SteelStacks Performing Arts Center | Bethlehem, Pennsylvania

Reevaluation of the SteelStacks Performing Arts Center

Bethlehem, Pennsylvania

THE PENNSYLVANIA STATE UNIVERSITY
SCHREYER HONORS COLLEGE

DEPARTMENT OF ARCHITECTURAL ENGINEERING

Sarah A Bednarcik
Spring 2013

A thesis submitted in
partial requirements for degree in Architectural Engineering
with honors in Architectural Engineering

Reviewed and approved* by the following:

Linda Hanagan
Associate Professor
Thesis Supervisor

Ali Memari
Hankin Chair Professor
Honors Advisor

*Signatures are on file in the Schreyer Honors College

SteelStacks Performing Arts Center

Bethlehem, Pennsylvania

Sarah Bednarcik

Structural

General Information:

Function: Arts & Cultural Center

Size: 67,167 sq. ft.

Height: 64 feet, 4 stories

Construction: Jan 2010 - Apr 2011

Project Cost: $48 million

Delivery: Design-Bid-Build

Construction:

Coordination between designers and builders important, as plans finished after breaking ground

Architecture:

Façade: Textured precast concrete panels & a glass curtain wall system

Emphasis: Exposed structural steel and large, open atrium spaces

Spaces: Creative commons, cinemas on lower floors; café & stage on upper floors

Challenges: Historic site requiring material approval

Structure:

Foundation: Concrete column piers and footings supporting slab-on-grade

Superstructure: Composite slab and metal decking supported by beam, girder, and truss system

Lateral: Braced frames and precast panels acting as shear walls

MEP Systems:

Mechanical: VAV with Reheat

6 Rooftop AHUs at 1650 to 23500 CFM, 1 interior AHU at 5300 CFM

150 total tons of Direct Expansion cooling

Lighting: Façade lit by halides

Interior includes a mixture of LEDs, fluorescents, incandescents

Electrical: 200 kW back-up generator

Main power 480 V, 3 phase, 4 wire

Fire protection: Wet pipe fire suppression system

Project Team:

Owner: ArtsQuest

Architect: Spillman Farmer Architects

Structural: Barry Isett, Assoc., Inc.

MEP: Brinjac Engineering

CM: Alvin H. Butz, Inc.

Civil: French & Parello

Acoustic: Acoustic Dimensions

Images courtesy of respective design professionals.

Abstract

The purpose of this report is to complete a thorough analysis and redesign of the structural system of the SteelStacks Performing Arts Center (SSPAC) and compare these results to the existing building, evaluating this redesign. The SSPAC is a 64-foot, 4 story, 67,000 square foot arts and cultural center with a steel gravity system and a dual lateral system comprised of braced frames and shear walls.

This report culminates the work of a semester of research and redesign, at the end of which a scenario was created in which the architect wanted to explore cast-in-place concrete as an alternative design option. The new design was decided to include a fully concrete gravity and lateral system. Additionally, the floor system was evaluated and chosen between different systems, a reinforced one-way slab and beam system and a prestressed system.

The goal of this redesign is to evaluate the benefits of both the existing steel system and a reinforced concrete system through a comparison of the benefits and issues with each. This analysis necessitated considering benefits and disadvantages including the structural benefits to each system, flexibility in design, cost, and construction.

The proposed redesign and change in materials resulted in a need to evaluate the acoustic performances of these spaces. This acoustic breadth considered both floor systems and the impact of these materials on the sound transmission as well as the reverberation time within each space.

Results from this analysis led to the conclusion that concrete benefits the system in terms of the many cantilevers and framing configurations seen throughout the SSPAC, while steel is a continued benefit in other areas if the layout is kept the same. If concrete were to be implemented, the building would benefit from seeing some slight changes to layout and structural design.

Cost and construction were seen as more effective in steel. The considerations on acoustics and architecture resulted in successful adjustments to the concrete structure to create a more effective design. Acoustics in particular, were improved through the use of concrete, as it is naturally a better system for the required sound isolation and reverberation of the spaces in the SSPAC.

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Acknowledgements

I am very grateful to Barry Isett Associates and ArtsQuest for providing me with the opportunity to use the SteelStacks Performing Arts Center. An especial thank you goes to Ross Sotak, who gave valuable insight and advice throughout this process.

The AE faculty also deserves special thanks, for their passion and help throughout my time at Penn State. This especially includes:

Dr. Hanagan

Dr. Vigeant

Kevin Parfitt & Robert Holland

My fellow classmates, who spent hours working and encouraging me and helping me, include:

Jon Fisher

Andrew Voorhees

Jon Gallis

Sean Felton

Dan Bodde

Matthew Neal

Cheuk Tsang

Rose Paddison

Allie Pladson

My family has been a never-ending support system, and through their encouragement and support, I have come through better reminded of why I enjoy what I study.

Most importantly, I thank Christ Jesus, for giving me a reason to enjoy my work and purpose for it all.

Chapter 1: Building Introduction

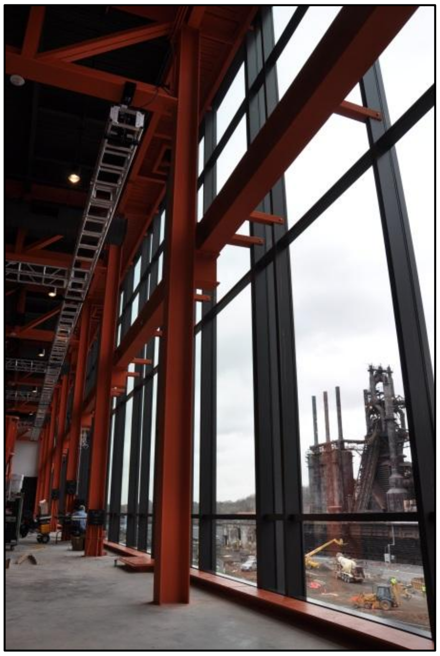


Figure 1: Interior atrium space, highlighting opening structural plan.

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.

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 A.

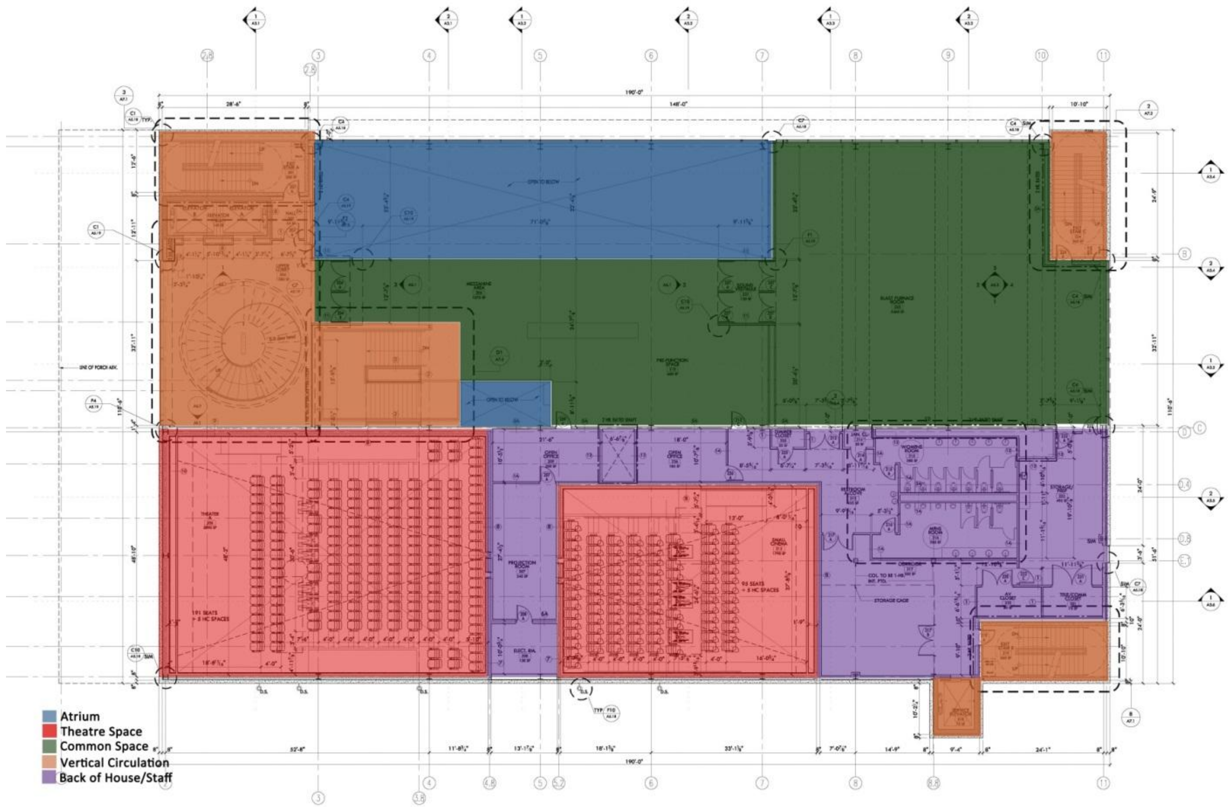


Figure 2: Floor Plan from A2.2

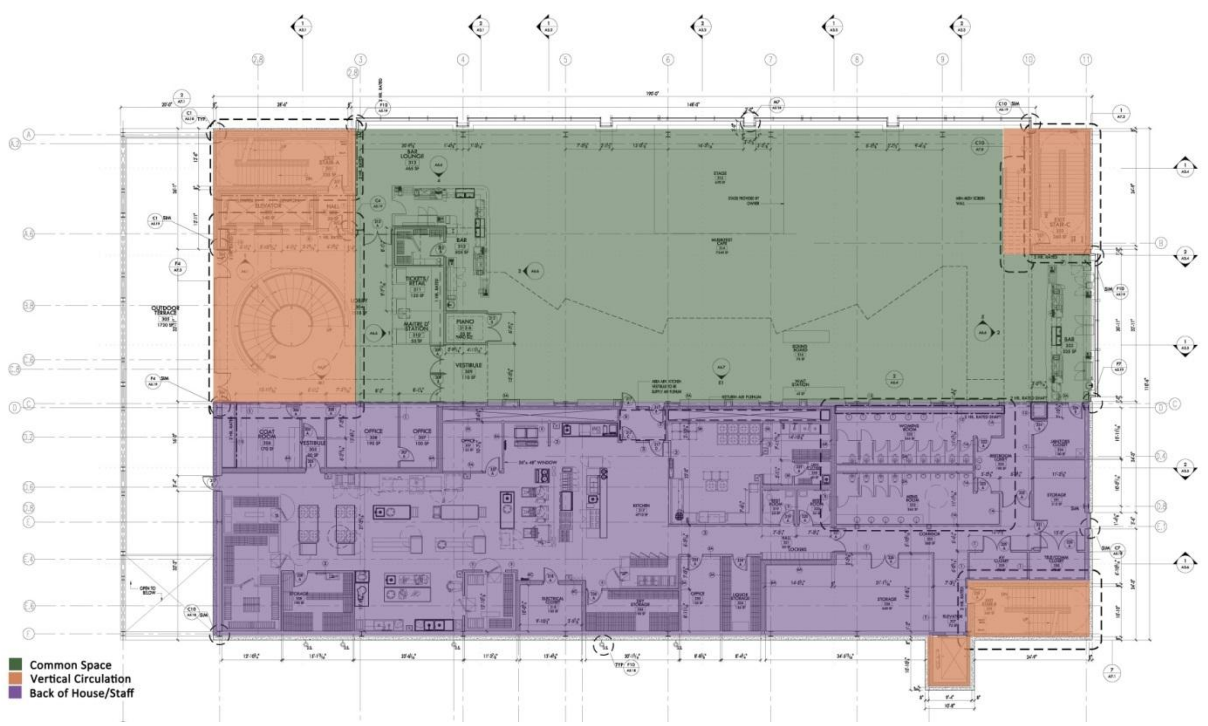


Figure 3: Third Floor Plan from A2.3

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 4.

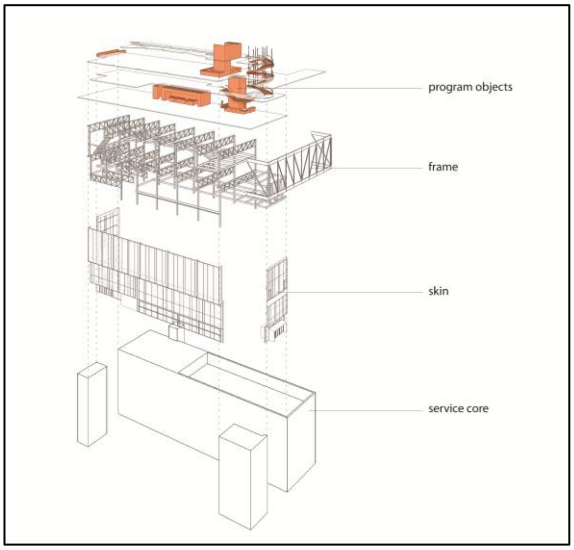


Figure 4 : Image displaying the separation of spaces through the structural design.

Images courtesy of Barry Isett, Inc. & Assoc.

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.

1.1 Existing Structural System

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.

1.1.2 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.

1.1.3 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 A.

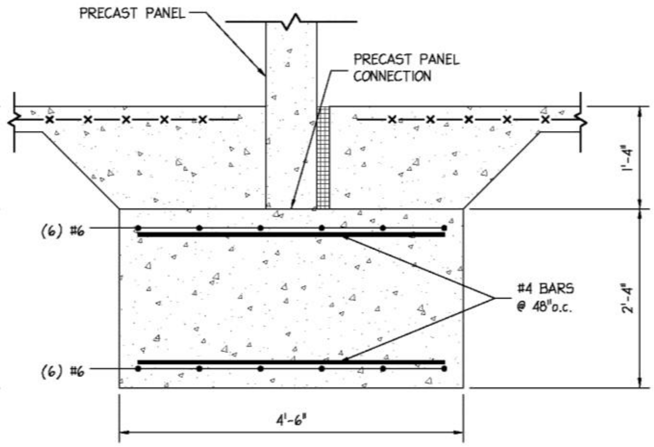


Figure 5 : Section of foundation to precast panel connection from S1.0.

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.

1.1.4 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

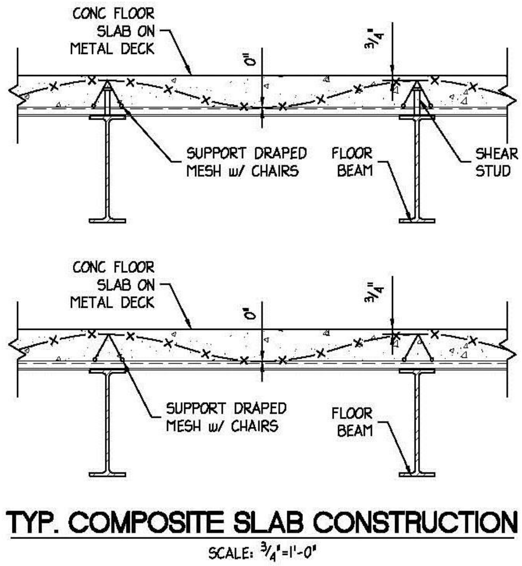


Figure 6 : Typical composite slab section for building from S2.8

spaces, requiring more vibration and sound isolation around the stage for band performances. Therefore it is a galvanized epicore 20GA roof deck, an acoustical decking and slab system.

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.

1.1.5 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.

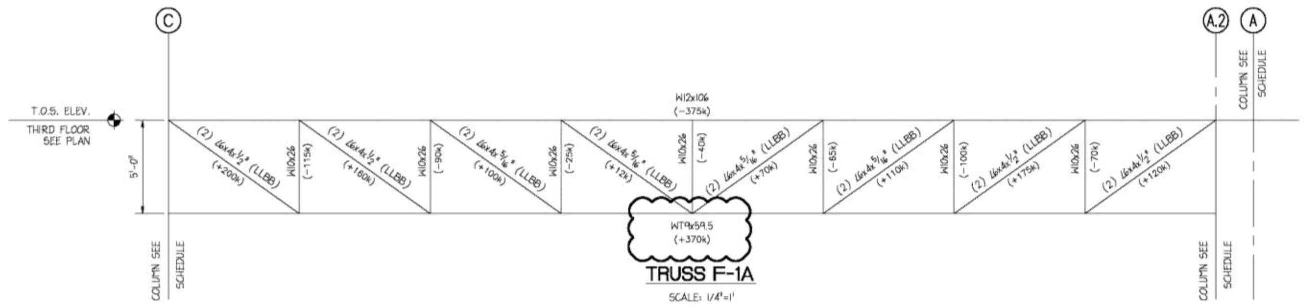


Figure 7 : Third floor representative framing system truss from S2.6.

Framing on the fourth floor is more irregular, as explained previously and included in Appendix A, 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.

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 A. 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.

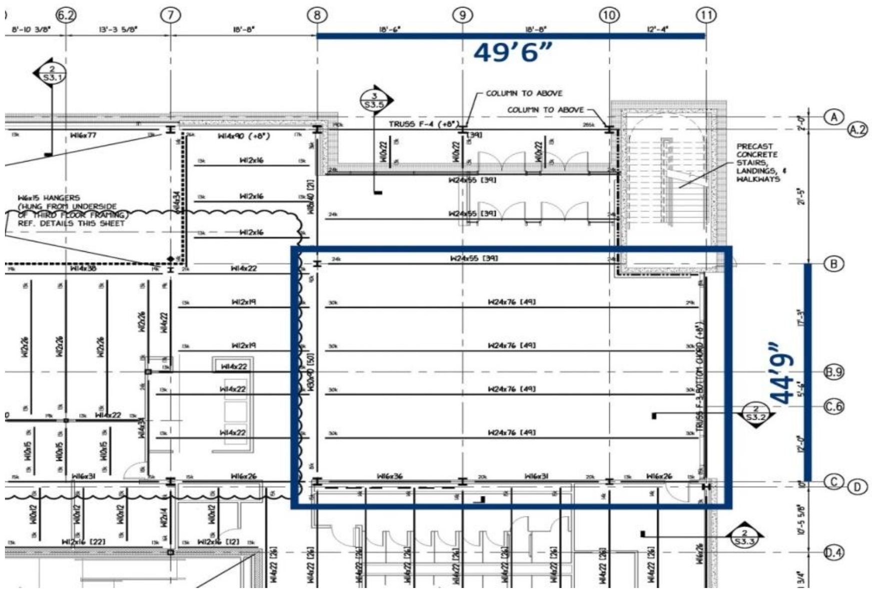


Figure 8: Second floor framing plan, representative bay of a typical frame, from S2.0

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 9for 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.

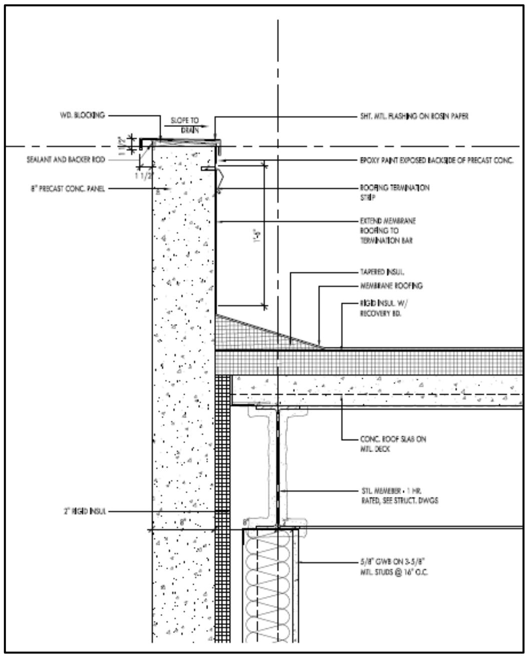


Figure 9 : Cross section of the roofing system.

1.1.6 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 10and are comprised of W10x33s for diagonal members and W16x36s for horizontal members. An elevation of these lateral systems is included in Appendix A.

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.

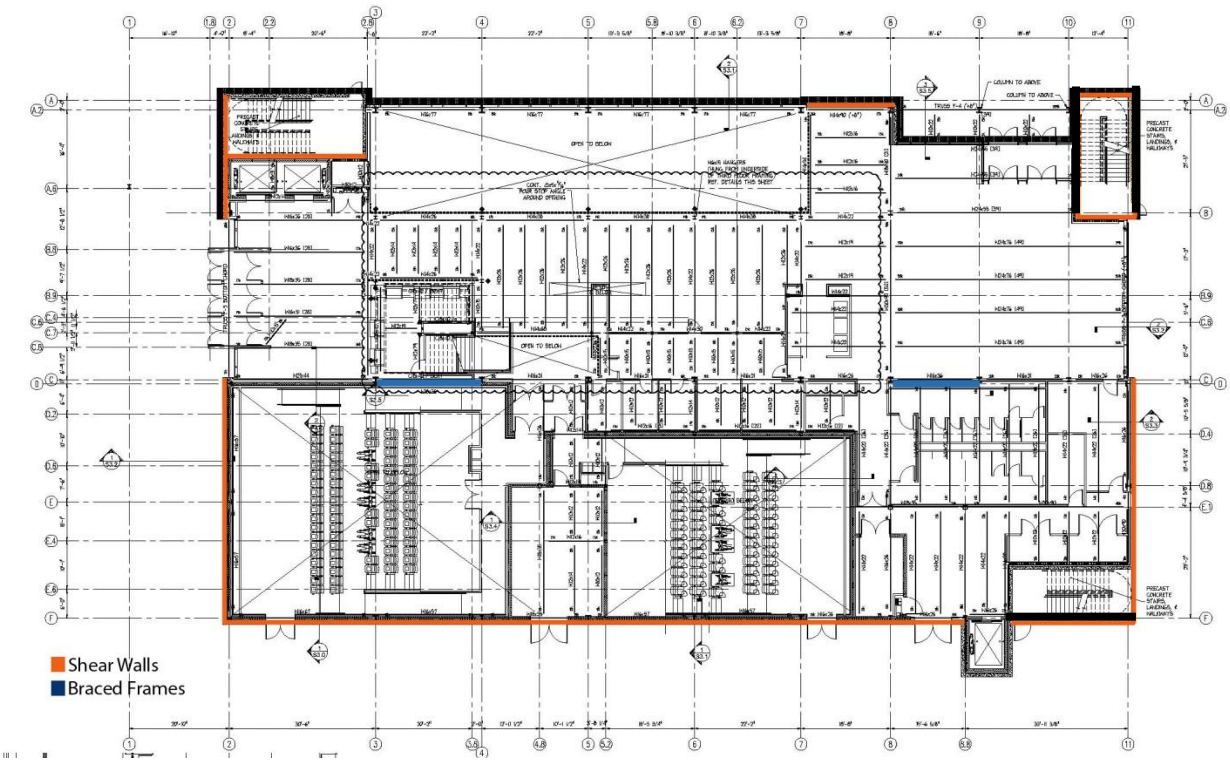


Figure 10 : Floor plan highlighting shear walls and braced frames, contributing to the lateral system.

1.2 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

1.3 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.

1.4 Gravity 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.

1.4.1 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 1.

Table 1: Table of Superimposed dead loads

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

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 2. 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.

Table 2: Table of live loads used on the structural plans and in this report

*Dashes designate values not provided in the structural drawings.

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

1.5 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 Appendix B (Wind) and Appendix C (Seismic).

1.5.1 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 11. 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.

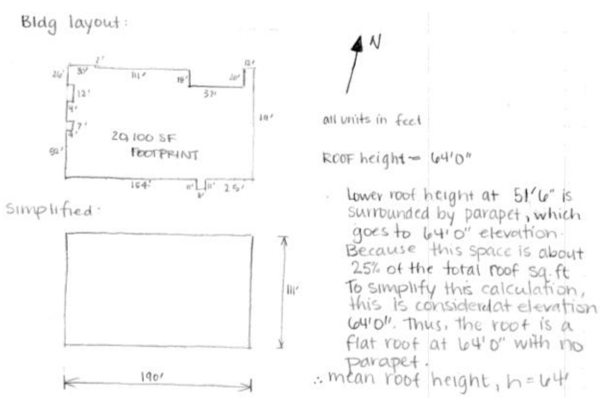


Figure 11 : Building dimensions simplified for wind load calculations following Method 2.

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 the tables and figures following. These results have been summarized for the East-West direction in Table 3, Table 4, Figure 12, and Figure 13 and highlight the base shear and overturning moment due to these wind pressures. Table 5, Table 6, Figure 14, and Figure 15 summarize similar results and drawings for the North-South direction. Table 7gives 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.

Table 3: Summary of wind pressure calculations in the East-West direction

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 4: Summary of overturning moment and base shear 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)	Story Load (k)	Story Shear (k)	Overturning Moment (k-ft)
Windward Wall	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
						Total Base Shear (k):	105.51	Total Overturning	3159
	Width (ft)	111							

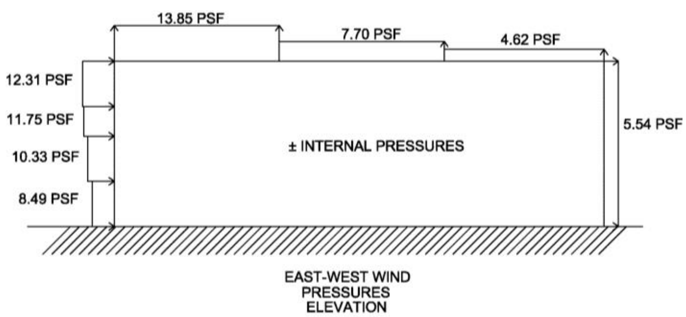


Figure 12: Summary of East-West wind pressures in elevation.

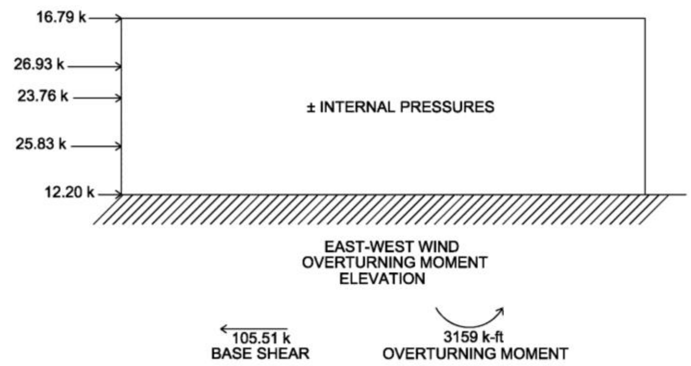


Figure 13: Summary of final forces in East-West direction in elevation.

Table 5: Summary of wind pressures in the North-South direction.

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

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							

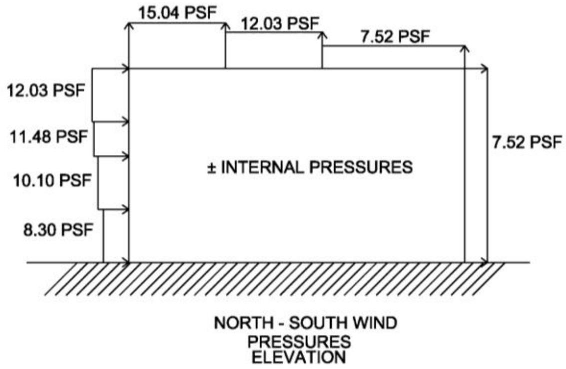


Figure 14: Summary of forces in the North-South direction in elevation.

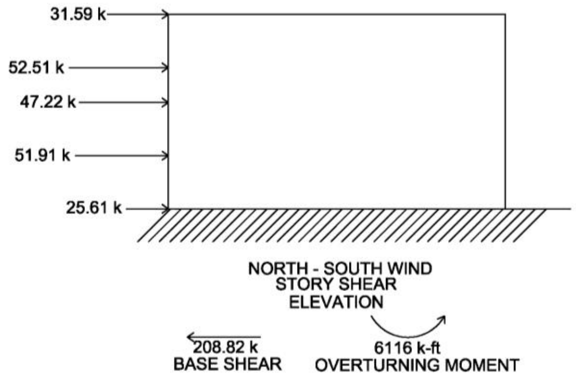


Figure 15: Summary of final forces in North-South direction in elevation.

Table 7: Hand calculations for wind loads per floor.

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

1.5.2 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 8, 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 C. 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

Table 8: Table of seismic load variables and values.

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

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 9 and summarized in Figure 16. 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 C. Accidental torsion impacted the seismic loads, and these values can be found later in this report.

Table 9: Summary of calculations for seismic load design.

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

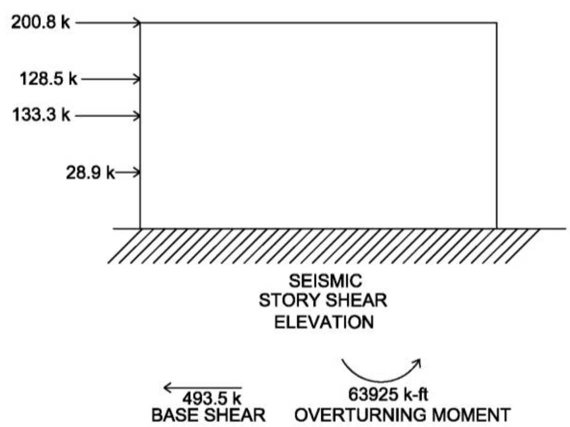


Figure 16 : Summary of forces due to seismic loads.

1.5.3 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 Table 10 and Table 11. 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.

Table 10: Comparison of lateral forces

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 11: Comparison of story shears

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

1.6 Problem Statement:

The SteelStacks Performing Arts Center is designed as a steel gravity system with braced frames acting as the lateral system. This is done effectively in the design by variations in floor plans, bays, structural components to result in a framing consists of a composite decking and steel system. The lateral system is designed as a dual system of shear walls and braced frames for the lateral structural system.

A scenario has been created in which the architect would like to explore an alternative option, and the building is required to be built in reinforced concrete. Through the observations made in Technical Report II, this is a viable system redesign for comparison to the existing system. Other alternatives, such as a precast plank floor system, have been disqualified due the inconsistency in bay layouts.

The goal of this redesign is to evaluate the benefits of both the existing system and a reinforced concrete system in a comparison of variables such as structural performance, cost, efficiency, aesthetics, and acoustic performance. With a concrete system in place, braced frames will no longer be a viable lateral system option, and therefore, shear walls will be reconfigured and replace braced frames. The gravity system will be evaluated and redesigned, with larger bays being considered for prestressing.

Therefore, a structural system will be designed with the existing gravity system and lateral system being converted to a reinforced concrete system. One ramification that will need to be considered is concerning floor to floor height, and this will be evaluated as part of the redesign. This redesign will impact the aesthetics, and the redesign will be evaluated for architecture and compared to the existing interior spaces. As the gravity system is being redesigned into concrete, these will also be considered a point of evaluation 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. All of this must be achieved while considering impact on the architectural and acoustical qualities of the structure.

1.7 Proposed Solution:

The redesign of the existing lateral and gravity systems will begin with the consideration of the new shear wall layout along the east-west axis. The new lateral system will be an entirely shear wall system, which will be compared to the existing system for design, construction, and cost while maintaining quality in architecture and acoustics.

The gravity system will then be designed in consideration of 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 a reinforced concrete system that mainstreams bay layouts on cost of materials and construction. 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 ACI 318-11.

Floor diaphragms will be redesigned while maintaining the necessary floor-to-floor dimensions currently in use, with acoustics and sound isolation being taken into consideration, as acoustics were a controlling factor in creating the existing design. Sound isolation issues will be considered for the mentioned floor and space design. Acoustics will be analyzed for Sound Transmission Contours related to each highlighted space, which will then be utilized for deciding on the most viable floor and space options. The ramifications of the new diaphragm design on the acoustic performance of the spaces are detailed in the Breadth section below.

1.7.1 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 braced frames and reconfiguring the framing system for a reinforced concrete system will directly impact the acoustics of the building spaces. Interior walls will need to be reevaluated, and acoustic paneling and materials will be adjusted according to calculation results, 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 altering 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, Reverberation Time values will be decided per room, based on 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 designing shear walls and changing the system to concrete, the interior spaces will be greatly altered, and this fact will need to be considered. The existing architecture also includes exposed ceilings with consistent beam, girder, and truss member sizes for a streamlined look. The proposed redesign continues to include constant sizes, but the use of a different material will impact the aesthetics. 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.

1.7.2 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 CE 543, *Prestressed Concrete Behavior and Design*, will also be applied the investigation of the gravity system design. Larger bays will be evaluated for the benefits of designing these bays for prestressing, and will be detailed for the appropriate design results.

Chapter 2: Structural Depth

The SSPAC is a braced steel frame system with precast concrete panels acting as shear walls. As a building with irregular bays, big curtain wall expanses, and heavy live loads, a steel system is a very effective option that allows for the structural components being designed to match the existing layout.

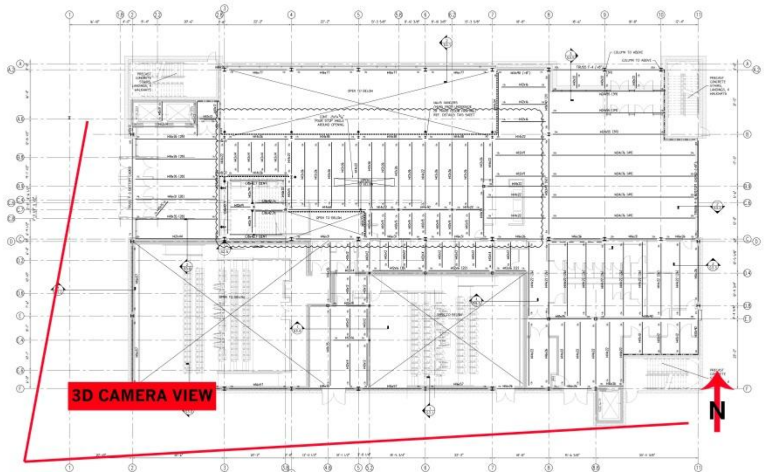


Figure 17: Camera view for 3D images

A fully concrete building can also be designed effectively from these same issues. The use of concrete lends easily to varying bay configurations, and is beneficial in areas of heavier loadings. Other benefits in a concrete redesign are in the areas of large cantilevers and atrium spaces. Concrete is known to be an effective solution for cantilevers where back spans are included, and is therefore a competitive option. Due to the large floor to floor height already in place because of the deep trusses used to support the large live loads and longer spans, ranging from 4'0" to 7'0" in depth, the use of concrete beams in these locations is an excellent area to explore in redesign. The lateral system exists as a dual system, comprised of shear walls and braced frames. Concrete fits into this framing scheme seamlessly, as shear walls can then act more integrally with the gravity system. Additionally, the construction equipment for the concrete gravity system will already be on site, so a cast-in-place, versus continuing with precast shear walls, is a more logical decision. The existing system is controlled by seismic loads, and with the conversion to an entirely concrete system, seismic loads are expected to continue controlling.

Therefore, the building redesign consists of a one-way slab and beam system with columns and gravity walls. This design is integrated with the lateral system, a set of shear walls in both directions, as seen in Figure 18 (gravity walls are hidden for ease of view), as viewed from the camera view in Figure 17.

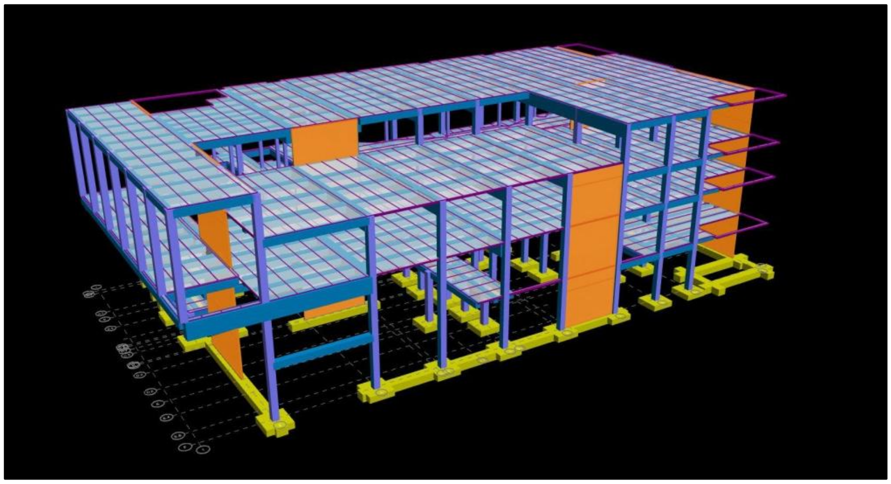


Figure 18: 3D view with shear walls in orange, other walls eliminated for visibility.

To follow a logical design process, the gravity system was first designed, starting with the slab and decking system being replaced by a one-way slab system, with beams and girders being redesigned in concrete. A controlling bay from each floor was chosen and designed, and these results are elaborated on below in the Gravity Redesign portion of this chapter. After the bays were designed, controlling columns were designed and the size estimated for the above calculations were confirmed or adjusted as necessary. Once the gravity system was designed, the lateral system, comprised of shear walls in both directions and highlighted in Figure 18, was analyzed and designed.

RAM Structural System (RAM) was used to develop full gravity and lateral models, and these were confirmed via the hand calculations. The use of RAM helps to meet the MAE requirements of this thesis. The development of the model will be explained more thoroughly in the appropriate portions of this thesis.

2.1 Gravity Redesign

This redesign minimizes changes in the architectural layout, while focusing on specific bays and structural components that control structurally. Due to the architectural features and design controlling layout, none of the floors have the same diaphragm, as seen in Figure 19. For a clearer understanding of the chosen bays and places of focus, it is necessary to develop a thorough understanding of where the controlling areas are and where concrete can benefit the current building layout.

As can be seen highlighted in Figure 19, fifty foot spans exist on many of the floors, as well as cantilevered areas highlighted in blue. These areas are benefitted by the use of concrete, which naturally utilizes back spans in the layout to strengthen cantilevered sections. The gravity redesign incorporates some of these controlling factors. Existing design includes hanging columns on the fourth floor and second floor, and elimination of these (and replacement by cantilevers) is evaluated.

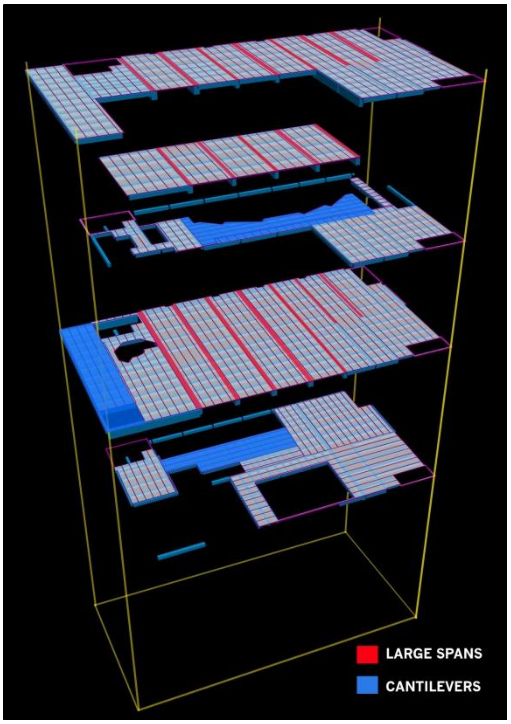


Figure 19: Expanded view highlighting focus spans and cantilevers.

Floor to floor height is another added benefit to concrete. Since these floors have large ceiling heights for space use and aesthetics, any additional space saved within the structural system is important. By utilizing concrete, structural floor to floor height could be saved while maintaining architectural ceiling height and minimizing some cost.

Acoustic isolation is naturally a benefit of concrete on each of the primary spaces. This is in contrast to the existing steel system, which requires additional acoustic control and sound isolation. The benefits to the acoustics of the spaces will further be considered in Chapter 3: Acoustic Breadth.

Since floor plans are not consistent from floor to floor, a controlling bay from each floor was first designed to develop a floor slab depth and typical bay design, as can be seen in Figure 20. A one-way slab system has been developed for the SSPAC, with series of beams and girders taking this load into the columns.

Ram was utilized for aiding in the design of the gravity system. Initially, the model was created with the existing dimensions and layouts, with gravity member sizes estimated based on preliminary hand calculations. Once this preliminary design was in place, it was run to check member sizes for the loadings assigned. Iterations followed, including reinforcement design and placement of members adjusted accordingly. These methods and results are elaborated below for each type of structural component.

Hand calculations, including the use of a spreadsheet, for the slab depth, beam sizes, and girder sizes were utilized to confirm the output loads and designs from RAM. These resulting structural component sizes were then updated in the RAM model and concrete and reinforcement design were confirmed through the RAM Concrete design module.

Due to the inconsistency of bay sizes, the use of ACI 318-11 §8.3 cannot be used for all moment estimations, except to confirm appropriate magnitude, which was done where possible. Therefore, the RAM model analysis was confirmed for output of moments and shear by modeling a beam line from the mechanical roof in STAAD. General assumptions made on the gravity model are as follows:

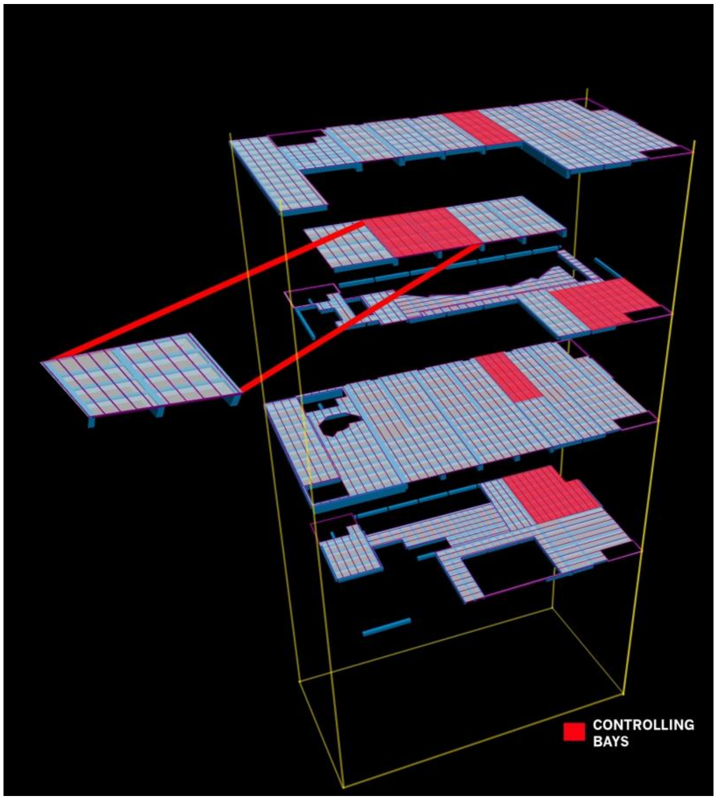


Figure 20: Primary spaces highlighted, mechanical roof bay as example.

This cast-in-place concrete system is monolithic. Therefore, connections between slabs, beams, and columns are fixed.

Deep beam issues, torsion, and shear are all taken into consideration.

For the purpose of this report, the bay chosen from the mechanical floor will be highlighted for explanation of the design procedure and confirmation of hand calculations and the RAM Structural System model. This can be seen in Figure 20. As the mechanical roof has a typical loading and bay width, with larger bay depths, it is a reasonable bay to use throughout the report. Full calculations and results for each of the controlling bays and structural components can be found in Appendix D.

2.1.1 Beams & Slabs

The highlighted bays still control for loading and beams, and therefore the beams adjoining each of these bays have been designed. The mechanical roof beams and girders are highlighted in red, with the slab highlighted in blue in Figure 21.

The diaphragm system was designed and chosen considering the results of a system comparison. Due to the results of past system comparisons (see Technical Report II), and the existence of large spans, the systems considered at greater depth in this report are a reinforced concrete one-way slab system and a prestressed one-way slab system. Because of the direct comparison desired, the column lines were not reconfigured to match each potential system, but reconfigured to be good for both. In reality, once the diaphragm system was chosen, the column lines would possibly be further adjusted. Each of the systems are elaborated on below and then compared for final design results. The prestrssed system lends towards completion of the MAE component of this thesis, as explained further.

The same area on the mechanical roof will be discussed for the most accurate comparison of the two systems. Important considerations for each of these designs included deflection, system depth, kept at or below the existing system depth, and cost considerations. These are evaluated and compared below.

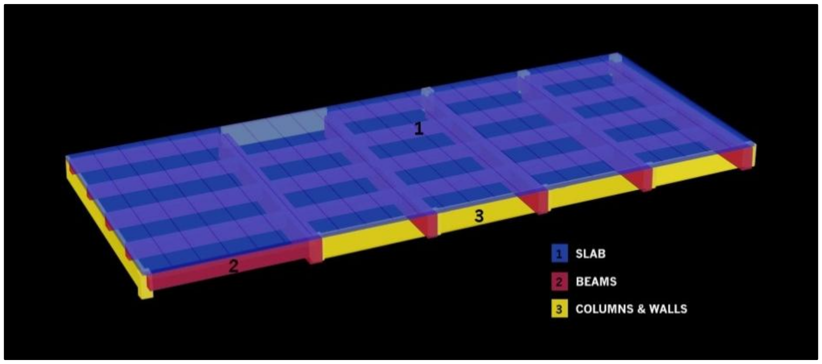


Figure 21: Mechanical roof

Reinforced Concrete Slab and Beam Design

The use of a one-way reinforced concrete system dictates the use of beams and girders within each bay for the layout seen throughout the SSPAC. Therefore, this design starts with a slab depth, used at the minimum for serviceability via ACI 318-11 §9.5.2 Table 9.5(a), which also considers deflection limits. For the mechanical roof, this was taken at 8 inches. Concrete strength for this system was taken at f'c = 4000 psi.

Table 12: Gravity redesign results

Gravity Design Results: Mechanical Roof			
Member	**Dimensions**	**Location**	**Reinforcement**
Slab	8"	Top/Bottom	#4s @ 8"
		Transverse	#4s @ 12"
Exterior Beam	26"x24"	Left Support	(4) #7s
		Midspan	(5) #6s
		Right Support	(4) #7s
Interior Beam	26"x24"	Left Support	(3) #9s
		Midspan	(3) #9s
		Right Support	(7) #10s
Girder	24"X54"	Left Support	(7) #10s
		Midspan	2 layers (8) #10s
		Right Support	(7) #10s

From here, the layout of the beams was decided, and modeled appropriately in RAM, being supported along column lines by girders. Due to the use of large trusses in the existing steel system, these girders require heavy designs to limit deflections and also maintain strength. As mentioned above, RAM was run through multiple iterations to design the most efficient system per bay.

To verify the integrity of the RAM output, hand calculations were performed simultaneously for the first set of beams and girders. These confirmations included that of loaded moments, designed sizes and reinforcement, and deflections. Due to the bay layouts, moments cannot be evaluated by the use of ACI Chapter 8 moment coefficient. Therefore, typical members were modeled in STAAD to confirm output values from the RAM Gravity Model. These programs resulted in output moments and forces with less than 5% variation. Hand calculations confirmed the accuracy of the RAM model for extrapolation to the design of the rest of the bays, as these design results saw a difference of results under 5%, which is considered adequate. These hand calculations and associated spreadsheets can be found in Appendix D.

This design resulted in 24" wide girders for the larger bays with a depth around 4' 6". This is comparable to the existing steel system, in that the depth of these girders is less than that of the trusses that are being replaced. A summary of the results for the mechanical roof can be seen in Table 12. The highlighted bay can be seen with typical dimensions in Figure 22.

Throughout the building, designs of the slabs and beams as a reinforced concrete system resulted in a system that is competitive with the steel framing system and the use of prestressed concrete system. As it is a cast-in-place concrete system, this also requires more on-site preparation during the construction process. Though this system does not eliminate many beams in some areas, it is an effective system for the larger spans, including both strength and serviceability considerations.

Yet, even with these concerns, benefits to this system are in primary locations of the structure. In using a

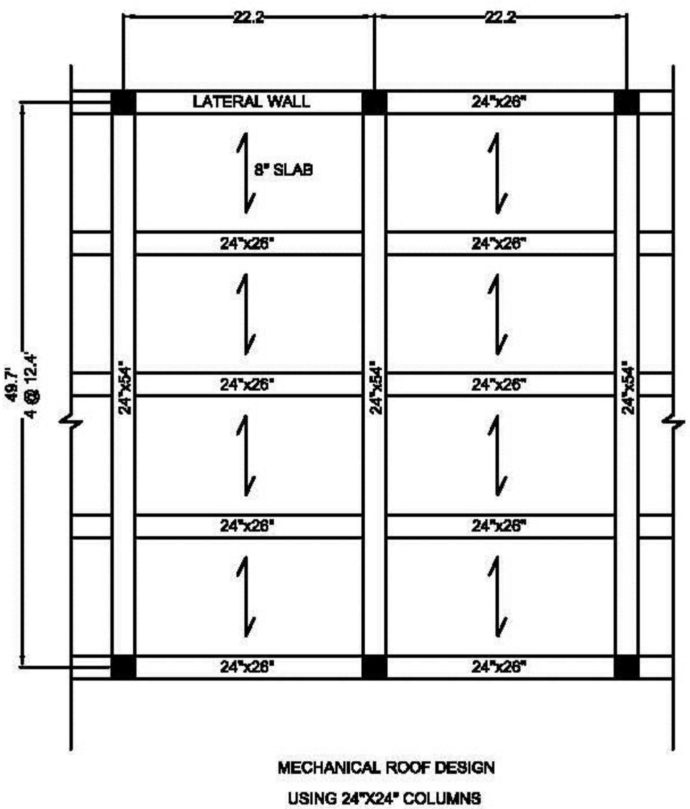


Figure 22: Reinforced concrete ne-way slab and beam system results

concrete system, a cantilever along the edge of the fourth floor mezzanine can be utilized. Compared to steel, which required the addition of beams along the edge of the slab, this is an added benefit, as edge beams can be eliminated and it would result in a visibly cleaner and more open look.

Prestressed Design for Girders & Slabs

The alternative to reinforced concrete for the slab and beam system is to prestress the slab and girders, primarily for the larger bays seen throughout the floors. As deflections and members sizes were large, this alternative is seen as a plausible possibility to a reinforced concrete system.

First, member sizes were estimated by rule of thumb based on the moments. This gave an accurate estimate to the sizing that would be input into ADAPT-PT, the program being utilized to assist in designing the prestressing of each of these structural components. Using rule-of-thumb, the results from ADAPT-PT were confirmed as accurate. And include some of the following assumptions:

Concrete strength was f'c= 5000 psi.

½" Ø 270 ksi strength prestressing strand are utilized throughout. With this, the PT force was kept under a rule-of-thumb maximum of 600 ksi.

Issues such as creep were considered throughout the design process.

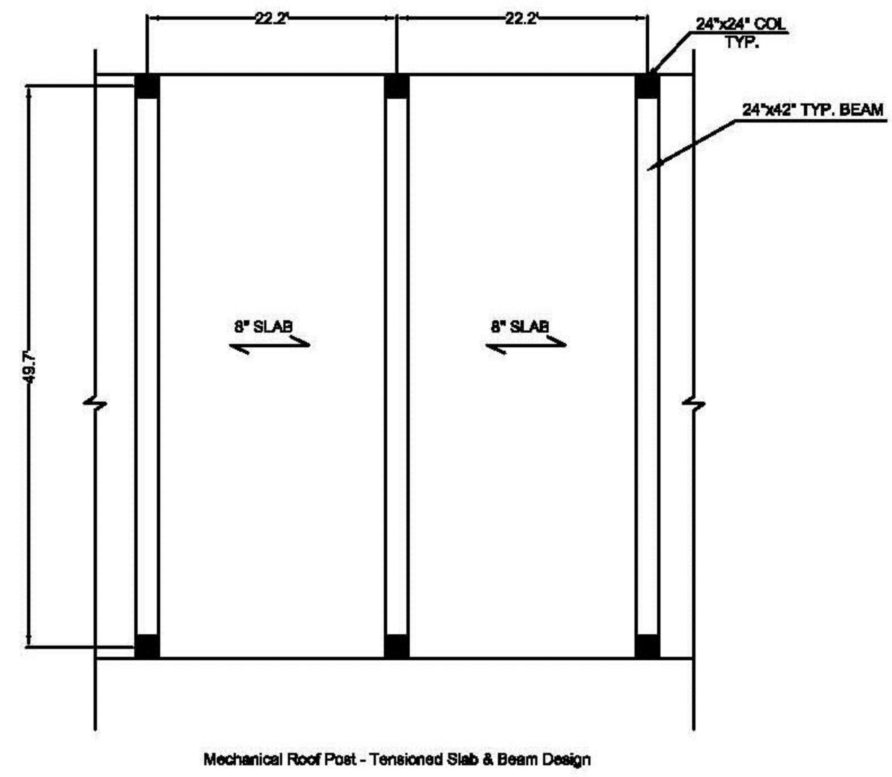


Figure 23: Prestressed one-way slab & beam system

Design resulted in one-way slabs spanning between column lines, on average at 22.2' spans, with 8" slabs. The girders were designed as 3'6" due to serviceability limitation for the approximately 50 foot spans. The tendon profiles for both slab and girder design are included in Figure 25 and Figure 24, respectively, below. Table 13 gives a summary of both the prestressing and the mild reinforcement necessary in this design.

Benefits to this design include both serviceability and strength criteria. Within the bays, elimination of interior bay beams minimizes concrete and therefore even more floor-to-floor height than the above reinforced concrete system. Most of the building has an exposed structural system; this will aid in a much cleaner aesthetic, and will open up space for mechanical, electrical, and plumbing in these larger spaces.

Where two adjoining large spans are prestressed, moments will also be minimized. Though this does not include all of the bays, it does include much of the third floor and some of the mechanical roof, and is an added benefit to each of these floor systems. Because of this benefit, other spaces not originally considered for larger spans, would benefit from a two-bay prestressed system, so columns could be eliminated, especially where the column already does not follow through to the slab-on-grade.

It is noted that this system will likely be more expensive than the reinforced concrete system. This cost comparison, as well as a more thorough comparison of the two systems, can be seen below.

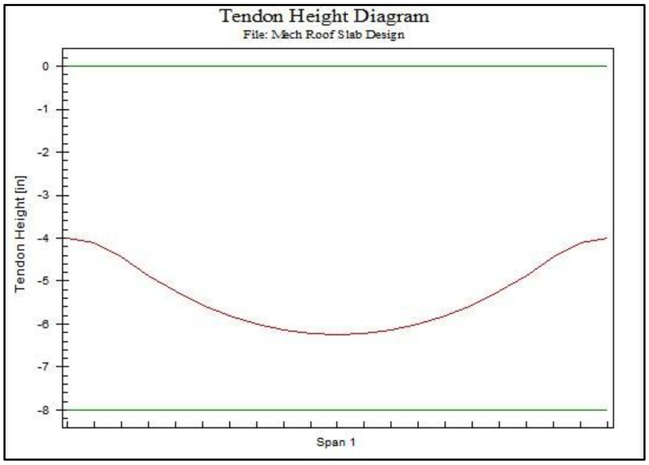


Figure 24: Tendon profile for prestressed slab

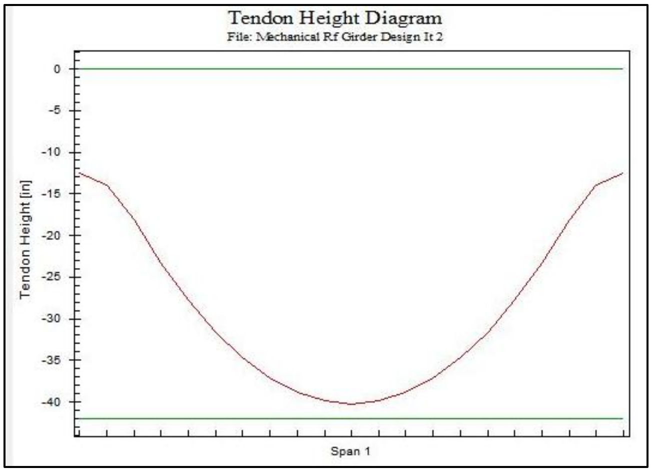


Figure 25: Tendon profile for prestressed beam

Table 13: Prestressed gravity design results

Prestrssed Gravity Design Results				
Member	**Dimensions**	**Location**	**Reinforcement**	**1/2" Ø Strands**
Slab	8"	Top Upper	#7s @ 9"	@ 10" o.c.
		Top Lower		
		Bottom Upper	#4s @ 12"	
		Bottom Lower		
Beam	42"x24"	Top Upper	(9) #7s x 40'0"	(15)
		Top Lower	---	
		Bottom Upper	(2) #7s x 22'0"	
		Bottom Lower	(3) #7s x 50'0"	

Comparison of Gravity Systems

These two slab and beam systems are more exhaustively compared through a cost analysis and through other design considerations. A side by side comparison of these systems is seen in Table 14.

The reinforced concrete system is the more viable system in terms of cost and construction. The prestressed system is a more challenging system in terms of cost and time, and would require additional on-site equipment. On the other hand, the prestressed system is an entire foot shorter per floor than the reinforced concrete system, and therefore allows for a higher architectural ceiling or additional material

Table 14: System comparison

Design Considerations		Existing Composite Steel System	Reinforced Concrete One-Way Slab and Beam	Prestressed One-Way Slab and Beam
Construction	Depth of Slab (in)	5	8	8
	Depth of System (ft)	7	4.6	3.6
	Cost ($/SF)	17.93	17.96	19.64
	Fire Rating (hr)	1	1	1
	Fire Protection	Spray Fireproofing	None	None
	Schedule	N/A	Curing & formwork time required	Slightly more lead time; more coordination required
	Constructability	Moderate	Easy	Moderate - Difficult
Structural	Foundation	N/A	Approx same weight, no change in foundation considerations	
	Seismic Increase	N/A	Negligible Difference	
	Lateral	N/A	Negligible Difference	
Architectural	Impact	N/A	Floor-to-floor height better	Floor-to-floor height better, elimination of some columns possible
Serviceability	Deflection (in)	0.77	0.60	0.32
	Vibration Control	Satisfactory	Satisfactory	Satisfactory

cost savings. Columns can also be eliminated while minimizing some of the moment seen in the building due to these larger spans.

In light of these considerations as summarized above, the final design results in the use of prestressed slabs and beams where appropriate for the larger spans and bays, and for smaller configurations, one-way slab and beam systems were utilized. As the SSPAC was designed with architecture being a main focus, the aesthetic benefits of a prestressed system over a one-way slab and beam system are a control in this structural decision.

2.1.2 Columns & Gravity Walls

Both columns and gravity walls were designed for the concrete structural system. Where walls already existed, gravity or lateral walls were designed (see Lateral Redesign section of this chapter) as appropriate. Both axial and flexural loadings were considered when designing the members and their appropriate reinforcement. Loads were evaluated based upon load transfer from the chosen gravity system above, with hand calculations confirming RAM output loads.

Columns

Typical columns, supporting the controlling bays, were chosen for hand calculation and design. As can be seen in Figure 18, a series of columns and walls support the gravity system. Members being focused on in this design overview are highlighted in red in Figure 26.

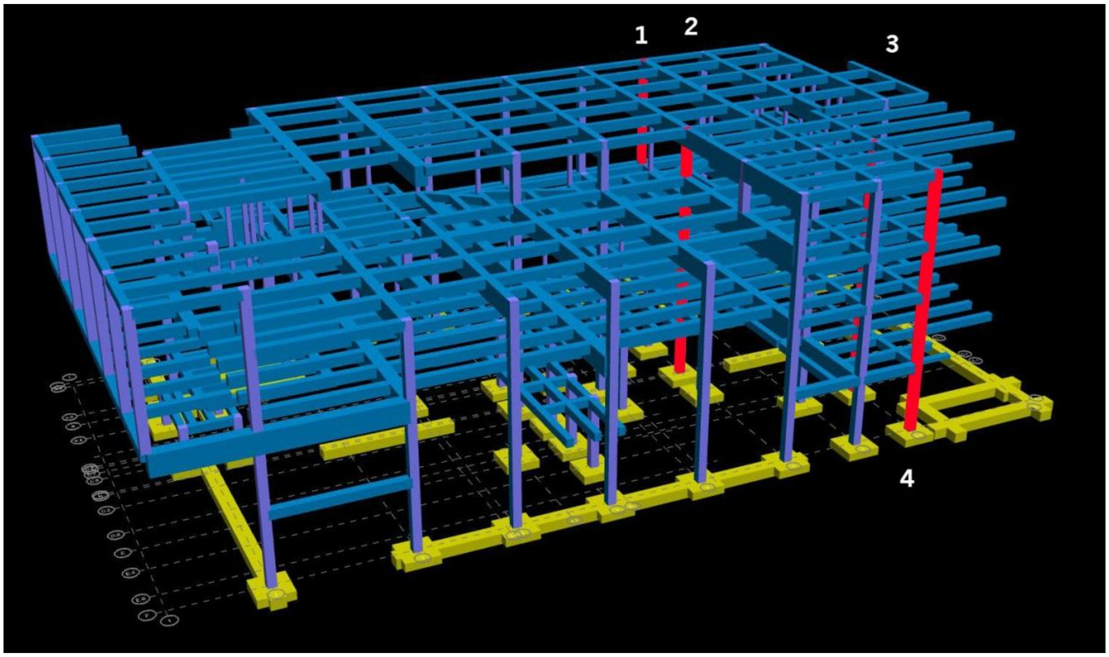


Figure 26: Highlighted columns designed by hand, walls eliminated

As above for the beam and slab system, primary columns were designed by hand calculations to confirm input and output from the computer model used, here StructurePoint Column, before continuing design. These hand results confirmed that, due to flexural loadings and the long unsupported length of many of the columns, some columns would require additional reinforcement and considerations.

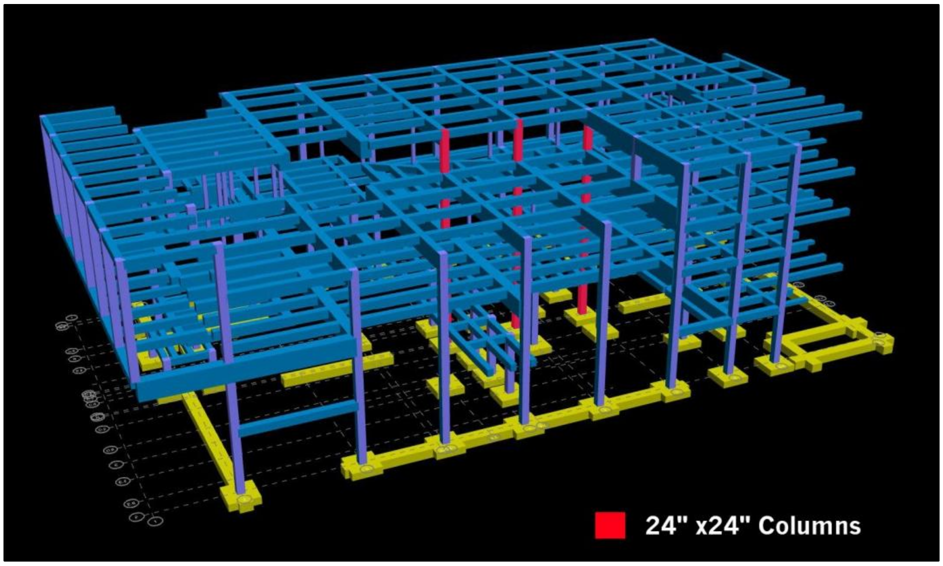


Figure 27: Resulting 24"x24" columns, walls eliminated

In all, columns were typically designed as 20"x20" members, with some variation in reinforcement based upon the loading. Columns along the central column line, as seen highlighted in Figure 27, were more heavily loaded, and design required these to be designed as 24x24" columns with f'c = 6000 psi. As these supports run along the center of the building and support larger spans on both sides, this is a logical outcome and does not impact the layout of the building. Results for the primary columns considered in this explanation are highlighted in Table 15.

Table 15: Gravity redesign results

Gravity Design Results: Mechanical Roof				
Member	**f'c (psi)**	**Dimensions**	**Location**	**Reinforcement**
F8.8	4000	20"X20"	Longitudinal	(6) #9s
(exterior)			Transverse	#4s @ 12"
E8.8	4000	20"X20"	Longitudinal	(6) #9s
(interior)			Transverse	#4s @ 12"
A-8	4000	20"X20"	Longitudinal	(6) #9s
(exterior)			Transverse	#4s @ 12"
C-7	6000	24"X24"	Longitudinal	(16) #9s
(interior)			Transverse	#4s @ 12"

One primary benefit to a concrete system over a steel system for columns is that the concrete columns have a higher stiffness and therefore resist more against flexure and therefore p-delta effects. It is noted however, that steel members are narrower and have less impact on the architecture.

Walls

Gravity walls were designed after the lateral system was decided on and required shear walls locations were confirmed, as discussed further in Lateral Redesign. The existing building includes gravity infill walls along the interior and exterior of the building, surrounding the cinemas and vertical circulation shafts. These walls were redesigned in concrete to maintain the same thicknesses as the original structure. A typical gravity wall result is seen below in Figure 28.

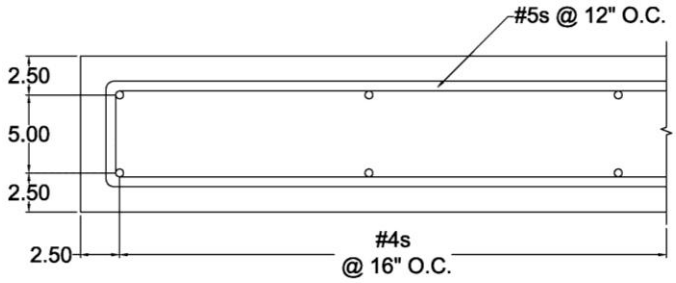


Figure 28: Typical gravity wall design

2.1.3 Foundations

Once the building had been designed, foundations were reevaluated for the new loadings. Due to a concrete system, the loadings on the foundations were slightly higher. A combination of axial and flexural load was evaluated for this heavier loading. In addition, uplift was considered where applicable. Some locations, with the heavy loading, eliminated uplift issues. For this, the foundations at the bases of the columns that were previously designed were designed. These design calculations can be found in Appendix D.

Overall, it was found that square foundations could be designed for a similar dimension, with additional reinforcement. Added benefits to this are in terms of the existing location, where existing site conditions are important and needed to be considered. Strip footings, under the walls, did not require additional dimensions, and were deemed adequate, with depth based on necessary site conditions. A summary of typical foundation results can be found in Table 16.

Table 16: Foundation design results, by hand

Foundation Design					
Type	Location	f'c (psi)	Dimensions	Reinforcement	Depth
Square	F8.8	4000	8' x 8'	(8) #6 ea. way	1'2"
Strip	Line F	4000	4.5' width	(7) #5s	1'4"

2.1.4 Design Summary

In conclusion, replacing the existing steel system with a concrete system has many benefits to the current building function needs. Strength requirements are met with a system that, though heavier, allows for flexibility in design of various cantilevers in the building. Also, due to the inconsistent layout of these spaces, concrete is a competitive option that has configuration flexibility that results in an effective design.

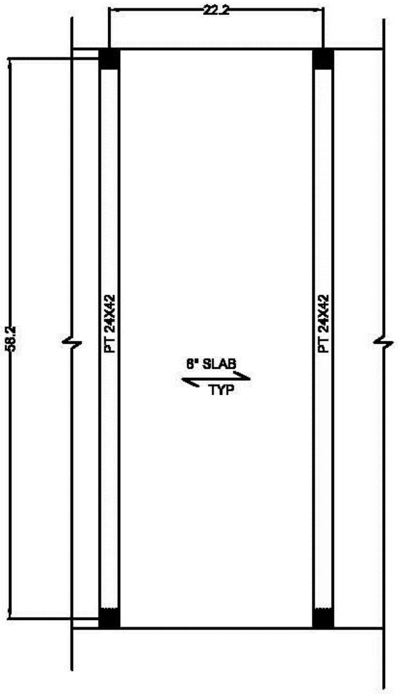
As has been developed above, a bay from the mechanical roof was detailed and designed, then explained thoroughly. The existing steel frame system included deep trusses to support these large spans. In place of these, large prestressed members were designed to a smaller depth. These met both strength and serviceability criteria. By using prestressed slabs and beams, many of the beams within bays were eliminated, freeing up space for mechanical, electrical, and plumbing, and giving the space a cleaner aesthetic. With the number of large spans over 45 feet summing up to more than 25 spans, the architectural requirements, and the forces seen on the system, the prestressed system was concluded as the better option.

Columns and gravity walls were designed effectively, with results for columns usually seen at 20”x20”, with the central column line being designed as 24”x24”. Gravity walls were designed similar to the existing in size and location, with the exception of now being cast-in-place instead of precast.

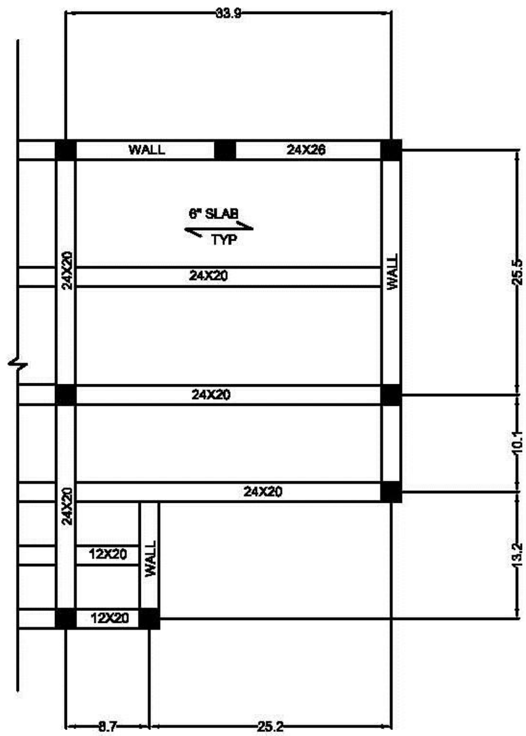
In relation to serviceability, 6 feet of floor-to-floor height is also saved throughout the total height of the structure. This floor-to-floor height already considers the layout of the mechanical, electrical, and plumbing systems, which reach the building spaces through a primary shaft in the middle of the building and is not largely impacted by the redesign. It is suggested that this additional space saved be eliminated from the building to minimize costs, but could also be utilized to create a higher architectural ceiling.

Even though this was a successful redesign, certain areas of the building lend more towards the existing steel system, and this was seen throughout the redesign. This includes the sets of hanging columns on the second and fourth floors, which would have to be adjusted into cantilevers, standard columns, or larger transfer girders. Another example of steel being a better system is in the spiral staircase, located between the second and fourth floors, and is a much better system in steel that would need to be changed to be efficiently designed in concrete.

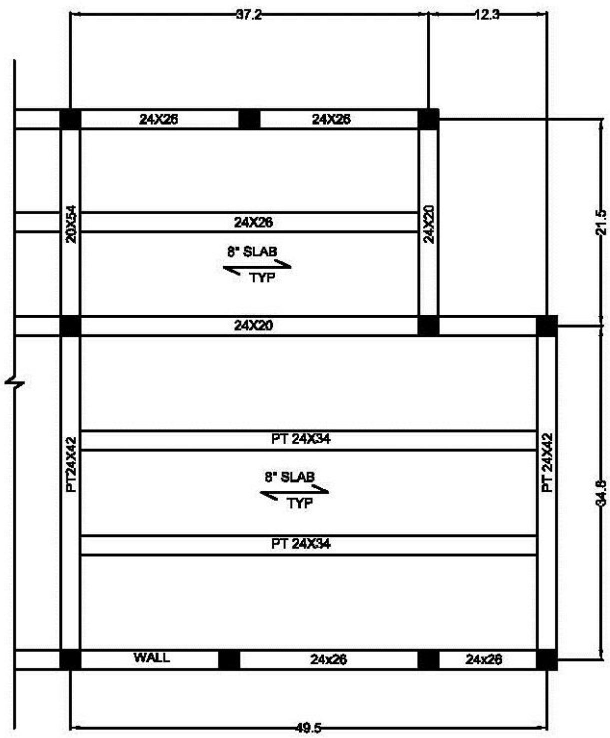
The final designs for each of the bays chosen, as highlighted previously are given in the images following. It should be noted that the bays taken from the roof and third floor utilize the prestressed system, the fourth floor bay using a typical reinforced one-way system, and the bay chosen from the second floor is an integrated approach. Overall, each of these varies uniquely from the others, but all show the integration of prestressed bays into the typical concrete system.



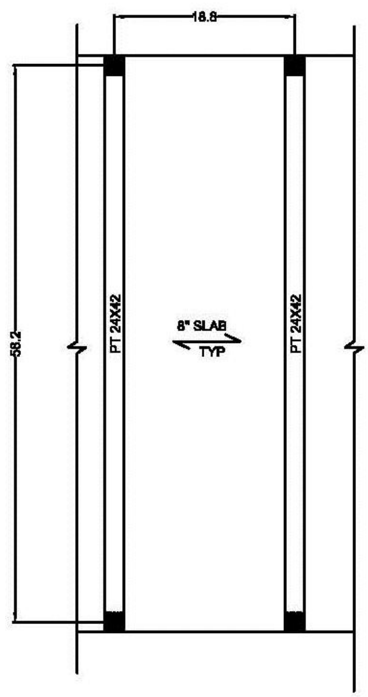
ROOF - PRESTRESSED



4TH - ONE-WAY SLAB AND BEAM



2ND - PRESTRESSED



3RD - PRESTRESSED

2.2 Lateral Redesign

Due to the gravity system being redesigned as a concrete system, the lateral system will also be changed. Braced frames, already existing in interior wall locations, were replaced by concrete shear walls. Shear walls are also slightly reconfigured to minimize torsion that contributed largely to lateral load in the existing building. Iterations were completed to find the best layout, as can be seen in Figure 29. While formerly shear walls were precast, they are being reevaluated as part of the cast-in-place system. For evaluation of the lateral loads, a RAM model was built, loads applied, and then shear walls were designed based on the resisting forces found in each wall. Supporting calculations can be found in Appendix E and Appendix F.

2.2.1 RAM Input & Confirmations

The lateral system was essentially left in the same layout as previously, and this new system can be seen in Figure 29, which highlights the shear wall configuration in orange. The lateral system was created in RAM, allowing for a complete frame analysis of the model. The input for this RAM model is similar to that of the existing structure, but with a more limited shear wall system, higher torsional issues are considered.

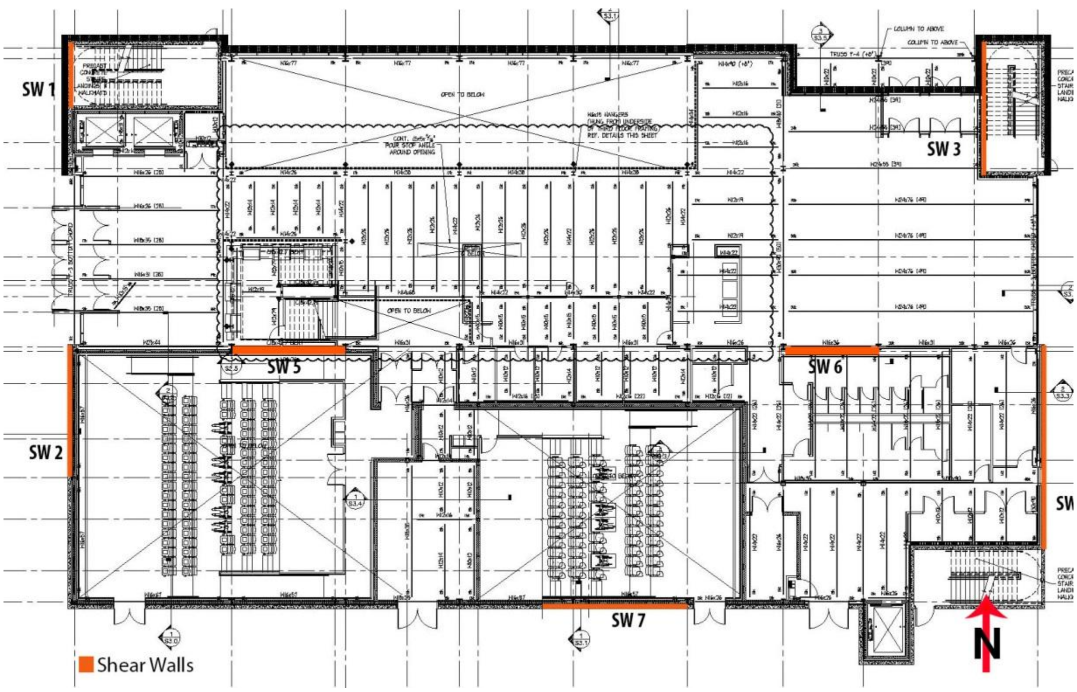


Figure 29: Lateral walls, highlighted

The RAM model incorporated into the analysis of the lateral system 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. Some of the primary assumptions are as follows:

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

All cast-in-place concrete was treated as a monolithic system, and assumptions for fixed joint connectivity followed ACI Code allowance. Connections at foundations were also assumed to be fixed.

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.

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

Torsion

Vertical structural irregularities were considered for the SSPAC, applicable ones from ASCE 7-05 Table 12.3-2 including in-plane discontinuities and weak story irregularities. Neither is an issue in the SSPAC, and both have been confirmed to not exist. Therefore, vertical structural irregularities do not apply to the structure.

After confirmation of the RAM model's accuracy through building property output and hand calculations, seen in Appendix E and Appendix F, 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.

Horizontal irregularities, as they existed in the existing model, were also an important factor in the lateral design. Both torsional irregularity and extreme torsional irregularity, as defined by ASCE 7-05 Table 12.3-1, needed to be considered for the SSPAC. Therefore, 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 Figure 12.8-1 and is more thoroughly explained in Technical Report III.

As was found in the existing structure, the X-Direction had no torsional irregularity. In the Y-Direction, 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. 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. Yet, compared to the existing system, some torsion was eliminated in the Y-direction; instead of creating an extreme torsional irregularity, only torsional irregularity now exists as an added benefit to the reconfigured shear walls. A summary of these results can be seen in Table 17, with detailed hand calculations found in Appendix E.

Table 17: Torsional amplification factors applied

	Y-Direction Accidental Torsion				Resulting Moment and Bx' (ft-k)	
	Bx	5% Bx	Ay	Mzy	Mzy'	Bx'
Roof	190	9.5	1.3	2208.8	2800.8	294.8
Mech Roof	190	9.5	1.0	1270.2	1270.2	133.7
4th	190	9.5	1.3	1352.8	1711.9	180.2
3rd	190	9.5	1.2	2883.3	3498.8	368.3
2nd	190	9.5	1.2	676.7	790.5	83.2
Ground	N/A	N/A	N/A	N/A	N/A	N/A
	Overturning Moment (ft-k)			8391.6	Overturning Moment (ft-k)	10072.1

2.2.2 Applied Building Loads

Once the building was modeled, both wind and seismic loads were calculated. Previously, as can be seen in the explanation in Chapter 1, seismic controlled. It was assumed that seismic would again control, especially as the building is heavier than before. As wind is based on height, these essentially changed very little. Seismic loads, based on the building weight, as can be seen below, were recalculated to be heavier loads than before. Serviceability requirements were checked with each lateral loading, and are elaborated below as well.

Wind

As stated previously, wind load calculations for the structure redesign do not result in much variation from the existing wind loads, as seen in Chapter 1.5. A summary of these loads, applied in both North-South and East-West directions can be seen in Table 18, with calculations for these adjusted values seen in Appendix E.

Table 18: Wind loading results

Wind Loadings						
	Location	Height	North-South		East-West	
			Story Shear (k)	Overturning Moment (k-ft)	Story Shear (k)	Overturning Moment (k-ft)
Windward Wall	Roof	58	21.59	1252	11.41	661.57
	Floor 4	45	63.47	1884	22.02	990.78
	Floor 3	32	110.06	1491	24.36	779.41
	Floor 2	15	159.57	743	25.87	388.11
	Ground	0	182.78	0	12.13	0.00
	Width (ft)	190				
	Total Base Shear (k):		182.78		95.78	
	Total Overturning (k-ft)		4628		2432	

Drift calculations for serviceability needed to be met under wind loads, as per the rule of thumb H/400 found in ASCE 7-5 §C-C. As found previously, the building is a fairly stiff building without drift issues. These results can be found in Table 19.

Table 19: Drift & displacement results, wind loading

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	58	11	0.01633	0.330	YES	0.05922	1.740	YES	W1
	Mech Roof	47	16.5	0.00125	0.495	YES	0.04289	1.410	YES	W1
	4th	45	13	0.01594	0.390	YES	0.04164	1.350	YES	W1
	3rd	32	17	0.01729	0.510	YES	0.0257	0.960	YES	W1
	2nd	15	17.5	0.00841	0.525	YES	0.00841	0.450	YES	W1
				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
Y Direction	Roof	58	11	0.00375	0.165	YES	0.04197	0.870	YES	W2
	Mech Roof	47	16.5	0.00968	0.248	YES	0.03822	0.705	YES	W2
	4th	45	13	0.00829	0.195	YES	0.02854	0.675	YES	W2
	3rd	32	17	0.01368	0.255	YES	0.02025	0.480	YES	W2
	2nd	15	17.5	0.00657	0.263	YES	0.00657	0.225	YES	W2

Seismic

Seismic loads controlled, as was anticipated, since when the weight of a building increases, the seismic loads also increase. For these load calculations, the building weight was recalculated and the RAM model was confirmed to be accurate in load and self-weight calculations. Seismic loads followed the same procedure outlined in Chapter 1.5 of this report, and results from this procedure can be found in Table 20.

Table 20: Seismic load results

Seismic Forces							
Level	Story Weight, w_x (kips)	Story Height, h_x	$w_x h_x^k$	C_{vx}	Story Force (k) $F_x=C_{vx}*V$	Story Shear (k)	Overturning Moment (k-ft)
Roof	3167.9	58	287,198	0.301	245.8	245.8	14254
Mech Roof	1361.13	47	97,708	0.103	83.6	329.4	15481
Floor 4	2025.8	45	138,566	0.145	118.6	447.9	20158
Floor 3	6366.1	32	298,256	0.313	255.2	703.2	22501
Floor 2	6495.0	15	131,232	0.138	112.3	815.5	12232
Ground	N/A	0	N/A	N/A	N/A	N/A	N/A
Cs	0.042			Base Shear [V=Cs*W] (k)			815.5
W(k)	19416			Total Overturning Moment (k-ft)			84626

Serviceability requirements were evaluated for seismic. Under heavier loadings, seismic drift increased from the previous system, yet these drifts are still well under drift allowances as per ASCE7-05. It can be remarked that less walls are being evaluated as shear walls, and therefore, while overall the building is stiffer as a fully concrete system, only the lateral system is considered as resisting load, and only a portion of the current shear walls system are necessary for lateral loads. These drift values are summarized in Table 21.

Since seismic loads control, these are the loads used in this design. Application of these specific loads to the SSPAC are elaborated on in the following few sections.

Table 21: Drift & displacement results, seismic loading

Seismic Drift & Displacement: Amplification Factor, (Cd/I) Factor Considered									
				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
X Direction	Roof	58	11	0.833	1.98	YES	2.849	10.440	YES
	Mech Roof	47	16.5	0.059	2.97	YES	2.016	8.460	YES
	4th	45	13	0.778	2.34	YES	1.957	8.100	YES
	3rd	32	17	0.826	3.06	YES	1.179	5.760	YES
	2nd	15	15	0.353	2.70	YES	0.353	2.700	YES
				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
Y Direction	Roof	58	11	0.098	1.98	YES	1.161	10.440	YES
	Mech Roof	47	16.5	0.286	2.97	YES	1.064	8.460	YES
	4th	45	13	0.244	2.34	YES	0.778	8.100	YES
	3rd	32	17	0.375	3.06	YES	0.534	5.760	YES
	2nd	15	17.5	0.159	3.15	YES	0.159	2.700	YES

2.2.3 Load paths

Because of the complexity and irregularity of the building layout, load paths need to be thoroughly understood and evaluated. Goals of understanding the load path more completely:

- → Look at areas of connection between shear walls and diaphragms throughout the floors
- → Highlight areas of larger concern (e.g., coupling beams, transfer girders)

For an easier explanation, the results of this load path evaluation will be elaborated on by applying the controlling lateral load in the east-west direction, as seen in Figure 30. This figure displays the lateral load, first applied in red, which transfers through each of the floor diaphragms and then to the shear walls.

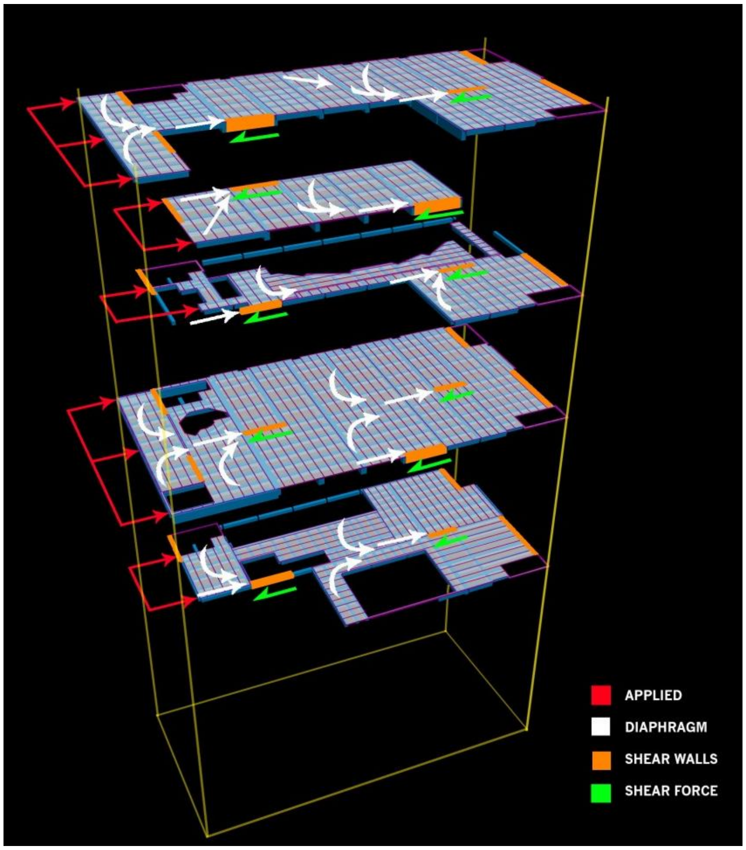


Figure 30: East-west load path

As can be seen in the load path figure, the diaphragms are not simply laid out, but are altered for each floor and a good portion can be considered irregular. These irregularities not only create torsional issues, but issues in transferring lateral loads through the diaphragms and into the shear walls by having good connectivity. By looking at this, locations can be seen where issues may arise out of poor connections.

Areas that cause concern are places where the diaphragm may be weaker, as seen on the fourth mezzanine by the thinner slab at the center, or where the walls carrying large amounts of lateral loads have a small connection point to the diaphragm. Here in the east-west direction, these places are highlighted in Figure 31.

Though diaphragms were considered rigid based on assumptions stated previously, further study would also look at additional stiffeners in some of these areas. Additionally, collectors at ends of shear walls and coupling beams between shear walls would require evaluation and further study. These areas are circled in Figure 31.

Once these areas are detailed, shear walls follow next in the load path, and seen highlighted in orange in Figure 30. Here, a typical shear wall, one that controls in torsion due the building eccentricity, is highlighted in Figure 32 The design of this and other shear walls are elaborated on in the next section.

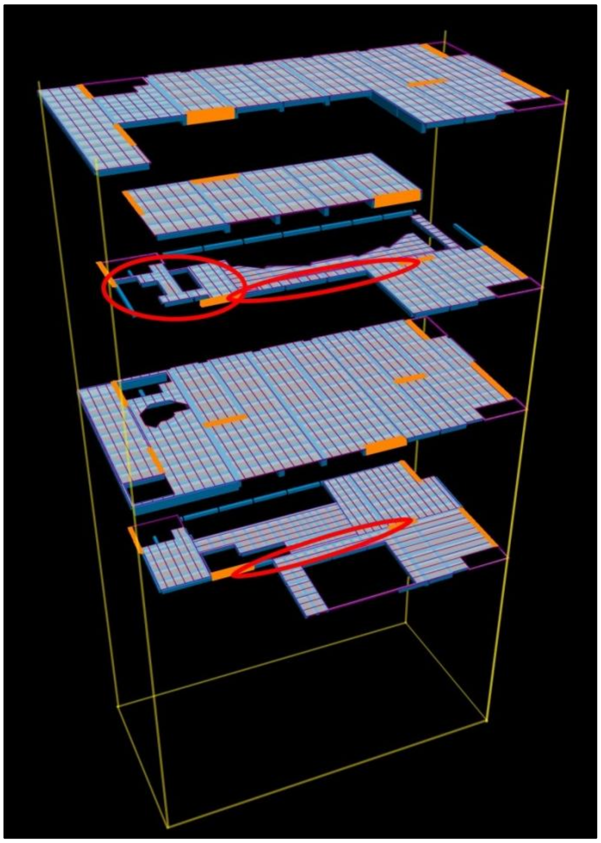


Figure 31: Lateral system details to be considered further

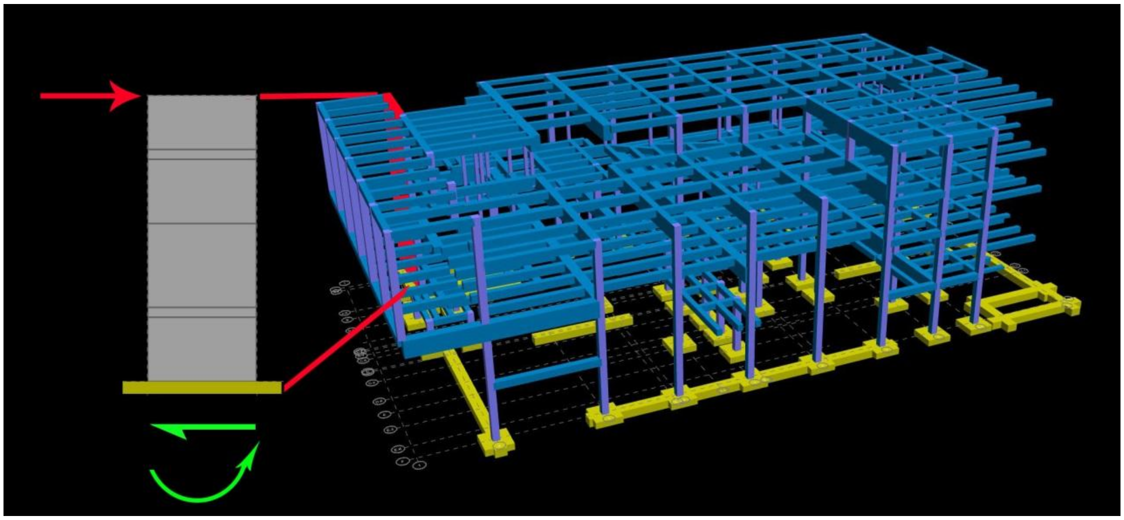


Figure 32: Shear wall highlighted

2.2.4 Design Results

Lateral loads, once taken through the diaphragm, transfer through to the shear walls, which are the primary component of the lateral system. RAM was not simply told to design shear walls, as it is not good to use a computer program without confirming assumptions the program is using. Loadings were confirmed via hand estimates of building loads and reactions. Hand calculations were completed on shear walls for design, with a thorough understanding of potential diaphragm issues related further as well.

Existing shear walls were designed as pre-cast panels. Because of the nature of the redesign, these will be cast-in-place, and were designed to be the same thicknesses as the existing walls. In addition to shear walls, locations along the diaphragm would also need to be detailed for higher stresses and loads. These are commented on previously.

Shear Walls

In designing shear walls, maximum shear, which included consideration of interstory shear, overturning moment, and flexural loadings were accounted for. Shear walls are similar to cantilevered beams, and therefore are designed similarly. The wall being elaborated on here is highlighted in Figure 29 above, designated "SW 1" was chosen for its critical location and higher torsional issues.

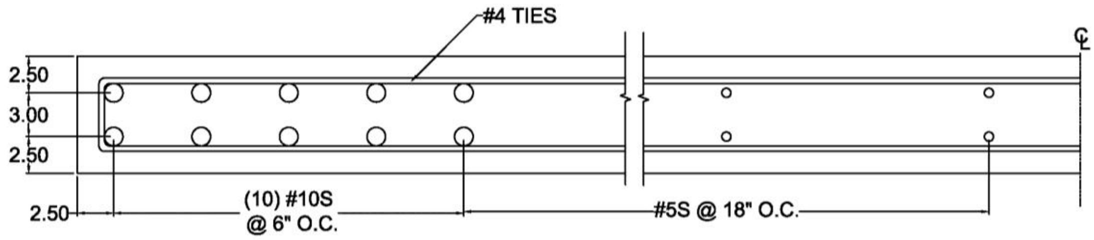


Figure 33: Typical shear wall detail

In summary, the lateral system, comprised of concrete shear walls, was redesigned to be 8" to 10" thick walls, matching thicknesses of the existing system. These shear walls are located at the existing braced frames and existing precast shear walls. Results are in Figure 33. See Appendix F for hand calculations.

Due to the torsional issues as explained more thoroughly above in Section 2.2.3 of this report, lateral loads create strong torsion in certain areas. The shear wall highlighted above is one with a larger torsional force due to its location and interaction with the existing diaphragms.

2.3 Comparison

As part of the focus of this thesis, a comparison of the existing system to the redesigned system is necessary to have a thorough understanding of the redesign. This portion of the comparison focuses on the structural advantages and disadvantages, with the implications this new system makes on other aspects of the building process.

While not every impact of the structural system was analyzed to great depth, each has been considered and evaluated for understanding of how these are impacted. Benefits and disadvantages on each of these systems to the redesign in concrete are discussed below.

Gravity Design

As discussed throughout the gravity redesign, concrete lends towards certain design elements in the building. Some of these include cantilevers with back spans, which are located in various parts of the SSPAC, varied bay configurations, and large spans. The use of a prestressed concrete system is a benefit to the larger spans and moments associated with these.

It is acknowledged that certain elements of the SSPAC are more adapted to a steel building. The existing system has multiple locations where hanging columns exist, to open spaces on the floors below. While various solutions exist in concrete – transfer girders, cantilevers, standard columns – these are not as ideal or adventitious as the steel counterpart. These can be designed, but may require the layout to be altered or result in a more expensive option.

Lateral Design

Benefits to using cast-in-place shear walls as the entire lateral system are seen in the stiffness of the building, as well as the cohesion between the gravity and lateral systems. There is less redundancy in this system, as compared to the existing design. This helps minimize cost, but in design against building failure, more redundancy is more conservative. The existing steel system has redundancy between the lateral and gravity systems, and though this is a good design, it is unnecessary for a concrete system.

Torsion, an issue in the existing system, was reevaluated for a slightly different shear wall configuration, and some of the extreme torsion inherent in the building was eliminated. Removal of torsion aided in making the redesign more efficient and therefore less expensive.

Cost

Two brief cost analyses were produced, as discussed above in this chapter, as well as following in the related breadth sections. These cost comparisons showed that the chosen designs of normal reinforced concrete and prestressed one-way concrete systems, at $17.96/ft^2 and $19.64/ ft^2 respectively, though more expensive, were still competitive with the existing system, as $17.93/ ft^2. The cost, per square foot, of the existing system, is estimated in Appendix D, with the redesigns found in Table 14.

Construction

Through the consideration of a concrete system, there is less lead time required, as opposed to steel. Concrete though, especially with the inclusion of prestressed bays, would take more construction time and require more site congestion.

Other

There are other factors that control in this system, and are integrated into the overall performance of this building. Two of these, acoustics and architecture, are further discussed below in Chapters 2 and 3, and are tied tightly into the structural design.

It was found that the use of a concrete system could be developed with minimal changes to the layout, and the floor to floor height minimized. Because building weight was also an important factor for minimizing seismic loads which control laterally, this height change is another benefit to this system, which overall, is a heavier system than a steel system.

2.4 MAE Coursework Integration

Coursework requirements for the MAE were integrated well into this thesis, through knowledge gained in AE 530, *Computer Modeling of Building Structures*, and CE 543, *Prestressed Concrete Behavior and Design*.

Both the gravity system and lateral system were designed through the benefits of utilizing RAM Structural System, a structural analysis and design program that aids a competent engineer through a building design. As learned in AE 530, this program cannot be treated simply as a black box of input and output, but as a tool. While this speeded up the design process, hand calculations were also utilized to confirm that the software was being used appropriately and that code was being met through the input assumptions. An explanation of how this was incorporated can be seen earlier in Chapter 2.1-2.2.

The gravity system included an in-depth study of two different diaphragm systems, incorporating information obtained from CE 543.The slab and beam systems were not only designed as one-way slab and beam simply reinforced systems, but also as a prestressed system. Due to the large spans throughout the building, this option has many benefits and therefore is a competitive option. In summary, this design stems from knowledge obtained in CE 543, looking at the benefits and issues the use of prestressed concrete brings to an entire building system. This design process is explained in Chapter 2.1.1 more thoroughly.

Chapter 3: Breadth I: Acoustics

As the SSPAC is a building full of various important acoustic spaces, a change in the structure and building materials will directly impact the success of each of these spaces. By eliminating steel and incorporating concrete, the new wall and floor systems will directly impact the effectiveness of sound isolation and room acoustics incorporated into the design. Therefore, the Reverberation Time (RT) of these spaces, and the Sound Transmission Criteria (STC) between these spaces and adjacent rooms will be evaluated. The RT is the amount of sound decay within a space due to the surface material reflectivity, and will be evaluated for the spaces most impacted by acoustics. With important spaces being adjacent to others, the Sound Transmission Criteria (STC), the sound transmitted between spaces, also needs to be evaluated, for confirmation that the space divisions meet requirements for the spaces. Each of these two components of the SSPAC acoustics is further discussed below. Supporting information and calculations can be found in Appendix G.

3.1 Reverberation Time

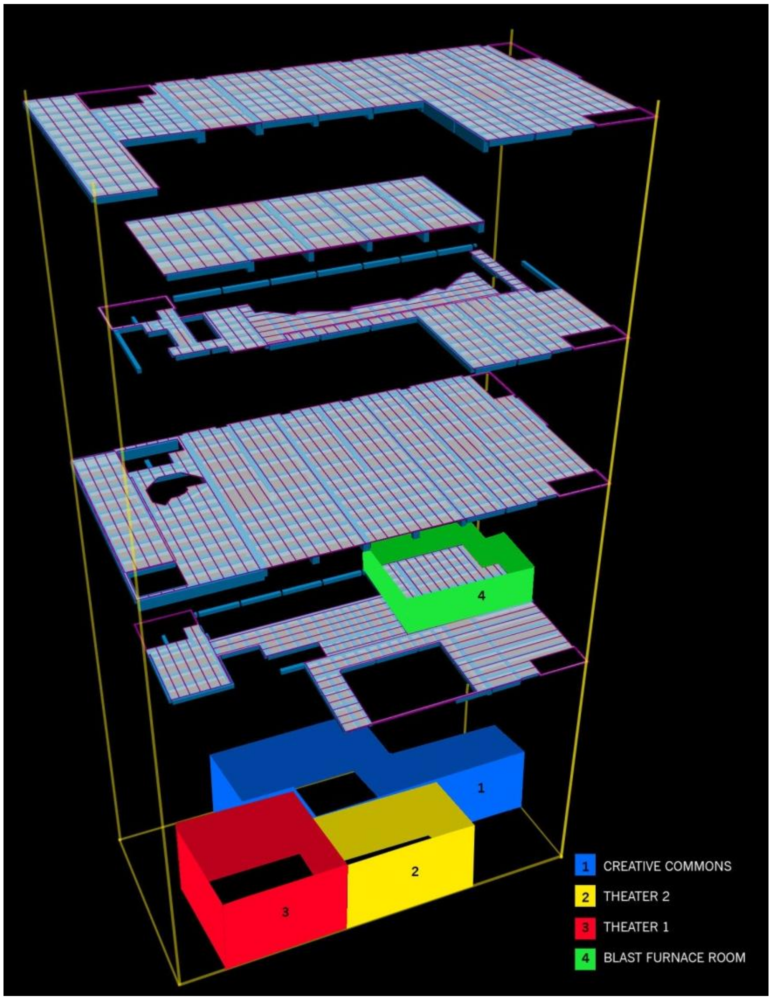


Figure 34: Spaces considered in acoustic analysis highlighted

As stated above, Reverberation Time (RT) evaluates the sound decay within a space. For a space to be considered appropriately reverberant, acceptable conditions for each space were chosen based on the bar graph shown in Appendix G. Rooms where surface materials changed from the existing design were reevaluated for performance and the room criteria set in the initial design phase. The spaces considered for this evaluation include the Blast Furnace Room, both Theaters, and the Creative Commons, as highlighted in Figure 34.

The reverberation time for each existing space was evaluated based on the current room materials and compared to the redesign with the new structural system and without any further acoustical attenuation. Comparing this analysis with the design targets, further acoustical considerations were made to ensure the redesign met the RT set points for each space. This is further discussed below.

Blast Furnace Room

The goal for this multipurpose room on the second floor in terms of the RT is in the range of 0.7-1.0 seconds, which allows for primarily speech, with potential for music use. Originally, this was a fairly dead space, and therefore, a slightly more 'live' room is allowable, as it allows for more variation in the space use. Table 22displays the RT results for both the existing and the redesigned space.

Table 22: Reverberation times for the Blast Furnace Room

Comparison of Reverberation Times: Blast Furnace Room							
Frequency (Hz)		125	250	500	1000	2000	4000
Existing							
Calculated Reverberation Time (s)	RT =	0.78	0.60	0.51	0.55	0.50	0.52
Mid range average Reverberation Time				0.52			
Redesign							
Calculated Reverberation Time (s)	RT =	0.77	0.61	0.57	0.63	0.58	0.59
Mid range average Reverberation Time				0.59			

By eliminating the acoustic panels where painted concrete will now be located, this design will be more cost effective, as fewer panels will be required. Also, with the improvement in the space acoustics, the Blast Furnace Room is now a higher quality space for a reduced material cost. Yet, this does not fit within the desired RT range, even though it is a more successful design. It is suggested to continue investigation of space materials to obtain a higher RT value for the space.

Theater 1 & Theater 2

The RT range for each of these theaters on the first floor is in the range of 1.0-1.2 seconds, which creates a more live or reverberant space, and is ideal for spaces like theaters and music halls. The goal for this redesign was to evaluate the impact of an unpainted concrete, as opposed to a painted concrete, as well as changes in the square foot of carpet. Table 23 and Table 24 display the RT results for both the existing and the redesigned space.

Table 23: Reverberation times for Theater 1

Comparison of Reverberation Times: Theater 1							
Frequency (Hz)		125	250	500	1000	2000	4000
Existing							
Calculated Reverberation Time (s)	RT =	1.00	0.40	0.34	0.35	0.32	0.33
Mid range average Reverberation Time				0.34			
Redesign							
Calculated Reverberation Time (s)	RT =	0.85	0.34	0.31	0.32	0.28	0.31
Mid range average Reverberation Time				0.30			

Table 24: Reverberation times for Theater 2

Comparison of Reverberation Times: Theater 2							
Frequency (Hz)		125	250	500	1000	2000	4000
Existing							
Calculated Reverberation Time (s)	RT =	1.08	0.41	0.35	0.36	0.33	0.33
Mid range average Reverberation Time				0.34			
Redesign							
Calculated Reverberation Time (s)	RT =	0.91	0.34	0.31	0.32	0.28	0.31
Mid range average Reverberation Time				0.31			

These results and the iterations behind them prove that both of these theater spaces designed to be very dead spaces. Generally, a cinema relies on the RT of the space for sound performance. This space, upon further investigation, relies on the sound system for acoustic performance, so a dead space is desired. The new system consequently, is adequate. Therefore, the theaters should not be redesigned for acoustic materials, and impact from the concrete redesign is negligible.

Creative Commons

This common space and atrium area has a goal RT of 0.8 – 1.0 seconds, which allows for a controlled reverberation to ensure clarify in sound patterns. With adjoining spaces, such as the Blast Furnace Room, lobby, and quieter seating area, a slightly deader space is allowable. Table 25 displays the RT results for both the existing and the redesigned space.

Table 25: Reverberation times for the Creative Commons

Comparison of Reverberation Times: Creative Commons							
Frequency (Hz)		125	250	500	1000	2000	4000
Existing							
Calculated Reverberation Time (s)	RT =	1.01	1.69	2.19	2.30	2.29	1.86
Mid range average Reverberation Time				2.26			
Redesign							
Calculated Reverberation Time (s)	RT =	0.60	0.60	0.91	1.01	0.74	1.04
Mid range average Reverberation Time				0.89			

By replacing the existing steel system with concrete, this space results in a higher mid-frequency RT value, of 0.89 seconds as compared to 0.56 seconds, which is more appropriately within the range of desired RT values for this space, than high above this range. Therefore, this space is a successful redesign and does not require additional acoustic material design.

3.2 Sound Transmission Criteria

A higher Sound Transmission Criteria (STC) value, a rating that approximates the Transmission Loss of a material, results in a more efficient design that reduces sound transmission. With floor systems being altered, primary spaces were evaluated for the acoustic development of the floor system. Target STC values were chosen based on acoustic requirements of each space based upon that space's requirements and the type of adjacent space.

Looking at the interaction between different spaces, it is noted that the Creative Lobby/Blast Furnace Room on the second floor and the Musikfest Café on the third floor have a floor system acoustically designed between them as a sound barrier. Therefore this flooring, as it will change, will impact the sound transmission between these two spaces.

The existing flooring consists of two different systems. Above the Creative Commons is an 8" concrete slab and metal decking system. Above the Blast Furnace Room, a more sound isolated space, the 8" of concrete and metal decking also includes acoustical ceiling tile. The new system, proposed as a 6" slab, and a 6" slab with acoustical ceiling tile, respectively, is being evaluated for effectiveness by use of the STC values.

A plan of the third and second floor overlaid is seen in Figure 35. A cross section of these two floor systems and their corresponding STC requirements can be seen in Figure 36.

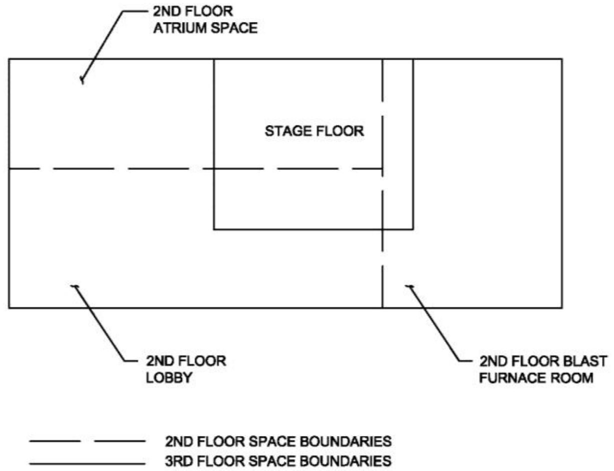


Figure 35: Second & third floor overlay

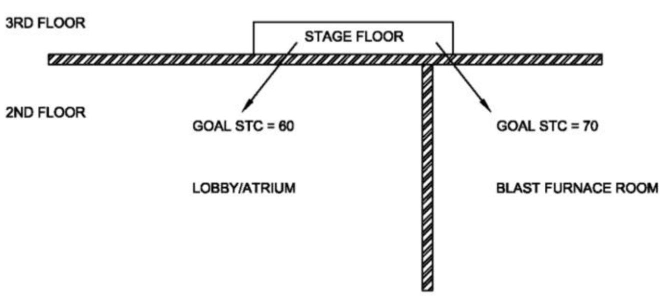


Figure 36: Section showing STC values

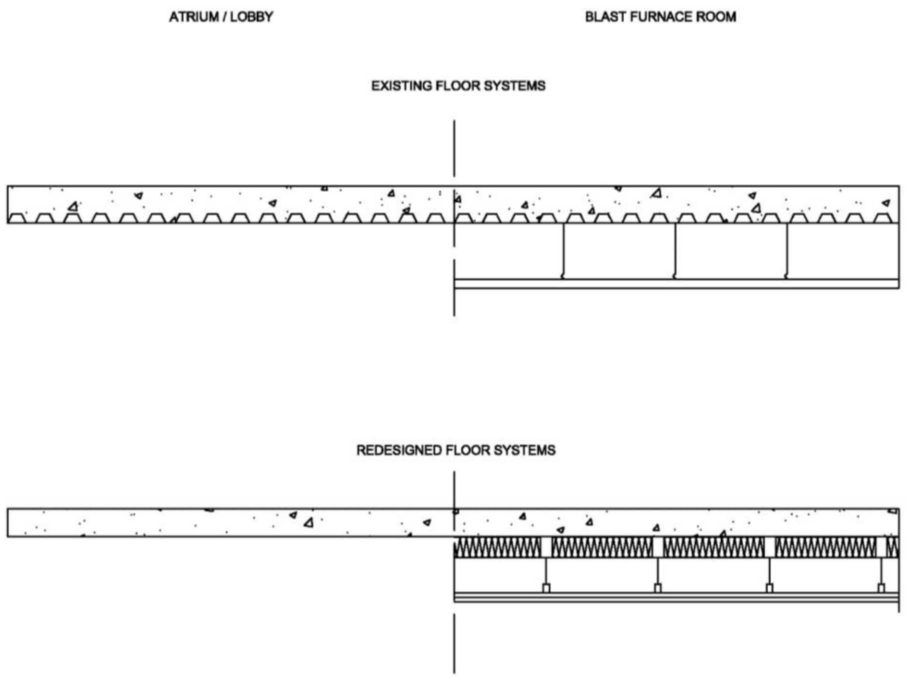


Figure 37: Floor composition options

Data is available only for a portion of floor cross sections, and therefore, the cross section used for the existing can be modeled as (a) an 8" slab and decking system and (b) an 8" slab and decking system hanging acoustical ceiling tile. These systems were redesigned as (c) a 6" concrete slab and (d) a 6" concrete slab with 3/8"plywood, 2 layers of gypsum wallboard, and 2" kinetics isolators. Each of these systems can be seen in Figure 37.

As can be seen in the STC graph in Figure 38 for the system spanning between the Creative Commons and Musikfest Café, the new structural floor system is above the existing and is therefore sufficient. For the other space, which is being redesigned with a thinner floor slab, no metal decking, and continued acoustical ceiling tiles, it is realized that the system is still sufficient. As can be seen in Figure 39, the STC Contour from this design results in 84 dB, which is higher than the existing 58 dB and above the desired 70 dB for this floor system.

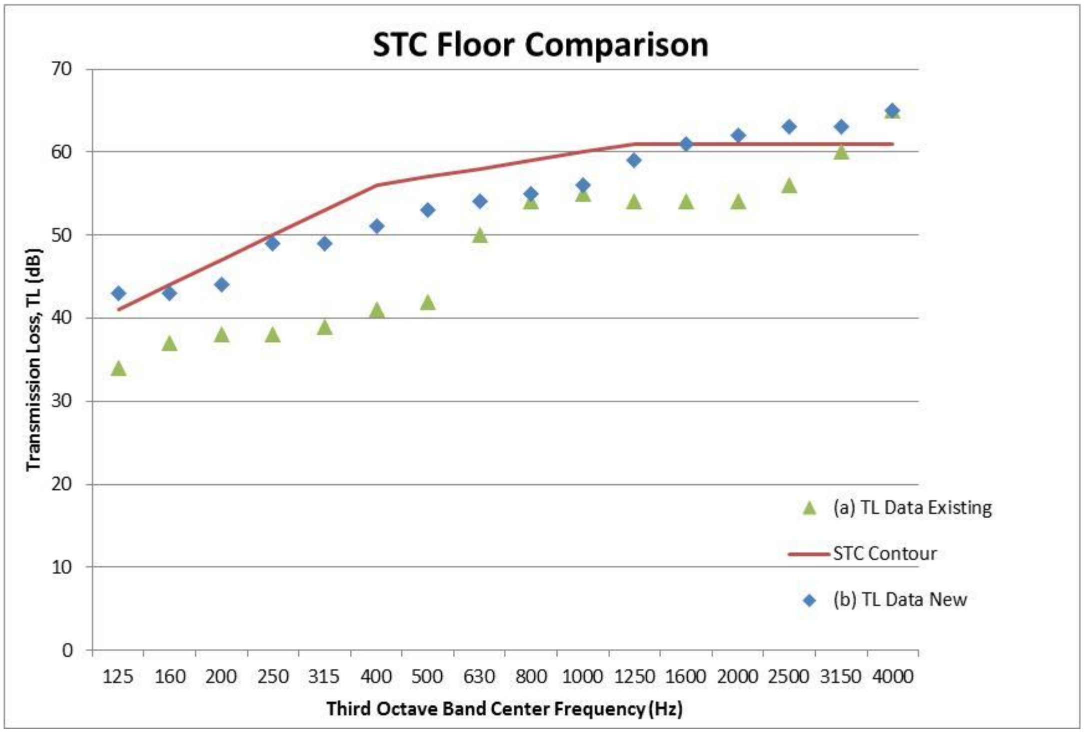


Figure 38: Comparison for Creative Commons and Musikfest Cafe

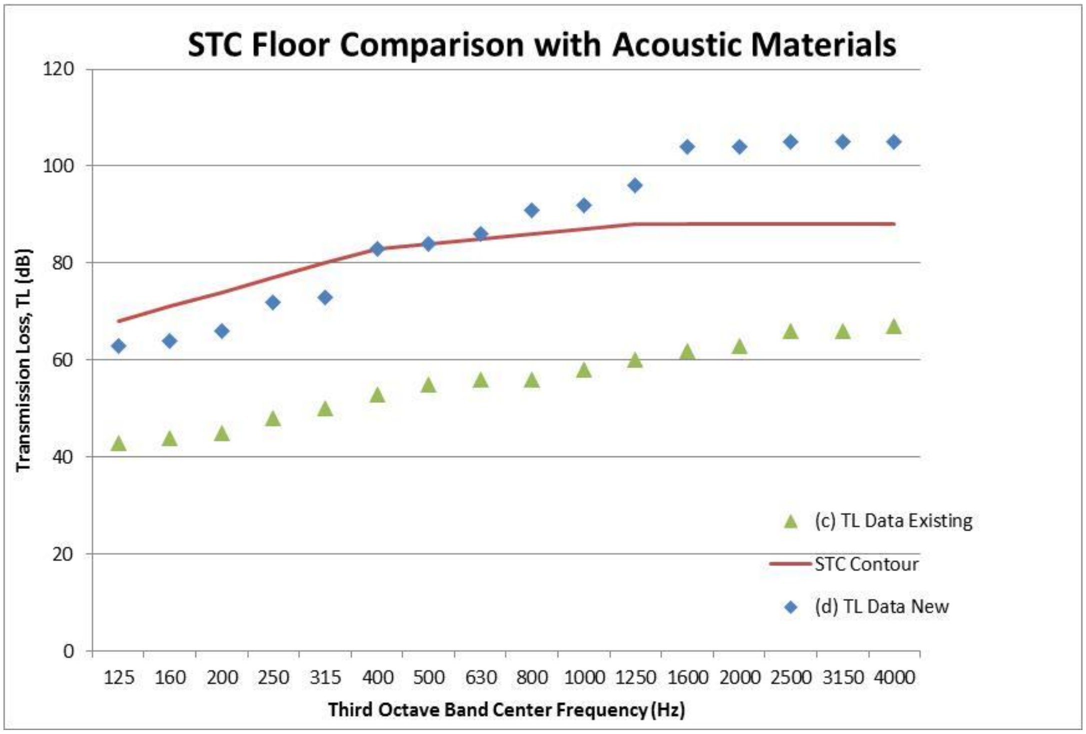


Figure 39: Comparison for Creative Commons and Blast Furnace Room

Table 26: Cost comparison of floor systems

Cost Comparison		
System		Total Cost per sq.ft.
5" slab & decking	EXISTING	10.16
6" concrete slab	REDESIGN	10.88
5" slab & decking w/ acoustic ceiling	EXISTING	15.18
6" concrete slab w/ spring isolators and acoustic ceiling	REDESIGN	15.21

After confirming the new design is acoustically sufficient, a cost comparison was done, per square foot, to estimate the monetary impact of the redesign as compared to the existing. As can be seen in Table 26, the redesigned system costs are comparable to the existing floor system. Though the new system costs 6% more for the simpler system, the floor system above the Blast Furnace Room is a competitive cost per square foot. Therefore, the structural redesign, with the addition of the necessary acoustic materials, is efficient and suggested as the appropriate design for the floor system in terms of both acoustics and cost with a concrete redesign.

Conclusions

In the reevaluation of the RT of each space, surface materials were evaluated and changed based on performance of the spaces. The Blast Furnace room resulted in a better performance, though the RT is still below the desired range. Further investigation of alternative materials is suggestive for a higher RT time. Both theater spaces were matched, and perform adequately. The Creative Commons were much improved for a lively space that does not allow for an abundance of echo.

In evaluation of the new floor material between the Musikfest Café and the spaces below, the floor system was matched to the performance of the existing system. This resulted in a slightly higher cost, due to the structural materials, though additional acoustic materials were eliminated.

Chapter 4: Breadth II: Architecture

Altering the building material from structural steel to a cast-in-place concrete system will directly impact the architecture of the SSPAC. This section will consider the impacts of the redesign on multiple interior spaces, looking at the impacts and adjusting accordingly. For each space, an image of the existing and a rendering of the redesign will be compared and the impacts briefly discussed. These spaces are:

The Creative Commons – first and second floor lobby & atrium

The Blast Furnace Room – second floor multipurpose room

While the building aesthetics change drastically, the building circulation is not majorly impacted. Even though the columns will be slighter larger than the existing steel, most are included in wall lines, and do not directly impact the spaces. The redesign considered this and maintained current building circulation. More of the impacts on each individual space are considered below.

4.1 Creative Commons

The Creative Commons is an interactive, open space spanning the first and second floor that was designed with the intent of displaying clear cuts and structural details, as well as highlighting the iconic orange color of the manufactured products of the original site. The original space can be seen in Figure 41, with the redesign seen in Figure 42. The camera view used on both images is seen in Figure 40.

This use of exposed steel also created a dialogue between some of the existing surrounding unused steel-mill buildings and the interior of the spaces. Through the constant rhythm created by the concrete beams, the perspective of the space and relation to it are kept. The initial design focuses on a compressed space opening into a larger one, and this is maintained through a redesign goal of a higher architectural ceiling. While the original space brought the outside into the space, the use of concrete creates a more secure envelope while still interacting with the exterior. Columns will be designed to not obstruct views within the space. These structural members create a more solid space, and maintain a similar rhythm to the existing steel, through a less busy, consistent beam and girder layout. Similar to the existing system, the final redesign included these exposed structural members as painted orange and accented with lighting.

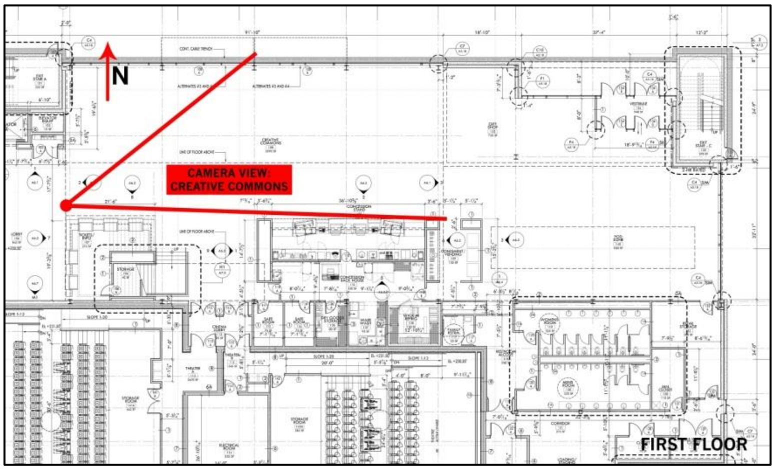


Figure 40: Camera view for Creative Commons

Figure 41: Existing Creative Commons space

Figure 42: Redesigned Creative Commons space

4.2 Blast Furnace Room

The Blast Furnace Room, located on the second floor of the SSPAC, is being viewed from the south east corner of the room, as can be seen in the camera view in Figure 43. The existing space, as seen in Figure 44, is being compared to the redesigned system, as seen in Figure 45. As originally designed, this room looks out towards the surrounding blast furnaces and has large trusses spanning across the room, highlighted by the orange color and focused lighting.

The change from trusses to deep concrete members is the primary impact on the architecture in the Blast Furnace Room. Trusses allow for a more spacious look, and are highlighted in the existing system by lighting and orange accent paint. With a concrete system, these structural members create a bolder, understandable space. To make this similar to the existing system, the final redesign included these exposed structural members as painted orange and accented with lighting, similar to the existing room. This also allows the room to take focus off of the trusses and be visually cleaner. With the concrete beams not taking as much focus, more can be driven towards the interaction of the room with the spaces outside – the blast furnaces, atrium, and second floor that adjoin this space.

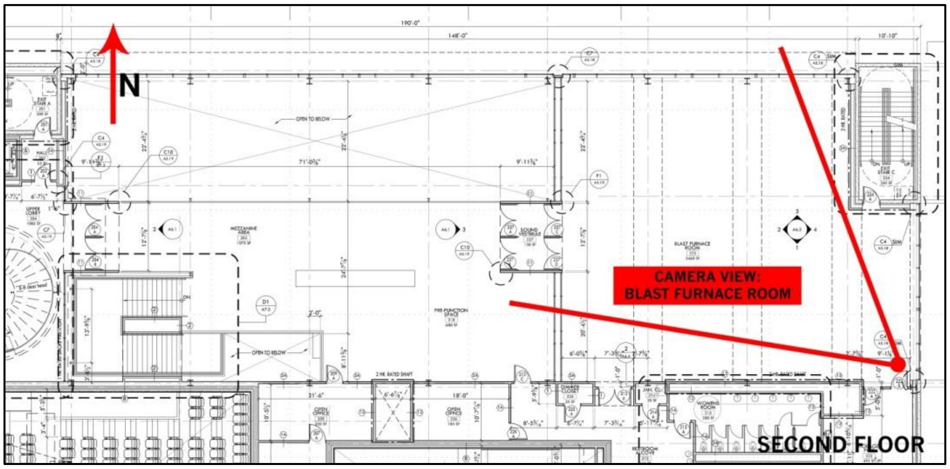


Figure 43: Camera view for Blast Furnace Room

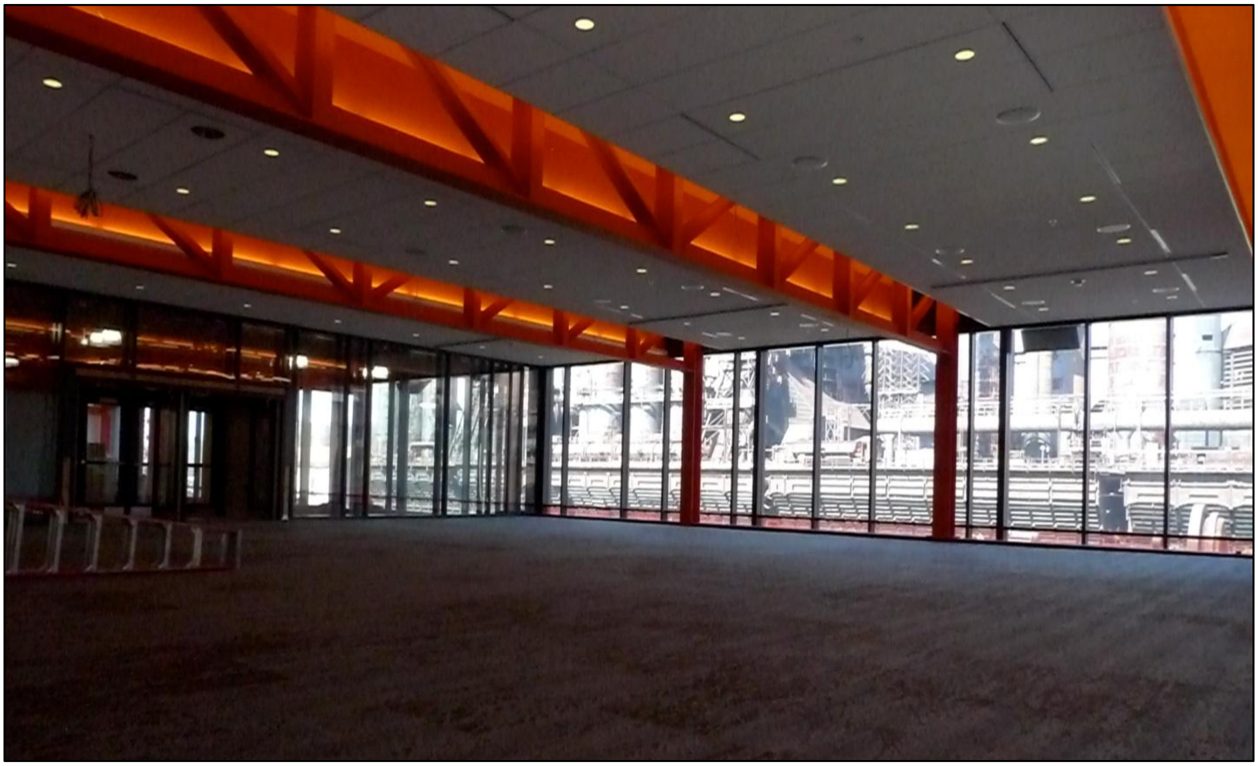


Figure 44: Existing Blast Furnace Room space

Figure 45: Redesigned Blast Furnace Room space

Chapter 5: Conclusion

The SSPAC was redesigned as a cast-in-place concrete gravity and lateral system. Through this design process following ACI 318-11, it can be seen that concrete is a plausible system with some architectural adjustments in certain areas, with seen benefits to some of the current architectural features.

For the gravity design, a bay from each floor was fully designed, with both a normal weight typical reinforced one-way slab-and-beam system and a prestressed system. This analysis resulted in the decision to use prestressed systems for the larger spanning bays. Typical columns, gravity walls, and foundations were successfully designed according to this controlling design. This design gave a better understanding to the design process and specific benefits and disadvantages to the corresponding systems, the computer modeling process, and associated rule-of-thumbs and was overall a success.

The lateral system was also redesigned as an entirely shear system. This redesign took advantage of the redesigned concrete gravity system, and was a successful design. Compared to the existing dual system utilizing braced frames and cast-in-place shear walls, this system takes a more integrated approach with the gravity system and therefore has less redundancy, even though both take advantage of the resources already being used for construction.

The use of a concrete system is beneficial in relation to the complex building layout and the flexibility of concrete to fit different floor diaphragm configurations. Locations where large cantilevers and open spaces were required and floor-to-floor height was desire, prestressed concrete was found to be a viable option. Steel though, was found to be more beneficial in terms of construction and cost and in places where hanging columns were utilized. These would be possible in concrete, but an altering of the current system would be required to be beneficial.

These structural redesigns also impacted other aspects of the building design and construction process. The first one considered in this thesis was the impact on the acoustic performance of each of the spaces, as appropriate for the building use. Reverberation time and sound transmission were evaluated. Concrete design resulted in better acoustic performance in terms of sound isolation and allowed for minimizing acoustic paneling in some rooms when analyzing them for reverberation time.

The architecture was also impacted and considered throughout the redesign process. Room layouts and space uses were not altered by additional columns or layout changes. Two of these spaces were further considered for architectural impact and adjusted as necessary. These analyses were successfully in maintaining the space and atmosphere first created.

In conclusion, the redesign of the SteelStacks Performing Arts Center structural system and evaluation of this redesign's impact on the acoustics and architecture was a success. The proposed thesis goals were exceeded both in depth and breadth, and provided ample knowledge into the design process of concrete and steel systems, as well as the details of prestressed systems, acoustics, and architectural impacts.

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