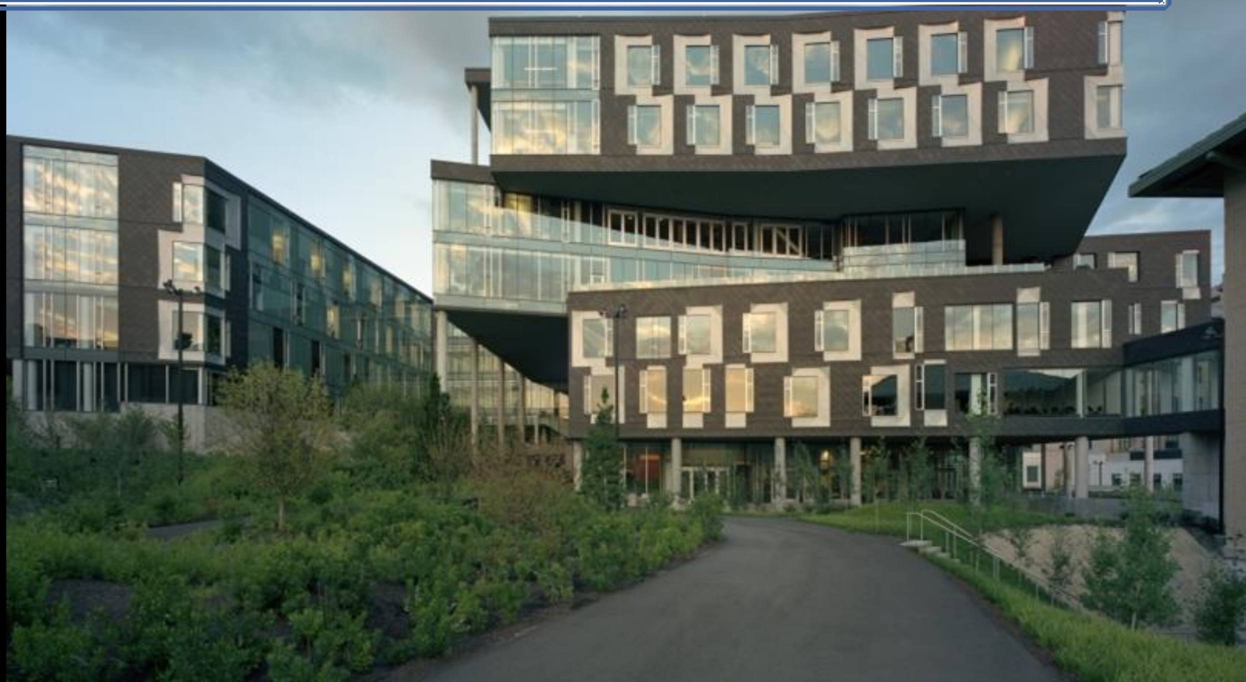
THE UNIVERSITY SCIENCES BUILDING

NORTHEASTERN, USA



Technical Report 2

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Executive Summary

The purpose of Technical Report 2 is to investigate the analysis and design of four (4) floor systems for The University Sciences Building. These systems include the existing composite slab and beam system, one way slab with beams, two way flat plate system, and hollow core planks on steel framing. Since the existing system utilizes multiple systems on different levels, although predominantly composite, the alternative systems of interest are to be implemented at every level. These investigations were executed by hand calculations and with assistance from spSLAB for the two concrete systems. These systems were designed and analyzed with respect to the same representative bay of 27'x30'. All calculations for technical Report 2 can be found in Appendices A through D.

These systems were compared to each other by topics of general impact (weight, cost, floor depth), architectural impact (fire rating, floor to floor height), structural impact (foundations and lateral system), serviceability (deflections and vibration), and construction impact (schedule and constructability). It is important to note that for the purpose of design the representative bay provides a simple and accurate design but it will be vital to consider these systems in areas of irregularity if the system is to be further analyzed.

The existing composite slab and beam floor is the system to which the alternative systems are compared to. This system is a 2" Vulcraft 2VLI18 composite deck with 4 ½" topping. It is framed into W14x22 infill beams spanning 27' that frame into a W24x68 girders spanning 30'. This system weighs nearly 79 psf and costs approximately $20.30 to construct. The main issue with this system is its depth of 30.2" but is still a very feasible system considering the designed floor to floor height of 14'.

The first alternative system to be designed is a one way concrete slab with longitudinal and transverse beams. A slab of 6" was designed with girder and beam sizes of 12"x20". At 102 psf, it costs nearly $17.90/SF to construct. Due to its constructability concerns for not only the formwork but the consideration of this system in areas or irregularity, it was eliminated from consideration. The hand calculations and spSLAB verification can be found in Appendix B.

Next is a two way flat plate system with a designed thickness of 12". This system is the shallowest of those considered but the heaviest at 150 psf; yet the cheapest at $16.35/SF. Due to its minimal thickness, cost, and rather easy constructability, it is a viable alternative.

Finally a precast hollow core plank on steel framing system was designed with a Nitterhouse 8" x 4' hollow core plank spanning 27'. The concerns with this alternative system are the depth of the system (29.4"), cost ($26.83/SF), and construction difficulty, which has eliminated the system from further consideration.

Building Introduction

The University Sciences Building is a pioneering sciences facility pushing the envelope on innovative research and education. The 209,000 square foot dual building is strategically nested on a 5.6 acre site on the urban university in Northeastern, USA. The building includes 300+ offices, state-of-the-art laboratories, classrooms, lecture halls, a 250 seat auditorium, and a 147 space parking garage. The University's standard building aesthetics include a symmetrical layout and typically a beige brick veneer. The USB's extravagant cantilevers and complex building enclosures express the University's commitment to innovative architecture and sustainability.

Figure 1 – Google Maps aerial view of site

The building was designed around the common idea of atrium space and the majority of other open spaces exposed to light, predominantly through curtain wall systems. The intent was to let these open areas serve as collaborative spaces for interaction among students, researchers, and professors. The featured atrium of the building is its 3 story helical structure, which serves as a ramp to levels 3–5 with classrooms intermediately located through its core (Figure 2).

Figure 2 – Helical ramp

The sophisticated and 'edgy' design of the façade expresses the University's movement to push the envelope for not only the sciences but also its architecture. The material used to clad the building is a unique zinc material. Both the black zinc molded squares and the sliver aluminum window trim give the building a different and uneven appearance which sparks interest towards the building.

Figure 3 – South Cantilever

Each floor's different floor plans presents one of a kind overhangs and cantilevers which really express the structure of the building (Figure 3). The placement of key structural components are carefully placed to preserve optimal function from floor to floor.

Structural Overview

The University Sciences Building sits upon a Site Class C (Geotechnical Report verified with ASCE 7-05 Chapter 11) with drilled 30'' caissons, caisson caps, spread, continuous, stepped footings, grade beams and column footings. Levels 1-3 of Building 1 and level 4 of Building 2 use concrete beams and slabs with a combination of concrete columns and steel encased columns. The upper floors of both buildings use a composite beam/slab system and continue with steel and encased columns. The lateral systems consists of shear walls and braced steel frames. The shear/retaining walls start from the grade and end at various heights around the building. The braced frames are composed of wide flange chords with HSS diagonals that also reach various heights.

Foundations

The design and analysis of foundations are in accordance with the geotechnical report provided by Construction Engineering Consultants, Inc and ASCE 7-05. Schematic and design development stages were conducted with a safe assumpiton that the soil class was solid rock. The majority of the University's soil has been geologic lly tested to show this. As time proceeded and the geotechincal report was released, it was found that the site class was actually C. This induced a complete redesign of Building 2's foundation along with using a new 'flowable fill' for backfill for Building 1. Flowable fill is entrained with fly ash, cement, and other agents to generate negliable lateral pressure on surrounding foundation walls but maintains a compressive strength of 500 psi (Calculations for this are not provided in this technical report).

In has been concluded from the structural drawings that the allowable soil/rock bearing pressures for spread footings on weathered shale are 6000 psf. Likewise for siltstone/sandstone allowable pressures are 12000 psf. In addition, caissons socketed 5' into siltstone/sandy stone are to have an allowable pressure of 50 ksf.

The building load path initiated from the floor systems to columns and then to their respective caissons or interior column footings. For exterior perimeter caissons, they are connected with grade beams to interior caissons or grade column foundations. The slab on grade (SOG) is to be poured onto compacted soil to withstand 500 psf and a minimum of 6" of compacted Penn DOT 2A or 2B material. Furthermore, the fill must be compacted to 95% of the dry density per ASTM D 1557. A vapor barrier is then required to lie between the fill and the slab.

Expansion joints should be used between the footings and floor slabs to minimize differential settlement stresses. The slab on grade is designed to have an f’c of 4500 psi of normal weight concrete and a mix class C.

Floor Systems

Due to the complexity of the floor layouts, typical bays occur irregularly and are comprised of a variety of beam sizes and lengths (Refer to appendix E for floor plans). In Building 1, floors 1 - 3 utilize concrete reinforced beams that range in size from 50”x24” to 10”x12”, integral with formed 6” reinforced slabs. The upper floors utilize composite and non-composite beam construction. These floor systems range from 1” x 20 gauge metal deck with 5” reinforced concrete topping to 2” x 18 gauge metal deck with 4.5” reinforced concrete topping. The most recurring slab is a composite 2”x18 GA deck with 4.5” normal weight concrete topping, which is found in both building 1 and 2 on floor 4-roof. Areas on levels 4 and 5 of Building 1 brace the metal decking between beams and girders with L4x4x3/8”.

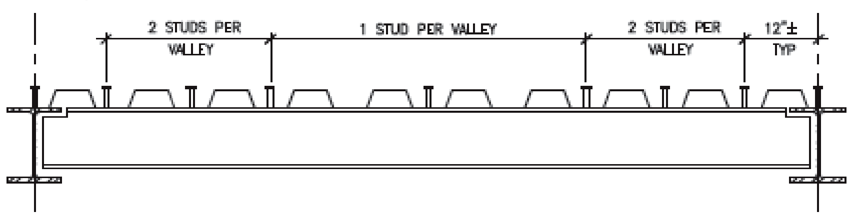


Figure 4. Perpendicular Decking Section – Case 3

The composite and non-composite decks are placed with the ribs of the deck perpendicular to the infill beams to maintain the rigidity of the system. This proved to be a conflict to construct with the placement of shear studs. Where it is efficient to place studs along the length of the beam uniformly normal to the valley and peaks of the deck, it was extremely difficult to maintain this layout with the odd angling placement of particular beams (Figure 4).

Framing System

The USB has three different types of columns, reinforced concrete, encased A992 steel with concrete, and A992 wide flange steel. Reinforced concrete columns vary in size from 24” to 18” diameter circular columns and 16”x18” to 33”x37” rectangular columns. Also, wide flange columns range from W12x40 to W21x210. Levels 1 and 2 of Building 1 have both circular and rectangular concrete columns. Level 3 of Building 1 uses circular/rectangular encased steel and circular reinforced concrete columns. This trend dissipates as you transverse up the building converting to steel columns, likewise with Building 2. Framing girders are then connected to these columns with simple and complex connections. (e.g. pin-pin, moment). The layout of the girders and beams have been arranged with much complexity and provide a challenge for analysis. This complexity not only produced adversity for the fabricators and erectors, increased the price of the building, but also delayed the floor to floor connection schedule. The most nearly identified typical bay has 30'x27' dimensions. .

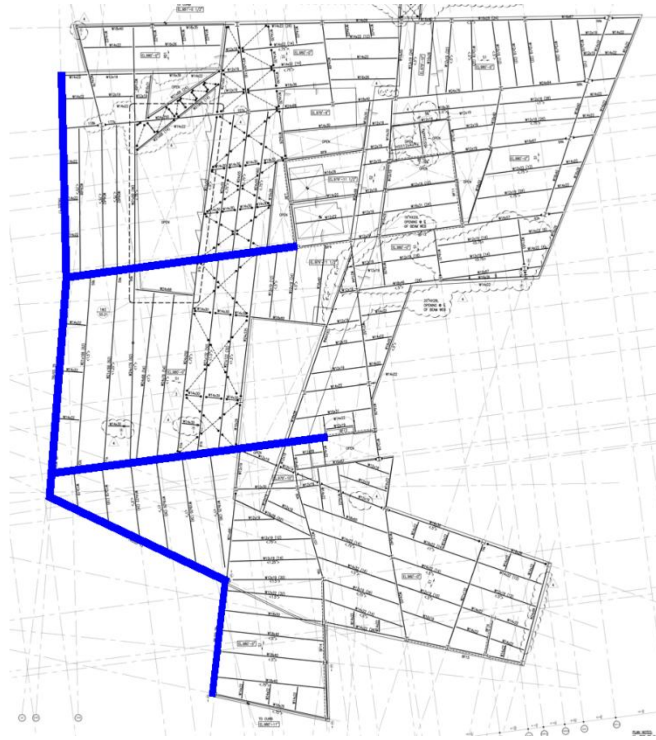


Figure 5. Highlighted truss elements from Building 1 Level 8.

An intricate and vital part of this structural framing system is the truss system in Building 1 which varies in height from Level 6 to the roof (Figure 5). These trusses are comprised of chord sizes as big as W30x292 and intermediate bracing elements as small as W14x53. Due to the complex cantilevers and floor plans, a system needed to be implemented to handle the buildings loads. The system is well hidden in the building and parts where it can be seen (through some windows) presents and interesting look for the building.

Lateral System

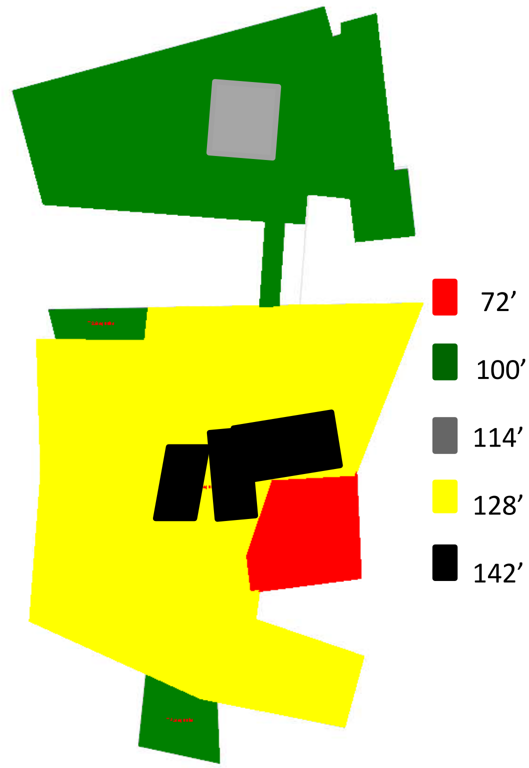
The most common lateral force resisting system in The USB is braced frames. The USB utilizes 16 different braced frames between the two buildings. The majority of these are framed within a single bay. Others are 'Chevron' braced frames between two bays and a few span through 3 or more bays.

Figure 6. Level 6 Braced Frames and Shear walls

In Building 1 these braced frames are connected to shear walls were the load is taken from steel elements to concrete elements. These concrete elements are generated from the formed concrete walls lining the 147 parking spot garage. This adds a considerable weight to the building. All shear/retaining walls employed in building are kept on the lower floors, which has been assumed to retain the majority of the weight on a lower elevation. This doesn't hold true for three shear walls that start with a connection to a caisson cap at grade and rise 72' to level 6. Refer to Figure 6 for the layout of brace frames (red) and shear walls (green) on Level 6. The challenge for Technical Report 3 will be to figure out how these lateral force resisting systems receive force on all floors of the building.

Roof System

This dual building system has 5 different roof heights which take into account mechanical penthouses. Figure 7 gives a discription of these varying heights in reference to grade elevation of 0'-0" (+880'). The framing of the roof is composed of wide flange framing with a 3" x 18 GA metal roof deck. The construction of the roof includes a modified bituminous roof system. This systems ranges in size from 3" to 12". This system is to undergo a flood test with 2" of ponding water for 24 hours to test for adaquacy.



Design Codes

In accordance with the specifications of structural drawing S0.01 the original design is to comply with the following codes:

- 2006 International Building Code with local amendments (IBC 2006)
- 2006 International Fire Code with local amendments (IFC 2006)
- Minimum Design Loads for Building and other structures (ASCE 7-05)
- Building Code Requirements for Structural Concrete (ACI 318)
- AISC Manual of Steel Construction LRFD 3rd Edition

These codes were also used in hand calculations and verifications in this Technical Report and those forthcoming.

Materials Used

The materials used for the construction of The USB are described in the following tables including relevant specifications:

Structural Steel			
Type	ASTM Standard	Grade	F_y (ksi)
Wide Flange	A992	50	50
Channels	A572	50	50
Rectangular and Round HSS	A500	B	46
Pipes	A53	E	35
Angles	A572	50	50
Plates	A572	50	50
Tees	A992	50	50

Concrete			
Location in the Structure	f'_c	Weight	Mix Class
Footings, Caissons, Grade Beams	4000	Normal	A
Slab On Grade	4500	Normal	C
Walls and Columns	4500	Normal	C
Beams and Slabs	4500	Normal	C
Slab on Metal Deck	4000	Normal	C
Equipment Pads and Curbs	4000	Normal	B
Lean Concrete	3000	Normal	E

- f'c is the concrete compressive strength at 28 days or at 7 days for high early strength concrete.
- Mix class as defined by project specifications

Aggregate	
Type	ASTM Standard
Normal Weight	C33
Light Weight	C330 and C157

Figure 8. Summary of Materials used on The USB Project with applicable specifications

Gravity Loads

Per the requirements of Technical Report 1, dead, live, and snow loads are to be calculated and verified with those provided on the structural drawings. Alongside these calculations and verifications spot check calculations of gravity members for adequacy are also provided. These calculations can be found in appendix A.

Dead and Live Loads

The structural drawings provide a schedule of superimposed dead and live loads for particular areas (Figure 9). Calculations of certain loads verify those provided in the table and in some cases are found to be conservative, which is typical practice for the structural engineer. This was perhaps a consideration due the complexity of the floor layout. Self-weights were also calculated to be applied in addition to the given dead and live loads

Provided Superimposed Dead Loads and Live Loads		
Locations	Superimposed Dead Load (psf)	Live Loads (psf)
Garage	35	50
Planetary Robotics	15	150
Loading Dock	5	250
Storage	35	125
Classroom	35	40
Halls, Assembly, Public Areas	35	80
Office, Meetings Rooms	35	50
Mechanical and Machine Room	75	100
Roof	35	30
Green Roof 1	35	30
Garage Roof	200	100
Green Roof 2	200	30
Mechanical Roof	35	50
Bridge 1	75	100
Roof Pavers	50	100
Roof River Rocks	55	30

Figure 9. Table of provided superimposed dead loads and live loads

Building Weight

The building weight was calculated considering superimposed dead loads, self-weights of columns, shear walls, braced frames, roofs, and exterior wall loads. This section is intended to provide weights for seismic calculations to generate total base shear. This value is then compared to the value provided on the drawings (See Seismic Section). Without the assistance of computer software to generate accurate weights, overall assumptions had to be made. First, from the provided schedules, pounds per square foot of reinforced concrete beams were tabulated considering weight of normal weight concrete (145 pcf) and supplemental reinforcement bars. Secondly, formed slab and metal deck slab pounds per square foot were calculated. Next linear takeoffs of steel beams were tabulated on floors 3-6 of building 1. This process reoccurred for floors 5-6 in building 2. Also counts of columns from the column schedule were made. A weight per lineal foot was noted per column. Next, the building enclosure is broken up into two groups; curtain walls and stud build out system. From assembly weight estimates it was assumed 15 psf for the curtain wall and 30 psf for the stud build out. Finally, the provided superimposed dead loads was summated and yielded a total pound per square foot for the floor. With all of the slabs, concrete beams, steel beams, columns, façade, and superimposed dead loads calculated to either a pound per square foot or linear foot, they are ready to be multiplied by its respective dimensions to result a total kilo pound per floor.

With a weight of kips per floor, it was then divided by that floors square footage resulting in a kip per square foot (ksf) for that floor. As stated before, level 3-6 in building 1 and levels 5-6 in building 2 were calculated with member accuracy. After investigation and grouping of these numbers per their typical floor layout, an average ksf was calculated to be applied to similar levels. This ksf was then applied to the remaining floors square footage once again resulting in kips per floor. The individual kips per floor were then summated to yield a total building weight. The following tables show numerical calculation. It is important to note that Technical Report 3 with provide a more detailed calculation of the building weight.

Building 1			
Level	~ Square Footage	Weight (K)	KSF
3	33,676	5,180.689	0.153839
4	20,983	2,644.86	0.126048
5	22,359	3,190.55	0.142697
6	27,633	3795.15	0.137342
7	21,018	2,592.60	0.123352
8	25,697	3,455.30	0.134463
9	21,970	2,954.15	0.134463
Total	**173,336**	**23,813.32**	**0.137382**

Building 2			
Level	~ Square Footage	Weight (K)	KSF
5	13413	1,654.52	0.1234 *
6	14,103	1,739.609	0.1234
7	13,438	1,657.604	0.1234
8	14,492	1,787.617	0.1234
Roof	14,915	1,839.795	0.1234
Total	**70,361**	**8,679**	**0.1234**

Figure 10. Table of floor approximate square footage, weights (K), and KSF.

* Note: Level 5 of Building 2 was calculated with member weight accuracy and its respective KSF was used as an average for the remaining floors.

From the structural loading diagrams, Live Loads were noted and compared to those provided in ASCE 7-05. Most of these values were verified by the code and others were found to be very conservative. A summary of these results can be found in Figure 11.

Live Loads			
Location	Design Live Load (psf)	ASCE 7-05 Live Load (psf)	Notes
Garage	50	40	May be from storage during construction
Planetary Robotics	150	N/A	N/A
Loading Dock	250	N/A	N/A
Storage	125	125	Anticipated light storage
Classroom	40	40	N/A
Halls, Assembly, Public Areas	80	80	N/A
Office, Meetings Rooms	50 (+20)	50 (+20)	+20 for Partition load
Mechanical and Machine Room	100	N/A	N/A
Roof	30	20	N/A
Green Roof 1	100	100	N/A
Garage Roof	30	30	N/A
Green Roof 2	50	60	Project green roof specifications may cause discrepancy
Mechanical Roof	100	N/A	N/A
Bridge	100	100	Serves as a corridor
Roof Pavers	100	100	N/A
Roof River Rocks	30	N/A	N/A

Figure 11. Comparison table of live loads from design documents and ASCE 7-05

Snow Loads

Snow loads were calculated in accordance with Chapter 7 of ASCE 7-05. This section highlights design criteria for The USB's location and design procedures. All design criteria and loads are summarized in Figure 12.

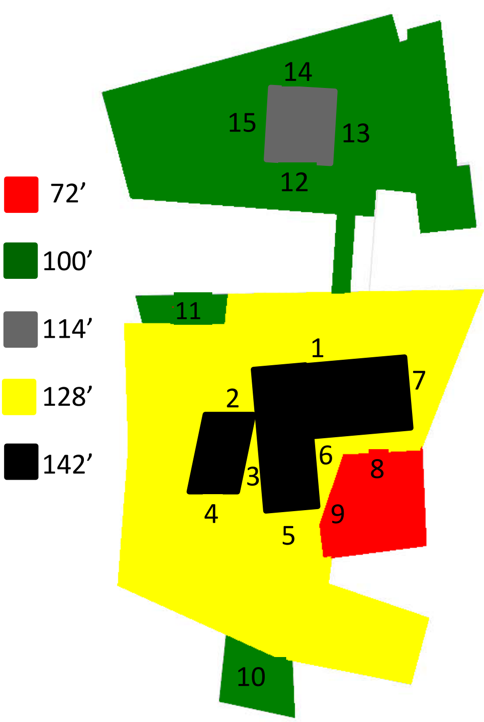
Flat Roof Snow Load Criteria			
Variable	Design Value	ASCE 7-05	Notes
Ground Snow Load, p_g (psf)	30	25	Fig -1 Conservative approach
Snow Exposure Factor, C_e	1.0	1.0	Table 7-2.
Snow Load Importance Factor, I_s	1.1	1.1	Table 7-4, Category III
Thermal Factor, C_t	1.0	1.0	Table 7-3, All other structures
Flat Roof Snow Load, p_f (psf)	27	23.1 (=$0.7C_eC_tIp_g$)	Eq 7-1, Conservative Approach
Snow Specific Gravity γ (pcf)	N/A	18	Eq 7-3
Base Snow Accumulation Heighg, h_b	N/A	1.3	N/A

Figure 12. Comparison table of snow load criteria from design documents and ASCE 7-05

The structural drawings provide design criterion that is accurate, but conservative in two locations. Figure 7-1 from ASCE 7-05 along with city building code clearly shows that the building location should be designed with a 25 psf ground snow load. This difference isn't necessarily bad as it is conservative. Likewise, the flat roof load calculation, with using a p_g of 30 psf, should yield 23.1 psf and not 27 psf. Once again this is a conservative approach but throughout this technical report and those forthcoming, a p_f of 23.1 psf will be used. Snow drift calculations were also performed for 15 potential locations on 5 different roof heights. Figure 13 shows snow drift calculations, along with Figure 14 and 15 providing a plan and elevation to assist drift calculations.

Snow Drift Calculations											
	General			Windward				Leeward			
Location	h_r	h_c	h_c/h_b	$L_{u\,(ft)}$	h_d (ft)	w_d (ft)	p_d (psf)	$L_{u\,(ft)}$	h_d (ft)	w_d (ft)	p_d (psf)
1	14	12.71	9.85	25	1.25	4.99	22.3	28.5	1.35	5.41	**24.2**
2	14	12.71	9.85	26.75	1.30	5.20	**23.3**	25	1.25	4.99	22.3
3	14	12.71	9.85		VOID				VOID		
4	14	12.71	9.85	68	2.19	8.74	**39.1**	25	1.25	4.99	22.3
5	14	12.71	9.85	25	1.25	4.99	22.3	39.5	1.64	6.55	**29.3**
6	14	12.71	9.85	25	1.25	4.99	**22.3**	25	1.25	4.99	**22.3**
7	14	12.71	9.85	25	1.25	4.99	22.3	54.75	1.95	7.82	**35.0**
8	56	54.71	42.39	35.25	1.53	6.14	27.5	41	1.67	6.69	**29.9**
9	56	54.71	42.39	37	1.58	6.31	28.2	70	2.22	8.87	**39.7**
10	28	26.71	20.70	25	1.25	4.99	22.3	35.25	1.53	6.14	**27.5**
11	28	26.71	20.70	25	1.25	4.99	22.3	99.5	2.63	10.53	**47.1**
12	14	12.71	9.85	25	1.25	4.99	**22.3**	25	1.25	4.99	**22.3**
13	14	12.71	9.85	43.75	1.73	6.93	**31.0**	25	1.25	4.99	22.3
14	14	12.71	9.85	25	1.25	4.99	**22.3**	25	1.25	4.99	**22.3**
15	14	12.71	9.85	58.5	2.02	8.09	**36.2**	25	1.25	4.99	22.3

Figure 13. Table of Snow Drift Calculations. Note: Snow Drift Loads are in addition to flat roof snow load. Total Snow @ max drift location = 23.1 psf + 47.1 psf = 70.2 psf



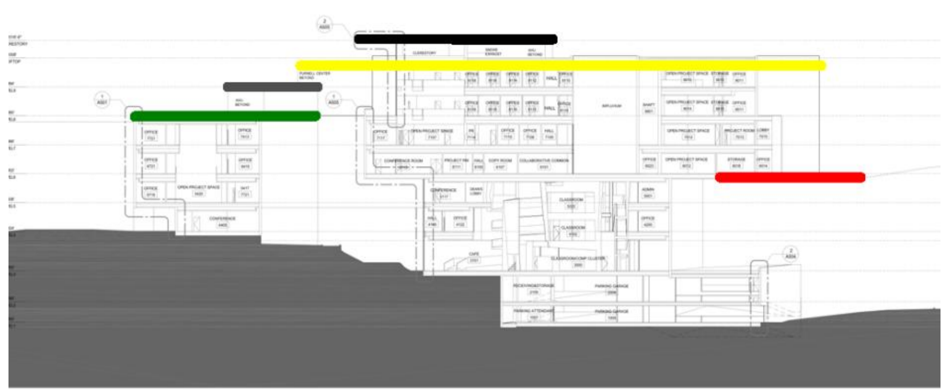


Figure 15. Elevation looking NE detailing roof elevations

Figure 14. Plan of varying roof elevations with potential drift locations

Floor Systems

Introduction

Technical Report 2 is intended to provide the design of alternative floor systems for consideration in an upcoming redesign proposal. Currently the floor system of The USB includes one way slabs with beams, along with composite and non-composite systems, although the majority being composite. The alternative systems under consideration will be investigated as one system throughout the building which will inherently adjust other systems in the building. These adjustments will be briefly analyzed in this section. The following are descriptions of the existing floor system and alternative systems.

Composite Slab and Beam

In the interest of performing calculations of the existing composite system in the complexity of the floor plans, a beam was chosen in a typical 27'x30' bay (refer to Appendix A). A pin pin connected W14x22 composite beam with a 2VLI18 Vulcraft deck (2" deck with 4.5" topping) was analyzed (Figure 16). Self-weights, superimposed dead loads, and live loads were used as the applicable loading on this particular beam with a tributary width of 7' 3/8".

Calculations were performed to check deck spans, unshored construction, flexure under construction load, composite design under full gravity load, shear stud allowance, live load deflection, and construction load construction, along with all necessary deflections (See Appendix A) . All of these checks proved the initial design to be adequate. Checking for composite action under full gravity load showed that the beam is more than appropriate for strength. A discrepancy in design moments way have resulted from constructability concerns. The required strength under construction loads controls the design of using a composite beam.

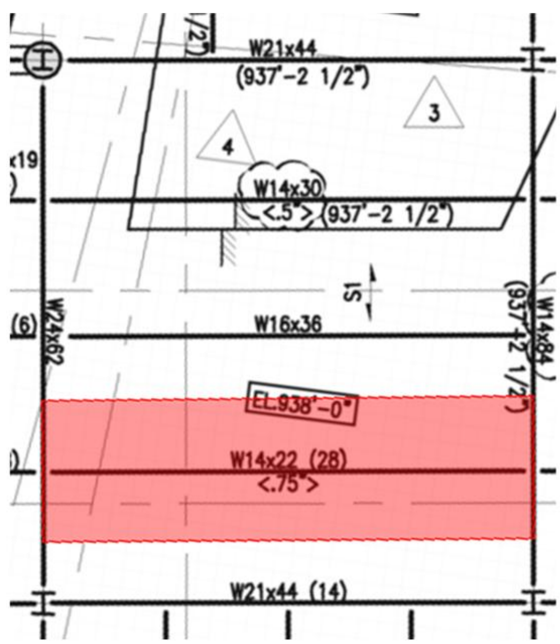


Figure 16. W14x22 Composite Beam and its tributary width

General

This existing composite slab and beam system is a point of reference in comparing the alternative floor systems. Since this system starts on level 4 and is intended to be on every level, the design of formed or masonry foundation walls will need to be considered. This particular system weighs approximately 79 psf and costs about $20.30 per square foot to construct. The weight of the system was found by calculating the total weight (lbs) of elements in a 27'x30' bay and then divided by the bays square footage (810 SF). The cost was calculated using the 2011 RS Means Assemblies Costs. The total depth of this system is 30.2" (not including floor finishes). This depth includes a 6.5" slab and a 23.7" deep W24x68 wide flange. These details are favorable when compared to other alternatives. MEP equipment is easily able to run or be hung from this system with minimal penetrations. This system is a feasible one to be further considered.

Architectural

This system is designed with a 2 hour fire rating. In most areas the steel beams and underside of the deck are coated with a spray-on fire proofing. The system allows for MEP runs to fit inside the plenum space contained by an acoustical drop ceiling. Since this is the existing system no further architectural impacts are to be considered.

Structural

The foundation consists of drilled caissons, column footings, and grade beams. The system is integral with the lateral systems; steel braced frames and shear walls. If chosen to remain, this lateral system will not change.

Serviceability

The maximum deflections are calculated from the girder with all applied loads including 3 infill beams that frame into the girder. This system's maximum deflection was found to be 1.19" and acceptable by IBC code (L/240). Although not considered in this report, vibrations may have a considerable effect on the serviceability of this system.

Construction

The construction of this system is given a 2 on a 1-5 scale (1 having the easiest constructability and 5 having the toughest constructability). This rating system, generated to provide a simple understanding of the system, is designed to rate the difficulty of constructing the system. For the purpose of this technical report, only this representative bay was considered in the design and analysis. Although this construction is rather routine for the daily trades, the complexity of the floor layout in other bays may take extra coordination to construct. Since this is the existing system no other constructability issues are considered.

Composite Slab and Beam System Pro-Con Analysis

Pros:

- Lightweight
- Easy connections for MEP equipment
- Rather easy to construct

Cons:

- Intermediate to long construction schedule
- Additional fire proofing needed
- Relatively large deflections at current span lengths.

If Considered. . .

If this system is further considered, the possibility of altering the bay sizes and complexity are of interest. Also, foundation walls will need to be designed for this system in the levels below grade.

One Way Slab with Beams

The investigation in this section is the use of a one way slab with beams throughout the whole building. The first two floors of Building 1 utilize a one way slab with beams. The design of this system was performed by hand calculations and spSLAB (figure 17) for verification which can be found in Appendix B.

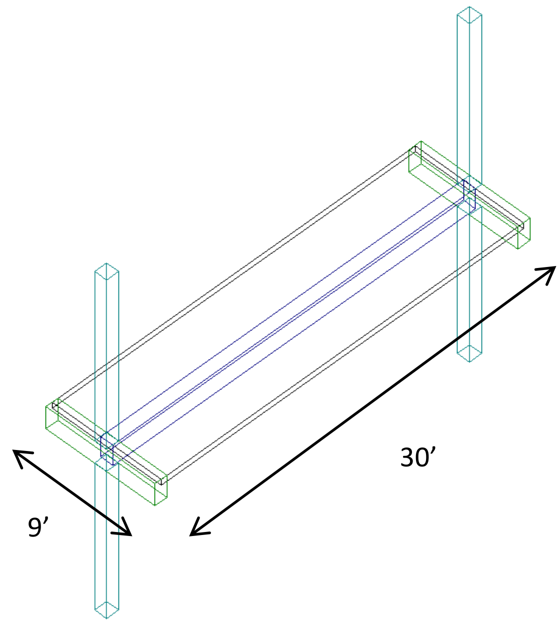


Figure 17: Frame analyzed in spSLAB

This system uses normal weight concrete with a specified compressive strength of 4000 psi and ASTM A615 Grade 50 reinforcement bars. The system is formed into its respective shape with timber formwork. For simplicity, calculations of preliminary slab and beam thicknesses were found with the assumption that deflections control, allowing the use of Table 9.5a from ACI 318-08. The bay of interest has dimensions of 27' x 30' (aspect ratio = 1.11) and considered with a minimum of 3 similar bays in each direction. The beam design resulted in three equal spans of 9' spanning the 30' direction with respective positive and negative moment reinforcement.

General

This system has many different characteristics in comparison to a composite slab and beam system. Its overall floor depth (22") is 8" less than the composite depth. Where this extra space can be utilized, it weighs approximately 102 lbs/SF, nearly 20 lbs/SF more than composite. This 2 hour fired rated system would cost approximately $17.90/SF to construct. This system will definitely have an impact on the design of the rest of the building's systems.

Architectural

This system provides a 2 hour rating from the depth of the concrete and the clear cover on reinforcement bars, not requiring any additional fire proofing. This system's depth of 22" can either reduce the overall height of the building or give more space between floors and/or the plenum space. There is a possibility of altercations to the façade do the complexity of the floor plans. Slab cantilevers may arise, presenting further investigation with the façade and floor system. Furthermore, concrete columns or encased steel columns will need to be implemented throughout the height of the building.

Structural

A one way slab with beams will alter the design of the structural system, primarily the foundations and lateral system. The use of concrete as a floor system will increase the system weight by nearly 29% consequently increasing the building weight causing a reconsideration of foundation design. Larger caissons and column footings will be of interest with this increase of building weight. Also, a change in the lateral system will need to be considered. First, due to the increase in building weight, seismic loading will increase resulting in the need of a stiffer and stronger lateral system. Secondly, the lateral system will need to connect to the floor system. Since the floor system is concrete, it would be most efficient to employ a shear wall lateral system throughout the building.

Serviceability

The design of this system was assumed to control by deflections, which allowed for the minimum thickness of the slab and beams to be calculated by ACI 318-08 Table 9.5a. Per the calculations found in Appendix B, it can be determined that the maximum deflection for the system is 0.93", performing second best among the considered systems. Also, due to the mass of this system it performs well under vibration, although corresponding calculations are not provided in this technical report which would also increase the weight of the building.

Construction

The construction of this system is given a 3.5 on a 1-5 scale (1 having the easiest constructability and 5 have the toughest constructability). The reasons for this rating are predominantly due to the amount of formwork needed. Every floor will need different formwork due to the variation in openings and floor profile. On the other hand, the construction of this system can be performed with two or less trades (carpenters for formwork and concrete trade for rebar and concrete). The complexity of floor plans will present challenges for the trades, primarily formwork. This will inherently increase the project schedule.

One Way Slab with Beams Pros Cons Analysis

Pros:

- Low cost per square foot
- Smaller system thickness
- Relatively low deflections

Cons:

- Heavy
    - Need for new foundations
    - Changes seismic loading
- Relatively more difficult to construct
- Will increase construction schedule

If Considered. . .

Investigation of a longer aspect ratio and a different bay layout are possibilities of making the system more efficient.

Two Way Flat Plate

A two way flat plate system is the third floor system to be considered in this technical report (Figure 18). Since the existing lower floors are concrete, there is no need for altercations in foundation walls. The design of this system was performed by hand calculations and spSLAB for verification which can be found in Appendix C.

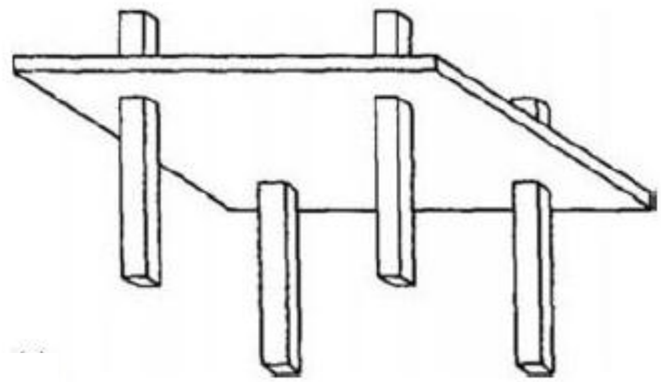


Figure 18: Image of a flat plate slab

This system uses normal weight concrete with a specified compressive strength of 4000 psi and ASTM A615 Grade 50 reinforcement bars. The system is formed into its respective shape with timber frame work, which is considerably easier compared to the one way system because it does not have beams to be formed. For simplicity, calculations of preliminary slab thicknesses were found with the assumption that deflections control, allowing the use of Table 9.5a from ACI 318-08. The bay of interest has dimensions of 27' x 30' with a minimum of 3 similar bays in each direction. The important design consideration for this system was punching shear, which was found to adequate per the design found in Appendix C.

General

This system has similar characteristics to the one way slab with beams. Its overall floor depth of 12" is the thinnest of all floor systems. Even though the system is thinner, it weights nearly 150 lbs/SF; by far the heaviest of the systems considered. This 2 hour fired rated system would cost approximately $16.35/SF to construct, the lowest of the four systems. This system will certainly have an impact on the design of the rest of the building's systems.

Architectural

This system provides a 2 hour rating from the depth of the concrete and the clear cover on reinforcement bars, not requiring any additional fire proofing. This system's depth of 12" can either reduce the overall height of the building or give more space between floors and/or the plenum space.

There is the possibility of altercations to the façade do the complexity of the floor plans. Since this system does not have beams, any odd dimensions and angles may employ cantilevers. Cantilevers where the façade is connected will either require more system strength or less weight for the façade. Furthermore, concrete columns or encased steel columns will need to be implemented throughout the height of the building.

Structural

A two way flat plate slab will alter the design of the structural system, primarily the foundations and lateral system. The use of concrete as a floor system will increase the system weight from 79 lbs/SF to 150lbs/SF, a 90% increase. Consequently, the increase to the building weight will induce reconsideration of the foundation design. Larger caissons and column footings will be of interest with an increase of building weight, possibly even a whole new foundation system. Also, a change in the lateral system will need to be considered. Since there will be an increase in building weight, seismic loading will increase, resulting in the need for a stiffer and stronger lateral system. Secondly, the lateral system will need to be connected to the floor system properly. Since the floor system is concrete, it will be most efficient to employ a shear wall lateral system throughout the building, also increasing the building weight.

Serviceability

The design of this system was assumed to control by deflections, which allowed for the minimum thickness of the slab to be calculated with ACI 318-08 Table 9.5a. Per the calculations found in Appendix C, it can be determined that the maximum deflection for the system is 0.89”, performing best among the considered systems. Also, due to the mass of this system it performs well under vibration, although corresponding calculations are not provided in this technical report.

Construction

The construction of this system is given a 2.5 on a 1-5 scale (1 having the easiest constructability and 5 have the toughest constructability). The reasons for this rating are due to the formwork and labor needed to construct. Every floor will need different formwork due to the variation in openings and floor outline. On the other hand, the construction of this system can be performed with two or less trades (carpenters for formwork and concrete trade for rebar and concrete). The complexity of floor plans will present challenges for the trades, primarily formwork. But without beam formwork due to being a flat slab, it will most likely not increase the project schedule.

Two Way Flat Plate Pros Cons Analysis

Pros:

- Low cost per square foot
- Smaller system thickness
- Low deflections

Cons:

- Heavy
  - Need for new foundations
  - Changes seismic loading
- Relatively more difficult to construct
  - Around edges and openings

If Considered. . .

Investigation into a thinner slab, the use of drop panels, and alternative bay sizes are all considerations to allow for a more efficient floor system.

Precast Hollow Core Planks on Steel Framing

In this section, precast hollow core planks on steel framing will be considered as an alternative floor system. Since the existing system has floors that are under grade, the system will need to incorporate foundation walls. If this system is chosen to be further evaluated, then such walls will be incorporated in the lateral system.

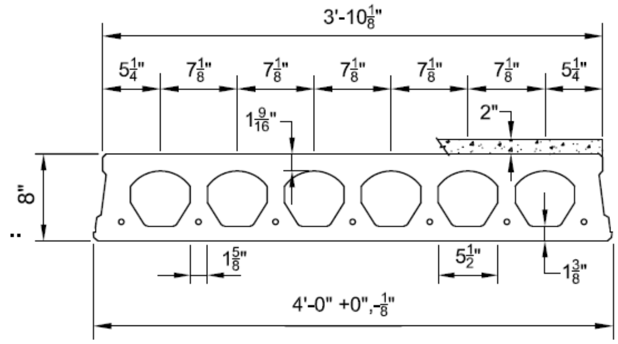


Figure 19: Cross section of Nitterhouse 8" hollow core

This system will utilize specified Nitterhouse precast hollow core planks (refer to figure 19 and Appendix D). For simplicity, calculations were performed under the assumption of simply supported system over a length of 27'. Planks with 7 1/2" reinforcing strands, a depth of 8", and a width of 4' will be spaced evenly over a 30' span, allowing for a uniform layout. Calculations and specifications can be found in Appendix D.

General

This system has similar characteristics to the existing system. The overall depth of the system is 29.4" (8" Slab and 21.4" Girder) and is almost 8 lbs/SF lighter than the composite system. In order to maintain a fire rating of 2 hours, the steel framing will need to be coated with a spray on fire proofing. This system cost is approximately $26.83/SF, the highest of all systems considered. Reasons for this cost can be most attributed to topics covered in the construction section. This system will have minimal impact on the design of the rest of the building's existing systems.

Architectural

This system performs well in typical bays of the building but will induce concern in areas of irregularity. Where the system has changing floor outlines and openings, specialized planks are needed to meet the architectural design. Also, the construction of sprayed fire rated steel beams will require a drop ceiling in most areas, similar to the existing system, which is feasible to fit within the plenum space. There is the possibility for altercations to the façade do the complexity of the floor plans and strength of the floor system in those areas.

Structural

The use of a precast hollow core planks on steel framing will most likely have no impact the design of the foundations or the lateral system. With a slight decrease in system weight, alternative foundations may be investigated but probably will not change. Also the lateral system will not change except for foundation walls needed to surround the system on the lower floors.

Serviceability

This system performed worst in deflections, reaching maximum deflection of 1.22”. Per deflection control from IBC 2006, the system is under the allowable deflection. Also, due to the light weight of this system it will most likely perform poorly under vibration, although corresponding calculations are not provided in this technical report.

Construction

The construction of this system is given a 3.5 on a 1-5 scale (1 having the easiest constructability and 5 have the toughest constructability). The considerations for this rating are the labor needed to construct, the individual picking of planks by a crane, and special cutting of planks in irregular locations. The construction of this system will potentially require two or more trades to construct (iron workers for steel erection and precast company trade for hollow core planks). The complexity of floor plans will present challenges for the trades but will most likely run a similar or possibly longer schedule to the existing composite system. Finally, due to the fabrication of the planks, MEP penetrations are very difficult because of the voids and reinforcing strands.

Precast Hollow Core Planks on Steel Framing Pros Cons Analysis

Pros:

- Less weight
- Potential for smallar foundations

Cons:

- Very high construction cost
- Relatively more difficult to construct
    - Around edges and openings
- Will increase construction schedule
- Difficult to make MEP penetrations

If Considered. . .

An investigation into shorter spans for the planks, column layout and alternative bay sizes, and a combination of formed slabs and planks in irregular areas are all considerations to allow for a more efficient floor system.

Summary of Systems

Considerations		System: Composite Slab and Beam	One way Slab With Beams	Two Way Flat Plate	Precast Hollow Core on Steel Framing
General	Weight (psf)	79	102	150	71
General	Cost ($/SF)*	$20.30	$17.90	$16.35	$26.83
General	Floor Depth	30.2" (6.5" Slab) (23.7" Girder)	22" (6.5" Slab) (19.5" Girder)	12" (12" Slab)	29.4" (8" Slab) (21.4" Girder)
Architectural	Fire Rating	2 Hour	2 Hour	2 Hour	2 Hour
Architectural	Additional Impacts	N/A	Considerable more floor height. Does not require additional F.P.	Considerable more floor height. Does not require additional F.P	Steel framing needs additional F.P.
Structural	Foundation Impact	Existing Caissons, column footings, and grade beams	May increase caisson and column footing sizes	Will Increase all foundation sizes	Will have minimal impact on foundations
Structural	Lateral System Impact	Existing Braced frames and shear walls	Shear walls would need to be implemented	Shear walls would need to be implemented	No Impact
Serviceability	Max. Deflection	1.19"	0.93"	0.89"	1.22"
Serviceability	Vibration	Average	Very Good	Very Good	Poor
Construction	Additional Fire Protection Required?	Spray-on Beams	None	None	Spray-on Beams
Construction	Schedule Impact	N/A	Will increase schedule	May possibly increase schedule	Will most likely not affect the schedule
Construction	Constructability**	2	3.5	2.5	3.5
Further Consider?		**Yes**	**No**	**Yes**	**No**

Note: * All of the estimated costs are in accordance with RS Means 2011 Assembly Costs and were interpolated to achieve a more accurate value for this projects conditions
** Constructability was rated on a scale from 1-5; 1 being the easiest to construct and 5 being the hardest to construct.

Figure 20: A chart summarizing the 4 floor systems in relation to relevant considerations

Conclusion

Technical Repot 2 was an accumulation of investigations on alternative floor systems to be considered for The University Sciences Building. These floor systems include the existing composite slab and beam, one way concrete slab with beams, two way concrete flat plate, and hollow core plank on steel framing. All of these systems were analyzed and designed per a typical 27'x 30' bay with consideration of these systems in areas of irregularity.

The existing composite slab and beam system remains as a feasible system as it is relatively both cheap and lightweight compared to the others. The systems depth of 30.2" is feasible with respect to the floor to floor height, as most areas are covered with drop ceilings.

The one way slab with beams is the least feasible of the alternative systems. The system works well for continual typical bays and preferably long aspect ratios. Since it is hard to continually achieve these conditions within The USB's architecture, it does not prove to be the very efficient. Its heavy weight (29% increase), potential increase to the project schedule, and construction difficulty concludes to eliminating of the system from further consideration.

The two way flat plate is worth further consideration after possible design altercations. The main concern for this system is its very heavy weight (90% increase), which will change the design of the foundations and lateral system, adversely affecting the seismic loads. It is of interest to implement drop panels and to change the bay sizes to minimize the slab thickness and overall weight.

The hollow core plank on steel framing system has proven to be ineffective as an alternative floor system. With a 32% increase in cost per square foot and the difficulty of construction due to the floor layout with respect to the desired architecture, the system has been eliminated from consideration.

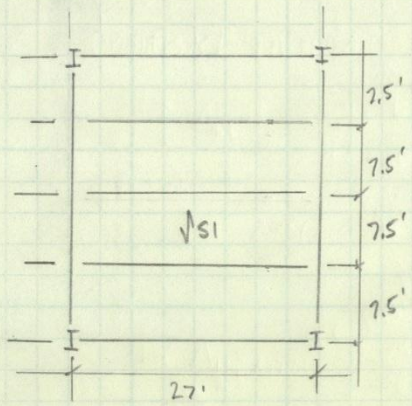
With the one way slab with beams and hollow core planks on steel framing eliminated from further consideration, possible variations to the composite slab and beam system and the two way flat plate system are worth further evaluation. This will induce further investigation into the foundations, lateral system and possibly other systems within The University Sciences Building.

Appendix

Appendix A: Composite Slab and Beam System

Slab on Metal Deck

1 OF 1 | CHRIS DUNLAY | TECHNICAL REPORT #1 | SPOT CHECK - SLAB



SLAB ON METAL DECK REQUIREMENTS

↳ 20 GAUGE MINIMUM

↳ VULCRAFT

↳ SPAN NO GREATER THAN 9'-8"

↳ 3-SPANS WHERE POSSIBLE

↳ S1 SPECIFIES A 2"x18 6½" (4½ TOPPING) NORMAL WEIGHT CONCRETE DECK

↳ USE 2VLI18

↳ 2HR UNPROTECTED

• TOTAL UNFACTORED LOAD

↳ DEAD = 35 PSF

↳ LIVE = 40 PSF

} TOTAL = 75 PSF

VULCRAFT 2008 MANUAL

• SDI MAX UNSHORED CLEAR SPAN

↳ 3-SPAN FOR 2VLI18

= 10'6" > 7'6" OK! ✓

• SUPERIMPOSE LOADS CLEAR SPAN

↳ 2VLI18 @ 7'6" CLEAR SPAN.

= 400 PSF

400 PSF > 75 PSF OK! ✓

• FIRE RATING

↳ 2HR UNPROTECTED DECK w/ 4½" TOPPING.

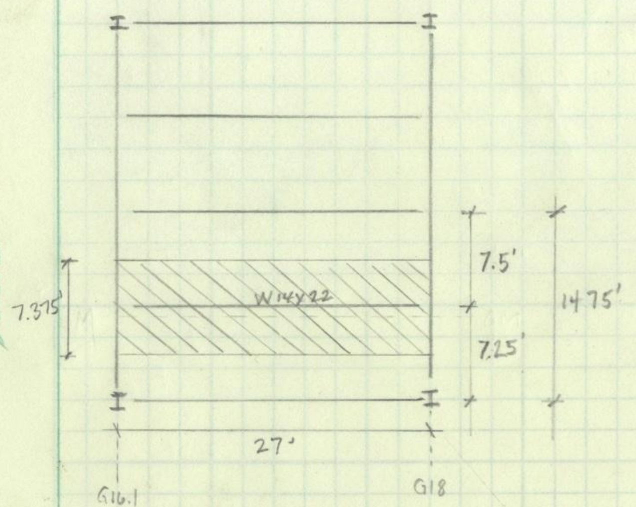
2VLI IS AVAILABLE OK! ✓

Composite Beam

1 OF 4 | CHRIS DUNLAY | TECHNICAL REPORT #1 | SPOT CHECKS–BEAM

SPOT CHECK #1: COMPOSITE BEAM

↳ LOCATION: BUILDING 1, LEVEL 5, BW G16.1 & G18 ON GM.



LOADING:

↳ w_D – S.I. DL = 35 PSF

– S.W. = 69 PSF
↳ 2VLI18 VULCRAFT COMP. DECK

\+ ———
104 PSF

↳ w_L = 40 PSF

LIVE LOAD REDUCTION:

↳ $A_T = 27' \times 7.375' = 199$ ft²

↳ $K_{LL} = 2$ (INT. BEAM)

↳ $K_{LL}A_T = 398$ ft² < 400 ft²

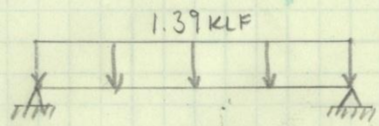
↳ NOT NEEDED!

↳ $w_u = 1.2D + 1.6L'$ → CONTROLING LOAD COMBO

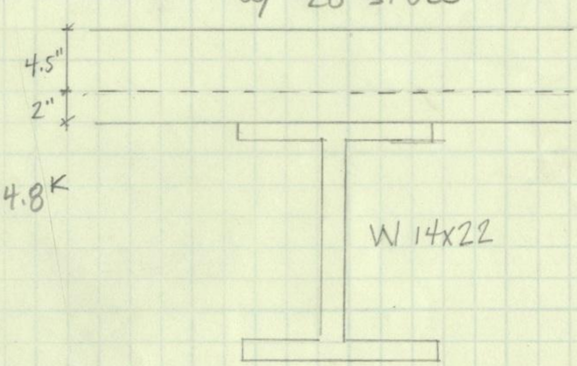
$= 1.2(104) + 1.6(40) = 188.8$ psf → CLEAR SPAN of 9.5' PERMISSES

$= \frac{(188.8)(7.375)}{1000} = 1.39$ KLF

243 psf > 188.8 psf
OK! ✓



CONFIRM SELECTION OF W14x22 w/ 28 STUDS



$V_u = w_u\left(\frac{l}{2}\right) = (1.39)\left(\frac{27}{2}\right)$

$\boxed{= 18.8^K} < \phi V_n = 94.8^K$

$M_u = \frac{w_u l^2}{8} = \frac{(1.39)(27)^2}{8}$

$\boxed{= 126.7 \text{ ft-K}}$

2 OF 4 | CHRIS DUNLAY | TECHNICAL REPORT #1 | SPOT CHECKS-BEAM

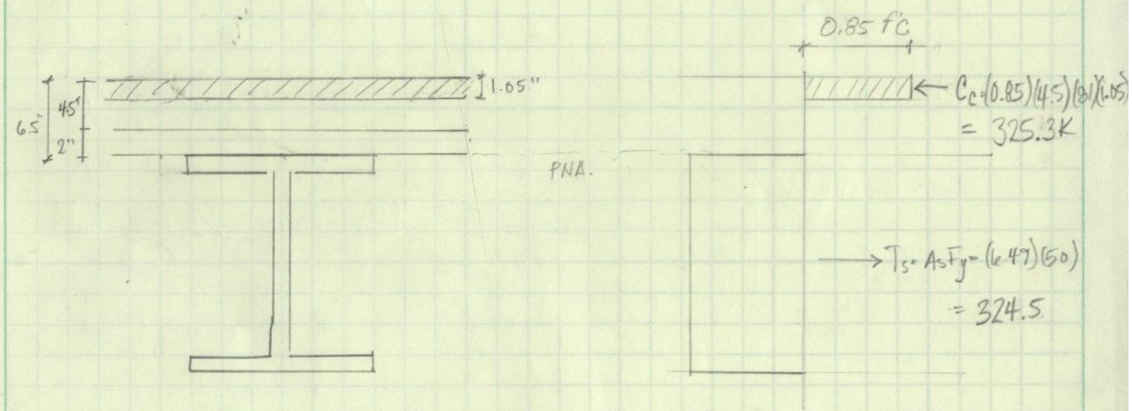
↳ $b_{eff} = \min \begin{cases} \frac{\text{SPAN}}{4} = \frac{(27)(12)}{4} = 81'' \longrightarrow \text{CONTROLS} \\ \frac{\text{BM SPACING}}{2} = \frac{(14.75)(12)}{2} = 88.5'' \end{cases}$

↳ REQUIRED STRENGTH UNDER UNSHORED CONSTRUCT. (D+L) (NON-COMP)

- $w_u = 1.2[(69\text{ psf} \times 7.5') + 22\text{ PLF}] + 1.6[(20\text{ psf} \times 7.5')] = 0.89\text{ PLF}$
- $M_u = \frac{(0.89)(27)^2}{8} = 81\text{ ft-K}$
- $\phi M_b = 125'\text{K} > 81\text{ ft-K}$ → DOES NOT NEED SHORING! ✓

↳ CONFIRM COMPOSITE DESIGN @ FULL GRAVITY LOAD

- $C_c = 0.85 f'_c b_{eff} t = (0.85)(4.5)(81)(4.5) = 1394^K$
- $T_s = A_s F_y = (6.49)(50) = 324.5^K$ → CONTROLS ΣQ_N
  PNA IS IN CONCRETE
- $a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{325}{(0.85)(4.5)(81)} = 1.05''$
- $Y_2 = 6.5'' - \frac{1.05}{2} = 5.98'' \approx 6.0''$ →
- $A_{s\text{-}c} = \frac{324.5 - 325.3}{2(50)} = 0$ → PNA IS TFL
- $\phi M_n = 313\text{ ft-K} > 126.7\text{ ft-K}$



3 OF 4 | CHRIS DUNLAY | TECHNICAL REPORT #1 | SPOT CHECKS-BEAM

↳ CONFIRM STUDS

- (28) $3/4''\phi$ SHEARS ACROSS 27' (1 PER RIB)
- $Q_N = 0.5A_{sc}\sqrt{f'_c E_c} \leq R_g R_p A_{sc} F_u$; $A_{sc} = \frac{\pi(3/4)^2}{4} = 0.442 \text{ in}^2$

  $0.5(0.442)\sqrt{4.5(3704)} \leq (1.0)(1.0)(0.44)(65)$ $E_c = 145^{1.5}\sqrt{4.5} = 3704$ KSI

  $= 28.53 \leq 28.73$ ✓ OK!
- # OF STUDS = $\frac{\Sigma Q_N}{Q_N} \times 2 = \frac{324.5}{28.53} = 11.3 \rightarrow 12 \times 2 = 24$ STUDS

  ↳ DESIGN REQUIRES 24 STUDS ALONG ITS LENGTH BUT STRUCTURAL DWGS CALL FOR 28. THIS AMOUNT WAS CHOOSEN SO A STUD CAN BE PLACED IN ONE RIB (12") OVER ITS 27' SPAN.

  OK! ✓

↳ CHECK LIVE LOAD DEFLECTIONS

- $\Delta_{LL} = \frac{5 w_{LL} L^4}{384 E_c I_{LB}}$; $I_{LB} = 737 \text{ in}^4 \rightarrow$ TABLE 3-20 AISC

  $E_c = 29,000$ KSI

  $w_{LL} = \frac{(40)(7.375)}{1000} = 0.295$ K/ft

  $= \frac{5(0.295)(27)^4}{384(29000)(737)}(1728)$

  $= 0.165$ in
- $\Delta_{LL, \text{ALLOWABLE FLOORS}} = \frac{L}{360} = \frac{(27)(12)}{360} = 0.9$ in

  $0.165 \text{ in} < 0.9 \text{ in}$ OK! ✓

↳ CHECK CONSTRUCTION LOAD DEFLECTIONS

- $w_D = 69$ PSF $\times$ (7.375') = 508.9 PLF SLAB WEIGHT

  + 22 PLF → BEAM WEIGHT

  530.9 PLF
- $w_{LL} = 20$ PSF $\times$ (7.375') = 147.5 PLF

4 of 4 | CHRIS DUNLAY | TECHNICAL REPORT #1 | SPOT CHECKS - BEAMS

- $w_u = 1.2(0.531) + 1.6(0.147) = 0.872$ plf
- $M_u = \dfrac{w_u l^2}{8} = \dfrac{(0.872)(27)^2}{8} = 79.5$ ft-k

  ↳ $\phi_b M_p = 125$ ft-k > 79.5 ft-k <u>OK!</u> ✓

- $\Delta_{DL} = \dfrac{5 w_D L^4}{384 E I}$ $E = 29000$ ksi, $I = 199$ in4 → FROM TABLE 1-1 AISC

  $= \dfrac{5(0.531)(27)^4}{384(29000)(199)}(1728) = 1.1'' - \overset{\text{CAMBER}}{0.75''} = 0.35''$

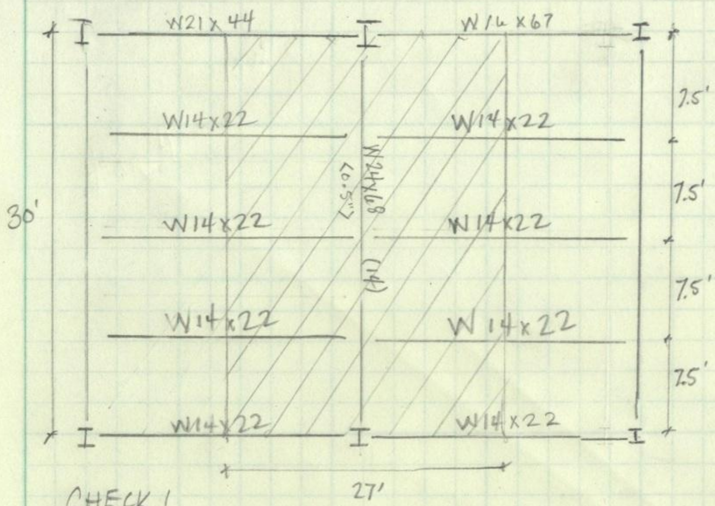
- $\Delta_{DL,\,ALLOW} = L/180 = \dfrac{(27)(12)}{180} = 1.8''$

  ↳ $1.8'' > 1.1'' > 0.35''$ <u>OK!</u> ✓

<u>CONCLUSION:</u> THESE CALCULATIONS CONFIRM THAT A W 14x22 COMPOSITE BEAM WITH 28 STUDS ACROSS ITS 27' SPAN IS ADEQUTE.

Girder

1 OF 4 | CHRIS DUNLAY | TECHNICAL REPORT #1 | SPOT CHECK - GIRDER



CHECK 1

- DEAD LOAD ON W14X22 BEAMS
  - ↳ SUPERIMPOSED = 35 PSF × (27') × (7.5') = 7.09^K
  - ↳ BEAM S.W. = 22 PLF × (27') = 0.59^K
  - ↳ SLAB = 69 PSF × (27') × (7.5') = 13.97^K
- LIVE LOAD
  - ↳ $K_{LL} = 2$ (INTERIOR BEAMS
  - ↳ $A_T = (27)(7.5) = 810$ □'

  } $K_{LL}A_T = 405$ □'

  - ↳ $L = 40_{psf}\left[0.25 \;\; \frac{15}{\sqrt{405}}\right] = 39.8$ PSF × (27') × (7.5') = 8.06^K
- $P_u = 1.2(7.09^K + 0.59^K + 13.97^K) + 1.6(8.06^K) = \boxed{38.9^K}$

3 OF 4 | CHRIS DUNLAY | TECHNICAL REPORT #1 | SPOT CHECK – GIRDER

• SOLVE FOR a

$$a = \frac{\Sigma Q_N}{0.85 f'_c b_{eff}} \quad , \quad b_{eff} = \min \begin{cases} \frac{(30)(12)}{4} = 90'' \\ \frac{7.5' + 7.5'}{2}(12) = 90'' \rightarrow \text{USE} \end{cases}$$

$$= \frac{199.7}{0.85(4.5)(90)} = 0.58''$$

• $Y_2 = 6.5'' - \frac{0.58''}{2} = 6.21''$

TOTAL SLAB DEPTH

• $M_N = 199.7\left(\frac{23.7}{2}\right) + 199.7\left(6.5 - \frac{0.58}{2}\right)$

$= 3606.\text{ IN-KIPS} = 300.5 \text{ ft-K}$

$\phi M_N = 0.9(300.5 \text{ ft-K}) = 270 \text{ ft-K}$

$\boxed{270 \text{ ft-K} << 664 \text{ ft-K}}$ OK! ✓

CHECK 2

• UNSHORED CONSTRUCTION

↳ $M_u = 587.5$ ft-K (FROM PAGE 2)

↳ $\phi M_p = 664$ ft-K (TABLE 3-19)

587.5 ft-K < 664 ft-K OK! ✓

• UNDER FULL GRAVITY LOAD

↳ $C_c = 0.85 f'_c b_{eff} t = 0.85(4.5)(90)(4.5) = 1550$ K

↳ $T_s = A_s F_y = (20.1)(50) = 1005$ K → CONTROLS ΣQ_N

• PNA IS CONC.

• TFL

↳ $a = \frac{\Sigma Q_N}{0.85 f'_c b_{eff}} = \frac{1005}{0.85(4.5)(90)} = 2.92''$

↳ $Y_2 = 6.5 - \frac{2.92}{2} = 5.04 \approx 5.00$

↳ WITH $Y_2 = 5.00$ & TFL → $\phi M_n = 1270$ ft-K >> 587 ft-K

OK! ✓

4 OF 4 | CHRIS DUNLAY | TECHNICAL REPORT #1 | SPOT CHECK - GIRDER

- CHECK LIVE LOAD DEFLECTIONS

↳ $\Delta_{LL} = \dfrac{5 w_{LL} L^4}{384 E I_{LB}}$; $I_{LB} = 4680 \text{ in}^4$ (TABLE 3-20 AISC)

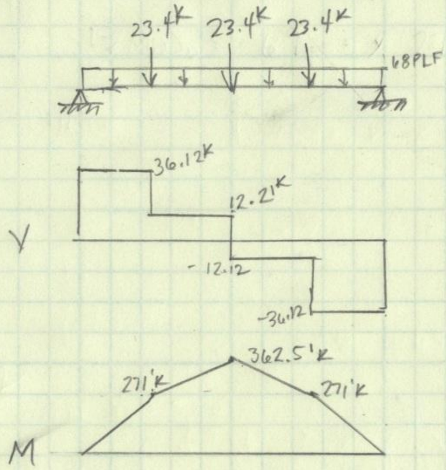
$w_{LL} = \dfrac{(40)(27')}{1000} = 1.08$ KLF

$= \dfrac{5(1.08)(30)^4}{384(29000)(4680)}(1728) = 0.145''$

↳ $\Delta_{LL,ALLOW} = \dfrac{l}{360} = \dfrac{(30)(12)}{360} = 1.00''$

$\underline{0.145'' < 1.00''}$ OK! ✓

- CHECK CONSTRUCTION LOAD DEFLECTIONS



$P_D = 69 \text{ PSF} \times 7.5' \times 27' = 13.97^K$

$w_D = 68$ PLF

$P_L = 20 \text{ PSF} \times 7.5' \times 27' = 4.05^K$

$P_u = 1.2(13.97) + 1.6(4.05) = 23.24^K$

$M_u = 362.5$ ft-K

$\phi M_p = 664'K$

$\underline{664 \text{ ftK} > 362.5 \text{ ft-K}}$ OK! ✓

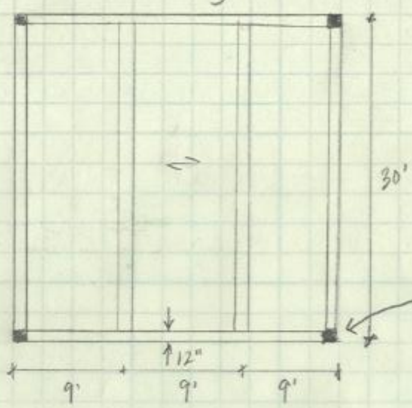
↳ $W_{DL} = (0.0198 \text{ PLF}) + (69\text{psf})(27') + 0.068 \text{ PLF} = 1.86$ PLF
→ W14X22 BEAMS

↳ $\Delta_{DL} = \dfrac{5 w_{DL} L^4}{384 EI}$; $E = 29000$ KSI ; $I = 1830 \text{ in}^4$

$= \dfrac{5(1.86)(30)^4}{384(29000)(1680)}(1728) = 0.63''$

↳ $\Delta_{DL,ALLOW} = \dfrac{l}{180} = \dfrac{(30)(12)}{180} = 2'' \rightarrow 2'' > 0.63''$ OK! ✓

Appendix B: One Way Slab with Beams



$f'_c = 4000$ psi

$f_y = 60000$ psi

$l_n = 30 - 2\left(\frac{12}{2}\right) = 29'$

- Assume deflections control design
- Assume 12" x 12" colums

↳ Minimum Slab thickness (Table 9.5a)

- $t = \dfrac{9'(12)}{24} = 4.5" \rightarrow$ use 6"

↳ Minimum Beam thickness (Table 9.5a)

- $h = \dfrac{30(12)}{18.5} = 19.5"$

↳ Loads & Moments

- Superimposed Dead = 35 psf
- Slab S.W. = $\frac{6}{12}(150) = 75$ psf
- Live Load = 60 psf
  - $LLR = 60\left(0.25 + \dfrac{15}{\sqrt{2(9)(30)}}\right) = 53.7$ psf

    $K_{LL} = 2$, $A_T = 9(30)$ } $K_{LL}A_T = 540\#$ > 400# ok! ✓
- $w_u = 1.2(75+35)9' + 1.6(53.7)9' = 1.96$ KLF
- $M_u = \dfrac{(1.96)(29)^2}{8} = 206$ k

Note: Moment was calculated under simply supported conditions. More accurate design would consider more bays & moment coefficients.

2 of 6

Chris Dunlay | Technical Report #2 One way Slab w/ Beams

↳ Reinforcement @ Negative moment

- $A_{s,min} = 0.66 \text{ in}^2$
- $A_{s,max} = 4.33 \text{ in}^2$
- $A_{s,Req} \approx \dfrac{M_u}{4d} = \dfrac{172.8}{4(17.5)} = 2.47''$

$\therefore$ Try (3) #9's $\rightarrow A_s = (1.0)(3) = 3 \text{ in}^2$ ($> A_{s,min}$ & $< A_{s,max}$) OK! ✓

↳ Calculate M_n

- Assume $\varepsilon_s \geq \varepsilon_y$

$$a = \frac{A_s \cdot f_y}{0.85 f'_c b} = \frac{(3)(60)}{0.85(4)(12)} = 4.41 \text{ in} \;=\; C = \frac{4.41}{0.85} = 5.19$$

- Check $\varepsilon_s > \varepsilon_y$

$$\varepsilon_s = \frac{0.003}{5.19}(17.5 - 5.19) = 0.0071 > \varepsilon_y = 0.00207$$

$$\varepsilon_t = \varepsilon_s = 0.0071 > 0.005 \rightarrow \phi = 0.9$$

$$\phi M_n = 0.9(3)(60)\left(17.5 - \frac{4.41}{2}\right)/12 = \underline{\underline{206.5'k}} > 172.8'k$$

↳ Shear

- $V_c = 2\sqrt{f'_c}\, b \cdot d = 2\sqrt{4000}\,(12)(17.5) = 26.6^k$
- $\phi V_c = 0.5\,\phi V_c = 0.5(.75)(26.6) = 9.96^k$

↳ Shear Strength

$$V_s = \frac{V_u}{\phi} - V_c \;;\quad V_u = \frac{(2.26)(29)}{2} = 32.8^k$$

$$V_s = \frac{32.8}{0.75} - 26.6 = 17.13^k$$

$$V_{s,max} = 8\sqrt{f'_c}\, b \cdot d = 8\sqrt{4000}\,(12)(17.5) = 106.3^k > 32.8^k \text{ OK! } ✓$$

3 of 6 Chris Dunlay | Technical Report #2 | One way with Beams

↳ Estimate Beam size

- $bd^2 = \frac{M_u}{\phi R}$
- where; $R = \omega f'_c (1 - 0.59\omega)$

$$\rho = \frac{\beta_1 f'_c}{4 f_y} = \frac{0.85(4000)}{4(60000)} = 0.0142$$

$$\omega = \rho \frac{f_y}{f'_c} = 0.0142\left(\frac{60000}{4000}\right) = 0.213$$

$$R = 0.213(4000)(1 - 0.59(.213)) = 0.745 \text{ ksi}$$

- $bd^2 = \frac{(206)(12)}{(0.9)(0.745)} = 3686.8 \text{ in}^3$

∴ Try $b = 12'' \rightarrow d = 17.5'' \rightarrow h = 17.5 + 2.5 = 20''$

↳ use a 12" x 20" Beam.

↳ New Moment

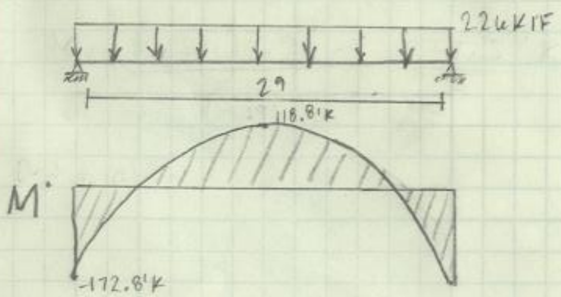
Slab = 75 psf

SDL = 35 psf

Beam = $\frac{(12)(20)}{144}(150) = 250$ plf

Live = 53.7 psf

$$w_u = 1.2\left[250 + (75+35)9\right] + 1.6(53.7)9' = 2.26 \text{ KLF}$$



Assume: · Continous ends

· interior beam

$$\therefore M^+ = \frac{w\ell^2}{16}$$

$$M^- = -\frac{w\ell^2}{11}$$

4 of 6 | Chris Dunlay | Technical Report #2 | One way with Beams

↳ Reinforcement @ Positive Moment.

- Check for T-Beam Behavior ($M_u = 118.8k$)

$$M_{u\text{-}T\text{-}Bm} = \phi 0.85 f'_c b\, h_f \left(d - \frac{h_f}{2}\right) = 0.9(0.85)(4)(12)(6)\left(17.5 - \frac{6}{2}\right)$$
$$= 266'k > 118.8k \quad \underline{\text{No T-Beam}}$$

- $$A_{s,min} = \min \begin{cases} \dfrac{3\sqrt{f'_c}\, b \cdot d}{f_y} = \dfrac{3\sqrt{4000}(12)(17.5)}{60{,}000} = \underline{\underline{0.66 \text{ in}^2}} \\[2ex] \dfrac{200\, b \cdot d}{f_y} = \dfrac{200(12)(17.5)}{60{,}000} = 0.7 \text{ in}^2 \end{cases}$$

- $$A_{s,max} = \rho \cdot b \cdot d \; ; \; \overset{f'_c = 4000}{\rho = 0.85\beta_1} \cdot \frac{f'_c}{f_y} \cdot \frac{\varepsilon_u}{\varepsilon_u + 0.004} = 0.0206$$
$$= 0.0206(12)(17.5) = \underline{\underline{4.326}} \text{ in}^2$$

- $$A_{s,req} \approx \frac{M_u}{4d} = \frac{118.8}{4(17.5)} = 1.69 \text{ in}^2$$

$\therefore$ Try (3) #8's $\rightarrow$ $A_s = 2.37 \text{ in}^2$ ($< A_{s,max}$ $> A_{s,min}$)
ok! ✓

↳ Calculate M_n

- Assume $\varepsilon_s \geq \varepsilon_y$

$$a = \frac{A_s \cdot f_y}{0.85 f'_c \cdot b} = \frac{2.37(60)}{0.85(4)(12)} = 3.49'' \rightarrow c = \overset{f'_c = 4000}{\frac{a}{\beta_1}} = \frac{3.49}{0.85} = 4.1''$$

- Check $\varepsilon_s > \varepsilon_y$

$$\varepsilon_s = \frac{\varepsilon_u}{c}(d - c) = \frac{0.003}{4.1}(17.5 - 4.1) = 0.0098$$
$$\varepsilon_y = \frac{60}{29000} = 0.00207$$

$$\varepsilon_y = 0.00207 < \varepsilon_s = 0.0098 \quad \text{ok! ✓}$$
$$\varepsilon_t = \varepsilon_s = 0.0098 > 0.005 \rightarrow \phi = 0.9$$

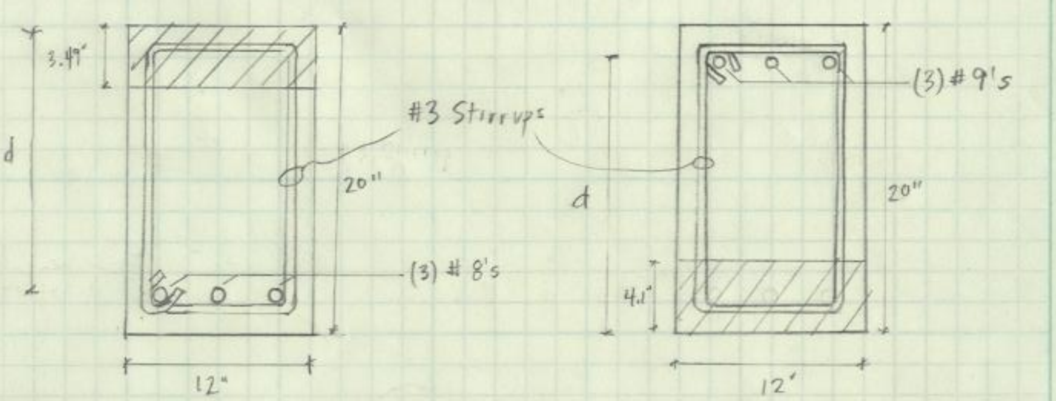
$$\phi M_n = \phi A_s \cdot f_y \left(d - \frac{a}{2}\right) = (0.9)(2.37)(60)\left(17.5 - \frac{3.49}{2}\right)/12$$
$$= \underline{\underline{168'k}} > 118'k \quad \underline{\text{ok! ✓}}$$

- S_{max} | $d/2 = 17.5/2 = 8.75"$ → controls
  | $24"$

- $A_{v,min} =$ | $0.75\sqrt{f'c}\, b \cdot S_{max}/f_{yT} = 47.4$
  max | $\frac{50\, b \cdot s_{max}}{f_{yT}} = \frac{50(12)(8.75)}{60000} = 0.0875\, in^2$

Try (2) Legs of #3 → $A_v = 2(0.11) = 0.22\, in^2 > 0.0875\, in^2$

Section @ M+ | Section @ M−



$d = 20" - 1.5" - (\frac{3}{8})" - (\frac{1.128}{2}) = 17.56"$ | $d = 20" - 1.5" - (\frac{3}{8})" - (\frac{1.128}{2})" = 17.56"$

↳Deflections

$W_{D+L} = (75)9' + 250 + (60)9' = 1.47$ KLF

$\Delta_{D+L} = \frac{5wL^4}{384EI}$; $E_c = 57000\sqrt{f'c} = 57000\sqrt{4000} = 3604$ ksi

$I = \frac{bh^3}{12} = \frac{12(20)^3}{12} = 8000\, in^4$

$= \frac{5(1.47)(30)^4}{384(3604)(8000)}(1728) = 0.93"$

$\Delta_{max} = \frac{l}{240} = \frac{30(12)}{240} = 1.5" > 0.93"$ OK

6 of 6 Chris Dunlay Technical Report #2 Oneway Slab w/ Beams

Slab Reinforcement

$W_u = 1.2(75+35)1' + 1.6(60)1' = 0.228$ klf

$M_n = \frac{(0.228)(9')^2}{8} = 2.31'k$

$A_s = \frac{M_u}{4d}$; $d = 6" - 2" = 4"$

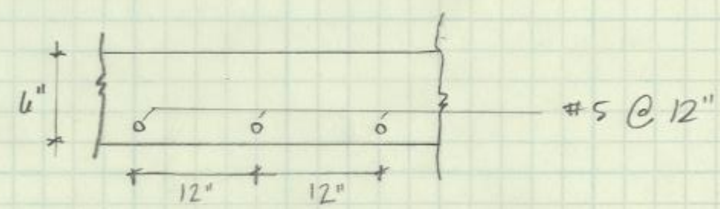
$= \frac{2.31}{4(4)} = 0.144 \text{ in}^2$ Try #5's @ 12" o.c.

$A_s = 0.31 \text{ in}^2$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.31)(60)}{0.85(4)(1)} = 5.5"$

$\phi M_n = 0.9(0.31)(60)\left(4 - \frac{5.5}{2}\right) = 20.9'k > 2.31'k$ ok! ✓

Slab Section



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A Computer Program for Analysis, Design, and Investigation of
Reinforced Concrete Beams, One-way and Two-way Slab Systems

[2] DESIGN RESULTS

Top Reinforcement

Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)

Span	Zone	Width	Mmax	Xmax	AsMin	AsMax	SpReq	AsReq	Bars
1	Left	3.00	119.36	0.500	0.728	3.942	6.115	1.556	6-#5
	Middle	3.00	0.00	15.000	0.000	3.942	0.000	0.000	---
	Right	3.00	119.36	29.500	0.728	3.942	6.115	1.556	6-#5

Top Bar Details

Units: Length (ft)

	Left				Continuous		Right			
Span	Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	3-#5	4.63	3-#5	2.77	---		3-#5	4.63	3-#5	2.77

Bottom Reinforcement

Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)

Span	Width	Mmax	Xmax	AsMin	AsMax	SpReq	AsReq	Bars
1	1.00	334.15	15.000	0.683	27.773	3.110	4.461	5-#9 2L

Bottom Bar Details

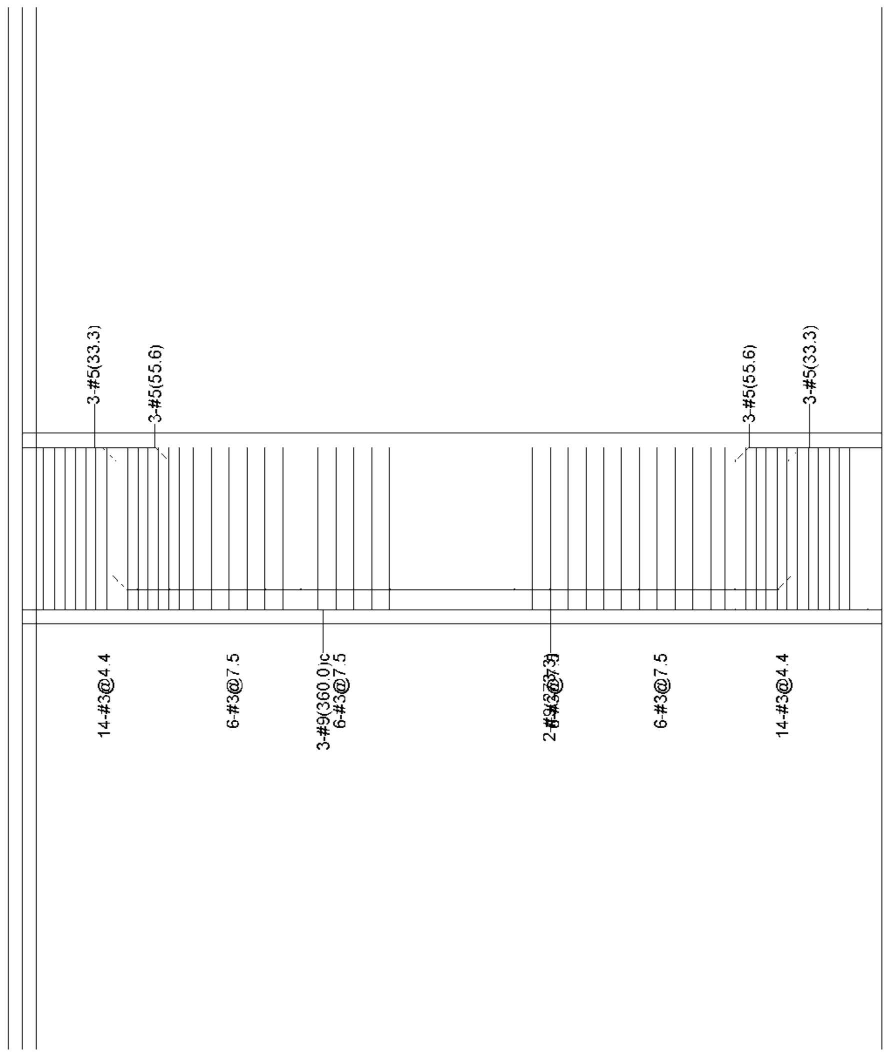
Units: Start (ft), Length (ft)

	Long Bars			Short Bars		
Span	Bars	Start	Length	Bars	Start	Length
1	3-#9	0.00	30.00	2-#9	3.61	22.77

Flexural Capacity

Units: x (ft), As (in^2), PhiMn (k-ft)

Span	x	AsTop	AsBot	PhiMn	PhiMn+
1	0.000	1.86	3.00	-140.78	238.17
	0.500	1.86	3.00	-140.78	238.17
	1.773	1.86	3.00	-140.78	238.17
	2.773	0.93	3.00	-73.25	238.17
	3.614	0.93	3.00	-73.25	238.17
	3.632	0.93	3.01	-73.25	238.73
	4.632	0.00	3.47	0.00	270.29
	7.942	0.00	5.00	0.00	373.38
	10.650	0.00	5.00	0.00	373.38
	15.000	0.00	5.00	0.00	373.38
	19.350	0.00	5.00	0.00	373.38
	22.058	0.00	5.00	0.00	373.38
	25.368	0.00	3.47	0.00	270.29
	26.368	0.93	3.01	-73.25	238.73



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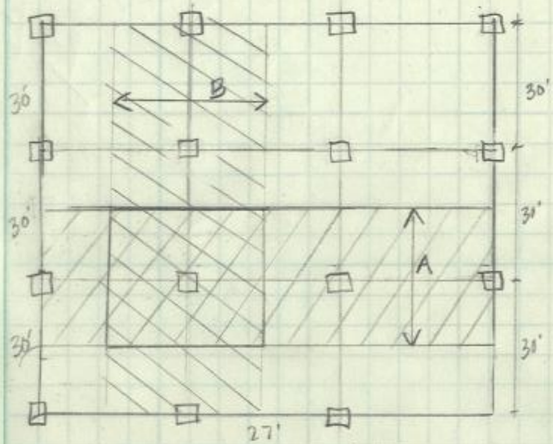
Project: One Way With Beams

Frame:

Engineer:

Appendix C: Two Way Flat Plate

1of6 | Chris Dunlay | Technical Report #2 | 2 way slab Direct Design



Assume 24"x24" columns

Location: Building 1

Level 6

Bay Sizes: 27' x 30'

Direct Design

- 3 spans ✓
- $\frac{30}{27} = 1.11 < 2$ ✓
- $w_L < 2 w_D$ ✓

Minimum Slab depth

↳ $t = \ell/33$ → Table 9.5b w/o drop panels interior panels w/ edge beams

$= \frac{(30'-2')12}{33}$

$= 10.2"$ → use 12" slab

Loads

↳ Self weight $= (1')(30')(151^{lb}/ft^3) = 4.5$ KLF

↳ Super Imposed Dead $= 35$ PSF $\times (30') = 1.05$ KIF

↳ Live Load $= 40$ PSF $\times (30') = 1.20$ KIF

↳ $W_u = 1.2(4.5+1.05)+1.6(1.2)$

$= 8.58$ KIF

Frame Moments

↳ Frame A : $M_{0A} = \frac{(8.58)(28)^2}{8} = 840.8$ ft-K

↳ Frame B : $M_{0B} = \frac{(8.58)(27-2)^2}{8} = 670.3$ ft-K

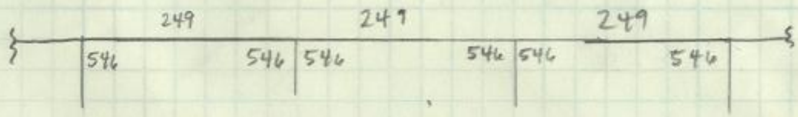
2 of 6 Chris Dunlay | Technical Report #2 | Two Way Flat Plate

↳ Moment Distribution

- Note: The bays under consideration are representative bays for calculations. Therefore consider all spans interior for calculation.

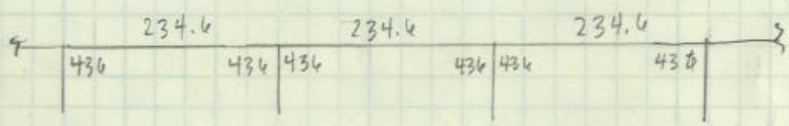
- $M_A^- = 0.65(840.8) = 546.5'k$

  $M_A^+ = 0.35(840.8) = 294.3'k$

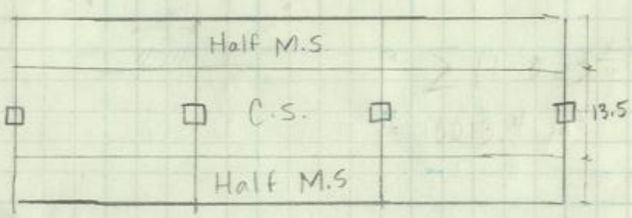


- $M_B^- = 0.65(670.3) = 435.7'k$

  $M_B^+ = 0.35(670.3) = 234.6'k$



↳ Determine Column & Middle Strips



Frame A

C.S.: $\frac{27'}{2}(12) = 162''$

M.S. $= [(30)(12) - 162'']2$
$= 99''$

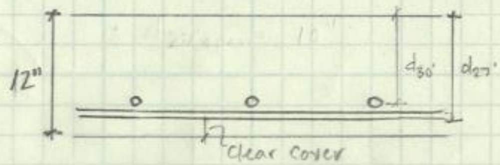
Frame B

C.S. $= 162''$

M.S $= 162''/2$
$= 81''$

3 of 4 Chris Dunlay Technical Report #2 Two way Flat Plate

↳ Calculate d



Assume using #5 rebar

$d_{27' span} = 12'' - 0.75'' - \frac{0.625}{2} = 10.94''$

$d_{30' span} = 10.94'' - 0.625'' = 10.31''$

↳ See spSlab for Reinforcement design.

- Frame A M^- Sample Calcs

- $M_o \times 12/b = \frac{-546(12)}{162} = 40.4 \frac{k\text{-}in}{in}$

- $M_n = \frac{M_o}{\phi} = \frac{-546}{0.9} = 606.7'k$

- $R = \frac{M_n}{bd^2} = \frac{(606.7)}{162(10.31)^2} = 422.8$ lb/in²

- ρ; $R = \rho f_y \left(1 - 0.59\rho\frac{f_y}{f'_c}\right)$

$422.8 = 40000\rho - 531000\rho^2$

$\rho = \frac{40000 \pm \sqrt{40000^2 - 4(531000)(422.8)}}{2(531000)}$

$= 0.00755 \rightarrow$ use

or

$= 0.08471$

- $A_s = \rho bd = 0.00755(162'')(10.31'') = 12.61\ in^2$

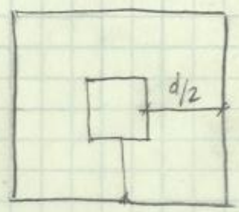
- $A_{s\,min} = 0.0018bt = 0.0018(162)(12) = 3.499\ in^2$

- $N = 12.61/0.31 = 40.68 \rightarrow$ use 41 Bars

- $N_{min} = \frac{b}{2t} = \frac{162}{2(12)} = 6.75$

Note: Refer to spSlab for remaing reinforcement.

Punching Shear



$$d/2 = \frac{10.94"}{2} = 5.47"$$

$$b_o = 4[24" + 2(5.47)] = 139.8 \text{ in}$$

$$V_c = 4\sqrt{4000}\,(139.8)(10.94) = 386.8^K$$

$$\phi V_c = 0.75(386.8) = \underline{\underline{290^K}}$$

$$w_u = 1.2(150 + 35) + 1.6(40) = 0.286 \text{ KSF}$$

$$V_u = w_u \cdot A_{ps} = (0.286)\left[(30)(27) - \frac{(24" + 10.94)^2}{144}\right] = \underline{\underline{229.2^K}}$$

$$\phi V_c > V_u \quad (290^K > 229^K)$$

OK! ✓

System is designed to withstand Punching Shear.

5 of 6 | Chris Dunlay | Technical Report #2 | Two Way Flat Plate

↳ Deflections

Column Strip (Assume 67.5% goes to this strip)

$$w_{DL} = \left(\frac{12}{12}\right)(150)(27)(0.675) = 2.73 \text{ KLF}$$

$$w_{LL} = (60)(27)(0.675) = 1.09 \text{ KLF}$$

$$I_{c.s} = \frac{162(12)^3}{12} = 23328 \text{ in}^4$$

$$E_c = 57000\sqrt{4000} = 3604 \text{ ksi}$$

$$\Delta_D = \frac{0.0026(2.73)(27)^4}{(3604)(23328)}(1728) = \underline{\underline{0.0775''}}$$

$$\Delta_L = \frac{0.0048(1.09)(27)^4}{(3604)(23328)}(1728) = \underline{\underline{0.057''}}$$

Middle Strip (Assume 32.5% goes to this strip)

$$w_{DL} = \left(\frac{12}{12}\right)(150)(30)(0.325) = 1.46 \text{ KLF}$$

$$w_{LL} = (60)(30)(0.325) = 0.59 \text{ KLF}$$

$$I_{M.S.} = \frac{81(12)^3}{12} = 11664 \text{ in}^4$$

$$E_c = 3604 \text{ ksi}$$

$$\Delta_D = \frac{0.0026(1.46)(30)^4}{(3604)(11664)}(1728) = \underline{\underline{0.126''}}$$

$$\Delta_L = \frac{0.0048(0.585)(30)^4}{(3604)(11664)}(1728) = \underline{\underline{0.0934''}}$$

6 of 6 | Chris Dunlay | Technical Report #2 | Two Way Flat Plate

Long Term Deflections

Column Strip

$$\Delta_{LT} = 3(\Delta_{DL} + 0.25\Delta_{LL}) = 3[0.0775 + 0.25(0.57)]$$
$$= 0.275''$$

Middle strip

$$\Delta_{LT} = 3[0.126 + 0.25(0.0934)]$$
$$= 0.448$$

$$\Delta_{TL} = 0.275'' + 0.448'' = 0.723''$$

$$\Delta_{max} = 0.1(\Delta_D) + \Delta_{TL} + \Delta_{LL}$$
$$= 0.1(0.0775 + 0.126) + 0.723 + (0.057 + 0.0934)$$
$$= 0.89''$$

$$\Delta_{Allowable} = \frac{l}{240} = \frac{30(12)}{240} = 1.5''$$

$$\boxed{0.89'' < 1.5''}$$

Frame A

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[2] DESIGN RESULTS*

*Unless otherwise noted, all results are in the direction of analysis only. Another analysis in the perpendicular direction has to be carried out for two-way slab systems.

Strip Widths and Distribution Factors

Units: Width (ft).

Span	Strip	Width Left**	Width Right**	Width Bottom*	Moment Factor Left**	Moment Factor Right**	Moment Factor Bottom*
1	Column	13.50	13.50	13.50	1.000	0.750	0.600
	Middle	16.50	16.50	16.50	0.000	0.250	0.400
2	Column	13.50	13.50	13.50	0.750	0.750	0.600
	Middle	16.50	16.50	16.50	0.250	0.250	0.400
3	Column	13.50	13.50	13.50	0.750	1.000	0.600
	Middle	16.50	16.50	16.50	0.250	0.000	0.400

*Used for bottom reinforcement. **Used for top reinforcement.

Top Reinforcement

Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)

Span	Strip	Zone	Width	Mmax	Xmax	AsMin	AsMax	SpReq	AsReq	Bars	
1	Column	Left	13.50	15.33	1.000	3.433	29.810	13.500	0.335	12-#5	*3
		Middle	13.50	0.00	13.500	0.000	29.810	0.000	0.000	---	
		Right	13.50	392.32	26.000	3.433	29.810	5.586	8.990	29-#5	
	Middle	Left	16.50	0.00	1.000	0.000	36.434	0.000	0.000	---	
		Middle	16.50	0.00	13.500	0.000	36.434	0.000	0.000		
		Right	16.50	130.78	26.000	4.277	36.434	14.143	2.889	14-#5	*3
2	Column	Left	13.50	352.44	1.000	3.433	29.810	5.586	8.033	29-#5	
		Middle	13.50	0.00	13.500	0.000	29.810	0.000	0.000	---	
		Right	13.50	352.44	26.000	3.433	29.810	5.586	8.033	29-#5	
	Middle	Left	16.50	117.48	1.000	4.277	36.434	14.143	2.592	14-#5	*3
		Middle	16.50	0.00	13.500	0.000	36.434	0.000	0.000	---	
		Right	16.50	117.48	26.000	4.277	36.434	14.143	2.592	14-#5	*3
3	Column	Left	13.50	392.32	1.000	3.433	29.810	5.586	8.990	29-#5	
		Middle	13.50	0.00	13.500	0.000	29.810	0.000	0.000	---	
		Right	13.50	15.33	26.000	3.433	29.810	13.500	0.335	12-#5	*3
	Middle	Left	16.50	130.78	1.000	4.277	36.434	14.143	2.889	14-#5	*3
		Middle	16.50	0.00	13.500	0.000	36.434	0.000	0.000	---	
		Right	16.50	0.00	26.000	0.000	36.434	0.000	0.000		

NOTES:
*3 - Design governed by minimum reinforcement.

Top Bar Details

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Top Bar Details
===============

Units: Length (ft)

		Left				Continuous		Right			
Span	Strip	Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column	12-#5	9.25	---		---		15-#5	9.25	14-#5	6.00
	Middle	---		---		---		14-#5	8.06	---	
2	Column	15-#5	9.25	14-#5	6.00	---		15-#5	9.25	14-#5	6.00
	Middle	14-#5	9.06	---		---		14-#5	9.06	---	
3	Column	15-#5	9.25	14-#5	6.00	---		12-#5	9.25	---	
	Middle	14-#5	8.06	---		---		---		---	

Bottom Reinforcement
====================

Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)

Span	Strip	Width	Mmax	Xmax	AsMin	AsMax	SpReq	AsReq	Bars	
1	Column	13.50	255.72	11.000	3.499	29.810	8.526	5.755	19-#5	
	Middle	16.50	170.48	11.000	4.277	36.434	14.143	3.781	14-#5	*3
2	Column	13.50	132.63	13.500	3.499	29.810	13.500	2.939	12-#5	*3
	Middle	16.50	88.42	13.500	4.277	36.434	14.143	1.945	14-#5	*3
3	Column	13.50	255.72	16.000	3.499	29.810	8.526	5.755	19-#5	
	Middle	16.50	170.48	16.000	4.277	36.434	14.143	3.781	14-#5	*3

NOTES:
*3 - Design governed by minimum reinforcement.

Bottom Bar Details
==================

Units: Start (ft), Length (ft)

		Long Bars			Short Bars		
Span	Strip	Bars	Start	Length	Bars	Start	Length
1	Column	19-#5	0.00	27.00	---		
	Middle	14-#5	0.00	27.00	---		
2	Column	12-#5	0.00	27.00	---		
	Middle	14-#5	0.00	27.00	---		
3	Column	19-#5	0.00	27.00	---		
	Middle	14-#5	0.00	27.00	---		

Flexural Capacity
=================

Units: x (ft), As (in^2), PhiMn (k-ft)

Span	Strip	x	AsTop	AsBot	PhiMn-	PhiMn+
1	Column	0.000	3.72	5.89	-167.15	261.52
		1.000	3.72	5.89	-167.15	261.52
		8.250	3.72	5.89	-167.15	261.52
		9.250	0.00	5.89	0.00	261.52
		9.750	0.00	5.89	0.00	261.52
		13.500	0.00	5.89	0.00	261.52
		17.250	0.00	5.89	0.00	261.52
		17.750	0.00	5.89	0.00	261.52
		18.935	4.65	5.89	-207.87	261.52
		20.999	4.65	5.89	-207.87	261.52
		22.185	8.99	5.89	-392.33	261.52
		26.000	8.99	5.89	-392.33	261.52
		27.000	8.99	5.89	-392.33	261.52
	Middle	0.000	0.00	4.34	0.00	195.18
		1.000	0.00	4.34	0.00	195.18
		9.750	0.00	4.34	0.00	195.18
		13.500	0.00	4.34	0.00	195.18
		17.250	0.00	4.34	0.00	195.18
		18.938	0.00	4.34	0.00	195.18
		19.938	4.34	4.34	-195.18	195.18
		26.000	4.34	4.34	-195.18	195.18
		27.000	4.34	4.34	-195.18	195.18
2	Column	0.000	8.99	3.72	-392.33	167.15
		1.000	8.99	3.72	-392.33	167.15
		4.941	8.99	3.72	-392.33	167.15
		6.001	4.65	3.72	-207.87	167.15
		8.191	4.65	3.72	-207.87	167.15
		9.250	0.00	3.72	0.00	167.15
		9.750	0.00	3.72	0.00	167.15
		13.500	0.00	3.72	0.00	167.15
		17.250	0.00	3.72	0.00	167.15
		17.750	0.00	3.72	0.00	167.15
		18.809	4.65	3.72	-207.87	167.15
		20.999	4.65	3.72	-207.87	167.15
		22.059	8.99	3.72	-392.33	167.15

Frame B

```
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======================================================================================
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                A Computer Program for Analysis, Design, and Investigation of
                 Reinforced Concrete Beams, One-way and Two-way Slab Systems
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[2] DESIGN RESULTS*

*Unless otherwise noted, all results are in the direction of analysis only. Another analysis in the perpendicular direction has to be carried out for two-way slab systems.

Strip Widths and Distribution Factors

Units: Width (ft).

Span	Strip	Width Left**	Width Right**	Width Bottom*	Moment Factor Left**	Moment Factor Right**	Moment Factor Bottom*
1	Column	13.50	13.50	13.50	1.000	0.750	0.600
	Middle	13.50	13.50	13.50	0.000	0.250	0.400
2	Column	13.50	13.50	13.50	0.750	0.750	0.600
	Middle	13.50	13.50	13.50	0.250	0.250	0.400
3	Column	13.50	13.50	13.50	0.750	1.000	0.600
	Middle	13.50	13.50	13.50	0.250	0.000	0.400

*Used for bottom reinforcement. **Used for top reinforcement.

Top Reinforcement

Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)

Span	Strip	Zone	Width	Mmax	Xmax	AsMin	AsMax	SpReq	AsReq	Bars	
1	Column	Left	13.50	62.35	1.000	3.499	29.810	13.500	1.370	12-#5	*3
		Middle	13.50	0.00	15.000	0.000	29.810	0.000	0.000	---	
		Right	13.50	402.28	29.000	3.499	29.810	5.400	8.231	30-#5	
	Middle	Left	13.50	0.09	1.280	3.499	29.810	13.500	0.002	12-#5	*3
		Middle	13.50	0.00	15.000	0.000	29.810	0.000	0.000		
		Right	13.50	134.10	29.000	3.499	29.810	13.500	2.972	12-#5	*3
2	Column	Left	13.50	323.12	1.000	3.499	29.810	5.400	7.336	30-#5	
		Middle	13.50	0.00	15.000	0.000	29.810	0.000	0.000	---	
		Right	13.50	277.20	29.000	3.499	29.810	6.750	6.256	24-#5	
	Middle	Left	13.50	107.71	1.000	3.499	29.810	13.500	2.380	12-#5	*3
		Middle	13.50	0.00	15.000	0.000	29.810	0.000	0.000	---	
		Right	13.50	92.40	29.000	3.499	29.810	13.500	2.038	12-#5	*3
3	Column	Left	13.50	322.47	1.000	3.499	29.810	6.750	7.321	24-#5	
		Middle	13.50	0.00	15.000	0.000	29.810	0.000	0.000	---	
		Right	13.50	41.70	29.000	3.499	29.810	13.500	0.914	12-#5	*3
	Middle	Left	13.50	107.50	1.000	3.499	29.810	13.500	2.375	12-#5	*3
		Middle	13.50	0.00	15.000	0.000	29.810	0.000	0.000		
		Right	13.50	0.06	28.720	3.499	29.810	13.500	0.001	12-#5	*3

NOTES:
*3 - Design governed by minimum reinforcement.

Top Bar Details

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Top Bar Details
===============

Units: Length (ft)

Span	Strip	Left Bars	Left Length	Left Bars	Left Length	Continuous Bars	Continuous Length	Right Bars	Right Length	Right Bars	Right Length
1	Column	12-#5	10.24	---		---		15-#5	10.24	15-#5	6.60
	Middle	12-#5	7.16	---		---		12-#5	8.35	---	
2	Column	15-#5	10.87	15-#5	6.60	---		12-#5	10.24	12-#5	6.60
	Middle	12-#5	10.87	---		---		12-#5	9.19	---	
3	Column	12-#5	10.24	12-#5	6.60	---		12-#5	10.24	---	
	Middle	12-#5	8.35	---		---		12-#5	7.16	---	

Bottom Reinforcement
====================

Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)

Span	Strip	Width	Mmax	Xmax	AsMin	AsMax	SpReq	AsReq	Bars	
1	Column	13.50	285.57	12.760	3.499	29.810	7.714	6.452	21-#5	
	Middle	13.50	190.38	12.760	3.499	29.810	11.571	4.249	14-#5	
2	Column	13.50	93.50	15.280	3.499	29.810	13.500	2.062	12-#5	*3
	Middle	13.50	62.33	15.280	3.499	29.810	13.500	1.370	12-#5	*3
3	Column	13.50	202.06	17.520	3.499	29.810	10.800	4.517	15-#5	
	Middle	13.50	134.71	17.520	3.499	29.810	13.500	2.986	12-#5	*3

NOTES:
*3 - Design governed by minimum reinforcement.

Bottom Bar Details
==================

Units: Start (ft), Length (ft)

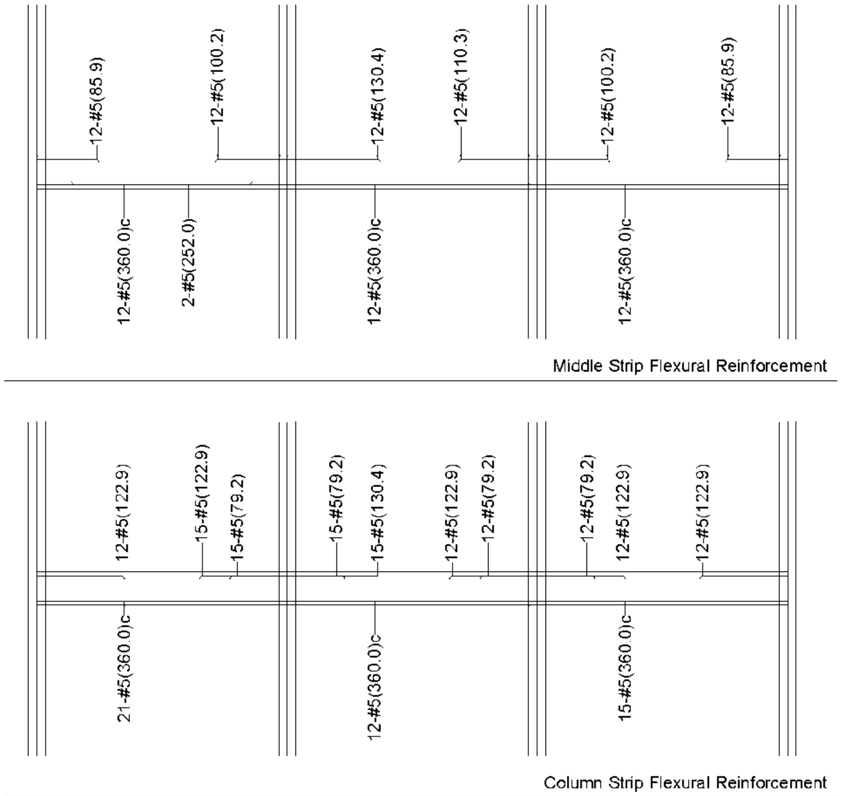
Span	Strip	Long Bars Bars	Long Bars Start	Long Bars Length	Short Bars Bars	Short Bars Start	Short Bars Length
1	Column	21-#5	0.00	30.00	---		
	Middle	12-#5	0.00	30.00	2-#5	4.50	21.00
2	Column	12-#5	0.00	30.00	---		
	Middle	12-#5	0.00	30.00	---		
3	Column	15-#5	0.00	30.00	---		
	Middle	12-#5	0.00	30.00	---		

Flexural Capacity
=================

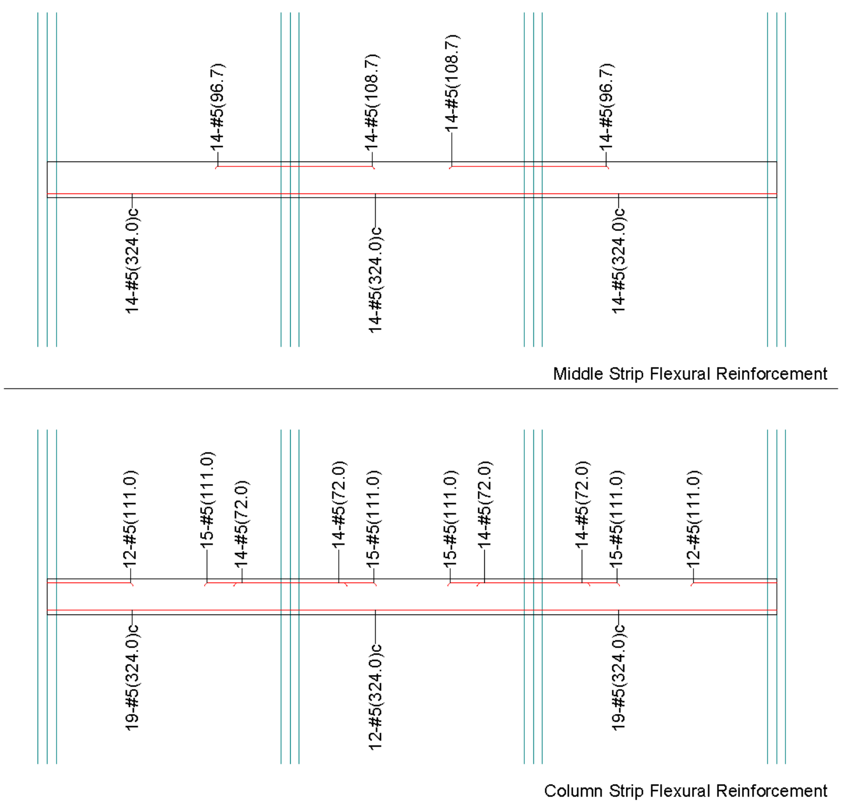
Units: x (ft), As (in^2), PhiMn (k-ft)

Span	Strip	x	AsTop	AsBot	PhiMn-	PhiMn+
1	Column	0.000	3.72	6.51	-167.15	288.06
		1.000	3.72	6.51	-167.15	288.06
		9.240	3.72	6.51	-167.15	288.06
		10.240	0.00	6.51	0.00	288.06
		10.800	0.00	6.51	0.00	288.06
		15.000	0.00	6.51	0.00	288.06
		19.200	0.00	6.51	0.00	288.06
		19.760	0.00	6.51	0.00	288.06
		20.937	4.65	6.51	-207.87	288.06
		23.399	4.65	6.51	-207.87	288.06
		24.576	9.30	6.51	-405.15	288.06
		29.000	9.30	6.51	-405.15	288.06
		30.000	9.30	6.51	-405.15	288.06
	Middle	0.000	3.72	3.72	-167.15	167.15
		1.000	3.72	3.72	-167.15	167.15
		4.500	3.72	3.72	-167.15	167.15
		5.661	3.72	4.34	-167.15	194.35
		6.161	3.72	4.34	-167.15	194.35
		7.161	0.00	4.34	0.00	194.35
		10.800	0.00	4.34	0.00	194.35
		15.000	0.00	4.34	0.00	194.35
		19.200	0.00	4.34	0.00	194.35
		21.650	0.00	4.34	0.00	194.35
		22.650	3.72	4.34	-167.15	194.35
		24.339	3.72	4.34	-167.15	194.35
		25.500	3.72	3.72	-167.15	167.15
		29.000	3.72	3.72	-167.15	167.15
		30.000	3.72	3.72	-167.15	167.15
2	Column	0.000	9.30	3.72	-405.15	167.15
		1.000	9.30	3.72	-405.15	167.15
		5.601	9.30	3.72	-405.15	167.15
		6.601	4.65	3.72	-207.87	167.15
		9.870	4.65	3.72	-207.87	167.15
		10.800	0.32	3.72	-14.87	167.15
		10.870	0.00	3.72	0.00	167.15

Frame A



Frame B

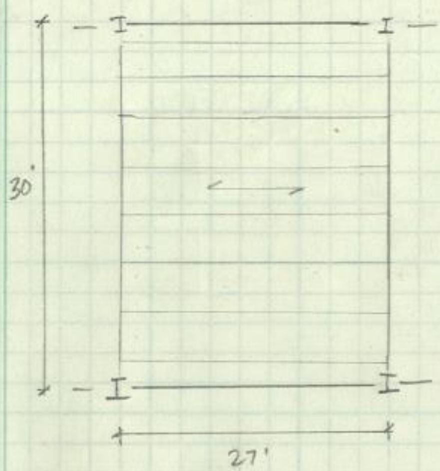


	Frame A							Frame B					
	Column Strip			Middle Strip				Column Strip			Middle Strip		
	M^-	M^+	M^-	M^-	M^+	M^-		M^-	M^+	M^-	M^-	M^+	M^-
Mo	-546	249	-546	-546	249	-546		-436	234.6	-436	-436	234.6	-436
Width (b)	162	162	162	99	99	99		162	162	162	81	81	81
d	10.31	10.31	10.31	10.31	10.31	10.31		10.94	10.94	10.94	10.94	10.94	10.94
Mux12/b	-40.44	18.44	-40.44	-66.18	30.18	-66.18		-32.30	17.38	-32.30	-64.59	34.76	-64.59
Mu	-606.67	276.67	-606.67	-606.67	276.67	-606.67		-484.44	260.67	-484.44	-484.44	260.67	-484.44
R	-422.77	192.80	-422.77	-691.80	315.49	-691.80		-299.83	161.33	-299.83	-599.66	322.66	-599.66
P	0.00755	0.00331	0.00755	0.01303	0.00553	0.01303		0.00524	0.00276	0.00524	0.01108	0.00566	0.01108
As	12.61	5.53	12.61	13.30	5.64	13.30		9.29	4.88	9.29	9.82	5.02	9.82
Asmin	3.50	3.50	3.50	2.14	2.14	2.14		3.50	3.50	3.50	1.75	1.75	1.75
N	40.68	17.84	40.68	42.91	18.20	42.91		29.96	15.76	29.96	31.68	16.18	31.68
Nmin	6.75	6.75	6.75	4.125	4.125	4.125		6.75	6.75	6.75	3.375	3.375	3.375
Use	**41**	**18**	**41**	**43**	**19**	**43**		**30**	**16**	**30**	**32**	**17**	**32**

5 Reinforcement Bars

Appendix D: Hollow Core Planks on Steel Framing

1of 2 | Chris Dunlay | Technical Report #2 | Hollow Core Planks on Steel Framing.



Try planks spanning 27'

- From Nitterhouse Specs

↳ use 8"x4' 7-½" strands
$W_{Allowable} = 161$ psf

Floor loads

- SDL = 35 psf
- Planks = 61.25 psf
- Live = 60 psf
- Beam Self doesn't load the planks

$w_u = 1.2(35) + 1.6(60) = 138$ psf

$w_u < w_{u,Allowable}$

$\boxed{\text{Use } 8'' \times 4' \text{ w/ } 2'' \text{ topping } 7\tfrac{1}{2}'' \text{ strands}}$

*Note: Only (7) planks will fully fit within a 30' span but since the other bays are 30' 2' overlap in each bay will make a feasible layout.

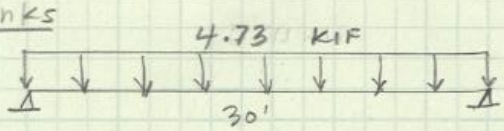
Live Load Reduction

$L = L_0\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$; $K_{LL} = 2$ (interior Bm) $A_T = (27)(30) = 810$

$= 60\left(0.25 + \frac{15}{\sqrt{2(810)}}\right) = 37.4$ psf

2 of 2

Design the Girders w/ planks



$W_u = 1.2(35+61.25)+1.6(37.4)$
$= 175.4 \text{ psf}$

$M_u = \dfrac{(175.4)(27)(30)^2}{8} = 533\ 'K$

$V_u = \dfrac{(175.4)(27)(30)}{2} = 63.2\ K$

From Table 3-10

· @ UBL = 30'

↳ use W 21 x 101 w/ $\phi M_n = 597\ 'K$
$I = 3100 \text{ in}^4$

Check Deflections

Live Load

$\Delta_{max} = l/360 = \dfrac{30(12)}{360} = 1.0''$

$\Delta_{LL} = \dfrac{5wL^4}{384EI} = \dfrac{5(0.06)(27)(30)^4}{384(29000)(2420)}(1728) = 0.42'' < 1.0''$ ok!

Total Lod

$\Delta_{max} = l/240 = \dfrac{30(12)}{240} = 1.5''$

$\Delta_T = \dfrac{5(4.73)(30)^4}{384(29000)(2420)} = 1.22'' < 1.5''$ ok!

$\boxed{\text{use W 21 x 101}}$

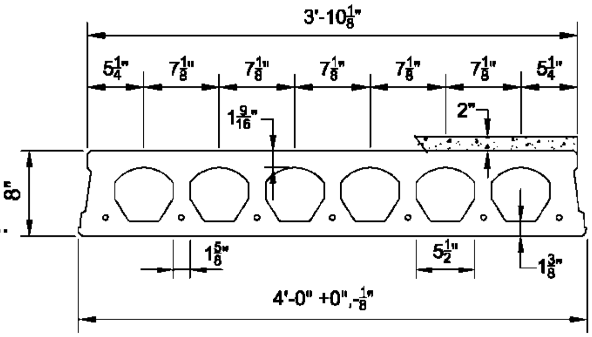
Prestressed Concrete
8"x4'-0" Hollow Core Plank

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

A_c = 301 in.^{2}	Precast b_w = 13.13 in.
I_c = 3134 in.^{4}	Precast S_{bcp} = 616 in.^{3}
Y_{bcp} = 5.09 in.	Topping S_{tct} = 902 in.^{3}
Y_{tcp} = 2.91 in.	Precast S_{tcp} = 1076 in.^{3}
Y_{tct} = 4.91 in.	Precast Wt. = 245 PLF
	Precast 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 at 60% jacking force
   6-1/2"Ø, 270K = 130.6 k-ft at 60% jacking force
   7-1/2"Ø, 270K = 147.8 k-ft at 60% jacking force
7. Maximum bottom tensile stress is $10\sqrt{f'c}$ = 775 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 2006 & ACI 318-05 (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)	280	248	214	185	159	138	118	102	87	74	62	52	42						
6 - 1/2"ø	LOAD (PSF)	366	341	318	299	271	239	211	187	165	146	129	114	101	88	77	67	58	50	42
7 - 1/2"ø	LOAD (PSF)	367	342	320	300	282	265	243	221	202	181	161	144	128	114	101	90	79	70	61

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.

NITTERHOUSE
CONCRETE PRODUCTS
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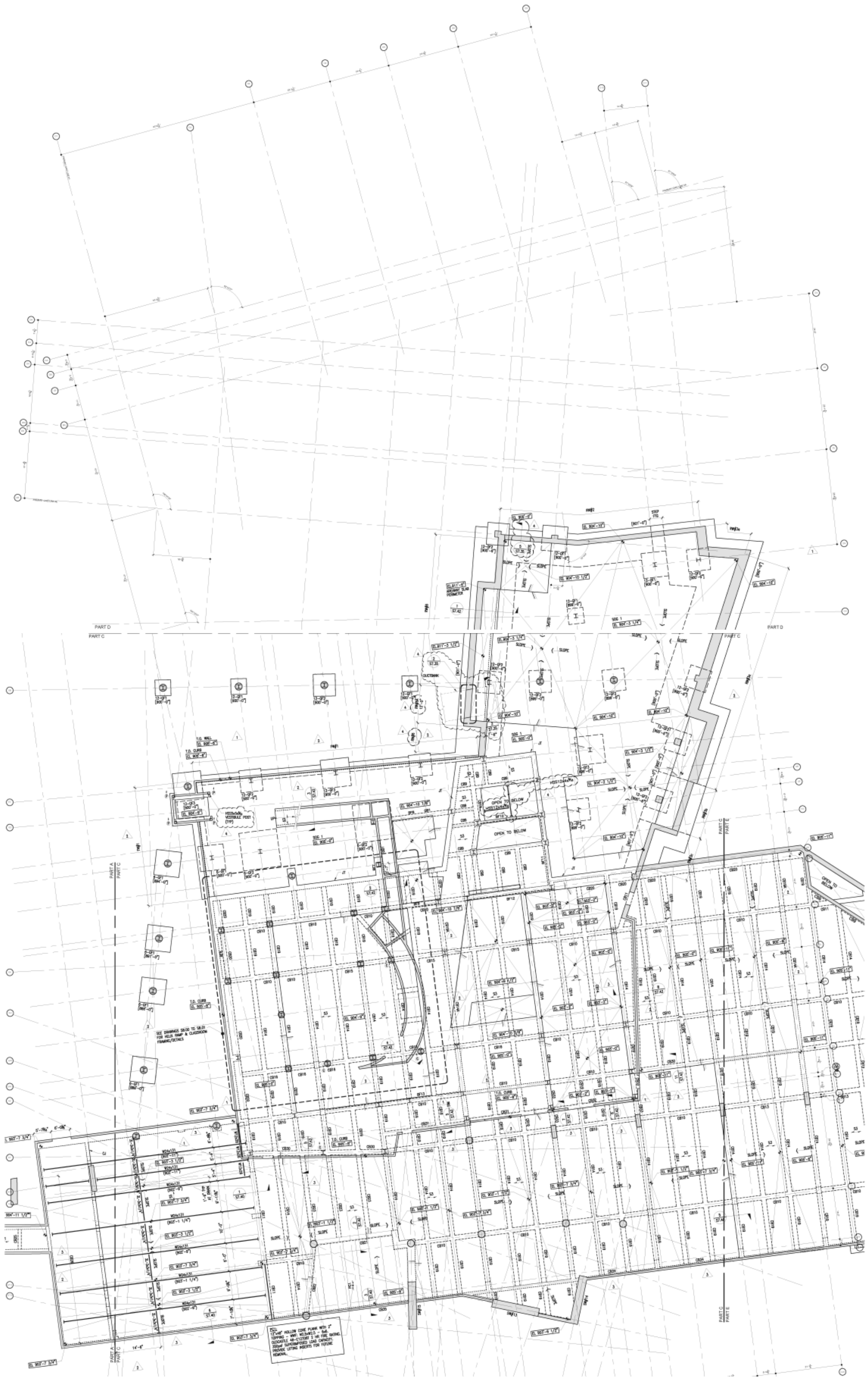
Chambersburg, PA 17202-9203
717-267-4505 Fax 717-267-4518

11/03/08

8SF2.0T

Appendix E: Typical Floor Plans

Level 3

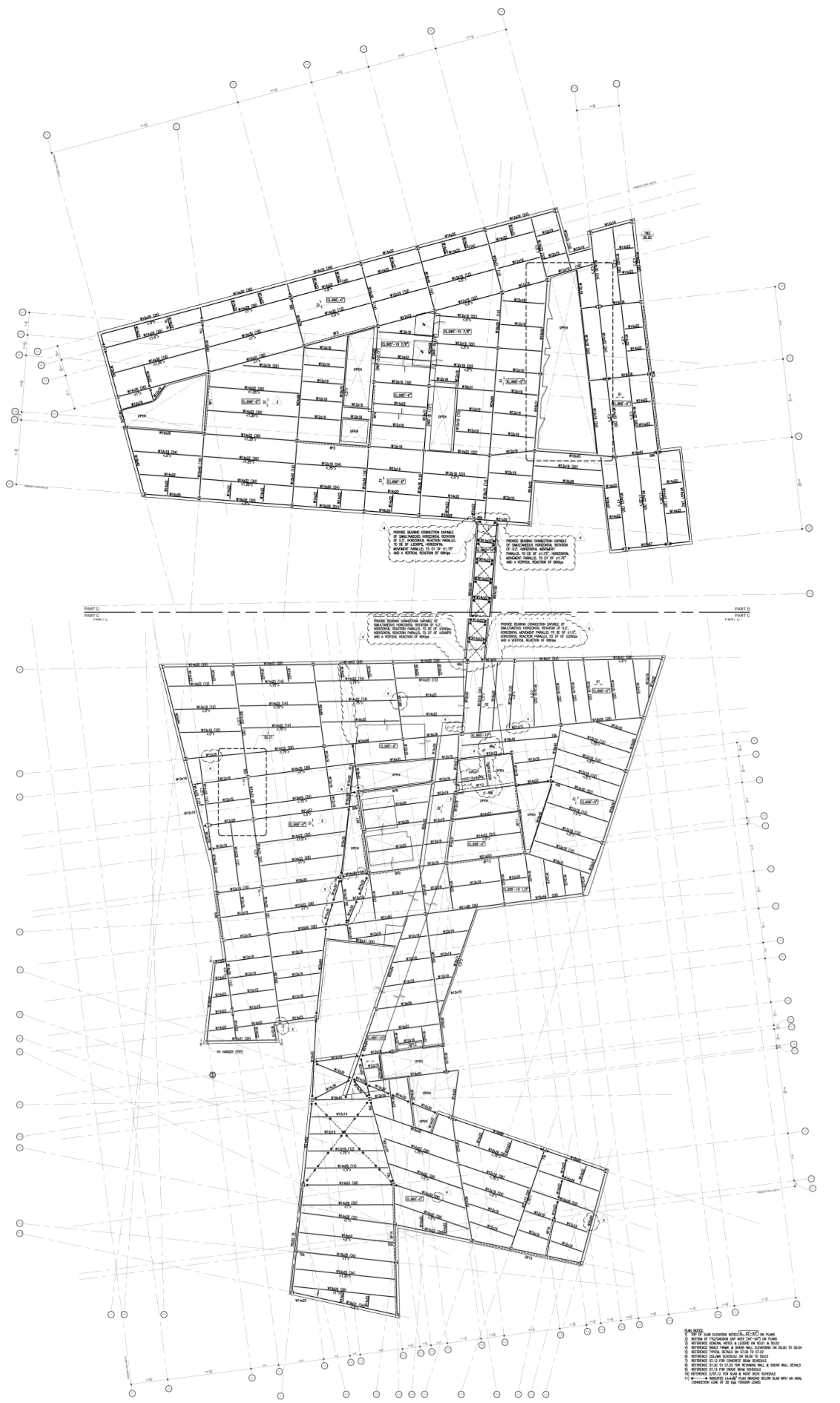


Level 4

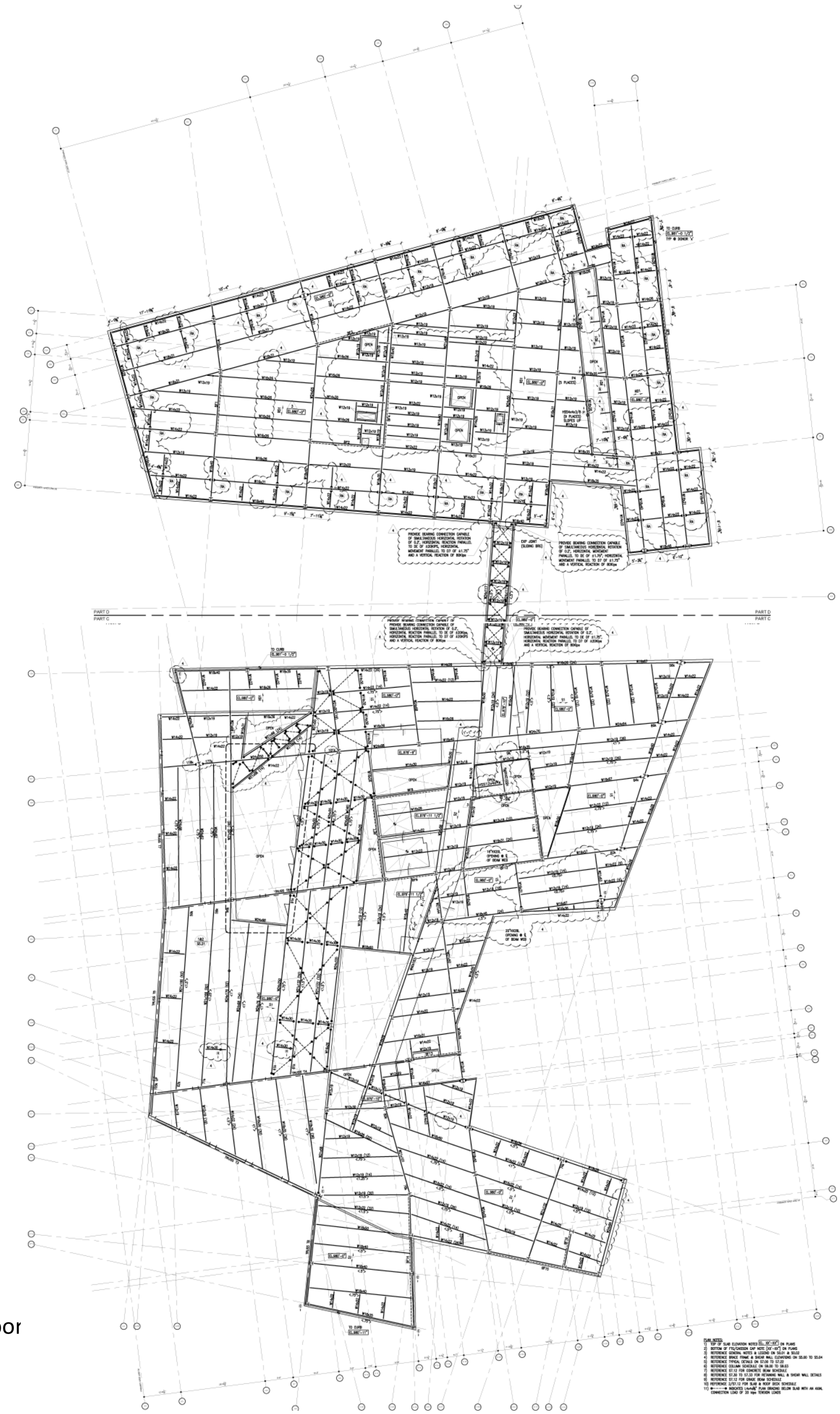
Level 5

Level 6

Level 7



Level 8



Level 9