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

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**Executive Summary**

A thorough and exhaustive analysis was conducted on the Milton S. Hershey Medical Center Biomedical Research Building. A RAM model was created, while attempting to adhere as close to the original building as possible for the most accurate results. To supplement the model, it was found that the maximum eccentricity was at 9”, due to slight irregularity in the column layout. This eccentricity only exists along the short axis of the building. Torsion was allowed to be neglected, as it was negligible, and this is supported by the model output file. An overturning analysis was conducted, and it was found that overturning from seismic forces controlled along the long axis of the building, and wind controlled along the short axis of the building. Both forces were both significantly less than the resisting moment from the weight of the building. Shear forces that acted upon the building were found from the 4 wind load cases found in ASCE 7-05, and found that wind load case 1 controlled overall for wind loads, 855 kips at the lowest floor along the short axis, 233, and seismic only controlled along the long axis of the building. Stiffness was found for the columns of the building. It was assumed that relative stiffness would not change along either access, and a stiffness check done in the RAM model through the application of a 1kip load in both directions validates this assumption. It was found that each column resists about 1.5% of the shear force at any given floor. A spot check under the worst case scenario was done to validate the findings from the RAM model, using the 1.5% force distribution, and it was found that lateral forces only take at most 35% of the moment capacity of a column. This, through interaction, allows 65% capacity for axial load, and it was shown that about 30% total capacity was utilized for axial, allowing for 35% room for error. Lack of significant torsional effects greatly simplified analysis of the building. Finally, total building drift, and story drift was also analyzed and found to be well under H/400, and 2% seismic requirements.

**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**

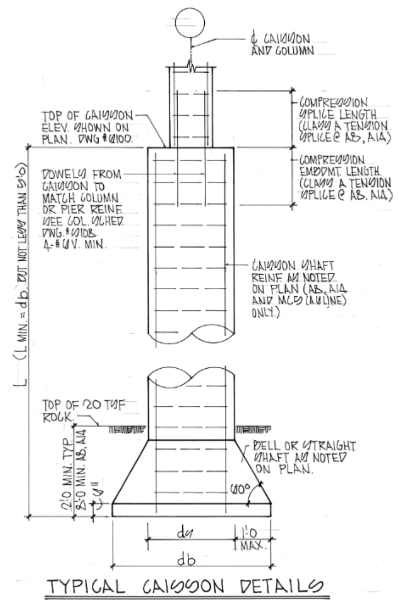


Figure 1. Typical Caisson Detail

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.

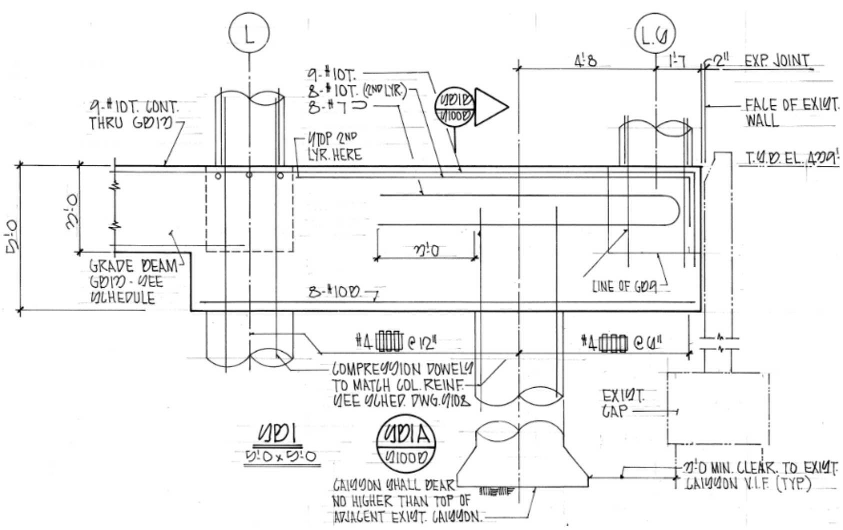


Figure 2. Example of caisson and column misalignment

**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.

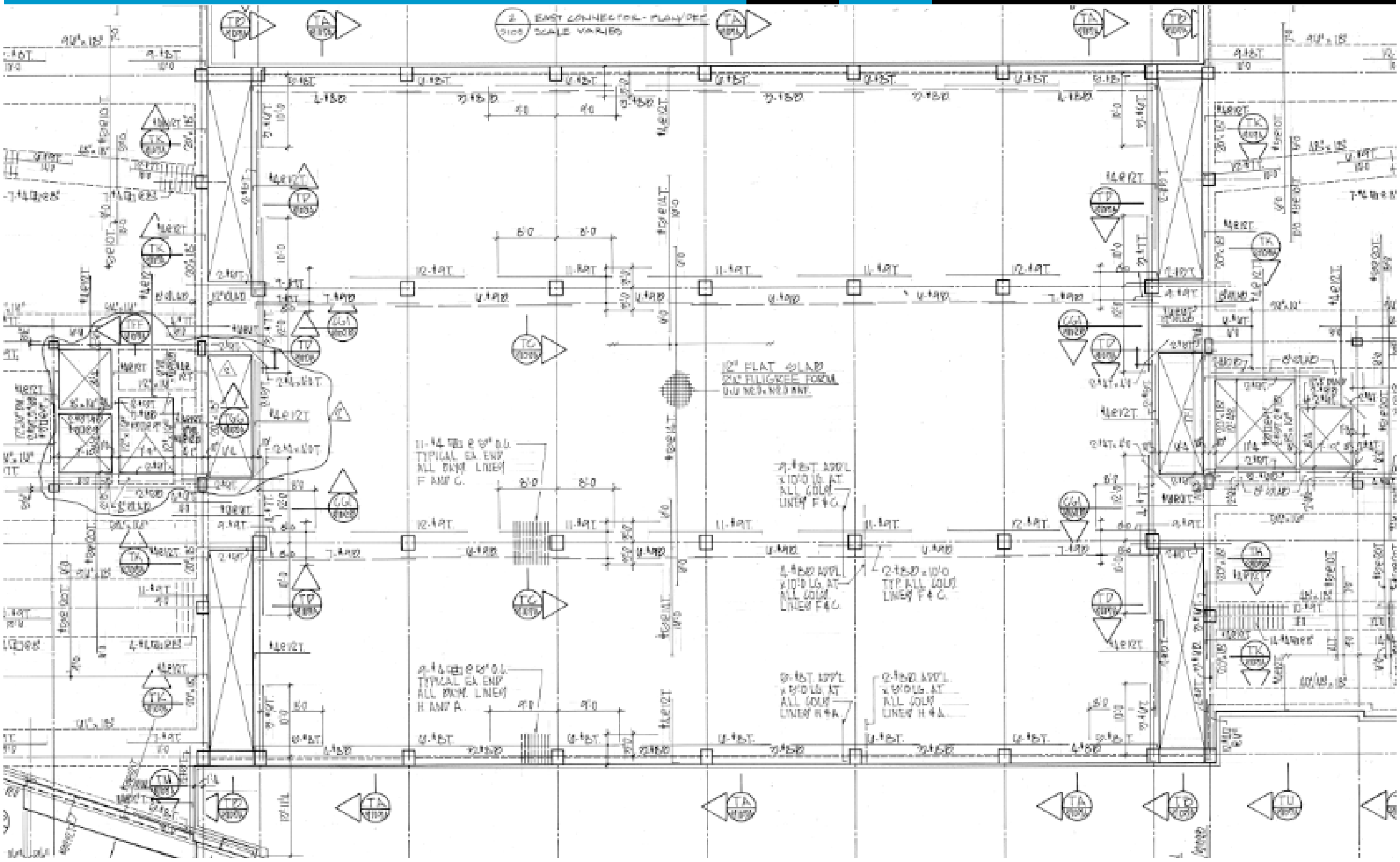


Figure 3. Typical Floor Plan - The three vertical openings on each side are for HVAC, electrical, and mechanical usage, and the openings just to the outside of these openings are elevator shafts.

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.

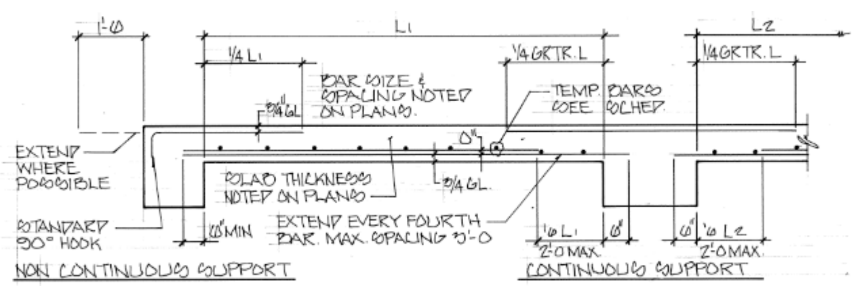


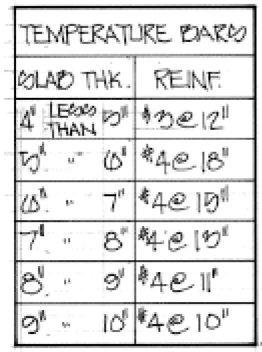

Figure 4. Typical Slab Detail

## 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

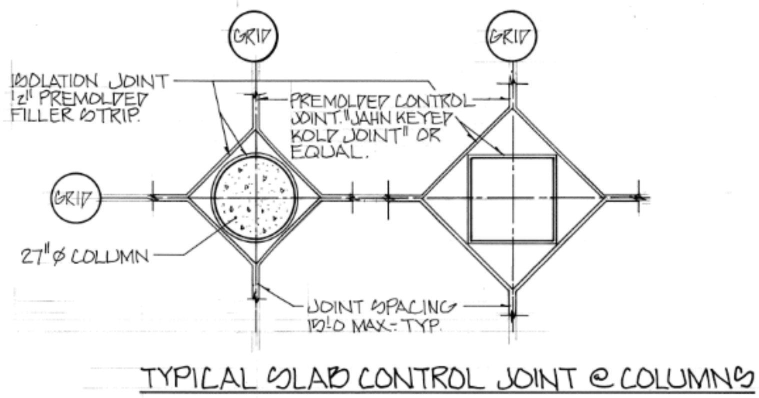



Figure 6. Typical Control Joint Detail

**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.

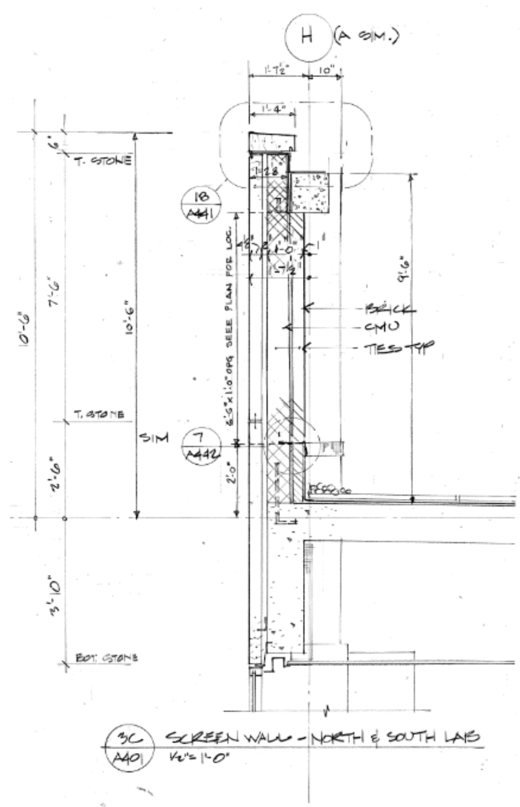



Figure 7. Example Section of a Parapet.

**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.

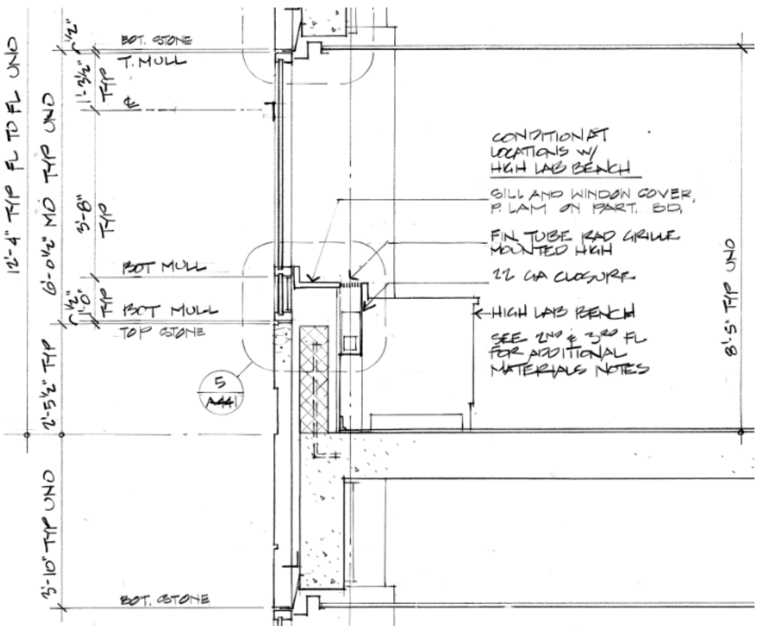



Figure 8. Example Section of Curtain Wall

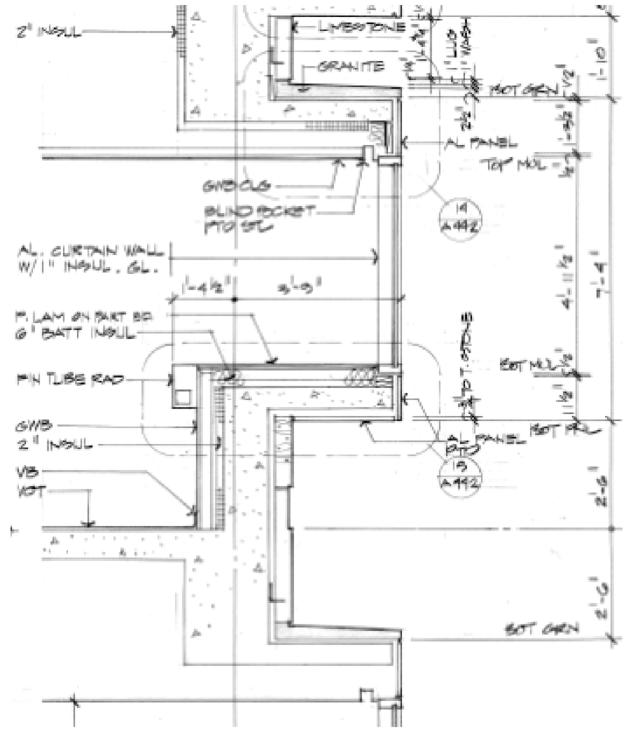



Figure 9. Example Section of Exterior Cladding

**Support of Architectural Cylinder on Corner of Building**

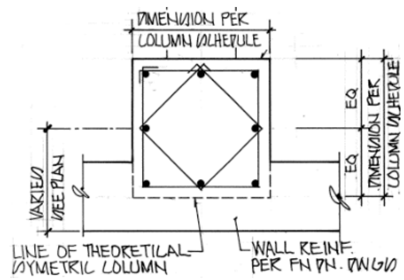



Figure 10. Illustration of Column Used for Support of Architectural Cylinder

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.

**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.

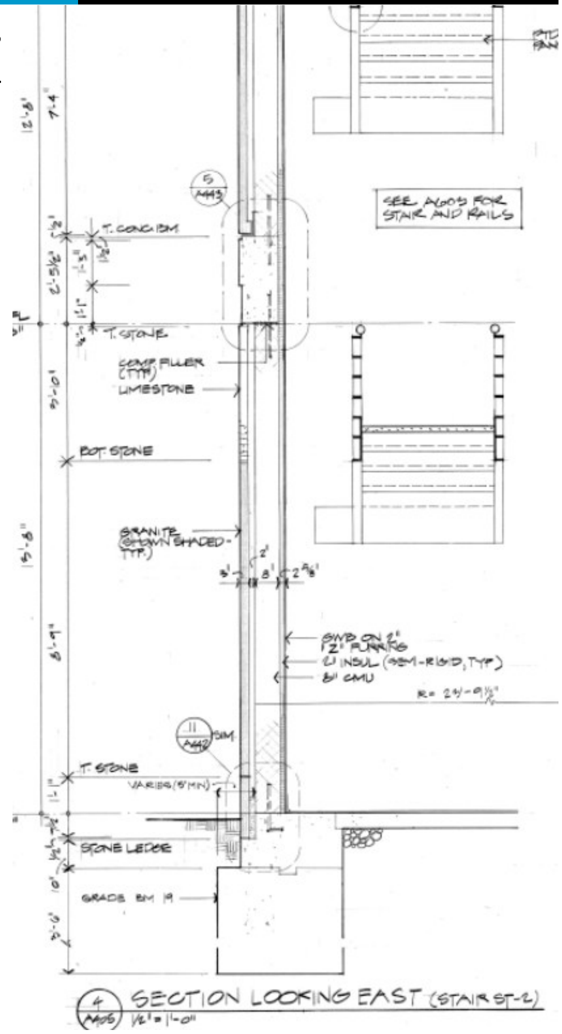


Figure 11. Section of Stairwell

**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.

**Lateral Loads**

A preliminary analysis was conducted during Tech Report 1, and the following tables were generated, using ASCE 7-05. Only applied loads were found, and no in depths analysis into the strength of the structure to resist these loads was completed. It was initially found, however, that seismic forces controlled the design of the structure. Once the in depth was complete, it was found the opposite was true, at least for the long side (280') of the building, while seismic controlled for the short side (90'). An initial model was also provided. Values found are in comprehensive tables in the appendix.

**RAM Model**

A model of the Biomedical Research Building was generated in RAM as a simplification of the lateral analysis process. Columns of the appropriate size and dimensions were placed, most of which were about 24" by 24", with slight variation. Slabs of 12" deep were modeled for diaphragms and beams were placed as appropriate. An isometric view of the model is below. As the main lateral force resisting system was composed of columns, extra attention was given to the accuracy of these columns' dimensions, shapes, and orientation to be as close to the original building as possible, for accuracy.

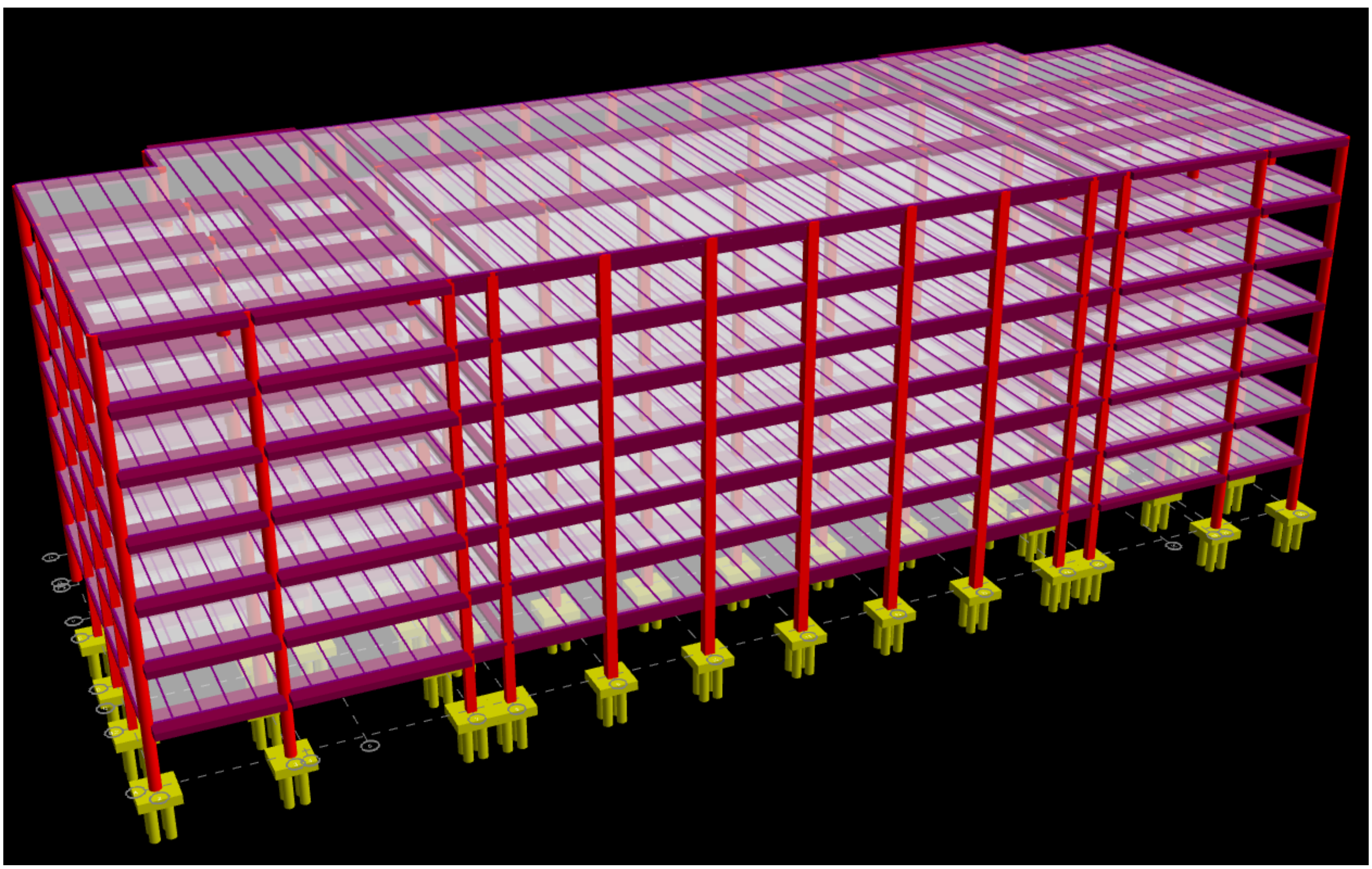


Figure 12. Model of BMR viewed from the south face. It should be noted that the north and east sides of the building have additional buildings that would impact the amount of wind that this building would see. The south and west sides are exposed.

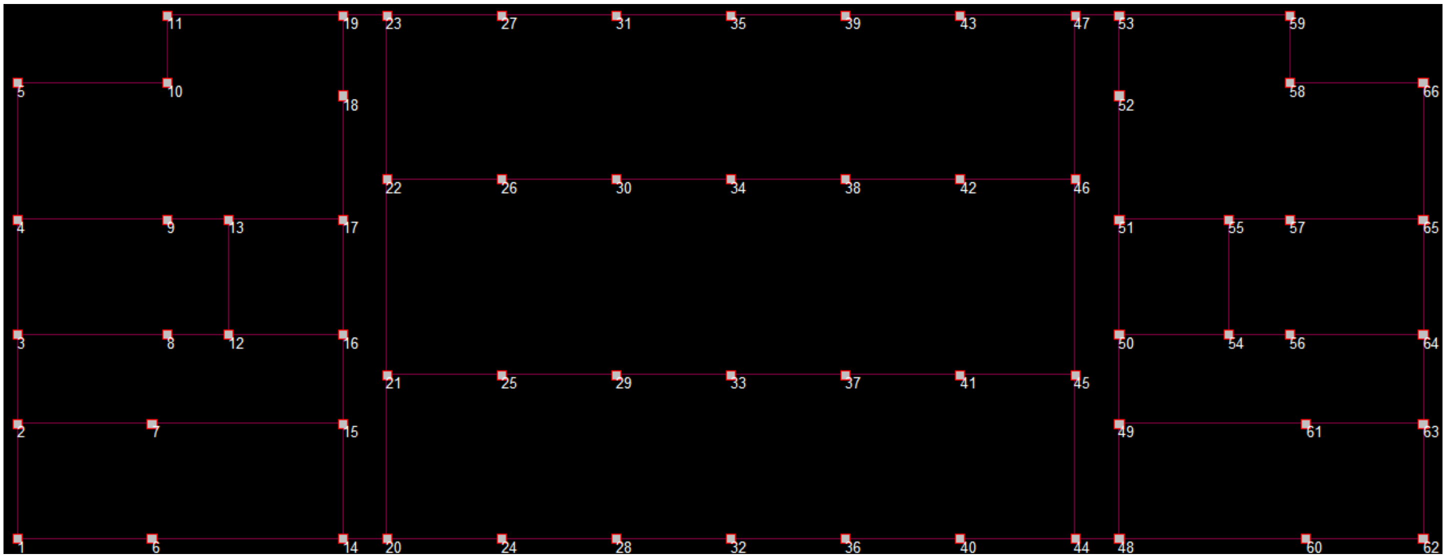


Figure 13. A typical floor layout of columns.

### Torsion

A preliminary check of eccentricity found that due to the column layout, there was only eccentricity of 9” in the short direction. Compared to the 96’ side, it was assumed to generate negligible torsion. Analysis of the computer model supports this assumption in its wind and earthquake load cases as well as in determining stiffness.

### Stiffness, Relative Stiffness, and Load Distribution

A 1 kip load was applied in both the long and short directions of the building. Relative stiffness was found by summing stiffnesses of each column and compared with each individual column, and then multiplied by the worst case scenario of 855 kips from the long side, and 407 kips from the short side. These loads were from wind and seismic, respectively, and both shear forces were found to be at the bottom story of the building. Relative stiffness supports initial assumptions that columns would be about the same stiffness, regardless of direction, due to columns maintaining about a 24” by 24” section, with little variation. Each column supports about 1.5% of the lateral force at any given story.

| Displacement Due To 1Kip Load | | |
|---|---|---|
| | X | Y |
| 1 | 0.00164 | 0.00615 |
| 2 | 0.00165 | 0.00615 |
| 3 | 0.00167 | 0.00615 |
| 4 | 0.00168 | 0.00615 |
| 5 | 0.0017 | 0.00615 |
| 6 | 0.00164 | 0.00616 |
| 7 | 0.00165 | 0.00616 |
| 8 | 0.00167 | 0.00616 |

| Relative Stiffness | | |
|---|---|---|
| | X | Y |
| 1 | 0.014844 | 0.015098 |
| 2 | 0.014935 | 0.015098 |
| 3 | 0.015116 | 0.015098 |
| 4 | 0.015206 | 0.015098 |
| 5 | 0.015387 | 0.015098 |
| 6 | 0.014844 | 0.015123 |
| 7 | 0.014935 | 0.015123 |
| 8 | 0.015116 | 0.015123 |

| Maximum Load Distribution | | |
|---|---|---|
| | X | Y |
| 1 | 12.7 | 6.1 |
| 2 | 12.8 | 6.1 |
| 3 | 12.9 | 6.1 |
| 4 | 13.0 | 6.1 |
| 5 | 13.2 | 6.1 |
| 6 | 12.7 | 6.2 |
| 7 | 12.8 | 6.2 |
| 8 | 12.9 | 6.2 |

| | | |
|---|---|---|
| 9 | 0.00168 | 0.00616 |
| 10 | 0.0017 | 0.00616 |
| 11 | 0.00171 | 0.00616 |
| 12 | 0.00167 | 0.00616 |
| 13 | 0.00168 | 0.00616 |
| 14 | 0.00164 | 0.00616 |
| 15 | 0.00165 | 0.00616 |
| 16 | 0.00167 | 0.00616 |
| 17 | 0.00168 | 0.00616 |
| 18 | 0.0017 | 0.00616 |
| 19 | 0.00171 | 0.00616 |
| 20 | 0.00164 | 0.00616 |
| 21 | 0.00166 | 0.00616 |
| 22 | 0.00169 | 0.00616 |
| 23 | 0.00171 | 0.00616 |
| 24 | 0.00164 | 0.00617 |
| 25 | 0.00166 | 0.00617 |
| 26 | 0.00169 | 0.00617 |
| 27 | 0.00171 | 0.00617 |
| 28 | 0.00164 | 0.00617 |
| 29 | 0.00166 | 0.00617 |
| 30 | 0.00169 | 0.00617 |
| 31 | 0.00171 | 0.00617 |
| 32 | 0.00164 | 0.00617 |
| 33 | 0.00166 | 0.00617 |
| 34 | 0.00169 | 0.00617 |
| 35 | 0.00171 | 0.00617 |
| 36 | 0.00164 | 0.00617 |
| 37 | 0.00166 | 0.00617 |
| 38 | 0.00169 | 0.00617 |
| 39 | 0.00171 | 0.00617 |
| 40 | 0.00164 | 0.00618 |
| 41 | 0.00166 | 0.00618 |
| 42 | 0.00169 | 0.00618 |
| 43 | 0.00171 | 0.00618 |
| 44 | 0.00164 | 0.00618 |
| 45 | 0.00166 | 0.00618 |
| 46 | 0.00169 | 0.00618 |
| 47 | 0.00171 | 0.00618 |
| 48 | 0.00164 | 0.00618 |
| 49 | 0.00165 | 0.00618 |

| | | |
|---|---|---|
| 9 | 0.015206 | 0.015123 |
| 10 | 0.015387 | 0.015123 |
| 11 | 0.015478 | 0.015123 |
| 12 | 0.015116 | 0.015123 |
| 13 | 0.015206 | 0.015123 |
| 14 | 0.014844 | 0.015123 |
| 15 | 0.014935 | 0.015123 |
| 16 | 0.015116 | 0.015123 |
| 17 | 0.015206 | 0.015123 |
| 18 | 0.015387 | 0.015123 |
| 19 | 0.015478 | 0.015123 |
| 20 | 0.014844 | 0.015123 |
| 21 | 0.015025 | 0.015123 |
| 22 | 0.015297 | 0.015123 |
| 23 | 0.015478 | 0.015123 |
| 24 | 0.014844 | 0.015147 |
| 25 | 0.015025 | 0.015147 |
| 26 | 0.015297 | 0.015147 |
| 27 | 0.015478 | 0.015147 |
| 28 | 0.014844 | 0.015147 |
| 29 | 0.015025 | 0.015147 |
| 30 | 0.015297 | 0.015147 |
| 31 | 0.015478 | 0.015147 |
| 32 | 0.014844 | 0.015147 |
| 33 | 0.015025 | 0.015147 |
| 34 | 0.015297 | 0.015147 |
| 35 | 0.015478 | 0.015147 |
| 36 | 0.014844 | 0.015147 |
| 37 | 0.015025 | 0.015147 |
| 38 | 0.015297 | 0.015147 |
| 39 | 0.015478 | 0.015147 |
| 40 | 0.014844 | 0.015172 |
| 41 | 0.015025 | 0.015172 |
| 42 | 0.015297 | 0.015172 |
| 43 | 0.015478 | 0.015172 |
| 44 | 0.014844 | 0.015172 |
| 45 | 0.015025 | 0.015172 |
| 46 | 0.015297 | 0.015172 |
| 47 | 0.015478 | 0.015172 |
| 48 | 0.014844 | 0.015172 |
| 49 | 0.014935 | 0.015172 |

| | | |
|---|---|---|
| 9 | 13.0 | 6.2 |
| 10 | 13.2 | 6.2 |
| 11 | 13.2 | 6.2 |
| 12 | 12.9 | 6.2 |
| 13 | 13.0 | 6.2 |
| 14 | 12.7 | 6.2 |
| 15 | 12.8 | 6.2 |
| 16 | 12.9 | 6.2 |
| 17 | 13.0 | 6.2 |
| 18 | 13.2 | 6.2 |
| 19 | 13.2 | 6.2 |
| 20 | 12.7 | 6.2 |
| 21 | 12.8 | 6.2 |
| 22 | 13.1 | 6.2 |
| 23 | 13.2 | 6.2 |
| 24 | 12.7 | 6.2 |
| 25 | 12.8 | 6.2 |
| 26 | 13.1 | 6.2 |
| 27 | 13.2 | 6.2 |
| 28 | 12.7 | 6.2 |
| 29 | 12.8 | 6.2 |
| 30 | 13.1 | 6.2 |
| 31 | 13.2 | 6.2 |
| 32 | 12.7 | 6.2 |
| 33 | 12.8 | 6.2 |
| 34 | 13.1 | 6.2 |
| 35 | 13.2 | 6.2 |
| 36 | 12.7 | 6.2 |
| 37 | 12.8 | 6.2 |
| 38 | 13.1 | 6.2 |
| 39 | 13.2 | 6.2 |
| 40 | 12.7 | 6.2 |
| 41 | 12.8 | 6.2 |
| 42 | 13.1 | 6.2 |
| 43 | 13.2 | 6.2 |
| 44 | 12.7 | 6.2 |
| 45 | 12.8 | 6.2 |
| 46 | 13.1 | 6.2 |
| 47 | 13.2 | 6.2 |
| 48 | 12.7 | 6.2 |
| 49 | 12.8 | 6.2 |

| | | |
|---|---|---|
| 50 | 0.00167 | 0.00618 |
| 51 | 0.00168 | 0.00618 |
| 52 | 0.0017 | 0.00618 |
| 53 | 0.00171 | 0.00618 |
| 54 | 0.00167 | 0.00619 |
| 55 | 0.00168 | 0.00619 |
| 56 | 0.00167 | 0.00619 |
| 57 | 0.00168 | 0.00619 |
| 58 | 0.0017 | 0.00619 |
| 59 | 0.00171 | 0.00619 |
| 60 | 0.00164 | 0.00619 |
| 61 | 0.00165 | 0.00619 |
| 62 | 0.00164 | 0.00619 |
| 63 | 0.00165 | 0.00619 |
| 64 | 0.00167 | 0.00619 |
| 65 | 0.00168 | 0.00619 |
| 66 | 0.0017 | 0.00619 |

| | | |
|---|---|---|
| 50 | 0.015116 | 0.015172 |
| 51 | 0.015206 | 0.015172 |
| 52 | 0.015387 | 0.015172 |
| 53 | 0.015478 | 0.015172 |
| 54 | 0.015116 | 0.015196 |
| 55 | 0.015206 | 0.015196 |
| 56 | 0.015116 | 0.015196 |
| 57 | 0.015206 | 0.015196 |
| 58 | 0.015387 | 0.015196 |
| 59 | 0.015478 | 0.015196 |
| 60 | 0.014844 | 0.015196 |
| 61 | 0.014935 | 0.015196 |
| 62 | 0.014844 | 0.015196 |
| 63 | 0.014935 | 0.015196 |
| 64 | 0.015116 | 0.015196 |
| 65 | 0.015206 | 0.015196 |
| 66 | 0.015387 | 0.015196 |

| | | |
|---|---|---|
| 50 | 12.9 | 6.2 |
| 51 | 13.0 | 6.2 |
| 52 | 13.2 | 6.2 |
| 53 | 13.2 | 6.2 |
| 54 | 12.9 | 6.2 |
| 55 | 13.0 | 6.2 |
| 56 | 12.9 | 6.2 |
| 57 | 13.0 | 6.2 |
| 58 | 13.2 | 6.2 |
| 59 | 13.2 | 6.2 |
| 60 | 12.7 | 6.2 |
| 61 | 12.8 | 6.2 |
| 62 | 12.7 | 6.2 |
| 63 | 12.8 | 6.2 |
| 64 | 12.9 | 6.2 |
| 65 | 13.0 | 6.2 |
| 66 | 13.2 | 6.2 |

**Drift**

Drift and Story drift was analyzed using the RAM model, and compared with an industry standard of H/400, and found to be held true under wind loads, and even seismic loads, which also adheres to the 2% of building height requirement for seismic. The following tables show total drift and story drift, and shows that the building does not fall outside of acceptable levels under either situation. Wind loads from load case 1 are used, as they control. All columns were assumed to drift the same amount due to lack of significant torsion.

| Story Drift | | | | | | |
|---|---|---|---|---|---|---|
| | Controlling Wind | | | Seismic | | |
| Floor | X | Y | Allowable | X | Y | Allowable |
| 7 | 0.0483 | 0.0038 | 0.37 | 0.0854 | 0.0169 | 2.9699 |
| 6 | 0.1364 | 0.0078 | 0.37 | 0.123 | 0.0292 | 2.9699 |
| 5 | 0.1659 | 0.0121 | 0.37 | 0.1725 | 0.0427 | 2.9699 |
| 4 | 0.2196 | 0.0163 | 0.37 | 0.2229 | 0.0563 | 2.9699 |
| 3 | 0.2607 | 0.0204 | 0.37 | 0.2621 | 0.0698 | 2.9699 |
| 2 | 0.2703 | 0.0243 | 0.38 | 0.2714 | 0.0839 | 3.04 |
| 1 | 0.1616 | 0.0197 | 0.41 | 0.1626 | 0.0685 | 3.28 |

| Drift | | | | | | |
|---|---|---|---|---|---|---|
| | Controlling Wind | | | Seismic | | |
| Floor | X | Y | Allowable | X | Y | Allowable |
| 7 | 1.2628 | 0.1044 | 2.64 | 1.2999 | 0.3673 | 21.1695 |
| 6 | 1.2145 | 0.1006 | 2.27 | 1.2145 | 0.3504 | 18.1996 |
| 5 | 1.0781 | 0.0928 | 1.9 | 1.0915 | 0.3212 | 15.2297 |
| 4 | 0.9122 | 0.0807 | 1.53 | 0.919 | 0.2785 | 12.2598 |
| 3 | 0.6926 | 0.0644 | 1.16 | 0.6961 | 0.2222 | 9.2899 |
| 2 | 0.4319 | 0.044 | 0.79 | 0.434 | 0.1524 | 6.32 |
| 1 | 0.1616 | 0.0197 | 0.41 | 0.1626 | 0.0685 | 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 below. 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 | 71.3 | 57.0 | 13.7 | 974.4 | 779.0 | 31400 | 47.5 |
| 2 | 141.2 | 62.2 | 26.3 | 3717.2 | 1636.9 | | |
| 3 | 146.3 | 66.3 | 38.7 | 5657.3 | 2564.8 | | |
| 4 | 150.0 | 70.2 | 51.0 | 7649.0 | 3581.2 | | |
| 5 | 148.0 | 71.9 | 63.3 | 9370.8 | 4555.6 | | |
| 6 | 131.3 | 66.5 | 75.7 | 9938.1 | 5034.1 | | |
| 7 | 67.1 | 35.4 | 88.0 | 5903.0 | 3117.8 | | |
| Total | | | | 43210 | 21269 | 1491500 | Good |

| Y Direction Overturning | | | | | | | |
|---|---|---|---|---|---|---|---|
| | | | | | | Resisting Moment | |
| | Wind | Seismic | Arm | Moment | | Self Wt | Arm |
| 1 | 19.8 | 54.5 | 13.7 | 271.1 | 745.0 | 31400 | 140 |
| 2 | 39.1 | 57.7 | 26.3 | 1030.7 | 1518.9 | | |
| 3 | 38.4 | 58.7 | 38.7 | 1484.8 | 2271.3 | | |
| 4 | 37.4 | 59.8 | 51.0 | 1907.9 | 3049.8 | | |
| 5 | 36.0 | 60.8 | 63.3 | 2279.4 | 3850.0 | | |
| 6 | 34.2 | 61.5 | 75.7 | 2590.1 | 4655.0 | | |
| 7 | 28.9 | 54.3 | 88.0 | 2542.3 | 4778.4 | | |
| Total | | | | 12106.3 | 20868.4 | 4396000 | Good |

**Spot Check**

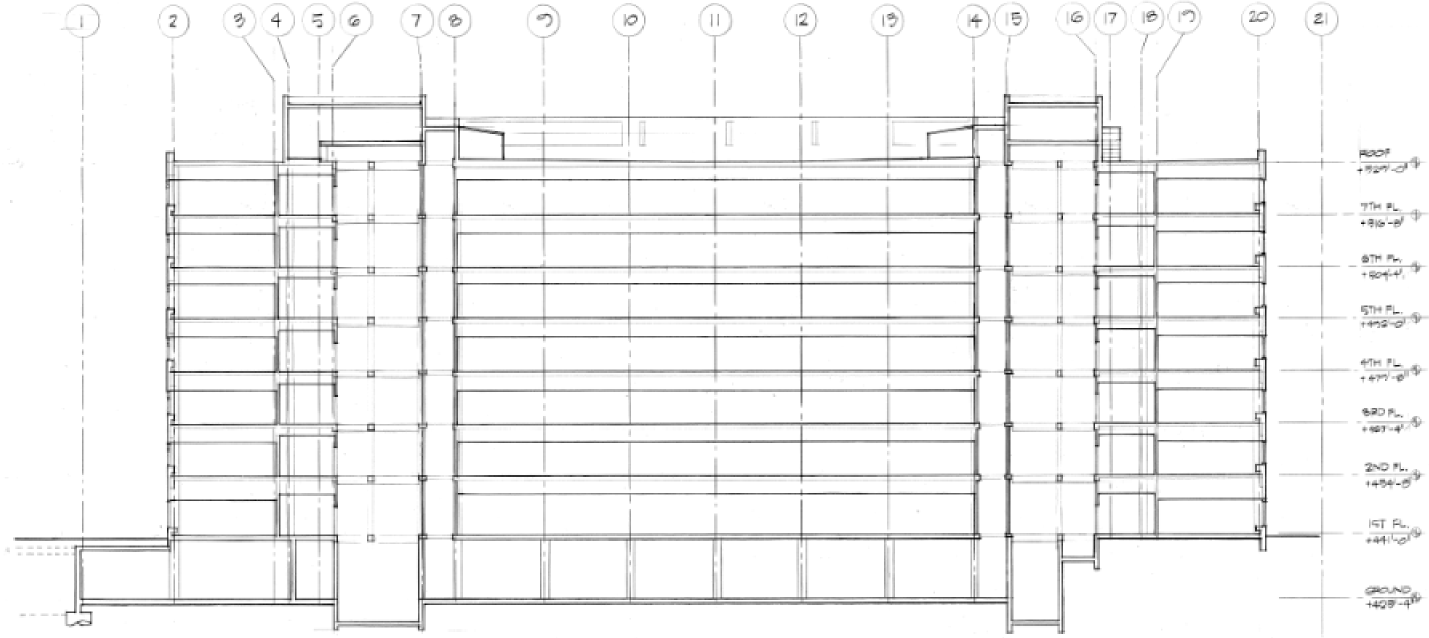
A spot check was done to verify the validity of the results found from the RAM model. A controlling shear force from the lowest floor was applied to a typical column at 1.5%, and found that the moment was not close to failing the column, and thus interaction would show that the column could support that as well as axial loads. This column was checked in both the strong and weak directions, with 480 ft-kip capacity resisting a 175.3 ft-kip moment from lateral wind forces, and a 175 ft-kip capacity resisting a 48 ft-kip moment from lateral wind forces. This leaves at least 64% axial capacity in interaction for the column. With the column resisting about 623 kips in axial load with a maximum capacity of 2000kips, this leaves about 30% in capacity.

**Conclusion**

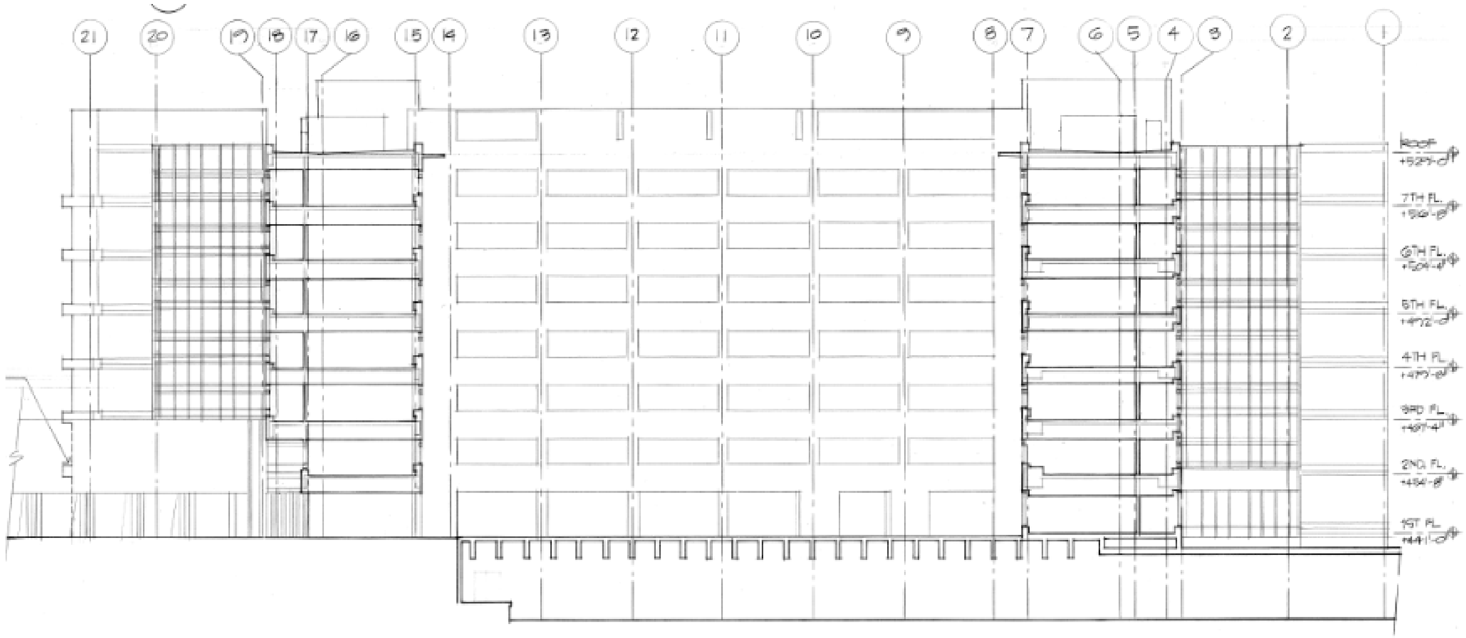
It is very much apparent that maintaining center of mass close to the center of rigidity to minimize eccentricity, and thus, effects of torsion made this building much more simple to design and analysis. Model analysis shows that minimal torsional effects exist by the fact that wind load case one, consisting of pure axial shear. Maintaining similar column cross sections also enforces this design method, by ensuring regular stiffness, and that torsion does not develop through irregularities. Ensuring a lack of torsional effects allows for a simplified design, and thus saving valuable time and money in erecting this building. Drift was found to be within acceptable values. Shear forces were thoroughly analyzed in both axis, and then distributed, under worst case scenarios, finding that these forces generated moments that were acceptable in overall design. Finally, overturning moment was analyzed, finding that resisting moment was much greater than the controlling seismic or wind force, depending on the axis being analyzed.

# Appendix

**Elevations**

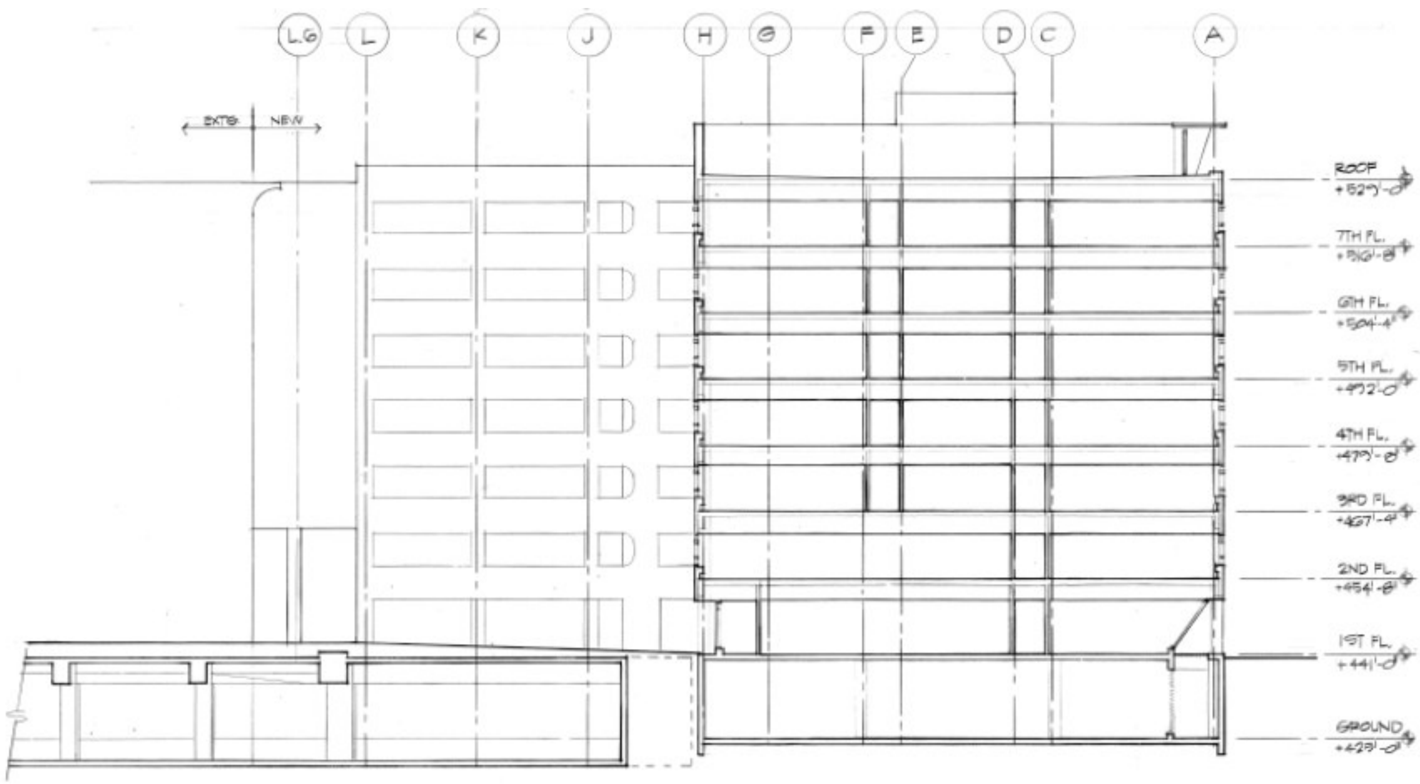



1 A208 LONGITUDINAL SECTION LOOKING NORTH
SCALE : 1/16" = 1'-0"



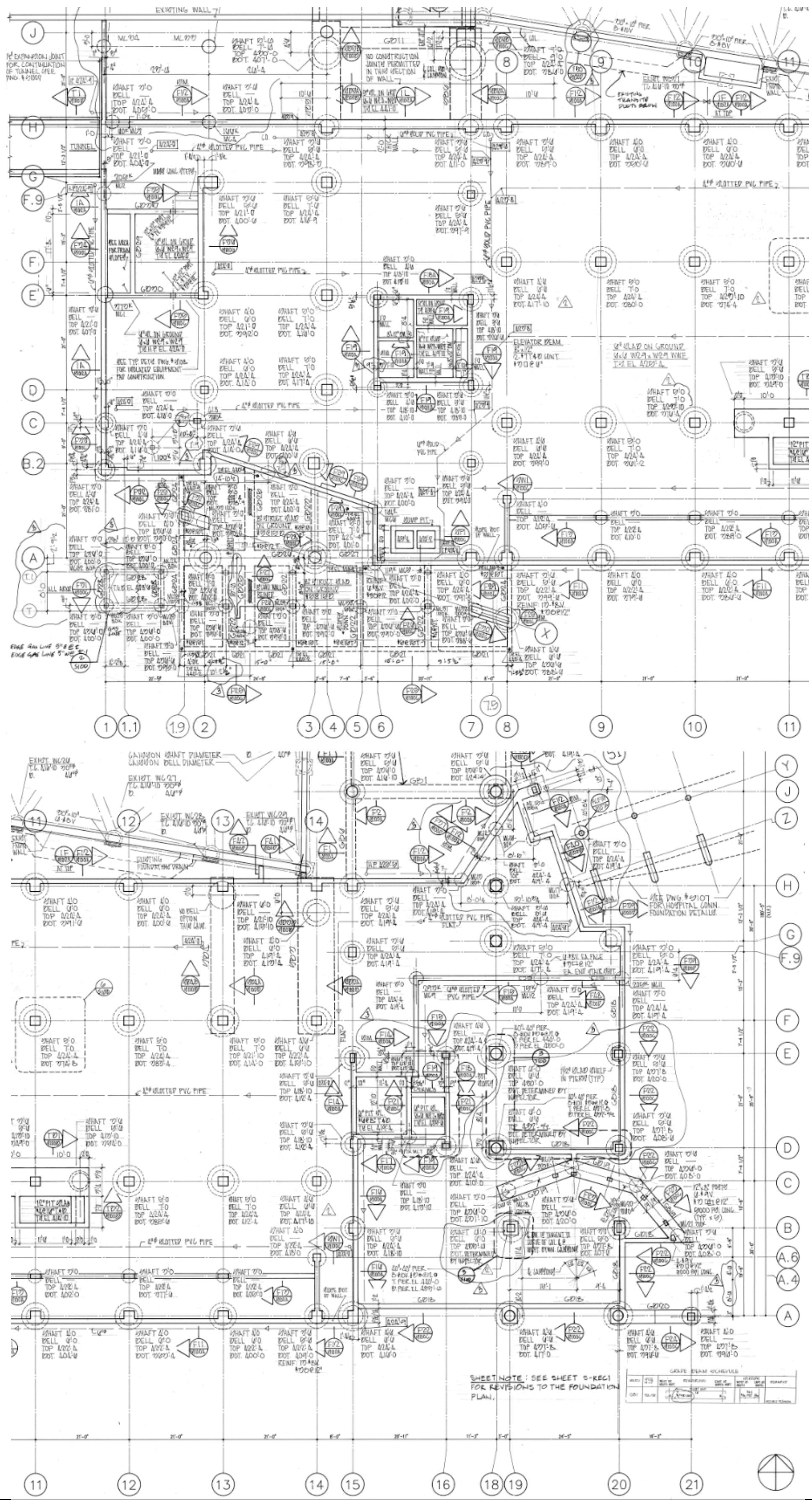

2 A208 SECTION THRU CONNECTORS LOOKING SOUTH
SCALE : 1/16" = 1'-0"

Elevations

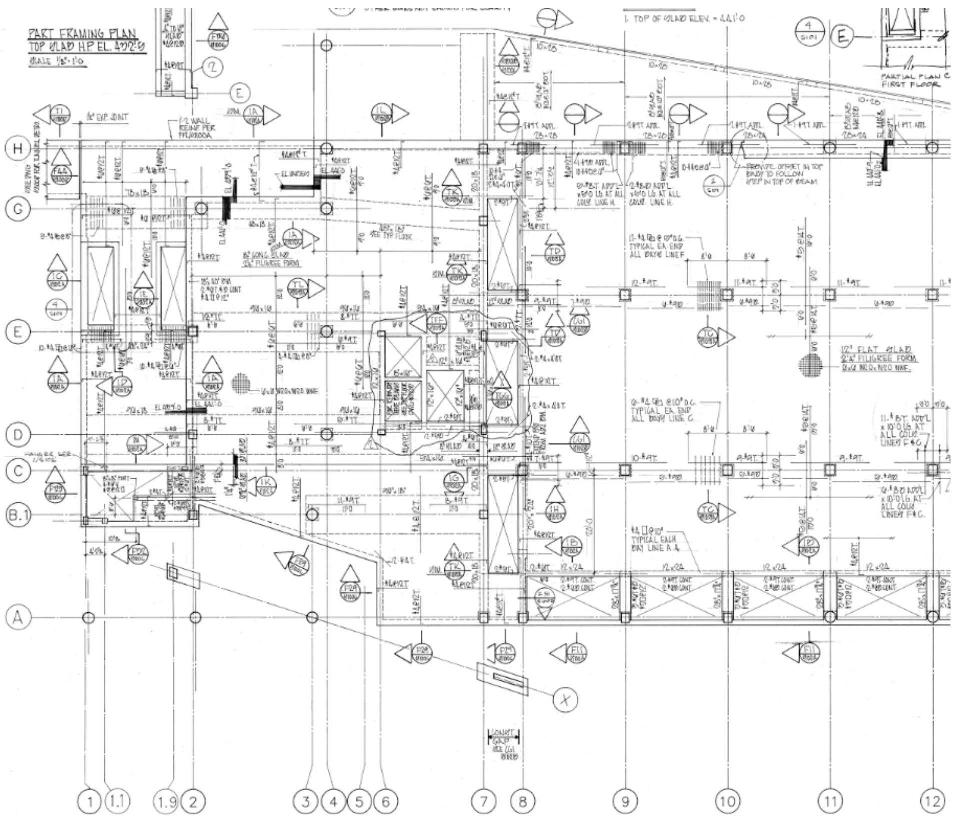


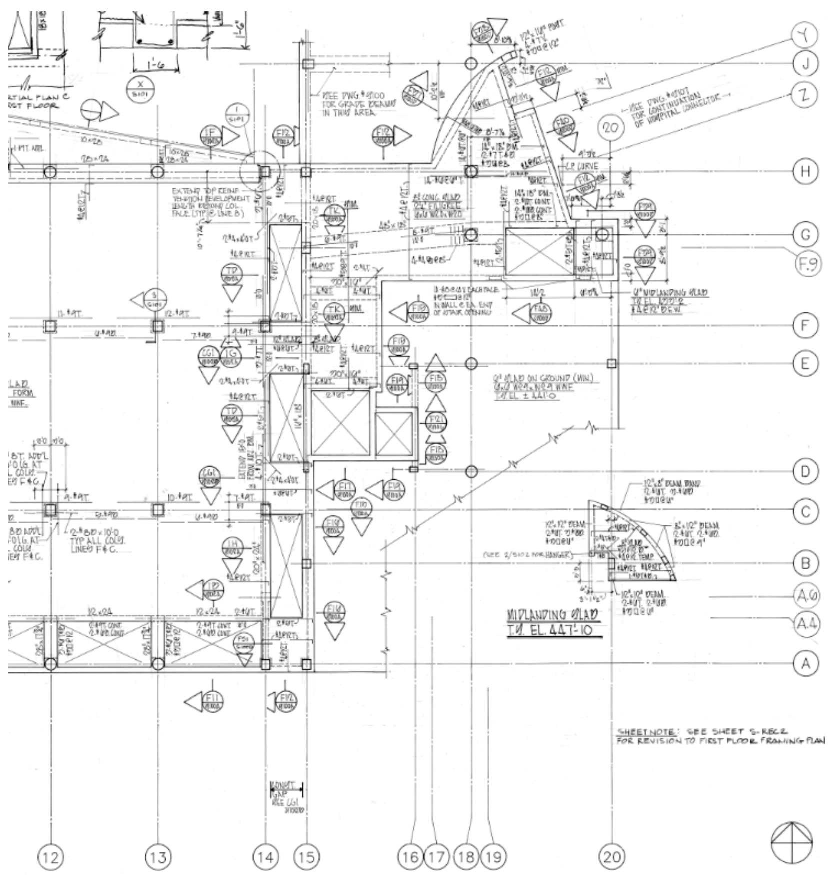

4
A208
TRANSVERSE SECTION LOOKING EAST
SCALE : 1/16" = 1'-0"

**Foundation Plan (Ground Floor)**

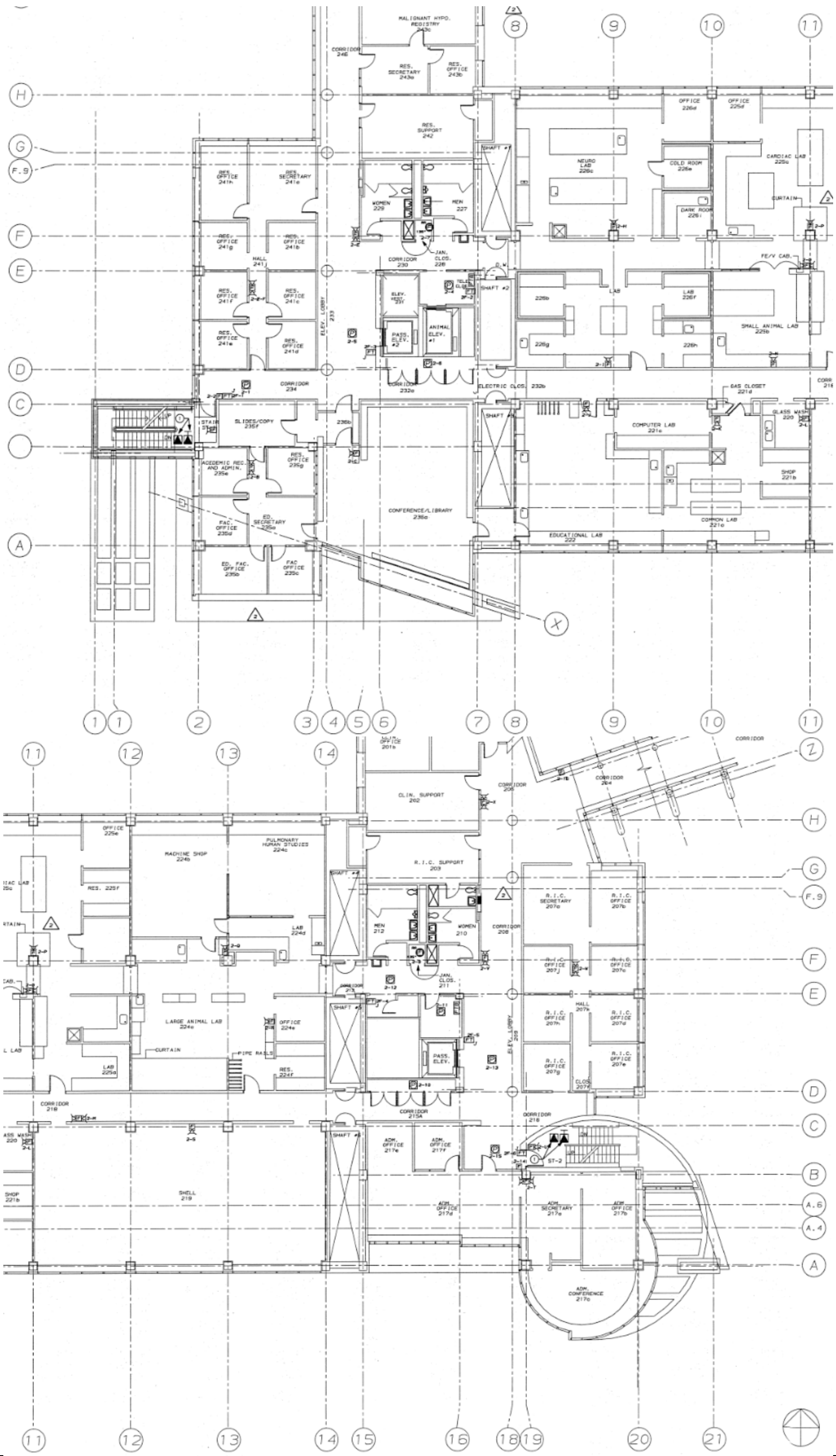


**First Floor Plan**

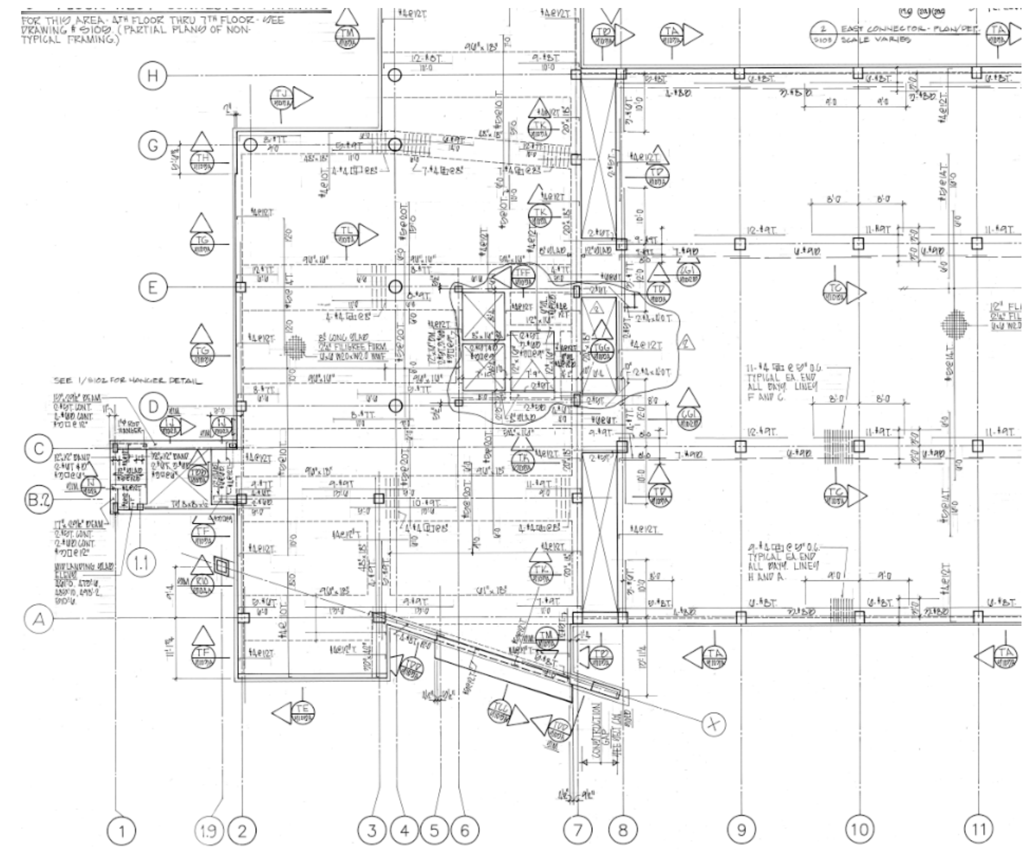


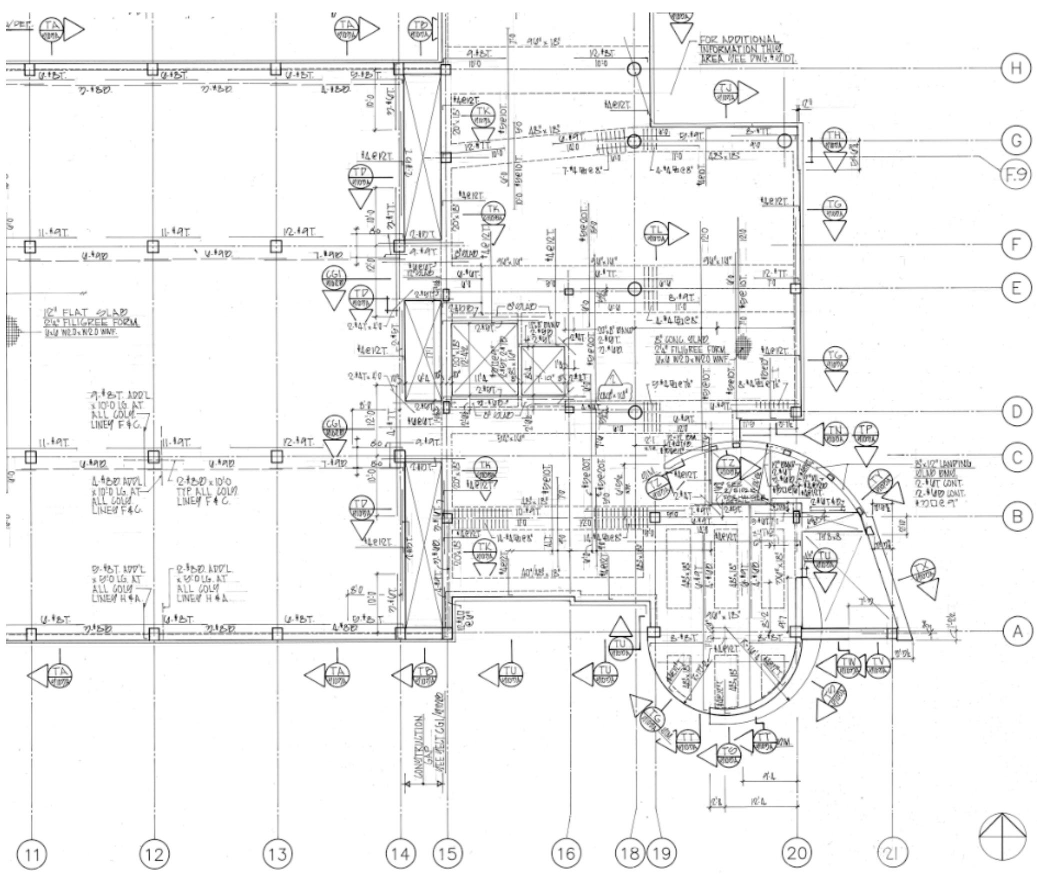


**Second Floor Plan**



**Typical 3rd through 7th Floor Plans**





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

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$^3$ = 169766 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)

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

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

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

$= 1.100 < T_L = 6$ ✓

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

$C_S$ 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.

$\frac{82720}{2} \cdot \frac{1}{3} \cdot 150 + \frac{82720}{2} \cdot \frac{1}{n} \cdot 150 = 3{,}100{,}000$ 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

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

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

$C_{vx}$ calculated on spreadsheet.

$F_x$ calculated on spreadsheet.

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

**Seismic Spreadsheet Calcula-**

| Cvx Calculations | | | | | |
|---|---|---|---|---|---|
| Floor | Wx | Hx | EWiHi^K | K | Cvx |
| g | 2535 | 0 | 0 | 2 | 0 |
| 1 | 3784 | 14.25 | 768388.5 | 2 | 0.00838 |
| 2 | 3700 | 27.91667 | 2883559 | 2 | 0.031448 |
| 3 | 2714 | 40.58333 | 4469977 | 2 | 0.048749 |
| 4 | 2714 | 52.91667 | 7599671 | 2 | 0.082882 |
| 5 | 2714 | 65.25 | 11555025 | 2 | 0.126019 |
| 6 | 2714 | 77.58333 | 16336037 | 2 | 0.17816 |
| 7 | 2714 | 89.91667 | 21942709 | 2 | 0.239306 |
| Roof | 2500 | 102.25 | 26137656 | 2 | 0.285056 |
| Total : | | | | | 1 |

| Fx Calculations | | |
|---|---|---|
| Cvx | V | Fx |
| 0.01 | 529 | 5.29 |
| 0.03 | 529 | 15.87 |
| 0.05 | 529 | 26.45 |
| 0.08 | 529 | 42.32 |
| 0.13 | 529 | 68.77 |
| 0.18 | 529 | 95.22 |
| 0.24 | 529 | 126.96 |
| 0.29 | 529 | 153.41 |
| Total Shear | | 534.29 |

| Overturning Moment | | |
|---|---|---|
| Fx (kip) | H (Feet) | M (kip ft) |
| 5.29 | 14 | 75.3825 |
| 15.87 | 28 | 443.0375 |
| 26.45 | 41 | 1073.429 |
| 42.32 | 53 | 2239.433 |
| 68.77 | 65 | 4487.242 |
| 95.22 | 78 | 7387.485 |
| 126.96 | 90 | 11415.82 |
| 153.41 | 102 | 15686.17 |
| Total: | | 42808 |

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

Location: Hershey, PA

Category IV (Table 1-1)

Exposure C (section 6.5.6)

V = 90 mph (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 spreadsheet using:

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

For $p = q G C_p - q_i (GC_{pi})$;

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

values found in spread sheet.

L/B for 95' side: $95/271 = .34 < 1$; $C_p = -.5$

L/B for 271' side: $271/95 = 2.92$; $C_p = -.3$

**Wind Spreadsheet Calculations**

| Spreadsheet for qz | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Floor | H | C | Kz | Kzt | Kd | V | V^2 | I | qz |
| 1 | 14.3 | 0.00256 | 0.85 | 1 | 0.85 | 90 | 8100 | 1.15 | 17.22902 |
| 2 | 13.7 | 0.00256 | 0.97 | 1 | 0.85 | 90 | 8100 | 1.15 | 19.66136 |
| 3 | 12.7 | 0.00256 | 1.04 | 1 | 0.85 | 90 | 8100 | 1.15 | 21.08022 |
| 4 | 12.3 | 0.00256 | 1.09 | 1 | 0.85 | 90 | 8100 | 1.15 | 22.09369 |
| 5 | 12.3 | 0.00256 | 1.15 | 1 | 0.85 | 90 | 8100 | 1.15 | 23.30986 |
| 6 | 12.3 | 0.00256 | 1.2 | 1 | 0.85 | 90 | 8100 | 1.15 | 24.32333 |
| 7 | 12.3 | 0.00256 | 1.24 | 1 | 0.85 | 90 | 8100 | 1.15 | 25.13411 |
| Parapet | 1.5 | 0.00256 | 1.26 | 1 | 0.85 | 90 | 8100 | 1.15 | 25.53949 |

| Windward Pressures | | | | | | | |
|---|---|---|---|---|---|---|---|
| q | G | Cp | qi | Gcpi (+/-) | Pressure | (+/-) | Resultant |
| 17.22902 | 0.85 | 0.8 | 17.22902 | 0.18 | 11.71574 | 3.101224 | 14.81696 |
| 19.66136 | 0.85 | 0.8 | 19.66136 | 0.18 | 13.36972 | 3.539044 | 16.90877 |
| 21.08022 | 0.85 | 0.8 | 21.08022 | 0.18 | 14.33455 | 3.794439 | 18.12899 |
| 22.09369 | 0.85 | 0.8 | 22.09369 | 0.18 | 15.02371 | 3.976864 | 19.00057 |
| 23.30986 | 0.85 | 0.8 | 23.30986 | 0.18 | 15.8507 | 4.195774 | 20.04648 |
| 24.32333 | 0.85 | 0.8 | 24.32333 | 0.18 | 16.53986 | 4.378199 | 20.91806 |
| 25.13411 | 0.85 | 0.8 | 25.13411 | 0.18 | 17.09119 | 4.524139 | 21.61533 |
| 25.53949 | 0.85 | 0.8 | 25.53949 | 0.18 | 17.36686 | 4.597109 | 21.96397 |

| Leeward Pressures | | | | | | | |
|---|---|---|---|---|---|---|---|
| LW (95' side) | | | | | | | |
| q | G | Cp | qi | Gcpi (+/-) | p | (+/-) | Resultant |
| 25.53949 | 0.85 | -0.5 | 25.53949 | 0.18 | -10.8543 | 4.597109 | -15.4514 |
| | | | | | | | |
| LW (277' side) | | | | | | | |
| q | G | Cp | qi | Gcpi (+/-) | p | (+/-) | Resultant |
| 25.53949 | 0.85 | -0.3 | 25.53949 | 0.18 | -6.51257 | 4.597109 | -11.1097 |

| Resultant wind loads | | | | | | | |
|---|---|---|---|---|---|---|---|
| | WW (95') | WW (277') | H | LW (95') | LW (277') | R (95') | R (277') |
| 1 | 20058.46 | 58486.25 | 14.3 | -20917.3 | -43852.7 | 40.97579 | 63.91115 |
| 2 | 21953.22 | 64010.96 | 13.7 | -20061.1 | -42057.5 | 42.01428 | 64.01076 |
| 3 | 21815.21 | 63608.57 | 12.7 | -18593.2 | -38980.2 | 40.40839 | 60.79538 |
| 4 | 22262.34 | 64912.29 | 12.3 | -18103.9 | -37954.4 | 40.36622 | 60.21671 |
| 5 | 23487.79 | 68485.44 | 12.3 | -18103.9 | -37954.4 | 41.59167 | 61.44216 |
| 6 | 24509 | 71463.07 | 12.3 | -18103.9 | -37954.4 | 42.61288 | 62.46337 |
| 7 | 25325.96 | 73845.18 | 12.3 | -18103.9 | -37954.4 | 43.42985 | 63.28033 |
| Parapet | 3129.865 | 9126.028 | 1.5 | -2201.82 | -4616.07 | 5.331689 | 7.745937 |

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: $21 - 22/12 = 19'$-2"

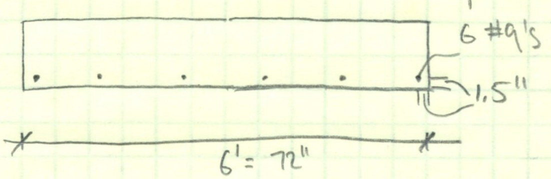
Distributed load:

$35.75 \cdot 400 = 14.3$ Kip/ft

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

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

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

Check assumption:

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

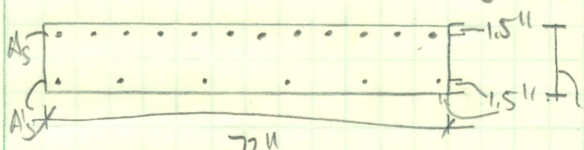
$M_n = A_s \cdot f_y (d - a/2)$

$= 6 \cdot 60000(10.5 - 1.47/2) = 4055400$ lbs·in $\Rightarrow$ 293 ft·Kips

293 ft·Kip < 329.5 ft·Kip ✗ 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.

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

$a = \frac{Asfy - A'sfy}{.85f'c(b)} = \frac{12(60000) - 6(60000)}{.85(4000)(72)} = 1.47"$ (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

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

Solve quadratic

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

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

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

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

$c = 2.47"$

Check yielding:

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

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

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

.97 9 9.45

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

$559 ftKips > 479.2 ftKips$ ✓

Joshua Zolko | Tech Report 1 | Spot Checks | 3/9

Check shear reinforcement: Plans call for 11 #4's $\boxed{\phantom{x}}$ @ 5" o.c. (4 legs) ⇒ $A_v = .8 in^2$

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

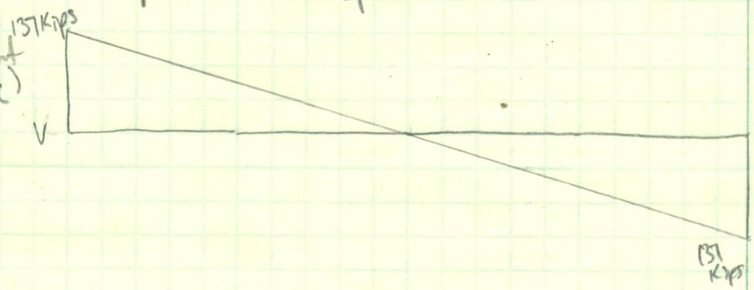
$= 2\sqrt{4000}(72)(10.5)$  R = 137 Kips  Clear span = 19'-2"

$V_c = 95.6$ 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:

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

122.1 Kips < 124.5 Kips ✗ shear reinforcement fails

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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)

$f_y = 60000\,psi$

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

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

$\lambda = 1.0$

$\frac{1.5 + 0}{1.128} = 1.3 < 2.5$ ✓

$f'c = 4000\,psi$

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

$\psi_s = 1.0$

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

$L_d = 62'' > 12$ ✓

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

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

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

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Typical slab check:

Punching Shear:

At face of support:

$b_o = 22\cdot 2 + 24\cdot 2 + 4\cdot 12 = 140''$

$V_n = (21\cdot 35.75 - (\frac{22+12}{12}\cdot\frac{24+12}{12}))400psf$

$= 297\ Kips$

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

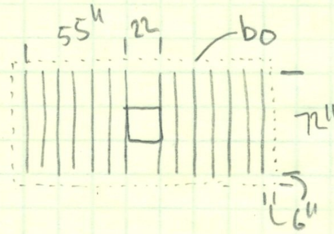
$V_u = .75(2)\sqrt{4000}(140)(10.5) + \frac{.75(40000)(10.5)}{5} \le .75(6)\sqrt{4000}(140)(10.5)$

$= 201.9\ Kips \le 416.7\ Kips$ ✓

but,

$V_n = 297 > 201.9\ Kips$ fails

At end of Stirrups:




$b_o = 22\cdot 2 + 110\cdot 2 + 72\cdot 2 + 12\cdot 4$

$= 456 in$

$V_n = (21\cdot 35.75 - 12\cdot 7)400$

$= 266.7\ Kips$

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

$= .75(2)\sqrt{4000}(456)\cdot 10.5$

$V_u = 452.5\ Kips$

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

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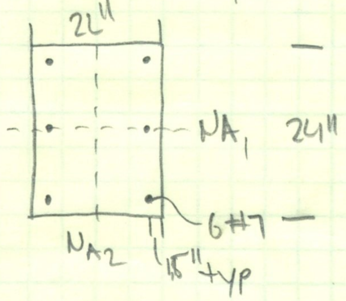
Column Check (typical)




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

Pure axial strength: $A_c = 22 \cdot 24 - 3.6 = 524.4 in^2$ $f'c = 4000 psi$

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

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

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

$P_o = 2000 Kips$ $P_n = 623 Kips$

$P_o > P_n$

$2000 > 623$ ✓

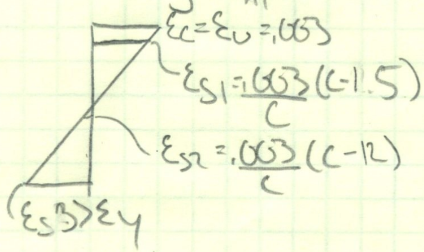
Pure axial tension:

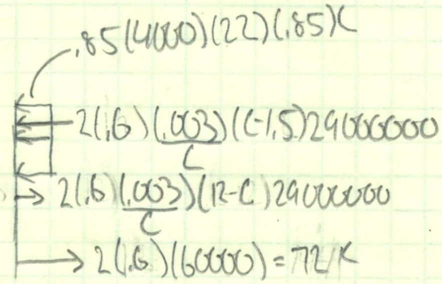
$T_o = A_s \cdot f_y$

$= 3.6 \cdot 60000$

$T_o = 216 Kips$

Pure bending @ $N_{A1}$




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

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

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$63580c + \frac{104400}{c}(c-1.5) = \frac{104400}{c}(c-12) + 172000$

$63580c^2 + 104400c - 156600 = 104400c - 1252800 + 172000c$

$63580c^2 - 172000c - 1096200 = 0$

$c = 4.8''$

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

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

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

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

$\varepsilon_{s3} = \frac{.003}{4.8}(4.8-24) > \varepsilon_y$ $\qquad$ Concrete $= .85(4000)(22)(6.5)$

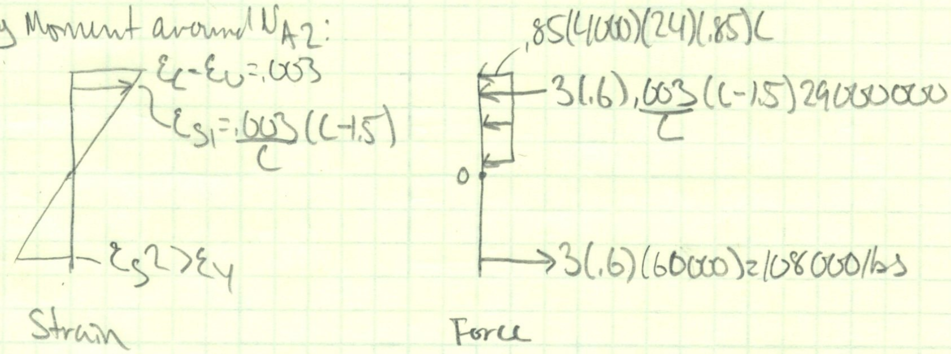
$F_{s3} = -60 ksi \Rightarrow -72 Kips$ $\qquad = 486.2 Kips$

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·Kips

Bending Moment around $N_{A2}$:




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

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

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

$c = 1.5''$

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$c = 1.5''$

$\varepsilon_{s1} = \frac{.003}{1.5}(1.5 - 1.5) = 0$

$F_{s1} = 0$ Concrete $= .85(4000)(24)(.85)(1.5)$

$F_{s2} = 108$ Kips $= 164$ Kips

Moment about 0 at $N_{A2}$:

$M_0 = 164\left(11 - \frac{.85(1.5)}{2}\right) + 108(11 - 1.5)$

$M_0 = 175.3$ ft·Kips

$479.2 > 175.3$ X 479.2 Kip ft from negative moment on beam.

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

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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.

| Case 1 | | | | | | |
|---|---|---|---|---|---|---|
| | X-Direction (280' Side) | | | Y-Direction (90' side) | | |
| Floor | | Shear | | | Shear | |
| 1 | | 855.1 | | | 233.9 | |
| 2 | | 788.05 | | | 205.01 | |
| 3 | | 656.71 | | | 170.78 | |
| 4 | | 508.75 | | | 134.79 | |
| 5 | | 358.77 | | | 97.38 | |
| 6 | | 212.46 | | | 58.98 | |
| 7 | | 71.3 | | | 19.84 | |

| Case 2 (+) | | | | | | |
|---|---|---|---|---|---|---|
| | X-Direction (280' Side) | | | Y-Direction (90' side) | | |
| Floor | | Shear | | | Shear | |
| 1 | | 641.03 | | | 175.42 | |
| 2 | | 590.77 | | | 153.77 | |
| 3 | | 492.41 | | | 128.1 | |
| 4 | | 381.51 | | | 101.1 | |
| 5 | | 269.06 | | | 73.04 | |
| 6 | | 159.34 | | | 44.23 | |
| 7 | | 53.48 | | | 14.88 | |

| Case 2 (-) | | | | | | |
|---|---|---|---|---|---|---|
| | X-Direction (280' Side) | | | Y-Direction (90' side) | | |
| Floor | | Shear | | | Shear | |
| 1 | | 641.66 | | | 175.43 | |
| 2 | | 591.31 | | | 153.75 | |
| 3 | | 492.66 | | | 128.07 | |
| 4 | | 381.62 | | | 101.09 | |
| 5 | | 269.1 | | | 73.04 | |
| 6 | | 159.35 | | | 44.23 | |
| 7 | | 53.47 | | | 14.88 | |

| Case 3 | | | | | | |
|---|---|---|---|---|---|---|
| | X-Direction (280' Side) | | | Y-Direction (90' side) | | |
| Floor | | Shear | | | Shear | |
| 1 | | 641.66 | | | 175.43 | |
| 2 | | 591.31 | | | 153.75 | |
| 3 | | 492.66 | | | 128.07 | |
| 4 | | 381.62 | | | 101.09 | |
| 5 | | 269.1 | | | 73.04 | |
| 6 | | 159.35 | | | 44.23 | |
| 7 | | 53.47 | | | 14.88 | |

| Case 4 (++) | | | | | | |
|---|---|---|---|---|---|---|
| | X-Direction (280' Side) | | | Y-Direction (90' side) | | |
| Floor | | Shear | | | Shear | |
| 1 | | 481.28 | | | 131.59 | |
| 2 | | 443.51 | | | 115.24 | |
| 3 | | 369.51 | | | 95.99 | |
| 4 | | 286.13 | | | 75.78 | |
| 5 | | 201.79 | | | 54.77 | |
| 6 | | 119.51 | | | 33.17 | |
| 7 | | 40.11 | | | 11.16 | |

| Case 4 (+-) | | | | | | |
|---|---|---|---|---|---|---|
| | X-Direction (280' Side) | | | Y-Direction (90' side) | | |
| Floor | | Shear | | | Shear | |
| 1 | | 481.28 | | | 131.55 | |
| 2 | | 443.51 | | | 115.41 | |
| 3 | | 369.51 | | | 96.15 | |
| 4 | | 286.22 | | | 75.87 | |
| 5 | | 201.83 | | | 54.8 | |
| 6 | | 119.51 | | | 33.18 | |
| 7 | | 40.1 | | | 11.16 | |

| Case 4 (-+) | | | | | | |
|---|---|---|---|---|---|---|
| | X-Direction (280' Side) | | | Y-Direction (90' side) | | |
| Floor | | Shear | | | Shear | |
| 1 | | -480.74 | | | 131.55 | |
| 2 | | -443.05 | | | 115.39 | |
| 3 | | -369.29 | | | 96.13 | |
| 4 | | -286.12 | | | 75.86 | |
| 5 | | -201.79 | | | 54.79 | |
| 6 | | -119.51 | | | 33.17 | |
| 7 | | -40.11 | | | 11.16 | |

| Case 4 (--) | | | | | | |
|---|---|---|---|---|---|---|
| | X-Direction (280' Side) | | | Y-Direction (90' side) | | |
| Floor | | Shear | | | Shear | |
| 1 | | -481.27 | | | 131.59 | |
| 2 | | -443.5 | | | 115.23 | |
| 3 | | -369.5 | | | 95.98 | |
| 4 | | -286.21 | | | 75.77 | |
| 5 | | -201.82 | | | 54.75 | |
| 6 | | -119.51 | | | 33.16 | |
| 7 | | -40.1 | | | 11.16 | |

| EQ | | | | | | |
|---|---|---|---|---|---|---|
| | X-Direction (280' Side) | | | Y-Direction (90' side) | | |
| Floor | | Shear | | | Shear | |
| 1 | | 429.6 | | | 407.34 | |
| 2 | | 394.17 | | | 353.04 | |
| 3 | | 327.64 | | | 291.52 | |
| 4 | | 255.71 | | | 230.73 | |
| 5 | | 185.49 | | | 170.93 | |
| 6 | | 119.16 | | | 112.19 | |
| 7 | | 57 | | | 54.51 | |

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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(109.6) +
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.

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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.