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Technical Report 2

Executive Summary

This report researches various framing alternatives to the existing composite steel floor system at the Swedish American Hospital – Heart and Vascular Center. This structure consists of 4 patient floors and a mechanical screen wall on the roof with the option of enclosing the roof into a 5th floor and adding an additional two floors on top of it.

A variety of floor systems were considered for this project, but many were ruled out for various reasons at the beginning of conceptual design. From those considered, four were chosen to be analyzed in comparison with the existing composite steel system. Of these four alternative systems, two are steel systems acting compositely with concrete and the other two are concrete systems. One steel system is a composite steel system, like the system already in use at the Heart Hospital, only utilizing a more efficient layout. The other steel system is a relatively new system (Girder-Slab) used predominately in residential and commercial high-rise construction. The first concrete system analyzed for this report was a one way slab with beams and girders. This system was the first attempt at using a structural material other than steel and tried to decrease the overall floor depth without compromising other structural aspects of the building. The final alternative framing plan is a continuation of the first concrete framing plan. The second concrete system is a two way post tensioned slab. This system further minimizes the floor depth and can handle the long spans in the hospital without the need for additional column supports.

These five systems (existing system included) are all compared simultaneously. They are weighed against each other addressing items such as: floor depth, floor weight, slab thickness, span, deflection, fire protection, cost, lead time, constructability, vibration, and structural changes relative to the existing structure. Finally, all the positives and negatives from each system are added up to determine whether a particular floor system is feasible to construct (or worth pursuing further).

After comparing all the framing systems, it was determined that the alternative composite steel system and the two way post tensioned slab were both viable alternatives to the existing composite system. The one way slab was rejected because it was limited by the maximum distance the slab could span and wasn't cost efficient enough compared to the other concrete system. It might have also required additional columns in some areas. The Girder-Slab system was rejected because the prefabricated members currently available by Girder-Slab Technologies are not large enough or stiff enough to carry the required loads. They don't meet the deflection requirements set by code and the concrete can't develop the required compressive stress needed.

The other two systems, two way post tensioned slab and the alternative composite steel system, are both considered potential possibilities and will be studied further to determine which is best suited to be used at the structural framing for the Heart Hospital at Swedish American.

It should be noted that the calculations herein are for schematic design used to develop preliminary sizes of members. This is not to be an exhaustive analysis, so some general assumptions were made and are noted at those locations in the report.

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Existing Structural System Overview

Floor System:

The typical building floor framing system is made up of beams and girders acting compositely with a concrete floor slab. Floor sections show 3”-20 gauge LOK Floor galvanized metal deck with 3¼” of lightweight concrete (110 pcf) resting on the steel framing below. Composite action is achieved through 5” long ¾” diameter shear studs welded to the steel framing. Concrete is reinforced with 6x6-W5xW5 welded wire fabric. The span of the metal deck varies depending on the bay location. However, the direction is limited to east-west or northeast-southwest. This assembly has a 2 hour fire rating without the use of spray on fireproofing.

There is no “typical” bay in the structural framing system. However, columns located on the wings are spaced approximately 22’-7 ½” on center. Columns in the interior core area are spaced approximately 32’-0” on center with additional columns located around the core perimeter framing into the wings. The most common and longest span is 32’-0”. Typical beam sizes range from W12x14’s (typically spanning 10’ to 12’) to W27x146 (spans ranging from 22’ to 32’) with the larger beams acting as part of the moment framing system.

Roof System:

The roof framing system is very similar to the building floor framing system. Composite design is still used with 3 ¼” of lightweight concrete and 3”-20gauge LOK Floor metal deck on top of steel framing. Deeper steel beams and girders are used to help carry the heavier loads of the mechanical equipment on the roof.

The lobby roof is slightly different from the typical roof framing. It uses composite action but has a 1 ½” deep 20 gauge metal deck spanning north-south instead of the 3” metal deck used elsewhere on the building. Lower portions of the roof that see a heavier snow loads due to drift use a 3” deep 20 gauge metal deck.

Lateral System:

The lateral load resisting system consists of steel moment frames. The majority of the moment frames extend around the perimeter of the building with a few added moment frames on the interior to help stiffen the structure. Larger girders are framed into columns with bolted flange plate moment connections. The prefabricated steel pieces were bolted in place rather than welded to eliminate the need of preheating for welds. Shear walls were not part of the original design analysis; therefore, masonry cores such as the elevator and stairwell cores were not assumed to provide lateral support during the structural analysis.

Foundation

The basement footprint is approximately one half of the square footage of the first floor plan. Hence, there are two slabs on grade: one for the basement and one for part of the first floor. Each slab on grade is 5” thick normal weight concrete (145pcf) with 4x4-W5xW5 welded wire fabric reinforcement.

Interior steel columns rest on spread footings with an allowable soil bearing capacity of 4ksf. Exterior columns and basement walls rest on continuous strip footings. Reinforced concrete pilasters are located where exterior columns rest on the basement wall. Footings below columns in the interior core area extend approximately 18’ deep whereas the perimeter strip footings and footings located beneath the wings extend approximately 8’ deep. All footings are required to extend a minimum of 4’ deep for frost protection.

Columns:

Columns are laid out on two different intersecting grids: one running east-west and the other running northwest-southeast. All columns are ASTM A992 Grade 50 wide flange steel shapes. Columns are spliced between the 3rd and 4th floor. Columns acting as part of a moment frame are spliced 5’-6” above the 3rd floor elevation. Columns acting only as gravity columns are spliced 4’-6” above the 3rd floor elevation. All interior columns that extend to the basement level are also spliced 5’-6” above the 1st floor elevation. Future columns for the 6th and 7th floors are designed to be spliced with existing columns at the 5th floor elevation (current mechanical floor and roof).

Typical Floor Loads

Floor Live Loads

Loaded Area	Building Design Load	ASCE 7-05 Section 4
Typical Floors (2nd, 3rd, 4th, 6th, 7th)	80 psf	Table 4-1

The typical floor live load used for the existing framing analysis was 80 psf (per ASCE 7-05, Table 4-1). The a 20 psf partition load is included in the 80 psf live load as follows: corridors on typical floors have a live load (LL) of 80 psf without partitions, operating rooms and patient rooms have a LL of 60psf + the 20psf partition load = 80psf. This uniform 80psf live load was also used during the analysis of the alternative framing systems.

Existing Floor Dead Loads

Typical Floors 1 though 4 and Future Floors 6 and 7

Item	Design Load
Steel Deck with LWC Slab	48 psf
Ponding due to Deflection	5 psf
Steel Self Weight	15 psf
MEP, Superimposed, Misc.	12 psf
Total	80 psf

The dead load for each alternative system varies since not all systems incorporate the same materials and amounts of material. A standard superimposed dead load (DL) of 12 psf is used in the analysis of the steel alternative framing plan and Girder-Slab framing plan. This superimposed load was assumed for various MEP systems and ceiling finishes. A superimposed dead load of 17 psf is used in the analysis of the alternative framing system with one-way slab with beams and girders. The extra 5 psf accounts for any concrete ponding due to beam or slab deflections.

Load Factors

Gravity loads were the only loads accounted for in this report. Each system was checked with the load case 1.2D + 1.6L. By inspection, this load case controlled over the load case 1.4D because of the high live load (80 psf) used for this occupancy. If columns were to be designed and sized for this report, the load case 1.4D might have controlled because of the allowable live load reduction. This situation especially applies to the one-way slab with beams and girders alternative framing system and the existing composite steel system, where the ratio of dead load to live load is higher the other alternative framing systems.

Floor Framing Systems

Steel Composite Floor (Existing)

The existing structural system carrying the gravity loads at the Heart Hospital is a 6¼”composite concrete slab on 3” metal deck supported by a system of steel beams and girders. Five inch long (3/4” dia.) shear studs transfer the forces between the concrete and steel shapes. Lightweight concrete (110psf) with an f'_c = 4ksi is used for the concrete slabs. A slab depth with a total of 6.25 inches (with 3.25 inches of concrete above the flutes of the metal deck) has a total weight of 48 psf.

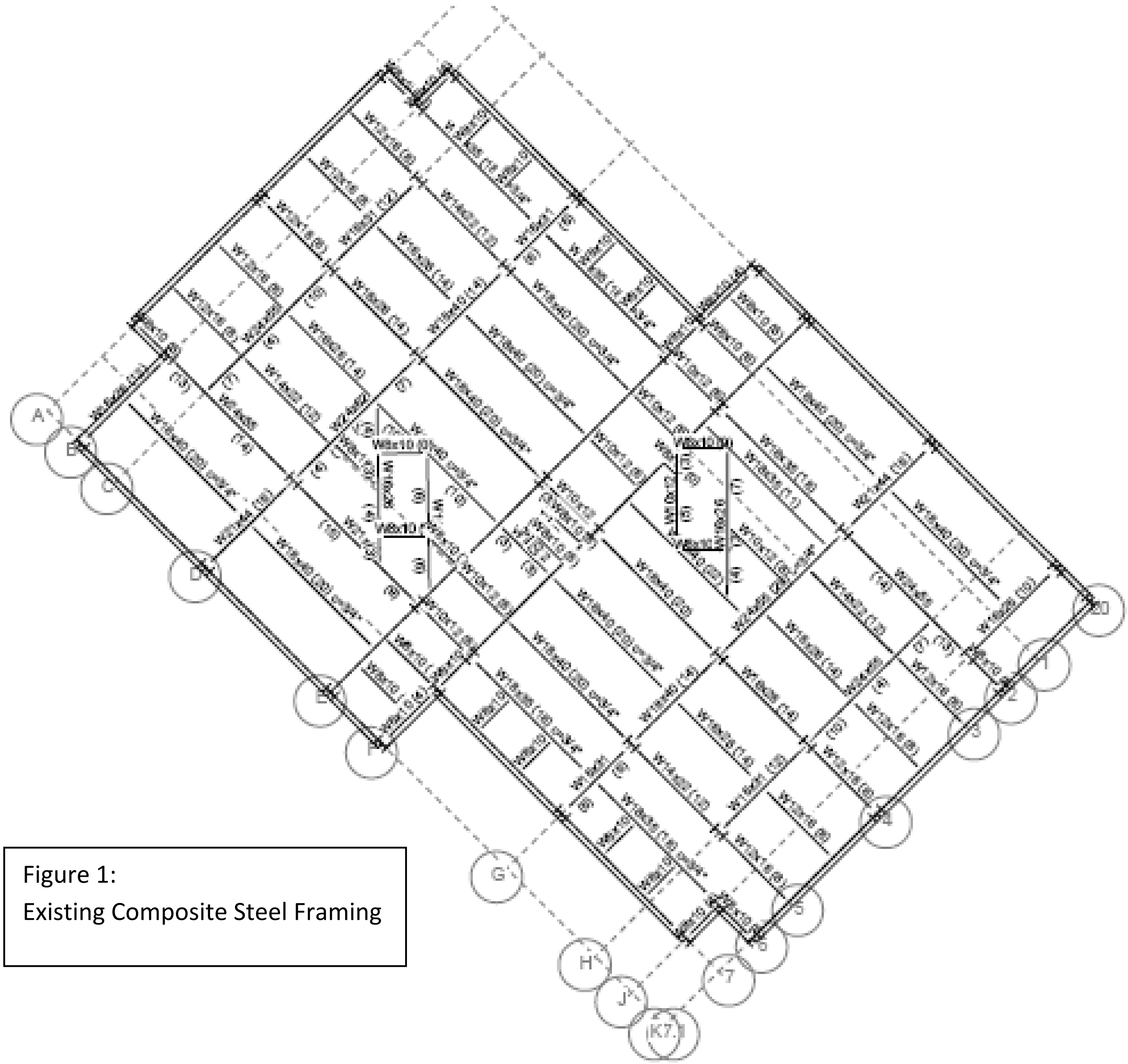


Figure 1:
Existing Composite Steel Framing

From the drawings, many of the beams and girders on a typical floor range from W14s to W24s. W14s, W16s and W18s are typically used as beams carrying a distributed load with spans ranging from 8' to 32' with the deeper beams running the longer spans (See Figure 1 above). W21s and W24s compose most of the girders and span either 22'7" or 32'. The largest gravity beam on a typical floor is a W24x62 and is the controlling member for framing depth. A W24x62 with 3" metal deck and 6 ¼" concrete has a maximum framing depth of 29.95 inches.

To check the existing floor framing, I created a RAM model of my structure using RAM Structural System. Analyzing only the gravity framing on a typical floor, RAM was able to replicate many of the member sizes seen on the structural drawings, including the W24x62. Using the RAM model, I found a maximum deflection of 1.03" on a typical 32' span gravity member. This value is less than the L/360 requirement for live loads and L/240 for total load deflections. (See Appendix A for RAM printouts on existing framing system)

Steel Composite Floor (Alternative)

The design of an alternative steel composite floor framing system included the removal of 4 columns. Columns B6, C3, H5, and J2 on the existing plans were removed and beams spanning NW-SE into those columns were made continuous to the exterior support for a total span of 40.5' (See Figure 2 below). This alternative framing system was also modeled in RAM Structural System using the same criteria as the existing framing system.

Beams and girders in this system remained relatively the same compared to the existing composite system. The only exceptions were the new beams and girders that previously had framed into the now removed columns. The increase in length and load on those new members increased the member depth to the next size (for example: W18s now became W21s, W21 became W24s and the W24s are now W27s). This increase in member size simultaneously increases the overall maximum depth of the floor framing from 29.95 inches to 32.95 inches. (See Appendix B for RAM printouts on the alternative steel framing system)

Larger deflections were also calculated as a result of the framing change. The larger girders (W24s and W27s) had controlling total load deflections of 1.0" to 1.2". However, these deflections still met the requirements L/240 and L/360 for total loads and live loads respectively. The increased depth of this system should not pose a problem since it is only at those localized areas where the 4 columns used to be. Therefore, if the increased floor depth does not cause an issue with the MEP systems, this system is a feasible alternative to the existing composite system already in place.

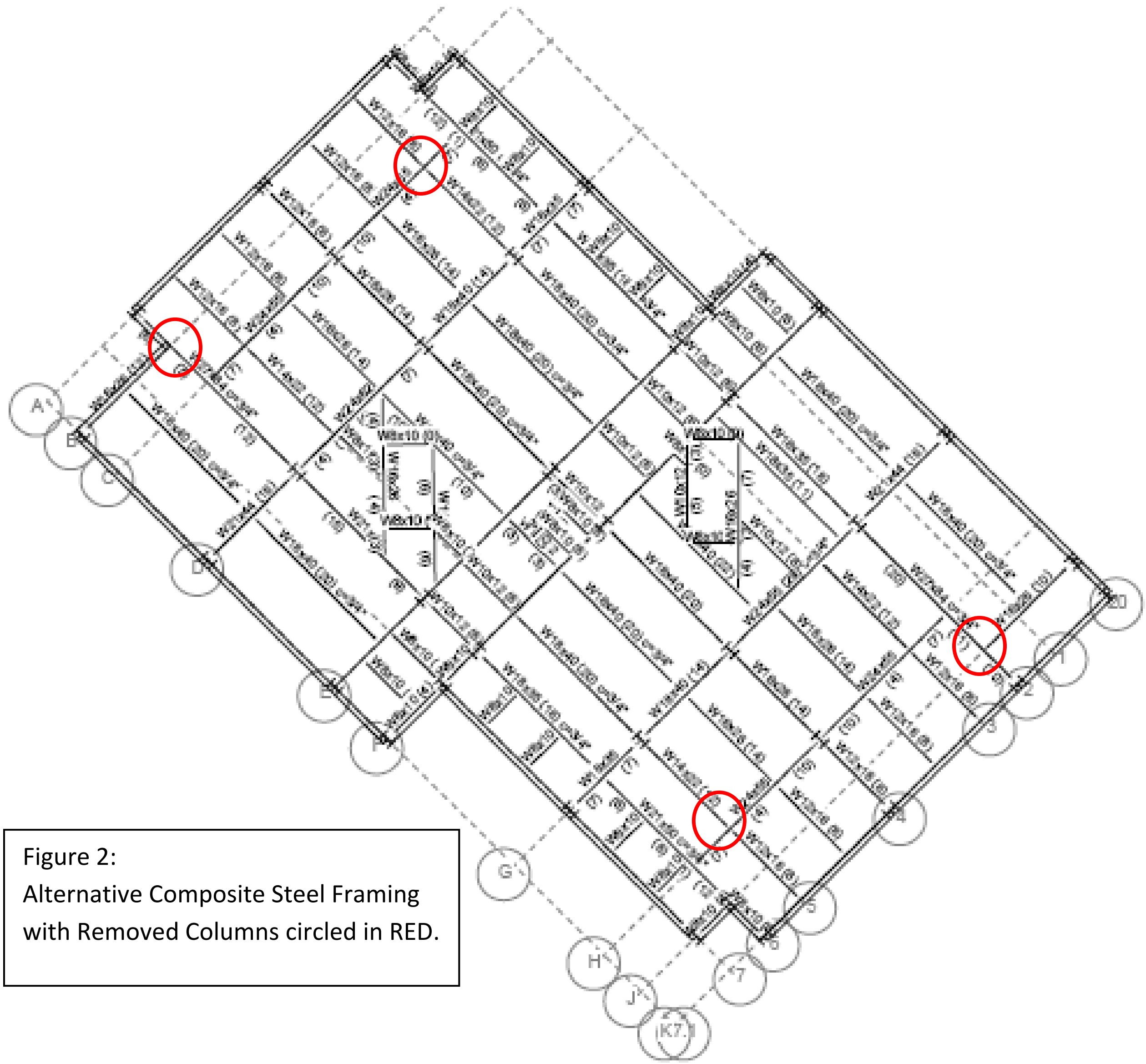


Figure 2:
Alternative Composite Steel Framing with Removed Columns circled in RED.

One Way Concrete Slab with Beams and Girders

For this system, I assumed the floor framing layout would not differ from the existing system, only the material would change. Concrete beams and girders would replace the existing steel beams and girders at their current locations (See Figure 3 below). Therefore, no additional columns or framing would be necessary for the alternative design. Typical spans are similar to those in the existing system (typically 22’-7” and 32’). For spans of 32’, a 16” wide by 19” deep reinforced concrete beam is necessary. For 22’-7” beams carrying a distributed load, a 12” wide by 16” deep beam is necessary. For 22’-7” girders carrying a point load(s), a 16” x 16” beam is necessary. (See Appendix C for detailed calculations)

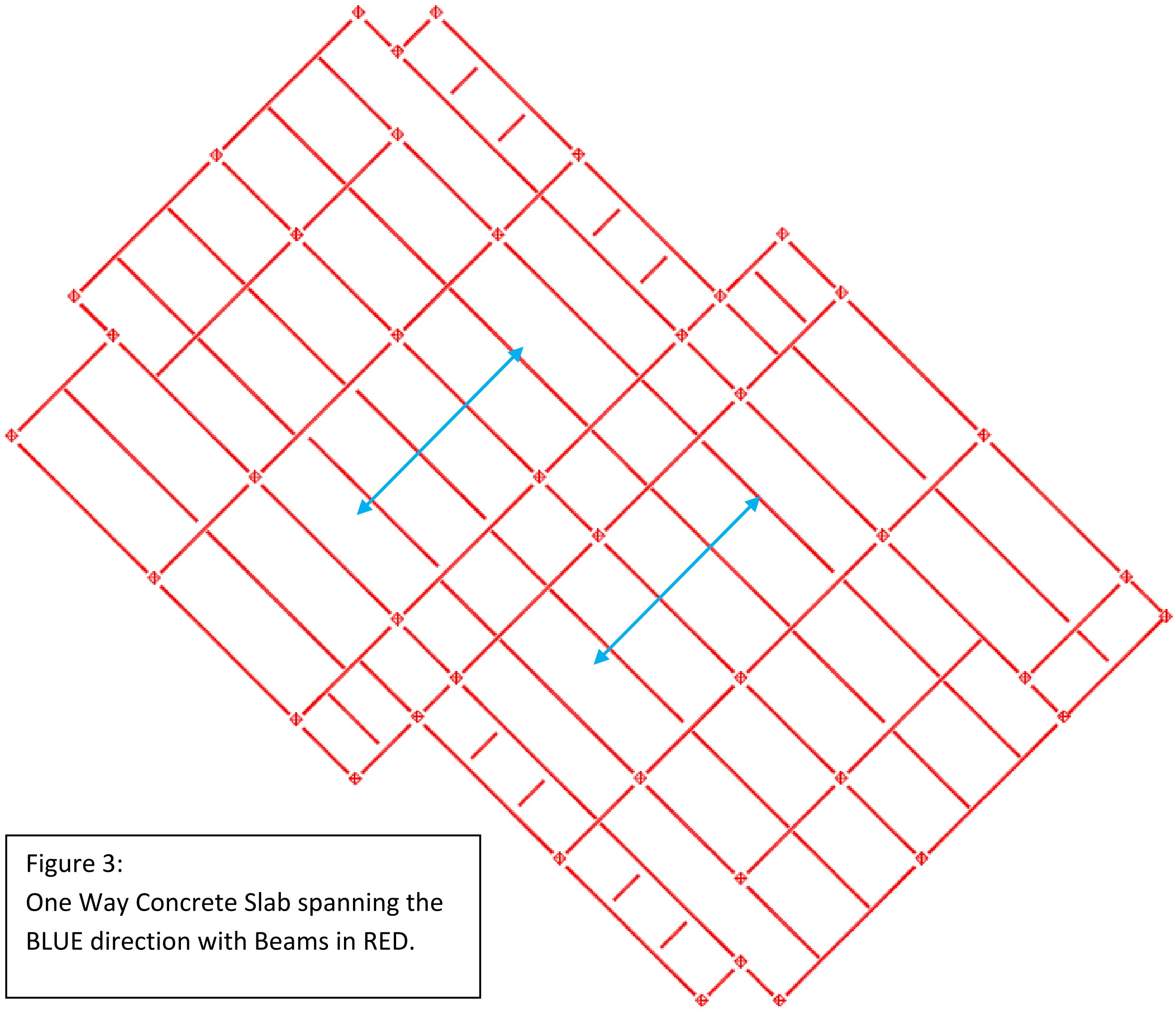


Figure 3:
One Way Concrete Slab spanning the BLUE direction with Beams in RED.

From ACI 318-05 Table 9.5(a), beam deflections in a one way slab design can be neglected if the depth of the beam (h) is > L/21 (assuming an interior span continuous on both ends). For spans of 32' and 22'-7", beam depths of 19" and 13" are required respectively and are less than or equal to the proposed beam and girder depths (19" and 16" respectively). For a one way slab (continuous on both ends) the slab deflection can be neglected if (h) > L/28. Maximum slab spans of 11'-4" yield a slab depth requirement of 4.86" which is less than the provided 5" slab.

The elements controlling the maximum framing depth are the beams and girders spanning 32'. They all have a depth of 19" which includes the 5" slab. Columns designs were not part of this assignment; however, it was assumed that 16"x16" columns would take the place of existing steel columns for calculation purposes. This is acceptable considering the maximum beam width is also 16". This system is a potential alternative; however, the poured concrete would require more framing for the various beam sizes. Also, if erection time is an issue, a steel structure is able to be erected quicker than a concrete structure.

Two Way Post-Tensioned Slab

The 6th Edition Post-Tensioning Manual published by PTI (Post Tensioning Institute) was a valuable asset in the design of a two way post tensioned flat plate. Referencing ACI 318-05 Table 9.5(c), slab deflections in a two way flat plate can be neglected if the slab depth (h) is > $l_n/33$. Conservatively assuming l_n = 32' yields a slab depth of 11.5". This is a conservative assumption to adjust for potentially large deflections. A more detailed analysis may require column capitals and/or spandrel beams in various locations to help carry the load over longer spans and resist punching shear over columns. These areas were not considered during this preliminary analysis. For the current design, the maximum framing depth is governed by the 11½" slab.

A flat plate was chosen over other post tension systems to take advantage of the required thickness of the concrete slab. Flat plate systems have optimum spans of 20'-30' and are useful in 'light' to 'medium' loading conditions (100psf – 200psf). Flat plates commonly have thicker slabs which result in more material and increased material cost. However, flat plates require less formwork that generates a cost savings in labor and formwork and assists in speeding up the construction process.

Following the procedure provided in the PTI Manual, 23 tendons are needed to provide adequate strength in the slab (See calculations in Appendix D). Of these 23, 6 are assumed to be banded together over the column supports. The remaining tendons are distributed evenly over the width of the bay (approximately 1'-6" o/c or about 2x the slab thickness). A mesh of rebar is required in areas of positive and negative moments to help carry and distribute the load. To minimize the number of tendons needed per slab, tendons spanning the long dimension of the building are evenly distributed over the width of each bay with a fraction of them banded together over the column supports. The tendons that span the short direction of the building (perpendicular to the designed tendons) should be placed in narrow bands over the columns supports. (See Figure 4 below for framing details)

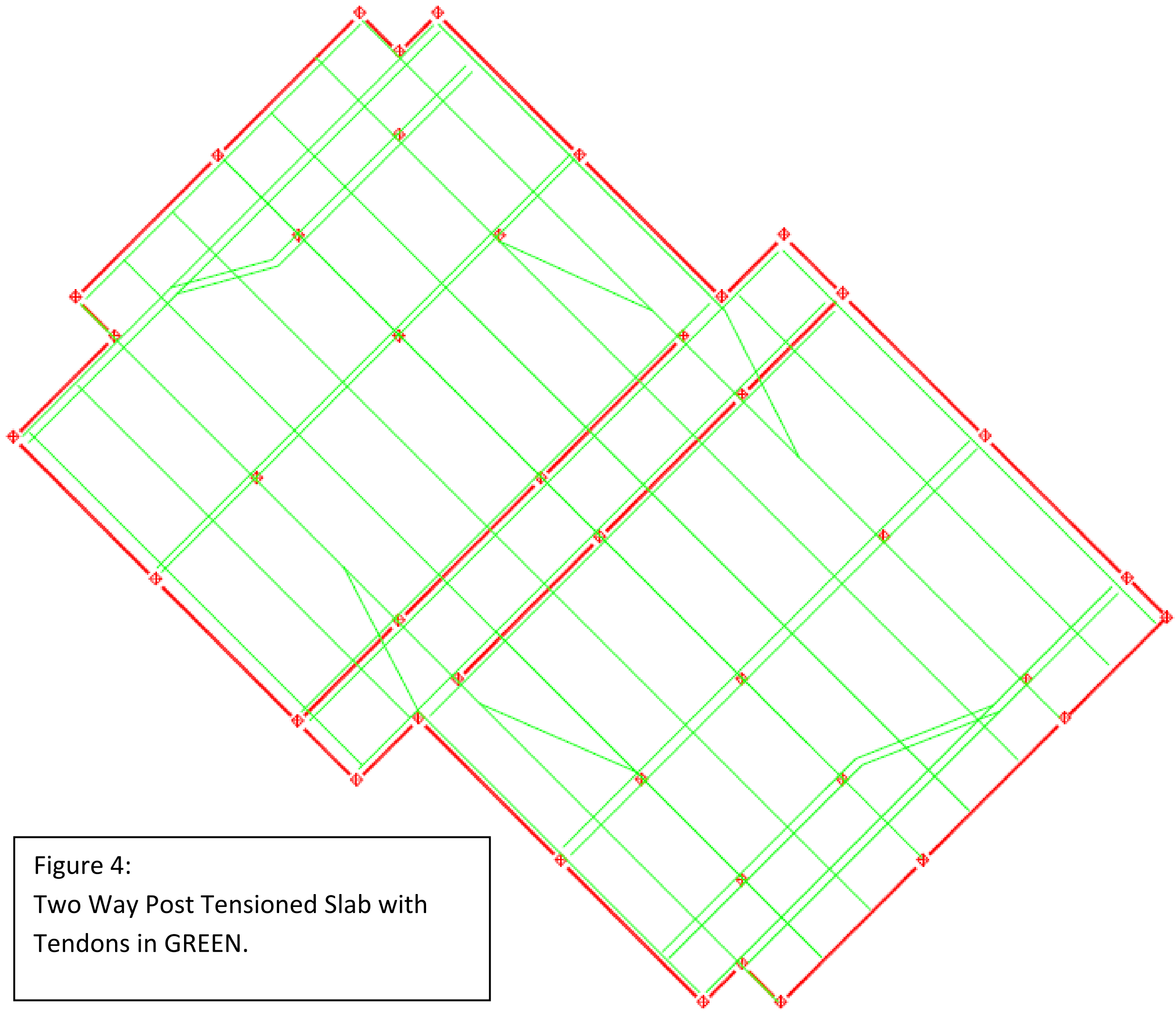


Figure 4:
Two Way Post Tensioned Slab with Tendons in GREEN.

This system is considered a feasible solution, but future analyses should investigate using a thinner slab with column capitals or drop panels to resist punching shear, which usually governs the slab thickness. A reduction in slab thickness and the addition of drop panels would maximize the cost efficiency of the design and create a lighter floor system. The lighter floor system would also help decrease the high seismic base shear that will have an impact on the lateral framing system.

Girder-Slab

The Girder-Slab system is a precast steel and concrete hybrid system. Precast concrete planks are supported by the bottom flange of special steel beams called D-Beams. D-Beams are available in two prefabricated depths: 8 inches and 9 inches with the largest size being a DB9x46. The two materials are assembled in place and connected together with a cementitious grout to achieve composite action (See Figure 5 below for more details). From their website (www.girder-slab.com), a design spreadsheet is an available download to aide in the preliminary design of the system.

The Girder-Slab system provides ideal performance in mid to high rise construction for residential structures. The D-Beams and precast planks work well with spans of 20' – 25' for lower residential live loads (40-50psf). To design the Girder-Slab system for the Heart Hospital, many adjustments must be made to the structure. By code, the live load of the hospital is 80psf, which is two times the optimum live load recommended by Girder-Slab Technologies. Therefore, additional columns must be installed to reduce the span lengths allow the system to handle the larger live load. The extra columns will also create a more symmetrical grid. However, additional columns would limit the flexibility of the spaces and interrupt the long open spans found in the hospital. In place of additional columns, additional wide flange beams could be installed in such a way to decrease the spans of the planks and D-Beams.

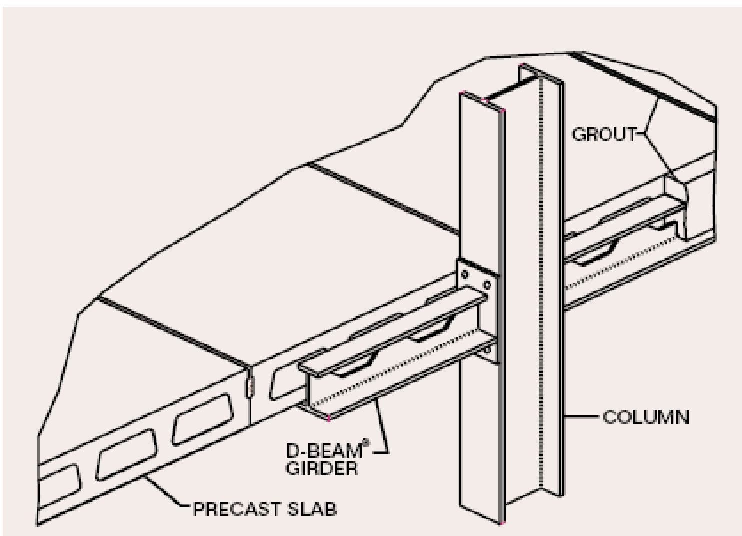


Figure 5:
Isometric view of Girder Slab construction. Image is provided by Girder-Slab Technologies

(www.girder-slab.com)

The additional beams would keep the flexible integrity of the open spans, but would still add additional cost and construction time to the project. If this system was chosen as a viable option, the largest D-Beam available (DB9x46) is needed to carry the large live loads. Once the D-Beams are in place and the precast planks are set, a 2" topping slab is poured on top of the planks and beams to create a smooth finish floor. This system has a maximum floor depth of 10" (not including the additional wide flange beams). To complete the design using additional beams, 4 additional columns are still needed. A total of 42 D-Beams are required with maximum spans of 22'-7". Not including wide flange beams acting as part of the lateral system, a total of 10 wide flange beams are need to transfer the gravity loads from the planks and D-Beams to the columns. (See Appendix E, Page 40 for a sketch of the Girder-Slab system layout)

This system is not a feasible option. The DB9x46 is not stiff enough to handle the heavy dead and live loads acting on the floor framing. First, the compressive stress in the concrete isn't able to handle the imposed stresses from the dead and live loads (see D-Beam Spreadsheet in Appendix F). Second, the live load deflection exceeds the acceptable value of L/360. Even if these two requirements were met, this system still requires 4 additional columns to minimize the span length of the D-Beams. These columns would increase the cost of the project and potentially decrease the flexibility of the open space on each floor.

Floor System Comparisons

Floor Depth

The city of Rockford, Illinois building code is such that minimizing the floor to floor height in the building was not an issue. The number of floors within the hospital was based on a "Certificate of Need". A Certificate of Need specifies the number of occupants a new medical facility must design for and is based on the surrounding population and expected growth.

The existing floor to floor height between typical floors is 13'-3". The total floor depth of an existing typical floor is 30.25" and is controlled by the deeper W24x62 members spanning 32'. The 3" LOK deck and 3 ¼" of concrete rest on top of the wide flange shapes and complete the 30.25" floor depth. The alternate steel framing system has a maximum floor depth of 33.25". It is identical to the existing system except the controlling member is W27x84.

The one way slab and beam system has a governing depth of 19". By code, 19" beams are needed to span 32' to meet deflection requirements without completing the deflection calculations. The smaller beams spanning 22'-7" have a maximum depth of 16". The maximum width for the concrete beams is 16" and is controlled by the assumed width of the columns (16"x16" concrete columns assumed for the one way slab with beams system).

The two way post tensioned slab has a maximum floor depth of 11½". This dimension is controlled by Table 9.5(c) in ACI 318-05. Further analysis of this system could reduce the slab dimension to create a lighter floor and minimize the slab depth even more. However, drop panels necessary around the columns would still control the maximum floor to floor depth.

The Girder-Slab system has the smallest floor depth of all 5 framing systems. The maximum floor depth of 10" is controlled by the 8" precast plank and 2" topping slab. However, this system is too small and doesn't provide the strength needed to satisfy the required loads. If this system was to be designed to handle the imposed loads, deeper members (D-Beams) would be needed to provide more stiffness. For now, the deepest D-Beam member provided by Girder-Slab Technologies is a DB9x46. Deeper members could provide the stiffness needed to satisfy the loading conditions, but it would also increase the floor depth of the system.

Maximum Floor Depth

Floor Type	Max Size	Max Defl. (d)	Camber	Max Depth (in)	Slab Width (in)
Existing	W24x62	1.03	0.75	30.25	3.25
Steel Alternative	W27x84	1.20	0.75	33.25	3.25
One Way Slab	19" Beam	0.72	--	19.00	5.00
Two Way PT	11.5" Slab	--	--	11.50	11.50
Girder Slab	10"	1.46	0.35	10.00	2.00

Cambers are used on the steel shapes in 3 of the 5 systems. A maximum camber of 0.75" is applied to a few beams and girders in the composite steel systems. In the Girder-Slab system, using their preliminary design spreadsheet available on their website (www.girder-slab.com) the maximum camber in a D-Beam is only allowed to be enough to equal the maximum deflection due to dead load. There won't be any noticeable camber in either of the concrete systems; however, tendons in the post tensioned slab will be draped within the slab in such a way that it will act as if the slab was cambered. Both of the concrete systems would require shoring until the forms are stripped and after the forms are pulled, re-shoring may be required.

Floor Weight

Assuming only the weight of the framing materials included in each floor system, the two composite steel systems have the lightest framing systems. Each assume a metal deck (with 3¼" lightweight concrete) to have a dead weight of 48psf acting over a 25,000sf floor plate. RAM Structural System provided a detailed takeoff for all beams and columns in each steel framing system. For shear studs, I assumed an equivalent weight of 10 lbs of steel for each stud.

Existing Framing: (48psf)x(25000sf) + (57kips) + (1043)x(10 lbs)/1000 = 1267k
Alternative Framing: (48psf)x(25000sf) + (63kips) + (1079)x(10 lbs)/1000 = 1274k

For the one way slab with beams and the two way post tension systems, only the self weight of the slab was taken into account. For the Girder-Slab system, only the 2" topping slab and the concrete planks were taken into account for floor weight (D-Beam steel was neglected). All slabs are assumed to be normal weight concrete (150pcf).

One Way Slab: (5/12)x(150pcf)x(25000) = 1563k
Two Way P-T: (11.5/12)x(150pcf)x(25000) = 3600k
Girder-Slab: [(2/12)x(150pcf) + (60 psf plank)]x25000sf = 2125k

All loads only take into account the floor weight (column weight was not considered). However, column take-offs in RAM for each steel system show 87.2k for the existing system and 70k for the alternative system. This difference is great enough to make the alternative system the lighter of the two systems and the lightest system overall.

Floor Weight

Floor Type	Area	Slab (t)	Steel (k)	Other (psf)	Total (k)	Total (k) w/ Columns
Existing	25000	-	57	48	1257	1344
Steel Alternative	25000	-	63.3	48	1263	1333
One Way Slab	25000	5	-	-	1563	-
Two Way PT	25000	11.5	-	-	3594	-
Girder Slab	25000	2	-	60	2125	-

Fire Protection

By code, a 2 hour fire rating is required between all typical floors at the Swedish American Hospital Heart and Vascular Center. For the proposed concrete systems (one way slab with beams and two way post tensioned) a minimum concrete slab depth of 5" will provide the necessary 2 hour fire rating. The two composite steel systems both have 3 ¼" of concrete above the flutes of the metal deck. For composite steel and concrete with metal deck, this depth of concrete will provide a 2 hour fire rating for the floor system. The exposed steel and metal deck must be encased in some type of fire protection to maintain the required fire rating. In this case, spray on fire proofing is acceptable because solid steel shapes are being used for the floor framing, as compared to an open web steel joist system. For the Girder-Slab system, Girder-Slab Technologies maintains that their floor systems provide a minimum 2 hour fire rating with 8" concrete planks. However, all exposed steel D-Beams must be coated in a cementitious fire resistive material or otherwise enclosed in a fire rated assembly.

Tabulating the work required for each assembly, the two concrete systems take no additional work to achieve the required fire rating. The Girder-Slab system includes a two inch topping slab and only the exposed steel D-Beams would need to be coated with a spray on fire protective layer. The two steel composite systems would take the most work and extra material because both the exposed steel shapes and the metal deck would need to be coated with spray on fire protection.

Cost

The cost of a floor system is one of the most important criteria for selecting a floor type. For high rise structures, or structure with repeating typical floors, lowering the cost of a typical floor system will save a large sum of money over the entire building. For the Heart Hospital, Swedish American was willing to pay a larger amount up front for a building that wanted and fit the needs for their current and future plans.

In the October issue of Modern Steel Construction (MSC), Swedish American Heart Hospital project engineers Michael Bolduc and John Thomsen say "to keep up with the constantly evolving 'state-of-the-art' modern medicine and clinical practices" private hospitals are willing to pay a little extra to get "as much future flexibility as possible."

Cost Analysis					
		Cost per ft^2			
Floor Type	Area (sf)	Mat. $/sf	Lab $/sf	Total/Floor	Total $/sf
Composite Steel	25000	$15.35	$6.30	$541,250.00	$21.65
One Way Slab	25000	$5.75	$10.35	$402,500.00	$16.10
Two Way PT	25000	$6.90	$7.90	$370,000.00	$14.80

The values used for the cost analysis were obtained from the 2007 R.S. Means Assemblies manual. All values assume a concrete compressive stress (f'_c) of 3000psi. The typical compressive stress used for this project is 4000psi. This difference could increase the overall cost of each systems based on the ratio of concrete used.

The cost of the Girder-Slab system was not listed in the R.S. Means Assemblies Manual. The cost could be estimated by pricing each individual item included in the system: precast planks, D-Beams, and concrete topping slab. However, a more precise way would be to contact Girder-Slab Technologies and request an approximate cost per square foot for their floor system.

This cost would also depend on location. Girder-Slab Technologies is based out of New Jersey and additional efforts would be need to find a prefabricator in the Chicago, Illinois area capable of completing their design.

Lead Time

Lead time was not a driving factor for this project. This project was phase 2 in a 3 phase construction plan, so the design was already completed and able to be sent to a steel fabricator when Swedish American decided to go forward with phase 2. Also, Swedish American Health System already had a functioning facility before the completion of the new Heart Hospital, so there wouldn't have been any profit loss due to slow construction. However, if lead time was crucial, steel fabrication could have taken 3 to 4 times the duration of concrete. It should be noted that concrete has a very short lead time and can be prepared by numerous companies, whereas, detailed steel fabrication many only be able to be completed by a select few companies nearby.

Constructability

Composite steel systems are typical framing systems used in steel construction and should not pose a major problem for any experienced contractor. Steel erection may require more skill than pouring concrete, but the erection process is much faster compared to concrete structures. Concrete systems can be difficult when workers are laying out rebar. In areas where the concrete is thin, rebar must be arranged and checked carefully to make sure it is at the correct depth and has enough cover. Post Tensioned systems can be problematic even for experienced contractors. Workers must make sure tendons are draped in the proper orientation before the concrete is poured and the tendons are stressed. It can take only one tendon in the wrong orientation to "blow out" the concrete on the outside of the slab. There is also the possibility of a tendon snapping and tearing up a section of concrete. Following the instructions on their website, the Girder-Slab system sounds like a quick and simple construction process. This is believable because of the precast members and cast in place topping slab.

It should be noted, spray on fireproofing necessary for all exposed steel members should not be applied until the concrete slabs have hardened above the beams. This is so the deflection due to the wet concrete weight does not crack the fireproofing on the beams.

Vibration

The vibration of a floor system is proportional to its overall stiffness. Therefore, it's understandable that both the concrete systems would have the smallest vibrations. The Girder-Slab system would experience vibration amplitudes between the concrete systems and the steel composite systems because it has a lower relative stiffness but is heavier than the steel systems. The steel systems would have the highest vibrations of all the systems. This is because they are the lightest and have a smaller stiffness than the concrete systems. To accurately compare the vibrations of each system, detailed calculations and computer models would need to be constructed to analyze each framing system.

Comparison Summary

Floor System Comparison Spreadsheet					
	Existing Composite	Alternative Composite	One Way Slab w/ Beams	Two Way PT Slab	Girder Slab
Total Depth	30.25"	33.25"	19"	11.5"	10"
Slab Thickness	6.25"	6.25"	5"	11.5"	2"
Weight (Materials)	48 psf	48 psf	62.5 psf	144 psf	85 psf
Max Span	32'	40.5'	11.5' (Slab)	32'	22.625'
Deflection	1.03"	1.2"	0.72"	N/A	1.05"
Cost per Floor	$541,250	$541,250.00	$402,500	$370,000	Contact Company
Fireproofing	Spray On	Spray On	None	None	Spray On
Lead Time	Long	Long	Short	Short	Contact Company
Constructability	Easy	Easy	Medium	Medium	Easy
Vibration	Average	Average	Above Average	Above Average	Average
Column Changes	None	Remove Columns	May Need Columns	None	Add Columns
Lateral Effects	None	None	Shear Walls	Shear Walls	None
Feasible Solution	Yes	Yes	No	Yes	No

Appendix A

- RAM Printouts for Existing Composite Steel Framing System

Gravity Beam Design Takeoff

RAM Steel v11.0
DataBase: Ram Model - Existing
Building Code: IBC

Page 2/3
10/23/07 14:04:41
Steel Code: AISC LRFD

Steel Grade: 50

Floor Type: Typical
Story Levels 1 to 3
Steel Grade: 50

SIZE	#	LENGTH (ft)	WEIGHT (lbs)
W8X10	32	301.52	3037
W10X12	9	127.04	1530
W12X16	10	178.75	2865
W14X22	5	116.11	2564
W16X26	10	238.32	6228
W16X31	4	81.00	2516
W18X35	6	192.00	6729
W18X40	14	429.25	17236
W21X44	3	77.00	3406
W24X55	5	160.00	8874
W24X62	1	32.00	1993
	99		**56979**

Total Number of Studs = **1043**

Gravity Column Design TakeOff

RAM Steel v11.0
DataBase: Ram Model - Existing
Building Code: IBC

10/23/07 14:44:56
Steel Code: AISC LRFD

Steel Grade: 50

I section

Size	#	Length (ft)	Weight (lbs)
W14X43	17	491.6	21076
W14X48	3	93.3	4477
W14X61	8	278.3	16951
W14X68	1	39.2	2665
W14X74	1	39.2	2905
W14X82	7	256.6	20958
W14X90	3	111.7	10068
W14X99	1	39.2	3878
W14X109	1	39.2	4264
	42		87242

Gravity Beam Design

RAM Steel v11.0
DataBase: Ram Model - Existing
Building Code: IBC

10/23/07 14:23:15
Steel Code: AISC LRFD

Floor Type: Typical **Beam Number = 92**

SPAN INFORMATION (ft): I-End (-20.73,81.80) J-End (1.90,59.18)

Beam Size (Optimum) = W18X40 Fy = 50.0 ksi
Total Beam Length (ft) = 32.00

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Concrete thickness (in)	3.25	3.25
Unit weight concrete (pcf)	110.00	110.00
f'c (ksi)	4.00	4.00
Decking Orientation	perpendicular	perpendicular
Decking type	USD 3" Lok-Floor	USD 3" Lok-Floor

beff (in)	=	96.00	Y bar(in)	=	18.28
Mnf (kip-ft)	=	702.90	Mn (kip-ft)	=	493.03
C (kips)	=	159.16	PNA (in)	=	14.00
Ieff (in4)	=	1399.80	Itr (in4)	=	2128.79
Stud length (in)	=	5.00	Stud diam (in)	=	0.75

Stud Capacity (kips) Qn = 15.9
\# of studs: Max = 64 Partial = 20 Actual = 20
Number of Stud Rows = 1 Percent of Full Composite Action = 26.98

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	CLL
1	0.000	0.905	0.735	0.905	19.3%	Red	0.453
	32.000	0.905	0.735	0.905			0.453
2	0.000	0.040	0.040	0.000	---	NonR	0.000
	32.000	0.040	0.040	0.000			0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 36.85 kips 0.90Vn = 152.24 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	PreCmp+	1.2DL+1.6LL	211.8	16.0	0.0	1.00	0.90	294.00
	Init DL	1.4DL	139.0	16.0	---	---		
	Max +	1.2DL+1.6LL	294.8	16.0	---	---	0.85	419.07
Controlling		1.2DL+1.6LL	211.8	16.0	0.0	1.00	0.90	294.00

REACTIONS (kips):

	Left	Right
Initial reaction	19.65	19.65
DL reaction	15.12	15.12
Max +LL reaction	11.69	11.69
Max +total reaction (factored)	36.85	36.85

DEFLECTIONS: (Camber = 3/4)

Initial load (in)	at	16.00 ft	=	-1.031	L/D =	373
Live load (in)	at	16.00 ft	=	-0.425	L/D =	904
Post Comp load (in)	at	16.00 ft	=	-0.523	L/D =	734
Net Total load (in)	at	16.00 ft	=	-0.804	L/D =	478

Appendix B

- RAM Printouts for Alternative Composite Steel Framing System

Gravity Beam Design Takeoff

RAM Steel v11.0
DataBase: RAM Alternative Floor Plan — 10/23/07 15:02:41
Building Code: IBC — Steel Code: AISC LRFD

STEEL BEAM DESIGN TAKEOFF:

Floor Type: Typical
Story Levels 1 to 3
Steel Grade: 50

SIZE	#	LENGTH (ft)	WEIGHT (lbs)
W8X10	30	284.52	2866
W10X12	9	127.04	1530
W12X16	10	178.75	2865
W14X22	5	116.11	2564
W16X26	10	238.32	6228
W18X35	6	163.75	5739
W18X40	14	429.25	17236
W21X44	3	77.00	3406
W21X50	2	64.00	3201
W24X55	5	160.00	8874
W24X62	1	32.00	1993
W27X84	2	81.00	6836
	97		**63338**

Total Number of Studs = **1079**

Gravity Column Design TakeOff

RAM Steel v11.0
DataBase: RAM Alternative Floor Plan — 10/23/07 15:22:06
Building Code: IBC — Steel Code: AISC LRFD

Steel Grade: 50

I section

Size	#	Length (ft)	Weight (lbs)
W14X43	8	213.3	9144
W14X48	3	93.3	4477
W14X53	1	26.7	1415
W14X61	4	145.0	8831
W14X68	2	66.7	4537
W14X82	4	145.0	11840
W14X90	6	223.3	20136
W14X120	2	78.3	9408
	30		69788

Gravity Beam Design

RAM Steel v11.0
DataBase: RAM Alternative Floor Plan
Building Code: IBC

10/23/07 15:02:
Steel Code: AISC LRF

Floor Type: Typical **Beam Number = 126**

SPAN INFORMATION (ft): I-End (32.53,89.80) J-End (61.16,61.16)
Beam Size (Optimum) = W27X84 Fy = 50.0 ksi
Total Beam Length (ft) = 40.50

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Concrete thickness (in)	3.25	3.25
Unit weight concrete (pcf)	110.00	110.00
f'c (ksi)	4.00	4.00
Decking Orientation	perpendicular	perpendicular
Decking type	USD 3" Lok-Floor	USD 3" Lok-Floor

beff (in)	=	117.00	Y bar(in)	=	23.42
Mnf (kip-ft)	=	1864.28	Mn (kip-ft)	=	1438.74
C (kips)	=	318.32	PNA (in)	=	20.27
Ieff (in4)	=	5137.87	Itr (in4)	=	7365.54
Stud length (in)	=	5.00	Stud diam (in)	=	0.75

Stud Capacity (kips) Qn[1] = 15.9 Qn[2] = 17.0
\# of studs per stud segment: Max = 66,27,24
Partial = 20,11,10
Actual = 20,11,10
Number of Stud Rows = 2 Percent of Full Composite Action = 25.67

POINT LOADS (kips):

Dist	DL	CDL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%	CLL
22.625	19.65	16.21	18.33	37.1	0.00	0.00	0.0	0.00	Snow	9.16
32.000	7.82	6.47	7.20	37.1	0.00	0.00	0.0	0.00	Snow	3.60

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	CLL
1	0.000	0.450	0.366	0.450	37.1%	Red	0.225
	32.000	0.450	0.366	0.450			0.225
2	32.000	0.900	0.731	0.900	37.1%	Red	0.450
	40.500	0.900	0.731	0.900			0.450
3	0.000	0.375	0.305	0.375	37.1%	Red	0.188
	40.500	0.375	0.305	0.375			0.188
4	0.000	0.084	0.084	0.000	---	NonR	0.000
	40.500	0.084	0.084	0.000			0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 83.05 kips 0.90Vn = 331.61 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	PreCmp+	1.2DL+1.6LL	737.5	22.6	0.0	1.00	0.90	915.00
	Init DL	1.4DL	493.6	22.6	---	---		
	Max +	1.2DL+1.6LL	906.9	22.6	---	---	0.85	1222.93
Controlling		1.2DL+1.6LL	737.5	22.6	0.0	1.00	0.90	915.00

DEFLECTIONS: (Camber = 3/4)

Initial load (in)	at	20.86 ft	=	-1.152	L/D =	422
Live load (in)	at	20.86 ft	=	-0.447	L/D =	1086
Post Comp load (in)	at	20.86 ft	=	-0.581	L/D =	837
Net Total load (in)	at	20.86 ft	=	-0.983	L/D =	494

Appendix C

- Hand Calculations for One Way Concrete Slab with Beams and Girders

Alternative: 1 WAY CONC SLAB W/ BEAMS & GIRDERS

* Assuming Typical Floor (2nd, 3rd, 4th, 6th, 7th)

Superimposed DL = 17 psf
LL = 80 psf

* Assume 5" thick slab (Normal Weight Conc = 150 pcf)
Self weight $= \frac{5}{12}(150) = 62.5$ psf

$1.2D + 1.6L$
$1.2(17 + 62.5) + 1.6(80) = 224$ psf

Typical BEAM / GIRDER SPANS: 22'-7½"
32'-0"

Typical SLAB SPANS: 11'-4"

DEPTH OF BEAMS / SLAB [ACI 318-05, TABLE 9.5(a)]
for Deflection Criteria

BEAMS:

32' SPAN ends cont $\Rightarrow \frac{l}{21} = \frac{32(12)}{21} = 18.3''$ $\Rightarrow$ USE h=19" for 32' SPANS

22' SPAN ends cont $\Rightarrow \frac{l}{21} = \frac{22.6(12)}{21} = 12.9'' \le 16''$ ✓OK

SLAB

11.33' SPAN ends cont $\Rightarrow \frac{l}{28} = \frac{11.33(12)}{28} = 4.86'' \le 5''$ ✓OK

ACI MOMENT COEFFICIENTS

* Assume INTERIOR SPANS

$$M^+ = \frac{w_u l_n^2}{16}, \quad M^- = \frac{w_u l_n^2}{11}$$

$b_{eff} = \begin{cases} \frac{SPAN}{4} = 8' \\ CLR\ SPA = 11.33' \\ 8h = 3.33' \ (40") \end{cases}$

* Assume h = 5"

- 32' BEAM * Assume $l_n = 32'$

Width = 11.33'

$$M^+ = \frac{(224)(11.33')(32')^2}{16 \cdot 1000} = \boxed{162.4 \text{ k-ft}}$$

$$M^- = \frac{(224)(11.33')(32^2)}{11 \cdot 1000} = \boxed{236.3 \text{ k-ft}}$$

$\boxed{M_{u\,BEAM\ SELF\ WT} = 19.9 \text{ kft}}$

Negative Moment:

$M_u = 256.3$ k-ft

$a = \frac{A_s f_y}{.85 f'_c b}$, $M_u = \phi A_s f_y (d - \frac{a}{2}) \Rightarrow \phi A_s f_y \left[d - \left(\frac{A_s f_y}{1.7 f'_c b}\right)\right]$

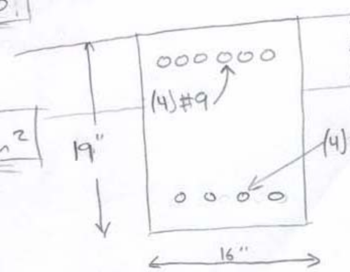
* Assume d = 16.5

$A_s = 3.98$ in² => use (4)#9 $\boxed{A_s = 4.0}$

Positive Moment

$M_u = 182.3$ k-ft

$A_s = 2.56$ in² => use (4)#8 $\boxed{A_s = 3.16 \text{ in}^2}$



- 22'-7½" BEAM * Assume $l_n = 22.625'$

width = 11.33'

h=5"
d=13.5"

$b_{eff} = \begin{cases} \frac{SPAN}{4} = 68" \\ CLR = 11.33' \\ 8h = 40" \end{cases}$

$$M^+ = \frac{(224)(11.33')(22.625^2)}{16 \cdot 1000} = 81.3 \text{ k-ft}$$

$$M^- = \frac{(224)(11.33')(22.625^2)}{11 \cdot 1000} = 118.3 \text{ k-ft}$$

$\boxed{M_{u\,BEAM\ SELF\ WT} = 5.8 \text{ k-ft}}$

Negative Moment:

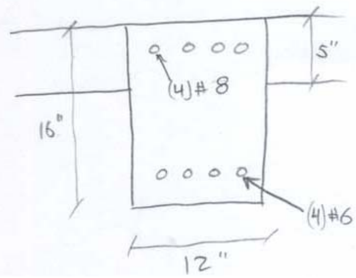
$M_u = 124.1$ k-ft

$A_s = 2.34$ in² => use (4)#8 $\boxed{A_s = 3.16 \text{ in}^2}$

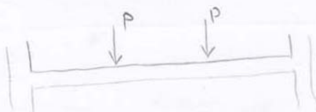
Positive Moment:

$M_u = 87.1$ k-ft

$A_s = 1.44$ in² => use (4)#6 $\boxed{A_s = 1.76 \text{ in}^2}$



- 32' GIRDER



$$P = (241.33\text{ sf})(224\text{ psf}) + \underbrace{\left(\tfrac{11" \times 16"}{144}\right)(22.625')(150\text{ pcf})}_{\text{SELF WT OF BEAM SPANNING INTO GIRDER}}$$
$$= 54.1^k + 4.15^k$$
$$= 58.2^k$$

$$b_{eff} = \begin{cases} \frac{SPAN}{4} = 8' \\ CLR = 22' \\ 8h = 40" \end{cases}$$

$$M^+ = .111(P)(L) \approx 207^{k\text{-ft}}$$
$$M^- = .222(P)(L) = 414^{k\text{-ft}}$$

Negative moment:

$M_u = 434^{k\text{-ft}}$

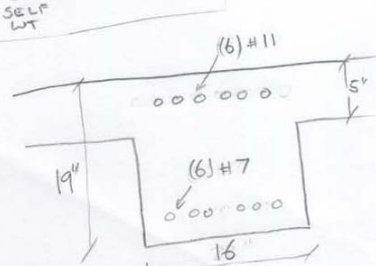
$A_s = 7.97 \Longrightarrow \boxed{(6)\#11 \quad A_s = 8.46}$

$\boxed{M_{u_{BEAM\ SELF\ WT}} = 20^{k\text{-ft}}}$

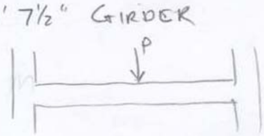
Positive Moment:

$M_u = 227$

$A_s = 3.2\ in^2 \Longrightarrow \boxed{(6)\#7 = 3.6}$



- 22' 7½" GIRDER



$$P = (224\text{ psf})(309\text{ sf}) + \left(\tfrac{11" \times 16"}{144"}\right)(27.3')(150\text{ pcf})$$
$$P = 69.22^k + 5$$
$$= 74.22^k$$

$\boxed{M_{u_{BEAM\ SELF\ WT}} = 7.8^{k\text{-ft}}}$

$$M^+ = \frac{PL}{8} = 210^{k\text{-ft}}$$
$$M^- = \frac{PL}{8} = 210^{k\text{-ft}}$$

Negative moment:

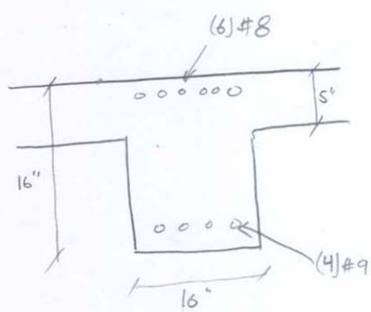
$M_u = 218\ k\text{-ft}$

$A_s = 4.63\ in^2 \Longrightarrow \boxed{(6)\#8 \quad A_s = 4.74}$

Positive Moment

$M_u = 218\ k\text{-ft}$

$A_s = 4.0\ in^2 \qquad \boxed{(4)\#9 \quad A_s = 4.0\ in^2}$



CHECK SLAB THICKNESS (5")

- 1' strip width , 11.3125' SPAN

$\left(\frac{5"}{12"}\right)(1')150 + (17\,psf)(1') = 80\,plf$

$m^+ = \frac{(80.0)(11.3125^2)}{16 \cdot 1000} = 0.64^{k-ft} \Longrightarrow \boxed{A_s = \#4 @ 12" o/c}$

$m^- = \frac{80.0(11.3125^2)}{11 \cdot 1000} = 0.93^{k-ft} \Longrightarrow \boxed{A_s = \#4 @ 12" o/c}$

$a = \frac{(.2)(60)}{.85(4)(12")} = 0.294"$

$\phi m_n = .9(.2)(60)(3.5" - \frac{.294}{2})$
$= 36.2^{k-in}$
$(3.0^{k-ft}) > 0.93^{k-ft}$ ✓OK

DEFLECTIONS

$\omega = 1.2(D) + 1.6(L)$
$= .224\,ksf$ x

$E_c = 57000\sqrt{4000}/1000$
$= 3605\,ksi$

16x16 $I_{Bm\ 32'} = K\left(\frac{b_w h^3}{12}\right)$

$\left.\begin{array}{l}\frac{b_e}{b_w} = \frac{40"}{16} = 2.5 \\ \frac{t}{h} = 5/16 = .3125\end{array}\right\} \rightarrow K = 1.49$

$I = 1.49\left(\frac{16 \times 16^3}{12}\right) = \underline{8137}\ in^4$

$\delta = \frac{(.224)(11.3')(32^4)\,1728}{384\,(3605)(8137)} = 0.41"$ ✓

* For FIXED - FIXED END CONNECTIONS

12x16 $I_{Bm\ 22'} = K\left(\frac{b_w h^3}{12}\right)$

$\left.\begin{array}{l}\frac{b_e}{b_w} = \frac{40}{12} = 3.3 \\ t/h = .3125\end{array}\right\} \rightarrow K = 1.67$

$I = 1.67\left(\frac{12 \times 16^3}{12}\right) = \underline{6840}\ in^4$

$\delta = \frac{(.224)(11.3)(22^4)(1728)}{384(3605)(6840)} = 0.12"$ ✓OK

* FOR FIXED- FIXED end connection

22x16 $I_{GR\ 32'} = K\left(\frac{b_w h^3}{12}\right)$

$\left.\begin{array}{l}\frac{b_e}{b_w} = \frac{40}{22} = 1.82 \\ t/h = .3125\end{array}\right\} \rightarrow K = 1.36$

$I = 1.36\left(\frac{22 \times 16^3}{12}\right) = \underline{10213\ in^4}$

$\delta = \frac{.008\,(P)(l^3)}{EI} = \frac{.008(58.2)(32^3)(1728)}{3605\,(10213)}$
$= 0.72"$

* For Fixed - Fixed end connections

16x16 $I_{GR\ 22'} = K\left(\frac{b_w h^3}{12}\right)$

$\frac{b_e}{b_w} = 2.5$
$t/h = .3125$ $I = 8137$

$\delta = \frac{.008(74.2)(22^3)1728}{3605\,(8137)} = 0.404"$ ✓OK

APPENDIX D

- Hand Calculations for Design of a Two Way Post Tensioned Slab

CLIENT: POST TENSIONING MANUAL
SUBJECT: 6th Ed.
DATE
BY
CHECKED BY

DESIGN OF A TWO-WAY P-T SLAB

$f'_c = 4000$ LL = 80 psf
$f_y = 60{,}000$ Partition Load = 10 psf
$\rho = 150$ pcf conc DL = 12 psf
$f_{pu} = 270{,}000$ psi

TWO WAY SLAB = SPAN / DEPTH

$$\frac{L}{45} = \frac{32'(12)}{45} = 8.5'' \Rightarrow 9''\text{ SLAB}$$

$\Rightarrow$ USE 11.5" per ACI TABLE 9.5(c)

LOADS

$$DL = \frac{9}{12}(150) = 144\text{ psf} + 17\text{ psf} = 161$$

$$1.2(DL) = 193\text{ psf}$$

LL = 32' x 32' bays

$$80 \times \left(1 - \frac{.08(32 \times 32 - 193)}{100}\right) = 80(.3)$$

≤ 40%, use 40%

$$80(.4) = 32\text{ psf}$$

22' x 22' bays

$$80 \times \left(1 - \frac{.08(22^2 - 193)}{100}\right) = 65.9\text{ psf}$$

$$TL_{32'} = 193 + 1.6(32) = 244\text{ psf}$$

$$TL_{22} = 193 + 1.6(58.6) = 291\text{ psf}$$

* LL 18 x 27 Bays

$$80\left(1 - \frac{.08(18 \times 27 - 193)}{100}\right) = 61.3\text{ psf}$$

LOAD BALANCING

Tendons balance 80% of SLAB WEIGHT

$$(.8)(.144) = 0.115\text{ ksf}$$

MAX TENDON SAG: 11.5" - 1" - 1" = 9.5"

$$F_e = \frac{w_{BAL} L^2}{8a} = \frac{(.115)(32')^2}{8(9.5/12)} = 18.6\text{ k/ft}$$

* Assume 1/2" Φ, 270 ksi Tendons (14 ksi long term stresses)

$.153 \times (.7 \times 270 - 14) = 26.8^K$ (per tendon)

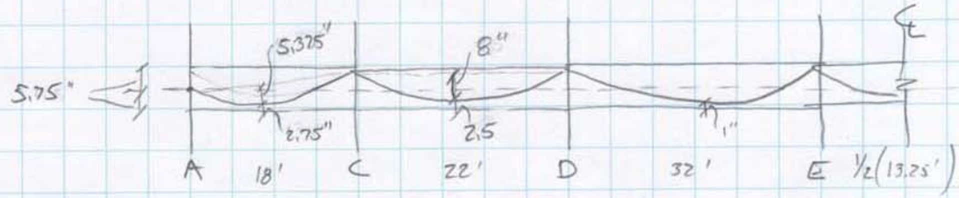
→ FOR 32' BAY:

$32 \times \left(\frac{18.6}{26.8}\right) = 22.5 = \boxed{\text{USE 23 TENDONS}}$

$F_e = 23\left(\frac{26.8}{32}\right) = 19.3\ \text{K/ft}$

$F/A = 19.3/(11.5 \times 12) = 0.14\ \text{ksi}$

TENDON PROFILE: (COLUMN LINE 4)



SPAN CD (22')

SPAN CD: 22'

$w_{BAL} = \frac{8 F_e a}{L^2} = \frac{8(19.3)(8/12)}{22^2} = \underline{0.213\ \text{ksf}}$

⇒ Adjust Tendon profile in span AC

$a = \frac{w_{BAL} L^2}{8 F_e} = \frac{0.213\ (18^2)}{8(19.3)} \times 12 = 5.4'' \leq 6.38''\ \checkmark OK$

MIDSPAN CGS:

$\frac{(5.75 + 10.5)}{2} - 5.4 = 2.72'' \Rightarrow 2.75''$

ACTUAL SAG IN SPANS AC (END SPANS)

$\frac{(5.75 + 10.5)}{2} - 2.75 = 5.375$

BALANCE LOAD IN SPANS

SPAN AC: 18'

$w_{BAL} = \frac{8 F_e a}{L^2} = \frac{8(19.3)(5.375/12)}{18^2} = \underline{0.213\ \text{ksf}}$

SPAN DE: 32'

$w_{BAL} = \frac{8(19.3)(10.5/12)}{32^2} = \underline{0.12\ \text{ksf}}$

NET LOAD CAUSING BENDING — TOTAL UNFACTORED LOAD

SPAN DE (32'): $W_{net} = .193 - .12 = 0.073$ ksf

SPAN AC (18'): $W_{net} = .222 - .213 = 0.009$ ksf

SPAN CD (22'): $W_{net} = .222 - .213 = 0.009$ ksf

EQUIVALENT FRAME PROPERTIES

COLUMN STIFFNESS (APPROX METHODS)

$$K_c = \frac{4EI}{L-2h}$$

* Assume All Columns = 16" x 16"
* Assume h = 11' = 132"

$$I_{col} = \frac{16 \times 16^3}{12} = 5461 \text{ in}^4 \qquad \frac{E_{COL}}{E_{SLAB}} = 1.0$$

$$K_c = \frac{4(1.0)(5461)}{132" - 2(11.5)} = 200.4 \text{ in}^3$$

TORSIONAL STIFFNESS OF SLAB

$$C = \left[1 - .63\frac{x}{y}\right]\frac{x^3 y}{3}$$
$$= \left[1 - .63\left(\frac{11.5}{16}\right)\right]\left(\frac{11.5^3 \cdot 16}{3}\right)$$
$$= 4438$$

$$K_t = \frac{9CE}{L_2\left(1 - c_2/L_2\right)^3} = \frac{9(4438)(1.0)}{(27.3' \times 12)\left(1 - 1.33/27.3\right)^3} = \frac{22510}{282}$$

$$= 141.6 \text{ in}^3$$

$$\frac{1}{K_{ec}} = \frac{1}{\Sigma K_T} + \frac{1}{\Sigma K_c}$$

$$K_{ec} = \left[\frac{1}{(2 \times 141.6)} + \frac{1}{(2 \times 200)}\right]^{-1} = 166 \text{ in}^3 \quad \text{FOR ALL COLUMNS}$$

SLAB STIFFNESS

width = 27.3'

$$K_S = \frac{4EI}{L_1 - \left(\frac{c_1}{2}\right)} \qquad K_c = 166 \text{ in}^3$$

$$K_{S_{22'}} = \frac{4(1.0)(27.3)(11.5^3)}{22'(12") - (16/2)} = 649 \text{ in}^3$$

$$K_{S_{32'}} = \frac{4(1.0)(27.3)(11.5^3)}{32'(12") - (16/2)} = 442 \text{ in}^3$$

$$K_{S_{18'}} = \frac{4(1.0)(27.3)(11.5^3)}{18(12) - 8} = 798 \text{ in}^3$$

DISTRIBUTION FACTORS

A $\omega = 0.009$ C $\omega = .009$ D $\omega = 0.073$ E

DF	.82	.495	.40	.51	.35	.31
FEM	-.513	+.513	-.726	+.726	-5.55	+5.55
	+.395	+.101	+.082	+2.364	+1.606	-1.665
	+.05	+.197	+1.182	+.041	-.832	+.803
	-.038	-.655	-.531	+.387	+.263	-.241
TOTAL	-.106	+0.05	+.007	3.52	-4.5	4.45

MAX

MAX NEGATIVE MOMENT

$$\text{AT COL FACE (D)} = -M_{MAX} + \frac{V_c}{3}$$

$$= -4.5 + \frac{1}{3}\left(\frac{.073 \times 32}{2}\right)\left(\frac{16}{12}\right)$$

$$-M_{MAX} = -4.00 \text{ K-ft}$$

$$S = \frac{bh^2}{6} = \frac{16 \times 11.5^2}{6} = 353 \text{ in}^3$$

F/A

$$\delta_{t,b} = -\delta_{pc} \pm \frac{M}{S} = -.145 \pm \frac{(4.00 \times 12)}{353}$$

$$= (.028), (-.604) \text{ ksi}$$

ALLOWABLE TENSION: (ACI 318-05)

$6\sqrt{f'_c} = 0.38$ KSI > 0.028 KSI ✓OK

ALLOWABLE COMPRESSION: (under total load)

$.6 f'_c = .6(4000) = 2.4$ KSI > -.004 KSI ✓OK

(under sustained loads)

$.45 f'_c = .45(4000) = 1.8$ KSI ≥ -.004 KSI ✓OK

MAX POSITIVE MOMENT

SPAN DE: $M = .073(32^2)/8 - 5.55 = \boxed{3.80 \text{ K-ft}}$ ← CONTROLS

SPAN CD: $M = .009(22^2)/8 - .726 = -0.183$ K-ft

$$f_{t,b} = -f_{pc} \pm \frac{M_{net}}{S} = -.145 \pm \frac{12(3.80)}{330}$$

$f_{t,b} = -0.011$ ksi, -0.27 ksi

COMPRESSION AT TOP = 0.27 KSI ≤ 1.8 KSI ✓OK

≤ 2.4 KSI

Tension AT Bottom = -.011 ≤ 0.474 ✓OK

≤ 0.38

BONDED REINFORCEMENT IN TENSION

$f_T = 2\sqrt{f'_c} = 2\sqrt{4000} = 0.126$

0.126 ksi ≥ T = -0.011 ksi

⇒ NO POSITIVE REINFORCEMENT REQUIRED

FLEXURAL CAPACITY

SPAN DE (32') : $FEM = \frac{w_{BAL} l^2}{12} = \frac{0.12\ 32^2}{12} = 10.24$ k-ft

SPAN CD (22') : $\frac{.213\,(22^2)}{12} = 8.6$ k-ft

SPAN AC (18') : $\frac{.213\,(18^2)}{12} = 5.75$ k-ft

	A		C		D		E
DF	.82	.49	.40	.51	.35	.31	
FEM	-5.75	5.75	-8.6	8.6	-10.24	+10.24	
	+4.71	+1.40	-1.14	+0.84	+.57	-3.00	
	+.70	+2.35	+.42	+.57	-1.50	+.28	
	-.57	-1.36	-1.11	+.47	+.33	-0.08	
	-.68	-.28	+.23	-.55	-.04	+.16	
	+.56	+.07	+.02	+.30	+.21	-.05	
SUM	-1.08	7.88	-7.9	10.23	-10.67	7.55	

$M_{BAL} = M_1 + M_2 \quad ; \quad M_2 = M_{BAL} - M_1$

Secondary Moments : (From Tendons)

@ A : $M_2 = 1.03 - 19.3\left(\frac{0}{12}\right) = 1.03$ k-ft

@ C_{ext} $M_2 = 7.88 - 19.3\left(\frac{5.75-1}{12}\right) = 0.24$ k-ft

@ C_{int} $= 7.9 - 19.3\left(\frac{4.75}{12}\right) = 0.26$ k-ft

@ D_{ext} $= 10.23 - 19.3\left(\frac{4.75}{12}\right) = 2.59$ k-ft

@ D_{int} $= 10.67 - 19.3\left(\frac{4.75}{12}\right) = 3.03$ k-ft

@ E_{ext} $= 7.55 - 19.3\left(\frac{4.75}{12}\right) = -0.09$ k-ft

FACTORED LOAD MOMENTS : 1.2(DL) + 1.6(LL)

SPAN DE (32') = $.244 \times 32^2/12 = 20.8$ k-ft
SPAN CD (22') : $.291 \times 22^2/12 = 11.7$ k-ft
SPAN AC (18') = $.291 \times 18^2/12 = 7.8$ k-ft

CHECKED BY

	A		C		D		E
DF	.82	.49	.40	.51	.35	.31	
FEM	-7.8	+7.8	-11.7	+11.7	-20.8	+20.8	
	+6.4	+1.91	+1.56	+4.64	+3.18	-6.24	
	+.95	+3.2	+2.32	+.78	-3.12	+1.59	
	-.78	-2.7	-2.21	+1.19	+.82	-.48	
	-1.35	-.39	+.59	-1.10	-.24	+.41	
	+1.11	-.10	-.08	+0.68	+.47	-.12	
TOTAL	-1.47	9.72	-9.52	17.89	-19.69	15.96	

DESIGN MOMENTS @ COL. FACE

	A		C		D		E
FACTORED MOMENT	-1.47	-9.72	-9.52	-17.89	-19.69	-15.96	
Secondary Moments	+1.03	+0.24	+0.26	+2.59	+3.03	-0.09	
MOMENT @ COL ℄	-0.44	-9.48	-9.26	-15.30	-16.66	-16.05	

CALCULATE/DESIGN FOR MIDSPAN MOMENTS

SPAN AC (18') : $V_{ext} = \frac{.291 \times 18}{2} - \frac{(9.48 - .44)}{18}$

$= 2.12$ K/ft ← governs

$V_{int} = 3.12$ K/ft

Point of ZERO shear from EXT. COL.

$X = 2.12/.291 = \underline{7.28}$ ft from ℄ of Ext Col

END SPAN POSITIVE MOMENT

$M_{max} = .5(2.12)(7.28) - .44 = \underline{7.27}$ ft-k/ft

SPAN CD (22') : $V = .291 \frac{(22')}{2} = \underline{3.46}$ K/ft

$M_{max} = .5(3.46)(11) - 15.30 = \underline{3.73}$ ft-k/ft

SPAN DE (32') : $V = .244 \frac{(32)}{2} = \underline{3.90}$ K/ft

$M_{max} = .5(3.90)(16) - 16.66 = \underline{\underline{14.54}}$ ft-k/ft

FLEXURAL STRENGTH

$A_s = .00075\ A_{cf}$

$= .00075\ (11.5)\left(\frac{22+32}{2}\right) \times 12 = 2.79\ in^2$

$\Rightarrow$ USE (5) #7 $\quad A_s = 3.0\ in^2 \geq 2.79\ in^2$

BAR LENGTH: $2 \times \frac{\left(32 - \frac{16}{12}\right)}{6} + \frac{16}{12} = 11.56'$ $\quad (11'-7'')$

ONE FOOT STRIP:

$A_s = 5 \times \frac{.60}{27.3'} = 0.11\ in^2/ft$

CALC DESIGN STRESS IN TENDON:

$f_{ps} = f_{pe} + 10000 + \frac{f'_c}{300\rho_p}$

$\rho_p = \frac{A_{ps}}{bd} = \frac{23\,(.153)}{(27.3)(12)(10.5)} = 0.0010$

$f_{se} = .7\,(270) - 14 = 175\ ksi$

$f_{ps} = 175 + 10 + \frac{4}{300\,(.001)}$

$= 198.3\ ksi$

f_{ps} NOT GREATER THAN:

$f_{py} = .85\ f_{pu} = 230\ ksi \geq 198.3$ ✓OK

$f_{se} + 30 = 205 \geq 198.3$ ✓OK

Tendon — $F_{su} = \frac{198.3 \times .153 \times 23}{27.3'} = 25.5\ k/ft$

Rebar — $F_u = 60 \times .11 = 6.6\ k/ft$

$F_{TOTAL} = 25.5 + 6.6 = 32.1\ k/ft$

Depth of compression block (a)

$$a = \frac{F}{.85 b f'_c} = \frac{32.1}{.85(12)(4)} = 0.79 \text{ in}$$

$$\varepsilon_t = (10.5 - .21) \times \frac{.003}{(.79/.85)} = 0.033$$

* Assume Reinforcing Bars & Tendons in SAME layer

$$d - a/2 = \frac{9.5 - \left(\frac{.79}{2}\right)}{12} = 0.76 \text{ ft}$$

MOMENT CAPACITY AT COL CENTERLINE

$$\phi M_n = .9(.76)(32.1) = 21.9 \text{ k-ft} \geq 16.66 \quad \checkmark \text{OK}$$

=> Redistribution of excess negative moment

Permissible change in negative moment

$$1000\,\varepsilon_t = 1000(.033) = 33\% \geq 20\% \text{ MAX}$$

AVAILABLE INCREASE IN NEG mom.

$$.2(16.66) = 3.33 \text{ ft-k/ft}$$

ACTUAL INCREASE IN NEG MOMENT CAPACITY

$$21.9 - 16.66 = 5.24 \leq 3.33 \quad \text{☒ NO GOOD}$$

$$\rightarrow \text{CAN ONLY INCREASE } 3.33 \text{ ft-k/ft}$$

SPAN DE : MIN DESIGN POSITIVE MOMENT
(32')

$$14.54 - 3.33 = 11.21 \text{ ft-k/ft}$$

CAPACITY AT MIDSPAN

$$A_{ps} f_{ps} = 25.5 \text{ k/ft}$$

$$a = \frac{25.5}{.85(12)(4)} = 0.625''$$

$$d - \frac{a}{2} = \frac{\left(9.5'' - \frac{.625}{2}\right)}{12} = 0.76 \text{ ft}$$

MOMENT CAPACITY @ CENTER SPAN

$$\phi M_n = .9(25.5)(.76) = \underline{\underline{17.6\ ^{k\text{-}ft}/_{ft}}} \geq 11.21\ ^{k\text{-}ft}/_{ft} \quad \underline{\underline{\checkmark OK}}$$

END SPAN:

(18')

$$d - \frac{a}{2} = \left[(11.5 - 2.75) - \frac{.625}{2}\right]/12 = 0.70\ ft$$

CHECK POSITIVE MOMENT:

$$\phi M_n = .9(25.5)(.70) = \underline{\underline{16.1}} \geq 7.29 \quad \underline{\checkmark OK}$$

EXTERIOR COLUMNS

$$A_{s_{min}} = .00075(27')(12")(11.5) = 2.79\ in^2$$

$$\rightarrow \text{USE } (5)\ \#7, \quad \underline{\underline{A_s = 3.0\ in^2}}$$

$$A_s = 5\left(\frac{.60}{27}\right) = 0.11$$

$$A_s f_y = .11(60) = 6.6\ ^k/_{ft}$$

$$\rho_p = \frac{29(.153)}{27.3(12)(5.75)} = 0.00187$$

$$f_c = 175 + 10 + \frac{4}{(.00187 \cdot 300)} = 192\ ksi$$

$$A_{ps} f_{ps} = 29(.153)(^{192}/_{27.3}) = 24.75\ k/ft$$

$$a = \frac{24.75 + 6.6}{.85(12)(4)} = 0.77\ in$$

TENDONS: $d - \frac{a}{2} = (5.75 - \frac{.77}{2})/12 = \underline{0.45\ ft}$

REBAR: $d - a/2 = (10.5 - \frac{.77}{2})/12 = \underline{0.84\ ft}$

$$\phi M_n = .9\left[(24.75 \times .45) + (6.6 \times .84)\right]$$

$$= \underline{\underline{15.01\ ^{k\text{-}ft}/_{ft}}} \geq 0.44\ ^{k\text{-}ft}/_{ft}$$

SHEAR CAPACITY

EXTERIOR COLUMN

$V_{u_{ext}} = 2.12\ k/ft$ * Assume EXTERIOR FACADE $= 0.5\ k/ft$

$V_u = [(1.2)(.5) + 2.12] \times 27.3'$
$= 74.25\ K$

Transferred Moment $= 27.3' \times .44 = 12.01\ ft\text{-}k$

Combined Shear Stress @ inside face

$d = .8(11.5) = 9.2''$
$c_1 = 16''$
$c_2 = 16''$
$b_1 = 16'' + \frac{9.2}{2} = 20.6''$
$b_2 = 16 + 9.2 = 25.2''$
$A_c = (2b_1 + b_2)d = [2(20.6) + 25.2]9.2 = 611\ in^2$

$J_c/c = [2(20.6)(9.2)[20.6 + 2(25.2)] + (9.2^3)[2(20.6) + 25.2]/20.6]/6$
$= 4904$

$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{20.6}{25.2}}} = 0.38$

$v_u = \frac{74250}{611} + \frac{.38(12.01 \times 1000 \times 12)}{4904}$
$= 133\ psi$

Permissible Shear Stress
$\phi V_n = \phi V_c / b_o d$

$\phi V_n = .75(4)\sqrt{4000} = 190\ psi \geq 133\ psi$ √OK

CHECK FLEXURAL MOMENT CAPACITY

$A_s = .00075(11.5)(27.3)(12) = 2.79\ in^2$

$\Rightarrow$ use (5) #7, $A_s = 3.0\ in^2$

CALC STRESS IN TENDONS

Eff slab width = $16 + 2(1.5 \times 11.5) = 50.5$ in

* Assume 6 Tendons ARE BUNDLED TOGETHER & Anchored within the column cage.

$\rho_p = \frac{6(.153)}{50.5 \cdot 5.75} = 0.0032$

$f_{ps} = 175 + 10 + \frac{4}{.0032 \times 300}$
$= 189.2$ ksi

Prestress force in critical width
$6(.153)(189.2) = \underline{173.7\ k}$

$A_s f_y = 5(.60)(60) = 180$ K

$A_{ps} f_{ps} + A_s f_y = 173.7 + 180 = \underline{353.7}$ k

$a = \frac{353.7}{.85(4)(50.5)} = 2.06''$

TENDON: $(d_p - a/2) = (5.75 - \frac{2.06}{2})/12 = 0.39$ ft
REBAR: $(d - a/2) = (10.5 - \frac{2.06}{2})/12 = 0.79$ ft

$\phi M_n = .9[(173.7)(.39) + (180)(.79)] = \underline{188.9}$ k-ft

$\gamma_F = 1 - \gamma_v = .62$
$\gamma_F M_u = .62(12.01) = 7.45 \ll 188.9$ ✓OK

INTERIOR COLUMN: 32' SPAN

DIRECT SHEAR: $(3.46 + 3.90)27.3' = 200.9$ K
Moment Transfer: $27.3(16.66 - 15.3) = 37.13$ k-ft

$d = 9.2''$
$c_1 = 16''$
$c_2 = 16''$
$b_1 = b_2 = 25.2$
$A_c = 2(25.2 + 25.2)11.5 = 1159$ in^2

$$J_c/c = \{(25.2)(9.2)[25.2 + 3b_2] + d^3\}/3$$
$$= 8049 \ in^3$$

$$\gamma_v = 1 - \frac{1}{1+\frac{2}{3}\sqrt{b/b_2}} = 1 - \frac{1}{1+\frac{2}{3}\sqrt{1}} = 0.4$$

$$V_u = \frac{200900}{1159} + \frac{(.4)(37.13)(1000)(12)}{8049} = \quad + 34$$
$$= 195.5 \ psi$$

Permissible Shear Stress

$$\phi V_c = \phi\left[4\sqrt{f'c} + .3f_{pc} + \frac{V_p}{b_o d}\right]$$

USE for SQUARE COLS

$$= .75\left[4\sqrt{4000} + .3(140)\right]$$
$$= 221 \ K$$

F/A TENDONS

$$221^k \geq 196^k \quad \checkmark OK$$

check moment transfer:

$$\gamma_v = 1 - .4$$
$$= 0.6$$
$$\gamma_v M_{vf} = .6(37.13) = 22.3 \ ft\text{-}K$$

EFF. SLAB WIDTH $= 16 + 2(1.5 \times 11.5) = 50.5$ in

$$A_{ps} f_{ps} = 173.7 \quad \text{(SAME AS EXT. COL)}$$
$$A_s = .00075(A_{CF})$$
$$= .00075(11.5)\left(\frac{32+22}{2}\right) \times 12 = 2.79 \ in^2$$

$\longrightarrow$ USE (5) #7 $\quad \underline{A_s = 3.0 \ in^2}$

$$A_s f_y = (3.0)(60) = \underline{180 \ K}$$
$$A_{ps} f_{ps} + A_s f_y = \underline{353.7^K}$$

$$a = \frac{353.7}{.85(4)(50.5)} = 2.06''$$

$d - a/2 = \left(10.5 - \frac{2.06}{2}\right)/12 = \underline{0.79\ ft}$

$\phi M_n = .9(353.7)(.79) = 251.5\ ^{k-ft} \geq 22.3\ ^{k-ft}$ /OK

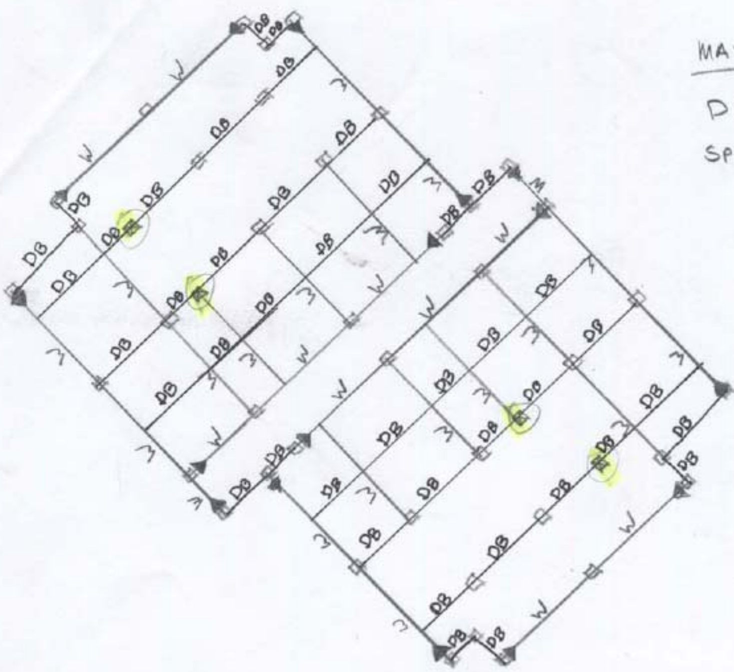
DISTRIBUTION OF TENDONS

- USE 23 TENDONS FOR 1 - 27.3' BAY
  W/ 6 TENDONS DIRECTLY THROUGH THE COLUMN
- SPACE THE REMAINING TENDONS (18) AT 1'-6" O/C.
  (ABOUT 2x slab thickness)
- TENDONS RUNNING ⊥ to TENDONS DESIGNED HEREIN SHOULD BE PLACED IN NARROW BANDS THROUGH COLUMN LINES.
- (14) #4 REBAR CAN BE USED IN PLACE OF (5) #7's. THIS WILL ALLOW FOR A MORE UNIFORM MESH OF REBAR OVER THE ENTIRE SLAB AS COMPARED TO LOCALIZED AREAS OF (5) #7's.

Appendix E

- Sketch of Girder Slab framing plan with additional columns and wide flange beams

ALTERNATIVE: GIRDER-SLAB



MAX SPAN
DB = 22.625' (DB-9)
SPAN = 22.5'

DL = 17 psf
Partition Load = 20 psf
LL = 60 psf
2' conc Topping = 18.33 psf   * Assume Lightweight conc ρ = 110 pcf

42 D-BEAMS
10 INTERIOR WIDE FLANGE BEAMS
4 ADDITIONAL COLUMNS MUST BE ADDED

Live Load Deflection (L/360)
(22.625*12/360) = 0.75"

Actual LL Deflection
= 1.05" from DBeam Spreadsheet
NO GOOD

Appendix F

- D-Beam design spreadsheet provided by Girder-Slab Technologies at girder-slab.com

D-Beam® Calculator Reference Tool

10/29/2007

Project Name: Swedish American Hospital
Job Number:

Design Information

Dead Load =	17	psf
Partition Load =	20	psf
Live Load =	60	psf
Topping Load =	18.33	psf
DB Span =	22.625	ft
Plank Span =	22.5	ft
Grout f'c =	5000	psi
Allowable Δ_{LL} = L /	240	
Allowable Δ_{LL} =	1.13	in

DB Properties

DB Size ------------> DB 9 x 46

Steel Section		Transformed Section	
I_s =	195 in^4	I_t =	356 in^4
S_t =	33.7 in^3	S_t =	68.6 in^3
S_b =	50.8 in^3	S_b =	80.6 in^3
M_{scap} =	84.0 ft-k		
t_w =	0.375 in		
b =	5.75 in		

Live Load Reduction (IBC 00/03/06)

Include LLR ☑		(Check for Yes)
% Reduction =	27.99	%
Reduced Load =	43.2	psf

Initial Load - Precomposite

M_{DL} =	24.5 ft-k	<	84.0 ft-k	OK
Δ_{DL} =	0.40 in			
Δ Ratio = L /	681			
Camber D-Beam ☑	(Check for Yes)			
D-Beam Camber	0.39 in			

Total Load - Composite

M_{sup} =	117.4 ft-k			
M_{TL} =	141.9 ft-k			
S_{REQ} =	56.7 in^3	<	68.6 in^3	OK
Δ_{SUP} =	1.05 in	<	1.13 in	OK
Δ_{TOT} =	1.06 in	= L/ 257		

Superimposed Compressive Stress on Concrete

N value =	7.20			
S_{tc} =	494 in^3			
f_c =	2.85 ksi			
F_c =	2.25 ksi	<	2.85 ksi	NO GOOD

Bottom Flange Tension Stress (Total Load)

f_b =	23.3 ksi			
F_b =	45 ksi	>	23.3 ksi	OK

Shear Check

Total Load =	99 psf			
w =	2.22 klf			
R =	25.1 k			
f_v =	11.6 ksi			
Fv =	20 ksi	>	11.6 ksi	OK