Technical Report II

MICA Gateway Residence

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Structural Option ~ Heather Sustersic ~ October 12, 2012

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Executive Summary:

Technical Report II of the Senior Thesis Project is an analysis of the existing floor framing system and a preliminary design of three alternative systems. This report includes an explanation of building codes, materials, and gravity loads and how they apply to the building structure. The existing floor system and three alternative designs were analyzed and then compared to determine the feasibility of each system.

The MICA Gateway Residence has a primarily concrete structure. The existing floor framing system has two-way flat plate concrete slabs, 8" thick. A flat plate system was chosen to maximize the architectural space between floors and also to better accommodate the irregular geometry of the building. The existing floor framing plan was approximated into a rectangular area and then analyzed using the equivalent frame method. Only gravity load was taken into account in this analysis.

Three alternate systems were designed, with framing members and decks/slabs sized and checked for strength and serviceability. The three alternate systems designed were a non-composite steel deck on steel beams and girders, a composite steel deck with lightweight concrete, and a one-way slab on concrete beams. Each system was chosen due to its unique design aspects and the differences between them. In each design, only the factored dead and live loads were taken into account. Lateral loads were not included in the designs.

Each floor framing design was then compared in a variety of different design categories, including slab/system depth, system weight, deflection, cost, formwork, fire protection and alterations to the lateral and foundation systems. RS Means data for assemblies was used to determine the square foot costs of each framing system. Based on these factors the feasibility of each system was determined. The results show that only the existing two-way flat plate concrete slab system is feasible in the Gateway. System depth and cost were the primary factors in arriving at this conclusion.

The appendices of this report include the hand calculations performed for each of the floor framing systems, the RS Means assembly cost estimates for each system, and the structural plans of each floor as well as building elevations.

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 was explored in Technical Report I. Beyond the multipurpose assembly room, Gateway features 64 student apartments, several art galleries and studios, and a café.

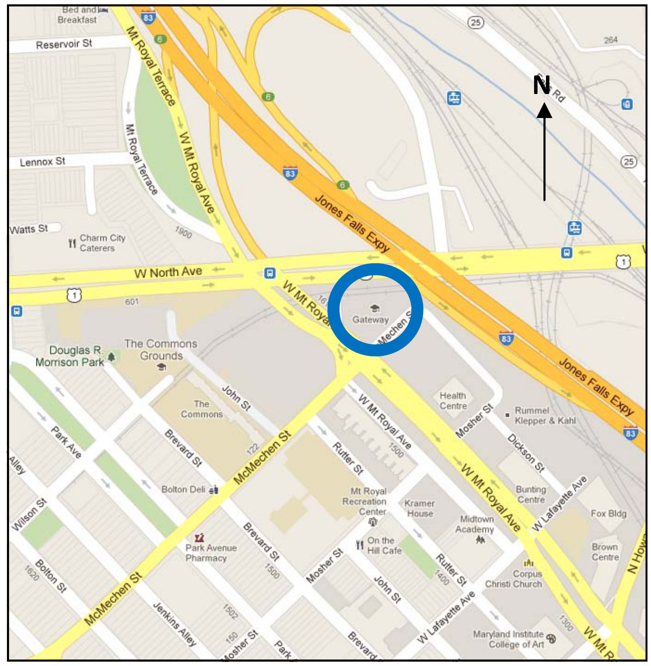


Figure 1: Gateway location in Baltimore. Courtesy of Google

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.

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.

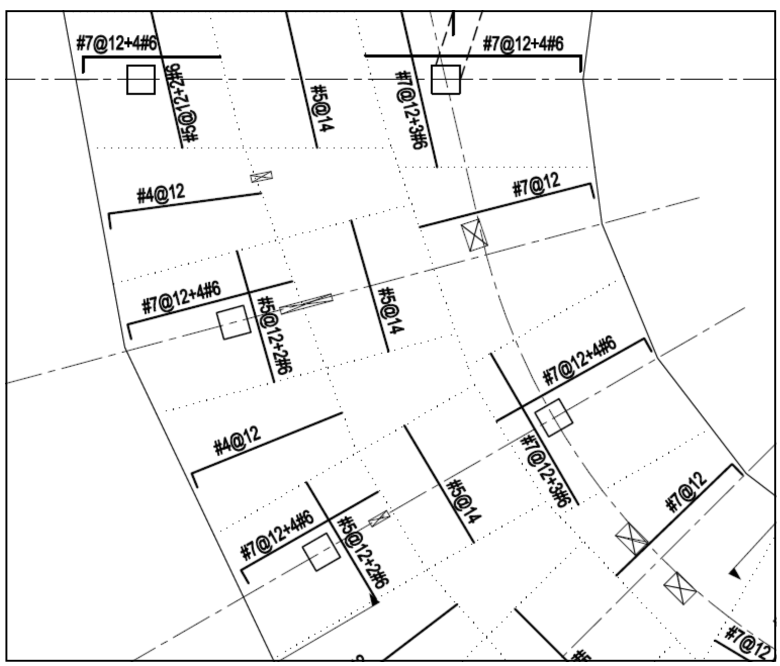


Figure 2: Typical two-way flat plate slab. Courtesy of RTKL

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.

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.

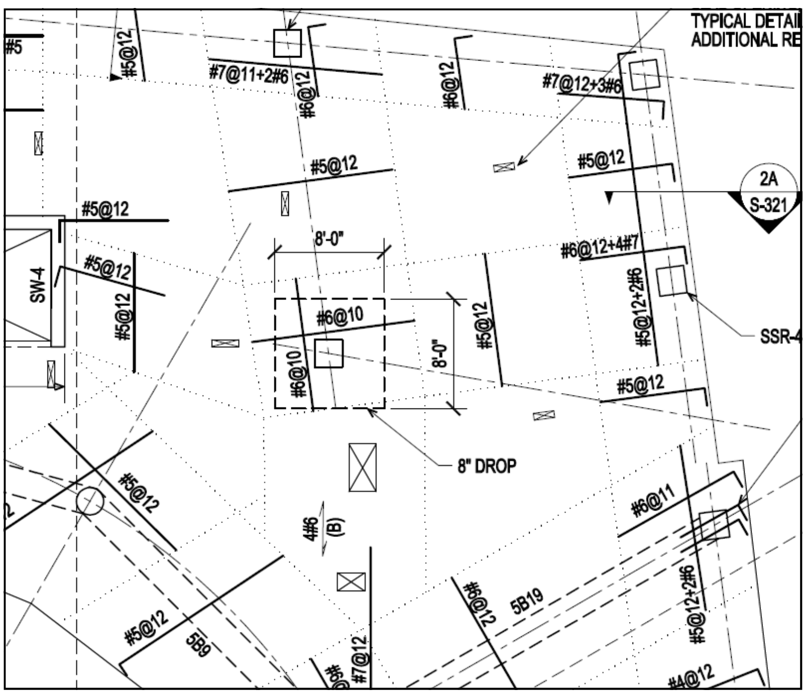


Figure 3: Two-way slab and drop panel around CO-7. Courtesy of RTKL

Floor Framing System Analysis:

Four different types of floor framing systems were analyzed and compared to determine which one of them was the most efficient in terms of design, cost, and constructability. The current two-way flat plate slab was analyzed, as well as three alternate systems; a non-composite deck with concrete topping on steel beams; a composite deck with lightweight concrete topping; and a one-way slab. Each of these systems has it's own unique advantages and disadvantages as described in the following pages.

All four systems were analyzed for a section of the framing plan on Level 5 of the structure. The area used for the analysis is outlined in Figure 4 below. The distance between the columns on the exterior (left) ring is 22' while the distance between the columns on the interior (right) ring is 25'. The distance between the two column rings is 22', the exterior cantilever is 6 ' wide and the interior cantilever is 10' wide. All dimensions are approximate. Only gravity loads were considered in the design of the floor members and in all cases were approximated as rectangular areas. All calculations are found in Appendix A.

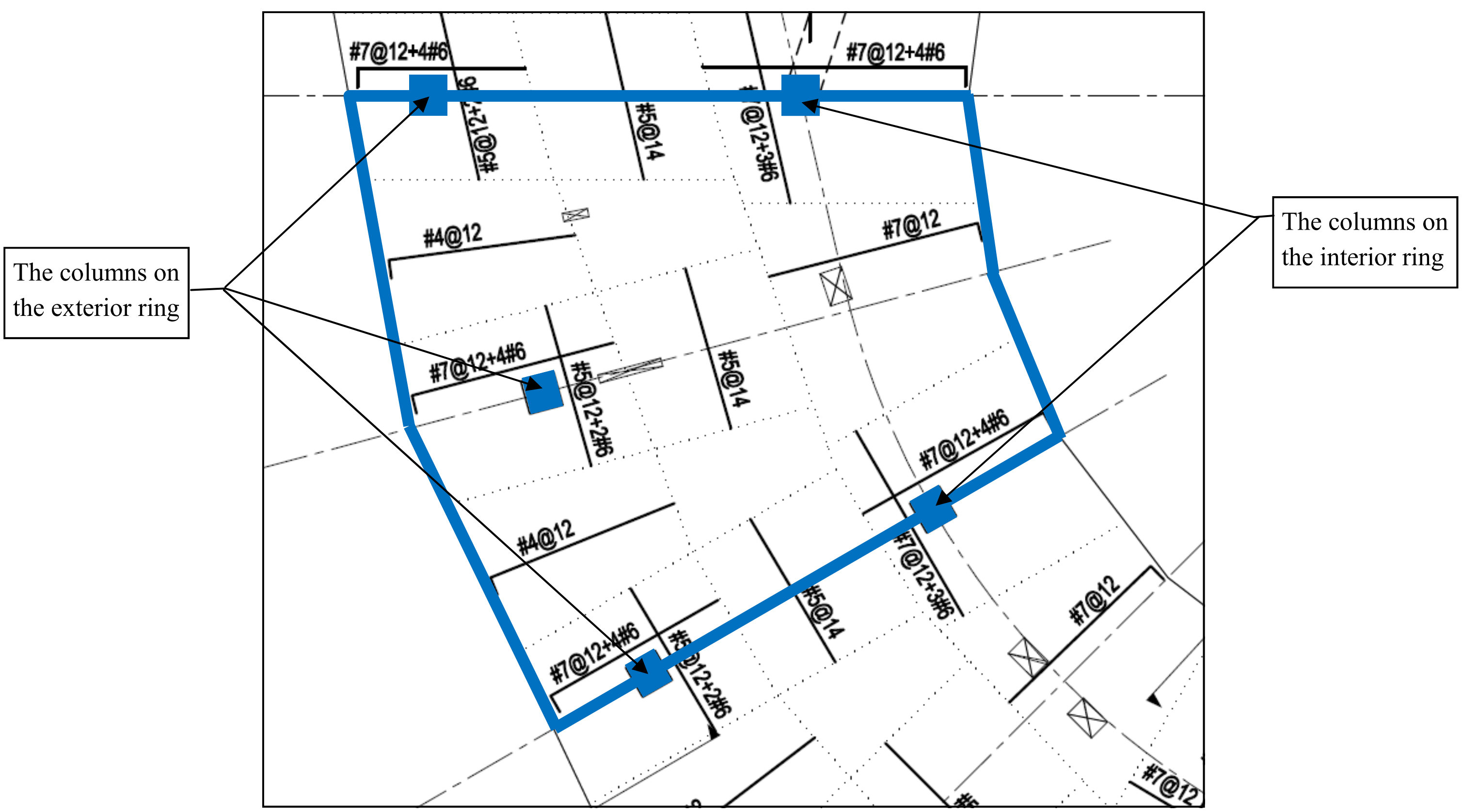


Figure 4: The area of analysis for all floor systems. Courtesy of RTKL

Existing System: Two-Way Flat Plate:

Description:

The existing floor framing system features a 8” two-way flat plate slab with a uniform thickness. This slab has a #5@12” each way continuous bottom mat reinforcement with appropriate top reinforcement called out on the drawings. Figure 5 shows the typical reinforcement layout in the two-way flat plate. For top reinforcement the middle strips feature #4@12” in the North South direction and #5@14” in the East West direction. In the column strips there are #7@12” in the North South direction and #5@12” in the East West direction. Around the columns there are an additional 4#6 in the North South direction and 2#6 in the East West direction to resist the greater negative moments and punching shear.

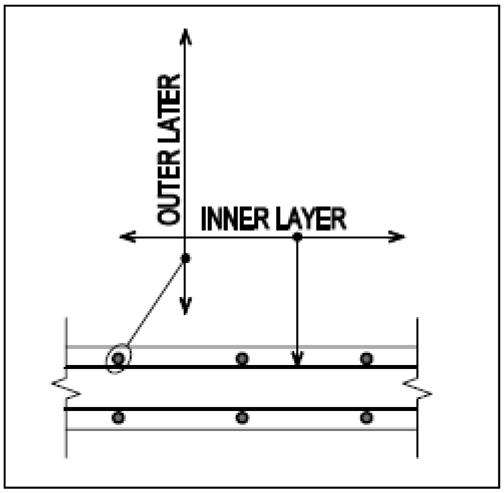


Figure 5: Rebar layering in two-way flat plate. Courtesy of RTKL

Advantages:

Two-way flat plate slabs are the thinnest floor systems available. The Gateway features 10’ floor to floor ceiling heights so an 8” thick flat plate provides the needed clearance. A flat plate replaces beams and drop panels that would take up space between floors. This system also features the advantage of being an all-concrete structure which requires no additional fireproofing. A two-way flat plate works well for areas with irregular bays in that no beams need be designed to span odd spaces.

Disadvantages:

As a concrete system a flat plate is heavier than a steel system, requiring larger column sizes and more robust foundations to accommodate the large dead loads of concrete construction. A concrete system also requires time to pour and set which increases the construction time of the structure. To pour concrete, formwork is required, creating more expenses.

Non-Composite Steel Deck on Steel Beams and Girders:

Description:

This system features 1.5C18 Non-composite deck with a 5.5” depth, including the 4” concrete topping. The deck runs perpendicular to the beams that support it. The beams used in this system are W12x16’s set at 5’-6” spacing. They span the two column rings and support the cantilevers as shown in red in Figure 6. Girders are W21x50’s and are shown in blue.

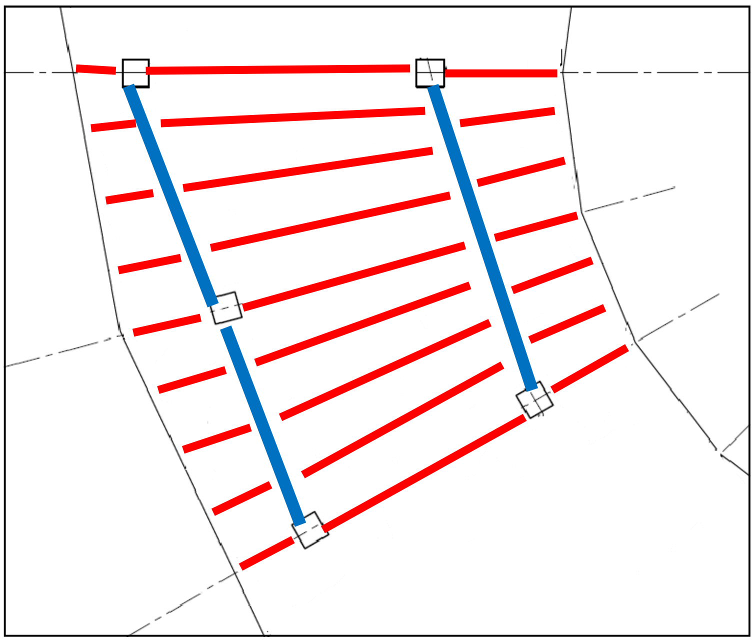


Figure 6: Beam and girder layout for non-composite system. Courtesy of RTKL

Advantages:

A non-composite system is lightweight, and requires less concrete in the deck/slab than a completely concrete slab. Due to this lightweight system, steel beams and girders tend to be smaller, thus creating additional cost savings. The steel decking also provides a formwork for the concrete slab, thus eliminating the need for other formwork.

Disadvantages:

Non-composite systems, while featuring a thinner deck/slab, require steel members which are inherently deep. In this case, a 21” deep girder significantly cuts into the floor to floor height. Steel deck, beams and girders also require some kind of additional fireproofing. The cost of labor for skilled welders to assemble the steel members is also a draw back of a non-composite steel deck system.

Composite Steel Deck with Lightweight Concrete:

Description:

A composite steel deck with lightweight concrete was designed with a 1.5VLR16 composite steel deck. This deck is 4” deep including a 2.5” lightweight concrete topping. The deck spans perpendicular to the beams, which are spaced at 11’. They are W12x19’s and are colored red in Figure 7. The girders are W16x26’s and are colored blue in Figure 7. Lightweight concrete was chosen so as to decrease the load, thus decreasing member size. Each beam and girder is attached to the deck with shear studs that were determined in the calculations.

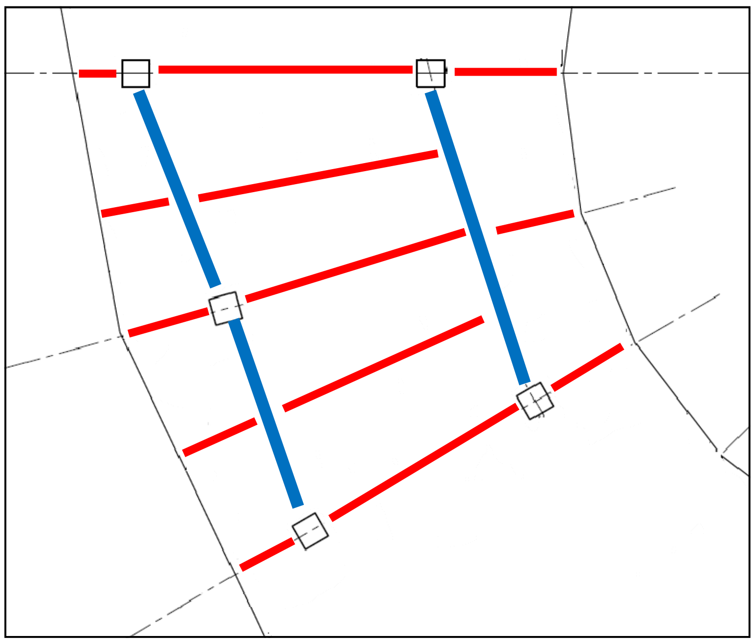


Figure 7: Beam and girder layout for composite system. Courtesy of RTKL

Advantages:

A composite deck has the advantage of keeping the concrete slab in compression while the steel members take all the tensile forces, thus efficiently utilizing both materials. Using lightweight concrete keeps the weight of the floor system down, in turn decreasing the member sizes. The strength of the composite system also allows for greater beam spacing, reducing the number of steel members used. Formwork is not needed either, providing another distinct advantage for a composite system.

Disadvantages:

Unfortunately a composite system still requires steel members which tend to be significantly deeper than any two-way flat plate or slab system. Steel members require fireproofing and skilled labor to erect and weld them in place. Lightweight concrete is also more expensive than normal weight.

One Way Concrete Slab on Beams:

Description:

The one way slab design features a 10" deep slab that spans the distance between the two column lines. In the slab #5@12" bottom bars provide the reinforcement across the 22' long span. Concrete beams span the distances between the columns parallel to the building exterior. Theses beams are 24" wide by 16" deep and have (4)#9 bottom bars as positive midspan reinforcement, (3)#10 and (2)#9 top bars for negative moment reinforcement at the beam ends. The beams also have (2)#4@5" spacing for shear reinforcement. Slab direction is indicated by the arrows on Figure 8 and the beams are drawn in blue. This design was deflection controlled and required two iterations after a beam depth of 15" failed to meet deflection criteria.

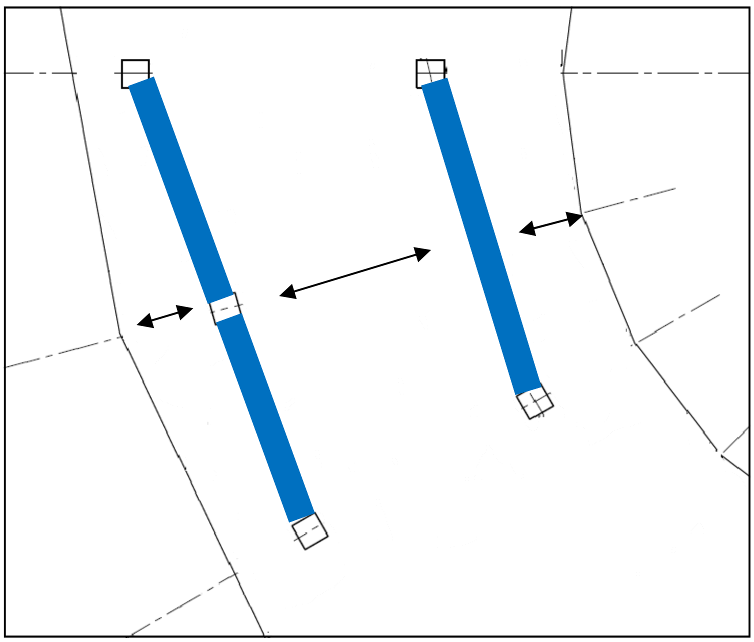


Figure 8: One way slab on beams layout. Courtesy of RTKL

Advantages:

A one-way slab system allows for a large spacing in a building, decreasing the amount of members obstructing the space below. For a building such as this with only two column rings, a one way slab makes sense because only one span needs to be accounted for. A concrete system always has the advantage of inherent fireproofing.

Disadvantages:

A one way slab on beams has a deeper system height than a flat plate. The system is also heavier and requires formwork to pour the concrete. There is also an increased construction time due to the time need to allow for the concrete to set and cure.

Floor Framing System Comparison:

The following table compares the existing and alternative floor framing systems in a variety of design categories. Cost data was determined through RS Means Cost Book by using approximate assemblies to the floor systems analyzed. The actual tables used to determine the system cost and the description of the various differences between the system analyzed and the assembly data are detailed in Appendix B of this report. Each of the design categories listed in the table are expanded upon in the following pages.

Floor Framing System Comparison				
Design Category	Existing Two-Way Flat Plate Slab	Non-Composite Steel Deck on Steel Beams and	Composite Steel Deck with Light-weight Concrete	One-Way Concrete Slab on Beams
Slab Depth	8”	5.5”	4”	10”
System Depth	8”	26.5”	20”	16”
System Weight	109 psf	77 psf	48 psf	334 psf
Deflection	0.12” (slab deflection)	1.22”	0.755”	1.14”
Fire Protection	Inherent	Spray-On	Spray-On	Inherent
Formwork	Yes	No	No	Yes
Lateral System Alterations	No	Yes	Yes	No
Foundation Alterations	No	Yes	Yes	No
System Cost	$14.69/sf	$22.99/sf	$17.86/sf	$17.14/sf
Feasibility	Yes	No	No	No

Slab/System Depth

All alternative systems proved to be too deep for the floor-to-floor height and intended architectural design of the structure. With only 10' between each floor, both steel systems left less than 8.5' of clearance and the one-way slab left less than 9'. The 8" flat plate slab proved to be the most feasible in this case due to its uniform thickness and unobtrusiveness into the architectural space.

System Weight

The steel deck framing systems were lighter than either concrete system, with the composite deck with lightweight concrete being by far the lightest at 48 psf. The problem with having such a light framing system is that it can be prone to vibration issues. The flat plate was a reasonable weight while the one-way slab was very heavy due to the thick slab and large beam sizes.

Deflection

The flat plate slab had a very small deflection at only 0.12". This most likely means that the existing system is very stiff. The steel framing systems had larger deflections likely due to their lighter construction. The one-way slab had the largest deflection because of the long distance (23') that the beam was expected to span. Deflection limits forced a deepening of the beam to fall below the code maximum.

Fire Protection

The two all concrete systems have an inherent fire protection equivalent to a 2-hr fire rating, which is the code minimum. The composite and non-composite steel decks would require additional spray-on fireproofing to reach the 2-hr code minimum.

Formwork

Both the existing system and the one-way slab alternative would require formwork in order to be placed. The formwork would be more extensive for the one-way slab due to the presence of beams, whereas the current flat plate would require formwork for a slab only. The two steel deck systems would not need formwork because the steel deck already acts as the formwork for laying the slab.

Lateral System Alterations

For the existing system no lateral alterations would be necessary. The one-way slab alternative utilizes the same columns as the existing system and would likewise not need any lateral alterations. The steel deck systems would more than likely frame into steel columns, instead of the concrete ones assumed in this analysis. Steel columns would require bracing of some kind to resist the lateral forces on the structure.

Foundation Alterations

The two-way flat plate and the one-way slab would utilize the same columns currently in the plans and thus would not see any significant change in the foundation structure. Since the composite and non-composite steel deck are both lighter structures, the building weight would be lower and thus might warrant a foundation redesign toward a less robust system.

System Cost

Based on RS Means assembly cost data, square foot estimates were obtained for approximate systems to the ones designed in this report. The differences between the two are detailed in Appendix B. The cheapest system was the existing two-way flat plate at $14.69 per square foot. The composite steel system and the one-way slab were more expensive at around $17-$18 per square foot. The most expensive system at $22.99, making it impractical when compared to the other systems.

Feasibility

Based on the intended architectural design of the building and the irregular floor plans the only truly feasible system is the existing two-way flat plate slab. The other systems are too deep and obtrusive to be considered in the design of the Gateway floor framing system. The irregular shape of the floor plan would be more difficult to design conventional steel or concrete beams on due to the changing angles around the building and the presence of numerous cantilevers. A two-way flat plate allows the contractors to build the precise formwork needed for each floor area, pour the concrete and then move on.

Conclusion:

Upon analysis of the calculations and comparison chart it is apparent that the existing two-way flat plate concrete slab system is the most efficient and feasible. The current system has the smallest system depth and the lowest cost of all the systems designed and analyzed in Technical Report II. These factors along with the architectural considerations of maximizing the spaces and creating a modern and progressive aesthetic make the original design decisions easy to understand.

However the other systems also provide some advantages over the existing system. Both the composite and non-composite systems are lighter than the current design which would lead to an overall decrease in building weight which could save money in the foundation design. They are also reasonably cheap compared to a potential one-way slab design. If floor-to-floor height were not such a pertinent design consideration in the Gateway, the composite steel deck would be a feasible alternative to the current system.

The least feasible alternative was the one-way concrete slab. Deflection control was a challenge in the design of the beams supporting the one-way slab, barely passing the deflection limit. The system was also far heavier than any of the others and much more expensive.

Other systems not analyzed in this report may still be feasible. A two-way flat slab with drop panels would likely not be much deeper than the current 8” flat plate. One-way slabs with pan joists and beams may also be a feasible option. Due to the height limitations between floors, it is unlikely that any steel framing design would be acceptable.

Technical Report II included the preliminary design and analysis of four different floor framing systems. While only gravity loads were considered, future analysis will reveal the design decisions made based on lateral loading, as well as potential gravity and lateral system redesigns based on a more complete analysis of the Gateway.

Appendices:

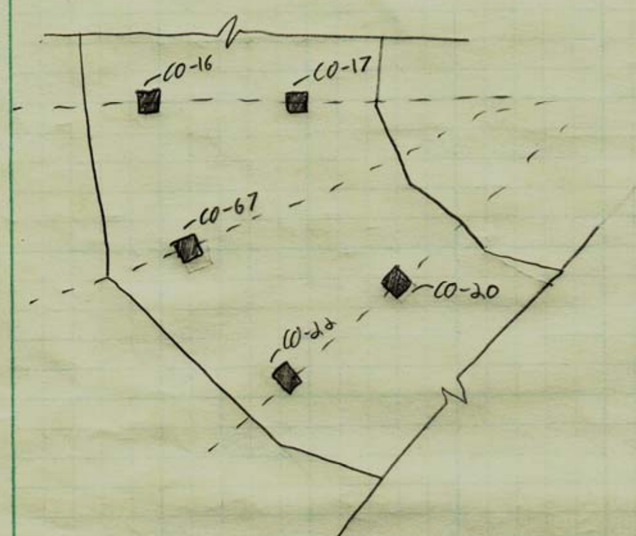
Appendix A: Hand Calculations

1 | Scott Molongoski | Tech Two | Existing System

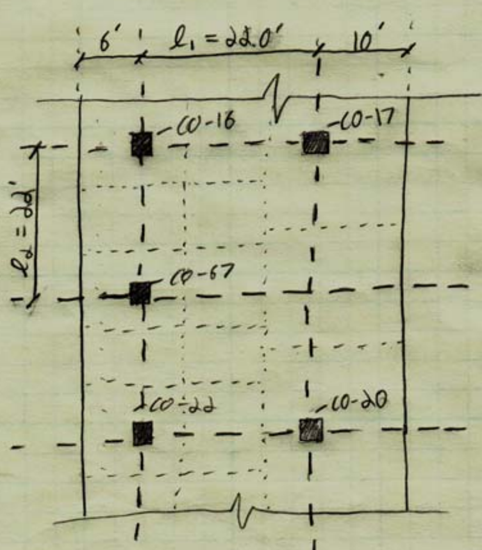
Existing System – Two Way Flat Plate

Level 5 - Southwest Quadrant

-8" Flat Plate



Due to the irregular nature of this floor framing layout, the plan to the left will be approximated as a rectangular system as shown below.



Column Locations

	(x)	(y)
CO-16	-71.83	0
CO-17	-49.92	0
CO-67	-65.18	-17.46
CO-22	-58.86	-33.99
CO-20	-42.15	-24.33

$l_1 = 71.83' - 49.92'$

$l_1 = 21.91 \sim 22.0'$

$l_2 = \sqrt{(65.18-78.83)^2 + (17.46)^2}$

$l_2 = 22.16 \sim 22.0'$

Assume: left cantilever = 6'
right cantilever = 10'

Small dotted lines represent middle & column strip boundaries

2	Scott Molongoski	Tech Two	Existing System

Slab Analysis using Equivalent Frame Method

Slab - Beam Stiffness:

$$I_1 = \frac{\ell_2 h^3}{12} = \frac{(22' \cdot 12)(8'')^3}{12} = \underline{11264 \text{ in}^4}$$

$$I_2 = \frac{I_1}{(1 - c_2/\ell_2)^2} = \frac{11264 \text{ in}^4}{(1 - 24''/22' \cdot 12)^2} = \underline{13629 \text{ in}^4}$$

$$K_{sb} = \frac{kEI_1}{\ell_1}$$

→ From Table A-14:

$c_2/\ell_2 = 0.091$
$c_1/\ell_1 = 0.069$ → by bi-linear interpolation → $k = 4.100$

$$K_{sb} = \frac{4.1(11264)E}{22' \cdot 12} = 175E$$

Carry over factor by bi-linear interpolation → $COF = 0.508$

Column Stiffness:

$$C = \Sigma\left(1 - 0.63\left(\frac{x}{y}\right)\right)\left(\frac{x^3 y}{3}\right) = \left(1 - 0.63\left(8/24\right)\right)\left(\frac{8^3 \cdot 24}{3}\right)$$

All columns are $24'' \times 24''$

$$C = 3236 \text{ in}^4$$

$$K_t = \Sigma \frac{9 \cdot E \cdot C}{\ell_2(1 - c_2/\ell_2)^3} = 2\left(\frac{9(3236)E}{22'(1 - 24/22 \cdot 12)^3}\right) \rightarrow K_t = \underline{293.7E}$$

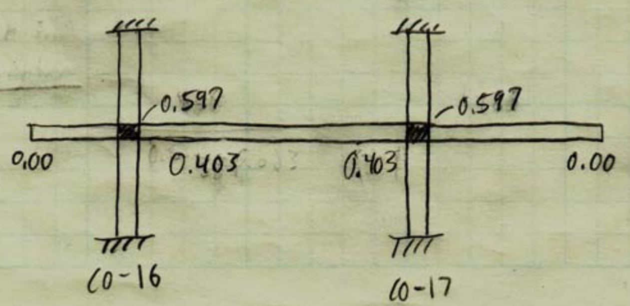
$\ell_c = 10' \cdot 12 = 120''$ $\quad \ell_c/\ell_u = 1.07 \quad t_a/t_b = 1$

$\ell_u = 120'' - 8'' = 112''$

From Table A-17: $\quad k = 4.75$

$$K_c = \Sigma \frac{kEI}{\ell_c} = 2\left(\frac{4.75\left(\frac{24 \cdot 24^3}{12}\right)E}{120}\right) = 2189E$$

$$\frac{1}{K_{ec}} = \frac{1}{K_c} + \frac{1}{K_t} = \frac{1}{2189E} + \frac{1}{293.7E} \rightarrow \underline{K_{ec} = 259E}$$



$$DF_{col} = \frac{259}{175 + 259}$$

$$DF_{col} = 0.597$$

$$1 - 0.597 = 0.403$$

3 | Scott Molongoski | Tech Two | Existing System

FEM:

$w = 1.2\left(\frac{8''}{12}\cdot 150\right) + 1.6(60) = 216\ psf$

$M = 0.084\, w\, \ell_2 \ell_1^2 = 0.084(216)(22)(22)^2$

$M = 193\ ft\text{-}k$

Cantilever Moment:

-Assume curtain wall = 100 lb/ft

Left Side:

$M = (1.2 \cdot 22 \cdot 0.1)(6) + 1.2(150 \cdot 8/12 \cdot 22' \cdot 6')(3') + 1.6(60 \cdot 22 \cdot 6)(3)$

$M = 101\ ft\text{-}k$

Right Side:

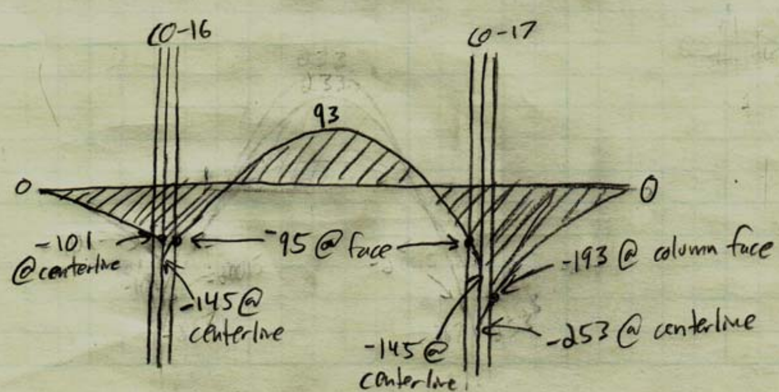
$M = 1.2(22 \cdot 0.1)(6) + 1.2(150 \cdot 8/12 \cdot 22 \cdot 10)(5) + 1.6(60 \cdot 22 \cdot 10)(5)$

$M = 253\ ft\text{-}k$

Moment Distribution

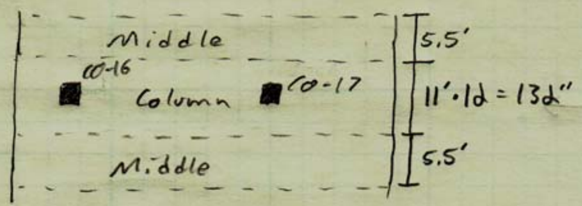
Joint	CO-16			CO-17		
DF	0	0.597	0.403	0.403	0.597	0
	Cant	Col	Slab	Slab	Col	Cant
COF			0.508			
FEM	-101		193	-193		-253
BAL		-115	-78	78	115	
CO			40	-40		
BAL		-24	-16	16	24	
CO			8.1	-8.1		
BAL		-4.8	-3.3	3.3	4.8	
CO			1.7	-1.7		
BAL		-1.0	-0.7	0.7	1.0	
Total	-101	-145	145	-145	145	-253

Moment Diagram:



4 | Scott Molongoski | Tech Two | Existing System

Moment Distribution per Direct Design Method



Left Cantilever Distribution:

$M = -101$ ft-k

$\% = 100 - 10\beta_t + 12\cancel{\beta_t\left(\frac{\alpha_{f1}\ell_2}{\ell_1}\right)\left(1-\frac{\ell_2}{\ell_1}\right)}$ $\alpha_{f1} = 0$

$C = 0$ because there are no edge beams

$\therefore \beta_t = 0$

$M^-_{o\,ext,col} = 1 \cdot 101 = -101$ ft-k

$M^-_{o\,ext,mid} = 0$ ft-k

Interior Span:

Exterior Negative Moment

$M = -95$ ft-k

$\% = 100 - 10\beta_t$; $\beta_t = 0$ because there are no edge beams

$\therefore$ $M^-_{o\,ext,col} = 1 \cdot 95 = -95$ ft-k

$M^-_{o\,ext,mid} = 0$ ft-k

Positive Moment

$M = 93$ ft-k

$\ell_2/\ell_1 = 22/29 = 0.76$

$\alpha_{f1}\,\ell_2/\ell_1 = 0$ $\therefore$ $M^+_{o\,col} = 0.6 \cdot 93 = 56$ ft-k

$M^+_{o\,mid} = 0.4 \cdot 93 = 37$ ft-k

Right Cantilever Distribution:

$M = -193$ ft-k

$\beta_t = 0$ $\therefore$

$M^-_{o\,col} = 1 \cdot 193 = -193$ ft-k

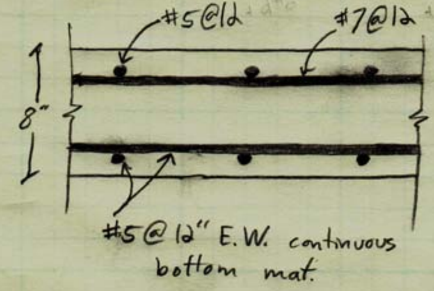
$M^-_{o\,mid} = 0$ ft-k

5 | Scott Molongoski | Tech Two | Existing System

Reinforcement:

Column Strip

Typical Rebar Layering →



Column Strip

Left Cantilever:

$d_{eff} = 8" - 3/4" - 0.5(0.625)$

$d_{eff} = 6.94"$

assume $j = 0.95$

$M^- = -101$ ft·k

$A_{s_{req'd}} = \frac{M_u}{\phi f_y jd} = \frac{101 \cdot 12000}{0.9(60000)(0.95)(6.94)}$

$A_{s_{req'd}} = 3.4$ in²

$A_{s_{min}} = 0.0018\, b \cdot h = 0.0018(11 \cdot 12)(8) = 1.9$ in²

#7@12" = 0.6 in² · 11 = 6.6 in² provided > 3.4 in² ✓

Exterior Negative Moment:

$M^- = -95$ ft·k

$A_{s_{req'd}} = \frac{M_u}{\phi f_y jd} = \frac{95 \cdot 12000}{0.9(60000)(0.95)(6.94)}$

$A_{s_{req'd}} = 3.2$ in²

$A_{s_{min}} = 1.9$ in²

#7@12" = 6.6 in² provided > 3.2 in² ✓

Positive Moment:

$M^+ = 56$ ft·k

$A_{s_{req'd}} = \frac{M_u}{\phi f_y jd} = \frac{56 \cdot 12000}{0.9(60000)(0.95)(6.94)} = 1.89$ in²

$A_{s_{min}} = 1.9$ in²

#5@12" = 0.31 · 11 = 3.41 in² provided > 1.89 in² ✓

Right Cantilever:

$A_{s_{req'd}} = \frac{M_u}{\phi f_y j \cdot d} = \frac{193 \cdot 12000}{0.9(60000)(0.95)(6.94)} = 6.51$ in²

$A_{s_{prov}} = 6.6$ in² provided > 6.51 in²

6 | Scott Molongoski: | Tech Two | Existing System

Reinforcement:

Middle Strip

Positive Moment

$M^+ = 37$ ft·k

$A_{s_{reqd}} = \frac{M_u}{\phi f_y \cdot j \cdot d} = \frac{37 \cdot 12000}{0.9 \cdot 60000 \cdot 0.95 \cdot 6.94} = 1.25$ in^2

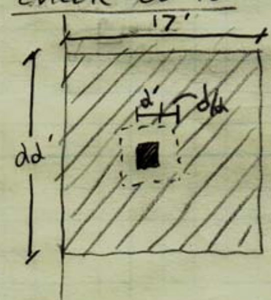
$A_{s_{min}} = 1.9$ in^2

#5 @ 12" = $0.31 \cdot 11 = 3.41$ in^2 provided > 1.25 in^2 ✓

Punching Shear:

$w = 216$ psf

Check C0-16



$d = 6.94''/2 = 3.47'' \cdot 2 + 24 \approx 31''$

$V_u = 0.216\left(17 \cdot 22 - \left(\tfrac{31}{12}\right)^2\right)$

$V_u = 79$ k

$\phi V_n = \phi 6\sqrt{f'_c}\, b_o \cdot d = 0.75\left(6\sqrt{4000} \cdot 31'' \cdot 4 \cdot 6.94\right)/1000$

$\phi V_n = 245$ k > 79 k ✓

Deflection Check:

Immediate deflection due to Dead Load

$I_y = \frac{22' \cdot 12 \cdot 8^3}{12} = 11264$ in^4 $\qquad E_c = 57000\sqrt{4000} = 3605$ ksi

$DL = 8/12 \cdot 150 + 9 = 109$ psf $\qquad w_D = \frac{(109)(22')}{1000} = 2.4$ klf

$\Delta_{D_{max}} = 0.0026\, w_D \ell^4 / E \cdot I = \frac{0.0026\left(\frac{2.4}{12}\right)(264'')^4}{3605 \cdot 11264}$

$\Delta_{D_{max}} = 0.06'' < L/480 = \frac{22 \cdot 12}{480} = 0.55''$

7	Scott Molongoski	Tech Two	Existing System

Deflection Check:

Immediate Deflection due to Live Load

$w_L = 60\,psf \cdot 22' = 1.32\ klf$

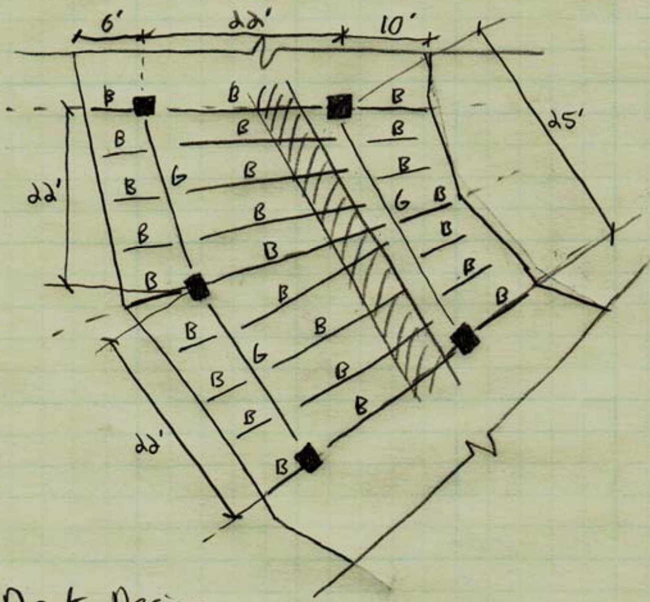
$$\Delta_{L\,max} = \frac{0.0048\, w_L\, \ell^4}{EI} = \frac{0.0048(1.32/12)(264)^4}{3605 \cdot 11264}$$

$\Delta_{L\,max} = 0.06'' < L/480 = 0.55''$

$\Delta_{tot} = 0.06'' + 0.06'' = 0.12''$

1 | Scott Molongoski | Tech Two | Non-Composite Steel Deck

Non-Composite Steel Deck w/ Steel Beams and Girders:



In this system there are 2 - 22' long girders on the outside column ring and 1 - 25' long girder on the inside column ring. The beams between them span 22' w/ 5.5' spacing in between them. Non-composite decking runs perpendicular to the beams.

Deck Design

w_D = 9 psf - superimposed
63 psf - deck/slab
5 psf - beam
77 psf

$w_L = 60$ psf

$w_u = 1.2(77) + 1.6(60) = 188$ psf

Spacing = 5.5'
3 Span

Use 1.5C18 Non-Composite Deck, 5.5" depth w/ t = 4.00"

Allowable Load = 252 psf > 188 psf ✓

Check construction span:

$5.5' \leq 8'\text{-}11''$ ✓

Check deflection:

$w_L = 1.6 \cdot 60 = 96$ psf ≤ 211 psf ✓

2 | Scott Molongoski | Tech Two | Non-Composite Steel Deck

Beam Design:

$w_u = 188$ psf — assume simply supported

$w_u = 188 \text{ psf} \cdot 5.5' = 1.03$ klf

$M_u = \frac{(1.03)(22)^2}{8} = 62.3$ ft-k

From Table 3-2 of AISC-14

Try W12×16 w/ $\phi M_p = 75.4$ ft-k > 62.3 ft-k ✓
$\phi V_n = 79.2$ k > 11.3 k ✓

Check Deflection:

$$\Delta = \frac{5wl^4}{384EI} = \frac{5(1.03)(22)^4(1728)}{384(29000)(103)} = 1.82''$$

$L/240 = 22 \cdot 12/240 = 1.1'' < 1.82'' \rightarrow$ N.G.

$I_{req'd} = 103\left(1.82/1.1\right) = 170.4 \text{ in}^4$

Try W12×26 w/ $\phi M_p = 140$ ft-k > 62.3 ft-k ✓

$$\Delta = \frac{5(1.03)(22)^4(1728)}{384(29000)(204)} = 0.92'' < 1.1'' \checkmark$$

Girder Design:

25' long Girder: assume simply supported

10' cantilever & ½ beam span = 11'

$w_u = 188 \text{ psf} \cdot 21' = 3.95$ klf

$M_u = \frac{(3.95)(25)^2}{8}$

$M_u = 309$ ft-k

$V_u = 49.4$ k

$M_u = 309$ ft-k → Try W21×44
w/ $\phi M_p = 358$ ft-k ✓
$\phi V_n = 217$ ft-k > 49.4 k ✓

Check Deflection

$$\Delta = \frac{5wl^4}{384EI} = \frac{5(3.95)(25)^4(1728)}{384(29000)(843)}$$

$\Delta = 1.42''$

$L/240 = 25 \cdot 12/240 = 1.25'' < 1.42''$ No good

$I_{req'd} = 843\left(1.42/1.25\right) = 958 \text{ in}^4$

3 | Scott Molongoski | Tech Two | Non-Composite Steel Deck

Try W21x50 w/ $I = 984 \text{ in}^4$

$$\Delta = \frac{5wl^4}{384EI} = \frac{5(3.95)(25)^4(1728)}{384(29000)(984)}$$

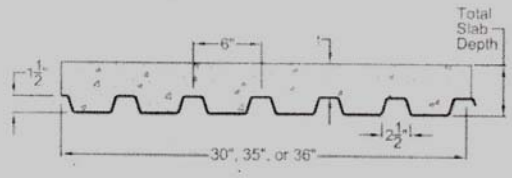
$$\Delta = 1.22'' < 1.25'' \checkmark$$

$$\boxed{\text{Use W21x50 w/ } \phi M_n = 413 \text{ ft-k}, \ \phi V_n = 237 \text{ ft-k}}$$

Assume 22' long girders will also be W21x50; due to similar loading and span lengths.

VULCRAFT

1.5 C CONFORM



NON-COMPOSITE

MAXIMUM CONSTRUCTION CLEAR SPANS (S.D.I. CRITERIA)

Total Slab Depth	DECK	WEIGHT PSF	NW CONCRETE N=9 145 PCF			WEIGHT PSF	LW CONCRETE N=14 110 PCF		
			1 SPAN	2 SPAN	3 SPAN		1 SPAN	2 SPAN	3 SPAN
3.5 (t=2.00)	1.5C24	37	5- 4	7- 1	7- 2	28	5- 10	7- 7	7- 9
	1.5C22	37	5- 9	7- 8	7- 9	29	6- 4	8- 2	8- 5
	1.5C20	38	6- 10	8- 9	9- 1	29	7- 5	9- 5	9- 9
	1.5C18	38	8- 5	10- 3	10- 8	30	9- 3	11- 1	11- 6
4 (t=2.50)	1.5C24	43	5- 1	6- 9	6- 10	33	5- 6	7- 4	7- 5
	1.5C22	43	5- 6	7- 3	7- 5	33	6- 0	7- 11	8- 1
	1.5C20	44	6- 5	8- 4	8- 8	34	7- 1	9- 1	9- 5
	1.5C18	44	7- 11	9- 9	10- 1	34	8- 9	10- 8	11- 0
4.5 (t=3.00)	1.5C24	49	4- 10	6- 5	6- 7	38	5- 4	7- 1	7- 2
	1.5C22	49	5- 3	6- 11	7- 1	38	5- 9	7- 7	7- 9
	1.5C20	50	6- 2	8- 0	8- 3	38	6- 9	8- 9	9- 0
	1.5C18	51	7- 6	9- 4	9- 8	39	8- 4	10- 3	10- 7
5 (t=3.50)	1.5C24	55	4- 8	6- 2	6- 4	42	5- 1	6- 9	6- 11
	1.5C22	56	5- 0	6- 8	6- 10	43	5- 6	7- 4	7- 6
	1.5C20	56	5- 10	7- 8	7- 11	43	6- 6	8- 5	8- 8
	1.5C18	57	7- 2	9- 0	9- 3	44	8- 0	9- 10	10- 2
5.5 (t=4.00)	1.5C24	61	4- 6	5- 11	6- 1	47	4- 11	6- 7	6- 8
	1.5C22	62	4- 10	6- 5	6- 7	47	5- 4	7- 1	7- 2
	1.5C20	62	5- 8	7- 4	7- 7	47	6- 3	8- 1	8- 5
	1.5C18	63	6- 11	8- 8	8- 11	48	7- 8	9- 6	9- 10
6 (t=4.50)	1.5C24	67	4- 4	5- 9	5- 11	51	4- 9	6- 4	6- 5
	1.5C22	68	4- 8	6- 2	6- 4	52	5- 2	6- 10	7- 0
	1.5C20	68	5- 6	7- 1	7- 4	52	6- 0	7- 10	8- 1
	1.5C18	69	6- 9	8- 4	8- 7	53	7- 5	9- 3	9- 6
6.5 (t=5.00)	1.5C24	73	4- 3	5- 6	5- 8	56	4- 7	6- 2	6- 3
	1.5C22	74	4- 7	6- 0	6- 2	56	5- 0	6- 8	6- 9
	1.5C20	74	5- 4	6- 10	7- 1	57	5- 10	7- 7	7- 10
	1.5C18	75	6- 7	8- 1	8- 4	57	7- 2	8- 11	9- 3

REINFORCED CONCRETE SLAB ALLOWABLE LOADS

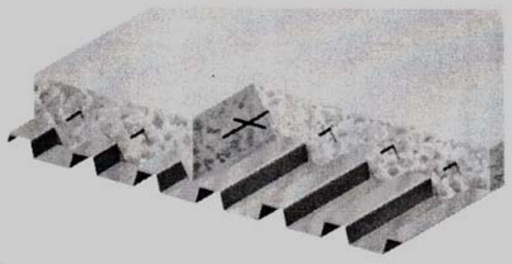
Slab Depth	REINFORCEMENT		Superimposed Uniform Load (psf) -- 3 Span Condition Clear Span (ft.-in.)										
	W.W.F.	As	4- 0	4- 6	5- 0	5- 6	6- 0	6- 6	7- 0	7- 6	8- 0	8- 6	9- 0
3.5 (t=2.00)	6X6-W2.1XW2.1	0.042*	108	86									
	6X6-W2.9XW2.9	0.058	147	116									
	4X4-W2.9XW2.9	0.087	214	169									
4 (t=2.50)	6X6-W2.1XW2.1	0.042*	136	108	87	72							
	6X6-W2.9XW2.9	0.058	185	147	119	98							
	4X4-W2.9XW2.9	0.087	272	215	174	144							
4.5 (t=3.00)	6X6-W2.1XW2.1	0.042*	164	129	**160**	**132**	**111**	**95**	**82**				
	6X6-W2.9XW2.9	0.058*	224	177	**215**	**177**	**149**	**127**	**110**				
	4X4-W2.9XW2.9	0.087	329	260	**318**	**263**	**221**	**188**	**162**				
5 (t=3.50)	6X6-W2.9XW2.9	0.058*	262	207	**264**	**218**	**183**	**156**	**135**	**117**			
	4X4-W2.9XW2.9	0.087	387	306	**392**	**324**	**272**	**232**	**200**	**174**			
	4X4-W4.0XW4.0	0.120	400	400	**400**	**400**	**363**	**310**	**267**	**233**			
5.5 (t=4.00)	6X6-W2.9XW2.9	0.058*	301	238	**313**	**259**	**217**	**185**	**160**				
	4X4-W2.9XW2.9	0.087	400	351	**400**	**385**	**323**	**275**	**237**				
	4X4-W4.0XW4.0	0.120	400	400	**400**	**400**	**400**	**370**	**319**				
6 (t=4.50)	6X6-W2.9XW2.9	0.058*	339	268	**358**	**296**	**249**	**212**	**183**				
	4X4-W2.9XW2.9	0.087*	400	397	**400**	**400**	**370**	**315**	**272**				
	4X4-W4.0XW4.0	0.120	400	400	**400**	**400**	**400**	**400**	**366**				
6.5 (t=5.00)	4X4-W2.9XW2.9	0.087*	400	400	**400**	**400**	**400**	**348**					
	4X4-W4.0XW4.0	0.120	400	400	**400**	**400**	**400**	**400**					
	4X4-W5.0XW5.0	0.150	400	400	**400**	**400**	**400**	**400**					

NOTES:
1. * As does not meet A.C.I. criterion for temperature and shrinkage.
2. Recommended conform types are based upon S.D.I. criteria and normal weight concrete.
3. Superimposed loads are based upon three span conditions and A.C.I. moment coefficients.
4. Load values for single span and double spans are to be reduced.
5. Vulcraft's painted or galvanized form deck can be considered as permanent support in most building applications. See page 23. If uncoated form deck is used, deduct the weight of the slab from the allowable superimposed uniform loads.
6. Superimposed load values shown in bold type require that mesh be draped. See page 23.

VULCRAFT

SLAB INFORMATION

Total Slab Depth, in.	Theo. Concrete Volume Yd^3 / 100 ft^2	ft^3 / ft^2	Recommended Welded Wire Fabric
3 1/2	0.92	0.247	6x6 - W1.4xW1.4
4	1.07	0.289	6x6 - W1.4xW1.4
4 1/2	1.22	0.331	6x6 - W1.4xW1.4
4 3/4	1.30	0.352	6x6 - W1.4xW1.4
5	1.38	0.372	6x6 - W2.1xW2.1
5 1/2	1.53	0.414	6x6 - W2.1xW2.1
5 3/4	1.61	0.435	6x6 - W2.1xW2.1
6	1.69	0.456	6x6 - W2.1xW2.1



SECTION PROPERTIES

Deck Type	Design Thickness in.	Deck Weight psf	Section Properties I_p in^4/ft	I_n in^4/ft	S_p in^3/ft	S_n in^3/ft	V_a lbs/ft	F_y ksi
1.5C24	0.0239	1.44	0.136	0.108	0.132	0.120	2634	60
1.5C22	0.0295	1.78	0.177	0.143	0.179	0.169	2754	50
1.5C20	0.0358	2.14	0.222	0.186	0.231	0.224	3322	50
1.5C18	0.0474	2.82	0.295	0.272	0.324	0.311	4350	50

NON-COMPOSITE

ALLOWABLE UNIFORM LOAD (PSF)

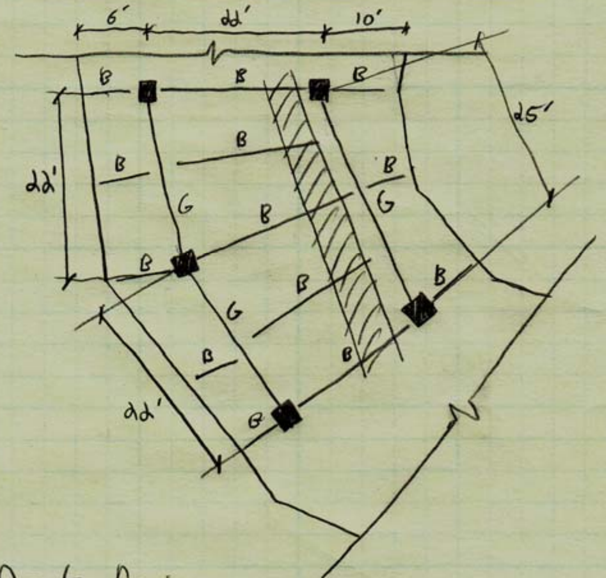
TYPE NO.	NO. OF SPANS	DESIGN CRITERIA	CLEAR SPAN (ft-in) 4- 0	4- 6	5- 0	5- 6	6- 0	6- 6	7- 0	7- 6	8- 0	8- 6	9- 0	9- 6	10- 0
1.5C24	1	Fb = 36,000	198	156	126	105	88	75	65	56	49	44	39	35	32
		Defl. = l/240	140	98	71	54	41	33	26	21	17	15	12	10	9
		Defl. = l/180	186	131	95	72	55	43	35	28	23	19	16	14	12
	2	Fb = 36,000	177	140	114	94	79	68	58	51	45	40	35	32	29
		Defl. = l/240	301	212	154	116	89	70	56	46	38	31	26	22	19
		Defl. = l/180	402	282	206	155	119	94	75	61	50	42	35	30	26
	3	Fb = 36,000	220	175	142	117	99	84	73	63	56	49	44	40	36
		Defl. = l/240	236	166	121	91	70	55	44	36	29	25	21	18	15
		Defl. = l/180	314	221	161	121	93	73	59	48	39	33	28	23	20
1.5C22	1	Fb = 30,000	223	176	143	118	99	85	73	64	56	49	44	40	36
		Defl. = l/240	182	128	93	70	54	42	34	28	23	19	16	14	12
		Defl. = l/180	242	170	124	93	72	56	45	37	30	25	21	18	15
	2	Fb = 30,000	207	164	133	110	93	79	68	60	52	47	41	37	34
		Defl. = l/240	395	278	202	152	117	92	74	60	49	41	35	29	25
		Defl. = l/180	527	370	270	203	156	123	98	80	66	55	46	39	34
	3	Fb = 30,000	257	204	166	137	116	99	85	74	65	58	52	47	42
		Defl. = l/240	309	217	158	119	92	72	58	47	39	32	27	23	20
		Defl. = l/180	412	290	211	159	122	96	77	63	52	43	36	31	26
1.5C20	1	Fb = 30,000	288	228	184	152	128	109	94	82	72	64	57	51	46
		Defl. = l/240	228	160	117	88	67	53	42	35	28	24	20	17	15
		Defl. = l/180	304	213	155	117	90	71	57	46	38	32	27	23	19
	2	Fb = 30,000	273	217	176	146	123	105	91	79	69	62	55	49	45
		Defl. = l/240	504	354	258	194	149	117	94	76	63	53	44	38	32
		Defl. = l/180	672	472	344	258	199	157	125	102	84	70	59	50	43
	3	Fb = 30,000	339	269	219	182	153	131	113	98	87	77	69	62	56
		Defl. = l/240	394	277	202	152	117	92	74	60	49	41	35	29	25
		Defl. = l/180	526	369	269	202	156	123	98	80	66	55	46	39	34
1.5C18	1	Fb = 30,000	404	319	259	214	180	153	132	115	101	90	80	72	65
		Defl. = l/240	303	213	155	116	90	71	56	46	38	32	27	23	19
		Defl. = l/180	404	283	207	155	120	94	75	61	50	42	35	30	26
	2	Fb = 30,000	379	301	244	203	171	146	126	110	96	85	76	68	62
		Defl. = l/240	700	492	359	269	207	163	131	106	88	73	61	52	45
		Defl. = l/180	934	656	478	359	277	218	174	142	117	97	82	70	60
	3	Fb = 30,000	468	373	304	252	212	181	157	137	120	107	95	85	77
		Defl. = l/240	548	385	281	211	162	128	102	83	68	57	48	41	35
		Defl. = l/180	731	513	374	281	216	170	136	111	91	76	64	55	47

Minimum exterior bearing length is 1.5 inches.
Minimum interior bearing length is 3.0 inches.

NUCOR VULCRAFT GROUP 31

1 | Scott Molongoski | Tech Two | Composite Beam w/ Deck

Composite Beams w/ steel Deck:



This system has the same girder layout as the Non-composite system. The beams between them span 22' w/ 11' spacing in between them. Composite decking runs perpendicular to the beams.

Deck Design

$w_L = 60\,psf + 9\,psf = 69\,psf$

Spacing = 11'

Span = 22'

Try 1.5VLR16 w/ Lightweight Concrete: 4" depth ($t = 2.50"$)

3 span max = 11'-2"

11'-0" spacing = 128 psf > 69 psf ✓

Beam Design

Deck/topping = 34 psf   LL = 60 psf

SDL = 9 psf

Beam = 5 psf

$w_u = 1.2(34+9+5) + 1.6(60) = 154\,psf \cdot 11'/1000 = 1.69\,klf$

$M_u = \frac{w\ell^2}{8} = \frac{(1.69)(22)^2}{8} = 102\,ft\text{-}k$

assume $a = 1"$

$Y2 = 4" - \frac{1}{2}" = 3.50"$

2 | Scott Molongoski | Tech Two | Composite Beam

Try:

- $W10\times 26 \rightarrow \Sigma Q_n = 190/17.2 = 11.0 \rightarrow 22$ studs/beam

  $\phi M_n = 195$ ft·k

  From Table 3-21;
  assuming: deck perpendicular
  3/4" stud
  $R_p = 0.6$
  1 stud per rib

  Economy Evaluation:

  1 stud ≈ 10 lbs of steel weight

  ∴ 26 lb/ft · 22' + 22 studs · 10'
  = 792 lbs

- $W10\times 19 \rightarrow \Sigma Q_n = 162/17.2 = 9.4 \rightarrow 20$ studs/beam

  $\phi M_n = 149$ ft-k

  Economy = 19·22 + 20·10 = 618 lbs

- $W12\times 19 \rightarrow \Sigma Q_n = 138/17.2 = 8.0 \rightarrow 16$ studs/beam

  $\phi M_n = 162$ ft-k

  Economy = 19·22 + 16·10 = 578 lbs

Check W12×19:

Unshored strength:

$\phi M_p = 92.6$ ft·k

$w_u = 1.4(34+9+5)(11) + 1.4(19) = 0.766$ klf

$w_u = 1.2(48\cdot 11 + 19) + 1.6(20\cdot 11) = 1.01$ klf ← controls

$M_u = \frac{(1.01)(22)^2}{8} = 61.1$ ft-k < 92.6 ft-k ✓ no shoring req'd

Wet Concrete Deflection:

$w_{wc} = 34(11) + 26 = 0.4$ klf

$$\Delta_{wc} = \frac{5 w l^4}{384 E I} = \frac{5(0.4)(22)^4(1728)}{384(29000)(130)} = 0.56''$$

$$\Delta_{wc} = \frac{22\cdot 12}{240} = 1.1'' > 0.56'' ✓$$

3 | Scott Molongoski | Tech Two | Composite Beam

Beam Design:

Check LL Deflection:

$w_L = 60 \cdot 11 = 0.66$ klf

$I_{LB} = 300$ in^4 @ $Y2 = 3.5''$ & BFL ($\Sigma Q_n = 138$ k) ← From Table 3-20

$$\Delta_{LL} = \frac{5(0.66)(22)^4(1728)}{384(29000)(300)} = 0.4''$$

$$\Delta_{LL_{max}} = 22 \cdot 12 / 360 = 0.73'' > 0.4'' \checkmark$$

Stud Spacing:

$22'' \cdot 12 = 264''/16 = 16.5''$ $> 4 \cdot 3/4'' = 3''$ min ✓

$< 8 \cdot 4'' = 32''$ max

Use W12×19 w/ 16 studs spaced @ 16.5''
578 lbs per member

Girder Design:

Check 25' long girder

$w_u = 154$ psf $\cdot 21'/1000 = 3.23$ klf

$$M_u = \frac{w\ell^2}{8} = \frac{(3.23)(25)^2}{8} = 252 \text{ ft-k}$$

assume $a = 1''$ → $Y2 = 3.5''$

Try:

- W14×26 → $\Sigma Q_n = 226/21.2 = 10.6$ → 22 studs/beam
  $\phi M_n = 258$ ft-k
  Economy $= 26 \cdot 25 + 22 \cdot 10 = 870$ lbs
- W16×26 → $\Sigma Q_n = 194/21.2 = 9.2$ → 20 studs/beam
  $\phi M_n = 275$ ft-k
  Economy $= 26 \cdot 25 + 20 \cdot 10 = 850$ lbs

Check W16×26

Stud Strength:
Deck parallel
3/4'' stud
$w_r/h_r = 4.25/1.5 = 2.8 \geq 1.5$ ∴
$Q_n = 21.2$ k

4 | Scott Molongoski | Tech Two | Composite Beam

Girder Design:

Unshored strength:

$\phi M_p = 166$ ft-k

$w_u = 1.4(48)(21) + 1.4(26) = 1.45$ kef

$M_u = \frac{(1.45)(25)^2}{8} = 113$ ft-k < 166 ft-k ✓ No shoring req'd

Check LL Deflection:

$\Delta_{LL} = \frac{5 w \ell^4}{48 E I}$

$I_{LB} = 506$ in^4 @ $Y2 = 3.5''$ & BFL ($\Sigma Q_n = 194$ k)

$\Delta_{LL} = \frac{5(1.26)(25)^4(1728)}{384(29000)(506)} = 0.755''$

$\Delta_{LL_{max}} = 25 \cdot 12 / 360 = 0.833'' > 0.755''$ ✓

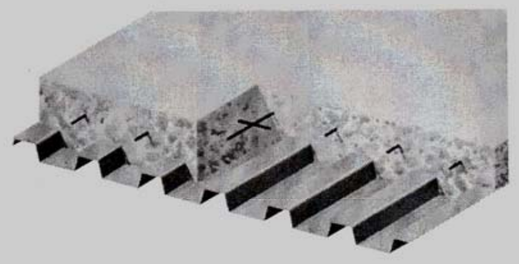
Stud Spacing:

$25 \cdot 12 / 20 = 15''$

Use W16×26 w/ 20 studs spaced @ 15"

850 lbs per member

VULCRAFT



SLAB INFORMATION

Total Slab Depth, in.	Theo. Concrete Volume Yd^3 / 100 ft^2	ft^3 / ft^2	Recommended Welded Wire Fabric
3 1/2	0.92	0.247	6x6 - W1.4xW1.4
4	1.07	0.289	6x6 - W1.4xW1.4
4 1/2	1.22	0.331	6x6 - W1.4xW1.4
4 3/4	1.30	0.352	6x6 - W1.4xW1.4
5	1.38	0.372	6x6 - W2.1xW2.1
5 1/2	1.53	0.414	6x6 - W2.1xW2.1
5 3/4	1.61	0.435	6x6 - W2.1xW2.1
6	1.69	0.456	6x6 - W2.1xW2.1

(N=14.15) LIGHTWEIGHT CONCRETE (110 PCF)

TOTAL SLAB DEPTH	DECK TYPE	SDI Max. Unshored Clear Span			Superimposed Live Load, PSF Clear Span (ft.-in.)														
		1 SPAN	2 SPAN	3 SPAN	5'-0	5'-6	6'-0	6'-6	7'-0	7'-6	8'-0	8'-6	9'-0	9'-6	10'-0	10'-6	11'-0	11'-6	12'-0
3.50 (t=2.00) 30 PSF	1.5VLR22	6'-4	8'-2	8'-5	278	247	222	182	164	149	136	125	115	103	88	76	66	58	51
	1.5VLR20	7'-5	9'-5	9'-9	305	271	243	220	201	165	151	139	128	110	94	82	71	62	55
	1.5VLR19	8'-6	10'-5	10'-9	329	292	262	237	216	198	183	163	137	117	100	86	75	66	58
	1.5VLR18	9'-3	11'-1	11'-6	350	311	279	252	230	211	195	171	144	123	105	91	79	69	61
	1.5VLR16	9'-3	11'-3	11'-8	352	312	280	253	231	212	195	181	158	135	115	100	87	76	67
4.00 (t=2.50) 34 PSF	1.5VLR22	6'-0	7'-11	8'-1	324	288	258	212	192	174	159	146	134	124	115	106	98	86	76
	1.5VLR20	7'-1	9'-1	9'-5	355	315	283	256	233	192	176	161	149	137	127	119	105	92	81
	1.5VLR19	8'-0	10'-0	10'-4	382	339	304	275	251	230	212	175	161	149	139	128	111	97	85
	1.5VLR18	8'-9	10'-8	11'-0	400	360	323	292	266	244	225	209	172	160	148	134	116	102	90
	1.5VLR16	8'-9	10'-10	11'-2	400	360	323	292	266	244	225	209	172	159	148	138	128	112	98
4.50 (t=3.00) 39 PSF	1.5VLR22	5'-9	7'-7	7'-9	372	330	272	244	220	200	183	167	154	142	132	122	114	106	99
	1.5VLR20	6'-9	8'-9	9'-0	400	361	324	293	243	221	202	185	171	158	146	136	127	118	111
	1.5VLR19	7'-8	9'-7	9'-11	400	388	348	315	287	264	219	201	185	171	159	148	138	129	120
	1.5VLR18	8'-4	10'-3	10'-7	400	400	369	334	305	279	258	214	198	183	170	158	148	138	126
	1.5VLR16	8'-4	10'-4	10'-9	400	400	369	334	304	279	257	213	197	182	169	158	147	138	129
4.75 (t=3.25) 41 PSF	1.5VLR22	5'-8	7'-6	7'-7	396	352	290	260	235	213	195	178	164	152	141	130	121	113	106
	1.5VLR20	6'-7	8'-7	8'-10	400	385	345	312	259	235	215	198	182	168	156	145	135	126	118
	1.5VLR19	7'-6	9'-5	9'-9	400	400	371	336	306	281	233	214	197	183	170	158	147	138	129
	1.5VLR18	8'-2	10'-1	10'-5	400	400	393	356	324	298	274	228	211	195	181	169	158	147	138
	1.5VLR16	8'-2	10'-2	10'-6	400	400	392	355	324	297	274	227	210	194	180	168	157	147	138
5.00 (t=3.50) 43 PSF	1.5VLR22	5'-6	7'-4	7'-6	400	374	308	276	250	227	207	190	175	161	149	139	129	120	112
	1.5VLR20	6'-6	8'-5	8'-8	400	400	367	332	275	250	229	210	193	179	166	154	144	134	126
	1.5VLR19	7'-4	9'-3	9'-6	400	400	394	356	325	271	248	227	210	194	180	168	157	146	137
	1.5VLR18	8'-0	9'-10	10'-2	400	400	400	378	344	316	291	242	224	207	192	179	167	157	147
	1.5VLR16	8'-0	10'-0	10'-4	400	400	400	377	343	315	291	241	223	206	192	178	167	156	146
5.75 (t=4.25) 50 PSF	1.5VLR22	5'-3	6'-11	7'-1	400	400	364	326	295	268	244	224	206	191	177	164	153	142	133
	1.5VLR20	6'-2	8'-0	8'-3	400	400	400	360	325	295	270	248	229	211	196	182	170	159	149
	1.5VLR19	6'-11	8'-9	9'-1	400	400	400	400	351	319	292	268	248	229	213	198	185	173	162
	1.5VLR18	7'-6	9'-4	9'-8	400	400	400	400	400	372	311	286	264	245	227	212	198	185	174
	1.5VLR16	7'-7	9'-6	9'-10	400	400	400	400	400	371	309	284	263	243	226	211	197	184	173

Notes: 1. Minimum exterior bearing length required is 1.50 inches. Minimum interior bearing length required is 3.00 inches. If these minimum lengths are not provided, web crippling must be checked.
2. Always contact Vulcraft when using loads in excess of 200 psf. Such loads often result from concentrated, dynamic, or long term load cases for which reductions due to bond breakage, concrete creep, etc. should be evaluated.
3. All fire rated assemblies are subject to an upper live load limit of 250 psf.

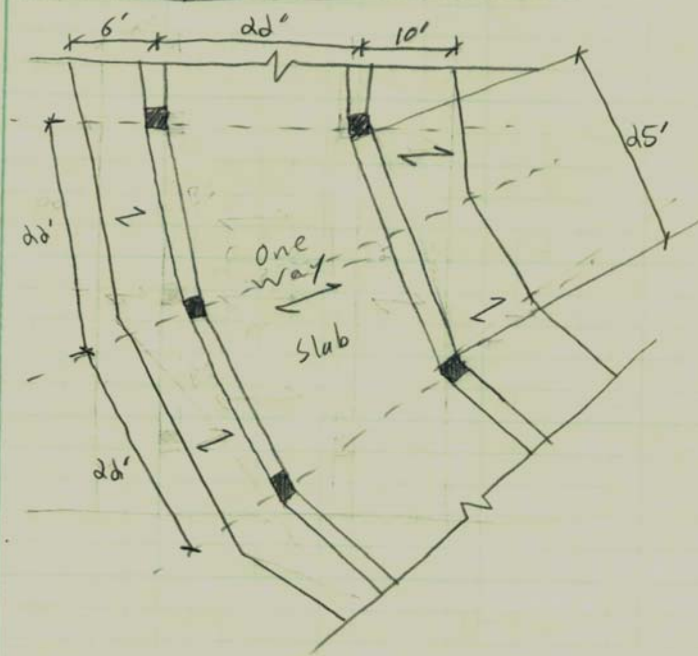
COMPOSITE

NUCOR VULCRAFT GROUP

51

1 Scott Molongoski: | Tech Two | One Way Slab

One Way Slab with Beams:



For this system, a typical bay includes 2-22' long beams on the outside column ring and 1-25' long beam on the inside column ring.

Beam Design:

25' long beam:

Check h_{min} from Table 9.5(a) of ACI 318-11

Beam w/ both ends continuous $\rightarrow h = \ell/21 = \frac{25' \cdot 12}{21}$

$h_{min} = 14.28$

Try $b = 24''$, $h = 16''$, $d = 13''$, $f'_c = 4000$ psi, $f_y = 60000$ psi

Loads

$w_D = 9$ psf SDL $\qquad w_L = 60$ psf

$(10/12 \cdot 150) = 125$ psf slab

$(16/12 \cdot 150) = 200$ psf beam

$w_u = 1.2\left[(9+125)(10'+11') + (200)(2')\right] + 1.6\left[(60)(10'+11')\right]$

$w_u = 5.87$ klf

Positive Midspan Moment

$M_u = \frac{w_u \ell_n^2}{16} = \frac{(5.87)(25-2)^2}{16} = 193$ ft-k

Negative Moment @ Face

$M_u = \frac{w_u \ell_n^2}{11} = \frac{(5.87)(23)^2}{11} = 281$ ft-k

2 Scott Molongoski | Tech Two | One Way Slab

Midspan Bottom Reinforcement:

$A_s = \frac{M_u}{4d} = \frac{193}{4(13)} = 3.71 \text{ in}^2$

-Try (4) #9 = 4.00 in^2

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(4.00)(60)}{0.85(4)(24)} = 2.94 \text{ in}$ $c = 3.34/0.85 = 3.46 \text{ in}$

$\varepsilon_s = \frac{0.003}{c}(d-c) = \frac{0.003}{3.46}(12-3.46)$

$\phi M_n = 0.9 A_s f_y (d - a/2)$ $\varepsilon_s = 0.008 \geq 0.002$

$\phi M_n = 0.9(4.00)(60)(13 - 2.94/2)$

$\phi M_n = 207 \text{ ft-k} \geq 193 \text{ ft-k}$ ✓

check min reinforcement:

$A_{s,min} \geq \begin{cases} \frac{3\sqrt{f'_c}}{f_y} b d = \frac{3\sqrt{4000}}{60000}(24)(13) = 0.91 \\ \frac{200 b d}{f_y} = \frac{200(24)(13)}{60000} = 0.96 \end{cases} \leq 4.00 \text{ in}^2$ ✓

$A_{s,max} = 0.018(24)(13) = 5.62 \geq \boxed{4.00 \text{ in}^2 = (4)\#9}$

Shear Reinforcement:

$V_u = \frac{w_u l_n}{2} = \frac{(5.84)(23)}{2} = 67.16 \text{ k}$

$V_c = 2\lambda\sqrt{f'_c} b_w d = 2(1)\sqrt{4000}(24)(13) = 36.43 \text{ k}$ → need shear reinforcement

$V_s = V_u/\phi - V_c = 67.16/0.75 - 36.43 = 53.12 \text{ k}$

$V_{s_{max}} = 8\sqrt{f'_c} b_w d = 8\sqrt{4000}(24)(13) = 145.7 > 53.12$ ✓

Max Spacing: $V_s \leq 4\sqrt{f'_c} b_w d = 4\sqrt{4000}(24)(13) = 72.85 \text{ k}$

→ use $s_{max} = \min \begin{cases} d/2 = 13/2 = 6.5 \\ 24'' \end{cases}$

$A_v = \frac{V_s}{f_{yt} \, d/s} = \frac{53.12}{60 \cdot 13/6.5} = 0.44 \text{ in}^2$ → too large, use (2) #4

$s = \frac{A_v f_{yt} d}{V_s} = \frac{(0.4)(60)(13)}{53.12} = 5.4''$ → $\boxed{\text{use (2) \#4 @ 5''}}$

3 Scott Molongoski | Tech Two | One Way Slab

Beam Deflection Check:

$$I_e = (M_{cr}/M_a)^3 I_g + [1 - (M_{cr}/M_a)^3] I_{cr}$$

$$f_r = 7.5\sqrt{4000} = 474 \text{ psi}$$

$$E_c = w_c^{1.5} 33\sqrt{f'_c} = (145)^{1.5} \cdot 33\sqrt{4000} \cdot 3644147 \text{ psi}$$

$$n_s = \frac{29000000}{3644147} = 8.0$$

$$I_g = \frac{bh^3}{12} = \frac{(24)(16)^3}{12} = 8192 \text{ in}^4$$

$$\bar{y} = \frac{b \cdot h(h/2) + (n-1)(A_s)(d)}{b \cdot h + (n-1)(A_s)} = \frac{24 \cdot 16(16/2) + (7)(4.00)(13)}{24 \cdot 16 + (7)(4.00)} = 8.3''$$

$$I_{cr} = \frac{bh^3}{12} + bh(h/2 - \bar{y})^2 + (n-1)(A_s)(d - \bar{y})^2$$

$$I_{cr} = \frac{(24)(16)^3}{12} + 24(16)(8.0 - 8.3)^2 + (7)(4.00)(13 - 8.3)^2$$

$$I_{cr} = 8846 \text{ in}^4$$

$$M_{cr} = \frac{f_r I_g}{y_{bot}} = \frac{(474)(8192)}{7.7} = 42.0 \text{ ft-k}$$

$$I_e = (42.0/193)^3 (8192) + [1 - (42.0/193)^3] 8846$$

$$I_e = 8839 \text{ in}^4$$

$$\Delta_T = \frac{5 w \ell^4}{384 E_c I_e} = \frac{5(5.87)(23)^4(1728)}{384(3644)(8839)} = 1.147$$

$$\Delta_T = L/240 = 23 \cdot 12/240 = 1.15 \geq 1.147 \checkmark$$

4 Scott Molongoski | Tech Two | One-Way Slab

Beam Design:

Negative Reinforcement:

$$A_s = \frac{M_u}{4d} = \frac{281}{4(13)} = 5.4 \text{ in}^2$$

Try (3) #10 & (2) #9 = 5.81 in^2

$$a = \frac{(5.81)(60)}{0.85 \cdot 4 \cdot 24} = 4.27$$

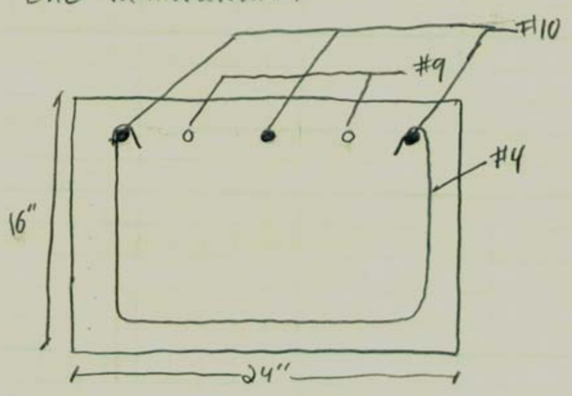
$$\phi M_n = 0.9(5.81)(60)(13 - 4.27/2)$$

$$\phi M_n = 284 \text{ ft-k} \geq 281 \text{ ft-k} \checkmark$$

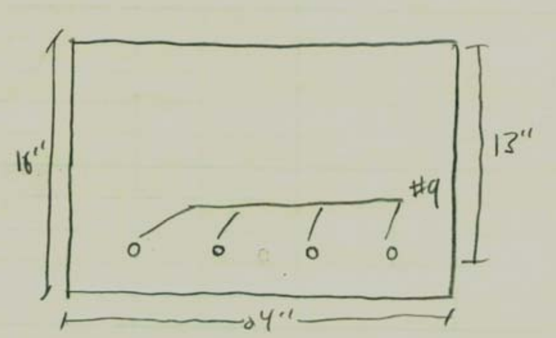
$$\boxed{\text{Use (3)\#10 \& (2)\#9} \quad (A_s = 5.81 \text{ in}^2)}$$

Beam Sections:

End Reinforcement:



Midspan Reinforcement:



5 | Scott Molongoski | Tech Two | One Way Slab

Slab Design:

Minimum Thickness: Central span $= \ell/28 = 22\cdot12/28 = 9.4'' \rightarrow 10''$

Left cantilever $= \ell/24 = 6\cdot12/24 = 3''$

Right cantilever $= \ell/24 = 10\cdot12/24 = 5''$

$h = 10''$, $d = 9''$

Loads

$w_u = 1.2\left(\frac{10}{12}\cdot150 + 9\right) + 1.6(60)$

$w_u = 257$ psf

$M_u = \frac{w_u \ell_n^2}{10} = \frac{257\text{psf}\cdot1\text{ft}\cdot20\text{ft}^2}{10} = 10.7$ ft-k/ft

$A_s \cong \frac{M_u}{\phi f_y(jd)}$ ← due to low ρ

$A_s = \frac{10.7\cdot12}{0.9\cdot60\cdot0.95\cdot9} = 0.278$ in²/ft

$A_{s,min} = 0.0018bh = 0.0018\cdot12\cdot10 = 0.216$ in²/ft < 0.278 in²/ft ✓

$\boxed{\text{Use \#5 @ 12''} \rightarrow A_s = 0.31 \text{ in}^2}$

Check Spacing:

$s = 15\left(\frac{40000}{f_s}\right) - 2.5c_c \leq 12\left(\frac{40000}{f_s}\right)$

$s = 15\left(\frac{40000}{40000}\right) - 2.5\cdot0.75 \leq 12\left(\frac{40000}{4000}\right) = 12''$ ✓

Check shear:

$V_u = \frac{1.15 w_u \ell_n}{2} = \frac{1.15(257)(1)(20)}{2} = 2956$ lb/ft

$V_c = 2\lambda\sqrt{f'_c}\,b_w d = 2(1)\sqrt{4000}(12)(9) = 13661$ lb/ft $\cdot 0.75 = 10246 > 2956$ ✓

Appendix B: RS Means Cost Estimates

B.1: Two-Way Flat Plate Assembly

Assembly B10102236000 **Based on National Average Costs**

Flat plate, concrete, 9" slab, 20" column, 20'x25' bay, 75 PSF superimposed load, 188 PSF total load

Description	Quantity	Unit	Material	Installation	Total
C.I.P. concrete forms, elevated slab, flat plate, plywood, to 15' high, 4 use, includes s...	0.98600	S.F.	1.12	5.57	6.69
C.I.P. concrete forms, elevated slab, edge forms, alternate pricing, to 6" high, 1 use, i...	0.03300	SFCA	0.02	0.21	0.23
Reinforcing Steel, in place, elevated slabs, #4 to #7, A615, grade 60, incl labor for acc...	2.65200	Lb.	1.49	1.14	2.63
Structural concrete, ready mix, normal weight, 3000 psi, includes local aggregate, san...	0.75100	C.F.	3.12	0.00	3.12
Structural concrete, placing, elevated slab, pumped, 6" to 10" thick, includes strike of...	0.75100	C.F.	0.00	0.97	0.97
Concrete finishing, floors, for specified Random Access Floors in ACI Classes 1, 2, 3 an...	1.00000	S.F.	0.00	0.86	0.86
Concrete surface treatment, curing, sprayed membrane compound	0.01000	C.S.F.	0.08	0.09	0.17
Total			$5.85	$8.84	$14.69

Figure B.1: RS Means 2012 Two-way flat plate cost estimate. Courtesy meansworks.com

This estimate uses a 20’x25’ bay size instead of the approximate 22’x22’ bay size used in this report. The slab is 9” here instead of 8” in the existing system and the load is also underestimated in this assembly.

B.2: Non-Composite Steel Deck Assembly

Assembly B10102540800 **Based on National Average Costs**

Floor, composite metal deck, 5" slab, 20'x25' bay, 21" total depth, 75 PSF superimposed load, 127 PSF total load

Description	Quantity	Unit	Material	Installation	Total
Welded wire fabric, sheets, 6 x 6 - W1.4 x W1.4 (10 x 10) 121 lb. per C.S.F., A185, incl...	0.01100	C.S.F.	0.17	0.40	0.56
Structural concrete, ready mix, normal weight, 3000 psi, includes local aggregate, san...	0.01100	C.Y.	1.23	0.00	1.23
Structural concrete, placing, elevated slab, pumped, less than 6" thick, includes strike...	0.01100	C.Y.	0.00	0.36	0.36
Concrete finishing, floors, for specified Random Access Floors in ACI Classes 1, 2, 3 an...	1.00000	S.F.	0.00	0.86	0.86
Concrete surface treatment, curing, sprayed membrane compound	0.01000	C.S.F.	0.08	0.09	0.17
Structural steel project, apartment, nursing home, etc, 100-ton project, 3 to 6 stories,...	6.75000	Lb.	9.45	2.90	12.35
Metal floor decking, steel, non-cellular, composite, galvanized, 3" D, 18 gauge	1.05000	S.F.	2.88	1.10	3.98
Metal decking, steel edge closure form, galvanized, with 2 bends, 12" wide, 18 gauge	0.05000	L.F.	0.20	0.12	0.32
Sprayed fireproofing, cementitious, normal density, beams, 1 hour rated, 1-3/8" thick...	0.68700	S.F.	0.40	0.68	1.08
Sprayed fireproofing, cementitious, normal density, corrugated or fluted decks, 1" thi...	1.00000	S.F.	0.87	1.18	2.05
Total			$15.30	$7.69	$22.99

Figure B.2: RS Means 2012 Non-composite steel deck cost estimate. Courtesy meansworks.com

This system uses a 20’x25’ bay size whereas the design used approximately a 22’x22’ bay. Loads were underestimated in this assembly. The total system depth is less than the design but the slab thickness is nearly the same.

Appendix B: RS Means Cost Estimates

B.3: Composite Steel Deck Assembly

Assembly B10102562500 **Based on National Average Costs**

Floor, composite metal deck, shear connectors, 5.5" slab, 20'x25' bay, 21.5" total depth, 75 PSF superimposed load, 115 PSF total load

Description	Quantity	Unit	Material	Installation	Total
Shores, vertical members, to 10' high, includes erect and strip by hand	0.01400	Ea.	0.00	0.28	0.28
Welded wire fabric, sheets, 6 x 6 - W1.4 x W1.4 (10 x 10) 121 lb. per C.S.F., A185, incl...	0.01100	C.S.F.	0.17	0.40	0.56
Structural concrete, placing, elevated slab, pumped, less than 6" thick, includes strike...	0.33300	C.F.	0.00	0.51	0.51
Structural concrete, ready mix, lightweight, 110 #/C.F., 3000 psi, includes local aggre...	0.33300	C.F.	2.41	0.00	2.41
Concrete finishing, floors, for specified Random Access Floors in ACI Classes 1, 2, 3 an...	1.00000	S.F.	0.00	0.86	0.86
Concrete surface treatment, curing, sprayed membrane compound	0.01000	C.S.F.	0.08	0.09	0.17
Weld shear connector, 3/4" dia x 4-7/8" L	0.13000	Ea.	0.10	0.26	0.36
Structural steel project, apartment, nursing home, etc, 100-ton project, 3 to 6 stories,...	4.65000	Lb.	6.51	2.00	8.51
Metal floor decking, steel, non-cellular, composite, galvanized, 3" D, 22 gauge	1.05000	S.F.	2.08	0.98	3.06
Metal decking, steel edge closure form, galvanized, with 2 bends, 12" wide, 18 gauge	0.04500	L.F.	0.18	0.11	0.29
Sprayed fireproofing, cementitious, normal density, beams, 1 hour rated, 1-3/8" thick...	0.52300	S.F.	0.30	0.52	0.82
Total			$11.85	$6.01	$17.86

Figure B.3: RS Means 2012 Composite steel deck cost estimate. Courtesy meansworks.com

The bay size in the assembly cost was 20'x25' while the design performed used approximately a 22'x22' bay size. The loads in this assembly are less than those used in the design. The slab and system depth is slightly larger in this assembly than the design.

B.4: One-Way Slab Assembly

Assembly B10102195100 **Based on National Average Costs**

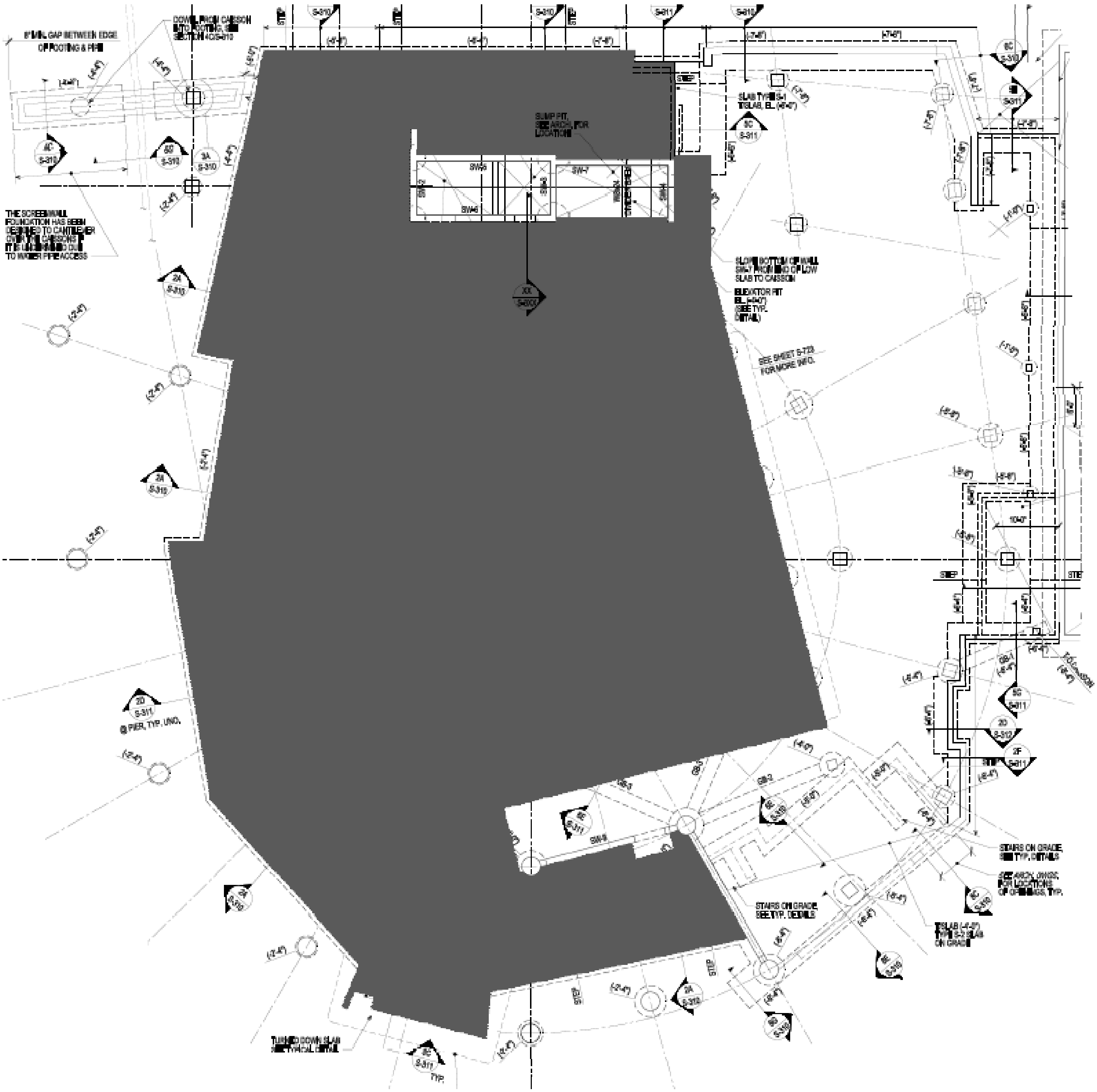
Cast-in-place concrete beam and slab, 5.5" slab, one way, 14" column, 20'x25' bay, 75 PSF superimposed load, 160 PSF total load

Description	Quantity	Unit	Material	Installation	Total
C.I.P. concrete forms, beams and girders, exterior spandrel, plywood, 12" wide, 4 use...	0.15100	SFCA	0.14	1.55	1.68
C.I.P. concrete forms, beams and girders, interior, plywood, 12" wide, 4 use, includes...	0.28700	SFCA	0.31	2.41	2.72
C.I.P. concrete forms, elevated slab, flat plate, plywood, to 15' high, 4 use, includes s...	0.84700	S.F.	0.97	4.79	5.75
Reinforcing Steel, in place, elevated slabs, #4 to #7, A615, grade 60, incl labor for acc...	2.71600	Lb.	1.52	1.17	2.69
Structural concrete, ready mix, normal weight, 3000 psi, includes local aggregate, san...	0.57000	C.F.	2.37	0.00	2.37
Structural concrete, placing, elevated slab, pumped, less than 6" thick, includes strike...	0.57000	C.F.	0.00	0.87	0.87
Concrete finishing, floors, for specified Random Access Floors in ACI Classes 1, 2, 3 an...	1.00000	S.F.	0.00	0.86	0.86
Concrete surface treatment, curing, sprayed membrane compound	0.01000	C.S.F.	0.08	0.09	0.17
Total			$5.40	$11.74	$17.14

Figure B.4: RS Means 2012 One-way slab cost estimate. Courtesy meansworks.com

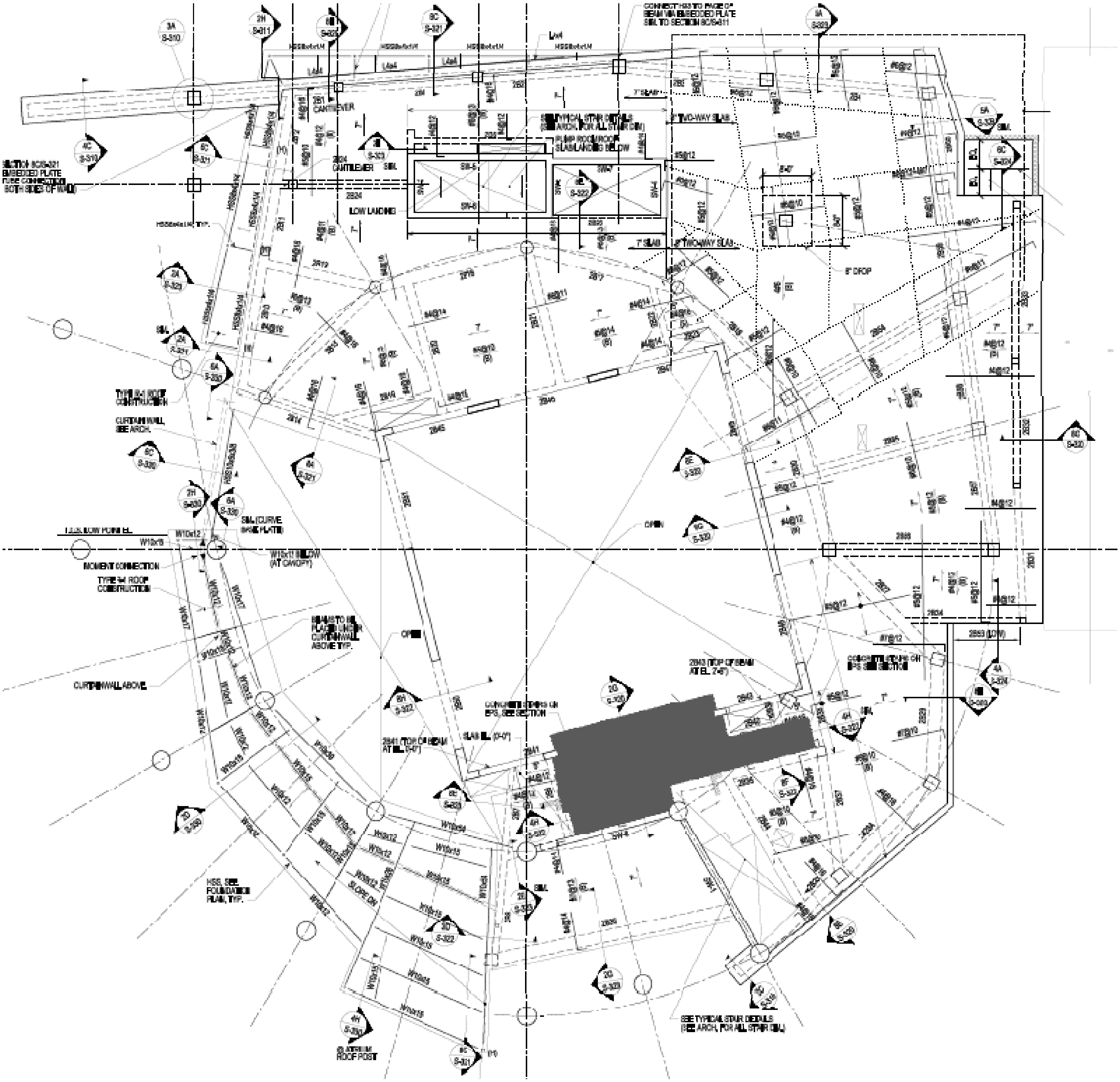
This assembly underestimated the total load used, as well as the slab thickness and column size (14" compared to 24" in the actually structure). The bay size was also different in this assembly than in the actual design.

Appendix C: Structural Plans



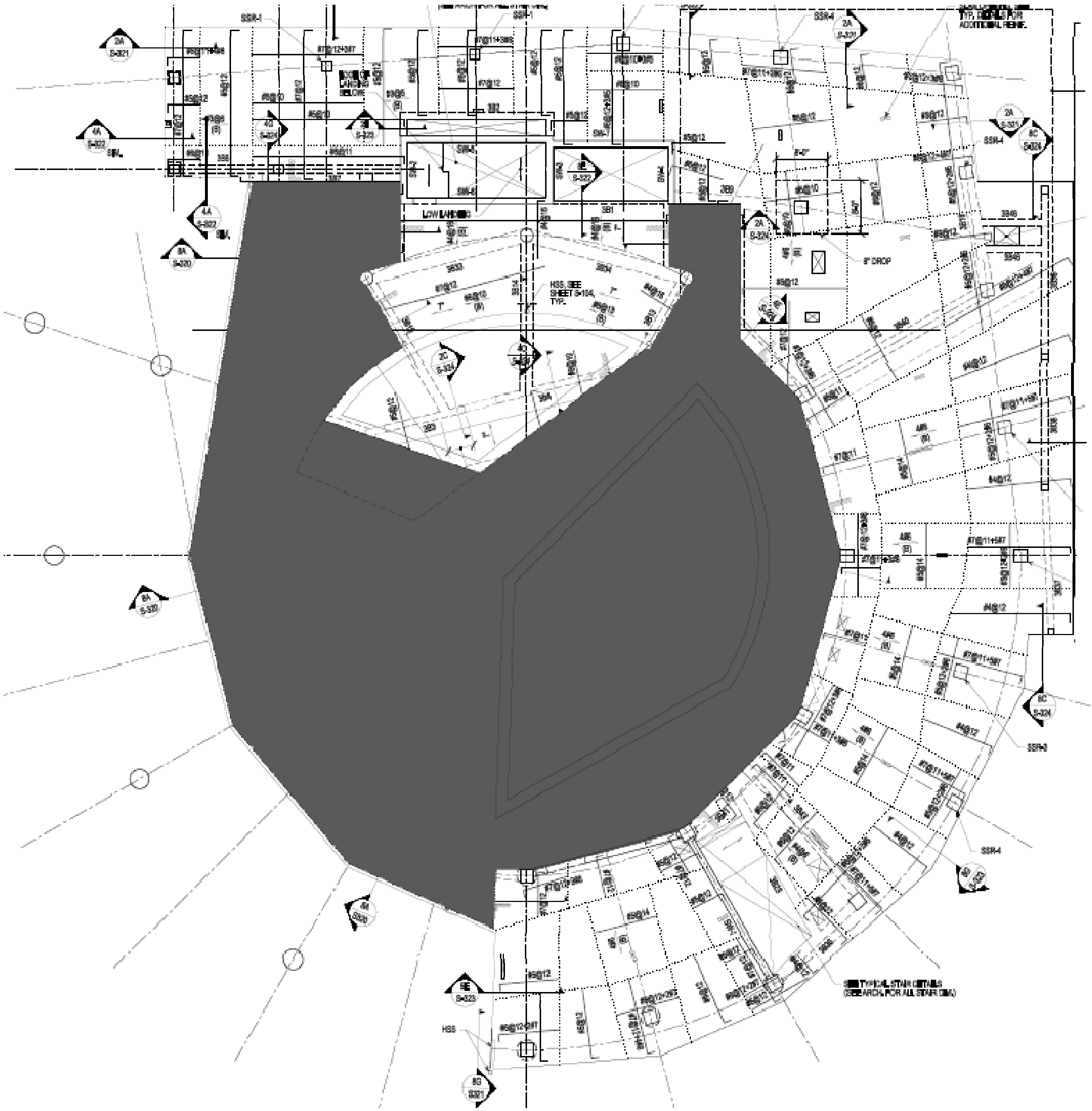
Level 1 Framing Plan– shaded area represents a depressed floor slab

Appendix C: Structural Plans



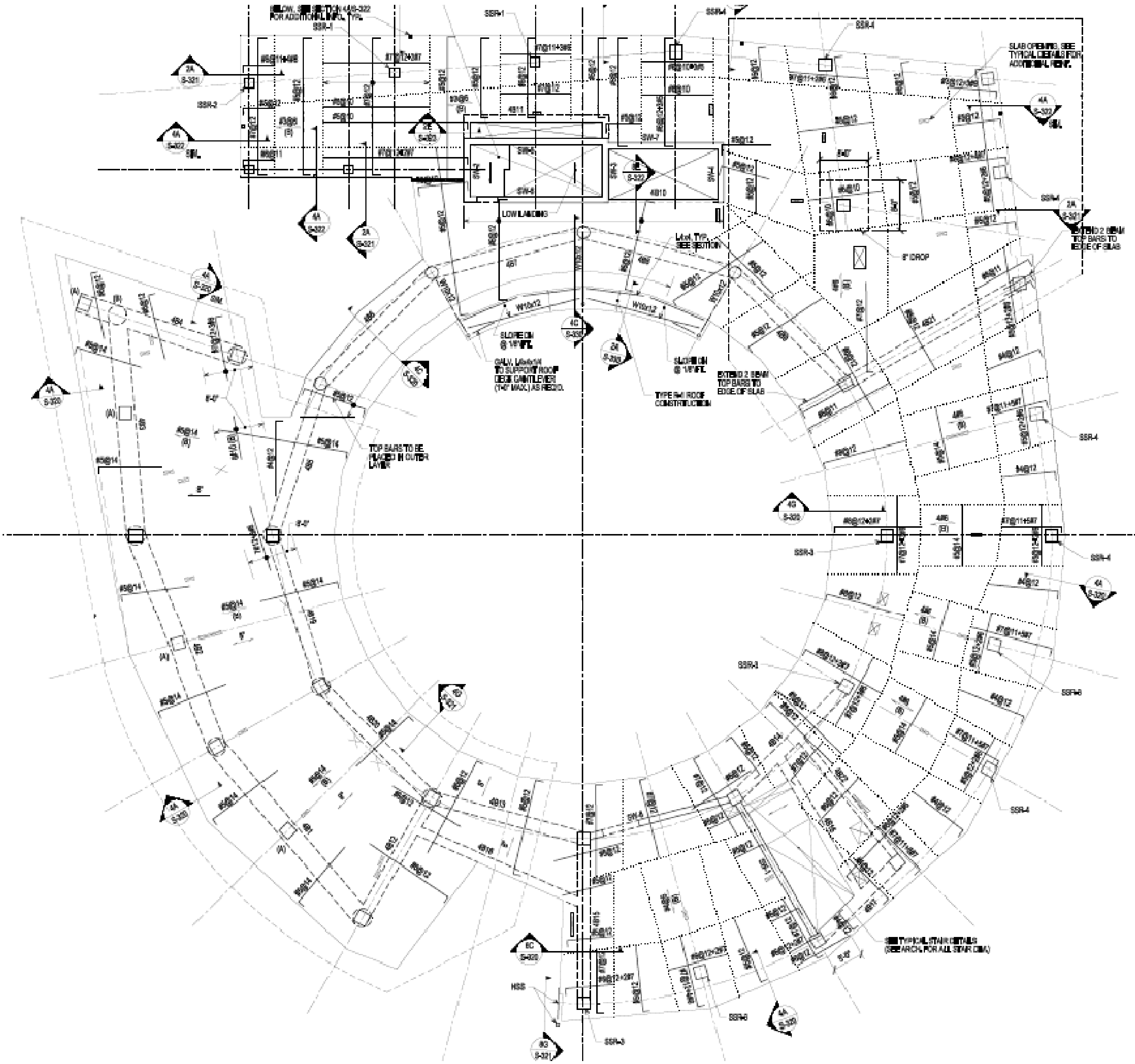
Level 2 Framing Plan

Appendix C: Structural Plans



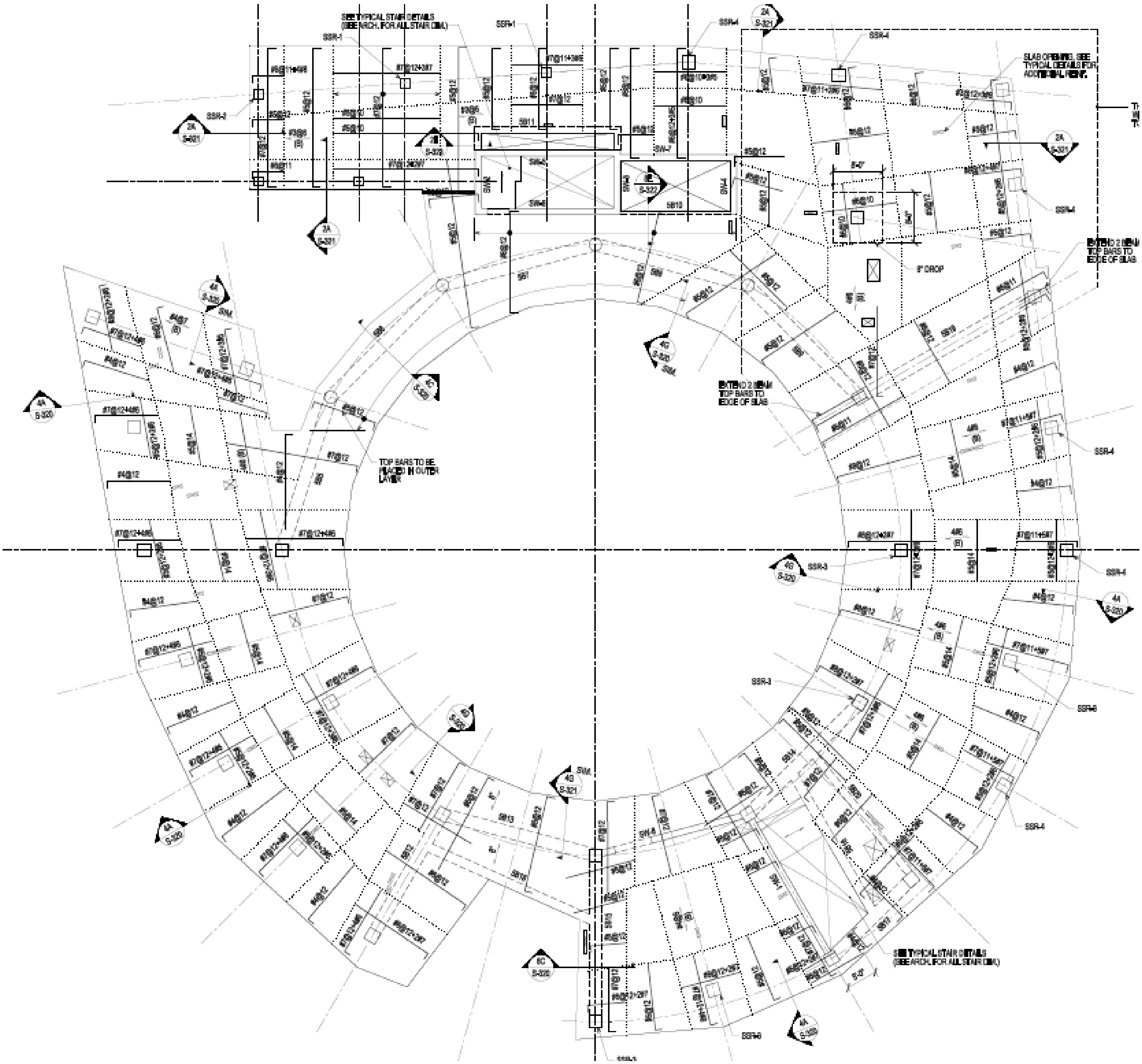
Level 3 Framing Plan– shaded area represents a depressed floor slab

Appendix C: Structural Plans



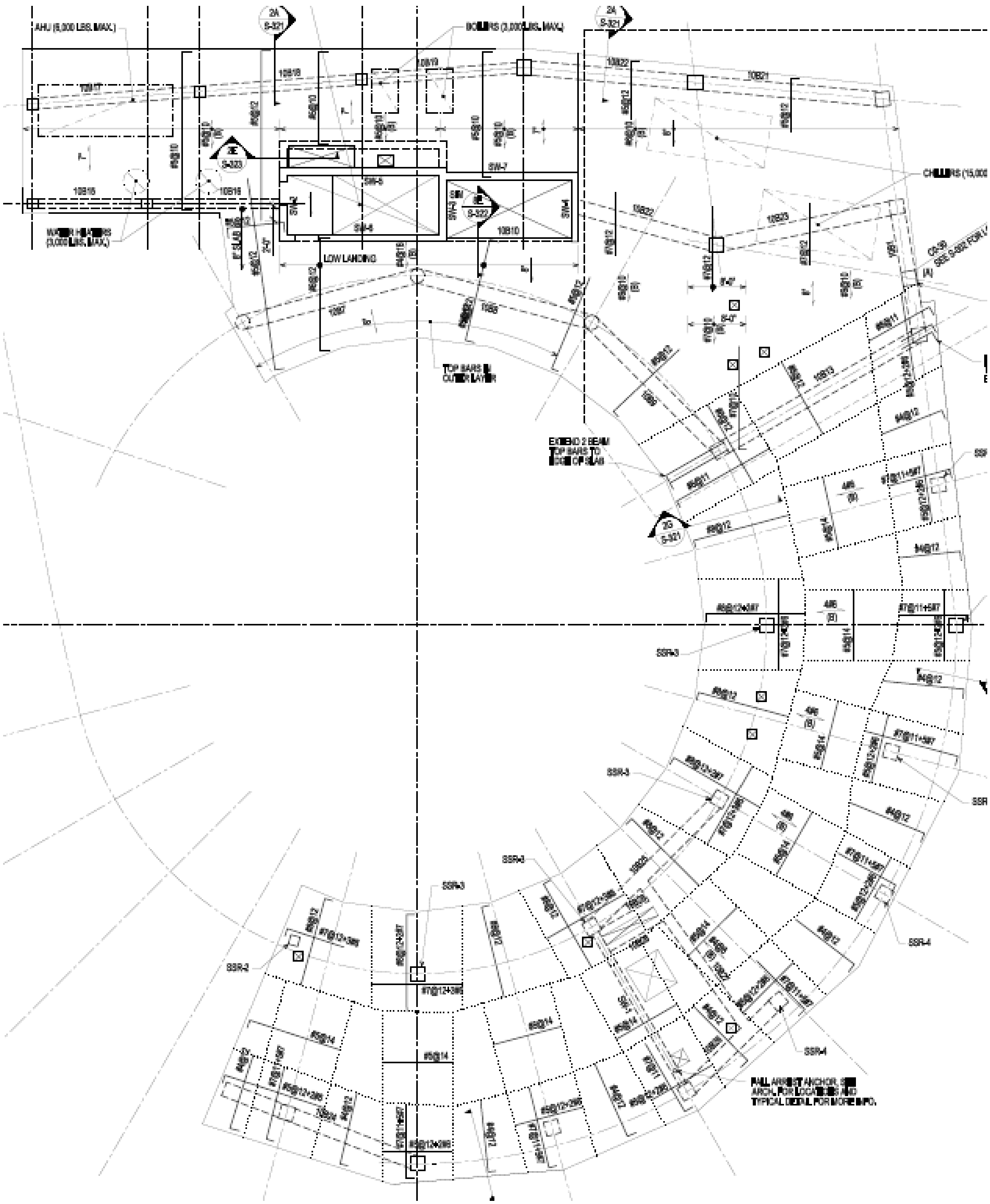
Level 4 Framing Plan

Appendix C: Structural Plans



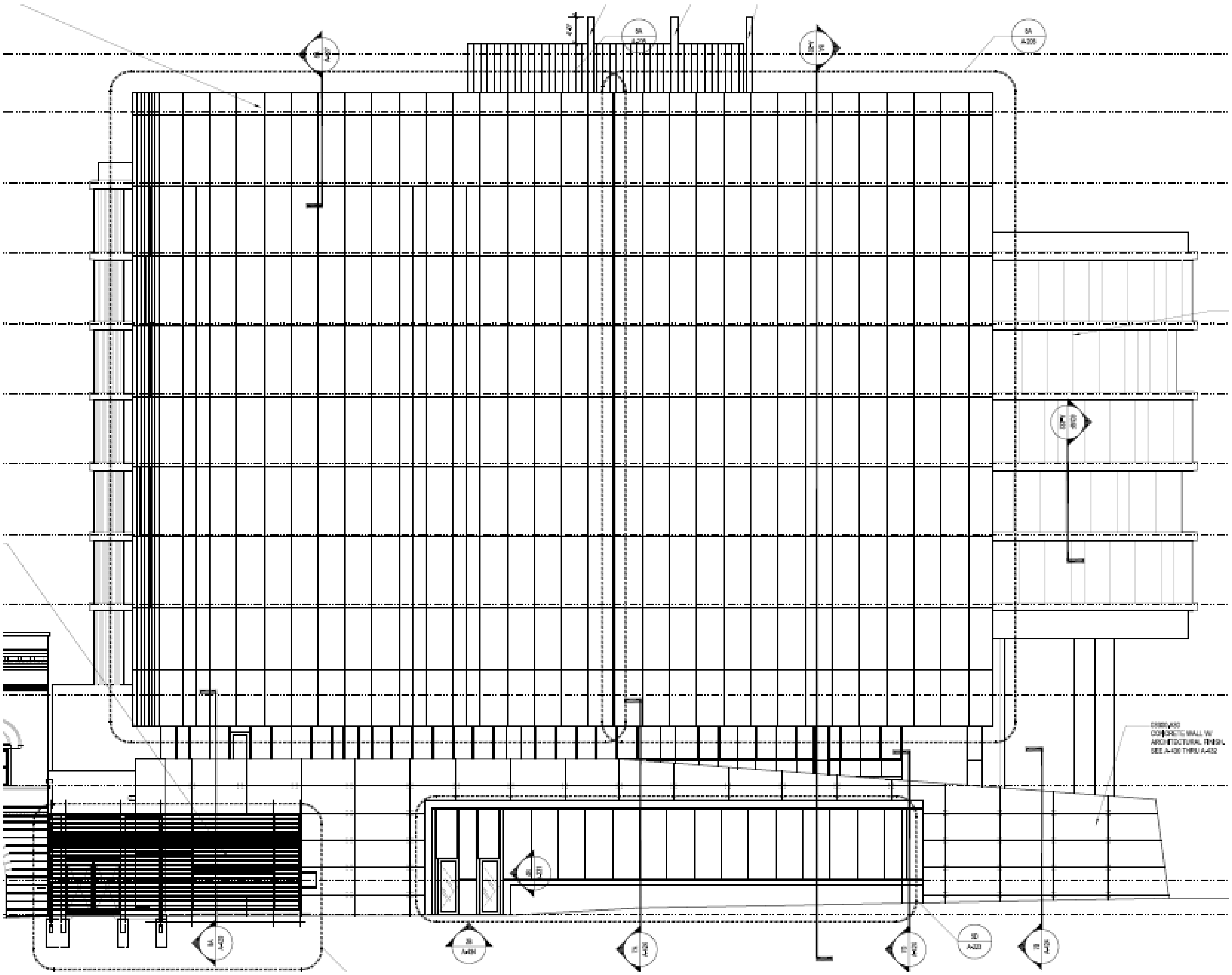
Level 5-9 Framing Plan

Appendix C: Structural Plans



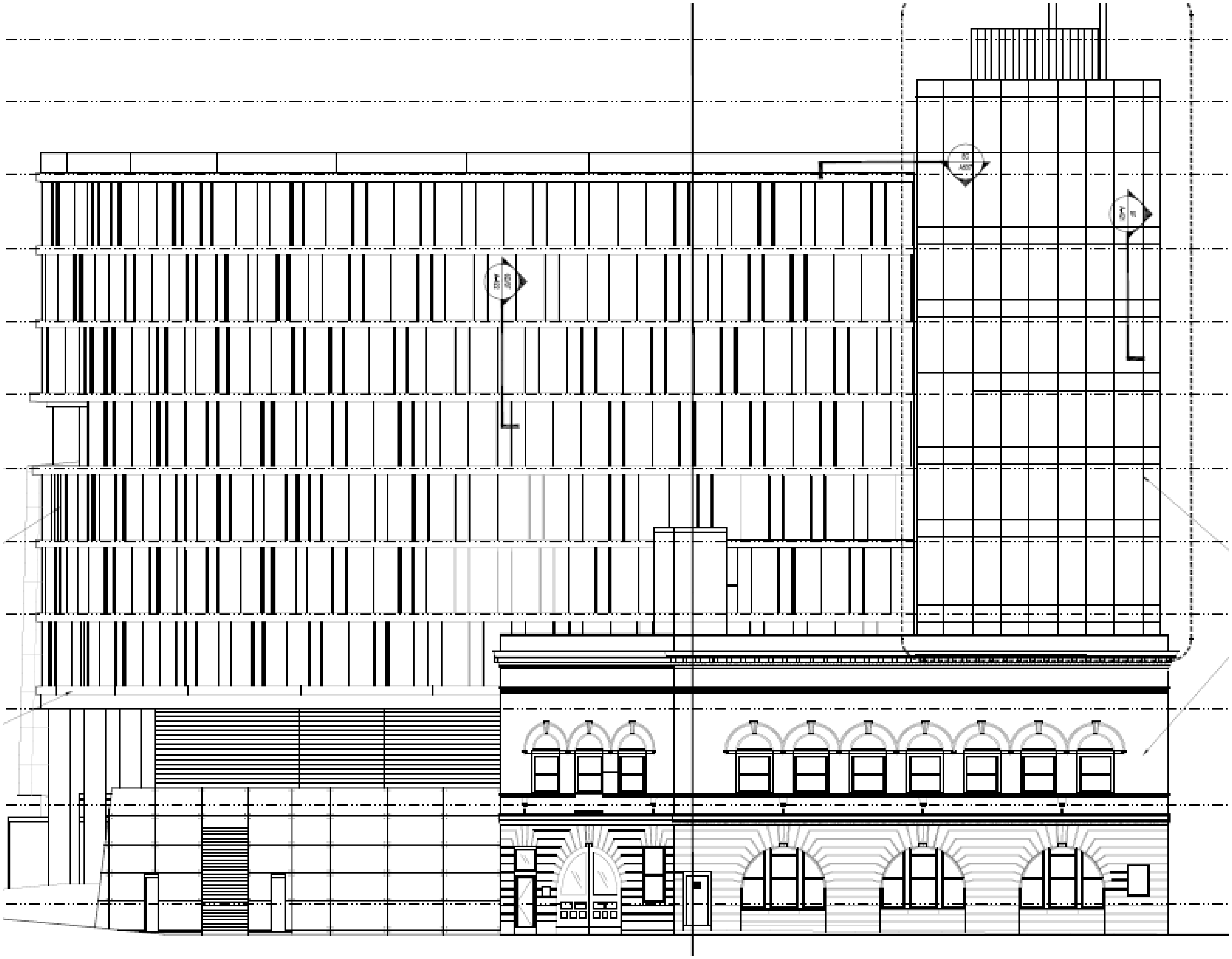
Level 10 Roof Framing Plan

Appendix C: Structural Plans



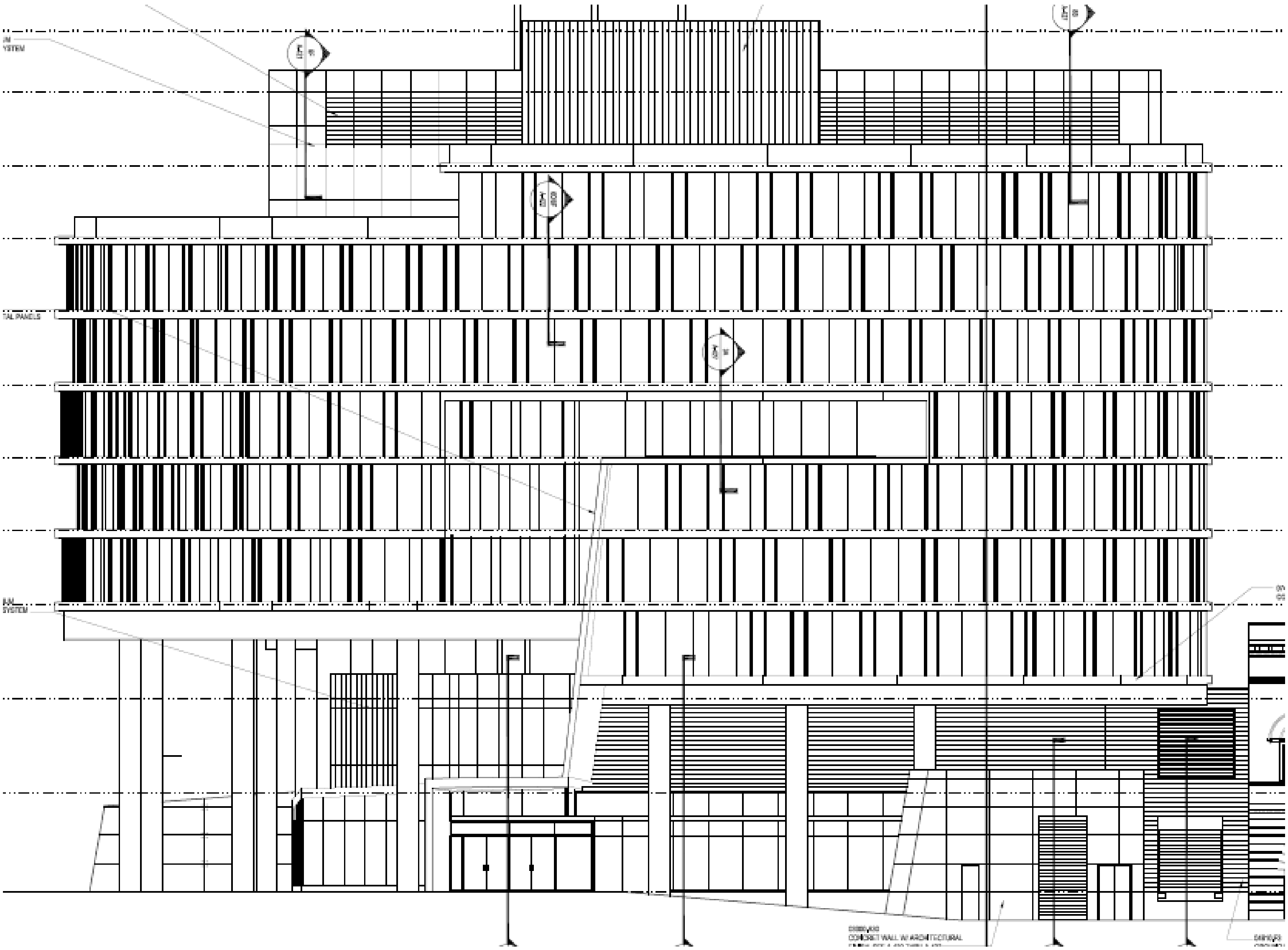
North Building Elevation

Appendix C: Structural Plans



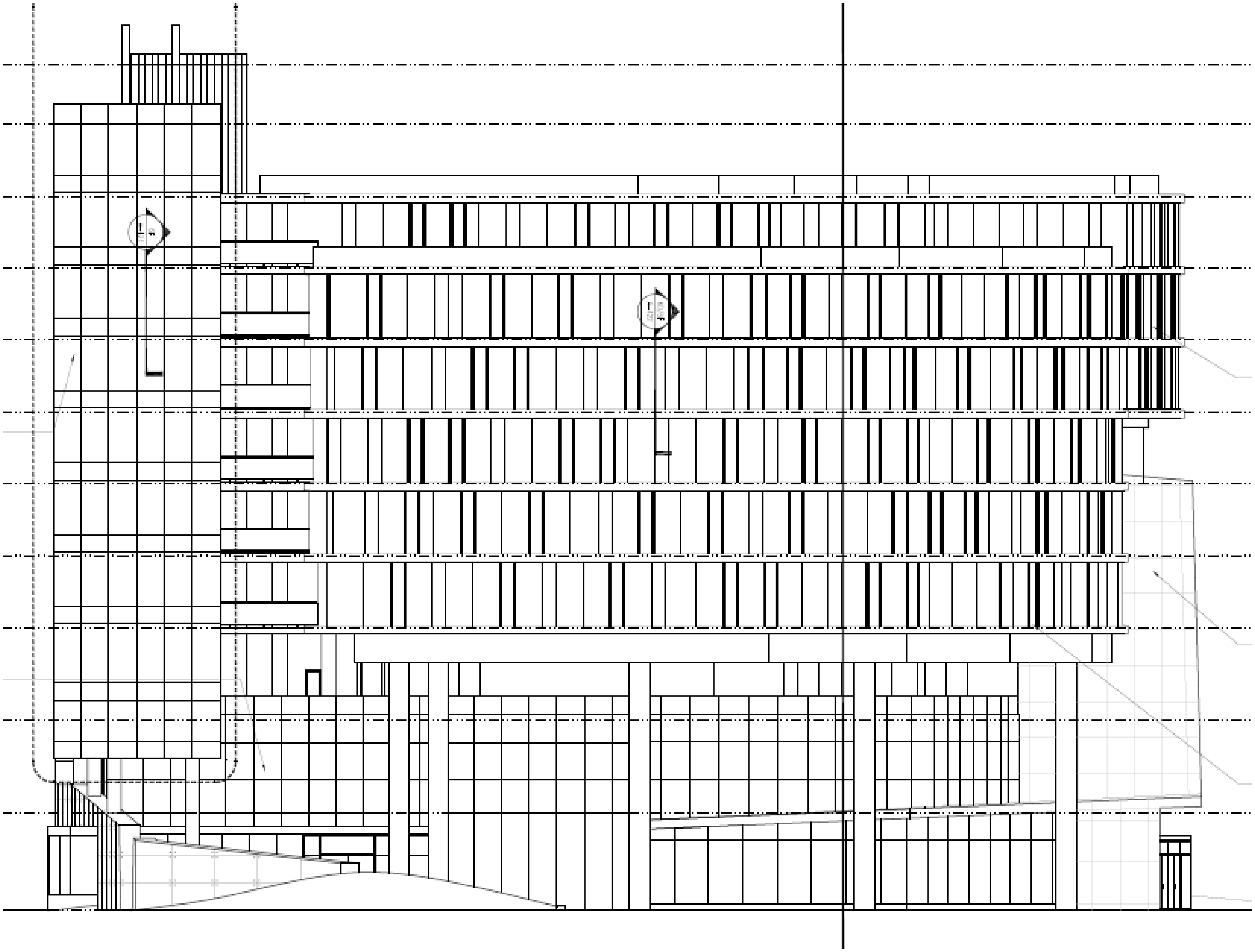
East Building Elevation

Appendix C: Structural Plans



South Building Elevation

Appendix C: Structural Plans



West Building Elevation