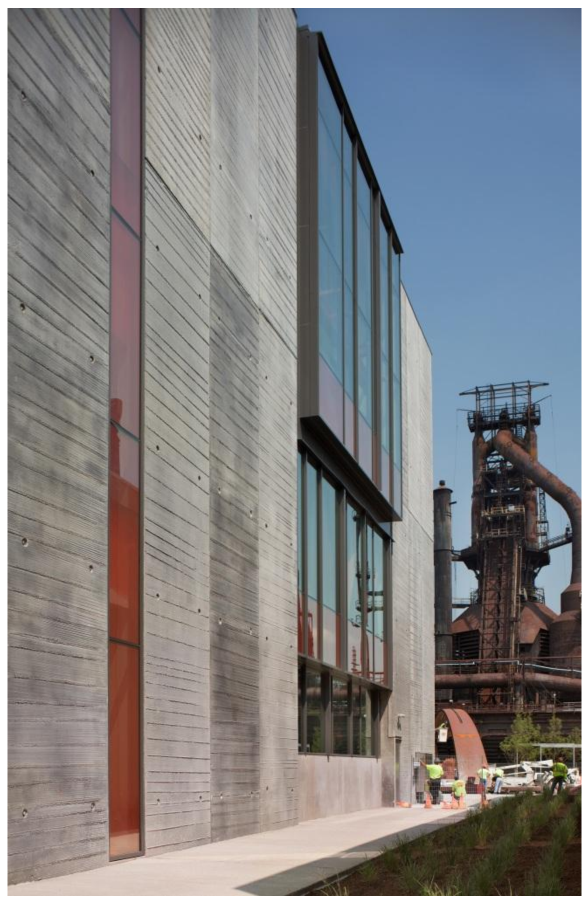
SteelStacks Performing Arts Center | Bethlehem, Pennsylvania

Technical Report III

Structural Study: Lateral System Analysis



Sarah Ashley Bednarcik
Advisor: Dr. Linda Hanagan
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Executive Summary

The purpose of this report is to complete a thorough analysis of the lateral structural system of the SteelStacks Performing Arts Center (SSPAC) and confirm that a sufficient design is detailed in the structural documents of the building. The SSPAC is a 64-foot, 4 story, 67,000 square foot arts and cultural center with a lateral system of braced frames and shear walls in the East-West direction and shear walls in the North-South direction.

A RAM model of the building was created to facilitate the analysis of the entire building, with parallel hand calculations utilized to confirm the appropriate use of the model. This model then was used to confirm that the structure met ASCE 7-05 requirements for wind and seismic loads. These checks included considerations for controlling lateral loads, torsion, drift, foundation considerations, and member checks.

Through the analysis detailed in this report, it was concluded that wind loads controlled on the lower floors, while seismic loads controlled on the upper floors. Story drifts and displacements met code requirements under both wind and seismic considerations, and led to the confirmation of the high stiffness of the building, which is also understood through the high amount of shear walls, low building height, and the values found for the period. By evaluating the resisting moment of the structure, the adequacy of the design of the foundation for the overturning moment was verified.

Member checks performed for confirmation of sufficient design of the lateral system focused on critical members found in Frame 2. These member checks confirmed that the building was sufficiently designed for the lateral loads found on the building through a thorough analysis of all portions of the lateral system.

Appendices are included with additional calculations, tables, and references as a supplementary resource beyond the scope of the report.

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Purpose

The purpose of this technical report is to consider the lateral system as designed by the professional engineers designing the SteelStacks Performing Arts Center (SSPAC). The appropriate lateral loads, in conjunction with the existing structural system, were then evaluated through the in-depth analysis detailed in this report. A structural system overview, as well as general load summaries, has been included for a better understanding of the system preceding the floor system analysis.

Introduction

The SSPAC is a new arts and cultural center designed to fit into the historic yet modern atmosphere of its location on the site of the previous Bethlehem Steel Corporation and situated near downtown Bethlehem. The owner is committed to uniting the community through the transformation of this brownfield into a revitalized historic site with LEED Silver status for the SSPAC is in progress. This has been achieved architecturally and structurally through the raw aesthetics of the steel and concrete structure, sitting amongst the skeletons of Bethlehem Steel as shown in Figure 1.

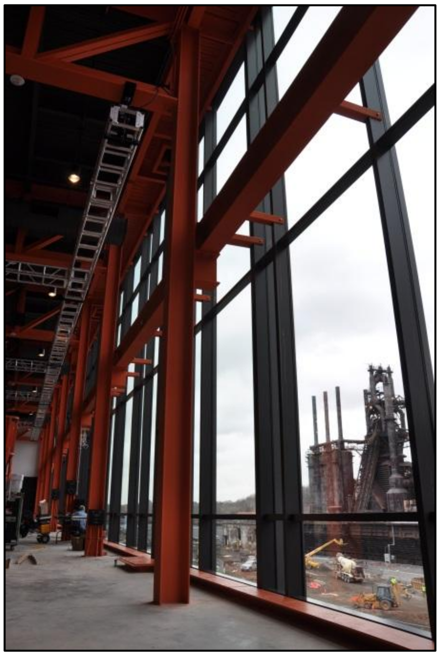


Figure 1: Interior atrium space, highlighting opening structural plan.

Exposed structural steel and large atrium spaces in the SSPAC imitate the existing warehouses and steel mill buildings for integration into the site. Yet in contrast, the SSPAC has an outlook on the community, with a large glass curtain wall system opening the interior atriums to the surrounding site. These atriums also look introspectively, uniting the various floors together as part of the mission to unite the community. These open spaces vary in size, location, and specific use, and yet all deliver similar results. The first floor consists of public spaces, such as a commons area open to above, and cinema spaces. The second floor is similar, with a mezzanine open to the common area on the first floor, as seen in the second floor plan in Figure 2. The third and fourth floors consist of a stage and small restaurant connecting the two floors via an atrium, and a cantilevered terrace adjoining the third floor, as seen in the third floor plan in Figure 3. The balcony portion of the restaurant on the fourth floor overlooks the third floor stage, as seen via outline on the third floor plan. Both the third and fourth floors have back-of-house spaces such as kitchens, offices, storage, and green rooms that service the public spaces. Other architectural floor plans are included in Appendix 1.

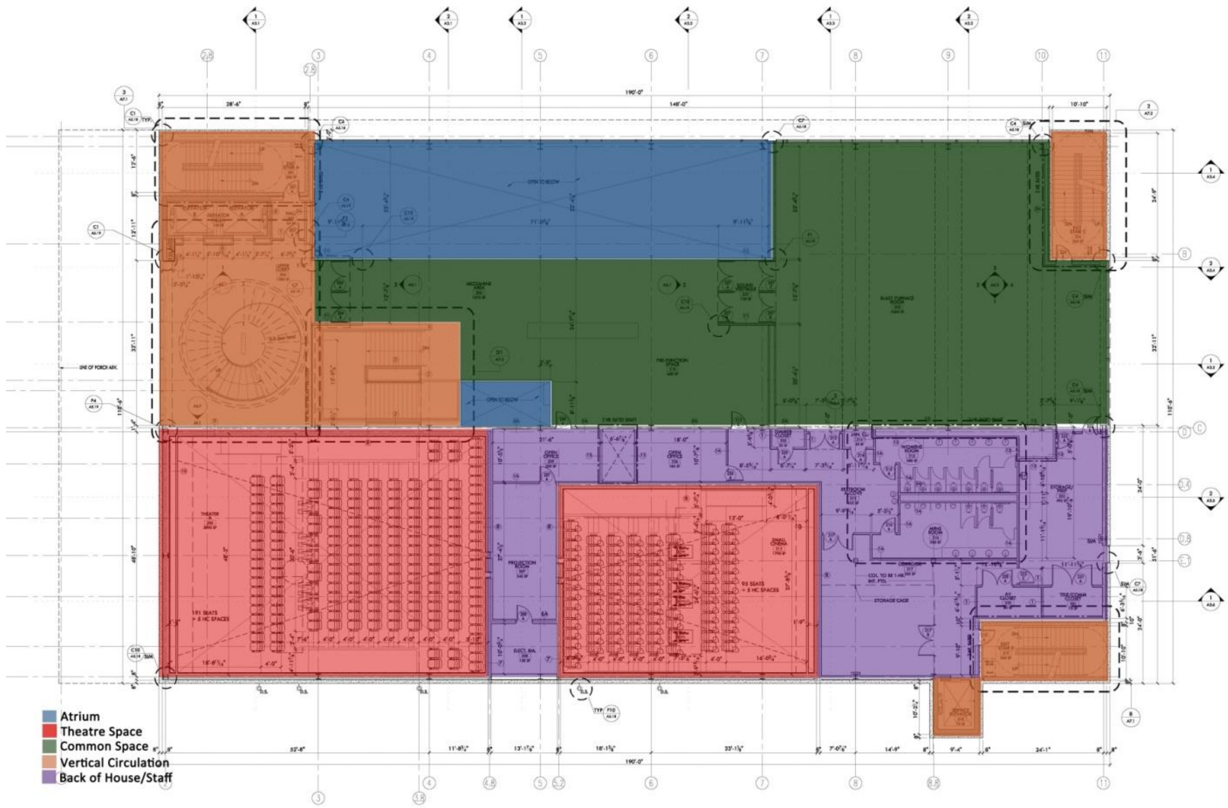


Figure 2: Floor Plan from A2.2

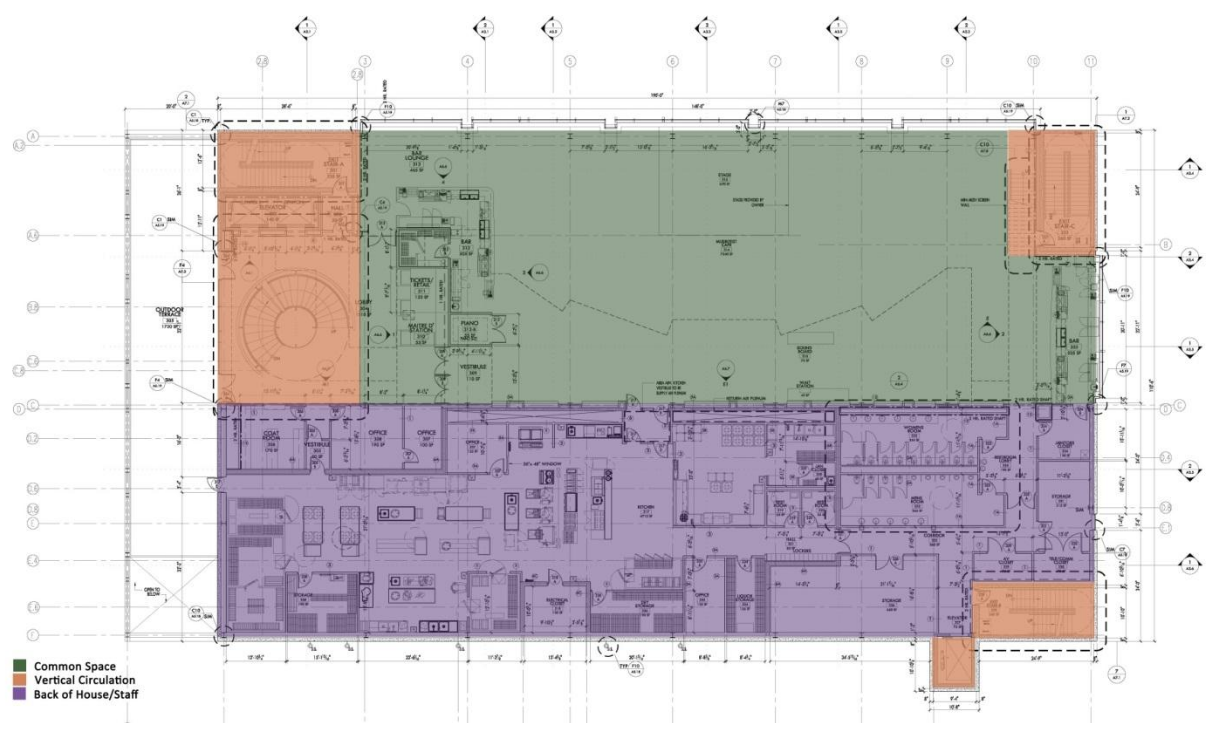


Figure 3: Third Floor Plan from A2.3

This $48 million project is approximately 67,000 square feet and is four stories above grade, with an integrated steel and concrete panel structural system. With a total building height of 64 feet, each level has a large floor-to-floor height, allowing for more open spaces and larger trusses to span the undersides of each floor system, mirroring the style of trusses found in an original warehouse. The spaces in the SSPAC include creative commons, theatres, a café, stage and performance area, production rooms, offices, and kitchens.

The main features of the façade are precast concrete panels with a textured finish, mimicking the aesthetics of the surrounding buildings, as well as a glass curtain wall system. The curtain wall system includes low E and fritted glazing along the northern facing wall that allows light to enter throughout the atrium common spaces on all floors. This is supported by the steel skeleton, which divides the building structurally into two acoustic portions, keeping vibrations from the north and south halves of the building from transferring, as seen in Figure 3.

While the SSPAC does not have any highlighted features that distinctly call to its LEED Silver certification, the integration towards sustainability of building design, use, and construction has been thoroughly developed in the structure and site. The overall building aesthetics and structural system can be attributed partially to sustainability, but also to the historical values that the site brings and the future purpose of the space integrating into these focuses.

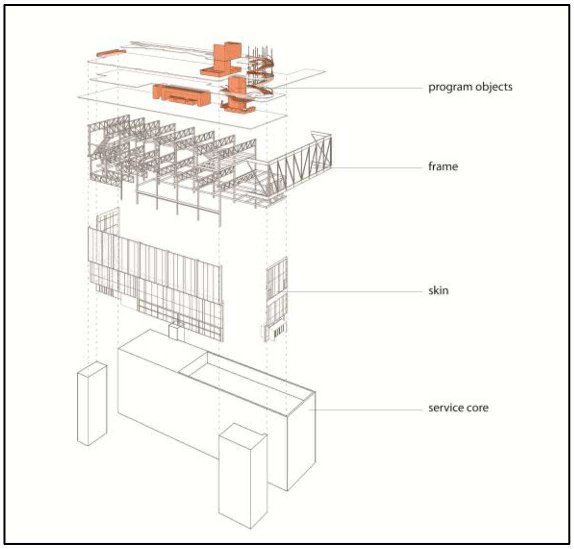


Figure 4 : Image displaying the separation of spaces through the structural design.

Courtesy of Barry Isett, Inc. & Assoc.

General Structural Information

This section provides a brief overview of the SSPAC in terms of the structural system, design codes, and materials, detailing the structural elements and factors associated with the structure's design and performance.

Structural System Overview

The structure of the SteelStacks Performing Arts Center consists of steel framing on a foundation of footings and column piers. Precast concrete panels and braced frames make up the lateral framing. The second, third, and fourth floors consist of normal weight concrete on metal decking, supported by a beam and truss system. The roof consists of an acoustical decking and slab system.

Foundation

French & Parrello Associates conducted field research on May 20, 2009, collecting the plan and topographic information shown on the civil drawings. The site of the SSPAC had an existing building, to be fully removed before start of construction. This demolition included the removal of the foundation and slab on the west side of the site. The location of an underground tunnel directly under the existing building was also taken into consideration when designing the foundation system for the SSPAC. The SSPAC is built above the original building portion that was demolished. A plan of this is included in Appendix 1.

Following the survey findings, provisions were supplied for instances of sink holes, accelerated erosion, and sediment pollution. The soil bearing pressure has been recommended on the subsequent plans as a minimum of 3000 psf, with precautions during construction required due to these results.

The foundation was then determined to be a system of column piers and footings supporting a slab-on grade. The column footings varying in size from 3'0"x3'0" to 20'0"x20'0" and vary in depth from 1'0" to 4'2". The variation in dimensions and depths of the column footings is due to the building design as well as the soil and other existing conditions that lead to settlement and strength issues. The foundations allow for a transfer of gravity loads into the soil, as seen in Figure 5, through connection with the first floor system and precast concrete panels.

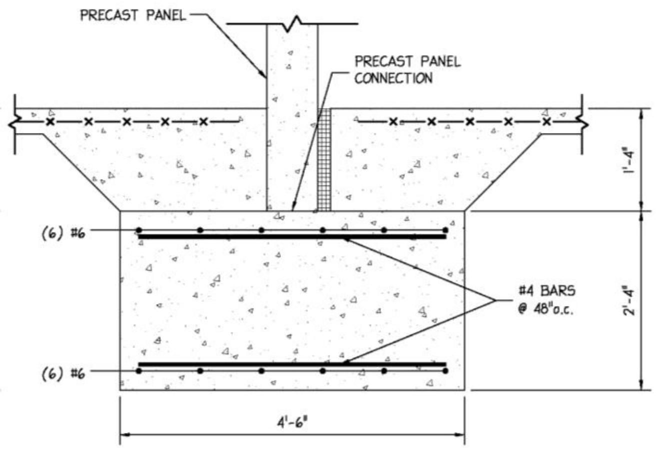


Figure 5 : Section of foundation to precast panel connection from S1.0.

Floor System

The first floor system is directly supported by the foundation of the building, with a 4" reinforced concrete slab sitting on top of a sub-floor composed of 4-6 inches of compacted gravel or crushed stone. The second and fourth floors consist of a 5" concrete slab on 2"x20 GA galvanized composite metal decking. This decking is supported by composite beams for smaller spans for the back-of-house spaces, while exposed trusses support this floor system for larger, public spaces. Uniquely, the third floor is comprised of an 8" concrete slab on 2"x16GA galvanized composite metal decking. This difference in slab thickness is due to acoustics of the spaces, requiring more vibration and sound isolation around the stage for band performances. The roof is a galvanized epicore 20GA roof deck, an acoustical decking and slab system.

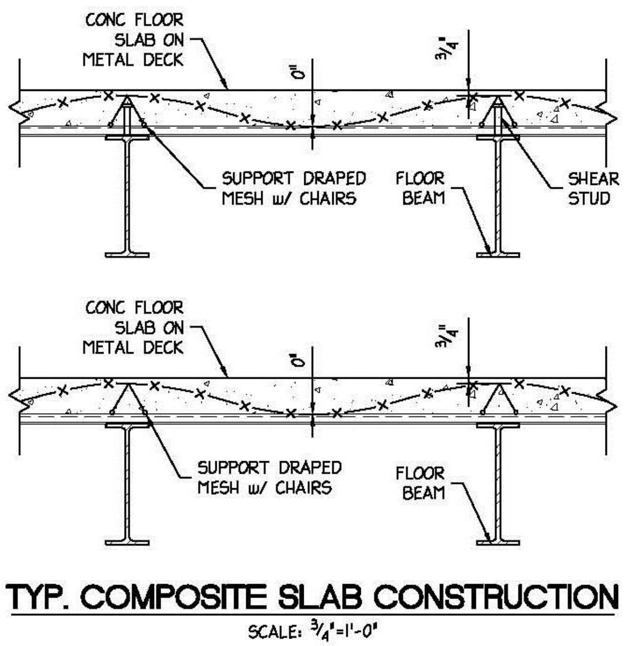


Figure 6 : Typical composite slab section for building from S2.8

Metal decking is connected to beams and girders with metal studs where appropriate. Decking is based on products from United Steel Deck, Inc. Depending on location, decking varies between roof decking, composite, and non-composite decking, but all decking is welded to supports and has a minimum of a 3-span condition. A section of the composite slab for this building can be seen in Figure 6.

Framing System

Supporting the floor systems are series of beams, girders, and trusses. Floor beams are spaced at a maximum of 7'6". The beams are also generally continuously braced, with ¾" x 4" long shear studs spaced along all beams connecting to the composite slabs. Trusses support larger spans in atrium and public spaces, while composite beams support the smaller spans for spaces such as hallways, meeting rooms, and back-of-house spaces.

Generally, the second floor consists of W12x26s for the mezzanine area and W24x76s for the blast furnace room. Beams for the third floor are W12x16s, spanning between 18'6" to 22'2". These beams are then supported by trusses, representative ones shown in Figure 7.

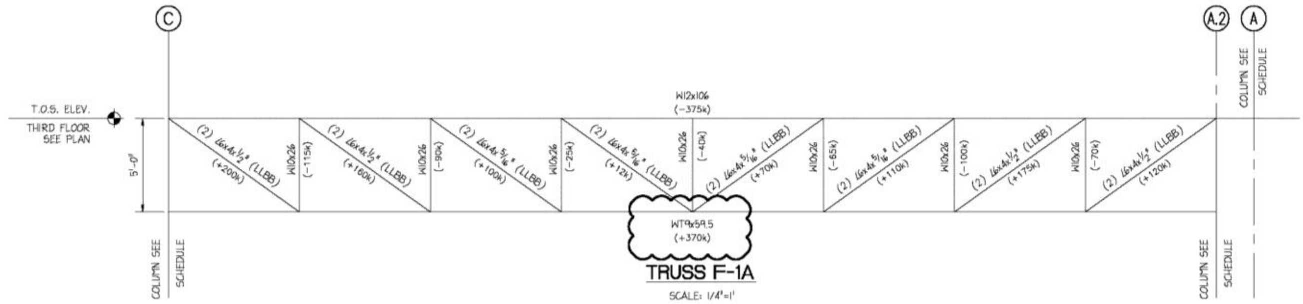


Figure 7 : Third floor representative framing system truss from S2.6.

Framing on the fourth floor is more irregular, as explained previously and included in Appendix 1, due to a large portion of the space open to the third floor, and approximately 25% of the square area excluded due to the mechanical roof. Yet even with the irregular framing plan, the beams are mostly W12x14 for public space, restroom facilities, and storage spaces and W18x35s supporting the green rooms and offices. The mechanical roof has typical framing members of W27x84s supported by Truss R-2, in a similar layout to that of Truss F-1A in Figure 7.

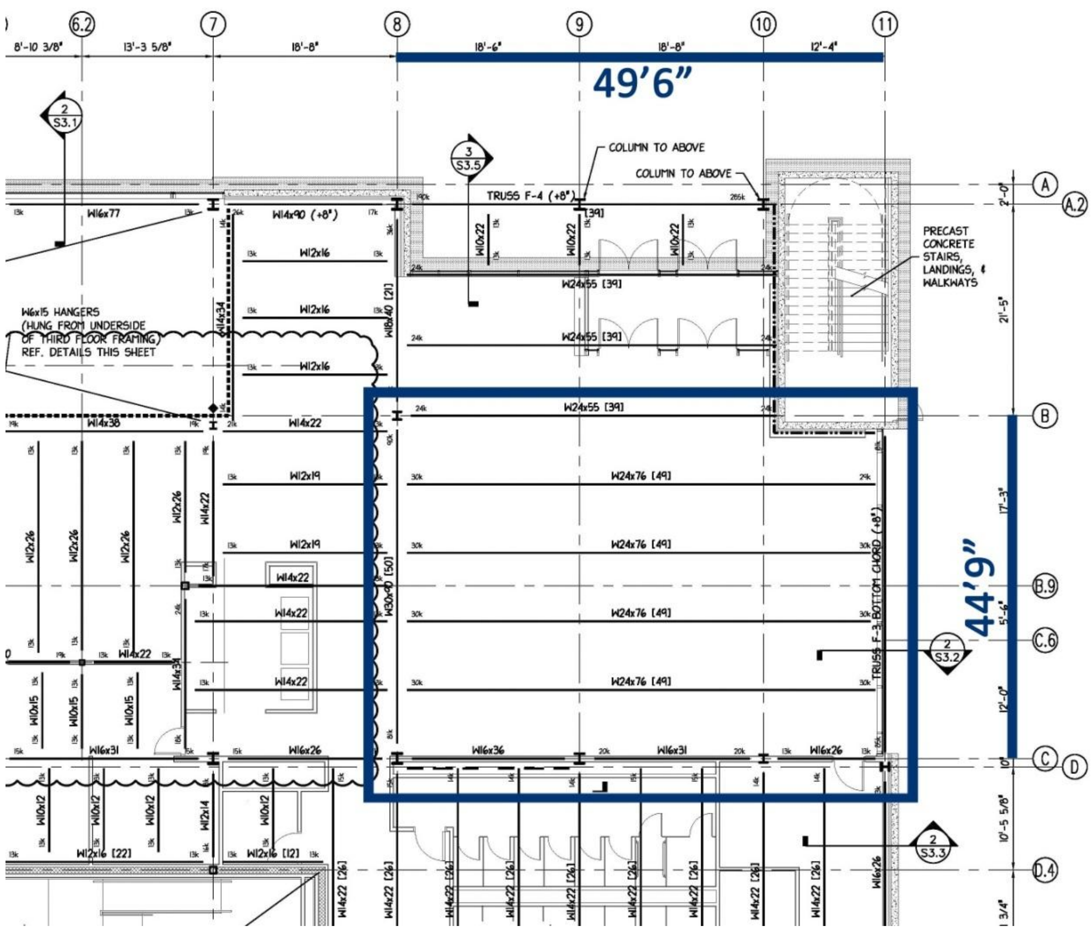


Figure 8: Second floor framing plan, with a representative bay of a typical frame, highlighted in blue, from S2.0

As explained above, this building has inconsistent framing from floor to floor, due to the variability in the space purposes. While no one framing plan is consistent throughout the building, a representative bay is highlighted in Figure 8. Structural framing plans for referenced floors are in Appendix 1. This bay is taken from the second floor, which uses the most consistent flooring and framing seen in other portions of the building and on the fourth floor and roofing plans.

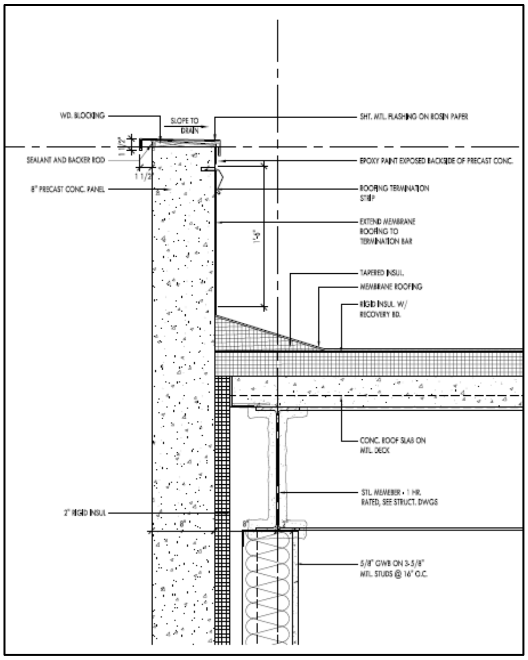


Figure 9 : Cross section of the roofing system.

The roof framing plan is similar to that of the third floor, both in layout of beams and supporting trusses. Typical beam members are W12x26s, with larger spans along the eastern side of the building leading to larger members.

Above all of the roof framing is the same finish, a fabric-reinforced Thermoplastic Polyolefin (TPO). This involves a light colored fully adhered roofing membrane on lightweight insulated concrete, lending to the LEED Silver status for the SSPAC. See Figure 9 for a cross section of the roof framing and system.

Supporting the floor systems is a combination of braced frames, columns, and precast panels. Columns are generally W12s, as the structural engineer focused on not only supporting the structure, but keeping the steel consistent dimensions. HSS columns were also used at varying locations, and varied from HSS4x4s to HSS10x10s.

Lateral System

The lateral system of this building varies per direction. In the North-South direction, the lateral system consists of shear walls. These shear walls are comprised of the precast concrete panels found along the exterior of the building, and highlighted in orange in Figure 10. These panels are 8" thick normal weight concrete and are anchored with L5x5x5/16" to the structure for deck support and into the foundation as discussed and detailed previously.

Braced frames along Column Line C in the East-West direction consist of the other component to the lateral framing system. These braced frames are highlighted in blue in Figure 10 and are comprised of W10x33s for diagonal members and W16x36s for horizontal members. An elevation of these lateral systems is included in Appendix 1.

The lateral loads on the structure first impact the exterior components and shear walls. Where braced frames are concerned, this load travels through the horizontal members into the diagonal and vertical members. These loads all then continue into the foundation.

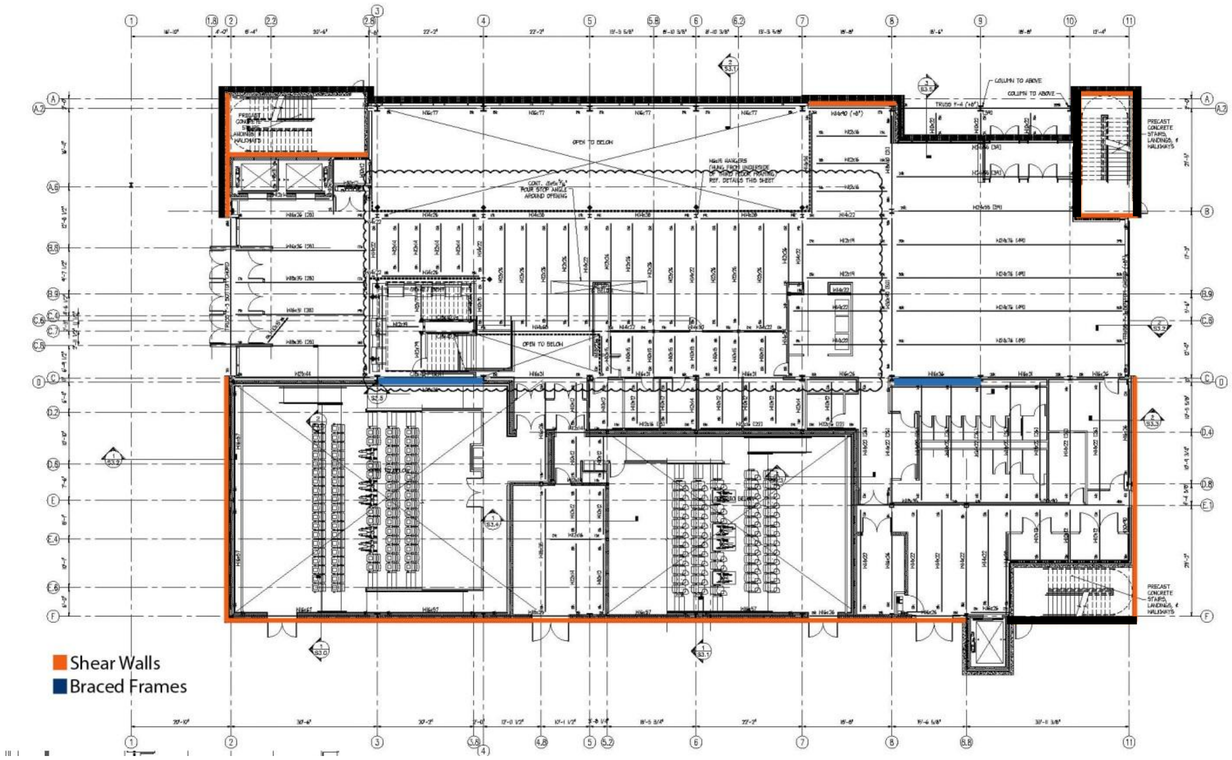


Figure 10 : Floor plan highlighting shear walls in orange and braced frames in blue, which contribute to the lateral system.

Design Codes

This section lists codes and design guides followed for the structural designs for the SSPAC, as well as applicable codes and design guides used throughout this report. Most recent code editions have been used for this report, and these differences should be noted below.

Design Codes:

- 2006 International Building Code (IBC 2006) with Local Amendments
- American Concrete Institute (ACI) 318-08, Specifications for Structural Concrete for Buildings
- American Concrete Institute (ACI) 530-2005, Building Code Requirements for Concrete Masonry Structures
- American Society of Civil Engineers (ASCE) 7-05, Minimum Design Loads for Buildings and Other Structures
- American Society of Civil Engineers (ASCE) 6-05, Specifications for Masonry Structures

Design Guides Used for Design:

- Steel Deck Institute (SDI), Design Manual for Floor Decks and Roof Decks
- Steel Deck Institute (SDI), Specifications for Composite Steel Floor Deck
- National Concrete Masonry Association (NCMA), Specifications for the Design and Construction of Load-Bearing Concrete Masonry

Thesis Codes & Design Guides:

- American Society of Civil Engineers (ASCE) 7-05, Minimum Design Loads for Buildings and Other Structures
- American Concrete Institute (ACI) 318-11, Specifications for Structural Concrete for Buildings
- American Institute of Steel Construction (AISC), Steel Construction Manual, 14th Edition
- Vulcraft Steel Decking Catalog, 2008

Materials

The following materials and their corresponding stress and strength properties have been listed below, as those used both in the existing building and for calculations for this report.

Concrete

Concrete slabs	f'c = 4000 psi @28 days
Reinforcing Bars Plain-Steel	f'c = 3000 psi
Other Concrete	fy = 60 ksi

Steel

W-Shapes	Fy = 50 ksi
Channels, Angles	Fy = 36 ksi
Plate and Bar	Fy = 36 ksi
Cold-formed hollow structural sections	Fy = 46 ksi
Hot-formed hollow structural sections	Fy = 46 ksi
Steel Pipe	Fy = 36 ksi

Other

Concrete Masonry Units	f'm = 1900 psi
Mortar, Type M or S	f'm = 2500 psi
Grout	f'm = 3000 psi
Masonry Assembly	f'm = 1500 psi
Reinforcing bars	Fy = 60 ksi

*Material properties are based on American Society for Testing and Materials (ASTM) standard rating.

Gravity Design Loads

This section details the provided designs loads for the SSPAC from the structural plans. Other loads have been derived as appropriate, with minimal differences in values calculated for this report and for initial design. It is noted that not all of these loads are applicable to the preceding comparisons, but have been included as a brief summary of the structural loadings.

Dead and Live Loads

Dead loads were not given on the structural drawings, and have therefore been assumed based on structural design textbooks. For a summary of the dead load values used in this report, see Table 11.

Superimposed Dead Loads	
Description	**Load (psf)**
Concrete Masonry Units (CMU)	91
Prefabricated Concrete Panels (8" thick)	100
Glazed Aluminum Curtain Walls	90
Roofing	30
Framing	7
MEP Allowance	5

Table 11 : Table of Superimposed dead loads

Conversely, the structural notes did provide partial live loads. These load values were compared with those found on Table 4-1 in American Society of Civil Engineers (ASCE) 7-05. As live loads on the plans are compiled to more overarching space divisions, other specific loads relevant to the building have been included for comparison in Table 12. One difference to note is the stage area on the third floor. If considered a stage floor by ASCE7-05, the loading here would be 150 psf. Yet, the structural drawings note all live loads, excluding mechanical, at 100 psf. This could be due to overestimating other spaces, such as theatre spaces, and using an average, yet still conservative, value. Live load reductions were not considered, as the SSPAC is considered under the "Special Occupancy" category, as a public assembly space, as per ASCE 7 -05 Chapter 4.8.4, and disallows the use of reduction factors on any live loads.

Live Loads*		
Space	**Structural Plan Load (psf)**	**Report Load (psf)**
Live Load	100	100
Corridor	100	100
Corridor, above 1st floor	---	80
Stairway	100	100
Mechanical Room/Light Manufacturing	125	125
Roof	30	20
Lobby	---	100
Theatre, stationary seating	---	60
Stage Floor	---	150
Restaurant/dining space	---	100
Balcony	---	100

Table 12: Table of live loads used on the structural plans and in this report.

*Dashes designate values not provided in the structural drawings.

Snow Loads

This section is a summary of the snow loads on the SSPAC; please see Technical Report I for a full expansion of these calculations.

Roof Snow Load Calculations	
Variable	**Value**
Roof Snow	30 + Snow Drift
Ground Snow - Pg	30 (psf)
Flat Roof Snow - Pf	30 (psf)
Terrain Category	B
Snow Exposure Factor - Ce	1.0
Snow Load Importance Factor - Is	1.2
Roof Thermal Factor - Ct	1.0
Roof Slope Factor -Cs	1.0

Table 13 : Summary of snow load variables.

The structural plans noted that the "Snow load controls roof design" and is therefore a primary focus of comparison in this section. The method of calculations follows ASCE 7-05, and factors used for the calculations are summarized in Table 13. The procedure for flat roofs was followed for the primary snow load of 30 psf, the value to be applied to the entire roof system, with drifts additional in certain areas.

With the height difference of 9.8 feet between the mechanical roof and the other roof and parapet heights, 5 locations on the mechanical roof were chosen for drift calculations. The magnitude of these drift heights led to an increase of the snow load from the base of 30 psf to 50 psf along the exterior 15 feet of the mechanical roof depression. Values assumed on the structural drawings coincide with the code allowances and results, reinforcing the statement that snow load controls roof design, with snow drifts being a primary concern on the mechanical roof. A summary of these results is given in Table 14.

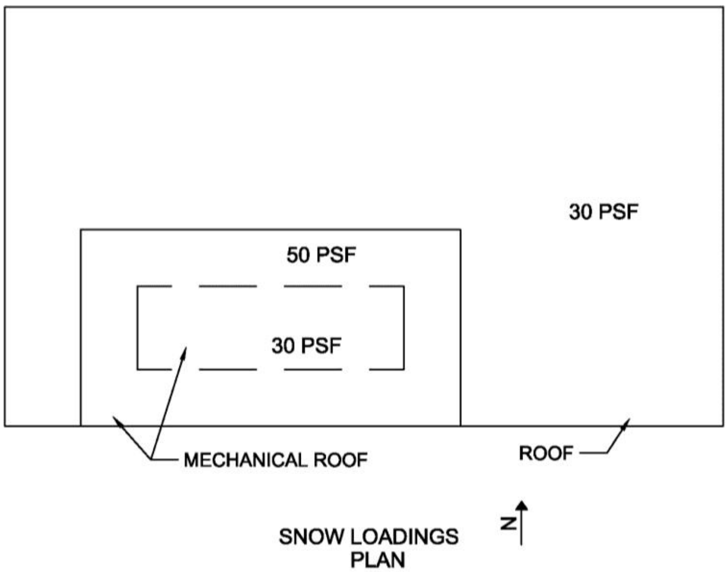


Figure 14 : Summary of snow loads.

Rain Loads

This section is a summary of the snow loads on the SSPAC; please see Technical Report I for a full expansion of these calculations.

Though rain load is not a determining load case for the SSPAC, the calculations for rain loads were followed, as a supplemental exercise in code interpretation and results, and as a preliminary step towards further analysis and discussion. Due to the roof slope being at the minimum allowance for not including ponding, rain loads needed only to be calculated for drainage system blocking. This procedure resulted in a rain load of 11 psf, and as compared to other roof loadings, did not control.

Lateral Loads

This section details the lateral loads that impact the structural system of the SSPAC, so that a more thorough understanding of the SSPAC would be obtained. For this report, both wind loads and seismic loads were calculated and applied to the model produced in RAM Structural System. Hand calculations for these load considerations can be found in Appendices 2 (Wind) and 3 (Seismic).

Wind

Wind loads were calculated using ASCE 7-05 Chapter 6, where Method 2 for Main Wind-Force Resisting Systems was applied to the structure. Due to the fact that the building is a low-rise building, with generally simple dimensions, this method was deemed appropriate. With this process of calculating the simplified design wind pressures, the dimensions of the building were simplified to the dimensions seen in Figure 15. The mechanical roof, realistically slightly lower than the rest of the roof, is surrounded by a parapet. With this scenario, the mechanical roof was considered to be at the same height at the adjoining roof for simplification and use of Method 2. Thus, the overall roof height is at an elevation of 64'0" relative to the ground.

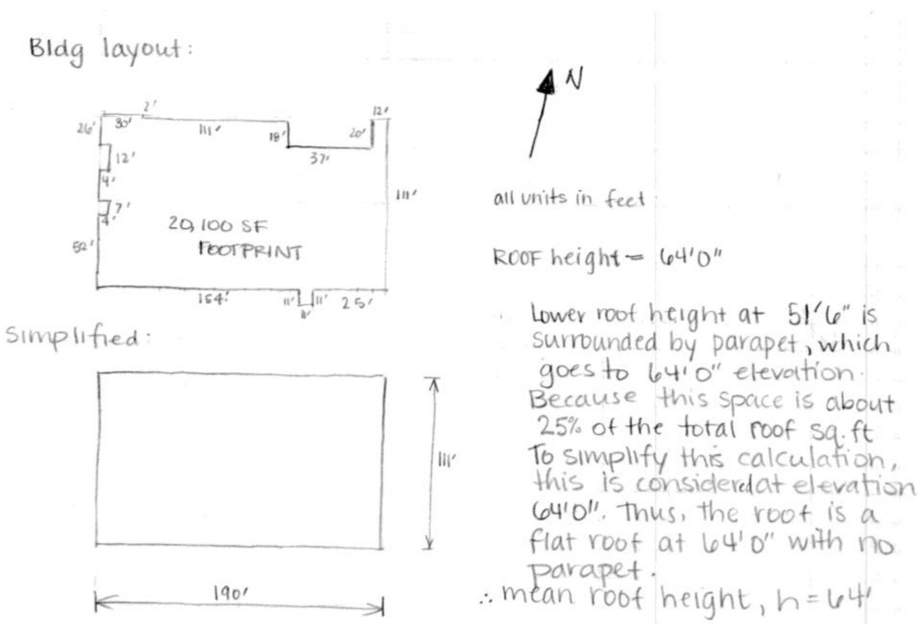


Figure 15 : Building dimensions simplified for wind load calculations following Method 2.

Calculations considered the wind coming along the East-West and North-South directions. The system is a rigid system, estimated by following the preferred method in the commentary of ASCE 7-05 Section C6. With this in mind, the gust effect factor was found to be .873 in the East-West direction and .853 in the North-South direction, which is slightly above the allowable minimum of G=.85 for rigid systems. Another portion of the calculations to highlight is the external pressure coefficient, Cp. This value varies per direction, as divided in Figure 6-6 of ASCE Chapter 6. A spreadsheet was formed for ease and accuracy of values for wind, and can be found in Appendix 2, along with the preceding hand calculations previously mentioned.

A summary of the wind pressures and variables going into these pressures in each direction are displayed below, in Figures 16 through 26. These results have been summarized for the East-West direction in Figures 16 through 20, and highlight the base shear and overturning moment due to these wind pressures. Figures 21 through 25 summarize similar results and drawings for the North-South direction. Table 26 gives a comparison of a summary of the loadings from each direction.

The structural drawings included input values and a total windward pressure. The input variables were compared with hand calculations and confirmed exact in most cases. For example, the maximum total windward pressure from the structural drawings was 38.9 psf, where the maximum value calculated below was 49.8 psf. The reason for these differences is that the value obtained by hand calculations did not include the internal pressures on the windward side, which would decrease the maximum loading seen.

The overall base shear for the East-West direction is 105.5 k, with an overturning moment of 3159 k-ft. These results can be compared with the North-South direction, where the base shear was higher, at 208.8 k, and the overturning moment at 6116 k-ft. When considering these results in relation to each other, and taking into account the building dimensions and direction, the proportion between building dimensions and base shear are fairly similar. Beyond the comparison between directions of the wind loading, these results, when considered in light of the building height and basic structure parameters, are reasonable values.

When finding the lateral loading on each floor due to the wind load, a factor of 1.6 was not applied, as per ASCE 7-05. The factor of 1.6 will be applied later for load combinations.

Wind Pressures East-West Direction									
Type		Location	Distance (ft)	Pressure Variables					Pressure
				Cp	qz	qh	G	GCpi	(psf)
Wall	Windward	Roof	64	0.8	17.63	17.63	0.873	0.18	12.31
		Floor 4	47.5	0.8	16.82	17.63	0.873	0.18	11.75
		Floor 3	35	0.8	14.80	17.63	0.873	0.18	10.33
		Floor 2	17.5	0.8	12.16	17.63	0.873	0.18	8.49
		Ground	0	0.8	10.05	17.63	0.873	0.18	7.02
	Leeward	All	All	-0.36	17.63	17.63	0.873	0.18	-5.54
	Side	All	All	-0.7	17.63	17.63	0.873	0.18	-10.77
Roof		0 to h/2	0 to 32	-0.9	17.63	17.63	0.873	0.18	-13.85
		h/2 to h	32 to 64	-0.9	17.63	17.63	0.873	0.18	-13.85
		h to 2h	64 to 128	-0.5	17.63	17.63	0.873	0.18	-7.70
		>2h	>128	-0.3	17.63	17.63	0.873	0.18	-4.62
							E-W load	Sum Wall	34.40
								Sum Roof	-40.02

Table 16 : Summary of wind pressure calculations in the East-West direction.

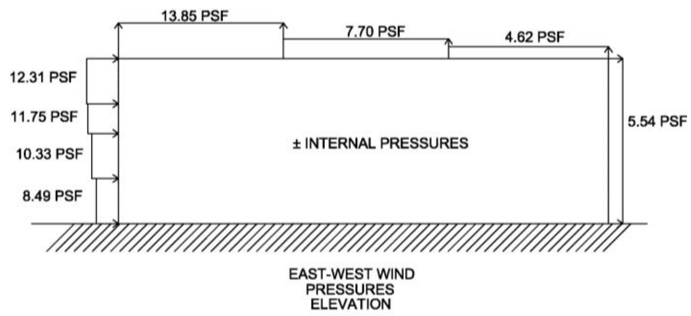


Figure 17: Summary of East-West wind pressures in elevation.

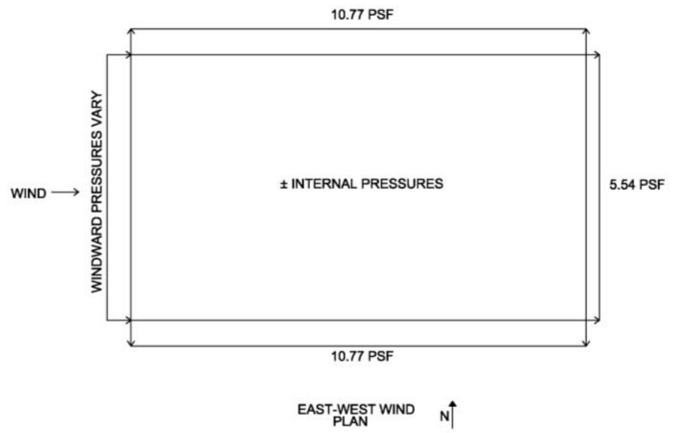


Figure 18: Summary of East-West wind pressures in plan.

	Overturning Moment/Base Shear East-West Direction								
Windward Wall	Location	Height	Area Below(ft^2)	Area Above (ft^2)	Pressure Below (psf)	Pressure Above (psf)	Story Load (k)	Story Shear (k)	Overturning Moment (k-ft)
	Roof	64	971.25	0	17.29	17.85	16.79	16.79	1075
	Floor 4	46.5	638.25	971.25	15.87	17.29	26.93	43.72	1252
	Floor 3	35	971.25	638.25	14.03	15.87	23.76	67.48	832
	Floor 2	17.5	971.25	971.25	12.56	14.03	25.83	93.31	452
	Ground	0	0	971.25	0	12.56	12.20	105.51	0
	Width (ft)	111				Total Base Shear (k):	105.51	Total Overturning	3159

Table 19 : Summary of overturning moment and base shear calculations in the East-West direction.

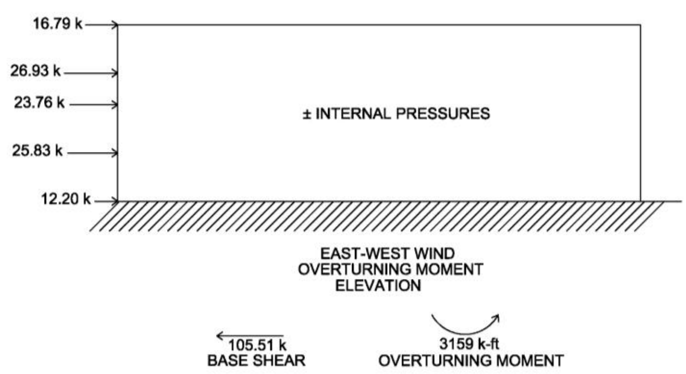


Figure 20 : Summary of final forces in East-West direction in elevation.

Wind Pressures North-South Direction									
Type		Location	Distance (ft)	Pressure Variables					Pressure
				Cp	qz	qh	G	GCpi	(psf)
Wall	Windward	Roof	**64**	0.8	17.63	17.63	0.853	0.18	12.03
		Floor 4	**47.5**	0.8	16.82	17.63	0.853	0.18	11.48
		Floor 3	**35**	0.8	14.80	17.63	0.853	0.18	10.10
		Floor 2	**17.5**	0.8	12.16	17.63	0.853	0.18	8.30
		Ground	**0**	0.8	11.55	17.63	0.853	0.18	7.88
	Leeward	All	**All**	-0.5	17.63	17.63	0.853	0.18	-7.52
	Side	All	**All**	-0.7	17.63	17.63	0.853	0.18	-10.53
Roof		0 to h/2	**0 to 32**	-1.0	17.63	17.63	0.853	0.18	-15.04
		h/2 to h	**32 to 64**	-0.8	17.63	17.63	0.853	0.18	-12.03
		h to 2h	**64 to 128**	-0.5	17.63	17.63	0.853	0.18	-7.52
		>2h	**>128**	N/A	17.63	17.63	0.853	0.18	N/A
							N-S load	Sum Wall	49.79
								Sum Roof	-34.59

Table 22: Summary of wind pressure calculations in the North-South direction.

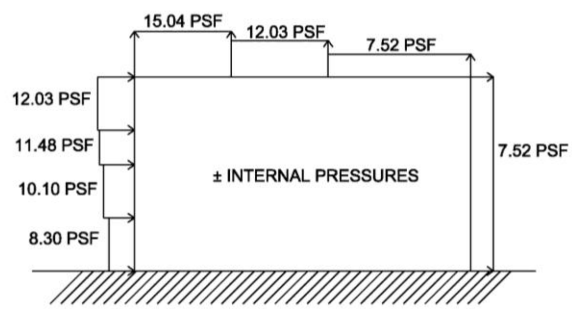


Figure 21: Summary of forces in the North-South direction in elevation.

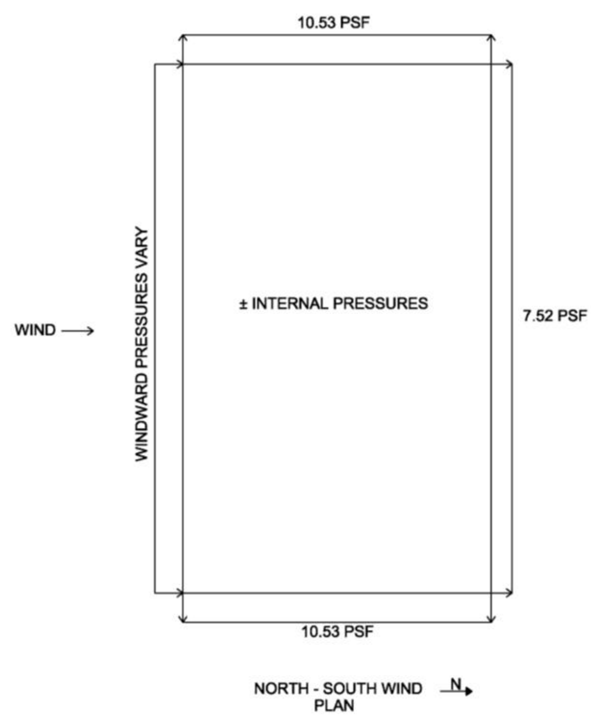


Figure 23: Summary of pressures in the North-South direction in plan.

Overturning Moment/Base Shear North-South Direction									
	Location	Height	Area Below (ft^2)	Area Above (ft^2)	Pressure Below (psf)	Pressure Above (psf)	Story Load (k)	Story Shear (k)	Overturning Moment (k-ft)
Windward Wall	Roof	64	1662.5	0	19.00	19.55	31.59	31.59	2022
	Floor 4	46.5	1187.5	1662.5	17.62	19.00	52.51	84.09	2442
	Floor 3	35	1662.5	1187.5	15.82	17.62	47.22	131.31	1653
	Floor 2	17.5	1662.5	1662.5	15.40	15.82	51.91	183.22	908
	Ground	0	0	1662.5	0	15.40	25.61	208.82	0
						Total Base Shear (k):	208.82	Total Overturning	6116
	Width (ft)	190							

Table 23 : Summary of overturning moment and base shear calculations in the North-South direction.

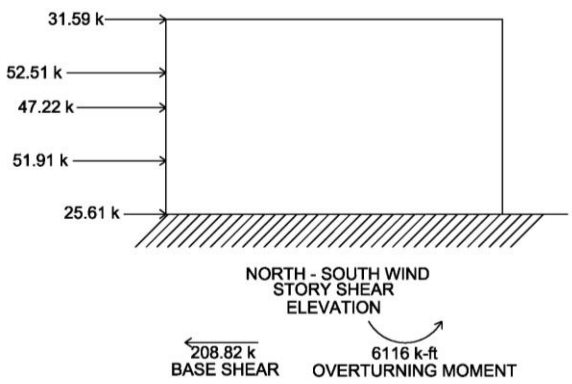


Figure 24 : Summary of final forces in North-South direction in elevation.

Wind Loads Per Floor - Summary					
Level	Height	North-South Direction		East-West Direction	
		Total Force (k)	Story Shear (k)	Total Force (k)	Story Shear (k)
Roof	**64**	31.59	31.59	16.79	16.79
4th	**46.5**	52.51	84.09	26.93	43.72
3rd	**35**	47.22	131.31	23.76	67.48
2nd	**17.5**	51.91	183.22	25.83	93.31

Table 25: Hand calculations for wind loads per floor

Seismic

Seismic calculations followed ASCE 7-05 Chapters 11 and 12, and used the Equivalent Lateral Force Procedure, which is also the method used for the structural plan designs. This procedure included the variables listed in Table 27, some of which were taken from the geo-technical report, while others were calculated. The calculations related to these variables and results are presented in Appendix 3. The lateral system for the SSPAC in the East-West direction is a braced-frame and shear wall system, while in the North-South direction, it is a shear wall system comprised of the precast concrete panels seen on the exterior of the building. This needed to be considered for certain variables, such as the response modification coefficient.

Variable	Value
S_s	1.5
S_1	0.26
Site Class	D
Sds	1.06
S_{D1}	0.28
Cd	3
T_s	0.347
Ta	0.6788
C_u	1.7
T	1.15
T_L	6
$C_{s\,(limit)}$	0.042

Figure 26: Table of seismic load variables and values.

Values calculated from this report were compared with those on the structural drawings; all values are exact excluding C_s. For this value, the structural drawings denote C_s=0.138, while the calculated value as C_s=0.140 before applying Section 12.8.1-1, which limits this value at 0.042. This maximum value of C_s was implemented for seismic calculations.

Once these values were obtained, the base shear needed to be calculated using V=C_s*W. The structure's weight, W, was estimated by hand, incorporating all dead weight, slab and framing weight, CMU walls, precast panels, and curtain walls supported by the structure. These calculations can be found in more detail in Appendix 3. This value for the building weight, W=11750 kips, when compared with those calculated by the engineer, were found to be off by less than 10%.

Using the values of C_s=0.042 and the building weight, W=11750 kips, were found, the base shear could then be calculated. The base shear calculated in this report is V=493.5 kips, with an overturning moment of approximately 63925 k-ft, as elaborated on in Table 28 and summarized in Figure 29. Structural drawing S2.8 denotes a base shear value, V=506.5 kips. The calculated base shear is only 2% lower than the value on the structural drawings. This minor difference in base shear can be attributed to the estimating required in hand calculations, while the structural engineer used a structural program to calculate the building weight. These calculations and values can be seen in Appendix 3. Accidental torsion impacted the seismic loads, and these values can be found later in this report.

Seismic Forces							
Level	Story Weight, w_x (lbs)	Story Height, h_x (ft)	$w_x h_x^k$	C_{vx}	Story Force (k) $F_x = C_{vx} * V$	Story Shear (k)	Overturning Moment (k-ft)
Roof	2731120.0	64	689,541,085	0.407	200.8	200.8	12850
Mech Roof	35934	51.5	6,795,309	0.004	2.0	202.8	10442
Floor 4	2598740.0	47.5	441,331,912	0.260	128.5	331.3	15735
Floor 3	4047240.0	35	457,898,750	0.270	133.3	464.6	16261
Floor 2	2206440.0	17.5	99,296,222	0.059	28.9	493.5	8637
Ground	N/A	0	N/A	N/A	N/A	N/A	N/A
Cs	0.042			Base Shear [V=Cs*W] (k)			493.5
W(k)	11750			Total Overturning Moment (k-ft)			63925

Table 27 : Summary of calculations for seismic load design.

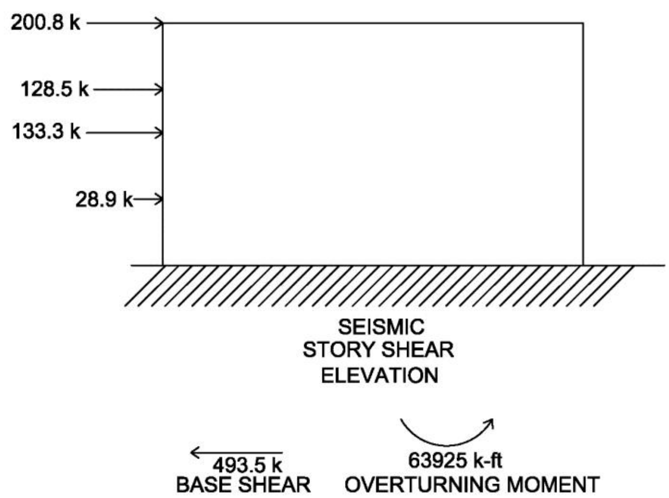


Figure 28 : Summary of forces due to seismic loads.

Comparison of Lateral Forces

When applying loads to the building, it was necessary to establish whether wind or seismic controlled. Comparisons of the factored wind and seismic loads follow in Tables 30 and 31. This comparison concluded that seismic loads controlled for base shear and loading on the upper individual floors, while Wind in the North-South direction controlled the overturning-moment and level 2. This can be explained by the seismic load correlation with height and weight of controlling lateral components.

In designing the structural components, the base shear and overturning moment will be important for the design of columns and shear walls. Story shears will be important for designing braces and the loads within them. The distribution of loads per member and confirmation of designed structural components will be discussed in more detail in the Lateral System Analysis section of this report.

Comparison of Lateral Forces			
	Wind, North-South	Wind, East-West	Seismic
Base Shear (k)	208.8	105.5	***493.5***
Overturning Moment (k-ft)	***6115.7***	3158.5	63925.2

Table 29: Comparison of lateral forces

Comparison of Story Shears (k)			
Level	Wind, North-South	Wind, East-West	Seismic
Roof	31.6	16.8	***200.8***
Mech Roof	Neglible	Neglible	***202.8***
Floor 4	84.1	43.72	***331.3***
Floor 3	131.31	67.48	***464.6***
Floor 2	183.22	93.31	***493.5***
Ground	N/A	N/A	N/A

Table 30: Comparison of story shears

Lateral System Analysis

After preliminary analysis of the structural system and loadings were completed, a thorough analysis of the lateral system was performed on the SSPAC. This was accomplished through the use of a RAM Structural System model in parallel with hand calculations as a verification of the output from RAM. These hand calculations were also paired with additional modeling in SAP2000 for displacement and stiffness verification. A more thorough description of how these models were utilized in the lateral analysis follows below.

RAM Model

The RAM model incorporated into the analysis of the lateral systems of the SSPAC allowed for several assumptions that impacted the results obtained from the model. The theory and code behind these assumptions dictated more accurate analysis results. These assumptions are as follows:

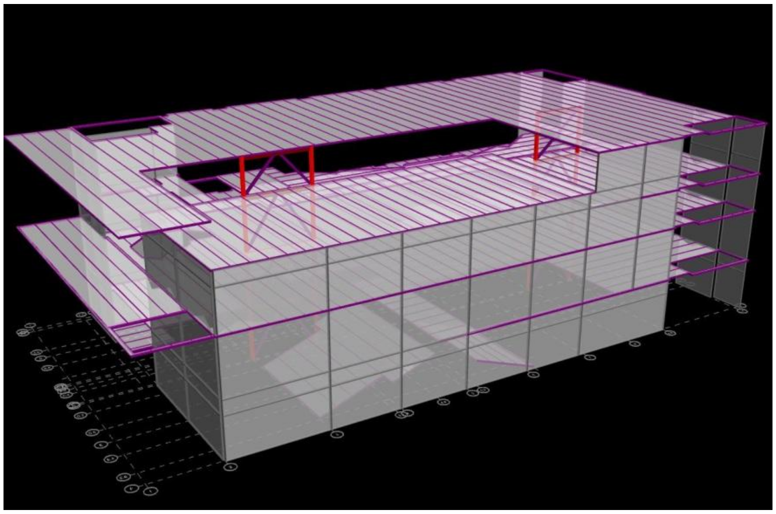


Figure 31: RAM lateral model from the Northeast corner of the building.

For the concrete shear walls, cracked sections were considered, as per ACI §8.8.2, wherein the gross section was minimized to 70% to model the area for the cracked section. The lateral system components modeled in SAP200 incorporated this rule as well. The shear walls were also meshed, with a membrane comprised of 96”x96” mesh. SAP models were utilized to confirm max displacements of walls, as well as stiffnesses of each component, as described below.

As can be noticed when comparing the model, as seen in Figure 32, and the structural drawings, which can be viewed in Appendix 1, only the precast walls resisting lateral forces were considered part of the RAM model. The use of the selected shear walls was confirmed with the project structural engineer at the onset of the analysis for this report.

P-Delta effects were considered in the lateral analysis, as required by chapters 12 (seismic) and 6 (wind) in ASCE 7-05.

Horizontal and diagonal structural components in the braced frames were given moment releases, to ensure that these members only saw axial forces as designed per the structural drawings.

Through the study of the structural drawings, pinned connections for the bases of the braced frames and shear walls were considered appropriate for the modeling. Due to the fact that the walls were precast concrete panels, the connection to the foundation was assumed to be pinned.

Floor systems were input into RAM as a rigid diaphragm, which guaranteed that all points would deflect together.

Figures 32 and 33 display the concentrically braced frame and shear walls modeled with rigid diaphragms in RAM.

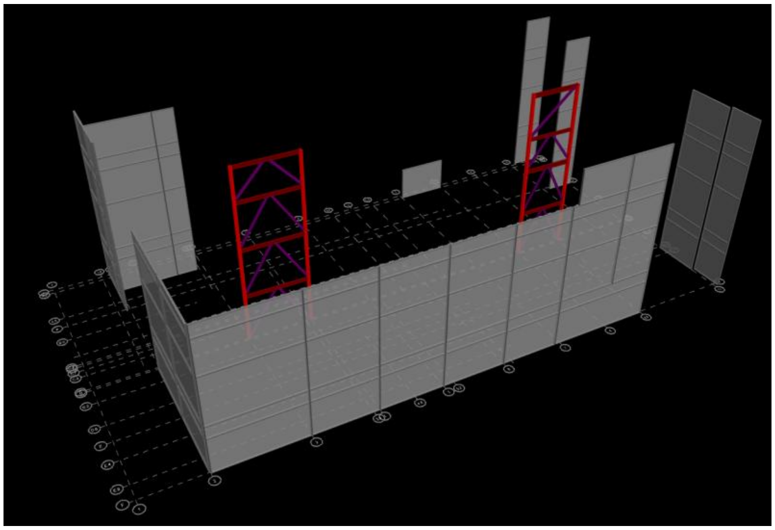


Figure 32: RAM model showing lateral system without diaphragms.

Building Properties

To confirm the accuracy of the RAM model for proper analysis, hand calculations were completed and compared with the output of the model before analyses were completed. These hand calculations included the center of mass, center of rigidity, load distribution, and torsion. The third floor, a typical floor for the lateral system, was considered for the hand calculated verification of the building properties.

Center of Rigidity & Center of Mass

The center of mass was found by first calculating the weight of the slab and central location of it, as well as the weight and location of shear walls. Braced frames were not considered in this calculation as a simplification, due to the symmetry of them, as can be seen in the floor plan in Figure 34. These weights and locations were then utilized in the equation for center of mass, where d = ∑(m*di)/ ∑m, with d being the direction considered. The hand calculations for the Y-Direction differed from RAM by 1.8% and those in the X-Direction by 5.9%. These are denoted by "X" in red and blue on the floor plans. These differences are off by less than 10% and are therefore acceptable. The values for the center of mass can be found in Table 36. The differences in these values can be allotted to the neglecting of the braced frames in hand calculations. These calculations and adjoining spreadsheets can be found in Appendix 4.

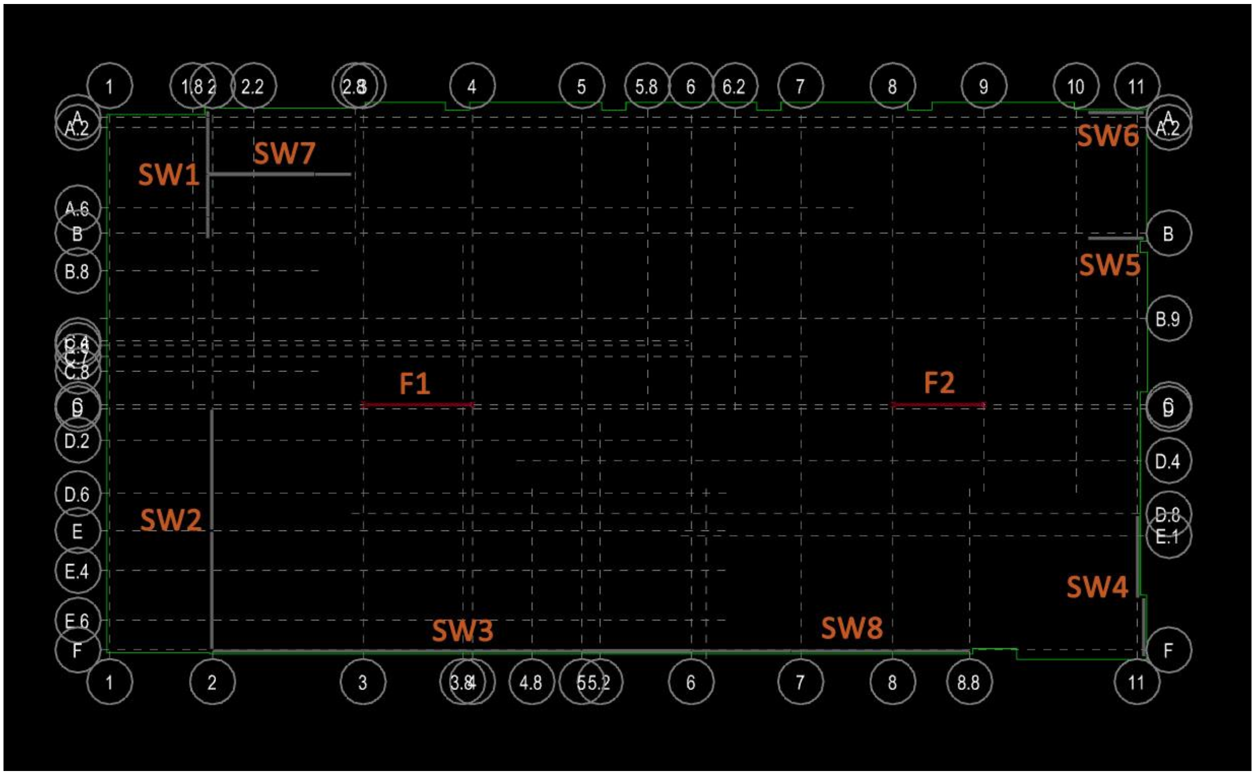


Figure 33: Floor plan displaying frame and shear wall designations.

Before finding the center of rigidity, the relative stiffness of each member needed to be confirmed and calculated. The relative stiffness of a member is related to the total shear it takes in relation to the total force applied at the level being considered. To find the stiffness of each member, a 1000 k load was applied at the center of mass at the roof level. This gave the shear in each wall, which was used in the spreadsheets seen in Appendix 4, to calculate the relative stiffness of each lateral member, which can be seen in Tables 35 and 36. Noting the stiffness of shear wall 2, the 4th floor relative stiffness can be explained by the lack of connectivity of the diaphragm to shear wall 2 (it connects to the mechanical roof). Other stiffnesses for shear wall 2 display how the shear wall is the largest mass in the Y-direction. These values from RAM were again confirmed by the modeling of each lateral member in SAP200 with the same load and stiffness calculation procedure. The spreadsheets for these values can also be found in Appendix 4.

Relative Stiffness by % of Total Direct Shear in Y-Direction			
Floor	SW1	SW2	SW4
Roof	66.8	---	31.6
Mech Roof	66.8	69.4	31.6
4th	81.0	6.9	10.8
3rd	15.0	73.2	11.8
2nd	4.6	73.2	23.1
Ground	4.0	73.2	23.1

Table 34: Relative stiffness in Y-Direction

Relative Stiffness by % of Total Direct Shear in X-Direction						
Floor	SW3	SW5	SW6	SW7	F1	F2
Roof	35.6	3.2	3.5	52.6	4.7	0.3
Mech Roof	38.0	3.3	3.6	52.9	2.3	0.0
4th	42.0	3.1	3.4	46.2	2.3	3.0
3rd	49.9	1.2	0.6	48.1	2.0	1.9
2nd	56.4	5.1	6.2	28.0	6.1	5.0
Ground	56.4	5.1	6.2	28.0	3.4	3.1

Table 35: Relative stiffness in X-Direction

From the relative stiffnesses calculated, it was then noted which shear walls and braced frames would take a higher amount of the lateral load, and how this load was transferred. In the X-direction, shear walls 3 and 7 saw the most lateral force. This is a reasonable answer, as shear walls 3 and 7 were the largest two lateral system components in the X-Direction. In the Y-direction, shear wall 1 saw more of the force from the fourth floor and above, whereas shear wall 2 saw more force below. As the model displays, shear wall 2 was the largest shear wall in the Y-Direction, and therefore took more load on the floors that it makes a contribution. Shear wall 2 did not contribute to the lateral load distribution on either the fourth floor or on the roof, as it connected to the mechanical roof, which was located four feet above the fourths floor. Therefore, the relative stiffnesses of these shear walls received from the calculations previously explained were confirmed reasonable.

These stiffnesses were utilized in the center of rigidity equation, and compared with the center of rigidity values found by RAM. The center of rigidity of the SSPAC was found in hand calculations via the use of the equation d = ∑[R*di]/∑R, with d being the direct of consideration and R as the stiffness of the structural component. These hand calculated values varied from the values obtained from RAM by only 8%, and therefore were found satisfactory. The differences in these values can be explained by the RAM model assuming the mass evenly distributed on the level.

Center of Mass & Center of Rigidity				
Level	**CM - X**	**CM - Y**	**CR - X**	**CR-Y**
Roof	-95.88	65.19	-165.48	23.95
Mech Roof	-114.38	36.00	-180.28	2.89
4th	-80.86	43.29	-166.86	16.39
3rd	-100.93	48.63	-146.50	14.17
2nd	-84.43	50.04	-146.65	17.83
1st	-99.21	13.85	-99.21	13.85

Table 36: Center of mass and center of rigidity

A summary of the results for the center of mass and center of rigidity can be seen in Table 37. Figure 38 displays the center of mass and center of rigidity located on the second floor plan. Other floor plans and their respective values can be found in Appendix 4.

The percent stiffnesses, center of rigidity, and center of mass values of each floor will be utilized to calculate the forces found in each lateral component as part of the discussion of individual members, found later on in this report.

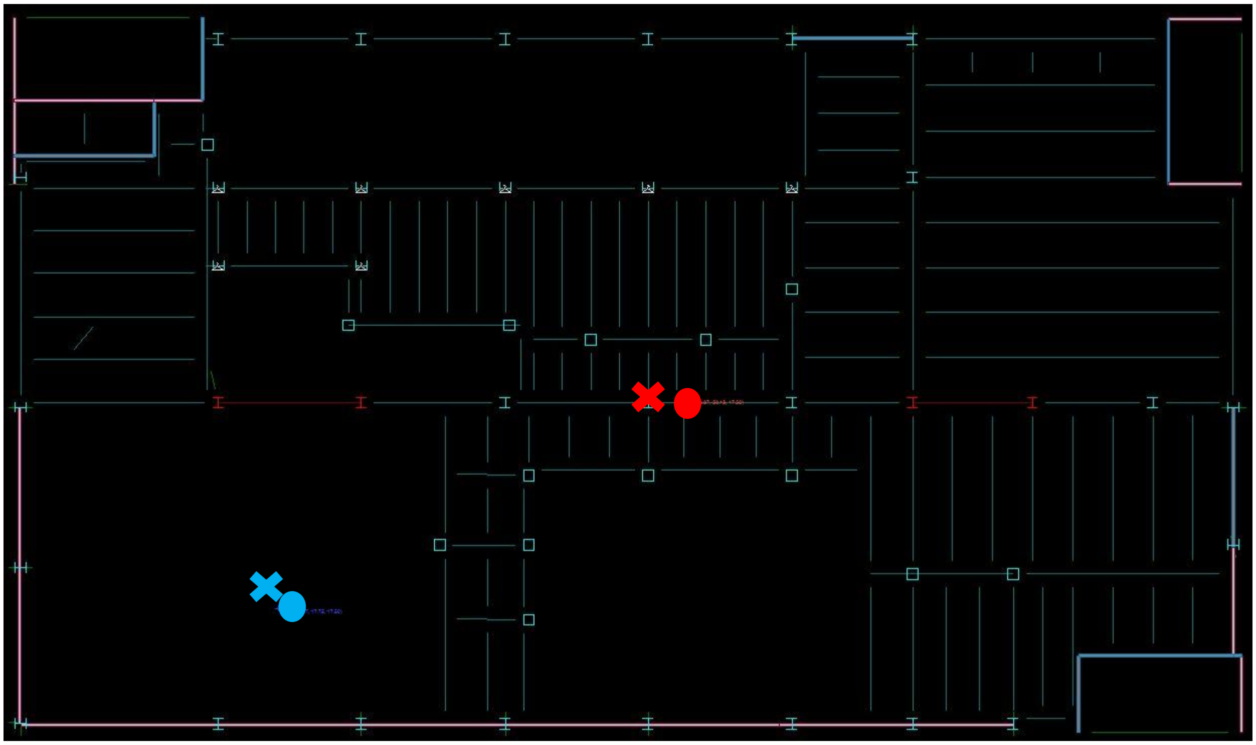


Figure 37: Second floor plan displaying center of rigidity (blue) and center of mass (red).

Vertical structural irregularities were considered for the SSPAC, and the applicable ones from ASCE 7-05 Table 12.3-2 are highlighted below in Figure 39. Neither in-plane discontinuities nor weak story irregularities are an issue in the SSPAC, and have been confirmed to not exist. Therefore, vertical structural irregularities do not apply to the structure.

TABLE 12.3-2 VERTICAL STRUCTURAL IRREGULARITIES

	Irregularity Type and Description	Reference Section	Seismic Design Category Application
1a.	**Stiffness-Soft Story Irregularity** is defined to exist where there is a story in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above.	Table 12.6-1	D, E, and F
1b.	**Stiffness-Extreme Soft Story Irregularity** is defined to exist where there is a story in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.	12.3.3.1 Table 12.6-1	E and F D, E, and F
2.	**Weight (Mass) Irregularity** is defined to exist where the effective mass of any story is more than 150% of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.	Table 12.6-1	D, E, and F
3.	**Vertical Geometric Irregularity** is defined to exist where the horizontal dimension of the seismic force–resisting system in any story is more than 130% of that in an adjacent story.	Table 12.6-1	D, E, and F
4.	**In-Plane Discontinuity in Vertical Lateral Force-Resisting Element Irregularity** is defined to exist where an in-plane offset of the lateral force-resisting elements is greater than the length of those elements or there exists a reduction in stiffness of the resisting element in the story below.	12.3.3.3 12.3.3.4 Table 12.6-1	B, C, D, E, and F D, E, and F D, E, and F
5a.	**Discontinuity in Lateral Strength–Weak Story Irregularity** is defined to exist where the story lateral strength is less than 80% of that in the story above. The story lateral strength is the total lateral strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	12.3.3.1 Table 12.6-1	E and F D, E, and F
5b.	**Discontinuity in Lateral Strength–Extreme Weak Story Irregularity** is defined to exist where the story lateral strength is less than 65% of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	12.3.3.1 12.3.3.2 Table 12.6-1	D, E, and F B and C D, E, and F

Figure 38: Table 12.3-2 from ASCE 7-05, highlighting applicable vertical structural irregularities.

Torsion

After confirmation of the RAM model's accuracy, torsion was then considered. Noting the differences in the center of mass and center of rigidity, it could be seen that torsion would impact the structure, with an average of 58 feet difference in the X-Direction and 28 feet difference in the Y-Direction of each of the stories.

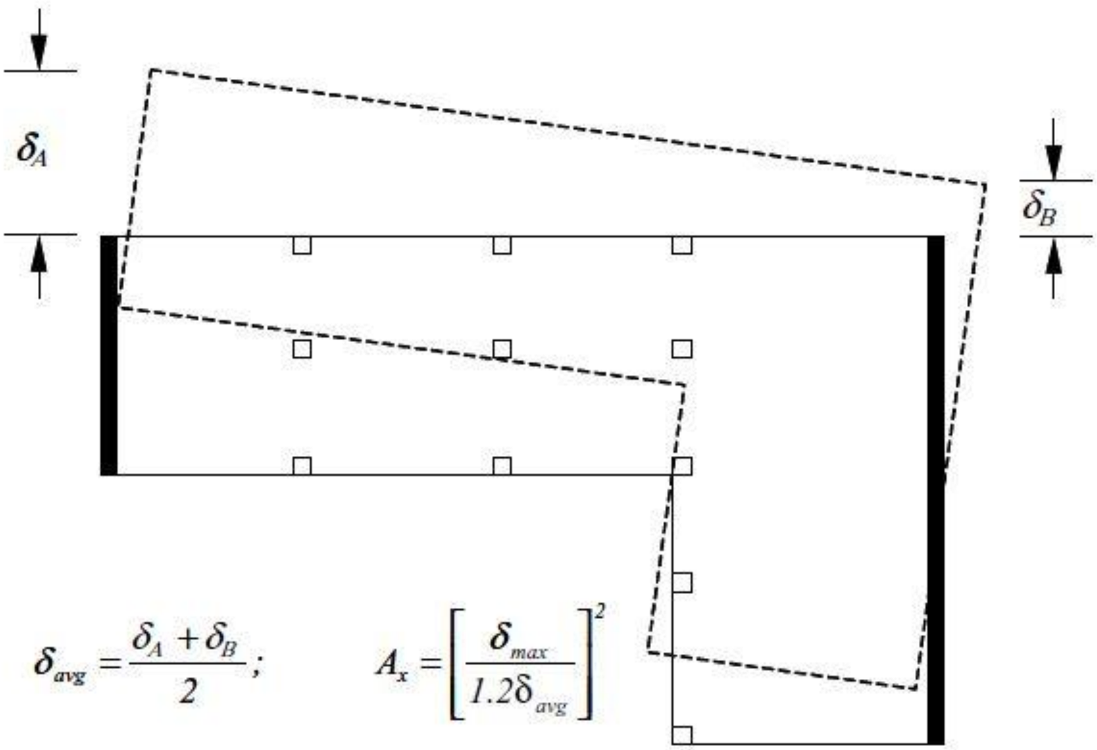


FIGURE 12.8-1 TORSIONAL AMPLIFICATION FACTOR, A_x

Figure 39: Torsional amplification, ASCE 7-05 chapter 12.

The RAM model considered a 5% eccentricity, but hand calculations were implemented to establish the need for use of the torsional amplification factor. The method utilized for this procedure is outlined in ASCE 7-05, and Figure 40 displays the equations for finding the amplification factors. The model was first run assuming Ax=1.0 to find the initial moment and displacements. These values were then applied to find the amplification factors at each story, in both X and Y-Directions. It was found that the amplification factor in the X-Direction continued at 1.0, as is understood through the fairly regular geometry and shorter cross section in this direction. In the Y-Direction, extreme torsional irregularity was found, and each of the corresponding amplification factors was then applied to recalculate the moment that was then reapplied to the SSPAC RAM model. Extreme torsional irregularity is a horizontal irregularity applicable to the SSPAC, as highlighted in Figure 42. Torsional irregularity in the Y-Direction is a result of the longer building cross section, large moment arm produced by the center of rigidity, and the irregularity of the geometry. A summary of these results can be seen in Table 41, with detailed hand calculations found in Appendix 4.

X-Direction Accidental Torsion				Y-Direction Accidental Torsion				Resulting Moment and Bx'	
By	5% By	Ax	Mzx	Bx	5% Bx	Ay	Mzy	Mzy'	Bx'
111	5.55	1.0	1114.3	190	9.5	1.4	1907.4	2737.1	288.1
111	5.55	1.0	11.0	190	9.5	1.4	18.8	27.2	2.9
111	5.55	1.0	713.2	190	9.5	1.5	1220.8	1781.6	187.5
111	5.55	1.0	740.0	190	9.5	1.5	1266.7	1917.8	201.9
111	5.55	1.0	160.5	190	9.5	1.6	274.7	437.3	46.0
111	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Overturning Moment (ft-k)			2739.0	Overturning Moment (ft-k)			4688.4	Overturning Moment (ft-k)	6901.0

Figure 40: Torsional amplification factors applied

TABLE 12.3-1 HORIZONTAL STRUCTURAL IRREGULARITIES

	Irregularity Type and Description	Reference Section	Seismic Design Category Application
1a.	**Torsional Irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.4 12.8.4.3 12.7.3 12.12.1 Table 12.6-1 Section 16.2.2	D, E, and F C, D, E, and F B, C, D, E, and F C, D, E, and F D, E, and F B, C, D, E, and F
1b.	**Extreme Torsional Irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.1 12.3.3.4 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 Section 16.2.2	E and F D B, C, and D C and D C and D D B, C, and D
2.	**Reentrant Corner Irregularity** is defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
3.	**Diaphragm Discontinuity Irregularity** is defined to exist where there are diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50% of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50% from one story to the next.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
4.	**Out-of-Plane Offsets Irregularity** is defined to exist where there are discontinuities in a lateral force-resistance path, such as out-of-plane offsets of the vertical elements.	12.3.3.4 12.3.3.3 12.7.3 Table 12.6-1 16.2.2	D, E, and F B, C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F
5.	**Nonparallel Systems-Irregularity** is defined to exist where the vertical lateral force-resisting elements are not parallel to or symmetric about the major orthogonal axes of the seismic force–resisting system.	12.5.3 12.7.3 Table 12.6-1 Section 16.2.2	C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F

Figure 41: Horizontal structural irregularities, ASCE 7-05 Table 12.3-1

Lateral Results

Once the model was completed and verified through hand calculations, the building was analyzed for controlling loads, drift, and the impact of torsion and foundations, with member checks performed as a final confirmation of the adequacy of the structural system design.

Load Combinations

First, load combinations from ASCE 7-05 §2.3.2 were evaluated to conclude which load cases would control for further lateral analysis. The load cases are as follow:

1. 1.4(D + F)
2. 1.2(D +F + T) + 1.6(L + H) + 0.5(L_r or S or R)
3. 1.2D + 1.6(L_r or S or R) + (L or 0.8W)
4. **1.2D + 1.6W + L + 0.5(L_r or S or R)**
5. **1.2D + 1.0E + L + 0.2S**
6. 0.9D + 1.6W + 1.6H
7. 0.9D + 1.0E + 1.6H

These load cases were considered, and it could be seen that load combination 4 controls for wind, with a 1.6 factor. Load combination 5 controls for seismic loads, with a 1.0E factor. The lateral forces that control per floor, with these load cases applied, are compared in Table 43. As is shown, seismic controls for all the floors for these story shears. This is explained through the fact that seismic loading is related to height and mass, and the SSPAC has been designed with the heavier upper floors, as can be understood through an example of the mechanical roof. This is important to note for designing of individual members per each floor, as discussed below.

Comparison of Factored Story Shears (k)			
Level	**Wind, North-South**	**Wind, East-West**	**Seismic**
Roof	50.5	26.9	***200.8***
Mech Roof	Neglible	Neglible	***202.8***
Floor 4	134.5	69.95	***331.3***
Floor 3	210.1	107.97	***464.6***
Floor 2	293.1	149.29	***493.5***
Ground	N/A	N/A	N/A

Table 42: Comparison of story forces.

Story Drift

Next, displacements and story drifts were computed for both wind and seismic loads and compared against the allowable deflections as per the respective portions of ASCE 7-05.

Wind

The story drifts and displacements due to the lateral wind load were compared against the allowable drift, using the rule of thumb H/400, as per ASCE 7-05 Chapter C Appendix C. The four load cases required for analysis by ASCE 7-05 can be viewed in Appendix 5, and are summarized in Table 43.

Design Wind Load Cases		
W1	Load Case 1	X Direction Only
W2	Load Case 1	Y Direction Only
W3	Load Case 2	X with E
W4	Load Case 2	X with -E
W5	Load Case 2	Y with E
W6	Load Case 2	Y with -E
W7	Load Case 3	X + Y
W8	Load Case 3	X - Y
W9	Load Case 4	X + Y with CW
W10	Load Case 4	X + Y with CCW
W11	Load Case 4	X - Y with CW
W12	Load Case 4	X - Y with CCW

Table 43: Wind load cases, as applied in model.

Displacement values for each of the load combinations were compared, and displacements that controlled are summarized in Table 44. All displacements for each level and for each load case were confirmed as passing. It was noted that the building, being stiff and only 4 stories, would have low displacements. The RAM model was confirmed via the modeling of lateral components in SAP2000, as discussed previously, and inputting hand-calculated loadings per floor. These additional calculations can be found in greater detail in Appendix 4.

Wind Drift & Displacement										
				Story Drift, Δ			Total Displacement, δ			
	Level	Story Height	h_{sx}	Story Drift, Δ (in)	Δmax, rel (in) = h/400	Δ < Δmax	Total Displ, δ (in)	δmax, rel (in) = h/400	δ < δmax	Controlling Load Case
X Direction	Roof	64	12.5	0.01153	0.375	YES	0.03113	1.920	YES	W11
	Mech Roof	51.5	16.5	0.00562	0.495	YES	0.0196	1.545	YES	W8
	4th	47.5	12.5	0.00317	0.375	YES	0.01398	1.425	YES	W8
	3rd	35	17.5	0.00666	0.525	YES	0.01081	1.050	YES	W11
	2nd	17.5	17.5	0.00415	0.525	YES	0.00415	0.525	YES	W11
				Story Drift, Δ			Total Displacement, δ			
	Level	Story Height	h_{sx}	Story Drift, Δ (in)	Δmax, rel (in) = .015 h_{sx}	Δ < Δmax	Total Displ, δ (in)	δmax, rel (in) = .015 h_{sx}	δ < δmax	Controlling Load Case
Y Direction	Roof	64	12.5	0.03138	0.188	YES	0.13247	0.960	YES	W2
	Mech Roof	51.5	16.5	0.00707	0.248	YES	0.10109	0.773	YES	W2
	4th	47.5	12.5	0.03941	0.188	YES	0.09402	0.713	YES	W2
	3rd	35	17.5	0.0352	0.263	YES	0.05461	0.525	YES	W2
	2nd	17.5	17.5	0.01941	0.263	YES	0.01941	0.263	YES	W2

Table 44: Controlling displacements for wind story drift & displacements.

Seismic

Allowable seismic story drift, as per ASCE 7-05 Table 12.12-1, under Occupancy Category IV, allows for maximum deflection of $\Delta_a = 0.015h_{sx}$, as shown in Figure 45. Because the structure had significant torsion, as seen in the calculation of the amplification factors previously in this report, these torsional effects were included when finding maximum drift values. For seismic drift considerations, as per chapter 12, a factor of Cd/I was applied to drift, where Cd=3 and I=1.5. These results were controlled by the amplification factor, and can be seen in Table 46.

TABLE 12.12-1 ALLOWABLE STORY DRIFT, Δ_a[a,b]

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

[a] h_{sx} is the story height below Level x.
[b] For seismic force–resisting systems comprised solely of moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.
[c] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived.
[d] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

Figure 45: Allowable story drift, ASCE 7-05 Table 12.12-1

X Direction				Story Drift, Δ			Total Displacement, δ		
	Level	Story Height	h_{sx}	Story Drift, Δ (in)	Δmax, rel (in) = .015 h_{sx}	Δ < Δmax	Total Displ, δ (in)	δmax, rel (in) = .015 h_{sx}	δ < δmax
	Roof	64	12.5	0.097	2.25	YES	0.195	11.520	YES
	Mech Roof	51.5	16.5	0.019	2.97	YES	0.098	9.270	YES
	4th	47.5	12.5	0.014	2.25	YES	0.079	8.550	YES
	3rd	35	17.5	0.041	3.15	YES	0.065	6.300	YES
	2nd	17.5	17.5	0.024	3.15	YES	0.024	3.150	YES

Y Direction				Story Drift, Δ			Total Displacement, δ		
	Level	Story Height	h_{sx}	Story Drift, Δ (in)	Δmax, rel (in) = .015 h_{sx}	Δ < Δmax	Total Displ, δ (in)	δmax, rel (in) = .015 h_{sx}	δ < δmax
	Roof	64	12.5	0.383	2.25	YES	0.898	11.520	YES
	Mech Roof	51.5	16.5	-0.104	2.97	YES	0.515	9.270	YES
	4th	47.5	12.5	0.280	2.25	YES	0.619	8.550	YES
	3rd	35	17.5	0.226	3.15	YES	0.339	6.300	YES
	2nd	17.5	17.5	0.113	3.15	YES	0.113	3.150	YES

Table 46: Controlling displacements for seismic story drift & displacements.

Lateral Frame Member Checks

Before determining loadings on individual members of the braced frames for the following member checks, the controlling loads per structural component were calculated. The story forces were applied to each floor, utilizing the stiffnesses of each lateral component and the center of masses and center of rigidities associated with each floor. The second floor results are highlighted in Figure 48. The results of all of these can be found in Appendix 5.

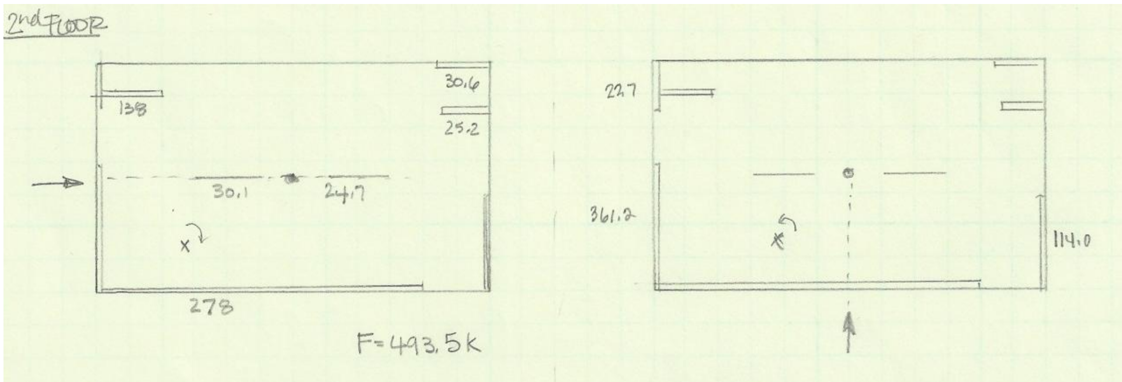


Figure 47: Floor 2 loading distribution per lateral component

To complete the analysis of the SSPAC lateral structural system, member sizes and loads were verified. Braced frame 2, as seen in Figure 49, was evaluated. The column and brace chosen out of this braced frame, considered at a critical section, and were found to be sufficient to carry the maximum lateral load each member supported. Loadings on each of the members within the braced frames followed a similar calculation procedure to how the loadings on each of the lateral structural components were found, as discussed previously. The load applied to the braced frame was divided amongst the members that supported the load by percent shear that each member carried. The calculations and supporting spreadsheet for these member checks and the appropriate loading distribution can be found in Appendix 5.

Column 8 was analyzed for combined loading through the interaction equation of ASIC 14th edition. Table 6-1 in AISC was utilized to find b_x, b_y, and p, which are variables in the equation for combined loading, $p*P_u + b_x*M_{ux} + b_y*M_{uy} < 1.0$. Moments and axial loads were calculated through the values found through the RAM model. Column 8 saw an M_{ux}=265.5 ft-k and axial load P_u = 349.1 k. Using these values, this equation resulted in 0.86 < 1.0 and therefore passed.

Brace 8, supporting the second floor, was also analyzed, for axial compression and tension. The loading found on this member was calculated at 24.7 k, compared to the 25 k axial load found on the structural drawings. Table 4-1 was again used, confirming that the brace was adequate in axial compression. For axial tension, Table 5-1 was used, comparing ϕP_n for both rupture and yield to the appropriate P_u. Compression controlled, with ϕP_n=143 k, which is more than adequate for the 25 k axial loading on the brace.

The original load values show a decrease in lateral loads on the frame at lower story levels, whereas hand calculations here saw a more constant increase in lateral load. It can be noted that the second floor load decreased from the floor above for both calculated and existing loads. This is logical as the braced frame takes less load on the second floor, due to the contribution of shear walls on the lower floor. The first floor again decreases for the existing loadings. The difference here is the incorporation of a short shear wall by the engineer. For force distribution, this shear wall was neglected for more conservative results in the other lateral components. Therefore, the succession of values going down the braced frame in both models is logical, though loadings in the hand calculations are slightly lower. This can be attributed to the stiffnesses of the braced frames being seen as much lower than the shear walls in the hand calculations and RAM model, which is less conservative.

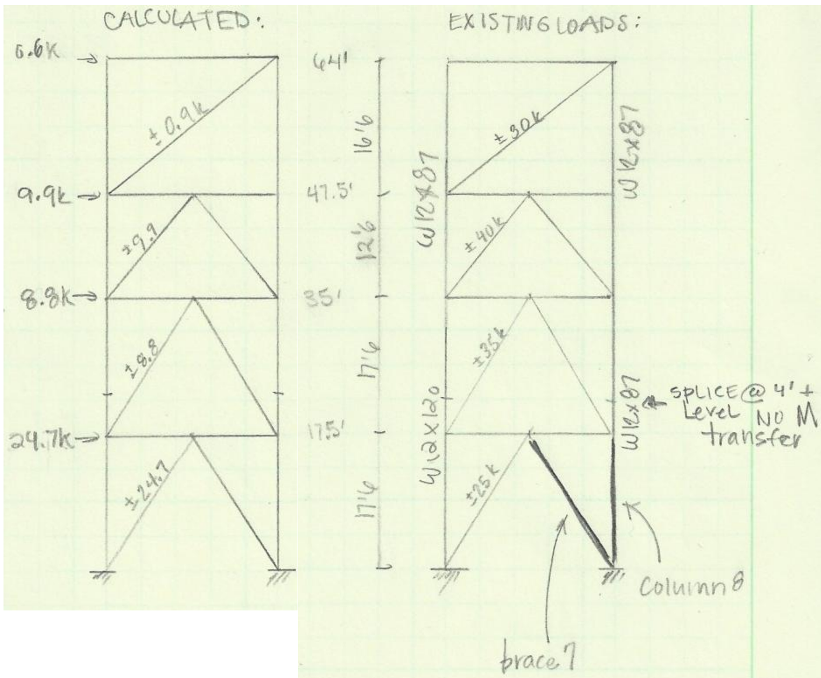


Figure 48: Elevation of braced frame 2, with verified members highlighted on right.

Though shear walls were part of the lateral system, one was not evaluated, as details for these structural components were not given. As these are precast concrete panels, the manufacturer designed them to support the lateral system, as opposed to the structural engineer.

Overturning of the structure was also evaluated. Overall, the resisting moment of the structure was 454,751 k-ft. This was calculated using the building weight, and can be seen in Appendix 5. The overturning moment of the building was 11, 549 k-ft, and comparing these values displays the fact that the building is designed sufficiently.

Considering the uplift in one braced frame, as seen per calculations in Appendix 5, the brace frame can resist uplift in Column 12 of 190 k with the load in Column 9 of 430 k. This displays a satisfactory brace design for uplift. Overturning for the braced frame was considered in the interaction equation check mentioned above.

Conclusion

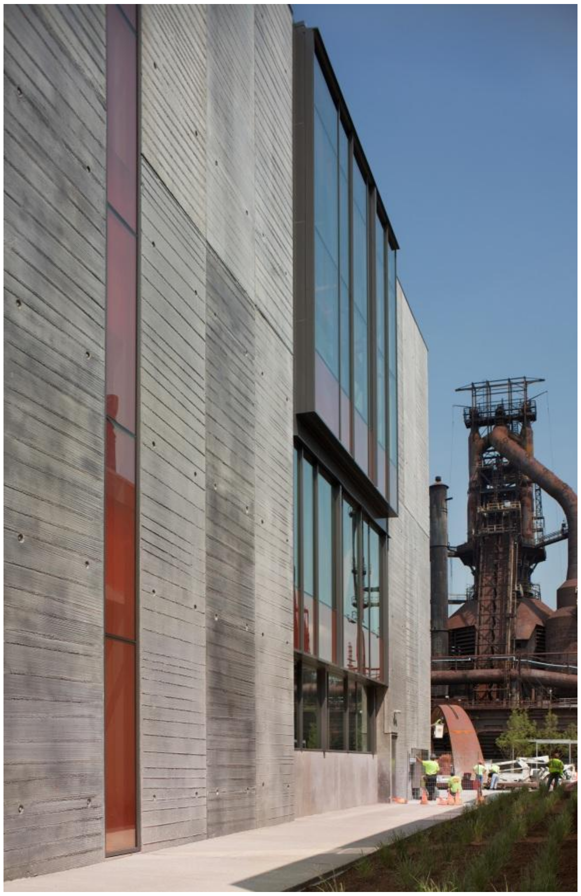
Through the comprehensive and in-depth analysis of the lateral system of the SteelStacks Performing Arts Center, a better understanding of the structural systems has been accomplished. This report has discussed the results of this analysis through the use of a RAM model of the lateral structural system and parallel hand calculations that confirmed the results of this model in relation to the controlling lateral loads, torsion, drift, and foundation considerations, and member checks. These design procedures relied heavily on ASCE 7-05.

Initially, a RAM model of the lateral system was created. Rigid diaphragms were implemented, as were braced frames and meshed walls representing the shear walls. These structural components were correctly modeled as pinned at the base. Once confirmed with hand calculations, the model was utilized to find the member stiffnesses and torsional amplification requirements.

Through this analysis, it was found that the lateral wind loads controlled on the lower floors, while seismic loads controlled on the upper floors. Story drifts and displacements met code requirements under both wind and seismic considerations, and led to the confirmation of the high stiffness of the building, which is also understood through the high amount of shear walls, low building height, and the values found for the period. By evaluating the foundation resisting moment, the adequacy of design for the overturning moment was verified.

Member checks performed for confirmation of sufficient design of the lateral system focused on the members found in Frame 2. These member checks confirmed that the building was sufficiently designed for the lateral loads found on the building through a thorough analysis of all portions of the lateral system.

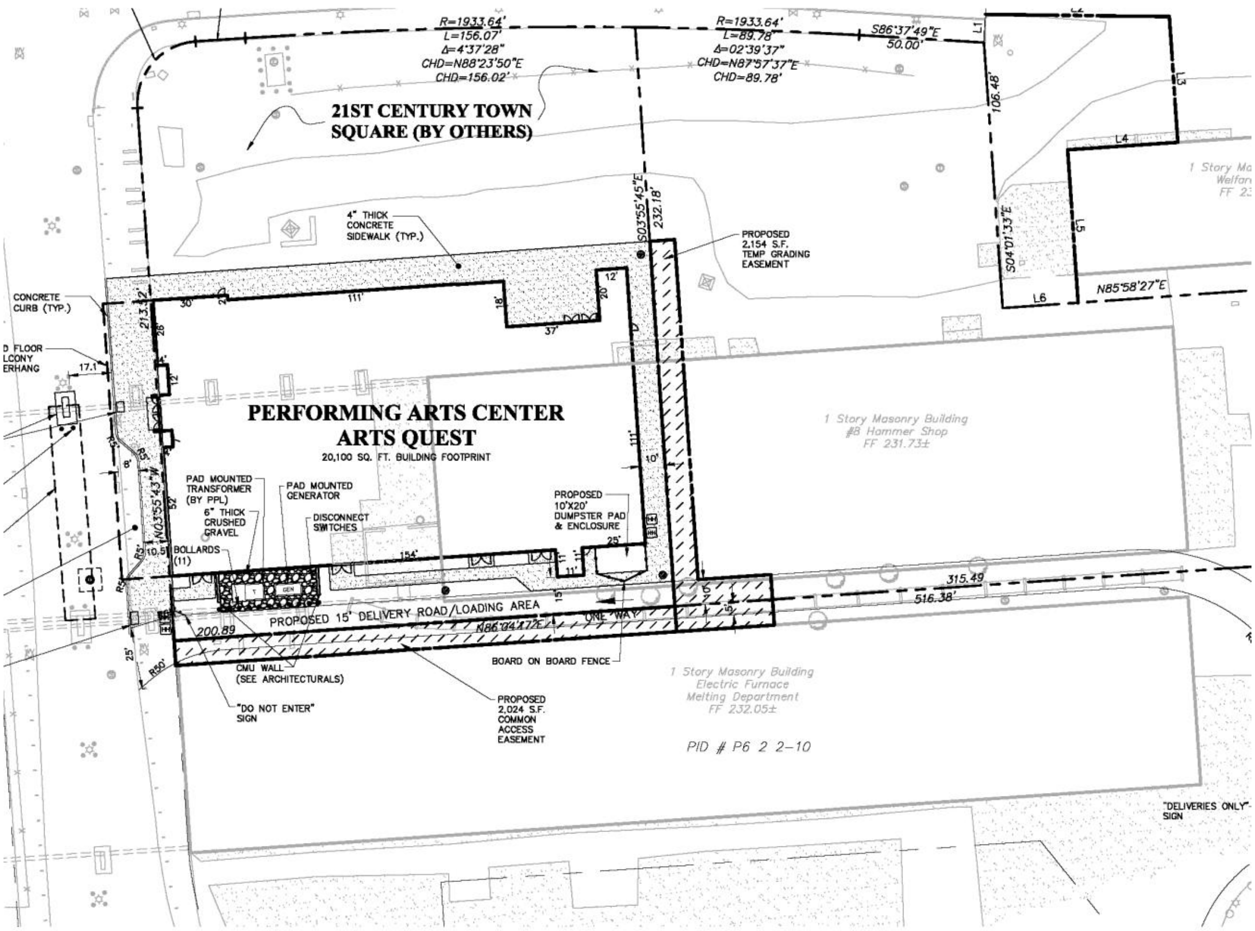
Appendices



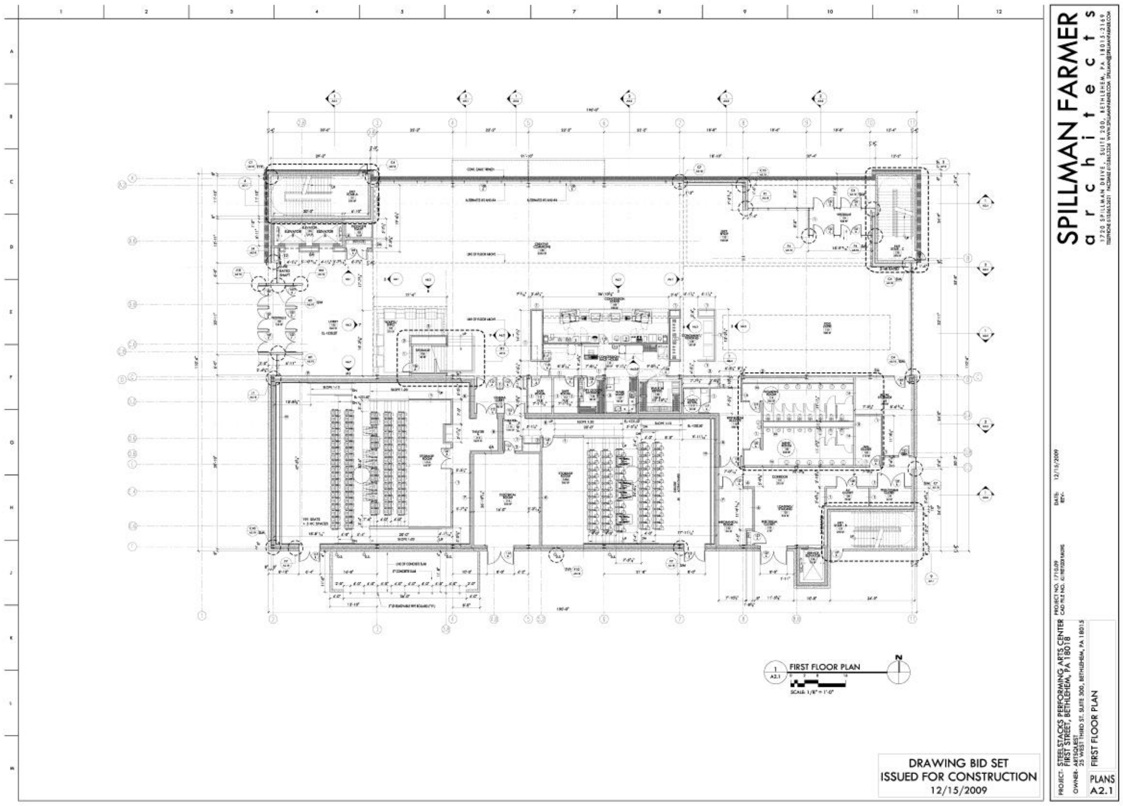
Appendix 1: Structural System Overview

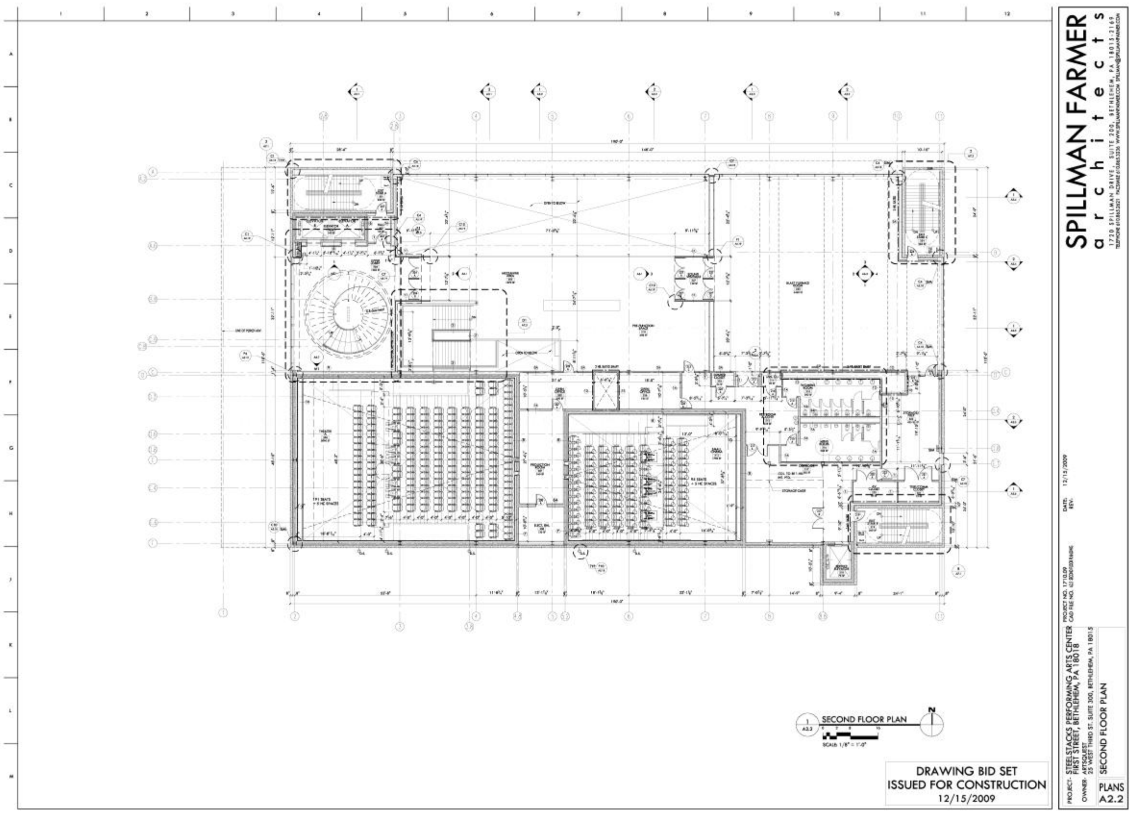
Site Plan Detail

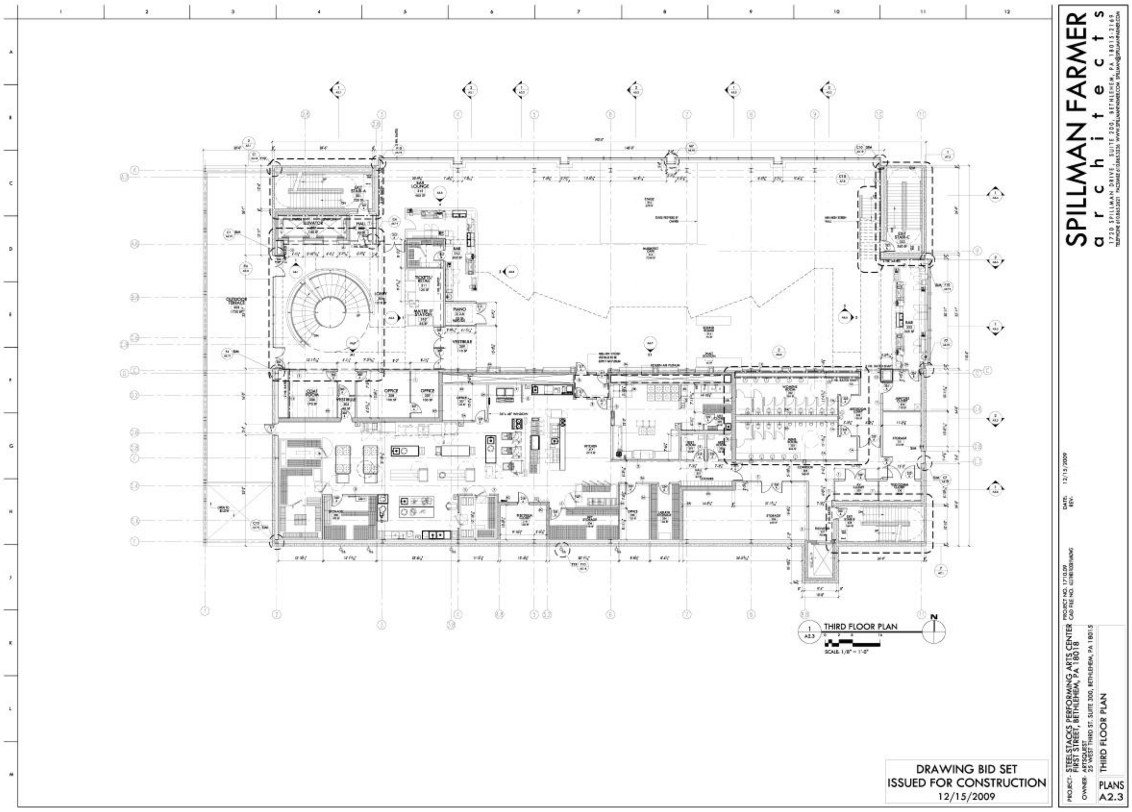
The location of the existing site at onset of project with current location overlaid.

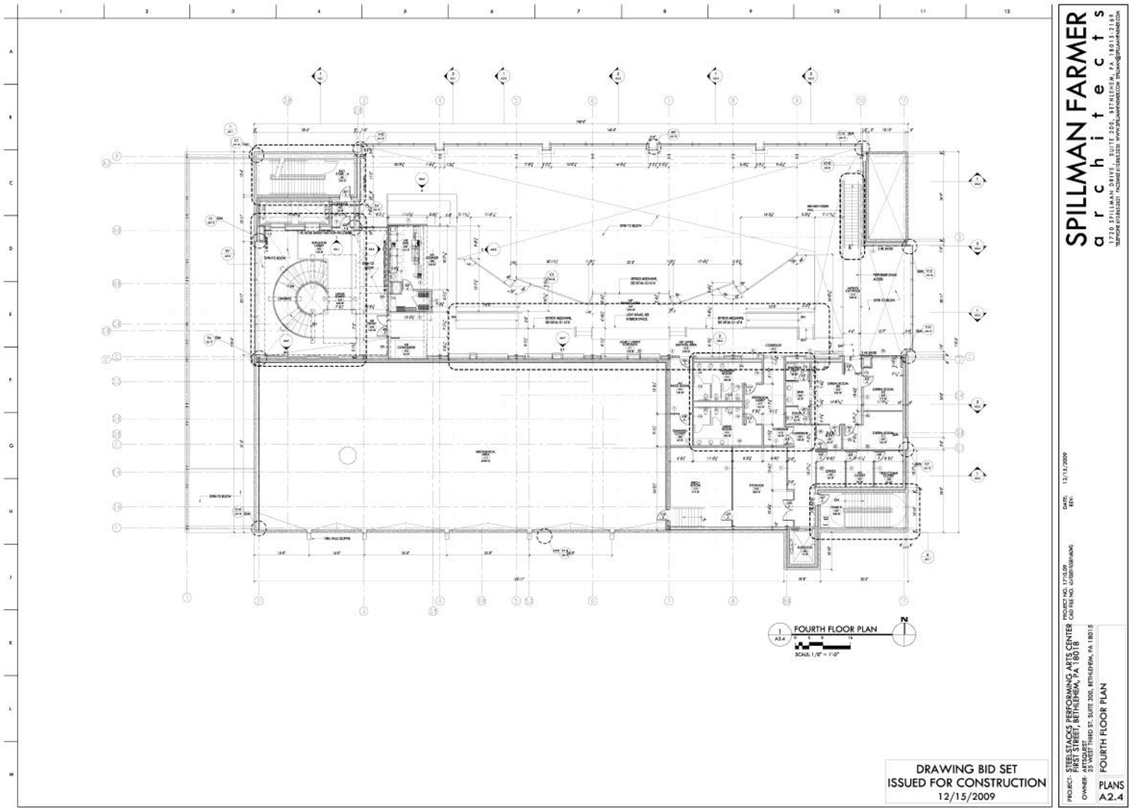


Architectural Floor Plans

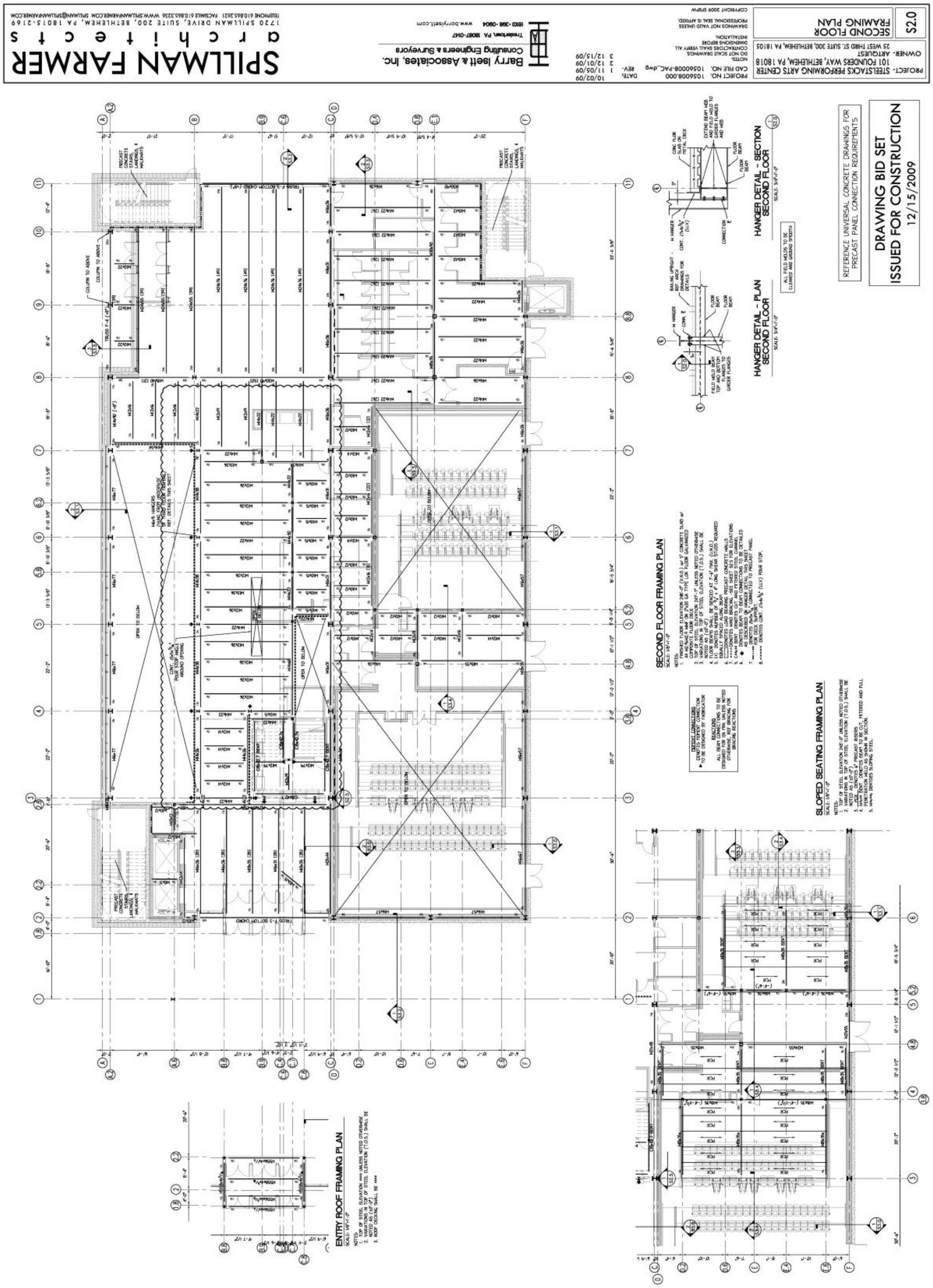




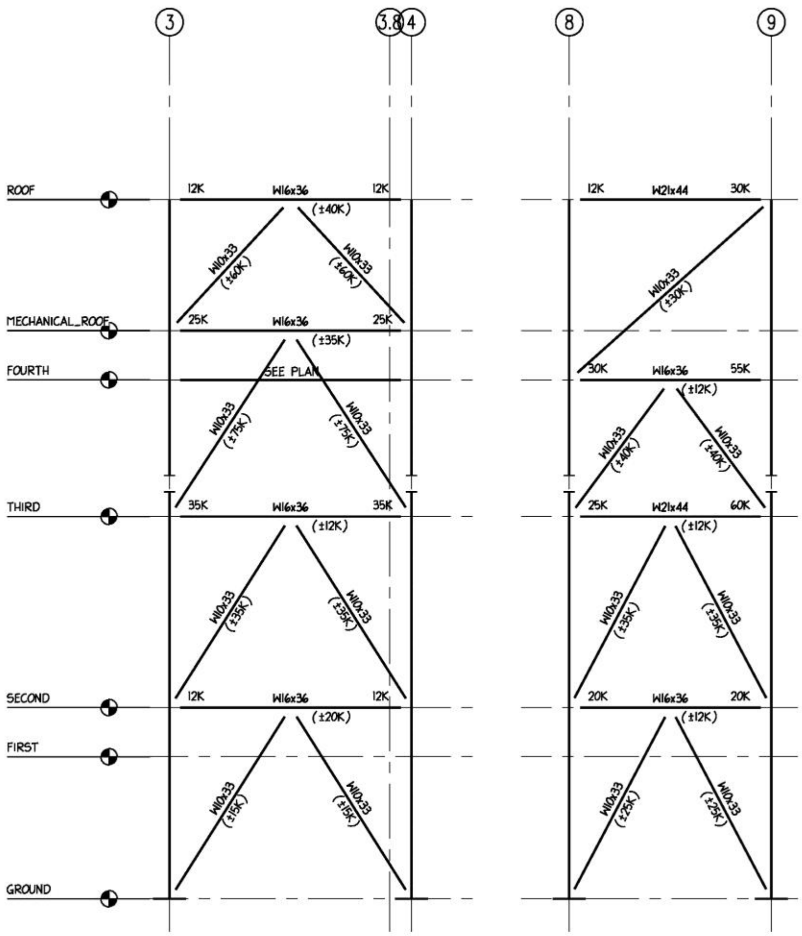

THIRD FLOOR PLAN
SCALE: 1/8" = 1'-0"
DRAWING BID SET
ISSUED FOR CONSTRUCTION
12/15/2009
SPILLMAN FARMER
architects
THIRD FLOOR PLAN
PLANS
A2.3


FOURTH FLOOR PLAN
SCALE: 1/8" = 1'-0"
DRAWING BID SET
ISSUED FOR CONSTRUCTION
12/15/2009
SPILLMAN FARMER
architects
FOURTH FLOOR PLAN
PLANS
A2.4

Structural Floor Plans



Lateral System



1 **ELEVATION AT LINE C**

SCALE: 1/8"=1'-0"

NOTES:

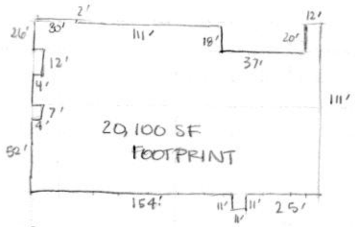
1. CONNECTIONS TO BE DESIGNED FOR FORCES INDICATED BY FABRICATORS ENGINEER.
2. (##k) DENOTES AXIAL FORCE IN MEMBER
   (+) TENSION
   (-) COMPRESSION
3. ##k DENOTES VERTICAL REACTION ON END OF BEAM

Appendix 2: Wind Calculations

Sarah Bednarcik | WIND LOAD CALCS | THESIS Sept 2012. | 1

USING: Chapter 6: Wind Loads ASCE 7-05
Method 2:

Bldg layout:

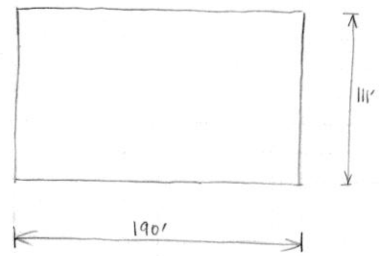


all units in feet

ROOF height = 64'0"

Lower roof height at 51'6" is surrounded by parapet, which goes to 64'0" elevation. Because this space is about 25% of the total roof sq. ft. To simplify this calculation, this is considerdat elevation 64'0". Thus, the roof is a flat roof at 64'0" with no parapet.

Simplified:



∴ mean roof height, h = 64'

For a N-S wind:
L = 111 ft
B = 190

For an E-W wind:
L = 190 ft
B = 111 ft

6.5.3 Design (Values) Procedure

1. Wind Speed $V = 90$ mph (Figure 6-1)
Directionality Factor $K_d = .85$ (Table 6-4)
2. Importance factor, $I = 1.15$ (Table 6-1)
Category III (by Table 1-1)
3. Exposure Category.
by Section 6.5.6. – Surface Roughness B. → Exposure B
Velocity Pressure Exp. Coeff (Table 6-3)
Case 2, Exp B.
interpolate z = 60 | .85, z = 70 | .89 → at z = 64' $K_z = .87$
other K_z values on excel.
4. Topographic factor, K_{zt}
by Section 6.5.7 $K_{zt} = 1.0$
5. Gust effect factor G or G_f.
by Section 6.5.8
rigid or flexible.

$n_1 = \frac{100}{64} = 1.56$ C6-17

$n_1 = \frac{75}{64} = 1.17$ C6-18

∴ treat as a rigid system. (follow §6.5.8.2)

RIGID

2/2

$G = .925 \left(\frac{1 + 1.7\, g_Q I_{\bar{z}} Q}{1 + 1.7\, g_v I_{\bar{z}}} \right)$

$g_Q = g_v = 3.4$

from Table 6-2 $\alpha = 7.0$ $z_g = 1200$ $\hat{a} = 1/4.0$ $\bar{b} = 0.45$
$c = 0.3$ $\ell = 320$ $\ell = 320$ $\bar{\epsilon} = 1/3.0$

$I_z = c\left(\frac{33}{z}\right)^{1/6} = 0.30\left(\frac{33}{38.4}\right)^{1/6} = .293$ $\bar{z}_{max} \Big| \frac{.6(64')}{30'} = 38.4'$

$Q = \sqrt{\frac{1}{1 + .63\left(\frac{B+h}{L_z}\right)^{.63}}} = \sqrt{\frac{1}{1 + .63\left(\frac{B+64}{336.6}\right)^{.63}}}$

N-S. $B = 190$ ft $B = 111$ ft E-W

$\therefore Q = .877$ $\therefore Q = .910$

$G = .925\left(\frac{1 + 1.7(3.4)(.293)(.877)}{1 + 1.7(3.4)(.293)}\right) = .853$ $= .925\left(\frac{1 + 1.7(3.4)(.293)(.910)}{1 + 1.7(3.4)(.293)}\right) = .873$

6. enclosure - Fully enclosed

7. internal pressure, $GC_{pi} = \pm 0.18$ by Fig. 6-5

8. ext. pressure coefs.
Fig 6-6 walls: windward $C_p = .80$
leeward
N/S $L/B = 111/190 = .584$ → $C_p = -.5$
E-W $L/B = 190/111 = 1.71$ → $C_p = -.36$ (interpolated)
Side $C_p = -.70$
roof:

$\theta = 0°$				$\theta = 0$
N-S	-1.0	0 to h/2	$-.9$	E-W
$.5 < h/L < 1.0$	-0.8	h/2 to h	$-.9$	$h/L \le .0.5$
∴ interpolate	-0.5	h to 2h	$-.5$	↓
	N/A	>2h	$-.3$	Roof Area = 190x111 > 1000 SF

9. velocity pressure q_z, q_h. §6.5.10 ∴ Reduction factor = .8
at h = 64':
$q_z = .00256 K_z K_{zt} K_d V^2 I = 17.63$ – others on excel spreadsheet

10. MWFRS.
$p = q\, G C_p - q_i (GC_{pi})$
pressure: where $q = q_z$ for windward. All else $q = q_h$.
$p = q_h G C_p - q_z G C_{pi}$

See excel for further calcs.

Overturning Moment/Base Shear North-South Direction

	Location	Height	Area Below (ft^2)	Area Above (ft^2)	Pressure Below (psf)	Pressure Above (psf)	Factored Story Load (k) Windward	Pressure Below (psf)	Pressure Above (psf)	Factored Story Load (k) Leeward	Factored Story Shear (k)	Overturning Moment (k-ft)	Factored Story Load
Windward Wall	Roof	64	1662.5	0	8.45	8.86	22.48	-10.69	-10.69	-28.44	50.93	3259	50.93
	Floor 4	46.5	1187.5	1662.5	7.43	8.45	36.61	-10.69	-10.69	-48.76	136.29	3969	85.37
	Floor 3	35	1662.5	1187.5	6.11	7.43	30.38	-10.69	-10.69	-48.76	215.43	2770	79.14
	Floor 2	17.5	1662.5	1662.5	5.80	6.11	31.69	-10.69	-10.69	-56.88	304.00	1550	88.58
	Ground	0	0	1662.5	0	5.80	15.44	0	-10.69	-28.44	347.89	0	43.88
	Width (ft)	190				Factored Total Base Shear (k):	347.89				Total Overturning Moment (k-ft):	11549	

Overturning Moment/Base Shear East-West Direction

	Location	Height	Area Below (ft^2)	Area Above (ft^2)	Pressure Below (psf)	Pressure Above (psf)	Factored Story Load (k) Windward	Pressure Below (psf)	Pressure Above (psf)	Factored Story Load (k) Leeward	Factored Story Shear (k)	Overturning Moment (k-ft)	Factored Story Load
Windward Wall	Roof	64	971.25	0	8.72	9.14	13.55	-8.71	-8.71	-13.54	27.09	1734	27.09
	Floor 4	46.5	638.25	971.25	7.67	8.72	21.39	-8.71	-8.71	-22.44	70.92	2038	43.83
	Floor 3	35	971.25	638.25	6.30	7.67	17.63	-8.71	-8.71	-22.44	110.99	1402	40.07
	Floor 2	17.5	971.25	971.25	5.21	6.30	17.89	-8.71	-8.71	-27.08	155.97	787	44.97
	Ground	0	0	971.25	0	5.21	8.09	0	-8.71	-13.54	177.60	0	21.64
	Width (ft)	111				Factored Total Base Shear (k):	177.60				Total Overturning Moment (k-ft):	5962	

Appendix 3: Seismic Calculations

S Bednarcik | Seismic Calcs | TR3

Ch 11 §11.4 Seismic Design Values — USING ASCE 7-05.

$S_s = 0.263g$
$S_1 = 0.062g$
From Geotech Report

Site Class D
SDC C
Occ. Cat. IV
§11.4.2

Structure
- eccentrically braced steel frame in E-W
- Shear walls in N-S

by table 11.4-1 $F_a = 1.6$
table 11.4-2 $F_v = 2.4$

$S_{MS} = F_a \cdot S_s = 1.6(0.263) = .4208 \longrightarrow S_{DS} = \frac{2}{3} S_{MS} = .281$
$S_{M1} = F_v S_1 = 2.4(0.062) = .0148 \qquad S_{D1} = \frac{2}{3} S_{M1} = .099$

$S_a = S_{DS}(0.4 + 0.6\,T/T_o)$
$T_o = 0.2\,S_{D1}/S_{DS} = .071$
$T_S = S_{D1}/S_{DS} = .356$

$T_L = 6$ by Fig 22-15

Finding T, fundamental period of bldg.

$T_a = C_t h_n^x = 0.03(64)^{.75} = .6788$ by T. 12.8-2

$C_u = 1.7$ T. 12.8-1

$T \le C_u T_a = 1.7(.6788) = 1.15 < T_L$ ✓ ok

by 12.8.2, allowed to use T_a.

$C_s = \frac{S_{DS}}{R/I} = \frac{.281}{3/1.5} = .140$ ← compare to engr's value = .139 ✓ GOOD.

$C_s = .139 \le \frac{S_{D1}}{T(R/I)} = \frac{.099}{1.15(2)} = .042$ by § 12.8.1-1 (12.8-3) can use $C_s = .042$

$V_s = C_s \cdot W$ ✓

Calculate Bldg Weight

SBednarcik | Seismic calcs | 13

Building weight §12.8.3

Floor 2,4
5" slab & deck
· slab 50 PSF
· MEP 7 PSF
· framing 10 PSF | 67 PSF

Floor 3
8" slab & deck
· slab 81.5 PSF
· MEP 7 PSF
· framing 10 PSF | 105 PSF

Roof
Lt wt conc 5"
· snow 30 PSF
· slab 40 PSF
· MEP 10 PSF | 80 PSF

Mech Roof
5" slab & deck
· slab 50 PSF
· MEP* 20 PSF
· framing 10 PSF | 80 PSF

*takes into account rooftop units

Materials
PCF from AISC Table 17-13

cmu wall 8" thick · $\left(\frac{8}{12}\right)$ 137 PCF = 92 PSF

Precast panels 8" thick · $\left(\frac{8}{12}\right)$ 150 PCF = 100 PSF

Curtain wall system glass w/ alum. mullions. = 20 PSF as per specs & manuf.

Mechanical System

RTU
2400
1000
1350
10550
10625
1400
| 27.3 k

ERU
1056
1536
1732
4285
| 8.6 K

total = 35.9 k

See excel for further calcs

Weight of Building	Area	PSF	Load (lbs)	Story Weight (lbs)
CMU	4310	91	392210	
Curtain Wall	2160	20	43200	
Concrete Panels	9610	100	961000	
Floor 2	12090	67	810030	2206440
CMU	9140	91	831740	
Curtain Wall	2160	20	43200	
Concrete Panels	9610	100	961000	
Floor 3	21060	105	2211300	4047240
CMU	5920	91	538720	
Curtain Wall	2300	20	46000	
Concrete Panels	6030	100	603000	
Floor 4	21060	67	1411020	2598740
Mechnical (RTU)			35934	35934
CMU	4520	91	411320	
Curtain Wall	3500	20	70000	
Concrete Panels	8530	100	853000	
Roof	17460	80	1396800	2731120
Columns	1870	70	130900	130900
			Total Weight (lbs)	11750374
			(k)	11750

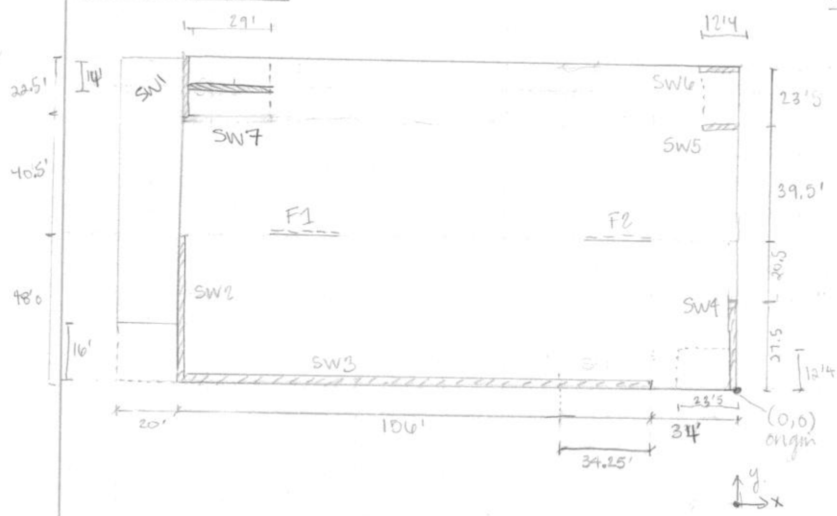
Appendix 4: RAM Model & Building Properties

Center of Mass & Center of Rigidity

S Bednarcik | C of Mass | T3

CENTER OF MASS:

Third floor
8" Slab @ 150 PCF
Shear walls @ 150PCF
8" thick
Story height = 12.5'



WEIGHT CALCS

SLAB:
[(210 x 111) - 2(23.5 x 12) - 29(22.9 - 20 x 16 - 20 x 20] 8/12 x 150 =

SHEAR WALLS:

		WEIGHT (lbs)	x' (in feet)	y' (in feet)
SLAB		2,137,350	-105	55.5
SW1	22.5 x 8/12 150 x 12.5' =	28,125	-190	99.75
SW2	48 x 8/12 150 x 12.5' =	60,000	-190	24
SW3	156 x 12/12 x 150 x 12.5' =	292,500	-112	0
SW4	27.5 x 8/12 x 150 x 12.5 =	34,375	0	13.75
SW5	12.67 x 8/12 x 150 x 12.5' =	15,834	-6.33	87.5
SW6	12.67 x 8/12 x 150 x 12.5' =	15,834	-6.33	111
SW7	29 x 8/12 x 150 x 12.5' =	36250	-175.5	88.5

$\bar{x} = \frac{\Sigma m x}{\Sigma m}$ $\bar{y} = \frac{\Sigma m y}{\Sigma m}$

SEE EXCEL FOR CALCS.

	$\bar{x}$	$\bar{y}$
HAND CALCS.	-106.89	49.50
RAM MODEL	-100.90	48.63
	5.9 % off	1.8 % off

REASONING.
- mass assumed evenly distributed by hand

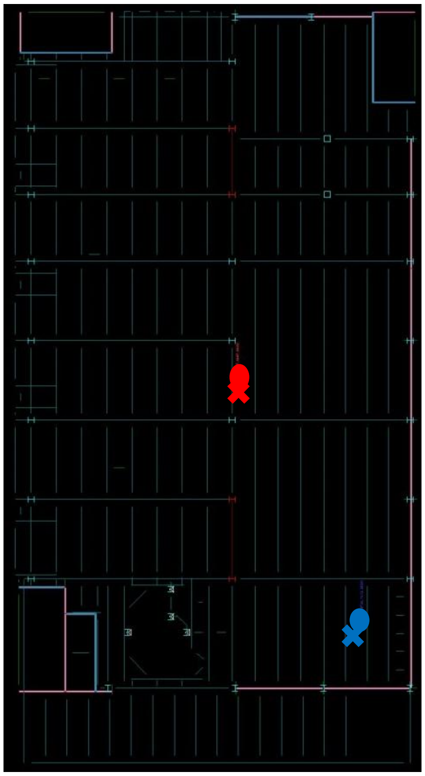
✓ Good.

Center of Mass Hand Calculations - 3rd Floor					
System	Mass	x (ft)	y (ft)	m*x	m*y
Slab	2137350	-105	55.5	-224421750	118622925
SW1	28125	-190	99.75	-5343750	2805468.75
SW2	60000	-190	24	-11400000	1440000
SW3	292500	-112	0	-32760000	0
SW4	34375	0	13.75	0	472656.25
SW5	15834	6.33	87.5	100229.22	1385475
SW6	15834	6.33	111	100229.22	1757574
SW7	36250	-175.5	88.5	-6361875	3208125
Sums	2620268			-280086916.6	129692224
		xbar=	-106.89		
		ybar=	49.50		

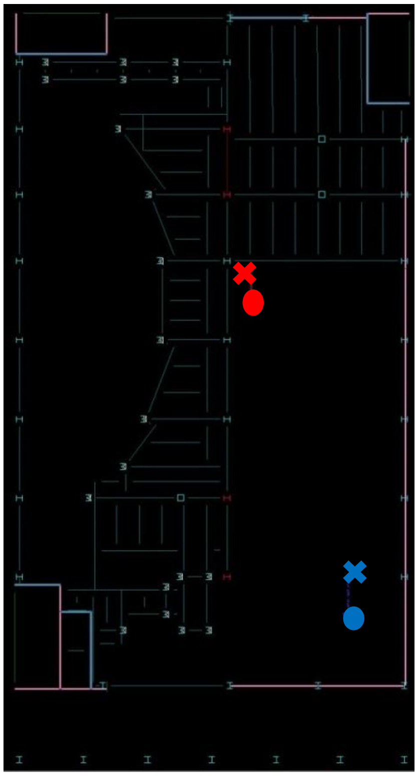
Periods of Vibration	
Tx=	0.8072
Ty=	1.1262
Ttors=	0.9004

Floor plans with associated Center of Mass (red) and Center of Rigidity (blue) marked. Hand calculated values are designated with a cross, while model values are designated with a circle.

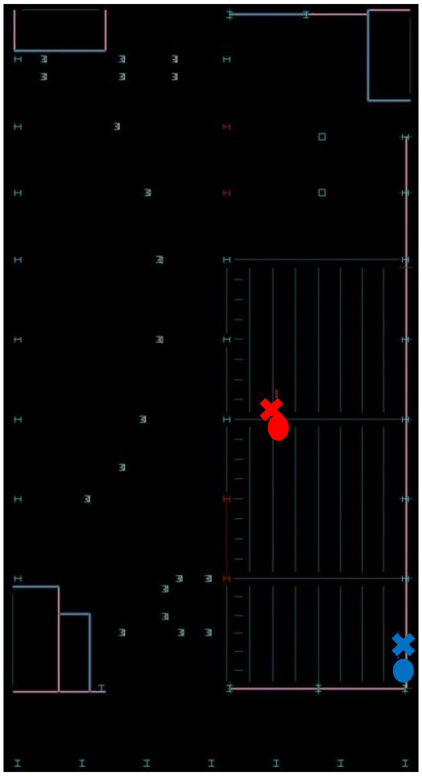
Third Floor:



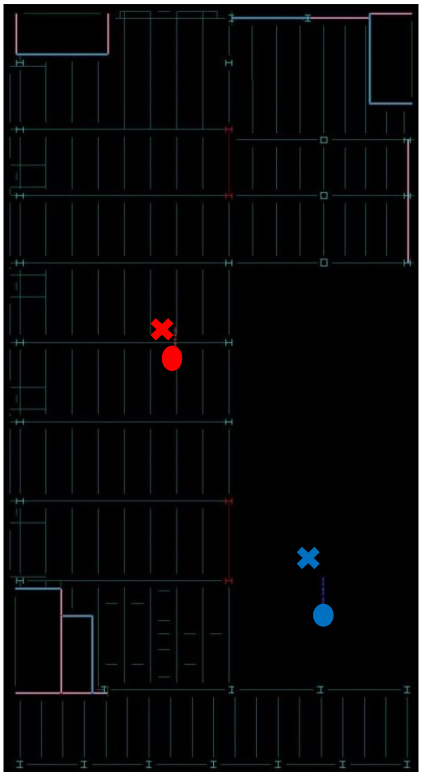
Fourth Floor:



Mechanical Roof:



Roof:



Load Transfer

Level	Direction	Frame	Horiz Force (k)	%V	Sum Check	% Error	Stiffness, X	Stiffness, Y
Roof	x	SW3	334.08	35.60			0.68	
		SW5	30.23	3.22			0.05	
		SW6	33.09	3.53			0.05	
		SW7	494.05	52.64			0.79	
		F1	43.86	4.67			0.07	
		F2	3.19	0.34	938.50	6.15	0.01	
	y	SW1	668.06	66.81				1.00
		SW4	316.26	31.63	1000.01	0.00		0.47
Mech Roof	x	SW3	352.47	37.99			0.72	
		SW5	30.23	3.26			0.06	
		SW6	33.09	3.57			0.07	
		SW7	491.03	52.92			1.00	
		F1	20.96	2.26			0.04	
		F2	0.07	0.01	927.85	7.21	0.00	
	y	SW1	668.05	66.80				1.00
		SW2	69.43	6.94				0.10
		SW4	316.26	31.62	1000.04	0.00		0.47
4th	x	SW3	406.43	41.98			0.91	
		SW5	30.23	3.12			0.07	
		SW6	33.09	3.42			0.07	
		SW7	447.56	46.23			1.00	
		F1	22.20	2.29			0.05	
		F2	28.70	2.96	968.21	3.18	0.06	
	y	SW1	814.09	80.96				1.00
		SW2	69.43	6.90				0.09
		SW4	108.61	10.80	1005.53	-0.55		0.13
3rd	x	SW3	494.32	49.90			1.00	
		SW5	-12.29	-1.24			-0.02	
		SW6	-6.10	-0.62			-0.01	
		SW7	476.62	48.11			0.96	
		F1	19.63	1.98			0.04	
		F2	18.40	1.86	990.59	0.94	0.04	
	y	SW1	149.66	14.97				0.20
		SW2	731.65	73.17				1.00
		SW4	118.17	11.82	1000.00	0.00		0.16
2nd	x	SW3	563.76	56.38			1.00	
		SW5	50.66	5.07			0.18	
		SW6	61.63	6.16			0.08	
		SW7	279.52	27.95			0.38	
		F1	61.41	6.14			0.08	
		F2	50.15	5.02	1067.13	-6.71	0.07	
	y	SW1	46.44	4.64				0.06
		SW2	731.65	73.17				1.00
		SW4	231.31	23.13	1006.33	-0.63		0.32
Ground	x	SW3	563.76	56.38			1.00	
		SW5	50.66	5.07			0.09	
		SW6	61.63	6.16			0.11	
		SW7	279.52	27.95			0.50	
		F1	33.61	3.36			0.06	
		F2	31.38	3.14	1020.55	-2.06	0.06	
	y	SW1	40.00	4.00				0.05
		SW2	731.65	73.17				1.00
		SW4	231.31	23.13	1001.58	-0.16		0.32
			V (total story shear)=			1000	k	Acting at Roof

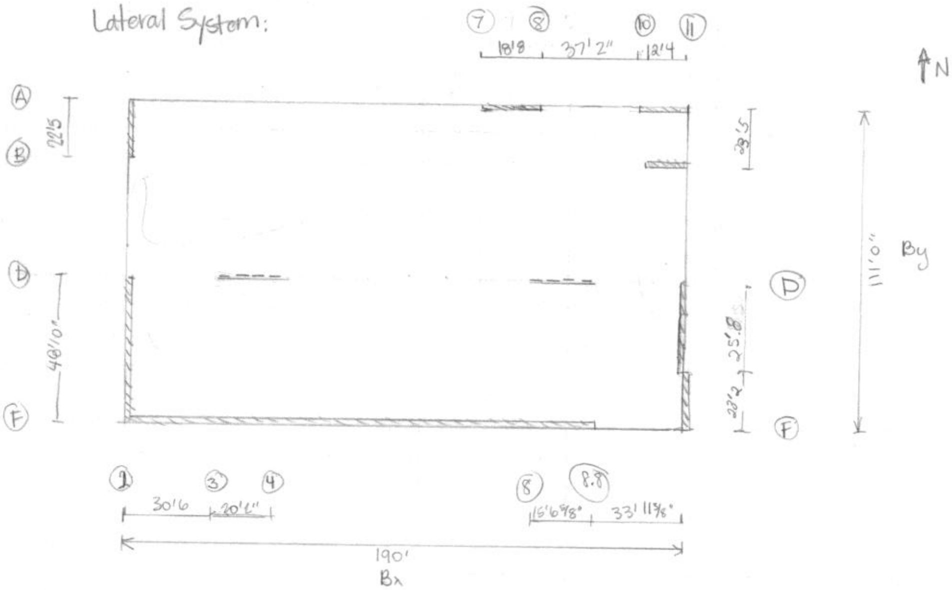
Output From SAP						
Direction Acting	Member	Load (kip)	Displacement (in)	Stiffness kip/in	x'	y'
X	Frame 1	1	0.0238	42.0	--	48
X	Frame 2	1	0.034	29.4	--	48
Y	SW1	1	0.0048	208.3	-190	--
Y	SW2	1	0.0007	1428.6	-190	--
X	SW3	1	0.0005	2000.0	--	0
Y	SW4	1	0.0027	370.4	0	--
X	SW5	1	0.027	37.0	--	87.5
X	SW6	1	0.027	37.0	--	111
X	SW7	1	0.0023	434.8	--	97
X	SW8	1	0.0014	714.3	--	0
X	SW9	1	0.0002	5000.0		111

**Blue rows denote members acting in the Y-Direction.

Torsion

SBednarcik T3

Lateral System:

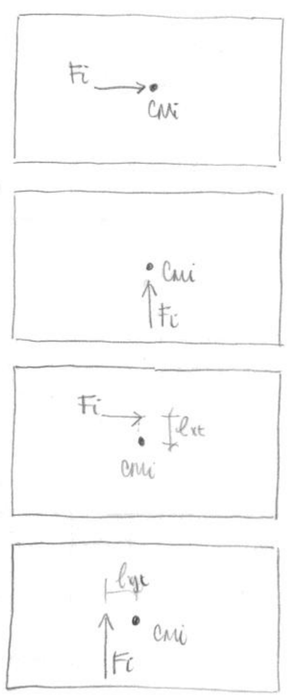


Ex:

Ey:

Ext

Exy



Apply story force to C of M of ea floor.

where $A_x = \left[\frac{\delta_{max}}{1.2\,\delta_{avg}}\right]^2$

$\delta_{avg} = \frac{\delta_A + \delta_B}{2}$

$\delta_A = \delta_{Ex} + \delta_{Ext}$

$e_{xt} = 5.6'$

$e_{yt} = 9.5'$

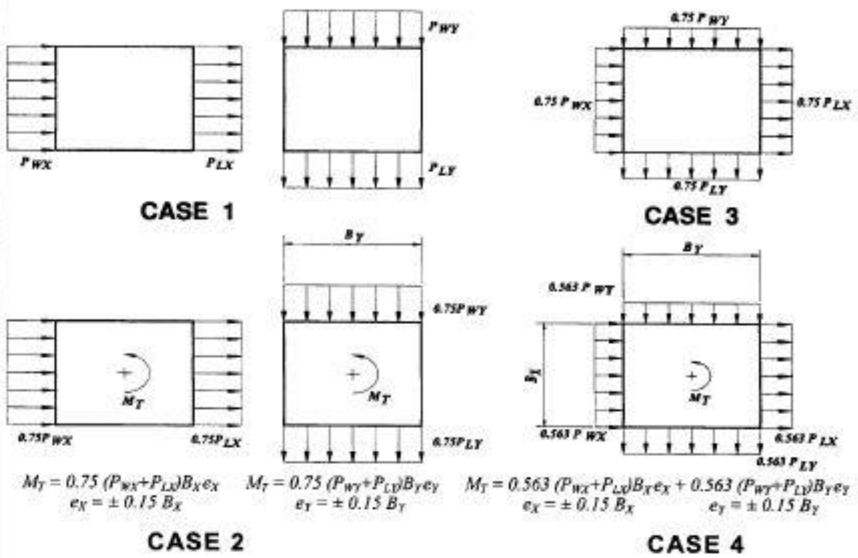
Calculation of Amplification Factor:

Direction		δ1			δ2					Irregularity Type
X Direction	Level	δ1 (Ex)	δ1 (Ext)	δ1	δ2 (Ex)	δ2 (Ext)	δ2	δavg	Ax	by Table 12.3-1
	Roof	0.189	0.206	0.395	0.188	0.206	0.394	0.395	1.0	1a
	Mech Roof	0.141	0.154	0.295	0.141	0.154	0.295	0.295	1.0	1a
	4th	0.124	0.136	0.260	0.124	0.135	0.259	0.260	1.0	1a
	3rd	0.079	0.083	0.162	0.079	0.086	0.165	0.164	1.0	1a
	2nd	0.027	0.030	0.057	0.027	0.030	0.057	0.057	1.0	1a
Y Direction		δ1			δ2					Irregularity Type
	Level	δ1 (Ey)	δ1 (Eyt)	δ1	δ2 (Ey)	δ2 (Eyt)	δ2	δavg	Ay	by Table 12.3-1
	Roof	0.345	0.337	0.682	0.843	0.900	1.743	1.212	1.4	1b
	Mech Roof	0.250	0.245	0.495	0.622	0.664	1.286	0.891	1.4	1b
	4th	0.215	0.211	0.426	0.542	0.579	1.121	0.773	1.5	1b
	3rd	0.124	0.121	0.245	0.334	0.357	0.691	0.468	1.5	1b
	2nd	0.035	0.034	0.069	0.104	0.111	0.215	0.142	1.6	1b

Appendix 5: Lateral Results – Supporting Information

Load Combinations

Main Wind Force Resisting System– Method 2		All Heights
Figure 6-9	Design Wind Load Cases	



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

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

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

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

Notes:

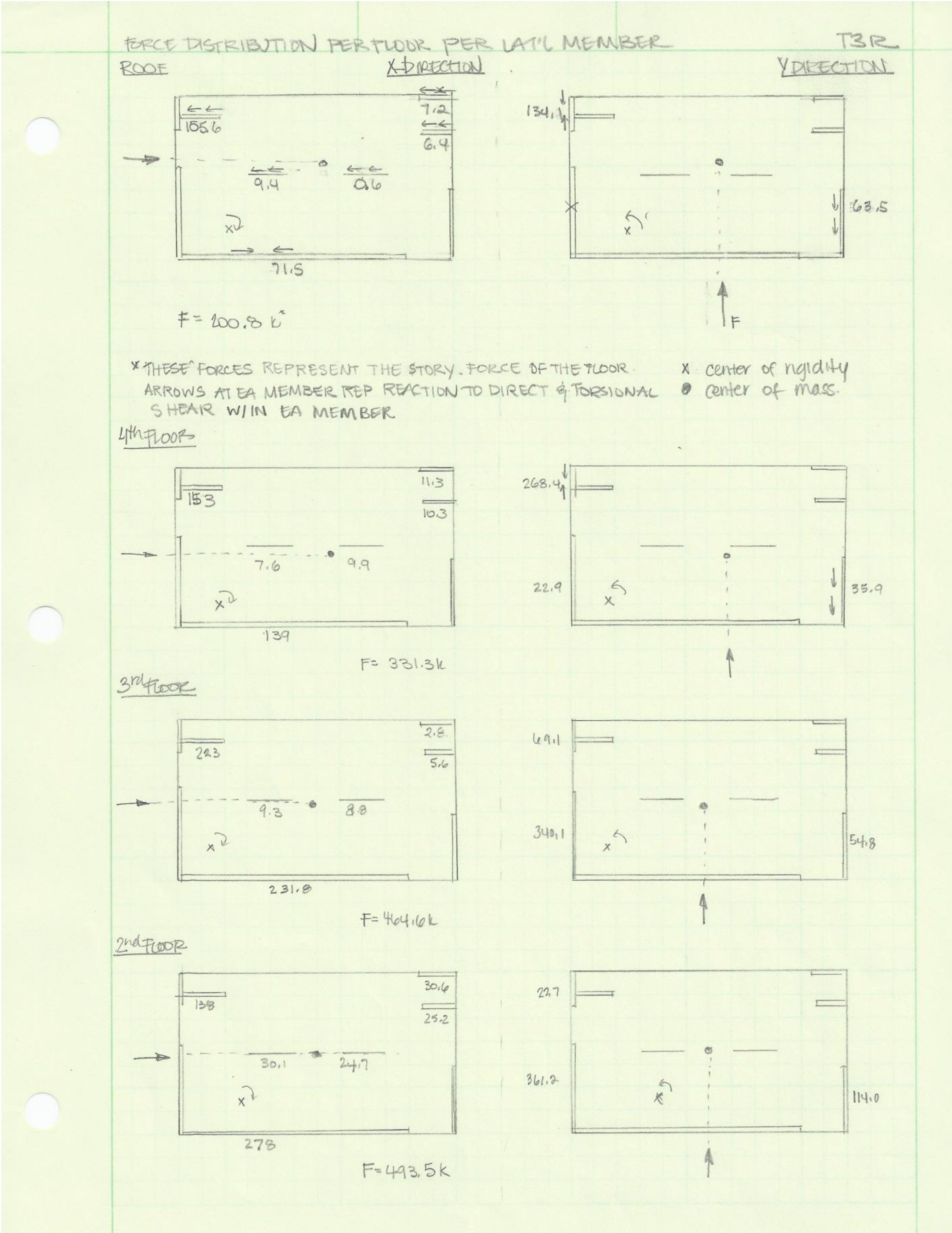
1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 6.5.12.2.1 and 6.5.12.2.3 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:
   - P_{WX}, P_{WY}: Windward face design pressure acting in the x, y principal axis, respectively.
   - P_{LX}, P_{LY}: Leeward face design pressure acting in the x, y principal axis, respectively.
   - $e\ (e_X, e_Y)$: Eccentricity for the x, y principal axis of the structure, respectively.
   - M_T: Torsional moment per unit height acting about a vertical axis of the building.

Story Drift Additional

These are included, as a comparison with SAP2000 model of shear wall 1, as explained in the "Story Drift" section of this paper.

Wind Drift & Displacement Confirmation					
Level	**Load**	**% on SW1**	**Load SW 1**	**Displacement (in)**	**Story Drift**
Roof	50.93	0.668	34.02	0.0331	0.0106
4th	85.37	0.81	69.15	0.0225	0.0088
3rd	79.14	0.15	11.87	0.0137	0.0093
2nd	88.58	0.046	4.07	0.0044	0.0044

Member Check Calculations



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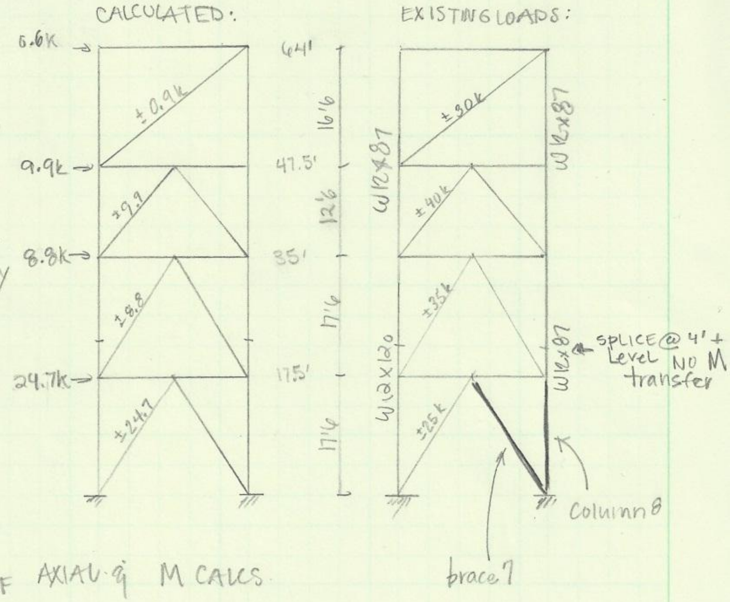
VERIFICATION OF MEMBER SIZES:

Designed at:

Braces - all are W10x33 - axial only

LOADS: USING VALUES FOR STORY FORCE ON PREVIOUS PG, THESE VALUES ALREADY INCLUDE FORCE FROM STORIES ABOVE.

COL. TAKE MINIMAL SHEAR, AND CONSERVATIVELY FOR BRACES, IT IS ASSUMED THEY TAKE NONE.



SEE EXCEL FOR SUMMARY OF AXIAL & M CALCS

*SEISMIC CONTROLS.

COLUMN 8: $KL = (1)(17.5) = 12.25'$ * use $KL = 13'$ MORE CONSERVATIVE

DEAD = 152
LIVE = 155
SNOW = 23

→ Axial $= 1.2(152) + (154.1) + 0.5(23.4) = 349.1k$

→ Moment $= (17.5+4!)(24.7) = \frac{531.1 \text{ ftk}}{2 \text{ col.}} = 265.5$ ftk

USING INTERACTION EQUATION:

$pP_u + b_x M_{rx} + b_y M_{ry} \leq 1.0$ (with $b_y M_{ry} \to 0$)

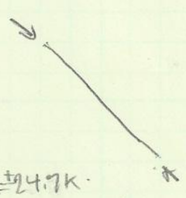
Table 6-1
$p = 1.05 \times 10^{-3}$
$b_x = 1.84 \times 10^{-3}$

$(349.1)(1.05\times10^{-3}) + (265.5)(1.84\times10^{-3})$

$.37 + .49 = \boxed{.86 \leq 1.0}$ ✓okay.

where $.37 > .2$ ✓ok

Brace 7:



W10x33 $L = 19.79'$

use $KL = 20'$ Table 4-1 in Compr.

(C) $\phi P_n = 143k \gg P_u = 24.7k$ ✓ok

$P_u = \pm 24.7k$
(see prev. calc)

Table 5-1 in Tension

(T) $\phi P_n = 437k$ rupture $\gg P_u$ ✓ok
$\phi P_n = 355k$ yield

was designed for $\pm 25k$

BRACE & COLUMN ARE ADEQUATE.

Structural Component Forces									
Height	Level	Load	Total Horiz Force (k)	Member	Designation	Seismic		Wind	
						Axial (k)	Shear (k)	Axial (k)	Shear (k)
64	Roof	E	5.02	Column	1	4.22	0.08	4.22	0.1
		W	0.68	Column	6	4.22	0.1	4.22	0.1
				Brace	26	7.12	5.04	1.68	1.19
47.5	4th	E	3.80	Column	15	4.22	0.24	4.22	0.10
		W	1.30	Column	10	4.22	0.25	4.22	0.10
				Brace	14	7.52	4.48	2.09	1.24
				Brace	15	7.52	4.48	2.09	1.24
35	3rd	E	2.48	Column	13	14.28	0.55	14.28	0.10
		W	0.76	Column	8	14.23	0.55	14.23	0.10
				Brace	9	9.25	4.35	2.45	1.15
				Brace	10	9.31	4.38	2.46	1.16
17.5	2nd	E	1.45	Column	12	28.03	0.76	28.03	0.11
		W	2.26	Column	8	27.97	0.56	27.97	0.1
				Brace	6	12.01	5.64	2.11	0.99
				Brace	7	12.07	5.67	2.121	1.00
						Moment on Column (k-ft)	E	Moment on Column (k-ft)	W
							47.33		16.40

Overturning and Foundation Impact

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§12.13 ASCE 7-05

OVERTURNING MOMENT

$M_{resist} = W \cdot (\frac{1}{2}B) \cdot F.S$

$= 11\,750k(\frac{1}{2}\ 111')\ \frac{2}{3} = 434{,}750$ kft

$M_{o,max} = 11{,}549$ k·ft (from wind load, N-S)

11,549 kft << 434,750 kft

∴ okay
foundation adequate.

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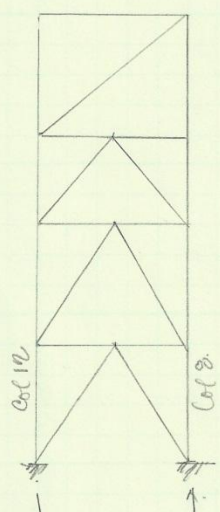
CK OVERTURNING:
CRIT @ Col 8

	COL 8	12
D	152 k	242
L	155 k	251
S	23 k	28.7
W	28.0	28
E	18	18

Ck 0.9D + 1.6W



0.9D + 1.6W 1.2D + 0.5L + 1.6W = 1.2(152) + 0.5(155) + 1.6(28) = 305

L+Lr

0.9D + 1.0E 1.2D + 1.6L = 1.2(152) + 1.6(155) = 430.

CONTROLLING:
= 0.9(242) − 1.0(28)
= 190.