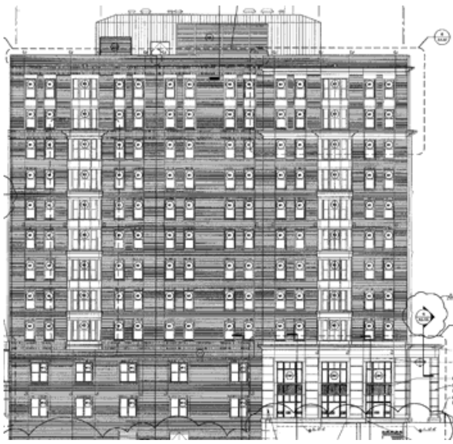
Existing Structure: Partially Post-Tensioned Structural Slabs

The concrete structure of Johns Hopkins University Charles Commons is mostly a conventional system applicable to most dormitories. There are two foundation systems, spread footings and shallow caissons. There are many continuous reinforced concrete columns that range from rectangular to square dimensions. Shear walls surround the many stair and elevator openings and the precast and brick façade is not load-bearing. Even the penthouse roof structured with typical wide-flange steel beams and metal decking. However, in the slab resides the largest complication for the Charles Commons project team.

The slab is partially post-tensioned, meaning that post-tensioning tendons coincide with rebar reinforcing. A conventional post-tensioning layout was prescribed for the Charles building since the building contains only dormitory space. In stark contrast, only the St. Paul building's top seven floors are exclusively dormitory space. The first three floors of the St. Paul building include a bookstore, a retail space, a conference area, and a full-service dining commons. In addition to the 8" thick post-tensioning slabs, perimeter edge beams and drop panels are implemented throughout to assuage deflection concerns.

Charles Building

The Charles building contains 12 floors that reaches an ultimate height of 153'-4", which is the tallest that the City of Baltimore and Historic Charles Street Association would permit. The 65'x35' footprint affords the Charles building only 100,000 sf. Its small footprint and rectangular shape allows for a structural plan that is nearly uniform throughout the building. The post-tensioned slabs and reinforced columns in the Charles building could be constructed in as little as five days a floor. The foundation of the Charles building began after the fourth floor of the St. Paul building due to staging and utility work. Since the Charles building afforded the construction team very few complications, this building will be spared detailed structural analysis, however, the systems applied on the St. Paul building can easily be extended to the Charles building.



Charles Building

St. Paul Building

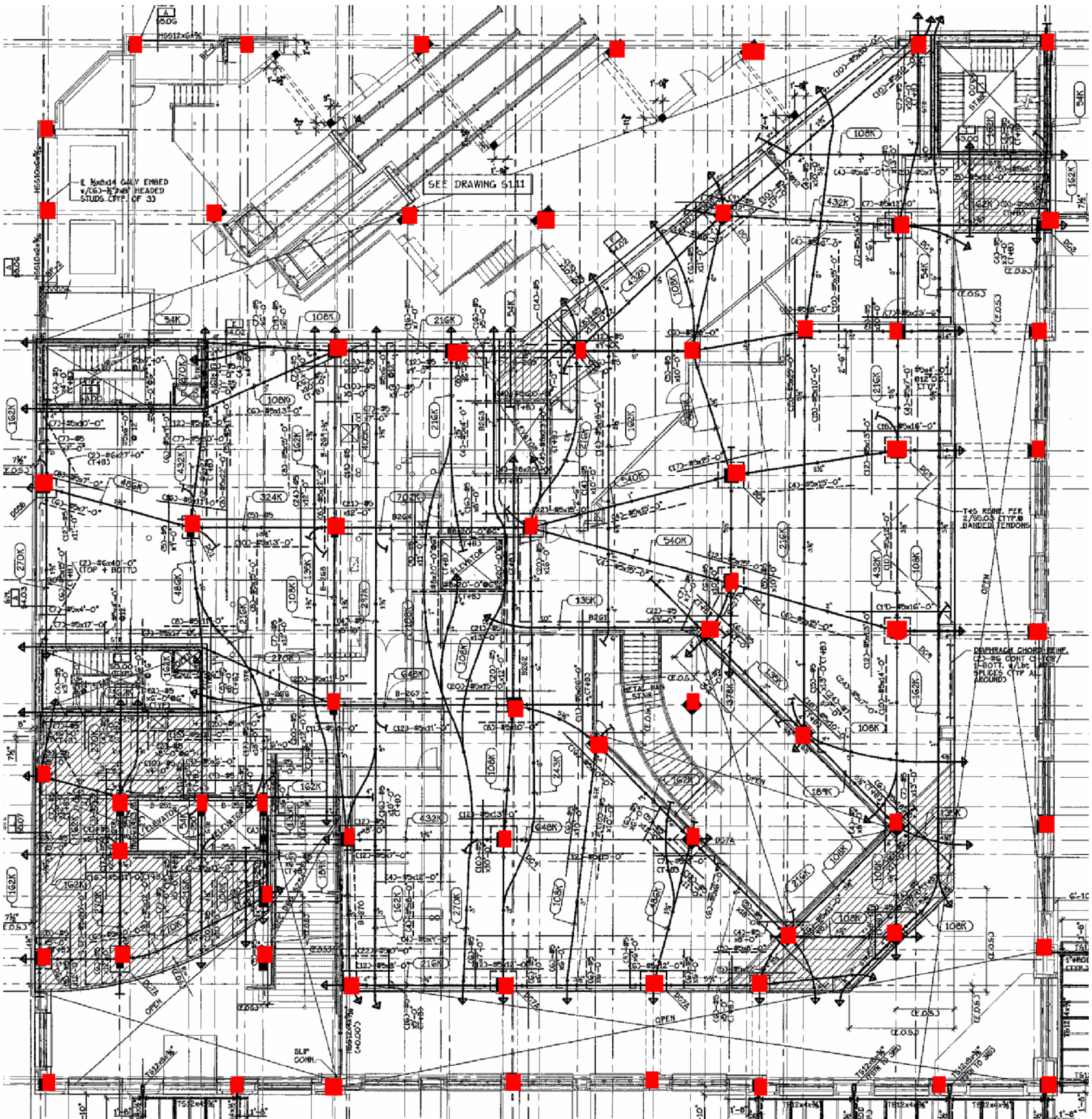
St. Paul Building

This 213,000 sf building towers 134'-8" and ten stories into the Baltimore cityscape. The difference between the two buildings is two stories, or 18'-8", which will be discussed later. Its footprint is quite large at 81'x87'. After the fourth floor, the building resembles a U-shape because of a large interior courtyard space. Before the fourth floor, the building maintains its square shell, but contains many large and odd-shaped floor openings for mezzanines, mechanical shafts, six elevators, four staircases, and a loading dock. To accommodate these large openings and a variety of functions, the engineers have specified two strengths of concrete for the slabs and beams, 6000 psi for the first two floors and 4500 psi for the remaining floors. In addition, the columns on the first two floors are 8000 psi, the next two floors are 6000 psi, and the remaining floors are 4000 psi.

The most frustrating aspect in redesigning the St. Paul building is its column layout. There are no typical bays. Spans range from 18'-29'. All of the columns are either covered in sheetrock and exposed or hidden inside walls. Realignment of columns more than two feet in any direction requires a redesign of the space function, a door or window realignment, and mechanical redesign. However, difficult design layout is not the reasoning to analyze the floors in the St. Paul building.

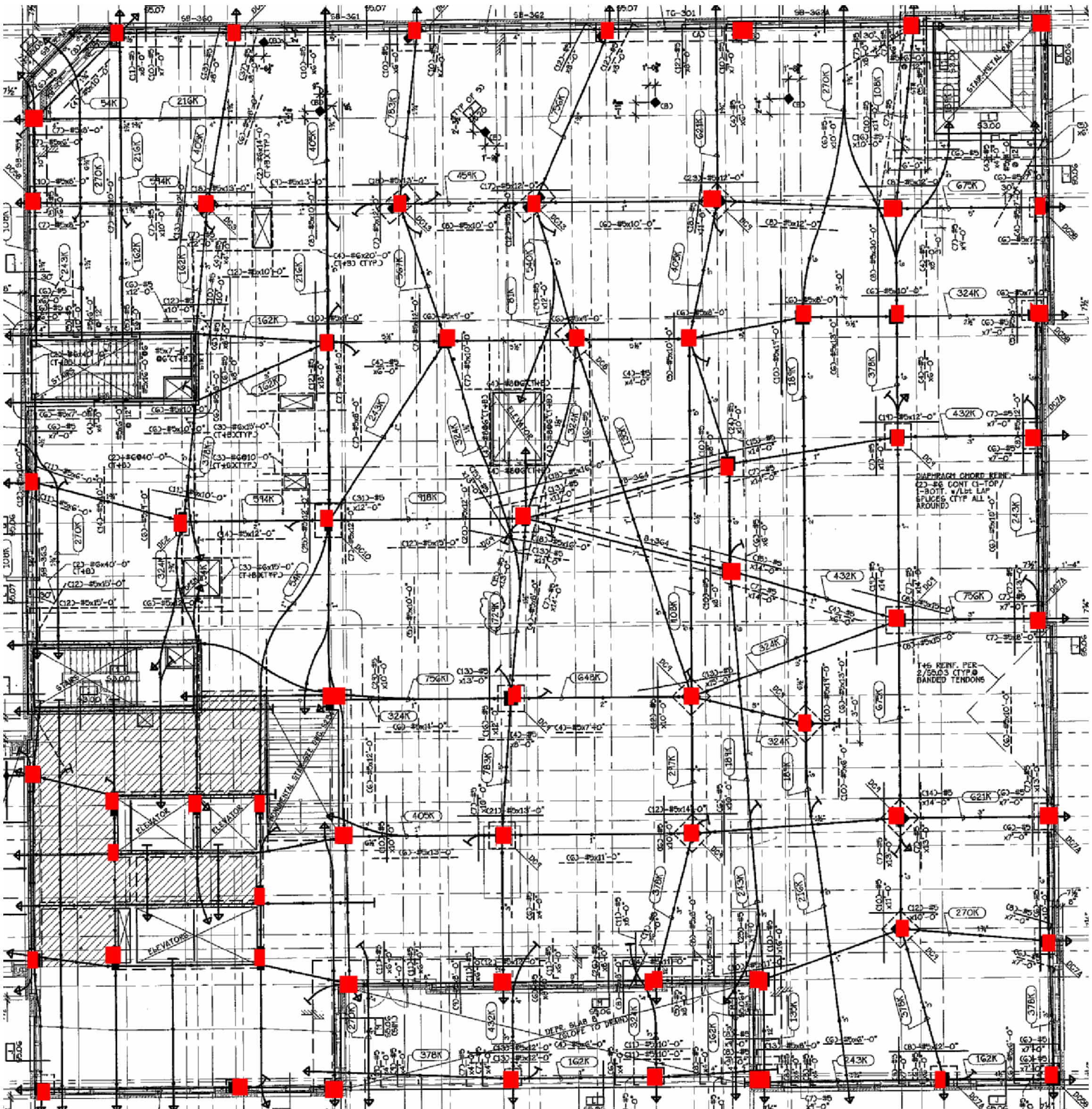
Three month construction for the first three floors of the St. Paul building is driving force for this analysis. The "custom" design of St. Paul makes it impossible to use the same formwork, the same rebar sequence, and the same concrete mixes. In addition, the problems relating to post-tensioned slabs resonated to the layouts of the mechanical, electrical, and plumbing systems. MEP coordination proved to be costly, delayed, and complicated. Hence, the third analysis will review the results from the structural breadth and propose solutions for MEP coordination success.

Second Floor Plan



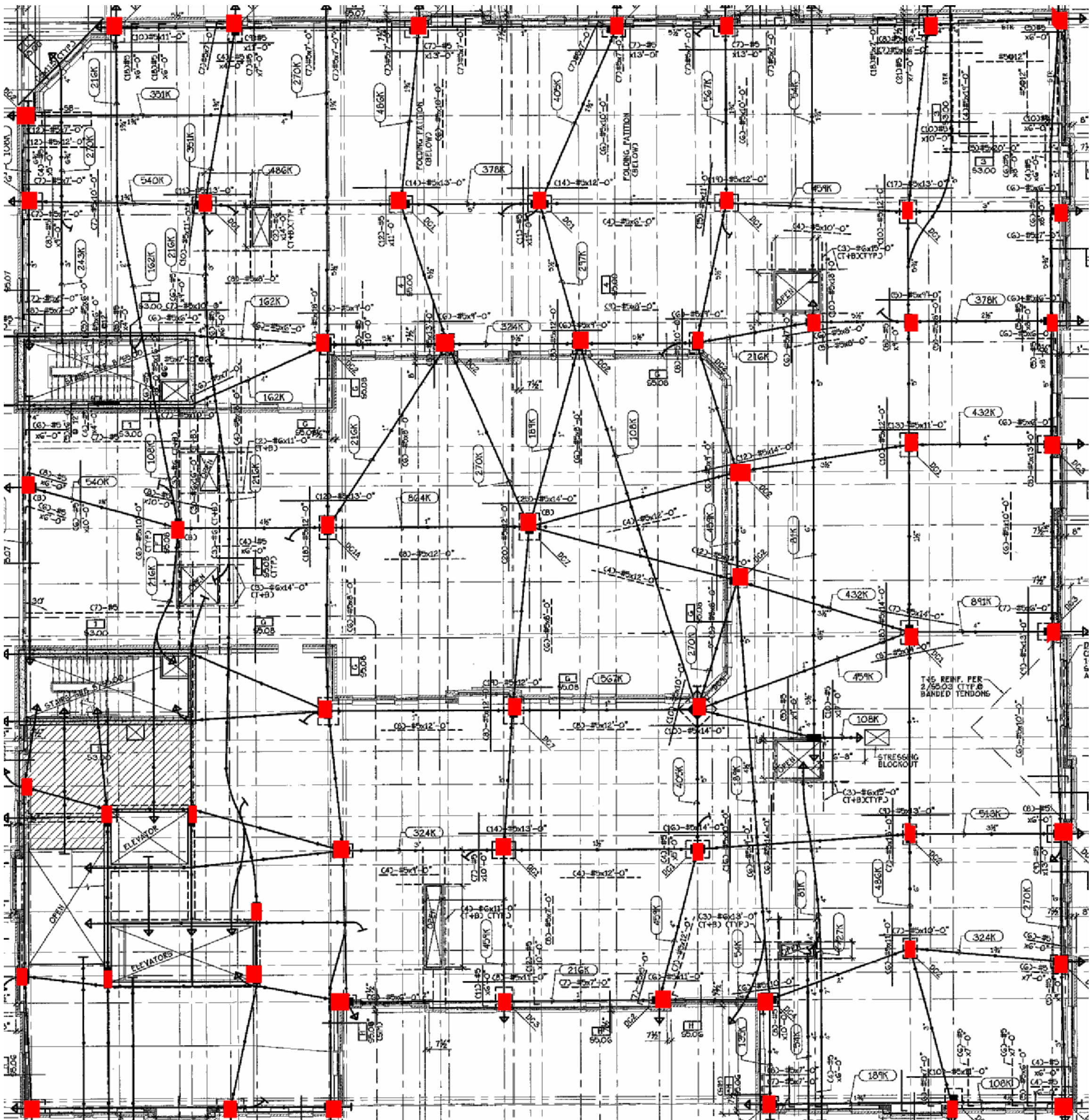
Structure Supports:	Loading docks, lobby, bookstore
Floor Height (1^{st}-2^{nd}):	15'-0"
Number of Columns:	67
Strength of Columns:	8000 psi
Strength of Slab:	6000 psi
Floor Completed in:	3 weeks

Third Floor Plan



Structure Supports:	Conference room, break-out rooms, dining hall
Floor Height (1st-2nd):	15'-0"
Number of Columns:	65
Strength of Columns:	8000 psi
Strength of Slab:	6000 psi
Floor Completed in:	6 weeks

Fourth Floor Plan



Structure Supports:	Courtyard, lounges, corridors, apartment suites
Floor Height (1^{st}-2^{nd}):	15'-4"
Number of Columns:	65
Strength of Columns:	6000 psi
Strength of Slab:	4500 psi
Floor Completed in:	6 weeks

Load Calculations

Loads and requirements as applicable to the design of the structural floors are:

I.)	Live Loads	
	A. Penthouse	30 psf
	B. Roof	30 psf
	C. Stairs	125 psf
	D. Public Rooms	100 psf
	E. Corridors	100 psf
	F. Dormitory Apartments	40 psf
	G. Dining Hall	125 psf
	H. Office	50 psf
	I. Retail	125 psf
II.)	Dead Loads	
	A. Slab – 8" thick	100 psf
	B. Bearing concrete shearwalls	20 psf
	C. Superimposed MEP	8 psf
III.)	Strength Requirements	
	A. Concrete (28 day strength)	
	i. Walls	4000 psi
	ii. Columns	4000, 6000, 8000 psi
	iii. Slabs, beams	4500, 6000 psi
	B. Steel (Yield Strength, F_y)	
	i. Reinforcement bars	60 ksi
IV.)	Steel Cover Requirements	
	A. Slab on Grade	1"
	B. Beams/Columns	1-1/2"
V.)	Post-Tensioning	
	A. Compressive strength at transfer	2,700 psf
	B. Steel yield strength	270,000 psf
	C. Effective stress after losses	189,000 psf
	D. Preliminary long term losses	15,000 psf

Existing Structural Floor System

The current floor system for the sampled floor, the second floor, is an 8" partially post-tensioned system. The loads on the floor slab are the 8 psi super-imposed dead load and 125 psi live load. The self-weight of the 8" slab is approximated at 100 psi. The spans vary from 18' to 29' between 24"x24" columns typically.

Post-Tensioned Slab	
Slab Thickness	8”
Concrete Strength	4500 psi, 6000 psi
Concrete Volume	5259 CY
Reinforcement Weight	350 ton
Self-weight	100 psf
Column Sizes	24”x24”, 24”x12”
Column Volume	609 CY
Building Height	134’-8”

Post-Tensioned Slab	Issue	Reason
Advantages	Structural Code	Does not limit depth of slab
	Rebar Placement	Needed in only one direction
	Formwork	Requires less edge formwork for thinner slab
	Building Height Requirements	Allows the maximum capacity of tenant space in areas with building height limits
Disadvantages	Safety	Snapped stressed tendons are catastrophic
	Complexity	Many different allowable stresses on cables, specialty contractors required
	Error	Slight margin for error, must retain prescribed height of tendon through pour.
	Equipment	Extra jacking equipment needed
	Slab Curing	More time is needed between pours to stress tendons and allow relaxation
	MEP Coordination	MEP penetrations must be planned and fabricated beforehand. Few core-drills allowed.
	Onsite laydown area	Large space, cables must be unraveled prior to setting them in place
	Labor	Must have experienced subcontractor and personnel
	Mistakes	Most problems relating to reinforcement in slabs require large-scale removal of concrete/reinforcing
	Weather	Cannot be performed in less than 45 deg. F. without slab heaters

Study of Alternate Floor Systems

Through discussions with the structural design firm, the existing floor system was found adequate. This shows that a live load of 125 psf and dead loads of 108 psf should be applied to alternate systems to compare the design inefficiencies. The three alternate systems I will analyze are the following:

- *Flat-plate reinforced concrete slab without drop panels*
- *Slab with one-way reinforced concrete beams with drop panels*
- *Precast concrete slab on cast-in-place beams*

Alternate I: Flat-plate reinforced concrete slab without drop panels

The flat-plate reinforced concrete slab idea was mentioned first by the general contractor on Charles Commons, Struever Bros, Eccles & Rouse. The foremost difficulty of post-tensioning is planning. The sequencing of trades before the slab pours proved much too difficult on the first four floors. In addition, slow-starting MEP Coordination could not effectively provide the dimensions in which slab penetrations would be needed. Since the first four structural slabs of St. Paul lie on the critical path of Charles Commons, it was no surprise that the overall project delayed more than three months.

The design of the flat-plate slab assumed all of the columns to be exactly where they had been designed. Only the flexibility of a flat-plate slab can allow the unequal spacing of columns and large openings. To move the column spacing to make the flat-plate slab design more efficient would have greatly compromised the architectural aspects of all of the floors of the building.

Methodology

For this exercise, a 29' span between two columns was analyzed using the current codes on a spreadsheet. From this data, a trial slab thickness was found and input into E-TABS, a program that make calculations for various load combinations to extricate the forces and moments associated with the entire building. This model includes all openings, columns, beams, shear walls, and cladding.

Calculations

The initial limiting factor for flat-plate slabs with 29' spans is the ACI limits on slab thickness. For a flat-plate design, the slab thickness is restricted by ACI (9.5.3.2) to be ln/33 without drop panels.

Lmax = 29' $\Rightarrow$ thickness = (29'*12)/33 = 10.54" > 8" existing slab thickness

It is quite doubtful that the designed slab without post-tensioning will be in the vicinity of 10" thick. It is most likely that the slab will be approximately 12-16" thick, so this limit factor does not limit this design.

At this point, the design was broken into interior and exterior spans to calculate the shear and moment capacities. As shown below, the design used 4000 psi concrete strengths and columns that are 24"x24". The total loads were calculated along with the allowable deflections. The following spreadsheet shows these values.

INTERIOR SPANS

slab thickness:	**14**	**in**	column width:	**24**	**in**
f'c:	**4000**	**psi**	column depth:	**24**	**in**
shear depth:	12.75	in	tributary width:	**29**	**ft**
			b for both strips:	14.5	ft
live load:	**125**	**psf**			
dead load:	**183**	**psf**	Longest clear span ln = 29 – (20/12) = 27.33 ft		
total unfactored load:	308	psf	Minimum h per ACI Table 9.5(c) = ln/33 = 10.54"		
total factored load:	**419.6**	**psf**			
Vu1	4169.8	lb for a 12" width	Allowable deflection for serviceability		
			L/240 =	1.45	in
PhiVc	1329.9	lb	L/360 =	0.97	in
			L/480 =	0.725	in
bo	147	in			
Vu2	237.8	k			
PhiVc	403029	lb			
PhiVc	403.0	k			
Static Moment ol	917.7	ft-k			
Static Moment os	917.7	ft-k			

Bars	*Area*	*Unit weight*
#4's	0.20	0.376
#5's	0.31	0.668
#6's	0.44	1.043
#7's	0.60	1.502
#8's	0.79	2.044
#9's	1	3.4

EXTERIOR SPANS

slab thickness:	**14 in**	column width:	**24 in**
f'c:	**4000 psi**	column depth:	**24 in**
shear depth:	12.75 in	tributary width:	**29 ft**
		b for both strips:	14.5 ft
live load:	**125 psf**		
dead load:	**183 psf**	Longest clear span ln = 29 – (20/12) = 27.33 ft	
total unfactored load:	308 psf	Minimum h per ACI Table 9.5(c) = ln/33 = 10.54"	
total factored load:	**419.6 psf**		

Vu1 4169.8 lb for a 12" width

Allowable deflection for serviceability

L/240 =	1.45 in
L/360 =	0.97 in
L/480 =	0.725 in

PhiVc 1329.9 lb

bo 147 in

Vu2 237.75 k

PhiVc 403029 lb

PhiVc 403.03 k

Static Moment ol 917.67 ft-k

Static Moment os 917.67 ft-k

Bars	*Area*	*Unit weight*
#4's	0.20	0.376
#5's	0.31	0.668
#6's	0.44	1.043
#7's	0.60	1.502
#8's	0.79	2.044
#9's	1	3.4

After the static moments in both directions are found, a chart is created using the moment equations for the column and middle strips depending on negative and positive moments from ACI 318 Section 8.3.3. These moments are checked for steel and the cross-section receives the selected bars.

For Interior Spans:

MOMENT CALCULATIONS	*Span ol = Span os*							
	Column Strip (12)				*Middle Strip (12)*			
	negative		positive		negative		positive	
Mu	-481.8	ft-k	275.3	ft-k	160.6	ft-k	183.5	ft-k
Mu/(phi)(bd^2)	-272.5	psi	155.7	psi	-90.8	psi	103.8	psi
rho (for 4000 psi)	0.0048		0.0033		0.0033		0.0033	
As	10.649	in^2	7.321	in^2	7.321	in^2	7.321	in^2
Bars Selected	14 8's		10 8's		10 8's		10 8's	

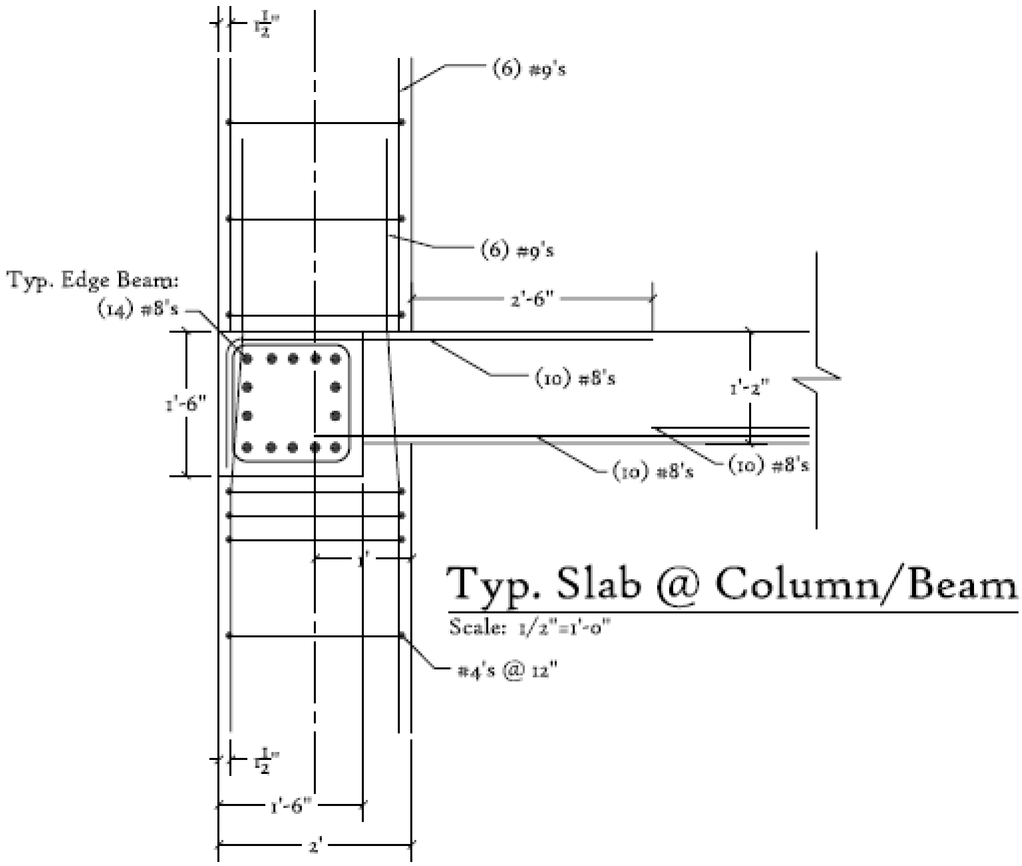
For Exterior Spans:

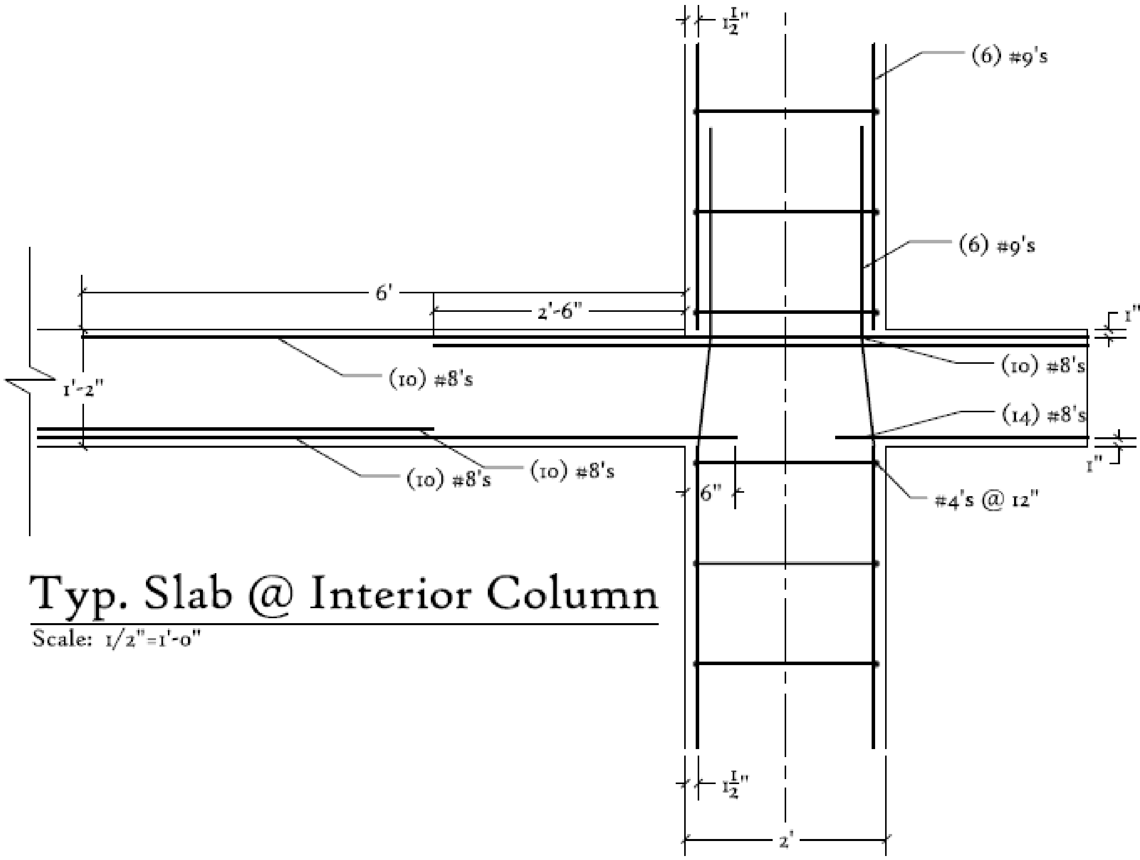
MOMENT CALCULATIONS	Span o1 = Span o5							
	Column Strip (12)				Middle Strip (12)			
	negative		positive		negative		positive	
Mu	-206.5	ft-k	275.3	ft-k	68.8	ft-k	183.5	ft-k
Mu/(phi)(bd^2)	-116.8	psi	155.7	psi	-38.9	psi	103.8	psi
rho (for 4000 psi)	0.0033		0.0033		0.0033		0.0034	
As	7.321	in^2	7.321	in^2	7.321	in^2	7.543	in^2
Bars Needed	10 8's		10 8's		10 8's		10 8's	

At the conclusion of the moment calculations, deflection calculations must be calculated since deflection will most likely limit the design of this slab. Three deflection calculations were made: dead load, live load, and total load. The equations used are found in ACI and are listed 9-8, 9-9, 9-10. These were compared to the allowable deflections specified by ACI 318, Table 9.5b. In addition, the long-term deflection (assumed greater than five years) ACI equation 9-11 was used and compared to the long-term limit of l/240. Since the deflections for the exterior spans were found to be equal with the interior spans, there is only one chart posted.

DEFLECTION CALCULATIONS		
y	6.86 in	
Inertia	3319.06 in^4	
Is	146.88 in^4	
Live Mom	630750 ft-k	
Dead Mom	923418 ft-k	
rho'	0.040	Deflection Limits
Live Deflection	0.885 in	0.967 in
Dead Deflection	0.476 in	0.725 in
Total Deflection	1.361 in	1.450 in
Long-Term Deflec.	0.671 in	0.725 in

These deflection calculations caused what had amounted from an 11" slab to a 14" slab. Specifically, the live load and total load deflection limits control since the long spans and the 125 psf live load is not the most efficient use of concrete. If the spans were decreased 5' throughout Charles Commons and the dining hall function removed, the slab would be controlled by the ACI span limits. The following is a cross-section of the designed 14" flat-plate slab.





Typ. Slab @ Interior Column

Scale: 1/2"=1'-0"

Modeling

In addition to small-scale checks on a long-span of 29', E-TABS modeled six loads in different combinations and analyzed the following:

- Strength-required reinforcement
- Unbalanced moments due to uneven dead load (column spacing)
- Axial forces due to large live loads
- Point deflections due to a variety of loads

E-TABS was chosen over RAM and other software for its ability to model multiple customized floors that the moment and axial forces from the floors above could be distributed evenly over all slabs. RAM would have been useful for calculating finite element mesh analyses for the slabs, but approximately five different slabs would have to be modeled to adequately determine the design for all of the slabs in the ten floors of the St. Paul building.

Strength-required reinforcement

The reinforcement required for the columns and beams were calculated based on load combinations of the loads: dead, live, super-imposed dead, cladding dead, wind, and earthquake. The beams were most affected by the 250 plf cladding dead loads while the columns were most affected by the 125 psf live loads. In addition, the largest loads were found to be on the perimeter of the building, where openings and uneven column spacing had controlled their design.

As an addendum to the original model, 18”x18” columns replace the 24”x24” columns on floors 5-10 to ensure efficient use of concrete. These changes are reflected in the following analyses.

Column design

The interior columns on floors 1-4 were shown to require 5.76 sq. in. reinforcing for their 24”x24” cross sections, which means approximately (6) #9 bars. The exterior columns on floors 1-4 are not as standard as the interior columns due to overloaded cross-sections in areas. Exterior columns are identical to the interior columns except:

- Column @ A1, at the southwest corner of the structure
- Column @ L1, near the southeast corner of the structure
- Column @ M1, at the southeast corner of the structure

At these columns, 36”x36” cross-sections were used for the first two floors of these columns with 18 sq. in. of reinforcing, approximately (13) #11 bars. This overloading can be attributed to the large opening on the second floor level and the transfer of load from the recessed area on the south side of the building.

On floors 5-10, all of the columns are 18”x18” and require 3.24 sq. in. of reinforcing, which is approximately (4) #9 bars. All columns including the columns on floors 5-10 have low requirements for shear reinforcing.

Beam design

Only exterior beams are used in this design since they are implemented as edge beams. The typical beams were 18”x18” and shown to require as much as 10.97 sq. in. longitudinal reinforcing. The shear and torsion reinforcing required was less than 1 sq. in. and made little impact on design. Typically (14) #8 bars were required for both the bottom and top longitudinal reinforcing. However, a few beams were exceptions such as the following:

- Beam spanning column line A, the first 4 floors, at the west elevation
- Beam spanning column line 1, the first floor, at the south elevation
- Beam spanning column line M, the first 4 floors, at the east elevation
- Beam spanning column line 2, floors 2-10, at the south recessed area

- The beams at column lines A and M are problem areas because of how the wind forces were applied to the model. The beam spanning column line 1 is an issue since it

braces a large cladding load and spans between the buildings largest columns. The beams spanning column line 2 are not in plane with the building's square footprint. All of these were deepened to 18"x22" and contain approximately the same reinforcing layout.

Unbalanced moments

Diagrams were produced that show the unbalanced moments that are introduced due to the uneven loading residual from the column spacing. This diagram shows the largest moments where the cross-sections of the columns and beams were increased due to unbalanced conditions as mentioned in strength-required reinforcement. These diagrams can be found in Appendix B.

Axial forces

As well as unbalanced moment diagrams, the resulting axial forces were compiled into a diagram. The over-sized beams and columns that resulted from the strength-required reinforcement show the greatest axial forces. These diagrams can be found in Appendix B as well.

Point Deflections

Point deflections were calculated at random places along all of the slabs to find if the largest deflections meet the 0.725" limit for dead load and the 0.967" limit for live load. The largest deflections were found at the midpoints along the exterior of the building due to the cladding dead loads and at the midpoints of middle strips in the slab due to live loads. The deflection values ranged from 0 to -0.33", which is much less than both limits. This can be attributed to the fact that most of the 29' spans are along the exterior of the building where edge beams assist in deflection control and in areas where the transverse span is much less than 29', creating much smaller deflections.

Flat-Plate Slab	
Slab Thickness	14"
Concrete Strength	4000 psi
Slab Concrete Volume	9204 CY
Reinforcement Weight	Approx. 450 ton
Self-weight	175 psf

Building Height	139'-8"
Column Sizes	24"x24", 18"x18", and a few 36"x36"
Column Volume	748 CY

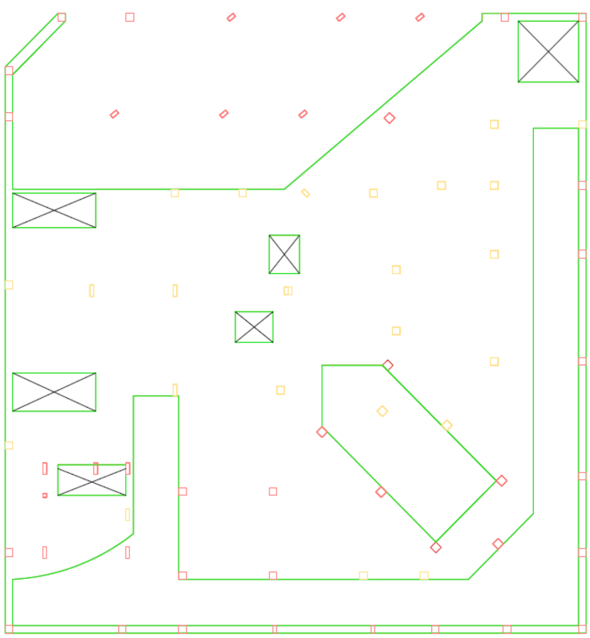
Flat-Plate Slab	Issue	Reason
Advantages	Safety	No special safety considerations
	Building Height Requirements	Relatively effective in areas with building height limits
	Complexity	Easy to duplicate construction, many contractors perform flat-plate
	Error	Larger margin for error, rebar only must maintain heights
	Equipment	No extra equipment needed
	MEP Coordination	MEP penetrations need not be planned beforehand. Core-drills are allowed on a limited spacing.
	Labor	Requires little subcontractor and personnel experience
	Mistakes	Most problems relating to reinforcement in slabs require minimal slab demolition that can be performed relatively easily
Disadvantages	Structural Code	Does limit slab thickness
	Rebar Placement	Needed in two directions
	Formwork	Requires more edge formwork for thicker slab
	Building Height Requirements	Little effectiveness in areas with building height limits
	Slab Curing	Time is needed between floors to allow for curing
	Onsite laydown area	Large space, different size rebar must be sorted prior to installation
	Weather	Cannot be performed in less than 45 deg. F. without slab heaters

Alternate II: Slab with one-way reinforced concrete beams and drop-caps

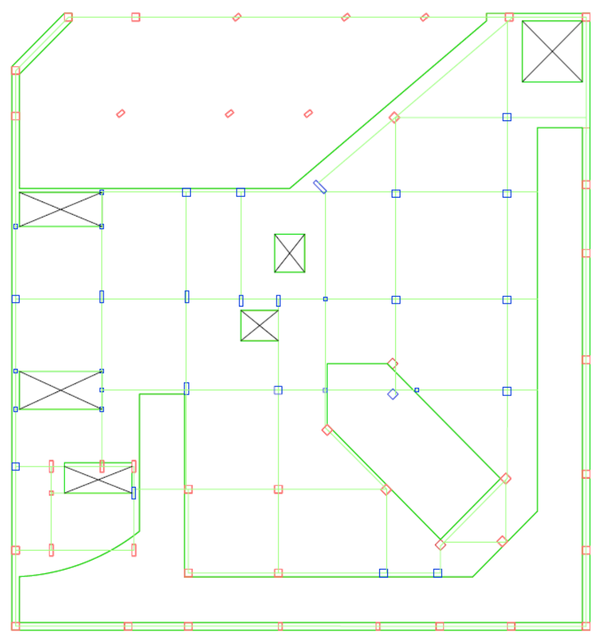
The reinforced concrete slab with one-way reinforced concrete beams idea was mentioned first by Dr. Parfitt. This idea was changed approximately two weeks after this report was completed in mid-March to additional drop-caps when a live load deflection calculation mistake provided the savings of a 12" slab to a 9" slab. Drop-caps were added to prevent the punching shear that results from slabs smaller than 12" without drop-caps. This design would have approximately the same flexibility found with the flat plate slab, but using less concrete between the ribs of concrete joists. However, the column spacing must be altered to make the one-way beams span in perpendicular directions and will subsequently compromise the architectural aspects of all of the floors of the building. First, an adequate column layout must be found and modeled.

Adjusting the Column Layout

Approximately half of the approximately 50-60 columns on each floor were adjusted as much as 12' to accommodate the one-way beam configuration. In addition, 17 columns were added creating smaller spans of 21' instead of 29'. However, the odd angled areas in the floor plan such as the loading dock, the grand staircase, and the lobby area will be left as a 10" slab. Approximately 25% more concrete will be used in the columns and 10% less concrete will be used in the slabs compared to the flat plate slab.

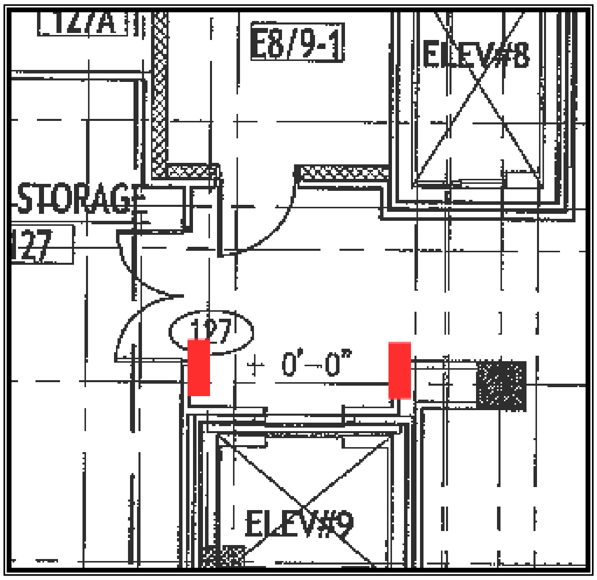


Existing Column Layout
(yellow columns are deleted)



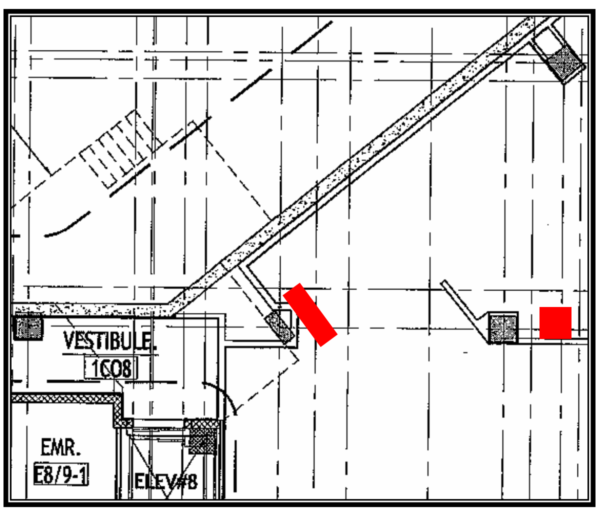
Adjusted Column Layout
(blue columns are added)

Architectural Adjustments



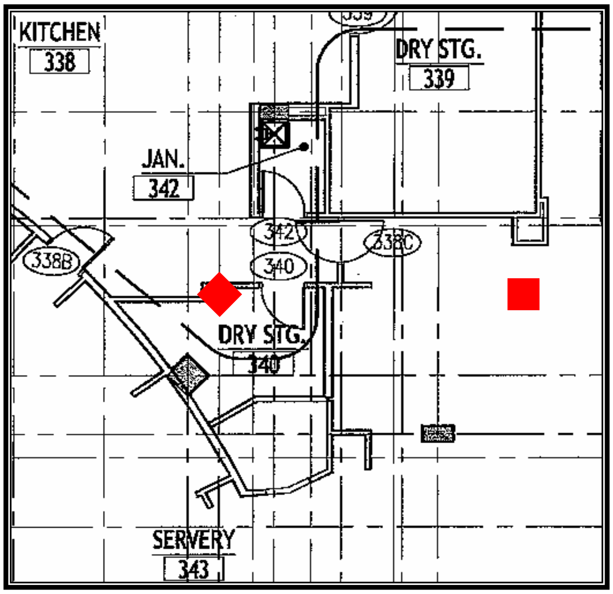
Second Floor, at elevator #9

In order to fit uniform bays around the openings on the second floor, a new column line was created running north to south along elevator #8 and #9. Another new column line ran along the west side of elevator #9. The conjunction of these new column lines lies in a storage corridor of the bookstore space. This arrangement impedes the opening of the doors into the storage room by 1', in which the doors must be adjusted 1' toward the north. Carts exiting elevator #9 should not have difficulty around the columns.



Second Floor, at the loading dock

In order to align the column lines next to the loading dock with those of the feature staircase, a service corridor near the loading docks needs to be adjusted 3' to the east. The full opening size is accounted for in this adjustment.



Third Floor, at the dining servery

All of the columns in the dining hall space have not been moved more than 2', except these in dry storage and a corridor beyond the servery. These were corrected by 5' to align the column lines from the dining hall to those near the conference room and loading docks. Both of these locations do not impede traffic through corridors or doorways.

Methodology

For this exercise, a 21' span between two columns was analyzed using the current codes on a spreadsheet. From this data, a trial slab thickness and joist thickness was found and input into E-TABS. The model that tested this information was quite different from the model used for the flat plate slab. This model does not include openings, shear walls, and cladding. A 5x5 column configuration spanning 21' in each direction was duplicated for 10 stories to model this alternate because E-TABS cannot place any beams that aren't perfectly perpendicular or perfectly even opening sizes.

Calculations

The initial limiting factor for one-way beams with 21' spans is the ACI limits on slab thickness. For a one-way beam design, the slab thickness is restricted by ACI (9.5.2.1) to be ln/28 for both end continuous spans.

Lmax = 21' ⇒ thickness = (21'*12)/28 = 9" > 8" existing slab thickness

Although 9" is much less limiting than the flat plate's 10.54", the overall design is still controlled by the deflections (the additional punching shear has already been remedied with 5'x5' drop panels). At this point, calculations were performed to find the moments at three locations and shear checks. As shown below, the design used 4000 psi concrete strengths and columns that are 24"x24". The total loads were calculated along with the allowable deflections. The following spreadsheet shows these values.

ONE-WAY SLAB WITH BEAMS AND DROP-CAPS					
beam depth:	**14**	**in**	beam width:	**6**	**in**
slab thickness:	**9**	**in**	column width:	**24**	**in**
f'c:	**4000**	**psi**	column depth:	**24**	**in**
shear depth:	7.75	in	first span:	21	ft
			second span:	21	ft
live load:	**125**	**psf**	b for both strips:	11	ft
dead load:	**132**	**psf**	Joists:	8 ea span or 30" clear spacing	
total unfactored load:	257	psf			
total factored load:	**359**	**psf**	Longest clear span ln = 21 - (20/12) = 19.33 ft		
beam self-weight:	12	psf	Minimum h per ACI Table 9.5(a) = ln/28 = 9"		
at interior span:	17.59	ft-k	Allowable deflection for serviceability:		
at midspan:	11.30	ft-k	L/240 =	1.05	in
at exterior support:	6.59	ft-k	L/360 =	0.53	in
			L/480 =	0.525	in
.75 rhob	0.02				
d^2	18.79	in^2			
d	4.33	in			
bo	8.26	in			
As	0.50	in^2			
a	0.74	in			
As	0.49	in^2			
As at midspan:	0.32	in^2	#5's @10" .37 in^2		
As at exterior:	0.19	in^2	#5's @15" .25 in^2		
Asmin	0.19	in^2			
Vu:	4102	lb			
Vn = Vc:	11764	lb			
phiVc:	9999	lb			
Punching Shear for 24"x24" column, ACI 318 11.12.2.1					
Vc	4754	*smallest of three			
Vc	28291				
Vc	9055				
Vlimit	11764	ok			

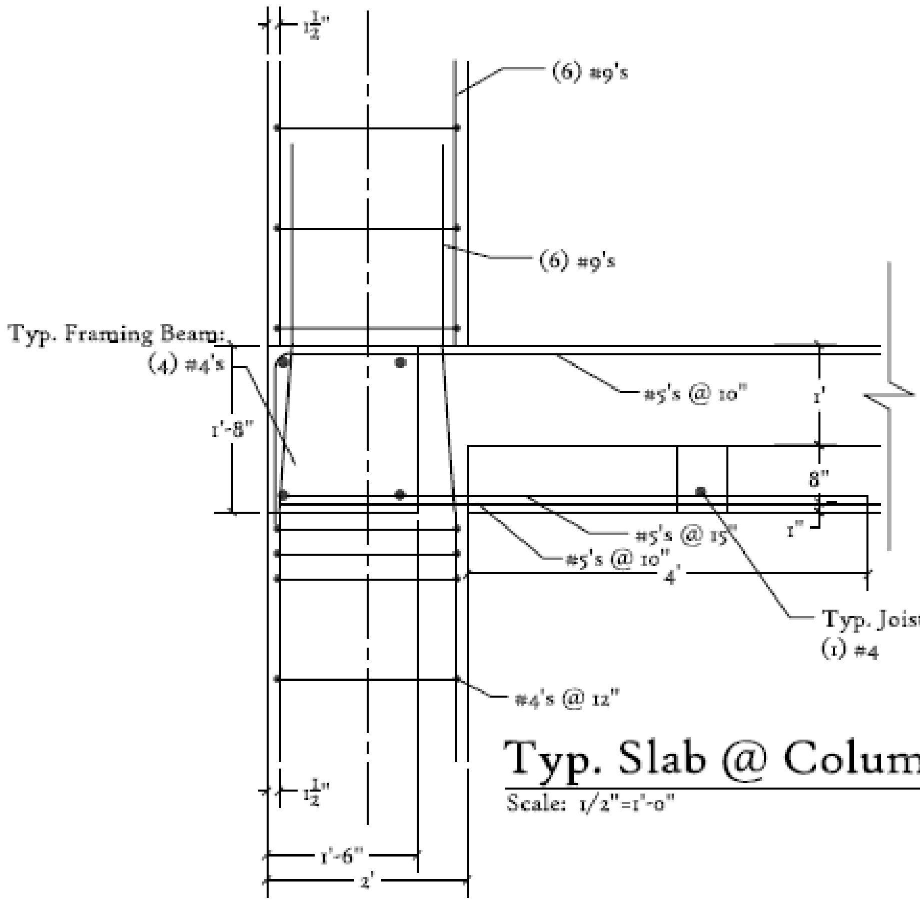
Try 9" slab with 6"x14" beams @ 30"

Try 5'x5' drop-caps with 14" depth

After finding the reinforcement, deflection calculations must be calculated since deflection will most likely limit the design of this slab. Three deflection calculations were made: dead load, live load, and total load. The equations used are found in ACI and are listed 9-8, 9-9, 9-10. These were compared to the allowable deflections specified by ACI 318, Table 9.5a. In addition, the long-term deflection (assumed greater than five years) ACI equation 9-11 was used and compared to the long-term limit of l/240.

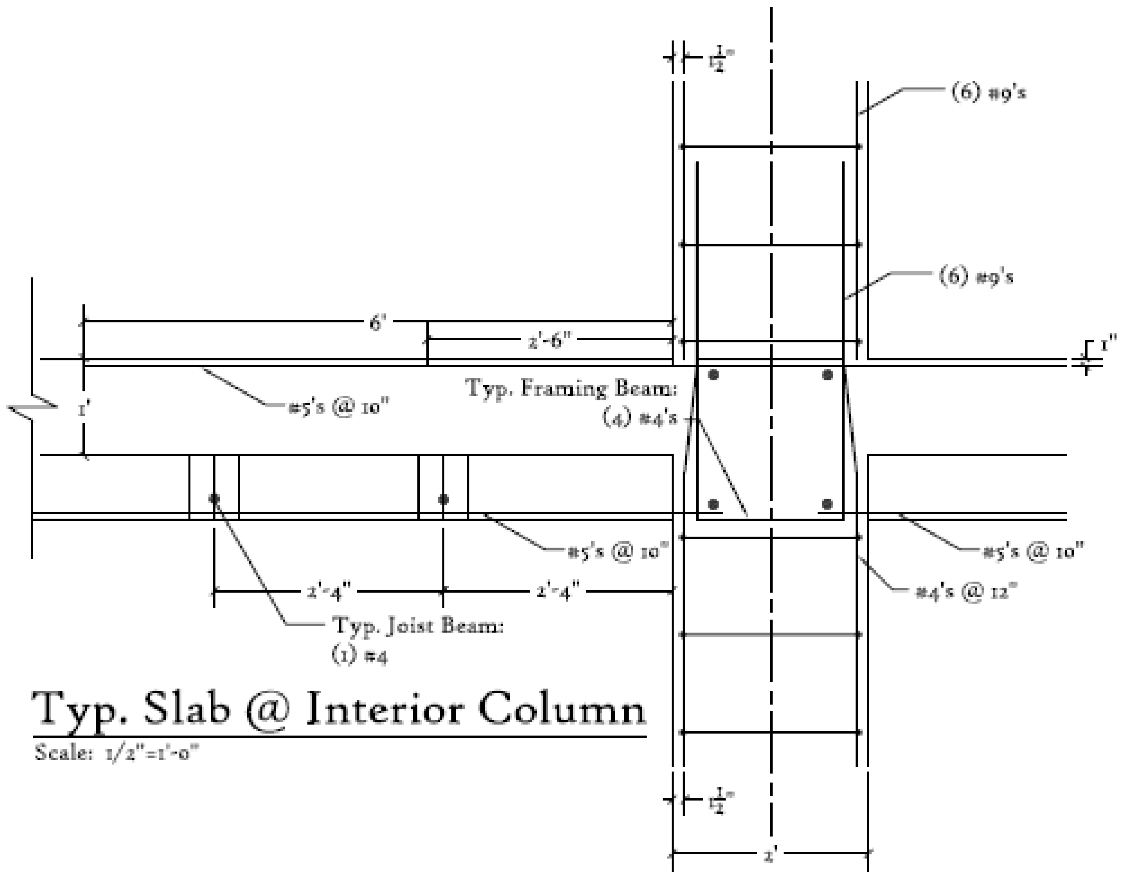
DEFLECTION CALCULATIONS

Inertia	771	in^4		
y	3.88	in		
Is	19	in^4		
Total Inertia	790	in^4		
x	114000			
Moment	50069	ft-k	Deflection Limits	
Live deflection	0.608	in	0.7	in
Dead deflection	0.405	in	0.53	in
Total deflection	1.013	in	1.05	in
Long-term deflection	0.668	in	0.70	in



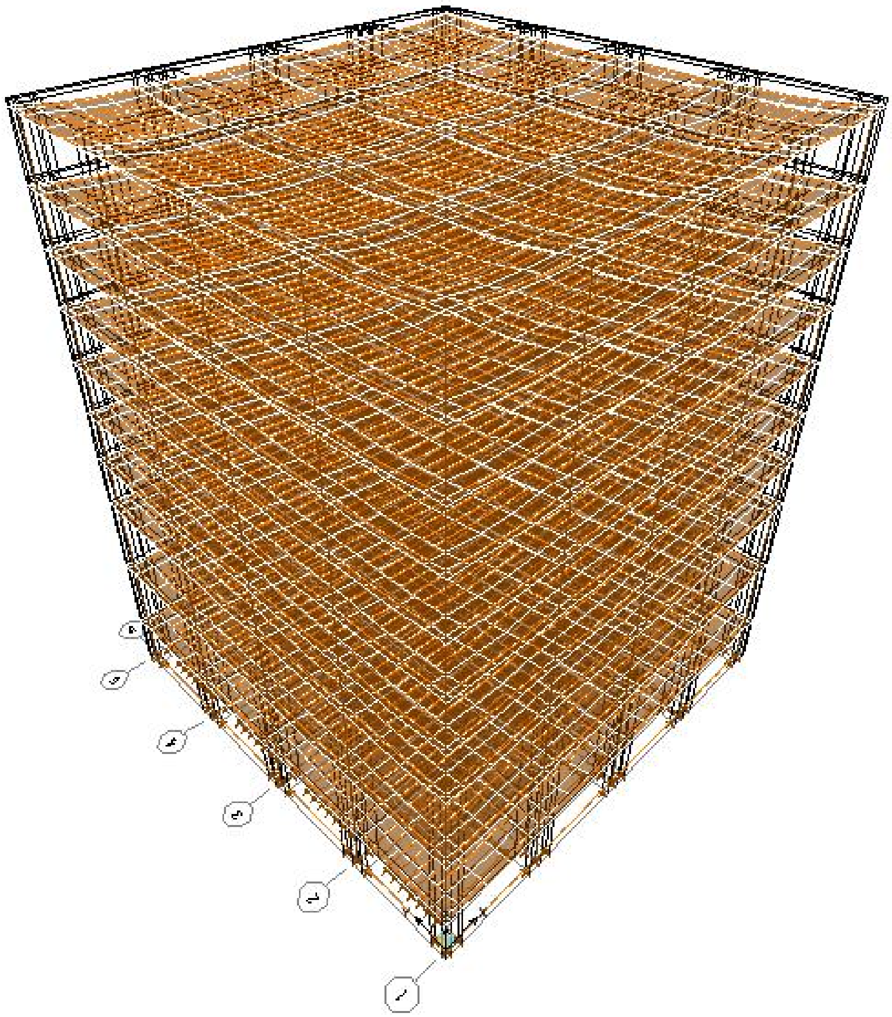
Typ. Slab @ Column/Beam

Scale: 1/2"=1'-0"



Typ. Slab @ Interior Column

Scale: 1/2"=1'-0"



Modeling

In addition to small-scale checks on a long-span of 21', E-TABS modeled six loads in different combinations and analyzed the following:

- Strength-required reinforcement
- Axial forces due to large live loads
- Point deflection due to a variety of loads

Although E-TABS is the best for this application, E-TABS does not allow beams to span outside of the initially specified grid. Therefore, a highly idealized model of the St. Paul building's new column layout must be used.

Strength-required reinforcement

The reinforcement required for the columns and beams were calculated based on load combinations of the loads: dead, live, super-imposed dead, and cladding dead

load. Consideration of wind and earthquake loads were omitted since this "ideal" condition does not realistically compensate for the differences in the exterior shape of the building (including the courtyard). The beams were most affected by the 250 plf cladding dead loads while the columns were most affected by the 125 psf live loads.

As an addendum to the original model, 18"x18" columns replace the 24"x24" columns on floors 5-10 to ensure efficient use of concrete. These changes are reflected in the following analyses.

Column design

The columns on floors 1-4 were shown to require 5.76 sq. in. reinforcing for their 24"x24" cross sections, which means approximately (6) #9 bars. All other columns require 4.27 sq. in. reinforcing for their 18"x18" cross sections. These results show that the one-way slab alternative is quite capable of holding the 125 psf live load under 21' spans.

Beam design

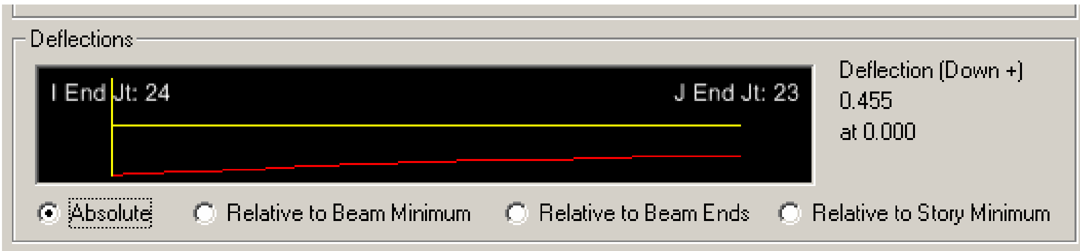
Two types of beams are used in this design: joist beams and framing beams. The framing beams were 12"x14" and shown to require as little as 0.7 sq. in. (or four #4's) longitudinal reinforcing. The shear and torsion reinforcing required was less than 1 sq. in. and made little impact on design. The joist beams were much smaller, where typically eight joists span 21' and have 6"x14" dimensions (or 28" clear spacing). These joist beams only require 0.15 sq. in! Each joist only requires one #4 bar.

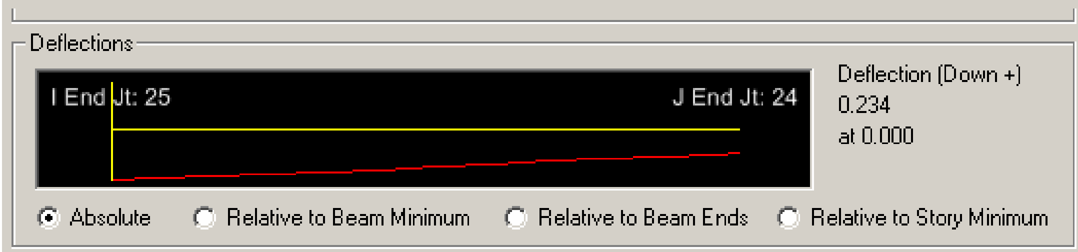
Axial forces

The resulting axial forces were compiled into a diagram. The under-sized beams and columns that resulted from the strength-required reinforcement show the least axial forces are located at the top of the building. These diagrams can be found in Appendix B.

Point Deflections

Point deflections were calculated at random places along all of the slabs to find if the largest deflections meet the 0.700" limit for live load. The largest deflections were found at the midpoints along the exterior of the building. The highest deflection values were 0.455" for live load, which is comfortably below the limits.





One-way Beams with Drop-caps	
Slab Thickness	9" with 14" drop-caps
Concrete Strength	4000 psi
Slab Concrete Volume	6518 CY
Reinforcement Weight	Approx. 500 ton
Self-weight	160 psf
Building Height	138'-0"
Column Sizes	24"x24", 18"x18", 12"x24"
Column Volume	948 CY

One-way Beams	Issue	Reason
Advantages	Safety	No special safety considerations
	Complexity	Easy to duplicate construction, many contractors perform one-way beam structures
	Error	Larger margin for error, rebar only must maintain heights
	Equipment	No extra equipment needed
	MEP Coordination	MEP penetrations need not be planned beforehand. Core-drills are allowed on a limited spacing.
	Labor	Requires little subcontractor and personnel experience
	Mistakes	Most problems relating to reinforcement in slabs require minimal slab demolition that can be performed relatively easily
	Building Height	Effective in areas with building height limits

Disadvantages	Structural Code	Does limit slab thickness greatly
	Rebar Placement	Needed in two directions, but separate in beams and slab
	Formwork	Requires more edge formwork for thicker slab and formwork for added beams
	Slab Curing	Time is needed between floors to allow for curing
	Onsite laydown area	Large space, different size rebar must be sorted prior to installation
	Weather	Cannot be performed in less than 45 deg. F. without slab heaters

Alternate III: Precast planks on cast-in-place beams

The precast plank idea was first developed when schedule problems began occurring on Charles Commons. This design would have limited flexibility as in post-tensioned slab, but by using less reinforcement. The column spacing will be identical to that of the one-way beam design in which all beams span in perpendicular directions, compromising the architectural aspects of all of the floors of the building.

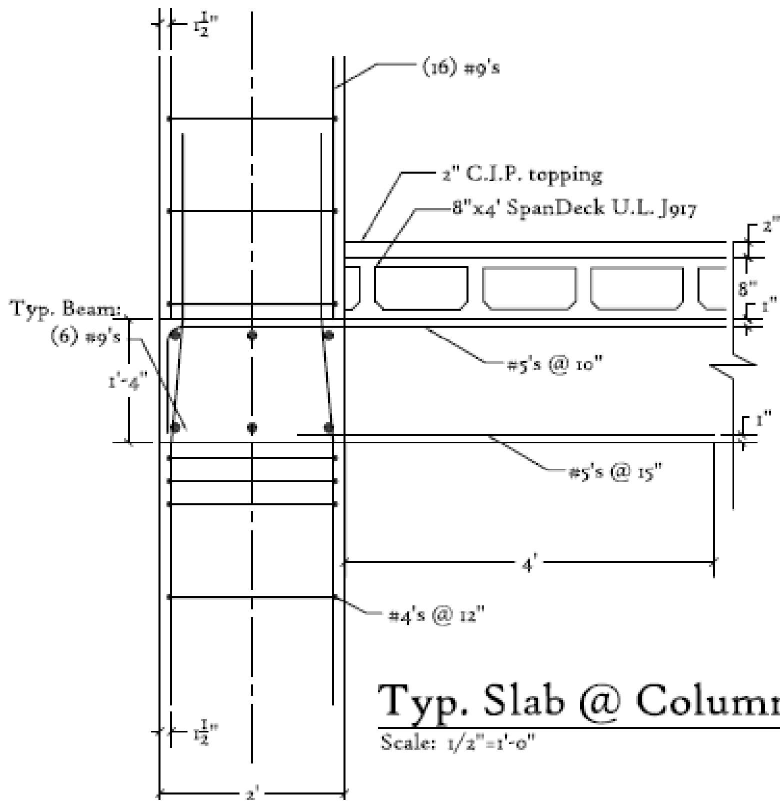
Methodology

For this exercise, a 21' span between two columns was analyzed using the current codes on a spreadsheet. From this data, a trial plank thickness and beam thickness was found and input into E-TABS. Again, a 5x5 column configuration spanning 21' in each direction was duplicated for 10 stories to model this alternate because E-TABS cannot place any beams that aren't perfectly perpendicular or perfectly even opening sizes.

Calculations

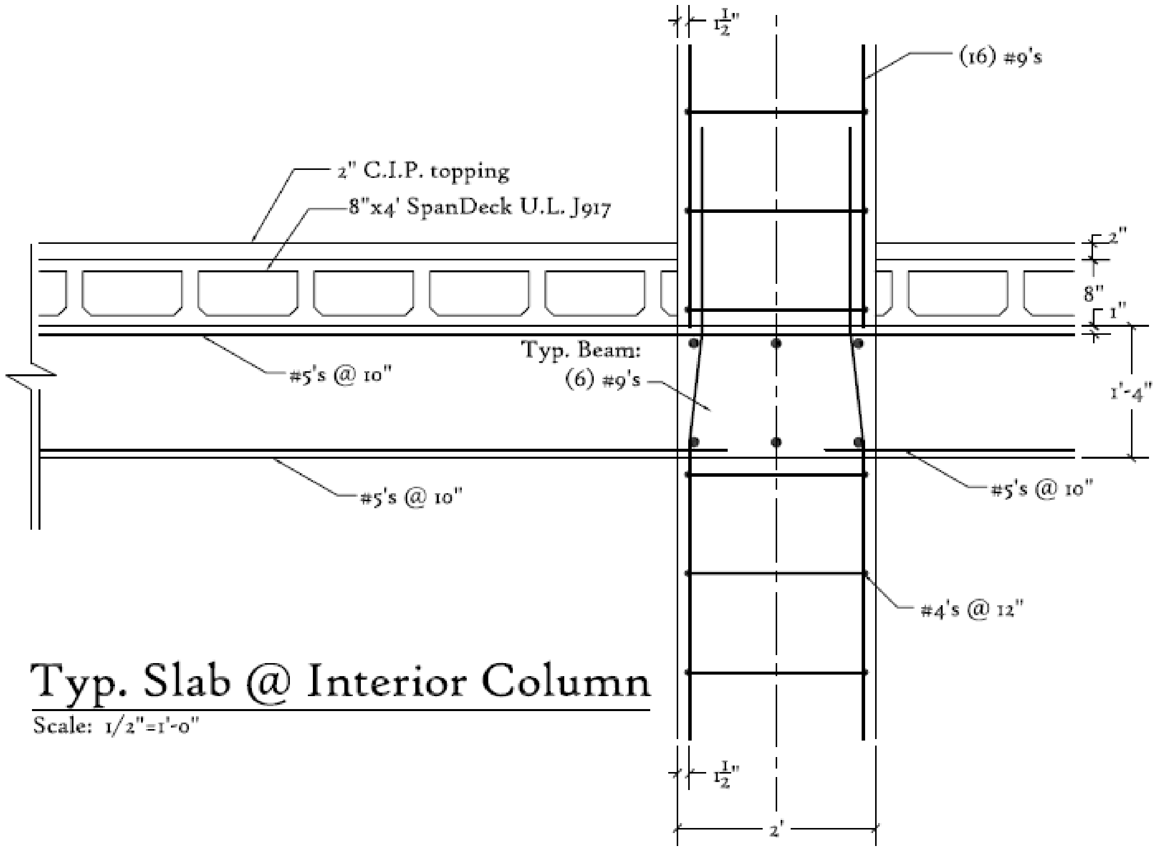
Sizing precast plank is customarily reserved for the manufacturer. Despite this limitation, simple axial load calculations were made and the applicable hollow core plank is Nitterhouse Concrete Products' 8"x4' SpanDeck U.L. J917, 6-strand model. Application of this product requires a 2" concrete topping. Since the specification for this product used allowable superimposed load instead of factored loads, I have included a 2.16 factor of safety at this capacity. The precast planks will be set on top of cast-in-place beams and columns that will be fluted to allow for the precast bearing.

PRECAST PLANK & BEAMS					
beam depth:	**16**	**in**	beam width:	**24**	**in**
slab thickness:	**8**	**in**	column width:	**24**	**in**
f'c:	**5000**	**psi**	column depth:	**24**	**in**
shear depth:	6.75	in	tributary width:	21	ft
span:	21	ft	b for both strips:	10.5	ft
live load:	125	psf			
dead load:	8	psf			
total factored load:	308.6	psf	Nitterhouse Concrete Products		
allowable superimp:	**287**	**psf**	UL J917 8"x4' SpanDeck, 6 strand		
calculated superimp:	**133**	**psf**	plus 2" CIP topping		
F.S:	**2.16**				
weight of precast:	*330*	*plf*			
precast self weight:	*82.5*	*psf*			
flat plate self weight:	*175*	*psf*			
one-way beam weight:	*132.4*	*psf*			



Typ. Slab @ Column/Beam

Scale: 1/2"=1'-0"

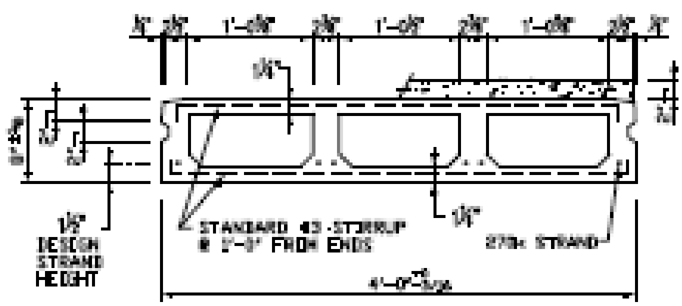


Typ. Slab @ Interior Column
Scale: 1/2"=1'-0"

Prestressed Concrete
8"x4' SpanDeck–U.L.–J917
(2" C.I.P. TOPPING)

PHYSICAL PROPERTIES
Composite

$A' = 254$ in.^{2}	$S'_b = 547$ in.^{3}
$I' = 2944$ in.^{4}	$S'_T = 1124$ in.^{3} (At Top of SpanDeck)
$Y'_b = 5.38$ in.	$S'_{tt} = 637$ in.^{3} (At Top of Topping)
$Y'_t = 2.62$ in. (To Top of SpanDeck)	Wt. = 330 PLF
$Y'_{tt} = 4.62$ in. (To Top of Topping)	Wt. = 82.5 PSF



DESIGN DATA
1. Precast Strength @ 28 days = 5000 PSI.
2. Precast Density = 150 PCF.
3. Strand = 1/2"φ, 270 K Lo-Relaxation.
4. Composite Strength = 3000 PSI.
5. Composite Density = 150 PCF.
6. Strand Height = 1.5 in.
7. Ultimate moment capacities (when fully developed)...
   4 – 1/2"φ, 270K = 94.6'K
   6 – 1/2"φ, 270K = 133.3'K
8. Maximum bottom tensile stress is $6\sqrt{f'c}$ =424 PSI.
9. All superimposed load is treated as live load in the strength analysis of flexure and shear.
10. Flexural strength capacity is based on stress/strain strand relationships.
11. Load values to the left of the solid line are controlled by ultimate strength. Load values to the right are controlled by service stress.
12. Shear values are the maximum allowable before shear reinforcement is required.
13. Deflection limits were not considered when determing allowable loads in this table.
14. All loads shown refer to allowable loads applied after the topping has hardened.

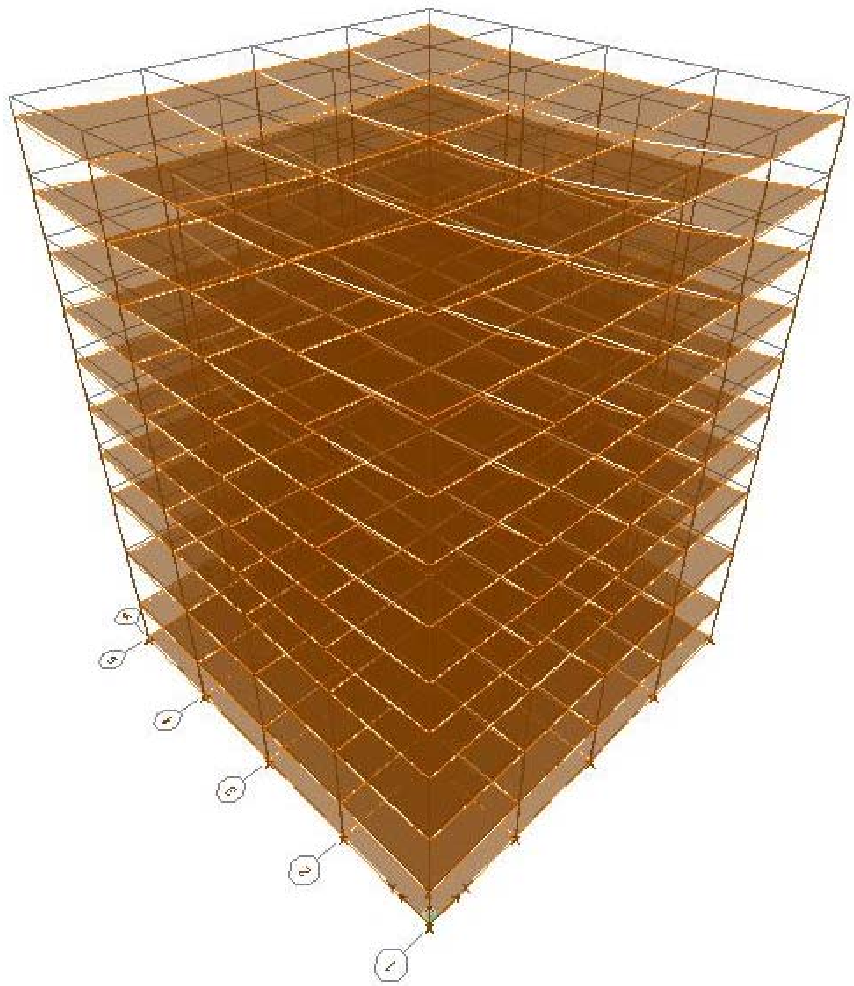
8" SPANDECK W/2" TOPPING	ALLOWABLE SUPERIMPOSED LOAD (PSF)																						
STRAND PATTERN	SPAN (FEET)																						
	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32
Flexure 4 – 1/2"φ	795	718	650	590	500	428	366	317	275	240	210	184	162	142	125	110	96	84	73	60	49	39	X
Shear 4 – 1/2"φ	571	509	458	415	378	347	320	298	275	257	240	222	199	178	160	145	133	126	115	103	93	84	X
Flexure 6 – 1/2"φ	1155	1049	945	859	732	629	544	474	416	366	324	287	256	228	204	183	164	147	132	118	103	90	77
Shear 6 – 1/2"φ	589	525	472	428	391	360	331	308	285	266	249	236	220	207	195	184	175	160	145	132	120	110	100

NITTERHOUSE CONCRETE PRODUCTS

This table is for simple spans and uniform loads. design data for any of these span–load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths.

2655 MOLLY PITCHER HWY. SOUTH, BOX N
CHAMBERSBURG, PA 17201–0813
717-267-4505 • FAX: 717-267-4518

REVISED 11/93



Modeling

In addition to small-scale checks on a long-span of 21', E-TABS modeled six loads in different combinations and analyzed the following:

- Strength-required reinforcement
- Axial forces due to large live loads
- Point deflection investigation for beams

Strength-required reinforcement

The reinforcement required for the columns and beams were calculated based on load combinations of the loads: dead, live, super-imposed dead, and cladding dead load. Consideration of wind and earthquake loads were omitted since this "ideal" condition does not realistically compensate for the differences in the exterior shape of the building (including the courtyard). The beams and the columns were most affected by the 125 psf live loads. In addition, the largest loads were found to be on the interior of the building, which is the opposite of the convention of large exterior loads

found in the models of the flat plate slab and one-way beams. This can only be explained by the pinned nature of the precast planks and the large live load being transferred to the interior columns instead.

Column design

The columns along the exterior of the building and the higher floors of the building were sized as 18”x18” with steel areas averaging 5.76 sq. in. Six #9 bars are specified for these columns. As shown in Appendix B, center sections were taken to display the larger interior columns. The interior columns on the first five floors for the entire structure are sized as 24”x24” and 28”x28”; the larger of which is closest to the center and grade level in the building. The 24”x24” columns require 16 sq. in. of reinforcing (or 16 #9’s) and the 28”x28” columns require 30 sq. in. of reinforcing (or 20 #11’s). Since no moment transfers in pinned connections of the precast hollow-core plank, the exterior of the building is relieved from the moment transfer experienced in the previous two models.

Beam design

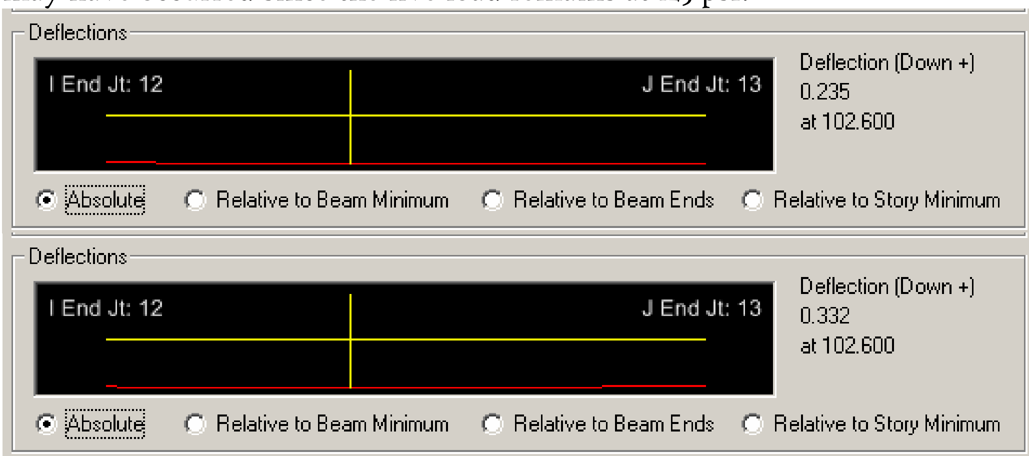
The beams are quite unlike the columns in this design. The capacity attained with a 24”x16” beam specified in the previous spreadsheet was plenty for the precast plank. I attribute this to the large width of the beam and the lighter hollow-core precast planks. These beams require up to 5.73 sq. in. of reinforcing which amounts to 6 #9’s. No substantial torsion or shear reinforcing was specified by the model since the earthquake and wind loads were not considered in this model.

Axial forces

The resulting axial forces were compiled into a diagram. The over-sized columns that resulted from the strength-required reinforcement show the greatest axial forces from the transfer of moment directly to the columns. These diagrams can be found in Appendix B as well.

Point Deflections

Point deflections were not calculated for the precast alternative prior. Despite this, I believe that it would still be prudent to show the maximum deflections for live and dead loads for the beams. Live load deflection is 0.235” and dead load deflections reach 0.332”. The values are the opposite from the findings for the one-way beams in which the live load deflection was larger as calculated. It is unknown for which this may have occurred since the live load remains at 125 psf.



Precast Beams and Columns

Precast beams and columns instead of cast-in-place beams and columns were not considered as an alternative structural system for a few very important reasons. First, the tolerance needed to install precast columns and beams so that they can accept the precast hollow-core planks without resorting to “making it work” or re-ordering the piece is very critical. Since many of the caissons were constructed in the wrong places on Charles building, the chance of losing time is always looming. Secondly, the site is quite small, allowing only for on-time delivery of the precast hollow-core planks. The addition of beams or columns can over-congest the site and require both cranes for the entire project. Also, the lengths of the columns can become too much for the delivery trucks to maneuver in downtown Baltimore. Finally, permits were refused by the City of Baltimore for temporary lane closures, which would undoubtedly be required for such an influx of deliveries.

Precast plank	
Slab Thickness	8” (2” topping)
Concrete Strength	5000 psi
Slab Concrete Volume	0 CY (all precast)
Reinforcement Weight	None, strands in planks
Self-weight	82.5 psf
Building Height	136’-4”
Column Sizes	24”x24”, 18”x18”, 12”x24”
Column Volume	948 CY

Precast plank	Issue	Reason
Advantages	Safety	No special safety considerations
	Complexity	Not difficult to duplicate construction, many contractors perform precast plank structures
	Weather	Can easily be performed in less than 45 deg. F. (with the exception of the cast-in-place columns and beams)
	Equipment	No extra equipment needed
	Structural Code	Does not limit slab thickness
	Rebar Placement	Only in the cast-in-place beams
	Slab Curing	Curing time is only needed for the cast-in-place beams
	Formwork	Requires no edge formwork, only formwork for beams
	Onsite laydown area	On-time delivery is needed for precast beams
	Building Height Requirements	Quite effective in areas with building height limits
Disadvantages	Error	Minimal margin for error, planks must meet tolerance
	MEP Coordination	MEP penetrations need to be planned beforehand. Small core-drills are allowed.
	Labor	Requires subcontractor and personnel experience
	Mistakes	Most problems relating to the precast planks require removal and recasting of whole plank sections

Constructability Review Summary

The previous four tables that describe the fourteen issues that affect each system were compiled into the following table. Each positive outcome to an issue is **green**, each fair outcome is **yellow**, and poor outcomes are shown in **red**. As expected, the less complex alternative systems are easier to construct than post-tensioned systems. The main problem with this analysis is its inability to quantify these issues into tangible cost and schedule impacts. If these issues could be quantified, it would be obvious that each alternative system saves over the existing post-tensioned system.

Issue	Post-Tensioning	Flat-Plate	One-Way Beams	Precast Plank
Safety				
Complexity				
Weather				
Equipment				
Structural Code				
Rebar Placement				
Slab Curing				
Formwork				
Onsite Laydown Area				
Building Height				
Error				
MEP Coordination				
Labor				
Mistakes				

Schedule Reduction

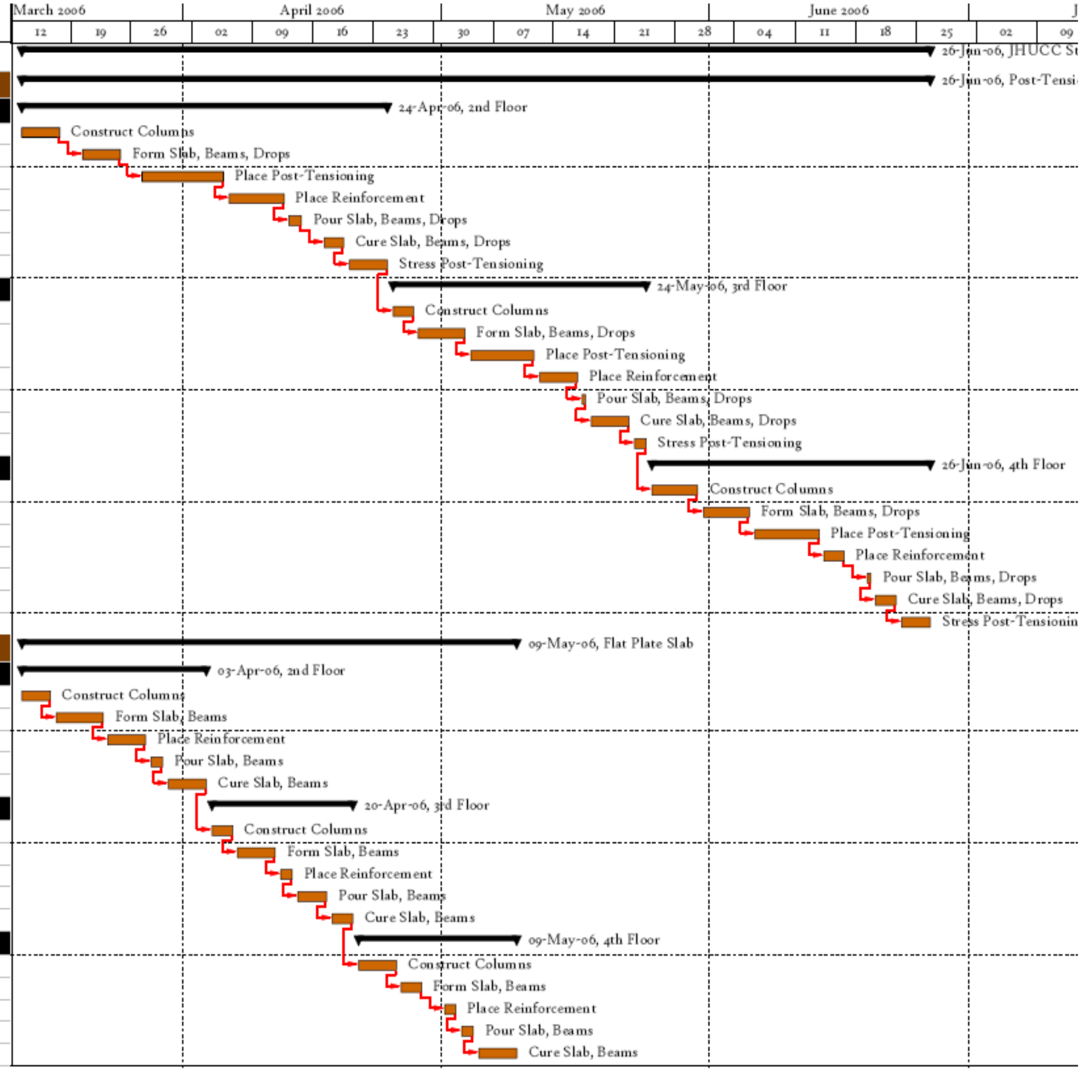
The impact of these different concrete structural systems to the sequencing of Charles Commons is not a very big issue. Since all of these systems incorporate the same (or slightly larger) cast-in-place foundation, columns, and edge beams into their construction, essentially all of the sequencing issues lie with the construction of the slab (or additional beams). The on-time delivery of precast planks can be handled by the dual tower cranes onsite since there is less reinforcing to handle onsite. The one-way beams and flat plate slab requires more reinforcing and concrete, but they do not need post-tensioning cables stored onsite. All of the alternatives have small sequencing issues related, but these issues are not comparable to the issues experienced with post-tensioning.

The structure of St. Paul is on the critical path of the project and any schedule savings found here can help the project get back on track. These alternative systems also have design and coordination schedule savings, however, these are difficult or impossible to consider from a structural stand-point. Later in this thesis report, the time allocated for MEP Coordination will be analyzed for each of these structural alternates.

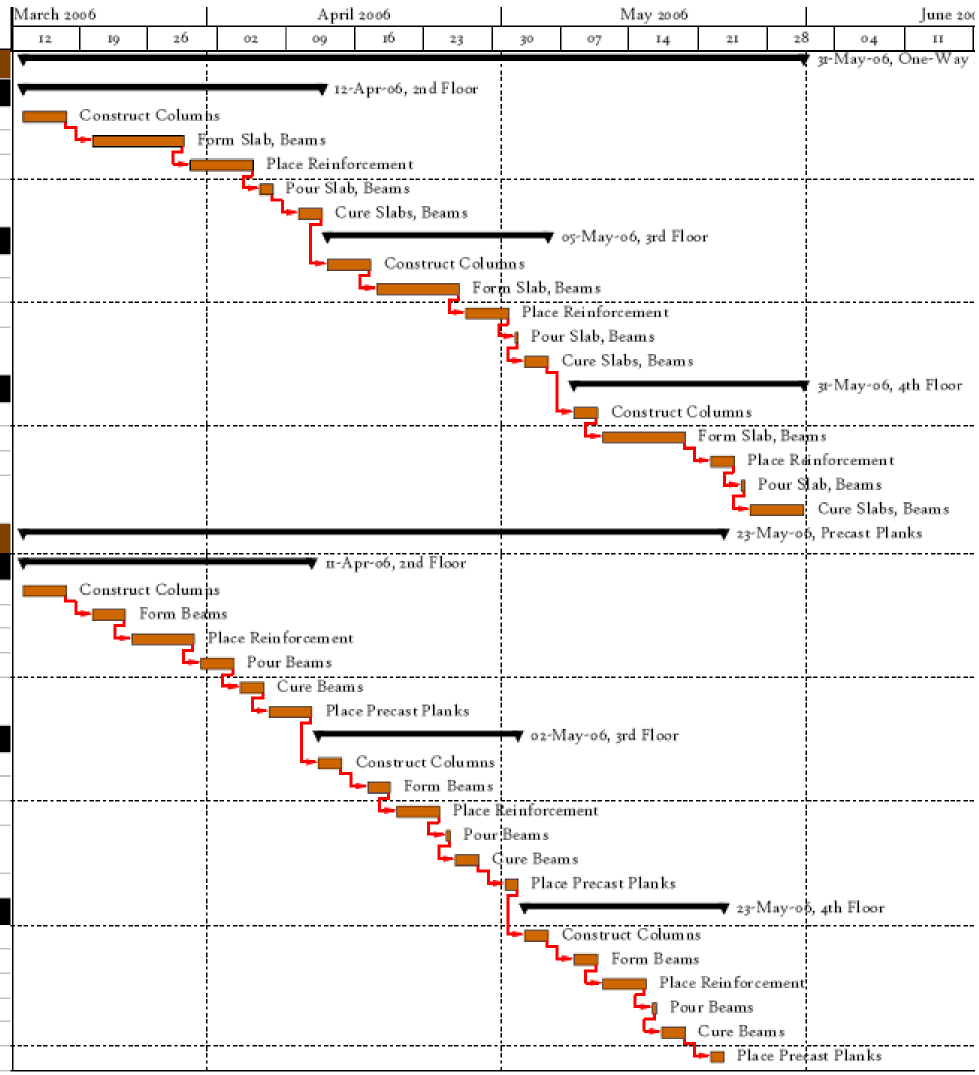
The following schedule shows how long it takes to complete the first three floor slabs as compared to the three months taken to complete the post-tensioned slabs. It is assumed that the concrete contractor will man the job similarly with all alternative systems. The standard work week in this exercise is 16 hours a day, 7 days a week. Approximately 180 men are present on each shift for the concrete contractor. Productivity losses associated with approximately 12 crews and 180 men are assumed to be 25%, or the shift's total production amounts to 135 men. Also, a 14% increase in labor is assumed for the one-and-a-half overtime work completed over the weekends. Since there is a learning-curve associated with concrete construction, the first floor is adjusted to take 150% longer than what has been calculated.

The schedule calculates the length of all of the activities if they were completed one-after-another. Some overlapping will occur with these activities, but estimating this is purely academic. The savings on this 3-floor schedule is broken into an individual floor savings and multiplied by ten to represent the savings over the entire St. Paul structure. This savings is shown on the cost estimate to calculate the reduced general conditions.

vity ID	Activity Name	Original	Start	Finish
JHUCC Structure		106	13-Mar-06	26-Jun-06
Post-Tensioned Slab		106	13-Mar-06	26-Jun-06
2nd Floor		43	13-Mar-06	24-Apr-06
A1000	Construct Columns	5	13-Mar-06	17-Mar-06
A1010	Form Slab, Beams, Drops	5	20-Mar-06	24-Mar-06
A1020	Place Post-Tensioning	8	27-Mar-06	05-Apr-06
A1030	Place Reinforcement	5	06-Apr-06	12-Apr-06
A1040	Pour Slab, Beams, Drops	2	13-Apr-06	14-Apr-06
A1050	Cure Slab, Beams, Drops	3	17-Apr-06	19-Apr-06
A1060	Stress Post-Tensioning	3	20-Apr-06	24-Apr-06
3rd Floor		30	25-Apr-06	24-May-06
A1070	Construct Columns	3	25-Apr-06	27-Apr-06
A1080	Form Slab, Beams, Drops	4	28-Apr-06	03-May-06
A1090	Place Post-Tensioning	6	04-May-06	11-May-06
A1100	Place Reinforcement	3	12-May-06	16-May-06
A1110	Pour Slab, Beams, Drops	1	17-May-06	17-May-06
A1120	Cure Slab, Beams, Drops	3	18-May-06	22-May-06
A1130	Stress Post-Tensioning	2	23-May-06	24-May-06
4th Floor		33	25-May-06	26-Jun-06
A1140	Construct Columns	3	25-May-06	30-May-06
A1150	Form Slab, Beams, Drops	4	31-May-06	05-Jun-06
A1160	Place Post-Tensioning	6	06-Jun-06	13-Jun-06
A1170	Place Reinforcement	3	14-Jun-06	16-Jun-06
A1180	Pour Slab, Beams, Drops	1	19-Jun-06	19-Jun-06
A1190	Cure Slab, Beams, Drops	3	20-Jun-06	22-Jun-06
A1200	Stress Post-Tensioning	2	23-Jun-06	26-Jun-06
Flat Plate Slab		58	13-Mar-06	09-May-06
2nd Floor		22	13-Mar-06	03-Apr-06
A1210	Construct Columns	4	13-Mar-06	16-Mar-06
A1220	Form Slab, Beams	4	17-Mar-06	22-Mar-06
A1230	Place Reinforcement	3	23-Mar-06	27-Mar-06
A1240	Pour Slab, Beams	2	28-Mar-06	29-Mar-06
A1250	Cure Slab, Beams	3	30-Mar-06	03-Apr-06
3rd Floor		17	04-Apr-06	20-Apr-06
A1260	Construct Columns	3	04-Apr-06	06-Apr-06
A1270	Form Slab, Beams	3	07-Apr-06	11-Apr-06
A1280	Place Reinforcement	2	12-Apr-06	13-Apr-06
A1290	Pour Slab, Beams	2	14-Apr-06	17-Apr-06
A1300	Cure Slab, Beams	3	18-Apr-06	20-Apr-06
4th Floor		19	21-Apr-06	09-May-06
A1310	Construct Columns	3	21-Apr-06	25-Apr-06
A1320	Form Slab, Beams	3	26-Apr-06	28-Apr-06
A1330	Place Reinforcement	2	01-May-06	02-May-06
A1340	Pour Slab, Beams	2	03-May-06	04-May-06
A1350	Cure Slab, Beams	3	05-May-06	09-May-06



Activity ID	Activity Name	Original	Start	Finish
One-Way Beams		**80**	**13-Mar-06**	**31-May-06**
2nd Floor		**31**	**13-Mar-06**	**12-Apr-06**
A1360	Construct Columns	5	13-Mar-06	17-Mar-06
A1370	Form Slab, Beams	8	20-Mar-06	29-Mar-06
A1380	Place Reinforcement	5	30-Mar-06	05-Apr-06
A1390	Pour Slab, Beams	2	06-Apr-06	07-Apr-06
A1400	Cure Slabs, Beams	3	10-Apr-06	12-Apr-06
3rd Floor		**23**	**13-Apr-06**	**05-May-06**
A1410	Construct Columns	3	13-Apr-06	17-Apr-06
A1420	Form Slab, Beams	7	18-Apr-06	26-Apr-06
A1430	Place Reinforcement	3	27-Apr-06	01-May-06
A1440	Pour Slab, Beams	1	02-May-06	02-May-06
A1450	Cure Slabs, Beams	3	03-May-06	05-May-06
4th Floor		**24**	**08-May-06**	**31-May-06**
A1460	Construct Columns	3	08-May-06	10-May-06
A1470	Form Slab, Beams	7	11-May-06	19-May-06
A1480	Place Reinforcement	3	22-May-06*	24-May-06
A1490	Pour Slab, Beams	1	25-May-06	25-May-06
A1500	Cure Slabs, Beams	3	26-May-06	31-May-06
Precast Planks		**72**	**13-Mar-06**	**23-May-06**
2nd Floor		**30**	**13-Mar-06**	**11-Apr-06**
A1510	Construct Columns	5	13-Mar-06	17-Mar-06
A1520	Form Beams	4	20-Mar-06	23-Mar-06
A1530	Place Reinforcement	5	24-Mar-06	30-Mar-06
A1540	Pour Beams	2	31-Mar-06	03-Apr-06
A1550	Cure Beams	3	04-Apr-06	06-Apr-06
A1560	Place Precast Planks	3	07-Apr-06	11-Apr-06
3rd Floor		**21**	**12-Apr-06**	**02-May-06**
A1570	Construct Columns	3	12-Apr-06	14-Apr-06
A1580	Form Beams	3	17-Apr-06	19-Apr-06
A1590	Place Reinforcement	3	20-Apr-06	24-Apr-06
A1600	Pour Beams	1	25-Apr-06	25-Apr-06
A1610	Cure Beams	3	26-Apr-06	28-Apr-06
A1620	Place Precast Planks	2	01-May-06	02-May-06
4th Floor		**21**	**03-May-06**	**23-May-06**
A1630	Construct Columns	3	03-May-06	05-May-06
A1640	Form Beams	3	08-May-06	10-May-06
A1650	Place Reinforcement	3	11-May-06	15-May-06
A1660	Pour Beams	1	16-May-06	16-May-06
A1670	Cure Beams	3	17-May-06	19-May-06
A1680	Place Precast Planks	2	22-May-06	23-May-06



Value Engineering

EXISTING POST-TENSIONING

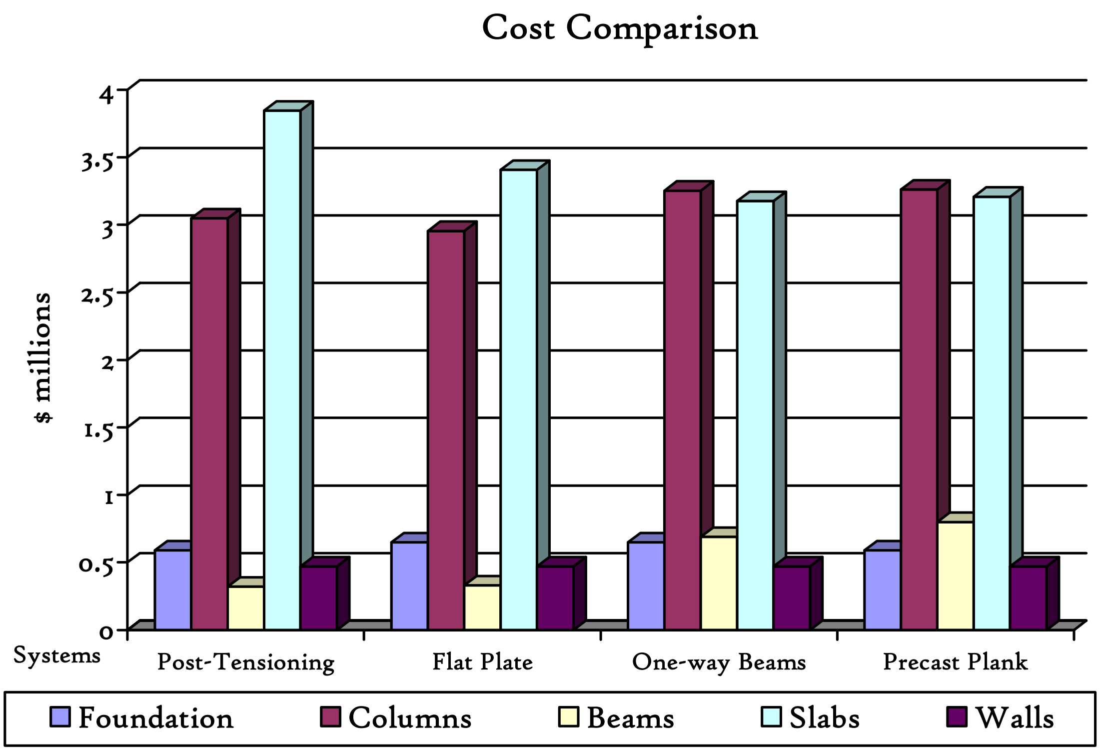
	Estimate	Projected	Bid		
Foundation				Total Building:	$54,310,854
Drilled Caissons	$471,916	$589,895	$471,916	Total St. Paul:	$35,845,164
Caisson Caps	$33,116	$33,116	$33,116	St. Paul per/ft heig	$266,177
Grade Beams	$35,626	$35,626	$35,626	Floors 1-4 Schedule	106 days
Footings	$49,649	$49,649	$49,649	General Conditions	$4,660,184
Foundation Total:	$590,307	$708,286	$590,307	16 month schedule	
				Gen Cond/month:	$291,262
Superstructure					
Concrete Columns	$3,047,695	$3,868,186	$2,536,298		
Concrete Beams	$324,555	$324,555	$324,555		
Shearwalls	$471,458	$471,458	$471,458		
Concrete Slabs	$3,845,175	$4,421,951	$3,359,948		
Superstructure Total:	$8,227,105	$9,086,151	$6,692,259		
Subtotal:	$8,817,412	$9,794,437	$7,282,566		
Location Factor of 92.7					
10% Markup	$8,991,115	$9,987,387	$7,426,033		
Coordination Allowance	$287,220	$350,000	$287,220		
Historical Cost Index	$8,424,728	$9,386,348			
Total:	**$8,711,948**	**$9,736,348**	**$7,713,253**		
	11%	**21%**			

ALT 1 FLAT PLATE

	Estimate	Assumptions		
Foundation			Added Height:	5 ft
Drilled Caissons	$519,108	10% increase	Added Height Cost	$1,330,884
Caisson Caps	$36,427	10% increase	Floors 1-4 Schedule	58 days
Grade Beams	$39,189	10% increase	Schedule Savings:	3.5 mo
Footings	$54,614	10% increase	Gen Cond Savings:	$1,019,415
Foundation Total:	$649,338	increase due to more weight		
Superstructure				
Concrete Columns	$2,953,637	add sizes, deduct strength		
Concrete Beams	$331,046	added beam sizes	Systems Difference	$311,469
Shearwalls	$471,458	same		
Concrete Slabs	$3,406,986	added concrete, formwork, rebar and deduct strength, PT		
Superstructure Total:	$7,664,546			
Subtotal:	$8,313,884			
Location Factor of 92.7				
10% Markup	$8,477,668			
Coordination Allowance	$0			
Systems Difference:	$311,469			
Total:	**$7,980,536**			
Difference:	**-$731,412**	**-8.40%**	compared to PT estimate	

ALT 2	ONE-WAY BEAMS			
Foundation	Estimate	Assumptions	Added Height:	3.333 ft
Drilled Caissons	$519,108	10% increase	Added Height Cost	$887,168
Caisson Caps	$36,427	10% increase	Floors 1-4 Schedule	80 days
Grade Beams	$39,189	10% increase	Schedule Savings:	2.0 mo
Footings	$54,614	10% increase	Gen Cond Savings:	$582,523
Foundation Total:	$649,338	increase due to more weight		
Superstructure				
Concrete Columns	$3,252,089	add sizes, ded. strength		
Concrete Beams	$692,046	add beams	Systems Difference	$304,645
Shearwalls	$471,458	same		
Concrete Slabs	$3,174,611	added concrete, formwork, rebar and deduct strength, PT		
Superstructure Total:	$8,121,519			
Subtotal:	$8,770,856			
Location Factor of 92.7				
10% Markup	$8,943,642			
Coordination Allowance	$71,805			
Systems Difference:	$304,645			
Total:	**$8,462,643**			
	-$249,305	**-2.86%**	compared to PT estimate	

ALT 3	PRECAST PLANK			
Foundation	Estimate	Assumptions	Added Height:	1.67 ft
Drilled Caissons	$471,916	no increase	Added Height Cost	$444,515
Caisson Caps	$33,116	no increase	Floors 1-4 Schedule	72 days
Grade Beams	$35,626	no increase	Schedule Savings:	2.2 mo
Footings	$49,649	no increase	Gen Cond Savings:	$640,775
Foundation Total:	$590,307	no increase due to lightweight hollow plank		
Superstructure				
Concrete Columns	$3,260,746	add bearing, deduct weight		
Concrete Beams	$795,853	add beam sizes	Systems Difference	-$196,260
Shearwalls	$471,458	same		
Concrete Slabs	$3,207,696	precast planks		
Superstructure Total:	$7,967,826			
Subtotal:	$8,558,133			
Location Factor of 92.7				
10% Markup	$8,726,728			
Coordination Allowance	$287,220			
Systems Difference:	-$196,260			
Total:	**$8,006,461**			
	-$705,488	**-8.10%**	compared to PT estimate	



Structural Conclusions

The redesign of the structural slabs for the St. Paul building is meant to find a system in which the project team has the most likely chance of success. After research in the ACI code, countless concrete books, and PCA online design examples, I made a spreadsheet for each system's design limit. Recommendations from these spreadsheets were fed into E-TABS, where models were created to check the reality of my calculations on the structure. Since axial forces and deflections were found to control the spreadsheet's output, these were used to design the slabs, beams, and columns for the St. Paul building.

Many different issues have been analyzed to make comparisons between the existing system and three alternatives. Quantitative analyses of the cost and schedule impacts show that the flat-plate slab and precast plank alternatives are the least expensive and take nearly ½ of the time required by the existing system. Qualitative analyses, such as the constructability review, were made for each system, in which all of the alternatives were found to be the best and the existing system was ranked worst. Therefore, the two best structural systems for this project from a structural standpoint are precast plank and flat-plate. Since these two systems are quite structurally comparable to each other, the limitations on the ceiling plenum will factor into the analysis.