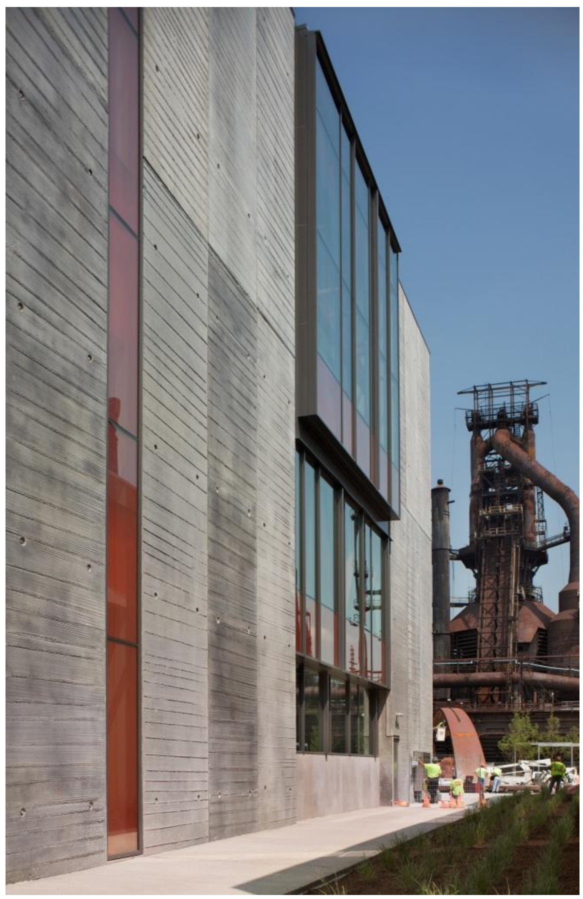
SteelStacks Performing Arts Center | Bethlehem, Pennsylvania

Technical Report I

Structural Existing Conditions



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Executive Summary

The purpose of this report is to establish an exhaustive understanding of the SteelStacks Performing Arts Center (SSPAC). As an arts and cultural center on the site of the Historic Bethlehem Steel, the SSPAC was designed as a composite steel gravity system with precast shear walls and a glass curtain wall system. The purposes of the spaces vary from cinema spaces, to open community spaces, and from a stage area and café to a more private banquet room.

To accomplish this analysis, the composition of the structural system is thoroughly described and explained through the use of images, sketches, and calculations. An understanding of the foundation, floor and roof systems, framing, and lateral systems is detailed in this report. These descriptions help to gain a complete understanding of the building with codes and structural textbooks complementing the study and calculations done to understand the design of the gravity and lateral systems. These results were then compared with the given values on the structural drawings where possible. The structural components and systems checked were found adequate.

The gravity system was evaluated through verification of gravity loads and further developed through checking multiple framing system members. The members considered were a typical bay, a beam, and an interior column. Each of the members was chosen due to being a typical member in the building, and were each found adequate, with values being comparable and within ten percent if not matching.

The lateral system is a combination of braced frames and shear walls. The lateral loadings were detailed and verified for this report, with seismic loads controlling over wind. Base shear for wind was calculated at V=74.7 k in the East-West direction and 124.6k in the North-South direction. Wind loads were 30% of the seismic, and this can be understood through considering the weight distribution of the building, with the third floor being particularly heavy due to a thicker floor slab and less atrium space than the second and fourth floors. The lateral loads and systems will be further considered and analysis more thoroughly developed in a later installment of this report.

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

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Purpose

The focus of this technical report is to analyze existing conditions and design parameters as noted by the professional engineers designing the SteelStacks Performing Arts Center (SSPAC). The gravity loads calculated (dead, live, snow, and rain loads) and framing members designed are compared and verified to the design team's loads and components. Other aspects of the building design considered are seismic and wind loadings, with analyses performed on these and other lateral loads as a point for comparison and understanding of the SSPAC.

Introduction

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

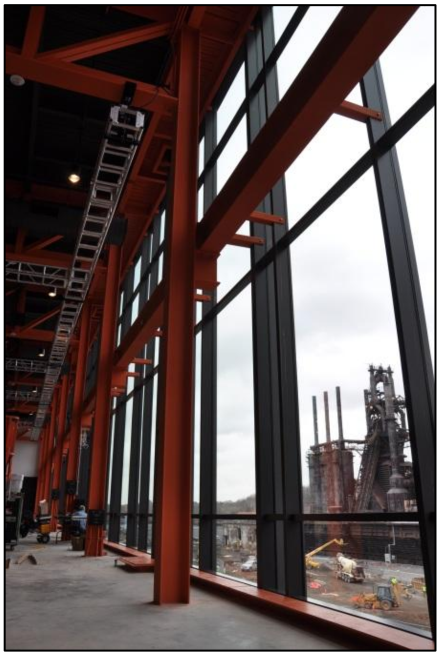


Figure 1: Interior atrium space, highlighting opening structural plan.

Courtesy of Barry Isett, Assoc.

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

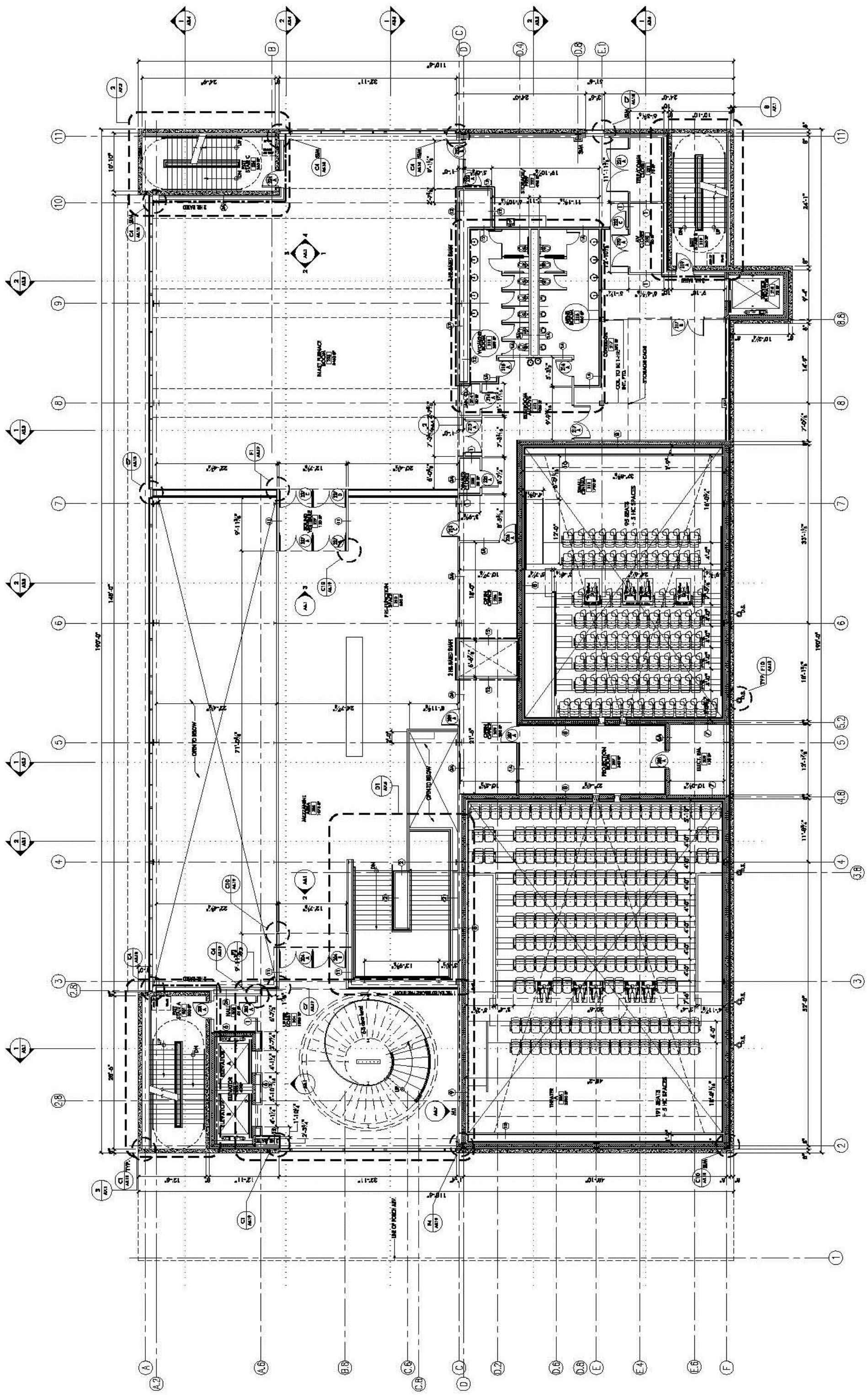


Figure 2: Floor Plan from A2.2

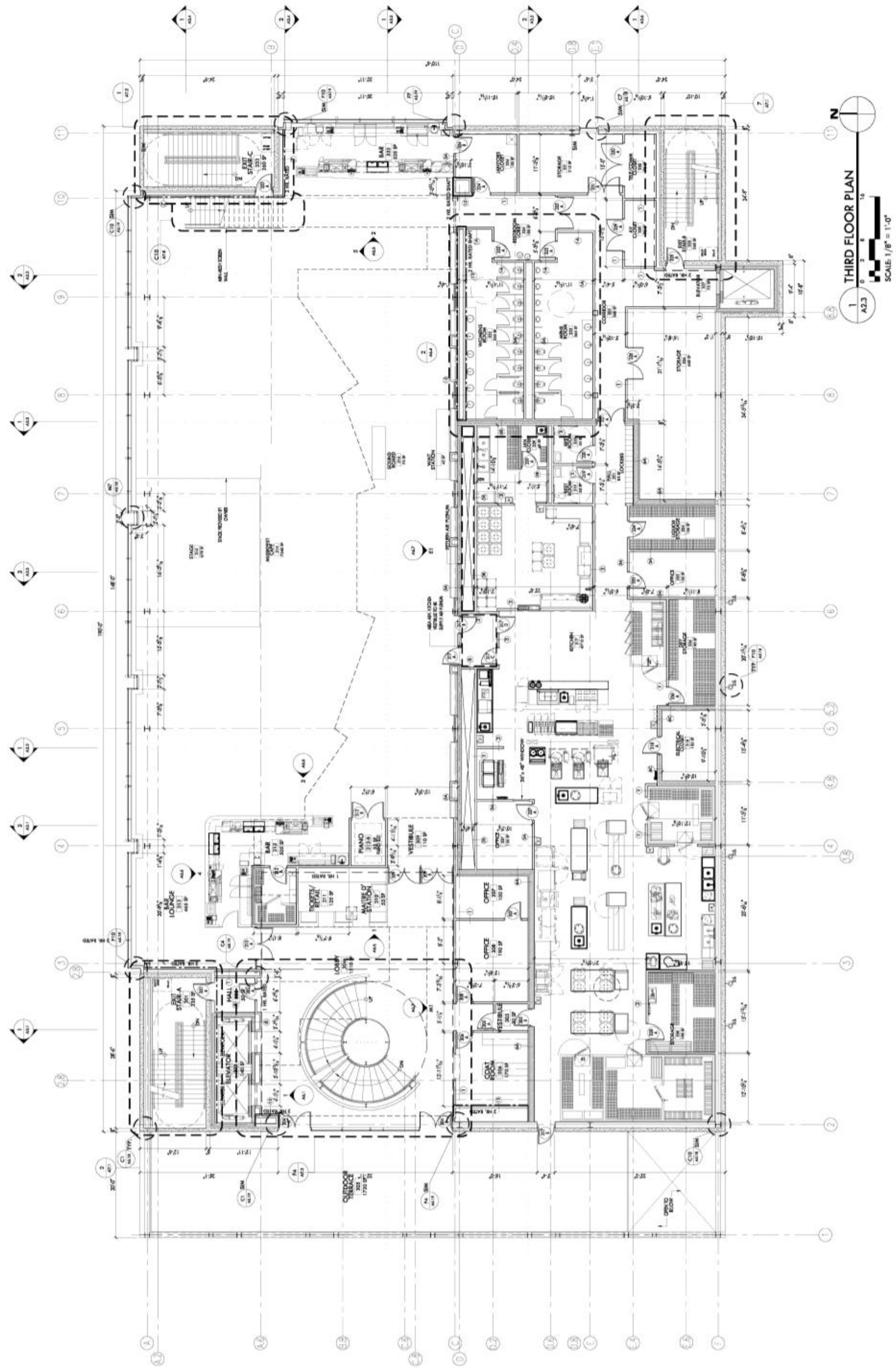


Figure 3: Third Floor Plan from A2.3

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

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

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

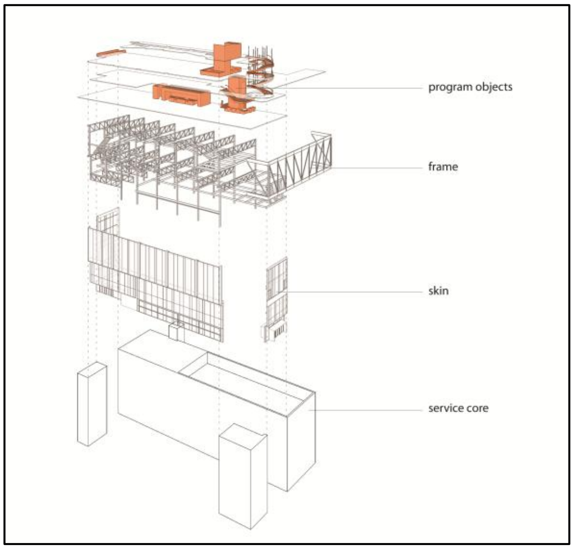


Figure 4 : Image displaying the separation of spaces through the structural design.

Courtesy of Barry Isett, Inc. & Assoc.

General Structural Information

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

Structural System Overview

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

Foundation

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

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

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

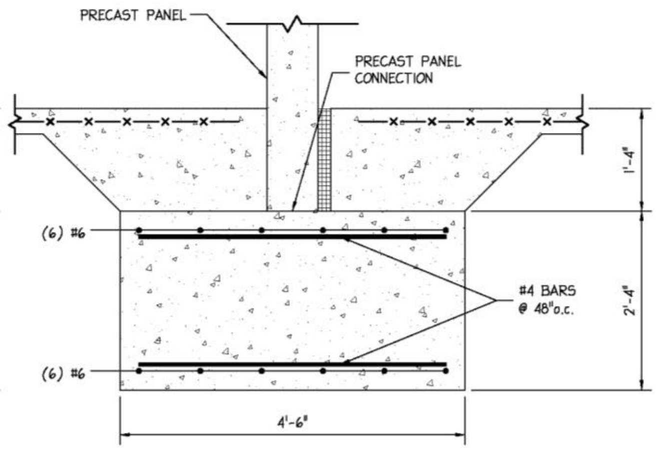


Figure 5 : Section of foundation to precast panel connection from S1.0

Floor System

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

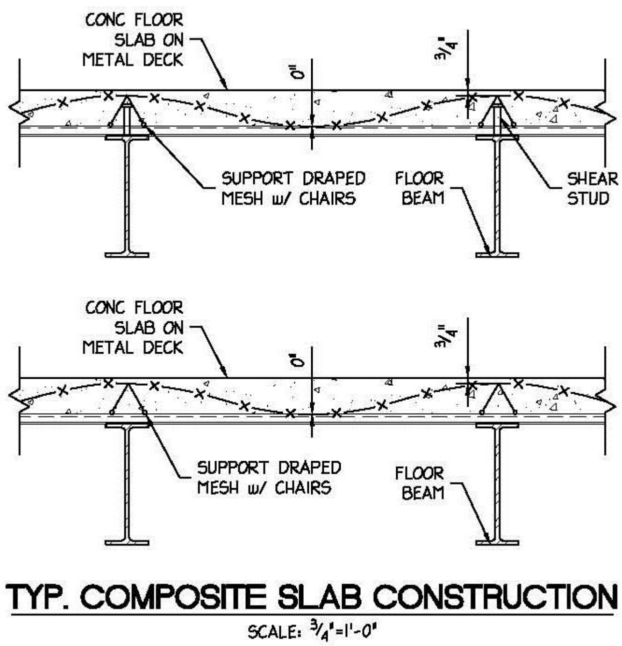


Figure 6 : Typical composite slab section for building from S2.8

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

Framing System

Supporting the floor systems are series of beams, girders, and trusses. Floor beams are spaced at a maximum of 7'6". The beams are also generally continuously braced, with ¾" x 4" long shear studs spaced along all beams connecting to the composite slabs. There are typical members related to each floor, though some spaces have consistent framing plans. The third floor is a primary example of this, as marked in Figure 6 and shown in greater detail in Figure 7.

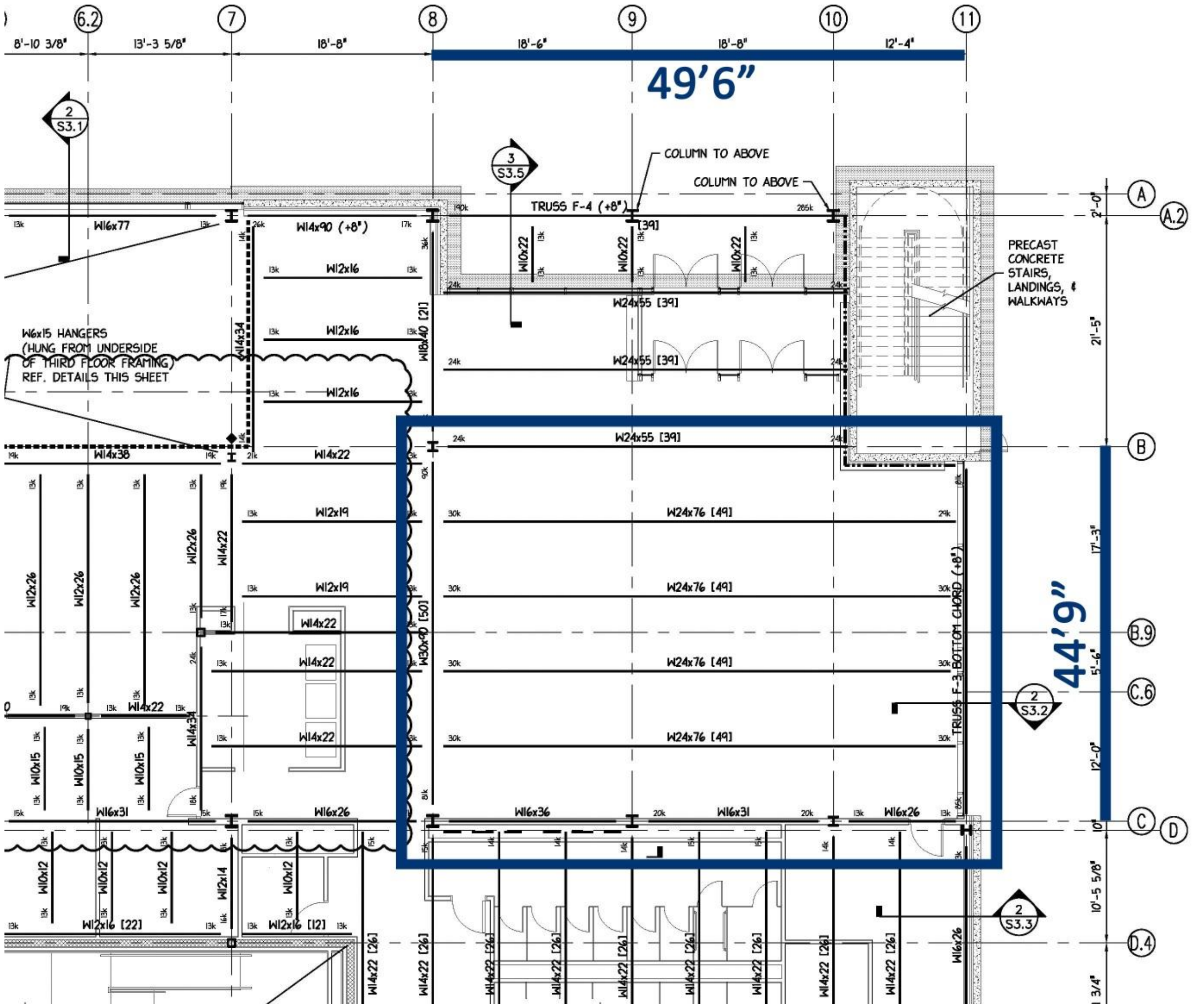


Figure 7 : Second floor framing plan, with a representative bay of a typical frame, highlighted in blue, from S2.0

Generally, the second floor consists of W12x26s for the mezzanine area and W24x76s for the blast furnace room. Typical 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 8.

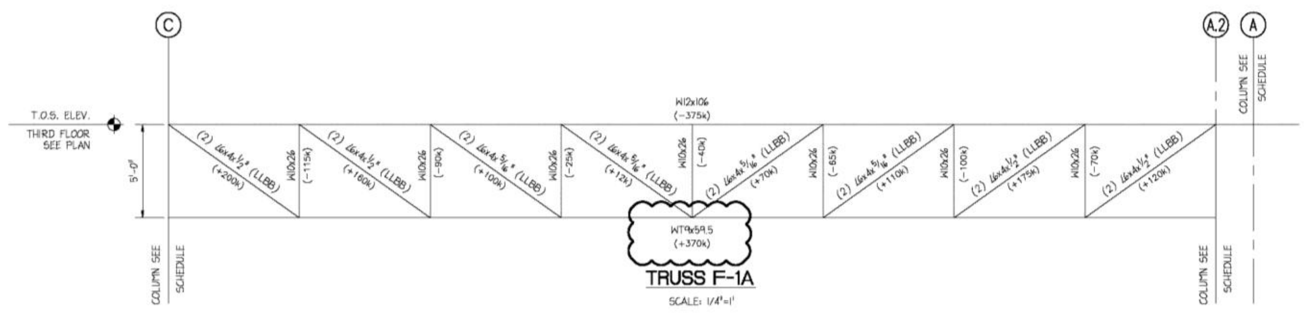


Figure 8 : Third floor representative framing system truss.

Framing on the fourth floor is more irregular, 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 8. Truss R-2 is included in Appendix 1.

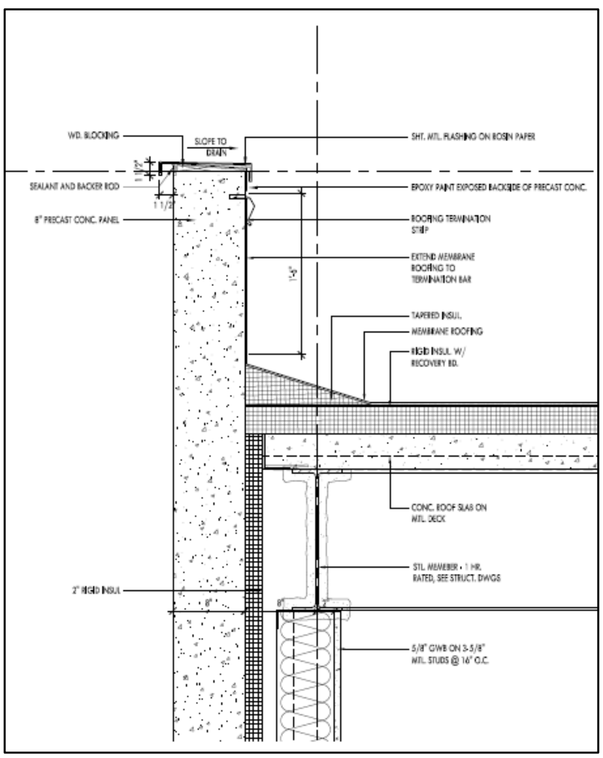


Figure 9 : Cross section of the roofing system.

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

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

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

Lateral System

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

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

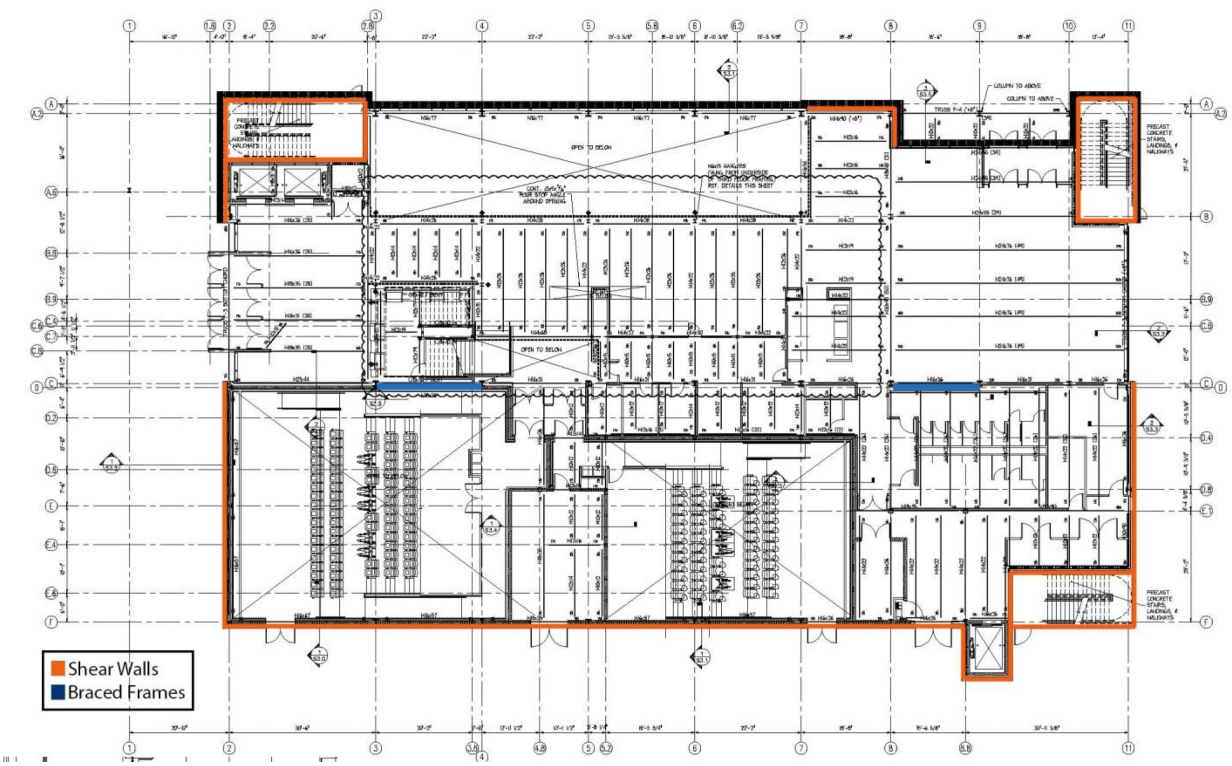


Figure 10 : Floor plan highlighting shear walls in orange and braced frames in blue, which contribute to the lateral system.

Design Codes

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

Design Codes:

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

Design Guides Used for Design:

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

Thesis Codes & Design Guides:

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

Materials

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

Concrete

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

Steel

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

Other

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

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

Determination of Design Loads

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

Dead and Live Loads

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

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

Table 11 : Table of Superimposed dead loads.

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

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

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

*Dashes designate values not provide in the structural drawings.

Wind Loads

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 13. Also, 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.

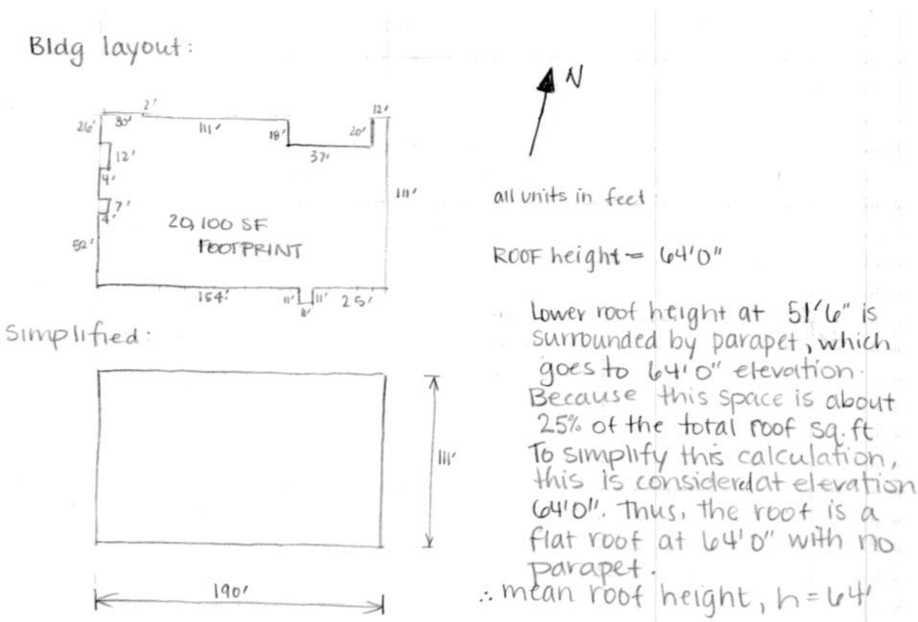


Figure 13 : Building dimensions simplified for wind load calculations following Method 2.

Calculations consider 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 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. An excel spreadsheet was formed for ease and accuracy of values for wind, and can be found in Appendix 2, along with the preceding hand calculations previously mentioned.

A summary of the wind pressures and variables going into these pressures in each direction are displayed below, in Figures 14 through 23. These results have been summarized for the East-West direction in Figures 14 through 18, and highlight the base shear and overturning moment due to these wind pressures. Figures 19 through 23 summarize similar results and drawings for the North-South direction.

As the structural drawings did not record wind loadings beyond some of the input variables, these results cannot be compared to those of the structural engineer. The input values that can be compared though, the pressure variables, are comparable and very similar, if not the same in most cases. For example, the maximum total windward pressure from the structural drawings is 38.9 psf, where the maximum value calculated below is 37.8 psf.

The overall base shear for the East-West direction is 266.3 k, with an overturning moment of 17040 k-ft. These results can be compared with the North-South direction, where the base shear was higher, at 445.8 k, and the overturning moment at 28530 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.

Wind Pressures East-West Direction									
Type		Location	Distance (ft)	Pressure Variables					Pressure
				Cp	qz	qh	G	GCpi (+/-)	(psf)
Wall	Windward	Ground	0	0.8	11.55	17.63	0.873	0.18	5.99
		Floor 2	17.5	0.8	12.16	17.63	0.873	0.18	6.30
		Floor 3	35	0.8	14.80	17.63	0.873	0.18	7.67
		Floor 4	46.5	0.8	16.82	17.63	0.873	0.18	8.72
		Roof	64	0.8	17.63	17.63	0.873	0.18	9.14
	Leeward	All	All	-0.36	17.63	17.63	0.873	0.18	-8.71
	Side	All	All	-0.7	17.63	17.63	0.873	0.18	-13.95
Roof		0 to h/2	0 to 32	-0.9	17.63	17.63	0.873	0.18	-17.03
		h/2 to h	32 to 64	-0.9	17.63	17.63	0.873	0.18	-17.03
		h to 2h	64 to 128	-0.5	17.63	17.63	0.873	0.18	-10.87
		>2h	>128	-0.3	17.63	17.63	0.873	0.18	-7.79
							E-W load (k)	Sum Wall	37.82
								Sum Roof	-52.71

Figure 14 : Summary of wind pressure calculations in the East-West direction.

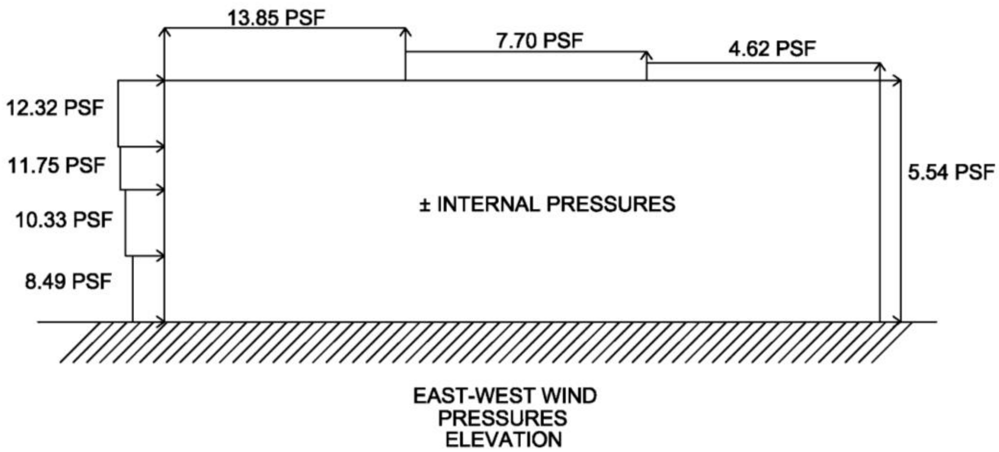


Figure 15 : Summary of East-West wind pressures in elevation.

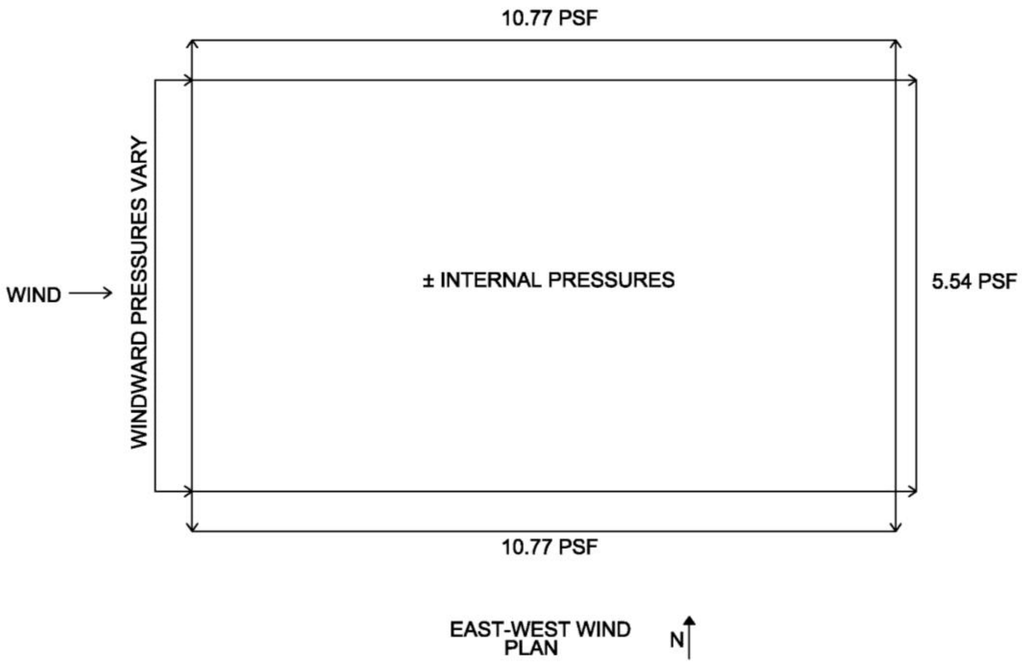


Figure 16 : Summary of East-West wind pressures in plan.

Overturning Moment/Base Shear East-West Direction					
Windward Wall	Total Force (psf)	Height (ft)	Width (ft)	Base Shear (k)	Overturning Moment (k-ft)
	37.82	64.0	110.0	266.3	17040.18

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

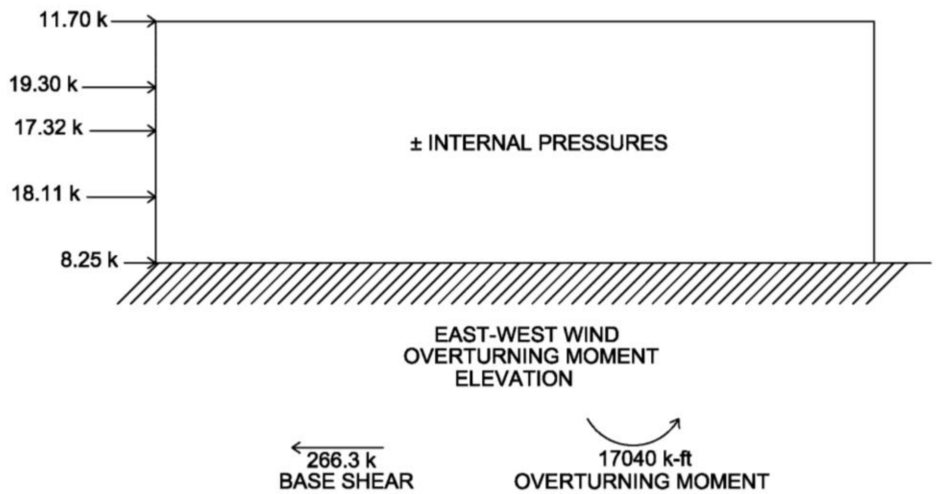


Figure 18 : Summary of final forces in East-West direction in elevation.

Wind Pressures North-South Direction									
Type		Location	Distance (ft)	Pressure Variables					Pressure
				Cp	qz	qh	G	GCpi (+/-)	(psf)
Wall	Windward	Ground	0	0.8	11.55	17.63	0.853	0.18	5.80
		Floor 2	17.5	0.8	12.16	17.63	0.853	0.18	6.11
		Floor 3	35	0.8	14.80	17.63	0.853	0.18	7.44
		Floor 4	46.5	0.8	16.82	17.63	0.853	0.18	8.45
		Roof	64	0.8	17.63	17.63	0.853	0.18	8.86
	Leeward	All	All	-0.5	17.63	17.63	0.853	0.18	-10.69
	Side	All	All	-0.7	17.63	17.63	0.853	0.18	-13.70
Roof		0 to h/2	0 to 32	-1.0	17.63	17.63	0.853	0.18	-18.21
		h/2 to h	32 to 64	-0.8	17.63	17.63	0.853	0.18	-15.20
		h to 2h	64 to 128	-0.5	17.63	17.63	0.853	0.18	-10.69
		>2h	>128	N/A	17.63	17.63	0.853	0.18	N/A
							N-S load (k)	Sum Wall	36.66
								Sum Roof	-44.11

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

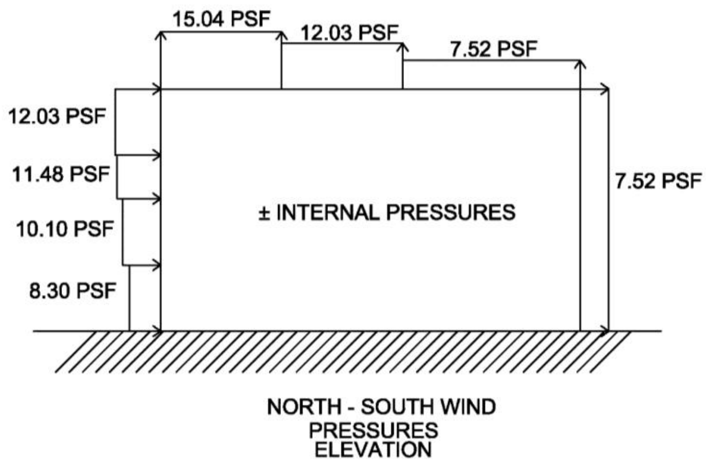


Figure 20 : Summary of forces in the North-South direction in elevation.

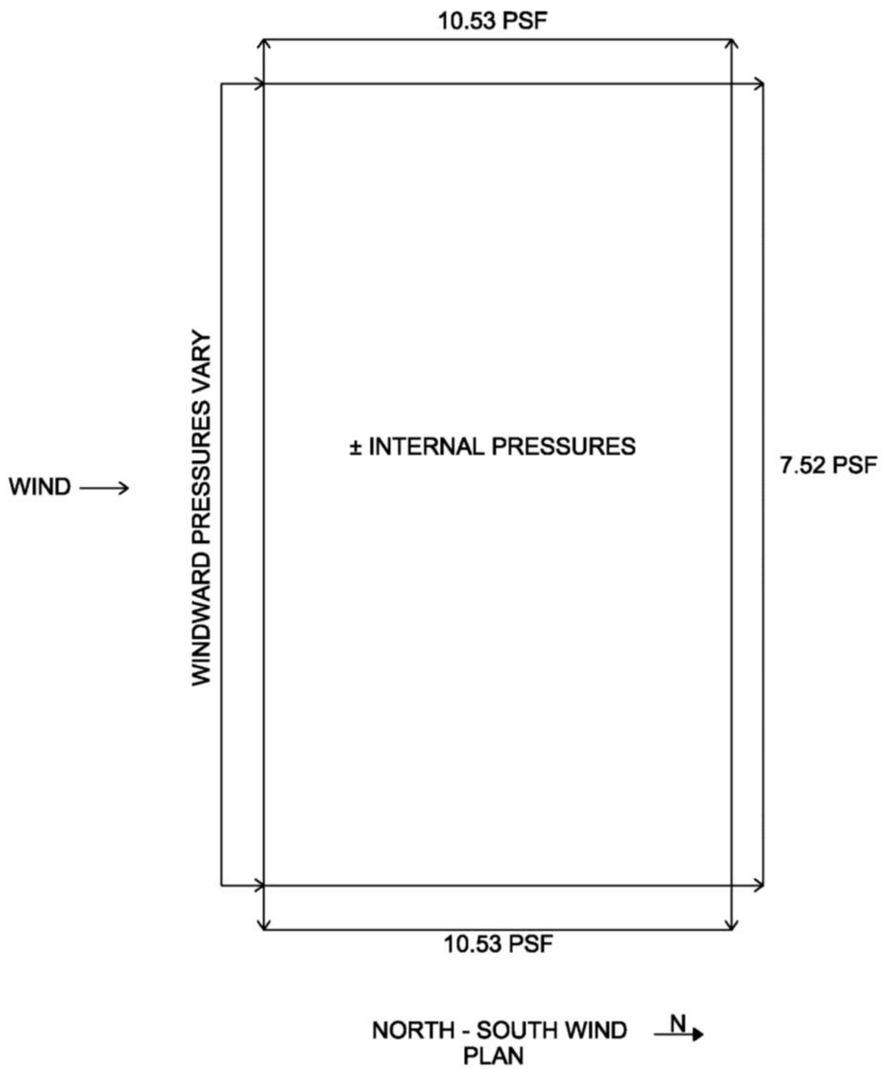


Figure 21 : Summary of pressures in the North-South direction in plan.

Overturning Moment/Base Shear North-South Direction					
Windward Wall	Total Force (psf)	Height (ft)	Width (ft)	Base Shear (k)	Overturning Moment (k-ft)
	36.66	64.0	190.0	445.8	28530.28

Figure 22 : Summary of overturning moment and base shear calculations in the North-South direction.

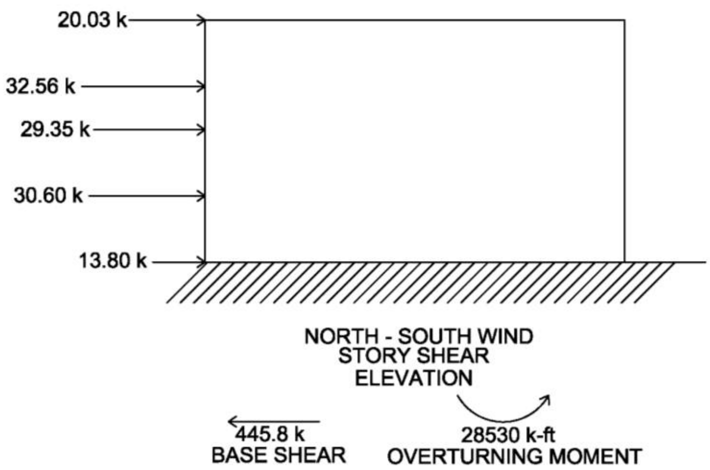


Figure 23 : Summary of final forces in North-South direction in elevation.

Seismic Loads

Seismic calculations follow ASCE 7-05 Chapters 11 and 12, using the Equivalent Lateral Force Procedure, which is also the method used for the structural plan designs. This procedure included the variables listed in Table 24, some of which were taken from the geo-technical report, while others were calculated. The calculations related to these variables and results are presented in Appendix 3. The lateral system for the SSPAC in the East-West direction is a braced-frame 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 R.

Variable	Value
S_s	1.5
S_1	0.26
Site Class	D
Sds	1.06
S_{D1}	0.28
Cd	3
T_s	0.347
Ta	0.6788
C_u	1.7
T	1.15
T_L	6
C_s	0.042

Table 24 : Table of seismic load variables and values.

Comparing values calculated from this report with those on the structural drawings, the values are exact excluding C_s. For this value, the structural drawings denote C_s=0.138, while the calculated value was C_s=0.139 before applying Section 12.8.1-1, which caps this value at 0.042. After discussing this with the structural engineer, it was concluded that the C_s cap was used for the structural drawings using RAM software, though not noted.

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, including the 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=11055 kips, was under 10% of the building weight calculated by the engineer through the use of a RAM model.

Using the value of C_s=0.042 and the building weight, W=11055 kips, the base shear could then be calculated. The base shear calculated in this report is V=464.3 kips, with an overturning moment of approximately 48600 k-ft, as elaborated on in Figure 25 and summarized in Figure 26. Structural drawing S2.8 denotes a base shear value, V=506.5 kips. The calculated base shear is only 9% 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 Figure 25, with a summary of the results displayed in Figure 26.

In comparison, the values for the seismic loadings controlled over the wind loads.

Seismic Forces							
Level	Story Weight, w_x (k)	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-
Ground	N/A	0	N/A	N/A	N/A	N/A	N/A
Floor 2	2353111.0	17.5	105,896,844	0.070	32.7	464.3	8125
Floor 3	4196745.0	35	474,813,524	0.315	146.4	431.6	15108
Floor 4	1874791.0	46.5	309,503,302	0.206	95.5	285.2	13262
Roof	2436715.0	64	615,211,014	0.409	189.7	189.7	12144
Cs	0.042				Base Shear [V=Cs*W] (k)		464
W(k)	11055				Total Overturning Moment (k-ft)		48639

Figure 25 : Summary of calculations for seismic load design.

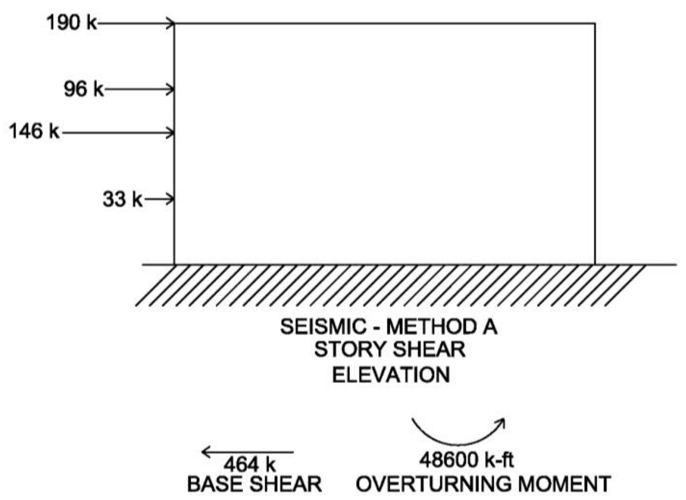


Figure 26 : Summary of forces due to seismic loads.

Snow Loads

The structural plans noted that the "Snow load controls roof design" and is therefore a primary focus of comparison in this section. The method of calculations follows ASCE 7-05, and factors used for the calculations are summarized in Table 27. The procedure for flat roofs was followed for the primary snow load of 30 psf, the value to be applied to the entire roof system, with drifts additional in certain areas.

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

Figure 27 : Summary of snow load variables.

With the height difference of 9.8 feet between the mechanical roof and the other roof and parapet heights, 5 locations on the mechanical roof were chosen for drift calculations. The magnitude of these drift heights led to an increase of the snow load from the base of 30 psf to 50 psf along the exterior 15 feet of the mechanical roof depression. Values assumed on the structural drawings coincide with the code allowances and results, reinforcing the statement that snow load controls roof design, with snow drifts being a primary concern on the mechanical roof. A summary of these results is given in Figure 28.

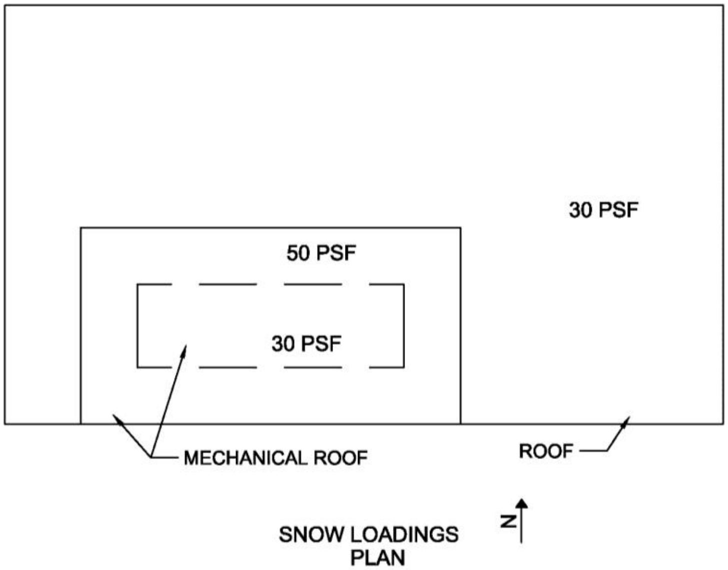


Figure 28 : Summary of snow loads.

Rain Loads

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

Evaluation of Systems

As a further analysis of the structural system, and a better understanding of the design, representative members were selected for spot checks and designs per the structural drawings were compared against these calculations and verified.

Floor System for Typical Bay

Considering the structural systems that support the gravity loads, two general bays have been selected for further analysis of the loading used for design. The first bay shown in Figure 29 has been selected from the second floor framing of the structural documents, supporting a public space entering from the main stairwell into a mezzanine area, as it is a general non-composite bay. The second bay, detailed later, is a primary example of a non-composite bay used within the SSPAC. Both systems were seen throughout the building structural system.

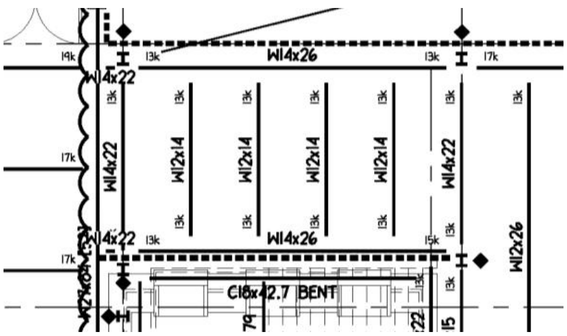


Figure 29 : Representative Bay for member analysis.

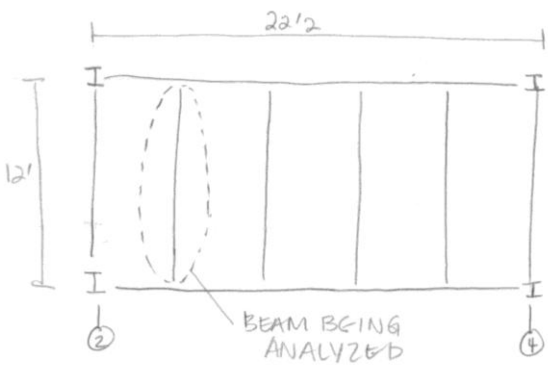


Figure 30 : Callout of beam used in second spot check.

The first structural member chosen for a more thorough analysis was the composite slab on this bay. These calculations, which can be found in Appendix 6, used the Vulcraft Steel Decking Catalog for design. This resulted in choosing a 2VLI20 composite deck, which is the same as was specific in the drawings. More specifically, the maximum span of 7'6"noted in the drawings was used for design, as well as the 3-span requirement. A superimposed live load of 249 psf exceeded the loading on the slab, calculated at 222 psf. Explanations for the use of this decking across the entire floor is logical in terms of production, where metal decking is manufactured in 42'0" length. It would therefore be economical to have metal decking consistent across the floors.

The second structural component chosen was the beam called out in Figure 30. This beam is a non-composite beam, representative of beams throughout the structure. The calculations completed for this report design the member as a W10x12 that sees a shear of 13 k and a moment of 38 k-ft. While the loading is consistent with that shown on the drawings, the member size chosen for the drawings was a W12x14. Considering the use of the building in coordination with acoustics, and the size of this bay compared to others, it can be noted that this specific bay did not control design. However, due to the fact that most beams are consistently W12s on this floor, the reason for upgrading to larger beam in this case can be attributed to consistency is member sizing.

The third member chosen was a composite beam on the second floor, called out in Figure 31. As a composite beam, calculations resulted in a member designed as a W24x76 with 34 studs. Comparatively, it was designed on the structural drawings as a W24x76 with 49 studs. The differences here in studs can be attributed to a difference in dead and live loads used throughout calculations.

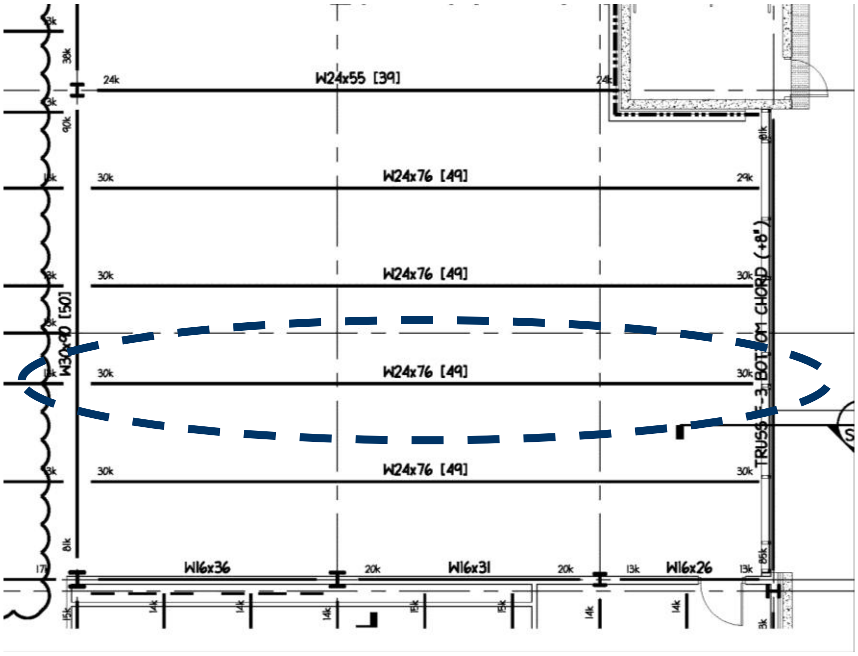


Figure 31 : Callout of composite beam chosen for spot check.

Typical Columns

This building also has a regular layout for columns supporting the building where shear walls do not. A spot check for a typical column is done on the column located at column line B8, and displayed in Figure 32. It can be seen that column B8 extends from the ground floor to the second floor, considering the column schedule on the plans, attached in Appendix 6 for further reference.

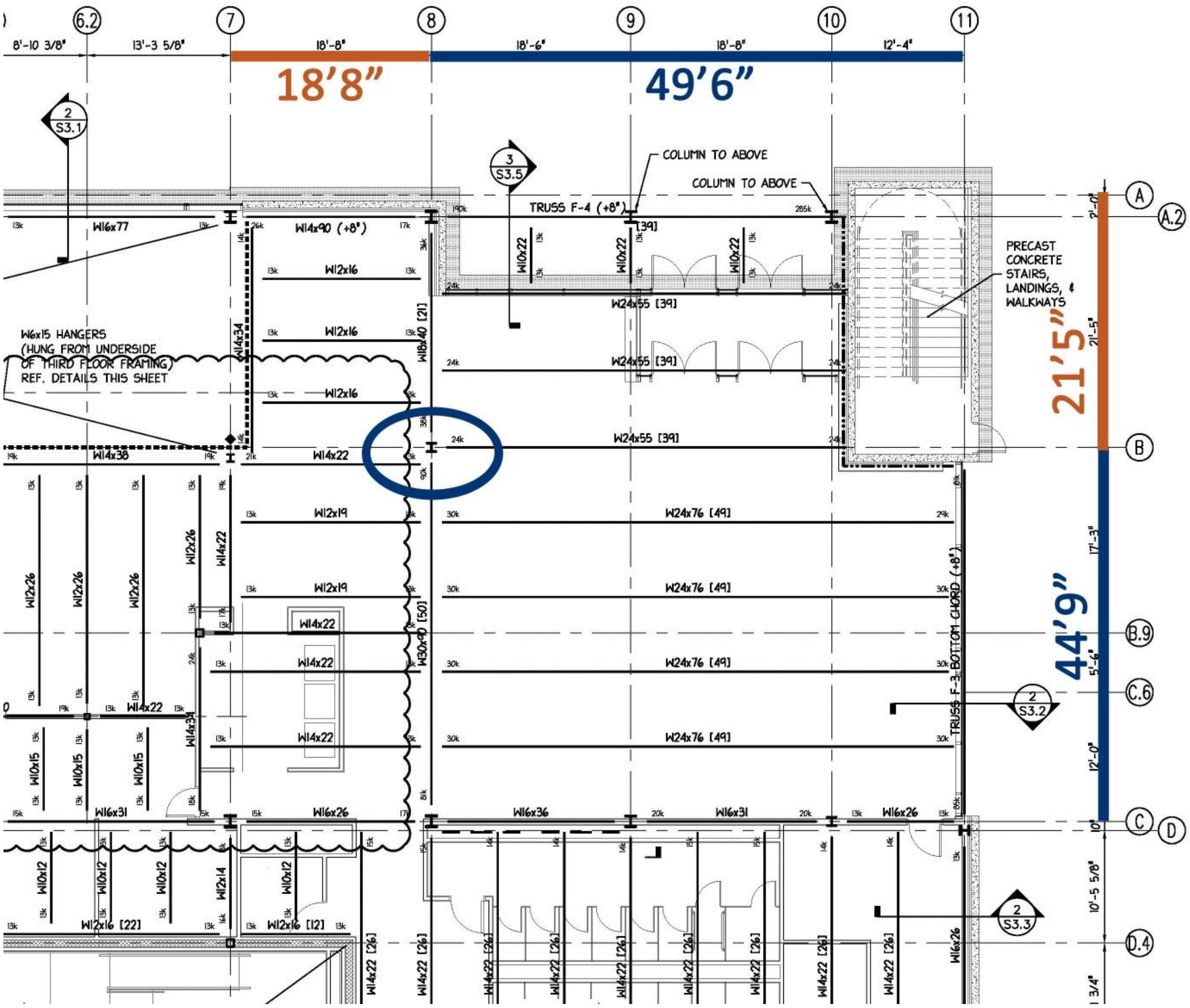


Figure 32 : Representative column used for further analysis

Calculations, as seen in Appendix 6, design the column to be a W10x45 by both axial loading and moment considerations. This design is comparable to the W10x49 chosen on the plans. This slight difference in weight can be due to this neglecting of P-delta effects, but is more likely due to conservative design, for vibration considerations and member consistency, by the engineer, as was the case in the beam analyzed above. 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.

Conclusion

Through the comprehensive and in-depth analysis of the SteelStacks Performing Arts Center, a better understanding of the structural systems and design has been accomplished. This report shows the results of this analysis through an overview of the structural system overview, calculation of the gravity and lateral loads, and comparisons of the original design values and the report's results. These design procedures relied heavily on ASCE 7-05 and AISC, 14^{th} edition. It can be seen that the SSPAC is a complex building that gives opportunity for further analysis of the interactions between structural design, architectural purpose and intent, and space needs.

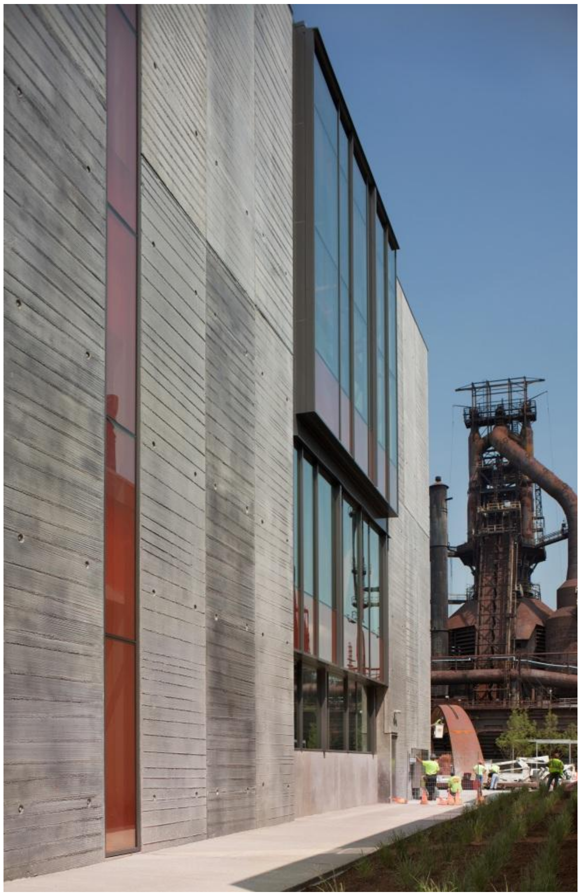
As a primarily steel structure, the gravity system is comprised a steel skeleton that employs both girders and deep trusses. Through analysis, the dead and live loads were found on the building, with the snow and rain loads looked at in greater detail. Dead, live, and snow loads were found within a reasonable percentage of those on the structural drawings, with snow continuing to control on the roof design. Though rain did not control design, it was an opportunity for further exploration of the code and possible direction for further analysis and redesign.

These gravity systems were further considered through the spot checks performed, with replication of design results within a reasonable percent difference, with the design team being slightly more conservative than the designs contained in this report. Their slightly more conservative designs can be attributed to logical reasoning of consistent framing, acoustic concerns, and architectural features.

The lateral systems are unique in direction and variety, combining the architect's vision for the building with the structural and performance needs of the space. The North-South direction of the structure employs a series of shear walls, seen in the precast concrete panels, and in the East-West direction, employs a set of braced frames. These lateral systems will be further considered in the second installment of this report.

The lateral loads analyzed were wind and snow loads. The wind loads used for design were not given on the drawings, but the variables input into wind load calculations were matched precisely, through the use of ASCE 7-05. Seismic loads were found to control, with the engineer's values found within 10%.

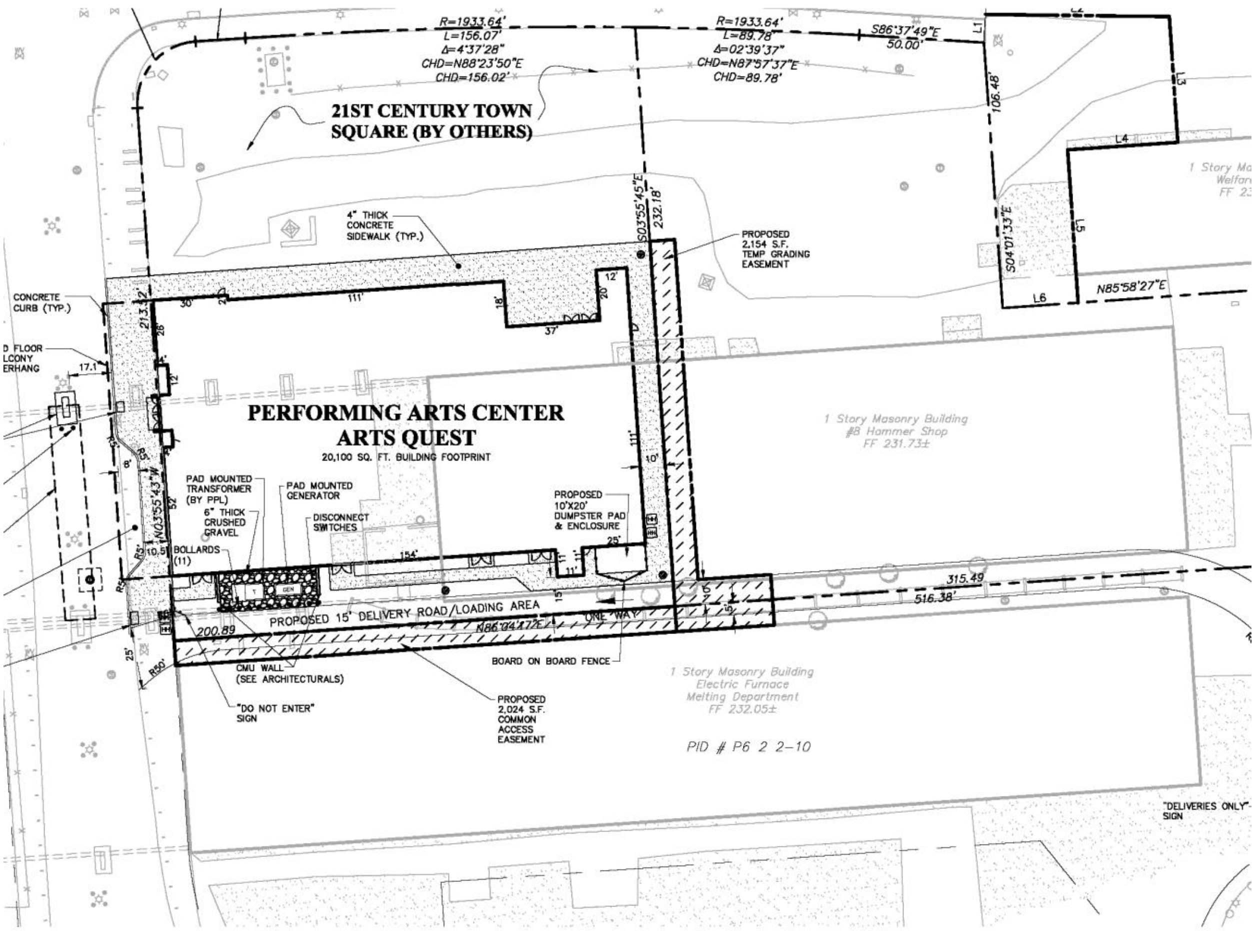
Appendices



Appendix 1: Structural System Overview

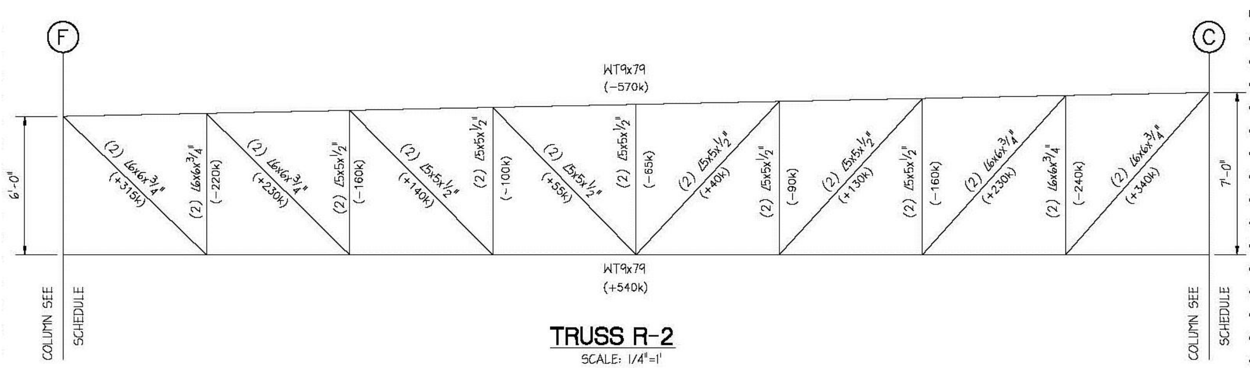
Site Plan Detail

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



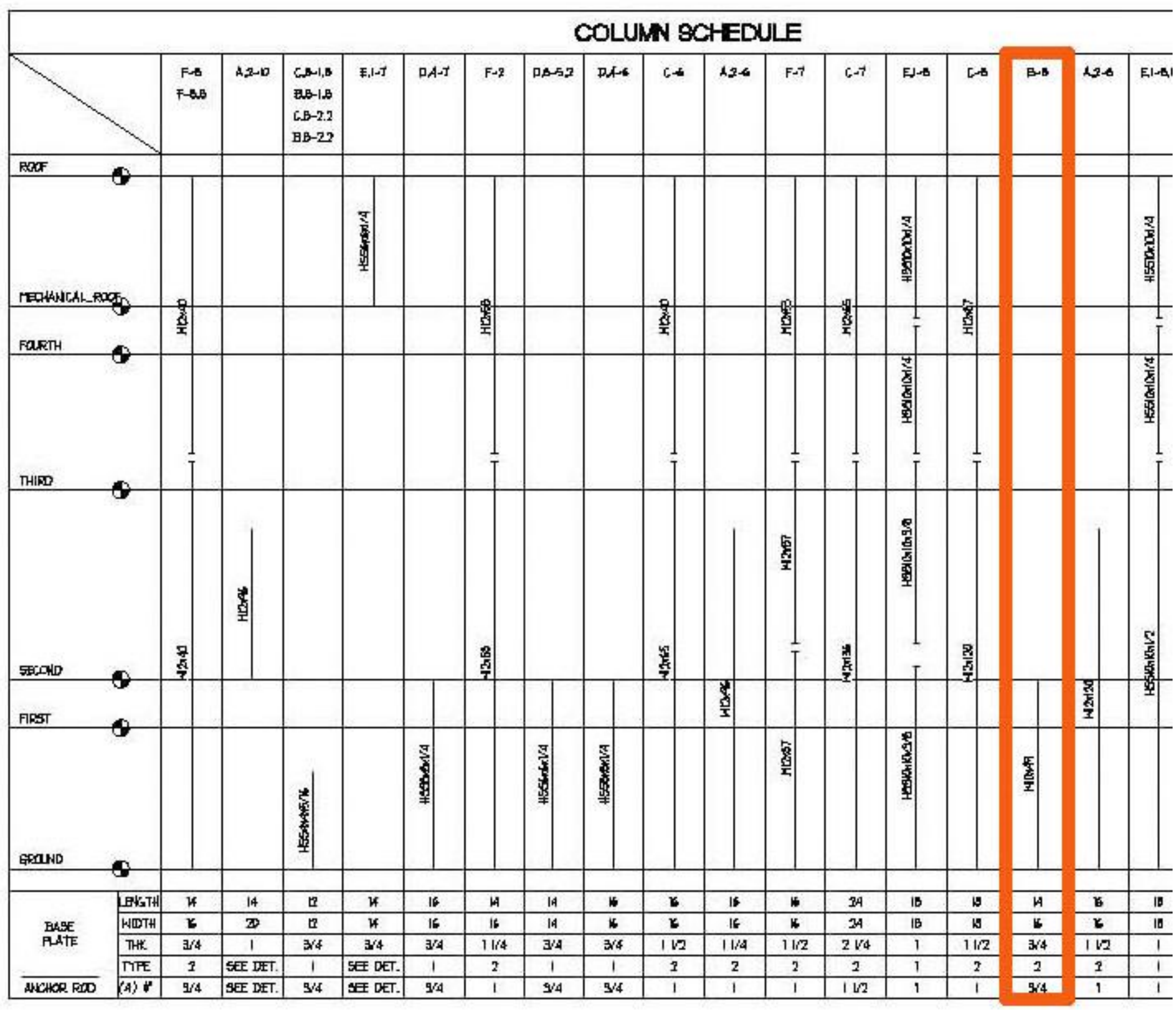
Representative Truss

This truss was referenced in the overview of the framing system.

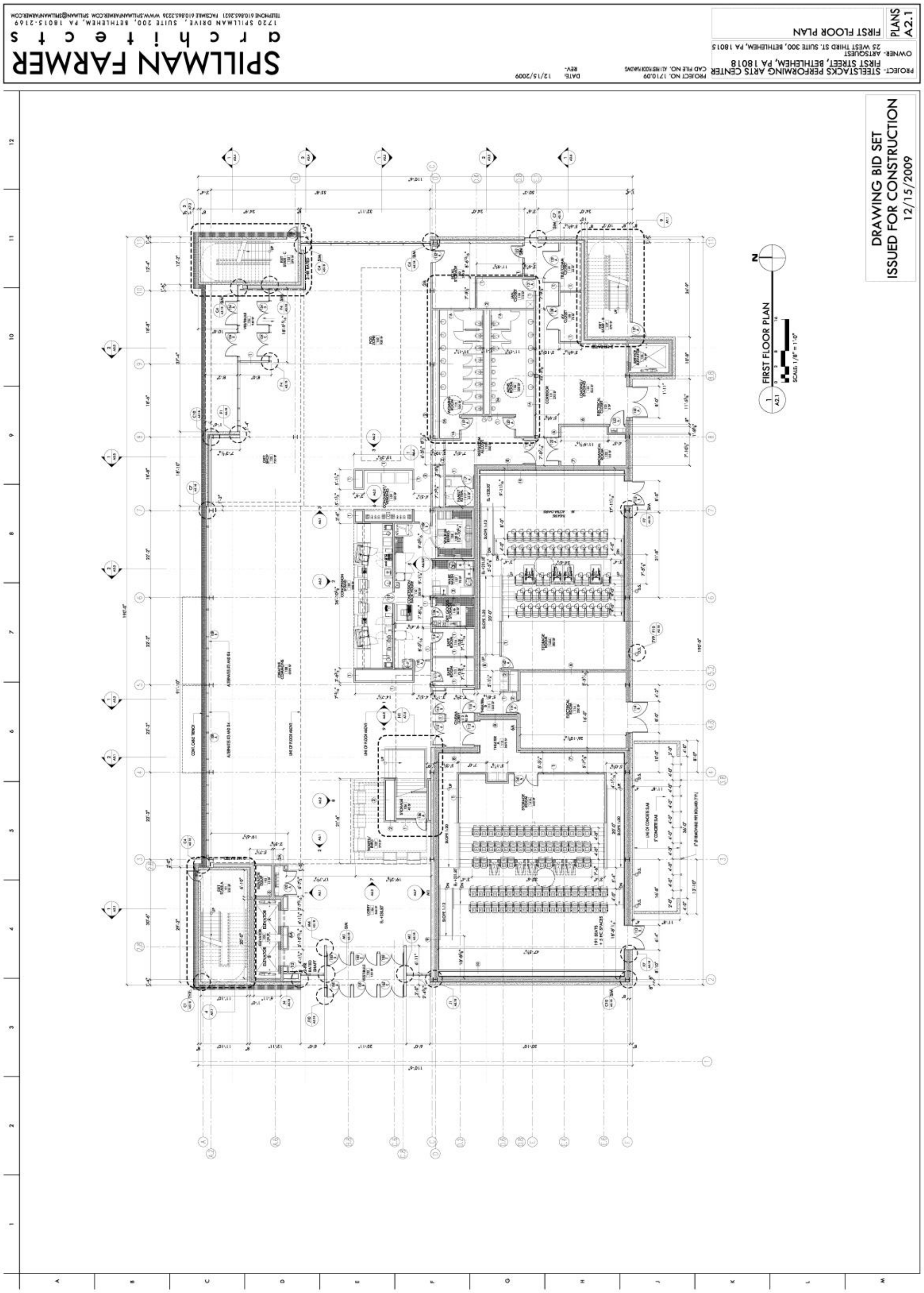


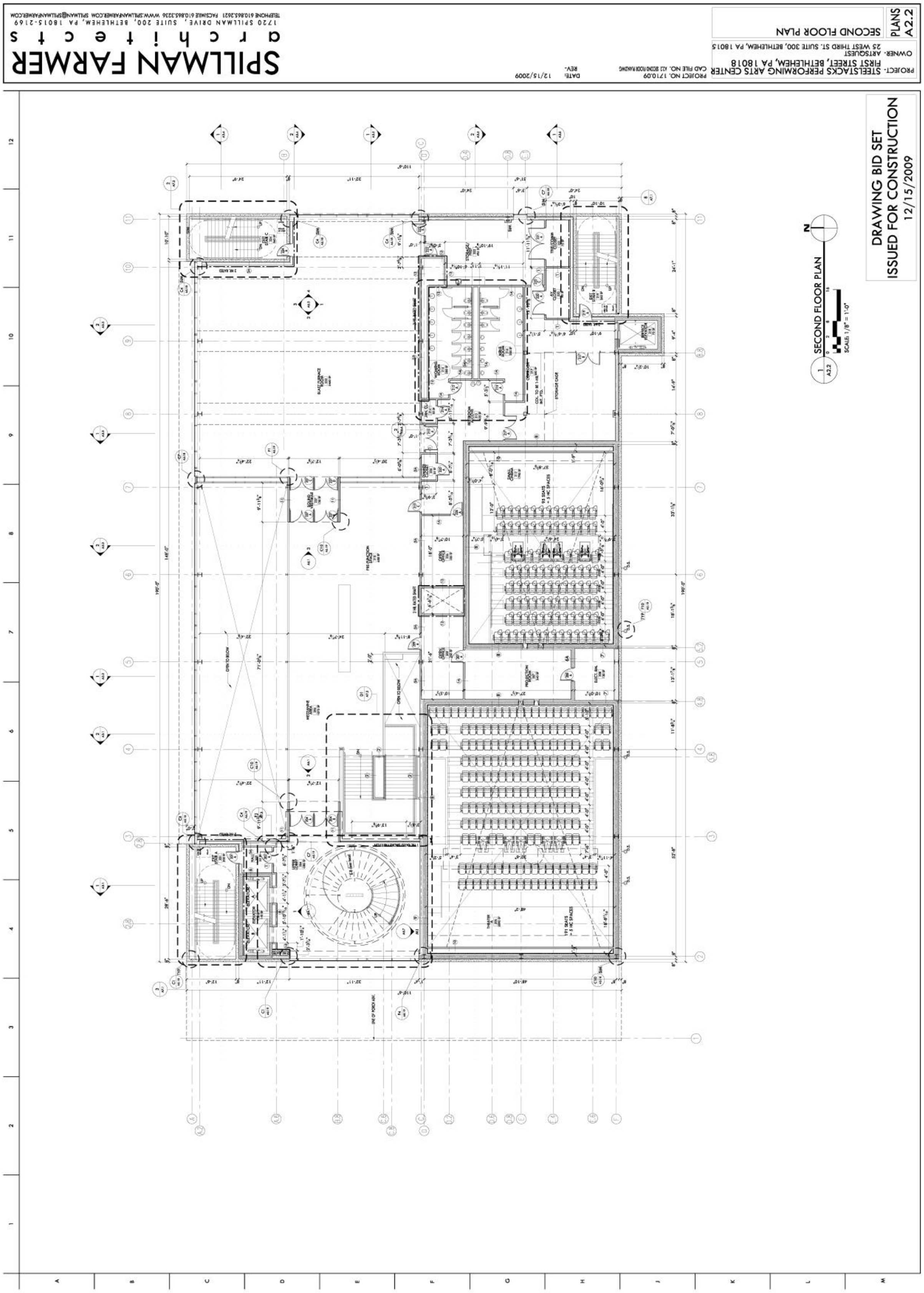
Column Schedule Excerpt

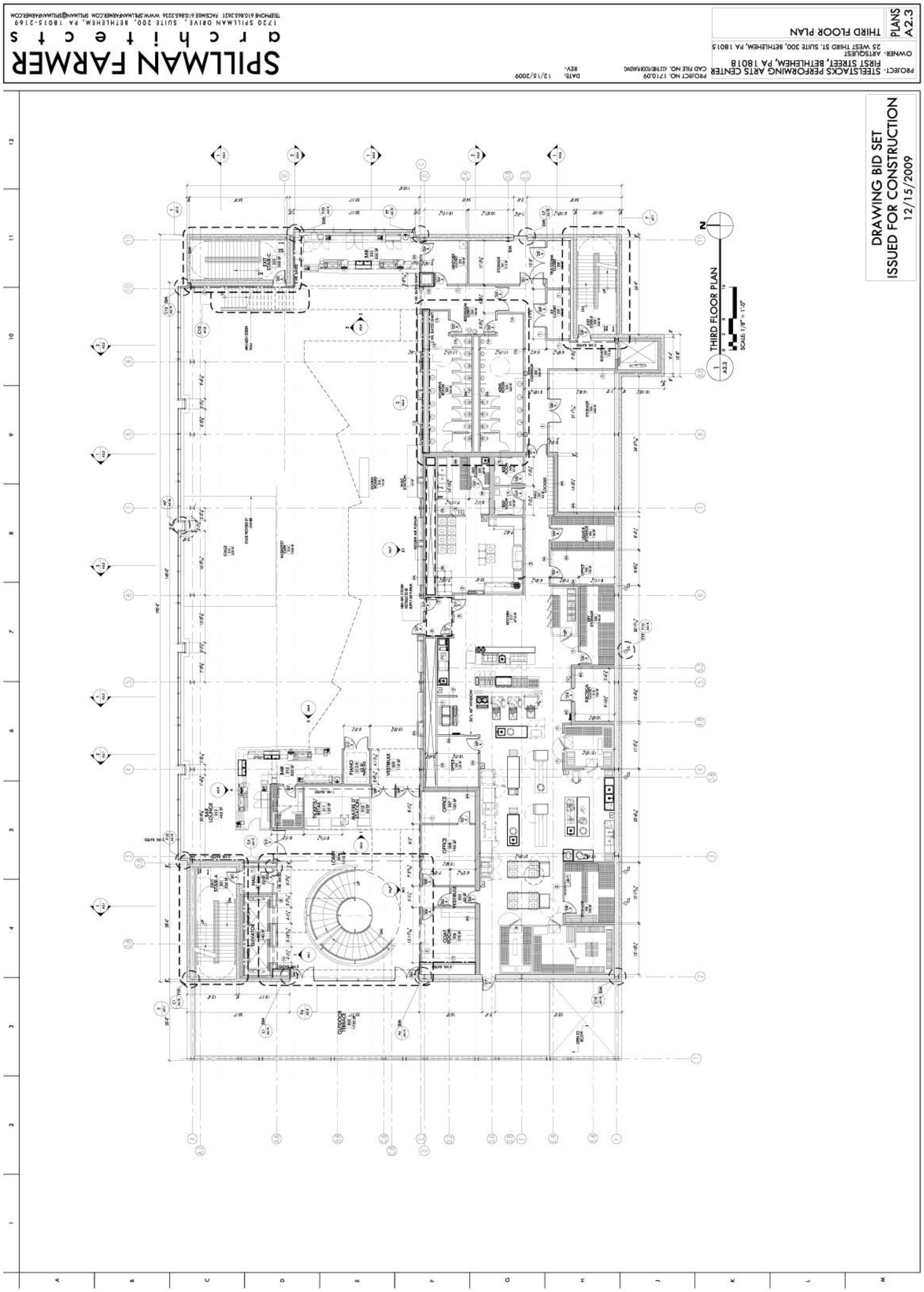
Referenced in the column spot check for column B8.

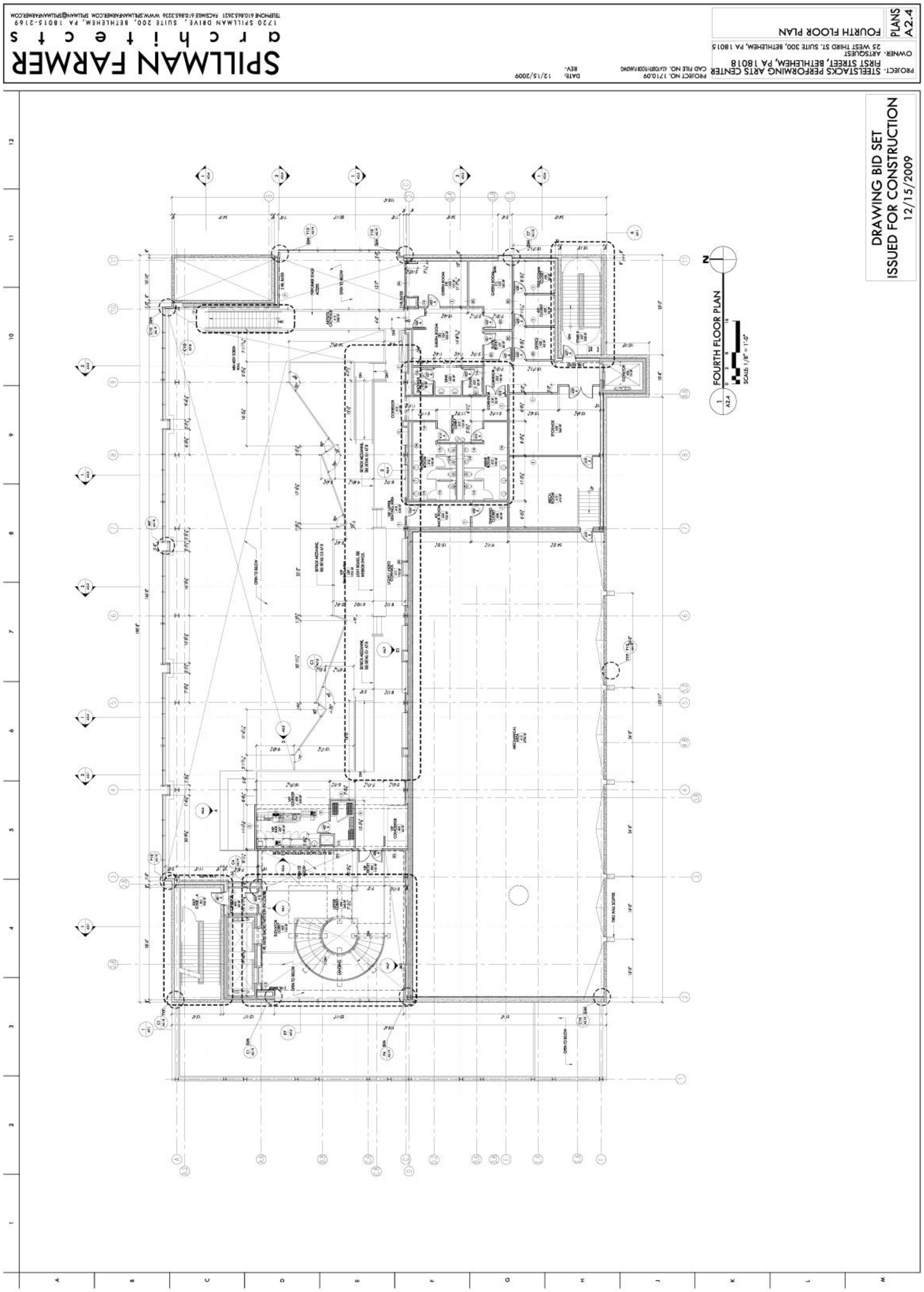


Architectural Plans

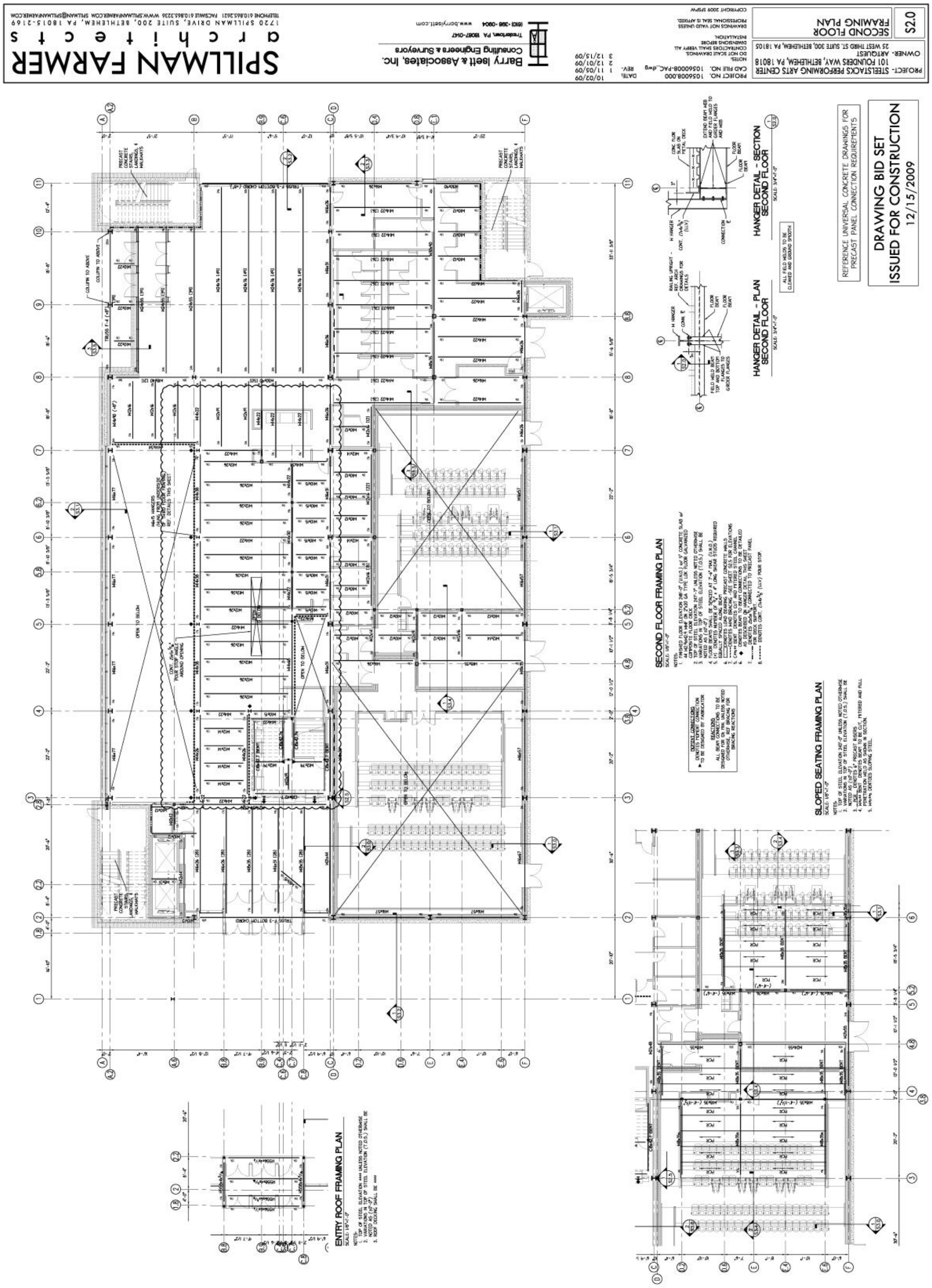



SPILLMAN FARMER
a r c h i t e c t s
SECOND FLOOR PLAN
PLANS
A2.2
DRAWING BID SET
ISSUED FOR CONSTRUCTION
12/15/2009

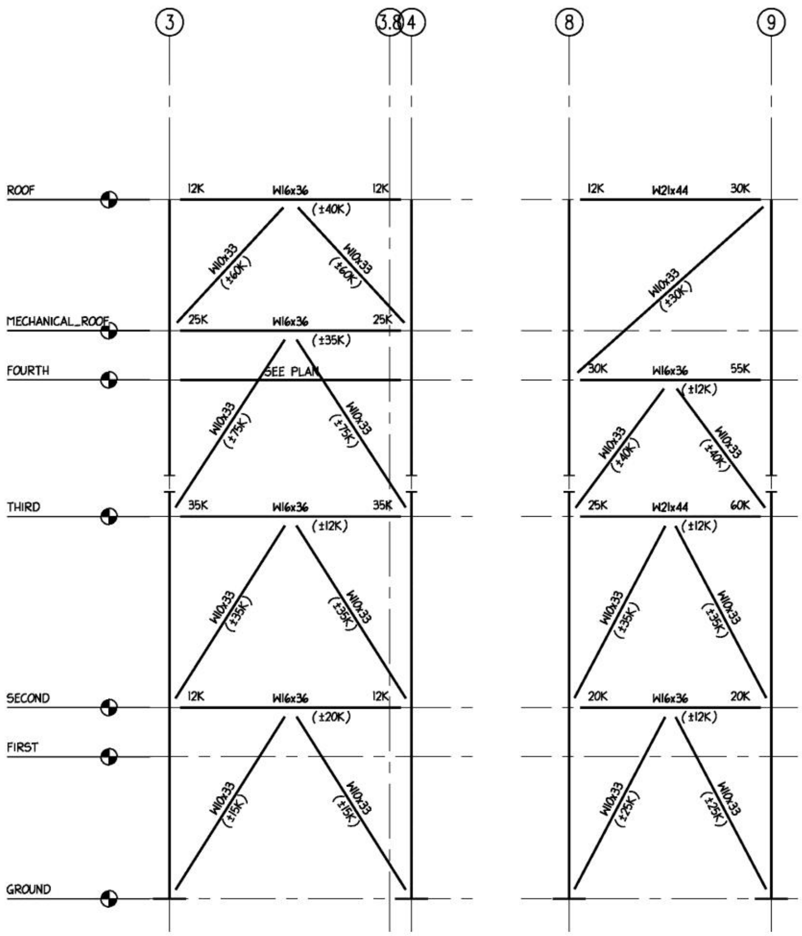

SPILLMAN FARMER
architects
THIRD FLOOR PLAN
PLANS
A2.3
DRAWING BID SET
ISSUED FOR CONSTRUCTION
12/15/2009


SPILLMAN FARMER
a r c h i t e c t s
PLANS
A2.4
FOURTH FLOOR PLAN
PROJECT: STEELSTACKS PERFORMING ARTS CENTER
FIRST STREET, BETHLEHEM, PA 18018
OWNER: ARTSQUEST
DRAWING BID SET
ISSUED FOR CONSTRUCTION
12/15/2009
FOURTH FLOOR PLAN

Structural Floor Plans



Lateral System



1 ELEVATION AT LINE C
SCALE: 1/8"=1'-0"

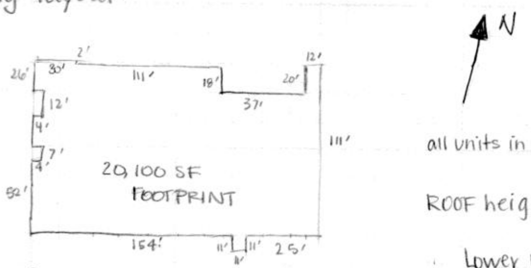
NOTES:
1. CONNECTIONS TO BE DESIGNED FOR FORCES INDICATED BY FABRICATORS ENGINEER.
2. (##k) DENOTES AXIAL FORCE IN MEMBER
(+) TENSION
(-) COMPRESSION
3. ##k DENOTES VERTICAL REACTION ON END OF BEAM

Appendix 2: Wind Load Calculations

Sarah Bednarcik | WIND LOAD calcs | THESIS Sept 2012 | 1

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

Bldg layout:



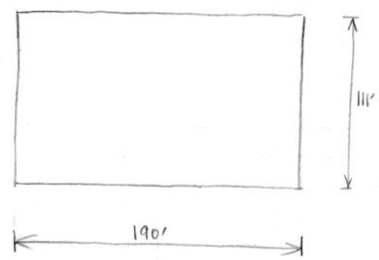
all units in feet

ROOF height = 64'0"

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

∴ mean roof height, h = 64'

Simplified:



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

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

6.5.3 Design (Values) Procedure

1. Wind Speed $V = 90$ mph (Figure 6-1)
Directionality Factor $K_d = .85$ (Table 6-4)

2. Importance factor, $I = 1.15$ (Table 6-1)
Category III (by Table 1-1)

3. Exposure Category:
by Section 6.5.6. – Surface Roughness B. → Exposure B
Velocity Pressure Exp. Coeff (Table 6-3)
Case 2, Exp B.
interpolate $z = 60$ | .85, $z = 70$ | .89 → at $z = 64'$ $K_z = .87$
other K_z values on excel.

4. Topographic factor, K_{zt}
by Section 6.5.7 $K_{zt} = 1.0$

5. Gust effect factor G or G_f
by Section 6.5.8
rigid or flexible.

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

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

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

RIGID

2/2

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

$g_Q = g_v = 3.4$

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

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

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

N-S. $B = 190$ ft

$\therefore Q = .877$

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

E-W $B = 111$ ft

$\therefore Q = .910$

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

6. enclosure - Fully enclosed

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

8. ext. pressure coefs.

Fig 6-6 walls: windward $C_p = .80$

leeward

N/S $L/B = 111/190 = .584$ → $C_p = -.5$

E-W $L/B = 190/111 = 1.71$ → $C_p = -.36$ (interpolated)

Side $C_p = -.70$

roof:

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

Roof Area = 190x111 > 1000 SF
∴ Reduction factor = .8

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

10. MWFRS.
$p = q\,GC_p - q_i(GC_{pi})$

pressure:
$p = q_h G C_p - q_z G C_{pi}$ where $q = q_z$ for windward. All else $q = q_h$.

See excel for further calcs.

	Height (ft)	Kz
Roof	64	0.87
Floor 4	46.5	0.83
Floor 3	35	0.73
Floor 2	17.5	0.6
Floor 1	0	0.57

Appendix 3: Seismic Load Calculations

Sarah Bednarcik | Seismic Calcs | THESIS Sept 2012 | T1

Ch 11: §11.4 Seismic Design Values — USING ASCE7-05 §12.

$S_s = 0.26$
$S_1 = 0.06$
Site Class D
FROM Geotech Report
§11.4.2

$F_a = 1.6$ (Table 11.4-1)
$F_v = 2.4$ (Table 11.4-2)
Ch. 11

$S_{MS} = F_a \cdot S_s = 1.6(0.26) = .416 \rightarrow S_{DS} = \frac{2}{3} S_{MS} = .277$
$S_{M1} = F_v S_1 = 2.4(0.06) = .144 \rightarrow S_{D1} = \frac{2}{3} S_{M1} = .096$

$S_a = S_{DS}\left(0.4 + 0.6\frac{T}{T_0}\right)$

$T_0 = 0.2\, S_{D1}/S_{DS} = 0.2(0.096)/.277 = .0693$
$T_S = S_{D1}/S_{DS} = .096/.277 = .347$

Finding T, fundamental period of bldg: §12.8.2

$T \leq C_u T_a$

$T_a = C_t h_n^x = (0.03)(64')^{.75} = .6788$ — Table 12.8-2

$C_u = 1.7$ (Table 12.8-1)

$\rightarrow T = 1.7(.6788) = 1.15$

*Structure is an eccentrically braced steel frame in E-W direction; shear walls in N-S direction

$\therefore S_a = .277(0.4 + 0.6(1.15/.0693)) = 2.87$

$C_s = \frac{S_{DS}}{(R/I)} = \frac{.277}{(3/1.5)} = .139$

$R = 3$ Table 2.1
$I = 1.5 \rightarrow$ Table 11.5-1 Occ. Cat IV

$T_L = 6$ (Fig 22-15) $\quad T < T_L \therefore C_s = .139 \leq \frac{S_{D1}}{T(R/I)} = \frac{.096}{1.15(2)}$
$\leq .042$

compare $C_s = .139$ to Structural's value of .138 ✓
but can be reduced to $= .042$ by §12.8.1.1 (12.8-3)

Building Weight Calculation:

Seismic Calcs 2

Building weight §12.8.3

Considering: Totals

Floor 2,4 — 5" slab & deck
- Slab 50 PSF
- MEP 5 PSF
- Framing 10 PSF

65 PSF

Floor 3 — 8" slab & deck
- Slab 87.5 PSF
- MEP 5 PSF
- framing 10 PSF

102.5 PSF

Roof — lightwt conc 5"
- Snow 30 PSF
- Slab 40 PSF
- MEP 10 PSF

80 PSF

Other materials:

CMU walls. 8" thick = $(\frac{8}{12} \cdot 137$ PCF$) = 91$ PSF

8" prefab conc panels $\cong \frac{8}{12}(150) = 100$ PSF

Curtain wall system $\cong$ 90 PSF

See excel spreadsheets for further calcs.

Weight of Building	Area	PSF	Load (lbs)	Story Weight (lbs)
CMU	4305	91	391755	
Curtain Wall	2152.5	90	193725	
Concrete Panels	9607.5	100	960750	
Floor 2	12043	65	782795	2329025
CMU	9135	91	831285	
Curtain Wall	2152.5	90	193725	
Concrete Panels	9607.5	100	960750	
Floor 3	21057	102.5	2158343	4144102.5
CMU	5911	91	537901	
Curtain Wall	2300	90	207000	
Concrete Panels	6026	100	602600	
Floor 4	7870	65	511550	1859051
CMU	4515	91	410865	
Curtain Wall	3500	90	315000	
Concrete Panels	8522.5	100	852250	
Roof	14310	80	1144800	2722915
			Total Weight (lbs)	11055093.5
			(k)	11055

Appendix 4: Snow Load Calculations

Sarah Bednarcik SNOW LOAD CALCS THESIS Sept 2012 T1

USING: Flat Roof Snow Loads $\theta < 5°$ ✓

Ground snow load, $p_g = 30$ PSF Fig 7-1, § 7.2

exposure factor, $C_e = 1.0$ Table 7-2 § 7.3

Thermal factor, $C_t = 1.0$ Table 7-3

Importance Factor, $I_e = 1.2$ Table 7-4

$p_{f\,min} = 20\,I = 20(1.2) = 24$ psf

$p_f = 0.7\,C_e C_t I p_g = 0.7(1.0)(1.0)(1.2)(30) = 25.2$ psf

Compare to engineer's conservative value of 30 PSF = p_f

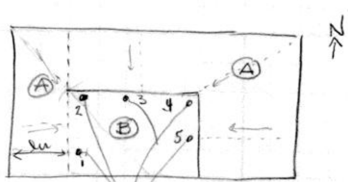
we will continue with 25.2 psf

Due to dif. Roof elevations: § 7.7
Drifts on Lower Roofs

$\gamma = 0.13\,p_g + 14 = .13(30) + 14 = 17.9 < 30$ pcf ✓

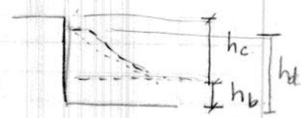
$h_b = \frac{p_s}{\gamma} = \frac{p_f}{\gamma} = \frac{30}{17.9} = 1.68$ ft } height of balanced snow load

snow drift



roof	height
A	64'
B (mechanical roof)	52' 6"

1/4 to 12 roof slope



All these locations (along lip bounding area B) are prone to snow drift.

$h_c = \Delta h - h_b = (64 - 52'6") - 1.68' = 9.82'$

by 7.7.1, if $h_c/h_b < .2$, then no need to design Not so

h_d: Fig 7-9

	leeward	x	windward
$l_{u1} = 20'$	1.5	100.5'	$\boxed{3.3}$ *
$l_{u2} = 62.3'$	$\boxed{2.6}$ *	54'	2.5
$l_{u3} = 91.2'$	$\boxed{3.3}$ *	52'	2.6
$l_{u4} = 59'$	2.6	80'	$\boxed{2.9}$ *
$l_{u5} = 69.5'$	2.7	100.5'	$\boxed{3.3}$ *

SNOW 2/2

these $h_d < h_c$ ∴ $w = 4h_d$ $h_c = 9.821$ = drift ht

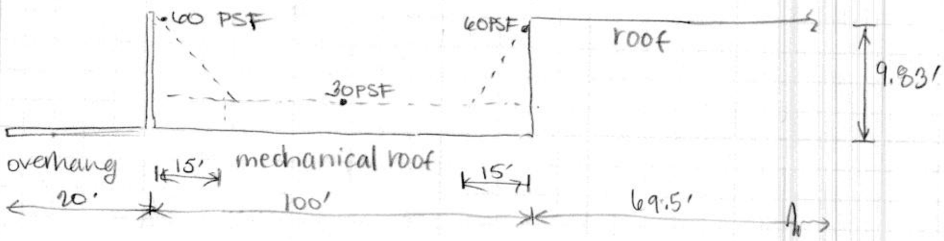
	h_d	w(ft)	p_d (psf)
1	3.3	13.2	59
2	2.6	10.4	47
3	3.3	13.2	59
4	2.9	11.6	52
5	3.3	13.2	59

$P_d = h_d \gamma$

+ 30 psf (snow, balanced)

considering mechanical roof, surrounded by higher roof and parapet:

$P_d = 60$ PSF, $P_b = 30$ PSF $W_{used} = 15'$



E-W Section
(N-S Similar)

∴ Will use 30 PSF except around 15' perimeter, where 50 PSF will be used, due to snow drift values above.

Appendix 5: Rain Load Calculations

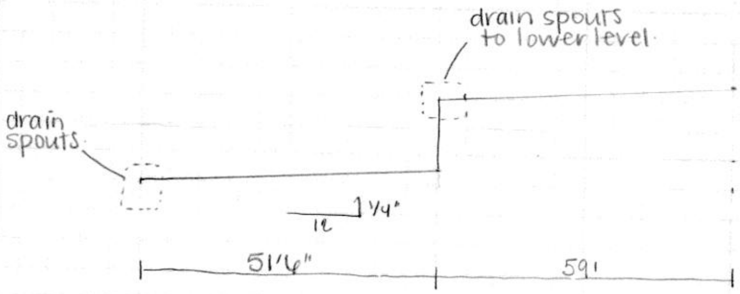
S Bednarcik   Rain calcs   THESIS   Sept 2012   11

Following ch 8 ASCE 7-05:

Considered for later application in Thesis report:

$R\ (\text{rain load}) = 5.2\,(d_s + d_h)$

Section of roof:



Assuming bottom inlet blocks:

If primary blocks:

d_s = .84' = 10"

If secondary blocks:

d_h = 1.23' = 14.75"

$\therefore R = 5.2\,(.84 + 1.23')$
$= 10.9$ PSF = $\boxed{11\text{ PSF}}$

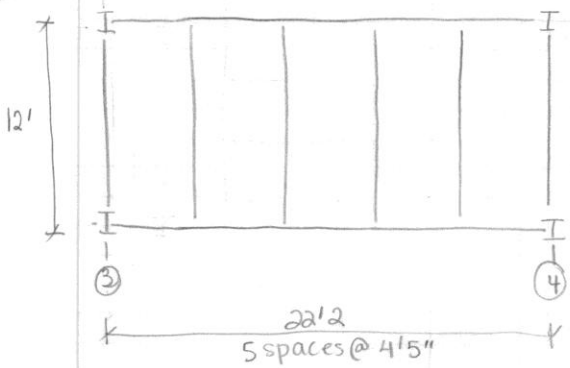
8.4 Ponding doesn't need to be considered

Slope is not less than 1/4 : 12

Appendix 6: Spot Check Calculations

Sarah Bednarcik SPOT CHECKS THESIS 11 1

LOOKING AT SECOND FLOOR FRAMING:



Floor System:
Composite steel floor deck

BAY LOCATED ON NORTH WEST QUADRANT OF 2nd FLOOR

- FIREPROOFING 1HR
- 3" CONC SLAB & 2" - 20 GA TYPE LOK FLOOR, COMPOSITE SLAB
- (2) Layers 44-W2.9xW2.9
- TOTAL THICKNESS = 5"

LOAD: Dead - MEP 5 PSF
- FRAMING 10 PSF
- SUPERIMP. 10 PSF

LIVE: - MEZZANINE 100 PSF (PER ASCE7-05)
- PARTITIONS 20 PSF

As per plans, max spacing allowed = 7'6". This is therefore the controlling spacing for the decking and will be used.

∴ Superimposed live load ⇒ 222 PSF

from 2VLI Vulcraft Superimposed @ 7'6 = 249 PSF > 222 ✓ok

Check 3 span, unshored : 10'2 ✓okay.

∴ $\boxed{\text{2VLI 20 (Vulcraft) should be used.}}$

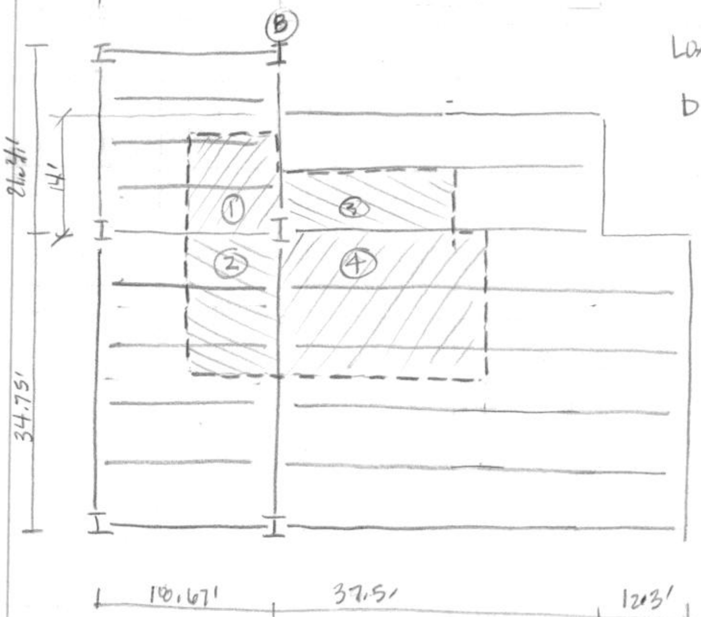
OTHER INFORMATION FOR FURTHER ANALYSIS:

SLAB = 3" slab $\left(\frac{1}{12''} \times 150 \text{ PCF}\right)$ ⇒ use. $\frac{4'' \text{ slab}}{12''}(150)$ = 48.3 PSF

SPOT CHECKS 2 TI

COLUMN:

LOCATED ON 2nd Floor Plan, at Column Line B8



LOADS: (PSF)

D = MEP 5
FRAME 10
SUPER. 10
SLAB 48
73 DL

L Mechanical Space 100
Partitions 20
= 120 LL

$W_u = 1.2(73) + 1.6(120)$
$= 87.6 + 192$
$= 279.6$ PSF

COLUMN CARRIES 2nd floor only.
Column height = 17' 6"

Trib Area: $TA_1 + TA_2 + TA_3 + TA_4$
$= \left(\frac{21.4}{2}\right)\left(\frac{18.67}{2}\right) + \left(\frac{34.75}{2}\right)\left(\frac{18.67}{2}\right) + \left(\frac{37.5}{2}\right)\left(\frac{14'}{2}\right) + \left(\frac{37.5+12.3}{2}\right)\left(\frac{34.75}{2}\right)$
$= 826$ SF

For Axial Compression

USING AISC 14th Table 4-1

USING K=1
$\therefore KL = 17.5' \cong 18'$

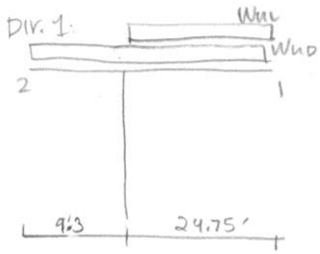
$P_u = 279.6$ PSF $\cdot$ 826 SF $= 231$ k.

$\phi P_n > P_u$
W10x45 $\phi P_n = 257 > 231$ ✓ ok

could also use:
W12x40 $\phi P_n = 235 > 231$ ✓ ok MORE ECONOMICAL?

CHECK Moment - Axial Interaction

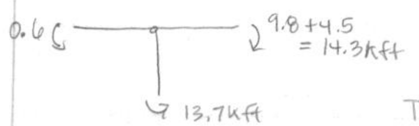
Assuming Fixed-end Moments



$M_{uL1} = \frac{W_{uL} l^2}{12} = \frac{192(24.75)^2}{12} = 9.8$ k ft

$M_{uD1} = 4.5$ k ft
$M_{uD2} = 0.6$ k ft

3 TI



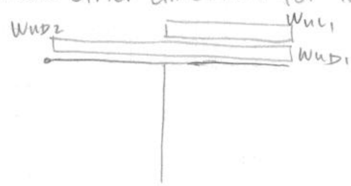
by $P_{eq} \geq P_u + 24\frac{M_u}{d}$

Try W10x45 → interpolate for KL = 17.5'

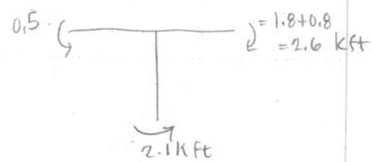
$P_u + 24\frac{M_u}{d} = 231 + 24\frac{(13.7)}{10} = 263.9 \text{ kft} < 269.5 \text{ kft}$

$\boxed{\text{Use a W10x45}}$

Check other direction for Moment:



$W_{uL1} = 1.8$ kft
$W_{uD1} = 0.8$ kft
$W_{uD2} = 0.5$ kft

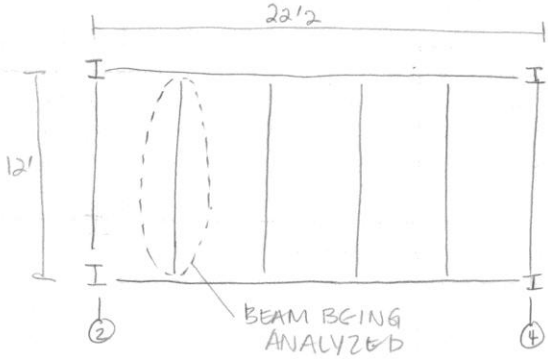


Check:
$\phi P_n = 269.5 \text{ kft} > 236 \text{ kft} = 231 + 24\frac{(2.1)}{10}$

✓ ok.

SPOT CHECKS 4 T1

USING SAME BAY AS IN STEEL DECK ANALYSIS



FOR THIS ANALYSIS, MAX SPACING OF 7'6" USED

LOAD: D: MEP 5 (PSF)
FRAMING 10
SUPERIMP 10
SLAB 48 | 73 PSF
L: MEZZANINE 100
PARTITIONS 20 | 120 PSF

LIVE LOAD REDUCTION DOES NOT APPLY

$\therefore W_u = 1.2(73) + 1.6(120) = 279.6$ PSF
$W_u = 279.7 * 7.5' = 2100$ PLF

checks with what member was designed for

$V_u = wl/2 = 2100 \cdot 12/2 = 12600 \cong 13k$
$M_u = wl^2/8 = 2100 \cdot 12^2/8 = 37800$ lbft $= 38$ kft

USING Zx Tables = $M_u < \phi M_{px}$.

considering economy.

W10x12 $\phi M_{px} = 46.9$ k ft > 38 K ft ✓ok

W12x14 $\phi M_{px} = 65.3$ K ft > ✓could also use

Check Shear $V_u \leq \phi V_{nx}$

W10x12 $\phi V_{nx} = 56.3$ k > 13k

W12x14 $\phi V_{nx} = 64.3$ k > 13k ✓ok.

W10x12
$I_x = 53.8$ $W_L = 1.6(120)(7.5') = 1.440$ KLF

W12x14
$I_x = 88.6$

5 TI

Check deflection $\Delta_{LL} \leq L/360$ $\Delta_{TL} \leq L/240$ for W10x12

LL: $\Delta_{LL} = \frac{5wl^4}{384EI} = \frac{5(1.4)(12^4)1728}{384(29000)(53.8)} = .25''$

compare $\Delta_{LL} = .25'' < 0.4'' = \frac{12 \times 12}{360}$ ✓ok

TL: $\Delta_{TL} = \frac{5(2.1)(12^4)(1728)}{384(29000)53.8} = .38''$

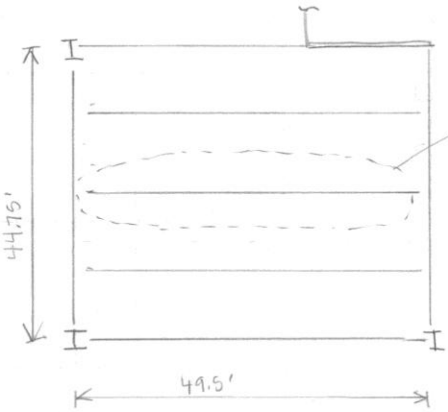
compare $\Delta_{TL} = .38'' < .6'' = \frac{12 \times 12}{240}$ ✓ ok

$\boxed{\text{use a W10x12 for the beam}}$

S Bednarcik SPOT CHECKS T2 1.

COMPOSITE BM SPOT CHECK:

USING 2nd Floor Bay in N-E Quadrant



Beam being evaluated.

Spaced at 7'6" = 7.5'

LL not reducible by ASCE 7-05 §4.7.

* EXISTING USES A W24x76 [49]

LOAD: D MEP 5 (PSF)
FRAMING 10
SUPERIMP. 10
SLAB 48 — 73 PSF DL
L VARYING USE 125 | 125 PSF LL

$W_L = 1.6(125)(7.5') = 1.5$ KLF
$W_u = 1.2(73) + 1.6(125) = 287.6 * 7'6 = 2.2$ kLF

$M_u = wl^2/8 = 674$ k·ft

use 3/4" studs → Table 3-21
k/ bolt strength = 17.2 k/stud =: Q_n

Table 3-19 AISC

$b_{eff} = \min \begin{cases} \text{span}/8 = 49.5 \times 12/8 = 74" \\ 1/2(7'6) \times 12 = 45 \end{cases} \rightarrow b_{eff} = 90"$

assume $a = 1"$ → $Y_2 = 5 - a/2 = 4.5"$

BM OPTIONS:

W21x44	$\phi M_n = 699 > 674$	$\Sigma Q_n = 577$	$n = 577/17.2 = 34 \rightarrow 68$ studs
W21x48	$\phi M_n = 691 >$ ✓	442	$n = \; = 26 \rightarrow 52$ studs
W21x50	696 > ✓	386	$n = \; = 23 \rightarrow 46$ studs

BM EQUIV. WT.

1	Wt	
W21x44 [68]	= 44(49.5) + 68(10) = 2858	← best, try this
W21x48 [52]	= 48 " + 52(10) = 2896	
W21x50 [46]	= 50 " + 46(10) = 2935	

S Bednarcik | S checks | TR | 2

$a = \frac{\Sigma Q_n}{.85 f'_c b_{eff}} = \frac{17.2(68/2)}{.85(4)(90)} = 2.5 > 1.0$ NG.

allowed 1 stud/ft.

49 studs → $17.2(49) = 842.8/2 = 421.4$

Solve for ΣQ_n.

$\Sigma Q_n = (1)(.85)(4)(90) = 306$ → go with W24x76
$\Sigma Q_n = 280$
$n = 34$ studs

req'd per deflection

$I_{LB} = \frac{5(.94)(49.5)^4(1728)}{384 \; 29000 (49.5 \times 12/360)} = 2654 \text{ in}^4$ req'd (.94 unfactored) using $\ell/360$ for LL unfactored.

check: $\frac{5(.94)(49.5)^4 1728}{384(29000)(3310)} = 1.3'' < 1.65'' = \frac{\ell}{360}$ ✓ok LL defl. check.

check: $a = \frac{280}{.85(4)90} = .92 < 1.0$ ✓ok

for a W24x76 ∴ $Y_2 = 5 - .92/2 = 4.54$ → use 4.5

$I_{LB} = 3310 > 2654$ ✓ok

good by deflection.

Check unshored strength
$\phi M_p = 750$ W24x76

$W_u = 1.4(76) + 1.4(63)(7.5) = .77$ KLF
$W_u = 1.2(76 + 63(7.5)) + 1.6(20 \times 7.5) = .88$ KLF ← controls.

$M_u = \frac{(.88)(49.5)^2}{8} = 269.5 < 750$ ok for unshored

Wet conc defl. ck.

$W_{wc} = 63(7.5) + 76 = .549$ KLF

$\delta_{wc} = \frac{5}{384}\frac{(.549)(49.5)^4 1728}{29000 \cdot 3310} = .77'' < 2.5'' = \frac{\ell}{240}$ ✓ok

$\delta_{tot} = \frac{5}{384}\frac{(2.2)(49.5)^4 1728}{29000 \cdot 3310} = 3.1''$ total deflection

$\boxed{\text{ok W24x76 at 34 studs}}$