HOWARD COUNTY GENERAL HOSPITAL

PATIENT TOWER ADDITION

COLUMBIA, MD

Technical Assignment 2:
Alternate Floor System Analysis

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October 29, 2007

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I. EXECUTIVE SUMMARY

This second technical report explores four possible floor systems and compares them to the existing composite system. The overall goal is to determine whether the existing system is the most appropriate floor system or if an alternate system would be better suited for this building.

The new framing systems that were investigated in addition to the existing composite system are:

1. Two-Way Concrete Flat Plate
2. Two-Way Concrete Flat Slab
3. Open Web Steel Joists
4. Hollow Core Precast Planks

Any available design aids and handbooks were utilized for simplicity. These references are included in the Codes and Standards portion of the report. An appendix is included at the end that consists of calculations for the various floor system alternatives.

The existing composite system was confirmed to be a practical choice of floor system as it provides a durable system with a low overall dead weight. Obviously this system accomplished the architectural layout required for the hospital, though the slab depressions at the upper levels were a bit of an inconvenience.

It was also found, however, that the flat slab system with drop panels is worth further investigation as it is cost effective and can efficiently carry the building's larger live loads. There will need to be a great deal of additional analysis as the tables used for the design of this system are ideal for simple rectangular buildings and the framing in this building is more complicated. In addition, the hollow core plank system proves to be another viable alternative though it would require fairly large steel beams and girders, which are both costly and difficult to maneuver. The flat plate system required too large of columns that would infringe with the architectural layout. The steel joists required large, heavy steel girders, like the precast system, and though it seems feasible, it does not seem to be the ideal system for this building due to sound attenuation and vibration issues. Obviously, all systems have both advantages and disadvantages, and a table is included later on for easy comparison.

II. INTRODUCTION TO BUILDING

Howard County Hospital is a member of Johns Hopkins Medicine located in Columbia, MD. It has been serving the surrounding community for over thirty years and grown significantly in the last decade. The most recent expansion, the 114,261 square foot patient tower, began construction in April 2007. This tower consists of one level partially below grade, four levels above grade, and a generously sized penthouse for a total building height of 88'-6" above grade (at the penthouse roof). The basement level consists mainly of offices for the hospital staff, storage areas, and mechanical/electrical rooms. The first floor is made up of a large gym along with cardio pulmonary and physical therapy areas. Patient rooms comprise the upper three levels, with each of the three floors providing thirty new beds for surgical or other medical patients.

The patient tower addition is part of a larger allover expansion known as the "Campus Development Plan." It is located on the south west side of the existing south building, close to Cedar Lane. Currently, the site consists of asphalt paved driveways and parking areas as well as a small landscape area. The topography gently slopes towards the west with an overall change in elevation of about 12 feet. The façade was selected to be horizontal bands of precast concrete, glass, and aluminum panels, similar to the existing hospital's exterior.

This expansion of the hospital was designed with large column bays and a 100 psf live load for flexibility in case of future renovations. Other portions of the hospital are currently undergoing renovations, demonstrating that designing for flexibility is a legitimate issue as the hospital grows and changes. This need for flexibility also contributed to the selection of moment frames as opposed to braced frames or another lateral system.

This report explains the existing structural system and checks that the selected members are capable of carrying the loads. Four alternate floor systems are then investigated and compared to determine if another structural system would be more ideal. Design handbooks and tables were used where applicable.

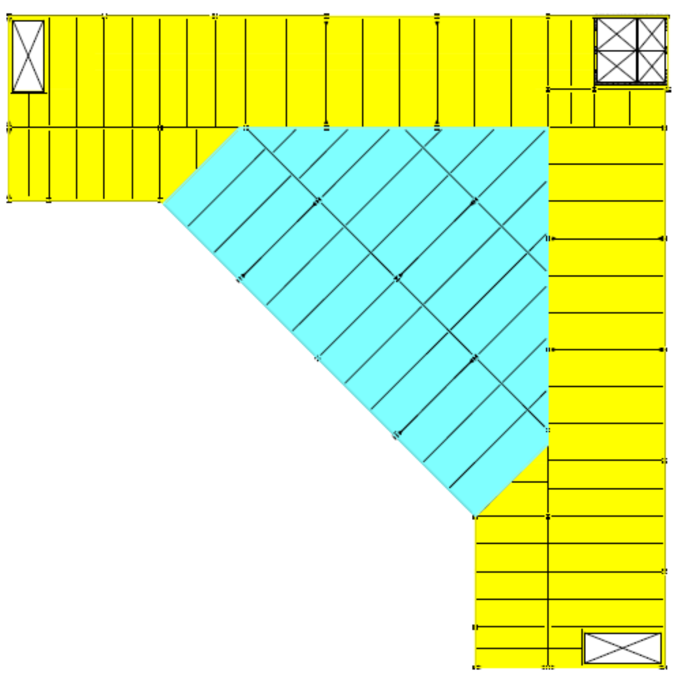
III. STRUCTURAL OVERVIEW

Existing Floor System:

The existing typical floor framing system is 3 ¼" lightweight concrete on 2" deep 18 gage composite metal deck for a total depth 5 ¼". Composite action is achieved with ¾" diameter by 4" shear studs evenly spaced along the length of supported beams. This total floor system attains a fire rating of two hours, according to the United Steel Deck catalog. There are three typical infill beam sizes – W12x19, W14x22, and W16x26. These beams vary from 19 feet to 30 ½ feet in length and are usually spaced at 7'-3" or 9'-8". In addition to the standard composite slab, additional reinforcing of 5 foot long #4 top bars are specified at 16" on center over all interior girders.

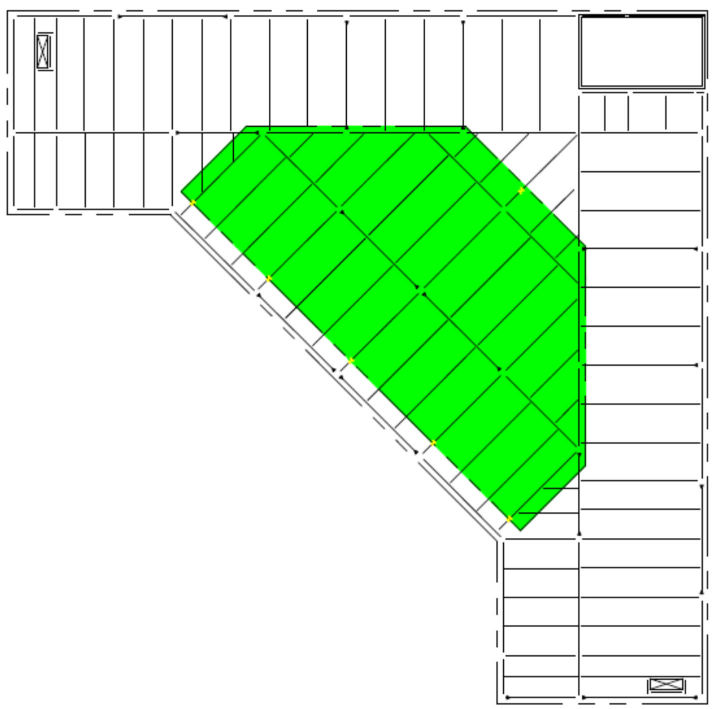
The first floor has a small 1-story extension on the north side of the building that connects to the existing hospital. This area is framed with W10x12 and W14x22 infill beams. The composite slab in this area is the same 5 ¼" thickness as the main addition.

The new addition is a uniquely shaped structure, so the floors are framed in two different directions. As you can see in the figure below, the "center" floor framing (shown in blue) is rotated at a 45 degree angle from the framing along the outer "L" of the building (shown in yellow). Please see the analysis of the existing floor system and Appendix A for more information.



Roof System:

The main roof is also a composite system since a considerable portion of it is occupied for the mechanical penthouse floor. This roof/floor system is composed of the same 3 ¼" lightweight concrete on 2" metal deck as the existing typical floors are. Infill beam sizes and lengths are similar to those mentioned above in the typical floor system. Transfer girders are also required at this level for 6 new columns that extend from the roof/penthouse floor up to the penthouse roof. You can see the portion of this level that is roof, shown in white below, and the portion that is penthouse, shown in green below.



The penthouse roof is the only floor system that varies from the typical system as it is 1 ½" wide rib 20 gage metal roof deck. The infill beams are typically either 24'-9" long W10x19s or 35'-4" long W16x36s. The framing at the penthouse roof is at a forty-five degree angle, the same direction as that in the "center" framing area of the typical floors. For the purpose of analyzing alternate floor systems, the roof will not be redesigned. I am assuming that the roof framing will utilize the same system as is selected for the floor framing.

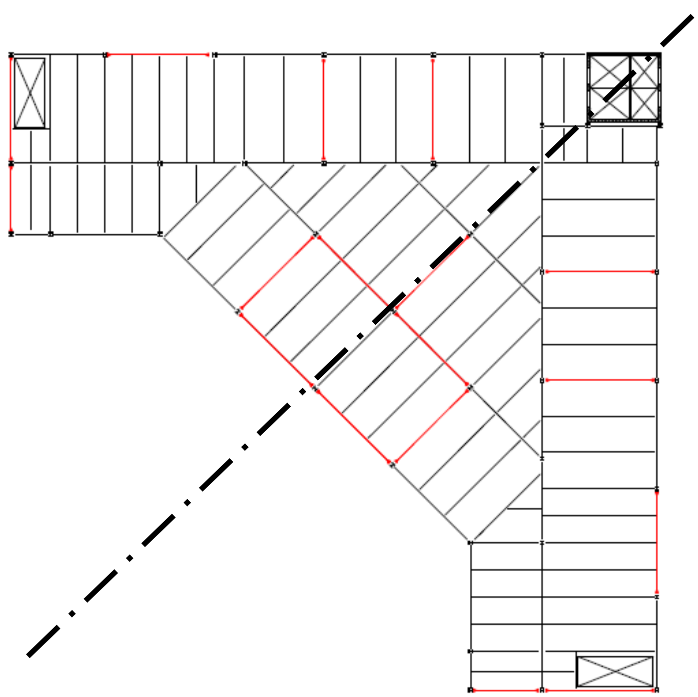
Exterior:

The exterior of the building is typically precast, metal and glass panels. The precast panels are 8" thick. At the first floor on the east side of the building, a curtain wall system is used similar to the curtain wall used on the existing hospital. The only variation to the precast, metal, and glass striping pattern is that the 39.5' true south and true north walls are made up of almost exclusively precast with a few punched out windows.

The walls that extend from the penthouse floor to the penthouse roof are composed of 6" metal studs at 16" on center with insulation. These walls have an exterior finish of "dryvit" on them for protection and aesthetics.

Lateral Load Resisting System:

In the existing system, steel moment frames were used at each level to resist lateral loads. Each floor contains 19 moment frames, 8 of which are along the perimeter of the building and 11 are interior beams. The moment frames are symmetrical about the same diagonal axis that the building is. These lateral force-resisting beams are highlighted in red in the diagram below with the axis of symmetry shown as a dashed line.



Moment frames were used to allow for floor plan flexibility. With the hospital constantly growing and the changing demands various branches (i.e. surgery, physical therapy, rehabilitation, etc.), the space initially designed for patient rooms could have an alternate use sometime down the road. If trusses or braced frames were used, the location of these braces would reduce the flexibility of the space.

The lateral system will not be redesigned or analyzed in this assignment, but it is important to note that changes may occur based on the different floor systems. For instance, the concrete systems analyzed will obviously not be able to utilize steel moment frames so the lateral system will change as well. This will be further investigated at a later time once a new system is decided on.

Foundation System:

Five soil test borings were taken at the site of the new patient tower. They were drilled to a depth of about 30 feet each according to ASTM D 1586 standards. It was found that the top layer of soil was fill soil consisting of sand and silt, but the basement floor elevation should generally fall below this layer of soil. Therefore, a new allowable bearing pressure of 6,000 psf was used to design the foundations.

The footing sizes of the main addition vary from 8 foot by 8 foot to 11 foot by 11 foot square footings along with a few rectangular footings. Smaller 4 and 5 foot square footings occur at columns located in the one-story extension to the north of the main tower. Along the north wall of the building, there is an existing retaining wall footing. This footing is to be field verified and any portions that interfere with the new footings are to be removed.

A 14” thick concrete foundation wall surrounds that building at the basement level. The wall is reinforced with #4 bars at 12” vertical on each face and #5 bars at 12” horizontal. Concrete piers protrude from the wall at the location of exterior columns from which steel columns extend from the first floor up.

The slab on grade is 5” thick reinforced with 6x6” WWF on a vapor retarder over a minimum 4” layer of clean, well graded gravel or crushed stone. There is a small area, approximately 20 by 40 feet, where the top of slab elevation is depressed one foot.

The foundation system may be affected by the alternate floor systems. If the building weight increases, the footing sizes will most likely increase and vice versa. More mention of this will be made later on when each individual system is analyzed.

IV. CODES

Codes and Standards:

Rathgeber/Goss Associates designed the Howard County General Hospital patient tower, which began design in 2004, according to the 2000 International Building Code and ASCE 7-98. Concrete design specifically references ACI 318-99 while steel design followed the AISC Load and Resistance Factor Design, Third Edition 2001.

My report will utilize the more recent versions of the building codes, the 2006 International Building Code, which references ASCE 7-05. For concrete analysis and design, I will be using ACI 318-05 and for steel design, I will be using the Load and Resistance Factor Design portion of the LRFD and ASD Combined AISC Thirteenth Edition Steel Manual, Copyright 2006.

For design of the flat plate and flat slab concrete systems, I will be utilizing the 2002 version of the CRSI Design Handbook. This version of the CRSI Handbook references ACI 318-99.

The Vulcraft steel joist and steel deck catalogs are being used for design of the open web steel joist system. For the hollow core planks, the Nitterhouse Concrete design tables were used. These sources will be referenced later in the report where the design tables are included. All catalogs and design tables are also available online.

As noted in Appendix F, RS Means 2006 Assemblies Cost Data was used to determine the floor systems' costs. All tables that were used are included in the Appendix. The cost per square foot is based solely on the cost of materials and installation for each floor assembly.

V. LOADS

Dead Loads:

The floor dead load will vary for each system that is analyzed. This load will be broken down and calculated for each analyzed floor system in a later section. This is the case for the roof dead load as well, though the roof will not be redesigned.

I am assuming that the building's exterior will remain the same, regardless of the change in floor system. From the existing design, the exterior dead load at the building perimeter is mainly the precast panel dead load listed below. The only exception is on the east side of the tower at the first floor, were the curtain wall system is present. A 10 psf dead load was assumed for the glass and aluminum panels.

Exterior Wall Dead Loads

Precast Panels (8" thick) .... 0.10 ksf
150 pcf*(8"/12) = 100 psf = 0.10 ksf
Glass/Aluminum .... 0.01 ksf
Curtain Wall (18' tall) .... 0.36 klf
Metal Stud Wall @ 16" oc .... 0.015 klf

Live Loads:

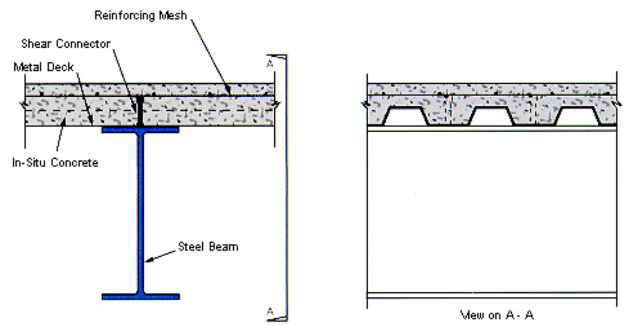
Most of the design live loads were included on the structural general notes and were verified with the newer code, ASCE 7-05. The live loads for storage areas and the roof, which were not listed in the structural general notes, were taken from chapter 4 of ASCE 7-05. A live load of 100 was used for the entire typical hospital load, though not required, for future flexibility reasons.

Location	Load	Comments
Framed Floor Areas	100 psf	80LL + 20 for Partitions
Lobbies/Stairs	100 psf	
Storage	125 psf	Unreducible
Penthouse	125 psf	Unreducible
Roof	30 psf	Unreducible

VI. FLOOR SYSTEM 1: EXISTING COMPOSITE SYSTEM

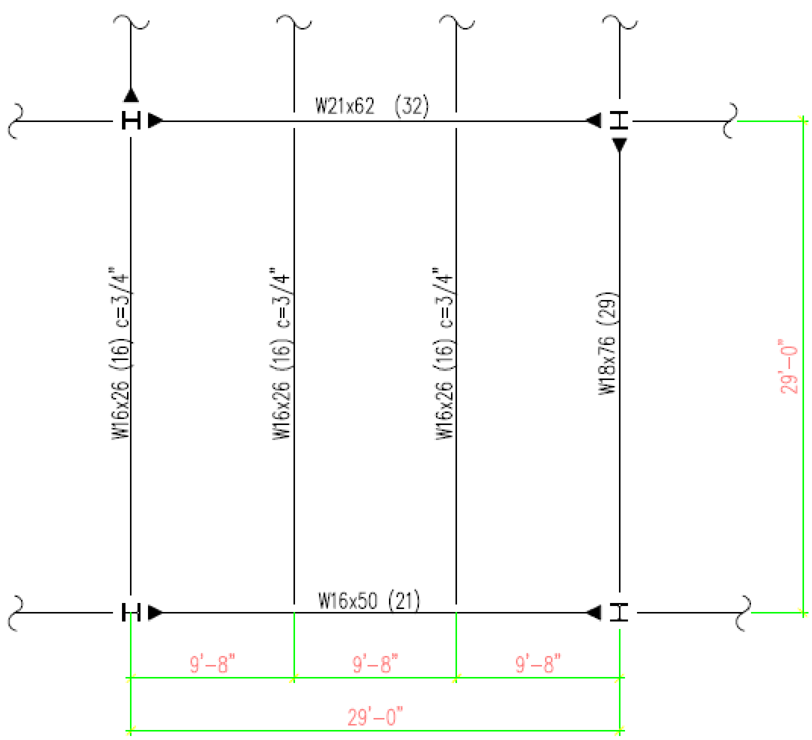
Material Properties:

3 ¼" lightweight concrete fill on metal deck
f'_c = 3500 psi
w = 110 pcf
2" 18 gage composite metal deck
¾" diameter by 4" shear studs
A992 Steel for Wide Flange Beams
f_y = 50 ksi



Typical Bay Framing:

A typical bay consists of 29' by 29' column bays with typical W16x26 infill beams and varying girder sizes. The designer's sizes were checked and considered to be adequate. Calculations can be found in Appendix A. An example of a typical bay can be seen below.



Advantages:

Composite concrete on metal deck is a good solution for the larger live loads such as those required in the hospital. Composite action allows for the reduction of beam sizes in comparison to a standard non-composite steel beam system. The 29-foot column bays are easily accomplished with manageably sized steel sections. This system is the basis to which each of the other systems will be compared.

Because of the metal deck, formwork is not required for the concrete slab. Also, shoring is not required with use of the selected deck. This greatly simplifies the construction process. In addition, a steel framed building skeleton can be erected quickly and efficiently.

The composite slab system achieves the required 2-hour fire rating without any additional fire proofing to the slab, though the beams do require spray-on fireproofing. Also, the system provides a good acoustical barrier and reduces any vibratory issues in comparison to some other systems, like open web steel joists.

Obviously, this system allows for the current architectural layout, which I am considering to be ideal from the architect's viewpoint considering it is the existing design. There may be some changes in layout or column grid for some of the new systems being investigated.

Disadvantages:

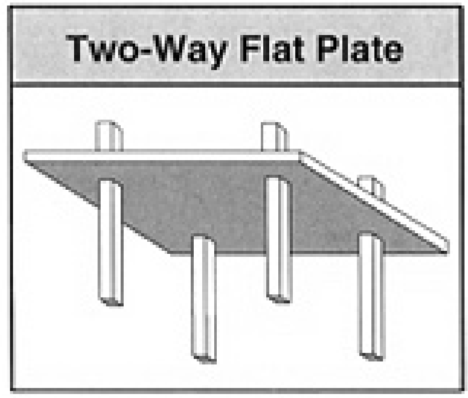
Even though compared to a non-composite steel system, the beam sections are reduced, in comparison to a concrete flat plate or flat slab system, the overall thickness is greater with the composite steel system. In some cases, W24 girders are required. Combined with the 5 ¼" composite slab, the maximum total thickness of this system is over 29" without any sort of finishes. In order to achieve the large floor-to-ceiling heights with a structural sandwich of 2 ½ feet plus, the overall building height is taller, which adds cost in a variety of ways including additional exterior materials and longer duct/piping runs. A thinner slab could potentially reduce costs.

Though concrete for the slab is readily available, the steel sections will require a longer lead-time than a fully concrete system. This could slow down the construction schedule, and though this project is not on the fast track, time is money in this business.

Where the slab depression was required for the stall-less showers, the depressed area had to be framed out with steel beams. Considering each typical floor has 30 patient rooms and hence 30 shower areas, this is a significant amount of additional steel sections, additional labor, and therefore additional cost. Concrete systems allow slab depressions to be achieved in a much simpler, less costly way.

VII. FLOOR SYSTEM 2: TWO WAY CONCRETE FLAT PLATE

Description:

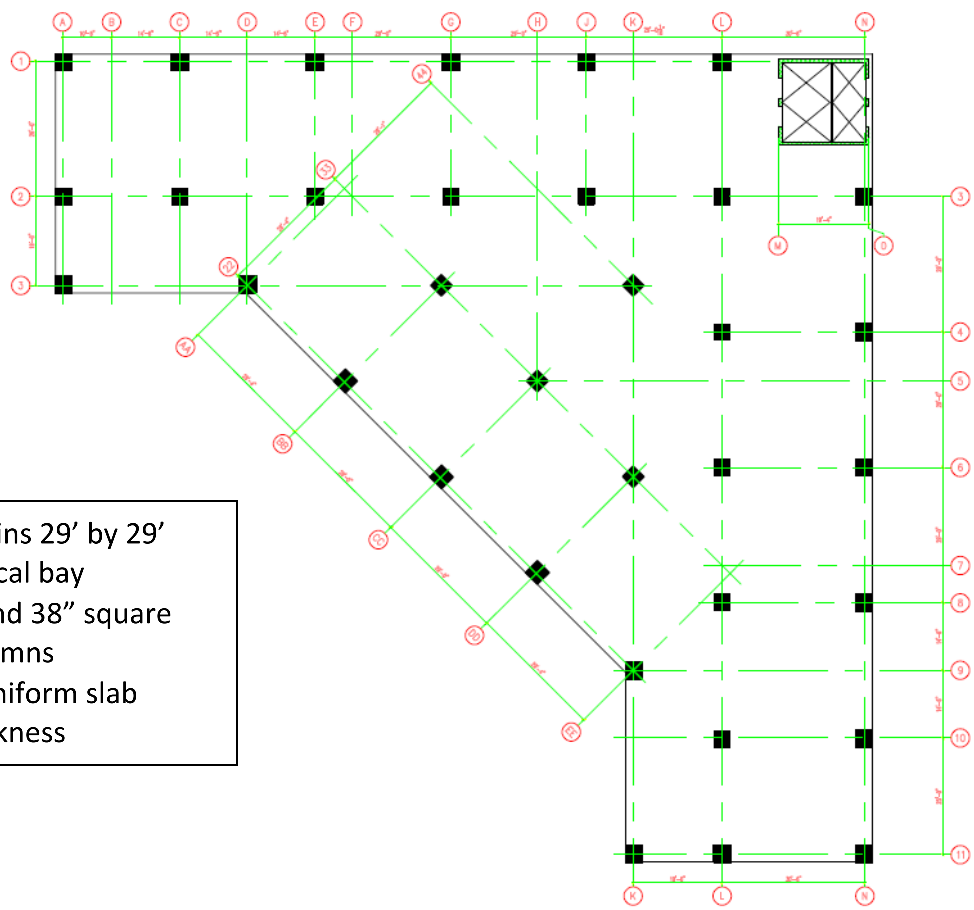


The first floor system to be analyzed was a concrete flat plate system. This consists of a reinforced slab of uniform thickness spanning between concrete columns. The following material properties were assumed:

F'_c = 4000 psi
w_{conc} = 150 pcf
F_y (reinforcing) = 60 ksi

For simplicity, Chapter 9 of the CRSI Handbook was used to design the slab thickness, column size, and required reinforcing. See Appendix B for tables and calculations.

Floor Plan:



- Remains 29' by 29' typical bay
- 42" and 38" square columns
- 10" uniform slab thickness

Advantages:

The flat plate floor system, shown in the diagram above, is primarily used in taller residential structures or hotels. The advantages and disadvantages differ greatly from that of the existing design of a composite slab on steel beams.

This floor system will allow for a decreased overall floor thickness. Also, the uniform concrete slab would eliminate the need for a drop ceiling or finishing unless desired. With a thinner slab, even larger floor to ceiling heights could be accomplished or, if they remain the same, the overall building height could be decreased. A decreased overall height would allow for savings in various areas including less exterior materials and shorter vertical runs of ducts, piping, etc.

In comparison to other concrete systems, the flat plate system requires simple formwork, which simplifies construction. Also, the slab depressions required for the prefabricated stall-less showers is much easier to accomplish in a concrete system rather than a steel system. It would eliminate the need for beams to frame out each of the 30 depressions on each floor.

The concrete slab easily accomplishes the required 2-hour fire rating. Vibration and acoustical performance should also not be an issue due to the increase in mass and stiffness.

Finally, concrete is readily available and does not require the same manufacturing process as steel members do. This would decrease the lead-time and project schedule, which could potentially reduce the overall construction time.

Disadvantages:

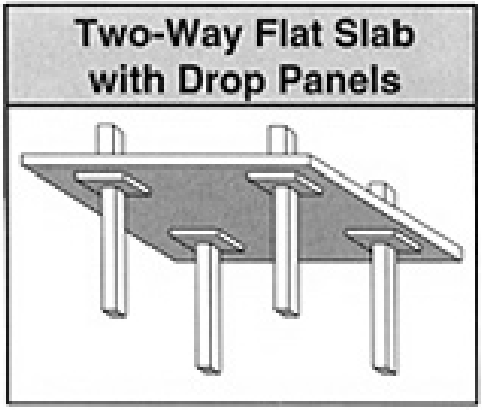
Though the advantages of a concrete flat plate system are significant, there are also several disadvantages that are important to identify. Compared to the existing lightweight concrete slab on metal deck, a normal weight concrete slab will be much heavier. This will increase the building weight and could have a fairly significant impact on the foundation system.

In order to maintain the large column bays, the flat plate system is required to span fairly large distances. This requires a thicker, and therefore heavier concrete slab and also very large column sections to prevent punching shear. The larger column sections could infringe on the architectural layout of the building and also add even more building weight. Also, a few column locations had to be moved from the initial layout in order to achieve two-way action. This could also affect the building's layout and would have to be further investigated.

In terms of construction, though the flat plate is simple for a concrete floor system, the concrete still needs appropriate time and weather to cure. This inconvenience is eliminated with steel systems, like the existing one.

VIII. FLOOR SYSTEM 3: TWO WAY CONCRETE FLAT SLAB

Description:

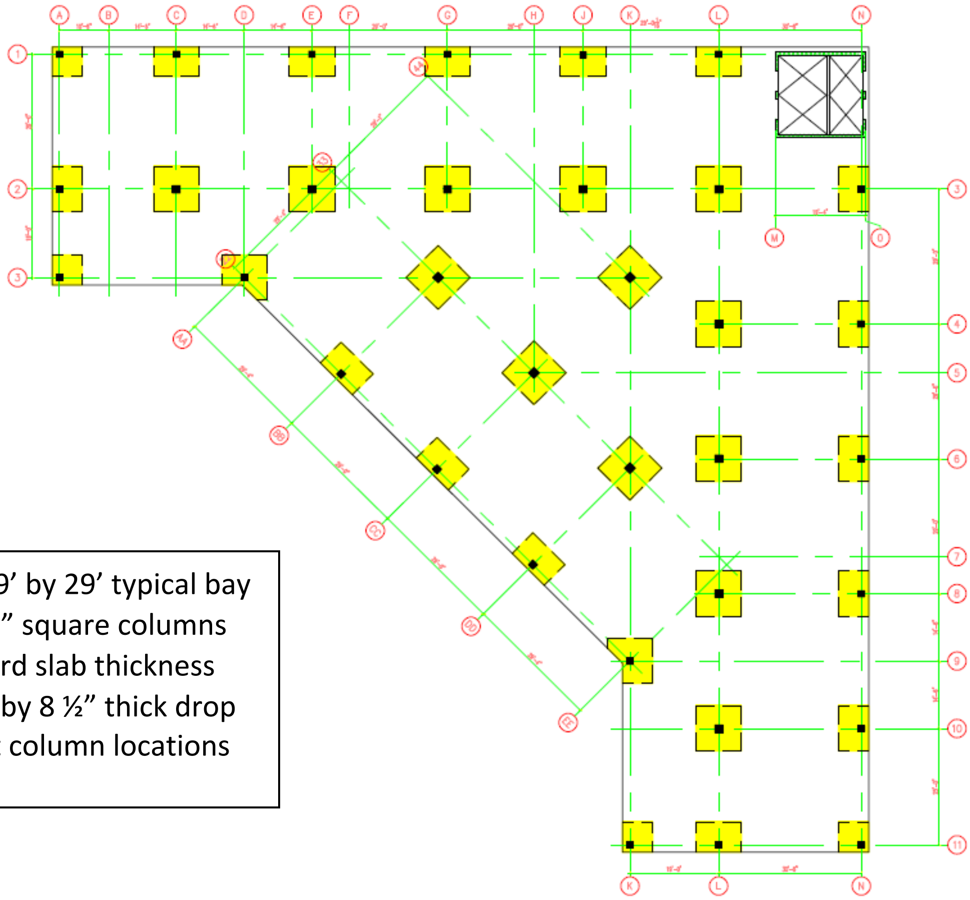


The two-way concrete flat slab system is similar to the flat plate as it consists of a concrete slab, reinforced in both directions, spanning between concrete columns. However, at the column locations, drop panels are added. The following material properties were assumed:

F'_c = 4000 psi
W_{conc} = 150 pcf
F_y (reinforcing) = 60 ksi

Chapter 10 of the CRSI Handbook was used to design the slab thickness, drop panel size, column size, and slab reinforcing. See Appendix C for tables and calculations.

Floor Plan:



- Remains 29' by 29' typical bay
- 19" and 16" square columns
- 10" standard slab thickness
- 9'-8" wide by 8 ½" thick drop panels at column locations

Advantages:

The flat slab system has some advantages that are inherent to concrete and therefore similar to those of the flat plate system. This includes the fact that concrete is more readily available and requires less lead-time than steel. Also, the slab thickness would be decreased in comparison to the composite steel system, allowing for the reduction of the overall building height or increasing the floor to ceiling heights. Finally, the slab depressions for the showers would be easily accomplished with concrete as opposed to steel.

In comparison to the flat plate system, the column sections required for the flat slab system are much smaller. The drop panels at the columns allow for the smaller sections because punching shear is less critical. Though still larger than the steel column sections, the concrete column sizes required are much more reasonable and would be less likely to infringe on the architectural layout. Also, the amount of reinforcing needed in the slab is less than in the flat plate.

Once again, fire protection, vibration, and acoustical performance are not a problem. The concrete slab provides the required 2-hour rating and the increase in mass and stiffness relieves the building of any vibratory or acoustical issues.

Disadvantages:

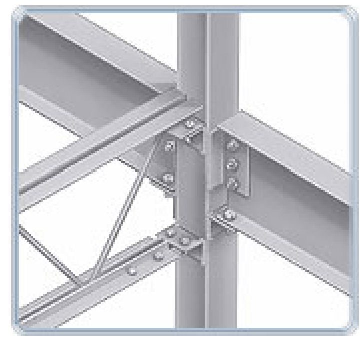
Similarly to the flat plate system, the building mass would increase by switching from lightweight concrete on metal deck to a normal weight concrete floor system. This could impact the foundation system and require much larger footings. Also, unforeseen construction issues are more likely given that the concrete requires time to cure.

Unlike the flat plate system, the flat slab system could require some sort of finished ceiling. The drop panels may not create the desired aesthetics and the architect may not want them exposed. This could require a solution such as a drop ceiling. Also, the drop panels pose a more difficult constructability issue than the flat plate system including formwork. This adds to labor and formwork costs.

Finally, the flat slab system requires some slight changes to the column layout just like the flat plate system does for framing simplicity, even though the columns aren't nearly as large. This may cause some changes in the architectural layout of the spaces or some inconvenient column locations, which could be undesirable for the architect.

IX. FLOOR SYSTEM 4: OPEN WEB STEEL JOISTS

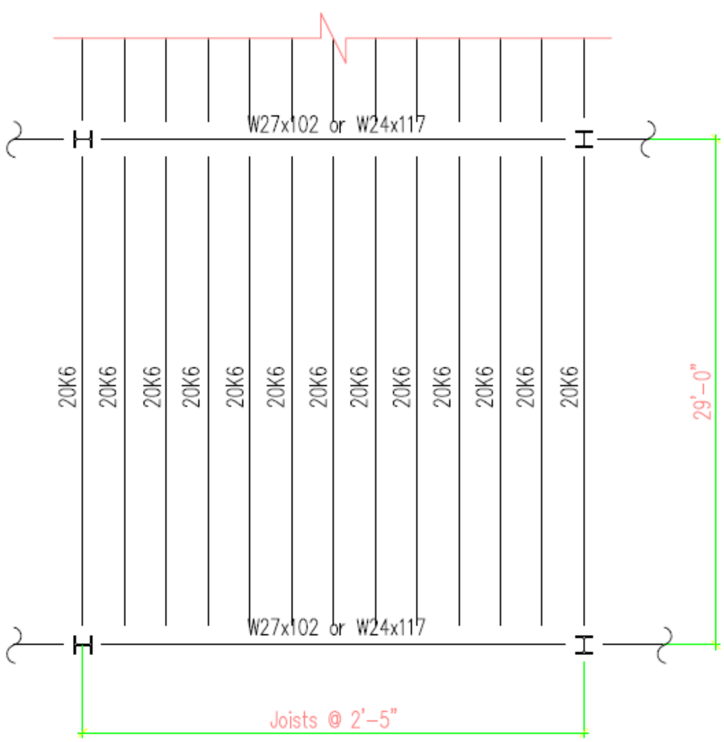
Description:



Open web steel joists are a light floor system that is most economical for regular bays where members can be mass-produced. The joists can support a variety of floor slabs, composite or non-composite. I chose to use a non-composite lightweight concrete slab on form deck with joists spaced at 2'-5" (to divide evenly into the typical 29 foot bay). Though composite systems allow you to space the joists further apart, with the hospital's large live load, increased spacing would greatly increase the depth, weight, and cost of the joists.

Typical Bay:

Use 20K6 Joists for the 29 foot span spaced at 2'-5" on center. The shorter 19-foot span (not shown below) requires 14K3 joists. A typical girder was sized to be a W27x102 or W24x117. These were sized for point loads from the long joists on both sides, so other girders (like those with the shorter joists framing into them on one side) would be smaller.



Advantages:

As mentioned above, open web steel joists are an inexpensive floor framing system to install. In addition, the thin concrete slab on light steel joists would reduce the building weight and could therefore decrease some of the foundation sizes, reducing the overall building cost even more.

Also, the building has a fairly regular grid, and joists could most likely be mass-produced in about 3 regular sizes, with the exception of special conditions. The joists will be supported by steel girders, similar to those in the composite system, so no changes to the current column grid are required.

The open webs of the joists allow for any mechanical ductwork and piping to be run through the structural framing. Therefore, no additional room is required for MEP runs, unlike in a concrete slab system.

In comparison to steel W-shapes, the joists are much lighter weight and easier to handle. They can be placed with less manpower and less crane costs than the composite system. All of this contributes to the efficient, quick, and simple constructability of an open web steel joists system.

Disadvantages:

Open web steel joists are very light, which reduces the building weight, but also increases the likelihood of vibrations. Also, such a light building could be more vulnerable to wind loads.

Also, the joists require fireproofing to accomplish the required 2-hour rating, unlike many of the other systems that inherently provide the rating. This process could include using a fireproofing spray, which in many cases does not easily adhere to the joists. They may need to be wrapped in chicken wire before being sprayed, which increases labor, time, and cost.

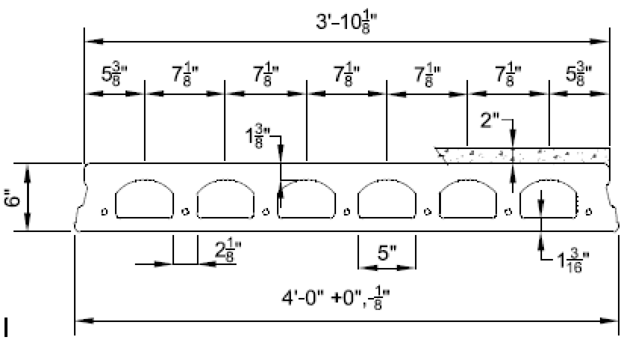
Similarly to the composite steel system, the 2" slab depression required for the stall-less showers would be much more difficult to accomplish with the joist system than a concrete system. I believe that it would require the openings to be framed out in steel beams with short joists spanning between them. This will increase the number of members and the cost of labor.

Finally, by switching to a non-composite system, the girder sizes are increased. In many cases the same depth girders can be used, so the floor thickness would not be altered much, but the weights of the girders would increase. These cross sections are harder to manage/place and cost more.

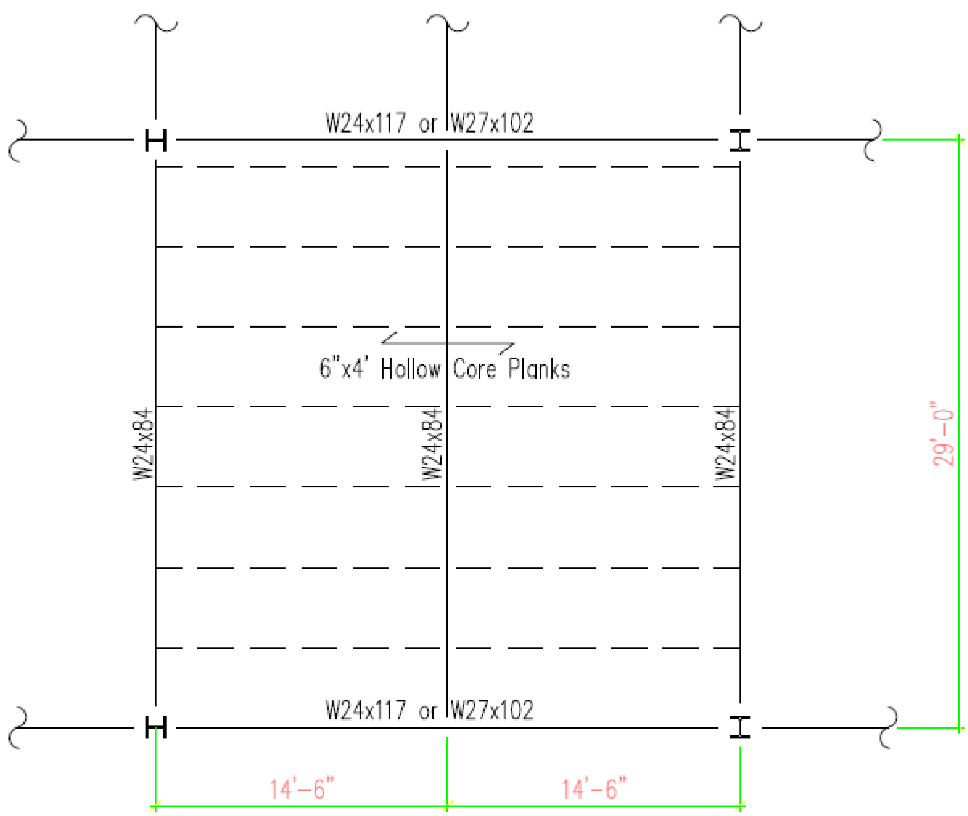
X. FLOOR SYSTEM 5: HOLLOW CORE PRECAST PLANKS

Description:

Hollow Core Precast Planks are very durable and simple to install. They can bear on a variety of structural members including steel girders (such as in this case), precast girders, and bearing walls. For this design, the planks will span 14'-6" as each typical column bay will receive one infill beam. The beam design can be found in the Appendix. With this span, I was able to select a 6" hollow core member with a 2" concrete topping.



Typical Bay:



Advantages:

The hollow core planks with the 2” topping achieve a 2-hour fire rating, so no additional fireproofing is required for the slab. The beams supporting the planks, however, will require spray-on fire proofing for protection.

The planks are prefabricated and allow for quick and easy erection. The steel skeleton is also easy to construct and the construction schedule, if planned properly, could allow for very efficient construction.

The planks minimize deflection and limit any vibratory issues. They also minimize sound attenuation. Compared to other systems, the planks are relatively cost effective and will most likely provide long-term durability. Since steel beams and girders will be used to support the planks, the same column grid can be used. Also, because the planks can span further than the composite slab, the spacing of infill beams is increased from 9’-8” to 14’-6”, which decreases the number of steel sections required.

The system with 2” topping was chosen because I figured that the areas the stall-less showers could be formed, and the topping could be omitted in these areas. This would allow for the necessary slab depression but would not affect the structural integrity of the floor.

Disadvantages:

Though the planks are hollow, the overall dead weight of the building is still greater than that of the existing composite steel system. This will have a variety of negative effects on cost and the building’s seismic performance.

Though the current column grid will be maintained, the varying framing directions will require many of the planks to be cut. This requires additional labor and inherently additional costs. Also, the planks come in 4-foot sections and the typical column bay is 29 feet, which is not divisible by 4. Therefore, even more planks will require cutting.

The planks require W24x84 and W24x68 beams girders, which are comparable to the composite steel system’s larger members. However, with the 6” planks and 2” topping in addition to the beams, the total floor thickness for structure only will be approximately 32”, which is several inches thicker than the existing system.

XI. OVERALL COMPARISON

	Floor System 1	Floor System 2	Floor System 3	Floor System 4	Floor System 5
	Existing Composite Steel	**Two Way Flat Plate**	**Two Way Flat Slab**	**Open Web Steel Joists**	**Hollow Core Precast Planks**
Weight (slab weight only)	41 psf	125 psf	137 psf (equivalent load including drops)	27 psf	74 psf
Slab Depth (inches)	5.25"	10"	10" + 8.5" drop	3"	6" + 2" topping
Total Maximum Floor Depth (inches)	24" max girder + 5.25" slab = approx 30"'	10"	18.5"	27" max girder + 3" slab = approx 30"'	24" max girder + 8" plank/topping = approx 32"'
Approximate Cost of Floor System	$17.80/SF	$13.85/SF	$16.00/SF	$16.85/SF	$17.40/SF
Construction Difficulty	Easy/Moderate	Easy	Moderate	Easy	Easy
Lead Time	Weeks	None - Concrete Readily Available	None - Concrete Readily Available	Weeks	Weeks
Column Grid Changes	n/a	Slight	Slight	None	None
Fire Protection Issues	Spray Beams and Girders	None	None	Requires wrap and spray to achieve 2 hr rating	Spray Girders
Architectural Layout Issues	Yes - Difficult to Achieve Slab Depression	Yes - Very Large Concrete Columns (approx 42”)	Maybe - Slight Increase in Column Sizes	Yes - Difficult to Achieve Slab Depression	Maybe - Possible Issues with Slab Depression

Based on this comparison, I have concluded that all floor systems except the flat plate seem feasible. The flat plate system requires too large of columns that would infringe on the existing layout of the spaces.

In terms of structural system depth, the existing composite system, open web steel joist system, and hollow core precast panel system all provide comparable total floor depths. The flat slab system is shallower, however duct work and pipes could not be accounted for within the structural depth like some of the other systems.

The four feasible systems are within about $2 of each other per square foot, though this could add up to a somewhat significant difference over the whole building. Also, the impacts on other systems will alter the costs, which was mentioned in the previous sections.

Though the open web steel joists are a possible option, I do not find them ideal for this building. With most of the floor area being comprised of patient rooms, and the use of medical equipment, sound attenuation could prove to be a problem. Also, because of fireproofing and the required slab depressions, overall construction is inconvenienced (though the joists themselves are easy to install).

With eliminating these two systems, I think the flat slab and hollow core precast panel systems are both worth more consideration. The flat slab would require a switch from steel to concrete columns, but they are reasonably sized and could be painted/exposed eliminating the need to "box out" the columns such as in the existing steel system. This is also the cheapest system according to the preliminary cost estimates. The hollow core precast panels seem to be a viable solution as well since they are easy to install and would most likely maintain the same column grid and similar column sizes. The weight of this system is less than the flat slab, so it would be less likely to impact the foundation. I do however I feel that the existing composite system was proven to be an appropriate choice.

APPENDIX A – FLOOR SYSTEM 1: EXISTING COMPOSITE SYSTEM

Deck Capacity:

From the structural general notes and plans, I concluded that the floor system was designed with 2" 18 gage Lok-Floor Deck by USD with 3 ¼" lightweight concrete for a total thickness of 5 ¼". The following table came from the USD website:

18 gage	4.50	62.08	32.6	0.292	34	1.53	5.4	42.99	4560	9.20	11.33	11.71	0.023
	5.00	72.04	37.5	0.333	38	1.81	7.3	50.72	5240	8.75	10.84	11.20	0.027
	5.25	77.02	40.0	0.354	41	1.95	8.3	54.72	5590	8.54	10.62	10.97	0.029
	5.50	82.00	42.6	0.375	43	2.10	9.5	58.78	5950	8.35	10.41	10.76	0.032
	6.00	91.95	48.0	0.417	48	2.39	12.1	67.07	6530	8.01	10.02	10.36	0.036
	6.25	96.93	50.8	0.438	50	2.54	13.6	71.29	6730	7.86	9.84	10.17	0.038
	6.50	101.91	53.6	0.458	53	2.69	15.2	75.55	6920	7.71	9.68	10.00	0.041
	7.00	111.87	59.5	0.500	58	3.00	18.8	84.17	7340	7.44	9.36	9.67	0.045
	7.25	116.85	61.9	0.521	60	3.16	20.7	88.52	7500	7.32	9.21	9.52	0.047
	7.50	121.83	64.3	0.542	62	3.31	22.8	92.91	7670	7.24	9.07	9.38	0.050

The red box highlights the maximum unshored spans, in feet, for 1, 2, or 3 span conditions. In the tower framing system, with a 5 ¼" total slab depth (far left column), the typical centerline to centerline beam spacing is 9.67'. You can see that the deck can span 10.62' and 10.97' for 2 or 3 span conditions, respectively. The one span condition does not apply to this building, so the deck fits this requirement.

Next, one must verify whether the composite deck can carry the floor loads. The typical hospital loading is a dead load of 65 psf and a live load of 100 psf (see Loads section). The deck tables are concerned only with live load, as you can see in this table:

	L, Uniform Live Loads, psf *														
	Slab Depth	φMn in.k	6.00	6.50	7.00	7.50	8.00	8.50	9.00	9.50	10.00	10.50	11.00	11.50	12.00
18 gage	4.50	62.08	400	400	400	400	375	330	290	260	230	205	180	155	135
	5.00	72.04	400	400	400	400	400	385	340	300	270	240	220	195	180
	5.25	77.02	400	400	400	400	400	400	365	325	290	260	235	210	190
	5.50	82.00	400	400	400	400	400	400	390	345	305	275	250	225	205
	6.00	91.95	400	400	400	400	400	400	400	385	345	310	280	250	230
	6.25	96.93	400	400	400	400	400	400	400	400	365	325	295	265	240
	6.50	101.91	400	400	400	400	400	400	400	400	385	345	310	280	255
	7.00	111.87	400	400	400	400	400	400	400	400	400	380	340	310	280

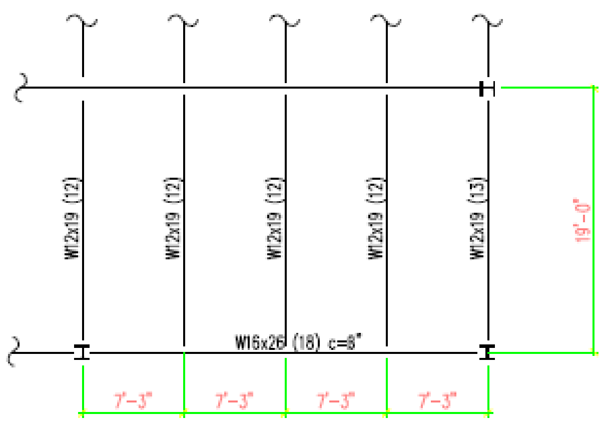
Rounding the 9.67' span up to 10.00', for 5 ¼" total thickness the capacity is 290 psf, which is much greater than the 100 psf live load for the typical office condition, or even the 125 psf live load for storage areas and the penthouse.

Typical Composite Beams:

There are three main infill beam sizes, a W12x19, W14x22, and W16x26. These beams are used in different locations based on span and spacing.

It must be verified that each of these beams are designed properly for the necessary loads which requires checking bending, number of shear studs, and deflection of the composite slab.

(A) W12x19 with 12 shear studs



CASE A

w_u = (1.2*65 + 1.6*100)*7.25' = 1726 lb/ft = 1.726 k/ft
M_u = (1.726*19^2)/8 = 77.9 ft-k
V_u = (1.726*19)/2 = 16.4 k

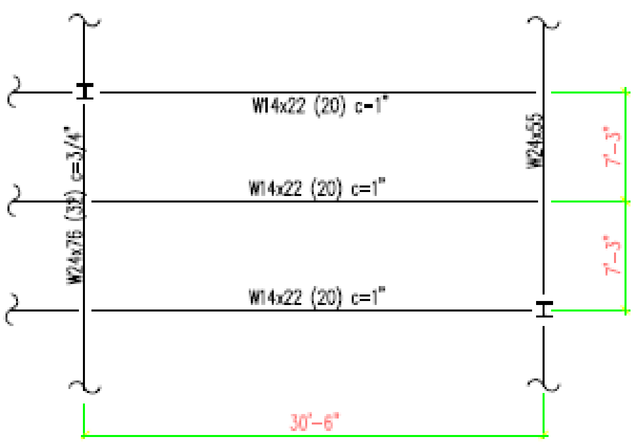
- Check Bending:
  Assume PNA at location 6 to reduce number of shear studs because available strength seems to be >> required strength
  $\sum Q_n$ = 104 k @ PNA 6 from Table 3-19
  b_{eff} = minimum of: ¼ span = ¼ *19 = $\boxed{4.75 \text{ ft}}$
  spacing = 7.25 ft

$a = \sum Q_n/(0.85*f'c*b_{eff}) = 104/(0.85*3.5*4.75*12) = 0.61''$
Y2 = 5.25" – 0.61"/2 = 4.95"
Use Y2 = 4.5" because rounding down is conservative
ϕMn = 162 ft-k > 77.9 ft-k, therefore **OK**

- Check Number of Shear Studs:
  Assume $\sum Q_n$ = 17.2 k from Table 3-21 (LW concrete, ¾" diameter studs, 1 stud/rib)
  # of studs required = (104/17.2)*2 = 12.1
  12 studs provided, therefore **OK**

- Check Deflection:
  Use Y2 = 4.5" because rounding down is conservative
  I_{LB} = 300 in^3 from Table 3-20
  $\Delta_{max} = (5*1.726*19^4*1728)/(384*29000*300) = 0.582''$
  0.582" = L/392 < L/240 = 0.95", therefore **OK**

(B) W14x22 with 20 shear studs and 1" camber



CASE B

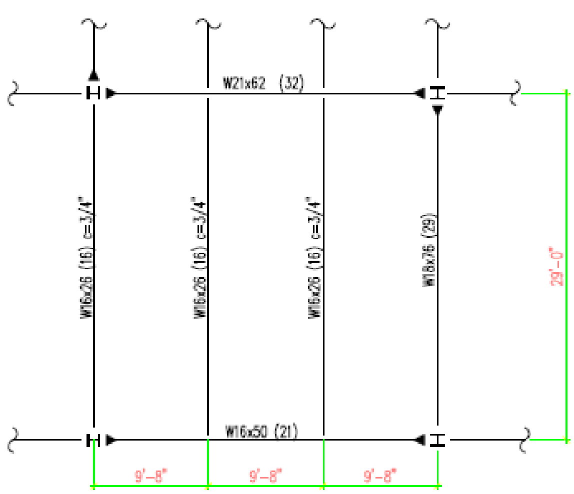
w_u = (1.2*65 + 1.6*100)*7.25' = 1726 lb/ft = 1.726 k/ft
$M_u = (1.726*30.5^2)/8$ = 200.7 ft-k
V_u = (1.726*30.5)/2 = 26.3 k

- Check Bending:
  Assume Y2 = 4.5”, which requires PNA @ BFL
  $\sum Q_n$ = 157 k @ BFL from Table 3-19
  b_{eff} = minimum of: ¼ span = ¼ *30.5 = 7.63 ft
  spacing = $\boxed{7.25\text{ ft}}$
  $a = \sum Q_n/(0.85*f'c*b_{eff})$ = 157/(0.85*3.5*7.25*12) = 0.61”
  Y2 = 5.25” – 0.61”/2 = 4.95”
  Use Y2 = 4.5” because rounding down is conservative
  ϕMn = 218 ft-k > 200.7 ft-k, therefore **OK**

- Check Number of Shear Studs:
  Assume $\sum Q_n$ = 17.2 k from Table 3-21 (LW concrete, ¾” diameter studs, 1 stud/rib)
  # of studs required = (157/17.2)*2 = 18.3
  20 studs provided, therefore **OK**

- Check Deflection:
  Use Y2 = 4.5” because rounding down is conservative
  I_{LB} = 473 in^3 from Table 3-20
  $\Delta_{max} = (5*1.726*30.5^4*1728)/(384*29000*473)$ = 2.45”
  2.45” – 1” camber = 1.45” = L/252 < L/240 = 1.525”, therefore **OK**

(C) W16x26 with 20 shear studs and ¾” camber



CASE C

w_u = (1.2*65 + 1.6*100)*9.67' = 2301 lb/ft = 2.301 k/ft
M_u = (2.301*30.5^2)/8 = 267.6 ft-k
V_u = (2.301*30.5)/2 = 35.1 k

- Check Bending:
  Assume Y2 = 4.5", which requires PNA @ BFL
  $\sum Q_n$ = 194 k @ BFL from Table 3-19
  b_{eff} = minimum of: ¼ span = ¼ *30.5 = $\boxed{\text{7.63 ft}}$
  spacing = 9.67 ft
  a = $\sum Q_n$/(0.85*f'c*b_{eff}) = 194/(0.85*3.5*7.63*12) = 0.71"
  Y2 = 5.25" – 0.53"/2 = 4.89"
  Use Y2 = 4.5" because rounding down is conservative
  ϕMn = 289 ft-k > 267.6 ft-k, therefore **OK**

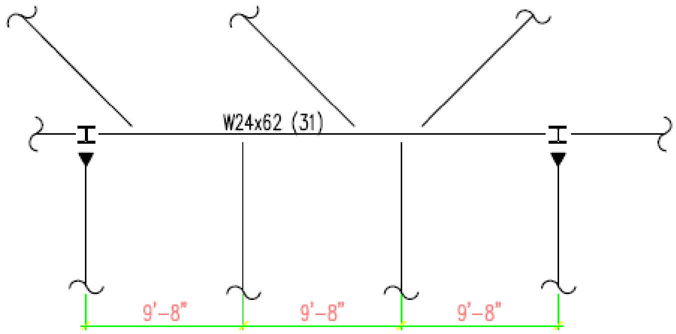
- Check Number of Shear Studs:
  Assume $\sum Q_n$ = 17.2 k from Table 3-21 (LW concrete, ¾" diameter studs, 1 stud/rib)
  # of studs required = (194/17.2)*2 = 22.6
  20 studs provided, therefore **NOT OK**, however they are close and I could have made different assumptions than the structural engineer

- Check Deflection:
  Use Y2 = 4.5"
  I_{LB} = 694 in^3 from Table 3-20
  Δ_{max} = (5*2.301*30.5^4*1728)/(384*29000*694) = 2.23"
  2.23" – ¾" camber = 1.48" = L/247 < L/240 = 1.525", therefore **OK**

Typical Composite Girder:

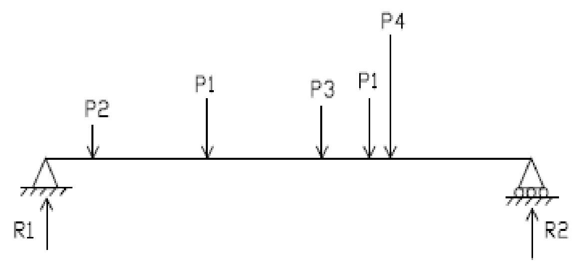
The girder sizes in the typical floor plan vary somewhat considerably between W16s and W27s. This variation becomes even greater on the upper floors where the slab is depressed for the shower stalls. However, the most typical sizes seem to be W21s and W24s with smaller sizes occurring only at short spans and larger sizes occurring only at unusually long spans.

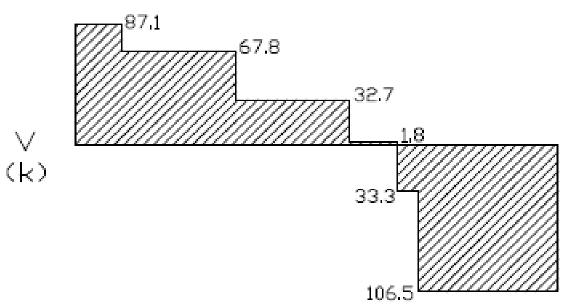
The points at which the girders are loaded vary because of the change in framing direction due to the building's unusual shape. I performed calculations on an interior 29-foot W24x62 that seemed to be a median sized girder with the typical condition of 3 beams framing into it on each side. The framing on each side of the girder at this location are in opposite directions, which is a fairly common condition for the interior girders in this building.

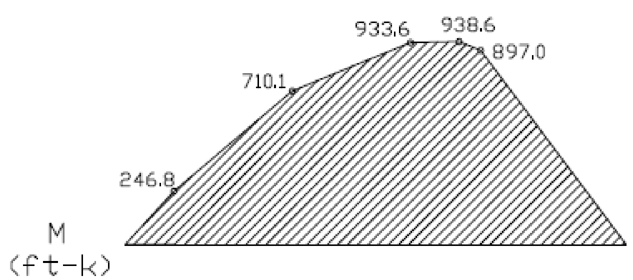


P_1 = 35.1 k (from typical W16x26 beam above)
P_2 = 2.301*16.75'/2 = 19.3 k
P_3 = 2.301*26.83'/2 = 30.9 k
P_4 = [(1.2*65+1.6*100)*46'/2]*26.75'/2 = 73.2 k

R_1 = 19.3*2.83'+35.1*(9.67'+19.3')+
30.9*16.5'+73.2*20.58']/29' = 106.5 k
R_2 = 193.6 k – 106.5 k = 87.1 k







V_{max} = 106.5 k M_{max} = 938.6 ft-k

Assume Y2 = 4.0", which requires PNA @ BFL
$\sum Q_n$ = 496 k @ BFL from Table 3-19
b_{eff} = minimum of: ¼ span = ¼ *19 = $\boxed{7.25 \text{ ft}}$
spacing of girders >> ¼ span
a = $\sum Q_n/(0.85*f'c*b_{eff})$ = 496/(0.85*3.5*7.25*12) = 1.92"
Y2 = 5.25" – 1.92"/2 = 4.29"
Use Y2 = 4.0" because rounding down is conservative
ϕMn = ft-k > 938.6 ft-k, therefore **OK**

Typical Column:

The column sizes used in the addition are a variety of wide flange shapes ranging in size from W12x40 to W14x159. The typical column bay is 29 feet by 29 feet, for a typical total tributary area of 841 square feet per column per floor. The live loads at the penthouse and roof are not reducible, as noted in the loads section, per the code. I am not going to analyze any specific column, rather a typical condition considering the standard bay size, and then I will compare that to the overall column size range.

Live load reduction per the code provides a new live load of:

A_T = 841 per floor*4 floors = 3364 SF
A_I = 4*3364 = 13,456 SF (assuming an interior column)
$L_R = 100*[0.25 + 15/(13456)^{1/2}] = 38 < 0.4*100 = 40$ therefore, use L_R = 40 psf

The total design live load for a typical column in a typical bay is:

P_L = 4*841*40 psf + 841*30 psf = 159.8 k

The typical dead load of 65 psf results in a total design dead load of:

P_D = 5*841*65 psf = 273.3 k

Therefore, the total factored design load for a typical column is:

$1.2*P_D + 1.6*P_L = 1.2*159.8 + 1.6*273.3 = 629$ k

The typical floor-to-floor height for the addition is 18 feet, so that will be used as the column unbraced length. Assuming that the column is pinned-pinned and therefore K=1.0, the required column size from Table 4-1 of the Steel Manual is a W12x72. This column size falls within the range of the designed columns.

From this check of a typical column, the designer’s column sizes are assumed to be appropriate.

APPENDIX B – FLOOR SYSTEM 2: TWO WAY CONCRETE FLAT PLATE

Material Properties:

F'_c = 4000 psi
F_y (reinforcing steel) = 60 ksi
w_{conc} = 150 pcf

Loads:

Live Load = 100 psf (typical floor)
Superimposed Dead Load = 15 psf
Total Factored Load (using old load factors)
w_u = 1.4*15 + 1.7*100 = 191 psf → use 200 psf in table to be conservative
Self-weight is already considered in load tables

Check Requirements for Using Table:

1. Minimum of 3 spans continuous in each direction……………OK
   The floor plan is somewhat irregular because it is framed in two different directions. However, in most cases it can be considered that there are 3 spans in each direction. For the preliminary design, I'm assuming it is reasonable to analyze the building with these tables, and the floor system will be looked at in more detail if and when required.

2. Ratio of panel length to width not to exceed 2.0……………OK
   29'/29' = 1.0 < 2.0 (typical bay)

3. Successive span lengths to differ not more than 1/3 the long span……………OK

4. Column centers not to be offset more than 10% of the span……………OK

5. Live load not to exceed 2x dead load……………OK
   Live load = 100 psf
   Dead load = SW + 15 psf > 50 psf for required minimum slab thickness

Choose Slab Thickness:

29' bay spacing
100 psf live load
Requires a minimum 9.5" slab thickness from CRSI Figure 9-10
Select a 10" slab

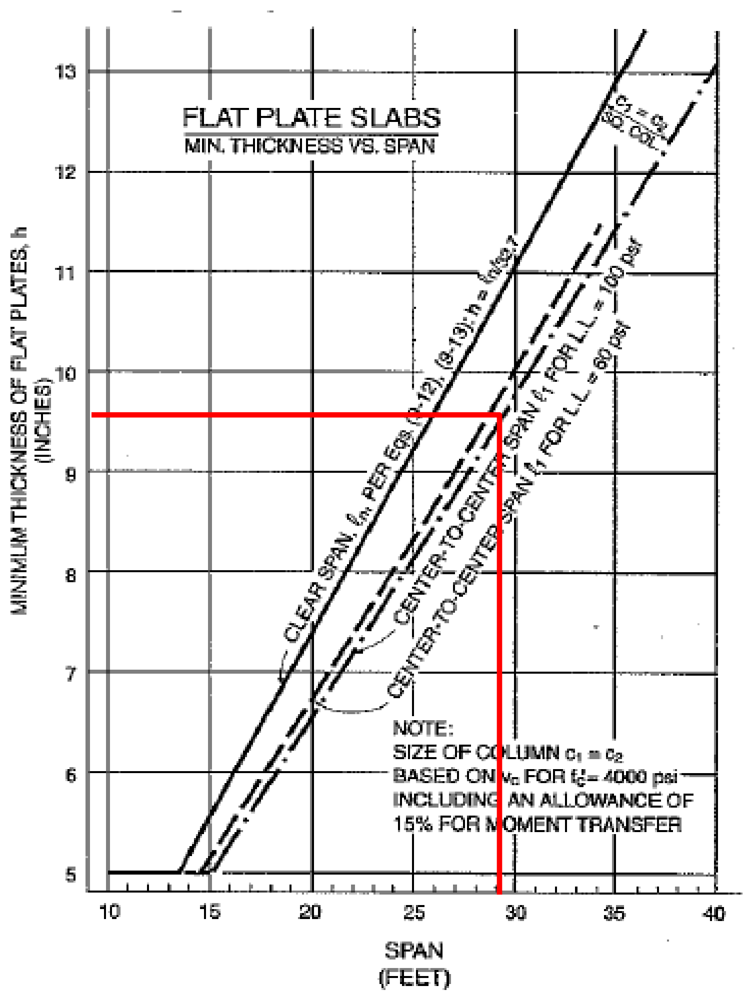


Figure 9-10 Minimum Thickness

Note: This figure assures slab deflection meets the appropriate requirements

TECHNICAL ASSIGNMENT #2

FLAT PLATE SYSTEM
(WITHOUT SHEARHEADS) SQUARE EDGE PANEL

SPAN c.-c. Cols. $\ell_1 = \ell_2$	Factored Superimposed Load	(1) Min. Square Column		Total Panel Moments			Reinforcing Bars						End Panel		
				−M Ext.	+M Int.	−M 1st. int.	Each Column Strip				Each Middle Strip		Steel (psf)		
													Location of Panel		
							Top Ext.	+	Bottom	Top Int.	Bottom	Top Int.	E	EC	C
(ft)	(psf)	(in.)	γ_f	(ft-kip)	(ft-kip)	(ft-kip)									
10 in. = TOTAL THICKNESS OF SLAB															**0.833 c.f./s.f.**
26	50	20	0.762	115	230	309	12-# 5	4	12-# 5	15-# 6	10-# 5	10-# 5	2.72	2.74	2.74
26	100	24	0.724	136	272	367	12-# 5	5	14-# 5	13-# 7	10-# 5	10-# 5	2.96	2.98	2.96
26	150	28	0.685	157	313	421	14-# 5	4	9-# 7	12-# 8	11-# 5	10-# 5	3.33	3.37	3.42
26	200	32	0.677	175	350	471	16-# 5	5	10-# 7	13-# 8	12-# 5	10-# 5	3.70	3.73	3.83
26	250	36	0.612	192	384	517	12-# 6	2	20-# 5	15-# 8	10-# 6	11-# 5	4.11	4.14	4.29
26	300	41	0.613	205	411	553	13-# 6	3	9-# 8	16-# 8	11-# 6	12-# 5	4.47	4.52	4.71
26	350	47	0.610	216	431	580	19-# 5	3	10-# 8	17-# 8	11-# 6	9-# 6	4.72	4.77	4.96
27	50	22	0.741	128	256	345	12-# 5	5	10-# 6	12-# 7	10-# 5	10-# 5	2.80	2.80	2.74
27	100	26	0.708	151	303	407	13-# 5	5	16-# 5	12-# 8	11-# 5	10-# 5	3.11	3.16	3.17
27	150	31	0.675	173	346	466	15-# 5	6	10-# 7	13-# 8	12-# 5	10-# 5	3.50	3.54	3.66
27	200	35	0.652	194	387	521	12-# 6	4	11-# 7	15-# 8	10-# 6	12-# 5	4.02	4.05	4.20
27	250	40	0.611	211	422	568	19-# 5	3	12-# 7	16-# 8	11-# 6	9-# 6	4.41	4.42	4.65
27	300	46	0.610	224	447	602	14-# 6	3	10-# 8	17-# 8	11-# 6	13-# 5	4.61	4.67	4.95
27	350	53	0.609	233	466	628	15-# 6	2	9-# 9	18-# 8	9-# 7	10-# 6	5.13	5.18	5.32
28	50	24	0.706	142	283	381	13-# 5	4	11-# 6	14-# 7	10-# 5	10-# 5	2.86	2.89	2.95
28	100	28	0.722	168	335	451	15-# 5	6	10-# 7	13-# 8	12-# 5	10-# 5	3.33	3.36	3.50
28	150	33	0.665	192	383	516	17-# 5	5	20-# 5	15-# 8	10-# 6	11-# 5	3.80	3.81	3.94
28	200	37	0.668	214	428	576	19-# 5	6	12-# 7	16-# 8	11-# 6	13-# 5	4.24	4.26	4.50
28	250	44	0.616	230	460	619	20-# 5	5	10-# 8	18-# 8	16-# 5	10-# 6	4.49	4.56	4.82
28	300	52	0.609	241	483	650	15-# 6	3	11-# 8	19-# 8	12-# 6	10-# 6	4.90	4.97	5.18
28	350	59	0.608	252	504	678	16-# 6	2	11-# 8	20-# 8	10-# 7	11-# 6	6.25	5.32	5.46
29	50	26	0.730	156	312	420	14-# 5	7	12-# 6	15-# 7	11-# 5	11-# 5	3.03	3.05	3.07
29	100	31	0.665	184	369	496	16-# 5	5	14-# 6	14-# 8	13-# 5	11-# 5	3.49	3.53	3.69
29	150	36	0.644	210	421	566	13-# 6	4	12-# 7	16-# 8	11-# 6	13-# 5	4.03	4.08	4.14
29	200	42	0.611	233	466	627	15-# 6	2	10-# 8	18-# 8	16-# 5	10-# 6	4.39	4.45	4.69
29	250	50	0.609	248	496	667	16-# 6	2	11-# 8	19-# 8	10-# 7	11-# 6	4.91	4.97	5.11
29	300	57	0.608	261	521	702	23-# 5	4	10-# 9	20-# 8	10-# 7	11-# 6	5.22	5.29	5.59
29	350	65	0.607	270	541	728	17-# 6	2	10-# 9	21-# 8	10-# 7	16-# 5	5.43	5.50	5.88
30	50	28	0.699	171	343	462	15-# 5	6	10-# 7	17-# 7	12-# 5	11-# 5	3.21	3.23	3.26
30	100	33	0.692	203	406	546	18-# 5	7	12-# 7	15-# 8	10-# 6	12-# 5	3.77	3.78	3.86
30	150	39	0.642	231	462	622	20-# 5	6	10-# 8	18-# 8	16-# 5	10-# 6	4.21	4.26	4.41
30	200	47	0.616	251	502	676	16-# 6	4	11-# 8	19-# 8	10-# 7	11-# 6	4.74	4.79	4.95
30	250	55	0.608	267	534	718	17-# 6	2	10-# 9	21-# 8	10-# 7	11-# 6	5.14	5.20	5.46
30	300	63	0.607	280	560	754	18-# 6	1	10-# 9	22-# 8	14-# 6	12-# 6	5.35	5.42	5.77
30	350	71	0.607	290	579	780	19-# 6	0	13-# 8	23-# 8	20-# 5	12-# 6	5.58	5.68	6.00
31	50	30	0.707	188	376	506	17-# 5	7	14-# 6	14-# 8	13-# 5	11-# 5	3.28	3.33	3.46
31	100	35	0.705	222	444	597	14-# 6	6	13-# 7	17-# 8	11-# 6	13-# 5	3.92	3.97	4.16
31	150	43	0.655	250	500	673	16-# 6	5	11-# 8	19-# 8	13-# 6	11-# 6	4.53	4.58	4.75
31	200	52	0.609	270	541	728	17-# 6	4	12-# 8	21-# 8	14-# 6	16-# 5	4.93	4.99	5.14
31	250	61	0.608	287	573	772	18-# 6	3	13-# 8	22-# 8	20-# 5	12-# 6	5.19	5.26	5.62
31	300	69	0.607	300	600	808	19-# 6	1	17-# 7	23-# 8	15-# 6	13-# 6	5.50	5.58	5.88
31	350	78	0.606	310	620	835	20-# 6	0	11-# 9	24-# 8	12-# 7	13-# 6	5.89	5.95	6.34

SQUARE INTERIOR PANEL							f'_c = 4,000 psi Grade 60 Bars		
(2)	(3)	(1)	Reinforcing Bars				Steel (psf)		
Span c.-c. (ft)	Load (psf	Min. Sq. Col. (in.)	Column Strip		Middle Strip		Location of Panel		
			Top	Bottom	Top	Bottom	I	IE	IC
10 in. = TOTAL THICKNESS OF SLAB							0.833 c.f./s.f.		
26	50	14	14-# 6	10-# 5	10-# 5	10-# 5	2.76	2.77	2.79
26	100	19	13-# 7	10-# 5	10-# 5	10-# 5	3.04	3.04	3.04
26	150	23	14-# 7	11-# 5	10-# 5	10-# 5	3.23	3.27	3.31
26	200	28	13-# 8	9-# 6	10-# 5	10-# 5	3.69	3.69	3.69
26	250	33	14-# 8	10-# 6	11-# 5	10-# 5	3.98	4.01	4.04
26	300	40	15-# 8	11-# 6	11-# 5	10-# 5	4.26	4.30	4.35
26	350	48	15-# 8	11-# 6	12-# 5	10-# 5	4.35	4.43	4.50
27	50	15	12-# 7	10-# 5	10-# 5	10-# 5	2.82	2.81	2.81
27	100	21	14-# 7	11-# 5	10-# 6	10-# 5	3.10	3.14	3.18
27	150	26	12-# 8	9-# 6	10-# 5	10-# 5	3.42	3.45	3.48
27	200	31	14-# 8	10-# 6	11-# 5	10-# 5	3.81	3.85	3.89
27	250	37	15-# 8	11-# 6	12-# 5	10-# 5	4.12	4.16	4.20
27	300	46	16-# 8	16-# 5	12-# 5	11-# 5	4.37	4.42	4.46
27	350	55	17-# 8	9-# 7	9-# 6	11-# 5	4.72	4.77	4.82
28	50	17	13-# 7	10-# 5	10-# 5	10-# 5	2.81	2.83	2.85
28	100	23	15-# 7	12-# 5	10-# 5	10-# 5	3.14	3.19	3.23
28	150	28	14-# 8	10-# 6	11-# 5	10-# 5	3.66	3.69	3.72
28	200	34	15-# 8	11-# 6	12-# 5	10-# 5	3.95	3.99	4.03
28	250	43	16-# 8	16-# 5	13-# 5	11-# 5	4.23	4.31	4.38
28	300	53	17-# 8	12-# 6	13-# 5	11-# 5	4.49	4.57	4.65
28	350	62	18-# 8	10-# 7	10-# 6	12-# 5	4.93	5.01	5.09
29	50	19	14-# 7	11-# 5	11-# 5	11-# 5	2.98	3.00	3.02
29	100	25	13-# 8	13-# 5	11-# 5	11-# 5	3.39	3.42	3.45
29	150	31	15-# 8	11-# 6	12-# 5	11-# 5	3.87	3.90	3.94
29	200	38	17-# 8	12-# 6	13-# 5	11-# 5	4.25	4.29	4.33
29	250	49	18-# 8	10-# 7	10-# 6	12-# 5	4.68	4.72	4.76
29	300	60	18-# 8	10-# 7	10-# 6	12-# 5	4.74	4.81	4.89
29	350	70	19-# 8	10-# 7	11-# 6	13-# 5	5.06	5.11	5.18
30	50	21	16-# 7	13-# 5	11-# 5	11-# 5	3.18	3.20	3.22
30	100	27	15-# 8	11-# 6	11-# 5	11-# 5	3.68	3.69	3.70
30	150	33	17-# 8	12-# 6	13-# 5	11-# 5	4.08	4.12	4.16
30	200	44	18-# 8	10-# 7	10-# 6	12-# 5	4.49	4.53	4.57
30	250	56	19-# 8	10-# 7	11-# 6	13-# 5	4.79	4.85	4.91
30	300	67	20-# 8	14-# 6	11-# 6	13-# 5	5.00	5.08	5.15
30	350	78	20-# 8	20-# 5	11-# 6	13-# 5	5.05	5.16	5.27
31	50	23	17-# 7	14-# 5	11-# 5	11-# 5	3.21	3.24	3.26
31	100	30	16-# 8	16-# 5	12-# 5	11-# 5	3.71	3.75	3.78
31	150	38	18-# 8	13-# 6	14-# 5	12-# 5	4.25	4.29	4.33
31	200	50	19-# 8	14-# 6	11-# 6	13-# 5	4.63	4.68	4.74
31	250	62	20-# 8	20-# 5	11-# 6	14-# 5	4.84	4.91	4.98
31	300	74	21-# 8	11-# 7	16-# 5	14-# 5	5.13	5.22	5.31
31	350	86	22-# 8	12-# 7	12-# 6	14-# 5	5.46	5.54	5.61

APPENDIX C – FLOOR SYSTEM 3: TWO WAY CONCRETE FLAT SLAB

Material Properties:

F'_c = 4000 psi
w_{conc} = 150 pcf
F_y (reinforcing steel) = 60 ksi

Loads:

Live Load = 100 psf (typical floor)
Superimposed Dead Load = 15 psf
Total Factored Load (using old load factors)
w_u = 1.4*15 + 1.7*100 = 191 psf → use 200 psf in table to be conservative
Self-weight is already considered in load tables

Check Requirements for Using Table:

1. Minimum of 3 spans continuous in each direction..........OK
   The floor plan is somewhat irregular because it is framed in two different directions. However, in most cases it can be considered that there are 3 spans in each direction. For the preliminary design, I'm assuming it is reasonable to analyze the building with these tables, and the floor system will be looked at in more detail if and when required.

2. Ratio of panel length to width not to exceed 2.0..........OK
   29'/29' = 1.0 < 2.0 (typical bay)

3. Successive span lengths to differ not more than 1/3 the long span..........OK

4. Column centers not to be offset more than 10% of the span..........OK

5. Live load not to exceed 2x dead load..........OK
   Live load = 100 psf
   Dead load = SW + 15 psf > 50 psf for required minimum slab thickness

Choose Slab Thickness:

From CRSI Design Handbook, Section 10:

Table 10-1 Minimum Thickness, *h*, of Two-Way Slabs*

(Expressed as fraction of longer clear span, ℓ_n)

1. Slabs without Interior Beams or With Interior Beams With $\alpha_m \leq 0.2$	
a. Flat Plates+ (no drop panels) Min. h = 5 inches	
Interior panels	$\ell_n/33$
Exterior panels, no edge beams	$\ell_n/30$
Exterior panels, with edge beams ≠	$\ell_n/33$
b. Flat Slab+ (with drop panels) Min. h = 4 inches	
Interior panels	$\ell_n/36$
Exterior panels, no edge beams	$\ell_n/33$
Exterior panels, with edge beams ≠	$\ell_n/36$

t_{min} = $l_n/36$ for interior panels = (29 – 16/12)/36 = 0.769’ → 9.5” thickness minimum
$l_n/33$ for exterior panels = (29 – 16/12)/33 = 0.838’ → 10” thickness minimum

10” minimum slab thickness required
16” square minimum interior columns and 19” square minimum exterior columns (from Table shown below)

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		(3) Square Column		REINFORCING BARS (E. W.)						MOMENTS		
						Column Strip (1)			Middle Strip		Total Steel (psf)	Edge (−) (f-k)	Bot. (+) (ft-k)	Int. (−) (ft-k)
		Depth (in.)	Width (ft)	Size (in.)	γ_f	Top Ext. +	Bottom	Top Int.	Bottom	Top Int.				
h = 10 in. = TOTAL SLAB DEPTH BETWEEN DROP PANELS														
25	100	5.50	8.33	12	0.776	12-#5 2	10-#6	14-#5	9-#5	9-#5	2.39	130.1	260.2	350.3
25	200	5.50	8.33	15	0.809	12-#5 4	13-#6	13-#6	12-#5	10-#5	2.95	171.3	342.6	461.2
25	300	7.00	8.33	18	0.664	12-#5 1	17-#6	15-#6	15-#5	9-#6	3.59	2'2.4	424.7	571.8
25	400	8.50	8.33	19	0.632	12-#5 1	15-#7	12-#7	10-#7	15-#5	4.25	254.3	508.6	684.6
25	500	8.50	10.00	21	0.744	13-#5 3	11-#9	26-#5	15-#6	10-#7	4.97	295.4	590.8	795.3
26	100	5.50	8.67	12	0.810	12-#5 3	11-#6	16-#5	11-#5	10-#5	2.60	146.8	293.7	395.3
26	200	7.00	8.67	15	0.704	12-#5 1	11-#7	14-#6	10-#6	12-#5	3.17	194.0	388.0	522.3
26	300	8.50	8.67	18	0.633	12-#5 1	11-#8	15-#6	9-#7	15-#5	3.88	240.6	481.1	647.6
26	400	8.50	8.67	19	0.745	13-#5 3	13-#8	18-#6	11-#7	9-#7	4.73	287.7	575.5	774.7
26	500	8.50	10.40	24	0.745	15-#5 4	13-#9	12-#8	10-#8	14-#6	5.49	330.9	661.8	890.9
27	100	7.00	9.00	12	0.746	12-#5 2	18-#5	16-#5	12-#5	10-#5	2.63	165.4	330.8	445.4
27	200	7.00	9.00	15	0.804	12-#5 5	17-#6	15-#6	11-#6	13-#5	3.37	2'8.2	436.3	587.4
27	300	8.50	9.00	18	0.674	12-#5 2	16-#7	13-#7	19-#5	16-#5	4.12	270.7	541.5	728.9
27	400	8.50	10.80	22	0.756	14-#5 5	12-#9	12-#8	10-#8	19-#5	5.09	321.6	643.2	865.8
27	500	8.50	10.80	27	0.682	16-#5 3	17-#8	13-#8	9-#9	9-#8	5.78	366.6	733.3	987.1
28	100	7.00	9.33	12	0.784	13-#5 2	14-#6	18-#5	13-#5	11-#5	2.76	185.0	370.0	498.1
28	200	8.50	9.33	16	0.714	13-#5 3	11-#8	15-#6	17-#5	15-#5	3.56	243.2	486.4	654.8
28	300	8.50	9.33	19	0.757	13-#5 5	11-#9	14-#7	12-#7	10-#7	4.56	302.4	604.8	814.1
28	400	8.50	11.20	25	0.692	16-#5 3	17-#8	13-#8	11-#8	12-#7	5.47	357.1	714.3	961.5
29	100	8.50	9.67	12	0.737	13-#5 2	22-#5	18-#5	15-#5	12-#5	2.91	206.7	413.4	556.5
29	200	8.50	9.67	16	0.758	13-#5 4	12-#8	13-#7	19-#5	16-#5	3.81	271.2	542.5	730.3
29	300	8.50	9.67	22	0.718	15-#5 4	20-#7	16-#7	10-#8	20-#5	4.92	334.3	668.6	900.1
29	400	8.50	11.60	28	0.639	17-#5 2	15-#9	14-#8	12-#8	10-#8	5.83	392.7	785.4	1057.3
30	100	8.50	10.00	12	0.774	14-#5 2	10-#8	20-#5	16-#5	10-#6	3.16	229.4	458.8	617.6
30	200	8.50	10.00	18	0.744	14-#5 4	11-#9	14-#7	21-#5	10-#7	4.16	299.6	599.1	806.5
30	300	8.50	10.00	24	0.675	16-#5 3	17-#8	14-#8	11-#8	12-#7	5.24	369.5	739.1	994.9

SQUARE INTERIOR PANEL
With Drop Panels(2)
No Beams

SPAN c.-c. $\ell_1 = \ell_2$ (ft)	Factored Superimposed Load (psf)	(3) Square Column Size (in.)	REINFORCING BARS (E. W.) Column Strip Top	Column Strip Bottom	Middle Strip Top	Middle Strip Bottom	Total Steel (psf)	Concrete $\left(\frac{\text{cu. ft}}{\text{sq. ft}}\right)$
	h = 10 in. = TOTAL SLAB DEPTH BETWEEN DROP PANELS							
25	100	12	13-#5	9-#5	9-#5	9-#5	2.19	0.884
25	200	18	12-#6	12-#5	10-#5	9-#5	2.63	0.884
25	300	21	14-#6	15-#5	12-#5	10-#5	3.10	0.898
25	400	23	15-#6	18-#5	10-#6	12-#5	3.63	0.912
25	500	25	13-#7	15-#6	16-#5	10-#6	4.26	0.947
26	100	12	15-#5	11-#5	10-#5	10-#5	2.40	0.884
26	200	18	17-#5	14-#5	11-#5	10-#5	2.73	0.898
26	300	21	14-#6	9-#7	13-#5	11-#5	3.31	0.912
26	400	23	13-#7	11-#7	16-#5	10-#6	4.17	0.912
26	500	25	27-#5	10-#8	10-#7	16-#5	4.65	0.947
27	100	12	15-#5	12-#5	10-#5	10-#5	2.37	0.898
27	200	18	14-#6	11-#6	12-#5	10-#5	2.92	0.898
27	300	21	12-#7	19-#5	15-#5	9-#6	3.56	0.912
27	400	24	26-#5	10-#8	10-#7	15-#5	4.35	0.947
27	500	27	16-#7	11-#8	11-#7	18-#5	5.02	0.947
28	100	12	17-#5	13-#5	10-#5	10-#5	2.42	0.898
28	200	19	14-#6	17-#5	13-#5	12-#5	3.02	0.912
28	300	21	13-#7	22-#5	12-#6	10-#6	3.85	0.912
28	400	24	16-#7	11-#8	20-#5	12-#6	4.71	0.947
29	100	12	17-#5	15-#5	12-#5	11-#5	2.58	0.912
29	200	19	16-#6	19-#5	15-#5	13-#5	3.27	0.912
29	300	21	15-#7	10-#8	10-#7	16-#5	4.34	0.912
29	400	26	13-#8	12-#8	12-#7	10-#7	5.06	0.947
30	100	12	14-#6	12-#6	13-#5	11-#5	2.77	0.912
30	200	19	18-#6	22-#5	12-#6	10-#6	3.57	0.912
30	300	21	16-#7	11-#8	11-#7	18-#5	4.56	0.912

APPENDIX D – FLOOR SYSTEM 4: OPEN WEB STEEL JOISTS

Choose a slab depth and deck type:

From the Vulcraft Steel Deck Catalog:

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

Total Slab Depth	Deck Type	Weight PSF	NW Concrete N=9 145 PCF 1 Span	2 Span	3 Span	Weight PSF	LW Concrete N=14 110 PCF 1 Span	2 Span	3 Span
2" (t=1 1/2")	0.6C28	23	2- 3	2- 10	2- 11	17	2- 4	3- 0	3- 0
	0.6C26	23	2- 8	3- 5	3- 5	18	2- 9	3- 6	3- 7
	0.6C24	23	3- 4	4- 3	4- 4	18	3- 6	4- 6	4- 7
	0.6C22	23	3- 10	5- 0	5- 1	18	4- 1	5- 4	5- 4
2 1/2" (t=2")	0.6C28	29	2- 2	2- 9	2- 10	22	2- 3	2- 10	2- 11
	0.6C26	29	2- 6	3- 3	3- 4	22	2- 8	3- 5	3- 6
	0.6C24	29	3- 2	4- 1	4- 2	22	3- 4	4- 4	4- 4
	0.6C22	29	3- 8	4- 9	4- 10	23	3- 11	5- 1	5- 2
3" (t=2 1/2")	0.6C28	35	2- 1	2- 8	2- 8	27	2- 2	2- 10	2- 10
	0.6C26	35	2- 5	3- 2	3- 2	27	2- 7	3- 4	3- 4
	0.6C24	35	3- 0	3- 11	4- 0	27	3- 2	4- 2	4- 2
	0.6C22	36	3- 6	4- 7	4- 7	27	3- 9	4- 10	4- 11
3 1/2" (t=3")	0.6C28	41	2- 0	2- 7	2- 7	31	2- 1	2- 9	2- 9
	0.6C26	41	2- 4	3- 0	3- 1	31	2- 6	3- 3	3- 3
	0.6C24	41	2- 10	3- 9	3- 10	32	3- 1	4- 0	4- 1
	0.6C22	42	3- 4	4- 5	4- 5	32	3- 7	4- 8	4- 9
4" (t=3 1/2")	0.6C28	47	1- 11	2- 6	2- 7	36	2- 1	2- 8	2- 8
	0.6C26	47	2- 3	2- 11	3- 0	36	2- 5	3- 2	3- 2
	0.6C24	47	2- 9	3- 8	3- 8	36	3- 0	3- 11	3- 11
	0.6C22	48	3- 2	4- 3	4- 3	36	3- 5	4- 6	4- 7
4 1/2" (t=4")	0.6C28	53	1- 10	2- 5	2- 6	40	2- 0	2- 7	2- 8
	0.6C26	53	2- 2	2- 10	2- 11	40	2- 4	3- 1	3- 1
	0.6C24	53	2- 8	3- 6	3- 7	41	2- 10	3- 9	3- 10
	0.6C22	54	3- 1	4- 1	4- 2	41	3- 4	4- 5	4- 5
5" (t=4 1/2")	0.6C28	59	1- 10	2- 5	2- 5	45	1- 11	2- 6	2- 7
	0.6C26	59	2- 1	2- 9	2- 10	45	2- 3	3- 0	3- 0
	0.6C24	59	2- 7	3- 5	3- 6	45	2- 10	3- 8	3- 9
	0.6C22	60	3- 0	3- 11	4- 0	46	3- 3	4- 3	4- 4

Choose a 3" total slab depth with lightweight concrete

Calculate Superimposed Loads:

w_{SDL} = 20 psf (assume 20 psf superimposed – self weight already considered in table)
w_L = 100 psf

w_{Total} = 100 + 20 = 120 psf

REINFORCED CONCRETE SLAB ALLOWABLE LOADS

Total Slab Depth	Reinforcement W.W.F.	As	Superimposed Uniform Load (psf) — 3 Span Condition, Clear Span (ft.-in.) 2-0	2-3	2-6	2-9	3-0	3-3	3-6	3-9	4-0
2" (t=1 1/2")	6X6-W1.4XW1.4	0.028*	194	153	124	103	86	74	63		
	6X6-W2.1XW2.1	0.042	285	225	183	151	127	108	93		
	6X6-W2.9XW2.9	0.058	384	304	246	203	171	146	125		
2 1/2" (t=2")	6X6-W1.4XW1.4	0.028*	268	212	172	142	119	102	88	76	67
	6X6-W2.1XW2.1	0.042	396	313	254	210	176	150	129	113	99
	6X6-W2.9XW2.9	0.058	400	400	344	284	239	204	176	153	134
3" (t=2 1/2")	6X6-W1.4XW1.4	0.028*	342	271	219	181	152	130	112	97	86
	6X6-W2.1XW2.1	0.042*	400	400	325	268	226	192	166	144	127
	6X6-W2.9XW2.9	0.058	400	400	400	366	307	262	226	197	173
3 1/2" (t=3")	6X6-W2.1XW2.1	0.042*	400	400	396	327	275	234	202	176	155
	6X6-W2.9XW2.9	0.058*	400	400	400	400	375	320	276	240	211
	4X4-W2.9XW2.9	0.087	400	400	400	400	400	400	400	353	310
4" (t=3 1/2")	6X6-W2.1XW2.1	0.042*	400	400	400	384	322	275	237	206	181
	6X6-W2.9XW2.9	0.058*	400	400	400	400	400	372	321	280	246
	4X4-W2.9XW2.9	0.087	400	400	400	400	400	400	400	400	358
4 1/2" (t=4")	6X6-W2.9XW2.9	0.058*	400	400	400	400	400	400	359	313	275
	4X4-W2.9XW2.9	0.087	400	400	400	400	400	400	400	400	400
	4X4-W4.0XW4.0	0.120	400	400	400	400	400	400	400	400	400
5" (t=4 1/2")	6X6-W2.9XW2.9	0.058*	400	400	400	400	400	400	396	345	303
	4X4-W2.9XW2.9	0.087*	400	400	400	400	400	400	400	400	400
	4X4-W4.0XW4.0	0.120	400	400	400	400	400	400	400	400	400
			0.6C28		0.6C26		0.6C24			0.6C22	

Span = 2'-5" → Use 2'-6" in table

Choose the 0.6C28 deck with total slab depth of 3" (concrete thickness = 2 ½") and 6x6-W1.4xW1.4 reinforcing for an allowable superimposed load of 219 psf > 120 psf.

Choose Steel Joists:

Floor Material Dead Loads

Material	Load
3" slab on form deck	27 psf
Framing	6 psf
MEP	10 psf
Miscellaneous	7 psf
Total	50 psf

Live Load = 100 psf (typical floor)

w_u = 1.2*50 + 1.6*100 = 220 psf*2.417' = 531.7 lb/ft

From Vulcraft Joist Catalog for a 29-foot span:

JOIST DESIGNATION	18K3	18K4	18K5	18K6	18K7	18K9	18K10	20K3	20K4	20K5	20K6	20K7	20K9	20K10	22K4	22K5	22K6	22K7	22K9	22K10	22K11
DEPTH (IN.)	18	18	18	18	18	18	18	20	20	20	20	20	20	20	22	22	22	22	22	22	22
APPROX. WT. (lbs./ft.)	6.6	7.2	7.7	8.5	9.0	10.2	11.7	6.7	7.6	8.2	8.9	9.3	10.8	12.2	8.0	8.8	9.2	9.7	11.3	12.6	13.8
SPAN (ft.) ↓																					
18	550	550	550	550	550	550	550														
	550	550	550	550	550	550	550														
19	514	550	550	550	550	550	550														
	494	523	523	523	523	523	523														
20	463	550	550	550	550	550	550	517	550	550	550	550	550	550							
	423	490	490	490	490	490	490	517	550	550	550	550	550	550							
21	420	506	550	550	550	550	550	468	550	550	550	550	550	550							
	364	426	460	460	460	460	460	453	520	520	520	520	520	520							
22	382	460	518	550	550	550	550	426	514	550	550	550	550	550	550	550	550	550	550	550	550
	316	370	414	438	438	438	438	393	461	490	490	490	490	490	548	548	548	548	548	548	548
23	349	420	473	516	550	550	550	389	469	529	550	550	550	550	518	550	550	550	550	550	550
	276	323	362	393	418	418	418	344	402	451	468	468	468	468	491	518	518	518	518	518	518
24	320	385	434	473	526	550	550	357	430	485	528	550	550	550	475	536	550	550	550	550	550
	242	284	318	345	382	396	396	302	353	396	410	448	448	448	431	483	495	495	495	495	495
25	294	355	400	435	485	550	550	329	396	446	486	541	550	550	438	493	537	550	550	550	550
	214	250	281	305	337	337	337	266	312	350	380	421	426	426	381	427	464	474	474	474	474
26	272	328	369	402	448	538	550	304	366	412	449	500	550	550	404	455	496	550	550	550	550
	190	222	249	271	299	354	361	236	277	310	337	373	405	405	338	379	411	454	454	454	454
27	252	303	342	372	415	498	550	281	339	382	416	463	550	550	374	422	459	512	550	550	550
	169	198	222	241	267	315	347	211	247	277	301	333	389	389	301	337	367	406	432	432	432
28	234	282	318	346	385	463	548	261	315	355	386	430	517	550	348	392	427	475	550	550	550
	151	177	199	216	239	282	331	189	221	248	269	298	353	375	270	302	328	364	413	413	413
29	218	263	296	322	359	431	511	243	293	330	360	401	482	550	324	365	398	443	532	550	550
	136	159	179	194	215	254	298	170	199	223	242	268	317	359	242	272	295	327	387	399	399
30	203	245	276	301	335	402	477	227	274	308	336	374	450	533	302	341	371	413	497	550	550
	123	144	161	175	194	229	269	153	179	201	218	242	286	336	219	245	266	295	349	385	385

The loads from the tables must be converted from the Steel Joist Institute's Specification for use in LRFD design. Following the conversion method on page 8 of the Joist Catalog:

w_{sji} = load per linear foot from table
w_u = ultimate load calculated with LRFD load factors

$w_{sji} > w_u/(1.65*0.9) = 531.7/(1.65*0.9) = 358$ lb/ft
$w_{sji} = 360$ lb/ft > 358 lb/ft therefore OK

$w_{sji,LL} > w_L/1.5 = (100*2.417')/1.5 = 161.1$ lb/ft
$w_{sji,LL} = 269 > 161.1$ lb/ft therefore OK

Use 20K6 joists for the 29-foot span

From Vulcraft Joist Catalog for 19 foot span:

JOIST DESIGNATION	8K1	10K1	12K1	12K3	12K5	14K1	14K3	14K4	14K6	16K2	16K3	16K4	16K5	16K6	16K7	16K9
DEPTH (IN.)	8	10	12	12	12	14	14	14	14	16	16	16	16	16	16	16
APPROX. WT. (lbs./ft.)	5.1	5.0	5.0	5.7	7.1	5.2	6.0	6.7	7.7	5.5	6.3	7.0	7.5	8.1	8.6	10.0
SPAN (ft.) ↓																
8	550															
	550															
9	550															
	550															
10	550	550														
	480	550														
11	532	550														
	377	542														
12	444	550	550	550	550											
	288	455	550	550	550											
13	377	479	550	550	550											
	225	363	510	510	510											
14	324	412	500	550	550	550	550	550	550							
	179	289	425	463	463	550	550	550	550							
15	281	358	434	543	550	511	550	550	550							
	145	234	344	428	434	475	507	507	507							
16	246	313	380	476	550	448	550	550	550	550	550	550	550	550	550	550
	119	192	282	351	398	390	467	467	467	550	550	550	550	550	550	550
17		277	336	420	550	395	495	550	550	512	550	550	550	550	550	550
		159	234	291	368	324	404	443	443	488	526	526	526	526	526	526
18		246	299	374	507	352	441	530	550	456	508	550	550	550	550	550
		134	197	245	317	272	339	397	408	409	456	490	490	490	490	490
19		221	268	335	454	315	395	475	550	408	455	547	550	550	550	550
		113	167	207	269	230	287	336	383	347	386	452	455	455	455	455
20		199	241	302	409	284	356	428	525	368	410	493	550	550	550	550
		97	142	177	230	197	246	287	347	297	330	386	426	426	426	426

The loads from the tables must be converted from the Steel Joist Institute's Specification for use in LRFD design. Following the conversion method on page 8 of the Joist Catalog:

w_{sji} = load per linear foot from table
w_u = ultimate load calculated with LRFD load factors

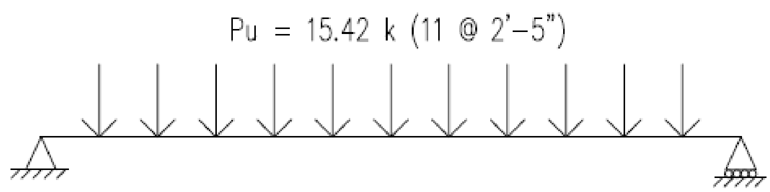
$w_{sji} > w_u/(1.65*0.9) = 531.7/(1.65*0.9) = 358$ lb/ft
$w_{sji} = 395$ lb/ft > 358 lb/ft therefore OK

$w_{sji,LL} > w_L/1.5 = (100*2.417')/1.5 = 161.1$ lb/ft
$W_{sji,LL} = 287 > 161.1$ lb/ft therefore OK

Use 14K3 joists for the 19-foot span

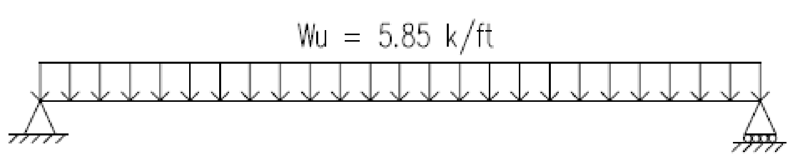
Design Steel Girder:

Point loads on typical girder = (531.7 lb/ft*29')/2 = 7.71 k/joist
29' joists framing into each side of girder = 7.71 k * 2 = 15.42 k



Since the joists are spaced closely together and covered with a 2" topping, it is reasonable to consider the point loads from the joists as a uniform distributed load.

Equivalent uniformly distributed load = 15.42 k*11 point loads/29' = 5.85 k/ft



V_u = 5.85*29/2 = 84.8 k
M_u = 5.85*29^2/8 = 615 ft-k
I_{req} = 5*5.85*29^4*1728/(384*29000*(29*12/360)) = 3321 in4

The joists brace the girder every 2'-5", so the unbraced length is assumed to be < L_p. Therefore, Table 3-2 can be used to select a girder.

Use either: W27x102 (ϕM_p = 701 ft-k; I_x = 3620 in^4)
W24x117 (ϕM_p = 764 ft-k; I_x = 3540 in^4)
W21x147 (ϕM_p = 643 ft-k; I_x = 3610 in^4)
**Assume the maximum girder depth to be 27" so that the total floor system = 30" (same as maximum existing floor depth)

Note: These girders were sized for the worst case – where the 29-foot 20K6 joists frame into both sides. In many cases, these joists frame into one side and the 19-foot 14K3 joists frame into the other, reducing the girder's load and thus the joist size.

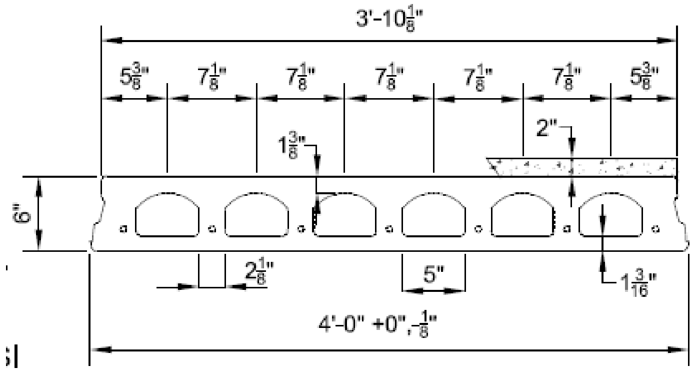
APPENDIX E – FLOOR SYSTEM 5: HOLLOW CORE PRECAST PLANKS

Choose Hollow Concrete Plank Size:

Try a 6" x 4'-0" Nitterhouse Hollow Core Plank with 2" topping spanning 14.5'

PHYSICAL PROPERTIES
Composite Section

$A_c = 253$ in.^{2} Precast $S_{bc} = 370$ in.^{3}
$I_c = 1519$ in.^{4} Topping $S_{tc} = 551$ in.^{3}
$Y_{bc} = 4.10$ in. Precast $S_{tc} = 799$ in.^{3}
$Y_{tc} = 1.90$ in. Wt.= 195 PLF
Wt.= 48.75 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 = 67.5 k-ft

Check Plank Capacity:

W_D = 48.75 psf (plank) + 25 psf (2" topping) + 15 psf (superimposed) = 90 psf
W_L = 100 psf
W_u = 1.2D + 1.6L = 1.2*90 + 1.6*100 = 268 psf
$W_{u,plank}$ = 268 psf * 4' = 1.07 k/ft

$M_u = (1.07\text{ k/ft})*14.5^2/8 = 28.1$ ft-k < 67.5 ft-k therefore OK

$E = w_c^{1.5}*33f'_c{}^{1/2} = 150^{1.5}*33*6000^{1/2} = 4695$ ksi
$\Delta = 5*1.07*29^4*1728/(384*4695*1519) = 0.149" < L/360 = 0.483"$ therefore OK

Design Beam:

W_u = 268 psf*14.5' = 3.89 k/ft

V_u = 3.89*29'/2 = 56.3 k
M_u = (3.89 k/ft)*29^2/8 = 408.5 ft-k
I_{req} = 5*3.89*29^4*1728/(384*29000*(29*12/360)) = 2208 in^4

Use a W24x84 (ϕM_p = 840 ft-k, I = 2370 in^4)

Design Girder:

P_u = 56.3*2 = 112.6 k @ mid span (from beam above)

V_u = 112.6/2 = 56.3 k
M_u = 112.6*29/4 = 816.4 ft-k
I_{req} = 112.6*29^3*1728/(48*29000*(29*12/360)) = 3527 in^4

Use either: W24x117 (ϕM_p = 1230 ft-k, I = 3540 in^4)
W27x102 (ϕM_p = 1140 ft-k, I = 3620 in^4)

APPENDIX F – RS MEANS COST ESTIMATES

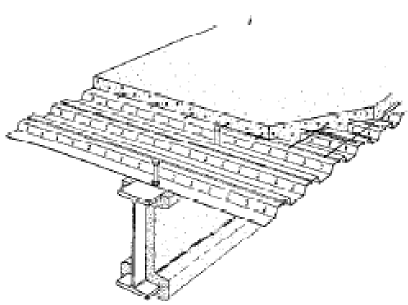
The following tables were taken from *RS Means 2006 Assemblies Cost Data*. The estimated cost per square foot is for the floor system only. The different systems may affect other aspects of the overall building costs such as foundation costs and exterior materials cost. The impacts of the alternate floor systems on these other issues were not calculated but were briefly discussed in the body of the report. Both the description of the floor construction, which includes any assumptions, and the tabulated costs were included for each floor system considered.

Floor System 1: Existing Composite System

The cost of the existing composite system given in the table includes the cost of composite beams, welded shear studs, composite steel deck, and a lightweight concrete slab reinforced with WWF. The table uses 3 ksi concrete and 36 ksi steel, while the designer used 3.5 ksi concrete for the slab and 50 ksi steel for the beams, so there could be some variation in cost. Also, the cost is based on the bay size and load, not the size of the members, so the cost per square foot is not exact.

B10 Superstructure

B1010 Floor Construction



Description: Table below lists costs ($/S.F.) for a floor system using composite steel beams with welded shear studs, composite steel deck, and light weight concrete slab reinforced with W.W.F. Price includes sprayed fiber fireproofing on steel beams.

Design and Pricing Assumptions:
Structural steel is A36, high strength bolted.
Composite steel deck varies from 22 gauge to 16 gauge, galvanized.
Shear Studs are 3/4".
W.W.F., 6 x 6 – W1.4 x W1.4 (10 x 10)
Concrete f'c = 3 KSI, lightweight.
Steel trowel finish and cure.
Fireproofing is sprayed fiber (non-asbestos).

Spandrels are assumed the same as interior beams and girders to allow for exterior wall loads and bracing or moment connections.

B1010 256 Composite Beams, Deck & Slab

	BAY SIZE (FT.)	SUPERIMPOSED LOAD (P.S.F.)	SLAB THICKNESS (IN.)	TOTAL DEPTH (FT.-IN.)	TOTAL LOAD (P.S.F.)	COST PER S.F. MAT.	INST.	TOTAL
2400	20x25	40	5-1/2	1 - 5-1/2	80	9.30	4.91	14.21
2500	RB1010 -100	75	5-1/2	1 - 9-1/2	115	9.65	4.92	14.57
2750		125	5-1/2	1 - 9-1/2	167	11.75	5.75	17.50
2900		200	6-1/4	1 - 11-1/2	251	13.25	6.20	19.45
3000	25x25	40	5-1/2	1 - 9-1/2	82	9.15	4.68	13.83
3100		75	5-1/2	1 - 11-1/2	118	10.15	4.75	14.90
3200		125	5-1/2	2 - 2-1/2	169	10.60	5.15	15.75
3300		200	6-1/4	2 - 6-1/4	252	14.30	6	20.30
3400	25x30	40	5-1/2	1 - 11-1/2	83	9.35	4.65	14
3600		75	5-1/2	1 11-1/2	119	10.05	4.70	14.75
3900		125	5-1/2	1 - 11-1/2	170	11.60	5.30	16.90
4000		200	6-1/4	2 - 6-1/4	252	14.35	6	20.35
4200	30x30	40	5-1/2	1 - 11-1/2	81	9.40	4.79	14.19
4400		75	5-1/2	2 - 2-1/2	116	10.10	5	15.10
4500		125	5-1/2	2 - 5-1/2	168	12.20	5.60	17.80
4700		200	6-1/4	2 - 9-1/4	252	14.60	6.50	21.10

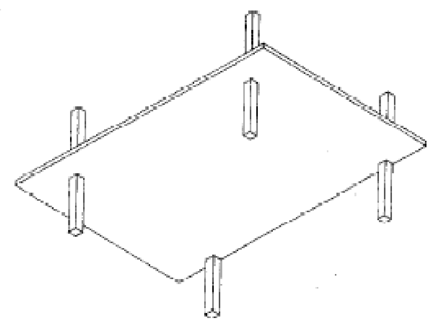
Cost of Materials = \$12.20/SF
Cost of Labor = \$5.60/SF
Total Cost = \$17.80/SF

Floor System 2: Two-Way Concrete Flat Plate

For a concrete flat plate system, the concrete is assumed to be 4 ksi, which is what I used for design purposes as well. However, the estimate is based on a 4-bay by 4-bay structure, which the hospital is not. Most likely, because of the building's irregular shape, the cost would be somewhat higher than that given in the table. Finally, an estimate had to be taken for a 25 by 25 foot bay because that is the maximum bay size given in the table. To somewhat offset the smaller bay size, a larger superimposed load of 125 psf was used.

B10 Superstructure

B1010 Floor Construction



General: Flat Plates: Solid uniform depth concrete two-way slab without drops or interior beams. Primary design limit is shear at columns.

Design and Pricing Assumptions:
Concrete f'c to 4 KSI, placed by concrete pump.
Reinforcement, fy = 60 KSI.
Forms, four use.
Finish, steel trowel.
Curing, spray on membrane.
Based on 4 bay x 4 bay structure.

B1010 223 Cast in Place Flat Plate

	BAY SIZE (FT.)	SUPERIMPOSED LOAD (P.S.F.)	MINIMUM COL. SIZE (IN.)	SLAB THICKNESS (IN.)	TOTAL LOAD (P.S.F.)	COST PER S.F. MAT.	INST.	TOTAL
2000	15 x 15	40	12	5-1/2	109	4.06	6.80	10.86
2200	RB1010 -010	75	14	5-1/2	144	4.09	6.80	10.89
2400		125	20	5-1/2	194	4.25	6.90	11.15
2600		175	22	5-1/2	244	4.33	6.90	11.23
3000	15 x 20	40	14	7	127	4.67	6.90	11.57
3400	RB1010 -100	75	16	7-1/2	169	4.96	7	11.96
3600		125	22	8-1/2	231	5.45	7.20	12.65
3800		175	24	8-1/2	281	5.45	7.20	12.65
4200	20 x 20	40	16	7	127	4.67	6.90	11.57
4400		75	20	7-1/2	175	5	7.05	12.05
4600		125	24	8-1/2	231	5.45	7.20	12.65
5000		175	24	8-1/2	281	5.50	7.25	12.75
5600	20 x 25	40	18	8-1/2	146	5.40	7.20	12.60
6000		75	20	9	188	5.60	7.25	12.85
6400		125	26	9-1/2	244	6	7.50	13.50
6600		175	30	10	300	6.25	7.60	13.85
7000	25 x 25	40	20	9	152	5.60	7.25	12.85
7400		75	24	9-1/2	194	5.90	7.45	13.35
7600		125	30	10	250	6.25	7.60	13.85
8000								

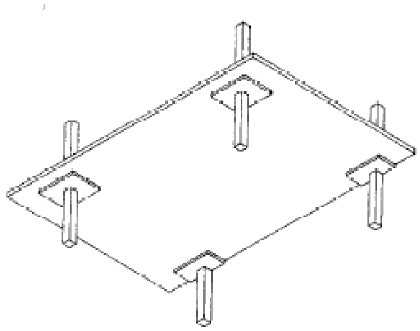
Cost of Materials = $6.25/SF
Cost of Labor = $7.60/SF
Total Cost = $13.85/SF

Floor System 3: Two-Way Concrete Flat Slab

For the flat slab system with drop panels, the same regular bay assumption is made as above. However, the concrete is assumed to be 3 ksi, and the floor system I designed was based on 4 ksi concrete, which could result in a slight cost variation. Also, a 30 by 30 foot bay with a 125 psf superimposed load was used in the table. The tabulated slab and drop thickness for this condition seemed to most closely match the actual thickness. This could slightly overestimate the cost of the system, but it will be accurate enough for a general comparison.

B10 Superstructure

B1010 Floor Construction



General: Flat Slab: Solid uniform depth concrete two-way slabs with drop panels at columns and no column capitals.

Design and Pricing Assumptions:
Concrete f'c = 3 KSI, placed by concrete pump.
Reinforcement, fy = 60 KSI.
Forms, four use.
Finish, steel trowel.
Curing, spray on membrane.
Based on 4 bay x 4 bay structure.

B1010 222 Cast in Place Flat Slab with Drop Panels

	BAY SIZE (FT.)	SUPERIMPOSED LOAD (P.S.F.)	MINIMUM COL. SIZE (IN.)	SLAB & DROP (IN.)	TOTAL LOAD (P.S.F.)	COST PER S.F. MAT.	INST.	TOTAL
6600	30 x 30	75	18	10-1/2 - 7-1/2	217	7.15	8.30	15.45
6800		125	22	10-1/2 - 9	269	7.50	8.50	16
7000		200	26	11 - 11	359	8	8.85	16.85
7400	30 x 35	40	16	11 1/2 - 9	196	7.30	8.30	15.60
7900		75	20	11-1/2 - 9	231	7.80	8.65	16.45
8000		125	24	11-1/2 - 11	284	8.10	8.85	16.95
9000	35 x 35	40	16	12 - 9	202	7.50	8.35	15.85
9400		75	20	12 - 11	240	8.10	8.75	16.85
9600		125	24	12 - 11	290	8.35	8.95	17.30

Cost of Materials = \$7.50/SF
Cost of Labor = \$8.50/SF
Total Cost = \$16.00/SF

Floor System 4: Open Web Steel Joists

For open web steel joists on steel columns and beams, basically all of the material property assumptions seem to be valid for my system except that A992 steel will be used for beams and girders. However, the table is based on a joist spacing of 2 feet, which is less than the designed joist spacing. This means the table will overestimate the number of joists required and hence overestimate the cost. Still, the tabulated cost is assumed to be sufficient for this general comparison. Once again, a 30 by 30 foot bay size with a 100 psf superimposed load is used.

B10 Superstructure

B1010 Floor Construction

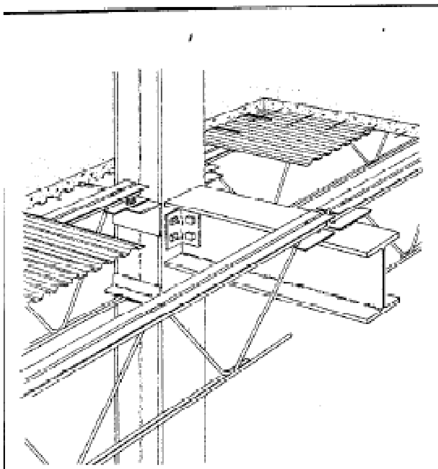


Table below lists costs for a floor system on steel columns and beams using open web steel joists, galvanized steel slab form, and 2-1/2" concrete slab reinforced with welded wire fabric.

Design and Pricing Assumptions:
Structural Steel is A36.
Concrete f'c = 3 KSI placed by pump.
WWF 6 x 6 – W1.4 x W1.4 (10 x 10)
Columns are 12' high.
Building is 4 bays long by 4 bays wide.
Joists are 2' O.C. ± and span the long direction of the bay.

Joists at columns have bottom chords extended and are connected to columns.

Slab form is 28 gauge galvanized.
Column costs in table are for columns to support 1 floor plus roof loading in a 2-story building; however, column costs are from ground floor to 2nd floor only. Joist costs include appropriate bridging.
Deflection is limited to 1/360 of the span.
Screeds and steel trowel finish.

Design Loads	Min.	Max.
S.S. & Joists	6.3 PSF	15.3 PSF
Slab Form	1.0	1.0
2-1/2" Concrete	27.0	27.0
Ceiling	3.0	3.0
Misc.	5.7	1.7
	43.0 PSF	48.0 PSF

B1010 250 Steel Joists, Beams & Slab on Columns

	BAY SIZE (FT.)	SUPERIMPOSED LOAD (P.S.F.)	DEPTH (IN.)	TOTAL LOAD (P.S.F.)	COLUMN ADD	COST PER S.F.		
						MAT.	INST.	TOTAL
6700	30x30	75	32	120		10.55	4.71	15.26
6800					column	.86	.29	1.15
6900	30x30	100	35	145		11.75	5.10	16.85
7000					column	1	.34	1.34
7100	30x30	125	35	172		12.80	6.35	19.15
7200					column	1.11	.37	1.48

Cost of Materials = $11.75/SF
Cost of Labor = $5.10/SF
Total Cost = $16.85/SF

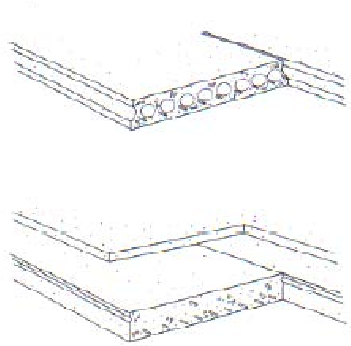
Floor System 5: Hollow Core Precast Planks

RS Means Assemblies did not have tabulated values for Hollow Core Precast Planks on Steel Beams and Girders. Instead, the cost of the precast planks (from one table) was added to the cost of W-Shape Beam and Girders (from another table). For the precast planks, prices were based on a much smaller project than this one, so most likely the price per square foot would decrease due to mass production. Also, it is possible that by summing the costs of these two assemblies, some costs are being accounted for twice or overestimated. Because of these two issues, both of which could have a considerable effect on cost, I am applying a 0.75 reduction factor to the cost of materials for the precast planks. This seems to be the only system from which the actual system varies enough from the assumptions to warrant a reduction factor.

For the hollow core planks with 2" topping, a 15-foot span with a 100 psf superimposed load was used. For the steel beams and girders, a 30 by 30 foot bay was used with one infill beam and the minimum superimposed load of 40 psf (because the load is already accounted for from the precast plank tables).

B10 Superstructure

B1010 Floor Construction



General: Units priced here are for plant produced prestressed members, transported to site and erected.

Normal weight concrete is most frequently used. Lightweight concrete may be used to reduce dead weight.

Structural topping is sometimes used on floors: insulating concrete or rigid insulation on roofs.

Camber and deflection may limit use by depth considerations.

Prices are based upon 10,000 S.F. to 20,000 S.F. projects, and 50 mile to 100 mile transport.

Concrete is f'c = 5 KSI and Steel is fy = 250 or 300 KSI

Note: Deduct from prices 20% for Southern states. Add to prices 10% for Western states.

Description of Table: Enter table at span and load. Most economical sections will generally consist of normal weight concrete without topping. If acceptable, note this price, depth and weight. For topping and/or lightweight concrete, note appropriate data.

Generally used on masonry and concrete bearing or reinforced concrete and steel framed structures.

The solid 4" slabs are used for light loads and short spans. The 6" to 12" thick hollow core units are used for longer spans and heavier loads. Cores may carry utilities.

Topping is used structurally for loads or rigidity and architecturally to level or slope surface.

Camber and deflection and change in direction of spans must be considered (door openings, etc.), especially untopped.

TECHNICAL ASSIGNMENT #2

B1010 230 Precast Plank with 2″ Concrete Topping

	SPAN (FT.)	SUPERIMPOSED LOAD (P.S.F.)	TOTAL DEPTH (IN.)	DEAD LOAD (P.S.F.)	TOTAL LOAD (P.S.F.)	COST PER S.F. MAT.	COST PER S.F. INST.	COST PER S.F. TOTAL
2000	10	40	6	75	115	5.70	4.35	10.05
2100		75	8	75	150	6.85	3.95	10.80
2200		100	8	75	175	6.85	3.95	10.80
2500	15	40	8	75	115	6.85	3.95	10.80
2600		75	8	75	150	6.85	3.95	10.80
2700		100.	8	75	175	6.85	3.95	10.80
2800	20	40	8	75	115	6.85	3.95	10.80
2900		75	8	75	150	6.85	3.95	10.80
3000		100	8	75	175	6.85	3.95	10.80
3100	25	40	8	75	115	6.85	3.95	10.80
3200		75	8	75	150	6.85	3.95	10.80
3300		100	10	80	180	7.15	3.66	10.81

B1010 241 W Shape Beams & Girders

	BAY SIZE (FT.) BEAM X GIRD	SUPERIMPOSED LOAD (P.S.F.)	STEEL FRAMING DEPTH (IN.)	FIREPROOFING (S.F. PER S.F.)	TOTAL LOAD (P.S.F.)	COST PER S.F. MAT.	COST PER S.F. INST.	COST PER S.F. TOTAL
7450	30x25	40	16	.637	50	5.85	2.33	8.18
7500		40	24	.839	90	8.30	3.26	11.56
7550		75	24	.919	125	10.05	3.88	13.93
7600		125	27	1.02	175	12.70	5.05	17.75
7650		200	30	1.160	260	15.90	4.87	20.77
7700	30x30	40	21	.52	50	6	2.31	8.31
7750		40	24	.629	103	9.30	3.44	12.74
7800		75	30	.715	138	11.05	4.07	15.12
7850		125	36	.822	206	14.50	5.55	20.05
7900		200	36	.878	281	16.15	4.75	20.90
7950	30x30	40	24	.619	50	6.30	2.45	8.75
8000		40	24	.706	90	8.45	3.22	11.67
8020		75	27	.818	125	10	3.80	13.80
8040		125	30	.910	175	12.85	5	17.85
8060		200	33	.999	263	15.80	4.75	20.55

Cost of Materials = \$6.85/SF *0.75 + \$6.00/SF = \$11.14/SF
Cost of Labor = \$3.95/SF + \$2.31/SF = \$6.26/SF
Total Cost = \$11.17/SF + \$6.26/SF = \$17.40/SF