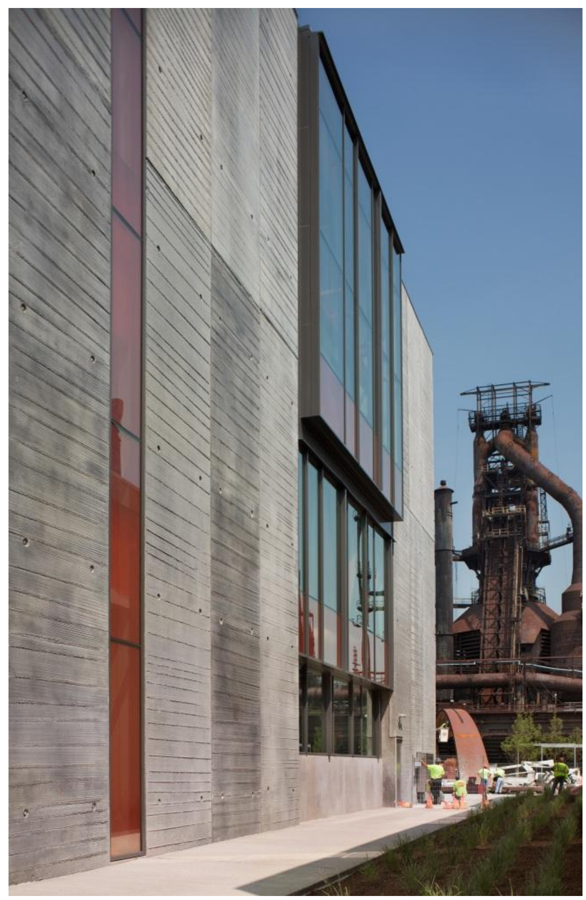
SteelStacks Performing Arts Center | Bethlehem, Pennsylvania

Technical Report II

Structural Study: Alternate Floor Systems



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Executive Summary

The purpose of this report is to complete a comparative analysis of the existing floor framing system against three alternative framing systems for a specific bay. The existing bay, a composite slab and deck on beam system, was compared against the systems listed below.

- Precast concrete planks on beam
- Post-tensioned two-way flat plate
- One-way slab on beam

These systems were evaluated in consideration of the structure, architecture, construction, and serviceability of each design. Evaluations of each system are presented in this report, with a comparative summary succeeding the individual system analyses.

The post-tensioned two-way flat plate system was not considered a viable redesign solution. The disadvantages of this system that resulted in it not being feasible included large moment due to span, inconsistent bay arrangements, and slab depth.

Other alternative systems were deemed viable possibilities for redesign. While the precast plank alternative had high costs and higher deflection, advantages included constructability, slab depth, and architectural impact. The one-way slab alternative is a comparable-weight system to the existing, with advantages in depth, constructability, deflection, and noise isolation.

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

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Purpose

The purpose of this technical report is to consider a typical bay of the framing system, as designed by the professional engineers designing the SteelStacks Performing Arts Center (SSPAC). This system was then reconsidered in three alternative flooring systems and in a comparative analysis discussed for potential further design. A structural system overview, as well as general load summaries, has been included for a better understanding of the system preceding the floor system analysis.

Introduction

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

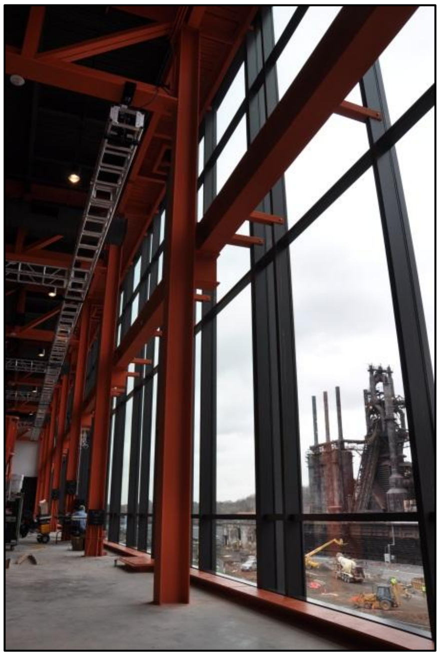


Figure 1: Interior atrium space, highlighting opening structural plan.

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

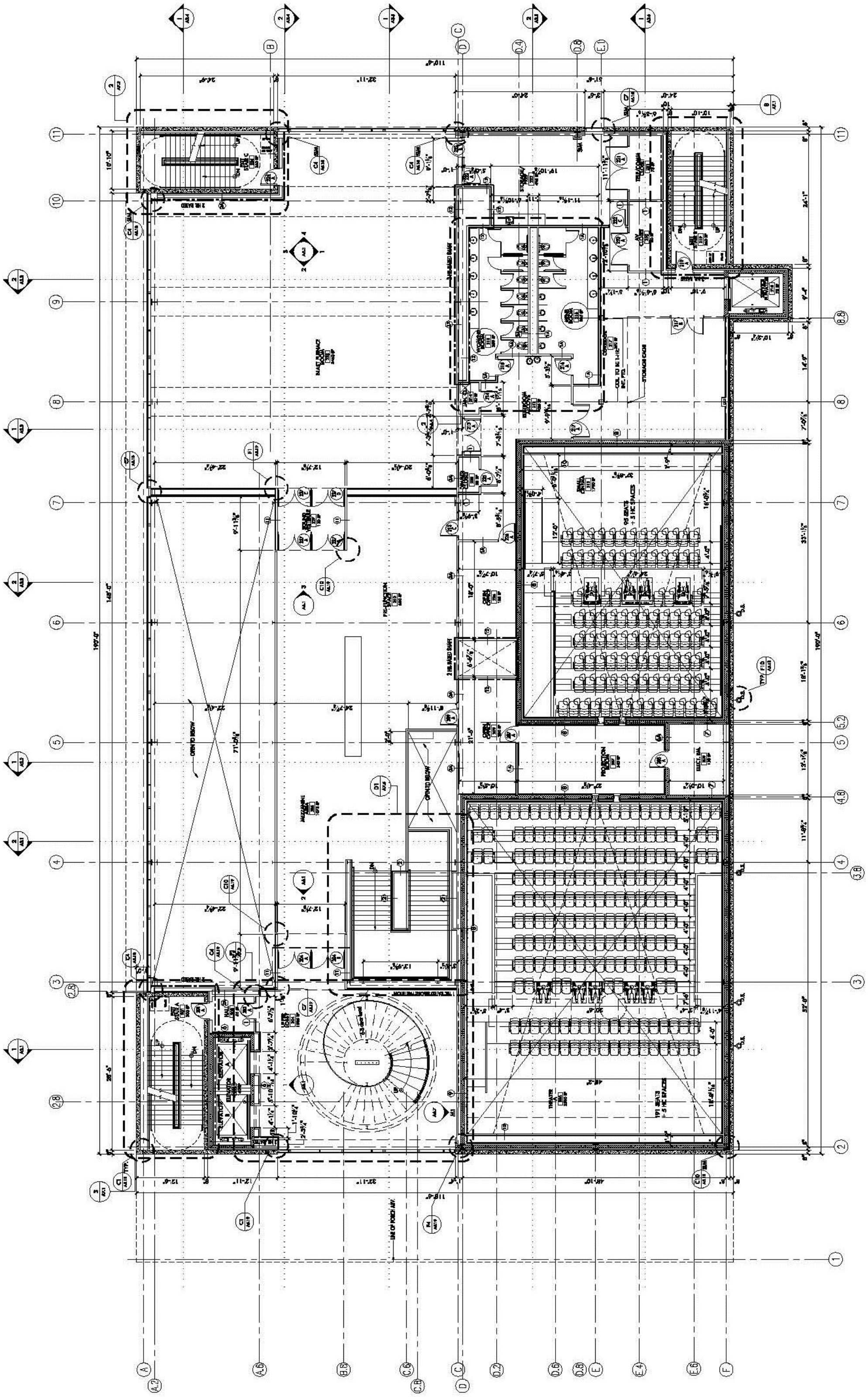


Figure 2: Floor Plan from A2.2

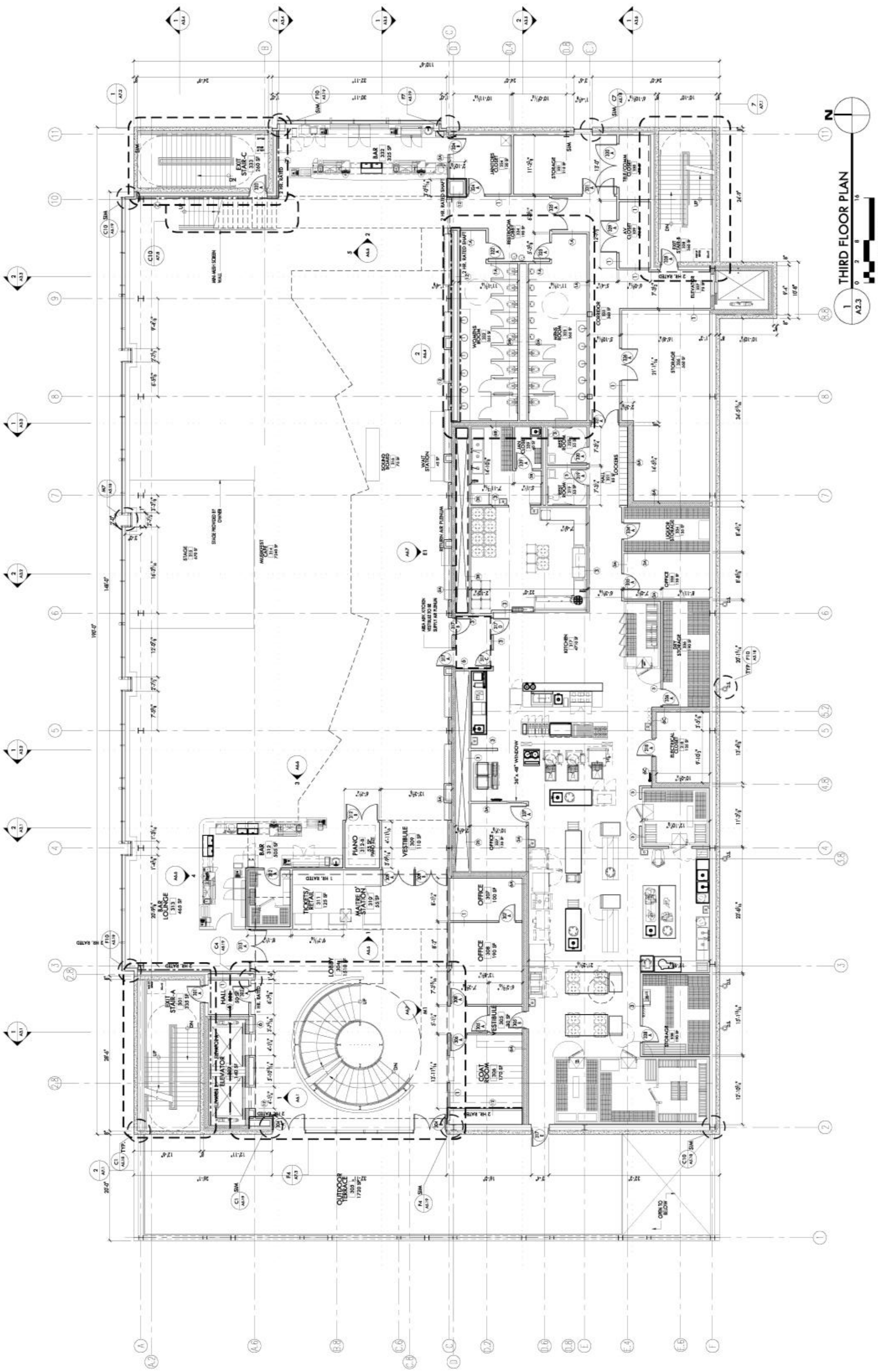


Figure 3: Third Floor Plan from A2.3

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

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

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

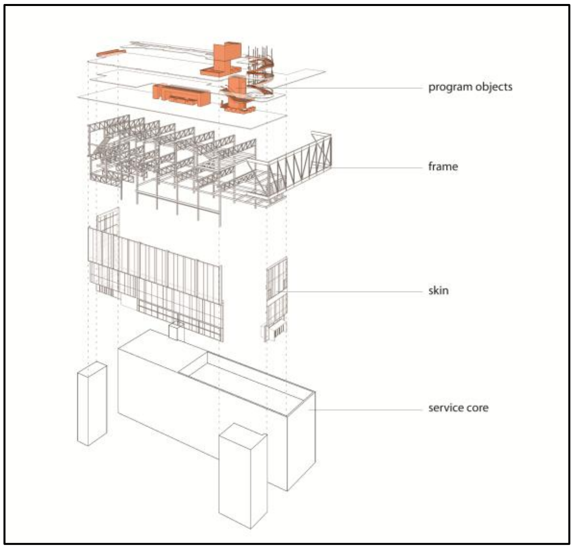


Figure 4 : Image displaying the separation of spaces through the structural design.

Courtesy of Barry Isett, Inc. & Assoc.

General Structural Information

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

Structural System Overview

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

Foundation

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

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

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

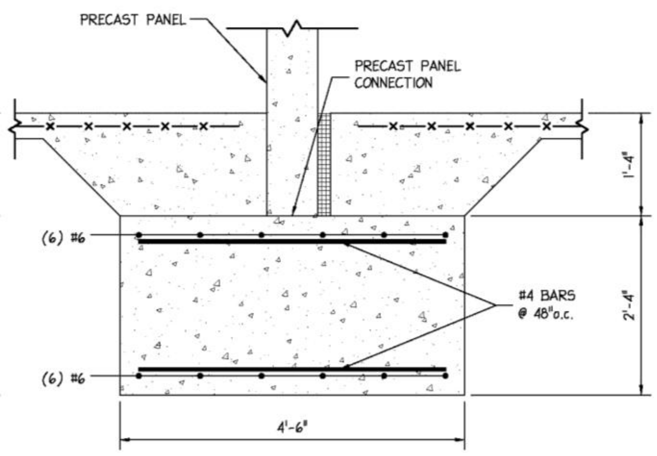


Figure 5 : Section of foundation to precast panel connection from S1.0.

Floor System

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

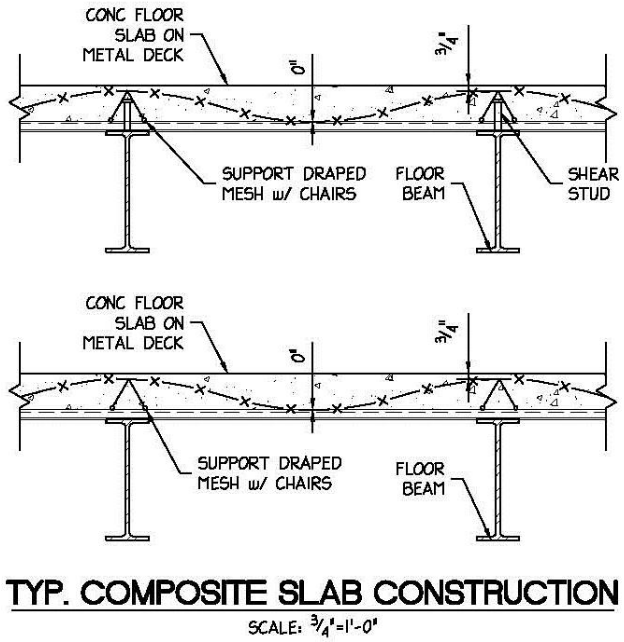


Figure 6 : Typical composite slab section for building from S2.8

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

Framing System

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

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 7. Structural framing plans for referenced floors are in Appendix 1. This bay is taken from the second floor, which uses the most consistent flooring and framing seen in other portions of the building and on the fourth floor and roofing plans.

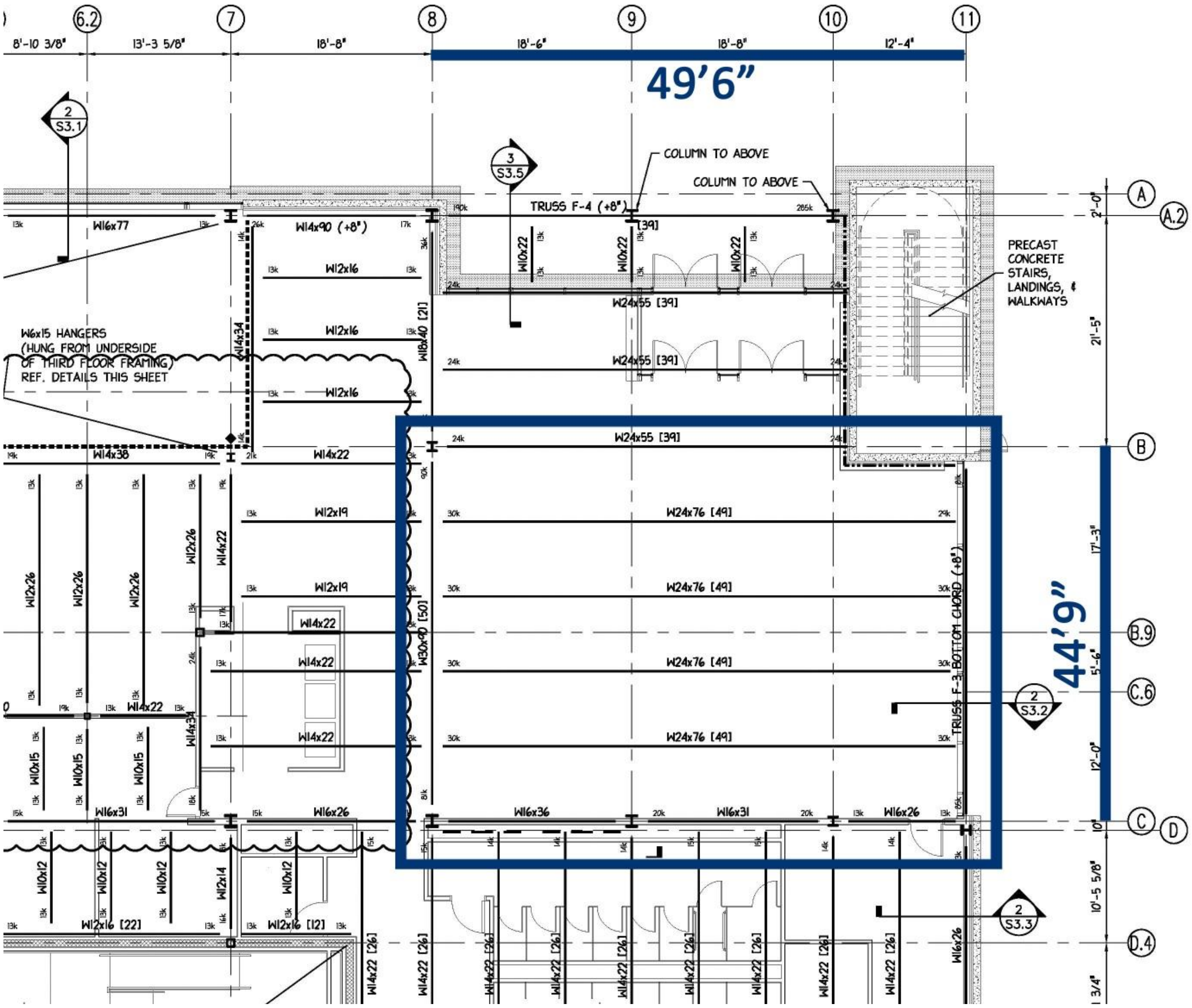


Figure 7 : Second floor framing plan, with a representative bay of a typical frame, highlighted in blue, from S2.0

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

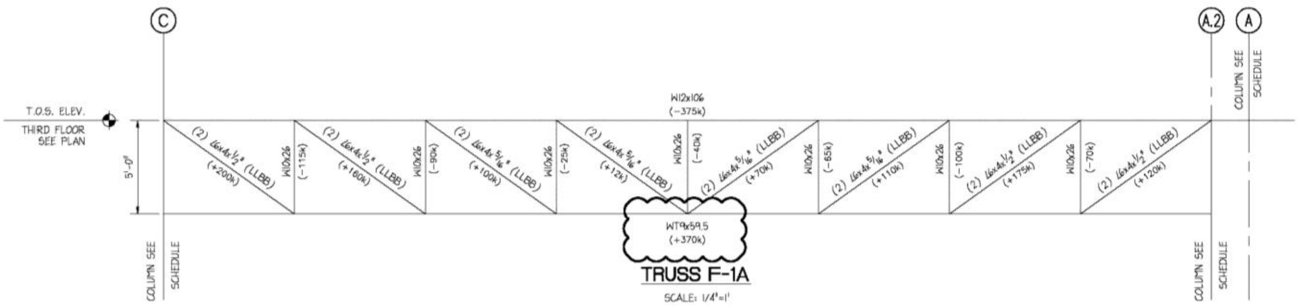


Figure 8 : Third floor representative framing system truss from S2.6.

Framing on the fourth floor is more irregular, as explained previously and included in Appendix 1, due to a large portion of the space open to the third floor, and approximately 25% of the square area excluded due to the mechanical roof. Yet even with the irregular framing plan, the beams are mostly W12x14 for public space, restroom facilities, and storage spaces and W18x35s supporting the green rooms and offices. The mechanical roof has typical framing members of W27x84s supported by Truss R-2, in a similar layout to that of Truss F-1A in Figure 8.

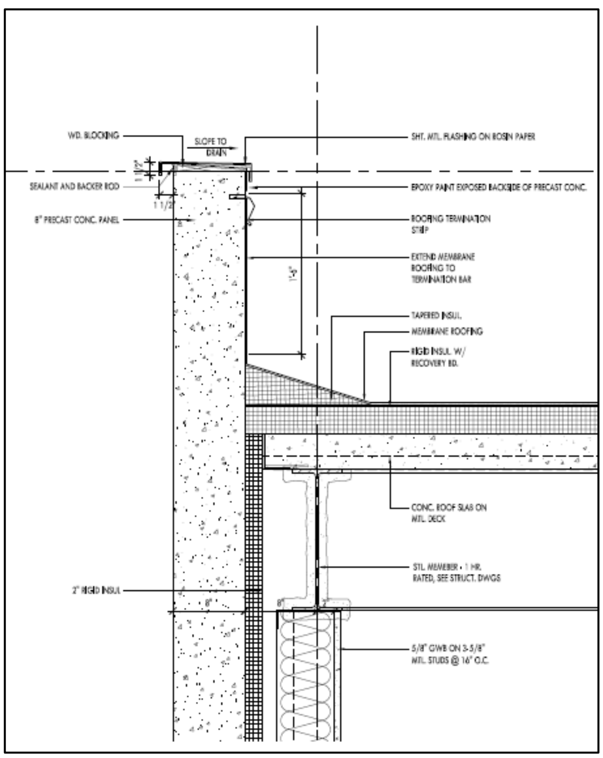


Figure 9 : Cross section of the roofing system.

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 systems is included in Appendix 1.

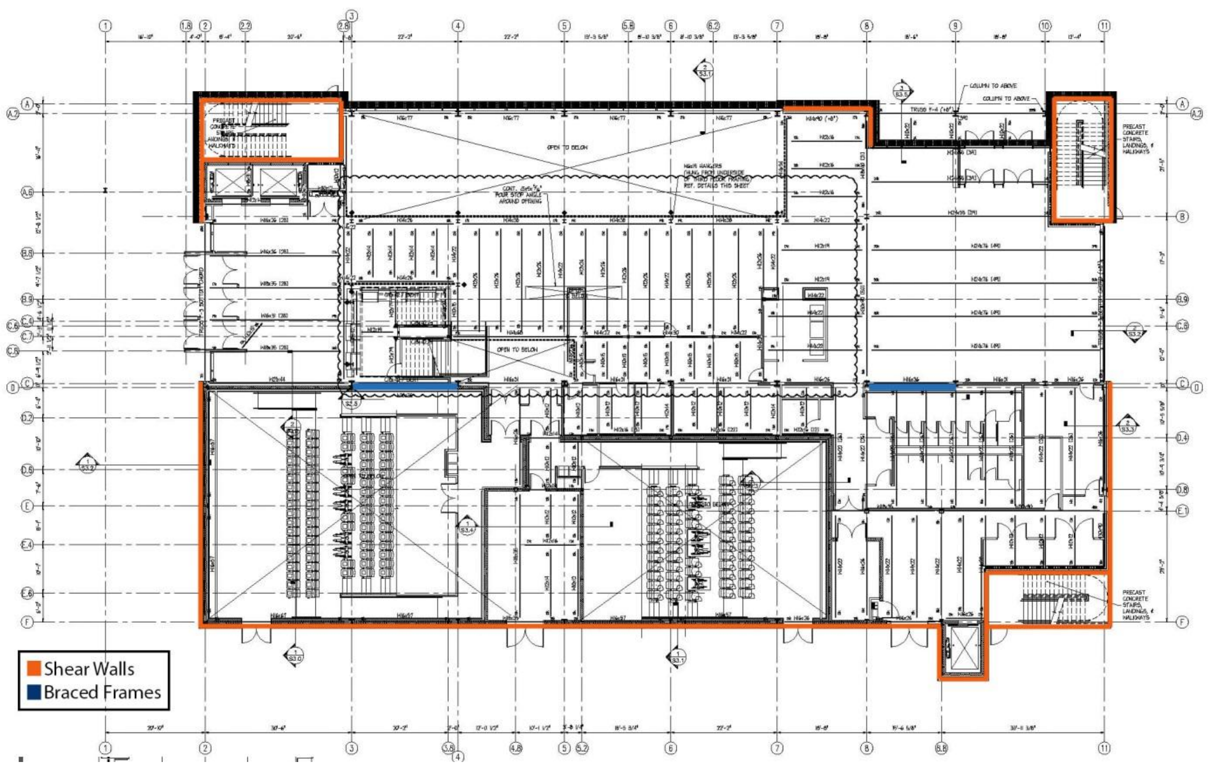


Figure 10 : Floor plan highlighting shear walls in orange and braced frames in blue, which contribute to the lateral system.

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.

Snow Loads

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

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

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

Table 13 : Summary of snow load variables.

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

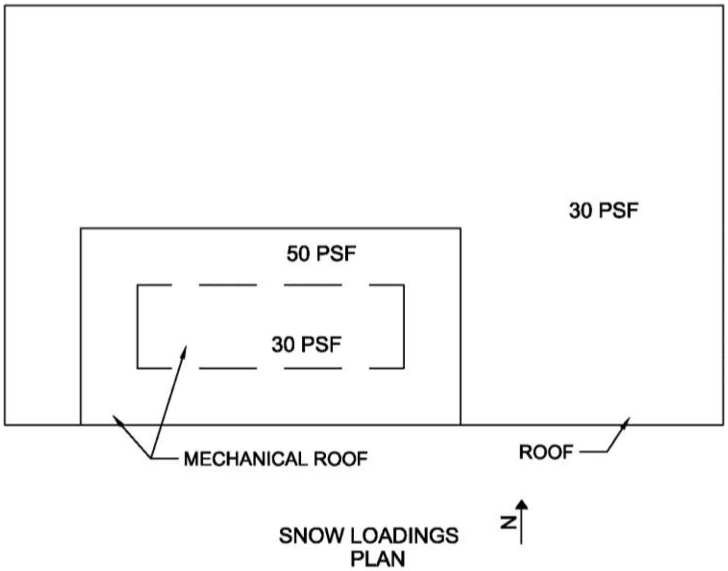


Figure 14 : Summary of snow loads.

Rain Loads

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

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

Floor System Analysis

The primary purpose of this report is to analyze the existing composite beam system of a second floor bay, as well as three alternative systems. These four systems are then compared through structure, constructability, serviceability, and architecture, as elaborated on in the chart following the descriptions and analyses of all four systems.

All four analyses considered the same interior bay on the second floor, spanning column lines B and C in the North-South direction and 8 and 11 in the East-West direction. As mentioned previously, the bay sizes are inconsistent throughout the building, as they are adjusted depending on the space purposes. This bay is an average one that spans a 49’6” by 44’9” space, and was adjusted according to the requirements of each system. These alternate systems are:

- Composite decking on beams (Existing)
- Precast concrete plank on beam
- Post tensioned two way flat plate
- One-way slab on beam

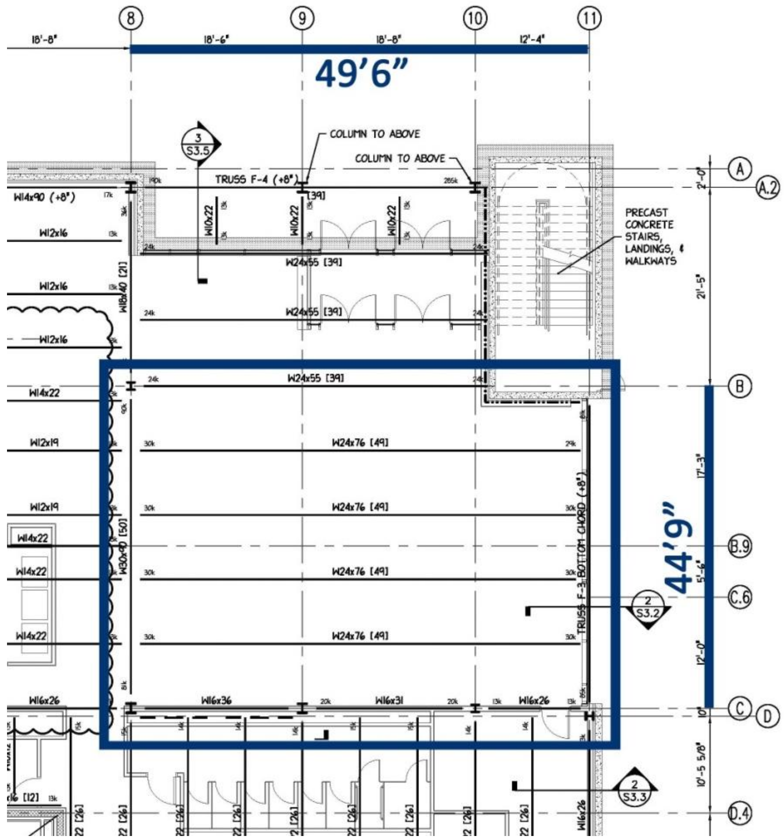


Figure 15: Bay from second floor used for analysis. Taken from S2.0.

Live load reduction was not considered in any of the framing system designs, as per ASCE 7-05 Section 4.8.4, the SSPAC is considered a public assembly space, and therefore live loads are not to be reduced. Fire rating for floor and structural framing requirements is at a one-hour fire rating, with the inclusion of a sprinkler system throughout the building, as per drawing CS-1.

Existing Framing System

The existing framing system has been evaluated through the selection of a representative bay from the second floor. Hand calculations were performed as a verification of this system, as elaborated on in Technical Report I. The results of the spot checks relevant to the purpose of this report are shown in Appendix 2.

Composite Slab & Decking on Composite Beams

The existing framing plan of the bay under consideration consists of a 5" 2VLI20 composite deck, designed using the Vulcraft Steel Decking Catalog meeting the three-span requirement. The decking is supported by W24x76 [49] beams at a maximum spacing of 7'6". The girders supporting these are W30x90s. Figure 16 shows the representative bay used for this comparative analysis.

General

This system has a slab depth of 5" and an overall floor depth of 2.9 feet (35"). Using this system as the baseline for comparison, the floor system weight is at 63.5 psf, and the cost is at $17.93/SF. This is the lightest system, and also is one of the least expensive systems. Cost breakdowns, using RS Means Building Construction Cost Data, can be seen in Appendix 6.

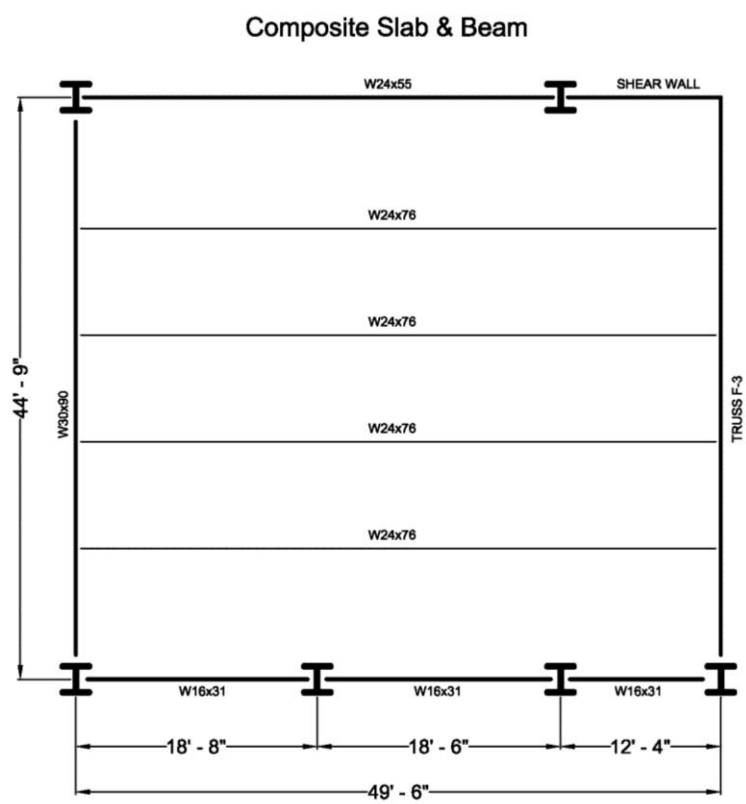


Figure 16: Layout of existing composite slab and beam system.

Architectural

Though this system is the existing, and therefore does not change the architecture, it can be noted that this has thin flooring, at 5" for total deck and slab, with larger spans and incorporates an aesthetic style similar to the surrounding steel mill buildings by using both trusses and beams.

Structural

The use of a composite steel system is beneficial towards the structure, as it is a lighter system that can use braced frames and shear walls for lateral loading. Considering the use of braced frames, connections can be less expensive, as moment connections are not required. This system also has minimal impact on the foundations. Column sizing is very flexible and can be adjusted to weight. This is more cost-effective, and maintains consistently sized members.

Construction

In light of construction, this system does not require highly skilled labor, and in this sense, is an inexpensive alternative. This method also requires less intense coordination between MEP and structural systems, as composite steel can easily leave more room for mechanical system. With this in mind, long lead times are not necessary, and the construction time for this portion of the system is short.

The metal decking, though unshored, does require curing time, as most other systems being considered. This system also necessitates fireproofing of all steel members, and this imbues both cost and time on the project.

Serviceability

This system has a larger deflection issue due to the large and variable spans, 0.77”, necessitating the use of studs and stronger members to eliminate this serviceability issue. Vibration control is also a hesitation this system brings, and the lack of density of the materials for the floor system does not help to dissipate vibration and noise issues very readily.

Conclusion

With advantages such as light weight, inexpensive cost, and ease of construction, it is easy to understand why this system was chosen for the SSPAC, even though this system could more easily have issues with noise isolation.

Alternative Floor Systems

This section details the three alternate framing systems considered for the chosen representative bay. Each system was chosen as an alternative design for potential benefits is terms of constructability, floor depth, and serviceability (deflection control). Throughout design, issues and benefits to each system were evaluated and are dialogued in terms of architecture, structure, construction, and serviceability.

Beyond live load reductions not being considered, acoustic controls on the systems were not considered a controlling design factor. As a rough design for each system, this report is a precursor to further redesign considerations, with more in-depth analyses being completed for the final redesign in future reports.

System 1: Precast Concrete Plank and Beam System

Hollow core planks on both steel and concrete framing were considered as the first alternative system. This was done to have a more thorough understanding of the impact on the floor depth and serviceability of the structure due to steel versus precast beams and girders. Using Nitterhouse catalogs for design, this precast concrete slab system is a series of 4' wide prestressed planks. The hollow core planks were chosen as a lighter slab system, and designs resulted in 10" thick hollow core planks, including a 2" topping, at 1 hour fireproofing as required in the Architectural Plans. Spans for these planks were considered for various configurations, but the use of two interior beams was deemed most advantageous, due to deflection and strength issues of these precast planks. To see the Nitterhouse table used, see Appendix 3. As two alterations on this system have been considered, they are elaborated more below. The design calculations for these two systems are included in Appendix 3 of this report.

A: Precast Concrete Plank on Steel Beams

Figure 17 displays the resulting layout for precast with steel beams and girders. Steel beams and girders were considered as the usual pairing with precast concrete slabs. Both beams and girders were designed as W33x130s, framing into the existing columns lines.

General

With a floor depth of 8", this system has an overall system depth of 3.6 feet (43"). Compared to the original floor system, precast on steel is fairly close, at 88.9 psf. The cost though, is higher, at $20.44/SF. This system is a fairly average system in this respect. See the cost breakdowns in Appendix 6.

Architectural

This system uses a bay size consistent with that of the existing system, with other bays in the system needing minor column line adjustments for the hollow core planks at 4 feet wide to fit bays. This alternative would also maintain similar aesthetics to the existing building, and would not make a huge impact on the architecture.

Structural

As a system that maintains a relatively close weight to the existing system, hollow core plank on steel does not impact the foundation immensely, using the composite beam system as a baseline. The lateral system also does not require much adjustment, as the braced frames and shear walls would still fit into this design.

Construction

In terms of constructability, precast concrete panels on steel beams would not require high level construction, and would therefore be an inexpensive, quick installation. In addition, longer lead times would be required, as hollow core planks do not allow for drilling through them for mechanical systems. This requires more front-end coordination between the structural and mechanical teams, and would also delay the project timeline. Fireproofing would also need to be considered, as steel beams and girders are still being used. This would increase the project cost and construction time.

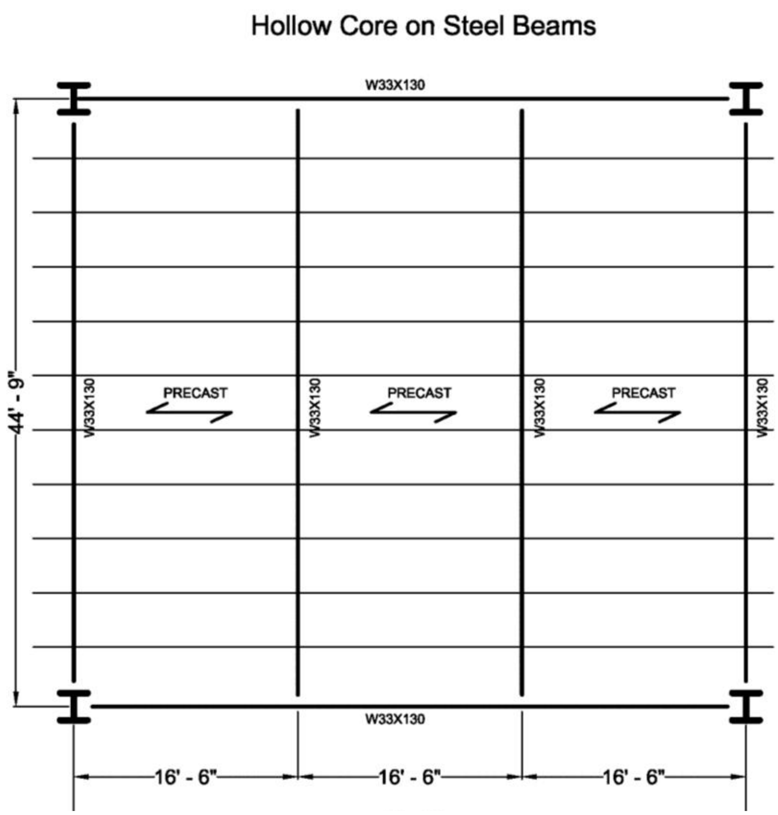


Figure 17: Layout for hollow core planks on steel beams.

Serviceability

Deflection in this system, though better than the existing system, is still a fairly high value, at 1.95". This is a visible deflection that could create issues amongst those utilizing the space. While this system has a denser floor system, it also will maintain better mitigation of noise and vibration between floors.

Conclusion

Though this system has issues in terms of system depth and deflection, this system has its advantages. These advantages come from vibration and noise isolation, ease of construction, and a relatively consistent cost. Though not seemingly the best system, this is still a viable option if other layouts are considered more thoroughly.

B: Precast Concrete Plank on Inverted T-Beams

Precast on concrete framing, with inverted T-beams as the main consideration, is seen in Figure 18. Inverted T-beams were considered as an alteration on the steel framing system, as a potential for minimizing floor depth. Inverted T-beams were designed as 40IT32s from Nitterhouse catalogs, included in Appendix 3, and were designed to rest on columns at the ends of the spans.

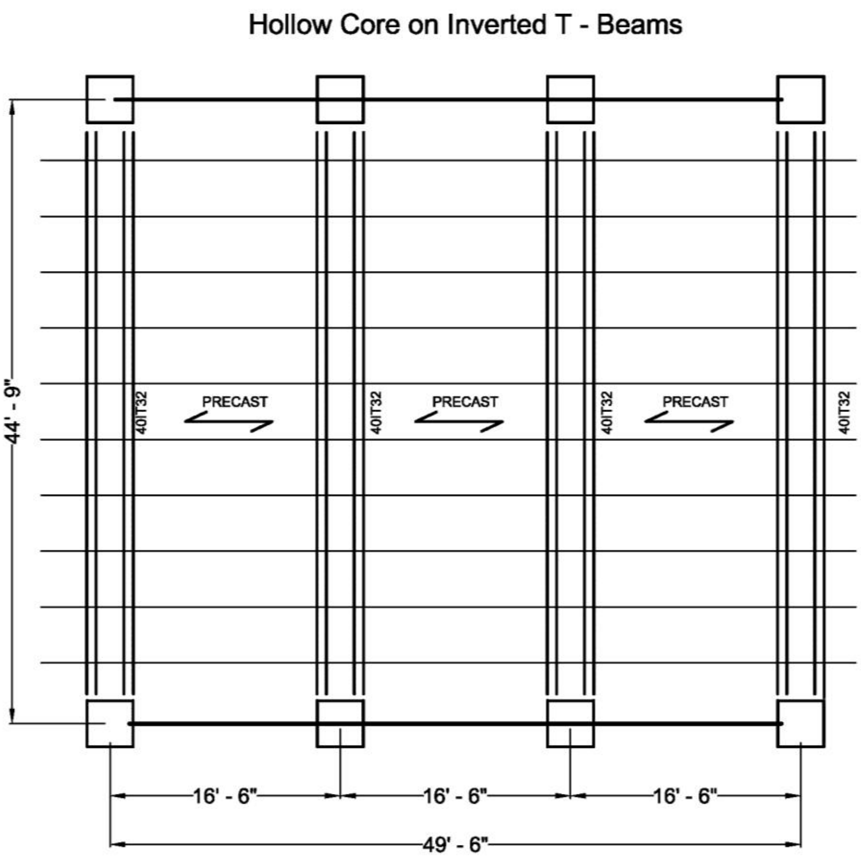


Figure 18: Layout of hollow core planks on inverted T-beams.

General

The hollow core plank flooring has an 8” depth, and an overall floor depth of 2.7 feet (32”). This alternative weighs 143.3 psf, which is primarily due to the use of the inverted T-beams. These members also increase the cost, which is at $24.06/SF. This is the most expensive system, and one of the heaviest systems. These calculations can be seen in Appendix 6.

Architectural

With the use of Inverted T-beams, the best solution for the weight and depth was the addition of columns along the northern column line. This impacts the architecture by confining some of the spaces. Yet, all columns added for this bay were along wall lines or existing space partitions, so did not restrict spaces. This is a further issue along the rest of the building. On the other hand, the floor depth is shallowest of all the systems, and allows for more space for required mechanical systems.

Structural

This system is a much heavier system as compared to the existing, as it includes the use of the inverted T-beams. Not only is the seismic loading increased, but the foundation is impacted as well. With additional columns supporting the bays, more spread footers will need to be included to support the additional columns and weight. The lateral system is no longer completely viable, as a concrete system would then require shear walls in each direction. Though shear walls are included in the design already, the braced frames would need to be replaced by additional shear walls to support the lateral system.

Construction

In terms of constructability, precast concrete panels on precast inverted T-beams would not require high level construction, and would therefore be an inexpensive, quick installation. Being precast, this also does not require the curing time for a cast-in-place system. Yet, longer lead times would be required, as hollow core planks do not allow for drilling through them for mechanical systems. This requires more front-end coordination between the structural and mechanical teams, and would also delay the project timeline. One added benefit to the use of this alternative system is that no additional fireproofing is required.

Serviceability

While the use of hollow core planks on steel beams resulted in a larger deflection, this system, by using a heavier system with a larger cross section, minimizes the deflection to 0.89”. This is less than half of the allowed deflection, and is an added benefit to the system. Noise and vibration isolation is also an added benefit to this system, as the materials have a satisfactory response to noise and movement dissipation.

Conclusion

This system includes benefits such as a shallow system at 2.7’, easy constructability, and good deflection and noise control. Though disadvantages include additional column and shear wall considerations, this system’s advantages keep this as a possible redesign option.

System 2: Post-Tensioned Concrete Flat Plate

A post-tensioned concrete design was selected for potential benefits in longer spans, minimizing column requirements, and helping to decrease slab depth. These designs and calculations followed design aids in *Prestressed Concrete: A Fundamental Approach* (5th Edition), written by Edward G. Nawy. Results of these calculations gave a 20" thick flat plate, with post-tensioning of ½" Φ 7-wire unbounded tendons at 8" spacing running North-South and at 9" spacing East-West. This layout can be seen in Figure 19. Calculations can be found in Appendix 4.

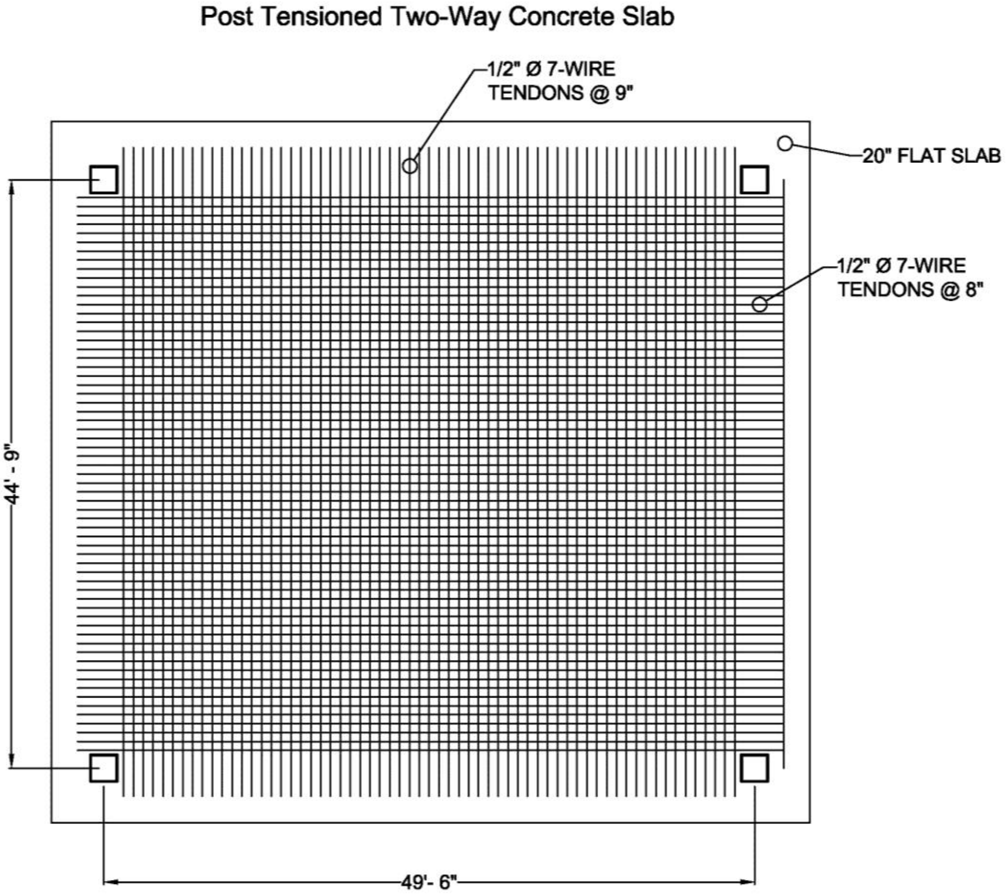


Figure 19: Layout of post-tensioned two-way concrete slab.

General

Because of the use of a flat plate post-tensioned system, the overall depth is the depth of the slab, which is 1.7 feet (20"). Because of this depth, it is the heaviest system at 250 psf. Yet, this system turned out to be one of the cheaper systems, at $21.04/SF, which can be accounted for with lack of formwork and fireproofing. Cost breakdowns, using RS Means Building Construction Cost Data, can be seen in Appendix 6.

Architectural

Overall, the post-tensioned slab allows for a more open ceiling space above floors, as it is a flat plate system. Though the floor is deeper and detracts from some of the floor-to-floor height, the lack of beams and drop panels is an added benefit to the system.

Structural

Cracking and deflection under service loads are more controlled by the use of post-tensioned slab, as seen by the use of tendons to allow for greater spans. Punching shear though, is an issue that a post-tensioned flat plate presents. This could be benefitted by the use of drop panels. Yet the positive moment of this system at mid-span, due to a large span and live load, is too great for drop panels to fully benefit the system. Beyond this bay, the bays do not show enough continuity to allow for ease in using post-tensioning. With the issues of large spans combined with the live load induced mid-span moment, it can be seen that this system is not a viable alternative for the SSPAC in terms of structure.

Construction

Due to the nature of post-tensioning, it requires a more specialized knowledge base for installation of the precast slabs with post-tensioning. After being placed and poured correctly, tensioning is required after a certain number of days. With this in mind, post-tensioning also requires a higher level of coordination between the structural team and the MEP teams for space allotment for systems before pouring. Core drilling cannot happen afterwards except at higher costs, as x-rays would need to be gathered to identify tendon location. These issues of a more specialized construction team and higher coordination would also impact the schedule, requiring more lead time and curing time.

Serviceability

Total load deflection, at .53", is the lowest of all of the systems. This is a huge benefit, as the long span is primarily controlled by its strength. With the slab being so thick, vibration and noise are not a concern.

Conclusion

Post-tensioning as an alternative system would be a viable system if all spans were more consistent, to be able to continue tendons. Other disadvantages to this system include the floor depth of 20", the high mid-span moment created by the high live load, a higher level of lead time and coordination between engineers, and the construction team's required experience in post-tension construction. These disadvantages outweigh the benefits of low deflection and slab depth, especially with the cost of the system not being any more inexpensive.

System 3: One-Way Slab on Beams

This third alternative system was a one-way slab-on-beam system, chosen due to the existence of concrete in the structure already, and the ease of construction and application of one-way on a series of irregular bays. The design process for this resulted in iterations of various dimensions, ending in a system that approximately matches a 24”x24” column size. Interior beams spanning the bay were chosen to keep the slab thinner while maintaining deflection control. The layout can be seen in Figure 20. Calculations for these designs can be found in Appendix 5.

General

The one-way slab and beam system has a slab depth of 5”, and an overall depth of 3.2’ (38”). The system costs $18.91/SF and weighs 97.4 psf. This is on the lower range of system weights, and, though a slightly thicker overall system, the one-way slab and beam system has a thin slab and a small overall cost. More detailed cost breakdowns can be found in Appendix 6.

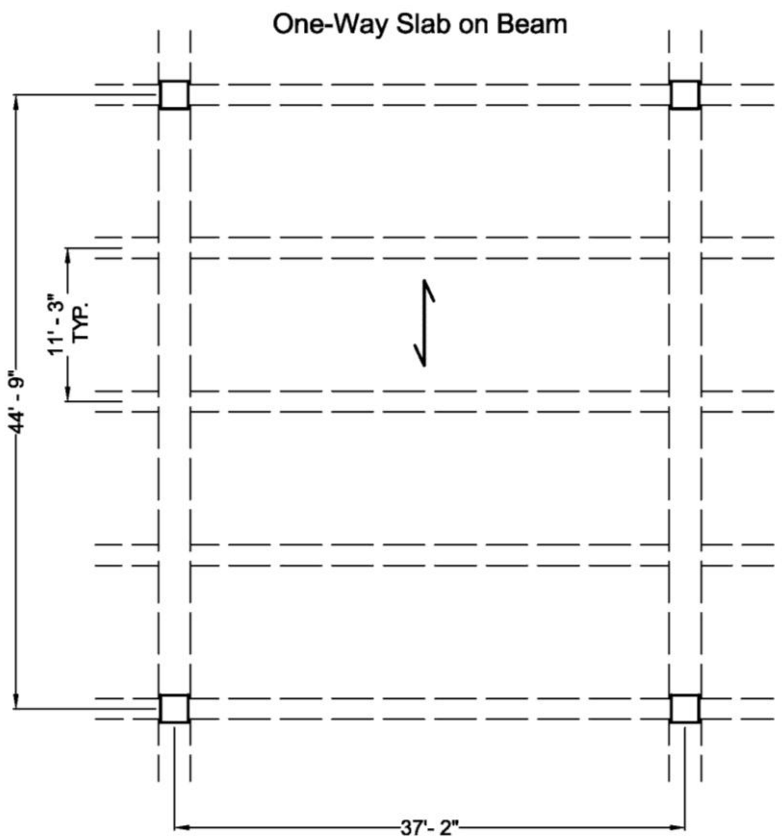


Figure 20: Layout of one-way slab on beam.

Architectural

One-way slab and beam is a viable system in terms of the architectural impacts, as it will not impact the bay sizes, and as in this bay, can be done without additional members. Though the aesthetics are taking a different interpretation than the existing building, it continues to tie into the culture of the area, with the history not only of the site as the previous Bethlehem Steel, but also tying this into the many cement and concrete mills in the area.

Structural

In terms of the structure, the one-way slab and beam system maintains fairly the same bay sizes as the existing system. This keeps the increased floor system loading going to the existing foundations, and therefore increasing the required strength of the foundation system. Looking at the entire structure, it is possible to continue this through the rest of the building.

Construction

The one-way slab and beam system requires the most formwork and shoring of any of the systems, with a larger amount of time on site. Though the construction schedule is impacted in terms of time needed for curing, the labor is also less expensive, allowing for an inexpensive system. With a shallower system, MEP has more flexibility in location, and does not require a high level of coordination between teams.

This site is also located in Bethlehem, which is a prime location for cement and concrete production. This would drive down costs of these materials, as they are more easily available.

Serviceability

This system is a beneficial one in terms of deflection, with overall deflection at 0.60”, at almost the same deflection as post-tensioning, which saw the least deflection. Due to this system being such a heavy one, it also minimizes vibration and noise isolation very well.

Conclusion

As the last alternative, this system has many advantages to being used for the design of the SSPAC. Not only is it a viable system in terms of constructability, ease of access to materials, and serviceability, it also is one of the least expensive of the systems analyzed and does not impact the structure in terms of weight and lateral very much.

Comparison of Systems

As each of these systems was considered as a design for the chosen bay of the SSPAC, various advantages and disadvantages to each system were considered. These considerations have been compiled in the following table for better understanding of these systems and a side-by-side perspective on the benefits of choosing one system over the other. The systems that are still deemed as viable systems for this structure are kept for investigation as a final redesign system.

	Design Considerations	Composite Beam (Existing)	Hollow Core Plank (A) - Steel Bm	Hollow Core Plank (B) - Invt. T-Bm	Post-Tensioned	One-Way Slab on Beam
Construction	Weight (psf)	63.5	88.9	143.3	250	97.4
	Depth of Slab (in)	5	8	8	20	5
	Depth of System (ft)	2.9	3.6	2.7	1.7	3.2
	Cost ($/SF)	17.93	20.44	24.06	21.04	18.91
	Fire Rating (hr)	1	1	1	1	1
	Fire Protection	Spray Fireproofing	Spray Fireproofing	None	None	None
	Schedule	N/A	Slightly more lead time; more coordination required	Slightly more lead time; increased coordination required	Extended lead time & coordination	Curing & formwork time required
	Constructability	Moderate	Easy	Easy	Challenging	Moderate
Structural	Foundation	N/A	Approx same weight, no change in foundation considerations	Add more columns, increase in spread footers amount and strength	Less columns required in some areas, increase in spread footers required	More weight, more impact on existing footers
	Seismic Increase	N/A	Minimal	Significant	Significant	Yes
	Lateral	N/A	Barely any adjustments required	Braced frames not viable, more shear walls required	Braced frames not viable, more shear walls required	Braced frames not viable, some additional shear walls required
Architectural	Impact	N/A	No significant adjustments required, some bays slightly adjusted	Additional columns for some bays	Less columns, more open spaces and flexibility of space	Interior bay members; somewhat less space to play with, more consistency in member sizes.
Serviceability	Deflection (in)	0.77	1.35	0.89	0.53	0.60
	Vibration Control	Fair	Satisfactory	Satisfactory	Fair	Best
Viable system		Yes	Yes	Yes	No	Yes

Conclusion

Through the comprehensive and in-depth analysis of the SteelStacks Performing Arts Center, by considering a typical bay on the second floor, a better understanding of the structural systems has been accomplished. This report shows the results of this better comprehension of the SSPAC through considering three alternative systems for the chosen typical bay. Previous analysis of gravity loads, lateral loads, and a structural overview have been summarized preceding this analysis for a better understanding of the results. These design procedures relied heavily on ASCE 7-05 and AISC, 14th edition.

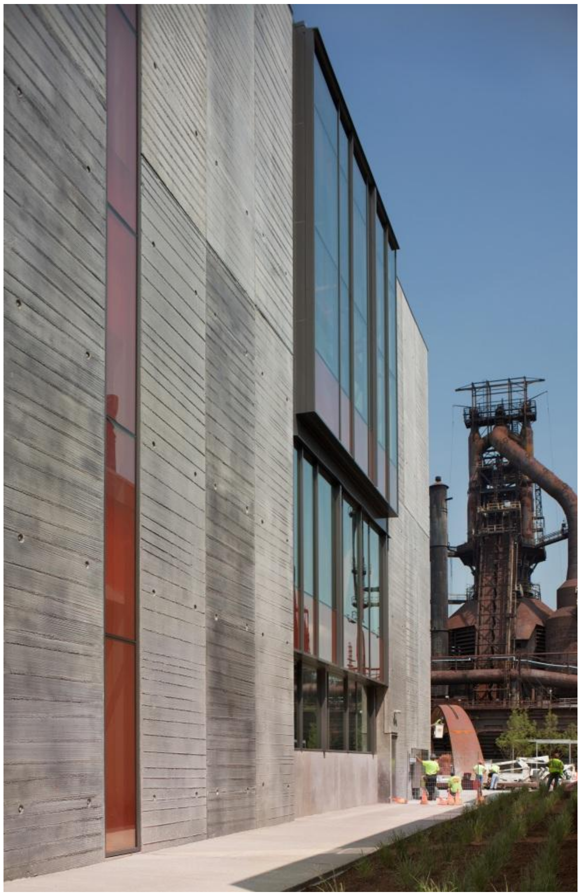
Initially, the existing system was analyzed. Advantages include light weight, inexpensive cost, and ease of construction. It is easy to understand why this system was chosen for the SSPAC, even though this system could have issues with noise isolation.

The next system considered as an alternative to the existing floor structure was precast concrete slab and beam system. The first design configuration for this system was designed with steel beams. Disadvantages for this system relate to overall depth and deflection. Advantages come from vibration and noise isolation, ease of construction, and a relatively consistent cost. This is currently not the most plausible system, but variations in the layout could keep this as a viable system. The second portion of this alternative system used precast beams supporting the hollow core planks, giving the system a much shallower overall depth. Constructability, minimal deflection and noise control are other advantages. Though disadvantages include additional column and shear wall considerations, this system's advantages keep this as a possible redesign option.

A post-tensioned two-way flat plate system was the second alternative design. Disadvantages of this system include the overall building's bay inconsistencies, thick floor depth, large mid-span moments, and more difficult construction. These disadvantages outweigh the benefits of low deflection and slab depth, especially with the cost of the system not being any more inexpensive.

The last alternative system was a one-way slab and beam design. As the last alternative, this system has many advantages to being used for the design of the SSPAC. Not only is it a viable system in terms of constructability, ease of access to materials, and serviceability, it also is one of the least expensive of the systems analyzed and does not impact the structure in terms of weight and lateral very much.

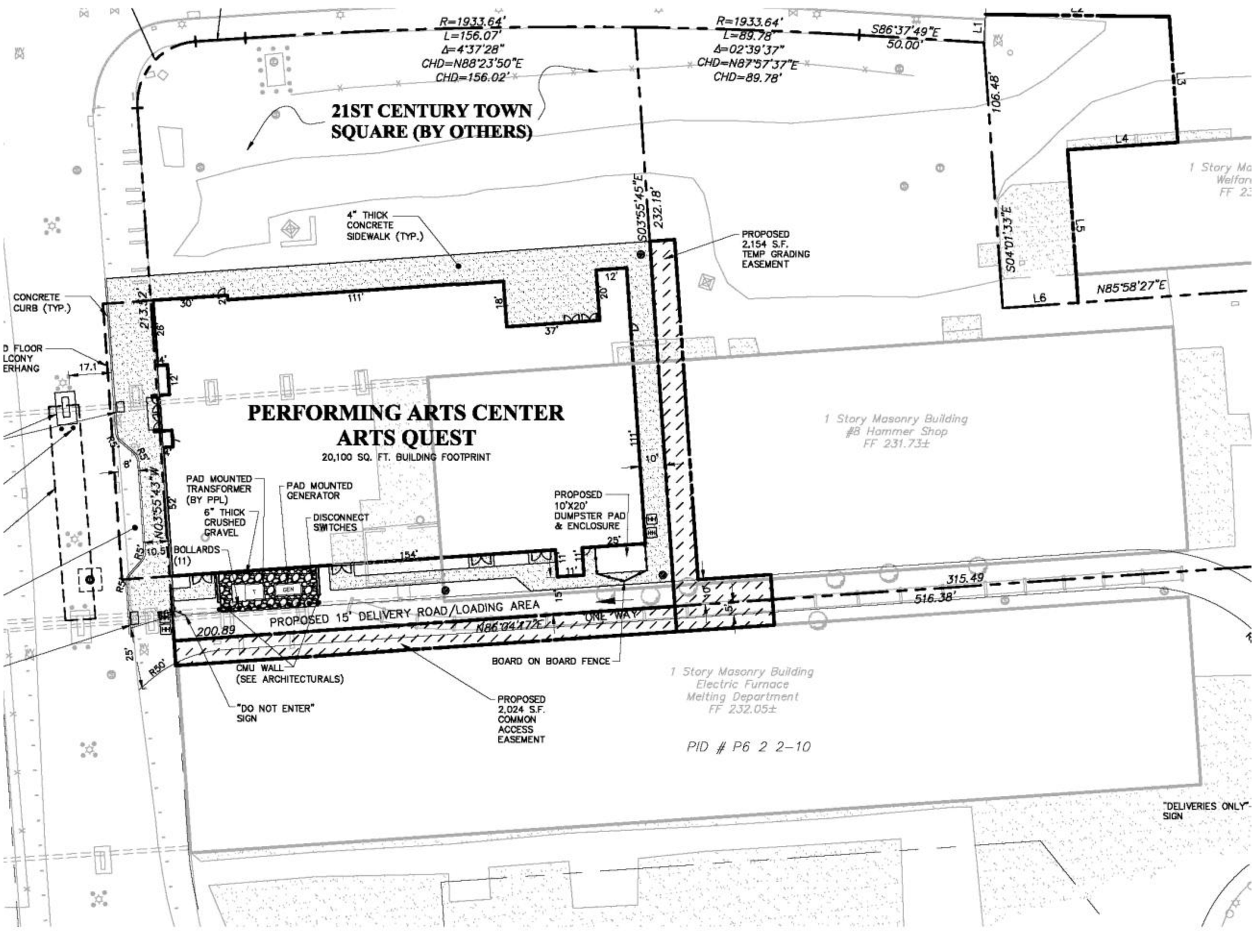
Appendices



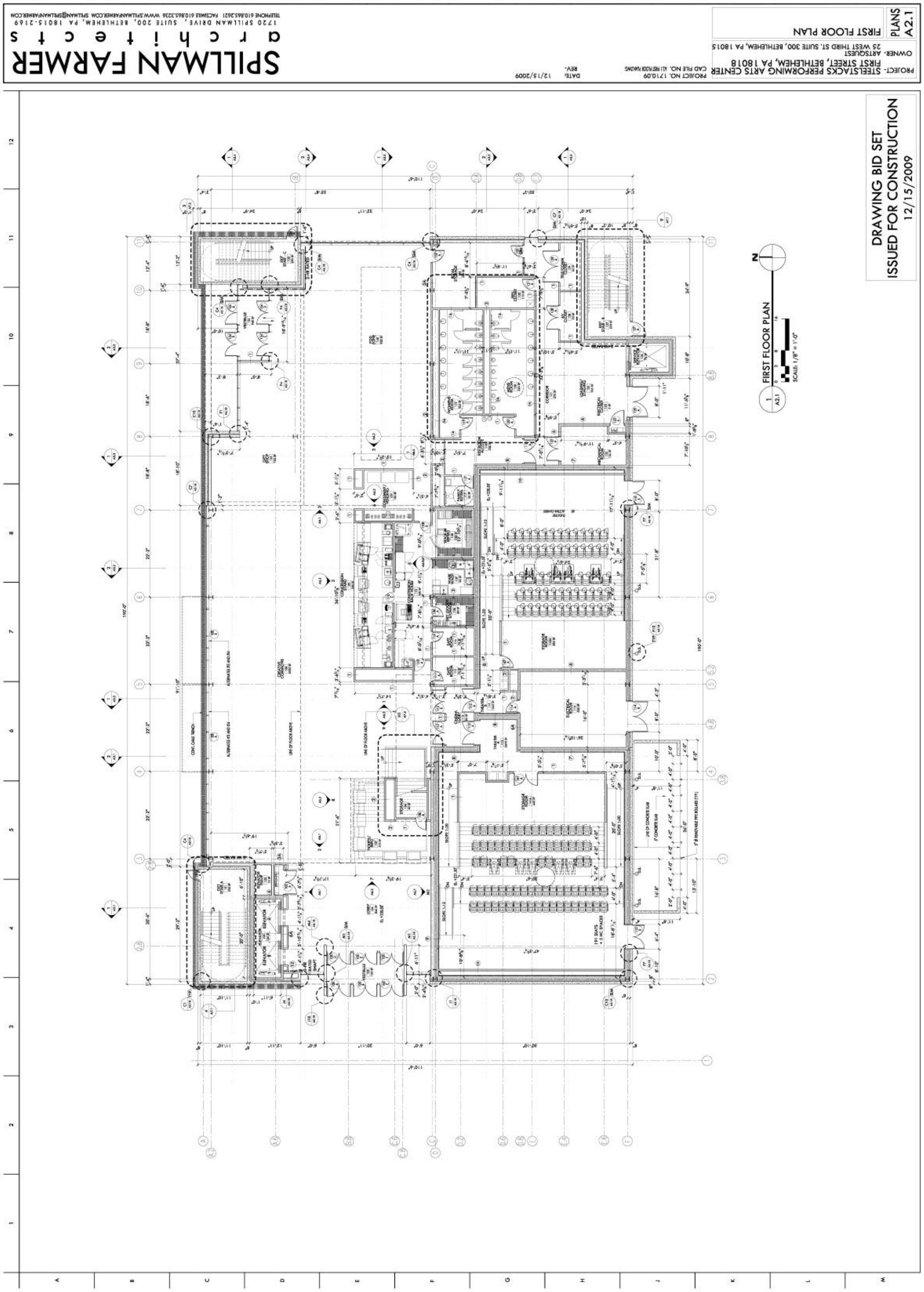
Appendix 1: Structural System Overview

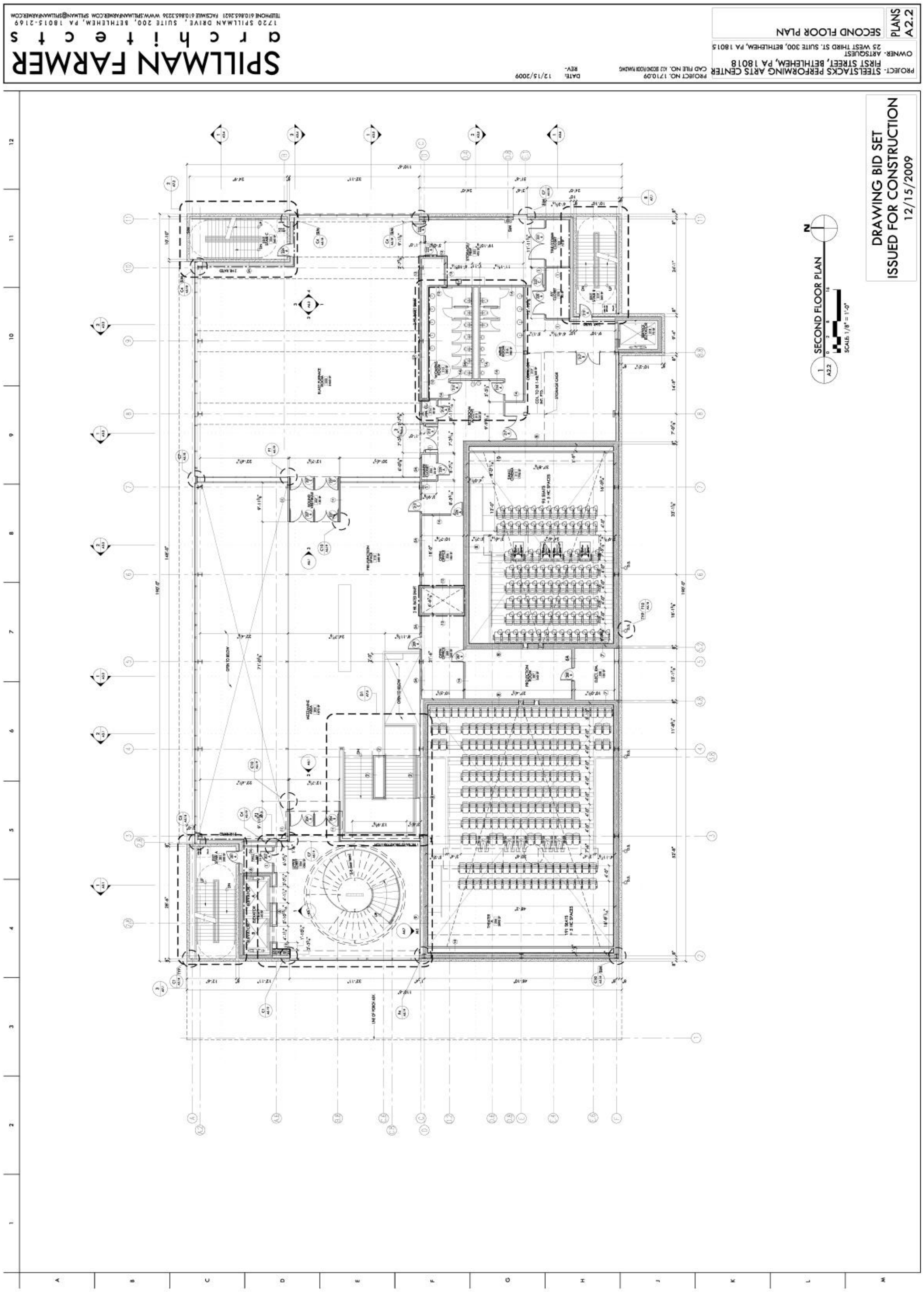
Site Plan Detail

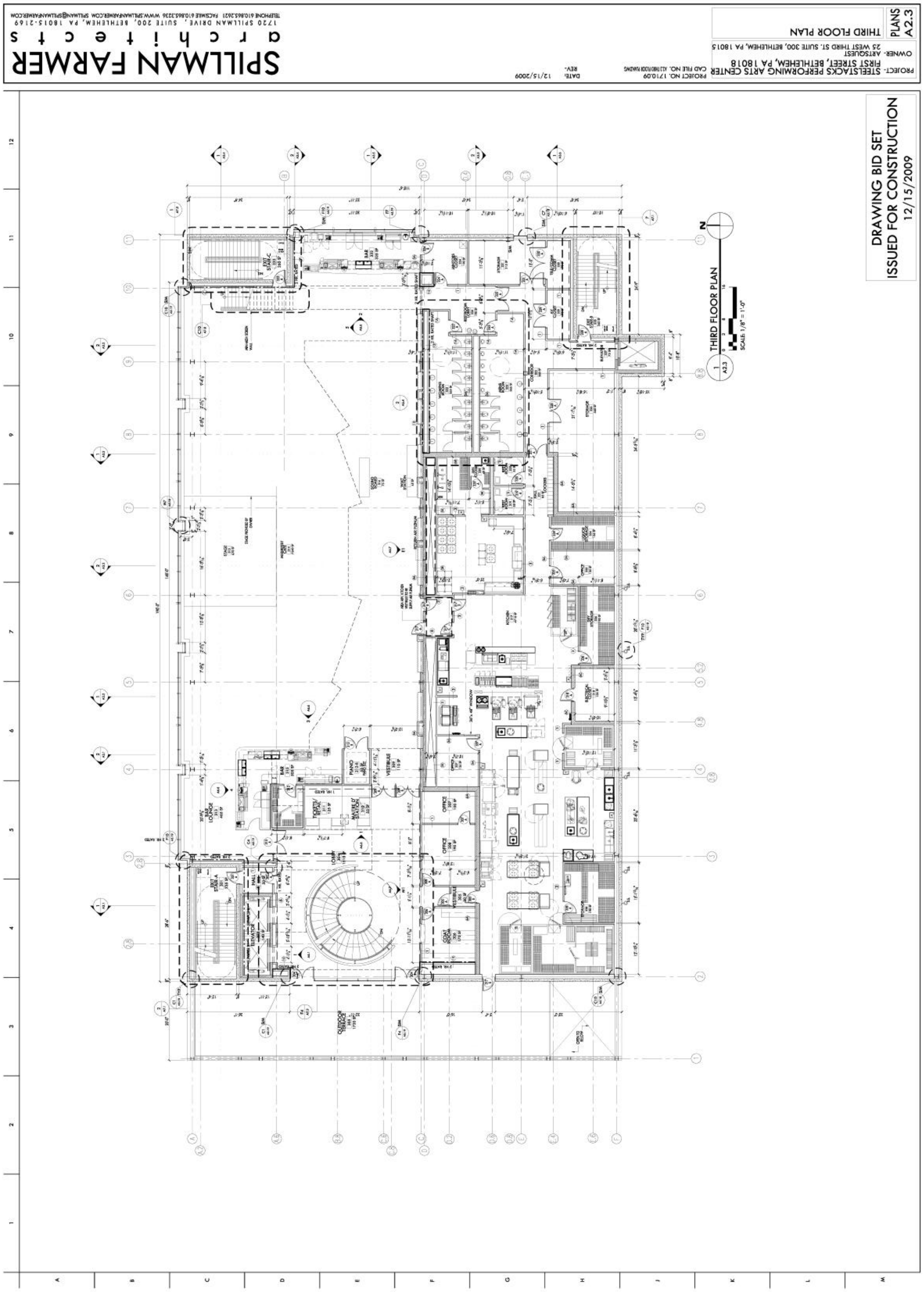
The location of the existing site at onset of project with current location overlaid.

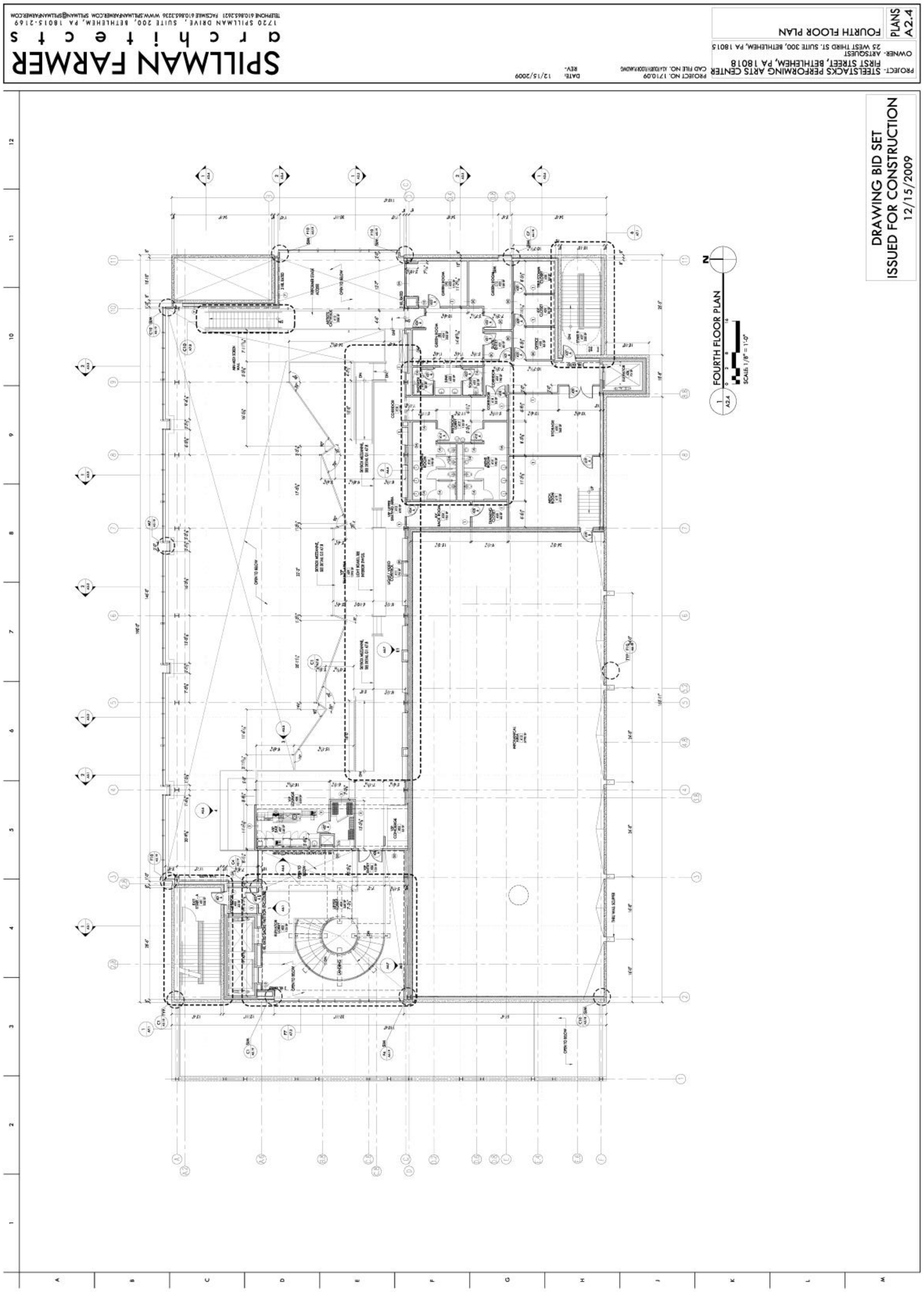


Architectural Floor Plans

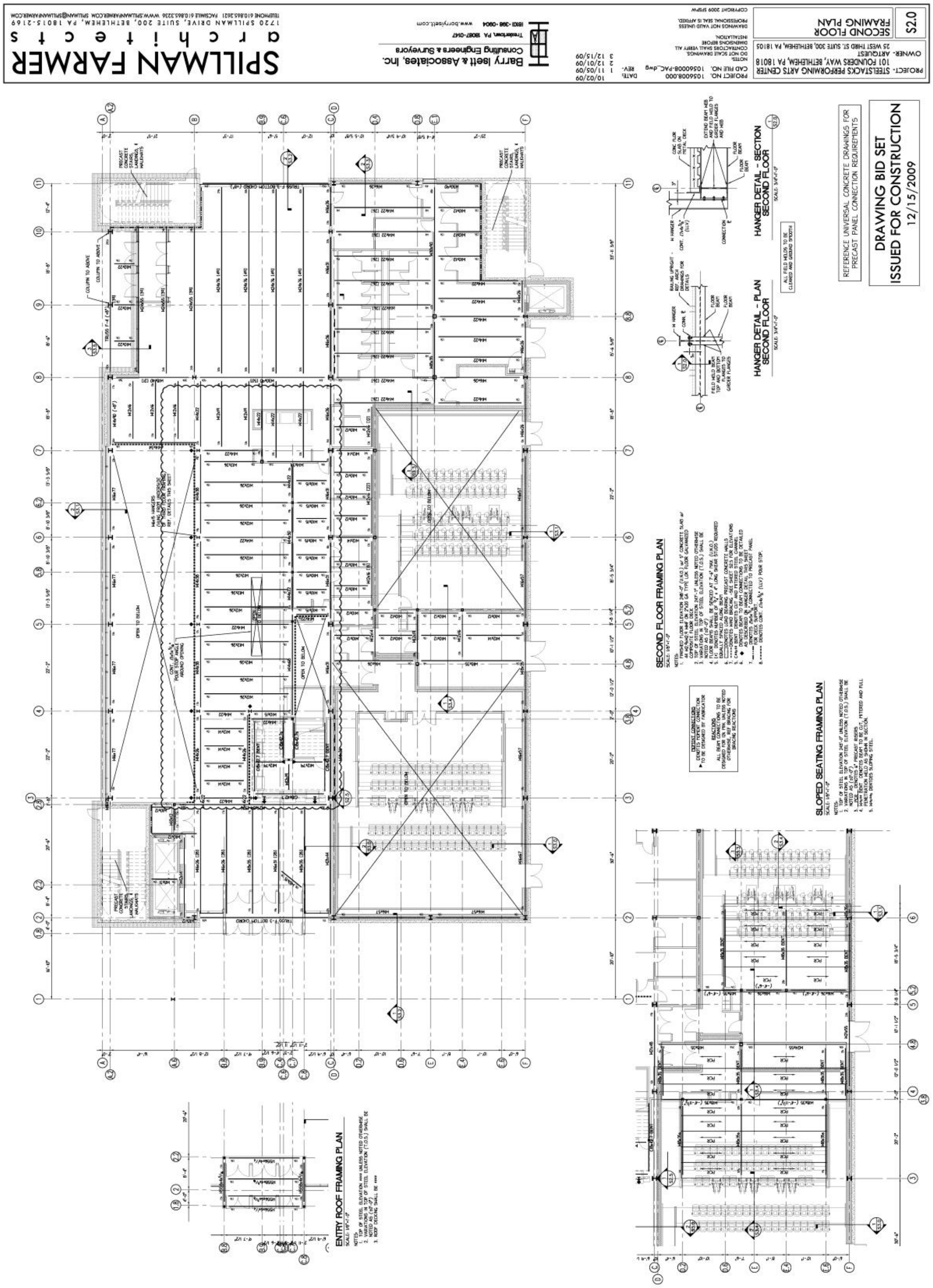



SPILLMAN FARMER
architects
SECOND FLOOR PLAN
PLANS
A2.2
DRAWING BID SET
ISSUED FOR CONSTRUCTION
12/15/2009

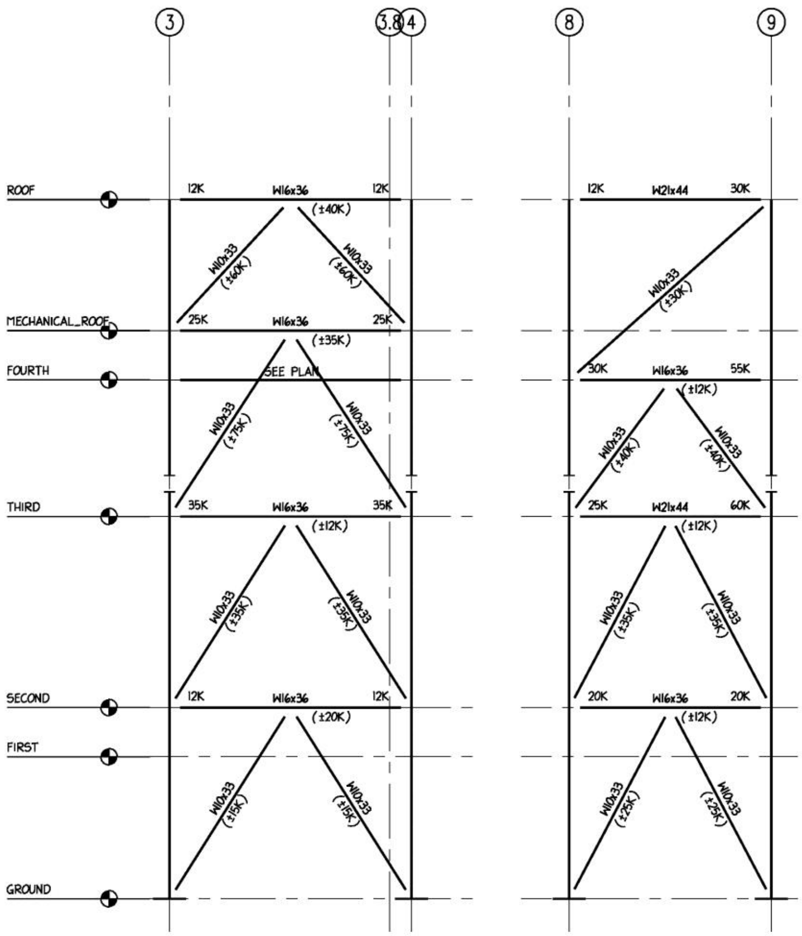

SPILLMAN FARMER
architects
THIRD FLOOR PLAN
PLANS
A2.3
DRAWING BID SET
ISSUED FOR CONSTRUCTION
12/15/2009



Structural Floor Plans



Lateral System



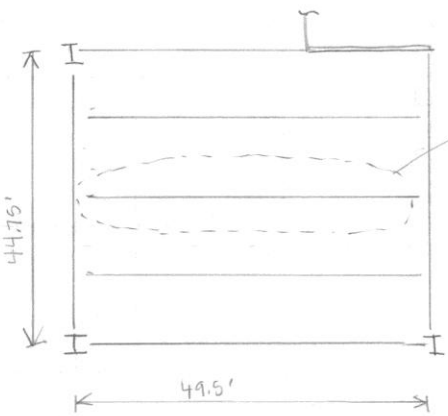
1 ELEVATION AT LINE C
SCALE: 1/8"=1'-0"

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: Existing: Composite Slab and Decking on Composite Beams

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	73 PSF DL
	SLAB	48	
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

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

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	$n = \quad = 26 \rightarrow 52$ studs
W21x50	696 > ✓	386	$n = \quad = 23 \rightarrow 46$ 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 (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 $\therefore Y_2 = 5 - .92/2 = 4.54$ → 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

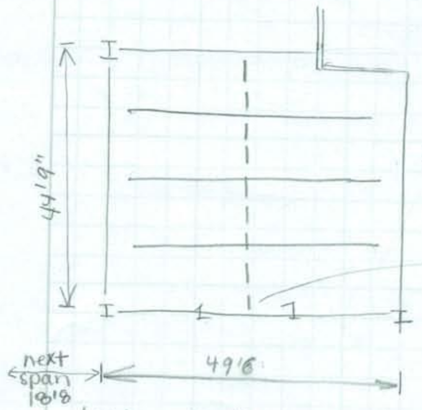
$\delta_{wc} = \frac{5}{384} \frac{(.549)(49.5)^4 1728}{29000 \cdot 3310} = .77'' < 2.5'' = \frac{\ell}{240}$ ✓ok

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

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

Appendix 3: Alternate System 1: Hollow Core Planks

Alt Frame 1: Hollow Core planks. T2



original beam spacing max at 7'6"

consider layout Ⓐ

X x 4' planks

- replace beams with one running N-S

$\longleftrightarrow$ planks spanning E-W

Load: D = 15 PSF

L = 125 PSF

Try 10" x 4' 2" topping 1hr fire rating

S.W = 68 PSF from Nitterhouse Charts

Design w/ MAX span @ 25'

$W_u = 1.2D + 1.6L = 218$ PSF $\rightarrow$ try 26' span (6) 1/2" ø strands. 240 PSF ≥ 218 ✓ ok

$M_u = wl^2/8 = 817.5$ kin

will control in flexure, deflection check.

flexure $f_t = -\frac{M}{S} - \frac{P}{A} + \frac{Pe}{S} = \frac{-817.5}{1340} - \frac{24.8(6)}{327} + \frac{24.8(6)(4.44)}{1340} = 572$ psi

$P = 60\%(270)(.153) = 24.8$

$A = .153$ in² of 1/2" ø (App E ACI 318-11)

$A_c = 327$

$e = 6.19 - 1.75 = 4.44$

$E = 57000\sqrt{f'c} = 4415$ ksi

total load $572 < .60 f'c = 3600$ psi ✓ ok by ACI §18.4.2

deflection:

$\Delta_L = \frac{5 w_L \ell^4 1728}{384 EI} = \frac{5(1.6(.125)4')25^4 1728}{384 \; 4415 \; 5102} = .31" \ll .83" = \frac{\ell}{360}$ ✓

$\Delta_{tot} = \frac{5(1.2(.015+.068)+1.6(.125))4(25^4)1728}{384 \; 4415 \; I} = .47" \ll 1.25" = \frac{\ell}{240}$ ✓

Moment:

$M_n = w\ell^2/8 = 93.6$ kft $\ll 168.1$ kft from Nitterhouse

$\boxed{\text{use } 10"\times 4' \text{ @ } 2" \text{ topping and 1 hr fire rating}}$

2

Beam Design:

includes assumed S.W.

$W_u = 1.2\,(22+68) + 1.6\,(125) = 308$ psf

$= 308\,(21.8) = 6.7$ klf

$\ell = 44.75'$

$w = 25/2 + 18'8/2 = 21.8'$

LL reduction not allowed by §4.8.4

$W_{LL} = 4.4$ klf

$E = 29\,000$

$V_u = w\ell/2 = 150$ k

$M_u = w\ell^2/8 = 1677$ kft

$$I_{req'd,\,LL} = \frac{5\,W_{LL}\,\ell^4\,1728}{384\,E\,(\ell/360)} = 9178\ \text{in}^4$$

$$I_{req'd\,TL} = \frac{\text{"}}{\text{"}\,(\ell/240)} = 9317\ \text{in}^4 \leftarrow \text{controls}$$

Shape	I
W 40 x 149	9800
W 36 x 160	9760
W 27 235	9100

by Table 3-3 AISC

most economical of listed

check S.W.

$\frac{135}{25'} = 5.4 < 7'$

use $\boxed{\text{W40x149 supporting planks}}$

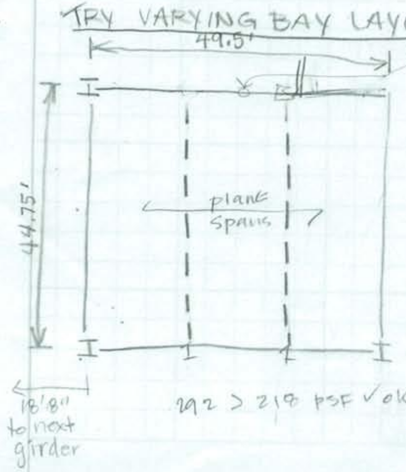
Table 3-2

$\phi M_n = 2240$ kft $> M_u = 1242$ kft

$\phi V_n = 650$ k $> V_u = 111$ k

(deflection controls span)

TRY VARYING BAY LAYOUT: (B)



Ⓐ / GIRDER

BEAMS SPACED AT 16.5'

span for planks = 16.5'

SAME BASE LOADS AS BEFORE

TRY 6" x 4' (6) 1/2"ϕ strands

2" topping 1hr fire rating

292 > 218 PSF ✓ok

using $\ell = 16.5' \rightarrow W_u = 218$ PSF

$M_u = \frac{w\ell^2}{8} = 356$ kin

S.W. = 49.75 PSF

3

will control in flexure, deflection:

$f_t = \frac{-356}{1799} - \frac{24.8(6)}{253} + \frac{24.8(6)(2.35)}{799} = -.596 = 596$ psi < 3600 psi ✓ok

$P = 60\%(270)(.153) = 24.8$; $A = .153$ in² of ½"ϕ

$A_c = 253$

$e = 4.10 - 1.75 = 2.35$

$E \sim 4415$ ksi

deflection:

$\Delta_u = \frac{5(1.6(125)4)(16.5)^4 1728}{384 \cdot 4415 \cdot 1519} = .06'' < .55''$ $\ell/360$

$\Delta_{TL} = \frac{5(1.6(125)+1.4(15+48.75))4(16.5)^4 1728}{384\,EI} = .08'' < .83''$ $\ell/240$

moment:

$M_n = \frac{w\ell^2}{8} = \frac{[1.2(15+48.75)+1.6(125)]4 \cdot (16.5)^2}{8} = 37.6$ k ft < 92.6 kft from Nitterhouse ✓ok

use 6" x 4' @ 2" topping & 1hr fire rating.

Design Beams:

$w = 18.6/2 + 16.5/2 = 17.6$

$\ell = 44.75'$

$w_u = 1.2(22+48.75)+1.6(125) = 284.9$ psf

$= (284.9)(17.6) = 5014 = 5.0$ klf

LL reduction

- Not allowed by § 4.8.4

$w_u' = [1.2(70.75)+1.6(125)]17.6 = 5.01$ klf → $V_u = 112$ k

$w_{LL} = 3.5$ klf $M_u = 1252$ k·ft

$I_{req'd,LL} = \frac{5 w_{LL} \ell^4 1728}{384\,E(12\ell/360)} = 7300$ in⁴

$I_{req'd,TL} = \frac{5 w_{TL} \ell^4 1728}{384\,E(\ell \cdot 12/240)} = 6953$ in⁴

Table 3-3 AISC:

Shape	I (in⁴)	
W36 x 135	7800	← most economical
W 24 x 229	7650	
W 33 x 141	7450	

TRY $\boxed{W36\times135}$ → check SW $\frac{167}{17} = 7 \le 7'$ ✓ok

Table 3-2

$\phi M_n = 1910$ kft > 1252 kft ✓ ok.

$\phi V_n = 577$ k > 112 k

$\delta = \frac{5w\ell^4 1728}{384EI} = 1.99''$

4

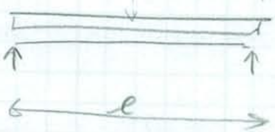
Now, USING LAYOUT B, DESIGN GIRDER

realigning column lines to match girder spacing $X = 16.5'$

GIRDER A:

$l = 37.2'$ consider load as distributed

pt load from Beam → distrib. load.



$P_u = W_u'(44.75')/2 = 112$ k.

$P_u = 78$ k

$W_u^* = (112)\times 2/37.2 = 6.0$ klf.

$W_u^* = 4.2$ klf

$M_u = 1038$ kft

$V_u = 112$ k E = 29000 ksi for steel.

$I_{req'd,\,LL} = \dfrac{5 W_u^* l^4 1728}{384 E (l/360)} = 5991\ in^4$ ← controls

$I_{req'd,\,DL} = 4793\ in^4$

by Table 3-3

Size	I
W36x135	7800 in⁴
W33x130	6710 in⁴

← most economical sizes.

Choose W33X130

check:

$\phi M_n = 1750$ kft > 1038 kft

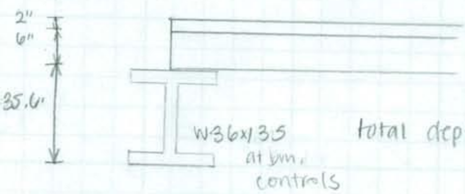
$\phi V_n = 576$ kft > 112 k ✓

Check: $\delta = \dfrac{5 w l^4 1728}{384 E I} = 1.33''$

$\boxed{\text{USE W33X130}}$

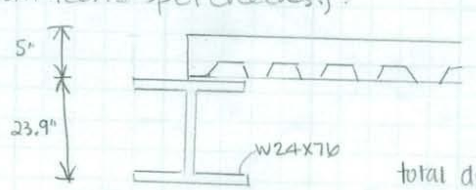
Layout B:

Overall depth: of System (1) layout B.



total depth = 43.6" = 3.6'

original system depth.
(from tech 1 spot checks.)



5" 2VLI20.

total depth = 28.9" = 2.4'

5

Design Inverted T-beam instead of Steel Beam : system(2) layout B.

same spacing as layout (B) :

$f'c$ - 6000 psi

$W_u' = 3.7$ klf → $V_u = 82.8$ k

$W_u' = 2.2$ klf $M_u = 926$ k-ft = 11,112 k-in

Span = 44.75'
use 46'

Try 40IT32 w/ strand pattern 16-60 top bars (6)#9

$\phi M_n = 29451 > 11,112$ k-ft

span 46' allows LL = 4.9 klf > 2.2 klf ✓ok

Controlled by flexure, deflection check: $S = 5926$ in³

flexure: $f_t = -\frac{M}{S} - \frac{P_n}{A} + \frac{P_e e''}{S} = -\frac{926}{5926} - \frac{58.6(28)}{1096} + \frac{58.6(28)\,11.3}{5926} = 1475$ psi

$P = 270(.217) = 58.6$

$A = .217$ for .600" ⌀ Gr. 270 by ACI 318-11

$A_c = 1096$

$n = 28$ prestressed bars

$e = 14.28 - 3 = 11.3''$

check:
1475 psi < 3600 psi ✓ ok

deflection: E = 4415 ksi

$\Delta_{LL} = \frac{5 w_L \ell^4 \,1728}{384\,EI} = \frac{5(2.2)(44.75)^4 1728}{384 \quad 4415 \quad 84622} = .53'' < 1.50'' = \frac{\ell}{360}$

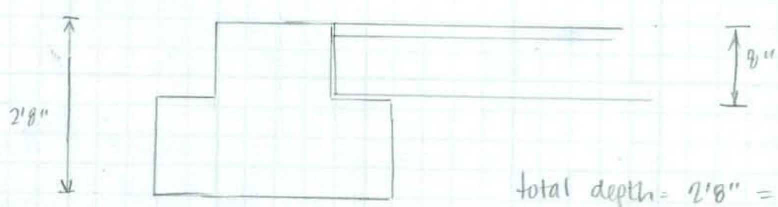
$\Delta_{TL} = \frac{5(3.7)}{\longrightarrow} = \longrightarrow = .89'' < 2.2'' = \frac{\ell}{240}$

✓ok by deflection

Moment:

$M_u = 926$ kft = 11 112 k in < 29 451 k in = ϕM_n ✓ok

∴ use 40 IT 32



total depth = 2'8" = 2.7"
at Inverted Tee

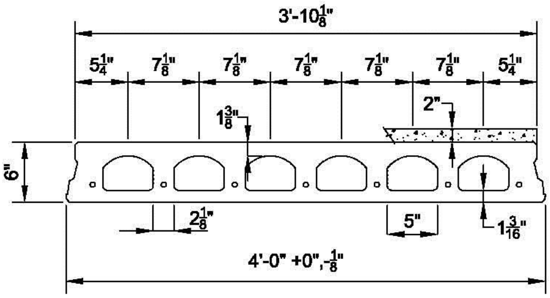
Members down the load path are not typical of the representative floor system.

Prestressed Concrete
6"x4'-0" Hollow Core Plank

1 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES Composite Section	
A_c = 253 in.^{2}	Precast b_w = 16.13 in.
I_c = 1519 in.^{4}	Precast S_{bcp} = 370 in.^{3}
Y_{bcp} = 4.10 in.	Topping S_{tct} = 551 in.^{3}
Y_{tcp} = 1.90 in.	Precast S_{tcp} = 799 in.^{3}
Y_{tct} = 3.90 in.	Precast Wt. = 195 PLF
	Precast Wt. = 48.75 PSF

DESIGN DATA



1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)..
   4-1/2"Ø, 270K = 67.4 k-ft at 60% jacking force
   6-1/2"Ø, 270K = 92.6 k-ft at 60% jacking force
   7-1/2"Ø, 270K = 95.3 k-ft at 60% jacking force
7. Maximum bottom tensile stress is $10\sqrt{f'c}$ = 775 PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI. Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span. A lesser thickness might occur if camber is taken into account during design, thus reducing the load capacity.
13. Load values to the left of the solid line are controlled by ultimate shear strength.
14. Load values to the right are controlled by ultimate flexural strength or allowable service stresses.
15. Load values will be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
16. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

SAFE SUPERIMPOSED SERVICE LOADS — IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)

Strand Pattern		SPAN (FEET)																		
		12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
4 - 1/2"ø	LOAD (PSF)	349	317	290	258	227	197	179	157	148	131	110	91	75	60					
6 - 1/2"ø	LOAD (PSF)	524	478	437	377	334	292	269	237	224	193	166	142	122	104	88	73	61	49	39
7 - 1/2"ø	LOAD (PSF)	541	492	451	416	364	331	293	274	242	214	190	167	144	124	107	91	77	64	53

NITTERHOUSE
CONCRETE PRODUCTS

2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17202-9203
717-267-4505 Fax 717-267-4518

This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths. The allowable loads shown in this table reflect a 1 Hour & 0 Minute fire resistance rating.

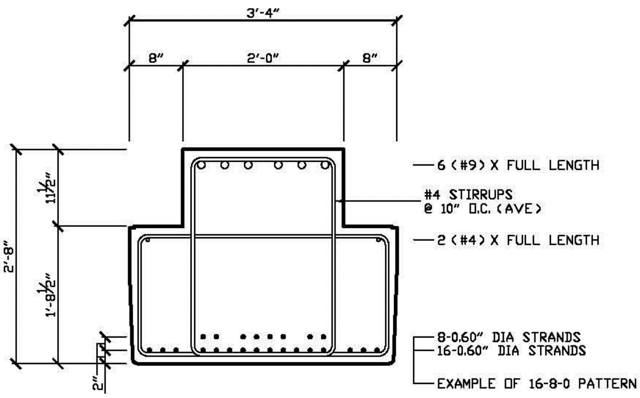
11/03/08

6F1.0T

Prestressed Concrete
Inverted Tee Beam 40IT32-B

PHYSICAL PROPERTIES

$A = 1{,}096\ in.^2$ $S_b = 5{,}926\ in.^3$
$I = 84{,}622\ in.^4$ $S_t = 4{,}775\ in.^3$
$Y_b = 14.28$ in. Wt.= 1,142 PLF
$Y_t = 17.72$ in.



DESIGN DATA

1. Precast Strength @ 28 days = 6,000 PSI
2. Precast Strength @ release = 4,000 PSI.
3. Precast Density = 150 PCF
4. Strand = 0.60"Ø 270K Lo-Relaxation.
5. Ultimate moment capacity shown below is for full strand development & tension controlled section.
6. Maximum bottom tensile stress is $12\sqrt{f'c} = 930$ PSI
7. Flexural strength capacity is based on stress/strain strand relationships and is slightly variable.
8. Deflection limits were not considered when determining allowable loads in this table.
9. All superimposed live loads listed are controlled by ultimate flexural strength, not allowable stresses.
10. All superimposed load is treated as live load in the flexural strength analysis. To determine the allowable live load if the amount of superimposed dead load is known use the following conversion method...

$$\text{Allowable Live Load} = \frac{(1.6)(\text{Load Table Value}) - (1.2)(\text{Superimposed Dead Load})}{1.6}$$

11. If the above conversion is used then allowable stress limits must be checked so they are not exceeded.
12. The concrete strength at release of prestress force increases to 4,500 psi for more than 22 strands.

ALLOWABLE SUPERIMPOSED LIVE LOADS (KLF)			IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)													
Strand Pattern	Top Bars	Moment Capacity	SPAN													
			24'	26'	28'	30'	32'	34'	36'	38'	40'	42'	44'	46'	48'	50'
8 - 0 - 0	2 - #9	11,915 "k	7.6	6.4	5.4	4.6	3.9	3.4	2.9	2.5	2.2	1.9	1.7	1.4	1.2	1.1
16 - 6 - 0	6 - #9	29,451 "k	20.1	17.2	14.7	12.7	11.1	9.7	8.6	7.6	6.8	6.0	5.4	4.9	4.4	4.0
16 - 8 - 0	6 - #9	31,294 "k	21.5	18.4	15.7	13.6	11.8	10.4	9.2	8.1	7.2	6.5	5.8	5.3	4.8	4.3

NITTERHOUSE
CONCRETE PRODUCTS

2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17201-0813
717-267-4505 Fax 717-267-4518

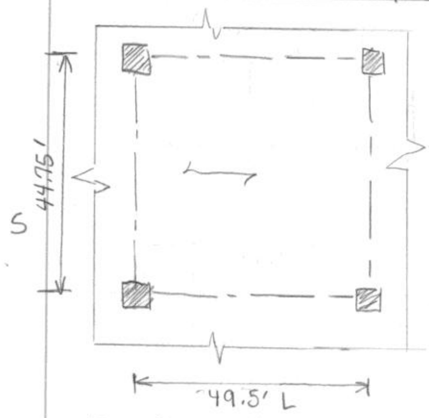
This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, etc...

04/04/08

40IT32-B

Appendix 4: Alternate System: Post Tensioned Slab

Sarah Bednarcik | POST TENSIONED | T2



N

Typical interior slab.

$f'c = 4000$ psi
$fy = 60\,000$ psi
$Es = 29\,000$ ksi

$LL = 125$ PSF
$SDL = 15$ PSF

Adding column at top right of bay, allowing for shorter span.

Column Sizes 24"x24"

Following Prelim. Analysis & Design by PTI "Design of Post-Tensioned Slabs Using Unbonded Tensions"

- 2 way
- Slab thickness:
  - trial size $\frac{\text{span}}{\text{depth}}$ ratio = 45

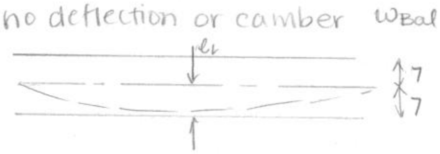
$h = \ell/45 = \frac{49.5}{45} \times 12 = 13.2"$ → trial depth 14"

- LOADINGS - see above

- <u>Section Properties</u>
  - a. Slab §18.3.3 of ACI 318-11
    - Class T - LL heavier, longer spans.
    - $f'ci = 3000$ psi → §7.6.7 min spac - {4db strands
    - $Eps = 29\,000$ ksi
    - $fpe = 159\,000$ psi
    - $fpy = 240\,000$ psi with: max $fc = .45f'c = 1800$ psi
    - $fps = 189\,000$ psi E-W max $fc \le 200$ psi
    - $Ec = 57\,000\sqrt{f'c} = 3605000$ psi N-S max $fc \le 350$ psi

<u>Balanced Loadings</u>

for no deflection or camber $w_{Bal} = w_D = 15\text{PSF} + \frac{8"}{12} 150 = 115$ PSF



$e_L = e_S = \frac{14"}{2} - 1 = 6"$

<u>Effective prestress</u>: $P_L = fc\,h\,12 = 200 \cdot 14 \cdot 12 = 33600$ lb/strip

$$W_{bal\,L} = \frac{8 P_L e_L}{L_L^2} = \frac{8(33600)(6)}{(49.5')^2 \times 12"/'} = 54.85\text{ PSF}$$

$$W_{bal\,(S)} = W_D - W_{bal\,L} = 115 - 54.85 = 60.15\text{ PSF}$$

S Bednarcik POST TENS. 2

$P_s = \frac{W_{bal\,s} L_s^2}{8 e_s} = \frac{60.15 (44.75)^2 \times 12}{8 \quad 6} = 30114$ lb/ft

$f_c (N\text{-}S) = \frac{P_s}{b \cdot h} = \frac{30114 \text{ lb/ft}}{12 \times 14} = 179.2 < 350$ psi ✓ok

use 1/2" ⌀ 7-wire 250K strands $A = .153 in^2$ by ACI318

w/ $P_e = f_{pe} A_\phi = 159000 (.153) = 24,327$ lb.

Req'd Spacing:

$S_s = \frac{P_e}{P_s} = \frac{24237}{30114} = .80 = 9.66'' \rightarrow 9''$

$S_L = \frac{P_e}{P_L} = \frac{24237}{33600} = .72 = 8.66'' \rightarrow 8''$

recommended spacing $3h = 3(14) = 42''$ under max range
$5h = 5(14) = 70''$ ✓ok

Service Load Stresses:

$W_L = 125$ PSF

$k = \frac{L_L}{L_s} = \frac{49.5}{44.75} = 1.11$

by Fig 9.10, see for values: $\alpha_y = \alpha_{E\text{-}W} = .042$ (long)
$\alpha_x = \alpha_{N\text{-}S} = .055$ (short)

$L_{s\,eff} = 44.75' - (24''/12) = 42.75'$

$L_{L\,eff} = 49.5 - (24''/12) = 47.5'$

LL Moments:

$M_s = \alpha_s \times W_L \times L_s^2 \times 12$

$= .055 (125)(42.75)^2 \times 12 = 150774$ lb·in/ft

$M_L = .042 (125)(47.5)^2 \times 12 = 142144$ in lb/ft

Moment of Inertia $I_s = \frac{12 (14)^3}{12} = 2744 \, in^4 = I_L$

Concrete Stresses due to LL

Short: $f_s = \frac{Mc}{I_s} = \frac{150774 \cdot 7''}{2744} = 384.6$ psi $= 385$ psi

long $f_L = \frac{142144 \cdot 7}{2744} = 362.6 = 363$ psi

$f_t = -\frac{P}{bh} - \frac{Mc}{I_s}$ $f_b = \frac{-P}{bh} + \frac{Mc}{I_s}$

S Bednarcik | POST TENS. | 3

Short/N-S: $f_t = \frac{-30114}{14 \cdot 12} - 383 = -542$ psi (C)

$f_b = \frac{-30114}{14 \cdot 12} + 383 = +184$ psi (T)

long/E-W: $f_t = \frac{-33600}{14 \cdot 12} - 363 = -563$ psi (C)

$f_b = \frac{-33600}{14 \cdot 12} + 363 = +163$ psi (T)

allowable compr stress $= f_c = 1800$ psi $\gg f_b, f_t$ in both directions ✓ok.

Deflection Check:

$\Delta_u = \frac{5 M l^2}{48 E_c I_s}$

$\Delta_{u\,NS} = \frac{5(150774)(44.75)^2\, 12^2}{48(3605000)(2744)} = .46''$

$\Delta_{u\,EW} = \frac{5(142144)(49.5)^2\, 12^2}{48(3605000)(2744)} = .53''$

$\Delta_{avg\,md} = .54''$

$\Delta_{u\,allow} = \frac{l}{360}$

$\Delta_{all\,N\text{-}S} = \frac{44.75 \times 12}{360} = 1.49''$ both ok ✓

$\Delta_{all\,E\text{-}W} = \frac{49.5 \times 12}{360} = 1.65''$

Nominal Moment Strength:

$w_u = 1.2D + 1.6L = 1.2(115) + 1.6(125) = 338$ psf

recall: $l_s = 42.75'$ $l_L = 47.5'$ $k = l_L/l_s = 1.1$

From Fig 9.11 $\alpha_s = .064$ $\alpha_L = .048$

North-South Direction:

$M_u = \alpha_s w_u l_s^2 \times 12 = .064(338)(42.75)^2 \times 12 = 474\,406$ in-lb/ft

req'd $M_n = \frac{M_u}{\phi} = \frac{474{,}406}{0.9} = 527\,118$ lb-in/ft

$A_{ps} = .153\text{ in}^2$ @ 9" (=.80')

$A_{ps}/ft = .153\text{ in}^2/.80' = .19\text{ in}^2/\text{ft}$

$\rho_{N\text{-}S} = \frac{A_{ps}/ft}{12 \times 14} = .0011$ span-to-depth ratio $\frac{44.75 \times 12}{14} = 38.4$

$f_{ps} = f_{pe} + 10\,000 + \frac{f'_c}{300 \rho_p} \le f_{py} \le f_{pe} + 30000$

S. Bednarcik POST TENS. 4

$f_{ps} = 159\,000 + 10\,000 + \frac{4000}{300\cdot(.0011)} = 181\,121 < f_{py} = 240\,000$
$< 159\,000 + 30\,000 = 189\,000$ ✓ ok

$a = \frac{A_{ps} f_{ps}}{.85 f'c\, b} = \frac{(.153)(181\,121)}{.85(4000)(12)} = .68$ $d = 14 - (.5/2 + 3/4) = 13$

available $M_n = A_{ps} f_{ps}(d - a/2)$
$= .153(181\,121)(13 - .68/2) = 350\,828$ in.lb.

not good
need to: increase d to:

$.153(181\,121)(d - .68/2) = 527\,118 \rightarrow d = 19.4''$ $h = 20.5''$

verify:
$M_{n\,req'd} = 527\,118 < M_{n\,avail}$.

$M_{n\,avail}$: $\rho_{N-S} = .00079$ using $d = 19$ $h = 20$
$f_{ps} = 185\,878$
$a = .70$ Still meet all checks from above.
$d = 20 - 1 = 19$
$M_{na} = 530\,394 > M_{nreqd}$ ✓ ok

East-West Direction:
$M_u = .048(338)(47.5)^2 \times 12 = 439\,265 \rightarrow M_{n\,req'd} = \frac{M_u}{\Phi} = 488\,072$.

$A_{ps}/ft = .153/.72 = .21$
$\rho_{E-W} = \frac{.21}{12\times 20} = .00088$ span to depth $= \frac{47.5\times 12}{20} = 28.5$

$f_{ps} = 169\,000 + \frac{4000}{300(.00088)} = 184\,238 < 189\,000$
$< 240\,000$ ✓ ok

$a = \frac{.153(f_{ps})}{.85(4000)(12)} = .69$ $d = 19''$
$M_n = .153(184\,238)(19 - .69/2) = 525\,855 > M_{n\,req'd} = 488\,072$

✓ ok

Minimum Reinf ACI 318-11 Ch 18.

N-S.

In positive moment areas
$A_s = \frac{N_c}{.5 f_y}$ where N_c = tensile force

$f_t = 184$ $y = \frac{(184)(20)}{184 + 542} = 5.07''$
$f_c = 542$

$M_n = \frac{474\,406 \text{ in lb}}{1000 \times 12} = 39.5$ kft

S Bednarcik | POST TENS | 5

$N_c = \frac{39.5}{12 \cdot 20^2/6}\; 0.5\,(5.07'')(44.75)\,144 = 770.5K$

$A_{smin} = \frac{770.5}{.5\;\;60\,000} = 25.7\ in^2 \Rightarrow .60\ in^2/ft$

Try #7 @ 12" o.c. = .60 in²/ft. at midspan

verify:

$\phi M_n = (A_s f_y + A_{ps} f_{ps})(d - a/2)$

$[(.60)(60k) + (.1539 in^2/9''\ 12'')(185\,878)\,(19 - .70/2)$

$\Rightarrow M_{req'd} = 527$ 118 inft ✓ ok.

Negative M reinf. areas:

$A_s = .00075\ A_{cf}$ $\quad A_{cf} = (44.75 \times 12)(20'')$

$= 8.06\ in^2/$ $\quad = 10740$

$\Rightarrow .126\ in^2/ft$

#4s @ 12" o.c. = .20 in²/ft. at columns

drop panels.

S Bednarcik   POST TENS   6

Shear Strength:

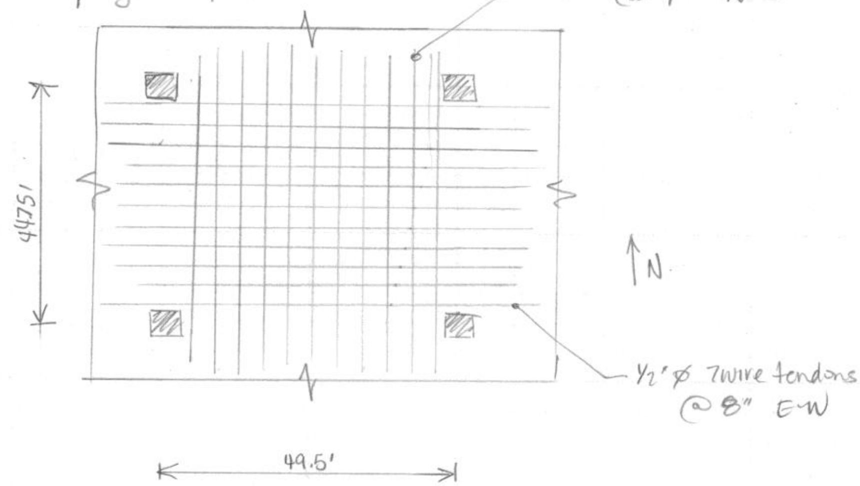
N-S. $V_u = \frac{1}{3} w_u l_s = \frac{1}{3}(338)\, 42.75' = 4817$ lb/ft

E-W $V_u = \frac{k w_u l_s}{2k+1} = \frac{1.2(338)(42.75)}{2.4+1} = 5100$ lb/ft

$V_c = 2\sqrt{f'_c}\, b_w d_p$
$= 2\sqrt{4000} \cdot 12 \cdot 20 = 30\,358$ lb / ft.

$\phi V_c = .75 V_c = 22768 \gg 4817$ lb/ft   ✓ ok in shear
$\gg 5100$

P.T. SLAB TYP DETAIL:
this doesn't show detailing of typ reinf at positive & negative positions.

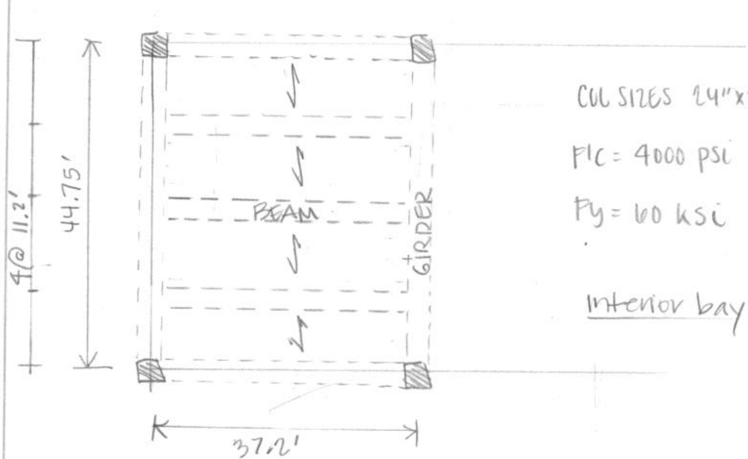


positive M   #7s @ 12" o.c.

Negative M   #4s @ 12" o.c.

Appendix 5: Alternate System: One-Way Slab on Beams

SBednarcik | ONE WAY SLABS | T2



COL SIZES 24"x24"

f'c = 4000 psi

fy = 60 ksi

Interior bay

Min Slab Thickness

Int Bay $l/28 = 11.2 \times 12/28 = 4.8"$ → use $t = 5"$

Assume #4 bars

$d = h - cc - d_b/2 = 5 - 3/4 - .5"/2 = 4"$

$W_D = (5/12")(150\ PCF) = 62.5\ PSF + 15\ PSF\ SDL = 77.5\ PSF$

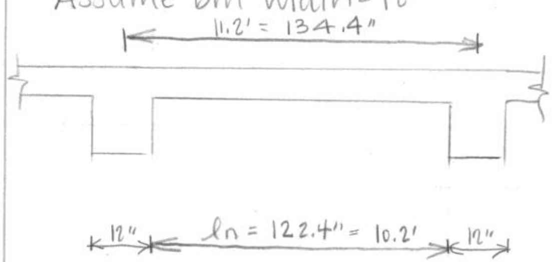
$W_L = 125\ PSF$

$W_u = 1.2(77.5) + 1.6(125) = 293\ PSF$

Assume tension-controlled $\phi = .9$

since $W_L < 3W_D$ can use ACI Moment Coeff.

Assume bm width = 12"



First Int Support:

$M_u = -\frac{W_u l_n^2}{10} = -\frac{293(10.2')^2}{10} \cdot 1'\text{ width} = 3048\ lb\cdot ft/ft = -3.05\ kft/ft$

S Bednarcik ONE WAY TR. 2

Second Int Support:

$$M_u = -\frac{W_u l_n^2}{11} = -\frac{293(10.2)^2}{11} = -2771 \text{ lbft/ft} = -2.77 \text{ Kft/ft}$$

Maximum neg design moment = −3.05 kft/ft

$$\text{Positive moment} = \frac{W_u l_n^2}{16} = \frac{293(10.2)^2}{16} = 1905 \text{ lbft/ft} = 1.91 \text{ k'ft/ft}$$

Reinforcement: Assume $jd = .9d$

$$A_s = \frac{M_u}{\phi f_y (d - a/2)} = \frac{M_u}{\phi f_y jd} = \frac{(3.05 \text{ kft})(12'')}{0.9(60)(.9)(4'')} = .19 \text{ in}^2/\text{ft}$$

$$a = \frac{A_s f_y}{.85 f'_c b} = \frac{.188 \cdot 60}{.85 \cdot 4 \cdot 12} = .276''$$

$$A_s \geq \frac{3.05 \cdot 12}{.9 \cdot 60 \cdot (4 - .276/2)} = .18 \text{ in}^2/\text{ft} \quad \checkmark \text{ ok}$$

$$\rho = \frac{A_s}{bd} = \frac{.18}{12 \cdot 4''} = .00365$$

Shear Check: at ext face of 1st int. support

$$V_u = \frac{1.15 W_u l_n}{2} = 1.15(293)(10.2)/2 = 1718 \text{ lb/ft} = 1.7 \text{ k/ft of slab}$$

at other supports

$$V_u = \frac{W_u l_n}{2} = (293)(10.2)/2 = 1494 = 1.5 \text{ k/ft width of slab}$$

normal wt conc.

$$\phi V_c = 0.75 \cdot 2\lambda\sqrt{f'_c}\, b_w d = 0.75(2)(1.0)\sqrt{4000} \cdot 12 \cdot 4 = 4554 \text{ lb/ft}$$

4.6 k/ft >> V_u ok

Design Reinforcement by ch 10 ACI 318-11

$$A_{s,min} = \frac{3\sqrt{f'_c}}{f_y} b_w d = \frac{3\sqrt{4000}}{60\,000} \cdot 12 \cdot 5 = .190 \text{ in}^2/\text{ft width of slab.}$$

$$A_{s,max} = \frac{200\, b_w d}{f_y} = .20 \text{ in}^2$$ ok for $A_{s\,used}$.

by 7.12.2.1

Temp reinf. transverse direction $\rho = .0018$ $A_s = .0018(12)(5) = .108 \text{ in}^2/\text{ft}$

spaced at min | 5h = 25" / 18" ← controls

we're okay ✓

S Bednarcik ONE WAY T2 3

	1st int. support	Int. Midspan	2nd int. support
l_n (ft)	11.2 →		
M coeff	$-1/10$	$1/11$	$1/16$
Mu (kft/ft)	-3.05	2.77	1.91
As req'd (in²)	.18	.16	.108
As min (in²) per ft width	.190 →		
Bars	#4 @ 12" →		
Final As	.20 →		

Spacing

$S_{max} = \min \begin{cases} 3h = 15'' \leftarrow \text{controls} \\ 18 \end{cases}$ $\quad f_s = 2/3 f_y = 40$ ksi

$S_{max} = 15\left(\frac{40000}{f_s}\right) - 2.5c_c = 15\left(40000/40000\right) - 2.5(3/4'') = 13.125$

$S_{min} = 12(40000/f_s) = 12''$

∴ 12" spacing is okay. §10.6.4 ACI 318-11

Transverse Direction

Req'rs As = .108 in²/ft using #4 bars @ 18"

5" SLAB:
#4 bars @ 12" o.c. TOP and BOT. flexural steel
#4 bars @ 18" o.c. for transverse reinf

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BEAM : INT. BAY, TYP.

$b = 18''$ by previous assumption

$h = \ell/21 = 37.2/21 = 1.77' \cong 22''$ Table 9.5

$W_{bm} = \frac{(22-5'')(18'')}{144} 150 = 212.5 = .2125\ k/ft.$ $h_{slab} = 5''$

$W_{slab+SDL} = (77.5\ PSF)(11.21) = 868 = .868\ k/ft$

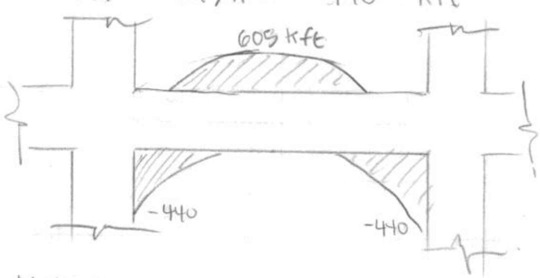
LL Reduction: Not allowed per ASCE7-05 §4.8.4

$W_L = 125\ PSF * 11.21'$
$= 1400\ PLF = 1.40\ k/ft$

$W_u = 1.2(.213 + .868) + 1.6(1.4) = 3.5\ k/ft$

$M_u^+ = \frac{wl^2}{8} = \frac{3.5(37.2')^2}{8} = 605\ kft$

$M_u^- = wl^2/11 = -440\ kft$



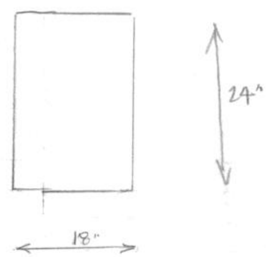
$d = h - cc - d_b/2$
$= 24 - 1.5 - 1.27/2 = 21.9$

Midspan

$A_s = \frac{M_u}{4d} = \frac{605}{4(21.9)} = 6.9\ in^2$

try (6) #10s $A_s = 6(1.27) = 7.62$ ✓ ok

one layer of #10s



$a = \frac{A_s f_y}{.85 f'_c b} = \frac{7.62 \cdot 60}{.85 \cdot 4 \cdot 18} = 7.5 \rightarrow c = a/\beta_1 = 8.8''$

$\varepsilon_s = \varepsilon_u \left(\frac{d-c}{c}\right) = .003\left(\frac{21.9-8.8}{8.8}\right) = .004 > .002$ ✓ ok

$\varepsilon_t \geq .005 \therefore \phi = .9$

S Bednarcik ONE WAY T2. 5

$M_n = A_s f_y (d - a/2)$
$= 7.62 (60)(21.9 - 7.5/2) = 8298 \text{ kin} = 692 \text{ kft}$

$\phi M_n = 622 \text{ kft} > M_u = 605 \text{ kft}$ Good

$A_{s,min} = \max \begin{cases} 3\sqrt{f'c}\, bwd/fy = 3\sqrt{4000}\ 18 \cdot 21.9/60\,000 = 1.25 \text{ in}^2 \\ 200\, bwd/fy = 200 \cdot 18 \cdot 21.9/60\,000 = 1.3 \leftarrow \text{controls} < A_s \text{ used} \end{cases}$ ✓ okay

At Supports

$M_u = -440 \text{ kft}$

$A_s = M_u/4d = 5.02 \text{ in}^2 \rightarrow$ try (4) #10s $A_{s,prov'd} = 5.08 \text{ in}^2$
$a = \frac{5.08(60)}{.85 \cdot 4 \cdot 18} = 4.98'' \rightarrow c = 5.86''$
$\varepsilon_s = .003/(21.9 - 5.86)/5.86 = .008 > \varepsilon_y$ ✓ ok

$M_n = 5.08 (60)(21.9 - 4.98/2) = 5916 \text{ kin} = 493 \text{ kft}$

$\phi M_n = 444 \text{ kft} > M_u = 440 \text{ kft}$ ✓ Good

also passes $A_{s,min}$.

Vertical Shear:

$A_v = .22 \text{ in}^2$

$V_u = \frac{W_u \ell_n}{2} = \frac{(3.11 \text{ k/ft})(37.2 - 2')}{2} = 54.7 \text{ k}$

$\phi V_n = \phi (V_c + V_s) = .75 [2\lambda \sqrt{f'c}\, bwd + 2 A_v f_y + d/s]$

$= .75 [2\sqrt{4000} \cdot 16 \cdot 20.73 + 2 \cdot (.2)(60000)(20.73/12'')] =$
$= 62.5 > V_u$
✓ ok

Good for both @ midspan and @ supports

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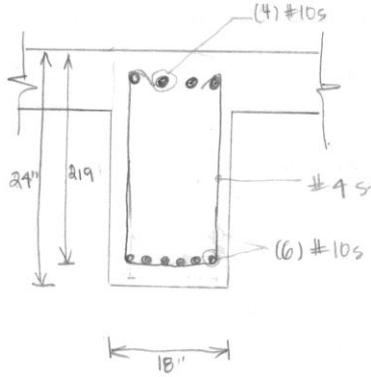
BEAM:



Deflection:

$\Delta_{u\,max} = \frac{\ell}{360} = \frac{37.2' \times 12}{360} = 1.24"$ $E = 57000\sqrt{4000} = 3.605E6$

$\Delta_{TL\,max} = \frac{\ell}{240} = \frac{37.2 \times 12}{240} = 1.86"$

$\bar{y} = 22 - \frac{(1/2)[(134.4-16)5^2 + 16(22^2)]}{(134.4-16)5 + 16(22)} = 11.4"$

$I_g = \frac{1}{12}(2(59.2)(5)^3 + 16(22)^3) + 2(59.2)5(11.4 - 5/2)^2 = 62{,}323 \text{ in}^4$

using $5/384\, w_u \ell^4 / EI$ as an approximate deflection check because I_g is being used, this will be more conservative

$\Delta_u = \frac{5 w \ell^4 1728}{384 EI} = \frac{5(1.078)(37.2)^4 1728}{384(3.605E6)62323} = .21"$ ✓ $\ll 1.24"$

$\Delta_{TL} = \frac{5(3.11)(37.2)^4 1728}{384(3.605E6)62323} = .60" \ll 1.86"$ ok ✓.

Also, by Table 9.5 (a) $h_{min} = \frac{\ell}{21} = 21.3" < 24"$ ok.

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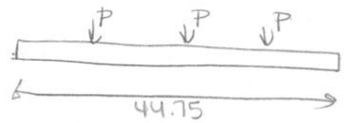
Girder Design:

Estimate Size:



$20 M_u = bd^2$

Assuming self weight = 10-15% of D.L.

$P_L = (1.078)\,37.2/2 = 20.1\,k$

$P_D = (3.11)\,37.2/2 - P_{LL} = 37.7\,k$

$W_{LL} = 125\,PSF * 37.2 = 4.7\ KLF$

$W_{DL} = 3P/L = 3(37.7)/(44.75) = 2.5\ KLF$

LL reduction cannot be used by § 4.8.4 ASCE 7-05

$W_u = 1.2(2.5)(1.15) + 1.6(4.7) = 11.0\ KLF$

$M_u = wl^2/12 = (11.0)(44.75)^2/12 = 1836\ k\,ft$

$20(1836\,k\,ft) = bd^2 = 28\,d^2$ ← try $b = 28''$

$b = 28''$

$d = 36''$ → $h = 38$

$W_{sw} = (38-5)(28)(150/144) = .96\ KLF$

→ $W_u = 10.5\ KLF$

$M_u^- = \frac{wl^2}{12} = \frac{10.5(44.75)^2}{12} = 1752\ k\,ft$

$V = wl/2 = 10.5(44.75)/2 = 235.\ k$

$M_u^+ = \frac{wl^2}{24} = 868\ k\,ft$

Reinf. Design: For Negative Moment

$A_s = \frac{M_u}{4d} = \frac{1752}{4 \cdot 36} = 12.2\,in^2$ → try (10) #10s = 12.7 in² 2 top row 8 bottom row

by Table A.7 fit ✓

$a = \frac{12.7(60)}{.85(4)(28)} = 8.0'' \rightarrow c = \frac{8.0}{.85} = 9.4''$

$\varepsilon_s = .003(36 - 9.4)/9.4 = .008 > \varepsilon_y$ ✓ ok

$\phi M_n = .9(12.7)(60)(36 - 8/2)\,1/12 = 1830\ K\,ft \geq M_u$ ✓ ok

use (10) #10s

8

Positive Reinf

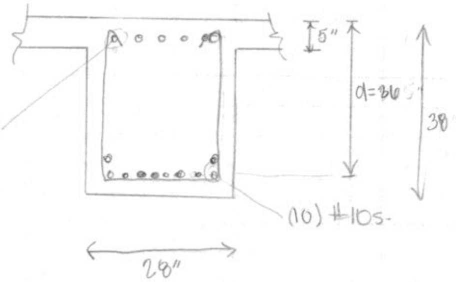
$A_s = \frac{848}{4.36} = 6.0\ in^2 \rightarrow$ try $(5)\#10 = 6.35 in^2$

$a = \frac{6.35 \cdot 60}{.85 \cdot 4 \cdot 28} = 4.0 \Rightarrow c = \frac{4.0}{.85} = 4.7$

$\varepsilon_s = .003\,(36 - 4.4)/4.7 = .020 > \varepsilon_y$ ✓

$\phi M_n = .9(6.35)(60)(36 - 4.\ /2)\ 1/12 = 972$ kft $> M_u$ ✓ ok

use (5) #9s



Deflection Check:

$\Delta_{u\,max} = \frac{l}{360} = \frac{44.75 \times 12}{360} = 1.5''$

$\Delta_{TL\,max} = \frac{l}{240} = \frac{44.75 \times 12}{360} = 2.2''$

$\bar{y} = 32 - \frac{(1/2)\left[(104-28)5^2 + 28 \cdot 32^2\right]}{(104-28)5 + 28 \cdot 32}$

$= 20.0'$

$I_g = \frac{1}{12}\left(2\left(\frac{104-14}{2}\right)5^3 + 28 \cdot 32^3\right)$

$+ 2\left(\frac{104-14}{2}\right)5\left(20 - 5/2\right)^2 = 211216$

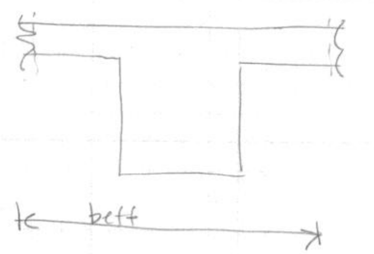
$\Delta_{LL} = \frac{wl^4}{384\,EI} = \frac{4.7\,(44.75)^4}{384 \cdot 57000\sqrt{4000} \cdot 211216} = .11''$

$= .11'' \ll 1.5''$ ok

$\Delta_{TL} = \frac{10.5\,(44.75)^4}{384\,EI} = .3'' \ll 2.2''$ ok

continuously cast.
fixed connections

treat as a T-beam cross section:



$b_{eff} = \min \begin{cases} 1/4\,(44.75) = 134'' \\ 24 + 16(5) = 104'' \text{ cont} \\ 24 + 1/2(42.75) = 281 \end{cases}$

use service loads

even considering creep, they deflections are well under limits

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Vertical Shear:

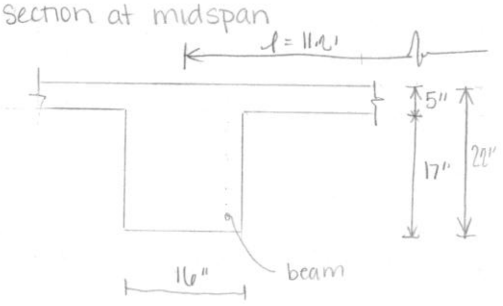
$$A_{v,min} = \max \begin{cases} 0.75\sqrt{f'_c}\, b_w s / f_y = .27 \text{ in}^2 \\ 50 b_w s / f_y = .250 \text{ in}^2 \end{cases}$$

use #5s $A = .3$

$$s = .3 (60) \frac{29.5}{172} \Rightarrow 3'' \text{ spacing}$$

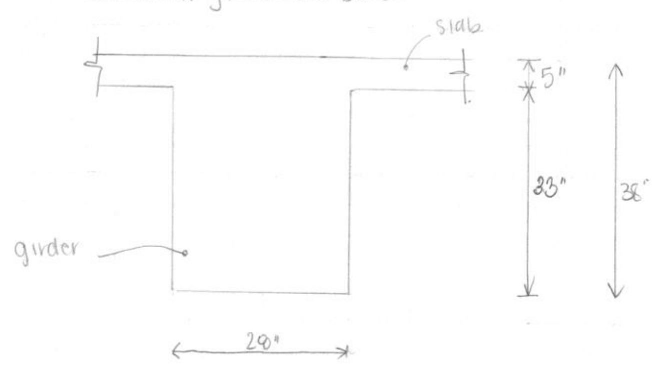
Depth

Section at midspan



reinf. not shown.

Section at girder and slab:



Appendix 6: System Comparisons

SBednarcik System Analysis T2 1

Weight Analysis

- Existing:
  - slab & decking 51 PSF
  - Steel framing W24x76 bm @ 7'6"
    W 30 x 90 girder @ 37'3

  Wsw = 51 PSF + 76 / 7.5' + 90 / 37.25 = 63.5 PSF

- System: Hollow Core

  1) planks 48.75 PSF + 25 PSF topping
  framing · bm W = 40x167 @ 16'5
  girder W = 40x149 @ 29.9'

  Wsw = 73.75 + 167 / 16.42 + 149/29.9 = 88.9 PSF

  2) planks 48.75 PSF + 25 PSF
  inv. T bms 1142 PLF @ 16'5

  Wsw = 73.75 + 1142/16.42 = 143.3 PSF

- System: Post-Tensioned

  Slab : 20"/12 depth (150 PCF) = 250 PSF
  includes rebar.

  Wsw = 250 PSF

- System: One-Way Slab
  - slab (5"/12")(150 PCF) = 62.5 PSF
  - bms [28] x [22-5"] x 150 @ 37.2' = 496 PLF @ 37.2'
  - girders [28] x [38 - 5"] x 150 @ 44.75' = 962.5 PLF @ 44.75'

  Wsw = 62.5 PSF + 496/37.2 + 962.5/44.75 = 97.4 PSF

S Bednarcik System Comparisons TR 2

Deflection Comparison

• Calculations presented in associated systems calcs. on previous pages

·Existing

$\delta_{TL} = 3.1''$

* $\delta_{SLL} = .77''$

* since camber included, comparing δ for LL

· System: Hollow Core (Layout B)

(1) $\delta_{planks,TL} = .47''$
$\delta_{bm\ LL} = 1.88''$ } at midspan = 1.35" ←
$\delta_{girder} = 1.38''$

(2) $\delta_{planks} = .47''$
$\delta_{IT32} = .89''$ ← controls.

·System: Post Tensioning

$\delta_{N\text{-}S} = .46''$

$\delta_{E\text{-}W} = .53''$ ← controls

· System: One-Way

$$\delta_{slab} = \frac{5(.293)(11.2)^4 1728}{384 \cdot 57000\sqrt{4000} \cdot \frac{1}{12}(12'')(5'')^3} = .23''$$

$\delta_{bm} = .60''$ ← controls

$\delta_{girder} = .30''$

S Bednarcik T2

Depth Comparison

• Original System:

slab & decking 5"
beam 23.9"
girder 29.5" ← controls

d = 34.5" = 2.9'

• System: Hollow Core

(1) planks 8"
beam 35.6" ← controls
girder 33.1"

d = 43.6" = 3.6'

(2) planks 8"
IT-beam 2'8" including planks
d = 2.7"

• System: Post-tensioned

Slab = 20" = 1.7" '

• System: One Way

slab 5"
beam 22"
girder 38"

d = 38" = 3.2"

Existing - Composite Steel					
System Components	Quantity	Unit	Cost per SF ($)		
			Material	Installation	Total
W24x55	37.2	LF	1.14	0.08	1.23
W24w76	198.0	LF	8.40	0.45	8.85
W16x31	49.5	LF	0.86	0.11	0.97
W30x90	22.4	LF	1.24	0.05	1.29
Welded Shear Connectors 3/4" diameter 3-7/8" long	240.5	Ea.	0.12	0.14	0.26
Metal decking, non cellular composite, galv. 2" deep, 20 gauge	2215.1	S.F.	1.83	0.47	2.30
Sheet metal edge closure form, 12" w/2 bends, 18 ga, galv	188.5	L.F.	0.09	0.09	0.17
Welded wire fabric rolls, 6 x 6 - W1.4xW1.4 (10x10), 21 lb/csf	22.2	C.S.F.	0.14	0.23	0.36
Concrete ready mix, normal weight, 3000 psi	20.5	CY	0.95	0.00	0.95
Place and vibrate concrete, elevated slab less than 6", pumped	20.5	CY	0.00	0.21	0.21
Curing with spread membrane curing compound	22.2	C.S.F.	0.07	0.06	0.13
Sprayed mineral fiber/cement for fireproof, 1" thick on beams	2215.1	S.F.	0.53	0.68	1.21
	Total SF	2215.13		Total ($/sf)	17.93

Hollow Core Plank with Steel Beams					
System Components	Quantity	Unit	Cost per SF ($)		
			Material	Installation	Total
Precast prestressed concrete roof/floor slabs 10" thick, grouted	2215.1	S.F.	7.40	0.97	8.88
Edge forms to 6" high on elevated slab, 4 uses	188.5	L.F.	0.01	0.23	0.24
Welded wire fabric 6x6 - W1.4xW1.4 (10x10), 21 lb/sf, 10% lap	22.2	C.S.F.	0.14	0.23	0.36
Concrete ready mix, regular weight, 3000 psi	13.7	CY	0.63	0.00	0.63
Place and vibrate concrete, elevated slab less than 6" pumped	13.7	CY	0.00	0.15	0.14
Curing with spreayed membraned curing compound	22.2	C.S.F.	0.07	0.06	0.13
Structural Steel - W33x130	134.25	LF	9.76	0.21	10.06
Sprayed mineral fiber/cement for fireproof, 1" thick on beams	2215.1	S.F.	0.53	0.68	1.21
	Total SF	2215.13		Total ($/sf)	21.65

Hollow Core Plank with Inverted T-Beams					
System Components	Quantity	Unit	Cost per SF ($)		
			Material	Installation	Total
Precast Concrete beam, T-shaped, 38' span, 40"x32"	2	Ea.	12.91	0.40	13.31
Precast prestressed concrete roof/floor slabs 10" deep, grouted	2215.1	S.F.	7.40	1.48	8.88
Edge forms to 6" high on elevated slab, 4 uses	188.5	L.F.	0.01	0.23	0.24
Forms in place, bulkhead for slab with keyway, 1 use, 2 piece	134.3	L.F.	0.11	0.25	0.36
Welded wire fabric 6x6 - W1.4xW1.4 (10x10), 21 lb/sf	22.2	C.S.F.	0.14	0.23	0.36
Concrete ready mix, regular weight, 4000 psi	13.7	CY	0.63	0.00	0.63
Place and vibrate concrete, elevated slab less than 6" pumped	13.7	CY	0.00	0.14	0.14
Curing with spreayed membraned curing compound	22.2	C.S.F.	0.07	0.06	0.13
	Total SF	2215.13		Total ($/sf)	24.06

Post Tensioned					
System Components	Quantity	Unit	Cost per SF ($)		
			Material	Installation	Total
Forms in place, flat plate to 15' high, 4 uses	2215.1	S.F.	2.06	8.32	10.38
Reinforcing in place, elevated slabs #4 to #7	2127.9	Lb.	0.51	0.26	0.77
Concrete ready mix, regular weight, 3000 psi	136.7	CY	6.36	0.00	6.36
Place and vibrate concrete, elevated slab over 10" thick, pump	136.7	CY	0.00	1.09	1.09
Cure with sprayed membrane curing compound	22.2	C.S.F.	0.07	0.06	0.13
Pre-Stressing Tendons	1703	Lb.	1.54	2.00	2.31
	Total SF	2215.13		Total ($/sf)	21.04

One Way Slab & Beam					
System Components	Quantity	Unit	Cost per SF ($)		
			Material	Installation	Total
Forms in place, flat plate to 15' high, 4 uses	1515.9	S.F.	0.94	3.79	4.73
Forms in place, interior beam. 12", 4 uses	1365.7	SFCA	0.81	4.47	5.28
Reinforcing in place, elevated slabs #4 to #7	1887.8	Lb.	0.60	0.31	0.91
Reinforcing in place, elevated beams #10	12504.5	Lb.	3.83	2.18	6.01
Concrete ready mix, regular weight, 4000 psi	26.28	CY	1.63	0.00	1.63
Place and vibrate concrete, elevated slab less than 6", pump	26.28	CY	0.00	0.36	0.36
Cure with sprayed membrane curing compound	0.26	C.S.F.	0.00	0.00	0.00
	Total SF	1664.70		Total ($/sf)	18.91