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

Joshua Zolko, Structural Option
9 April 2014

Table of Contents

Executive Summary

The goal of this report was to figure out if changing from a concrete structural system to a steel structural system was a viable option. First a viable steel structure was designed, then evaluated for its HVAC, cost and environmental impacts. The proposed steel structure was to maintain a H/400 building drift maximum, while keeping the deflection small on the north side of the building where it connects with another building. It was also to conform to serviceability limits and perform under various loading combinations.

After designing a viable steel structure system, the system was then compared with the concrete system in terms of HVAC, cost and CO_2 emissions. The new design did not perform well for HVAC with the changes, and the cost remained the same, while CO_2 emissions were decreased by 16.4% at a cost of losing 6" of ceiling height.

These changes overall did not impact the building to a significant degree in terms of cost, but making the building greener makes the change very much appealing, and is as such, recommended by this report.

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

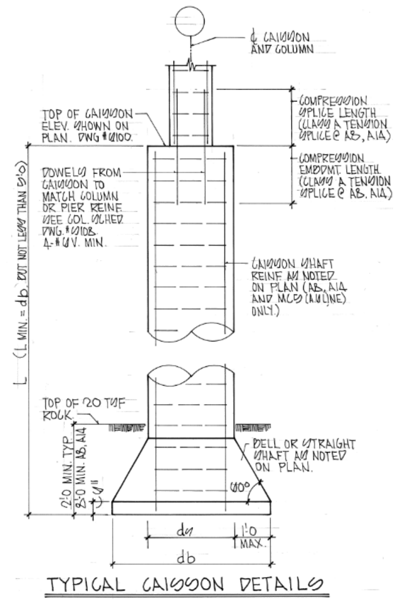


Figure 1. Typical Caisson Detail

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.

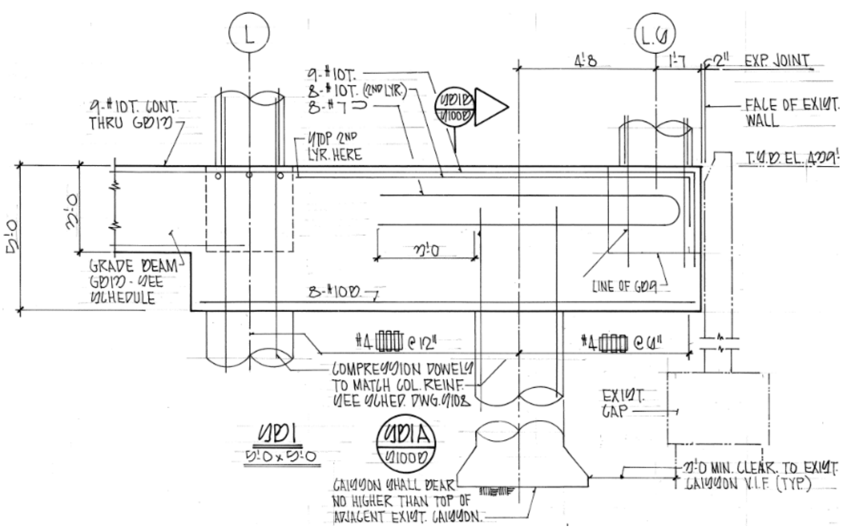


Figure 2. Example of caisson and column misalignment

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.

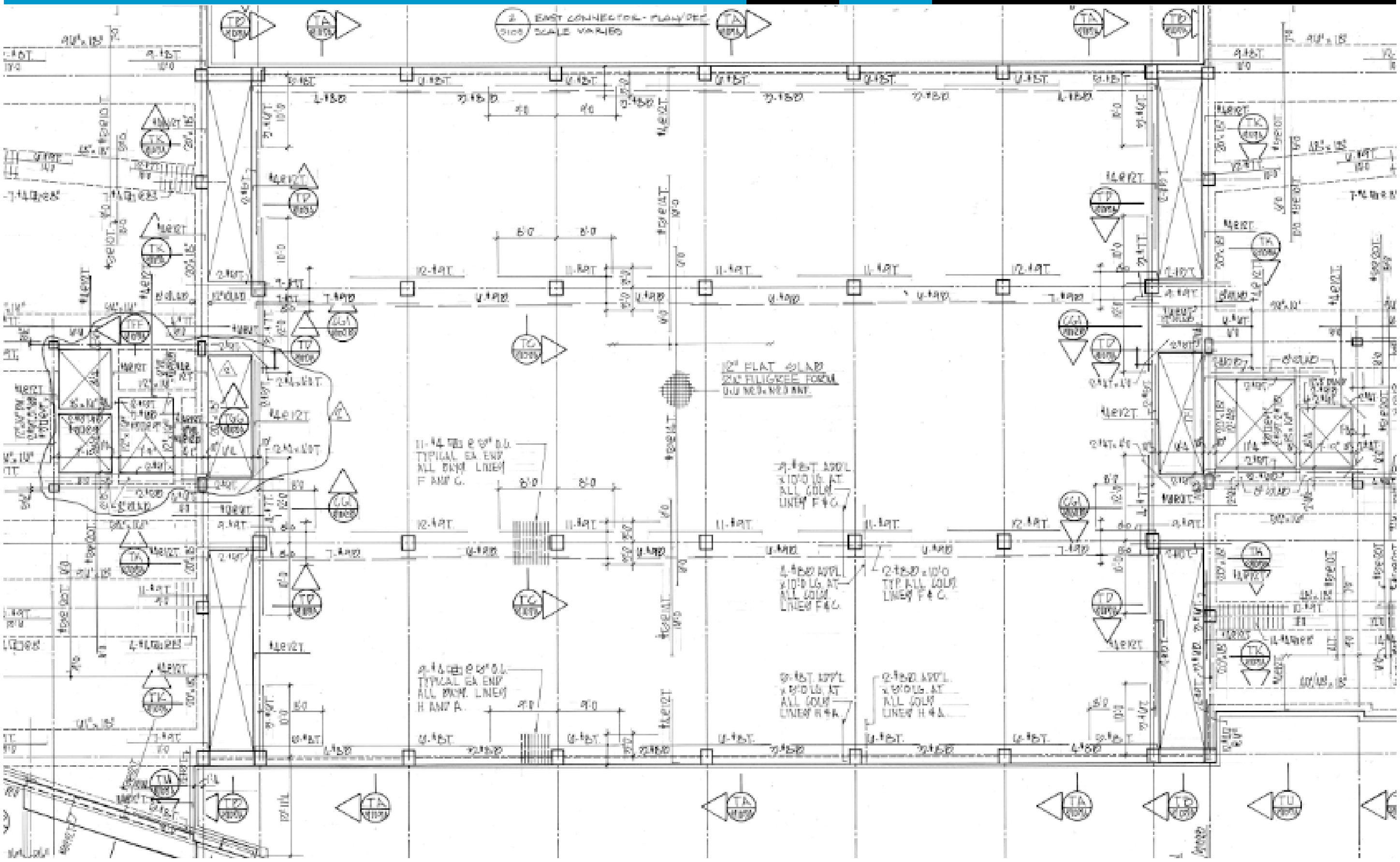


Figure 3. Typical Floor Plan - The three vertical openings on each side are for HVAC, electrical, and mechanical usage, and the openings just to the outside of these openings are elevator shafts.

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.

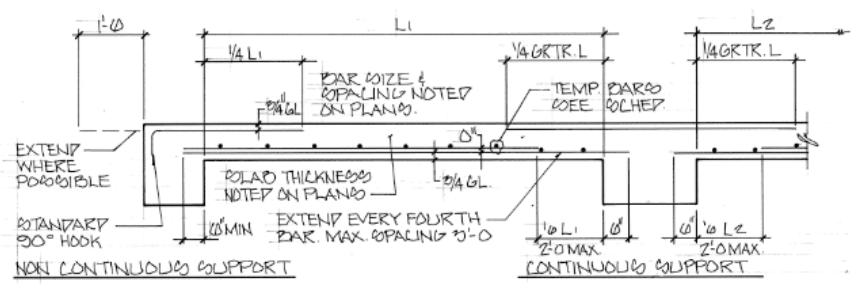


Figure 4. Typical Slab Detail

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

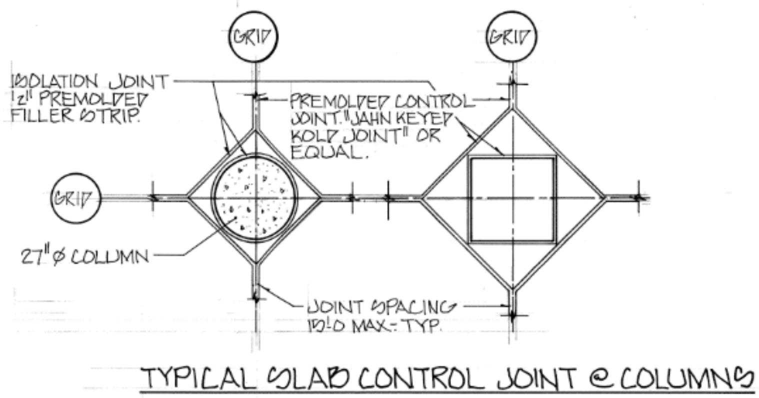


Figure 6. Typical Control Joint Detail

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.

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.

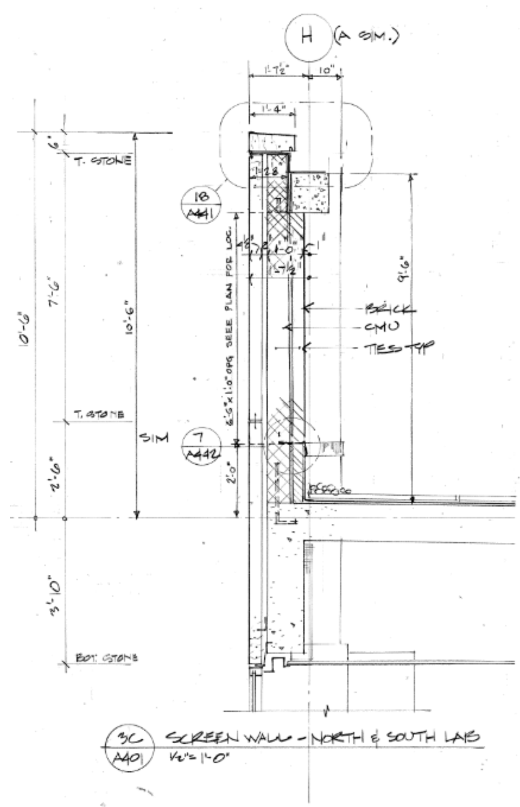


Figure 7. Example Section of a Parapet.

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.

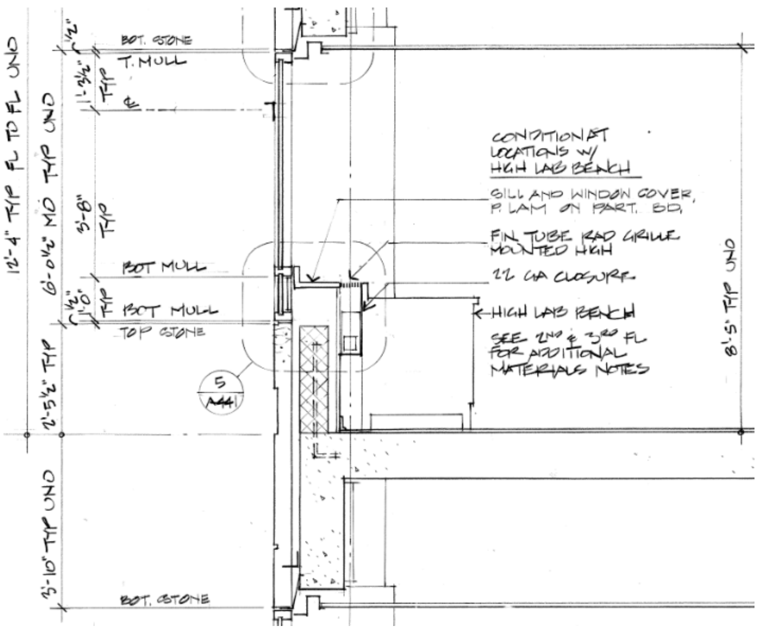


Figure 8. Example Section of Curtain Wall

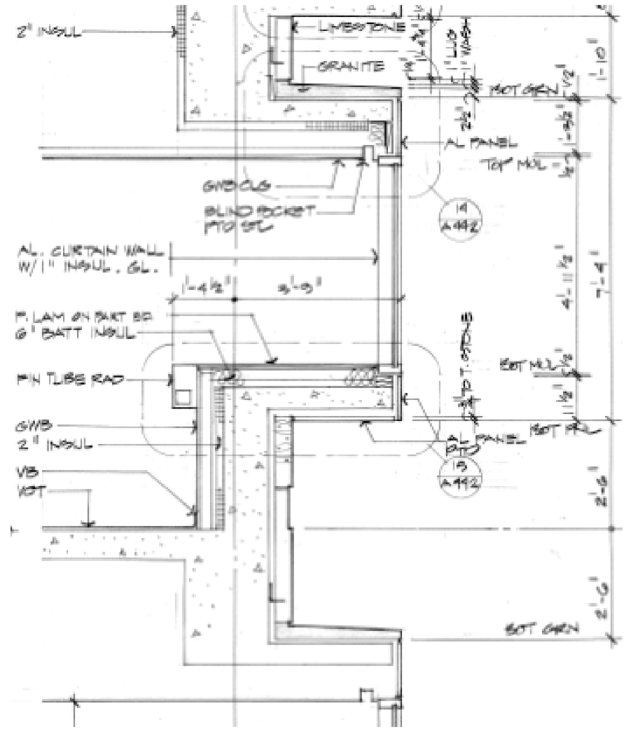


Figure 9. Example Section of Exterior Cladding

Support of Architectural Cylinder on Corner of Building

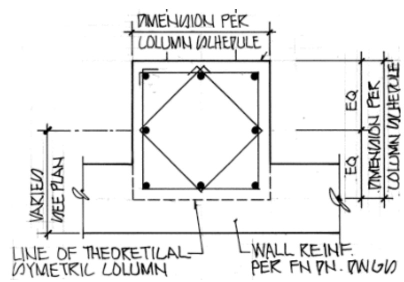


Figure 10. Illustration of Column Used for Support of Architectural Cylinder

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 2nd 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.

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.

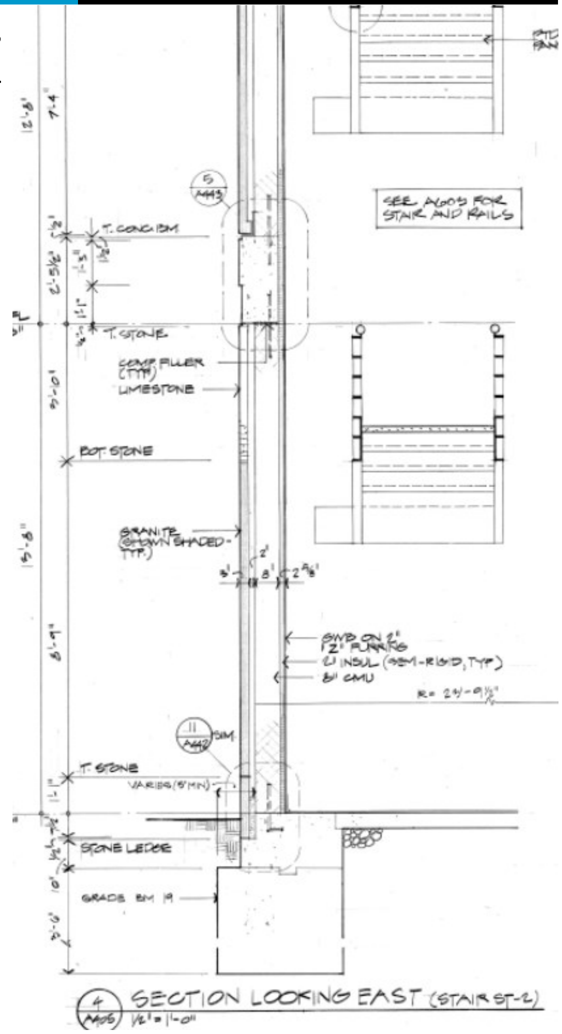


Figure 11. Section of Stairwell

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)

The proposed redesign of the Biomedical Research Building is of steel construction using both moment and braced frames as its lateral systems. It was important to maintain the same floor plan and column layout by extension. The BMR is also directly attached to another building by two hallways, so keeping the deflections due to lateral loads low on the north side of the building is important.

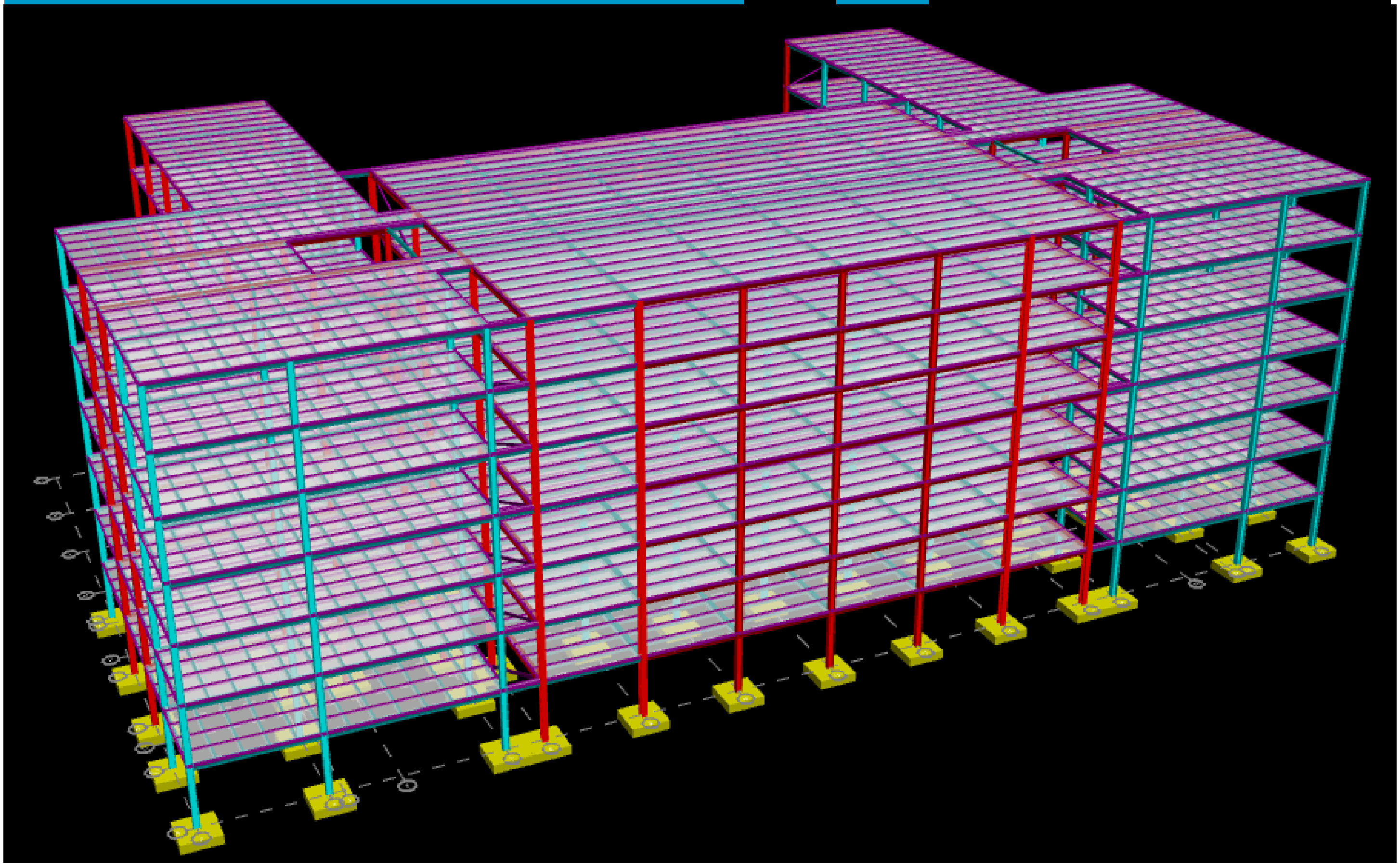
Starting off with designing the core 30' by 21' bays, assuming a 5' joist span, it was found using the Vulcraft catalogue, that a non composite system using a 3 inch thick slab on top of a 1.0C CSV Conform with 4x4-W2.9xw2.9 to support the 125 psf live load from the laboratories and classrooms. This floor slab, along with superimposed dead loads gave a total dead load of 65 psf. Using load combinations from ASCE 7-10, the total load acting on the slab was found to be 278 psf. Repeating the process for the two side office spaces in the building yields a total load of 238 psf, using a 100 psf live load, and using the same slab system with the same 65 psf unfactored dead load.

The joists spanning 21' in the core were assumed to be simply supported and unbraced. Applying the 278 psf core load to the beam yielded a 70 ft*kip moment. In the office spaces, longer spans exist up to 24', but under the lower load of 238 psf, they use the same size joists of W14x22. A significant drawback of the proposed joist depth being at 14" nominal, increases the depth of the structural system by 6". The existing system was a 12" thick slab for comparison. Joists under this loading condition deflect .625", satisfying L/360.

Gravity beams are W12x120 and are either fixed – fixed in the middle span 36' span or fixed on one end when they join a lateral frame either at either 30' end span to prevent loading the moment or braced frame with moments in two directions, as this will be seen where two lateral frames overlap. With these support conditions and loading in the core, the beam with fixed-fixed supports a286 ft*kip positive moment and a 572ft*kip negative moment. The fixed at one end beam supports a 336ft*kip positive moment and a 597ft*kip negative moment. Beams in the office spaces were also to be the same size to maintain uniformity in the project. Under worst case scenario in the core, beams support 83.4kips on each side. Beams under this loading condition deflect a maximum of .47", satisfying L/360.

Gravity columns vary in size from the top of the building at W14x90 up to W14x159 at the lowest story as axial forces grew. Reduced live loads were valuable in reducing the size of the columns, and were utilized from the 6th story to the bottom floor where columns were subject to the two floor support requirement to allow the reduction of heavy live loads over 125 psf. Interaction formulas were used in spreadsheets in developing sizes for both the core and office space columns. Both sets of columns were found to use the same sizes aside from the 4th story, so all columns were made to be the same in both spaces .

Story	Shape
7	w14x90
6	w14x99
5	w14x109
4	w14x120
3	w14x145
2	w14x145
1	w14x159



Lateral System

The lateral system, using RAM, underwent a few iterations to reach the goal of keeping drift under H/400, which was approximately 2.6” for the 88’ tall building. These iterations can be seen in the appendix. The original plan had 1 moment frame in each hall and 4 moment frames in the core to resist lateral forces in the North-South direction for a total of 6 moment frames, and a total of 6 moment frames in the East-West direction. All frames were to be pinned at the foundation due to the soil’s poor ability to resist loads. This initial design seemed intuitive as it kept the center of rigidity close to the center of mass, but due to the irregular shape of the building, the building developed large torsional forces that made the building drift well over the 2.6” limit.

Follow up designs add braced frames in the halls in the East-West directions to move the center of rigidity towards the north side of the building and exchange the parallel moment frames for a pair of braced frames on both sides of the core. This rearrangement led to reducing the drift to 1.4”, and lower on the north side where the BMR connect to the Crescent, fulfilling the goal of maintaining a small drift to keep the drift differential small between the two buildings.

These braced frames used only one diagonal member as to not block the path of halls that would be located underneath. There are two braced frames placed in front of glazing on the north side of the building, facing a courtyard, opposite of the southern architectural features, and relates to their statements in its own way as an exposed frame, architecturally.

Lateral columns were made to be larger than the gravity columns. Lateral beams were bumped up to a W14x120, and fixed-fixed to develop the moment frame action. The table of column sizes can be seen below.

Story	Shape
7	w14x233
6	w14x233
5	w14x257
4	w14x257
3	w14x257
2	w14x257
1	w14x283

The braced frames used the same beam and column sizes as the moment frames, but use HSS6.625X0.250 up to HSS9.625X0.250 for its bracing, size increasing as the bracing approaches the ground floor. These braces were pinned at both ends and only subject to axial loads from 30 kips up to 110 kips. STAAD was used extensively to develop the braced frames.

Story	Shape
7	HSS6.625X0.250
6	HSS6.625X0.250
5	HSS7.0X0.250
4	HSS7.5X0.250
3	HSS8.625X0.250
2	HSS8.625X0.322
1	HSS9.625xX.250

Overturning

A serious concern while reducing the building weight by almost a factor of 3, from approximately 33,000,000 lbs to about 12,000,000 lbs, it is very much necessary to check overturning moments from both wind and seismic . Results from this check can be seen below and on the next page.

X Direction Overturning in ft kips							
				Moment		Resisting Moment	
	Wind	Seismic	Arm	Wind	Seismic	Self Wt	Arm
7	24.8	53.89	13.7	2031	738	12000	47.5
6	27.4	56.12	26.3	2027	1475		
5	29.8	56.12	38.7	1917	2171		
4	32.6	56.12	51	1829	2862		
3	34.2	56.12	63.3	1675	3552		
2	36.1	56.13	75.7	1539	4249		
1	37.7	56.12	88	1392	4938		
Total				12413	19988	570000	Good

Y Direction Overturning in ft kips							
				Moment		Resisting Moment	
	Wind	Seismic	Arm	Wind	Seismic	Self Wt	Arm
7	14.5	53.89	13.7	1184.3	738.3	12000	47.5
6	16.0	56.12	26.3	1182.0	1476.0		
5	17.4	56.12	38.7	1117.7	2171.8		
4	19.0	56.12	51	1066.3	2862.1		
3	19.9	56.12	63.3	976.4	3552.4		
2	21.1	56.13	75.7	897.4	4249.0		
1	22.0	56.12	88	811.5	4938.6		
Total				7235.57	19988.21	570000	Good

HVAC

The proposed steel system make the slab thinner, and as a result, decreases the thermal mass of the building, making it more susceptible to temperature changes throughout the day. The building envelope remains the same other than the roof. Using the ASHRAE handbook to calculate the Cooling Load Temperature Differences for both systems, it shows that the proposed system increases the cooling load by 29%.

Roof Energy					
	Temp Range	Inside Temp	Outside temp	CLTD	Cooling Load
Existing roof	22	75	84	49	0.29
Proposed Roof	22	75	84	78	0.38
Percent Increase					29%

Cost

A major goal of the proposed design was to decrease in the cost of materials for the project. It was assumed that decreasing the weight of the building would lead to saving money. Takeoffs from the RAM Model were tallied up and totaled, and then combined with pricing from RS Means to find a total value of the proposed structural system. It was not necessary to calculate the entire price of the building as it was assumed that the envelope and the foundation would remain the same between the two systems . The table tallying all the steel can be seen on the next page.

Cost Totals				
Proposed	Num-ber	Length	Weight	Cost
W8X31	56	1109.1	34418	$50,250.28
W12X120	126	3938.2	473043	$690,642.80
W14X22	1967	38898	859029	$1,254,182.00
W14X38	112	2352	89636	$130,868.60
W14X90	55	678.3	61168	$89,305.28
W14X99	70	863.3	85487	$124,811.00
W14X109	245	5383.8	586270	$855,954.20
W14X120	133	2741.1	329263	$480,724.00
W14X132	16	197.3	26053	$38,037.38
W14X145	90	1817.3	224824	$328,243.00
W14X159	70	940.7	149480	$218,240.80
W14X176	16	218.7	38542	$56,271.32
W14X233	21	259	60369	$88,138.74
W14X257	28	350.7	90207	$131,702.20
W14X283	6	82	23243	$33,934.78
HSS6.625X0.250	12	395.7	6302	$9,200.92
HSS7.000X0.250	6	197.9	3333	$4,866.18
HSS7.500X0.250	6	197.9	3582	$5,229.72
HSS8.625X0.250	6	197.9	4134	$6,035.64
HSS8.625X0.322	6	198.6	5306	$7,746.76
HSS9.625X0.250	6	201	4699	$6,860.54
			3158388	$4,611,246.00

The assumed cost per pound of steel erected is $1.46, from averages obtained from RS Means. Add in the cost of the decking and the slab, which averages out to 3.5” and costs about $5.60 per square foot, puts the cost of the proposed structural system at about $5,229,500, or about $26 per square foot.

As for the concrete system, the volume of reinforced concrete was put at 8260 cubic yards, with 876 cubic yards being in columns. The cost per cubic yard of slab concrete was $582.50, while the cost per cubic yard in a column was $896.50, putting the cost of the concrete structural system at $5087000, or about $25.50 per square foot.

That leaves the proposed steel structure about $140,000 over the concrete system. However, the weight of the concrete is at 16750 tons, vs the total weight of the steel system of 6500 tons. One form of cost savings to be seen is in transportation of materials. The closest producers of concrete and steel are 50 and 30 miles away approximately from the building site, respectively. Using a 25 cent ton*mile fee for these materials, and the proportions of steel to concrete in each system produces the chart on the next page.

Transportation Costs				
	Price	Distance	Weight	Cost
Steel	0.25	30	6479	$73,000
Concrete	0.25	50	16728	$209100

This brings the cost of each system to be nearly identical, with each system being approximately $5,300,000, or about $26.56 per square foot for each structural system. But with the steel system being 6" thicker with its beam and joist depths, it still does not seem appealing as a proposal.

Carbon Footprint

While not a direct cost in terms of money, another potential benefit of decreasing the weight of the building is reducing its carbon footprint. All the materials that go into the building require energy to make and that energy requires fuel to be burned, and more often than not, that fuel releases CO2 into the air. The more energy the materials take, the more CO2 released. While the concrete system takes much less energy to produce than steel per pound, the system is about 2 and a half times heavier than the steel system, and can potentially have even more of an environmental impact than the steel system. The transportation of these materials can also impact the carbon footprint of the building.

Carbon Footprint							
	Material	Weight (lbs)	Emissions (lbs/lbs)	CO2 (lbs)	Emissions (CO2/ (Tons*mile))	Transportation	Total Emissions (lbs)
Steel System	Concrete	8750000	0.192	1680000			
	Steel	3150000	1.7	5355000			
				7035000	0.6	158507	7193507
Concrete System	Concrete	32339930	0.192	6209267			
	Steel	1115170	1.7	1895789			
				8105056	0.6	299034	8404089

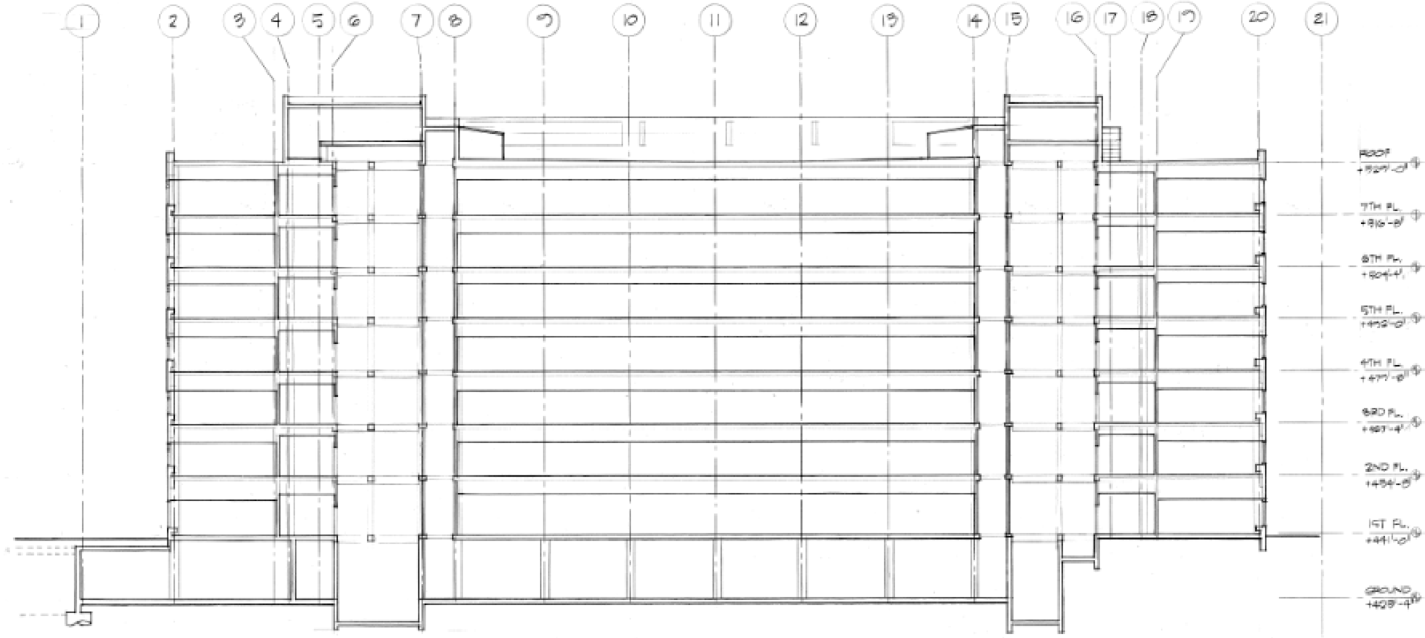
As shown above, the steel system provides a 16.4% improvement in overall greenhouse gasses emitted in construction of this building for virtually no additional construction cost. In a previous section, it was mentioned that the steel structural system increased the HVAC load of the roof by 29%, and thus would have a negative environmental impact that would negate the benefits of switching over the building's lifespan. However, the building receives its power from a nearby nuclear power plant, and thus produces no emissions from consuming power.

Conclusion

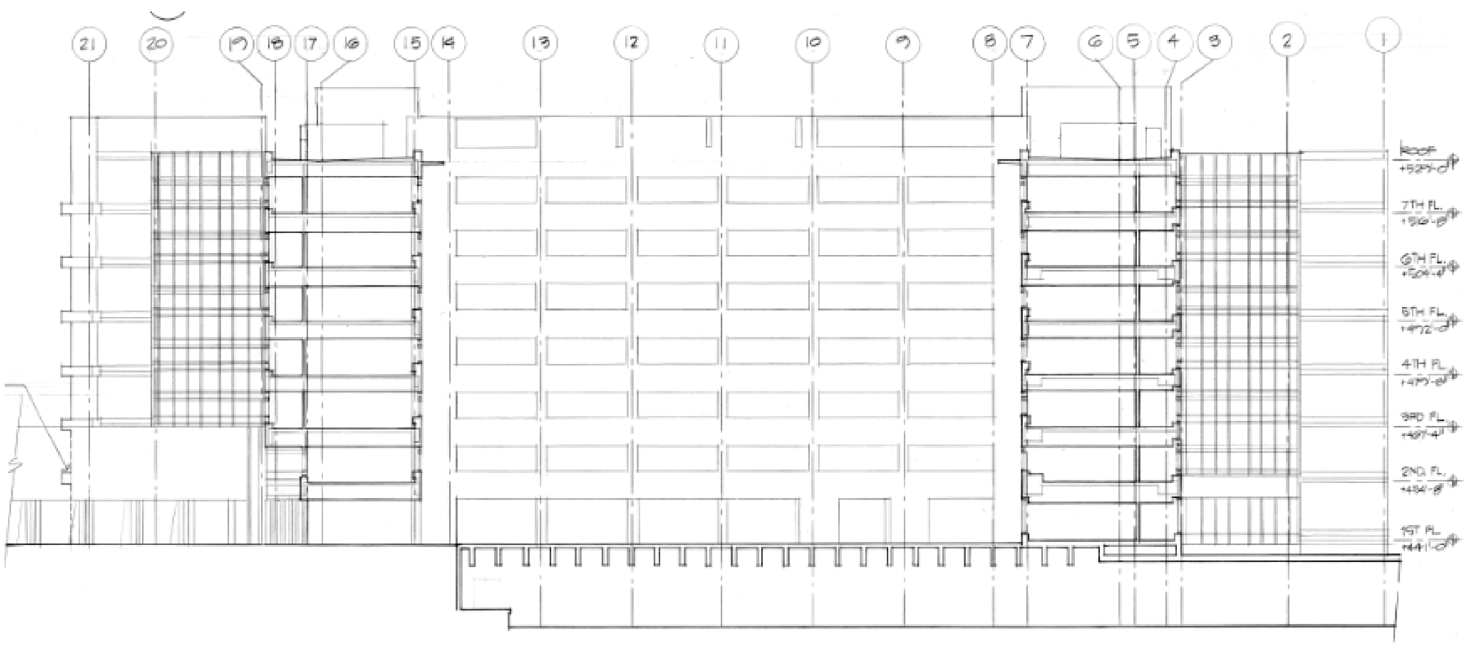
With no difference of cost between the two structural systems, a decision to choose between these systems is purely based on the pros and cons of each system, and the values of the building owner. Losing only 6 inches of ceiling height out of a 12' – 4" floor to floor height and approximately $500 a month over the increased HVAC loads from switching from a concrete system to a steel system in order to prevent 600 tons of CO2 being released sounds like a fair trade. Even if the pros and cons are considered to be too small to be concerned about, every little bit helps in preventing the emissions of additional CO2 into the atmosphere, and as such this report recommends the switching from concrete to steel as a structural system.

Appendix

Elevations

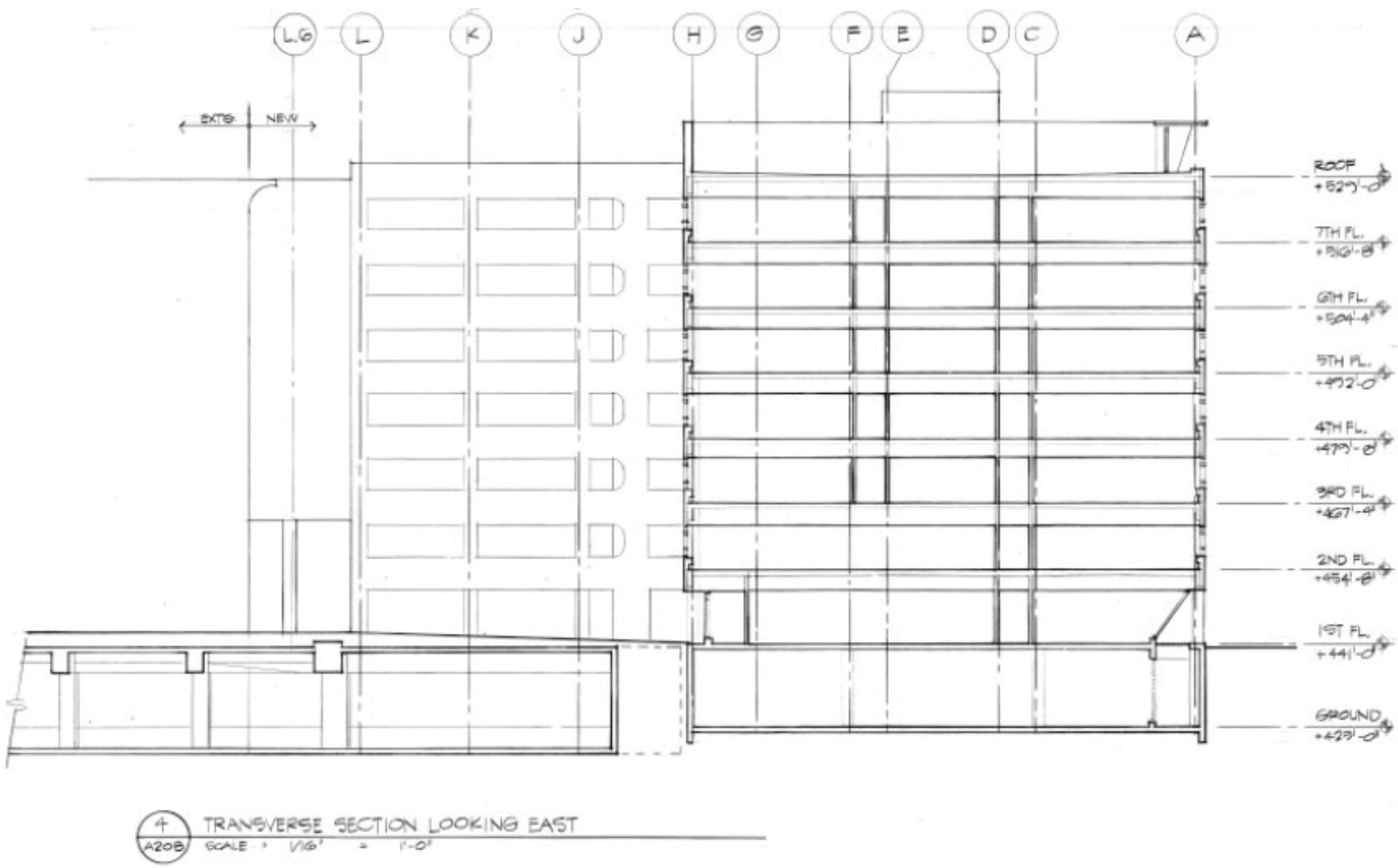


1 A208 LONGITUDINAL SECTION LOOKING NORTH
SCALE : 1/16" = 1'-0"

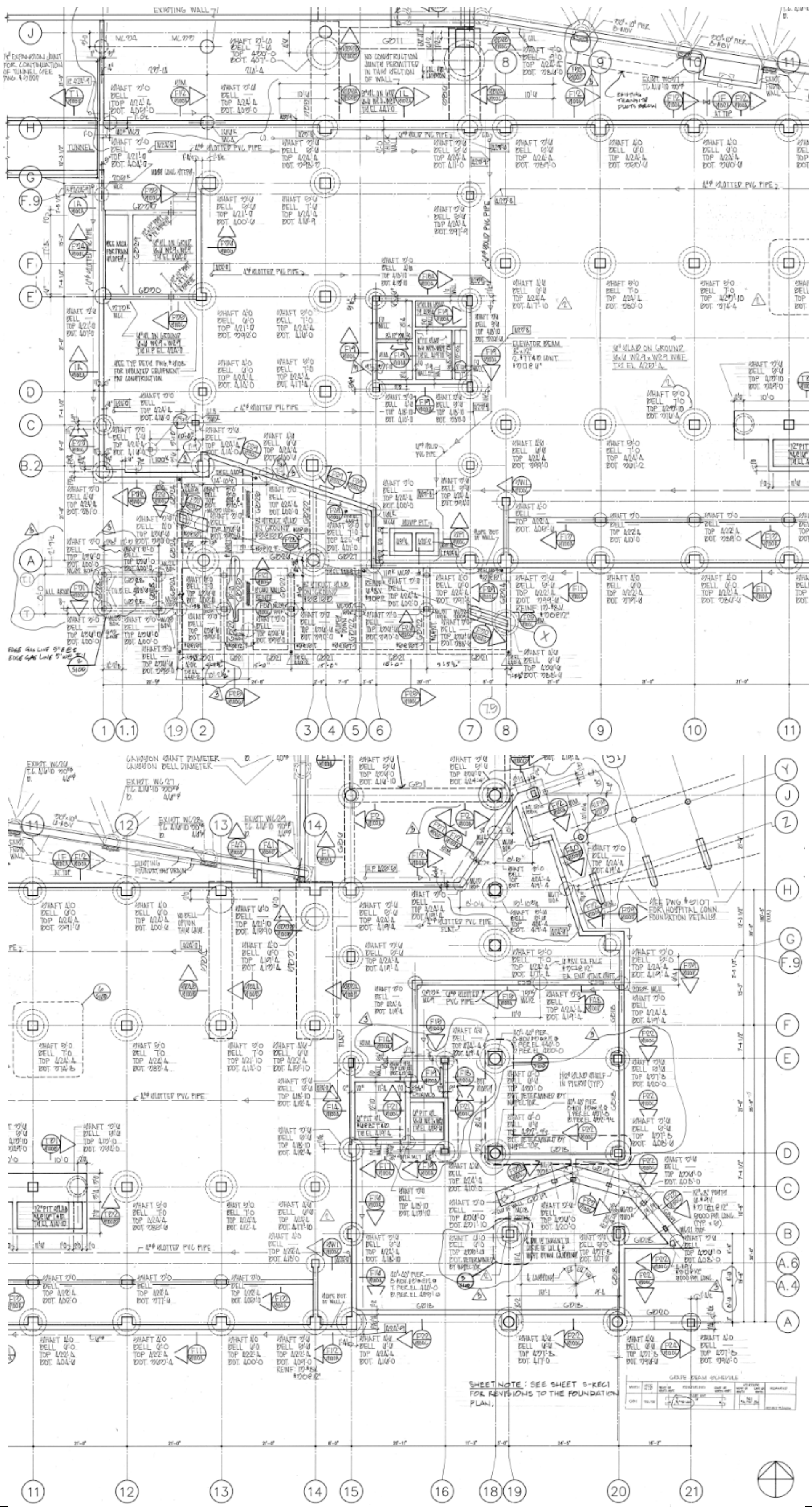


2 A208 SECTION THRU CONNECTORS LOOKING SOUTH
SCALE : 1/16" = 1'-0"

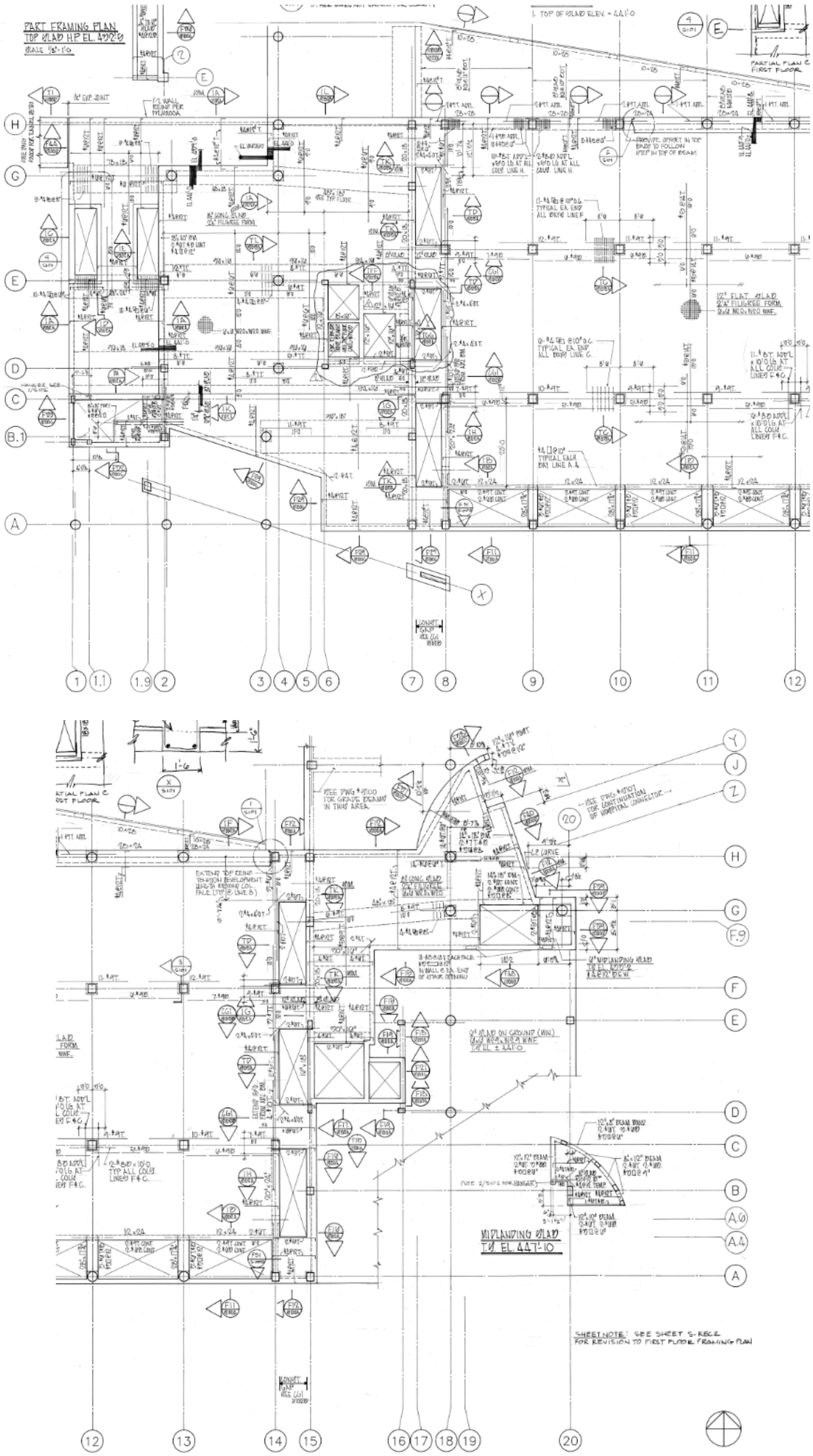
Elevations


L.G
L
K
J
H
G
F
E
D
C
A
EXTG
NEW
ROOF
+529'-0"
7TH FL.
+516'-8"
6TH FL.
+504'-4"
5TH FL.
+492'-0"
4TH FL.
+479'-8"
3RD FL.
+467'-4"
2ND FL.
+454'-8"
1ST FL.
+441'-0"
GROUND
+429'-0"
4
A208
TRANSVERSE SECTION LOOKING EAST
SCALE : 1/16" = 1'-0"

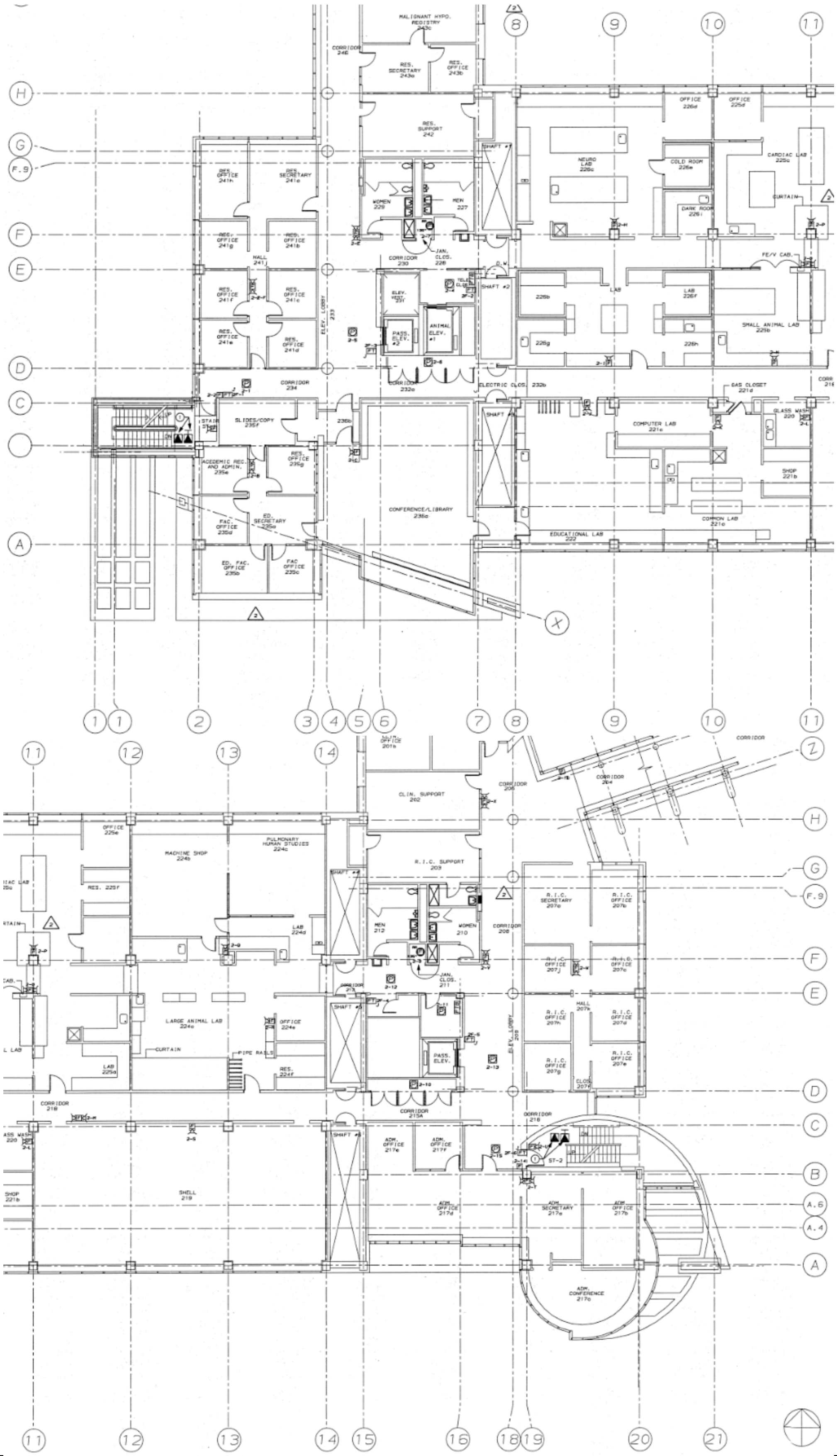
Foundation Plan (Ground Floor)



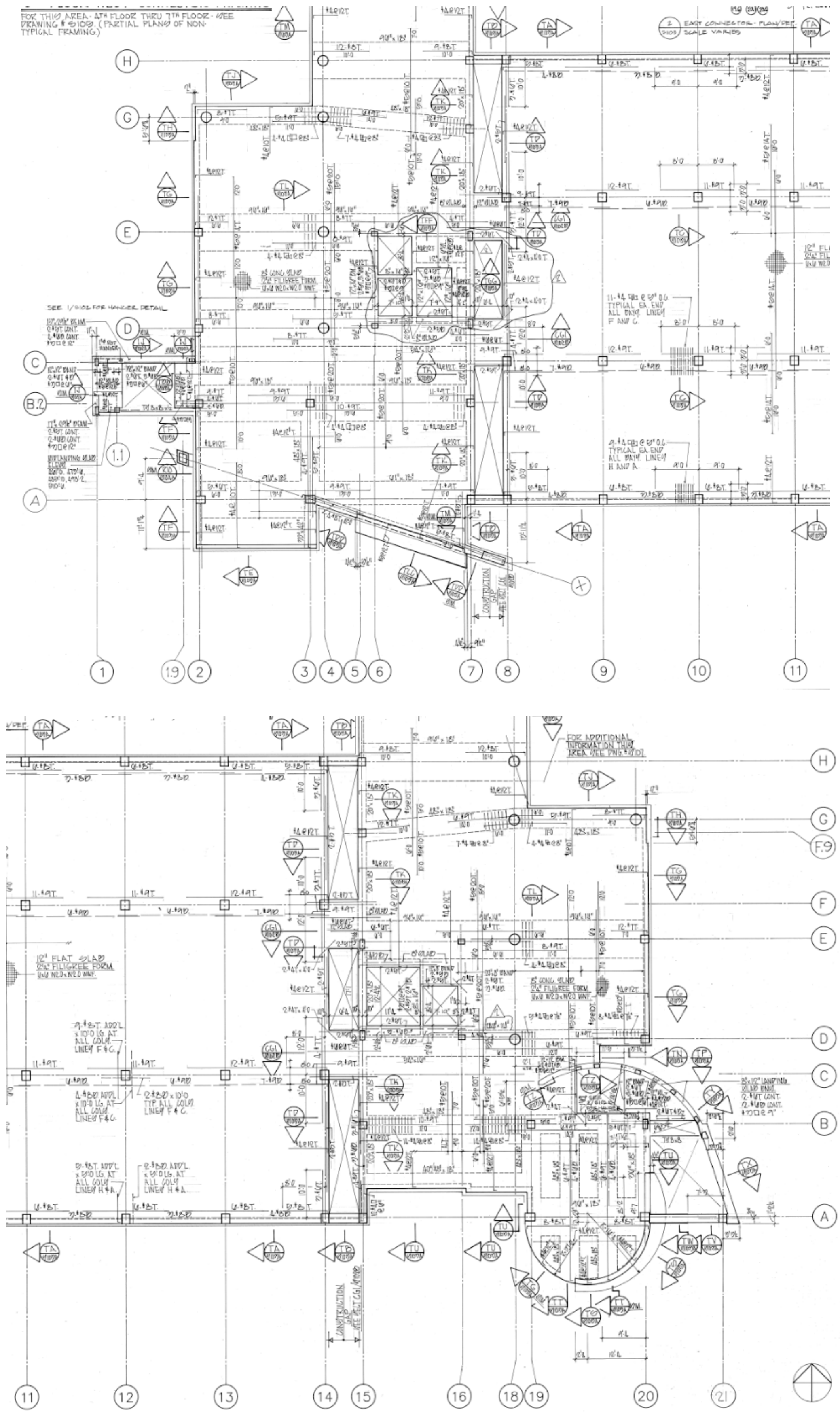
First Floor Plan



Second Floor Plan



Typical 3rd through 7th Floor Plans



20-24x25

1. Joist:

$w = 252 \cdot 5 = 1265$ plf

Assume SS:

$m = \frac{1265 \cdot 21^2}{8} = 69.7 \Rightarrow 70$ K·ft

use a W8x31

Beam:

12psf dead

$w = 252 \cdot 21 = 5300$ plf

Assume fixed at one end

$m^+ = \frac{9(5300) \cdot 30^2}{128} = 336$ Kip·ft

$m^- = \frac{5300 \cdot 30^2}{8} = 597$ Kip·ft

use a W14x120 / W18x119

facade 12.33·20 Assume 40psf

$\frac{3 \cdot (1.265 + 1.6 \cdot 125) + 12.33 \cdot 20 \cdot 40 \cdot 1.2}{12} = 422.36$

$+80 = 502.4$ ftKips

W14x90 @ facade joists

417 ftKips

W14x38-21

W14x68-36

2. Beam: Joists are to be the same

$w = 252 \cdot 21 = 5300$ plf

Assume fixed-fixed

$m^+ = \frac{5300 \cdot 36^2}{24} = 286.2$

$m^- = \frac{5300 \cdot 36^2}{12} = 572.4$ → 715 ft·Kips W14x109?

use a W12x120

3. Joist:

$w = 205 \cdot 5 = 1025$

Assume SS:

$m = \frac{1025 \cdot 21^2}{8} = 56.5$ ft·Kip

use a W8x28

3. beam: $w = 21 \cdot 205 = 4305$ lbs/ft

Assume fixed on one end

$$M^+ = \frac{9(4305)32^2}{128} = 310 \text{ ft}\cdot\text{Kips}$$

$$M^- = \frac{4305(32)^2}{8} = 551 \text{ ft}\cdot\text{Kips}$$

use a W12x120

4. Joists: $w = 205 \cdot 5 = 1025$ lbs/ft

Assume fixed on one end

$$M^+ = \frac{9(1025)25^2}{128} = 45 \text{ ft}\cdot\text{K}$$

$$M^- = \frac{1025(25)^2}{8} = 80 \text{ ft}\cdot\text{K}$$

use a W8x35

Beam: $w = 205 \cdot 23 = 4715$

Assume fixed-fixed:

$$M^+ = \frac{4715(24)^2}{12} = 226.3 \text{ ft}\cdot\text{Kips}$$

$$M^- = \frac{4715(24)^2}{24} = 113.2 \text{ ft}\cdot\text{Kips}$$

use a W10x49

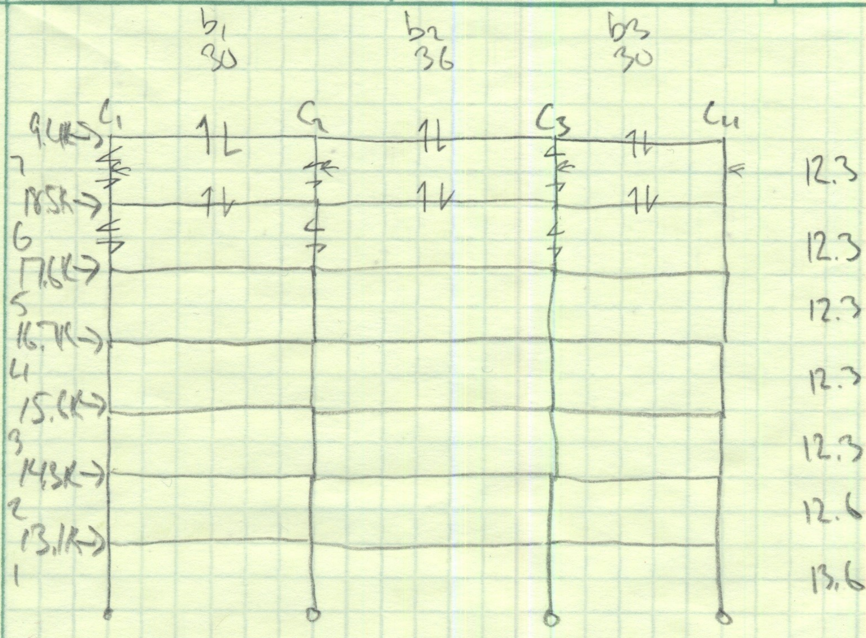
5. $w = 5 \cdot 205 = 1025$ lbs/ft

Joists:

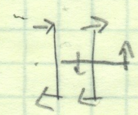
SS

$$\frac{1025 \cdot 16^2}{8} = 32.8 \text{ ft}\cdot\text{Kips}$$

use a W8x21



$\frac{9.4}{6} = 1.57K$ $\frac{2(9.4)}{6} = 3.13K$



$\frac{27.5}{2}$ 11 21

= 13.1 5.5 10.5

6.3
12.3
11.7
11.1
10.4
9.5
8.7
70 — ~~2~~ 280

$\frac{6365}{280} = 22 \rightarrow$ table 2-1 gives $B_2 = 1.1$ with $H/400$

$$\frac{37.4\left(\frac{1}{.062}\right) + \left(\frac{1}{.062}\right)58.4}{\left(\frac{1}{.062}\right)+\left(\frac{1}{.062}\right)} = \cancel{66.6}\ 47.9$$

1545.2

$\frac{1545.2}{66.6} = 23.2$

$$\frac{37.4\left(\frac{1}{.062}\right)+\left(\frac{1}{.062}\right)58.4 + 156.75\left(\frac{1}{x}\right)}{\left(\frac{1}{.062}\right)\left(\frac{1}{.062}\right)+\frac{1}{x}} = 57.4$$

$$\frac{1545.2 + \frac{158.75}{x}}{32.3 + \frac{1}{x}} = 57.4$$

$$1545.2 + \frac{158.75}{x} = 127.4(32.3) + \frac{157.4}{x}$$

4115

$1545.2x + 158.75 = 4115x + 57.4$

$2569.8x = 101.35$

$x = .039'' = \Delta$

127.4, 46.1 (153840

$-\left(\begin{array}{l}15 \cdot 36.8.2 \\ 48 \cdot 28 \cdot 20 \cdot 2 \\ 81 \cdot 30 \cdot 8 \cdot 2 \\ 12 \cdot 25 \cdot 2 \cdot 90\end{array}\right.$

11.84256 — 4153600

$\frac{257.96 \cdot 48 + 2.30 \cdot 60 \cdot 126}{257.96 + 2.3060}$

257.4'

$R_1 = 6.18$

$R_2 = 25.64$

$R_4 = 115.8$

Main

561

1390

5/384 SS

W8x31 110

$\frac{5wL^4}{384EI} = \frac{5(1390)(20)^4(20736)}{384(29000000)(110)} = 1.57''$ 1.13'' 7.66

1000 lLonly

$(L/360)$

use W14x22 Δ = .625

10x39

W12x120

~~W14x211~~

~~W12x252~~

for 30' span

+ 36' span

$\frac{(5560)(30)^4(1728)}{384(29000000)(1070)} = 3.3''$

~~2.3''~~ .47'' < 1.0 ✓

.47 <

1/384 f.f

1/185 1 side fixed → .975 < 1.0

.974 < 1.2

Δ/good

and

W8x31 → W14x22

HSS6.625x.28

~.28

35.7

.43

HSS7x.125

0 1 ~~HSS6x.188~~ HSS6.625x.125

2

3

4 ~~HSS6x.250~~ HSS6.625x.188

5

6 HSS6.875x.188

7 ~~HSS6.625x.312~~ ~~HSS6.875x.250~~ → HSS7.5x.25

.36

node 42 | 232.92, 58.4

Interaction Tables

Story	Axial	Moment X	Shape	I	p	bx	Moment Y	by	
			Frame.core.edge						
7	73.5138	502.4	w14x90	0.910501	0.937	1.55	0	3.26	0.068882
6	147.0276	502.4	w14x90	0.944942	0.937	1.55	0	3.26	0.137765
5	220.5414	502.4	w14x90	0.985367	0.937	1.55	0	3.26	0.206647
4	294.0552	502.4	w14x99	0.944141	0.853	1.38	0	2.85	0.250829
3	367.569	502.4	w14x109	0.90245	0.774	1.23	0	2.56	0.284498
2	441.0828	502.4	w14x109	0.95935	0.774	1.23	0	2.56	0.341398
1	514.5966	502.4	w14x120	0.923935	0.702	1.12	0	2.32	0.361247
			Frame.core						
7	123.354	562	w14x99	0.925115	0.853	1.38	0	2.85	0.105221
6	246.708	562	w14x99	0.986002	0.853	1.38	0	2.85	0.210442
5	370.062	562	w14x109	0.977688	0.774	1.23	0	2.56	0.286428
4	493.416	562	w14x120	0.975818	0.702	1.12	0	2.32	0.346378
3	616.77	562	w14x132	0.961119	0.638	1.01	0	2.1	0.393499
2	740.124	562	w14x145	0.936635	0.573	0.912	0	1.78	0.424091
1	863.478	562	w14x159	0.915811	0.523	0.826	0	1.62	0.451599
			Frame.core.joint						
7	123.354	562	w14x233	0.970603	0.355	0.544	502.4	1.07	0.043791
6	246.708	562	w14x233	0.992499	0.355	0.544	502.4	1.07	0.087581
5	370.062	562	w14x257	0.912154	0.321	0.487	502.4	0.964	0.11879
4	493.416	562	w14x257	0.931952	0.321	0.487	502.4	0.964	0.158387
3	616.77	562	w14x257	0.95175	0.321	0.487	502.4	0.964	0.197983
2	740.124	562	w14x257	0.995587	0.321	0.487	502.4	0.964	0.23758
1	863.478	562	w14x283	0.931442	0.291	0.437	502.4	0.865	0.251272

Interaction Tables

Story	Axial	Moment X	Shape	I	p	bx	Moment Y	by	
			frame.side.hall						
7	99.54	497	w14x90	0.913278	0.937	1.55	0	3.26	0.093269
6	199.08	497	w14x90	0.956888	0.937	1.55	0	3.26	0.186538
5	298.62	497	w14x99	0.940583	0.853	1.38	0	2.85	0.254723
4	398.16	497	w14x109	0.919486	0.774	1.23	0	2.56	0.308176
3	497.7	497	w14x120	0.99653	0.774	1.23	0	2.56	0.38522
2	597.24	497	w14x132	0.883009	0.638	1.01	0	2.1	0.381039
1	696.78	497	w14x132	0.946516	0.638	1.01	0	2.1	0.444546
			frame.side.core						
7	99.54	432.7	w14x90	0.801155	0.937	1.55	0	3.26	0.093269
6	199.08	432.7	w14x90	0.857223	0.937	1.55	0	3.26	0.186538
5	298.62	432.7	w14x99	0.851849	0.853	1.38	0	2.85	0.254723
4	398.16	432.7	w14x109	0.840397	0.774	1.23	0	2.56	0.308176
3	497.7	432.7	w14x120	0.917441	0.774	1.23	0	2.56	0.38522
2	597.24	432.7	w14x132	0.818066	0.638	1.01	0	2.1	0.381039
1	696.78	432.7	w14x132	0.881573	0.638	1.01	0	2.1	0.444546
			frame.side.hall.joint						
7	99.54	497	w14x233	0.920096	0.355	0.544	497	1.07	0.035337
6	199.08	497	w14x233	0.937764	0.355	0.544	497	1.07	0.070673
5	298.62	497	w14x233	0.955433	0.355	0.544	497	1.07	0.10601
4	398.16	497	w14x233	0.973101	0.355	0.544	497	1.07	0.141347
3	497.7	497	w14x233	0.99077	0.355	0.544	497	1.07	0.176684
2	597.24	497	w14x257	0.907147	0.321	0.487	497	0.964	0.191714
1	696.78	497	w14x257	0.89906	0.291	0.437	497	0.964	0.202763

Gravity Columns

Story	Axial	Moment	Shape	Interaction	p	b
7	164.934	450	w14x90	0.852043	0.937	1.55
6	329.868	450	w14x99	0.902377	0.853	1.38
5	494.802	450	w14x109	0.936477	0.774	1.23
4	659.736	450	w14x120	0.967135	0.702	1.12
3	824.67	450	w14x145	0.882936	0.573	0.912
2	989.604	450	w14x145	0.977443	0.573	0.912
1	1154.538	450	w14x159	0.975523	0.523	0.826

Story	Axial	Moment	Shape	Interaction	p	b
7	129.78	535.5	w14x90	0.951629	0.937	1.55
6	259.56	535.5	w14x99	0.960395	0.853	1.38
5	389.34	535.5	w14x109	0.960014	0.774	1.23
4	519.12	535.5	w14x120	0.964182	0.702	1.12
3	648.9	535.5	w14x145	0.860196	0.573	0.912
2	778.68	535.5	w14x145	0.93456	0.573	0.912
1	908.46	535.5	w14x159	0.917448	0.523	0.826

story	floor h	q	Kz	qz	p	F	each lateral frame y	x
7	12.3333	88	0.96	27.6265	23.75879	37653.62	6.3	5.779733
6	12.3333	75.6667	0.92	26.47539	22.76884	36084.71	6.0	5.538911
5	12.3333	63.3333	0.87	25.03651	21.5314	34123.59	5.7	5.237883
4	12.3333	51	0.83	23.88541	20.54145	32554.69	5.4	4.997061
3	12.3333	38.6667	0.76	21.87098	18.80904	29809.11	5.0	4.575622
2	12.6667	26.3333	0.68	19.56877	16.82914	27392.3	4.6	4.204648
1	13.6667	13.6667	0.57	16.40323	14.10678	24773.92	4.1	3.802732

	c1		c2		c3		c4	
frame 1	V K	M Kft	V K	M Kft	V K	M Kft	V K	M Kft
7	1.0	6.4	2.1	12.9	2.1	12.9	1.0	6.4
6	2.0	12.6	4.1	25.3	4.1	25.3	2.0	12.6
5	3.0	18.5	6.0	37.0	6.0	37.0	3.0	18.5
4	3.9	24.1	7.8	48.1	7.8	48.1	3.9	24.1
3	4.7	29.2	9.5	58.3	9.5	58.3	4.7	29.2
2	5.5	34.8	11.0	69.5	11.0	69.5	5.5	34.8
1	6.2	42.2	12.4	84.4	12.4	84.4	6.2	42.2

b1		b2		b3	
V K	M Kft	V K	M Kft	V K	M Kft
0.4	6.4	0.4	6.4	0.4	6.4
1.3	19.1	1.1	19.1	1.3	19.1
2.1	31.1	1.7	31.1	2.1	31.1
2.8	42.5	2.4	42.5	2.8	42.5
3.5	53.2	3.0	53.2	3.5	53.2
4.3	63.9	3.6	63.9	4.3	63.9
5.1	77.0	4.3	77.0	5.1	77.0

frame 2	c1		c2		c3		c4	
7	1.0	6.4	2.1	12.9	2.1	12.9	1.0	6.4
6	2.0	12.6	4.1	25.3	4.1	25.3	2.0	12.6
5	3.0	18.5	6.0	37.0	6.0	37.0	3.0	18.5
4	3.9	24.1	7.8	48.1	7.8	48.1	3.9	24.1
3	4.7	29.2	9.5	58.3	9.5	58.3	4.7	29.2
2	5.5	34.8	11.0	69.5	11.0	69.5	5.5	34.8
1	6.2	42.2	12.4	84.4	12.4	84.4	6.2	42.2

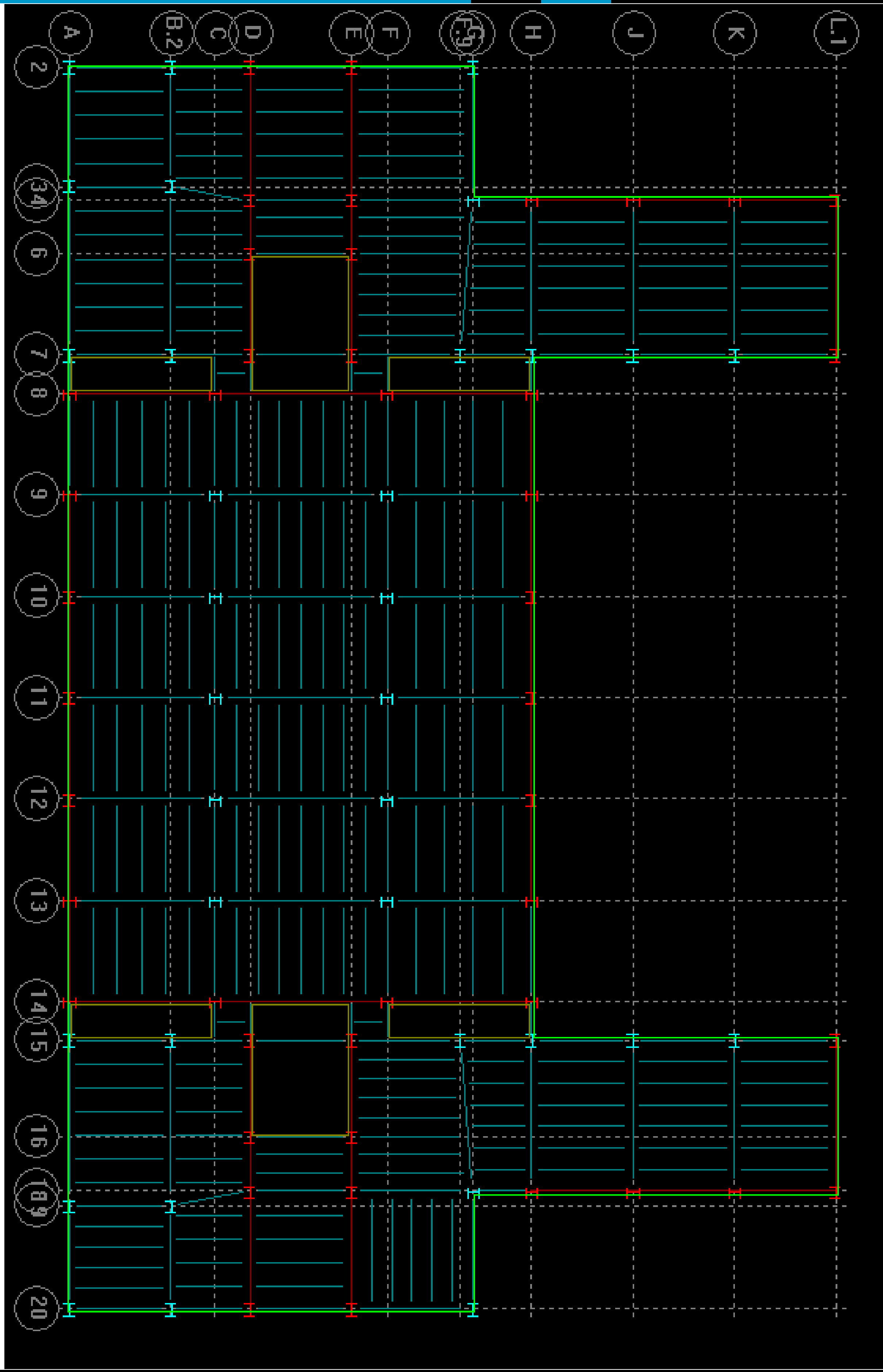
b1		b2		b3	
0.6	6.4	0.6	6.4	0.6	6.4
1.8	19.1	1.8	19.1	1.8	19.1
3.0	31.1	3.0	31.1	3.0	31.1
4.1	42.5	4.1	42.5	4.1	42.5
5.1	53.2	5.1	53.2	5.1	53.2
6.1	63.9	6.1	63.9	6.1	63.9
7.3	77.0	7.3	77.0	7.3	77.0

frame 3	c1		c2		c3		c4	
7	1.0	5.9	1.9	11.9	1.9	11.9	1.0	5.9
6	1.9	11.6	3.8	23.3	3.8	23.3	1.9	11.6
5	2.8	17.0	5.5	34.0	5.5	34.0	2.8	17.0
4	3.6	22.2	7.2	44.3	7.2	44.3	3.6	22.2
3	4.4	26.9	8.7	53.7	8.7	53.7	4.4	26.9
2	5.1	32.0	10.1	64.0	10.1	64.0	5.1	32.0
1	5.7	38.9	11.4	77.8	11.4	77.8	5.7	38.9

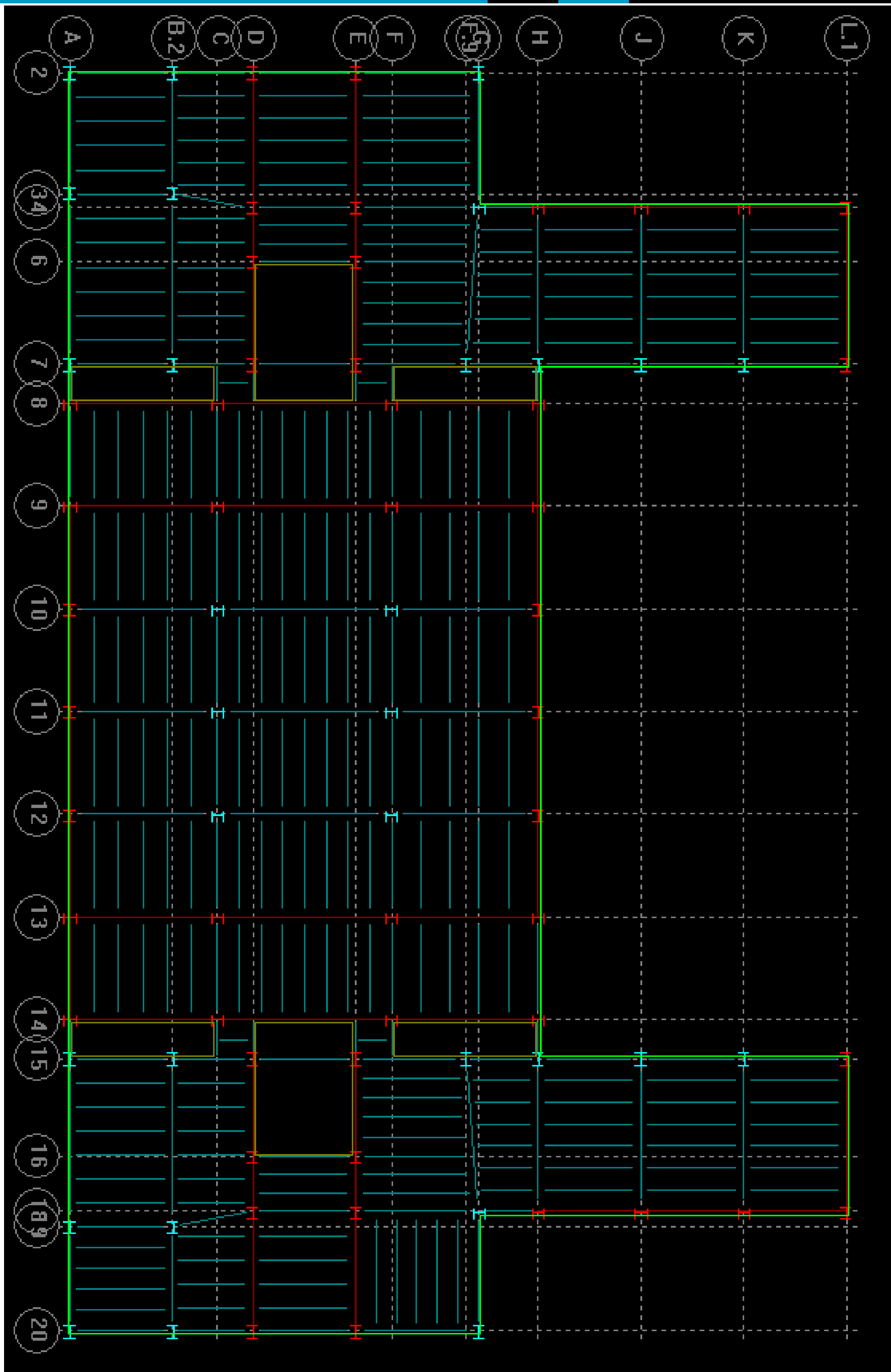
b1		b2		b3	
0.4	5.9	1.1	5.9	0.6	5.9
1.3	17.6	3.2	17.6	1.7	17.6
2.0	28.6	5.2	28.6	2.7	28.6
2.8	39.2	7.1	39.2	3.7	39.2
3.5	49.0	8.9	49.0	4.7	49.0
4.2	58.9	10.7	58.9	5.6	58.9
5.1	70.9	12.9	70.9	6.8	70.9

rame 4	c1		c2		c3		c4		c5		c6
7	0.6	3.6	1.2	7.1	1.2	7.1	1.2	7.1	1.2	7.1	0.6
6	1.1	7.0	2.3	14.0	2.3	14.0	2.3	14.0	2.3	14.0	1.1
5	1.7	10.2	3.3	20.4	3.3	20.4	3.3	20.4	3.3	20.4	1.7
4	2.2	13.3	4.3	26.6	4.3	26.6	4.3	26.6	4.3	26.6	2.2
3	2.6	16.1	5.2	32.2	5.2	32.2	5.2	32.2	5.2	32.2	2.6
2	3.0	19.2	6.1	38.4	6.1	38.4	6.1	38.4	6.1	38.4	3.0
1	3.4	23.3	6.8	46.7	6.8	46.7	6.8	46.7	6.8	46.7	3.4

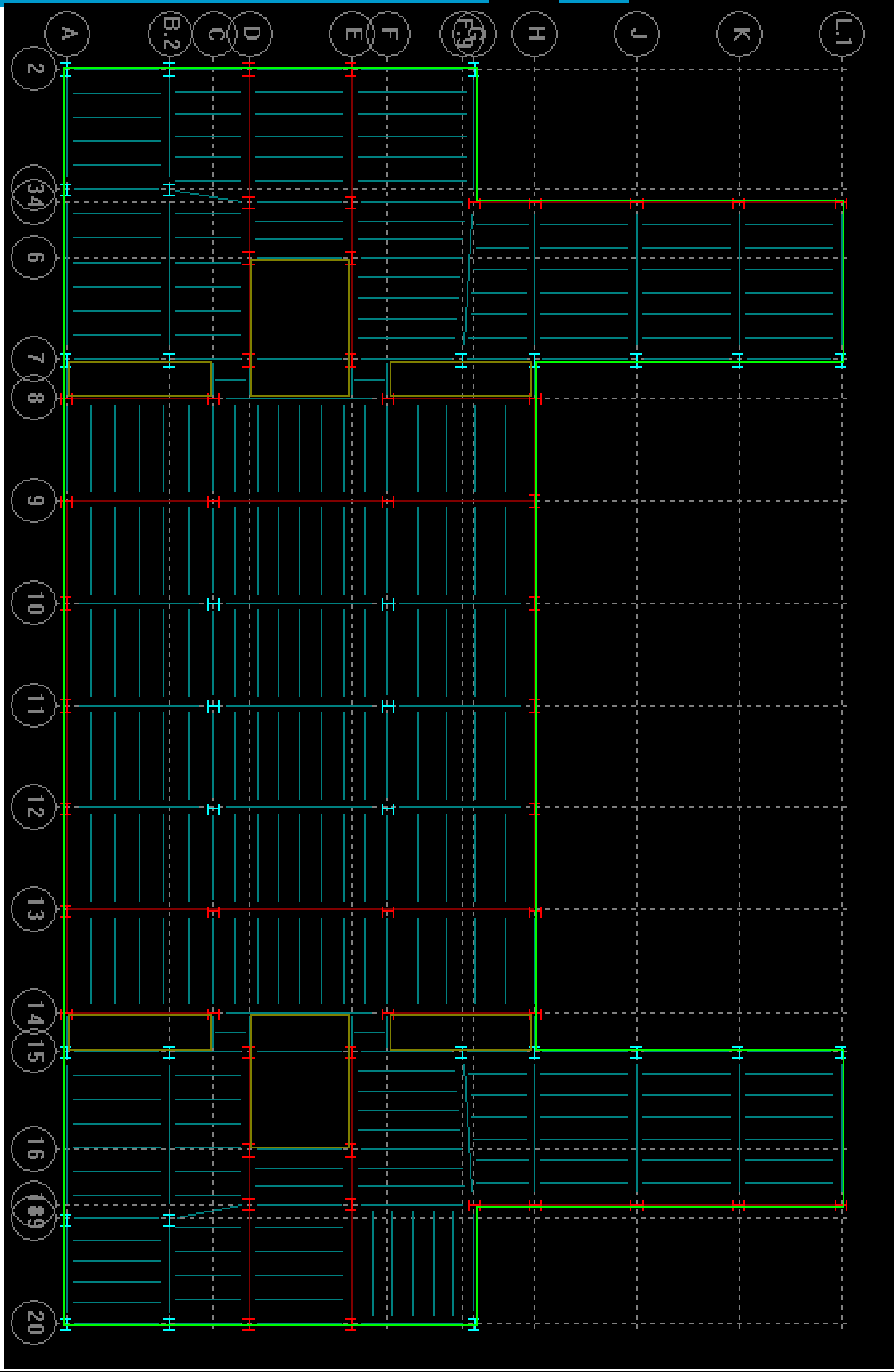
b1		b2		b3		b4		b5	
0.3	3.6	0.3	3.6	0.3	3.6	0.3	3.6	0.3	3.6
1.0	10.5	1.0	10.5	1.0	10.5	1.0	10.5	1.0	10.5
1.6	17.2	1.6	17.2	1.6	17.2	1.6	17.2	1.6	17.2
2.2	23.5	2.2	23.5	2.2	23.5	2.2	23.5	2.2	23.5
2.8	29.4	2.8	29.4	2.8	29.4	2.8	29.4	2.8	29.4
3.4	35.3	3.4	35.3	3.4	35.3	3.4	35.3	3.4	35.3
4.1	42.5	4.1	42.5	4.1	42.5	4.1	42.5	4.1	42.5



Final Design



2nd Design


A
B.2
C
D
E
F
H
J
K
L.1
2
6
7
8
9
10
11
12
13
14
15
16
20

CRITERIA:

Rigid End Zones: Ignore Effects
Member Force Output: At Face of Joint
P-Delta: Yes Scale Factor (DL): 1.00 Scale Factor (LL): 1.00
Scale Factor (Roof): 1.00 Scale Factor (Snow): 1.00
Ground Level: Base
Mesh Criteria :
Max. Distance Between Nodes on Mesh Line (ft) : 4.00
Merge Node Tolerance (in) : 0.0100
Geometry Tolerance (in) : 0.0050
Walls Out-of-plane Stiffness Not Included in Analysis.
Sign not considered for Dynamic Load Case Results.

LOAD CASE DEFINITIONS:

E1	Seismic	EQ_IBC09_X_+E_F
E2	Seismic	EQ_IBC09_X_-E_F
E3	Seismic	EQ_IBC09_Y_+E_F
E4	Seismic	EQ_IBC09_Y_-E_F
W1	Wind	Wind_IBC09_1_X
W2	Wind	Wind_IBC09_1_Y
W3	Wind	Wind_IBC09_2_X+E
W4	Wind	Wind_IBC09_2_X-E
W5	Wind	Wind_IBC09_2_Y+E
W6	Wind	Wind_IBC09_2_Y-E
W7	Wind	Wind_IBC09_3_X+Y
W8	Wind	Wind_IBC09_3_X-Y
W9	Wind	Wind_IBC09_4_X+Y_CW
W10	Wind	Wind_IBC09_4_X+Y_CCW
W11	Wind	Wind_IBC09_4_X-Y_CW
W12	Wind	Wind_IBC09_4_X-Y_CCW

Level: Roof, Diaph: 1

Center of Mass (ft): (127.36, 46.10)

LdC	Disp X in	Disp Y in	Theta Z rad
E1	3.14417	-0.05256	0.00159
E2	3.33878	-0.05898	0.00178
E3	0.20400	1.88350	0.00019
E4	-0.31549	1.90062	-0.00030
W1	0.75619	-0.01338	0.00038
W2	-0.02517	1.45898	-0.00003
W3	0.51663	-0.00823	0.00024
W4	0.61767	-0.01184	0.00034
W5	0.42526	1.07835	0.00042
W6	-0.46301	1.11012	-0.00046
W7	0.54827	1.08420	0.00027

W8	0.58602	-1.10427	0.00030
W9	0.04021	0.82641	-0.00017
W10	0.78219	0.79988	0.00057
W11	0.06852	-0.81494	-0.00014
W12	0.81051	-0.84146	0.00060

Level: 7th, Diaph: 1

Center of Mass (ft): (127.36, 46.10)

LdC	**Disp X** **in**	**Disp Y** **in**	**Theta Z** **rad**
E1	2.37282	-0.04271	0.00112
E2	2.51140	-0.04823	0.00127
E3	0.13952	1.39279	0.00015
E4	-0.23040	1.40752	-0.00024
W1	0.60173	-0.01086	0.00029
W2	-0.02163	1.14473	-0.00002
W3	0.41309	-0.00660	0.00017
W4	0.48951	-0.00970	0.00026
W5	0.32060	0.84484	0.00034
W6	-0.35305	0.87225	-0.00038
W7	0.43508	0.85040	0.00020
W8	0.46753	-0.86669	0.00023
W9	0.04503	0.64924	-0.00015
W10	0.60759	0.62636	0.00045
W11	0.06936	-0.63858	-0.00013
W12	0.63192	-0.66146	0.00048

Level: 6th, Diaph: 1

Center of Mass (ft): (127.36, 46.10)

LdC	**Disp X** **in**	**Disp Y** **in**	**Theta Z** **rad**
E1	1.53222	-0.02703	0.00063
E2	1.61074	-0.03107	0.00073
E3	0.07597	0.86465	0.00010
E4	-0.13363	0.87546	-0.00017
W1	0.40872	-0.00741	0.00017
W2	-0.01588	0.75549	-0.00002
W3	0.28338	-0.00437	0.00010
W4	0.32970	-0.00675	0.00016
W5	0.19347	0.55604	0.00025
W6	-0.21730	0.57719	-0.00027
W7	0.29463	0.56106	0.00012
W8	0.31845	-0.57218	0.00014
W9	0.04956	0.42962	-0.00013
W10	0.39238	0.41196	0.00031

W11	0.06743	-0.42030	-0.00011
W12	0.41025	-0.43796	0.00033

Level: 5th, Diaph: 1

Center of Mass (ft): (127.36, 46.10)

LdC	Disp X in	Disp Y in	Theta Z rad
E1	1.35220	-0.02334	0.00055
E2	1.42144	-0.02678	0.00063
E3	0.06729	0.71114	0.00008
E4	-0.11754	0.72032	-0.00014
W1	0.36210	-0.00644	0.00015
W2	-0.01383	0.62710	-0.00002
W3	0.25106	-0.00381	0.00009
W4	0.29209	-0.00585	0.00014
W5	0.17182	0.46127	0.00021
W6	-0.19257	0.47938	-0.00023
W7	0.26121	0.46549	0.00010
W8	0.28195	-0.47516	0.00013
W9	0.04387	0.35668	-0.00011
W10	0.34794	0.34156	0.00026
W11	0.05943	-0.34881	-0.00009
W12	0.36349	-0.36392	0.00028

Level: 4th, Diaph: 1

Center of Mass (ft): (127.36, 46.10)

LdC	Disp X in	Disp Y in	Theta Z rad
E1	1.13594	-0.01959	0.00047
E2	1.19450	-0.02236	0.00053
E3	0.05717	0.55664	0.00007
E4	-0.09916	0.56405	-0.00012
W1	0.30435	-0.00540	0.00013
W2	-0.01150	0.49221	-0.00001
W3	0.21090	-0.00323	0.00008
W4	0.24563	-0.00488	0.00012
W5	0.14582	0.36183	0.00017
W6	-0.16307	0.37648	-0.00019
W7	0.21964	0.36510	0.00009
W8	0.23689	-0.37320	0.00011
W9	0.03588	0.27994	-0.00009
W10	0.29358	0.26771	0.00022
W11	0.04881	-0.27379	-0.00007
W12	0.30652	-0.28602	0.00023

Level: 3rd, Diaph: 1

Center of Mass (ft): (127.36, 46.10)

LdC	Disp X in	Disp Y in	Theta Z rad
E1	0.87899	-0.01516	0.00037
E2	0.92472	-0.01721	0.00042
E3	0.04482	0.39392	0.00005
E4	-0.07725	0.39938	-0.00008
W1	0.23521	-0.00417	0.00010
W2	-0.00883	0.34839	-0.00001
W3	0.16287	-0.00252	0.00006
W4	0.18995	-0.00373	0.00009
W5	0.11401	0.25589	0.00013
W6	-0.12725	0.26668	-0.00014
W7	0.16979	0.25817	0.00007
W8	0.18303	-0.26441	0.00008
W9	0.02671	0.19812	-0.00006
W10	0.22797	0.18912	0.00016
W11	0.03664	-0.19381	-0.00005
W12	0.23790	-0.20281	0.00017

Level: 2nd, Diaph: 1

Center of Mass (ft): (127.37, 46.11)

LdC	Disp X in	Disp Y in	Theta Z rad
E1	0.54707	-0.00927	0.00023
E2	0.57562	-0.01047	0.00026
E3	0.02808	0.22225	0.00003
E4	-0.04814	0.22544	-0.00005
W1	0.14615	-0.00254	0.00006
W2	-0.00543	0.19645	-0.00001
W3	0.10117	-0.00155	0.00004
W4	0.11805	-0.00226	0.00006
W5	0.07122	0.14418	0.00007
W6	-0.07936	0.15049	-0.00008
W7	0.10554	0.14543	0.00004
W8	0.11368	-0.14924	0.00005
W9	0.01635	0.11171	-0.00003
W10	0.14195	0.10644	0.00010
W11	0.02246	-0.10930	-0.00003
W12	0.14806	-0.11456	0.00010