Gea Johnson | Structural Option
Faculty Advisor: Dr. Linda Hanagan
April 4, 2012

1000 CONNECTICUT AVENUE

Washington DC

FINAL THESIS REPORT:
STEEL DESIGN AND ANALYSIS

1000 CONNECTICUT AVENUE, NW OFFICE BUILDING

WASHINGTON DC

Gea Johnson|Structural Option

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 typical office with floor to ceiling windows

Perspective of roof-top terrace with green roof

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

Table of Contents

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.

Acknowledgments

I would like to thank the following people and companies for their support during the completion of this final report.

SK&A Structural Engineers, for providing me with the project and the owner permission form. I would like to thank them for answering all of my questions and providing me with the structural drawings and geotechnical report to assist in my thesis project. I would like to specifically thank

Guy Razzi

Kamil Akcali

Michele Robertson

MJ Tyler and Associates, for providing me permission to use 1000 Connecticut Avenue for my thesis project. I would like to specifically thank

Michael Tyler

WDG Architecture, for answering all of my questions, providing me with architectural renderings, and providing me access to the architectural drawings and specifications to assist in my thesis project. I would like to specifically thank

Carl Skooglund

Eric Schlegel

Girard Engineering, for providing me access to the MEP drawings to assist in my thesis project. I would like to specifically thank

Susan Sheridan

Family and friends for their support

The entire AE faculty, but specifically

Dr. Linda Hanagan, for being my advisor and assisting in answering all of my questions

Dr. Kevin Parfitt, for assisting in answering all of my questions

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".

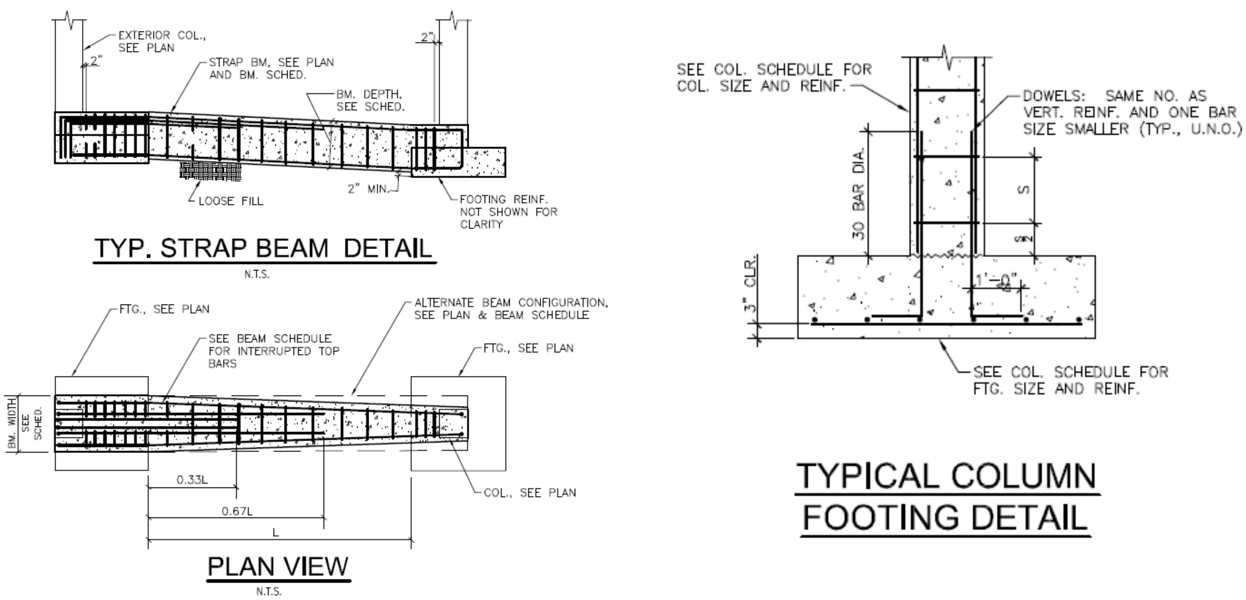


Figure 6 Details of typical strap beam and column footing

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.

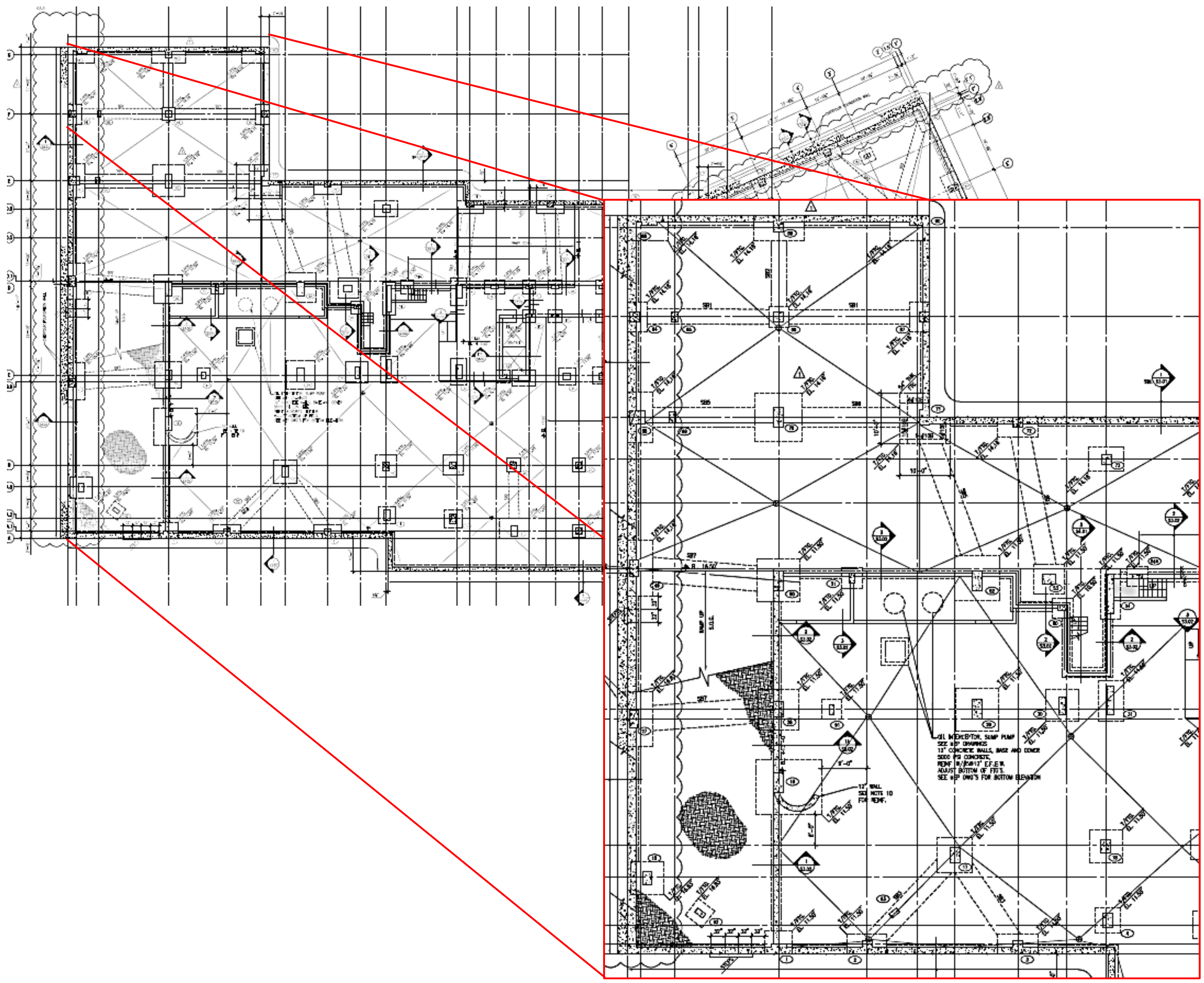


Figure 7 Foundation plan

Framing and Floor System

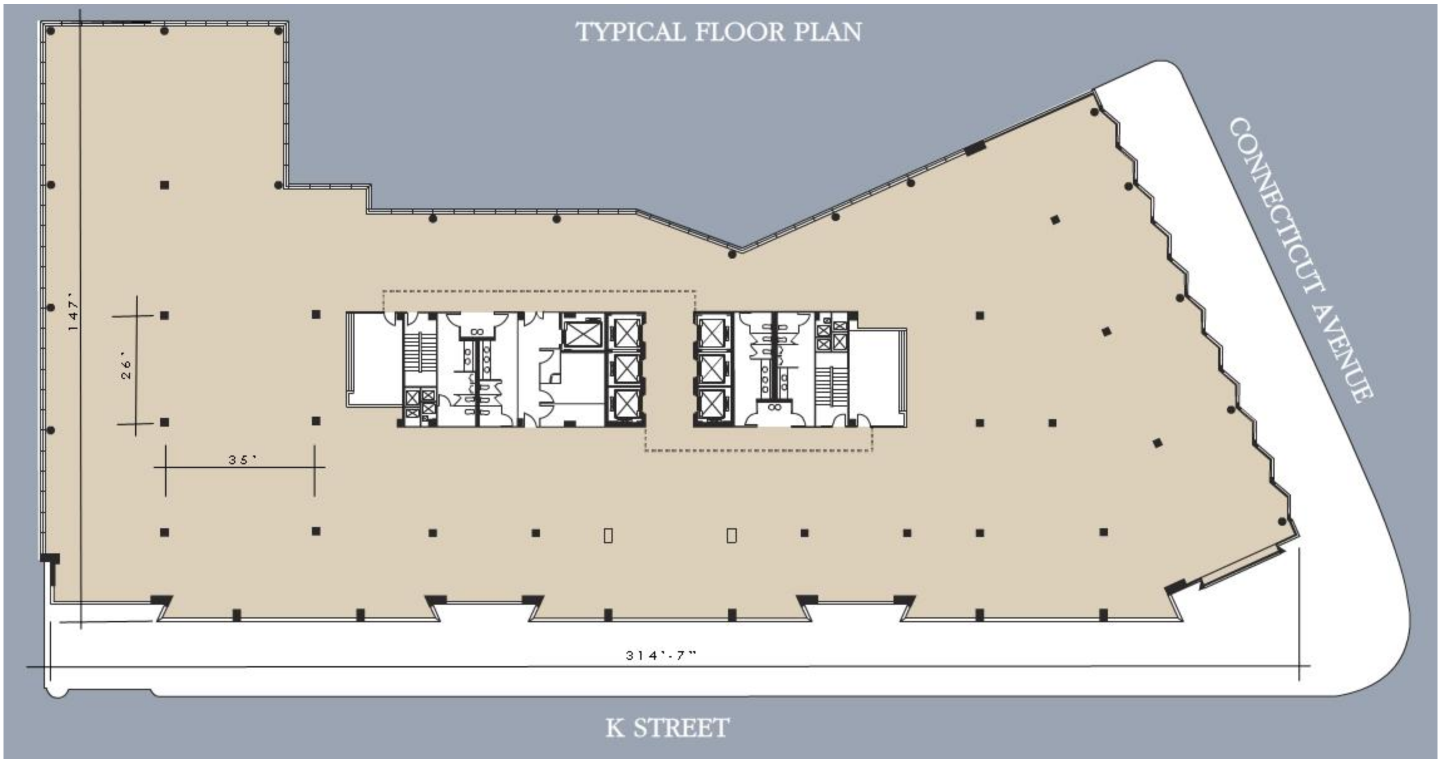
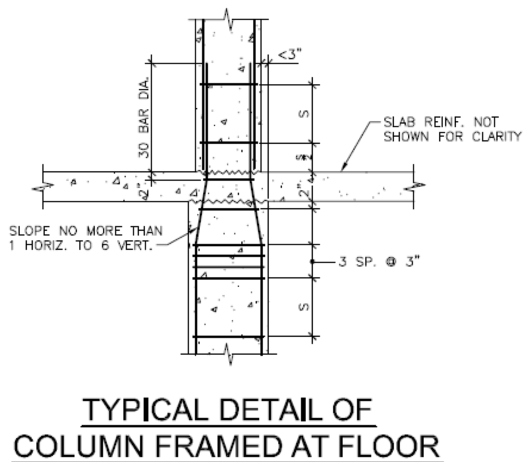


Figure 8 Floor plan displaying column locations and bays

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.



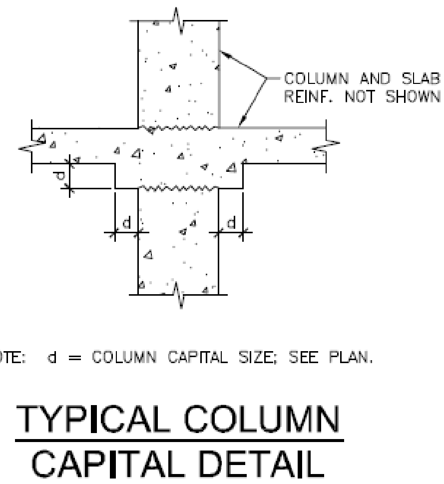


Figure 9 Typical Detail of column framed at the floor

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.

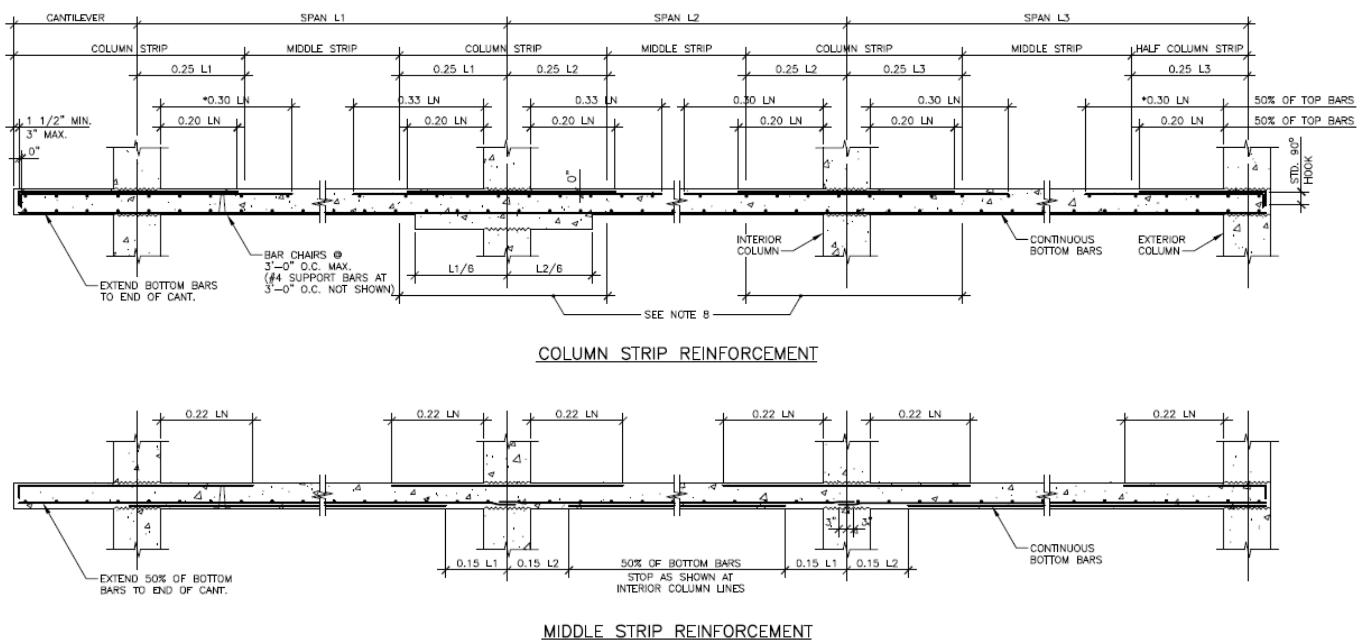


Figure 11 Typical two-way slab reinforcing detail

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.

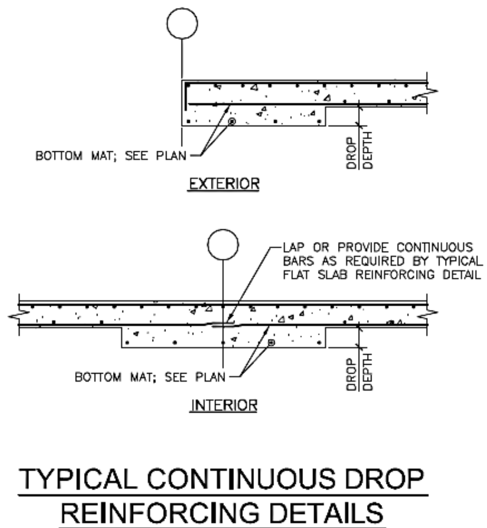


Figure 12 Typical Continuous drop panel

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.

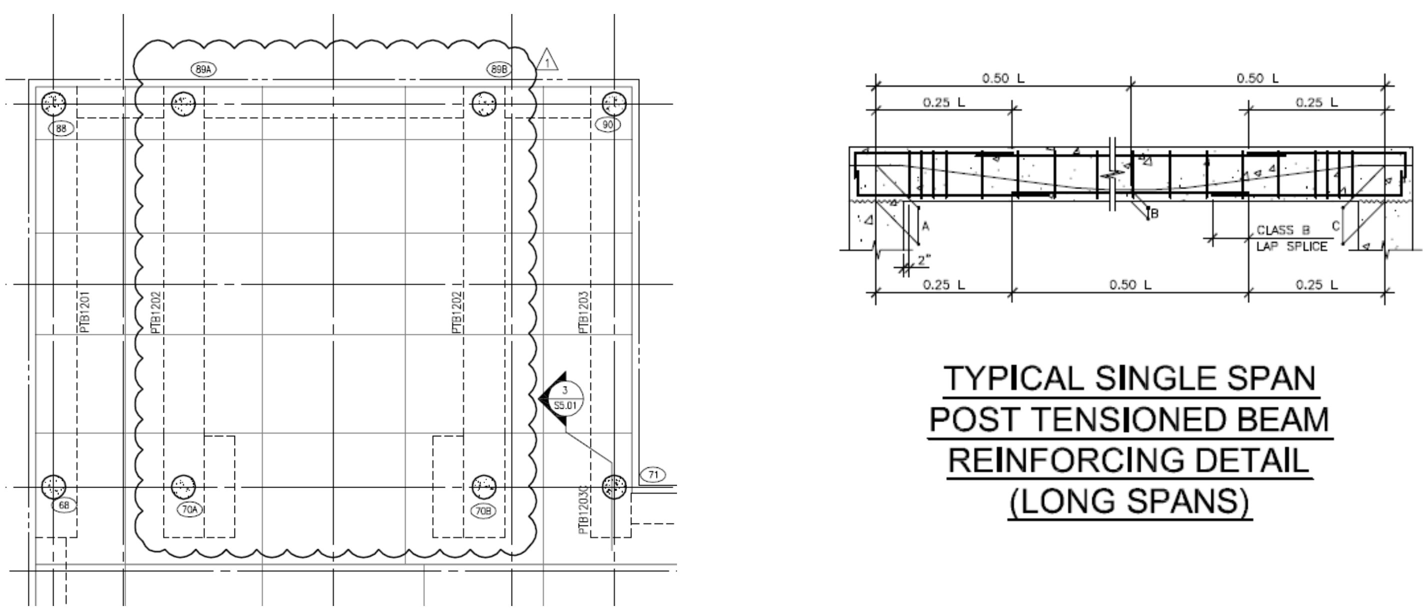


Figure 13 Plan view and typical detail of Post-tension beams supporting slab on levels above two-story loading dock

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.

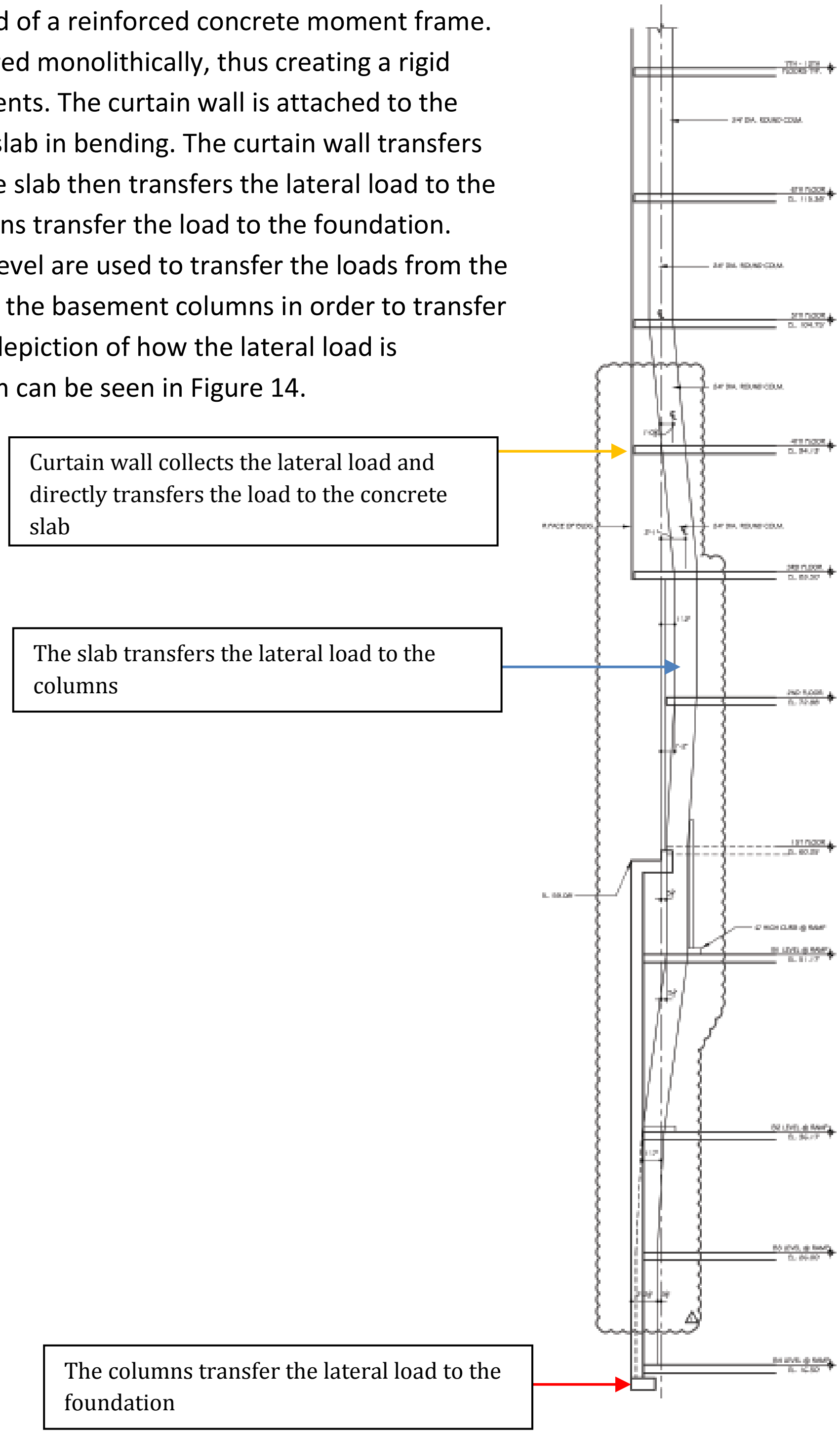


Figure 14 Lateral load path depiction

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.

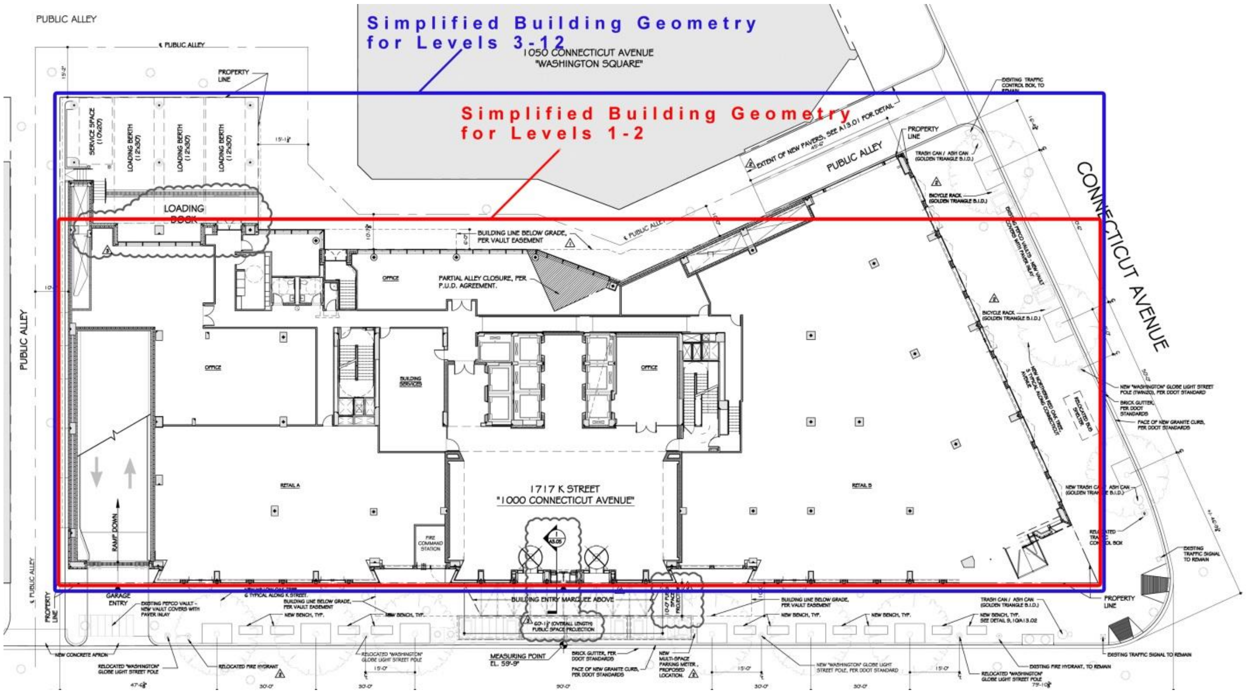


Figure 17 Simplified building shape for wind load analysis

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)
	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
Windward Walls	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
	N/A	0 to 65	-32.52
Roof	N/A	65 to 130	-20.20
	N/A	130-260	-17.61
	N/A	>260	N/A

Table 7 N-S Wind Pressures

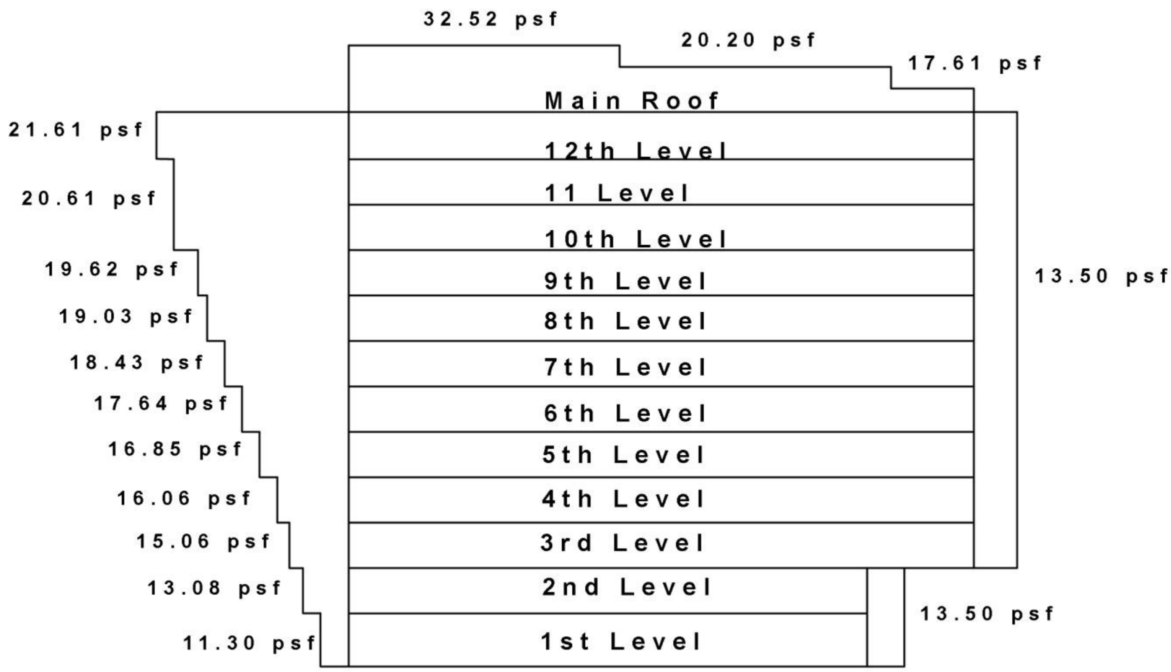


Figure 18 N-S wind pressure vertical pressure sketch

Wind Pressures - E-W Direction			
Type	Floor	Distances (ft)	Wind Pressure (psf)
	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
Windward Walls	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
	N/A	0 to 65	-26.14
Roof	N/A	65 to 130	-26.14
	N/A	130-260	-14.52
	N/A	>260	-8.71

Table 8 E-W wind pressures

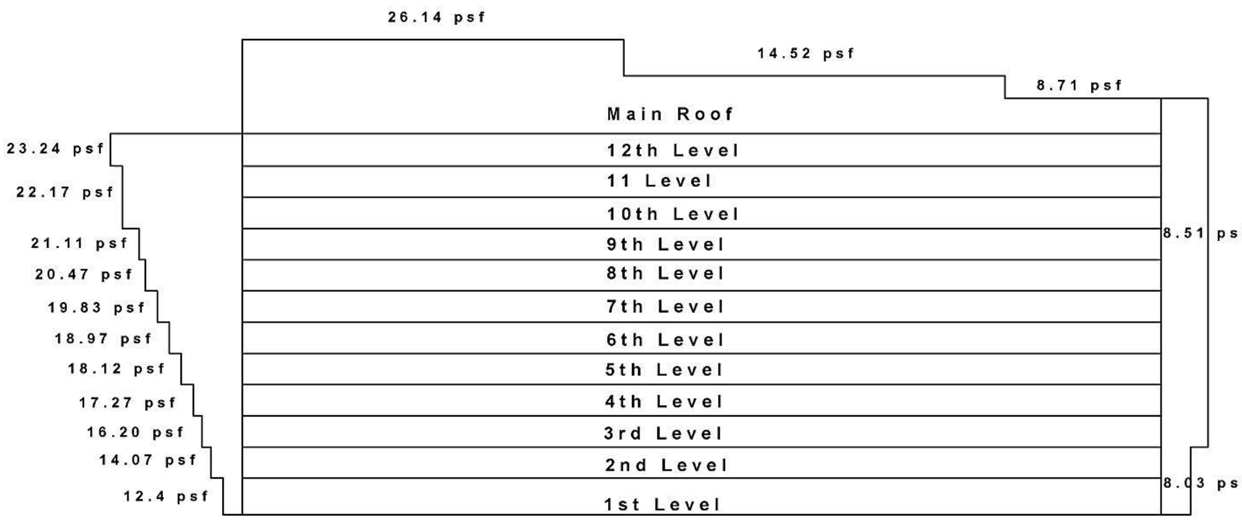


Figure 19 E-W vertical wind pressure profile

Wind Forces - N-S Direction										
		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	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

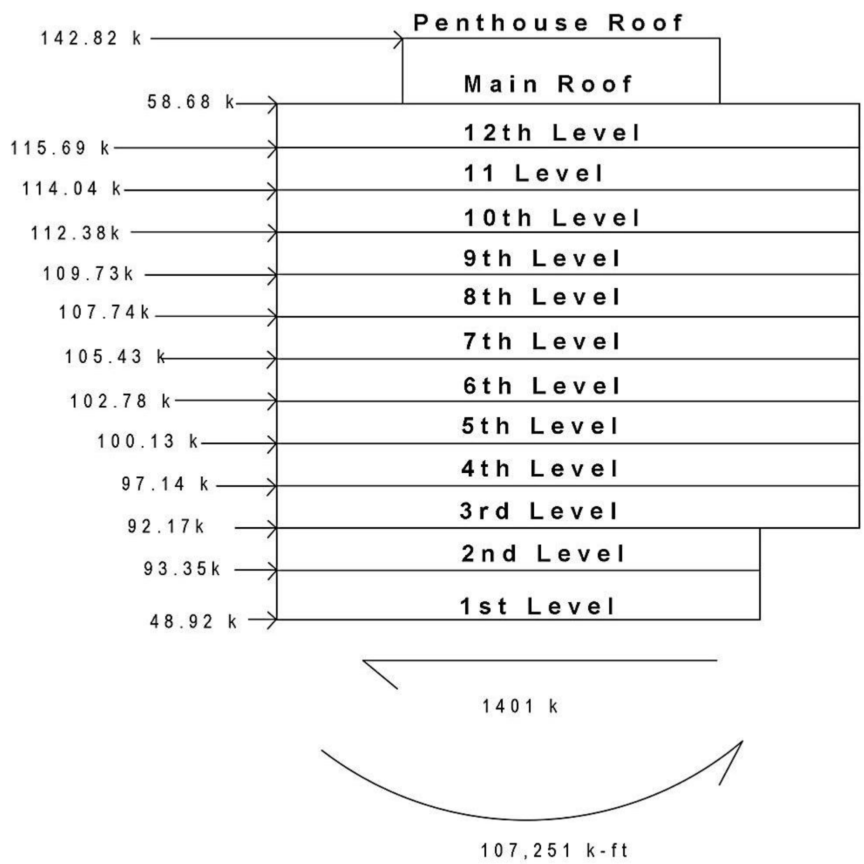


Figure 20 Vertical profile of story forces in N-S direction

Wind Forces - E-W Direction										
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	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

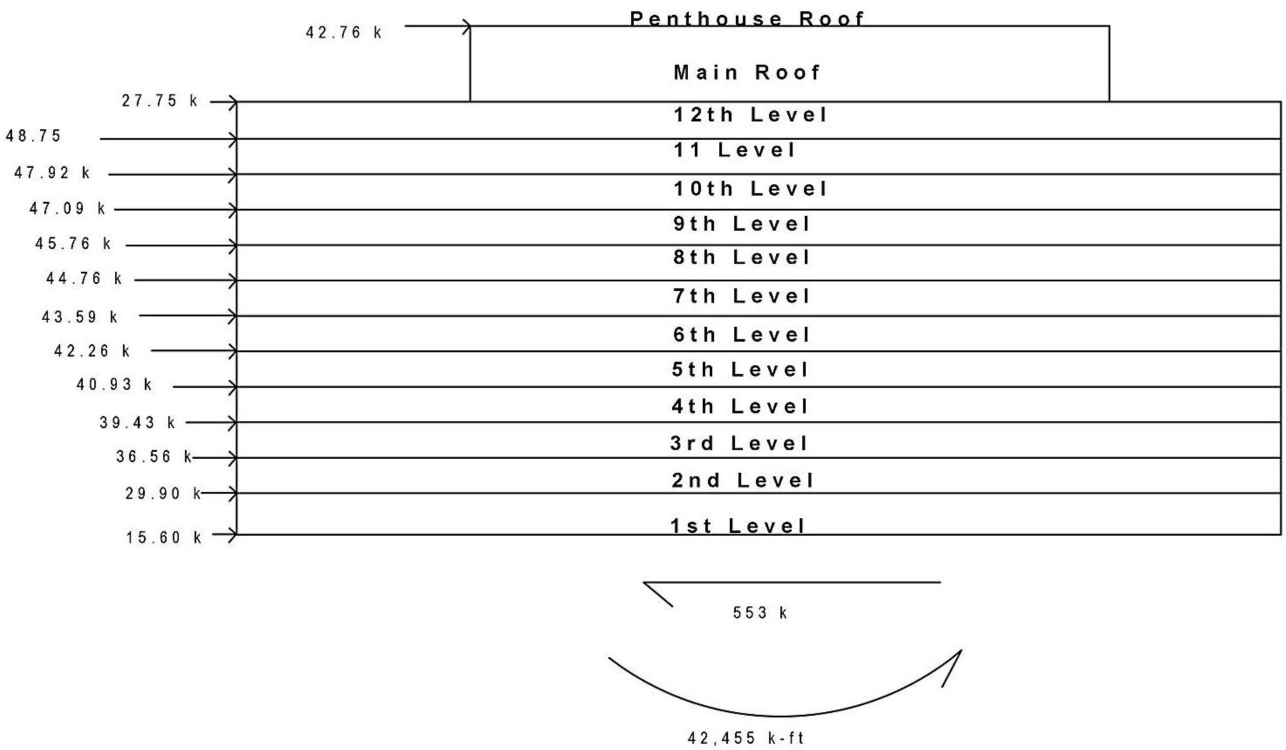


Figure 21 Vertical profile of story forces in E-W direction

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)	$w_x{*}h_x^k$	C_{VX}	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.

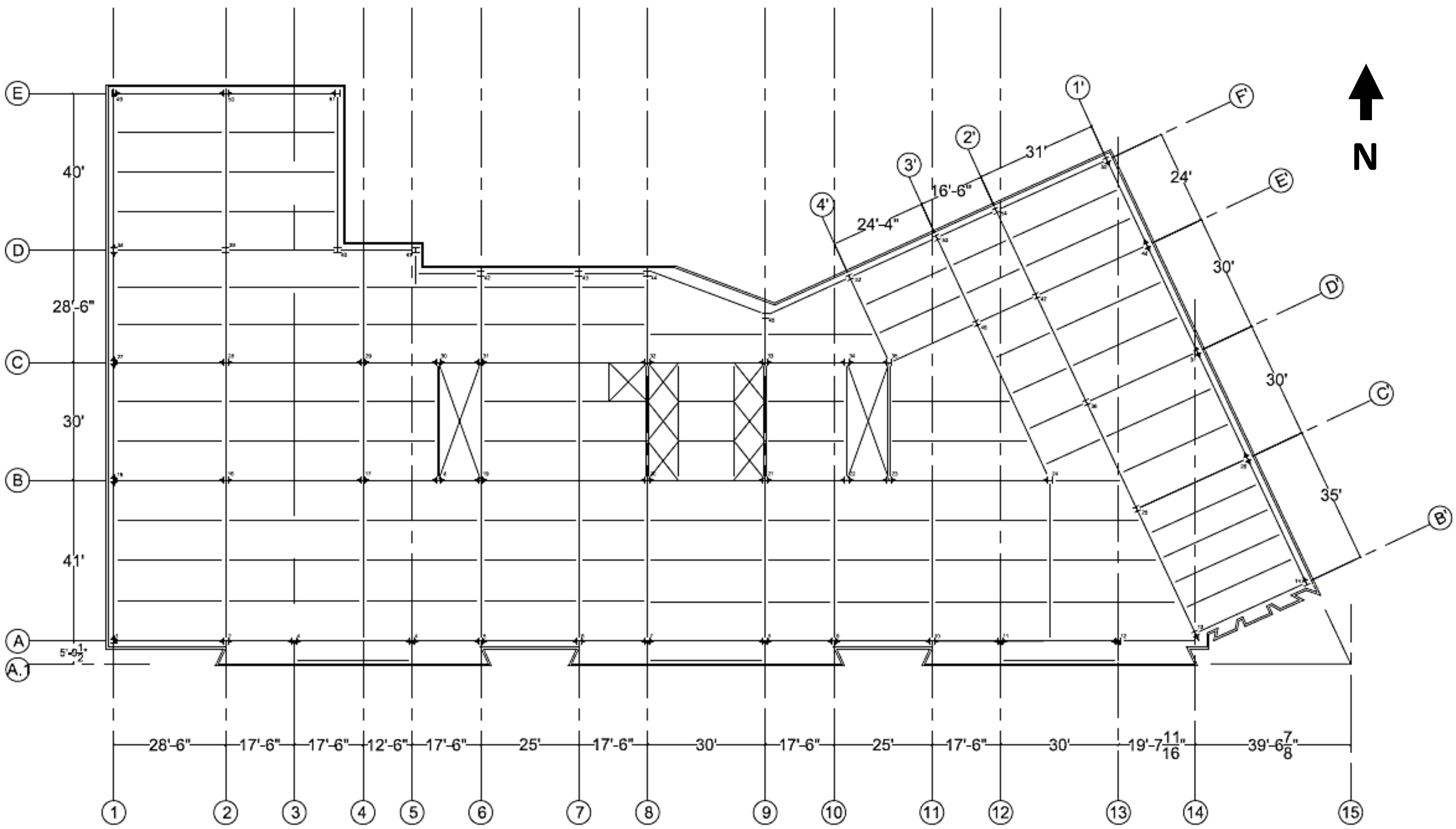


Figure 22 Typical framing plan layout

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.

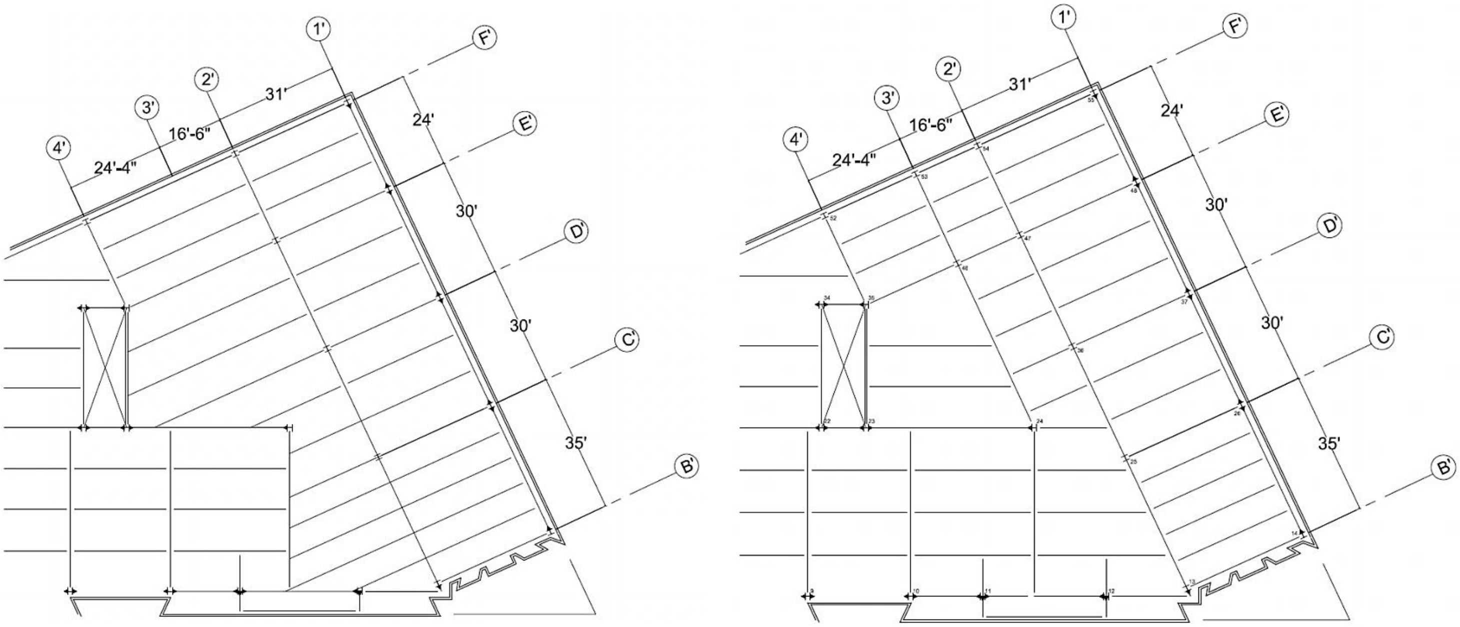


Figure 23 Original (left) and final (right) design of framing layout in tilted building region

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.

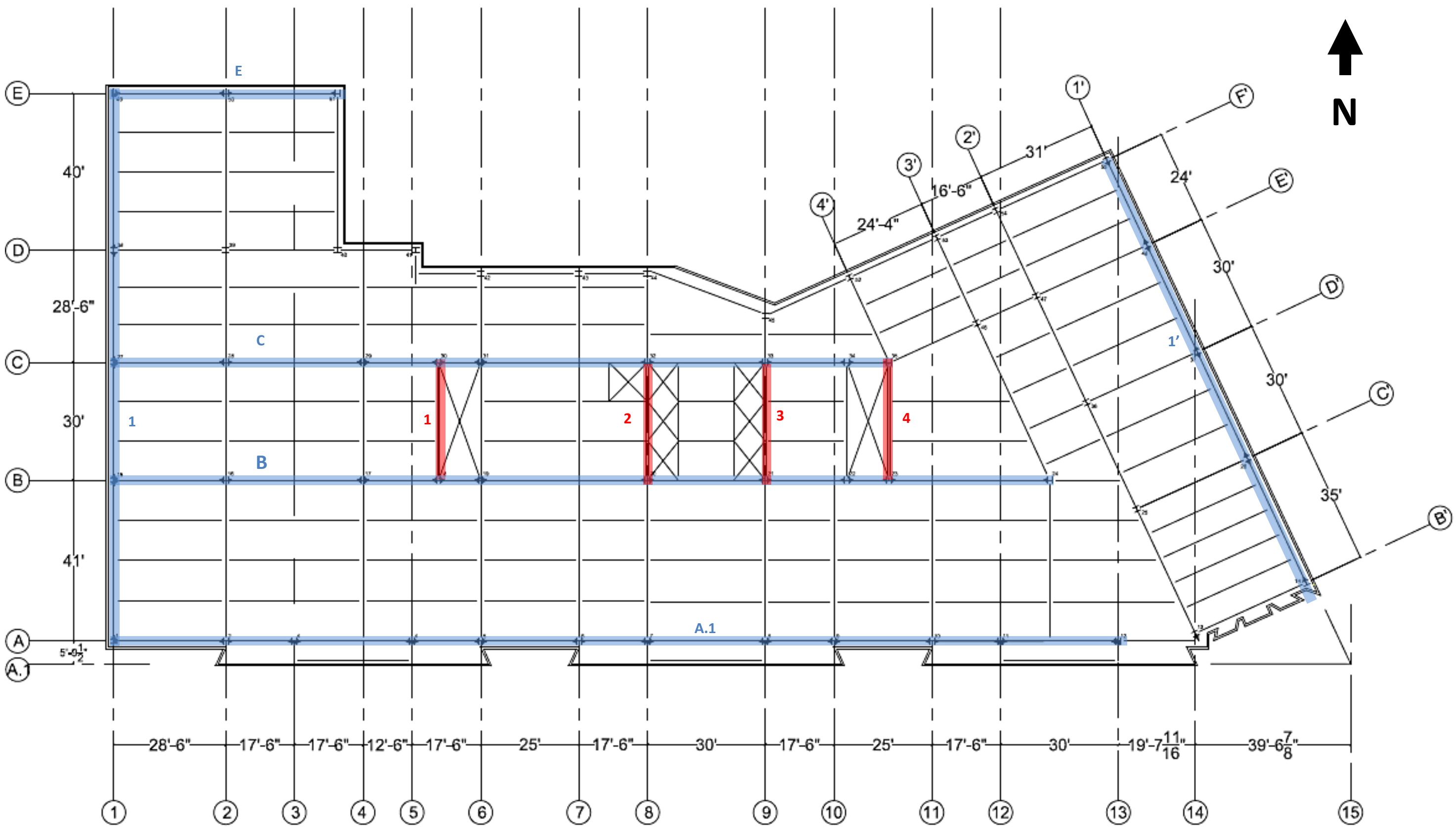


Figure 24 Floor plan with moment frames indicated in blue and braced frames indicated in red

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.

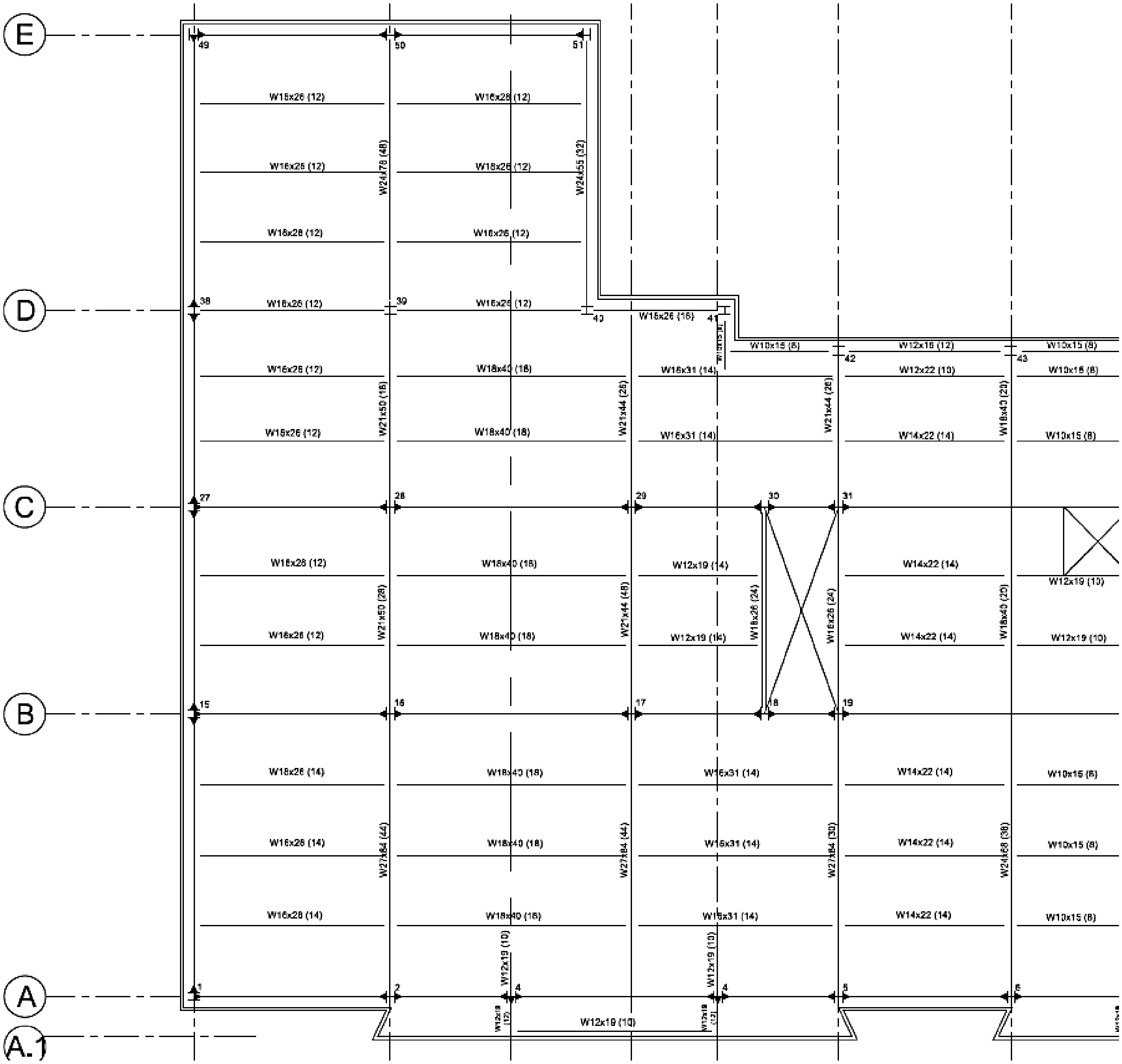


Figure 25 Typical framing plan A with frame sections

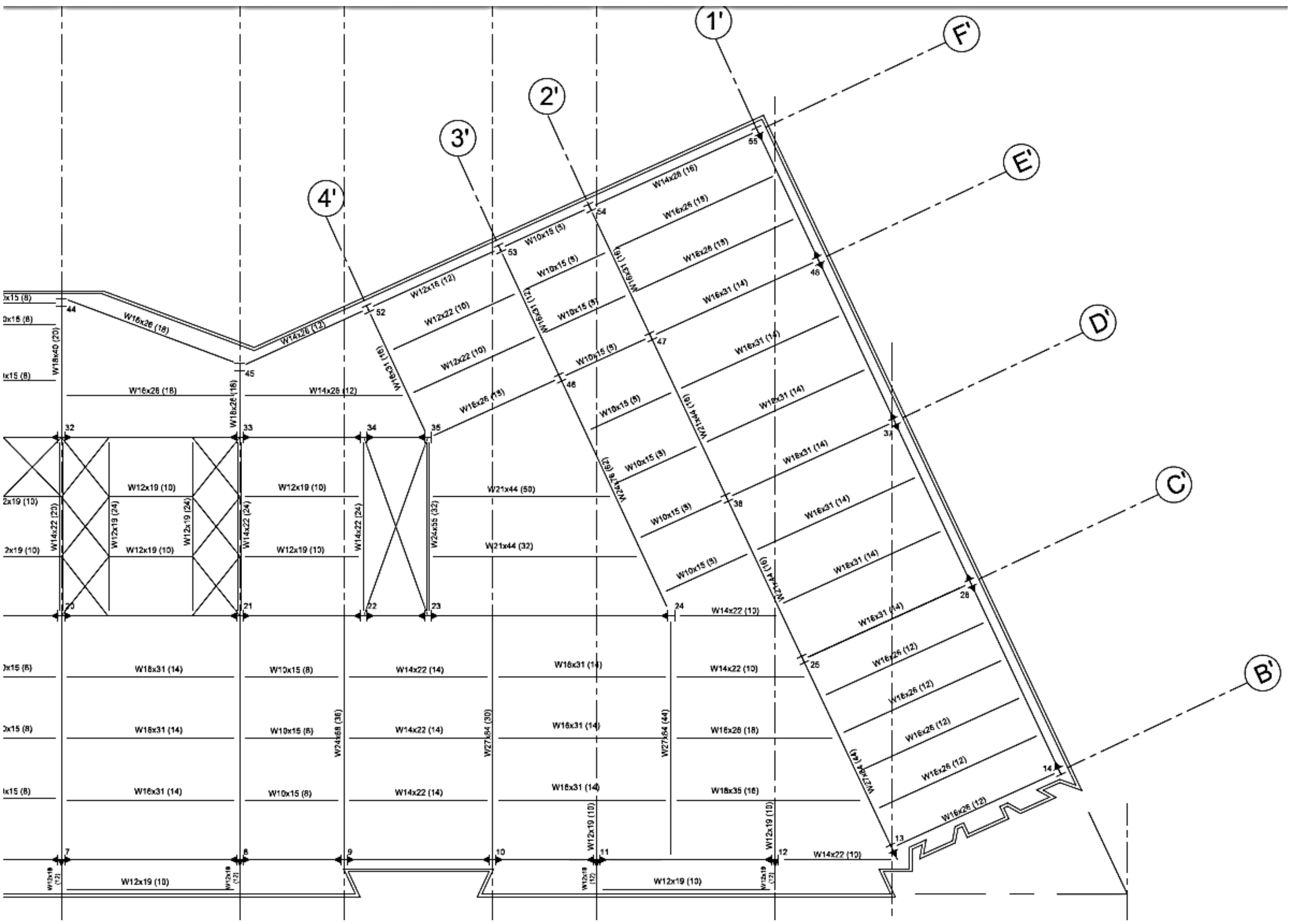


Figure 26 Typical framing plan B with frame sections

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 11TH FLOOR	W14x43	W14x61	W14x48	W14x53	W14x43	W14x43	W14x43	W14x43	W14x48	W14x43	W14x43	W14x43	W14x43	W14x43	W14x43
10 FLOOR 9TH FLOOR	W14x61	W14x74	W14x68	W14x82	W14x61	W14x43	W14x43	W14x43	W14x68	W14x43	W14x61	W14x61	W14x43	W14x43	W14x43
8TH FLOOR 7TH FLOOR	W14x61	W14x90	W14x90	W14x99	W14x90	W14x43	W14x43	W14x43	W14x90	W14x48	W14x68	W14x82	W14x53	W14x48	W14x53
6TH FLOOR 5TH FLOOR	W14x82	W14x109	W14x99	W14x132	W14x99	W14x43	W14x43	W14x61	W14x109	W14x61	W14x90	W14x90	W14x61	W14x61	W14x61
4TH FLOOR 3RD FLOOR	W14x90	W14x132	W14x120	W14x159	W14x120	W14x53	W14x53	W14x61	W14x132	W14x61	W14x90	W14x109	W14x68	W14x61	W14x68
2ND FLOOR 1ST FLOOR	W14x99	W14x159	W14x145	W14x193	W14x145	W14x61	W14x61	W14x68	W14x145	W14x74	W14x109	W14x132	W14x82	W14x68	W14x82

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.

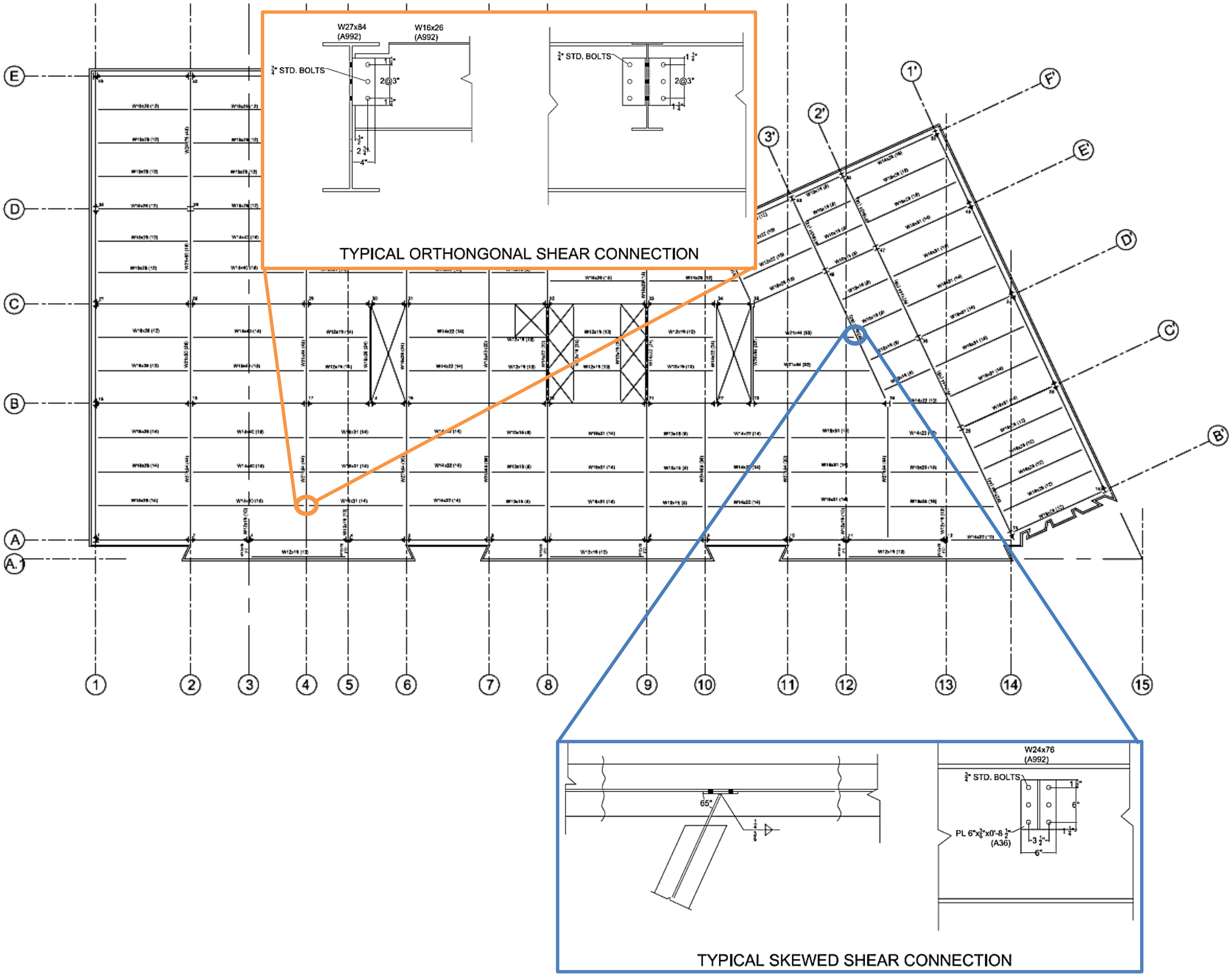


Figure 27 Typical shear connections

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)
	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
Windward Walls	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
	N/A	0 to 90	-39.13
Roof	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

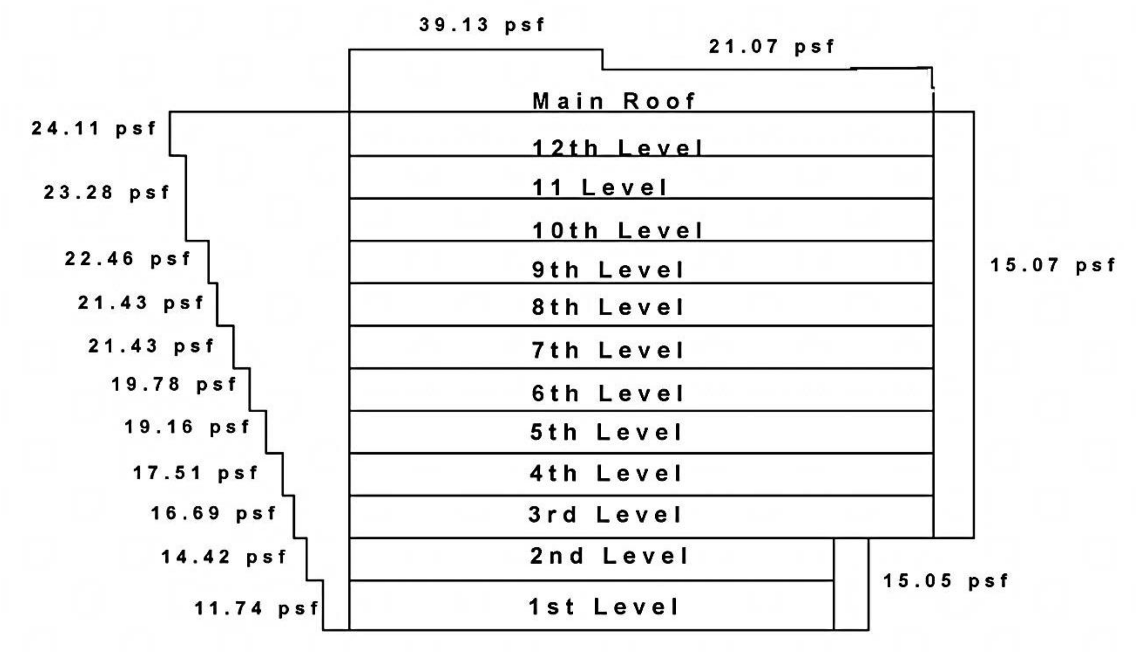


Figure 28 Vertical profile of wind pressure distribution in North-South direction

Wind Pressures - E-W Direction			
Type	Floor	Distances (ft)	Wind Pressure (psf)
	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
Windward Walls	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
	N/A	0 to 90	-31.29
Roof	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

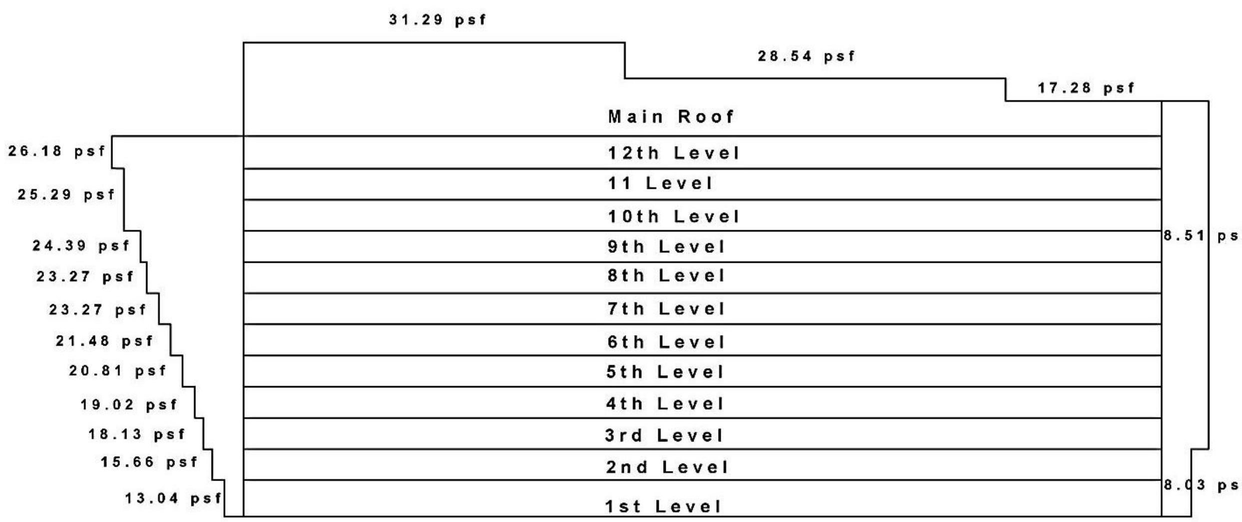


Figure 29 Vertical profile wind pressure distribution in East-West direction

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

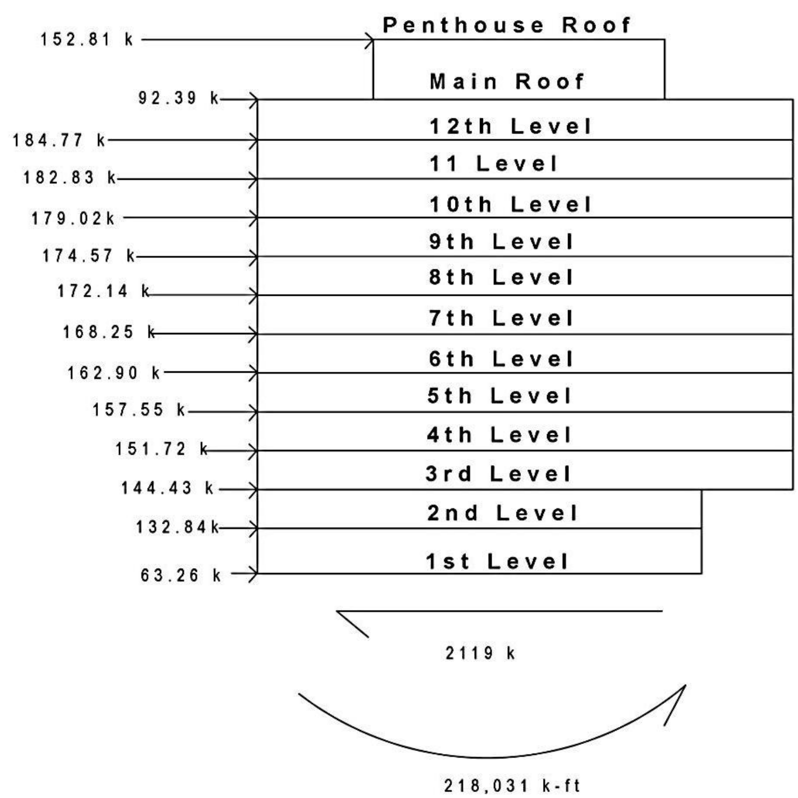


Figure 30 Vertical profile of story forces in N-S direction

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

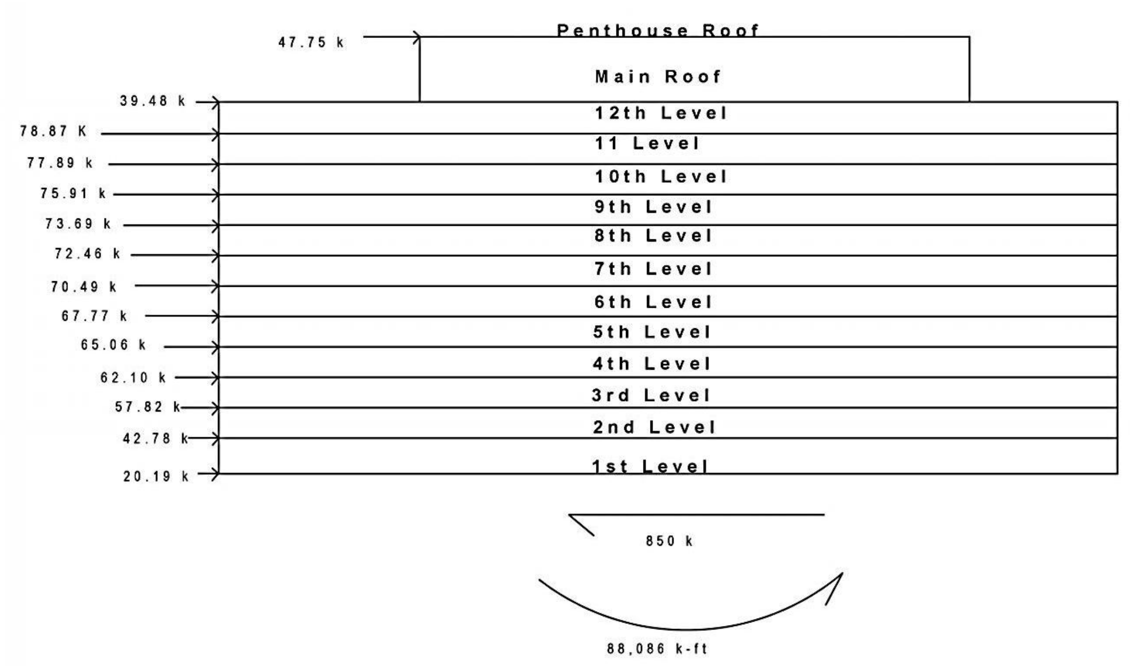


Figure 31 Vertical profile of story forces in E-W direction

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)	$w_x*h_x^k$	C_{vx}	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

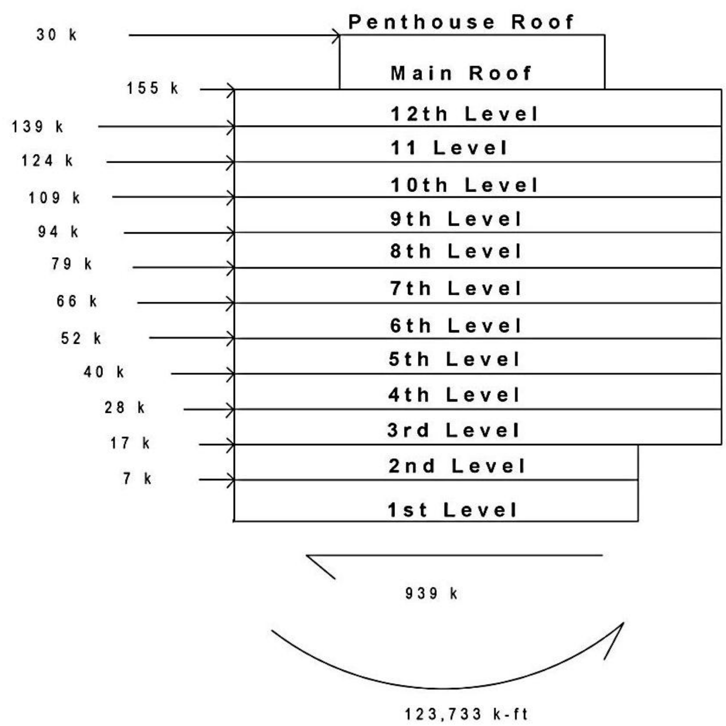


Figure 32 Vertical profile of story forces in N-S direction

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

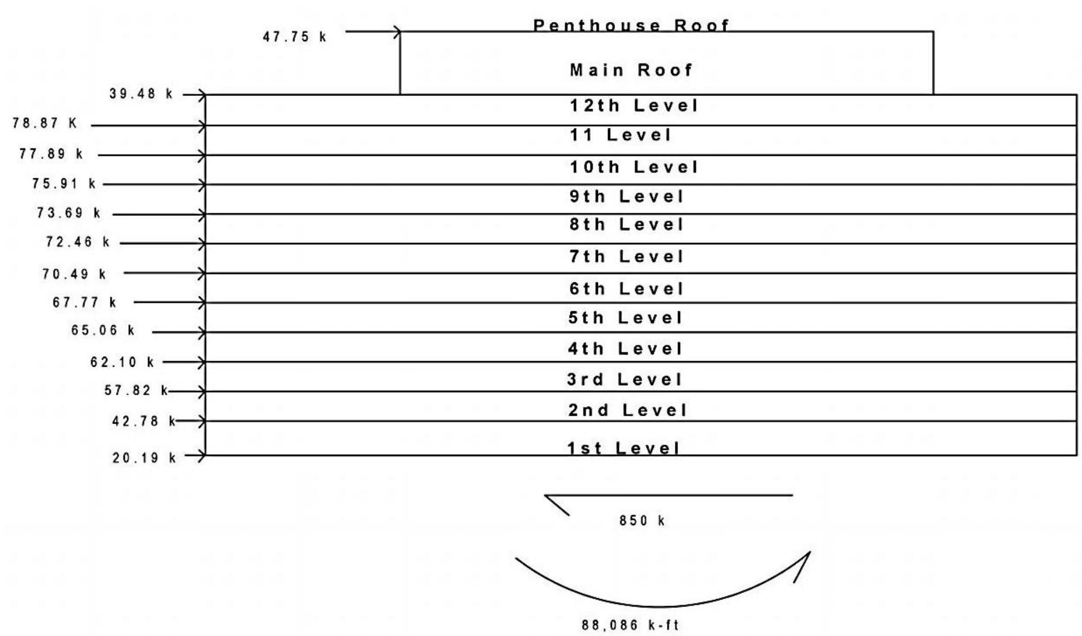


Figure 33 Vertical profile of story forces in E-W direction

Computer Model

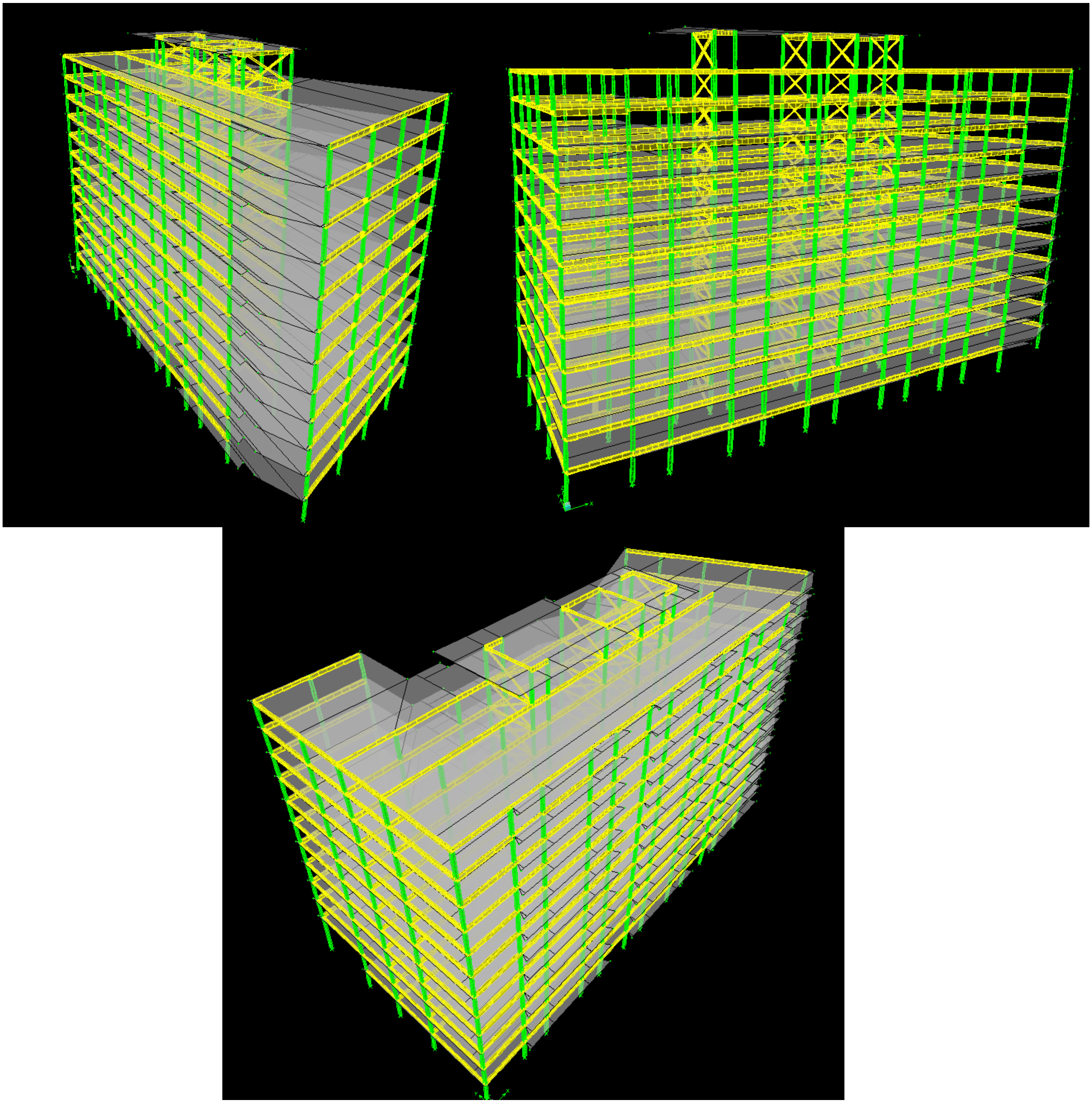
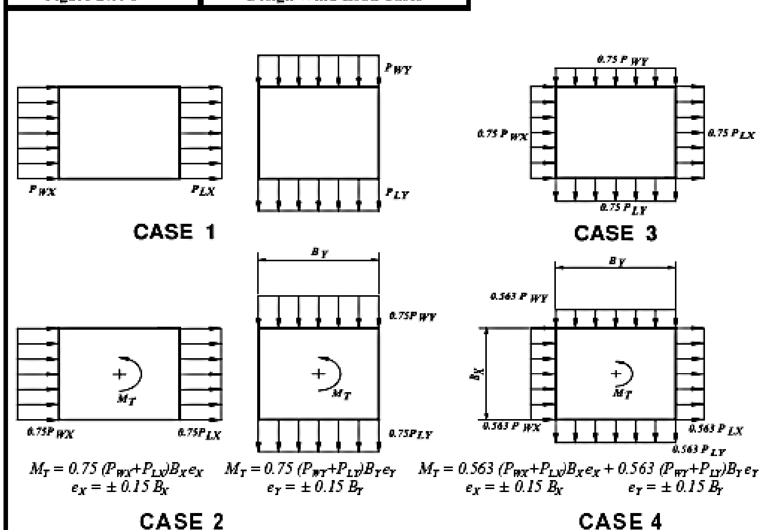


Figure 34 3D perspectives of the new lateral system modeled in ETABS

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.



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 Shear Force (kips)	Y-Direction 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 Shear Force (kips)	Y-Direction 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.

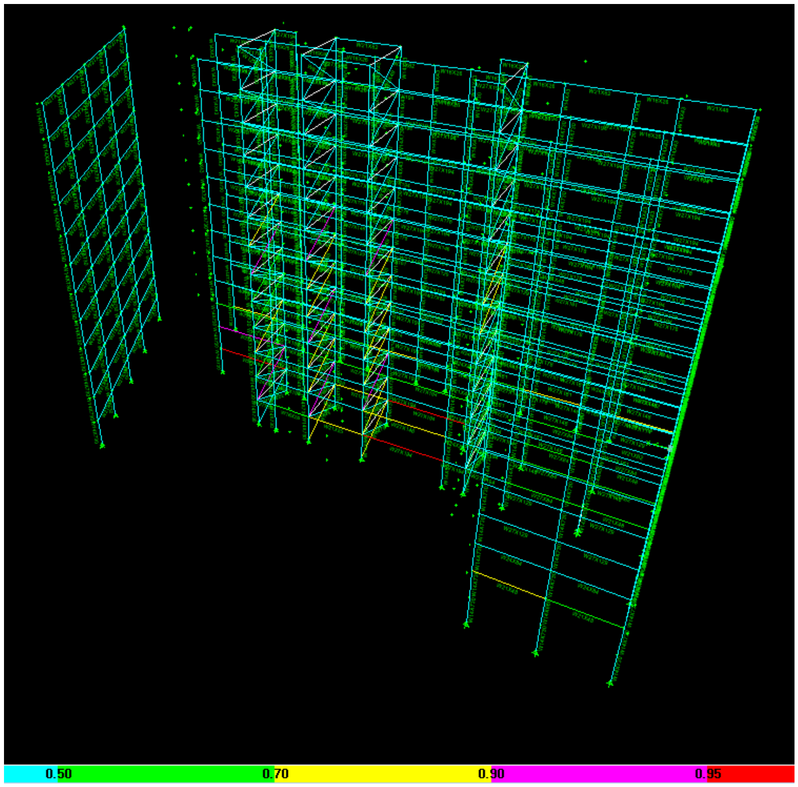


Figure 37 Strength of the design members with the use of the interaction diagram

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.

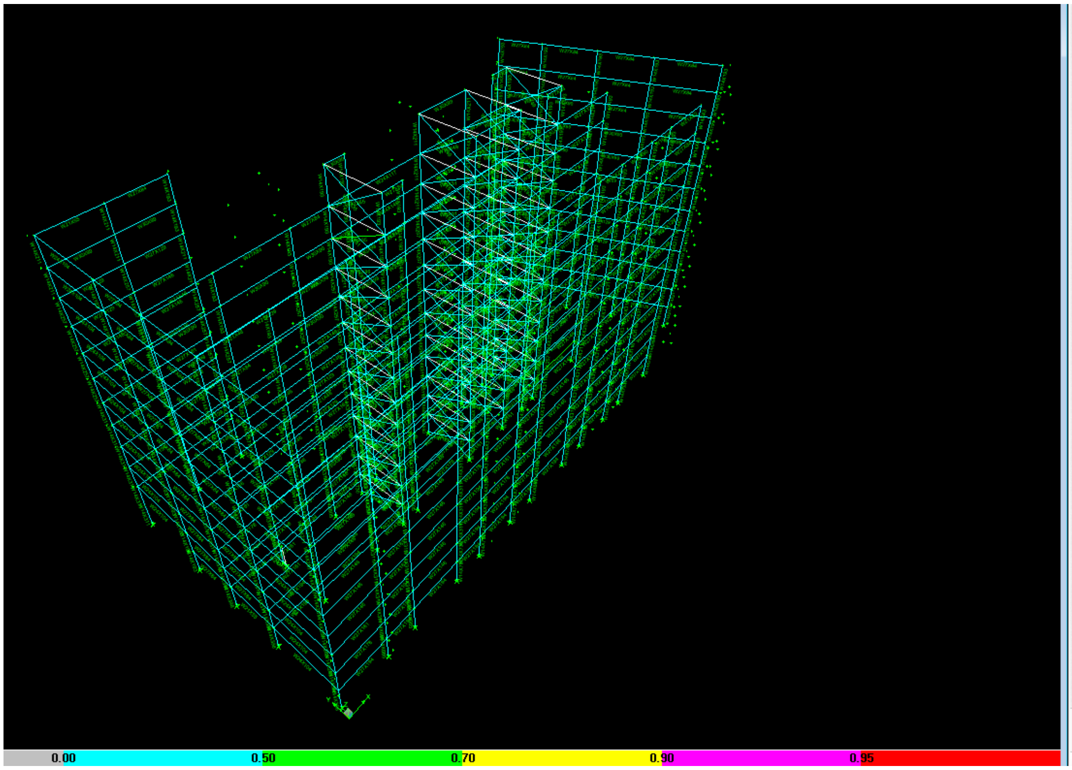


Figure 38 Final lateral system member selection

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

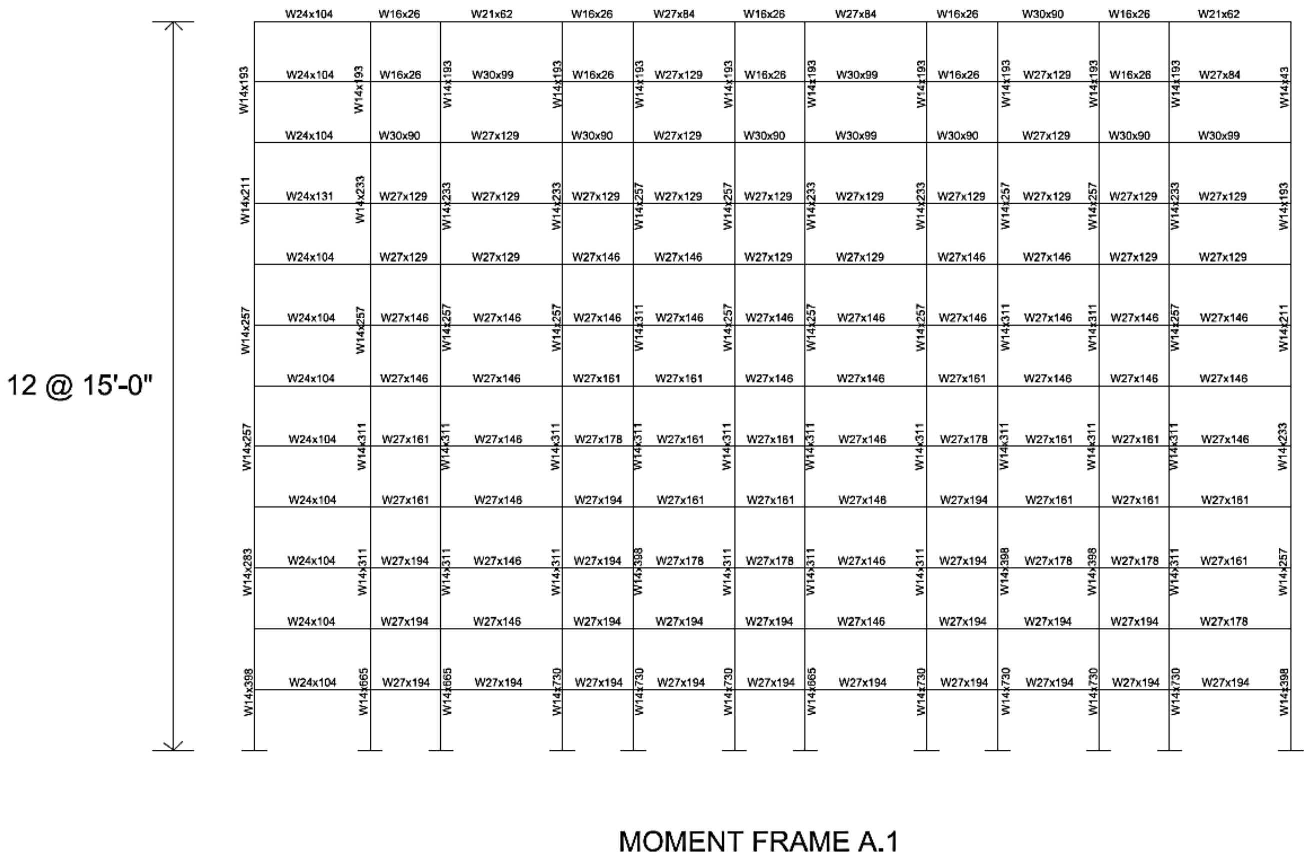


Figure 39 Moment Frame A.1

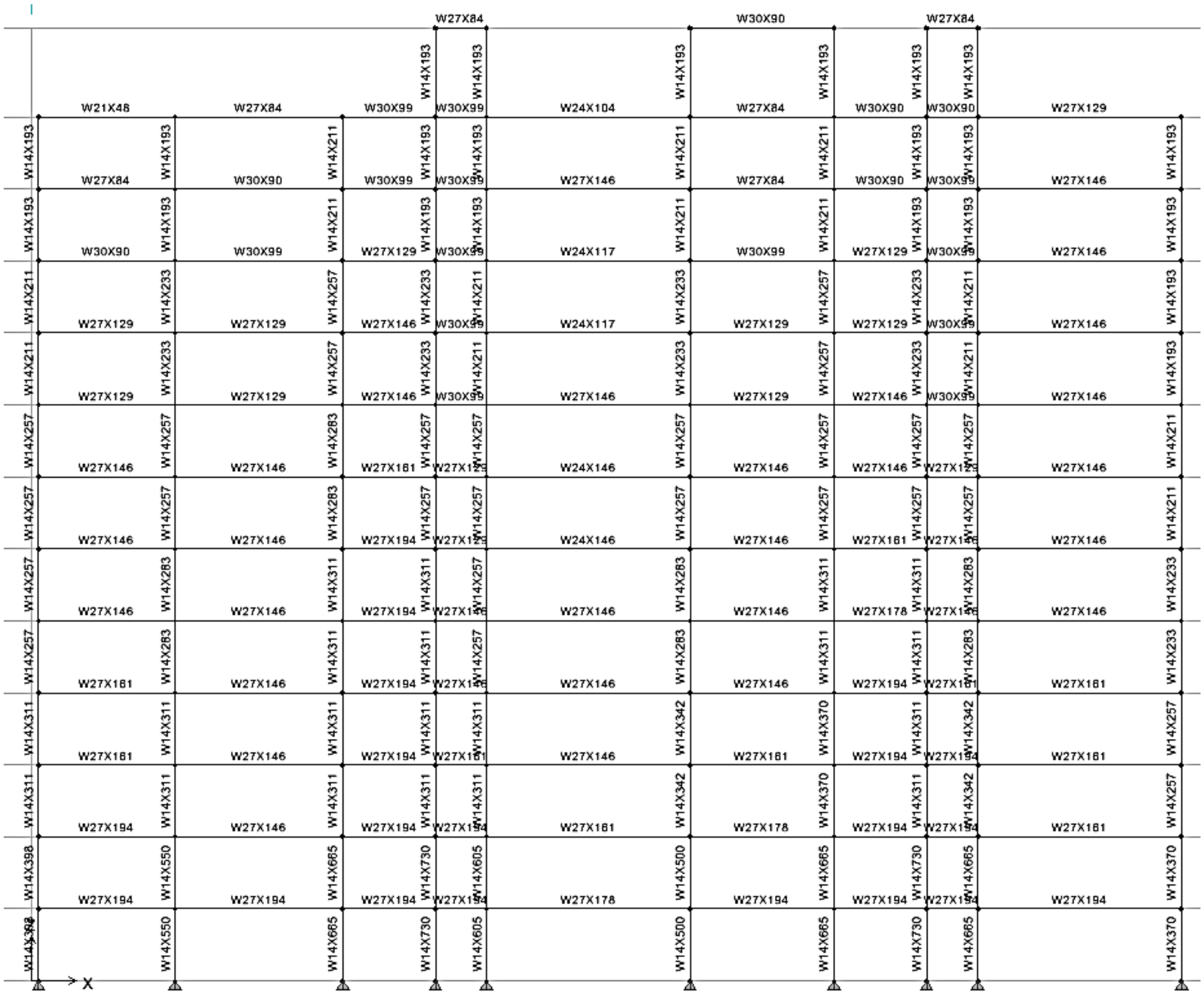


Figure 40 Moment Frame B

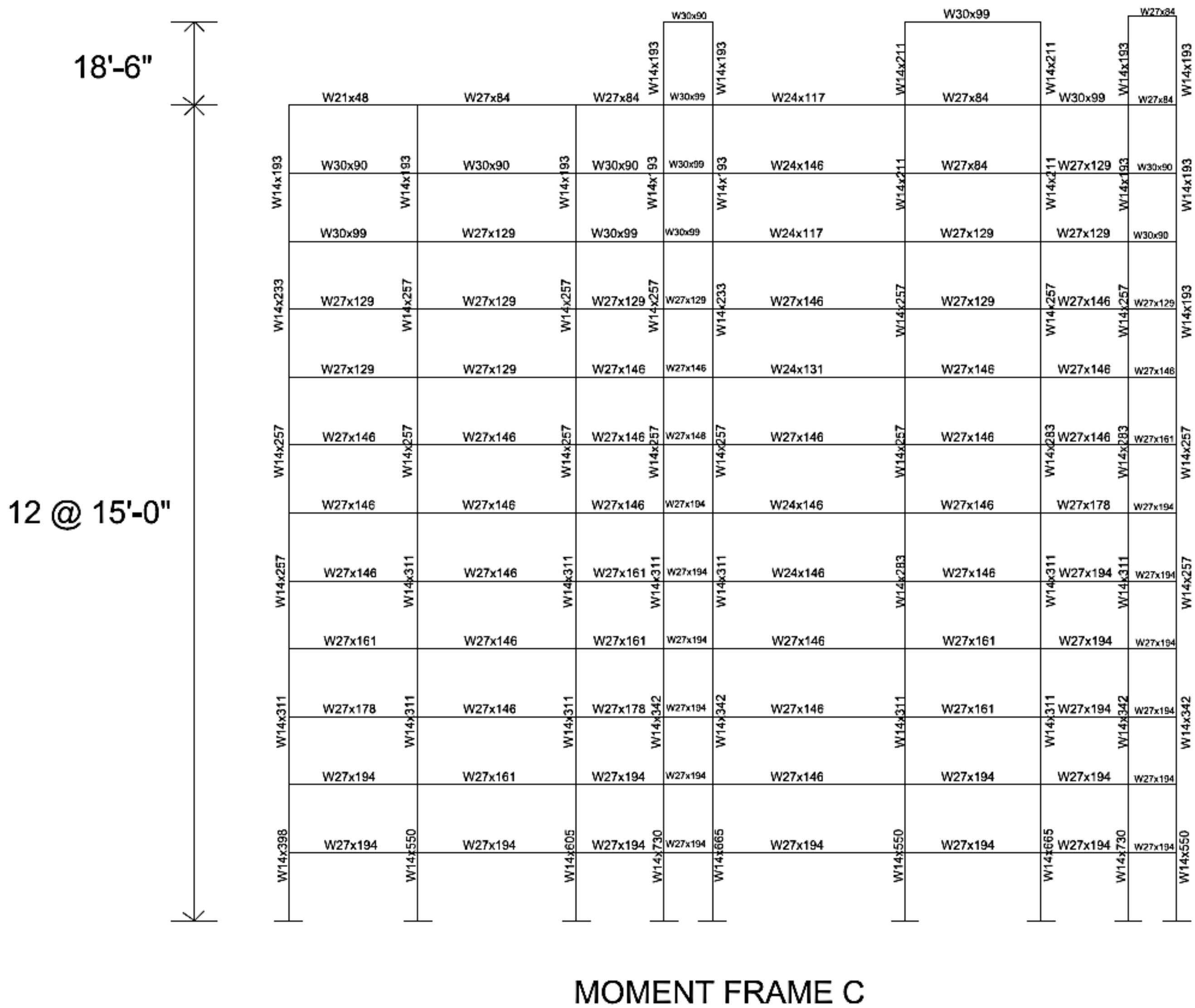


Figure 41 Moment Frame C

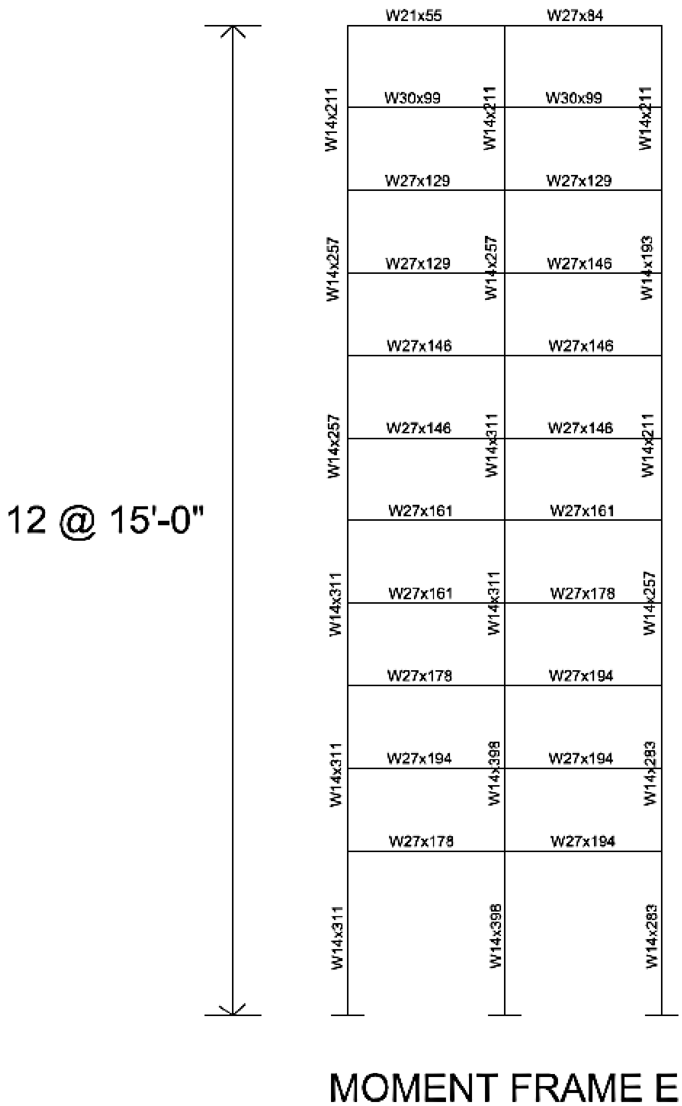


Figure 42 Moment Frame E

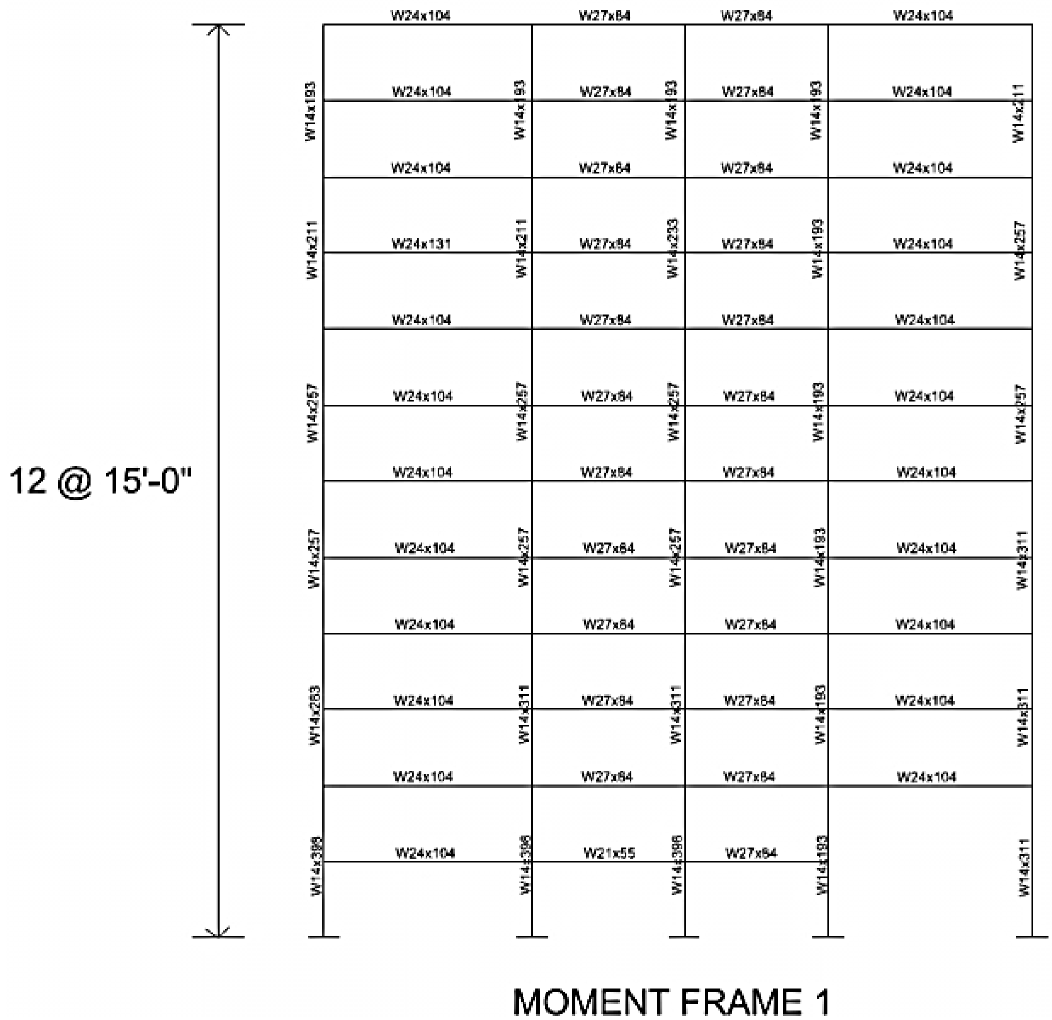


Figure 43 Moment Frame 1

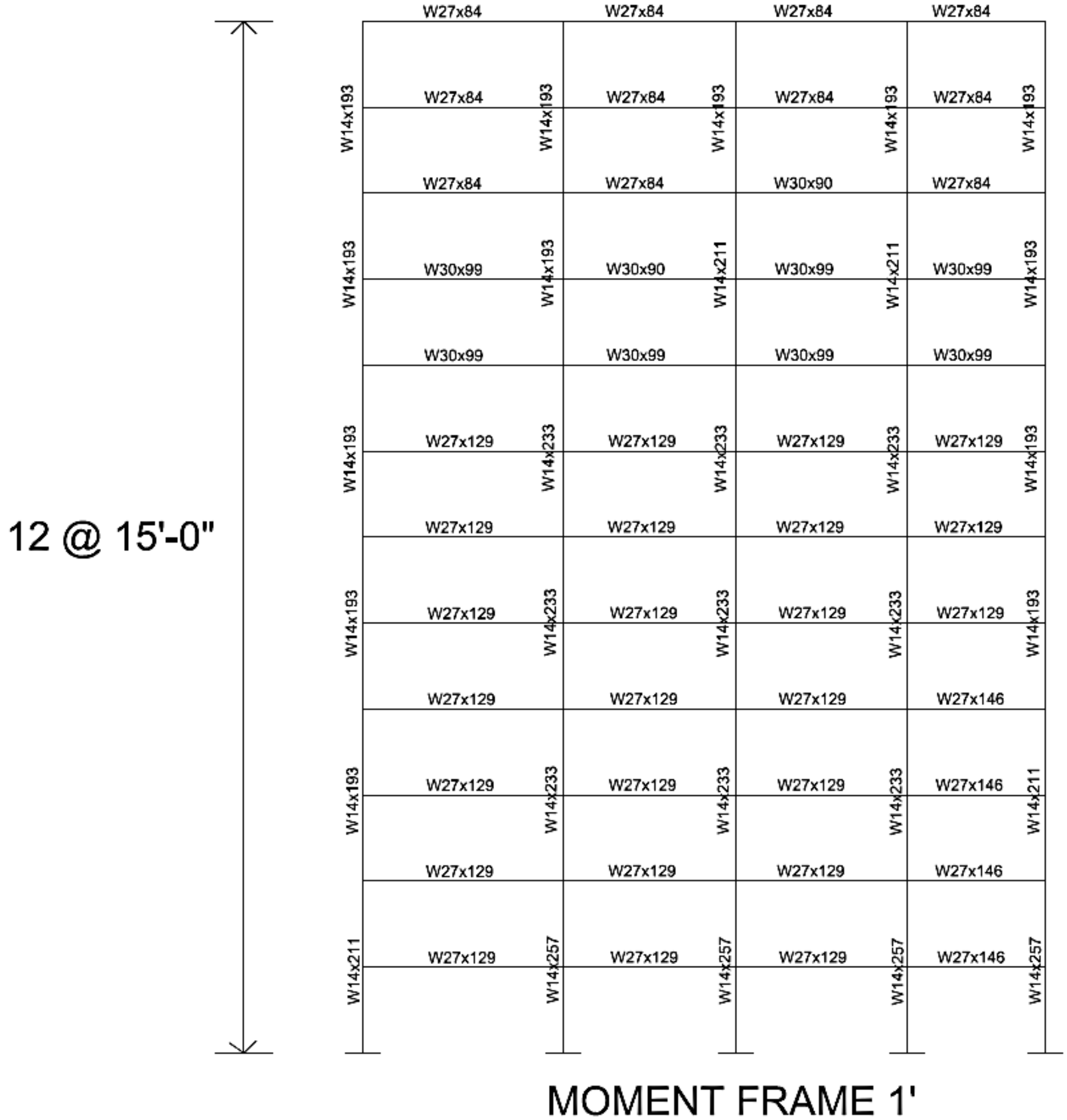


Figure 44 Moment Frame 1'

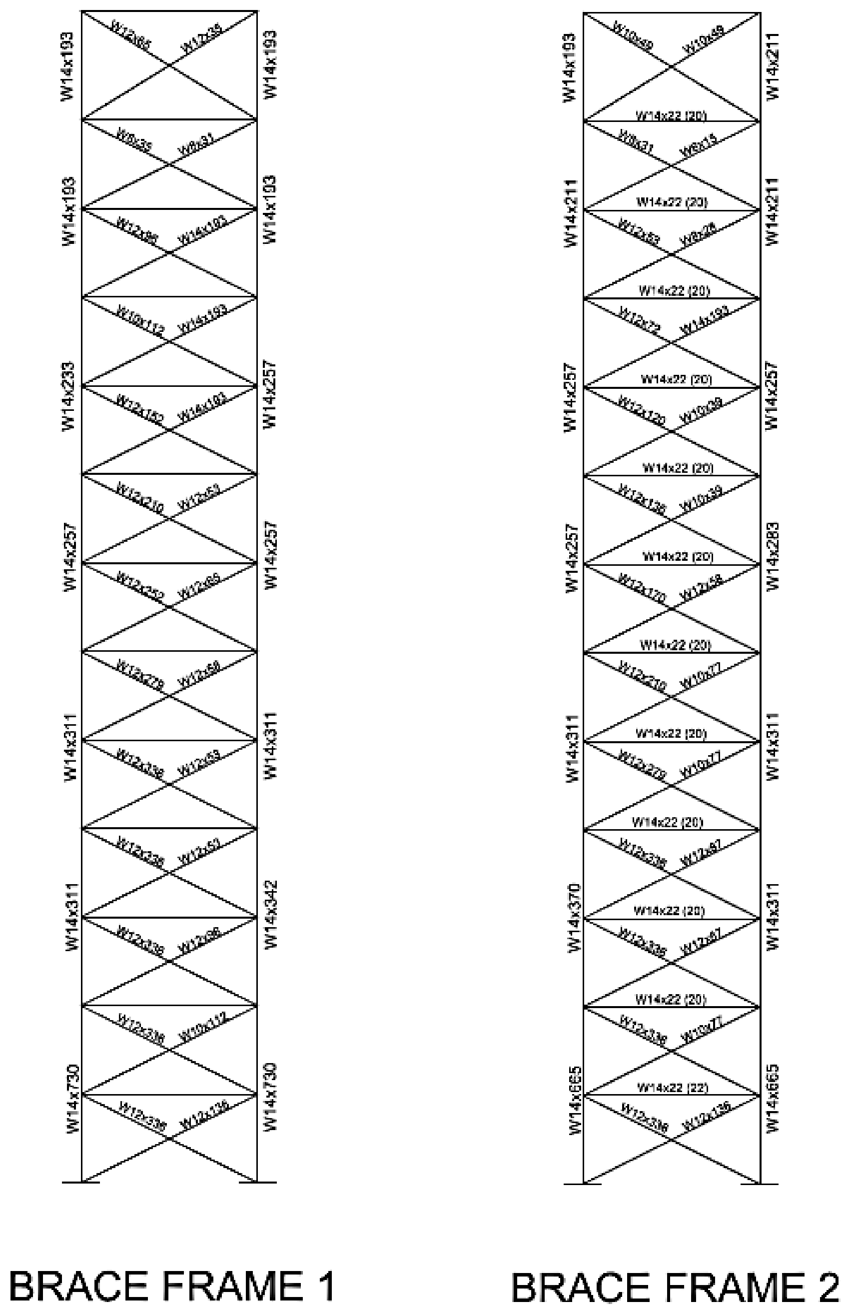


Figure 45 Braced frames 1 and 2

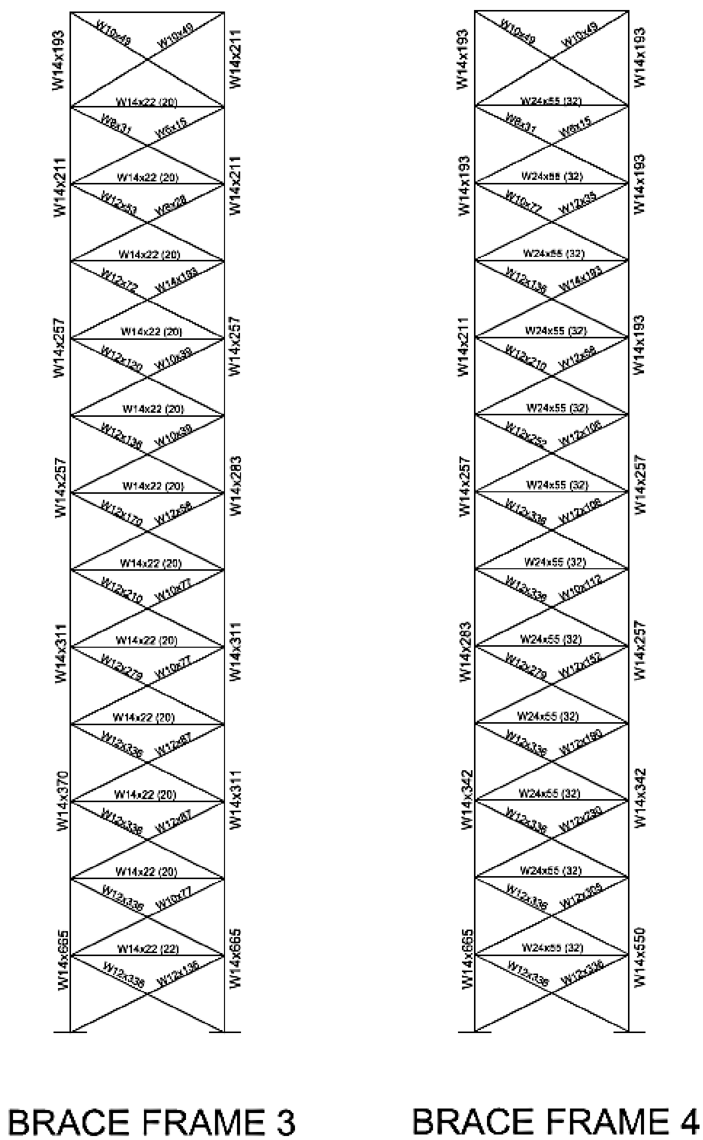


Figure 46 Braced frames 3 and 4

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.

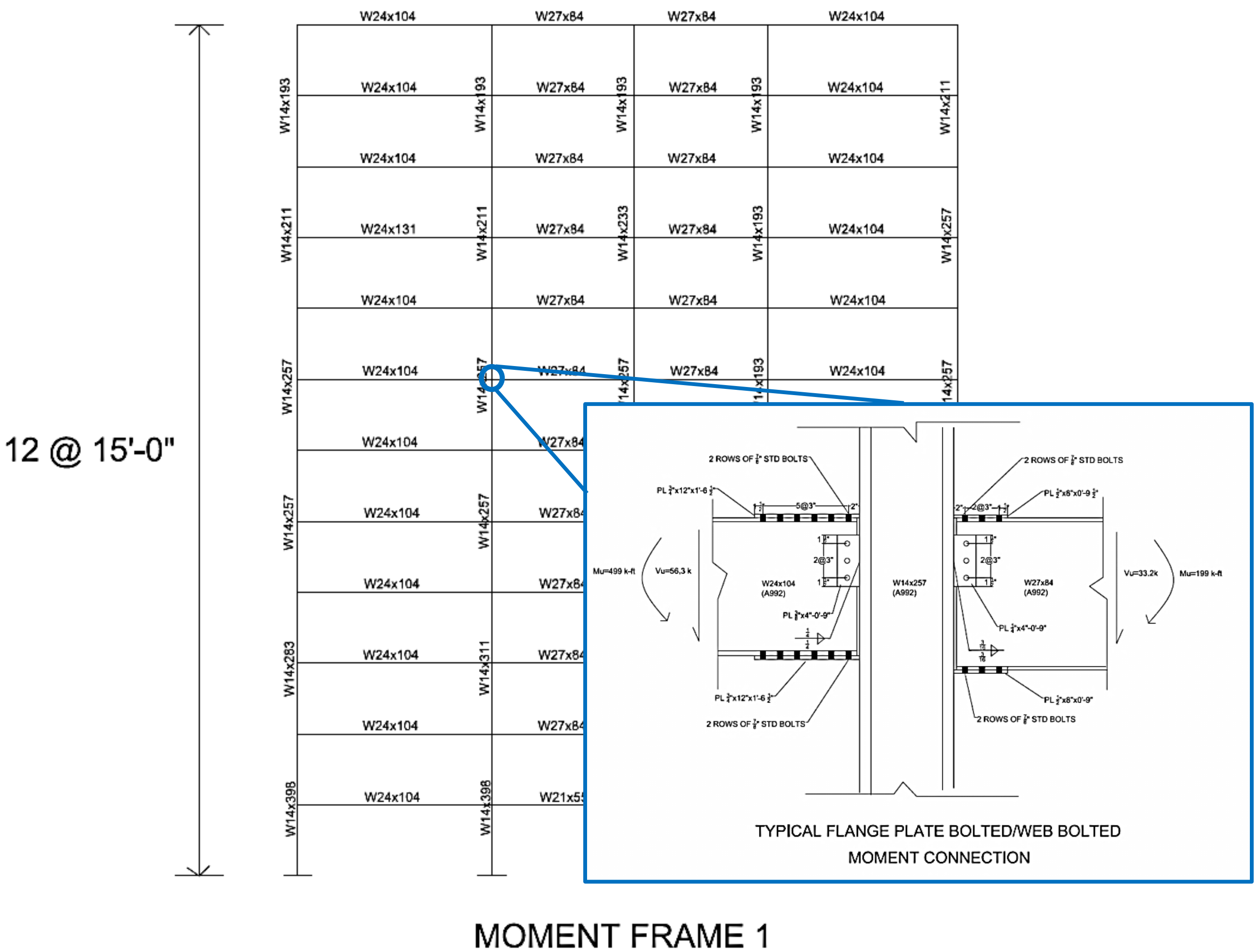


Figure 47 Typical moment frame connection

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

Building Torsion

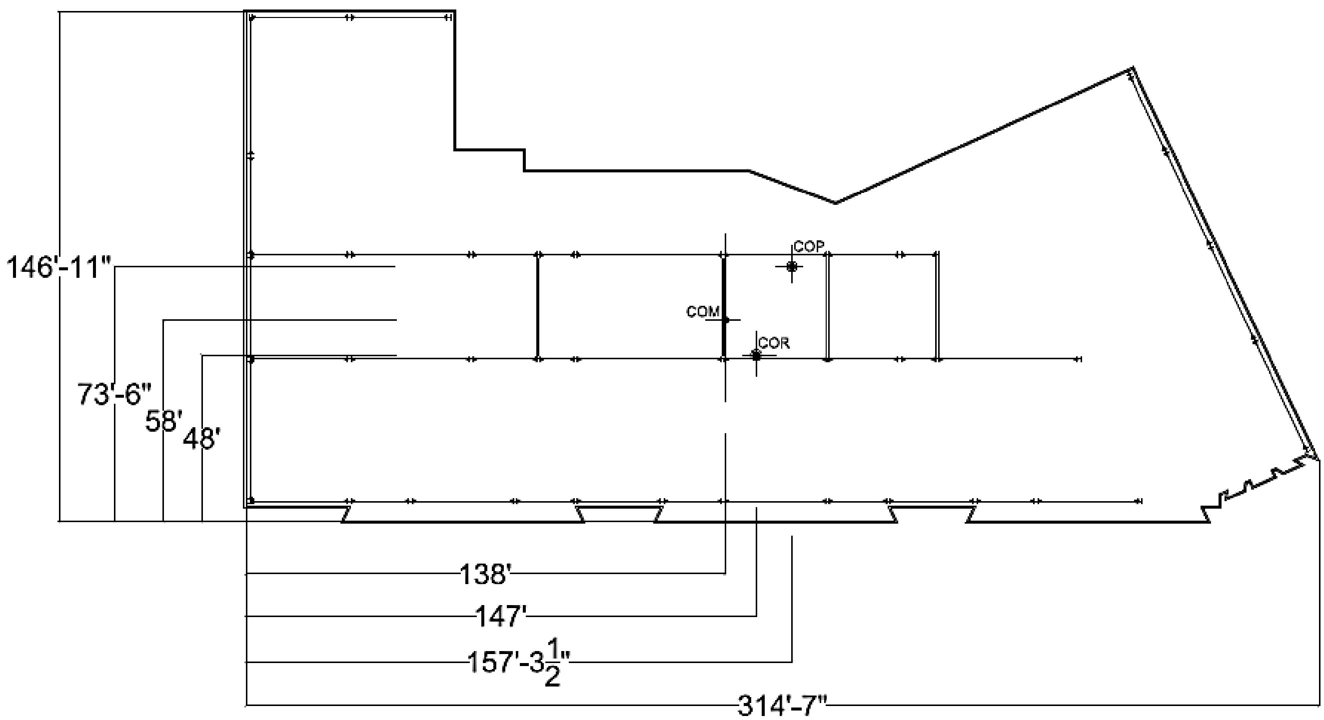


Figure 48 Plan view showing the location of the Center of Mass (COM), Center of Rigidity (COR), and Center of Pressure (COP)

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

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.

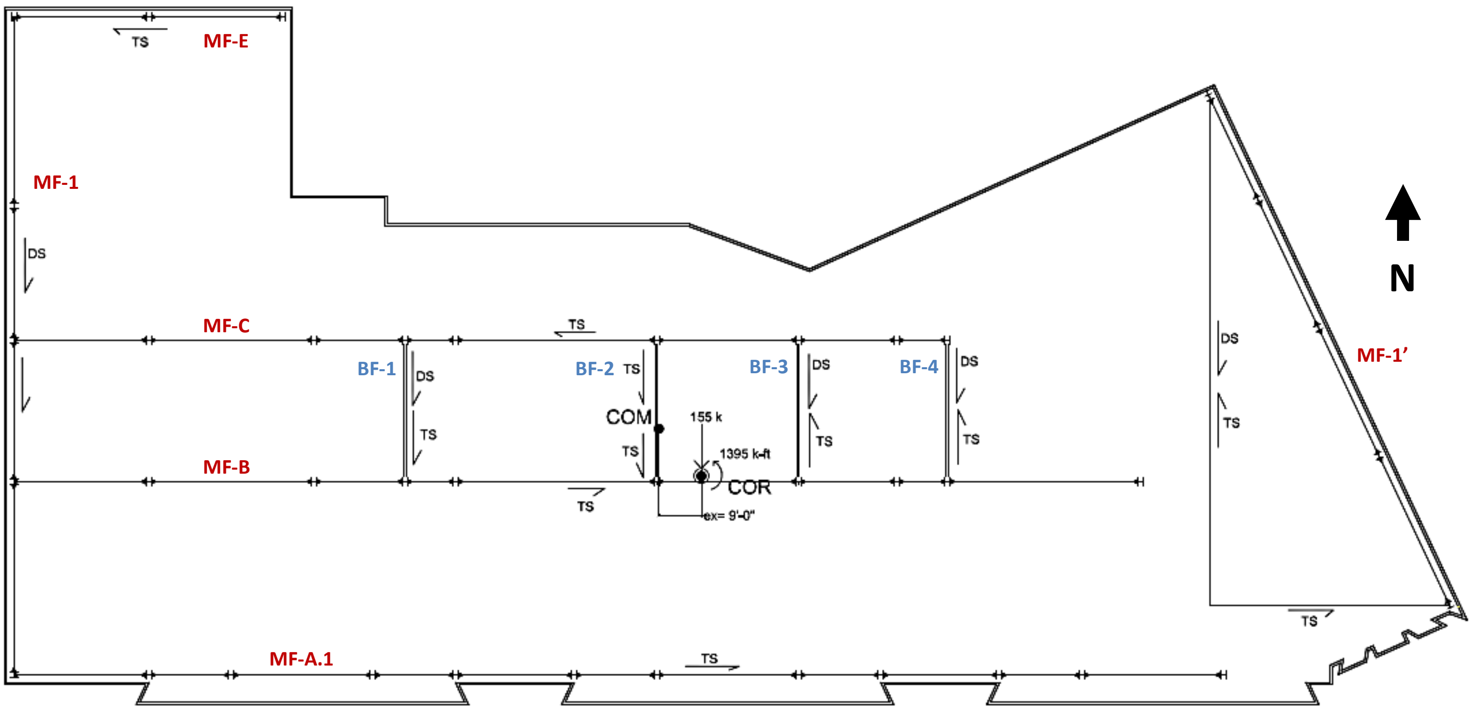


Figure 49 Plan view of the direction of the direct shear (DS) and torsional shear (TS) forces acting on the frames subjected to a 155 kip seismic lateral load acting on the main roof level in the N-S direction

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, $\Delta_a^{a,b}$

Structure	Risk Category I or II	Risk Category III	Risk Category 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	$\boxed{0.020h_{sx}}$	$0.015h_{sx}$	$0.010h_{sx}$

[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.

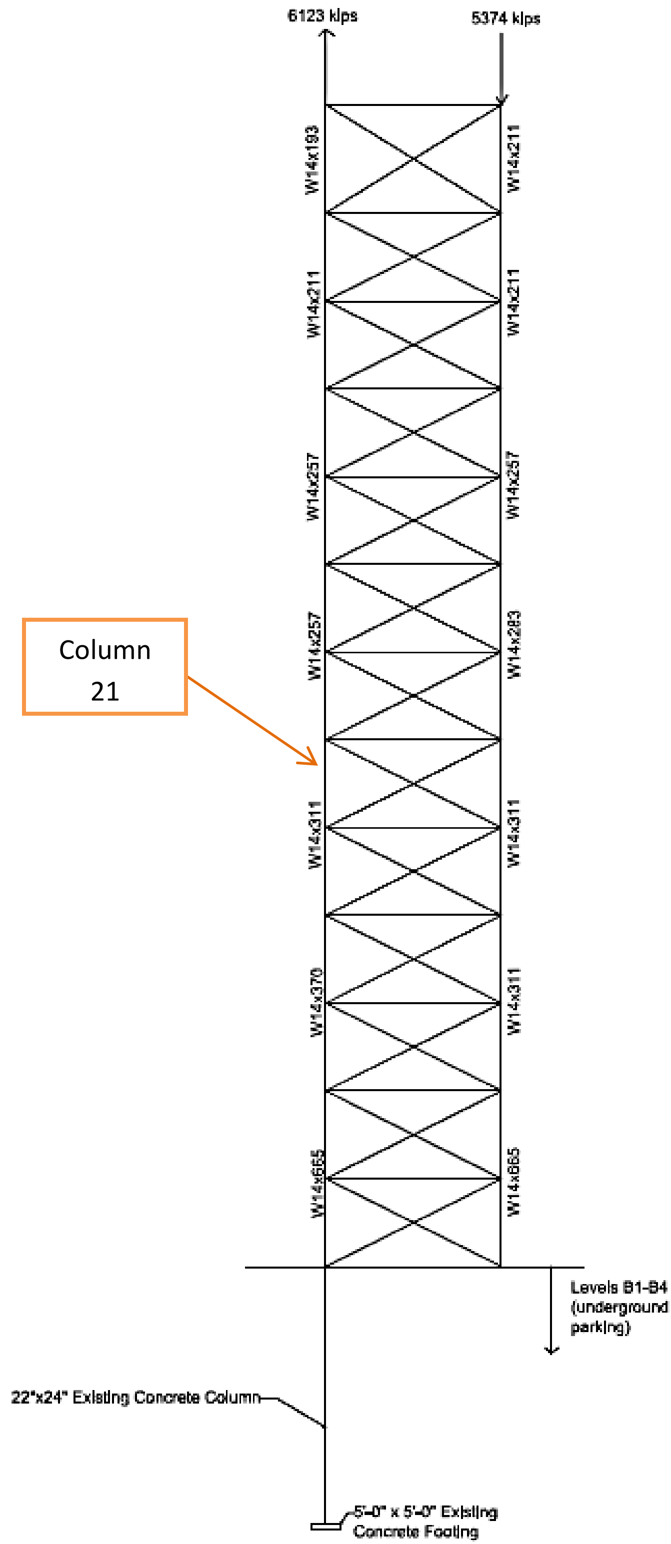


Figure 51 Uplift force braced frame is subjected to due to wind load case 1 acting in the N-S direction

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.

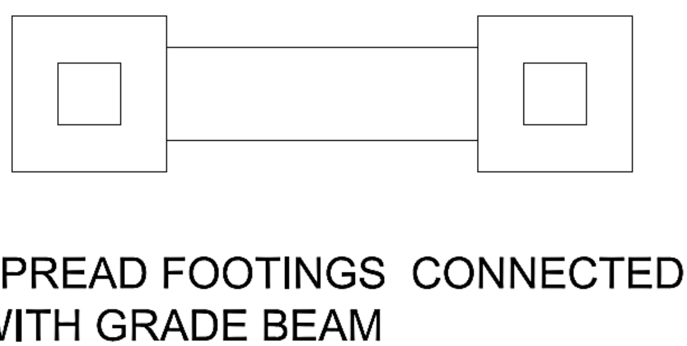


Figure 52 Spread footings connected with a grade beam

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

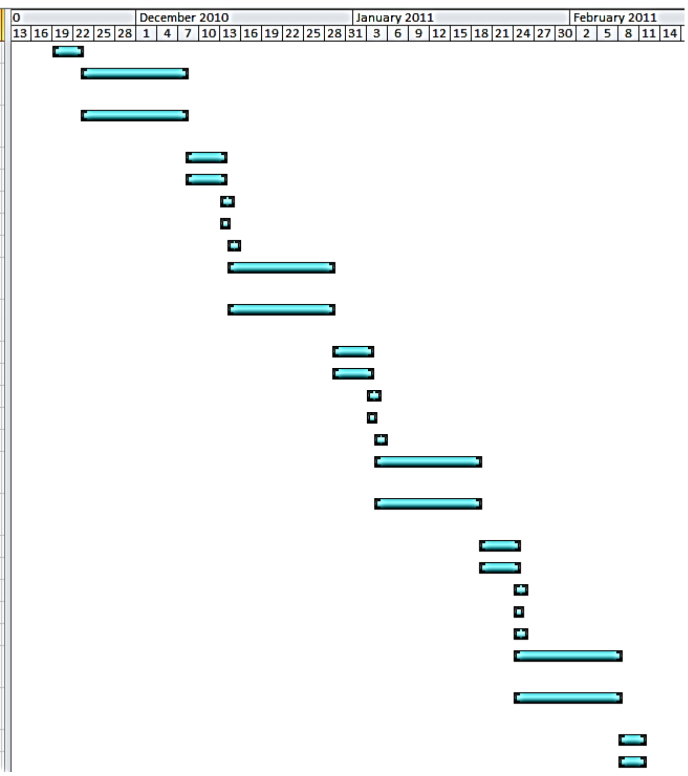
Construction Schedule

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.

	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.

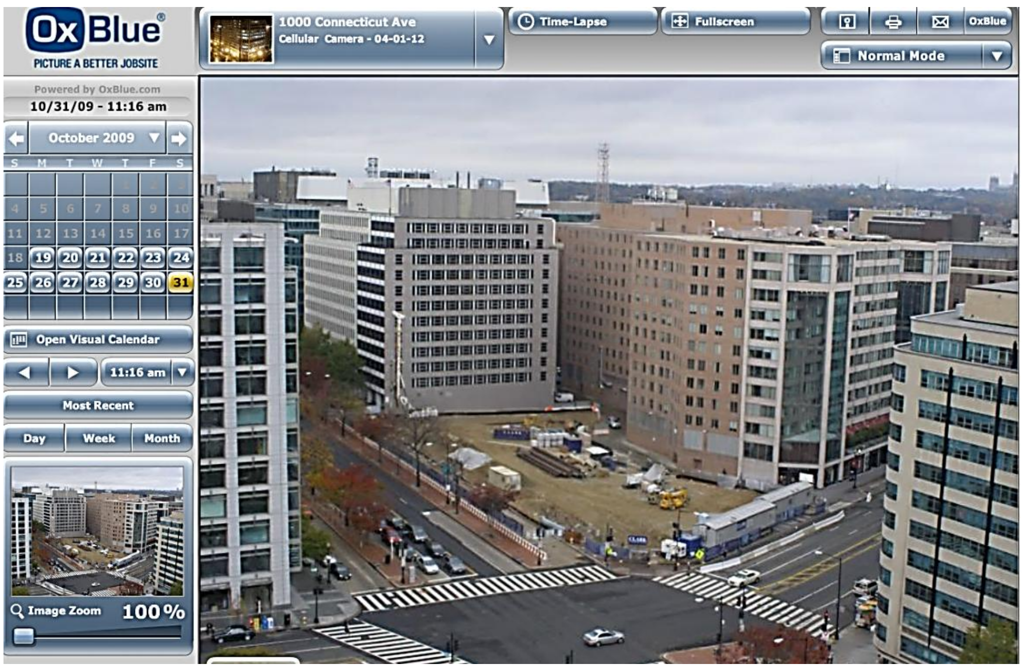


Figure 56 construction site before excavation (October 2009)

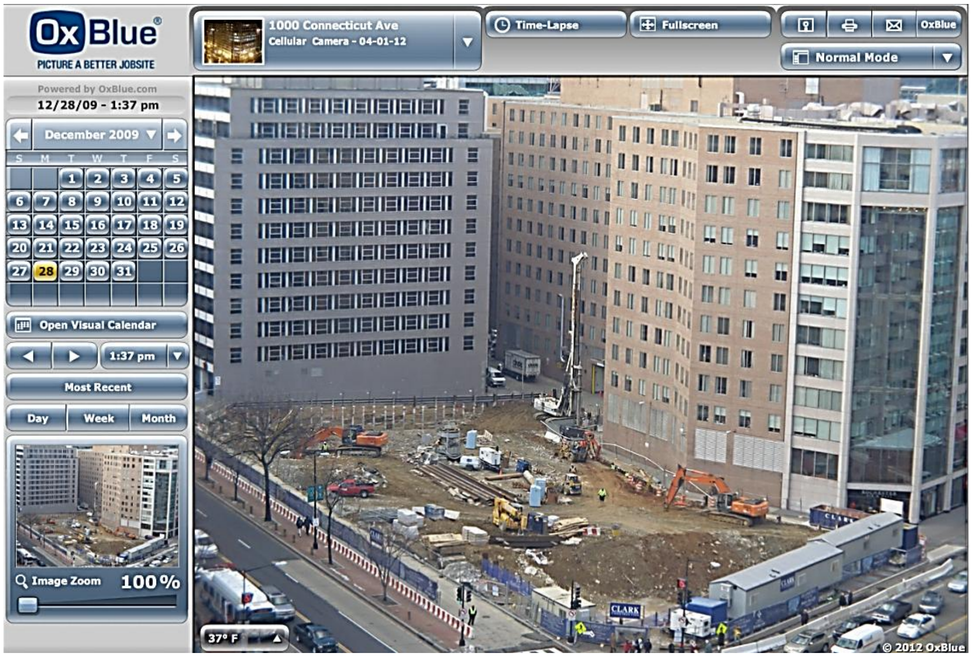


Figure 57 Beginning stages of excavation (December 2009)

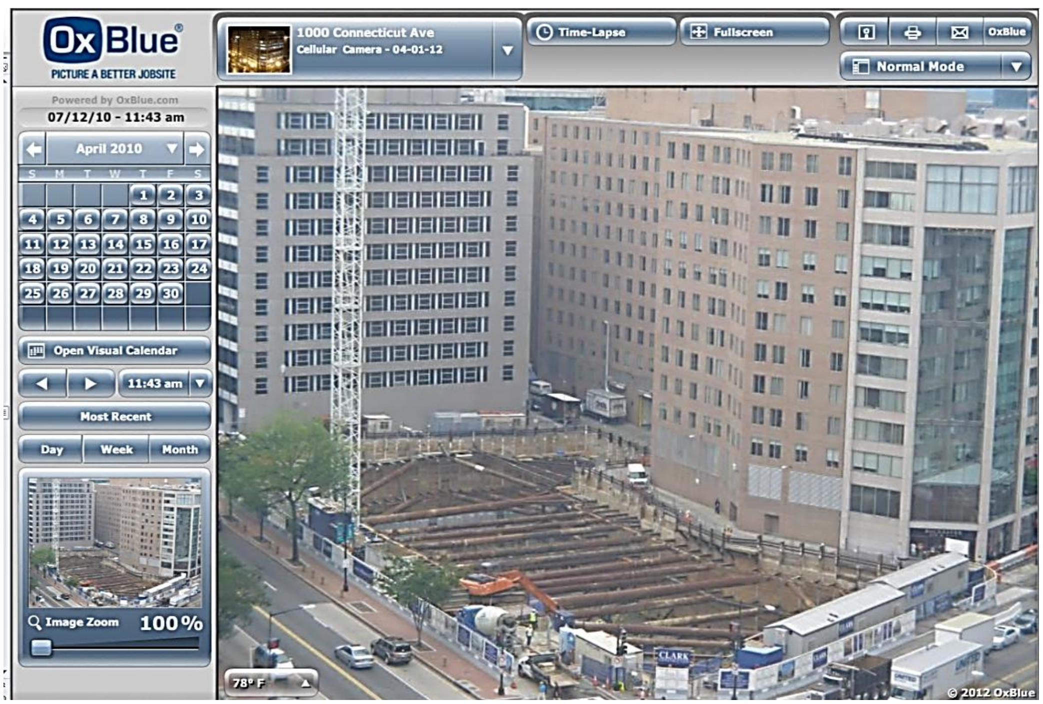


Figure 58 Construction site after excavation (April 2010)

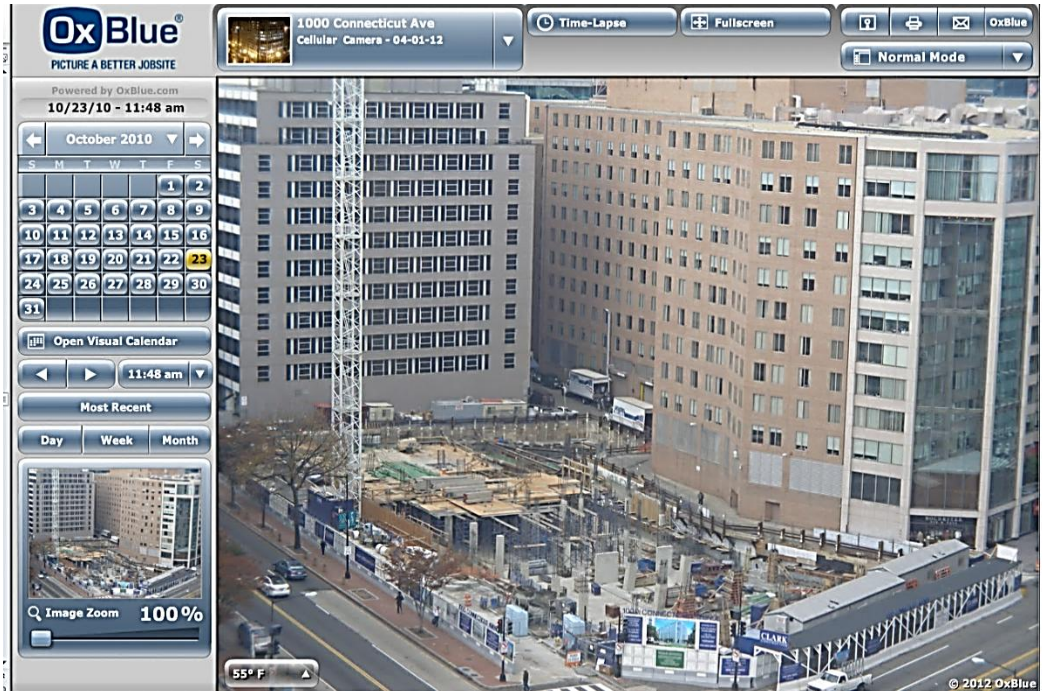


Figure 59 erection of the subgrade four - level underground parking garage (October 2010)

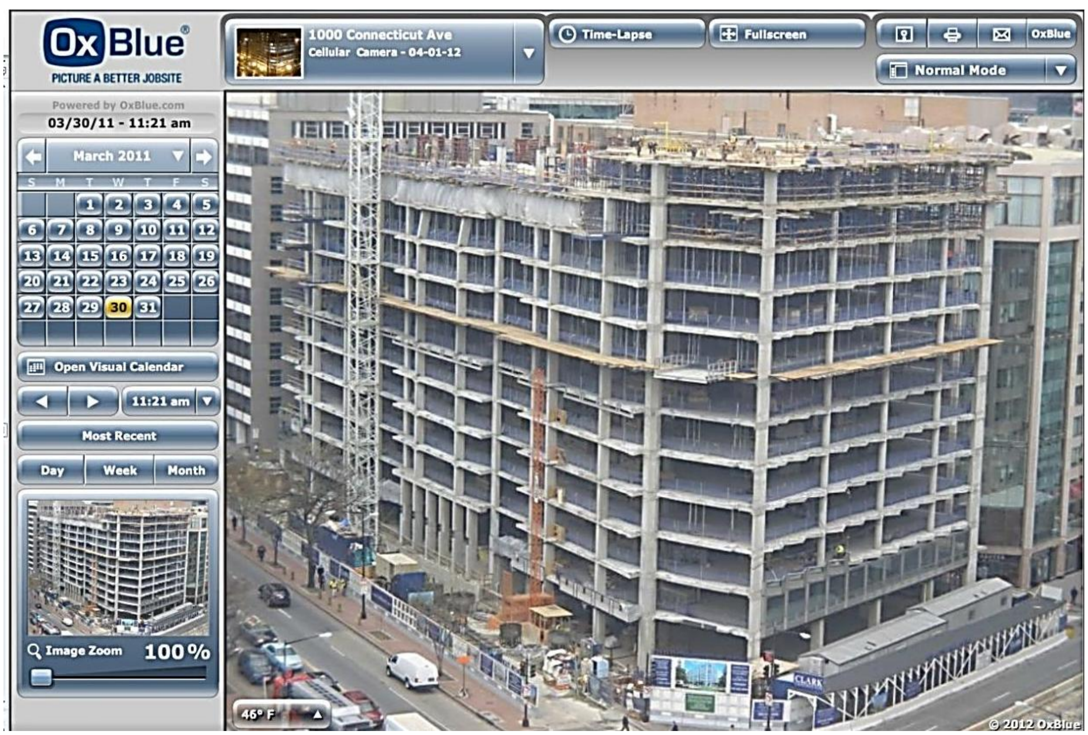


Figure 60 Erection of the twelfth story (main roof) (March 2011)

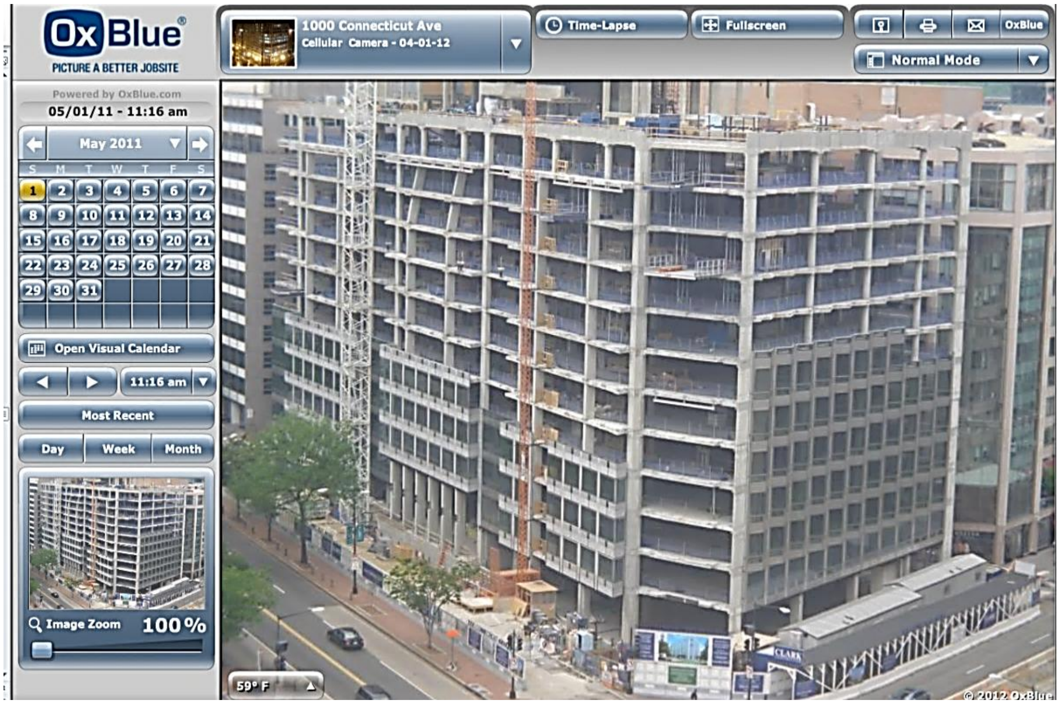


Figure 61 Concrete tops out in May 2011 and the early stages of glass curtain wall installation

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.

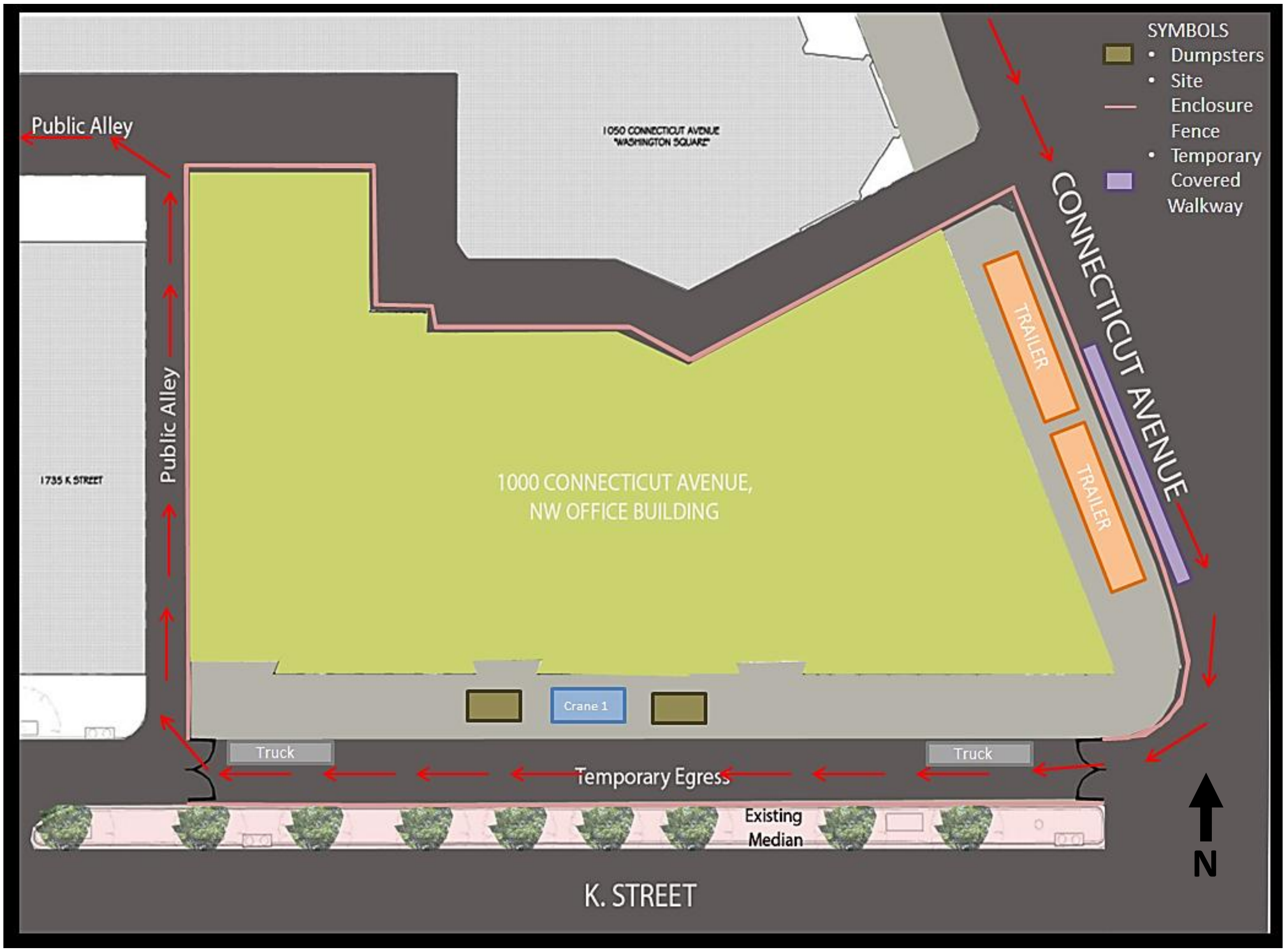


Figure 62 Existing concrete system's site logistics

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.

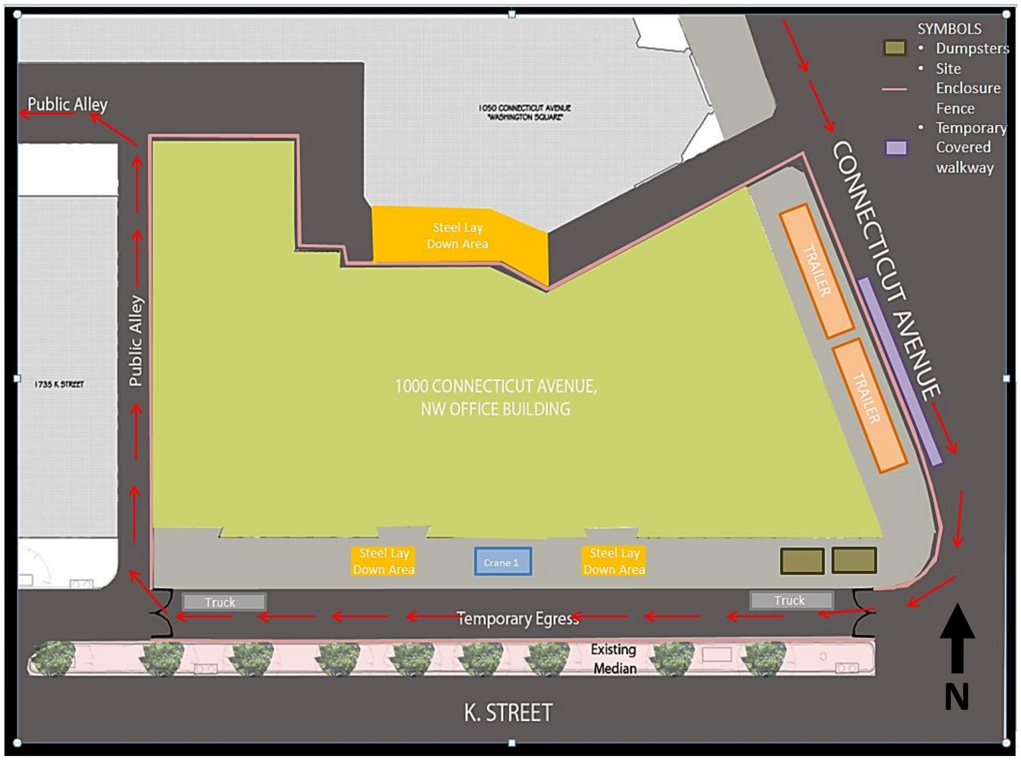


Figure 63 proposed site logistics plan for steel construction

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.

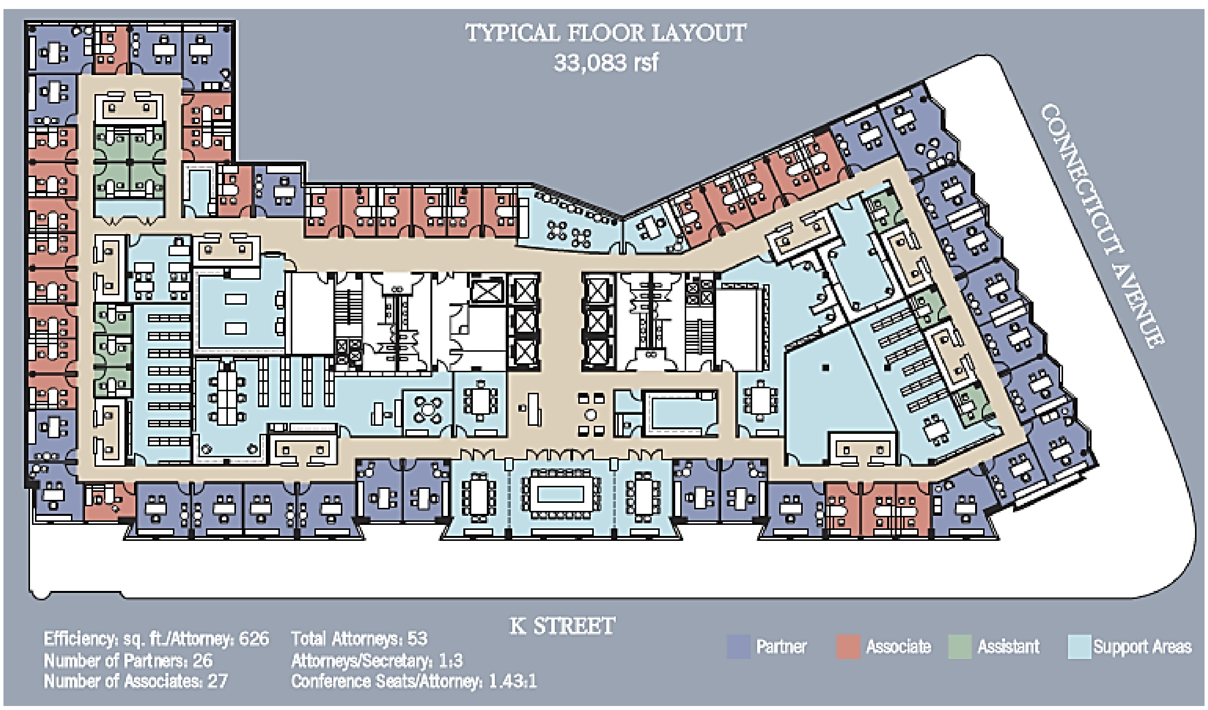


Figure 64 Typical floor plan layout

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 David M. Egan

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

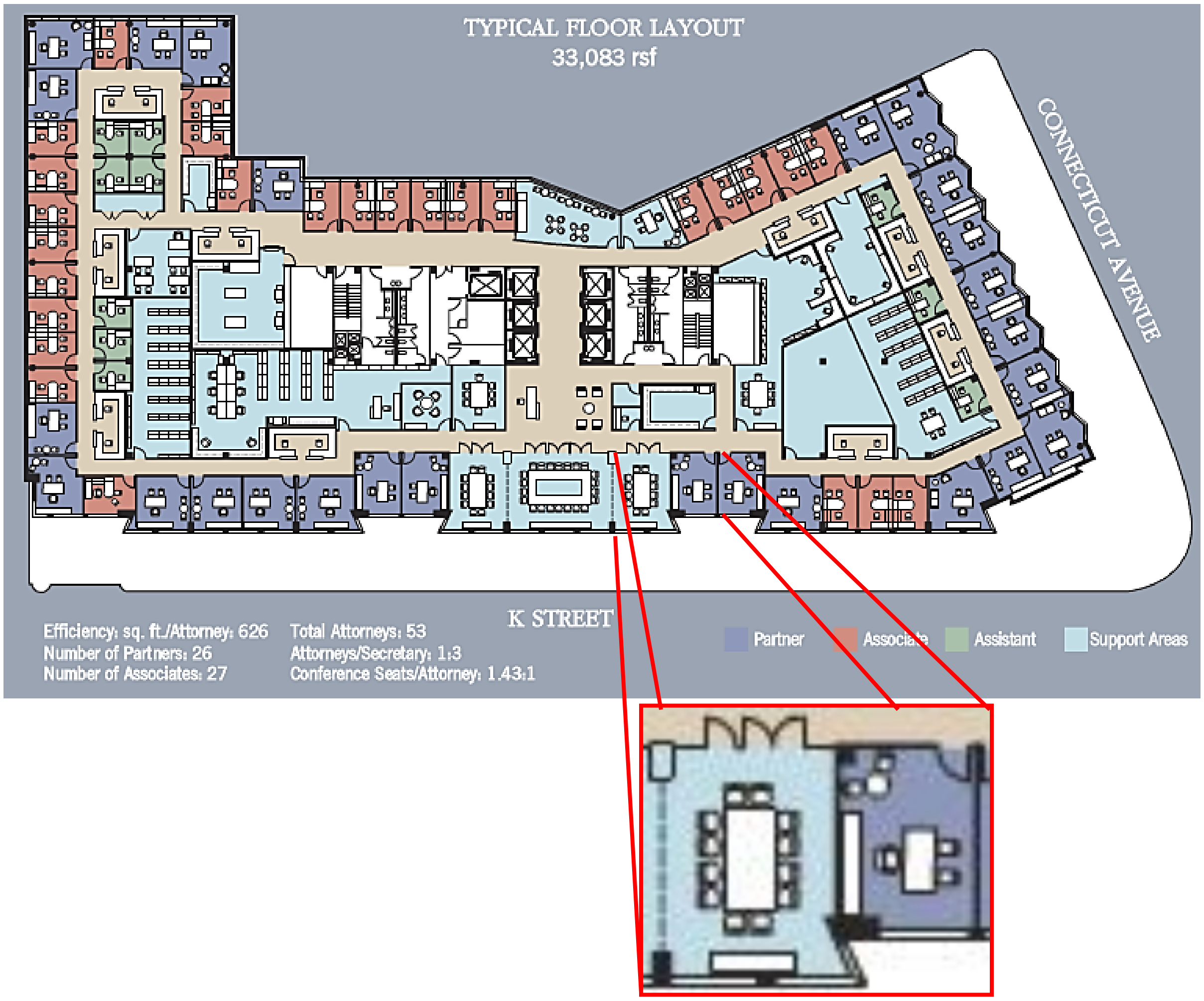


Figure 66 Plan of an attorney's private office (right) and adjacent conference room (left)

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

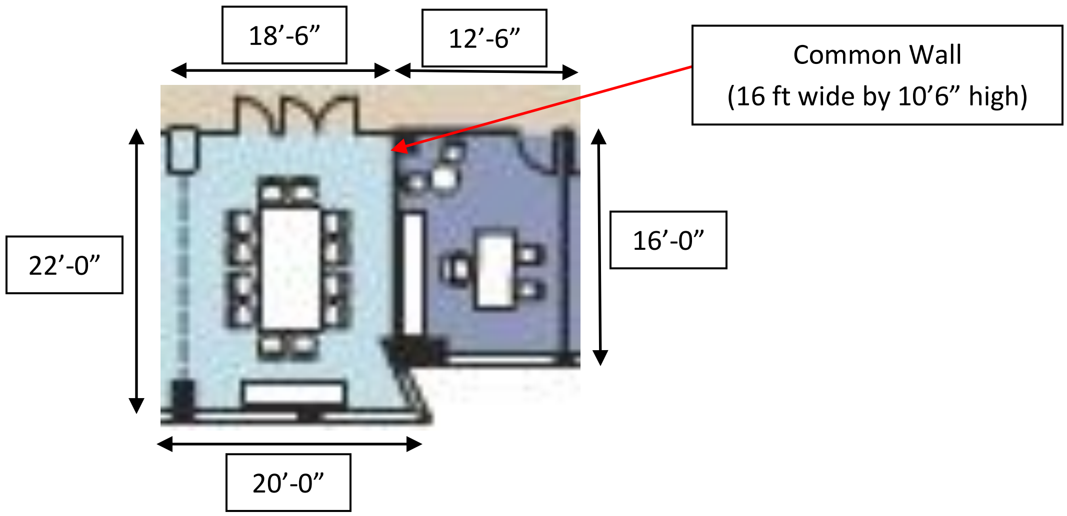


Figure 67 Private office room and conference room dimensions

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.

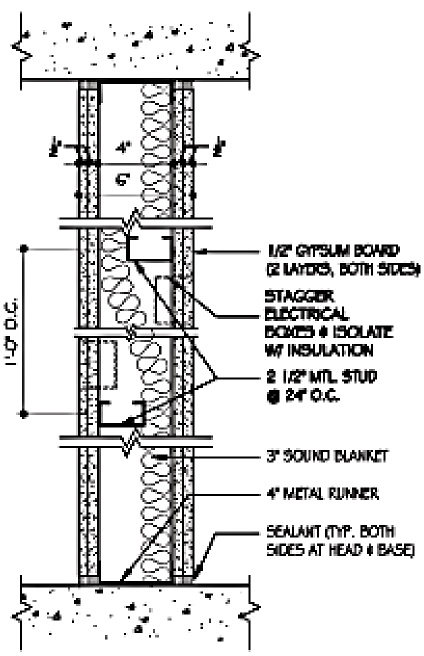


Figure 68 Common partition wall barrier between the private offices and conference rooms with an STC rating of 54

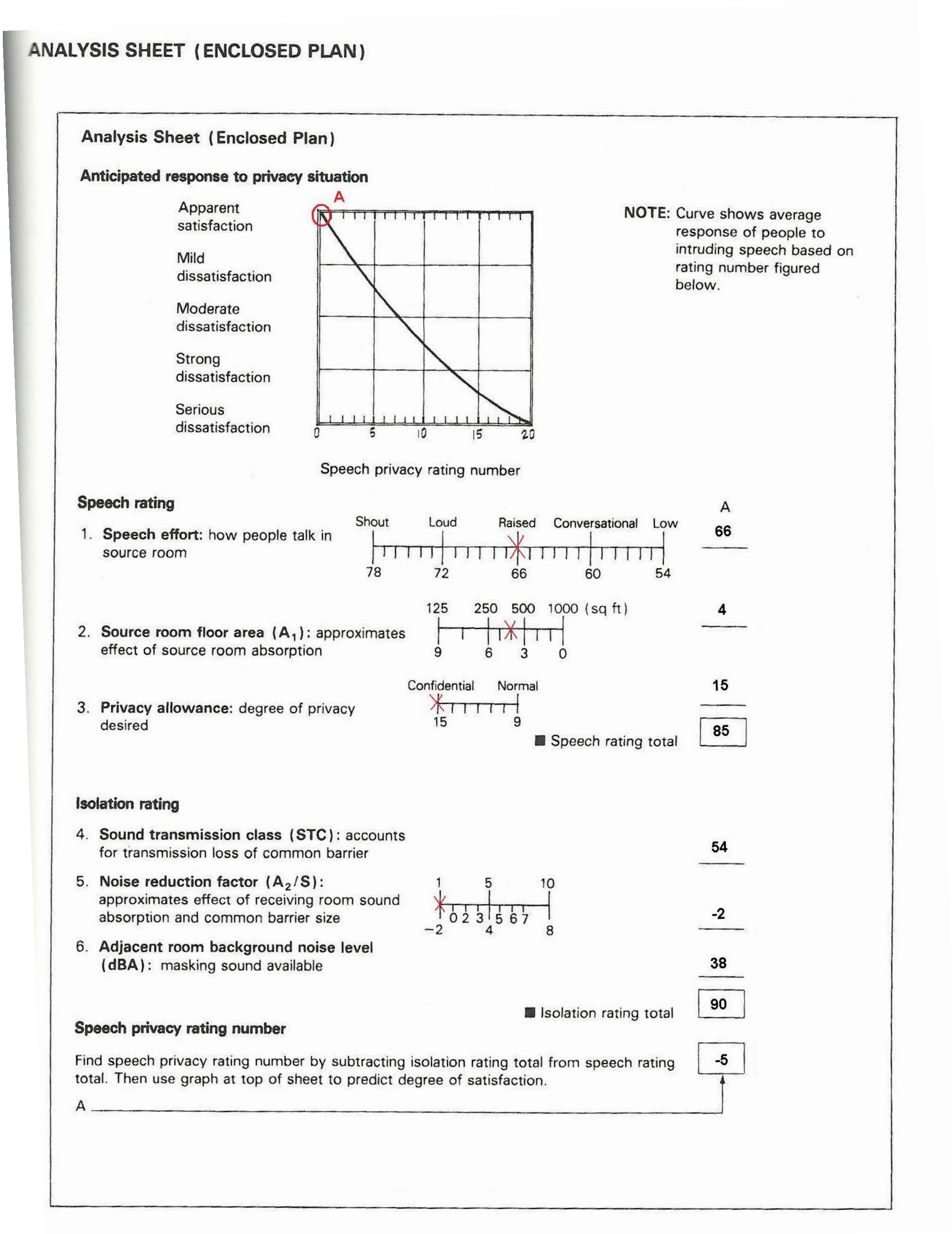
Image obtained from the existing partition wall schedule sheet A1.50

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

A

0 5 10 15 20

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.

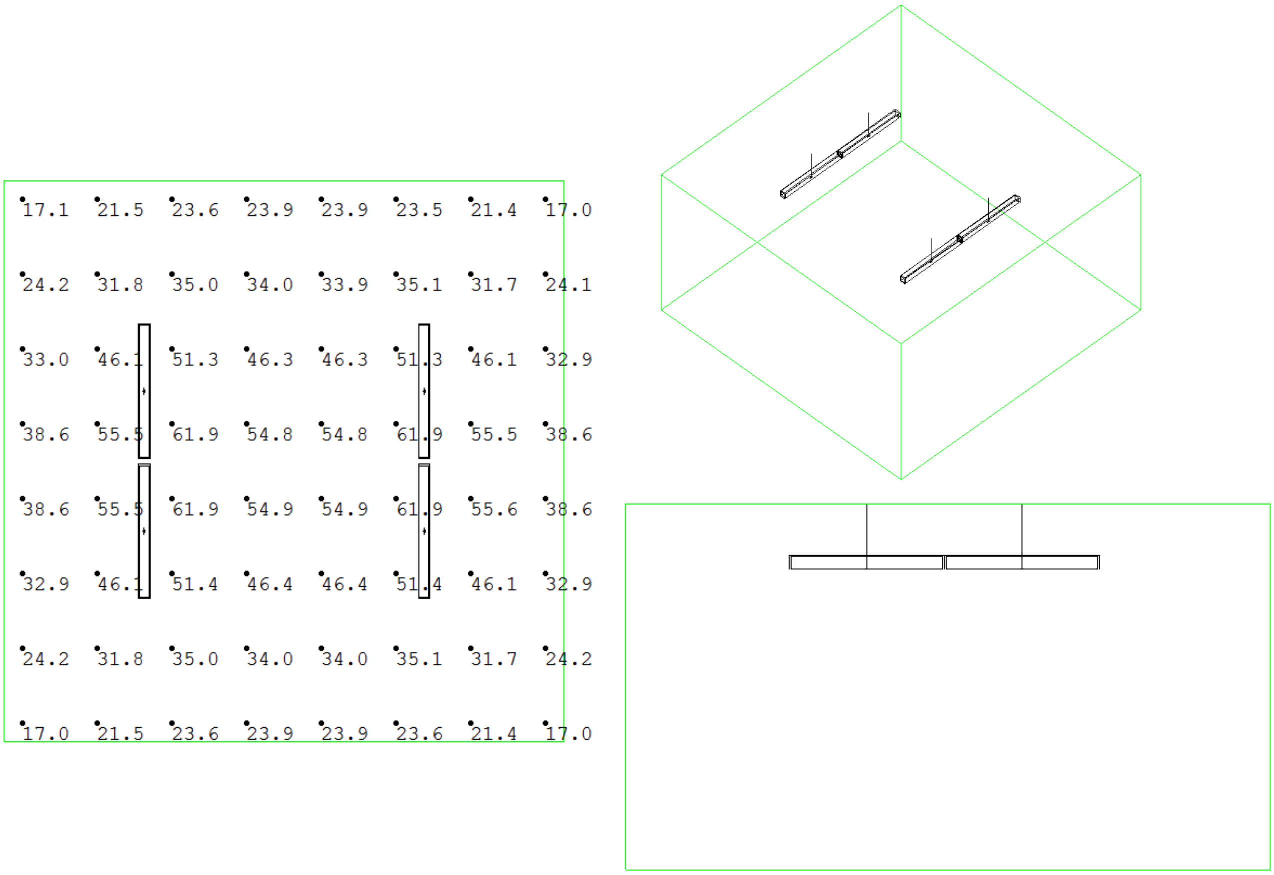


Figure 73 office plan with luminaire layout and illuminance values

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.

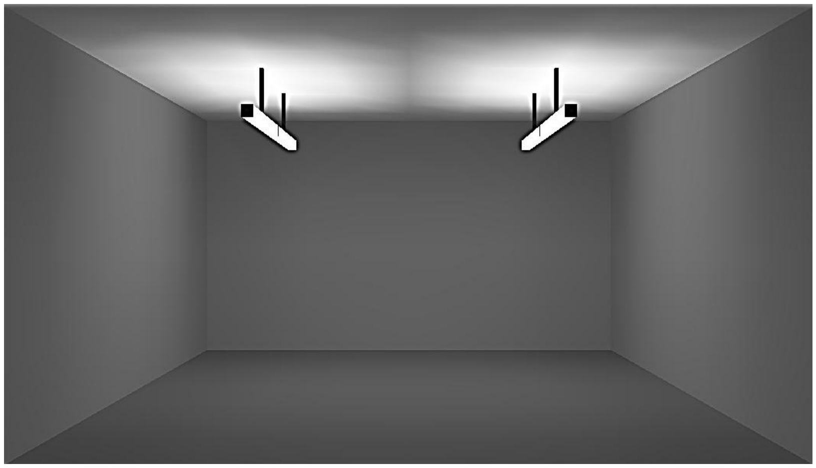


Figure 74 Rendering of the office space with 8’-6” floor-to-ceiling height

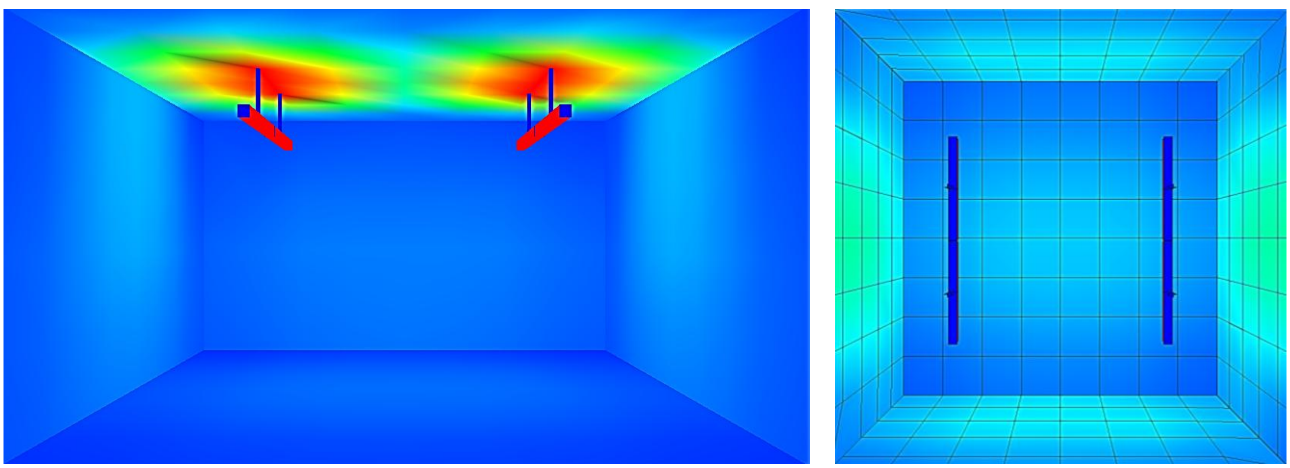


Figure 75 A thermograph of the of the office space with 8’-6” floor-to-ceiling height

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, $\rho_{wall\ avg}$=	$\frac{0.30\,(21ft^2)+0.08(127.5ft^2)+0.60\,(361.5ft^2)}{510ft^2}$ =	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)	$\frac{\text{Mean Luments}}{\text{Initial Lumens}}$	$\frac{2418}{2600}$ =	0.93
	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.

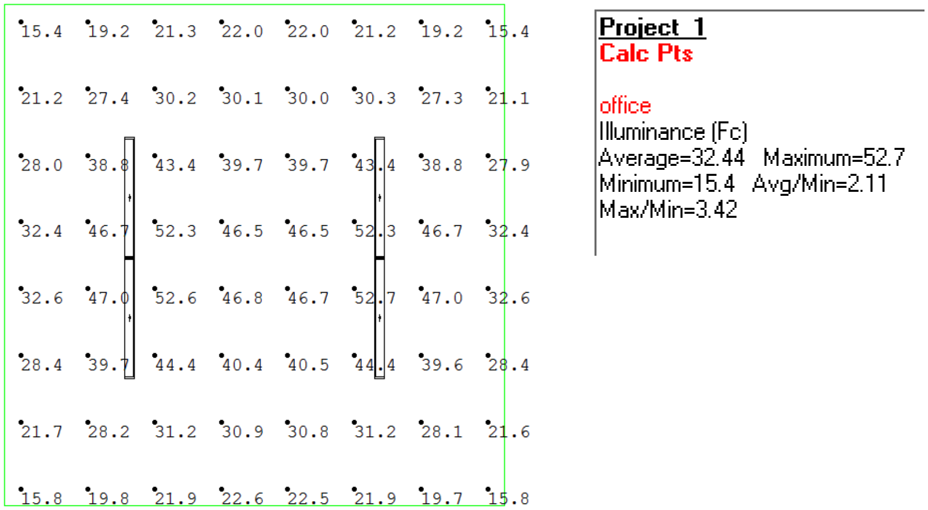


Figure 76 Illuminance values of the office space with a 10'-6" floor-to-ceiling height

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

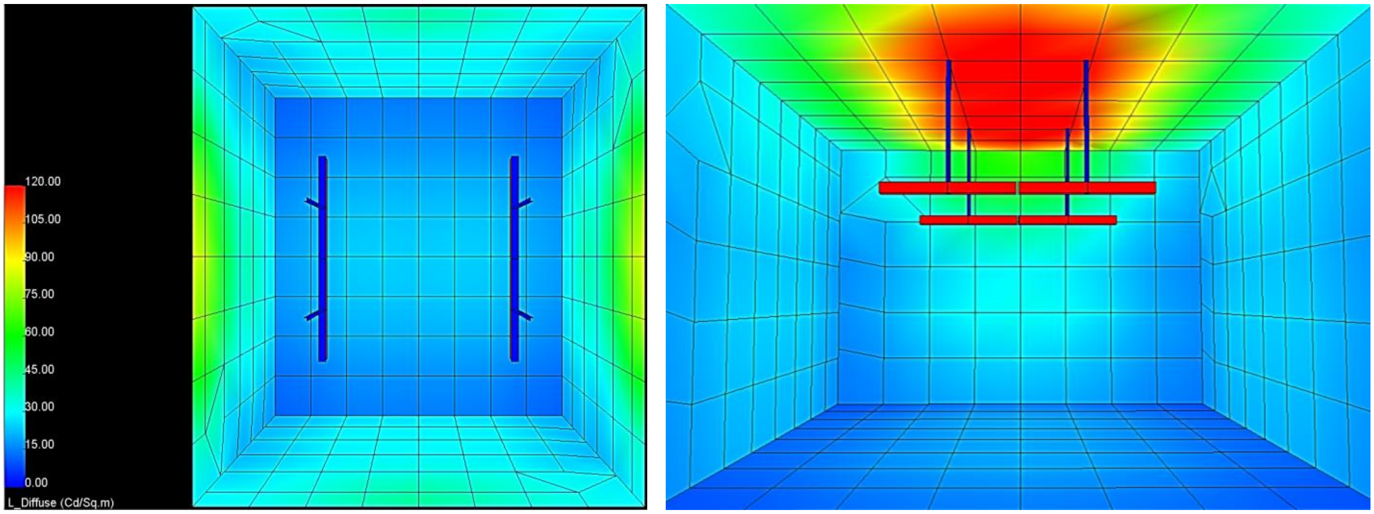


Figure 78 A thermograph of the office space with 10’-6” floor-to-ceiling height

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, $\rho_{wall\ avg}$=	$\frac{0.30(21ft^2)+0.08(157.5ft^2)+0.60(451.5ft^2)}{630ft^2}$ =	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)	$\frac{\text{Mean Luments}}{\text{Initial Lumens}}$	$\frac{2418}{2600}$ =	0.93
	Ballast Factor (BF)	1.0		
	Total LLF	LDD*LBO*LLD*BF=	0.865	
Calculations:				
The lumen method				
E_{avg}=	$\frac{\text{lamps per luminaire x \# of luminaires x CU x LLFs}}{\text{Floor area}}$ =		32.4	(obtained from AGI analysis)
Power Density used=	$\frac{\text{1 ballast/lum(4 lum)(65 W)}}{630 ft^2}$ =	0.413	W/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.

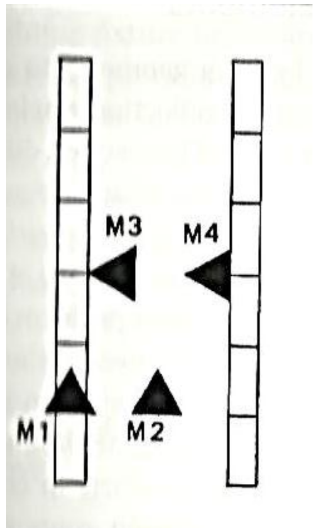


Figure79 preferred and non-preferred arrangements of four possible work planes
Image obtained from "Mechanical and Electrical Equipment for Buildings"

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.

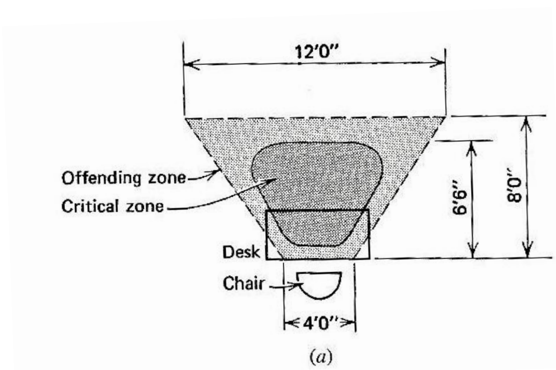


Figure 80 Offending and critical zones for the work plane
Image obtained from "Mechanical and Electrical Equipment for Buildings"

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".

References

American Insititute of Steel Construction (AISC). (2005). *Manual of Steel Construction, 13 ed. Design Supplement.* Chicago, IL: AISC.

American Institute of Steel Construction (AISC). (2010). *Manual of Steel Construction, 14th ed.* Chicago, IL: AISC.

American Society of Civil Engineers (ASCE). (2010). *Minimum Design Loads for Buildings and Other Structures.* Reston, VA: ASCE.

Dilaura, David L., Houser, Kevin W., Mistrick, Richard G., Steffy, Gary R. (2011). *Illuminating Engineering Society: The Lighting Handbook, Reference and Application, 10th ed.* New York, NY: Illuminating Engineering Society of North America (IESNA).

Egan, M. D. (1988). *Architectural Acoustics.* The McGraw-Hill Companies, Inc.

Gould, Frederick E. (2005). *Managing the Construction Process, 3rd ed.* Upper Saddle River, NJ: Pearson Education, Inc.

Kloiber, Lawrence A. (2004). *Orthogonal and Skewed Shear Connections Design and Detailing Requirements.* Retrieved February 12, 2012

RS Means. (2012). *RS Means Building Construction Cost Data, 70th ed.* Kingston, MA: RS Means Company.

Schwinger, Clifford W.,Campbell, Todd R. (2010). *Tips for Designing Constructible Steel-Framed Structures*. Structure Magazine, 33-35.

Steel Deck Institute. (2008). *Vulcraft Steel Roof & Floor Deck.* Florence, SC.

Stein, Benjamin, Reynolds, John S., Grondzik, Walter T., Kwok, Alison G. (2006). *Mechanical and Electrical Equipment for Buildings, 10th ed.* Hoboken, NJ: John Wiley & Sons, Inc.

Appendix A: Gravity System Design

Appendix B: Wind Load Calculations

Appendix C: Seismic Load Calculations

Appendix D: Typical Connections Design and Analysis

Appendix E: Construction Management Breadth Analysis

Appendix F: Acoustics and Lighting Breadth Analyses

Appendix G: Typical Floor Plans