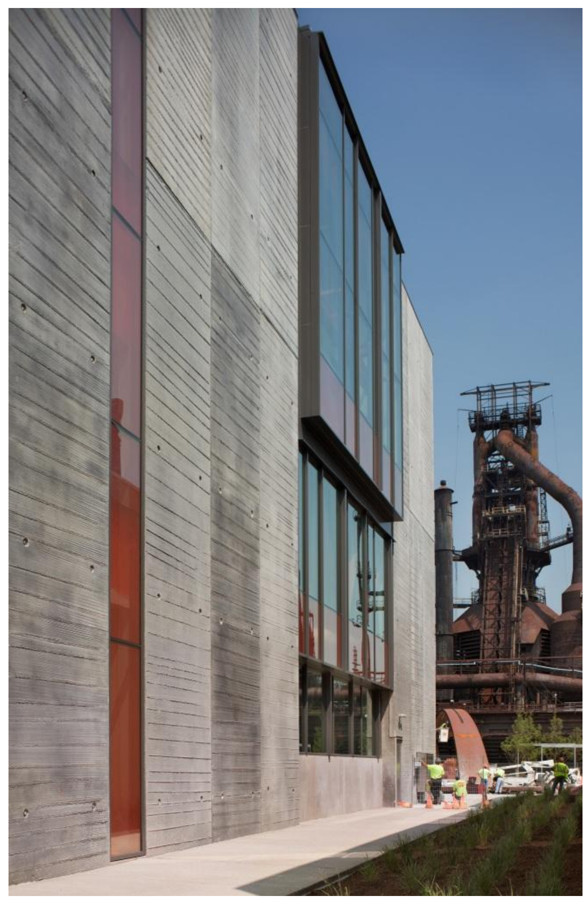
SteelStacks Performing Arts Center | Bethlehem, Pennsylvania

Thesis Proposal



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Executive Summary

The purpose of this report is to provide a solution and detailed process for the scenario that has developed concerning the structural system of the Steel Stacks Performing Arts Center (SSPAC), while outlining the tasks, tools, and schedule for the proposed solution. The SSPAC is a 64-foot, 4 story, 67,000 square foot arts and cultural center in Bethlehem, Pennsylvania. The existing lateral system consists of braced frames and shear walls in the East-West direction and shear walls in the North-South direction, with concrete slabs taking additional lateral loads as the floor diaphragms.

The existing system, though an efficient and successful design, saw torsional irregularity due to the necessity of providing more shear walls along the south west side of the building. The engineer, in meeting these difficulties, designed a building that provided a stiff lateral system with minimal deflection. In response to this, a scenario has been developed in which the lateral system is no longer allowed to include shear walls, but must implement only a steel lateral system.

The structural depth solution to this scenario implements additional braced frames in both directions, allowing for more ductility to be designed into the system. Existing walls will become nonstructural, with alternatives, including Exterior Insulation and Finish Systems (EIFS), being considered. The gravity system will be redesigned with a goal of eliminating superfluous cost and material. Floor diaphragms will be redesigned to lighten the building weight, considering options such as lightweight concrete.

The solution for the structural redesign will impact the architecture and acoustics of the spaces. With both aesthetics and acoustics being important factors in the original design, they are also necessary considerations for the redesign. Public spaces will be impacted by the structural changes due to acoustic issues, such as in the stage and Musikfest Café area. With the floor system being redesigned, acoustical performance of each floor system will be studied and taken into account when choosing the final gravity system. In architectural design, the façade and public spaces will be altered due to the wall and gravity systems being redesigned. For this to be thoroughly evaluated, an architectural model will be utilized to compare the existing and redesigned spaces, with major impacts being studied for possible alteration.

In addition, MAE coursework will be included in this thesis project, and has been incorporated into this proposal. Material from AE 530, *Computer Modeling of Building Structures,* and AE 534, *Analysis and Design of Steel Connections*, will be utilized to provide a more complete project, and the implementation has been elaborated on in this report.

In preparation for the thesis project, this proposal includes the tasks and tools used for each aspect of the proposed solution, as well as a detailed schedule outlining the specific steps required to complete the redesign process.

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Purpose

The purpose of this report is to propose a particular scenario that has developed for the SteelStacks Performing Arts Center. This problem necessitates redesign of different aspects of the building, and a proposed solution is elaborated in the following report. To display capabilities of accomplishing this solution, the solution method and timeline are also elaborated. Precursory information has been presented to provide a better understanding of the existing building and design information.

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.

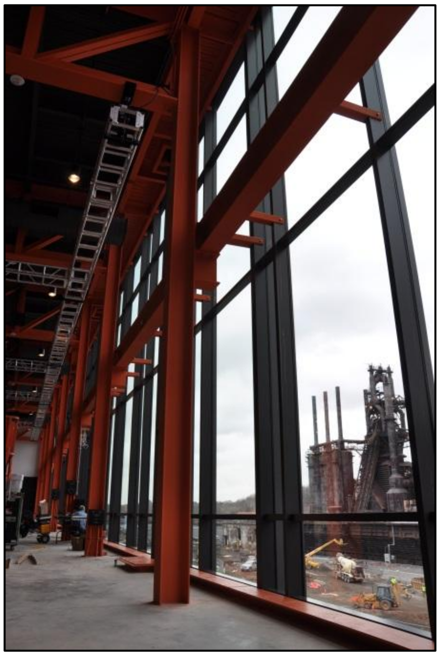


Figure 1: Interior atrium space, highlighting opening structural plan.

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.

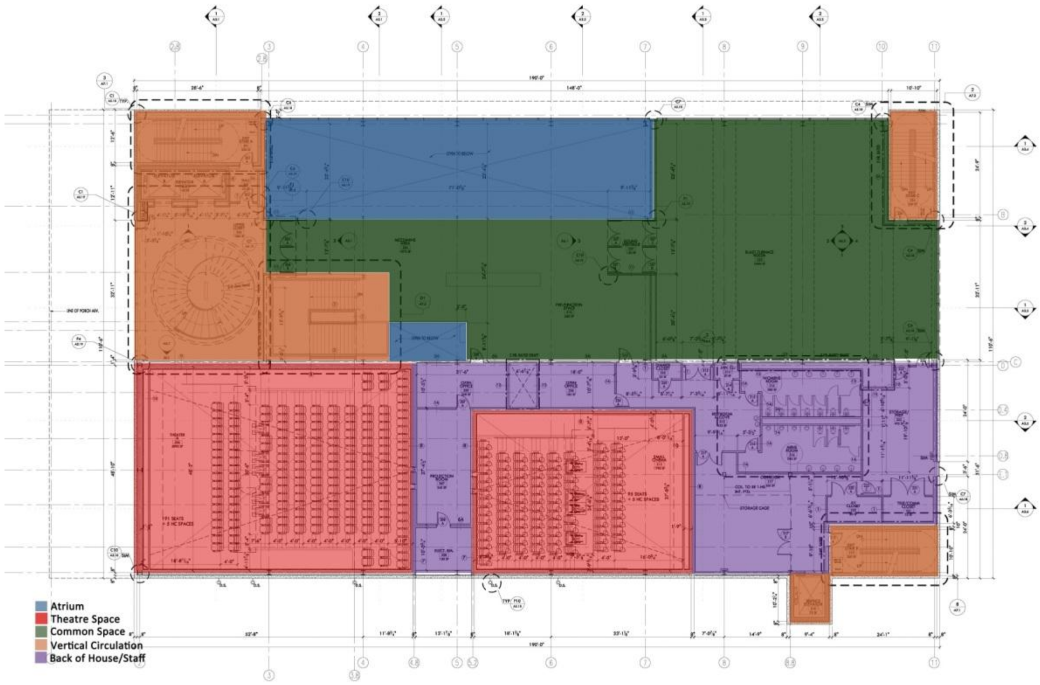


Figure 2: Floor Plan from A2.2

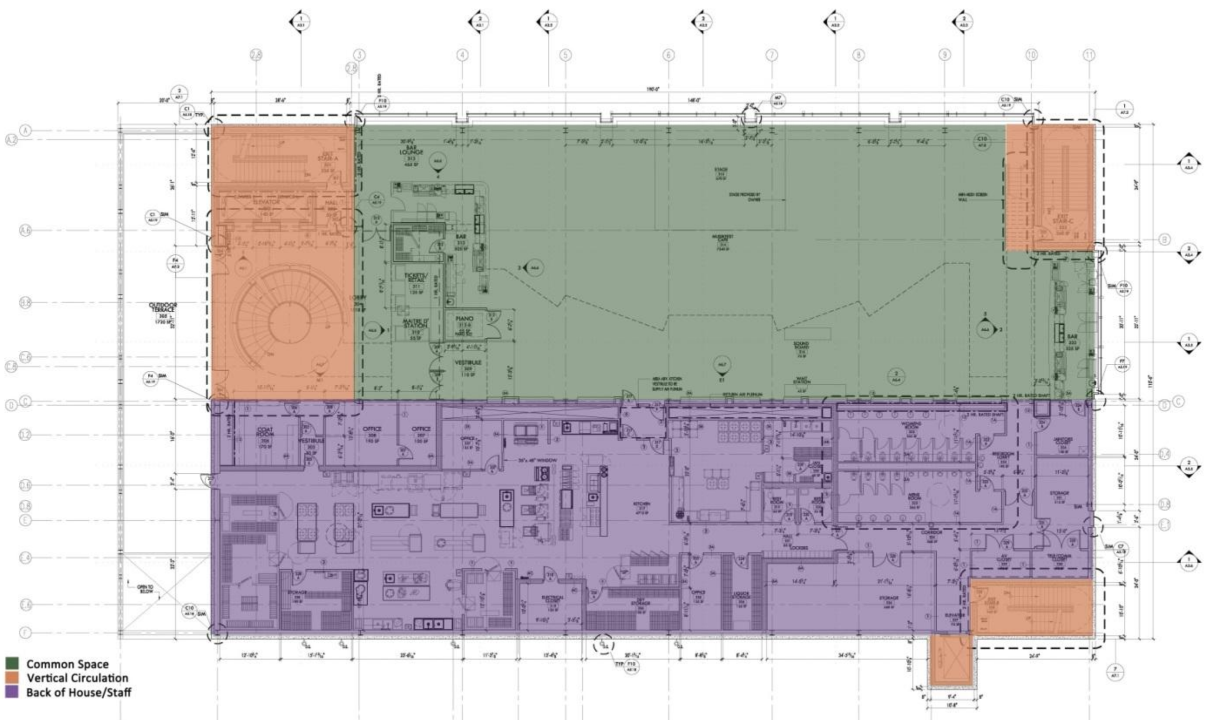


Figure 3: Third Floor Plan from A2.3

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.

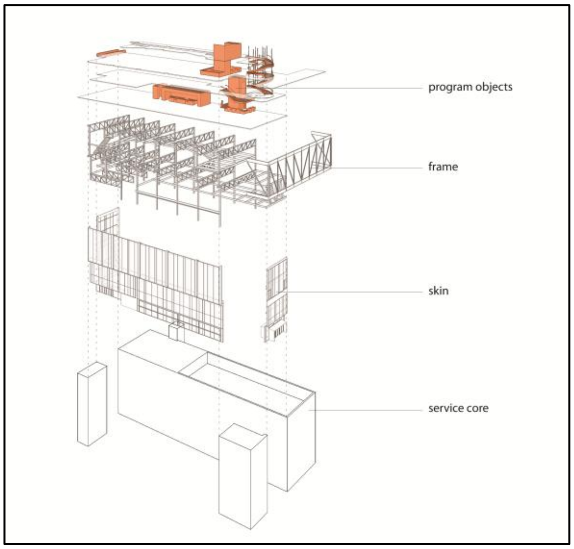


Figure 4 : Image displaying the separation of spaces through the structural design.

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.

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.

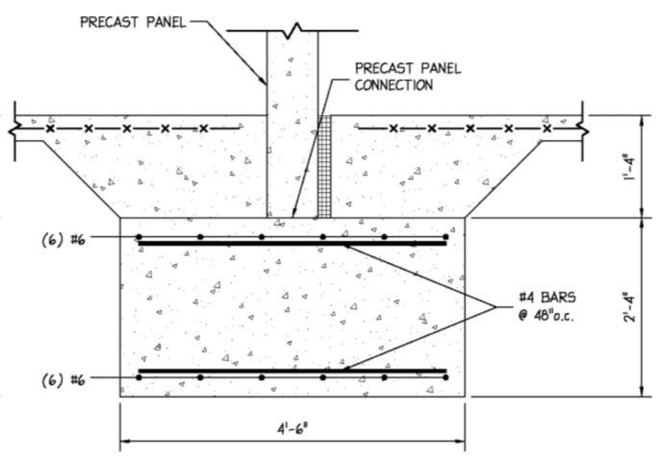


Figure 5 : Section of foundation to precast panel connection from S1.0.

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.

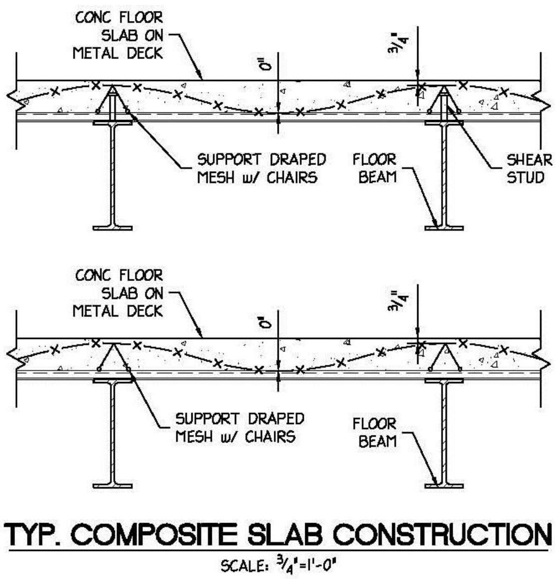


Figure 6 : Typical composite slab section from S2.8

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, a representative one, Truss F-1A, shown in Figure 7.

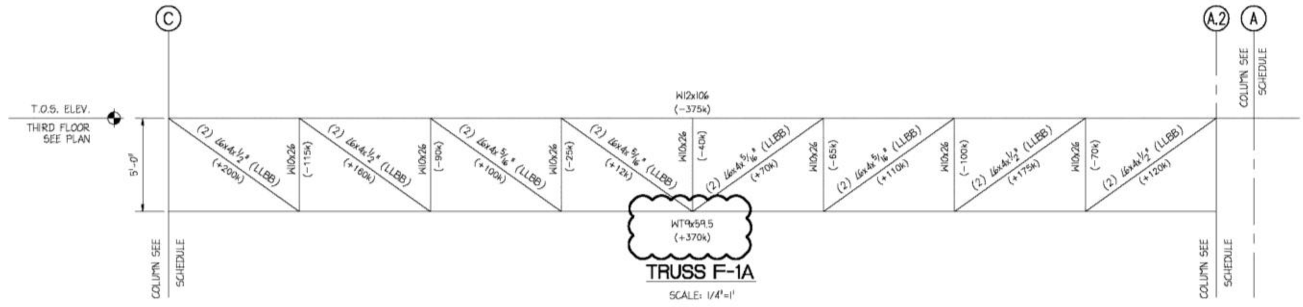


Figure 7 : Third floor representative framing system truss from S2.6.

Framing on the fourth floor is more irregular, as explained previously, 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.

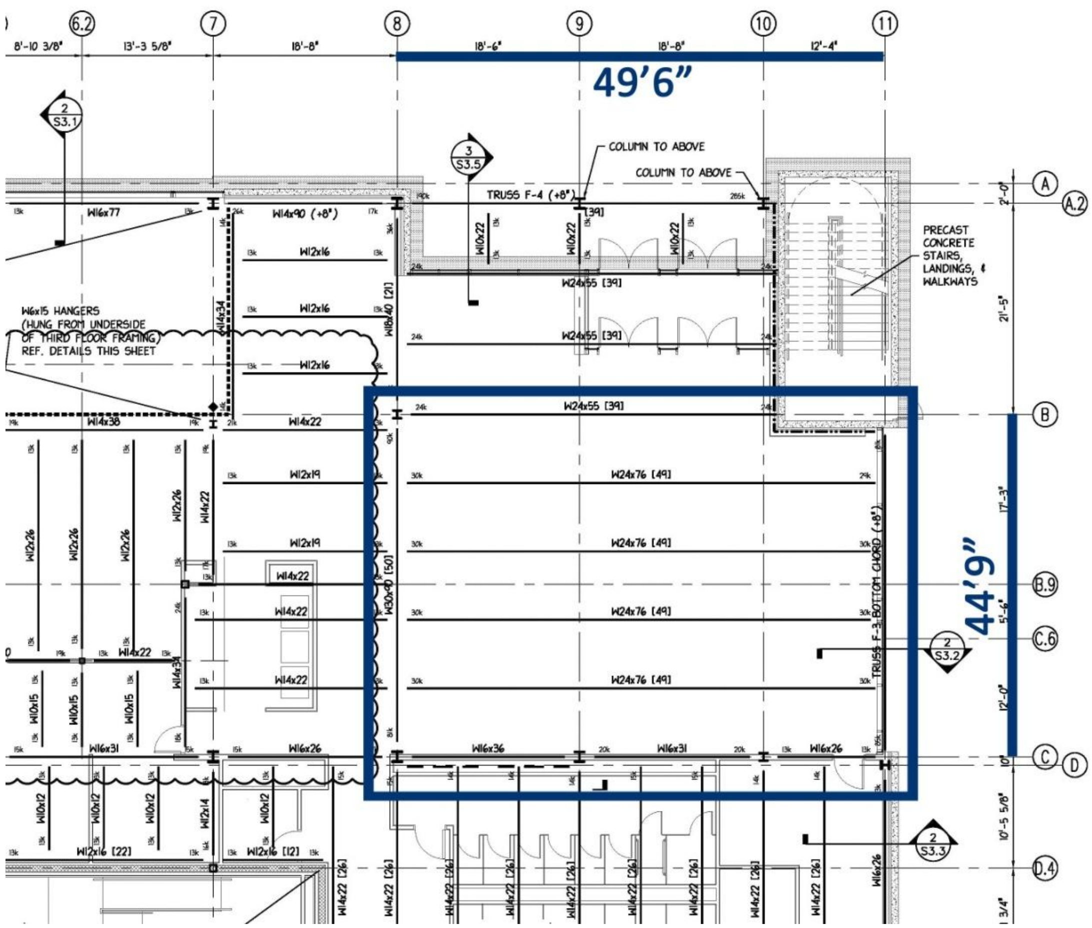


Figure 8: Second floor framing plan, with a representative bay of a typical frame, highlighted in blue, from S2.0

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. 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.

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.

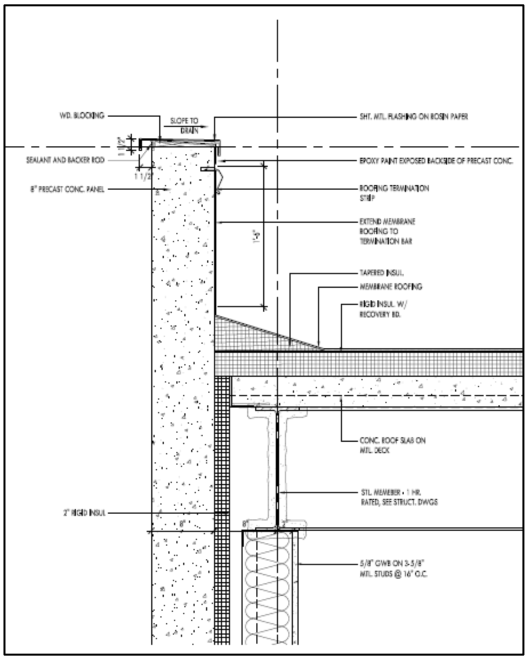


Figure 9 : Cross section of the roofing system.

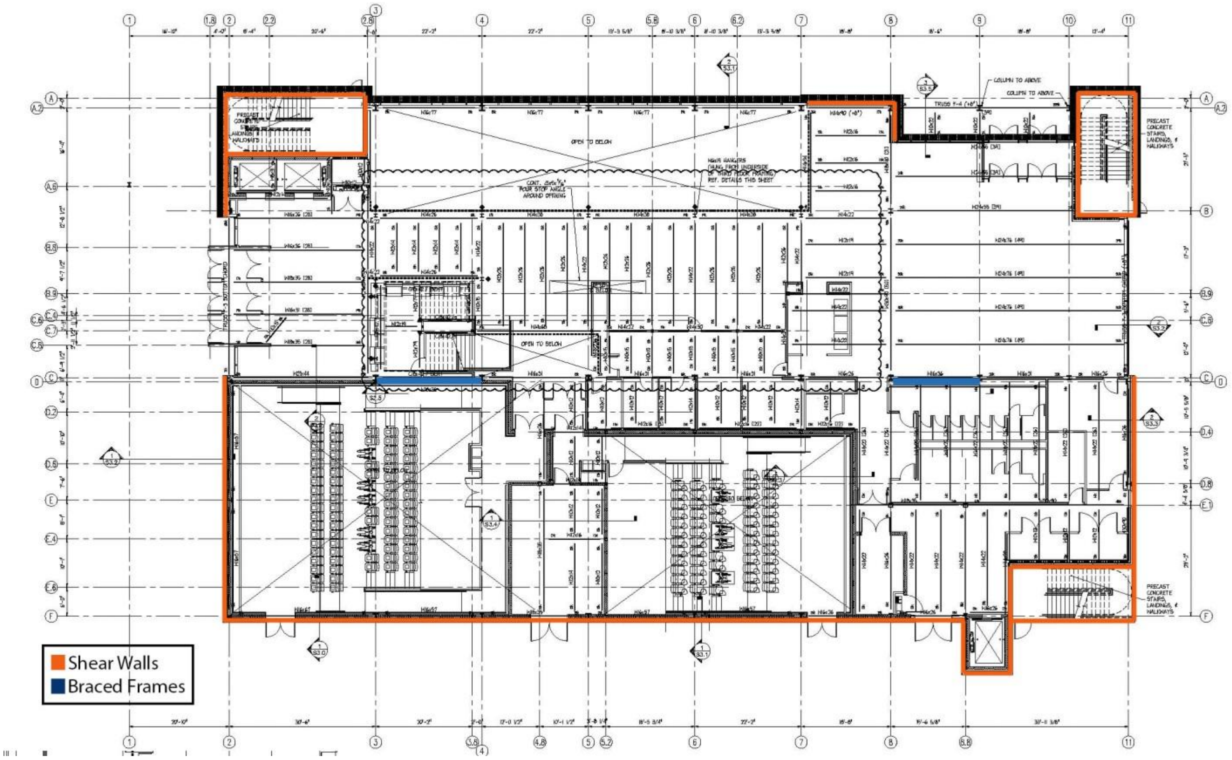
Supporting the floor systems is a combination of braced frames, columns, and precast panels. Columns are generally W12s, as the structural engineer focused on not only supporting the structure, but keeping the steel consistent dimensions. HSS columns were also used at varying locations, and varied from 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 Figure 10.

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.



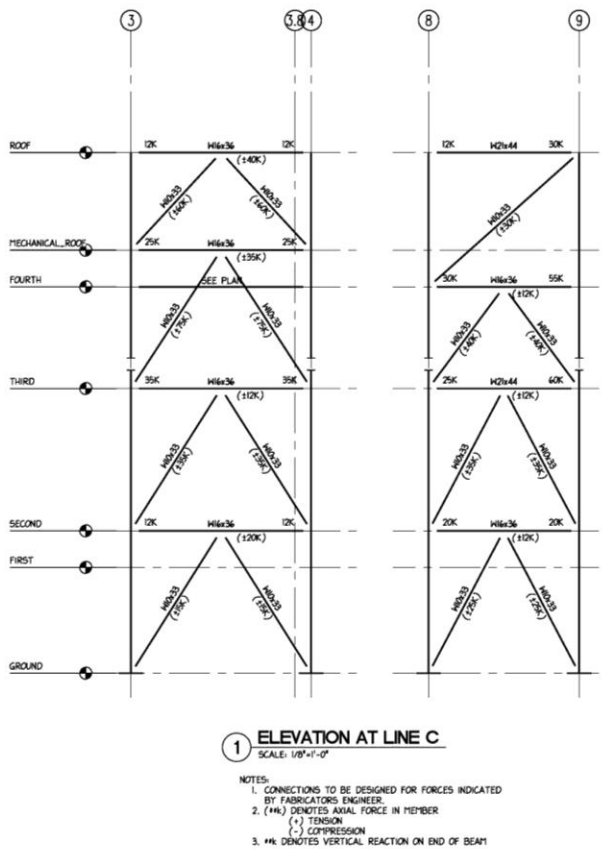


Figure 10: Floor plan highlighting shear walls in orange and braced frames in blue, which contribute to the lateral system, with braced frame elevations shown.

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.

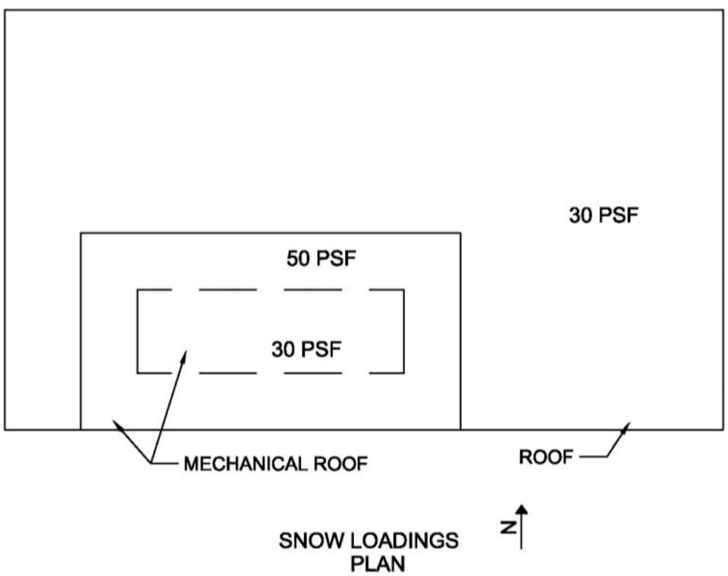


Figure 14 : Summary of snow loads.

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. For further information concerning these loads, see Technical Report III.

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.

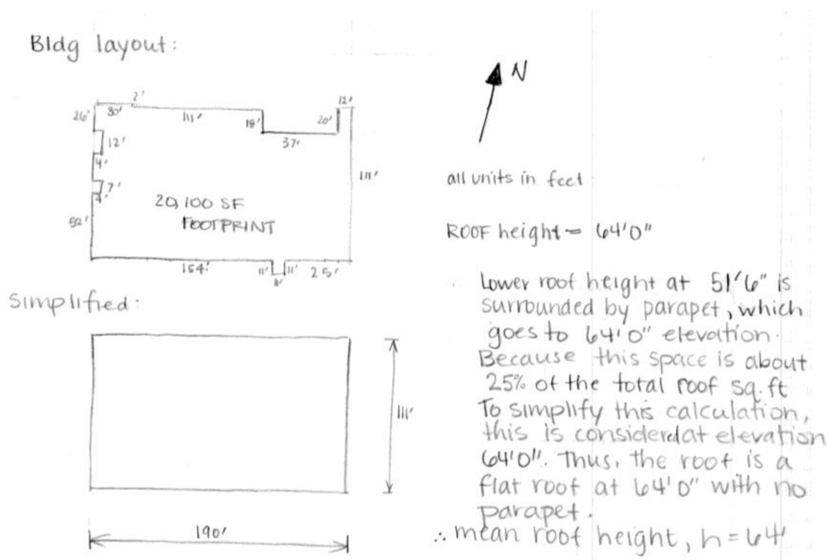


Figure 15 : Building dimensions simplified for wind load calculations following Method 2.

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. Wind calculations in their entirety are included in Technical Report III.

A summary of the wind pressures and variables going into these pressures in each direction are displayed below, in Figures 16 through 21. These results have been summarized for the East-West direction in Figures 16 through 18, and highlight the base shear and overturning moment due to these wind pressures. Figures 19 through 21 summarize similar results and drawings for the North-South direction. Table 22 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 36.7 psf.

The overall base shear for the East-West direction is 177.6 k, with an overturning moment of 5175 k-ft. These results can be compared with the North-South direction, where the base shear was higher, at 347.9 k, and the overturning moment at 9998 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 applied, as per ASCE 7-05. The values in the following tables included this factor.

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	9.14
		Floor 4	47.5	0.8	16.82	17.63	0.873	0.18	8.72
		Floor 3	35	0.8	14.80	17.63	0.873	0.18	7.67
		Floor 2	17.5	0.8	12.16	17.63	0.873	0.18	6.30
		Ground	0	0.8	10.05	17.63	0.873	0.18	5.21
	Leeward	All	All	-0.36	17.63	17.63	0.873	0.18	-8.71
	Side	All	All	-0.7	17.63	17.63	0.873	0.18	-13.95
Roof		0 to h/2	0 to 32	-0.9	17.63	17.63	0.873	0.18	-17.03
		h/2 to h	32 to 64	-0.9	17.63	17.63	0.873	0.18	-17.03
		h to 2h	64 to 128	-0.5	17.63	17.63	0.873	0.18	-10.87
		>2h	>128	-0.3	17.63	17.63	0.873	0.18	-7.79
							E-W load	Sum Wall	25.53
								Sum Roof	-52.71

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

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)	Factored Story Shear (k)	Overturning Moment (k-ft)
Windward Wall	Roof	64	971.25	0	17.44	17.85	27.09	27.09	1734
	Floor 4	46.5	638.25	971.25	16.38	17.44	43.83	70.92	2038
	Floor 3	35	971.25	638.25	15.02	16.38	40.07	110.99	1402
	Floor 2	17.5	971.25	971.25	13.92	15.02	44.97	155.97	787
	Ground	0	0	971.25	0	13.92	21.64	177.60	0
	Width (ft)	111				Factored Total Base Shear (k):	177.60	Total Overturning Moment (k-ft):	5175

Table 17 : Summary of overturning moment and base shear calculations in the East-West direction.

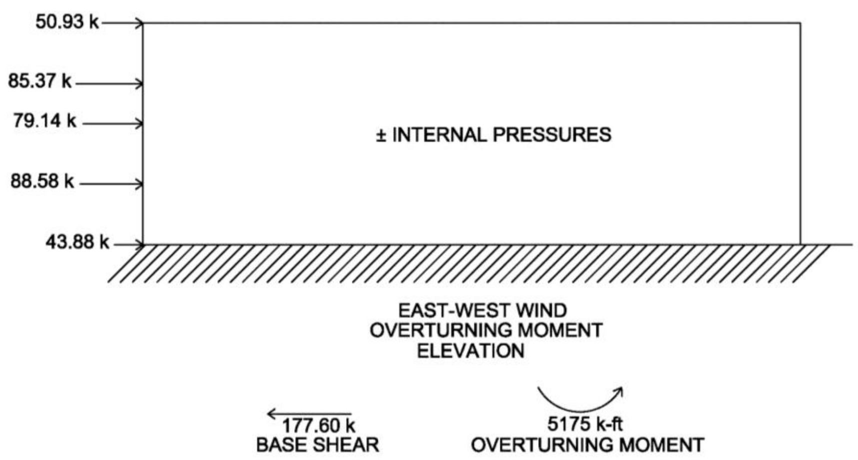


Figure 18 : Summary of final forces in East-West direction in elevation.

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	8.86
		Floor 4	47.5	0.8	16.82	17.63	0.853	0.18	8.45
		Floor 3	35	0.8	14.80	17.63	0.853	0.18	7.43
		Floor 2	17.5	0.8	12.16	17.63	0.853	0.18	6.11
		Ground	0	0.8	11.55	17.63	0.853	0.18	5.80
	Leeward	All	All	-0.5	17.63	17.63	0.853	0.18	-10.69
	Side	All	All	-0.7	17.63	17.63	0.853	0.18	-13.70
Roof		0 to h/2	0 to 32	-1.0	17.63	17.63	0.853	0.18	-18.21
		h/2 to h	32 to 64	-0.8	17.63	17.63	0.853	0.18	-15.20
		h to 2h	64 to 128	-0.5	17.63	17.63	0.853	0.18	-10.69
		>2h	>128	N/A	17.63	17.63	0.853	0.18	N/A
							N-S load	Sum Wall	36.66
								Sum Roof	-44.11

Table 19 : Summary of wind pressure calculations in the North-South direction.

Overturning Moment/Base Shear North-South Direction									
Windward Wall	Location	Height	Area Below (ft^2)	Area Above (ft^2)	Pressure Below (psf)	Pressure Above (psf)	Factored Story Load (k)	Factored Story Shear (k)	Overturning Moment (k-ft)
	Roof	64	1662.5	0	19.14	19.55	50.93	50.93	3259
	Floor 4	46.5	1187.5	1662.5	18.13	19.14	85.37	136.29	3969
	Floor 3	35	1662.5	1187.5	16.80	18.13	79.14	215.43	2770
	Floor 2	17.5	1662.5	1662.5	16.50	16.80	88.58	304.00	1550
	Ground	0	0	1662.5	0	16.50	43.88	347.89	0
	Width (ft)	190				Factored Total Base Shear (k):	347.89	Total Overturning Moment (k-ft):	9998

Table 20 : Summary of overturning moment and base shear calculations in the North-South direction.

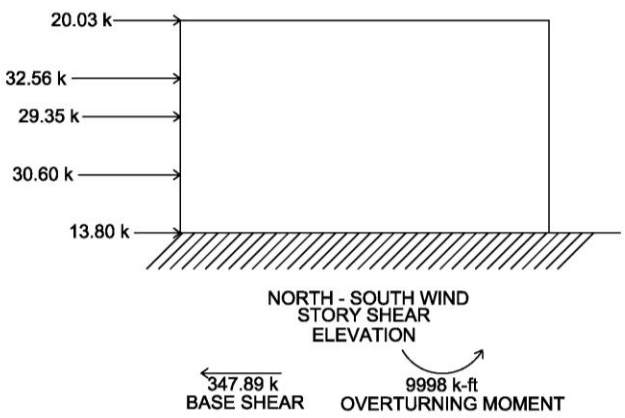


Figure 21 : Summary of final forces in North-South direction in elevation.

Wind Loads Per Floor - Hand Calculations							
Level	Height	North-South Direction			East-West Direction		
		Story Load Windward	Story Load Leeward	Total Force	Story Load	Story Load	Total Force
Roof	**64**	22.48	-28.44	50.93	13.55	-13.54	27.09
4th	**46.5**	36.61	-48.76	85.37	21.39	-22.44	43.83
3rd	**35**	30.38	-48.76	79.14	17.63	-22.44	40.07
2nd	**17.5**	31.69	-56.88	88.58	17.89	-27.08	44.97

Table 22: Hand calculations for hand 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 23, 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 Technical Report III. 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.

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 Technical Report III. 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%.

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	0.042

Figure 23: Table of seismic load variables and values.

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 24 and summarized in Figure 25. 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 Technical Report III. 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 24 : Summary of calculations for seismic load design.

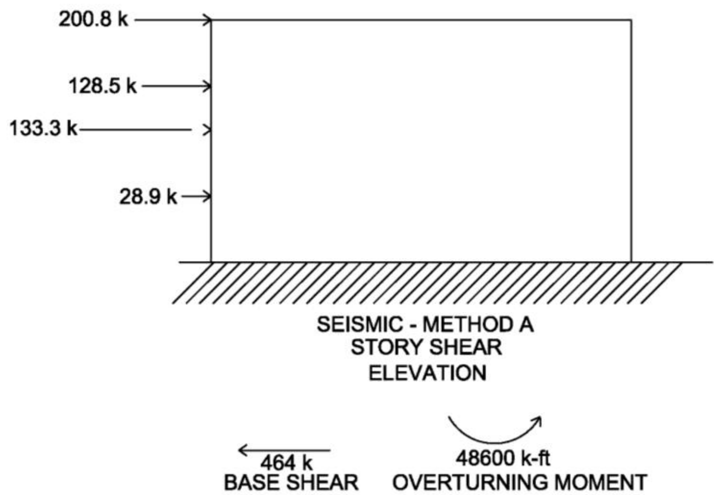


Figure 25 : Summary of forces due to seismic loads.

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 26 and 27. 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.

Comparison of Lateral Forces			
	Wind, North-South	**Wind, East-West**	**Seismic**
Base Shear (k)	347.9	177.6	***493.5***
Overturning Moment (k-ft)	***11548.5***	5961.6	6392.5

Table 26: Comparison of Lateral Forces

Comparison of Story Forces (k)			
Level	**Wind, North-South**	**Wind, East-West**	**Seismic**
Roof	50.9	27.1	***200.8***
Mech Roof	Neglible	Neglible	***2.0***
Floor 4	85.4	43.83	***128.5***
Floor 3	79.14	40.07	***133.3***
Floor 2	***55.58***	***44.97***	28.9
Ground	N/A	N/A	N/A

Table 27: Comparison of Story Forces

Problem Statement

The SteelStacks Performing Arts Center is designed to satisfy the purposes of the space through variations in floor plans, bays, structural components, and a dual system of shear walls and braced frames for the lateral structural system. While this is a successful design, this layout creates an extreme torsional irregularity, as defined by Chapter 12 of ACSE 7-05 with an amplification factor greater than 1.2, with this factor reaching 1.6 for the North-South axis, as per Technical Report III. Because of the large quantity of shear walls and braced frames, the building is well designed and stiffened for this loading and minimal deflection is seen. Yet, shear walls are more difficult to make ductile, and the existing walls take a majority of the lateral load. In addition, the shear walls in the SSPAC require more connection detailing where the diaphragms connect to the shear walls due to the building layout.

In response to these difficulties met by the engineer, a scenario has been created in which the walls are no longer allowed to be laterally load bearing. The goal of this redesign is to eliminate connection issues between the shear walls and diaphragms by going through the steel beams and girders into an entirely steel lateral system. With walls no longer being load bearing, a new lateral system will need to be designed that can withstand the torsional irregularity, with the torsional irregularity being minimized through the redesign of the gravity and lateral systems.

Therefore, a structural system will be designed that contains solely steel braced frames as the lateral system, with the existing walls all being evaluated as nonstructural. These walls will be evaluated for architectural use, and for their feasibility in reaching the architect's intent for the façade. The floor diaphragms will also be considered a point of evaluation for redesign due to the weight impacts on the lateral system. With upper floors being heavier due to acoustic issues, these will be redesigned with acoustics as a consideration. This alteration in the gravity system will consider the viable systems discussed in Technical Report II. All of this must be achieved while considering impact on the architectural and acoustical qualities of the structure.

Proposed Solution

The redesign of the existing lateral system will begin with the design of braced frames along the north-south axis. The new lateral system will be an entirely steel system, which will aid in creating a more efficient structure for design, construction, and cost while maintaining quality in architecture and acoustics. An entirely steel system allows for more ductility to be included in the design, and is thus an added benefit to the design requirements.

Existing walls will all become nonstructural, and the use of Exterior Insulation and Finish Systems (EIFS), a lightweight curtain wall panel system, will be considered as a viable cladding option to replace the precast concrete panels. This will aid in elimination of extreme torsional irregularity through removing the stiff shear walls. Evaluation of the impact of these walls on the torsional irregularity will implement the use of structural modeling software, primarily RAM Structural, for analysis of the torsional irregularity, combined with a spreadsheet for confirmation of these calculated values.

The gravity system will then be designed to minimize cost and weight. Currently, the system is designed for consistent size members for aesthetics, as ceilings are exposed. This redesign will consider the impact of altering sizes and minimizing overdesign to eliminate cost. This will influence the architecture and aesthetics of the building, and this impact will be considered and is detailed below in the Breadth section. The structural framing members will be designed using the 14^{th} edition of AISC.

Floor diaphragms will be redesigned to lighten the building self-weight, with acoustics and floor vibrations being taken into consideration, as acoustics were a controlling factor in creating the existing design. Variations in the floor system will be considered, such as lightweight concrete slab, and those discussed in Technical Report II. Vibration issues will be analyzed for performance, and will be considered for the mentioned floor diaphragm options. Acoustics of each of these options will be analyzed for Impact Isolation Class, which will then be utilized for deciding on the most viable floor option. The ramifications of the new diaphragm design on the acoustic performance of the spaces are detailed in the Breadth section below. The Vulcraft Steel Decking Catalog will be utilized for diaphragm design for existing normal weight and possible lightweight concrete.

Breadth Study

Redesign of the SSPAC for the above mentioned limitations will have a direct impact on various other aspects of the building design, as previously stated. These influences include architectural design, acoustics of each of the altered spaces, construction, and mechanical location and vibration issues. The breadths being considered for this proposal are acoustics and architecture and are elaborated below.

Acoustics:

Eliminating shear walls and reconfiguring the framing system will directly impact the acoustics of the building spaces. Interior walls will need to be reevaluated, and a double wall system will need to be installed surrounding public spaces, such as the cinemas, to maximize noise isolation. By changing the framing plan arrangement, a primary influence would be on the acoustical performance of each of the spaces where the floor diaphragms are designed for sound isolation. One such space that will be impacted is the third floor Musikfest Café and Stage area. A heavier floor system allows for better sound isolation between floors. By lightening the floor system, the chosen design might no longer provide a satisfactory acoustic design. Therefore, the floor diaphragms will be analyzed for effectiveness as sound barriers. To analyze the acoustic performances of the space in each option, Impact Isolation Classes will be decided per wall and floor material. Existing and alternatives options will be compared, to conclude on the most viable option according to acoustic performance for the spaces.

Architecture:

By changing the bay layouts and exterior wall system, architectural features will be impacted. By replacing the existing precast concrete wall panels with an alternative nonstructural wall system, the façade will be greatly altered, and this fact will need to be considered while making replacement choices. The existing architecture also includes exposed ceilings with consistent beam, girder, and truss member sizes for a streamlined look. The proposed redesign includes less constant member sizes to eliminate superfluous building weight and cost, and will result in fewer members all the same size. The impacts of these system alterations will be visually considered through the use of a Revit model, giving the ability to compare the existing with the new design more exhaustively. A final architectural view will be provided to display the impacts of the design.

MAE Component

As a requirement for the MAE program, the coursework from multiple MAE classes will be incorporated into the completion of this thesis. For completion of the depth, a structural building model will be built in RAM Structural System. This follows the material learned in AE 530, *Computer Modeling of Building Structures*. Use of a detailed structural model will aid in the analysis of building and member loads. Concepts implemented include panel zones, and rigid diaphragm constraints. With the further details of the structural system redesign, material from AE 534, *Analysis and Design of Steel Connections*, will also be applied the investigation of connection design. Typical connections will be designed, including those for the gravity system, such as truss connections, and for the lateral system, such as connections for the braced frames.

Tasks & Tools

I. Structural System – Lateral System
   a. Establish most effective location for east-west braced frames
      i. Consider existing wall and column line locations for braced frame locations
      ii. Take architectural features (walls, windows, spaces) into consideration
   b. Establish lateral loads on system
   c. Using a computer modeling program, determine member loads, confirming with hand calculations
   d. Redesign lateral system for fully concentrically steel braced system, incorporating EIFS
   e. Analyze effectiveness of eliminating nonstructural walls

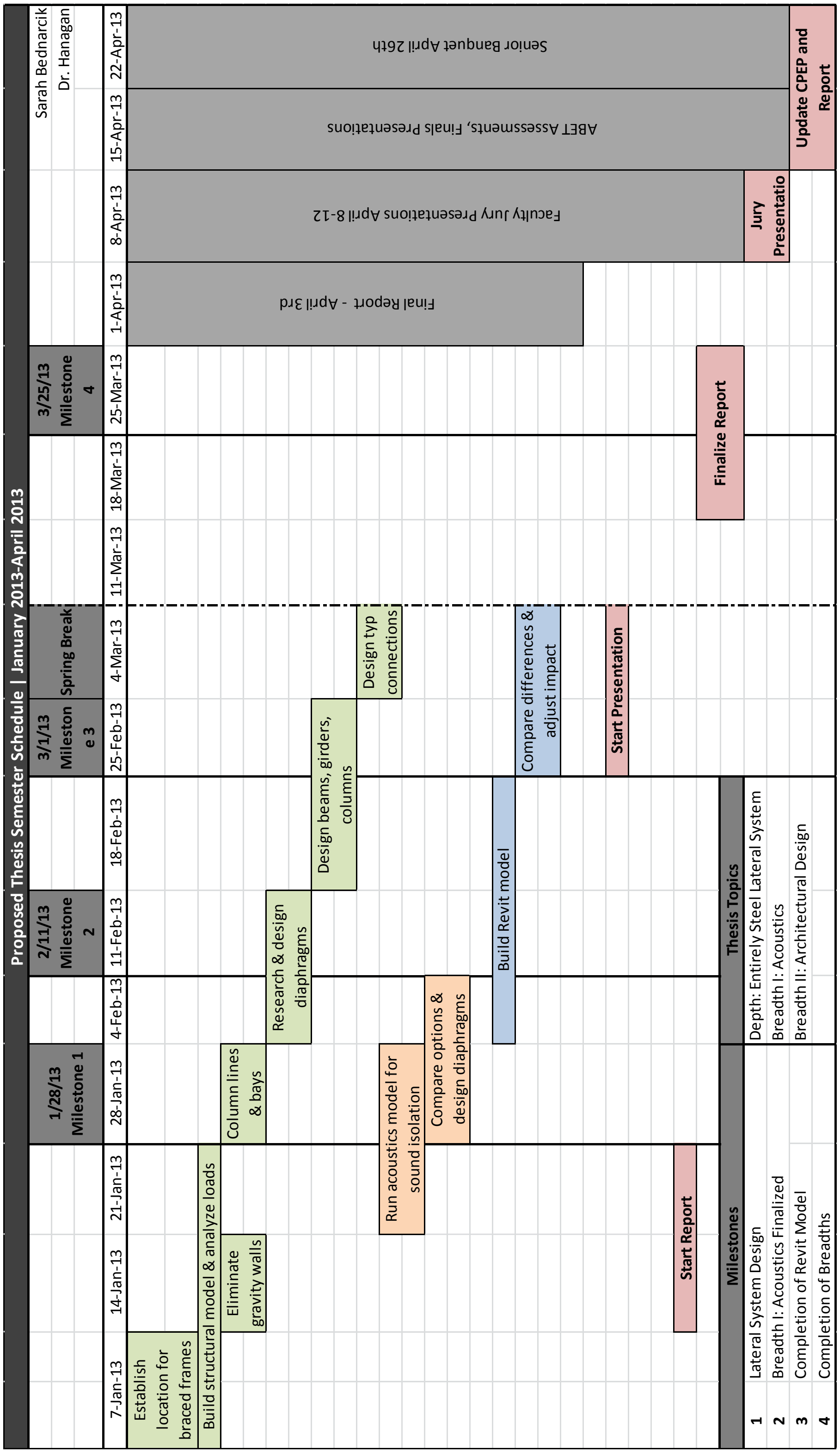
II. Structural System – Gravity System
   a. Adjust column lines & bay configurations, due to impact from eliminated walls
      i. Reconfigure diaphragm for more effective lateral load transfer
      ii. Consider ramifications of this on space requirements, if any
   b. Analyze loading from above spaces on beams and girders
   c. Design diaphragms
      i. Research options for comparison of other effective systems
      ii. Design chosen system
   d. Design beams/girders/columns in typical bay by hand for loading
   e. Design typical connections
   f. Consider ramifications on foundation design

III. Breadth II: Acoustics: Musikfest Café and Stage Area
   a. Research impact of different floor systems on acoustics
   b. Compare options through use of acoustics analysis for sound isolation
   c. Include this in analysis and design of floor system

IV. Breadth X: Architectural
   a. Build initial Revit model for direct comparison of existing building design with new lateral and gravity systems
   b. Look at primary façade impacts – compare to existing
      i. Evaluate issues new structural design may bring
   c. Look at impact on primary interior spaces via comparison of Revit Models
      i. Impact of beam and girder sizing
      ii. Impact of column and shear wall locations
   d. Adjust wall and framing configuration for major architectural issues to minimize impacts, if necessary

Timetable

A weekly schedule has been developed, summarizing the main tasks discussed above, with semester and target dates provided to give a representation of individual-led goals throughout the thesis process.



Conclusion

In response to difficulties met by the structural engineer, a scenario has been developed in which the lateral system is no longer allowed to include shear walls, but must implement only a steel lateral system. The structural depth solution to this scenario implements additional braced frames in both directions, allowing for more ductility to be designed into the system. Existing walls will become nonstructural, with alternatives, including Exterior Insulation and Finish Systems (EIFS), being considered. The gravity system will be redesigned with a goal of eliminating superfluous cost and material. Floor diaphragms will be redesigned to lighten the building weight, considering options such as lightweight concrete.

The solution for the structural redesign will impact the architecture and acoustics of the spaces. With both aesthetics and acoustics being important factors in the original design, they are also necessary considerations for the redesign. Public spaces will be impacted by the structural changes due to acoustic issues, such as in the stage and Musikfest Café area. With the floor system being redesigned, acoustical performance of each floor system will be studied and taken into account when choosing the final gravity system. In architectural design, the façade and public spaces will be altered due to the wall and gravity systems being redesigned. For this to be thoroughly evaluated, an architectural model will be utilized to compare the existing and redesigned spaces, with major impacts being studied for possible alteration.

In addition, MAE coursework will be included in this thesis project, and has been incorporated into this proposal. Material from AE 530, *Computer Modeling of Building Structures,* and AE 534, *Analysis and Design of Steel Connections*, will be utilized to provide a more complete project, and the implementation has been elaborated on in this report.