University of North Carolina's

Imaging Research Building

Technical Report 2

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Structural Option

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Executive Summary

This report was created to be a pro-con study of the existing floor system, and three additional floor systems. Each option was examined using a typical 21'-3" by 31'-4" bay as the basis for analysis. The existing floor system is a 6" one-way cast-in-place slab with #4@12 top and bottom for the typical interior bay. An overview of this system and its advantages and disadvantages are provided in the report.

The three alternate systems that were analyzed were:

- Non-composite steel framing
- Composite steel framing
- Hollow core precast concrete on steel beams

The non-composite steel framing was designed using the AISC 13th edition Steel Construction Manual and the Vulcraft Steel Roof Floor Deck Guide. The design for the typical interior bay was composed of 2C18 metal deck with a 6" slab, W16X31 beams, and W24X76 girders. The composite steel framing was designed using RAM Structural System and Vulcraft 2VL composite deck. It was found that W10X12's with (14) ¾" shear studs and a ¾" camber would work for the beams, and W21x44's with (50) shear studs a ¾" camber would work for the girders. The 4'-0" x 6" hollow core precast plank with 2" topping were selected from the PCI Design Handbook and the supporting girders were determined to be W24X76's when optimized, the same as the non-composite floor system.

The advantages and disadvantages of each of the floor systems were discussed for each framing system, and it was determined that both the non-composite steel framing and the hollow core system were not feasible due to several reasons outlined in the report. However, the composite steel system was determined to be a viable option for future exploration. It's lightweight, not as labor intensive as a cast-in-place concrete slab, and therefore relatively cost effective to construct. The one major disadvantage though is that on the lower levels, stainless steel would need to be used for the framing due to the magnetic interference with the imaging equipment. This problem can be overcome though if a concrete base is used for the subfloors (where the imaging equipment is located), and a steel frame system is used for the floors above.

Introduction

This pro-con structural study of alternate floor systems examines the existing floor framing of the University of North Carolina Imaging Research building that was designed by Mulkey Engineers and Consultants, and analyzes three other possible systems. The existing design is a 6" one-way cast-in-place concrete slab while the alternate systems that were studied include non-composite steel framing, hollow core precast planks on steel beams, and a two-way cast-in-place slab. Gravity loads determined in Technical Report 1 were used in the design to help determine slab thicknesses, member sizes and necessary reinforcement. The main focus of this report is compare and contrast the advantages and disadvantages concerning constructability, system depth, system weight, fire protection, cost, and various other criteria to determine which systems may be possible topics for the structural proposal required by Senior Thesis.

The relevant codes used for this analysis are:

Codes & Design Standards

Applied to original design:

2009 North Carolina State Building Code (2006 International Building Code with revisions)

American Concrete Institute (ACI 318-05), Building Code Requirements for Structural Concrete

Substituted for thesis analysis:

American Society for Civil Engineers (ASCE 7-05), Minimum Design Loads for Buildings and Other Structures, 2005

American Concrete Institute (ACI 318-08), Building Code Requirements for Structural Concrete

Material Strength Requirement Summary:

Concrete/Reinforcing Steel (28 day compressive strength)

- Elevated Slabs on Metal Deck: 3500 psi
- Elevated Slabs and Beams: 5000 psi
- Columns, Shear Walls: 7000 psi
- Basement Walls, Site Walls: 7000 psi
- Slab on Grade, Footings, Grade Beams: 4000 psi
- Reinforcement: 60 ksi

Architectural Design Concepts

The Imaging Research Building at UNC Chapel Hill was designed by the architecture firm Perkins + Will. Its primary usage is the driving force behind many of the structural decisions for the project. Once it is open, it will contain the most advanced imaging equipment in any one spot in the world. First, the two subgrade floors house several heavy pieces of imaging research equipment that have large Gaussian fields. Because of this, foundations, walls, and slabs were made thicker than usual, which will result in the use of mass concrete pouring techniques to be required when constructed. For example, the foundation where a 1.5GHZ NMR machine will sit required a 6' thick mat footing.

Above grade you will find typical bays sizes of 21'-4" by 21'-4", and 21'-4" by 31'-4" driven by the laboratory space requirements on every floor. A bridge also connects the new imaging research facility to existing Lineberger Cancer Center on the second floor. At the eighth floor, a large area houses all of the mechanical equipment with a partial mezzanine at the floor above, which services all of the imaging and laboratory equipment below. These architectural and usage restraints have a generous effect on the structural system as noted below, and hopefully in future technical reports.

Structural System

Foundation

The geotechnical engineering study was performed by Tai and Associates on November 12, 2008. The study indicates that the subsurface materials on the site consist of pavement and topsoil, fill, residual soil, weathered rock, and rock and boulders. Based on this composition, Tai and Associates were confident in giving Mulkey a net allowable bearing pressure of 6000 pounds per square foot to use in their foundation calculations.

Because of this allowable bearing pressure, Mulkey had to be creative with their foundation design. The result is a mixture of spread footings under the columns, and a combination of spread and mat footings under the large imaging research equipment and shear walls. The walls below grade range from 18" to 36" in thickness¸ and in one location a 36" wall spans both subgrade floors to the first floor unbraced. An example of a typical mat footing can be seen in Figure 1.1. As with the other mat footings, this one is combined and sits under two pieces of large imaging equipment. It is 6'-0" thick and also services a shear wall that steps 6' in elevation. Another area of note in the foundation design is a 6'-0" thick concrete footing which will service a cyclotron, another heavy piece of imaging equipment.

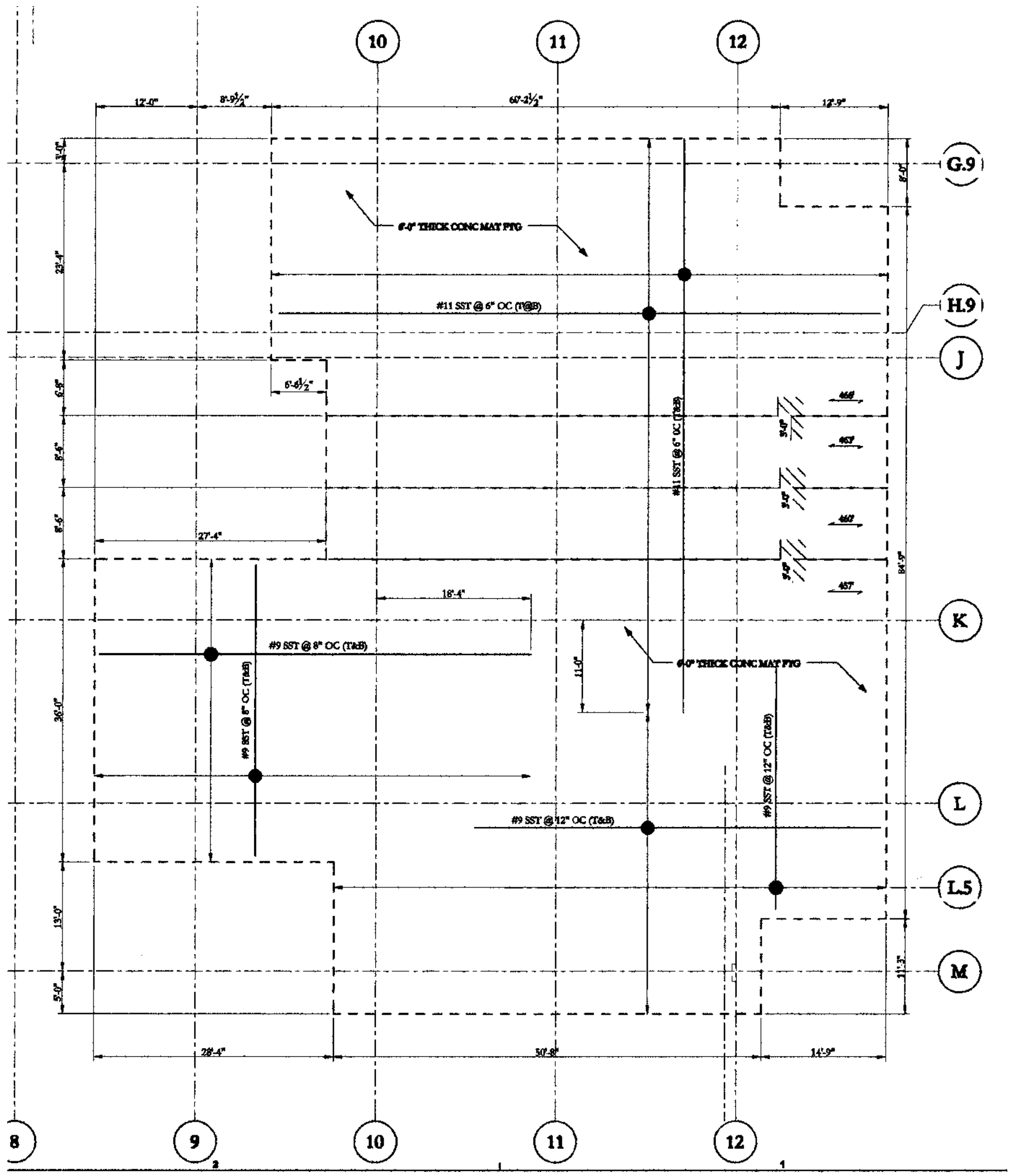


Figure 1.1 – Typical Mat Foundation under Imaging Equipment

Superstructure

The first floor and the floors above to the eighth floor is a 6" one-way cast-in-place slab (NWC) with a compressive strength (f'c) of 5 ksi. The beams on these levels are mostly 18"x20" T-Beams, which change directions at the re-entrant corner where the building changes directions. The girder dimensions vary, but are typically 28"x30".

Most of the columns in the Imaging Research Building are 20"x20" square columns with #3 ties above the first floor, and 24"x24" below grade, with all them having a compressive strength of 7 ksi. The typical frame consists of four bays with three of them being approximately twenty feet in width and the other being thirty feet in width to accommodate the laboratories that occupy these spaces on almost every floor of the building.

For more detail on the superstructure a section of the third floor framing is provided in figure 2.1 for reference.

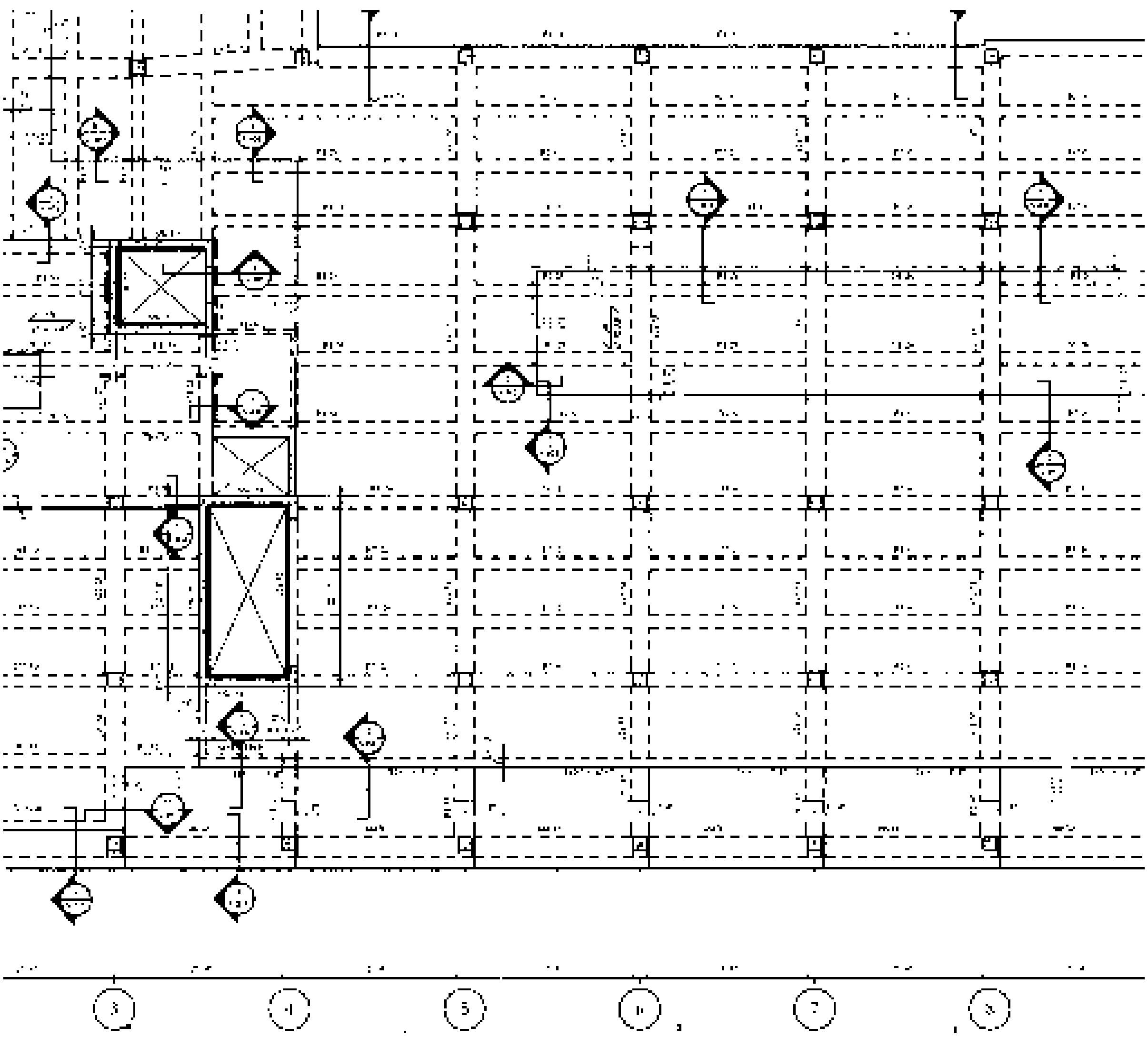


Figure 1.2 - Third Floor Framing

Lateral System

Ordinary reinforced concrete shearwalls are used as the lateral force resisting system in the UNC Imaging Research Building. The largest shearwalls are wrapped around the main elevator and stairwell cores while the other ones encase mechanical closets. Most of the shearwalls exist from the mechanical mezzanine to the foundation with others picking up in between. There are forty-one shearwalls either 12" or 16" thick. Figure 1.3 shows an example of the shearwalls around the main stair and elevator core, while Figure 1.4 is an example of a typical shearwall elevation.

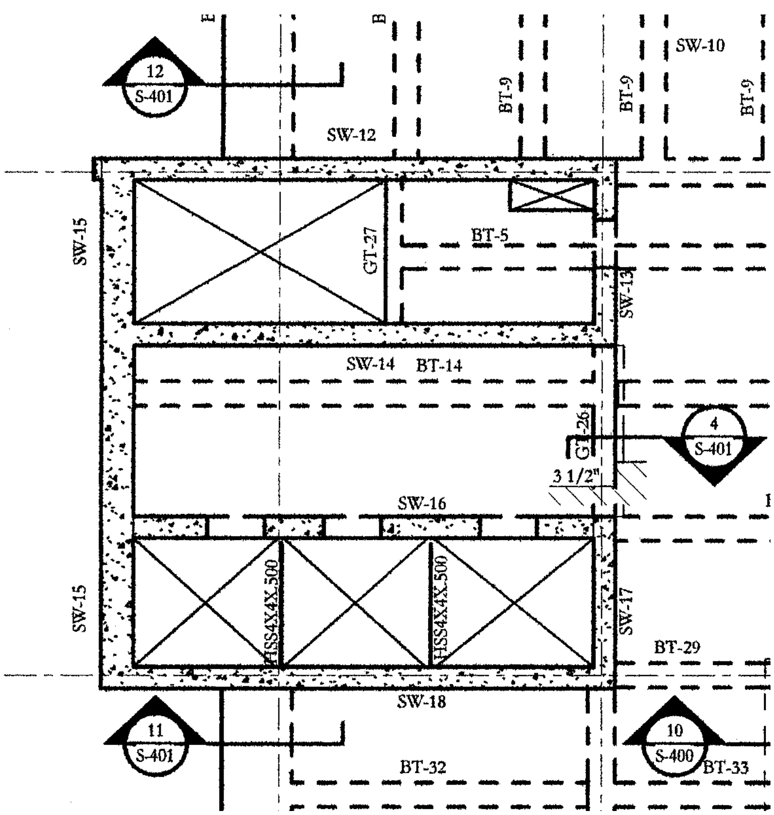


Figure 1.3 - Shearwalls around Elevator Core

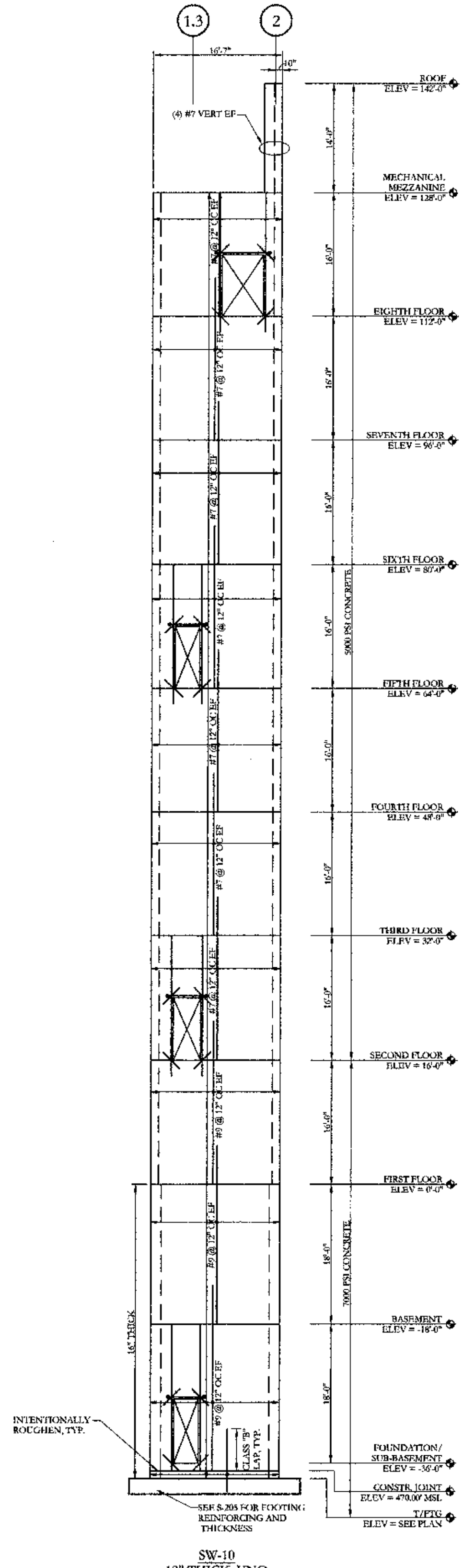


Figure 1.4 - Typical Shearwall Elevation

Loads

Gravity Loads

The determination of gravity loads by Mulkey Engineers and Consultants was done using the 2009 North Carolina State Building Code (2006 International Building Code with Revisions), which adopts ASCE 7-05 for its minimum design loads for buildings. This report also uses ASCE 7-05 as the main reference in accordance with the requirements of AE Senior Thesis. In several places, Mulkey chose to use higher design loads than what was stipulated by the building code. These differences along with the rest of the design loads are noted in the Mulkey column of Table 1, while the code loads are in the ASCE 7-05 column. Calculations of the snow load are provided in Appendix A.

Table 1 -Gravity Loads		
Description	Mulkey	ASCE 7-05
DEAD (DL)		
Reinforced Normal Weight Concrete	150 pcf	150 pcf
LIVE (LL)		
Roof	30 psf	20 psf
Offices	50 psf	50 psf
Public Areas, Lobbies	100 psf	100 psf
Laboratories	100 psf	60 psf
Corridors, 2nd & Above	100 psf	100 psf
Corridors Ground	100 psf	100 psf
Stairs	100 psf	100 psf
Catwalk	40 psf	40 psf
Storage	125 psf	125 psf
Heavy File Storage	200 psf	250 psf
Mechanical Rooms	150 psf	150 psf
Level B1	150 psf	N/A
SNOW (S)		
Snow	16.5 psf	16.5 psf
SUPERIMPOSED (SDL)		
Finishes, MEP, Partitions	25 psf	25 psf
Bathroom Terrazo	40 psf	N/A
Lobby Terrazo	60 psf	N/A
Mechanical Courtyard	300 psf	N/A
3T MRI Room	250 psf	N/A
7T Sheilding	75 psf	N/A
Hot Cells	350 psf	N/A
Water Tank	350 psf	N/A

Floor Systems

One-Way Reinforced Cast-In-Place – Existing

Material Properties:

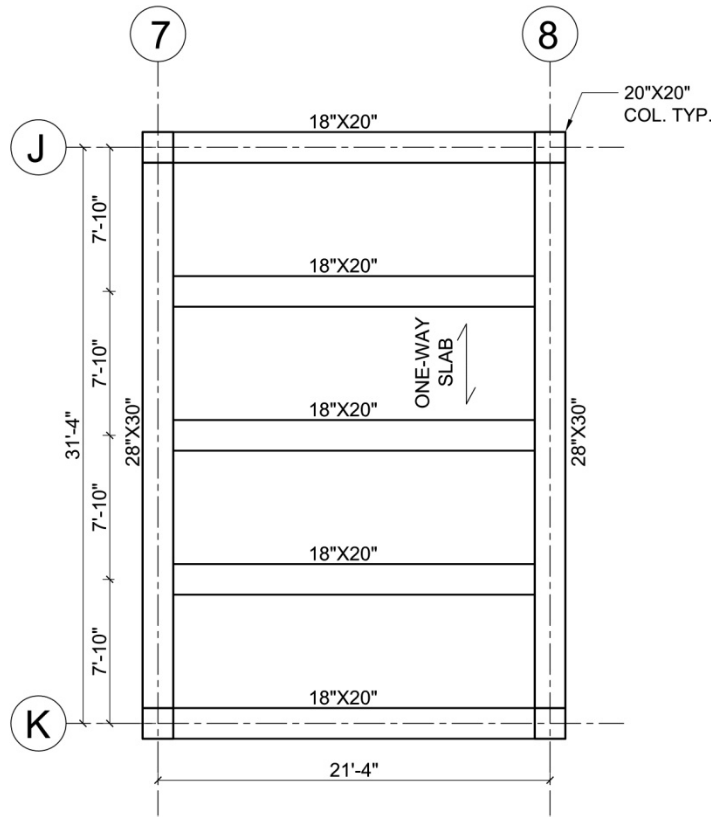
Concrete:	6” slab (NWC)
	20”x20” columns
	f’c = 5000 psi
Reinforcement:	fy = 60,000 psi

Loading:

Dead (self weight):	75 psf
Live:	100 psf
Superimposed:	25 psf

Description

This one-way reinforced cast-in-place floor system designed by Mulkey Engineering is a 6” NWC slab with ______________. The typical interior bay that was considered for this analysis features 18”x20” beams at 7’-10” on center and 28”x30” girders. Because of the size of the floorplan of the building, a detail of the bay analyzed is shown in figure ___.



An analysis for this floor system was done at the ___ floor using RAM Structural System software. Included for this technical report from the RAM output are the stresses and code checks while basic calculations to check minimum thickness for deflection control are done by hand. Both the computer output and hand calculations can be found in Appendix ___.

Advantages

The one-way cast-in-place slab is a simple floor system to design and construct. Therefore, it is relatively inexpensive both in design and construction. Also, it is works for heavier live loads as in the Imaging Research Building because there is very little deflection when used in combination with beams. But more importantly, penetrations in the slab cause few structural problems because there is not a lot of large rebar or tendons running through the slab, and it is easy to reinforce around them after they have been created. This is very important on a job like the Imaging Research Building where there are a number of mechanical systems and equipment lines for the imaging laboratory equipment penetrating through the floors. Therefore, the one-way cast-in-place slab was a logical choice for Mulkey.

Disadvantages

While there are some obvious advantages that make the one-way floor slab a logical choice for the Imaging Research Building, there are a couple disadvantages to it as well. First, since it is a cast-in-place beam and slab system it's going to require a lot of formwork that will be time consuming and costly. This results in a longer construction schedule which will delay the opening of the building. Also, the one-way floor system is typically a deeper floor system then some of its concrete counterparts. The two-way flat plate, and flat slab systems have a smaller overall depth to them.

Another disadvantage is the quality of concrete work that can be expected. After speaking with several individuals who have years of experience designing structures in the south, I have found that it is common judgment that the quality of concrete placement in the south is inferior to that above the Mason-Dixon Line. While obviously not a make or break factor, it is one that must be considered none-the-less.

Non-Composite deck on steel – Option #1

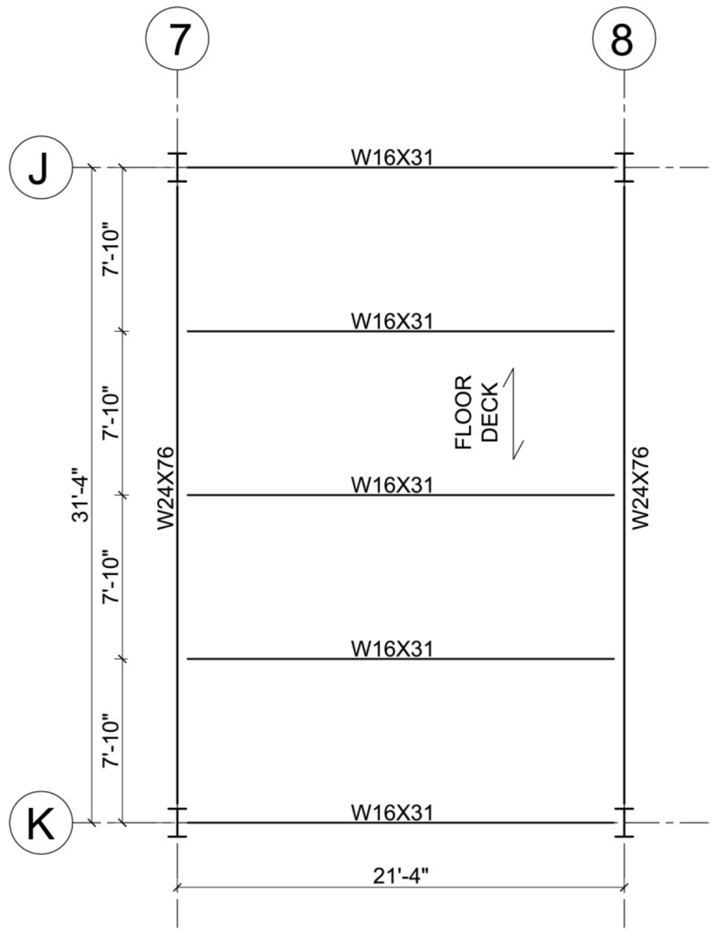
Material Properties:

Concrete:	6" slab (2" deck with 4" of concrete)
	f'c = 3000 psi
Steel:	fy = 50,000 psi
Reinfrocement:	fy = 60,000 psi
Metal Deck:	2C18 – 3 span

Loading:

Dead (self weight):	75 psf
Live:	100 psf
Superimposed:	25 psf

Description



This non-composite steel system was designed using a typical interior bay of 21'-4" by 31'-4" with beams spaced 7'-10" on center as in the original concrete framing system. With a 3-span condition the Vulcraft 2C18 non-composite deck is able to span 11'-7" during construction, which is greater than the 7'-10" spacing proposed for this framing layout. The 2", 18 gauge deck is also topped with 4" of concrete for a 6" total slab depth. According to the allowable uniform load table in the Vulcraft manual, this system satisfies the system satisfies the bending stress and deflection limit design criteria given.

Calculations for this system were done using the AISC thirteenth edition *Steel Construction Manual* and RISA-3D. The steel manual was used to determine the sizes for the beams and girders and for efficiency of time RISA was used to determine the deflections of the girders to check that they did not exceed the deflection limits.

It was found that that the beams were controlled by the total load deflection of the system, and the girders were controlled by their moment capacity. The calculations supporting these findings can be found in Appendix B.

Advantages

There are several advantages to a steel frame system with a non-composite deck over a cast-in-place concrete slab. First, a steel frame system has a quicker erection time because it can arrive at the site prefabricated and there is no need for formwork. The lack of formwork will also reduce the cost of labor, although this is not as big of deal in North Carolina because it not unionized like it is up North. Another advantage is that it is that decking is able to span 11’-7” during construction; therefore there will be no need for shoring. Finally, although a composite system was not looked at (and there are merits to a composite system), the non-composite system will also save money due to the absence of shear studs.

Disadvantages

The biggest disadvantage to this system in regards to the Imaging Research Building is that on the lower floors where there are a number of pieces of imaging equipment, the steel used would have to be stainless steel. This is due to the fact that the imaging equipment is magnetized and any ferrous material used can disrupt the magnetic field and ruin the equipment. Therefore using a steel frame system for the lower floor would be a large cost increase considering as of September 2009, the North American stainless steel price was 2945 US$/tonne compared to the carbon steel price of 680 US$/tonne.

Besides the cost increase for having to use stainless steel on the lower floors, there are several other disadvantages as well. Although probably not likely, one disadvantage could be possible floor vibrations. The reason that this is relatively unlikely though is that there is going to be a lot a heavy equipment used in the building that would act as a natural damper for the system. One problem though that would need to be addressed is the need for additional fire protection to obtain a 2 hour fire rating if a steel frame was used. Finally there is the issue with the existing lateral force resisting system of ordinary reinforced concrete shear walls. If the shear walls are to stay, special connections will need to be designed to frame the two materials together. Otherwise, a steel lateral system will have to be designed.

Feasibility

For the Imaging Research Building, I believe it is a tossup whether or not a steel floor framing system would be a feasible option or not. Perhaps, if a concrete base was used for the two subgrade floors and steel framing was used above, this could be an economical and reasonable option. Further investigation is needed to determine the validity of this argument.

Composite steel framing – Option #2

Material Properties:

Concrete:	6" slab (2" deck with 4" of concrete)
	f'c = 3000 psi
Steel:	fy = 50,000 psi
Reinfrocement:	fy = 60,000 psi
Metal Deck:	Vulcraft 2VL

Loading:

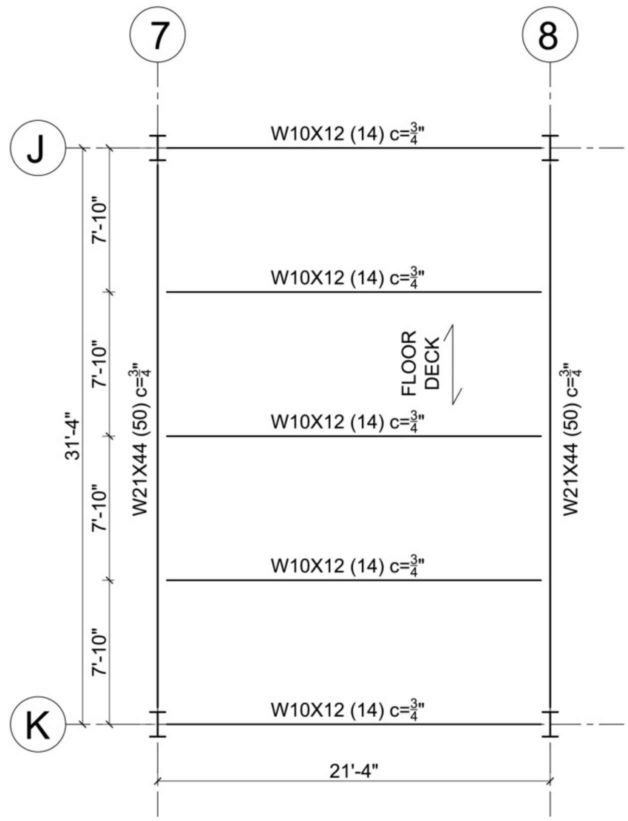
Dead (self weight):	75 psf
Live:	100 psf
Superimposed:	25 psf

Description

Again the composite steel system was designed using a typical interior bay of 21'-4" by 31'-4" with beams spaced 7'-10" on center as in the original concrete framing system. Contrary to the non-composite framing though

Calculations for this system were done using the AISC thirteenth edition *Steel Construction Manual* and RISA-3D. The steel manual was used to determine the sizes for the beams and girders and for efficiency of time RISA was used to determine the deflections of the girders to check that they did not exceed the deflection limits.

It was found that that the beams were controlled by the total load deflection of the system, and the girders were controlled by their moment capacity. The calculations supporting these findings can be found in Appendix B.



Advantages

There are several advantages to a steel frame system with a non-composite deck over a cast-in-place concrete slab. First, a steel frame system has a quicker erection time because it can arrive at the site prefabricated and there is no need for formwork. The lack of formwork will also reduce the cost of labor, although this is not as big of deal in North Carolina because it not unionized like it is up North. Another advantage is that it is that decking is able to span 11'-7" during construction; therefore there will be no need for shoring. Finally, although a composite system was not looked at (and there are merits to a composite system), the non-composite system will also save money due to the absence of shear studs.

Disadvantages

The biggest disadvantage to this system in regards to the Imaging Research Building is that on the lower floors where there are a number of pieces of imaging equipment, the steel used would have to be stainless steel. This is due to the fact that the imaging equipment is magnetized and any ferrous material used can disrupt the magnetic field and ruin the equipment. Therefore using a steel frame system for the lower floor would be a large cost increase considering as of September 2009, the North American stainless steel price was 2945 US$/tonne compared to the carbon steel price of 680 US$/tonne.

Besides the cost increase for having to use stainless steel on the lower floors, there are several other disadvantages as well. Although probably not likely, one disadvantage could be possible floor vibrations. The reason that this is relatively unlikely though is that there is going to be a lot a heavy equipment used in the building that would act as a natural damper for the system. One problem though that would need to be addressed is the need for additional fire protection to obtain a 2 hour fire rating if a steel frame was used. Finally there is the issue with the existing lateral force resisting system of ordinary reinforced concrete shear walls. If the shear walls are to stay, special connections will need to be designed to frame the two materials together. Otherwise, a steel lateral system will have to be designed.

Feasibility

For the Imaging Research Building, I believe it is a tossup whether or not a steel floor framing system would be a feasible option or not. Perhaps, if a concrete base was used for the two subgrade floors and steel framing was used above, this could be an economical and reasonable option. Further investigation is needed to determine the validity of this argument.

Hollow core precast on steel beams – Option #3

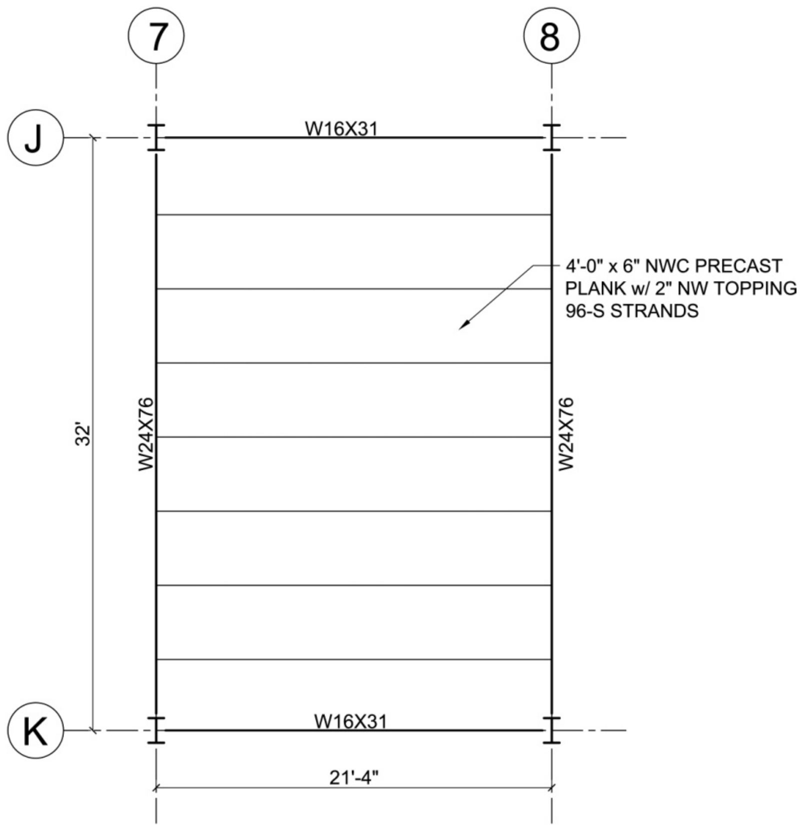
Material Properties:

Concrete:	4'-0"x6" plank (NWC)
	20"x20" columns
	f'c = 5000 psi
Reinfrocement:	fy = 60,000 psi

Loading:

Dead (self weight):	75 psf
Live:	100 psf
Superimposed:	25 psf

Description



The hollow core precast concrete system can be used if there is a slight adjustment to all of the bays within the building. Because the precast panels come in 4'-0" wide sections, it seems logical to set the typical interior bay size to 32'x21'-4" as shown in figure ___. Of course, the other bays in the building will also have to be adjusted, but for all intensive purposes of this technical report just the 32'x21'-4" bay was examined.

Using the PCI Design Handbook, a 6" thick plank with 2" topping was chosen for this floor system. The span of 21'-4" was satisfied using 96-S strands within the hollow core panel. In other words, the panel will contain 9 strands a 6/16ths, and that the strands are straight (S). This floor system is capable of supporting a superimposed service load of 160 psf which is greater than 140 psf which calculated using the 100 psf live load, the 25 psf superimposed dead load, and 15 psf for the 2" topping according to the PCI Design Handbook.

As seen in figure ___, the steel sections that the precast hollow core planks will frame into are W24x76's. This was determined using the AISC 13th edition *Steel Construction Manual*. The calculations supporting these sizes can be found in Appendix ___.

Advantages

There are a number of advantages to the hollow core precast plank system. First, the system is durable and it is a low maintenance assembly. Not only that, but it takes little time to construct because no curing time is needed. Therefore, construction can be completed quicker than with a cast-in-place slab which could allow for earlier occupancy. Besides that added construction benefits it also attenuates noise and is a recognized as a LEED rated system.

Disadvantages

In regards to the Imaging Research Building, there are also some disadvantages to using the hollow core precast floor system. The most glaring one would be that the bay sizes would have to be adjusted to accommodate the width of the planks. In turn, this would result in an increase in building footprint that may or may not be acceptable.

Also, again a steel framing system would be used which adds the factor of vibrations. It is unknown at this time the vibration that is associated with this system. Another disadvantage is the added depth to the framing system. Currently, the maximum depth for the bay anaylzed is 30" at the girders. If precast planks were used instead the depth would increase to 31.9". While this isn't a large increase, it is still one that the other trades on the project such as the MEP would have to contend with when trying to design their systems.

Again, as with non-composite floor system there is also a concern with the connections required at the concrete shear walls if that was to remain as the lateral force resisting system. Unless another lateral system was used, these connections could be more time consuming and costly.

Feasibility

This floor system does not seem like a candidate for further investigation. The disadvantages to it outweigh the benefits for its use in the Imaging Research Building.

Conclusions

After reviewing the advantages and disadvantages of each of the four floor systems is appears that the two-way cast in place slab, and the non-composite steel framing are the most feasible alternate floor framing systems. Both are options that have enough positives to be reviewed further. However, since the non-composite steel framing seems to be a viable option it is decided that a composite steel frame system should also be investigated in the future.

First, there are several benefits to the two-way flat slab that make it a feasible option as an alternate floor system. There is no additional fireproofing required and the layout of the building does not need to change.

Appendix A- Gravity Load Calculations

MULKEY
ENGINEERS & CONSULTANTS
P.O. Box 33127 • Raleigh, NC 27636-3127
Phone: (919) 851-1912 • Fax: (919) 851-1918

PROJECT NAME UNC Imaging Research Building
PROJECT NO. SHEET 1 OF 1
SUBJECT Snow Load
PREPARED BY DRH DATE 10/1/09 CHECKED BY DATE

Snow Load Calculations

Ground Snow load $p_g = 15$ psf (Fig. 7-1)

Flat roof snow load $p_f = 0.7\, C_e C_t I p_g$

Exposure factor: $C_e = 1.0$ (parapet) → partially exposed

Thermal factor: $C_t = 1.0$

Importance Factor: $I = 1.1$ Occupancy Cat: III

$p_f = 0.7(1.0)(1.0)(1.1)(15) = 11.55$ psf

but not less than...

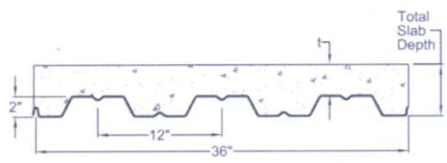
$p_f = (I)p_g = 1.1(15) = \boxed{16.5 \text{ psf}}$

Appendix B- Option #1

VULCRAFT

NON-COMPOSITE

2 C CONFORM



Interlocking side lap is not drawn to show actual detail.

MAXIMUM CONSTRUCTION CLEAR SPANS (S.D.I. CRITERIA)

Total Slab Depth	DECK	WEIGHT PSF	NW CONCRETE N=9 145 PCF			WEIGHT PSF	LW CONCRETE N=14 110 PCF		
			1 SPAN	2 SPAN	3 SPAN		1 SPAN	2 SPAN	3 SPAN
4.5 (t=2.50)	2C22	44	6- 11	9- 0	9- 4	34	7- 8	9- 10	10- 2
	2C20	45	8- 2	10- 3	10- 7	34	9- 0	11- 3	11- 7
	2C18	45	10- 2	12- 4	12- 4	35	11- 2	13- 1	13- 1
	2C16	46	10- 5	12- 6	12- 11	36	11- 7	13- 8	13- 10
5 (t=3.00)	2C22	50	6- 7	8- 7	8- 11	39	7- 4	9- 5	9- 9
	2C20	51	7- 9	9- 10	10- 2	39	8- 7	10- 9	11- 2
	2C18	51	9- 7	11- 10	11- 11	40	10- 9	12- 9	12- 9
	2C16	52	9- 11	12- 0	12- 4	40	11- 0	13- 1	13- 5
5.5 (t=3.50)	2C22	56	6- 4	8- 0	8- 6	43	7- 0	9- 1	9- 5
	2C20	57	7- 5	9- 5	9- 9	43	8- 3	10- 4	10- 9
	2C18	57	9- 2	11- 4	11- 7	44	10- 3	12- 5	12- 5
	2C16	58	9- 5	11- 6	11- 10	45	10- 6	12- 7	13- 0
6 (t=4.00)	2C22	62	6- 1	7- 5	8- 2	48	6- 9	8- 9	9- 1
	2C20	63	7- 1	9- 1	9- 4	48	7- 11	10- 0	10- 4
	2C18	63	8- 10	10- 11	11- 3	49	9- 10	12- 0	12- 1
	2C16	64	9- 1	11- 1	11- 5	49	10- 1	12- 2	12- 7
6.5 (t=4.50)	2C22	68	5- 11	6- 11	7- 11	52	6- 6	8- 6	8- 9
	2C20	69	6- 11	8- 9	9- 0	53	7- 7	9- 8	10- 0
	2C18	69	8- 7	10- 6	10- 11	53	9- 6	11- 8	11- 10
	2C16	70	8- 10	10- 8	11- 0	54	9- 9	11- 10	12- 2
7 (t=5.00)	2C22	74	5- 10	6- 6	7- 5	57	6- 4	8- 0	8- 6
	2C20	75	6- 9	8- 6	8- 9	57	7- 4	9- 5	9- 8
	2C18	75	8- 4	10- 2	10- 6	58	9- 2	11- 4	11- 7
	2C16	76	8- 7	10- 4	10- 8	59	9- 5	11- 5	11- 10

REINFORCED CONCRETE SLAB ALLOWABLE LOADS

Slab Depth	REINFORCEMENT		Superimposed Uniform Load (psf) -- 3 Span Condition Clear Span (ft.-in.)										
	W.W.F.	As	5- 0	5- 6	6- 0	6- 6	7- 0	7- 6	8- 0	8- 6	9- 0	9- 6	10- 0
4.5 (t=2.50)	6X6-W2.1XW2.1	0.042*	84	69									
	6X6-W2.9XW2.9	0.058	114	94									
	4X4-W2.9XW2.9	0.087	167	138									
5 (t=3.00)	6X6-W2.1XW2.1	0.042*	**153**	**127**	**107**	**91**	**78**						
	6X6-W2.9XW2.9	0.058*	**206**	**170**	**143**	**122**	**105**						
	4X4-W2.9XW2.9	0.087	**305**	**252**	**212**	**180**	**155**						
5.5 (t=3.50)	6X6-W2.9XW2.9	0.058*	**255**	**211**	**177**	**151**	**130**	**113**	**100**				
	4X4-W2.9XW2.9	0.087	**378**	**313**	**263**	**224**	**193**	**168**	**148**				
	4X4-W4.0XW4.0	0.120	**400**	**400**	**351**	**299**	**258**	**224**	**197**				
6 (t=4.00)	6X6-W2.9XW2.9	0.058*	**304**	**251**	**211**	**180**	**155**	**135**	**119**	**105**	**94**		
	4X4-W2.9XW2.9	0.087	**400**	**374**	**314**	**267**	**231**	**201**	**177**	**156**	**140**		
	4X4-W4.0XW4.0	0.120	**400**	**400**	**400**	**359**	**309**	**270**	**237**	**210**	**187**		
6.5 (t=4.50)	6X6-W2.9XW2.9	0.058*	**353**	**292**	**245**	**209**	**180**	**157**	**138**	**122**	**109**	**98**	**88**
	4X4-W2.9XW2.9	0.087*	**400**	**400**	**365**	**311**	**268**	**234**	**205**	**182**	**162**	**146**	**131**
	4X4-W4.0XW4.0	0.120	**400**	**400**	**400**	**400**	**361**	**315**	**277**	**245**	**219**	**196**	**177**
7 (t=5.00)	4X4-W2.9XW2.9	0.087*	**400**	**400**	**400**	**355**	**306**	**266**	**234**	**207**	**185**	**166**	**150**
	4X4-W4.0XW4.0	0.120	**400**	**400**	**400**	**400**	**400**	**360**	**316**	**280**	**250**	**224**	**202**
	4X4-W5.0XW5.0	0.150	**400**	**400**	**400**	**400**	**400**	**400**	**389**	**344**	**307**	**276**	**249**

NOTES:
1. * As does not meet A.C.I. criterion for temperature and shrinkage.
2. Recommended conform types are based upon S.D.I. criteria and normal weight concrete.
3. Superimposed loads are based upon three span conditions and A.C.I. moment coefficients.
4. Load values for single span and double spans are to be reduced.
5. Vulcraft's painted or galvanized form deck can be considered as permanent support in most building applications. See page 23. If uncoated form deck is used, deduct the weight of the slab from the allowable superimposed uniform loads.
6. Superimposed load values shown in bold type require that mesh be draped. See page 23.

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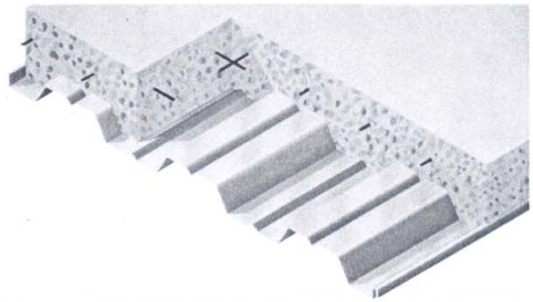
VULCRAFT

SLAB INFORMATION

Total Slab Depth, in.	Theo. Concrete Volume Yd^3 / 100 ft^2	Theo. Concrete Volume ft^3 / ft^2	Recommended Welded Wire Fabric
4	0.93	0.250	6x6 - W1.4xW1.4
4 1/2	1.08	0.292	6x6 - W1.4xW1.4
5	1.23	0.333	6x6 - W1.4xW1.4
5 1/4	1.31	0.354	6x6 - W1.4xW1.4
5 1/2	1.39	0.375	6x6 - W2.1xW2.1
6	1.54	0.417	6x6 - W2.1xW2.1
6 1/4	1.62	0.438	6x6 - W2.1xW2.1
6 1/2	1.70	0.458	6x6 - W2.1xW2.1



SECTION PROPERTIES

Deck Type	Design Thickness in.	Deck Weight psf	I_p in^4/ft	I_n in^4/ft	S_p in^3/ft	S_n in^3/ft	V_a lbs/ft	F_y ksi
2C22	0.0295	1.62	0.324	0.321	0.263	0.266	1832	50
2C20	0.0358	1.97	0.409	0.406	0.341	0.346	2698	50
2C18	0.0474	2.61	0.559	0.558	0.495	0.504	3608	50
2C16	0.0598	3.29	0.704	0.704	0.653	0.653	3618	40

NON-COMPOSITE

ALLOWABLE UNIFORM LOAD (PSF)

TYPE NO.	NO. OF SPANS	DESIGN CRITERIA	CLEAR SPAN (ft-in) 5- 0	5- 6	6- 0	6- 6	7- 0	7- 6	8- 0	8- 6	9- 0	9- 6	10- 0	10- 6	11- 0
2C22	1	Fb = 30,000	210	174	146	124	107	93	82	73	65	58	52	48	43
		Defl. = l/240	170	128	98	77	62	50	42	35	29	25	21	18	16
		Defl. = l/180	227	170	131	103	83	67	55	46	39	33	28	25	21
	2	Fb = 30,000	200	167	141	121	105	92	81	72	64	58	52	47	43
		Defl. = l/240	408	306	236	186	149	121	100	83	70	59	51	44	38
		Defl. = l/180	544	409	315	248	198	161	133	111	93	79	68	59	51
	3	Fb = 30,000	243	204	173	149	129	113	100	89	80	72	65	59	54
		Defl. = l/240	319	240	185	145	116	95	78	65	55	47	40	34	30
		Defl. = l/180	426	320	246	194	155	126	104	87	73	62	53	46	40
2C20	1	Fb = 30,000	272	225	189	161	139	121	106	94	84	75	68	62	56
		Defl. = l/240	215	161	124	98	78	64	52	44	37	31	27	23	20
		Defl. = l/180	286	215	166	130	104	85	70	58	49	42	36	31	27
	2	Fb = 30,000	263	219	185	159	137	120	106	94	84	75	68	62	56
		Defl. = l/240	515	387	298	235	188	153	126	105	88	75	64	56	48
		Defl. = l/180	687	516	398	313	250	204	168	140	118	100	86	74	65
	3	Fb = 30,000	322	269	228	196	170	149	131	117	104	94	85	77	70
		Defl. = l/240	403	303	233	184	147	119	98	82	69	59	50	44	38
		Defl. = l/180	538	404	311	245	196	159	131	109	92	78	67	58	50
2C18	1	Fb = 30,000	395	327	274	234	202	176	154	137	122	109	99	90	82
		Defl. = l/240	294	221	170	134	107	87	72	60	50	43	37	32	28
		Defl. = l/180	392	294	227	178	143	116	96	80	67	57	49	42	37
	2	Fb = 30,000	380	317	268	230	199	174	154	136	122	110	99	90	82
		Defl. = l/240	706	531	409	321	257	209	172	144	121	103	88	76	66
		Defl. = l/180	942	708	545	429	343	279	230	192	161	137	118	102	88
	3	Fb = 30,000	464	389	330	283	246	215	190	169	151	136	123	112	102
		Defl. = l/240	553	415	320	252	201	164	135	113	95	81	69	60	52
		Defl. = l/180	737	554	426	335	269	218	180	150	126	107	92	80	69
2C16	1	Fb = 24,000	417	345	290	247	213	185	163	144	129	116	104	95	86
		Defl. = l/240	370	278	214	168	135	110	90	75	63	54	46	40	35
		Defl. = l/180	493	370	285	224	180	146	120	100	85	72	62	53	46
	2	Fb = 24,000	392	328	277	238	206	180	159	141	126	114	103	93	85
		Defl. = l/240	890	669	515	405	324	264	217	181	153	130	111	96	84
		Defl. = l/180	1187	892	687	540	433	352	290	242	204	173	148	128	111
	3	Fb = 24,000	479	401	341	293	254	223	197	175	156	141	127	116	106
		Defl. = l/240	697	523	403	317	254	206	170	142	119	102	87	75	65
		Defl. = l/180	929	698	538	423	339	275	227	189	159	135	116	100	87

Minimum exterior bearing length is 2.0 inches.
Minimum exterior bearing length is 4.0 inches.

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PROJECT NAME UNC Imaging Research Building
PROJECT NO. SHEET 1 OF
SUBJECT Non Composite Steel Framing
PREPARED BY DRH DATE 10/21/09 CHECKED BY DATE

Non Composite Steel

- Catalog From Vulcraft Website
- Loads: LL = 100 psf (Corridors & Laboratories according to Mulkey's values)
  SDL = 25 psf
  DL = ?

Three Span Condition, Current span = 7'-10"

∴ $\boxed{\text{Use 2C18 (2C Conform)}}$

DL = 63 psf
↳ 6" slab depth (t = 4") n = 9
NWC; 3 span = 11'-3", 145 pcf
f'c = 3000 psi, Fy, reinf = 60,000 psi

Total Load = 100 + 25 + 63 = 188 psf

- 2C18 3 span; 8'-0" span

Fb = 30,000, 18 Ga, Allowable Load = 190 psf > 188 psf ∴ OK!
$\ell/240$ = 18 Ga, Allowable load = 135 psf > 100 psf ∴ ok
$\ell/180$ = 18 Ga, Allowable load = 180 psf > 63 psf ∴ ok
Compared to load of wet concrete

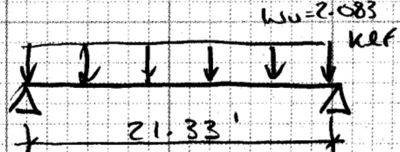
- For Beams

Load = 1.2 (25 + 63) + 1.6 (100) = 265.6 psf = 0.266 ksf
Trib width = 7'-10" = 7.833'

$W_u = 7.833(0.266) = 2.084$ klf

$M_u = \frac{w\ell^2}{8} = \frac{2.084(21.33)^2}{8} = 118.5$ 'k

$V_u = \frac{2.084(21.33)}{2} = 22.2$ K



* Assume Fully Braced

ME-02

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PROJECT NAME UNC Imaging Research Building
PROJECT NO. ______ SHEET ______ OF ______
SUBJECT ______
PREPARED BY ______ DATE ______ CHECKED BY ______ DATE ______

Beams - Table 3-2 (AISC Steel Manual - 13th Ed)

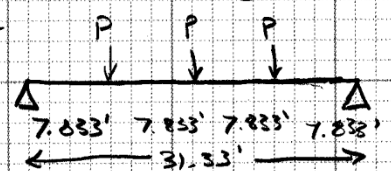
W14x22 $\phi M_n = 125 > 118.5'k \therefore ok!$ $I_x = 199 \text{ in}^4$

$\Delta_{LL} = l/360 = \frac{21.33(12")}{360} = 0.711"$ (allowable)

$\Delta_{LL} = \frac{5wl^4}{384EI} = \frac{5(100)(7.833)(21.33)^4(1728)}{384(29,000)(199)} = 0.632"$

$0.632" < 0.711" \therefore ok!$

Girder



$P = \frac{(2.0833)(21.33)}{2} = (22.2)(2) = 44.4k$

For interior girder

$V_u = 66.6^k$

$M_{max} = V_u(7.833) + P/2(7.833) = 695.6'k$

Try W24x76 $\phi M_n = 750 > 695.6'k \therefore ok$

Δ_{LL} (allowable) $= \frac{l}{360} = \frac{31.33(12")}{360} = 1.044"$

Δ_{LL} max $= 0.702$ (From RISA)

$0.702 < 1.044" \therefore ok!$

For all interior Beams:

$\Delta_{TL} = \frac{l}{240} = \frac{21.33(12")}{240} = 1.0665"$ (allowable)

Δ_{TL} max $= \frac{5(188)(7.833)(21.33)^4(1728)}{384(29,000)(199)} = 1.19"$

- Need to increase I_x!

ME-02

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PROJECT NAME UNC IRB
PROJECT NO. SHEET 3 OF
SUBJECT
PREPARED BY DATE CHECKED BY DATE

Solve for I_x: $I_x = \frac{5(2.084)(21.33)^4(1728)}{384(29{,}000)(1.0665)} = 313.8 \text{ in}^4$

Try W16x31 $\phi M_n = 203'^k > M_u = 118.5'^k$ ∴ ok

$I_x = 375 \text{ in}^4$

∴ $\Delta_{max} = \frac{5(2.084)(21.33)^4(1728)}{384(29{,}000)(375)} = 0.893''$

$0.893'' < 1.07''$ ∴ ok

$\boxed{\text{∴ Use W16x31 for interior beams}}$

For all Interior Girders:

$\Delta_{TL} = \frac{l}{240} = \frac{(31.33)(12)}{240} = 1.567''$

$\Delta_{TL\,max} = 1.32''$ (From RISA, W24x76, $I_x = 2100$)

$1.32'' < 1.567''$ ∴ ok!

$\boxed{\text{∴ Use W24x76 for interior girders}}$

ME-02

Appendix C – Option #2

Gravity Beam Design

RAM Steel v14.00.03.00
DataBase: IRB two-way trial
Building Code: IBC

10/28/09 17:28:15
Steel Code: AISC360-05 LRFD

Floor Type: Floor 3 **Beam Number = 26**

SPAN INFORMATION (ft): I-End (21.33,23.50) J-End (42.67,23.50)

Beam Size (Optimum) = W10X12 Fy = 50.0 ksi
Total Beam Length (ft) = 21.33

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Concrete thickness (in)	4.00	4.00
Unit weight concrete (pcf)	115.00	115.00
f'c (ksi)	3.00	3.00
Decking Orientation	perpendicular	perpendicular
Decking type	VULCRAFT 2.0VL	VULCRAFT 2.0VL

beff (in)	=	64.00	Y bar(in)	=	12.51
Mnf (kip-ft)	=	153.29	Mn (kip-ft)	=	129.06
C (kips)	=	120.61	PNA (in)	=	9.73
Ieff (in4)	=	273.17	Itr (in4)	=	319.56
Stud length (in)	=	4.00	Stud diam (in)	=	0.75

Stud Capacity (kips) Qn = 17.2 Rg = 1.00 Rp = 0.60
\# of studs: Max = 21 Partial = 14 Actual = 14
Number of Stud Rows = 1 Percent of Full Composite Action = 68.14

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	CLL
1	0.000	0.399	0.399	0.000	---	NonR	0.000
	21.333	0.399	0.399	0.000			0.000
2	0.000	0.196	0.000	0.783	---	NonR	0.000
	21.333	0.196	0.000	0.783			0.000
3	0.000	0.012	0.012	0.000	---	NonR	0.000
	21.333	0.012	0.012	0.000			0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 21.13 kips 1.00Vn = 56.26 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu	@	Lb	Cb	Phi	Phi*Mn
			kip-ft	ft	ft			kip-ft
Center	PreCmp+	1.4DL	32.7	10.7	0.0	1.00	0.90	46.90
	Init DL	1.4DL	32.7	10.7	---	---		
	Max +	1.2DL+1.6LL	112.7	10.7	---	---	0.90	116.16
Controlling		1.2DL+1.6LL	112.7	10.7	---	---	0.90	116.16

REACTIONS (kips):

	Left	Right
Initial reaction	4.38	4.38
DL reaction	6.47	6.47
Max +LL reaction	8.36	8.36
Max +total reaction (factored)	21.13	21.13

DEFLECTIONS: (Camber = 3/4)

Initial load (in) at 10.67 ft = -1.227 L/D = 209

Gravity Beam Design

RAM Steel v14.00.03.00 Page 2/2
DataBase: IRB two-way trial 10/28/09 17:28:15
Building Code: IBC Steel Code: AISC360-05 LRFD

Live load (in)	at	10.67 ft	=	-0.461	L/D	=	556
Post Comp load (in)	at	10.67 ft	=	-0.576	L/D	=	444
Net Total load (in)	at	10.67 ft	=	-1.053	L/D	=	243

Gravity Beam Design

RAM Steel v14.00.03.00
DataBase: IRB two-way trial 10/28/09 17:28:15
Building Code: IBC Steel Code: AISC360-05 LRFD

Floor Type: Floor 3 **Beam Number = 19**

SPAN INFORMATION (ft): I-End (21.33,0.00) J-End (21.33,31.33)

Beam Size (Optimum) = W21X44 Fy = 50.0 ksi
Total Beam Length (ft) = 31.33

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Concrete thickness (in)	4.00	4.00
Unit weight concrete (pcf)	115.00	115.00
f'c (ksi)	3.00	3.00
Decking Orientation	parallel	parallel
Decking type	VULCRAFT 2.0VL	VULCRAFT 2.0VL

beff (in)	=	94.00	Y bar(in)	=	20.25
Mnf (kip-ft)	=	812.18	Mn (kip-ft)	=	745.02
C (kips)	=	442.07	PNA (in)	=	20.38
Ieff (in4)	=	2397.50	Itr (in4)	=	2727.97
Stud length (in)	=	4.00	Stud diam (in)	=	0.75

Stud Capacity (kips) Qn = 17.7 Rg = 1.00 Rp = 0.75
# of studs: Full = 74 Partial = 50 Actual = 50
Number of Stud Rows = 1 Percent of Full Composite Action = 68.01

POINT LOADS (kips):

Dist	DL	CDL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%	CLL
7.833	6.47	4.38	0.00	0.0	8.36	0.00	0.0	0.00	Snow	0.00
7.833	6.47	4.38	0.00	0.0	8.36	0.00	0.0	0.00	Snow	0.00
15.667	6.47	4.38	0.00	0.0	8.36	0.00	0.0	0.00	Snow	0.00
15.667	6.47	4.38	0.00	0.0	8.36	0.00	0.0	0.00	Snow	0.00
23.500	6.47	4.38	0.00	0.0	8.36	0.00	0.0	0.00	Snow	0.00
23.500	6.47	4.38	0.00	0.0	8.36	0.00	0.0	0.00	Snow	0.00

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	CLL
1	0.000	0.044	0.044	0.000	---	NonR	0.000
	31.333	0.044	0.044	0.000			0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 64.23 kips 1.00Vn = 217.35 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	PreCmp+	1.4DL	199.8	15.7	7.8	1.11	0.90	333.85
	Init DL	1.4DL	199.8	15.7	---	---		
	Max +	1.2DL+1.6LL	668.7	15.7	---	---	0.90	670.51
Controlling		1.2DL+1.6LL	668.7	15.7	---	---	0.90	670.51

REACTIONS (kips):

	Left	Right
Initial reaction	13.84	13.84
DL reaction	20.11	20.11

Gravity Beam Design

RAM Steel v14.00.03.00
DataBase: IRB two-way trial
Building Code: IBC

Page 2/2
10/28/09 17:28:15
Steel Code: AISC360-05 LRFD

	Left	Right
Max +LL reaction	25.07	25.07
Max +total reaction (factored)	64.23	64.23

DEFLECTIONS: (Camber = 3/4)

Initial load (in)	at	15.67 ft	=	-0.982	L/D	=	383
Live load (in)	at	15.67 ft	=	-0.632	L/D	=	595
Post Comp load (in)	at	15.67 ft	=	-0.790	L/D	=	476
Net Total load (in)	at	15.67 ft	=	-1.022	L/D	=	368

Appendix D – Option #3

Strand Pattern Designation
76-S

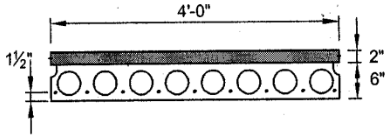
S = straight
Diameter of strand in 16ths
No. of Strand (7)

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.
0.2 – 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

Table of safe superimposed service load (psf) and cambers (in.) — **No Topping**

Strand Designation Code	Span, ft 10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
66-S	444	382	333	282	238	203	175	151	131	114	100	88	77	68	59	52	46	40	33	28	
	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.5	–0.7		
	0.2	0.2	0.2	0.2	0.3	0.3	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.3	–0.5	–0.7	–0.9	–1.2	–1.5	–1.9	
76-S		445	388	328	278	238	205	178	155	136	120	105	93	82	73	65	57	49	42	36	31
		0.2	0.2	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	–0.4	–0.6
		0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.7	–0.9	–1.2	–1.6	–2.0
96-S		466	421	386	338	292	263	229	201	177	157	139	124	110	99	88	78	68	60	53	46
		0.3	0.3	0.3	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
		0.3	0.4	0.4	0.5	0.5	0.5	0.6	0.6	0.6	0.5	0.5	0.4	0.3	0.2	0.1	–0.1	–0.3	–0.6	–0.9	–1.3
87-S		478	433	398	362	322	290	264	240	212	188	167	149	134	119	107	95	85	76	68	60
		0.3	0.4	0.4	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
		0.4	0.5	0.5	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.3	0.2	0.0	–0.3	–0.6
97-S		490	445	407	374	346	311	276	242	220	203	186	166	148	133	119	107	96	86	78	70
		0.4	0.4	0.5	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
		0.5	0.6	0.6	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.0	1.0	0.9	0.9	0.8	0.7	0.5	0.3	0.1	–0.2

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.

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PROJECT NAME UNC Imaging Research Building
PROJECT NO. SHEET OF
SUBJECT Hollow Core Precast on Steel
PREPARED BY DRH DATE 10/21/09 CHECKED BY DATE

Hollow Core Precast on Steel

Load: LL = 100 psf
SDL = 25 psf
DL = 15 psf (From PCI Handbook for topped members)
140 psf Superimposed Service Load

Try 4'-0" x 6" NWC w/ 2" topping
f'c = 5000 psi Span = 21'-4" → 22' (simplified)
f'ci = 3500 psi fpu = 270,000 psi

96-S → 160 psf > 140 psf ∴ OK
0.5" camber @ erection
0.2" Camber (long period)

9 Strands @ 6/16" ϕ - Straight
Self Wt = 74 psf

Girders:

Load = 1.2 (25+74) + 1.6 (100) = 278.8 psf = 0.279 ksf

$$M_u = \frac{w_u l^2}{8} = \frac{0.279(21.33)(31.33)^2}{8} = 730.2\ 'k$$

Try W24 x 76 $\phi M_n = 750\ 'k > 730\ 'k$ ∴ OK

$$\Delta_{LL}\text{(allowable)} = l/360 = \frac{31.33(12)}{360} = 1.044''$$

$$\Delta_{LL} = \frac{5wl^4}{384EI} = \frac{5(100)(21.33)(1/1000)(31.33)^4(1728)}{384(29{,}000)(2100)}$$

$\Delta_{LL} = 0.76''$

0.76" < 1.044" ∴ OK

ME-02

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PROJECT NAME UNC IRB
PROJECT NO. SHEET 2 OF
SUBJECT
PREPARED BY DRH DATE 10/21/09 CHECKED BY DATE

Check TL Δ:

Δ_{TL} (allowable) $= \ell/240 = \frac{31.33(12")}{240} = 1.57"$

Service load $= 100 + 25 + 74 = 199$ psf

Δ_{TL} max $= \frac{5w\ell^4}{384EI} = \frac{5(199)(21.33)(1/1000)(31.33)^4(1728)}{384(29{,}000)(2100)}$

$\Delta_{TL} = 1.51"$

$1.51" < 1.57" \therefore$ ok

Check Shear: 0.279 ksf $(21.33) = 5.95$ klf

$V_u = 5.95(31.33)/2 = 93.2^k$

$\phi V_n = 316^k > 93.2^k \therefore$ OK

$\therefore$ Use 4'-0" x 6" NWC w/ 2" Topping
4HC6 + 2, 96-S on W24x76 Girders

ME-02