Existing Typical Floor Plan

The typical floor plan of the Gen*NY*Sis Center for Excellence in Cancer Genomics (CFG) consists of mainly laboratories and offices with an atrium in the center. The hallway was designed to have a minimum clearance of 9’-6” throughout the whole building. Displayed at the right in Figure 1 is the breakdown of the sections of the building used for construction of the project. Figure 2 shows the location of the offices with a view from the curtain wall façade along the North side of the building in sections II and III, while section I contains only one floor for a seminar room and sections IV and V are laboratories, classrooms, conference rooms and storage rooms. The green represents laboratories and classrooms, the blue is offices, red is stairwells and elevators, yellow is corridor space, and gray is mechanical rooms.

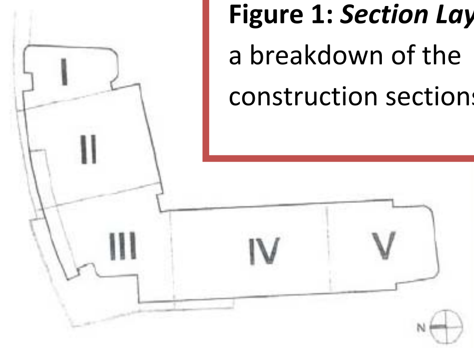


Figure 1: ***Section Layout*** a breakdown of the construction sections

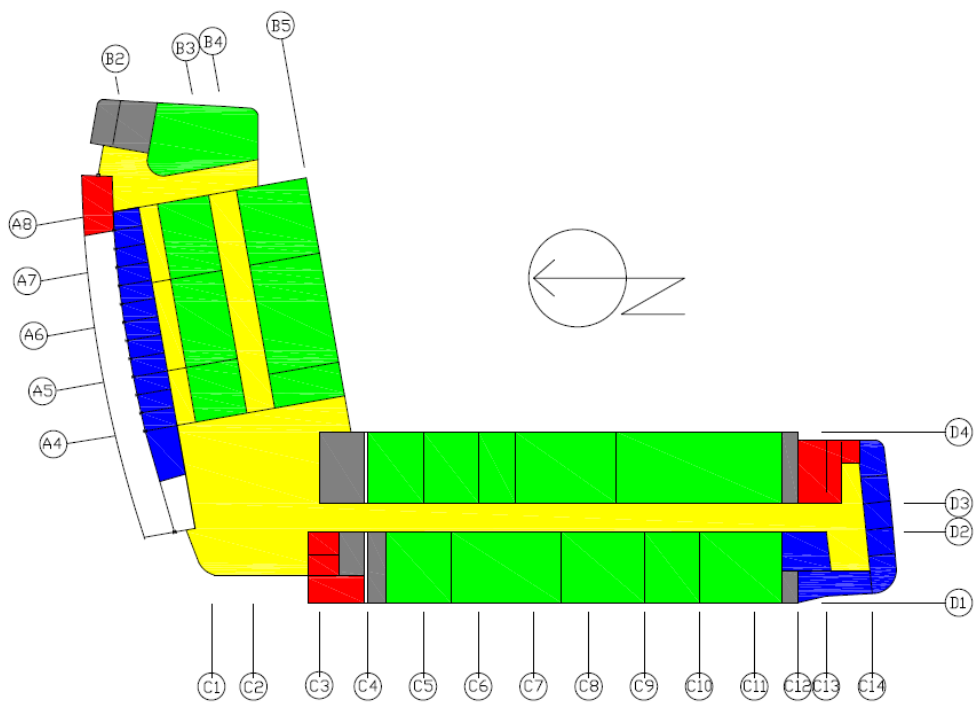


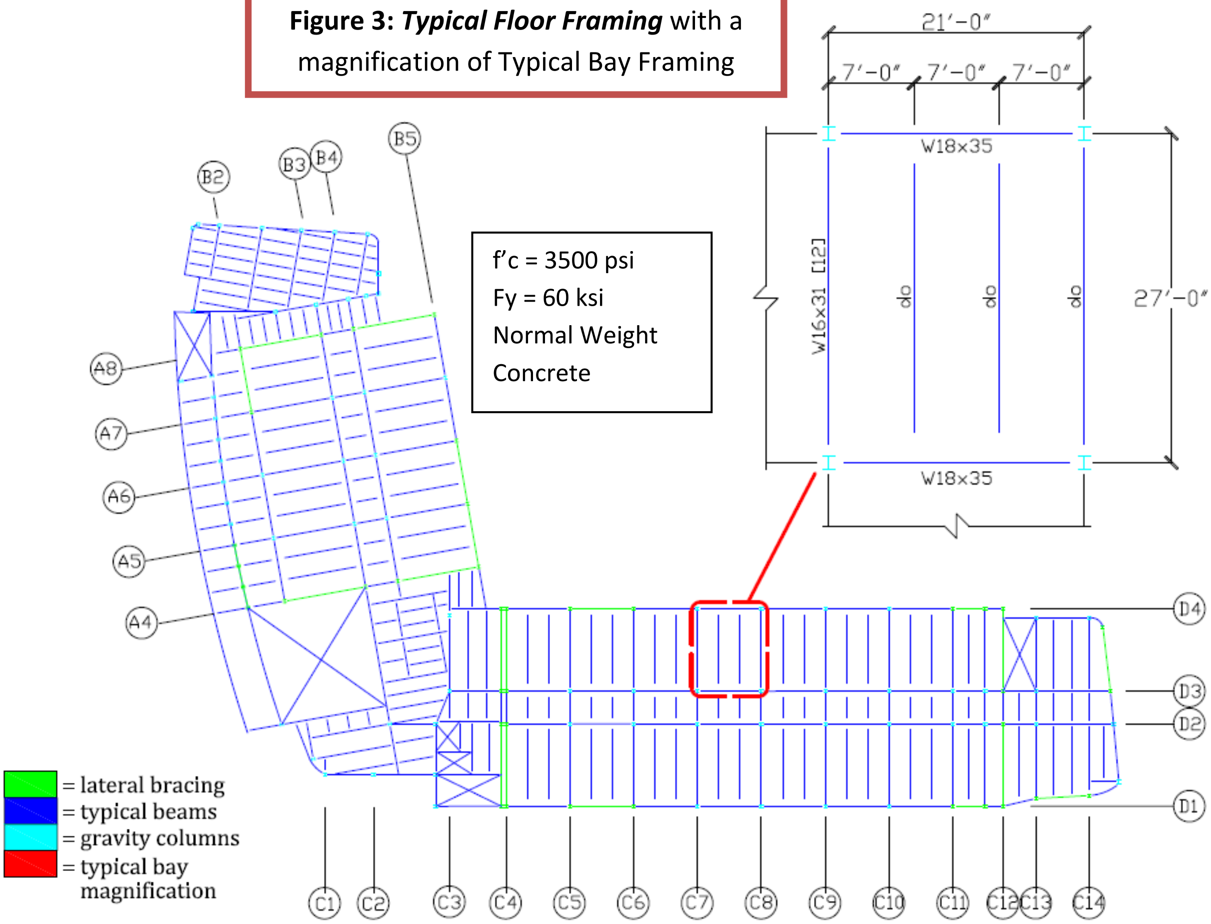
Figure 2: ***Architectural Room Layout*** Green is labs and classrooms, blue is offices, yellow is corridors, red is stairways and elevators, gray is mechanical rooms

Floor Framing

The structural layout is displayed in Figure 3 with typical beams in blue, gravity columns in cyan, and lateral bracing in green. A section of the structural grid is magnified to show a typical bay with dimensions and beam sizes. The typical floor system consists of composite metal decking which spans the north-south direction across sections IV and V and east-west across sections I, II and III. Typical floor framing includes 2-inch, 20-gauge, galvanized composite metal deck with 4½-inches normal weight concrete (total slab thickness of 6½-inches) with 6x6-W2.9xW2.9 wire welded fabric. Normal weight concrete was chosen over lightweight for vibration control. The structural steel used has a weight of 8 psf of floor area. Typical floor beams are W16x31 spaced 7-feet apart with 20 shear connectors. Filler beams across the 10-foot corridor are W10x12 spaced 7-feet apart. Girders along the interior column lines and along the exterior walls are W18x35 with 32 shear connectors. Camber will not be accounted for due to relatively short spans. Atypical framing is located in the lobby and offices along the North wall. Transfer girders are required in the lobby and mechanical rooms along the North wall to maintain column-free areas. Offices along the North wall are cantilevered over columns along the First Floor terrace.

Figure 3: ***Typical Floor Framing*** with a magnification of Typical Bay Framing

f'c = 3500 psi
Fy = 60 ksi
Normal Weight Concrete



Foundation

The geotechnical report indicates that the allowable bearing capacity is 4000 psf. Typical column footings are 9-feet square and 25-inches deep calling for (11)#9 reinforcing bars each way on the bottom. Typical continuous wall footings are 1-foot deep by 2-feet wide calling for (3)#5 continuous bars and (1)#5 bar at 12-inches on center, transverse. The 20-inch thick basement walls retain 20-feet of soil (see diagram for reinforcement). Typical slab-on-grade is 5-inch thick with steel fiber reinforcement. The mechanical room slabs are 6-inch thick with steel fiber reinforcement. All steel fibers in slab-on-grade are at 30 pounds/cy. Weights for cast-in-place concrete, footings, foundation walls and piers, and slabs on metal deck are 4000 psi, 3000 psi, 4000 psi, and 3500 psi, respectively.

Figure 4: *Typical Column Pier* and foundation layout

Roof

To satisfy the extra HVAC loading on the roof, a concrete slab is set on the metal deck framing that is supported by steel beams. The 6½-inch slab is on 2-inch, 20-gauge, galvanized composite metal deck with 4½-inches of normal weight concrete reinforced with 6x6xW2.9xW2.9 wire welded fabric. Roof framing supports a screen wall set back from the face of the building, extending 15 to 20-feet above the roof slab. Typical roof framing filler beams are W16x31 spaced 7-feet apart with 20 shear connectors. Deeper beams will be required at bearing points of the penthouse posts. Filler beams spanning the corridor bay will be W10x12 spaced 7-feet apart with no shear

Figure 5: *Penthouse Mechanical Screen* with structural tube braces

connectors. Girders along the interior column lines and along the exterior walls will be W18x40 with 32 shear connectors. The structural steel used in the Main Roof framing is 10 psf of roof area. Penthouses on the roof have cross-braced steel-frames supporting steel joists and 1½", 22-gauge, galvanized, wide-rib (type B) roof deck. The structural steel used in the Penthouse Roof framing is 5 psf of penthouse area.

Columns

Typical columns are W12x72 members at the lower tier and W12x53 members at the top tier. Using W12 columns as a minimum size simplifies fabrication of connections of beams framing into the columns and allows the OSHA-required four anchor bolts to fit within the flanges at the base. This minimizes both base plate and pier sizes. A column splice with a bolted web and welded flanges is required 4-feet above the Second Floor for all columns. Perimeter columns will bear on piers 1-foot below the First Floor elevation of 195.0'. Interior columns will bear on footings 1-foot below the Ground Floor elevation of 175.0 feet.

Lateral Force Resisting System

Steel braced frames, shown in Figure 6, will resist wind and seismic lateral loads. An expansion joint at the intersection of the two building wings will isolate the two sections from each other. The expansion joint will require a row of columns along each side of the joint, with the building structures separated by a distance sufficient to provide seismic isolation—approximately 6 to 8-inches. Each building section has braced frames across the ends, and two bays of bracing along the length of each exterior wall. Bracing diagonals are tube-shaped steel members in non-moment-resisting eccentrically braced frames. The building is designed for wind loading drift criteria of H/400, including second order effects.

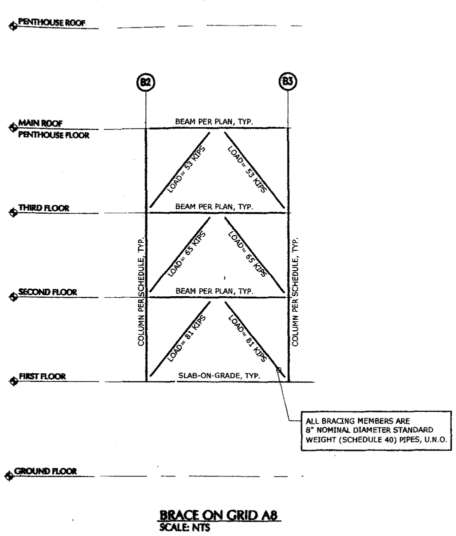


Figure 6: ***Typical Lateral Brace***

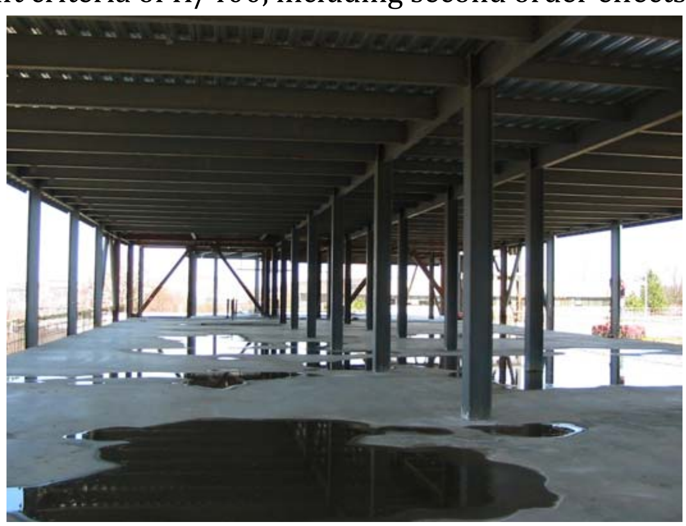


Figure 7: ***Floor Elevation*** of a typical bay size

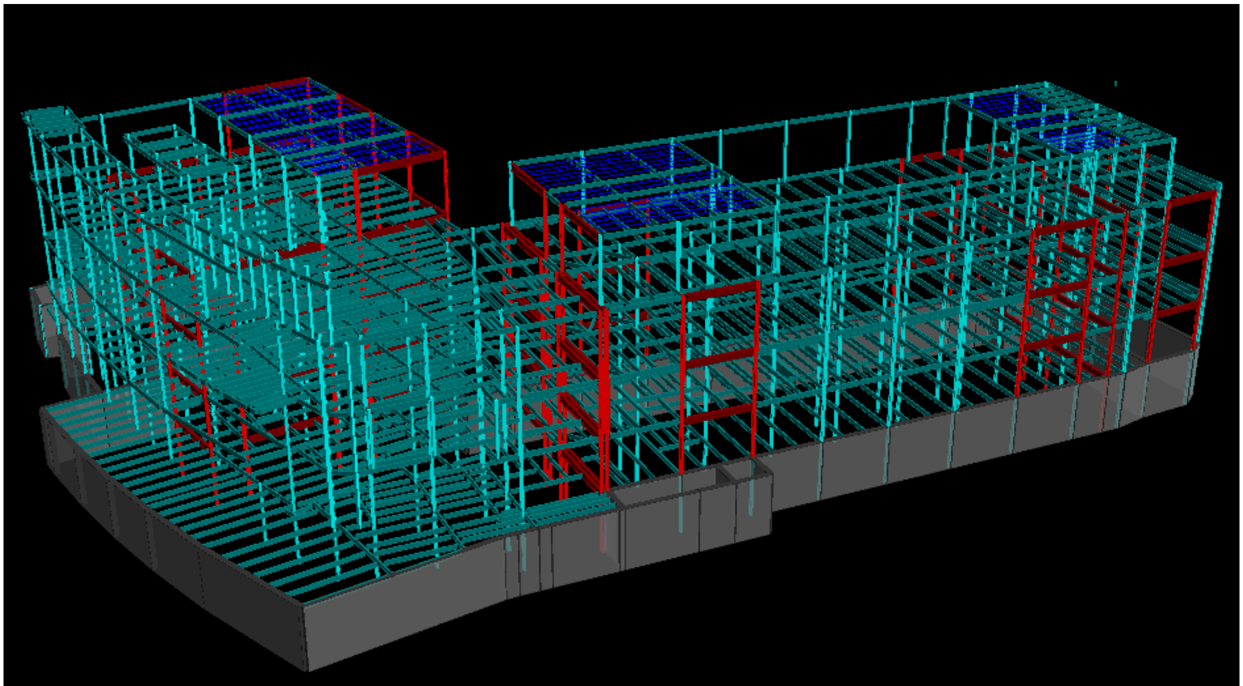


Figure 8: ***RAM 3-D View of Structural System with Lateral Bracing Highlighted* (Northwest corner)**

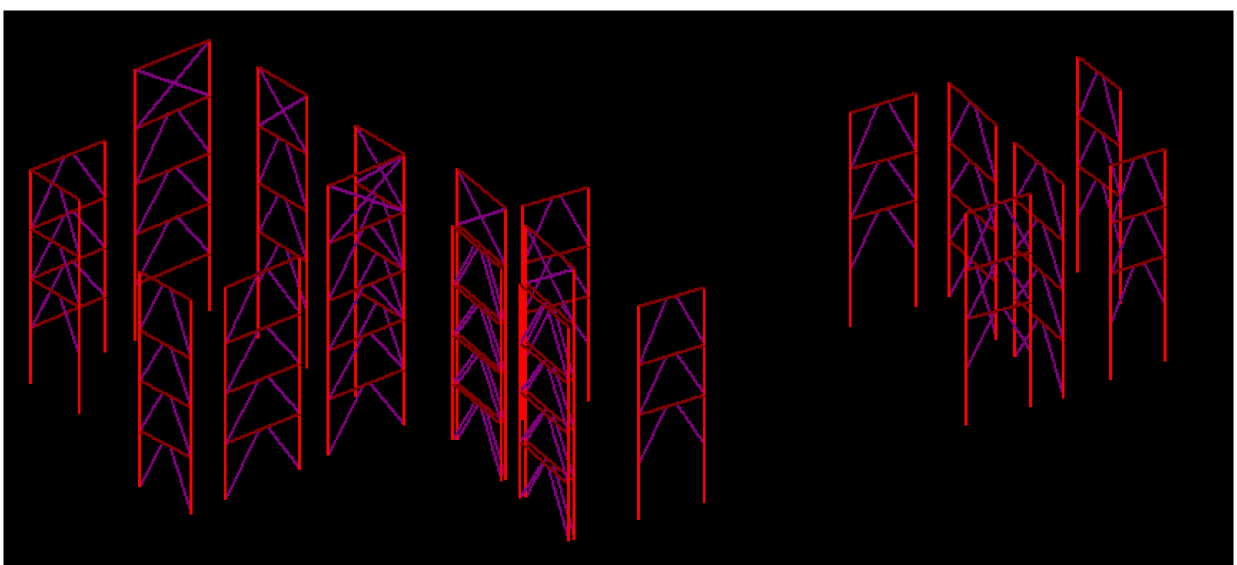


Figure 9: ***RAM 3-D View of Lateral Braces***

LRFD Load Combinations

1.4(Dead)
1.2(Dead) + 1.6(Live) + 0.5(Roof Live or Snow)
1.2(Dead) + 1.6(Roof Live or Snow) + 1.0(Live) or 0.8(Wind)
1.2(Dead) + 1.6(Wind) + 1.0(Live) + 0.5(Roof Live or Snow)
1.2(Dead) + 1.0(Seismic) + 1.0(Live) +0.2 (Snow)
0.9(Dead) + 1.6(Wind)
0.9(Dead) + 1.0(Seismic)

Figure 10: ***Elevation from the South***

Original Design Loads

Construction Dead Load	
Concrete	150 pcf
Steel	490 pcf
Dead Load	
Partitions	20 psf
M.E.P.	10 psf
Finishes	5 psf
Windows and Framing	20 psf
Roof System without slab	30 psf
Roof System with slab	85 psf
Typical Elevated Floor System	85 psf
Elevated Terrace Floor System	170 psf maximum
Live Loads	
Office/Laboratory flexibility	70 psf

Lobbies and first floor corridors	100 psf
Corridors above first floor	80 psf
Stairs and Exits	100 psf
Seminar Room	100 psf
Balcony/Terrace	100 psf
Mechanical Penthouse	200 psf
Roof Live Load/Roof Snow Load	
Ground Snow Load, p_g	65 psf
Flat-roof Snow Load, p_f	50 psf
Snow Exposure Factor, C_e	1.0
Snow Load Importance Factor, I	1.1
Thermal Factor, C_t	1.0
Wind Load	
Basic Wind Speed (3-sec gust), V	90 mph
Building Category	II
Wind Importance Factor, I	1.15
Wind Exposure Category	B
Internal Pressure Coefficient, GCp_i	±0.18
Height and Exposure Adjustment Coefficient, λ	1.16
Component & Cladding Design Wind Pressure	30 psf
Seismic Load	
Seismic Use Group	II
Importance Factor	1.0
Spectral Response Acceleration, S_s	0.220
Spectral Response Acceleration, S_1	0.076
Site Class	C
Site Class Factor, F_a	1.2
Site Class Factor, F_v	1.7
Spectral Response Acceleration, S_{MS}	0.264g
Spectral Response Acceleration, S_{M1}	0.129g
Spectral Response Coefficient, S_{DS}	0.159
Spectral Response Coefficient, S_{D1}	0.073
Seismic Design Category	B
Response Modification Factor, R	7.0
Nonmoment-Resisting Eccentrically Braced Frames	
Seisimc Period Coefficient, Ct	0.03
Seismic Response Coefficient, Cs	0.0251 sec
Period Coefficient, x	0.75

Problem Statement

Receiving a grant in September of 2002, the University at Albany was given $45 million to create the cornerstone to New York State's Gen*NY*Sis (Generating Employment Through New York Science) program, which is an initiative by government and private investors to lure jobs in life science into New York. Given the location of the building near Albany, NY, it makes sense to use a structural steel system. Also, the foundation loads were not able to stand the heavy weight of a concrete system. Furthermore, a fast-track method was desired and a concrete system generates a longer construction period.

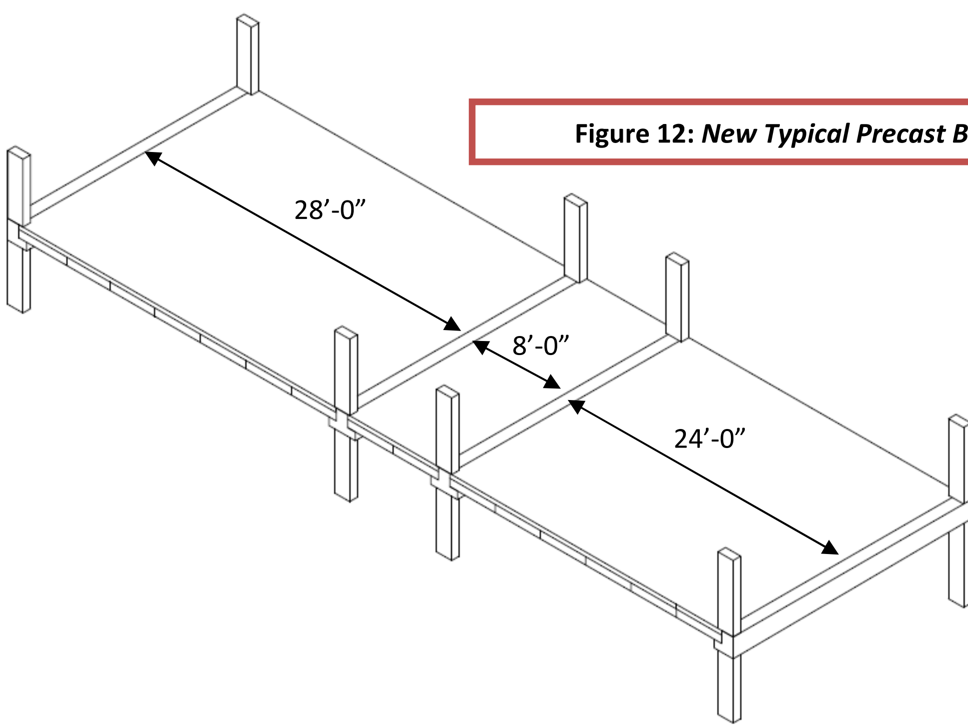
However, steel decking is not always the ideal situation for vibration control, which is important for a laboratory that deals with cell experiments. A concrete system is much more ideal for this type of building. Therefore, a redesign in a new location, at a new site with a different soil bearing capacity might be able to handle such a structure. For the purposes of this research, the building has been moved from Rensselaer, NY to the Penn State University Hershey Medical Center to fulfill different design criteria. In addition, the fast track delivery can still be utilized with the use of precast panels. Because of this elimination of steel, a new lateral design must be employed. Also, to go along with the precast panels, concrete shear walls are a good compliment to resist the building's lateral loads. Furthermore, the effect on the vibration control will be investigated and compared in the original composite steel deck and the precast concrete. Because of the heavier structure, a look into the changes needed in the foundation will be conducted.

Figure 11: ***Map of Location Change***

Preliminary Redesign

The redesign with concrete began with a recalculation of the wind loads and the seismic loads to see what condition controls in Hershey, PA. With this recalculation, it was determined that the new wind load controlled, as seen in Appendix D. An addition of an inhabitable green roof made the gravity columns and beam capacities change as well. The new roof load needed to include a roof with assembly live loads, green roof live loads, and the saturated soil weight of the proposed green roof. To further make the penthouse a place of refuge, the original mechanical screen was removed to allow a better view of the surroundings. Also, all staircases and the elevators were extended to 18’-0” above the Main Roof level to provide egress to the roof. The use of lightweight concrete was an option in preliminary design to represent the use of fly ash, which improves the workability of concrete by decreasing its water demand, reducing segregation and bleeding and lowering the heat of hydration. However, it was eliminated based on some more criteria seen later in the vibration section of this report. Once the wind and seismic loads were determined, seen in Appendix D, the PCI Industry Handbook, 6th Edition was used to size hollow-core precast planks. Upon further calculations, it was decided that it would be best to use a hollow-core plank with a 4” topping to further dampen the vibration effects on the laboratory equipment. The edge beams are made up of 20LB24 L-beams and T-beams of 28IT20 which can be seen in Appendix E.

Figure 12: ***New Typical Precast Bay***



The original layout of the CFG has rather small bay sizes most likely due to a better control of the vibration frequencies. In an attempt to minimize the architectural layout changes, the bay sizes were kept close to the original. Since the precast panels are cast in increments of 4’-0” wide, minor changes to the floor plan was necessary. It was necessary to keep the hallway width to at least 60”. The hallway was recreated to be 96” wide, which is slightly narrower than the original. Also, the bay sizes changed to 20’-0” by 24’-0” and 20’-0” by 28’-0”. The column lengths remained the same with a penthouse level bringing the overall height to 90’-0”. In section III, the typical bay size is not used. Part of the grid is laid out at an angle presenting a challenge for how to lay down flooring in a typical size. Therefore, a few unique panels would need to be ordered.

New Design Loads

To begin the new RAM model, the dead loads and live loads were calculated from the roof load first. To try and keep the bay sizes low, the span was set at 21’-0” to stay consistent with the old architectural plan. Once the load and moment of the roof and penthouse were calculated, a hollow-core plank was selected from PCI Industry Handbook, 6th Edition, see Appendix E. Once the planks were chosen, the weight was divided up into tributary areas in the 21’-0” spans and the L-and T- beams were selected to hold up the weight of the slab. Once the appropriate sizes were approximated, the total weight was tabulated and divided up amongst the tributary areas to be loaded through the columns. From there it was determined that an overall use of a 20” x 20” column would be most appropriate. To model the precast in RAM, the Concrete Beam analysis was set to only use #8 to reinforce the slab and beams since those are the sizes that are used in the prestressed strands of the precast planks. With an estimated size for each part at each level, the loads were then carried down and the foundation could be resized. To help carry these loads and

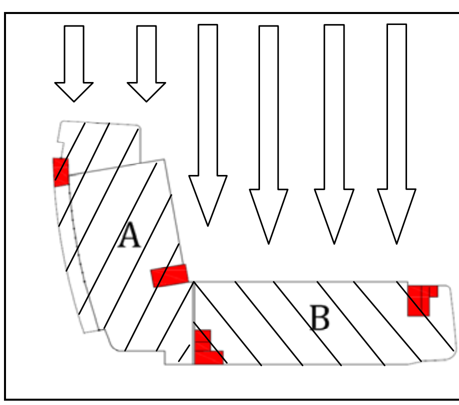


Figure 13: ***Distribution of Lateral Forces to the building frame***

to enclose the interior, structural insulating panels were chosen to use as the load bearing wall system. In particular, Murus foam core SIPs not only can take the place of for instance a metal gauge studs and drywall system but they also provide better acoustical and temperature characteristics which could be very helpful for a building full of lab experimenting. See Appendix F for more spec information.

Lateral Framing

Figure 14: ***Allowable and Actual Story Drift***

Floor	Story Height (ft)	Allowable Story Drift (in)	RAM Story Drift (in)		
Penthouse	18.42	0.553	0.134	✓	1
Roof	18.58	0.557	0.121	✓	1
3rd	16	0.480	0.084	✓	1
2nd	16	0.480	0.081	✓	1
1st	18	0.540	0.036	✓	1

To go along with the precast panels, concrete shear walls were designed to take on the new lateral loading and continue with the concrete theme. Since the building location moved from Rensselaer, NY to Hershey, PA, the wind and seismic loads changed. Also, upon further inspection, the building was originally designed so that there is a disconnection between sections III and IV. Therefore, the analysis of the loading was re-calculated with this in mind so that there are two diaphragms with separate loading cases. This was modeled in RAM by laying out two different slabs around Building A and Building B. Also with the new dimensions and variables, the new main roof height was increased to 67'-0" as opposed to the 30'-0" originally used by the engineer. Also, the wind and seismic direction were as displayed in Figure 13. The main differences between the original system and the new system design loads are shown in Appendix D. In the new situation, the wind load controls which was not the case for the original system, which makes drift more of a concentration than before.

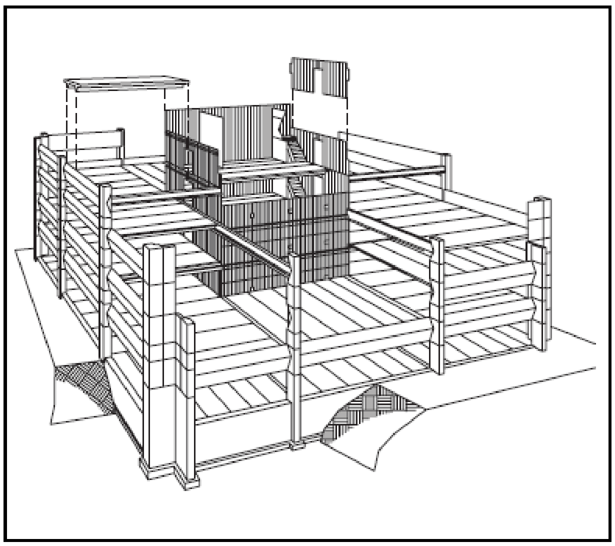


Figure 15: ***Interior Shear Wall System*** as shown and defined by the PCI Handbook, 6th Edition

To firm up the diaphragm it makes sense to use cast-in-place concrete shear walls. Even though it is a slower method, pouring concrete is most likely a faster method than placing a CMU wall. Another method is to use precast load bearing walls for the shear walls, which is also an option that was looked into. However, that was dismissed because it would make the shear wall elements to reliant on the rest of the structure and thus lose the continuity. Since there are already plenty of vertical egress components available, it makes sense to continue them up through the roof and make them the shear wall components. Another stairwell was added as access to the roof for the green roof but also to take on more of the lateral forces. With the addition of the shear walls, the center of rigidity changed relative to each Building Section. However, this data was inputted into RAM as four separate diaphragms at the Penthouse level and then breaks down into the two Building Section diaphragms. Therefore, the center of rigidity is off and the hand calculation was used to size the shear wall. To simplify the shear walls, only the continuous walls were considered to take lateral forces, so there were no wall openings accounted for, and no coupling beams were designed. Each shear wall was designed with a boundary element to increase the amount of shear capacity withheld. Displayed below is the drift of one of the more severe shear walls. For shear wall calculations see Appendix G.

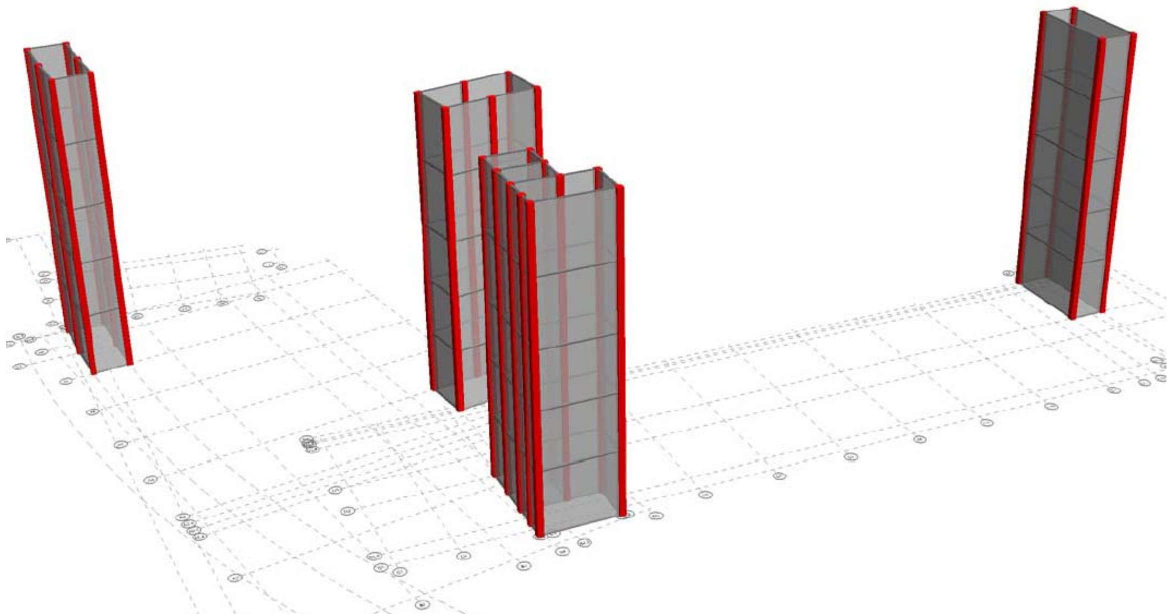


Figure 16: ***RAM 3-D View of Lateral Walls***

Vibration Analysis

Another factor in design was the effect that vibration of the floor. Because this building is used for very precise medical experimentation with sensitive equipment, vibration can be considered as a key piece of design guideline. Concrete is more of solid and dense material so it would make sense that it would have a smaller effect from vibration compared to the original steel design. Unfortunately, there is very little information as for how to calculate vibration in concrete while there is a full steel design guide for the analysis of vibration caused by footsteps. However, a paper entitled "Vibration of Precast

Prestressed Concrete floors" by Robert F. Mast, was also used to help determine the vibration figures to compare with steel. Figure 17 shows the results of this analysis.

		Steel System	Concrete System
	fn	7.85	5.62
Vibration Velocity at:	Fast Walking	8870	5204
(micro in/ sec)	Moderate Walking	1951	1145
	Slow Walking	532	312

Figure 17: ***Comparison of Vibration Analysis***

Looking at the table, it can be seen that actually while the concrete system can withstand the vibrations at one more severe of a level than the original steel system could. The concrete system is able to handle sensitive equipment up to electron microscopes at 30,000x magnification, while the steel system can only safely use bench microscopes up to 400x magnification. This analysis also shows that the best environment for the laboratories is one in which there is predominantly slow walking and absolutely no running. STAAD was also used to analyze this and Figure 18 shows those results of the natural frequency felt amongst continuous spans of the typical bay size. Another option that was investigated was the use of lightweight concrete for purposes of sustainable design. However, it was deemed that lightweight concrete was a little too light and registered a natural frequency that went below 5 Hz, which is the limit to which this vibration analysis can be used with.

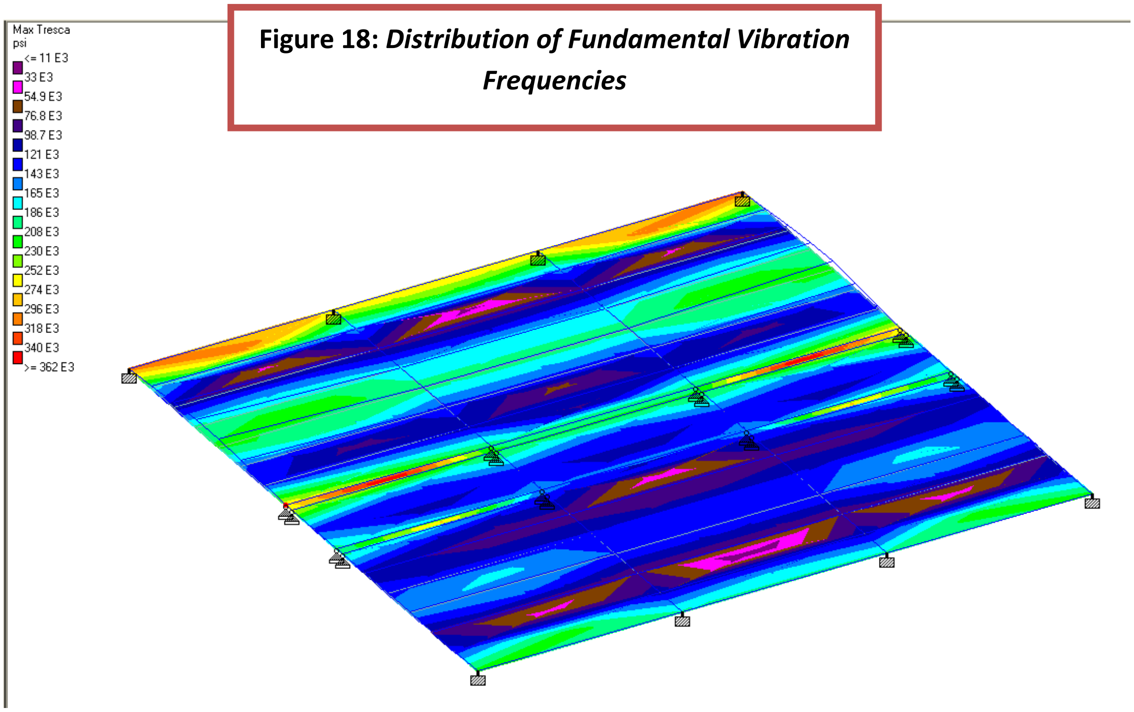


Figure 18: ***Distribution of Fundamental Vibration Frequencies***

Foundation Redesign

Because the overall weight of the system is much heavier with the massive amounts of concrete being added, a foundation redesign was in order. Only the worst case scenario shear wall was recalculated and designed with a footing foundation. The bearing capacity of the soil in Hershey, PA was estimated based on the soil reports on the USDA records available online. It was assumed to be much lower than the bearing capacity available in Albany, NY. Sturdier foundations are needed, and the option of caissons was looked into.