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1000 CONNECTICUT AVENUE

Washington DC

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 roof-top 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 faming systems were design 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-tensioned 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.

Figure 1 Building Site

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.

Figure 2 Main Lobby Entrance facing K Street (left) and perspective of curtain wall system (right)

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

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

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".

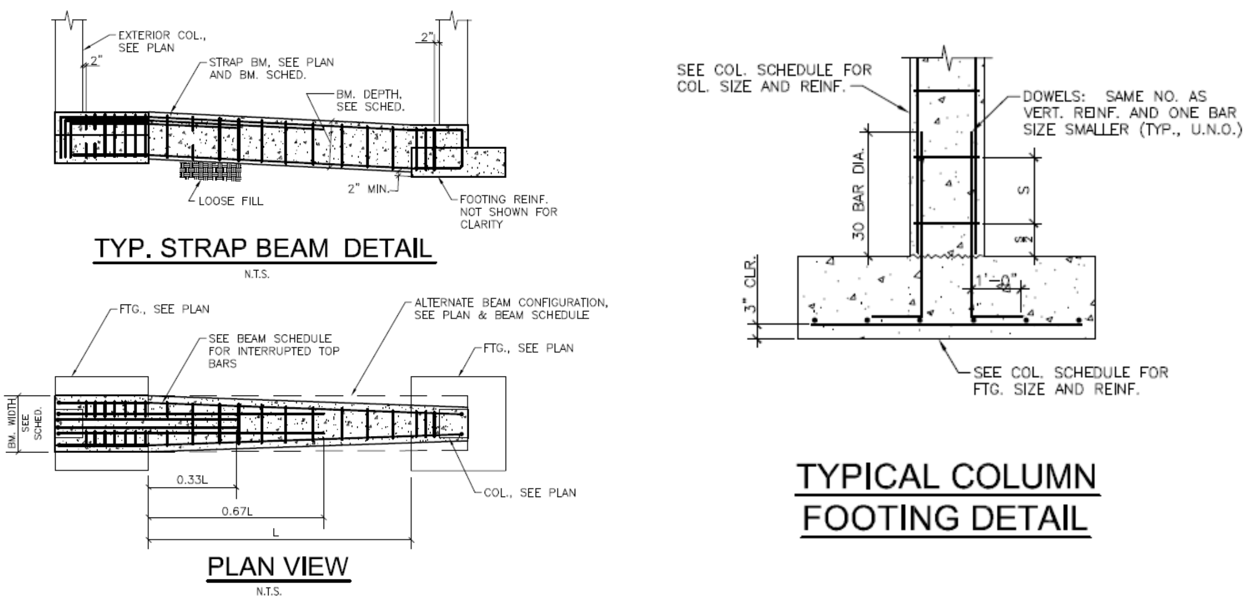


Figure 6 Details of typical strap beam and column footing

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.

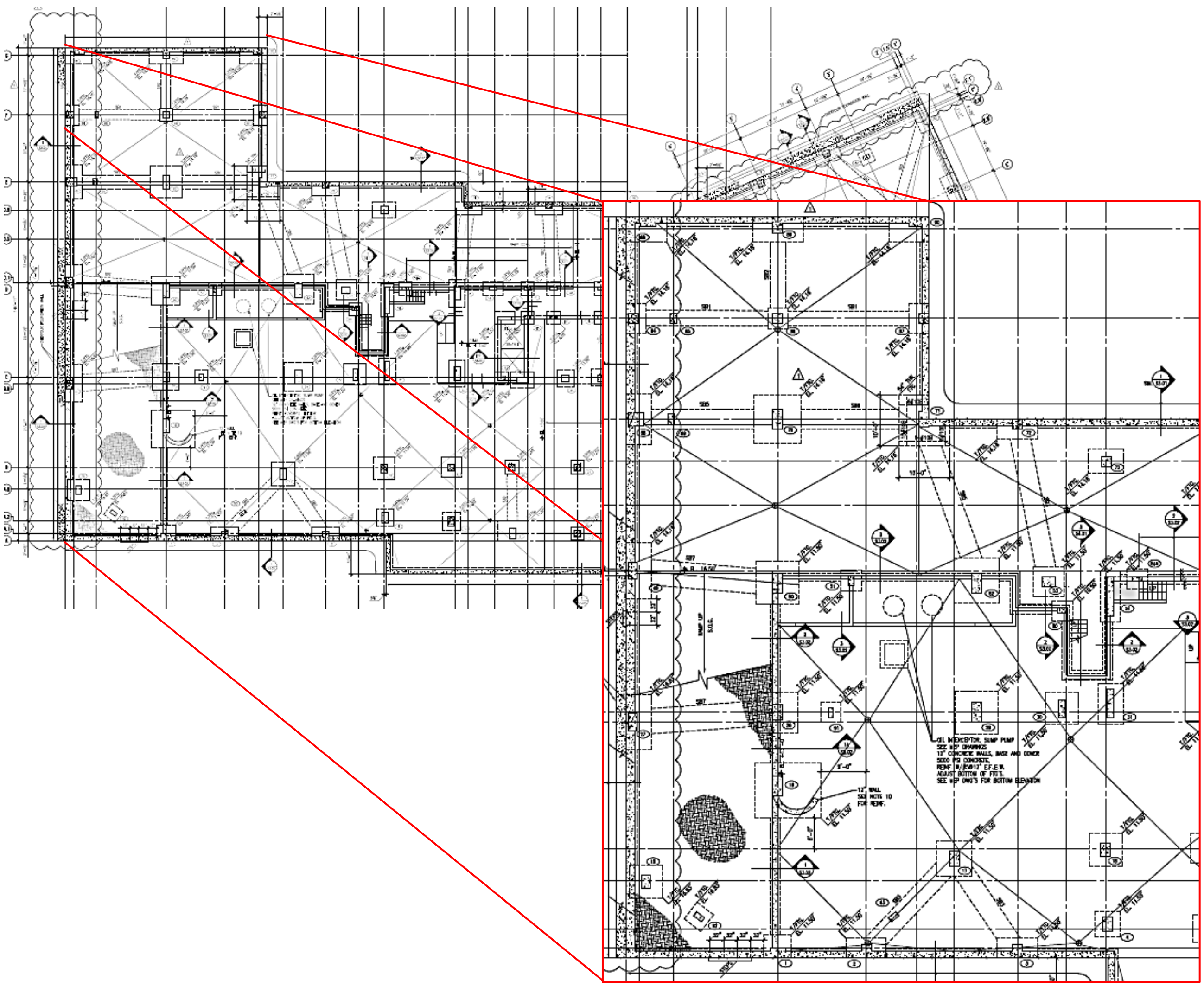


Figure 7 Foundation plan

Framing and Floor System

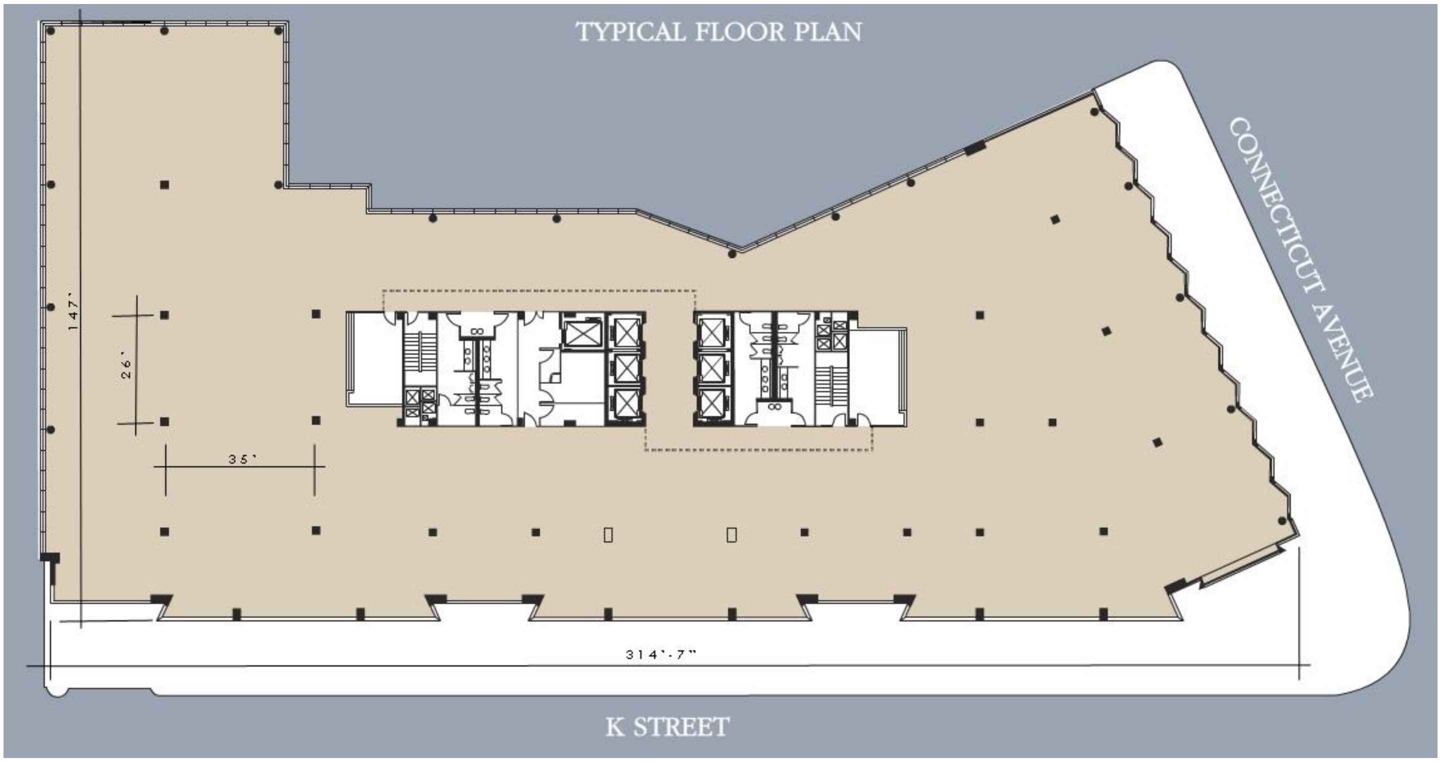
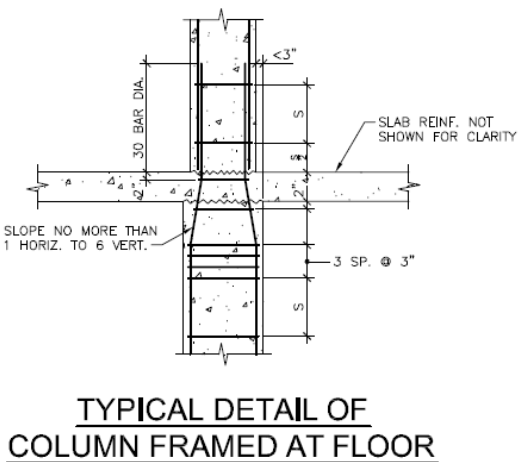


Figure 8 Floor plan displaying column locations and bays

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.



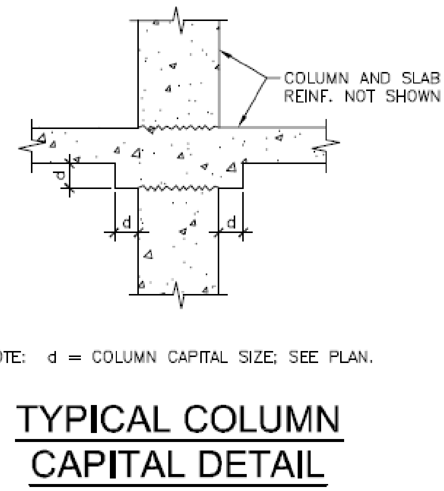


Figure 9 Typical Detail of column framed at the floor

Figure 10 Typical column capital detail

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.

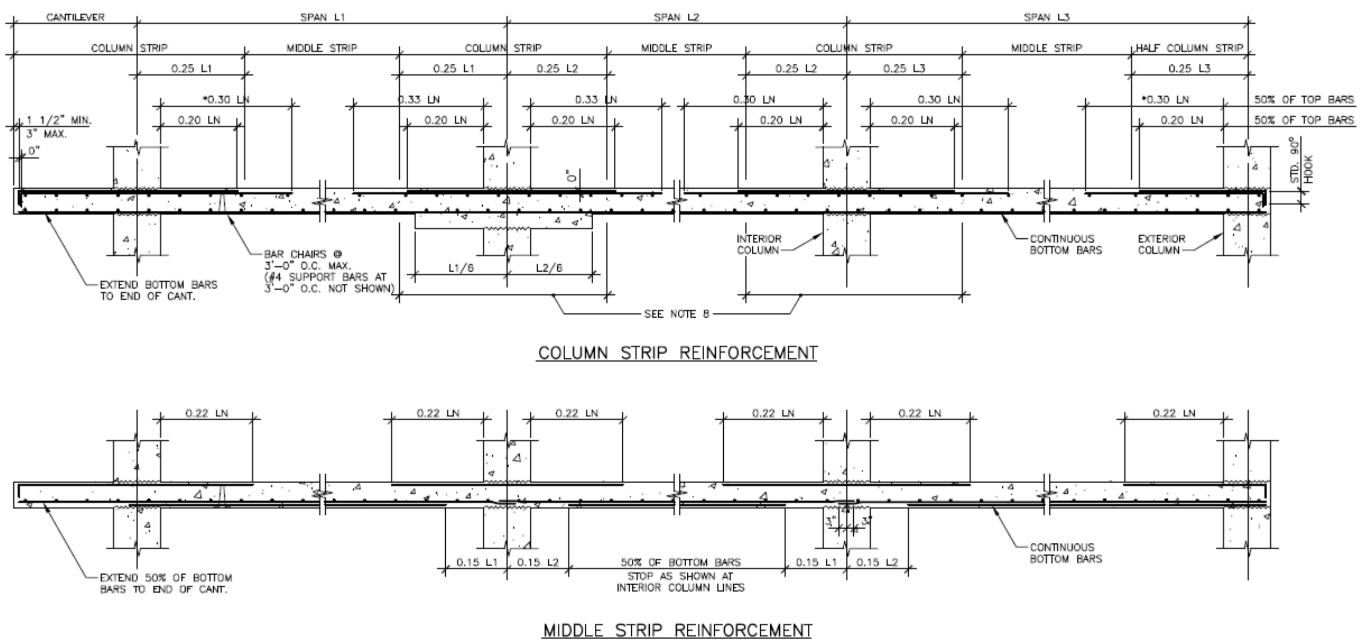


Figure 11 Typical two-way slab reinforcing detail

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.

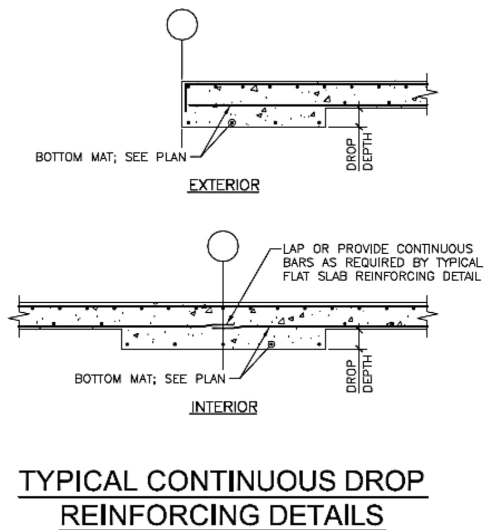


Figure 12 Typical Continuous drop panel

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.

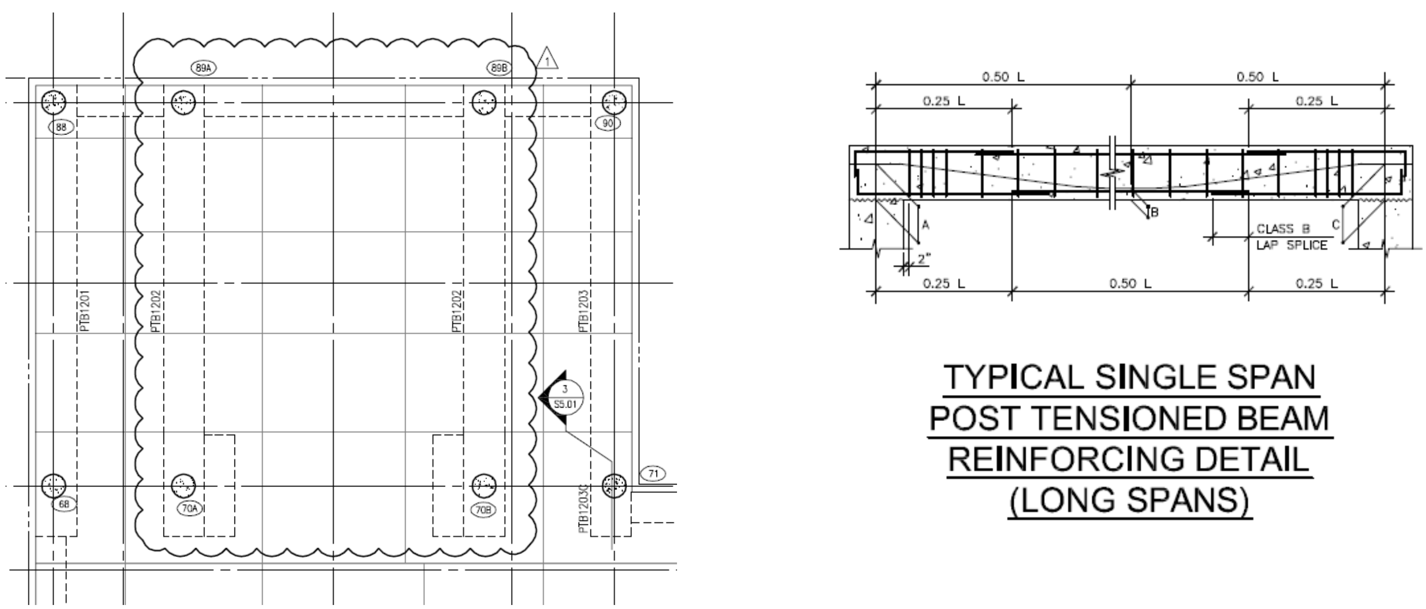


Figure 13 Plan view and typical detail of Post-tension beams supporting slab on levels above two-story loading dock

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.

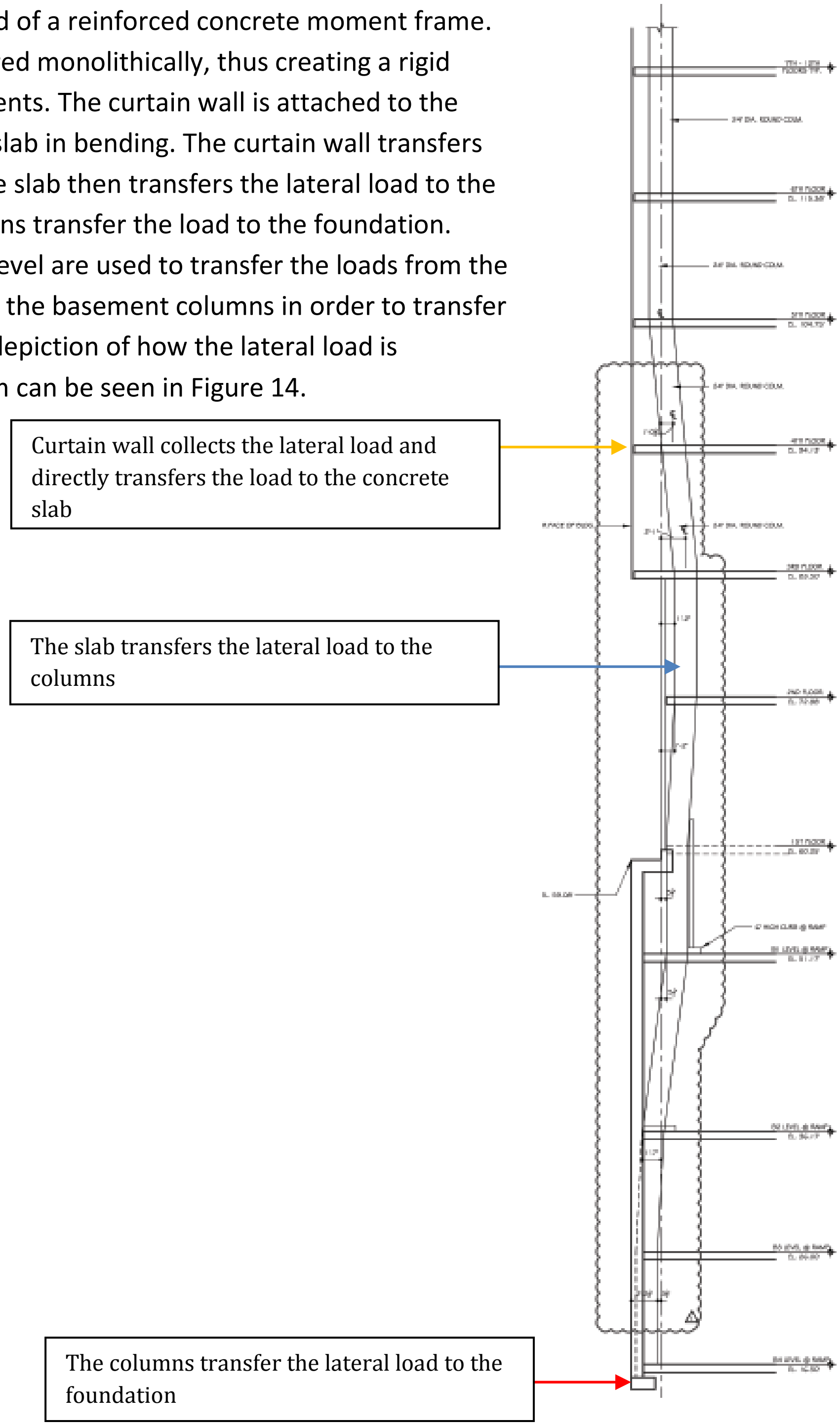


Figure 14 Lateral load path depiction

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:

- 2000 International Building Code (IBC 2000)
- 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)
- AISC Steel Construction Manual, 14th Edition, Load and Resistance Factor Design (LRFD) method
- 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.

Concrete (Cast-in-Place)		
Usage	**Weight**	**Strength (psi)**
Spread Footings	Normal	4000
Strap Beams	Normal	4000
Foundation Walls	Normal	4000
Formed Slabs and Beams	Normal	5000
Columns	Normal	Varies (based on column schedule)
Concrete Toppings	Normal	5000
Slabs on Grade	Normal	5000
Pea-gravel concrete (or grout)	Normal	2500 (for filling CMU units)
All other concrete	Normal	3000
Reinforcing Steel		
Type	**Standard**	**Grade**
Deformed Reinforcing Bars	ASTM A615	60
	ASTM A775	N/A
Welded Wire Fabric	ASTM A185	N/A
Reinforcing Bar Mats	ASTM A184	N/A
Post-Tensioning (Unbonded)		
Type	**Standard**	**Strength (ksi)**
Prestressed Steel (seven wire low-relaxation or stressed relieved strand)	ASTM A416	270
Miscellaneous Steel		
Type	**Standard**	**Grade**
Structural Steel	ASTM A36	N/A
Bolts	ASTM A325	N/A
Welds	AWS	N/A

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.

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

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.

Flat Roof Snow load Calculations	
Variable	Value
Ground Snow, p_g (psf)	25
Temperature, Factor C_t	1.0
Exposure Factor, C_e	0.9
Importance Factor, I_s	1.0
Flat Roof Snow Load, p_f	15.75

Table 3 Summary of roof snow calculations

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, $p_f=0.7C_eC_tI_sP_g$, whereas according to IBC 2000, $p_f=C_eC_tI_sP_g$. 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.

Dead Loads	
Normal Weight Concrete	150 pcf
Curtain Wall	250 plf
Precast Panels	450 plf
Floor Superimposed Dead Load (ceiling, lights, MEP, miscellaneous)	10 psf
Main Roof Superimposed Dead Load (ceiling, lights, MEP, miscellaneous)	10 psf
Penthouse Roof Superimposed Dead Loads	5 psf

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)

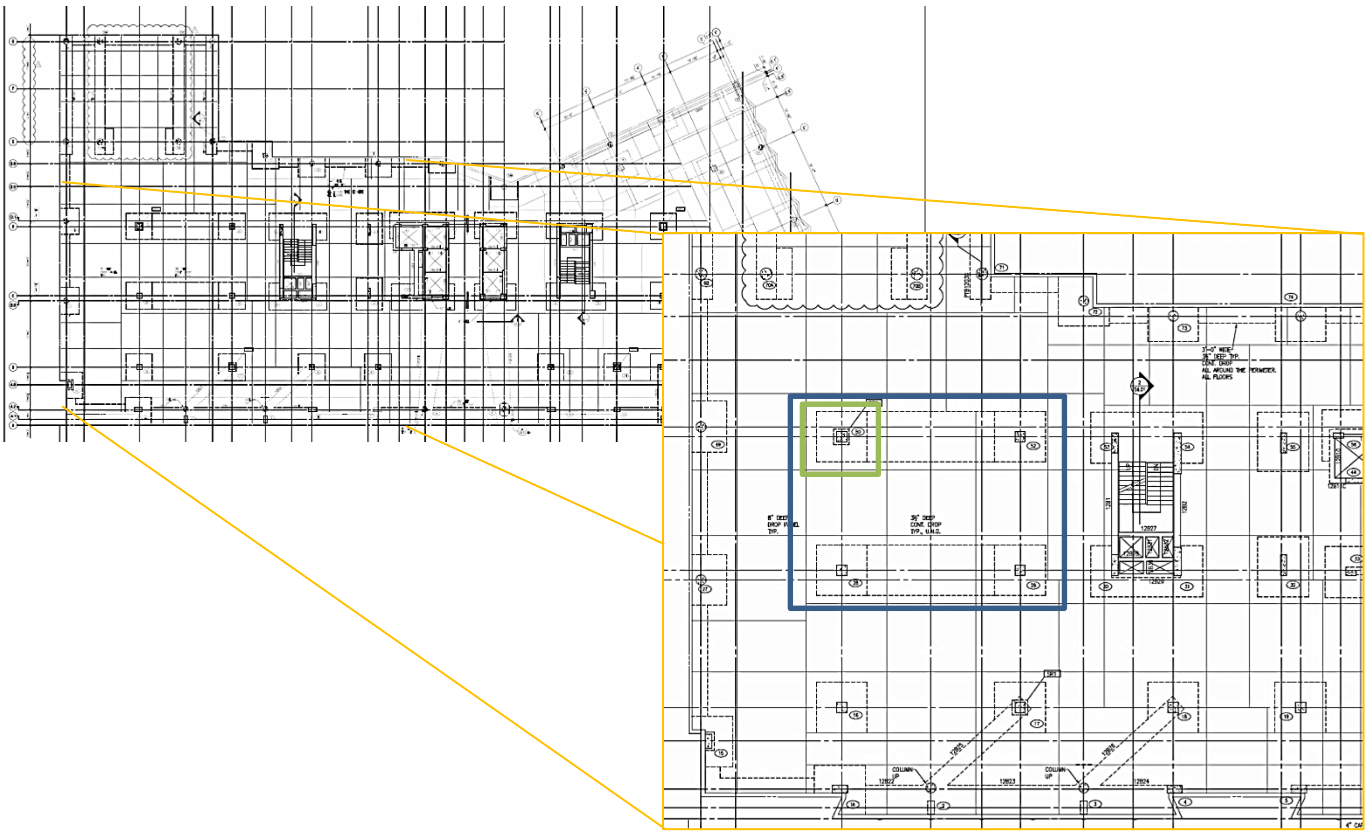


Figure 15 Existing system typical structural layout (left) and an enlarged layout of the interior panel used for gravity spot check analyses (right)

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

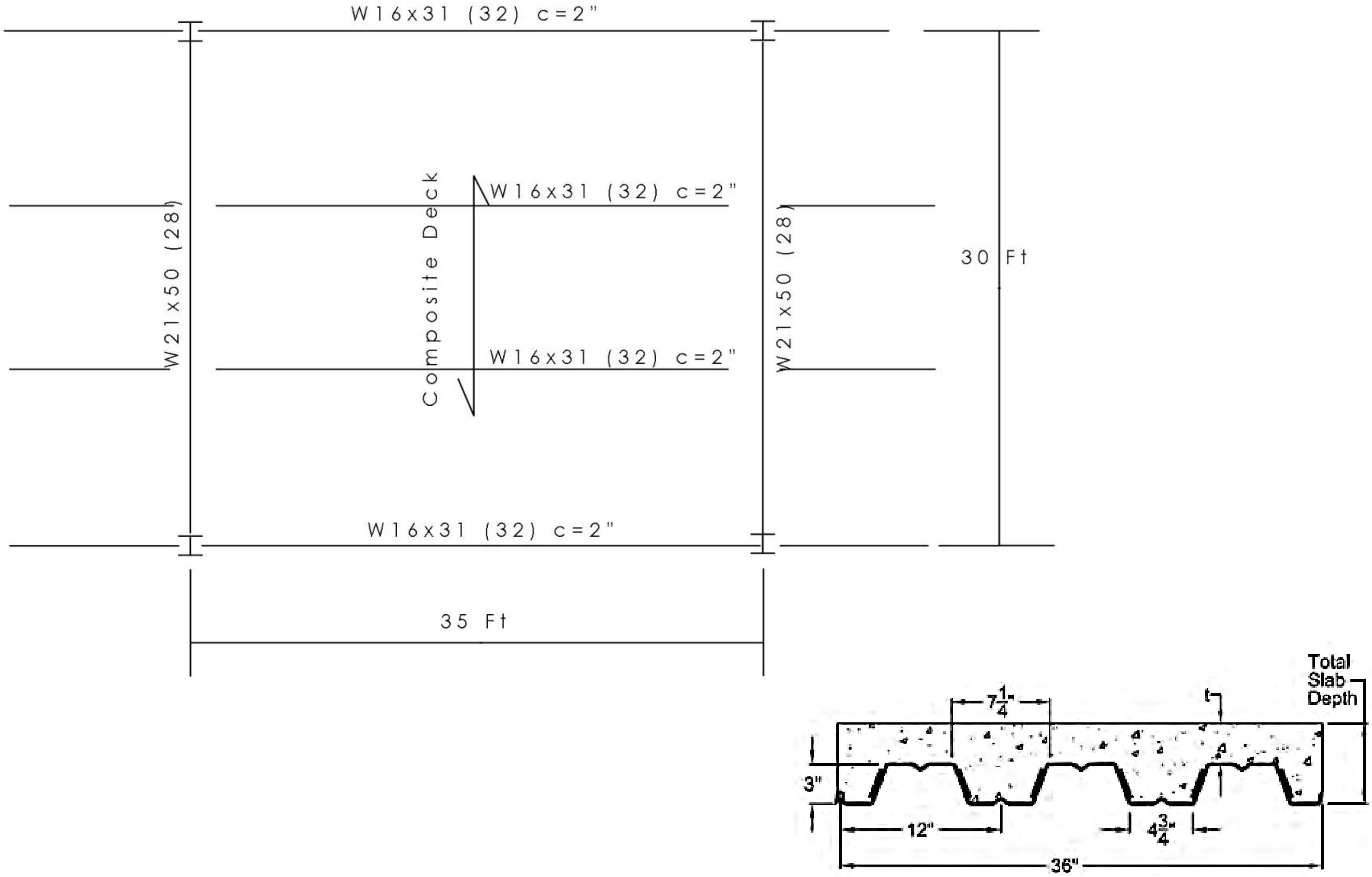


Figure 16 Composite Steel System Layout (top left) and composite deck section taken from Vulcraft 2008 catalog (lower right)

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.

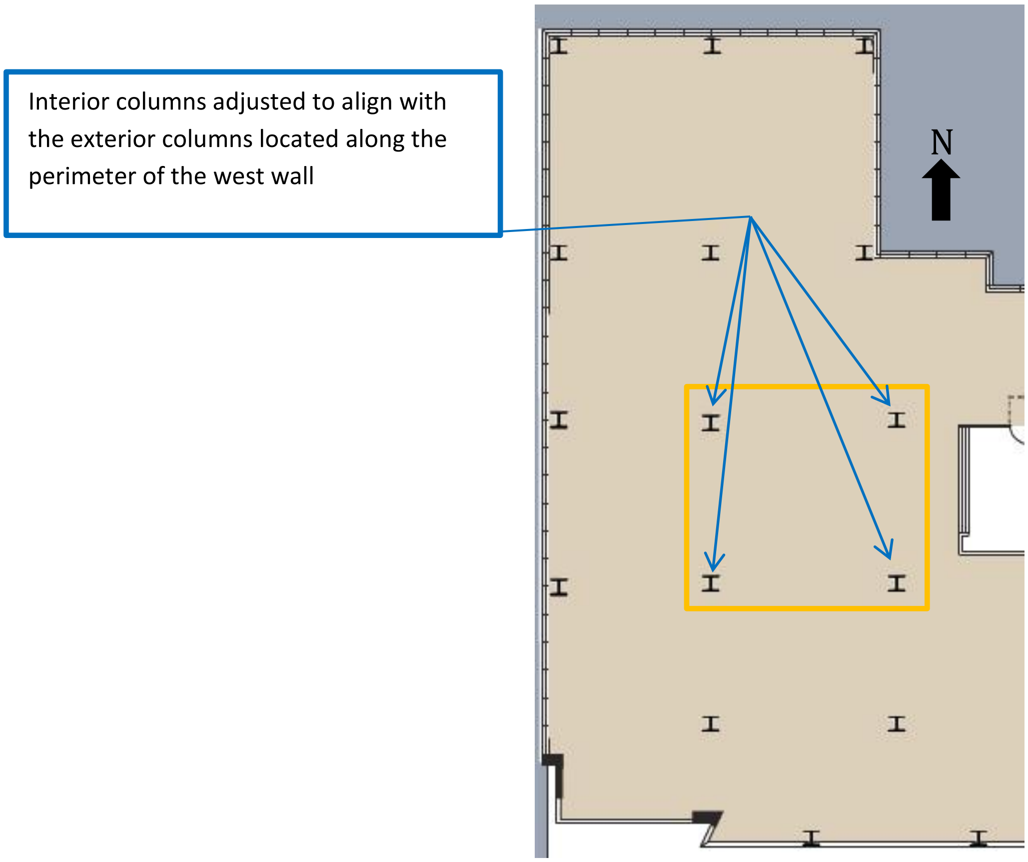


Figure 17 Proposed steel column layout for composite steel system

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 celling 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

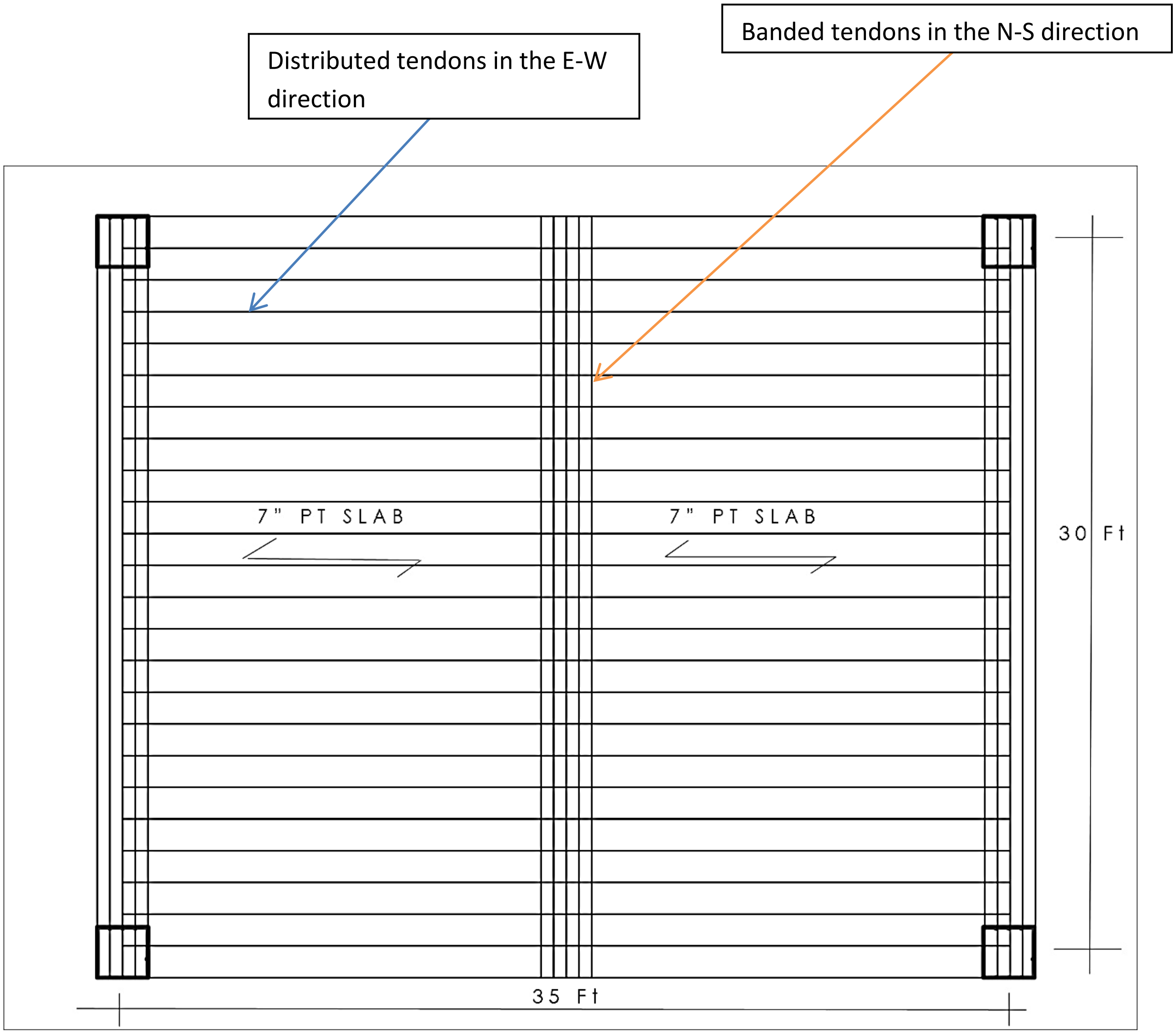


Figure 18 post-Tension tendon layout

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.

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.

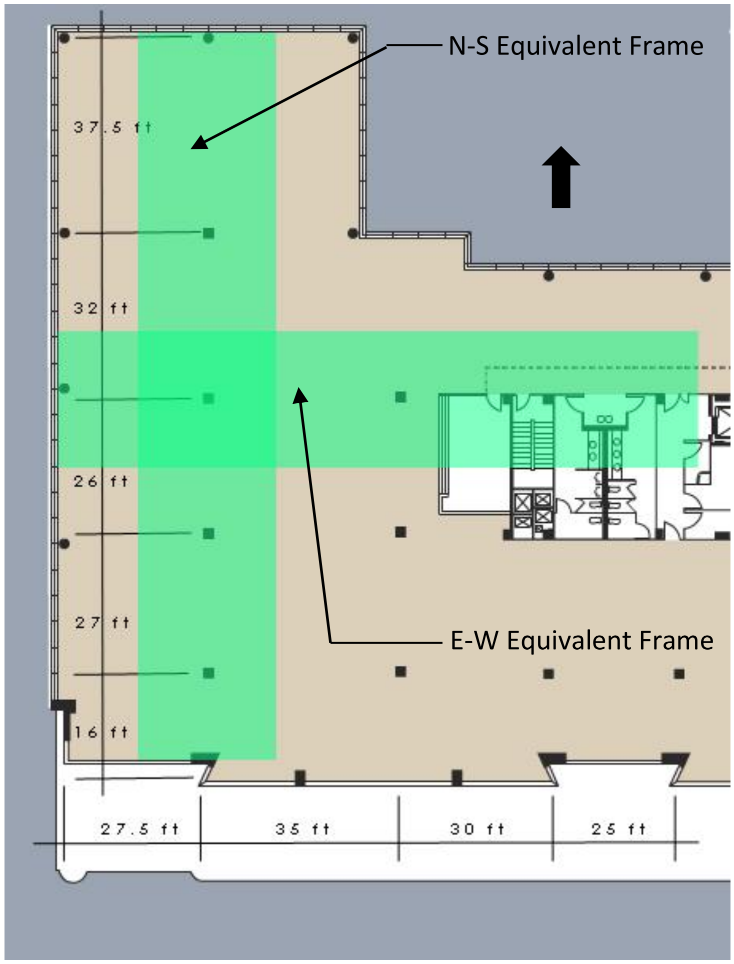


Figure 19 Equivalent frames chosen for PT design highlighted in green

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.

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

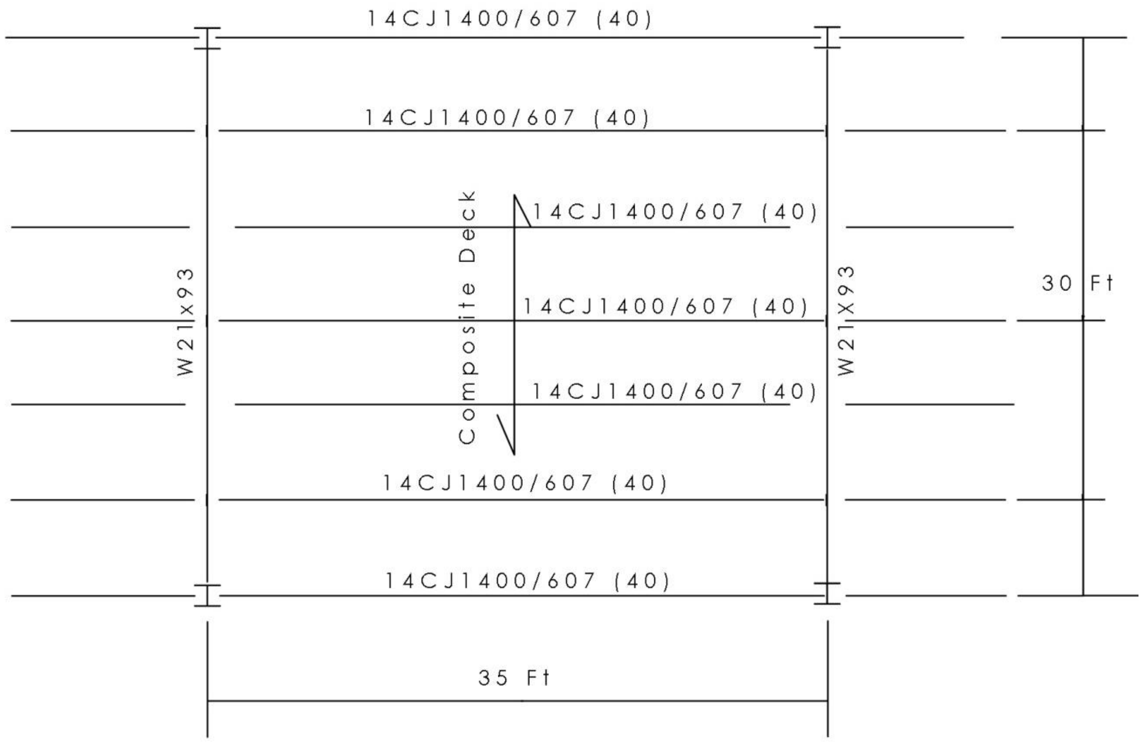


Figure 20 composite joist/steel girder layout

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 14CJ1400/607 with (40) - ⅝” ϕ 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 celling 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.

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

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

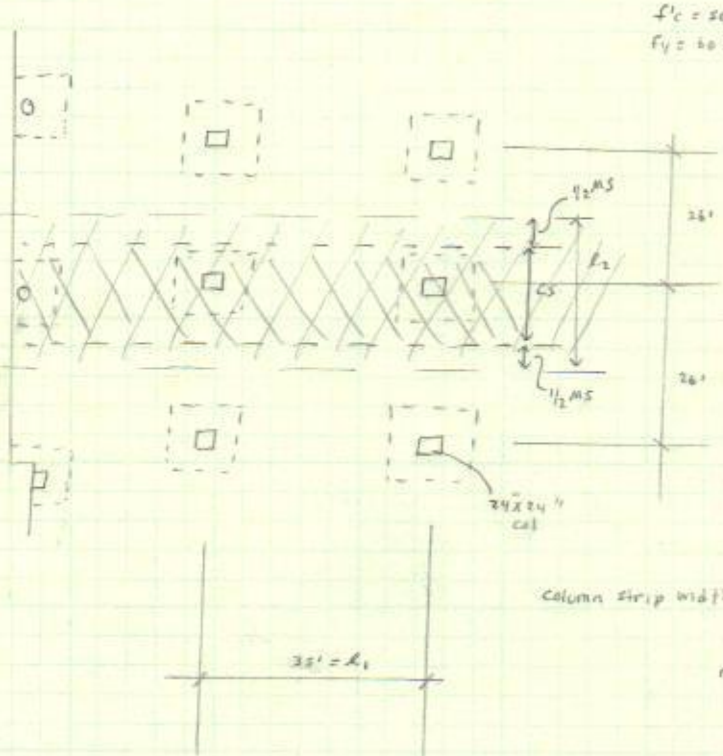
After designing each system and using the above criteria for system comparison, it was found that all 3 alternative systems were viable and will be further investigated to determine which one would be the better floor framing alternative.

Appendix A: Existing System Gravity Load Calculations

Gravity spot check – flat slab interior panel	Tech 1	page 1 of 4

check typical interior flat slab panel in E-W direction for slab thickness and column strip reinforcement

$f'_c = 5000$ psi
$f_y = 60$ ksi



column strip width = min $\left| \frac{\ell_2}{2} = 17.5' \right.$, $\frac{\ell_1}{2} = 13'$

step 1: slab thickness

from Table 9.5 (c) in ACI 318-08;

interior panel with drop panels and $f_y = 60$ ksi

$$t_{min} = \frac{\ell_n}{36} = \frac{(35-2)\text{ft}\,(12\text{ in/ft})}{36} = 11''$$

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)

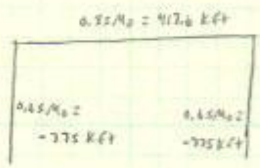
Gravity spot check - flat slab interior panel	Tech 1	page 2 of 4

- total load
  - dead:
    - slab - 150 pcf ($^{11}/_{12}$ ft) = 137.5 psf
    - SDL = 10 psf
  - live load - 100 psf

$w_u = 1.2\,(137.5 + 10)\,\text{psf} + 1.6\,(100\ \text{psf}) = 337\ \text{psf}$

- $M_0 = \dfrac{w_u \ell_2 \ell_n^2}{8} = \dfrac{1}{8}(337\ \text{psf})(26\ \text{ft})(33\ \text{ft})^2 = 1193\ \text{k}\cdot\text{ft}$

- distribute M_0 longitudinally using ACI direct design moment coefficients, sect 13.6.3.2



Interior span

- transverse distribution of moments in column strip → from ACI sect 13.6.4

percentage of longitudinal moment going to col. strip

negative moment @ interior support

$\dfrac{\ell_2}{\ell_1} = \dfrac{26}{35} = 0.74 \qquad \dfrac{\alpha \ell_2}{\ell_1} = 0$ (no longitudinal beam support)

	ℓ_2/ℓ_1 0.5	0.74	1.0
$\dfrac{\alpha \ell_2}{\ell_1}$	75	75	75

-775 kft —— 75 % to col. strip = -581.3 kft

positive moment

$\dfrac{\ell_2}{\ell_1} = 0.74 \qquad \alpha \ell_2/\ell_1 = 0$

	0.50	0.74	1.0
$\alpha \ell_2/\ell_1$	60	60	60

+417.6 kft —— 60 % to col. strip = 250.6 kft

Gravity Spot check – flat slab interior panel | Tech 1 | page 3 of 4

step 3: reinforcement design of column strip in interior panel

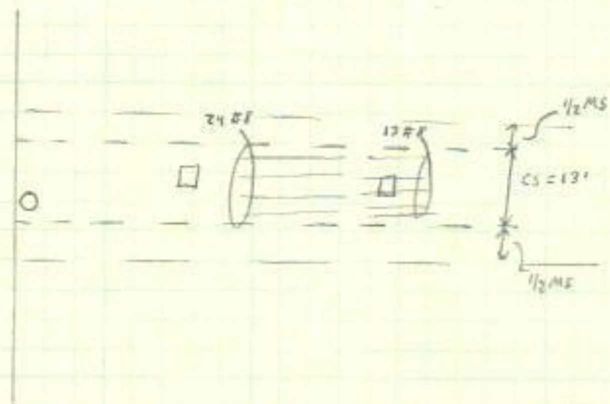
item no.	description	interior span M_u^-	interior span M_u^+
1.	M_u (k-ft)	-775	+417.6
2.	width of strip, b (in)	156"	156"
3	effective depth, d (in) $11 - 0.75 - 0.5(0.625)$ t, clear cover, reinf. #5 bar dia.	9.94"	9.94"
4.	$M_n = \frac{M_u}{0.9}$ (k-ft)	-861	464
5	$R = \frac{M_n}{bd^2}$ (psi) $= \frac{M_n \times 12000}{156"(9.94")^2}$	670	361
6.	ρ from table A-3 in Reinforced concrete, 8th edit from interpolation, $\rho = \left(\frac{R-R_1}{R_2-R_1}\right)(\rho_2-\rho_1)+\rho_1$	0.0122	0.0062
7.	$A_s = \rho bd$ (in²) $= \rho(156)(9.94)$	18.92	9.77
8.	$A_{s,min} = 0.0018bt$ $= 0.0018(156)(11)$	3.1	3.1
9.	$N = \frac{\text{larger of 7 or 8}}{0.31}$ #5 bar	61	$31.5 \cong 32$
10.	$N_{min} = \frac{\text{width of strip}}{2t} = \frac{156}{2(11)}$	$7.09 \cong 8$	8

To minimize the number of reinforcing bars required, increase bar size to #8.

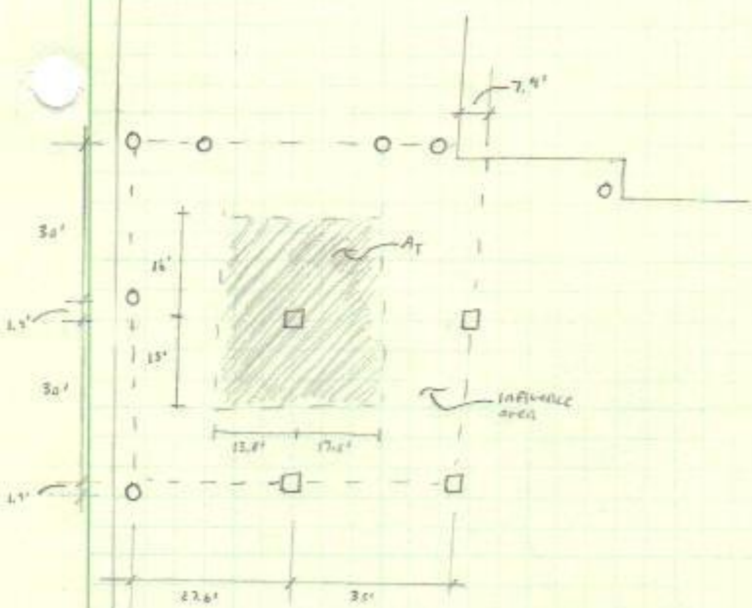
Therefore to resist the positive moment in the middle of the interior slab span, use $N = \frac{18.92}{0.79} = 24$ bars

To resist the negative moment at the supports use $N = \frac{9.77}{0.79} = 13$ bars

Gravity spot check - Flat slab interior panel | Tech 1 | page 4 of 4



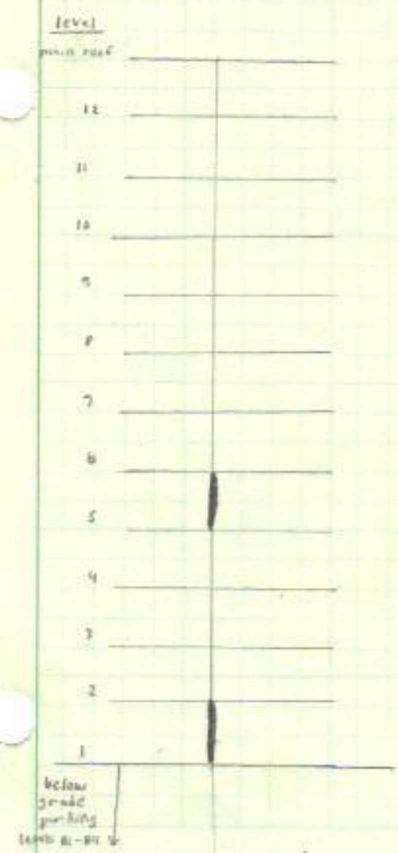
Gravity spot check – column #50 | Tech 1 | Page 1 of 6



tributary area / floor, A_T

$A_T = (31.3')(31') = 970.3\ ft^2$

influence area/floor $= (70')(61.9')$
$= 4333\ ft^2$



Loads

main roof: $L_r = 150\ psf$ ← pump / generator room

assumption: $L_r = 150\ psf > S = 30\ psf$, ∴ use L_r in all applicable load combinations

$DL = 100\ psf + 10\ psf = 110\ psf$

levels 2–12: $LL = 100\ psf$

$DL = 100\ psf + 10\ psf = 110\ psf$

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

Gravity spot check - Column #50	Tech 1	page 2 of 6

Axial load on column at level 1

- load above level 1: roof + 11 floors

$$LL_{reduction}\ factor = \max \begin{cases} 0.4 \\ 0.25 + \frac{15}{\sqrt{11 \times 4 \times 970}} = 0.32 \end{cases} \quad \therefore \text{use } LL_{red} = 0.40$$

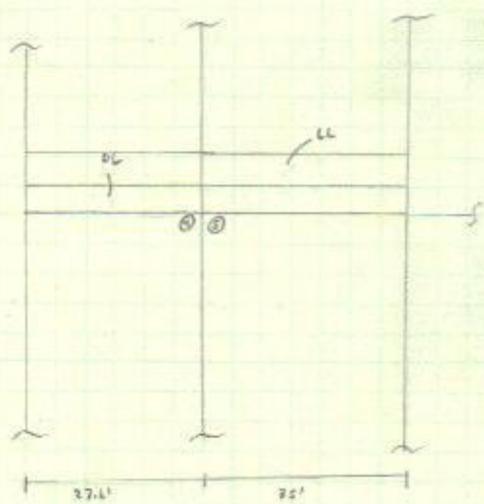
$P_L = LL_{red} \times LL \times A_T = 0.40\,(100\,psf)(11\,flrs)(970.3\,ft^2/flr) = 427\,K$

$P_D = 110\,psf\,(970.3\,ft^2/flr)(11\,flrs) + 110\,psf\,(970.3\,ft^2/roof) = 1281\,K$

$P_{Lr} = 150\,psf\,(970.3\,ft^2/roof) = 145.5\,K$

$P_u = 1.2P_D + 1.6P_L + 0.5L_r = 1.2(1281\,K) + 1.6(470\,K) + 0.5(145.5\,K) = 2362\,K$

Unbalanced moment for column at level 1



use the ACI moment coefficient method to determine the maximum moments and shears at the critical sections

⑤ negative moment at exterior face of 1st interior support

$FEM = \frac{w_u \ell_n^2}{10}$ – more than 2 spans

⑥ negative moment at other faces of interior supports

$FEM = \frac{w_u \ell_n^2}{11}$

note: ℓ_n is the clear distance between the supports, but for preliminary sizing purposes, I will use the ctr-to-ctr distance with the assumption that the column sizes are unknown at this stage of preliminary column sizing

$w_{LL} = 100\,psf\,(31\,ft) = 3100\,plf = 3.1\,klf$

$w_{DL} = 110\,psf\,(31\,ft) = 3400\,plf = 3.4\,klf$

$w_u = 1.2(3.4\,klf) + 1.6(3.1\,klf) = 9.04\,klf$

Gravity Spot check – column #50 | Tech 1 | page 3 of 6

$$Mu_{\text{left of support}} = \frac{9.04\ \text{klf}\left(\frac{27.6+35}{2}\right)^2}{10} = 886\ \text{kft}$$

$$Mu_{\text{right of support}} = \frac{9.04\ (31.3)^2}{11} = 805\ \text{kft}$$

use $Mu = 886$ kft to be conservative

<u>Preliminary column size for level 1</u>

assume bars on all 4 faces, $f'c = 8000$ psi and $f_y = 60$ ksi

$$e = \frac{Mu}{Pu} = \frac{886 \times 12}{2362} = 4.5''$$

assume $d' = 2.5''$ $\qquad \gamma = \frac{h - 2d'}{h}$

– set target reinforcement ratio to about $\rho_g = 3\% = 0.03$

h	γ	e/h
22	0.773	0.205
24	0.791	0.188
30	0.833	0.15
36	0.861	0.125

assumptions: to enable use of design aid interaction diagrams for determining the preliminary column size for level 1, use a specific concrete strength of $f'c = 6000$ psi in place of the existing column's specific concrete strength of $f'c = 8000$ psi

– using fig. A-11b from Reinforced Concrete: Mechanics and Design, 5th edition, for $\gamma = 0.75$, $\rho_g = 0.03$ and $e/h \approx 0.17$

then $\frac{\phi P_n}{bh} = 2.9$ for which $b \cdot h = \frac{\phi P_n}{2.9} = \frac{2362}{2.9} = 814.5\ \text{in}^2$

$\Rightarrow b = h = 28.5''$ → try $30'' \times 30''$ column

from fig. A-11b

$$\frac{\phi P_n}{b \cdot h} = \frac{2362}{30 \times 30} = 2.62$$

$$\frac{\phi M_n}{bh^2} = \frac{886 \times 12}{30 \times 30^2} = 0.40$$

requires $\rho_g = 1.3\%$

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from fig. A-11c ($\gamma = 0.90$)

$\frac{\phi P_n}{bh} = 2.62$

$\frac{\phi M_n}{bh^2} = 0.40$

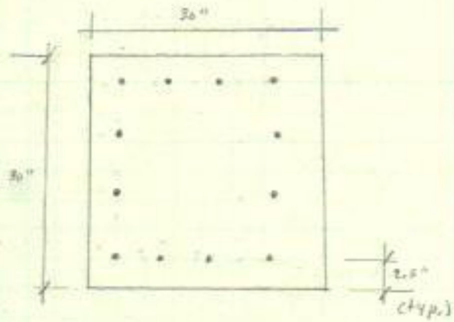
requires $\rho_g \approx 1.1\ \%$

interpolation gives $\rho = \frac{0.833 - 0.75}{0.90 - 0.75}(1.1 - 1.3) + 1.3 = 1.19\ \%$ for $\gamma' = 0.833$

$- \ A_{s,min} = \rho_g (A_g) = 0.0119(30 \times 30) = 10.71\ in^2$

$\frac{10.71\ in^2}{8\ bars} = 1.34\ in^2$ use (12) #9 = 12 in²

use a 30" x 30" column reinforced with (12) #10



assumption: column is subjected to only gravity load ∴ check the column for pure axial compressive strength

pure compression: ($c = \infty$, $\varepsilon_c = \varepsilon_s = 0.003$ (section is compression controlled)

for tied colums, $\phi P_n = 0.8\phi\,[f'_c\,(bh - \Sigma A_{si}) + \Sigma A_{si} f_y]$
$= 0.80(0.65)[6(30 \times 30 - 12) + 12(60)]$
$= 3145\ K > P_u = 2362\ K$ ok ✓

existing design uses a 24"x36" column on level 1 with f'c = 8000 psi and (16) #11 reinforcing bars. In addition, the column at level 1 has a slope, but for simplification purposes the slope was neglected

cross-section percent error $= \frac{|24 \times 36 - 30 \times 30|}{24 \times 36} \times 100 = 4\ \%$

Gravity spot check – column # 50	Tech 1	page 5 of 6

Axial load on column at level 5

- load above level 5: roof + 7 floors

$LL_{reduct.}$ factor $= \max \begin{cases} 0.40 \\ 0.25 + \dfrac{15}{\sqrt{7 \times 4333}} = 0.336 \end{cases}$ ∴ use 0.40

$P_L = 0.40 (100\ psf)(7\ flrs)(970.3\ ft^2/flr) = 272$

$P_D = 110\ psf\ (970.3\ ft^2/flr)(7\ flrs) + 110\ psf\ (970.3\ ft^2/roof) = 854\ K$

$P_{Lr} = 150\ psf\ (970.3\ ft^2/roof) = 145.5\ k$

$P_u = 1.2(854) + 1.6(272) + 0.5(145.5) = 1533\ K$

Unbalanced moment for column at level 5

- same as UBM for column at level 1 (refer to pg. 2)

Preliminary column size for level 5

assume bars on all 4 faces, $f'c = 6000$ psi and $f_y = 60$ ksi

$e = \dfrac{M_u}{P_u} = \dfrac{886 \times 12}{1533} = 6.94''$

assume $d' = 2.5''$

set target reinforcement ratio to $\rho_g = 0.03$

h	γ	e/h
22"	0.773	0.315
24"	0.792	0.289
30"	0.833	0.231
36"	0.861	0.193

- using fig. A-11b from Reinforced Concrete: Mechanics and Design, 5th edition, for $\gamma = 0.75$, $\rho_g = 0.03$ and $e/h \approx 1.86$ → (average of e/h values above)

then $\dfrac{\phi P_n}{bh} \approx 2.8$ for which $bh = \dfrac{1533}{2.8} = 547.5\ in^2$

$\Rightarrow b = h = 23.4 \longrightarrow$ try 24" x 30" column

Gravity Spot check – column #50 | Tech 1 | Page 6 of 6

from Fig. A-11b

$$\frac{\phi P_n}{bh} = \frac{1533}{24 \times 30} = 2.13$$

$$\frac{\phi M_n}{bh^2} = \frac{856 \times 12}{24 \times 30^2} = 0.50$$

requires $\rho_g = 1.5\ \%$

from Fig. A-11c ($\gamma = 0.80$)

$$\frac{\phi P_n}{bh} = 2.13$$

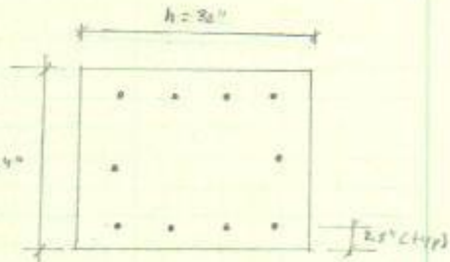
$$\frac{\phi M_n}{bh^2} = 0.50$$

requires $\rho_g = 1.2\ \%$

$$\rho_g = \frac{0.833 - 0.75}{0.80 - 0.75}(1.2 - 1.5) + 1.5 = 1.33$$

– $A_{s,min} = \rho_g (A_g) = 0.0133\ (24 \times 30) = 9.58\ in^2$

$$\frac{9.58\ in^2}{8\ bars} = 1.20\ in^2 \quad \text{use } (10)\ \#9 = 10.0\ in^2$$



assumption: column is subjected to only gravity load ∴ check the column for pure axial compressive strength

Pure compression: $c = \infty$, $\varepsilon_c = \varepsilon_s = 0.003$

for tied columns, $\phi P_n = 0.8(0.65)[6(24 \times 30 - 10) + 10(60)]$
$= 2527\ k > P_u = 1533\ k$ ok ✓

The existing column at level 5 is 24"×24" with (12)#11 reinforcing bars and $f'_c = 6000$ psi.

$$\text{cross-section percent error} = \frac{|24 \times 24 - 24 \times 30|}{24 \times 24} \times 100 = 25\ \%$$

Snow Drift | tech 1 | page 1 of 2

step 1: ground snow load, p_g → from fig. 7-1, $p_g = 25$ psf

step 2: Exposure Factor, C_e, → from table 7-2, Terrain category B
roof fully exposed
⇒ $C_e = 0.90$

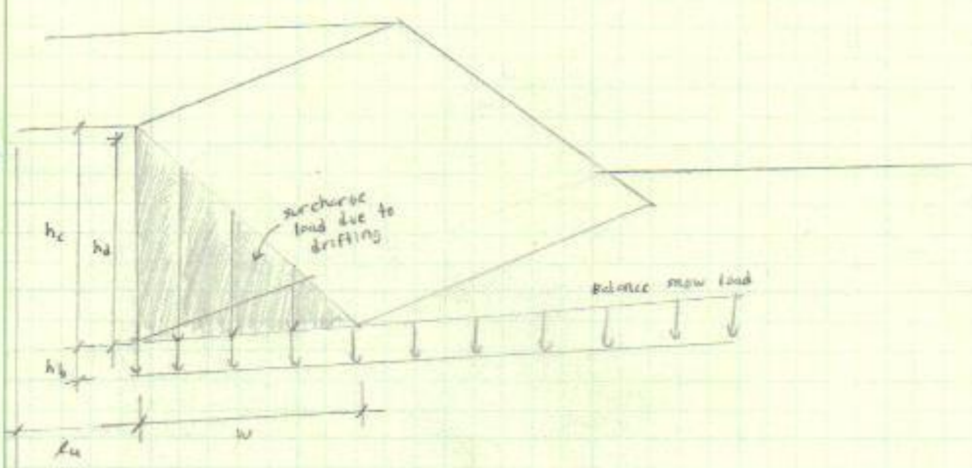
step 3: Thermal Factor, C_t → from table 7-3, $C_t = 1.0$

step 4: importance factor, I → from table 1.5-1, $I = 1.0$ (occ. II)

step 5: flat roof snow load, p_f → sect 7.3,

$$p_f = 0.7\, C_e C_t I_s p_g$$
$$= 0.7(0.90)(1.0)(1.0)(25\text{ psf})$$
$$= 15.75\text{ psf}$$

Snow drift: Penthouse level



step 6: maximum intensity of the drift surcharge load, p_d → sect 7.7.1

$p_d = h_d \gamma$ ⇒ snow density $\gamma = 0.13 p_g + 14$
$= 0.13(25\text{ psf}) + 14$
$= 17.25\text{ pcf} < \gamma_{max} = 30\text{ pcf}$

leeward drift
E-W wind: $\ell_u = 135'$ ⇒ snow drift height, h_d → from fig. 7-9,

$$h_d = 0.43\sqrt[3]{135}\ \sqrt[4]{25+10} - 1.5 = 3.7\text{ ft}$$

leeward drift
N-S wind: $\ell_u = 55.9'$ ⇒ $h_d = 0.43\sqrt[3]{55.9}\ \sqrt[4]{25+10} - 1.5 = 2.5$ ft

snow drift | Tech 1 | page 2 of 2

leeward drift
E-W wind: $P_d = 3.9\ ft(17.25\ pcf) = 67.3\ psf$; width of snow drift, $w = 4h_d = 4(3.9\ ft) = 15.6\ ft$

leeward drift
N-S wind: $P_d = 2.5\ ft(17.25\ pcf) = 43.1\ psf$; $w = 4(2.5) = 10\ ft$

step 7: balanced snow load height, h_b → from sect 2.1

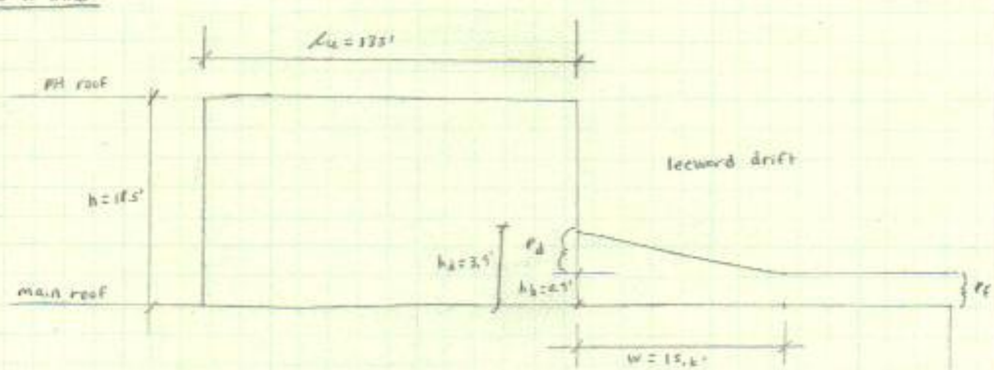
$$h_b = \frac{P_f}{\gamma} = \frac{15.75\ psf}{17.25\ pcf} = 0.9\ ft$$

step 8: $h = \underset{\text{PH roof}}{148.5'} - \underset{\text{main roof}}{130'} = 18.5\ ft$

step 9: $h_c = h - h_b = 18.5' - 0.9' = 17.6\ ft$

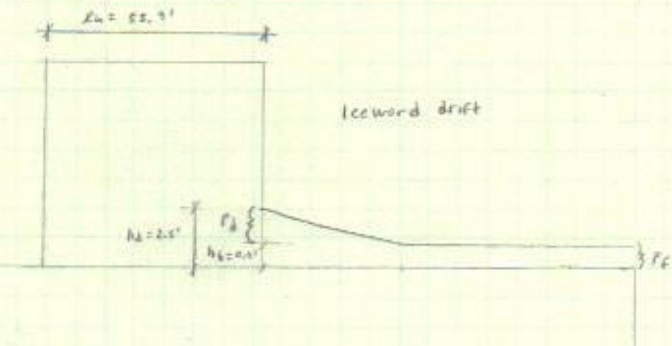
step 10: total snow load, $P = P_d + P_f$

E-W wind



$P_{total} = 67.3\ psf + 15.75\ psf = 83\ psf$

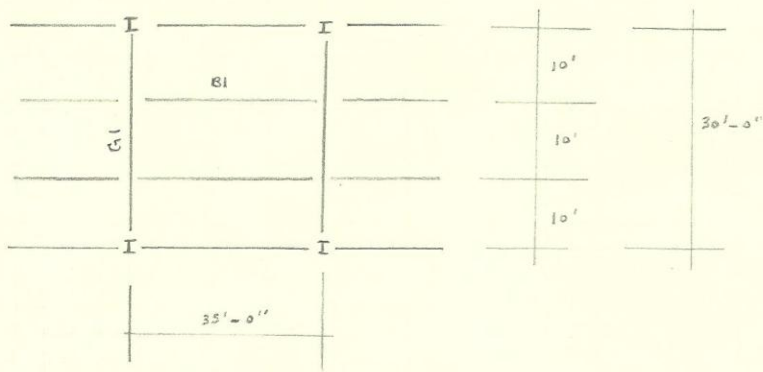
N-S wind



$P_{total} = 43.1 + 15.75 = 59\ psf$

since $P_{tot,E-W} > P_{tot,N-S}$, use $P_{tot} = 83\ psf$ for a conservative design

Appendix B: Alternative 1 – Composite Steel Floor System with Composite Deck



composite floor deck design – use Vulcraft steel Roof 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

↑ SDL

total load = 110 psf + slab + deck

② ctr-to-ctr span of supports – 10'-0"

③ number of spans – 3+ span condition

④ deck type

select 1st composite flr deck in which

- clear span ≥ ctr-to-ctr span of supports
- superimposed uniform load ≥ required total load
- deck max SDI const. span ≥ ctr-to-ctr span of supports

for a 2 Hr fire assembly rating, use an unprotected deck with a 4.5" normal wt concrete topping

use 3VLI20 composite deck with 7.5" thick slab (topping = 4.5") and self-wt = 75 psf

- clear span = 10'-0"
- superimposed uniform load = 190 psf ≥ required total load = 110 psf ∴ ok
- deck max SDI const. span = 11'-0" ≥ ctr-to-ctr span of supports = 10'-0" ∴ ok

Interior composite beam design B1

① total load

dead load:

SDL – 10 psf

slab – 75 psf

$w_D = (10\ psf + 75\ psf)(10\ ft) = 850\ plf$

live load:

$L_o = 100\ psf$

influence area $= K_{LL} A_T = 2(10\ ft)(35\ ft) = 700\ ft^2 > 400\ ft^2$ ∴ must reduce live load

$$L = 100 \times \max \left| \begin{array}{l} 0.5 \\ 0.25 + \dfrac{15}{\sqrt{700}} = 0.817 \end{array} \right.$$

$L = 100(0.817) = 81.7\ psf$

$w_L = 81.7\ psf(10\ ft) = 817\ plf$

② required moment for the composite beam → assume beam is simply supported

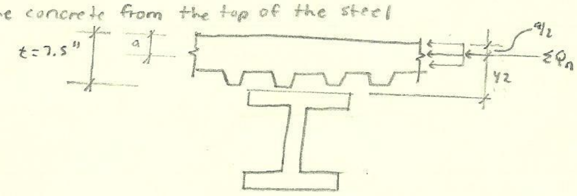
→ add 5 psf for bm self-wt

$$W_u = 1.2(5\ psf)(10\ ft) + 1.2(850\ plf) + 1.6(817\ plf)$$
$$= 2387\ plf$$

$M_u = \dfrac{w_u L^2}{8} = \dfrac{2387\ plf(35\ ft)^2}{8} = 365{,}509\ lb\text{-}ft = 366\ k\text{-}ft$; $V_u = \dfrac{w_u L}{2} = \dfrac{2.387\ klf(35\ ft)}{2} = 41.8\ k$

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

assume $a = 1.0$ in



$Y2 = t - \dfrac{a}{2} = 7.5 - \dfrac{1}{2} = 7$ in

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④ determine the lower bound inertia (I_{LB}) based on $\Delta_{LL,max}$ and $\Delta_{TL,max}$

$$\Delta_{LL,max} = \frac{L}{360} = \frac{35(12)}{360} = 1.17''$$

$$\Delta_{LL} = \frac{5w_L L^4 (1728)}{384 E I_{LB}} \Rightarrow I_{LB,min} = \frac{5 w_L L^4 (1728)}{384 E \Delta_{LL,max}} = \frac{5(0.817)(35)^4 (1728)}{384 (29000)(1.17)} = 813 \text{ in}^4$$

$$\Delta_{TL,max} = \frac{L}{240} = \frac{35(12)}{240} = 1.75''$$

$$\Delta_{TL} = \frac{5 w_{TL} L^4 (1728)}{384 E I_{LB}} \Rightarrow I_{LB,min} = \frac{5(0.817 + 0.850)(35)^4 (1728)}{384 (29000)(1.75)} = 1109 \text{ in}^4$$

⑤ select potential W-shapes from tables 3-19 and 3-20 in steel manual
(select members with $\phi M_n \geq 366$ Kft and $I_{LB} \geq 1109 \text{ in}^4$)

W14x30, $I_{LB} = 1150 \text{ in}^4$, $\phi M_n = 461$ Kft, $\Sigma Q_n = 443$ K (Y1 = TFL)

W16x26, $I_{LB} = 1150 \text{ in}^4$, $\phi M_n = 428$ Kft, $\Sigma Q_n = 384$ K (Y1 = TFL)

W16x31, $I_{LB} = 1140 \text{ in}^4$, $\phi M_n = 414$ Kft, $\Sigma Q_n = 274$ K (Y1 = 4)

W18x35, $I_{LB} = 1220 \text{ in}^4$, $\phi M_n = 433$ Kft, $\Sigma Q_n = 194$ K, (Y1 = 6)

⑥ determine horizontal shear strength for shear stud

- using table 3-21 in steel manual, determine Q_n for shear stud
  - deck perpendicular
  - assume weak stud position
  - assume 1 stud/rib
  - use 3/4" ϕ stud
  - assume $f'_c = 4$ ksi normal wt conc.

$\Rightarrow Q_n = 17.2$ K
↑
1 stud/rib

⑦ determine # of studs/bm for shapes listed in step 5

W14x30: $\frac{\Sigma Q_n}{Q_n} \times 2 = \frac{443}{17.2}(2) = 52 > 35'$ ∴ use 2 studs/rib $\Rightarrow \Sigma Q_n = 14.6$ K

$= \frac{443}{14.6}(2) = 62$ studs/bm

W16x26: $\frac{384}{17.2} \times 2 = 46 > 35'$ ∴ use 2 studs/rib $\Rightarrow \frac{384}{14.6}(2) = 54$ studs/bm

W16x31: $\frac{274}{17.2} \times 2 = 32 < 35'$ ∴ use 32 studs/rib

W18x35: $\frac{194}{17.2} \times 2 = 24 < 35'$ ∴ use 24 studs/rib

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⑧ evaluate bm listed in step 5 for economy

W 14x30 w/ 62 studs:

30 # x 35' + 62 studs x 10 #/stud = 1670 #

W16x26 w/ 54 studs:

26 # x 35' + 54 studs x 10 #/stud = 1450 #

W16x31 w/ 32 studs

31 # x 35' + 32 studs x 10 #/stud = 1405 #

W18x35 w/ 24 studs:

35 # x 35' + 24 studs x 10 #/stud = 1465 #

try W16x31 since it's more economical

⑨ check the depth of the compressive concrete block, a

$$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}}$$

$$W16x31: \; a = \frac{274}{0.85(4)(105)} = 0.77'' < \text{assumed } 1'' \therefore ok$$

b_{eff} for interior bm $= \min\begin{cases}\frac{span}{8}\\ \frac{1}{2}trib_w\\ \frac{span}{8}\end{cases} + \min\begin{cases}\frac{span}{8}\\ \frac{1}{2}trib_w\\ \frac{1}{2}trib_w\end{cases}$

$= \min\begin{cases}\frac{35(12)}{8} = 52.5''\\ \frac{10(12)}{2} = 60''\\ \frac{35(12)}{8} = 52.5''\end{cases} + \min\begin{cases}\frac{35(12)}{8} = 52.5''\\ \frac{10(12)}{2} = 60''\\ \frac{10(12)}{2} = 60''\end{cases}$

$b_{eff} = 105''$

⑩ check unshored strength

W16x31, $\phi_b M_p = 203$ Kft (obtained from table 3-19 in steel manual)

$w_u = 1.4 D_{slab} + 1.4 b_{bm\ self-wt} = 1.4(75\ psf)(10\ ft) + 1.4(31\ plf) = 1093\ plf = 1.093\ klf$ DL only

$w_u = 1.2 b_{slab} + 1.2 b_{bm\ self-wt} + 1.6 L_{construct.\ load} = 1.2(75)(10) + 1.2(31) + 1.6(\underbrace{20}_{construct.\ LL})(10) = 1257\ plf = 1.257\ klf$

$$M_u = \frac{1.257\ klf\ (35)^2}{8} = 192\ Kf < 203\ Kft \;\; \therefore \text{ ok for no shoring}$$

(11) check wet concrete deflection

W_{wc} = slab wt + bm self-wt = 75 psf (10 ft) + 31 plf = 781 plf = 0.781 klf

$$\Delta_{wc} = \frac{5(0.781\ klf)(35)^4(1728)}{384(29000)(375)} = 2.42''$$

$$\Delta_{wc_{max}} = \frac{35(12)}{240} = 1.75''$$

since $\Delta_{wc} > \Delta_{wc,max}$, camber beam

camber, $c = 0.80(2.42'') = 1.94''$ → use $c = 2''$

(12) check LL and TL deflection

$w_{LL} = 0.817$ klf

$Y2 = 7$, and point $Y1 = 4$ $(\Sigma Q_n = 274\ k)$, $I_{LB} = 1140\ in^4$

$$\Delta_{LL} = \frac{5(0.817)(35)^4(1728)}{384(29000)(1140)} = 0.83'' < \Delta_{LL,max} = 1.17'' \therefore ok$$

$$\Delta_{TL} = \frac{5(DL+LL)L^4(1728)}{384\ E\ I_{LB}} = \frac{5[850+31+817](35)^4(1728)/1000}{384(29000)(1140)} = 1.73'' \le \Delta_{TL} = \frac{35(12)}{240} = 1.75''$$

(13) check M_u, V_u, and bm self wt assumption for W16X26

$M_u = 366$ kft $< \phi M_n = 414$ kft $\therefore$ ok

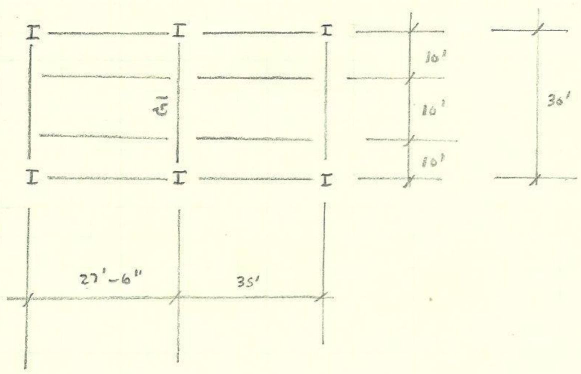
$V_u = 41.8$ k $< \phi V_n = 131$ k $\therefore$ ok

self-wt assumption $= \dfrac{31\ plf}{10\ ft} = 3.1$ psf < 5 psf $\therefore$ ok

$\nearrow w_{trib.}$

$\boxed{\text{use W16 X 31 beam w/ 32 studs and } c = 2''}$

interior composite girder design G1



① total load

dead load:

slab – 75 psf

SDL – 10 psf

bm self-wt allowance – 5 psf

girder self-wt – 5 psf

live load:

$L_o = 100$ psf

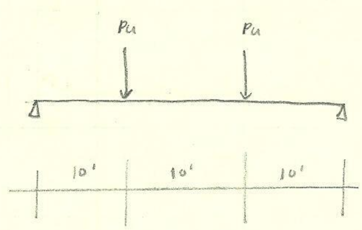
influence area $= (27.5 + 35)(30) = 1875\ ft^2$

$$L = 100 \times \max \begin{cases} 0.50 \\ 0.25 + \frac{15}{\sqrt{1875}} = 0.60 \end{cases}$$

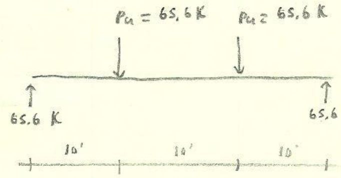
$L = 100(0.60) = 60$ psf

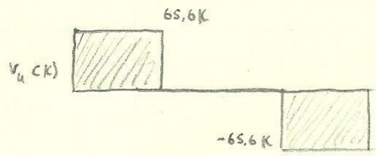
$w_u = 1.2(75 + 10 + 5 + 5) + 1.6(60) = 210$ psf

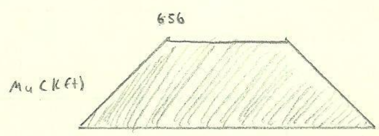
$P_u = 210\ psf\left(\frac{27.5 + 35}{2}\right)(10) = 65.6\ k$



② M_u and V_u







$V_u = 65.6$K
$M_u = 656$ Kft

③ determine I_{LB} based on $\Delta_{LL,max}$ and $\Delta_{TL,max}$

$$\Delta_{LL,max} = \frac{L}{360} = \frac{30(12)}{360} = 1'' \qquad \Delta_{TL,max} = \frac{L}{240} = \frac{30(12)}{240} = 1.5''$$

$$\Delta_{LL} = \frac{0.036\, P_L L^3 (1728)}{E I_{LB}} \qquad \Delta_{TL} = \frac{0.036\, P_{TL} L^3 (1728)}{E I_{LB}}$$

$$I_{LB,min} = \frac{0.036(18.8K)(30)^3(1728)}{29000(1)} = 1089 \text{ in}^4 \qquad I_{LB,min} = \frac{0.036(48.4K)(30)^3(1728)}{29000(1.5)} = 1869 \text{ in}^4$$

$$P_L = 60\text{psf}\left(\frac{27.5+35}{2}\right)(10) = 18.8K \qquad P_{TL} = (95+60)\text{psf}\,(31.25')(10') = 48.4\text{ K}$$

④ select potential W-shapes from tables 3-19 and 3-20 in steel manual
(select members with $\phi M_n \geq 656$ Kft, $I_{LB} \geq 1869$ in^4)
assume $Y2 = 6.5''$ ($a = 2''$)

W18X40, $I_{LB} = 2020$ in^4, $\phi M_n = 684$, $\Sigma Q_n = 590$ K, $Y1 =$ TFL

W18X46, $I_{LB} = 2090$ in^4, $\phi M_n = 696$ Kft, $\Sigma Q_n = 492$ K, $Y1 = 3$

W21X44, $I_{LB} = 2310$ in^4, $\phi M_n = 712$ Kft, $\Sigma Q_n = 431$ K, $Y1 = 4$

W21X48, $I_{LB} = 2290$ in^4, $\phi M_n = 712$ Kft, $\Sigma Q_n = 355$ K, $Y1 =$ BFL

W21 X50, $I_{LB} = 2160$ in^4, $\phi M_n = 693$ Kft, $\Sigma Q_n = 285$ K, $Y1 = 6$

Composite Steel Flr System | Tech 2 | page 8 of 9

⑤ determine horizontal shear strength for shear stud

- deck parallel
- $\frac{w_r}{h_r} = \frac{4.75}{3} = 1.58 > 1.5$
- assume weak position $\Rightarrow Q_n = 21.5\,K$
- use 3/4" ϕ stud
- assume $f'c = 4$ ksi normal wt conc.

⑥ determine # of studs/girder for shapes listed in step 4

W18X40: $\frac{\Sigma Q_n}{Q_n} \times 2 = \frac{590}{21.5} \times 2 = 56$ studs/girder

W18X46: $\frac{492}{21.5} \times 2 = 46$ studs/girder

W21X44 $\frac{431}{21.5} \times 2 = 42$ studs/girder

W21X48: $\frac{355}{21.5}(2) = 34$ studs/girder

W21X50: $\frac{285}{21.5}(2) = 28$ studs/girder

⑦ evaluate girders listed in step 4 for economy

W18X40 w/ 56 studs

$40\# \times 30' + 56 \text{ studs} \times 10\#/\text{stud} = 1760\#$

W18X46 w/ 46 studs

$46\# \times 30' + 46 \text{ studs} \times 10\#/\text{stud} = 1840\#$

W21X44 w/ 42 studs

$44\# \times 30' + 42 \text{ studs} \times 10\#/\text{stud} = 1740\#$

W21X48 w/ 34 studs

$48\# \times 30' + 34 \text{ studs} \times 10\#/\text{stud} = 1780\#$

W21X50 w/ 28 studs

$50\# \times 30' + 28 \text{ studs} \times 10\#/\text{stud} = 1780\#$

try W18X40 to reduce structural depth

⑧ check the compressive concrete block depth, a

$$b_{eff} = \min\begin{vmatrix} \frac{span}{8} \\ \frac{1}{2}\,trib_w \end{vmatrix} + \min\begin{vmatrix} \frac{span}{8} \\ \frac{1}{2}\,trib_w \end{vmatrix} = \frac{30(12)}{8} \times 2 = 90'' \qquad \frac{27.5(12)}{2} + \frac{35(12)}{2} = 292.5''$$

$$a = \frac{590}{0.85(4)(90)} = 1.93'' < \text{assumed } 2'' \therefore \text{ok}$$

⑨ check unshored strength

W18x40 , $\phi_b M_p = 294\ K$

$P_u = 1.2[75(10) + 40](31.25) + 1.6(20)(10)(31.25) = 39.6\ K$

$M_u = P_u a = 39.6\ K(10\ ft) = 396\ Kft > \phi_b M_p$ ∴ girder must be shored

select larger W-shape

try W21x50 , $\phi_b M_p = 413\ Kft$, $P_u = 40K$ (incl. 50 self wt)

$M_u = 400\ Kft < \phi_b M_p$ ∴ ok for no shoring

⑩ check LL and TL deflection

$$\Delta_{LL} = \frac{0.036(18.8\ K)(30)^3(1728)}{29000(2160)} = 0.50'' < \Delta_{LL,max} = 1''$$

$$\Delta_{TL} = \frac{0.036(48.4)(30)^3(1728)}{29000(2160)} = 1.30'' > \Delta_{TL,max} = 1.5''$$

⑪ check M_u, V_u, and self girder wt assumption

$M_u = 656\ Kft < \phi M_n = 693\ Kft$ ∴ ok
$V_u = 65.6\ K < \phi V_n = 237K$ ∴ ok

self-wt assumption $= \frac{50\ plf}{\underset{w_{trib}}{31.25\ ft}} = 1.60\ psf < 5\ psf$ ∴ ok

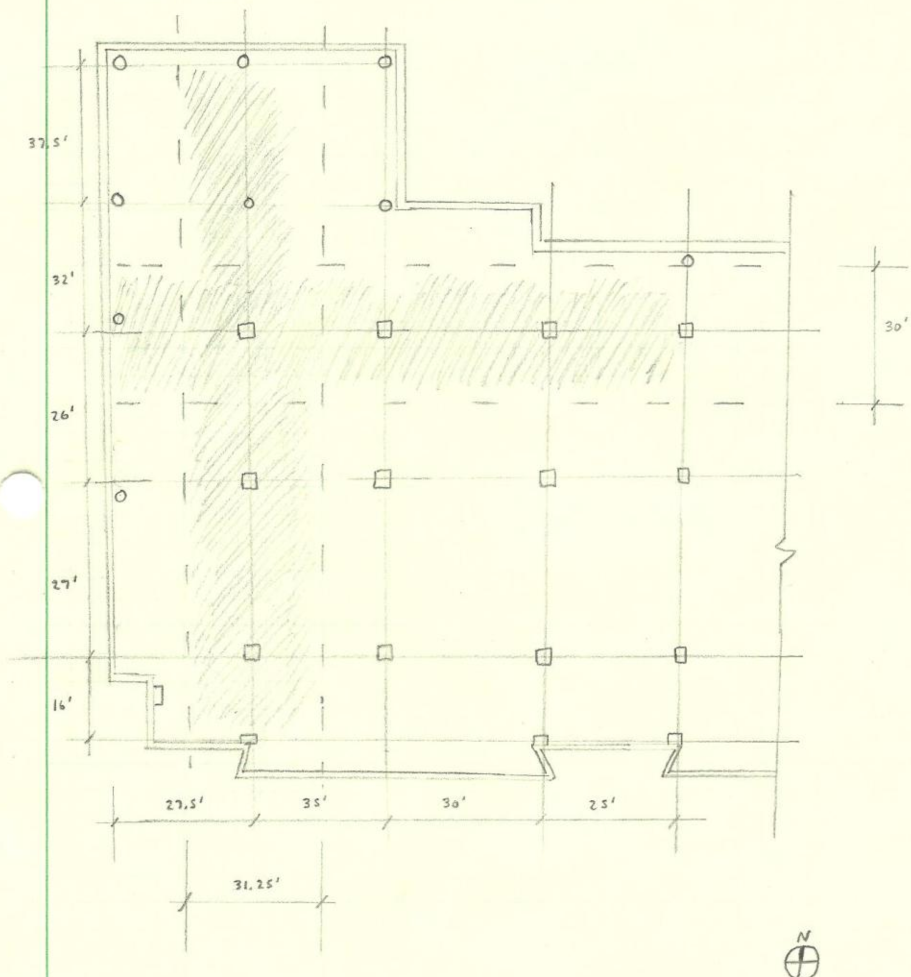
$\boxed{\text{use W21x50 girder w/ 28 studs}}$

Appendix C: Alternative 2 – Two-Way Post-Tensioned Slab Floor System

Two-way Post-tension slab	Tech 2	page 1 of 15

simplifying assumptions: design two-way post-tension slab for half of the building to determine whether this system will be a possible alternative for the entire building flr

Grid Layout



Two-way Post-tension slab | Tech 2 | page 2 of 15

Loads:

framing dead load = self wt

superimposed dead load = 10 psf

live load = 100 psf

2 hour fire rating

Materials

concrete : NW 150 pcf

$f'_c = 5000$ psi

$f'_{ci} = 3000$ psi

rebar: $f_y = 60{,}000$ psi

PT : unbonded tendons

$1/2''$ Ø, 7-wire strand, $A = 0.153$ in²

$f_{pu} = 270$ ksi

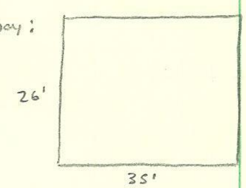
estimated prestress losses = 15 ksi (ACI 18.6)

$f_{se} = 0.7(270\text{ ksi}) - 15\text{ ksi} = 174$ ksi (ACI 18.5.1)

$P_{eff} = A \cdot f_{se} = (0.153)(174) = 26.6$ k/tendon

Preliminary slab thickness

typical bay:



start with $\frac{L}{h} = 45$

shortest span = 26'

$h = \frac{26(12)}{45} = 7''$ preliminary slab thickness

Loading

DL = self wt = 150 pcf (7/12 ft) = 87.5 psf

SDL = 10 psf

LL_o = 100 psf → reduce if $A_T > 400$ ft²

37.5' x 31.25' frame : $\underline{\text{N-S LL reduction}}$

$K_{LL} A_T = 1(37.5)(31.25) = 1171.9$ ft² > 400 ft² ∴ ok to reduce LL

$L = 100 \times \left(0.25 + \frac{15}{\sqrt{1171.9}}\right) = 100(0.69) = 69$ psf

32' x 31.25' frame : $K_{LL} A_T = 1(32)(31.25) = 1000$ ft² > 400 ft² ∴ ok to reduce LL

$L = 100(0.72) = 72$ psf

26' x 31.25' frame : $K_{LL} A_T = 1(26)(31.25) = 812.5$ ft² > 400 ft² ∴ ok to reduce LL

$L = 100(0.78) = 78$ psf

16' x 31.25' frame : $K_{LL} A_T = 1(16)(31.25) = 500$ ft² > 400 ft² ∴ ok to reduce LL

$L = 100(0.92) = 92$ psf

27' x 31.25' frame : $K_{LL} A_T = 1(27)(31.25) = 843.75$ ft² > 400 ft² ∴ ok to reduce LL

$L = 100(0.77) = 77$ psf

Two-way Post-tension slab	Tech 2	page 3 of 15

E-W LL reduction

30' x 30' bay : $K_{LL} A_T = 1(30)(30) = 900 > 400$ ft² ∴ ok to reduce LL

$L = 100(0.75) = 75$ psf

35' x 30' bay : $K_{LL} A_T = 1(35)(30) = 1050 > 400$ ft² ∴ ok to reduce LL

$L = 100(0.71) = 71$ psf

25' x 30' bay : $K_{LL} A_T = 1(25)(30) = 750 > 400$ ft² ∴ ok to reduce LL

$L = 100(0.80) = 80$ psf

Design of N-S interior frame

use Equivalent Frame method according to ACI 13.7
total bay width between centerlines = 32.5'
ignore column stiffness in equations for hand calculation simplicity
since $\frac{LL_{max}}{DL} = \frac{92}{97.5} = 0.94 > 3/4$ ∴ pattern loading not required
(to simplify preliminary calcs, neglect load patterning)

① calculate section properties

two way slab must be designed as class U → from ACI 318 section 18.3.3

gross cross-sectional properties allowed → from ACI 318 sect 18.3.4

$A = bh = (31.25')(12)(7\text{ in}) = 2625\text{ in}^2$

$S = \frac{bh^2}{6} = \frac{(375\text{ in})(7\text{ in})^2}{6} = 3062.5\text{ in}^3$

② set design parameters

allowable stresses : class U

at time of jacking (ACI 318 sect 18.4.1) :

$f'_{ci} = 3000$ psi

compression $= 0.6 f'_{ci} = 0.6(3000) = 1,800$ psi

Tension $= 3\sqrt{f'_{ci}} = 3\sqrt{3000} = 164$ psi

at service loads (ACI 318 sect 18.4.2 (a) and 18.3.3)

$f'_c = 5000$ psi

compression $= 0.45 f'_c = 0.45(5000) = 2,250$ psi

Tension $= 6\sqrt{f'_c} = 6\sqrt{5000} = 424$ psi

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Average precompression limits:

$\frac{P}{A} =$ 125 psi min (ACI 318 sect 18.12.4)

= 300 psi max

Target load balances:

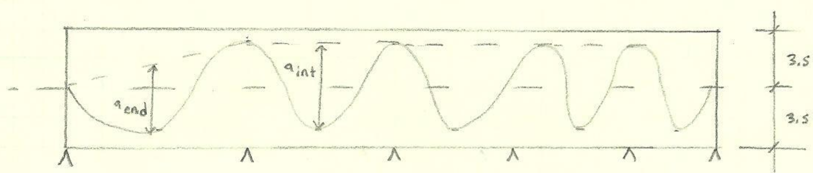
60-80% of DL (self wt) for slabs

use $0.65 w_{DL} = 0.65(87.5) = 57$ psf

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

restrained slabs = 3/4" bottom cover

③ initially assumed tendon profile



tendon ordinate	tendon (CGS) location
exterior support - anchor	3.5"
interior support - top	7 - 1 = 6"
interior support - bottom	1"
end span - bottom	1.75"

$a_{int} = 6 - 1 = 5"$

$a_{end} = \frac{3.5+6}{2} - 1.75 = 3"$

④ prestress force required to balance 65% of self-wt DL

$w_{balance} = w_b = 0.65 w_{DL} = 0.65(87.5\text{psf})(31.25\text{ ft}) = 1777\text{ plf} = 1.78\text{ klf}$

force needed in tendons to counteract the load in the 32' bay (note: use post-tension beams to support the 37.5' bay)

$P = \frac{w_b L^2}{8 a_{end}} = \frac{(1.78\text{ klf})(32\text{ft})^2}{8(3/12)} = 911\text{ K}$

note: since $a_{end} < a_{int}$ and the end 37.5' bay is the largest bay, the 37.5' end span will govern the max. required post-tension force

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⑤ check precompression allowance

determine number of tendons to achieve 911 K

$$\#\text{ of tendons} = \frac{(911\ k)}{26.6\ k/\text{tendon}} = 34.2 \text{ (round downward)}$$

use 34 tendons ; the actual force for bonded tendons $P_{actual} = (34 \text{ tendons})(26.6\ k)$
$= 904.4\ k$

the balanced load for the end span is slightly adjusted

$$w_b = \frac{904.4}{911}(1.78\ klf) = 1.77\ klf$$

determine actual precompression stress

$$\frac{P_{actual}}{A} = \frac{904.4\,(1000)}{2625\ in^2} = 344\ psi$$

> 125 psi min ∴ ok

< 300 psi max ∴ not ok ∴ reduce balance DL percentage

assumption; 50 % of load to be balanced in E-W direction and 50 % to be balanced by the N-S direction

– prestress force req'd to balance 50 % of self-wt DL

$$w_b = 0.50\,(87.5)(31.25) = 1.37\ klf$$

– force needed in tendons to counteract the load in the 32' bay

$$p = \frac{1.37\,(32)^2}{8\,(3/12)} = 701\ K$$

– check precompression allowance

$$\#\text{ tendons} = \frac{701}{26.6} = 26.4 \Rightarrow \text{use } 26 \text{ tendons}$$

$$P_{actual} = 26\,(26.6) = 691.6\ K$$

the balanced load for the end span

$w_b = \frac{691.6}{701}(1.37) = 1.35$ klf

actual precompression stress

$\frac{P_{actual}}{A} = \frac{691.6(1000)}{2625} = 263.5$ psi $>$ 125 psi min ∴ ok

$<$ 300 psi min ∴ ok

- check interior spans

$P = \frac{1.35(27)^2}{8(5/12)} = 295$ k < 691.6 k ∴ less force req'd for int. 26' bay

$w_b = \frac{691.6(8)(5/12)}{27^2} = 3.16$ klf ← 27' interior span

$\frac{w_b}{w_{DL}} = \frac{3.16}{2.73} = 1.16 > 1.0$ ∴ unacceptable for design

$2.73 = (7/12)(150)(31.25)$

$w_b = \frac{P8a_{int}}{L^2} \Rightarrow a_{int} \le \frac{2.73\text{ klf }(27\text{ft})^2}{691.6\text{k }(8)} \times 12 = 4.3''$

use $a_{int} = 4.25''$ for 27' span

$a_{int} \le \frac{2.73(26)^2}{691.6(8)} = 4.0''$ for 26' span

- check exterior spans

16' ext. span: $w_b = \frac{P8a_{end}}{L^2} \Rightarrow a_{end} \le \frac{2.73\text{ klf }(16\text{ft})^2}{691.6\text{k }(8)} \times 12 = 1.52 \Rightarrow$ use $a_{end} = 1.5''$

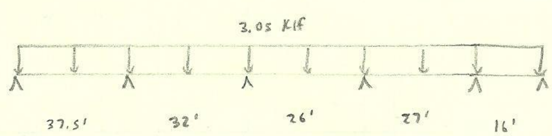
37.5' ext. span: $a_{end} \le \frac{2.73(37.5)^2}{691.6\text{k}(8)} \times 12 = 8.33 \Rightarrow$ use $a_{end} = 3''$

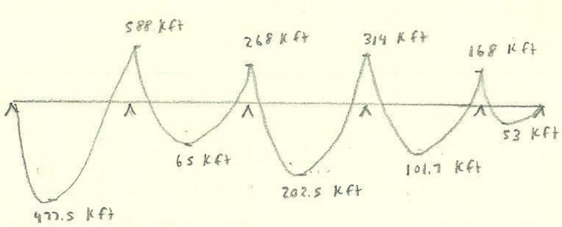
N-S interior frame:

effective prestress force, $P_{eff} = 691.6$ K

(7) check slab stresses

- dead load moments : $w_{DL} = (87.5 + 10)/_{1000}\ (31.25\text{ ft}) = 3.05$ klf

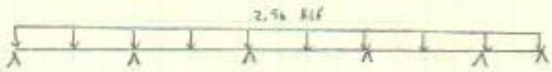


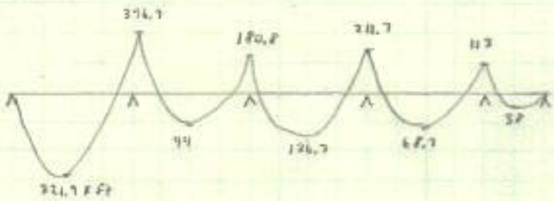


Two-way post-tension slab	Tech 2	page 7 of 15

- live load moments

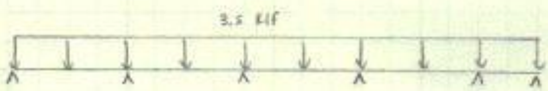
w_{LL} = 91 psf (32.5 ft) = 2.96 klf

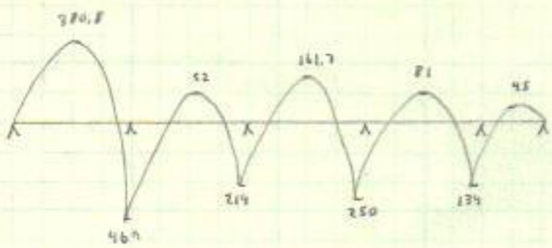




- total balancing moment

w_b = 3.5 klf (average of 5 bays)





Two-way post-tension slab	Tech 2	page 8 of 15

stage 1: stress immediately after jacking (DL + PT)

+ tension
− compression
+ moment creates ten (+) @ bot and comp (−) @ top

mid span stresses

$$f_{top} = (-M_{DL} + M_{bal})/S - P/A$$

$$f_{bot} = (+M_{DL} - M_{bal})/S - P/A$$

interior spans:

$$36' \text{ span} - f_{top} = \frac{(-65 + 52)(12\ \text{in/ft})(1000)}{6500} - 125 = -145 \text{ compression} < 0.6 f'_{ci} < 0.6 f'_{ci} = 1800 \text{ psi} \therefore ok$$

$$f_{bot} = 24 - 125 = -101 \text{ psi compression} < 1800 \text{ psi} \therefore ok$$

$$30' \text{ span} - f_{top} = \frac{(-202.5 + 162.7)(12)(1000)}{6500} - 125 = -200 \text{ compression} < 0.6 f'_{ci} = 1800 \text{ psi} \therefore ok$$

$$f_{bot} = 75 - 125 = -50 \text{ psi compression} < 1800 \text{ psi} \therefore ok$$

$$25' \text{ span} - f_{top} = \frac{(-101.7 + 81)(12)(1000)}{6500} - 125 = -163 \text{ compression} < 0.6 f'_{ci} = 1800 \therefore ok$$

$$f_{bot} = 38 - 125 = -87 \text{ compression} < 1800 \text{ psi} \therefore ok$$

exterior spans:

$$37.5' \text{ span} - f_{top} = \frac{(-477.5 + 380.8)(12)(1000)}{6500} - 125 = -303 \text{ compression} < 1800 \text{ psi} \therefore ok$$

$$f_{bot} = 178.5 - 125 = 53.5 \text{ tension} < 3\sqrt{f'_{ci}} = 164 \text{ psi} \therefore ok$$

$$16' \text{ span} - f_{top} = \frac{(-53 + 45)(12)(1000)}{6500} - 125 = -139.8 \text{ compression} < 1800 \text{ psi} \therefore ok$$

$$f_{bot} = 14.8 - 125 = -110 \text{ compression} < 1800 \text{ psi} \therefore ok$$

Stage 2: stresses at service load (DL + LL + PT) after losses

midspan stresses

$$f_{top} = (-M_{DL} - M_{LL} + M_{bal})/S - P/A$$

$$f_{bot} = (+M_{DL} + M_{LL} - M_{bal})/S - P/A$$

interior spans

30' span – $f_{top} = \dfrac{(-65 - 44 + 52)(12)(1000)}{6500} - 125 = -230 < 0.45 f'_c = 2250$ ∴ ok (comp)

$f_{bot} = 105 - 125 = -20 < 2250$ psi ∴ ok (comp.)

30' span – $f_{top} = \dfrac{(-202.5 - 136.7 + 161.2)(12)(1000)}{6500} - 125 = -452.7 < 2250$ ∴ ok (comp)

$f_{bot} = 327.7 - 125 = 202.7$ psi $< 6\sqrt{f'_c} = 424$ psi ∴ ok (tension)

25' span – $f_{top} = \dfrac{(-101.7 - 68.7 + 81)(12)(1000)}{6500} - 125 = -290$ psi < 2250 ∴ ok (comp)

$f_{bot} = 168 - 125 = 40$ psi < 424 psi ∴ ok (tension)

exterior spans

37.5' span – $f_{top} = \dfrac{(-477.5 - 321.7 + 380.8)(12)(1000)}{6500} - 125 = -897 < 2250$ psi ∴ ok

$f_{bot} = 772 + 125 = 697$ psi (tension) > 424 psi not good

16' span – $f_{top} = \dfrac{(-53 - 38 + 45)(12)(1000)}{6500} - 125 = -210 < 2250$ psi ∴ ok (comp)

$f_{bot} = 85 - 125 = -40 < 2250$ psi ∴ ok (comp)

Two-way post-tension slab | Tech 2 | page 10 of 15

⑧ ultimate strength

determine factored moments

primary moments, $M_1 = Pe$; $e = 0$ for ext. support

$e = 9 - 5 = 4"$ @ int. support

$$M_1 = \frac{479(4)}{12} = 160 \text{ kft}$$

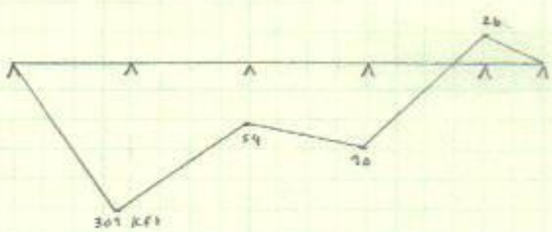
secondary moments, $M_{sec} = M_{bal} - M_1$

$= 469 - 160 = 309$ ← support 1 (interior)

$= 214 - 160 = 54$ ← support 2 (interior)

$= 250 - 160 = 90$ ← support 3 (interior)

$= 134 - 160 = -26$ ← support 4 (interior)



the typical load combination for ultimate strength design

$$M_u = 1.2M_{DL} + 1.6M_{LL} + 1.0M_{sec}$$

@ midspan : $M_u = 1.2(477.5) + 1.6(321.7) + 1.0(159.5) = 1242 \text{ kft}$

@ support : $M_u = 1.2(588) + 1.6(-396.7) + 1.0(309) = -1031 \text{ kft}$

Two-way post-tension slab	Tech 2	page 11 of 15

determine minimum bonded reinforcement

positive moment region

30' interior span : $f_t = 202.7 > 2\sqrt{f'c} = 2\sqrt{5000} = 141.4$ psi $\therefore$ reinf. positive req'd

minimum pos. mom. reinf. req'd

$$y = \frac{f_t}{(f_t + f_c)} h = \frac{202.7}{(202.7 + 452.7)} (10) = 3.1 \text{ in}$$

$$N_c = \frac{M_{DL} + M_{LL}}{S} 0.5 \times y\, \ell_2 = \frac{(202.5 + 136.7)\text{ k ft} (12)(0.5)(3.1)(32.5)(12)}{6500}$$
$= 378.5$ K

$$A_{s,min} = \frac{N_c}{0.5 f_y} = \frac{378.5}{0.5(60)} = 12.6 \text{ in}^2$$

distribute reinf. over 32.5 ft width

$$A_{s,min} = \frac{12.6 \text{ in}^2}{32.5 \text{ ft}} = 0.39 \text{ in}^2/\text{ft}$$

pos reinf. – use #7 @ 12" o.c. bottom = 0.60 in²/ft

note: use same reinf. in 25' interior bay

37.5' exterior span : $f_t = 647$ psi $> 2\sqrt{f'c} = 141.4$ psi $\therefore$ reinf. positive req'd

$$y = \frac{647}{(647 + 897)} (10) = 4.2 \text{ in}$$

$$N_c = \frac{(477.5 + 321.7)\text{ k ft}}{6500} (12)(0.5)(4.2)(32.5)(12) = 1208 \text{ K}$$

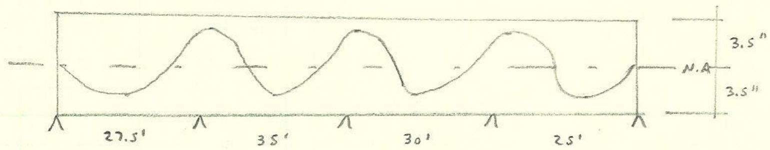
$$A_{s,min} = \frac{1208}{0.5(60)} = 40 \text{ in}^2$$

distribute over 32.5 ft width

$$A_{s,min} = \frac{40}{32.5} = 1.23 \text{ in}^2/\text{ft}$$

Two-way post-tension slab | Tech 2 | page 12 of 15

design of E-W interior frame



note: initially assumed tendon profile same as N-S initially assumed tendon profile

① prestress required to bance 50% of self-wt bL

$W_b = 0.50\,(87.5\text{ psf})(30\text{ft}) = 1312.5\text{ plf} = 1.31\text{ klf}$

② force needed in tendons to counteract the load in the end bay

$P = \frac{1.31(27.5)^2}{8(3/12)} = 495.3\text{ K}$ $P = \frac{1.31(25)^2}{8(3/12)} = 409.4\text{ K}$ $\Rightarrow$ 495.3 K controls

30' end bay   25' end bay

③ check precompression allowance

$\#\text{ tendons} = \frac{495.3\text{K}}{26.6\text{ K/tend.}} = 18.6 \Rightarrow$ use 18 tendons

$P_{actual} = 26.6(18) = 479\text{ K}$

the balance load for the end span

$W_b = \frac{479}{495.3}(1.31) = 1.27\text{ klf}$

actual precompression stress

$\frac{P_{actual}}{A} = \frac{479(1000)}{2520\text{ in}^2} = 190\text{ psi}$ > 125 psi min ∴ ok

$7" \times 30'(12)$ < 300 psi max ∴ ok

- check 25' end span

$P = \frac{1.27(25)^2}{8(3/12)} = 397\text{ K} < 479\text{ K}$ ∴ less force req'd

$W_b = \frac{479(8)(3/12)}{25^2} = 1.53\text{ klf} \Rightarrow \frac{W_b}{W_{DL}} = \frac{1.53}{2.63} = 0.58 < 1.0$ ∴ acceptable for design

- check interior spans

35' span : $P = \frac{1.27(35)^2}{8(5/12)} = 467 \text{ k} < 479$ ∴ less force req'd

$$w_b = \frac{479(8)(5/12)}{35^2} = 1.30 \text{ klf}$$

$$\frac{w_b}{w_{DL}} = \frac{1.30}{2.63} = 0.49 < 1.0$$ ∴ acceptable for design

30' span : $P = \frac{1.27(30)^2}{8(5/12)} = 343 \text{ k} < 479\text{K}$ ∴ less force req'd

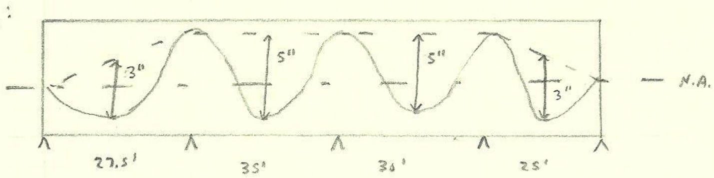
$$w_b = \frac{479(8)(5/12)}{30^2} = 1.77 \text{ klf}$$

$$\frac{w_b}{w_{DL}} = \frac{1.77}{2.63} = 0.67 < 1.0$$ ∴ acceptable for design

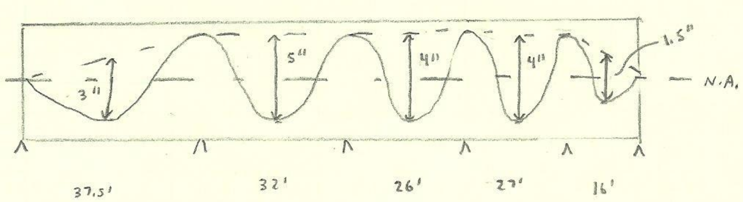
E-W interior frame effective prestress, $P_{eff} = 479$ k

final tendon profile

E-W direction:



N-S direction:

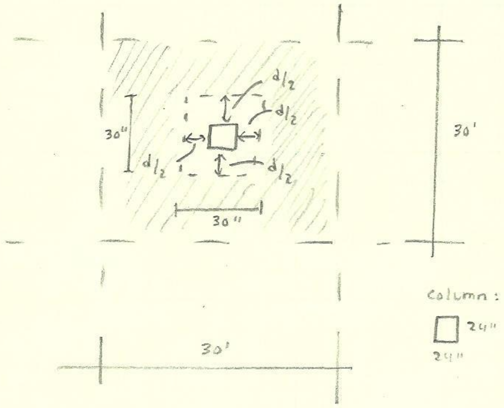


Two-way post-tension slab	Tech 2	page 14 of 15

check punching shear (two-way action)

- use average typical 30' x 30' bay to check punching shear

$V_u = w_u \cdot Area$



$w_u = 1.2(87.5 + 10)\,psf + 1.6(100)\,psf$

↑ slab wt ↑ SDL

$= 277\ psf = 0.277\ ksf$

$d = 7" - 1" = 6"$

$\frac{d}{2} = 3"$

$V_u = 0.277\ ksf\ (30 \times 30 - 2.5 \times 2.5)$
$= 247.6\ K$

perimeter at critical section, $b_o = 30"(4) = 120"$

$\frac{b_o}{d} = \frac{120"}{6"} = 20$; $\alpha_s = 40$ for interior col. ; $\beta_c = \frac{b_L}{b_s} = \frac{24}{24} = 1.0$

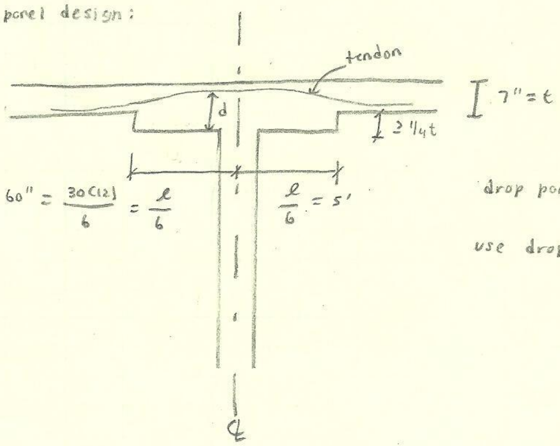
$V_c = 4\sqrt{f'c}\ b_o d = 4\sqrt{5000}\ (120)(6)/_{1000} = 203.6\,K$ ← governs

$V_c = (2 + \frac{4}{\beta_c})\sqrt{f'c}\ b_o d = 305.5\ K$

$V_c = \left(\frac{\alpha_s}{b_o/d} + 2\right)\sqrt{f'c}\ b_o d = 4\ \sqrt{f'c}\ b_o d$

$\phi V_c = 0.75(204) = 153\ K < V_u = 248\ K$ ∴ drop panels are needed

drop panel design:



$5' = 60" = \frac{30(12)}{6} = \frac{\ell}{6}$ $\frac{\ell}{6} = 5'$

drop panel depth ≥ 1/4 t = 1/4 (7") = 1.75"

use drop panel thickness of 3"

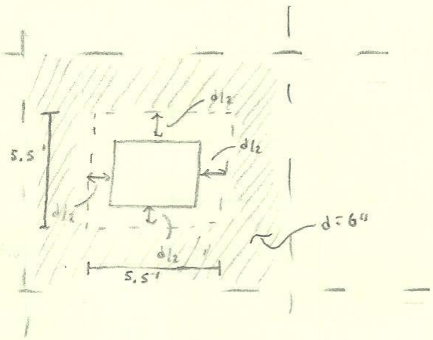
effective depth = 6"

d = 6"

$b_o = (5' + 6/12')\,4 = 22' = 264"$

$V_c = \left(\frac{40}{44} + 2\right)\sqrt{5000}\,(264)(6) = 326\,K$

$\phi V_c = 0.75(326\,K) = 244.5\,K$



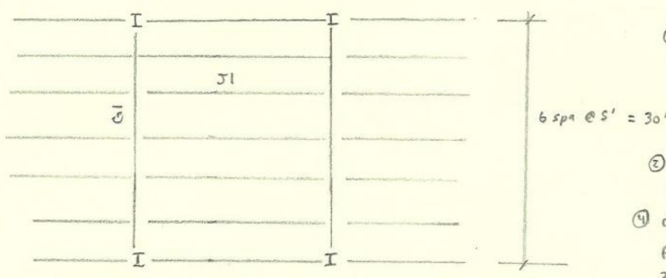
$V_u = 0.277(900 - 30.25)$

$= 241\,K < \phi V_c \therefore OK$

$\boxed{\text{Use 3" drop panels}}$

Appendix D: Alternative 3 – Composite Joist/ Steel Girder System

composite steel joist/ girder flr system | Tech 2 | page 1 of 5



composite floor deck design

① superimposed load on deck

total load = 110 psf + slab + deck

② - ctr-to-ctr span of supports = 5'-0"

- 3 + span condition

④ deck type - select deck type from vulcraft steel deck composite 2010 catalog

use 1.5 VL22 with 6" thick slab

(4.5" topping for 2 hr fire rating)

- superimposed uniform load: 400 psf > 110 psf ∴ ok

- deck max SDI const. span = 6'-4" > 5'-0" ∴ ok

- self wt = 63 psf

interior composite joist design J1

① total load

dead load:

SDL - 10 psf

slab - 63

$w_D = (10+63)\text{psf}\,(5\text{ft}) = 365\text{ plf}$

live load:

$L_o = 100\text{ psf}$

influence area $= K_{LL} A_T = 2(5\text{ft})(35\text{ft}) = 350\text{ ft}^2 < 400\text{ ft}^2$ ∴ LL cannot be reduced

$w_L = 100\text{ psf}\,(5\text{ft}) = 500\text{ plf}$

total load, $w_u = 1.2(365) + 1.6(500) = 1238\text{ plf}$

② joist span - 35'-0"

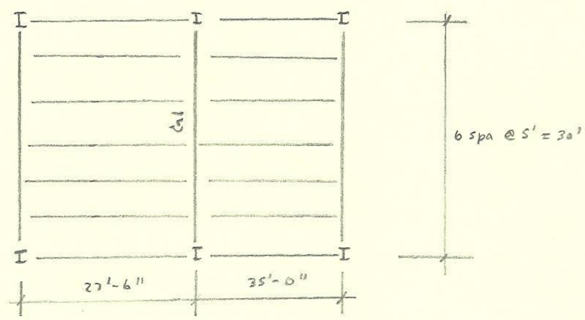
③ ctr-to-ctr joist spacing - 5'-0" ; number of spaces $= \frac{30'}{5'} = 6$

④ select joist from vulcraft composite joists 2009 catalog

- 14CJ :
  - allowable load = 1400 plf > 1238 plf ∴ ok
  - allowable LL that produces a deflection of $L/360$ = 607 plf > 500 plf ∴ ok
  - allowable total load that produces a deflection of $L/240$ $= 607(360/240) = 910.5\text{ plf} > 865\text{ plf}$ ∴ ok
  - number of shear studs/ span = 40 - 5/8" ϕ studs
  - 2 rows of bridging

$\boxed{\text{use a 14CJ 1400/607 composite joist with (40) - 5/8" ϕ studs}}$

composite steel joist / girder flr system | Tech 2 | page 2 of 5



① total load

dead load:

slab – 63 psf

SDL – 10 psf

joist self-wt – 22 plf

girder self-wt estimate – 5 psf

live load:

$L_0 = 100$ psf

LL reduces to 60 psf (from pg. 6 of "composite steel flr system" calcs)

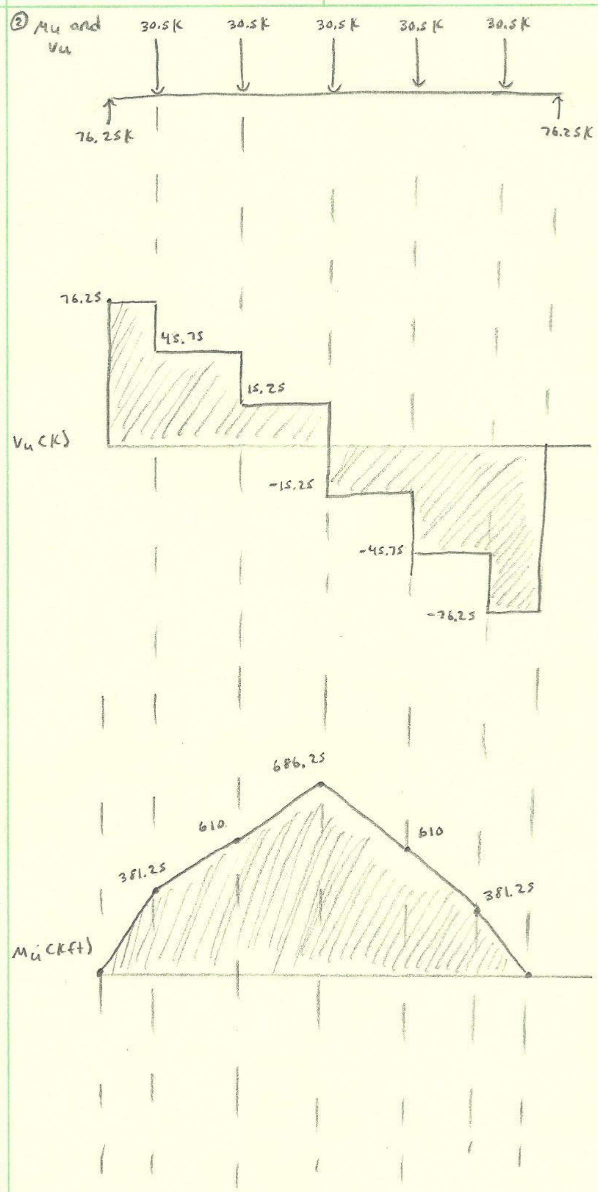
$P_u = 1.2(63+10+5)\text{ psf}(5\text{ft})(31.25\text{ ft}) + 1.2(22\text{ plf})(31.25\text{ft}) + 1.6(60)(5\text{ft})(31.25\text{ft})$

$= 14625 + 825 + 15000 = 30.5\text{ k}$

composite steel joist / girder flr system

Tech 2

② Mu and Vu



$V_u = 76.25$ K

$M_u = 686.25$ k ft

composite steel joist / girder flr system | Tech 2 | page 4 of 5

③ determine I_x based on $\Delta_{LL,max}$ and $\Delta_{TL,max}$

assumption: four or more equal point loads may be assumed distributed

$w_u = 1.2\,(63+10+5)\,psf\,(31.25\,ft) + 1.2\,(4.4\,psf)(31.25\,ft) + 1.6\,(60\,psf)(31.25\,ft)$
$= 6.1\,plf$

$\frac{22\,plf}{5\,ft}$ joist tributary width

$\Delta_{L,max} = \frac{L}{360} = \frac{30(12)}{360} = 1''$

unfactored LL

$I_x \geq \frac{5(1.875\,klf)(30\,ft)^4(1728)}{384(29000)(1'')} = 1178\,in^4$

$\Delta_{TL,max} = \frac{L}{240} = \frac{30(12)}{240} = 1.5''$

unfactored DL

$I_x \geq \frac{5(1.875+2.56)(30)^4(1728)}{384(29000)(1.5)} = 1858\,in^4$

using table 3-3 in manual
try W21x93, $I_x = 2070\,in^4$

④ $M_u \leq \phi M_n$

assumption: girder is simply supported and laterally braced at joist locations only ∴
the unbraced length $L_b = 5'$

$M_u = \frac{(6.1\,plf)(30\,ft)^2}{8} = 686.3\,kft$

$L_b = 5' < L_p = 6.50'$ (from table 3-2) ∴ there is no unbraced length problem

$\phi_b M_{px} = 829\,kft > M_u$ ∴ ok

$\boxed{\text{use W21 x 93 girder}}$

⑤ check $V_u \leq \phi V_n$

$V_u = \frac{w_u L}{2} = \frac{6.1\,plf\,(30\,ft)}{2} = 91.5\,K < \phi V_n = 376\,K$ (from table 3-2) ∴ ok

composite steel joist / girder flr system	Tech 2	page 5 of 5

⑥ check steel joist and steel girder deflections

steel joist: $\Delta_{LL} = \frac{5(0.50 \text{ klf})(35 \text{ ft})^4 (1728)}{384 (29000)(606 \text{ in}^4)} = 0.96'' < L/360 = 1.17''$ ∴ ok

$\Delta_{TL} = \frac{5(0.865 \text{ klf})(35')^4 (1728)}{384 (29000)(606 \text{ in}^4)} = 1.66'' < L/240 = 1.75''$ ∴ ok

steel girder: $\Delta_{LL} = \frac{5(1.875 \text{ klf})(30 \text{ ft})^4 (1728)}{384 (29000)(2070 \text{ in}^4)} = 0.57'' < L/360 = 1''$ ∴ ok

$\Delta_{TL} = \frac{5(4.44 \text{ klf})(30 \text{ ft})^4 (1728)}{384 (29000)(2070 \text{ in}^4)} = 1.35'' < L/240 = 1.5''$ ∴ ok

Vulcraft 2009 Composite Joist Table

NORMAL WEIGHT CONCRETE

DESIGN GUIDE LRFD WEIGHT TABLE FOR COMPOSITE STEEL JOISTS, C

Based on a 50 ksi Maximum Yield Strength

BEARING HEIGHT	2 1/2"	5"	7 1/2"

Joist Span (ft.)	Joist Depth (in.)												
			Concrete Slab Parameters										
			Normal Weight Concrete (145 pcf) f'c = 4.0 ksi										
		hr (in.)	1.5	1.5	1.5	2	2	2	2	2	3	3	3
		tc (in.)	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
		Js (ft.)	5	5.5	6	7	7.5	8	9	10	11	12	13
			Total Safe Factored Uniformly Distributed Joist Load in Pounds Per Linear Foot										
		TL	1400	1600	1800	2000	2200	2400	2700	3000	3300	3600	3900
35	14	Wt(plf)	22	25	27	29	33	33	38	41	43	47	53
		W360(plf)	607	695	750	871	984	994	1130	1229	1386	1478	1627
		N-ds	40-5/8"	50-5/8"	54-5/8"	44-3/4"	52-3/4"	52-3/4"	60-3/4"	70-3/4"	62-3/4"	70-3/4"	80-3/4"
		Ieff(in4)	606	694	748	870	982	992	1130	1230	1380	1480	1620
		Bridging	(2)H	(2)H	(2)H	(2)H	(2)H	(2)H	(2)H	(2)H	(1)H	(1)H	(1)X
	16	Wt(plf)	21	23	25	28	29	31	34	38	41	42	46
		W360(plf)	654	724	819	950	1012	1108	1160	1304	1524	1557	1660
		N-ds	38-5/8"	40-5/8"	50-5/8"	38-3/4"	44-3/4"	52-3/4"	52-3/4"	60-3/4"	62-3/4"	62-3/4"	70-3/4"
		Ieff(in4)	652	722	817	949	1010	1110	1160	1300	1520	1550	1660
		Bridging	(2)H	(2)H	(2)H	(2)H	(2)H	(2)H	(1)H	(1)H	(1)H	(1)H	(1)X
	18	Wt(plf)	16.5	19.0	21	22	25	27	33	36	38	40	44
		W360(plf)	691	785	857	957	1065	1141	1374	1509	1651	1687	1855
		N-ds	48-1/2"	38-5/8"	40-5/8"	42-5/8"	34-3/4"	38-3/4"	52-3/4"	62-3/4"	52-3/4"	52-3/4"	62-3/4"
		Ieff(in4)	689	783	855	955	1060	1140	1370	1510	1650	1680	1850
		Bridging	(2)H	(2)H	(2)H	(2)H	(2)H	(2)H	(1)H	(1)H	(1)H	(1)H	(1)X
	20	Wt(plf)	15.6	17.8	19.5	21	24	26	30	33	35	39	41
		W360(plf)	783	877	966	1100	1212	1292	1520	1659	1804	2026	2054
		N-ds	44-1/2"	34-5/8"	36-5/8"	38-5/8"	32-3/4"	34-3/4"	44-3/4"	52-3/4"	44-3/4"	52-3/4"	52-3/4"
		Ieff(in4)	781	875	964	1100	1210	1290	1520	1660	1800	2020	2050
		Bridging	(2)H	(2)H	(2)H	(2)H	(2)H	(2)H	(1)H	(1)H	(1)H	(1)H	(1)X
	22	Wt(plf)	14.8	16.9	19.1	20	23	24	29	31	35	36	40
		W360(plf)	869	968	1129	1207	1358	1438	1694	1794	2120	2161	2415
		N-ds	40-1/2"	30-5/8"	38-5/8"	34-5/8"	30-3/4"	32-3/4"	38-3/4"	44-3/4"	44-3/4"	44-3/4"	52-3/4"
		Ieff(in4)	868	966	1130	1200	1360	1440	1690	1790	2120	2160	2410
		Bridging	(2)H	(2)H	(2)H	(2)H	(2)H	(2)H	(1)H	(1)H	(1)H	(1)H	(1)X
	24	Wt(plf)	14.3	16.2	18.1	20	22	23	28	29	32	36	37
		W360(plf)	950	1052	1209	1395	1525	1579	1835	1960	2234	2491	2524
		N-ds	36-1/2"	28-5/8"	34-5/8"	34-5/8"	40-5/8"	30-3/4"	34-3/4"	38-3/4"	36-3/4"	44-3/4"	44-3/4"
		Ieff(in4)	948	1050	1210	1390	1520	1580	1830	1960	2230	2490	2520
		Bridging	(2)H	(2)H	(2)H	(2)H	(2)H	(2)H	(1)H	(1)H	(1)H	(1)H	(1)X

Appendix E: R.S. Means 2010 Cost Details

Two-Way Flat Slab System

B1010 222 — Cast in Place Flat Slab with Drop Panels

	BAY SIZE (FT.)	SUPERIMPOSED LOAD (P.S.F.)	MINIMUM COL. SIZE (IN.)	SLAB & DROP (IN.)	TOTAL LOAD (P.S.F.)	COST PER S.F.		
						MAT.	INST.	TOTAL
1960	20 x 20	40	12	7 - 3	132	4.99	8.25	13.24
1980		75	16	7 - 4	168	5.30	8.45	13.75
2000		125	18	7 - 6	221	5.85	8.75	14.60
3200	25 x 25	40	12	8-1/2 - 5-1/2	154	5.85	8.65	14.50
4000		125	20	8-1/2 - 8-1/2	243	6.55	9.30	15.85
4400		200	24	9 - 8-1/2	329	6.90	9.55	16.45
5000	25 x 30	40	14	9-1/2 - 7	168	6.35	9	15.35
5200		75	18	9-1/2 - 7	203	6.75	9.35	16.10
5600		125	22	9-1/2 - 8	256	7.05	9.55	16.60
5400	30 x 30	40	14	10-1/2 - 7-1/2	182	6.90	9.20	16.10
5600		75	18	10-1/2 - 7-1/2	[illegible]	[illegible]	[illegible]	[illegible]
5800		125	22	10-1/2 - 9	269	7.60	9.85	17.45

Composite Beam with Composite Deck System

B1010 256 — Composite Beams, Deck & Slab

	BAY SIZE (FT.)	SUPERIMPOSED LOAD (P.S.F.)	SLAB THICKNESS (IN.)	TOTAL DEPTH (FT.-IN.)	TOTAL LOAD (P.S.F.)	COST PER S.F.		
						MAT.	INST.	TOTAL
2400	20x25	40	5-1/2	1 - 5-1/2	80	10	5.70	15.70
2500		75	5-1/2	1 - 9-1/2	115	10.40	5.70	16.10
2750		125	5-1/2	1 - 9-1/2	167	12.80	6.75	19.55
2900		200	6-1/4	1 - 11-1/2	251	14.50	7.25	21.75
3000	25x25	40	5-1/2	1 - 9-1/2	82	9.65	5.45	15.10
3100		75	5-1/2	1 - 11-1/2	118	10.80	5.55	16.35
3200		125	5-1/2	2 - 2-1/2	169	11.30	6	17.30
3300		200	6-1/4	2 - 6-1/4	252	15.35	7	22.35
3400	25x30	40	5-1/2	1 - 11-1/2	83	9.85	5.40	15.25
3600		75	5-1/2	1 - 11-1/2	119	10.65	5.50	16.15
3900		125	5-1/2	1 - 11-1/2	170	12.50	6.20	18.70
4000		200	6-1/4	2 - 6-1/4	252	15.40	7	22.40
4200	30x30	40	5-1/2	1 - 11-1/2	81	10.10	5.55	15.65
4400		75	5-1/2	2 - 2-1/2	116	10.90	5.80	16.70
4500		125	5-1/2	2 - 5-1/2	168	13.30	6.50	19.80
4700		200	6-1/4	2 - 9-1/4	252	16.10	7.55	23.65

Steel Joist/ steel girder floor system

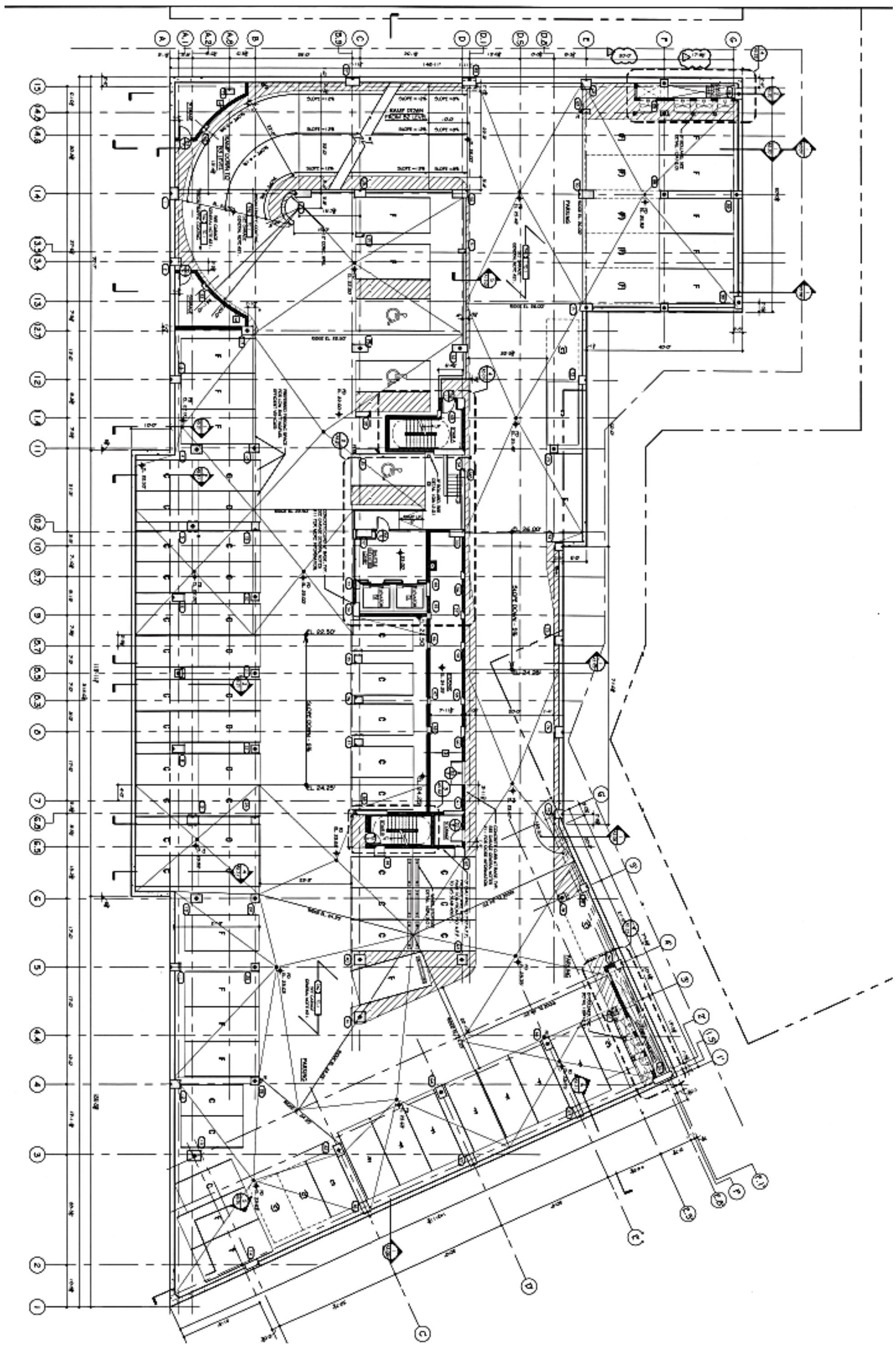
B10 Superstructure

B1010 Floor Construction

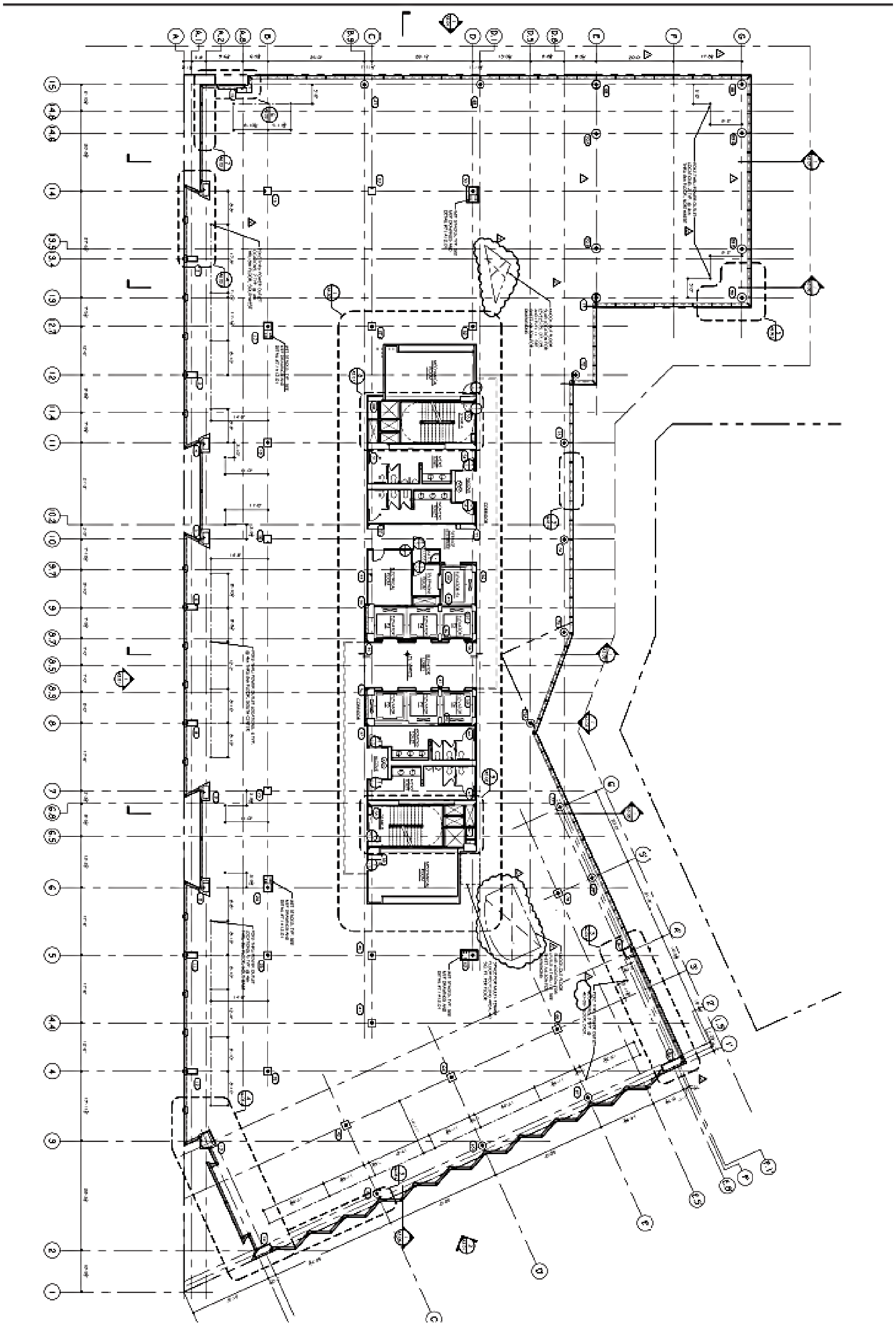
B1010 250 Steel Joists, Beams & Slab on Columns

	BAY SIZE (FT.)	SUPERIMPOSED LOAD (P.S.F.)	DEPTH (IN.)	TOTAL LOAD (P.S.F.)	COLUMN ADD	COST PER S.F. MAT.	INST.	TOTAL
5300	25x25	100	29	145		11.25	5.90	17.15
5400					column	1.15	.38	1.53
5500	25x25	125	32	170		11.90	6.20	18.10
5600					column	1.27	.42	1.69
5700	25x30	40	29	84		9.45	5.50	14.95
5800					column	.95	.32	1.27
5900	25x30	65	29	110		9.75	5.75	15.50
6000					column	.95	.32	1.27
6050	25x30	75	29	120		10.60	5.25	15.85
6100					column	1.06	.35	1.41
6150	25x30	100	29	145		11.45	5.50	16.95
6200					column	1.06	.35	1.41
6250	25x30	125	32	170		12.30	6.80	19.10
6300					column	1.22	.40	1.62
6350	30x30	40	29	84		9.90	4.98	14.88
6400					column	.88	.29	1.17
6500	30x30	65	29	110		11.25	5.45	16.70
6600					column	.88	.29	1.17
6700	30x30	75	32	120		11.50	5.50	17
6800					column	1.01	.33	1.34
6900	30x30	100	35	145		12.75	5.95	18.70
7000					column	1.18	.40	1.58
7100	30x30	125	35	172		13.90	7.40	21.30
7200					column	1.31	.44	1.75
7300	30x35	40	29	85		11.15	5.40	16.55
7400					column	.75	.24	.99
7500	30x35	65	29	111		12.30	6.85	19.15
7600					column	.97	.32	1.29
7700	30x35	75	32	121		12.35	6.85	19.20
7800					column	.99	.33	1.32
7900	30x35	100	35	148		13.40	6.20	19.60
8000					column	1.22	.40	1.62
8100	30x35	125	38	173		14.80	6.70	21.50
8200					column	1.24	.41	1.65
8300	35x35	40	32	85		11.45	5.50	16.95
8400					column	.87	.28	1.15
8500	35x35	65	35	111		13	7.10	20.10
8600					column	1.04	.35	1.39
9300	35x35	75	38	121		13.35	7.20	20.55
9400					column	1.04	.35	1.39
9500	35x35	100	38	148		14.40	7.65	22.05
9600					column	1.29	.42	1.71
9750	35x35	125	41	173		15.65	6.95	22.60
9800					column	1.31	.44	1.75

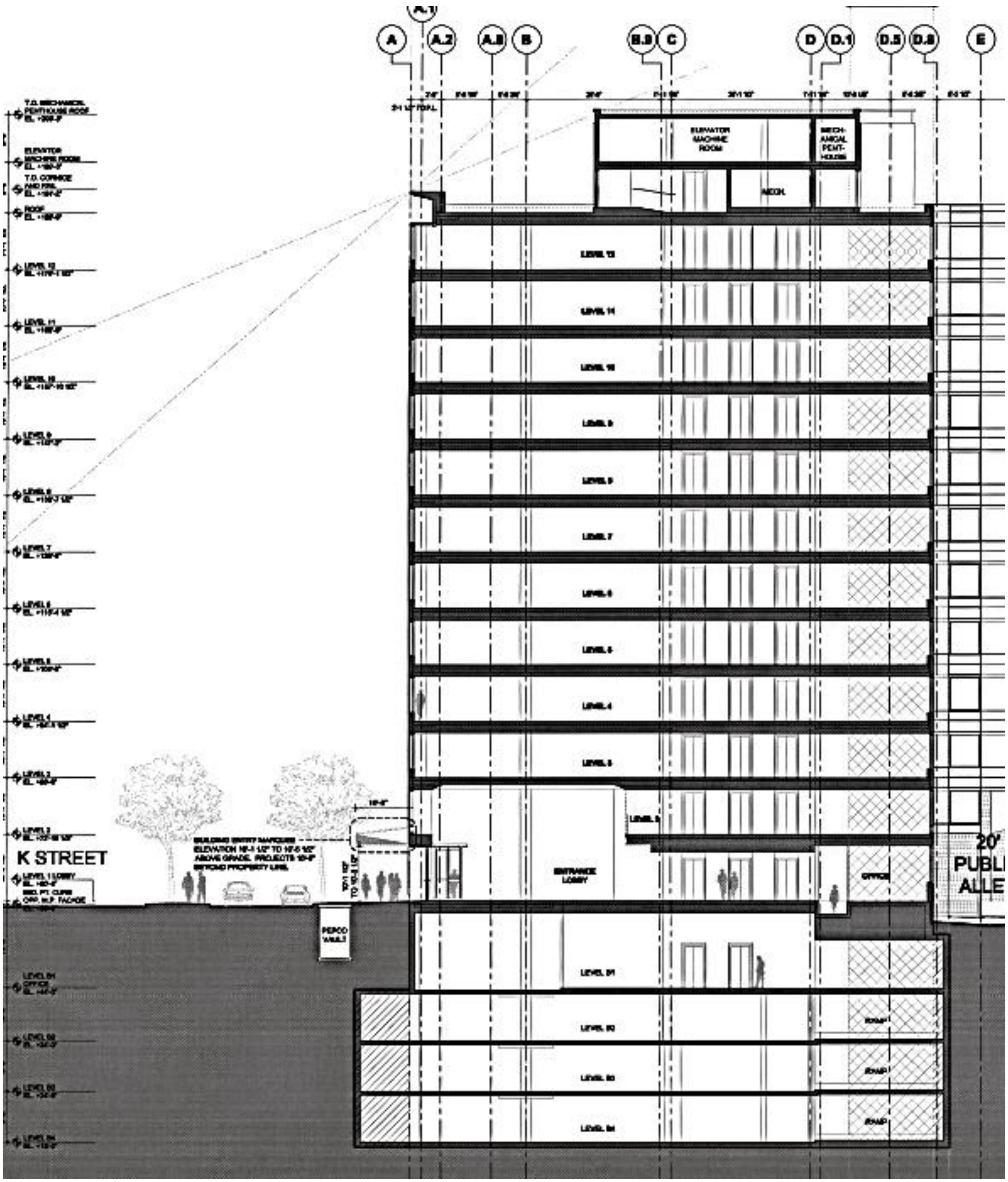
Appendix F: Typical Floor Plans



Typical underground parking plan rotated 90 degrees CW



Typical Floor plan oriented 90 degrees CW



Building Section