Structural System Design

STRUCTURAL SYSTEM DESIGN

DESIGN CRITERIA

The primary goals and criteria governing the alternate concrete superstructure design are as follows:

-maintain an open office layout, free of column obstructions
-maintain an overall floor system depth equal to or less than the existing steel floor system depth of 23-1/4”
-limit live load deflection of floor to $\ell/360$
-limit the total building drift to $\ell/400$ or 6.42”

DESIGN PROCESS

The alternate concrete structural system for Eight Tower Bridge was designed keeping in mind several of the same performance and design criteria that the original steel system was designed under. A desirable attribute of any office tower design, especially a multi-tenant office tower, is to maintain an open floor plan with minimal interruption from columns. An open floor plan allows for the space to be configured to suite the tenant’s needs as the space is rented. A multi-tenant office tower also means there is little to no set floor plan prior to or during construction, again requiring flexibility of the space to be modified once rented.

In order to preserve the open floor span of the existing steel structure, the long spans of 44’4” from the building exterior columns to the building core columns must be preserved without adding additional column lines. The bay size of 44’4”x28’ starts to approach the upper limits for allowable two-way concrete action (a length to width ratio less than two is required). Even though this bay size is below the *l/w* ratio of two, it still may not be a very economical design.

A one-way beam and slab system will carry the floor loads in a similar fashion to the existing steel system, which will keep the long spans of the bay in tact. A T-beam design would be possible, with the added flange width allowing for the decrease of the depth of the concrete stress block, a. The moment capacity of a T-beam section is given by the equation below:

$$M_n = (.85f'c(b-b_w)h_f)(d-h_f/2) + (.85f'cb_wa)(d-a/2)$$

When the depth of the stress block a is reduced, the overall moment capacity of the section will increase. While this seems like the optimum solution to the concrete design problem, T-beam design is only possible at midspan, as negative moments at the end span won't be resisted by forces in the flange. A solution to this problem would be to increase the effective depth, d, thus increasing the overall system depth. While this may be a viable design solution for applications where structure depth may not be a limiting criterion such as a bridge span, minimizing the structure depth in building design is an important design factor. Additionally, with an increase in beam size, controlling deflection will also become difficult due to an increase in beam self weight. A design solution that will decrease the beam depth and provide adequate flexural strength is to introduce post tensioning to the section.

POST TENSIONING DISCUSSION

Post tensioning of concrete sections involves balancing a certain percentage of the floors permanent load dead load with an external tension force at the end of the beam transferred to the concrete through a stressing tendon. These tendons are laid inside the concrete formwork, usually protected by plastic duct work or sheathing to prevent the concrete from bonding to the tendons initially. The tendons are anchored at one end of section, and tensioned using a hydraulic jack at the other, or in long span cases (longer than 120') are tensioned at both ends once the concrete has cured to a specified strength, usually taken to be 0.6f'c. The tendon ducts are sometimes injected with a grout in order for the tendons to more effectively transfer the prestressing force to the concrete section. These tendons are known as bonded tendons. Post-tensioned systems that don't use this method are said to have unbonded tendons.

Figure 9: Post tensioned slab
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Pulling on the end of the anchored tendon creates a compression force at either end of the beam, inducing a compression force in area of section where tension would occur in the concrete element under dead loading. If designed properly, this compression force will create a camber of the concrete element, which is then balanced by the addition of finishing loads,

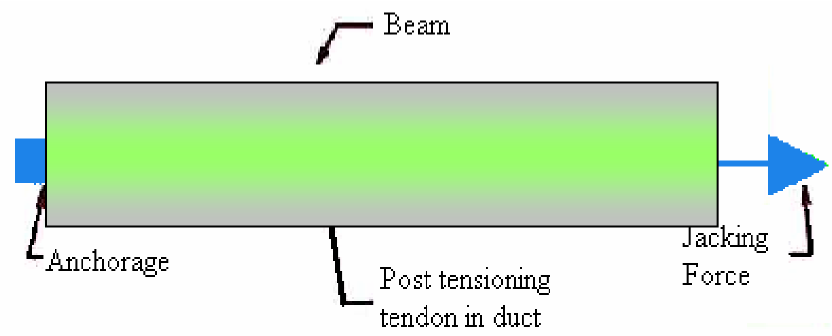
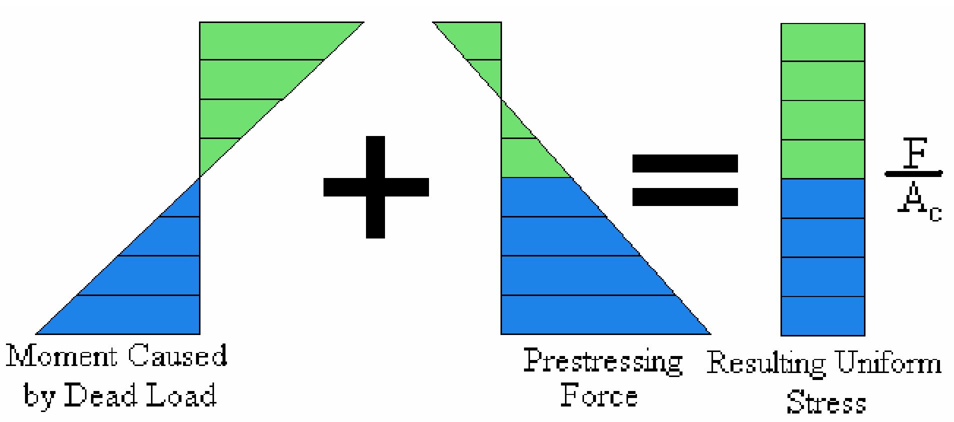


Figure10: Post-tensioned beam



miscellaneous MEP system loads and live loading after the concrete has cured. This additional loading on the beam or slab will act against the upward camber, yielding minimum deflections across the member length. Prestressing tendons are usually draped across the section in a parabolic profile in order to evenly balance the dead load along the length of the beam. The tendon profile and magnitude can be designed to create a member that is uniformly stressed under flexural forces.

The floor system design of any building has the most significant impact on the rest of the building's structure, and additionally the building's overall cost. The weight of each floor ultimately determines the size of the columns, walls and foundations. The depth of the structural system determines the overall building height, thereby affecting the total quantities of cladding components as well as mechanical, electrical and plumbing work. Additionally, in areas of high wind forces, a 6" reduction of flooring system depth over 20 stories would result in a 10' reduction in building height, thus reducing the total area the wind has to act on the building. This results in reduced story shears, base shears and overturning moment. In seismic controlled areas, a reduction of floor system weight could ultimately reduce the lateral force resisting system of the building.

There are both advantages and disadvantages of post-tensioning concrete. In general practice, post-tensioning concrete can result in thinner, more aesthetic sections without sacrificing strength. Post-tensioning concrete allows for greater span/depth ratios, thus decreasing the total material impacting both cost and weight. When concrete is post-tensioned in buildings, it is possible to strip formwork earlier than regularly reinforced concrete once the slab or beam has been post-tensioned, thus decreasing the lag time between floor construction cycles. In systems where a considerable amount of the load is reduced by post-tensioning, the amount of regular steel reinforcing is decreased, reducing raw steel tonnage and material handling costs.

Figure 11: Post-tensioned bridge span

Disadvantages of post-tensioned system are largely construction related. You must wait for the concrete to cure to a specified strength before the tendons can be stressed. This can prolong the floor construction duration, despite the ability to speedily remove formwork shortly after the tendons are stressed. When post-tensioning slabs, additional labor is required to actually tension each of the strands which can slow construction time if not done properly. This usually requires hiring a special post-tensioning subcontractor. Post-tensioning tendons can also wreak havoc on a site if they are not tensioned to the proper strength or placed incorrectly. Too little tensioning can drastically reduce the effectiveness of the tendon. This can create increased deflections seen under wet concrete loads or over the course of the building's life. If a tendon is over stressed, it can snap, ripping through an entire concrete slab or beam, ruining the section, in addition to threatening the safety of crews on site. Also, a contractor can only perform post-tensioning when the temperature is above 45˚F. If a post-tensioning structure falls behind schedule into the winter months, concrete heaters will be required, adding to equipment costs. It is essential the contractor hired to perform post-tensioning in any building be experience to ensure safety and quality.

FLOOR SYSTEM DESIGN #1

Multiple types of post-tensioned systems were reviewed to determine what concrete flooring system would be most suitable for Eight Tower Bridge. After weighing possible options, a one-way post-tensioned beam and slab design alternative was selected. A typical bay spans 28'x44'4" with beams spaced at 14" on center. A 6" thick concrete slab with reinforcement in the orthogonal direction was found to be adequate with regard to ACI 318-05, Table 9.5(a). The flexural reinforcement for the

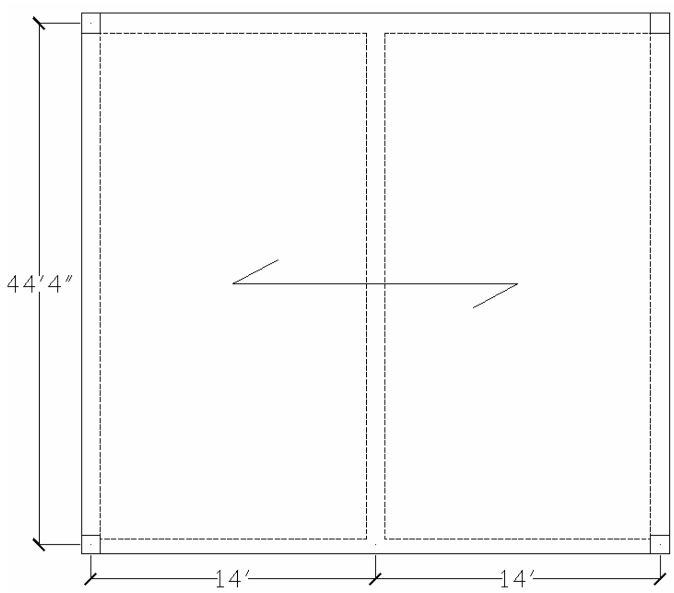


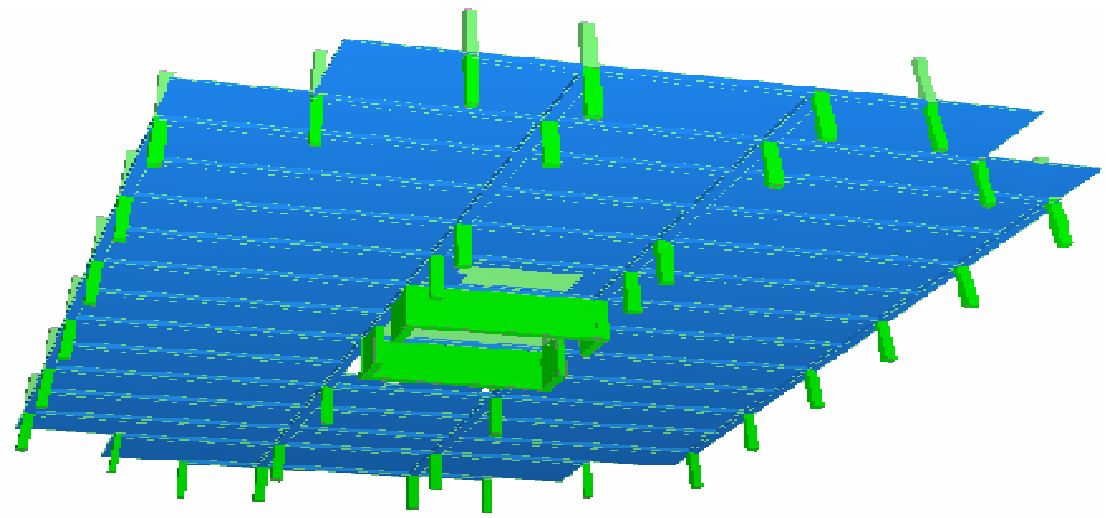
Figure 12: Typical concrete bay

slab was found to be #5@12" on center through the hand calculations found in Appendix B.

Due to the rather complex nature of calculating post-tensioning by hand, the structural software package RAM Conc*ept* was used to model the beam and slab floor system. RAM Conc*ept* allows the user to model a single floor of a building and either design or analyze post-tensioning system for the flooring system. A basic model for the program would include a floor slab, beams, drop panels if necessary and any columns or shear walls above and below the floor. However, at this point in the design, there was no trial section for either columns or beams to be entered into Conc*ept*.

In order to obtain a trial size for modeling the floor system in the program, a moment distribution based on relative stiffness with E and I held constant was performed on each bent of the proposed concrete frame to determine the approximate magnitude of the moments each frame would need to be designed for. Alternate and adjacent bay loadings were used to determine maximum and minimum design moments. The upper limit of any size beam selected, trial or final, was set at 24" deep, including the 6" slab in attempt to keep the total system design under the existing 23-1/4". A trial beam size of 20x20 was selected, and seeing as both E and I were kept constant for both columns and beams in the moment distribution, a 20x20 trial column size was also selected. The moment distribution tables can be found in Appendix B.

With trial sizes selected for the flooring system, an initial model could be



2-D rendering of post-tensioned beam and slab system

constructed in Conc*ept*. Shear walls were also placed in the model, but were not designed, as Concept does not consider lateral loads in design. A 3-D rendering of the floor system and supporting columns can be seen above. Please refer to Appendix B for more views the flooring system.

Post-tensioning tendons were added to beams spanning both directions, and the following design assumptions were made:

1. The concrete beams were designed as "T Class" sections, with an allowable extreme fiber stress of $7.5\sqrt{f'c} < f_t < 12\sqrt{f'c}$ in precompressed tensile zone at service loads (ACI 318-05, 18.3.3b.)

2. The design strips that Concept uses to design concrete elements were designed as T or L beams in the column strip, and as elevated slabs in the middle strip

3. All slabs and beams are 5000psi normal weight concrete

4. Tendons are unbonded, 270 ksi, ½"ø 7-wire stands with an effective force of 26.6kip/tendon after losses. Loss calculations can be found in Appendix B

The model was then run to test the initial section size, number of tendons and tendon profiles in each element. Through multiple trial and error design iterations, final beam member sizes were assigned and appropriate number of tendons and tendon profiles were placed in each section. The deepest section was found to be 20" including the 6" slab. This system had an overall depth 3-1/4" less than the existing steel system. The final framing plan can be found below with member sizes noted.

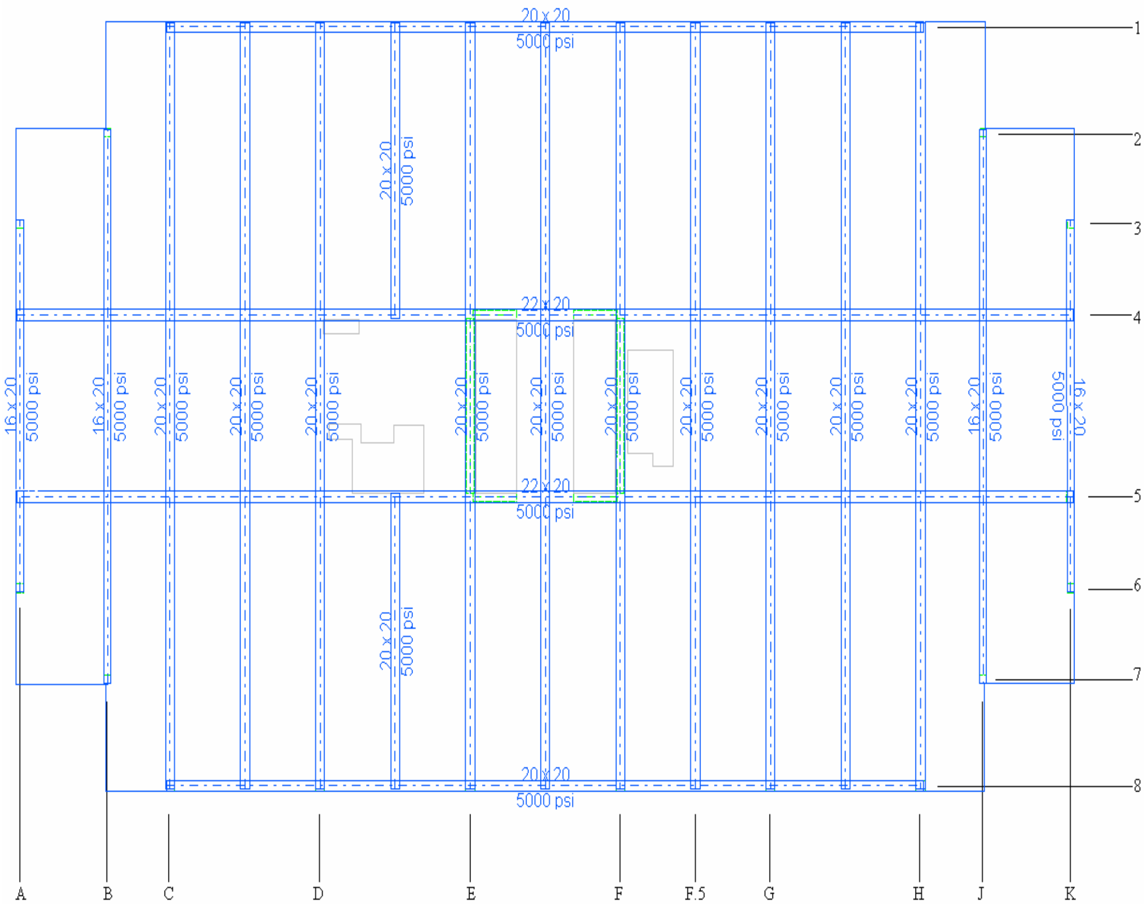


Figure 13: Concrete framing plan of typical floor

Post-tensioning tendons were originally placed in each of the beams in the above framing plan. However, in order to reduce post-tensioning costs, tendons were entirely removed from beams along column lines 1, 8, A, B, J and K, as well as along column lines 4 and 5 between lines A and C, and between lines H and K. All of these spans are under 28' in length and can be designed as regularly reinforce concrete sections. Although RAM Conc*ept* designs regularly reinforced concrete sections, the design was verified through hand calculations which can be found in Appendix B.

In the design of post-tensioned concrete beams, the load balanced in by the post-tensioning force tends to be in the range of 80%-110% of the dead load. The design of a tendon spanning multiple lengths like the spans found in the system depicted above, the longest span is usually designed first and labeled as the "critical span". For the design in this system, the critical span was 44'4" in the longitudinal direction. Using the maximum possible drape in the section as one limiting criterion, and the minimum precompression force (7.5√f'c by assumption #1) as the other, an efficient number of tendons and tendon profile should be designed for this span.

For the 28' span adjacent to the critical span, it is practical to design for a smaller percentage of the dead load because less upward force in this adjacent span reduces the design of the critical span. This can be done by either reducing the number tendons in the section, or decreasing the tendon drape. It is usually preferred to design using the latter method, as it simplifies the constructability of the system. The overall goal of modifying the tendon drape profile through a section over a varying length is to find a constant jacking force that will be applied throughout the length of the tendon and will resist an acceptable percentage of the design dead load. For an illustration of the effect sag has on tendon tension, see Appendix B.

The tendon drape profile and number of tendons in each beam were designed using the method above in order to maximize strength of each tendon over varying spans. The depth of the drape or "sag" of each tendon is related to the tensioning force required through the equation $F = w_{pre}L^2/8s$, where the term "s" is the sag of the tendon in inches. The other terms, "w_{pre}" and "L" are the balanced load in design and length of span, respectively. The final tendon profile for a typical beam spanning the longitudinal building direction can be seen below. Beams in the latitude direction were designed using the iterative process. Tendon plans for both directions can be found in Appendix B.

The tendon profile below uses 16 unbonded ½"ø 7-wire stands throughout the

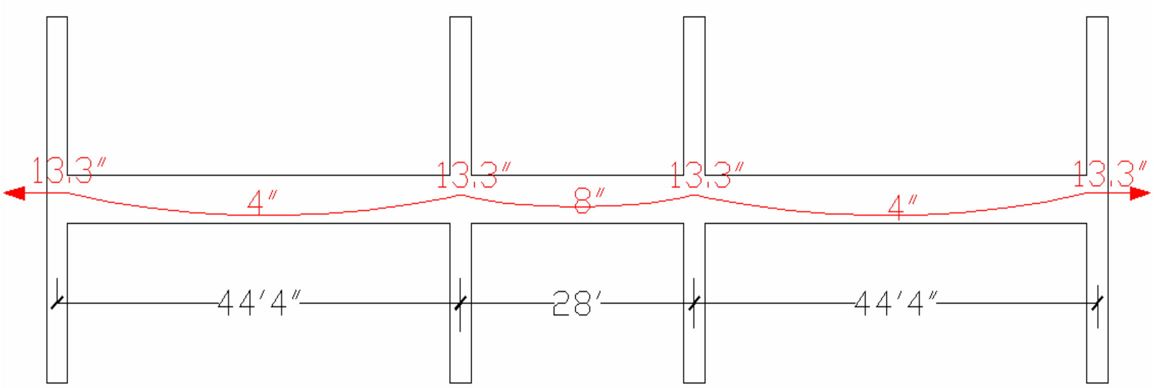


Figure 14: Tendon profile for typical beam in longitudinal direction. Drape dimensions are from bottom of concrete soffit.

entire profile with a capacity 26.6kips/strand, resulting in a total jacking force of 425kips. The total span of this tendon is 116'8", which falls under the 120' maximum length for jacking a tendon from one side only. This will allow for simplified constructability by the post-tensioning contractor.

As previously mentioned, one advantage of post-tensioning concrete is the ability to minimize deflections due to the upward camber created by the compressive forces created from tensioning the tendon. For this design, the deflection of the floor system was limited by $\ell/360$ from ACI 318. This equates to a maximum deflection of 1.47" in the 44'4" spans; a limitation which the system meets. It should also be noted that designing the beams of this system as a "Class T" member allows for higher precompression stresses, but also uses the cracked section as the basis for deflection calculations. The cracked moment of inertia for any section tends to be around half the uncracked section, effectively doubling any deflections using the cracked section. This is a serious design consideration, and would have to be counteracted by adding strands to the section or increasing the tension force in the strand. Below are the deflection plans for the flooring system design. The plans include deflection under initial service loading, sustained service loading, and long term loading. The long term loading plan takes into account creep in the concrete and the post-tensioning tendons over a considerable length of the building's lifetime. While it important to minimize the deflection under long term loads and know how the building reacts under long term loads, deflections are expected to be slightly higher and may slightly exceed deflection design criteria. However, over the lifetime of the building, a deflection of 1.6" will be noticeable, but not incredibly uncomfortable to the tenant.

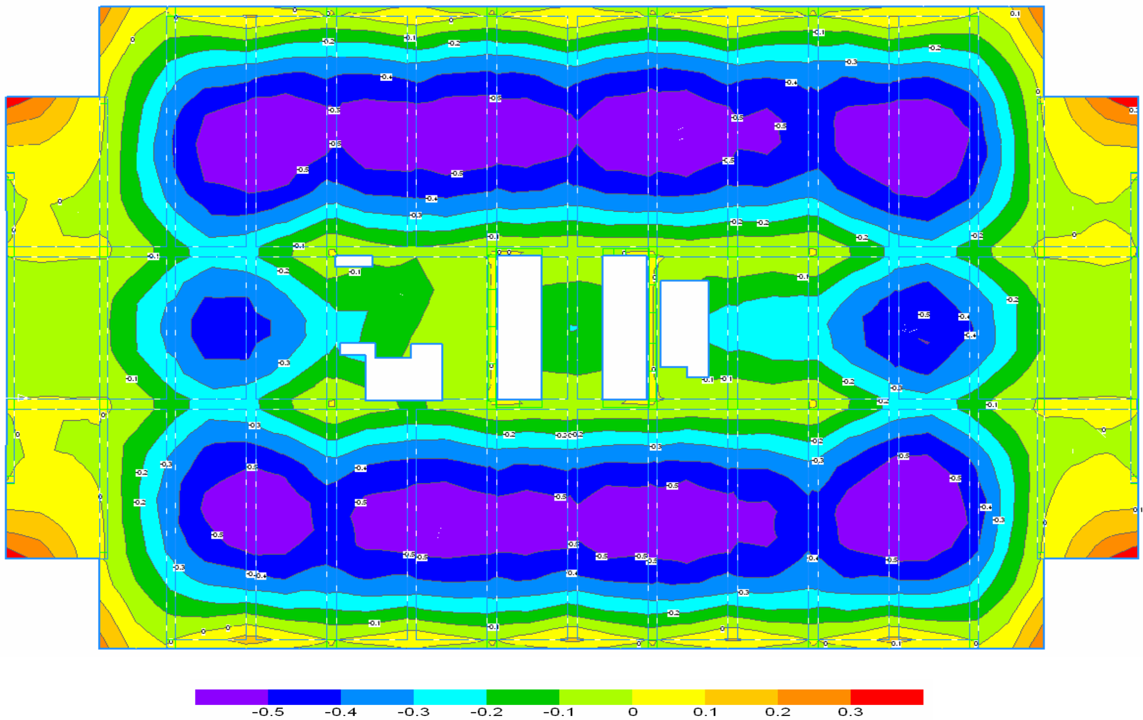


Figure 15: Deflection plan under initial service loading

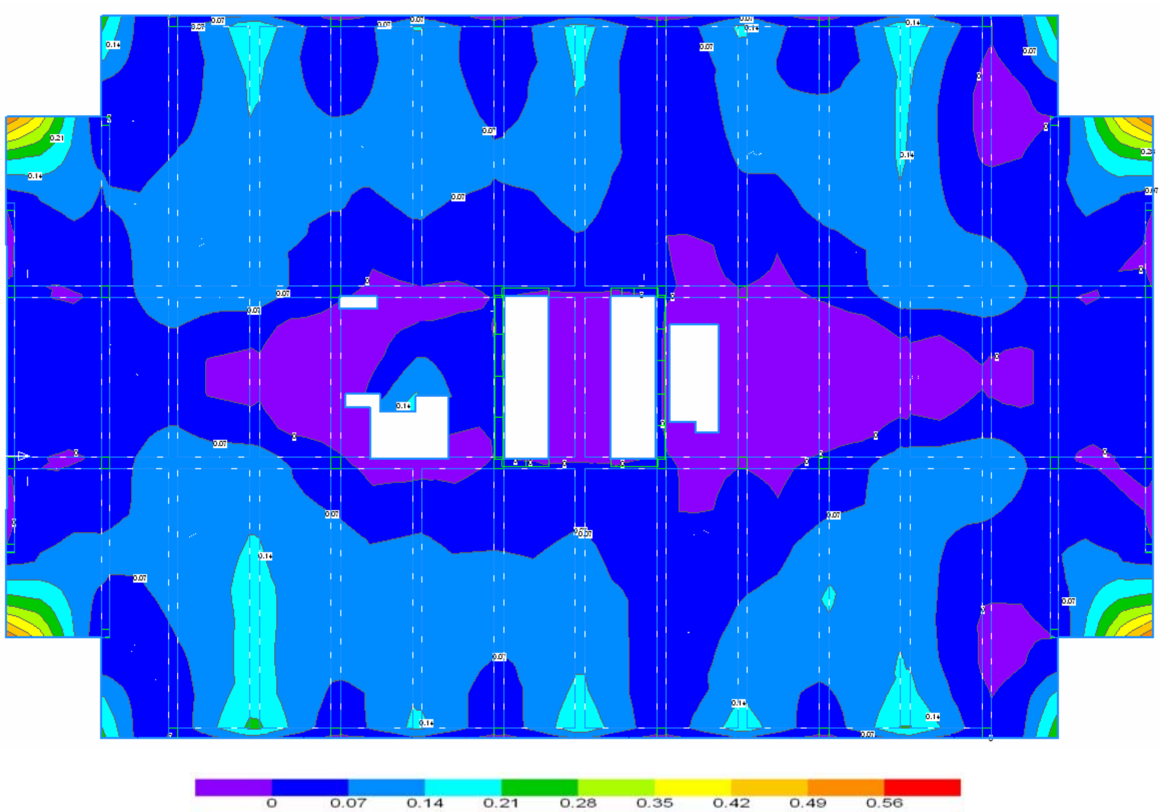


Figure 16: Deflection plan under sustained load

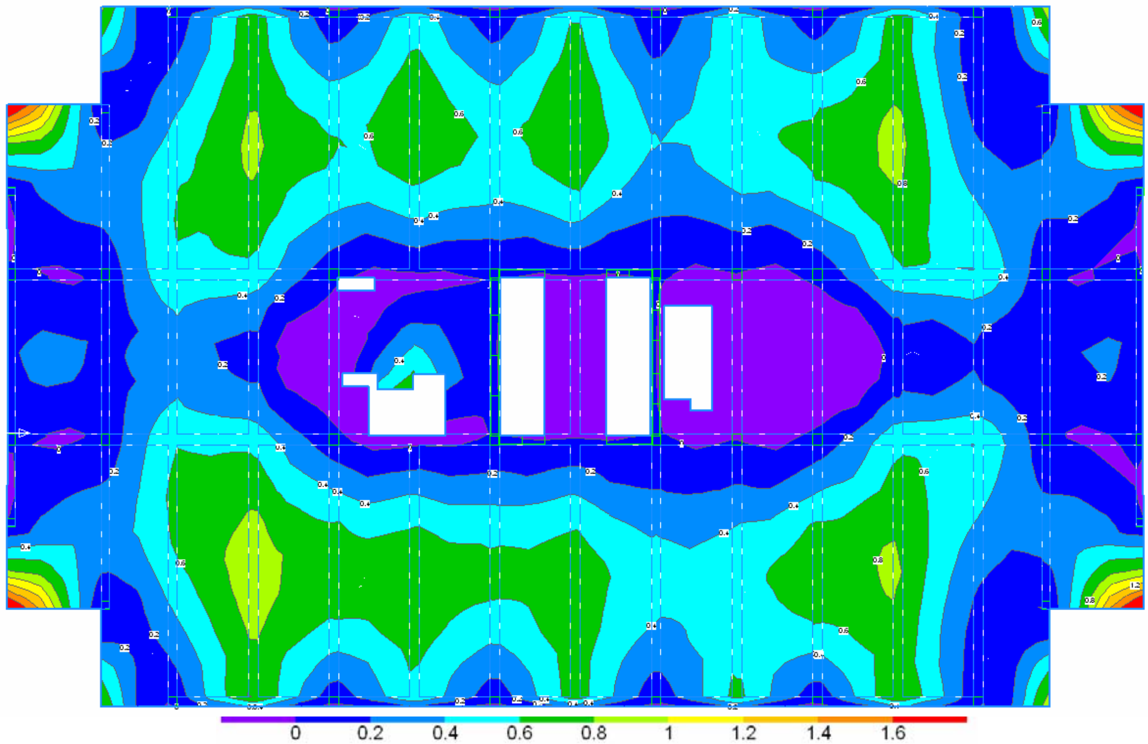
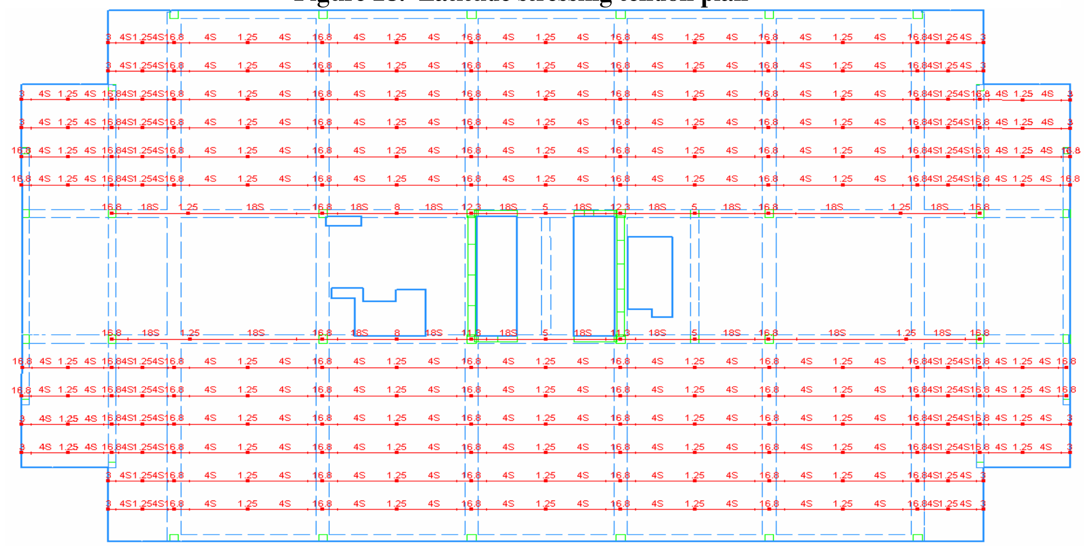


Figure 17: Deflection plan under long term loading

FLOOR SYSTEM DESIGN #2

Once an acceptable flooring system was designed in RAM Conc*ept*, and a better understanding of how post-tensioning works was obtained, it became evident that a design with post-tensioning found only in the beams could be altered to include post-tensioning in the slab and beam a spacing of twice the distance. These beams would need to be designed as wide beams, and additional post-tensioning tendons would needed to be added to resist the dead load from the increased self weight of the wide beams, as well as the concrete weight being carried from double the tributary area. As the number of design iterations increased, it was found that a wider beam with additional post-tensioning added could resist the same loads as a deeper section with less post-tensioning. A wide beam design would decrease the depth of the system, thus shortening the floor to floor height of the building and overall height. This design required bundles of 4 ½"ø 7-wire stands spaced a little over 6' apart in the slab, and an average of 26 strands in the 18"x30" wide beams spanning in the

Figure 18: Latitude stressing tendon plan



longitudinal direction. The latitude tendon plan is shown in the diagram above. The final depth of the post-tensioned slab floor system was found to be 18" including a 6" slab; a reduction of 5-1/4" from the original steel system and a decrease of 2" from the first post-tensioned flooring system. The post-tensioned beam and slab design adequately met the L/360 deflection rating, and even only slightly exceeded the rating over long term loading, which speaks well for the strength of the system over the life of the building. Below are the initial, sustained and long term service load deflection plans for a post-tensioned beam and slab system.

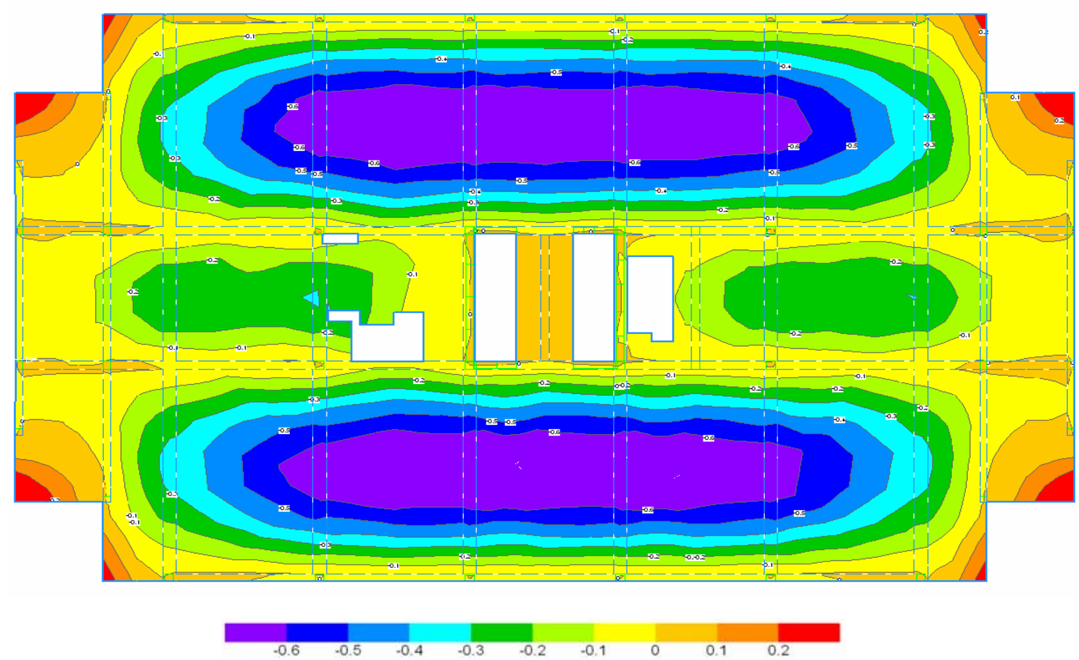


Figure 20: Deflection plan under initial service load

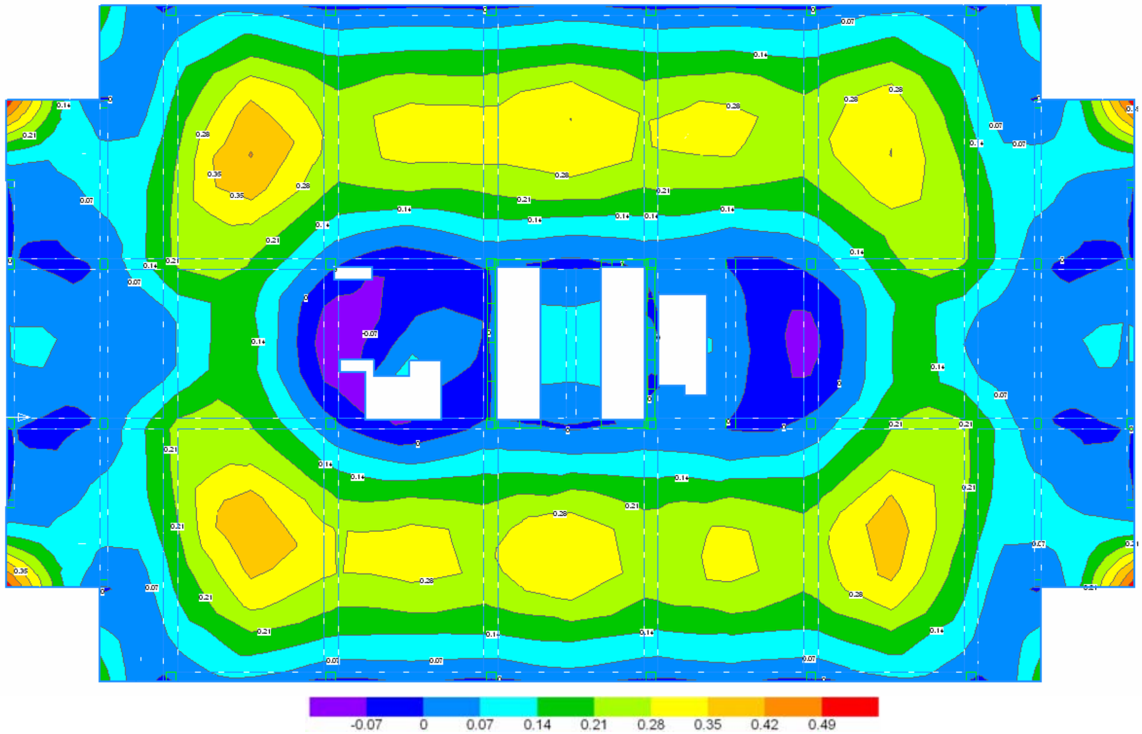


Figure 21: Deflection plan under sustained service loads

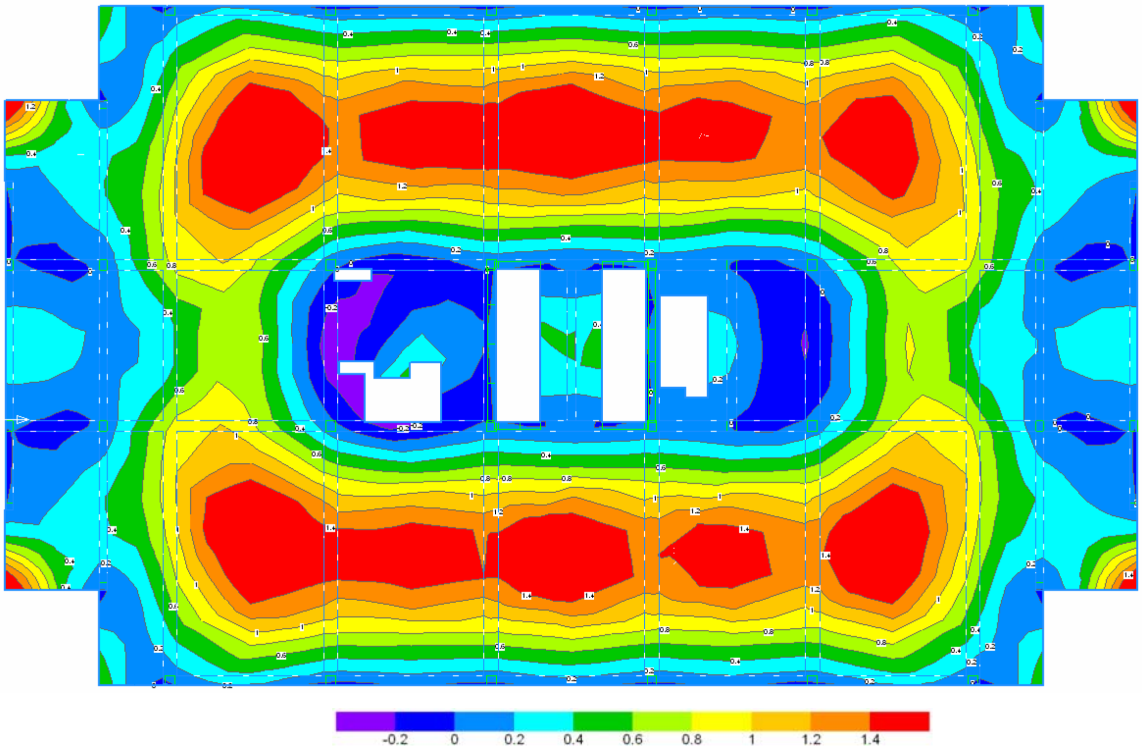


Figure 22: Deflection plan under long term loading

CONCRETE COLUMN DESIGN

As previously mentioned, columns were assigned a trial section of 20" x 20" in order to create a post-tensioned floor system model in RAM Con*cept*. Again, this section was determined by finding the end moments on the beams from a moment distribution, initially estimating a beam size, and then keeping the dimensions of the column the same as the beam to keep a constant moment of inertia. However, it is evident by inspection that a base column with dimensions of 20"x20" wouldn't be nearly large enough to support 15 stories of weight above it given the current column layout of Eight Tower Bridge.

To get a better idea of the axial forces carried in each column at every level, a simple spreadsheet was created that factored in tributary area, dead load, reduced live load, roof load and mechanical rooftop loading. The weight of the concrete slab was also taken into account over the tributary area. These spreadsheets can be found in Appendix C.

The columns must also be designed to resist bending moment about both axes. In order to obtain bending moments due to gravity loads, a reaction plan created by RAM Conc*ept* was used. The moments from this plan were determined to be more accurate than the moments derived through the moment distribution spreadsheets created. However, it was recognized that the moment outputs from this plan were for 20"x20" columns only, and did not take into account that a larger column would take more moment from the beams in a distribution. In order to estimate the increased moment a larger column size would take, the moment distribution spread sheet used to determine the trial section size was run with a constant beam size and an increasing column size. Although the distribution of moments is not on a linear scale, a rough "moment multiplier" was determined for columns larger and smaller than the 20"x20" column moments obtained from RAM Con*cept*.

With both axial loads and bending moments obtained for all columns, the program PCA column was used to obtain column sizes and reinforcement that could withstand the given moments. The columns loads were entered as service loads, and the load cases of 1.2D+1.6L and 1.4D were used. Below are the service loads entered into and an output from PCA COL for the base columns at D4 and D5. These two column marks carried the largest load, having the largest tributary area and supporting half of the load created from the mechanical system room located on every floor.

D4, D5	Axial (kips)	M_{x-x} (ft-kips)	M_{y-y} (ft-kips)
Dead	2051	271	125
Live	803	185	78

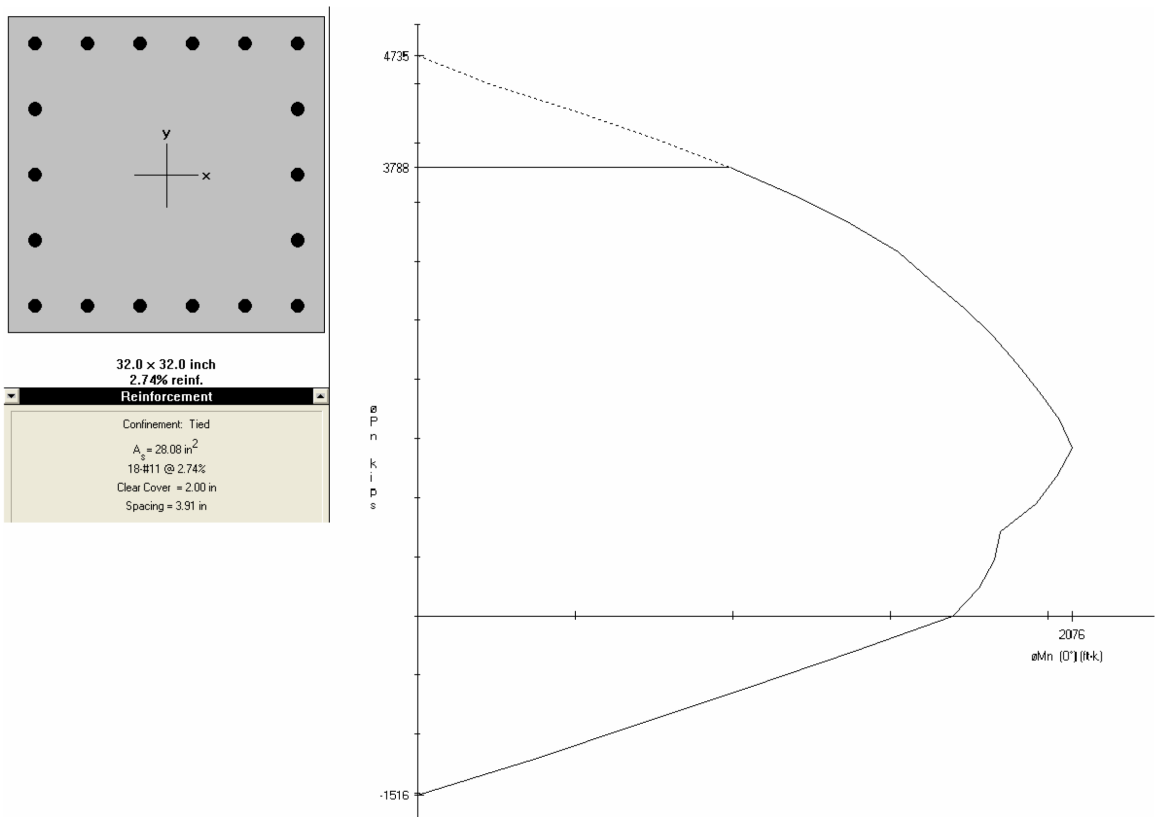


Figure 23: Initial PCA COL output with interaction chart

The output suggests that a 32"x32" 6000psi concrete column reinforced with 18-#11 bars be used to resist the given moments. From discussion with design professionals and referencing similar sized concrete office building plans, it was determined that a column size of 32"x32" for the given tributary area is common. It was also previously mentioned that RAM Conc*ept* does not take into account lateral forces, so the moments caused by both gravity and lateral loads needed to be obtained. However, the initial 32"x32" column size seems to work well for the given axial loads, so the reinforcement will need to be verified for bending.

The above design process was used for each column mark, changing the loads every three levels. For contractibility, it is more efficient to re-size columns every few floors rather than at every level. Column sizes were obtained for every level using PCA COL.

With the floor system designed and a tentative column schedule, a complete building model could now be constructed using ETABS. This model took into account moments created from lateral loads, and helped refine the column sizes and reinforcement obtained through PCA COL. There is also a design feature on ETABS that outputs the suggested area of steel to include in each column. This feature was used as a check against the PCA COL output as well. Below flexural summary from the concrete design feature used in ETABS.

```
ACI 318-99 COLUMN SECTION DESIGN  Type: Sway Special  Units: Kip-Ft  (Flexural Details)

Level        : LEVEL 2          L=11.000
Element      : C26              B=2.667          D=2.667         dc=0.150
Station Loc  : 10.167           E=518400.000     fc=864.000      Lt.Wt. Fac.=1.000
Section ID   : 6KSI-32X32COL    fy=8640.000      fys=8640.000
Combo ID     : DCON18           RLLF=1.000

Phi(Compression-Spiral): 0.750  Overstrength Factor: 1.25
Phi(Compression-Tied):   0.700
Phi(Tension):            0.900
Phi(Bending):            0.900
Phi(Shear/Torsion):      0.850

AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR PU, M2, M3
                            Rebar      Rebar    Design    Design    Design
                             Area          %        Pu       Mu2       Mu3
                            0.071      1.000   339.217   -58.105    44.098

Factored & Minimum Biaxial Moments
                         Non-Sway       Sway  Factored   Minimum   Minimum
                              Mns         Ms        Mu      Mmin Eccentrcty
    Major Bending(M3)      18.772      0.235    19.007    44.098     0.130
    Minor Bending(M2)      -7.210    -50.894   -58.105    44.098     0.130

Axial Force & Biaxial Moment Factors
                               Cm   Delta_ns   Delta_s         K         L
                           Factor     Factor    Factor    Factor    Length
    Major Bending(M3)       0.600      1.000     1.000     1.000    10.167
    Minor Bending(M2)       0.600      1.000     1.000     1.000    10.167
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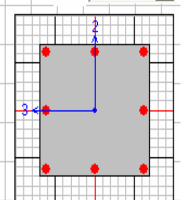
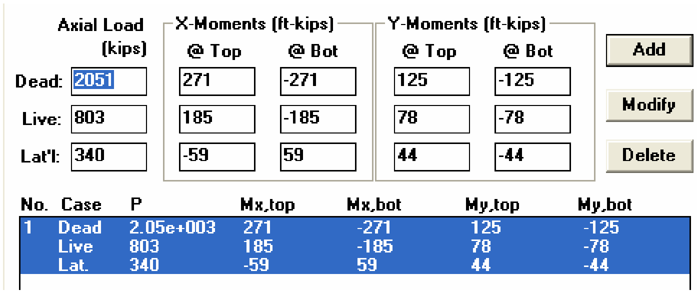
Units Kip-ft



These moments were added to the gravity moments in PCA COL to verify that the reinforcement in the column will be enough to resist later loads. Below is a view of the loads that were put into PCA COL as well as an output with these loads.



No.	Case	P	Mx,top	Mx,bot	My,top	My,bot
1	Dead	2.05e+003	271	-271	125	-125
	Live	803	185	-185	78	-78
	Lat.	340	-59	59	44	-44

The input of lateral loads

Above: Table output of loads input to PCA COL
Below: Output and interaction diagram from PCA COL

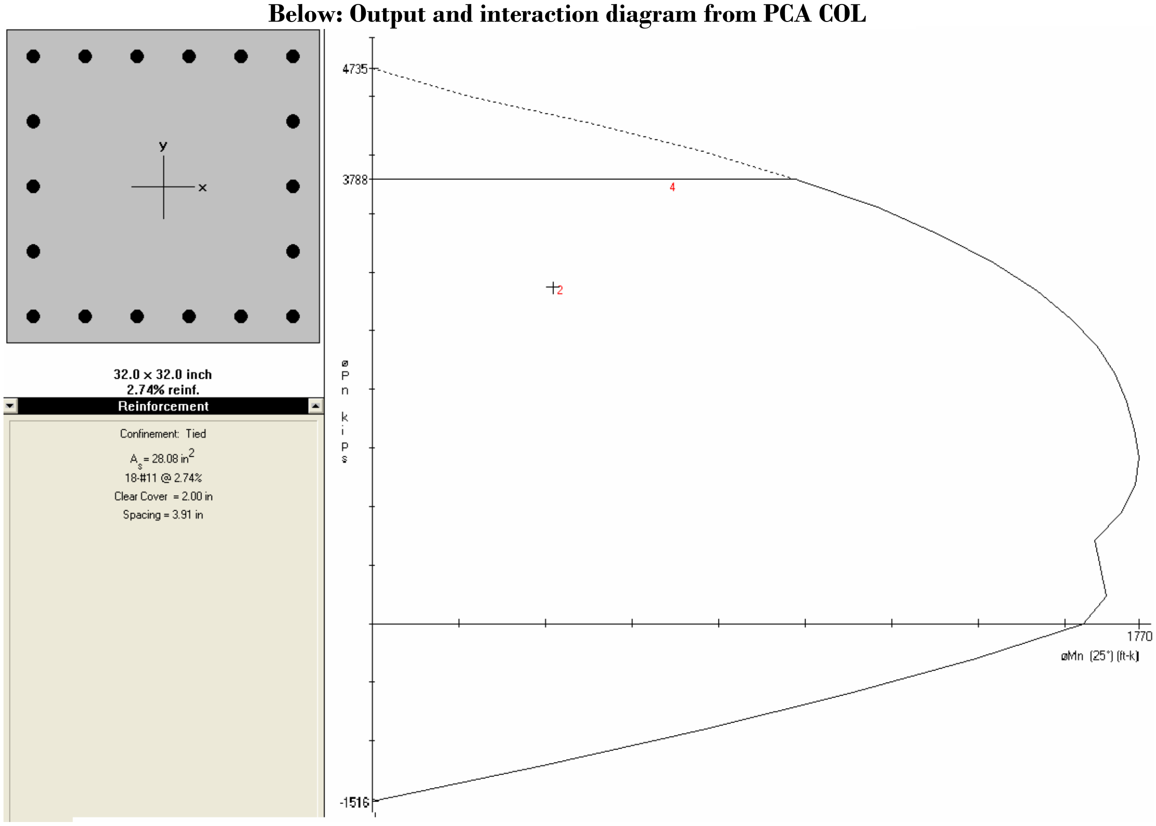


Figure 24: Refined PCA COL output using lateral loads from ETABS

for this design shows that a 32"x32" column with 18-#11 bars is adequate reinforcement for this column. The process was repeated for the rest of the columns, and a full column schedule was produced. This schedule can be found in Appendix C. It was found that the strength of the concrete could be reduced at the 10th level to 5000psi, which is the same strength as the other structural elements, making concrete placement easier for construction crews.

It should be noted that the columns were not designed as the main lateral force resisting members, even though the concrete frames will act as rigid frames and take moment. The shear wall design is discussed in the next section of this report.

SHEAR WALL DESIGN

The main wind lateral force resisting system of this alternate concrete design is comprised of 8, 12" thick shear walls located around the building core throughout the entire building height. The lateral system was modeled using ETABS.

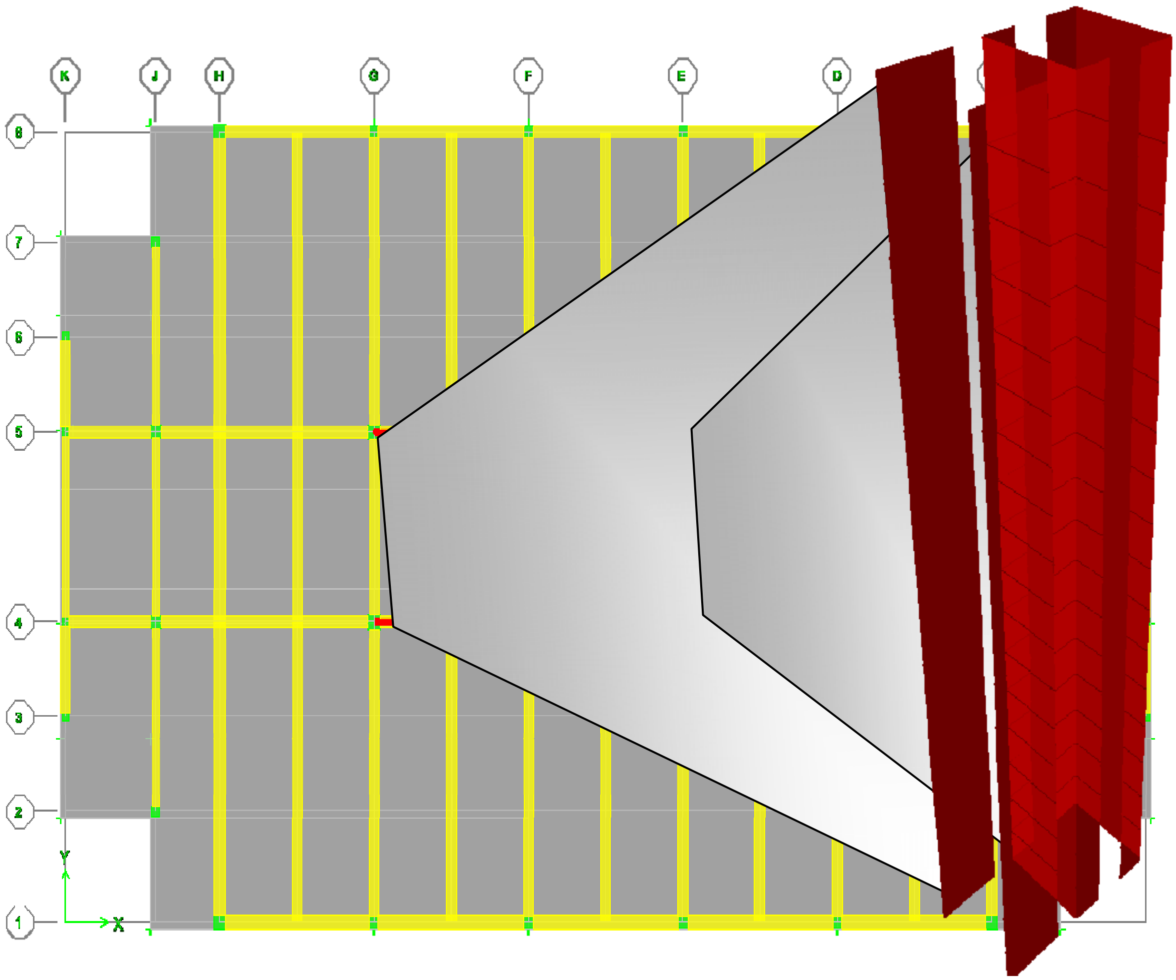


Figure 25: 3-D view of shear walls over building plan

The first design model run included 2 more shear walls along column lines D and G. They were removed after it was found that the shear wall plan above was enough to resist the lateral loads on the building. The two shear walls resisting load in the Y-direction are 28' long, while the four short walls in the X-direction are each 9'4". The two additional walls in the X-direction span 20' and have been intentionally cut short of the full 28' bay length in order to allow for doorway openings to the stair tower found within that bay. All walls are 5000psi normal weight concrete.

There were five main load cases input into ETABS to obtain the deflection of the building and the forces on each shear wall. They are as follows:

1. Seismic in both X and Y directions
2. ASCE7-02 Wind Case 1
3. ASCE7-02 Wind Case 2
4. ASCE7-02 Wind Case 3
5. ASCE7-02 Wind Case 4

Wind Case 2 and Case 3 were input into ETABS without eccentricity. An eccentricity of 15% of the building length was then added by hand to account for the torsion created from eccentric loading. Please refer to Appendix D for the ASCE7-02 description of load cases, as well as additional load cases run in ETABS.

The deflection found in each of these load cases is summarized in the table below:

Lateral Load Deflection Summary		
	△ X	△ Y
Wind Case 1X	1.76"	-
Wind Case 1Y	-	1.65"
Seismic X	4.66"	-
Seismic Y	-	4.55"
Wind Case 2X	1.32”	-
Wind Case 2Y	-	1.23"
Wind Case 3	1.29"	1.23"
Wind Case 4	0.98"	0.93"

The controlling deflection case in both directions was found to be seismic. This differs from the controlling cases found for the original steel building (both were wind) due to the increased weight of the building. However, the building is not located in a very heavy seismic region, so the deflections resulting from earthquake loads will be at a minimum. Below is an elevation along column line D of the deflected shape of the building under the Seismic X loading.

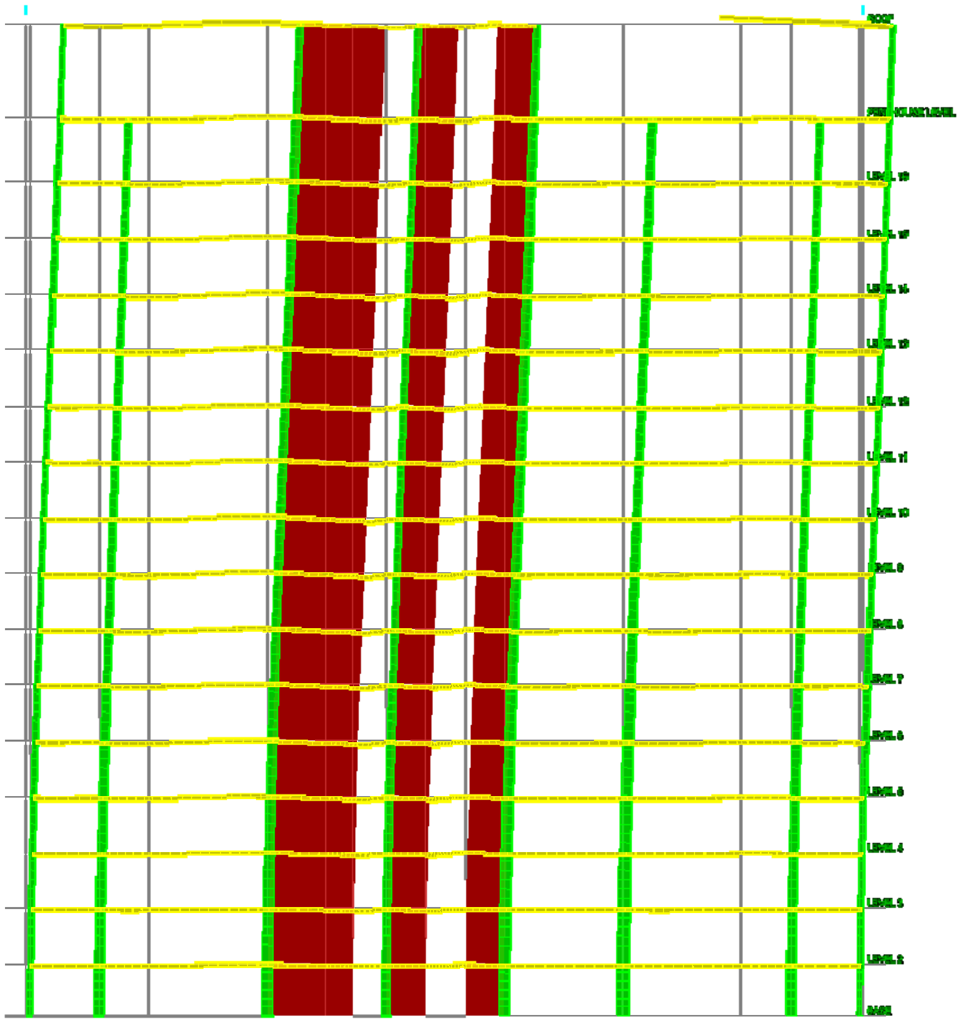


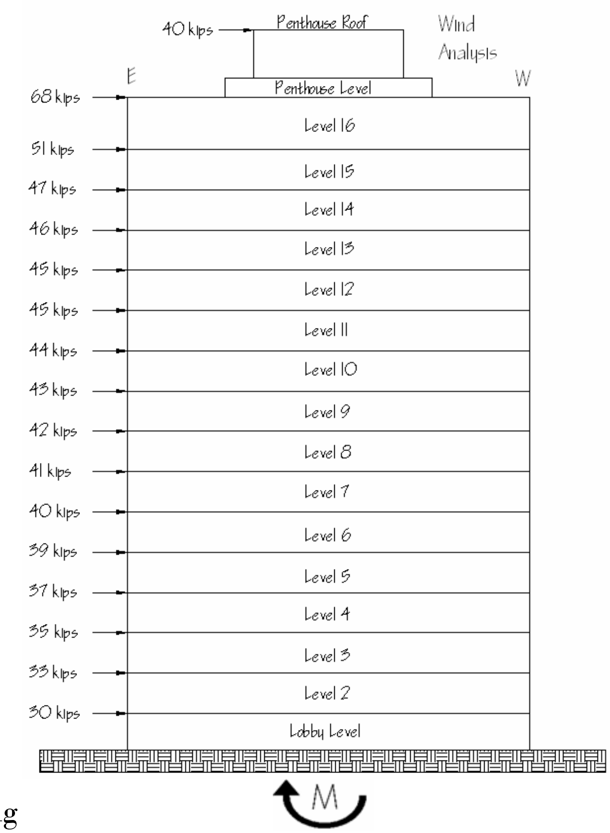
Figure 26: Deflection under seismic loading in the X-direction

The base force for each shear wall under the above loading is summarized in the table below. The shear wall forces for the remaining load cases considered can be found in Appendix D. These forces were verified by spreadsheet calculations, also found in Appendix D.

Shear Wall Forces under Seismic X Loading (kips)								
	Wall							
Level	**A**	**B**	**C**	**D**	**E**	**F**	**G**	**H**
Level 2	99.95	N/A	99.36	97.15	N/A	97.77	266	264.8

Shear wall reinforcement ratios were also designed in ETABS and fell between 2.34% and 2.92% at the wall base and .25% at the top story of the wall.

Although the largest deflection was found to be under seismic loading, Eight Tower Bridge will be primarily resisting wind loads, as it is not located in an area of high seismic activity. The wind story forces acting in the y-dimension are shown below. The total of these forces added to a total overturning moment 84,320 ft-k at the base. This overturning moment is resisted by a total building dead weight of 38,733 kips, which performs adequate resistance. It should also be noted that these calculations were also performed for a reduced story height for both alternative flooring systems. The first alternative flooring system, which reduced the overall building height 4'8" (3-1/4" per floor) reduced the overturning moment by 4,400 ft-kips. The second floor system, which reduced the overall building height by 7' (5-1/4" per floor) subtracted over 7,700 ft-kips from the overturning moment.



FOUNDATION DISCUSSION

Although the foundation was not redesigned for the alternate concrete system, it should be noted that the foundation would need to be redesigned slightly. Concrete buildings are generally heavier structures despites a 3:1 ration in weight per cubic foot of steel to concrete, which results in an overall increased building dead load. The increased dead load was seen when performing seismic calculations. Increasing the foundation strength capacity could be done by increasing the concrete strength from 4000psi to 5000psi, increase the dimensions of the pile caps or increase the quantity of piles driven. All of these foundation design options could be explored independently at critical locations or in combination to increase foundation performance.

OTHER CONSIDERATIONS

There are a few other considerations involving the design of Eight Tower Bridge in concrete, specifically post-tensioned concrete rather than steel. The first issue concerns the post-tensioning. When the concrete is being post-tensioned, there are increased forces formed from pulling on the tendons. This could be a serious problem if not designed for, especially at points on the structure that are not as laterally stable. For example, in the first concrete flooring system, post-tensioning tendons would have to be run through beams in between column line, falling at the mid span of the perimeter beams. This could result in added torsion and lateral bending effects in the beam during construction. Moments can also be created in columns when post-tensioning tendons run through column-beam joints.

Another general concern when designing any concrete structure is rebar crowding. This issue can become particularly difficult when dealing with post-tensioning tendons that vary their profile throughout the member section. This design concern was evaluated in the design of this concrete system, as additional space was left towards the bottom of each beam in the longitudinal direction. Even without rebar crowding, this space will still allow for easier concrete placement to the soffit and in between tendon bundles

A third concern is the rooftop mechanical penthouse located on the roof of Eight Tower Bridge. While it is possible to construct a rooftop penthouse out of concrete, they are more easily constructed out of steel. A RAM Concept model was run with point loads placed along the length of beams to model transfer columns from a penthouse design, and met strength requirements after additional post-tensioning tendons were added to the beam. Moving the penthouse HVAC equipment to the basement was considered for the concrete system, but with the close site proximity to the Schuylkill River, even the slightest flood could costs million of dollars in HVAC equipment damage, eliminating the feasibility of this move.

Finally, Philadelphia is not particularly well known as a "high post-tensioned building" area, and contractors are not prominent from the searches that were conducted. Therefore, a post-tensioning contractor would have to be carefully selected if this were to become a post-tensioned concrete project.

ALTERNATE SYSTEM SUMMARY

An alternate concrete superstructure was designed for Eight Tower Bridge. The structure will be comprised of a post-tensioned concrete beam and slab system. Two alternate systems were designed. The first system employs a 6" reinforced concrete slab cast monolithically with post tensioned beams spaced 14' apart. The second system involves a 6" post-tensioned concrete slab cast monolithically with post-tensioned beams spaces 28' apart. A summary of both systems can be seen in the table above. More information about these systems can be found in Appendix B.

	Typical Beam Size	Overall Depth	Sustained Deflection	2 hour fire rating?
System #1	20"x20"	20"	.57"	✔
System #2	18"x30"	18"	.55"	✔

Cast in place concrete columns were designed to support both of these floor systems. The largest of these columns was found to be a 32" square column reinforced with 18-#11 bars, and was found at the building base. The most prominent column selection was a 20" square column with varying amounts of reinforcement, decreasing as level location increases. A complete column schedule can be found in Appendix C.

The main lateral force resisting system is comprised of 8, 12" thick shear walls. Six of these shear walls are arranged in a "channel" formation around the building elevator core, while the additional two walls span along the building's y-directions. This shear wall formation yielded a maximum deflection of 4.66" under seismic loading in the X-direction. Calculations and computer output related the design of these shear walls can be found in Appendix D.