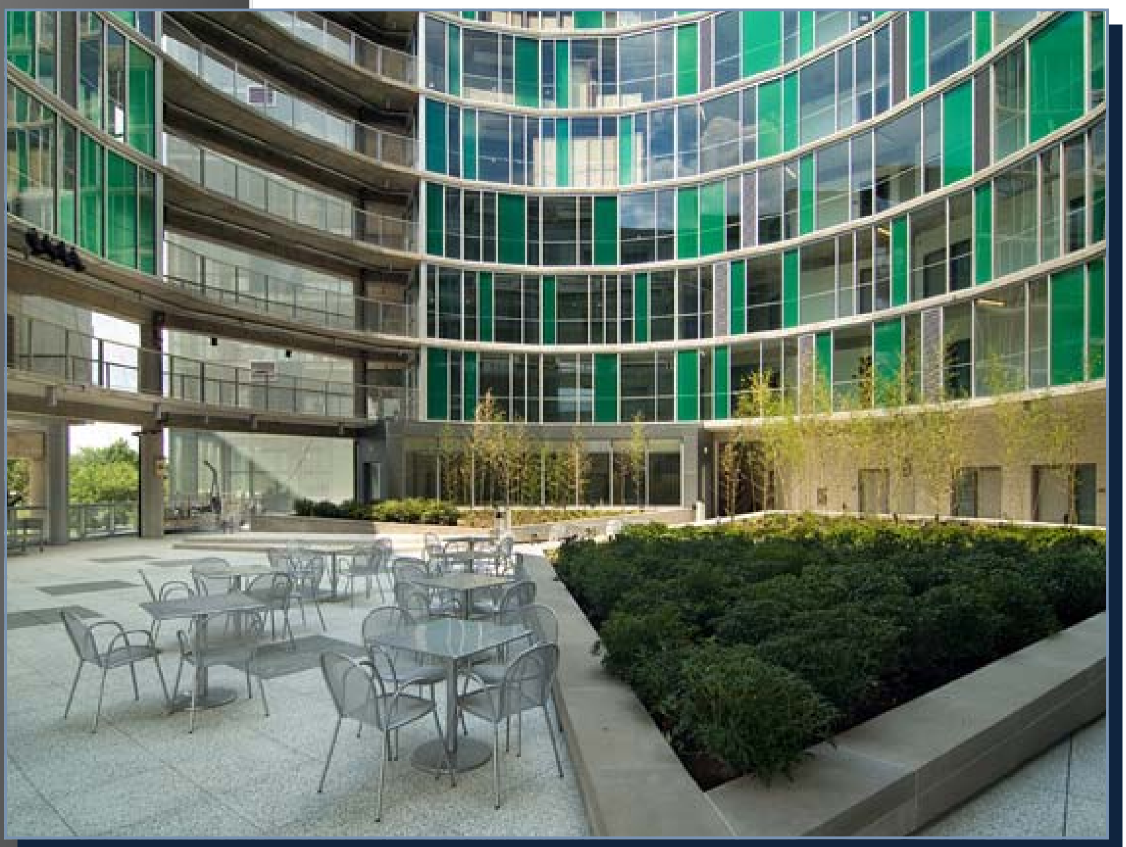
Technical Report III



MICA Gateway Residence

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Structural Option ~ Heather Sustersic ~ November 12, 2012

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Executive Summary:

Technical Report III of the Senior Thesis Project is an analysis of the lateral load bearing system of the MICA Gateway Residence. This report includes information from Technical Report I regarding building codes, materials, and gravity loading. Preliminary analysis of the seismic and wind loading is included as well.

The Gateway lateral system features 8 shear walls arranged around the two elevator and stair cores in the building. Two shear walls are grouped together in the drum portion of the building while the other six are located in the tower portion of the building. Theses eight shear walls take all of the lateral forces from both seismic and wind loads and distribute them to the foundations.

The lateral system was analyzed more accurately in this technical report than in previous reports. An ETABS computer model was constructed to accurately represent the structure. The ETABS model featured an accurate estimate of the building mass, as well as accurate gravity loading conditions. The model was then analyzed under a variety of gravity, wind, and seismic load combinations to find the greatest displacements, drifts, and forces in the lateral system.

Displacement, drift, building torsion, overturning moment, and effects of the lateral system on the foundation design were all considered in this report. Through the ETABS analysis it was determined that the structure was under the acceptable drift limits for both seismic and wind loads. The structure was also deemed adequate to resist the overturning moments of seismic and wind loads. From the overturning moment calculations it was found that wind in the North-South direction is the controlling lateral force on the Gateway.

Spot checks were performed to check that the computer analysis was accurate. Through spot checks it was determined that the drilled caissons of the foundation were adequate, as well as the strength and displacement of one of the shear walls. These calculations, as well as wind, seismic, stiffness spreadsheets, and structural plans are found in the appendices.

Building Introduction:

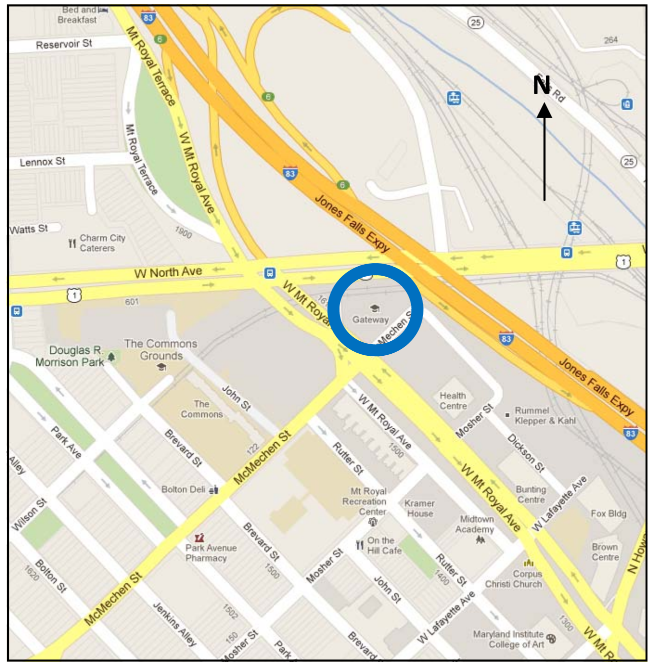


Figure 1: Gateway location in Baltimore. Courtesy of Google

The Gateway residence hall at the Maryland Institute College of Arts was designed to be a cornerstone of their campus in downtown Baltimore, Maryland. Gateway is 122' tall, with 9 stories and a mechanical penthouse and has a useable floor area of 108,000 square feet. The building is located on a constricted site near the intersection of several major roads and Interstate 83. Due to its visibility from all directions, the building has a full 360 degree façade. Gateway is primarily circular in plan with a rectangular tower on the side that faces the highway. The circle, or drum component of the building encloses an open-air courtyard that actually begins on the third floor of the structure. This plaza is located directly above a large "black-box" multipurpose room capable of multiple arrangements to fit a variety of functions. This unique condition was explored in Technical Report I. Beyond the multipurpose assembly room, Gateway features 64 student apartments, several art galleries and studios, and a café.

RTKL Associates Inc. were the architects and engineers on the project, with KCW Engineering Technologies as the civil engineer, and Whiting Turner as the general contractor. The project was delivered with the design-bid-build method for an approximate cost of $30 million. The initial design began in 2005, with construction starting in August 2006 and concluding in August 2008. The building was designed using the Baltimore City Code, which at the time was in accordance with IBC 2000. Due to its various functions, the building has the occupancy types R-2, A-3, and B.

The building structure is primarily concrete, consisting of two-way flat plate slabs, beams, and columns. There are a few steel framed sections of the building, including the entrance vestibule and lobby. Being a prominent building, Gateway has a full 360 degree façade made almost entirely of glass curtain wall panels. The façade has clear, fritted, and frosted glass panels of white, gray, and mint green. Besides the glass curtain wall the superstructure is exposed in a number of places, such as the vertical cuts through the building and the 40' columns holding up a section of the fourth floor. The edge of each concrete floor slab is also exposed.

Design Codes:

MICA Gateway was designed in compliance with the following:

- Baltimore City Code in accordance with IBC 2000
- ASCE 7-05– Minimum Design Loads for Buildings and Other Structures
- ACI 318-05– General Design of Reinforced Concrete
- AISC 13th Edition– Specifications for Structural Steel Buildings
- AWS D1.1– Structural Welding Code– Steel
- ACI 530-05– masonry structures

Building Materials:

MICA Gateway was designed and constructed using the following materials as specified on the General Notes Sheet S001:

- 3500 psi Concrete*– used in spread footings, drilled caissons, and slab on grade
- 4000 psi Concrete*– used in walls, piers, grade beams, columns, slabs, and beams
- ASTM A615, Grade 60– deformed bars
- ASTM A185– welded wire fabric
- ASTM A992– W and WT shapes
- ASTM A36– channels and angles
- ASTM A500, Grade B– rectangular and square HSS, and round HSS
- ASTM A53, Grade B– steel pipe
- ASTM A36 2, Grade 50– steel plates
- ASTM A325 or A490– high strength bolts
- ASTM F1554, Grade 36– anchor bolts
- ASTM A307– standard fasteners
- ASTM A653, Quality SS, Grade 33– metal roof deck
- ASTM C476– grout
- ASTM C270, Type S– mortar
- 1500 psi Masonry– used in masonry walls

*Normal weight concrete shall have a maximum dry unit weight of 150 pcf

Gravity Loads:

Dead Loads:

In the General Notes (S001) the designers provided a loading schedule of superimposed dead loads which varied by location. That schedule lists each component of the dead load separately, but the following table lists only the total superimposed dead load for each building space. Concrete slab, column, beam, etc. self weights are not included in this table.

Area	Dead Load (psf)
Residences	9
Circulation Ring	10
Storage Rooms	9
Roof	13
Level 3 Planters	258*
Planters on Multi Use Room Space Roof	283†
Level 3 Plaza	38‡
Mechanical Rooms	9
Multi Use Room Space Roof	67§
Offices	9
Gallery Roof	17
Level 2 Balcony	37

* Takes into account a 240 psf saturated soil load. Only applies to structure supporting planters that are not above the multi-use performance space.

† Takes into account a 240 psf saturated soil load and the multi-use performance space roof ceiling components (steel grid, lighting, etc.). Only applies to structure supporting planters above the multi-use performance space.

‡ Takes into account pavers of the plaza not above the multi-use performance space.

§ Takes into account pavers of the plaza above the multi-use performance space.

Gravity Loads:

Live Loads:

The Generals Notes also provided a table of live load values for the various areas of the building. Partitions are included in the live load for the residence and office areas. Oddly no live load was given for the floor of the multi-use performance room space on the loading schedule. Therefore a 100 psf live load for dance halls and ballrooms will be assumed, as per IBC 2006.

Area	Dead Load (psf)
Residences	60
Circulation Ring	100*
Storage Rooms	125*
Roof	30*
Level 3 Planters	240
Planters on Multi Use Room Space Roof	40
Level 3 Plaza	100*
Mechanical Rooms	150*
Multi Use Room Space Roof	100*
Offices	70
Gallery Roof	30*
Level 2 Balcony	100*
Multi-Use Performance Space	100 (per IBC 2006)

* Indicates that live load reduction was not allowed.

Snow Load:

Based on ASCE 7-05, which assumes a ground snow load of 25 psf, the roof snow load was calculated at 20 psf. This was checked against ASCE 7-10 and no change in snow load requirements between the two codes was noted.

Structural Overview:

The Mica Gateway Residence is a predominately concrete structure with some steel members in certain places. Due to the unique circular shape of the building, the designers developed a radial grid with columns located by their X and Y coordinates in the four quadrants of the Cartesian coordinate system. The zero-zero point of the grid is located in the exact center of the courtyard. Thus a column located in the lower left of the plan will have a negative X and Y coordinate while a column in the upper right will have a positive X and Y coordinate. This was done to avoid an unreasonable amount of column lines clustered together at odd intervals.

Foundation:

The geotechnical report was prepared by D.W. Kozera, Inc. They submitted the geotechnical report on February 23, 2005. In their report they found that the site had very dense soil and soft rock, earning a site soil classification of C.

The foundation of the MICA Gateway features drilled caissons that bear directly on bedrock and have a safe bearing capacity of 100 ksf. All columns that start at ground level start at the top of a drilled caisson. Caissons are also located directly under the walls that support the load from the long span beams over the "black box" theater. All caissons are between 3'-0" and 4'-6" in diameter

Where exterior walls meet the foundation, strip footings are incorporated and are a minimum of 30" below the finished grade. For the steel framed entrance vestibule and lobby, steel columns are supported by spread footings with a minimum safe bearing capacity of 1.5 ksf.

Gravity System:

The gravity load system for the Gateway features numerous two-way flat plate slabs as well as several one-way slabs and two-way slabs with drop panels. Below Level 4, there are several one way slabs of 7" thickness that span the areas below the courtyard. They work in conjunction with concrete beams that span very irregular areas. On Level 3, the courtyard spans over the "black-box" theater, to give a column free space for intended use. As such, 48"x48" beams were designed to span over the almost 60' of the theater and accommodate the large dead and live loads from the plaza and planters in the courtyard above. These beams have (16)#10 bottom reinforcing bars to resist the large moments produced by the load.

On Level 4 there is an area featuring one-way slabs and beams. This area is supported by large exterior columns that rise nearly 40' from grade to the bottom of the slab. Here a transfer beam runs between columns so as to support new columns that rise to support the upper floors. Beams are also used extensively to support the exterior walkways that connect the various parts of the drum.

The rest of Level 4 and all floors above have 8" two-way flat plate slabs between radial column lines as shown in Figure 2 to the right. The dotted lines represent the boundaries between the column and middle strips.

Other unique floor framing conditions include a section of the slab on each floor that frames into a column with a drop panel. This area is located in the northeast quadrant of the plans centered around column 7, as seen in Figure 3 below. The only uses of steel framing in this building are over the entrance and lobby, using mainly W10x15, W10x12, and HSS8x3x3/16.

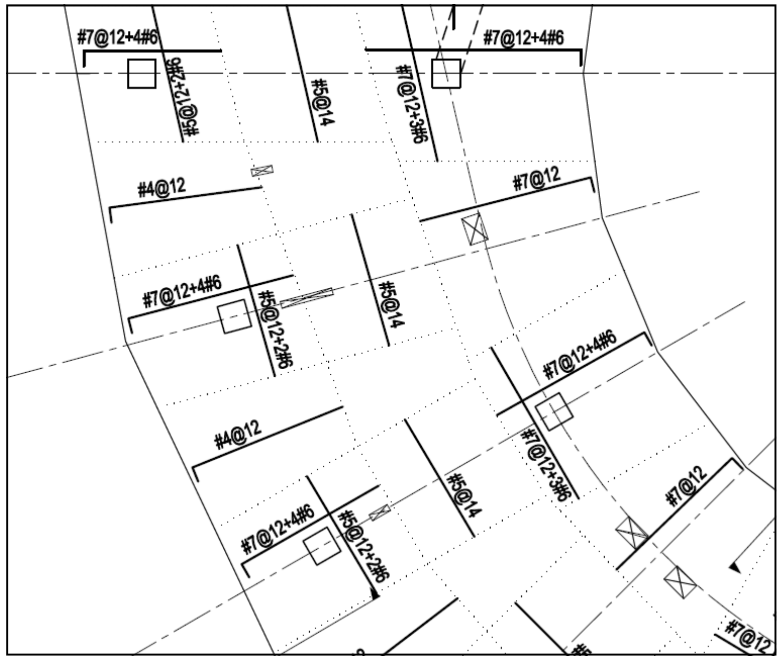


Figure 2: Typical two-way flat plate slab. Courtesy of RTKL

The slabs and beams of the Gateway are all supported by concrete columns that form two concentric circular lines around the drum of the building. In most interior areas and on the upper floors these columns are rectangular, with sizes ranging from 12x12 to 24x24. In other places where the columns are on the exterior of the building, such as the 40' slender columns that support Level 4, the columns are circular with sizes ranging from 24" diameter to 36" diameter.

The roof system of the Gateway is no different from a normal floor. One-way slabs frame into beams that transfer load to the columns. The main difference is the smaller slab thicknesses, between 6"-7" due to the smaller loads on the roof areas.

Lateral Systems:

The lateral system of the Gateway features two concrete shear wall groups located near the stair and elevator cores, one in the tower and the other in the drum. Due to the low seismic risk of the region, it was assumed that the lateral system was primarily ordinary concrete shear walls. Each of the eight shear walls extend from the ground to the highest point in their respective part of the building; 122' in the tower and 103' in the drum. The walls are all 12" thick and from 9' to 24' long. The shear walls are highlighted in Figure 7 below.

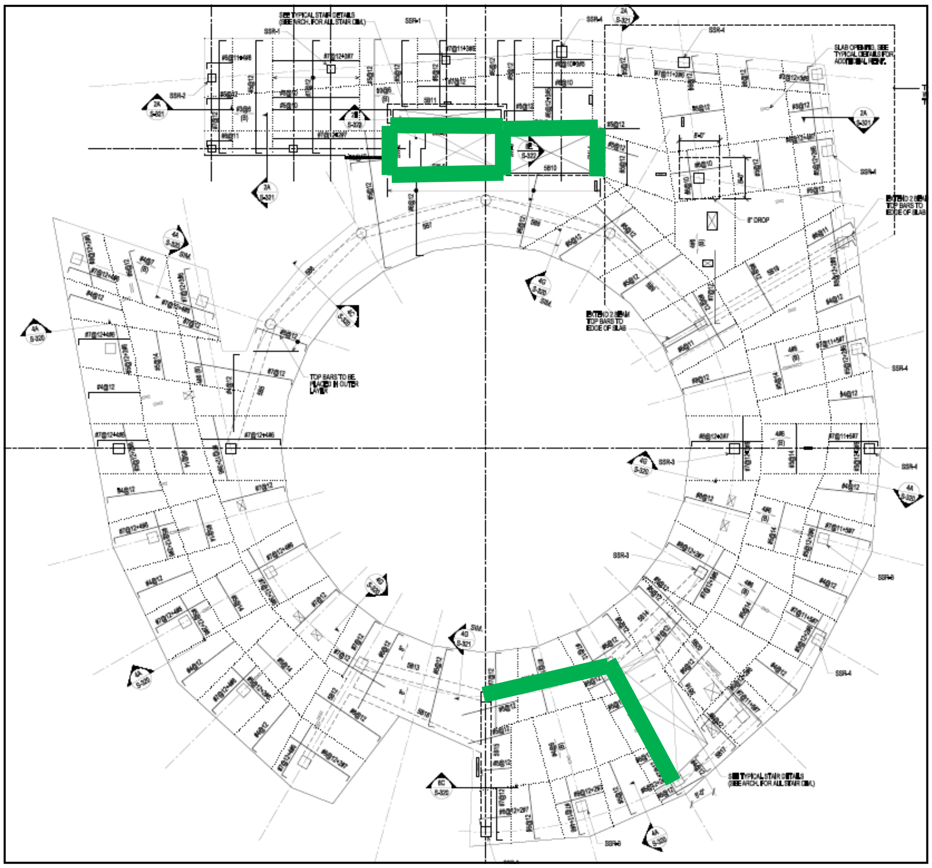


Figure 3: Shear wall locations. Courtesy of RTKL.

The lateral load path is as follows: wind pressure bears on the building cladding, which is supported by the edge slab. From here the slab transfers the load into shear walls either directly or through beams. The shear walls then direct the load into the foundation. Shear walls prevent unwanted torsion and large displacements of the building from occurring in the event of an earthquake or a severe storm with high winds.

Wind Design Loads:

The wind analysis of the Gateway building was originally computed using ASCE 7-05. This report uses ASCE 7-10 to determine wind design pressures on the building facades. Appendix A includes the hand calculations associated with the wind analysis. Appendix B contains the Excel spreadsheets used to determine the wind loads, story forces, and overturning moment.

Due to the unique shape and presence of numerous different surface planes, a number of assumptions and approximations were done to analyze the wind load on the Gateway. The building geometry was simplified to a 160' by 160' square with the analyzed faces being the projected area in elevation. Wind pressures were considered for each of the four "sides" of the building due to their unique profiles and cutouts. The various cuts that extend from the façade to the interior courtyard were subtracted from the tributary area to reach more accurate story forces. Due to the variety of opening that penetrate into the central part of the building, the Gateway is assumed to be partially enclosed. Other effects such as uplift underneath the overhanging floors and the wind effects in the inner courtyard were ignored for simplicity. The building height was simplified to 113' for three sides, while the fourth side was considered to be 103' tall because the tower portion of the building was on the leeward side.

Other assumptions included; Risk category III due to the large assembly space and an internal pressure coefficient reduction factor which is applicable to a partially enclosed building that contains a single partitioned large volume; in this case the courtyard. One unique difference between ASCE 7-05 and ASCE 7-10 was an increase in the Basic Wind Speeds for all building risk categories. In the original design, a basic wind speed of 90 mph was assumed, while this report assumed a basic wind speed of 120 mph in accordance with ASCE 7-10.

The following are wind load diagrams associated with the four building sides.

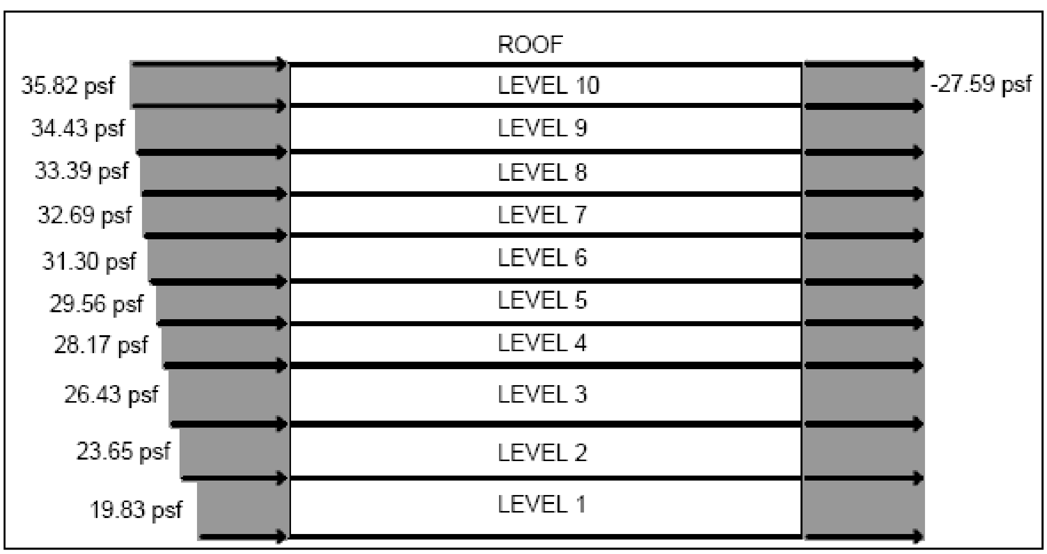


Figure 4: North-South Wind Design Pressure

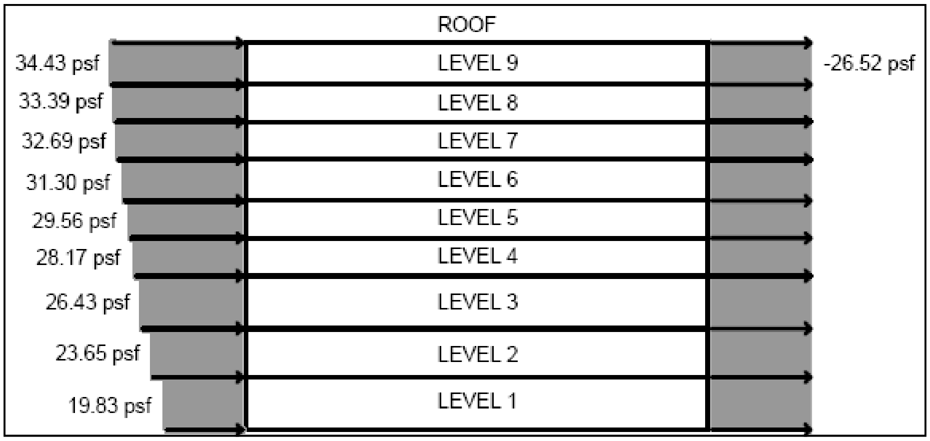


Figure 5: South-North Wind Design Pressure

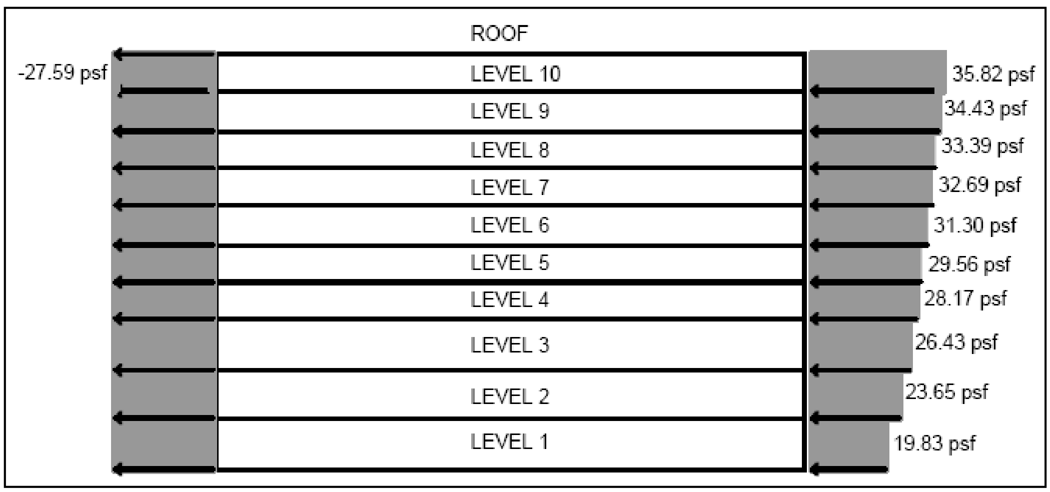


Figure 6: East-West Wind Design Pressure

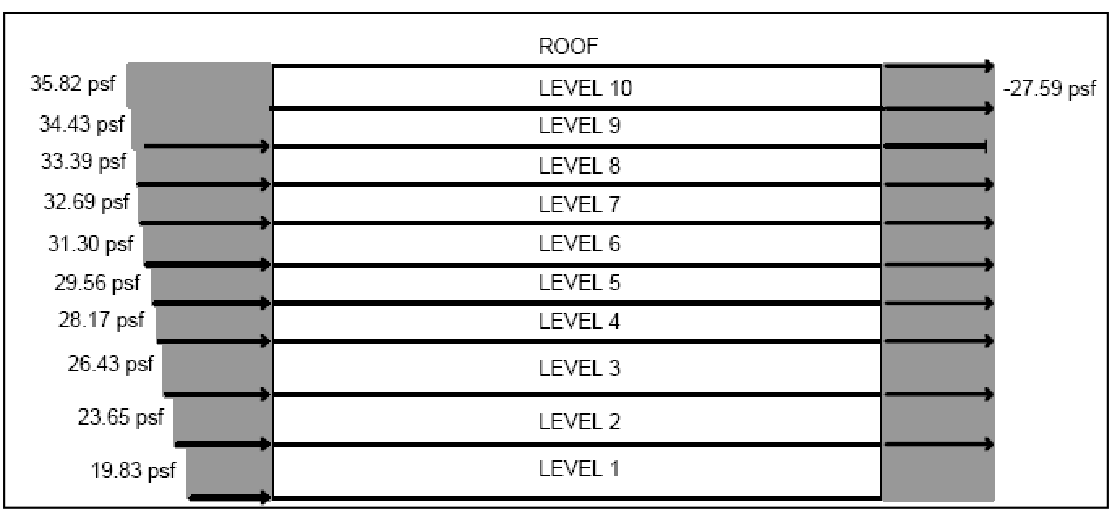


Figure 7: West-East Wind Design Pressure

Seismic Design Load:

For seismic analysis, ASCE 7-10 Chapters 11 and 12 were followed. Based on the geotechnical report a site class of C was used in the analysis. Using the United States Geological Survey website, which determines spectral response acceleration parameters based on site location and class, a S_{ds} of 0.104g and a S_{d1} of 0.059g were found. Using Tables 11.6-1 and 11.6-2 of ASCE 7-10, a Seismic Design Category of A was determined. This is contrary to the actual design of the building, which considered SDC B. This discrepancy could be due to the difference in code used at the time of design. Therefore SDC B will be assumed for the seismic load calculations.

The building was assumed to have ordinary concrete shear walls as its primary lateral resisting system, warranting a Response Modification Factor of 4. Further calculations are detailed in Appendix A.

In determining the seismic base shear and overturning moment, the weight of each story was approximated as 150 pcf of concrete multiplied by 8" and the entire floor area of that story. An additional 50 percent was added onto that weight to approximate the weight of the concrete beams, column, etc. This data was then entered into an Excel spreadsheet that can be found in Appendix B. The below figure summarizes the results of the seismic analysis.

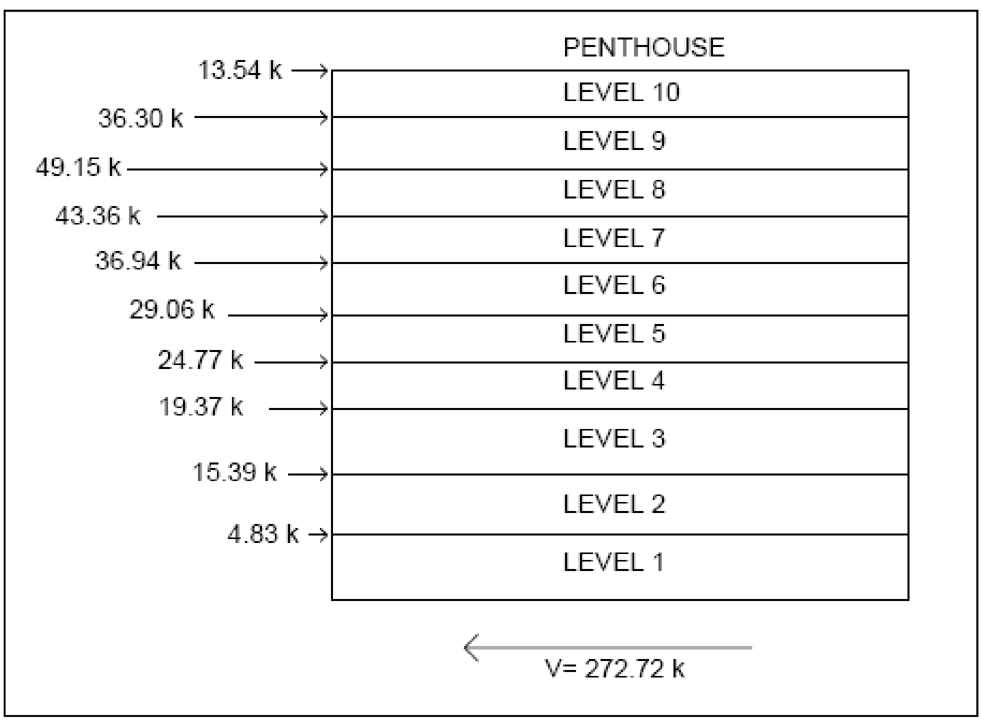


Figure 8: Seismic Story Force and Base Shear

ETABS Model:

The Gateway residence was modeled with ETABS to analyze the building's lateral system. All structural members of the building were included in the model to accurately determine the correct mass of the structure. The floor slabs were modeled as a rigid diaphragm. All shear walls and floor slabs were meshed with a maximum size of 48." All members also feature a cracked section property to more realistically model the structure. Beams have a moment of inertia modifier of 35%, columns have a moment of inertia modifier of 70%, the shear walls have bending modifiers of 70%, and the slabs have bending modifiers of 25%. Below are Figures 9 and 10 which show a typical floor frame in ETABS and a three dimensional projection of the model.

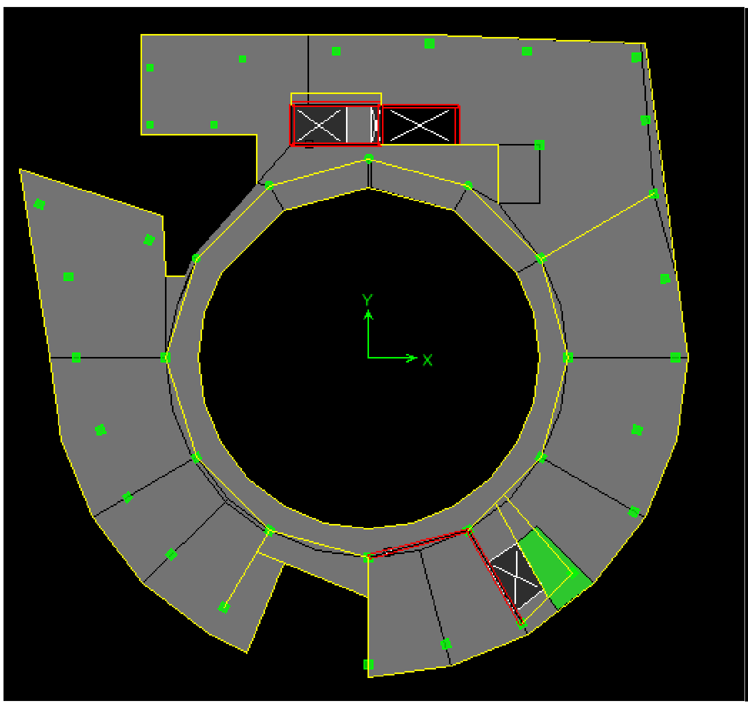


Figure 9: ETABS model floor plan

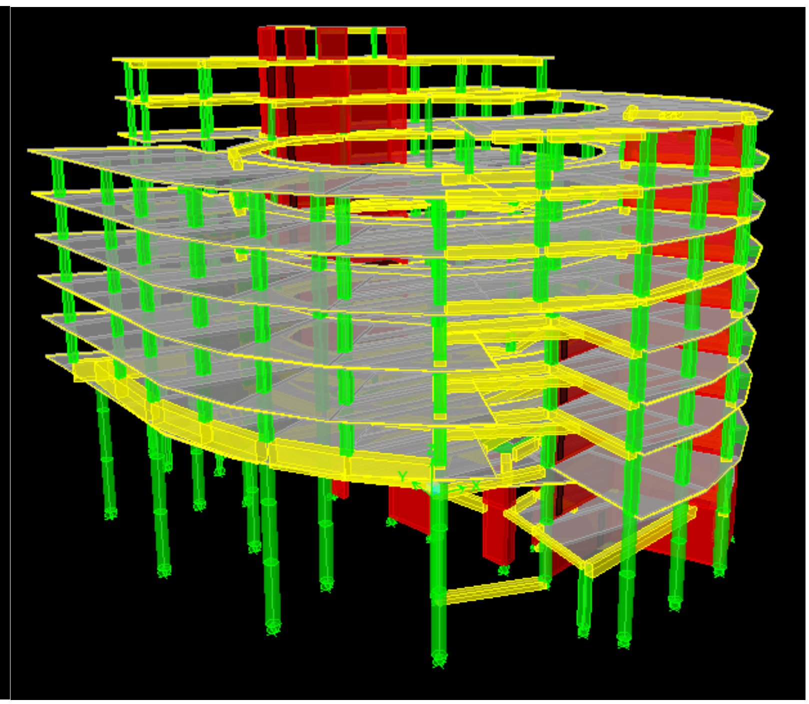


Figure 10: ETABS model perspective

Load Combinations:

ASCE 7-05 provides a variety of load combinations that could potentially control the lateral system design. Loads considered in the ETABS model for this report included dead, live, snow, wind, and earthquake. The load combinations from ASCE 7-05 are:

1.4D

1.2D+1.6L+0.5S

1.2D+1.6L+05.W

1.2D+1.0W+1.0L+0.5S

1.2D+1.0E+1.0L

0.9D+1.0W

0.9D+1.0E

Each of the above load combinations are applied in multiple directions where applicable. For example the wind loads can be considered from a variety of directions both singularly and simultaneously. Wind load cases are defined by ASCE 7-05, Chapter 6 under Method 2. Earthquake loads were also considered in multiple directions both with and without accidental torsion, a topic that will be covered later in this report.

Lateral Load Distribution:

Lateral loads are distributed by several shear walls located around the elevator and stair cores within the Gateway structure. There are eight shear walls in total that distribute the lateral loads down to the foundations. The shear walls are highlighted in Figure 11 below and assigned a number based on the actual building design documents. All shear walls except for Shear Wall 8 extend the full height of the building. Shear Wall 8 ends at Level 6.

Each shear wall takes a portion of the lateral load based on its relative stiffness at each story height. The stiffer walls take a larger proportion of the story force based on the amount of deflection caused by that load. The stiffness of each shear wall at each level is determined in spreadsheets found in Appendix B.

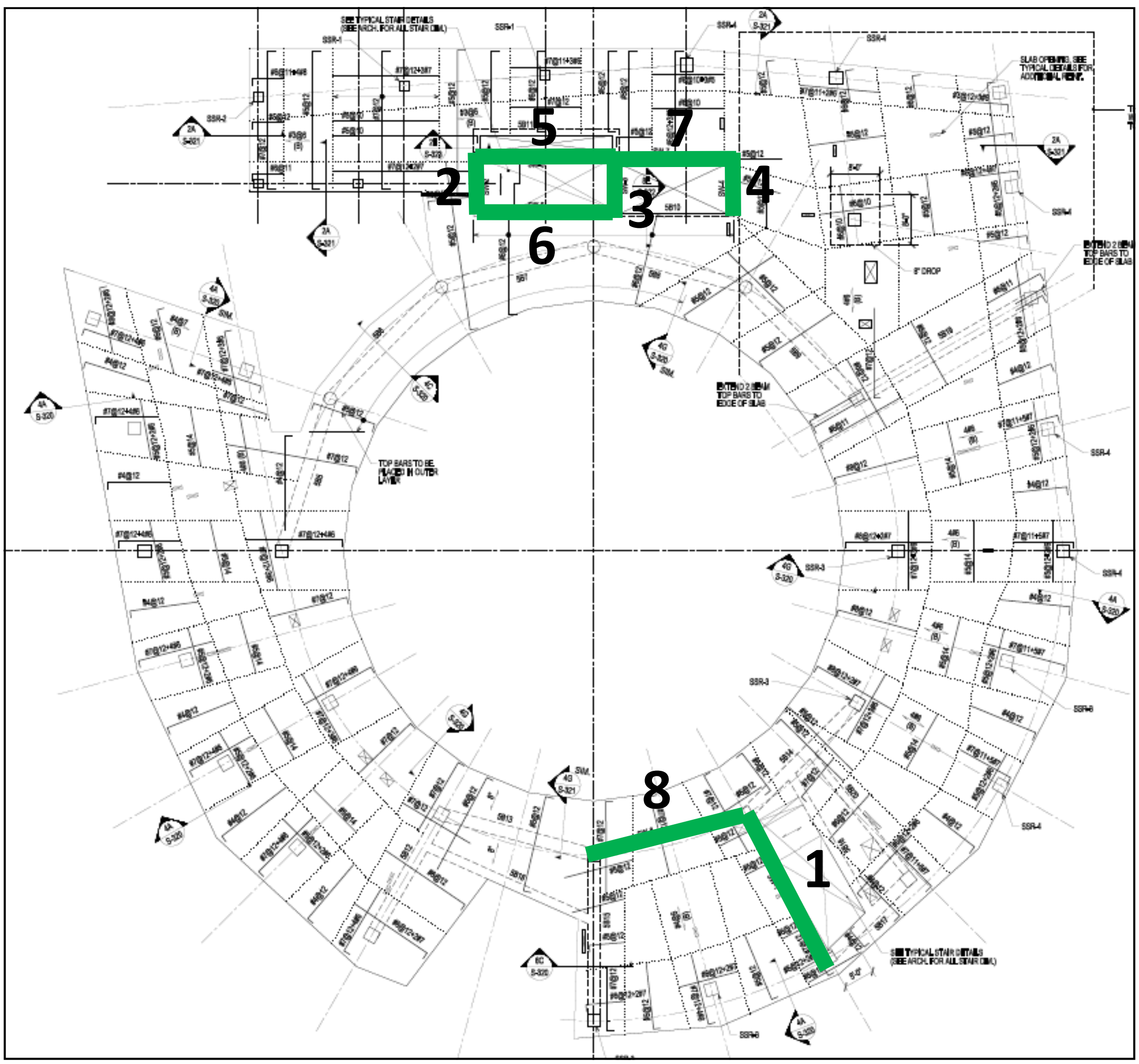


Figure 11: Shear wall locations. Courtesy of RTKL.

Drift and Displacement:

The governing load cases that caused the most drift and displacement are considered controlling and are listed in the following Figures. The ETABS analysis data shows the highest lateral drift for each story. Both the controlling seismic load combination and controlling wind load combination are considered separately. The story drifts calculated by ETABS are also checked against the allowable code limits for seismic based on ASCE 7-05 Table 12.12-1. For a risk category III, the allowable story drift is 0.015h, where h is the story height. The displacements for seismic loads fall below the allowable code amount based on the ETABS analysis. The standard practice for story drift for wind loads is H/400.

Wind Drift and Displacement					
Story	X Displacement (in)	Y Displacement (in)	X Story Drift (in)	Y Story Drift (in)	Allowable Drift (in)
Roof	0.123	0.0732	0.0001	0.0003	3.39
10	0.164	0.858	0.0007	0.0004	3.39
9	0.158	0.046	0.0005	0.0003	3.39
8	0.132	0.039	0.0005	0.0003	3.39
7	0.106	0.038	0.0004	0.0002	3.39
6	0.084	0.03	0.0002	0.0001	3.39
5	0.071	0.021	0.0002	0.0001	3.39
4	0.059	0.021	0.0011	0.0013	3.39
3	0.048	0.013	0.0022	0.0007	3.39
2	0.045	0.001	0.0018	0.0011	3.39

Seismic Drift and Displacement					
Story	X Displacement (in)	Y Displacement (in)	X Story Drift (in)	Y Story Drift (in)	Allowable Drift (in)
Roof	1.211	0.542	0.0007	0.0007	1.8
10	1.229	0.536	0.0021	0.0011	1.98
9	1.106	0.445	0.0023	0.0013	1.8
8	0.954	0.388	0.0025	0.0015	1.8
7	0.788	0.326	0.0022	0.0013	1.98
6	0.649	0.27	0.0007	0.0003	1.8
5	0.572	0.239	0.0006	0.0003	1.8
4	0.494	0.244	0.0016	0.0019	2.52
3	0.43	0.142	0.0026	0.0015	2.34
2	0.404	0.049	0.0036	0.0025	2.52

Building Torsion:

Torsional forces occur within the Gateway structure due to the different locations of the building's center of mass and center of rigidity. The center of mass and center of rigidity vary on every floor due to the presence of shear walls, beams, and other structural elements. There are two types of torsion that occur on the building structure. The first is the torsion caused by the eccentricity between the center of mass and the center of rigidity. The other form of torsion is called accidental torsion, which is caused by the eccentric application of the seismic loads on the structure. This eccentricity is applied at 5% of the building length in both the N-S and E-W directions. The center of mass and center of rigidity were calculated by ETABS to provide the most accurate locations. Below are the tables where the torsional forces are calculated.

N-S Torsional Seismic Loading

Story	Story Force	Location of CM	Location of CR	e (ft)	M_t (ft-k)	M_a (ft-k)	M_{total} (ft-k)
Roof	14.44	66.65	38.02	28.63	413.4172	115.4478	**528.865**
10	32.46	15.32	30.05	14.73	478.1358	259.5177	**737.6535**
9	37.29	5.27	23.67	18.4	686.136	298.1336	**984.26955**
8	32.77	5.49	15.18	9.69	317.5413	261.9962	**579.53745**
7	28.36	4.75	4.03	0.72	20.4192	226.7382	**247.1574**
6	23.6	6.75	-6.27	13.02	307.272	188.682	**495.954**
5	19.22	6.56	-12.77	19.33	371.5226	153.6639	**525.1865**
4	17.82	4.33	-19.53	23.86	425.1852	142.4709	**567.6561**
3	20.11	8.52	-27.79	36.31	730.1941	160.7795	**890.97355**
2	5.56	17.11	-33.24	50.35	279.946	44.4522	**324.3982**
						Total=	**5881.6513**

E-W Torsional Seismic Loading

Story	Story Force	Location of CM	Location of CR	e (ft)	M_t (ft-k)	M_a (ft-k)	M_{total} (ft-k)
Roof	14.44	4.28	4.6	0.32	4.6208	106.7206	**111.34143**
10	32.46	23.47	3.88	19.59	635.8914	239.8997	**875.79109**
9	37.29	3.21	0.156	3.054	113.8837	275.5964	**389.48007**
8	32.77	2.38	5.98	3.6	117.972	242.1908	**360.16278**
7	28.36	2.57	15.17	12.6	357.336	209.5981	**566.93413**
6	23.6	2.69	26.05	23.36	551.296	174.4188	**725.71475**
5	19.22	2.67	36.56	33.89	651.3658	142.0478	**793.41361**
4	17.82	-6.1	47.54	53.64	955.8648	131.7009	**1087.5657**
3	20.11	11.83	59.46	47.63	957.8393	148.6255	**1106.4648**
2	5.56	28.85	65.77	36.92	205.2752	41.09188	**246.36708**
						Total=	**6263.2354**

Overturning Moment:

Overturning moments due to lateral forces are important when considering foundation design. The overturning moment was calculated for both seismic and North-South and East-West wind directions. The story forces were determined from the ETABS analysis data and then multiplied by the height of each story. The moments were then summed to determine a total overturning moment. According to the ETABS data, the controlling lateral force was the North-South wind force, being slightly larger than the seismic overturning moment. Below is the summary table of information regarding overturning moment.

The Gateway's columns rest on top of drilled caissons that have straight shafts that bear on bedrock, with a required minimum safe bearing capacity of 100 ksf. In the ETABS model, the column to caisson connections are modeled as pinned connections. A spot check was performed on a caisson located below Shear Wall 2 to determine the adequacy of the foundation under gravity and lateral loads. The calculations are detailed in Appendix A.

Overturning Moment							
		Seismic		N-S Wind		E-W Wind	
Story	Height	Story Force	**Moment**	Story Force	**Moment**	Story Force	**Moment**
Roof	113	14.44	**1631.72**	7.9	**892.7**	3.95	**446.35**
10	103	32.46	**3343.38**	31.21	**3214.63**	15.61	**1607.83**
9	92	37.29	**3430.68**	30.66	**2820.72**	15.32	**1409.44**
8	82	32.77	**2687.14**	30.04	**2463.28**	15.02	**1231.64**
7	72	28.36	**2041.92**	29.37	**2114.64**	14.69	**1057.68**
6	61	23.6	**1439.6**	28.63	**1746.43**	14.31	**872.91**
5	51	19.22	**980.22**	27.79	**1417.29**	13.9	**708.9**
4	41	17.82	**730.62**	30.59	**1254.19**	15.3	**627.3**
3	27	20.11	**542.97**	32.56	**879.12**	16.27	**439.29**
2	14	5.56	**77.84**	28.77	**402.78**	14.39	**201.46**
		Total=	**16906.09**	**Total=**	**17205.78**	**Total=**	**8602.8**

Shear Wall Spot Check:

To determine the structural stability and strength of the lateral force resisting system, a spot check was performed for Shear Wall 2. The loads applied in the spot check were taken from the ETABS analysis as shown below in Figure 12. Figure 13 is a elevation view of Shear Wall 2. Based on the calculations detailed in Appendix A, the shear wall was found to be adequate to support the applied loads.

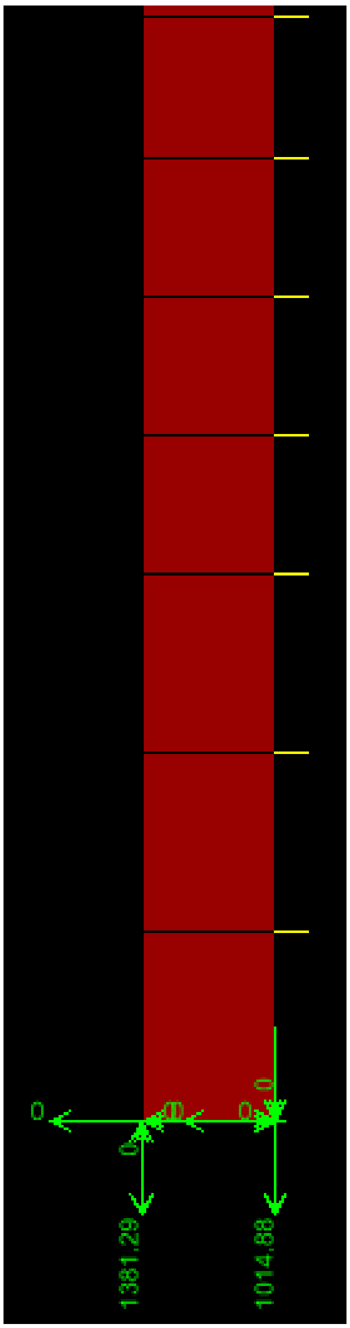


Figure 12: Shear Wall 2 reactions from ETABS model

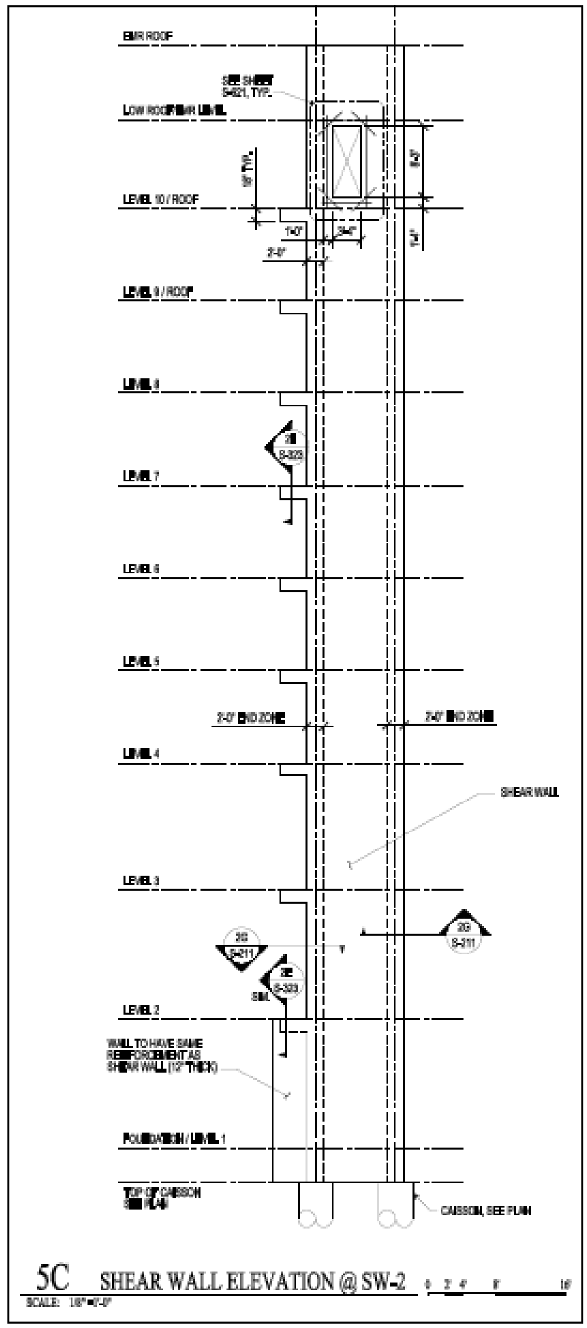


Figure 13: Shear Wall 2 elevation. Courtesy of RTKL.

Conclusion:

Based on the data obtained from the ETABS model and the spot checks performed it was determined that the Gateway structure is adequate to resist lateral loads from both seismic and wind forces. The ETABS model was essential to analyzing the Gateway's lateral system, in particular determining the location of the center of rigidity and center of mass, as well as the shear forces in the shear walls. From the analysis results the story displacement and drift, torsion, and stiffness were found. The story drift for each level was well under the allowable code maximum. Based on the results it was found that the North-South wind force creates the largest overturning moment on the structure.

The analysis performed in Technical Report III varied from the analysis performed in Technical Report I. The ETABS analysis provided a more complete understanding of how the Gateway lateral system distributes lateral loads. The ETABS model also featured more accurate loading conditions and a more accurate estimate of the buildings mass than Technical Report I.

The results of this technical report will be essential for future work. The proposal for the Gateway structure will likely require a re-design of the lateral system. As such, the understanding gained of the existing lateral system, as well as the construction of the ETABS model will be invaluable.

Appendicies:

Appendix A: Hand Calculations

11 | Scott Molongoski | Tech One | Wind Load Calcs

Wind Load Calculations

- Based on ASCE 7-10
  - Risk Category III (Table 1.5-1)
  - Basic Wind Speed, $V = 120$ mph (Fig 26.5B)
  - Directionality Factor, $K_d = 0.85$ (Table 26.6-1)
  - Exposure Category: B (Sect. 26.7)
  - Topographic Factor, $K_{zt} = 1.0$ (Sect. 26.8)
  - Gust Effect Factor, $G = 0.85$ (Sect. 26.9)
  - Enclosure Classification: Partially Enclosed (Sect. 26.10)
    - → Based on the numerous openings between the exterior of the drum and the interior courtyard
  - Internal Pressure Coefficient; $GC_{pi} = \pm 0.43$ (Sect. 26.11)
    - → Reduction Factor:

$$R_i = 0.5\left[1 + \frac{1}{\sqrt{1 + \frac{V_i}{22.8 A_{og}}}}\right] \leq 1.0$$

→ Applicable for partially enclosed bldgs containing a single unpartitioned large volume; in this case the courtyard.

V_i = Volume of space = 326755 ft³

A_{og} = total area of openings = 7462 ft³ (includes roof & wall slots)

$$R_i = 0.5\left[1 + \frac{1}{\sqrt{1 + \frac{326755}{22.8 \cdot 7462}}}\right] = 0.79 \leq 1.0$$

$$\rightarrow GC_{pi} = \pm 0.55 \cdot 0.79 = \pm 0.43$$

(0.55 from Table 26.11-1)

- Refer to Excel spreadsheets for
  - → Velocity pressure exposure coefficients, K_z
  - → Velocity pressure, q_z
  - → Wind pressure, p
- Wind pressures were analyzed on 4 faces of the building, due to their unique heights and openings.

See Excel spreadsheets for further calculations.

Appendix A: Hand Calculations

Scott Molongoski | Tech Three | Seismic Calcs

Seismic Load Calculations:

- Based on ASCE 7-10
- From USGS website U.S. Seismic Design Maps: $S_s = 0.130g$, $S_1 = 0.052g$, $S_{MS} = 0.15g$, $S_{M1} = 0.088g$, $S_{DS} = 0.104g$, $S_{D1} = 0.059g$
- Based on this data and Tables 11.6-1 and 11.6-2, the building falls into Seismic Design Category A, contrary to the building plans which state Seismic Design Category B. SDC B will be used for the remainder of the calculations.
- Assume building has ordinary reinforced concrete shear walls:
  - Response Modification Coefficient, $R = 4$, per Table 12.2-1
  - Importance Factor $= 1.25$, per Table 1.5-2
  - No height limitations per Table 12.2-1
- $T_L = 6s$ per Fig. 22-12
- $C_u = 1.7$ per Table 12.8-1
- $C_t = 0.02$ per Table 12.8-2
- $x = 0.75$ per Table 12.8-2
- $h = 113$ ft

$$T_a = C_t h_n^x = 0.02(113)^{0.75} = 0.69s$$

$$T = C_u \cdot T_a = 1.7 \cdot 0.69 = 1.18s$$

$$C_s \le \begin{cases} S_{DS}/(R/I) = 0.104/(4/1.25) = 0.021 \\ S_{D1}/[T(R/I)] = 0.059/[1.18(4/1.25)] = 0.016 \end{cases}$$

$$k = 1 + (1.18-1)\left(\frac{2-1}{2.5-0.5}\right) = 1.09$$

For each story, the weight of the 150 pcf 8" concrete slab was taken for the entire floor area (found on Sheet C101) and then an additional 50% of that weight was added on to get an assumed value for the weight of the slabs, beams, columns, etc.

Appendix A: Hand Calculations

Scott Molongoski | Tech Three | Foundation Check

Foundation Check:

From ETABs Model, the reaction under Shear Wall 2 is 1381 k. This is the axial load taken by the caisson.

Check Load Bearing Capacity of caisson

$$q_u = A_p(c'N_c F_{cs} F_{cd} F_{cc} + q N_q F_{qs} F_{qd} F_{qc})$$

$$A_p = \pi/4\, D_b^2 = \pi/4 (3.5)^2 = 9.6\ ft^2$$

$\phi' = 27°$ $\quad c' = 20$

$N_c = 23.94$

$N_q = 13.20$

$$F_{cs} = 1 + (B/L)\left(\frac{N_q}{N_c}\right) = 1 + (1)\left(\frac{13.2}{23.94}\right) = 1.55$$

$$F_{qs} = 1 + (B/L)(\tan\phi') = 1 + (1)\tan(27) = 1.51$$

$$F_{qd} = 1 + 2\tan\phi'(1-\sin\phi')^2 \tan^{-1}\left(\frac{D_f}{B}\right) = 1 + 2\tan(27)(1-\sin(27))^2 \tan^{-1}(50/3.5)$$

$$F_{qd} = 2.8$$

$$F_{cd} = F_{qd} - \frac{1-F_{qd}}{N_c \tan\phi'} = 2.8 - \frac{1-2.8}{23.94 \cdot \tan(27)} = 2.95$$

$F_{cc} = 1$

$F_{qc} = 1$

$$q_u = 9.6(20(23.94)(1.55)(2.95)(1) + (100)(13.2)(1.51)(2.8)(1))$$

$$q_u = 74600\ k >> 1381\ k \checkmark$$

Caisson is more than adequate to support loads from the shear wall.

Appendix A: Hand Calculations

Scott Molongoski | Tech Three | Shear Wall Check

Shear Wall Strength:

check Shear Wall 2:

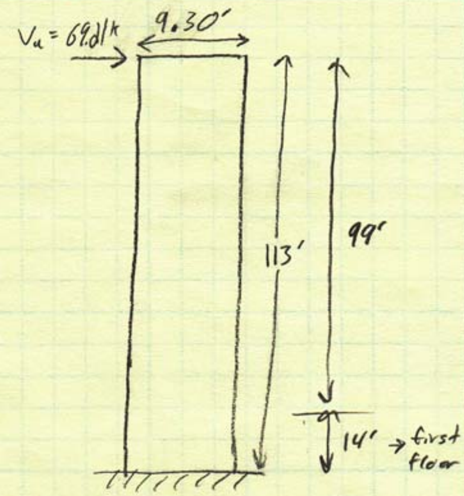
$\ell = 113'$

$h = 12''$

$V_u = 69.21^k$ (from ETABS)

$f'_c = 4000$ psi

$f_y = 60000$ psi



Max Strength:

$\phi V_n = \phi 10\sqrt{f'_c} \cdot h \cdot d$

$\phi V_n = 0.75(10)\sqrt{4000}(12'')(89'')/1000$

$\phi V_n = 507\,k > V_u = 69.21^k$ ✓

Shear Strength provided by Concrete:

$V_c = 2\sqrt{f'_c}\, h \cdot d = 2\sqrt{4000}(12'')(89'')/1000 = 135\,k$

$V_c \le 3.3\sqrt{f'_c}\, h \cdot d + N_u \cdot d/\ell_w$ (→ 0) No axial load

$3.3\sqrt{4000}(12'')(89'') = 222.9k$

$V_c \le \left[0.6\sqrt{f'_c} + \left[\ell_w(1.25\sqrt{f'_c} + 0.2 N_u/\ell_w h)\right] / \left(M_u/V_u - \ell_w/2\right)\right](h \cdot d)$

$M_u = 99' \cdot 69.21k \cdot 12 = 82222$ in-k

$V_c \le \left[0.6\sqrt{4000} + \left[9.3 \cdot 12(1.25\sqrt{4000})\right] / \left(82222/69.21 - 9.3 \cdot 12/2\right)\right](12'')(89'')$

$V_c \le 48.9\,k$

Deflection Check:

Δ_{shear} & $\Delta_{flexure}$

$\Delta_v = Ph^3/3E_m \cdot I + 1.2Ph/E_v A$

$I = \frac{td^3}{12} = \frac{(12)(89)^3}{12} = 705000$

$= (69.21)(113 \cdot 12)^3/3(57000\sqrt{4000})(705000)$
$+ (1.2 \cdot 69.21)(113 \cdot 12)/0.4 \cdot (57000\sqrt{4000})(14'' \cdot 89'')$

$1.73 \cdot 10''$

$\Delta_v = 0.23''$

0.005

$\Delta_f = Ph^3/12 E_m I + 1.2Ph/E_v A$

$= (69.21)(113 \cdot 12)^3/12(57000\sqrt{4000})(705000) + 1.2(69.21)(113 \cdot 12)/0.4(57000\sqrt{4000})(14 \cdot 89)$

$\Delta_f = 0.005''$

$\Delta_T = 0.23 + 0.005 = \boxed{0.235''} < \ell/400 = 113 \cdot 12/400 = 3.39''$ ✓

Appendix B: Wind, Seismic, and Stiffness Tables

Wind Tables:

Table B-1:

North-South MWFRS									
Level	Elevation	z	K_z	q_z	q_h	Windward P (psf)	Leeward P (psf)	Tributary Area (ft^2)	Story Force (kip)
1	112	0	0.57	17.86	32.27	19.83	-27.59	0.00	0.00
2	126	14	0.57	17.86	32.27	19.83	-27.59	1660.50	32.92
3	139	27	0.68	21.31	32.27	23.65	-27.59	1660.50	39.27
4	153	41	0.76	23.81	32.27	26.43	-27.59	1476.00	39.02
5	163	51	0.81	25.38	32.27	28.17	-27.59	1230.00	34.65
6	173	61	0.85	26.63	32.27	29.56	-27.59	1291.50	38.18
7	184	72	0.9	28.20	32.27	31.30	-27.59	1291.50	40.43
8	194	82	0.94	29.45	32.27	32.69	-27.59	1230.00	40.21
9	204	92	0.96	30.08	32.27	33.39	-27.59	1291.50	43.12
10	215	103	0.99	31.02	32.27	34.43	-27.59	1291.50	44.47
Roof	225	113	1.03	32.27	32.27	35.82	-27.59	615.00	22.03
								Base Shear	374.31
								Overturning Moment	24463.08

Table B-2:

South-North MWFRS									
Level	Elevation	z	K_z	q_z	q_h	Windward P (psf)	Leeward P (psf)	Tributary Area (ft^2)	Story Force (kip)
1	112	0	0.57	17.86	31.02	19.83	-26.52	0.00	0.00
2	126	14	0.57	17.86	31.02	19.83	-26.52	2160.00	42.82
3	139	27	0.68	21.31	31.02	23.65	-26.52	2077.00	49.12
4	153	41	0.76	23.81	31.02	26.43	-26.52	1778.40	47.01
5	163	51	0.81	25.38	31.02	28.17	-26.52	1482.00	41.75
6	173	61	0.85	26.63	31.02	29.56	-26.52	1556.10	46.00
7	184	72	0.9	28.20	31.02	31.30	-26.52	1615.10	50.56
8	194	82	0.94	29.45	31.02	32.69	-26.52	1600.00	52.31
9	204	92	0.96	30.08	31.02	33.39	-26.52	1383.00	46.18
10	215	103	0.99	31.02	31.02	34.43	-26.52	585.00	20.14
								Base Shear	395.90
								Overturning Moment	23041.71

Appendix B: Wind, Seismic, and Stiffness Tables:

Table B-3:

East-West MWFRS									
Level	Elevation	z	K_z	q_z	q_h	Windward P (psf)	Leeward P (psf)	Tributary Area (ft^2)	Story Force (kip)
1	112	0	0.57	17.86	31.02	19.83	-27.59	0.00	0.00
2	126	14	0.57	17.86	31.02	19.83	-27.59	877.50	17.40
3	139	27	0.68	21.31	31.02	23.65	-27.59	877.50	20.75
4	153	41	0.76	23.81	31.02	26.43	-27.59	1070.00	28.28
5	163	51	0.81	25.38	31.02	28.17	-27.59	1230.00	34.65
6	173	61	0.85	26.63	31.02	29.56	-27.59	1291.50	38.18
7	184	72	0.9	28.20	31.02	31.30	-27.59	1291.50	40.43
8	194	82	0.94	29.45	31.02	32.69	-27.59	1230.00	40.21
9	204	92	0.96	30.08	31.02	33.39	-27.59	1291.50	43.12
10	215	103	0.99	31.02	31.02	34.43	-27.59	676.50	23.29
Roof	225	113	1.03	32.27	32.27	35.82	-27.59	120.00	4.30
								Base Shear	290.63
								Overturning Moment	18634.91

Table B-4:

West-East MWFRS									
Level	Elevation	z	K_z	q_z	q_h	Windward P (psf)	Leeward P (psf)	Tributary Area (ft^2)	Story Force (kip)
1	112	0	0.57	17.86	32.27	19.83	-27.59	0.00	0.00
2	126	14	0.57	17.86	32.27	19.83	-27.59	2227.50	44.16
3	139	27	0.68	21.31	32.27	23.65	-27.59	1437.50	34.00
4	153	41	0.76	23.81	32.27	26.43	-27.59	1757.60	46.46
5	163	51	0.81	25.38	32.27	28.17	-27.59	1427.60	40.22
6	173	61	0.85	26.63	32.27	29.56	-27.59	1510.10	44.64
7	184	72	0.9	28.20	32.27	31.30	-27.59	1510.10	47.27
8	194	82	0.94	29.45	32.27	32.69	-27.59	1427.60	46.67
9	204	92	0.96	30.08	32.27	33.39	-27.59	934.50	31.20
10	215	103	0.99	31.02	32.27	34.43	-27.59	252.00	8.68
Roof	225	113	1.03	32.27	32.27	35.82	-27.59	120.00	4.30
								Base Shear	334.63
								Overturning Moment	18317.04

Appendix B: Wind, Seismic, and Stiffness Tables:

Seismic Table:

Table B-5:

Seismic Loads				
Story	Story Weight (k)	Height (ft)	C_{vx}	Story Force (k)
2	1509	14	0.0177	4.83
3	2349	27	0.0564	15.39
4	1875	41	0.0710	19.37
5	1890	51	0.0908	24.77
6	1824	61	0.1066	29.06
7	1935	72	0.1354	36.94
8	1971	82	0.1590	43.36
9	1971	92	0.1802	49.15
10	1287	103	0.1331	36.30
Penthouse	434	113	0.0497	13.54
Total	17045	Overturning Moment		20319.07
Base Shear	272.72			

Appendix B: Wind, Seismic, and Stiffness Tables:

Stiffness Tables:

Table B-6:

N-S Shear Walls			
Story	Shear Wall	Shear	Relative Stiffness (K)
2	1	47.82	0.206
	2	69.21	0.299
	3	65.01	0.281
	4	49.61	0.214
	Total=	231.65	1
3	1	50.45	0.223
	2	64.73	0.286
	3	60.31	0.267
	4	50.58	0.224
	Total=	226.07	1
4	1	42.13	0.205
	2	51.44	0.250
	3	59.28	0.288
	4	53.11	0.258
	Total=	205.96	1
5	1	39.52	0.210
	2	51.52	0.274
	3	49.46	0.263
	4	47.64	0.253
	Total=	188.14	1
6	1	36.01	0.213
	2	48.66	0.288
	3	45.32	0.268
	4	38.93	0.230
	Total=	168.92	1
7	1	36.87	0.254
	2	39.7	0.273
	3	38.36	0.264
	4	29.39	0.202
	Total=	145.32	1

Table B-7:

E-W Shear Walls			
Story	Shear Wall	Shear	Relative Stiffness (K)
2	5	54.91	0.237
	6	57.43	0.248
	7	53.67	0.232
	8	65.64	0.283
	Total=	231.65	1
3	5	52.55	0.232
	6	54.45	0.241
	7	53.02	0.235
	8	66.05	0.292
	Total=	226.07	1
4	5	49.56	0.241
	6	50.11	0.243
	7	48.78	0.237
	8	57.51	0.279
	Total=	205.96	1
5	5	43.02	0.229
	6	44.97	0.239
	7	45.6	0.242
	8	54.55	0.290
	Total=	188.14	1
6	5	55.87	0.331
	6	57.14	0.338
	7	55.91	0.331
	8	xx	xxxxx
	Total=	168.92	1
7	5	45.55	0.313
	6	46.08	0.317
	7	53.69	0.369
	8	xx	xxxxx
	Total=	145.32	1

Appendix B: Wind, Seismic, and Stiffness Tables:

Stiffness Tables:

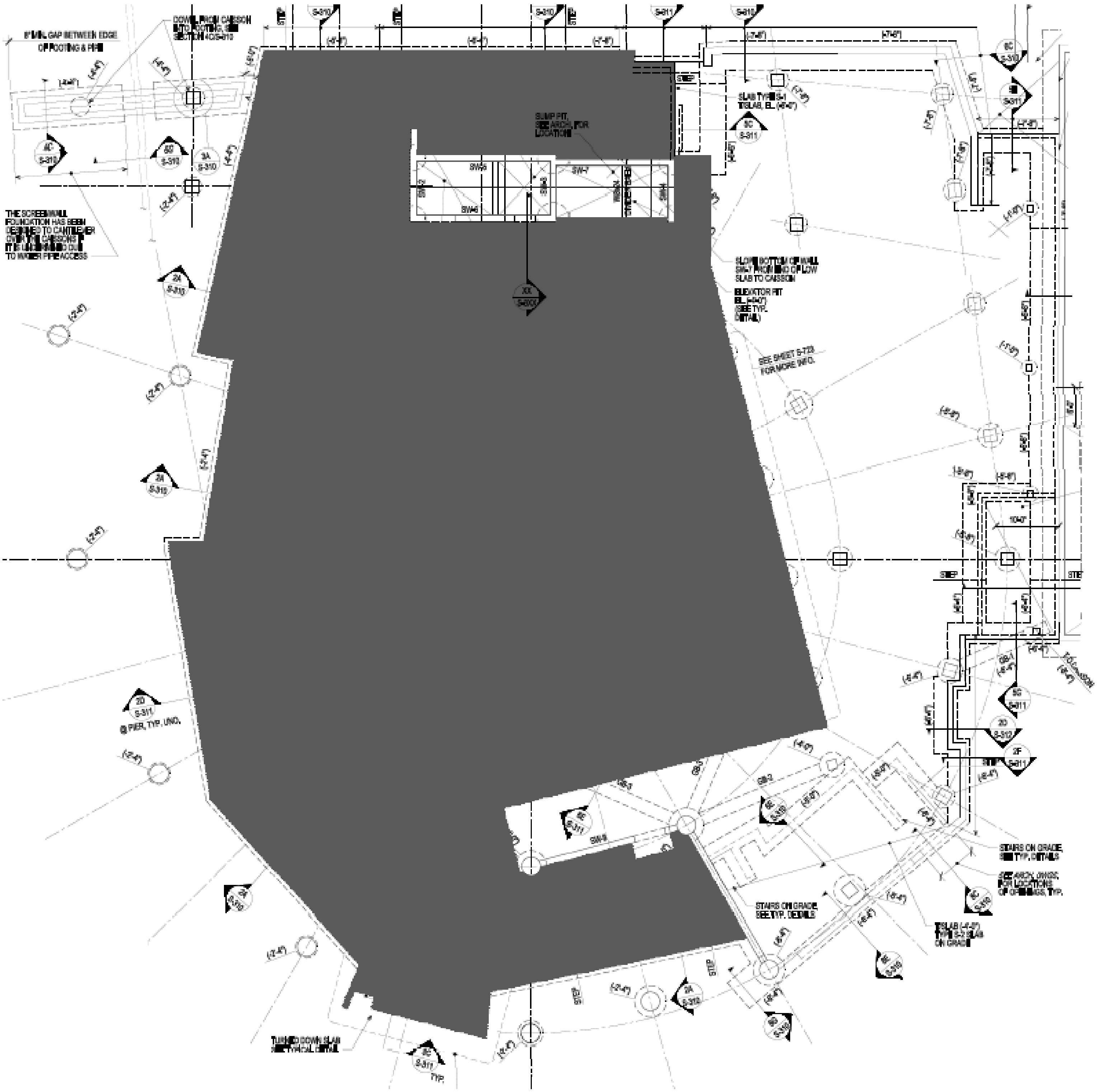
Table B-6 Cont.:

N-S Shear Walls			
Story	Shear Wall	Shear	Relative Stiffness (K)
8	1	31.38	0.268
	2	32.14	0.275
	3	31.29	0.268
	4	22.15	0.189
	Total=	116.96	1
9	1	20.13	0.239
	2	24.07	0.286
	3	22.25	0.264
	4	17.74	0.211
	Total=	84.19	1
10	1	xx	xxxxx
	2	15.31	0.326
	3	15.78	0.336
	4	15.81	0.337
	Total=	46.9	1
Roof	1	xx	xxxxx
	2	4.65	0.322
	3	5.13	0.355
	4	4.66	0.323
	Total=	14.44	1

Table B-7 Cont.:

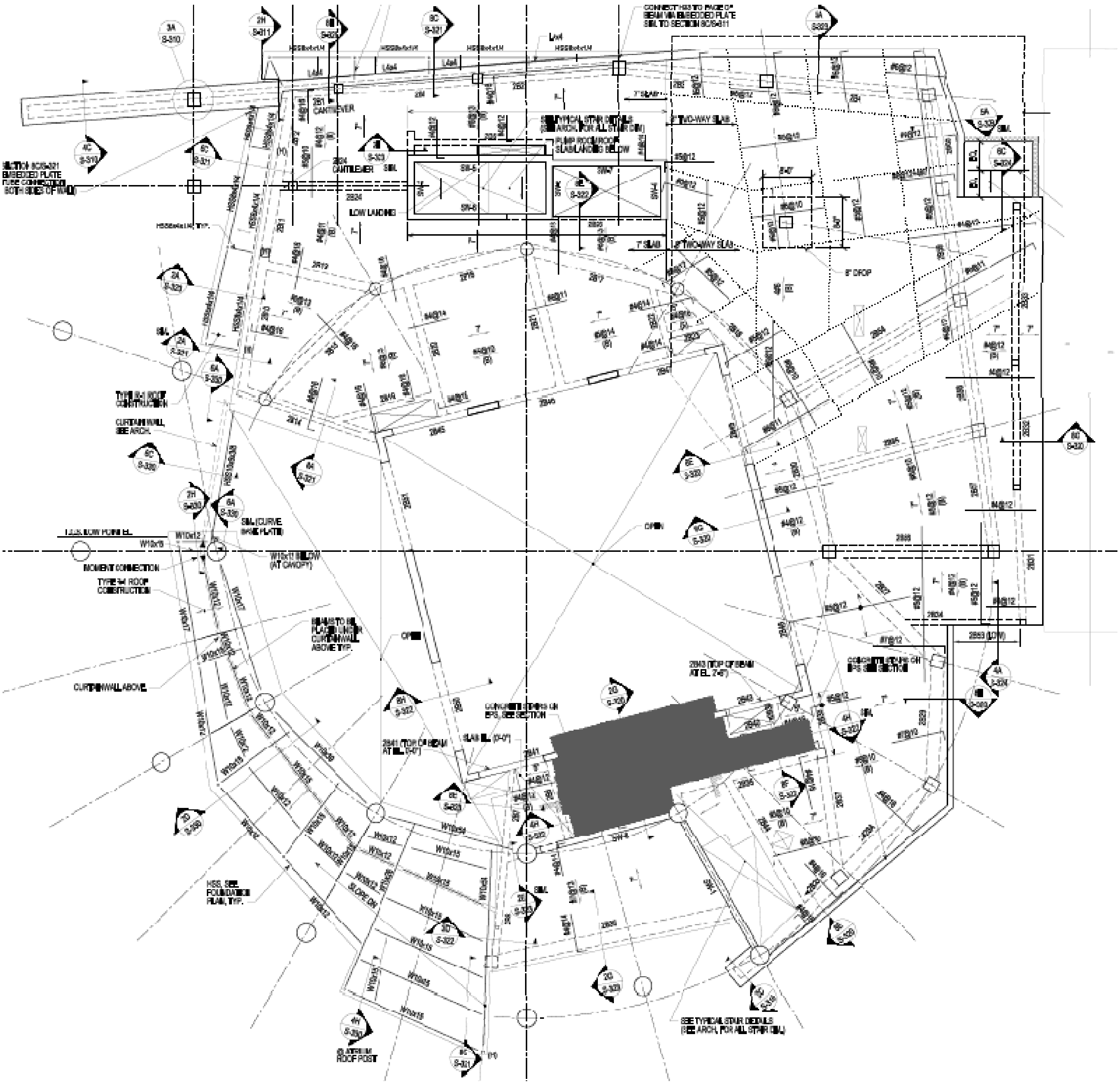
E-W Shear Walls			
Story	Shear Wall	Shear	Relative Stiffness (K)
8	5	39.35	0.336
	6	38.11	0.326
	7	39.5	0.338
	8	xx	xxxxx
	Total=	116.96	1
9	5	26.67	0.317
	6	28.05	0.333
	7	29.47	0.350
	8	xx	xxxxx
	Total=	84.19	1
10	5	15.26	0.325
	6	14.99	0.320
	7	16.65	0.355
	8	xx	xxxxx
	Total=	46.9	1
Roof	5	4.65	0.322
	6	4.01	0.278
	7	5.78	0.400
	8	xx	xxxxx
	Total=	14.44	1

Appendix C: Structural Plans



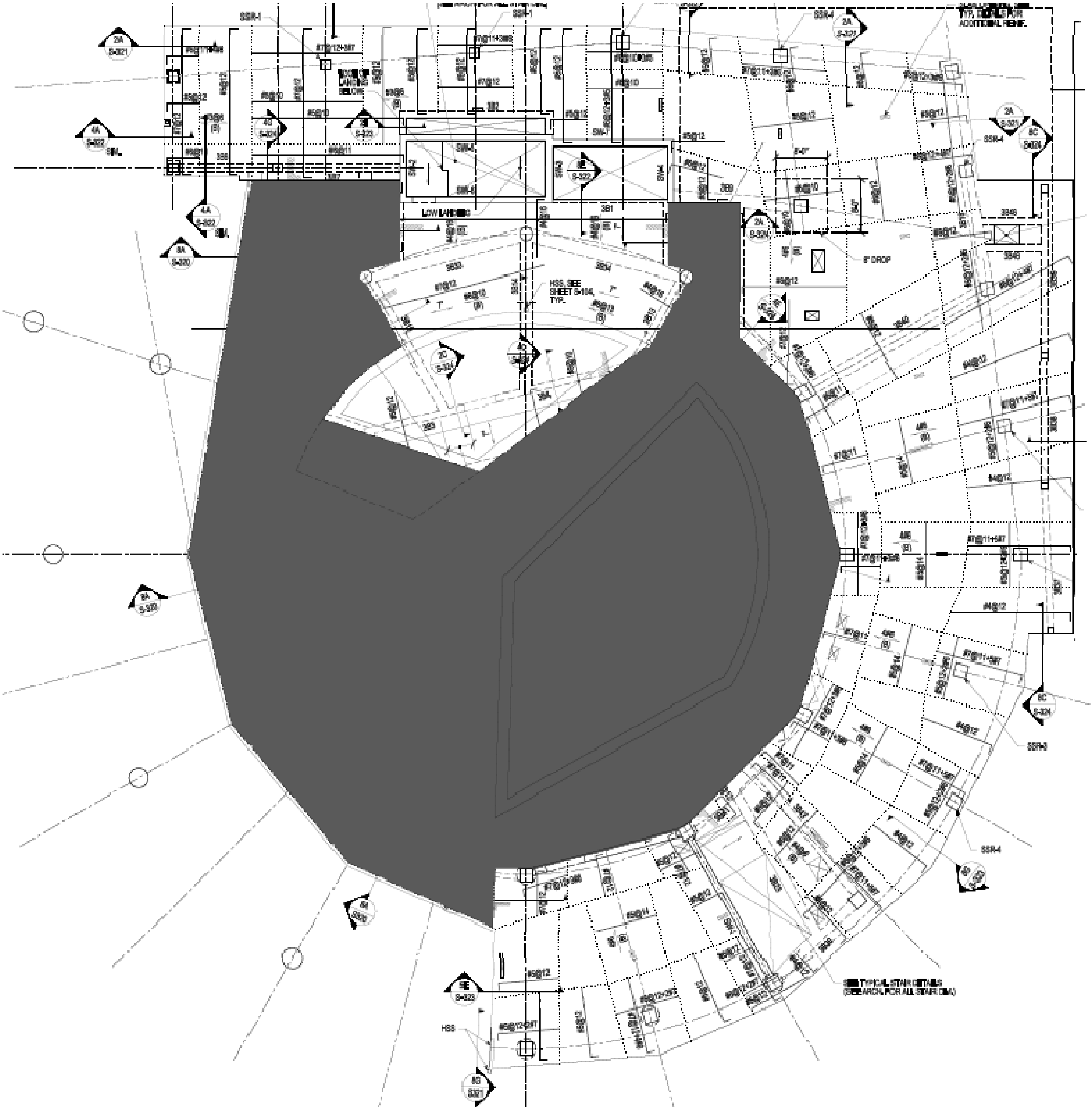
Level 1 Framing Plan– shaded area represents a depressed floor slab

Appendix C: Structural Plans



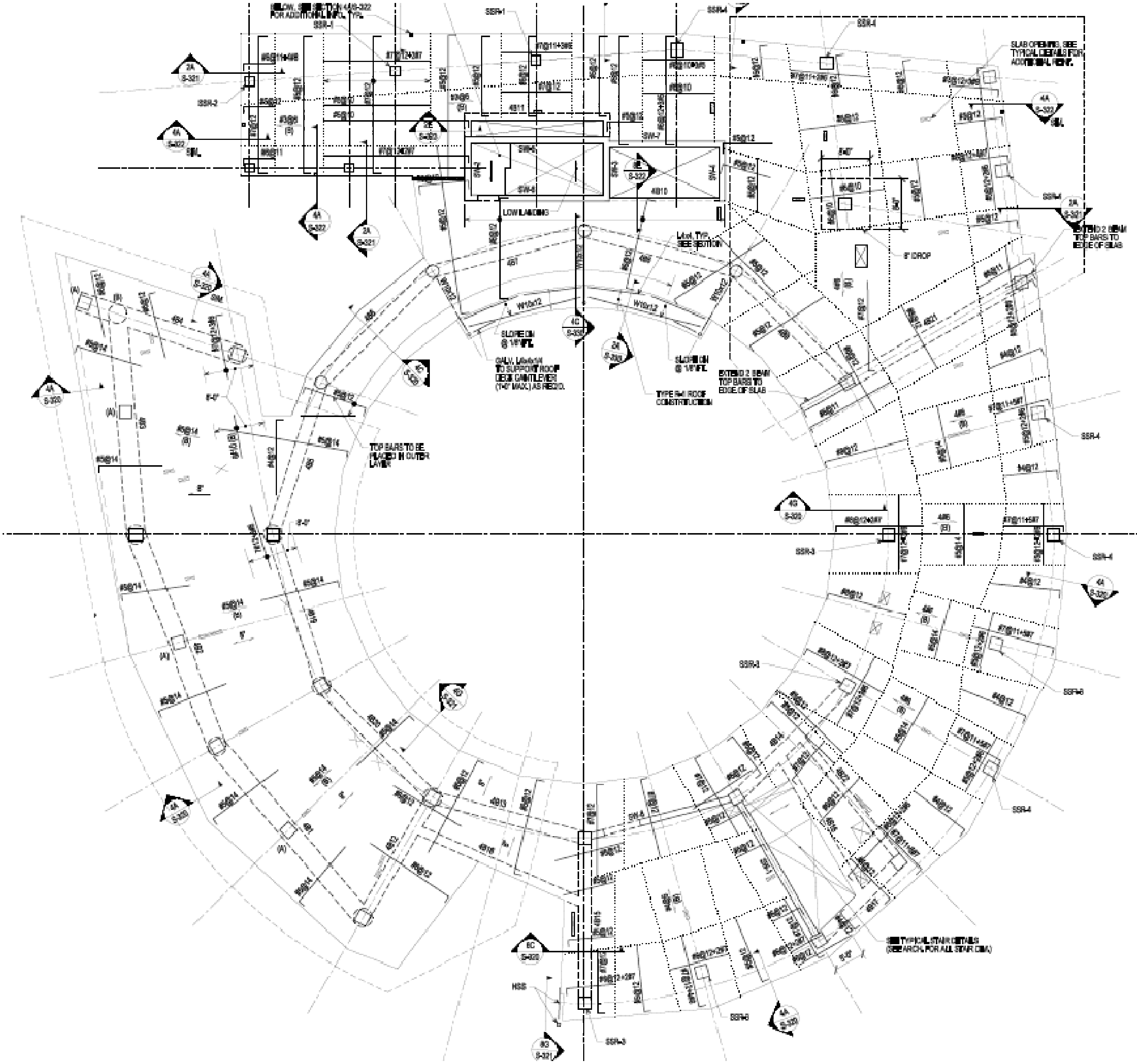
Level 2 Framing Plan

Appendix C: Structural Plans



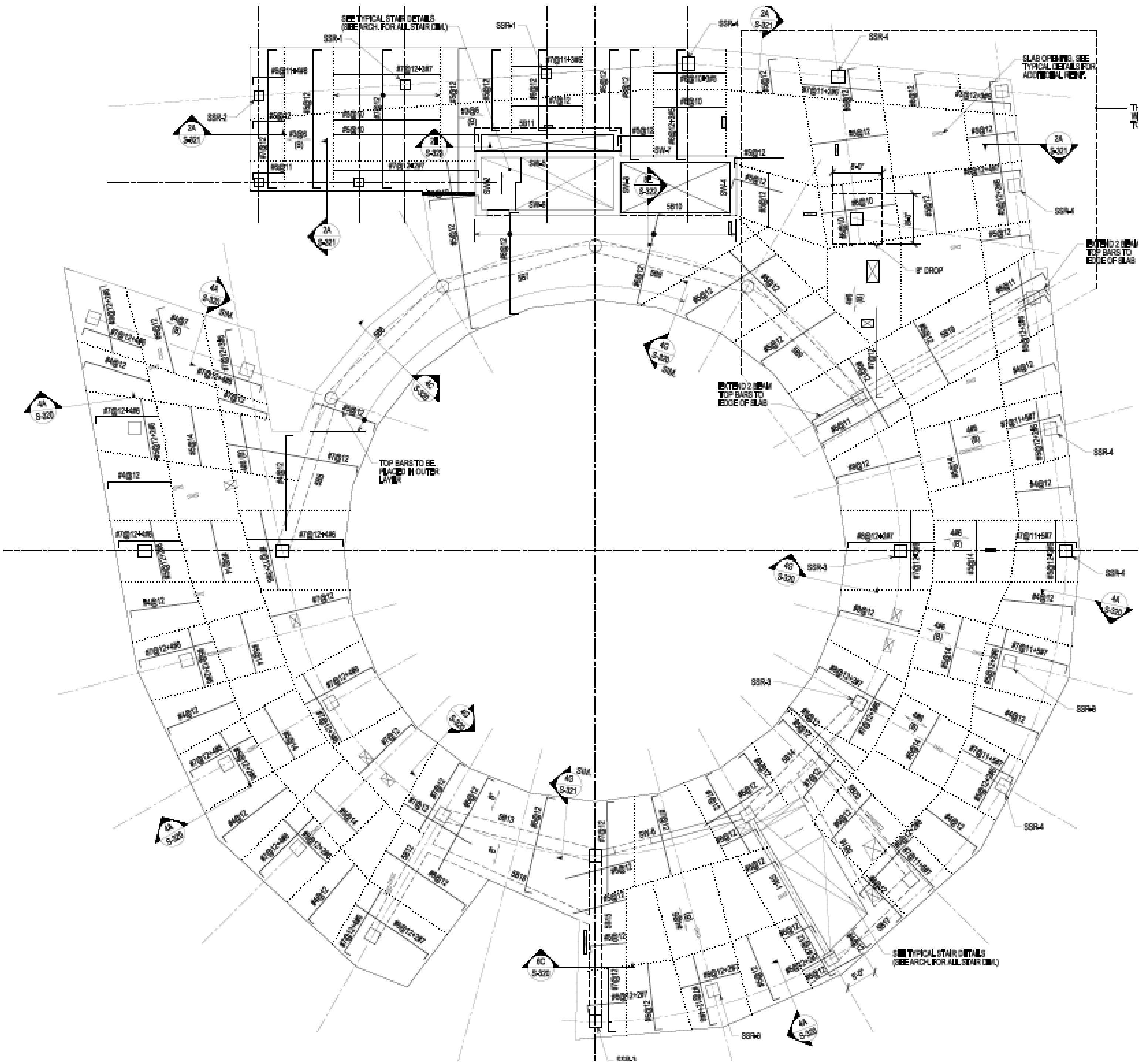
Level 3 Framing Plan– shaded area represents a depressed floor slab

Appendix C: Structural Plans



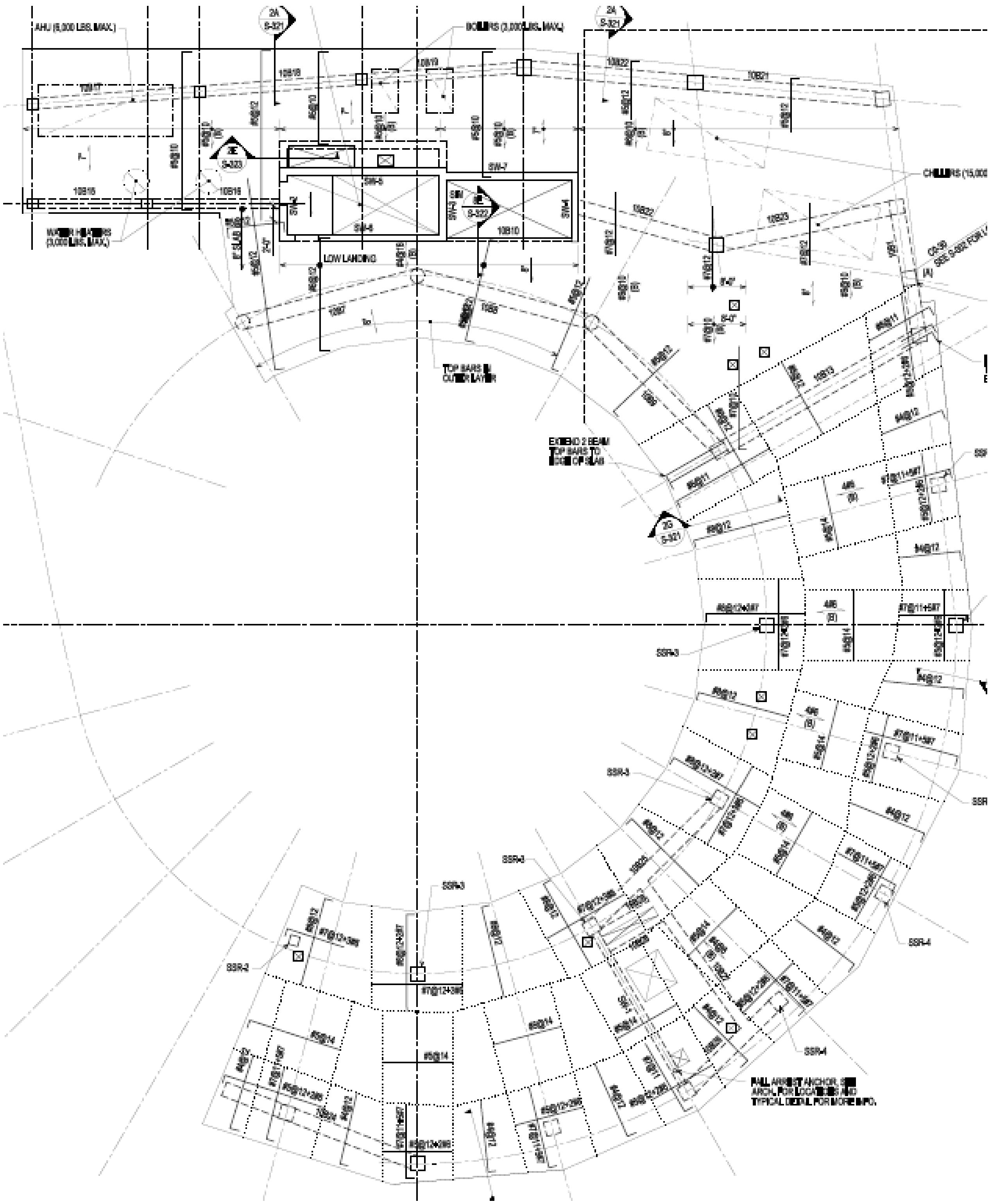
Level 4 Framing Plan

Appendix C: Structural Plans



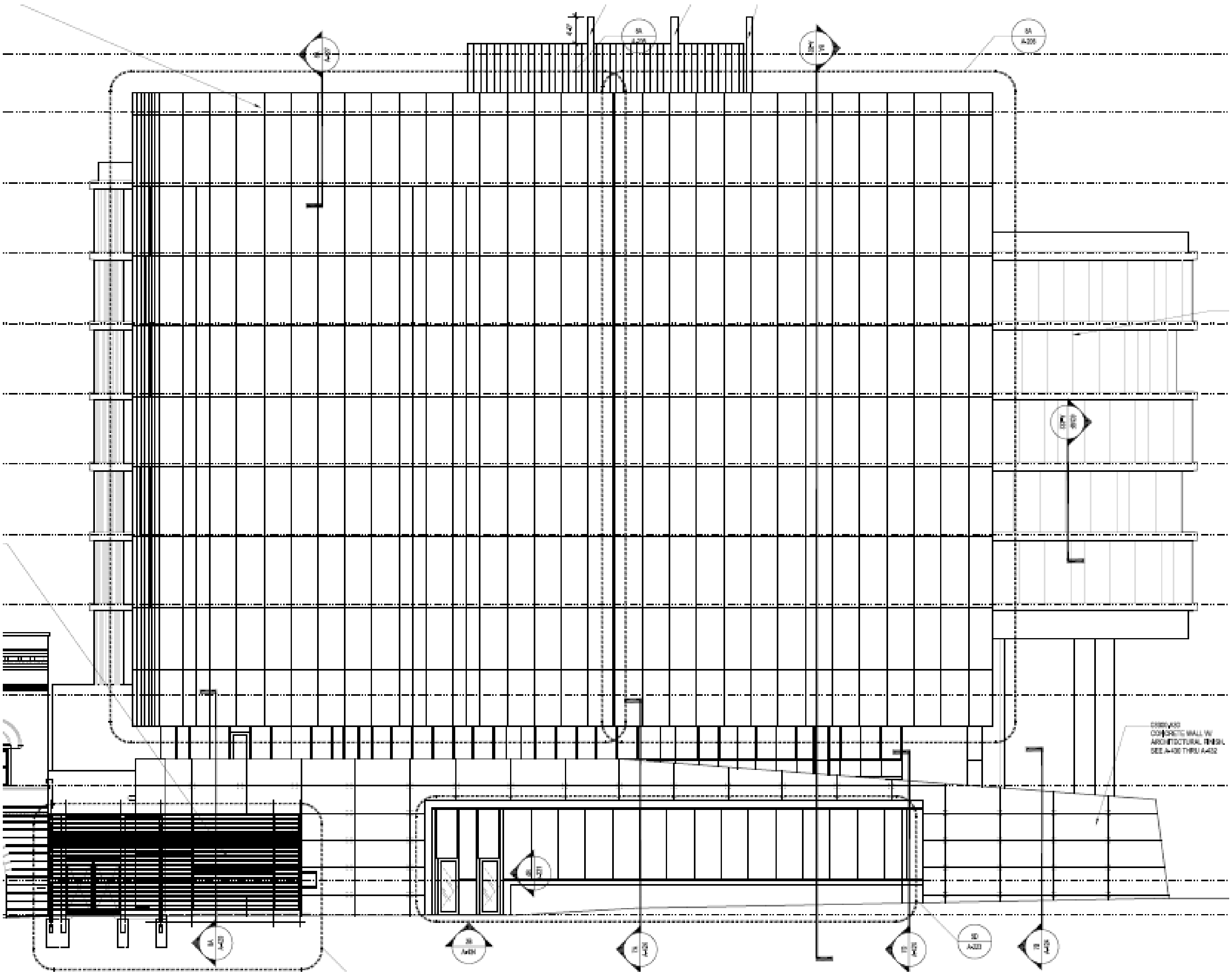
Level 5-9 Framing Plan

Appendix C: Structural Plans



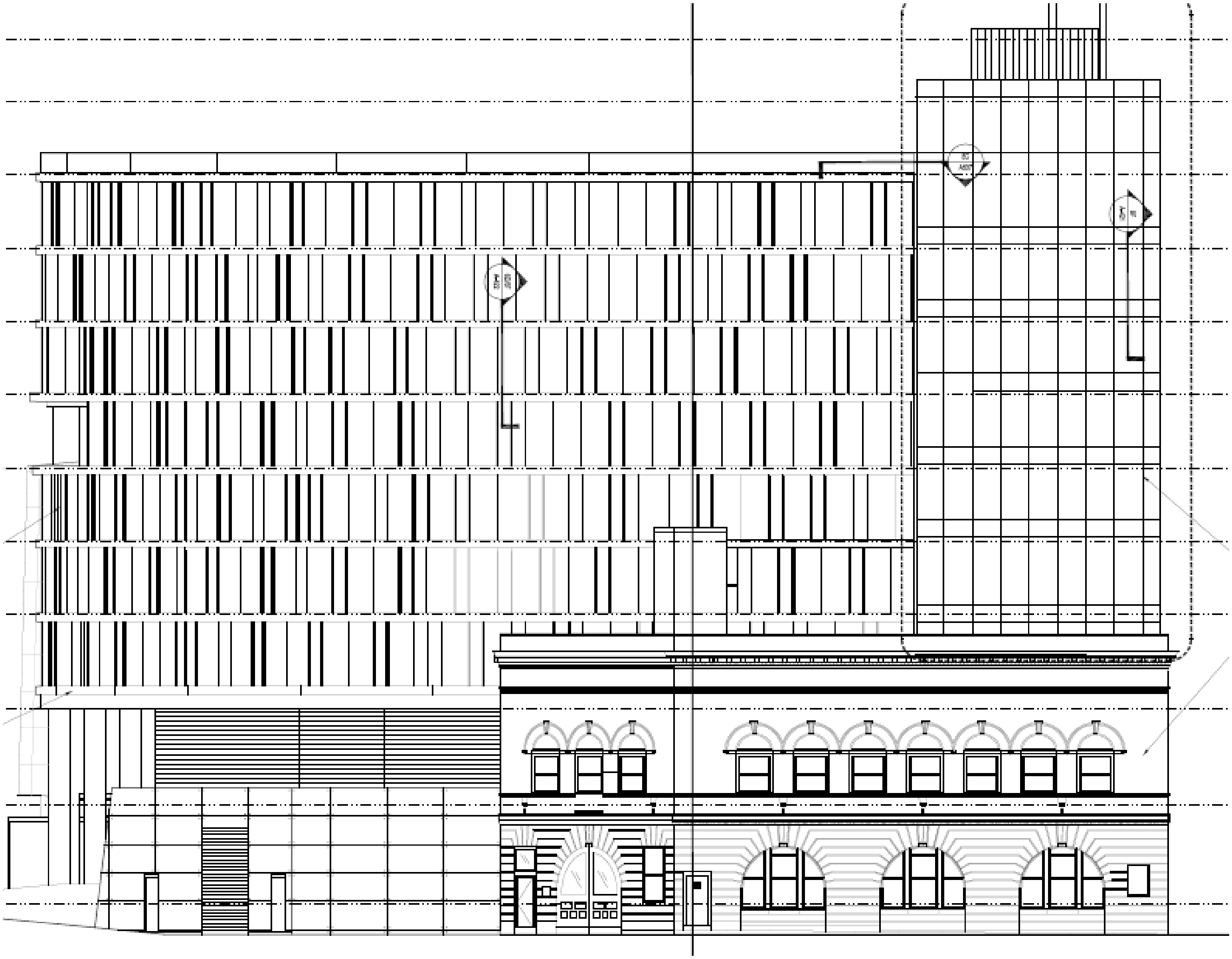
Level 10 Roof Framing Plan

Appendix C: Structural Plans



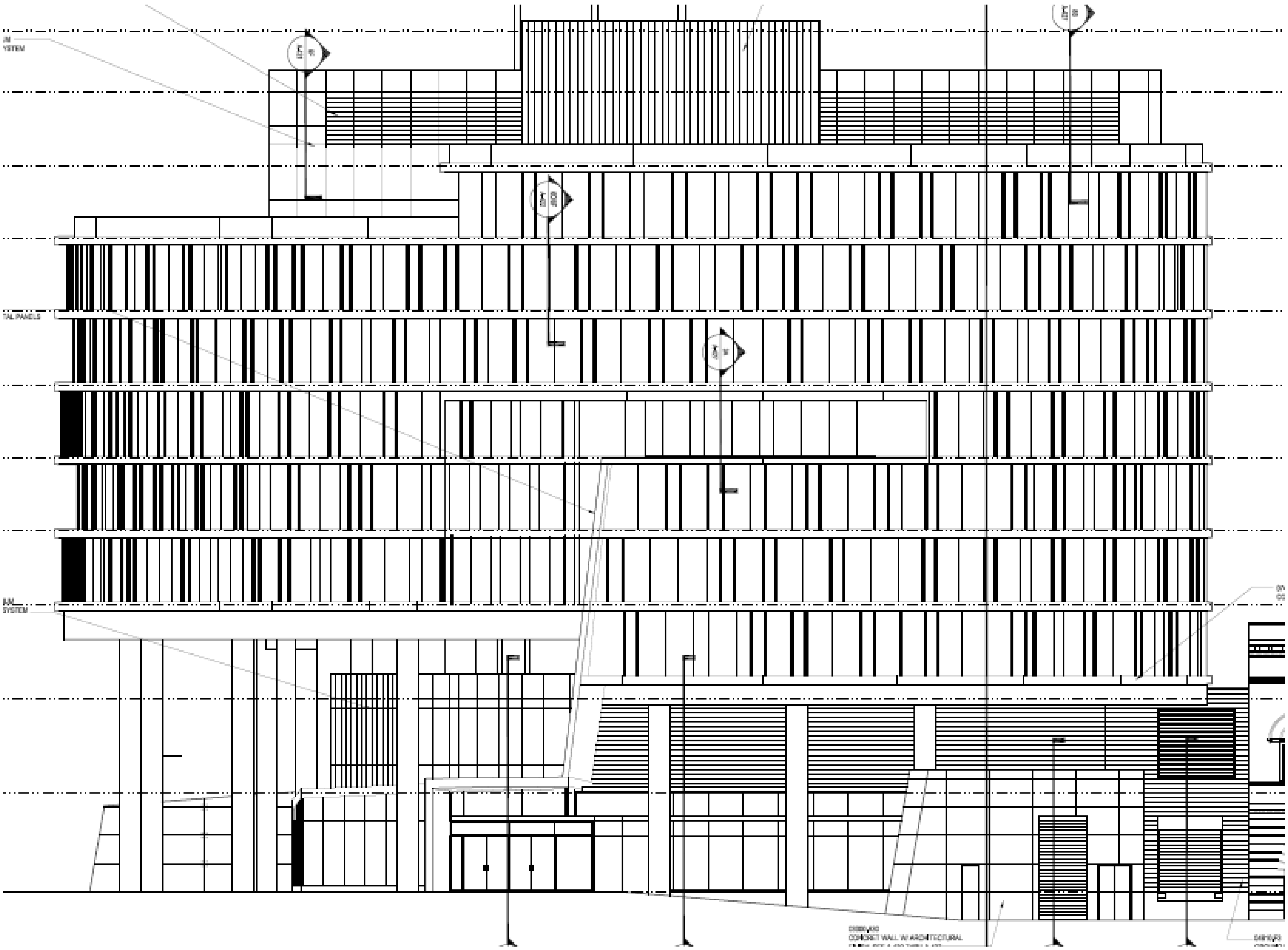
North Building Elevation

Appendix C: Structural Plans



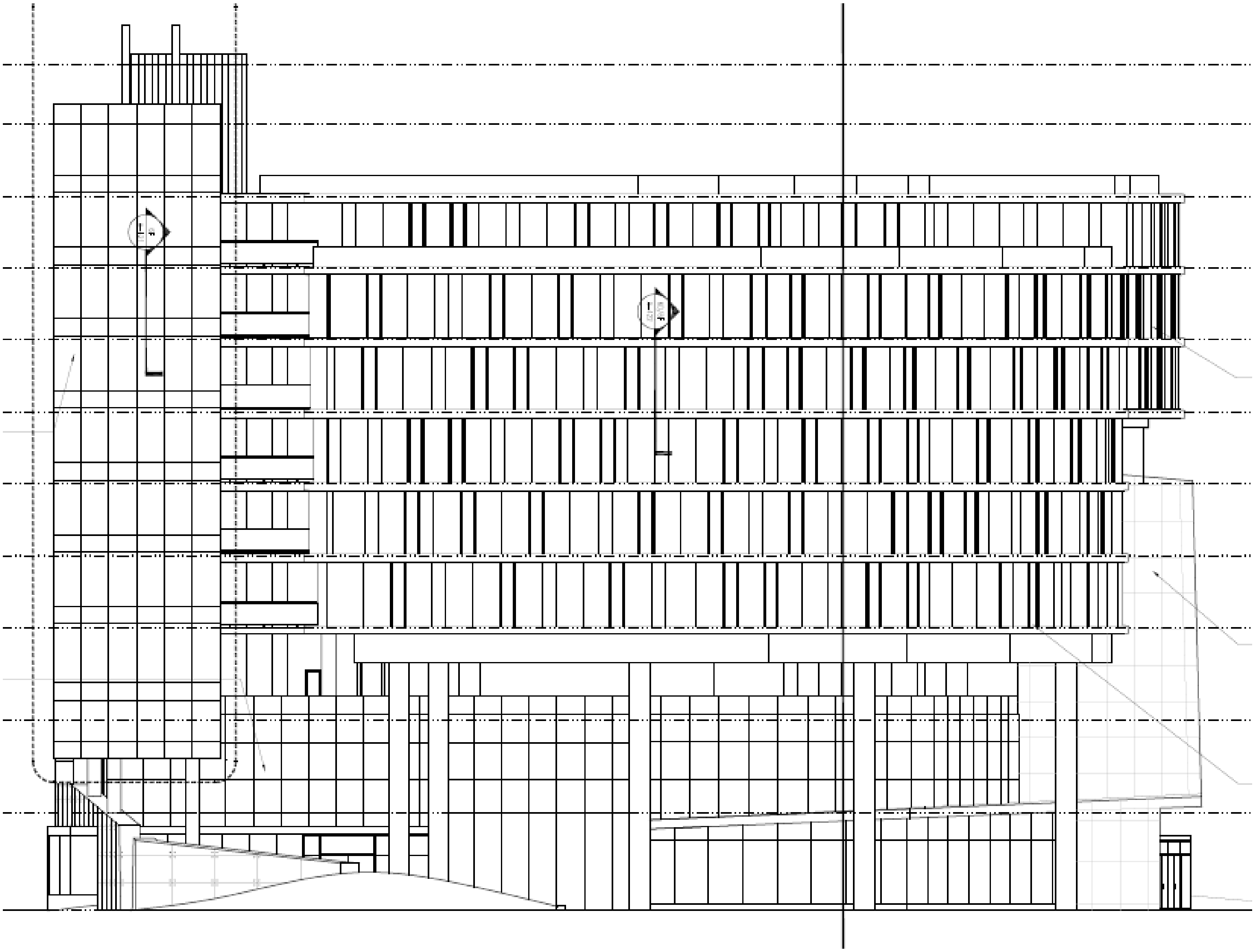
East Building Elevation

Appendix C: Structural Plans



South Building Elevation

Appendix C: Structural Plans



West Building Elevation