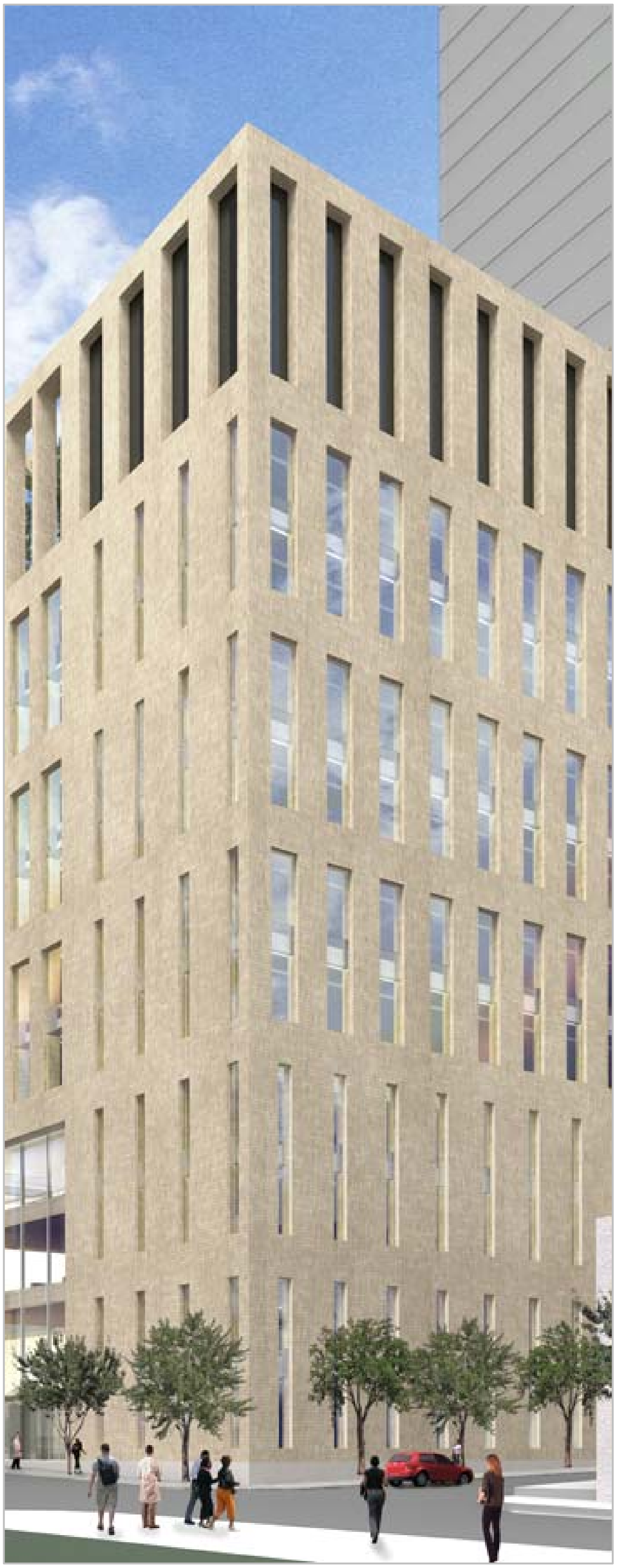
center for science & medicine

new york, ny



Technical Assignment 2

October 29, 2007

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Structural Option
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Executive Summary

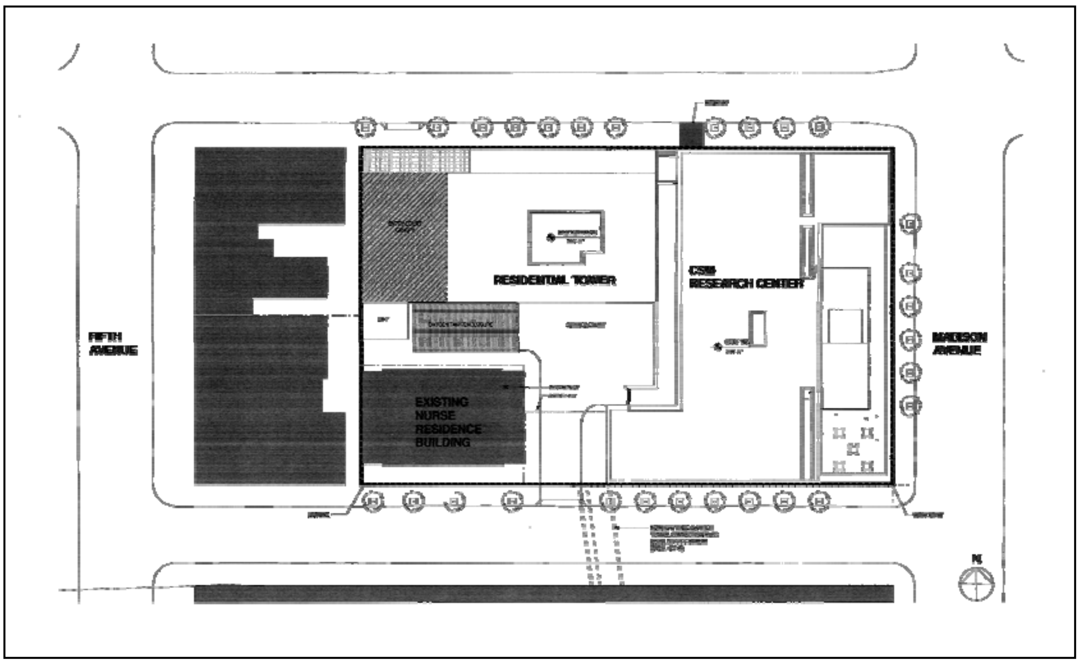
This report is a study of alternative floor framing systems for the Center for Science & Medicine in New York, NY. Five different floor systems were designed and analyzed to be compared for their viability. Comparisons between the systems are based on factors such as cost, fire rating, serviceability, architecture, and ease of construction. Currently, the design for CSM incorporates a composite metal deck floor system on steel beams. Spans are relatively long and heavily loaded, and stringent vibration requirements have been placed on the structure. Although the composite floor system is able to meet these demands, it is worthwhile to investigate other floor framing options. These alternative solutions, each studied in the following pages, include:

1. One-Way Concrete Slab
2. Pre-Cast Double Tees
3. Pre-Cast Hollow Core Slab on Steel
4. Post-Tensioned One-Way Slab

Based on my preliminary analyses, it appears as though the composite metal deck (existing) system and the post-tensioned one-way slab system are the best framing options. Each system has its own advantages. A composite metal deck system is fast and easy to construct (once steel has been delivered), it is capable of long spans and heaving loading, and it is able to control floor vibration. It is a common framing choice among designers today because of its economy and efficiency. Similarly, a PT slab is also able to handle heavy loads and long spans, it has a minimal required floor depth, and it lightens the structure's total weight. Both systems would be good options to investigate further by studying their impacts on vibration, the foundation system, and the lateral system of the building. Such analyses will be conducted in future reports.

Introduction

The Center for Science & Medicine is a research laboratory designed for scientific investigation, discovery, and treatment. Located in New York City's Upper Manhattan, the building is organized and shaped by its architectural program. On the north and south edges of the site, two linear lab bars encompass a core of support spaces. The building's east edge links the inside to the outside with a window-covered, multi-story atrium. Situated within the building are 6 additional floors of wet lab research space, 1½ floors of clinical space, a clinical trial area, and space for research imaging. The building is 11 stories above grade with a typical floor to floor height of 15'-0", giving a total building height of 184'-0." A 40-story residential tower will also rise on the site adjacent to the lab, but the buildings are clearly defined as two separate entities. Below is a site plan showing the CSM research center, the adjacent residential tower, outdoor service areas, and surrounding buildings.



It is important to note that the Center for Science & Medicine, or CSM, is only at the 50% design development phase. Thus, the existing structural design and calculated quantities are not absolute or finalized.

This report will examine four alternate floor systems for the CSM research center. Each analysis includes an evaluation of the system's effectiveness in terms of cost, serviceability, ease of construction, and others. The purpose of this paper is to gain an understanding of potential alternate framing options that are viable for a more detailed study. Thus, all calculations and designs are preliminary and will need to be adjusted and extended if taken to a more comprehensive level.

Existing Structural System

Foundation

The foundation will consist of reinforced concrete spread footings ranging from 4'x4'x2' to 8'x8'x4' (l x w x h) in size, with a concrete compressive strength of $f'_c = 5000$ psi. Maximum footing depth will be 49'-0" below grade, and all footings will bear on sound bedrock (Class 2-65 rock with bearing capacity 40TSF or Class 1-65 rock with bearing capacity 60TSF, according to New York City Building Code). Seven (7) of the total forty-three (43) footings will be designed to support columns from both the research center and the residential tower, as dictated by their location at the CSM / tower interface. Foundation loads vary from 400 to 3200 kips.

Below grade perimeter walls will consist of cast-in-place, reinforced concrete ($f'_c = 5000$ psi) braced by the below-grade floor slabs. The walls will stand 48 ft in height (equivalent to 2 basement levels). These walls are designed to resist lateral loads from soil and surcharge in addition to the vertical loads transferred from perimeter columns above. On the north and south perimeter walls, reinforced concrete pilasters will support perimeter columns above. A continuous grade beam ($f'_c = 5000$ psi) will be constructed under these perimeter basement walls.

The lowest level basement floor will be an 8" concrete slab on grade with a compressive strength of $f'_c = 4000$ psi, typically reinforced with #5 bars@12" each way. At typical columns, additional slab reinforcement will be provided with (4)#4 bars oriented diagonally in the horizontal plane around the column base. At lateral columns located around the building core, the slab will be reinforced with (12)#5 bars oriented diagonally with additional longitudinal bars arranged in a grid pattern around the column base.

Lateral System

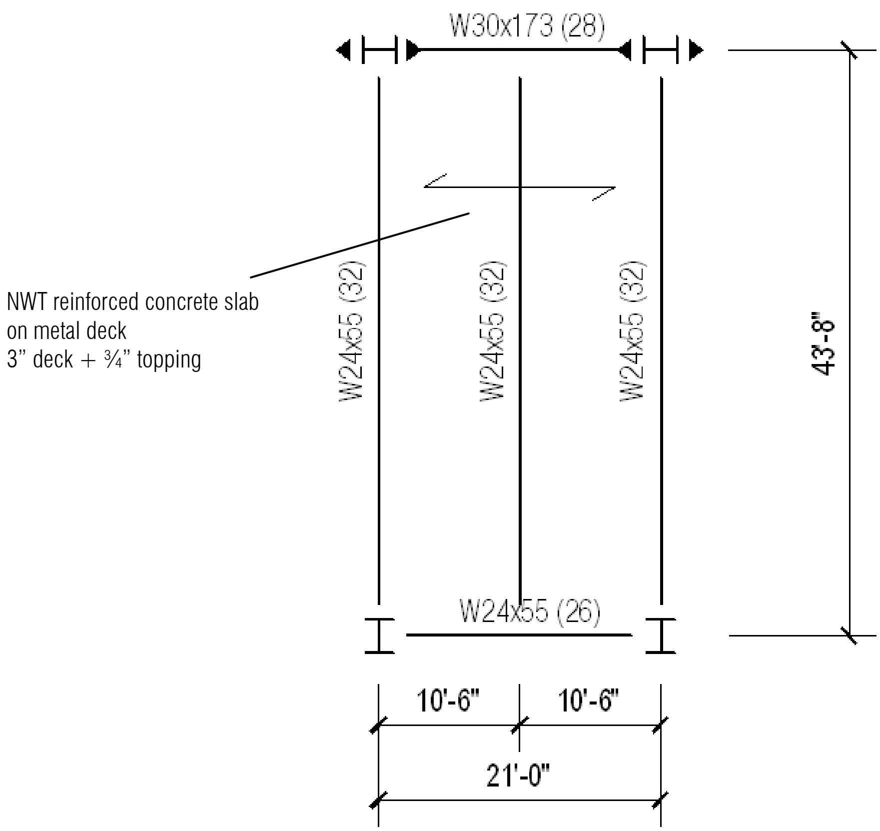
Lateral resistance to wind and seismic loads is provided by a combination of braced and moment resisting steel frames. In the North/South direction, lateral loads are resisted by a system of diagonally-braced frames around the service core area of the building's interior. The core is made up of (6) column bays spaced at approximately 20'x20' and using W14 column sections. Heavy double tee sections used as diagonal braces provide the lateral resistance at the core and vary from WT6x39.5 to WT6x68 in size.

In the East/West direction, lateral loads are taken by a dual system of perimeter beam/column moment frames and the diagonally-braced frame around the service core. Thus, it is assumed that the moment frames in this system are capable of resisting 25% of the design lateral forces. These moment frames have been designed to use W14 or W24 column sections spaced approximately 21'-0" on center and W30 wide flange beams. The frames first occur on the third level and then alternate levels up through the building's roof (a total of five floors with moment frames).

Floor Framing System

CSM's existing floor system uses composite metal deck. The floor slabs typically consist of 3" metal deck with 4 ¾" normal-weight concrete topping, giving a total slab depth of 7 ¾". Thicker, normal-weight concrete slabs will be provided in spaces such as mechanical floors to meet acoustic and vibration criteria. These thickened slabs will be designed with 3" metal deck and 8" NWT concrete topping with reinforcement, giving a total slab depth of 11". Full composite action is created by 6" long, ¾" diameter shear studs, and concrete compressive strength is to be $f'_c = 4000$ psi. The composite metal deck is supported by wide flange steel beams ranging from W12x14 to W36x150 in size and spaced approximately 10'-6" on center.

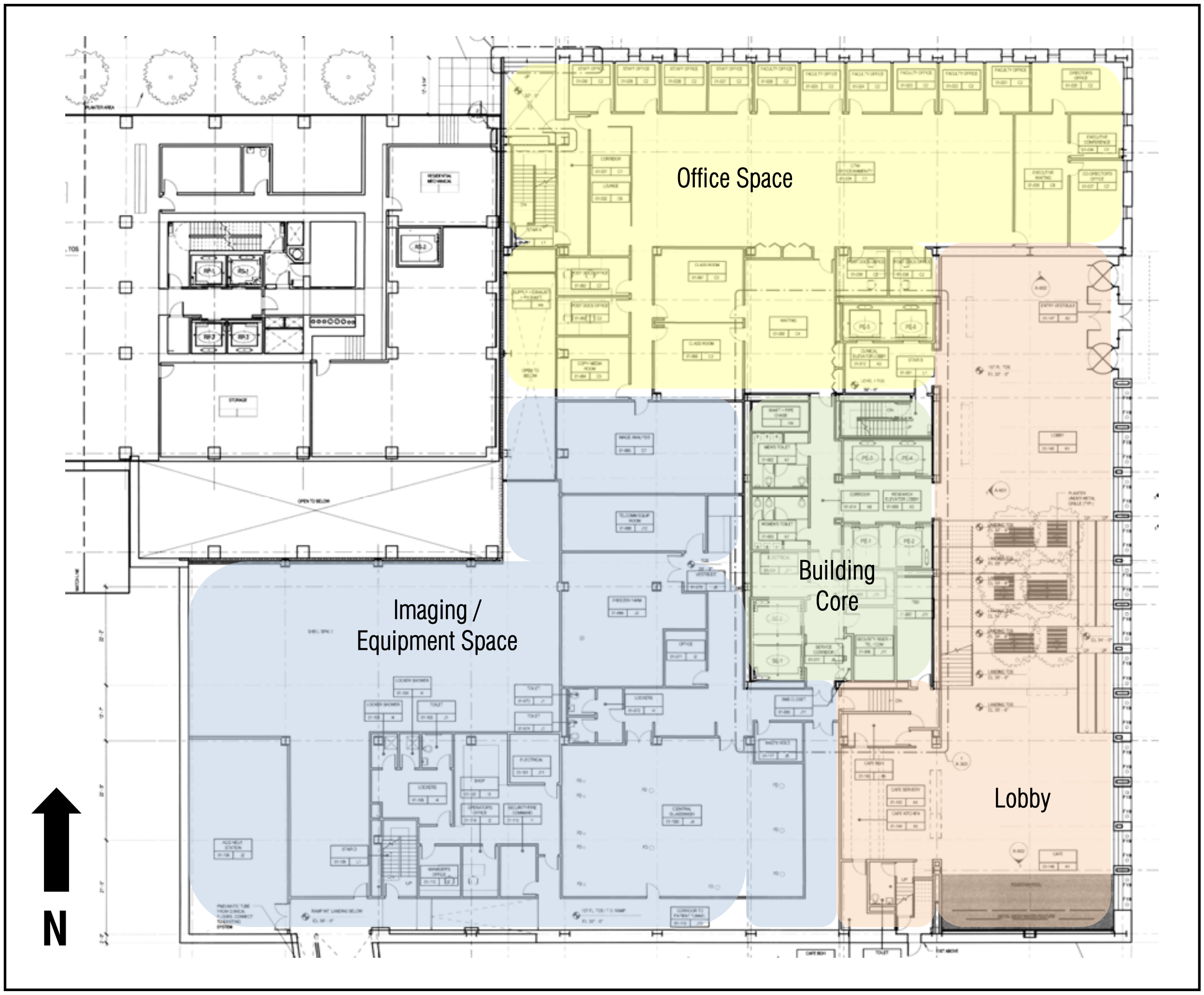
There are two typical bay sizes used throughout the building, 21'-0"x 21'-0" and 43'-0" x 21'-0." For this study, I have chosen the larger bay size to analyze in order to obtain results that can be applied throughout the entire structure. This particular bay, shown below, is located on the North end of the building and occurs on typical lab floors (level 3 and levels 5-10). It is designed for 100 psf live load and 25 psf superimposed dead load (see table on page 9). Also, the lab areas must meet the strict serviceability requirement of a 2000 micro-inch/sec vibration velocity, which is another reason why I have chosen to study this typical bay.



Typical Floor Plans

Architectural

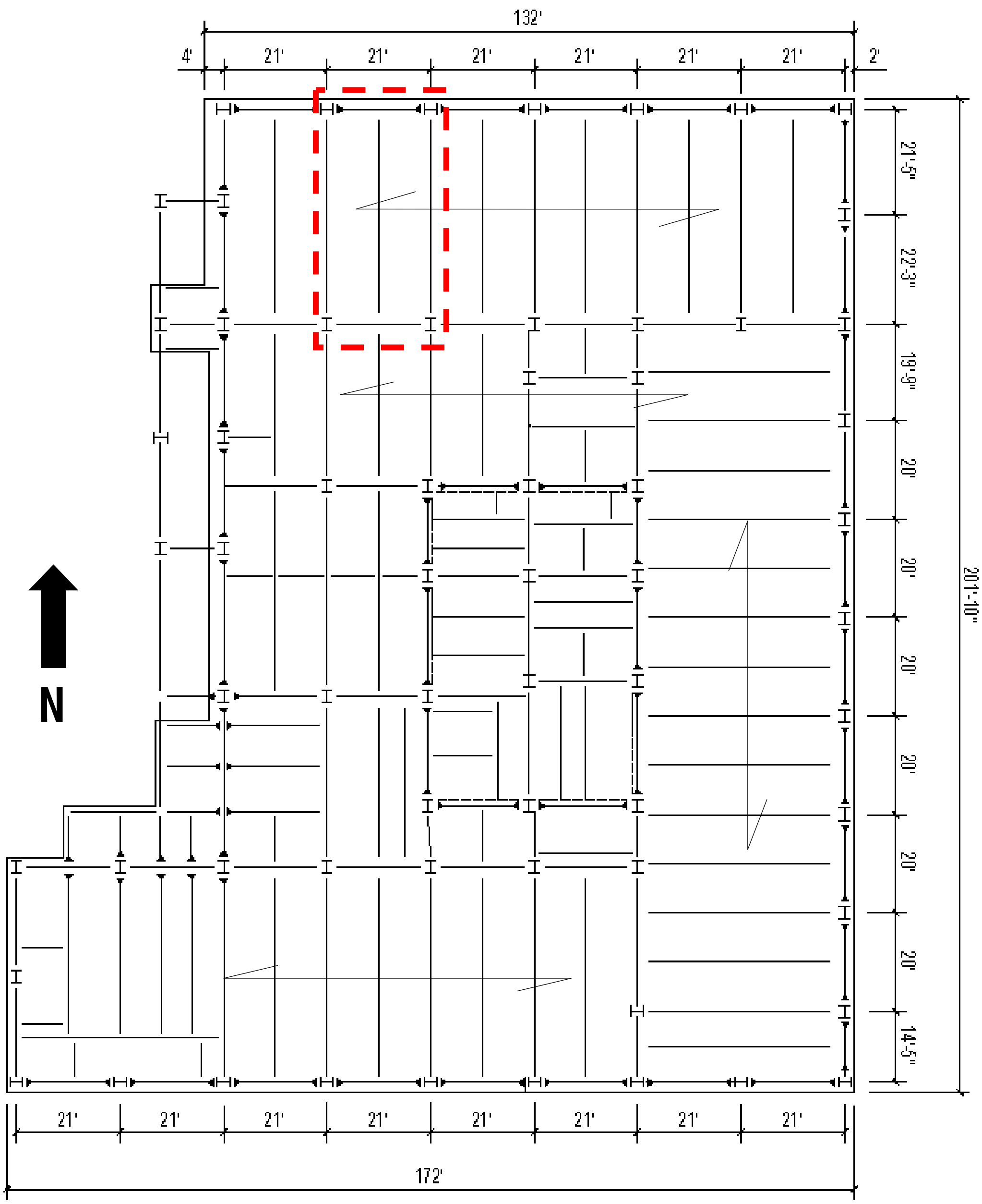
Below is the architectural floor plan for the first level of CSM. Colored zones indicate the functions of each area. The building footprint stays basically the same with increasing height, except for a slight decrease in area on the southwest corner beginning on the 3rd floor.



Level 1, Architectural Plan

Framing

Typical floor framing is shown in the figure below (laboratory floor). Composite metal deck spans the floor in the east-west direction in most areas and in the north-south direction above the atrium. Perimeter columns are spaced approximately 20'-0" - 22'-3" on center, and the longest span is 43'-8" (located on the south side of the building). The typical bay chosen for study in this report, as discussed on page 5, is noted with a dashed line.



Level 5, Floor Framing Plan

Code & Design Requirements

Applicable design standards

International Building Code 2006

ACI 318-05 (Reinforced Concrete Design)

AISC LRFD-2005, 13th Edition (Structural Steel)

ASCE 7-05

Deflection Criteria

Floor to Floor Deflection

Typical live load deflection	L/360
Typical total deflection	L/240
Typical exterior spandrel deflection	½"

Vibration Criteria

Imaging rooms / laboratories	2000 Micro inches / sec
Patient rooms	4000 Micro inches / sec
Offices / seminar rooms	8000 Micro inches / sec

Gravity Loads

Below is a table summarizing the load values of the structural designer and of IBC 2006 (which references ASCE 7-05).

Floor / Description	Superimposed Dead Load	Design Live Load	IBC Live Load	Vibration Velocity
SC1 & SC 2				
· Vivarium	30 psf	50 psf	-	2000 μin/s
· Stair	5 psf	100 psf	100 psf	-
SC1 & SC2 Interstitial				
· Mechanical Service	10 psf	50 psf	-	-
· Stair	5 psf	100 psf	100 psf	-
Level 1				
· Lobbies, Corridors	110 psf	100 psf	100 psf	-
· Office	30 psf	50 psf	50 psf	8000 μin/s
· Glass Wash	10 psf	125 psf	-	2000 μin/s
· Stair	5 psf	100 psf	100 psf	-
Level 2				
· Wet Lab	25 psf	100 psf	-	2000 μin/s
· Loading Dock	75 psf	250 psf	250 psf	-
· Auditorium	40 psf	60 psf	60 psf	-
· Stair	5 psf	100 psf	100 psf	-
Level 3				
· Wet Lab	25 psf	100 psf	-	2000 μin/s
· Stair	5 psf	100 psf	100 psf	-
Level 4				
· Lobbies, Corridors	110 psf	100 psf	100 psf	-
· Office	30 psf	50 psf	50 psf	8000 μin/s
· Stair	5 psf	100 psf	100 psf	-
Levels 5 - 10				
· Office	30 psf	50 psf	50 psf	8000 μin/s
· Wet Lab	25 psf	100 psf	-	2000 μin/s
· Stair	5 psf	100 psf	100 psf	-
Level 11				
· Roof Terrace	235 psf	100 psf	100 psf	-
· Mechanical	80 psf	125 psf	-	-
· Stair	5 psf	100 psf	100 psf	-
Roof				
· Green Roof	60 psf	100 psf	100 psf	-
· Snow Load	-	30 psf	22 psf (see calcs)	-
Superimposed Loads				
· Partitions	10-20 psf	-	-	-
· CMEP	10 psf	-	-	-
· Finishes / Screed	5-15 psf	-	-	-
· Roofing Membrane / Insul.	10 psf	-	-	-

Alternate Framing Systems

System 1: Existing Composite Metal Deck

Loading:

Live load = 100 psf
Dead load (superimposed) = 25 psf

Material Properties:

f'c = 4,000 psi
fy = 50 ksi (beams / girders)
= 60 ksi (shear studs)

3" metal deck, 16 gage
4.75" normal weight concrete topping
¾" diameter, 6" long shear studs

Special Requirements:

2-hour fire rating
2,000 μin/sec vibration limit

Framing Layout:

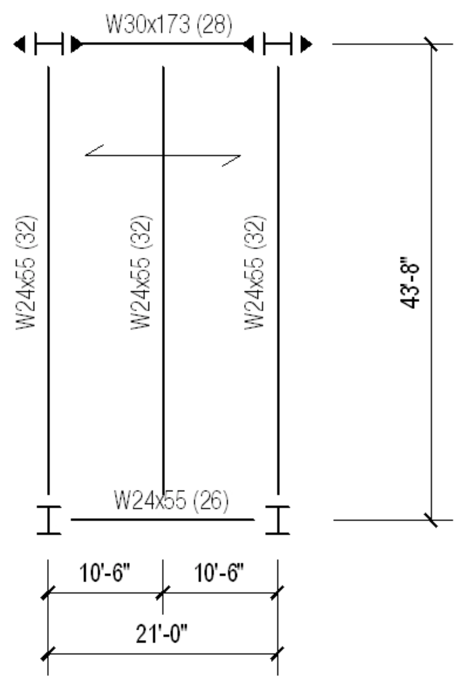


Figure 4: Existing Framing for Typical Bay

System Evaluation:

Structural:

This system of composite metal deck presents itself to be an effective framing option for CSM. The ability of the steel and concrete to work together allows for the heavy live load and long spans. Although the 2-hour fire rating of this system is met by the 7.75" total slab thickness, steel members must also receive spray-on fireproofing to meet the code. Also, steel sections are on the heavier side due to the vibration requirements that must be met. The approximate total floor depth is about 32," which is on the larger side.

Construction:

Composite concrete is generally a cost effective means of construction. Forms are not required, which eases the process. The floor slab does not need to be cut in many areas based on this design, minimizing time between concrete pours. Erection of the steel is also a quick and efficient process, and it is able to be sequenced strategically as a part of the project's construction schedule.

Architectural

The composite system allows for long, rectangular bays. This kind of column grid is very desirable for laboratory layout. However, since the member depth is required to be relatively deep, less floor-to-ceiling height is able to be achieved at each level, which is an undesirable feature of this system.

Conclusion

A composite metal deck floor framing system is a viable option for CSM's structure.

Positive	Negative
+ Easy to construct	- Heavy steel sections required
+ 2 hour fire rating (with spray-on fireproofing)	- Thick total floor depth (2'-8")
+ Cost effective	
+ Meets vibration requirements	
+ Fast erection time	
+ Carries large live loads	
+ Large, rectangular column bays allow for lab layout	

System 2: One Way Slab

Loading:

Live load = 100 psf
Dead load (superimposed) = 25 psf

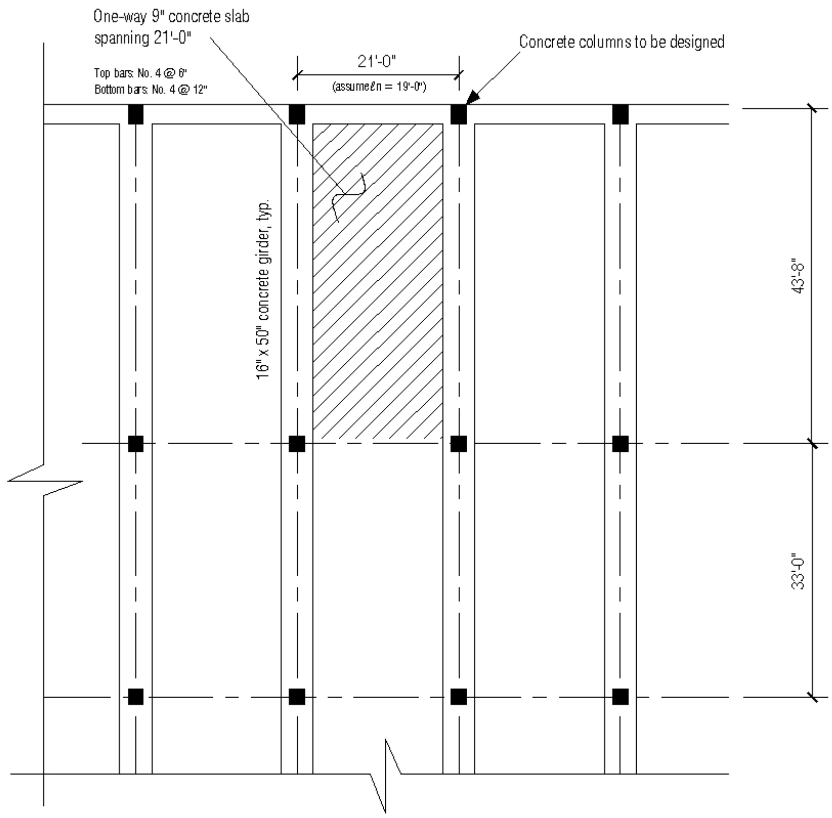
Material Properties:

f'c = 4,000 psi
fy = 50 ksi (beams / girders)
= 60 ksi (reinforcement)

Special Requirements:

2-hour fire rating
2,000 μin/sec vibration limit

Framing Layout:



System Evaluation:

Structural:

This system of a one-way concrete slab and wide, shallow beams appears to be a somewhat effective framing option for CSM. A 9" concrete slab is required for a 21'-0" span, plus another 16" for the depth of beams running in the north-south direction, giving an overall floor depth of 25 inches. The system is able to carry the heavy live loads and remains consistent with the original large column spacing. The 2-hour fire rating requirement is met by the 9" slab thickness, and no additional fireproofing is required since there are no structural steel members. Although concrete structures are typically able to effectively minimize vibration, there was no in-depth vibration study performed for this report. If this system were to be analyzed further, vibration requirements would need to be checked and all columns would need to be redesigned as concrete.

Construction:

Cast-in-place concrete presents a longer time schedule for construction. Instead of being able to pour a floor and proceed to the next soon after, workers must wait for the concrete to cure. Only after the concrete has reached a certain strength can workers strip the forms and progress to upper levels. The regularity of the floor plan allows the reuse of concrete forms from floor to floor.

Architectural

The concrete one-way slab system allows for long, rectangular bays in accordance with the original grid layout, which is desirable for the laboratory function of this space. Also, the overall floor depth is slightly less than the composite system, which is desirable from an architectural standpoint.

Conclusion

A one-way slab floor framing system could be a viable option for CSM's structure and is worth further investigation.

Positive	Negative
+ Thinner floor depth	- Heavy steel sections required
+ 2 hour fire rating (no spray-on fireproofing)	- Thick total floor depth (2'-8")
+ Will likely meet vibration requirements	- Slowed erection time
+ Carries large live loads	- More expensive to construct
+ Large, rectangular column bays allow for lab layout	

System 3: Precast Double Tees

Loading:

Live load = 100 psf
Dead load (superimposed) = 25 psf

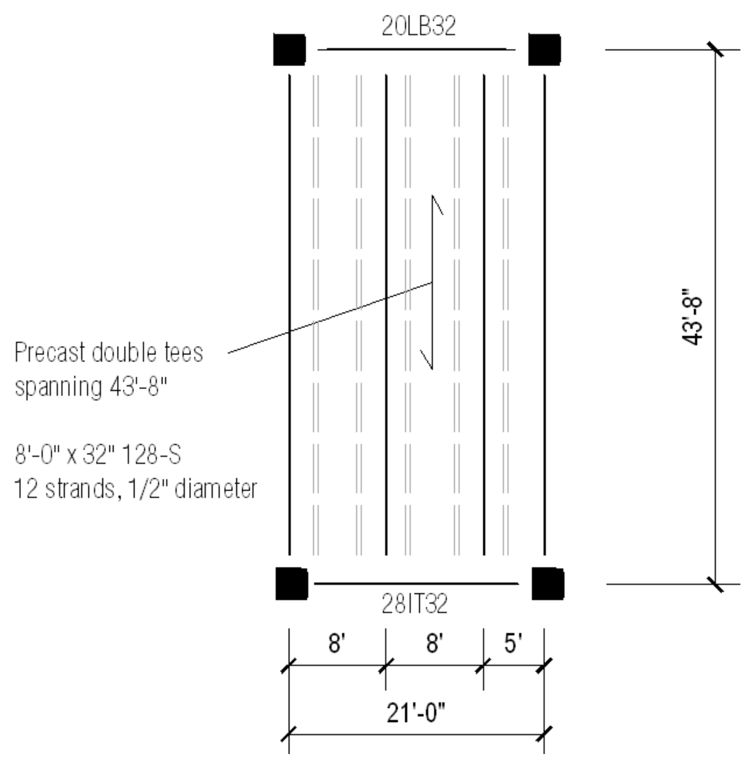
Material Properties:

f'c = 5,000 psi
fpu = 270,000 psi (reinforcement)

Special Requirements:

2-hour fire rating
2,000 μin/sec vibration limit

Framing Layout:



System Evaluation:

Structural:

The most obvious disadvantage to this system is limited variety of double tee shapes in terms of dimensions. My design incorporates 8'-0" wide double tees, but they do not fit perfectly into the existing column grid. To solve this problem, either specially-fabricated double tees would need to be ordered to fit into 21'-0" bays, or the column grid would need to be re-configured. Also, the slab is only 4" thick between joists, so spray-on fireproofing would be required. Overall floor depth is 32," including the 2" topping on the double tees, which is relatively large. Aside from these disadvantages, the double tees are efficient in carrying heavy loads on long spans. Deflection and vibration would likely be kept to a minimum, since the system is entirely concrete, but a more detailed study would be required to confirm this. Also, concrete columns would need to be redesigned as well.

Construction:

Pre-cast construction is a much faster process than cast-in-place, as all of the elements are fabricated in a shop. However, pre-cast concrete construction requires a longer lead time for ordering these pre-fabricated members (sometimes up to five months). The front-ended schedule impacts may or may not prove to be a better option than a cast-in-place system.

Architectural

Overall floor depth is actually greater than the existing composite system. Also, the restrictive dimensions of the double tees prevent an even fit into a 21'-0" bay. This issue would require a reorganization of the column grid, which would be undesirable from an architectural viewpoint.

Conclusion

A pre-cast double tee system is probably not the most feasible or economical option for CSM's floor framing system.

Positive	Negative
+ Fast erection time	- Long lead time
+ 2 hour fire rating (no spray-on fireproofing)	- Thick total floor depth (32")
+ Will likely meet vibration requirements	- Possible reconfiguration of column grid
+ Carries large live loads	- Special fabrication of unique double tee sizes
+ Large, rectangular column bays allow for lab layout	

System 4: Precast Hollow Core Plank

Loading:

Live load = 100 psf
Dead load (superimposed) = 25 psf

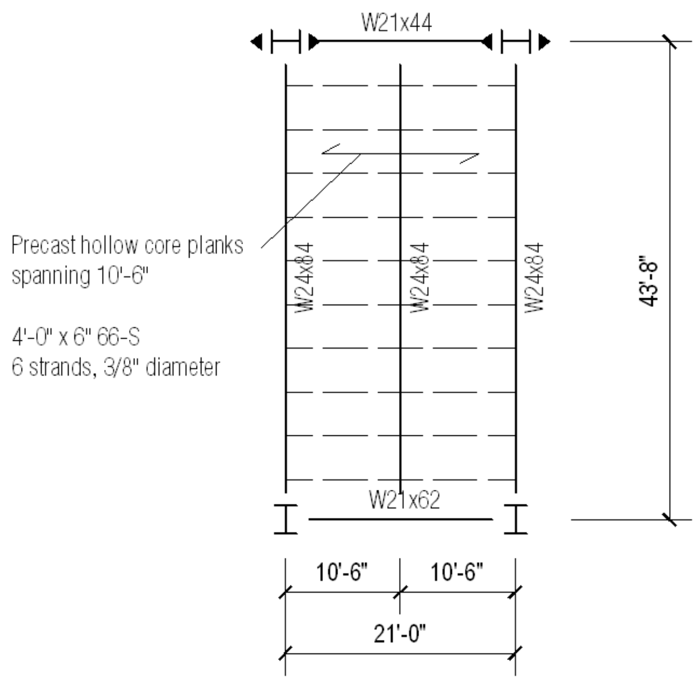
Material Properties:

f'c = 5,000 psi
fpu = 270,000 psi (reinforcement)

Special Requirements:

2-hour fire rating
2,000 μin/sec vibration limit

Framing Layout:



System Evaluation:

Structural:

A system of hollow core plank allows large loads, long spans, and desirable fire rating. The pre-fabricated elements also fit well into the existing column grid, just 4" short on one end of the 43'-8" span. The 8" slab itself meets the 2-hour fire rating requirement, but additional spray-on fireproofing is required on the steel beams and girders. Total floor depth of this system is about 32." Since this system is both steel and concrete, it is difficult to predict vibration effects. Thus, a more detailed analysis is required.

Construction:

Like the double tees, hollow core planks will need significant lead time to be pre-ordered and shipped to the construction site. However, once all materials have been gathered, erection of the steel and installation of the slab should be a fast process. The front-ended schedule impacts of the hollow core system may or may not prove to be a better option than a cast-in-place system.

Architectural

Overall floor depth is fairly significant, which is architecturally undesirable. Also, like the pre-cast double tees, the restrictive dimensions of the planks prevent an even fit into a 43.667" bay length. This issue would require a reorganization of the column grid, which would be undesirable from an architectural viewpoint as well, or a special ordering of unique plank sizes.

Conclusion

A pre-cast hollow core system is probably not the most feasible or economical option for CSM's floor framing system.

Positive	Negative
+ Fast erection time	- Long lead time
+ 2 hour fire rating (extra spray-on fireproofing)	- Thick total floor depth (32")
+ Carries large live loads	- Possible reconfiguration of column grid
+ Large, rectangular bays allow for lab layout	- Unknown vibration effects
	- Possible expense in ordering unique plank sizes

System 5: Post-Tensioned Concrete Slab

Loading:

Live load = 100 psf
Dead load (superimposed) = 25 psf

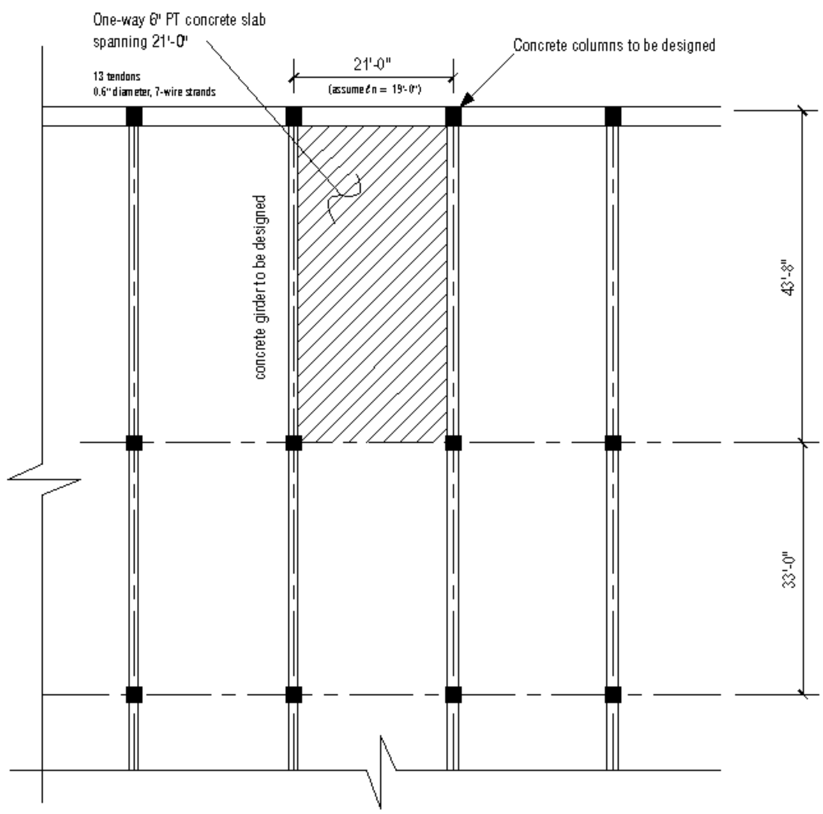
Material Properties:

f'c = 5,000 psi
fpu = 270,000 psi (reinforcement)

Special Requirements:

2-hour fire rating
2,000 μin/sec vibration limit

Framing Layout:



System Evaluation:

A very basic, preliminary design was done for a PT system. Due to my limited knowledge of this subject, results may not be as accurate as they could be. Thus, this system will be studied at a later time when I have been more educated on this design method.

Based on my preliminary analysis, it seems that a PT system would be worthy of further investigation. Total floor thickness is only 20" (including drop panels around columns). Fire rating requirements are met by the 6" concrete slab, and no further treatment is necessary. The system is able to handle the heavy live load and large column spacing, and the increased strength of the floor due to post-tensioning allows the beam spacing to increase so that no infill beams are required between columns. Also, the pre-compression within the slab section may help in meeting the strict vibration criteria.

The laying of tendons during the construction process could potentially slow down the process. Aside from tendons though, the construction of the remainder of the slab is relatively fast. Additionally, because of the large jacking forces applied to the slab after 75% curing, safety on the jobsite is of utmost importance. It might be worthwhile to have an inspection agency onsite during post-tensioning to monitor the hazardous environment.

Conclusion

Based on this analysis, a post-tensioned slab system is a feasible option for CSM's floor framing system and should be investigated further.

Positive	Negative
+ Medium-length erection time	- Formwork required
+ Thinner floor depth (20")	- Laying of tendons is labor intensive
+ 2 hour fire rating (extra spray-on fireproofing)	- Extra safety procedures required on the job site
+ Carries large live loads	
+ Large, rectangular bays allow for lab layout	
+ Vibration effects likely subdued by PT slab	

Comparison of Systems

	System 1 Composite Steel (existing)	System 2 One-Way Slab with Wide, Shallow Beams	System 3 Pre-Cast Double Tees	System 4 Pre-Cast Hollow Core Slab on Steel	System 5 PT One-Way Slab & Beams
Relative Cost	Medium	Medium	Medium	High	Medium
Structure Depth	32"	25"	34"	29"	20"
Structure Weight	78 psf	115 psf	63 psf	76 psf	75 psf
Fireproofing	SOFP required	No additional FP required	SOFP required	SOFP required	No additional FP required
Vibration	Satisfactory	(Additional study required)			
Lead Time	Long	Short	Long	Long	Short
Effect on Column Grid	None	None	Possible rearrangement	Possible Rearrangement	None
Construction Difficulty	Medium	Medium-Hard	Easy	Easy	Medium-Hard
Formwork	No	Yes	No	No	Yes
Fire Rating	Satisfied	Satisfied	Satisfied	Satisfied	Satisfied
Overall Feasibility	(existing system)	Possible for investigation	Few advantages over existing system	Few advantages over existing system	Should be investigated

Conclusion

The preliminary designs conducted in this report were aimed to generate an understanding of basic floor framing systems and how they might work in the structural system of the CSM research center. The existing framing system is composite metal deck, and the four alternate systems studied were: one-way concrete slab with wide, shallow beams, pre-cast double tees, pre-cast hollow core slab on steel, and a post-tensioned one-way slab.

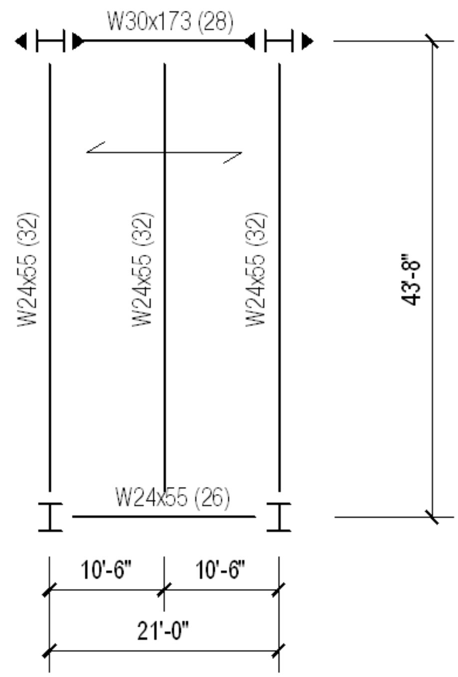
Each framing system was designed using basic, preliminary methods and then examined for its feasibility. While none of the systems should be altogether eliminated, some are better than others. It appears that the existing composite system and the post-tensioned system hold the most potential for effective framing schemes. A PT system will lighten the floor load, decrease the floor depth, and still be able to carry heavy loads over long spans. A composite system is both economical and efficient, easy to construct, and makes good use of the tensile properties of steel in addition to the compressive properties of concrete.

Further investigation of both systems will be conducted. In these studies, vibration will be examined in depth, and any ramifications on the building's lateral and foundation system will be accounted for as well. After such investigations, final conclusions can be drawn.

Appendix

System 1: Composite Metal Deck (existing)

Referenced: ACI 318-05



Loading:
Live load = 100 psf
Superimposed dead load = 25 psf
$w_u = 1.2(25) + 1.6(100) = 190$ psf

Materials:
$f'_c = 4{,}000$ psi
$f_y = 50$ ksi (beams/girders)
$f_y = 60$ ksi (reinforcement)

3" metal deck, 16 gage
4.75" NW concrete topping
¾" diameter, 6" long shear studs

W30x173, $A_s = 51.0$ in^2
W24x55, $A_s = 16.2$ in^2

Special Requirements:
2-hour fire rating
2000 μin/sec vibration limit

Check composite deck:

$w_u = 1.2(25) + 1.6(100) = 190$ psf

From United Steel Deck Catalogue,

Max unshored span allowed = 12.04' for 2 span condition > 10'-6" OK
Max uniform live load for 10'-6" span = 400 psf > 1.6(100) = 160 psf OK
Fire rating: 2 hours OK

Check composite beam:

$w_u = (190 \text{ psf})(10.5')/1000 = 1.995$ klf → without load factor, w = 1.31 klf

$M_u = (1.995 \text{ klf})(43.667^2 \text{ ft})/8 = 475.5$ ft-k

$V_u = (1.995 \text{ klf})(43.667')/2 = 43.6$ kips

$\sum Q_n = F_y A_s = 50(16.2) = 810$ kips

$b_{eff} = ½(43.667') = 21.8'$
OR
$= 10.5'$ → controls.

$a_{required} = \sum Q_n/(0.85f'_c b) = 810/(0.85)(4)(10.5 \times 12) = 1.89"$

$Y2 = 7.75 - a/2 = 6.8"$

From Table 3-19,

$\sum Q_n = 810 \text{ k} \geq 810 \text{ k}$ OK

$\phi Mn = 1128 \text{ ft-k} \geq Mu = 475.5 \text{ ft-k}$

$\phi Vn = 251 \text{ k} > Vu = 43.6$ OK

$I_{LB} = 4150$ (for $Y_2 = 6.8"$)

$\Delta_{max} = 5(1.31)(43.667)^4(1728)/[(384)(29000)(4150) = 0.89"$

$\Delta_{D+L} \leq \ell/240 = (43.667 \times 12)/240 = 2.18"$

$\Delta_{max} = 0.89" < \Delta_{D+L} = 2.18"$ OK

Check girder, W24x55:

$P_u = (11.995 \text{ ksf})(43.667')/2 = 43.6 \text{ k}$

Without load factors, $P_u = 28.6 \text{ k}$

$M_u = (43.6 \text{ k})(21')/4 = 228.9 \text{ ft-k}$

$V_u = 43.6 \text{ k}/2 = 21.8 \text{ kips}$

Without load factors, $V_u = 14.3 \text{ k}$

ϕMp (W24x55) $= 503 \text{ ft-k} > M_u = 228.9 \text{ ft-k}$

ϕVn (W24x55) $= 251 \text{ k} > V_u = 21.8 \text{ k}$

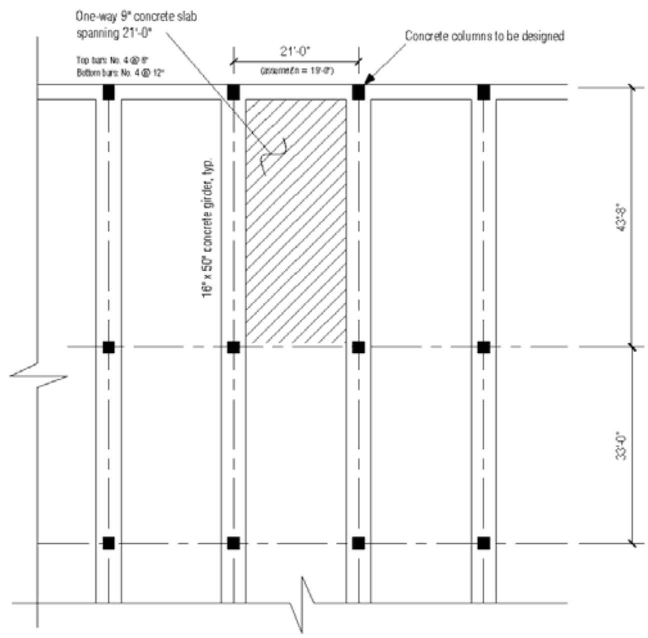
$\Delta_{max} = 28.6(21)^3(1728)/[(48)(29000)(1350) = 0.24"$

$\Delta_{D+L} \leq \ell/240 = (21 \times 12)/240 = 1.05"$

$\Delta_{max} = 0.24" < \Delta_{D+L} = 1.05"$ OK

System 2: One-Way Concrete Slab

Referenced: ACI 318-05



Loading: Live load = 100 psf
Superimposed dead load = 25 psf
$w_u = 1.2(25) + 1.6(100) = 190$ psf

Materials: $f'_c = 4{,}000$ psi
$f_y = 60$ ksi (reinforcement)

Special Requirements: 2-hour fire rating
2000 μin/sec vibration limit

One-Way Slab Design:

Due to my limited knowledge at this point and a restriction on time, there are no vibration checks in the calculations below. Thus, members are designed based on flexure and deflection only, giving smaller designs than what will likely be required. Vibration analysis will be considered at a later date.

Minimum slab thickness

Assuming columns are 24"x24" concrete,
$\ell_n = 21' - (2 \times 12)/12 = 19'\text{-}0"$

From ACI 318-05, Table 9.5(a), $h \geq \ell_n/28$
$h \geq (19' \times 12)/28 = 9.0"$ → **9" slab** → meets 2-hour fire rating (h = 5")

Slab Contribution

Slab weight = 150 pcf x 9"/12 = 112.5 psf
$w_{slab} = 1.2(112.5) = 135$ psf

Total Load
$w_u = 190 \text{ psf} + 135 \text{ psf} = 325$ psf

Moment Values using ACI Coefficients

At both interior supports: $-M = (1/10)w_u\ell_n^2 = (1/10)(0.325)(19)^2 = 11.7$ ft-k
At midspan: $+M = (1/16)\, w_u\ell_n^2 = (1/16)(0.325)(19)^2 = 7.3$ ft-k

Required Reinforcement

$\rho_{max} = 0.85(0.85)(4/60)[0.003/(0.003+0.004)] = 0.021$

Effective depth:
$d = 9" - 1" = 8"$ controls
OR
$d^2 = Mu/(\phi\rho f_y b(1\text{-}0.59(\rho f_y/f'_c) = (10.6 \times 12)/[(0.9)(0.021)(60)(12)(1\text{-}0.59(0.021)(60/4))$
$d = 3.4"$

Area of steel required per foot in top of slab:

Assume a = 1
$A_s = (10.6 \times 12)/[(0.9)(60)(8-1/2) = 0.314\ in^2$

Check a = 1:
$a = A_s f_y/(0.85f'_c b) = (0.314)(60)/[(0.85)(4)(12)] = 0.46"$

For a = 0.46",
$A_s = (10.6 \times 12)/[(0.9)(60)(8-0.46/2) = 0.303\ in^2$

$A_s = 0.303\ in^2$ per foot
Use No. 4 @ 6"

Area of steel required per foot at midspan:
For a = 0.46",
$A_s = (6.7 \times 12)/[(0.9)(60)(8-0.46/2) = 0.19\ in^2$

Minimum As for control of shrinkage and cracking:
$A_s = 0.0018(12)(9) = 0.194$ controls.

$A_s = 0.194\ in^2$ per foot
Use No. 4 @ 12"

Check Shear: Shear Values using ACI Coefficients
Shear in end members at first interior support:
$Vu = 1.15 w_u \ell_n/2 = 1.15(0.325)(19)/2 =$ **3.6 kips**

Shear at all other supports:
$Vu = w_u \ell_n/2 = (0.325)(19)/2 =$ **3.1 kips**

Allowable Shear:
$\phi Vn = 0.75(2)\sqrt{(f'c)}bd = 0.75(2)\sqrt{(4000)}(12)(8)/1000 = 9.12$ kips

$Vu = 3.6$ kips $< \phi Vn = 9.12$ kips OK

Girder Design:

Loading
Dead load = 112.5 psf *(slab)* + 25 psf *(superimposed)* = 137.5 psf
Live load = 100 psf, reduce
$A_I = 42' \times 43.667' = 1834$ sq. ft.
Reduction factor $= 0.25 + 15/(\sqrt{1834}) = 0.60 > 0.4$, OK
LL = 0.6(100) = 60 psf
$w_u = 1.2(137.5) + 1.6(60) = 261$ psf

Maximum Moment
M_{max} @ ends $= w\ell^2/12 = (261 \times 21')(43.667)^2/12 =$ **870.8 ft-k**
M_{max} @ midspan $= w\ell^2/24 = (261 \times 21')(43.667)^2/24 = 435.4$ ft-k

Girder Size

$\rho = 0.85(0.85)(4/60)(0.003/(0.008) = 0.0181$

$Mu = \phi Mn$

$870.8(12) = 0.9(0.0181)(60)(bd^2)[(1-0.59(0.0181)(60)/4]$

$bd^2 = 12730.4 \text{ in}^3$

For a shallow beam, try b = 3d

d = 16"

b = 50"

$\phi M_n = 875.6$ ft-k $> M_u = 870.8$ ft-k, OK

Note: Columns will also need to be redesigned if the analysis of this system is pursued further.

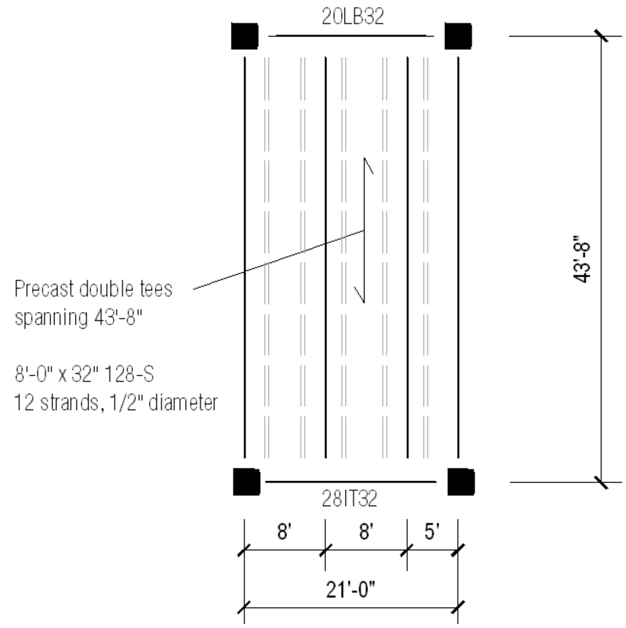
System 3: Precast Double Tees

Referenced: PCI Design Handbook, 6th Edition

Loading:
Live load = 100 psf
Dead load = 25 psf
wu = 1.2(25) + 1.6(100) = 190 psf

Materials:
f'c = 5,000 psi
fpu = 270,000 psi (reinforcement)

Special Requirements:
2-hour fire rating
2000 μin/sec vibration limit



Due to my limited knowledge at this point and a restriction on time, there are no vibration or deflection checks in the calculations below. Thus, members are designed based on flexure and deflection only, giving smaller designs than what will likely be required. Vibration analysis and deflection checks will be considered at a later date.

Joist Slab Design:

From PCI handbook, select Double Tee 128-S with 2" topping:

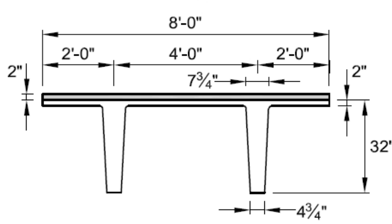
Strand Pattern Designation

1 8 8 - D 1
No. of strand (18)
S = straight D = depressed
No. of depression points
Diameter of strand in 16ths

Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

Key
192 – Safe superimposed service load, psf
1.1 – Estimated camber at erection, in.
1.4 – Estimated long-time camber, in.

DOUBLE TEE
8'-0" x 32"
Normal Weight Concrete



f'_c = 5,000 psi
f_{pu} = 270,000 psi

Section Properties

		Untopped		Topped	
A	=	567	in.^{2}	–	
I	=	55,464	in.^{4}	71,886	in.^{4}
y_b	=	21.21	in.	23.66	in.
y_t	=	10.79	in.	10.34	in.
S_b	=	2,615	in.^{3}	3,038	in.^{3}
S_t	=	5,140	in.^{3}	6,952	in.^{3}
wt	=	591	plf	791	plf
DL	=	74	psf	99	psf
V/S	=	1.79	in.		

8DT32 + 2

Table of safe superimposed service load (psf) and cambers (in.) **2 in. Normal Weight Topping**

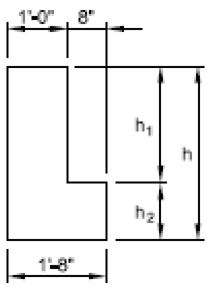
Strand Pattern	y_s(end) in. y_s(center) in.	Span, ft 42	44	46	48	50	52	54	56	58	60	62	64	66	68	70	72	74	76	78	80	82	84	86	88	90	92	94
128-S	7.00	270	240	214	190	170	152	136	121	108	97	86	76	67	56	47	38	30										
	7.00	1.0	1.0	1.1	1.1	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.1	1.1	1.0	0.9	0.7	0.6										
		1.0	1.1	1.1	1.1	1.0	1.0	0.9	0.9	0.8	0.6	0.4	0.2	0.0	–0.3	–0.7	–1.1	–1.5										
148-S	7.00		269	259	232	208	187	168	152	137	123	111	100	88	77	66	57	47	39	31								
	7.00		1.3	1.4	1.4	1.5	1.5	1.6	1.6	1.7	1.7	1.7	1.7	1.6	1.6	1.5	1.4	1.3	1.2	1.0								
			1.4	1.4	1.4	1.4	1.4	1.4	1.3	1.3	1.2	1.0	0.9	0.7	0.5	0.2	–0.2	–0.6	–1.0	–1.5								
168-S	8.00			288	259	233	210	190	171	155	138	124	110	98	87	76	67	57	49	40	33	25						
	8.00			1.4	1.5	1.6	1.7	1.7	1.8	1.8	1.8	1.9	1.9	1.9	1.8	1.8	1.7	1.6	1.5	1.3	1.2	1.0						
				1.5	1.6	1.6	1.6	1.6	1.5	1.5	1.4	1.3	1.1	1.0	0.7	0.5	0.2	–0.2	–0.6	–1.1	–1.6	–2.2						
188-S	9.00				282	254	230	208	186	166	148	133	119	106	94	83	74	65	56	48	40	32	25					
	9.00				1.6	1.6	1.7	1.8	1.8	1.9	1.9	2.0	2.0	2.0	2.0	1.9	1.9	1.8	1.7	1.6	1.4	1.2	1.0					
					1.6	1.7	1.7	1.7	1.6	1.6	1.5	1.4	1.3	1.1	0.9	0.7	0.4	0.1	–0.3	–0.8	–1.3	–1.9	–2.5					
188-D1	14.39								233	213	194	175	157	141	126	113	100	89	78	69	61	54	47	41	35	30		
	4.00								2.2	2.2	2.3	2.4	2.4	2.5	2.5	2.5	2.5	2.4	2.4	2.3	2.2	2.0	1.8	1.6	1.4	1.1		
									2.1	2.0	2.0	1.9	1.8	1.7	1.5	1.3	1.1	0.8	0.4	0.0	–0.5	–1.0	–1.5	–2.1	–2.8	–3.6		
208-D1	15.50													159	143	129	115	103	92	81	72	63	55	48	42	36	31	26
	4.25													2.7	2.7	2.8	2.8	2.8	2.7	2.7	2.6	2.5	2.3	2.1	1.9	1.7	1.4	1.1
														2.0	1.9	1.7	1.5	1.2	0.9	0.5	0.1	–0.4	–1.0	–1.6	–2.2	–3.0	–3.8	–4.6

Strength is based on strain compatibility; bottom tension is limited to $12\sqrt{f'_c}$; see pages 2–7 through 2–10 for explanation.
Shaded values require release strengths higher than 3500 psi.

Exterior Girder Design:

w_u = (190 psf)(43.667'/2) = 4,148 plf
From PCI handbook, select L-Beam 20LB32 148-S with 2"concrete topping:

L-BEAMS



f'_c = 5,000 psi
f_{pu} = 270,000 psi
½ in. diameter
low-relaxation strand

Normal Weight Concrete

Designation	h in.	h_1/h_2 in./in.	A $in.^2$	I $in.^4$	y_b in.	S_b $in.^3$	S_t $in.^3$	wt plf
20LB20	20	12/8	304	10,160	8.74	1,163	902	317
20LB24	24	12/12	384	17,568	10.50	1,673	1,301	400
20LB28	28	16/12	432	27,883	12.22	2,282	1,787	450
20LB32	32	20/12	480	41,600	14.00	2,971	2,311	500
20LB36	36	24/12	528	59,119	15.82	3,737	[illegible]	550
20LB40	40	24/16	608	81,282	17.47	4,653	3,608	633
20LB44	44	28/16	656	108,107	19.27	5,610	4,372	683
20LB48	48	32/16	704	140,133	21.09	6,645	5,208	733
20LB52	52	36/16	752	177,752	22.94	7,749	6,117	783
20LB56	56	40/16	800	221,355	24.80	8,926	7,095	833
20LB60	60	44/16	848	271,332	26.68	10,170	8,143	883

1. Check local area for availability of other sizes.
2. Safe loads shown include 50% superimposed dead load and 50% live load. 800 psi top tension has been allowed, therefore, additional top reinforcement is required.
3. Safe loads can be significantly increased by use of structural composite topping.

Key
6566 – Safe superimposed service load, plf.
0.3 – Estimated camber at erection, in.
0.1 – Estimated long-time camber, in.

Table of safe superimposed service load (plf) and cambers (in.)

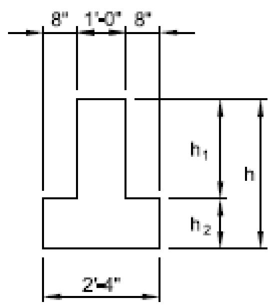
Desig-nation	No. Strand	y_s(end) in. y_s(center) in.	Span, ft 16	18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	48	50
20LB20	98-S	2.44	6566	5131	4105	3345	2768	2318	1961	1674	1438	1243	1079							
		2.44	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.0	1.1	1.2							
			0.1	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.2							
20LB24	108-S	2.80	9577	7495	6006	4904	4066	3414	2896	2479	2137	1854	1617	1416	1244	1097	969			
		2.80	0.3	0.3	0.4	0.5	0.5	0.6	0.7	0.8	0.9	0.9	1.0	1.0	1.1	1.1	1.2			
			0.1	0.1	0.1	0.1	0.1	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.1	0.0	0.0			
20LB28	128-S	3.33			8228	6733	5596	4711	4009	3443	2979	2595	2273	2000	1768	1567	1394	1243	1110	992
		3.33			0.4	0.4	0.5	0.6	0.6	0.7	0.8	0.9	0.9	1.0	1.1	1.1	1.2	1.2	1.2	1.3
					0.1	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	0.0
20LB32	148-S	3.71				8942	7446	6281	5356	4611	4001	3495	3071	2712	2406	2143	1914	1715	1540	1386
		3.71				0.4	0.5	0.5	0.6	0.7	0.7	0.8	0.9	1.0	1.0	1.1	1.2	1.2	1.3	1.3
						0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.2	0.2	0.2	0.2	0.1
20LB36	168-S	4.25					[illegible]457	7988	6823	5883	5113	4476	3941	3489	3103	2771	2483	2231	2011	1816
		4.25					0.4	0.5	0.5	0.6	0.7	0.8	0.8	0.9	1.0	1.1	1.1	1.2	1.2	1.3
							0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2
20LB40	188-S	4.89						9812	8386	7235	6293	5513	4858	4305	3832	3425	3073	2765	2495	2257
		4.89						0.4	0.5	0.6	0.6	0.7	0.8	0.8	0.9	1.0	1.0	1.1	1.1	1.2
								0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3
20LB44	198-S	5.05								8959	7803	6845	6042	5363	4783	4284	3851	3474	3143	2850
		5.05								0.5	0.6	0.6	0.7	0.8	0.8	0.9	0.9	1.0	1.1	1.1
										0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
20LB48	218-S	5.81									9226	8100	7158	6360	5678	5092	4584	4140	3751	3408
		5.81									0.5	0.6	0.6	0.7	0.8	0.8	0.9	0.9	1.0	1.1
											0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3
20LB52	238-S	6.17										9634	8521	7578	6774	6082	5482	4958	4499	4094
		6.17										0.6	0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.0
												0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3
20LB56	258-S	6.64											9954	8860	7927	7124	6427	5820	5287	4816
		6.64											0.6	0.7	0.7	0.8	0.8	0.9	1.0	1.0
													0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3
20LB60	278-S	7.33													9089	8173	7380	6688	6080	5544
		7.33													0.7	0.7	0.8	0.9	0.9	1.0
															0.3	0.3	0.3	0.3	0.3	0.3

Interior Girder Design:

w_u = (190 psf)(43.667'/2 + 33'/2) = 7,283 plf
From PCI handbook, select 28IT32 158-S with 2"concrete topping:

INVERTED TEE BEAMS

Normal Weight Concrete



f'_c = 5,000 psi
f_{pu} = 270,000 psi
½ in. diameter
low-relaxation strand

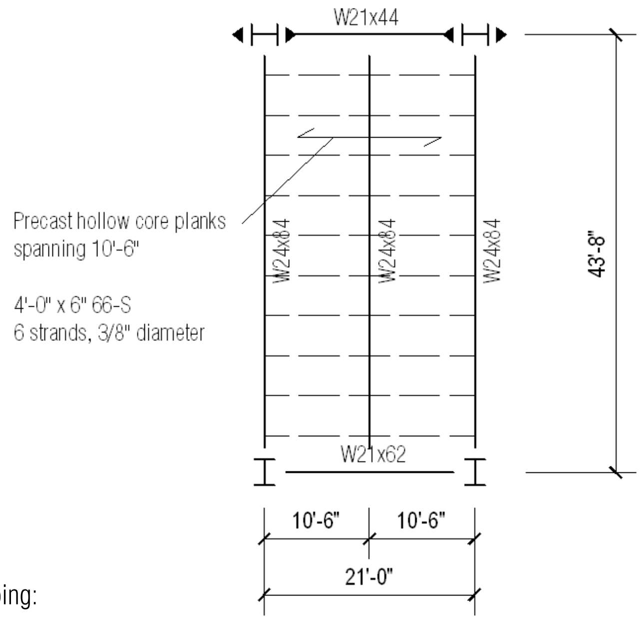
Section Properties								
Designation	h in.	h_1/h_2 in./in.	A $in.^2$	I $in.^4$	y_b in.	S_b $in.^3$	S_t $in.^3$	wt plf
28IT20	20	12/8	368	11,688	7.91	1,478	967	383
28IT24	24	12/12	480	20,275	9.60	2,112	1,408	500
28IT28	28	16/12	528	32,078	11.09	2,892	1,897	550
28IT32	32	20/12	576	47,872	12.67	3,778	2,477	600
28IT36	36	24/12	624	68,101	14.31	4,759	3,140	650
28IT40	40	24/16	736	93,503	15.83	5,907	3,869	767
28IT44	44	28/16	784	124,437	17.43	7,139	4,683	817
28IT48	48	32/16	832	161,424	19.08	8,460	5,582	867
28IT52	52	36/16	880	204,884	20.76	9,869	6,558	917
28IT56	56	40/16	928	255,229	22.48	11,354	7,614	967
28IT60	60	44/16	976	312,866	24.23	12,912	8,747	1,017

1. Check local area for availability of other sizes.
2. Safe loads shown include 50% superimposed dead load and 50% live load. 800 psi top tension has been allowed, therefore, additional top reinforcement is required.
3. Safe loads can be significantly increased by use of structural composite topping.

Key
6511 – Safe superimposed service load, plf.
0.2 – Estimated camber at erection, in.
0.1 – Estimated long-time camber, in.

Table of safe superimposed service load (plf) and cambers (in.)

Desig-nation	No. Strand	y_e(end) in. y_e(center) in.	Span, ft																	
			16	18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	48	50
28IT20	98-S	2.44 2.44	6511 0.2 0.1	5076 0.3 0.1	4049 0.4 0.1	3289 0.4 0.1	2711 0.5 0.1	2262 0.5 0.1	1905 0.6 0.0	1617 0.7 0.0	1381 0.7 0.0	1186 0.7 0.0	1022 0.8 –0.1							
28IT24	188-S	2.73 2.73	9612 0.2 0.1	7504 0.3 0.1	5997 0.3 0.1	4882 0.4 0.1	4034 0.4 0.1	3374 0.5 0.1	2850 0.6 0.1	2427 0.6 0.1	2081 0.7 0.1	1795 0.7 0.1	1555 0.7 0.0	1351 0.8 0.0	1178 0.8 –0.1	1029 0.8 –0.2				
28IT28	138-S	3.08 3.08			8353 0.3 0.1	6822 0.3 0.1	5657 0.4 0.1	4750 0.5 0.1	4031 0.5 0.1	3451 0.6 0.1	2976 0.6 0.1	2582 0.7 0.1	2252 0.7 0.1	1973 0.8 0.1	1735 0.8 0.0	1530 0.8 0.0	1352 0.9 –0.1	1197 0.8 –0.2	1061 0.8 –0.2	
28IT32	158-S	3.47 3.47				9049 0.3 0.1	7521 0.4 0.1	5333 0.4 0.1	5389 0.5 0.1	4628 0.5 0.1	4006 0.6 0.1	3490 0.6 0.1	3057 0.7 0.1	2691 0.7 0.1	2379 0.8 0.1	2110 0.8 0.1	1876 0.9 0.0	1673 0.9 0.0	1495 0.9 0.0	1337 0.9 –0.1
28IT36	168-S	3.50 3.50					9832 0.3 0.1	8295 0.4 0.1	7075 0.4 0.1	6092 0.5 0.1	5287 0.5 0.1	4619 0.6 0.1	4060 0.6 0.1	3587 0.7 0.1	3183 0.7 0.1	2835 0.8 0.1	2534 0.8 0.0	2271 0.9 0.0	2040 0.9 0.0	1836 0.9 –0.1
28IT40	198-S	4.21 4.21							8638 0.4 0.1	7440 0.5 0.1	6460 0.5 0.1	5647 0.6 0.1	4966 0.6 0.1	4390 0.7 0.1	3898 0.7 0.1	3474 0.8 0.1	3107 0.8 0.1	2787 0.8 0.1	2506 0.9 0.1	2258 0.9 0.1
28IT44	208-S	4.40 4.40								9186 0.4 0.1	7989 0.5 0.1	6997 0.5 0.1	6165 0.6 0.1	5462 0.6 0.1	4861 0.7 0.1	4344 0.7 0.1	3896 0.7 0.1	3505 0.8 0.1	3162 0.8 0.1	2859 0.8 0.0
28IT48	228-S	4.55 4.55									9719 0.4 0.1	8525 0.5 0.1	7523 0.5 0.1	6676 0.6 0.1	5953 0.6 0.1	5330 0.7 0.1	4791 0.7 0.1	4320 0.8 0.1	3907 0.8 0.1	3542 0.9 0.1
28IT52	248-S	5.17 5.17										9987 0.5 0.1	8823 0.5 0.1	7838 0.6 0.1	6998 0.6 0.1	6274 0.6 0.1	5647 0.7 0.1	4100 0.7 0.1	4619 0.8 0.1	4196 0.8 0.1
28IT56	268-S	5.23 5.23												9307 0.5 0.2	8319 0.6 0.2	7469 0.6 0.2	6731 0.7 0.2	6088 0.7 0.2	5524 0.8 0.2	5026 0.8 0.2
28IT60	288-S	5.57 5.57													9645 0.6 0.2	8668 0.6 0.2	7820 0.7 0.2	7081 0.7 0.2	6432 0.8 0.2	5859 0.8 0.2



System 4: Precast Hollow Core Plank

Referenced: PCI Design Handbook, 6th Edition

Loading: Live load = 100 psf
Dead load = 25 psf
wu = 1.2(25) + 1.6(100) = 190 psf

Span: 21'-0"

Materials: f'c = 5,000 psi
fpu = 270,000 psi (reinforcement)

Special Requirements: 2-hour fire rating
2000 μin/sec vibration limit

Hollow Core Slab Design:

From PCI handbook, select 4'-0" x 6" Hollow Core 66-S with 2" topping:

$w_{allowable}$ = greater than 470 psf (for 10.5' span) > w_u = 190 psf OK

Strand Pattern Designation
76-S

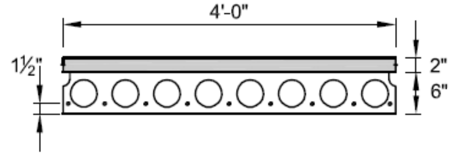
S = straight
Diameter of strand in 16ths
No. of Strand (7)

Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

Capacity of sections of other configurations are similar. For precise values, see local hollow-core manufacturer.

Key
444 – Safe superimposed service load, psf
0.1 – Estimated camber at erection, in.
0.1 – Estimated long-time camber, in.

HOLLOW-CORE
4'-0" x 6"
Normal Weight Concrete



f'_c = 5,000 psi
f_{pu} = 270,000 psi

Section Properties

		Untopped		Topped	
A	=	187	in.^{2}	283	in.^{2}
I	=	763	in.^{4}	1,640	in.^{4}
y_b	=	3.00	in.	4.14	in.
y_t	=	3.00	in.	3.86	in.
S_b	=	254	in.^{3}	396	in.^{3}
S_t	=	254	in.^{3}	425	in.^{3}
wt	=	195	plf	295	plf
DL	=	49	psf	74	psf
V/S	=	1.73	in.		

4HC6 + 2

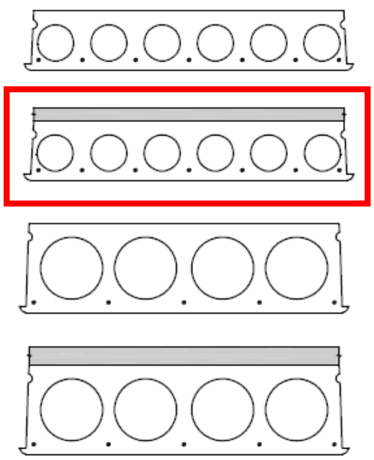
Table of safe superimposed service load (psf) and cambers (in.) **2 in. Normal Weight Topping**

Strand Designation Code	Span, ft																		
	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
66-S	470	396	335	285	244	210	182	158	136	113	93	75	59	46	34				
	0.2	0.2	0.2	0.2	0.2	02	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2				
	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2	–0.3	–0.5	–0.7	–0.9	–1.2				
76-S		461	391	334	287	248	216	188	163	137	115	95	78	63	50	38	27		
		0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.1	–0.0	–0.1	–0.3		
		0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.2	–0.3	–0.5	–0.7	–0.9	–1.2	–1.5		
96-S			473	424	367	319	279	245	216	186	160	137	116	98	82	68	55	43	33
			0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.1	0.0	–0.1
			0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.1	–0.1	–0.3	–0.5	–0.7	–1.0	–1.4	–1.7
87-S			485	446	415	377	331	292	258	224	195	169	147	127	109	94	80	67	55
			0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.7	0.8	0.8	0.7	0.7	0.7	0.6	0.5	0.4	0.3
			0.5	0.5	0.5	0.6	0.6	0.6	0.5	0.5	0.4	0.4	0.2	0.1	–0.1	–0.3	–0.5	–0.8	–1.2
97-S			494	455	421	394	357	327	288	251	219	192	168	146	127	110	95	82	70
			0.5	0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9	1.0	0.9	0.9	0.9	0.8	0.7	0.6
			0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.5	0.4	0.2	0.0	–0.2	–0.5	–0.8

Strength is based on strain compatibility; bottom tension is limited to $7.5\sqrt{f'_c}$; see pages 2–7 through 2–10 for explanation.

Figure 2.5.6 Section Properties – Normal Weight Concrete **Ultra Span**

Trade Name: Ultra Span ®
Licensing Organization: Ultra Span Technologies Inc., Winnipeg, Manitoba, Canada



Section	Untopped				With 2 in. topping		
width x depth	A in.^{2}	y_b in.	I in.^{4}	wt psf	y_b in.	I in.^{4}	wt psf
4'-0" x 4"	154	2.00	247	40	2.98	723	65
4'-0" x 6"	188	3.00	764	49	4.13	1,641	74
4'-0" x 8"	214	4.00	1,666	56	5.29	3,070	81
4'-0" x 10"	259	5.00	3,223	67	6.34	5,328	92
4'-0" x 12"	289	6.00	5,272	75	7.43	8,195	100

Note: All sections are not available from all producers. Check availability with local manufacturers.

Due to my limited knowledge at this point and a restriction on time, there are no vibration checks in the calculations below. Thus, members are designed based on flexure and deflection only, giving smaller designs than what will likely be required. Vibration analysis will be considered at a later date.

Steel Beam Design:

Intermediate Beams (spanning 43.667'-0"):
Slab self-weight = 74 psf (from table above)
Total load = 1.2(74) + 1.6(100) = 249 psf

Flexure:

$$Mu = \frac{(249 \text{ psf} \times 10.5 \text{ ft})(43.667)^2}{8} = 623 \text{ ft-k}$$

Deflection:

$$\Delta L_{allowable} = \ell/360 = (43.667 \times 12)/360 = 1.45''$$

$$\Delta L = \frac{5(100 \times 10.5)(43.667)^4}{384(29{,}000)(I)} \times 1728/1000 \leq 1.45''$$

$$I \geq 2042 \text{ in}^4$$

$$\Delta D{+}L_{allowable} = \ell/240 = (43.667 \times 12)/240 = 2.18''$$

$$\Delta D{+}L = \frac{5(100+74 \text{ psf})(10.5)(43.667)^4}{384(29{,}000)(I)} \times 1728/1000 \leq 2.18''$$

$$I \geq 2364 \text{ in}^4$$

Without considering vibration, choose W24x84.
$\phi M_n = 840$ ft-k > $M_u = 623$ ft-k OK
I = 2370 > 2364 OK

Interior Girder (spanning 21'-0"):

Total load = (249 psf x 10.5' x 43.667')/2 + (249 psf x 10.5' x 33')/2 = 100.2 k at midspan

Unfactored:

P_L = (100 x 10.5' x 43.667'/2) + (100 x 10.5' x 33'/2) = 40.3 k

P_{D+L} = (174 x 10.5' x 43.667'/2) + (174 x 10.5' x 33'/2) = 70.0 k

Flexure:

$$Mu = \frac{(100.2)(21)}{4} = 526 \text{ ft-k}$$

Deflection:

$\Delta L_{allowable} = \ell/360 = (21 \times 12)/360 = 0.7"$

$$\Delta L = \frac{(40.3)(21)^3}{48(29{,}000)(I)} \times 1728 \leq 0.7"$$

$I \geq 662 \text{ in}^4$

$\Delta D+L_{allowable} = \ell/240 = (21 \times 12)/240 = 1.05"$

$$\Delta D+L = \frac{(70)(21)^3}{48(29{,}000)(I)} \times 1728 \leq 2.18"$$

$I \geq 369 \text{ in}^4$

Without considering vibration, choose W21x62.
$\phi M_n = 540$ ft-k $> M_u = 526$ ft-k OK
$I = 1330 > 662$ OK

Exterior Girder (spanning 21'-0"):

Total load =(249 psf x 10.5' x 43.667')/2 = 57.1 k at midspan

Unfactored:

P_L = (100 x 10.5' x 43.667'/2) = 22.9 k

P_{D+L} = (174 x 10.5' x 43.667'/2) = 39.9 k

Flexure:

$$Mu = \frac{(57.1)(21)}{4} = 300 \text{ ft-k}$$

Deflection:

$\Delta L_{allowable} = \ell/360 = (21 \times 12)/360 = 0.7"$

$$\Delta L = \frac{(22.9)(21)^3}{48(29{,}000)(I)} \times 1728 \leq 0.7"$$

$I \geq 376 \text{ in}^4$

$\Delta D+L_{allowable} = \ell/240 = (21 \times 12)/240 = 1.05"$

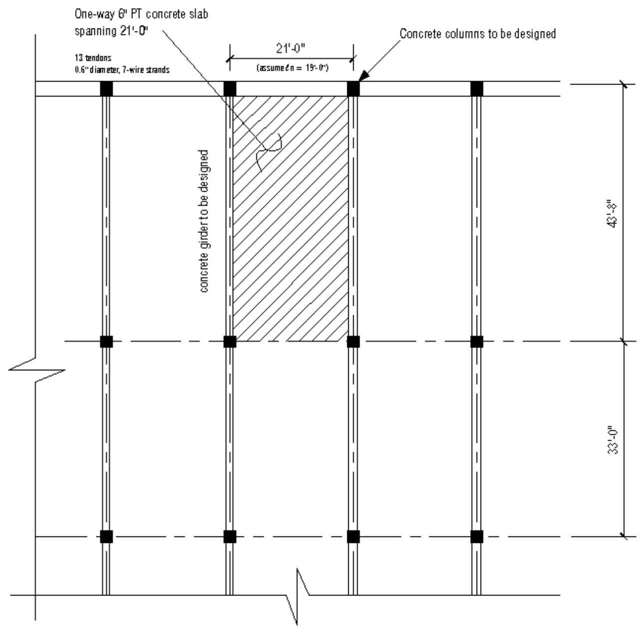
$$\Delta D+L = \frac{(39.9)(21)^3}{48(29{,}000)(I)} \times 1728 \leq 2.18"$$

$I \geq 437 \text{ in}^4$

Without considering vibration, choose W21x44.
$\phi M_n = 358 > M_u = 300$ OK
$I = 612 > 437$ OK

System 5: Post-Tensioned One-Way Slab and Beams

Referenced: ACI 318-05



Loading:

Live load = 100 psf

Reduced live load:

$A_I = 21' \times (43.667' + 33') = 1610$ sq. ft.

Reduction factor $= 0.25 + 15/(\sqrt{1610})$

$= 0.62 > 0.4$, OK

$LL = 0.62(100) = 62$ psf

Dead load (superimposed) = 25 psf

Dead load (self) = (6"/12)(150) = 75 psf

Check: $LL/DL = 62/100 = 0.62 < 0.75$

No pattern loading required (ACI 13.7.6)

Total Load, w = 162 psf

Factored Load, $w_u = 1.2(100) + 1.6(62) = 219$ psf

$w_{pre} = 0.9(75 \text{ psf}) = 68$ psf

$w_{net} = 162 - 68 = 94$ psf

Materials:

$f'_c = 5{,}000$ psi

Unbonded tendons:

0.6" diameter, 7-wire strands

$A = 0.217 \text{ in}^2$

$f_{pu} = 270{,}000$ psi

Estimated pre-stress losses = 15 ksi (ACI 18.6)

Effective stress in steel:

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

$P_{eff} = A(f_{se}) = (0.217)(174) = 37.8$ kips / tendon

Special Requirements:

2-hour fire rating

2000 μin/sec vibration limit

System Geometry:

$\ell = 21'$-0"

$\ell_n = 19'$-0" assuming 2'-0" x 2'-0" columns

ℓtributary = 43.667'/2 + 33'/2 = 38.33'

cover = ¾" (restrained slab, 2-hour fire rating) (IBC 2006)

Due to my limited knowledge at this point and a restriction on time, there are no vibration or deflection checks in the calculations below. Thus, members are designed based on flexure and deflection only, giving smaller designs than what will likely be required. Vibration analysis and determination of deflection will be considered at a later date.

Post-Tensioned One-Way Slab Design:

Preliminary Slab Thickness

h (slab thickness) → ℓ/h = 45

h = 21(12)/45 = 5.6

h = 6” preliminary slab thickness

Section Properties

$A = bh = (12)6 = 72 \text{ in}^2$

Allowable Stresses

$f'_c = 5000 \rightarrow \beta_1 = 0.80$ …………………………………………………….. (ACI 10.2.7.3)

$f'_{ci} = 3000$ psi

Stresses in concrete at time of jacking:

Compression = $0.6f'_{ci} = 0.6(3000) = 1800$ psi …………………. (ACI 18.4.1a)

Tension = $3\sqrt{f'ci} = 3\sqrt{3000} = 164$ psi ………………………….. (ACI 18.4.1b)

Stresses in concrete at service loads:

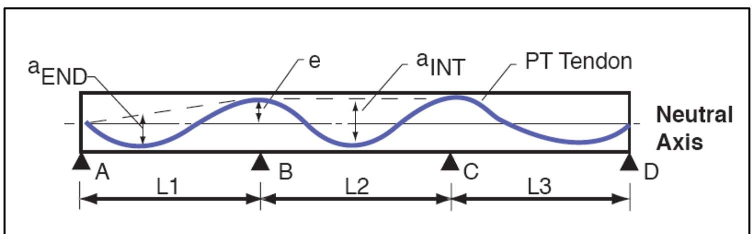
Compression = $0.45f'c = 0.45(5000) = 2250$ psi ………………. (ACI 18.4.2a)

Tension = $6\sqrt{f'c} = 6\sqrt{5000} = 424$ psi ………………………….. (ACI 18.4.2b)

Since $f_t = 424 \leq 7.5\sqrt{f'c} = 530$, **Design as Class U.** ……………. (ACI 18.3.3)

Tendon Profile:

Parabolic shape: Tendons will typically be located at the highest allowable point at the interior columns, the lowest allowable point at the midspans, and the neutral axis at the anchor locations. See figure below.



Courtesy of: Portland Cement Association Concrete Design Resources

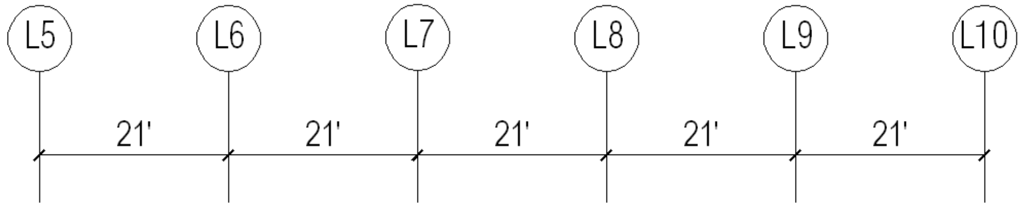
Tendon Ordinate	Tendon Location (center of gravity) from bottom of slab
Exterior support: anchor	3.0”
Interior support: top	5.0”
Interior span: bottom	1.0”
End span: bottom	1.75”

$a_{int} = 6 - 1.25” = 4.75”$

$a_{end} = (3.0 + 5.0)/2 – 1.75 = 2.25”$

The eccentricity, e, is the distance from the center of the tendon to the neutral axis. It varies along the span.

Calculation of Stresses



	Span 1	Span 2	Span 3	Span 4	Span 5
W_{pre} (psf)	68 psf	68 psf	68 psf	68 psf	68 psf
M_{pre} (ft-k)	3.7 ‘k	3.7 ‘k	3.7 ‘k	3.7 ‘k	3.7 ‘k
a (in)	2.25”	4.7”	4.7”	4.7”	2.25”
F (kips)	19.7 k	9.4 k	9.4 k	9.4 k	19.7 k
F/A (psi)	273.6	130.5	130.5	130.5	273.6

Check: 273.6 $>$ 125 psi min (ACI 18.12.4)
$<$ 300 psi max

130.5 $>$ 125 psi min (ACI 18.12.4)
$<$ 300 psi max

OK

Required Tendons

19.7 k/ft x 38.33 ft = 755 k
$A_{reqired}$ = 755 k / 270 ksi = 2.8 in^2
Number of Tendons = 2.8 / (0.217 kips / tendon) = 12.9
Use 13 tendons spanning the short direction (21’-0”).

Check Punching Shear
$Vc = 4\sqrt{f'c}(b_o d) = 4\sqrt{5000}(28.75 \times 4)(4.75) = 154.5$ kips
$\phi Vc = 0.75(154.5\ k) = 115.9$ kips

Vu = (21’ x 38.33’)(219 psf) = 176.3 kips

Since ϕVc $<$ Vu, drop panels are required.

ϕVc needed = 176.3 k / 0.75 = 236 kips
236 k = $4\sqrt{5000}(b_o)(4.75)$
b_o = 175.7”

175.7” = 4(b + d) → If b = 24”, d = 20”
20” - 6” slab = 14” drop panel

Need minimum 14” drop panel at each column, assuming 24” x 24” columns.