Technical Report I

MICA Gateway Residence

Scott R.
Molongoski

Structural Option ~ Heather Sustersic ~ September 17, 2012

Table of Contents:

Executive Summary:

Technical Report I of the Senior Thesis Project is a preliminary analysis of the building structure as a whole. The report details the existing structural conditions of the building and explores the design decisions that led to the final building product. Strong emphasis is placed on the codes, materials, gravity and lateral framing systems, as well as the gravity, wind, and seismic loads that govern the building design.

The MICA Gateway Residence building is a 9 story mixed use building located in Baltimore, Maryland. The building includes 64 student apartments, art galleries, studios, a café, and a multipurpose “black-box” theater facility. The building is circular in plan with a large open air courtyard in the center starting on the third floor. There are two main components of the building plan; a rectangular tower and a circular drum.

Structurally the building is primarily concrete, with two way flat plate slabs forming most of the floor framing systems. Ordinary concrete shear walls form the buildings lateral resisting system. There are also a variety of unique conditions in the structure, including slender columns nearly 40’ in height, and long span beams that measure 48” wide by 48” deep.

Gravity load spot checks were preformed on a variety of structural members to determine the structural adequacy of the system. A typical concrete beam, two-way flat plate slab, typical column, and long span beam were analyzed. Based on the preformed analysis, the building is deemed structurally adequate.

Wind and seismic loads were also analyzed based on ASCE 7-10. Wind design pressures were calculated on all four primary faces of the structure, with story force, base shear, and overturning moment also calculated. A similar analysis was preformed for seismic loads to determine seismic base shear and overturning moment. The largest overturning moment due to wind was found to be 24463 k-ft on the North face of the building. The overturning moment due to seismic forces was found to be 8343 k-ft. The conclusion was therefore that wind forces controlled the design of the Gateway lateral system.

The appendices of Technical Report I include hand calculations, wind and seismic load spreadsheets, and select structural framing plans.

Building Introduction:

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 will be explored in-depth later in the report. Beyond the multipurpose assembly room, Gateway features 64 student apartments, art galleries, studios, and a café.

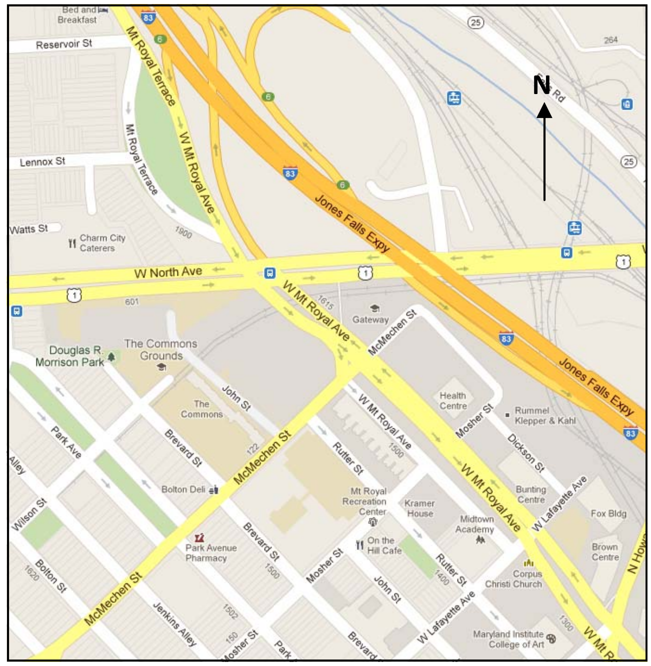


Figure 1: Gateway location in Baltimore

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, most prominently 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, most prominently in 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 design in compliance with the following:

- Baltimore City Code in accordance with IBC 2000
- ASCE 7-05– Minimun Design Loads for Buildings and Other Structures
- ACI 318-05– General Design of Reinforced Concrete
- AISC 12th Edition– Specifications for Structural Steel Buildings
- AWS D1.1– Structural Welding Code– Steel
- ACI 530– masonry structures

Building Materials:

MICA Gateway was designed and constructed using the following materials as specified in General Notes 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 on the various floor locations. 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 taken into account.

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 the 240 psf saturated soil load. Only applies to planters that are not above the multi-use performance space.

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

*** Takes into account walking areas of the plaza not above the multi-use performance space.

**** Takes into account walking areas 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 taken into account.

Snow Load:

Based on ASCE 7-05, which assumes a ground snow load of 25 psf, the roof snow load was calculated at 19.25 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’ 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 cover the areas below the courtyard. They work in conjunction with concrete beams that span very irregular areas. On Level 3, the courtyard sits directly on top of the “black-box” theater, which requires a space completely devoid of column and other obstructions. As such 48x48 beams were designed to span the almost 60’ of the theater and accommodate the large dead and live load from the plaza and planters in the courtyard above. These beams have 16#10 bottom reinforcing bars to resist the gigantic moments produced by the load. They are outlined in green in Figure 2 to the right.

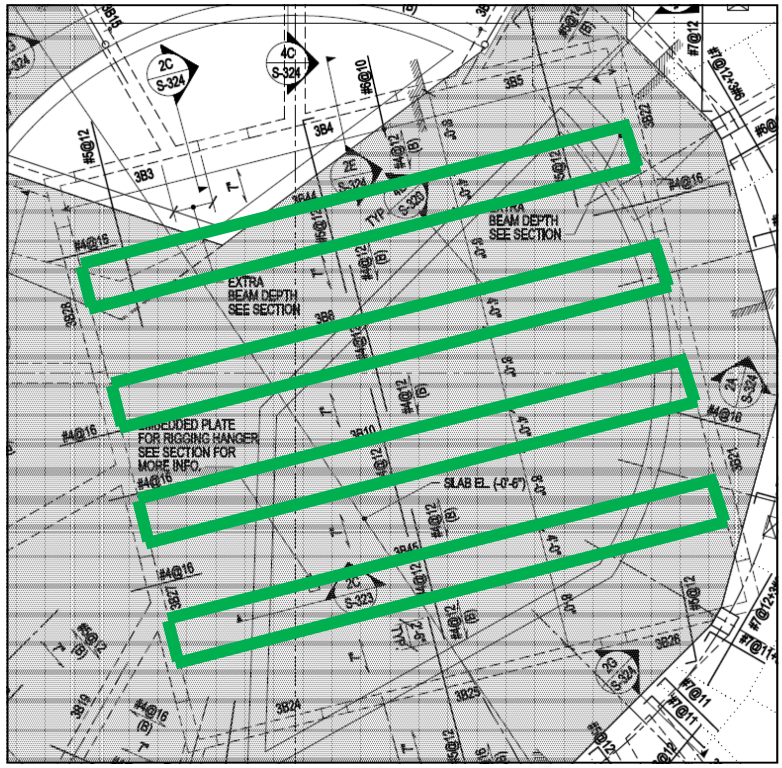


Figure 2: Long-span beams supporting the plaza

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 large beams run between columns so as to support new columns that rise to support the upper floors. These beams are 36x60 to take the load from the upper floors. Other typical beams in the building have sizes ranging from 8x18 to 24x24. 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 3 to the right. The dotted lines represent the boundaries between the column and middle strips.

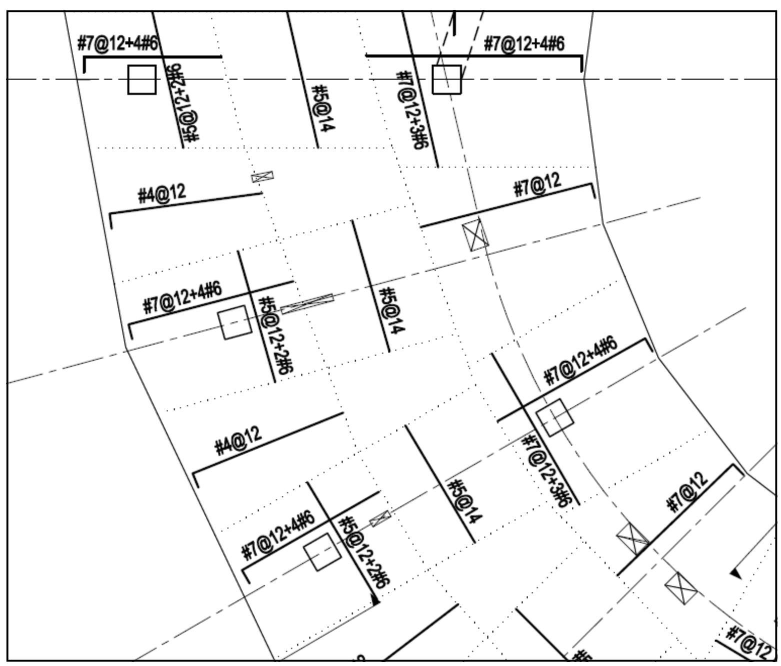


Figure 3: Typical two-way flat plate slab

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 4 below. The only uses of steel framing in this building are over the entrance and lobby, using mainly W10x15, W10x12, and HSS8x3x3/16.

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.

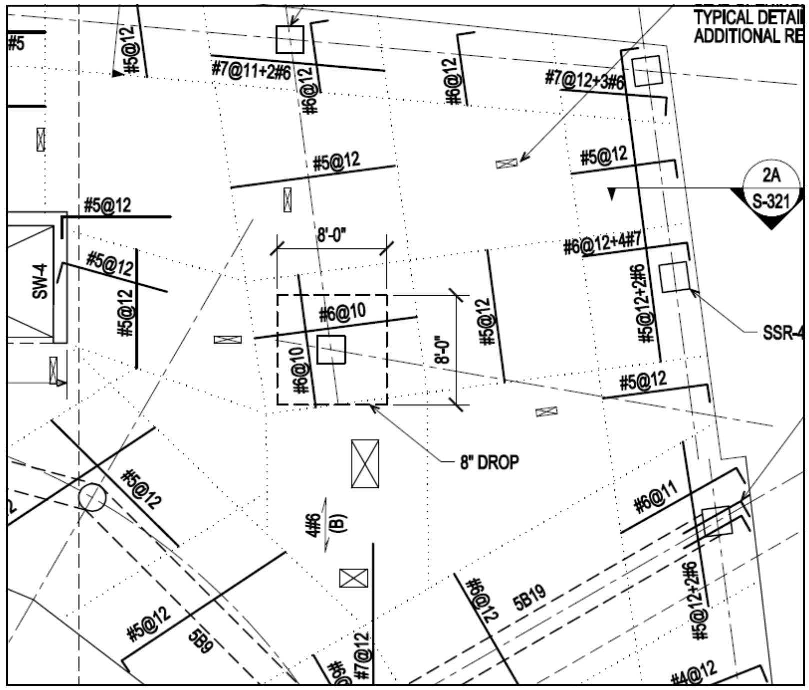


Figure 4: Two-way slab and drop panel around CO-7

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" that come from the smaller loads on the roof areas.

Gravity Spot Checks:

Gravity spot checks were preformed on several structural components to assess the structural adequacy of the Gateway. The long span beams were analyzed to determine whether the rebar used in the design was adequate. The calculations determined that (14) #10 bars are to be used for bottom reinforcement, (8) #10 bars for top reinforcement and shear reinforcement (2) #4 bars at 7". The actual design called for (16) #10 bottom bars, a discrepancy probably due to different assumptions of the loads. The same is true for the shear reinforcement, with the actual design requiring (2) #4 at 6".

A spot check was also done on a more typical beam supporting the Level 2 mechanical space. In this spot check the tributary area of the beam was estimated by finding the distance between the surrounding columns and then simplifying an irregular area into a rectangle. The resulting reinforcement calculated for the beam was similar to the actual design, with the discrepancy again due to differing assumptions of the load conditions and the tributary area.

A column supporting the roof slab was also analyzed to determine its structural capacity. The column was analyzed for pure axial strength, the balanced strain condition, and pure bending. This data was then organized into an interaction diagram. The column loading was then determined and then axial and bending strengths were calculated. The results proved that the column was structurally adequate.

A final spot check was done on the two-way flat plate slab using spSlab. The computer analysis approximated the irregular column spans into rectangular spans as illustrated below. The results of the computer analysis showed that the design requirements of a continuous reinforcing bottom mat of #5 bars at 12" was adequate, as well as the top reinforcement of #5 bars. Only the reinforcement parallel to the column line was analyzed. Deflections were also calculated by spSlab, with maximum instantaneous deflections of 0.262" and maximum long-term deflections of 0.238", reasonable when checked against ACI 318-11 Table 9.5(b), which states that for a floor supporting nonstructural elements likely to be damaged by large deflections, the deflection limit must be L/240, which in this case is 1.1"

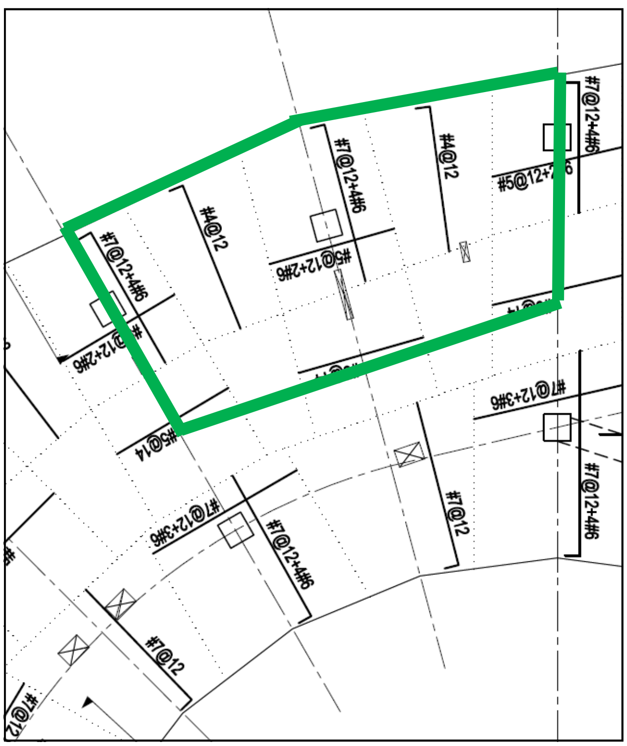


Figure 5: Actual area of two-way slab analysis

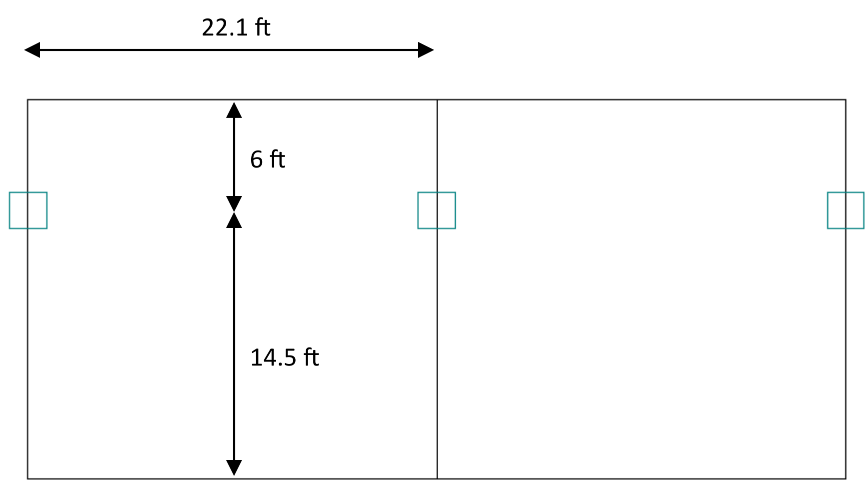


Figure 6: Approximation of two-way slab area using spSlab

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. The reinforced concrete moment frame is also assumed to take a significant amount of the lateral force (especially wind).

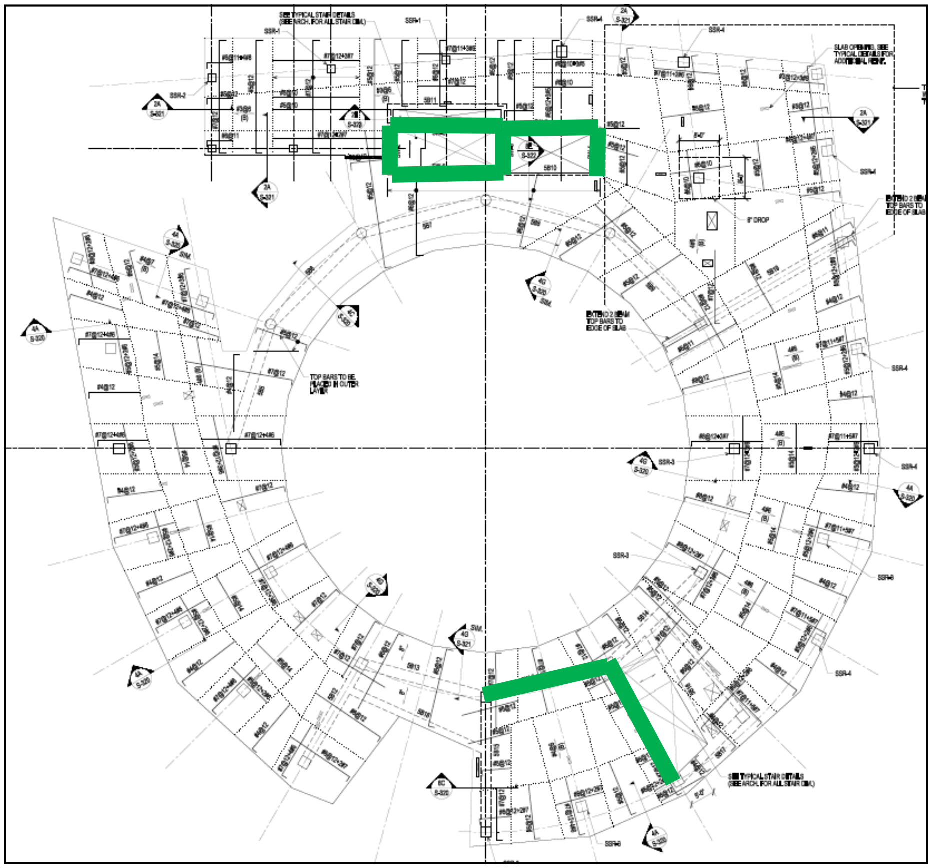


Figure 7: Shear wall locations

The lateral load path is as follows: load bears on the glass curtain wall, which is supported by the edge slab. From here the slab transfers the load into columns either directly or through beams. The columns then direct the load into the foundation. The 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.

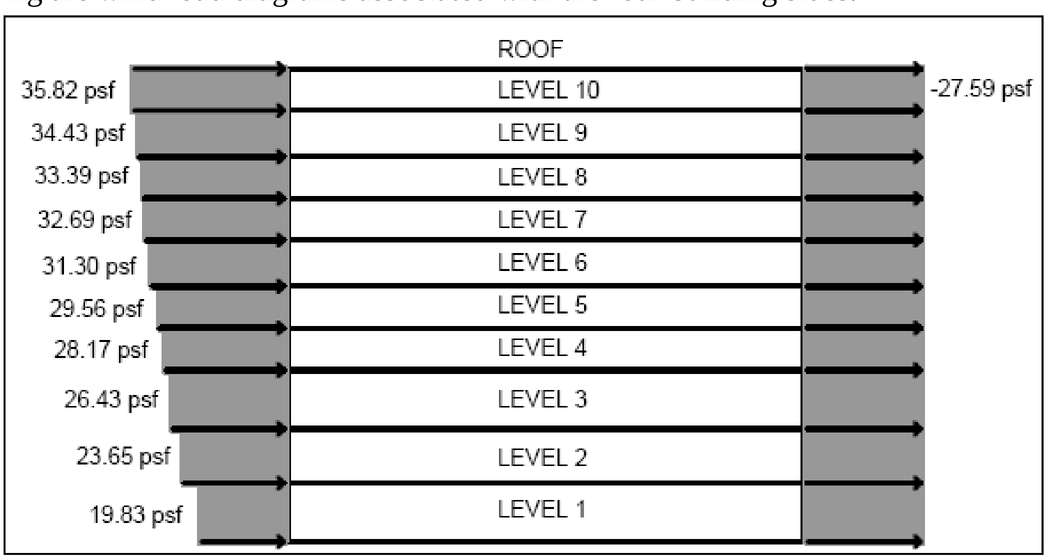


Figure 8: North-South Wind Design Pressure

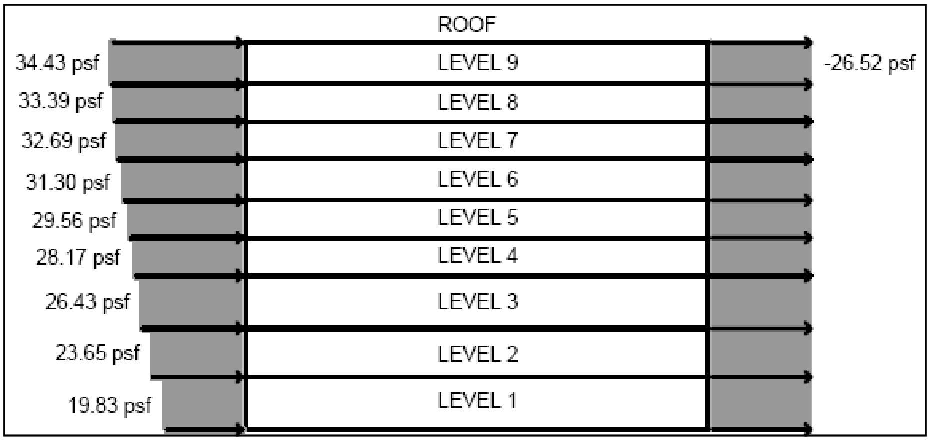


Figure 9: South-North Wind Design Pressure

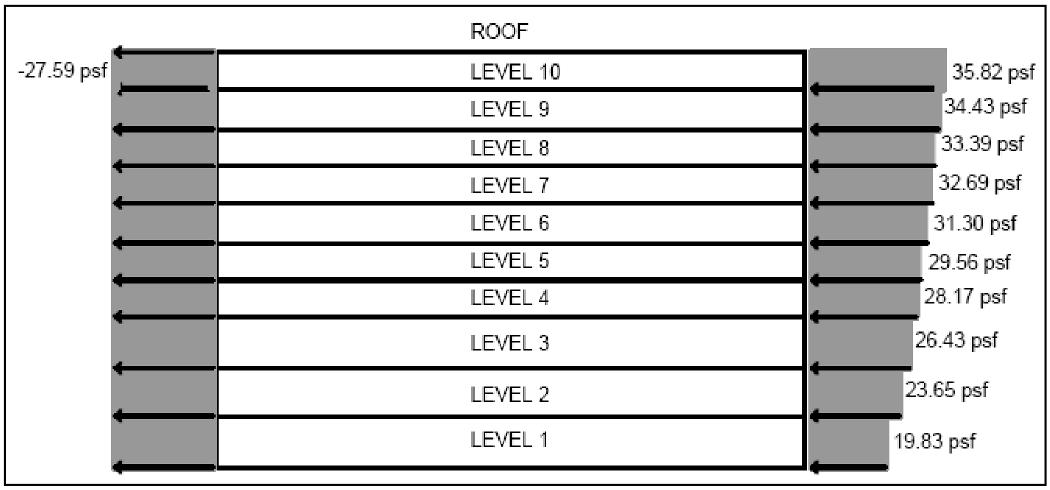


Figure 10: East-West Wind Design Pressure

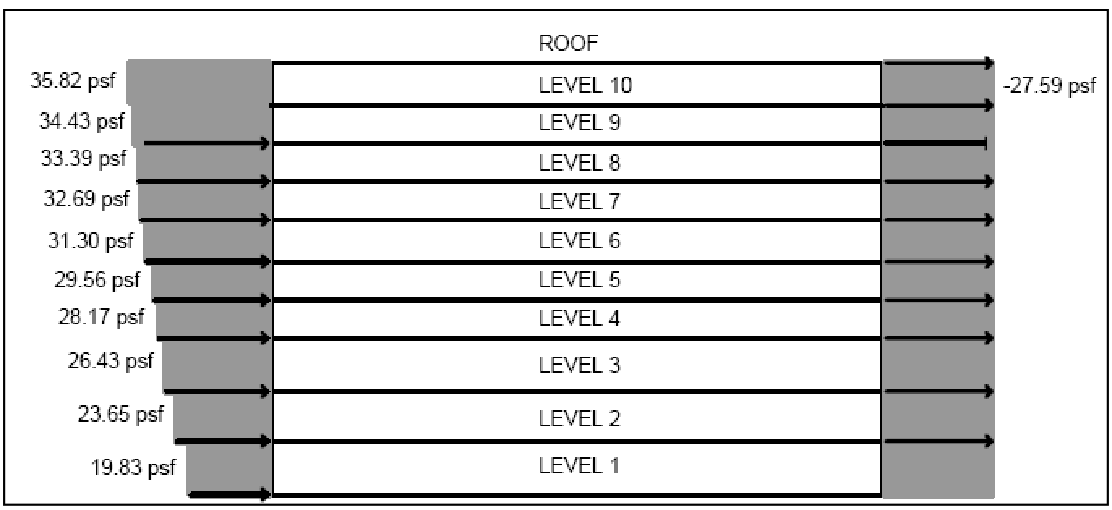


Figure 11: West-East Wind Design Pressure

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 different data at the time of the original design, or error from the USGS website. 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 5. Further calculations are detailed in Appendix B.

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.

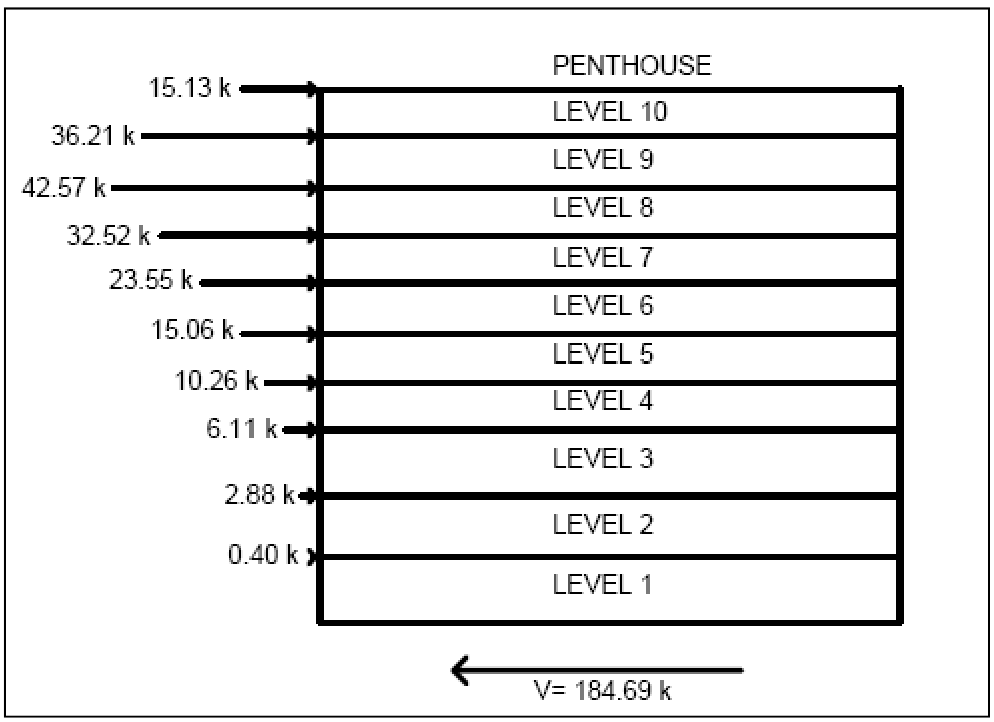


Figure 12: Seismic Story Force and Base Shear

Conclusion:

From this technical report, it was determined that wind loads caused an overturning moment of 24463 k-ft whereas seismic loads caused and overturning moment of 8343 k-ft. This proves that wind loads dictated the design of the Gateway's lateral force resisting system. Although the wind loads determined via this report are only approximations of the computer analysis performed by the designer, it can be assumed based on location and the large difference between the wind and seismic overturning moment that wind forces still governed design.

Based on the variety of spot checks done on the gravity resisting system, the structural adequacy of the building can be safely assumed. Differences in actually designed members and spot check results arose from different assumptions of loading, tributary areas, and code changes between then and now.

Completion of Technical Report One has provided a sufficient understanding of the structural systems that make the MICA Gateway Residence work. Further analysis of certain members and systems through computer software will yield an even greater understanding of the structure. The investigation performed for Technical Report One has shown that the Gateway is a thoroughly unique and intriguing building to work with.

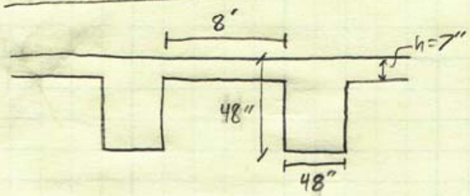
Appendicies:

Appendix A: Hand Calculations

1	Scott Molongoski	Tech One	Beam Calcs

Black-Box Theater Beam Calculations

3B8 on plans



- Beam is 58' long
- Beam supports the plaza and planters of the courtyard above

Loads

150 pcf concrete slab · 7/12 = 87.5 psf
150 pcf concrete beam · 4' = 600 psf
Saturated soil = 240 psf
Multi performance space roof, lighting, etc = 32 psf
Other DL = 15 psf

Live Load = 100 psf for plaza area

Snow Load = 25 psf

Total Dead Load = 974.5 psf

87.5 psf · 8' = 700 plf
600 psf · 4' = 2400 plf
(240+32+15)·12' = 3,444 plf

100 · 12' = 1200 plf
25 · 12' = 300 plf

Load Combination

$U = 1.2D + 1.6L + 0.5S$

$U = 1.2(700 + 2400 + 3444) + 1.6(1200) + 0.5(300)$

$U = 9.92\ ^{k}/_{ft}$

Minimum Thickness (h) → per Table 9.5(a) ACI 318-11

$\ell/16$ for beam $= 58/16 = 3.625' < 4'$ ∴ minimum thickness met ✓

$M_u^+ = \frac{w_u \cdot \ell^2}{12} = \frac{(9.92\ ^k/_{ft})(58\ ft)^2}{12} = \boxed{2781\ k\text{-}ft}$

$M_u^- = \frac{w_u \ell^2}{24} = \frac{(9.92)(58)^2}{24} = \boxed{1390\ k\text{-}ft}$

Midspan Reinforcement

Assume $d = 42"$

$A_s = \frac{M_u}{4d} = \frac{2781}{4(42)} = 16.55\ in^2$ → use 14 #10 = 17.78 in²

Assume over $A_{s,min}$

$A_{s,max} = 0.0181(48)(42) = 36.5\ in^2$ ✓

2 | Scott Molongoski | Tech One | Beam Calcs

Black-Box Theater Beam Calculations

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(17.78)(60)}{0.85(4)(48)} = 6.54''$

$c = a/\beta_1 = 6.54''/0.85 = 7.69''$

$\varepsilon_s = \frac{\varepsilon_u}{c}(d - c/2)$

$\varepsilon_s = \frac{0.003}{7.69}(42 - 7.69/2)$

$\varepsilon_s = 0.149 > 0.005$ ✓

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

$\phi M_n = 0.9(17.78)(60)(42 - 6.54/2)$

$\phi M_n = 3099 \text{ k-ft} > M_u = 2781 \text{ k-ft}$ ✓

End Reinforcement

$A_s = \frac{1390}{4(42)} = 8.27 \text{ in}^2 \rightarrow$ use 8 #10 bars $= 10.16 \text{ in}^2$

$a = \frac{(10.16)(60)}{0.85(4)(48)} = 3.74''$ $c = 3.74/0.85 = 4.39''$

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

$\phi M_n = 0.9(10.16)(60)(42 - 3.74/2)$

$\phi M_n = 1835 \text{ k-ft} > M_u = 1390 \text{ k-ft}$ ✓

Shear Reinforcement

$V_u = \frac{w_u \ell}{2} = \frac{(9.92)(58)}{2} = 288 \text{ k}$

$V_c = 2\lambda\sqrt{f'_c}\, b \cdot d = 2(1)\sqrt{4000}(48)(42) = 255 \text{ k} \rightarrow$ need shear reinforcement

$V_u > 0.5 \cdot 0.75 \cdot 255 = 95 \text{ k}$

$V_s = V_u/\phi - V_c = \frac{288}{0.75} - 255 = 129 \text{ k} \leq 8\sqrt{4000} \cdot 48 \cdot 42 = 1020 \text{ k}$

Use (2) #4, $A_v = 0.40$

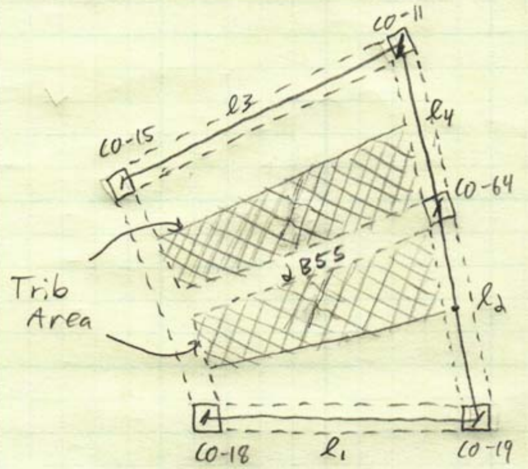
$s = \frac{A_v f_{yt} d}{V_s} = \frac{(0.4)(60)(42)}{129 \text{ k}} = 7.8 \rightarrow$ use (2) #4 @ 7"

3 | Scott Molongoski | Tech One | Beam Calcs

Typical Concrete Beam Calculations

Level 2 - Northeast Quadrant

- Beam 2B55
- Beam is 24"x24"
- Beam supports a mechanical space



Span lengths

	Location (ft)	
	X	Y
CO-18	48.67	0
CO-19	75.17	0
CO-64	72.67	19.47
CO-15	42.15	24.33
CO-11	69.67	40.40

l_1 distance

$l_1 = 75.17 - 48.67 = 26.50'$

l_2 distance

$l_2 = \sqrt{(75.17-72.67)^2 + 19.47^2} = 19.67'$

l_3 distance

$l_3 = \sqrt{(69.67-42.15)^2 + (40.4-24.33)^2} = 31.87'$

l_4 distance

$l_4 = \sqrt{(72.67-69.67)^2 + (40.4-19.47)^2} = 21.14'$

- Approximate beam to be 28' long.

Loads

Slab selfweight = 150pcf · 7/12 = 87.5 psf

Beam selfweight = 150pcf · 2 · 2 = 600 plf

Other DL = 9 psf

LL = 150 psf

$87.5 + 9 = 96.5 \text{ psf} \rightarrow$

$96.5 \cdot \tfrac{1}{2} \cdot l_2 + 96.5 \cdot \tfrac{1}{2} \cdot l_4 = w_D$

$96.5(\tfrac{1}{2})(19.67) + 96.5(\tfrac{1}{2})(21.14) = 1969.08 \text{ plf}$

$150 \cdot \tfrac{1}{2} l_2 + 150 \cdot \tfrac{1}{2} l_4 = w_L$

$150(\tfrac{1}{2})(19.67) + 150(\tfrac{1}{2})(21.14) = 3060.75 \text{ plf}$

$w_u = 1.2D + 1.6L$

$w_u = 1.2(1969.08 + 600) + 1.6(3060.75)$

$w_u = 7.98 \text{ k/ft}$

$M_u^+ = \frac{w_u l^2}{12} = \frac{(7.98)(28)^2}{12} = 521 \text{ k/ft}$ ← positive midspan moment

$M_u^- = \frac{w_u l^2}{24} = \frac{(7.98)(28)^2}{24} = 261 \text{ k/ft}$ ← negative end moments

4 | Scott Molongoski | Tech One | Beam Calcs

Typical Concrete Beam Calculations

Midspan Reinforcement

$d = h - \text{cover} - d_{stirrup} - d_{bar}/2 = 24'' - 1.5'' - 0.5 - 1'' = 21''$

$A_s = \frac{M_u}{4d} = \frac{521}{4(21)} = 6.2\ in^2 \rightarrow$ Use $5\ \#10 = 6.35\ in^2$

$A_{s_{min}} \geq \begin{cases} \frac{3\sqrt{f'_c}}{f_y} b \cdot d = \frac{3\sqrt{4}}{60}(24)(21) = 1.59\ in^2 \\ \frac{200}{f_y} b \cdot d = \frac{200}{60}(24)(21) = 1.68\ in^2 \end{cases}$ ✓

$A_{s_{max}} = \rho_{max}\, b \cdot d = 0.85(0.85)\left(\frac{4}{60}\right) \cdot \frac{0.003}{0.008} \cdot 24 \cdot 21 = 9.1\ in^2$ ✓

$a = \frac{A_s f_y}{0.85 f'_c \cdot b} = \frac{(6.35)(60)}{0.85(4)(24)} = 4.67''$

$c = a/\beta_1 = \frac{4.67}{0.85} = 5.49''$

$\varepsilon_s = \frac{\varepsilon_u}{c}(d - c)$

$= \frac{0.003}{5.49}(21 - 5.49) = 0.0085$ ✓

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

$\phi M_n = 0.9(6.35)(60)\left(21 - \frac{4.67}{2}\right)$

$\phi M_n = 533\ k\text{-}ft > M_u = 521\ k\text{-}ft$ ✓

Check Deflection

$h_{min} = \ell/16 = \frac{28 \cdot 12}{16} = 21'' < 24''$ ✓

End Reinforcement

$A_s = \frac{M_u}{4d} = \frac{261}{4(21)} = 3.11\ in^2 \rightarrow$ use $4\ \#8 = 3.16\ in^2$

$a = \frac{3.16(60)}{0.85(4)(24)} = 2.32''$

$c = a/\beta_1 = 2.32/.85 = 2.73''$

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

$\phi M_n = 0.9(3.16)(60)(21 - 2.32/2)$

$\phi M_n = 282\ k\text{-}ft > M_u = 261\ k\text{-}ft$ ✓

5 | Scott Molongoski | Tech One | Beam Calcs

Typical Concrete Beam Calculations

Shear Reinforcement Check

$$V_u = \frac{w_u \ell}{2} = \frac{(7.98)(28)}{2} = 112\ k$$

$$V_c = 2\lambda\sqrt{f'_c}\,b_w \cdot d = 2(1)\sqrt{4000}\,(24)(21)$$

$V_c = 63.75\ k$ → need shear reinforcement

$$V_s = V_u/\phi - V_c = \frac{112}{0.75} - 63.75 = 85.58\ k$$

Check $V_{s_{max}}: = 8\sqrt{f'_c}\,b_w \cdot d = 8\sqrt{4000}\cdot 24 \cdot 21 = 255\ k$ ✓

Max spacing: $V_s \leq 4\sqrt{f'_c}\,b_w \cdot d = 4\sqrt{4000}(24)(21) = 127.5\ k$

→ use $s_{max} = \min \begin{cases} d/2 = 10.5'' & \text{— controls} \\ 24'' \end{cases}$

$$A_v = \frac{V_s}{f_{yt}\,d/s} = \frac{85.58}{60 \cdot 21/10.5}$$

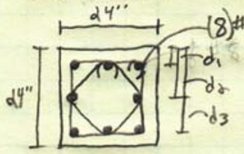
$A_v = 0.71\ in^2$ → too large, use (2) #4

$$s = \frac{A_v f_{yt}\,d}{V_s} = \frac{(0.4)(60)(21)}{85.58} = 5.89''$$ → use (2) #4 @ 5"

6 | Scott Molongoski | Tech One | Column Calcs

Column Calculations

Column CO-22 - Level 8



$f'_c = 4$ ksi $\quad \varepsilon_y = 0.00207 \quad d_1 = 2.5"$

$f_y = 60$ ksi $\quad d_2 = 12" \quad d_3 = 21.5"$

Pure Axial Strength

$P_o = 0.85 f'_c \cdot A_c + A_s \cdot f_y$

$P_o = 0.85(4)(24 \cdot 24 - 8 \cdot 0.79) + 8 \cdot 0.79 \cdot 60$

$\boxed{P_o = 2316 \text{ k}}$

Balanced Condition

$c = \frac{\varepsilon_u}{\varepsilon_u + 0.00207}(d) = \frac{0.003}{0.003+0.00207}(21.5) = 12.7"$

$\varepsilon_{s1} = \frac{0.003}{c}(c-d) = \frac{0.003}{12.7}(12.7-2.5) = 0.0024 > \varepsilon_y \quad \therefore f_{s1} = 60$ ksi

$\varepsilon_{s2} = \frac{0.003}{12.7}(12.7-12) = 0.00017 < 0.00207$

$f_{s2} = 0.00017 \cdot 29000 = 4.93$ ksi

$\varepsilon_{s3} = \frac{0.003}{12.7}(12.7-21.5) = -0.00208 > \varepsilon_y \quad \therefore \quad f_{s_i} = -60$ ksi

$P_b = 0.85 f'_c \cdot b \cdot \beta_1 \cdot c + \sum A_{si} f_{si}$

$P_b = 0.85(4)(24)(0.85)(12.7) + (3)(0.79)(60) + (2)(0.79)(4.93) + (3)(0.79)(-60)$

$\boxed{P_b = 889 \text{ k}}$

$M_b = 0.85 f'_c \cdot b \cdot \beta_1 \cdot c\left(\frac{h}{2} - \frac{\beta_1 \cdot c}{2}\right) + \sum\left[A_{si} f_{si}\left(\frac{h}{2} - d_i\right)\right]$

$M_b = 0.85(4)(24)(0.85)(12.7)\left(\frac{24}{2} - \frac{0.85 \cdot 12.7}{2}\right) + (3)(0.79)(60)\left(\frac{24}{2} - 2.5\right)$

$+(2)(0.79)(4.93)\left(\frac{24}{2} - 12\right) + (3)(0.79)(-60)\left(\frac{24}{2} - 21.5\right)$

$\boxed{M_b = 710 \text{ k-ft}}$

7 | Scott Molongoski | Tech One | Column Calcs

Column Calculations

Column C0-22 – Level 8

Pure Bending

– Assume f_{s2} & f_{s3} yield, f_{s1} does not

$$f_{s1} = \frac{0.003}{c}(c-2.5)(29000)$$

$$f_{s2} = -60 \text{ ksi}$$

$$f_{s3} = -60 \text{ ksi}$$

$$P_n = 0 = 0.85(4)(24)(0.85)c + 3(0.79)\left(\frac{0.003}{c}(c-2.5)(29000)\right) + 5(0.79)(-60)$$

$$0 = 69.36c + \frac{-515.5}{c} - 30.8$$

$$0 = 69.4c^2 - 30.8c - 515.5$$

$$c = 3.0$$

verify assumptions:

$$f_{s1} = \frac{0.003}{3}(3-2.5)(29000) = 14.5 \text{ ksi} \checkmark$$

$$f_{s2} = \frac{0.003}{3}(3-12) = -0.009 < -0.00207 \checkmark$$

$$f_{s3} = \frac{0.003}{3}(3-21.5) = -0.0185$$

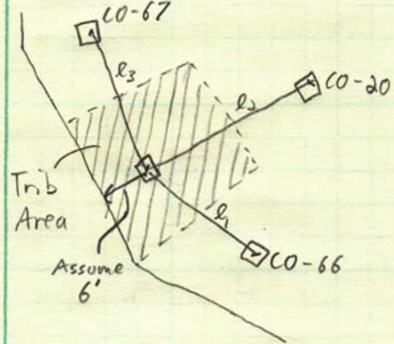
$$M_o = 0.85(4)(24)(0.85)(3.0)\left(12 - \frac{0.85 \cdot 3}{2}\right) + (3)(0.79)(14.5)(12-2.5)$$
$$+ (2)\cancel{(0.79)(60)(12}-12) + (3)(0.79)(-60)(12-21.5)$$

$$\boxed{M_o = 326 \text{ k-ft}}$$

Column Calculations

Column CO-22 Level 8

Loads



Take ½ distance to Columns 67, 66, 20 and estimate tributary area as a rectangle.

	x	y
	Location	
CO-22	-58.86	-33.99
CO-20	-42.15	-24.33
CO-66	-47.43	-47.43
CO-67	-65.18	-17.46

l_1 distance

$l_1 = \sqrt{(58.86-47.43)^2+(47.43-33.99)^2}$

$l_1 = 17.6'$

l_2 distance

$l_2 = \sqrt{(58.86-42.15)^2+(33.99-24.33)^2}$

$l_2 = 19.2'$

l_3 distance

$l_3 = \sqrt{(65.18-58.86)^2+(33.99-17.46)^2}$

$l_3 = 17.7'$

$\frac{1}{2} l_1 = 8.8'$ $\quad 8.8+8.8 = 17.6'$

$\frac{1}{2} l_2 = 9.6'$ $\quad 9.6+6 = 15.6'$

$\frac{1}{2} l_3 = 8.8'$

Trib Area $= 17.6' \cdot 15.6' = 275\ ft^2$

Superimposed DL = 13 psf

Slab DL $= 150\ pcf \cdot 8/12 = 100\ psf + 13\ psf = 113\ psf \cdot 275 = 31.1\ k$

Live Load $= 30\ psf \cdot 275 = 8.25\ k$

Roof Snow Load $= 19.25\ psf \cdot 275 = 5.29\ k$

$P_u = 1.2(31.1k) + 0.5(8.25) + 0.5(5.29) = \boxed{44.1\ k}$

$w_{DL} = 113\ psf \cdot 15.6' = 1763\ plf$

$w_{LL} = 30\ psf \cdot 15.6' = 468\ plf$

$w_{SL} = 19.25\ psf \cdot 15.6' = 300\ plf$

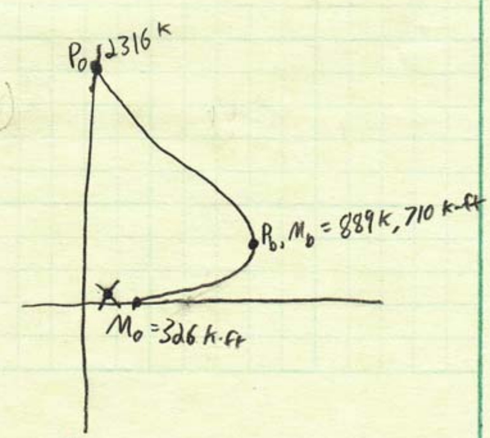
$w_u = 1.2(1763) + 0.5(468) + 0.5(300)$

$w_u = 2.5\ klf$

$M_u = \frac{w_u l^2}{12} + Pe = \frac{(2.5)(17.6)^2}{12} + (44.1)(2.8)$

$\boxed{M_u = 187.5\ k\text{-}ft}$

Interaction Diagram

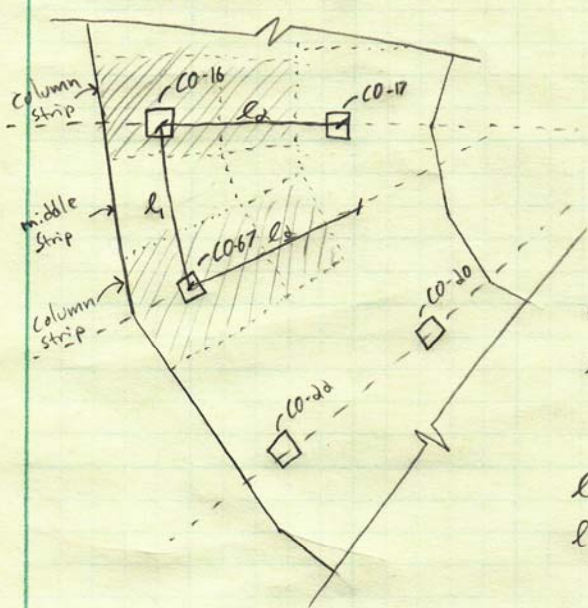


Column within Interaction Diagram curve

Typical Two-Way Slab Calculation

Level 5 - Southwest Quadrant

- 8" Flat Plate



	Location (ft) (X)	(Y)
CO-16	-78.83	0
CO-17	-49.92	0
CO-67	-65.18	-17.46

ℓ_1 distance

$$\ell_1 = \sqrt{(65.18' - 78.83')^2 + (17.46' - 0')^2}$$

$$\ell_1 = 22.16'$$

ℓ_2 distance

$$\ell_2 = 78.83' - 49.92$$

$$\ell_2 = 28.91'$$

★ See spSlab computer data for remainder of two-way slab spot check.

10 | Scott Molongoski | Tech One | Snow Load Calcs

Snow Load Calculations

Based on ASCE 7-10

$p_f = 0.7 C_e C_t I_s p_g$

$C_e = 1.0$ (Partially exposed, category B)

$C_t = 1.0$

$I_s = 1.10$

$p_g = 25$ psf

$p_f = 0.7(1.0)(1.0)(1.10)(25)$

$p_f = 19.25$ psf

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 ft^3

A_{og} = total area of openings = 7462 ft^3 (includes roof & wall slots)

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

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

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.

12 | Scott Molongoski | Tech One | Seismic Calcs

Seismic Load Calculations

- Based on ASCE 7-10
- From USGS website: $S_S = 0.130g$ $S_{MS} = 0.15g$ $S_{DS} = 0.104g$
  U.S. Seismic Design Maps $S_1 = 0.052g$ $S_{M1} = 0.088g$ $S_{D1} = 0.059g$
- 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. Seismic Design Category B will be used for the remainder of the calculations.
- Assume that the building has ordinary reinforced concrete shear walls ∴
  - Response Modification Coefficient, R = 5, per Table 12.2-1
  - Importance Factor = 1.25, per Table 1.5-2
  - No height limitations per Table 12.2-1

- $T_L = 6$ sec per Fig 22-12
- $C_u = 1.7$ per Table 12.8-1
- $C_t = 0.016$ per Table 12.8-2
- $x = 0.9$ per Table 12.8-2
- Height = 113 ft

$$T_a = C_t h_n^x = 0.016(113)^{0.9} = 1.13$$

$$T = C_u \cdot T_a = 1.7 \cdot 1.13 = 1.92$$

$$C_s \le \begin{cases} S_{DS}/(R/I) = 0.104/(5/1.25) = 0.026 \\ S_{D1}/[T(R/I)] = 0.059/[1.92(5/1.25)] = 0.007 \leftarrow C_s = 0.007 \\ \dfrac{S_{D1} \cdot T_L}{T^2(R/I)} = \dfrac{(0.059)(6)}{(1.92)^2(5/1.25)} = 0.024 \end{cases}$$

For each story, the weight of the 150 pcf 8" concrete slab was taken for the entire floor area 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 B: Wind and Seismic Tables

Wind Tables:

Table A-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 A-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 and Seismic Tables

Wind Table:

Table A-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 A-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 and Seismic Tables

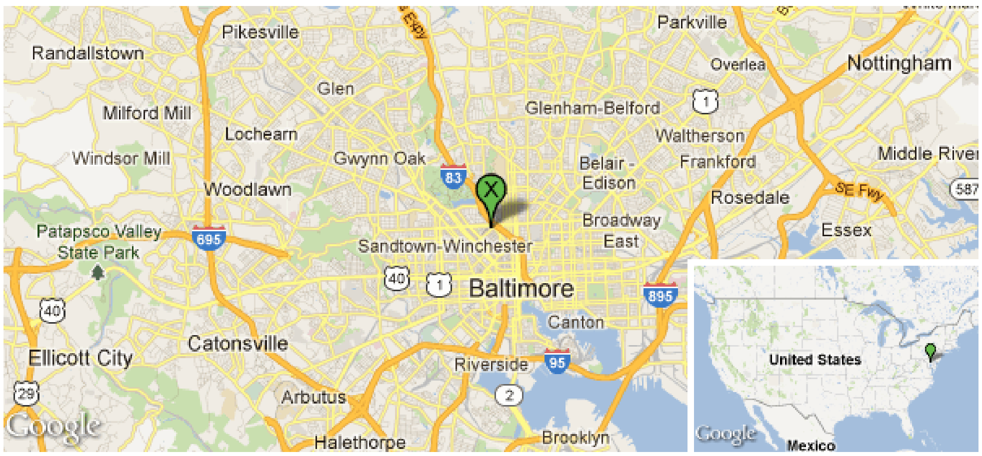
Seismic Design Information from USGS:

USGS Design Maps Summary Report

Print   View Detailed Report

User–Specified Input

Building Code Reference Document	2012 International Building Code (which makes use of 2008 USGS hazard data)
Site Coordinates	39.31°N, 76.625°W
Site Soil Classification	Site Class C – "Very Dense Soil and Soft Rock"
Risk Category	I/II/III

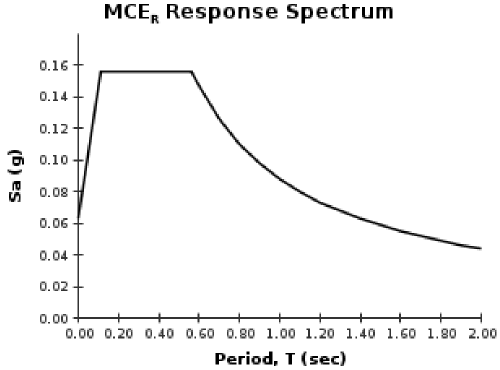


USGS–Provided Output

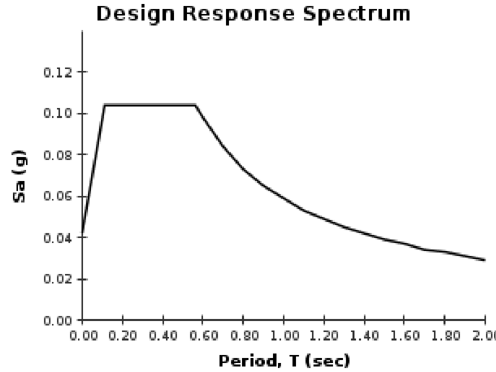
S_S = 0.130 g	S_{MS} = 0.156 g	S_{DS} = 0.104 g
S_1 = 0.052 g	S_{M1} = 0.088 g	S_{D1} = 0.059 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.

MCE_R Response Spectrum



Design Response Spectrum



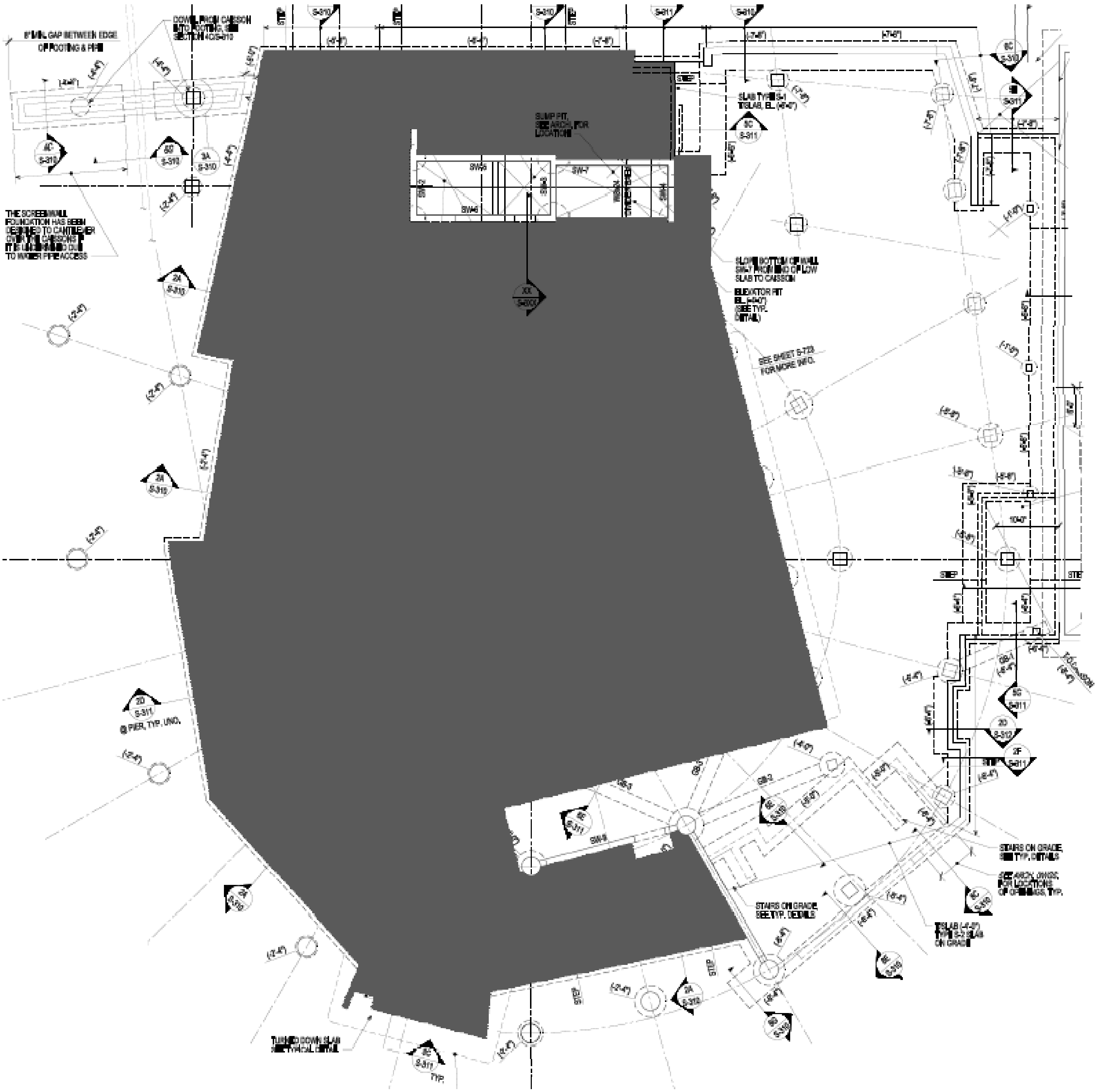
Appendix B: Wind and Seismic Tables

Seismic Table:

Table A-B.3

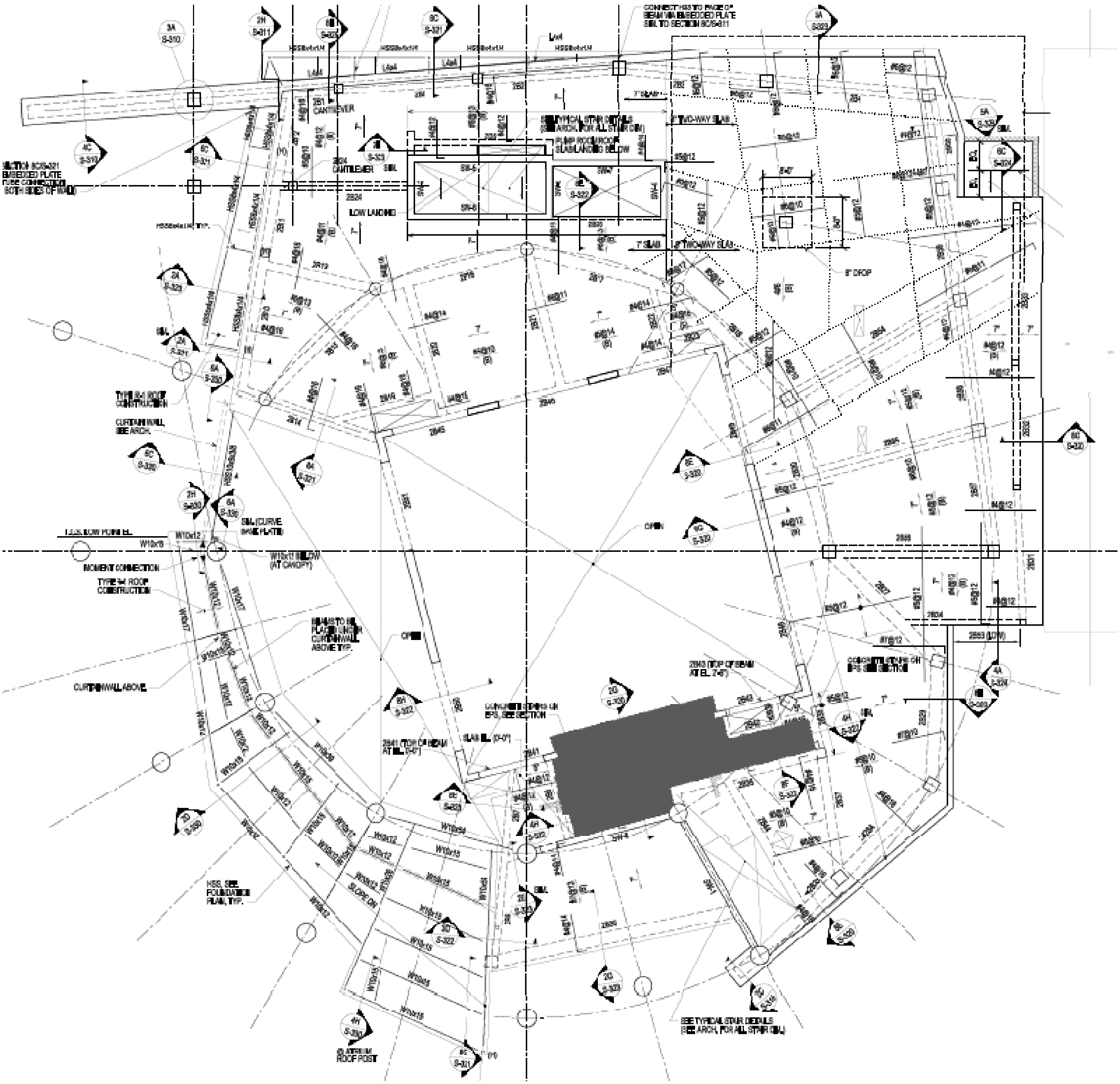
Seismic Loads				
Story	**Story Weight (k)**	Height (ft)	C_{vx}	**Story Force (k)**
2	**1258**	14	0.0022	**0.21**
3	**1958**	27	0.0156	**1.55**
4	**1563**	41	0.0331	**3.29**
5	**1575**	51	0.0556	**5.53**
6	**1520**	61	0.0815	**8.11**
7	**1613**	72	0.1275	**12.68**
8	**1643**	82	0.1761	**17.51**
9	**1643**	92	0.2305	**22.92**
10	**1073**	103	0.1961	**19.50**
Penthouse	**361**	113	0.0819	**8.15**
Total	**14207**	Overturning Moment		**8342.99**
Base Shear	**99.45**			

Appendix C: Structural Plans



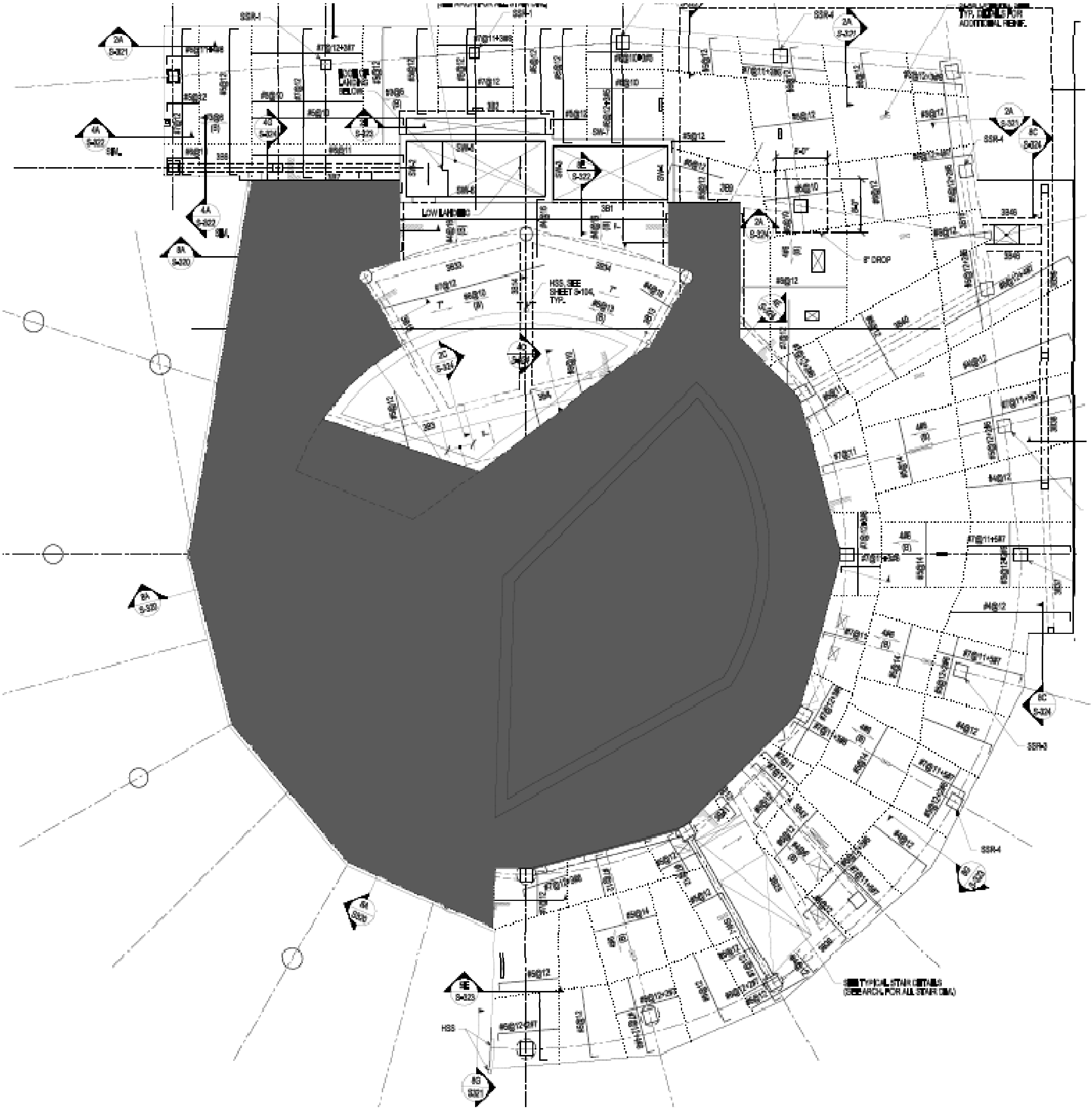
Level 1 Framing Plan– shaded area represents a depressed floor slab

Appendix C: Structural Plans



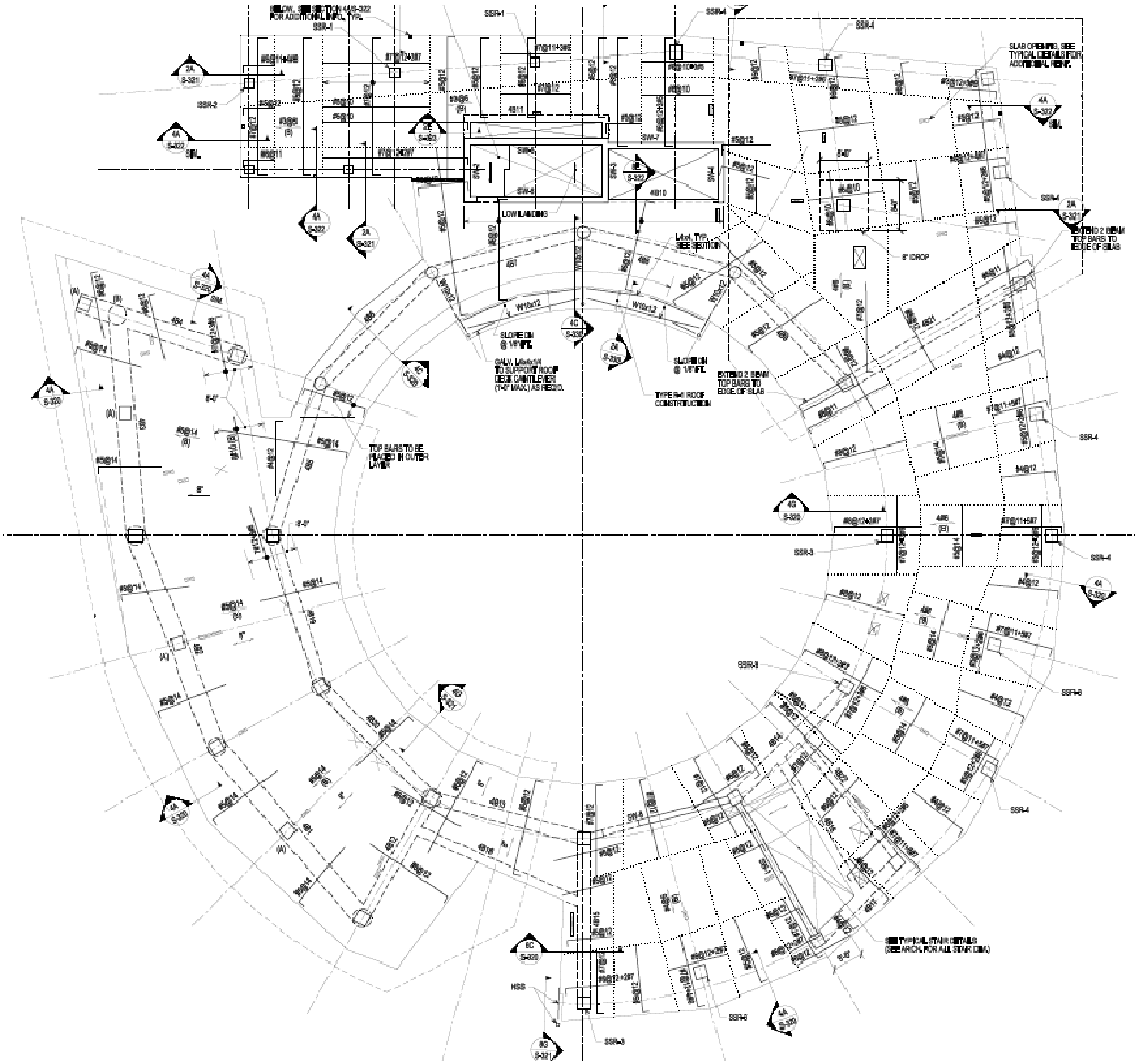
Level 2 Framing Plan

Appendix C: Structural Plans



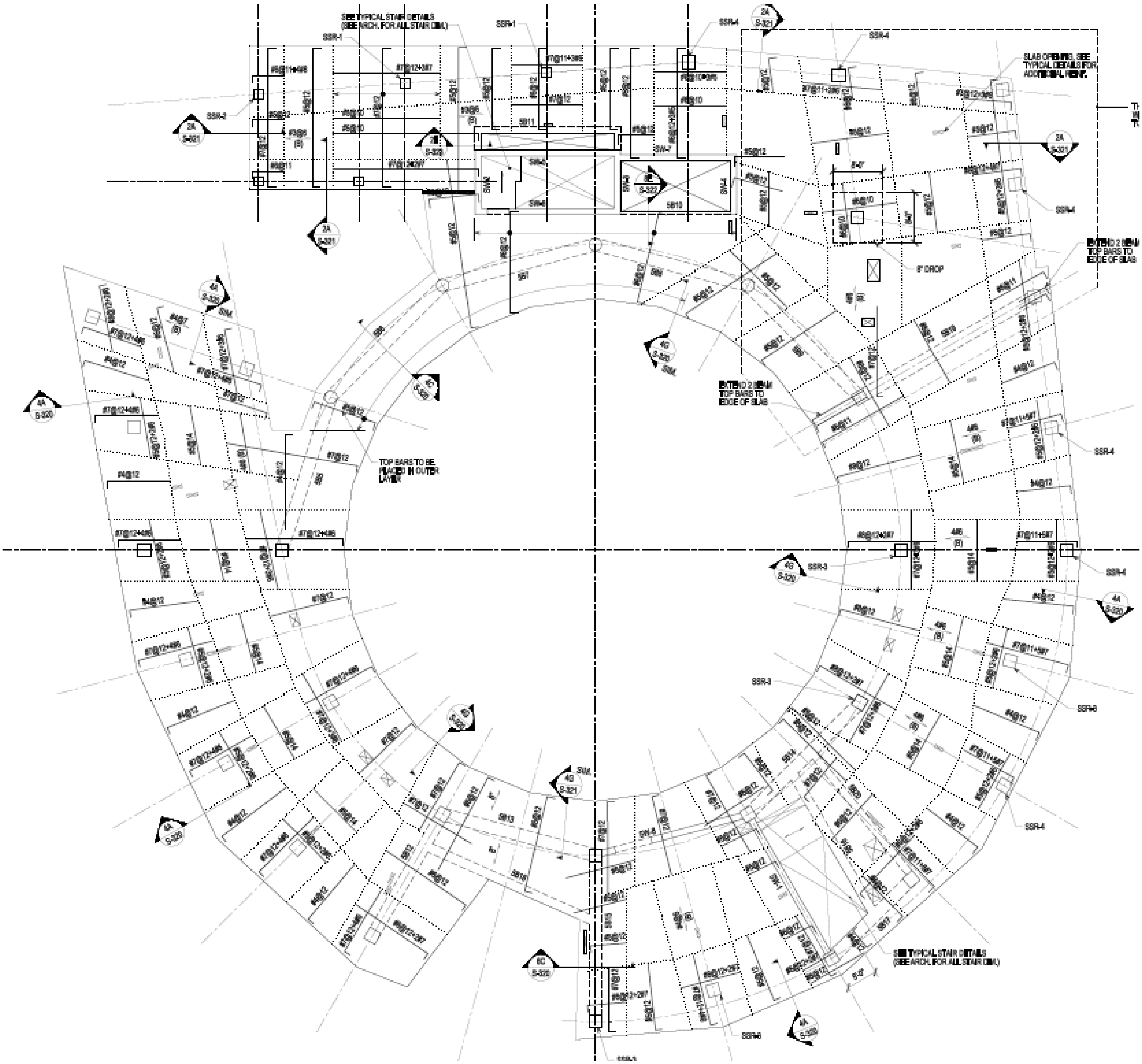
Level 3 Framing Plan– shaded area represents a depressed floor slab

Appendix C: Structural Plans



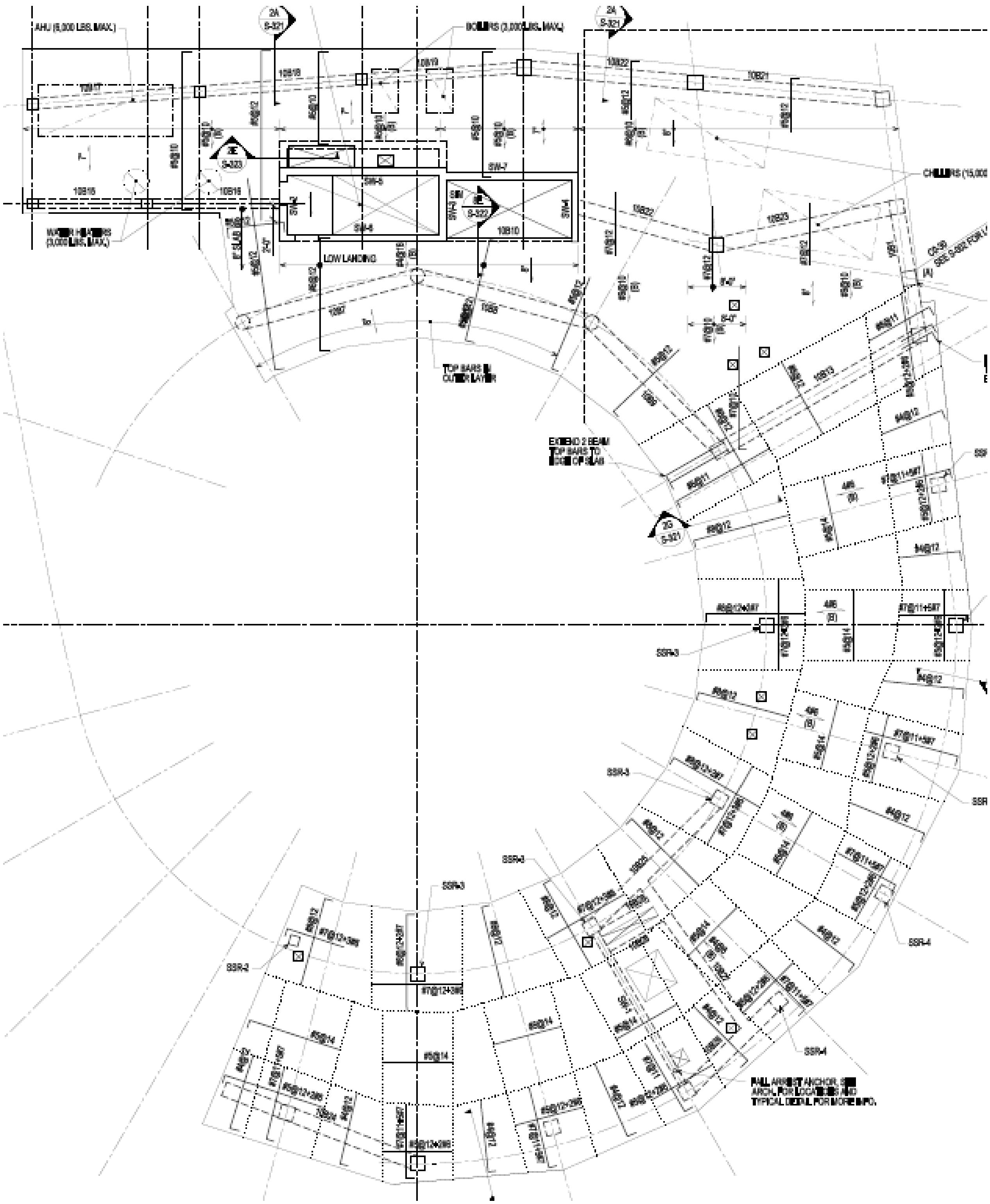
Level 4 Framing Plan

Appendix C: Structural Plans



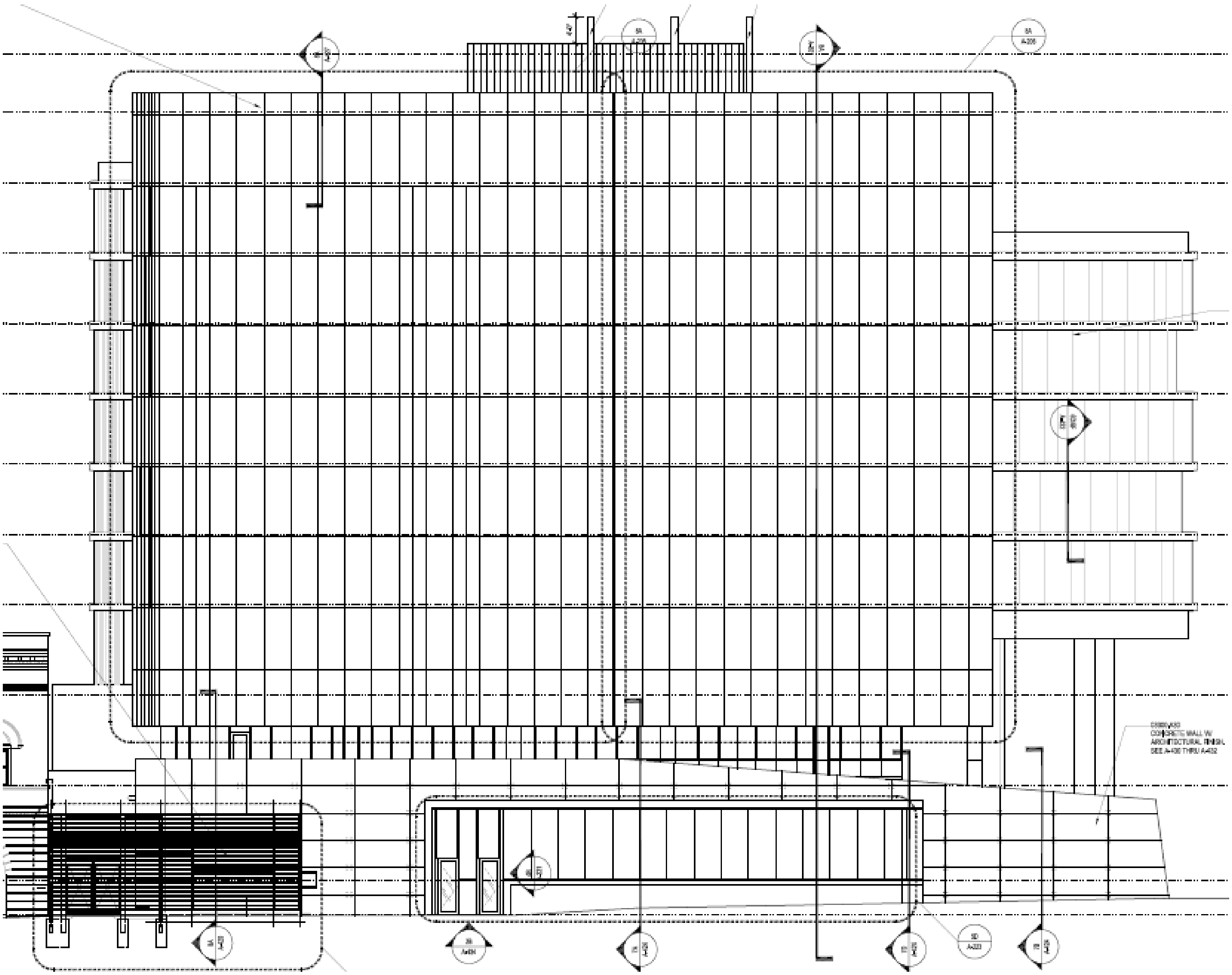
Level 5-9 Framing Plan

Appendix C: Structural Plans



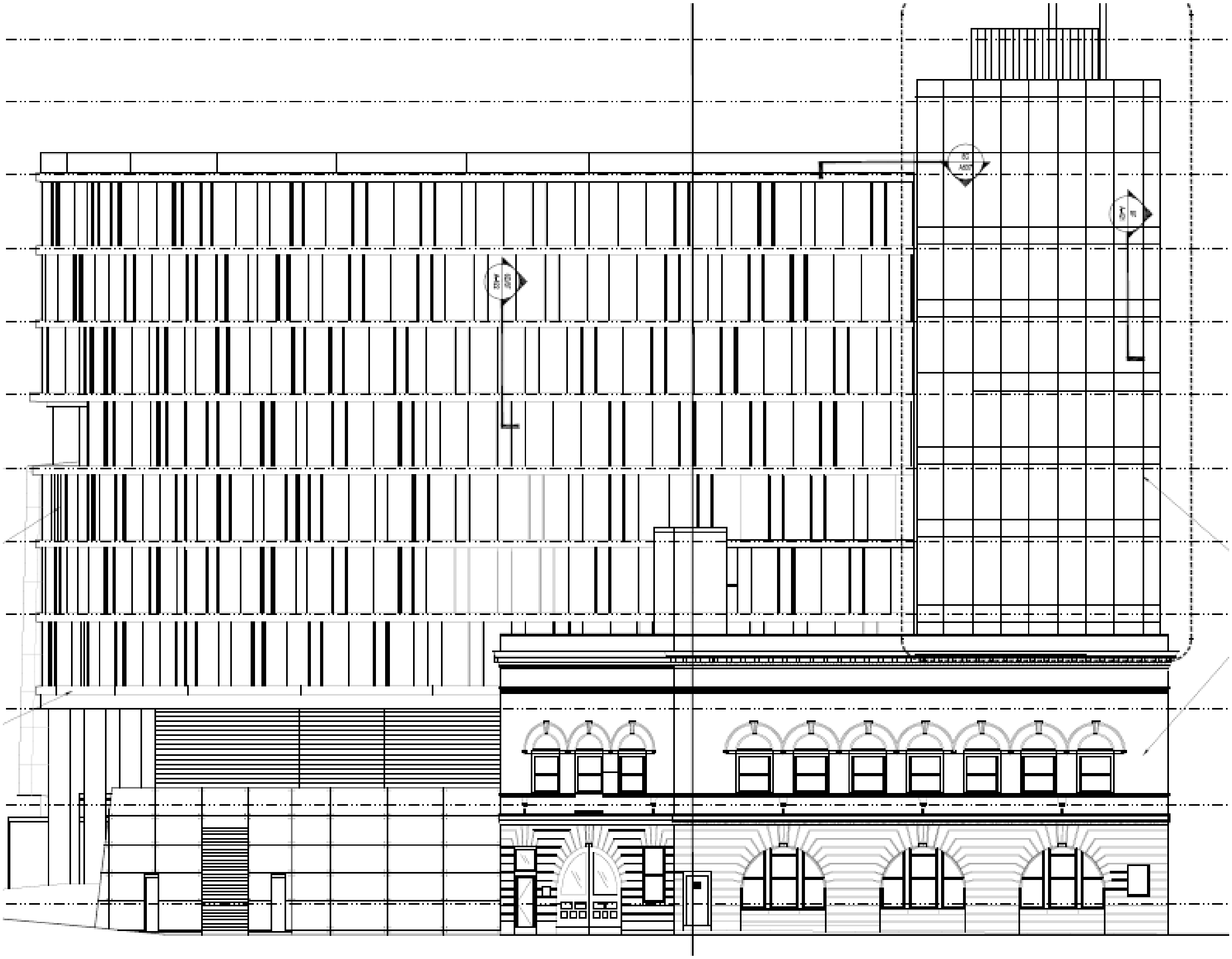
Level 10 Roof Framing Plan

Appendix C: Structural Plans



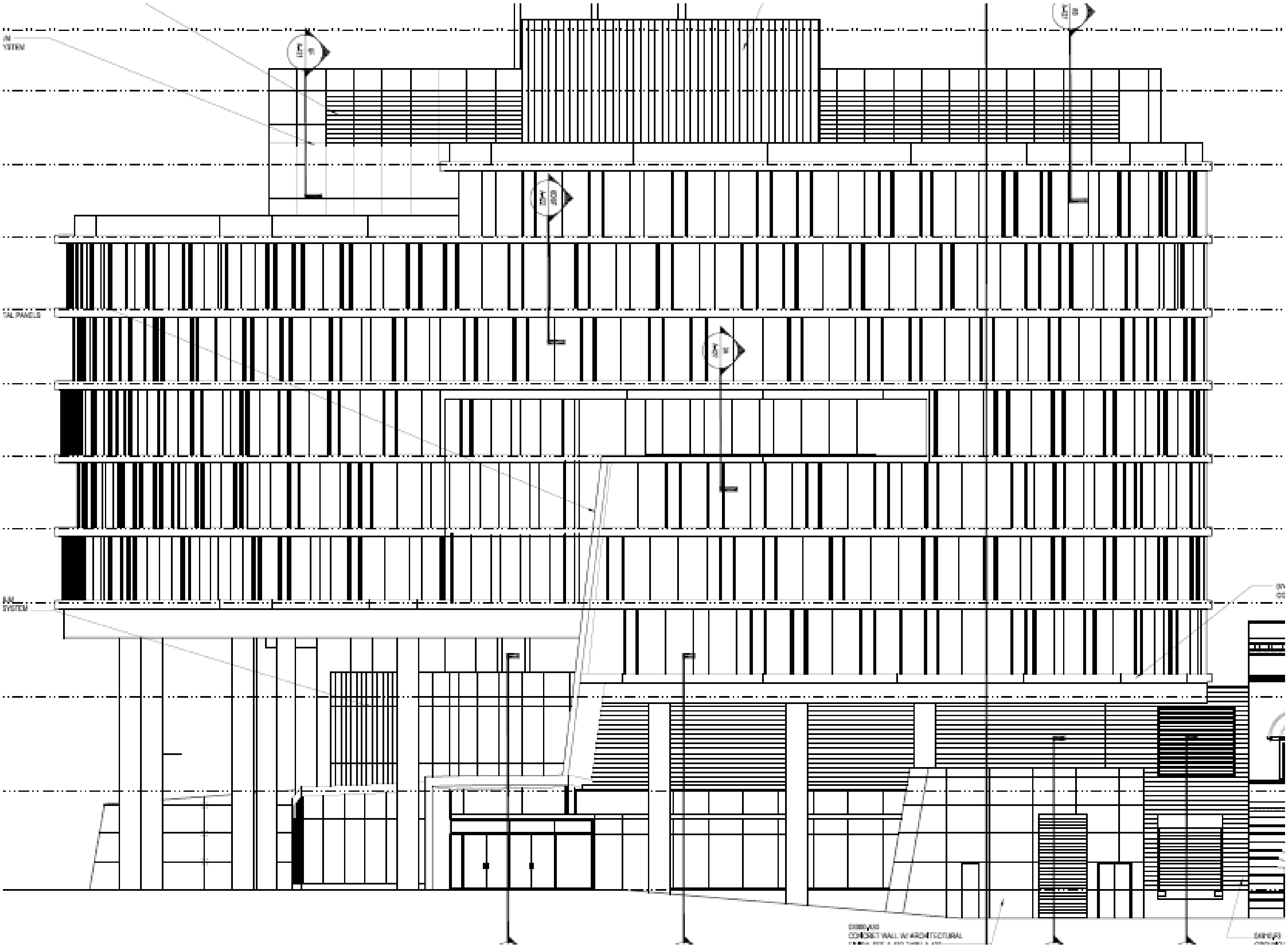
North Building Elevation

Appendix C: Structural Plans



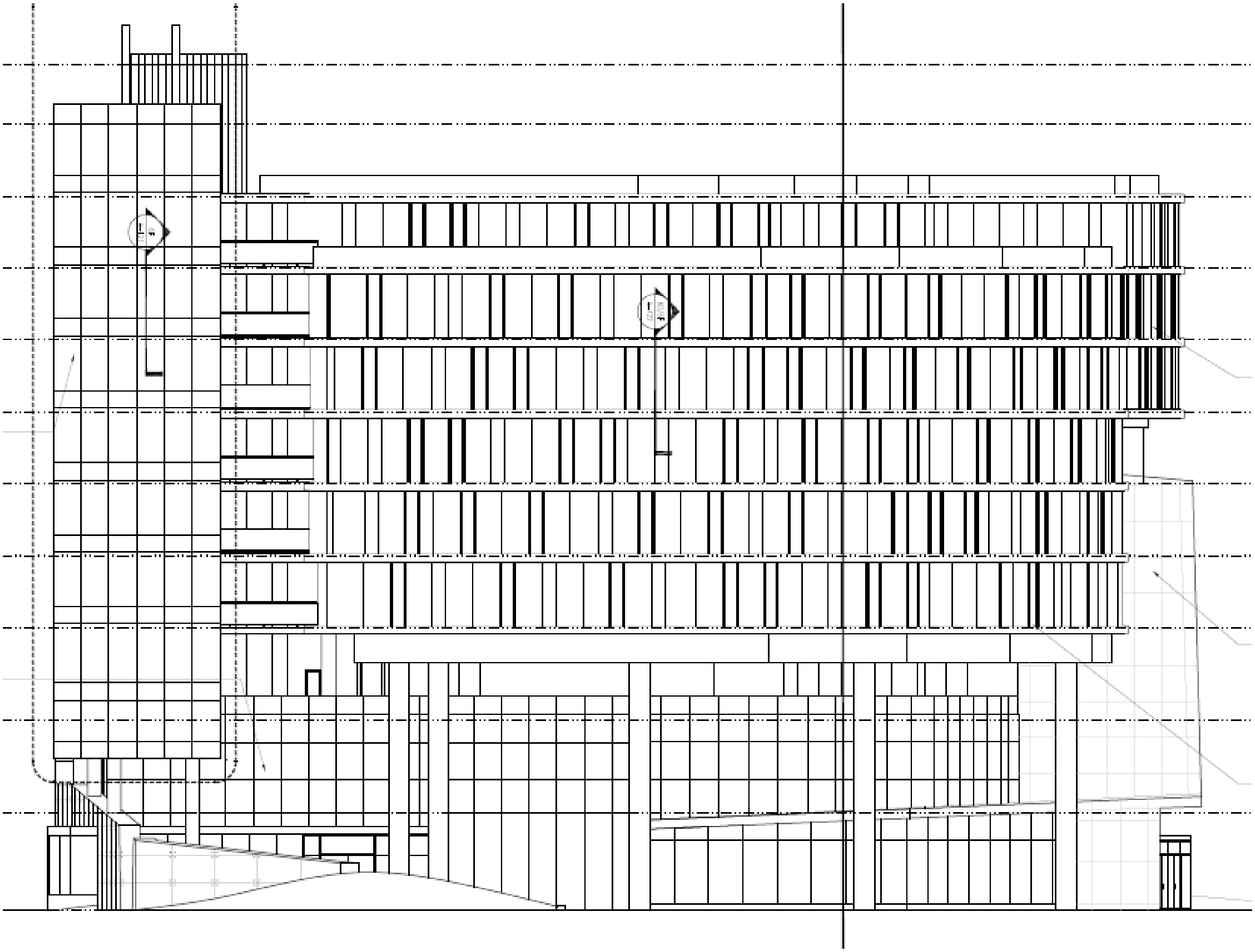
East Building Elevation

Appendix C: Structural Plans



South Building Elevation

Appendix C: Structural Plans



West Building Elevation