shear / bearing / tear-out

  bolt shear, $\phi r_n = 24.3 \text{ K/bolt}$ ← all bolts

  bearing, $\phi r_n = 102 \text{ K/in/bolt } (0.75 \text{ in}) = 76.5 \text{ K/bolt}$ ← all bolts

  $\uparrow$ from table 7-4

  tear-out, $\phi r_n = 0.75(1.2)[1.5 - 0.5(7/8 + 1/16)](65)(0.75) = 45.2$ ← bolts 1 and 2

$\phi R_n = 24.3 \text{ K/bolt } (12)$

$= 291.6 \text{ K} > 241 \text{ K} \therefore \text{ok}$

Typical Moment Frame

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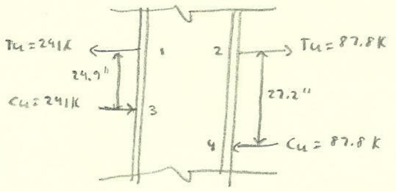
Design the column side stiffener

member information : column W14X257 , $d = 16.4''$, $t_w = 1.18''$, $b_f = 16''$, $t_f = 1.89''$, $K_{design} = 2.49''$

$h/t_w = 7.41''$, $A = 75.6 \text{ in}^2$



$Mu_{left} = 499 \text{ Kft}$

$Mu_{right} = 199 \text{ Kft}$

column flange bending :

$$\phi R_n = 0.9(6.25)t_{fc}^2 F_{yc} = 0.9(6.25)(1.89)^2(50) = 1005 K > Tu = 241K$$

$$> Tu = 87.8K$$

∴ do not need half depth stiffeners @ 1 and 2 for this loading condition

column web yielding

$$\phi R_n = 1.0(5K_{des} + t_p)F_{yc}t_{wc} = 1.0(5(2.49)+0.5)(50)(1.18) = 764.1 K > Tu_{right} \text{ and } Cu_{right} = 87.8K$$

$$= 1.0(5(2.49)+0.75)(50)(1.18) = 778.8 K > Tu_{left} \text{ and } Cu_{left} = 241 K$$

∴ do not need half depth stiffeners @ 1, 2, 3, or 4 for this limit state

column web crippling

$$\phi R_n = 0.75(0.80)t_{wc}^2\left[1 + 3\left(\frac{\overset{t_p \text{ for this problem}}{\ell_b}}{d_c}\right)\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_{fc}}{t_{wc}}}$$

for 1/2" thick plate :

$$= 0.75(0.80)(1.18)^2\left[1 + 3\left(\frac{0.5}{16.4}\right)\left(\frac{1.18}{1.89}\right)^{1.5}\right]\sqrt{\frac{29000(50)(1.89)}{1.18}} = 1273(1.045) = 1330.3 > Cu_{right} = 87.8K$$

∴ do not need half depth stiffener at 4 (compression flange)

for 3/4" thick plate:

$= 1273(1.068) = 1359 K > Cu_{right} = 241 K$ ∴ do not need half depth stiffener at 3

Typical moment
connection

page 11 of 11

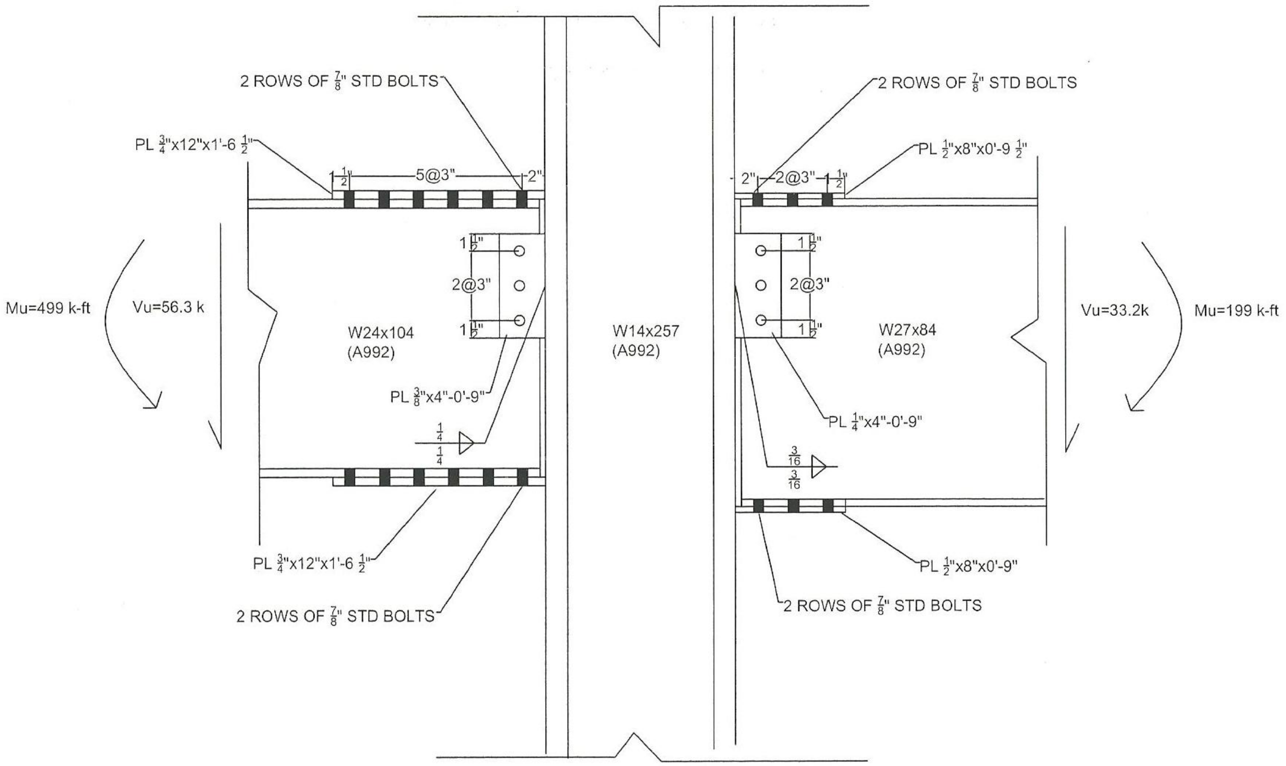
column web buckling

$$\phi R_n = \frac{0.9(24)t_{wc}\sqrt{EF_{yc}}}{h/t_{wc}} = \frac{0.9(24)(1.18)^2\sqrt{29000(50)}}{9.41} = 3848.7\text{ K} > Cu_{max} = 241\text{ K}$$

∴ do not need full depth stiffeners for this limit state

the w14x257 column subjected to the design loads do not require stiffeners

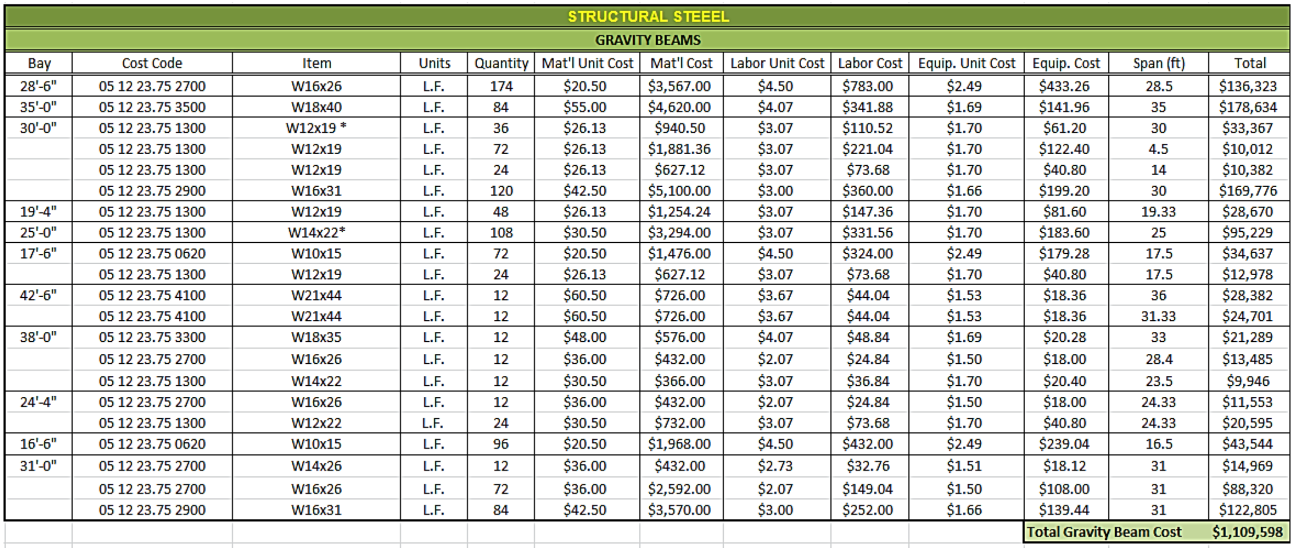
- refer to autoCAD drawing for final design



TYPICAL FLANGE PLATE BOLTED/WEB BOLTED
MOMENT CONNECTION

Appendix E: Construction Management Breadth Analysis

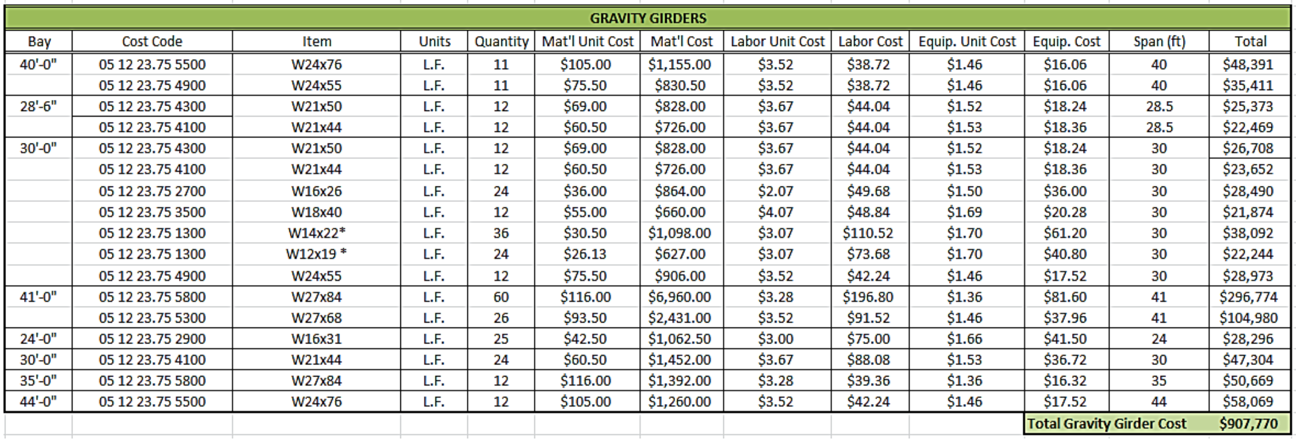
Detailed Superstructure Cost



STRUCTURAL STEEEL

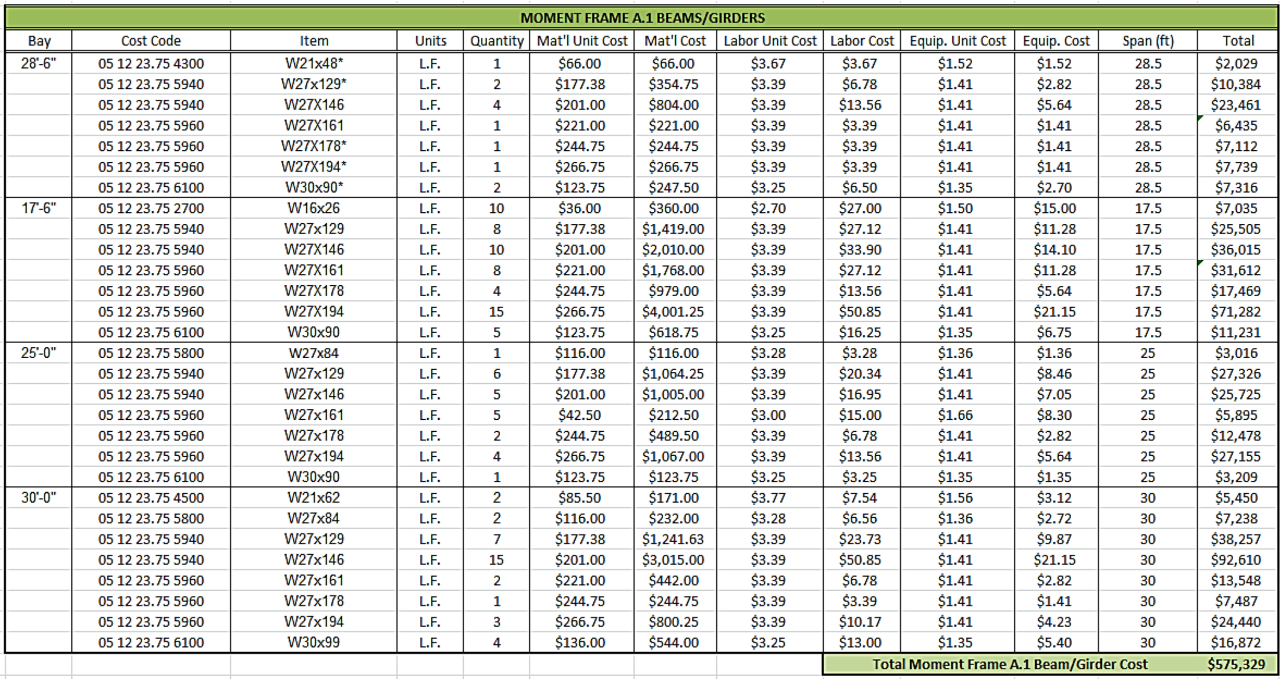
GRAVITY BEAMS

Bay	Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Span (ft)	Total
28'-6"	05 12 23.75 2700	W16x26	L.F.	174	$20.50	$3,567.00	$4.50	$783.00	$2.49	$433.26	28.5	$136,323
35'-0"	05 12 23.75 3500	W18x40	L.F.	84	$55.00	$4,620.00	$4.07	$341.88	$1.69	$141.96	35	$178,634
30'-0"	05 12 23.75 1300	W12x19 *	L.F.	36	$26.13	$940.50	$3.07	$110.52	$1.70	$61.20	30	$33,367
	05 12 23.75 1300	W12x19	L.F.	72	$26.13	$1,881.36	$3.07	$221.04	$1.70	$122.40	4.5	$10,012
	05 12 23.75 1300	W12x19	L.F.	24	$26.13	$627.12	$3.07	$73.68	$1.70	$40.80	14	$10,382
	05 12 23.75 2900	W16x31	L.F.	120	$42.50	$5,100.00	$3.00	$360.00	$1.66	$199.20	30	$169,776
19'-4"	05 12 23.75 1300	W12x19	L.F.	48	$26.13	$1,254.24	$3.07	$147.36	$1.70	$81.60	19.33	$28,670
25'-0"	05 12 23.75 1300	W14x22*	L.F.	108	$30.50	$3,294.00	$3.07	$331.56	$1.70	$183.60	25	$95,229
17'-6"	05 12 23.75 0620	W10x15	L.F.	72	$20.50	$1,476.00	$4.50	$324.00	$2.49	$179.28	17.5	$34,637
	05 12 23.75 1300	W12x19	L.F.	24	$26.13	$627.12	$3.07	$73.68	$1.70	$40.80	17.5	$12,978
42'-6"	05 12 23.75 4100	W21x44	L.F.	12	$60.50	$726.00	$3.67	$44.04	$1.53	$18.36	36	$28,382
	05 12 23.75 4100	W21x44	L.F.	12	$60.50	$726.00	$3.67	$44.04	$1.53	$18.36	31.33	$24,701
38'-0"	05 12 23.75 3300	W18x35	L.F.	12	$48.00	$576.00	$4.07	$48.84	$1.69	$20.28	33	$21,289
	05 12 23.75 2700	W16x26	L.F.	12	$36.00	$432.00	$2.07	$24.84	$1.50	$18.00	28.4	$13,485
	05 12 23.75 1300	W14x22	L.F.	12	$30.50	$366.00	$3.07	$36.84	$1.70	$20.40	23.5	$9,946
24'-4"	05 12 23.75 2700	W16x26	L.F.	12	$36.00	$432.00	$2.07	$24.84	$1.50	$18.00	24.33	$11,553
	05 12 23.75 1300	W12x22	L.F.	24	$30.50	$732.00	$3.07	$73.68	$1.70	$40.80	24.33	$20,595
16'-6"	05 12 23.75 0620	W10x15	L.F.	96	$20.50	$1,968.00	$4.50	$432.00	$2.49	$239.04	16.5	$43,544
31'-0"	05 12 23.75 2700	W14x26	L.F.	12	$36.00	$432.00	$2.73	$32.76	$1.51	$18.12	31	$14,969
	05 12 23.75 2700	W16x26	L.F.	72	$36.00	$2,592.00	$2.07	$149.04	$1.50	$108.00	31	$88,320
	05 12 23.75 2900	W16x31	L.F.	84	$42.50	$3,570.00	$3.00	$252.00	$1.66	$139.44	31	$122,805
										Total Gravity Beam Cost		$1,109,598



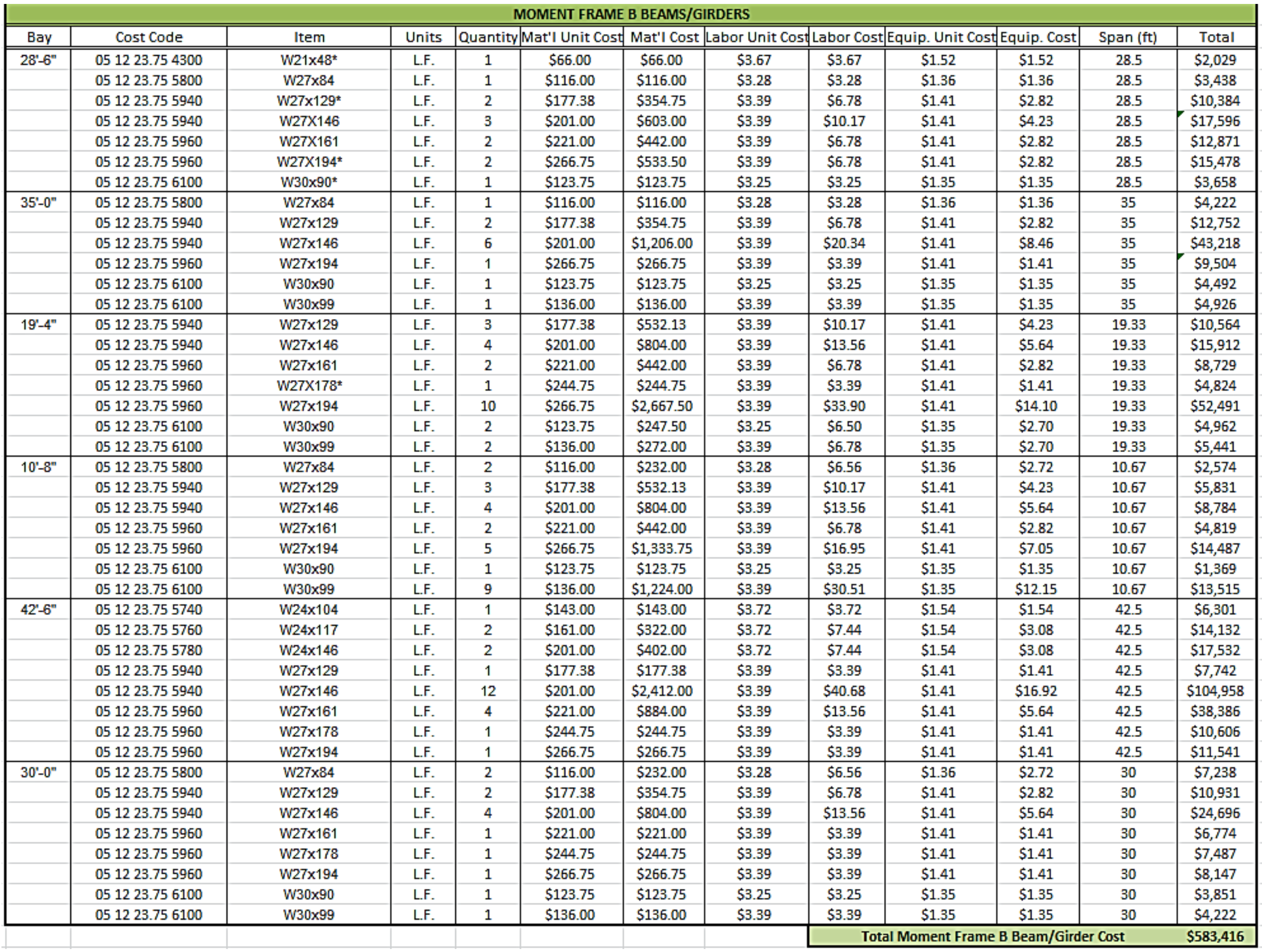
GRAVITY GIRDERS

Bay	Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Span (ft)	Total
40'-0"	05 12 23.75 5500	W24x76	L.F.	11	$105.00	$1,155.00	$3.52	$38.72	$1.46	$16.06	40	$48,391
	05 12 23.75 4900	W24x55	L.F.	11	$75.50	$830.50	$3.52	$38.72	$1.46	$16.06	40	$35,411
28'-6"	05 12 23.75 4300	W21x50	L.F.	12	$69.00	$828.00	$3.67	$44.04	$1.52	$18.24	28.5	$25,373
	05 12 23.75 4100	W21x44	L.F.	12	$60.50	$726.00	$3.67	$44.04	$1.53	$18.36	28.5	$22,469
30'-0"	05 12 23.75 4300	W21x50	L.F.	12	$69.00	$828.00	$3.67	$44.04	$1.52	$18.24	30	$26,708
	05 12 23.75 4100	W21x44	L.F.	12	$60.50	$726.00	$3.67	$44.04	$1.53	$18.36	30	$23,652
	05 12 23.75 2700	W16x26	L.F.	24	$36.00	$864.00	$2.07	$49.68	$1.50	$36.00	30	$28,490
	05 12 23.75 3500	W18x40	L.F.	12	$55.00	$660.00	$4.07	$48.84	$1.69	$20.28	30	$21,874
	05 12 23.75 1300	W14x22*	L.F.	36	$30.50	$1,098.00	$3.07	$110.52	$1.70	$61.20	30	$38,092
	05 12 23.75 1300	W12x19 *	L.F.	24	$26.13	$627.00	$3.07	$73.68	$1.70	$40.80	30	$22,244
	05 12 23.75 4900	W24x55	L.F.	12	$75.50	$906.00	$3.52	$42.24	$1.46	$17.52	30	$28,973
41'-0"	05 12 23.75 5800	W27x84	L.F.	60	$116.00	$6,960.00	$3.28	$196.80	$1.36	$81.60	41	$296,774
	05 12 23.75 5300	W27x68	L.F.	26	$93.50	$2,431.00	$3.52	$91.52	$1.46	$37.96	41	$104,980
24'-0"	05 12 23.75 2900	W16x31	L.F.	25	$42.50	$1,062.50	$3.00	$75.00	$1.66	$41.50	24	$28,296
30'-0"	05 12 23.75 4100	W21x44	L.F.	24	$60.50	$1,452.00	$3.67	$88.08	$1.53	$36.72	30	$47,304
35'-0"	05 12 23.75 5800	W27x84	L.F.	12	$116.00	$1,392.00	$3.28	$39.36	$1.36	$16.32	35	$50,669
44'-0"	05 12 23.75 5500	W24x76	L.F.	12	$105.00	$1,260.00	$3.52	$42.24	$1.46	$17.52	44	$58,069
										Total Gravity Girder Cost		$907,770



MOMENT FRAME A.1 BEAMS/GIRDERS

Bay	Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Span (ft)	Total
28'-6"	05 12 23.75 4300	W21x48*	L.F.	1	$66.00	$66.00	$3.67	$3.67	$1.52	$1.52	28.5	$2,029
	05 12 23.75 5940	W27x129*	L.F.	2	$177.38	$354.75	$3.39	$6.78	$1.41	$2.82	28.5	$10,384
	05 12 23.75 5940	W27X146	L.F.	4	$201.00	$804.00	$3.39	$13.56	$1.41	$5.64	28.5	$23,461
	05 12 23.75 5960	W27X161	L.F.	1	$221.00	$221.00	$3.39	$3.39	$1.41	$1.41	28.5	$6,435
	05 12 23.75 5960	W27X178*	L.F.	1	$244.75	$244.75	$3.39	$3.39	$1.41	$1.41	28.5	$7,112
	05 12 23.75 5960	W27X194*	L.F.	1	$266.75	$266.75	$3.39	$3.39	$1.41	$1.41	28.5	$7,739
	05 12 23.75 6100	W30x90*	L.F.	2	$123.75	$247.50	$3.25	$6.50	$1.35	$2.70	28.5	$7,316
17'-6"	05 12 23.75 2700	W16x26	L.F.	10	$36.00	$360.00	$2.70	$27.00	$1.50	$15.00	17.5	$7,035
	05 12 23.75 5940	W27x129	L.F.	8	$177.38	$1,419.00	$3.39	$27.12	$1.41	$11.28	17.5	$25,505
	05 12 23.75 5940	W27X146	L.F.	10	$201.00	$2,010.00	$3.39	$33.90	$1.41	$14.10	17.5	$36,015
	05 12 23.75 5960	W27X161	L.F.	8	$221.00	$1,768.00	$3.39	$27.12	$1.41	$11.28	17.5	$31,612
	05 12 23.75 5960	W27X178	L.F.	4	$244.75	$979.00	$3.39	$13.56	$1.41	$5.64	17.5	$17,469
	05 12 23.75 5960	W27X194	L.F.	15	$266.75	$4,001.25	$3.39	$50.85	$1.41	$21.15	17.5	$71,282
	05 12 23.75 6100	W30x90	L.F.	5	$123.75	$618.75	$3.25	$16.25	$1.35	$6.75	17.5	$11,231
25'-0"	05 12 23.75 5800	W27x84	L.F.	1	$116.00	$116.00	$3.28	$3.28	$1.36	$1.36	25	$3,016
	05 12 23.75 5940	W27x129	L.F.	6	$177.38	$1,064.25	$3.39	$20.34	$1.41	$8.46	25	$27,326
	05 12 23.75 5940	W27x146	L.F.	5	$201.00	$1,005.00	$3.39	$16.95	$1.41	$7.05	25	$25,725
	05 12 23.75 5960	W27x161	L.F.	5	$42.50	$212.50	$3.00	$15.00	$1.66	$8.30	25	$5,895
	05 12 23.75 5960	W27x178	L.F.	2	$244.75	$489.50	$3.39	$6.78	$1.41	$2.82	25	$12,478
	05 12 23.75 5960	W27x194	L.F.	4	$266.75	$1,067.00	$3.39	$13.56	$1.41	$5.64	25	$27,155
	05 12 23.75 6100	W30x90	L.F.	1	$123.75	$123.75	$3.25	$3.25	$1.35	$1.35	25	$3,209
30'-0"	05 12 23.75 4500	W21x62	L.F.	2	$85.50	$171.00	$3.77	$7.54	$1.56	$3.12	30	$5,450
	05 12 23.75 5800	W27x84	L.F.	2	$116.00	$232.00	$3.28	$6.56	$1.36	$2.72	30	$7,238
	05 12 23.75 5940	W27x129	L.F.	7	$177.38	$1,241.63	$3.39	$23.73	$1.41	$9.87	30	$38,257
	05 12 23.75 5940	W27x146	L.F.	15	$201.00	$3,015.00	$3.39	$50.85	$1.41	$21.15	30	$92,610
	05 12 23.75 5960	W27x161	L.F.	2	$221.00	$442.00	$3.39	$6.78	$1.41	$2.82	30	$13,548
	05 12 23.75 5960	W27x178	L.F.	1	$244.75	$244.75	$3.39	$3.39	$1.41	$1.41	30	$7,487
	05 12 23.75 5960	W27x194	L.F.	3	$266.75	$800.25	$3.39	$10.17	$1.41	$4.23	30	$24,440
	05 12 23.75 6100	W30x99	L.F.	4	$136.00	$544.00	$3.25	$13.00	$1.35	$5.40	30	$16,872
								Total Moment Frame A.1 Beam/Girder Cost				**$575,329**



MOMENT FRAME B BEAMS/GIRDERS

Bay	Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Span (ft)	Total
28'-6"	05 12 23.75 4300	W21x48*	L.F.	1	$66.00	$66.00	$3.67	$3.67	$1.52	$1.52	28.5	$2,029
	05 12 23.75 5800	W27x84	L.F.	1	$116.00	$116.00	$3.28	$3.28	$1.36	$1.36	28.5	$3,438
	05 12 23.75 5940	W27x129*	L.F.	2	$177.38	$354.75	$3.39	$6.78	$1.41	$2.82	28.5	$10,384
	05 12 23.75 5940	W27X146	L.F.	3	$201.00	$603.00	$3.39	$10.17	$1.41	$4.23	28.5	$17,596
	05 12 23.75 5960	W27X161	L.F.	2	$221.00	$442.00	$3.39	$6.78	$1.41	$2.82	28.5	$12,871
	05 12 23.75 5960	W27X194*	L.F.	2	$266.75	$533.50	$3.39	$6.78	$1.41	$2.82	28.5	$15,478
	05 12 23.75 6100	W30x90*	L.F.	1	$123.75	$123.75	$3.25	$3.25	$1.35	$1.35	28.5	$3,658
35'-0"	05 12 23.75 5800	W27x84	L.F.	1	$116.00	$116.00	$3.28	$3.28	$1.36	$1.36	35	$4,222
	05 12 23.75 5940	W27x129	L.F.	2	$177.38	$354.75	$3.39	$6.78	$1.41	$2.82	35	$12,752
	05 12 23.75 5940	W27x146	L.F.	6	$201.00	$1,206.00	$3.39	$20.34	$1.41	$8.46	35	$43,218
	05 12 23.75 5960	W27x194	L.F.	1	$266.75	$266.75	$3.39	$3.39	$1.41	$1.41	35	$9,504
	05 12 23.75 6100	W30x90	L.F.	1	$123.75	$123.75	$3.25	$3.25	$1.35	$1.35	35	$4,492
	05 12 23.75 6100	W30x99	L.F.	1	$136.00	$136.00	$3.39	$3.39	$1.35	$1.35	35	$4,926
19'-4"	05 12 23.75 5940	W27x129	L.F.	3	$177.38	$532.13	$3.39	$10.17	$1.41	$4.23	19.33	$10,564
	05 12 23.75 5940	W27x146	L.F.	4	$201.00	$804.00	$3.39	$13.56	$1.41	$5.64	19.33	$15,912
	05 12 23.75 5960	W27x161	L.F.	2	$221.00	$442.00	$3.39	$6.78	$1.41	$2.82	19.33	$8,729
	05 12 23.75 5960	W27X178*	L.F.	1	$244.75	$244.75	$3.39	$3.39	$1.41	$1.41	19.33	$4,824
	05 12 23.75 5960	W27x194	L.F.	10	$266.75	$2,667.50	$3.39	$33.90	$1.41	$14.10	19.33	$52,491
	05 12 23.75 6100	W30x90	L.F.	2	$123.75	$247.50	$3.25	$6.50	$1.35	$2.70	19.33	$4,962
	05 12 23.75 6100	W30x99	L.F.	2	$136.00	$272.00	$3.39	$6.78	$1.35	$2.70	19.33	$5,441
10'-8"	05 12 23.75 5800	W27x84	L.F.	2	$116.00	$232.00	$3.28	$6.56	$1.36	$2.72	10.67	$2,574
	05 12 23.75 5940	W27x129	L.F.	3	$177.38	$532.13	$3.39	$10.17	$1.41	$4.23	10.67	$5,831
	05 12 23.75 5940	W27x146	L.F.	4	$201.00	$804.00	$3.39	$13.56	$1.41	$5.64	10.67	$8,784
	05 12 23.75 5960	W27x161	L.F.	2	$221.00	$442.00	$3.39	$6.78	$1.41	$2.82	10.67	$4,819
	05 12 23.75 5960	W27x194	L.F.	5	$266.75	$1,333.75	$3.39	$16.95	$1.41	$7.05	10.67	$14,487
	05 12 23.75 6100	W30x90	L.F.	1	$123.75	$123.75	$3.25	$3.25	$1.35	$1.35	10.67	$1,369
	05 12 23.75 6100	W30x99	L.F.	9	$136.00	$1,224.00	$3.39	$30.51	$1.35	$12.15	10.67	$13,515
42'-6"	05 12 23.75 5740	W24x104	L.F.	1	$143.00	$143.00	$3.72	$3.72	$1.54	$1.54	42.5	$6,301
	05 12 23.75 5760	W24x117	L.F.	2	$161.00	$322.00	$3.72	$7.44	$1.54	$3.08	42.5	$14,132
	05 12 23.75 5780	W24x146	L.F.	2	$201.00	$402.00	$3.72	$7.44	$1.54	$3.08	42.5	$17,532
	05 12 23.75 5940	W27x129	L.F.	1	$177.38	$177.38	$3.39	$3.39	$1.41	$1.41	42.5	$7,742
	05 12 23.75 5940	W27x146	L.F.	12	$201.00	$2,412.00	$3.39	$40.68	$1.41	$16.92	42.5	$104,958
	05 12 23.75 5960	W27x161	L.F.	4	$221.00	$884.00	$3.39	$13.56	$1.41	$5.64	42.5	$38,386
	05 12 23.75 5960	W27x178	L.F.	1	$244.75	$244.75	$3.39	$3.39	$1.41	$1.41	42.5	$10,606
	05 12 23.75 5960	W27x194	L.F.	1	$266.75	$266.75	$3.39	$3.39	$1.41	$1.41	42.5	$11,541
30'-0"	05 12 23.75 5800	W27x84	L.F.	2	$116.00	$232.00	$3.28	$6.56	$1.36	$2.72	30	$7,238
	05 12 23.75 5940	W27x129	L.F.	2	$177.38	$354.75	$3.39	$6.78	$1.41	$2.82	30	$10,931
	05 12 23.75 5940	W27x146	L.F.	4	$201.00	$804.00	$3.39	$13.56	$1.41	$5.64	30	$24,696
	05 12 23.75 5960	W27x161	L.F.	1	$221.00	$221.00	$3.39	$3.39	$1.41	$1.41	30	$6,774
	05 12 23.75 5960	W27x178	L.F.	1	$244.75	$244.75	$3.39	$3.39	$1.41	$1.41	30	$7,487
	05 12 23.75 5960	W27x194	L.F.	1	$266.75	$266.75	$3.39	$3.39	$1.41	$1.41	30	$8,147
	05 12 23.75 6100	W30x90	L.F.	1	$123.75	$123.75	$3.25	$3.25	$1.35	$1.35	30	$3,851
	05 12 23.75 6100	W30x99	L.F.	1	$136.00	$136.00	$3.39	$3.39	$1.35	$1.35	30	$4,222
								Total Moment Frame B Beam/Girder Cost				**$583,416**

MOMENT FRAME C BEAMS/GIRDERS

Bay	Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Span (ft)	Total
28'-6"	05 12 23.75 4300	W21x48*	L.F.	1	$66.00	$66.00	$3.67	$3.67	$1.52	$1.52	28.5	$2,029
	05 12 23.75 5940	W27x129*	L.F.	2	$177.38	$354.75	$3.39	$6.78	$1.41	$2.82	28.5	$10,384
	05 12 23.75 5940	W27X146	L.F.	3	$201.00	$603.00	$3.39	$10.17	$1.41	$4.23	28.5	$17,596
	05 12 23.75 5960	W27X161	L.F.	1	$221.00	$221.00	$3.39	$3.39	$1.41	$1.41	28.5	$6,435
	05 12 23.75 5960	W27X178*	L.F.	1	$244.75	$244.75	$3.39	$3.39	$1.41	$1.41	28.5	$7,112
	05 12 23.75 5960	W27X194*	L.F.	2	$266.75	$533.50	$3.39	$6.78	$1.41	$2.82	28.5	$15,478
	05 12 23.75 6100	W30x90*	L.F.	1	$123.75	$123.75	$3.25	$3.25	$1.35	$1.35	28.5	$3,658
	05 12 23.75 6100	W30x99	L.F.	1	$136.00	$136.00	$3.25	$3.25	$1.35	$1.35	28.5	$4,007
35'-0"	05 12 23.75 5800	W27x84	L.F.	1	$116.00	$116.00	$3.28	$3.28	$1.36	$1.36	35	$4,222
	05 12 23.75 5940	W27x129	L.F.	3	$177.38	$532.13	$3.39	$10.17	$1.41	$4.23	35	$19,128
	05 12 23.75 5940	W27x146	L.F.	5	$201.00	$1,005.00	$3.39	$16.95	$1.41	$7.05	35	$36,015
	05 12 23.75 5960	W27x161	L.F.	1	$221.00	$221.00	$3.39	$3.39	$1.41	$1.41	35	$7,903
	05 12 23.75 5960	W27x194	L.F.	1	$266.75	$266.75	$3.39	$3.39	$1.41	$1.41	35	$9,504
	05 12 23.75 6100	W30x90	L.F.	1	$123.75	$123.75	$3.25	$3.25	$1.35	$1.35	35	$4,492
19'-4"	05 12 23.75 5800	W27x84	L.F.	1	$116.00	$116.00	$3.28	$3.28	$1.36	$1.36	19.33	$2,332
	05 12 23.75 5940	W27x129	L.F.	3	$177.38	$532.13	$3.39	$10.17	$1.41	$4.23	19.33	$10,564
	05 12 23.75 5940	W27x146	L.F.	6	$201.00	$1,206.00	$3.39	$20.34	$1.41	$8.46	19.33	$23,869
	05 12 23.75 5960	W27x161	L.F.	2	$221.00	$442.00	$3.39	$6.78	$1.41	$2.82	19.33	$8,729
	05 12 23.75 5960	W27X178	L.F.	2	$244.75	$489.50	$3.39	$6.78	$1.41	$2.82	19.33	$9,648
	05 12 23.75 5960	W27x194	L.F.	7	$266.75	$1,867.25	$3.39	$23.73	$1.41	$9.87	19.33	$36,743
	05 12 23.75 6100	W30x90	L.F.	1	$123.75	$123.75	$3.25	$3.25	$1.35	$1.35	19.33	$2,481
	05 12 23.75 6100	W30x99	L.F.	2	$136.00	$272.00	$3.39	$6.78	$1.35	$2.70	19.33	$5,441
10'-8"	05 12 23.75 5800	W27x84	L.F.	2	$116.00	$232.00	$3.28	$6.56	$1.36	$2.72	10.67	$2,574
	05 12 23.75 5940	W27x129	L.F.	2	$177.38	$354.75	$3.39	$6.78	$1.41	$2.82	10.67	$3,888
	05 12 23.75 5940	W27x146	L.F.	3	$201.00	$603.00	$3.39	$10.17	$1.41	$4.23	10.67	$6,588
	05 12 23.75 5960	W27x161	L.F.	1	$221.00	$221.00	$3.39	$3.39	$1.41	$1.41	10.67	$2,409
	05 12 23.75 5960	W27x194	L.F.	12	$266.75	$3,201.00	$3.39	$40.68	$1.41	$16.92	10.67	$34,769
	05 12 23.75 6100	W30x90	L.F.	2	$123.75	$247.50	$3.25	$6.50	$1.35	$2.70	10.67	$2,739
	05 12 23.75 6100	W30x99	L.F.	4	$136.00	$544.00	$3.39	$13.56	$1.35	$5.40	10.67	$6,007
42'-6"	05 12 23.75 5760	W24x117	L.F.	2	$161.00	$322.00	$3.72	$7.44	$1.54	$3.08	42.5	$14,132
	05 12 23.75 5780	W24x131*	L.F.	1	$180.13	$180.13	$3.72	$3.72	$1.54	$1.54	42.5	$7,879
	05 12 23.75 5780	W24x146	L.F.	3	$201.00	$603.00	$3.72	$11.16	$1.54	$4.62	42.5	$26,298
	05 12 23.75 5940	W27x146	L.F.	5	$201.00	$1,005.00	$3.39	$16.95	$1.41	$7.05	42.5	$43,733
	05 12 23.75 5960	W27x194	L.F.	1	$266.75	$266.75	$3.39	$3.39	$1.41	$1.41	42.5	$11,541
30'-0"	05 12 23.75 5800	W27x84	L.F.	2	$116.00	$232.00	$3.28	$6.56	$1.36	$2.72	30	$7,238
	05 12 23.75 5940	W27x129	L.F.	2	$177.38	$354.75	$3.39	$6.78	$1.41	$2.82	30	$10,931
	05 12 23.75 5940	W27x146	L.F.	4	$201.00	$804.00	$3.39	$13.56	$1.41	$5.64	30	$24,696
	05 12 23.75 5960	W27x161	L.F.	2	$221.00	$442.00	$3.39	$6.78	$1.41	$2.82	30	$13,548
	05 12 23.75 5960	W27x194	L.F.	2	$266.75	$533.50	$3.39	$6.78	$1.41	$2.82	30	$16,293
	05 12 23.75 6100	W30x99	L.F.	1	$136.00	$136.00	$3.39	$3.39	$1.35	$1.35	30	$4,222
								Total Moment Frame C Beam/Girder Cost				**$487,257**

MOMENT FRAME E BEAMS/GIRDERS

Bay	Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Span (ft)	Total
28'-6"	05 12 23.75 3900	W21x55*	L.F.	1	$75.50	$75.50	$4.28	$4.28	$1.77	$1.77	28.5	$2,324
	05 12 23.75 5800	W27x84	L.F.	1	$116.00	$116.00	$3.28	$3.28	$1.36	$1.36	28.5	$3,438
	05 12 23.75 5940	W27x129*	L.F.	3	$177.38	$532.13	$3.39	$10.17	$1.41	$4.23	28.5	$15,576
	05 12 23.75 5940	W27X146	L.F.	5	$201.00	$1,005.00	$3.39	$16.95	$1.41	$7.05	28.5	$29,327
	05 12 23.75 5960	W27X161	L.F.	3	$221.00	$663.00	$3.39	$10.17	$1.41	$4.23	28.5	$19,306
	05 12 23.75 5960	W27X178*	L.F.	3	$244.75	$734.25	$3.39	$10.17	$1.41	$4.23	28.5	$21,337
	05 12 23.75 5960	W27X194*	L.F.	4	$266.75	$1,067.00	$3.39	$13.56	$1.41	$5.64	28.5	$30,957
	05 12 23.75 6100	W30x99	L.F.	2	$136.00	$272.00	$3.25	$6.50	$1.35	$2.70	28.5	$8,014
								Total Moment Frame E Beam/Girder Cost				**$130,278**

MOMENT FRAME 1 BEAMS/GIRDERS

Bay	Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Span (ft)	Total
41'-0"	05 12 23.75 5740	W24x104	L.F.	11	$143.00	$1,573.00	$3.72	$40.92	$1.54	$16.94	41	$66,865
	05 12 23.75 5780	W24x131*	L.F.	1	$180.13	$180.13	$3.72	$3.72	$1.54	$1.54	41	$7,601
30'-0"	05 12 23.75 3900	W21x55*	L.F.	1	$75.50	$75.50	$4.28	$4.28	$1.77	$1.77	30	$2,447
	05 12 23.75 5800	W27x84	L.F.	11	$116.00	$1,276.00	$3.28	$36.08	$1.36	$14.96	30	$39,811
28'-6"	05 12 23.75 5800	W27x84	L.F.	12	$116.00	$1,392.00	$3.28	$39.36	$1.36	$16.32	28.5	$41,259
40'-0"	05 12 23.75 5740	W24x104	L.F.	11	$143.00	$1,573.00	$3.72	$40.92	$1.54	$16.94	40	$65,234
										Total Gravity Girder Cost		**$223,217**

MOMENT FRAME 1' BEAMS/GIRDERS

Bay	Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Span (ft)	Total
35'-0"	05 12 23.75 5800	W27x84	L.F.	3	$116.00	$348.00	$3.28	$9.84	$1.36	$4.08	35	$12,667
	05 12 23.75 5940	W27x129*	L.F.	7	$177.38	$1,241.63	$3.39	$23.73	$1.41	$9.87	35	$44,633
	05 12 23.75 6100	W30x99	L.F.	2	$136.00	$272.00	$3.39	$6.78	$1.35	$2.70	35	$9,852
30'-0"	05 12 23.75 5800	W27x84	L.F.	5	$116.00	$580.00	$3.28	$16.40	$1.36	$6.80	30	$18,096
	05 12 23.75 5940	W27x129	L.F.	14	$177.38	$2,483.25	$3.39	$47.46	$1.41	$19.74	30	$76,514
	05 12 23.75 6100	W30x90*	L.F.	2	$123.75	$247.50	$3.25	$6.50	$1.35	$2.70	30	$7,701
	05 12 23.75 6100	W30x99	L.F.	3	$136.00	$408.00	$3.25	$9.75	$1.35	$4.05	30	$12,654
24'-0"	05 12 23.75 5800	W27x84	L.F.	3	$116.00	$348.00	$3.28	$9.84	$1.36	$4.08	24	$8,686
	05 12 23.75 5940	W27x129	L.F.	3	$177.38	$532.13	$3.39	$10.17	$1.41	$4.23	24	$13,117
	05 12 23.75 5940	W27x146	L.F.	4	$201.00	$804.00	$3.39	$13.56	$1.41	$5.64	24	$19,757
	05 12 23.75 6100	W30x99	L.F.	2	$136.00	$272.00	$3.25	$6.50	$1.35	$2.70	24	$6,749
										Total Gravity Girder Cost		**$230,425**

GRAVITY COLUMNS											
Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Span (ft)	Total
05 12 23.75 2320	W14x43	L.F.	23	$59.00	$1,357.00	$3.34	$76.82	$1.85	$42.55	30	$44,291
05 12 23.17 7350	W14x48*	L.F.	4	$66.00	$264.00	$2.75	$11.00	$1.52	$6.08	30	$8,432
05 12 23.75 2340	W14x53	L.F.	5	$73.00	$365.00	$3.38	$16.90	$1.87	$9.35	30	$11,738
05 12 23.17 7350	W14x61*	L.F.	16	$83.88	$1,342.00	$2.75	$44.00	$1.52	$24.32	30	$42,310
05 12 23.17 7350	W14x68*	L.F.	6	$93.50	$561.00	$2.75	$16.50	$1.52	$9.12	30	$17,599
05 12 23.17 7350	W14x74	L.F.	2	$102.00	$204.00	$2.75	$5.50	$1.52	$3.04	30	$6,376
05 12 23.17 7400	W14x82*	L.F.	5	$112.75	$563.75	$2.82	$14.10	$1.56	$7.80	30	$17,570
05 12 23.75 2380	W14x90	L.F.	8	$124.00	$992.00	$3.65	$29.20	$2.02	$16.16	30	$31,121
05 12 23.75 6100	W14x99	L.F.	4	$136.00	$544.00	$3.25	$13.00	$1.35	$5.40	30	$16,872
05 12 23.17 7400	W14x109*	L.F.	4	$149.88	$599.50	$2.82	$11.28	$1.56	$6.24	30	$18,511
05 12 23.17 7400	W14x120	L.F.	2	$165.00	$330.00	$2.82	$5.64	$1.56	$3.12	30	$10,163
05 12 23.17 7450	W14x132*	L.F.	4	$181.50	$726.00	$2.96	$11.84	$1.64	$6.56	30	$22,332
05 12 23.17 7450	W14x145*	L.F.	3	$199.38	$598.13	$2.96	$8.88	$1.64	$4.92	30	$18,358
05 12 23.17 7450	W14x159*	L.F.	2	$218.63	$437.25	$2.96	$5.92	$1.64	$3.28	30	$13,394
05 12 23.17 7450	W14x193*	L.F.	1	$265.38	$265.38	$2.96	$2.96	$1.64	$1.64	30	$8,099
									Total Gravity Column Cost		**$287,164**

MOMENT FRAME COLUMNS											
Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Span (ft)	Total
05 12 23.17 7350	W14x43*	L.F.	1	$59.13	$59.13	$2.75	$2.75	$1.52	$1.52	30	$1,902
05 12 23.17 7450	W14x193*	L.F.	53	$265.38	$14,065	$2.96	$156.88	$1.64	$86.92	30	$429,260
05 12 23.17 7450	W14x211*	L.F.	19	$290.13	$5,512.38	$2.96	$56.24	$1.64	$31.16	30	$167,993
05 12 23.17 7450	W14x233*	L.F.	23	$320.38	$7,368.63	$2.96	$68.08	$1.64	$37.72	30	$224,233
05 12 23.17 7450	W14x257*	L.F.	54	$353.38	$19,082	$2.96	$159.84	$1.64	$88.56	30	$579,920
05 12 23.17 7450	W14x283*	L.F.	10	$389.13	$3,891.25	$2.96	$29.60	$1.64	$16.40	30	$118,118
05 12 23.17 7450	W14x311*	L.F.	46	$427.63	$19,671	$2.96	$136.16	$1.64	$75.44	30	$596,471
05 12 23.17 7450	W14x342*	L.F.	6	$470.25	$2,821.50	$2.96	$17.76	$1.64	$9.84	30	$85,473
05 12 23.17 7450	W14x370*	L.F.	2	$508.75	$1,017.50	$2.96	$5.92	$1.64	$3.28	30	$30,801
05 12 23.17 7450	W14x500*	L.F.	1	$687.50	$687.50	$2.96	$2.96	$1.64	$1.64	30	$20,763
05 12 23.17 7450	W14x550*	L.F.	4	$756.25	$3,025.00	$2.96	$11.84	$1.64	$6.56	30	$91,302
05 12 23.17 7450	W14x605*	L.F.	2	$831.88	$1,663.75	$2.96	$5.92	$1.64	$3.28	30	$50,189
05 12 23.17 7450	W14x665*	L.F.	8	$914.38	$7,315.00	$2.96	$23.68	$1.64	$13.12	30	$220,554
05 12 23.17 7450	W14x730*	L.F.	11	$1,003.75	$11,041	$2.96	$32.56	$1.64	$18.04	30	$332,756
								Total Moment Frame Column Cost			**$2,350,577**

BRACES											
Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Span (ft)	Total
05 12 23.75 0120	W6x15	L.F.	5	$20.50	$102.50	$4.50	$22.50	$2.49	$12.45	33.54	$4,610
05 12 23.75 0370	W8x28	L.F.	1	$38.50	$39	$4.91	$4.91	$2.72	$2.72	33.54	$1,547
05 12 23.75 0500	W8x31	L.F.	6	$42.50	$255.00	$4.91	$29.46	$2.72	$16.32	33.54	$10,088
05 12 23.75 0520	W8x35	L.F.	1	$48.00	$48.00	$4.91	$4.91	$2.72	$2.72	33.54	$1,866
05 12 23.75 0900	W10x39*	L.F.	2	$53.63	$107	$4.91	$9.82	$2.72	$5.44	33.54	$4,109
05 12 23.75 0900	W10x45*	L.F.	3	$61.88	$185.63	$4.91	$14.73	$2.72	$8.16	33.54	$6,994
05 12 23.75 0900	W10x49	L.F.	6	$67.50	$405	$4.91	$29.46	$2.72	$16.32	35.25	$15,890
05 12 23.75 0900	W10x77*	L.F.	4	$105.88	$423.50	$4.91	$19.64	$2.72	$10.88	33.54	$15,228
05 12 23.75 1520	W12x35	L.F.	1	$48.00	$48.00	$3.34	$3.34	$1.85	$1.85	33.54	$1,784
05 12 23.75 1520	W12x35	L.F.	1	$48.00	$48.00	$3.34	$3.34	$1.85	$1.85	35.25	$1,875
05 12 23.75 2340	W12x53	L.F.	3	$73.00	$219.00	$3.38	$10.14	$1.87	$5.61	33.54	$7,874
05 12 23.75 1580	W12x58	L.F.	5	$80.00	$400.00	$3.60	$18.00	$2.00	$10.00	33.54	$14,355
05 12 23.75 3920	W12x65	L.F.	3	$89.50	$268.50	$4.34	$13.02	$1.80	$5.40	33.54	$9,623
05 12 23.75 3920	W12x65	L.F.	1	$89.50	$90	$4.34	$4.34	$1.80	$1.80	35.25	$3,371
05 12 23.75 1700	W12x72	L.F.	1	$99.00	$99.00	$4.22	$4.22	$2.34	$2.34	33.54	$3,540
05 12 23.75 1740	W12x87	L.F.	2	$120.00	$240.00	$4.22	$8.44	$2.34	$4.68	33.54	$8,490
05 12 23.75 1740	W12x96*	L.F.	2	$132.00	$264.00	$4.22	$8.44	$2.34	$4.68	33.54	$9,295
05 12 23.75 3980	W12x106	L.F.	2	$60.50	$121.00	$3.67	$7.34	$1.52	$3.04	33.54	$4,406
05 12 23.75 2500	W12x120	L.F.	4	$165.00	$660.00	$3.75	$15.00	$2.08	$8.32	33.54	$22,919
05 12 23.75 1740	W12x136*	L.F.	4	$187.00	$748.00	$4.22	$16.88	$2.34	$9.36	33.54	$25,968
05 12 23.75 1740	W12x152*	L.F.	2	$209.00	$418.00	$4.22	$8.44	$2.34	$4.68	33.54	$14,460
05 12 23.75 7600	W12x170	L.F.	2	$234.00	$468.00	$3.39	$6.78	$1.41	$2.82	33.54	$16,019
05 12 23.75 1740	W12x190*	L.F.	1	$261.25	$261.25	$4.22	$4.22	$2.34	$2.34	33.54	$8,982
05 12 23.75 1740	W12x210*	L.F.	4	$288.75	$1,155.00	$4.22	$16.88	$2.34	$9.36	33.54	$39,619
05 12 23.75 1740	W12x230*	L.F.	1	$316.25	$316.25	$4.22	$4.22	$2.34	$2.34	33.54	$10,827
05 12 23.75 1740	W12x252*	L.F.	3	$346.50	$1,039.50	$4.22	$12.66	$2.34	$7.02	33.54	$35,525
05 12 23.75 1740	W12x279*	L.F.	2	$383.63	$767.25	$4.22	$8.44	$2.34	$4.68	33.54	$26,174
05 12 23.75 1740	W12x305*	L.F.	1	$419.38	$419.38	$4.22	$4.22	$2.34	$2.34	33.54	$14,286
05 12 23.75 1740	W12x336*	L.F.	23	$462.00	$10,626	$4.22	$97.06	$2.34	$53.82	33.54	$361,457
05 12 23.75 2500	W14x193*	L.F.	7	$265.38	$1,857.63	$3.75	$26.25	$2.08	$14.56	33.54	$63,674
									Total Brace Cost		**$764,853**

*- the section was unavailable in the RS means, therefore the next higher section was used as a reference. To find the material cost, the sections unit weight was multiplied by $1.375/lb

COLUMN BASE PLATE CONNECTIONS

Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Area (ft^2)	Total
05 12 23.65 0500	1"x20" plate**	S.F.	55	$51.00	$2,805.00	-	-	-	-	0.139	$389.58
05 05 23.25 1380	A490-7/8" Φ bolt**	Ea.	220	$16.35	$3,597.00	$4.39	$965.80	-	-	-	$4,562.80
								Total Column Base Plate Connections			$4,952

** Design assumption

COLUMN SPLICE CONNECTIONS

Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Area (ft^2)	Total
05 12 23.65 0450	3/4"x12" plate**	S.F.	550	$38.50	$21,175.00	-	-	-	-	0.0625	$1,323.44
05 05 23.25 1380	A490-7/8" Φ bolt**	Ea.	6600	$16.35	$107,910.00	$4.39	$28,974.00	-	-	-	$136,884.00
								Total Column Splice Connections			$138,207

** Design assumption

Braced Frame Connections

Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Area (ft^2)	Total
05 12 23.65 0300	3/8"x6"plate	S.F.	132	$19.15	$2,527.80	-	-	-	-	0.0156	$39
05 05 23.25 0220	A325-7/8" Φ bolt x 3" long	Ea.	792	$3.20	$2,534	$3.43	$2,717	-	-	-	$5,251
Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Weight (lb)	Total
05 12 23.40 0400	L4x4x1/4 angle	Lb.	5730	$0.73	$4,182.90	$2.73	$15,643	$0.28	$1,604.40	6.6	$141,439
Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Length (ft)	Total
05 05 21.90 1500	1/4" thick weld	L.F.	132	$0.63	$83	$8.20	$1,082	$2.43	$320.76	0.708	$1,053
								Total Skewed Shear Cost			$147,783

ORTHOGONAL SHEAR CONNECTIONS

Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Weight (lb)	Total
05 12 23.40 0400	L4x4x1/4 angle	Lb.	5730	$0.73	$4,182.90	$2.73	$15,643	$0.28	$1,604.40	6.6	$141,439
05 05 23.25 0220	A325-3/4" Φ bolt	Ea.	17190	$3.20	$55,008	$3.43	$58,962	-	-	-	$113,970
								Total Orthogonal Shear Cost			$255,409

SKEWED SHEAR CONNECTIONS

Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Area (ft^2)	Total
05 12 23.65 0300	3/8"x6"plate	S.F.	132	$19.15	$2,527.80	-	-	-	-	0.0156	$39
05 05 23.25 0220	A325-3/4" Φ bolt x 3" long	Ea.	792	$3.20	$2,534	$3.43	$2,717	-	-	-	$5,251
Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Length (ft)	Total
05 05 21.90 1500	1/4" thick weld	L.F.	132	$0.63	$83	$8.20	$1,082	$2.43	$320.76	0.708	$1,053
05 05 21.90 1800	3/8" thick weld	L.F.	132	$1.05	$139	$13.70	$1,808	$4.05	$534.60	0.708	$1,758
								Total Skewed Shear Cost			$8,101

MOMENT CONNECTIONS

Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Area (ft^2)	Total
05 12 23.65 0450	3/4"x12"plate	S.F.	1980	$38.50	$76,230.00	-	-	-	-	0.0625	$4,764
05 12 23.65 0300	3/8"x4"plate	S.F.	990	$19.15	$18,958.50	-	-	-	-	0.0104	$197
05 05 23.25 0365	A325-7/8" Φ bolt x 3"long	Ea.	26730	$4.41	$117,879	$3.59	$95,961	-	-	-	$213,840
Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Length (ft)	Total
05 05 21.90 1500	1/4" thick weld	L.F.	1980	$0.63	$1,247	$8.20	$16,236	$2.43	$4,811.40	0.750	$16,721
								Total Moment Connection Cost			$235,523

STEEL FLOOR DECK

Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Area (ft^2)	Total
05 31 13.50 5800	3" deep, 20 gauge	S.F.	1	$2.01	$2.01	$0.53	$0.53	$0.04	$0.04	381965	$985,470
								Total Steel Floor Deck Cost			$985,470

SHEAR STUDS

Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	-	Total
05 05 23.85 0900	3/4" diameter stud, 6" long	Ea.	25296	$0.77	$19,477.92	$0.89	$22,513	$0.43	$10,877.28	-	$52,869
								Total Shear Stud Cost			$52,869

SPRAYED CEMENTITIOUS FIREPROOFING

Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Area (ft^2)	Total
07 81 16.10 0700	flat decking, beams and columns	S.F.	1	$0.59	$0.59	$0.81	$0.81	$0.12	$0.12	381965	$580,587
								Total Fireproofing Cost			$580,587

SHEAR STUDS

Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	-	Total
05 05 23.85 0900	3/4" diameter stud, 6" long	Ea.	25296	$0.77	$19,477.92	$0.89	$22,513	$0.43	$10,877.28	-	$52,869
								Total Shear Stud Cost			**$52,869**

SPRAYED CEMENTITIOUS FIREPROOFING

Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Area (ft^2)	Total
07 81 16.10 0700	flat decking, beams and columns	S.F.	1	$0.59	$0.59	$0.81	$0.81	$0.12	$0.12	381965	$580,587
								Total Fireproofing Cost			**$580,587**

ELEVATED SLABS

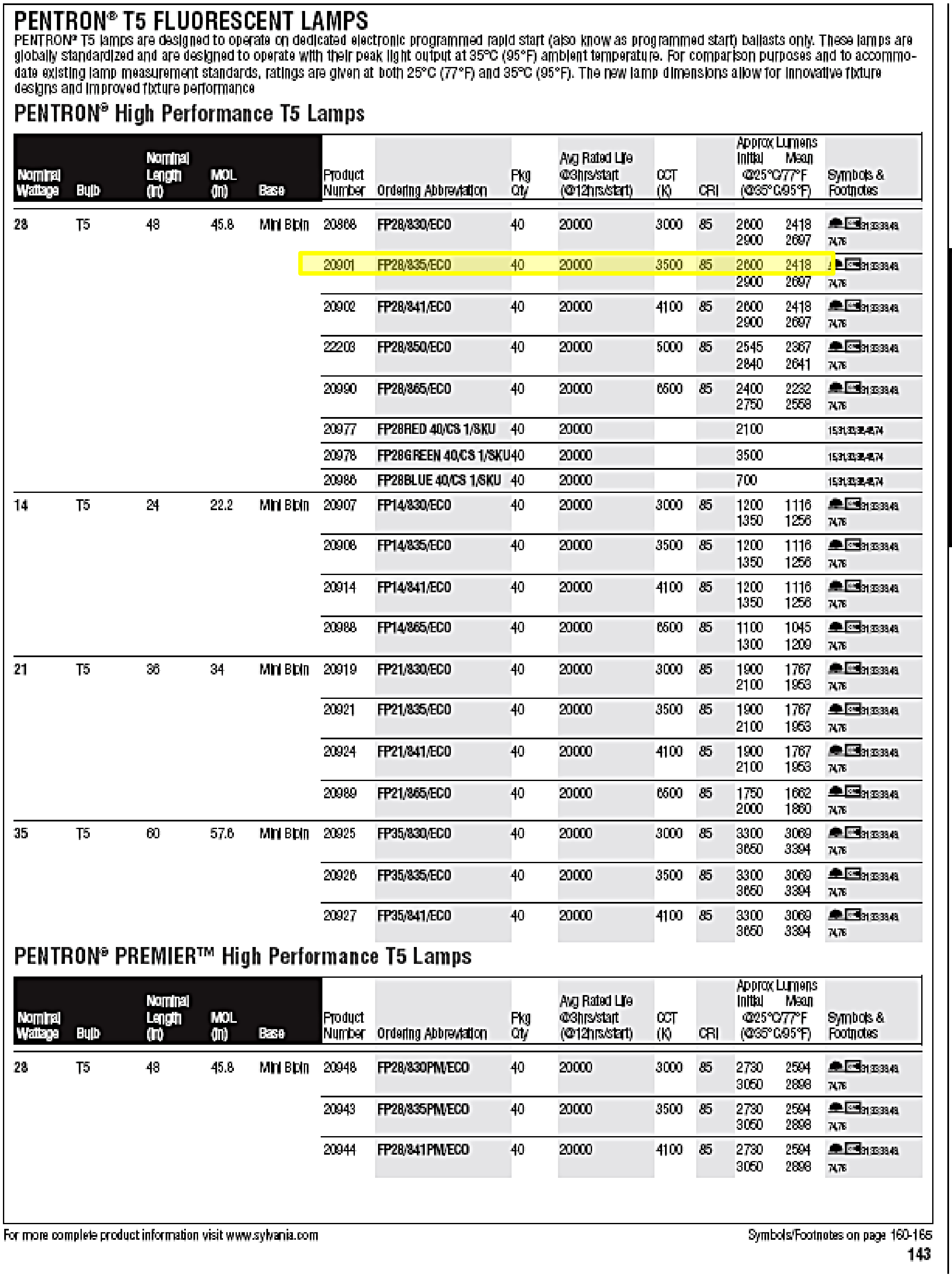
Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Volume (y^3)	Total
03 31 05.35 0300	Normal wt. concrete ready mix, 4000 psi	C.Y.	1	$103.00	$103.00	-	-	-	-	14146.9	$1,457,126
03 31 05.70 1500	elevated slab, 6-10" thick pumped with crane	C.Y.	1	-	-	$15.10	$15.10	$4.82	$4.82	14146.9	$281,805
03 31 05.70	high rise, for more than 5 stories, pumped, add pe story	C.Y.	1	-	-	$1.15	$1.15	$0.37	$0.37	14146.9	$21,503
								Total Elevated Slab Cost			**$1,760,434**

SYSTEM	COST
B-4 SOG	$400,000
Building Foundations (footings & strap beams)	$725,000
Lower level (B-4 to 1st flr) foundation walls	$1,200,000.00
Columns and elevated decks (B-4 to 1st flr)	$3,140,000.00
Misc. subcontractor costs (submittals, gen. conditions, tower crane, etc.)	$250,000.00

Note: Refer to final report for total superstructure cost

Appendix F: Acoustics and Lighting Breadth Analyses

Lighting Analysis



PENTRON® T5 FLUORESCENT LAMPS

PENTRON® T5 lamps are designed to operate on dedicated electronic programmed rapid start (also know as programmed start) ballasts only. These lamps are globally standardized and are designed to operate with their peak light output at 35°C (95°F) ambient temperature. For comparison purposes and to accommodate existing lamp measurement standards, ratings are given at both 25°C (77°F) and 35°C (95°F). The new lamp dimensions allow for innovative fixture designs and improved fixture performance

PENTRON® High Performance T5 Lamps

Nominal Wattage	Bulb	Nominal Length (in)	MOL (in)	Base	Product Number	Ordering Abbreviation	Pkg Qty	Avg Rated Life @3hrs/start (@12hrs/start)	CCT (K)	CRI	Approx Lumens Initial @25°C/77°F (@35°C/95°F)	Approx Lumens Mean	Symbols & Footnotes
28	T5	48	45.8	Mini Bipin	20868	FP28/830/ECO	40	20000	3000	85	2600 2900	2418 2697	31,33,38,48 74,78
					20901	FP28/835/ECO	40	20000	3500	85	2600 2900	2418 2697	31,33,38,48 74,78
					20902	FP28/841/ECO	40	20000	4100	85	2600 2900	2418 2697	31,33,38,48 74,78
					22203	FP28/850/ECO	40	20000	5000	85	2545 2840	2367 2641	31,33,38,48 74,78
					20990	FP28/865/ECO	40	20000	6500	85	2400 2750	2232 2558	31,33,38,48 74,78
					20977	FP28RED 40/CS 1/SKU	40	20000			2100		15,31,33,38,48,74
					20978	FP28GREEN 40/CS 1/SKU	40	20000			3500		15,31,33,38,48,74
					20986	FP28BLUE 40/CS 1/SKU	40	20000			700		15,31,33,38,48,74
14	T5	24	22.2	Mini Bipin	20907	FP14/830/ECO	40	20000	3000	85	1200 1350	1116 1256	31,33,38,48 74,78
					20908	FP14/835/ECO	40	20000	3500	85	1200 1350	1116 1256	31,33,38,48 74,78
					20914	FP14/841/ECO	40	20000	4100	85	1200 1350	1116 1256	31,33,38,48 74,78
					20988	FP14/865/ECO	40	20000	6500	85	1100 1300	1045 1209	31,33,38,48 74,78
21	T5	36	34	Mini Bipin	20919	FP21/830/ECO	40	20000	3000	85	1900 2100	1767 1953	31,33,38,48 74,78
					20921	FP21/835/ECO	40	20000	3500	85	1900 2100	1767 1953	31,33,38,48 74,78
					20924	FP21/841/ECO	40	20000	4100	85	1900 2100	1767 1953	31,33,38,48 74,78
					20989	FP21/865/ECO	40	20000	6500	85	1750 2000	1662 1860	31,33,38,48 74,78
35	T5	60	57.6	Mini Bipin	20925	FP35/830/ECO	40	20000	3000	85	3300 3650	3069 3394	31,33,38,48 74,78
					20926	FP35/835/ECO	40	20000	3500	85	3300 3650	3069 3394	31,33,38,48 74,78
					20927	FP35/841/ECO	40	20000	4100	85	3300 3650	3069 3394	31,33,38,48 74,78

PENTRON® PREMIER™ High Performance T5 Lamps

Nominal Wattage	Bulb	Nominal Length (in)	MOL (in)	Base	Product Number	Ordering Abbreviation	Pkg Qty	Avg Rated Life @3hrs/start (@12hrs/start)	CCT (K)	CRI	Approx Lumens Initial @25°C/77°F (@35°C/95°F)	Approx Lumens Mean	Symbols & Footnotes
28	T5	48	45.8	Mini Bipin	20946	FP28/830PM/ECO	40	20000	3000	85	2730 3050	2594 2898	31,33,38,48 74,78
					20943	FP28/835PM/ECO	40	20000	3500	85	2730 3050	2594 2898	31,33,38,48 74,78
					20944	FP28/841PM/ECO	40	20000	4100	85	2730 3050	2594 2898	31,33,38,48 74,78

FLUORESCENT

For more complete product information visit www.sylvania.com

Symbols/Footnotes on page 160-165

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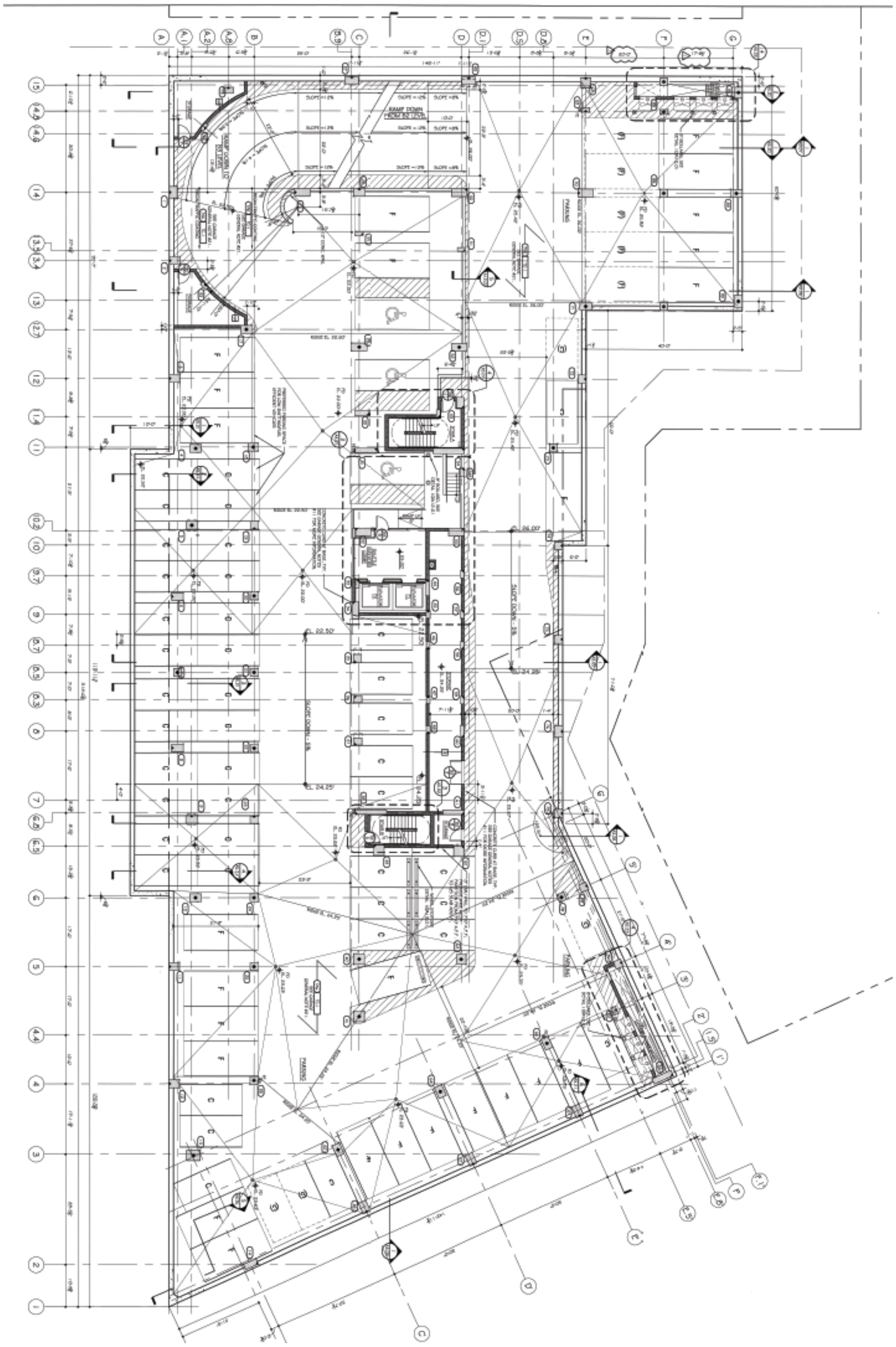
The luminaire/Lamp catalog sheet with selected lamp highlighted can be seen in yellow

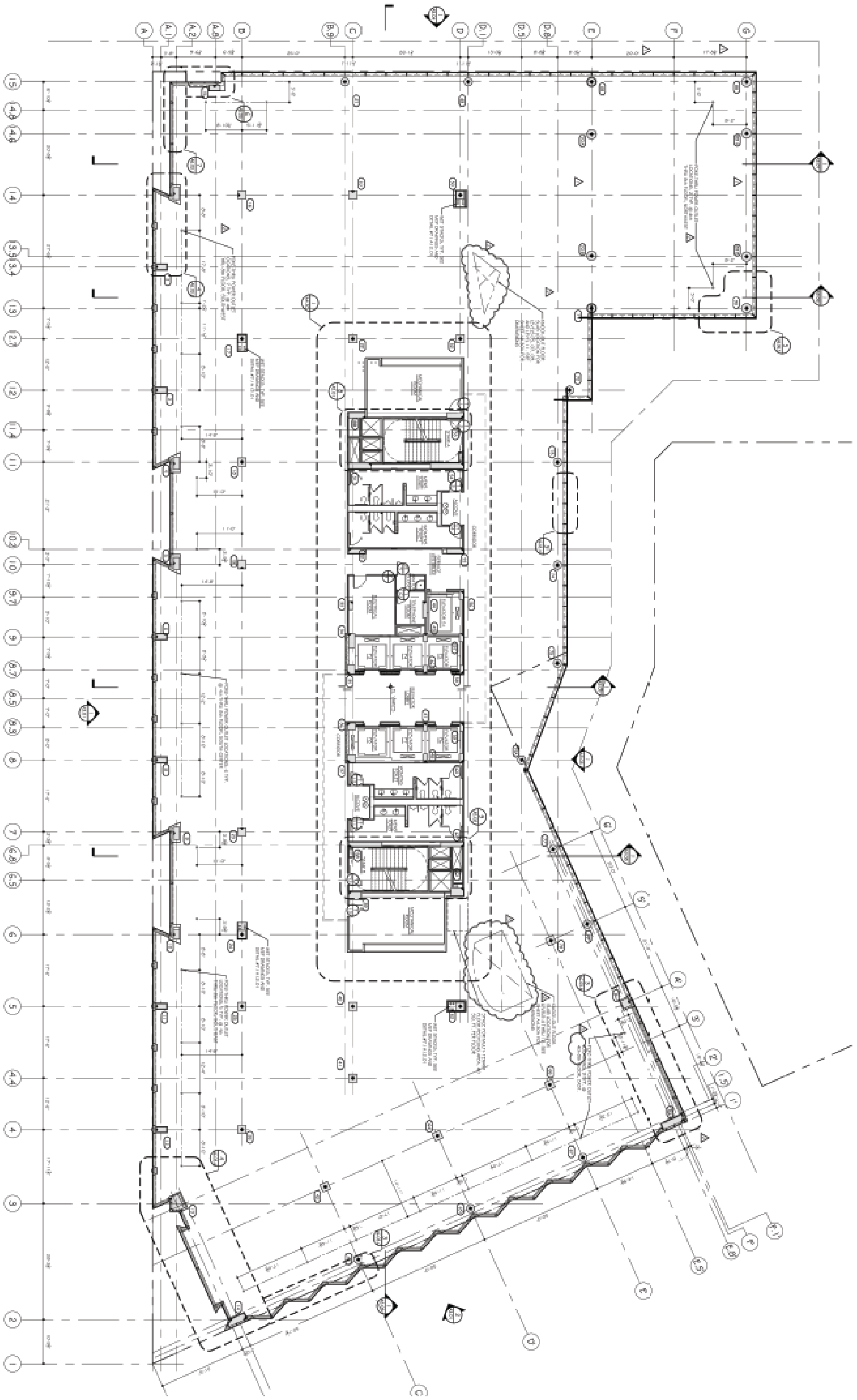
Appendix G: Typical Floor Plans

Existing System Typical Floor Plans and Building Sections



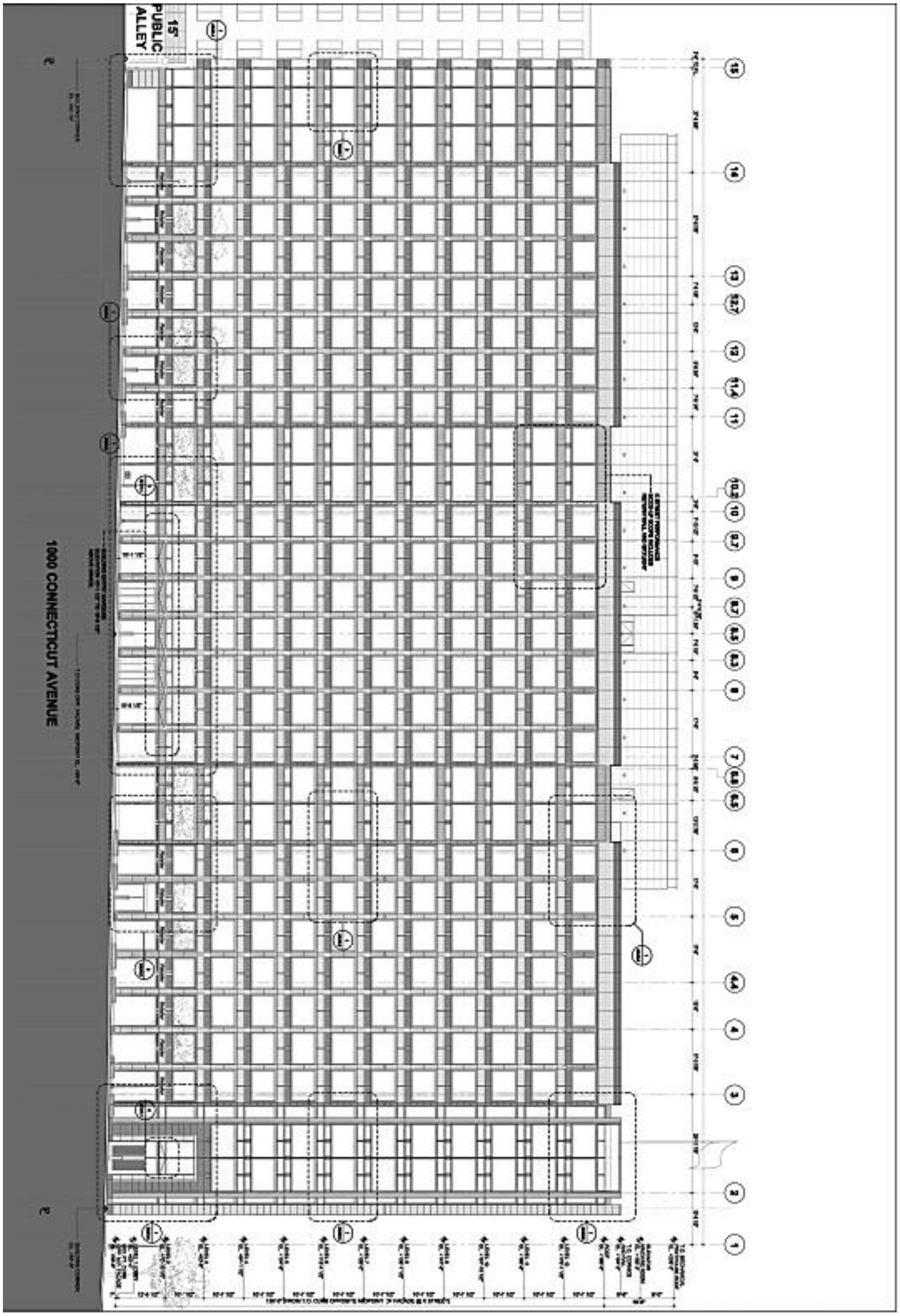
Typical underground parking plan rotated 90 degrees CW

Image obtained from Sheet A2.03 from existing Architectural drawings



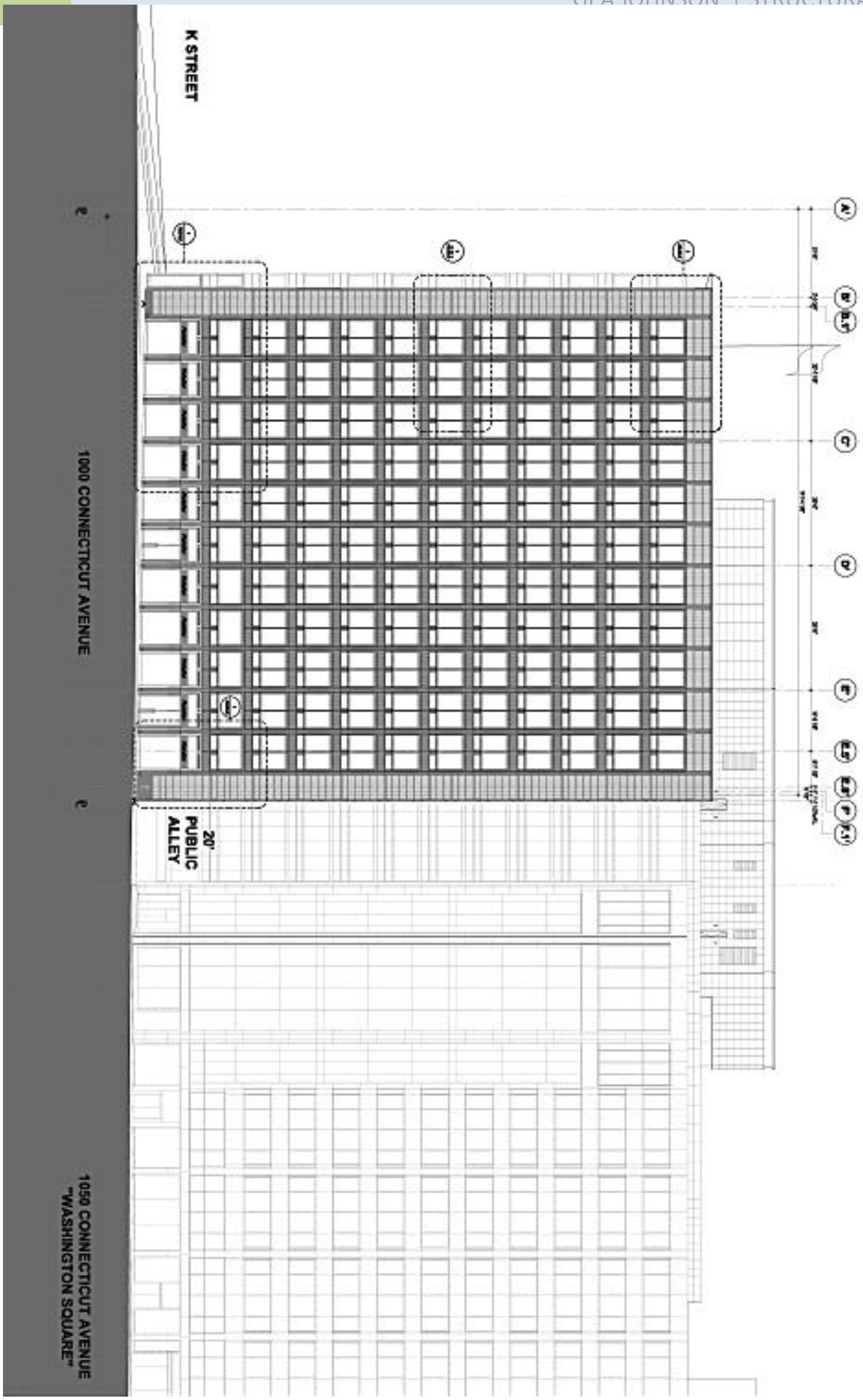
Typical Floor plan oriented 90 degrees CW

Image obtained from Sheet A2.08 from existing Architectural drawings



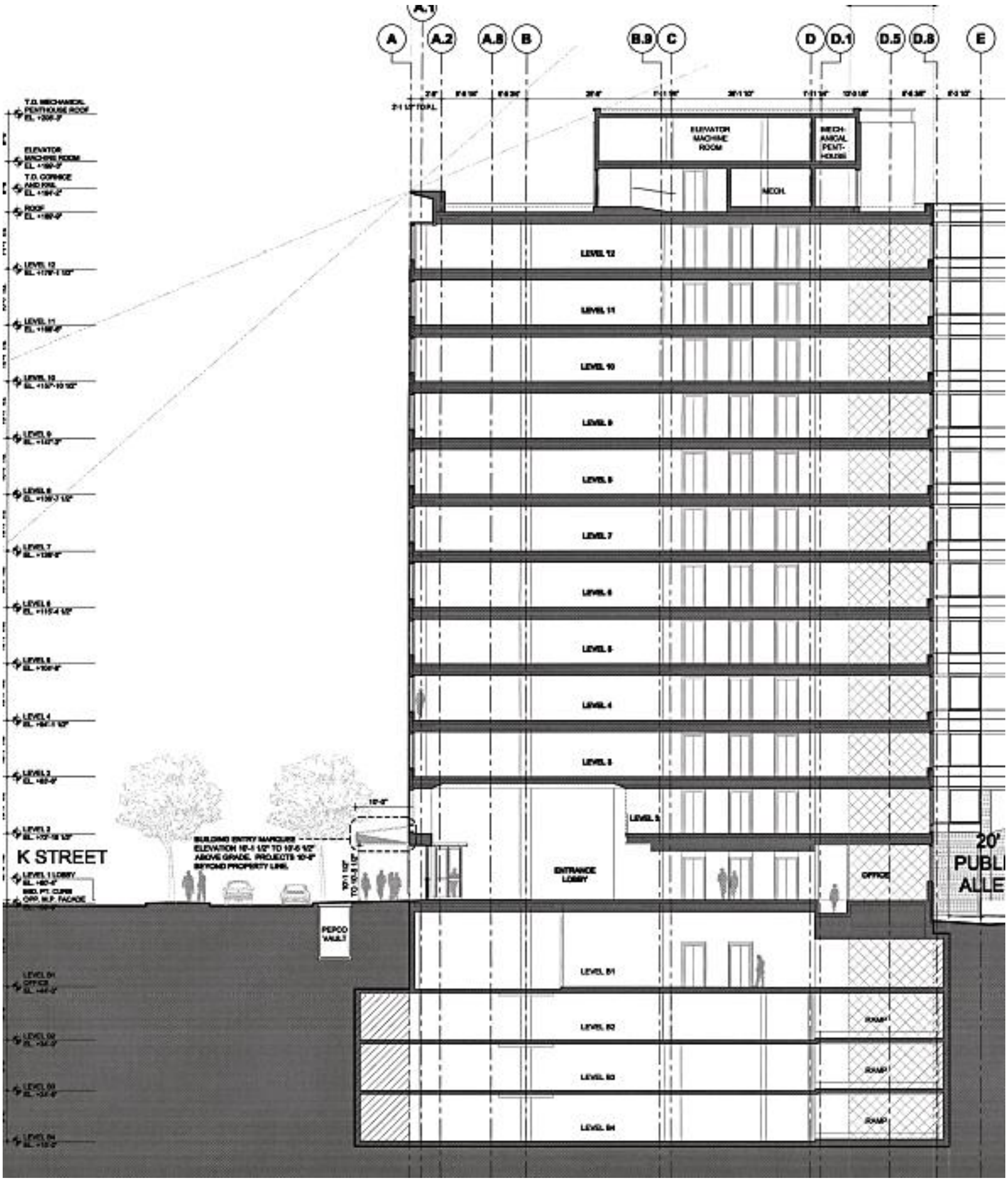
Elevation K Street oriented 90 degrees CW

Image obtained from Sheet A3.01 from existing Architectural drawings



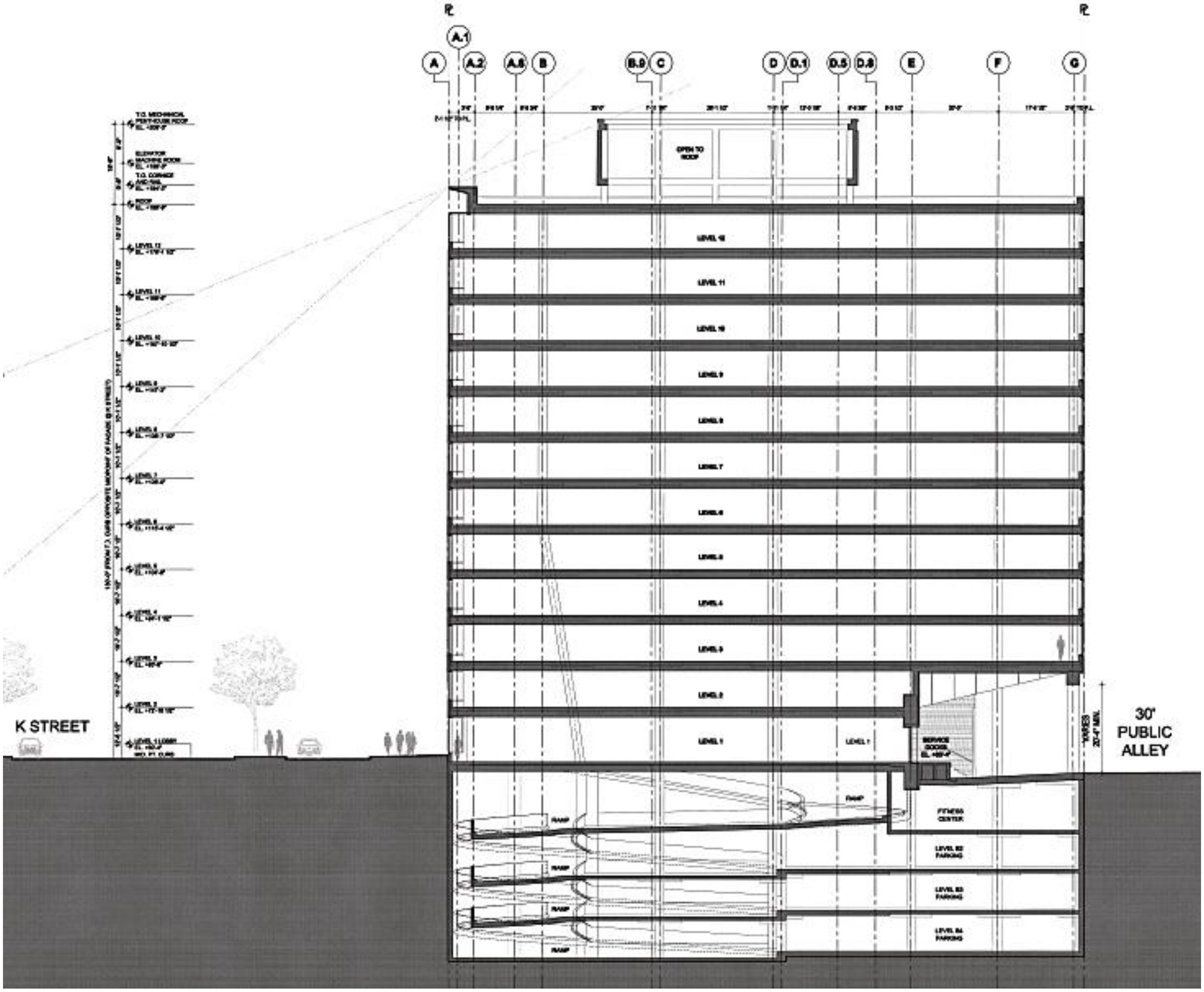
Elevation Connecticut Avenue oriented 90 degrees CW

Image obtained from Sheet A3.02 from existing Architectural drawings



Building section A

Image obtained from Sheet A3.05 from existing Architectural drawings



Building section B

Image obtained from Sheet A3.06 from existing Architectural drawings