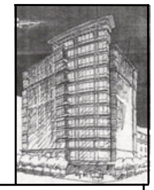


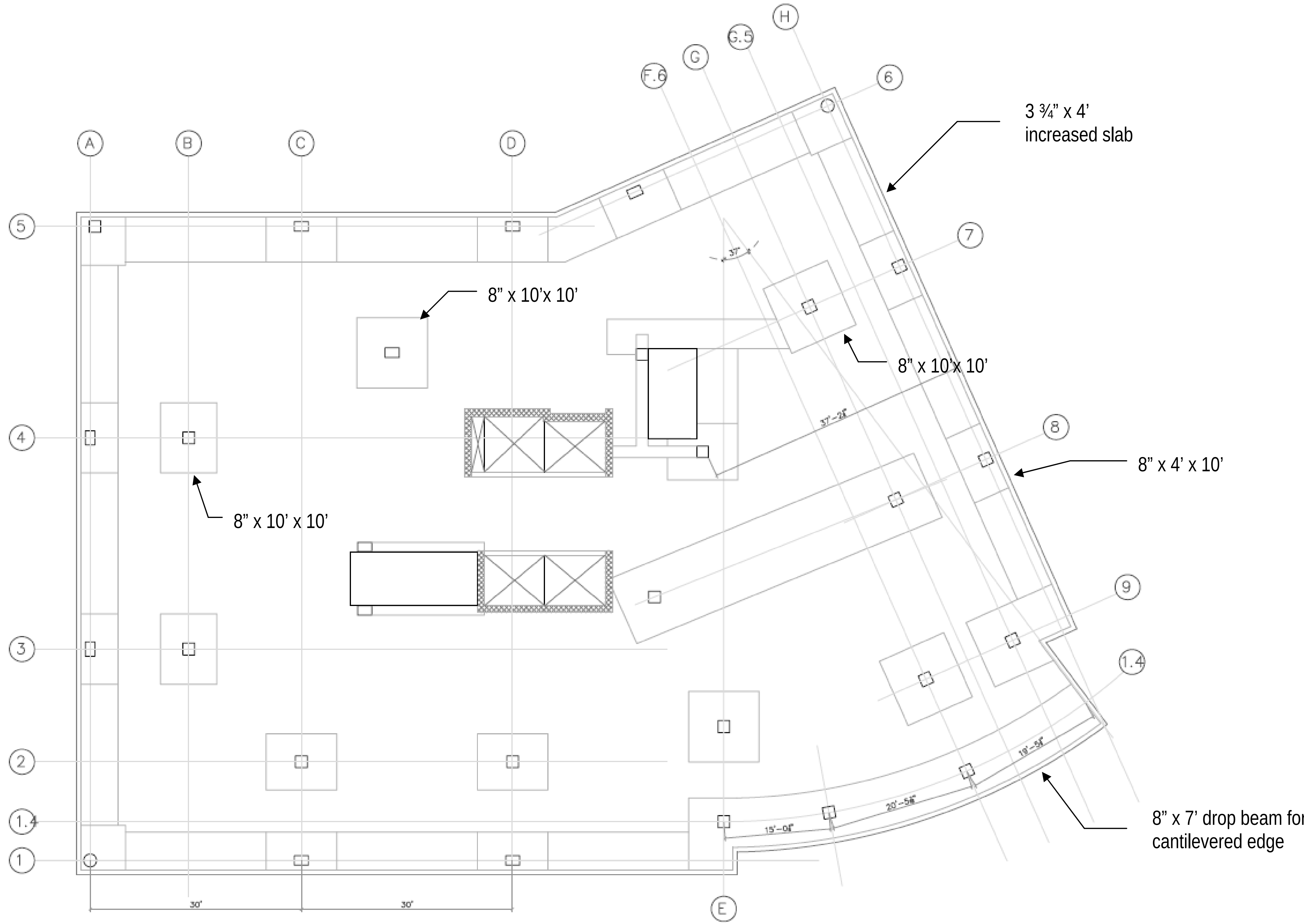
POST TENSIONING

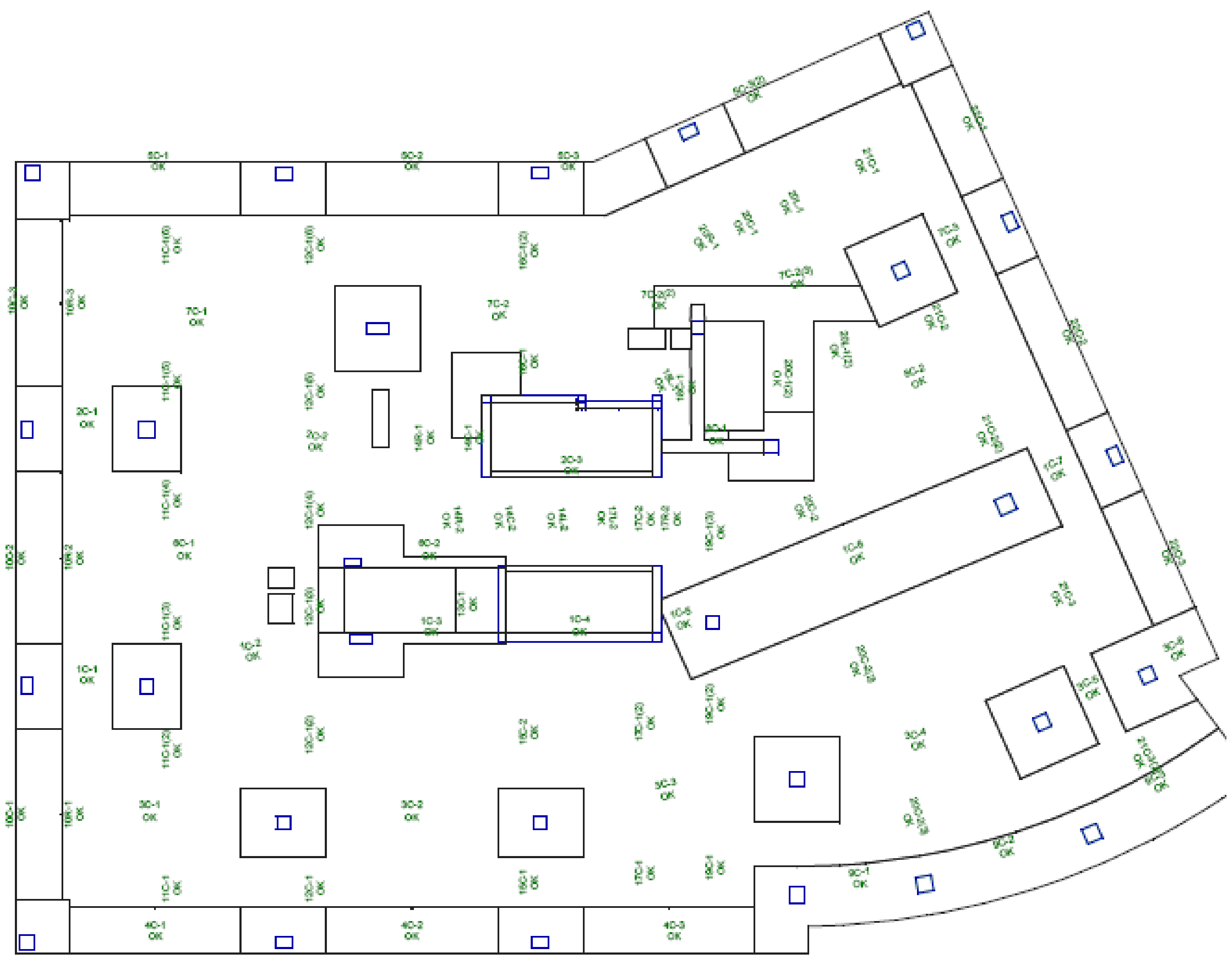
The third and final step in completing the proposal is the application of a two-way post tensioned slab in order to reduce the depth of the framing system for the Executive Tower by three inches. The existing system is an eight inch two-way flat slab with eight inch drop panels at all column locations. An increased slab thickness of three and three quarter inches acts at a perimeter beam around the entire building except for in one place. The curve perimeter section is supported by three columns with a 19 foot cantilever on the south end. This section of the slab has an eight inch by seven foot drop beam added to the thickness of the slab. A detailed drawing of the structural floor plan can be found on the following page (24).

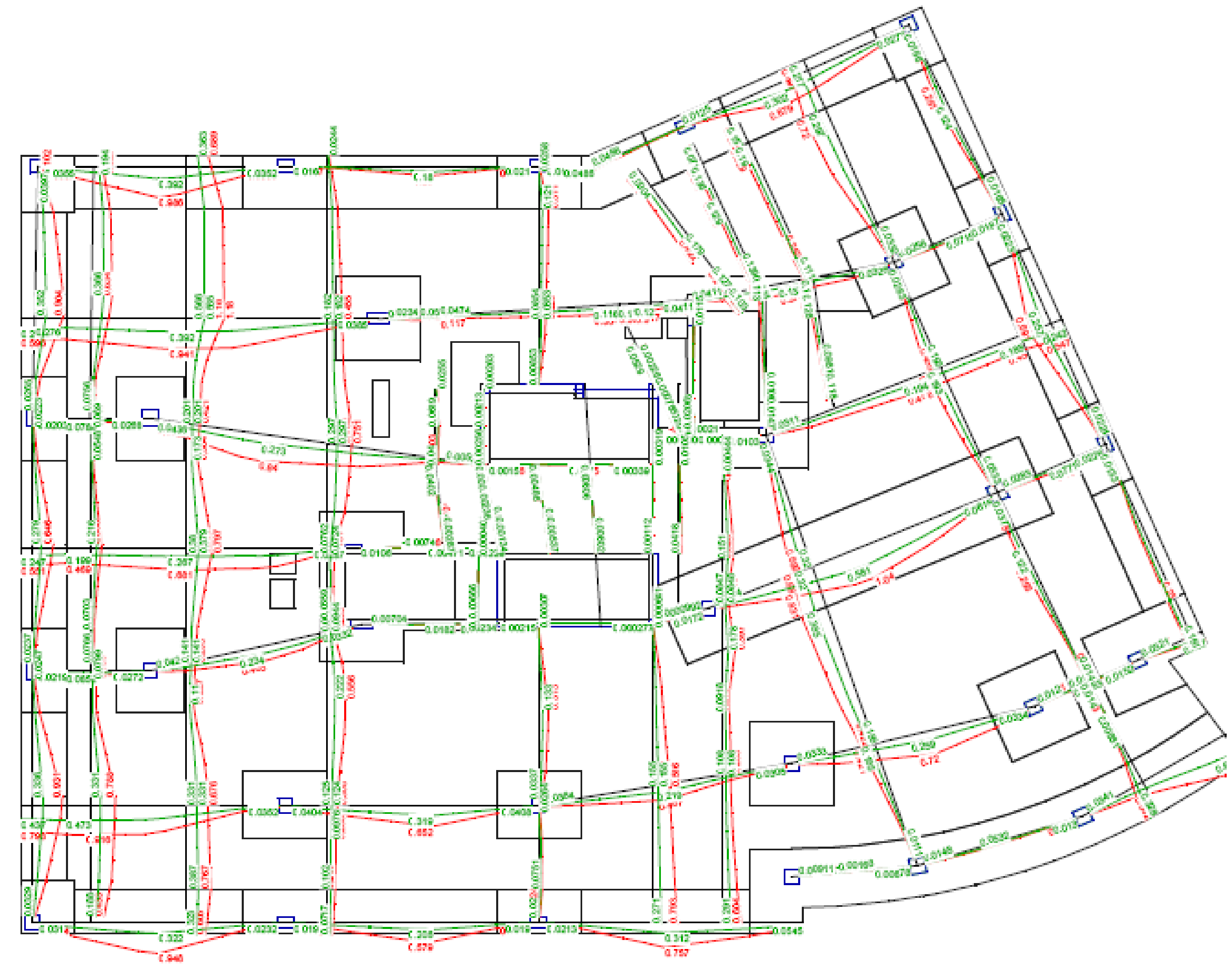
In order to achieve the goal of a three inch reduction, it was decided as of Technical Report 2 to convert the current system to a two-way post tensioned slab. In order to analyze the post tensioning due the Executive Tower's disorganized column layout, a structure program that undertook a finite analysis was used.

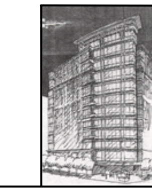
The Executive Tower was constructed in RAM Concept by developing the original system without any post tensioning tendons and then allowing it to run its analysis. The results were conclusive, the original system worked for the most part in RAM Concept. The areas of failure are due to sections of the slab that were reinforced more because the #4 @ 12" web was insufficient. The results of this analysis can be seen including the deflections on page 25. This is in agreement with the findings from Technical Report 1.

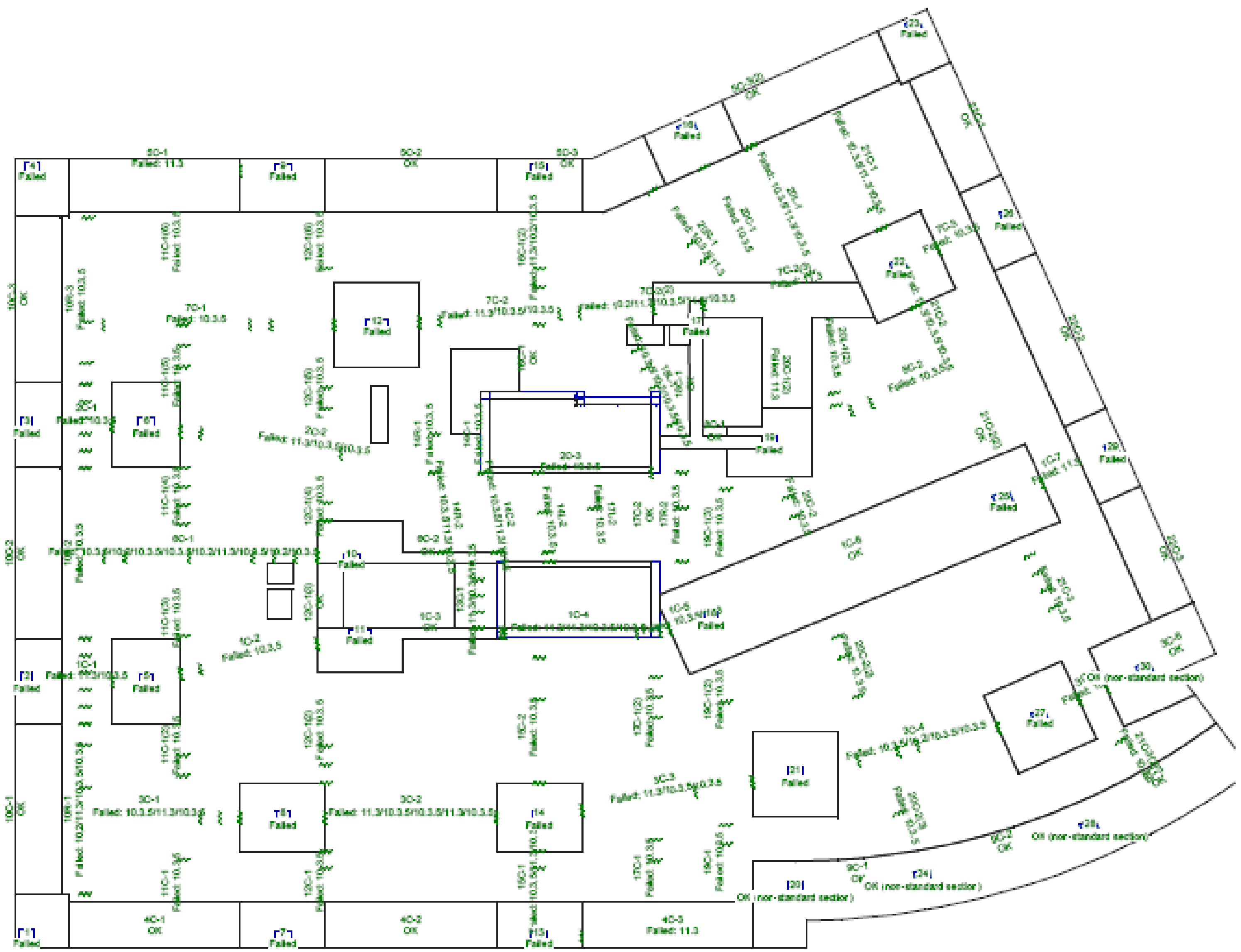
On page 26, RAM was then run with a flat slab system with the slab reduced by three inches proving the application of a post tensioning system is necessary to achieve the goal of a thinner slab. Note the slab fails in multiple places and where it does not fail the deflections in the five inch slab are considerably greater, some as high as five inches.

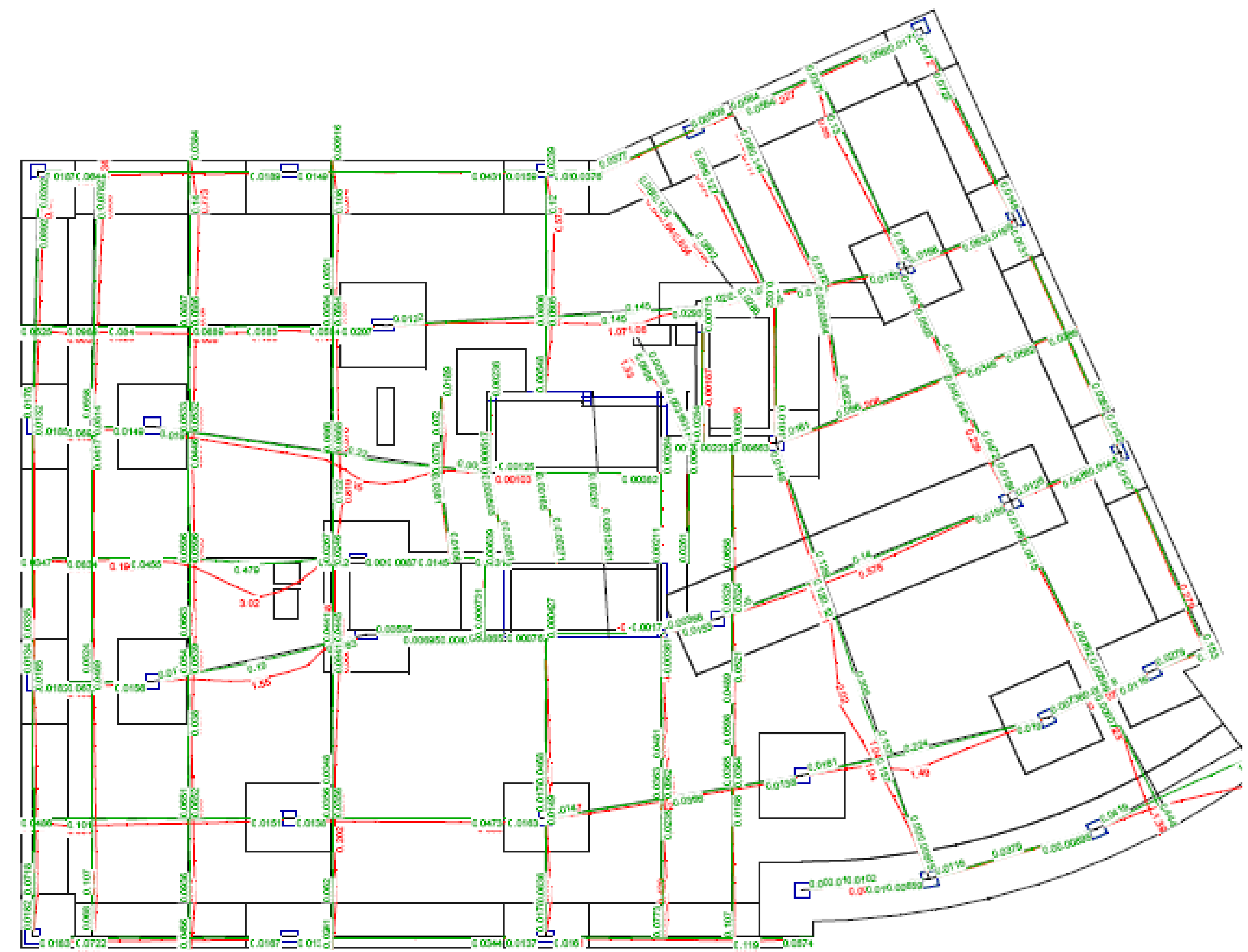

3 ¾" x 4'
increased slab
8" x 10'x 10'
8" x 10'x 10'
8" x 4' x 10'
8" x 10' x 10'
8" x 7' drop beam for
cantilevered edge
A
B
C
D
E
F.6
G
G.5
H
1
1.4
2
3
4
5
6
7
8
9
30'
30'

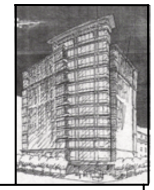










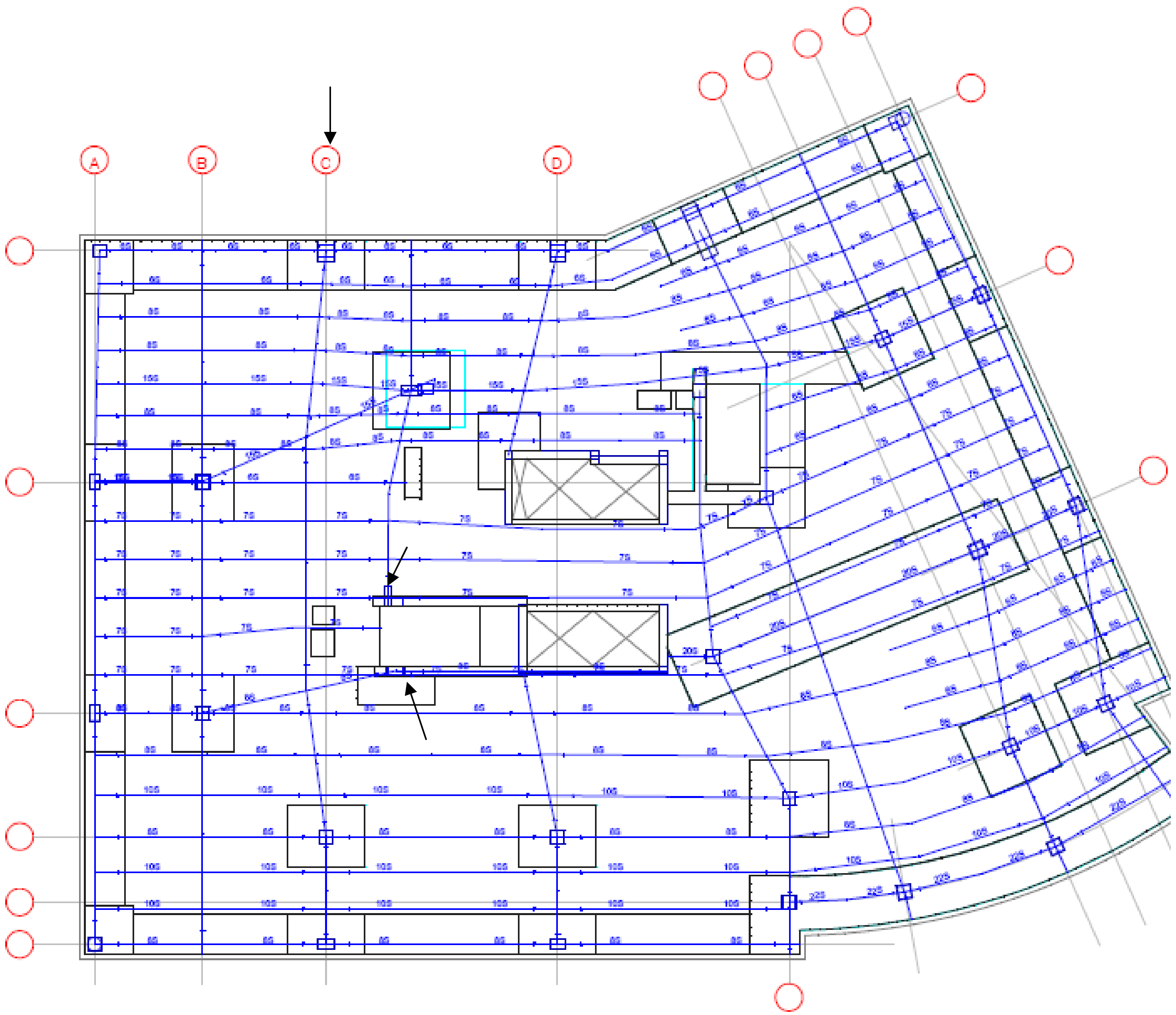


TRIAL 1

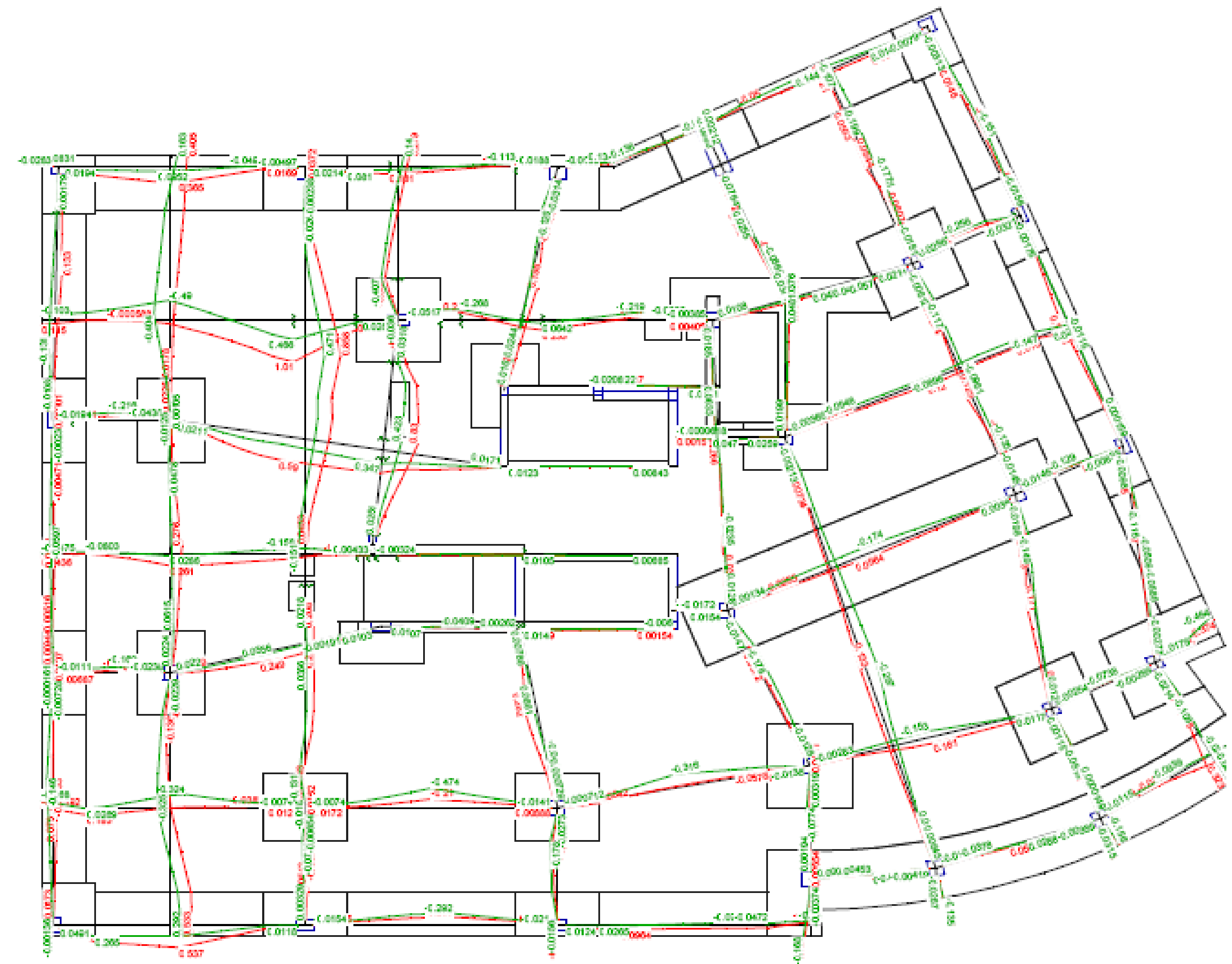
To develop a workable post tensioning system, the column strips need to be laid out meeting as many columns as possible. In the case of the Executive Tower, the columns do not line up along one column line grid. The column strips needed to be skewed in several places. The end result was a tendon layout just as irregular. The longitudinal tendons were bundle in groups of 15 making the longitude direction the strong direction and the distributed tendons in the latitude. Running the strong tendons in this direction proved to be next to impossible. First, the tendon along column line C was too long of a distance to make the section work (see next page). It was impossible to trend the tendon to the right of the opening to the two columns indicated by the arrows due to the stairwell in between them, so two tendons (out of plane of the latitude direction) were laid out span from one column to the other with the low point of the tendon underneath the low point of column line C in an attempt to help support this section of the slab. After extending 15 strands at both of these locations, the slab continued to fail. Any more strands at these points and the slab would have been compressively stressed to the maximum resulting in failure again. Second, many of the longitudinal tendons take too steep of directional changes making it less effective and constructible. It is ideal the tendon stay perfectly straight to properly jack the tendons to their necessary stresses. Third, the distributed tendons in the latitude direction are spread out evenly but some of the spans were too long to work under service loads; also, the latitude tendons were unable to be design to effective following the curve of the building.

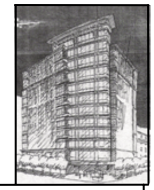
The advantage of constructing this layout was the discovery that a post tension is ideal for the Executive Tower's unique column layout and necessary in cutting the slab thickness. Also shown below is the deflection plan with this post tensioning layout on page 28. Even though some spans failed and were unable to be constructed to pass, most of the floor plan was acceptable and the largest deflection was 1.01 inches on a 37 foot span calculating a deflection ratio of L/439.

Due to the orientation of the slab openings and the column layout it was decided to try running the tendons in the opposite directions. By doing this, the longitude tendons (now the distributed tendons) can be stopped at the elevator cores leaving the slab in the corridor without post tensioning.



A
B
C
D





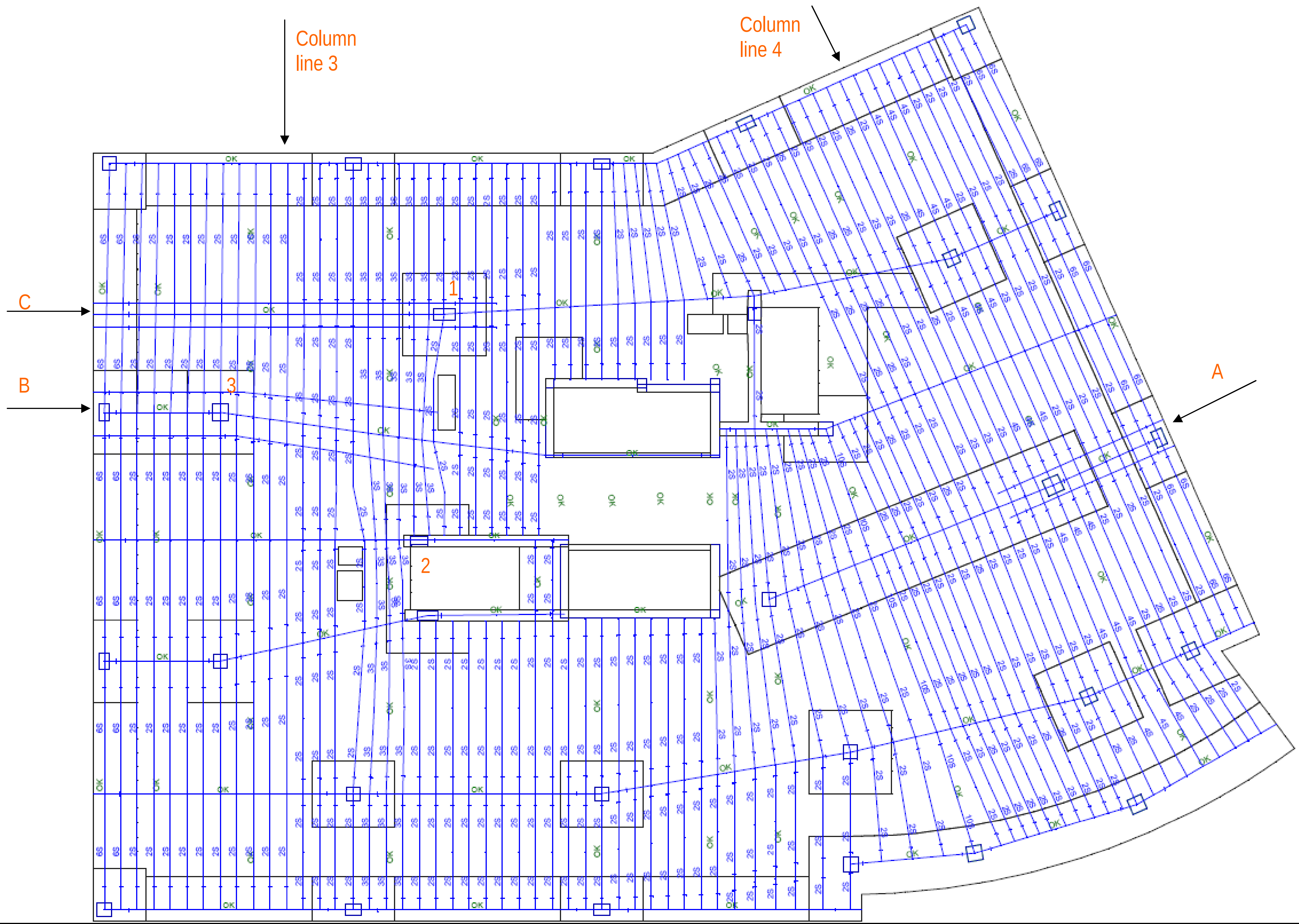
TRIAL 2

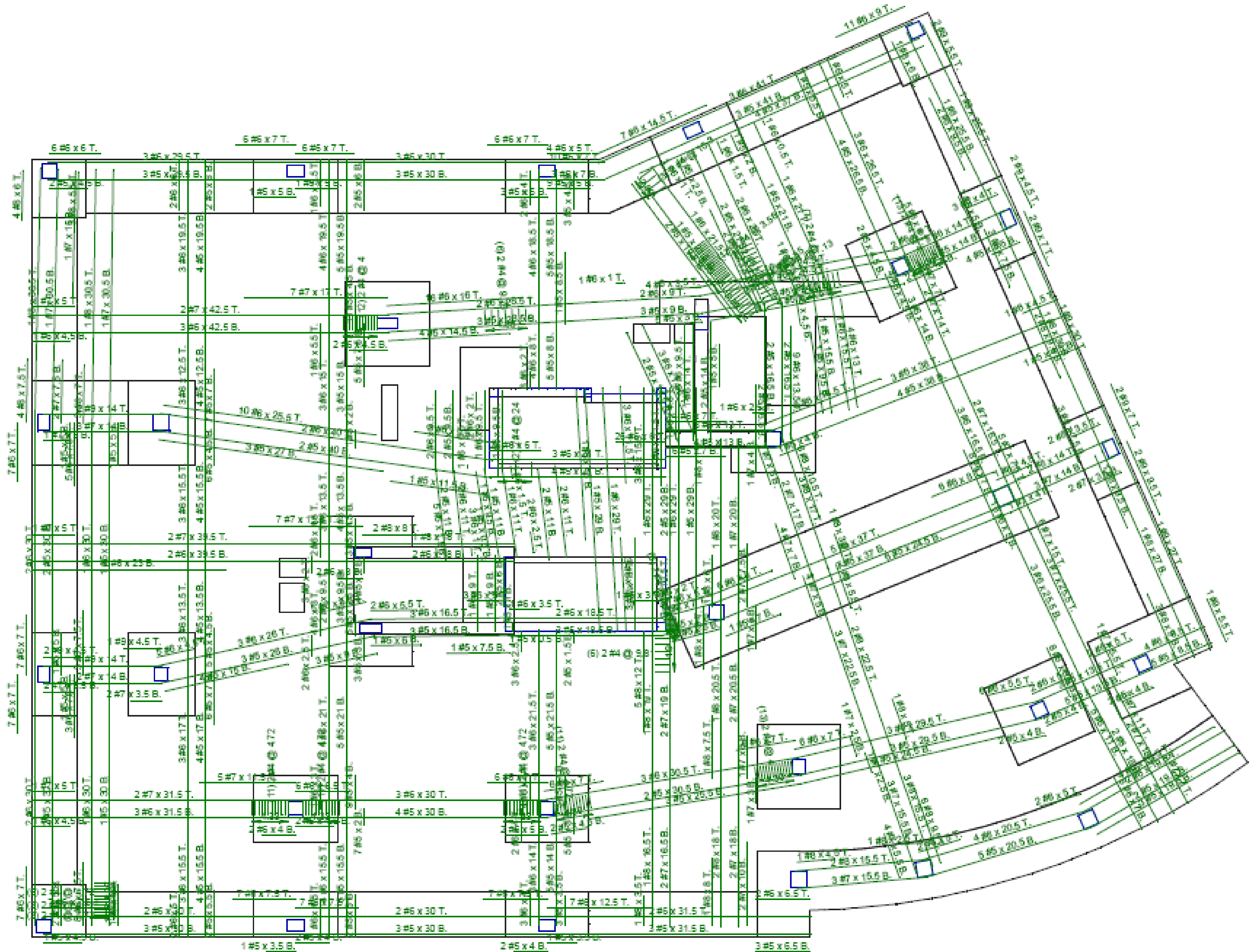
In trial two, the tendons were rotated 90 degrees to attempt to create shorter and straighter column strip spans, a tendon free corridor and enforce a deflection criterion of L/360 or better. With the exception of a few spans that needed a creative design solution, the trial two created a significantly better layout than that of trial one. The Trial two plan is on page 30.

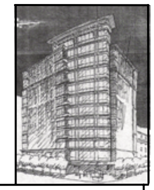
Trial two is a more realistic construction plan compared to trial one. The strong tendons run in the latitudinal direction which has few turns and produces natural breaks in the building structure to anchor tendons. Only four latitude tendons stretch the entire length of the building. The remaining five are anchored along the right side of the two elevator cores. This creates a smoother transition in designing for the 24 degree skew the building plan takes in the middle of the floor plan and allows the use of fewer tendons in slabs that do not required large stress to be sufficiently supported. In trial two by spanning the strong tendons in the latitudinal direction, the strong tendons are now in line with several beams in the Executive Tower floor plan making it ideal for these beams to support the distributed tendons in the other directions. The beams at the stairwells are great places to stop distributed tendons. Most of the MEP openings in the slab are oriented parallel to the distributed tendons. Having these openings in the same direction makes it easier to spread tendons to still support the slab without disrupting the MEP duct work.

In the process of laying out the column strips, it was assumed the edge beams around the perimeter would act compositely with the slab creating a tee beam. Also due to the Executive Tower's column arrangement, when designing the column strips for the distributed direction (longitude) it was assumed the columns strips along column lines three and four would act as equivalent frames. The column strips were drawn perfectly straight stopping at each strong tendon that runs the in latitude direction to insure the slab is checked at each span of the distributed tendons.

A few disadvantages are places in the slab where even with substantial post tensioning and reinforcement would still fail. These areas are discussed further in the design section.


Column line 3
Column line 4
C
B
A
1
2
3





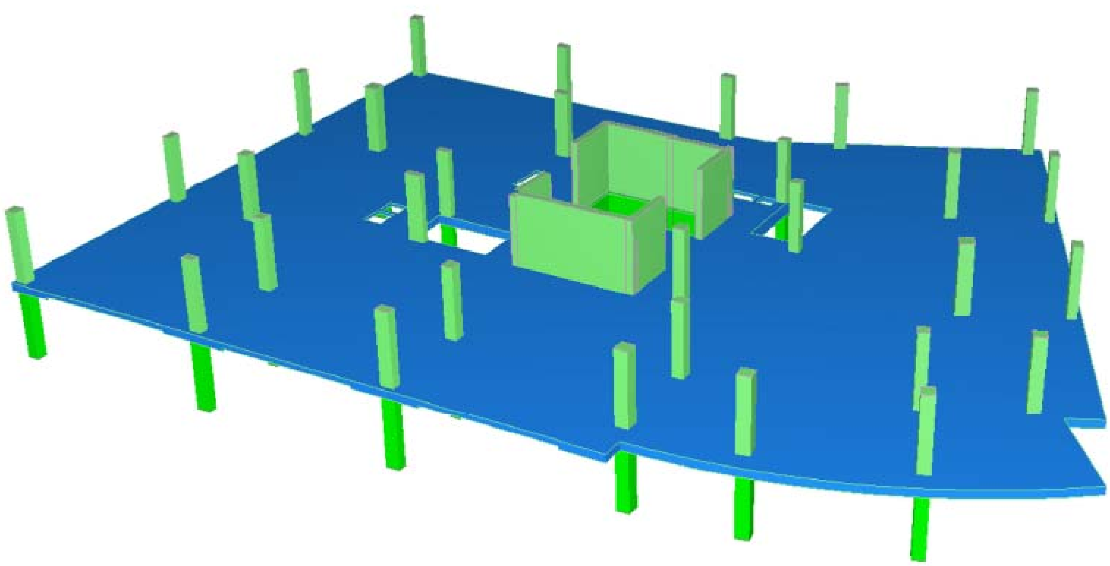
DESIGN

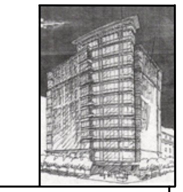
After designing the second post tension plan in RAM, it was found that the initial goal of reducing the slab to five inches was too aggressive. With the thickness reduced this much, the slab still continuously failed in similar locations as trial one. It was decided to only reduce the slab thickness by two inches. This however, does not sway opinion of using trial two over trial one. Trial two still proves to be the more suitable design solution for the Executive Tower.

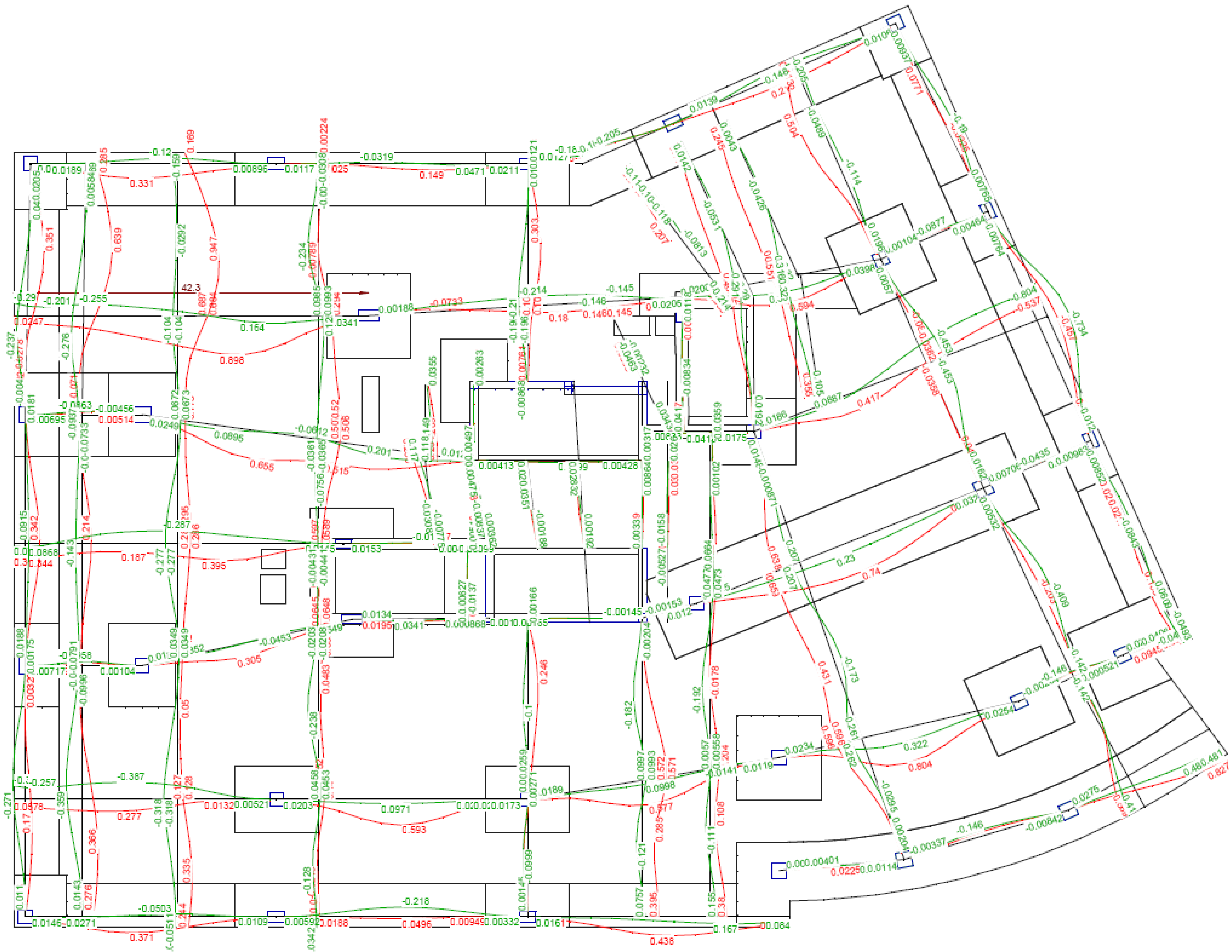
Three areas initially caused problems in the design phase in the RAM Concept model. These areas are marked by the arrows on the previous page (30). Section A is a 10 foot span at the end of a 37 foot span. Along the 37 foot span is an eight inch drop panel to help control the deflection in this area. Without tendons in this section, the 37 foot span would deflect up to 0.98” causing the 14 foot span to have an upwards deflection of 0.3”. Due to the large deflection over a short distance, the slab was cracking in both tension and compression at the edge of the drop beam. The first design solution was to add more tendons at this area to help carry the loads. However, after extending 27 tendons, the slab would begin to reach its pre-compressive limit and would fail. As a result of this, the main tendon was cut down to nine strands and set at its maximum uplift balancing load for the 37 foot span and inverted over the 14 foot span developing a downward balancing load. This caused a combination of uplift for the 37 foot span and a downward loading for the 14 foot span resulting in an improved deflection over the 14 foot span however still failing. Six strands were then run over the 14 foot span and anchored just after the column to increase the downward load in this area. The results were verified by the deflection plan now show only -0.74 and +0.044 which has a control deflection of L/600 between the two of them.

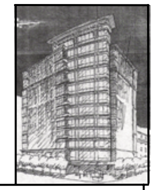
A similar area of failure occurred at section B indicated by the arrow on the previous page. This area was deflecting too much from the long span of 40’ compared to the short span of 14’. Similarly, the main tendon was reduced to 10 strands and two four strand tendons were placed on either side creating uplift in the long span and downward load in the short span. The result improved the short span but still failed, plus the reduction of tendons in the long span was now causing flexural failure. To fix the short span, the slab was increased in thickness equivalent to the edge of 9 ¾". The new section passed and stiffened the connection of the column and long causing it the long span to deflect less, but still fail in flexural. A creative solution to this involved revising the distributed tendons in the longitudinal direction. Fifteen strands spread evenly at one foot spacing were altered to span from column 1 to column 2 instead of resting on the main tendon in the 40 foot span. The result of this is an uplifting point load at these crossing tendons equivalent to their balancing load times the width of the 40 foot span column strip which is 13.5'.

Section C was failing in deflection as a result of a 44 foot span. The conclusion was to apply the same solution of section B and have the distributed tendons span from the edge beam to column 3. The result for both sections was a deflection limit of L/732 and L/587, respectively.









PUNCHING SHEAR

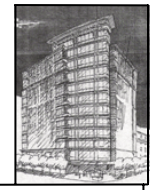
Punching shear in the Executive Tower was found to be the controlling factor in determining the size of columns. The punching shear equation for a prestressed concrete was used from ACI 318-05 11.12.2.2, without being in excessive of 11.12.3.1 (both shown below).

$$V_c = (\beta_p (f'c)^{(1/2)} + 0.3 f_{pc}) * b_o * d + V_p \qquad 11.12.2.2$$

$$2 * (f'c)^{(1/2)} * b_o * d \qquad 11.12.3.1$$

The results from this spreadsheet can be found in Appendix E, but three columns are shown below and discussed. In the existing structure, shear reinforcement was not necessary since at every column location had 16" of concrete due to drop panels. Punch shear was checked however to determine if this holds true for 14" of concrete. In all but three columns, punch shear passed without the use of steel reinforcement. Columns 1, 8 and 24 were test without steel reinforcement and failed mostly by only a few kips. The formula was then calculated again this time factoring in #4 bars at six inch spacings, which was found to be acceptable.

	Size (in x in)	d (in)	b_o (in)	f_{pc} (psi)	f'c (psi)	α_s	β_p	Vc (lb)	ØVc (lb)	Vu (lb)	check?	Vs w/ #4@6'	new ØVc (lb)	check?
1	20x20	14	58	260	4000	20	3.5	102711	77033	86300	no good	24000	95033.1	OK
8	20x20	14	136	225	4000	40	3.5	240839	180629	181000	no good	24000	198629.3	OK
24	24x24	14	152	200	4000	40	3.5	269173	201880	207000	no good	24000	219879.8	OK



LATERAL DESIGN

The shear walls were developed using the same method from Technical Report 3. Six shear walls are located enclosing the elevator core and five frames lining the perimeter of the building due to the thickened slab acting as a perimeter beams. The frames were modeled in STAAD with 100 kips point loads at each floor to find the relative stiffnesses. One hundred kips virtual loads were used instead of one to get a deflection off of STAAD with two more significant figures. The shear wall stiffnesses were found through the following equation:

$$R= Et/(4*(h/L)^3+3*(h/L))$$

Through an excel spreadsheet the shear walls and frames were all simultaneously calculated for direct shear and torsion. These loads were calculated for each floor. The loads per floor per element were then divided by the relative stiffness for those points to find the story drift and building drift. By designing this way, it is assumed the frames and shear walls will be taking all of the lateral loads, and as a result, the concrete strength for the shear walls needed to be increased to have a building deflection of less than the L/400 limit. In reality, the slab and all the columns would contribute to resisting the lateral loads which is why the shear walls on the original plan were sized smaller.

POST TENSION CONCLUSIONS

It has been found that converting to a post tensioned floor system was the correct process in order to meet the proposal. However, to much disappointment, reducing to a five inch slab proved inadequate to support the floor in flexure or deflections. Punch shear was not checked for a five inch slab, just a six inch slab, but by observation many more of the column in Appendix E were within a few kips of failure. Had the slab been kept at five slabs, punch shear would be become a reoccurring problem in several columns. As for the slab itself, accept in the areas discussed above the slab was sufficiently supported with one strand per foot distributed tendons in the longitudinal direction and strong tendon in the latitudinal direction mark on the tendon layout on page 30.