Milton S. Hershey Medical Center Biomedical Research Building
Hershey, Pennsylvania

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Executive Summary

As stated in the proposal, 3 additional stories are to be added to the top of the Biomedical Research Building, the top most to be of double height, with 22’ from floor to ceiling, and make this addition a viable option through HVAC requirements, lighting design and acoustical management. Also to be explored are the effects of the 3 additional stories, with a total height of about 50’, on the existing structure, and testing the hypothesis that the columns currently existing in the building are indeed designed for additional stories should the need arise in the future.

After designing a viable 3 story expansion based on the existing structure for compatibility, the existing structure was analyzed for any exceeded design values, and after reiteration, was found that the existing structure is sufficient for the additional 3 stories, both under gravity and lateral loads. After this, it was simply a matter of creating a usable space for HVAC, lighting and acoustics, to provide students with plenty of work space to do research and recreation without disruption.

HVAC was found to require a system that provides an additional 86000 CFM and approximately 5.5 million BTU/HR for both heating and cooling, with improved insulation and special glazing. Lighting requirements for the 500 lux recommended, necessitated the use of 200 T8 58W flourescent luminaries over 18 21’ by 35’ bays in addition to indirect lighting for the ceiling to be illuminated yet not create shadows with the mid height bracing beams. Acoustic management found the balance between noise reduction and prevention of creating an uncomfortably quiet space, as well as prevention of echoes. Cost was also computed to compare the addition to the cost of the existing structure, showing the return is just as good, if not better, should inflation be factored in, per story.

Existing Building Summary

The Milton S. Hershey Medical Center Biomedical Research Building in Hershey, Pennsylvania, is an education and research facility. It is owned by the Milton S. Hershey Medical Center, and is part of Penn State Hershey, and thus is a branch campus of Pennsylvania State University. It is a 110’ tall structure with 7 stories and 245000 total square feet of floor space. It was constructed by Alexander Building and Shoemaker Construction Companies and managed by Alvin H. Butz, Inc. between 1991 and 1993, costing $49 million. It was designed by Geddes Brecher Qualls Cunningham, and engineered by The Sigel Group and Earl Walls Associates. The most distinguishing architectural aspect of the building is a large cylinder that extends from the 2nd floor up to the roof on one of the corners of the building.

Foundation System

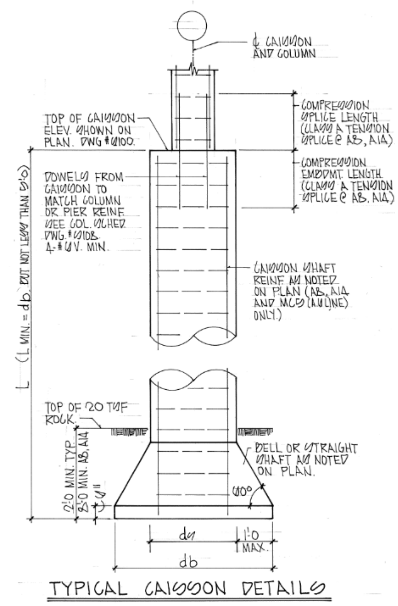


Figure 1. Typical Caisson Detail

The Biomedical Research Building at Penn State Hershey utilizes a simple monolithic concrete structure to serve its load distribution needs. This structure stands on a series of large, 3 to 7 and a half foot diameter caissons which loads ranging from 250 kips to 1610 kips, with most loads around 1000 kips expected by the building's original engineers. These caissons have a 40 kip per square foot requirement, using 3000 psi 28 day strength concrete, and are set into the bedrock below. It should be noted that even though 3000 psi concrete was called for, there was an instance where 1000 psi concrete was called for in the plans. A variety of different sized 60ksi steel rebar are utilized in reinforcing both the caissons and the grade beams, with clear cover at 2.5 inches, given its exposure to ground.

Caissons were chosen as the building's foundation, as the area is known to have large sink holes develop within the limestone deposits. This prevents future sinkhole development underneath or nearby to have any drastic effect on the Biomedical Research Building's safety, especially as sinkholes are not usually detected until it is too late. As seen in figure 2, grade beams act to transfer forces from the columns into the caissons when columns and caissons do not line up, and to further the idea of sink hole damage prevention, using beams varying from 14 inches wide by 30 inches deep to 7 feet by 16 foot 8 inches deep.

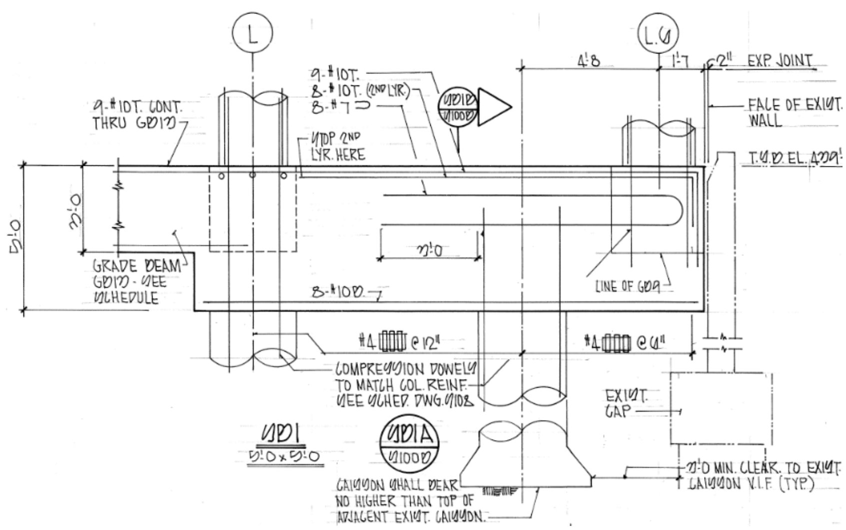


Figure 2. Example of caisson and column misalignment

General Floor Framing

Floors of the Biomedical Research building are supported by large beams typically spanning 20' that predominately go in the longitudinal direction of the building for the central part, and in the far ends of the building. These beams vary from 12 to 36 inches deep, and 3 to 8 feet wide. There obviously were some depth restrictions where the 8 foot wide beams are located. Shown in Figure 3 on the next page, the building is effectively cut into 3 sections by two set of three openings in the floors, with columns and beams on all sides of these openings. These openings are to serve the building in its HVAC, plumbing and electrical needs. Additional openings in the floor are directly adjacent to these service openings, for elevator shafts that serve the entirety of the building. These elevator shafts have two additional columns to help support the concentrated load of the elevator and its machinery, distributing the load around the openings.

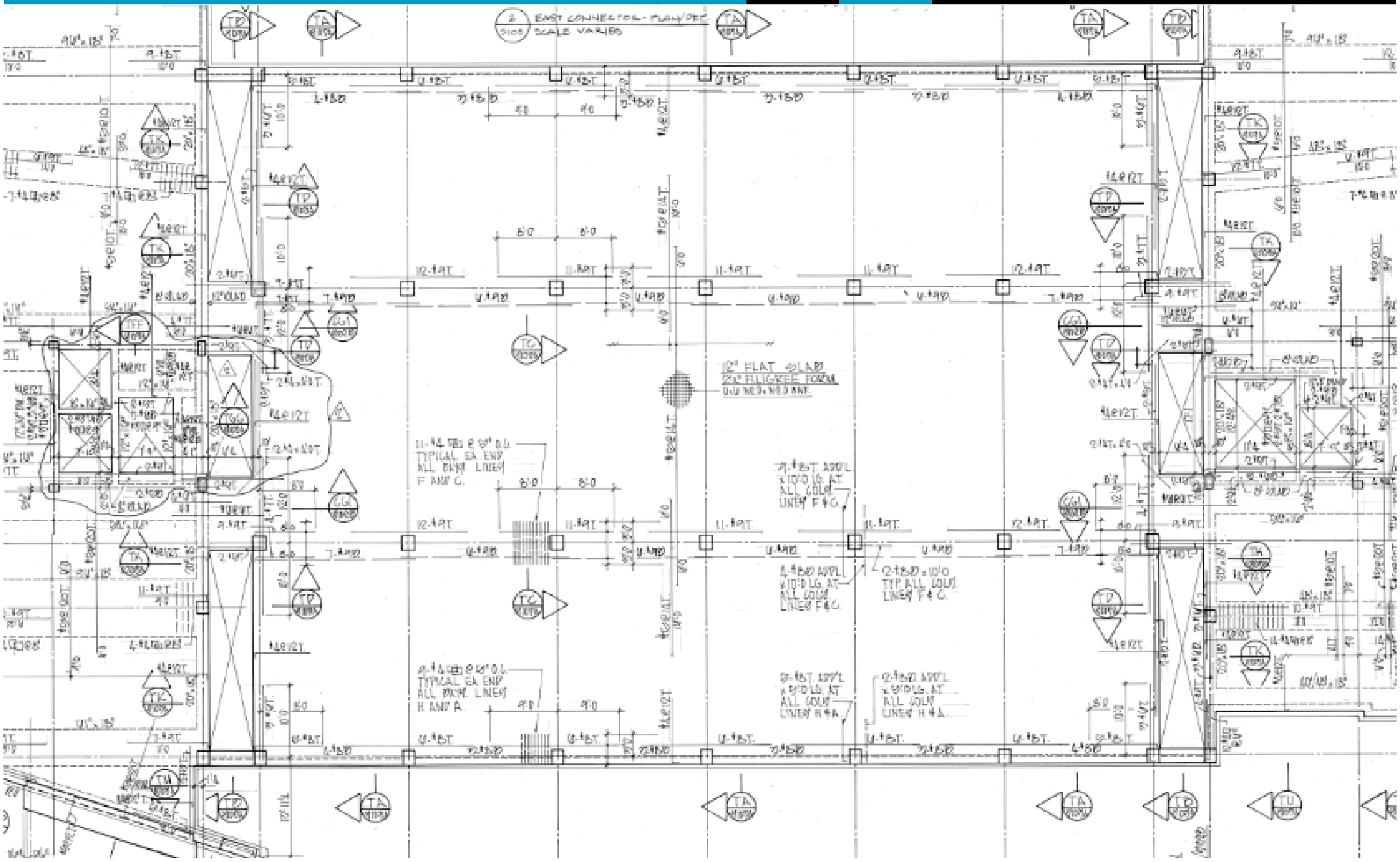


Figure 3. Typical Floor Plan - The three vertical openings on each side are for HVAC, electrical, and mechanical usage, and the openings just to the outside of these openings are elevator shafts.

Beams use rebar at the top and bottom of the beam to resist positive and negative moments, and such reinforcement is usually discontinued at some point after development length has been achieved. Shear reinforcement is used in the form of stirrups, using #3 or #4 sized rebar with 40ksi steel. There are no drop panels used, and as found in the calculations on page 30 in the Appendix, the building would benefit from drop panels.

Supporting the beams are a multitude of columns, averaging about 2 feet by 2 feet in dimension. Circular columns are also used, and average about 30 inches in diameter. 60ksi rebar are used to reinforce the columns, with varied sizes and number of rebar utilized. Clear cover for the columns and beams inside of the building is at 1.5 inches.

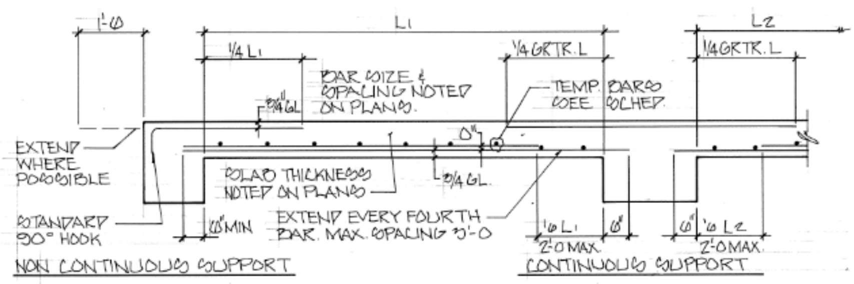


Figure 4. Typical Slab Detail

Floor Systems

On these beams are a system of one way slabs designed to support 100 to 125 psf floor loads, using 4000 psi 28 day strength concrete, with temperature reinforcement and a 6x6 W2.0xW2.0 WWF. The one way slabs are oriented perpendicular to the beams, and are treated as beams in that direction. On the ground level, where large mechanical equipment is located, slabs are thickened according to the size and weight of the machinery, as applicable.

Expansion joints

There are no expansion joints, but there is temperature reinforcement to handle the stresses of expansion and contraction of the building. In addition, there are also control joints that are designed to mitigate and control potential cracking in the building, which would include crack development due to temperature change. A typical control joint detail is shown below.

TEMPERATURE BARS	
SLAB THK.	REINF.
4" LESS THAN 5"	#3 @ 12"
5" " 6"	#4 @ 18"
6" " 7"	#4 @ 15"
7" " 8"	#4 @ 15"
8" " 9"	#4 @ 11"
9" " 10"	#4 @ 10"

Figure 5. Temperature Reinforcement Schedule

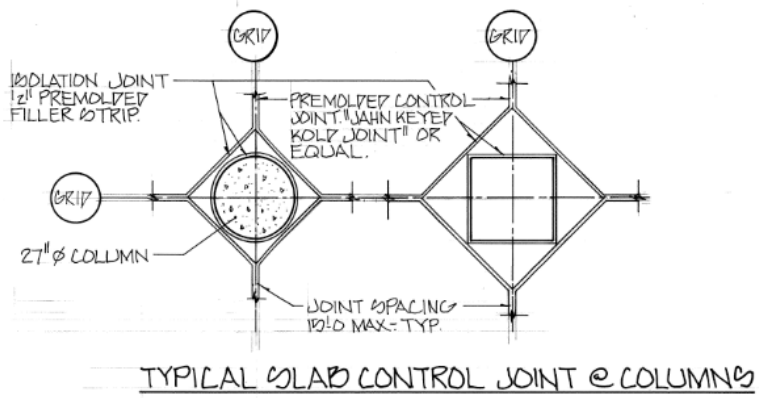


Figure 6. Typical Control Joint Detail

Roof system

Elevator machinery and miscellaneous other HVAC machinery is stationed on the roof, as typical. These must be supported in addition to snow loads, and were designed also to manage rain water, diverting it to drainage pipes on the roof. There are parapets of varying heights also located on the roof, preventing water run off on the sides of the building. The 8 inch thick roof is sloped slightly to aid in rain water management, preventing it from pooling, and potentially causing a collapse. Calculations on page # in Appendix # for snow loads show that the design load of 30 psf is in excess of the 21 psf snow load that would accumulate on the roof should snow drifts come into play during winter months.

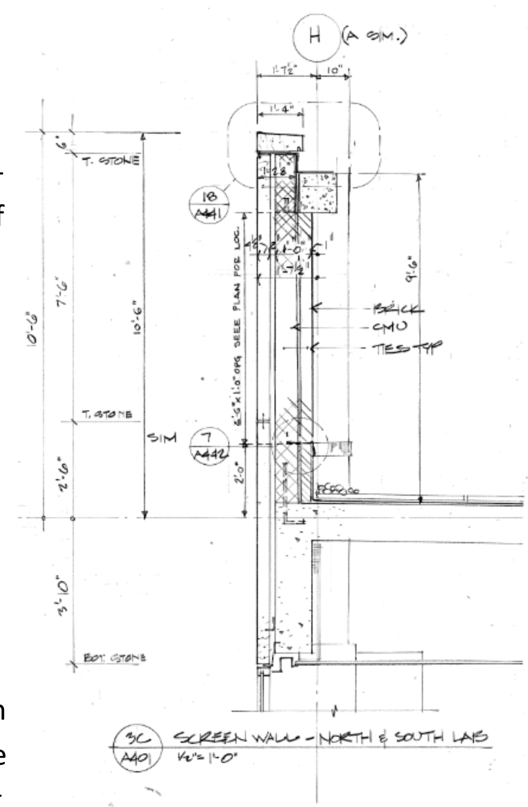


Figure 7. Example Section of a Parapet.

Secondary Structural System for Mechanical Equipment

As mentioned before, for the ground level, slabs are thickened for the additional weight, and elevator equipment has its own columns around the elevator shaft to handle both the weight of the machinery, the elevator carriage, and the people that may be using the elevator at any given time.

Support of Curtain Walls

Curtain walls and cladding for this building consist of limestone, granite and glass panels. These are often anchored directly into the concrete structure where they are applied. Two inches of clearing between the panel and the building are in place to insure that moisture has a way to weep and not accumulate behind the panel. Slabs have beams or some other support at the edge of their spans of varying depths and widths to support additional weight where panels are installed.

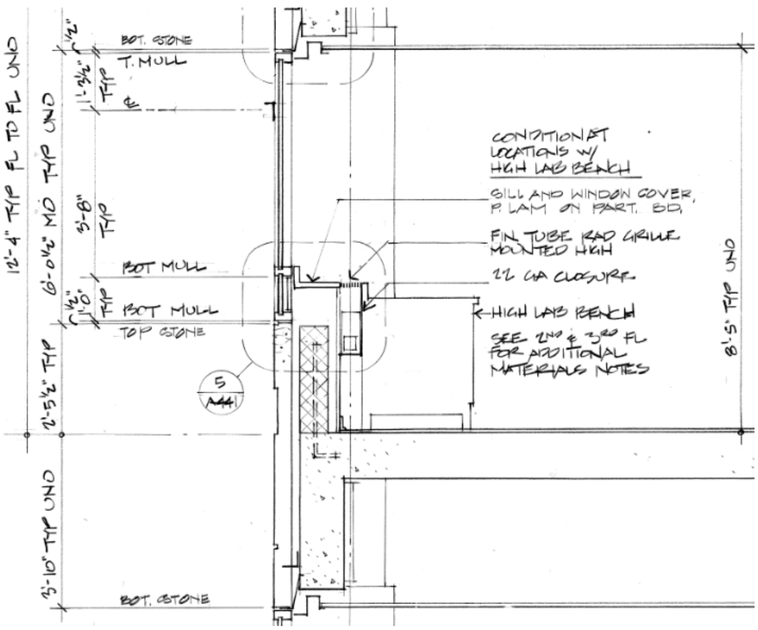


Figure 8. Example Section of Curtain Wall

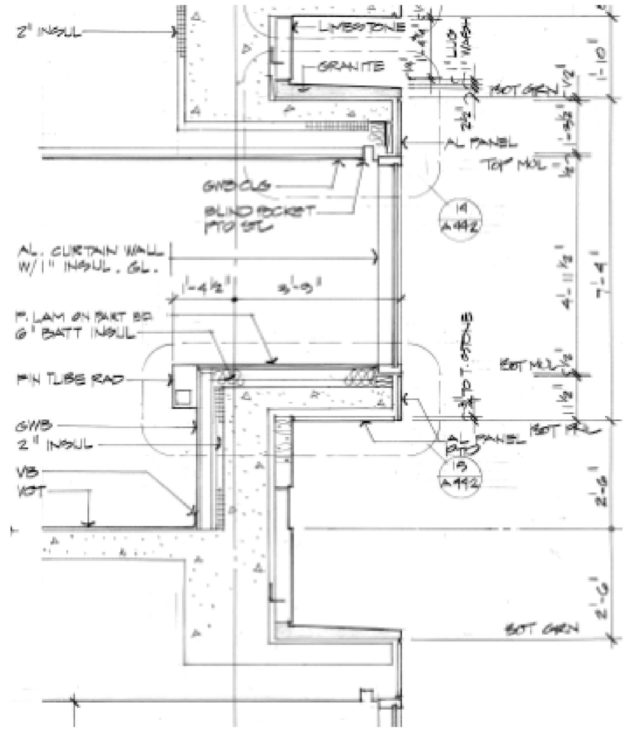


Figure 9. Example Section of Exterior Cladding

Support of Architectural Cylinder on Corner of Building

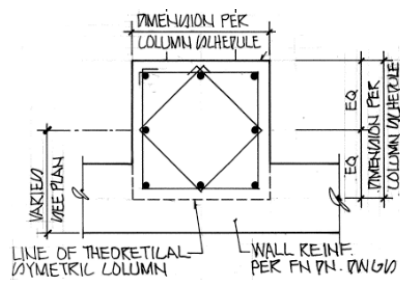


Figure 10. Illustration of Column Used for Support of Architectural Cylinder

There is an architectural cylinder on the corner of the building that is supported by 4 - 33” by 33”columns reinforced with 8 #11’s as in Figure 10. The column is 125% larger than the columns above it, possibly from a safety standpoint. From the 2^{nd} floor to the roof, the slabs on the interior support its glass, granite and limestone facade, and on the other face, a solid wall supports additional aesthetic wall panels along the stairwell, as seen in a section in Figure 11.

Lateral system

Wind plays a large factor in the surrounding buildings, especially the Crescent, the main hospital building of the Hershey Medical Center. Its long and unique shape plays a direct role in sheltering the Biomedical Research Building from direct wind, as well as other surrounding buildings in the area. As for the Biomedical Research building, it has an oblong shape, making wind forces to be manageable in one direction by a smaller area for wind to push up, and a large structure to resist this wind load, but leaves a larger area to resist a larger wind load. Wind forces are directly resisted by the curtain on the building, and forces are then transferred to the 8”-12” thick concrete slabs. Slabs then transfers the load into the columns and shear walls, and eventually down into the ground, through the caissons. For the short side of the building, there are large concrete beams that would play a strong role in resist wind forces.

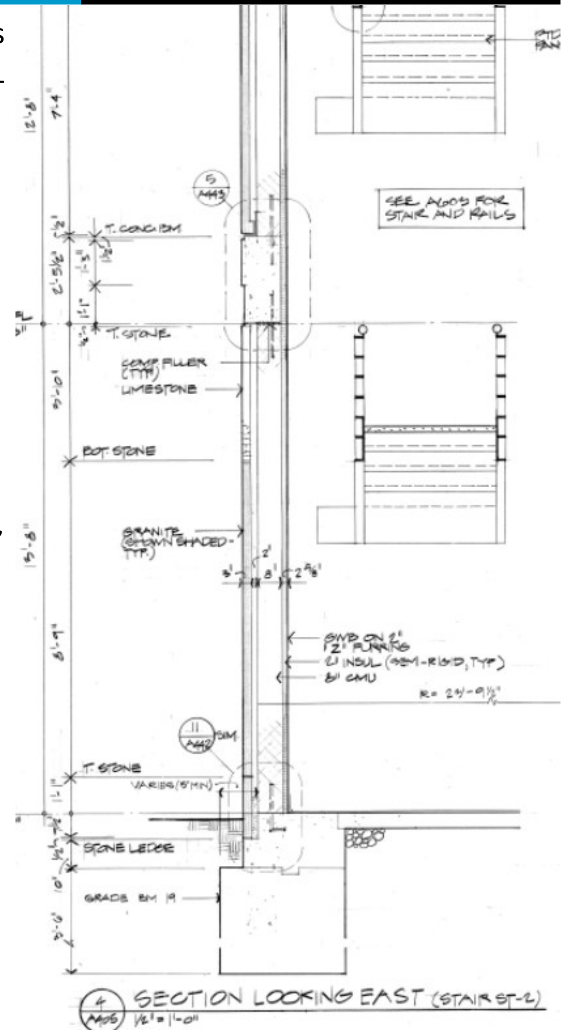


Figure 11. Section of Stairwell

Overall Interaction of Systems

Ultimately, all existing systems rely heavily on the largely straightforward concrete structure, with lateral forces, going through the curtain walls, and most live and gravity loads behind handled by the floor slabs. The one way slabs transfer the loads to the beams and shear walls, and subsequently into various columns, which also support equipment loads and resulting roof loads. Excessive cracking in the slabs are controlled by control joints, temperature reinforcement maintains the effectiveness of the slabs under various temperature related stresses. Large grade beams then take the loads from the columns, as well as the thickened ground slab, supporting various heavy machinery, and redistribute the loads to the caissons below.

Design Codes

The original codes used by the original plans were BOCA, 1987 Edition, ACI 318-83, AISC, 1980 Edition, A. W. S. D1.1, 1986 or 1988 Edition and CRSI, 1986 edition. This technical report uses ACI 318-08, and ASCE-05 for its reference calculations.

Typical Materials Used

Typical materials that were utilized were varying strengths of concrete. Those specifically specified in the typical details were 4000-5000 psi 28 day strength concrete, with most concrete being 4000 psi strength, while further investigation into the plans revealed at least one call for 1000 psi concrete for use in caissons. Reinforcing steel bars for #4-#11 sizes were to adhere to ASTM A615-60, and stirrups being #3 and #4 were to be of grade 40 steel. For the one way slabs, unless 6x6-w2.0xw2.0 WWF was called for, 6x6-w2.9xw2.9 WWF was the typical wire mesh used.

Gravity Loads

Gravity loads were a combination of dead, live, and superimposed loads. Dead loads were calculated based on existing slab thicknesses and a 150 pcf concrete density. Live loads from plans were used, 125 psf for laboratories, and 100 psf for everywhere else, but for simplicity’s sake, 125 psf was used for all locations except the roof. A 30 psf roof load was used for a guideline for calculated snow drift loads. Lastly, a 15 psf superimposed dead load was included for miscellaneous lighting, electrical, HVAC, and plumping fixtures that may have been otherwise excluded from calculations.

Structural— Depth

It was fundamental for the design of the expansion to maintain certain design considerations as the existing structure. Specifically, maintaining negligible eccentricity would prevent new and potentially unexpected loads being applied or managed differently throughout the building. Columns were to be kept uniform and placed on top of existing columns for simplicity.

For design of the three additional stories to be added to the top of the building, the existing snow load of 50 psf, a mechanical load of 20 psf, in addition to a 115 psf dead load, which includes a 15 psf superimposed dead load, with a total factored load of 215 psf was used. This load produced an axial load of 143.4 kips with a bay size of 667 sq ft, and it was found that a 24" by 24" column, using concrete with a f'c of 4000 psi, reinforced with 8 #7 rebar was sufficient to support this load as well the possible moment of 393.1 ft*kips.

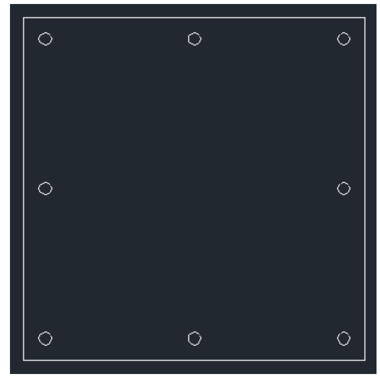


Figure 12. A typical 24" by 24" column with 8 #7 rebar.

This column, assumed to be typical, however is required to be 24.6' tall, and requires bracing at mid height to mitigate bending effects over its length. A beam of 24" by 24" was used to match the dimensions of the columns as this beam was to be exposed, and not supporting a slab. However, this beam was required to support its own weight as well as miscellaneous air ducts and lighting equipment. 15 psf dead load was assumed for mechanical and lighting equipment, and was combined with the 600 plf of the beam to find a total of a 1100 plf load. Analysis found that this load was managed by 2 #7 rebar, where moment was found to be a positive 66 ft*kips, and 2 #6's for the negative 96.1 ft*kip moment. Shear was checked for the beams, and due to the size of the beam, no specific shear reinforcement was required, according to section 11.4.6.1 in ACI 318-07. Torsion for the exterior bracing beams, supporting the façade, was predicted to be problematic, and thus was checked. Due to the large size of the beam, it was found that torsion was negligible according to section 11.5.1, also in ACI 318-07. Effects from these bracing beams were checked on the lower half of the 24.6' tall columns and it was found that no changes were required.

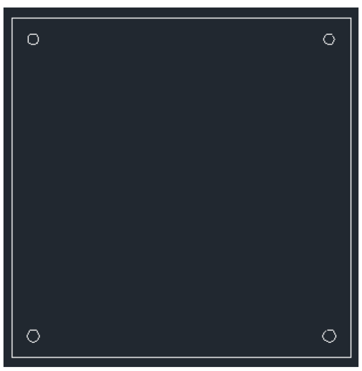


Figure 13. A typical cross section of the 24" by 24" bracing beam used. It has 2 #6's at top and 2#7's at bottom.

In the addition, there are 3 floors, all of which were to be assumed typical, all with a design load of 80 psf, due to the student atmosphere and recreation/studio environment, as opposed to most lower floors requiring a 125 psf design load for their multiple labs. Only thing that would change from floor to floor would be an axial load increase in the columns, but no actual change required as the maximum allowable axial load is 2000 kips with a 24" by 24" cross section. Under the new load requirements, and thicker slab, the factored load becomes 326 psf, both from the increased live load of 80 psf, and the 150 psf from the 12" thick slab, along with a 15 psf superimposed load. A one-way slab system was developed to handle these loads.

21 foot long beams were designed to handle the loads distributed over the slab, in which wire mesh fabric is used to reinforce the slab. Under the load from the slab, which causes a positive 321.2 ft*kips, and a negative 467.2 ft*kips, the 6 foot wide, 1 foot thick beam was found to require 6 #10 rebar on the bottom of

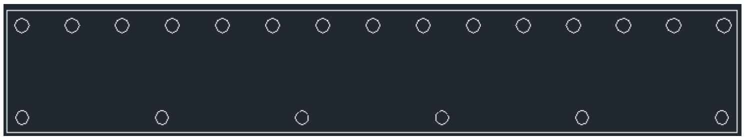


Figure 14. Typical 72” by 12” beam cross section supporting the slab. Top is reinforced by 15 #11 rebar, and the bottom has 6 #10 rebar.

the beam, and 15 #11 rebar at the top. This beam is considered to be typical, and is used on all 3 floors. As for the slab, it was sliced into foot wide sections, and treated as a 12” by 12” beam with a 330 plf load. This creates a positive moment of 19.8 ft*kips and a negative moment of 28.8 ft*kips, on this foot wide beam, which requires a W15 WMF spaced at 4”, and W25 WMF at 4”, respectively. The slab design is considered to be typical for all floors.

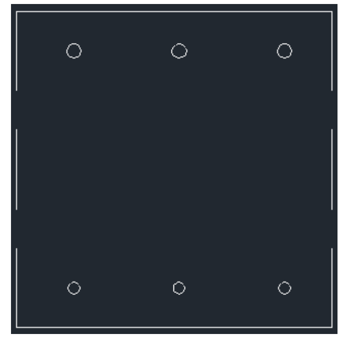


Figure 15. Slice of section of slab, reinforced on top with W25 WMF at 4” and W15

Development Length

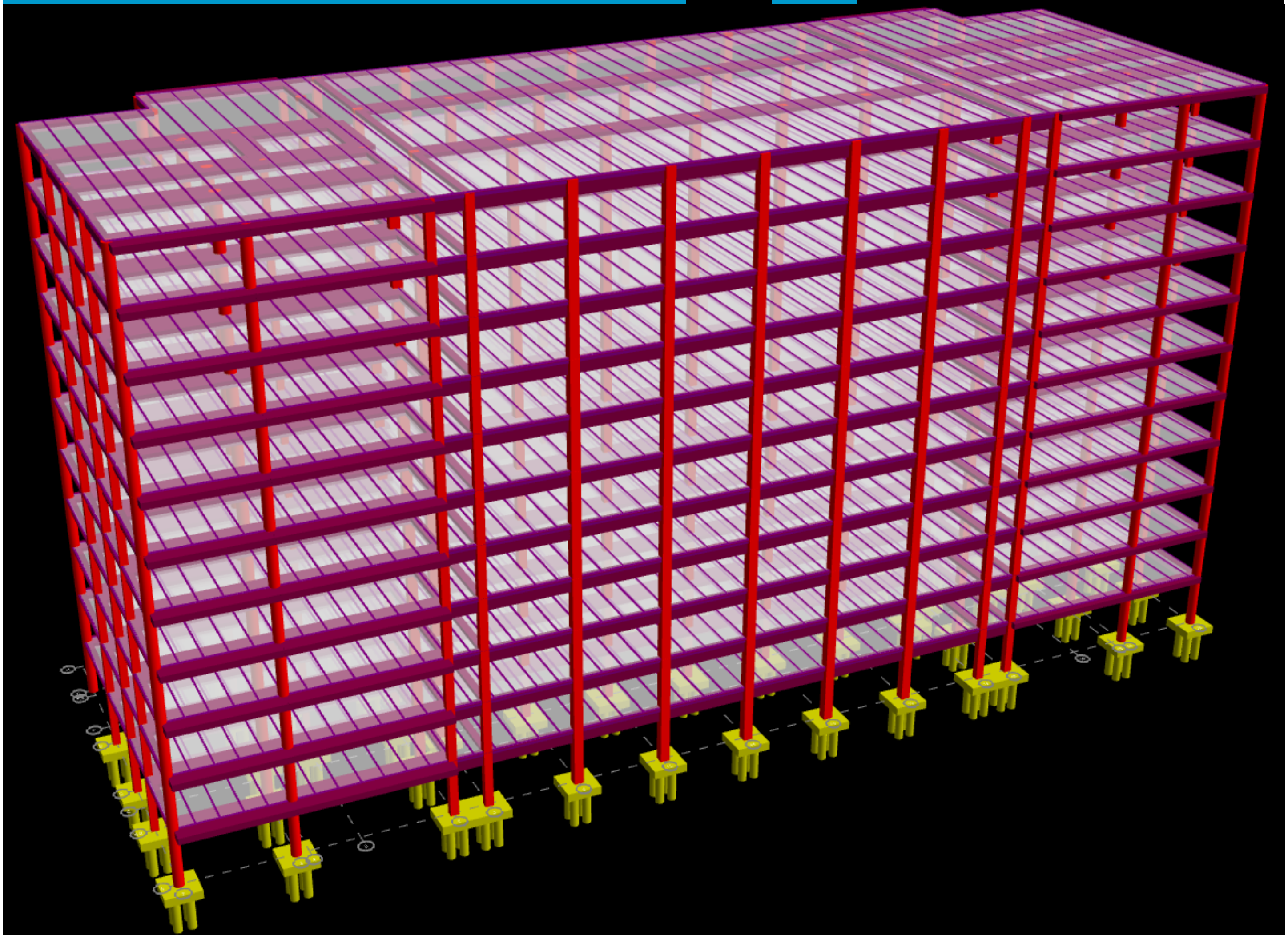
Development length was explored to ensure that reinforcement used was utilized properly, and was done for column bracing, columns, beams, and slabs. For column bracing on the top floor, a development length of 29” was required for positive reinforcement and 26” for negative. A typical, 6 foot wide beam requires 54” for positive reinforcement and 65” for negative. Columns require 29” and slab reinforcement required 12” of development length.

Deflections

A deflection analysis was done to check short and long term effects of various loads on the concrete system. Minimum height requirements were met to use table 9.5 in ACI 318-07. Deflections were found for both the typical beams and the typical slabs. Beams were found to a total of .6” which was found to be allowable by the 1.4” requirement, and the slab was found to deflect .3”, which was much less than the 2.3” allowed by table 9.5 (b) in the ACI Manual.

Existing Structure

Spot checks were done on the existing structure to ensure that this new addition would not exceed the capabilities of the existing structure. According to previous exploratory tech reports, it was found there was a 30-35% extra capacity in the columns. This thesis tests that extra capacity, and checking axial capacity for the existing structure, axial capacity was almost fully utilized, especially at the lowest floors. Moments would have exceeded that capacity, and analysis was done again taking maximum live load reduction allowable by code, taking into effect that certain loads could not take advantage of maximum reductions, specifically loads over 100 psf not being allowed to take more than a 20% reduction, and it was found that under these conditions, that the existing structure satisfies requirements.



Lateral Check

An additional 3 floors were added to the existing RAM model, as shown above, to provide analysis for the effects that the expansion would have on the existing structure as well as the effects of lateral on the expansion itself. As expected from previous tech reports, it was found that direct forces on the building control, in the X and Y directions. Still, it was also found that wind forces controlled on the North-South faces of the building and seismic controlled on the East-West faces. Using the 1.5% stiffness distribution factor found in Tech Report 3, combined with a maximum story shear of 624kips, leaves each column on the bottom floor to handle 9.4 kips. Over a 13.6' column height, this produces a 127.8 ft*kip moment on each column, much lower than the 450 ft*kip capacity of each column.

Drift

Drift was checked again, with aid from the modified RAM model, to ensure that the extra 50' in building height did not cause the building to exceed allowable drifts. Using H/400 to maintain drift control, the building was shown to pass the limitations, even with the extra 50 addition feet on top of the building, although the total drift doubled in comparison to the original building. Drift tables follow on the next page, displaying how much each story drifts under which circumstances, and total building drift.

Drift (Continued)

Story Drift						
	Controlling Wind			Seismic		
Floor	X	Y	Allowable	X	Y	Allowable
10	0.04	0.003	0.74	0.04	0.02	5.94
9	0.14	0.010	0.37	0.14	0.03	2.97
8	0.17	0.012	0.37	0.17	0.04	2.97
7	0.22	0.016	0.37	0.22	0.06	2.97
6	0.26	0.020	0.37	0.26	0.07	2.97
5	0.30	0.024	0.37	0.30	0.08	2.97
4	0.35	0.029	0.37	0.35	0.10	2.97
3	0.38	0.031	0.37	0.38	0.12	2.97
2	0.39	0.032	0.38	0.39	0.14	3.04
1	0.25	0.027	0.41	0.25	0.12	3.28

Drift						
	Controlling Wind			Seismic		
Floor	X	Y	Allowable	X	Y	Allowable
10	2.50	0.204	4.12	2.50	0.78	33.05
9	2.46	0.201	3.38	2.46	0.76	27.11
8	2.32	0.191	3.01	2.32	0.73	24.14
7	2.15	0.179	2.64	2.15	0.69	21.17
6	1.93	0.163	2.27	1.93	0.63	18.2
5	1.67	0.143	1.90	1.67	0.56	15.23
4	1.37	0.119	1.53	1.37	0.48	12.26
3	1.02	0.090	1.16	1.02	0.38	9.29
2	0.64	0.059	0.79	0.64	0.26	6.32
1	0.25	0.027	0.41	0.25	0.12	3.28

Overturning

Overturning was taken into account to ensure the building's ability to resist the applied lateral loads as a whole. The building in this case uses its self weight to resist lateral loads. Again, the two sides of the building varied in what controls the design of the structure. The long side had wind controlling, and the short side had seismic controlling, but both worst case scenarios of wind and seismic were done for the two sides for comparison purposes. Results found from the model are shown on the next page. Wind and seismic forces are in kips, and moments are in ft-kips.

X Direction Overturning							
						Resisting Moment	
	Wind	Seismic	Arm	Moment		Self Wt	Arm
1	33.39	53.89	13.7	457.4	738.3	38300	47.5
2	129.7	56.12	26.3	3411.1	1476.0		
3	127.16	56.12	38.7	4921.1	2171.8		
4	124.36	56.12	51	6342.4	2862.1		
5	121.28	56.12	63.3	7677.0	3552.4		
6	58.9	56.13	75.7	4458.7	4249.0		
7	113.78	56.12	88	10012.6	4938.6		
8	108.96	56.12	100.3	10928.7	5628.8		
9	104.28	56.19	112.6	11741.9	6327.0		
10	51.65	56.43	124.9	6451.1	7048.1		
Total				66402	38992	1819250	Good

Y Direction Overturning							
						Resisting Moment	
	Wind	Seismic	Arm	Moment		Self Wt	Arm
1	10.25	53.89	13.7	140.4	738.3	38300	140
2	19.77	56.12	26.3	520.0	1476.0		
3	19.29	56.12	38.7	746.5	2171.8		
4	18.77	56.12	51	957.3	2862.1		
5	18.19	56.12	63.3	1151.4	3552.4		
6	17.54	56.13	75.7	1327.8	4249.0		
7	16.79	56.12	88	1477.5	4938.6		
8	15.89	56.12	100.3	1593.8	5628.8		
9	14.95	56.19	112.6	1683.4	6327.0		
10	14.52	56.43	124.9	1813.5	7048.1		
Total				11412	38992	5362000	Good

Structural Depth Conclusion

From the analysis of both hand calculations and results from the RAM model and developed spreadsheets, it shows that the addition of 3 stories on top of the existing structure is indeed a viable option, as was intended by the owner. The live load was reduced from 125 psf to 80 psf, but would still serve the designed purpose without any modifications to the existing structure.

HVAC (Breadth 1)

In order to make the new addition an effective place to work and study, it is necessary to make it comfortable. One way to ensure comfort is to make sure that the temperature is managed through ventilating the addition with treated air for heating or cooling. First step that was taken was designing the insulation. A table is shown on the next page starting with the outside material and working its way in.

Enclosure		
Insulation:	Thickness	R-Value
Limestone	4.5"	0.8
Insulation	2"	8
CMU	7.625"	1.11
Insulation	2"	8
GWB	.625"	0.56
Total:	16.75"	18.47

Temperature differences where then calculated for temperature extremes, with the difference for cooling being 33 degrees, and the difference for heating being 77 degrees. Dividing by the R value found about and the R value for the triple pane glazing, which was found to be worth the investment, giving an addition 60% efficiency for 50% more glass, within .75” in total thickness, it was found that heat gain was 1.8 BTU/HR per square foot for non-glazing, and 10.2 BTU/HR for glazing, while heat loss was found to be 4.2 BTU/HR for non-glazing, and 23.8 BTU/HR for glazing. At a ratio of 50:50 for glazing to non glazing, this results in a 175000 BTU/HR heat gain in the summer, and 400000 BTU/HR heat loss in the winter. Person loads were also calculated based on occupancy charts, which put 50 people in 1000 square feet of space, according to the IBC. With 3 stories at 90’ by 300’, approximately, this results in 4050 people in this addition, and at 500 BTU/HR per person, is 2000000 BTU/HR.

Ventilation

Ventilation requirements for 4050 people were found to be 81000 CFM, and ventilation requirements for the general square footage of the expansion were found to be about 5000 CFM. This gives us a total of 86000 CFM for the expansion that must be pumped in addition to the existing system. This 86000 CFM must be heated or cooled, requiring 7.2 million BTU/HR and 3.1 million BTU/HR respectively.

Total HVAC Loads

Cooling in the summer was found to be 5.3 million BTU/HR, and heating in the summer was needed to be 5.6 million BTU/HR. This requires an 8 row 35ws Serpentine from McQuay with 13 FPI. This machine provides a 10’ by 18’ tunnel that needs to move air at 478 FPM to provide the required 86000 CFM, thought it can provide a maximum of 554 FPM.

Lighting (Breadth 2)

Two separate lighting systems were developed using TTX 188581 1xTL-D58W HFE WK Flourescent light fixtures, using a 58W T8, with 200 being required for the 90’ by 120’ room, allowing approximately 11 fixtures per 21’ by 35’ bay, to provide the recommended 500 lux for the space. A system was developed for both being supported by the ceiling at 22’ up, and one at mid height. The system at the ceiling level would require many more luminaries and create many, many shadows from the bracing beams, and require changes to the HVAC system to ensure that it did not block light and deliver the appropriate air treatment to occupants. The lower system however, creates a cramped feel for having proportionately low light in comparison to the overall height, and the luminaries used are considered direct lighting. So to open up the room, indirect lighting was used to illuminate the upper part of the room, while using the main mid height system, giving occupants the illusion the room is more open than it is.

Lighting Continued

In addition to the luminaries at the ceiling, a low reflectance ceiling should be used to prevent shadows from the bracing beams.

Acoustics (Breadth 3)

Noise reduction is important to make sure this space is usable for students, even if they do not keep quiet, and as such, acoustic management was checked. A maximum decibel level of 100 was assumed for the top floor, and 80 dB for the other two floors. First, insulating everything for maximum reduction was attempted, but it was noted that the side effect of such a room would deaden the atmosphere, and disturb occupants, which would be just as distracting as a room that was too loud. So a happy medium was set with an average sabin level of .4, erring on the dead side, but still allowing some life in the room. This was achieved by insulating the bracing beams with a material with a insulating coefficient of .29, to soak up a significant portion of the sound, while the ceiling was insulated with a coefficient of .95, to prevent echoes. It was found, under this configuration that the reverb time was about .6 seconds.

Noise Reduction

Using a simple equation, this insulation configuration was checked for how efficient the noise reduction was. The bare room was found to have 400 sabins, from the amount of glazing, and after the insulation, had 19700 sabins. Using this, it was found that the top floor had a reduction of 17 dB, reducing the 100 dB to 83 dB, while on the other two floors with an after insulation sabin level of 11200, gave a reduction of 15 dB, reducing the 80 dB to 65 dB.

Cost

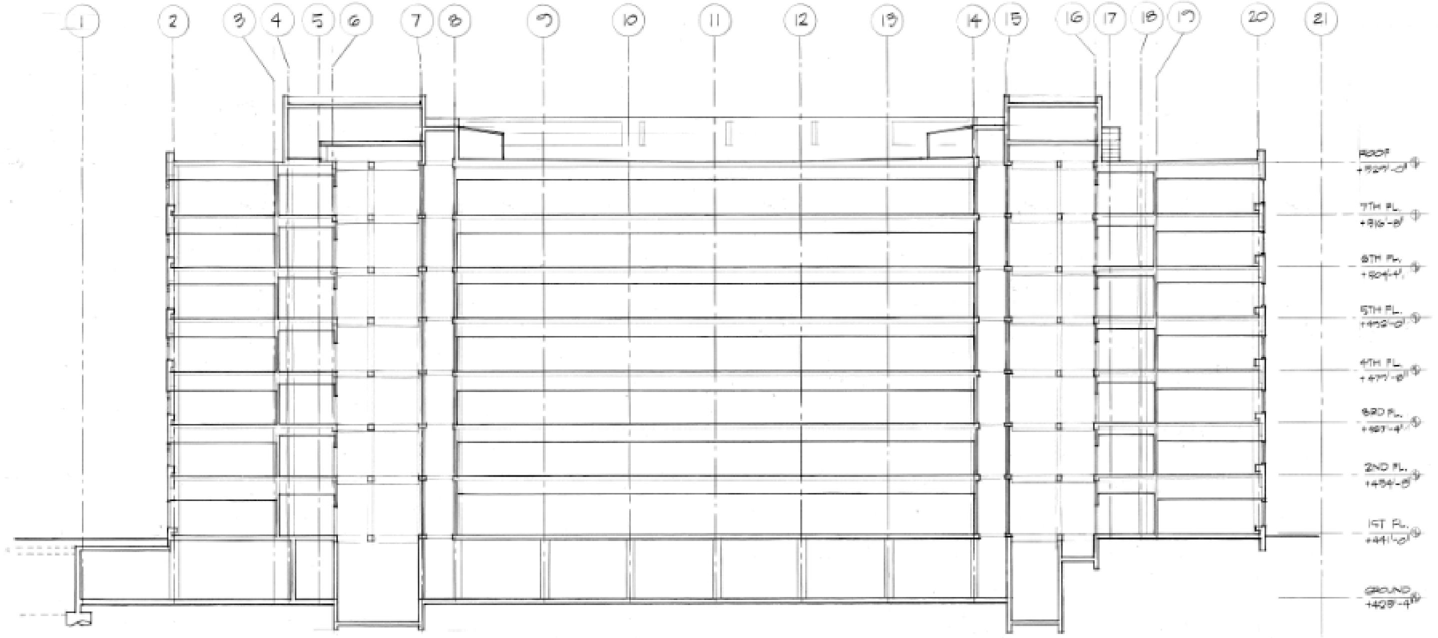
According to values pulled from RS Means, using values for a 5 to 10 story medical office building in Philadelphia as a reference point, at approximately 250 dollars per square foot, it would cost about $23.3 million dollars, including 15% extra for contractor fees, to build this addition for the Biomedical Research Building.

Conclusion

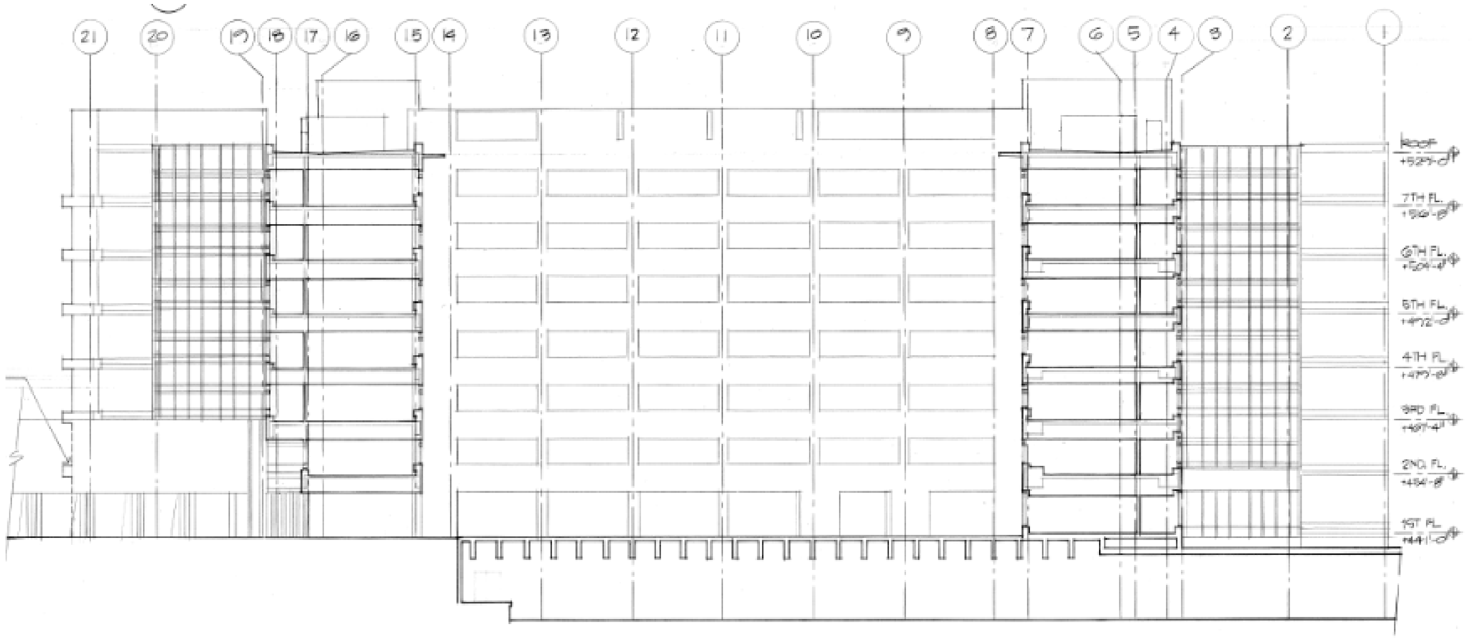
The initial structural depth analysis and exploration of the effects on the existing structure with the hypothetical addition of the 3 additional stories concludes that it can support the extra gravity and lateral loads, and that an expansion is viable. The breadths and extraneous exploration provides a sound and realistic option for the expansion, making it a living, breathing space for occupants, supplying comfort through HVAC and acoustic management, as well as an operable space with appropriate lighting. The forethought of designing a building for additional floors and maintaining ease of analysis for these additional floors through maintaining negligible eccentricity, will pay off should the need ever arise to expand upon an existing building on the Milton S. Hershey Medical Center campus instead of constructing a new building altogether.

Appendix

Elevations

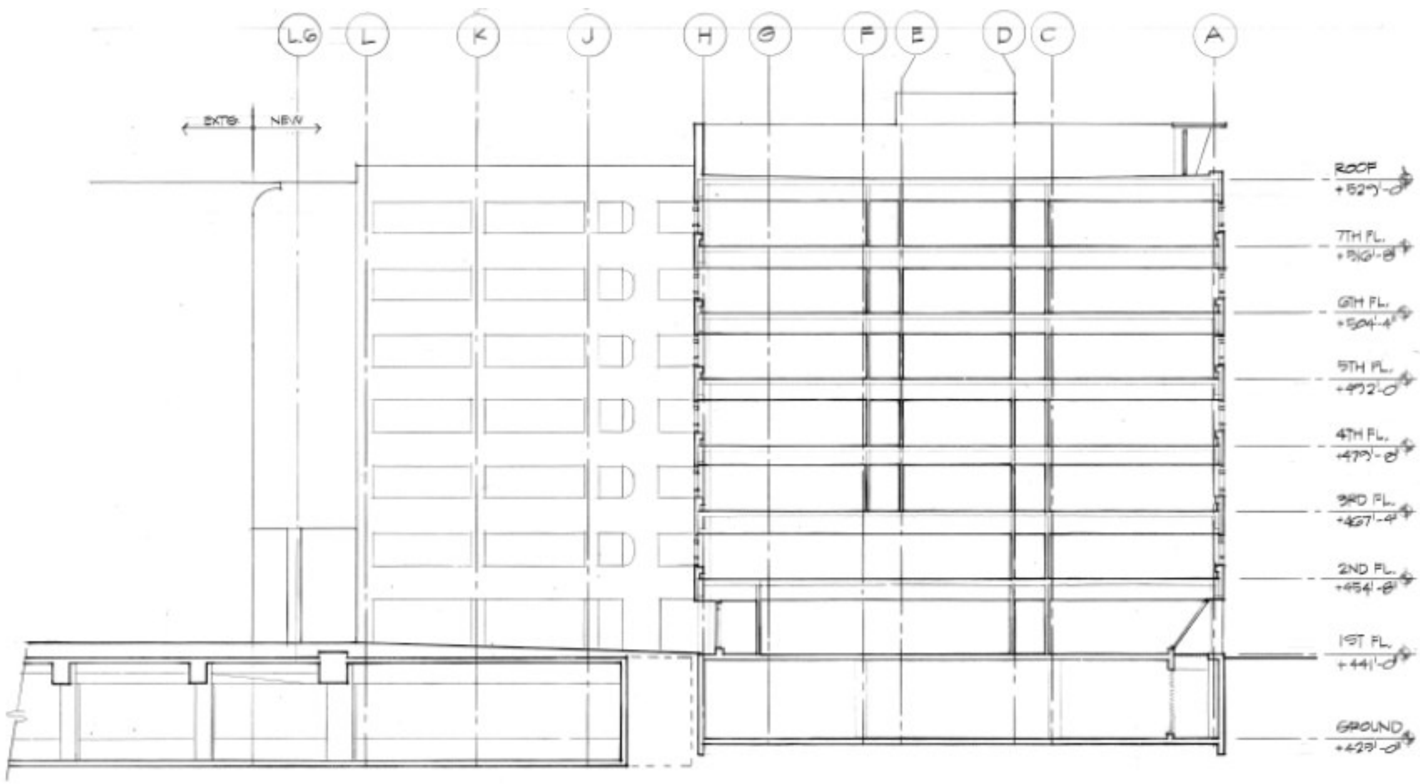


1 A208 LONGITUDINAL SECTION LOOKING NORTH
SCALE: 1/16" = 1'-0"



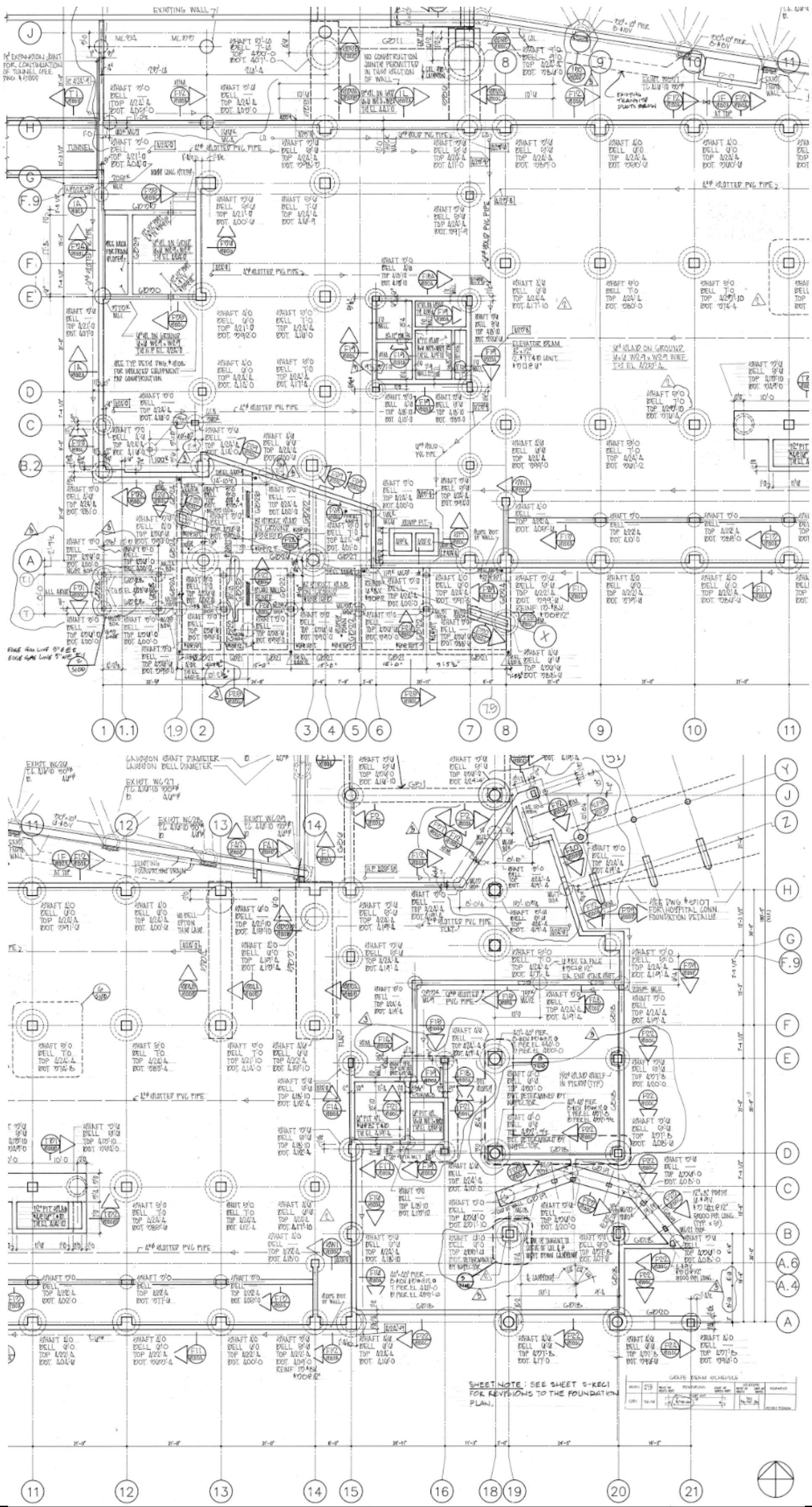
2 A208 SECTION THRU CONNECTORS LOOKING SOUTH
SCALE: 1/16" = 1'-0"

Elevations

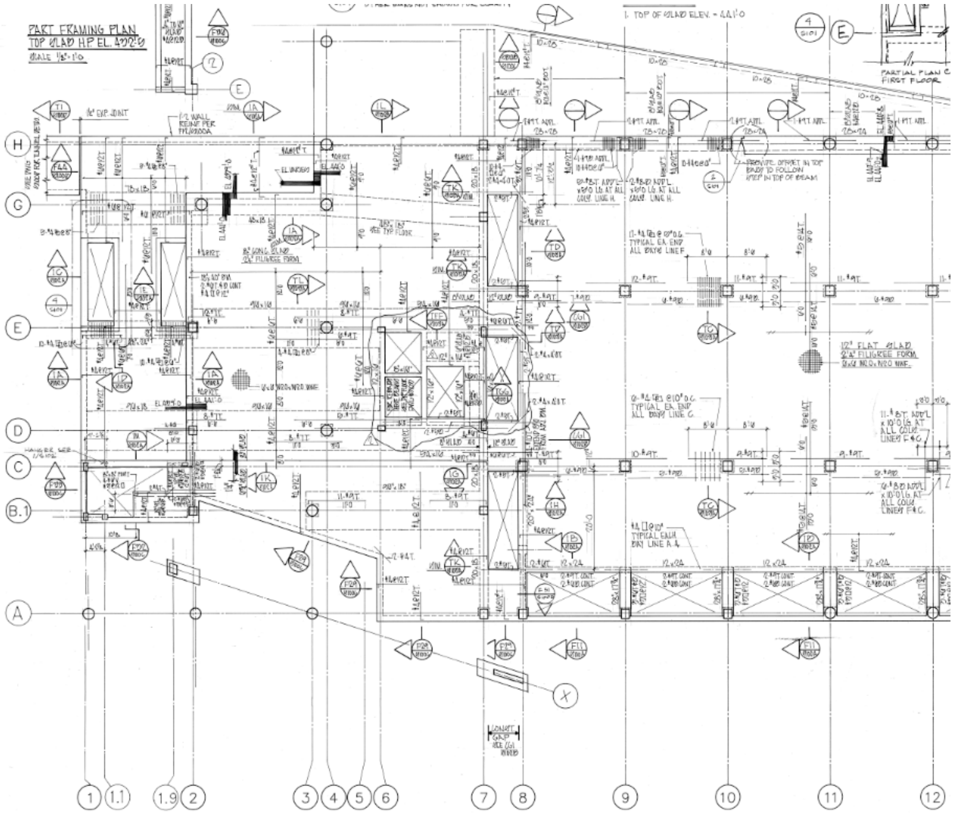


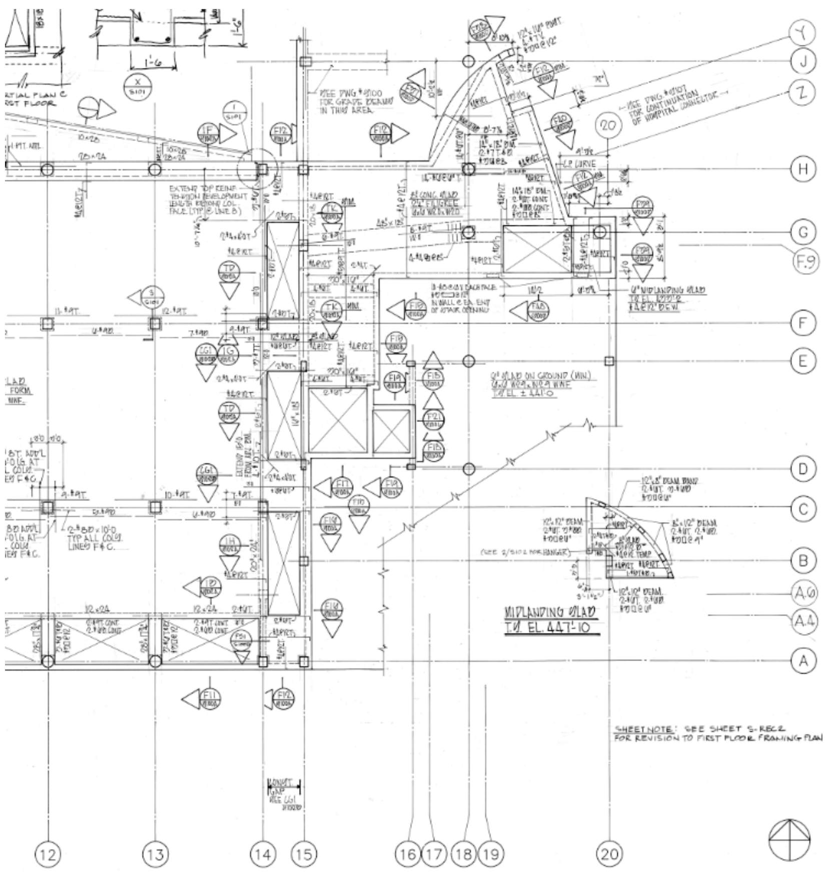
4 / A208 TRANSVERSE SECTION LOOKING EAST
SCALE: 1/16" = 1'-0"

Foundation Plan (Ground Floor)

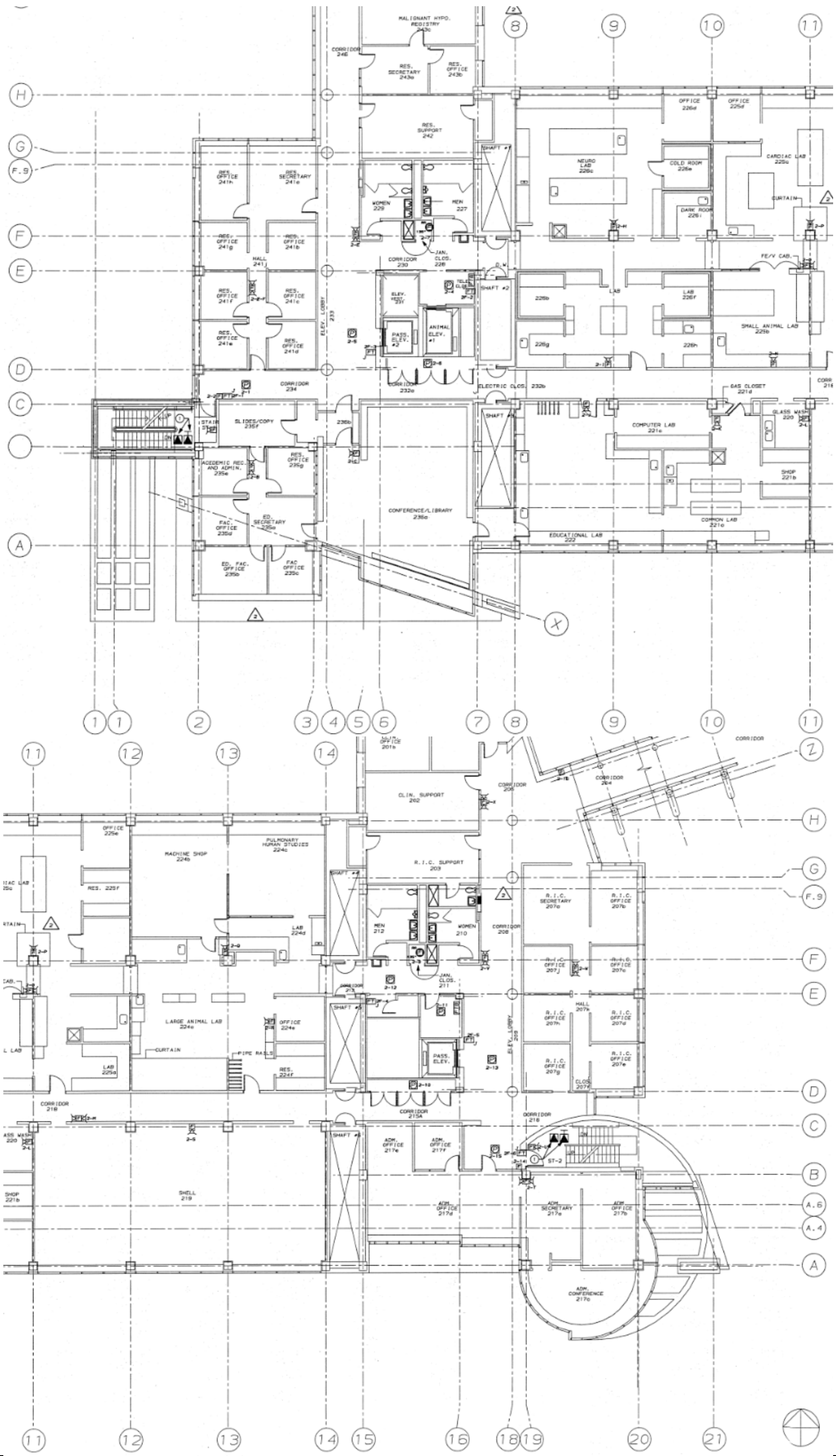


First Floor Plan

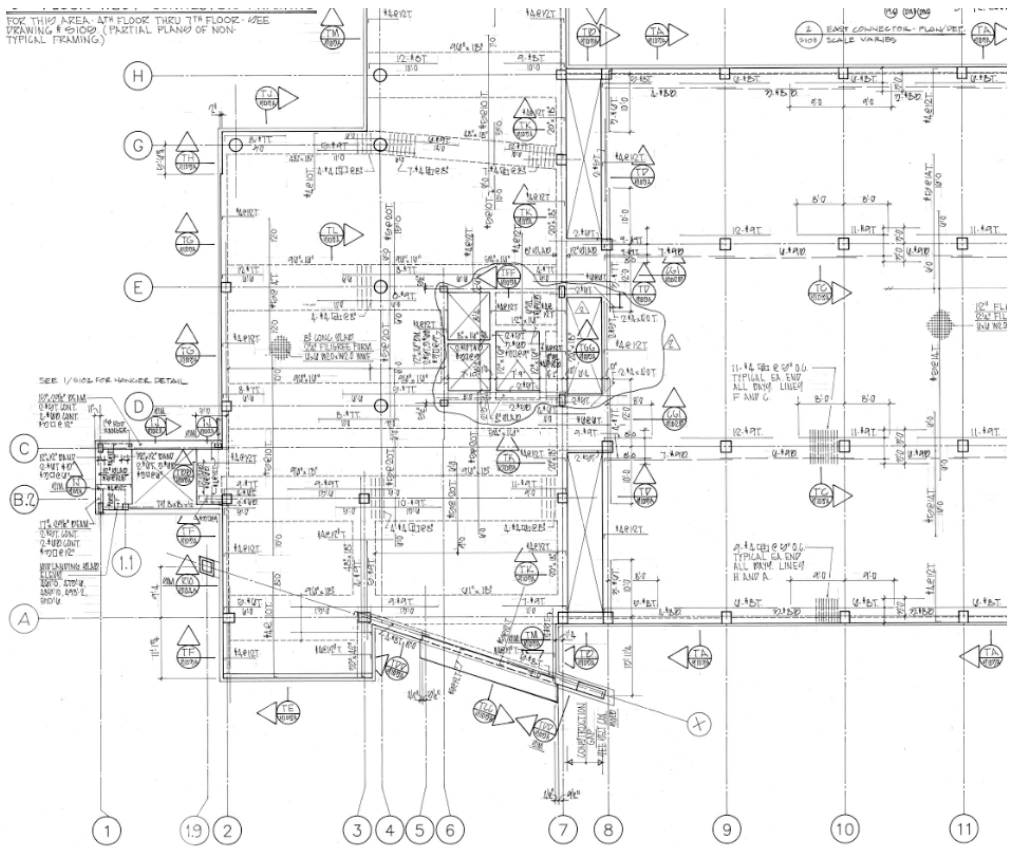


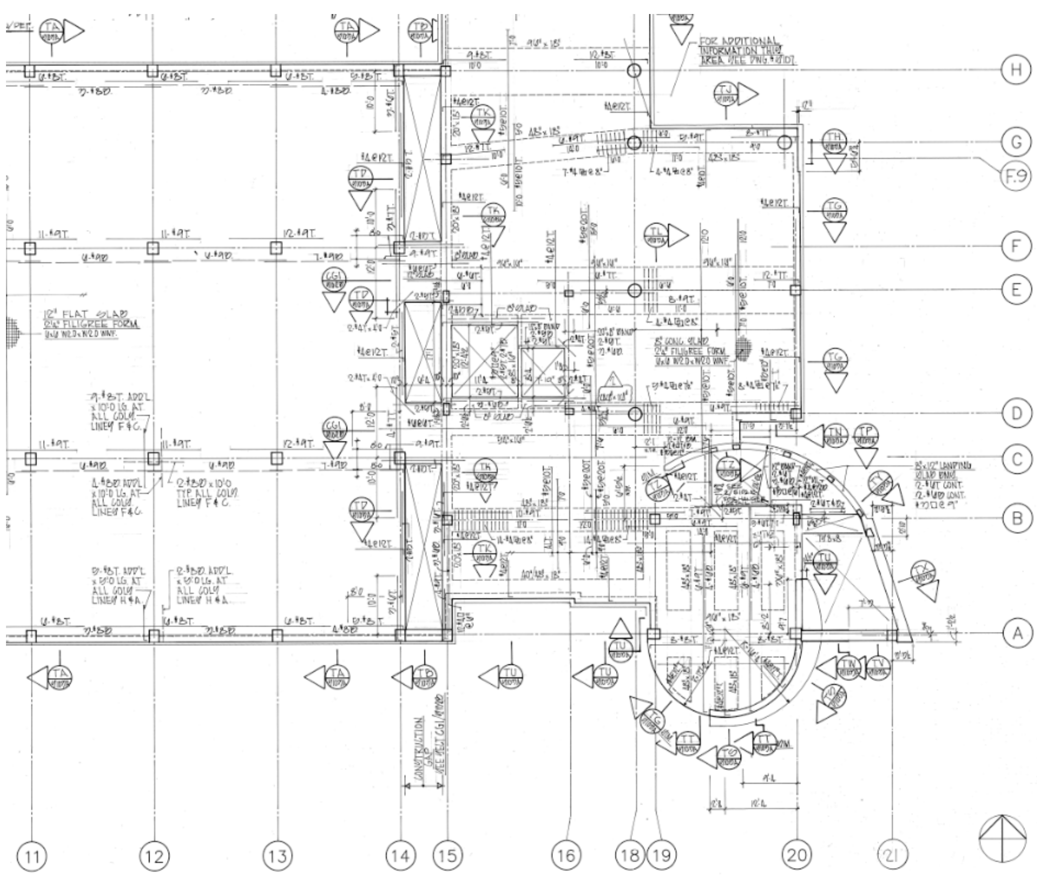


Second Floor Plan



Typical 3rd through 7th Floor Plans





Self weight of building:

Assume 8" slabs typical for floors 3-7.

Assume 12" slabs for 1st floor. Assume 6" slab for ground floor.

Assume 12" slabs for 2nd floor. Assume 8" slab for roof

Assume 150 lbs per cubic foot of concrete.

Assume columns are uniform in size along entire length

Assume curtain walls to be typical – 6' 3.5" high limestone sections, 4" thick
– 6' 4.5" glass, 1" thick

Approximate area of ground floor:

(95.75)·(264.75) = 25350 sq ft.

Approximate area of 1st floor:

(88.2)(286) = 25225 sq ft.

Approximate area of 2nd floor:

(257)(96) = 24672 sq ft.

Approximate area of 3rd – 7th floors:

(282.75)(96) = 27144 sq ft.

5·27144 = 135720 sq ft.

total area: 211000 sq ft

Approximate roof area: 25000 sq ft.

height of columns: 110' from top of caissons to bottom of roof.

Average column size is 20"x20". => 1.6'x1.6'

# of columns: 67

volume of columns: 18870 ft^3

volume of floors + roof:

$$\frac{25350}{2} ft^3 + 25225 ft^3 + 24670 ft^3 + 2\left(\frac{135720}{3}\right) ft^3 + 2\left(\frac{25000}{3}\right) ft^3 = 169700 ft^3$$

Perimeter of building: 752 ft

Joshua Zolko | Tech Report 1 | Seismic Loads | 2/3

Site location: Hershey, PA

1. Determine Design Spectral Response Acceleration

Values obtained from usgs.gov, referencing ASCE 7-10

Assume Class "B" Soil Classification, Risk Category facility

$S_s = .154g$ $S_{MS} = .154g$ $S_{DS} = .103g$

$S_1 = .055g$ $S_{M1} = .055g$ $S_{D1} = .037g$

2. Determine Seismic Design Category (SDC)

Building is Category IV so $I = 1.5$ (table 1.5-2)

For Hershey, PA.

For $S_{DS} = .103g$ and Category IV ⇒ SDC = A (table 11.6-1)

For $S_{D1} = .037g$ and Category IV ⇒ SDC = A (table 11.6-2)

3. Identify the analysis procedure

Equivalent Lateral Force Procedure

Seismic base Shear

$V = C_s W$ $R = 3$ (Table 12.2-1, Ordinary reinforced concrete moment frame)

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{.103}{\left(\frac{3}{1.5}\right)} = .0515$$

$T = C_T h_n^x$ $C_T = .016$ $x = .9$ (Table 12.8-2)

$= .016(110)^{.9}$ $h_n = 110'$

$= 1.100 < T_L = 6$ ✓

$$C_s < \frac{S_{D1}}{\left(\frac{R}{I}\right)T} = \frac{.037}{\left(\frac{3}{1.5}\right)1.1} = .017$$

C_s is not lower than .017, therefore C_s is .017.

$C_s = .017 > .01$ ✓

4. Calculate total building weight

Joshua Zolko | Tech Report 1 | Seismic Loads | 3/3

Surface area of building:

752.110 = 82720 Sq ft.

$\frac{82720}{2} \cdot \frac{1}{3} \cdot 150 + \frac{82720}{2} \cdot \frac{1}{12} \cdot 150 = 3{,}100{,}000$ lbs curtain wall weight

$(169700 + 18870)150 = 28300000$ lbs

Total Approximate building weight: 31400000 lbs

roof LL = 30 psf (from plans)

$.2 \cdot 30 = 6$ psf

$6 \cdot 25000 = 150000$ lbs

$w = 31400000 + 150000 = 31450000$ lbs $\Rightarrow$ 31450 Kips

$V = C_s \cdot w = .017 \cdot 31450$

$V = 529$ Kips

6. Determine vertical distribution of seismic forces

$F_x = C_{vx} \cdot V$ $V = 529$ Kips

$C_{vx} = \frac{w_x h_x^k}{\Sigma w_i h_i^k}$ $k = 2$, for $T = 1.17.5$s

C_{vx} calculated on spreadsheet.

F_x Calculated on spreadsheet.

Overturning moment also calculated on spreadsheet, at 42808 ft-Kips

Joshua Zolko | Tech Report 1 | Wind Loads | 1/2

Location: Hershey, PA

Category IV (Table 1-1)

Exposure C (section 6.5.6)

V=90mph (Figure 6-1)

I=1.15 (Table 6-1)

K_d=.85 (Table 6-4)

K_{zt}=1.0 (flat elevation)

$K_z = \frac{1.26+1.31}{2} = 1.29$ (Table 6-3) (h=110') (varies)

qz calculated on spread sheet using:

$q_z = .00256 K_z K_{zt} K_d V^2 I$

For $p = qGC_p - q_i(GC_{pi})$;

G=.85
C_p from table 6-6
$q_i = q$
GC_{p_i} from table 6-5

values found in Spread sheet.

L/B for 95' side: 95/277 = .34 < 1; C_p = -.5

L/B for 277' side: 277/95 = 2.92; C_p = -.3

Joshua Zolko | Tech Report 1 | Spot Checks | 1/a

Beam Check

Rebar: $f_y = 60{,}000$ psi Stirrups: $f_y = 40{,}000$ psi $f'c = 4000$ psi

LL = 125 psf (Lab spaces require 125 psf as specified on drawings)

DL = 150 psf (1 ft deep concrete slab)

Superimposed DL = 15 psf

$1.2D + 1.6L = w$

$1.2(150 + 15) + 1.6(125) = 398 \text{ psf} \approx 400 \text{ psf}$

Trib length: 35'-9'' Clear span: $21 - {}^{22}/_{12} = 19'\text{-}2''$

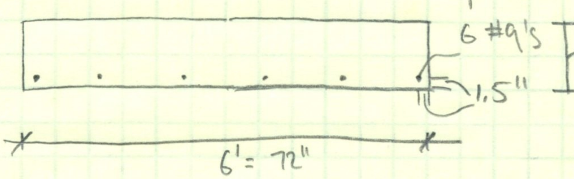
Distributed load:

$35.75 \cdot 400 = 14.3$ Kip/ft

$$M^+ = \frac{14.3(19.2)^2}{16} = 329.5 \text{ ft·Kips}$$

$$M^- = \frac{14.3(19.2)^2}{11} = 479.2 \text{ ft·Kips}$$

Section of beam @ maximum positive moment: 329.5 ft·Kips



note: beam is continuous with slab; both have a depth of 12''

$d = 12 - 1.5 = 10.5''$

$A_s = 6 \cdot (1) = 6 \text{ in}^2$ $f_y = 60000$ psi $f'c = 4000$ psi (assuming rebar is yielding)

$$a = \frac{A_s \cdot f_y}{.85(f'c)(b)} = \frac{6 \cdot 60000}{.85(4000)(72)} = 1.47'' \qquad c = \frac{a}{.85} \Rightarrow \frac{1.47}{.85} = 1.73''$$

Check assumption:

$$.003\left(\frac{d-c}{c}\right) = .003\left(\frac{10.5 - 1.73}{1.73}\right) = .015 > .002 \checkmark$$ and $.015 > .004$, section is allowed.

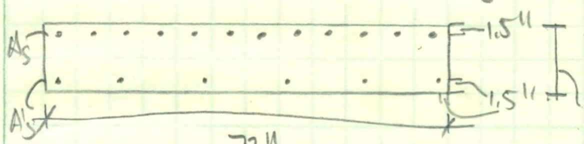
$M_n = A_s \cdot f_y (d - a/2)$

$= 6 \cdot 60000(10.5 - {}^{1.47}/_2) = 4055400 \text{ lbs·in} \Rightarrow 293 \text{ ft·Kips}$

$293 \text{ ft·Kip} < 329.5 \text{ ft·Kip}$ X section fails

Joshua Zolko | Tech Report 1 | Spot Checks | 2/9

Section of beam @ maximum negative moment = 479.2 ft Kips



Treat section as upside-down, checking negative section.

$A's = 6 in^2$ $d' = 1.5"$ $f_y = 60000 psi$ Assume both rows of rebar are yielding.
$As = 12 in$ $d = 10.5"$ $f'c = 4000 psi$

$a = \frac{As f_y - A's f_y}{.85 f'c (b)} = \frac{12(60000) - 6(60000)}{.85(4000)(72)} = 1.47"$ (measured from bottom of section)

$c = \frac{1.47}{.85} = 1.73"$

Verify assumptions:

$\varepsilon's = \frac{.003}{c}(c-d') = \frac{.003}{1.73}(1.73-1.5) = .00039 < .002$; not true

$.85 f'c \beta_1 c \cdot b + A's \frac{.003}{c}(c-d')E_s = As f_y$ $E_s = 29000000 psi$

Solve quadratic

$.85 f'c \beta_1 c^2 b + A's .003(c-d')E_s - As f_y \cdot c = 0$

$.85 f'c \beta_1 c^2 b + A's .003 c E_s - As f_y c - A's .003(d')E_s = 0$

$.85(4000)(.85)(72)c^2 + 6 \cdot .003 \cdot 29000000c - 12 \cdot 60000 \cdot c - 6 \cdot .003(29000000)1.5 = 0$

$208080c^2 - 198000c - 783000 = 0$

$c = 2.47"$

Check yielding:

$\varepsilon_s = \frac{.003}{2.47}(10.5 - 2.47) = .0097 > .002$ ✓ and $.0097 > .004$, section is allowed.

$M_n = A's \cdot \frac{.003}{c}(c-d')E_s(d-d') + .85 f'c a b (d - a/2)$ $a = \beta_1 c = 2.1"$

$M_n = 6 \cdot \frac{.003}{2.47}(2.47-1.5)(29000000)(10.5-1.5) + .85(4000)(2.1)(72)(10.5 - 2.1/2)$

.97 9 9.45

$= 6703020 lb \cdot in \Rightarrow 559 ft \cdot Kips$

559 ft Kips > 479.2 ft Kips ✓

Check shear reinforcement: Plans call for 11 #4's @ 5" o.c. (4 legs) => $A_v = .8 in^2$

$V_c = 2\sqrt{f'c}(b)(d)$ $f'c = 4000$ $b = 72"$ $d = 10.5"$

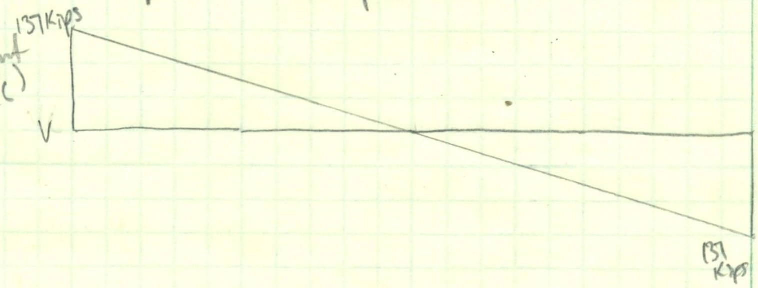
$= 2\sqrt{4000}(72)(10.5)$ R = 137 Kips Clear span = 19'-12"

$V_c = 95.6$ Kips (no reinforcement required below V_c)

$\phi V_n = .5\phi V_c$ $\phi = .75$

$= .5(.75)(95.6)$

$\phi V_n = 35.9$ Kips



$V_s = \frac{V_u}{\phi} - V_c$ $V_u = 137 - \frac{10.5}{12} \cdot 14.3 = 124.5$ Kips

$V_s = \frac{124.5}{.75} - 95.6 = 70.4$ Kips

Maximum spacing

$V_s \leq 4\sqrt{f'c}\, b_w d = 4\sqrt{4000}(72)(10.5)$

$V_s \leq 191.3$ ✓

therefore $S_{max} = \min\begin{cases} d/2 \\ 24" \end{cases}$

$S_{max} = \min\begin{cases} 10.5/2 \\ 24" \end{cases}$

$= 5.25" > 5"$ called for on plans ✓

Check reinforcement discontinue:

Appears shear reinforcement starts at 5" of support.

$137 - \frac{11.5}{12} \cdot 14.3 = 71.5$ Kips

$V_c = 95.6 > 71.5$ ✓

reinforcement discontinued at appropriate spot.

minimum shear reinforcement

$A_{vmin} = \max\begin{cases} .75\sqrt{f'c}\, b_w S/f_{yt} \\ 50 b_w S/f_{yt} \end{cases} = \max\begin{cases} .75\sqrt{4000}(72)(5)/40000 \\ 50(72)(5)/40000 \end{cases} = \max\begin{cases} .43 \\ .45 \end{cases}$

$A_{vmin} = .45 in^2 < .8 in^2$ ✓

Check required reinforcement:

$\phi V_n = \phi V_c + \phi V_s = .75(95.6 + \frac{.8(40)(10.5)}{5}) = 122.1$ Kips

122.1 Kips < 124.5 Kips ✗ shear reinforcement fails

Joshua Zolko | Tech Report 1 | Spot Checks | 4/9

Deflection check.

$h_{min} = \frac{L}{21} = \frac{19.2 \cdot 12}{21} = 10.97'' < 12''$ ✓ unnecessary to check deflection.

Check development length of top reinforcement. (16' long total, so 8' into beam from center of column.)

$L_d = \frac{3}{40} \frac{f_y}{\lambda \sqrt{f'c}} \frac{\psi_t \psi_e \psi_s}{\left(\frac{c_b + K_{tr}}{d_b}\right)} d_b \left(\frac{A_{s,req'd}}{A_{s,prov}}\right) \geq 12''$

$c_b = 1.5''$ $K_{tr} = 0$ (conservative)

$f_y = 60000\,psi$

$d_b = 1.128''$ $\frac{A_{s,req'd}}{A_{s,prov}} = 1$ (conservative)

$\lambda = 1.0$

$f'c = 4000\,psi$

$\psi_t = 1.0$ $\psi_e = 1.0$ (assume uncoated bars, reasonable given this is an interior bar)

$\psi_s = 1.0$

$\frac{c_b + K_{tr}}{d_b} \leq 2.5$

$\frac{1.5 + 0}{1.128} = 1.3 < 2.5$ ✓

$L_d = \frac{3}{40} \frac{60000}{1.0\sqrt{4000}} \frac{1.0 \cdot 1.0 \cdot 1.0}{1.3} (1.128) \geq 12$

$L_d = 62'' > 12$ ✓

$\frac{62}{12} = 5'\text{-}2''$

Length given $= 96 - 11 = 7'\text{-}1''$

$5'\text{-}2'' < 7'\text{-}1''$ ✓ length is sufficient for reinforcement development.

Joshua Zolko | Tech Report 1 | Spot Checks | 5/9

Typical slab check:

Punching Shear:

At face of support:

$b_o = 22\cdot2 + 24\cdot2 + 4\cdot12 = 140''$

$V_n = \left(21\cdot35.75 - \left(\frac{22+12}{12}\cdot\frac{24+12}{12}\right)\right)400\,psf$

$= 297\,Kips$

$V_u = \phi 2\lambda\sqrt{f'c}\,b_o d + \frac{\phi A_v f_y d}{S} \le \phi 6\sqrt{f'c}\,b_o d$ $\lambda = 1.0$ $\phi = .75$ $A_v = 1.6 in^2$ $f_y = 40000\,psi$ $S = 5''$ $d = 10.5$

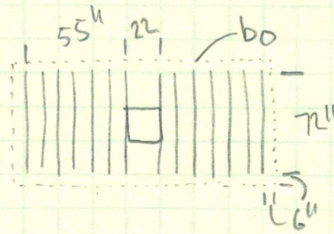
$V_u = .75(2)\sqrt{4000}(140)(10.5) + \frac{.75(40000)(10.5)}{5} \le .75(6)\sqrt{4000}(140)(10.5)$

$= 201.9\,Kips \le 416.7\,Kips$ ✓

but,

$V_n = 297 > 201.9\,Kips$ fails

At end of Stirrups:



$b_o = 22\cdot2 + 110\cdot2 + 72\cdot2 + 12\cdot4$

$= 456\,in$

$V_n = (21\cdot35.75 - 12\cdot7)400$

$= 266.7\,Kips$

$V_u = \phi 2\lambda\sqrt{f'c}\,b_o\cdot d$

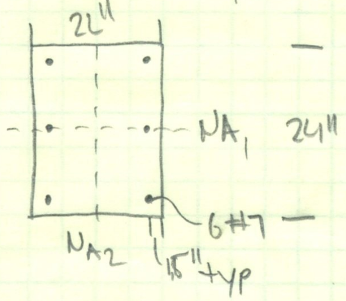
$= .75(2)\sqrt{4000}(456)\cdot10.5$

$V_u = 452.5\,Kips$

$V_n = 266.7 < 452.5$ ✓ does not fail to punching shear.

Joshua Zolko | Tech Report 1 | Spot Checks | 8/9

Column Check (typical)



$A_s = 6 \cdot .6 = 3.6 \text{ in}^2$ $f_y = 60000$ psi

Pure axial strength:

$A_c = 22 \cdot 24 - 3.6 = 524.4 \text{ in}^2$ $f'c = 4000$ psi

$P_o = .85 f'c A_c + A_s f_y$

$= .85(4000)524.4 + 3.6(60000)$

$P_o = 2000$ Kips

$P_n = \underbrace{2 \cdot 400 \cdot 35.75 \cdot 21}_{\text{2 floors of load 6th + 7th}} + \underbrace{30 \cdot 35.75 \cdot 21}_{\text{roof load}}$

$P_n = 1623$ Kips

$P_o > P_n$

$2000 > 623$ ✓

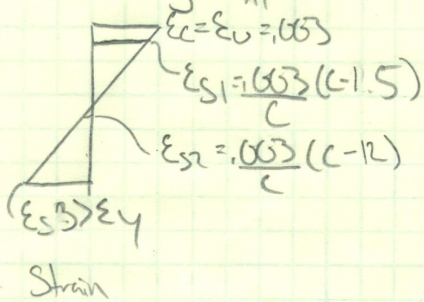
Pure axial tension:

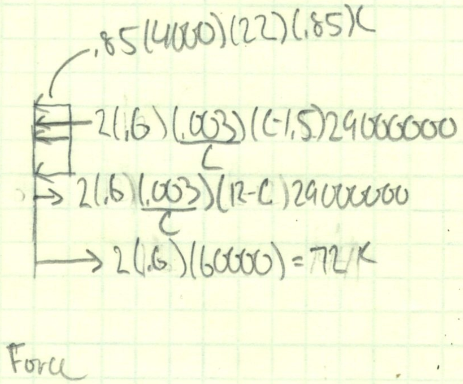
$T_o = A_s \cdot f_y$

$= 3.6 \cdot 60000$

$T_o = 216$ Kips

Pure bending @ N_{A1}





$\frac{104400}{c}(c-1.5)$

$\frac{104400}{c}(c-12)$

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$63580c + \frac{104400}{c}(c-1.5) = \frac{104400}{c}(c-12) + 72000$

$63580c^2 + 104400c - 156600 = 104400c - 1252800 + 72000c$

$63580c^2 - 72000c - 1096200 = 0$

$c = 4.8''$

$\varepsilon_{s1} = \frac{.003}{4.8}(4.8 - 1.5) = .00207 = \varepsilon_y$ $\qquad \varepsilon_y = \frac{60}{29000} = .00207$

$F_{s1} = 60\,ksi \Rightarrow 72\,Kips$

$\varepsilon_{s2} = \frac{.003}{4.8}(4.8 - 12) = -.0045$

$F_{s2} = -60\,ksi \Rightarrow -72\,Kips$ $\qquad a = .85c = .85(7.7) = 6.5''$

$\varepsilon_{s3} = \frac{.003}{4.8}(4.8 - 24) > \varepsilon_y$

$F_{s3} = -60\,ksi \Rightarrow -72\,Kips$

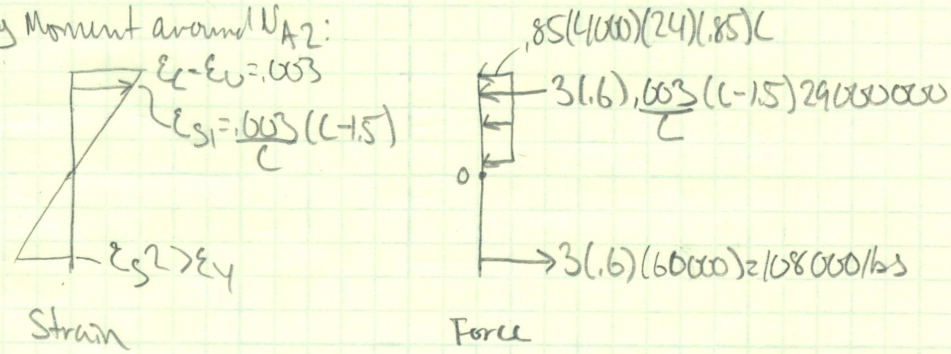
Concrete $= .85(4000)(22)(6.5) = 486.2\,Kips$

Pure bending moment around N_{A1}:

$M_0 = 486.2\left(12 - \frac{6.5}{2}\right) + 72(12 - 1.5) + 72(12 - 1.5)$

$M_0 = 480\,ft \cdot Kips$

Bending Moment around N_{A2}:



$.85(4000)(24)(.85)c + 3(.6)\frac{.003}{c}(c-1.5)29000000 = 108000$

$69360c^2 + 156600c - 234900 = 108000c$

$69360c^2 + 48600c - 234900 = 0$

$c = 1.5''$

Joshua Zolko | Tech Report 1 | Spot Checks | 7/9

$c = 1.5''$

$\varepsilon_{S1} = \frac{.003}{1.5}(1.5 - 1.5) = 0$

$F_{S1} = 0$ Concrete $= .85(4000)(24)(.85)(1.5)$

$F_{S2} = 108$ Kips $= 104$ Kips

Moment about 0 at N_{A2}:

$M_0 = 104\left(11 - \frac{.85(1.5)}{2}\right) + 108(11 - 1.5)$

$M_0 = 175.3$ ft·Kips

$479.2 > 175.3$ X 479.2 Kipft from negative moment on beam.

but 479.2 would include almost 1/2 DL, mixed with different calcs and Design methods, could account for drastic difference.

Joshua Zolko | Tech Report 1 | Spot Checks | 9/9

Caisson Check

Pure axial: (Assume negligible moment, as caisson is in ground.)

$P_n = 8 \cdot 400 \cdot 35.75 \cdot 21 + 30 \cdot 35.75 \cdot 21$

$P_n = 2424.9$ Kips (plans only mention a load of 1550K)

Plans call for 1000 psi concrete ⇒ $f'c = 1000$ psi

$84'' \phi \Rightarrow \pi 42^2 = A_c = 5539 in^2$

$.85 \cdot 1000 \cdot 5539 = 4708$ Kips > 2425 Kips ✓

second part of caisson below first is $54'' \phi \Rightarrow \pi 27^2 = A_c = 2289.06 in^2$

$.85(1000) \cdot 2289 = 1945.7$ Kips < 2425 Kips X fails, but works with 1550 Kip load marked on plans. Also, step created by shrinking of diameter + caisson is 85' deep with step at 63' deep.

Joshua Zolko Tech Report #3 Eccentricity check 1/1

Finding Center of Rigidity:

X-direction length: 260.3'

y-direction length: 95.75'

X coordinate of Center of Rigidity:

$5(0)+2(24.7)+4(27.4)+2(38.7)+6(59.6)+4(67.6)+4(88.6)+4(101.6)+$
$4(130.6)+4(151.6)+4(172.6)+4(193.6)+6(201.6)+2(221.7)+$
$4(232.9)+2(235.9)+5(260.3)=8576.7$

$\frac{8576.7}{66}=130'$

Y coordinate of Center of Rigidity:

$13(0)+6(21)+7(30)+8(37.4)+8(58.4)+7(65.8)+2(81.1)+4(83.5)$
$11(95.75)=3112.5$

$\frac{3112.5}{66}=47.2$

Center of Rigidity: (130.0, 47.2)

Center of Mass: (130.15, 47.9)

Since Center of Rigidity is very close to the Center of Mass, Moments due to torsion are assumed to be negligible.

Joshua Zolko | Tech Report #3 | Spot Check | 1/1

Relative stiffness from table for typical column: 1.5%

Maximum possible shear for a story occurs on first floor: 855.1 Kips

855.1 · .015 = 12.8 Kips → 12.8 · 13.7 = 175.3 Kip·ft

Moment acting on typical column for first floor: 175.3 Kip·ft

This moment is much lower than the Mp load of 480 ft·Kips.

Interaction with Axial load shows that this would not cause a problem

for the other direction:

2339 · .015 = 3.5 Kips → 3.5 · 13.7 = 48 ft·Kips

This value is also much lower than the bending in the other direction

of 175 Kips. Interaction would show no problems.

Joshua Zolko | Thesis | Expansion Design | 1/11

Top most Story height

Snow Load: 50 psf

Mechanical: 20 psf

Dead Load: 100 psf

Superimposed d: 15 psf

Total: $1.2(135)+50(1.0)=212 \rightarrow 215\,psf$

Bay Size: $31.75' \times 21' = 667\,ft\,sq$

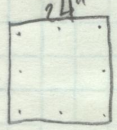
Axial load: $215 \cdot 667 = 143.4\,Kips$

Moment: $\frac{45.31^2}{11} = 393.1\,ft\,Kip$

Try a 24x24 column with f'c = 4000 and 8 #7's (TYP)

$A_s = 4.8\,in^2$ $f_y = 60\,Ksi$

24"



24"

$A_c = 24 \cdot 24 - 4.8 = 571.2\,in^2$

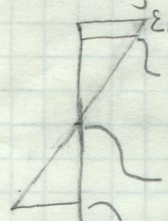
Pure axial:

$P_o = .85(4000)571.2 + 4.8(60000)$

$= 2230\,Kips > 143.4\,Kips$ ✓

$T_o = 4.8 \cdot 60000 = 288K$

Pure bending:



$\varepsilon_c = \varepsilon_u = .003$

$\varepsilon_{s1} = \frac{.003}{c}(c-1.5)$

$\varepsilon_{s2} = \frac{.003}{c}(c-12)$

$\varepsilon_{s3} > \varepsilon_y$

Strain

$.85(4000)(24)(.85)c$

$3(.60)\left(\frac{.003}{c}\right)(c-1.5)29000000 \rightarrow 114840$

$2(.60)\left(\frac{.003}{c}\right)(c-12)\,29000000 \rightarrow 76560$

$3(.60)(60000) = 79.2K$

Force

$69360c^2 - 156600 - 423400 = 0$

$c = 3.8"$

Joshua Zolko | Thesis | Expansion Design | 2/4

$\varepsilon_{S1} = \frac{.003}{3}(3.4-1.5) = .002$

$F_{S1} = 108$

$\varepsilon_{S2} \geq .207$

$F_{S2} = 72$

$\varepsilon_{S3} \geq .207$

$F_{S3} = 108$

Concrete $= .85(4000)(24)(.85)(3.8) = 2636$ K

$M_o = 2636(12 - \frac{3.8}{2}) + 108(12-1.5) + 108(12-1.5)$

$= 410.8 \text{ k-ft} > 393.1 \text{ ft·kip}$ ✓

Mid Height bracing

24"x24" beam supporting self weight + superimposed dead Load

Dead: 600 plf

Super: 15 psf ⇒ 15 · 21 = 315 plf

Total = 1.2(600+315) = 1100 plf

$m^+ = \frac{1.1(31)^2}{16} = 66 \text{ K·ft}$

$M^- = 1.1\frac{(31)^2}{11} = 96.1 \text{ K·ft}$

Try 2(#7) = 1.2 in² = A_s d = 24-1.5 = 22.5 $C = \frac{.88}{.85} = 1"$

$a = \frac{1.2(60000)}{.85(4000)(24)} = .88"$ $.003\left(\frac{22.5-1}{1}\right) = .0645 > .002$ ✓

$M_n = 1.2 \cdot 60000(22.5 - .44) = 132 \text{ K·ft} > 66 \text{ K·ft}$ ✓

For M^-, Try 2(#7) = 1.2 in² = A_s

$.85(4000).85(24)c^2 + 1.2(.003)(c-1.5)29000000 - 1.2(60000)c = 0$

$69360c^2 + 32400c - 156600 = 0$

$c = 1.3"$

3/11

Check A's is not yielding:

$\varepsilon'_s = \frac{.003}{5.8}(6-22.5) = .0085 > .002$; yielding

a = 0, using A's = 1.2; Reduce A's

Try 2 #6's → A's = .88

$a = \frac{(1.2-.88)60000}{.85(4000)24} = .235''$ $C = \frac{.235}{.85} = .276''$

$M_n = .88(60000)(21) + .85(4000)(.235)(24)(22.5 - \frac{.235}{2})$

$= 1108800 + 429207 = 128.2$ Kip·ft > 96.1 K·ft ✓

Check shear

$V_c = 2\sqrt{f'c}\,24 \cdot 24 = 72.9$ Kips (f'c → 4000) $R_A = R_B = 1.1 \cdot 31/2 = 17.05$ K

$\phi V_n = .5(.75)72.9 = 27.3$ Kips

$17.05 - 3 \cdot 1.1 = 13.75$

$V_s = 13.75/.75 - 72.9 =$ negative; no shear reinforcement required.

27.3 > 13.75, no shear reinforcement required §11.4.6.1

Check torsion

$(6.29 \cdot .5 \cdot 31 + (.375 \cdot .083 \cdot 31))\,130\,(14/12) = 13.1 = T_u = 157.2$ (K·ft, K·in)

$A_{cp} = 24^2 = 576$ $P_{cp} = 24 \cdot 4 = 96$ $\phi = .75$ $\lambda = 1.0$

$.75(1.0)\sqrt{4000}\left(\frac{(24^2)^2}{96}\right) = 163.9$ K·in > 157.2 K·in ✓ §11.5.1

Torsion is considered to be negligible ∴ no special design required.

Moment @ mid height is assumed to be negligible due to equilibrium from dead load → M=0

Axial Load

$143.4 + 1.2(\underbrace{7.2 + 55}_{\text{beam loading into column}}) = 218.0$ Kips

2230 > 218 ✓ no cross sectional change required

4/11

Assume a Load of 80psf → typ

$(1.2(150+15)+1.6(80))\frac{667}{1000}+218K = 435.4K$

Using current 24x24 cross section, $P_u \approx 2000K >> 435.4K$ ✓

PSF = 326 PSF

$31 \cdot 326 = 10.1 KLF$

$M^+ = \frac{10.1(21)^2}{16} = 321.2 K \cdot ft$

$M^- = \frac{10.1(21)^2}{11} = 467.2 K \cdot ft$

Assume a beam w/ b=72", h=12", d=10.5", For M^+

Try 6 #10's

$A_s = 6 \cdot 1.27 = 7.62 in^2$ $f_y = 60 ksi$ $f'_c = 4000 psi$

$a = \frac{7.62 \cdot 60000}{.85(4000)(72)} = 1.8"$ $c = \frac{1.8}{.85} = 2.2"$

Check yielding:

$.003\left(\frac{10.5-2.2}{2.2}\right) = .011 > .004 > .002$ ✓

$M_n^+ = 7.62 \cdot 60000(10.5 - \frac{1.8}{2}) = 365.7 K \cdot ft > 321.2 K \cdot ft$ ✓

For M^-:

Try 12 #10's

$A's = 12 \cdot 1.27 = 15.24 in^2$

Assume ε_s and $\varepsilon'_s > \varepsilon_y$

$a = \frac{15.24 \cdot 60000 - 7.62 \cdot 60000}{.85(4000)72} = 1.87"$ $c = 2.2"$

$\varepsilon'_s = \frac{.003}{2.2}(2.2-1.5) = .00095 < .002 \rightarrow$ Solve quadratic

5/11

Quadratic:

$.85(4000).85c(72) + 7.62\left(\frac{.003}{c}\right)(c-1.5)29000000 - 15{,}2460000 = 0$

$208080c^2 + 662940c - 1908810 = 0$

$c = 1.83''$ $a = 1.83 \cdot .85 = 1.56$

$M_n = 7.62 \cdot \left(\frac{.003}{1.83}\right)(1.83-1.5)(29000000)(9'') + .85(4000)(1.56)(72)(10.5 - \frac{1.56}{2})$

$M_n = 374.2\,ft\cdot Kips < 467.2\,ft\cdot Kips$ ✗

Try 15 #10's

$A_s = 15 \cdot 1.27 = 19.1\,in^2$

$208080c^2 + 662940c - 2137410 = 0$

$c = 2.0''$

$M_n = 7.62 \cdot \left(\frac{.003}{2.0}\right)(2-1.5)(29000000)(9) + .85(4000)(1.7)(72)(10.5-1)$

$= 453.8 < 467.2$ ✗

Try 15 #11's

$A_s = 15 \cdot 1.56 = 23.4\,in^2$

$208080c^2 + 662940c - 2398410$

$c = 2.16''$

$M_n = 7.62\left(\frac{.003}{2.16}\right)(2.16-1.5)(29000000)(9) + .85(4000)(1.836)(72)(10.5 - \frac{1.836}{2})$

$M_n = 510.8\,k\cdot ft < 467.2$ ✓

∴ Use 72"x12" beams w/ 15 #11's in M⁻ and 6 #10's in M⁺ TYP

Joshua Zolko Thesis Expansion Design 6/11

8th Floor Column

$(1.2(150+15)+1.6(80))\frac{667}{1000}+435.4K=652.7K$

Using Typ 24x24 column cross section; $P_U \approx 2000K >> 652.7K$ ✓

Slab/beams/loads remain Typ, Moments are OK by previous checks

7th Floor Column

$(1.2(150+15)+1.6(80))\frac{667}{1000}+652.7K=870K$

$P_U=2000K >> 870K$ ✓

Development Length:

Top floor bracing: K_{tr} assumed to be 0.

$M^+: L_d=\frac{3}{40}\frac{60000}{\sqrt{4000}}\frac{(1.0)(1.0)(1.0)}{\left(\frac{1.94+0}{.875}\right)}.875=29''$

$M^-: L_d=\frac{3}{40}\frac{60000}{\sqrt{4000}}\frac{(1.3)(1.0)(.8)}{2.2}.75=26''$

Typical beam:

$M^+: L_d=\frac{3}{40}\frac{60000}{\sqrt{4000}}\frac{(1.0)(1.0)(1.0)}{\left(\frac{2.15+0}{1.27}\right)}1.27=54''$

$M^-: L_d=\frac{3}{40}\frac{60000}{\sqrt{4000}}\frac{(1.0)(1.0)(1.0)}{\left(\frac{2.2+0}{1.41}\right)}1.41=65''$

Typical column:

$L_d=\frac{3}{40}\frac{60000}{\sqrt{4000}}\frac{(1.0)(1.0)(1.0)}{\left(\frac{1.94+0}{.875}\right)}.875=29''$

Joshua Zolko | Thesis | Expansion Design | 7/11

Slab: treat as foot wide section:

$1.2(150+15)+1.6(80)=326\,plf$

$M^+ = \frac{.33(31)^2}{16} = 19.8\,K\cdot ft$

$M^- = \frac{.33(31)^2}{11} = 28.8\,K\cdot ft$

Try W11 W.W.M. $A_s = .33$ $F_y = 65000\,psi$

$a = \frac{.33(65000)}{.85(4000)12} = .53''$ $\quad .003\left(\frac{10.5-.62}{.62}\right) = .048 > .002\checkmark$

$c = \frac{.53}{.85} = .62$

$M_n = .33(65000)(10.5-\frac{.53}{2}) = 18.3 < 19.8\ X$

Try W15 W.W.M. $A_s = .45$ $F_y = 65000\,psi$

$a = \frac{.45(65000)}{.85(4000)12} = .72$ $\quad .003\left(\frac{10.5-.84}{.84}\right) = .0345 > .002\checkmark$

$c = \frac{.72}{.85} = .84$

$M_n = .45(65000)(10.5-\frac{.72}{2}) = 24.7 > 19.8\,K\cdot ft\checkmark$

For $M^- = 28.8\,K\cdot ft$, Try W25 W.W.M. $A_s = .75$ $F_y = 65000\,psi$

$a = \frac{.75(65000)}{.85(4000)12} = 1.2''$ $\quad .003\left(\frac{10.5-1.41}{1.41}\right) = .0193 > .002\checkmark$

$c = \frac{1.2}{.85} = 1.41''$

$M_n = .75(65000)(10.5-\frac{1.2}{2}) = 40.2 > 28.8\,K\,ft\checkmark$

∴ use W15 spaced @ 4" at M+, W25 spaced @ 4" at M−.

Development length (WWM)

$l_d = .27\left(\frac{.15}{4}\right)\left(\frac{60000}{\sqrt{4000}}\right) = 10'' < 12''$; 12" min

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Deflections:

Beams:

$h_{min}: l/18.5 = 21/18.5 = 1.14 \cdot 12 = 13.6 > 12 \checkmark$

$w_D = 35 \cdot 165 = 5.8$ Klf

$w_l = 35 \cdot 80 = 2.8$ Klf

$M_d^+ = \frac{5.8(21)^2}{16} = 159.9$ K ft

$M_l^+ = \frac{2.8(21)^2}{11} = 77.2$ K ft

$M^+ = 237.1$ K·ft

$M_{sus}^+ = 159.9 + .3(77.2) = 183.1$ K·ft

$M_d^- = \frac{5.8(21)^2}{11} = 232.5$ K·ft

$M_l^- = \frac{2.8(21)^2}{11} = 112.3$ K·ft

$M^- = 344.8$ K·ft

$M_{sus}^- = 232.5 + .3(112.3) = 266.2$ K·ft

$f_r = .85(7.5)\sqrt{4000} = 404$ psi

$E_c = 57000\sqrt{4000} = 3591000$ psi

$n = \frac{29000000}{3591000} = 8.08$

$Kd = \frac{\sqrt{2(10.5)1.17+1} - 1}{1.17} = 3.47$

$y_t = 6''$

$B = \frac{72}{8.08(7.62)} = 1.17$

$I_g = 72(12)^3/12 + \cancel{72(12)(6-6)}^{0}$

$I_g = 10368$ in^4

$I_{cr} = 72(3.47)^3/3 + 8.08(7.62)(10.5-3.47)^2$

$I_{cr} = 4046$ in^4

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Deflections continued:

$M_{cr} = \frac{404(10368)}{6 \cdot 12000} = 58.2 \text{ k-ft}$

$M_{cr}/M_d = 58.2/159.9 = .364 \rightarrow .364^3 = .048$

$(M_{cr}/M_{sus})^3 = (58.2/183.1)^3 = .032$

$(I_e)_{sus} = (.032)(10368) + (1-.032)4046$ $(I_e)_d = .048(10368) + (1-.048)4046$

$(I_e)_{sus} = 4248.3 \text{ in}^4$ $(I_e)_d = 4349.5 \text{ in}^4$

$(M_{cr}/M_{d+l})^3 = (58.2/237.1)^3 = .0148$

$(I_e)_{d+l} = .0148(10368) + (1-.0148)4046$

$(I_e)_{d+l} = 4139.5$

$K = 1.2 - .2(2) = .8$

$\Delta_i)_{d+l} = \frac{.8(5/48)237.1(21)^2(1728)}{3591(4140)} = 1.01''$

$\Delta_i)_d = \frac{.8(5/48)159.9(21)^2(1728)}{3591(4350)} = .65''$

$\Delta_i = \Delta_{d+l} - \Delta_{id} = 1.01 - .65 = .36''$

$\Delta_{iLs} = .3(.36) = .12''$

$\Delta = .65 + .36 + .65 \cdot 2 + .12 \cdot 2$

$\Delta = 2.55 \text{ in}$

$\Delta_L = .36 + .24 = .6 < 1.4 = \frac{21 \cdot 12}{180}$ ✓ (Table 9.5(b))

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Deflections continued

Slab: (1 foot section)

$w_D = 165\text{plf}$

$w_l = 80\text{plf}$

$M_d^+ = \frac{.165(35)^2}{16} = 12.6\text{k}\cdot\text{ft}$

$M_l^+ = \frac{.08(35)^2}{16} = 6.13\text{k}\cdot\text{ft}$

$M^+ = 18.7\text{k}\cdot\text{ft}$

$M_{sus}^+ = 12.6 + .3 \cdot 6.13 = 14.4\text{ k}\cdot\text{ft}$

$f_r = 404\text{psi}$

$E_c = 3541000\text{psi}$

$n = 8.08$

$D = \frac{12}{8.08(.45)} = 3.3$

$Kd = \frac{\sqrt{2(10.5)3.3+1} - 1}{3.3} = 2.24$

$y_t = 6$

$I_g = 1728\text{ in}^4$

$I_{cr} = 12(2.24)^3/3 + 8.08(.3.3)(10.5 - 2.24)^2$

$I_{cr} = 1864\text{ in}^4$

$M_{cr} = \frac{404(1728)}{6(12000)} = 9.7\text{k}\cdot\text{ft}$

$(M_{cr}/M_d)^3 = (9.7/12.6)^3 = .456$

$(I_e)_d = .456(1728) + (1 - .456)(1864)$

$(I_e)_d = 1801\text{ in}^4$

Joshua Zolko Thesis Expansion design 11/11

Deflections Continued:

$(M_{cr}/M_{sus})^3 = (9.7/14.4)^3 = .306$

$(I_e)_{sus} = .306(1728) + (1-.306)(1864)$

$= 1821.8\ in^4$

$(M_{cr}/M_{d+l})^3 = (9.7/18.7)^3 = .140$

$(I_e)_{d+l} = .140(1728) + (1-.140)(1864)$

$= 1845\ in^4$

$$\Delta_{id+l} = \frac{.8(5/48)18.7(35)^2(1728)}{3591(1845)}$$

$= .498\ in$

$$\Delta_{id} = \frac{.8(5/48)12.6(35)^2(1728)}{3591(1821)}$$

$= .344\ in$

$\Delta_{il} = .498 - .344 = .154\ in$

$\Delta_{ils} = .154 \cdot .3 = .051\ in$

$\Delta = .344 + .154 + .344 \cdot 2 + .051 \cdot 2 = 1.288$

$\Delta_L = .154 + .051 \cdot 2 = .256 < \frac{35 \cdot 12}{180} = 2.3$ ✓ (table 9.5(b))

Joshua Zolko Thesis Existing design Check 1/2

Initial axial Column Check (to be used in conjunction w/ Ram Model output)

Typical column: 22"x24" (# of rebar varies w/ floor)

P=870K @ bottom of expansion column.

(22·24 − 6.6)(.85)(4000) + 36·60000 = 2000K

@ 7th floor, P = 870 < 2000K ✓

@ 6th floor, P = 870 + 294 = 1164K < 2000K ✓

Column @ 5th floor:

(22·24 − 10·.79)(.85)(4000) + 7.9·60000 = 2242K ✓

@ 5th floor, P = 1164 + 294 = 1458K < 2242K ✓

@ 4th floor, P = 1458 + 294 = 1752K < 2242K ✓

Column @ 3rd floor:

(22·24 − 12·1.56).85(4000) + 12·1.56·60000 = 2855K

@ 3rd floor, P = 1752 + 294 = 2046K < 2855K ✓

@ 2nd floor, P = 2046 + 294 = 2340K < 2855K ✓

@ 1st floor, P = 2340 + 294 = 2634K < 2855K ✓ — May cause problems with Moment/axial interaction

Column @ Ground level:

(24·26 − 14·1.56).85(4000) + 14·1.56·60000 = 3358K

@ Ground level, P = 2634 + 294 = 2928K < 3358K ✓

Caisson check: (f'c = 1000 psi

.85 · 1600 · π 42² = 4708K > 2928K ✓

No changes required due to pure axial loads

Joshua Zolko | Thesis | Existing Design Check | 2/2

Assume rebar near NA have minimal impact on Moment resistance

$.85(4000)(24)(.85)c$

$5(1.56)\left(\frac{.003}{c}\right)(c-1.5)29000000$

$\rightarrow 5(1.56)(60000)=468K$

$69360c^2+678666c-1485900=0$

$c=1.84''$

Steel $=125.4K$

Concrete $=127.6K$

$M=125.4(21)+127.6\left(22.5-\frac{.85\cdot 1.84}{2}\right)=450\ ft\cdot K$

$M_L=112.3$

$\frac{112.3}{450.4}=.249$

Int:

2nd $\frac{2340}{2855}=.82+.249<1$ (requires additional concrete)

1st $\frac{2634}{2855}=.922+.249<1$ (requires additional concrete)

G $\frac{2928}{3358}=.872+.249<1$ (requires additional concrete)

Check LL reduction

$.25+\frac{15}{\sqrt{K_{LL}(A)}}$ $\quad A=35\cdot 21=735$

$.25+\frac{15}{\sqrt{4(735)}}=.53$ (applies to top 3 floors only, LL < 100psf)

.20 for 125psf.

Dead: $1.2(9)(150)(735)=1190.7K$

RLL: $1.6[(3)(80)(.53)+(5)(125)(.8)]735=737.6K$

$P=1190.7+737.6=1928.3K$

Joshua Zolko | Thesis | HVAC | 1/2

High: 103-70 = 33 = Δ_H

low: -22-55 = 77 = Δ_C

Insulation:

	R	Δ_H	Δ_C	
4 1/2" limestone	.8			
2" insulation	8			
7 5/8" CMU	1.11			
2" insulation	8			
5/8" GWB	.56			
Total:	18.47	1.8	-4.2	
Glazing:				
double pane:	2	16.5	38.5	
Triple pane:	3.23	10.2	23.8	(use triple pane)

Surface Area:

37·90·2+37·300·2 = 28860 (Half is glazing)

Cooling

14430·1.8 = 26000
14430·10.2 = 157000

175000 BTU/HR

Heating

14430·4.2 = 60000
14430·23.8 = 340000

400000 BTU/HR

People Load:

Occupancy:

1 person per 20 sq ft, 20 CFM per Person

500 BTU/HR · 4050 = 2m/HR

@ 90·300 → 4050 in 3 stories = 81000 CFM

↳ .06 CFM/sq ft = 4860 CFM

86000 CFM Total.

Cooling: 1.08·86000(103-70) = 3.1m/HR

Heating: 1.08·86000(55+22) = 7.2m/HR

Joshua Zolko | thesis | HVAC | 2/2

Total Loads

Cooling: 3.1 + .18 + 2 = 5.3 M

Heating: 7.2 + .4 − 2 = 5.6 M

→ Requires an 8 row 35 WS Serpentine @ 13 FPI From McQuay

8600 CFM Through 10X18 Duct = 478 FPM.

Joshua Zolko | Thesis | Acoustics | 1/1

Expected sound levels: volume of room = 22·90·120 = 238000 ft^3

100dB for top story

80dB for other floors

sabins before treatment: 400

Sabins after treatment: 19660

Noise Reduction:

$10\log\frac{19660+400}{400} = 17\,db$ Top floor

$100\,dB - 17\,db = 83\,dB$

$10\log\frac{11222.8+400}{400} = 15\,dB$ all other floors

$80-15 = 65\,dB$

Reverb time = .6 s