AE Senior Thesis

Final Report

8^{th} Street Office Building | Richmond, VA

Carol Gaertner | Structural Option

AE Consultant: Professor Kevin Parfitt

April 7, 2010

8th Street Office Building

Richmond, VA

carol lynn gaertner

structural option

www.engr.psu.edu/ae/thesis/portfolios/2010/clg5017

general data

- project area: 307,178 ft^2
- height: 4 levels below grade, 10 above + penthouse
- total cost: $113 million
- construction dates: not scheduled
- delivery method: design-bid-build

project team

- owner: Commonwealth of Virginia
- architects: Commonwealth Architects; Perkins + Will
- landscape architect: Snead Associates, P.C.
- preconstruction services: W.M. Jordan
- civil: Draper Aden Associates
- structural: Rathgeber/Goss Associates
- MEP: Integral Performance Engineering
- lighting: Grenald Waldron Associates
- telecommunications: WB Engineers|Consultants
- fire protection: The Protection Engineering Group

arch

- spaces: retail, secure main lobby, assembly areas and offices
- atrium: 6 stories, acts as a connector to the adjacent building
- facade: glass curtain walls and precast concrete panels
- terraces: planters atop granite on the 3rd, 7th and 10th floors
- roofing: standing seam stainless steel system around perimeter

struc

- materials: concrete parking garage, steel superstructure
- foundation: 48” thick mat reinforced with #10 bars
- floors: lightweight concrete over 18 gage composite metal deck
- lateral system: 12” thick concrete shear walls around 4 cores
- roof: structural slab with 20 gage metal roof deck

mech

- heating: three 250 HP boilers for 14,500,000 MBH load
- cooling: three 500 ton centrifugal chillers for 1,450 ton load
- air systems: variable air volume air handling units
- smoke control: atrium smoke removal fans on adjacent roof

ltg/elec

- office lighting: indirect/direct pendants to 40-45 footcandles
- circulation lighting: compact fluorescent downlights
- interior distribution: three 5kV to 480/277V unit substations
- exterior distribution: two transformers in a vault on 8th Street provided by Dominion Virginia Power

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Acknowledgements

I am very grateful to the following firms and individuals for their support and assistance:

Rathgeber/Goss Associates

Commonwealth Architects

- Robert Burns

American Hydrotech, Inc.

- Matthew Carr
- Nathan Griswold
- Jeff Thomas

The Pennsylvania State University:

- Professor M. Kevin Parfitt
- Dr. Louis Geschwinder
- Professor Robert Holland
- Dr. Andres Lepage

Thank you to all of my family and friends, especially my parents.

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Executive Summary

The 8th Street Office Building is a government office building located in Richmond, Virginia whose main purpose is to serve the needs of the Virginia General Assembly during renovations to the Virginia Capitol. Once the renovations are completed, it is expected that the office building will be utilized by various Virginia government agencies. The 8th Street Office Building consists of four underground parking garage levels, ten floors above grade, and a mechanical penthouse. The building is approximately 307,000 square feet in size and 176'-5" in height, with floor to floor heights ranging from 10'-0" for the parking garage levels to 18'-10" for the second floor.

The structural system of the 8th Street Office Building is comprised of composite steel framing and a lightweight concrete slab on metal deck floor system. A typical bay through the interior of the building is 20'-0" by 30'-0", and a typical bay around the perimeter of the building is 20'-0" by 40'-6". However, several variations on these dimensions exist throughout the building due to façade and floor plan irregularities. The lateral system consists of sixteen reinforced concrete shear walls located around four transportation cores within the building.

Unfortunately, the 8th Street Office Building is not currently scheduled for construction due to a deficit in the Virginia state budget. In fact, the design of the building has been stalled since 2008 at approximately 85-90% completion until funds are allocated for the remainder of the project. Through discussions with the structural design engineers, it was discovered that the design for the current lateral system for the 8th Street Office Building has not been finalized. Therefore, this final thesis report investigates several steel lateral systems as alternative options to the reinforced concrete shear walls.

Steel plate shear walls were eliminated as a feasible system for the 8th Street Office building due to their incompatibility with openings, despite having many advantages similar to those of braced frames. Therefore, the first steel lateral that was designed in depth for the 8th Street Office Building consists of braced frames. However, it was discovered that seismic drift limitations significantly governed the design of the braced frames. For example, it was necessary to increase column sizes at the ground level from W14x283 to W14x255 in order meet drift requirements. Moment frames were also evaluated for feasibility since they do not obstruct openings. Unfortunately, a schematic design of the moment frames indicated that slenderness issues resulting from large floor to floor heights could not be resolved. Finally, two dual steel systems of braced frames and moment frames were analyzed in order to evaluate drift control. Ultimately, one of the dual systems was chosen as the optimal steel lateral system for the 8th Street Office Building. The computer modeling program RAM Structural System was utilized extensively in the designs of the various steel lateral systems.

The architectural breadth study involves a redesign of the overall service core in order to maximize the amount of useable space for the tenants. Ultimately, 1,440 square feet were gained as a result of eliminating a corridor. The sustainable breadth study involves the selection of an extensive green roof as well as the sizing of three 1,000 gallon collection tanks for greywater use within the 8th Street Office Building.

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Building Introduction

The new 8th Street Office Building will be located in the bustling Richmond, VA commercial district near the Virginia State Capitol Building. It is intended to be a legacy building that will serve both the needs of the state government and the general public. Initially, the Virginia General Assembly will occupy the 8th Street Office Building for approximately five years while renovations to the Capitol Building are being completed. After that time, it is expected that various Virginia government agencies will move into the new office building.

The 8th Street Office Building will be comprised of four underground parking garage levels with spaces for 201 cars, ten floors above and a mechanical penthouse. The completed building will stand 176'-5" tall and will enclose approximately 307,000 square feet. Rooftop terraces with planters will be an integral part of the construction on the 3rd, 7th and 10th floors.

A secure main lobby on the first floor will efficiently handle high volume traffic to the large assembly areas. Ground level retail will be located on the corner of East Broad Street and 9th Street. The remainder of the floors will be open office spaces with meeting areas that can be flexibly rearranged to meet the needs of the various tenants. Finally, a six story atrium will connect the building along its southern edge to the existing 9th Street Office Building. The 9th Street Office Building is another Virginia government office building, and the atrium is expected to provide seamless passage between the two buildings. See Figure 1 on the next page for a general site plan.

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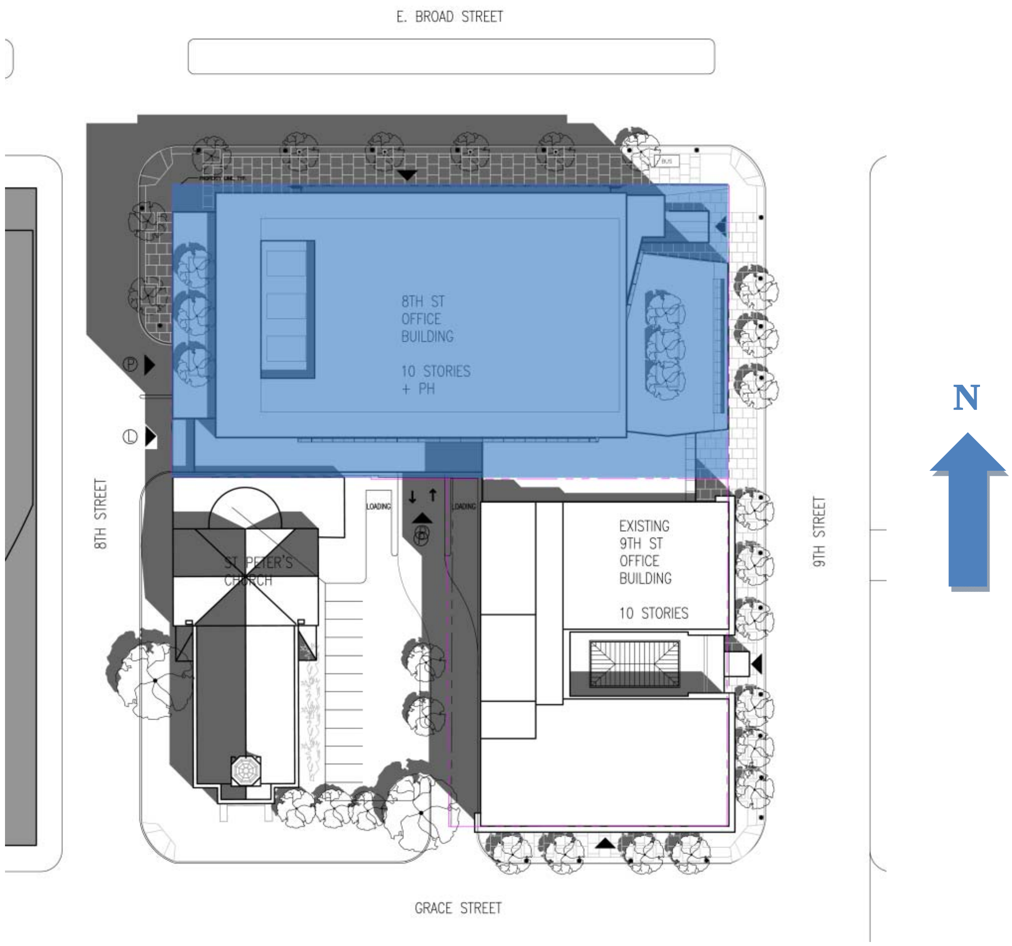

Figure 1 – Site plan

The façade will consist of several different glass curtain walls at varying angles and precast concrete panels. Aluminum will be used to frame individual windows and doorways. Finally, a standing seam stainless steel roof will cantilever dramatically over 30'-0" off of the mechanical penthouse. See Figures 2 and 3 for elevations that display façade materials and the cantilevered roof. Also, select architectural plans are available in Appendix A.

Figure 2 – Broad Street Elevation

Figure 3 – 9th Street Elevation

Existing Structural System

Foundation

The geotechnical engineering study was conducted by Froehling & Robertson, Inc. of Richmond, VA. A total of nine test borings ranging from 50 to 100 feet were performed in September, 2006 and June-July, 2007. Based on the data from the borings and experience with other buildings located in Richmond, it was recommended in the geotechnical report that the 8th Street Office Building be supported on a mat foundation system. The mat foundation is located at elevations of 130'-0" and 140'-0" since the fourth and lowest level of the underground parking garage is only located on the western half of the site. See Figures 4 and 5 for visual representations of the mat foundations locations. Based on the elevations, it was recommended that the 4000 pounds per square inch concrete mat foundation be designed for a maximum allowable bearing pressure of 3,500 pounds per square foot. Ultimately, the mat foundation was designed to be 48" thick reinforced with #10 bars at 12" each way on the top and the bottom throughout the entire foundation.

According to the geotechnical report, the mat foundation system at the proposed elevations will be above the permanent groundwater table. However, the permanent perched water system may cause a substantial flow of water. Therefore, it was recommended that the 12" thick foundation walls be constructed with a minimum of 6" of free-draining granular filter material. Furthermore, the 48" thick mat should be placed on a 12" layer of free-draining aggregate for drainage and to provide uniform bearing pressure.

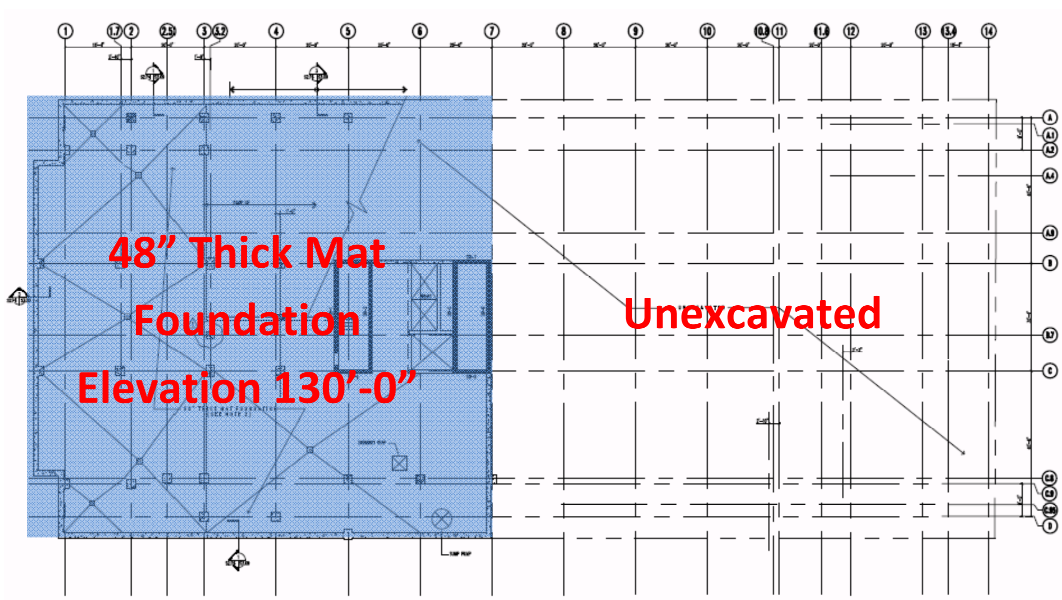

Figure 4 – 4th Level of Parking Garage with General Mat Foundation Location

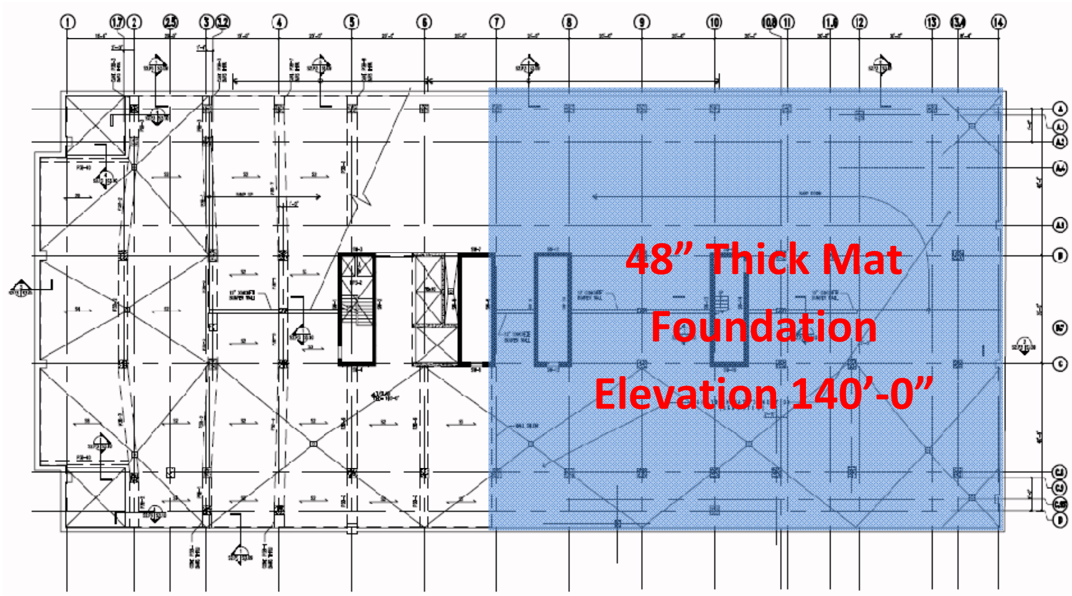

Figure 5 – 3rd Level of Parking Garage with General Mat Foundation Location

Parking Garage

The 8th Street Office Building's underground parking garage is comprised of 3 ½ levels and can accommodate 201 vehicles. The concrete columns are sized to be 30"x30" and tend to be reinforced with 16 #10 bars. Typical bay sizes are either 20'-0" by 40'-6" or 20'-0" by 30'-0". The concrete beams are typically sized to be 30"x30" although there are several exceptions. The longest span of the beams is approximately 40'-6". Primary reinforcement for the beams ranges anywhere from #7 to #11 bars. The one way concrete slabs span in the 20'-0" direction, and the majority of the slabs are 8" thick and reinforced with #5 bars spaced at 12".

Superstructure

The most typical bay sizes for the 8th Street Office Building are either 20'-0" by 40'-6" around the perimeter or 20'0" by 30'-0" through the middle portion of the building. However, there are several variations due to the shape of the building from floor to floor. The composite floor system consists of 3 ¼" of lightweight concrete and 2" deep, 18 gage metal deck for a total depth of 5 ¼". The deck spans W-shape infill beams spaced at 10'-0" on center. The beams tend to be W16x31, W18x35, or W18x40 depending on the length of their span, which most commonly ranges from 30'-0" to 40'-6". Composite action is achieved between the floor system and the beams through ¾" diameter, 4" long headed shear studs. See Figure 6 on the following page for a detail of the floor system. The beams then transfer their loads to W-shape girders whose sizes vary greatly. The girders are connected to W14 columns that range in size from W14x43 to W14x283. The columns are typically spliced every three floors. See Appendix A for select typical floor framing plans.

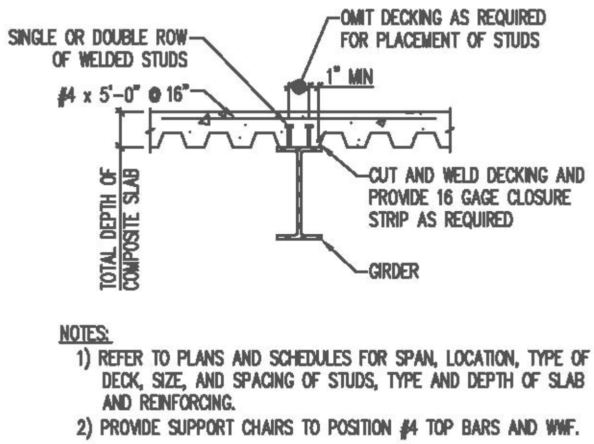

Figure 6 – "Concrete Steel Deck Parallel to Beam" Detail

Lateral System

The primary lateral load resisting system for the 8th Street Office Building consists of reinforced concrete shear walls surrounding four cores within the building. The cores are the locations of the main elevators and stairwells for the building. Therefore, openings are provided in the walls for doorways. There are a total of 16 shear walls. Shear Walls 1 thru 4 extend from the 4th floor foundation of the parking garage below grade to the roof. Shear Walls 5 thru 8 extend from the 4th floor foundation of the parking garage below grade to the penthouse mezzanine. Shear Walls 9 thru 12 extend from the 3rd floor foundation of the parking garage below grade to the penthouse mezzanine. Finally, Shear Walls 13 thru 16 extend from the 3rd floor foundation of the parking garage below grade to the penthouse. See Figure 7 for the exact locations of the shear walls in plan.

The shear walls are 12" thick and reinforced horizontally with #6 bars spaced at 12" on each face and vertically with #8 bars spaced at 12" on each face. The shear walls are a constant 12" thickness throughout without larger boundary elements. There is, however, heavier reinforcement of four #10 bars in each of the shear wall corners.

It is assumed that the floor system of the 8th Street Office Building acts as a rigid diaphragm and transfers the lateral loads due to wind and seismic activity completely to the shear walls in relation to their relative stiffness. The shear walls then carry those loads down to the mat foundation.

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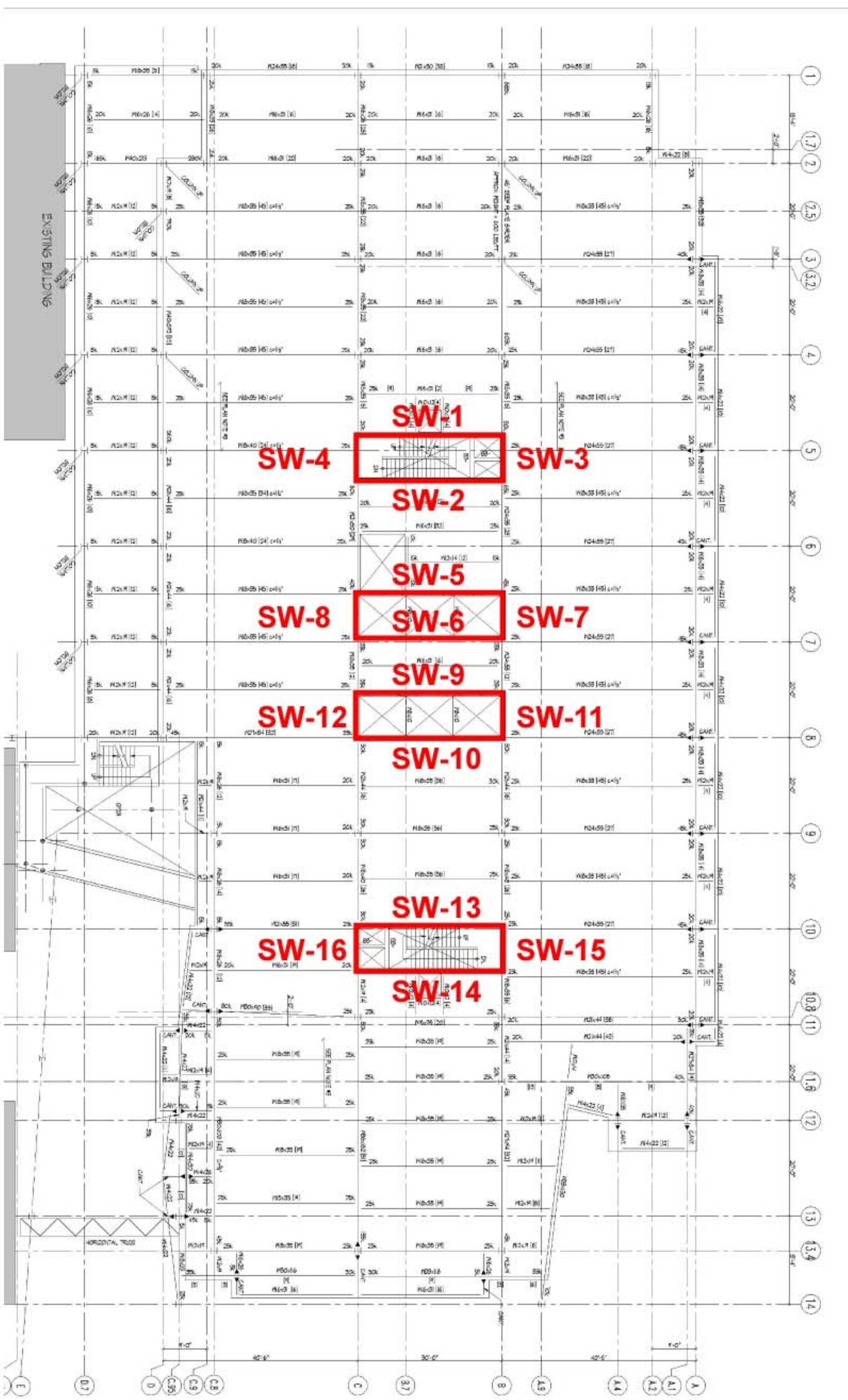

Figure 7 – Locations of Reinforced Concrete Shear Walls

Materials

The following materials were designated by the structural design engineers for the construction of the 8th Street Office Building:

Structural Steel

Rolled Shapes	ASTM A992, Grade 50
Channels, Angles, and Plates	ASTM A36
Pipe	ASTM A53, Grade B, F_y=35 ksi
Round HSS Shapes	AST A500, Grade B, F_y=42 ksi
Tubing (Square and Rectangular HSS)	ASTM A500, Grade B, F_y=46 ksi
Headed Shear Studs $^3/_4$" diameter	ASTM A108
High Strength Bolts $^3/_4$" diameter	ASTM A-325N
Welding Electrodes E70XX	Tensile Strength = 70 ksi

Steel Deck

3 $^1/_4$" Lightweight Concrete over 2" Composite Deck (5 $^1/_4$" total depth)	ASTM A653, 18 Gage
1 $^1/_2$" Roof Deck	ASTM A653, 20 Gage

Concrete

Slabs on Grade (Interior)	f'_c=3000 psi
Slabs on Grade (Exterior)	f'_c=3500 psi
Reinforced Slabs	f'_c=5000 psi
Reinforced Beams	f'_c=5000 psi
Fill on Metal Deck	f'_c=3500 psi
Columns	f'_c=5000/7000 psi
Walls	f'_c=4000 psi
Mat Foundation	f'_c=4000 psi

Reinforcement

Deformed Reinforcing Bars	ASTM A615, Grade 60
Welded Wire Fabric	ASTM A185

Design Codes and Standards

The following code references were used by the structural design engineers:

Model Codes
Virginia Uniform Statewide Building Code 2003
International Building Code 2003

Structural Standards
ASCE 7-02, Minimum Design Loads for Buildings and Other Structures

Design Codes
ACI 318-02, Building Code Requirements for Structural Concrete
AISC Manual of Steel Construction – Allowable Stress Design, 9th Edition
AISC Manual of Steel Construction – Volume II, Connections – ASD, 9th Edition/LRFD, 3rd Edition

Design Loads

Gravity Loads:

The following superimposed dead and live loads were specified by the structural design engineers:

Gravity Loads Specified by the Structural Design Engineers	PSF
Superimposed Dead Loads	
Floor	20
Garage	5
Live Loads	
Framed Floor Areas	70 (50 + 20 partitions)
Lobbies / Stairs / Exits	100
Parking	50
Corridors Above 1st Floor	100
Mechanical Rooms	150
Ordinary Roof	30 (use snow load when greater)

All live loads meet or exceed the minimum requirements set forth by ASCE 7-02.

Table 1 – Existing Design Gravity Loads

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Table 2 shows the snow loads determined by the structural design engineers. Notice that the snow load of 15.4 psf is less than the minimum roof load of 30 psf.

Snow Variables and Loads Specified by the Structural Design Engineers	
P_g	20 psf
C_e	1.0
C_t	1.0
I	1.1
P_f	15.4 psf

Table 2 – Existing Design Snow Variables and Loads

Lateral Loads:

The following wind variables were used by the structural design engineers:

Wind Variables Specified by the Structural Design Engineers	
Basic Wind Speed V (mph)	90
Importance Factor I	1.15
Exposure Category	B
Internal Pressure Coefficient GC_{pi}	± 0.18

Table 3 – Existing Design Wind Variables

The seismic variables that were used by the structural design engineers are not available.

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Proposal

Structural Depth Study:

It was discovered through analyses performed in Technical Report #3 as well as through discussions with the structural design engineers that the existing lateral system of concrete shear walls for the 8th Street Office Building has not been fully optimized. Because the design of the building has been on hold since 2008 at approximately 85-90% completion until funds are allocated for the remainder of the project, it was suggested that an investigation into various steel alternatives for the lateral system could be useful. Therefore, the following steel lateral systems will be the focus of the structural depth study for this thesis:

- Steel Plate Shear Walls
- Braced Frames
- Moment Frames
- Dual System – Braced Frames combined with Moment Frames

Initially, each type of lateral system will be researched generally based on a variety of criteria including, but not limited to, serviceability, constructability, and cost. Then, each system will be given consideration as it specifically relates to the 8th Street Office Building, especially with regard to configurations within the bounds of the architecture. It is expected that a minimum of two of the proposed steel alternative lateral systems will display a sufficient level of feasibility to permit a full design for the 8th Street Office Building. Structural computer modeling will be performed extensively when completing designs, and RAM Structural System is the software program of choice. Ultimately, one of the proposed steel lateral systems will need to be chosen as the optimal steel lateral system for the 8th Street Office Building, so it can be incorporated into the new architecture and roof loads resulting from the two breadth studies to be discussed next. Due to the glass curtain walls that surround the building, serviceability is a large concern that will be given priority in the decision.

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Architectural Breadth Study:

In conjunction with the investigation of alternative steel lateral systems, the architecture of the overall service core of the 8th Street Office Building will be analyzed and redesigned. The goal of the redesign is to minimize the effect of the service core on the useable floor space for the tenants. It is anticipated that restrooms, means of transportation, main lobby aesthetics, and the number of parking spaces may be affected by the redesign of the overall service core. Finally, it will be necessary to maintain all means of egress as required by code. The new architecture will then be used to design the optimal steel lateral system chosen for the 8th Street Office Building.

Sustainable Breadth Study:

The design of the 8th Street Office Building incorporates several sustainable strategies in order to achieve Silver Certification under the U.S. Green Building Council's LEED for New Construction Version 2.2 Rating System. In order to achieve an even more sustainable building, a green roof will be designed for the terraces on the 3rd, 7th, and 10th floors as an alternative to the existing planters. The green roof will be designed with the intention of reducing stormwater runoff and providing a pleasant outdoor atmosphere for the tenants. In addition to the green roof, a rainwater harvest system will be considered for the inaccessible roof areas. The rainwater harvest system will collect greywater that will be used within the building to flush toilets and urinals in the restrooms. Finally, the new loads on the terraces due to the green roof will be used to design the optimal steel lateral system chosen for the 8th Street Office Building.

Structural Redesign of Lateral System

Design Codes and General Criteria

The following code references were used throughout the duration of this structural depth study:

Model Codes
Virginia Uniform Statewide Building Code 2006
International Building Code 2006

Structural Standards
ASCE 7-05, Minimum Design Loads for Buildings and Other Structures

Design Codes
AISC Manual of Steel Construction, 13th Edition

The following general criteria were adopted throughout the duration of this structural depth study:

- All material properties designated by the design structural engineers and previously listed will be used.
- All new columns will be limited to W14 shapes in order to remain consistent with the existing design.
- Total displacement will be limited to 1/400 of the total building height as recommended in Section CC.1.2 of ASCE 7-05.

Design Loads

Gravity Loads:

The following dead, superimposed dead, and live loads were used throughout the duration of this structural depth study:

Gravity Loads Utilized for the Design of the Lateral Systems	PSF
Dead Loads	
2" Composite Metal Deck with 3 $^{1}/_{4}$" Lightweight Concrete Slab	41
Self Weight of Steel Framing	8
Curtain Walls and Precast Concrete Panels	25
Mechanical Rooms (inclusive)	150
Roof/Terrace (inclusive)	100
Atrium (inclusive)	60
Superimposed Dead Loads	
Fireproofing	2
Finishes	10
Partitions	20
Ceiling	5
Mechanical/Electrical/Plumbing	5
Live Loads	
Lobbies & First Floor Corridors	100
Corridors above First Floor	100*
Assembly Areas	100
Offices	50
Ordinary Roof	30*
Roofs used for Roof Gardens or Assembly Purposes	100

*Live load exceeds ASCE 7-05 requirements in order to replicate conditions stipulated by the engineer of record.

Table 4 – Design Gravity Loads for Alternative Lateral Systems

The weight of the floor system was determined from a United Steel Deck Catalog using the properties specified by the structural design engineers for the 8th Street Office Building. Furthermore, the weight of the steel framing was determined by performing spot checks. An initial value of 7 psf was obtained, and it was decided to use 8 psf in order to conservatively account for an increase in steel members from the proposed steel lateral systems. Please refer to Appendix B for more information and calculations. Appendix B also contains snow load calculations that confirm the use of a minimum ordinary roof live load of 30 psf in accordance with the structural design engineers' recommendation. Table 5 on the following page displays the total weight of the 8th Street Office Building as it was summed for each level. These values were later utilized in the calculation of seismic loads for the building.

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Level	Floor-to-Floor Height (ft)	Floor Area (sq ft)	Atrium Area (sq ft)	Terrace/ Roof Area (sq ft)	Floor Loading (psf)	Atrium Loading (psf)	Terrace/Roof Loading (psf)	Weight (kips)
Roof	-	0	0	22904	0	0	100	2290
PH Mezz.	16.58	5715	0	0	150	0	0	857
PH	13.42	11664	0	6212	150	0	100	2371
10	14.08	22883	0	1781	116	0	100	2833
9	13.50	24649	0	0	116	0	0	2859
8	13.50	24635	0	0	116	0	0	2858
7	13.50	24615	0	6886	116	0	100	3544
6	14.25	28517	3233	0	116	60	0	3502
5	14.25	28724	2968	0	116	60	0	3510
4	14.25	28534	3159	0	116	60	0	3499
3	14.25	28697	2968	2469	116	60	100	3754
2	18.83	29130	4296	0	116	60	0	3637
							Total W =	35514

Table 5 – Building Weight by Floor

Lateral Loads:

The following hand calculated lateral loads were used throughout the duration of this structural depth study. The main intent was to create a basis for comparison with the computer generated loads from RAM Structural System.

Wind loads for the 8th Street Office Building were determined using Method 2, also known as the Analytical Procedure, in ASCE 7-05 Section 6.5. Because the building has a significant setback that occurs at the 7th floor, two analyses were conducted. The first analysis utilized the first floor dimensions, and the second analysis utilized average dimensions from the 7th through the 10th floors. The wind variables common to both of the analyses conducted can be found below in Table 6. These variables confirm the wind variables specified by the structural design engineers. The controlling pressure was then selected for each floor in order to calculate the forces. Generally, the second analysis produced the controlling pressures, although the results were not significantly different. Please see Figures 8 and 9 on the following page for wind pressure diagrams. Detailed calculations for each of the analyses can be found in Appendix B.

Wind Variables		ASCE 7-05 Reference
V	90	(Fig. 6-1)
K_d	0.85	(Table 6-4)
I	1.15	(Table 6-1)
Exposure Category	B	
K_{zt}	1	(Sec. 6.5.7.1)
Enclosure Classification	Enclosed	(Sec. 6.2)
GC_{pi}	± 0.18	(Fig. 6-5)

Table 6 – Wind Variables

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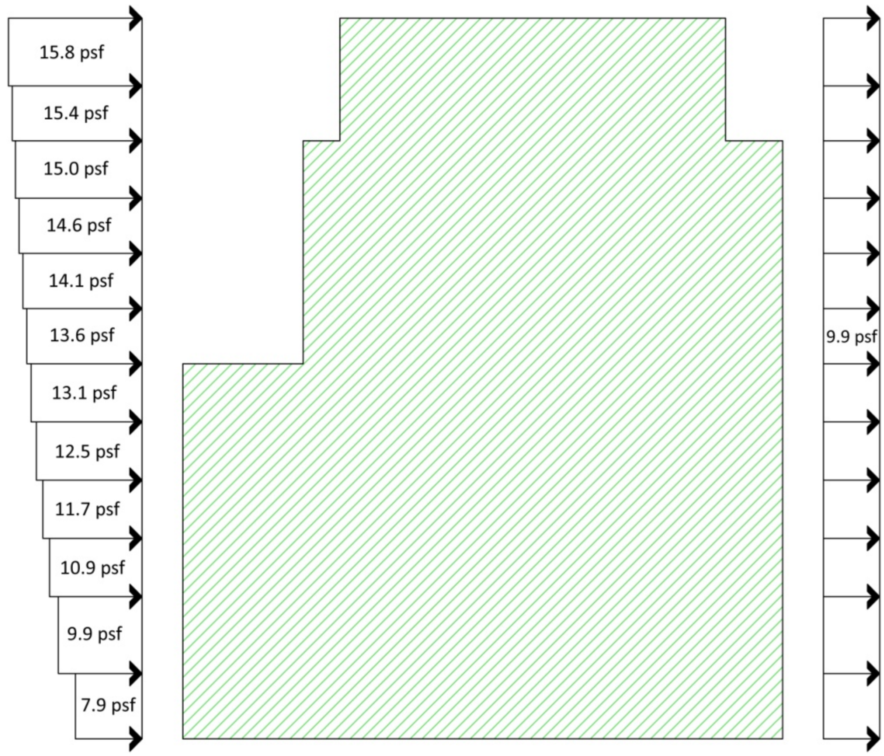

Figure 8 – North-South Wind Pressure Diagram

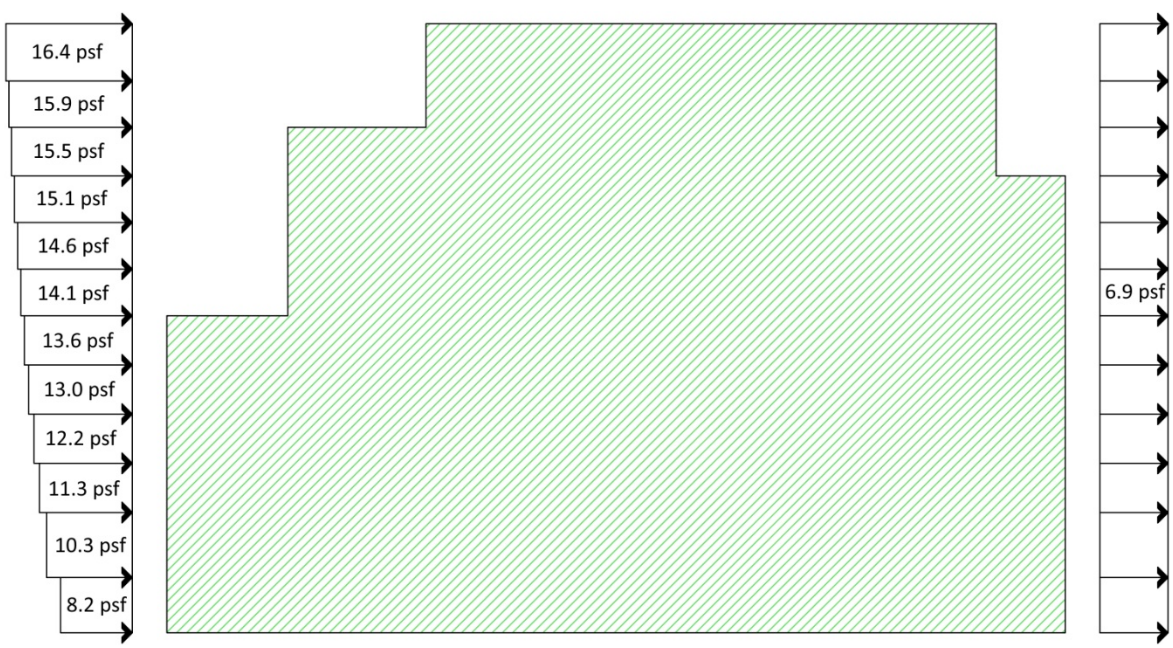

Figure 9 – East-West Wind Pressure Diagram

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Wind pressures were converted to concentrated loads by utilizing the tributary area of the building's façade at each level. See Figures 10 and 11 for the distribution of the wind loads and the base shear in each direction. Finally, it should be noted that the 9th Street Office Building and St. Peter's Church abut the 8th Street Office Building and block the wind on the lower levels. However, wind was still examined in these areas in the event that the adjacent buildings no longer exist.

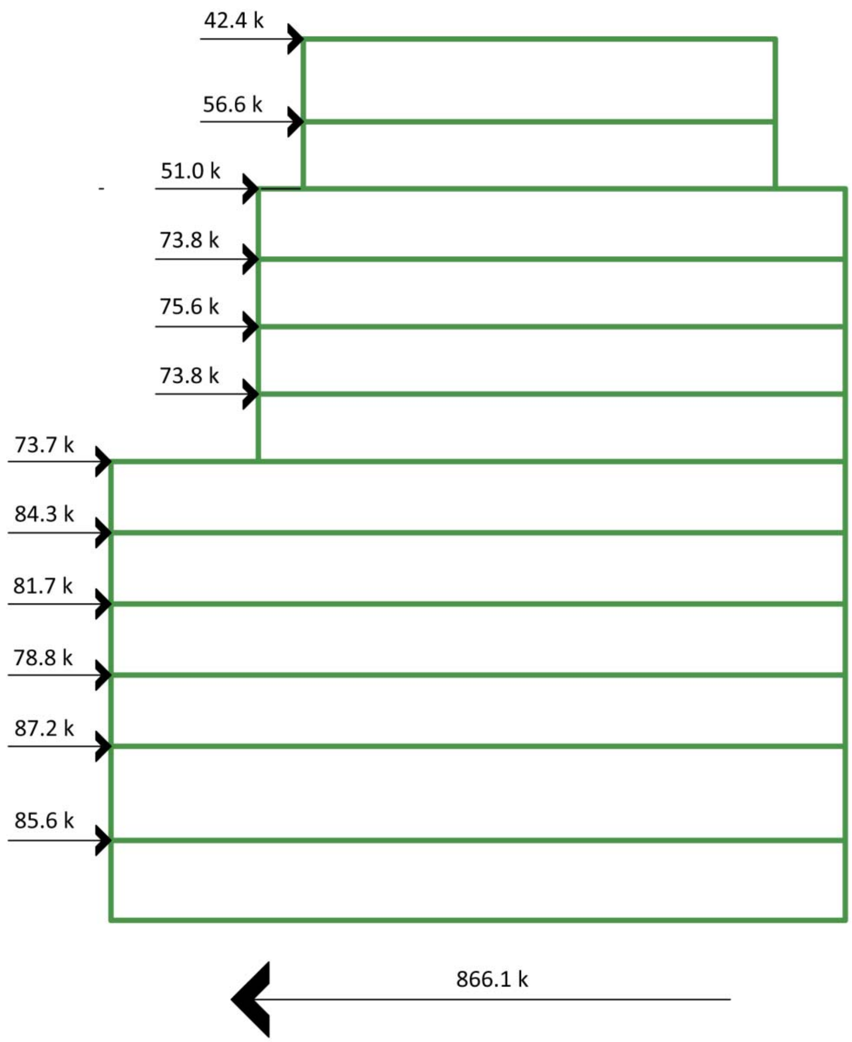

Figure 10 – North-South Service Wind Loads

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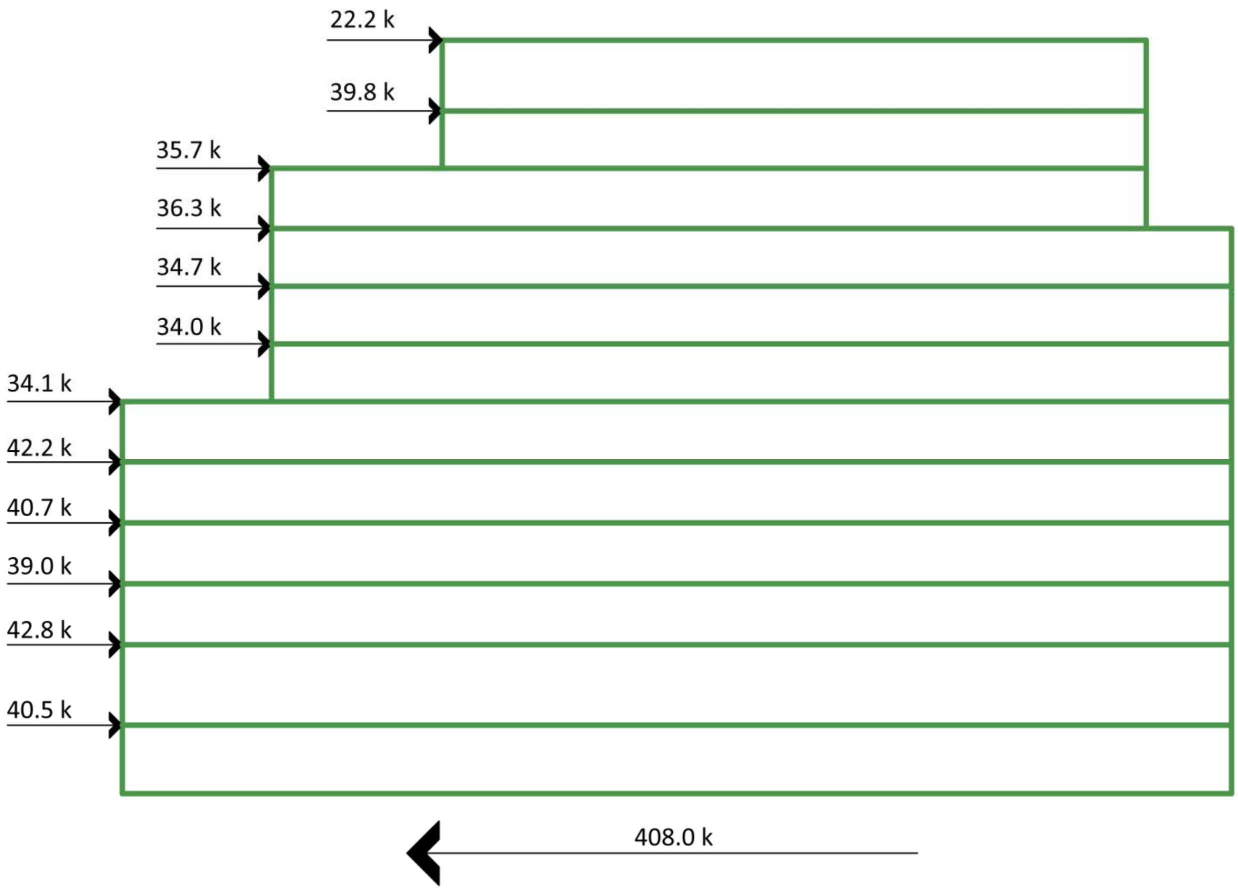

Figure 11 – East-West Service Wind Loads

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Seismic loads for the 8th Street Office Building were determined using Chapters 11 and 12 of ASCE 7-05. It was determined that the Equivalent Lateral Force Procedure could be used in the calculation of seismic forces. In order to accommodate the various alternative steel lateral systems, two analyses were conducted. The results from the first analysis were used for the braced frame system and dual system, while the results from the second analysis were used for the moment frame system. The analyses include dead loads from floor slabs, steel framing, glass curtain walls and superimposed dead loads. An additional allowance was also provided for the penthouse mechanical areas and the roof terraces due to built-up concrete and planters respectively. A summary of the seismic variables common to both of the analyses can be found below in Table 7. Differences in variables obtained from Tables 12.8-2 in ASCE 7-05 resulted in much lower seismic forces for the moment frame system. Please see Figures 12 and 13 for seismic loading diagrams. Detailed calculations related to both seismic analyses are available in Appendix B.

Seismic Variables		ASCE 7-05 Reference
S_s	0.23	(Fig. 22-1)
S_1	0.06	(Fig. 22-2)
Site Classification	C	(Table 20.3-1)
F_a	1.2	(Table 11.4-1)
F_v	1.7	(Table 11.4-2)
S_{MS}	0.276	(Eq. 11.4-1)
S_{M1}	0.102	(Eq. 11.4-2)
S_{DS}	0.184	(Eq. 11.4-3)
S_{D1}	0.068	(Eq. 11.4-4)
Occupancy Category	III	(Table 1-1)
I	1.25	(Table 11.5-1)
Seismic Design Category	B	(Tables 11.6-1 & 11.6-2)
Equivalent Lateral Force Procedure permitted by (Table 12.6-1)		
T_L	8	(Fig. 22-15)

Table 7 – Seismic Variables

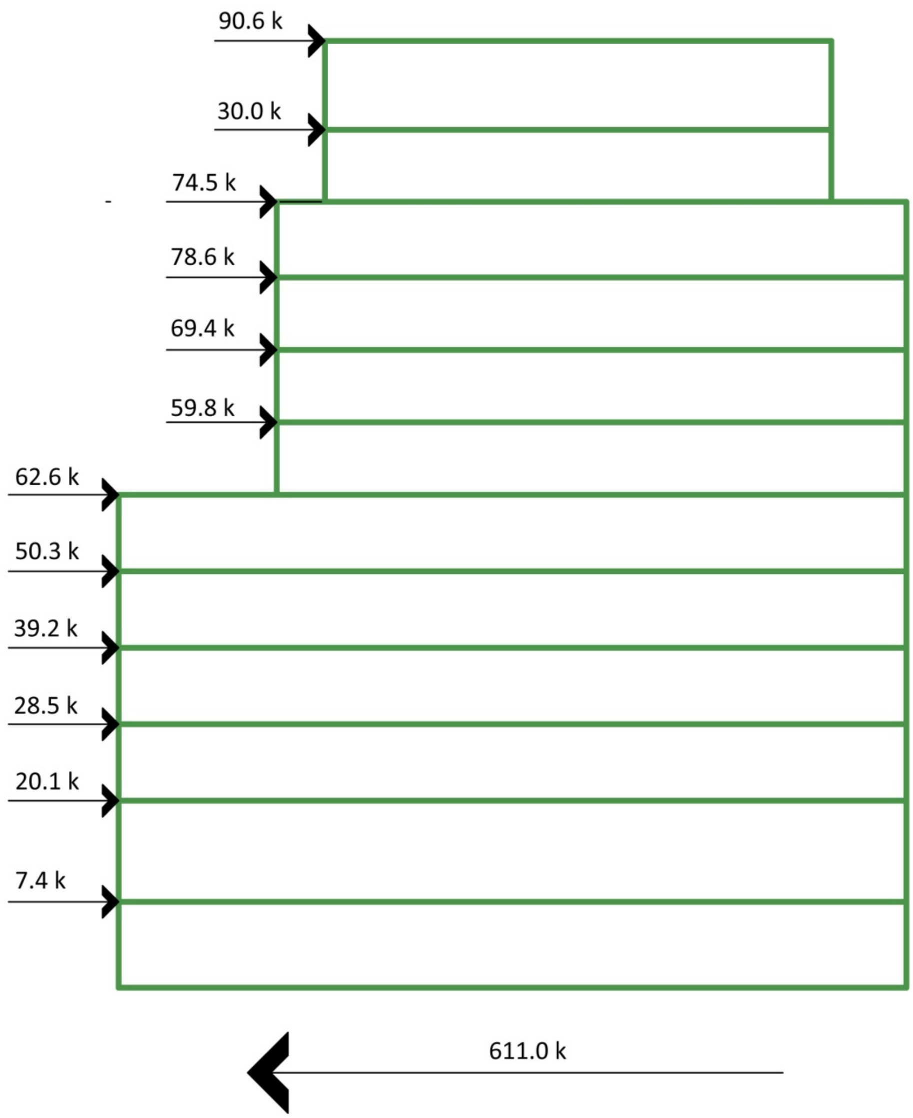

Figure 12 – Braced Frames/Dual System Ultimate Seismic Loads

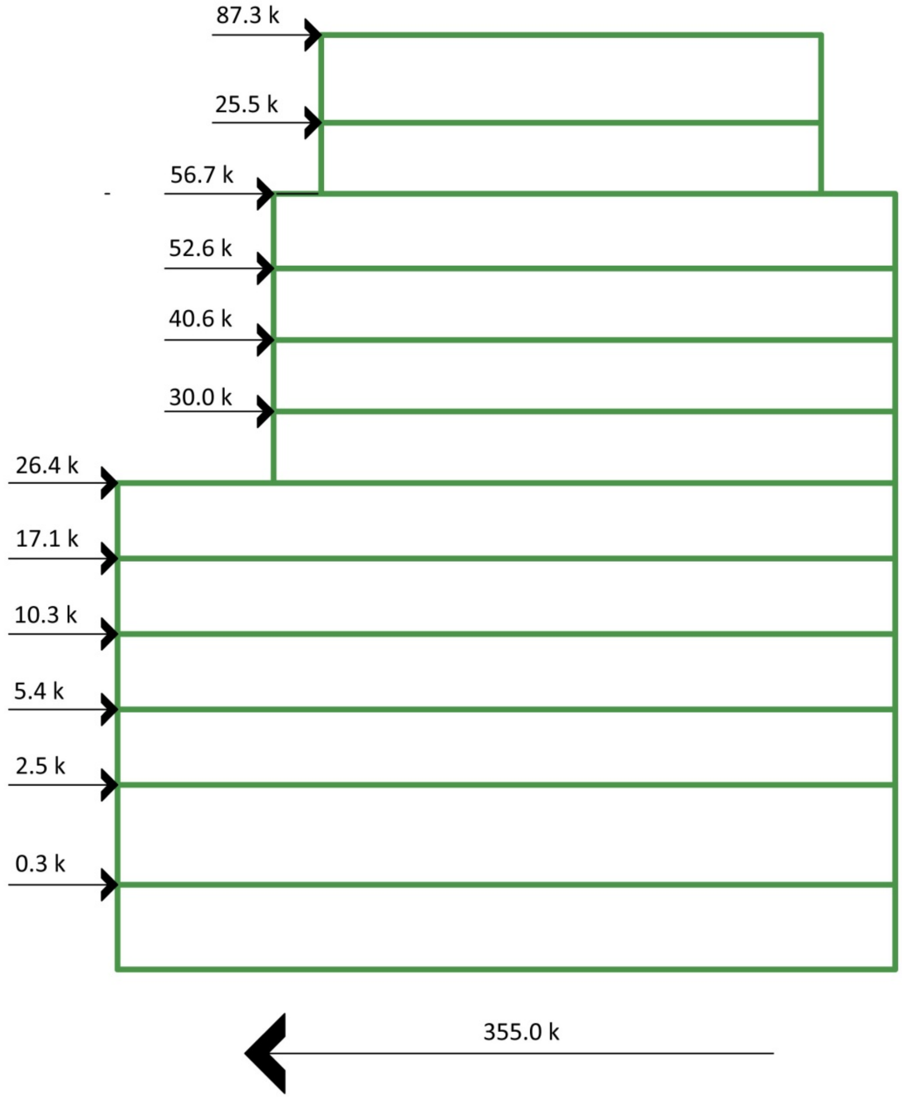

Figure 13 – Moment Frames Ultimate Seismic Loads

Steel Plate Shear Walls

The first steel lateral system that was investigated consists of steel plate shear walls. The steel plate shear wall (SPW) system was chosen as a possible alternative lateral system because it has been successfully used in a significant number of buildings for more than three decades. However, it has only been four years since the American Institute of Steel Construction published *Design Guide 20, Steel Plate Shear Walls*, which develops a complete design methodology for both low-seismic and high-seismic applications. Therefore, the SPW system is considered relatively new and interesting among practicing engineers today.

Several different types of steel plate shear walls are recognized globally in countries such as Canada, Mexico, and Japan. Stiffened steel plate shear walls are primarily found in Japan to resist seismic forces. See Figure 14 for an example of a building in Japan that utilizes a stiffened SPW system. Additionally, Figure 15 features steel plate shear walls under construction in Canada. Stiffening is typically achieved by welding plates to one or both sides of the steel web plate, which increases the shear-buckling strength. Steel web plates can also be stiffened by adding concrete to one or both sides, which results in a composite SPW system. However, both of these types of stiffened SPW systems are significantly less economical than walls that utilize an unstiffened, slender web plate. Furthermore, the thickness of the wall must be increased with the stiffened systems. Therefore, SPW systems with unstiffened, slender web plates are the most popular in the United States and are the focus of the remainder of the discussion on steel plate shear walls.

Figure 14 – Nippon Steel Building in Tokyo, Japan (courtesy of AISC)

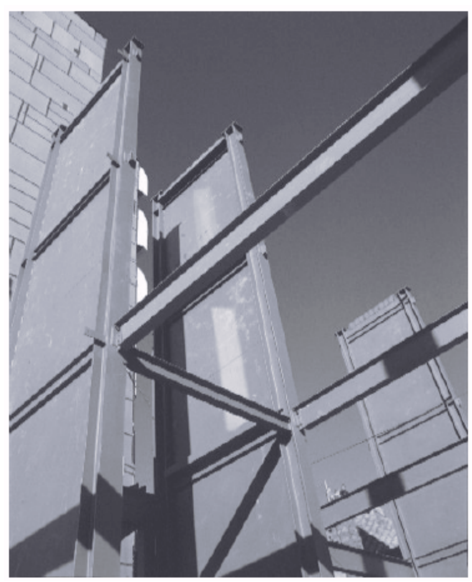

Figure 15 – Canam Manac Group headquarters expansion in St. George, Quebec, Canada (courtesy of Richard Vincent, Canam Manac Group and AISC)

Steel plate shear walls are designed to resist lateral loads through diagonal tension in the web plate and overturning forces in the columns, also known as vertical boundary elements (VBE). Likewise, beams are referred to as horizontal boundary elements (HBE). In particular, unstiffened, slender web plates can resist rather large tension forces but very little compression, which is analogous to tension-only bracing. However, the frame members around the web plate in a steel plate shear wall are designed differently from the frame members around tension-only bracing. The tension in the web plate is present along the entire length of the boundary elements, as opposed to simply at the intersection of beams and columns. Therefore, the boundary elements can be subjected to large inward forces, which must be resisted through flexure. The flexural forces can be especially large for tall stories and may govern the design of the VBE. See Figure 16 below for an illustration of the flexural deformation of boundary elements due to these inward forces.

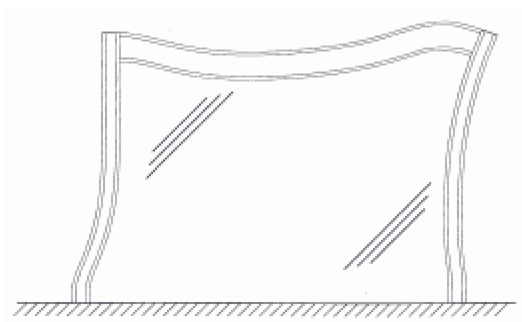

Figure 16 – Inward Flexure of Boundary Elements
(courtesy of AISC)

In the United States, unstiffened steel plate shear walls are typically designed as Special Plate Shear Walls (SPSW), and high-seismic design provisions for this system are found in AISC 341-05. However, no low-seismic design provisions exist for the system when the redundancy factor R is less than or equal to 3. Therefore, ASCE 7-05 and AISC 360-05 are assumed to be the governing coded for low-seismic design, although certain equations from AISC 341-05 are typically used as well.

There are several advantages to using a SPW system in building construction compared to concrete shear walls. Wall thicknesses can be reduced, which results in more useable plan area. Overall building mass can also be reduced, which is beneficial with regard to the design of the foundation. Steel plate shear walls can also be erected much quicker than concrete shear walls. These advantages of an SPW system can be considered comparable to the advantages of braced frames.

Unfortunately, the open plan architecture of the 8th Street Office Building limits the locations of any steel plate shear walls to the four transportation cores where the existing concrete shear walls are located. As a result of this required configuration, there are several disadvantages to using a SPW system in the 8th Street Office Building. For taller buildings, drift control may be very difficult to achieve, with respect to either the allowable seismic drift at the center of mass or the recommended limit of

H/400. Therefore, longer bays may be required in order to attain acceptable drifts, but many of the shear walls in the 8th Street Office Building are limited to 10 feet in width. Furthermore, less torsional resistance can be achieved by locating the steel plate shear walls at the building core rather than around the perimeter. Therefore, it is usually recommended that a perimeter moment frame be used in conjunction with the SPW system. However, the irregular floor plans prohibit perimeter moment frames. It should be noted that these disadvantages may also be considered disadvantages of braced frames.

However, there is one disadvantage in particular of the SPW system that causes braced frames to be a better lateral system for the 8th Street Office Building. Each opening in a steel plate shear wall must be framed by local vertical and horizontal boundary elements that extend the full story height and full bay width respectively. Please see Figure 17 below. By locating the shear walls around the transportation cores, a significant number of openings are required for stairwell and elevator doors. Steel tonnage and thus cost will be increased if every opening is to be framed by local boundary elements. The only other option is to eliminate between five and seven of the sixteen total shear walls in order to avoid most of the openings. If braced frames are used around the transportation cores, only four of the sixteen must be eliminated in order to accommodate the openings. In addition, a relative symmetry can be maintained when braced frames are used. Therefore, it was decided not to fully design steel plate shear walls as an alternative lateral system for the 8th Street Office Building in favor of braced frames instead.

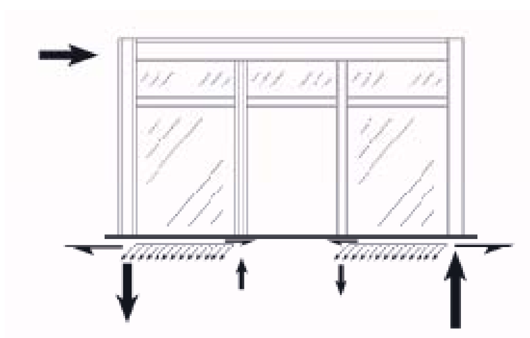

Figure 17 – Steel Plate Shear Wall with an Opening Framed by Local Boundary Elements
(courtesy of AISC)

Braced Frames

A braced frame system was the first steel lateral system to be fully designed for the 8th Street Office Building as indicated in the previous section. The following advantages played a significant role in the selection of braced frames:

- Lightweight
- Simple connections
- Quick construction time
- Economical
- Configuration (specific to the 8th Street Office Building)
  - More suited to handle openings than steel plate shear walls
  - More capable of maintaining relative symmetry than steel plate shear walls

The design of the braced frames involved a five step process that utilized structural computer modeling:

1. Determine locations of the braced frames and finalize ideal bracing configurations.
2. Initially, size the columns and beams as gravity members in RAM Structural System. Then, modestly increase the gravity sizes for the first iteration as lateral members. Choose HSS members for the braces.
3. Analyze the lateral members in RAM Structural System by manually inputting the wind and seismic loads that were determined using the Analytical Procedure and Equivalent Lateral Force Procedure from ASCE 7-05 respectively. Resize members for strength when necessary.
4. Analyze the lateral members using wind and seismic loads calculated within RAM Structural System. Compare these loads to the previously calculated loads. Upon determination of accuracy of loads, tweak member sizes for strength when necessary.
5. Check that drift limitations are met. Utilize the Drift Control Module within RAM Structural System to determine the contributions of individual lateral members to drift resistance and resize members when necessary.

As previously discussed, the open and irregularly shaped floor plans of the 8th Street Office Building limited the location of the lateral system to the four transportation cores. In Step 1 of the design process, it was determined that four of the potential sixteen braced frames had to be completely eliminated due to openings. Please see Figure 18 on the following page for the locations of the twelve remaining braced frames. The locations that could not accommodate braced frames due to openings are indicated as new gravity framing. It was also determined that one-story X-bracing would be used for the approximately 10’ wide frames in the East-West direction, and two-story X-bracing would be used for the 30’ wide frames in the North-South direction, with the exception of chevron bracing for Braced Frame 10 due to openings.

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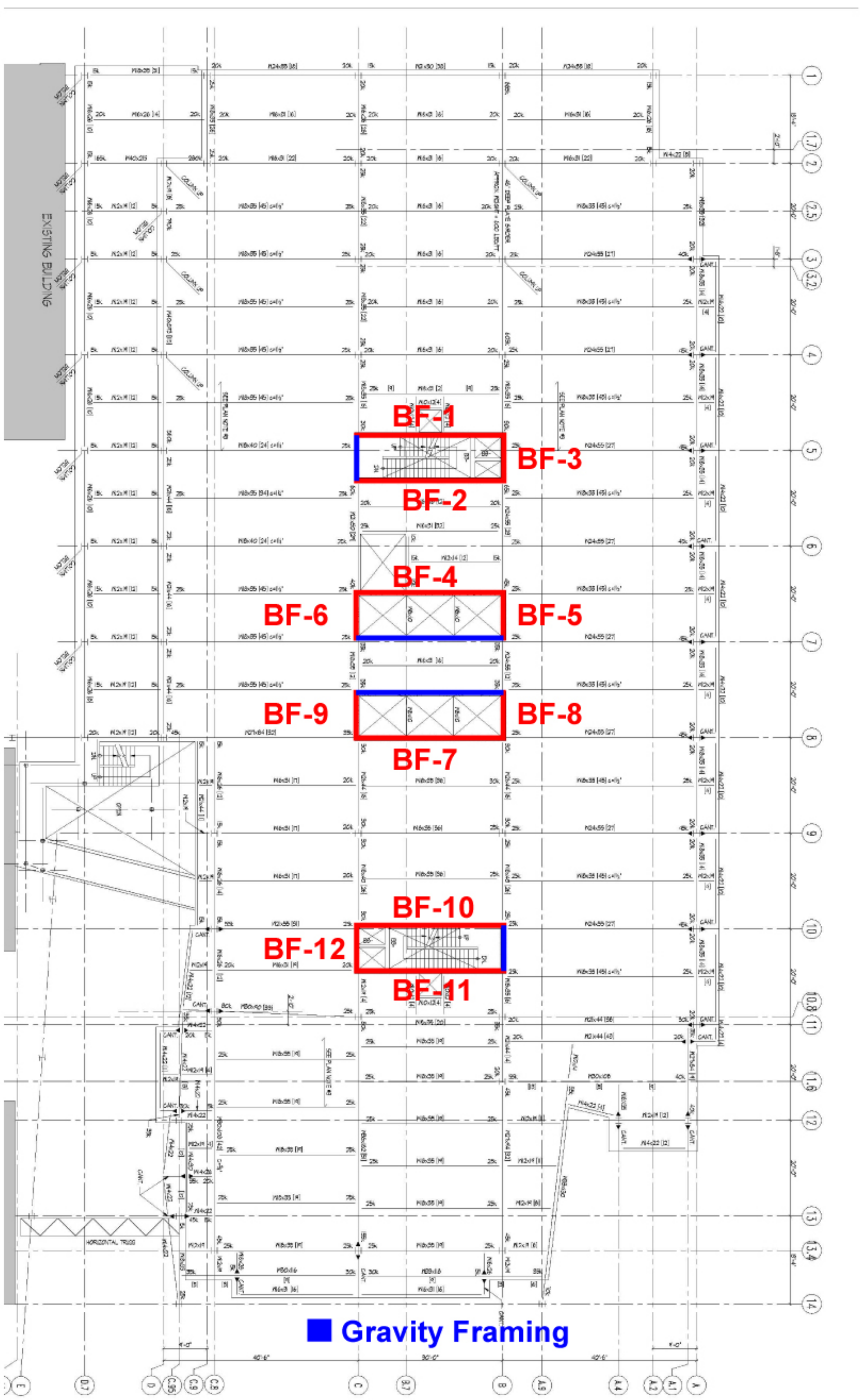

Figure 18 – Locations of Braced Frames

In order to help resist torsion at the roof level, several of the braced frames were extended to a higher elevation than their concrete shear wall counterparts. Specifically, Braced Frames 5, 6, 7, and 9 were extended one story from the penthouse mezzanine level to the roof level.

The 8th Street Office Building was modeled in RAM Structural System in order verify the lateral forces that were previously calculated. In addition, the RAM Frame module made it possible to perform multiple iterations of lateral member analysis, which would have been time prohibitive to perform by hand. The RAM Frame module was also vital to the obtainment of accurate drift values, which governed the final design. The following assumptions were made during the modeling process:

- All floors were modeled as rigid diaphragms. Therefore, inherent torsion, per Section 12.8.4.1 of ASCE 7-05, due to the eccentricity between the locations of the center of mass and center of rigidity for each floor was considered.
- Columns were spliced every three floors.
- All columns were assumed to be pinned at the base.
- Columns in the braced frames were assumed to be fixed (except at the base), while the beams and braces were considered to be pinned.
- Accidental torsion of seismic loads, per Section 12.8.4.2 of ASCE 7-05, was considered by applying a 5% eccentricity. (RAM Structural System calculates the period at the center of mass, and the seismic story forces that are derived from said period are then applied at the specified eccentricity.) It was not necessary to consider amplification of the accidental torsional moment, per section 12.8.4.3 of ASCE 7-05, since the structure is assigned to Seismic Design Category B.
- 131 load combinations were generated within the program. An additional 48 load combinations were derived from the generic combination D + 0.5L + 0.7W, which is recommended in Section CC.1.2 of ASCE 7-05 for evaluating the recommended drift limit of H/400, and manually input.
- P-Delta effects were taken into account within the model.

Please see Figure 19 on the following page for a three-dimensional representation of the gravity members and braced frames that were modeled in RAM Structural System for the purposes of designing the braced frames and assessing the serviceability concern of drift.

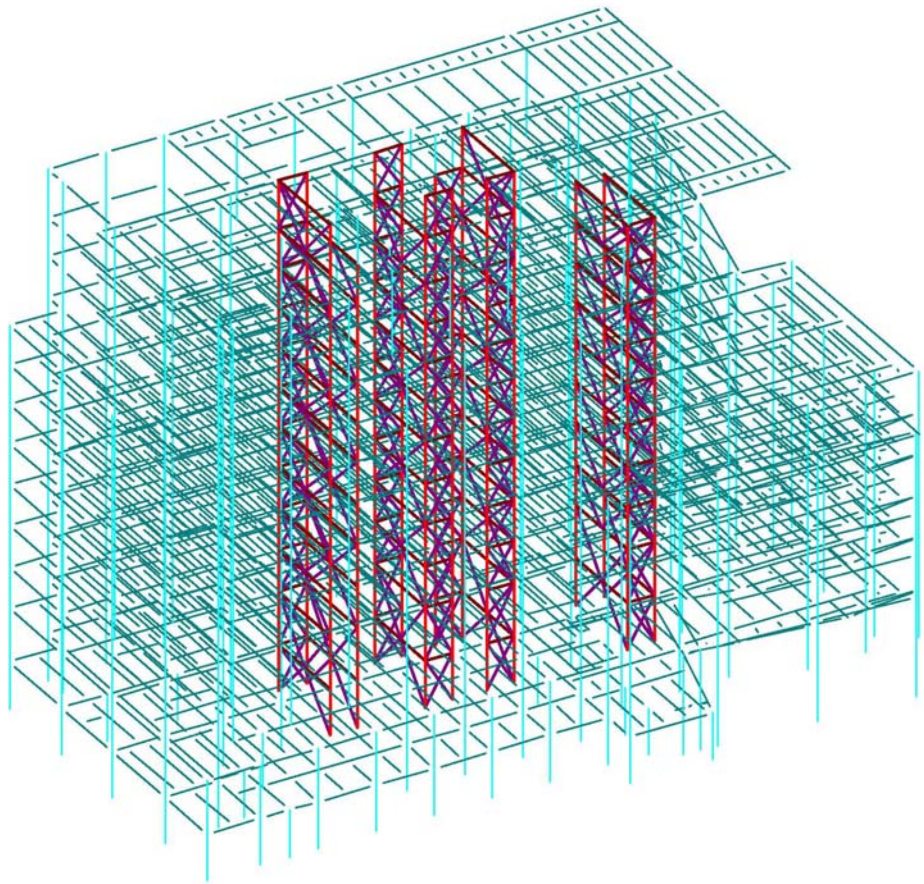

Figure 19 – RAM Model of Steel Gravity Framing and Braced Frames

In order to verify the adequacy of the lateral loads used in the design of the braced frames, the hand calculated wind and seismic loads were compared to the RAM generated loads through the base shears and periods. See Table 8 below for the percent difference between the hand calculated values and the RAM generated values. All percent differences were less than 10%, which was deemed an acceptable accuracy to proceed with the design of the braced frames. It was ultimately decided to finalize the braced frame designs with the RAM calculated loads due to the geometry simplifications that were made during the hand calculations.

Load Check	Hand Calculated	RAM calculated	Percent Difference
Wind East-West Base Shear (k)	487.2	529.0	8.27%
Wind North-South Base Shear (k)	866.1	786.3	9.67%
Seismic Period T	1.645	1.529	7.31%

Table 8 – Comparison of Hand Calculated versus RAM Structural System Calculated Lateral Loads

When designing the frame members, the interaction value of each member was based on the controlling interaction equation from all of the load combinations. The target interaction value for the columns was approximately 0.85 and was calculated from equations H1-1a and H1-1b in the AISC 360-05 Specification. Upon completing the design of the braced frames for strength, typical member sizes ranged from W14x283 to W14x48 for the columns, W12x14 in the 10' direction to W21x44 in the 30' directions for the beams, and HSS10x10x$^5/_8$ to HSS6x6x$^5/_{16}$ for the braces.

Next, seismic story drift was evaluated according to Section 12.8.6 of ASCE 7-05 and compared to the story drift limit, per Section 12.12.1 of ASCE 7-05.

$$\text{(Equation 12.8-15)} \quad \delta_x = \frac{C_d \delta_{xe}}{I} \leq 0.015 h_{sx} \quad \text{(Table 12.12-1)}$$

$$\frac{3\delta_{xe}}{1.25} \leq 0.015 h_{sx}$$

$$drift\ ratio = \frac{\delta_{xe}}{h_{sx}} \leq 0.00625$$

It was discovered that the design of the braced frames based solely on strength exceeded the allowable drift ratio by almost 50%. In order to rectify the problem, the Drift Control module in RAM Structural System was utilized. A virtual load case consisting of 100 kips applied at the roof was created for both the East-West and North-South directions. Then, each virtual load case was paired with the appropriate seismic load case in order to determine which lateral members provided the most resistance to drift. The lateral members were viewed as a function of total displacement divided by volume, so it could be determined where it would be most beneficial to increase a member in size. The appropriate members were increased in size until seismic drift limitations were met. The final seismic story drifts and corresponding drift ratios are tabulated in Table 9 below.

Level	Story Height Below h_{sx} (ft)	Center of Mass Coordinates		Story Drift (in)		Drift Ratio		≤ 0.00625
		X (ft)	Y (ft)	X (E-W)	Y (N-S)	X (E-W)	Y (N-S)	
Roof	16.58	124.36	85.16	1.21	0.20	0.0061	0.0010	Yes
PH Mezzanine	13.42	117.16	84.87	0.97	0.11	0.0060	0.0007	Yes
Penthouse	14.08	117.33	88.48	0.96	0.14	0.0057	0.0008	Yes
10	13.50	90.78	87.51	0.87	0.18	0.0054	0.0011	Yes
9	13.50	98.62	87.67	0.83	0.16	0.0051	0.0010	Yes
8	13.50	98.52	87.65	0.75	0.15	0.0046	0.0009	Yes
7	14.25	141.71	74.20	0.70	0.10	0.0041	0.0006	Yes
6	14.25	122.75	83.59	0.62	0.10	0.0036	0.0006	Yes
5	14.25	122.35	83.76	0.51	0.10	0.0030	0.0006	Yes
4	14.25	122.33	83.76	0.39	0.09	0.0023	0.0005	Yes
3	18.83	116.69	76.05	0.34	0.14	0.0015	0.0006	Yes
2	16.00	119.43	71.43	0.00	0.00	0.0000	0.0000	Yes
Total	**176.41**			**8.15**	**1.46**			

Table 9 – Braced Frames Seismic Drift

Finally, the recommended total displacement limitation of 1/400 of the building height found in Section CC.1.2 of ASCE 7-05 was evaluated. The load combination of D + 0.5L + 0.7W was used since it is considered to be conservative. Displacements were checked at the center of mass of the roof level and at eight points around the perimeter of the roof. These values were all found to be less than the limit of 5.29" and can be found in Table 10.

Control Point	Coordinates X (ft)	Coordinates Y (ft)	Maximum Displacement (in)*	≤ 5.29"
Center of Mass	124.36	85.16	4.70	Yes
Southwest Corner	18.33	29.42	4.60	Yes
West Midpoint	18.33	55.50	4.44	Yes
Northwest Corner	18.33	140.42	5.29	Yes
North Midpoint	95.94	140.42	5.29	Yes
Northeast Corner	210.21	140.42	5.29	Yes
East Midpoint	210.21	55.50	4.44	Yes
Southeast Corner	210.21	29.42	4.60	Yes
South Midpoint	95.94	29.42	5.29	Yes

*All maximum displacements occurred in the East-West direction.

Table 10 – Braced Frames H/400 Displacement

The final designs of the braced frames, after taking into account drift, can be found in Appendix C. Final member sizes ranged from W14x455 to W14x53 for the columns, W12x14 in the 10' direction to W21x44 in the 30' directions for the beams, and HSS10x10x$^{5}/_{8}$ to HSS6x6x$^{5}/_{16}$ for the braces.

Moment Frames

The next alternative lateral system that was considered consists of sixteen ordinary steel moment frames in the same locations as the existing concrete shear walls. The primary reason for investigating a moment frame system is its compatibility with open floor plans since there are no obstructions such as braces or walls associated with moment frames. Therefore, the openings around the four transportation cores did not cause any problems as they did in the steel plate shear wall system and braced frame system. Unfortunately, moment frames are much less economical than steel plate shear walls and braced frames for the following reasons:

- Complex connections that often involve more field-welding
- Larger column and beam sizes (higher steel tonnage) in order to achieve the required stiffness for serviceability concerns

A preliminary design was modeled in RAM Structural System according to the same assumptions listed earlier for the braced frames, with the exception of end conditions. All columns and beams in the moment frames were assumed to be fixed (except for the columns at the base). Furthermore, different seismic loads were manually input to reflect adjustments in variables due to the change in lateral system.

However, it was quickly discovered that an ordinary steel moment frame system is less than ideal for the 8th Street Office Building. In fact, the chosen configuration was unable to meet the slenderness limit of KL/r = 200 primarily due to large story heights, such as more than 18' for the 3rd story. While the limit of 200 is no longer mandatory, it is still recommended in Section E2 of the AISC 360-05 Specification for the following reasons:

- The flexural buckling stress for a very slender column is extremely low, resulting in an inefficient column.
- Extra care must be taken with very slender columns to avoid damage during fabrication, transportation, and erection.

A new configuration was briefly considered that involved linking the 10' wide moment frames in the East-West direction. However, it was still not possible to place moment frames around the perimeter of the building due to irregularities. In addition, increasing the number of moment frames would only serve to create an even more cost prohibitive lateral system, which would outweigh the advantage of being able to avoid door openings. Therefore, it was ultimately decided not to finalize a moment frame system for the 8th Street Office Building.

Dual System

The final steel lateral system that was investigated combines braced frames and moment frames into a dual system. The desire was to maintain the advantages of a braced frame system that were previously discussed, while addressing the serviceability issue of drift in more detail. The locations of the braced frames were kept the same as before, and two moment frames were added linking Braced Frames 5 and 8 and Braced Frames 6 and 9 in the East-West direction. See Figure 21 on the following page for the locations of the braced frames and moment frames in the proposed dual lateral system. The reasons for deciding to locate the moment frames as indicated are the following:

- The drift analysis from the lateral system consisting of only braced frames indicated that drift was more of a concern in the East-West direction, presumably due to the small 10' widths of the braced frames in that direction.
- Symmetry was maintained as much as possible.
- It was possible to extend the moment frames to the roof level.

Two options were developed for designing the dual system in RAM Structural System after finalizing the locations of the frames. The same assumptions were made during the modeling process as before, with the end conditions depending on the frame type. The RAM generated wind and seismic loads were also used again since accuracy was already been confirmed. See Figure 20 below for a three-dimensional representation of the gravity members, braced frames, and moment frames that were modeled in RAM Structural System for the purposes of designing the dual lateral system.

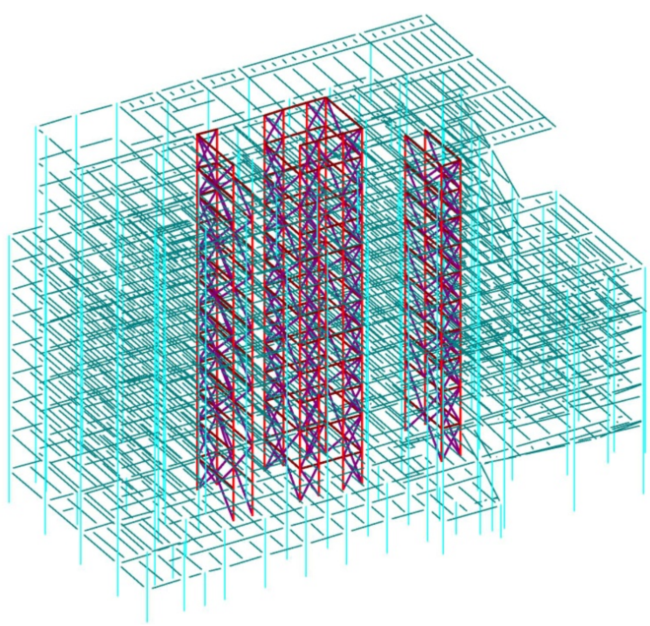

Figure 20 - RAM Model of Steel Gravity Framing, Braced Frames, and Moment Frames

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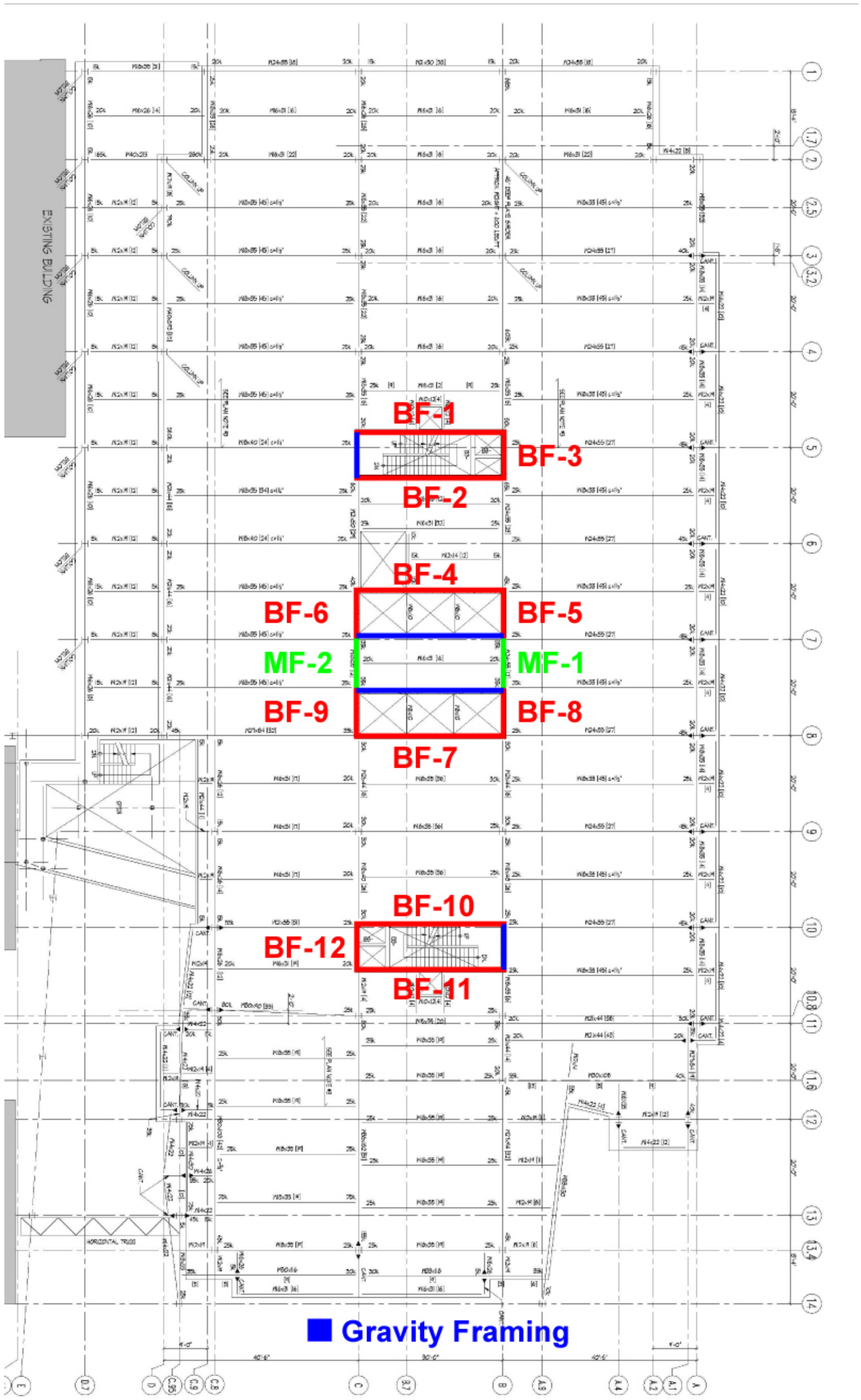

Figure 21 – Locations of Braced Frames and Moment Frames for the Dual Lateral System

Option 1:

1. Keep the sizes of the braced frames that were previously designed for strength, without regard for drift limitations.
2. Analyze the lateral members using RAM Structural System, using the gravity sizes for the moment frame beams for the first iteration. Increase any sizes as necessary.
3. Evaluate drift limitations. Use the Drift Control module to assist in resizing of members if story drift is not acceptable.

Again, the target interaction value for each member was approximately 0.85. Table 11 below shows that the drift ratio at the center of mass of each level due to seismic loading was less than the allowable drift ratio by code. Furthermore, Table 12 shows that the maximum total displacement from the control points checked at the roof level decreased to 3.5 inches, which is well below the recommended limit 5.29 inches. Therefore, the design of the dual system was governed by strength rather than drift, and the Drift Control module did not need to be utilized.

Level	Story Height Below h_{sx} (ft)	Center of Mass Coordinates		Story Drift (in)		Drift Ratio		≤ 0.00625
		X (ft)	Y (ft)	X (E-W)	Y (N-S)	X (E-W)	Y (N-S)	
Roof	16.58	124.35	85.16	0.76	0.22	0.0038	0.0011	Yes
PH Mezzanine	13.42	117.16	84.92	0.61	0.13	0.0038	0.0008	Yes
Penthouse	14.08	117.33	88.48	0.64	0.15	0.0038	0.0009	Yes
10	13.50	90.74	87.52	0.62	0.19	0.0038	0.0012	Yes
9	13.50	98.50	87.69	0.62	0.18	0.0038	0.0011	Yes
8	13.50	98.41	87.67	0.57	0.16	0.0035	0.0010	Yes
7	14.25	141.75	74.17	0.56	0.12	0.0033	0.0007	Yes
6	14.25	122.69	83.59	0.53	0.12	0.0031	0.0007	Yes
5	14.25	122.29	83.77	0.46	0.12	0.0027	0.0007	Yes
4	14.25	122.29	83.75	0.36	0.09	0.0021	0.0005	Yes
3	18.83	116.61	75.95	0.34	0.14	0.0015	0.0006	Yes
2	16.00	119.36	71.24	0.00	0.00	0.0000	0.0000	Yes
Total	**176.41**			**6.06**	**1.61**			

Table 11 – Dual System Option 1 Seismic Drift

The final designs of the braced frames and moment frames for the first option of the dual system can be found in Appendix C.

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Control Point	Coordinates X (ft)	Coordinates Y (ft)	Maximum Displacement (in)	≤ 5.29"
Center of Mass	124.35	85.16	3.29 (E-W)	Yes
Southwest Corner	18.33	29.42	3.43 (N-S)	Yes
West Midpoint	18.33	55.50	3.43 (N-S)	Yes
Northwest Corner	18.33	140.42	3.50 (E-W)	Yes
North Midpoint	95.94	140.42	3.50 (E-W)	Yes
Northeast Corner	210.21	140.42	3.50 (E-W)	Yes
East Midpoint	210.21	55.50	3.20 (E-W)	Yes
Southeast Corner	210.21	29.42	3.20 (E-W)	Yes
South Midpoint	95.94	29.42	3.20 (E-W)	Yes

Table 12 – Dual System Option 1 H/400 Displacement

Option 2:

1. Keep the sizes of the final design of the braced frame system, in which all story drifts and total displacements are acceptable.
2. Analyze the lateral members using RAM Structural System, using the gravity sizes for the moment frame beams for the first iteration. Increase any sizes as necessary.
3. Examine the extent to which drift was reduced.

The second option was examined with the sole intention of reducing drift as much as possible since drift presents a concern to the various glass curtain walls that clad the 8th Street Office Building. It was discovered that the largest drift ratio, which occurs at the roof level, was almost halved from 0.0061 for the braced frame system to 0.0032. See Table 13 below for the drift ratios due to seismic loading for the second dual system option.

Level	Story Height Below h_{sx} (ft)	Center of Mass Coordinates X (ft)	Center of Mass Coordinates Y (ft)	Story Drift (in) X (E-W)	Story Drift (in) Y (N-S)	Drift Ratio X (E-W)	Drift Ratio Y (N-S)	≤ 0.00625
Roof	16.58	124.36	85.16	0.64	0.20	0.0032	0.0010	Yes
PH Mezzanine	13.42	117.16	84.87	0.52	0.11	0.0032	0.0007	Yes
Penthouse	14.08	117.33	88.48	0.54	0.12	0.0032	0.0007	Yes
10	13.50	90.78	87.51	0.52	0.16	0.0032	0.0010	Yes
9	13.50	98.62	87.67	0.50	0.16	0.0031	0.0010	Yes
8	13.50	98.52	87.65	0.47	0.15	0.0029	0.0009	Yes
7	14.25	141.71	74.20	0.46	0.10	0.0027	0.0006	Yes
6	14.25	122.75	83.59	0.41	0.10	0.0024	0.0006	Yes
5	14.25	122.35	83.76	0.36	0.10	0.0021	0.0006	Yes
4	14.25	122.33	83.76	0.27	0.09	0.0016	0.0005	Yes
3	18.83	116.69	76.05	0.27	0.14	0.0012	0.0006	Yes
2	16.00	119.43	71.43	0.00	0.00	0.0000	0.0000	Yes
Total	**176.41**			**4.96**	**1.43**			

Table 13 – Dual System Option 2 Seismic Drift

See Table 14 below for the total displacements at the roof level at the center of mass and several points located around the perimeter of the roof. It is interesting to note that while the maximum displacement of 3.73" is well below the recommended limit of 5.29", it is actually greater than the 3.5" displacement from the first dual system option. The conclusion to be drawn from this observation is that the increased column sizes are more effective at limiting drift at the center of mass than the total displacement due to factored wind loading. This is expected though since the virtual load cases that were created for the Drift Control module in the design of the braced frame system were applied at the center of mass and paired with the seismic load cases.

Control Point	Coordinates		Maximum Displacement (in)	≤ 5.29"
	X (ft)	Y (ft)		
Center of Mass	124.36	85.16	2.65 (E-W)	Yes
Southwest Corner	18.33	29.42	3.73 (N-S)	Yes
West Midpoint	18.33	55.50	3.73 (N-S)	Yes
Northwest Corner	18.33	140.42	3.73 (N-S)	Yes
North Midpoint	95.94	140.42	3.17 (E-W)	Yes
Northeast Corner	210.21	140.42	3.17 (E-W)	Yes
East Midpoint	210.21	55.50	2.52 (E-W)	Yes
Southeast Corner	210.21	29.42	2.83 (E-W)	Yes
South Midpoint	95.94	29.42	2.83 (E-W)	Yes

Table 14 – Dual System Option 2 H/400 Displacement

The final designs of the braced frames and moment frames for the second option of the dual system can be found in Appendix C.

Comparisons and Conclusions

The various steel lateral systems that were proposed for the 8th Street Office Building have been investigated based on a variety of criteria. Table 15 provides a general summary of the advantages and disadvantages of each system. The steel plate shear wall system and the moment frame system have been eliminated as contenders for the optimal steel lateral system for the 8th Street Office Building based on one or more disadvantages that could not be easily overcome, as indicated by the negative signs in Table 15. It would appear from Table 15 that the braced frame system is the obvious choice for optimal lateral system. However, serviceability is a significant concern for the 8th Street Office Building, which should be given greater priority than other criteria.

General Steel Lateral System Comparison					
Criterion	**Steel Plate Shear Walls**	**Braced Frames**	**Moment Frames**	**Dual System Option 1**	**Dual System Option 2**
Serviceability (Drift)	✓	✓	–	+	+
Ease of Construction	+	+	–	✓	✓
Speed of Construction	+	+	–	✓	✓
Foundation Impact (Weight)	✓	✓	✓	✓	✓
Compatibility with Architecture	–	✓	+	✓	✓
Cost	+	+	–	✓	✓

Table 15 – General Comparison of the Alternative Steel Lateral Systems

Both dual systems have proven to be more successful at story drift and total displacement control than the braced frame system. Please see Figures 22 and 23 on the following page for visual comparisons of the maximum drift ratios and displacements among the braced frame system and dual systems. While the braced frame system is more advantageous with regard to constructability and cost, the support for better constructability and lower cost is simply derived from the fact that the braced frame system does not contain moment connections. In fact, the first dual system option may be equal in cost to the braced frame system because the smaller amount of steel tonnage may actually offset the complexity of the moment connections. However, it would be necessary to perform a complete steel takeoff for each fully designed lateral system as well as consult steel fabricators and contractors in order to make a specific determination, which is beyond the scope of this study. Therefore, it has been concluded that one of the dual systems should be chosen as the optimal system based on drift control. Furthermore, the first dual system option has been chosen as the optimal steel lateral system for the 8th Street Office Building because it uses less steel tonnage than the second dual system option.

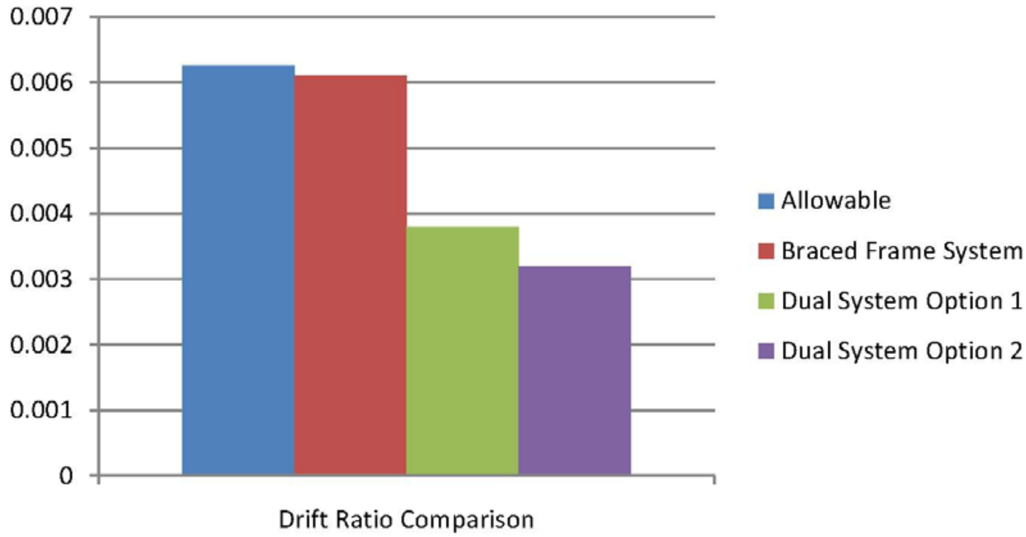

Figure 22 – Bar Graph Comparison of Drift Ratios due to Seismic Loading

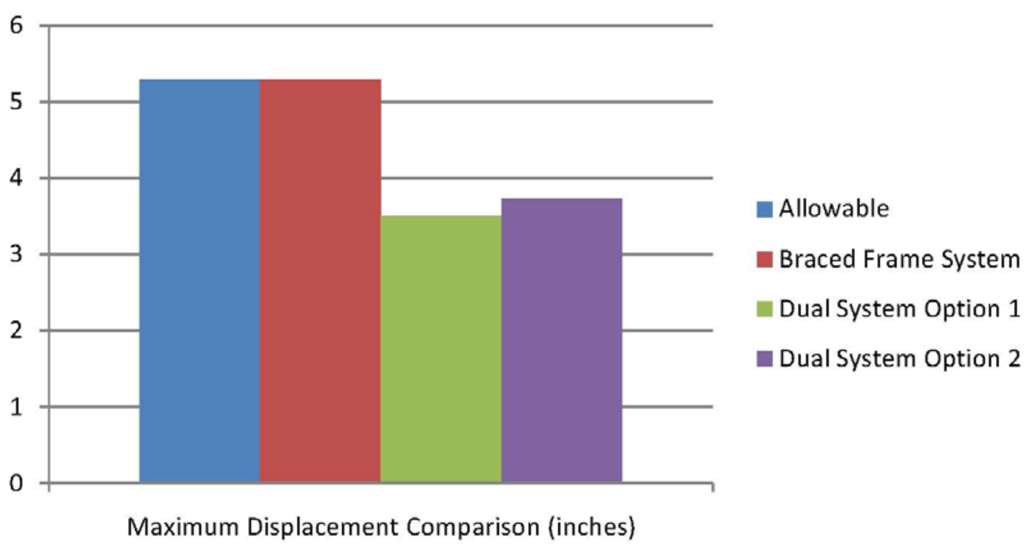

Figure 23 – Bar Graph Comparison of Maximum Displacements due to Factored Wind Loading

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Architecture Study: Redesign of the Service Core

The main goal of the architecture study was to reduce the impact of the overall service core of the 8th Street Office Building on the useable space available to the tenants. Because the layout of the existing service core is already very efficient, there were few possible options to reduce the area of the core. Ultimately, the primary solution was to eliminate the corridor between the restrooms and the farthest east transportation core consisting of Stair A on the 3rd floor through the 10th floor. As a result, the transportation core consisting of Stair A was shifted west by 6'-0". Therefore, each floor gained 180 square feet of office space since the transportation core is 30' long, which is a total increase of 1,440 square feet for the tenants of the 8th Street Office Building. Please see Appendix D for the typical floor plans of the overall service core. Existing plans and the revised plans that resulted from this study have both been provided for the sake of comparison.

The following are the effects of the relocation of the transportation core on the surrounding architecture of the parking garage:

- The fourth level of the parking garage below grade was not affected since it is unexcavated on the eastern half of the site.
- On the third level of the parking garage below grade, parking spaces 117 and 151 were relocated to the opposite side of Stair A as a result of the 6' shift.
- On the second level of the parking garage below grade, parking spaces 54 and 72 were relocated to the opposite side of Stair A as a result of the 6' shift. It is no longer possible to designate parking space 54 as a handicapped space due to lack of pedestrian area adjacent to the parking space. However, parking space 1 on the level above has the potential to replace parking space 54 as a handicapped space.
- On the first level of the parking garage below grade, parking space 15 was relocated to the opposite side of Stair A as a result of the 6' shift. Furthermore, the number and size of bicycle racks were maintained, albeit less concentrated in one space.

The following are the effects of the relocation of the transportation core on the surrounding architecture of the 1st and 2nd floors:

- The assembly area to the east of the transportation core gained 180 square feet. While the existing design provides enough space for seating for 321 people, the additional 180 square feet will increase the space for gathering and greeting near the main entrance to the assembly area.
- The exit passageway from the assembly area was increased in length by 6'-0". However, means of egress as required by code was not violated because the length of exit travel

only increased from 143 feet to 149 feet, and the maximum length of travel for use group A with a sprinkler system is 250 feet.

- The major drawback to the shift of the transportation core is the loss of 180 square feet in the main lobby, which results in loss of symmetry with respect to the security entrance and the escalators.

The following are the effects of the relocation of the transportation core on the surrounding architecture of the 3rd through the 9th floors:

- The entrance to the women's restroom was relocated to the southern wall, with the door swing oriented to minimize public view into the restroom.
- Likewise, the entrance to the men's restroom was relocated to the northern wall, with the door swing oriented to minimize public view into the restroom.
- The water fountains were also relocated to the outside of the southern wall of the women's restroom and the northern wall of the men's restroom.
- The entrance to Stair A was relocated to the northern wall of the transportation core.

All of the above effects occurred in the redesign of the 10th floor service core with the exception of the relocation of the restroom entrances and water fountains. An existing access stairwell to the penthouse mechanical room located adjacent to the restrooms on the 10th floor required the restroom entrances and water fountains to be located on the southern and northern walls before the redesign of the overall service core. The penthouse, penthouse mezzanine, and roof levels were not affected by the relocation of the transportation core.

After evaluating the increase of 1,440 square feet in useable space for the tenants versus the effects on the architecture of the 8th Street Office Building, it was concluded that the new overall service core layouts are feasible. All means of egress, number of parking spaces, restrooms, and water fountains were maintained. One minor drawback is the location of the entrances to the restrooms, but the line of sight into the restrooms is still minimal and considered acceptable. The only significant negative effect is the lack of symmetry in the main lobby, which may not be as aesthetically pleasing as the original design. The pros and cons would need to be presented to the owner before making any final decisions. For the purposes of this study, the new overall service core plans will be used in the redesign of the optimal steel lateral system for the 8th Street Office Building.

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Sustainability Study: Green Roof and Rainwater Harvesting

Green Roof:

After consulting with American Hydrotech, Inc., it was decided that an extensive green roof would be utilized on the 3rd, 7th, and 10th floor terraces of the 8th Street Office Building. Please refer to Appendix E for the exact locations of the green roofs. The terraces were chosen as the locations for the green roofs in order to provide the tenants with access to a relaxing and pleasant outdoor space. Initially, a shallow-intensive green roof was desired due to its ability to support sod lawns and perennials. However, the lower cost, lighter weight, and minimal maintenance associated with an extensive green roof dictated the final decision. Furthermore, extensive green roofs are still capable of creating an aesthetically pleasing outdoor space for the tenants, as shown by the green roof atop the Milton Hershey School Health Center in Figure 24 below.

Figure 24 – Milton Hershey School Health Center
(courtesy of American Hydrotech, Inc.)

The Hydrotech Extensive Garden Roof Assembly will provide the following benefits to the 8th Street Office Building:

- An enjoyable outdoor space for the use of the tenants
- Stormwater retention
- Additional thermal resistance
- Reduced noise levels
- Mitigation of the Urban Heat Island Effect

The following figure shows a schematic design for a layout of the extensive green roof elements on the 10th floor terrace. The green indicates a variety of sedums that are provided by American Hydrotech, Inc. The paths shown are composed of gravel, but other materials may be specified by the owner, such as concrete pavers or wooden slats. Finally, benches have been provided for the use of the tenants.

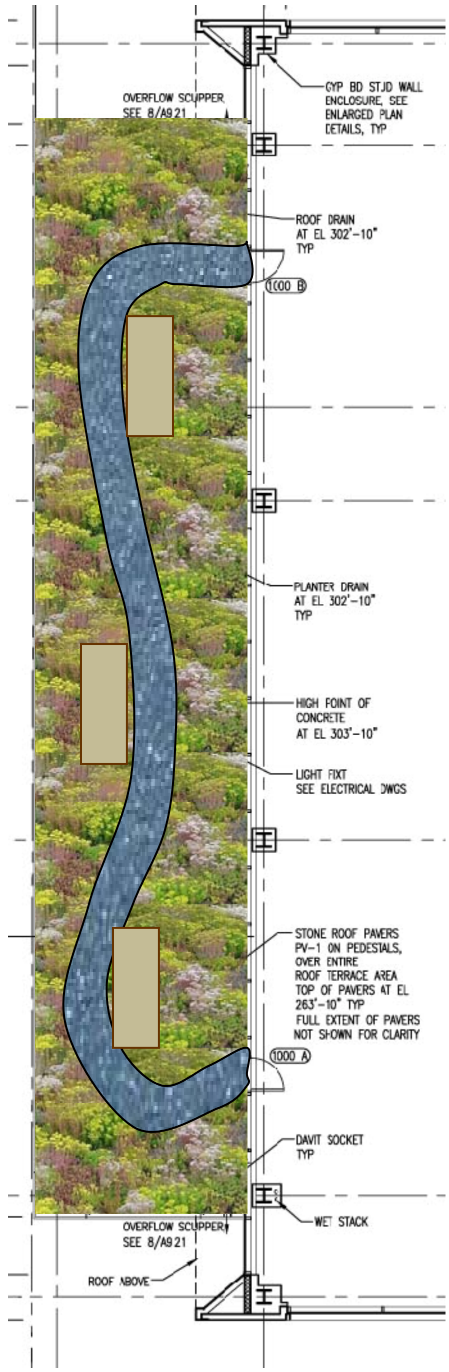

Figure 25 – Schematic Design of the 10th Floor Green Roof

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The Hydrotech Extensive Garden Roof Assembly typically consists of the following components:

- Vegetation
- LiteTop Growing Media (soil)
- System filter
- Gardendrain GR30
- Styrofoam
- Root Stop with Hydroflex 30
- Monolithic Membrane 6125

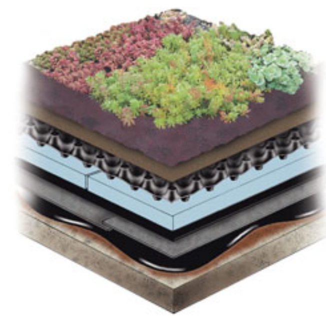

Figure 26 – Extensive Garden Roof Assembly
(courtesy of American Hydrotech, Inc.)

Please refer to Figure 27 below for a detail of a typical transition from a path for the tenants to the green roof assembly.

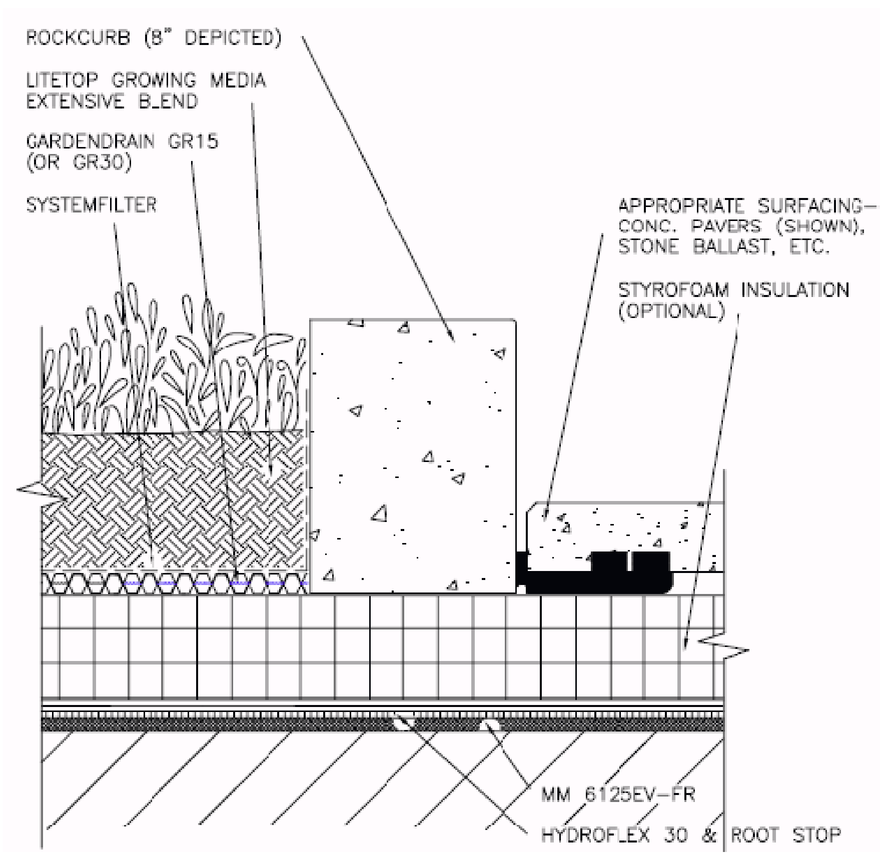

Figure 27 – Detail of a Possible Transition Option for an Extensive Garden Roof Assembly
(courtesy of American Hydrotech, Inc.)

In accordance with the earlier stated goal of reducing stormwater runoff, it was decided to design an extensive green roof that would result in a 50% decrease in the volume of stormwater runoff from a two-year 24-hour design storm for the green roof areas. For Richmond, VA, the rainfall from a two-year 24-hour design storm is 2.75 inches, so a 50% reduction in volume corresponds to approximately 1.38 inches of rainfall. Table 16 displays an extensive garden roof design courtesy of American Hydrotech, Inc. that is capable of holding 1.57 inches of moisture, which exceeds the target 1.38 inches. It is estimated that the designed extensive green roof will cost $32 per square foot, which results in a total cost of $257,600 for the total area of 8,051 square feet.

Element	MM6125	Root Stop with Hydroflex 30	Gardendrain GR30 (filled)	Filter Fabric	Intensive Soil	Sedum Carpet	Total
Profile Height (inches)	0.25	0.1	1.2	0.01	4.0	n/a	5.56
Saturated Weight (psf)	1.5	0.8	3.8	0.03	27.0	5.0	38.13

Table 16 – Hydrotech Extensive Garden Roof Assembly Design

Unfortunately, the green roofs alone do not satisfy the criteria for the LEED Sustainable Site Credit 6.1: Stormwater Design: Quantity Control. For Case 2, in which the existing imperviousness is greater than 50%, a 25% decrease in the volume of stormwater runoff from a two-year 24-hour design storm is required. While a 50% decrease was achieved for the green roof areas, the terraces where the green roofs are located only constitute a total area of 8,051 square feet, which is approximately 23% of the total roof area.

The stormwater retention achieved by the green roofs is sufficient to meet the requirements of the LEED Sustainable Site Credit 6.2: Stormwater Design: Quantity Control, in which the stormwater runoff from 90% of the average annual rainfall must be treated using acceptable best management practices. Of course, the Hydrotech Extensive Garden Roof Assembly is considered a best management practice. It was determined that the average annual rainfall for Richmond, VA is 39.2 inches, and 0.75 inches must be treated in order to meet the requirements of this credit. The green roofs retain 1.57 inches, which is more than double the required amount.

See Figure 28 on the following page for a summary graph of the annual retention of the green roof area in comparison to the retention capabilities of a “bare” roof and the total precipitation for Richmond, VA.

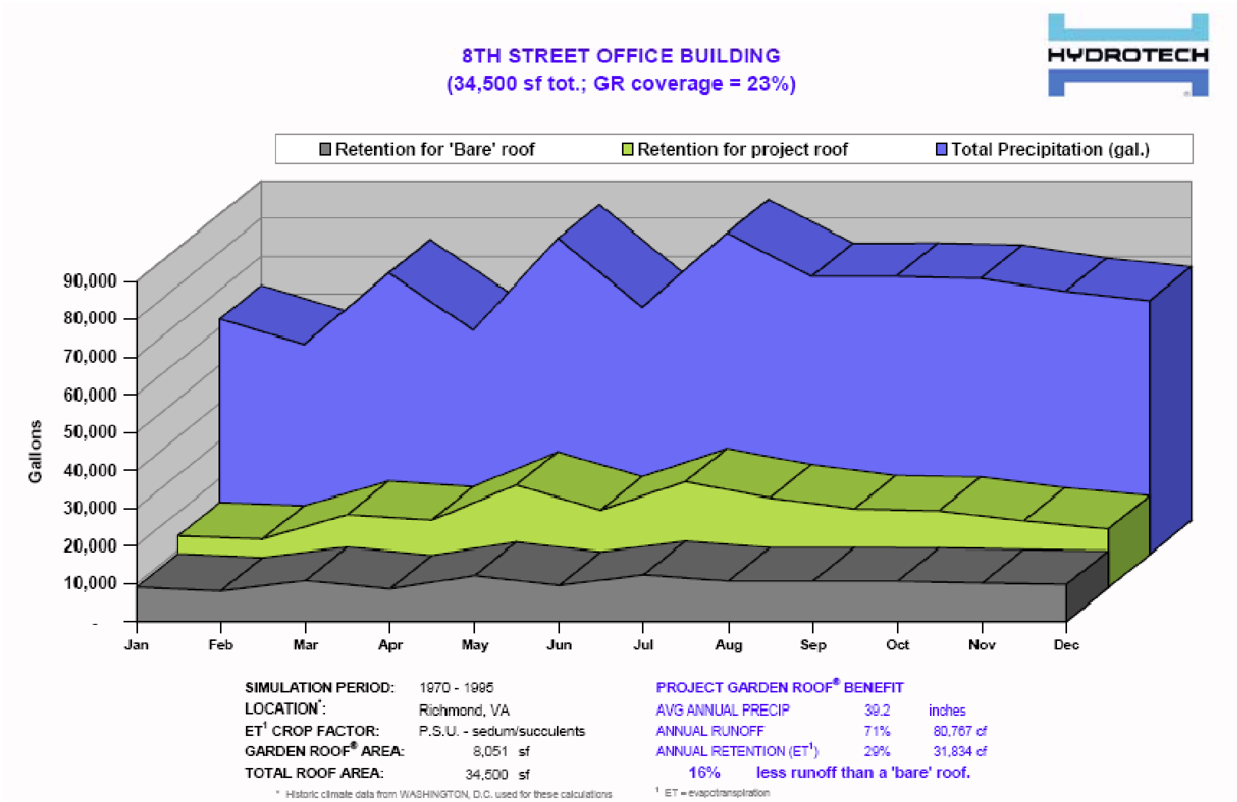

Figure 28 – Summary Graph of the Stormwater Retention Capabilities of the Designed Green Roof

Rainwater Harvest System:

After investigating the use of a green roof on the terraces, a water collection system was considered to take advantage of the significant amount of rainfall on the inaccessible roof areas. Once again, an average annual precipitation of 39.2 inches was used. However, it was assumed that the rainfall is uniformly distributed throughout the year in order to approximately size the water holding tanks. Furthermore, it was assumed that little to no runoff will occur where the green roofs are being implemented. Therefore, the total roof area that was considered for the rainwater harvest system was determined by subtracting the green roof area from the total roof area. Please see Table 17 below for the sizing of the water holding tanks for the 8th Street Office Building.

Roof Area (square feet)	Rainfall (inches)	Volume of Water (cubic ft)	Average Water Supply per Month (cubic ft)	Average Water Supply per month (gallons)	Sizes of Provided Tanks (gallons)
26,550	39.2	86730	7227.50	54065.46	3 @ 1000

Table 17 – Design of Water Collection Tanks

It should be noted that the tanks were not sized to treat the entirety of the anticipated stormwater. Once the tanks are filled, the excess water will be redirected into the stormwater removal system. It was determined that the best location for the water collection tanks is in the southern unexcavated area of the first parking garage level below grade adjacent to Elevator 10 and an air exhaust shaft/plenum. Please refer to Figure 29 for the exact location.

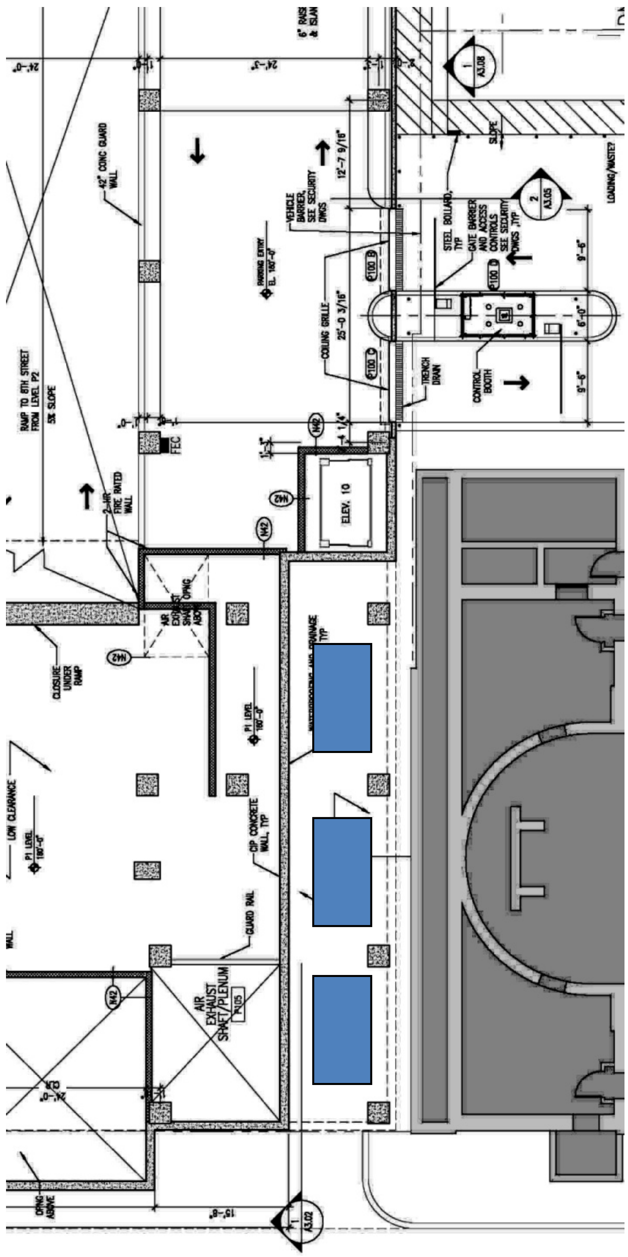

Figure 29 – Proposed Locations of the Water Collection Tanks

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In order to calculate the amount of water savings for the building's sanitary system as a result of the rainwater harvest system, it was assumed that the entirety of the uniform monthly rainfall will be collected by the tanks and transferred to the system. Of course, the amount of rainfall that is actually collected depends on the exact distribution of rainfall for Richmond, VA. Furthermore, it was assumed that each occupant will utilize the restroom three times per workday and that an average of 23 workdays occur each month. See Table 18 below for the calculations that resulted in a water savings of approximately 13.8% for the 8th Street Office Building's sanitary system.

Occupancy	Average Days Occupied per Month	Number Toilet Flushes per Month	Number Urinal Flushes per Month	Water Used per Month (gallons)	Monthly Rainfall (gallons)	Savings
3910	23	202343	67448	391200	54065	13.8%

Table 18 – Calculated Water Savings Attributed to the Rainwater Harvest System

Structural Design of Optimal Lateral System

Once the architecture and sustainability studies were completed, the chosen optimal steel lateral system for the 8th Street Office Building was designed utilizing the revised architecture and roof loads. Since the first dual system option of combined braced and moment frames was chosen, the only change as a result of the revised architecture was a 6' shift west of braced frames 10, 11 and 12. Because a dead load of 100 psf was initially utilized for the terraces in order to account for pavers and other various materials, an extensive garden roof assembly dead load of approximately 38 psf did not warrant an increase in the overall terrace dead load. RAM Structural System was utilized once again in the design of the dual system. See Figure 30 below for a three-dimensional representation of the gravity members, braced frames, and moment frames that were modeled in RAM Structural System for the purposes of designing the optimal dual system. The same assumptions were made in the design of the dual system as indicated previously. Please refer to Appendix F for the final design of the optimal system. Braced Frame 11 was the only frame that changed as a result of its relocation in accordance with the revised architecture. This was expected since its new location caused the braced frame to receive more load from the mechanical penthouse.

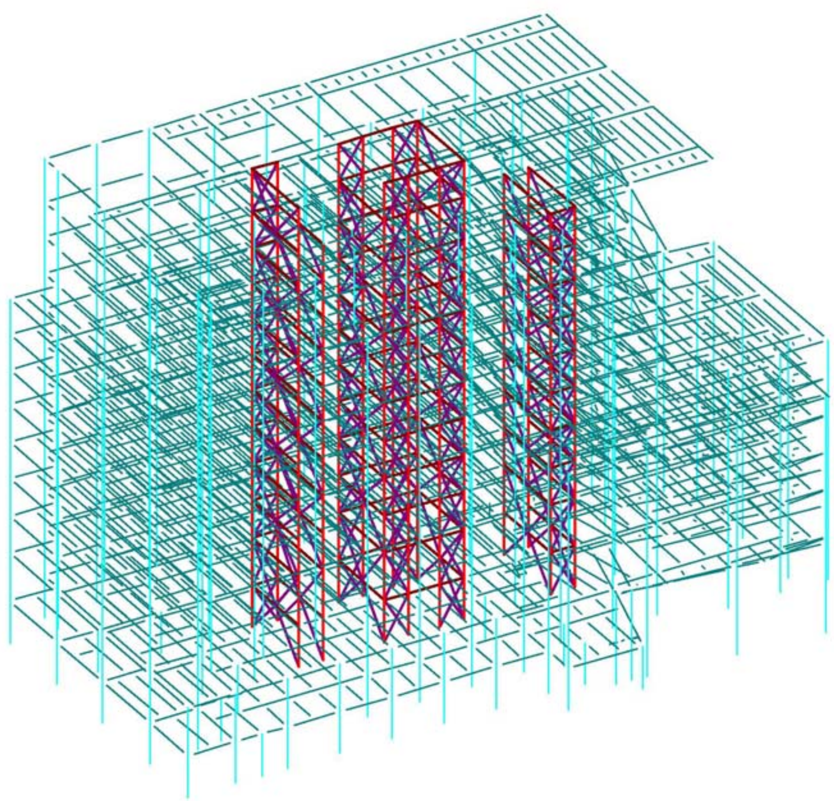

Figure 30 - RAM Model of Optimal Steel Lateral System combined with Revised Architecture and Loads

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Table 19 below shows that the drift ratio at the center of mass of each level due to seismic loading was less than the allowable drift ratio by code. Furthermore, Table 12 shows that the maximum total displacement from the control points checked at the roof level was 3.81 inches, which is well below the recommended limit 5.29 inches. Notice that the seismic drift and maximum displacement results are very similar to the results prior to the revised architecture as expected.

Level	Story Height Below h_{sx} (ft)	Center of Mass Coordinates X (ft)	Y (ft)	Story Drift (in) X (E-W)	Y (N-S)	Drift Ratio X (E-W)	Y (N-S)	≤ 0.00625
Roof	16.58	124.35	85.16	0.76	0.22	0.0038	0.0011	Yes
PH Mezzanine	13.42	117.16	84.92	0.61	0.13	0.0038	0.0008	Yes
Penthouse	14.08	117.30	88.49	0.64	0.15	0.0038	0.0009	Yes
10	13.50	90.40	87.54	0.62	0.19	0.0038	0.0012	Yes
9	13.50	98.14	87.71	0.62	0.18	0.0038	0.0011	Yes
8	13.50	98.05	87.69	0.57	0.16	0.0035	0.0010	Yes
7	14.25	141.65	74.14	0.56	0.14	0.0033	0.0008	Yes
6	14.25	122.47	83.60	0.53	0.12	0.0031	0.0007	Yes
5	14.25	122.06	83.79	0.44	0.12	0.0026	0.0007	Yes
4	14.25	122.05	83.77	0.36	0.09	0.0021	0.0005	Yes
3	18.83	116.38	75.93	0.00	0.00	0.0000	0.0000	Yes
2	16.00	119.10	71.14	0.00	0.00	0.0000	0.0000	Yes
Total	**176.41**			**5.71**	**1.50**			

Table 19 – Optimal Steel Lateral System Seismic Drift

Control Point	Coordinates X (ft)	Y (ft)	Maximum Displacement (in)	≤ 5.29"
Center of Mass	124.35	85.16	3.37 (E-W)	Yes
Southwest Corner	18.33	29.42	3.81 (E-W)	Yes
West Midpoint	18.33	55.50	3.76 (N-S)	Yes
Northwest Corner	18.33	140.42	3.76 (N-S)	Yes
North Midpoint	95.94	140.42	3.14 (E-W)	Yes
Northeast Corner	210.21	140.42	3.14 (E-W)	Yes
East Midpoint	210.21	55.50	3.60 (E-W)	Yes
Southeast Corner	210.21	29.42	3.81 (E-W)	Yes
South Midpoint	95.94	29.42	3.81 (E-W)	Yes

Table 20 – Optimal Steel Lateral System H/400 Displacement

Conclusion

The main focus of this final thesis report was to analyze a variety of steel lateral systems as they apply to the 8th Street Office Building to be located in Richmond, VA. The existing lateral system of reinforced concrete shear walls has not been fully optimized, so steel lateral systems were examined as possible alternatives. Steel plate shear walls, braced frames, moment frames, and a dual system of combined braced and moment frames were considered as viable options.

Both the steel plate shear walls and braced frames were considered because they are economical, lightweight, and simple and quick to construct. However, the architecture of the 8th Street Office Building prevented the systems from being placed in locations free of openings. Therefore, a braced frame system was fully designed for the building, rather than a steel plate shear wall system, since it can be more fully integrated with the openings. Unfortunately, the braced frame system was governed by a seismic drift ratio of 0.00625 and a recommended total displacement limit of 5.29 inches.

The moment frame system was considered because it can be located and designed without regard to placement of openings. Ultimately, a schematic design indicated that the moment frames alone could not overcome the large floor to floor height of 18'-10". Finally, the dual system of braced and moment frames was fully designed. The dual system was conceived by locating two central moment frames within the braced frame system. Ultimately, the best dual system was able to reduce the seismic drift ratio to 0.0038, the maximum displacement to 3.5 inches, and the overall steel tonnage from the braced frame system.

The architecture study involved a redesign of the overall service core for the 8th Street Office Building. It was decided that the elimination of a corridor between the restrooms and Stair A on the 3rd through the 9th floors would govern the redesign. Ultimately, 1,440 square feet were gained in useable space by the tenants and all parking spaces and means of egress were maintained. The major drawback from the revised service core was a loss of symmetry in the main ground floor lobby.

The sustainability study involved the selection of an extensive garden roof assembly for the terraces and the implementation of a rainwater harvest system. The extensive green roof was chosen because of its light weight, hardiness, stormwater retention capabilities, and pleasing aesthetic. The extensive garden roof assembly provided by American Hydrotech, Inc. is capable of retaining 1.57 inches of rainfall and contributes to the LEED certification of the 8th Street Office Building. The rainwater collection system consists of three 1,000 gallon holding tanks that are capable of saving the building approximately 13.8% of its sanitary system water usage.

The goals of this thesis were to analyze alternative steel lateral systems, maximize useable space for the tenants, and implement sustainable strategies. Based on the results discussed, these goals have been met. All design methods and values utilized are in accordance with the applicable codes. Please refer to the appendices for further review of detailed notes, figures, and tables.

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Appendix A – Existing Drawings

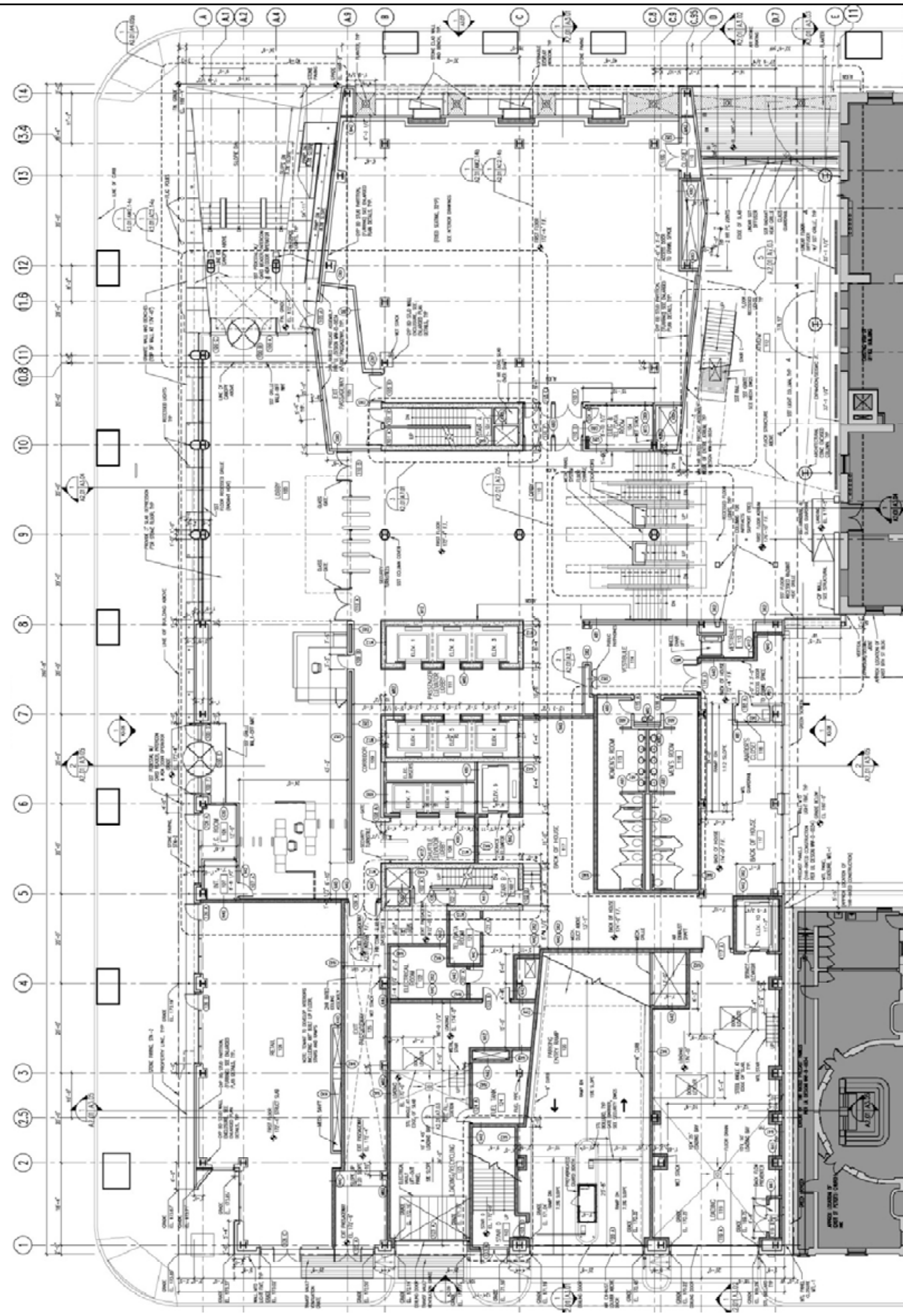

Figure 31 – 1st Floor Plan

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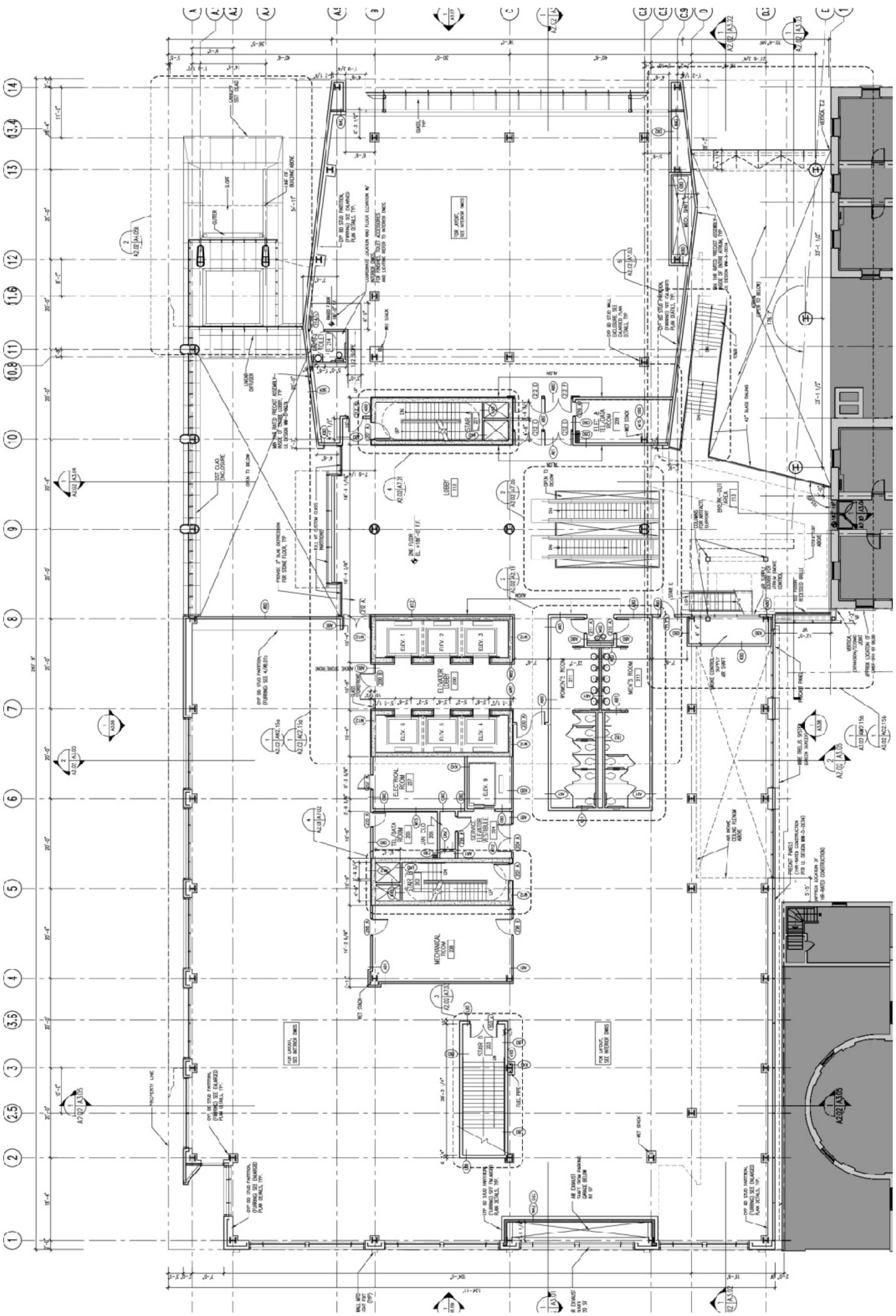

Figure 32 – 2nd Floor Plan

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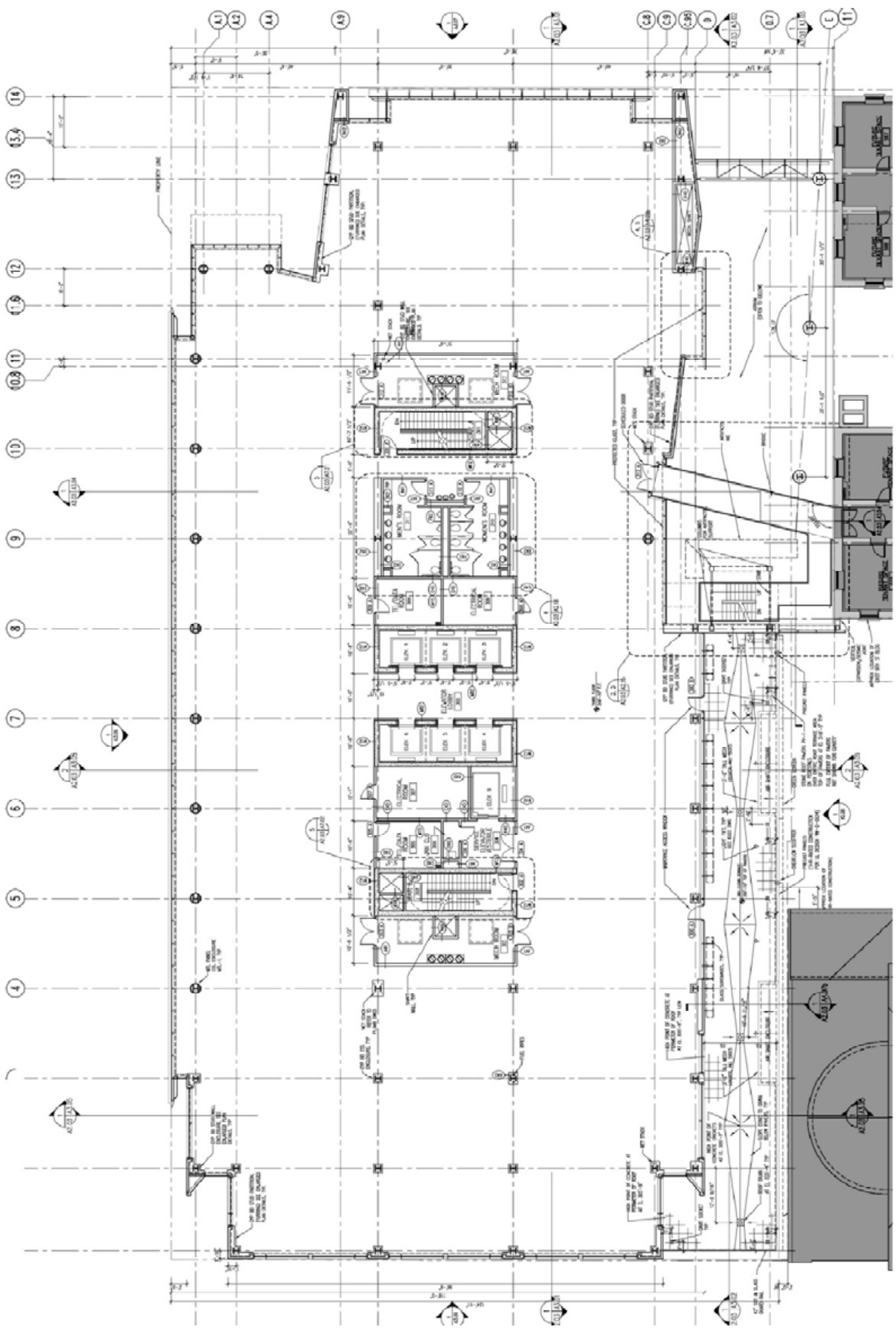

Figure 33 – 3rd Floor Plan

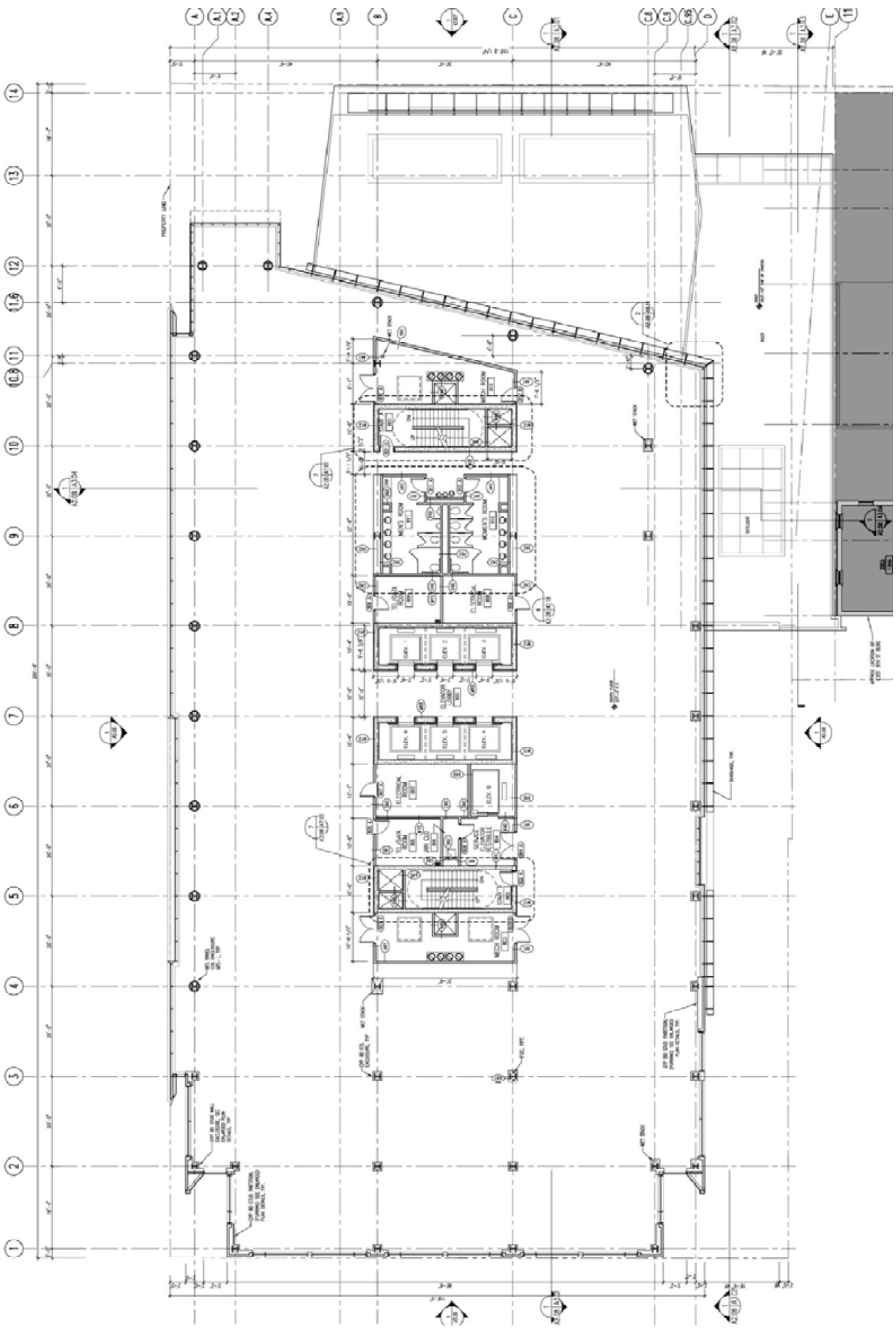

Figure 34 – 8th Floor Plan

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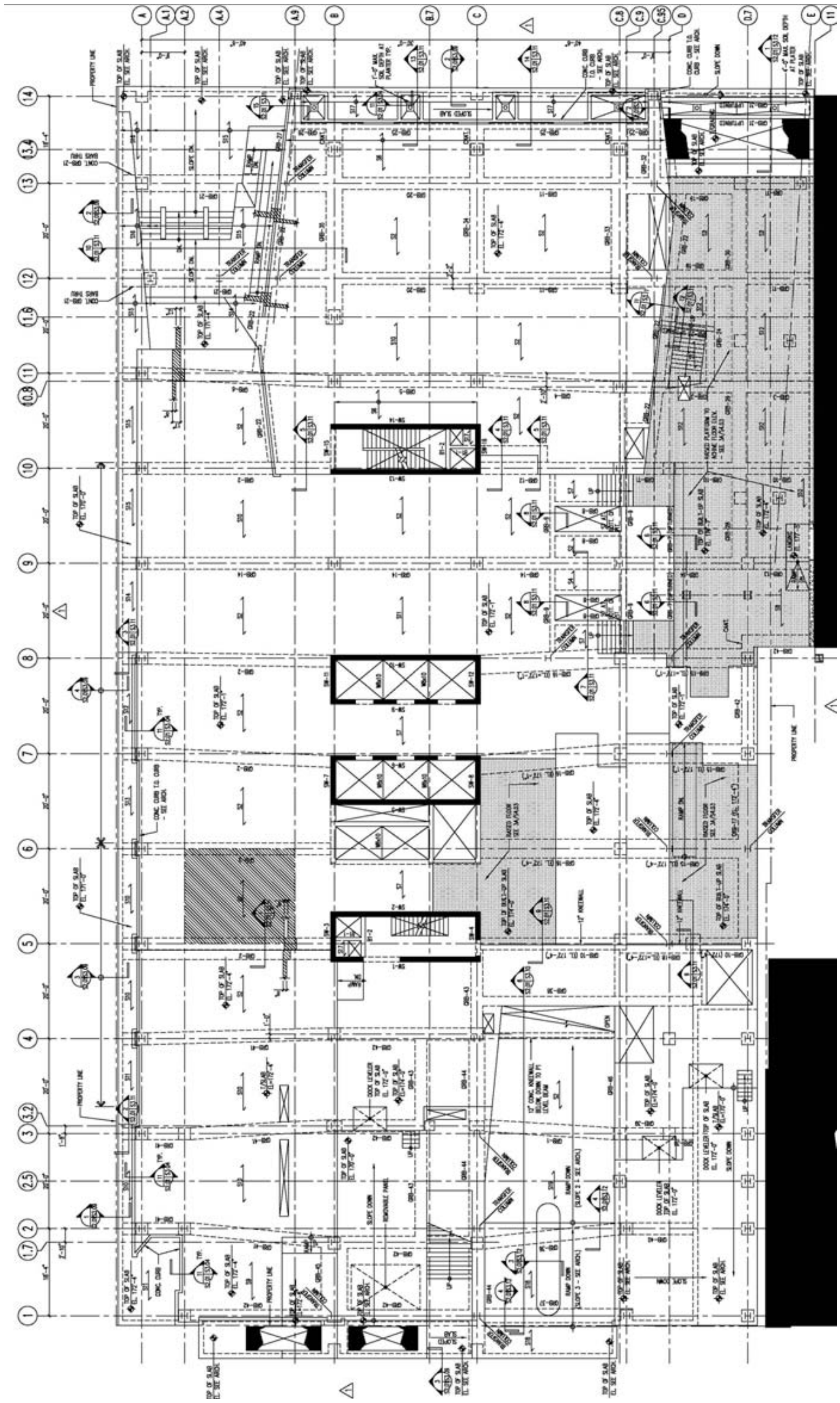

Figure 35 – 1st Floor Framing Plan

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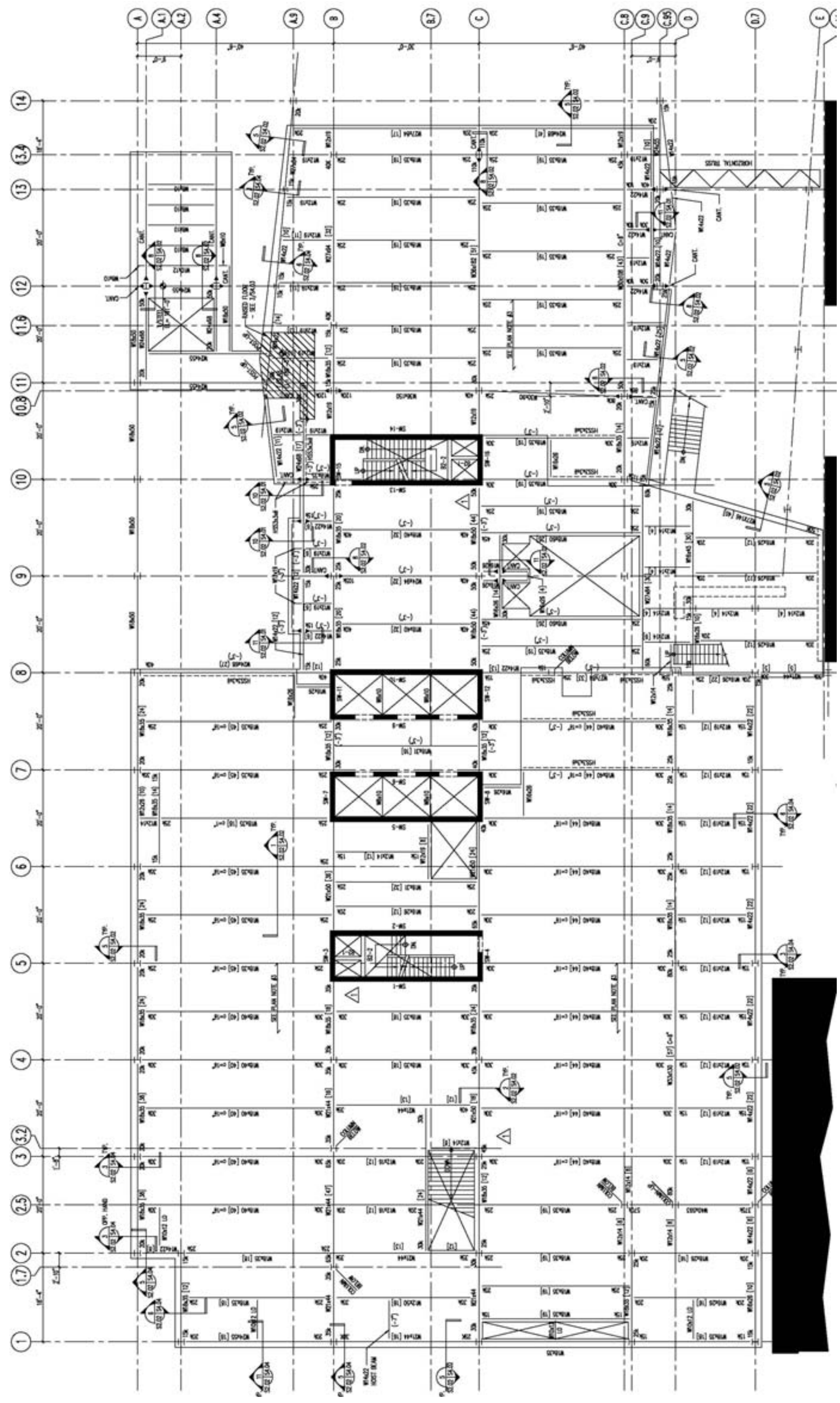

Figure 36 – 2nd Floor Framing Plan

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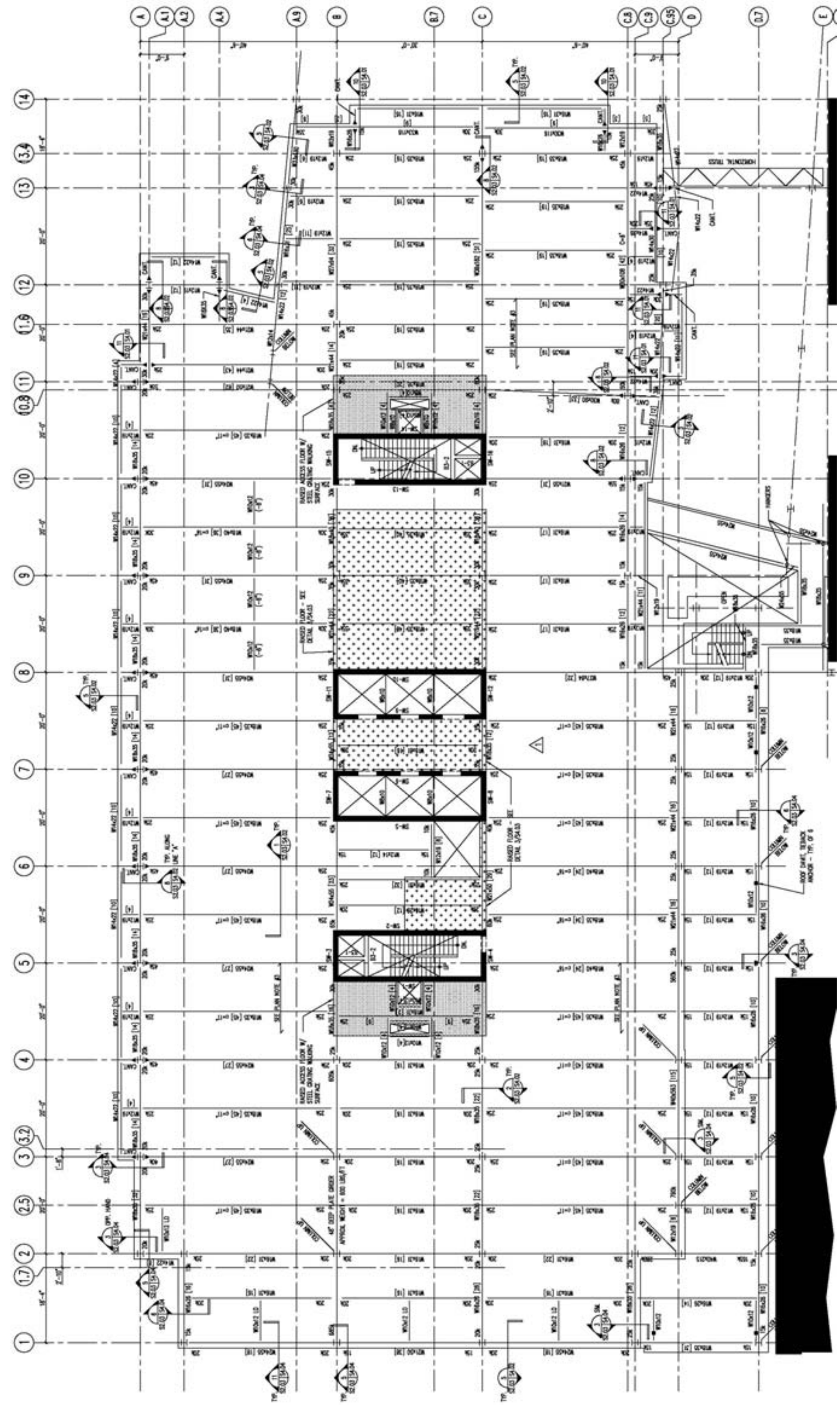

Figure 37 – 3rd Floor Framing Plan

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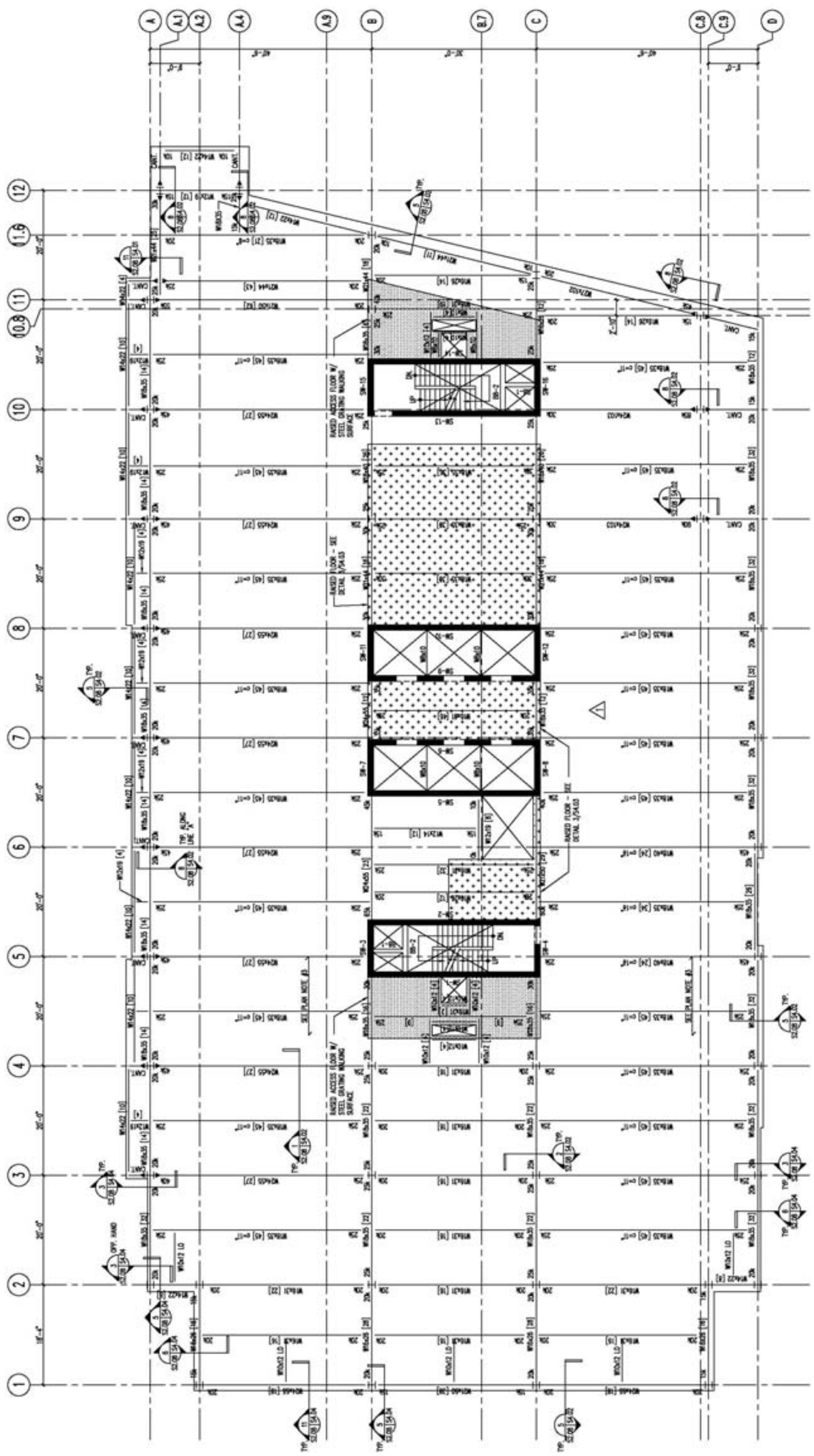

Figure 38 – 8th Floor Framing Plan

Appendix B – Design Loads for Alternative Lateral Systems

Dead

	COMPOSITE PROPERTIES												
	Slab Depth	ϕM_{nf} in.k	A_c in²	Vol. ft³/ft²	W psf	S_c in³	I_{av} in⁴	ϕM_{no} in.k	ϕV_{nt} lbs.	Max. unshored spans, ft. 1span	2span	3span	A_{wwf}
18 gage	4.50	62.08	32.6	0.292	34	1.53	5.4	42.99	4560	9.20	11.33	11.71	0.023
	5.25	77.02	40.0	0.354	41	1.95	8.3	54.72	5590	8.54	10.62	10.97	0.029
	6.00	91.95	48.0	0.417	48	2.39	12.1	67.07	6530	8.01	10.02	10.36	0.036
	6.25	96.93	50.8	0.438	50	2.54	13.6	71.29	6730	7.86	9.84	10.17	0.038
	6.50	101.91	53.6	0.458	53	2.69	15.2	75.55	6920	7.71	9.68	10.00	0.041
	7.00	111.87	59.5	0.500	58	3.00	18.8	84.17	7340	7.44	9.36	9.67	0.045
	7.25	116.85	61.9	0.521	60	3.16	20.7	88.52	7500	7.32	9.21	9.52	0.047
	7.50	121.83	64.3	0.542	62	3.31	22.8	92.91	7670	7.24	9.07	9.38	0.050

Table 21 – United Steel Deck Composite Properties for 2" Metal Decking with 3 ¼" Lightweight Topping

→ Steel Framing

Perform a couple spot calculations:

3rd floor between gridlines C-D and 6-8

beams { (3) W18×35 40.5' long
(1) W18×40 40.5'
(2) W21×44 20'

columns (3) W14×99 14.25' tall

weight = 3(35)(40.5) + 1(40)(40.5) + 2(44)(20)
+ 3(99)(14.25) = $\frac{11505 \text{ lb}}{40' \times 40.5''}$ = 7.1 psf

Area

8th floor between gridlines C-D and 6-8

beams same as above

columns (3) W14×68 13.5' tall

weight = 3(35)(40.5) + 1(40)(40.5) + 2(44)(20)
+ 3(68)(13.5) = $\frac{10027 \text{ lb}}{40' \times 40.5'}$ = 6.2 psf

∴ Use $\boxed{7 \text{ psf}}$ total for steel framing

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Snow

Snow Analysis

ground snow load (Fig. 7-1) $\quad p_g = \boxed{20 \text{ psf}}$

flat roof snow load (Eqn 7-1) $\quad p_f = 0.7\, C_e C_t I p_g$

exposure factor (Table 7-2) $\quad C_e = \boxed{1.0}$ Category B, partially exposed

thermal factor (Table 7-3) $\quad C_t = \boxed{1.0}$

Importance factor (Table 7-4) $\quad I = \boxed{1.1}$ Category III

$\Rightarrow p_f = 0.7(1.0)(1.0)(1.1)(20) = \boxed{15.4 \text{ psf}}$

$p_g \leq 20 \text{ psf}$ so $p_f \geq I p_g = 1.1(20) = \boxed{22 \text{ psf}}$

CONTROLS

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Wind

Analysis 1:

Wind Analysis - Method 2 in Chapter 6 of ASCE 7-05

Location: Richmond, VA

Basic wind speed (Figure 6-1) $\boxed{V = 90 \text{ mph}}$

Wind directionality factor (Table 6-4) $\boxed{K_d = 0.85}$

Importance Factor (Table 6-1) $\boxed{I = 1.15}$ ... based on • Occupancy Category III (Table 1-1)

$\boxed{\text{Exposure Category} = B}$ .... based on • V = 85-100 mph • Surface Roughness B (urban & suburban areas)

Velocity pressure exposure coeff.: (Table 6-3) ... based on Exposure B, Case 2

Height above ground level, z (ft)	K_z
0-15	0.57
20	0.62
25	0.66
30	0.70
40	0.76
50	0.81
60	0.85
70	0.89
80	0.93
90	0.96
100	0.99
120	1.04
140	1.09
160	1.13
176'-5"	1.16

interpolate in spreadsheet to get values by level

Topographic factor $\boxed{K_{zt} = 1.0}$ per 6.5.7.1 & 6.5.7.2

Gust effect factor: frequency $n_1 = \frac{100}{h \text{ (ft)}}$ (Eqn C6-17)

$$n_1 = \frac{100}{176'\text{-}5''} = \boxed{0.567 \text{ Hz}}$$

$\Rightarrow \boxed{\text{FLEXIBLE}}$ since $n_1 < 1$

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Gust effect factor for Flexible Structures:

→ $g_Q = g_v = 3.4$

$$g_R = \sqrt{2\ln(3600 n_1)} + \frac{0.577}{\sqrt{2\ln(3600 n_1)}} \quad \text{(Eqn. 6-9)} \quad ; n_1 = 0.567$$

→ $\boxed{g_R = 4.052}$

Section 6.3: B = horiz. dim. of building measured normal to wind direction

L = horiz. dim of bldg measured parallel to wind direction

h = mean roof height

	N-S	E-W
B	260'-8" = 260.67	145.25'
L	145.25'	260'-8" = 260.67
h	176'-5" = 176.42	176'-5" = 176.42

$\bar{z}$ = equivalent height of structure

$\bar{z}$ = max of 0.6h or z_{min}

$0.6h = 0.6(176.42) = \boxed{105.85\ ft}$ ← controls

$z_{min} = 30\ ft$ (Table 6-2)

$\bar{\alpha} = \frac{1}{4.0}$; $\bar{b} = 0.45$ (Table 6-2)

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) \quad \text{(Eqn 6-14)}$$

$$\bar{V}_{\bar{z}} = 0.45\left(\frac{105.85}{33}\right)^{1/4}(90)\left(\frac{88}{60}\right) = \boxed{79.49\ ft/s}$$

mean hourly wind speed

Intensity of Turbulence $I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6}$ (Eqn 6-5)

c = 0.30 (Table 6-2) $I_{\bar{z}} = 0.3\left(\frac{33}{105.85}\right)^{1/6} = \boxed{0.247}$

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the integral length scale of turbulence $L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}$

$\ell = 320$ ft ; $\bar{\epsilon} = \frac{1}{3.0}$ (Eqn 6-7)

(Table 6-2)

$$L_{\bar{z}} = 320\left(\frac{105.85}{33}\right)^{1/3} = \boxed{471.93}$$

Background response $Q = \sqrt{\dfrac{1}{1+0.63\left(\dfrac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$ (Eqn 6-6)

N-S $$Q = \sqrt{\dfrac{1}{1+0.63\left(\dfrac{260.67+176.42}{471.93}\right)^{0.63}}} = \boxed{0.790}$$

E-W $$Q = \sqrt{\dfrac{1}{1+0.63\left(\dfrac{145.25+176.42}{471.93}\right)^{0.63}}} = \boxed{0.818}$$

R_h : use $\eta = \dfrac{4.6 n_1 h}{\bar{V}_{\bar{z}}} = \dfrac{4.6(0.567)(176.42)}{79.49} = \boxed{5.789}$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1-e^{-2\eta}\right) \quad \text{(Eqn 6-13a)}$$

$$R_h = \frac{1}{5.789} - \frac{1}{2(5.789)^2}\left(1-e^{-2(5.789)}\right) = \boxed{0.158}$$

R_B : use $\eta = \dfrac{4.6 n_1 B}{\bar{V}_{\bar{z}}}$; $R_B =$ (Eqn 6-13a)

N-S $$\eta = \frac{4.6(0.567)(260.67)}{79.49} = \boxed{8.553}$$

$$R_B = \frac{1}{8.553} - \frac{1}{2(8.553)^2}\left(1-e^{-2(8.553)}\right) = \boxed{0.110}$$

E-W $$\eta = \frac{4.6(0.567)(145.25)}{79.49} = \boxed{4.766}$$

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R_B: E-W $\quad R_B = \frac{1}{4.766} - \frac{1}{2(4.766)^2}\left(1 - e^{-2(4.766)}\right)$

$= \boxed{0.188}$

R_L: use $\eta = \frac{15.4\, n_1 L}{\bar{V}_{\bar{z}}}$

N-S $\quad \eta = \frac{15.4(0.567)(145.25)}{79.49} = \boxed{15.955}$

$R_L = \frac{1}{15.955} - \frac{1}{2(15.955)^2}\left(1 - e^{-2(15.955)}\right) = \boxed{0.061}$

E-W $\quad \eta = \frac{15.4(0.567)(260.67)}{79.49} = \boxed{28.634}$

$R_L = \frac{1}{28.634} - \frac{1}{2(28.634)^2}\left(1 - e^{-2(28.634)}\right) = \boxed{0.034}$

$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}}$ (Eqn 6-12) $\quad N_1 = \frac{0.567(471.93)}{79.49} = \boxed{3.366}$

$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}}$ (Eqn 6-11)

$R_n = \frac{7.47(3.366)}{[1 + 10.3(3.366)]^{5/3}} = \boxed{0.065}$

$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$ (Eqn 6-10)

β = damping ratio, percent of critical

According to pg. 294 in ASCE 7-05 under Structural Damping: 1 to 2% for damping ratio

$\Rightarrow$ assume $\beta = 1.5\% = 0.015$

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N-S $R = \sqrt{\frac{1}{0.015}(0.065)(0.158)(0.110)(0.53 + 0.47 \times 0.061)}$

$\boxed{R = 0.205}$

E-W $R = \sqrt{\frac{1}{0.015}(0.065)(0.158)(0.188)(0.53 + 0.47 \times 0.034)}$

$\boxed{R = 0.265}$

Finally, gust effect factor $G_f = 0.925\left(\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right)$

N-S $G_f = 0.925\left[\frac{1 + 1.7(0.247)\sqrt{(3.4)^2(0.79)^2 + (4.052)^2(0.205)^2}}{1 + 1.7(3.4)(0.247)}\right]$

$\boxed{G_f = 0.831}$

E-W $G_f = 0.925\left(\frac{1 + 1.7(0.247)\sqrt{(3.4)^2(0.818)^2 + (4.052)^2(0.265)^2}}{1 + 1.7(3.4)(0.247)}\right]$

$\boxed{G_f = 0.858}$

Enclosure classification (6.5.9) – Enclosed (definition in 6.2)

Internal pressure coeff. GC_{pi} (Fig. 6-5)

$\boxed{= \pm 0.18}$

External pressure coeff. C_p (Fig. 6-6)

Windward wall $\boxed{C_p = 0.8}$ use with q_z

Leeward wall N-S $\frac{L}{B} = \frac{145.25}{260.67} = 0.557$

$\Rightarrow \boxed{C_p = -0.5}$ use w/ q_h

Leeward wall E-W $\frac{L}{B} = \frac{260.67}{145.25} = 1.795 \Rightarrow \boxed{C_p = -0.341}$

from interpolation
use w/ q_h

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Velocity pressure (section 6.5.10)

(Eqn 6-15) q_z evaluated @ z $= 0.00256\ K_z K_{zt} K_d V^2 I$ $\left(\frac{lb}{ft^2}\right)$

q_h = velocity pressure using Eqn 6-15 @ mean roof height h

already found

K_z = veloc. press. exposure coeff, VARIES

$K_{zt} = 1.0$

$K_d = 0.85$

$V = 90$ mph

$I = 1.15$

Design wind pressures: (for Flexible Bldg)

(Eqn 6-19) $p = q G_f C_p - q_i (GC_{pi})$ answer in psf

windward: $q = q_z$ @ each level

$q_i = q_h$

Leeward: $q = q_h$; $q_i = q_h$

See spreadsheet for K_z, q_z, pressures for each level

Final Report

Summary of Analysis 1:

Gust Effect Factor			
	N-S	E-W	ASCE 7-05 Reference
B	260'-8"	145'-3"	(Sec. 6.3)
L	145'-3"	260'-3"	(Sec. 6.3)
h	176'-5"		(Sec. 6.3)
n_1	0.567		(Eq. C6-17)
Structure	Flexible		(Sec. 6.2)
g_r	4.052		(Eq. 6-9)
$\bar{z}$	105.85		(Table 6-2)
$\bar{V}_{\bar{z}}$	79.49		(Eq. 6-14)
$I_{\bar{z}}$	0.247		(Eq. 6-5)
$L_{\bar{z}}$	471.93		(Eq. 6-7)
Q	0.790	0.818	(Eq. 6-6)
R_h	0.158		(Eq. 6-13a)
η=	5.789		
R_B	0.110	0.188	(Eq. 6-13a)
η=	8.553	4.766	
R_L	0.061	0.034	(Eq. 6-13a)
η=	15.955	28.634	
N_1	3.366		(Eq. 6-12)
R_n	0.065		(Eq. 6-11)
β	1.50%		(Sec. C6.5.8)
R	0.205	0.265	(Eq. 6-10)
G_f	0.831	0.858	(Eq. 6-8)

Table 22 – Wind Analysis 1 Gust Effect Factor and Corresponding Variables

External Pressure Coefficient C_p			
	N-S	E-W	ASCE 7-05 Reference
Windward Wall	0.8	0.8	(Fig. 6-6)
Leeward Wall	-0.5	-0.341	(Fig. 6-6)

Table 23 – Wind Analysis 1 External Pressure Coefficients

	Level	Elevation	Floor-to-Floor Height (ft)	Height Above Ground (ft)	K_z	q_z	Wind Pressure (psf)					
							N-S			E-W		
							+ 0.18	- 0.18	Net	+ 0.18	- 0.18	Net
Windward	1	172'-0"	16.00	0	-	-	-	-	-	-	-	-
	2	188'-0"	18.83	16.00	0.58	11.76	3.57	12.06	7.82	3.83	12.31	8.07
	3	206'-10"	14.25	34.83	0.73	14.78	5.58	14.07	9.82	5.90	14.38	10.14
	4	221'-1"	14.25	49.08	0.81	16.33	6.61	15.10	10.85	6.96	15.45	11.21
	5	235'-4"	14.25	63.33	0.86	17.50	7.39	15.88	11.63	7.77	16.25	12.01
	6	249'-7"	14.25	77.58	0.92	18.65	8.16	16.64	12.40	8.56	17.05	12.80
	7	263'-10"	13.50	91.83	0.97	19.57	8.77	17.25	13.01	9.19	17.68	13.43
	8	277'-4"	13.50	105.33	1.00	20.34	9.28	17.76	13.52	9.72	18.20	13.96
	9	290'-10"	13.50	118.83	1.04	21.02	9.73	18.22	13.97	10.19	18.67	14.43
	10	304'-4"	14.08	132.33	1.07	21.71	10.19	18.67	14.43	10.66	19.14	14.90
	PH	318'-5"	13.42	146.42	1.10	22.35	10.62	19.10	14.86	11.10	19.59	15.34
	PF Mezz.	331'-10"	16.58	159.83	1.13	22.90	10.98	19.46	15.22	11.47	19.96	15.72
	Roof	348'-5"	-	176.42	1.16	23.57	11.43	19.91	15.67	11.94	20.42	16.18
Leeward	All	348'-5"	-	176.42	1.16	23.57	-14.04	-5.55	-9.79	-11.14	-2.65	-6.90

Table 24 – Wind Analysis 1 Pressures

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Analysis 2:

Do second wind analysis with different B & L because levels 7 thru roof are smaller.

	N-S	E-W
B	≈ 228'	≈ 118'
L	≈ 118'	≈ 228'
h	176.42'	176.42'

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} \quad \text{(Eqn 6-6)}$$

N-S $$Q = \sqrt{\frac{1}{1+0.63\left(\frac{228+176.42}{471.93}\right)^{0.63}}} = \boxed{0.798}$$

E-W $$Q = \sqrt{\frac{1}{1+0.63\left(\frac{118+176.42}{471.93}\right)^{0.63}}} = \boxed{0.825}$$

R_B : $$\eta = \frac{4.6 n_1 B}{\bar{V}_{\bar{z}}} \; ; \; R_B = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1-e^{-2\eta}\right) \quad \text{(Eqn. 6-13a)}$$

N-S $$\eta = \frac{4.6(0.567)(228)}{79.49} = \boxed{7.481}$$

$$R_B = \frac{1}{7.481} - \frac{1}{2(7.481)^2}\left(1-e^{-2(7.481)}\right) = \boxed{0.125}$$

E-W $$\eta = \frac{4.6(0.567)(118)}{79.49} = \boxed{3.872}$$

$$R_B = \frac{1}{3.872} - \frac{1}{2(3.872)^2}\left(1-e^{-2(3.872)}\right) = \boxed{0.225}$$

R_L : $\eta = \dfrac{15.4\, n_1 L}{\bar{V}_{\bar{z}}}$; R_L = (Eqn 6-13a)

N-S $\eta = \dfrac{15.4(0.567)(118)}{79.49} = \boxed{12.962}$

$R_L = \dfrac{1}{12.962} - \dfrac{1}{2(12.962)^2}\left(1 - e^{-2(12.962)}\right) = \boxed{0.074}$

E-W $\eta = \dfrac{15.4(0.567)(228)}{79.49} = \boxed{25.045}$

$R_L = \dfrac{1}{25.045} - \dfrac{1}{2(25.045)^2}\left(1 - e^{-2(25.045)}\right) = \boxed{0.039}$

External Pressure Coeff. C_p (Fig. 6-6)

Leeward wall N-S $\dfrac{L}{B} = \dfrac{118}{228} = 0.518 \Rightarrow \boxed{C_p = -0.5}$

Leeward wall E-W $\dfrac{L}{B} = \dfrac{228}{118} = 1.932 \Rightarrow \boxed{C_p = -0.314}$

$R = \sqrt{\dfrac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$ (Eqn 6-10)

N-S $R = \sqrt{\dfrac{1}{0.015}(0.065)(0.158)(0.125)(0.53 + 0.47 \times 0.074)} = \boxed{0.220}$

E-W $R = \sqrt{\dfrac{1}{0.015}(0.065)(0.158)(0.225)(0.53 + 0.47 \times 0.039)} = \boxed{0.291}$

$G_f = 0.925\left(\dfrac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right)$ (Eqn 6-8)

N-S $G_f = 0.925\left(\dfrac{1 + 1.7(0.247)\sqrt{(3.4)^2(0.798)^2 + (4.052)^2(0.220)^2}}{1 + 1.7(3.4)(0.247)}\right)$

$\boxed{G_f = 0.838}$

E-W $G_f = 0.925\left(\dfrac{1 + 1.7(0.247)\sqrt{(3.4)^2(0.825)^2 + (4.052)^2(0.291)^2}}{1 + 1.7(3.4)(0.247)}\right)$

$\boxed{G_f = 0.868}$

All other variables remain unchanged from the first analysis.
See spreadsheet for new pressures.

Summary of Analysis 2:

Gust Effect Factor			
	N-S	E-W	ASCE 7-05 Reference
B	260'-8"	145'-3"	(Sec. 6.3)
L	145'-3"	260'-3"	(Sec. 6.3)
h	176'-5"		(Sec. 6.3)
n_1	0.567		(Eq. C6-17)
Structure	Flexible		(Sec. 6.2)
g_r	4.052		(Eq. 6-9)
$\bar{z}$	105.85		(Table 6-2)
$\bar{V}_{\bar{z}}$	79.49		(Eq. 6-14)
$I_{\bar{z}}$	0.247		(Eq. 6-5)
$L_{\bar{z}}$	471.93		(Eq. 6-7)
Q	0.798	0.825	(Eq. 6-6)
R_h	0.158		(Eq. 6-13a)
η=	5.789		
R_B	0.125	0.225	(Eq. 6-13a)
η=	7.481	3.872	
R_L	0.074	0.039	(Eq. 6-13a)
η=	12.962	25.045	
N_1	3.366		(Eq. 6-12)
R_n	0.065		(Eq. 6-11)
β	1.50%		(Sec. C6.5.8)
R	0.22	0.291	(Eq. 6-10)
G_f	0.838	0.868	(Eq. 6-8)

Table 25 – Wind Analysis 2 Gust Effect Factor and Corresponding Variables

External Pressure Coefficient C_p			
	N-S	E-W	ASCE 7-05 Reference
Windward Wall	0.8	0.8	(Fig. 6-6)
Leeward Wall	-0.5	-0.314	(Fig. 6-6)

Table 26 – Wind Analysis 2 External Pressure Coefficients

	Level	Elevation	Floor-to-Floor Height (ft)	Height Above Ground (ft)	K_z	q_z	Wind Pressure (psf)					
							N-S			E-W		
							+ 0.18	- 0.18	Net	+ 0.18	- 0.18	Net
Windward	1	172'-0"	16.00	0	-	-	-	-	-	-	-	-
	2	188'-0"	18.83	16.00	0.58	11.76	3.64	12.12	7.88	3.92	12.41	8.16
	3	206'-10"	14.25	34.83	0.73	14.78	5.66	14.15	9.91	6.02	14.50	10.26
	4	221'-1"	14.25	49.08	0.81	16.33	6.70	15.19	10.94	7.09	15.58	11.34
	5	235'-4"	14.25	63.33	0.86	17.50	7.49	15.97	11.73	7.91	16.39	12.15
	6	249'-7"	14.25	77.58	0.92	18.65	8.26	16.75	12.51	8.71	17.20	12.95
	7	263'-10"	13.50	91.83	0.97	19.57	8.88	17.36	13.12	9.35	17.83	13.59
	8	277'-4"	13.50	105.33	1.00	20.34	9.39	17.88	13.63	9.88	18.36	14.12
	9	290'-10"	13.50	118.83	1.04	21.02	9.85	18.34	14.09	10.35	18.84	14.60
	10	304'-4"	14.08	132.33	1.07	21.71	10.31	18.79	14.55	10.83	19.31	15.07
	PH	318'-5"	13.42	146.42	1.10	22.35	10.74	19.23	14.99	11.28	19.77	15.52
	PH Mezz.	331'-10"	16.58	159.83	1.13	22.90	11.11	19.59	15.35	11.66	20.14	15.90
	Roof	348'-5"	-	176.42	1.16	23.57	11.56	20.04	15.80	12.12	20.61	16.37
Leeward	All	348'-5"	-	176.42	1.16	23.57	-14.12	-5.63	-9.88	-10.67	-2.18	-6.42

Table 27 – Wind Analysis 2 Pressures

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Combined Wind Analyses Results:

Level	Floor-to-Floor Height (ft)	Height Above Ground (ft)	Controlling Windward Pressure (psf)		Controlling Leeward Pressure (psf)		Total Controlling Pressure (psf)		Wind Forces					
									Load (kips)		Shear (kips)		Moment (ft-kips)	
			N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W
1	16.00	0	-	-	-	-	-	-	0.0	0.0	866.1	408.0	0	0
2	18.83	16.00	7.88	8.16	-9.88	-6.90	17.76	15.06	85.6	40.5	866.1	408.0	1370	648
3	14.25	34.83	9.91	10.26	-9.88	-6.90	19.79	17.16	87.2	42.8	778.8	367.5	3039	1492
4	14.25	49.08	10.94	11.34	-9.88	-6.90	20.82	18.24	78.8	39.0	691.6	324.7	3868	1916
5	14.25	63.33	11.73	12.15	-9.88	-6.90	21.61	19.05	81.7	40.7	612.8	285.6	5176	2579
6	14.25	77.58	12.51	12.95	-9.88	-6.90	22.39	19.85	84.3	42.2	531.1	244.9	6540	3276
7	13.50	91.83	13.12	13.59	-9.88	-6.90	23.00	20.49	73.7	34.1	446.8	236.8	6764	3131
8	13.50	105.33	13.63	14.12	-9.88	-6.90	23.51	21.02	73.8	34.0	373.1	202.7	7776	3580
9	13.50	118.83	14.09	14.60	-9.88	-6.90	23.97	21.50	75.6	34.7	299.3	168.7	8981	4129
10	14.08	132.33	14.55	15.07	-9.88	-6.90	24.43	21.97	73.8	36.3	223.7	134.0	9764	4798
PH	13.42	146.42	14.99	15.52	-9.88	-6.90	24.87	22.42	51.0	35.7	149.9	97.7	7463	5234
PH Mezz.	16.58	159.83	15.35	15.90	-9.88	-6.90	25.23	22.80	56.6	39.8	99.0	62.0	9041	6358
Roof	-	176.42	15.80	16.37	-9.88	-6.90	25.68	23.27	42.4	22.2	42.4	22.2	7482	3914

Table 28 – Design Wind Pressures and Forces

Seismic

Braced Frames / Dual System:

Seismic Analysis - Equivalent Lateral Force Procedure

Occupancy Category : III (Table 1-1)

S_s = 23% g for Richmond, VA (Fig. 22-1)

S_1 = 6% g (Fig. 22-2)

Site Class : C (Table 20.3-1) from geotech. report

F_a = 1.2 (Table 11.4-1)

F_v = 1.7 (Table 11.4-2)

$S_{MS} = F_a S_s = 1.2(0.23) = 0.276$ (Eqn. 11.4-1)

$S_{M1} = F_v S_1 = 1.7(0.06) = 0.102$ (Eqn. 11.4-2)

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.276) = 0.184$ (Eqn. 11.4-3)

$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.102) = 0.068$ (Eqn. 11.4-4)

Importance Factor I = 1.25 (Table 11.5-1)

Seismic Design Category : (Table 11.6-1) ⇒ B
(Table 11.6-2) ⇒ B

∴ Equivalent Lateral Force Procedure permitted by (Table 12.6-1)

T_L = 8 (Fig. 22-15)

for "All other structural systems" (Table 12.8-2)
i.e. braced frames in this case
C_t = 0.02
x = 0.75

$T_a = C_t h_n^x$ (Eqn 12.8-7) where h_n = height = 176.42'

$T_a = 0.02(176.42)^{0.75} = 0.968$ sec.
↖ approximate fundamental period

$T = T_a = 0.968$ per Section 12.8.2

Check $T \leq C_u T_a$ where $C_u = 1.7$ (Table 12.8-1)

so use $T = (1.7)(0.968) = 1.645$

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$R = 3$ (Table 12.2-1) "Steel Systems not Specifically Detailed for Seismic Resistance"

for $T \le T_L$ $\quad C_s \le \dfrac{S_{D1}}{T\left(\frac{R}{I}\right)}$ (Eqn. 12.8-3)

$1.645 < 8$

$C_s = \dfrac{S_{DS}}{\left(\frac{R}{I}\right)}$ (Eqn. 12.8-2)

$C_s \ge 0.01$ (Eqn. 12.8-5)

$C_s = \dfrac{0.184}{\left(\frac{3}{1.25}\right)} = 0.077 > \dfrac{0.068}{1.645\left(\frac{3}{1.25}\right)} = 0.0172$

↑ controls

also $0.0172 > 0.01$ so okay

Seismic Base Shear $V = C_s W$ (Eqn. 12.8-1)

W = effective seismic weight per Sec. 12.7.2

W = DL + 25% LL + min 10 psf partitions in offices + 20% snow load where $P_f > 30$ psf

* Partitions were included in DL
$P_f = 22$ psf < 30 psf
Storage areas are negligible

$\therefore W = DL = 35514$ k see the Gravity Analysis

$V = 0.0172(35514) = 611$ k

$k = 1$ for $T = 0.5$ and 2 for $T = 2.5$

for $T = 0.968$ interpolate to get $k = 1.234$

Level	Weight w_x (kips)	Height h_x (ft)	$w_x h_x^k$	C_{vx}	Lateral Force Fx (kips)	Story Shear Vx (kips)	Moment Mx (ft-k)
Roof	2290	176.42	1355662	0.148	90.6	0.0	15980
PH Mezzanine	857	159.83	449181	0.049	30.0	90.6	4797
Penthouse	2371	146.42	1114925	0.122	74.5	120.6	10908
10	2833	132.33	1175711	0.129	78.6	195.1	10396
9	2859	118.83	1039241	0.114	69.4	273.6	8251
8	2858	105.33	895035	0.098	59.8	343.1	6299
7	3544	91.83	937150	0.102	62.6	402.9	5750
6	3502	77.58	752075	0.082	50.3	465.5	3899
5	3510	63.33	586815	0.064	39.2	515.8	2483
4	3499	49.08	427150	0.047	28.5	555.0	1401
3	3754	34.83	300086	0.033	20.1	583.5	698
2	3637	16	111329	0.012	7.4	603.6	119
Total	35514	176.42	9144360	1.000	611.0	611.0	70981

Table 29 – Design Seismic Forces for Braced Frames/Dual System

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Moment Frames:

for "Steel moment-resisting frames" (Table 12.8-2)

$C_t = 0.028$
$x = 0.8$

$T_a = C_t h_n^x = 0.028\,(176.42)^{0.8} = 1.755$ sec.
(Eqn 12.8-7)

$T = T_a = 1.755$ per Section 12.8.2

Check $T \le C_u T_a$ where $C_u = 1.7$ (Table 12.8-1)

so use $T = 1.7\,(1.755) = 2.984$

$R = 3$ (Table 12.2-1) "Steel Systems not Specifically Detailed for Seismic Resistance"

for $T \le T_L$ $\quad C_s \le \dfrac{S_{D1}}{T\left(\frac{R}{I}\right)}$ (Eqn 12.8-3)
$2.984 < 8$

$C_s = \dfrac{S_{DS}}{\left(\frac{R}{I}\right)}$ (Eqn 12.8-2)

$C_s \ge 0.01$ (Eqn. 12.8-5)

$C_s = \dfrac{0.184}{\left(\frac{3}{1.25}\right)} = 0.077 > \dfrac{0.068}{2.984\left(\frac{3}{1.25}\right)} = 0.0095$
controls

but $0.0095 < 0.01$ so use $C_s = 0.01$

Seismic Base Shear $V = C_s W$ (Eqn. 12.8-1)

$W = 35514$ k see the Gravity Analysis

$V = 0.01\,(35514) = 355$ k

$k = 2$ for $T \ge 2.5$

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Level	Weight w_x (kips)	Height h_x (ft)	$w_x h_x^k$	C_{vx}	Lateral Force Fx (kips)	Story Shear Vx (kips)	Moment Mx (ft-k)
Roof	2290	176.42	946849337	0.246	87.3	0.0	15409
PH Mezzanine	857	159.83	276855555	0.072	25.5	87.3	4082
Penthouse	2371	146.42	615029468	0.160	56.7	112.9	8307
10	2833	132.33	570584530	0.148	52.6	169.6	6965
9	2859	118.83	440121452	0.114	40.6	222.2	4824
8	2858	105.33	325379934	0.085	30.0	262.8	3161
7	3544	91.83	286383387	0.074	26.4	292.9	2426
6	3502	77.58	185645586	0.048	17.1	319.3	1329
5	3510	63.33	112031140	0.029	10.3	336.4	654
4	3499	49.08	59056186	0.015	5.4	346.7	267
3	3754	34.83	26875622	0.007	2.5	352.2	86
2	3637	16	3724124	0.001	0.3	354.7	5
Total	35514	176.42	3848536321	1.000	355.0	355.0	47515

Table 30 – Design Seismic Forces for Moment Frames

Appendix C – Designs of Alternative Lateral Systems

Braced Frames

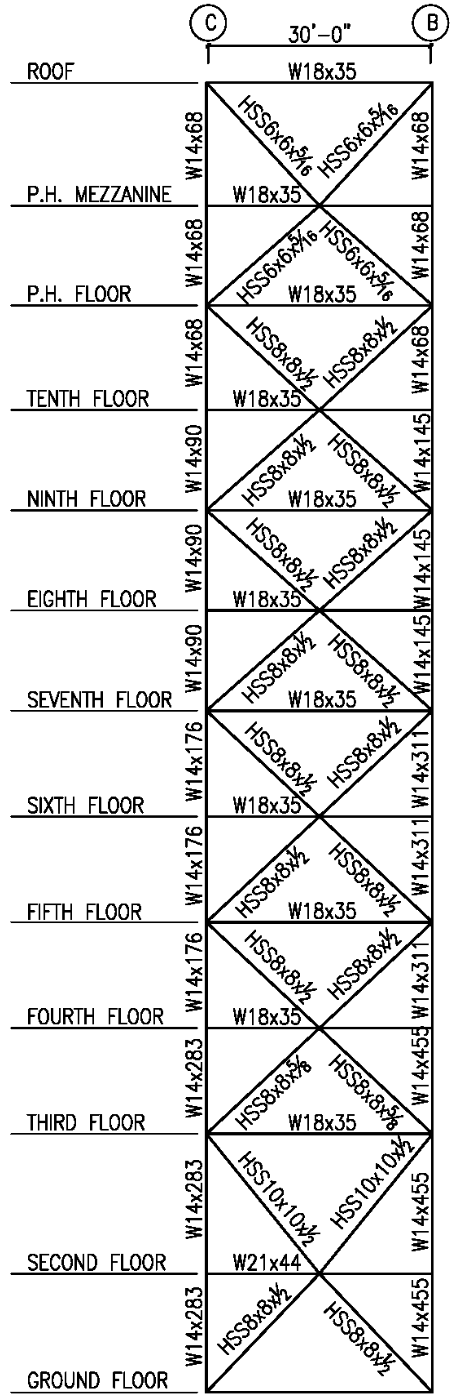

Figure 39 – BF-1 Braced Frame Elevation

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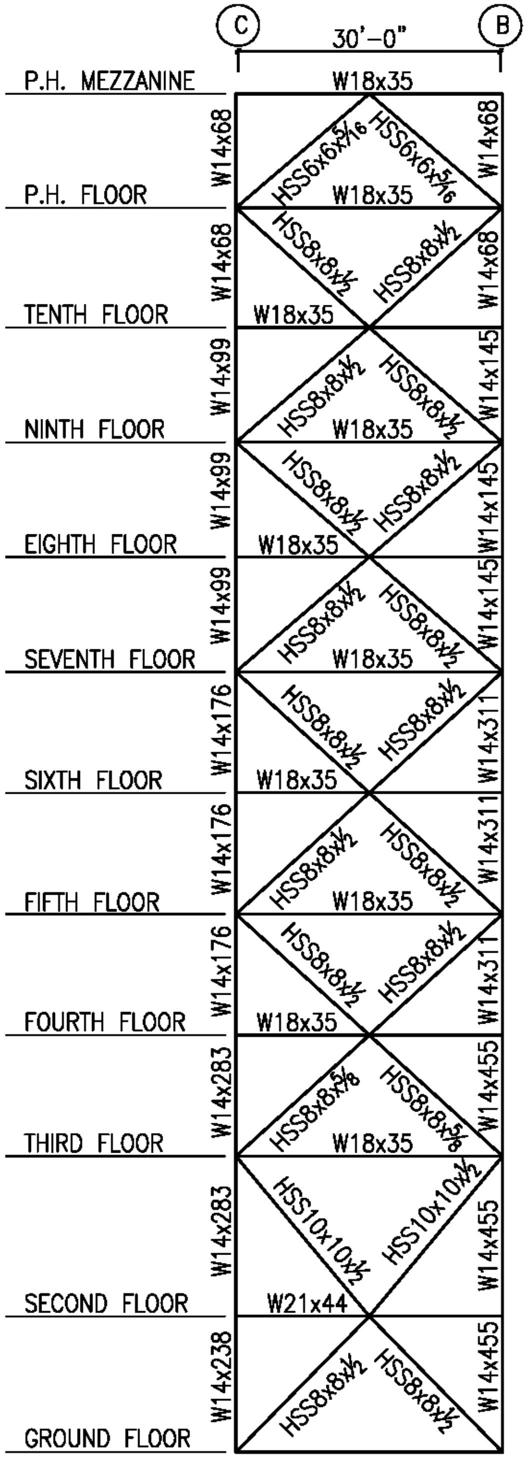

Figure 40 – BF-2 Braced Frame Elevation

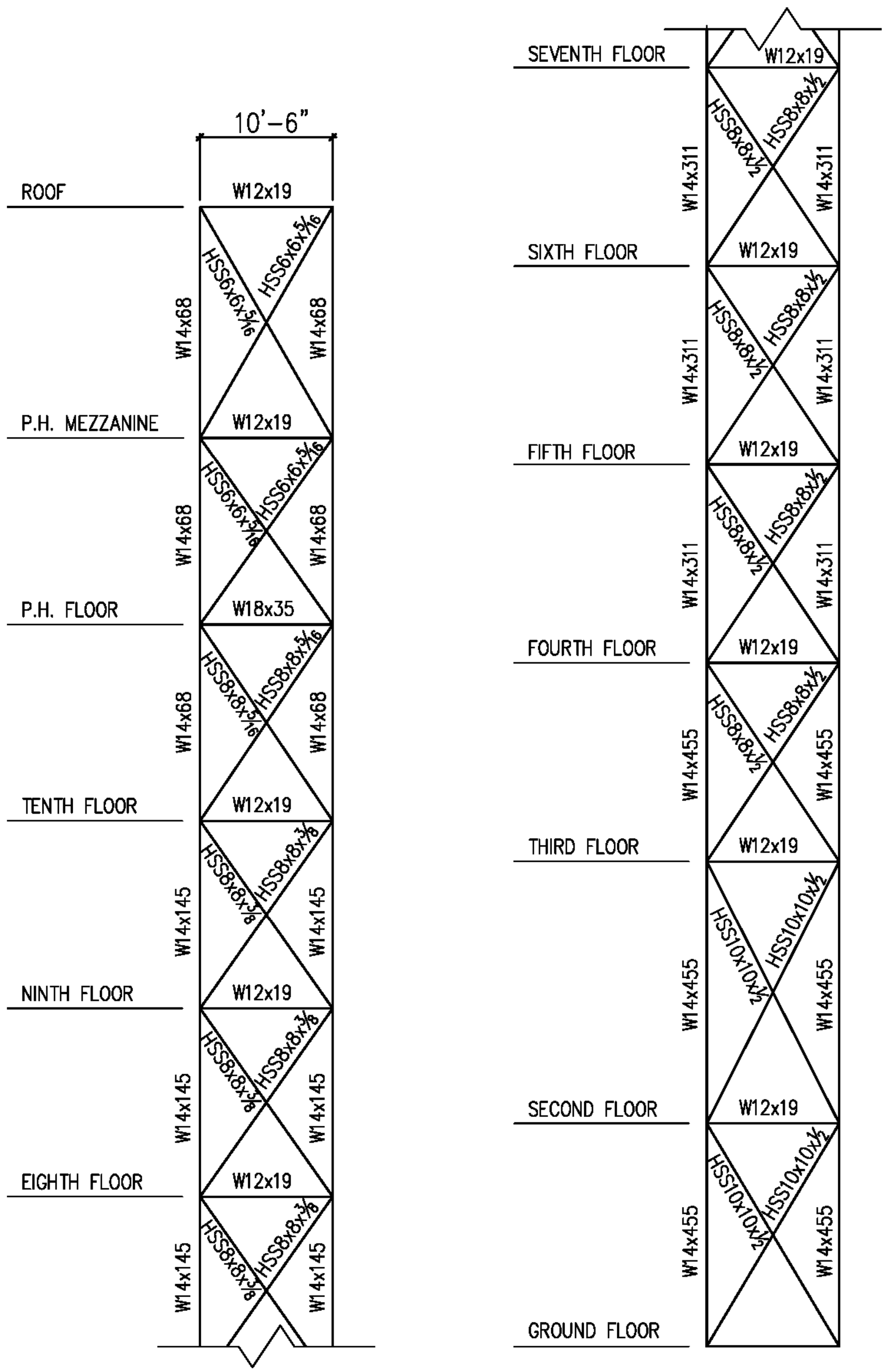

Figure 41 – BF-3 Braced Frame Elevation

Final Report

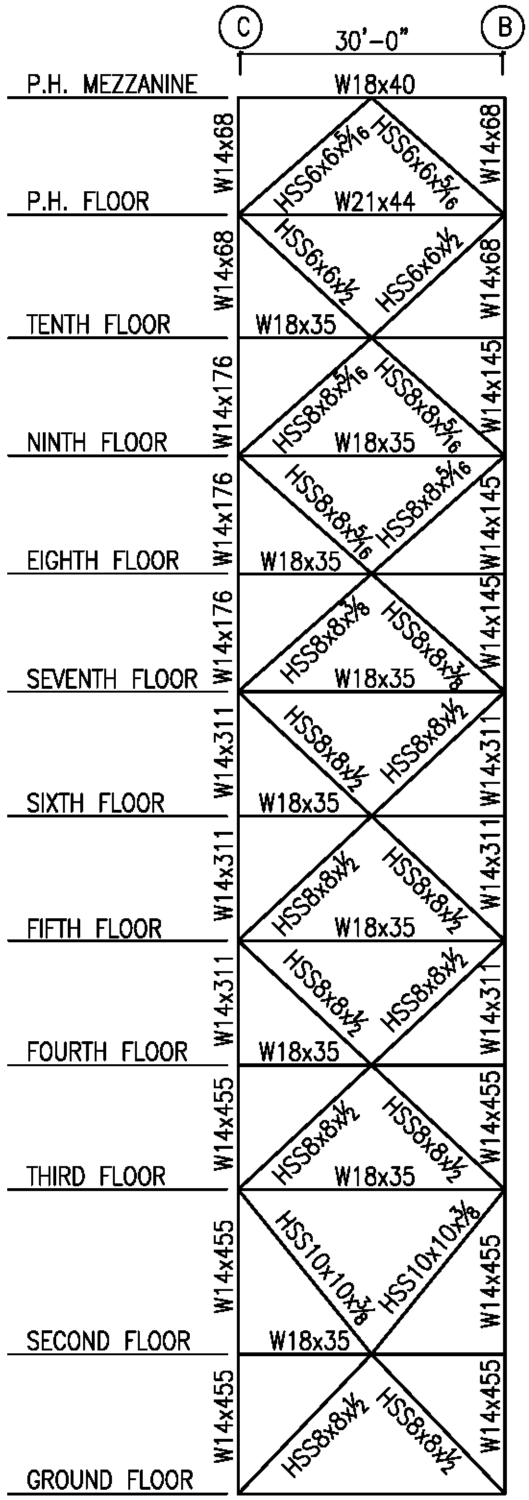

Figure 42 – BF-4 Braced Frame Elevation

Final Report

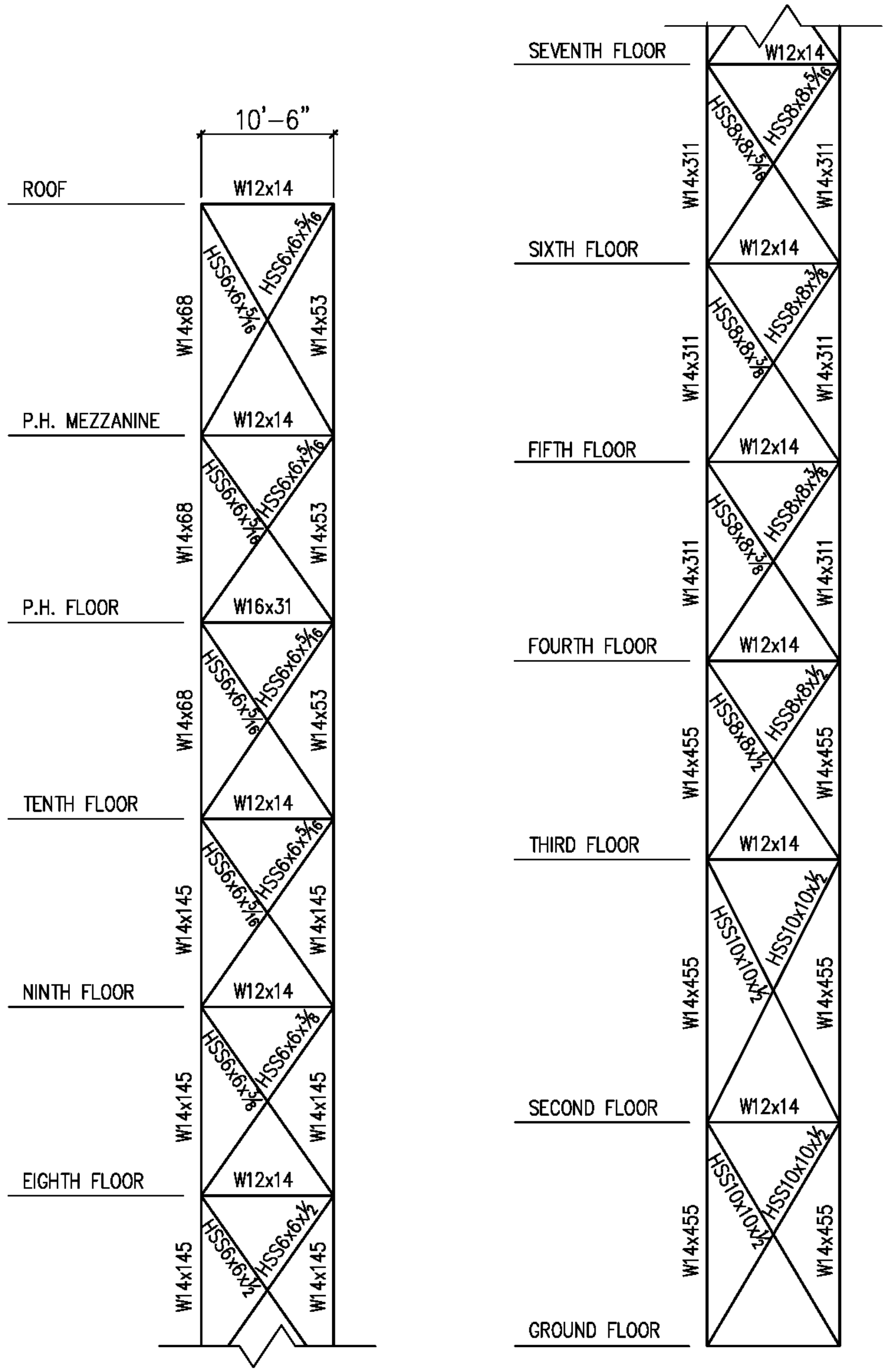

Figure 43 – BF-5 Braced Frame Elevation

Final Report

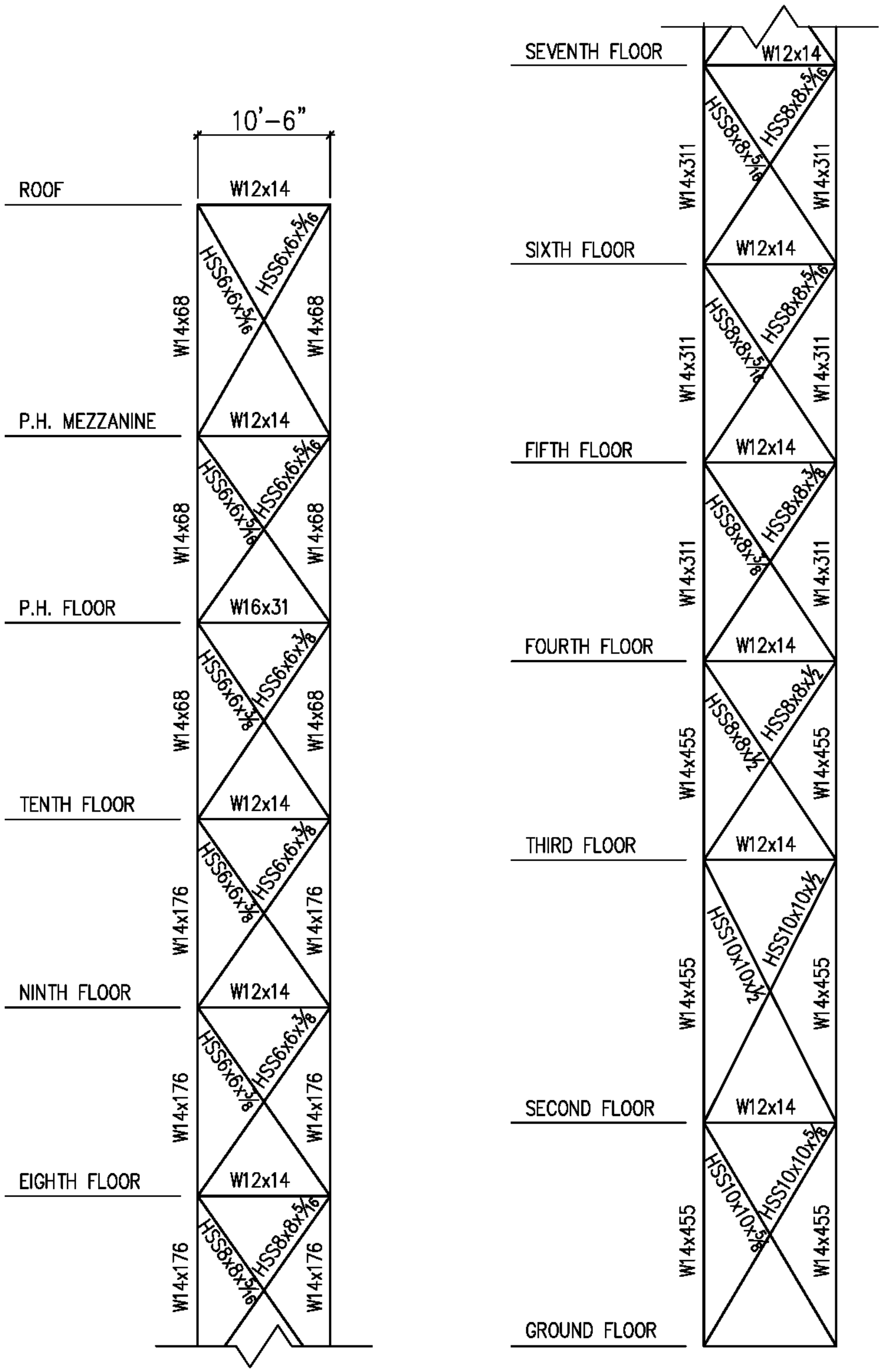

Figure 44 – BF-6 Braced Frame Elevation

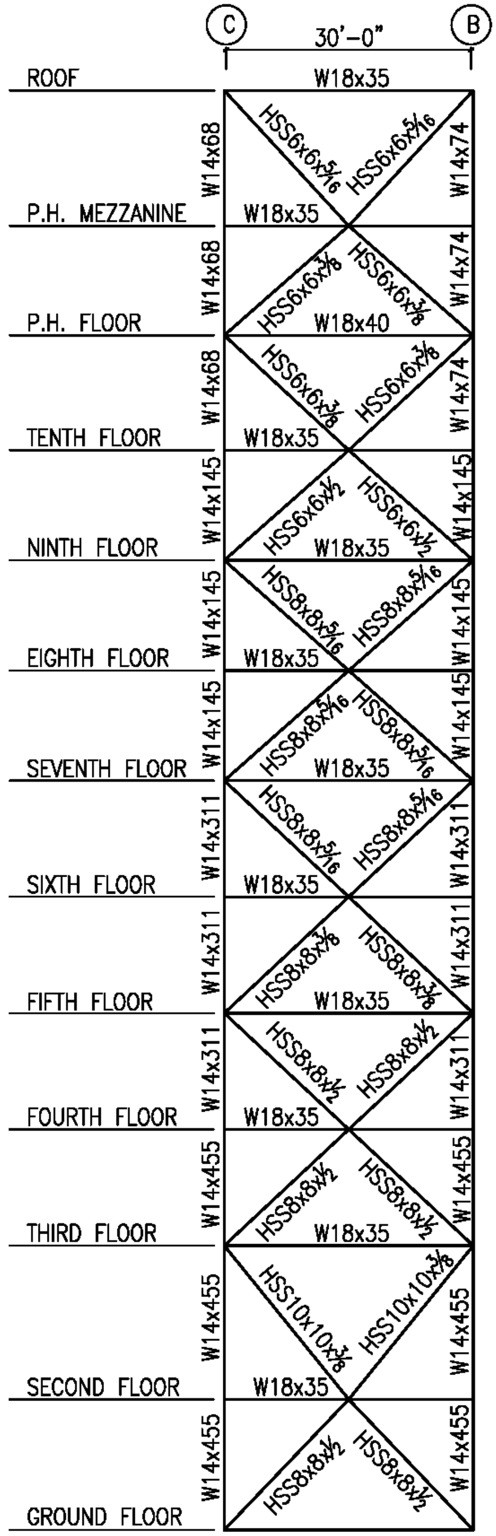

Figure 45 – BF-7 Braced Frame Elevation

Final Report

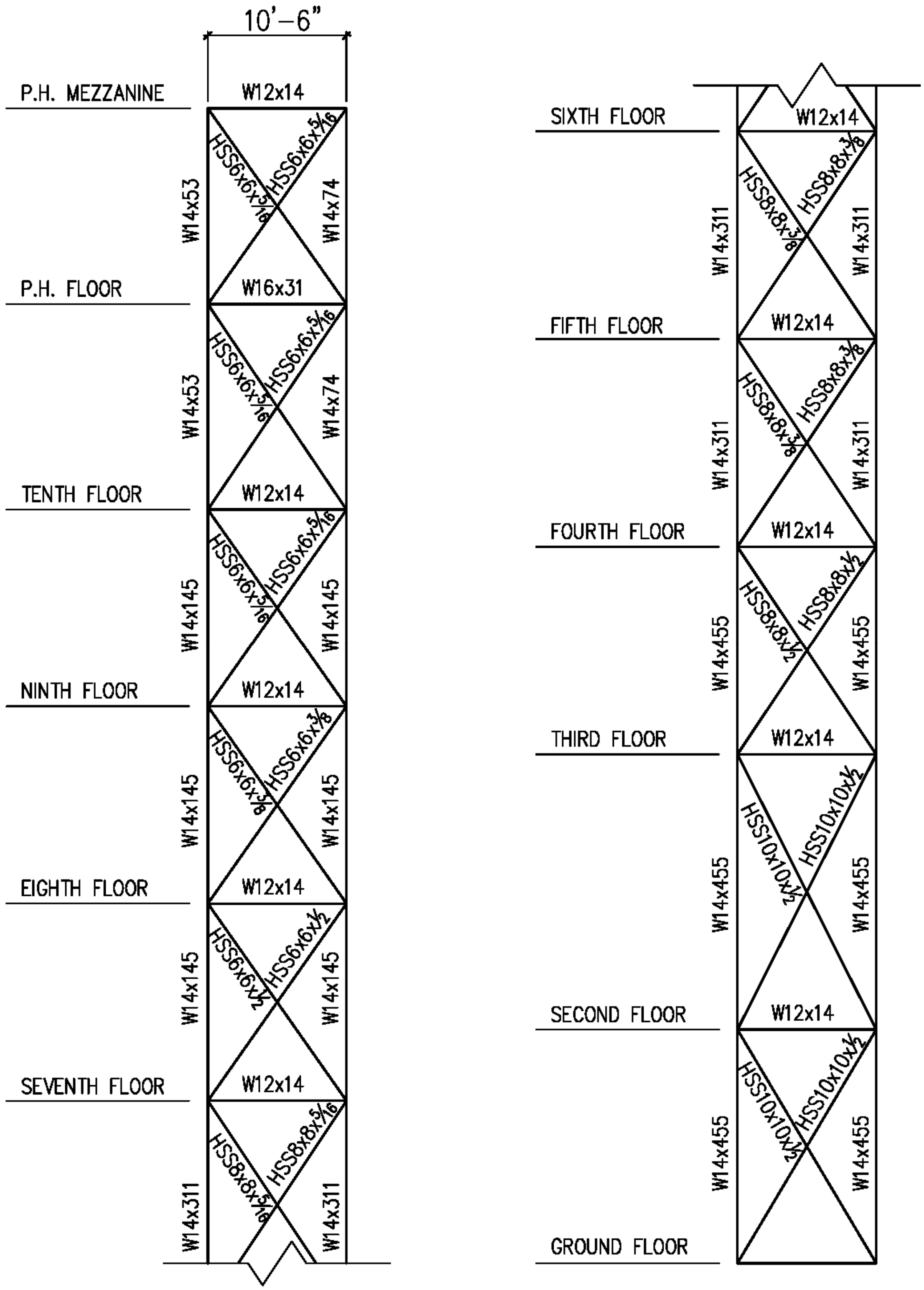

Figure 46 – BF-8 Braced Frame Elevation

Final Report

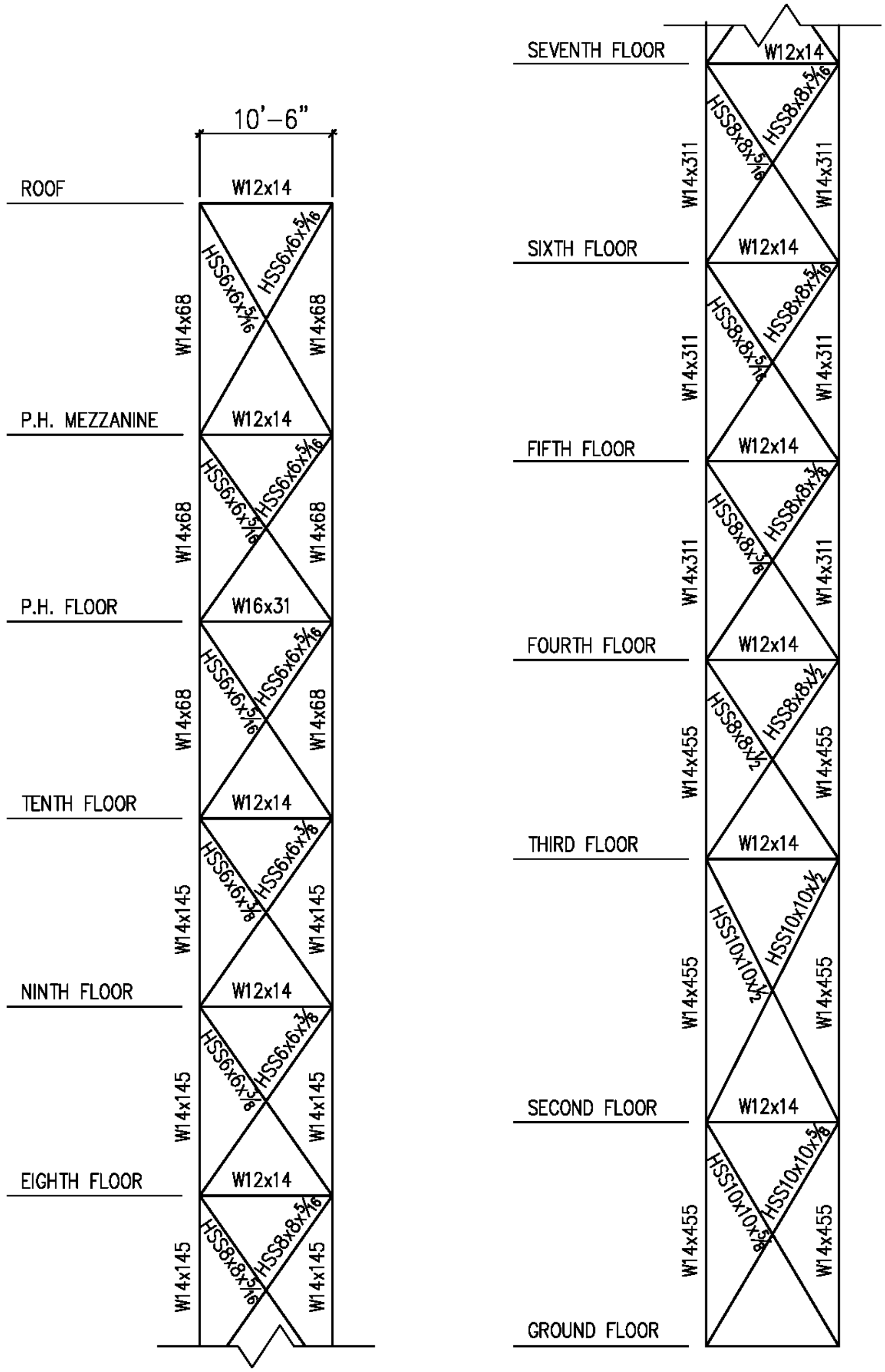

Figure 47 – BF-9 Braced Frame Elevation

Final Report

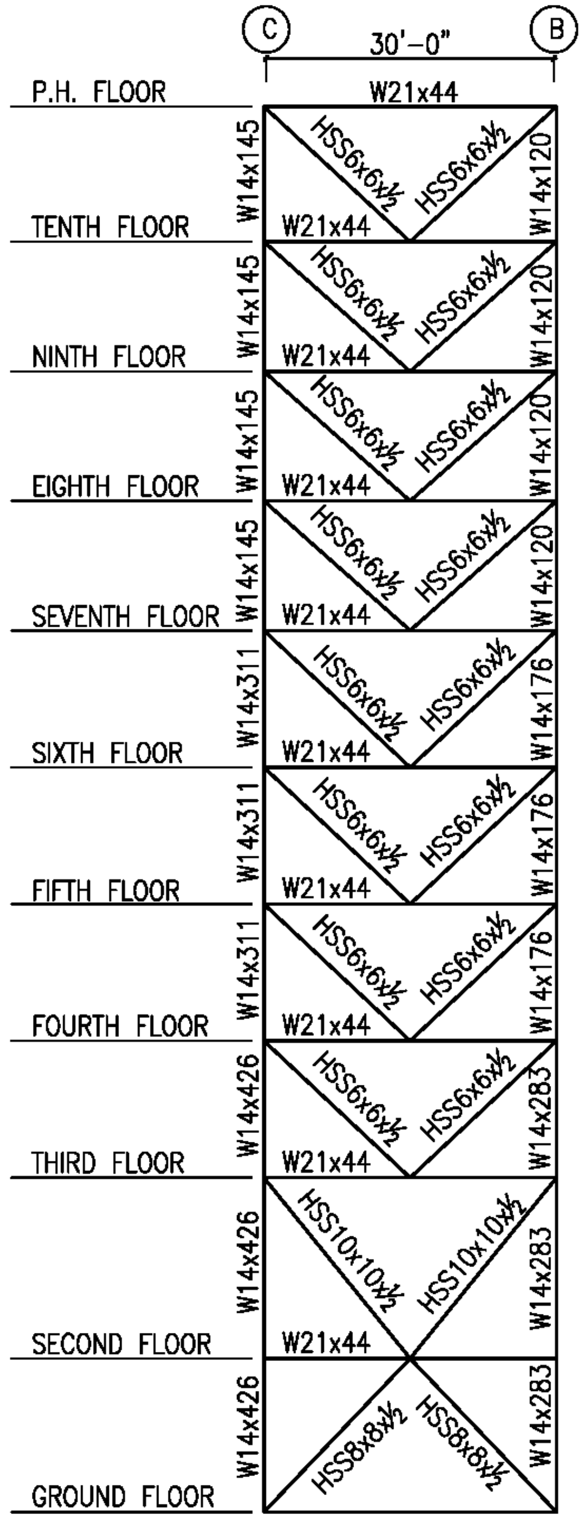

Figure 48 – BF-10 Braced Frame Elevation

Final Report

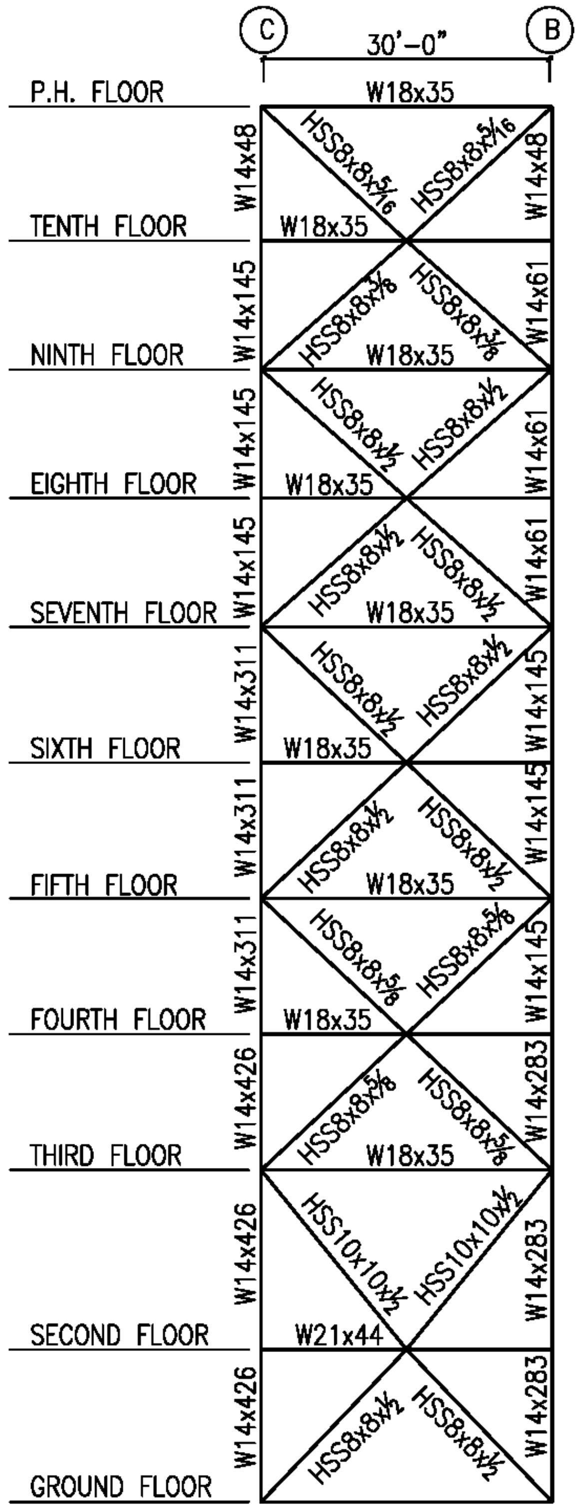

Figure 49 – BF-11 Braced Frame Elevation

Final Report

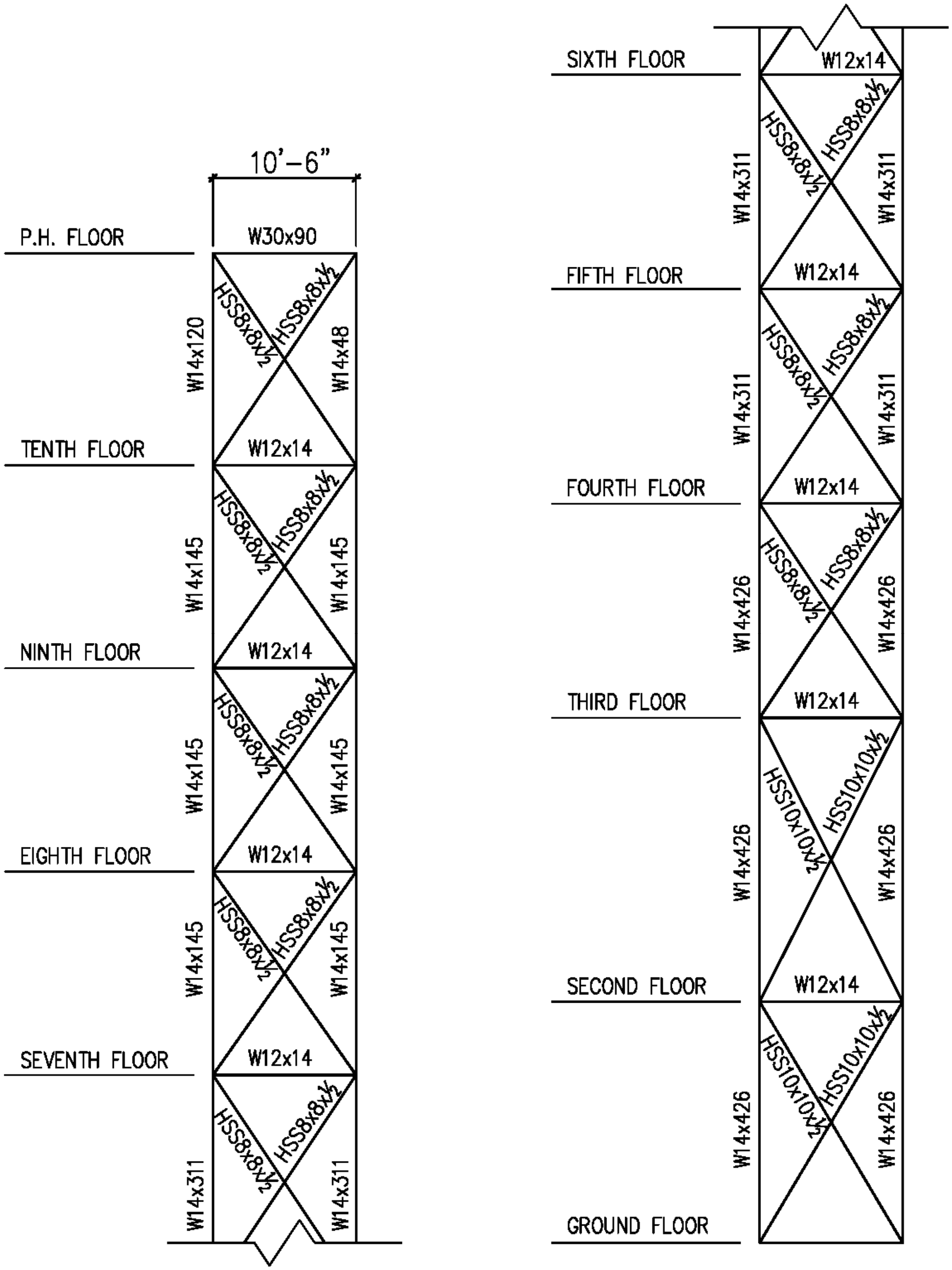

Figure 50 – BF-12 Braced Frame Elevation

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Dual System Option 1

(The braced frames are sized independently of the moment frames to carry the lateral load without regard to limiting drift. Then the moment frames are added to reduce drift.)

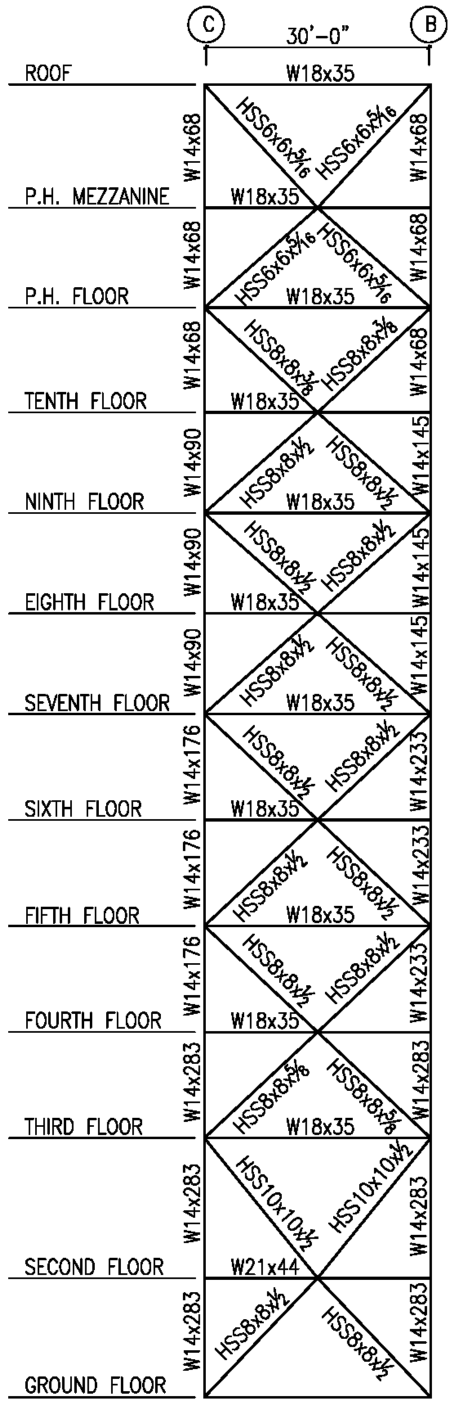

Figure 51 – BF-1 Braced Frame Elevation

Final Report

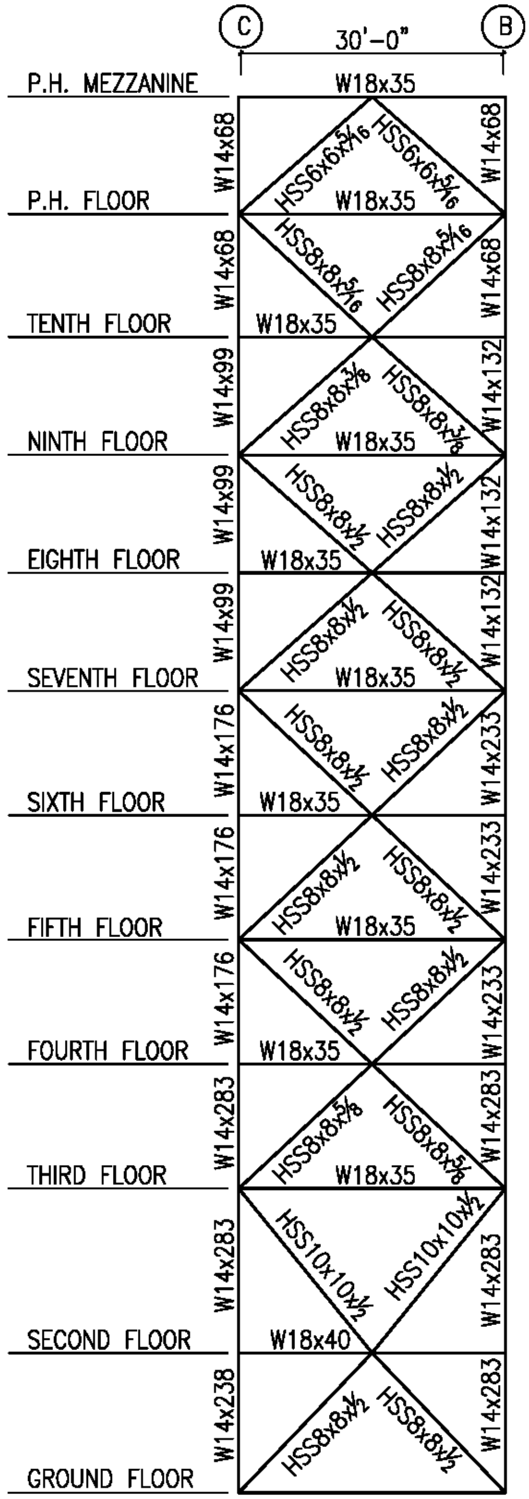

Figure 52 – BF-2 Braced Frame Elevation

Final Report

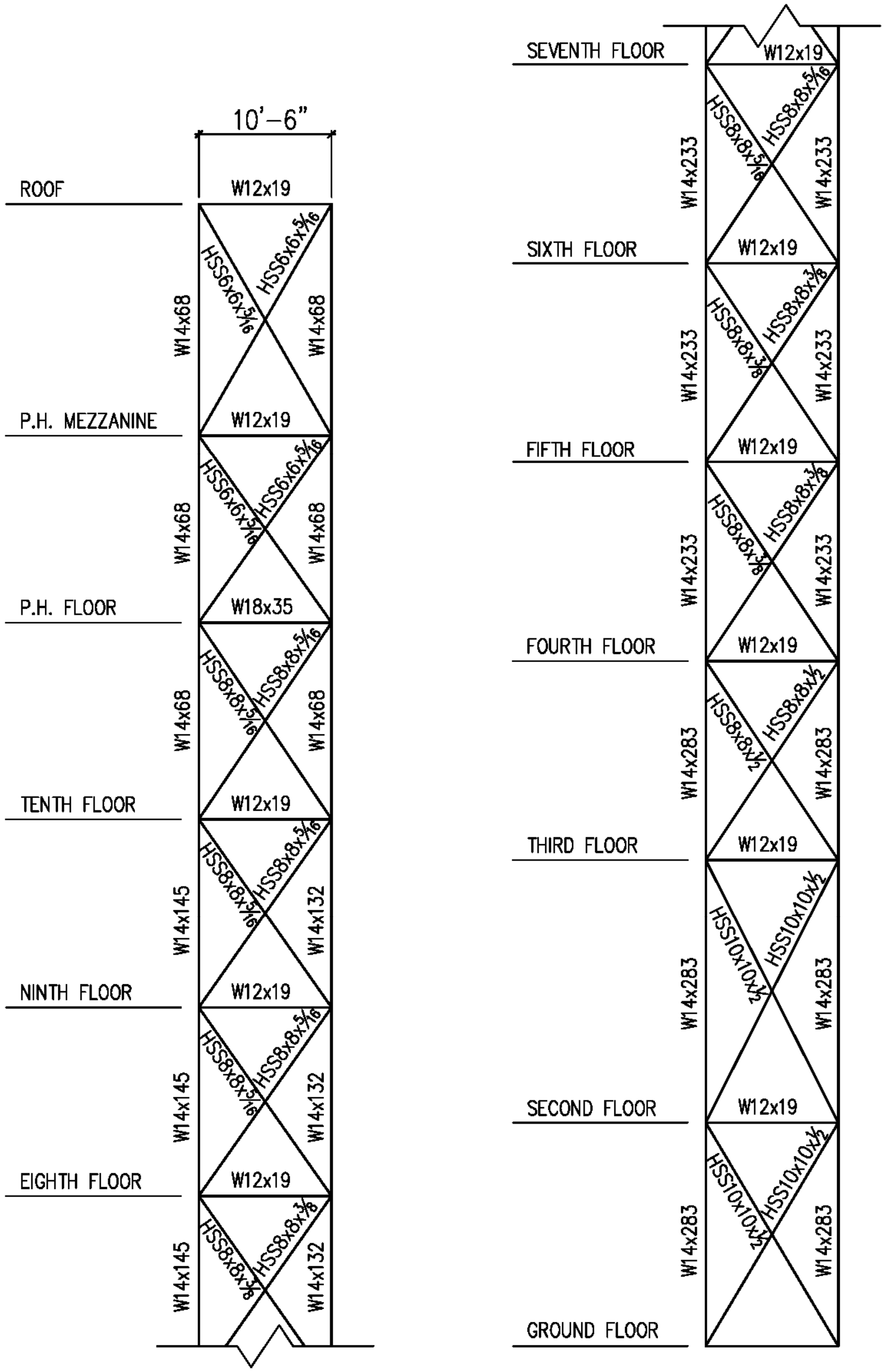

Figure 53 – BF-3 Braced Frame Elevation

Final Report

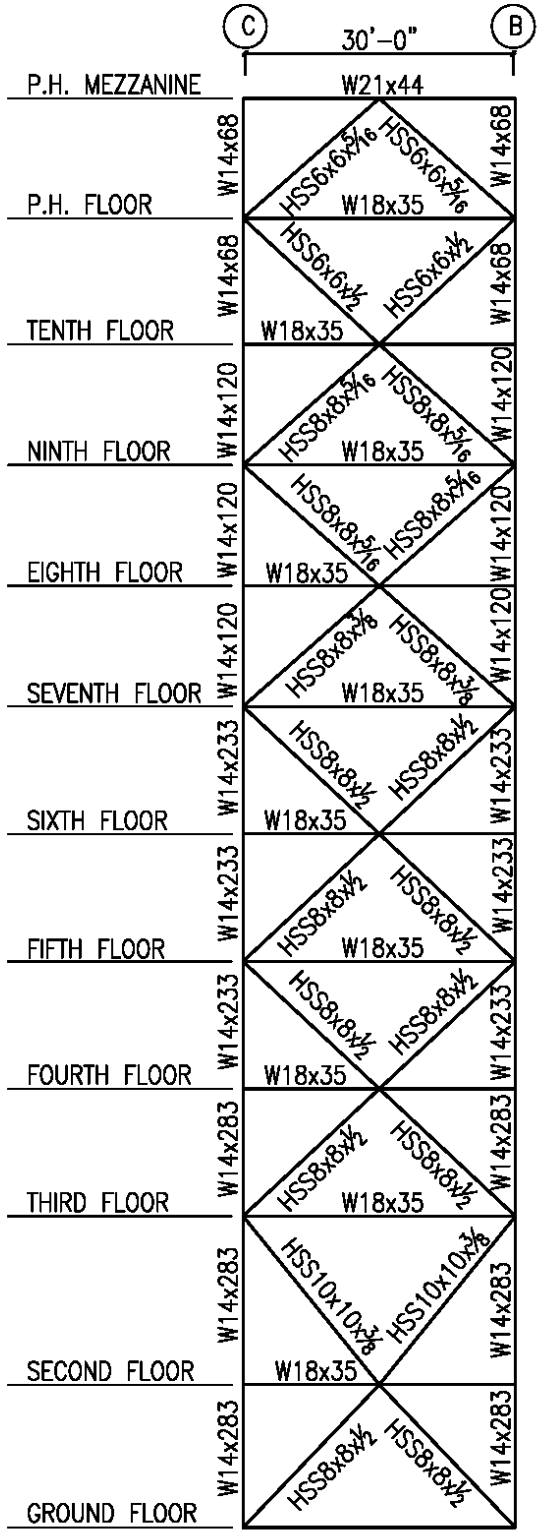

Figure 54 – BF-4 Braced Frame Elevation

Final Report

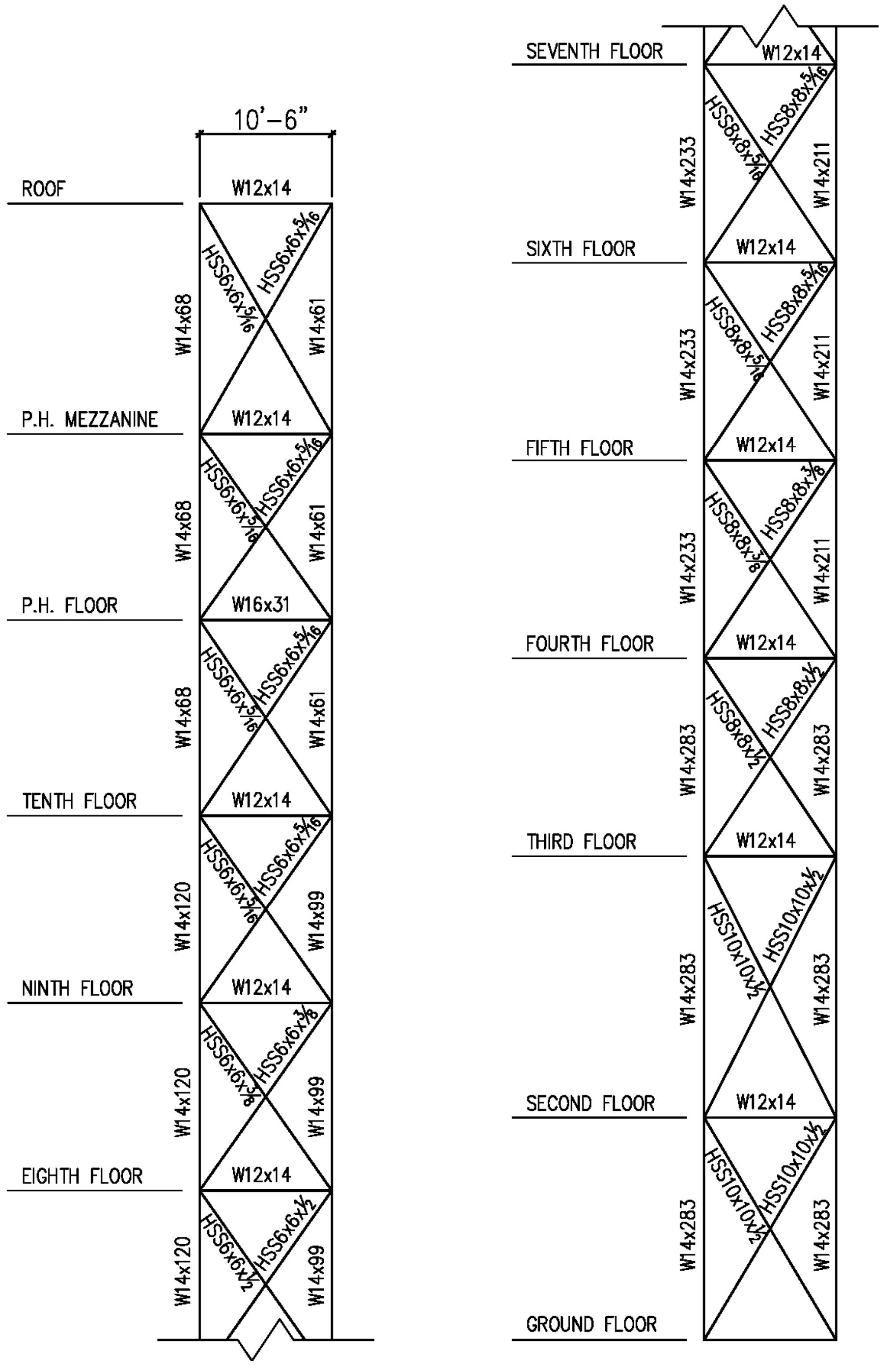

Figure 55 – BF-5 Braced Frame Elevation

Final Report

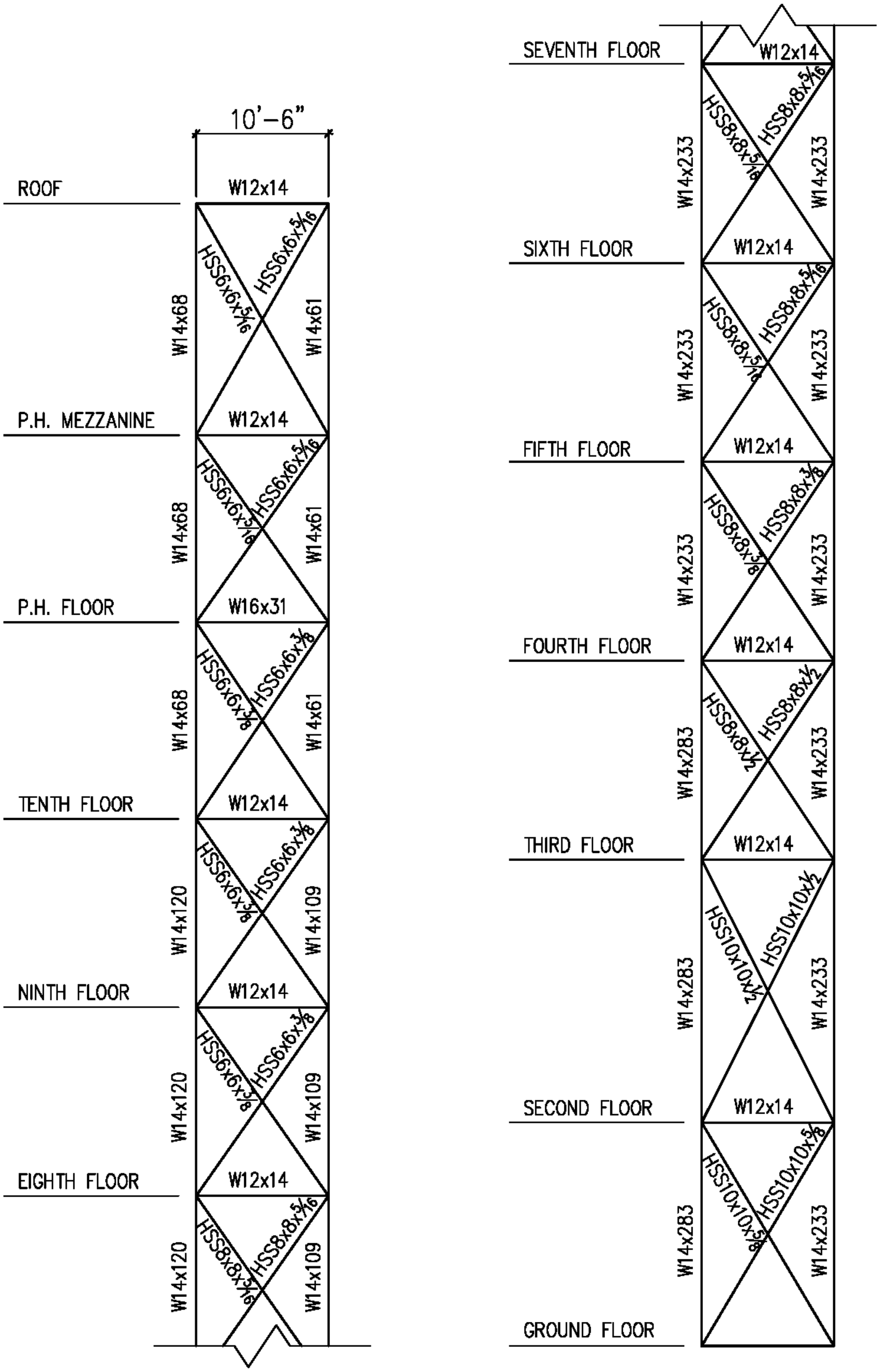

Figure 56 – BF-6 Braced Frame Elevation

Final Report

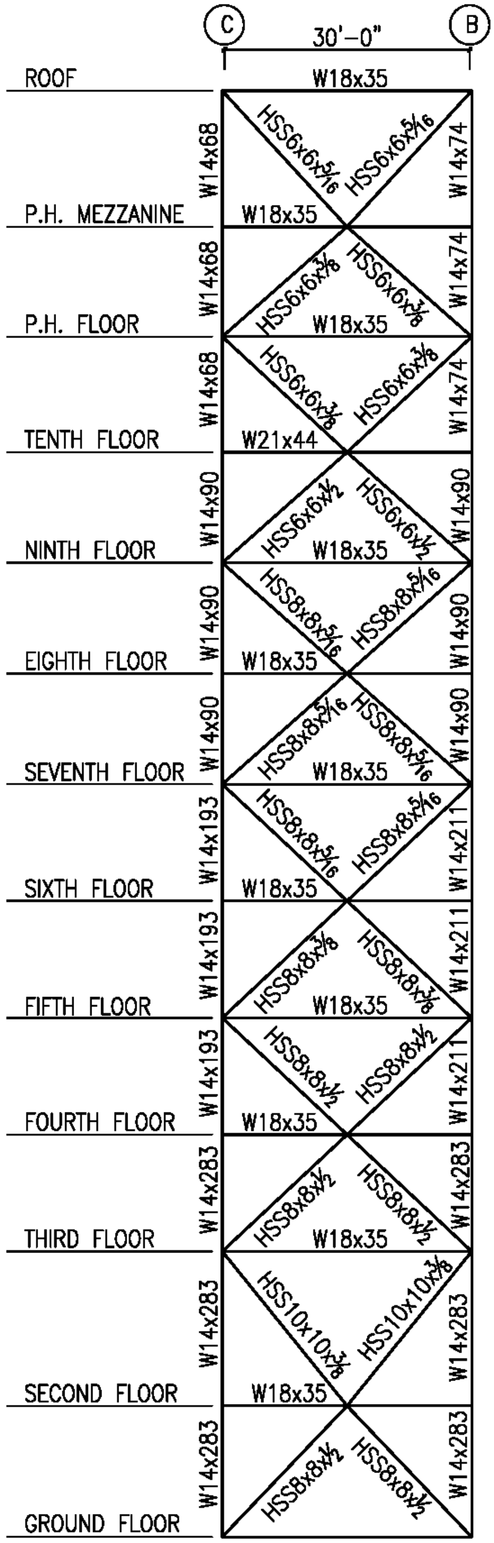

Figure 57 – BF-7 Braced Frame Elevation

Final Report

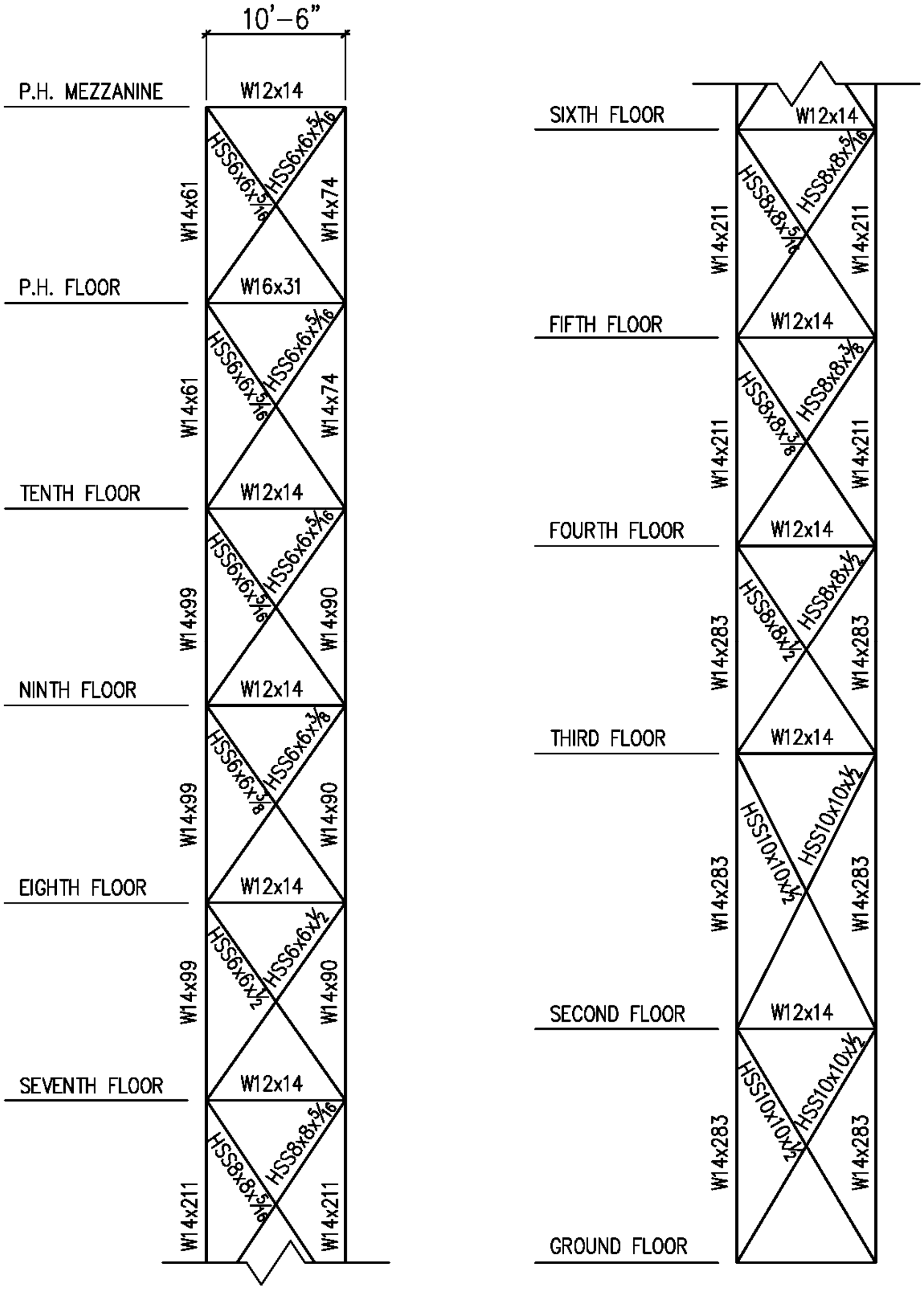

Figure 58 – BF-8 Braced Frame Elevation

Final Report

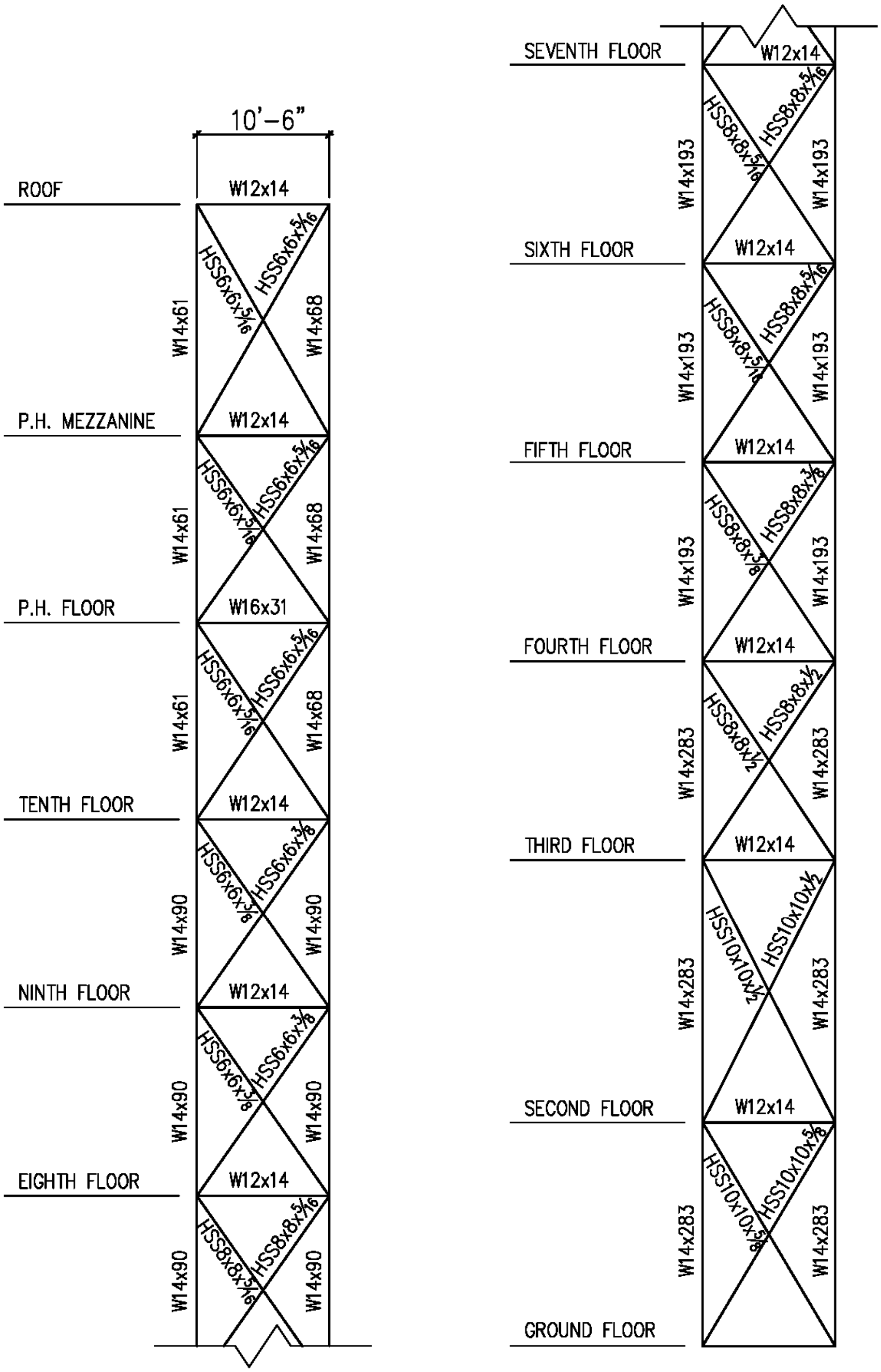

Figure 59 – BF-9 Braced Frame Elevation

Final Report

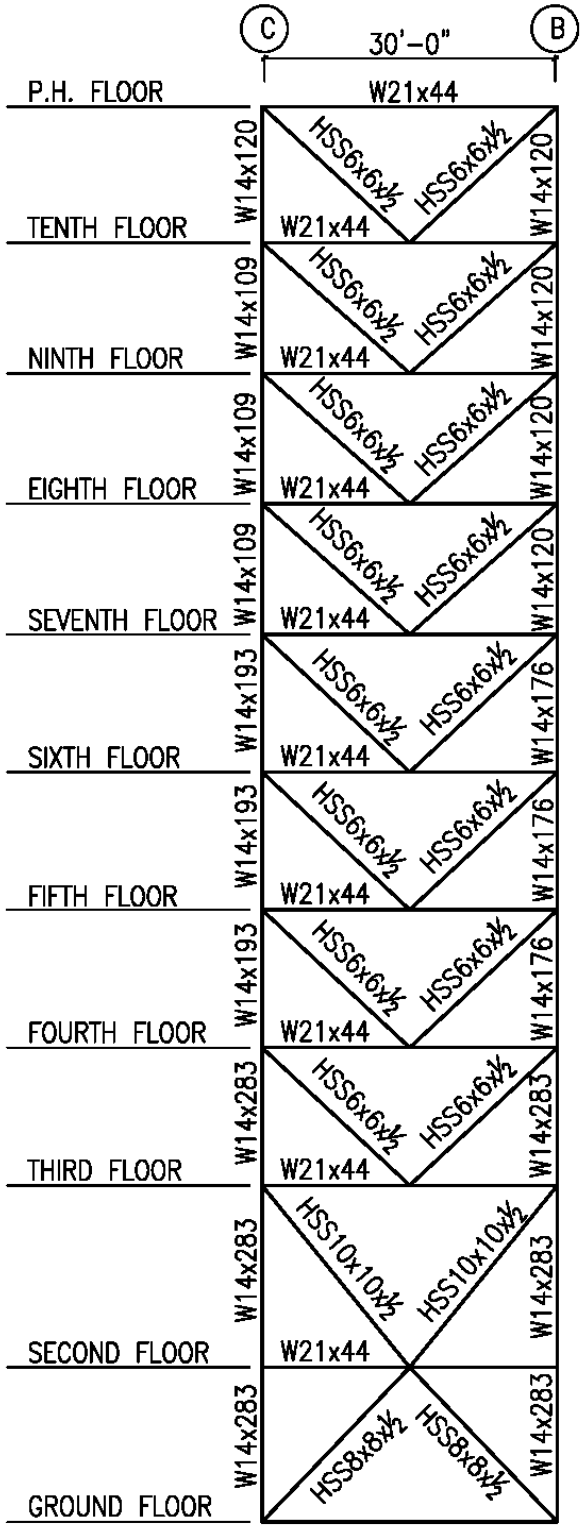

Figure 60 – BF-10 Braced Frame Elevation

Final Report

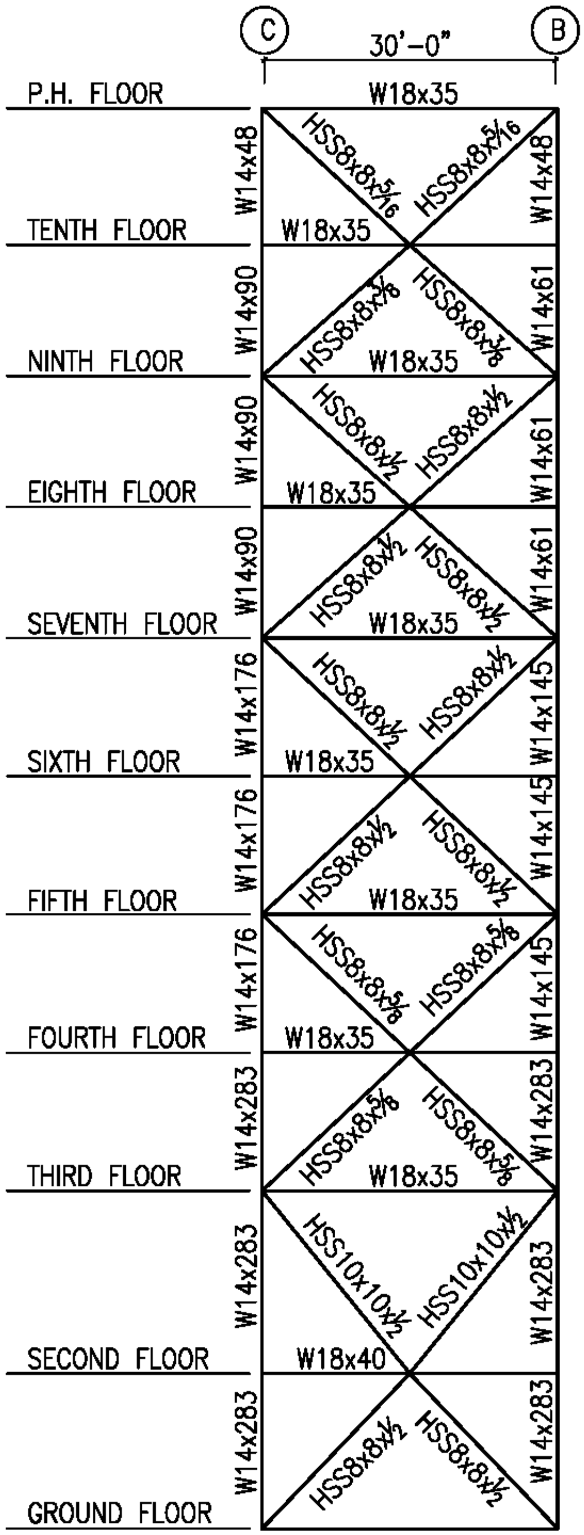

Figure 61 – BF-11 Braced Frame Elevation

Final Report

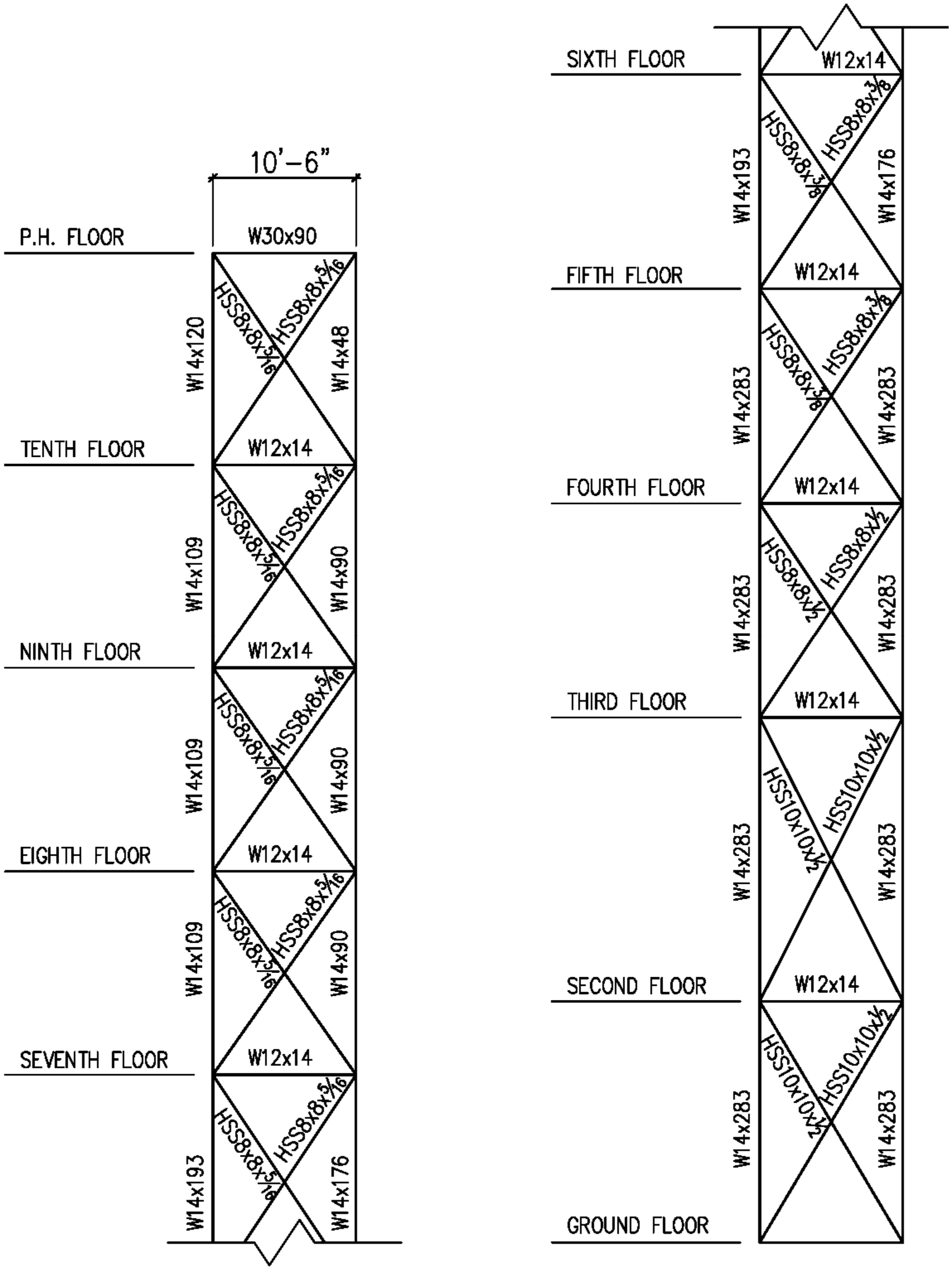

Figure 62 – BF-12 Braced Frame Elevation

Final Report

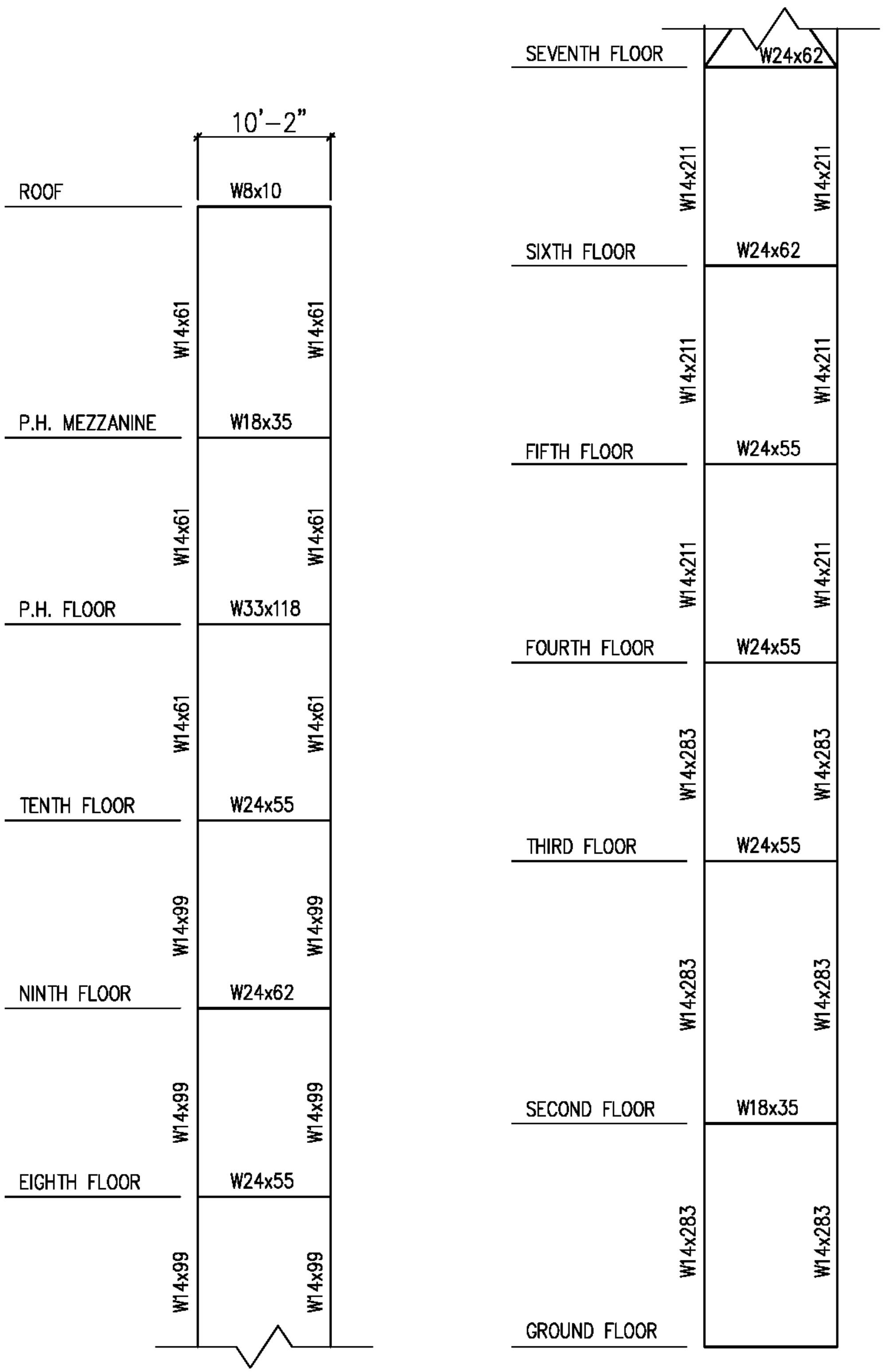

Figure 63 – MF-1 Moment Frame Elevation

Final Report

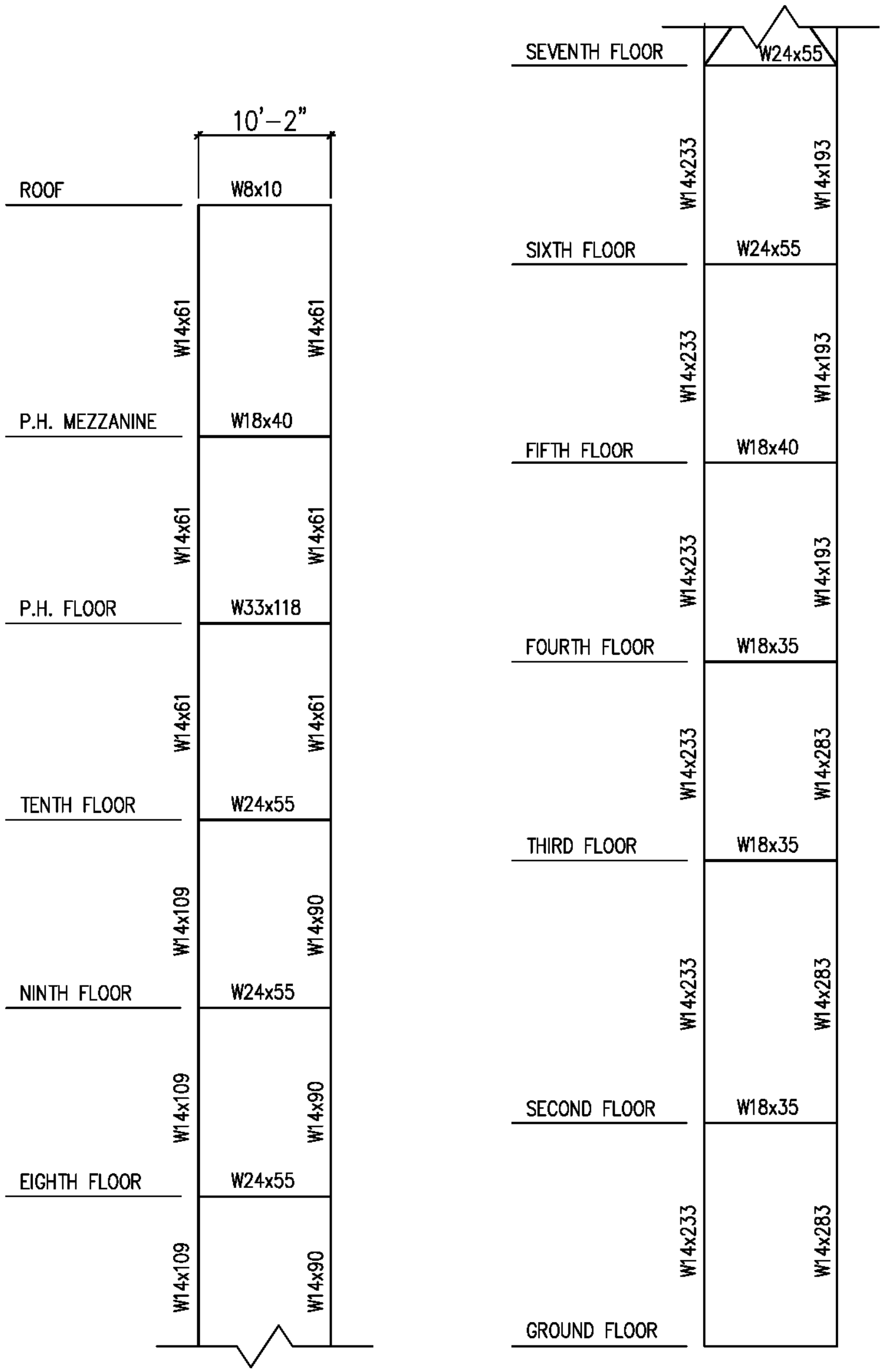

Figure 64 – MF-2 Moment Frame Elevation

Dual System Option 2

(The braced frames from the final design of the braced frame lateral system remain unchanged, and the following centrally located moment frames are added.)

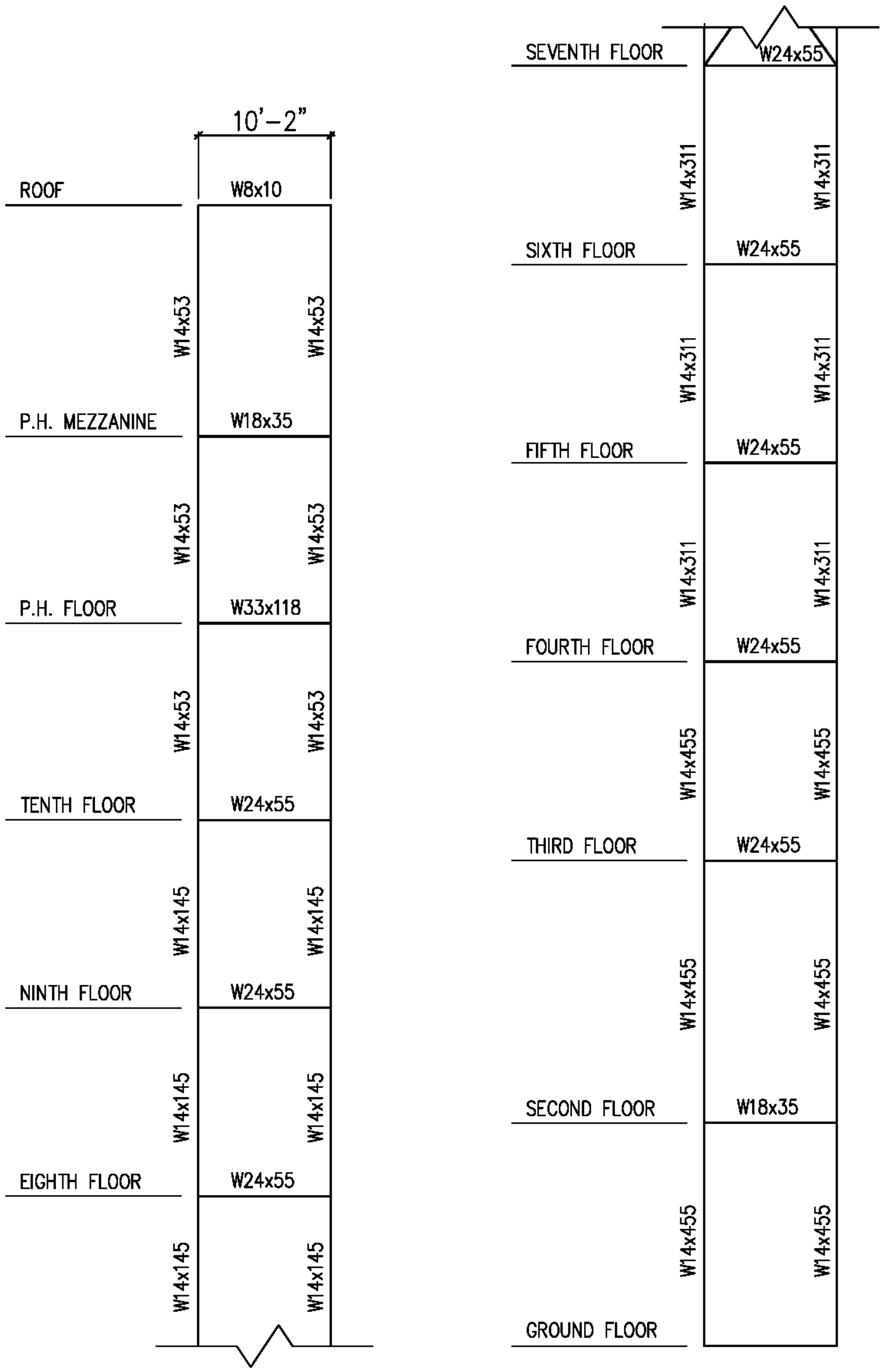

Figure 65 – MF-1 Moment Frame Elevation

Final Report

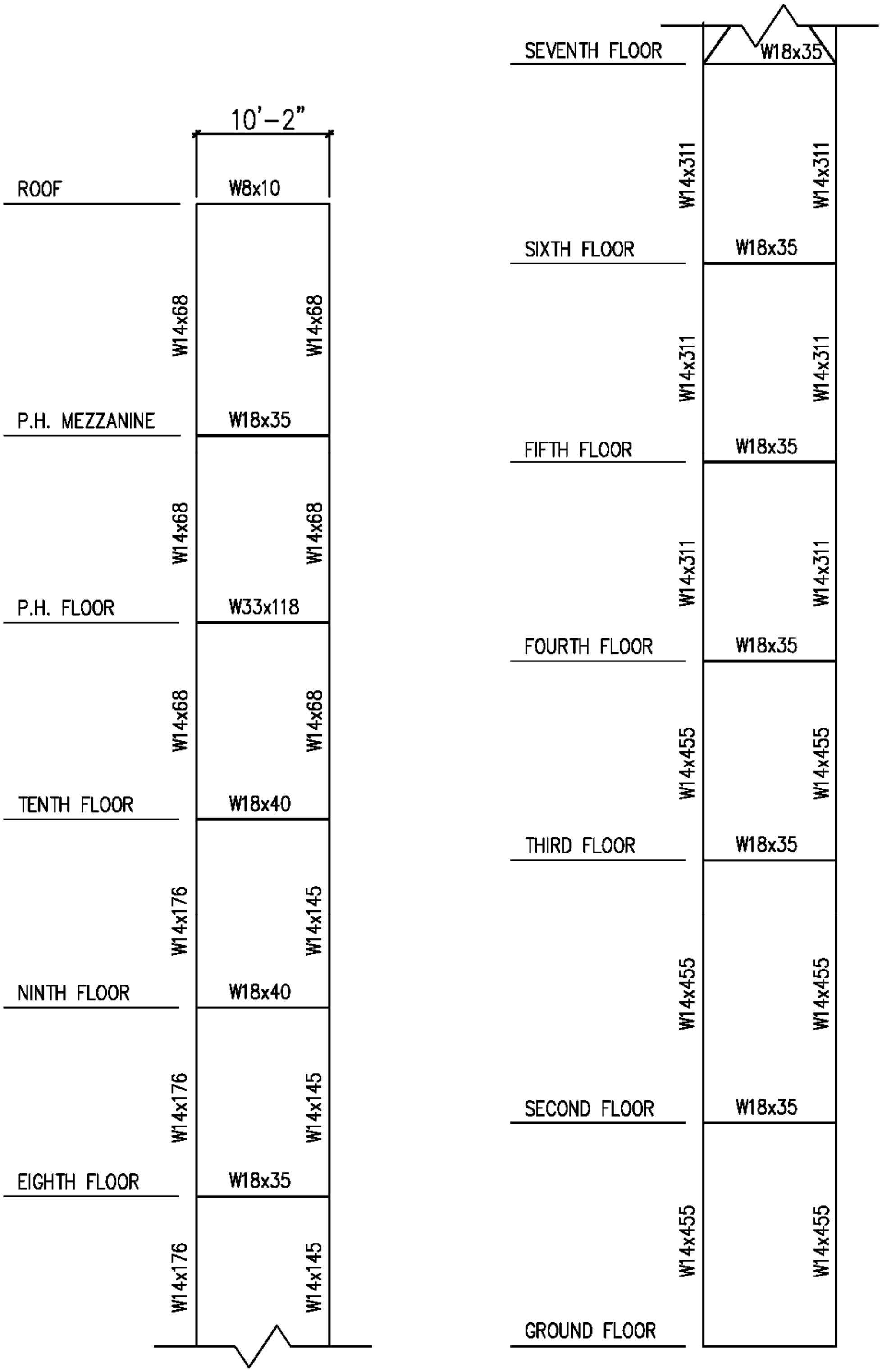

Figure 66 – MF-2 Moment Frame Elevation

Appendix D – Architectural Plans: Service Core

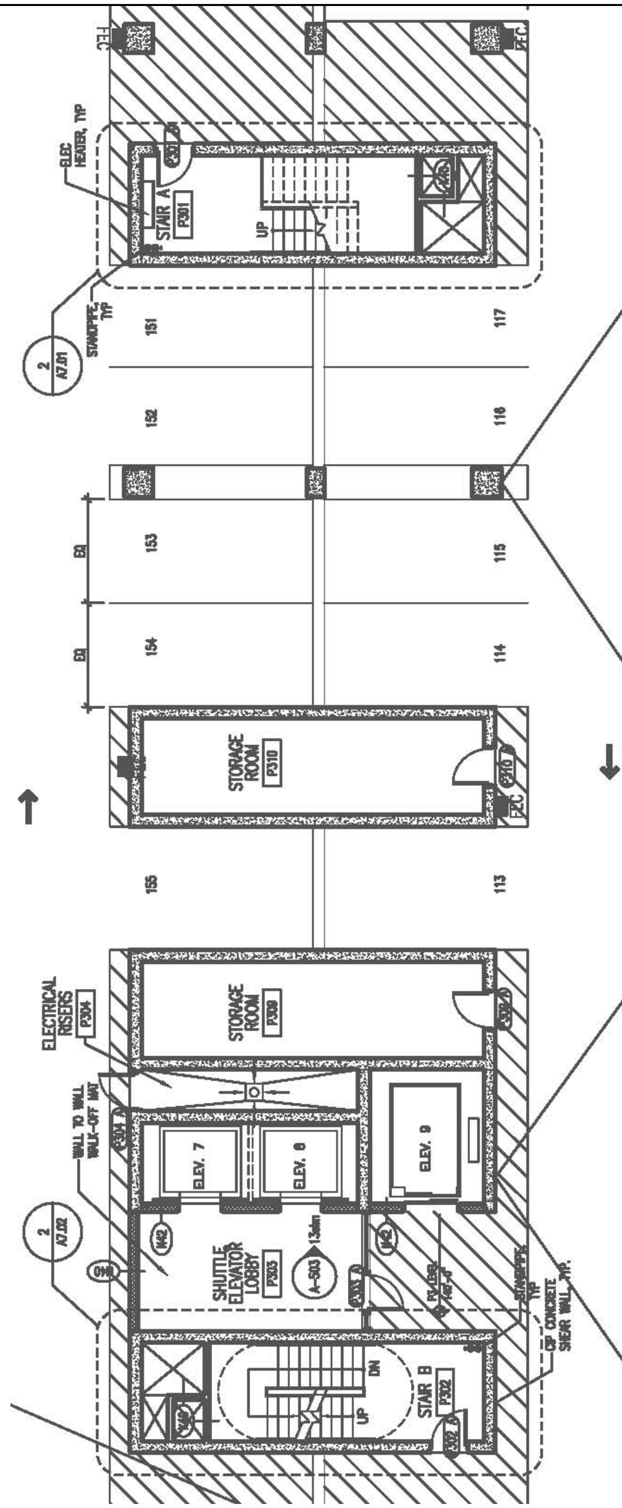

Figure 67 – Original Parking Garage P3 Level Service Core Plan

Final Report

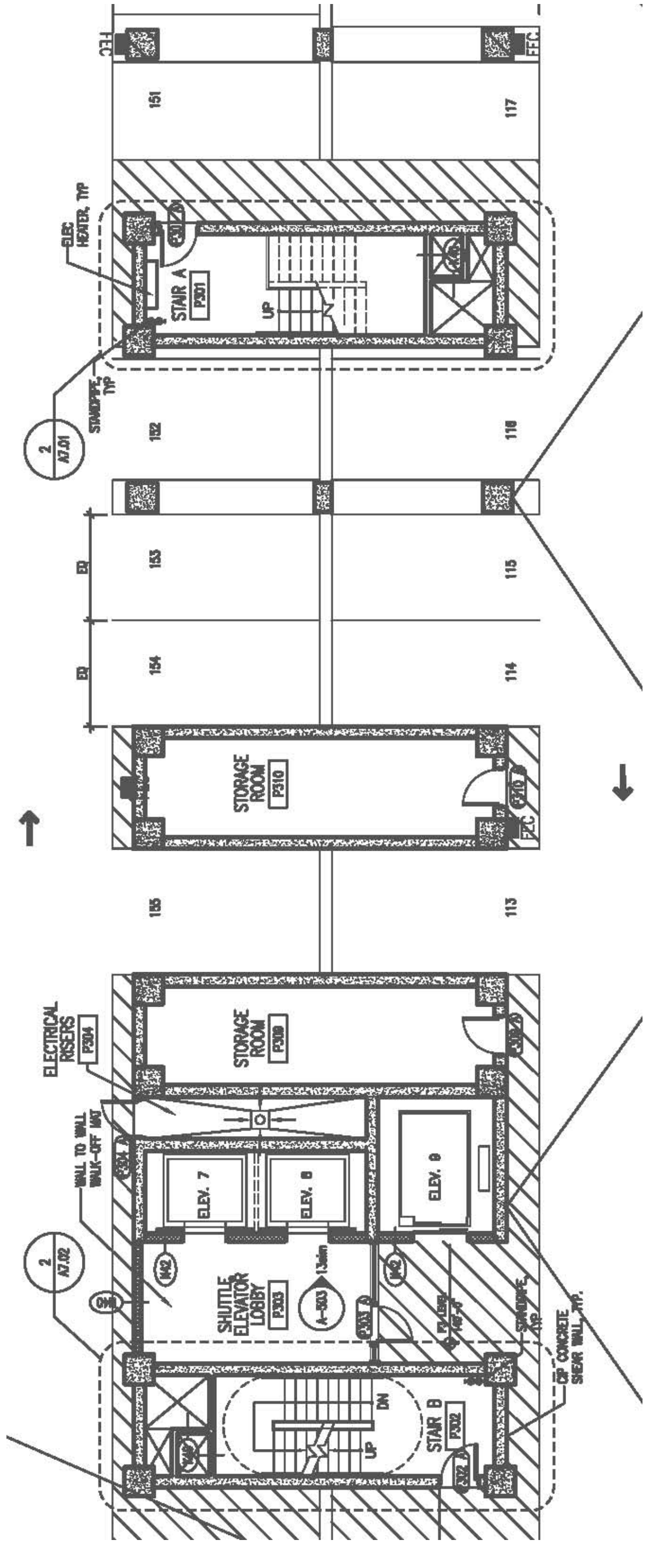

Figure 68 – Revised Parking Garage P3 Level Service Core Plan

Final Report

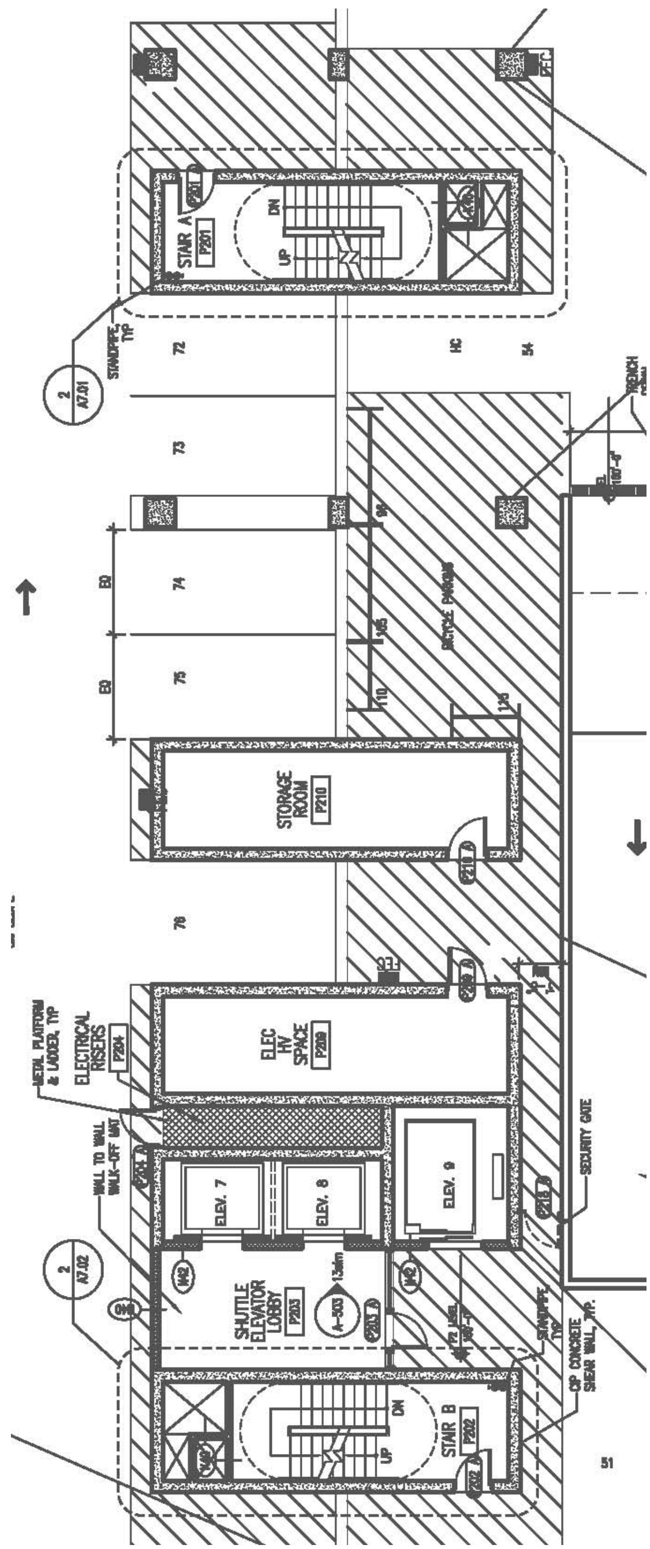

Figure 69 – Original Parking Garage P2 Level Service Core Plan

Final Report

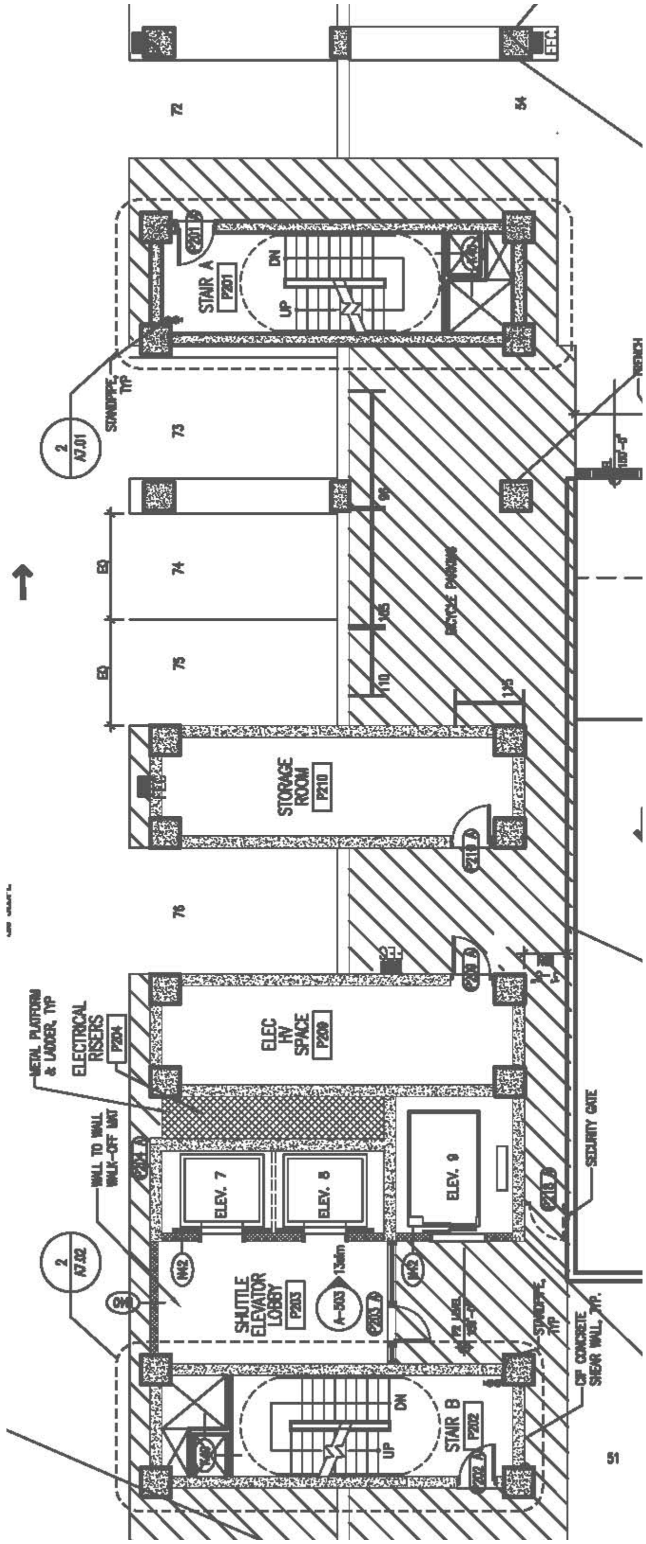

Figure 70 – Revised Parking Garage P2 Level Service Core Plan

Final Report

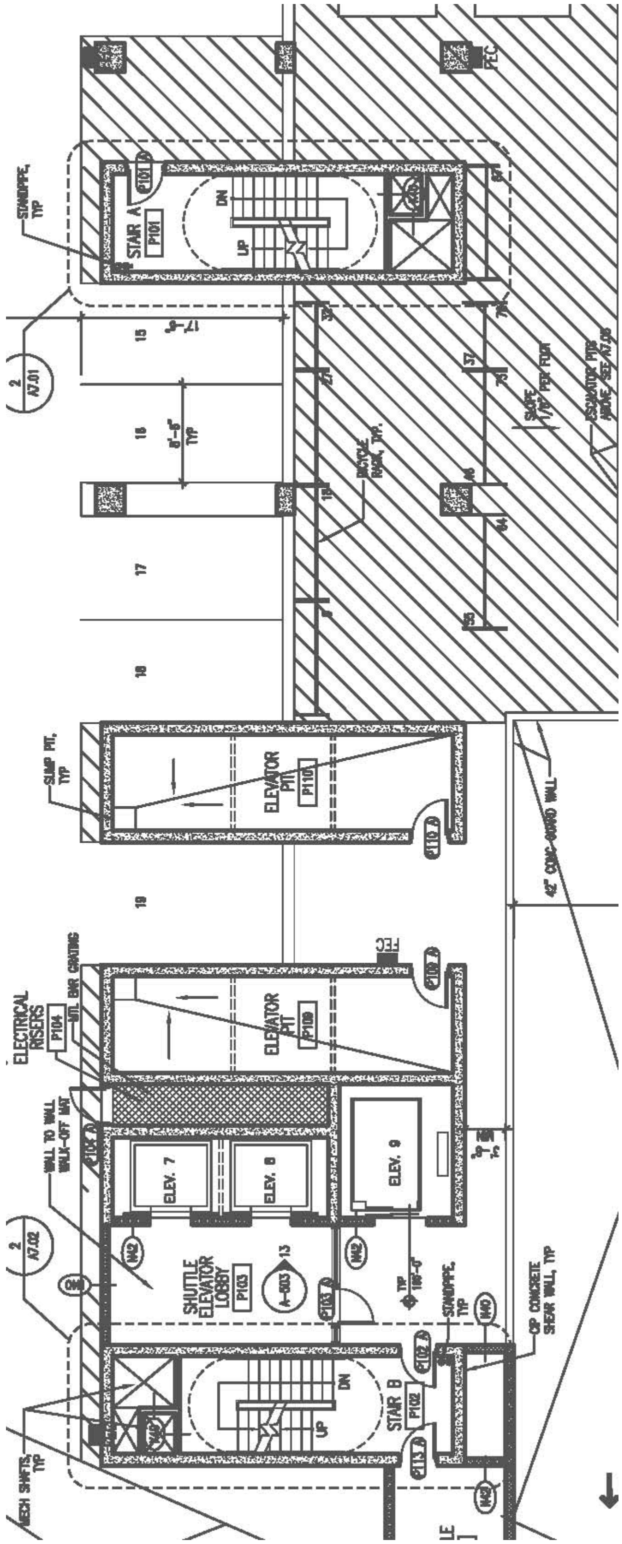

Figure 71 – Original Parking Garage P1 Level Service Core Plan

Final Report

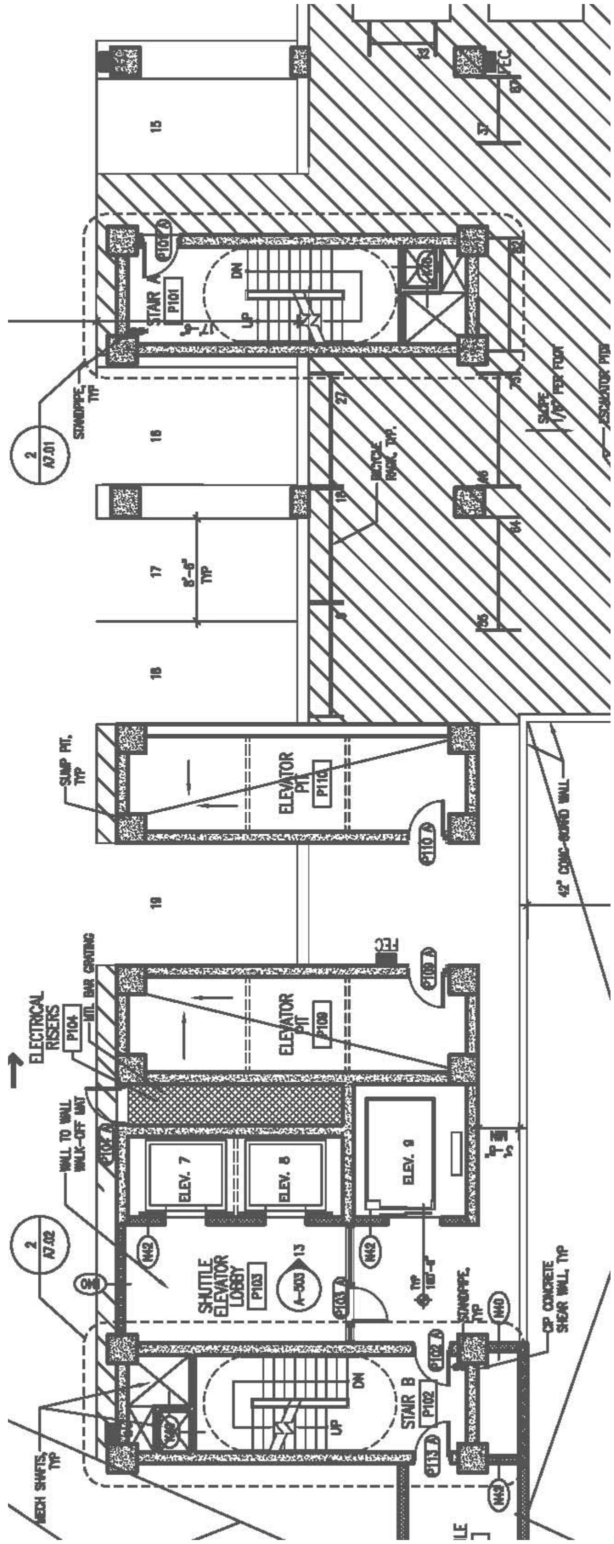

Figure 72 – Revised Parking Garage P1 Level Service Core Plan

Final Report

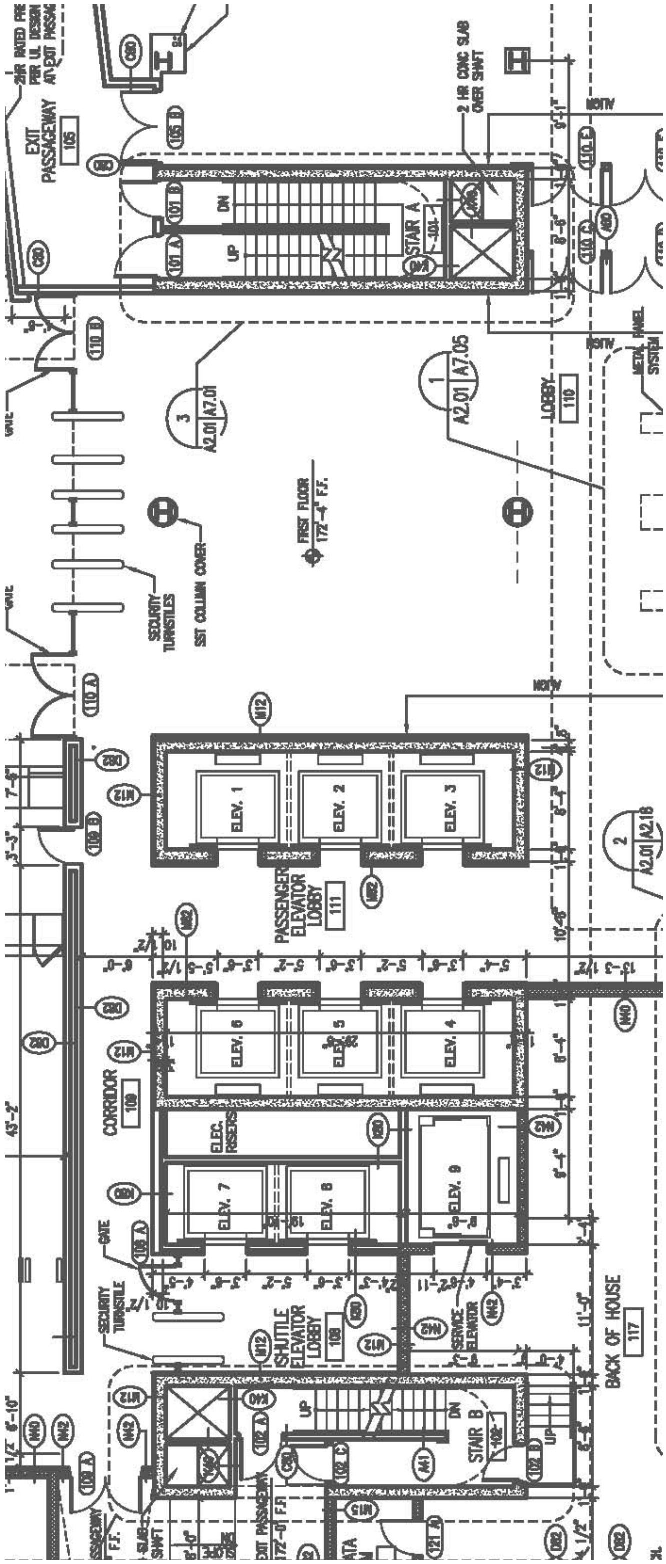

Figure 73 – Original 1st Floor Service Core Plan

Final Report

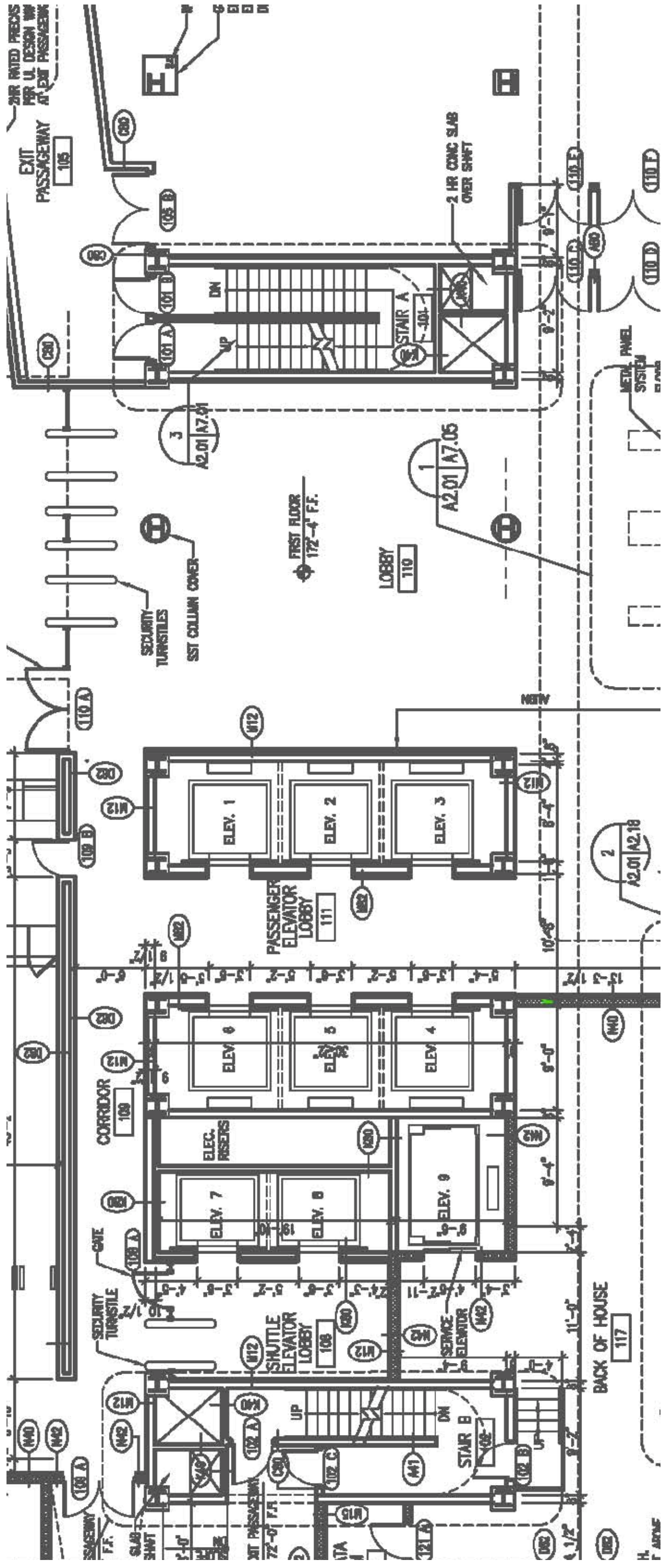

Figure 74 – Revised 1st Floor Service Core Plan

Final Report

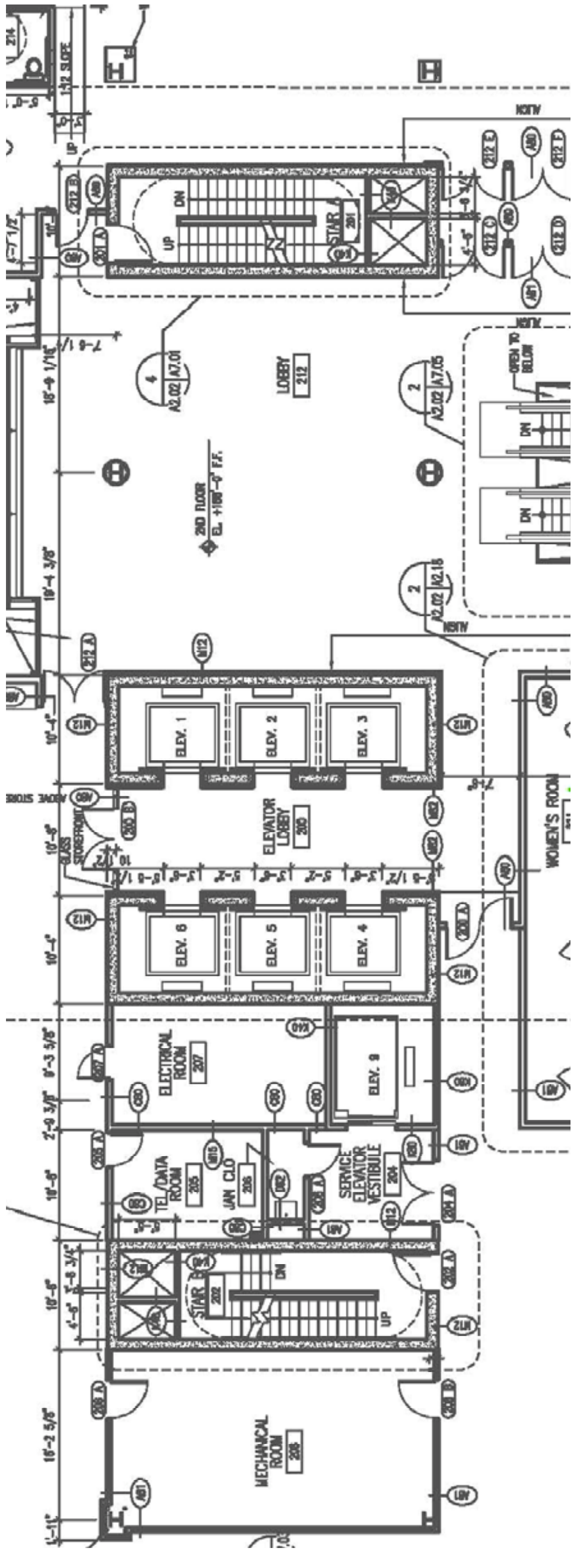

Figure 75 – Original 2nd Floor Service Core Plan

Final Report

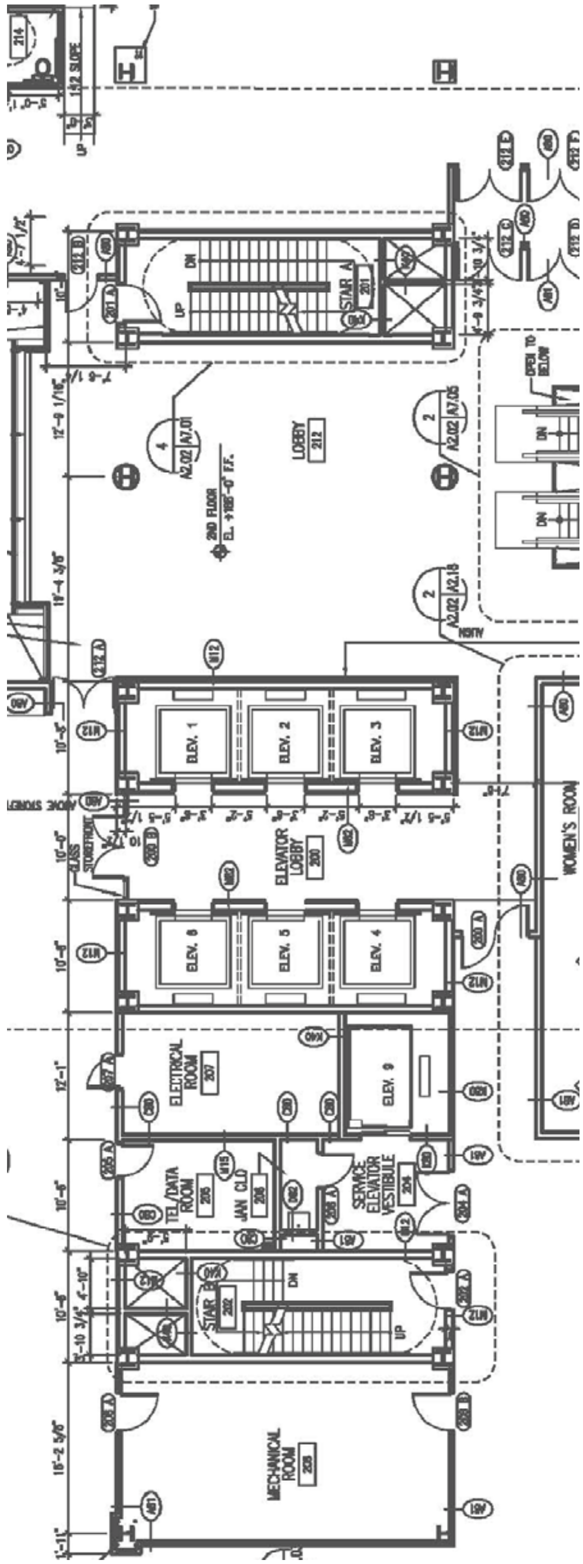

Figure 76 – Revised 2nd Floor Service Core Plan

Final Report

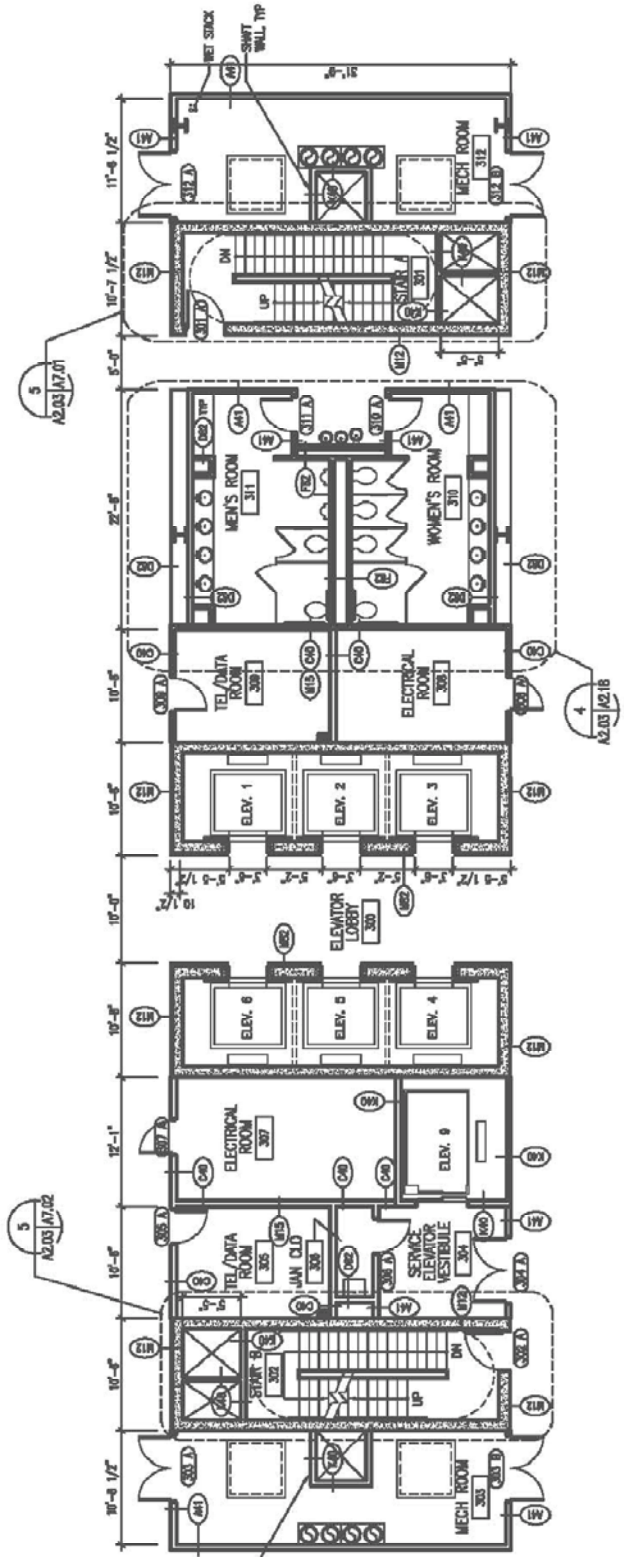

Figure 77 – Original 3rd Floor Service Core Plan

Final Report

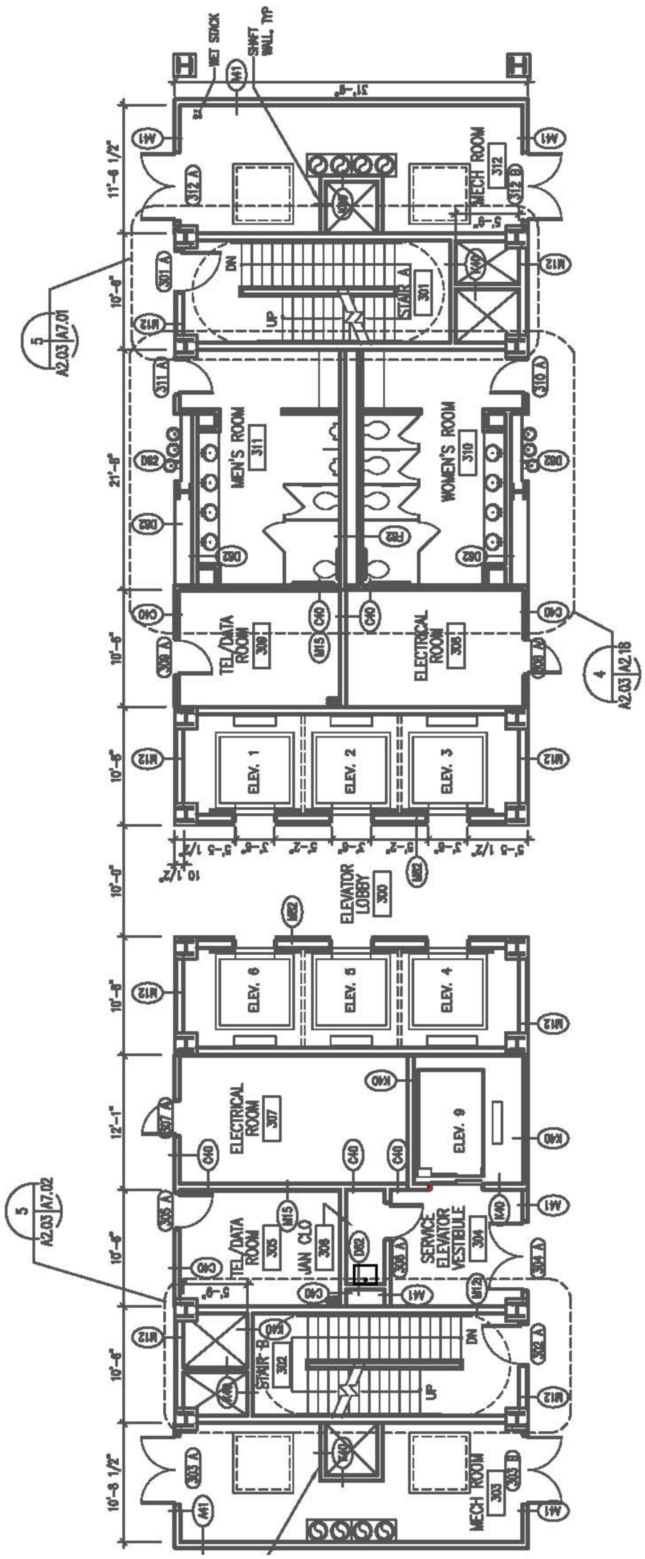

Figure 78 – Revised 3rd Floor Service Core Plan

Final Report

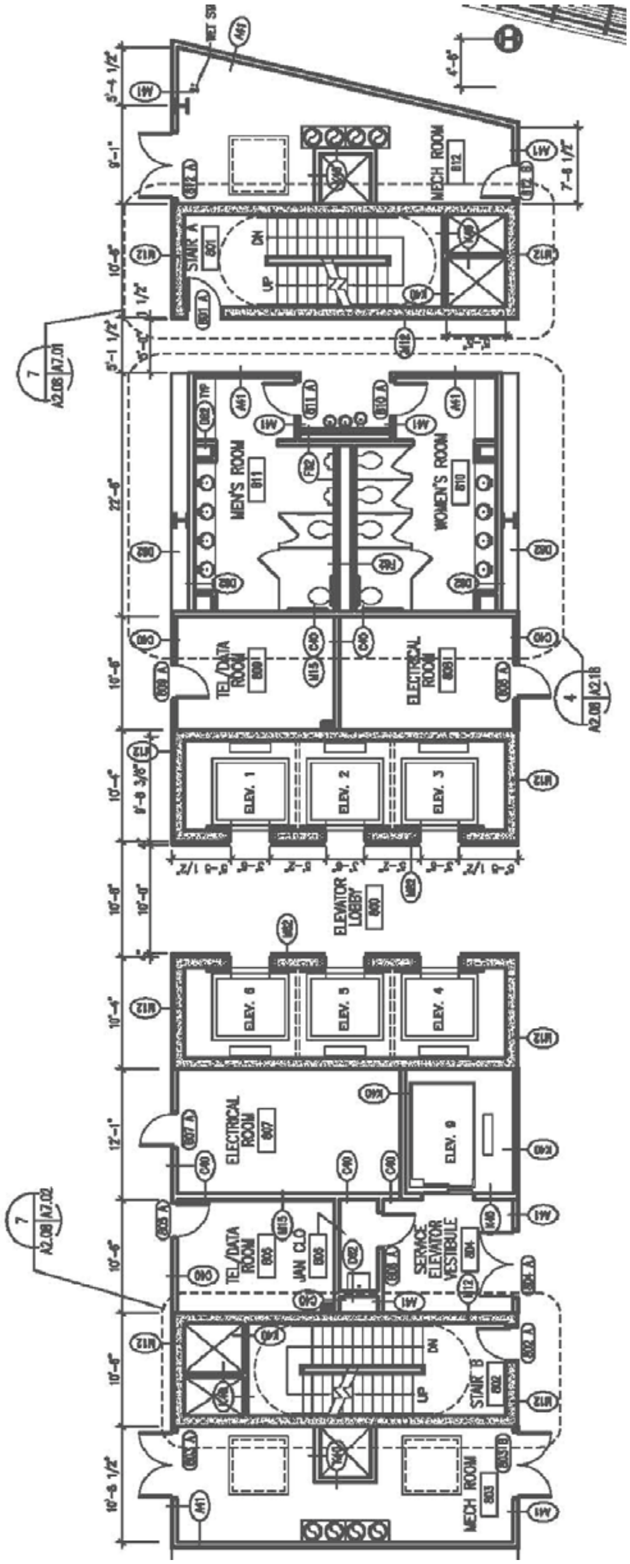

Figure 79 – Original 8th Floor Service Core Plan

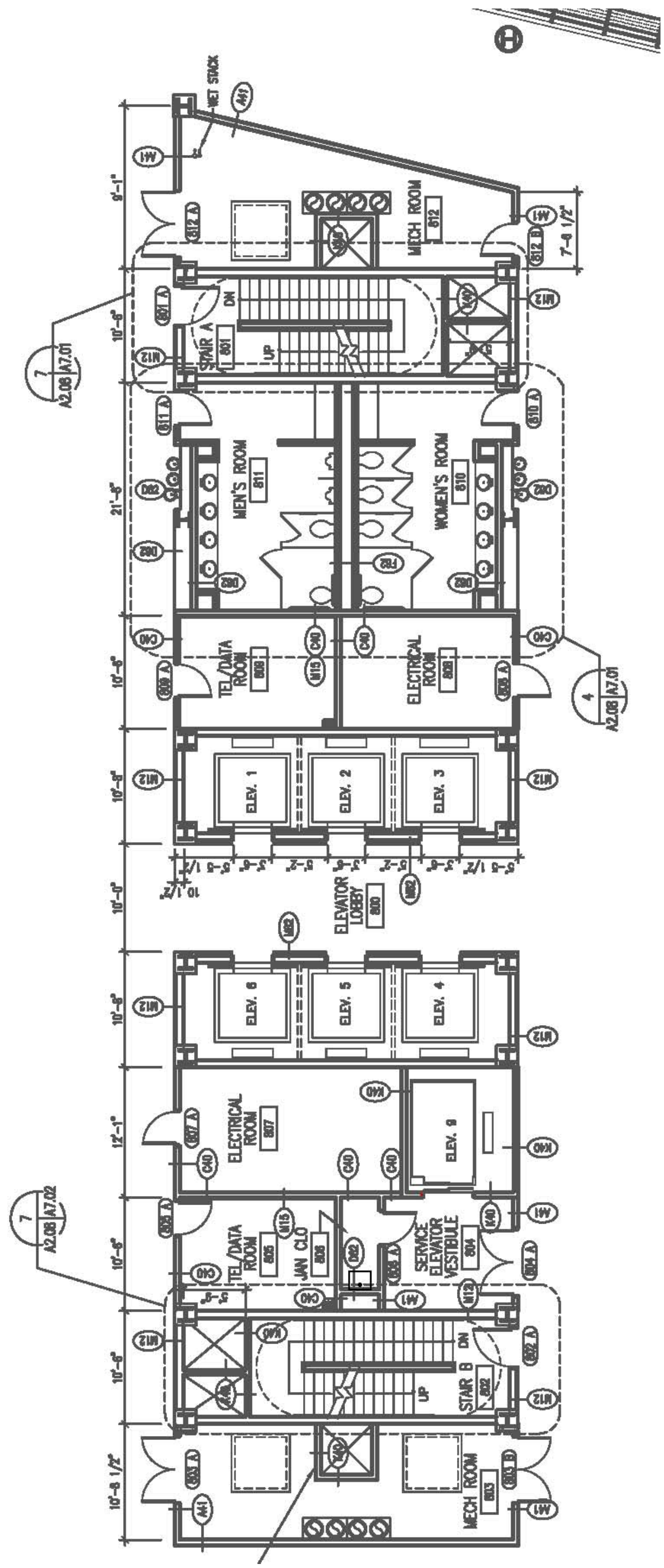

Figure 80 – Revised 8th Floor Service Core Plan

Final Report

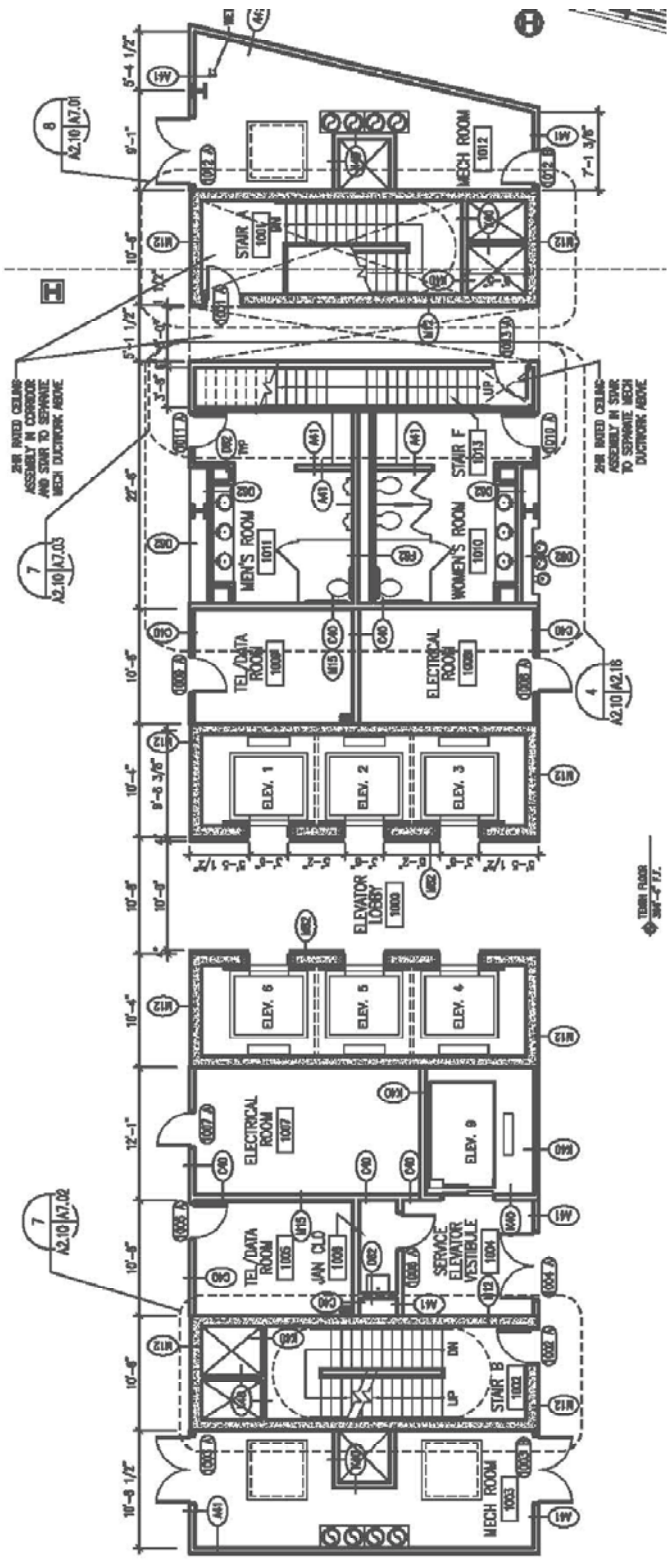

Figure 81 – Original 10th Floor Service Core Plan

Final Report

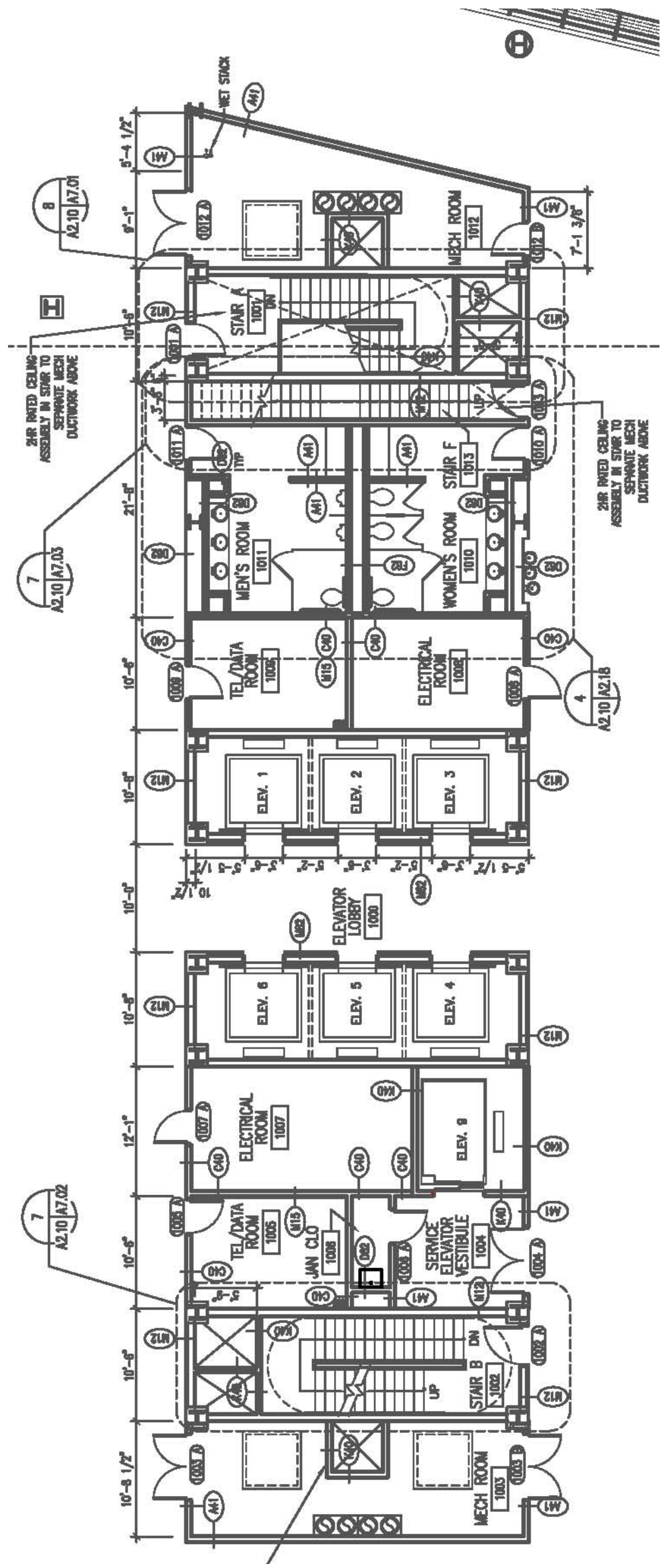

Figure 82 –Revised 10th Floor Service Core Plan

Appendix E – Green Roof Locations

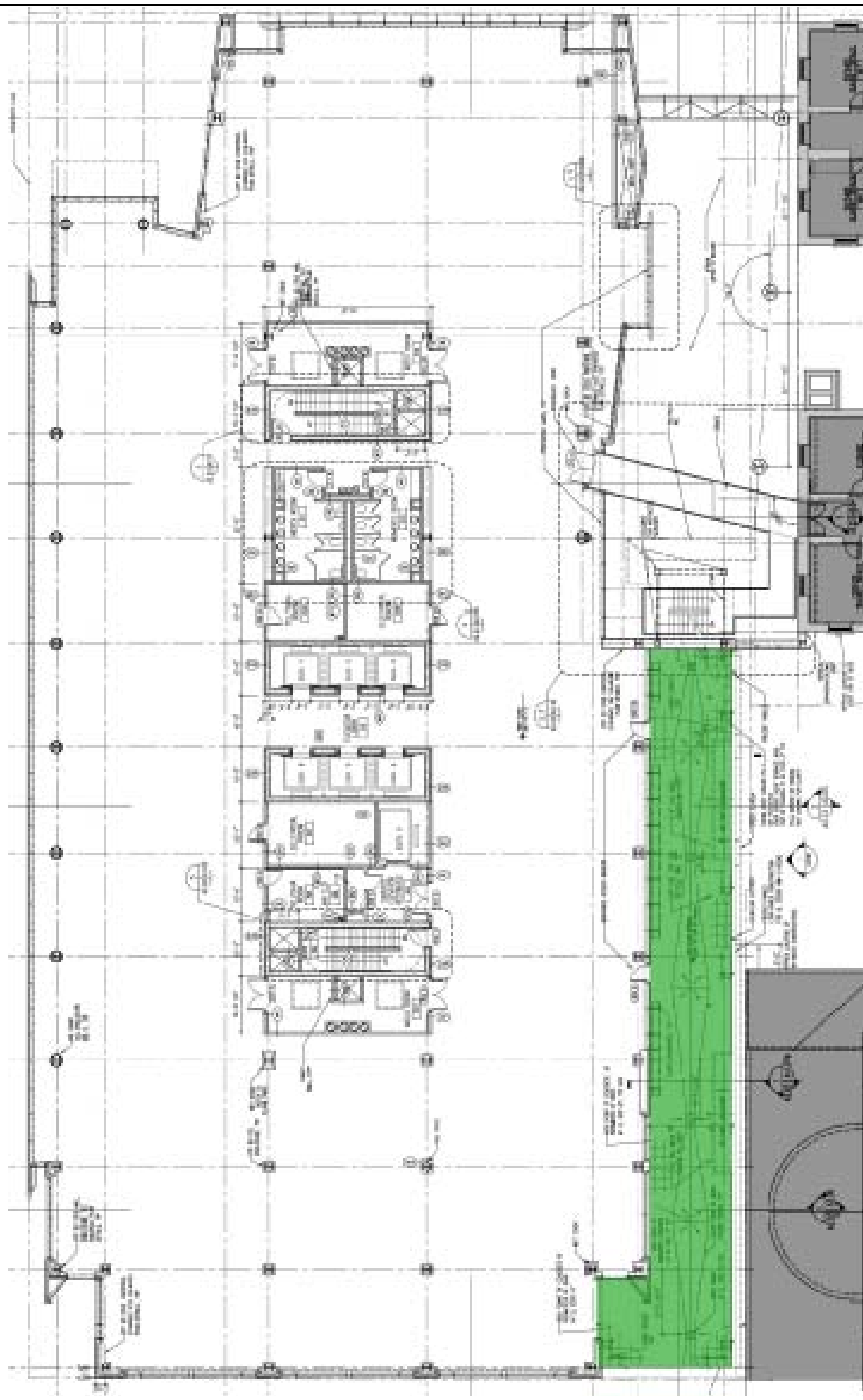

Figure 83 – Location of Green Roof on 3rd Floor Terrace

Final Report

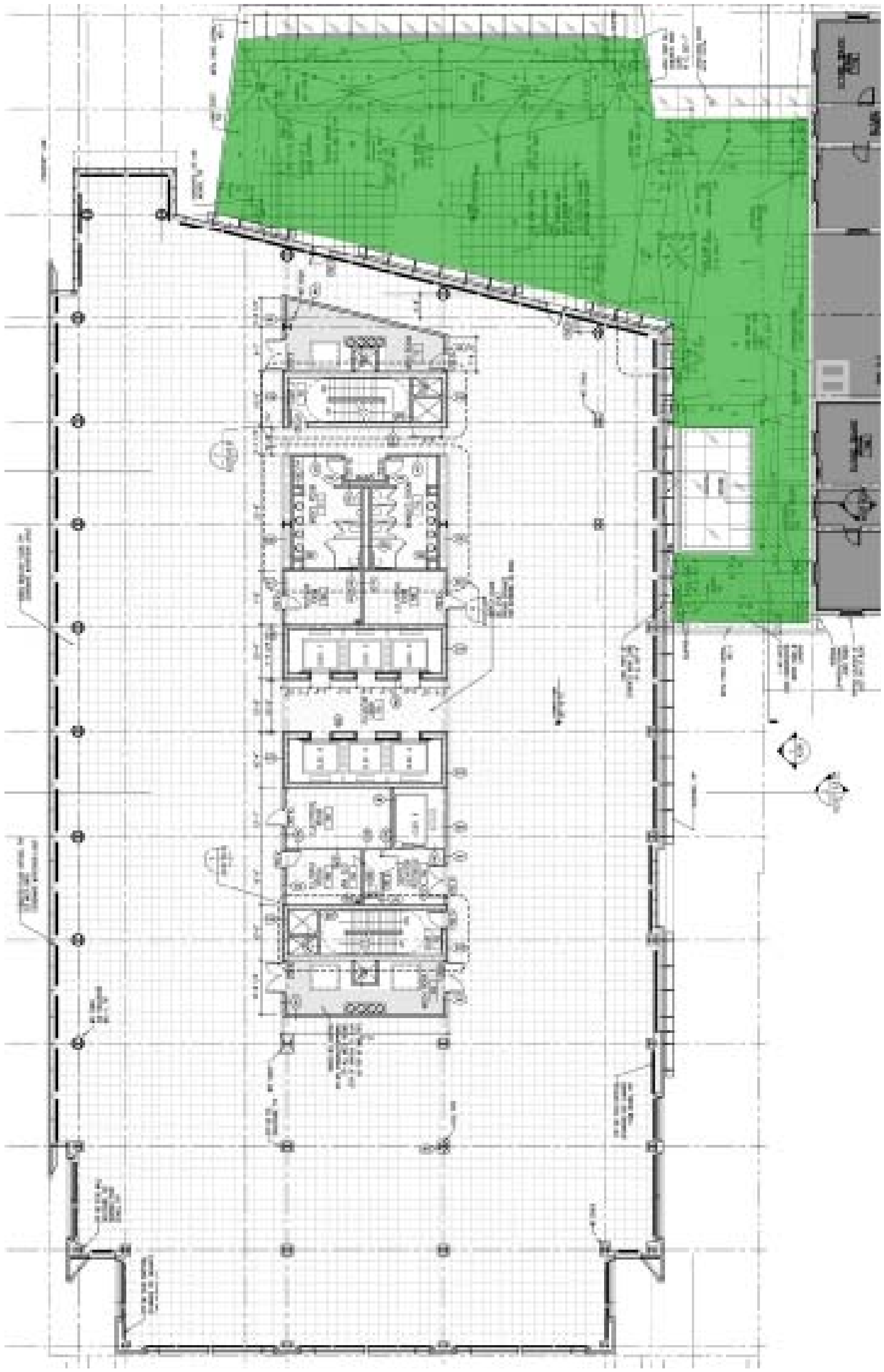

Figure 84 – Location of Green Roof on 7th Floor Terrace

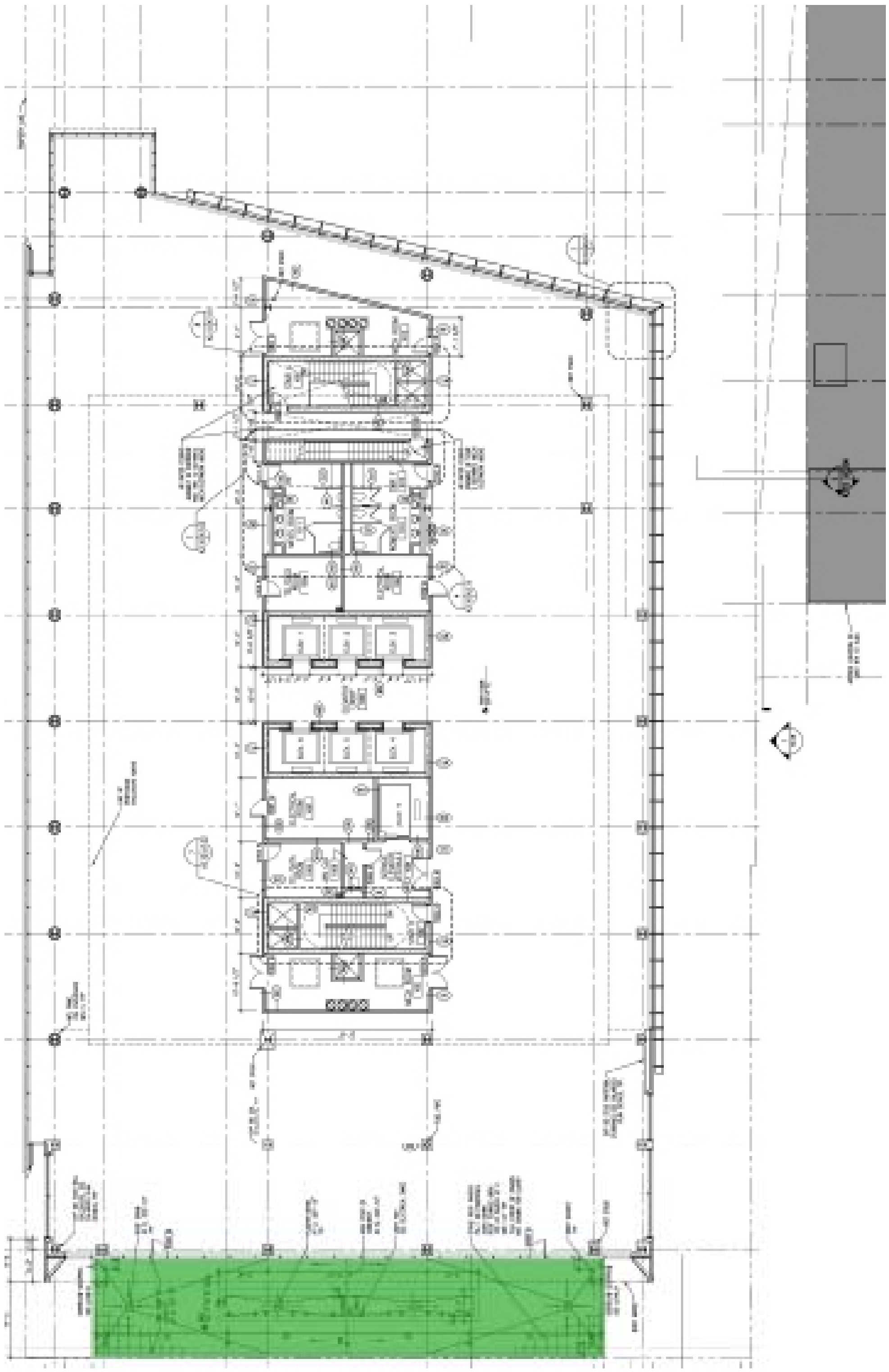

Figure 85 – Location of Green Roof on 10th Floor Terrace

Appendix F – Design of Optimal Steel Lateral System

Dual System Option 2 with Revised Architecture and Roof Loads

(All frames are the same as those for Dual System Option 2 in Appendix C except for Braced Frame 11.)

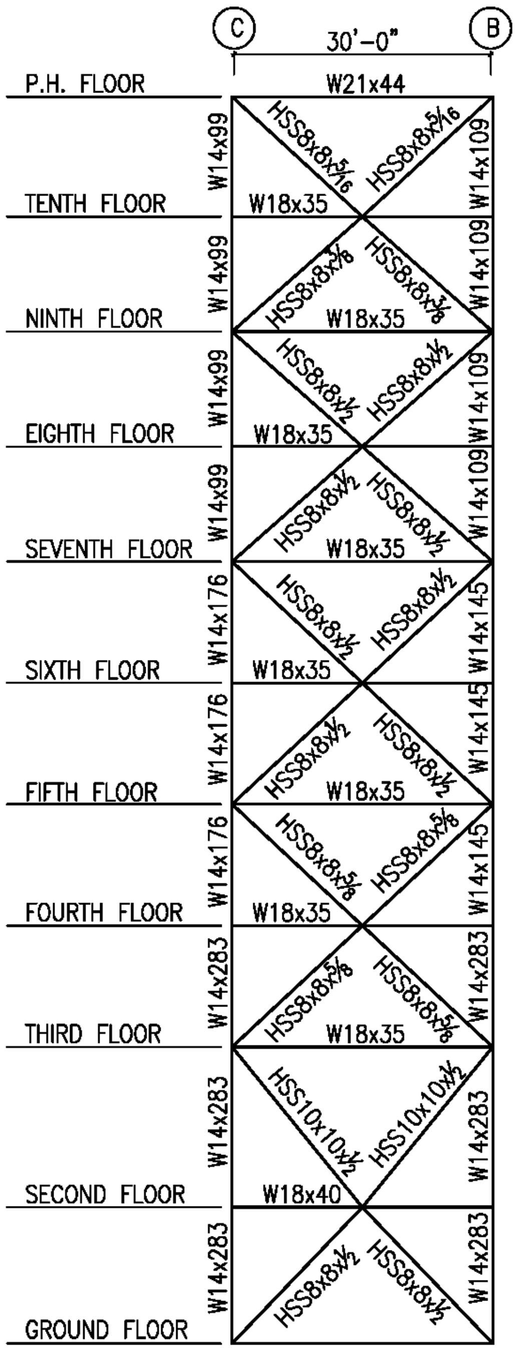

Figure 86 – BF-11 Braced Frame Elevation