Analysis 3: Structural Analysis (Breadth)

Issue 3:

The new green roof that is to be implemented will add an additional dead load to the roof. A structural analysis must be made to be sure the roof can handle the new load, or if more reinforcement or concrete is needed. The original analysis was to see if steel would have been a better choice for the superstructure than cast-in-place concrete. After speaking to some advisors and having the 3D model unusable, a new analysis was put together. Also further investigation is to be put into the perimeter beams and post tension girders, because of the new structural loads to the building.

Analysis:

The original proposal was to be: By using steel instead of cast-in-place concrete more of the steel pieces will be able to be fabricated before brought to the site; this will ensure a faster erection time. After changing the building envelope, the building will have a lesser load on the entire structure. With that information, smaller columns and beams can be used to support the entire building. This can affect the size of the crane, which in turn can save money. Research will be put into the cost of steel erection compared to the cast-in-place concrete used. BIM was going to be incorporated in the structural design of the steel frame in order to show BIM's value engineering and work sequencing abilities.

Expectations:

Original expectations were: After constructing a BIM superstructure of Plaza East, the software will be used for take offs and work sequencing to show increases in savings. The money and time saved on the steel erection will be put to other aspects of the building. New expectations are to find out how much more rebar is to be placed in the roof slab to hold the new dead load of the green roof.

Outcome:

When trying to create a 3D model, my analysis has changed. I no longer had a chance to use my 3D model to test steel erection over concrete, because the analysis was too big for a breadth. I also spoke with Dale Kopnitsky from Gilbane about the idea of using steel in the Chantilly, VA area. Mr. Kopnitsky basically told me concrete was the material for the job. He explained this to me with the following information. Some believe steel could possibly be better than concrete because of the erection time, but I was told different. With steel, the skeleton would be up quicker but you would still have to fire proof and poor the slab before MEP, which ends up having the MEP for concrete and steel starting around the same time. The fire proofing spray also ends up being about $2.25/ft^2, concrete does not use any. With steel, the deeper members end up having bigger floor heights, further more increasing costs. You would need to add at least a foot per floor, which would add about 5 feet to the exterior of the building, increasing the price of the building envelope, interior partitions, and piping. Also using concrete you tend to get an extra floor, than when using steel. This is nice for the DC area where no buildings can be taller than the capital. Mr. Kopnitsky went on saying the deflection is greater in the slab, even if the slab is cambered. This would then have the need for a self leveling agent. Mr. Kopnitsky said another advantage is that all engineers are trained to use concrete frames.

The new structural breadth will be to ensure the green roof, that is to be applied to the building, will be able to be held up by the original roof or if additional reinforcement or concrete is needed. I've come to two separate types of green roofs and plan on doing a calculation on an extensive sedum and herbs (3" think) or sedum, herbs, and perennials (5" thick). Each will be using drainage plates instead of granular drainage or drainage mats. A slab calculation will be done to see if the current 5 ½" slab will hold or if there is a need to have more reinforcement and if the slab needed to be increased up to the 7" slab equal to the rest of the floors or if .

Plans to contact the structural engineer of Plaza East have been made to further investigate what should or could be done on the perimeter concrete beams and the post tension girders to see what actions are usually taken when the building envelope load is reduced and the roof load is increased. No help was given from the structural engineer from Plaza East, but a few general suggestions were given to me from Richard Apple, P. E. and Vice President of Holbert Apple Associates, Inc.

After seeing the post tension girders schedule I developed a few questions for Mr. Apple. He is not completely clear with the building design so his answers were general for all post tensioning buildings. Looking at the schedule I saw Force (Kips) and a tendon profile section with A, B, and C positions on it (**Fig 18**). Mr. Apple explained the force is usually given as the total effective post tension force that you want in the tendons after all losses. This can be equated to a total number of tendons by dividing by approximately 26.5k per tendon. For the two beams my slab calculations go across, this seems reasonable. My exterior post tension girder has 110 kips. Divided by 26.5 kips, this gives me 4.15 tendons, and the next interior girder has 530 kips divided by 26.5 kips which gives me exactly 20 tendons. The exterior also has rebar stated in the drawings to hold the extra 0.15 tendons needed. The tendon profiles A, B, and C was explained as positions measured from either the top or bottom of the post tension girder. Profile A can be the end of the member, which usually has the tendons anchored at the center of gravity of the member cross section. Profile B could be the low point of the tendons at the mid-span and profile C could be the tendon high point at the supports.

BUILDING 2

MARK	WIDTH	DEPTH	FORCE (KIPS)	TENDON PROFILE			
				A	B	C	L.E.
PTTB1	24	12	110	7 1/8	3 1/2	9 7/8	36
PTTB2	24	12	110	9 7/8	4	7 1/8	---
PTTB3	24	12	110	7 1/8	4 1/2	9 7/8	36
PTTB4	24	12	110	9 7/8	5	9 7/8	---
PTTB5	24	12	110	9 7/8	3 1/2	7 1/8	---
PTTB6	24	12	110	9 7/8	5 1/2	9 7/8	---
PTTB7	48	19	500	11 7/8	2 1/8	16 7/8	87
PTTB8	48	19	500	16 7/8	12 1/2	16 7/8	97
PTTB9	48	19	500	16 7/8	2 1/8	11 7/8	---
PTTB10	48	19	530	11 7/8	2 1/8	16 7/8	97
PTTB11	48	19	530	16 7/8	12 1/2	16 7/8	107
PTTB12	48	19	530	16 7/8	2 1/8	11 7/8	---

Fig 18

Inquiring about how to increase the capacity for the green roof load, I was told to analyze the beams for their capacity as they are scheduled, so that I can determine the tension and compression stresses at the critical locations (midspan, at supports, etc.). The beam size may or may not be able to accommodate more post tensioning force to get additional capacity. The beam could end up having too high of compression forces for the cross section available, so an increase in either the depth of the beams, or an additional width may help if the net increase in loads are not very significant overall. Not much information was given about the perimeter beams and the new building envelope loads.

Typed out hand calculations for the roof slab is presented below:

Roof Loads

Live Load: 35 psf

Dead Loads: (150 pcf)(5 ½”/12”per ft) = 68.75 psf

Green Roof 37 psf

Snow Load: 27 psf

Total Load: 1.2(68.75+37) + 1.6(35) + 27 = 209.9 => 210psf

210 psf/1ft = 210 plf

Moments

At Exterior Support: -M = 1/12 (210) $(20')^2$ = 7,000 ft-lb

At Mid-Span 1: +M = 1/14 (210) $(20)^2$ = 6,000 ft-lb

At Interior Support: -M = 1/12 (210) $(20')^2$ = 7,000 ft-lb

At Mid-Span 2: +M = 1/16 (210) $(20)^2$ = 6,250 ft-lb

Area of Steel (A_s)

d = 5.5" – 0.75" = 4.75"

a = [(As)(f_y)] / [0.85(f'c)(b)]

Using a=1"

A_s = M_U / [ϕ(f_y)(d – a/2)] = [(7 kips)(12)] / [.9(60)(4.75 – 1/2)] = 0.366 in^2

Check a

a = [0.366(60)] / [0.85(3.5)(12)] = 0.615"

A_s at Exterior Support:

A_s = [(7 kips)(12)] / [.9(60)(4.75 – 0.615/2)] = 0.35 in^2

A_s at Mid Span 1:

A_s = [(6 kips)(12)] / [.9(60)(4.75 – 0.615/2)] = 0.30 in^2

A_s at Mid Span 2:

A_s = [(5.25 kips)(12)] / [.9(60)(4.75 – 0.615/2)] = 0.26 in^2

Minimal Reinforcement for Shrinkage & Temperature Cracking

A_s = (0.0018)(12")(5.5") = 0.12 in^2 < 0.35, 0.30, 0.26 in^2 GOOD

Minimal Reinforcement for Shrinkage & Temperature Cracking in Perpendicular Direction

Use #3 @ 10" = 0.13 in^2, 10" < Five times slab thickness & less than 18" GOOD

Existing Reinforcement: #4 @ 15"

A_s = #4 @ 15" = 0.20in^2(12"/15") = 0.16 in^2 < 0.35 in^2 NOT GOOD

New Reinforcement

For Ext. Support use: #4 @ 6" = 0.40 in^2 or #5 @ 10" = 0.37 in^2

For Mid Span 1 use: #4 @ 7 ½" = 0.31 in^2 or #5 @ 12" = 0.31 in^2

For Int. Support use same as Ext Support

For Mid Span 2 use: #3 @ 5" = 0.26 in^2 or #4 @ 9" = 0.26 in^2

Factor Shear @ d

$V_U = (1.15)(210)(20)/2 – (210)(4.75/2) = 2{,}332$ lb

$V_n = V_c = 2(f'c)^{0.5}(b)(d) = 2(3{,}500)^{0.5}(12)(4.75) = 6{,}744.33$ lb

$\phi V_c = 0.75(6{,}744.33) = 4{,}721$ lb > $V_U = 2{,}332$ lb GOOD

For hand calculations of the roof slab see Appendix D

Conclusion:

Using the new upgraded area of steel will let the roof slab hold the load. The cost will increase but only slightly. This extra cost is found in the next section with the conclusion to all three analyses.

Conclusions

Each analysis has had its own individual conclusions stated in each section, but when combining the conclusions you can see that the project can save money with the new façade in order to add a green roof. The energy and cost savings can be minor, but every little bit helps. With these new materials the building is much more environmentally safe and the savings shown in **Table 5**, proves a green roof is feasible and the project still saves money.

I've also decided to add a **very rough estimate** of how much more it would cost to add the rebar. I used the existing building value of #4 @ 15", 0.668 lb/ft and compared it to the biggest difference in steel area from the hand calculations #5 @ 10", 1.043 lb/ft. Using these numbers over the area of the entire roof, with a length of 214.5' and width of 121.5' is not