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.

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

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.](image)

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.](image)

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](image-url)

*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  
\[ f'c = 4000 \text{ psi @28 days} \]
Reinforcing Bars Plain-Steel  
\[ f'c = 3000 \text{ psi} \]
Other Concrete  
\[ f_y = 60 \text{ ksi} \]

Steel
W-Shapes  
\[ f_y = 50 \text{ ksi} \]
Channels, Angles  
\[ f_y = 36 \text{ ksi} \]
Plate and Bar  
\[ f_y = 36 \text{ ksi} \]
Cold-formed hollow structural sections  
\[ f_y = 46 \text{ ksi} \]
Hot-formed hollow structural sections  
\[ f_y = 46 \text{ ksi} \]
Steel Pipe

Other
Concrete Masonry Units  
\[ f'm = 1900 \text{ psi} \]
Mortar, Type M or S  
\[ f'm = 2500 \text{ psi} \]
Grout  
\[ f'm = 3000 \text{ psi} \]
Masonry Assembly  
\[ f'm = 1500 \text{ psi} \]
Reinforcing bars  
\[ f_y = 60 \text{ ksi} \]

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

This section details the provided design 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.

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 ASCE 7-05, the loading here would be 150 psf. Yet, the structural drawings note all live loads, excluding mechanical, at 100 psf. This could be due to overestimating other spaces, such as theatre spaces, and using an average, yet still conservative, value.

Live load reductions were not considered, as the SSPAC is considered under the “Special Occupancy” category, as a public assembly space, as per ASCE 7 -05 Chapter 4.8.4, and disallows the use of reduction factors on any live loads.

<table>
<thead>
<tr>
<th>Superimposed Dead Loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>Description</td>
</tr>
<tr>
<td>Concrete Masonry Units (CMU)</td>
</tr>
<tr>
<td>Prefabricated Concrete Panels (8&quot; thick)</td>
</tr>
<tr>
<td>Glazed Aluminum Curtain Walls</td>
</tr>
<tr>
<td>Roofing</td>
</tr>
<tr>
<td>Framing</td>
</tr>
<tr>
<td>MEP Allowance</td>
</tr>
</tbody>
</table>

Table 11: Table of Superimposed dead loads.

<table>
<thead>
<tr>
<th>Live Loads*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Space</td>
</tr>
<tr>
<td>Live Load</td>
</tr>
<tr>
<td>Corridor</td>
</tr>
<tr>
<td>Corridor, above 1st floor</td>
</tr>
<tr>
<td>Stairway</td>
</tr>
<tr>
<td>Mechanical Room/Light Manufacturing</td>
</tr>
<tr>
<td>Roof</td>
</tr>
<tr>
<td>Lobby</td>
</tr>
<tr>
<td>Theatre, stationary seating</td>
</tr>
<tr>
<td>Stage Floor</td>
</tr>
<tr>
<td>Restaurant/dining space</td>
</tr>
<tr>
<td>Balcony</td>
</tr>
</tbody>
</table>

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.

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.

<table>
<thead>
<tr>
<th>Variable</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof Snow</td>
<td>30 + Snow Drift</td>
</tr>
<tr>
<td>Ground Snow - Pg</td>
<td>30 (psf)</td>
</tr>
<tr>
<td>Flat Roof Snow - Pf</td>
<td>30 (psf)</td>
</tr>
<tr>
<td>Terrain Category</td>
<td>B</td>
</tr>
<tr>
<td>Snow Exposure Factor - Ce</td>
<td>1.0</td>
</tr>
<tr>
<td>Snow Load Importance Factor - Is</td>
<td>1.2</td>
</tr>
<tr>
<td>Roof Thermal Factor - Ct</td>
<td>1.0</td>
</tr>
<tr>
<td>Roof Slope Factor - Cs</td>
<td>1.0</td>
</tr>
</tbody>
</table>

Table 13: Summary of snow load variables.

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

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.

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 in 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.

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.

**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.

![Diagram](image)

*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.

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.

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

1. ELEVATION AT LINE C

Scale: 1"=1'-0"

Notes:
1. Connections to be designed for forces indicated by fabricator's engineer.
2. (') denotes axial force in member
   (') tension
   ('') compression
3. (+) denotes vertical reaction on end of beam
Appendix 2: Existing: Composite Slab and Decking on Composite Beams

Table 3.40 AISC

\[ \text{Bending moment factor} = \frac{30\text{in} \times 17.4}{28.74} = 7.6 \rightarrow \theta = 90^\circ \]

**BM OPTIONS:**

- **W12 x 41**
  - **Min. =** \( 9.9 \times 6.74 \)
  - \( 80 \times n = 577 \)
  - \( n = \frac{577}{7.6} \approx 74 \) + 60 studs

- **W12 x 48**
  - \( 9.9 \times 6.74 > 74 \)
  - \( 948 \)
  - \( n = \frac{948}{7.6} \approx 123 \) → 48 studs

**BM EQUIV: WL**

\[ \text{WL} = \frac{W12 \times 41}{W21 \times 41} \]

- \( W21 \times 41(498) + 65(10) = 2985 \) → best stud thus
- \( 48 \times 52(10) = 2894 \)
- \( 50 \times 61(10) = 2935 \)
\[
a = \frac{2 \text{ in}}{.65 \text{ ft load}} = \frac{1184}{65 (4)(24)} = 2.5 > 1.0 \text{ NG.}
\]

allowed load/ft:

44 studs \( \Rightarrow \) 17.2 (44) = \( \frac{84}{44} \) = 44.8

Solve for \( a \):

\[
\text{Eqn} = \left( \frac{11.85}{99.95} \right) (4/90) = 2.0 \Rightarrow 90 \text{ with } W24\times14
\]

\( \text{Eqn} = 280 \) \( \Rightarrow \) 34 studs

 required per deflection

\[
I_w = \frac{57.94}{99.5} (4/19) = 2554 \text{ in}^4 \text{ reqd.}
\]

Using \% of def for UC

\[
\text{check: } \frac{0.994 (4/19)}{99.5} < 1.0^\circ
\]

\[
\Rightarrow \text{ok}
\]

check:

\[
a = \frac{2.72}{85 (4/90)} = .92 < 1.0 \text{ ok}
\]

for a \( W24\times14 \)

\[
I_w = 57.94 = 9.5 \Rightarrow \text{ use } 9.5
\]

\( I_w = 3810 > 2554 \text{ vok} \)

Good by deflection:

\[
\text{check unshored strength}
\]

\[
0.75 P = 750 \text{ \( W24\times14 \)}
\]

\[
W_{sc} = 1.4 (7.6) + 1.4 (4.9) (1.5) = 177 \text{ KLF}
\]

\[
W_{sc} = 1.2 (7.6 + 3.5 (1.5) + 1.4 (20 (1.5)) = 188 \text{ KLF} \text{ v olk treatment.}
\]

\[
W_{sc} = \frac{(188) (49.5)^2}{8} < 269.5 \text{ vok for unshored}
\]

Wet (wet def calc)

\[
W_{wc} = 63 (7.5) + 7.6 = 544 \text{ KLF}
\]

\[
8_{wc} = \frac{5 (1044) (49.5)^{1/2}}{58420000 \cdot 3310} = .77^\circ < 2.5^\circ = \frac{L}{240}
\]

\[
S_{tot} = \frac{5 (22) (49.5)^{1/2}}{38420000 \cdot 3310} = 21^\circ \text{ total deflection}
\]

OK \( W24\times14 \) at 34 studs \( \checkmark \)
Appendix 3: Alternate System 1: Hollow Core Planks

Original beam spacing max at 7' 4".

Consider layout:

X x 4' planks

Replace beams with one running N-S:

Planks spanning E-W

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

Design w/ max span @ 25'

\[ W_0 = 12 \times 14 \times 0.7 = 145 \text{ psf} \]

\[ M_{max} = W_0 \times 9.1 = 1318 \text{ ft-lb} \]

\[ S_{w} = 6.5 \text{ psf from Nutterhouse Charts} \]

Will control in flexure, deflection: check.

Flexure:

\[ f = \frac{M}{I} + \frac{Pc}{c} = \frac{-917.5}{1540} - \frac{2.4 \times 6}{387} + \frac{2.4 \times 6}{12} = 572 \text{ psf} \]

\[ C_1 = 3.27 \]

\[ C_2 = \frac{1619 - 175}{175} = 4.4 \]

Total load:

\[ 572 < 0.6 \times 6.5 \times 3870 = \text{OK} \]

by ACI 8.4.2

Deflection:

\[ \Delta_1 = \frac{5W_0}{384E} = \frac{5(145)(1.728)}{384 \times 1415} = 0.31" \ll 0.83" = W \]

\[ \Delta_2 = \frac{fL^4}{384E} = \frac{5.12 \times 105 + 0.63 + 1.8 \times 1.728}{384 \times 1415} = 0.47" \ll 1.25" = W \]

Moment:

\[ M_{max} = W \times 12 = 13.6 \text{ k-ft} \ll 16.8, 11 \text{ ft from Nutterhouse} \]

Use 10" x 4' 2" topping and 1 hr fire rating.
Beam Design:

\[ V = \frac{1}{2} (2\text{Ew}) + 1.6 (175) = 308 \text{ psf} \]

\[ = 308 (21.8^2) = 6.7 \text{ kft} \]

II reduction: not allowed by 540.4

\[ W = 4.4 \text{ kK} \]

\[ V_a = W/2 = 150 \text{ k} \]

\[ M_a = W/4 = 87.5 \text{ kft} \]

\[ I_{a,LT} = \frac{5 W_a L^4}{384 E (d/300)} = 9179 \text{ in}^4 \]

\[ I_{a,LT} = \frac{3 d}{12490} = 93.17 \text{ in}^4 \text{ controls} \]

Shape:

<table>
<thead>
<tr>
<th>Material</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>W 40 x 149</td>
<td>1300</td>
</tr>
<tr>
<td>W 35 x 120</td>
<td>1100</td>
</tr>
<tr>
<td>W 27 x 93</td>
<td>750</td>
</tr>
<tr>
<td>W 27 x 93</td>
<td>675</td>
</tr>
</tbody>
</table>

by Table 3-9 AISC

most economical of listed

\[ \frac{155}{25} = 6.2 \leq 7' \]

check sw:

\[ \frac{155}{25} = 6.2 \leq 7' \]

use W40x149 supporting planks

Table 3-2

<table>
<thead>
<tr>
<th>Material</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>M 14 x 10</td>
<td>240</td>
</tr>
<tr>
<td>M 12 x 10</td>
<td>175</td>
</tr>
<tr>
<td>M 10 x 8</td>
<td>111</td>
</tr>
</tbody>
</table>

(deflection controls span)

Try varying bay layout:

Beams spaced at 16.5'

Span for planks = 16.5'

Same base loads as before

Try 6" x 4" (6) ½ 4 strands

2" topping 1hr fire rating

Using \( L = 16.5' \rightarrow W = 210 \text{ psf} \)

S.W. = 49.76 psf

\[ M_a = W L^2 = 354 \text{ kK} \]
will control in flexure, deflection:

\[ f_t = \frac{354 - 248(6) + 248(6)(7.5)}{797} = -598 \text{ psi} < 596 \text{ psi} \]

\[ p = 122 \times 12 = 244 \text{ psi} \]

\[ A_c = 153 \text{ in}^2 \quad \text{of} \quad \frac{1}{2}" \]

\[ e = 4.10 - 1.75 = 2.35 \quad \text{in} \]

\[ E = 4415 \text{ ksi} \]

\[ \Delta u = \frac{5}{384} \left[ \frac{1.0(15+48.75)}{(15+48.75)(12)} \right] \frac{(6)}{(128)} = 0.6" < 0.55" \frac{1}{340} \]

\[ \Delta u = -\frac{5}{384} \left[ \frac{1.0(15+48.75)+1.0(12)}{(15+48.75)(12)} \right] \frac{(6)}{(128)} = 0.08" < 0.83" \frac{1}{240} \]

Moment:

\[ M_n = \frac{Wl^2}{8} = \frac{[1.0(15+48.75)+1.0(12)](128)(6)}{8} = 57.6 \text{ k-ft} < 92.6 \text{ k-ft} \]

Use 6" x 4" 2" topping & 1/2" fire rating.

Design Beams:

\[ W_n = 1.7(20+48.75) + 1.0(12) = 289.4 \text{ psf} \]

\[ f = (289.4)(11.4) = 3244 \text{ psi} \]

LL reduction not allowed by 8.4.2.4.

\[ W_n = \frac{[1.0(70.75)+1.0(12.5)](128)}{8} = 5.0 \text{ k-ft} \]

\[ M_n = 3.5 \text{ k-ft} \]

\[ W_n = 1.0(112.7) \quad \text{in}^2 \]

\[ h = 1252 \text{ k-ft} \]

Isec = \[ \frac{5W_n}{6} \left( \frac{1}{128} \right) \]

\[ = 953 \text{ in}^4 \]

Shape

| W80 x 185 | 7500 |
| W34 x 299 | 7650 |
| W33 x 141 | 7950 |

[Table 3-3 AISC]

\[ \text{TKE} \rightarrow \text{CHECK MN 112.7 \quad 7 \leq 7 \quad \checkmark} \]

\[ \phi \text{ Min} = 1252 \text{ k-ft} \]

\[ \phi \text{ Vn} = 577 \text{ k-ft} \]

\[ \phi \text{ Mn} = 112.7 \text{ k-ft} \]

\[ \phi \text{ Vn} = 577 \text{ k-ft} \]

\[ \text{TKE} \text{ Min} = 1252 \text{ k-ft} \]

\[ \text{TKE} < 577 \text{ k-ft} \]

\[ \text{Table 3-1} \]

\[ \phi \text{ Vn} = 577 \text{ k-ft} \]

\[ \phi \text{ Mn} = 112.7 \text{ k-ft} \]

\[ \phi \text{ Vn} = 577 \text{ k-ft} \]

\[ \phi \text{ Mn} = 112.7 \text{ k-ft} \]

\[ \text{TKE} < 577 \text{ k-ft} \]
Now, using Layout B, design girder:

- Realigning column lines to match girder spacing
- Consider load as distributed

GIRDER A:

\[ x = 16.5' \]

\[ A = 372' \]

\[ \text{P}_{u} = \text{Wu} \left( \frac{44.75}{h} \right) \]

\[ \text{Pu} = 78 \text{ kips} \]

\[ \text{Wu}^2 = \left( \frac{112}{2} \right)^2 \cdot 37.2 = 6.0 \text{ klf} \]

\[ \text{Wu}^2 = 3.8 \text{ klf} \]

\[ M_u = 1038 \text{ kipt} \]

\[ V_u = 112 \text{ kips} \]

\[ \text{E} = 29,000 \text{ kpsi for steel} \]

\[ \text{I}_{xx} = 5 \text{ Wu}^3 \left( \frac{4}{12} \right) \]

\[ \text{I}_{xx} = 5 \times 5191 \text{ in}^4 \]

\[ \text{by Table 3-3} \]

\[ \text{Size 1} \]

\[ W_{35} 	imes 150 \text{ most economical sizes} \]

\[ W_{35} 	imes 150 \text{ provings} \]

\[ \text{Check:} \]

\[ f_{\text{Min}} = 1950 \text{ ksi} > 1038 \text{ kipt} \]

\[ f_{V_u} = 516 \text{ kips} > 112 \text{ kips} \]

\[ \text{Check:} \]

\[ f_{\text{Min}} = 1.53' \]

USE W_{35} \times 150

Layout B:

Overall depth:

- 2''
- 8.5''
- 33.4''

At spans, controls

Original system depth:

(from tech 1 spot checks)

- 5''
- 2.4''
- Total depth = 28.7'' = 2.4'
Design Inverted T-beam instead of Steel Beam:

**System(2)**

**Layout-B**

- Same spacing as layout B

\[ Fc = 6000 \text{ psi} \]

- \( Wd = 3.7 \text{k} \)
- \( V = 7.8 \text{k} \)
- \( M = 9.2 \text{k} \)
- \( L = 11.1 \text{ ft} \)
- \( k = 11.1 \text{ k} \)

**Spacing:** 44.75\(^\circ\)

**Use:** 46\(^\circ\)

- \( \text{Min} = 144 \text{ k} \)

**Span:**

- \( L = 4.9 \text{k} \)
- \( > 22 \text{k} \)

**Reinforcement:**

- \( \text{Strand pattern: 16-6} \)
- \( \text{top bars: 6-9} \)

\[ \text{Min} = 144 \text{ k} \geq 11.1 \text{ k} \]

**Span:**

- \( L = 4.9 \text{k} \)
- \( > 22 \text{k} \)

**Deflection:**

- \( \delta = L \cdot (12) \cdot E \cdot I \)

- \( \delta = 0.53 < 1.50 \text{ in} \)

**Moment:**

- \( M = 920 \text{ k} \cdot \text{ft} \)
- \( 1110 \text{ k} \cdot \text{in} \)

**Check:**

- \( Fc = 4415 \text{ k} \)

**Use:** 46\(^\circ\)

- \( \text{Total depth: } 2.8'' = 2.7'' \)

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
\( A_c = 253 \text{ in}^2 \)  \( \text{Precast} \ b_w = 16.13 \text{ in.} \)
\( L_c = 1519 \text{ in.} \)  \( \text{Precast} \ S_{x0} = 370 \text{ in}^3 \)
\( \gamma_{c0} = 4.10 \text{ in.} \)  \( \text{Topping} \ S_{x0} = 551 \text{ in}^3 \)
\( \gamma_{c0} = 1.90 \text{ in.} \)  \( \text{Precast} \ S_{z0} = 799 \text{ in}^3 \)
\( \gamma_{c0} = 3.90 \text{ in.} \)  \( \text{Precast} \ WL = 195 \text{ PLF} \)
\( \text{Precast} \ Wt. = 48.75 \text{ PSF} \)

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 \( \sqrt{f_c} = 775 \text{ PSI} \)
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

<table>
<thead>
<tr>
<th>Strand Pattern</th>
<th>SPAN (FEET)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30</td>
</tr>
<tr>
<td>4 - 1/2&quot;Ø LOAD (PSF)</td>
<td>349 317 290 258 227 197 179 157 148 131 110 91 75 60</td>
</tr>
<tr>
<td>6 - 1/2&quot;Ø LOAD (PSF)</td>
<td>524 476 437 377 334 292 269 237 224 183 166 142 122 104 88 73 61 49 39</td>
</tr>
<tr>
<td>7 - 1/2&quot;Ø LOAD (PSF)</td>
<td>541 492 451 416 364 331 293 274 242 214 190 167 144 124 107 91 77 64 53</td>
</tr>
</tbody>
</table>

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, f a n g e o r s t e m openings and narrow widths.  The allowable loads shown in this table reflect a 1 Hour & 0 Minute fire resistance rating.

6F1.0T

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

11/03/08
Prestressed Concrete
Inverted Tee Beam 40IT32-B

PHYSICAL PROPERTIES
A = 1,096 in.²
I = 84,622 in.⁴
Y₅ = 14.28 in.
Y = 17.72 in.
Sₐ = 5,926 in.³
Sᵢ = 4,775 in.³
Wₕ = 1,142 PLF

DESIGN DATA
1. Precast Strength @ 28 days = 6,000 PSI
2. Precast Strength @ release = 4,000 PSI.
3. Precast Density = 150 PCF
4. Strand = 0.80"Ø 270K Lo-Relaxation.
5. Ultimate load shown below is for full strand development & tension controlled section.
6. Maximum bottom tensile stress is 12√(fc) = 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...

   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 to ensure they are not exceeded.
12. The concrete strength at release of prestress force increments to 4,500 psi for 22 strands.

<table>
<thead>
<tr>
<th>ALLOWABLE SUPERIMPOSED LIVE LOADS (KLF)</th>
<th>IBC 2006 &amp; ACI 318-05 (1.2 D + 1.6 L)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strand Pattern</td>
<td>Top Bars</td>
</tr>
<tr>
<td>8 - 0 - 0</td>
<td>2 - #9</td>
</tr>
<tr>
<td>16 - 0 - 0</td>
<td>6 - #9</td>
</tr>
<tr>
<td>16 - 8 - 0</td>
<td>6 - #9</td>
</tr>
</tbody>
</table>

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...
Appendix 4: Alternate System: Post Tensioned Slab

Following Prelim Analysis & Design by Pei's Design of Post-Tensioned Slabs Using Unbonded Tension

Two Way Slab: Thickness:

\[ \text{Span} \cdot \text{Depth} = \frac{45}{45} \]

\[ h = \frac{45}{45} \times 13.2'' \rightarrow \text{trial depth} 14'' \]

LOADINGS - See above.

Section Properties

- Slab S18.3.3 or AC 318-11
- Class 1 - LL heavier, longer spans:
  - \( f_{cu} = 3000 \text{ psi} \)
  - \( f_{ps} = 159,000 \text{ ksi} \)
  - \( f_{yy} = 190,000 \text{ psi} \)
  - \( f_{yp} = 189,000 \text{ psi} \)
  - \( f_{Ec} = 57,000 \text{ psi} \)
  - \( E_W \) max: \( f_c \leq 4000 \text{ psi} \)
  - \( E_N \) max: \( f_c \leq 2000 \text{ psi} \)

Balanced Loadings

for no deflection or camber

\[ \text{Load} = W_0 = 15 \text{ PSF} \text{ max} = 115 \text{ PSF} \]

Effective Prestress:

\[ \text{Pl: } f_c \cdot h \cdot 14' = 98,000 \text{ lb/strip} \]

\[ W_{bal} = \frac{8 \cdot P_{pl} \cdot f_c}{L_i} = \frac{8 \cdot (98,000) \cdot (14')}{(49.6')^2} = 54,850 \text{ PSF} \]

\[ W_{bal} (s) = W_0 - W_{bal} = 115 - 54,850 = 60,150 \text{ PSF} \]
SteelStacks Performing Arts Center | Bethlehem, Pennsylvania

12 October 2012 | Tech Report II

Sarah Bednarcik | Structural Option

**Test Tens.**

\[
P_s = \frac{W_{Load}}{S_3} = \frac{6045}{44.75^2} \times 12 = 1201.4 \text{ lb/ft}
\]

\[
fe (N-S) = \frac{P_s}{bh} = \frac{1201.4}{12 \times 14} = 9.79 \text{ ksf} < 980 \text{ psi OK}
\]

Use \( \frac{1}{6} \) in \#14 wire 400K strands \( A = 0.159 \text{ in}^2 \) by Acisite

\[
w_1 = \frac{P}{fe} = \frac{159000}{0.159} = 249,947 \text{ lb}
\]

**Reinforcement Spacing:**

\[
S_3 = \frac{P}{P_s} = \frac{1201.4}{44.75} = 0.80 \text{ ft} = 9.60'' \to 9''
\]

\[
S_1 = \frac{P}{P_t} = \frac{1201.4}{54.83} = 0.22 = 8.46'' \to 8''
\]

Recommended spacing: \( 3h = 3(14) - 92'' \) under max range \( \frac{b}{h} = \frac{5(14)}{92} = 70'' \)

**Service Load Stresses:**

\[
W_L = 125 \text{ psi}
\]

\[
k = \frac{W_L}{S_3} = \frac{49.95}{44.75} = 1.11
\]

by Fig 9.10, see for values: \( \alpha_y = \alpha_{E-W} = 0.42 \quad \text{long} \)

\( \alpha_x = \alpha_{N-S} = 0.055 \quad \text{short} \)

\[
L_{self} = 44.75'' \times \frac{4}{18} = 14.75''
\]

\[
L_{eff} = 49.95'' - 14.75'' = 35.2''
\]

**LL Moments:**

\[
M_s = \alpha_x S_3 \times W_L \times L_3^2 \times 12
\]

\[
= 0.055 \times (125)(44.75)^2 \times 12 = 150,774 \text{ lb-in/ft}
\]

\[
M_i = 0.042 (125)(44.75)^2 \times 12 = 142,144 \text{ lb-in/ft}
\]

**Moment of Inertia:**

\[
I_i = I_1 \left( \frac{14}{12} \right)^3 = 2744 \text{ in}^4 = I_0
\]

**Concrete Stresses due to LL**

**Short:**

\[
f_s = \frac{M_s}{I_0} = \frac{150,774}{2744} = 55.4 \text{ psi}
\]

**Long:**

\[
f_l = \frac{M_i}{I_0} = \frac{142,144}{2744} = 51.8 \text{ psi}
\]

\[
f_t = \frac{P}{bh} - \frac{M_s}{I_0} - \frac{M_s}{I_0} = \frac{P}{bh} - \frac{M_s}{I_0}
\]

\[
f_{ab} = \frac{P}{bh} - \frac{M_s}{I_0}
\]
S. Bednarcik

POST TENS.

Short/N-S:
\[ f_t = \frac{-30114}{14.12} - 383 = -542 \text{ psi (C)} \]
\[ f_b = \frac{-30114}{14.12} + 383 = +184 \text{ psi (T)} \]

Long/E-W:
\[ f_t = \frac{-33400}{14.12} - 383 = -565 \text{ psi (C)} \]
\[ f_b = \frac{-33400}{14.12} + 383 = +163 \text{ psi (T)} \]

Allowable Compressive Stress:
\[ f_c = 1800 \text{ psi} \]

Deflection Check:
\[ \Delta u = \frac{5Mw^2}{48E I_x} \]
\[ \Delta u_{NS} = \frac{5(15074)(49.75)^2 10^6}{48(365000)(2.744)} = 0.46" \]
\[ \Delta u_{EW} = \frac{5(142144)(49.75)^2 10^6}{48(365000)(2.744)} = 0.53" \]
\[ \Delta u_{NW} = 0.54" \]
\[ \Delta u_{N-S} = \frac{44760 \times 10}{360} = 1.24" \text{ both OK} \]
\[ \Delta u_{E-W} = \frac{44760 \times 10}{360} = 1.26" \]

Nominal Moment Strength:
\[ Wu = 1.2D + 1.6L = 1.2(115) + 1.6(115) = 338 \text{ ft-lb} \]
Recall: \[ L_s = 49.75', \quad L = 47.5', \quad k = \frac{L_s}{L} = 1.1 \]
From Fig 9.11: \[ \phi_s = 0.64, \quad \alpha = 0.48 \]

North-South Direction:
\[ M_u = \phi_s Wu \times L_s^2 / 12 = 0.644(338)(49.75)^2 / 12 = 474 \text{ ft-lb} \text{ (Min)} \]
\[ Aps = 0.15^2 \text{ in}^2 / 12 = 0.0012 ft^2 \]
\[ Aps/ft = 0.15^2 / 120 = 0.19 \text{ in}^2/ft \]
\[ Pw/S = Aps/ft \cdot 0.001 \quad \text{span-to-depth ratio} \quad \frac{49.75 \times 10}{14} = 36.1 \]
\[ fps = f_p + k'000 + \frac{f_c}{300} \leq f_p \leq f_p + 30000 \]
S. Bednarcik

POST TENS

\[ f_p = \frac{159,000 + 10,000 + \frac{4000}{300 \cdot (0.011)}}{181,181} \leq f_{pr} = \frac{340,000}{159,000 + 30,000} = 189,000 \]

\[ a = \frac{A_p f_p}{b \cdot 0.85} = \frac{185 (181,181)}{0.85 (4000)(0.11)} = 108 \]

\[ d = 14 - \left( \frac{15}{4} + 0.1 \right) = 13 \]

Available Mn = Aps \( f_p (d - 0.5) \)

\[ = 153 (181,181)(13 - 0.68/2) = 950,888 \text{ in lb.} \]

Not good

Need to increase \( d \) to:

\[ 115.5 (181,181)(13 - 0.68/2) = 927,118 \rightarrow d = 19.4'' h = 0.5'' \]

Verify:

\[ M_{reqd} = 590,344 < M_{avail}. \]

\[ M_{reqd} = 590,344 < M_{avail}. \]

East-West Direction:

\[ M_{uc} = \frac{0.45(338)(47.5)(72)}{12} = 439,265 \rightarrow M_{uc} = \frac{M_{reqd}}{3} = 488,072. \]

\[ Aps/ft = 153/72 = 2.1 \]

\[ P_{cw} = \frac{21}{12 \times 20} = 0.00088 \text{ span to depth} = 47.5 \times 12 = 28.5 \]

\[ f_p = \frac{110,000 + 4000}{300 (0.0088)} = 184,238 \leq 189,000 \leq 240,000 \text{ ok} \]

\[ a = \frac{153(f_p)}{0.85(4000)(0.11)} = 0.61 \]

\[ d = 19'' \]

\[ M_{uc} = 153 (181,238)(19 - 0.68/2) = 525,855 \rightarrow M_{uc} = 488,072 \text{ reqd} \]

Minimum Reinf. AC1 318'11 Ch18:

\[ A_s = \frac{N_e}{f_{y}} \text{ where } N_e = \text{tensile force} \]

\[ f_t = 184 \]

\[ f_c = 540 \]

\[ y = \frac{(184)(12)}{184 + 540} = 5.01'' \]

\[ N = \frac{474,400 \text{ in lb.}}{1000 \times 10} = 34.647 \text{ kips} \]

\[ N_e = \frac{f}{1000} \text{ kips} \]
\[ N_c = \frac{39.5}{18.10^{3/6}} \]

\[ A_{min} = \frac{770.5}{60000} = 25.7 \text{ in}^2 \Rightarrow 0.60 \text{ in}^2/\text{ft} \]

\[ I_{ty} = \frac{7}{12} @ 10 \text{ in} \Rightarrow 0.60 \text{ in}^3/\text{ft} \]

\[ M_{min} = 527 \text{ in}^3/\text{ft} \]

Negative M remote areas:

\[ A_s = 0.00076 \times A_{cf} \]

\[ = 8.06 \text{ in}^2/\text{ft} \]

\[ \Rightarrow 0.80 \text{ in}^3/\text{ft} \]

\[ 4S @ 10 \text{ in} \Rightarrow 0.90 \text{ in}^3/\text{ft} \]

drop panels...
Shear Strength:

N-S: \[ V_u = \frac{1}{3} W_u L_u = \frac{1}{3}(198)(44.75) = 481.7 \text{ lb/ft} \]

E-W: \[ V_u = \frac{L_u W_u L_u}{2K+1} = \frac{1.9(198)(44.75)}{9.41} = 5100 \text{ lb/ft} \]

\[ V_c = 20\% L_u W_u \text{ lb/ft} \]

Positive Moment:

\[ +75 \text{ ft-lbf @ 12' 0" O.C.} \]

Negative Moment:

\[ -45 \text{ ft-lbf @ 12' 0" O.C.} \]
Appendix 5: Alternate System: One-Way Slab on Beams

Min Slab Thickness

\[ \text{Int Bay} \quad \frac{d}{2} = \frac{11.2 \times 10.28}{2} = 4.8^\prime \rightarrow \text{Use} \quad t = 5^\prime. \]

Assume #4 bars,

\[ d = h - c_c - d/2 = 5 - \frac{3}{4} - \frac{1}{2} = 4^\prime. \]

\[ W_b = \frac{3}{8}(150 \text{ psi}) = 62.5 \text{ PSF} + 15 \text{ PSF SOL} = 77.5 \text{ PSF} \]

\[ W_L = 115 \text{ PSF} \]

\[ W_u = 1.2(77.5) + 1.6(125) = 293 \text{ PSF} \]

Assume tension-controlled \( \phi = 0.9 \)

Since \( W_u < 3W_b \), can use ACI moment coeff.

Assume beam width = 12^\prime

\[ 11.2 = 12 + 4^\prime \]

First int Support:

\[ M_n = \frac{W_u L_n^2}{10} = \frac{293(10.28)^2}{10} \text{ ft}^2 \text{ in}^2 = 304.8 \text{ kN/m} \text{ ft}^2 = 3.05 \text{ kN/m} \text{ ft} \]
Sarah Bednarcik | Structural Option

Second Int Support:

\[ M_n = \frac{W_u h_n^2}{11} = \frac{-293(10.2)^2}{11} = -2771 \text{ lbft/ft} = -2.77 \text{ kft/ft} \]

Maximum neg design moment = -3.05 kft/ft

Positive moment = \[ \frac{W_u h_n^2}{16} = \frac{193(10.2)^2}{16} \text{ lbft/ft} = 1905 \text{ kft/ft} - 1.91 \text{ kft/ft} \]

Reinforcement:

Assume \( \phi_d = \frac{d}{1.4} \)

\[ A_s = \frac{M_n}{f_y (d-\phi_d)} = \frac{M_n}{f_y (d - \frac{d}{1.4})} = \frac{3.05 \times 1.1}{0.60 \times (3.75)} = 0.274 \text{ in}^2/\text{ft} \]

\[ \phi = \frac{A_s}{\phi_d} = \frac{18}{1.4 \times 4.12} = 0.0345 \]

Shear Check:

at ext face of 1st int. Support

\[ V_u = 1.15 \frac{W_u h_n}{2} = 1.15 \frac{(293)(10.2)}{2} = 1718 \text{ lb/ft} = 1.7 \text{ kft of Slab} \]

at other supports

\[ V_u = \frac{W_u h_n}{2} = \frac{(193)(10.2)}{2} = 1494 = 1.5 \text{ kft} \text{ width of slab normal at conc.} \]

\[ V_u = 0.75 2 \sqrt{f_{fc} b w d} = 0.75(2)(10)(40000 \times 12.4 = 4554 \text{ lb/ft}} \]

Design Reinforcement

by C110 ACI 318-11

\[ A_{min} = 3 \sqrt{f_{fc}} \frac{b w d}{f_y} = 3 \sqrt{10000 \times 1.5} \times \frac{190}{160000} = 0.190 \text{ in}^2/\text{ft width of slab} \]

\[ A_{max} = 200 \frac{b w d}{f_y} = 0.20 \text{ in}^2 \text{ ok for As used.} \]

by 7.19.11

Temp Reint.

transverse direction \( \phi = 0.0018 = \frac{A_s}{0.0018(10)(15)} = 108 \text{ mil/ft} \)

spaced at min \( 5h = 8.5^\circ \) controls. We're okay.
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Expression</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>L1 (ft)</td>
<td>- \frac{1}{10}</td>
<td>y_1</td>
</tr>
<tr>
<td>M_0 (kft)</td>
<td>- 3.05</td>
<td>2.77</td>
</tr>
<tr>
<td>N_0 (kft-ft)</td>
<td>1.01</td>
<td>1.91</td>
</tr>
<tr>
<td>A_{reqd} (in^2)</td>
<td>1.18</td>
<td>y_2</td>
</tr>
<tr>
<td>A_{min} (in^2)</td>
<td>1.90</td>
<td>1.18</td>
</tr>
<tr>
<td>Bars</td>
<td>#4 @ 15&quot;</td>
<td>1.18</td>
</tr>
<tr>
<td>Final As.</td>
<td>#4 @ 15&quot;</td>
<td>1.18</td>
</tr>
</tbody>
</table>

**Spacing**

\[ S_{max} = \min \left( \frac{3L}{15}, \frac{c}{3} \right), \quad f_s = \frac{f_y}{40} \, ksf \]

\[ S_{min} = 12 \left( \frac{40000}{f_s} \right) = 18" \]

18" spacing is okay.

**Transverse Direction**

Reinforcement:

\[ A_s = 0.10 \, \text{in}^2/ft; \quad \text{using \#4 bars @ 18"} \]

5" SLAB:

\#4 bars @ 18" o.c. 11/2 and Bot. Flexural steel

\#4 bars @ 18" o.c. for transverse reinft.
BEAM: INT BAY, TYP:

- $b=12\"$ by previous assumption
- $h=\frac{f_{w}}{f_{y}} = \frac{97.5}{1.05} = 92.7\"\rightarrow 100\"$

**Table 9.5**

- $W_{om} = (9.5\times 5)(12) = 550$ lb $= .925$ k/ft
- $W_{dek} = (77.5 (22))(6) = 966 = .868$ k/ft

LL Reduction: Min allowed per ASCE 7-05 54.8.4

- $W_{L} = 145$ PIFk 11.4
- $= 1400$ PIF = 1.40 k/ft
- $W_{N} = 1.2 (W_{om}+W_{dek})+1.0 (11.4) = 3.5 = 3.5$ k/ft

**Mn** = $\frac{W_{L}^{2}}{8} = \frac{3.5 (92.7)^{2}}{8} = 605$ kft

**Wn** = $\frac{W_{L}^{2}}{11} = \frac{605}{11} = 54.0$ kft

**d** = $h - \frac{C}{C} - d/4$

- $d = 1.5 - 0.7/2 = 21.9\ldots$

**Mudpan**

- $A_{c} = \frac{A_{w}}{11} = \frac{605}{4 (2.5)} = 6.1$ in$^{2}$

- Thr. (6) #10s: $A_{c} = 6 (141) = 7.69\sqrt{aw}$

- One layer of #6s

- $a = \frac{A_{c}}{.85 A_{w} b} = \frac{7.69 \times 60}{.85 \times 4 \times 10} = 7.5 \rightarrow C = a/b = 8.8\"$

- $E_{u} = E_{u} \left( \frac{1-\nu}{2} \right) = .005 \left( \frac{219 - 9.8}{22.8} \right) = .0042 > .002$ ok.

- $E_{u} > .005 \rightarrow f = .9$
$M_n = A_s f_y (d - a/2)$

$= 7.62 (60) (0.1 - 75/2) = 8240 \text{ kN} = 692 \text{ kft}$

$\phi M_n = 692 \text{ kft} > M_n = 440 \text{ kft} \quad \text{GOOD}$

$A_{\text{min}} = \max \left\{ \frac{200 \text{ kN}/\text{f.y.}}{200 \cdot 16 - 21.9/60} \cdot 1.3 \right\} = 1.3 \text{ kN/ft^2} \quad \text{t - controls < As used, Vok}$

$A_{\text{t, supports}}$

$M_n = 440 \text{ kft}$

$A_o = \frac{M_n}{4d} = 5.02 \text{ in}^2 \rightarrow \alpha_y (4) f_{y t} \quad A_{\text{spun, hole}} = 5.08 \text{ in}^2$

$\alpha = 5.08 / (60) = 0.08 \rightarrow \frac{c}{d} = 5.08'$

$\frac{E_o}{E_o} = 0.03 / 0.03 = 5.78$, $5.78 / 5.78 = 1.00 > 2.5 \quad \text{Vok}$

$M_n = 5.08 (60) (21.9 - 0.08/2) = 5916 \text{ kN} = 493 \text{ kft}$

$\phi M_n = 440 \text{ kft} > M_n = 440 \text{ kft} \quad \text{GOOD}$

Also passes $A_{\text{min}}$.

Vertical Shear:

$V_n = \frac{W_n L_o}{2} \left( \frac{5.11 \text{ kft}}{2} \right) = 54.7 \text{ k}$

$\phi V_n = \phi (V_c + V_o) = \frac{16}{20} \left[ 2 \cdot \frac{16 \text{ kN}}{200 \text{ kN}} \cdot \frac{16}{200 \text{ kN}} + 2 \cdot \frac{0.8 \text{ kN}}{200 \text{ kN}} \cdot \left( \frac{20.75}{200 \text{ kN}} \right) \right]$

$= 0.15 > V_n \quad \text{Vok}$

Good for both $\theta$ Ebdspan and $\theta$ Supports.
Deflections:

$A_{ul max} = \frac{f}{360} = \frac{37.2}{360} = 0.104''$

$\Delta_{ul max} = \frac{f}{240} = \frac{37.2}{240} = 0.155''$

$V = 22 - \left( \frac{1}{2} \right) \left( \frac{54.4 - 14}{3} \right) \left( \frac{5 + 16}{22} \right) = 11.9''$

$I_0 = 14 \left( \frac{2(54.4) + 14(77)^2)}{22} \right) = 63.875''^4$

Using $5/804 \times \frac{144}{E_1}$ as an approximate deflection check because $I_0$ is being used, this will be more conservative

$\Delta_1 = \frac{5 \times 144}{384} \times \frac{5(1.058)(87.2)^4}{384(3.06586)} = 0.06'' < 0.155''$ OK

$\Delta_2 = \frac{5(3.11)(87.2)^4}{384(5.605E6)} = 0.10'' < 1.86''$ OK

Also, by Table 9.5 (a) $h_{min} = \frac{f}{21} = 21.3'' < 24''$ OK.
**Girder Design:**

**Estimate Size:**

\[ w_n = 10 \text{ ft} \]

**Assuming Self weight = 10-15% of D.L.**

\[ \text{W}_{u} = 105 \text{ psf} \times 37.4 = 3,977 \text{ kbf} \]
\[ \text{W}_{b} = 5 \times 2 = 30 \text{ klf} \]
\[ \text{W}_{w} = 1.2 (37.4 + 30) = 110 \text{ klf} \]
\[ \text{W}_{u} = \frac{w_{uf}}{1.4} = \frac{110}{1.4} = 78.6 \text{ klf} \]
\[ 2d (180 \text{ lb/ft}) = 2b = 2d \]
\[ b = 28'' \]
\[ d = 56'' \rightarrow h = 98'' \]
\[ \text{wsw} = (30-5)(28) \times (150/44) = 96 \text{ klf} \]
\[ \rightarrow \text{W}_{u} = 4.5 \text{ klf} \]
\[ \text{W}_{u} = \frac{w_{uf}}{1.4} = \frac{110}{1.4} = 78.6 \text{ klf} \]
\[ V_{u} = \frac{w_{uf}}{1.4} = \frac{110}{1.4} = 78.6 \text{ klf} \]
\[ V_{u} = \frac{w_{uf}}{1.4} = \frac{110}{1.4} = 78.6 \text{ klf} \]

**Reinf Design:** For Negative Moment

\[ A_s = \frac{M_u}{f_y} = \frac{182}{4} = 45.5 \text{ in}^2 \]
\[ f_y = 60,000 \text{ psi} \]
\[ \phi A_s = 0.9 (45.5) = 40.9 > \frac{f_y}{f_y} = 1.4 \]
\[ \phi M_n = 0.9 (12.7) (180) (28 - 91/2) = 19,300 \text{ kbf-2} \]

**Use (10) #10s**
Positive Reinforcement:

\[ N = \frac{644}{4.36} = 6.0 \text{ in}^2 \rightarrow 4y \cdot (5) \times 10 = 6.30 \text{ in}^2 \]

\[ a = \frac{6.30 - 6.0}{0.85 - 1.28} = 4.0 \Rightarrow C = 4.0 = 4.7 \frac{185}{185} \]

\[ \delta_s = 1000 \times (36 - 4.4) \div 4.7 = 1000 \times 31 \]

\[ \delta_s \text{ max} = 0.9 \times (60) \times (36 - 4.4) \times 2 \times 0.5 = 97.2 \text{ ft} \]  

\[ \text{h} > \delta_s \text{ max} \Rightarrow \text{ok} \]

\[ \text{Use (5) #48} \]

Deflection Check:

\[ \Delta_1 \text{ max} = \frac{7}{360} = 0.875 \times 11 = 15.2'' \]

\[ \Delta_T \text{ max} = \frac{7}{240} = 0.475 \times 11 = 2.2'' \]

\[ \bar{y} = 3 \frac{1}{3} \left( \frac{104 - 28}{5^2 + 19.32} \right) \]

\[ = 20.6'' \]

\[ \bar{y} = 12 \left( \frac{5}{3} \right) \sqrt{5^2 + 19.32} \]

\[ + 2 \left( \frac{1}{3} \right) \sqrt{5 (20 - 36)^2} = 21.12'' \]

\[ \Delta_1 = \frac{5Wl^4}{384EI} = \frac{4.9 \times (44.75)^4}{384 \times 57600 \times 57000} = 11'' \]

\[ = 11'' \leq 1.5'' \text{ ok} \]

\[ \Delta_T = \frac{10.6 (44.75)^4}{384 EI} = 3'' < 2.2'' \text{ ok} \]

Even considering creep, the deflections are well under limits.

Continuously cast:

Fixed connections

Treat as a 7-beam cross section:

\[ \text{b x h =} \]

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Vertical Shear:

\[ A_{\text{min}} = \max \left\{ 127.9 \sqrt{f_e} \frac{b_m}{f_y} \right\} = 279 \text{ in}^2 \]

\[ S = 3 \left( \frac{24\text{ in}}{12\text{ in}} \right) = 36 \text{ in} \text{ spacing} \]

Depth:

Section at midspan:

\[ d = 114\text{ in} \]

Section at girdler and slab:

\[ 36^\circ \]

Point not shown.
Appendix 6: System Comparisons

Weight Analysis

- **Existing:**
  - Slab & decking 51 psf
  - Steel framing: W44×74 beam @ 7.5’
    - W30×90 girder @ 37.5’
    - W = 51 psf + 7.5/7.5’ + 9.0/37.25 = 63.5 psf

- **System: Hollow Core**
  1) Planks: 48.75 psf + 25 psf topping
     - Framing: beam W=40×149 @ 14.5’
     - Girder W=40×149 @ 24.9’
     - W = 73.75 + 146/14.46 + 149/24.9 = 88.9 psf
  2) Planks: 48.75 psf + 25 psf
     - Inv 7 bms 114’ PLF @ 14.5’
     - W = 73.75 + 146/14.46 = 143.3 psf

- **System: Post-Tensioned**
  - Slab: 20” depth (150pcf) = 250 psf
    - Includes rebar

    - W = 250 psf

- **System: One-Way Slab**
  - Slab: (9/16”) (150pcf) = 62.5 psf
  - Bmns: (28.1” x 32.5”) x 150 @ 87.2’ = 49.6 PLF @ 37.2’
  - Girder: (28”) x (38 – 5”) x 150 @ 44.75’ = 96.5 PLF @ 44.75’

    - W = 62.5 psf + 49.6/37.2 + 96.5/44.75 = 274 psf
Reflection Comparison

Existing

\[ \delta_h = 3.1'' \]

\[ \delta_{sec} = 77.0'' \]

System: Hollow Core (Layout B)

1. \[ \delta_{planks} = 47'' \]
   \[ \delta_{bm} = 1.88'' \]
   \[ \delta_{goder} = 1.38'' \]

2. \[ \delta_{planks} = 47'' \]
   \[ \delta_{1728} = 1.89'' \]
   \[ \delta_{controls} \]

System: Post Tensioning

\[ \delta_{m} = .46'' \]

\[ \delta_{ew} = .55'' \]
\[ \delta_{controls} \]

System: One-Way

\[ \delta_{slab} = \frac{6(1.293)(11.2/1728)}{384 \cdot 5100(4000 \cdot \frac{7}{12''})} \cdot 28'' \]

\[ \delta_{bm} = .60'' \]
\[ \delta_{goder} = .30'' \]

Calculations presented in associated systems' calcs. on previous pages.

*Since camber included, composing for \( \delta \).
Depth Comparison

**Original System:**
- Slab & decking: 5”
- Beam: 23.9”
- Girder: 59.5” (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”
   - I-beam: 21.5” including planks
   - \( d = 2.7” \)

**System: Post-tensioned**
- Slab: 20” = 1.7”

**System: One Way**
- Slab: 5”
- Beam: 22”
- Girder: 58”
- \( d = 38” = 3.2’ \)
| Structural Option | SteelStacks Performing Arts Center | Bethlehem, Pennsylvania | 12 October 2012 | Tech Report II |

### Existing - Composite Steel

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

### Hollow Core Plank with Steel Beams

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

### Hollow Core Plank with Inverted T-Beams

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

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

### One Way Slab & Beam

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<thead>
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<th>System Components</th>
<th>Quantity</th>
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<th>Material</th>
<th>Installation</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Forms in place, flat plate to 15' high, 4 uses</td>
<td>1515.9</td>
<td>S.F.</td>
<td>0.94</td>
<td>3.79</td>
<td>4.73</td>
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<td>Forms in place, interior beam. 12&quot;, 4 uses</td>
<td>1365.7</td>
<td>SFCA</td>
<td>0.81</td>
<td>4.47</td>
<td>5.28</td>
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<tr>
<td>Reinforcing in place, elevated slabs #4 to #7</td>
<td>1887.8</td>
<td>Lb.</td>
<td>0.60</td>
<td>0.31</td>
<td>0.91</td>
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<tr>
<td>Reinforcing in place, elevated beams #10</td>
<td>12504.5</td>
<td>Lb.</td>
<td>3.83</td>
<td>2.18</td>
<td>6.01</td>
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<tr>
<td>Concrete ready mix, regular weight, 4000 psi</td>
<td>26.28</td>
<td>CY</td>
<td>1.63</td>
<td>0.00</td>
<td>1.63</td>
</tr>
<tr>
<td>Place and vibrate concrete, elevated slab less than 6&quot;, pump</td>
<td>26.28</td>
<td>CY</td>
<td>0.00</td>
<td>0.36</td>
<td>0.36</td>
</tr>
<tr>
<td>Cure with sprayed membrane curing compound</td>
<td>0.26</td>
<td>C.S.F.</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Total SF 1664.70 Total ($/sf) 18.91