Technical Report 2:
Alternative Floor Framing Systems
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Executive Summary

1000 Connecticut Avenue is an 11 story, 565,000 GSF commercial office building located at the corner of K Street and Connecticut Avenue in Washington D.C. The building is used primarily for office space, but also contains retail space on the first level, commercial office space on levels 3-12, a rooftop terrace with a green roof, and four levels of underground parking. The purpose of this technical report is to further understand the existing structural system by spot checking existing structural members and designing three alternative floor framing systems and comparing each to the existing floor system to determine which one more is a more viable alternative.

Spot checks were performed for an interior flat slab panel and an interior column. Both analyses resulted in different member sizes relative to the existing structural members. This difference can be explained through a combination of simplifying assumptions and assumed dead loads.

Further, alternative floor framing systems were designed for this tech report and the system comparisons were based on architecture (fire rating and other impacts); structural (foundation and lateral system impact); serviceability (maximum system deflection and vibration control); and construction (additional fire protection and schedule impact). Each system’s feasibility was determined based on these four listed criteria. A summary chart of these system comparisons are provided at the end of this report.

For this tech report, the four systems analyzed and designed were the following:

- Two-way flat slab (existing)
- Composite beam/girder system with composite steel deck
- Two-way post-tension slab
- Composite steel joist/steel girder system with composite steel deck

The final design of the alternative floor systems resulted in the following:

- Two-way flat slab system (existing): 8” thick slab with 8” thick drop panels
- Composite steel beam/girder system: a W16x31 beam with (32)- ¾” ϕ shear studs and a 2” camber and a W21x50 girder with (28)- ¾” ϕ shear studs
- Two-way post-tension slab: 7” thick slab with 3” thick drop panels and (26) - ½” ϕ 7-wire unbounded tendons in the N-S direction and (18) - ½” ϕ 7-wire unbounded tendons in the E-W direction.
- Composite joist/steel girder system: 14CJ1400/607 composite joist with (40)-⅝” ϕ shear studs and a W21x93 girder

After designing each system and using the above criteria for system comparison, it was found that the composite steel beam/girder system, composite joist/steel girder system, and the post-tensioned slab were all viable alternatives. As a result, these three systems will be further investigated to determine which one would be the better floor framing alternative.
The appendices in this report include hand calculations for gravity spot checks and the three alternative floor system designs, as well as typical floor plans and a building section.
Introduction

1000 Connecticut Avenue, NW Office Building is a new 12 story office building located at the northwest intersection of K Street and Connecticut Avenue in Washington DC, as can be seen in Figure 1. The 1000 Connecticut Avenue Office building is designed to achieve LEED Gold certification upon completion. Despite being used primarily for office space, the building is comprised of mix occupancies, which include: office space, a gymnasium, retail, and parking garages. The structure has 4 levels of underground parking. The building’s total square footage is 555,000 SF with 370,000 SF above grade and 185,000 SF below grade.

To create a new Washington landmark, the building is designed to complement surrounding institutions by blending both traditional and modern materials. The facade consists of a glass, stainless steel and stone panel curtain wall system. Exterior and interior aluminum and glass storefront windows and doors are on the ground level. The lobby and retail space are located on the 1st level, which has a 12’-6 1/2” floor-to-floor story height. A canopy facing K Street brings attention to the main lobby entrance, as can be seen in Figure 2.
Beyond the main entrance is a two story intricate lobby space with carrera marble and Chelmsford granite flooring, aluminum spline panels integrated with glass fiber reinforced gypsum (GFRG) ceiling tiles and European white oak wood screens, as can be seen in Figure 3.

![Figure 3 Perspective of lobby](image)

The retail space is broken down into several retail stores facing K Street and Connecticut Avenue. These retail stores are housed behind storefront glass to enable display of merchandise to potential customers. The 2nd-12th levels have 10’-7 ½” floor-to-floor story heights. Housed on the typical levels (3rd-12th) is the office space. A combination of tall story heights and a continuous floor to ceiling glass façade enables natural daylight to enter the building space as well as provides scenery to the Washington monuments, Farragut Park, and the White House, as can be seen in Figure 4.

![Figure 4 Perspective of typical office with floor-to-ceiling windows that supply views to the city](image)
In addition, located on the penthouse level is a roof-top terrace with a green roof and a mechanical penthouse, as can be seen in Figure 5.

Figure 5 Perspective of green roof on roof-top terrace and mechanical penthouse

Housed on the basement levels (B1-B4) are underground parking and a fitness center. A total of 253 parking spaces are provided; level B1 has 19 parking spaces; level B2 has 74 parking spaces; level B3 has 78 parking spaces; level B4 has 82 parking spaces. In addition, the fitness center is located on level B1.
Structural Overview

1000 Connecticut Avenue Office Building’s structural system is comprised of a reinforced concrete flat slab floor system with drop panels and a bay spacing of approximately 30 feet by 30 feet. The slab and columns combined perform as a reinforced concrete moment frame. The substructure and superstructure floor systems are both comprised of an 8” thick two-way system with #5 reinforcing bars spaced 12” on center in both the column and middle strips and 8” thick drop panels. The below grade parking garage ramp is comprised of a 14” thick slab with #5 reinforcing bars provided both top and bottom with a spacing of 12” on center.

Foundation

ECS Mid-Atlantic, LLC performed a geotechnical analysis of the building’s site soil conditions as well as provided recommendations for the foundation. A total of five borings were observed in the geotechnical analysis. It was determined that a majority of the site’s existing fill consists of a mixture of silt, sand, gravel, and wood. The natural soils consisted of sandy silt, sand with silt, clayey gravel, silty gravel, and silty sand. The soil varies from loose to extremely dense in relative density. Based on the samples recovered from the rock coring operations, the rock is classified as completely to moderately weathered, thinly bedded, and hard to very hard gneiss.

At the time of the study, the groundwater was recorded at a boring depth of 7.5 feet below the existing ground surface. The shallow water table is located at an elevation of 35 to 38 feet in the vicinity of the site.

1000 Connecticut Avenue, NW Office Building is supported by a shallow foundation consisting of column footings and strap beams, as can be seen in Figure 6. The typical column footing sizes are 4'-0” x 4'-0”, 5'-0” x 5'-0”, and 4'-0” x 8'-0”.

Figure 6 Details of typical strap beam and column footing
The footings bear on 50 KSF competent rock. The Strap beams (cantilever footings) are used to prevent the exterior footings from overturning by connecting the strap beam to both the exterior footing and to an adjacent interior footing. A simplified foundation plan can be seen in Figure 7.

The slab on grade is 5” thick, 5000 psi concrete with 6x6-W2.9xW2.9 wire welded fabric on a minimum 15 mil Polyethylene sheet over 6” washed crushed stone. The foundation walls consists of concrete masonry units vertically reinforced with #5 bars at 16” on center and horizontally reinforced with #4 bars at 12” on center and are subjected to a lateral load (earth pressure) of 45 PSF per foot of wall depth.

Figure 7 Foundation plan
Framing and Floor System

The framing system is composed of reinforced concrete columns with an average column-to-column spacing of 30’x30’, as can be seen in Figure 8. The columns have a specified concrete strength of $f'_c$=8000 psi for columns on levels B4 to level 3, $f'_c$=6000 psi for columns on levels 4-7, and $f'_c$=5000 psi for columns on levels 8-mechanical penthouse. The columns are framed at the concrete floor, as can be seen in Figure 9, and the columns vary in size. The most common column sizes are 24”x24”, 16”x48”, and 24”x30”. The column capitals are 6” thick, measured from the bottom of the drop panel, extending 6” all around the face of the column, as can be seen in Figure 10.
The typical floor system is comprised of an 8" thick two-way flat slab with drop panels reinforced with #5 bottom bars spaced 12" on center in both the column and middle strips, as can be seen in Figure 11.
The individual drop panels are 8" thick, extending a distance d/6 from the centerline of the column, as can be seen in Figure 12.

**Figure 12** Typical Continuous drop panel

A 36" wide by 3 ½" deep continuous drop panel is located around the perimeter on all floor levels. Levels 3-12 are supported by four post-tension beams above the lobby area. Due to the two story lobby, there’s a large column-to-column spacing. As a result, post tension beams are used to support the slab on levels 3-12 located above the lobby. In addition, four post-tension beams support the slab on levels 3-12 that are located above the two-story parking deck, which also has a large column-to-column spacing, as can be seen in Figure 13.

**Figure 13** Plan view and typical detail of Post-tension beams supporting slab on levels above two-story loading dock
Lateral System
The lateral system is comprised of a reinforced concrete moment frame. The columns and slab are poured monolithically, thus creating a rigid connection between the elements. The curtain wall is attached to the concrete slab, which puts the slab in bending. The curtain wall transfers the lateral load to the slab. The slab then transfers the lateral load to the columns and in turn the columns transfer the load to the foundation. Transfer girders on the lower level are used to transfer the loads from the columns that do not align with the basement columns in order to transfer the load to the foundation. A depiction of how the lateral load is transferred through the system can be seen in Figure 14.
Roof System
The main roof framing system is supported by an 8” thick concrete slab with #5 bars spaced 12” on center at the bottom in the east-west direction. The slab also has 8” thick drop panels. The penthouse framing system is separated into two roofs: Elevator Machine Room roof and the high roof. The elevator machine room roof framing system is supported by 14” and 8” thick slab with #7 bars with 6” spacing on center top and bottom in the east-west direction.

Design Codes

According to sheet S601, the original building was designed to comply with the following:

- Building Code Requirements for Structural Concrete (ACI 318)
- Specifications for Structural Concrete (ACI 301)
- Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315)
- Specification for the Design, Fabrication and Erection of Structural Steel for Buildings (AISC manual), Allowable Strength Design (ASD) method

The codes that were used to complete the analyses within this technical report are the following:

- ACI 318-08
- Minimum Design Loads for Building and Other Structures (ASCE 7-10)
- Vulcraft Steel Roof and Floor Deck Catalog, 2008
- Vulcraft Composite and Non-Composite Floor Joist Catalog, 2009
**Structural Materials**

Table 1 below shows the several types of materials that were used for this project according to the general notes page of the structural drawings on sheet S601.

<table>
<thead>
<tr>
<th>Usage</th>
<th>Weight</th>
<th>Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spread Footings</td>
<td>Normal</td>
<td>4000</td>
</tr>
<tr>
<td>Strap Beams</td>
<td>Normal</td>
<td>4000</td>
</tr>
<tr>
<td>Foundation Walls</td>
<td>Normal</td>
<td>4000</td>
</tr>
<tr>
<td>Formed Slabs and Beams</td>
<td>Normal</td>
<td>5000</td>
</tr>
<tr>
<td>Columns</td>
<td>Normal</td>
<td>Varies (based on column schedule)</td>
</tr>
<tr>
<td>Concrete Toppings</td>
<td>Normal</td>
<td>5000</td>
</tr>
<tr>
<td>Slabs on Grade</td>
<td>Normal</td>
<td>5000</td>
</tr>
<tr>
<td>Pea-gravel concrete (or grout)</td>
<td>Normal</td>
<td>2500 (for filling CMU units)</td>
</tr>
<tr>
<td>All other concrete</td>
<td>Normal</td>
<td>3000</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Type</th>
<th>Standard</th>
<th>Grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deformed Reinforcing Bars</td>
<td>ASTM A615</td>
<td>60</td>
</tr>
<tr>
<td>Welded Wire Fabric</td>
<td>ASTM A775</td>
<td>N/A</td>
</tr>
<tr>
<td>Reinforcing Bar Mats</td>
<td>ASTM A185</td>
<td>N/A</td>
</tr>
</tbody>
</table>

**Reinforcing Steel (Unbonded)**

<table>
<thead>
<tr>
<th>Type</th>
<th>Standard</th>
<th>Strength (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestressed Steel (seven wire low-relaxation or stressed relieved strand)</td>
<td>ASTM A416</td>
<td>270</td>
</tr>
</tbody>
</table>

**Miscellaneous Steel**

<table>
<thead>
<tr>
<th>Type</th>
<th>Standard</th>
<th>Grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural Steel</td>
<td>ASTM A36</td>
<td>N/A</td>
</tr>
<tr>
<td>Bolts</td>
<td>ASTM A325</td>
<td>N/A</td>
</tr>
<tr>
<td>Welds</td>
<td>AWS</td>
<td>N/A</td>
</tr>
</tbody>
</table>

**Table 1 Design materials**
Gravity Loads

For this technical report, live loads and snow loads were compared to the loads listed on the structural drawings. In addition, dead loads were calculated and assumed in order to spot check gravity members and typical columns. The system evaluations were then compared to the original design. The hand calculations for the gravity member checks can be found in Appendix A.

Dead and Live Loads

Table 2 below is a list of the live loads in which the project was designed for compared to the minimum design live loads outlined in ASCE 7-10.

<table>
<thead>
<tr>
<th>Occupancy</th>
<th>Design Load (psf)</th>
<th>ASCE 7-10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parking Levels</td>
<td>50</td>
<td>40</td>
</tr>
<tr>
<td>Retail</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>Vestibules &amp; Lobbies</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>Office Floors</td>
<td>100=(80 psf+ 20 psf partitions)</td>
<td>70= (50 psf + 20 psf partitions)</td>
</tr>
<tr>
<td>Corridors</td>
<td>100</td>
<td>100 on ground level 80 above 1st level</td>
</tr>
<tr>
<td>Stairs</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>Balconies &amp; Terraces</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>Mechanical Room</td>
<td>150</td>
<td>-</td>
</tr>
<tr>
<td>Pump Room, Generator Room</td>
<td>150</td>
<td>-</td>
</tr>
<tr>
<td>Light Storage</td>
<td>125</td>
<td>125</td>
</tr>
<tr>
<td>Loading Dock, Truck Bays</td>
<td>350</td>
<td>250</td>
</tr>
<tr>
<td>Slab On Grade</td>
<td>100</td>
<td>-</td>
</tr>
<tr>
<td>Green Roof Areas</td>
<td>30</td>
<td>-</td>
</tr>
<tr>
<td>Terrace</td>
<td>100</td>
<td>100</td>
</tr>
</tbody>
</table>

Table 2 Summary of design live loads compared to minimum design live loads on ASCE 7-10

Note: - Means the load for the specified occupancy was not provided

Based on the above design live loads, certain spaces were designed for higher loads to create a more conservative design and to allow for design flexibility. For this technical report, the design live loads were used for the gravity member analyses.
Snow Load

The snow load was determined in conformance to chapter 7 in ASCE 7-10. A summary of the snow drift parameters are shown in table 3.

![Table 3 Summary of roof snow calculations](image)

According to structural drawing sheet S601, the flat roof snow load was 22.5 psf whereas 15.75 psf was calculated in this technical report. According to ASCE 7-10, \( p_f = 0.7 C_e C_t I_s P_g \), whereas according to IBC 2000, \( p_f = C_e C_t I_s P_g \). The difference in the calculated flat roof snow load and the design flat roof snow load is due to a 0.7 reduction factor. The 15.75 psf value was used to determine the snow load and snow drifts. These subsequent calculations can be found in Appendix A.

Table 4 below is a list of the dead loads that were used for the gravity spot checks. The superimposed dead loads for the floor levels and roofs were assumed.

<table>
<thead>
<tr>
<th>Dead Loads</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Weight Concrete</td>
<td>150 pcf</td>
</tr>
<tr>
<td>Curtain Wall</td>
<td>250 plf</td>
</tr>
<tr>
<td>Precast Panels</td>
<td>450 plf</td>
</tr>
<tr>
<td>Floor Superimposed Dead Load (ceiling, lights, MEP, miscellaneous)</td>
<td>10 psf</td>
</tr>
<tr>
<td>Main Roof Superimposed Dead Load (ceiling, lights, MEP, miscellaneous)</td>
<td>10 psf</td>
</tr>
<tr>
<td>Penthouse Roof Superimposed Dead Loads</td>
<td>5 psf</td>
</tr>
</tbody>
</table>

Table 4 Summary of dead loads
Floor System Analysis

Comparisons were made between the existing floor system and three alternative floor framing systems, which were designed for this report. Hand calculations were used to design the alternative floor systems. The four systems that were analyzed in this report were:

- Two-way flat slab (existing system)
- Composite beam/girder system with composite steel deck
- Two-way post-tensioned slab
- Composite joist/steel girder system with composite steel deck

The cost of each system was determined by using a square foot estimate, which has a ±20% error, based on data obtained from R.S. Means Square Foot Costs 2010. Appendix E provides the R.S. Means charts that summarize the cost of each system. The Cost for two-way post-tensioning slabs was not found, but the cost of the system was assumed to cost the same as the two-way flat system plus the cost of tendons.
Two-Way Flat Slab (Existing System)

The four levels below grade and twelve levels above grade consist of a two-way flat slab floor system with an 8” thick slab, 8” thick drop panels and 6” thick column capitals. The parking garage ramp consists of a 14” thick slab. The 8” slab consists of #5 reinforcing bars spaced 12” on center in both the column and middle strips. This system is assembled and shored on site and formwork is used to construct the concrete slabs, columns, drop panels and column capitals.

For this technical report, gravity checks were performed on a typical interior panel and column 50 was checked on both the 1st and 5th levels. The slab panel and column used for analyses can be seen in Figure 15 outlined in blue (interior panel) and green (column 50). The hand calculations can be found in Appendix A.
General

The two-way slab system weighs 100 pounds per square foot. Based on the R.S. Means data, this system was found to cost $17.45 per square foot, which includes cost of the material and installation.

The structural depth of this system is 8” in the slab region and 22” in the column region (which includes thickness contributed by the drop panels and column capitals). The remaining ceiling cavity towards the center of the building is used for mechanical ductwork. As a result, any additional structural depth will either require an increase in building height or a redesign of the mechanical layout. Since the building height is limited to 130 ft. by zoning and the existing structure is currently 130 ft., an alternative system that will require additional building height cannot be used with the existing 11 story structure.

Architectural

This system achieves a minimum 2 hour fire rating and since the entire structure was designed to achieve this rating, there are no additional architectural impacts to consider.

Structural

This system is supported on a shallow foundation consisting of spread footings and a slab on grade. If this system were chosen as the final design, the existing foundation system will remain unchanged.

Serviceability

Deflections were not directly calculated for this system, instead the slab thickness was determined based on a span-to-depth ratio used in design practice and it was found that an 8” slab would be required to control deflections, which is the existing slab thickness. In addition, through research it was found that two-way concrete slabs are effective in absorbing sound thus decreasing sound transmission as well as vibration control. Therefore it is apparent that this system will not create any serviceability issues.

Construction

Additional fire proofing does not need to be provided for this system, but formwork will be required for the slab, drop panels, column capitals, and columns. In addition, the concrete will require time for curing to enable the concrete to reach its full strength. The formwork needed to construct this system along with the time required for concrete curing will increase the construction schedule. The existing concrete structural system began construction in July 2010 and was complete by March 2011. The four levels below grade plus the twelve levels above grade were completed within an 8 month period. This rapid construction may be attributed to the fact that Washington D.C. has a very competitive concrete market with many tradesmen that specialize in concrete construction, thus resulting in shorter construction time.
Advantages

- Long spans
- Shallow structural depth thus low floor-to-floor story heights
- Simple formwork
- Protects against corrosion
- Very good vibration and sound transmission control

Disadvantages

- Slight increase in construction schedule due to formwork and concrete curing
- Difficult to drill through the slab core for future services
- Increase in cost due to formwork being labor intensive

Despite the fact that this system is relatively heavy, it still only requires a shallow foundation and performs well in all of the above analyzed categories. This system provides long spans with low structural depth and low floor-to-floor heights, making this system ideal for the 1000 Connecticut Avenue Office Building by providing less structural obstructions and thus more open, rentable office space.
Composite Beam and Girder Framing with Composite Steel Deck

Figure 16 Composite Steel System Layout (top left) and composite deck section taken from Vulcraft 2008 catalog (lower right)

The first system designed was a composite steel system, which was chosen because it was the more practical alternative steel system to use to span the long bays and still maintain a lower structural depth. The composite action between the steel beam/girder and slab results in an efficient system. The layout for this system can be seen in Figure 16.

The design was performed by hand calculations, which can be found in Appendix B. The Vulcraft 2008 Manual was used to specify the deck and AISC, 14th edition was used to design the steel beams and girders.

For this system, the column grid was slightly adjusted by increasing the column spacing between two interior bays by aligning the interior columns with the exterior columns (located along the perimeter of the west wall). This column spacing adjustment increased the two interior bay widths from 26’ to 30’ in the N-S direction. This slight change to the column grid was to create a more consistent frame layout throughout the building. The new steel column layout can be seen in Figure 17.
Figure 17 Proposed steel column layout for composite steel system

The final design resulted in a 3VLI20 composite metal deck with a 3 span condition and a 7 ½” total slab thickness. To achieve a 2 hour fire assembly rating, an unprotected deck with 4 ½” normal concrete topping was used. A W16x31 with (32) - ¾” shear studs and a 2” camber was chosen for the beam and a W21x50 with (28) - ¾” shear studs was chosen for the girder.

General

With a 7 ½” total thick composite deck combined with the beams in Figure 16, this system was found to weigh 81.2 pounds per square foot and costs $19.83 per square foot. The most important impact of this system is its structural depth increase of 23 ½” in the slab region due to the beams and 28 ½” in the slab region due to the girders. The controlling 20 ½” increase in the slab region will be difficult to absorb in the mechanical layout without increasing the building height or decreasing the floor-to-ceiling height.
**Architectural**

The steel beams, girders, and deck will need to be fire proofed with spray on fireproofing. A drop ceiling can be used as a ceiling finish and the additional space supplied by the drop ceiling will provide additional mechanical and electrical space.

To use this floor system and achieve an 8’-6” minimum floor-to-ceiling height, the building height will need to increase. Since 1000 Connecticut Avenue is currently 130 ft. and is located in Washington DC, which has a zoning height restriction of 130 ft., the existing structure cannot be increased in height. As a result, this system will have to be designed for a fewer number of stories to achieve high floor-to-ceiling heights and to stay within the height limit.

In addition, a steel framing system will require a uniform layout, therefore to use this system in place of the existing gravity system will require certain columns to be relocated and removed to achieve a uniform framing layout. As a result, the existing architectural layout may need to be changed to accommodate the structural system layout.

**Structural**

This system weight is 19% lighter than the two-way flat slab system. As a result, the existing shallow foundation can still be used. Since the vertical columns are steel, the lateral force resisting system will either consist of steel moment frames, or braced frames, or a combination of these two systems. The below grade construction will still be comprised of cast-in-place concrete, which is a better material to use for parking garages.

**Serviceability**

The maximum deflection of this system was calculated in this report to be 1.73” for the beams and 1.3” for the girders, which are both within the permissible limits.

A vibration analysis for this system was not performed, but if this system were chosen for further investigation, vibration analysis will have to be performed to ensure this system will be able to control vibrations throughout the structure.

**Construction**

To achieve a 2 hour fire rating, the steel beams, girders, and deck must be fire proofed with spray on fireproofing. Despite this, steel member erection is more rapid than cast in place concrete construction, therefore the construction schedule should be significantly reduced.
Advantages

- Low system weight resulting in a reduction in frame loading and foundation cost
- Composite action between the concrete slab and steel member decreases structural depth
- Decrease in construction schedule
- Adaptable system that can be drilled and/or cut out for service requirements
- Increase rentable space due to wider bays created by longer spans

Disadvantages

- Building height increase
- Construction cost increase due to fire proofing
- Requires columns to be relocated and removed to create a uniform framing layout

The composite beam/girder floor system increases the current structural depth to 28 ½” and requires an increase in the overall building height to achieve high floor-to-ceiling heights. Since the existing building is limited to a 130 ft., this alternative floor system cannot be used with the current 11 story structure. As a result, this system is feasible if either the building were designed for a reduced number of stories or relocated to a region that does not have a height limit.
Two-Way Post-Tension Slab

Post-tensioning design is often used to achieve longer spans and reduce structural depth, which was particularly important for 1000 Connecticut Avenue due to the 130 ft. zoning height restriction. The design was performed by hand calculations, which can be found in Appendix C. An example by the Portland Cement Association (PCA) was used as a design reference. The post-tensioned slab layout can be seen in Figure 18.

Figure 18 post-Tension tendon layout
Two interior equivalent frames were chosen for design to determine whether this system would be viable. 5 spans were designed in the N-S direction and 4 spans were designed in the E-W direction. The two equivalent frames chosen for design can be seen highlighted in green in Figure 19.

The final design resulted in a 7” thick slab with 3” thick drop panels and (26) - ½” ϕ 7-wire unbounded tendons in the N-S (banded) direction and (18) - ½” ϕ 7-wire unbounded tendons in the E-W (distributed) direction.

**Figure 19** Equivalent frames chosen for PT design highlighted in green
General

This system weighs 87.5 pounds per square foot, which is 12.5% lighter than the existing two-way slab system, and costs $17.45 (not including post-tensioning material). The structural depth in the slab region decreases to 7” and the structural depth in the column region decreases to 3”. Due to the slight decrease in structural depth in the slab region, both the floor-to-ceiling height and existing overall building height will be unaffected.

Architectural

This system achieves a 2 hour fire resistance rating from cover requirements on the reinforcing. If this system were used, the existing structural layout can remain the same and therefore the current architecture layout will be unaffected. Further, despite to the slight decrease in the slab system, the existing floor-to-ceiling height will remain the same.

Structural

Since this system weighs less than the existing two-way flat slab system, the foundation will be unaffected. The lateral load system will remain the same as the existing lateral system; a concrete moment frame consisting of the concrete columns and slab. Thus if this system were chosen for further investigation, lateral loads will have to be considered for designing the slab. The below grade construction will still consist of cast-in-place concrete, with the possibility of using post-tensioned slabs for the underground four level parking garage and slab on grade.

Serviceability

Deflections were not directly calculated for this system, but they were limited by acceptable span-to-depth ratios from industry practice outlined in the Portland Cement Association example, which was used to assist in designing the slab. In addition, through research it was found that post-tensioned slabs are effective in decreasing sound transmission and providing vibration control, thus it is likely this system will not have any serviceability issues.

Construction

Additional fire proofing does not need to be provided for this system, but formwork will be required for the slab, drop panels, and columns. The construction time for this system may potentially lengthen due to the fact that specialized tradesmen familiar with post-tensioning will be required to construct this slab system productively and successfully.
Advantages

- Longer spans achieved with thinner slab depths
- Low structural depth
- Reduced deflection due to service loads
- Good crack control
- High punching shear strength obtainable through appropriate tendon layout
- Increased design flexibility without the need for transverse or longitudinal beams for irregular building geometries
- Lighter system weight

Disadvantages

- May lengthen construction schedule
- Difficult to drill through slab due to tendons
- Additional construction difficulty due to post-tensioning requirements

This system weighs less than the existing two-way flat slab system, as a result the foundation will be unaffected. This system provides long spans with low structural depth and low floor-to-floor heights, making this system ideal for the 1000 Connecticut Avenue Office Building by providing less structural obstructions and thus more open, rentable office space. Therefore this system merits further investigation.
Composite Joist/ Steel Girder System

Figure 20 composite joist/steel girder layout

The last system designed was a composite steel joist system, which was chosen to span long distances while maintaining a low floor-to-floor building height and to reduce overall system weight. The composite action between the steel joist and slab results in an efficient system with reduced live load deflections. The layout for this system can be seen in Figure 20.

The design was performed by hand calculations, which can be found in Appendix D. The Vulcraft 2008 Steel Roof and Floor Deck Catalog was used to specify the deck, Vulcraft 2009 Composite and Non-Composite Floor Joists Catalog was used to specify the composite joist and AISC, 14th edition was used to design the steel girders.

Since it’s more efficient and less expensive for steel frames to have a uniform framing layout, the column grid was slightly adjusted by increasing the column spacing between two interior bays by aligning the interior columns with the exterior columns. This slight change to the column grid was to create a more consistent frame layout throughout the building. The new steel column layout can be seen in Figure 17.

The final design resulted in a 1.5VLI22 composite metal deck with a 3 span condition and a 6” total slab thickness. To achieve a 2 hour fire assembly rating, an unprotected deck with 4 ½” normal concrete
topping was used. A 14C1400/607 with (40) - \( \frac{3}{8} \)" \( \phi \) shear studs was chosen for the composite joist and a W21x93 was chosen for the girder.

**General**

With a 6" total thick composite deck combined with the joists in Figure 20, this system was found to weigh 85 lbs. per square foot and costs $22.05 per square foot. The joist has a depth of 14” and the girder has a 21.6” depth. This 19.6” increase in structural depth in the slab region will require an increase in building height to maintain a minimum 8'-6" floor-to-ceiling height.

**Architectural**

The steel joists, girders, and deck will need to be fire proofed with spray on fireproofing to achieve a 2 hour fire rating. A drop ceiling can be used as a ceiling finish and the open webs can be used as raceways for mechanical ducts and piping, which will reduce the amount of space needed in the ceiling cavity.

To use this floor system and achieve an 8'-6" minimum floor-to-ceiling height, the building height will need to increase. Due to 1000 Connecticut Avenue having a restricted 130 ft. height limit, the existing structure will not be able to increase to accommodate for the additional height needed to maintain high floor-to-ceiling heights. As a result, this system will have to be designed for a fewer number of stories to stay within the height limit.

In addition, a steel framing system will require a uniform layout, therefore to use this system in place of the existing gravity system will require certain columns to be relocated and removed to achieve a uniform framing layout. As a result, the existing architectural layout may need to be rearranged to accommodate the new structural system layout.

**Structural**

This system weight is 15% lighter than the two-way flat slab system. As a result, the existing shallow foundation can still be used. Since the vertical columns are steel, the lateral force resisting system will either consist of steel moment frames or braced frames. The levels below grade will remain constructed of cast-in-place concrete.

**Serviceability**

The deflection of this system was calculated in this report to be 1.66” for the joists and 1.35” for the girders, which are both within the permissible limits.

**Construction**

To achieve a 2 hour fire rating, the joists, girders, and deck must be fire proofed with spray on fireproofing. Despite this, steel joist erection is more rapid and efficient than cast in place concrete construction, therefore the construction schedule should be significantly reduced.
**Advantages**

- Potential reduction in construction schedule due to simply erection
- Shallow structural depth in the slab region
- Reduced structural weight
- Open webs can be used as raceways for mechanical and electrical pipes
- Increase rentable space due to wider bays created by longer spans

**Disadvantages**

- Lightweight floor system prone to vibration
- Increase in construction cost due to required fire proofing
- Requires uniform column framing layout

The composite joist/steel girder floor system increases the current structural depth to 27.6" and requires an increase in the overall building height to achieve high floor-to-ceiling heights. Since the existing building is limited to a 130 ft. height, this alternative floor system cannot be used with the existing 11 story structure. As a result, this system is feasible if either the building were designed for a reduced number of stories or relocated to a region that does not have a height limit.
# Floor System Summary

Table 5 summarizes the results that were discussed in this technical report.

<table>
<thead>
<tr>
<th>Consideration</th>
<th>Two-Way Flat Slab</th>
<th>Composite Steel Beam/Girder</th>
<th>Post-Tensioned Concrete Slab</th>
<th>Composite Steel joist/ Steel Girder</th>
</tr>
</thead>
<tbody>
<tr>
<td>General Weight (psf)</td>
<td>100</td>
<td>81.2</td>
<td>87.5</td>
<td>85</td>
</tr>
<tr>
<td>Cost ($/SF)</td>
<td>17.45</td>
<td>19.83</td>
<td>17.45 + Post-tensioning</td>
<td>22.05</td>
</tr>
<tr>
<td>Floor Depth (inches)</td>
<td>8 slab/8 drop panel</td>
<td>7.5 slab/21 girder</td>
<td>7 slab/3 drop panel</td>
<td>6 slab/21.6 girder</td>
</tr>
<tr>
<td>Architectural Fire rating (hour)</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Other impacts</td>
<td>N/A</td>
<td>20.5” increase in structural depth; beams, girders, and deck must be fireproofed</td>
<td>Under side of slab Can be left exposed as a finishing; 5” decrease in column region</td>
<td>19.6” increase in structural depth; joists, girders, and deck must be fireproofed</td>
</tr>
<tr>
<td>Structural Foundation Impact</td>
<td>Existing shallow foundation with spread footings and strap beams</td>
<td>May not impact foundation</td>
<td>May not impact foundation</td>
<td>May not impact foundation</td>
</tr>
<tr>
<td>Lateral System Impact</td>
<td>Existing concrete moment frame</td>
<td>Steel moment/braced frames</td>
<td>Concrete moment frame consisting of slab and columns</td>
<td>Steel moment/steel braced frames</td>
</tr>
<tr>
<td>Serviceability Maximum Deflection (inches)</td>
<td>N/A</td>
<td>1.73 beams/1.3 girders</td>
<td>N/A</td>
<td>1.66 joists/1.35 girders</td>
</tr>
<tr>
<td>Vibration Control</td>
<td>Very Good</td>
<td>Average</td>
<td>Very Good</td>
<td>average</td>
</tr>
<tr>
<td>Construction Additional Fire Protection Required</td>
<td>None</td>
<td>Spray on fireproofing for beams, girders and deck</td>
<td>None</td>
<td>Spray on fireproofing for joists, girders, and deck</td>
</tr>
<tr>
<td>Schedule Impact</td>
<td>N/A</td>
<td>May reduce construction schedule</td>
<td>May reduce construction schedule</td>
<td>May reduce construction schedule</td>
</tr>
<tr>
<td>Feasibility</td>
<td>Yes</td>
<td>Easy</td>
<td>Moderate</td>
<td>Easy</td>
</tr>
</tbody>
</table>

**Table 5** Floor System summary chart
Conclusion

This technical report further investigated the existing structural system by spot checking existing structural members as well as designing three alternative floor framing systems to determine which alternative system would be most viable. Each system was compared based on the following criteria:

- Architecture (fire rating and other impacts);
- Structural (foundation and lateral system impacts);
- Serviceability (maximum system deflection, vibration control and sound transmission);
- Construction (additional fire protection and schedule impact)

For the existing system, spot checks were performed for an interior flat slab panel and an interior column. Spot checks performed on a typical interior flat slab panel showed that the analysis simplifications resulted in a conservative slab design, which can be explained through both simplifying and dead load assumptions. On the other hand, the interior column spot check showed that the preliminary designed cross sections for levels 1 and 5 were very close to the existing cross-sections.

The three alternative systems designed for this tech report were:

- Composite beam/girder system with composite steel deck
- Two-way post-tensioned slab
- Composite joist/steel girder system with composite steel deck

The final design of the alternative floor systems resulted in the following:

- Two-way flat slab system: 8” thick slab with 8” thick drop panels
- Composite steel beam/girder system: a W16x31 beam with (32)- ¾” ϕ shear studs and a 2” camber and a W21x50 girder with (28)- ¾” ϕ shear studs
- Two-way post-tension slab: 7” thick slab with 3” thick drop panels and (26) - ½” ϕ 7-wire unbounded tendons in the N-S (banded) direction and (18) - ½” ϕ 7-wire unbounded tendons in the E-W (distributed) direction.
- Composite joist/steel girder system: 14CJ1400/607 composite joist with (40)-⅝” ϕ shear studs and a W21x93 girder

After designing each system and using the above criteria for system comparison, it was found that all 3 alternative systems were viable and will be further investigated to determine which one would be the better floor framing alternative.
Appendix A: Existing System Gravity Load Calculations

Step 1: Slab thickness

From Table 9.5.5.2: in ACI 318-08:
interior panel with drop panels and Fy = 60 ksi:

\[
L_{\text{min}} = \frac{L}{2} = \frac{26}{2} = 13\text{ in.}
\]

Step 2: Determine moments in column strip:

Simplifying assumption - determine column strip moments by analyzing the slab as a flat plate system (neglecting the drop panels)

Calculation strip width:

\[
\frac{L}{2} = 17.5\text{ in.}
\]

Step 1:Slab thickness check:

Check typical interior slab panel in E-W direction for slab thicknesses and column strip reinforcement.

\[
L_x = 8000\text{ psi},
F_y = 60\text{ ksi}
\]
Gravity spot check -
Flat slab interior panel

- total load
  dead:
  slab - 180 psf \( \times \frac{6}{12} \text{ ft} \) = 90 psf
  conv. - 10 psf
  live load - 100 psf

  \[ W_d = 0.2 \left( 90 \text{ psf} + 1.4 \times 100 \text{ psf} \right) = 220 \text{ psf} \]

- \( M_0 = \frac{W_d L_1}{12} \left( C_{0.29} \text{ kips} \right) \left( \frac{1}{2} \text{ ft} \right) = 1138 \text{ kip-ft} \)

- distribute \( M_0 \) longitudinally using ACI direct design moment coefficients, Sec 11.4.8.2

\[
\begin{align*}
\delta M_0 &= 417.4 \text{ kip-ft} \\
-776 \text{ kip-ft} \\
-353 \text{ kip-ft}
\end{align*}
\]

Interior span

- Transverse distribution of moments on column strip - from ACI Sec 11.4.4

\[
\begin{align*}
\frac{E_{c1}}{\alpha} &= 0.74 \\
\frac{E_{c2}}{\alpha} &= 0 \text{ (no longitudinal beam support)}
\end{align*}
\]

\[
\begin{align*}
\frac{\delta M_0}{x_1} &= 77.3 \text{ kip-ft} \\
\text{to col. strip} &= 281.5 \text{ kip-ft}
\end{align*}
\]

Positive

\[
\begin{align*}
\frac{\delta M_0}{x_1} &= 0.74 \\
\frac{\delta M_0}{x_1} &= 0
\end{align*}
\]

- 0.74 to 1.0

147.6 kip-ft - to col. strip = 281.5 kip-ft
Gravity Spot Check -
First Slab interior panel

**Step 2: Reinforcement design of column strip in interior panel**

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
<th>Interior Span</th>
<th>$M_u$</th>
<th>$M_u T$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>$R_e (kN/m)$</td>
<td>775</td>
<td>472.6</td>
<td></td>
</tr>
<tr>
<td>2.</td>
<td>Width of Strip, $b$ (in)</td>
<td>131</td>
<td>121</td>
<td></td>
</tr>
<tr>
<td>3.</td>
<td>Effective depth, $d$ (in)</td>
<td>7.74</td>
<td>7.59</td>
<td></td>
</tr>
<tr>
<td>4.</td>
<td>$M_u = \frac{A_u (kN)}{a_m}$</td>
<td>$-R_{sl}$</td>
<td>114</td>
<td></td>
</tr>
<tr>
<td>5.</td>
<td>$R = \frac{M_u}{\text{Area}}$</td>
<td>670</td>
<td>361</td>
<td></td>
</tr>
<tr>
<td>6.</td>
<td>From Eq. 2.4.1 (Reinforced concrete, 1st Ed.)</td>
<td>0.0022</td>
<td>0.0022</td>
<td></td>
</tr>
</tbody>
</table>

From interpolation,

$$f = \frac{R - R_1}{R_{2c} - R_1} f_{c2}$$

| 7.   | $A_e = \phi d^2$ (in$^2$) | 18.92    | 9.27  |
| 8.   | $A_{f, min} = 0.004$ | 3.1      | 3.1   |
| 9.   | $N = \text{larger of } T + R_1$ | 91.5     | 91.5  |

To minimize the number of reinforcing bars required, increase bar size to $\bar{A}_e$.

Therefore to resist the positive moment in the middle of the interior slab span, use $N = \frac{18.92}{0.75}$ bars

To resist the negative moment at the supports use $N = \frac{114}{0.75}$ bars.
Gravity split check - column B5a

Element area, Ar
Ar = (9.8')^2(3/1') = 33.8 sq ft

Influence area/Floor = (7.9')^2(1.1')
= 55.9 sq ft

Level:
- Main roof: 11' = 100 psf + 10 psf = 110 psf
- Levels 2-12: 14' = 100 psf + 10 psf = 110 psf

Analysis procedure:
1. Determine axial load and unbalanced moment on column at levels 1 and 5
2. Determine preliminary size of column at levels 1 and 5 based on gravity loads
Axial load on column at level 1

1. Load above level 1: roof + 11 floors

\[ \text{Axial load factor} = \frac{0.14}{1 + 1.5} = 0.08 \]

1. Use Axial load = 0.14

\[ P_c = L \times A_c \times f_c = 0.14 \times (100 \text{ psi}) \times (11 \text{ floor}) \times (0.7 \text{ ft}^2/\text{psi}) = 487 \text{ kips} \]

\[ \begin{align*}
P_u &= 110 \text{ psi} \times (720 \text{ ft}^3/\text{ft}^2) = 80,940 \text{ kips} \\
P_e &= 100 \text{ psi} \times (720 \text{ ft}^3/\text{ft}^2) = 72,000 \text{ kips} \\
P &= 1.2P_u + 1.6P_e + 0.25P_e = 1.2(487 \text{ kips}) + 1.6(720 \text{ kips}) + 0.25(720 \text{ kips}) = 2,442 \text{ kips} 
\end{align*} \]

- Use the ACI moment coefficient method to determine the maximum moments and shears at the critical sections.

1. Negative moment at exterior face of 1st interior support:

\[ F_{max} = \frac{wL^2}{10} \]

1. Negative moment at other faces of interior supports:

\[ F_{max} = \frac{wL^2}{11} \]

Notes: L is the clear distance between the supports, but for preliminary design purposes, C will use the clear distance with the assumption that the column lines are unknown at this stage of preliminary column sizing.
Gravity Spot Check -
column #50

Check 1

Midleft of support = \( \frac{9.14 \times 16}{2} \left( \frac{12.5 + 12.5}{2} \right) \) = 98.6 kip

Midright of support = \( \frac{9.14 \times 16}{2} \left( \frac{21.2}{2} \right) \) = 85.6 kip

Use 98.6 kip to be conservative

Preliminary column size for level 1

Assume load on no. 1 floor = t'c = 8000 psf and A'c = 10"

\( c = \frac{A'u}{A'} = \frac{800 \times 11}{2302} = 0.35 \) in

Assume \( d'' = 2.0 \) in

Set target reinforcement ratio to about \( \phi_p = 0.75 \times 0.03 \)

<table>
<thead>
<tr>
<th>h</th>
<th>y</th>
<th>cd'</th>
</tr>
</thead>
<tbody>
<tr>
<td>22</td>
<td>0.773</td>
<td>16.5</td>
</tr>
<tr>
<td>24</td>
<td>0.791</td>
<td>15.8</td>
</tr>
<tr>
<td>26</td>
<td>0.811</td>
<td>15.5</td>
</tr>
<tr>
<td>28</td>
<td>0.830</td>
<td>15.2</td>
</tr>
<tr>
<td>30</td>
<td>0.850</td>
<td>14.9</td>
</tr>
</tbody>
</table>

Assumption: No use of design aid interaction diagrams for determining the preliminary column size for level 1. Use a specific concrete strength of 6000 psi in place of the existing column's specific concrete strength of 4000 psi.

Using Fig. 8-16b from Reinforced Concrete Mechanics and Design, 5th edition, for \( f'_c = 0.72, f'_u = 0.03 \) and \( V_p = 0.17 \)

Then \( \phi_p/n = 2.9 \) for which \( b' = \frac{\phi_p}{n} = \frac{2.9}{0.17} = 171.5 \text{ in} \)

Thus \( 28 \times 36 \) = try 30 x 36" column

Ex. Fig. 8-16b

\( \phi_p/n = \frac{23.3}{20.50} = 1.14 \)

\( \phi_p/n = \frac{384 \times 12}{28 \times 36} = 0.90 \)

returns \( \phi_p = 1.35 \)
G. Johnson

Structural Option

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October 23, 2011

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From Eq. A.18c (δ' = 0.70)

\[ \frac{b}{h} = 2.62 \]

\[ \frac{d_{min}}{b} = 0.40 \]

\[ \frac{A_{min}}{b} = 0.11 \]

Interpolation gives \( \rho = 0.113 - 0.75 (C_{11} - 1.2) + 1.19 \% \) for \( C_{11} = 0.12 \)

- \( A_{min} = \rho \cdot A_{g} = 0.019 (30.3 	imes 30) = 10.51 \text{ in}^2 \)

- \( \frac{d_{min}}{b} = 0.22 \text{ in} \)

Use a 30" x 30" column reinforced with (6) #10

Use a 30" x 30" column reinforced with (6) #10

Assumption: Column is subjected to only

- gravity load: check the column

- pure axial compressive strength

- for fixed column, \( \delta_{net} = 0.8 \theta \left( \frac{1}{C_{11}} \left( h - A_{g} \right) + \frac{1}{E_{g}} \right) \)

- \( C_{11} = 0.04 \text{ in} \), \( h = 30 \text{ in} \), \( A_{g} = 0.35 \text{ in}^2 \)

Existing design was a 30" x 30" column on level 1 with \( f' = 5000 \text{ psi} \) and (6) #10 reinforcing bars. In addition, the column at level 1 has a shear, but for simplification purposes, the shear was neglected.

Cross-section percent error: \( \frac{124.32 - 93.0}{24.32} \) is also \( \approx 4 \) %.
Gravel Spot Check -
column E5o

Axial load on column at level 2

- Load above level 5: roof + 7 floors

\[ F_{\text{red}} = \frac{1.40}{1.22 + 0.5} = 0.336 \]

\[ F_{\text{red}}/\text{code} = 0.336 \]

\[ R = 0.48 \text{ ksi} (270 \text{ psi}) (2.7 \text{ ft}^2) (270 \text{ ksi}/\text{ft}) = 772 \text{ kN} \]

\[ P_1 = \frac{1.10 \text{ psi}}{1.22 + 0.5} (270 \text{ psi}) = 230 \text{ kN} \]

\[ P_2 = \frac{1.10 \text{ psi}}{1.22 + 0.5} (270 \text{ psi}) = 195.5 \text{ kN} \]

\[ P_3 = \frac{1.10 \text{ psi}}{1.22 + 0.5} (270 \text{ psi}) = 183.3 \text{ kN} \]

Unbalanced Moment for column at level 3

Some as 4'K for column at level 1 (Refer to pg. 2).

Preliminary column size for level 2

- Assume butt at all 4 faces, \( F_{\text{c}} \geq 6000 \text{ psi} \) and \( F_{\text{c}} = 60 \text{ ksi} \)

\[ \epsilon = \frac{M_u}{P_u} = \frac{480 \times 12}{1533} = 23.2\% \]

- Assume \( d' = 2.3' \)

Set target reinforcement ratio to \( 2\% \)

<table>
<thead>
<tr>
<th>h</th>
<th>( f_y )</th>
<th>( f/h )</th>
</tr>
</thead>
<tbody>
<tr>
<td>21&quot;</td>
<td>0.772</td>
<td>0.28</td>
</tr>
<tr>
<td>25&quot;</td>
<td>0.772</td>
<td>0.28</td>
</tr>
<tr>
<td>30&quot;</td>
<td>0.772</td>
<td>0.28</td>
</tr>
<tr>
<td>32&quot;</td>
<td>0.772</td>
<td>0.28</td>
</tr>
</tbody>
</table>

- Using \( f_{\text{ck}} = 4.1 \text{ ksi} \), Reinforced Concrete: Mechanics and Behavior, 5th edition,
  for \( f_y = 0.25 \), \( f_y = 0.07 \text{ ksi} \) and \( f_{\text{ck}} = 4.1 \text{ ksi} \) (average of \( f_{\text{ck}} \) values above).

- Then \( f = 0.8 \) for which \( h = 13/2 \times 0.07 = 0.440 \text{ in} \)

\[ = 3 \text{ ksi} = 23.5 \rightarrow \text{try 2' x 4', 36" column} \]
Gravity spot check -
column 930

\[ \frac{f_{ct}}{f_{t}} = \frac{f_{ct}}{f_{c}} = 2.13 \]

\[ \frac{f_{ct}}{f_{t}} = \frac{f_{ct}}{f_{c}} = 0.50 \]

required $f_{ct} = 1.89$.

\[ P_0 = 0.933 - 0.75 \] 

\[ \rho_5 = 0.012 \] 

\[ A_{s, min} = \rho_5 \] 

\[ 0.95 \] 

\[ m = 1.0 \text{ in}^2 \]

8 bars

assumptions: column is subjected to only
gravity load. Check the column for pure axial compressive strength.

Pure compression: $C = 0$, $E_C = E_t = 0.003$

For tied columns, $\phi P_t = 0.8 f_{t} A_{s} \left[ 0.624 \left( 0.050 \right) + 0.005 \left( 0.600 \right) \right]$

$\phi = 0.816 K > P_0 = 0.816 K << \phi$

The existing column at level 5 is 20 x 20 in with (2) #11 reinforcing bars and $f_t = 6000$ psi.

cross-section moment factor = \[ \frac{180000 - 75000}{20 \times 20} = 7.5 \]
Step 1: Ground snow load, \( p_g \) — from Fig. 7-1, \( p_g = 15 \) psf

Step 2: Exposure factor, \( C_e \) — from Table 7-2, Terrain category II, roof fully exposed
\[ C_e = 0.9 \]

Step 3: Thermal factor, \( C_t \) — from Table 7-2, \( C_t = 1.0 \)

Step 4: Importance factor, \( C_i \) — from Table 1.2-1, \( C_i = 1.0 \) (Sec. II)

Step 5: Flat roof snow load, \( R_f \) — see Sect. 7.2.1
\[
R_f = 0.7\ C_e\ C_t\ p_g \\
\leq 0.7\ \times 0.9\ \times 15 = 9.45\ \text{psf}
\]

Snow drift: Penthouse level

Step 6: Maximum intensity of the drift surface load, \( P_d \) — see Sect. 7.2.1
\[
P_d = \frac{h d^2}{2g} = \text{snow density,} \ 8 = 0.13 p_d \text{ vs.} \ 8 = 0.12\ C_e\ C_t\ p_g \\
\geq 19.22\ \text{psf} \times \text{roof}
\]

Leftward drift

\( \theta = 30\ \text{deg.} \)
\[ h_d = \frac{0.53 \times 183 - 25.10 - 15}{1.35} = 5 \text{ ft}
\]

Rightward drift

\( \theta = 45\ \text{deg.} \)
\[ h_d = \frac{0.43 \times 183 - 25.10 - 15}{1.35} = 6 \text{ ft}
\]
Snow drift

E-W wind: 

\[ \frac{P_r}{k} = \frac{15.38 \text{ psf}}{13.24 \text{ psf}} = 0.90 \text{ ft} \]

Step 2: balanced snow load height, \( h_b \) from sect 7.1

Step 3: \( h = 14.85 - 13.01 = 1.84 \text{ ft} \)

Step 4: \( h_c = 1 - h_b = 18.5' - 0.1' = 17.6' \)

Step 5: total snow load \( P = P_b + P_c \)

E-W wind

N-S wind

Since \( P_{tot} E-W > P_{tot} N-S \), use \( P_{tot} E-W \) for a conservative design.
Appendix B: Alternative 1 – Composite Steel Floor System with Composite Deck

<table>
<thead>
<tr>
<th>Composite Steel Floor System</th>
<th>Tech 2</th>
<th>page 1 of 9</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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</tr>
</tbody>
</table>

Composite floor deck design - use Varistet steel floor and floor deck manual to select composite deck.

1. Superimposed load on deck:
   - total live load = 100 psf
   - total dead load = 10 psf + slab + deck
   - total live = 110 psf + slab + deck

2. Clear-to-clear span of supports = 10' - 0"

3. Number of spans = 8 + span condition

4. Deck type
   - Select 1st composite floor deck in which
     - clear span = clear-to-clear span of supports
     - superimposed uniform load 2 required total load
     - deck max see clear span = clear-to-clear span of supports

   For 2 hr fire assembly ratings, use an unprotected deck with a 3/8" normal weight concrete topping.

   Use steel composite deck with 3.5" thick slab (topping = 3/8") and steel wire 76 psf.

   - clear span = 10' - 0"
   - superimposed uniform load = 110 psf ≥ required total load = 110 psf, OK
   - deck max see clear span = 10' - 0" ≥ clear-to-clear span of supports ≥ 10' - 0", OK
Composite Steel Floor System

1. Total Load

   a. Dead Load:
      - 500 lb
      - 75 lb
   
      \[ W_d = (10 \text{ psf} + 75 \text{ psf}) \cdot 10 \text{ ft} = 850 \text{ psf} \]

   b. Live Load:
      - 100 psf
   
      \[ L = 100 \times 0.817 = 81.7 \text{ psf} \]

\[ W = W_d + L \cdot 0.817 = 837.7 \text{ psf} \]

2. Required moment for the composite beam - Assumes beam is simply supported

   a. Add 5 psf for live load
   
   \[ W_u = 1.2 \cdot (5 \text{ psf}) \cdot 10 \text{ ft} + 1.6 \cdot (837.7 \text{ psf}) \]
   
   \[ M_u = \frac{W_u \cdot L^2}{4} = \frac{23 \cdot 10^3 \text{ psf} \cdot 10^2 \text{ ft}^2}{4} = 236,654 \text{ ft}^3 \leq 366 \text{ ft} \cdot \text{in} \]

3. The starting moment arm for the concrete from the top of the steel

   a. Assume \( h = 10 \text{ in} \)
   
   \[ \gamma = \frac{6 - \frac{h}{2}}{12} \approx 7.5 - \frac{7.5}{3} = 7 \text{ in} \]
- determine the lower bound moment (\( M_{LB} \)) based on \( \delta_{L,\text{max}} \) and \( \delta_{L,\text{max}} \):

\[
\delta_{L,\text{max}} = \frac{V}{E I} = \frac{F_1}{E I} = L.3
\]

\[
M_{LB} = \frac{E I L^4}{4} = \frac{F_1 L^4}{4} = 118 \text{ in}^4
\]

- Select potential W-shapes from tables 2-19 and 2-20 in steel manual (select members with \( \phi A_n = 270 \text{ kft} \), and \( L = 118 \text{ in} \)):

<table>
<thead>
<tr>
<th>Shape</th>
<th>( \phi A_n ) (kft)</th>
<th>( L ) (in)</th>
<th>( Q_n ) (kft)</th>
<th>( C ) (F)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W14 x 30</td>
<td>270</td>
<td>118</td>
<td>349</td>
<td>17</td>
</tr>
<tr>
<td>W14 x 36</td>
<td>270</td>
<td>118</td>
<td>397</td>
<td>17</td>
</tr>
<tr>
<td>W16 x 36</td>
<td>270</td>
<td>118</td>
<td>397</td>
<td>17</td>
</tr>
<tr>
<td>W16 x 38</td>
<td>270</td>
<td>118</td>
<td>419</td>
<td>17</td>
</tr>
</tbody>
</table>

- Determine horizontal shear strength for shear stud:

- Using table 2-21 in steel manual, determine \( Q_n \) for shear stud:
  - Deck parallel to, assumed weak stud position
  - Assume 1 stud/16
  - Use 7/16 stud:

\[
\phi Q_n = 17.3 \text{ k}
\]

- Determine # of studs/ft for shapes listed in Step 5:

<table>
<thead>
<tr>
<th>Shape</th>
<th>( \phi Q_n ) (k)</th>
<th>( \frac{x}{L} )</th>
<th>( \phi Q_n \frac{x}{L} ) (kft)</th>
<th>( \frac{x}{L} ) (kft)</th>
<th>Use studs/ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>W14 x 30</td>
<td>17.3</td>
<td>0.25</td>
<td>4.3</td>
<td>0.25</td>
<td>2</td>
</tr>
<tr>
<td>W14 x 36</td>
<td>17.3</td>
<td>0.25</td>
<td>4.3</td>
<td>0.25</td>
<td>2</td>
</tr>
<tr>
<td>W16 x 36</td>
<td>17.3</td>
<td>0.25</td>
<td>4.3</td>
<td>0.25</td>
<td>2</td>
</tr>
<tr>
<td>W16 x 38</td>
<td>17.3</td>
<td>0.25</td>
<td>4.3</td>
<td>0.25</td>
<td>2</td>
</tr>
<tr>
<td>W18 x 38</td>
<td>17.3</td>
<td>0.25</td>
<td>4.3</td>
<td>0.25</td>
<td>2</td>
</tr>
</tbody>
</table>
1. Evaluate beam listed in step 3 for economy

\[ \text{w} = 1.4 \times 3.0 \text{ w} \text{ 6\# steel:} \]

\[ 3.0 \text{" x } 3.0\text{" + 6\# steel x } 15'\text{/stud x } 1.870 = \]

\[ 1.4 \times 3.0 \text{ w} 24\# steel : \]

\[ 2.2 \text{" x } 2.2\text{" + 24\# steel x } 10'\text{/stud} = 1450 \]

\[ \text{w} = 6.0 \text{ w} 3.0\# steel \]

\[ 7.2 \text{" x } 7.2\text{" + 3.0\# steel x } 15'\text{/stud} = 1482 \]

Try \text{w} = 3.0\# since it's more economical.

2. Check the depth of the compressive concrete block, \( a \)

\[ a = \frac{2F_{Oa}}{0.85 \pi D} \]

\[ \text{w} = 3.0\#; a = 2.74 \text{" assumed 1" , ok } \]

\[ \text{min} \left( \frac{1}{4} h_{eb}, \frac{1}{2} h_{eb} \right) \]

\[ = \frac{2F_{Oa}}{0.85 \pi D} \]

\[ = 2\pi \text{ 2.74"} \]

\[ = \frac{2\pi (2.74)}{0.85} \]

\[ = 38.5 \pi \]

\[ = 120 \text{"} \]

\[ h_{eb} = 120 \text{"} \]

3. Check unshored strength

\[ \text{w} = 3.0\#; D_B = 203 \text{ kft} (\text{obtained from table 3-19 in steel manual}) \]

\[ W_U = 1.3D_B + 1.0 \text{ kip/mc} = 1.3(203) + 1.0(55) = 273 \text{ psi} = 1.23 \text{ kip/ft only} \]

\[ W_U = 1.2D_B + 1.0 \text{ kip/mc} = 1.2(203) + 1.0(55) = 273 \text{ psi} = 1.23 \text{ kip/ft} \]

\[ M_U = \frac{1.3D_B (2.74)^2}{2} = 192 \text{ kip} < 203 \text{ kip} \text{ ok for no shoring} \]
5. Check wet concrete deflection

\[ W_{pc} = 1266 \text{ kfl} + 6.6 \text{ psi sec-ut} = 75 \text{ psi (10 psi)} + 31 \text{ psi} = 78 \text{ psi} = 0.78 \text{ kfl} \]
\[ \Delta_{wc} = \frac{560,311 \text{ ksi} (12.7)}{373} \approx 2.41\text{ in} \]
\[ \Delta_{wc,max} = \frac{373}{900} = 1.75\text{ in} \]

Since \( \Delta_{wc} > \Delta_{wc,max} \), consider beam:

\[ c = 0.70 (2.41\text{ in}) = 1.71\text{ in} \quad \rightarrow \quad \text{use } c = 1.71\text{ in} \]

6. Check LL and TL deflection

\[ W_{LL} = 0.87 \text{ kfl} \]
\[ \frac{13}{7} \text{ in point } V = 96 \text{ ksi} = 231 \text{ kfl}, \quad \Gamma_{LL} = 340 \text{ in}^4 \]
\[ \Delta_{LL} = \frac{560,311 \text{ ksi} (12.7)}{340} \approx 1.81\text{ in} \]
\[ \Delta_{LL,max} = 1.77\text{ in} \quad \rightarrow \quad \Delta_{LL} = \frac{373}{900} = 1.75\text{ in} \]

\[ \Delta_{TL} = \frac{560,311 \text{ ksi} (12.7)}{340} \approx 1.77\text{ in} \]

7. Check \( M_y \), \( V_y \), and \( b_n \) sec wt assumption for \( wibx26 \):

\[ M_y = 344,661 \text{ kfl} < 750,000 \text{ kfl} \quad \rightarrow \quad \text{ok} \]
\[ V_y = 41,666 \text{ kfl} < 750,000 \text{ kfl} \quad \rightarrow \quad \text{ok} \]

Sec wt assumption:

\[ 31 \text{ psi} = 3.1 \text{ psi} \quad \rightarrow \quad \text{ok} \]

Use \( wibx26 \) beam with 32 studs and \( c = 2\text{ in} \)
Composite steel girder system

Interior composite girder design 61

10'

25'-6"

25'

Dead load:
- Slab = 25 psf
- 50c = 10 psf
- Bm self wt allowance = 5 psf

Live load:
- 60 c 100 psf

Influence span 2(25 + 25) / 120 = 1.89 ft

L = 100 x \( \frac{0.30}{0.35 + \frac{15}{1875}} - 0.40 \)

L = 100 (0.64) = 64 psf

Wu = L x (25 + 10 + 5) + 1.4 (60) = 210 psf

Pw = 2.10 psf (22.5 + 22) / 2 = 68.6 k

\[ P_w \downarrow P_{ei} \downarrow \Delta \]

10'

10'

10'
(a) Determine $T_{EL}$ based on $A_{LV}$, $V_{LV}$ and $A_{LV \text{ max}}$

\[ A_{LV, \text{ max}} = \frac{1}{200} \times \frac{300(1)}{200} = 1.5'' \]

\[ A_{LV, \text{ max}} = \frac{1}{200} \times \frac{300(1)}{200} = 1.5'' \]

\[ T_{EL, \text{ min}} = \frac{0.025 L_{EL}(C_{11} + C_{22})}{E_{LV}} \]

\[ T_{EL, \text{ min}} = \frac{0.025 L_{EL}(C_{11} + C_{22})}{E_{LV}} \]

\[ T_{EL} = 0.025 L_{EL}(C_{11} + C_{22}) \]

\[ T_{EL} = 0.025 L_{EL}(C_{11} + C_{22}) \]

\[ T_{EL} = 0.025 L_{EL}(C_{11} + C_{22}) \]

\[ T_{EL} = 0.025 L_{EL}(C_{11} + C_{22}) \]

\[ T_{EL} = 0.025 L_{EL}(C_{11} + C_{22}) \]

(b) Select potential $w_{PV}$ from Table 7-14 and 7-20 in steel max

Assume $T_{EL} = 6.2''$ ($n = 2$)

\[ w_{PV} = 90'' \]

\[ T_{EL} = 700'' \]

\[ w_{PV} = 100'' \]

\[ T_{EL} = 800'' \]

\[ w_{PV} = 110'' \]

\[ T_{EL} = 900'' \]

\[ w_{PV} = 120'' \]

\[ T_{EL} = 1000'' \]

\[ w_{PV} = 130'' \]
2) Determine horizontal shear strength for shear stud
   - Deck parallel
     \[ \frac{V}{A} = \frac{9,720}{3.5} = 2,800 \text{ kips} \]
   - Assume weak position
   - Use 3/4" 3B stud
   - Assume 4 x 10' beams
   - Min at 

3) Determine # of 3/8" border studs listed in step 2
   - 76.25" stud border
   - 27.25" stud border
   - 15.25" stud border

5) Evaluate girders listed in step 4 for economy
   - 8' 3/8" with 56 studs
   - 8' 3/8" with 56 studs
   - 8' 3/8" with 56 studs
   - 8' 3/8" with 56 studs
   - 8' 3/8" with 56 studs
   - Try 8' 3/8" to reduce structural depth

6) Check the compressive concrete block depth, \( h \)
   - \( h = \left( \frac{p}{w} \right)^{1/2} \)
   - \( h = \left( \frac{1,750}{2,800} \right)^{1/2} \approx 2.75 \text{ ft} \)
(3) Check stressed strength

\[ \begin{align*}
W_{1/2} & = 40, \quad \phi_{b}\cdot M_{p} > 274 \text{k}
\end{align*} \]

\[ P_{u} = 1.2 \left[ 75 \text{ kft} + 40 \right] (31.25) + 1.6(30 \times 10^3) (31.25) \approx 39,600 \text{k}
\]

\[ M_{u} = 525 \text{ kft} \times 12 = 6,300 \text{kft} \text{ in.} \quad \text{ok since } M_{u} < P_{u} \]

\[ \text{Select larger w-shape} \]

(4) Check EE and EI deflection

\[ \begin{align*}
\Delta_{le} & = \frac{0.086(90)(3)(3)}{27000 (31.25)} < 0.06 < \Delta_{l, \text{max}} \quad \text{ok}
\end{align*} \]

\[ \Delta_{lc} = \frac{0.086(90)(50)^{3}}{27000 (31.25)} < 0.36 < \Delta_{l, \text{max}} \quad \text{ok}
\]

(5) Check \(M_{u}, V_{u}\) and self girders at assumption

\[ M_{u} = 656 \text{ kft} < \phi_{b} M_{p} = 693 \text{ kft} \quad \text{ok}
\]

\[ V_{u} = 656 \text{ kft} < \phi_{b} V_{p} = 832 \text{ kft} \quad \text{ok}
\]

Self-wt assumption = 50 psf \( \frac{50 \times 12}{31.25} = 1.60 \text{ psf} < 5 \text{ psf} \quad \text{ok}
\]

Use \(W_{1/2}\) girder with tee stud
Appendix C: Alternative 2 – Two-Way Post-Tensioned Slab Floor System
Two-way post-tensioned slab

Loads:
- Framing dead load + self wt
- Superimposed dead load = 10 psf
- Live load = 100 psf
- 2 hour fire rating

Materials:
- Concrete: New 120 psi
  - $f_c = 5000$ psi
  - $E_c = 2000$ psi
- Rebar: $f_y = 65,000$ psi
- PT: Unbonded tendons
  - 9/16" 7-wire strand, $A_t = 0.183$ in²
  - $f_p = 270$ ksi
- Estimated prestress losses = 15 ksi
- $f_p = 0.7(180 ksi) = 126$ ksi
- $f_p = 0.7(150 ksi) = 105$ ksi
- Prestress = $A_t f_p = 0.183(126) = 23.3$ kf/ft/ton

Preliminary slab thickness:
- Short with $h = \frac{8b}{k}$
- Shortest span = 26'
- $h = \frac{26(1.15)}{2} = 8.5''$ preliminary slab thickness

Loading:
- $w_t = 120$ psi + 100 psi + 100 psi = 220 psi
- $w_c = 10$ psi
- $L_0 = 100$ psi
- $A_t = 0.183$ in²
- $f_{pt} = 270$ ksi

- 32' x 31.5' frame: $k_{ll} A_t = 0.71(0.232) = 0.16$ ksi > 0 psf, ok to reduce LL
  - $L = 1000 \sqrt{\frac{0.16}{0.71}} = 72$ psf
- 32' x 31.5' frame: $k_{ll} A_t = 0.71(0.232) = 0.16$ ksi > 0 psf, ok to reduce LL
  - $L = 1000 \sqrt{\frac{0.16}{0.71}} = 72$ psf
- 32' x 31.5' frame: $k_{ll} A_t = 0.71(0.232) = 0.16$ ksi > 0 psf, ok to reduce LL
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- 32' x 31.5' frame: $k_{ll} A_t = 0.71(0.232) = 0.16$ ksi > 0 psf, ok to reduce LL
  - $L = 1000 \sqrt{\frac{0.16}{0.71}} = 72$ psf
Two-way Post-tensioned Slab

E-W LL reduction

30' x 30' bay:
\[ k_{LL} = \frac{14(30)}{200} \geq 0.05 \text{in.} \text{ psf to reduce LL} \]
\[ L = 100 \text{ in.} = 7.5 \text{ psf} \]

25' x 30' bay:
\[ k_{LL} = \frac{14(25)}{200} \geq 0.05 \text{in.} \text{ psf to reduce LL} \]
\[ L = 100 \text{ in.} = 7.5 \text{ psf} \]

25' x 25' bay:
\[ k_{LL} = \frac{14(25)}{200} \geq 0.05 \text{in.} \text{ psf to reduce LL} \]
\[ L = 100 \text{ in.} = 7.5 \text{ psf} \]

Design of H-2 interior frame

Use Equivalent Frame Method according to ACI 318-11
Total bay width between centerlines is 52.5' 
Ignore column stiffness in equations for hand calculation simplicity.
Since \[ \frac{k_{c}}{k_{s}} = 0.99 \geq 0.75 \] : pattern loadings not required
(to simplify preliminary calculations, neglect load patternings)

1. Calculate section properties
   Two-way slab must be designed as class V - see ACI 318 section 18.7.3
   Gross cross-sectional properties allowed - see ACI 318 section 18.7.4
   \[ A = \frac{bh^{2}}{6} = \frac{30(25)^{2}}{6} = 196.25 \text{ in.}^{2} \]
   \[ s = \frac{bh^{2}}{6} = \frac{30(25)^{2}}{6} = 392.5 \text{ in.}^{2} \]

2. Set design parameters
   Allowable stresses: class V
   at time of casting (ACI 318 section 18.7.1):
   \[ f'_{c} = 3000 \text{ psi} \]
   Compression: \[ f'_{c} \geq 0.4f'_{c} = 0.4(2000) = 800 \text{ psi} \]
   Tension: \[ 0.85f'_{c} = 0.85(2000) = 1640 \text{ psi} \]
   at service loads (ACI 318 section 18.7.1(a) and 18.7.2):
   \[ f'_{c} = 2000 \text{ psi} \]
   Compression: \[ f'_{c} \geq 0.4f'_{c} = 0.4(2000) = 800 \text{ psi} \]
   Tension: \[ 0.85f'_{c} = 0.85(2000) = 1640 \text{ psi} \]
Average precompression limits:

\[ P = 125 \text{ psi} \text{ min (ACI 318 Sect 17.2.8)} \]
\[ P = 700 \text{ psi} \text{ max} \]

Target load balances:

60% of BL (96,000 lb) for slabs
Use \( 0.6 \times 96,000 = 57,600 \text{ lb} \) for slab.

Cover requirements (assume 2-hr fire rating and carbonate aggregate)

Restraint slip = \( \frac{1}{6} \text{ in} \) bottom cover.

Initially assumed tendon profile

<table>
<thead>
<tr>
<th>Tendon ordinate</th>
<th>Tendon C.C.G. location</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exterior support-anchor</td>
<td>3.5&quot;</td>
</tr>
<tr>
<td>Interior support-top</td>
<td>7&quot;</td>
</tr>
<tr>
<td>Interior support-bottom</td>
<td>1.75&quot;</td>
</tr>
<tr>
<td>End span-bottom</td>
<td>1.75&quot;</td>
</tr>
</tbody>
</table>

\[ a_{min} = 0.5 \times 5" \]
\[ a_{end} = 3.5 + 1.75 = 7" \]

Presress force required to balance 45% of self-weight.

\[ W_{balance} = W_b = 0.45 \times W_{self} = 0.45 \times (96,000 \text{ lb}) = 43,200 \text{ lb} = 1.77 \text{ kif} \]

Force needed in tendons to counteract the load in the 32' bay (note: use post-tension beams to support the 32' bay).

\[ F = \frac{W}{P_{end}} = \frac{43,200 \text{ lb}}{3.5 \text{ ft}} = 12,342 \text{ lb} \]

Notes: since \( a_{end} < a_{min} \) at the end 32' bay to the longest bay, the 27.5' end span will sever the max. required post-tension force.
Check precompression allowance

determine number of tendons to achieve 711 k

\[ \text{# of tendons} = \frac{711 \text{ k}}{240 \text{ k/tendon}} \]

use 34 tendons; the actual force for bonded tendons \( P_{\text{actual}} = 5 \times 34 \text{ tendons} \times 23.4 \text{ k} \)

the balanced load for the end span is slightly adjusted

\[ W_b = \frac{209.6 \times 6.70 \text{ ksi}}{71} \times 1.37 \text{ ksf} \]

determine actual precompression stress

\[ P_{\text{actual}} = \frac{5 \times 34 \times 1000}{2425 \text{ in}^2} = 394 \text{ psi} \]

> 115 psi min.: ok

< 500 psi max.: not ok. Reduce balance or pretension.

Assumption: 50% of load to be balanced in E-W direction and 50% to be balanced by the N-S direction.

- Prestress force reqd. to balance 50% of self-wt k

\[ W_b = 0.50 \times (23.7 \times 31.23) \times 1.37 \text{ ksf} \]

- Force needed in tendons to counteract the load in the E-W bay

\[ P = 1.57 \times \frac{23.7^2}{2} \times 701 \text{ k} \]

- Check precompression allowance

\[ \text{# of tendons} = \frac{701}{24.4} = 28.4 \text{ use } 26 \text{ tendons} \]

\[ P_{\text{actual}} = 26 \times (24.6) = 648 \text{ k} \]
the balanced load for the end span

\[ W_b = \frac{691.6}{70} = 9.86 \text{ kfs} \]

**actual precompression stress**

\[ \frac{P_{\text{actual}}}{A} = \frac{691.6(1000)}{7025} = 9.86 \text{ psi} \]

- check interior spans

\[ f = 1.35 C (25)^{1/2} = 2.25 k < 691.6 k \text{ psi} \]

\[ \frac{w_b}{w_{\text{tot}}} = \frac{691.6(CP)(y_{10}^{1/2})}{27^2} = 1.36 \text{ kfs} \]

\[ w_b = \frac{w_{\text{tot}}}{f} \]

\[ \text{unacceptable for design} \]

- check exterior spans

\[ 14' \text{ exl. span}; \quad w_b = \frac{PP_{\text{tot}}}{L} \Rightarrow \theta_{\text{end}} = \frac{2.23 (275)}{691.6(CP)} \]

\[ \text{use } \theta_{\text{end}} = 1.0^o \]

\[ 37.5' \text{ exl. span}; \quad \theta_{\text{end}} = \frac{2.23 (375)^{1/2}}{691.6(CP)} \]

\[ \text{use } \theta_{\text{end}} = 3^o \]

**W-5 interior frame:**

- effective preivces force, \( P_{\text{eff}} = 691.6 k \text{ psi} \)

\( \theta \) check slab stress:

- dead load moment:

\[ W_{\text{tot}} = \frac{30 \times 103}{1000} (31.52 \text{ fts}) = 9.08 \text{ kifs} \]

\[ \begin{align*}
A & \quad A & \quad A & \quad A & \quad A \\
1' & \quad 21' & \quad 27' & \quad 37.5' & \quad 47' \\
\end{align*} \]

**Diagram:**

- 507.5 kft
- 707.5 kft
- 101.7 kft
- 72 kft
Two-way post-tension slab

- live load moments

\[ w_{LL} = 91 \text{ psf} (2200 \text{ lb/ft}) = 2.0 \text{ kF} \]

- total bending moment

\[ w_{H} = 3.5 \text{ kF} \text{ (coverage of 3 bays)} \]
Two-way post-tension slab

Stress: stress immediately after jacking (C, T, P)

- Tension
- Compression

+ Moment creates tensile stress at top and compresion at bottom

Mid-span stresses:

$E_{guy} = \frac{-M_{L} + M_{U}}{L} - P/A$

- $f_{guy} = E_{guy} + P/A$

Interior spans:

- Top:
  $E_{top} = \frac{C(4x+12)}{C(12)}(13/18, 12)$
  $\ell_{top} = -135 < 0.6 \ell_{c} = 1500 \text{ psi} \geq \ell_{c} = 1500 \text{ psi}$
  Compression

- $f_{top} = 220 - 135 = 85 \text{ psi} \leq 1500 \text{ psi} \geq \ell_{c} = 1500 \text{ psi}$

- $f_{bottom} = \ell_{bottom} = 135 - 135 = 0 \text{ psi} \leq 1500 \text{ psi} \geq \ell_{c} = 1500 \text{ psi}$

- $f_{bottom} = 135 - 135 = 0 \text{ psi} \leq 1500 \text{ psi} \geq \ell_{c} = 1500 \text{ psi}$

- $f_{bottom} = 135 - 135 = 0 \text{ psi} \leq 1500 \text{ psi} \geq \ell_{c} = 1500 \text{ psi}$

- $f_{bottom} = 135 - 135 = 0 \text{ psi} \leq 1500 \text{ psi} \geq \ell_{c} = 1500 \text{ psi}$

Compressiv

- $f_{bottom} = 135 - 135 = 0 \text{ psi} \leq 1500 \text{ psi} \geq \ell_{c} = 1500 \text{ psi}$

- $f_{bottom} = 135 - 135 = 0 \text{ psi} \leq 1500 \text{ psi} \geq \ell_{c} = 1500 \text{ psi}$

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- $f_{bottom} = 135 - 135 = 0 \text{ psi} \leq 1500 \text{ psi} \geq \ell_{c} = 1500 \text{ psi}$

Compressiv
Stage 2: Service at service load (W + L + P) after latter.

Midspan stresses:

\[ f_{tep} = \frac{(C_{\text{M}0} + C_{\text{M}L} + M_{\text{Lw}})}{S} = P/A \]

\[ f_{bl} = \frac{(C_{\text{M}0} + C_{\text{M}L} - M_{\text{Lw}})}{S} = P/A \]

Exterior spans:

8':
\[ f_{tep} = \frac{(-12 - 44 + 82)}{600} \text{ksi} \]
\[ f_{bl} = 135 - 152 = 20 \text{ ksi} \text{ OK} \]

9':
\[ f_{tep} = \frac{(-20 - 12 - 181 + 141)}{600} \text{ksi} \]
\[ f_{bl} = 320 - 152 = 168 \text{ ksi} \text{ OK} \]

25':
\[ f_{tep} = \frac{(-187 - 187)}{600} \text{ksi} \]
\[ f_{bl} = 135 - 152 = 98 \text{ ksi} \text{ OK} \]

Exterior spans:

77.5':
\[ f_{tep} = \frac{(-127.5 - 254.7 + 254.7)}{600} \text{ksi} \]
\[ f_{bl} = 772 + 125 = 997 \text{ psi} \text{ not exceed} \]

16':
\[ f_{tep} = \frac{(25 - 30 + 42)}{600} \text{ksi} \]
\[ f_{bl} = 82 - 125 = 0 \text{ ksi} \text{ OK} \]
Ultimate strength

determine factored moments
primary moments, \( M_u = f_c \)
\[ M_{u} = \frac{779 \times 41}{12} = 140 \text{ kft} \]

Secondary moments, \( M_{uc} = M_{bd} - M_{u} \)
\[ M_{uc} = 989 - 140 = 849 + \text{support 1 (interior)} \]
\[ = 234 - 180 = 54 + \text{support 2 (interior)} \]
\[ = 420 - 180 = 240 + \text{support 3 (interior)} \]
\[ = 778 - 180 = 598 + \text{support 4 (interior)} \]

The largest load combination for ultimate strength design

\[ M_{u} = 1.2M_{u} + 1.6M_{lc} + 1.0M_{sc} \]

- midspan: \( M_{u} = 1.2(31.7) + 1.6(30.7) + 1.0(159.5) = 1242 \text{ kft} \)
- support: \( M_{u} = 1.1(31.7) + 1.6(-35.7) + 1.0(30.2) = -1051 \text{ kft} \)
Two-way post-tensioned slab

**determine minimum required reinforcement**

**positive moment region**

**6th interior span**: \( f_k = 202.2 > \sqrt{f_c} = 2 \sqrt{5000} = 141.4\) psi; reinf. positive

\[
\text{Minimum per. mom. reinf. reqd.}
\]

\[
y = \frac{f_k}{C_{f(1.0)}} = \frac{202.2}{1027.8(1.0)} = 2.1 \text{ in}
\]

\[
N_c = \frac{M_{el} + M_{fl}}{s} = \frac{(202.2 + 122.7) \times 12}{3} (0.5) (0.9) (0.24) (610) = \frac{378.5}{600}
\]

\[
A_{s, min} = \frac{N_c}{\phi f_y} = \frac{378.5}{0.85(50)} = 154 \text{ in}^2
\]

**distribute reinf. over 32 ft width**

\[
A_{s, min} = \frac{122.7}{s} = \frac{4.77 \text{ in}^3}{16 \text{ in}}
\]

**roc: use same reinf. in 35 ft interior bay**

**7th exterior span**: \( f_k = 617.4\) psi; \( 2 \sqrt{f_c} = 141.4\) psi; reinf. positive

\[
y = \frac{f_k}{C_{f(1.0)}} = \frac{617.4}{1027.8(1.0)} = 0.2 \text{ in}
\]

\[
N_c = \frac{(202.2 + 122.7) \times 12}{3} (0.9) (0.5) (0.24) (610) = \frac{328.8}{600}
\]

\[
A_{s, min} = \frac{N_c}{\phi f_y} = \frac{328.8}{0.85(50)} = 76.1 \text{ in}^2
\]

**distribute reinf. over 32 ft width**

\[
A_{s, min} = \frac{76.1}{32 \text{ in}} = 2.4 \text{ in}^2
\]
Two-way post-tension slab

Design of B-W interior frame

1. Initial assumed tendon profile same as N-S initially assumed tendon profile

2. Prestress required to balance 50% of self-weight:
   \[ P_b = 0.50 \times \frac{27.5 \text{ psf}}{140} \times 1382.5 \text{ psi} = 1.31 \text{ kif} \]

3. Force needed in tendons to counteract the load in the end bay:
   \[ P = 1.31 \left( \frac{27.5}{54} \right)^2 = 455.3 \text{ kips} \]
   \[ \frac{P}{F(t_{12})} = \frac{455.3}{741} < 2 \text{ end bay} \]

4. Check precompression allowance:
   \[ \frac{W_b}{\text{tendons}} = \frac{572.5 \text{ kips}}{30 \text{ tendons}} = 19.1 \text{ kif} \]
   Use 18 tendons

   \[ P_{\text{actual}} = 26.6 \text{ kips} \]
   The balance load for the end span:
   \[ W_b = \frac{429}{455.3} \times 1.77 \text{ kif} \]

   Actual precompression stress:
   \[ \frac{P_{\text{actual}}}{A} = \frac{475 \text{ kips}}{7520 \text{ in}^2} = 63.3 \text{ psi} < 700 \text{ psi: min. 0.6 kif} \]
   Check 25' end span:
   \[ P = 1.31 \left( \frac{27.5}{54} \right)^2 = 777 \text{ kips} < 475 \text{ kips} \]
   Less force required
   \[ \frac{W_b}{\text{tendons}} = \frac{475 \times 1.77}{2 \times 26.6} = 2.63 \text{ kif} \]
   \[ W_b = 1.63 \text{ kif} \quad \frac{W_b}{W_{\text{tol}}} = \frac{1.63}{2.63} < 0.6 \text{ acceptable for design} \]
Two-way post-tension
tensile slab

- check interior spans

32' span:

\[ P = 1.27 \times 32.5^3 = 492 \text{ kips less force reqd} \]

\[ W_b = 492 \left( \frac{8}{5} \right) \left( \frac{5}{12} \right) = 1.30 \text{ kips} \]

\[ \frac{W_i}{W_{ol}} = \frac{1.30}{2.63} = 0.49 < 1.0 \text{ i.e. acceptable for design} \]

36' span:

\[ P = 1.27 \times 36.5^3 = 543 \text{ kips less force reqd} \]

\[ W_b = 543 \left( \frac{8}{5} \right) \left( \frac{5}{12} \right) = 1.77 \text{ kips} \]

\[ \frac{W_i}{W_{ol}} = \frac{1.77}{2.63} = 0.67 < 1.0 \text{ i.e. acceptable for design} \]

- EW interior frame effective prestress, \( P_{eff} = 479 \text{ kips} \)

E-W direction:

[Diagram showing benton profile]

N-S direction:

[Diagram showing benton profile]
Two-way post-tensioned slab

check punching shear (two-way action)
- use average type 51 4x31 boy to check punching shear

\[ V_u = w_u \cdot A_{ec} \]

\[ w_u = 12 \left( \frac{77 + 10}{2} \right) \text{psf} + 16 \left( \frac{100}{2} \right) \text{psf} \]
\[ = 277 \text{ psf} = 0.277 \text{ ksf} \]
\[ d = \frac{7}{11} = b'' \]
\[ \frac{d}{2} = \frac{3}{2} \]

\[ V_{ul} = 0.277 \text{ ksf} \left( 30 \times 30 - 2.0 \times 2.0 \right) \]
\[ = 247.4 \text{ k} \]

perimeter at critical section, \( b_o = 30'' \), \( c_o = 12'' \)

\[ \frac{b_o}{d} = \frac{120''}{6''} = 20 \]
\[ \frac{d}{2} = \frac{3}{2} \text{ for interior} \]
\[ \frac{b_o}{d} = \frac{20}{3} \approx 1.0 \]

\[ V_c = \frac{3}{2} \sqrt{\frac{f_c}{d} \cdot b_o d} \approx \frac{3}{2} \sqrt{\frac{3000}{2000}} \left( \frac{110}{6} \right) \approx 703.6 \text{ k} \]

\[ V_c = 2.15 \left( \frac{3000}{d} \right) \sqrt{\frac{f_c}{d} \cdot b_o d} \approx 305.6 \text{ k} \]

\[ V_c = \left( \frac{3000}{d} + 1 \right) \sqrt{\frac{f_c}{d} \cdot b_o d} \approx 4 \sqrt{f_c \cdot b_o d} \]

\[ \delta V_c = 0.7 \left( 205 \right) \approx 133 \text{ k} \leq V_u = 247.4 \text{ k} \]

use drop panel design

\[ \int_0^1 \gamma'' = 6'' \]

\[ \gamma = \frac{60''}{4} = \frac{30}{2} \]

\[ \delta = \frac{30}{2} + \gamma = 2'' \]

\[ \delta'' = \frac{30}{2} + \gamma = 1.72'' \]

use drop panel thickness of 3''
Two-way post-tensioned slab

-effective depth \( d = 6" \)

- \( b = (C'F + M/l)^{1/2} \approx 22' = 244" \)

- \( V_c = \left( \frac{40}{9} + \frac{1}{I} \right) \frac{L}{I} = 826 \text{k} \)

- \( \phi V_c = 0.75(826 \text{k}) = 624.5 \text{k} \)

- \( V_u = 0.75(720 - 30.15) = 241 \text{k} < \phi V_c \approx 624.5 \text{k} \)

-use 3" drop panel
Appendix D: Alternative 3 – Composite Joist/ Steel Girder System

<table>
<thead>
<tr>
<th>Composite Steel Joist/</th>
<th>Steel Girder</th>
<th>Tech 2</th>
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<tbody>
<tr>
<td>System</td>
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</tbody>
</table>

- Computed floor deck design
  - Composite beam load on deck
  - Total load = 1.0 psf + slab + deck
  - Clear-clear span of supports = 5'-11"
  - 2 spans condition
  - Deck type – select deck type:
    - From various steel deck composite 2010 catalog
    - Use 41232S with 6" thick slab
    - (9.5" topping for 1 hr fire rating)
    - Superimposed uniform load: 400 psf + 100 psf
    - Deck max. cond. span = 9'-5" x 8'-6"
    - Self wt: 47 psf

Interior composite joint design

- Total load:
  - Dead load:
    - 10 psf
  - 62
  - Live load:
    - 60 psf

- Influence men = \( R_A I = \frac{2 (C_{441})}{12} \) = 750 ft.\(^3\) \( f_{13} \) at 4.4 ft.\(^3\) \( f_{13} \) cannot be reduced
  - Live load = 100 psf \( f_{411} \) = 200 psf
  - Total load = 1.2(201) + 1.6(300) = 1108 psf

- Joist span = 3'-9"
- Adjacent joist spacing = 5'-4"
- Number of spaces = \( \frac{39}{5} \) = 7

- Select joist from Vulcan composite joist 2009 catalog
  - 14C3:
    - Allowable load = 1400 psf + 1.0 psf = 10.0 psf
    - Allowable "I" that produces \( \leq 0.07 \) deflection of \( I_{1400} \)
    - Allowable "I" that produces deflection of \( I_{1400} \)
    - Number of joists per span = 7 - 5' 12" of bridging

Use a 14C3(1400/62) composite joist with 5'-5" of slab.
Composite steel joists/strut

Tech 2

Page 2 of 5

1. Total load

2. Dead load:
   - Sill - 62 psf
   - Slab - 10 psf
   - Wind - 22 psf

3. Live load:
   - Wy - 100 psf
   - Reduces to 60 psf (from pg. 6 of "Composite steel fir system" codes)

4. Pu = 1.2 (C12 + 0.6) psf (55 ft) (3.25 ft) + 1.2 (22 psf) (3.25 ft) + 1.6 (60) (65 ft) (7.5 ft)
   = 1565 + 168 + 15000 = 29,243
5. Determine $T_X$ based on $\alpha_{1\gamma, \max}$ and $\rho_{1\gamma, \max}$.

Assumption: four or more equal point loads may be assumed distributed.

$$W_u = 1.2 (1.1 + 1.0) \text{pcf} \left(21.2 \text{ ft}^3 + 31.2 \text{ ft}^3 + 14.4 \text{ ft} \times 31.2 \text{ ft} \times 1.0 \text{ pcf} + 6.0 \text{ ft} \times 31.2 \text{ ft} \times 1.75 \text{ pcf} \right) = 4.1 \text{ kip}$$

$$\alpha_{1\gamma, \max} = \frac{L}{240} = \frac{30(31.2)}{240} = 0.38$$

$$T_X = \frac{5.0 \times \text{pcf} \times (30 \text{ ft}) \times (31.2 \text{ ft})}{240} = 1178 \text{ in}^4$$

$$\rho_{1\gamma, \max} = \frac{L}{240} = \frac{30(31.2)}{240} = 0.38$$

$$T_X = \frac{5.0 \times \text{pcf} \times (30 \text{ ft}) \times (31.2 \text{ ft})}{240} = 1178 \text{ in}^4$$

Use Table 7.2 in manual.

Try $W_21x92$; $T_X = 2070 \text{ in}^4$.

6. Assumption: girder is simply supported and laterally braced at joint locations only.

The unbraced length $L_b = 5\text{ ft}$.

$$M_u = \frac{(4.1 \text{ kip})(30 \text{ ft})(31.2 \text{ ft})}{8} = 638.2 \text{ kft}$$

$$L_b < L_p = 6.50 \text{ ft} \text{ (from Table 7.2)} \Rightarrow \text{ there is no unbraced length problem}$$

$$\alpha M_u = 32 \text{ kft} > \alpha \rho \Rightarrow \alpha > 0$$

\[ \text{use W21x92 girder} \]

7. Check $W_{u\gamma} = \rho_{u\gamma}$.

$$W_u = \frac{M_u}{L} = \frac{638.2 \text{ kft}}{2} = 319 \text{ k} \Rightarrow \rho_{u\gamma} = 319 \text{ k} \text{ (from Table 7.2)} \Rightarrow \text{ OK}$$
Check steel joint and steel girder deflections

Steel joint: \[ d = \frac{5(0.52\times10^6)(32\times10^5)}{2\times10^6(6.0)(64.0)} \approx 0.064 < \frac{1}{130} = 0.1 \text{ in} \leq 0.25 \text{ in.} \leq 0.25 \text{ in.} = 0.0 \]

Steel girder: \[ d = \frac{5(1.89\times10^6)(24\times10^5)}{2\times10^6(6.0)(64.0)} \approx 0.574 < \frac{1}{130} = 0.1 \text{ in} \leq 0.25 \text{ in.} \leq 0.25 \text{ in.} = 0.0 \]

\[ d = \frac{5(1.99\times10^6)(24\times10^5)}{2\times10^6(6.0)(64.0)} \approx 1.764 < \frac{1}{130} = 0.1 \text{ in} \leq 0.25 \text{ in.} \leq 0.25 \text{ in.} = 0.0 \]
**Vulcraft 2009 Composite Joist Table**

### NORMAL WEIGHT CONCRETE

#### DESIGN GUIDE LRFD WEIGHT TABLE FOR COMPOSITE STEEL JOISTS,

<table>
<thead>
<tr>
<th>Joist Span (TL)</th>
<th>Joist Depth (in.)</th>
<th>TL 1400</th>
<th>1500</th>
<th>1800</th>
<th>2000</th>
<th>2200</th>
<th>2400</th>
<th>2700</th>
<th>3000</th>
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<td>814</td>
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**Total Safe Factored Uniformly Distributed Joist Load in Pounds Per Linear Foot**

- Based on a 50 ksi Maximum Yield Strength
- Concrete Slab Parameters
  - Normal Weight Concrete (146 psf) Fc = 4.0 ksi

October 23, 2011
1000 Connecticut Avenue | Washington DC
77
Appendix E: R.S. Means 2010 Cost Details

Two-Way Flat Slab System

![Flat Slab System Table]

Composite Beam with Composite Deck System

![Composite Beam Table]
# Steel Joist/steel girder floor system

**B1010 Superstructure**

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Appendix F: Typical Floor Plans
Typical underground parking plan rotated 90 degrees CW

Typical Floor plan oriented 90 degrees CW