## Gravity System (Depth Topic I)

### Post Tensioned Slab

A new floor system was designed in an attempt to create a more consistent flooring system throughout the entire building. This new design consists of a two way post-tensioned flat plat slab with no drop panels. The post-tension would help reduce the number of columns by allowing the slab to span larger distances. It would also decrease the thickness of the slab which would in turn increase the floor to ceiling height. The flat plat system is ideal for the residential building since it would eliminate the beams and provide a finished ceiling.

The slabs were designed using ADAPT-PT which uses the equivalent moment frame method. Hand calculations were also used to check the results obtained from ADAPT. The floor plan was divided into strips running in both the E-W and N-S direction. The following plans shows the strips generated with the typical strip designed highlighted:

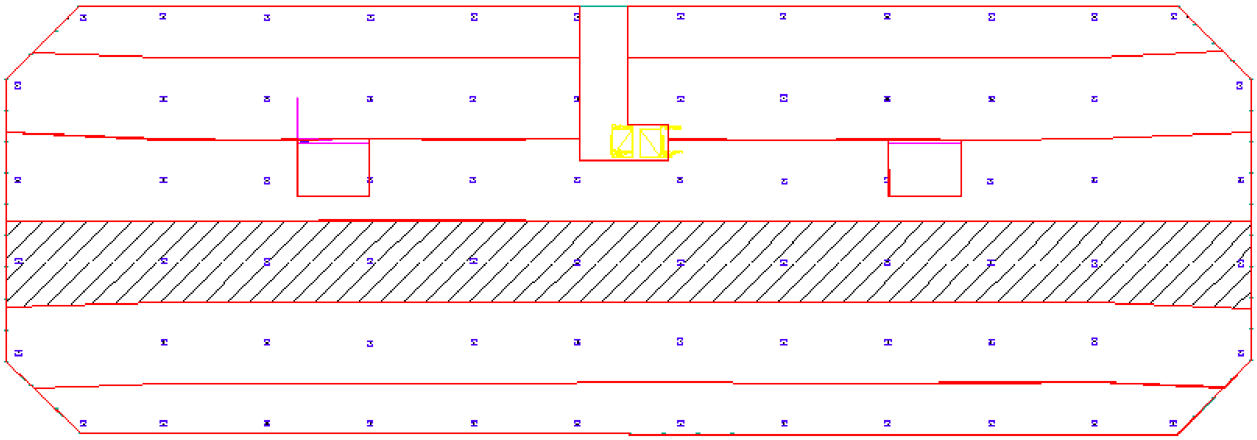


Figure 9: Post-Tension design Strips in E-W Direction
(Residential floor)

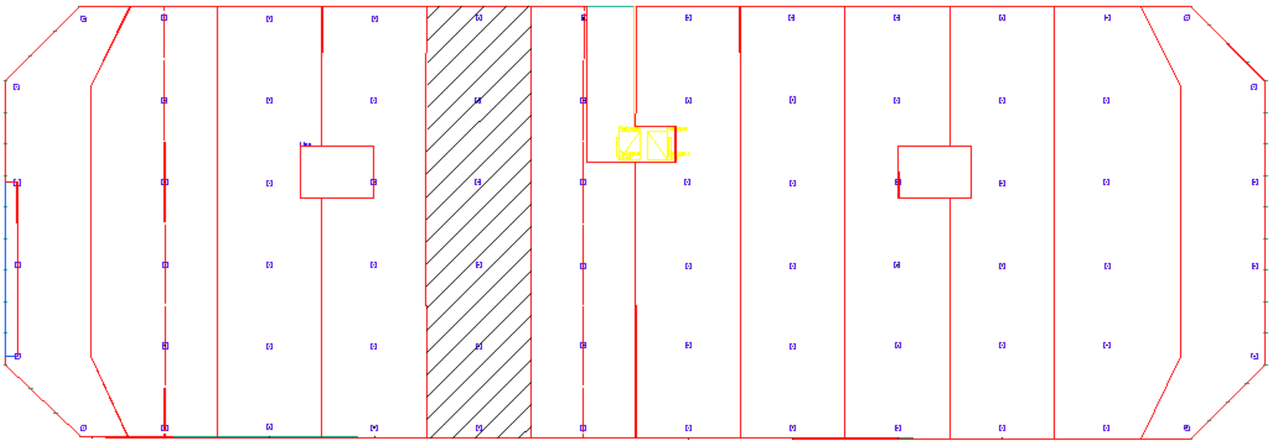


Figure 10: Post-Tension design Strips in N-S Direction
(Residential floor)

The following table summarizes the design parameters. Notice that the concrete strength used is kept at 5000 psi, in order to compare it with the original design (also designed at 5000 psi). The balanced dead load percentage was kept at fewer than 100% while the average precompression was bounded by a maximum value of 350 psi. The strand used is a 270ksi, 7-wire prestressing steel strand. Pattern loading was not considered since the LL/DL < ¾.

| Parameter | Value | Parameter | Value |
|---|---|---|---|
| Concrete | | Minimum Cover at BOTTOM | 0.75 in |
| F'c for BEAMS/SLABS | 5000.00 psi | Post-tensioning | |
| For COLUMNS/WALLS | 5000.00 psi | SYSTEM | UNBONDED |
| Ec for BEAMS/SLABS | 4031.00 ksi | Fpu | 270.00 ksi |
| For COLUMNS/WALLS | 4031.00 ksi | Fse | 175.00 ksi |
| CREEP factor | 2.00 | Strand area | 0.153 in 2 |
| CONCRETE WEIGHT | NORMAL | Min CGS from TOP | 1.00 in |
| UNIT WEIGHT | 150.00 pcf | Min CGS from BOT for interior spans | 1.00 in |
| Tension stress limits / (f'c)1/2 | | Min CGS from BOT for exterior spans | 1.75 in |
| At Top | 6.000 | Min average precompression | 125.00 psi |
| At Bottom | 6.000 | Max spacing / slab depth | 8.00 |
| Compression stress limits / f'c | | Analysis and design options | |
| At all locations | 0.450 | Structural system - Equiv Frame | TWO-WAY |
| Reinforcement | | Moments reduced to face of support | YES |
| Fy (Main bars) | 60.00 ksi | Moment Redistribution | NO |
| Fy (Shear reinforcement) | 60.00 ksi | DESIGN CODE SELECTED | ACI-318 (2005) |
| Minimum Cover at TOP | 0.75 in | | |

Table 1:Post-Tension Design Parameters

## E-W Direction strip:

Due to the shape of the building, there are two 30 ft exterior spans at each end of the strip while the rest of the spans are about 20ft. More detail regarding the column layout will be covered in the column design section of the report. The two long exterior spans resulted in an increase in stress compared to the interior spans. After several trials it was discovered that an 8in slab with 22 strands works for the flexure stresses and deflection. Punching shear was also checked when designing the columns. The following graphs illustrate the tendon profile and deflections produced:

Figure 11

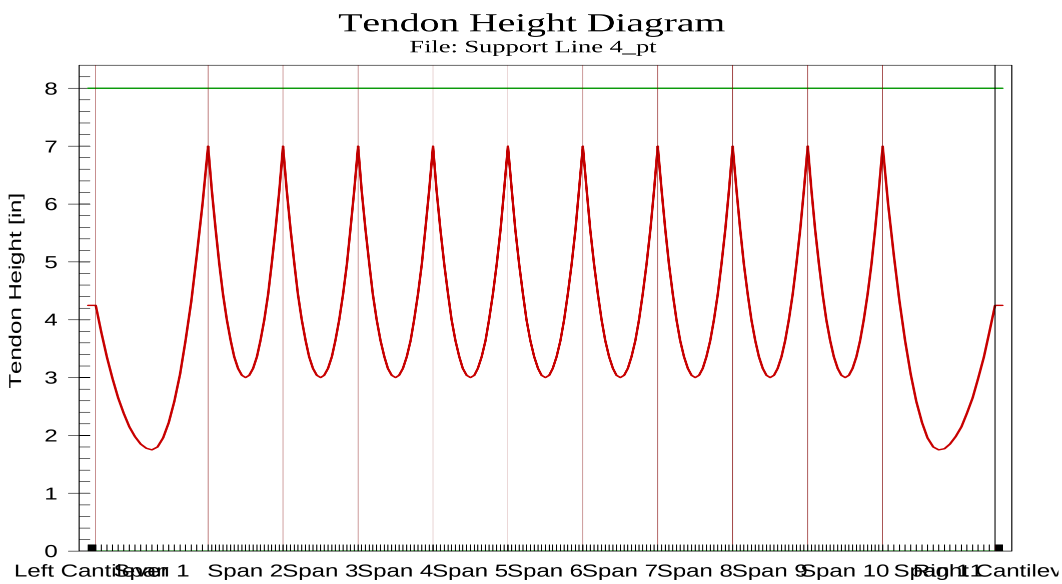
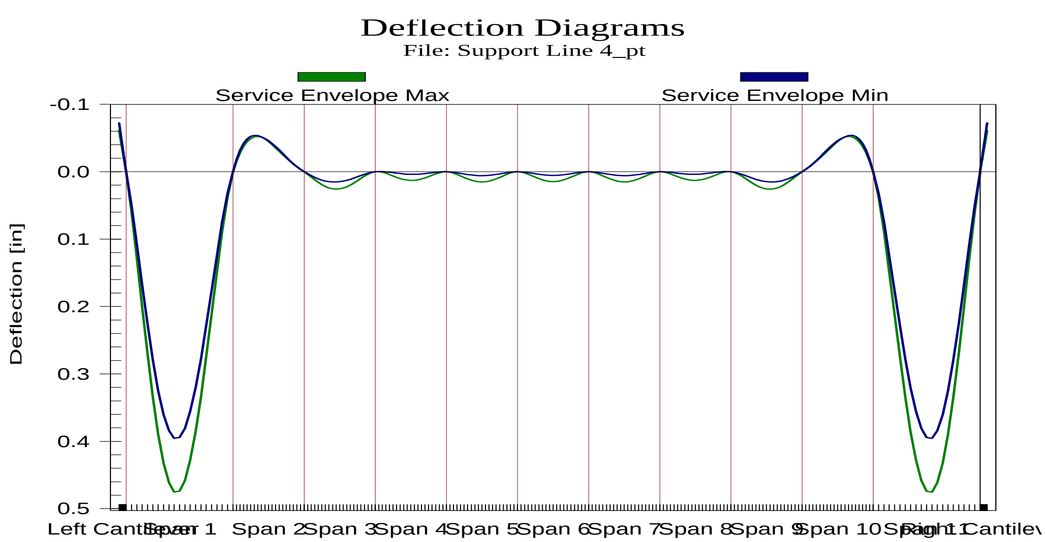



Figure 12



The deflection limit used in the design is L/360. This is due to the assumption that no deflection is induced by the dead load since it is mostly balanced by the tendons. Hence, the deflection would generally be caused by the live load which is limited at L/360. Notice that the largest deflection is under 0.5in which is acceptable based on our assumptions.

Hand calculations were used to check the results obtained from ADAPT-PT. The results from ADAPT-PT yielded larger forces and thus the design from ADAPT-PT is more conservative. The difference in results is due to the fact that the hand calculation is simplified and based on many assumptions. Below is the summary of the hand calculation. More details and calculations are provided in Appendix A:

| | | | |
|---|---|---|---|
| $a_{INT}$ | 4 | | |
| $a_{end}$ | 3.875 | | |
| $W_b$(k/ft) | 1.44 | | |
| P(k) | 501.6774194 | < 574.5 From Adapt | hence Adapt conservative |
| No. of tendons | 18.8445 | | |
| $P_{actual}$(k) | 505.818 | | |
| $W_b$(k/ft) | 1.42821229 | | |
| P/A(psi) | 329.3085938 | | |

Table 2:Post-Tension parameters

Stage 1: Stresses after jacking

| | Interior span | End span | Support stresses | |
|---|---|---|---|---|
| $f_{top}$(psi) | -223.8398438 | 121.8632813 | -798.0585938 | ok |
| $f_{bot}$(psi) | -434.7773438 | -780.4804688 | 139.4414063 | ok |

Table 3:Post-Tension stresses after jacking

Stage 2: Stresses at service load

| | Interior span | End span | Support stresses | |
|---|---|---|---|---|
| $f_{top}$(psi) | -305.8710938 | -171.1054688 | -475.7929688 | ok |
| $f_{bot}$(psi) | -352.7460938 | -487.5117188 | -182.8242188 | ok |

Table 3:Post-Tension stresses at service load

**N-S Direction Strip:**

Since the N-S direction is the short span direction, the resulting stresses were much smaller as expected. The spans are uniform causing the tendon profile to be uniform as well. Furthermore, a resulting deflection of less than 0.03in was compared to the L/360 and checked out as acceptable. Only 6 strands were needed for the short spans. The following graph illustrates the tendon profile and deflection in the N-S direction:

Figure 13

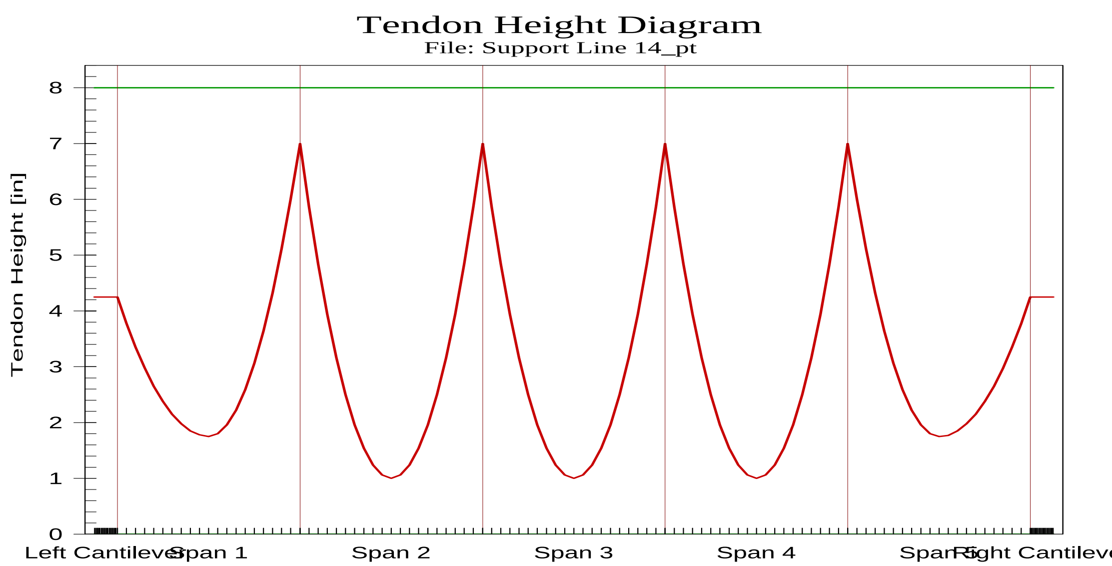
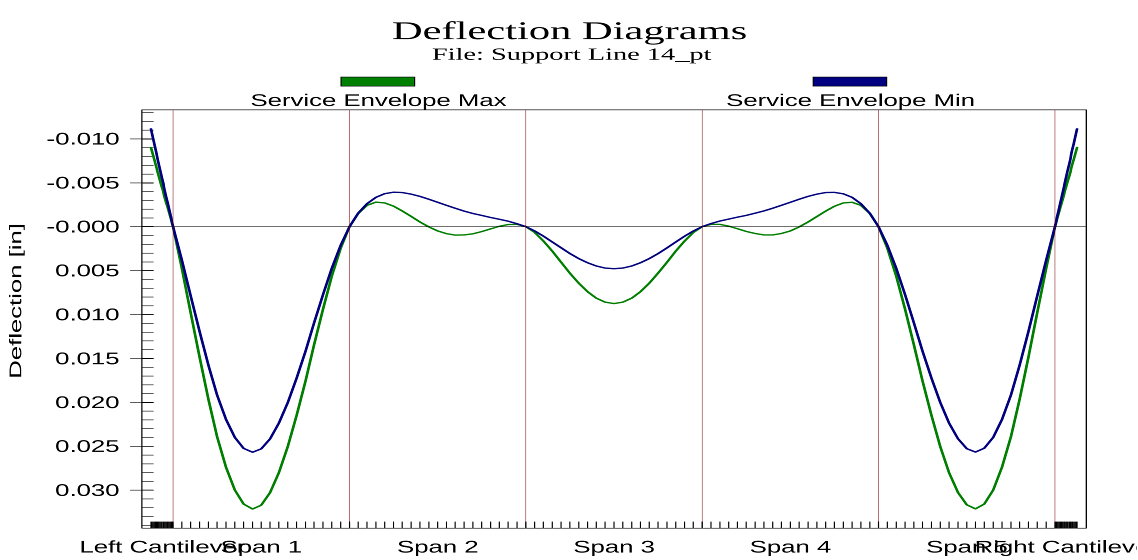



Figure 14



## Slab Design Summary

The new post-tensioned flat plat design did not decrease the thickness of the slab as expected. On the other hand, it still served as a better flooring system since the beams were eliminated and fewer columns were used. The slab thickness, however, did decrease on the office floor from 14in flat plate with drop panels to 9in flat plate with no drop panels with the post tensioning. The elimination of the beams will decrease the weight of the building while significantly impacting the cost and schedule of the building which will be discussed later on in the report.

Keep in mind that the design discussed is a typical strip in the floor plan. Further study would be needed to determine the exact design of the other strips, especially the ones with an opening which is not included in the scope of this report. The following diagram summarizes the design of a typical interior bay in a residential floor:

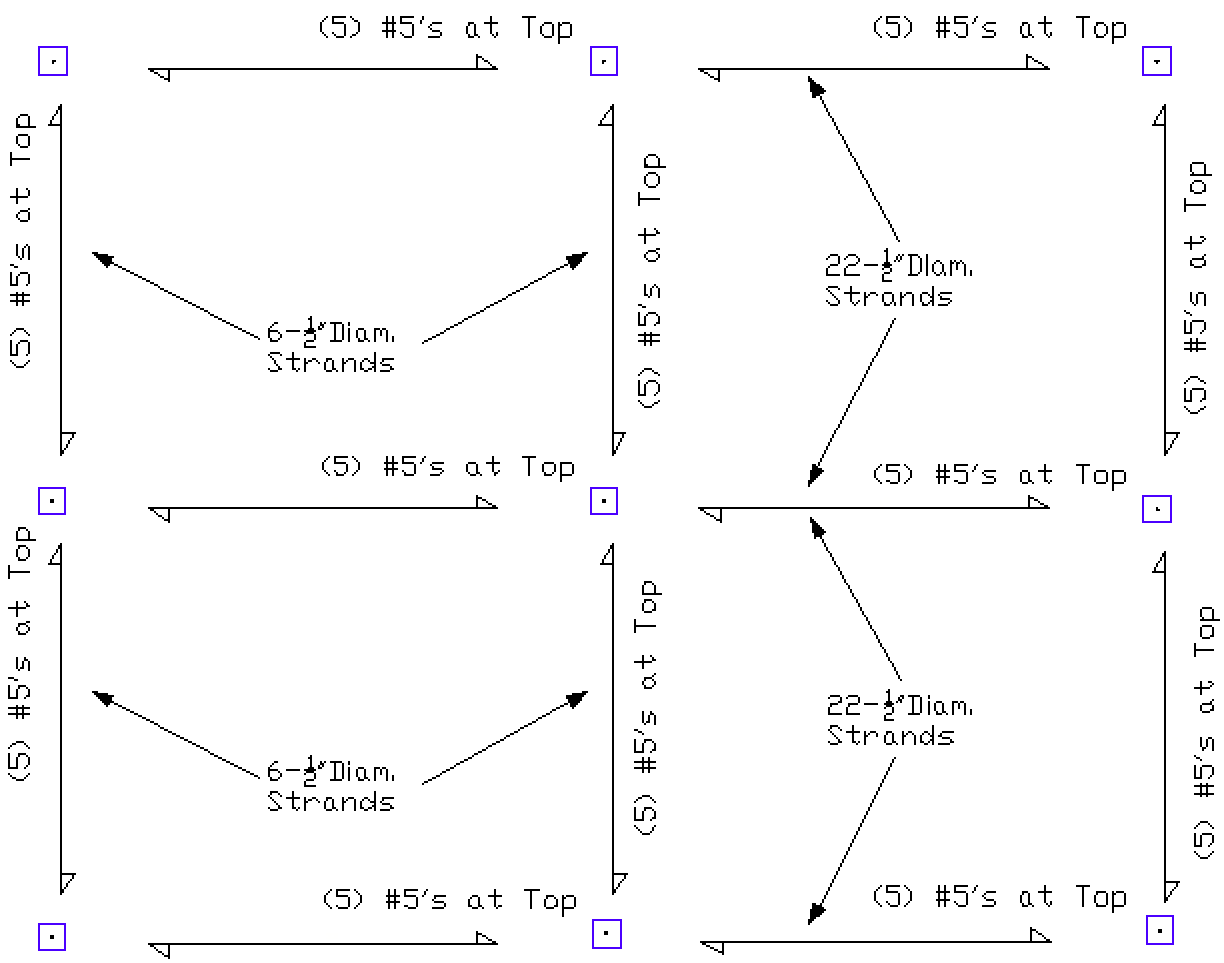



Figure 15: Design of a Typical Interior bay on the residential floor

## Column Design:

The column grid for the existing building was designed to complement the two-way beams running in the residential area. The spans ranged between 10ft and 30ft where the columns at the exterior span were considerably too close to each other for a post-tensioned design. Many different column sizes were also used within the same floor which would increases the time of erecting the form work during the construction phase. The building had 14 different column sizes within a single floor. Figure 16 shows the existing column layout:

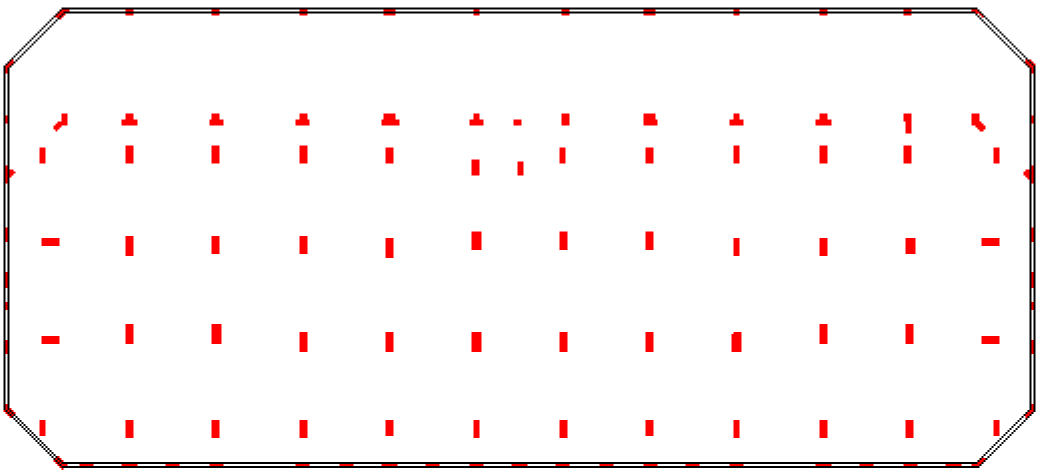


Figure 16: Column Layout of Original Design

A new Column Layout had to be designed in order to increase the spans to justify using a post-tensioned slab. A slab thickness of less than 8in could have been achieved if the same layout was used. However, an engineering decision was made to increase the spans and decrease the number of columns at the expense of the thinner slab. The reasoning behind this decision is that the form work for the slab is very basic and would not take more time to construct if you increase the slab thickness. The column form work on the other hand would cost more money and time to construct if more columns were designed. A more flexible space layout is also achieved when using fewer columns especially in the office spaces. Figure 17 shows the column layout for the new design:

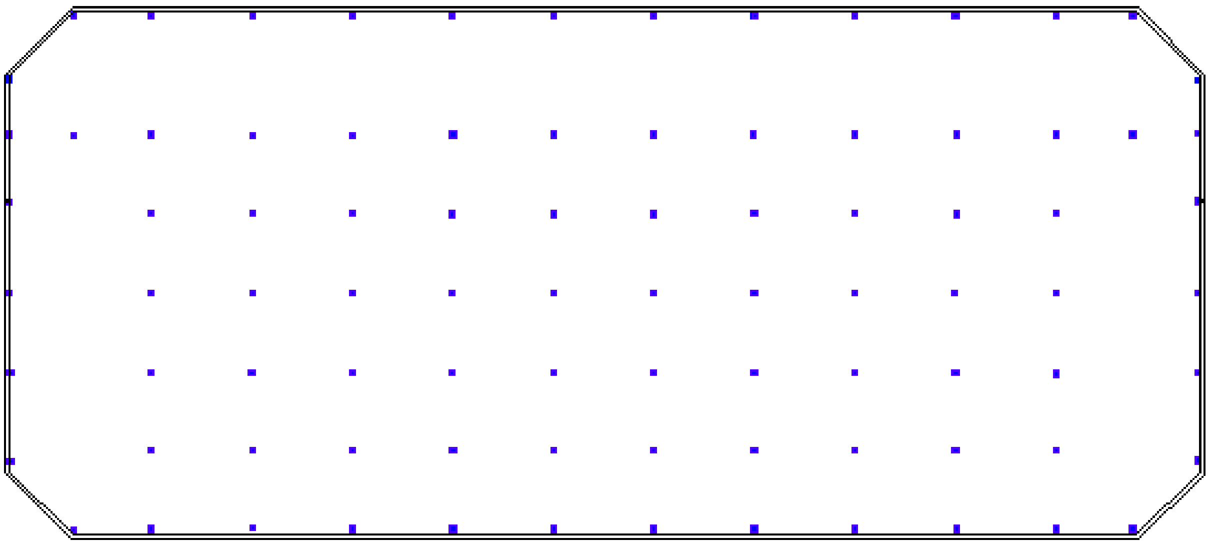


Figure 17: Column Layout of New Design

The new column layout was designed with larger more uniform spans in order to efficiently incorporate the post tensioning slab into the building. There are still larger spans at the exterior bays and that's due to the shape of the building which forces to use on of the following; two smaller spans, one large span, or the same span with a larger cantilever. The number of different column sizes was also reduced to two sizes only; exterior columns and interior columns sizes. Total number of columns was reduced from 112 to 88 columns.

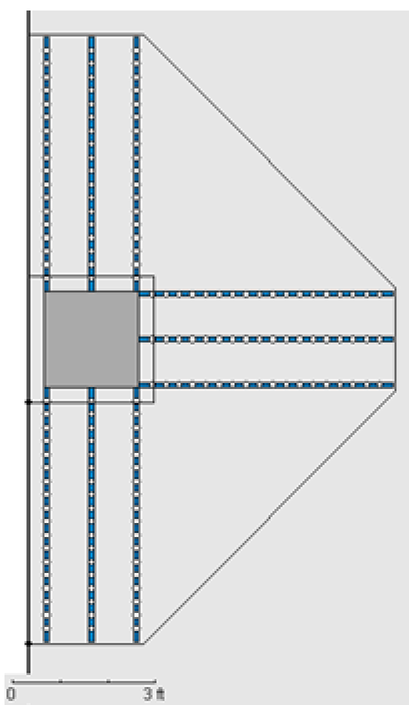
A column takedown of the loads was generated by hand in order to design the columns. PCA column was then used to design the individual columns using the interaction diagrams. The size of the building column going up the different levels was also kept constant in order to better facilitate the construction process. Hence all columns were designed by the loads applied at the bottom level. The moments generated on the columns were minimal since the frame of the building was assumed as a non-sway frame and only the shear walls are used to resist the lateral loads. However, these minimal moments were still checked with the design. The following table summarizes the loads and sizes of the columns. Refer to Appendix A for more detailed calculations:

| Column Type | Axial Load (k) | Size (in) |
|---|---|---|
| Interior Column | 1052 | 20 x 20 |
| Exterior Column (All floors) | 1193 | 24 x 24 |
| Exterior Column (North of 1st and 2nd Floor) | 491 | 14 x 14 |

Table 4: Column Design Summary

## Punching Shear:

The design of flat plate slabs was checked for punching shear to make sure the columns and slab were adequate to carry the loads. As expected, the exterior spans did not satisfy the shear check and hence a solution had to be determined. Possible solutions to the punching shear problem are; using drop panels, increasing column size, and using stud rails. Assuming the architect would not be very happy with the idea of drop panels in the residential floors, the column sizes where increased. In addition, to minimize the increase in column size shear studs were used. The software used to design the shear studs is called STDesign 3.1 provided by Decon. The following figure illustrates the design of the shear studs:



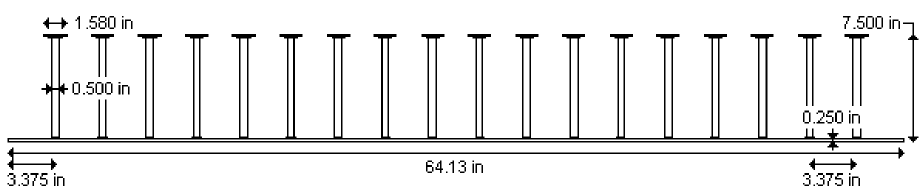




Figure 18: Elevation of Studrails

There are 9 studrails per column with 18 studs on each studrail. The studs are spaced at 3.38 in with an overall height of 7.5in.

Figure 19: Plan of Studrails

## External Beams:

When designing the post tensioned slabs, the façade of the building was not taking into consideration. Hence a quick hand calculated estimate was used to design the external beams to carry the façade. These beams are not designed to carry any loads from the slab. Table 5 summarizes the beam design:

| | | |
|---|---|---|
| Façade Load (psf) | 30 | |
| $W_u$ (PLF) | 300 | |
| $M_u$ (K-FT) | 20.19798 | |
| Try b= 4/5d | | |
| $d^3$ | 7.963109 | |
| use h=11, d=8.5 | | |
| b | 7 | |
| d | 8.5 | |
| h | 11 | |
| $bd^2$ | 505.75 | |
| $W_{sw}$ (PLF) | 80.20833 | |
| $W_u$ (PLF) | 456.25 | |
| $M_u$ (K-FT) | 23.27103 | |
| $20M_u$ | 465.4206 | < 506 o.k |
| $A_s$ | 0.684442 | |
| $A_s$ | 0.21 | |
| | 0.2 | |
| a | 1.61 | |
| c | 2.02 | |
| $M_n$ (K-FT) | 30.78 | |
| $\phi M_n$ (K-FT) | 27.7 | o.k |

Table 5: External beam frame

Hence a 7in x11in beam with (4) # 4's is sufficient to carry the load of the façade.