EDENWALD NEW TOWER

Technical Report #2

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EXECUTIVE SUMMARY

The purpose of this report is to investigate four alternative floor solutions to the post tensioned system used in the Edenwald New Tower. This 12 story addition will house independent and assisted living apartment units, as well as amenities for residents such as an indoor pool, a walking track, a fitness center, an outdoor terrace with putting green, and a pub & lounge.

Before selecting any design solutions, two design considerations were noted. First, for reasons fully explained later in this report, floor depth was considered a priority. Systems which would hopefully provide shallow depths would be better suited for this building. Secondly, inconsistencies in bay dimensions provided various aspect ratios, the lower of which ruled out one way systems, which are better suited for rectangular bays.

Keeping these considerations in mind, the four alternative floor solutions were selected. They are two way concrete slab with drop panels, two way concrete joist (waffle slab), hollow core concrete planks, and concrete slab with composite steel. To compare these systems, a typical interior panel was selected from the northeast wing, which is highlighted in Figure 1 on page 7. When appropriate, the bays immediately adjacent in the E-W dimension were used for a frame analysis. RAM Structural Systems was used for the steel design, and the PCI Design Handbook and the CRSI Handbook were used for the concrete designs, as explained later in the report.

The resulting designs were compared according to ten criteria: cost, fire rating, lead time, constructability, slab depth, total depth, aesthetics, column grid changes, minimum column size, and lateral system effects. Only after examining each system in comparison with the others were judgments made on whether or not the system would suffice as a reasonable substitute. However, it is important to note that this comparison is preliminary. Various assumptions were required to arrive at general conclusions about each system, and these are documented in the following pages.

The results of the following comparisons lead me to conclude that the existing system is the best overall choice for the Edenwald New Tower. The post-tensioned system allows for thin ceiling-floor sandwiches while also sufficiently supporting the longer spans reaching up to 30 feet. This is also an economical choice in comparison with other concrete flat plate systems. The steel design was immediately rejected because of floor depth and the lack of necessity to change from a concrete design. The hollow core slab was also rejected, again for unsatisfactory floor depth. Though the remaining systems are plausible substitutes, the failed to provide enough evidence to suggest they would make a better overall selection than post tensioning. The reasons for this, and further commentary on the post tensioning system, can be found in the conclusions following the comparison table on page 14.

STRUCTURAL SYSTEM OVERVIEW

Foundation:
The geotechnical analysis of the sub-surface conditions prior to construction revealed great variances in soil type and depth to bedrock, ranging from 50 to 150 feet deep, making deep foundations impractical. Given two recommendations from the geotechnical engineer, it was decided by the designers to use a geopier system as opposed to an alternative of driven HP 12x74 piles. Comprised of densified "rammed" stone aggregate piers, geopiers are referred to as "intermediate foundation systems" in that they strengthen, stiffen and reinforce soil layers beneath the building. The use of this option provided the opportunity to utilize a shallow foundation system of typical spread footings. (It should be noted, however, that pre-existing utilities only discovered upon excavation in the north end of the site required the use of the HP piles, in that localized area only.) The geopiers were determined to require a 30 inch diameter, and range from 20 to 30 feet in length. The allowable bearing pressure of the strengthened soil beneath the building was then determined to be 6 ksf beneath the tower, and 4 ksf beneath the parking garage. Total settlement expected from the geopier design amounts to one inch.

All concrete used in the Edenwald New Tower is normal weight (145 pcf dry unit weight). Footings, grade beams and slabs on grade have a minimum 28-day strength of 3000 psi. Shear wall footings have a minimum 28-day strength of 4000 psi. The slab on grade is reinforced with 6x6-W2.9x2.9 WWF on a vapor barrier on 4 inches of granular subbase.

Floor System:
The typical floor system used is a 9 inch, post-tensioned concrete slab having a minimum 28-day strength of 5000 psi. In specific locations where the post tensioned system is not feasible and/or practical, reinforced one way slabs were used, ranging in thickness from 8 to 9 inches, with cast in place concrete beams, both requiring a minimum 28-day strength of 5000 psi.

Roof System:
The flat roof system is almost identical to the typical floor system. Still utilizing the post-tension reinforcement, the slab thickness reaches up to 16 inches underneath the penthouse. The penthouse is supported by a steel braced frame and is covered by 1.5 inch deep, wide rib, 20 gage galvanized metal deck. The pentouse roof is supported by a combination of steel W shapes and 12k3 joists. The columns supporting the penthouse are W8x31 shapes.

Columns:
The building is supported by rectangular concrete columns laid out in a geometric grid. The columns range in size, the most common being 22x22 and 22x36. The largest column found in the building is 22x60. Column loads range from 203 kips in the garage to 1600 kips at the base of the tower. From the ground level to the seventh floor, the columns are required to have a minimum 28-day strength of 6000 psi. From the seventh floor to the roof, that value decreases to 5000 psi.

Lateral System:
The building is laterally supported in both the N-S and E-W directions by a total of 15 simply reinforced concrete shear walls, with thickness ranging from 12 to 14 inches. These shear walls are required to have a minimum 28-day strength of 5000 psi. Located throughout the building, the shear walls are often conveniently placed around stair and elevator shafts. All but one of the 15 shear walls run the entire height of the building.

ORIGINAL DESIGN LOADS

Gravity: Superimposed Dead Loads

Item	Design Value
Typical Floor Areas	30 psf
Typical Parking	5 psf
Parking above occupied space	35 psf
Garage Roof	35 psf
Main Roof	30 psf

Gravity: Live Loads

Item	Design Value	Comment (Values found in Table 4-1 of ASCE 7)
Framed Floor Areas	40 psf	Code Minimum: 40 psf (residential)
Lobbies/Stairs/Exits	100 psf	Code Minimum: 100 psf
Corridors above 1st Floor	100 psf	Code Minimum: 100 psf
Parking Decks	50 psf	Code Minimum: 40 psf
Balconies	100 psf	Code Minimum: 100 psf
5th Floor Terrace/Roof	100 psf	Code Minimum: 100 psf (roofs used for assembly purposes)

Gravity: Roof Live Loads

Item	Design Value	Comment
Roof Live Load (snow load used when greater than 30 psf)	30 psf	Code Minimum: 20 psf (ordinary flat roof) (See Table 4-1 of ASCE 7
Roof Snow Load	P_f = 19.25 psf C_e = 1.0 I = 1.1 C_t = 1.0	Calculated Snow Load: Pf = 19.25 psf (See Appendix, calculated according to chapter 7 of ASCE 7)

Original Design Loads

Lateral Loads: Seismic

Seismic Use Group: II
Sesimic Importance Factor: Ie = 1.25
Mapped Spectral Response Coefficients: SDS = 0.210 g SD1 = 0.070 g
Site Class: D
Spectral Response Coefficients: SDS = 0.224 g SDS = 0.112 g
Sesimic Design Category: B
Design Base Shear: V = 997 kips
Seismic Response Coefficient: Cs = 0.022
Response Modification Factor: R = 5.0
Analysis Procedure: Equivalent Lateral Force Procedure

Lateral Loads: Wind

Basic wind speed (3-sec gust) = 90 mph
Importance Factor: 1.15
Exposure Category: B
Internal Pressure Coefficient: Gcpi = ±0.18

Selection of Typical Bay

Figure 1:
Typical Tower Floor
(Bay of interest in red)

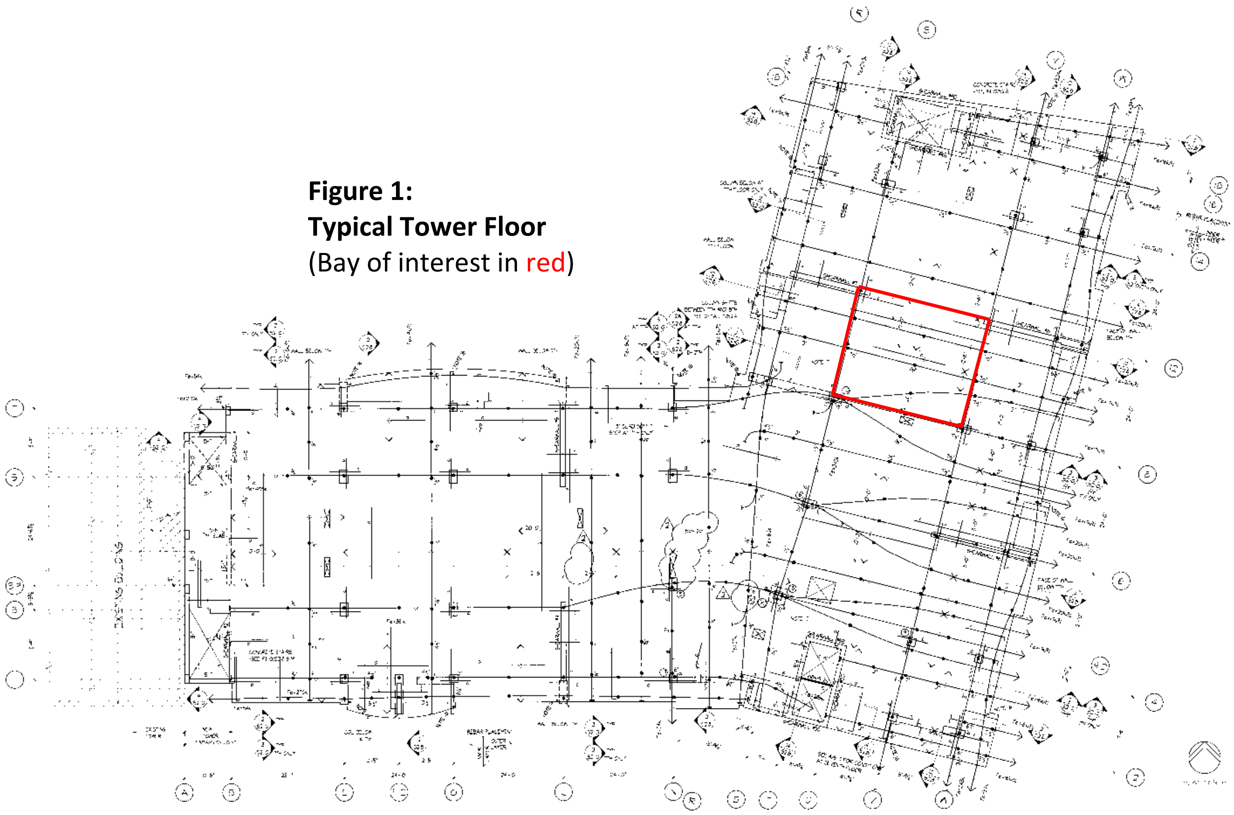


Figure 2:
Existing Column Grid

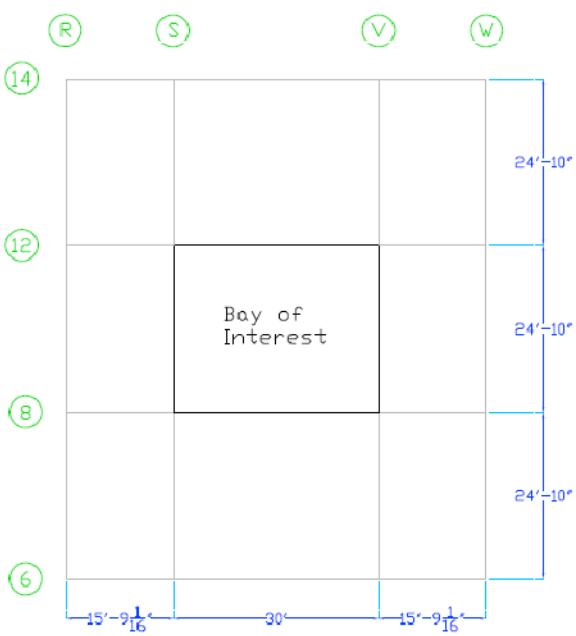
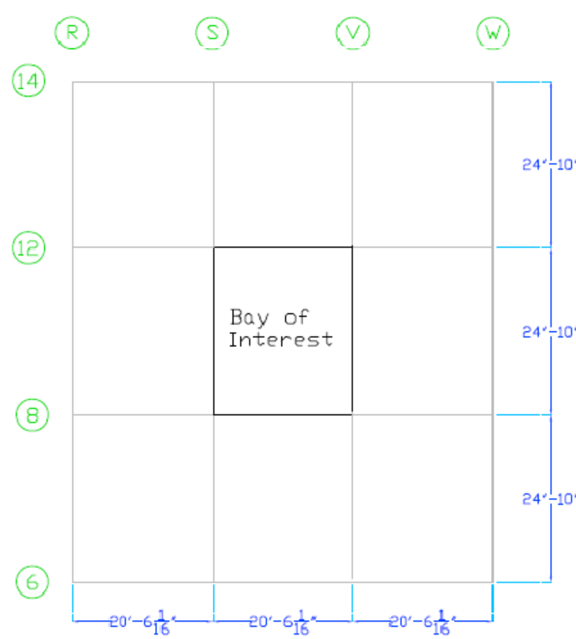


Figure 3:
Revised Column Grid



SYSTEM ANALYSIS OVERVIEW

In the following paragraphs, I will compare four floor systems to the existing post tensioned flat plate system: steel composite, two-way concrete joists, hollow core concrete slab, and two way concrete slab with drop panels.

Though many factors go into selecting floor systems, floor depth was the one on which most of my immediate decisions were based upon. The existing system of post-tensioned concrete has a slab thickness of 9 inches, making it difficult to justify switching to a system that dramatically increases this value. Additionally, it was made clear to me by Whiting-Turner representatives on site that complications had arisen from a slim envelope through which the MEP systems were placed between the dropped ceiling and the bottom of the slab. Though this may be in part due to relatively short story heights (in some floors only 9'-4"), I took their comments into consideration when considering floor depth.

On the preceding page, Figure 1 shows the location of the typical bay located on a typical tower floorplan. Beneath that are Figures 2 and 3: two enlargements to show existing column line locations and revised column line locations. In the following paragraphs, each floor system will be explained and it will be determined whether a revised column grid would be advantageous on a case by case basis.

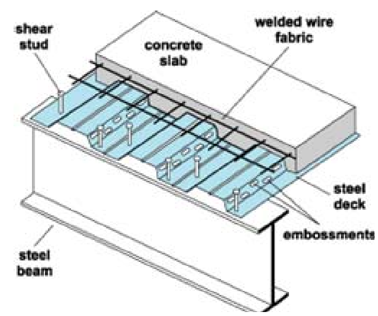
Though specific assumptions will be listed for each design solution, there are two general assumptions made for the following analyses.

1. For CRSI tabulated values, the existing bay will be considered square. The typical bay has an existing l_1/l_2 ratio of 1.2. The tables found in the CRSI manual are based upon square panels ($l_1/l_2 = 1$). However, the manual states that for rectangular panels, as this value approaches 1, the panel may assumed to be square using the longer dimension. Clearly the design would differ for aspect ratios of 1.2, however, the differences would be minor, and most likely found in reinforcement detailing. For the purpose of comparing the overall system, the manual will provide sufficient information.
2. For the sake of this report, it will be assumed that columns exist at all column line intersections found in Figure 2 and Figure 3. In reality, shear walls exist from R-12 to S-12 and from V-12 to W-12. The presence of a shear wall increases the stiffness of the support infinitely, but there are not shear walls located adjacent to every panel. Therefore the systems must be designed according to the less conservative condition of column supports.

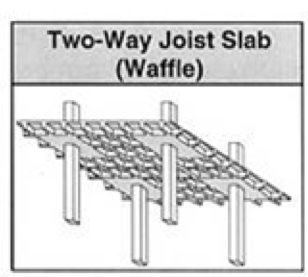
Post Tensioned Flat Plate Concrete Floor

This is the existing floor system, and so the following systems will be more or less compared to this one. The drawings indicate the necessary force in kips required from the tendons throughout the slab, and the contractor is required to provide the adequate reinforcing to meet that value. Using values correlating to unbounded, ½" diameter, 7-wire strands, I performed a spot check in an attempt to get matching results. Choosing the E-W west direction, along the 30 foot span, I found the drawings dictated a force be provided (P_{eff}) that varied from 17 kips/foot to 20 kips/foot along the span for a 9 inch slab. (I am not entirely sure why this value changes at about midspan, but the building's complex shape is most likely the cause.) Through calculations found in the Appendix, I found P_{eff} to be 17 kips/foot for an 8 inch slab. The extra inch of slab thickness in the design can be accounted for by a number of reasons, and the most likely one is that my panel probably did not provide the controlling condition in the building, specifically in regards to punching shear. However, the post-tensioning force calculated is identical to that provided. Typical design span/depth ratios for post tensioned, two way solid slabs supported by columns are between 40 and 45. For the longest span, 30 feet, the existing system provides a span/depth ratio of 40 – right on target.

Steel Composite Floor
Designed with RAM Structural Systems



This design solution departs the most radically from the existing design because the fundamental structural material is now steel, not concrete. A composite system was investigated over non-composite because of the additional depth required in non-composite design. Using the USD design manual, I first selected 20 gauge, 3" LOK floor deck with a 5.5" slab in accordance with tabulated unshored, 3-span condition values. Using this data, I then constructed a RAM model to design the rest of the composite system. Though the revised column grid may not be necessary, it seemed appropriate to check both. The details of both reports can be found in the Appendix. Keeping with the original framing layout resulted in a slab depth of 5.5 inches, but at the girder cross section yielded a total depth of 19.25 inches. However, adjusting the column grid to achieve smaller spans (25 feet as opposed to 30) resulted in smaller girder sizes, reducing the maximum total depth to 15.37 inches.

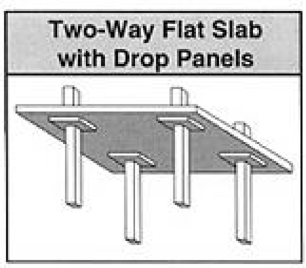


Two Way Concrete Joists (Waffle Slab)
Designed with CRSI Handbook

Waffle slabs are only advantageous with longer spans and heavier loads. Figure 2 shows the bays adjacent to the bay of interest to be very rectangular, which suggests inefficient two-way behavior, making the heavy waffle system unpractical. Therefore, a change to the revised grid layout became justified for the two way waffle system. (Using the new layout, the l_1/l_2 value now equals 1.21.) A total load of 110 psf was derived from a 1.4D + 1.7L load combination, which is the one used in the table design. I found that a design of 19"x19" voids with 5" ribs spaced at 24" on center, with a total depth of 11 inches was adequate for the desired load. See the Appendix for reinforcing details.

Hollow Core Concrete Slab
Designed with PCI Design Handbook

For this alternative floor solution, I found that 4'x6" panels with a two inch concrete topping could carry a safe superimposed service load of 98 psf for a span of 25 feet. This unfactored load is greater than the designed unfactored load of 70 psf for planks running in the direction of the 24'-10" span. Reinforcement was determined to be (9) 6/16" diameter straight bars per slab. While the total slab thickness is only 8 inches, the depth of the beams must be considered when evaluating the solution as a whole. As designed, the longest beam span is 30 feet. A shift in the column grid to achieve smaller spans (and identical bays) would put the maximum beam span at just over 25 feet, but for this design I will retain the original column layout. The hollow core system can be supported by various types of beams, however for this design the most practical and shallowest type would be a precast, prestressed , inverted tee beam. The linear unfactored load to be carried (including the weight of the planks) was found to be 3571 plf. Specifications for the required beam can be found in the appendix. The maximum depth of the system now becomes 24 inches. This, of course, is only located at the beam cross section, but still presents a significant challenge to the MEP system designers, especially on floors with shorter heights.



Two Way Concrete Slab with Drop Panels
Design with CRSI

For this design, a shift in the column grid became advantageous to reduce overall slab thickness. (Additionally, for this design, the longer span resulting in thicker slabs will have a significant cost increase when carried through the entire building.) As stated above, the CRSI uses a 1.4D + 1.7 load combination, and so using a factored superimposed load of 110 psf, I found that a 8.5 inch slab would suffice for a square panel based upon the my 24'-10" span. Square columns of at least 18 inches in depth are required for this design. See the Appendix for reinforcement.

SYSTEM COMPARISON

The following factors have been selected to compare the four floor systems: cost, fire rating, constructability, deflection, total depth, lead time, aesthetics, column grid changes, and lateral system effects. Though some of these criteria will have more importance than others, it is appropriate to consider them all.

Cost: Cost may be one of the most influential factors, for obvious reasons. To use the 2007 RS Means Assemblies Manual, I first made an assumption. Because this is simply a comparison of cost, (as opposed to a detailed cost analysis) all systems will be considered in the unchanged column grid of a 24'-10" x 30' bay. This then will allow me to compare to tabulated values for a 25' x 30' bay, which will provide a reasonable comparison of the systems. According to RS Means, the total costs of these assemblies, to include material and labor, are as follows:

Flat Slab with Drop Panels	\$14.70/sq. ft.
Waffle Slab	\$18.77/sq. ft.
Hollow Core	\$19.56/sq. ft.
Composite Steel	\$14.62/sq. ft.

Calculating a standard cost of the post tensioning system is not entirely possible. Not only does the assemblies manual fail to provide an estimate, but this system's cost is directly effected by geography. Contractors who are familiar and comfortable with post tensioning construction will often offer bids which make it more economical than a standard flat plate system, however where the technology is less commonly used, the price can rise. In Baltimore, where post tensioning is common, it is reasonable to assume there is no advantage or disadvantage economically to using post-tensioning when compared with the flat plate system.

Fire Rating: The building is required to have a 2 hour fire rating, and so it is imperative that all design solutions meet or exceed that value. Concrete slabs with depths greater than 5 inches provide this level of protection standing alone. Thus the existing design, the hollow core and the flat slab systems are all sufficient in of themselves. However, the waffle slab, with its 3 inch slab thickness, and the composite steel system will both need spray on fireproofing. While this is common and not enough of a reason to exclude a particular design, it will have significant cost ramifications for the owner.

Lead Time: Because the apartment units have been leased already and construction is still months away from completion, lead time is significant for this project, as slower construction means a direct loss in profits if it causes the completion date to be pushed back. The only systems which have lead time issues would be steel and precast concrete. Lead time for fabrication can be 10 weeks, or even several months when considering shop drawing phases. This makes change orders difficult, to say the least. In contrast, cast-in-place concrete needs literally no lead time at all.

Constructability: Though all the systems considered are reasonable for an experienced contractor, there is some variance in difficulty. Steel design requires more skilled labor than does concrete, for welds and other connection details. The different types of cast-in-place concrete design are mainly a function of reinforcement placement and formwork layout, which presents no great obstacles alone. Though due to complex formwork, the waffle slab system is considered labor intensive. The hollow core system is perhaps the simplest, although the presence of a crane to move the planks into position is obviously required. The exisiting system is unique in that while the creation of the slab is not signficantly challenging, the system must be post-tensioned after the concrete has cured. However, this process is neither time-intensive nor difficult, meaning there are no real constructability concerns here either. One specfic negative to post tensioning, however, is the danger that comes with drilling through the slab. Rupturing a tendon can not only be dangerous from the violence of the released energy, it can also cause failure in an entire frame if the bays were not designed conservatively. This makes placing vertical elements through the slab tricky if the contractors failed to place sleeves in the formwork before the pour.

Deflection: All the systems were designed from manuals or software to meet or exceed deflection requirements based on L/240 total load deflections and L/360 live load deflections. Because of the insensitive nature of the occupancy and lack of specific vibration criteria, passing code requirements will be enough for this report, and no further investigation will be necessary.

Floor Depth: As stated before, this criteria is of particular interest in this report. Because of the relatively slim floor depth of the existing system (nine inches uniformly), alternative systems of greater depth present problematic conditions in a building where contractors are already dealing with slim ceiling envelopes to run MEP systems due to small story heights. There are two values to compare the systems with: slab depth, and total depth. Total depth results from systems which are supported by beams or girders, or utitlize drop panels. These points are the ones which present contractors with the greatest challenges.

	Slab Depth	Total Depth
Flat Slab with Drop Panels	8.5”	14”
Waffle Slab	3”	11”
Hollow Core	8”	24”
Composite Steel	5.5”	19.25”

Aesthetics: While it is true some of the systems provide inherent challenges for contractors, all do provide necessary depth for MEP systems. None of the systems, based on a cursory overview, prevent the use of recessed lighting or other aesthetic lighting options. That being said, this criteria finds no alternative with any specific advantage or disadvantage. The only exception may be on floors where girders cannot be completely hidden from sight, and MEP systems are also forced into unsightly locations.

Column Grid Changes: Changing the column grid is not an ideal condition by any means. It would require revisions to the architectural and foundation plans and could potentially impact the lateral force resisting system. However, investigation of alternative floor systems required a reduction in span length for some cases, as stated above. Also, while not all spans require the change, even the ones that do not would most likely benefit from it. For instance, in the case of the steel composite and hollow core systema, the longer spans are plausible, but a shorter span means a smaller beam, which reduces total floor depth. Thus changing the column grid must be considered alongside other criteria to determine whether the positives outweigh the negatives. For the existing column spacing, however, the post tensioned flat plate clearly provides an excellent solution to the long, 30 foot spans.

Minimum Column Size: Many of these systems are required to have minimum column sizes to address the issue of punching shear. Though this factor is not of significant importance in comparing the designs directly, it is considerably important that the designer checks against this should a particular system be selected, in the case that the column sizes need to be enlarged. For this building, that is highly unlikely as the columns are already quite conservatively large, as discussed in Technical Report 1.

In an attempt to ratify the difference in size between the designed columns and my column from Technical Report 1, I ran a check to see if punching shear dictated the size of the columns. Calculations which are found in the Appendix show that a typical 22"x36" column provides 1.7 times more shear capacity than required. Next I checked the column I designed in Technical Report 1, which was 14"x22", with an 8" slab. Though only by a hair, even this combination provides adequate shear capacity. The initial thought was that the lack of shear caps (common in apartment and hotel buildings) would force the designers to use bigger columns to resist punching shear, but still my calcuations cannot account for the size that was provided. The only other reason I can suggest is that, as said in the last report, the shear walls will not take all the lateral force and a dual system will require columns to be designed for axial and bending interaction. This will be more fully investigated in Technical Report 3.

Lateral System Effects: For the cast-in-place concrete systems, shearwalls will most likely remain the ideal lateral resisting system. For the steel, and possibly the precast systems, the use of moment frames needs to be considered. These systems will be investigated further in Technical Report 3.

SYSTEM COMPARISON CHART AND CONCLUSIONS

Factor	Existing	Flat Slab with Drop Panels	Hollow Core	Waffle Slab	Composite Steel
Cost	n/a	$14.70	$19.56	$18.77	$14.62
2 Hr. Fire Rating	Yes	Yes	Yes	No	No
Lead Time	No	No	Yes	No	Yes
Constructability	Medium	Easy	Easy	Medium -Hard	Medium
Slab Depth	9”	8.5”	8”	3”	5.5”
Total Depth	9”	14”	24”	11”	19.25”
Aesthetics	Good	Good	Possible Beam Problems	Good	Possible Beam Problems
Column Grid Changes	No	Yes	No	Maybe	Maybe
Minimum Column Size	14”x22” (or square equivalent)	18”x18”	n/a	13”x13”	n/a
Lateral System Effects	n/a	No	Maybe	No	Yes

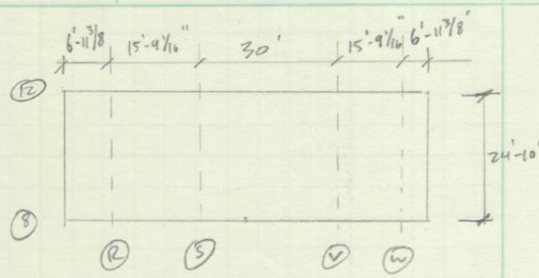
Conclusions: The only system which can be immediately ruled out is composite steel. Concrete design seems to make much more sense for this building. Though the weight of concrete is greater than steel, it is relatively easy to construct and makes for thinner floors. There would need to be a compelling reason to switch to steel design, and there is not. After taking a more detailed look at the above table, the only other system which I will rule out is hollow core. This is largely because a total depth of 24 inches is unsatisfactory for this building design, for all the reasons mentioned previously. Though the remaining two alternatives, flat slab with drop panels and waffle slab, are viable substitutes, *the existing system seems to be the best overall solution to meet the building’s design requirements*.

Having said that, there are several other items to note about the post tensioned floor not mentioned above. One advantage is the system’s crack control and water-tightness. (The cracks from the positive moment are still present, but the camber introduced to the slab keeps them tighter than in a conventional slab.) Though the water-tightness is not of great value here (parking garages are excellent examples of where this facet becomes valuable in design), the crack control means increased durability and lifespan of the building. A second advantage is that the high strength of the tendons provide for superb structural integrity when considering catastrophic loading. Should the slab fail, the tendons in many cases would still keep it from collapsing to the floor below.

On the other hand, there are some unique concerns with a post tensioning system not addressed in the criteria above. Perhaps the most noteworthy is the shortening caused from shrinkage. Though the shrinkage cracks are closed from the post tension force, the slab shortens which causes cracks to develop near the ends. This becomes particularly problematic when considering restraint to shortening: how the slab is connected to shear walls or other rigid members. In the case of this building, the slabs connection to the shear walls may need to be monitored as there will be some shortening from the slab in the direction of the shear walls' stiffer dimension. It is also probably due to this behavior that the first floor is designed as a series of one way slab with beams and two way flat plates. If a post-tensioned floor was used, there could be damage at the connection to the foundation wall due to the stiffness of that wall as the slab shortens.

APPENDIX: POST TENSIONING FORCE

Post Tension (E-W Panel)



Loads:

Super imposed Dead : 30 psf

Live : 40 psf

Framing : Slf wT

2hr Fire Rating

Materials:

Conc: Normal Weight = 150 pcf

$f'_c = 5000$ psi

$f'_{ci} = 3000$ psi

Rebar: $f_y = 60$ ksi

PT : Unbonded Tendons ASTM A416, Grade 270

1/2" ϕ, 7wire strands, $A = 0.153 \text{ in}^2$

$f_{pu} = 270$ ksi

Estimated pre stress losses = 15 ksi

$f_{se} = 0.7(270) - 15 = 174$ ksi

$P_{eff} = A^* f_{se} = (0.153)(174 \text{ ksi}) = 26.6$ k/tendon

Preliminary Slab Thickness

$L/h = 45$

Longest span: 30'

$h = 30(12)/45$

= 8" preliminary thickness ⟹ Slf wT = 8"(150 pcf) = 100 psf

Loading

LL Reduction

Interior Bay

$A_T = (30')(24'-10") = 745 \text{ ft}^2$

$K_{LL} = 1$

$LL = 0.83 LL_o = .80(40) = 32.0$ psf

Tendon Profile:

CG measured from bottom of slab

APPENDIX: POST TENSIONING FORCE

Design East West Panel

Use Equiv Frame Method

Bay width: 24'-10"

Ignore col stiffnesses in equations for simplicity

Calc Section Properties

$A = bh = (298")(8") = 2384 \text{ in}^2$

$S = bh^2/6 = (298)(8)^2/6 = 3179 \text{ in}^2$

Set Design Parameters

Allowable stresses

At time of jacking

$f'_{ci} = 3000$ psi

Comp $= (.6)(3000) = 1800$ psi

Tens: $3\sqrt{3000} = 164$ psi

At service loads

$f'_c = 5000$ psi

Comp $= 0.45(5000) = 2{,}250$ psi

Tens $= 6\sqrt{5000} = 424$ psi

Avg Precomp Limits

$P/A = 125$ psi min (ACI 18.12.4

$= 300$ psi max

Target Load Balances

60% – 80% of DL (slf wt) for slabs (approx for hand calcs)

$0.75\, w_{DL} = .75(100\text{psf}) = 75$ psf

Cover Requirements

Restrained slab – 3/4" top & bottom

Tendon Profile (see prev page)

$a_{Cant} =$

$a_{Ext} = 7.5" - 3" = 4.5"$

$a_{Int} = 7.5" - 1.5" = 6"$

APPENDIX: POST TENSIONING FORCE

Prestress Force required to balance 75% of self wt DL

Int Bay

$w_b = 75\ psf\,(24.833') = 1862.5\ plf = 1.863\ klf$

P = force needed to counteract load

$P = w_b L^2 / 8 a_{Int}$

$= 1.863(30)^2 / 8(6in/12)$

$= 419\ k$

Ext Bay

$P = 1.863(15.755')^2 / 8(4.5/12)$

$= 154^k$

$P = 419^k$ governs

Distributed Force

$419^k / 24'\text{-}10'' = 17^k/ft$

Force needed on Drawings: varies btwn $17^k/ft$ and $20^k/ft$

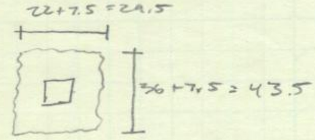
9" slab would merit more force, I used 8"

$\underline{P_{eff} = 419^k \text{ or } 17^k/ft}$

APPENDIX: POST TENSIONED SLAB PUNCHING SHEAR

Punching Shear

Check Interior 22x36 col



Slab $h = 9''$

$d = 7.5''$

$b_o = 2(29.5) + 2(43.5) = 146$

Factored Load $= 1.2(30) + 1.2\left(\frac{9}{12}(150)\right) + 1.6(40)$

$= 235$ psf

$V_u = 235\,(22.89' \times 24.83') = 133582\,\text{lb}$

↳ trib area

$\phi V_c = 0.75(4)\sqrt{f'_c}\, b_o\, d$

$= 0.75(4)\sqrt{5000}(146)(7.5)$

$= 232284\ \text{lb} > V_u$ ok

$\phi V_c = \left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f'_c}\, b_o\, d\,(.75)$

$= \left(\frac{40(7.5)}{146} + 2\right)\sqrt{5000}\,(146)(7.5)$

$= 313955\ \text{lb} > V_u$ ok

$\phi V_c = \left(2 + \frac{4}{\beta_c}\right)\sqrt{f'_c}\, b_o\, d(.75)$ $\quad \beta_c = \frac{36}{22}$

$= \left(2 + \frac{4}{36/22}\right)\sqrt{5000}(146)(7.5)(.75)$

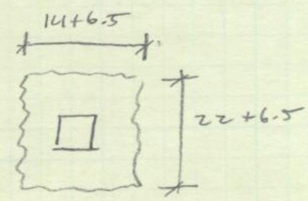
$= 344125.33\ \text{lb} > V_u$ ok

APPENDIX: POST TENSIONED SLAB PUNCHING SHEAR

Punching shear

Check col designed in Tech 1 with 8" slab

14 x 22



$h = 8"$

$d = 6.5"$

$b_o = 2(14+6.5) + 2(22+6.5)$

$= 98$

$V_u = 133582 \text{ lb}$ (from prev page)

$\phi V_c = \phi 4 \sqrt{f'_c}\, b_o d$

$= 0.75(4)\sqrt{5000}\,(98)(6.5)$

$= 135128 \text{ lb} > V_u$ ok

$\phi V_c = \phi \left(2 + \frac{4}{\beta_c}\right) \sqrt{f'_c}\, b_o d$

$= 0.75\left(2 + \frac{4}{(22/14)}\right)\sqrt{5000}\,(98)(6.5)$

$= 153554 > V_u$ ok

$\phi V_c = \left(\frac{\alpha_s d}{b_o} + 2\right) \sqrt{f'_c}\, b_o d$

$= \left(\frac{(40)(6.5)}{98} + 2\right)\sqrt{5000}\,(98)(6.5)(.75)$

$= 157189 \text{ lb} > V_u$ ok

APPENDIX: FLAT SLAB (FROM CRSI HANDBOOK)

f'_c = 4,000 psi
Grade 60 Bars

FLAT SLAB SYSTEM

SQUARE EDGE PANEL — With Drop Panels — No Beams

h = 8.5 in. = TOTAL SLAB DEPTH BETWEEN DROP PANELS

SPAN c.-c. $\ell_1 = \ell_2$ (ft)	Factored Superimposed Load (psf)	Square Drop Panel Depth (in.)	Square Drop Panel Width (ft)	Square Column (3) Size (in.)	Square Column (3) γ_f	Reinforcing Bars (E. W.) Column Strip (1) Top Ext.	+	Column Strip (1) Bottom	Column Strip (1) Top Int.	Middle Strip Bottom	Middle Strip Top Int.	Total Steel (psf)	Moments Edge (−) (ft-k)	Moments Bot. (+) (ft-k)	Moments Int. (−) (ft-k)
22	100	4.00	7.33	12	0.676	11-#4	1	10-#5	11-#5	10-#4	11-#4	2.01	79.1	158.2	213.0
22	200	4.00	7.33	15	0.746	11-#4	4	10-#6	15-#5	9-#5	8-#5	2.57	107.1	214.2	288.4
22	300	5.50	7.33	17	0.638	12-#4	2	18-#5	25-#4	12-#5	10-#5	3.08	135.1	270.1	363.6
22	400	7.00	7.33	18	0.629	13-#4	2	9-#8	18-#5	14-#5	12-#5	3.70	163.8	327.7	441.1
22	500	8.50	8.80	20	0.629	14-#4	1	12-#8	14-#6	8-#8	10-#6	4.67	192.3	415.9	517.8
23	100	4.00	7.67	12	0.786	12-#4	4	18-#4	19-#4	12-#4	12-#4	2.13	90.8	181.5	244.4
23	200	5.50	7.67	15	0.697	12-#4	4	16-#5	15-#5	16-#4	9-#5	2.64	123.3	246.7	332.1
23	300	7.00	7.67	17	0.630	13-#4	3	11-#7	12-#6	10-#6	17-#4	3.40	155.6	311.2	418.9
23	400	7.00	9.20	19	0.738	15-#4	5	18-#6	20-#5	9-#7	10-#6	4.00	187.3	374.5	504.2
23	500	8.50	9.20	20	0.658	16-#4	3	13-#8	12-#7	20-#5	16-#5	4.83	220.6	451.7	593.8
24	100	5.50	8.00	12	0.706	13-#4	2	21-#4	19-#4	9-#5	13-#4	2.27	103.9	207.7	279.6
24	200	7.00	8.00	15	0.633	13-#4	3	19-#5	15-#5	12-#5	10-#5	2.80	140.6	281.3	378.7
24	300	7.00	8.00	17	0.722	15-#4	4	17-#6	14-#6	11-#6	13-#5	3.71	177.9	355.7	478.9
24	400	8.50	9.60	19	0.630	16-#4	2	16-#7	15-#6	10-#7	16-#5	4.43	214.8	429.5	578.2
–25	100	5.50	8.33	12	0.766	13-#4	4	23-#4	14-#5	10-#5	13-#4	2.34	117.8	235.6	317.2
25	200	7.00	8.33	15	0.686	13-#4	4	15-#6	12-#6	10-#6	12-#5	3.11	159.8	319.5	430.1
25	300	8.50	8.33	17	0.643	15-#4	3	15-#7	14-#6	10-#7	15-#5	4.08	202.5	405.0	545.2
25	400	8.50	10.00	20	0.723	18-#4	5	14-#8	13-#7	9-#8	10-#7	5.01	243.4	486.8	655.3

SQUARE INTERIOR PANEL — With Drop Panels(2) — No Beams

h = 8.5 in. = TOTAL SLAB DEPTH BETWEEN DROP PANELS

SPAN c.-c. $\ell_1 = \ell_2$ (ft)	Factored Superimposed Load (psf)	Square Column (3) Size (in.)	Reinforcing Bars (E. W.) Column Strip Top	Column Strip Bottom	Middle Strip Top	Middle Strip Bottom	Total Steel (psf)	Concrete (cu. ft / sq. ft)
22	100	12	16-#4	11-#4	11-#4	11-#4	1.94	0.745
22	200	17	14-#5	9-#5	11-#4	11-#4	2.31	0.745
22	300	20	15-#5	18-#4	9-#5	8-#5	2.71	0.759
22	400	21	16-#5	14-#5	8-#6	14-#4	3.16	0.773
22	500	22	17-#5	13-#6	13-#5	12-#5	3.84	0.822
23	100	12	12-#5	12-#4	12-#4	11-#4	2.01	0.745
23	200	17	14-#5	16-#4	13-#4	11-#4	2.33	0.759
23	300	20	11-#6	21-#4	16-#4	9-#5	2.91	0.773
23	400	23	28-#4	16-#5	19-#4	16-#4	3.35	0.802
23	500	23	14-#6	20-#5	15-#5	13-#5	4.09	0.822
24	100	12	12-#5	9-#5	13-#4	12-#4	2.09	0.759
24	200	18	14-#5	18-#4	14-#4	12-#4	2.38	0.773
24	300	20	18-#5	11-#6	12-#5	10-#5	3.22	0.773
24	400	23	14-#6	10-#7	22-#4	12-#5	3.76	0.822
–25	100	12	13-#5	10-#5	13-#4	12-#4	2.12	0.759
25	200	18	16-#5	21-#4	16-#4	9-#5	2.65	0.773
25	300	20	18-#5	18-#5	10-#6	12-#5	3.43	0.787
25	400	23	12-#7	9-#8	9-#7	10-#6	4.32	0.822

NOTES: (1) 50 percent of these bars may be placed in the middle third of column strip. (2) Drop panels same size as for edge panels. (3) Same column size above and below slab.

APPENDIX: HOLLOW CORE (FROM PCI DESIGN MANUAL)

Strand Pattern Designation
76-S

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

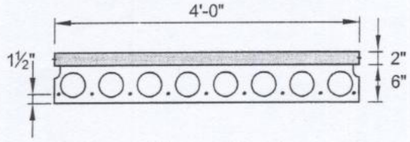
Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

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	Span, ft 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**

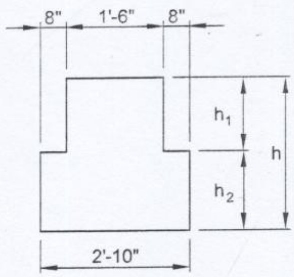
Strand Designation Code	Span, ft 12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
66-S	470	396	335	285	244	210	182	158	136	113	93	75	59	46	34				
	0.2	0.2	0.2	0.2	0.2	02	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2				
	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2	–0.3	–0.5	–0.7	–0.9	–1.2				
76-S		461	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.

APPENDIX: PRESTRESSED BEAM (FROM PCI MANUAL)

INVERTED TEE BEAMS

Normal Weight Concrete



Section Properties								
Designation	h in.	h_1/h_2 in./in.	A $in.^2$	I $in.^4$	y_b in.	S_b $in.^3$	S_t $in.^3$	wt plf
34IT20	20	12/8	488	16,082	8.43	1,908	1,390	508
34IT24	24	12/12	624	27,825	10.15	2,741	2,009	650
34IT28	28	16/12	696	44,130	11.79	3,743	2,722	725
34IT32	32	20/12	768	65,856	13.50	4,878	3,560	800
34IT36	36	24/12	840	93,616	15.26	6,135	4,514	875
34IT40	40	24/16	976	128,656	16.85	7,635	5,558	1,017
34IT44	44	28/16	1,048	171,157	18.58	9,212	6,733	1,092
34IT48	48	23/16	1,120	221,906	20.34	10,910	8,023	1,167
34IT52	52	36/16	1,192	281,504	22.13	12,721	9,424	1,242
34IT60	60	44/16	1,336	439,623	25.78	17,053	12,847	1,392

1. Check local area for availability of other sizes.
2. Safe loads shown include 50% superimposed dead load and 50% live load. 800 psi top tension has been allowed, therefore, additional top reinforcement is required.
3. Safe loads can be significantly increased by use of structural composite topping.

$f'_c = 5{,}000$ psi
$f_{pu} = 270{,}000$ psi
½ in. diameter
low-relaxation strand

Key

7822 – Safe superimposed service load, plf.
0.4 – Estimated camber at erection, in.
0.1 – Estimated long-time camber, in.

Table of safe superimposed service load (plf) and cambers (in.)

Desig-nation	No. Strand	y_s(end) in. y_s(center) in.	Span, ft 16	18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	48	50
34IT20	148-S	2.29		7822	6253	5092	4209	3522	2977	2537	2177	1879	1629	1417	1237	1081				
		2.29		0.4	0.5	0.6	0.7	0.7	0.8	0.9	1.0	1.1	1.1	1.2	1.2	1.2				
				0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1				
34IT24	178-S	2.59			9221	7524	6233	5229	4432	3789	3262	2826	2461	2151	1887	1660	1463	1291	1140	1007
		2.59			0.4	0.5	0.6	0.7	0.7	0.8	0.9	1.0	1.1	1.1	1.2	1.2	1.2	1.3	1.3	1.2
					0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.1	0.0	–0.1
34IT28	208-S	3.00					8641	7271	6183	5306	4589	3994	3495	3073	2713	2403	2134	1900	1694	1513
		3.00					0.5	0.6	0.7	0.7	0.8	0.9	1.0	1.0	1.1	1.2	1.2	1.3	1.3	1.3
							0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.1
34IT32	238-S	3.48						9589	8174	7032	6097	5323	4674	4124	3655	3252	2902	2597	2329	2093
		3.48						0.5	0.6	0.7	0.8	0.8	0.9	1.0	1.0	1.1	1.2	1.2	1.3	1.3
								0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2
34IT36	248-S	3.50								9223	8016	7015	6176	5466	4860	4338	3886	3492	3146	2840
		3.50								0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.1	1.1	1.2	1.2
										0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
34IT40	308-S	4.40									9720	8510	7497	6639	5907	5277	4731	4254	3836	3467
		4.40									0.6	0.7	0.8	0.9	0.9	1.0	1.1	1.1	1.2	1.3
											0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.4	0.4	0.4
34IT44	308-S	4.40											9362	8307	7406	6630	5958	5372	4857	4403
		4.40											0.7	0.7	0.8	0.9	0.9	1.0	1.0	1.1
													0.2	0.2	0.2	0.2	0.2	0.3	0.2	0.2
34IT48	338-S	4.73													8963	8037	7234	6533	5919	5376
		4.73													0.8	0.8	0.9	1.0	1.0	1.1
															0.3	0.3	0.3	0.3	0.3	0.3
34IT52	368-S	5.22														9503	8564	7745	7026	6392
		5.22														0.8	0.9	0.9	1.0	1.0
																0.3	0.3	0.3	0.3	0.3
34IT56	398-S	5.59																	8269	7532
		5.59																	1.0	1.0
																			0.3	0.3
34IT60	408-S	6.00																	9564	8721
		6.00																	0.8	0.9
																			0.3	0.3

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APPENDIX: WAFFLE SLAB (FROM CRSI MANUAL)

WAFFLE FLAT SLAB SYSTEM 19″ X 19″ Voids: 5″ Ribs @ 24″

f'_c = 4,000 psi
Grade 60 Bars

SQUARE EDGE PANELS

Total Depth = 11 in. Rib Depth = 8 in. Total Slab Depth = 3 in.

Span c.-c. Columns $\ell_1 = \ell_2$ (ft)	Factored Super-imposed Load (psf)	(1) Steel (psf)	Square Edge Column $c_1 = c_2$ (in.)	Square Edge Column γ_f	Square Edge Column (2) Stirrups	Column Strip Top Edge No.-size +	Column Strip Bottom No. Ribs	Column Strip Bottom Bars per Rib	Column Strip Top Interior No.-size	Middle Strip Bottom No. Ribs	Middle Strip Bottom Long Bars	Middle Strip Bottom Short Bars	Middle Strip Top Interior No.-size	Moments −M Edge (ft-k)	Moments +M Bot. (ft-k)	Moments −M Int. (ft-k)
24'- 0" D= 8.417 RIB ON COLUMN LINE 0.571 CF/SF	50	2.26	12	0.855		18-#5+ 0	5	2-#4	18-#5	7	#4	#4	7-#5	67	135	181
	100	2.39	12 *	0.889		18-#5+ 0	5	2-#5	18-#5	7	#4	#4	7-#5	88	182	237
	150	2.54	12 *	0.926		18-#5+ 1	5	1-#5 and 1-#6	18-#5	7	#4	#4	7-#5	109	251	292
	200	2.96	13 *	0.629		18-#5+ 0	5	1-#6 and 1-#7	21-#5	7	#4	#5	7-#5	129	315	347
	300	3.73	18 *	0.623	3 S 4 1	18-#5+ 0	5	1-#7 and 1-#8	19-#6	7	#5	#5	9-#5	166	417	446
26'- 0" D=10.417 RIB NOT ON COLUMN LINE 0.585 CF/SF	50	2.32	13	0.863		19-#5+ 0	6	1-#4 and 1-#5	19-#5	7	#4	#4	8-#5	86	173	233
	100	2.40	13 *	0.911		19-#5+ 2	6	2-#5	19-#5	7	#4	#4	8-#5	113	227	303
	150	2.85	14 *	0.936		19-#5+ 4	6	2-#6	22-#5	7	#4	#5	8-#5	138	315	373
	200	3.31	17 *	0.624		19-#5+ 2	6	1-#6 and 1-#7	26-#5	7	#5	#5	8-#5	163	393	439
28'- 0" D=10.417 RIB NOT ON COLUMN LINE 0.577 CF/SF	50	2.39	14 *	0.881		21-#5+ 1	6	2-#5	21-#5	8	#4	#4	8-#5	107	215	289
	100	2.55	14 *	0.931		21-#5+ 4	6	1-#5 and 1-#6	22-#5	8	#4	#4	8-#5	140	280	377
	150	3.14	17 *	0.624		21-#5+ 0	6	1-#6 and 1-#7	20-#6	8	#4	#5	9-#5	171	378	461
	200	3.45	23 *	0.619		21-#5+ 2	6	2-#7	23-#6	8	#4	#5	10-#5	198	414	533
30'- 0" D=10.417 RIB NOT ON COLUMN LINE 0.571 CF/SF	50	2.50	15 *	0.893		22-#5+ 2	6	1-#5 and 1-#6	22-#5	9	#4	#4	9-#5	132	263	354
	100	2.77	16 *	0.934		22-#5+ 6	6	2-#6	28-#5	9	#4	#4	9-#5	171	343	461
	150	3.37	23 *	0.619		22-#5+ 1	6	2-#7	24-#6	9	#4	#5	11-#5	207	417	556

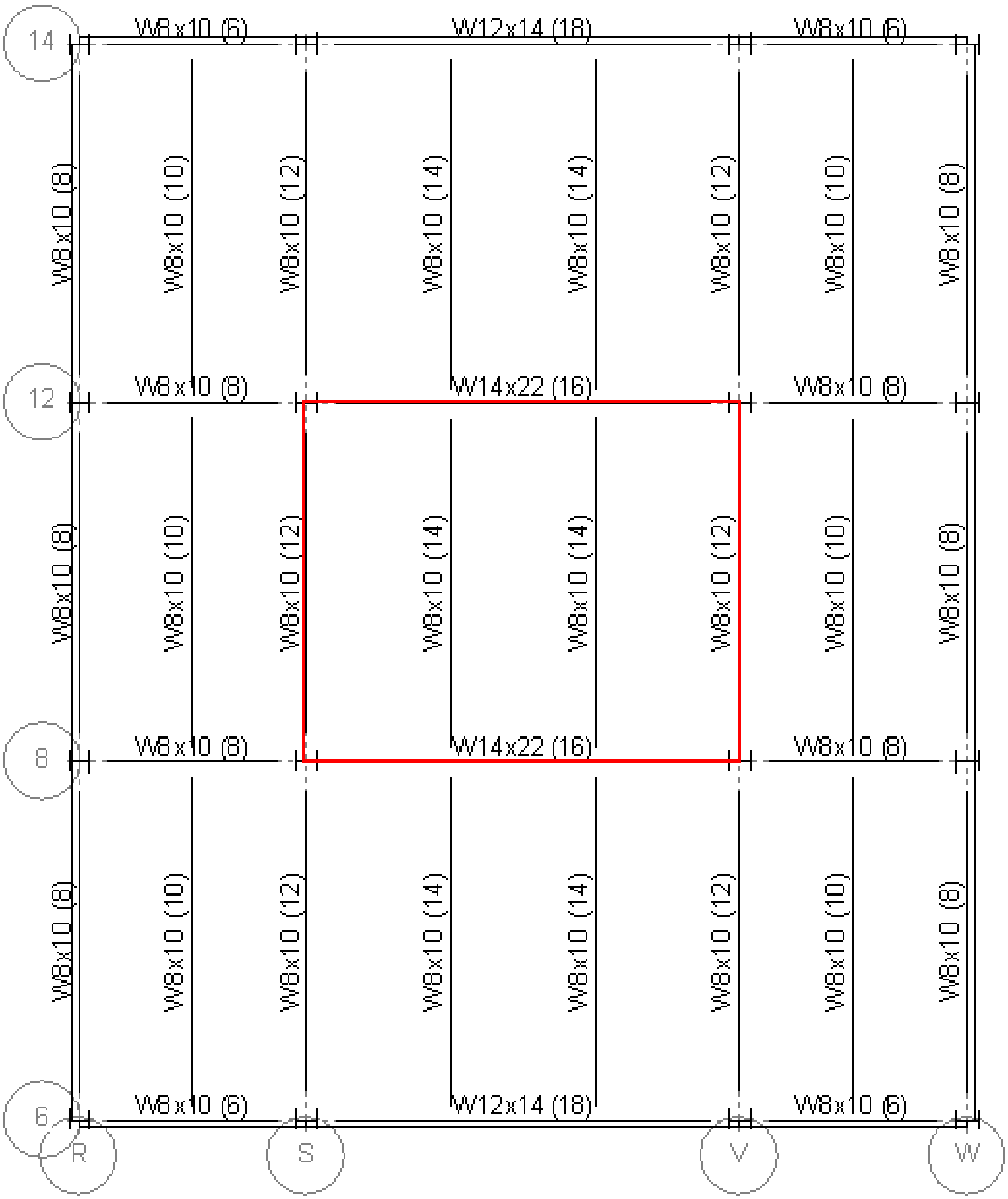
SQUARE INTERIOR PANELS

Total Depth = 11 in. Rib Depth = 8 in. Total Slab Depth = 3 in.

Span c.-c. Columns $\ell_1 = \ell_2$ (ft)	Factored Super-imposed Load (psf)	(1) Steel (psf)	Square Interior Column $c_1 = c_2$ (in.)	Square Interior Column (2) Stirrups	Column Strip Bottom No. Ribs	Column Strip Bottom Bars per Rib	Column Strip Top Interior No.-size	Middle Strip Bottom No. Ribs	Middle Strip Bottom Long Bars	Middle Strip Bottom Short Bars	Middle Strip Top Interior No.-size
24'- 0"	50	2.23	12		5	2-#4	18-#5	7	#4	#4	7-#5
	100	2.23	12		5	2-#4	18-#5	7	#4	#4	7-#5
	150	2.35	12 *	3 S 4 1	5	1-#4 and 1-#5	18-#5	7	#4	#4	7-#5
	200	2.53	12 *	3 S 4 1	5	2-#5	19-#5	7	#4	#4	7-#5
	300	3.21	14 *	3 S 4 2	5	2-#6	18-#6	7	#4	#4	8-#5
26'- 0"	50	2.23	13		6	2-#4	19-#5	7	#4	#4	8-#5
	100	2.23	13		6	2-#4	19-#5	7	#4	#4	8-#5
	150	2.52	13 *	3 S 4 1	6	2-#5	20-#5	7	#4	#4	8-#5
	200	2.94	13 *	3 S 4 1	6	1-#5 and 1-#6	18-#6	7	#4	#4	8-#5
28'- 0"	50	2.23	14		6	2-#4	21-#5	8	#4	#4	8-#5
	100	2.35	14 *	3 S 4 1	6	1-#4 and 1-#5	21-#5	8	#4	#4	8-#5
	150	2.81	14 *	3 S 4 1	6	1-#5 and 1-#6	26-#5	8	#4	#4	8-#5
	200	3.13	17 *	3 S 4 2	6	1-#5 and 1-#6	22-#6	8	#4	#4	10-#5
30'- 0"	50	2.33	15		6	1-#4 and 1-#5	22-#5	9	#4	#4	9-#5
	100	2.60	15 *	3 S 4 1	6	2-#5	26-#5	9	#4	#4	9-#5
	150	3.06	16 *	3 S 4 2	6	1-#5 and 1-#6	23-#6	9	#4	#4	10-#5

See the notes on Page 11-19.

APPENDIX: STEEL COMPOSITE FRAMING LAYOUT, EXISTING COLUMN GRID, DESIGNED WITH RAM STRUCTURAL SYSTEMS



APPENDIX: STEEL COMPOSITE, EXISTING COLUMN GRID

Beam Deflection Summary

RAM Steel v11.0
DataBase: floor plan existing
Building Code: IBC

10/22/07 14:05:39
Steel Code: AISC LRFD

STEEL BEAM DEFLECTION SUMMARY:

Floor Type: 7 floor typical bay

Composite / Unshored

Bm #	Beam Size	Initial in	PostLive in	PostTotal in	NetTotal in	Camber in
25	W8X10	0.096	0.406	0.710	0.806	
37	W8X10	0.035	0.166	0.291	0.326	
26	W8X10	0.096	0.406	0.710	0.806	
40	W8X10	0.055	0.259	0.453	0.508	
27	W8X10	0.096	0.406	0.710	0.806	
43	W8X10	0.055	0.259	0.453	0.508	
46	W8X10	0.035	0.166	0.291	0.326	
58	W8X10	0.096	0.606	1.060	1.157	
59	W8X10	0.096	0.606	1.060	1.157	
60	W8X10	0.096	0.606	1.060	1.157	
28	W8X10	0.096	0.614	1.094	1.190	
38	W12X14	0.181	0.695	1.272	1.453	
29	W8X10	0.096	0.614	1.094	1.190	
41	W14X22	0.142	0.661	1.344	1.485	
30	W8X10	0.096	0.614	1.094	1.190	
44	W14X22	0.142	0.661	1.344	1.485	
47	W12X14	0.181	0.695	1.272	1.453	
53	W8X10	0.096	0.619	1.122	1.219	
51	W8X10	0.096	0.619	1.122	1.219	
49	W8X10	0.096	0.619	1.122	1.219	
54	W8X10	0.096	0.619	1.122	1.219	
52	W8X10	0.096	0.619	1.122	1.219	
50	W8X10	0.096	0.619	1.122	1.219	
31	W8X10	0.096	0.614	1.094	1.190	
39	W8X10	0.035	0.166	0.291	0.326	
32	W8X10	0.096	0.614	1.094	1.190	
42	W8X10	0.055	0.259	0.453	0.508	
33	W8X10	0.096	0.614	1.094	1.190	
45	W8X10	0.055	0.259	0.453	0.508	
48	W8X10	0.035	0.166	0.291	0.326	
57	W8X10	0.096	0.606	1.060	1.157	
55	W8X10	0.096	0.606	1.060	1.157	
56	W8X10	0.096	0.606	1.060	1.157	
34	W8X10	0.096	0.406	0.710	0.806	
35	W8X10	0.096	0.406	0.710	0.806	
36	W8X10	0.096	0.406	0.710	0.806	

APPENDIX: STEEL COMPOSITE FRAMING LAYOUT, NEW COLUMN GRID, DESIGNED WITH RAM STRUCTURAL SYSTEMS

APPENDIX: STEEL COMPOSITE, NEW COLUMN GRID

Beam Deflection Summary

RAM Steel v11.0
DataBase: floor plan new
Building Code: IBC

10/22/07 16:49:53
Steel Code: AISC LRFD

STEEL BEAM DEFLECTION SUMMARY:

Floor Type: 7 floor typical bay

Composite / Unshored

Bm #	Beam Size	Initial in	PostLive in	PostTotal in	NetTotal in	Camber in
25	W8X10	0.096	0.358	0.627	0.723	
37	W8X10	0.119	0.516	0.902	1.021	
26	W8X10	0.096	0.358	0.627	0.723	
40	W10X12	0.115	0.438	0.835	0.951	
27	W8X10	0.096	0.358	0.627	0.723	
43	W10X12	0.115	0.438	0.835	0.951	
46	W8X10	0.119	0.516	0.902	1.021	
73	W8X10	0.096	0.534	0.935	1.032	
71	W8X10	0.096	0.534	0.935	1.032	
69	W8X10	0.096	0.534	0.935	1.032	
74	W8X10	0.096	0.534	0.935	1.032	
72	W8X10	0.096	0.534	0.935	1.032	
70	W8X10	0.096	0.534	0.935	1.032	
28	W8X10	0.096	0.574	1.005	1.101	
38	W8X10	0.119	0.516	0.902	1.021	
29	W8X10	0.096	0.574	1.005	1.101	
41	W10X12	0.115	0.438	0.835	0.951	
30	W8X10	0.096	0.574	1.005	1.101	
44	W10X12	0.115	0.438	0.835	0.951	
47	W8X10	0.119	0.516	0.902	1.021	
53	W8X10	0.096	0.574	1.005	1.101	
51	W8X10	0.096	0.574	1.005	1.101	
49	W8X10	0.096	0.574	1.005	1.101	
54	W8X10	0.096	0.574	1.005	1.101	
52	W8X10	0.096	0.574	1.005	1.101	
50	W8X10	0.096	0.574	1.005	1.101	
31	W8X10	0.096	0.574	1.005	1.101	
39	W8X10	0.119	0.516	0.902	1.021	
32	W8X10	0.096	0.574	1.005	1.101	
42	W10X12	0.115	0.438	0.835	0.951	
33	W8X10	0.096	0.574	1.005	1.101	
45	W10X12	0.115	0.438	0.835	0.951	
48	W8X10	0.119	0.516	0.902	1.021	
75	W8X10	0.096	0.534	0.935	1.032	
77	W8X10	0.096	0.534	0.935	1.032	
67	W8X10	0.096	0.534	0.935	1.032	
76	W8X10	0.096	0.534	0.935	1.032	
78	W8X10	0.096	0.534	0.935	1.032	
68	W8X10	0.096	0.534	0.935	1.032	