901 NEW YORK Avenue

Structural Technical Report 2

Pro-Con Structural Study of Alternate Floor Systems

901 New York Avenue
Washington, D.C.
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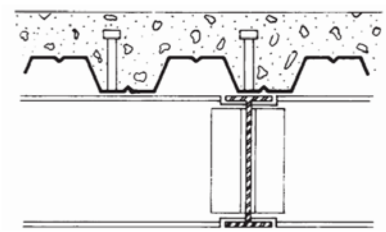
Executive Summary

901 New York Avenue Multi-Use Facility

The Report on Pro-Con Structural Study of Alternate Floor Systems is the second thesis study. It encompasses the analysis of 4 alternate systems to the current structural framing system. These systems are analyzed only with gravity loads, as lateral shall be assessed in the third and final report. As this is only a preliminary investigation into alternative systems, analysis has been simplified to simple 20' by 40' bays (or as described in the report). This is not a real-case scenario, as 901 NYA's unique shape and style causes it to have a great number of non-typical bays. The four systems proposed are: steel framing, composite steel framing, pre-cast concrete panels, and 1-way slab system.



Steel framing turned out to be the worst of the four alternative systems due to its large beam sizes, lack of integration of MEP, and its lightweight design. Typical bay sizes for analysis was 20' by 40', with a 2" deck and 5.5" concrete slab. Another factor was its requirement of a large number of beams to be fabricated and delivered.

The composite system fared well compared to the non-composite frame. Beams sizes varied from 14" to 16", and the slab and deck combination was only a total of 5.5". Although the addition of shear studs increase construction time, the fact that it requires a lower number of beams could save time (since steel connections can become complicated and take a lot of time to do). However, its inability to integrate MEP systems into the floor system still makes it a major setback.

Pre-cast panels are already being used in the current system for the outside façade. However, using pre-cast panels for structural design is still relatively new (since the early 1980's). There are several benefits of pre-cast design, and it is very apparent in this analysis. Slab thickness was only 8" thick, which include a 2" topping (that you may choose to have or not have). However, the long span of 40' caused a great deal of problems, requiring a 32" steel girder or a 42"+ concrete girder. There is the possibility of fabricating pre-stressed girders to reduce the size of the concrete beam. Another setback is that pre-cast costs can be kept at a minimal through repetitive design. In the case of 901 NYA, only a few bays per floors have the actual 20' by 40' dimension. Most other pieces are uniquely trapezoidal.

Finally, a 1-way slab was proposed. Although initial calculations showed good numbers (5" and 8" slabs, for 13' and 20' spans), the girders once again was the main problem. They were about 10" to 31", which is more than desired. A 1-way joist-and-girder system was also briefly entertained, and it may be another possibility to an alternative.

Further calculations and tables can be found in the Appendix.

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Introduction

901 New York Avenue is an 11-story multi-use facility located in the heart of Washington, D.C. Because of its location, the building has many restrictions that it must follow, one of which is its height restriction. This was the major factor that moved the structural engineers to follow through with the current system (two-way post-tensioning flat slab design with moment framing). The following report will review the current floor system (from Technical Assignment 1) and investigate four alternative systems to the current.

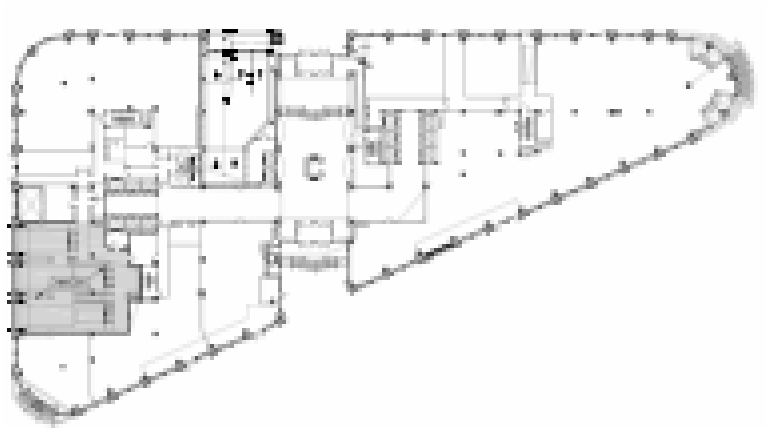


Figure 1 *– 1st Floor plan of 901 New York Avenue*

Existing Conditions

901 NYA is primarily used as office space for a number of law firms. As a result, the loads on the floors are office space and lobby/corridor loads. Also, to maximize space on each floor without clutter, typical bays were laid out to be 20' by 40'.

Dead Loads	11" slab	137.5 psf
	8" slab	100 psf
	MEP	15 psf
	Miscellaneous	5 psf

Live Loads	SOG	100 psf
	Parking	50 psf
	Office (w/partitions)	100 psf
	Lobby, Corridors	100 psf
	Heavy Mech.	150 psf
	Loading Truck Bay	250 psf

Referenced Materials	CRSI Design Handbook
	PCI Handbook (6th Edition)
	RS Means Construction Cost Data (2006)
	LRFD Steel Manual (13rd Edition)

Floor System: Because of the large span of the slabs, shear caps were also put into application, along with post-tensioning. This system was also used to resist lateral loads, as construction of shear walls would have been much too costly (lateral system is moment framing). Slabs are typically 11" above the parking levels, poured with a compressive strength of f'c = 5,000 psi. Columns supporting this system are typically in the range of 26" by 26" to 32" by 32". Figure 1 below shows a typical bay in 901 New York Avenue (ground level up).

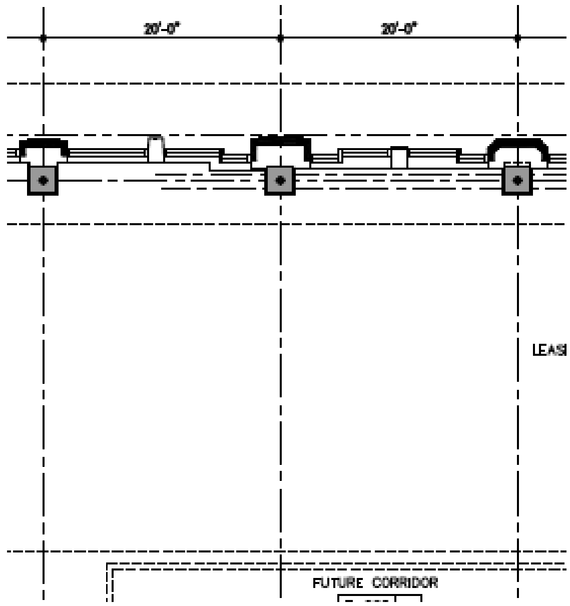


Figure 2 *– Typical bay layout from ground level up*

Foundation: The foundation of the current building consists of single and continuous footings. Strap beams are used to sometimes tie one footing to another. Caissons or piles were not necessary, due to the already satisfactory soil conditions. This benefit is possible due to the 4-level parking garage. Walls of the sub-grade levels are typically 36" thick. Levels below the 2nd floor have the same 20' by 40' bay layout, but these bays are intermittently interrupted by other columns, sometimes sloped.

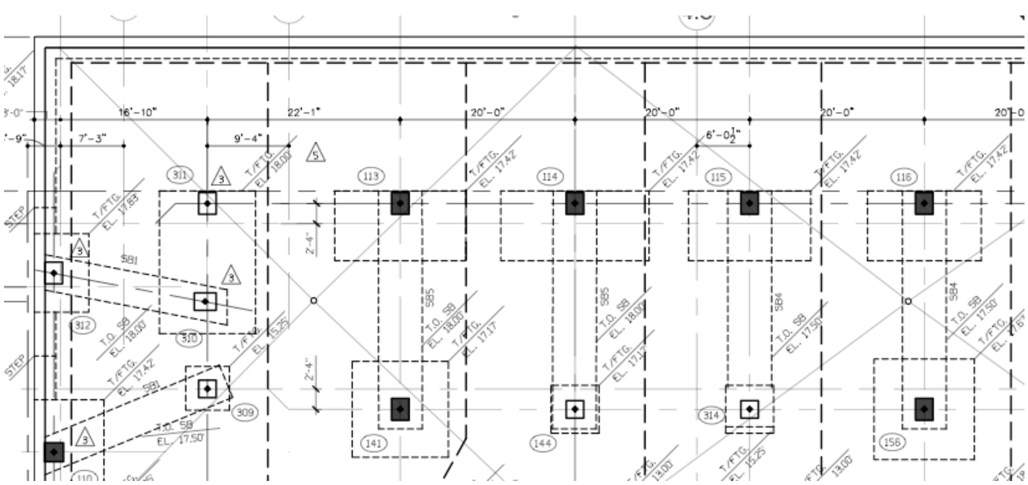


Figure 3 *– Example of footings (single and continuous) and strap beams*

Lateral System: The most unique structural feature of the building is its lateral system. Although concrete buildings typically have shear walls to resist lateral loads, 901 NYA was different to its physical shape. The triangular shape of the building helped distributed loads in unusual patterns to the typical rectangular building. It was believed that moment framing was a sufficient to resist lateral loads without forcing too much of an increased design in both the slab and column. Tech 1 already explained that the slab satisfies as long as it had shear caps, and that the columns were extremely over-designed if only axial loads are considered.

Explanation of Current Design: There are several reasons as to the current design of 901 NYA. The building is located in a very valuable location, minutes from D.C.'s Convention Center, located in the heart of the city, and just outside the Chinatown border. The owner desired to have as many floors as possible without sacrificing good space per floor (current design has 11'-8" floor-to-floor heights for nominal floor-to-ceiling height [without finishes] at 10'-9") to maximize the number of leasers. Post-tensioning lessened the thickness of slab, allowed the possibility of moment framing, and opened up bays to a full 40' by 20' area. Due to shape and design of the concrete structure, shear walls were not necessary for later support. This contributes to the 4-story atrium opening on the 1st floor.

Disadvantage to Current Design: There are many setbacks to the current system. Since it is a concrete building, pouring, curing, and settling of concrete consumes a large amount of the construction process. Also, post-tensioning requires a tremendous amount of extra work than traditional slab design, even if a slab design consisted of draped reinforcement. Schedules are extended due to the fact you must wait until a satisfied strength of the concrete before applying any tensioning to the tendons. And finally, even with a two-way flat slab system, space from the bottom of slab to recessed ceiling is necessary to house the MEP systems. So the two-way slab ensures the thinnest slab, but it doesn't necessarily ensure the thinnest overall solution.

Summary

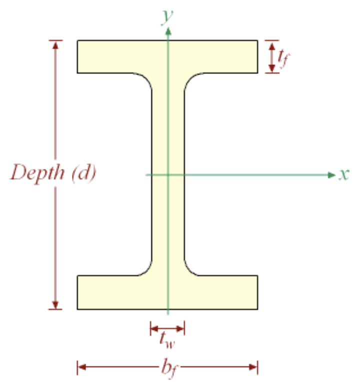
It is no wonder that the structural design of 901 NYA was a great feat in itself, but it is possible that a simpler design could have satisfied the owner as well. The following systems have been briefly entertained to see if it would be a feasible alternative to the current system:

1. Steel-Framed Building with Metal Decking and Concrete Slab
2. Steel-Framed Composite System with Metal Decking and Concrete Slab
3. Pre-Cast Slabs resting on Steel Girder
4. 1-way Concrete Slab with Joists

Each system has an explanation of the system, a step-by-step process of design, summary of advantages and disadvantages, and the probability whether or not it can be considered as an alternative.

Alternative System 1: Steel Beam and Column with Metal Deck and Concrete Slab

Description: The first alternative system to be analyzed was a steel-framed building, using wide-flanged beams and columns with metal form deck and a concrete slab. Structural steel has many benefits in design and construction, from strength in both compression and tension to very quick erection. Although typically composite systems are known to have stronger qualities, construction time on composite systems take a significantly longer time than a non-composite system. As a result, both systems were analyzed. The composite option will be described in the following alternative system.



in × lbf/ft	Area (in^2)	d (in)	b_f (in)	t_f (in)	t_w (in)	Ixx (in^4)	Zxx (in^3)	kxx (in)	Iyy (in^4)	Zyy (in^3)	kyy (in)
W24 × 76	22.4	23.92	8.990	0.680	0.440	2100	176	9.69	82.5	18.4	1.92

Figure 4 *– Dimension of a W24 x 76 beam*

The greatest factor will be the depth of the beams. Although steel opens up space in between beams and girders, the greatest depth of the beams will most likely control the floor-to-ceiling thickness (since you cannot cut through a steel beam without significantly losing the integrity of the beam).

Loads: Similar loads were used for the steel framing. It was assumed that this would only be a preliminary design, so lateral loads were, for the most part, not considered.

Live Load:	Lobby/Office Space	100 psf
Dead Load:	Metal Deck	3 psf
	Concrete Slab	(5.5" + 2"/2)*145 = 78.54 psf
	Beam Weight (assumed)	50 plf
	MEP and Finishes	20 psf

Bay Size: The same bay size was used as the original system at 20' by 40'. The metal decking spanned a complete distance of 8'-0", which also spread the beams out evenly within the bay at 8'. Sample design in RAM featured 3 bays horizontally (40' span) and 4 bays vertically (20' span). As already discussed, lateral loads were not considered. All beams and columns only take gravity loads.

Design: The metal decking used for design had to withstand at least 100 psf service loads. Vulcraft's catalog was used to find a suitable deck, and their 2C Conform deck was best fit for the 8' span.

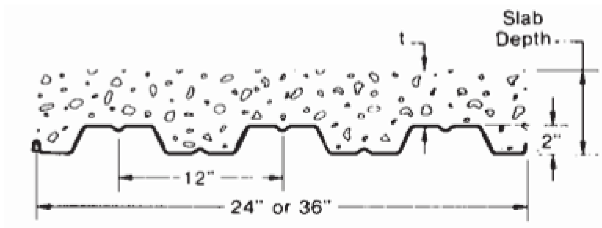


***Figure 5** – 2C Conform deck courtesy of Vulcraft*

Because of the deck's design, the total thickness slab is half the thickness of the deck and the cover on top of the deck. In this case, it was considered to be 5.5" + 2"/2 to make a total load of 78.54 psf by the slab and deck combination. It will be reinforced with 4x4-W2.9xW2.9 welded wire fabric.

Most of the beams that were spaced at 8' were typically designed at W14 x 22, while the girders were sized at W24 x 55 on the outside perimeter and W24 x 76 on the inside. Sample hand calculations were done to check the values of the RAM model. All calculations were done according to the LRFD Steel Manual (3rd Edition). The calcs showed that these estimated values are correct (see Appendix). Because there is nothing outside of the lateral system, the columns do not take on a heavy load. As a result, most of the columns were found to be either W10 x 33 or W10 x 39. Sample hand calculations show that these estimated values are also correct (see Appendix).

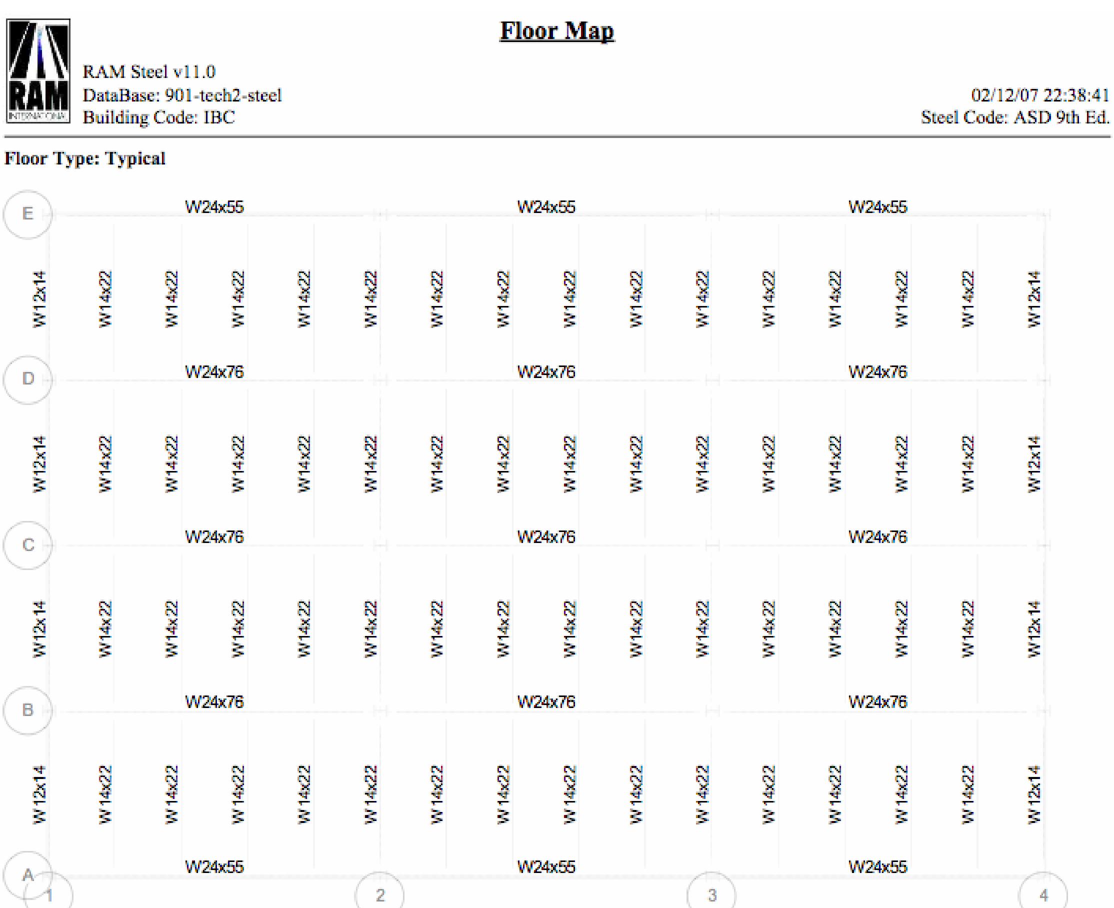


***Figure 5** – Beam Design of Steel System*

Gravity Column Design Summary

RAM Steel v11.0
DataBase: 901-tech2-steel
Building Code: IBC

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Steel Code: ASD 9th Ed.

Column Line 1 - A

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
11	18.1	8.8	1.8	1	0.25 Eq H1-3	0.0	50	W10X33
10	29.8	3.6	0.7	1	0.22 Eq H1-3	0.0	50	W10X33
9	40.5	3.3	0.7	3	0.24 Eq H1-1	0.0	50	W10X33
8	50.7	3.1	0.6	3	0.29 Eq H1-1	0.0	50	W10X33
7	60.5	2.9	0.6	3	0.34 Eq H1-1	0.0	50	W10X33
6	70.1	2.8	0.6	3	0.38 Eq H1-1	0.0	50	W10X33
5	79.4	2.7	0.6	3	0.43 Eq H1-1	0.0	50	W10X33
4	88.7	2.7	0.5	3	0.48 Eq H1-1	0.0	50	W10X33
3	97.8	2.6	0.5	3	0.52 Eq H1-1	0.0	50	W10X33
2	106.8	2.6	0.5	3	0.57 Eq H1-1	0.0	50	W10X33
1	115.7	2.5	0.5	1	0.62 Eq H1-1	0.0	50	W10X33

Column Line 1 - B

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
11	25.6	13.7	1.4	6	0.33 Eq H1-3	0.0	50	W10X33
10	47.7	5.8	0.6	3	0.29 Eq H1-1	0.0	50	W10X33
9	65.7	5.3	0.5	3	0.38 Eq H1-1	0.0	50	W10X33
8	81.1	5.0	0.5	6	0.46 Eq H1-1	0.0	50	W10X33
7	98.0	4.8	0.5	6	0.55 Eq H1-1	0.0	50	W10X33
6	114.5	4.7	0.5	11	0.63 Eq H1-1	0.0	50	W10X33
5	133.2	4.7	0.5	6	0.73 Eq H1-1	0.0	50	W10X33
4	152.5	4.7	0.5	6	0.83 Eq H1-1	0.0	50	W10X33
3	171.8	4.7	0.5	6	0.94 Eq H1-1	0.0	50	W10X33
2	191.1	4.7	0.5	6	0.86 Eq H1-1	0.0	50	W10X39
1	210.5	4.7	0.5	6	0.93 Eq H1-1	0.0	50	W10X39

Column Line 1 - C

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
11	25.6	13.7	1.4	6	0.33 Eq H1-3	0.0	50	W10X33
10	47.7	5.8	0.6	3	0.29 Eq H1-1	0.0	50	W10X33
9	65.7	5.3	0.5	3	0.38 Eq H1-1	0.0	50	W10X33
8	81.1	5.0	0.5	6	0.46 Eq H1-1	0.0	50	W10X33
7	98.0	4.8	0.5	6	0.55 Eq H1-1	0.0	50	W10X33
6	114.5	4.7	0.5	11	0.63 Eq H1-1	0.0	50	W10X33
5	133.2	4.7	0.5	6	0.73 Eq H1-1	0.0	50	W10X33
4	152.5	4.7	0.5	6	0.83 Eq H1-1	0.0	50	W10X33
3	171.8	4.7	0.5	6	0.94 Eq H1-1	0.0	50	W10X33
2	191.1	4.7	0.5	6	0.86 Eq H1-1	0.0	50	W10X39
1	210.5	4.7	0.5	6	0.93 Eq H1-1	0.0	50	W10X39

Column Line 1 - D

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
11	25.6	13.7	1.4	6	0.33 Eq H1-3	0.0	50	W10X33

Figure 6 – *Column Design of Steel System*

Advantages
- Off-site fabrication
- Very quick erection
- Tension/Compression benefits
- Longer lifetime integrity than concrete
- Different dead loads due to different materials could lead to different foundation and lateral system

Disadvantages:
- Fireproofing not included
- Moment framing much more complicated, otherwise braced framing needed
- Very thick beam-and-deck combination may not be a better solution. Beam itself is 24", and that doesn't include the 7.5" slab and deck.

Summary: It can be quickly assumed that a simple steel-framed building (no composite or other contribution to distributing loads) would not be in the best interest of the owner. A total floor thickness of 31.5" is more than acceptable, as the MEP systems have not even been considered. It is possible that perhaps a composite system may prove much more efficient for steel design. That option will be assessed in the next alternative system.

Alternative System 2: Steel Composite System w/Metal Deck and Concrete Slab

Description: In the previous alternative system, a steel system was proposed, but the sizes were coming out much too large to be considered as a true alternative. A composite system may help reduce the thickness of slab, deck, and beam.

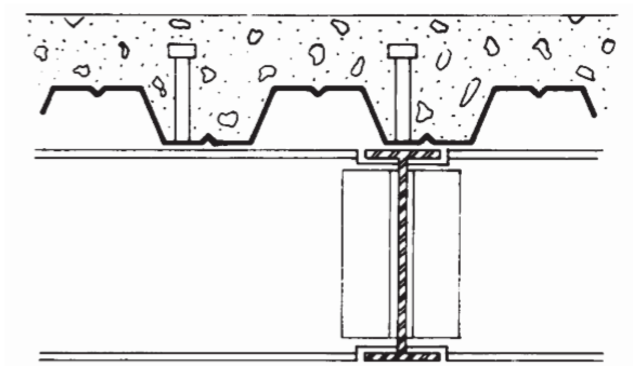


Figure 7 – *Example of Composite System*

A composite system works by distributing the loads of the beam to the deck along with itself. In this manner, stress on the beam is lessened, and a smaller beam is possible. There is a setback to this design, however. The deck and beam are connected through a mechanism called the shear stud, and the installation and application of these studs into the deck and beam is very time consuming. Also, the positive benefits of a composite system don't really come into effect until about 28' and more. This may actually be helpful in 901 NYA's case, as its span is as long as 40'-0".

Outside of these special conditions, a composite has mainly the same advantages and disadvantages of a regular steel system.

Loads: Once again, lateral loads were not considered for simplicity purposes. A composite steel framing considers the beam, slab, and deck weight, along with MEP and finishes. Live loads are still the same as the existing system:

Live Load:	Lobby/Office Space	100 psf
Dead Load:	Metal Deck and Slab (comb.)	2 psf
	Beam Weight (assumed)	50 plf
	MEP and Finishes	20 psf

Bay Size: Two different designs were considered for the composite system. Although the current bay length of 40'-0" is a positive benefit for composite beams, very long distances can still force the beam's depth to be too deep. As a result, composite beams at 32' and 40' were both analyzed through a RAM model. The current frame's short distance is 20', but for the fitting of the deck, a preferred distance would either be 7'-0" to 8'-0" between beams. As such, both distances were also tried, one at 3 beams @ 24'-0" and the other at 3 beams @ 21'-0".

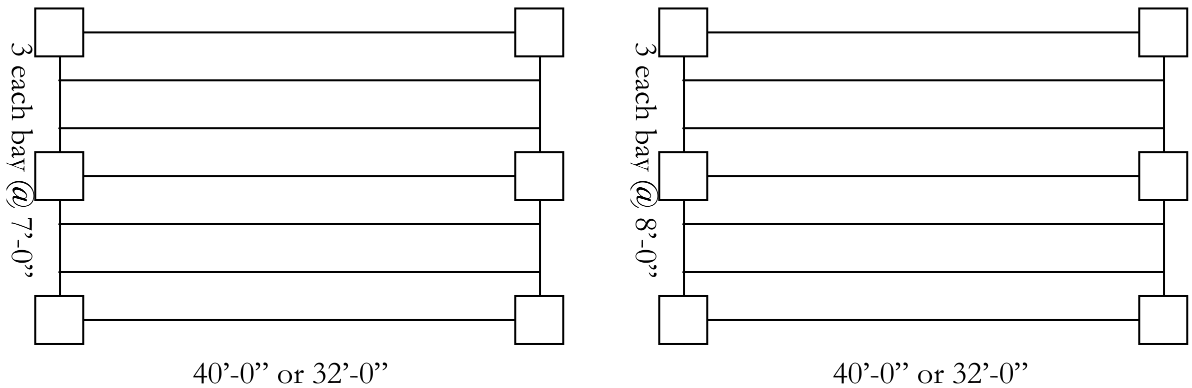


Figure 8 *– Bay designs for composite systems*

Design: Once again, the first step is to find the size of the deck. This time things are different from the previous design in that the length being spanned is 32'-0" and 40'-0" instead of 20'-0". Vulcraft's decking catalog also has a section for composite-use decks along with roof and non-composite decks. Distances of 7'-0" and 8'-0" were the span of the deck, with service loads as the considered loads in the tables. 1.5 VL/VLI was found to satisfy both distances, with the 7'-0" length needing 3.5" with 22-gage steel and the 8'-0" length requiring 3.5" with 21-gage steel. Already there is a significant difference from the regular steel framing. The non-composite system required 6.5" of slab and deck, whereas the composite system only requires 5" of slab and deck with a lower weight (1.97 psf).

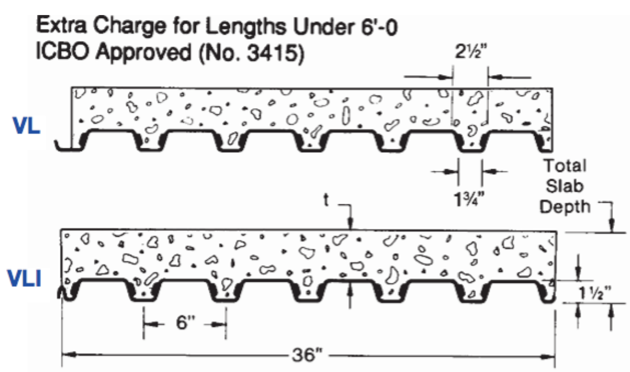


Figure 9 *– 1.5 VL/VLI deck courtesy of Vulcraft*

Several different designs were tried. Because it was desired to have a span of 7'-0" and 8'-0" for the deck, the bays had to be readjusted to 21' and 24' bay widths, respectively. This allows for 3 divisions in the bays.

The first trial in RAM of 40' x 7' (length by width) resulted in a typical layout with W16 x 26 beams throughout the frame (girders included). The lightest beams are found around the perimeter at W12 x 19. This is simply because of the fact that perimeter beams take half the load. It should be noted that there are numbers in parenthesis next to the beam size. These are the number of studs required for satisfactory design. The more studs, the better composite action, but longer construction time.

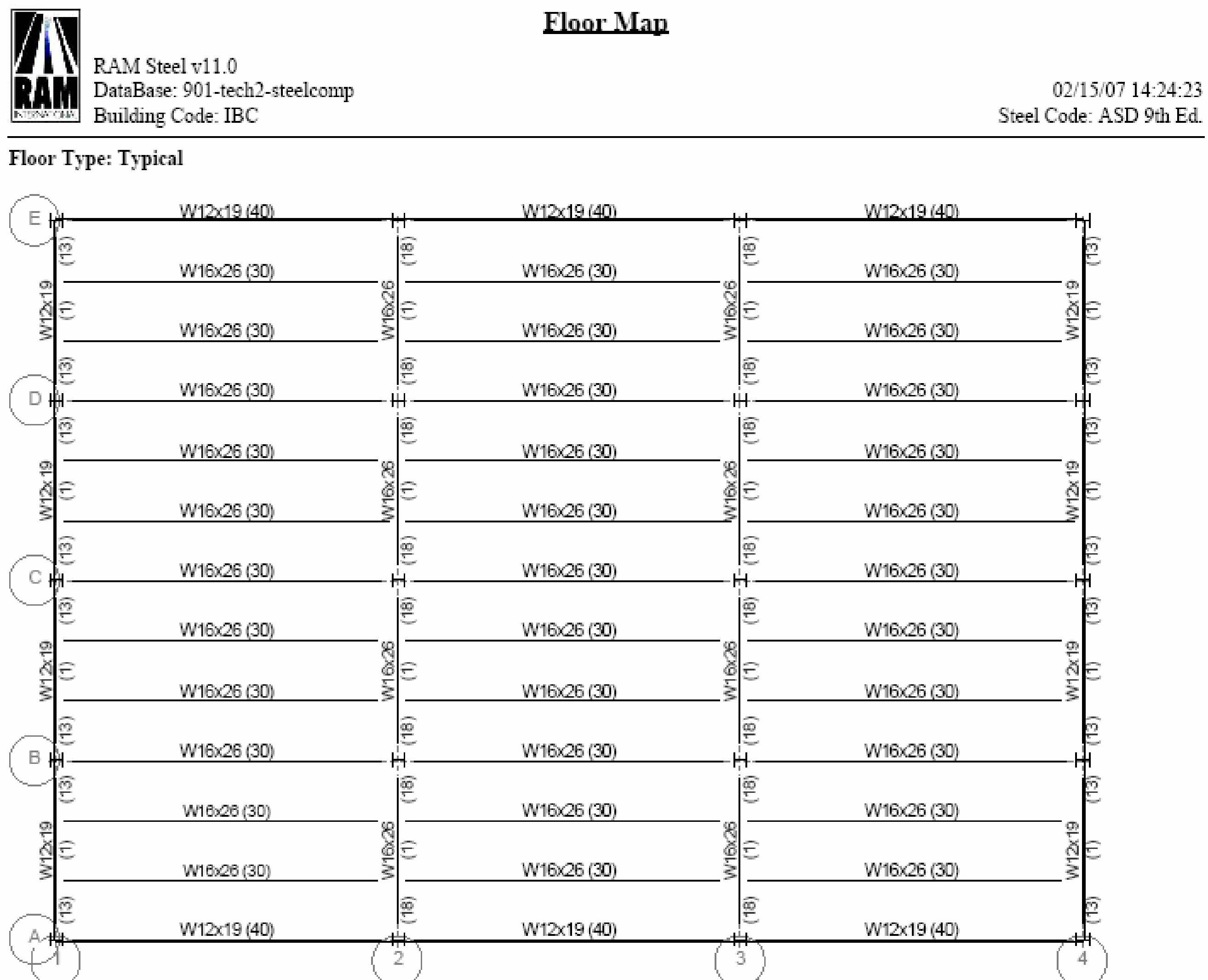


Figure 10 – Composite Layout with 21' by 40' bays

The second trial had a 24' by 40' bay using 8' divisions within the bay. This opened up the possibility of even larger bay spans than the current design. Although the beam sizes are the same (W16 x 26), it's observed that the wider bays require more studs. Not only that, but the girders are also larger sizes. In terms of fabrication and delivery to site, it is much easier to have pieces in the same size to reduce fabrication time. Also, an increase of shear studs can also greatly increase construction time. So far, the first trial is the better solution.

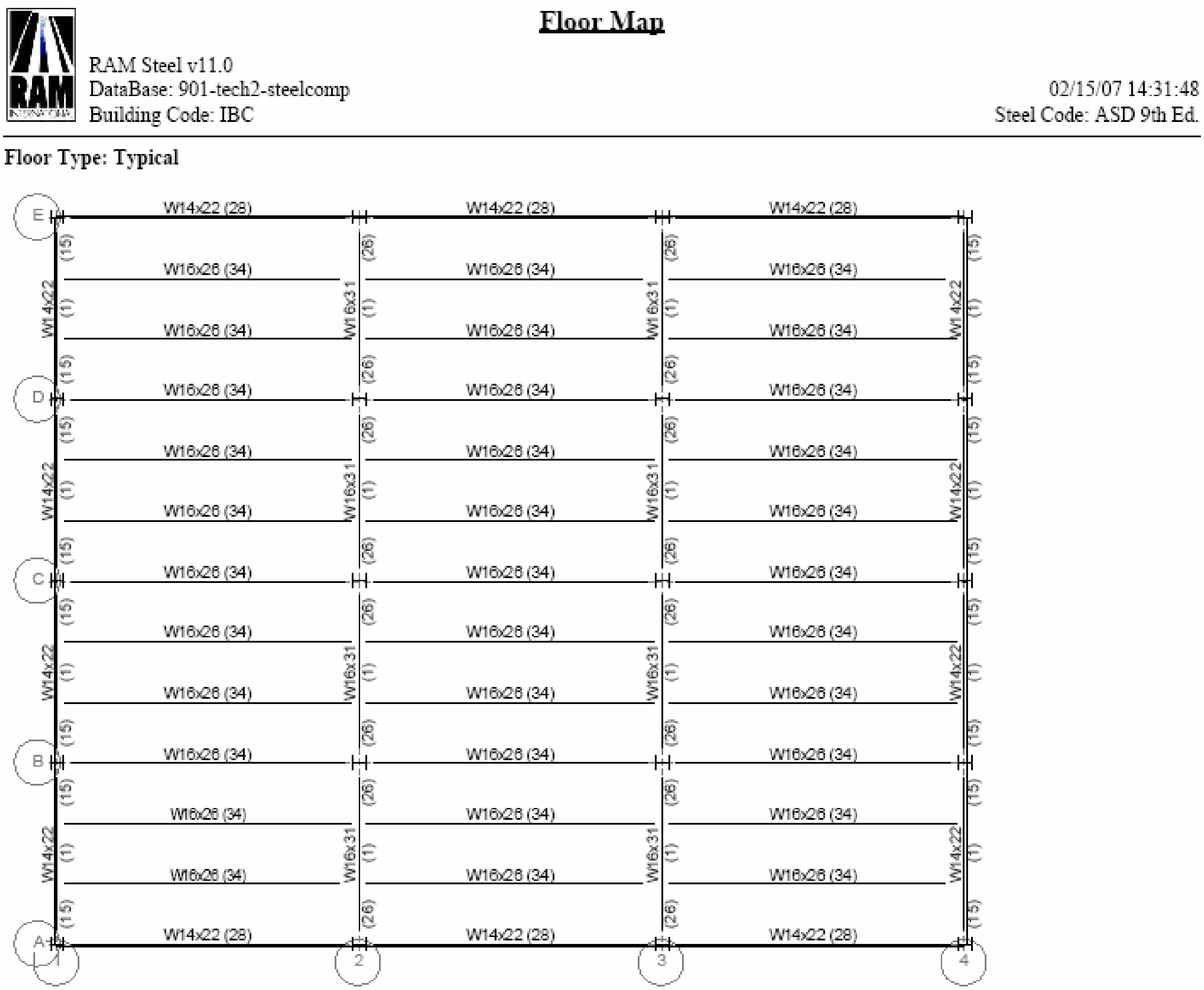


***Figure 11** – Composite Layout with 24' by 40' bays*

The final trial was an attempt to see if a smaller bay length would affect the size of the beams and the number of studs. The third trial had a bay size of 21' by 32'. Beams within the bay came out to about 2" smaller than the first trial and required less studs for composite action. However, it can be noted also that the girders stay the same size at W16 x 26. So even with the smaller beams, the total depth of the system is still 16". The first trial still has the best outcome (longer span, same sized beams and girders, average amount of studs).

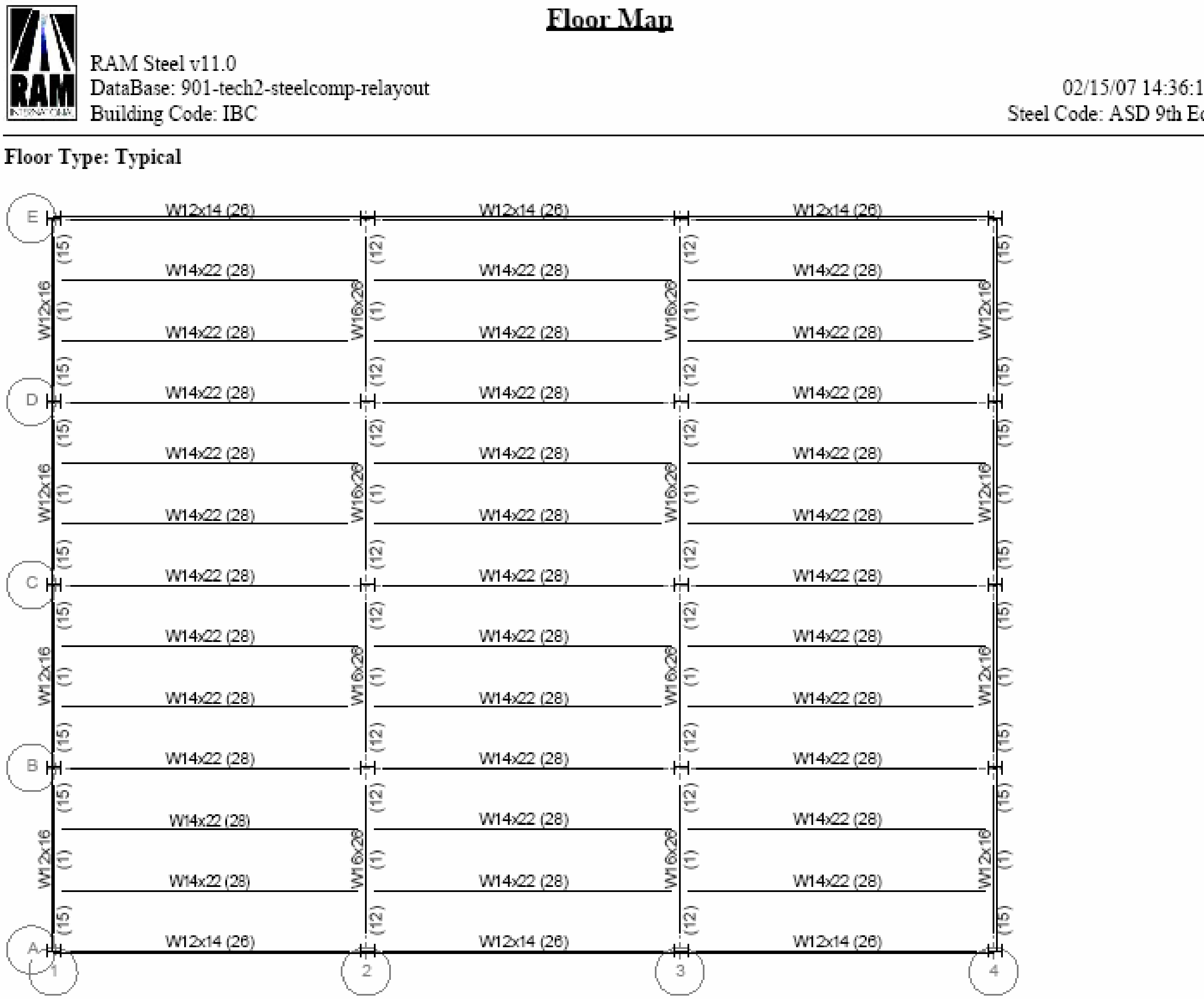


Figure 12 – *Composite Layout with 21' by 32" bays*

Advantages
- Much smaller sandwich system than the non-composite system
- Smaller slab and deck system than the non-composite system
- Much smaller beam at 16”
- Shoring is not needed
- A lighter system may lighten foundation design and lateral resistance necessities

Disadvantages
- Shear studs require much more construction time and work
- Same general disadvantages of steel structure as the non-composite system

Summary: Of all the steel systems, it seems that the 21’ by 40’ bay composite structure is the best solution. It is also important to note that the number of connections in the composite system is greatly decreased due to the fact that the beams run long-way instead of short-way in the non-composite alternative. There is still the setback of composite systems because of shear studs. But to my observation, if the owner was willing to pay extra cash for an extremely complicated post-tensioning system, extra money for a composite system would definitely be a possibility.

Another setback is that even the smallest system of 16” beams does not include the integration of the MEP system. So it can be assumed that the total depth of the system would be larger than the 16” of just the beam.

Alternative System 3: Pre-Cast Hollow Core Concrete Slab

Description: Another alternative system considered was a pre-cast, hollow-core concrete slab. Pre-cast (P/C) concrete is already used on the building for the outside façade. Because it is yet a young method of construction, pre-cast concrete brings in a great number of benefits atypical to steel and cast-in-place (CIP) concrete. Concrete is typically known for its time-consuming on-site construction and some tendencies of having unsatisfactory concrete batches (that would require re-pouring and a huge delay on many projects). P/C concrete benefits from CIP in the following ways: better controlled conditions, fire resistance, and durability (more benefits in the AS-1 summary). It is also just as shapeable as CIP concrete. These are the reasons as to why P/C concrete was considered.

Loads:

Bay Size: The slab is proposed to span 20'-0" (short direction) in the typical bay. Another option was to span the full 40'-0", but P/C slabs cannot be loaded to support more than 122 psf @ 40'-0". Thus the 20'-0" span was selected over the 40'-0" span.

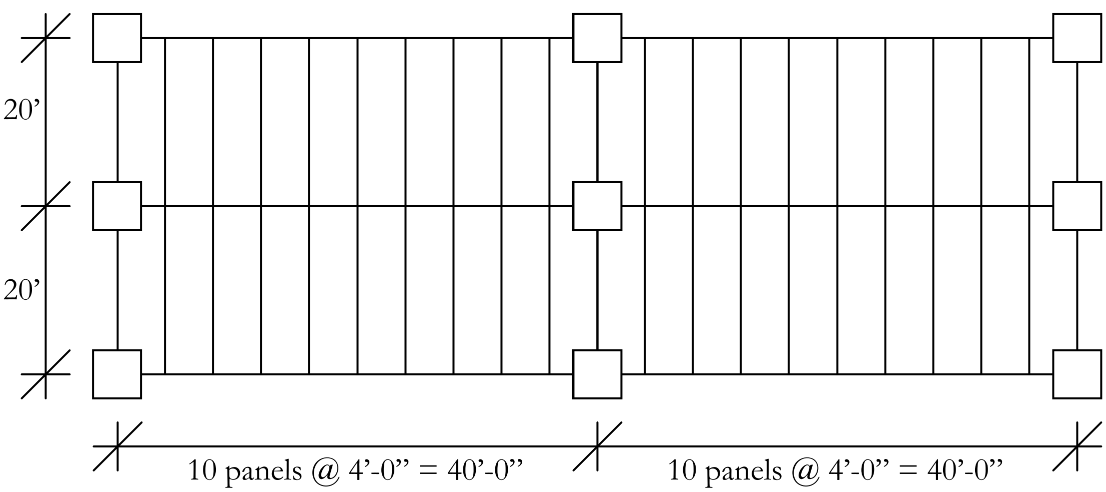


Figure 13 *– Layout of 4'-0" P/C Slabs*

Design: Factored loads included the pre-cast slab and 2" topping (73.75 psf), and the live load (160 psf) to get a total of 241 psf. Example slabs were found in the PCI Handbook (6th Edition), and a hollow core slab was found to best suit the current system (better long span conditions). Design guidelines were followed in conjunction with the PCI Handbook. 4HC6 + 2 was chosen, with 7-3/8" strands. Safe superimposed service loads come out to be 163 psf, with a camber of .3" during erection and 0.1" longtime camber. 4HC6 + 2 was chosen over 4HC6 because the deflection for 4HC6 was assumed to be 0.5". Although this is still in the acceptable range of deflection for its length,

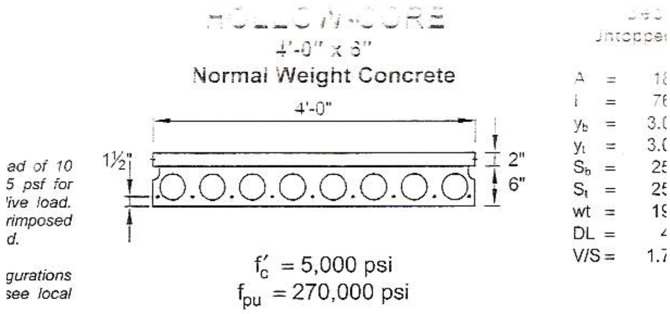


***Figure 14** – Sample of 6" Hollow Core Slab w/ 2" topping*

Although P/C beams can also be used, a concrete beam was assumed to be much too large at a 40' span, so both steel and concrete beams were considered. RAM Structural System was used to analyze the steel beams.

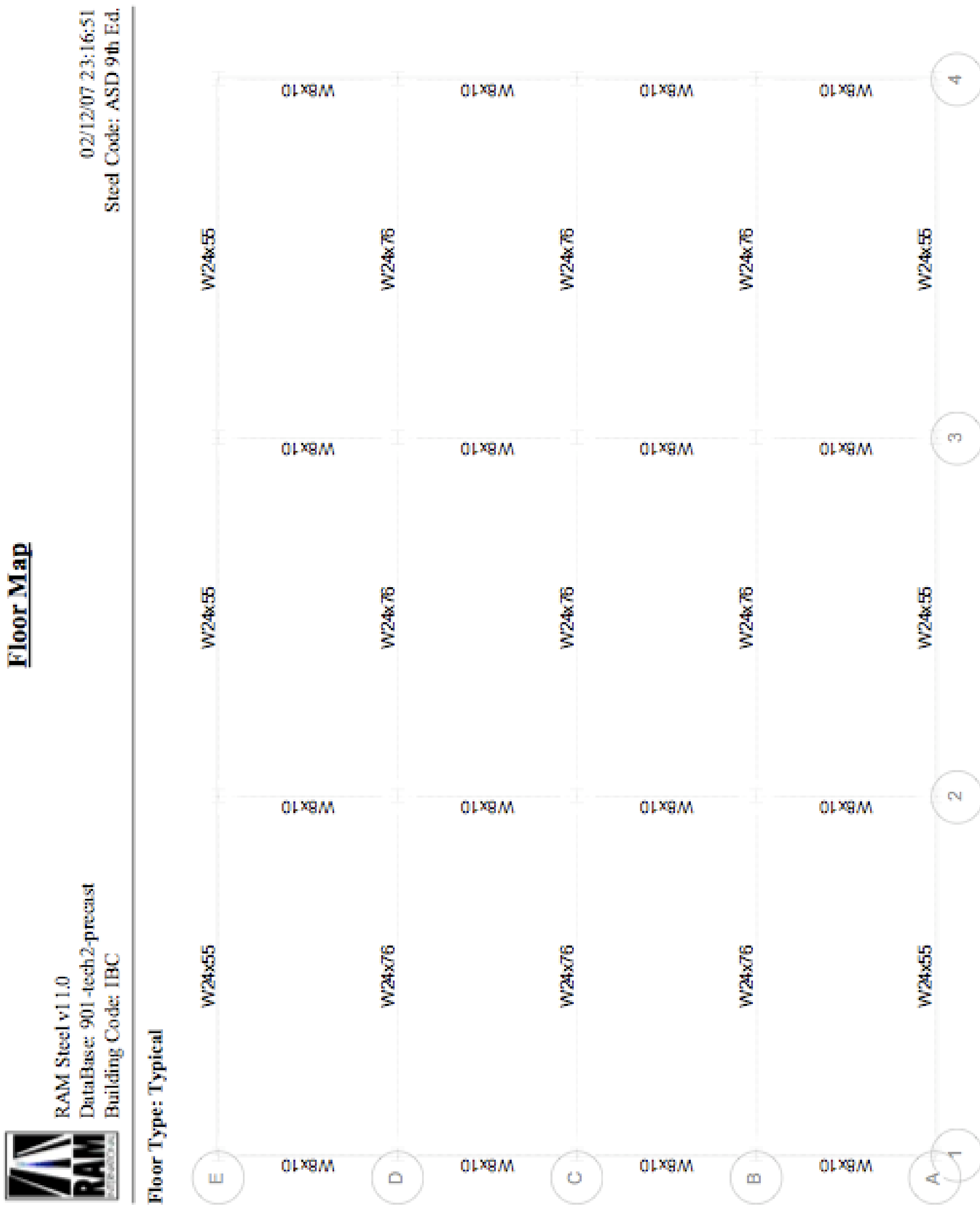
***Figure 15** – Steel beam layout for P/C panels*

Advantages

- Very quick to erect
- Off-site construction of panels
- Very quick scheduling
- Better integrity than CIP
- Lighter system may help lighten loads for foundation

Disadvantages

- Fireproofing not included for steel
- Lighter system may cause a whole new series of issues (different lateral system may control)
- Connections and details can become very complicated with hybrid systems
- Cannot "cut through" beams w/o losing significant strength


Floor Map
RAM Steel v11.0
DataBase: 901-tech2-precast
Building Code: IBC
02/12/07 23:16:51
Steel Code: ASD 9th Ed.
Floor Type: Typical
W24x55
W24x76
W8x10
A
B
C
D
E
1
2
3
4

Gravity Column Design Summary

RAM Steel v11.0
DataBase: 901-tech2-precast
Building Code: IBC

02/12/07 23:18:18
Steel Code: ASD 9th Ed.

Column Line 1 - A

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
11	17.6	10.4	0.2	1	0.21 Eq H1-3	0.0	50	W10X33
10	28.8	4.2	0.1	1	0.20 Eq H1-3	0.0	50	W10X33
9	39.1	3.8	0.1	3	0.22 Eq H1-1	0.0	50	W10X33
8	48.7	3.6	0.1	3	0.27 Eq H1-1	0.0	50	W10X33
7	58.1	3.4	0.1	3	0.31 Eq H1-1	0.0	50	W10X33
6	67.2	3.3	0.1	3	0.36 Eq H1-1	0.0	50	W10X33
5	76.1	3.2	0.1	3	0.40 Eq H1-1	0.0	50	W10X33
4	84.8	3.1	0.1	3	0.45 Eq H1-1	0.0	50	W10X33
3	93.5	3.0	0.1	3	0.49 Eq H1-1	0.0	50	W10X33
2	102.0	2.9	0.1	3	0.53 Eq H1-1	0.0	50	W10X33
1	110.4	2.9	0.1	1	0.58 Eq H1-1	0.0	50	W10X33

Column Line 1 - B

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
11	27.2	16.2	0.1	6	0.32 Eq H1-3	0.0	50	W10X33
10	45.8	6.8	0.0	3	0.28 Eq H1-1	0.0	50	W10X33
9	62.9	6.2	0.0	3	0.36 Eq H1-1	0.0	50	W10X33
8	79.0	5.8	0.0	6	0.44 Eq H1-1	0.0	50	W10X33
7	94.8	5.6	0.0	6	0.52 Eq H1-1	0.0	50	W10X33
6	110.3	5.4	0.0	6	0.59 Eq H1-1	0.0	50	W10X33
5	128.1	5.4	0.0	6	0.68 Eq H1-1	0.0	50	W10X33
4	146.4	5.4	0.0	6	0.78 Eq H1-1	0.0	50	W10X33
3	164.7	5.4	0.0	6	0.87 Eq H1-1	0.0	50	W10X33
2	183.0	5.4	0.0	6	0.96 Eq H1-1	0.0	50	W10X33
1	201.4	5.4	0.0	6	0.88 Eq H1-1	0.0	50	W10X39

Column Line 1 - C

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
11	27.2	16.2	0.1	6	0.32 Eq H1-3	0.0	50	W10X33
10	45.8	6.8	0.0	3	0.28 Eq H1-1	0.0	50	W10X33
9	62.9	6.2	0.0	3	0.36 Eq H1-1	0.0	50	W10X33
8	79.0	5.8	0.0	6	0.44 Eq H1-1	0.0	50	W10X33
7	94.8	5.6	0.0	6	0.52 Eq H1-1	0.0	50	W10X33
6	110.3	5.4	0.0	6	0.59 Eq H1-1	0.0	50	W10X33
5	128.1	5.4	0.0	6	0.68 Eq H1-1	0.0	50	W10X33
4	146.4	5.4	0.0	6	0.78 Eq H1-1	0.0	50	W10X33
3	164.7	5.4	0.0	6	0.87 Eq H1-1	0.0	50	W10X33
2	183.0	5.4	0.0	6	0.96 Eq H1-1	0.0	50	W10X33
1	201.4	5.4	0.0	6	0.88 Eq H1-1	0.0	50	W10X39

Column Line 1 - D

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
11	27.2	16.2	0.1	6	0.32 Eq H1-3	0.0	50	W10X33

Summary: The hollow core pre-cast system has many benefits. For one, the simplicity of design of erecting pre-cast panels instead of casting in place would save an immense amount of time. An 8" panel is sufficient to withstand gravity loads, which is thinner than the current system. The only setback is that if the same bay area is used, the depth of the beams becomes much too deep. If a concrete girder is used, it can be expected to exceed more than 42". Even a steel beam would be a depth of 33". An alternative to a simple girder is a pre-stressed concrete girder. This may help in the size of the beam.

Another setback is the fact that 901 NYA is not a simple rectangular building. The greatest benefit from pre-cast concrete is the repetition of panels. Because of so many different actually bay sizes and dimensions, pre-cast may not be the best alternative to the current system.

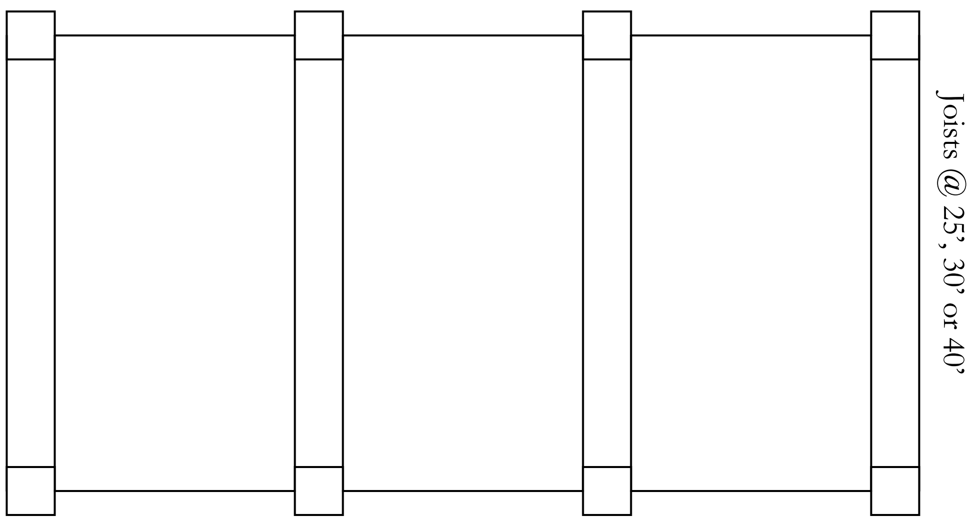
Alternative System 4: 1-way Concrete Slab w/Joists

Description: The final alternative system is the possibility of using a one-way slab supported on running joists. This is the only other concrete alternative that was assessed. One-way slab and joist systems are known for its low dead weight and need for reinforcement. It is also best suited at long distances, so it is beneficial that our current system uses a bar dimension of 20' by 40'.

Loads: Loads for the slab were first found before finding possible loads. Then the dead load of the slab was added to the total load to find the loading on the joists.

Live Load:	Lobby/Office Space	80 psf
Dead Load:	Slab (8")	100 psf
	Slab (5")	67.5 psf
	MEP and finishes	20 psf

Bay Size: Several different bay sizes were used to see what bay size might be best for a one-way joist. For initial calculations, I looked at a 13' and 20' slab span. For the 13' span, a 13' by 25' bay was selected (to maintain rectangular properties and not square). For the 20' span, a 20' by 30' bay and a 20' by 40' bay was selected.



Slab Spans 13' or 20' @ 5" and 8", respectively
Figure 18 *– 1-way joist dimensions*

Design: The CRSI Handbook was used to find acceptable sizes for different factored loads on a slab. At a 13'-0" span, the handbook allowed a 5" slab with #4's @ 10" OC on top and #3's @ 7" OC on the bottom. The slab is considered to be normal weight concrete, and the dead weight of the slab is 63 psf. At a 20'-0" span, the accepted design was an 8" slab with #5's @ 9" OC on top and #4's @ 8" OC on the bottom. All calculations can be found in the Appendix.

Advantages

- Simple design means simple construction and formwork
- Fireproofing is already implemented
- Generally about the same weight as current system; new foundation design wouldn't be necessary
- Much quicker construction than post-tensioning

Disadvantages

- Thinner slab brings new serviceable issues, like vibration
- At columns, the thickness of floor system ranges from 21" to 42", for 5" slab and 8", respectively
- Shear walls may need to be designed into building

Summary: Although the slab design came out beneficial for this alternative, the girders supporting the slabs were much too thick. Compared to the current building, it is a difference of 10"-31", which is perhaps more than permissible by the owner. As already explained, sacrificing ceiling space causes a "cramped" feel to the building floor, which would not be a comfortable environment to work in.

A joist-and-girder system has also been briefly viewed from the CRSI Handbook to see the possibilities of using a multi-joist system (8" deep rib + 3" slab is the smallest found in the handbook). The benefits of a joist-and-girder 1-way slab is it increases stiffness to the floor, MEP systems can be easily integrated into the floor system, and additional weight would factor out vibration as being an issue. The setbacks are that a new floor layout would be required, along with the fact that it will still be deeper than the current system. If a 1-way slab is to be considered for an alternative to the current system, it would be a 1-way joist-and-girder system.

Summary of Investigation

	Steel Framing	Composite System	Hollow Core Pre-Cast Concrete	1-way Slab (w/ and w/o Joists)
Floor Depth	Slab and Deck: 7.5” Beam: 24” **Total: 31.5”**	Slab and Deck: 6.5” Beam: **Total:**	Panel: 8” Girder: 33” – 42” **Total: 41” – 50”**	Slab: 5” – 8” Girder: 16” – 34” **Total: 21” – 42”**
Floor Weight (psf)	~ 70 psf	~ 40 psf	~ 60 psf	~ 125 psf
Fireproofing	No	No	If concrete girder used	Yes
Vibration	Relatively light systems have vibration issues	No	Relatively light systems have vibration issues	Most likely no
Cost (RS Means)				
Lead Time	Yes	Yes	Yes	No
Feasibility of Design	Fabrication off-site, quick erection, braced framing, complicated connections, lighter weight may cause re-design of foundation	Fabrication off-site, semi-quick erection (shear studs), complicated connections, lighter wt may cause re-design of foundation	Fabrication off-site, quick erection, possible pre-stressed designs may help, perhaps not enough repeat of panels	Cast-in-place, long construction time, pre-stressed designs may help some, MEP implemented into floor system
General Comments	Not considered as an alternative	Possible consideration, but redesign of columns	Prestressed beams? Possible consideration for alternative	Possible consideration, but redesign of columns

In summary, there are many things to note. First, it is important that whatever alternative system is chosen must begin with a redesign of column layout. A 20' by 40' is very difficult to work with, especially to have a girder supporting 800 square feet of loads. The current bay is only fitting for the two-way post-tensioned slab.

Another consideration is a re-assessment of lateral load-resisting systems. Currently, it is the moment framing of the post-tensioned system that resists lateral loads. Other systems can involve shear walls, braced frames, or steel moment framing.

The foundation may also need to be re-designed, depending on the alternative system chosen. For example, a steel-framed building would have a total weight of 3,365 kips, while the current system has a total weight of 6,610 kips. Half the weight will change the size of footings, the need for strap beams, etc.

Finally, it is important to note and remember the fact that there is still a 4-level parking garage sub-grade. In my personal experience, I have yet to see a steel-framed parking garage. Most above-grade parking garages are usually made of pre-cast or cast-in-place concrete. Although it is possible to make a parking garage of steel, it is not a usual practice to do so.

Overall, whatever system is chosen, it must meet the general criteria of the building. From building height limitations to desired floor-to-ceiling heights to exposed MEP systems, all of these must be considered before calling any other alternative system a true possibility.

The quick overview of all the systems above shows that either the pre-cast or the 1-way concrete systems may be the best options. Although steel can be used, composite systems are complicated, from its connections to application of shear studs. Concrete also has better flexibility in terms of integration of MEP systems into the floor system.

Appendix

TECH ASSIGNMENT #2 | 901 NYA | TIMOTHY H PARK

GIRDER DESIGN FOR HOLLOW CORE

• CONCRETE

USING $\boxed{4HC6+2}$ (DL = 74 PSF) ASSUME UNFACTORED

LOADS ON GIRDERS DL 20 + 74 = 94
LL ~~100~~ 80 = 80] 1.2(94) + 1.6(80) = 240.8 PSF

$(240.8)(20') = 4.816$ KLF

$\frac{(4.816)(40)^2}{8} = 963.2'^K = 11558''^K$

$\rho = 0.6\,\rho_{max} = 0.6\,(0.0206) = 0.01236$

$f'_c = 5000$ PSI
$f_y = 60$ KSI

ρ	R
0.0120	659
0.0125	684

— $\rho = 0.0124$, R = 679

$M_u = \phi R b d^2 \quad bd^2 = \frac{M_u}{\phi R} = \frac{11558}{0.9(679)} = 18913$ IN³

TRY 27"×27": $I = \frac{27 \times 27^3}{12} = 44287$ IN⁴

$\Delta = \frac{5 w \ell^4}{384 EI} = \frac{(5)(963)(40 \times 12)^4}{384\,(3.6E6)(44287)} = 4.17''$

$\Delta_{max} = \frac{\ell}{240} = 2''$ 2 < 4.17 ×

20"×32": I = 54613 IN⁴
Δ = 3.39"

15"×36" I = 58320 IN⁴
Δ = 3.17"

11"×42" I = 67914 IN⁴
Δ = 2.72"

W33×90
$\phi M_n = 1060'^K$

CONCRETE GIRDER NOT GOOD SOL FOR 40' SPAN

@ 28': $M_u = 472'^K = 5664''^K$ $bd^2 = 9268$

12×28: I = 16207 IN⁴
Δ = 2.74"

8"×36" WORKS @ 28'-0" SPAN

8"×36": I = 31105 IN⁴
Δ = 1.4"

TECH ASSIGNMENT #2 | 901 NYA | TIMOTHY H PARK

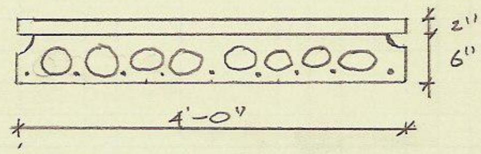
HOLLOW CORE SLAB DESIGN

FROM PCI DESIGN HANDBOOK 6TH EDITION

SPAN = 20'

SERVICEABLE LOADS: LL 100 PSF, DL 20 PSF] $1.2(20)+1.6(80)=152$ PSF

1)

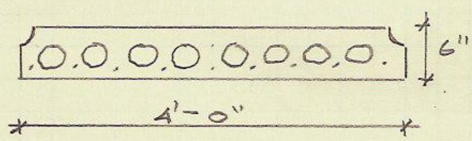


$\boxed{4HC6+2}$

w/ 76-S STRANDS

HOLLOW CORE PROP: MAX LOAD = 163 PSF
DL = 74 PSF
f'_c = 5000 PSI
STRANDS = 7 - 3/8" STRANDS
Δ = 0.1"

2)



$\boxed{4HC6}$

w/ 96-S STRAND

HOLLOW CORE PROP: MAX LOAD = 157 PSF
DL = 49 PSF
f'_c = 5000 PSI
STRANDS = 9 - 3/8" STRANDS
Δ = 0.5"

FROM RS MEANS

8" HC SLAB: \$9.60/SF × (48070)(11) = \$5,076,192.00

6" HC SLAB: \$9.50/SF × (48070)(11) = \$5,023,315.00

NOTE: $\Delta_{MAX} = \frac{l}{240} = \frac{20 \times 12}{240} = 1"$ BOTH SATISFY Δ REQUIREMENT

Strand Pattern Designation
76-S

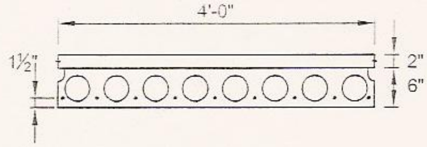
— S = straight
— Diameter of strand in 16ths
— No. of Strand (7)

Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

Capacity of sections of other configurations are similar. For precise values, see local hollow-core manufacturer.

Key
444 – Safe superimposed service load, psf
0.1 – Estimated camber at erection, in.
0.2 – Estimated long-time camber, in.

HOLLOW-CORE
4'-0" x 6"
Normal Weight Concrete



f'_c = 5,000 psi
f_{pu} = 270,000 psi

Section Properties

	Untopped	Topped
A =	187 in.²	283 in.²
I =	763 in.⁴	1,640 in.⁴
y_b =	3.00 in.	4.14 in.
y_t =	3.00 in.	3.86 in.
S_b =	254 in.³	396 in.³
S_t =	254 in.³	425 in.³
wt =	195 plf	295 plf
DL =	49 psf	74 psf
V/S =	1.73 in.	

4HC6

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

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

4HC6 + 2

Table of safe superimposed service load (psf) and cambers (in.) — **2 in. Normal Weight Topping**

Strand Designation Code	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
66-S	470	396	335	285	244	210	182	158	136	113	93	75	59	46	34				
	0.2	0.2	0.2	0.2	0.2	02	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2				
	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2	–0.3	–0.5	–0.7	–0.9	–1.2				
76-S		461	391	334	287	248	216	188	163	137	115	95	78	63	50	38	27		
		0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.1	–0.0	–0.1	–0.3		
		0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.2	–0.3	–0.5	–0.7	–0.9	–1.2	–1.5		
96-S			473	424	367	319	279	245	216	186	160	137	116	98	82	68	55	43	33
			0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.1	0.0	–0.1
			0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.1	–0.1	–0.3	–0.5	–0.7	–1.0	–1.4	–1.7
87-S			485	446	415	377	331	292	258	224	195	169	147	127	109	94	80	67	55
			0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.7	0.8	0.8	0.7	0.7	0.7	0.6	0.5	0.4	0.3
			0.5	0.5	0.5	0.6	0.6	0.6	0.5	0.5	0.4	0.4	0.2	0.1	–0.1	–0.3	–0.5	–0.8	–1.2
97-S			494	455	421	394	357	327	288	251	219	192	168	146	127	110	95	82	70
			0.5	0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9	1.0	0.9	0.9	0.9	0.8	0.7	0.6
			0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.5	0.4	0.2	0.0	–0.2	–0.5	–0.8

Strength is based on strain compatibility; bottom tension is limited to $7.5\sqrt{f'_c}$; see pages 2–7 through 2–10 for explanation.

PCI Design Handbook/Sixth Edition | 2–31

2500	40′ span, 12″ x 52″			20	3.600		4,175	146	92.50	4,413.50	4,925
2550	18″ x 52″			16	4.500		4,950	183	116	5,249	5,900
2600	24″ x 52″		↓	12	6	↓	5,425	243	154	5,822	6,575

03 41 05.15 Precast Columns

0010	**PRECAST COLUMNS**	R034105-30									
0020	Rectangular to 12′ high, small columns		C-11	120	.600	L.F.	47	24.50	15.45	86.95	111
0050	Large columns			96	.750		82	30.50	19.30	131.80	165
0300	24′ high, small columns			192	.375		47	15.20	9.65	71.85	88.50
0350	Large columns			144	.500	↓	82	20.50	12.85	115.35	140
0700	24′ high, 1 haunch, 12″ x 12″			32	2.250	Ea.	1,125	91.50	58	1,274.50	1,475
0800	20″ x 20″		↓	28	2.571	″	1,975	104	66	2,145	2,425

03 41 05.25 Precast Joists

0010	**PRECAST JOISTS**	R034105-30									
0015	40 psf L.L., 6″ deep for 12′ spans		C-12	600	.080	L.F.	7.80	2.90	1.21	11.91	14.40
0050	8″ deep for 16′ spans			575	.083		13	3.02	1.26	17.28	20.50
0100	10″ deep for 20′ spans			550	.087		23	3.16	1.32	27.48	31.50
0150	12″ deep for 24′ spans		↓	525	.091	↓	31	3.31	1.38	35.69	41

03 41 13 – Precast Concrete Hollow Core Planks

03 41 13.50 Precast Slab Planks

0010	**PRECAST SLAB PLANKS**	R034105-30									
0020	Prestressed roof/floor members, grouted, solid, 4″ thick		C-11	2400	.030	S.F.	4.89	1.22	.77	6.88	8.40
0050	6″ thick			2800	.026		6.30	1.04	.66	8	9.50
0100	Hollow, 8″ thick			3200	.023		6.70	.91	.58	8.19	9.60
0150	10″ thick			3600	.020		7	.81	.51	8.32	9.70
0200	12″ thick		↓	4000	.018	↓	7.95	.73	.46	9.14	10.55

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SOLID ONE-WAY SLABS—INTERIOR SPAN — Top Steel for $-M_u$

f'_c = 3,000 psi — Grade 60 Bars — ρ = 0.0050

Thickness (in.)	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10
Top Bars	#4	#4	#4	#4	#5	#5	#5	#5	#5	#6	#6	#6	#6
Spacing (in.)	12	11	10	9	12	11	10	10	9	12	11	10	10
Bottom Bars	#3	#3	#3	#4	#4	#4	#4	#4	#4	#5	#5	#5	#5
Spacing (in.)	10	9	7	12	11	10	10	9	8	12	11	10	10
T-S Bars	#3	#3	#3	#3	#4	#4	#4	#4	#4	#4	#4	#5	#5
Spacing (in.)	15	13	12	11	18	17	15	14	13	13	12	18	17
Areas of Steel (in.²/ft) Top Interior	.200	.218	.240	.267	.310	.338	.372	.372	.413	.440	.480	.526	.526
Bottom	.132	.147	.189	.200	.218	.240	.240	.267	.300	.310	.338	.372	.372
Slab Wt. (psf)	50	56	63	69	75	81	88	94	100	106	113	119	125
CLEAR SPAN	**FACTORED USABLE SUPERIMPOSED LOAD (psf)**												
6'-0"	703	923											
6'-6"	589	775											
7'-0"	498	657	907										
7'-6"	425	562	778	986									
8'-0"	365	485	673	856									
8'-6"	315	420	586	747	935								
9'-0"	273	367	513	656	822								
9'-6"	238	321	452	579	727	894	980						
10'-0"	208	282	399	513	646	795	872						
10'-6"	181	243	317	410	539	661	779	882					
11'-0"	159	214	281	365	482	592	699	792	964				
11'-6"	139	189	249	326	432	532	629	713	870	994			
12'-0"	122	167	222	291	388	479	568	644	787	901			
12'-6"	107	148	197	261	349	433	514	583	715	819	967		
13'-0"	94	131	176	234	315	392	465	529	650	746	862		
13'-6"	82	116	157	210	285	355	423	481	593	681	806	959	
14'-0"	71	102	139	188	257	322	384	438	541	623	730	880	939
14'-6"	61	90	124	169	233	293	350	400	495	570	678	809	863
15'-0"	53	79	110	151	210	266	319	365	453	523	623	745	795
15'-6"	45	69	97	136	190	242	291	333	416	480	573	688	733
16'-0"		60	86	121	172	220	265	305	381	442	528	635	678
16'-6"		51	76	108	156	200	242	279	350	406	487	587	627
17'-0"		44	66	96	140	182	221	255	322	374	450	543	580
17'-6"			57	86	127	165	201	233	296	345	416	503	538
18'-0"			49	76	114	150	184	213	272	318	384	467	499
18'-6"			42	66	102	136	167	195	250	293	355	433	463
19'-0"				58	91	123	152	178	230	270	329	402	429
19'-6"				50	81	111	138	162	211	249	304	373	399
20'-0"				43	72	100	125	147	194	229	281	346	370

Note: See Fig. 7-1 for reinforcing bar details.

SOLID ONE-WAY SLABS—END SPAN | **Top Steel for $-M_u$**

f'_c = 3,000 psi | Grade 60 Bars | ρ = 0.0050

Thickness (in.)	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10
Top Bars	#4	#4	#4	#4	#5	#5	#5	#5	#5	#6	#6	#6	#6
Spacing (in.)	12	12	11	9	12	11	10	10	9	12	11	10	10
Bottom Bars	#4	#4	#4	#4	#4	#5	#5	#5	#5	#5	#6	#6	#6
Spacing (in.)	12	11	10	8	8	12	11	11	10	9	12	11	11
Top Bars Free End	#4	#4	#4	#4	#4	#4	#4	#4	#4	#4	#4	#4	#4
Spacing (in.)	12	12	12	12	12	12	12	12	12	12	12	12	12
T-S Bars	#3	#3	#3	#3	#4	#4	#4	#4	#4	#4	#4	#5	#5
Spacing (in.)	15	13	12	11	18	17	15	14	13	13	12	18	17
Areas of Steel (in.^{2}/ft) Top Interior	.200	.200	.218	.267	.310	.338	.372	.377	.413	.440	.480	.528	.528
Bottom	.200	.218	.240	.300	.300	.310	.338	.338	.372	.413	.440	.480	.480
Slab Wt. (psf)	50	56	63	69	75	81	88	94	100	106	113	119	125
CLEAR SPAN	**FACTORED USABLE SUPERIMPOSED LOAD (psf)**												
6'-0"	700	906											
6'-6"	586	761	967										
7'-0"	496	645	821										
7'-6"	423	552	704	988									
8'-0"	363	475	608	856	986								
8'-6"	314	412	528	747	861	976							
9'-0"	272	359	462	656	757	858							
9'-6"	237	314	405	579	669	759	916						
10'-0"	207	276	357	513	593	674	814	890					
10'-6"	158	191	248	364	481	591	722	790	957				
11'-0"	138	167	218	323	429	528	647	708	859	987			
11'-6"	120	146	192	287	383	473	582	636	774	890			
12'-0"	105	127	169	256	343	426	524	574	700	806	952		
12'-6"	91	111	149	228	308	383	473	518	634	731	865		
13'-0"	79	97	131	204	277	346	428	469	575	664	787	937	999
13'-6"	68	84	115	182	249	312	388	426	523	605	719	857	914
14'-0"	58	73	101	162	224	282	352	386	477	552	657	785	837
14'-6"	49	62	88	145	202	256	320	351	435	505	602	721	769
15'-0"	42	53	76	129	182	231	291	320	397	462	552	662	707
15'-6"		45	66	115	163	209	264	291	363	423	507	610	651
16'-0"			56	102	147	190	241	265	332	388	466	562	600
16'-6"			48	90	132	171	219	241	304	356	429	519	554
17'-0"			40	79	118	155	199	220	278	327	395	479	511
17'-6"				69	105	140	181	200	255	300	363	442	473
18'-0"				60	94	126	164	182	233	275	335	409	437
18'-6"				51	83	113	149	165	213	253	309	378	405
19'-0"				44	73	101	135	149	195	232	284	350	374
19'-6"					64	90	122	135	178	213	262	324	347
20'-0"					56	80	109	122	162	195	241	300	321

Note: See Fig. 7-1 for reinforcing bar details.

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STANDARD ONE-WAY JOISTS (1) MULTIPLE SPANS	30" Forms + 5" Rib @ 35" c.-c. (2) FACTORED USABLE SUPERIMPOSED LOAD (PSF)											f'_c = 4,000 psi f_y = 60,000 psi
	8" Deep Rib + 3.0" Top Slab = 11.0" Total Depth											
TOP BARS Size @	# 4 12	# 4 12	# 4 11	# 4 9	# 5 11	End Span Defl. Coeff. (3)	# 4 12	# 4 12	# 4 10.5	# 4 8		Int. Span Defl. Coeff. (3)
BOTTOM BARS # #	# 3 # 4	# 4 # 4	# 4 # 5	# 5 # 5	# 5 # 6		# 3 # 3	# 3 # 4	# 4 # 4	# 4 # 5		
Steel (psf)	.50	.60	.72	.89	1.09		.56	.63	.78	1.00		
CLEAR SPAN	END SPAN						INTERIOR SPAN					
14'-0"	184	258	274*	285*	298*	.450	194	302	312*	322*		.277
	0	0	346	436	464*		0	0	410	538		
15'-0"	150	215	244*	253*	263*	.593	159	253	280*	289*		.365
	0	0	292	370	428*		0	0	347	459		
16'-0"	123	180	219*	226*	235*	.767	131	213	253*	261*		.472
	0	0	247	316	382*		0	0	296	394		
17'-0"	100	151	197*	203*	211*	.978	107	180	230*	237*		.602
	0	0	210	271	339		0	0	254	340		
18'-0"	81	126	178*	184*	190*	1.229	87	152	210*	216*		.756
	0	0	179	233	295		0	0	218	295		
19'-0"	65	105	153	167*	172*	1.525	71	129	188	198*		.939
	0	0	0	202	257		0	0	0	257		
20'-0"	51	88	131	152*	157*	1.873	56	109	162	181*		1.153
	0	0	0	175	224		0	0	0	225		
21'-0"		72	112	139*	143*	2.276	44	92	140	167*		1.401
		0	0	151	196		0	0	0	197		
22'-0"		59	95	128*	131*	2.742		77	121	154*		1.687
		0	0	131	172			0	0	173		
23'-0"		48	80	113	120*	3.276		64	104	143*		2.016
		0	0	0	151			0	0	151		
24'-0"			68	98	110*	3.884		52	89	132*		2.390
			0	0	132			0	0	133		
25'-0"			56	84	102*	4.572		42	76	116		2.814
			0	0	116			0	0	0		
26'-0"			46	72	94*	5.349			65	102		3.292
			0	0	102				0	0		
27'-0"				61	86*	6.221			55	89		3.828
				0	89				0	0		

(1) For gross section properties, see Table 8-1.
(2) First load is for standard square joist ends; second load is for special tapered joist ends.
(3) Computation of deflection is not required above horizonal line (thickness ≥ $\ell_n/18.5$ for end spans, $\ell_n/21$ for interior spans).
(4) Exclusive of bridging joists and tapered ends.
*Controlled by shear capacity. +Capacity at elastic deflection = $\ell_n/360$.

PROPERTIES FOR DESIGN (CONCRETE .36 CF/SF) (4)												
NEGATIVE MOMENT STEEL AREA (SQ. IN.)	.58	.58	.64	.78	.99		.58	.58	.67	.88		
STEEL % (UNIFORM)	1.03	1.03	1.12	1.37	1.75		1.03	1.03	1.18	1.54		
(TAPERED)	.55	.55	.60	.74	.94		.55	.55	.63	.83		
EFF. DEPTH, IN.	9.8	9.8	9.8	9.8	9.7		9.8	9.8	9.8	9.8		
– ICR/IGR	.208	.208	.222	.256	.298		.208	.208	.230	.278		
POSITIVE MOMENT STEEL AREA (SQ. IN.)	.31	.40	.51	.62	.75		.22	.31	.40	.51		
STEEL %	.09	.12	.15	.18	.22		.06	.09	.12	.15		
EFF. DEPTH, IN.	9.8	9.8	9.7	9.7	9.6		9.8	9.8	9.8	9.7		
+ICR/IGR	.164	.207	.254	.303	.353		.121	.164	.207	.254		

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