Technical Report II

University of Maryland College Park Dorm Building 7

College Park, MD

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Executive Summary

Technical Report 2 is a pro-con structural study of alternate floor systems. This report describes the physical existing conditions of the current structure of University of Maryland College Park Dorm Building 7. This report will addresses three alternative floor framing systems and the existing.

In this technical report the systems analyzed were chosen for further investigation because they are best represented systems for providing maximum floor to ceiling height. Constructability was also taken into an account when choosing them. The systems chosen are:

1. Hambro Composite Floor System (existing)
2. Two way Flat Slab with Drop Panels
3. Composite Steel and Deck Framing System
4. Girder Slab with Prestressed Hollow Core Planks

After designing each of the four systems, it appears as if the composite steel and deck system and the girder slab with hollow core planks are the best choices for Building 7. Each of these systems is relatively light in weight and also has minimal thickness to allow for the low floor to floor height. The two way drop plate could have potential to be viable but the relative weight of the system and other all thickness it has are a disadvantage, the thickness could be reevaluated if concrete was a last choice. Also the current system in Building 7, Hambro Composite Floor System, is a good choice from a strength point of view; it however has other issues dealing with construction and fire protection that make it less desirable compared to the others.

Overall it is felt that system 3 and 4 have the greatest potential and benefits to Building 7. A more detailed and through analysis and design of the composite steel and deck system and the girder slab with hollow core planks are need to see other implications such as lateral load distribution of the diaphragm, connections, vibrations and the floor effects on the lateral system. These considerations will be looked at in future reports.

Introduction

The University of Maryland College Park Dorm Building 7 (Building 7) is the final stage of the south campus master plan at the University of Maryland. Building 7 is the corner stone of the south campus entrance for all to take part of as they approach the campus. Building 7 is an eight story residential dorm in the shape of an unsymmetrical-U that compliments the adjacent two existing dorm buildings in architectural styles with its shape and material usage.

This eight story-133,000 square feet residential building, houses 370 bedrooms, study lounges, seminar spaces and resident life offices. The average floor to floor height is 10 feet on each floor with an average floor area of 12,000-15,500 square feet per floor, depending on shifts in the vertical plane. The layout of each floor is such that all of the rooms have an exterior view of the surrounding campus with a central corridor running the length of the building. The roof level houses the mechanical equipment along with the elevator and stair towers.

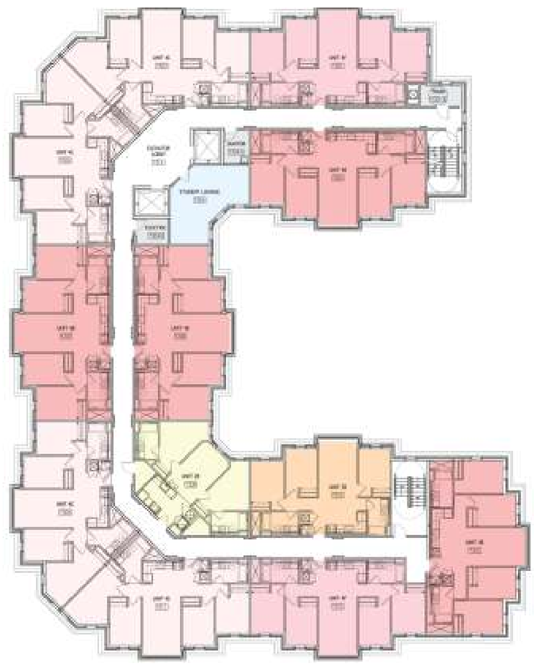


Figure 1. (Typical Floor Plan)

The façade and building envelope is comprised of light gage studs with a brick masonry veneer exterior around the entire building. There is rigid insulation on the exterior of the studs between the veneer with a 1.5 inch air cavity. The walls are filled with batt insulation and covered in drywall.

The windows are fixed casement aluminum windows with cast stone sills to accent them. In the regions where the wall sections are pulled away from the primary facade, the wall system is composed of composite metal panel and cast stone veneer panels. The roof system is an EPDM classification which is a fully adhered system comprised of a waterproof membrane that is bonded to rigid insulation by mechanical and chemical means with appropriate flashing at the base of the parapets and where the brick meets the top of the parapet.

Structural Systems

Foundation

The foundation system is composed of reinforced concrete grade beams 24”x30” with 3#8’s on the top and bottom with number #4 stirrups placed every 14”. The deep foundation portion is auger cast grout piles 16” in diameter. These piles are to be 65’ below elevation and are to be able to carry at 65 ton allowable load capacity. The pile configurations range from 2-4 piles per cap. The slab on grade for the foundation is 4” thick normal weight concrete reinforced with 6x6-1.4xW1.4 welded wire fabric. All foundation concrete is 4ksi except for the SOG which is 3.5 ksi. Due to the site’s soil conditions it was necessary that the differential settlement over the entire building was limited, because of this the allowable soil bearing capacity was held to 500 psf.

Column and Bearing Wall Systems

The concrete columns support the lower two floors of Building 7. They arranged to form a typical bay of 15’x20’. These columns are gravity bearing only due to the type of lateral system in the building. The typical size of the columns range from 18x14 to 64x14 with the reinforcing ranging in each from 4#9’s to 10#9’s for vertical bars with #4 stirrups spaced at 14” O.C.. The concrete compressive strength for the columns is 6 ksi.

The bearing walls in Building 7 support the upper 6 floors and run along the outside perimeter of the building as well as along the corridors. The typical spans for the floor joists are 20’. Dealing with the concerns that the joists may not line up with the studs causing the header to buckle, this problem was solved by placing a distribution tube across the tops of all bearing walls. These walls are also to be designed by the contractor who is given general criteria to follow along with a loading diagram for all the different bearing walls. The general criteria are: a maximum stud spacing of 16” O.C., a minimum G90 galvanized coating, and have a minimum 16 gage thickness.

Roof System

The roof system is made of the same Hambro Composite Floor System bearing on light gage walls. This Hambro Composite Floor System is also to be designed by the contractor instead of the Engineer just as the other floors are to be designed. Here are the criteria for the roof: overall depth of the members is 16” deep typically throughout except in the corridors which it drops to 8”deep with a 3” thick concrete slab reinforced with 6x6-2.9xW2.9 welded wire fabric. The mechanical unit weights are listed and are placed close to the corridors for they are formed by the bearing walls. The elevator towers and stair towers are made of the same light gage studs.

Lateral Systems

The primary lateral system for Building 7 is shear walls. On each floor there are 16 shear walls spanning both directions of the building, 9 in the north-south direction and 7 in the east-west direction. The lower two stories shear walls are 10" thick reinforced concrete with 10#5's on each end for flexure and for shear reinforcement there is #5@12" each way, each face. All concrete shear walls are 6 ksi normal weight concrete. The upper floors shear walls are to be light gage studs with maximum stud spacing of 16" O.C. they are also have a minimum G90 galvanized coating and have a minimum gage of 16 for the studs while the tracks are permitted to have a 20 gage minimum. There is to be bridging at 4' spacing throughout the shear walls. Since these are light gage it was determined that steel strapping was needed and is being provided in an X pattern connecting to the farthest opposite ends. The light-gage shear walls not designed by the Structural Engineer but rather is to be designed by the Contractor. The Structural Engineer has however given detailed loading diagrams of each load and the type of load on every shear wall.

Floor Systems

Lower 2 Floors

The lower two floors are made of reinforced concrete beams spanning between the columns. The intermediate members between these beams are made up of the Hambro Composite Floor System, which includes the steel joists and the slab system. The concrete beams range from 16x36 to 18x18 to 24x36 with the reinforcing ranging in each from 3#5's to 6#10's for longitudinal bars with #4 stirrups spaced from 8" to 16" O.C.

The Hambro Composite Floor System in Building 7 is not designed by the Structural Engineer but rather is to be designed by the Contractor. The Structural Engineer has however given detailed criteria that the contractor must follow. The following is the criteria: are overall depth of the members is 16" deep typically throughout except in the corridors which it drops to 8"deep, the slab on top is to be 5" thick reinforced with 6x6-W4.0xW4.0 welded wire fabric.

Upper 6 Floors

The floor system is made of the same Hambro Floor System but instead of them bearing on concrete girders they bear on light-gage stud bearing walls. This Hambro Floor System is also to be designed by the contractor instead of the Engineer. Here are the criteria for these 7 stories: overall depth of the members is 16" deep typically throughout except in the corridors which it drops to 8"deep with a 3" thick concrete slab reinforced with 6x6-2.9xW2.9 welded wire fabric.

Here is a typical Upper Floor plan that will be utilized throughout this technical report. The upper floors were chosen due to the majority of the building is structurally supported in this manner. The arrows on the floor plans indicate the way the Hambro joists are laid out. The area shaded in blue is the typical bay that will be studied for the alternate systems.

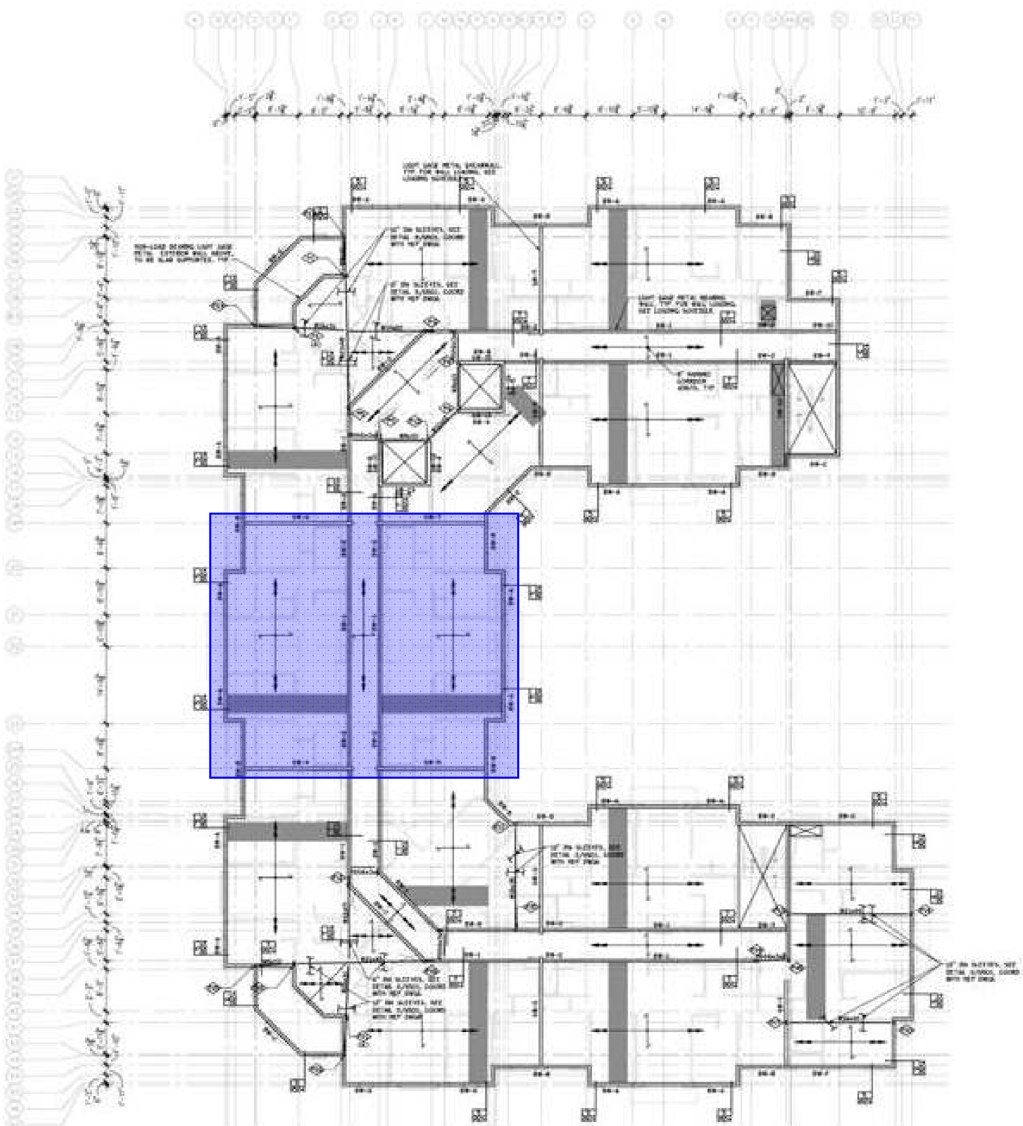


Figure 2.

Shown below in Figure 3 is an enlargement of the typical bay. The larger area shown in green will be the primary typical bay which all the designs are based off of. Depending on the different systems that have been chosen to be studied, the area in yellow may also have significant impact in the overall design of a system. In some cases only one half of the green area will be considered while for other systems this may change to the entire area from outer wall to outer wall. The reason for this is because of requirements and limitations of the system.

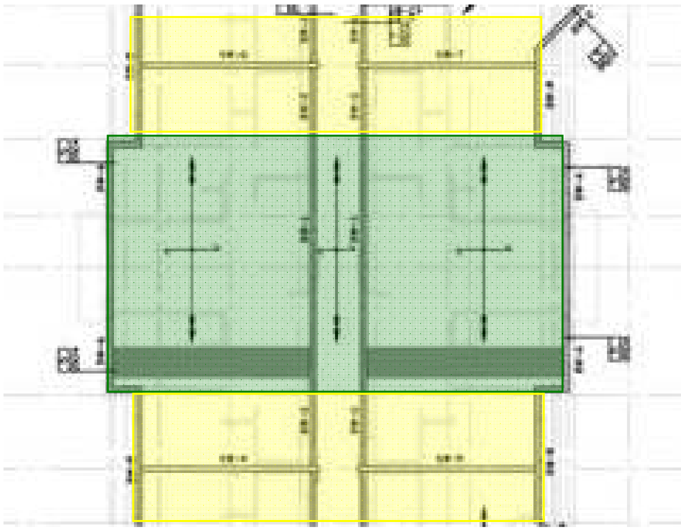


Figure 3.

Design Codes & Guides

1. AISC Unified Manual 13th Edition
2. ACI 318-08
3. ASCE 7-05
4. International Building Code (IBC) 2006
5. Girder-Slab Design Guide v1.4
6. Hambro Floor System Design Guide
7. Vulcraft floor and Deck Catalog
8. CRSI Design Handbook 2002
9. RS Means Square Foot Costs 2008

Deflection Criteria

Typical live load deflections limited to: L/360
Typical total deflections limited to: L/240
Typical construction load deflections limited to: L/360

Gravity Loads

Live Loads

The live loads for Building 7 were calculated in accordance with IBC 2006 which references ASCE 7-05, Chapter 6. In the event that ASCE did not list loads needed a close equivalent was chosen to meet that space.

Live Loads			
Occupancy	Design Load	Code Required Loads	
		Load	Code
Corridors	100 psf	100 psf	ASCE 7
Offices	100 psf	50 psf	ASCE 7
Seminar Room	100 psf	40 psf	ASCE 7
Mechanical Room	250 psf	125 psf	Light manufacturing
Partition	15 psf	-	-
Roof	30 psf	20 psf	ASCE 7
Dormitory Rooms	40 psf	40 psf	ASCE 7
Lobby	100 psf	100 psf	ASCE 7

Dead Loads

The dead loads for Building 7 were determined by referencing various standards and textbooks to find the corresponding values of their weights. Approximate values were assumed when ranges were listed depending on how dense the layouts were.

Dead Loads		
Roof Dead Load	Material	Design Weight
	Rigid Insulation	4 psf
	3" Hambro Slab	38 psf
	M/E/P	5 psf
	Ceiling Finishes	3 psf
	Roofing Finish	4 psf
	Total Dead Load	54 psf
Typ. Floor Dead Load	Material	Design Weight
	3" Hambro Slab	38 psf
	5" Hambro Slab	63 psf
	M/E/P	5 psf
	Ceiling Finishes	3 psf
	Total Dead Load	46-71 psf

Alternate Framing Systems

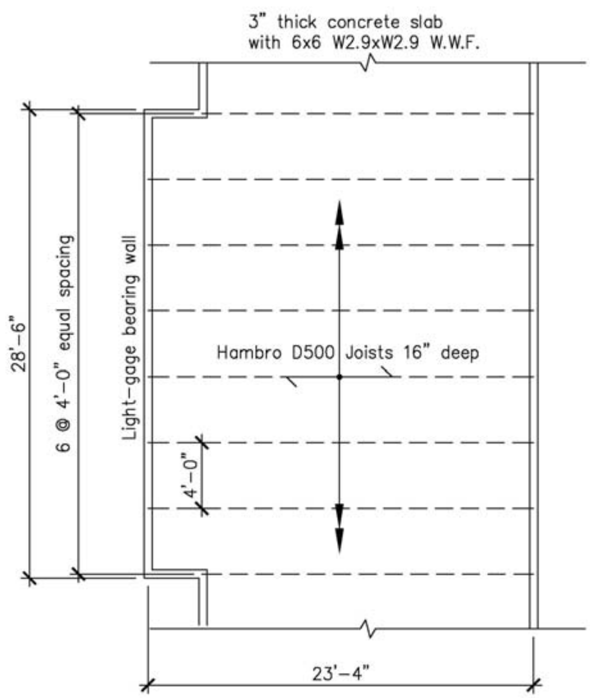
System 1: Hambro Floor System (Existing)

Description of the System

The Hambro Floor System is a proprietary product developed by Canam Group. This system consists of an open web joists and a concrete slab with W.W.F. as its reinforcing. The joists are shaped with a special bar on the top that is designed to protrude into the slab and help form composite action. The joists run a single direction and can rest on many various other structural supports such as masonry walls, concrete beams, steel beams, precast walls, etc. The slab behaves as a continuous one-way that carries the loads transversely to the joists.

System Design & Evaluation

Designed System



F'c (of the slab) = 3000 psi
Fy (of the W.W.F) = 60,000 psi
Fy (of the joist) = 50,000 psi

Overall system depth = 19 inches

Structural Assumptions:

The structural assumptions for this case are that the design is based off of the requirements so to fit within the scope that the engineer prescribed. The recommended live loads were used and matched to Hambro but Hambro used a larger dead load then we needed. Also Hambro's design chart takes a load factor of 1.7 for both live and dead. Finally we chose the four 4'-0" spacing because this is the same size at typical formwork to fit between the joists when pouring the slab due to no decking is used in the end result. Finally the light-gage bearing wall was not considered in this design.

<u>Evaluations</u>

Structural:

Structurally this system seems reasonable for the design and layout of Building 7. The joists and slab (19”deep overall) meet the required depth (24” deep) to be fitted into the ceiling cavity. The designed joists are over designed “depth-wise” to allow larger for opening in the web so ductwork can be placed through it, this should be more than adequate to control live loads.

On the other hand due to the thin slab thickness (3”) and relative flimsiness of the joists, vibration can be an issue. Also the connections need to be welded to the distribution tubes on the bearing walls thus leaving more error for mistakes. The W.W.F. also needs to be draped over the joists and be laid in the wave pattern; this reason could pose a problem for getting W.W.F. to lay properly. This can leave room for a structural weakness of the slab.

Architectural:

This system, on the basis of not impacting the architecture is very good. The main reason for this is that the system has the ability to sit on any wall as long as they can carry the load. This leaves more freedom for the architect to no have to worry about the columns interfering with their space layout. It is felt that this is a key reason why this system is chosen. This system also has very good acoustic properties as described by the technical manual.

Construction:

From a construction stand point this system can be fast to build depending on the supports the joists bear on. In the case of Building 7 the bearing members are bearing walls. This system has draw backs for you need the bearing walls up before the joists can be placed and the slab must be poured before the next floor is erected. This can be time consuming and difficult especially when moving equipment around the floor plan do to the many bearing walls.

Advantage & Disadvantage

Advantages

- Lightweight system
- Can obtain high fire ratings
- Good acoustic properties

Disadvantages

- Possible vibration issues
- Harder to apply fire proofing
- Limited configurations of joists, per design guide

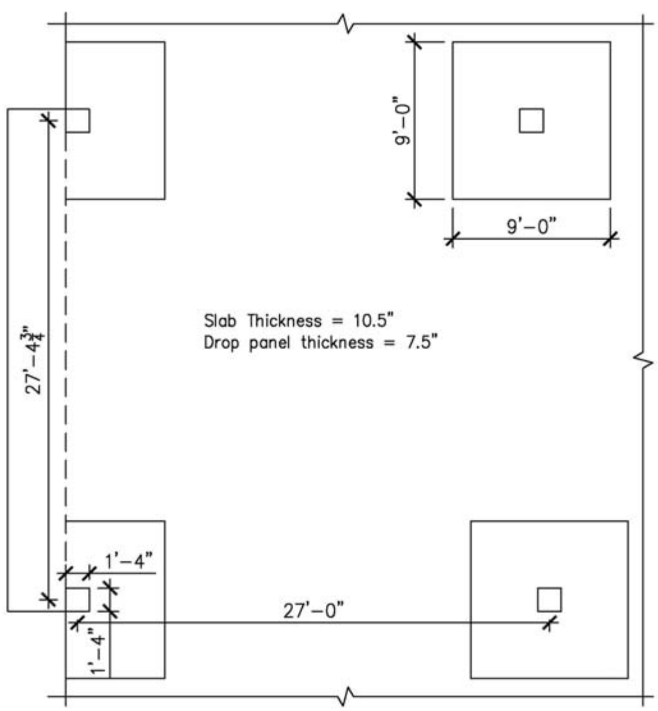
System 2: Two-Way Flat Slab with Drop Panels

Description of the System

The two-way flat slab system with drop panels is an all concrete floor system reinforced with standard size reinforcing bars. Edge beams can be added around the perimeter of the floor if needed to help carry and transfer the loads near the outer bays. The system is based on the fact that the column carries the entire load directly from the slab. The slab is a single thickness except where the drop panels form around the column. The drop panels are used to help increase the stiffness and also resist critical shear issues near the column.

System Design & Evaluation

Designed System



F'c = 4000psi
Fy (rebar) = 60,000psi

Overall system depth = 10.5 inches
depth with drop panels = 18 inches

Column Strip Reinforcing Bars:
- Top Ext. = (12) #5 bars
- Bottom = (12) #7 bars
- Top Int. = (20) #5 bars

Middle Strip Reinforcing Bars:
- Bottom = (15) #5 bars
- Top = (9) #6 bars

Structural Assumptions:

The structural assumptions for this system are that we are able to use the CRSI design manual to design the bay. This manual is based on the direct design method (DDM). The current building's layout does not meet the requirements of DDM. The Equivalent Frame Method (EFM) is required for we don't have 3 continuous bays in each direction. The DDM method was chosen for simplicity given this report deals with schematic design but if this system seems viable a more rigorous model and the use of EFM would need to be done.

The bay size of this system changed in the building so that there are only two spans in the short direction instead of having a third tiny bay. The small 3”-4”cantilever was ignored at this stage but would have an effect on the moments and reinforcing bars supporting the cantilever. On this bay there is a corridor live load near the right columns that is higher than the rest of the bay’s live load. For this technical report an average based on area was used to determine an effective live load over the entire bay.

Evaluations

Structural:

This system has the potential for a good alternative floor system for Building 7. The majority thickness at the center of the bay is 10.5” thick which will allow for more MEP space. This thickness is rather large for the bay size but was based off of Table 9.5C so deflections were not needed. If viable for Building 7 then a thinner slab can be analyzed and deflection calculations can be performed.

The down side to this system is that it is very heavy and can lead to foundation issues especially since the bearing capacity is rather low. This system may require a completely different foundation configuration. Also note that the reinforcing was based off of CRSI and it uses different bar sizes in different areas. If chosen a more uniform bar size throughout would be chosen for constructability.

Architectural:

The only primary effect of this system on the architecture is that the columns maybe become large as you travel down the building. The larger the columns become the harder they will be to conceal within the walls or placed where the arrangement of the spaces conceal their locations. Should this system be chosen as a viable alternative then an architecture breath may be needed to consider the impact of large columns in spaces.

Construction:

This system has both benefits and disadvantages. A benefit is that the formwork is reusable and the construction of the formwork is fast. Also the availability of the concrete itself is easy to come by for it doesn’t have any special admixtures. A disadvantage of concrete flat system is that it needs to be shored in place until the concrete has developed enough strength to carry its own load. This will limit how fast the floors can be constructed and occupied thus possibly resulting in a longer overall construction schedule.

Advantage & Disadvantage

Advantages

* Shallow floor depth & no beams to work MEP systems around
* Decreased vibrations due to concrete
* No fireproofing needed
* Reusable formwork

Disadvantages

* Heavier system can cause foundation issues
* Shoring and longer concrete placing time is needed
* More formwork around drop panels needed

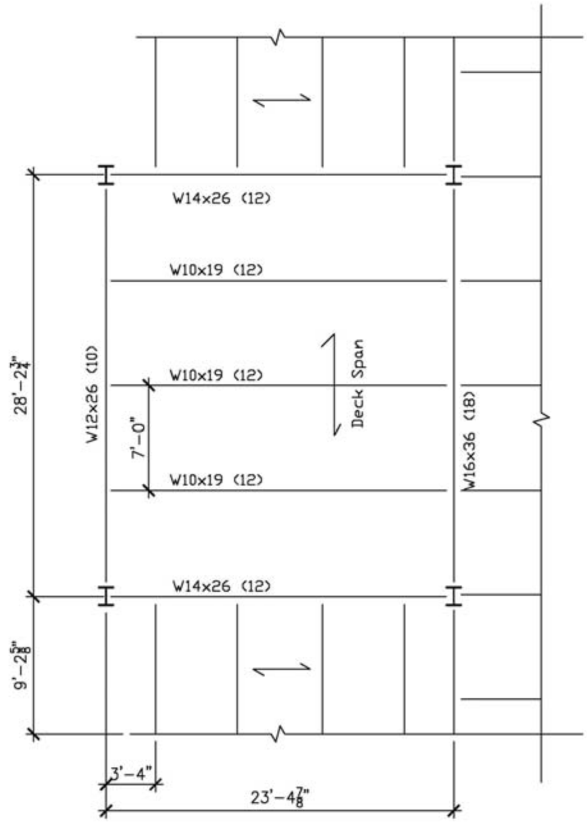
System 3: Composite Steel Deck & Beams

Description of the System

The composite steel system is a combination of steel columns, typically, at the corners of the bays with rolled steel W-shapes as girders spanning from column to column. From here in a chosen direction are infill beams spanning that are also rolled W-Shapes. Each beam is design to act compositely so that the concrete takes part of the compression force. The shear force needs to be transferred between the beam and the deck for composite action to work. This is typically done with either composite deck designed to transfer shear or by the use of shear studs.

System Design & Evaluation

Designed System



F'c (of the slab) = 3000psi
Fy (of the studs) = 60,000psi
Fy (of the steel) = 50,000psi

Majority system depth = 16inches

2VLI22 composite metal deck (3 span) with LWC
Total depth of deck = 5.25"
Stud size = ¾ Dia 4" long

Structural Assumptions:

The structural assumption taken when designing this system is that we can reduce the live load when permitted. Table 3-19 was used to design the section based on a guess of the PNA, then confirmed that this was satisfied. Only live load deflections were considered for this design and no construction live load. Finally 5 psf was added into the dead load to account for the beams and girders, this number was chosen by an average stated in past class examples. All beams and girders were assumed to be fully braced against lateral-torsional buckling.

<u>Evaluations</u>

Structural:

This system seems to be a very good choice for Building 7. The members are relatively small, W10x19 for the beams and W14 and W16 for the girders. This system is heavier than the existing system but less than concrete and will affect the foundation less. The current layout would have a small series of beams spanning and connecting the two larger bays on each side but would have a smaller depth allowing for an excellent spot for the mechanical ducts to be run.

The decking chosen, 2VL22 with 3.25” LWC topping, provides the required fire rating such that the deck need not be sprayed with fire proofing. The down side whoever is that the exposed steel need to have sprayed on fire proofing to gain the required 2 hr rating.

Architectural:

This system does not seem to affect the architecture of the building from looking at the layout of the spaces. Where concerns about the girder depth taking up the entire floor cavity or extra, this was considered in the layout of the spaces and the girders were strategically placed directly about the wall cavities so if need be, they can be hidden within the wall. In the case a wall is to thin it could be thickened to conceal the girders.

Construction:

This system has many advantages. A primary advantage is that the erection time for steel is fast and stories can be built quick succession. There is no need to have walls up before the next floor, allowing for free movement of the construction machinery around on that floor as compared to the other systems. If the floor system on take gravity loads only as it does in this case then the steel connections are simple pinned connections and can be made at a cheap price.

Advantage & Disadvantage

Advantages

* Faster construction
* Thinner floor thickness compared to non-composite
* Good against vibrations
* At times no deck shoring is needed
* Lighter steel shapes

Disadvantages

* Expensive connections
* Deep beams can obstruct mechanical ducts
* Installation of shear studs

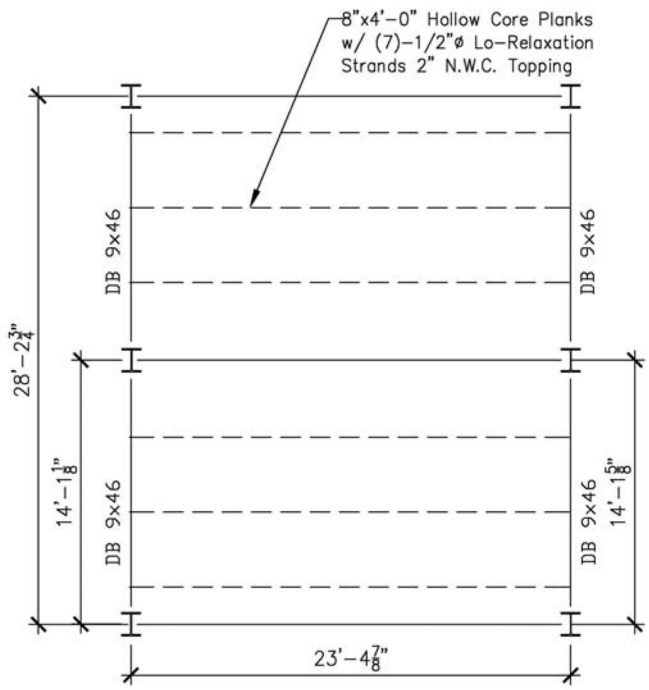
System 4: Girder-Slab

Description of the System

The Girder-Slab System is a proprietary product developed by Girder-Slab Technologies LLC. This system provides a composite action between the special steel girders that support hollow-core concrete planks on their bottom flange. These girders are open-web dissymmetric beams (D-Beams). Castellated sections of the beam are grouted solid after the planks are laid to provide the interaction and connection between the two materials. Typically on top of the planks is a poured concrete topping as a finish. The underside of this system can be exposed to the open as the finished ceiling if the correct hollow-core plank is chosen.

System Design & Evaluation

Designed System



F'c (of the H.C.P) = 6000psi
Fy (of the steel) = 50,000psi
1/2" Dia, 270K Lo-Relaxation Strands

Majority system depth = 10inches
2" N.W.C. Topping

Prestressed 8"x4'-0" Hollow Core Plank with 2-Hr. fire rating

Structural Assumptions:

For this system the primary structural assumption were that the deflections for this system were met based on the chart values given for the hollow core planks from Nitterhouse Concrete. No live load reductions were performed on this system to give a worse case result when choosing out of the tables. The beams running parallel to the planks were not designed because they are not supporting any load; instead they connect the columns only to provide stability.

<u>Evaluations</u>

Structural:

This system seems to be very reliable and feasible for Building 7. The primary benefits are that the floors are extremely thin (10” total) resulting in allowing more floor cavity of other building systems. A negative side to this system is that the span of the D-Beam is limited in load carrying and deflections requirements. In the design it was necessary to add extra columns. A further look at this implication and also the limited D-Beam sizes will need to be considered if this system is viable.

Architectural:

This system doesn’t affect the architecture of Building 7 except where the extra columns would be required. In this case a architectural breath would be need to see if all extra required columns can be hidden with spaces and wall or if the spaces themselves need to be redesigned to properly accommodate this new column gird. Hollow core planks do provide better acoustic properties due to their mass and this could be of benefit for this system has a great floor slab thickness than the original, being a dorm this could have a great impact.

Construction:

This system is has some great advantages for Building 7 is that the erection and construction time to build this system are relatively short allowing for the floors to be erected in a shorter time. The negative side to this system is that since there are two proprietary products, the planks and the D-Beams, the lead time associated with these will be much higher than other systems.

Advantage & Disadvantage

Advantages

* Very shallow floor depth
* Light weight
* Ease of construction
* Noise reduction form hollow core plank

Disadvantages

* Smaller column grid spacing
* Steel fire protection is required
* Possible vibration issues
* Limited D beam sizes

Floor Systems Comparison

typical Bay Systems				
Criteria	Hambro Floor system	Two-Way Flat Slab with Drop panels	Composite Steel Framing	Girder-Slab
Relative Cost	$10.34 per S.F.	$16.70 per S.F.	$19.00 per S.F	$13.08 per S.F
Structure Depth	19" throughout the bay	10.5" @ the center of the bay	16"@ the center of the bay	10" throughout the bay
Structure Weight	43 psf	131.3 psf	50 psf	63 psf
Fireproofing	No spray FP but gypsum board ceiling req.	No additional FP required	SOFP needed	SOFP needed
Vibration	Average	Good	Good	further investigation needed
Lead Time	Long	Short	Medium	Long
Construction Difficulty	Easy	Medium	Easy	Easy
Formwork	Yes for between joists	Yes for the entire system	No	No
Fire Rating	2 hr with UL Design G-229	2hr with carbonate Aggregate needs 3/4" clear cover	2 hr with UL Design No. 916	2 hr with UL Design K912

Conclusion

The results of the preliminary designs conducted in this report were aimed to generate a better understanding of basic floor framing systems and how they might be a better alternative structural floor system for Building 7. Each framing system was designed using basic preliminary (schematic) methods and assumptions, and then examined for its feasibility on different discipline fronts. While none of the systems should be altogether eliminated, some are better than others.

None of the systems should be eliminated completely, but some systems have greater advantages over other systems. The two-way flat slab system was designed based on certain constraints that could be adjusted in an attempt to lighten the system and also thin the slab more if this system is to be kept. This system would impact the foundations but also give more room in the ceiling cavity. The existing hambro system is naturally acceptable for a floor system but has limitations on building speed and also stability related to vibrations and fire ratings.

The two best systems that show enough feasibility to further look at that are: the girder slab system with hollow core planks and the composite steel and deck system. These systems are less thick in the ceiling cavity allowing for more room. Also they are two lightest systems after the existing. The disadvantages to these are they need spray one fire proofing. The cost involved could be offset from the original system due to each floor can be built without bearing walls and the floors plans can be open to allow for faster construction. The construction of these systems are relatively easy compared to the over systems. So in conclusion it is recommended that these two systems are the best alternative for Building 7 and a more advanced analysis and design considering more parameters will be done in the future to see which the best is.

Appendices

The pages following this page contain the following Appendices:

A: System 1, Hambro Composite Floor System

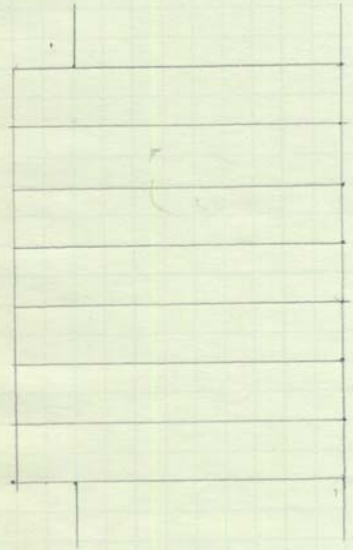
B: System 2, Two-Way Flat Slab with Drop Panels

C: System 3, Composite Steel Framing

D: System 4, Girder-Slab

Appendix A: System 1, Hambro Composite Floor System

System 1: Hambro Floor System.



LL = 40 psf
DL = 3"/12 (150) = 37.5 psf
15 psf mep / finishing

53 psf

Using the Hambro Technical manual

Slab Design

Using Table 1

3" Thickness req for Fire rating of 2 hrs.

4' spacing at the external bay/panel

$W_{tot} = 1.7(40 + 53) = 158$ psf

their recommended factor based for the chart.

Table 1 says

3" thick slab with a 1/2" Φ rod at the top of the joist

W.W.F with $f_y = 60$ ksi 6×6 W2.9×W2.9

can carry $W_u = 206$ psf > 158 psf so good.

Joist Design

using Hambro D500 Joists

(recommended for "residential" construction)

Using Table 6.

- This table considers $\Delta LL = \ell/360$, a LL = 40 & DL = 65 psf
- our LL is the same & the DL is greater than ours so this is ok to use,
- I will chose the 16" Deep Joist so openings are larger for mechanical work.

1/2

with the given Data we can span up to 33'-6"

(note: this is considering uniform loads only no concentrated loads)

Final Design

slab = t = 3"

W.W.F. with $F_y = 60ksi$

6x6 W2.9 x W2.9 , Draped between joists.

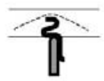
Joist = Hambro D500

16" Deep with an extra 1/2" ϕ bar at the top chord.

2/2

UL DESIGN #	RATING (hr.)	SLAB THICKNESS (in.)	CEILING	BEAM RATING (hr.)
G-003	2	2 1/2	Suspended or panel	-
G-213	2 3	3 4	Suspended or panel Suspended or panel	2 3
G-227	2	2 1/2	Suspended or panel	3
G-228	2	3 1/4	Suspended or panel	2
G-229	2	3	Suspended or panel	2
	3	4	Suspended or panel	3
G-524	1 - 2 3	2 1/2* 3 1/2*	Gypboard 1/2" Gypboard 1/2"	2 3
G-525	3	3 1/4	Gypboard 5/8"	3
G-702	1 - 2 - 3	Varies*	Spray on	-
G-802	1 - 2 - 3	Varies*	Spray on	-

Table 1 - Slab Capacity Chart (Total Load in psf)

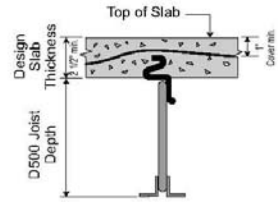
SLAB THICKNESS (t)	d	MESH SIZE F_y = 60,000 psi	4'-1 1/4" JOIST SPACING	
			Exterior	Interior
t ≥ 2 1/2"  No chair	1.6"	6 x 6 W2.0 x W2.0 6 x 6 W2.0 x W2.9 6 x 6 W4.0 x W4.0	114 157 210	123 172 230
t ≥ 3" with 1/2" Rod  (shop welded to top chord)	2.1"	6 x 6 W2.9 x W2.9 6 x 6 W4.0 x W4.0	206 279	226 306
t ≥ 3 1/2" with 2 1/2" Chair 	2.6"	6 x 6 W2.9 x W2.9 6 x 6 W4.0 x W4.0	256 347	280 380

Note: Slab capacities are based on mesh over joists raised as indicated.

TABLE 6: D500™ Clear Span Table

	Residential		Commercial		
Slab Thickness	2 1/2"	3"	3"	3 1/2"	3 3/4"
Joist Depth*	LL = 40 psf DL = 59 psf	LL = 40 psf DL = 65 psf	LL = 50 psf DL = 65 psf	LL = 50 psf DL = 71 psf	LL = 50 psf DL = 74 psf
8"	20' - 0"	20' - 0"	20' - 0"	20' - 0"	20' - 0"
10"	25' - 0"	25' - 0"	25' - 0"	25' - 0"	25' - 0"
12"	30' - 0"	30' - 0"	30' - 0"	28' - 0"	26' - 6"
14"	33' - 0"	31' - 0"	31' - 0"	31' - 0"	29' - 0"
16"	36' - 0"	33' - 6"	33' - 6"	33' - 6"	31' - 0"
18"	38' - 6"	36' - 0"	36' - 0"	36' - 0"	33' - 0"
20"	41' - 0"	38' - 6"	38' - 6"	38' - 6"	35' - 6"
22"	43' - 0"	40' - 6"	40' - 6"	40' - 6"	37' - 0"
24"	43' - 0"	43' - 0"	43' - 0"	43' - 0"	39' - 0"

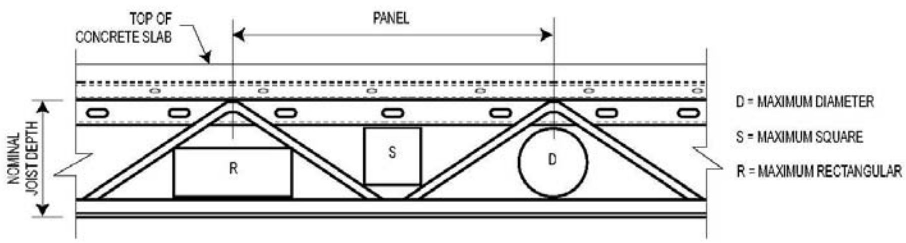
* Total floor depth = D500™ Joist depth plus slab thickness



NOTES:

- Minimum slab thickness = *2 1/2"*
- Minimum top chord cover = *1"*
- $f'_c = 3{,}000\ psi,\ F_y = 50\ ksi$
- Table reflects uniform loads only.
- Standard spacing is *4'-1 1/4"*
- Live load deflection design standard: *L / 360*
- Design clear spans, other than those shown in the above table, require additional structural review.

Maximum Duct Openings



DEPTH (in.)	PANEL (in.)	D (in.)	S (in.)	R (in. x in.)
8	20	4	4	6 x 3
10	20	6	5	7 x 4
12	24	8	6	9 x 5
14	24	9	7	9 1/2 x 6 11 x 5
16	24	10	8	10 1/2 x 6 1/2 13 x 5
18	24	11	8 1/2	11 x 7 12 1/2 x 6
20	24	11 1/2	9	12 x 7 13 x 6
22	24	12	9 1/2	12 x 8 14 x 6
24	24	12 1/2	10	13 x 8 14 x 7

NOTE: For other configurations, the maximum limits will be defined by the joist geometry.

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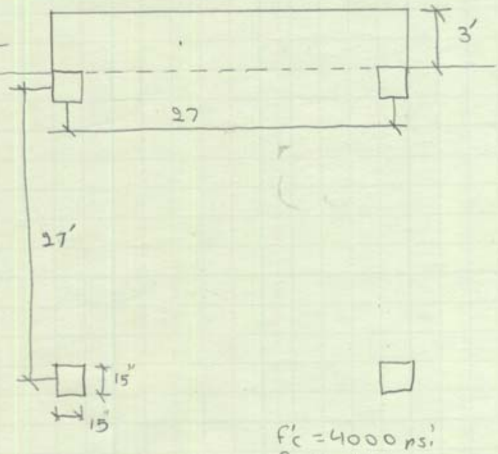
Appendix B: System 2, Two-Way Flat Slab with Drop Panels

System 2: Flatslab with drop panels



Note: for simplicity reasons
The 3' cantilever is going to be
ignored for this tech report.

This typical bay contains a
corridor with a 100 psf LL while
The rest of the bay has a 40 psf LL.
For this design a 50 psf Live load
will be used as an average based on
areas.

$f'_c = 4000$ psi
$f_y = 60$ ksi

Determine slab thickness

ACI Table 9.5C

exterior panel with drop panels/without edge beams.

$$t_{min} = \frac{l_n}{33} = \frac{(27' - 15/12)\,12}{33} = 9.4'' \rightarrow 9.5''$$

since no edge beam, increase by 10%

$9.4(1.1) = 10.3 \rightarrow 10.5''$ ← using this for CRSI chart determination.

USING CRSI pg 10-25

$w_u = 1.2(15 \quad) + 1.7(15 + 50 \quad) = 128.5$ psf
↑ old code.

using CRSI table with

$l_1 = l_2 = 27'$ (worse case)
Factored super imposed dead load = 200 psf (more conservative to round up)

For exterior panel no beams with drop panels.

Drop panel depth = 7.5"
width = 9'
16" sq column.

reinf on next page.

1/3

Reinforcing

- Column Strip

Top ext $= (12)^{\#}5$

Bottom $= (12)^{\#}7$

Top int $= (20)^{\#}5$

- Middle Strip

Bottom $= (15)^{\#}5$

Top $= (9)^{\#}6$

Moments

- Edge $= 222.6^{k-ft}$
Bottom $= 445.2^{k-ft}$
Interior $= 559.3^{k-ft}$

Note! This Design is based on a DDM. This condition is met in one direction but not the other. (the shorter direction is only 2 bays deep which does not meet the 3-continuous bay limit)

This bay size was chosen because the reinforcing will be the same in both directions.

A more accurate model will need to be done/created using EFM, if this system is viable for building 7.

Shear Check

Assuming $d = 1''$ shorter than Slab thickness

$d = 18 - 1 = 17''$

crit section $= 13.5 - 0.625 - 1.42$
$= 11.5'$

Wide beam action

$\omega L = 55$

$\omega D = \frac{10.5}{12}(150) = 132\ psf \quad 15_{psf}$

$\omega_u = 1.2(147) + 1.6(55) = 264.5\ psf$

$V_u = 0.265(11.5)(27) = 82.3^k$

2/3

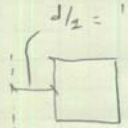
$V_n = V_c = 2\sqrt{f'_c}\, b_w d$

$= 2\sqrt{4000}\,(27\times12)(9.5)$
$= 389^k$

$\phi V_c = 0.75(389)$
$= 292^k > 82.3^k$ so ok.

Punching Shear

$d/2 = 17/2 = 8.5''$



Interior column.

$V_u = w_u\, Area.$

(crit sect dia/area)

$= 0.265(27\times27 - 5.33)$

$= 192^k$

$b_o = 132''$

$\frac{b_o}{d} = \frac{132}{17} = 7.76$

Equ 1.)

$V_c = \frac{4\sqrt{4000}\,(132)(17)}{1000} = 567.7$

$V_c = \frac{(2 + 4/1)\sqrt{4000}\,(132)(17)}{1000} = 852$

$V_c = \frac{\left(\frac{40}{7.76} + 2\right)\sqrt{4000}\,(132)(17)}{1000} = 1015$

$\phi V_c = 0.75(568) = 426^k > 192^k$ so good

Since Table 9.5C takes into account for deflections they were not calculated.

3/3

f'_c = 4,000 psi
Grade 60 Bars

FLAT SLAB SYSTEM
SQUARE EDGE PANEL — With Drop Panels
No Beams

SPAN c.-c. $\ell_1 = \ell_2$ (ft)	Factored Superimposed Load (psf)	Square Drop Panel Depth (in.)	Square Drop Panel Width (ft)	(3) Square Column Size (in.)	γ_f	Reinforcing Bars (E. W.) Column Strip (1) Top Ext. +	Column Strip Bottom	Column Strip Top Int.	Middle Strip Bottom	Middle Strip Top Int.	Total Steel (psf)	Moments Edge (−) (ft-k)	Bot. (+) (ft-k)	Int. (−) (ft-k)
h = 10.5 in. = TOTAL SLAB DEPTH BETWEEN DROP PANELS														
26	100	6.00	8.67	12	0.760	12-#5 2	15-#5	15-#5	10-#5	10-#5	2.46	151.6	303.2	408.1
26	200	6.00	8.67	15	0.798	12-#5 4	11-#7	14-#6	13-#5	11-#5	3.08	198.2	396.4	533.6
26	300	7.50	8.67	18	0.679	12-#5 2	18-#6	12-#7	9-#7	10-#6	3.83	244.7	489.4	658.8
26	400	9.00	8.67	20	0.632	12-#5 2	16-#7	13-#7	14-#6	9-#7	4.39	291.2	582.3	783.9
26	500	9.00	10.40	22	0.707	14-#5 2	12-#9	12-#8	12-#7	10-#7	5.17	336.6	673.1	906.1
26	600	9.00	10.40	26	0.701	16-#5 3	17-#8	13-#8	9-#9	9-#8	6.00	379.8	772.7	1022.5
27	100	6.00	9.00	12	0.797	12-#5 3	9-#7	12-#6	12-#5	10-#5	2.66	170.3	340.6	458.5
27	200	7.50	9.00	16	0.651	12-#5 1	12-#7	20-#5	15-#5	9-#6	3.25	222.6	445.2	599.3
27	300	9.00	9.00	18	0.634	12-#5 2	15-#7	12-#7	10-#7	11-#6	3.96	274.9	549.8	740.1
27	400	9.00	9.00	20	0.741	14-#5 4	14-#8	12-#8	9-#8	10-#7	4.88	327.9	655.8	882.8
27	500	9.00	10.80	25	0.694	16-#5 3	13-#9	13-#8	9-#9	15-#6	5.70	375.4	750.8	1010.7
28	100	7.50	9.33	12	0.750	13-#5 2	19-#5	18-#5	13-#5	11-#5	2.74	191.0	382.0	514.2
28	200	7.50	9.33	16	0.767	13-#5 4	18-#6	16-#6	12-#6	10-#6	3.50	249.3	498.5	671.1
28	300	9.00	9.33	18	0.745	13-#5 5	13-#8	26-#5	11-#7	17-#5	4.32	308.1	616.1	829.4
28	400	9.00	11.20	23	0.722	15-#5 4	13-#9	16-#7	10-#8	11-#7	5.20	365.1	730.3	983.1
28	500	9.00	11.20	28	0.644	17-#5 2	18-#8	14-#8	12-#8	10-#8	5.95	415.8	831.6	1119.4
29	100	7.50	9.67	12	0.787	13-#5 3	22-#5	14-#6	10-#6	12-#5	2.88	212.8	425.5	572.8
29	200	9.00	9.67	16	0.702	13-#5 3	15-#7	23-#5	10-#7	11-#6	3.67	277.7	555.4	747.6
29	300	9.00	9.67	19	0.763	14-#5 5	12-#9	15-#7	10-#8	19-#5	4.75	342.7	685.5	922.7
29	400	9.00	11.60	25	0.702	17-#5 3	14-#9	14-#8	12-#8	10-#8	5.68	405.3	810.5	1091.1
30	100	9.00	10.00	12	0.722	14-#5 1	17-#6	14-#6	16-#5	13-#5	3.00	236.8	473.6	637.6
30	200	9.00	10.00	16	0.763	14-#5 4	13-#8	18-#6	11-#7	17-#5	3.99	308.5	617.1	830.7
30	300	9.00	10.00	22	0.691	16-#5 3	13-#9	17-#7	18-#6	15-#6	5.07	377.6	755.2	1016.6
30	400	9.00	12.00	28	0.700	18-#5 5	16-#9	15-#8	10-#9	18-#6	5.96	444.1	888.3	1195.7
31	100	9.00	10.33	12	0.777	14-#5 3	11-#8	16-#6	13-#6	15-#5	3.29	261.9	523.8	705.1
31	200	9.00	10.33	18	0.749	14-#5 5	12-#9	15-#7	12-#7	19-#5	4.29	339.6	679.2	914.3
31	300	9.00	10.33	24	0.731	17-#5 6	18-#8	14-#8	12-#8	13-#7	5.38	416.0	832.0	1120.0
31	400	9.00	12.40	31	0.697	14-#6 4	17-#9	14-#9	11-#9	12-#8	6.43	483.9	967.9	1302.9

Table 2.3—Minimum cover for concrete floor and roof slabs

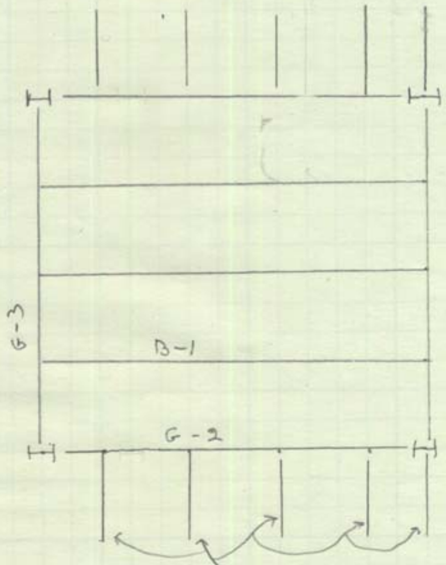
Aggregate type	Cover[A,B] for corresponding fire resistance, in. Restrained 4 or less	Unrestrained 1 hr	1 1/2 hr	2 hr	3 hr	4 hr
Nonprestressed						
Siliceous	3/4	3/4	3/4	1	1 1/4	1 5/8
Carbonate	3/4	3/4	3/4	3/4	1 1/4	1 1/4
Semi-lightweight	3/4	3/4	3/4	3/4	1 1/4	1 1/4
Lightweight	3/4	3/4	3/4	3/4	1 1/4	1 1/4
Prestressed						
Siliceous	3/4	1 1/8	1 1/2	1 3/4	2 3/8	2 3/4
Carbonate	3/4	1	1 3/8	1 5/8	2 1/8	2 1/4
Semi-lightweight	3/4	1	1 3/8	1 1/2	2	2 1/4
Lightweight	3/4	1	1 3/8	1 1/2	2	2 1/4

A. Shall also meet minimum cover requirements of 2.3.1
B. Measured from concrete surface to surface of longitudinal reinforcement

Appendix C: System 3, Composite Steel System

System 3 : Composite Steel.

equally spaced beams at 7'-0"



only considered loads from these did not design

Deck Design

LL = 40 psf or 100 psf , 15 psf
DL = 15 psf superimposed

Total Liveload :
40 + 15 = 55 psf
or 100 + 15 = 115 psf

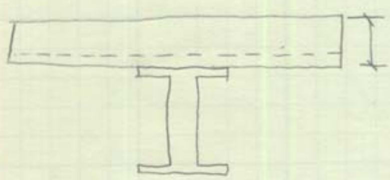
need 3 1/4" min t for fire rating
Lightweight

use the larger load so the same
Deck can be used throughout

3-Span Condition

2VLI22 has unshored length 7'-4" & can carry 236 psf at 7'-0" spacing

Beam B-1 Design.



5 1/2" using this for ease of table usage.

F_y = 50 ksi
f'_c = 3 ksi

ω_{LL} = 15 psf
ω_{DL} = 15 + 42 psf

ω_u = 1.2(15+42) + 1.6(55)
= 156.4

$b_{eff} = \begin{cases} \frac{7'}{2} = 3.5' = 42" & \text{controls} \\ \frac{23.3}{4} = 5.83' = 70" \end{cases}$

$M_u = \frac{0.174\,(7')(23.3)^2}{8} = 74.1^{k\text{-}ft}$

Try a = 1.0 in

$Y_2 = 5.5 - {}^{1.0}/_{2} = 5"$

1/6

From Table 3-19 pg. 1 W10x19 most econ.

$Y_2 = 5$
$\phi M_{pc} = 140$
$\Sigma q_n = 96.1$

check

$$a = \frac{96.1}{0.85(3)(42'')} = 0.9 < 1.0 \text{ so good.}$$

Studs: 3/4" Stud L.W.C., 3 ksi

$Q_n = 17.2$ Deck ⊥ work direction 1 stud

$$\text{studs} = \frac{2(96.1)}{17.2} = 11.2 \rightarrow 12 \text{ studs.}$$

LL Deflection

$\omega_{LL} = 55$ psf

$$\Delta = \frac{5(50)(7)(23.3)^4(1728)}{384(29000)(243)} = 0.36''$$

Limit to $\ell/360$

$$\frac{23.3(12)}{360} = 0.77''$$

9/6

Girder G-1

LL = 40 & 100 psf, 15 psf

DL = 57 psf

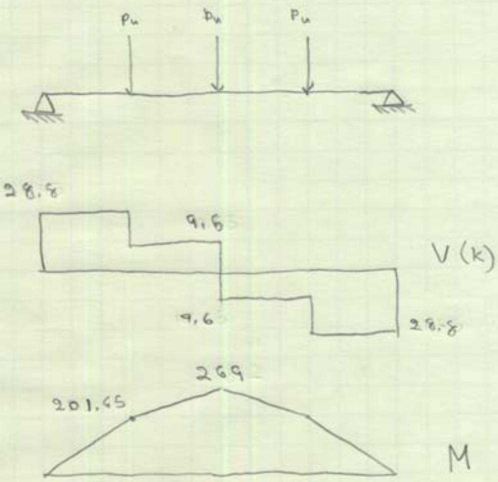
2(17.6' x 28') = 985.6 ≥ 400 ✓

$$LL_{reduction} = L_o\left(0.25 + \frac{15}{\sqrt{986}}\right)$$

= 0.73 L_o

= 55 (0.73) = 40.2 psf
= 115 (0.73) = 84 psf

w_u = 1.2(57) + 1.6(84) = 203 psf
1.2(57) + 1.6(40.2) 133



$$P_u = 0.133(7')\left(\frac{23.3'}{2}\right) +$$

$$0.203(7')(5.83')$$

$$= 19.2^k$$

M_u = 269 k-ft

Try a = 1.0"

Y_2 = 5.5 − 1/2 = 5.0 in

Table 3-19 PS 1 : W16x36

Y_2 = 5
ϕM_{pc} = 373
Σq_n = 180

$$b_{eff} = \begin{cases} \dfrac{17.6'}{2}(12) = 106'' \\ \dfrac{28'(12)}{4} = 84'' \text{ controls} \end{cases}$$

check!

$$a = \frac{180}{0.85(3)(84'')} = 0.84 < 1.0''$$

3/6

Studs:

$Q_n = 21^k$

$\text{studs} = \dfrac{2(180)}{21} = 17.2 \rightarrow 18$ studs.

LL Deflection

$w_{LL} = 55$ psf (ignoring the 100 due to very small area)

$$\Delta = \frac{5(55)(17.6')(28')^4(1728)}{384(29000)(897)} = 0.51''$$

$$\frac{l}{360} = \frac{28(12)}{360} = 1.13''$$

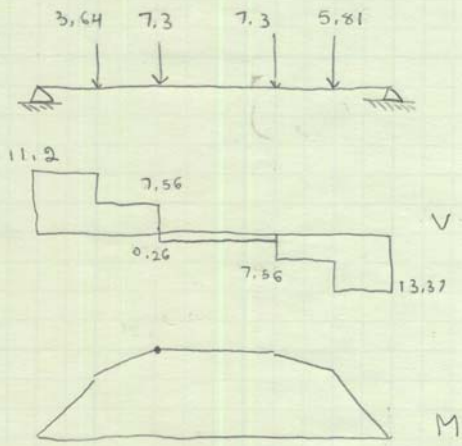
4/6

Girder G-2 Design

LL = 55 psf
DL = 57 psf

$w_u = 1.2(57) + 1.6(55) = 147$ psf

$P_u = 0.147(2.75')(9') = 3.64^k$
$P_u = 0.147(4.4)(9') = 5.81^k$
$P_u = 0.147(5.5')(9') = 7.3^k$



$M_u = 88^{k\text{-}ft}$

Try $a = 1.0''$

$Y_2 = 5.5 - \frac{1}{2} = 5.0''$

Table 3-19 pg 11! W14x26

choose high/deeper to ensure beams into it are shallower.

$Y_2 = 5$
$\phi M_{pc} = 224$
$\Sigma Q_n = 96.1$

check !
$$a = \frac{96.1}{0.85(3)(42)} = 0.9 < 1''$$

LL reduction $\left(0.25 + \frac{15}{\sqrt{500}}\right)$

$2(20 \times 12.5) = 500 > 400$

$0.92 L_o$

$w_{LL} = 51$ psf

$$b_{eff} = \begin{cases} \frac{7'}{2} = 42'' & \text{controls} \\ \frac{20(12)}{4} = 60'' \end{cases}$$

studs:

$Q_n = 17.2$

$$\text{studs} = \frac{2(96.1)}{17.2} = 11.2 \rightarrow 12 \text{ studs.}$$

LL Deflection

$w_{LL} = 40$

$$\Delta = \frac{5(55)(12)(23.5)^4(1727)}{384(29000)(465)} = 0.33$$

$$\frac{l}{360} = \frac{20(12)}{360} = 0.78''$$

5/6

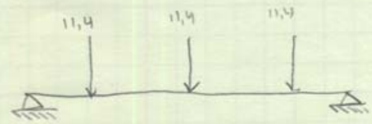
G-3 Design

LL = 55
DL = 57

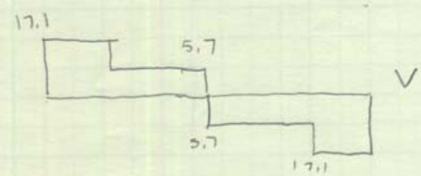
LL reduction $= L_o\left(0.25 + \frac{15}{\sqrt{653}}\right) = 0.84$ $\quad 2\left(28 \times \frac{23.3}{2}\right) = 653 > 400$

LL = 46.2

$P_u = (1.2(57) + 1.6(46.2))(7')(11.6') = 11.4^k$



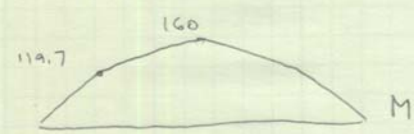
$M_u = 160^{k\text{-}ft}$



Try $a = 1.0''$

$Y_2 = 5.5 - 1/2 = 5.0$

Table 3-19, pg 1 : W12x26



Y_2 : 5
ϕM_{pc} : $204^{k\text{-}ft}$
ΣQ_n : 95.6^k

b eff = Same as G1
42" controls.

Check: $\frac{95.6}{0.85(3)(42'')} = 0.89 < 1.0''$

Studs:

$Q_n = 21$

studs $= \frac{2(95.6)}{21} = 9.1 = 10$ studs.

LL Deflections

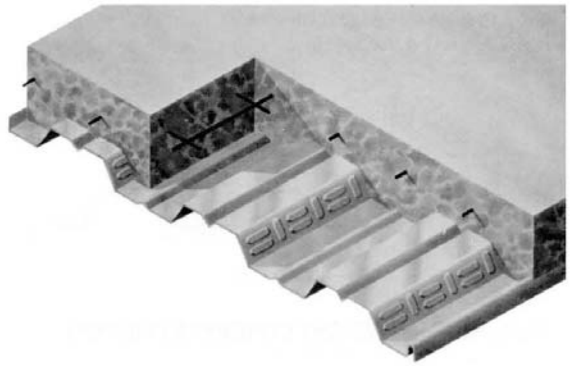
$\Delta = \frac{5(55)(11.6)(29)^4(1728)}{384(29000)(393)} = 0.77 < 1.13''$ so good

6/6

VULCRAFT

SLAB INFORMATION

Total Slab Depth	Theo. Concrete Volume Yds./ 100 Sq. Ft.	Theo. Concrete Volume Cu. Ft./ Sq. Ft.	Recommended Welded Wire Fabric
4"	0.94	0.253	6x6-W1.4xW1.4
4 1/2"	1.09	0.294	6x6-W1.4xW1.4
5"	1.24	0.336	6x6-W1.4xW1.4
5 1/4"	1.32	0.357	6x6-W1.4xW1.4
5 1/2"	1.40	0.378	6x6-W2.1xW2.1
6"	1.55	0.419	6x6-W2.1xW2.1
6 1/4"	1.63	0.440	6x6-W2.1xW2.1
6 1/2"	1.71	0.461	6x6-W2.1xW2.1



(N=14) LIGHTWEIGHT CONCRETE (110 PCF)

Total Slab Depth	Deck Type	SDI Max. Unshored Clear Span			Superimposed Live Load, PSF Clear Span (ft.-in.)														
		1 Span	2 Span	3 Span	6'-0	6'-6	7'-0	7'-6	8'-0	8'-6	9'-0	9'-6	10'-0	10'-6	11'-0	11'-6	12'-0	12'-6	13'-0
4"	2VLI22	7'-2	9'-6	9'-8	238	209	186	149	133	120	108	98	90	82	75	69	64	59	55
	2VLI21	7'-10	10'-2	10'-6	254	223	198	178	142	128	115	105	96	87	80	74	68	63	58
	2VLI20	8'-5	10'-9	11'-1	268	235	209	187	169	135	122	110	101	92	84	78	72	66	61
(t=2")	2VLI19	9'-6	11'-11	12'-4	297	260	230	206	185	168	153	141	111	101	93	86	79	73	68
	2VLI18	10'-6	12'-10	13'-3	324	285	253	227	205	187	171	158	146	136	107	99	92	86	80
30 PSF	2VLI17	11'-5	13'-8	14'-0	352	308	273	245	221	201	184	169	156	145	135	107	99	92	86
	2VLI16	12'-1	14'-4	14'-4	377	330	292	261	235	214	195	179	165	153	143	133	118	98	91
4 1/2"	2VLI22	6'-9	9'-1	9'-3	276	243	195	173	155	139	126	114	104	96	88	81	75	69	64
	2VLI21	7'-5	9'-9	10'-1	295	259	231	185	165	149	134	122	111	102	93	86	79	73	68
	2VLI20	8'-0	10'-4	10'-8	312	273	243	217	196	157	141	128	117	107	98	90	84	77	72
(t=2 1/2")	2VLI19	9'-0	11'-5	11'-9	346	302	268	239	215	195	178	142	129	118	108	100	92	85	79
	2VLI18	10'-0	12'-3	12'-8	376	331	294	264	238	217	199	183	170	136	125	116	107	100	93
35 PSF	2VLI17	10'-10	13'-1	13'-6	400	358	318	284	256	233	213	196	181	168	134	124	115	107	100
	2VLI16	11'-5	13'-8	13'-10	400	384	340	303	273	248	227	208	192	178	166	132	123	114	106
5"	2VLI22	6'-6	8'-8	8'-10	315	277	222	197	176	159	144	130	119	109	100	92	85	79	73
	2VLI21	7'-1	9'-4	9'-8	337	296	263	211	189	169	153	139	127	116	107	98	91	84	78
	2VLI20	7'-7	9'-11	10'-3	355	312	276	248	199	179	161	146	133	122	112	103	95	88	82
(t=3")	2VLI19	8'-7	10'-11	11'-4	394	345	305	272	245	223	178	162	147	135	124	114	105	97	90
	2VLI18	9'-6	11'-10	12'-2	400	377	335	300	272	247	227	209	168	155	143	132	122	114	106
40 PSF	2VLI17	10'-3	12'-7	13'-0	400	400	362	324	292	266	243	223	207	166	153	142	131	122	114
	2VLI16	10'-11	13'-2	13'-5	400	400	387	346	311	283	258	237	219	203	163	151	140	130	121
5 1/4"	2VLI22	6'-4	8'-6	8'-8	334	268	236	209	187	168	152	138	126	116	106	98	90	84	78
	2VLI21	7'-0	9'-2	9'-6	357	314	279	224	200	180	163	148	135	123	113	104	96	89	83
	2VLI20	7'-6	9'-8	10'-0	377	331	293	263	211	190	171	155	142	130	119	110	101	94	87
(t=3 1/4")	2VLI19	8'-5	10'-9	11'-1	400	366	324	289	260	210	189	172	156	143	131	121	111	103	95
	2VLI18	9'-3	11'-7	12'-0	400	400	355	319	288	263	241	195	179	164	151	140	130	121	113
42 PSF	2VLI17	10'-1	12'-4	12'-9	400	400	384	344	310	282	258	237	219	177	163	151	140	130	121
	2VLI16	10'-8	12'-11	13'-3	400	400	400	367	330	300	274	252	232	215	173	160	148	138	128
5 1/2"	2VLI22	6'-3	8'-5	8'-6	353	284	250	222	198	178	161	147	134	122	113	104	96	89	82
	2VLI21	6'-10	9'-0	9'-4	378	332	268	237	212	190	172	156	142	130	120	110	102	94	87
	2VLI20	7'-4	9'-6	9'-10	399	350	310	250	223	201	181	165	150	137	126	116	107	99	92
(t=3 1/2")	2VLI19	8'-3	10'-6	10'-11	400	387	342	306	275	222	200	182	165	151	139	128	118	109	101
	2VLI18	9'-1	11'-4	11'-9	400	400	376	337	305	278	254	206	189	174	160	148	138	128	119
44 PSF	2VLI17	9'-10	12'-1	12'-6	400	400	400	363	328	298	273	251	204	187	172	159	148	137	128
	2VLI16	10'-5	12'-8	13'-1	400	400	400	388	350	317	290	266	246	199	184	170	157	146	136
6 1/4"	2VLI22	5'-11	7'-10	8'-0	380	331	291	258	231	208	188	171	156	143	131	121	112	103	96
	2VLI21	6'-5	8'-7	8'-10	400	355	312	276	247	222	200	182	166	152	140	129	119	110	102
	2VLI20	6'-11	9'-1	9'-4	400	400	329	292	260	234	211	192	175	160	147	135	125	115	107
(t=4 1/4")	2VLI19	7'-10	10'-0	10'-4	400	400	398	356	288	259	233	212	193	176	162	149	137	127	118
	2VLI18	8'-7	10'-10	11'-2	400	400	400	392	355	323	264	240	220	202	187	173	160	149	139
51 PSF	2VLI17	9'-3	11'-6	11'-11	400	400	400	400	381	347	317	259	237	218	201	186	172	160	149
	2VLI16	9'-10	12'-1	12'-6	400	400	400	400	400	369	337	310	253	232	214	198	183	170	158

Notes:
1. Minimum exterior bearing length required is 2.0 inches. Minimum interior bearing length required is 4.0 inches. If these minimum lengths are not provided, web crippling must be checked.
2. Always contact Vulcraft when using loads in excess of 200 psf. Such loads often result from concentrated, dynamic, or long term load cases for which reductions due to bond breakage, concrete creep, etc. should be evaluated.
3. All fire rated assemblies are subject to an upper live load limit of 250 psf.
4. Inquire about material availability of 17, 19 & 21 gage.

COMPOSITE

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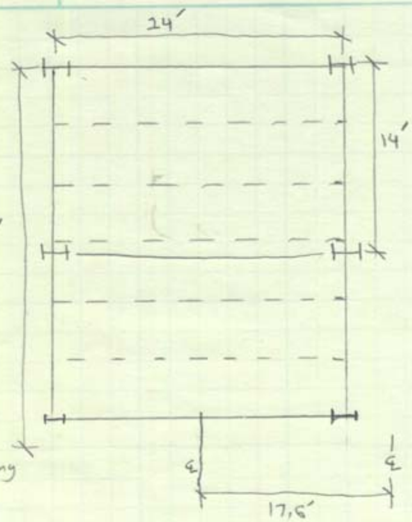
Restrained Assembly Rating	Type of Protection	Concrete Thickness & Type (1)	U.L. Design No. (2,3,4)	Classified Deck Type		Unrestrained Beam Rating
				Fluted Deck	Cellular Deck (5)	
2 Hr. (continued)	Sprayed Fiber	2" NW&LW	859 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,3 Hr.
		2 1/2" NW&LW	822 *	2VLI,3VLI	2VLP, 3VLP	1 Hr.
			825 *	1.5VLI,2VLI,3VLI	2VLP, 3VLP	1,1.5,2 Hr.
			831 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2 Hr.
			832 *	1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2,3 Hr.
			833 *	1.5VLI,2VLI,3VLI	2VLP, 3VLP	1.5 Hr.
			847 *	2VLI,3VLI	3VLP	1,1.5,3 Hr.
			858 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,4 Hr.
			861 *	12VLI,3VLI		1,1.5 Hr.
			870 *	1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,2 Hr.
			871 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,3 Hr.
		2 1/2" LW	862 *	2VLI,3VLI		1 Hr.
		2 1/2" NW	864 *	3VLI	3VLP	1.5 Hr.
		3 1/4" LW	860 *	2VLI,3VLI		1,1.5,2 Hr.
	Unprotected Deck	3 1/4" LW	733 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5Hr.
			826 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2 Hr.
			840 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
			902 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
			907 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,2 Hr.
			913 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1 Hr.
			916 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2,3 Hr.
			918 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
			919 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
			920 #	2VLI,3VLI	2VLP, 3VLP	1.5 Hr.
		4 1/2" NW	902 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
			916 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2,3 Hr.
			918 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
			919 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.

Appendix D: System 4, Girder-Slab System

System 4 Girder - Slab



Precast Hollow Core Plank Design

DL = 15 psf
LL = 40 psf + 15 psf
partition.

Topping 25 psf

$W_u = 1.2(15) + 1.6(55) = 106$ psf

Try 7 - 1/2" ϕ 8" x 4' plank, with 2" Topping

24' span.

106 < 216 psf so good.

Plank capacity

$W_u = 1.2(15 + 25 + 61.3) + 1.6(55) = 209.5$ psf x 4' = 838 plf

$M_n = \frac{0.838(24')^2}{8} = 60.4^{k\text{-}ft} < 147.7 = \phi M_n$ so good.

$\boxed{\text{Use } 8" \times 4' \text{ with } 2" \text{ Topping w/ (7) } 1/2\phi \text{ lo-relax strand}}$

Beam Design

Try DB 9x46

+ Initial load Precomposite:

$M_{DL} = \frac{(61.3^{psf} \cdot 17.6)(14')^2}{8} = 26.4 < 84^{k\text{-}ft}$

$\Delta_{DL} \; \frac{5(0.061)(17.6')(14')^4(1728)}{384(29000)(195)} = 0.2"$

+ Total load Composite.

$M_{super} \; \frac{(17.6')(0.055\text{ klf})(14)^2}{8} = 41^{k\text{-}ft}$

$M_{tot} = 26.4 + 41 = 67.4^{k\text{-}ft}$

1/2

$S_{req} = \frac{(67.4^{k\text{-}ft})(12)}{0.6(50)} = 27 in^3 < 68.6 in^3$, so ok.

$\Delta_{sup} = \frac{5(17.6)(0.095)(14)^4(1728)}{384(29000)(356)} = 0.14 in < 0.47 in$ so good.

+ Compressive Strength on concrete.

$N = \frac{29,000}{57,000\sqrt{6000}} = 6.57 \rightarrow S_{tc} = 6.57(68.6) = 450.7 in^3$

$f_c = \frac{41(12)}{450.7} = 1.09$ ksi

$F_c = 0.45(6000) = 2.7^{ksi} > 1.09$ so good.

+ Bottom flange tension Stress

$f_b = \frac{26.1(12)}{50.8} + \frac{41(12)}{80.6} = 12.3$ ksi

$F_b = 0.9(50) = 45$ ksi > 12.3 so good.

+ Check Shear

$w_{tot} = 61.3 + 15 + 55 + 25 = 156.3$ psf

$w = \frac{156.3(17.6')}{1000} = 2.75$ k/ft

$R = 2.75(14)/2 = 19.25^k$

$f_v = \frac{19.25}{0.375(5.75)} = 8.9^{ksi}$

$F_v = 0.4(50) = 20$ ksi > 8.9 so good.

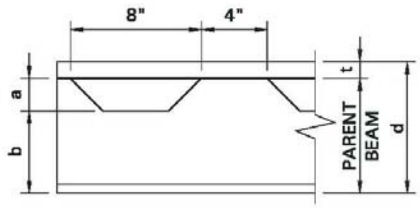
For simplicity use same beam on both ends even though lefts trib area is slightly smaller.

$\boxed{\text{Use DB } 9 \times 46}$

2/2

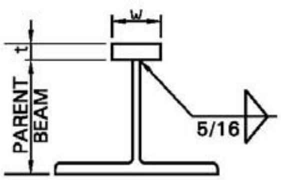
D-Beam® Dimensions Table

Designation	Web Included		Depth	Web	Parent Beam			Top Bar
	Weight	Avg. Area	d	Thickness t_w	Size	a	b	w x t
	lb/ft	in^2	in	in		in	in	in x in
DB 8 x 35	34.7	10.2	8	.340	W10 x 49	4	3	3 x 1
DB 8 x 37	36.7	10.8	8	.345	W12 x 53	2	5	3 x 1
DB 8 x 40	39.8	11.7	8	.340	W10 x 49	3	3.5	3 x 1.5
DB 8 x 42	41.8	12.3	8	.345	W12 x 53	1	5.5	3 x 1.5
DB 9 x 41	40.7	11.9	9.645	.375	W14 x 61	3.375	5.25	3 x 1
DB 9 x 46	45.8	13.4	9.645	.375	W14 x 61	2.375	5.75	3 x 1.5



D-Beam® Reference Calculator is Available on Website. www.girder-slab.com

D-Beam® Properties Table



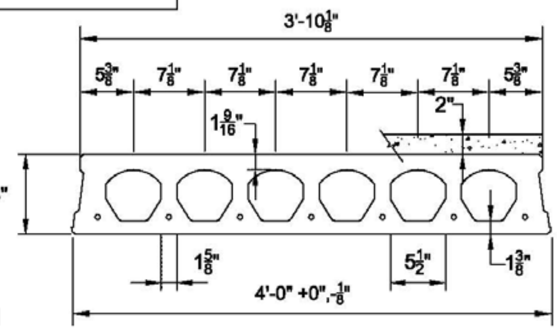
Designation	Steel Only / Web Ignored						Transformed Section / Web Ignored				
	Ix	C bot	C top	S bot	S top	Allowable Moment Fy=50 KSI f_b=0.6 Fy	Ix	C bot	C top	S bot	S top
	in^4	in	in	in^3	in^3	kft	in^4	in	in	in^3	in^3
DB 8 x 35	102	2.80	5.20	36.5	19.7	49	279	4.16	4.40	67.1	63.5
DB 8 x 37	103	2.76	5.24	37.3	19.7	49	282	4.16	4.42	67.7	63.8
DB 8 x 40	122	3.39	4.61	36.1	26.5	66	289	4.26	4.30	67.9	67.2
DB 8 x 42	123	3.35	4.65	36.9	26.5	66	291	4.26	4.32	68.4	67.5
DB 9 x 41	159	3.12	6.51	51.0	24.4	61	332	4.27	5.35	77.7	62.1
DB 9 x 46	195	3.84	5.79	50.8	33.7	84	356	4.43	5.20	80.6	68.6

Prestressed Concrete
8"x4'-0" Hollow Core Plank

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

$A_c = 301$ in.^{2} Precast $S_{bc} = 617$ in.^{3}
$I_c = 3134$ in.^{4} Topping $S_{tc} = 902$ in.^{3}
$Y_{bc} = 5.09$ in. Precast $S_{tc} = 1076$ in.^{3}
$Y_{tc} = 2.91$ in. Wt.= 245 PLF
Wt.= 61.25 PSF



DESIGN DATA

1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI.
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   4-1/2"Ø, 270K = 92.3 k-ft
   7-1/2"Ø, 270K = 147.7 k-ft
7. Maximum bottom tensile stress is $7.5\sqrt{f'c} = 580$ PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI. Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span. A lesser thickness might occur if camber is not taken into account during design, thus reducing the load capacity.
13. Load values to the left of the solid line are controlled by ultimate shear strength.
14. Load values to the right are controlled by ultimate flexural strength or fire endurance limits.
15. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
16. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

SAFE SUPERIMPOSED SERVICE LOADS — IBC 2003 & ACI 318-02 (1.2 D + 1.6 L)

Strand Pattern		SPAN (FEET) 17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35
4 - 1/2"ø	LOAD (PSF)	275	236	203	175	150	129	111	95	81	68	57	47	38						
7 - 1/2"ø	LOAD (PSF)	367	342	319	299	281	265	243	216	193	171	153	136	121	107	95	84	74	63	53

NITTERHOUSE CONCRETE PRODUCTS
2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17201-0813
717-267-4505 Fax 717-267-4518

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. The allowable loads shown in this table reflect a 2 Hour & 0 Minute fire resistance rating.

08/10/07

8SF2.0T