ASHA National Office

Rockville, MD

2011 AE Senior Thesis
Final Report

Photo Courtesy of Boggs & Partners Architects

Ryan Dalrymple
Structures Option

Advisor: Dr. Thomas Boothby

ASHA National Office

Rockville, MD

Ryan Dalrymple Structures Option

Size

133,870 sq. ft.

Number of Stories

5 stories above grade

2 stories of underground parking

Dates of Construction

April 2006 – December 2007

Project Cost

$48,000,000

Project Delivery Method

Design-Bid-Build

Project Team

Owner

ASHA

Development Manager

AtSite Real Estate

Construction Manager

Davis Construction

Architect

Boggs & Partners Architects

Structural Engineer

Cagley & Associates

M.E.P. Engineers

Vanderweil Engineers

Architecture

The ASHA National Office building was designed with the employees in mind. There is a generous amount of workspace for the workers and the meeting rooms are very flexible. The meeting rooms have adjustable partitions and movable furniture so that they can be altered to accommodate any type of meeting or event that is held. On the first floor of the office building there is a kitchen, café and gym for the employees to use throughout the work day.

Structural System

Two floors of parking make up the substructure of the ASHA National Office. The parking structure is composed of a two way flat slab concrete system that is comprised of a 9" thick slab and 5 ½" thick drop panels. The steel framing for the five story office tower consists of steel columns and beams with a composite concrete floor slab on metal deck. The composite slab consists of 3 ½" normal weight concrete on top of 2" deep metal deck.

Mechanical System

The mechanical system is powered by two 200 ton chillers with variable frequency drives. These are located in the chiller room on level B2. There is a variable air volume air handler on each floor of the building. Series fan-powered variable air volume terminal units provide air to all occupied spaces. The terminal units on the perimeter of the building have heating coils to provide heat to those spaces.

Lighting/Electrical System

The electrical system used to power the AHA National Office Building is a 277/480 Volt 3-Phase 4-Wire conduit system. A 300 kW diesel-fueled emergency generator is located outside. Interior lighting is mostly fluorescent type lighting fixtures. The lighting in the parking garage is provided by HID fixtures.

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Executive Summary

The ASHA National Office building is an office building located in Rockville, MD. The office tower is five stories and there are two floors of subgrade parking. The parking structure is composed of a flat slab system with drop panels and the superstructure is composite steel. The lateral system consists of four braced frames in the office tower with shear walls in the subgrade parking garage. The gross area of the building is 133,870 square feet.

The goal of this thesis was to redesign the structural system of the office tower as reinforced concrete. Using reinforced concrete would eliminate the need for the baseplates and anchor bolts that are needed to connect the steel office tower to the concrete parking structure below. By designing the entire structure as a reinforced concrete structure, the issue of connecting the steel office tower structure to the concrete parking structure below will be eliminated. In addition, the continuity of the concrete structure will create natural moment connections. The concrete structure will also eliminate the need for spray fire proofing. Reinforced concrete does not require any additional fire proofing treatments which will help reduce the cost of the structure.

Two different concrete floor systems were considered for this thesis redesign. The first floor system that was considered was a two-way flat slab system with drop panels, and the second is a one-way slab and beam system. Both systems were modeled and designed using SPBeam. Due to the irregular shape of the floor plan of the office tower, all column lines had to be modeled. It was determined that the two-way flat slab system would be slightly cheaper, but would create limitations on floor plan flexibility due to the additional columns that are required for this system. For this reason, the one-way slab and beam system was ultimately chosen.

After the structure was designed for the gravity loads, multiple checks were done to determine if the inherent moment connections of the reinforced concrete structure were adequate to resist the lateral loads on the building. ETABS was used to create a computer model of the office building, which was used to analyze the building for the lateral loads. If the structure did not meet these requirements, then shear walls would have to be implemented in the structure of the office tower. It was ultimately determined that the inherent moment connections of the concrete structure are adequate to resist the lateral loads, and shear walls are not needed for the office tower.

A study that explores the architectural affects of changing the structure to concrete was done. If the two-way flat slab system was chosen, it would require the need for two more column lines. The impact of these additional columns on the open office floor space was considered, and the plaza level floor plan was redesigned to accommodate these extra columns. A cubicle layout was also created for part of the plaza level.

A cost estimate and construction schedule was created for the redesigned concrete structure, and compared with the existing steel structure. It was determined that the existing steel structure is cheaper and the construction time is less than the redesigned concrete structure.

Acknowledgements

I would like to thank the American Speech-Language-Hearing Association for allowing me to use the ASHA National Office building for my thesis project.

I would like to thank the following companies and people for providing me with all of the documents and reports needed to complete this project:

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Introduction

The ASHA National Office building is a five story office building in Rockville, MD. The American Speech-Language-Hearing Association owns and operates the building. The building was designed with the employees in mind. There is a generous amount of workspace for the employees and the conference rooms are very flexible. A café and kitchen are provided for the employees on the first floor of the office building. There are two levels of subgrade parking beneath the building in addition to surface parking. There are 201 parking spaces in the subgrade parking structure and 224 spaces above grade.

One of the main architectural themes that Boggs & Partners incorporated throughout the building is curves. This was done to mimic the sound waves in the ASHA logo which is shown below. The pre-function space has the curve incorporated into it, and there is a curved piece of art on the landing of the stairway that leads from the lobby to the second floor. The exterior façade has a large three story curved glass curtain wall above the main entrance, and the sidewalks on the exterior of the building are curved as well to further emphasize the main theme of the building.

The five story office building has a total floor area of 133,870 square feet and the roof the building is 69 feet above grade. The top of the penthouse roof is 85 feet above grade. The building façade of the office tower consists of a window wall system and precast concrete spandrels.

American Speech-Language-Hearing Association

www.asha.org

Structural System

Substructure

The substructure of the ASHA National Office building is comprised of two floors of subgrade parking. There is parking underneath the office tower along with a section of the parking structure that is adjacent to the office tower. See Figure 1: Overall Parking Floor Plan. The parking below the office tower is shown in blue and the parking adjacent to the office tower is shown in yellow.

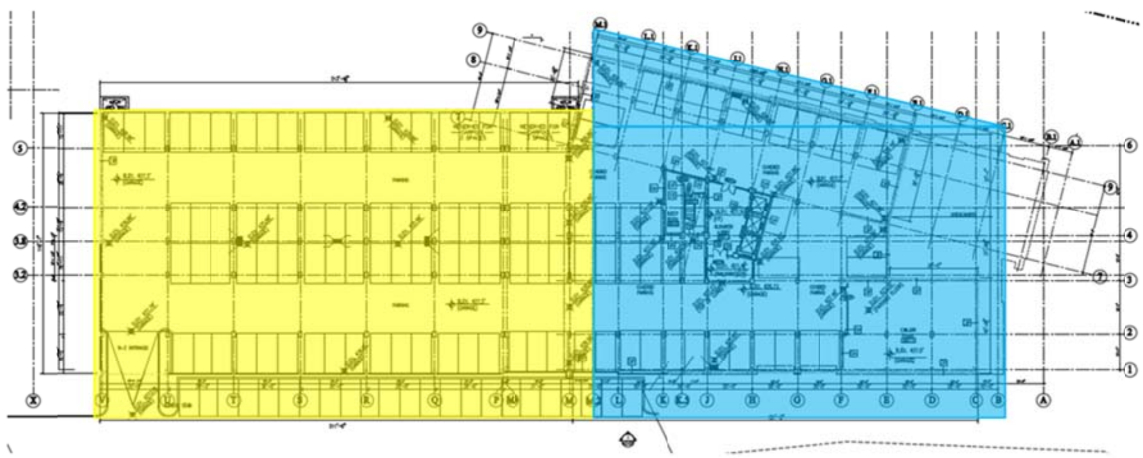


Figure 1: Overall Parking Floor Plan

Foundation

The foundation of the ASHA National Office building consists of a 5" thick reinforced concrete slab with strip footings around the perimeter of the building. There are also footings at the base of all concrete columns. The foundations for the building were designed in accordance with the recommendations included in the geotechnical report prepared by ESC Mid-Atlantic, LLC. See Figure 2: Partial Foundation Plan. The interior column footings are generally 6'x6' and range from 12" to 18" thick. See Figure 3: Column Footing Schedule.

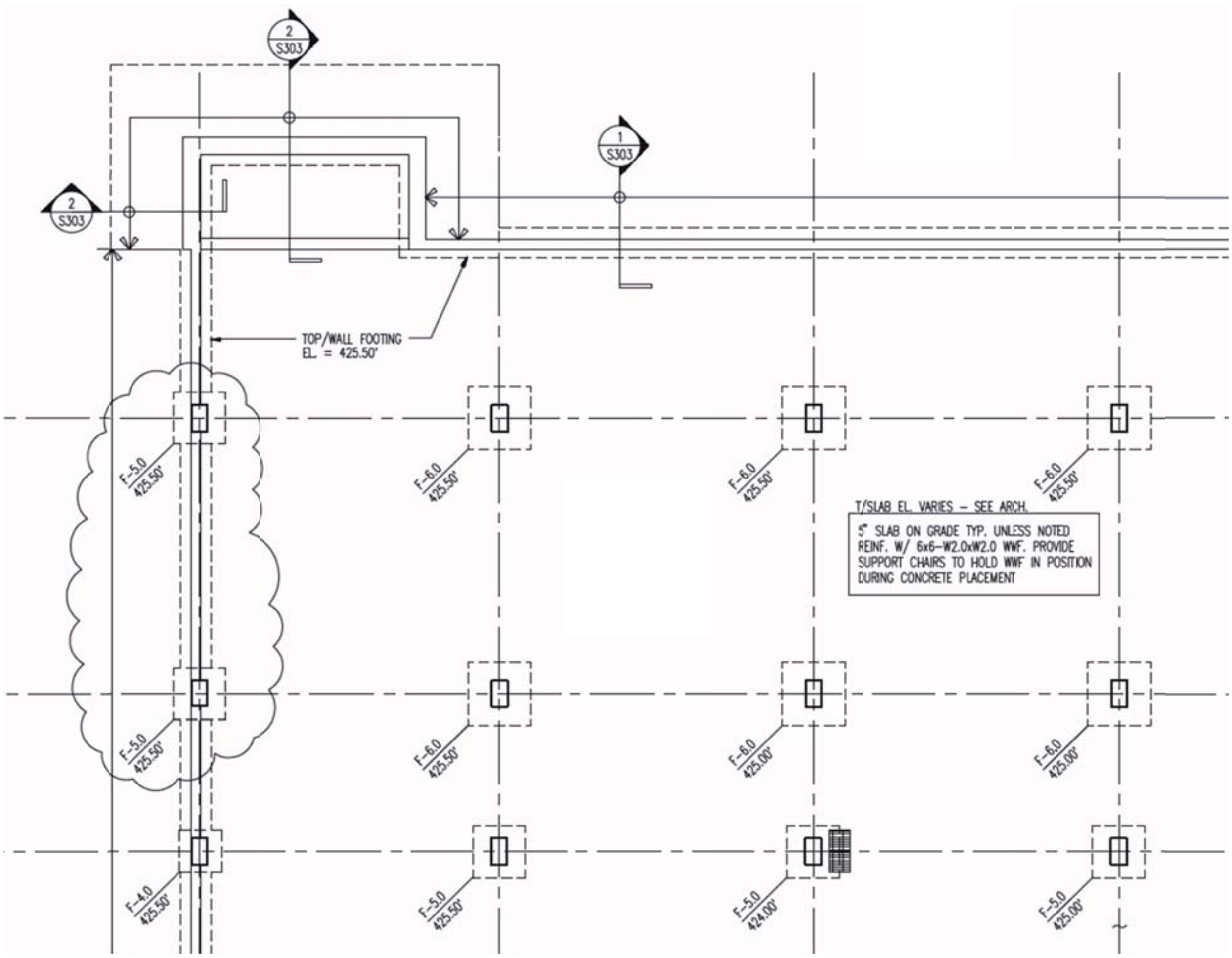


Figure 2: Partial Foundation Plan

COLUMN FOOTING SCHEDULE					
MARK	DIMENSIONS			REINFORCEMENT	REMARKS
	WIDTH	LENGTH	DEPTH		
F-4.0	4'-0"	4'-0"	12"	5#5 EWB	
F-4.5	4'-6"	4'-6"	15"	6#5 EWB	
F-5.0	5'-0"	5'-0"	15"	6#6 EWB	FOR F5.0A-SEE 2/S301 FOR F5.0B-SEE 3/S301
F-5.5	5'-6"	5'-6"	18"	7#6 EWB	
F-6.0	6'-0"	6'-0"	20"	8#6 EWB	FOR F6.0A-SEE 2/S301
F-7.0	7'-0"	7'-0"	24"	7#7 EWB	
F-7.5	7'-6"	7'-6"	26"	8#7 EWB	
F-8.0	8'-0"	8'-0"	27"	10#7 EWB	
F-8.5	8'-6"	8'-6"	29"	10#7 EWB	
F-9.0	9'-0"	9'-0"	30"	9#8 EWB	
F-9.5	9'-6"	9'-6"	31"	10#8 EWB	
F-10.0	10'-0"	10'-0"	33"	11#8 EWB	
F-10.5	10'-6"	10'-6"	36"	12#8 EWB	
F-11.0	11'-0"	11'-0"	36"	13#8 EWB	
F-3.0x8.0	3'-0"	8'-0"	18"	4#6 LWB 11#6 SWB	SEE PLAN FOR ORIENTATION

ABBREVIATIONS: EWB = EACH WAY BOTTOM EWT = EACH WAY TOP
SW = SHORT WAY LW = LONG WAY

NOTE: ALL FOOTINGS ARE DESIGNED FOR 8 KSF ALLOWABLE BEARING UNLESS OTHERWISE NOTED.

Figure 3: Column Footing Schedule

Floor Structure

The parking structure is a two way reinforced concrete flat slab system that is comprised of a 9" thick slab and 5 ½" thick drop panels. Unless otherwise noted on the plans, the drop panels are 7'-0"x9'-0" and 10'-0"x10'-0". The bay sizes vary depending on the part of the building, but the typical span ranges from 20' to 40'. The bottom reinforcing mat consists of #5 bars at 12" or 14" each way. The top reinforcing bars vary depending on the location, but are typically #5, #6 or #7 bars. See Figure 4: Parking Level Framing Plan.

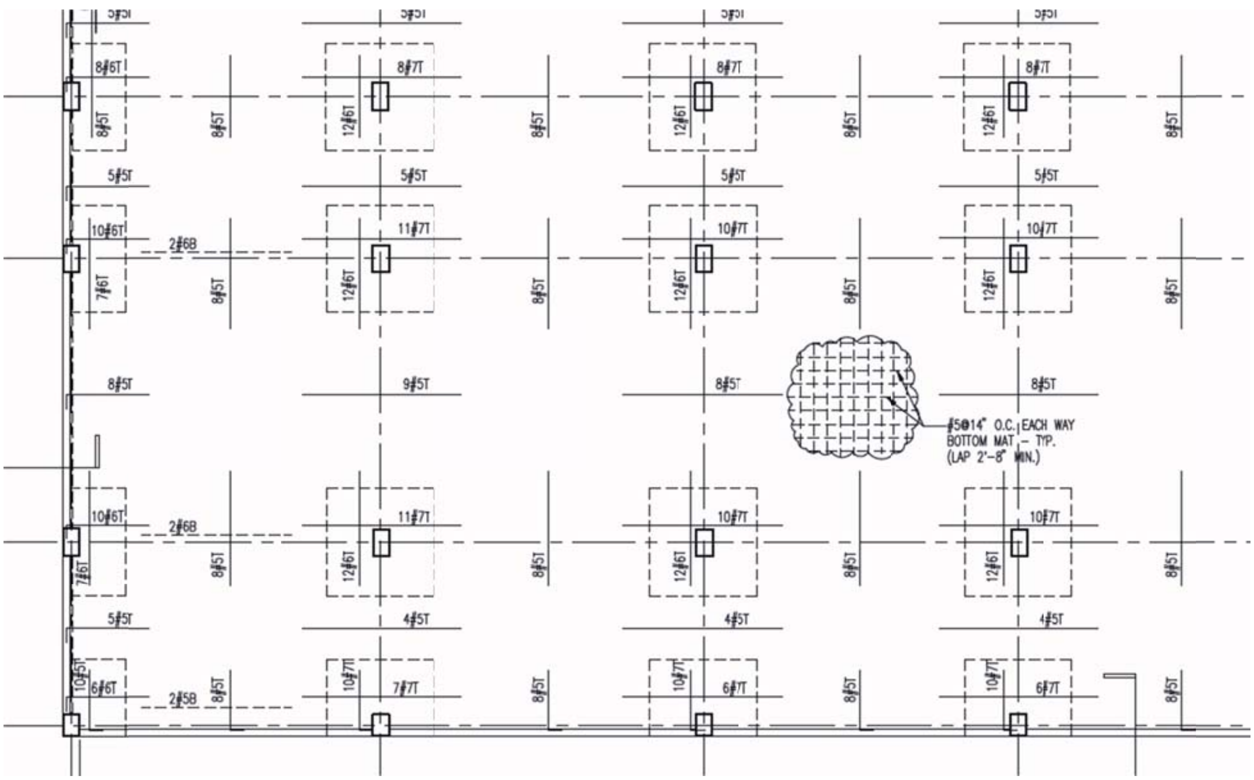
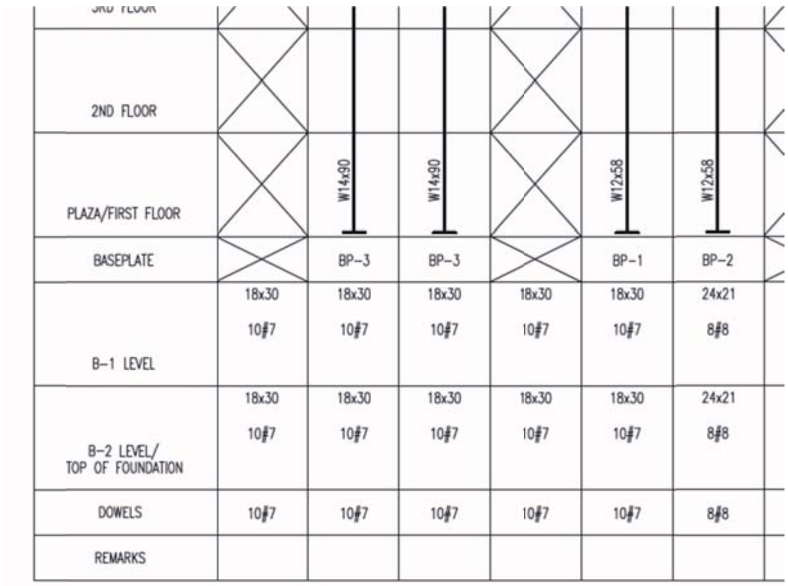


Figure 4: Parking Level Framing Plan

<u>Columns</u>

The concrete columns in the parking structure are generally 18”x30” with 10 #7 bars, and 24”x21” with 8 #8 bars. The columns have a minimum 28 day compressive strength of 4000 psi. See Figure 5: Partial Column Schedule. The concrete columns of the parking structure are connected to the steel columns in the office tower above with column base plates. See Figure 6: Baseplate Pocket Detail.



2ND FLOOR						
PLAZA/FIRST FLOOR		W14x90	W14x90		W12x58	W12x58
BASEPLATE		BP-3	BP-3		BP-1	BP-2
B-1 LEVEL	18x30 10#7	18x30 10#7	18x30 10#7	18x30 10#7	18x30 10#7	24x21 8#8
B-2 LEVEL/ TOP OF FOUNDATION	18x30 10#7	18x30 10#7	18x30 10#7	18x30 10#7	18x30 10#7	24x21 8#8
DOWELS	10#7	10#7	10#7	10#7	10#7	8#8
REMARKS						

Figure 5: Partial Column Schedule

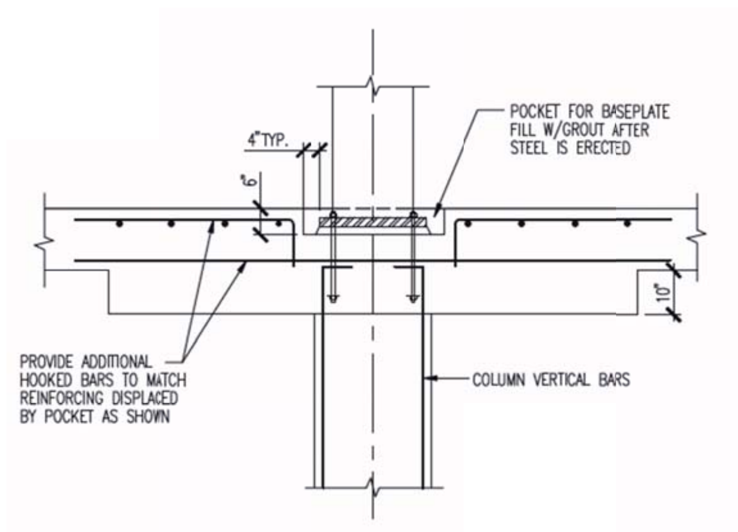


Figure 6: Baseplate Pocket Detail

Superstructure

A five story office tower is the superstructure of the ASHA National Office building. The first level has a large conference room that can be subdivided into five smaller conference rooms. The upper four floors are composed of offices in the central core of the building, and open office space with cubicles on the exterior of the building. There is a penthouse on top of the office tower that houses mechanical and elevator equipment.

Floor Structure

The floor structure for the tower consists of cambered steel beams with a composite concrete floor slab on metal deck. The composite slab consists of 3 ½” normal weight concrete on top of 2” deep 18 gauge galvanized composite steel deck. The composite beams are generally W21x44 and W14x22 members with ¾” diameter shear studs. The girders running along the exterior of the building vary in size, but are mostly W18x35’s. See Figure 7: Partial Framing Plan.

Figure 7: Partial Framing Plan

Columns

The columns for the office tower are steel wide flange shapes. The columns are all W12 and W14 members. The columns are spliced above level 3. The columns that extend to the penthouse roof are spliced again above level 5. See figure 8: Partial Column Schedule.

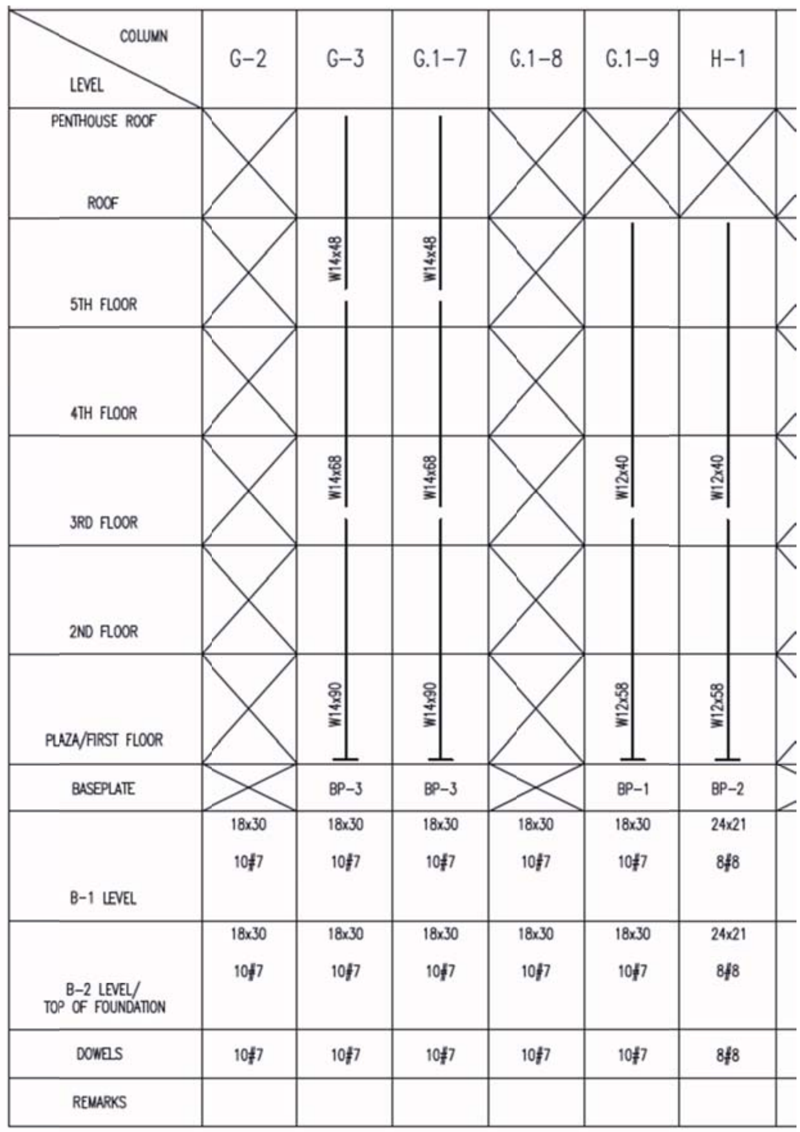


Figure 8: Partial Column Schedule

Roof System

The roof structure consists of K series open web joists and wide flange shapes. The structural roof slab consists of 3 ½" normal weight concrete on top of 2" deep 18 gauge composite steel deck. See Figure 9: Partial roof framing plan.

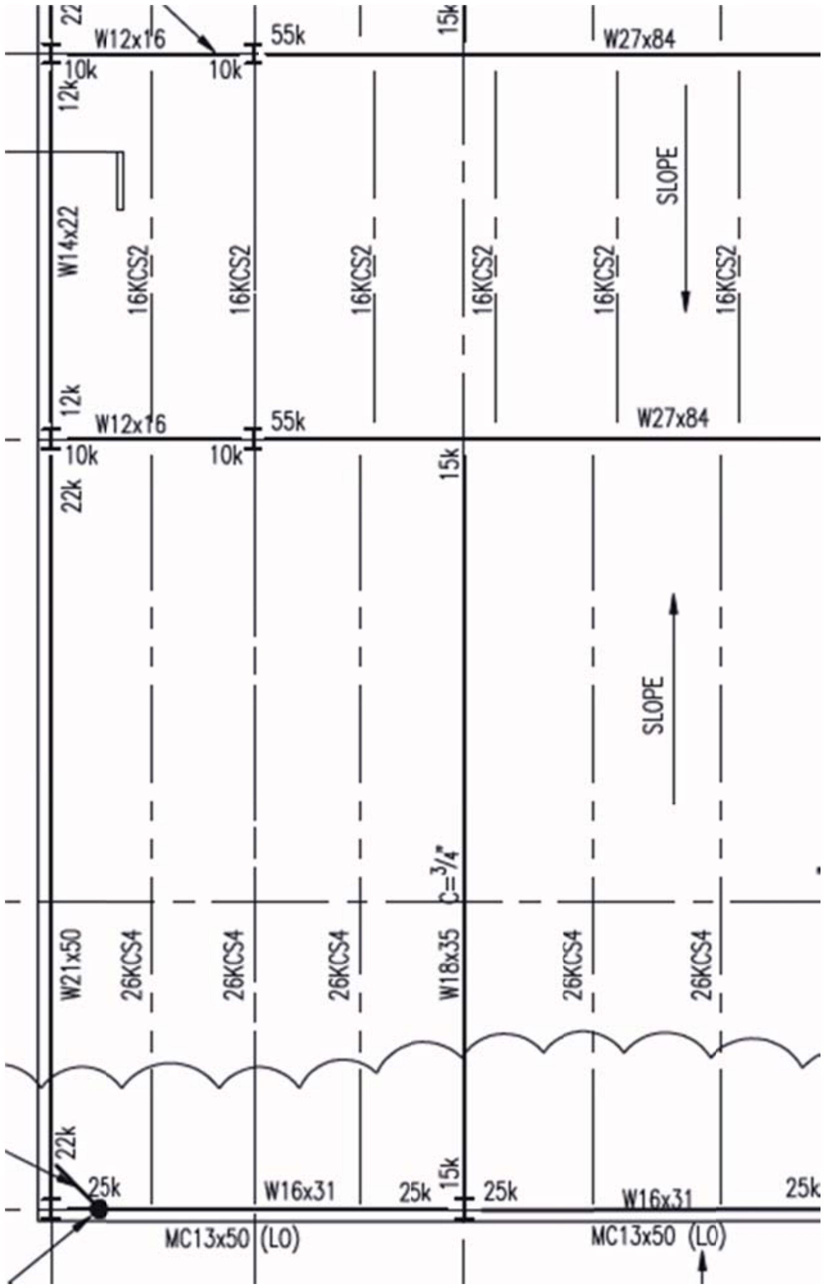


Figure 9: Partial Roof Framing Plan

Lateral System

The lateral force resisting elements in the ASHA National Office building consist of shear walls in the subgrade parking structure of the building and braced frames in the office tower. The shear walls below work in combination with the braced frames above to resist the lateral loads on the building. The wind loads are collected by the precast concrete spandrels that make up the façade of the building. These loads are then distributed to the composite floor slabs and beams which then are transmitted to the braced frames in the core of the building. These loads are then transfered to the shear walls below and to the footings at the base of the shear walls. See figure 10: Braced Frame and Shear Wall Elevation.

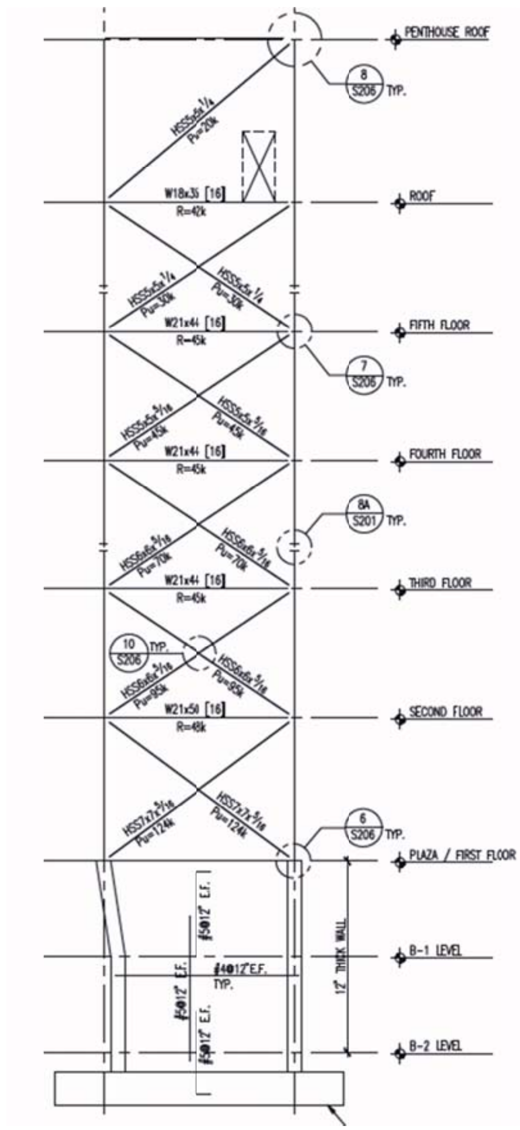


Figure 10: Braced Frame and Shear Wall Elevation

Thesis Objective

Structural Depth

Currently the structure for the subgrade parking garage for the ASHA National Office building is a two-way reinforced concrete flat slab system. The office tower that is above grade has a composite steel structure. The structure was found to be adequate for the gravity and lateral loads on the building, but having both a reinforced concrete system and a composite steel system in the building creates some complications for the design and construction of the building. One issue is that the steel structure above has to be connected to the concrete structure below. In the current design, this is done with baseplates and anchor bolts. These baseplates must be leveled and positioned accurately so that the steel columns are plumb and in the right location. By altering the structural system of the office tower, the cost of the project may be able to be decreased.

The ASHA National office tower will be redesigned as a reinforced concrete structure. Two floor systems will be explored. The first floor system that will be considered is a one-way slab and beam system. The beams will span the 40’ direction and will be wide and shallow to reduce the floor system depth as much as possible. The columns will also be changed from steel W-Flange shapes to reinforced concrete columns. The second system that will be investigated is a two-way flat slab system with drop panels. This type of floor system will be considered because the subgrade parking structure consists of this type of floor system. By continuing this type of floor system in the office tower, the design and construction costs may be reduced.

By designing the entire structure as a reinforced concrete structure, the issue of connecting the steel office tower structure to the concrete parking structure below will be eliminated. In addition, the continuity of the concrete structure will create natural moment connections. The concrete structure will also eliminate the need for spray fire proofing. Reinforced concrete does not require any additional fire proofing treatments which will help reduce the cost of the structure. It will be determined if a reinforced concrete office tower is an economical option when compared to a composite steel structure.

By changing the design of the structure to reinforced concrete, the lateral system will have to be completely changed. If the inherent moment connections in the reinforced concrete structure are not adequate to resist the lateral loads, then concrete shear walls may have to implemented. The heavier weight of the structure will also increase the seismic loads on the building.

The impact on the foundation will also have to be considered. Because the structure will be redesigned as a reinforced concrete structure, the weight of the building will increase resulting in higher loads on the lower parking structure and foundation below. This will most likely require the size of the foundations to be increased. A spread footing will be redesigned for the higher dead loads in order to determine the cost and schedule impact of the larger foundations.

Breadth Studies

Redesigning the structure as reinforced concrete will affect the cost and schedule of the project. For this reason, an in-depth study will be done on the cost and schedule impacts of redesigning the structure. The overall cost and a construction schedule will be determined for the concrete structure. The cost and schedule of the redesigned concrete structure will be compared to that of the existing composite steel structure, and feasibility of the redesign will be determined.

Another study that explores the architectural affects of changing the structure to concrete will be done. The two-way flat slab system will require the need for two more column lines. The impact of these additional columns on the office tower floor plan will be considered. The plaza level will be the floor that is most affected by these additional columns. This is due to the conference rooms that are located on this level. The floor plan will be rearranged in order to work with the new structural layout. Research on cubicle sizes will be done, and a cubicle layout will be created for part of the plaza level.

MAE Requirement

The MAE requirement for this class was met by utilizing ETABS to create a computer model of the building. The model was used to analyze the building under the lateral loads. By generating and utilizing the ETABS computer model of the building, the course material taught in AE 597A was directly applied to this thesis project.

Structural Depth

Floor System Comparison

Two concrete floor systems were explored for this thesis project. The first floor system that was considered was a one-way slab and beam system. This floor system was considered because of the long 40' spans in the office tower. The second floor system that was considered was a two-way flat slab system with drop panels. This system was considered because this is the type of floor system that is used in the parking structure below. In order for the flat slab system to work, additional columns had to be added.

Both floor systems were designed using StructurePoint software. Because of the irregular shape of the floor plan of the building, every column line had to be modeled. For the one-way slab and beam system, all column lines in the north-south direction had to be modeled in spBeam. For the two-way flat slab system, all column lines in both directions had to be modeled using spSlab. The minimum slab thickness and drop panel sizes were determined using ACI 318-08. These calculations can be seen in Appendix A. It was determined that the drop panel thickness is 4 ¼" and are generally 9'x7'. Some drop panels were increased to 10'x7' at the center of the floor plan due to the longer spans. The floor system was designed using a concrete compressive strength of 5000 psi to be consistent with the parking garage below. The spSlab software was then used to determine the amount of reinforcing that is required for the two-way system. Column lines 1 through 9 in the east-west direction, and column lines B through M were modeled in the north-south direction. Figures 11 and 12 show a top view and a 3D view of the spSlab model of column line C. Edge beams were also designed on the exterior of the building to support the precast concrete spandrel façade. The spSlab output that shows the middle strip and column strip reinforcing for column line C is shown in Figure 13. The spSlab models and the required reinforcing for all other column lines are shown in Appendix B.

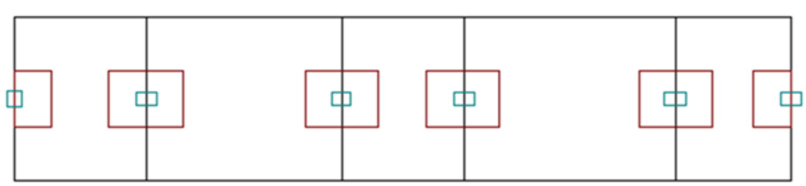


Figure 11: spSlab Column Line C: Top View

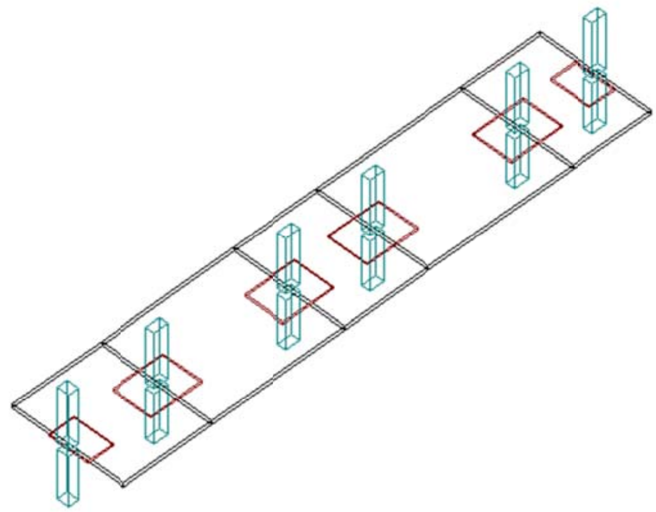


Figure 12: spSlab Model Column Line C: 3D View

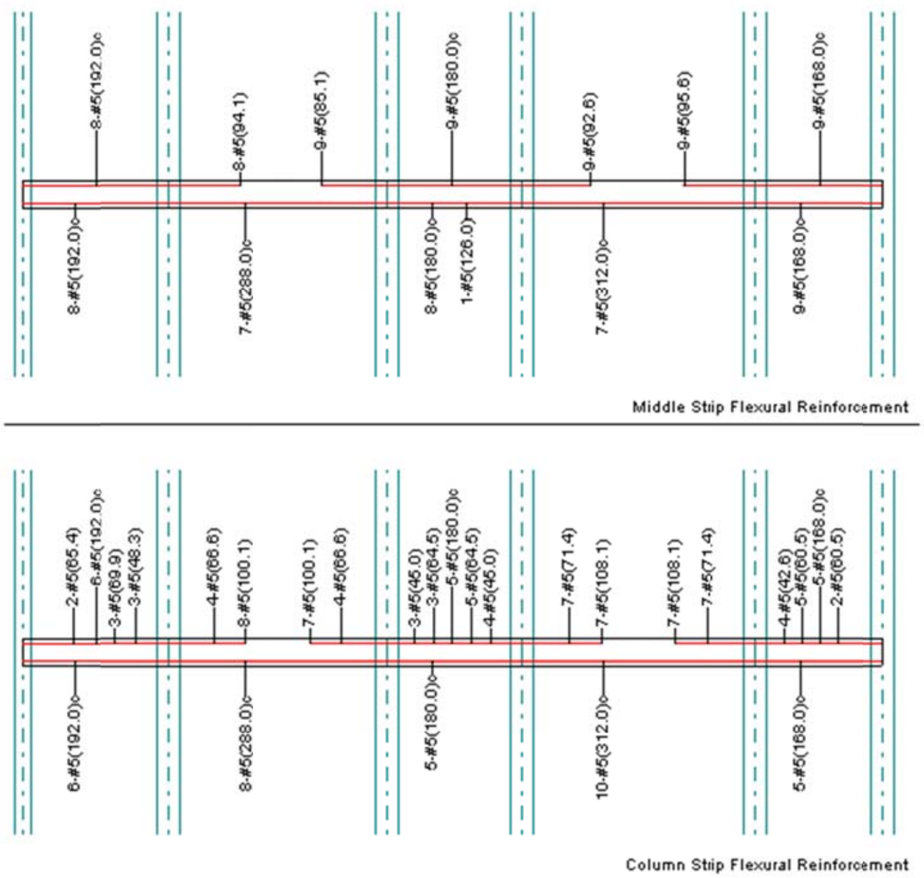


Figure 13: spSlab Column Line C Reinforcing Diagram

The one-way slab and beam system was designed using spBeam. The one-way slab was designed to run in the east-west direction and the beams were designed to run in the north-south direction. Similar to the flat slab floor system, compressive strength of the concrete beams and slabs was designed to be 5000 psi to be consistent with the parking garage below. In order to determine an economical beam size for the 40' span, a simple cost analysis was done. The calculations can be seen in Appendix A. It was determined that the most economical beam is one that is 18" wide by 26" deep. Additional edge beams were included in the floor design to support the façade that consists of precast concrete spandrels. Figures 14 and 15 show a top view and 3D view of the spBeam model of the beams in column line C. Figure 16 shows the reinforcing diagram for the beams in column line C. The spBeam models and reinforcing diagrams for the beams for all other column lines are shown in Appendix C.

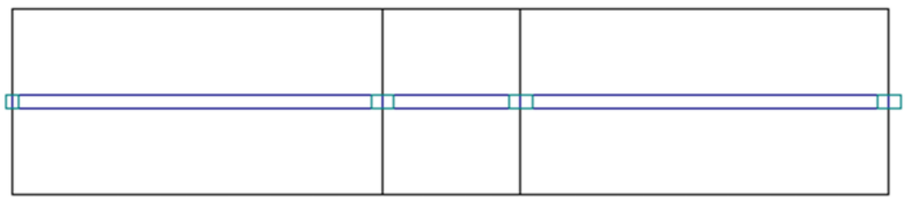


Figure 14: spBeam Model Column Line C: Top View

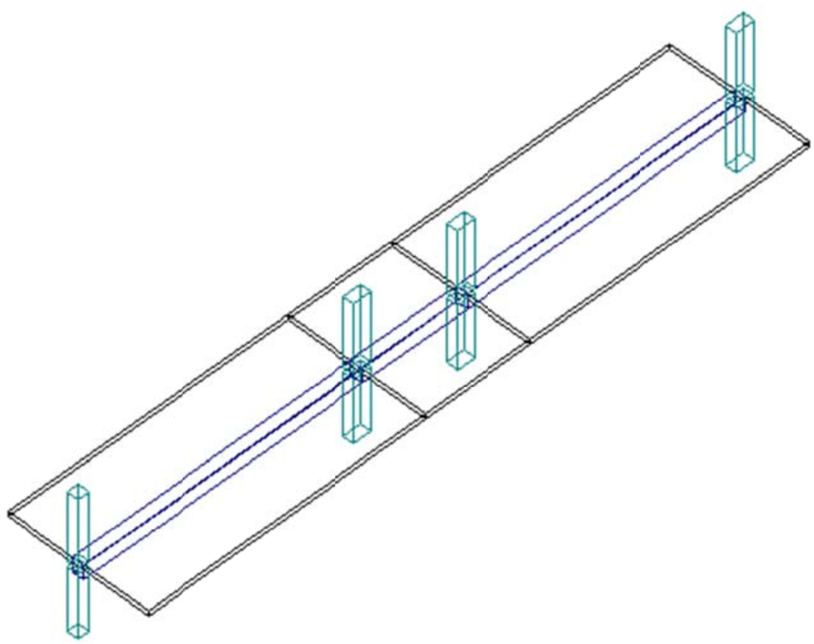


Figure 15: spBeam Model Column Line C: 3D View

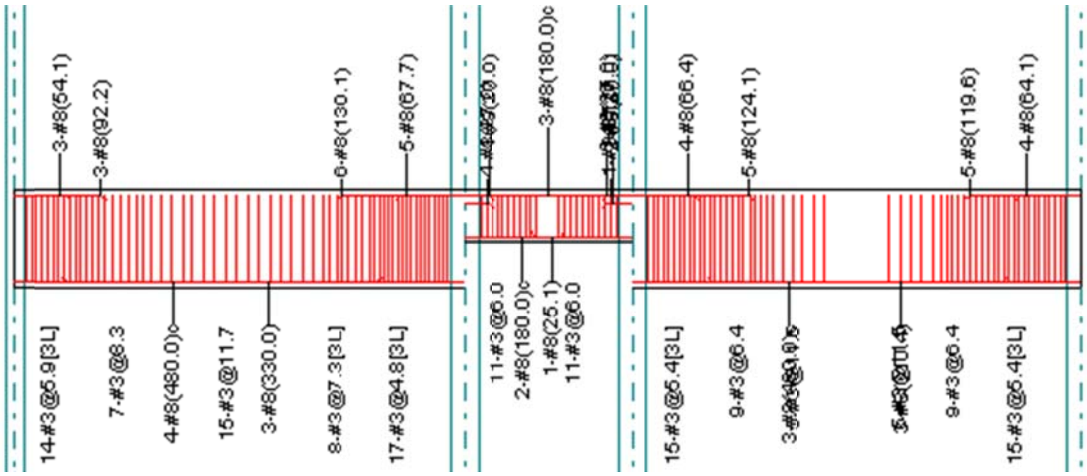


Figure 16: spBeam Column Line C Reinforcing diagram

A cost analysis was done to determine which floor system would be more economical. A cost estimate was created for both floor systems using RSMeans. The estimated cost of the two-way flat slab system is $20.05/sq. ft. and the one-way slab and beam system is approximately $20.29/sq. ft. Detailed hand cost estimate calculations can be seen in Appendix A. The two-way flat slab system is slightly more economical than the one-way slab and beam system, but the costs are very similar. If the two-way flat slab floor system was chosen, 24 additional columns would be needed on every floor. The flexibility of the open floor plan created by using the one-way slab and beam system is worth the extra cost. For this reason, the one-way slab and beam floor system was ultimately chosen for this thesis redesign.

One-Way Slab and Beam System Design

After the one-way slab and beam system was chosen, a beam layout for the floor plan was created. Figures 17 and 18 below show the beam layout for a typical floor along with the sizes of the beams. As seen on the layout there are four transfer girders that had to be designed. These girders were also designed using spBeam. The spBeam models and reinforcing diagrams for the transfer girders can be seen in Appendix D. The table below shows the beam sizes and the flexural reinforcing required for the gravity loads.

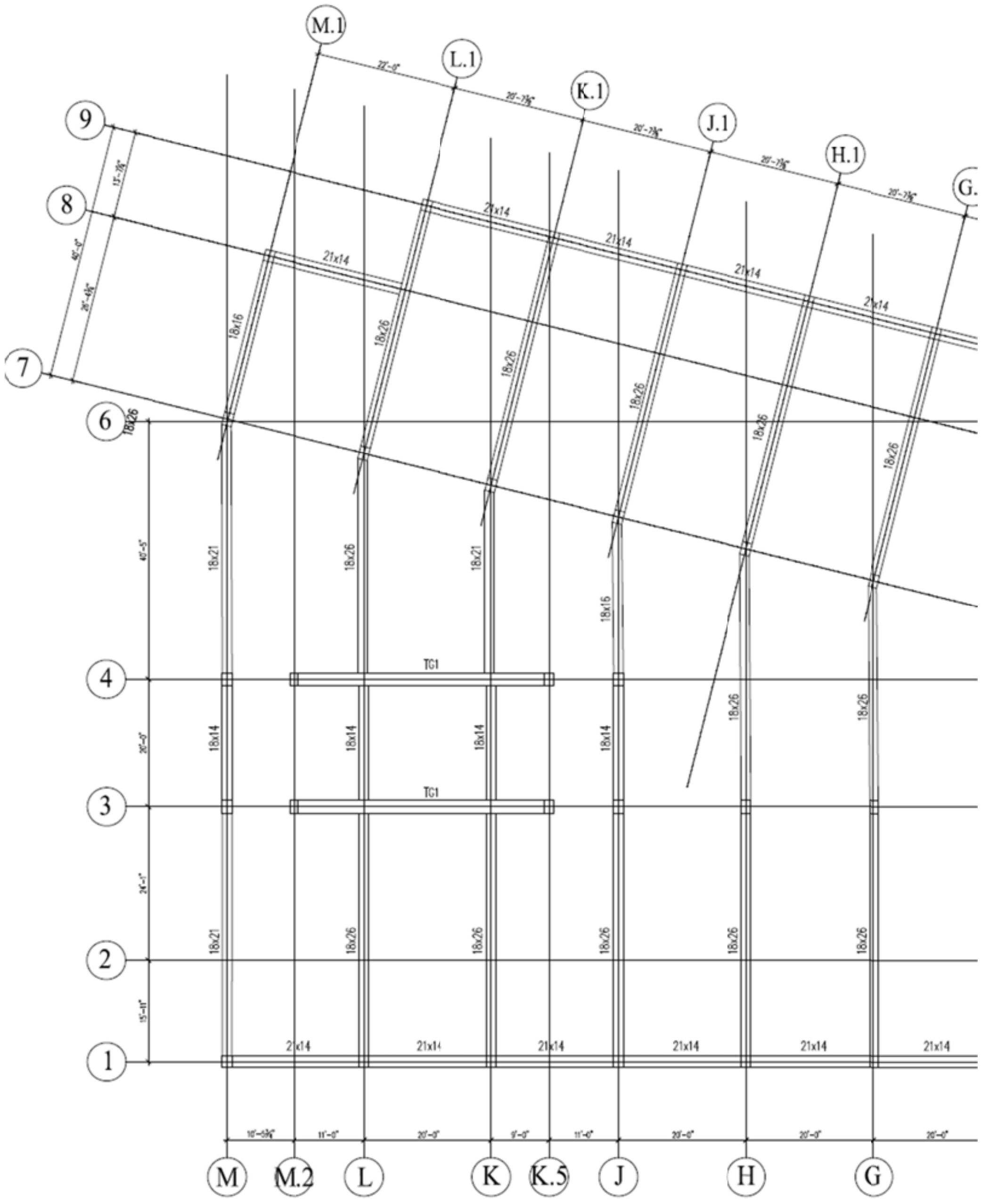


Figure 17: Typical Beam Layout Part 1

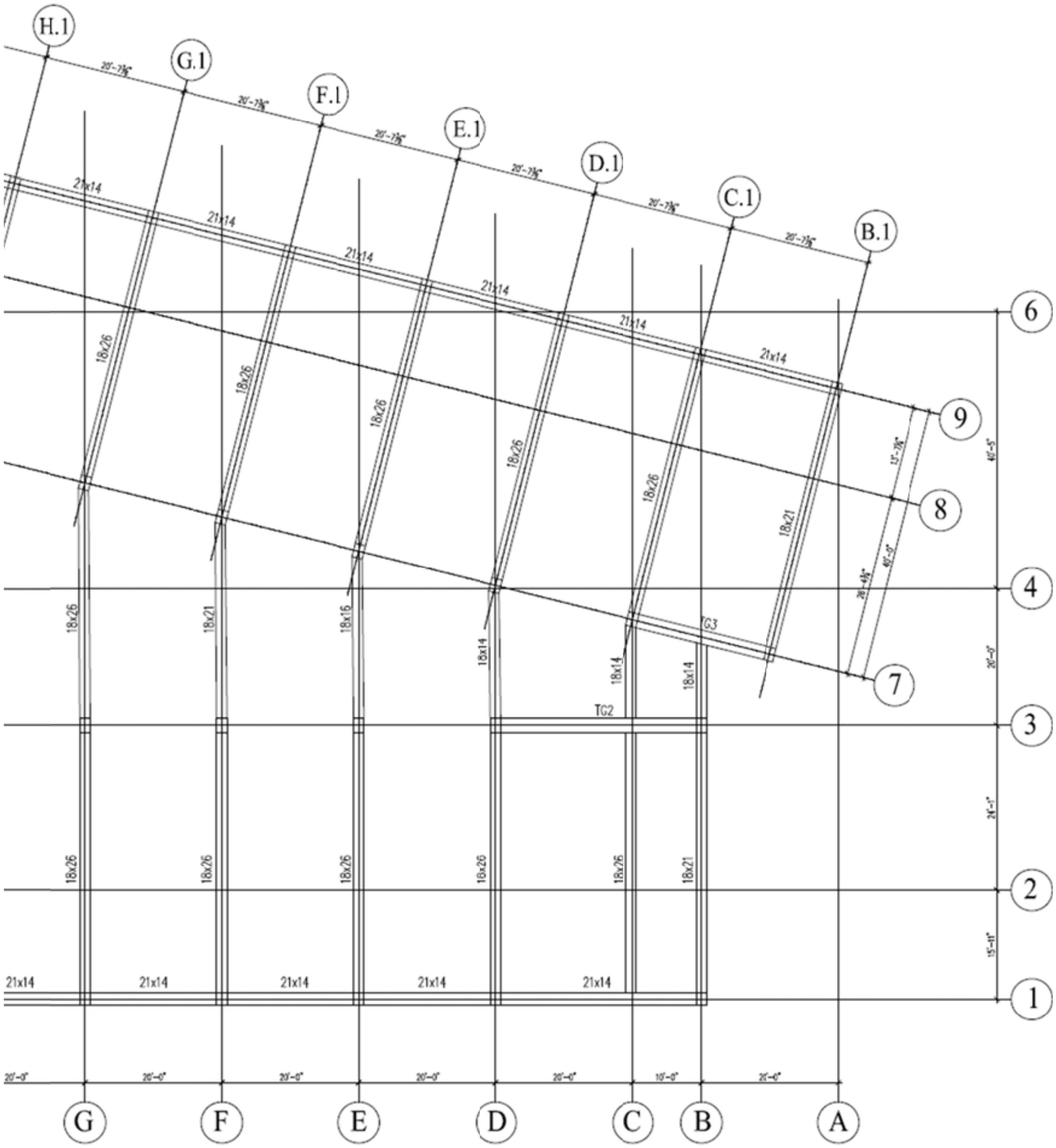


Figure 18: Typical Beam Layout Part 2

Beam Reinforcement Details (From spBeam)							
Reinforcing is #8 Bars Unless Otherwise Noted							
Column Line C							
Start	End	Length	Size	Top Reinf. Left	Top Reinf. Mid.	Top Reinf. Right	Bottom Reinf.
C-1	C-3	40'	18x26	6	-	11	7
C-3	C.1-7	14'	18x14	11	3	9	3
C.1-7	C.1-9	40'	18x26	9	-	9	6
Column Line D							
Start	End	Length	Size	Top Reinf. Left	Top Reinf. Mid.	Top Reinf. Right	Bottom Reinf.
D-1	D-3	40'	18x26	8	-	10	7
D-3	D.1-7	20'	18x14	10	-	9	3
D.1-7	D.1-9	40'	18x26	9	-	9	6
Column Line E							
Start	End	Length	Size	Top Reinf. Left	Top Reinf. Mid.	Top Reinf. Right	Bottom Reinf.
E-1	E-3	40'	18x26	8	-	10	7
E-3	E.1-7	25'	18x16	10	-	10	4
E.1-7	E.1-9	40'	18x26	10	-	9	6
Column Line F							
Start	End	Length	Size	Top Reinf. Left	Top Reinf. Mid.	Top Reinf. Right	Bottom Reinf.
F-1	F-3	40'	18x26	8	-	11	7
F-3	F.1-7	29'	18x21	11	-	10	4
F.1-7	F.1-9	40'	18x26	10	-	9	6
Column Line G							
Start	End	Length	Size	Top Reinf. Left	Top Reinf. Mid.	Top Reinf. Right	Bottom Reinf.
G-1	G-3	40'	18x26	8	-	11	7
G-3	G.1-7	35'	18x26	11	-	10	5
G.1-7	G.1-9	40'	18x26	10	-	9	6
Column Line H							
Start	End	Length	Size	Top Reinf. Left	Top Reinf. Mid.	Top Reinf. Right	Bottom Reinf.
H-1	H-3	40'	18x26	8	-	12	7
H-3	H.1-7	40'	18x26	12	-	11	6
H.1-7	H.1-9	40'	18x26	11	-	9	6
Column Line J							
Start	End	Length	Size	Top Reinf. Left	Top Reinf. Mid.	Top Reinf. Right	Bottom Reinf.
J-1	J-3	40'	18x26	8	-	10	7
J-3	J-4	20'	18x14	10	-	6	3
J-4	J.1-7	25'	18x16	6	-	10	4
J.1-7	J.1-9	40'	18x26	10	-	9	6

Column Line K							
Start	End	Length	Size	Top Reinf. Left	Top Reinf. Mid.	Top Reinf. Right	Bottom Reinf.
K-1	K-3	40'	18x26	8	-	10	7
K-3	K-4	20'	18x14	10	-	6	3
K-4	K.1-7	29'	18x21	6	-	10	4
K.1-7	K.1-9	40'	18x26	10	-	9	6
Column Line L							
Start	End	Length	Size	Top Reinf. Left	Top Reinf. Mid.	Top Reinf. Right	Bottom Reinf.
L-1	L-3	40'	18x26	8	-	10	7
L-3	L-4	20'	18x14	10	-	7	3
L-4	L.1-7	35'	18x26	7	-	12	5
L.1-7	L.1-9	40'	18x26	12	-	10	7
Column Line M							
Start	End	Length	Size	Top Reinf. Left	Top Reinf. Mid.	Top Reinf. Right	Bottom Reinf.
M-1	M-3	40'	18x21	8	4	9	5
M-3	M-4	20'	18x14	9	3	8	3
M-4	M.1-7	40'	18x21	8	4	9	5
M.1-7	M.1-9	26'	18x16	9	3	4	3
Column Line 9 & 8							
Start	End	Length	Size	Top Reinf. Left	Top Reinf. Mid.	Top Reinf. Right	Bottom Reinf.
M.1-8	L.1-8	20'	21x14	3	3	3	3
C-9	B-9	20'	21x14	5	3	5	4
Transfer Girders							
Start	End	Length	Size	Top Reinf. Left	Top Reinf. Mid.	Top Reinf. Right	Bottom Reinf.
M.2-3	K.5-3	40'	18x26	5-#11	-	5-#11	7
M.2-4	K.5-4	40'	18x26	5-#11	-	5-#11	7
D-3	B-3	30'	24x16	5	-	8	7
C.1-7	B.1-7	20'	12x16	2	-	2	2
Column Line 1							
Start	End	Length	Size	Top Reinf. Left	Top Reinf. Mid.	Top Reinf. Right	Bottom Reinf.
M-1	L-1	20'	21x14	5	3	5	4
D-1	B-1	30'	21x21	10	3	11	8
Column Line B							
Start	End	Length	Size	Top Reinf. Left	Top Reinf. Mid.	Top Reinf. Right	Bottom Reinf.
B-1	B-3	40'	18x21	5	4	5	4
B-3	B-7	12'	18x14	5	3	3	3
B.1-7	B.1-9	40'	18x21	8	4	8	5

Column Design for Gravity Loads

The concrete columns for the office tower were designed using spColumn. The columns and beams were generally sized to be the same width to save time and money on formwork. The columns are spliced above level 4. In the lower floors, the exterior columns are mostly 18”x21” and the interior columns 18”x24”. In the upper floors above the splice at level 4, the exterior columns are generally 18”x18” and the interior columns 18”x20”. The spColumn output for a typical interior column is shown in Figure 19 below. As seen in the figure below, the typical exterior column is 18”x21” and has 12 # 10 bars. Additional column designs from spColumn are shown in Appendix D. Two tables are presented below that show the axial loads, moments and gravity column designs for all of the columns in the office tower. The first table is for the columns below the splice at level 4 and the second table is for the columns above the splice at level 4. The last two columns which are highlighted in yellow show the size of each column and reinforcing required under the gravity loads.

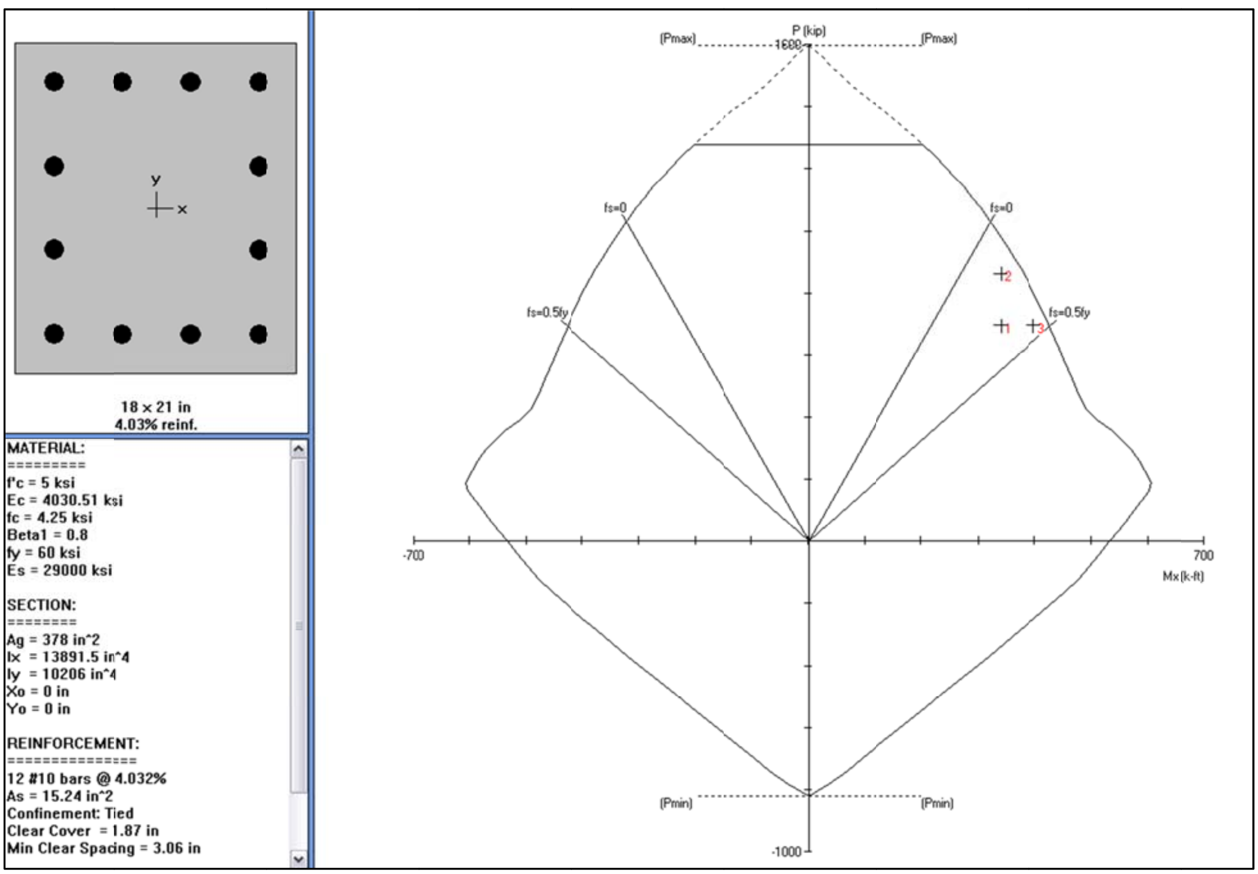


Figure 19: spColumn Typical Exterior Column Design

Column Design Table										
	ft^2		kip	kips	kips	kips	kips	ft-kips	Size	Reinf
Column Location	A_T	Type	Self wt.	P_{live}	P_{dead}	$P_{dead+self}$	Pu	Mu		
B-1	300	Corner	21.3	129.0	282.8	304.0	571	185	18x21in	4-#9
D-1	500	Exterior	21.3	215.0	411.3	432.5	863	340	18x21in	12-#10
E-1	400	Exterior	21.3	172.0	329.0	350.3	696	339	18x21in	12-#10
F-1	400	Exterior	21.3	172.0	329.0	350.3	696	336	18x21in	12-#10
G-1	400	Exterior	21.3	172.0	329.0	350.3	696	333	18x21in	12-#10
H-1	400	Exterior	21.3	172.0	329.0	350.3	696	331	18x21in	12-#10
J-1	400	Exterior	21.3	172.0	329.0	350.3	696	340	18x21in	12-#10
K-1	400	Exterior	21.3	172.0	329.0	350.3	696	340	18x21in	12-#10
L-1	400	Exterior	21.3	172.0	329.0	350.3	696	340	18x21in	4-#9
M-1	200	Corner	21.3	86.0	200.5	221.8	404	239	18x21in	12-#10
B-3	525	Exterior	21.3	225.8	431.4	452.6	904	161	18x21in	4-#9
D-3	750	Interior	24.3	322.5	549.4	573.7	1204	325	18x24in	12-#10
E-3	640	Interior	24.3	275.2	468.8	493.1	1032	273	18x24in	12-#10
F-3	700	Interior	24.3	301.0	512.8	537.1	1126	213	18x24in	12-#10
G-3	740	Interior	24.3	318.2	542.1	566.4	1189	115	18x24in	12-#10
H-3	800	Interior	24.3	344.0	586.0	610.3	1283	35	18x24in	12-#10
J-3	600	Interior	24.3	258.0	439.5	463.8	969	323	18x24in	12-#10
K.5-3	750	Interior	24.3	322.5	549.4	573.7	1204	409	18x26in	12-#10
M.2-3	750	Interior	24.3	322.5	549.4	573.7	1204	400	18x24in	12-#10
M-3	300	Exterior	21.3	129.0	273.8	295.0	560	228	18x21in	4-#9
J-4	440	Interior	24.3	189.2	322.3	346.6	719	56	18x24in	12-#10
K.5-4	625	Interior	24.3	268.8	457.8	482.1	1009	409	18x24in	12-#10
M.2-4	750	Interior	24.3	322.5	549.4	573.7	1204	400	18x26in	12-#10
M-4	300	Exterior	21.3	129.0	273.8	295.0	560	208	18x21in	4-#9
B.1-7	225	Corner	21.3	96.8	218.8	240.1	443	283	18x21in	4-#9
C.1-7	505	Interior	24.3	217.2	369.9	394.2	820	339	18x24in	12-#10
D.1-7	640	Interior	24.3	275.2	468.8	493.1	1032	302	18x24in	12-#10
E.1-7	700	Interior	24.3	301.0	512.8	537.1	1126	249	18x24in	12-#10
F.1-7	740	Interior	24.3	318.2	542.1	566.4	1189	381	18x26in	12-#10
G.1-7	800	Interior	24.3	344.0	586.0	610.3	1283	96	18x24in	12-#10
H.1-7	600	Interior	24.3	258.0	439.5	463.8	969	15	18x24in	12-#10
J.1-7	640	Interior	24.3	275.2	468.8	493.1	1032	246	18x24in	12-#10
K.1-7	700	Interior	24.3	301.0	512.8	537.1	1126	183	18x24in	12-#10
L.1-7	740	Interior	24.3	318.2	542.1	566.4	1189	114	18x24in	12-#10
M.1-7	350	Exterior	21.3	150.5	319.4	340.6	650	162	18x21in	4-#9
B.1-9	200	Corner	21.3	86.0	200.5	221.8	404	283	18x21in	4-#9
C.1-9	400	Exterior	21.3	172.0	329.0	350.3	696	399	18x21in	12-#10
D.1-9	400	Exterior	21.3	172.0	329.0	350.3	696	398	18x21in	12-#10
E.1-9	400	Exterior	21.3	172.0	329.0	350.3	696	396	18x21in	12-#10
F.1-9	400	Exterior	21.3	172.0	329.0	350.3	696	393	18x21in	12-#10
G.1-9	400	Exterior	21.3	172.0	329.0	350.3	696	389	18x21in	12-#10
H.1-9	400	Exterior	21.3	172.0	329.0	350.3	696	386	18x21in	12-#10
J.1-9	400	Exterior	21.3	172.0	329.0	350.3	696	395	18x21in	12-#10
K.1-9	400	Exterior	21.3	172.0	329.0	350.3	696	392	18x21in	12-#10
L.1-9	400	Exterior	21.3	172.0	329.0	350.3	696	448	18x21in	12-#10
M.1-8	180	Corner	21.3	77.4	176.9	198.1	362	114	18x21in	4-#9

Column Design Table - Above Splice at Level 4										
	ft^2		kip	kips	kips	kips	kips	ft-kips	Size	Reinf
Column Location	A_T	Type	Self wt.	P_{live}	P_{dead}	$P_{dead+self}$	Pu	Mu		
B-1	300	Corner	10.6	69.0	166.9	177.5	323	185	18x18in	4-#9
D-1	500	Exterior	10.6	115.0	244.8	255.4	490	340	18x18in	12-#10
E-1	400	Exterior	10.6	92.0	195.8	206.4	395	339	18x18in	12-#10
F-1	400	Exterior	10.6	92.0	195.8	206.4	395	336	18x18in	12-#10
G-1	400	Exterior	10.6	92.0	195.8	206.4	395	333	18x18in	12-#10
H-1	400	Exterior	10.6	92.0	195.8	206.4	395	331	18x18in	12-#10
J-1	400	Exterior	10.6	92.0	195.8	206.4	395	340	18x18in	12-#10
K-1	400	Exterior	10.6	92.0	195.8	206.4	395	340	18x18in	12-#10
L-1	400	Exterior	10.6	92.0	195.8	206.4	395	340	18x18in	12-#10
M-1	200	Corner	10.6	46.0	117.9	128.5	228	239	18x18in	12-#10
B-3	525	Exterior	10.6	120.8	256.7	267.4	514	161	18x18in	4-#9
D-3	750	Interior	12.2	172.5	329.6	341.8	686	325	18x20in	10-#10
E-3	640	Interior	12.2	147.2	281.3	293.4	588	273	18x20in	10-#10
F-3	700	Interior	12.2	161.0	307.7	319.8	641	213	18x20in	10-#10
G-3	740	Interior	12.2	170.2	325.2	337.4	677	115	18x20in	10-#10
H-3	800	Interior	12.2	184.0	351.6	363.8	731	35	18x20in	10-#10
J-3	600	Interior	12.2	138.0	263.7	275.9	552	323	18x20in	10-#10
K.5-3	750	Interior	12.2	172.5	329.6	341.8	686	409	18x21in	12-#10
M.2-3	750	Interior	12.2	172.5	329.6	341.8	686	400	18x21in	12-#10
M-3	300	Exterior	10.6	69.0	161.9	172.5	317	228	18x18in	4-#9
J-4	440	Interior	12.2	101.2	193.4	205.5	409	56	18x20in	10-#10
K.5-4	625	Interior	12.2	143.8	274.7	286.8	574	409	18x21in	12-#10
M.2-4	750	Interior	12.2	172.5	329.6	341.8	686	400	18x20in	10-#10
M-4	300	Exterior	10.6	69.0	161.9	172.5	317	208	18x18in	4-#9
B.1-7	225	Corner	10.6	51.8	128.9	139.5	250	283	18x18in	12-#10
C.1-7	505	Interior	12.2	116.2	369.9	382.1	644	339	18x20in	10-#10
D.1-7	640	Interior	12.2	147.2	281.3	293.4	588	302	18x20in	10-#10
E.1-7	700	Interior	12.2	161.0	307.7	319.8	641	249	18x20in	10-#10
F.1-7	740	Interior	12.2	170.2	325.2	337.4	677	381	18x21in	12-#10
G.1-7	800	Interior	12.2	184.0	351.6	363.8	731	96	18x20in	10-#10
H.1-7	600	Interior	12.2	138.0	263.7	275.9	552	15	18x20in	10-#10
J.1-7	640	Interior	12.2	147.2	281.3	293.4	588	246	18x20in	10-#10
K.1-7	700	Interior	12.2	161.0	307.7	319.8	641	183	18x20in	10-#10
L.1-7	740	Interior	12.2	170.2	325.2	337.4	677	114	18x20in	10-#10
M.1-7	350	Exterior	10.6	80.5	188.8	199.5	368	162	18x18in	4-#9
B.1-9	200	Corner	10.6	46.0	117.9	128.5	228	283	18x18in	12-#10
C.1-9	400	Exterior	10.6	92.0	195.8	206.4	395	399	18x18in	12-#10
D.1-9	400	Exterior	10.6	92.0	195.8	206.4	395	398	18x18in	12-#10
E.1-9	400	Exterior	10.6	92.0	195.8	206.4	395	396	18x18in	12-#10
F.1-9	400	Exterior	10.6	92.0	195.8	206.4	395	393	18x18in	12-#10
G.1-9	400	Exterior	10.6	92.0	195.8	206.4	395	389	18x18in	12-#10
H.1-9	400	Exterior	10.6	92.0	195.8	206.4	395	386	18x18in	12-#10
J.1-9	400	Exterior	10.6	92.0	195.8	206.4	395	395	18x18in	12-#10
K.1-9	400	Exterior	10.6	92.0	195.8	206.4	395	392	18x18in	12-#10
L.1-9	400	Exterior	10.6	92.0	195.8	206.4	395	448	18x18in	12-#10
M.1-8	180	Corner	10.6	41.4	104.1	114.7	204	114	18x18in	4-#9

ETABS Model

After the new seismic loads were determined, a computer model of the building was created using ETABS. All of the columns, beams were modeled using line elements. The slabs were modeled using rigid diaphragms, with an added area mass to account for the self-weight. This was done to determine if the gravity system is adequate to resist the lateral loads. Two grids were created for this model. One of the grids is rotated 14.04 degrees clockwise off of the global axis. Figure 20 and 21 show three dimensional views of the ETABS model that was created for this report. Figure 22 is a typical floor plan of the ETABS model that shows the locations of the beams and columns. There were multiple assumptions that were made in order to model the ASHA National Office tower.

Assumptions

- The self-weight of the columns and beams is accounted for in the model
- Rigid end zones are applied to all beams with a reduction of 50%
- The slabs are considered to act as rigid diaphragms
- The self-weight of the slab is applied as an additional area mass on the rigid diaphragm
- P-Δ effects are considered
- The moment of inertia for each element is:

  Columns = 0.7Ig
  Beams = 0.35Ig
  Slabs = 0.25Ig
- The compressive strength of all concrete is 5000 psi

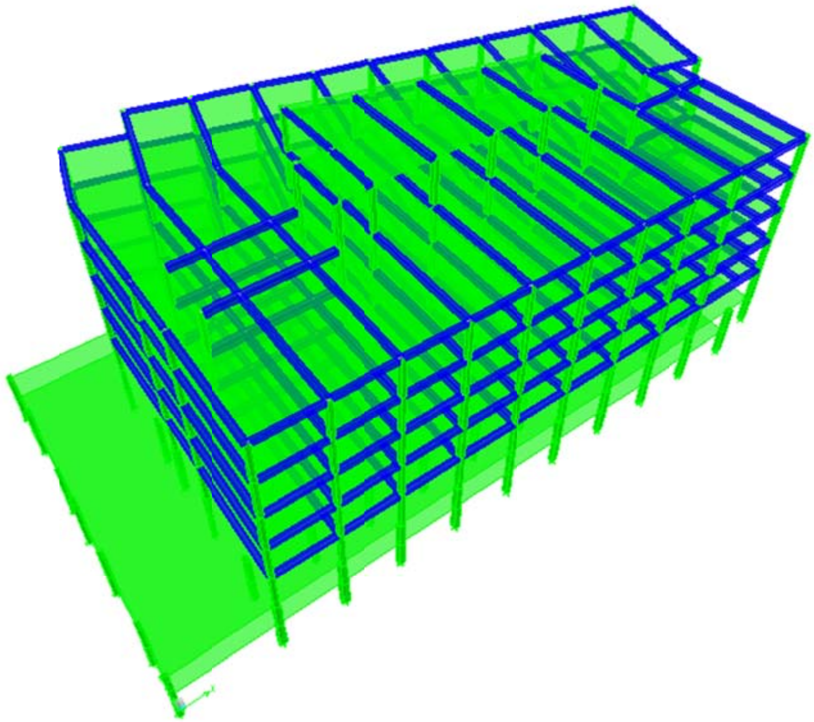


Figure 20: ETABS Model 3-D View 1

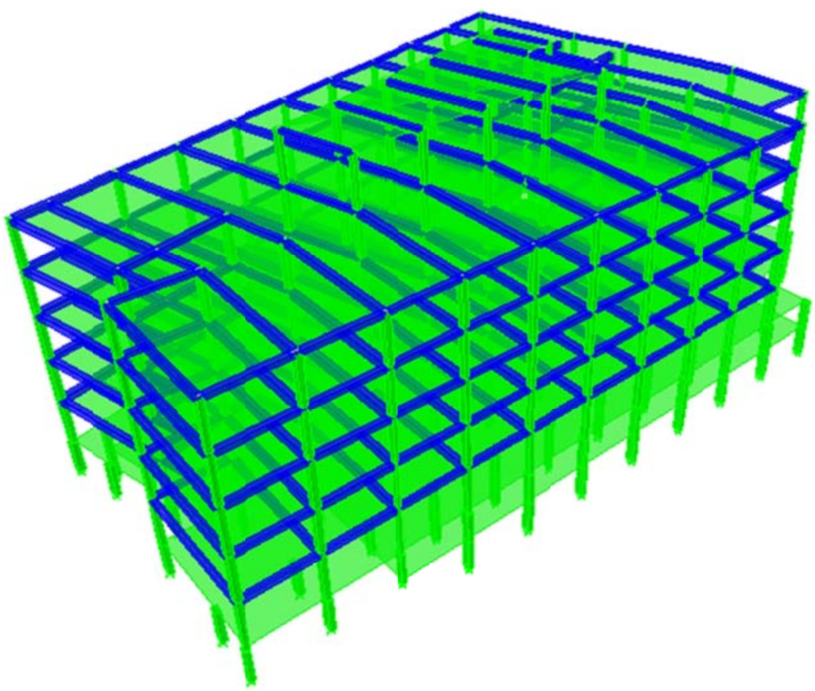


Figure 21: ETABS Model 3-D View 2

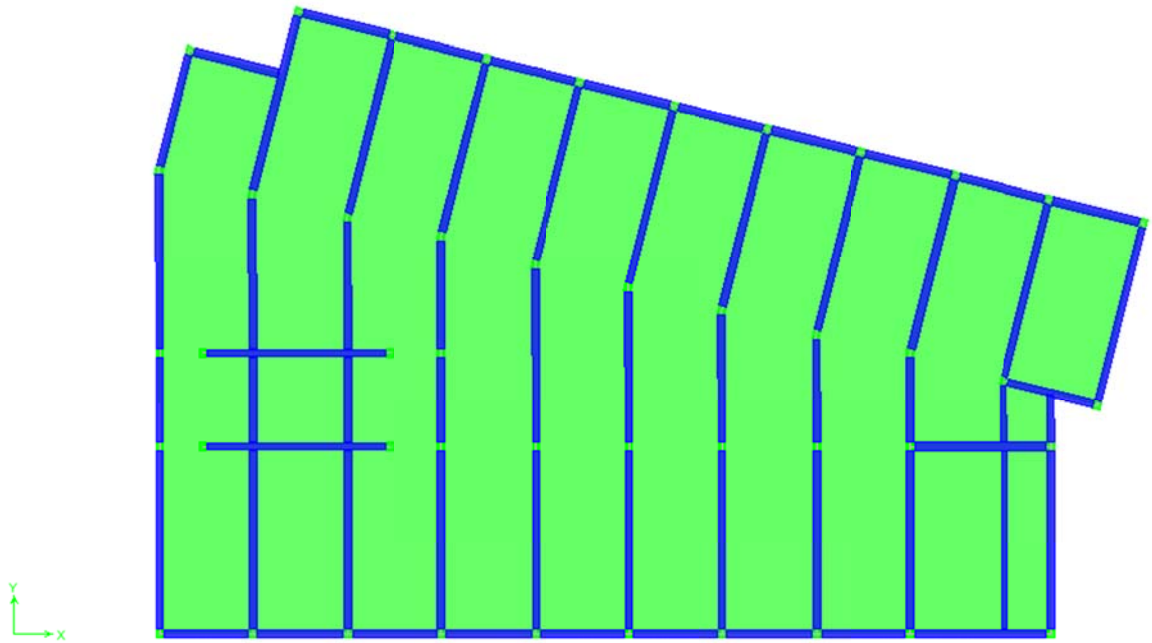


Figure 22: ETABS Model Typical Floor Plan

Recalculation of Seismic Loads

After the building was designed for the gravity loads, then the lateral loads were considered. Because the building was changed to concrete from steel, the seismic loads on the building will change. The seismic loads were recalculated using The Equivalent Lateral Force Procedure of ASCE 7-10. The ETABS model that was created was used to find the fundamental periods along the principle axes. The design period must not exceed C_uT_a from chapter 12 of ASCE 7-10, which was calculated to be 1.19s. As seen in the table below, the calculated period C_uT_a is less than the all three of the first modes of vibration; therefore it was used as the design period. The table below shows the new seismic loads on each floor of the building. Detailed floor weight and seismic load calculations can be seen in Appendix A.

Fundamental Periods Along Principle Axes		
Direction	**T**	**Mode**
X	3.224 s	1
Y	2.152 s	2
Z	1.955 s	3

Veritical Distribution of Seismic Forces						
Floor	**w_x**	**h_x (ft)**	**w_xh_x^k**	**C_{vx}**	**F_x**	
Parking	3007.7	10.0	65801.0	0.015	5.3	k
Plaza	2960.0	20.0	163935.9	0.037	13.3	k
2nd	3354.5	35.0	393265.0	0.090	32.0	k
3rd	3339.9	48.5	606217.7	0.138	49.3	k
4th	3294.0	62.0	830852.9	0.190	67.5	k
5th	3191.7	75.5	1048252.4	0.239	85.2	k
Roof	3105.9	89.0	1271638.1	0.290	103.4	k
		Sum	4379963.0	1.000	**356.1**	**k**

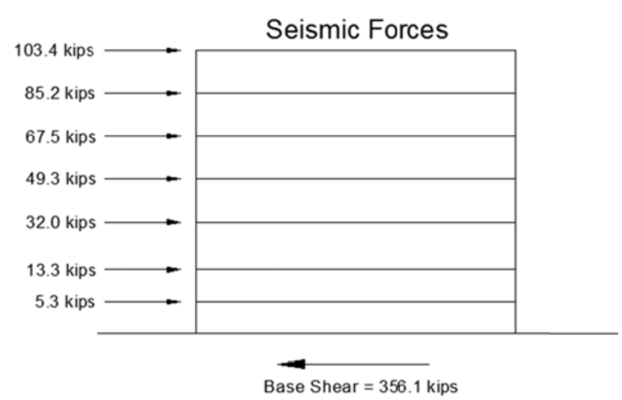


Figure 23: Seismic Story Forces

Lateral Design

Multiple checks were done to determine if the inherent moment connections of the reinforced concrete structure are adequate to resist the lateral loads on the building. Story drifts and displacements were checked, and the strength of the beams and columns were checked to see if they are sufficient to resist moments caused by the wind and seismic loads. If the structure does not meet these requirements, then shear walls will have to be added to the office tower.

Drift and Displacement Check

After the ETABS model was created, the wind and seismic loads were applied to the office tower to determine if the gravity system is adequate for the lateral loads. Story drift and the total lateral displacement of the building were then checked. According to ASCE 7-10, the allowable seismic story drift for a building in the occupancy category II is $0.020h_{sx}$. The accepted standard for total building displacement for wind loads is L/400. The ETABS building model was utilized to determine the story drifts and displacements. The unfactored loads were used to determine the seismic story drift, and the factored loads were used to determine the wind drift. The tables below show the story drifts for the wind and seismic loads versus the allowable drifts. As seen, the actual drifts are within the limits of the code and accepted standards.

Seismic Story Drift N-S Direction				
Floor	**Displacement (in)**	**Story Drift (in)**	**Allowable Story Drift (in)**	**Okay?**
PH Roof	1.596	0.079	3.84	Yes
Roof	1.517	0.158	3.24	Yes
Fifth	1.359	0.249	3.24	Yes
Fourth	1.110	0.304	3.24	Yes
Third	0.806	0.350	3.24	Yes
Second	0.456	0.366	3.6	Yes
Plaza	0.090	0.090	2.4	Yes
Parking	0.000	0.000	2.4	Yes

Seismic Story Drift E-W Direction				
Floor	**Displacement (in)**	**Story Drift (in)**	**Allowable Story Drift (in)**	**Okay?**
PH Roof	3.879	0.354	3.84	Yes
Roof	3.525	0.383	3.24	Yes
Fifth	3.142	0.561	3.24	Yes
Fourth	2.581	0.710	3.24	Yes
Third	1.871	0.811	3.24	Yes
Second	1.060	0.836	3.6	Yes
Plaza	0.224	0.224	2.4	Yes
Parking	0.000	0.000	2.4	Yes

Wind Story Displacement N-S Direction			
Floor	**Displacement (in)**	**Allowable Displacment (in)**	**Okay?**
PH Roof	1.491	3.150	Yes
Roof	1.443	2.670	Yes
Fifth	1.343	2.265	Yes
Fourth	1.146	1.860	Yes
Third	0.866	1.455	Yes
Second	0.510	1.050	Yes
Plaza	0.101	0.600	Yes
Parking	0.000	0.300	Yes

Wind Story Displacement E-W Direction			
Floor	**Displacement (in)**	**Allowable Displacment (in)**	**Okay?**
PH Roof	1.564	3.150	Yes
Roof	1.560	2.670	Yes
Fifth	1.342	2.265	Yes
Fourth	1.141	1.860	Yes
Third	0.853	1.455	Yes
Second	0.496	1.050	Yes
Plaza	0.106	0.600	Yes
Parking	0.000	0.300	Yes

Lateral Design of One-Way Beams

After the drift and displacements were checked, the beams were checked to determine if they able to resist the wind and seismic loads. The moments on the one-way beams due to the wind and seismic loads were obtained from ETABS and then input into spBeam. The tables below show the moments on the one-way beams due to the wind and seismic loads that were obtained from ETABS. The proper load cases were used for ASCE7-10. Every column line from B to M in the N-S direction was reanalyzed and new reinforcing diagrams for all of the beams were created. Detailed calculations and spreadsheets can be seen in Appendix A. The new reinforcing diagrams for all of the beams are shown in Appendix E. None of the beams in the N-S direction had to be increased in size for the lateral loads, although the amount of flexural reinforcing had to be increased in a number of the beams. As seen in the new reinforcing diagrams, the amount of shear reinforcing also often had to be increased to resist the lateral loads. The edge beams in column line 9 that run in the E-W direction had to be made deeper by 2 inches. The 30' long edge beam in column line 1 had to be made deeper by 3 inches. The four transfer girders in the building were also redesigned for the gravity loads. The reinforcing diagrams for the transfer girders that have been redesigned for the lateral loads are shown in Appendix E.

Moments on Beams Due to N-S Wind Loads (ft-kips)								
Column Line	Beam 1-3		Beam 3-7		Beam 7-9			
B	44.3	44.4	29.9	19.2	47.0	46.8		
C	38.6	30.3	18.8	27.7	76.7	76.9		
D	75.4	76.3	26.6	26.9	75.6	75.2		
E	73.9	74.5	31.0	31.1	70.3	70.4		
F	69.7	69.1	51.5	51.6	64.8	66.0		
G	65.3	63.5	73.4	73.6	59.2	61.5		
H	62.4	61.0	61.6	61.8	56.5	58.4		
	Beam 1-3		Beam 3-4		Beam 4-7		Beam 7-9	
J	60.8	61.3	21.7	22.3	25.9	25.2	57.5	57.5
K	44.7	31.9	8.4	8.8	26.3	34.1	51.8	52.4
L	41.6	28.2	7.1	7.1	29.4	43.0	52.6	44.3
	Beam 1-3		Beam 3-4		Beam 4-7		Beam 7-8	
M	29.3	29.2	18.6	18.6	30.0	30.0	18.1	18.3

Moments on Beams Due to N-S Seismic Loads (ft-kips)								
Column Line	Beam 1-3		Beam 3-7		Beam 7-9			
B	48.8	49.0	32.9	21.0	51.3	51.1		
C	42.8	33.7	20.9	30.7	84.8	85.0		
D	84.7	85.6	29.8	30.2	84.7	84.3		
E	84.2	84.8	35.3	35.5	80.0	80.1		
F	80.6	79.9	59.5	59.6	75.1	76.4		
G	76.8	74.6	86.3	86.5	69.8	72.5		
H	74.6	73.0	73.7	73.9	68.1	70.2		
	Beam 1-3		Beam 3-4		Beam 4-7		Beam 7-9	
J	74.0	74.8	26.5	27.2	31.5	30.6	70.7	70.7
K	55.4	39.4	10.4	11.0	32.5	42.2	65.3	66.0
L	53.1	36.2	9.0	9.0	37.7	55.0	67.6	58.4
	Beam 1-3		Beam 3-4		Beam 4-7		Beam 7-8	
M	38.2	38.2	24.4	24.4	39.2	39.1	24.2	24.4

Lateral Design of Concrete Columns

The concrete columns were also checked to determine if they can resist the lateral loads. The moments on the columns caused by the wind and seismic loads were obtained from ETABS. These moments were then put into spColumn in order to check to see if the columns were sufficient to resist the loads. Figure 24 shows the spColumn design of a typical exterior column for gravity and lateral loads. Additional column designs for both gravity and lateral loads are shown in Appendix F. The table below shows the loads caused by the dead, live, wind and seismic loads. The last two columns of the table, which are highlighted in green, show the size of each column and the required reinforcing for gravity and lateral loads. None of the columns had to be upsized for the lateral loads. The majority of the columns had sufficient reinforcing to resist the lateral loads, although the reinforcing in some of the exterior 18x21 inch columns had to be increased from 4 #9 bars to 8 #9 bars to resist the wind and seismic loads.

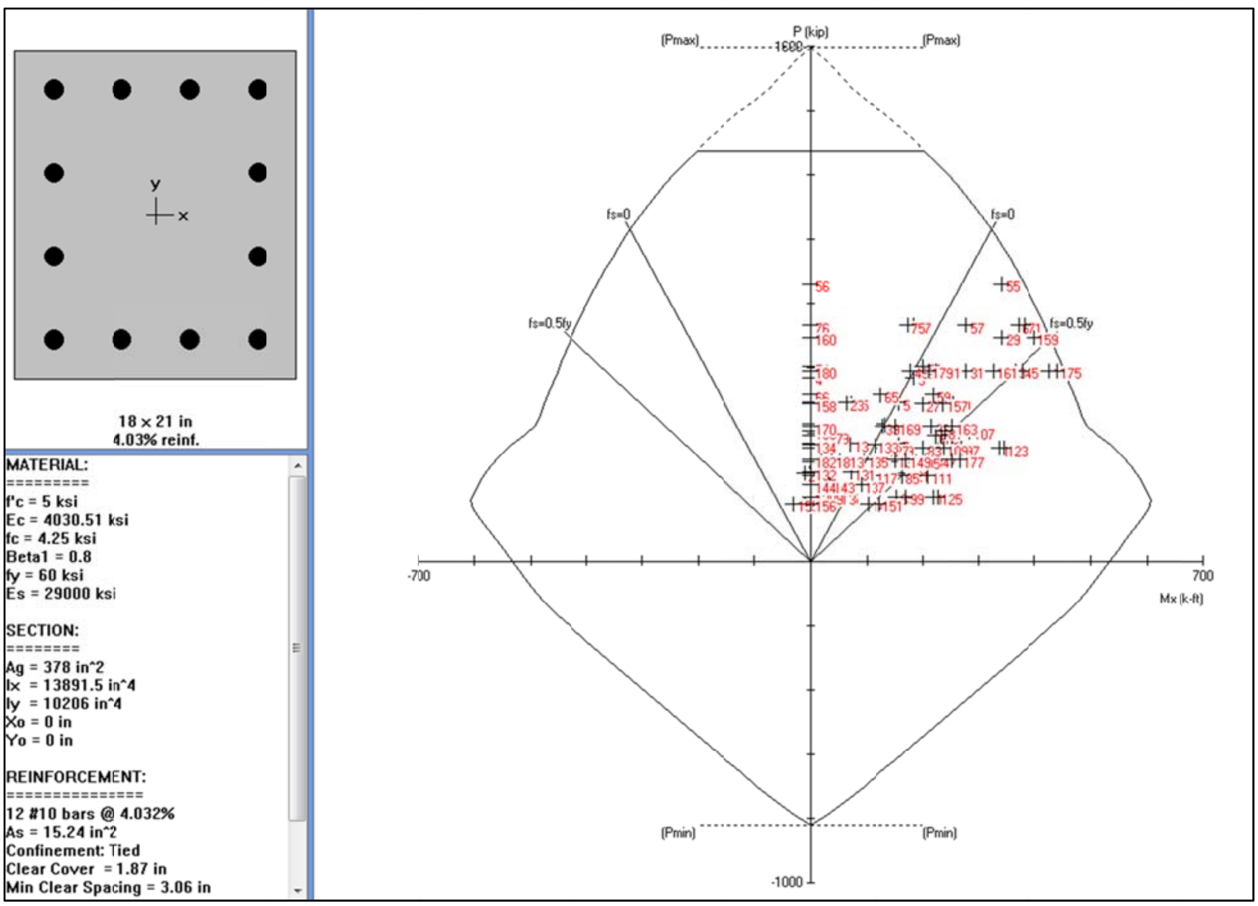


Figure 24: spColumn Typical Exterior Column Design (Lateral Loads)

Column Design Table - Lateral Loads								
Column Location	Plive	Pdead	M_L	M_D	M_{WY}	M_{EY}	Size	Reinf
B-1	129.0	304.0	44.8	94.5	54.7	62.9	18x21in	8-#9
D-1	215.0	432.5	105.2	143.4	58.2	66.9	18x21in	12-#10
E-1	172.0	350.3	105.2	142.1	54.3	62.4	18x21in	12-#10
F-1	172.0	350.3	105.0	140.5	51.4	59.1	18x21in	12-#10
G-1	172.0	350.3	104.7	138.0	48.5	55.8	18x21in	12-#10
H-1	172.0	350.3	104.8	135.9	45.8	52.7	18x21in	12-#10
J-1	172.0	350.3	105.1	143.3	43.2	49.7	18x21in	12-#10
K-1	172.0	350.3	105.1	143.3	38.9	44.7	18x21in	12-#10
L-1	172.0	350.3	105.2	143.3	36.1	41.5	18x21in	12-#10
M-1	86.0	221.8	63.6	114.3	32.2	37.0	18x21in	8-#9
B-3	225.8	452.6	38.7	82.7	90.7	104.3	18x21in	8-#9
D-3	322.5	573.7	97.5	141.2	95.6	109.9	18x24in	12-#10
E-3	275.2	493.1	80.6	119.6	89.9	103.4	18x24in	12-#10
F-3	301.0	537.1	61.6	95.0	88.3	101.5	18x24in	12-#10
G-3	318.2	566.4	31.5	53.9	86.8	99.8	18x24in	12-#10
H-3	344.0	610.3	7.3	19.5	80.9	93.0	18x24in	12-#10
J-3	258.0	463.8	97.4	139.2	71.2	81.9	18x24in	12-#10
K.5-3	322.5	573.7	186.4	84.5	62.9	72.3	18x26in	12-#10
M.2-3	322.5	573.7	191.1	86.0	53.9	62.0	18x26in	12-#10
M-3	129.0	295.0	58.5	112.4	54.5	62.7	18x21in	8-#9
J-4	189.2	346.6	17.5	22.6	66.0	75.9	18x24in	12-#10
K.5-4	268.8	482.1	186.4	84.5	62.4	71.8	18x24in	12-#10
M.2-4	322.5	573.7	191.1	86.0	53.9	62.0	18x26in	12-#10
M-4	129.0	295.0	54.0	101.1	54.6	62.8	18x21in	8-#9
B.1-7	96.8	240.1	76.7	133.8	81.8	94.1	18x21in	8-#9
C.1-7	217.2	394.2	101.8	146.8	86.3	99.2	18x24in	12-#10
D.1-7	275.2	493.1	90.1	131.3	93.0	107.0	18x24in	12-#10
E.1-7	301.0	537.1	73.4	109.9	85.0	97.8	18x24in	12-#10
F.1-7	318.2	566.4	54.8	85.9	83.1	95.6	18x26in	12-#10
G.1-7	344.0	610.3	25.3	45.7	81.4	93.6	18x24in	12-#10
H.1-7	258.0	463.8	2.0	11.3	75.4	86.7	18x24in	12-#10
J.1-7	275.2	493.1	73.3	106.5	61.1	70.3	18x24in	12-#10
K.1-7	301.0	537.1	54.6	79.8	62.7	72.1	18x24in	12-#10
L.1-7	318.2	566.4	39.5	42.5	60.2	69.2	18x24in	12-#10
M.1-7	150.5	340.6	41.8	78.9	49.3	56.7	18x21in	8-#9
B.1-9	86.0	221.8	76.7	133.8	61.3	70.5	18x21in	8-#9
C.1-9	172.0	350.3	122.8	168.5	63.2	72.7	18x21in	12-#10
D.1-9	172.0	350.3	122.8	167.4	60.5	69.6	18x21in	12-#10
E.1-9	172.0	350.3	122.7	165.8	63.1	72.6	18x21in	12-#10
F.1-9	172.0	350.3	122.5	164.0	59.4	68.3	18x21in	12-#10
G.1-9	172.0	350.3	122.3	161.2	55.7	64.1	18x21in	12-#10
H.1-9	172.0	350.3	122.4	158.6	52.3	60.1	18x21in	12-#10
J.1-9	172.0	350.3	122.6	165.6	49.2	56.6	18x21in	12-#10
K.1-9	172.0	350.3	122.3	163.6	45.5	52.3	18x21in	12-#10
L.1-9	172.0	350.3	150.8	172.2	41.2	47.4	18x21in	12-#10
M.1-8	77.4	198.1	32.8	51.2	35.7	41.1	18x21in	8-#9

Lateral Design Summary

After checking the drifts and displacements due to the lateral loads and designing the beams and columns for the lateral loads, it was determined that shear walls are not necessary in the ASHA National Office building. Because the office tower is only five stories high, the inherent moment connections of the reinforced concrete structure are sufficient to resist the wind and seismic loads on the building. Reinforcing had to be added to some of the beams and columns and a small number of beams had to be upsized to resist the lateral loads. Using reinforced concrete rather than steel for the structure of the building provides a significant advantage, because it eliminates the architectural impacts of the braced frames of the existing steel office tower. The fact that shear walls are not needed allows for more flexibility for the layout of each floor in the office tower.

Parking Structure Column Check

Because the structure of the office tower was changed from steel to concrete for this report, the loads on the parking structure below will increase due to the larger self-weight of the reinforced concrete. A spot check was done to determine if the columns in the subgrade parking structure would have to be upsized due to the larger dead loads. Hand calculations were done to determine the new loads on a typical parking level column. These calculations are shown in Appendix A. The column in the subgrade parking structure was then analyzed using spColumn. Because the floor system in the parking structure is a two-way flat slab with drop panels, the column was analyzed for biaxial bending. As seen in Figure 25, the existing concrete column is adequate for the additional dead load caused by the reinforced concrete office tower.

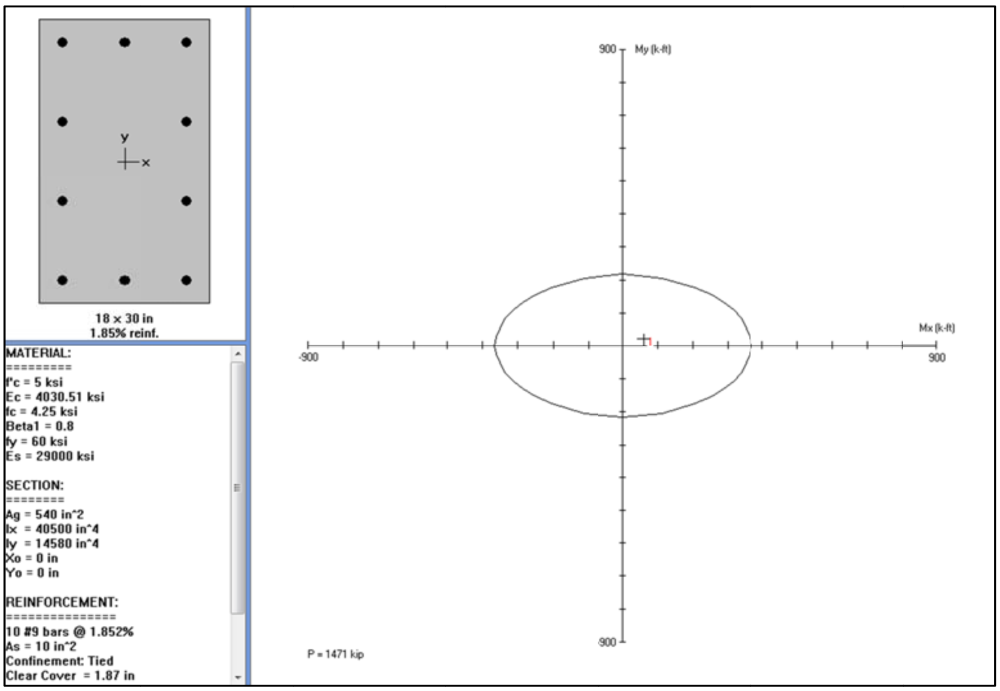


Figure 25: spColumn Typical Parking Garage Column Check

Foundation Check

A check was done to determine if the foundations can support the additional dead load from the concrete office tower. The spread footing under interior column G-3 was spot checked and it was determined that the footing would have to be upsized for the additional load. Figure 26 shows a plan view of the spread footing that was redesigned. According to the geotechnical report, the allowable soil bearing capacity is 8000 psf. It was determined that the existing 11x11 ft footing would have to be increased to 12x12 ft. The reinforcing was also designed for the footing and the footing was checked for punching shear. The 3 foot deep footing would require 12 #8 bottom bars in both directions. The footing was found to be adequate for the punching shear. Detailed hand calculations can be seen in Appendix A.

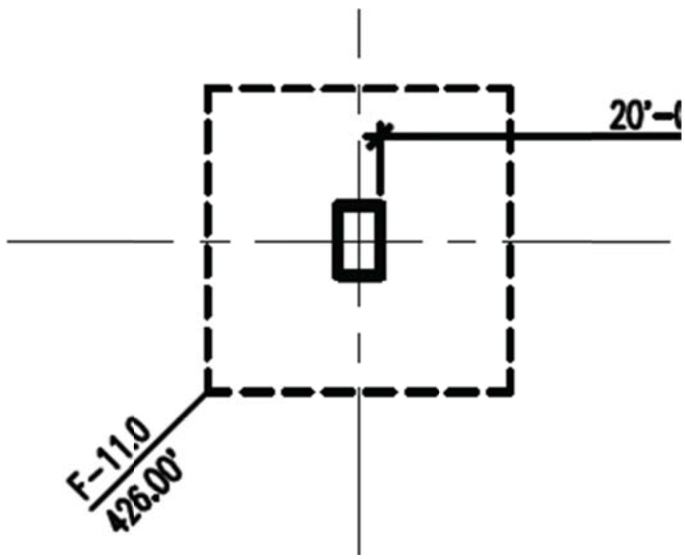


Figure 26: Plan View of Spread Footing at G-3

Architectural Breadth

The architectural breadth for this thesis report was done assuming that the two-way flat slab floor system was chosen for the office tower. This was not the system that was ultimately chosen for the building, but the impact of the additional columns created by this system was explored. The two-way flat slab system with drop panels would create two extra column lines in the E-W direction. These additional columns would create complications with the layouts of the floors of the building. The floor that is impacted the most the plaza level, therefore a floor plan was laid out for this level. The existing floor plan that was provided by the architect does not have much detail other than the locations of the conference rooms and the layout of the core of the office tower. This floor plan can be seen in Figure 27 below. The additional columns needed for the two-way flat slab system are highlighted in red.

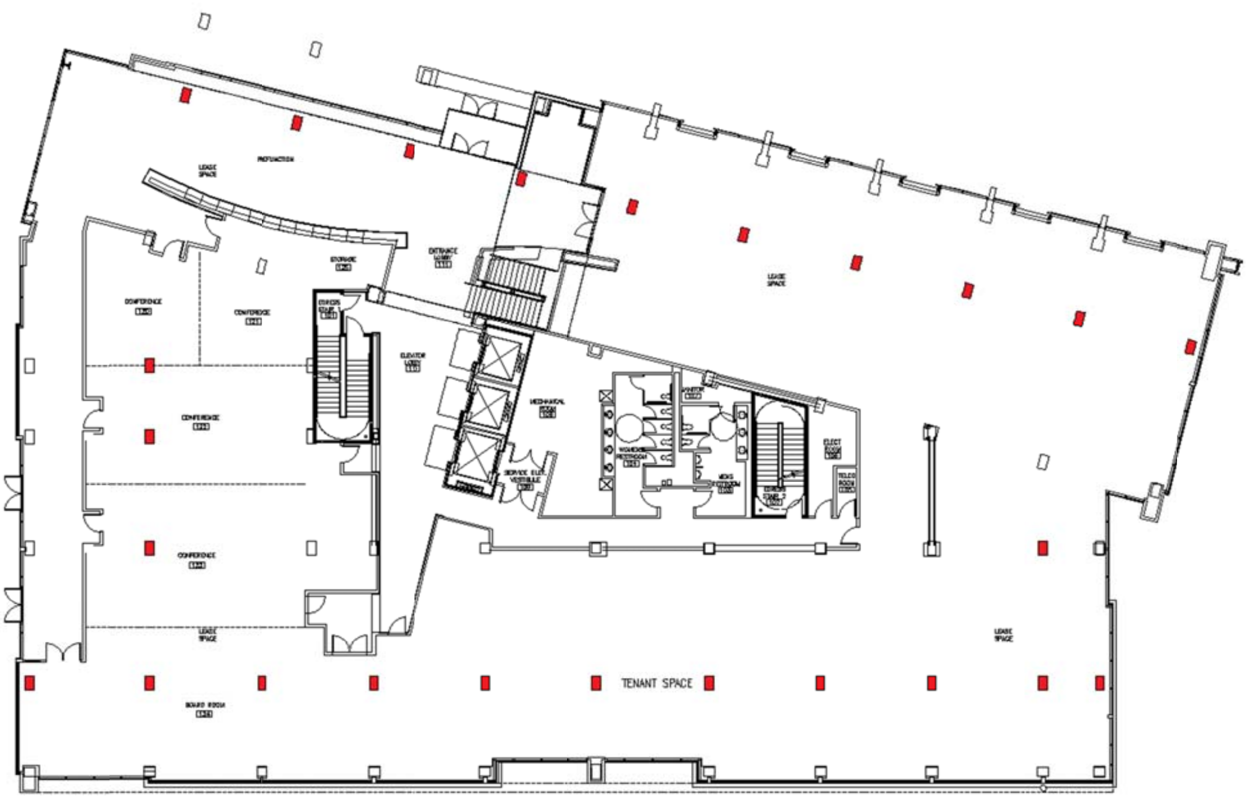


Figure 27: Original Plaza Floor Plan

Figure 28 is an enlarged floor plan of the conference rooms. It can be seen there are multiple columns in the middle of the conference rooms and the large board room. This is not acceptable, so the conference rooms would have to be rearranged so that they are free of columns.

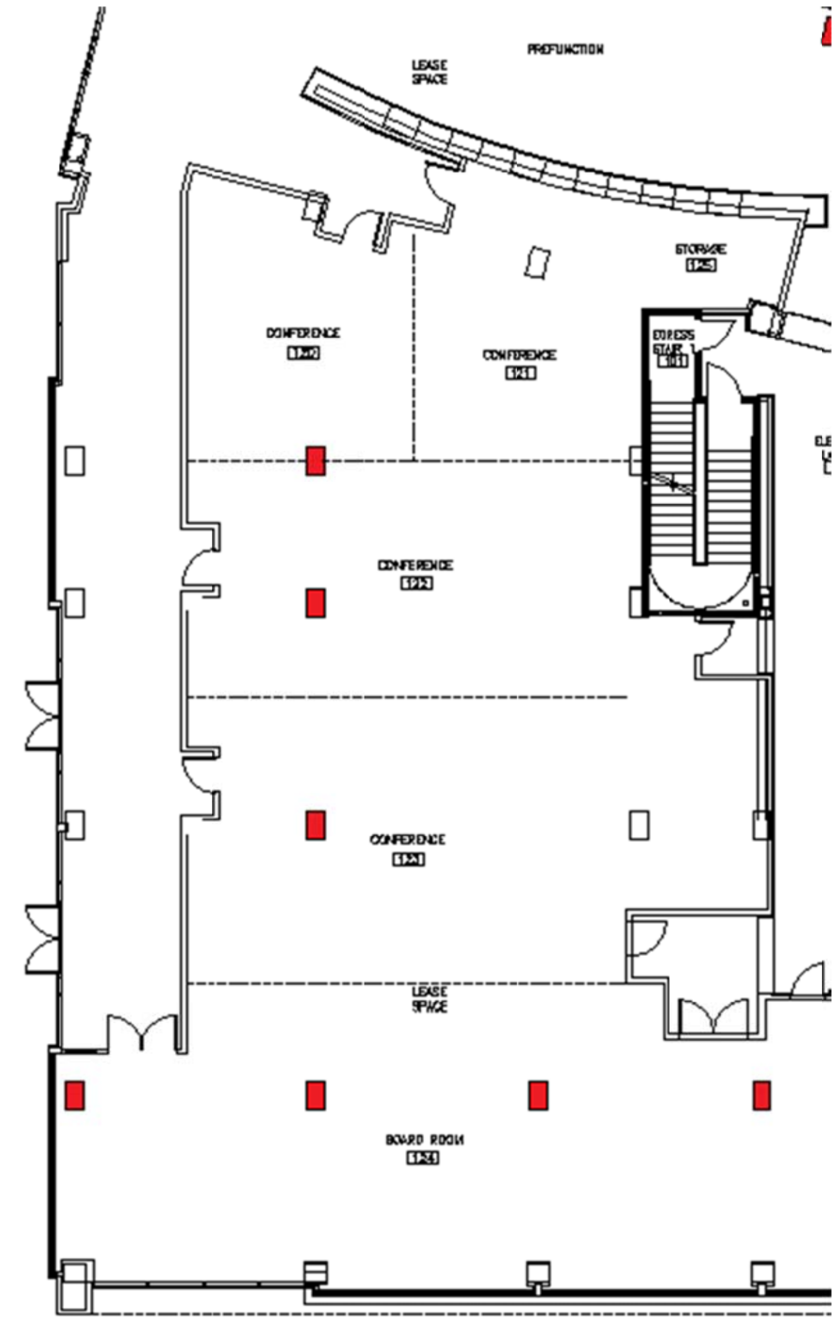


Figure 28: Enlarged Conference Room Floor Plan

Figure 29 below shows the layout for the plaza level that was created for this thesis report. The plaza level includes offices, cubicles, a café and kitchen. All of these items were included in the layout that was created. The sizes of spaces, the location of the spaces and the flow of people throughout the plaza level were all considered when this layout was created. Figure 30 is a color coded floor plan of the plaza level

Figure 29: Plaza Level Floor Plan – New Layout

Figure 30: Color Coded Plaza Level Layout

The spaces are color coded as follows:

Conference Rooms

Offices

Cubicles

Café and Kitchen

Copy Room

Storage Spaces

Lobby (Unchanged)

Core of Building (Unchanged)

Prefunction Area (Unchanged)

Circulation

Figure 31 below shows an enlarged view of the conference rooms. The number of conference rooms was kept the same, although the orientations of them changed. The large board room was moved closer to the prefunction space, and the conference rooms were moved farther to the back of the building. Possible table set-ups are shown for each of the conference rooms. The tables can be moved to accommodate any type of meeting that is being held.

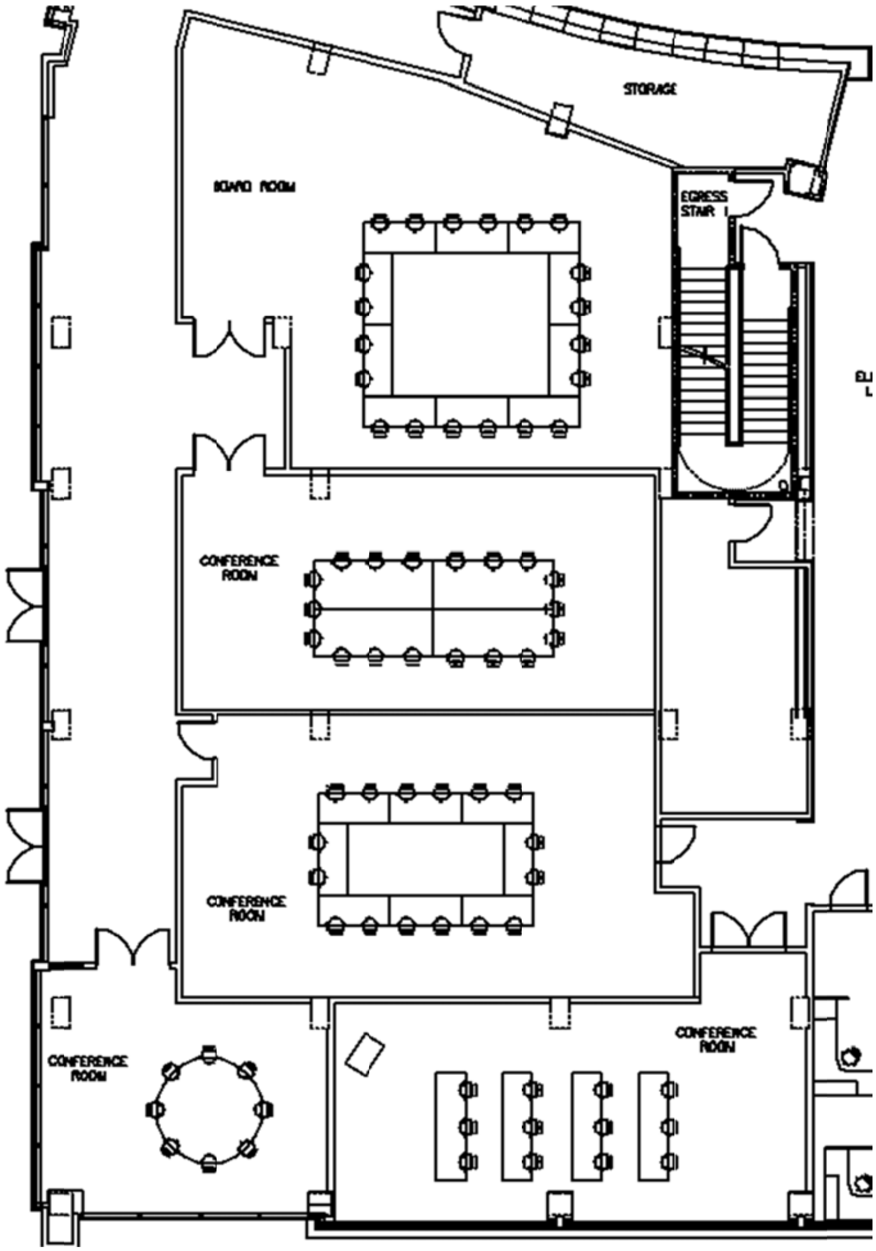


Figure 31: Enlarged Conference Room Floor Plan – New Layout

A cubicle layout was also created for part of the plaza level. Two cubicle sizes were used for the open office floor. There are 23 small cubicles that are 6 x 8 ft, and 4 larger cubicles that are 8 x 9 ft. The cubicles were arranged to maximize the number of cubicles in the open office space, and to allow for adequate circulation within the space. Figure 32 shows an enlarged view of the cubicle layout.

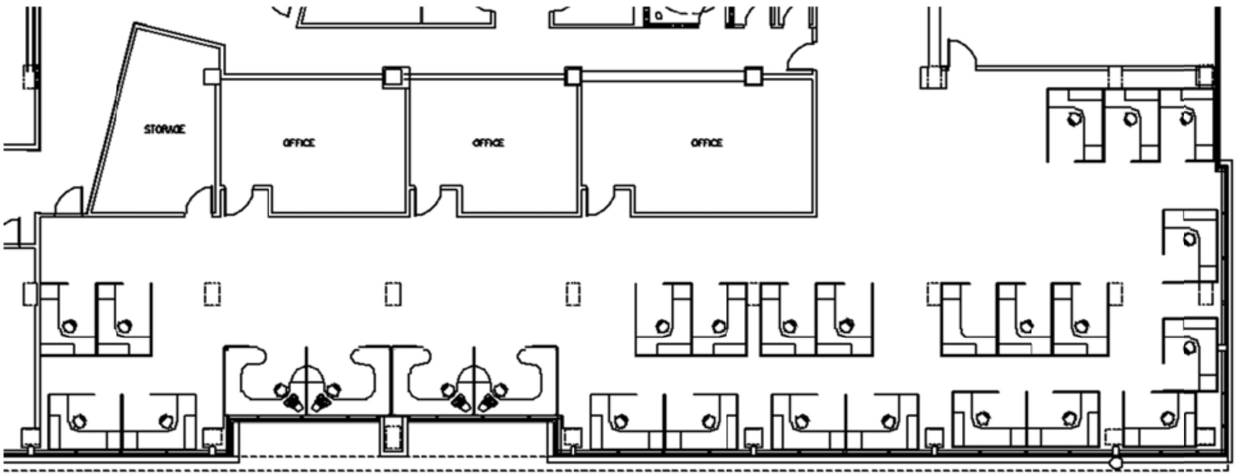


Figure 32: Enlarged Cubicle Layout

Construction Management Breadth

Redesigning the structure of the ASHA National Office building as reinforced concrete affects the construction costs and scheduling of the project. For this thesis report, a detailed cost estimate was done for the reinforced concrete structure and compared with the cost of the existing composite steel structure in order to determine the viability of the redesign. RS Means Building Construction Cost Data 2011 was used to obtain the unit prices for the concrete structure. The cost and schedule information for the existing structure was obtained from Davis Construction who was the construction manager on the project. The ASHA National Office building was constructed in 2007, so the costs obtained from Davis Construction were adjusted using the historical cost indices found in RS Means. The cost of the concrete redesign was adjusted for location by multiplying by the city cost index. The cost of the foundations and subgrade columns in the parking structure for the redesign were increased by 20% to account for the higher dead loads due to the concrete structure. Takeoffs for the concrete, formwork and reinforcement were done by hand. Detailed takeoff calculations can be seen in Appendix A. The tables below show the cost calculations for the concrete beams, columns and slabs.

Concrete Beam Cost Estimate										
Floor	Conc. Vol. (CY)	Conc Cost/CY	Placing Cost/CY	Formwork (SFCA)	Formwork Cost/SFCA	Reinf #7 & Below (Tons)	Cost/Ton #7 & Below	Reinf #8 & Above (Tons)	Cost/Ton #8 & Above	Total Cost
2nd	101.97	$122.00	$74.00	6289.50	$10.70	2.94	$2,550.00	7.94	$1,900.00	$101,087.07
3rd	101.97	$122.00	$74.00	6289.50	$10.70	2.94	$2,550.00	7.94	$1,900.00	$101,087.07
4th	101.97	$122.00	$74.00	6289.50	$10.70	2.94	$2,550.00	7.94	$1,900.00	$101,087.07
5th	101.97	$122.00	$74.00	6289.50	$10.70	2.94	$2,550.00	7.94	$1,900.00	$101,087.07
Roof	78.38	$122.00	$74.00	5453.83	$10.70	2.94	$2,550.00	7.94	$1,900.00	$88,606.83
PH Roof	6.11	$122.00	$74.00	511.00	$10.70	0.59	$2,550.00	1.59	$1,900.00	$10,289.36
									Total	$462,984.91

Concrete Column Cost Estimate										
Below Floor	Conc. Vol. (CY)	Conc Cost/CY	Placing Cost/CY	Formwork (SFCA)	Formwork Cost/SFCA	Reinf #7 & Below (Tons)	Cost/Ton #7 & Below	Reinf #8 & Above (Tons)	Cost/Ton #8 & Above	Total Cost
2nd	64.50	$122.00	$49.00	4185.00	$10.75	0.76	$2,650.00	14.55	$2,075.00	$81,151.03
3rd	64.50	$122.00	$49.00	4185.00	$10.75	0.76	$2,650.00	14.55	$2,075.00	$81,151.03
4th	64.50	$122.00	$49.00	4185.00	$10.75	0.76	$2,650.00	14.55	$2,075.00	$81,151.03
5th	54.44	$122.00	$49.00	3822.75	$10.75	0.76	$2,650.00	14.55	$2,075.00	$75,985.34
Roof	54.44	$122.00	$49.00	3822.75	$10.75	0.76	$2,650.00	14.55	$2,075.00	$75,985.34
PH Roof	10.89	$122.00	$49.00	764.55	$10.75	0.15	$2,650.00	2.91	$2,075.00	$15,197.07
									Total	$410,620.83

Concrete Slab Cost Estimate								
Below Floor	Conc. Vol. (CY)	Conc. Cost/CY	Placing Cost/CY	Formwork (SF)	Formwork Cost/SFCA	Reinf #7 & Below (Tons)	Cost/Ton #7 & Below	Total Cost
2nd	669.89	$122.00	$28.00	20499	$7.85	8.00	$1,900.00	$254,469.56
3rd	669.89	$122.00	$28.00	20499	$7.85	8.00	$1,900.00	$254,469.56
4th	669.89	$122.00	$28.00	20499	$7.85	8.00	$1,900.00	$254,469.56
5th	669.89	$122.00	$28.00	20499	$7.85	8.00	$1,900.00	$254,469.56
Roof	655.97	$122.00	$28.00	20073	$7.85	7.83	$1,900.00	$249,176.41
PH Roof	82.64	$122.00	$28.00	2529	$7.85	1.60	$1,900.00	$32,463.60
							Total	$1,299,518.23

The tables below show cost estimates of the existing composite steel structure and the redesigned concrete structure. As seen, the original steel structure is less costly than the reinforced concrete structure. This is most likely why the composite steel structure was chosen for the ASHA National Office building. The concrete redesign is approximately $500,000 more than the existing composite steel structure. This is a relatively small difference, so it can be concluded that the concrete structure is very comparable to the composite steel structure with respect to cost.

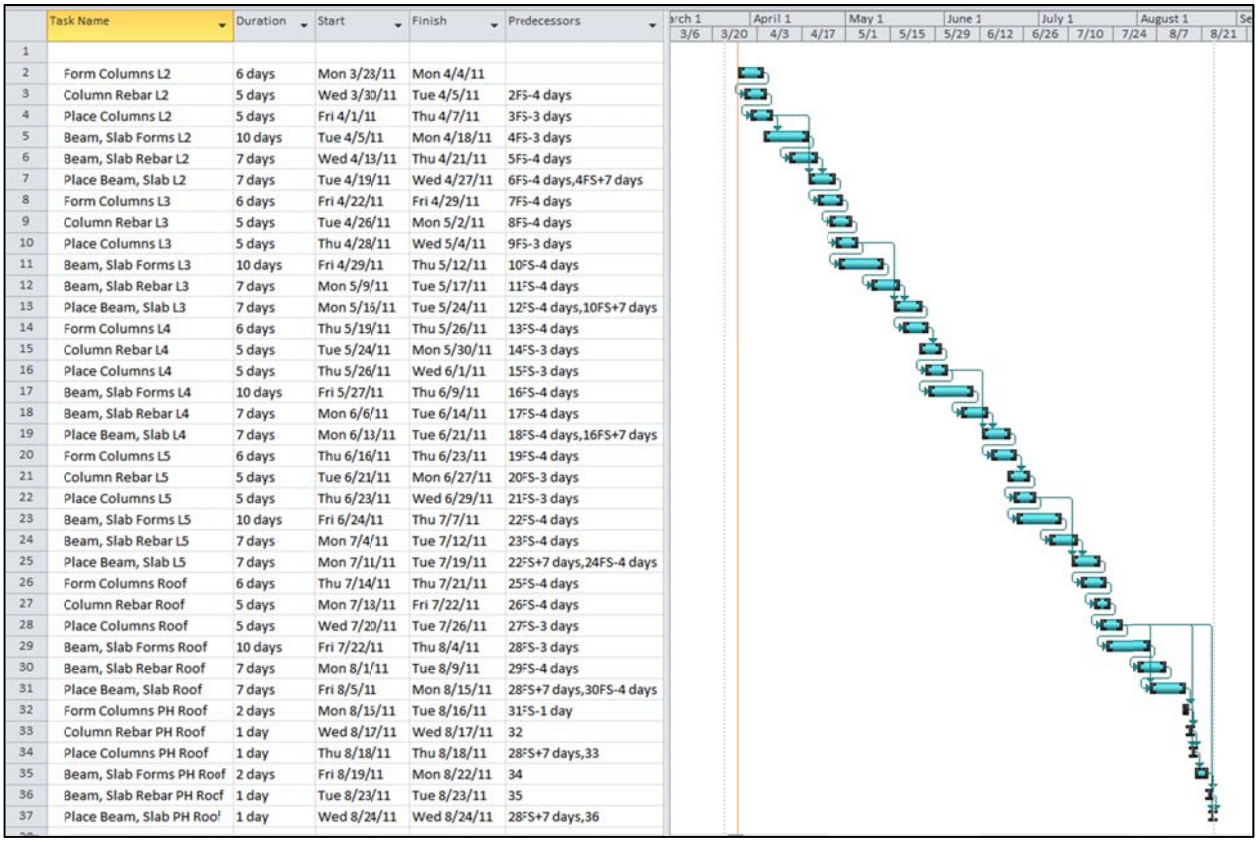
Original Steel Structure Cost		
Description	**Cost**	**Adjusted 2011 Cost**
Mobilization & Cranes	$299,498.00	$326,963
B2 Level	$1,596,426.00	$1,742,823
B1 Level	$1,096,252.00	$1,196,782
Plaza Level	$341,649.00	$372,979
2nd Floor	$62,086.00	$67,779
3rd Floor	$51,969.00	$56,735
4th Floor	$51,969.00	$56,735
5th Floor	$51,199.00	$55,894
Roof	$9,852.00	$10,755
Total Steel	$1,372,852.00	$1,498,747
Fireproofing	$82,000.00	$89,520
Total	$5,015,752.00	**$5,475,712**

Concrete Structure Cost	
Description	**Cost**
Mobilization & Cranes	326,963
B2 Level	1,887,782
B1 Level	1,239,164
Plaza Level	372,979
Beams	462,985
Columns	410,621
Slabs	1,299,518
Total	**6,000,013**

In order to further examine the feasibility of the concrete redesign, a construction schedule was created for the concrete structure and compared with the construction schedule for the existing steel structure. The construction schedule for the redesign was created using Microsoft Project. The daily output for each construction task obtained from RS Means along with the takeoffs that were previously calculated were used to calculate approximate durations for each task. The pouring of concrete beams and slabs are not poured until 7 days after the placement of concrete columns below to allow the columns to gain the necessary strength. The weather conditions were not taken into account when the construction schedule was created. Construction on the steel office tower in Rockville, MD was started in mid-December. For this reason, the construction time may increase due to the fact that extra measures may have to be taken for the pouring of concrete in cold weather. Figures 33 and 34 show the construction schedules for existing steel structure and for the redesigned concrete structure.

Steel & SOMD		61	0	61		18-Dec-06 A	14-Mar-07 A
A3080	Setup Steel Crane / Shakeout / Anchor Bolt Check	9	0	3	100%	18-Dec-06 A	20-Dec-06 A
A1320	Steel 2nd & 3rd Floors	10	0	20	100%	21-Dec-06 A	22-Jan-07 A
A1340	Steel 4th & 5th Floors	10	0	27	100%	02-Jan-07 A	07-Feb-07 A
A1330	Install Deck, Angle, Studs - 2nd & 3rd	13	0	15	100%	05-Jan-07 A	26-Jan-07 A
A1450	Install Steel Stair #1	25	0	23	100%	23-Jan-07 A	23-Feb-07 A
A1460	Install Steel Stair #2	25	0	21	100%	25-Jan-07 A	23-Feb-07 A
A1390	Pour 2nd Deck	3	0	2	100%	31-Jan-07 A	01-Feb-07 A
A1350	Install Deck, Angle, Studs - 4th & 5th	9	0	7	100%	07-Feb-07 A	16-Feb-07 A
A1360	Steel at Roof & Penthouse	5	0	7	100%	08-Feb-07 A	19-Feb-07 A
A1400	Pour 3rd Deck	0	0	1	100%	19-Feb-07 A	20-Feb-07 A
A1370	Install Deck, Angle, Studs - Roof & PH	5	0	10	100%	21-Feb-07 A	06-Mar-07 A
A1410	Pour 4th Deck	2	0	1	100%	28-Feb-07 A	28-Feb-07 A
A1420	Pour 5th Deck	2	0	1	100%	08-Mar-07 A	08-Mar-07 A
A1430	Pour Penthouse / Mechanical Pads	1	0	1	100%	14-Mar-07 A	14-Mar-07 A
A1440	Complete Concrete Pours	0	0	0	100%		14-Mar-07 A

Figure 33: Construction Schedule for Existing Steel Structure



	Task Name	Duration	Start	Finish	Predecessors
1					
2	Form Columns L2	6 days	Mon 3/28/11	Mon 4/4/11	
3	Column Rebar L2	5 days	Wed 3/30/11	Tue 4/5/11	2FS-4 days
4	Place Columns L2	5 days	Fri 4/1/11	Thu 4/7/11	3FS-3 days
5	Beam, Slab Forms L2	10 days	Tue 4/5/11	Mon 4/18/11	4FS-3 days
6	Beam, Slab Rebar L2	7 days	Wed 4/13/11	Thu 4/21/11	5FS-4 days
7	Place Beam, Slab L2	7 days	Tue 4/19/11	Wed 4/27/11	6FS-4 days,4FS+7 days
8	Form Columns L3	6 days	Fri 4/22/11	Fri 4/29/11	7FS-4 days
9	Column Rebar L3	5 days	Tue 4/26/11	Mon 5/2/11	8FS-4 days
10	Place Columns L3	5 days	Thu 4/28/11	Wed 5/4/11	9FS-3 days
11	Beam, Slab Forms L3	10 days	Fri 4/29/11	Thu 5/12/11	10FS-4 days
12	Beam, Slab Rebar L3	7 days	Mon 5/9/11	Tue 5/17/11	11FS-4 days
13	Place Beam, Slab L3	7 days	Mon 5/16/11	Tue 5/24/11	12FS-4 days,10FS+7 days
14	Form Columns L4	6 days	Thu 5/19/11	Thu 5/26/11	13FS-4 days
15	Column Rebar L4	5 days	Tue 5/24/11	Mon 5/30/11	14FS-3 days
16	Place Columns L4	5 days	Thu 5/26/11	Wed 6/1/11	15FS-3 days
17	Beam, Slab Forms L4	10 days	Fri 5/27/11	Thu 6/9/11	16FS-4 days
18	Beam, Slab Rebar L4	7 days	Mon 6/6/11	Tue 6/14/11	17FS-4 days
19	Place Beam, Slab L4	7 days	Mon 6/13/11	Tue 6/21/11	18FS-4 days,16FS+7 days
20	Form Columns L5	6 days	Thu 6/16/11	Thu 6/23/11	19FS-4 days
21	Column Rebar L5	5 days	Tue 6/21/11	Mon 6/27/11	20FS-3 days
22	Place Columns L5	5 days	Thu 6/23/11	Wed 6/29/11	21FS-3 days
23	Beam, Slab Forms L5	10 days	Fri 6/24/11	Thu 7/7/11	22FS-4 days
24	Beam, Slab Rebar L5	7 days	Mon 7/4/11	Tue 7/12/11	23FS-4 days
25	Place Beam, Slab L5	7 days	Mon 7/11/11	Tue 7/19/11	22FS+7 days,24FS-4 days
26	Form Columns Roof	6 days	Thu 7/14/11	Thu 7/21/11	25FS-4 days
27	Column Rebar Roof	5 days	Mon 7/18/11	Fri 7/22/11	26FS-4 days
28	Place Columns Roof	5 days	Wed 7/20/11	Tue 7/26/11	27FS-3 days
29	Beam, Slab Forms Roof	10 days	Fri 7/22/11	Thu 8/4/11	28FS-3 days
30	Beam, Slab Rebar Roof	7 days	Mon 8/1/11	Tue 8/9/11	29FS-4 days
31	Place Beam, Slab Roof	7 days	Fri 8/5/11	Mon 8/15/11	28FS+7 days,30FS-4 days
32	Form Columns PH Roof	2 days	Mon 8/15/11	Tue 8/16/11	31FS-1 day
33	Column Rebar PH Roof	1 day	Wed 8/17/11	Wed 8/17/11	32
34	Place Columns PH Roof	1 day	Thu 8/18/11	Thu 8/18/11	28FS+7 days,33
35	Beam, Slab Forms PH Roof	2 days	Fri 8/19/11	Mon 8/22/11	34
36	Beam, Slab Rebar PH Roof	1 day	Tue 8/23/11	Tue 8/23/11	35
37	Place Beam, Slab PH Roof	1 day	Wed 8/24/11	Wed 8/24/11	28FS+7 days,36

Figure 34: Construction Schedule for Redesigned Concrete Structure

From the construction schedule obtained from Davis Construction, it can be seen that the structure of the steel office tower was constructed in 61 days. From the construction schedule that was created, the estimated duration for the construction of the concrete office tower is 108 days. It is typical that a steel building can be built faster than a concrete building, but the lead time for steel is considerably longer than for concrete due to the fact that the steel members have to be fabricated for the building. With respect to cost and schedule, the composite steel structure appears to be a better option than the reinforced concrete structure.

Final Summary

The purpose of this project was to investigate the feasibility of changing the structural system of the ASHA National Office building from composite steel to reinforced concrete. Two different floor systems were explored; a two-way flat slab system with drop panels and a one-way slab and beam system. The one-way slab and beam system was ultimately chosen due to the additional columns that would have to be added for the two-way system. The structural system was analyzed for the lateral wind and seismic loads. It determined that the inherent moment connections of the reinforced concrete structure are sufficient to resist the lateral loads. For this reason, shear walls do not need to be implemented in the concrete structure. This will help reduce the cost and will allow for more floor plan flexibility.

An architectural study was done for the ASHA National office. The study was done assuming that the two-way flat slab system was chosen for the building. A layout for the plaza level was created taking into account the additional columns that would be required. The study shows additional columns greatly decrease the flexibility of the floor plan, and is one of the main reasons why the two-way flat slab system was not chosen for the building.

The cost estimate that was created for the concrete redesign shows that the existing steel structure is a more economical choice for the structure. The concrete structure is only approximately $500,000 more than the steel structure, so concrete is a viable alternative with respect to cost. The construction time for the concrete structure is significantly longer than for the steel structure. For this reason, if time is crucial then the existing steel structure is the best choice.

References

American Concrete Institute. “Building Code Requirements for Structural Concrete” ACI, Farmington Hills, MI, 2008

American Society of Civil Engineers, “ASCE7-05: Minimum Design Loads for Buildings and Other Structures” ASCE, Reston, VA, 2005

Wight, James K., MacGregor, James G. “Reinforced Concrete Mechanics and Design” Pearson Education, Inc., Upper Saddle River, New Jersey 2009

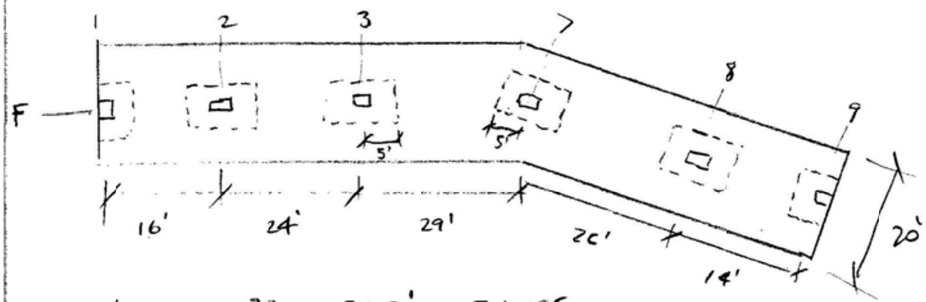
RSMeans Company, Inc. “Building Construction Cost Data” RSMeans, Kingston, Ma, 2011

Fanella, David A. “Concrete Floor Systems: Guide to Estimating and Economizing” PCA, Skokie, IL, 2000

Appendix A: Calculations

TWO WAY FLAT SLAB 1/19/11

COLUMN LINE F



$l_n = 29 - \frac{30}{12} = 26.5'$ Interior

$l_n = 16 - \frac{30}{12} = 13.5'$ Exterior

MIN SLAB THICKNESS

Int $l_n/36 = 26.5/36 \cdot 12 = 8.8''$ => Try 9" thick Slab

Ext $l_n/33 = 13.5/33 \cdot 12 = 4.9''$

Drop panel Size

$l/6 = 29/6 = 4.83'$ => USE 5' for center span

$24/6 = 4'$
$26/6 = 4.33'$ } USE 4.5' for all other spans for uniformity

* SP slab used to design reinforcing (Results Attached)

Drop panel thickness used: $4\frac{1}{4}''$ => $9 + 4.25 = 13.25$

4x Lumber = $3\frac{1}{2}'' + 3/4'' = 4\frac{1}{4}''$ ∴ OKAY FOR FORMWORK (3/4" = plywood)

$\frac{13.25}{9} = 1.47 > 1.25$ ∴ OK

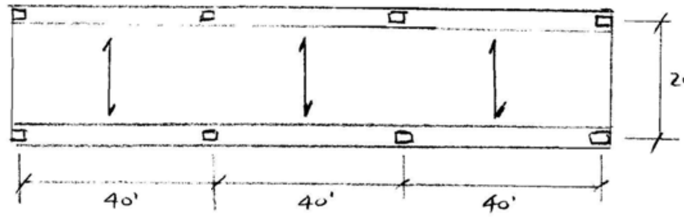
COLUMN LINE G

CENTER SPAN = 35'

$l_n/36 = 35/36 \cdot 12 = 11.7''$ => MUST USE 12" AT THIS SPAN

* EXTEND DROP PANEL TO SPAN ENTIRE LENGTH TO SIMPLIFY

One way Slab Design 1/19/11



SDL = 25 psf

LL = 100 psf

f'_c = 5 ksi

f_y = 60 ksi

$l/28 = 0.714' = 8.57'' \approx 9''$

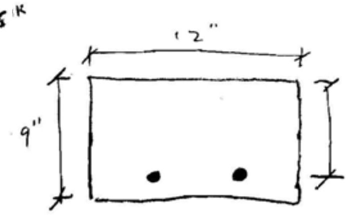
$A_{s,min} = 0.0018 \cdot 12 \cdot 6 = 0.13 \text{ in}^2$

Try 9" Slab

$w_u = [1.2(25 \cdot 1 + 150 \cdot 9/12 \cdot 1) + 1.6 \cdot 100]/1000 = 0.325 \text{ k/ft}$

$M_u = \dfrac{0.325 \cdot 20^2}{8} = 16.25^{\text{'k}}$

$\dfrac{16.25}{4 \cdot 7.94} = 0.52 \text{ in}^2$



$d = 9 - 3/4 - 1/2 \cdot 5/8 = 7.94$

Try #5 bars @ 6"

$A_s = 2 \cdot 0.31 = 0.62 \text{ in}^2$

$a = \dfrac{0.62 \cdot 60}{0.85 \cdot 5 \cdot 12} = 0.729 \text{ in}$

$c = \dfrac{0.729}{0.85} = 0.86 \text{ in} < 0.375d = 0.375 \cdot 7.94 = 2.98$

$\therefore \varepsilon_t > 0.005$

$\phi = 0.9$

$\phi M_n = 0.9 \cdot 0.62 \cdot 60 \left(7.94 - \dfrac{0.73}{2}\right)/12$

$\phi M_n = 21.1^{\text{'k}} > M_u = 16.95^{\text{'k}}$ $\therefore$ okay

$\therefore$ Use #5 bars @ 6" O.C.
for 9" deep slab

Economical Floor Design 1/26/11

RSMeans Concrete + Masonry Cost Data 2011
29th Annual Edition
Norwell, MA
Reed Construction Data

03 31 05.35 Normal weight concrete
0400 5000psi Concrete $122/CY

03 21 10.60 Reinforcing in Place
0100 Beams + Girders #3 - #7 $2,500/TON
0150 Beams + Girders #8 - #18 $1,900/TON

03 31 05.70 Placing Concrete
0200 Large beams, pumped $49/CY
1500 Elevated Slabs Pumped 6"-10" thick $31.50/CY
0800 24" thick columns pumped $48.50/CY

03 11 13.20 Forms in place, Beams and girders
0500 Ext. Spandrel 12" wide 2 use $12.70/SFCA
1000 " " 18" " " $12.60/SFCA
1500 " " 24" " " $11.90/SFCA
2000 INT Beam 24" wide 2 use $9.70/SFCA

03 11 13.35 Forms in place, Elevated Slabs
2050 Flat Slab, drop panels to 15' high 2 use $8.20/SF

03 11 13.25 Forms in place, Columns
6550 24x24 col 2 use $10.75/SFCA

Economical Beam Design 1/26/11

40 FT SPAN

LIMIT Depth to 26" because depth of composite steel floor is 26.5"

24x26

#8 bars

$4 \cdot 102.5$
$3 \cdot 56.7$
$6 \cdot 143.4$
$5 \cdot 70.3$
$4 \cdot 480$
$3 \cdot 267.2$

$\left.\right\} \frac{4513.6}{12} = 376.13'$

#3 bars

$51 \cdot (26.2 + 24) = \frac{3876''}{12} = 323'$

concrete $-(24 \times 26)/12^2 \cdot 40/27$ 6.42 CY

$376.13 \cdot 2.67 = 1004.28$ lb $= 0.5021$ TON $\cdot 1900 = \$954.06$
$323 \cdot 0.376 = 121.45$ lb $= 0.06073$ TON $\cdot 2500 = \$151.83$

COST EST. 2011

CONC $122 \cdot 6.42 = \$783.24$
placing $49 \cdot 19.26 = \$943.74$
forms $9.70 \cdot 200$ SFCA $= \$1940$

$$\begin{array}{r} 783.24 \\ 943.74 \\ 1940.00 \\ 954.06 \\ +\ 151.83 \\ \hline \$4772.9 \end{array}$$

per 40' beam

Economical Beam Design 1/26/11

20 x 26

#8 bars

4 · 58.7
4 · 107.1
6 · 143.4
5 · 69.9
4 · 480
3 · 279.1

$\frac{385.87 \cdot 2.67}{2000} \cdot 1900 = \$978.76$

#3 bars

$49(26 \cdot 2 + 20) = \frac{3528}{12} = \frac{294' \cdot 0.376}{2000} \cdot 2500 = \$138.18$

concrete $\frac{20 \cdot 26}{12^2} \cdot \frac{40}{27} = 5.35\ cy\,(122 + 49) = \$914.85$

forms $9.70 \cdot 180\ SFCA = \$1746.00$

978.76
138.18
914.85
1746.00

\$3777.79 per 40' beam

18x26

#8

4 · 59.8
4 · 107.1
6 · 143.4
5 · 69.7
4 · 480
3 · 289.1

$\frac{388.65 \cdot 2.67}{2000} \cdot 1900 = \$985.80$

#3 $57 \cdot (26 \cdot 2 + 18) = \frac{3990}{12} \cdot \frac{0.376}{2000} \cdot 2500 = \$156.3$

concrete $\frac{18 \cdot 26}{12^2} \cdot \frac{40}{27} = 4.81\ cy\,(122 + 49) = \$823.33$

forms $9.70 \cdot 173.33 = \$1681.33$

$\boxed{\$3646.76 \text{ per } 40' \text{ beam}}$

Floor System Comparison 2/3/11

Flat Slab system

9" slab

$\frac{9}{12} \cdot 20 \cdot 40 = 600\ ft^3/27 = 22.2\ CY$

4¼" drop panels

$3.5 \cdot 4.5 \cdot \frac{4.25}{12} = 5.58\ ft^3/27 = 0.207\ CY$

5 bars

$$\left.\begin{array}{l} 8 \cdot 192 \\ 8 \cdot 192 \\ 8 \cdot 88.1 \\ 7 \cdot 288 \\ 14 \cdot 121.1 \\ 6 \cdot 192 \\ 2 \cdot 65.4 \\ 6 \cdot 192 \\ 4 \cdot 69.9 \\ 7 \cdot 288 \\ 2 \cdot 66.6 \\ 8 \cdot 100.1 \\ 13 \cdot 121.1 \\ 13 \cdot 66.6 \end{array}\right\} \frac{15592.7''}{12} = 1299.4'$$

$1299.4 \cdot 1.043 = 1355.3\ lb$

$\frac{1355.3}{2000} = 0.678\ ton$

$0.678 \cdot 2500.00$

$= \$1695.00$

columns

$$\left.\begin{array}{l} \frac{18 \times 30}{12^2} \cdot 2 \cdot 13.5 = \frac{101.25\ ft^3}{27} = 3.75\ CY \\ \frac{21 \times 24}{12^2} \cdot 13.5 = \frac{47.25\ ft^3}{27} = 1.75\ CY \end{array}\right\} 5.5\ CY$$

Floor System Cost Comparison 2/3/11

cost est.

conc.

$122(5.5 + 22.2 + 0.207) = \$3404.65$

placing

$31.50(22.2 + 0.207) + 48.50 \cdot 5.5 = \$972.57$

forms

$8.20(20 \cdot 40) + 10.75(108 \cdot 2 + 101.25) = \$9970.44$

$$\begin{array}{r} 1695.00 \\ 3404.65 \\ 972.57 \\ +\ 9970.44 \\ \hline \$16042.66 \end{array}$$ per bay

one way beam system

9" slab

$600 - 1.5 \cdot \frac{9}{12} \cdot 40 = 555\ ft^3 / 27 = 20.6\ cy$

18x26 Beam

\$3646.76 (previously calculated)

#5 bars in slab

$80 \cdot 20 = 1600\ ft$

$1600 \cdot 1.043 = 1668.8\ lb$

$\frac{1668.8}{2000} = 0.8344\ ton$

columns

$\frac{3.75}{2} + 1.75 = 3.62\ cy$

cost est.

conc (slab & col)

$122 \cdot (3.62 + 20.6) = \$2954.84$

placing

$31.50 \cdot 20.6 + 48.50 \cdot 3.62 = \$824.45$

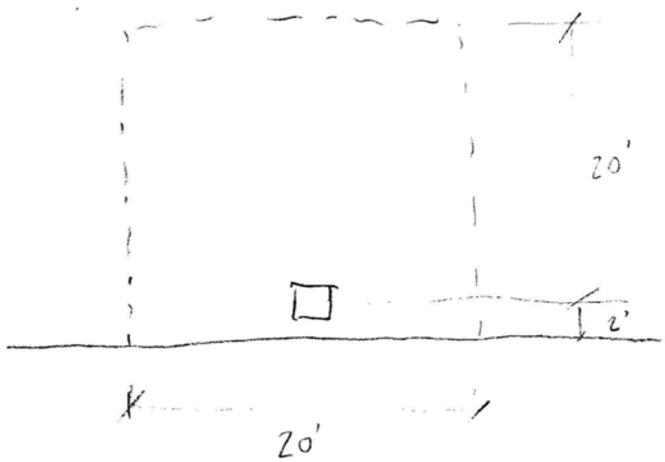
forms

$10.75 \cdot (108 + 101.25) + 8.20 \cdot (20 \cdot 40) = \$8809.44$

Total = \$16,235.49 per bay

Column Design 2/7/11

Edge Column Below Level 2 ie. col F1



$A_T = 22 \times 20 = 440 \text{ ft}^2$

Slab / floor

$\frac{9}{12} \text{ ft} \cdot 440 \text{ ft}^2 \cdot 150 \frac{\text{lb}}{\text{ft}^3} = 49{,}500 \text{ lb} = 49.5 \text{ k}$

beams / floor

$\frac{18}{12} \text{ ft} \cdot \frac{26}{12} \text{ ft} \cdot 20 \text{ ft} \cdot 150 \frac{\text{lb}}{\text{ft}^3} = 9{,}750 \text{ lb} = 9.75 \text{ k}$

ext. facade / fl.

$400 \frac{\text{lb}}{\text{ft}} \cdot 20 \text{ ft} = 8000 \text{ lb} = 8 \text{ k}$

Total Dead Load

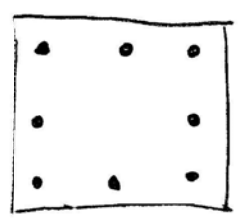
$\underbrace{49.5 \cdot 5}_{\text{Slab}} + \underbrace{9.75 \cdot 5}_{\text{beams}} + \underbrace{8 \cdot 4.5}_{\text{facade}} + \underbrace{\frac{10 \cdot 440}{1000} \cdot 5}_{\text{SDL}} = 354 \text{ k}$

Total Live Load

$\frac{100 \cdot 440 \cdot 4}{1000} + \frac{30 \cdot 440}{1000} = 189.2 \text{ k}$

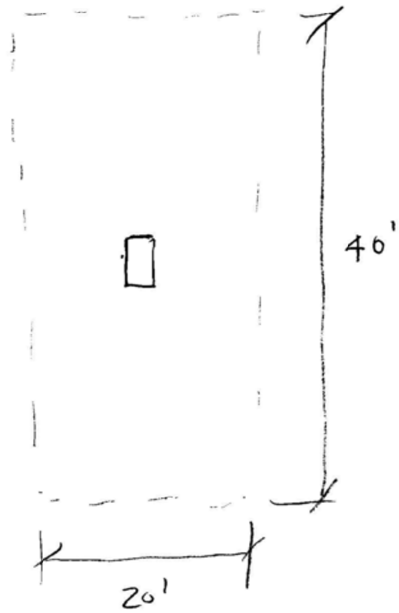
$M_x = 391.4 \text{ 'k}$
SPBeam Results

SPColumn Design
21" × 21"
8 - #10 BARS



Column Design 2/7/11

Interior column below level 2 i.e. col H3



$A_T = 20 \cdot 40 = 800\ ft^2$

Slab $\frac{9}{12} \cdot 800 \cdot 150 = 90\ K$

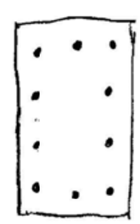
beams $\frac{18}{12} \cdot \frac{26}{12} \cdot 40 \cdot 150 = 19.5\ k$

Total Dead Load

$90 \cdot 5 + 19.5 \cdot 5 + \frac{10 \cdot 800}{1000} \cdot 5 = 588\ k$

Total Live load

$\frac{100 \cdot 800 \cdot .4}{1000} + \frac{30 \cdot 800}{1000} = 344\ k$



SP Column Design

18" x 24"

10 - #10 Bars

Transfer Girders 2/16/11

Beam from L.1-7 to L.1-9

Point Load on beam

$$\frac{21}{12}\cdot\frac{14}{12}\cdot 11'\cdot 150\,\frac{lb}{ft^3} = 3370\ lb \text{ (Self wt. of Bm)}$$

$P_D = 4.47\ k$

$P_L = 11\ k$

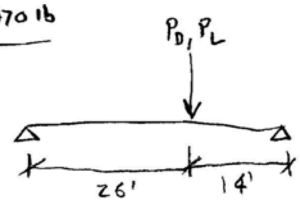
SDL

$10\,\frac{lb}{ft^2}\cdot 10'\cdot 11' = 1100\ lb$

Total dead = 4470 lb

Live Load

$100\,\frac{lb}{ft^2}\cdot 10'\cdot 11' = 11{,}000\ lb$



Beam From M.2-3 to K.5-3

Point Loads

$$\frac{18}{12}\cdot\frac{26}{12}\cdot 150\cdot 20 = 9750\ lb \text{ (self wt. of beam) } 18\times 26$$

$P_D = 18.4\ k$

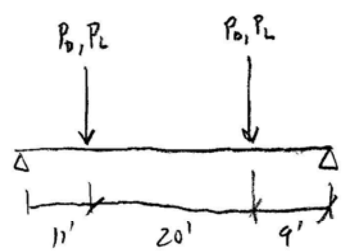
$P_L = 60\ k$

SDL

$10\,\frac{lb}{ft^2}\cdot 20\cdot 20 = 4000\ lb$

Live

$100\cdot 20\cdot 20 = 40{,}000\ lb$



$$\frac{18}{12}\cdot\frac{14}{12}\cdot 150\cdot 10 = 2625\ lb$$

SDL

$10\cdot 20\cdot 10 = 2000\ lb$

Live

$100\cdot 20\cdot 10 = 20{,}000\ lb$

Total Dead = 9750 + 4000 + 2625 + 2000 = 18.4 k

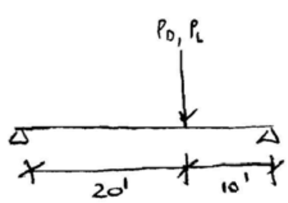
Total Live = 60 k

Beam From D-1 to B-1

Self = 9750 lb

SDL $10\cdot 20\cdot 15 = 3000\ lb$

Live $100\cdot 20\cdot 15 = 30{,}000\ lb$



$P_D = 12.8\ k$

$P_L = 30\ k$

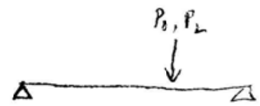
Transfer Girders 2/16/11

Beam From D-3 to B-3

$\frac{14}{12} \cdot \frac{18}{12} \cdot 150 \cdot 7 = 525\ lb$ self

SDL $10 \cdot 15 \cdot 7 = 1050\ lb$

Live $100 \cdot 15 \cdot 7 = 10{,}500\ lb$



$P_D = \frac{525 + 1050}{1000} + 12.8 = 14.4\ k$

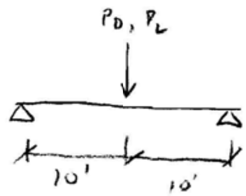
$P_L = 40.5\ k$

Beam from C.1-7 to B.1-7

$\frac{18}{12} \cdot \frac{14}{12} \cdot 150 \cdot 6 = 1575\ lb$ self

SDL $10 \cdot 5 \cdot 6 = 300\ lb$

Live $100 \cdot 5 \cdot 6 = 3000\ lb$



$P_D = 1.9\ k$

$P_L = 3\ k$

Seismic Calcs 2/20/11

Seismic Requirements from Drawings:

$S_s = 0.16$	Site class C	$R = 3$
$S_1 = 0.05$	$I_e = 1.0$	$C_s = 0.019$
$S_{DS} = 0.128$	Seismic Use group I	
$S_{D1} = 0.06$	Design Category A	PROCEDURE: Equiv. Lateral Force Procedure

Seismic Force Resisting System: Steel Braced Frames + Shear walls

Mapped Acceleration Parameters ASCE 7-10

$S_s = 0.14$ (Figure 22-1)

$S_1 = 0.05$ (Figure 22-2)

Spectral Response Acceleration Parameters - Site class C

$F_a = 1.2$ Table 11.4-1

$F_v = 1.7$ Table 11.4-2

$S_{MS} = F_a S_s = 1.2 \cdot 0.14 = 0.168$ (Eq. 11.4-1)

$S_{M1} = F_v S_1 = 1.7 \cdot 0.05 = 0.085$ (Eq. 11.4-2)

Design Spectral Acceleration Parameters

$S_{DS} = 2/3\, S_{MS} = 2/3 \cdot 0.168 = 0.112$ (Eq. 11.4-3)

$S_{D1} = 2/3\, S_{M1} = 2/3 \cdot 0.085 = 0.057$ (Eq. 11.4-4)

Seismic Design Category

Risk Category II

$S_{DS} < 0.167$

$S_{D1} < 0.067$ $\therefore$ SDC A

Response Modification Coeff - Ordinary Concrete Moment Frame

$R = 3$ (TABLE 12.2-1)

Approximate Fundamental Period

$T_a = C_t h_n^x$ (Eq. 12.8-7)

$C_t = 0.016$ $x = 0.9$ $h_n = 89$ ft

$T_a = 0.016 \cdot 89^{0.9} = 0.91$ s

$T_a = 0.1N = 0.1 \cdot 7 = 0.70$ s (12.8-8)

Seismic Calcs 2/20/11

Equivalent Lateral Force Procedure (Both Directions)

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{0.112}{3/1} = 0.037 \qquad (\text{Eq. } 12.8\text{-}2)$$

$C_u = 1.7$ (Table 12.8-1)

$C_u T_a = 1.7 \cdot 0.7 = 1.19\ s$

From ETABS Model:

$T_x = 3.224\ s$

$T_y = 2.152\ s$ ∴ use $T = 1.19\ s$

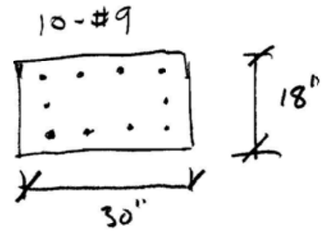
$T_z = 1.955\ s$

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{0.057}{1.19\left(\frac{3}{1}\right)} = 0.016 < 0.037 \quad (\text{Eq } 12.8\text{-}3)$$

$$\boxed{C_s = 0.016}$$

Parking Garage Column Check 2/25/11

Column D-3



$M_{ux} = 61\,'k \quad M_{uy} = 20\,'k$

$P_{u\,above} = 1189$

$(16+13)\cdot 20 = 580\ ft^2$

$P_L = 322.5 + 100 \cdot 580 \cdot 2/1000 = 438.5\ k$

$P_D = 493.1 + \frac{18 \cdot 30}{12^2} \cdot 150 \cdot 10/1000 + (112.5+10)\cdot 580 \cdot 2/1000 = 640.8$

$P_U = 1.2 \cdot 640.8 + 1.6 \cdot 438.5 = 1471\ k$

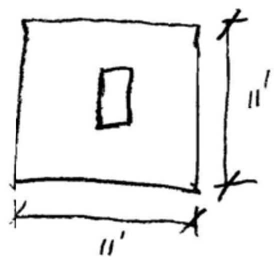
SP column was used to determine that the column is adequate for the gravity loads

10 - # 9 BARS OKAY

Foundation Check 3/18/11

11x11 spread footing G-3 36" thick

13 # 8 EWB (EACH WAY BOTTOM)



Load $P = 1080$ k from above

soil bearing capacity 8000 psf (FROM GEOTECHNICAL REPORT)

self wt. of footing $= \dfrac{11 \cdot 11 \cdot 3 \cdot .150}{1000} = 55\text{ k}$

Total load on soil $= 55 + 1080 = 1135\text{ K}$

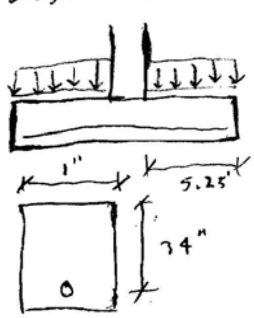
Footing capacity $= 8000 \frac{\text{lb}}{\text{ft}^2} \cdot 11\text{ft} \cdot 11\text{ft} / 1000 = 968\text{ k}$

Reqd. Dim of Footing

$\dfrac{1135\text{ k}}{b^2\ \text{ft}^2} = 8\text{ ksf}$

$b = 11.9$ ft $\Rightarrow$ use $12' \times 12'$ footing

Design of Reinforcing $f'_c = 3000$ psi



Soil wt. $100 \frac{\text{lb}}{\text{ft}^3} \cdot 1.5' = 150 \frac{\text{lb}}{\text{ft}^2}$

weight of footing $= 150 \frac{\text{lb}}{\text{ft}^3} \cdot 3' = 450 \frac{\text{lb}}{\text{ft}^2}$

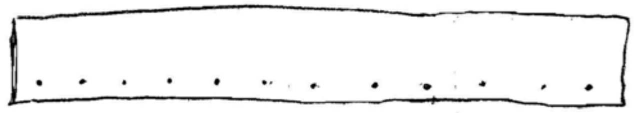
$M_u = \dfrac{wL^2}{2} = \dfrac{1.2(1.5 + 4.5) \cdot 5.25^2}{2} = 99^{\text{ik}}$

$\dfrac{99}{4.34} = 0.72'' \Rightarrow$ Try 1 # 8 $A_s = 0.79$

$a = \dfrac{0.79 \cdot 60}{0.85 \cdot 3 \cdot 12} = 1.55$ $\quad \phi M_n = 0.9 \cdot 0.79 \cdot 60 \left(34 - \frac{1.55}{2}\right) / 12$

$\phi M_n = 118^{\text{ik}} > M_u = 99^{\text{ik}}$ $\therefore$ OK

Foundation Check 3/21/11

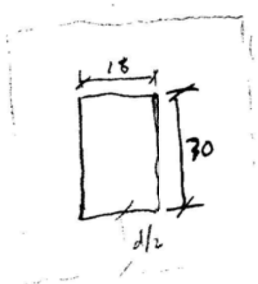


Reinforcing Reqd. 12 #8 Bottom Bars Each Way

Punching Shear Check

3' deep footing

column 18" x 30"



$d/2 = 34"/2 = 17"$

$b_o = (17 \cdot 2 + 30) \cdot 2 + (18 + 2 \cdot 17) \cdot 2 = 232"$

↑ perimeter

$\phi V_c = 0.75 \cdot 4\sqrt{f'_c} \cdot b_o d = 0.75 \cdot 4 \cdot \sqrt{3000} \cdot 232 \cdot 34 / 1000$

$\phi V_c = 1296$ K

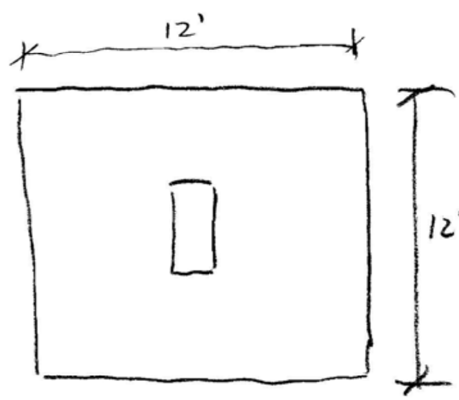
$V_u = 1471 - 8\left(\frac{64}{12} \times \frac{52}{12}\right) = 1286\text{ k} < \phi V_c = 1296\text{ k}$ ∴ OKAY

Design of footing

12x12 ft

3' deep

12 #8 EWB



Effective Seismic Weight Calculations

Concrete Beams / typical floor			
Size	Number	Length (ft)	Weight (kips)
26x18	24	40	306.0
14x18	3	20	5.6
16x18	3	25	9.8
21x18	3	29	19.6
26x18	2	35	22.3
		Total	**363.4**

Slabs			
Floor	Area (ft^2)	Thickness (in)	Weight (kips)
Parking	23285	9	2619.6
Plaza	23285	9	2619.6
2	24116	9	2713.1
3	24116	9	2713.1
4	24116	9	2713.1
5	23615	9	2656.7
Roof	23615	9	2656.7

Columns		
Size	Number	Weight/ft (kip/ft)
18x30	69	38.8
18x21	21	8.3
18x24	25	11.3
18x18	21	7.1
18x20	25	9.4

Floor	Height Below (ft)	Height Above (ft)	Weight of Cols. (kips)
Parking	5	5	388.1
Plaza	5	7.5	340.5
2	7.5	6.75	278.1
3	6.75	6.75	263.5
4	6.75	6.75	242.9
5	6.75	6.75	222.2
Roof	6.75	0	111.1

Effective Seismic Weight		
Floor	**Weight**	
Parking	3007.7	k
Plaza	2960.0	k
2nd	3354.5	k
3rd	3339.9	k
4th	3294.0	k
5th	3191.7	k
Roof	3105.9	k
Total	22253.6	k

$V=C_sW=$ 578.6 k

Vertical Distribution of Seismic Forces						
Floor	**w_x**	**h_x (ft)**	**w_xh_x^k**	**C_{vx}**	**F_x**	
Parking	3007.7	10.0	38746.5	0.024	14.2	k
Plaza	2960.0	20.0	82307.1	0.052	30.1	k
2nd	3354.5	35.0	173600.0	0.110	63.4	k
3rd	3339.9	48.5	248260.7	0.157	90.7	k
4th	3294.0	62.0	321568.9	0.203	117.5	k
5th	3191.7	75.5	387737.7	0.245	141.6	k
Roof	3105.9	89.0	452901.6	0.286	165.4	k
		Sum	1584068.9	1.000	**578.6**	**k**

Takeoff Calculations

Concrete Beam Takeoffs (Typical Floor)				
Width (in)	**Depth (in)**	**Length (ft)**	**# of Beams**	**Volume (CY)**
18	26	40	20	62.96
18	14	14	1	0.32
18	14	20	5	2.31
18	16	25	3	2.43
18	21	29	2	3.22
18	26	35	2	5.51
18	21	40	4	8.89
21	16	20	10	7.56
12	16	20	1	0.43
24	16	30	1	1.30
21	14	20	8	4.32
21	24	30	1	2.43
18	14	12	1	0.28
			Total	**101.97**

Concrete Beam Takeoffs (Roof)				
Width (in)	**Depth (in)**	**Length (ft)**	**# of Beams**	**Volume (CY)**
18	21	40	20	44.44
18	14	14	1	0.32
18	14	20	5	2.31
18	16	25	3	2.43
18	18	29	2	2.42
18	21	35	2	3.89
18	21	40	4	8.89
21	14	20	10	5.40
12	16	20	1	0.43
24	16	30	1	1.30
21	14	20	8	4.32
21	21	30	1	1.94
18	14	12	1	0.28
			Total	**78.38**

Concrete Beam Takeoffs (PH Roof)				
Width (in)	**Depth (in)**	**Length (ft)**	**# of Beams**	**Volume (CY)**
18	14	20	2	0.93
18	14	25	2	1.16
18	21	40	1	2.22
18	16	35	1	1.13
18	14	29	1	0.67
			Total	**6.11**

Concrete Beam Takeoffs	
Floor	**Volume**
2nd	101.97
3rd	101.97
4th	101.97
5th	101.97
Roof	78.38
PH Roof	6.11
Total	**492.38**

Concrete Column Takeoffs (Lower Floors)				
Column Dimensions (in x in)		**Height (ft)**	**# of Cols.**	**Volume (CY)**
18	21	13.5	26	34.13
18	24	13.5	17	25.50
18	26	13.5	3	4.88
			Total	**64.50**

Concrete Column Takeoffs (Above Splice)				
Column Dimensions (in x in)		**Height (ft)**	**# of Cols.**	**Volume (CY)**
18	18	13.5	26	29.25
18	20	13.5	17	21.25
18	21	13.5	3	3.94
			Total	**54.44**

Concrete Column Takeoffs	
Below Floor	**Volume**
2nd	64.50
3rd	64.50
4th	64.50
5th	54.44
Roof	54.44
PH Roof	10.89
Total	**302.38**

Concrete Slab Takeoffs			
Floor	**Thickness (in)**	**Area (sq. ft.)**	**Volume (CY)**
2nd	9	24116	669.89
3rd	9	24116	669.89
4th	9	24116	669.89
5th	9	24116	669.89
Roof	9	23615	655.97
PH Roof	9	2975	82.64
		Total	**3418.17**

Beam Formwork Takeoffs (Typical Floor)				
Width (in)	**Depth (in)**	**Length (ft)**	**# of Beams**	**Formwork (SFCA)**
18	26	40	20	3466.67
18	14	14	1	32.67
18	14	20	5	233.33
18	16	25	3	200.00
18	21	29	2	203.00
18	26	35	2	303.33
18	21	40	4	560.00
21	16	20	10	583.33
12	16	20	1	43.33
24	16	30	1	95.00
21	14	20	8	413.33
21	24	30	1	127.50
18	14	12	1	28.00
			Total	**6289.50**

Beam Formwork Takeoffs (Roof)				
Width (in)	**Depth (in)**	**Length (ft)**	**# of Beams**	**Formwork (SFCA)**
18	21	40	20	2800.00
18	14	14	1	32.67
18	14	20	5	233.33
18	16	25	3	200.00
18	18	29	2	174.00
18	21	35	2	245.00
18	21	40	4	560.00
21	14	20	10	516.67
12	16	20	1	43.33
24	16	30	1	95.00
21	14	20	8	413.33
21	21	30	1	112.50
18	14	12	1	28.00
			Total	**5453.83**

Beam Formwork Takeoffs (PH Roof)				
Width (in)	**Depth (in)**	**Length (ft)**	**# of Beams**	**Formwork (SFCA)**
18	14	20	2	93.33
18	14	25	2	116.67
18	21	40	1	140.00
18	16	35	1	93.33
18	14	29	1	67.67
			Total	**511.00**

Beam Formwork Takeoffs	
Floor	**Formwork**
2nd	6289.50
3rd	6289.50
4th	6289.50
5th	6289.50
Roof	5453.83
PH Roof	511.00
Total	**31122.83**

Column Formwork Takeoffs (Lower Floors)				
Column Dimensions (in x in)		**Height (ft)**	**# of Cols.**	**Formwork (SFCA)**
18	21	13.5	26	2281.50
18	24	13.5	17	1606.50
18	26	13.5	3	297.00
			Total	**4185.00**

Column Formwork Takeoffs (Above Splice)				
Column Dimensions (in x in)		**Height (ft)**	**# of Cols.**	**Formwork (SFCA)**
18	18	13.5	26	2106.00
18	20	13.5	17	1453.50
18	21	13.5	3	263.25
			Total	**3822.75**

Concrete Column Takeoffs	
Below Floor	**Volume**
2nd	4185.00
3rd	4185.00
4th	4185.00
5th	3822.75
Roof	3822.75
PH Roof	764.55
Total	**20200.50**

Concrete Slab Takeoffs			
Floor	**Thickness (in)**	**Area (sq. ft.)**	**Volume (CY)**
2nd	9	24116	669.89
3rd	9	24116	669.89
4th	9	24116	669.89
5th	9	24116	669.89
Roof	9	23615	655.97
PH Roof	9	2975	82.64
		Total	**3418.17**

Column Reinf. Takeoffs (Tons)		
Below Floor	**# 7 & Below**	**#8 & Above**
2nd	0.76	14.55
3rd	0.76	14.55
4th	0.76	14.55
5th	0.76	14.55
Roof	0.76	14.55
PH Roof	0.15	2.91
Total	**3.93**	**75.65**

Beam Reinforcing Takeoffs (Tons)		
Floor	**# 7 & Below**	**#8 & Above**
2nd	2.94	7.94
3rd	2.94	7.94
4th	2.94	7.94
5th	2.94	7.94
Roof	2.94	7.94
PH Roof	0.59	1.59
Total	**15.31**	**41.28**

Appendix B: spSlab Models and Reinforcing Diagrams

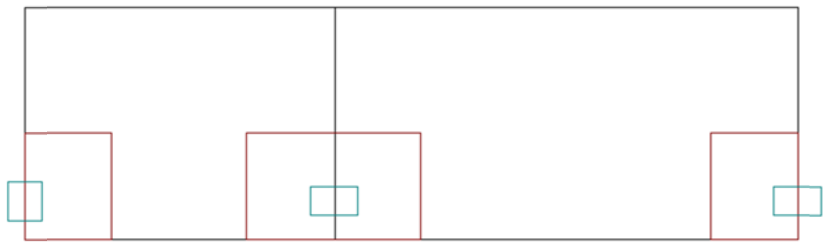


Figure 35: spSlab Model Column Line B: Top View

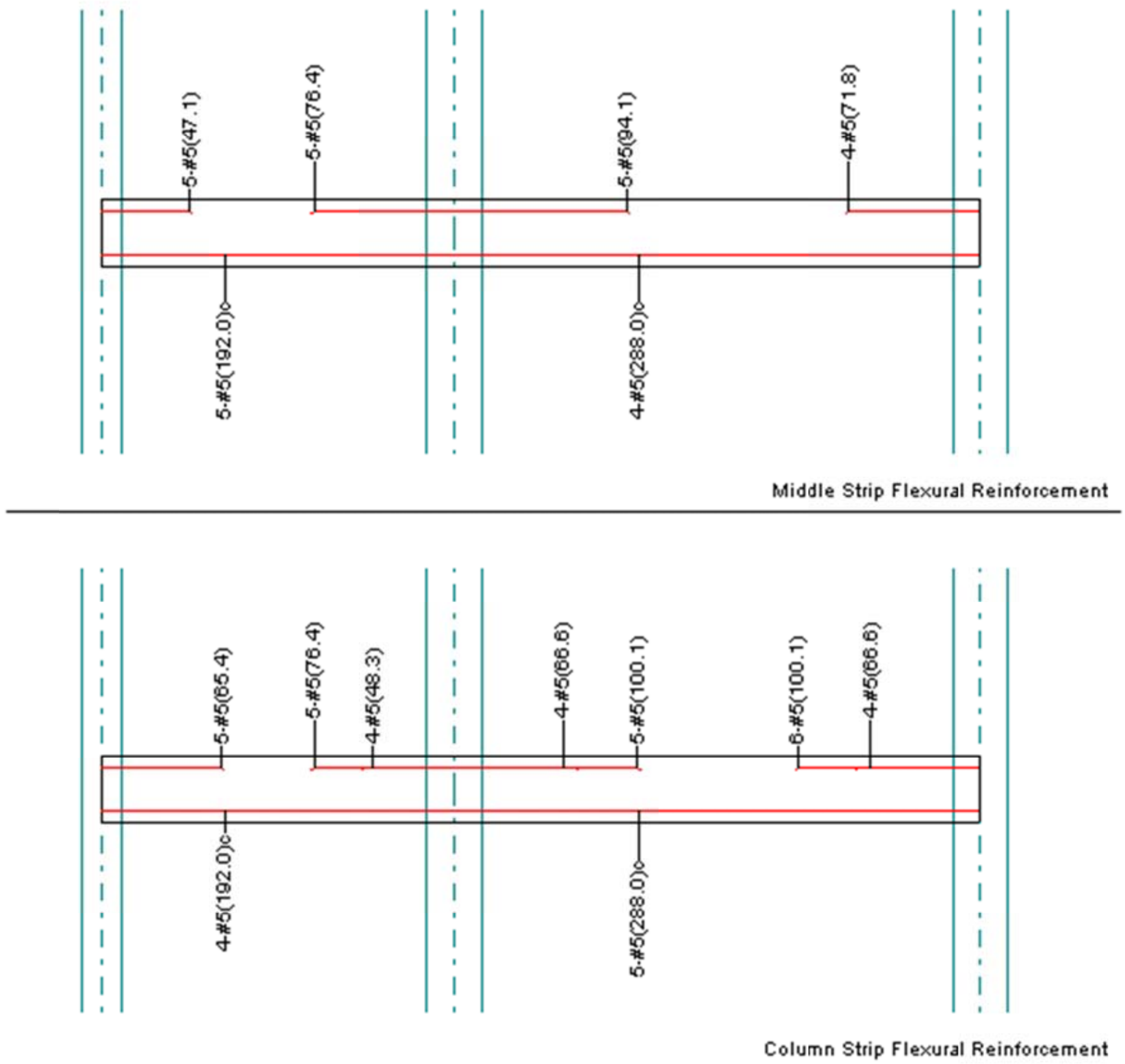


Figure 36: spSlab Column Line B Reinforcing Diagram

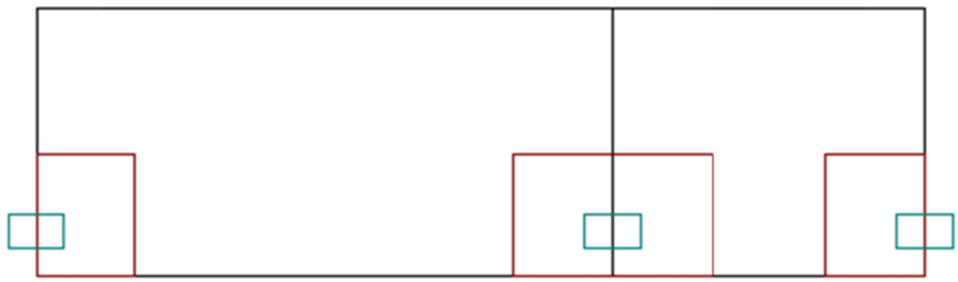


Figure 37: spSlab Model Column Line B.1: Top View

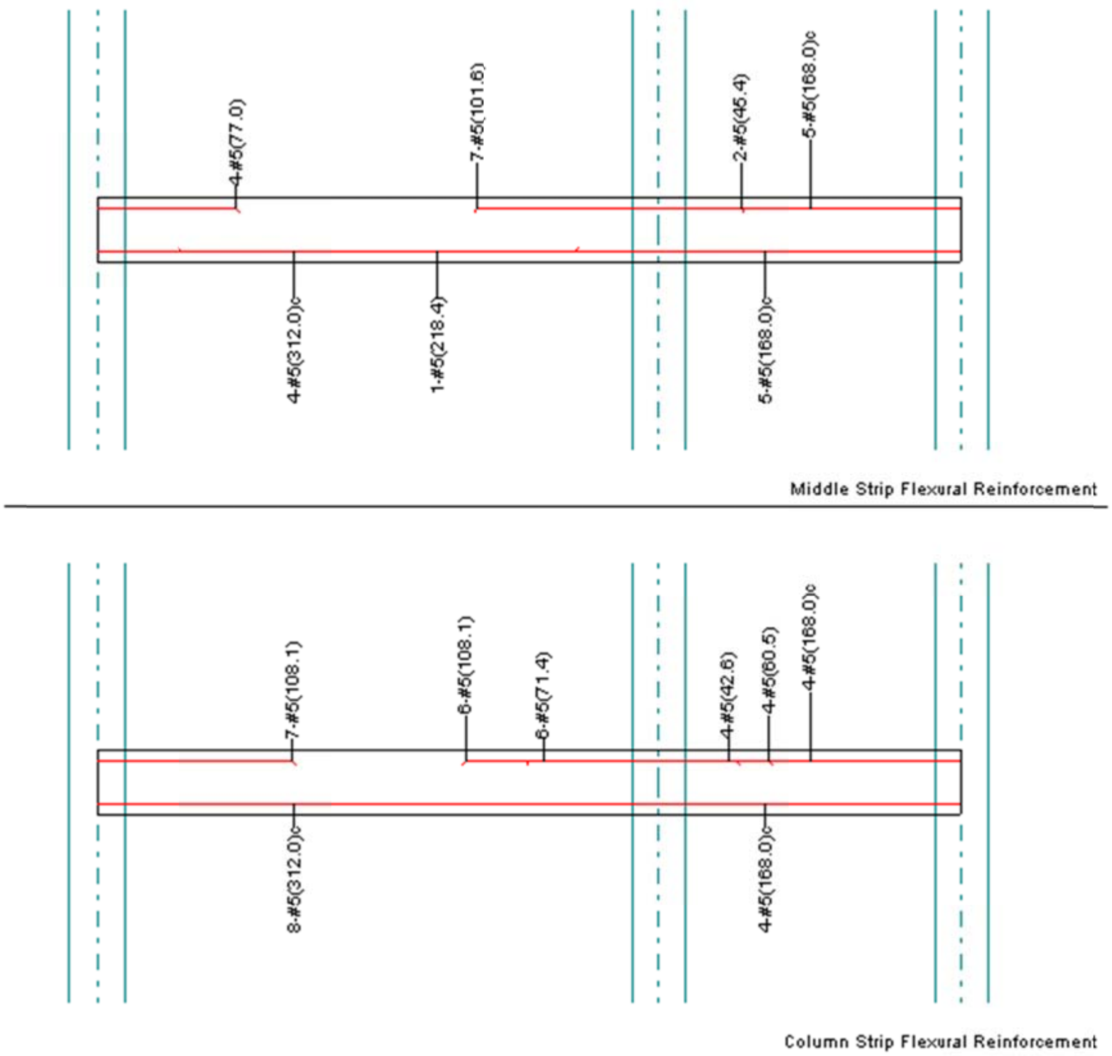


Figure 38: spSlab Column Line B.1 Reinforcing Diagram

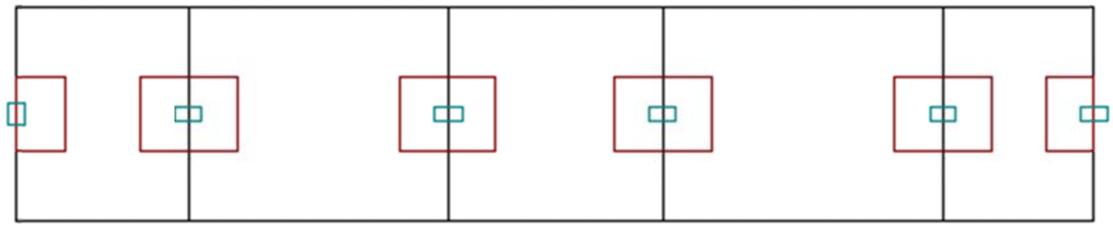


Figure 39: spSlab Model Column Line D: Top View

Figure 40: spSlab Column Line D Reinforcing Diagram

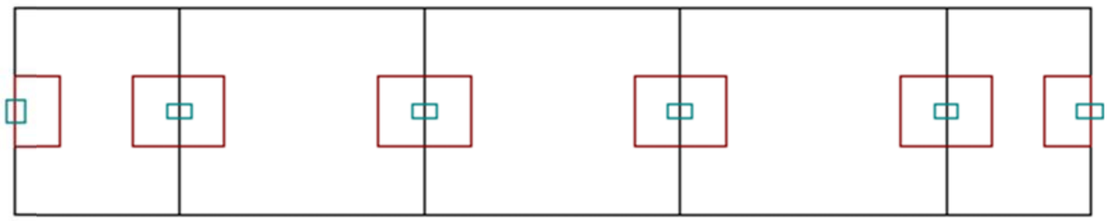


Figure 41: spSlab Model Column Line E: Top View

Figure 42: spSlab Column Line E Reinforcing Diagram

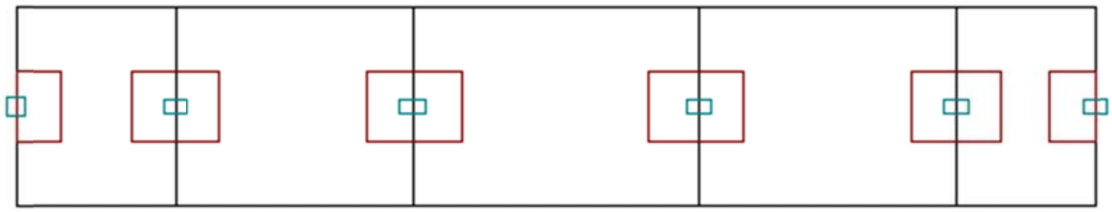


Figure 43: spSlab Model Column Line F: Top View

Figure 44: spSlab Column Line F Reinforcing Diagram

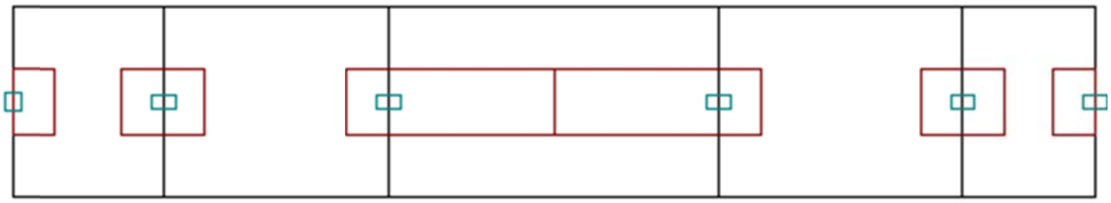


Figure 45: spSlab Model Column Line G: Top View

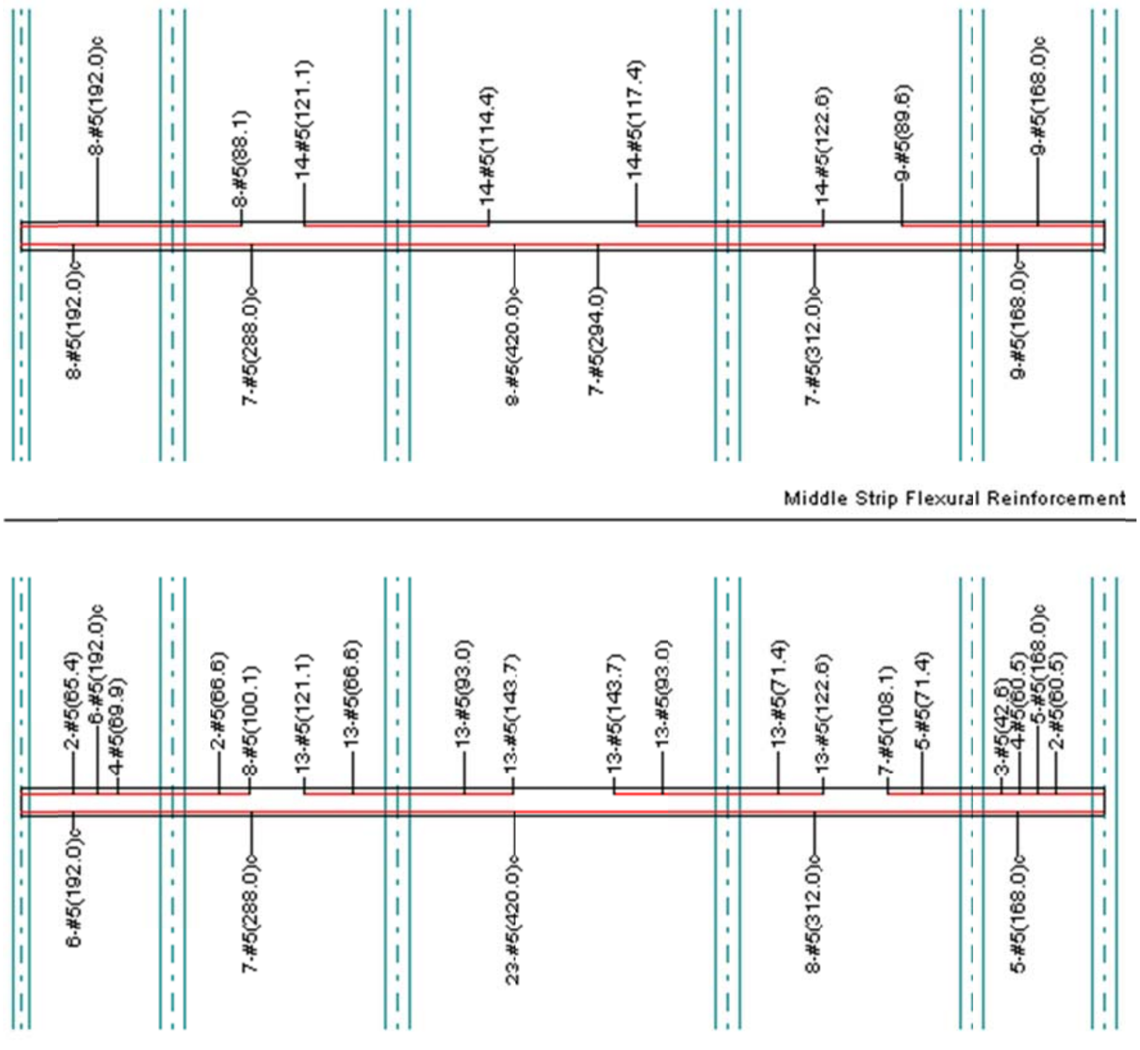


Figure 46: spSlab Column Line G Reinforcing Diagram

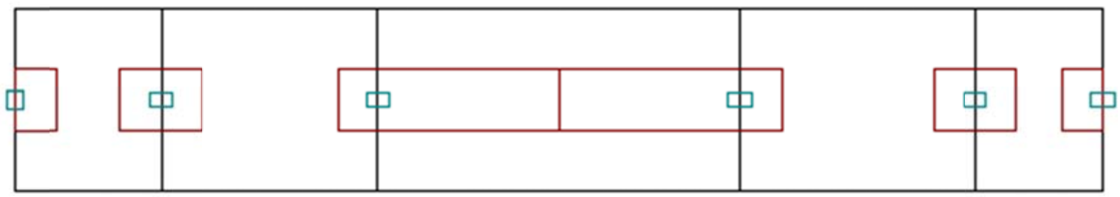


Figure 47: spSlab Model Column Line H: Top View

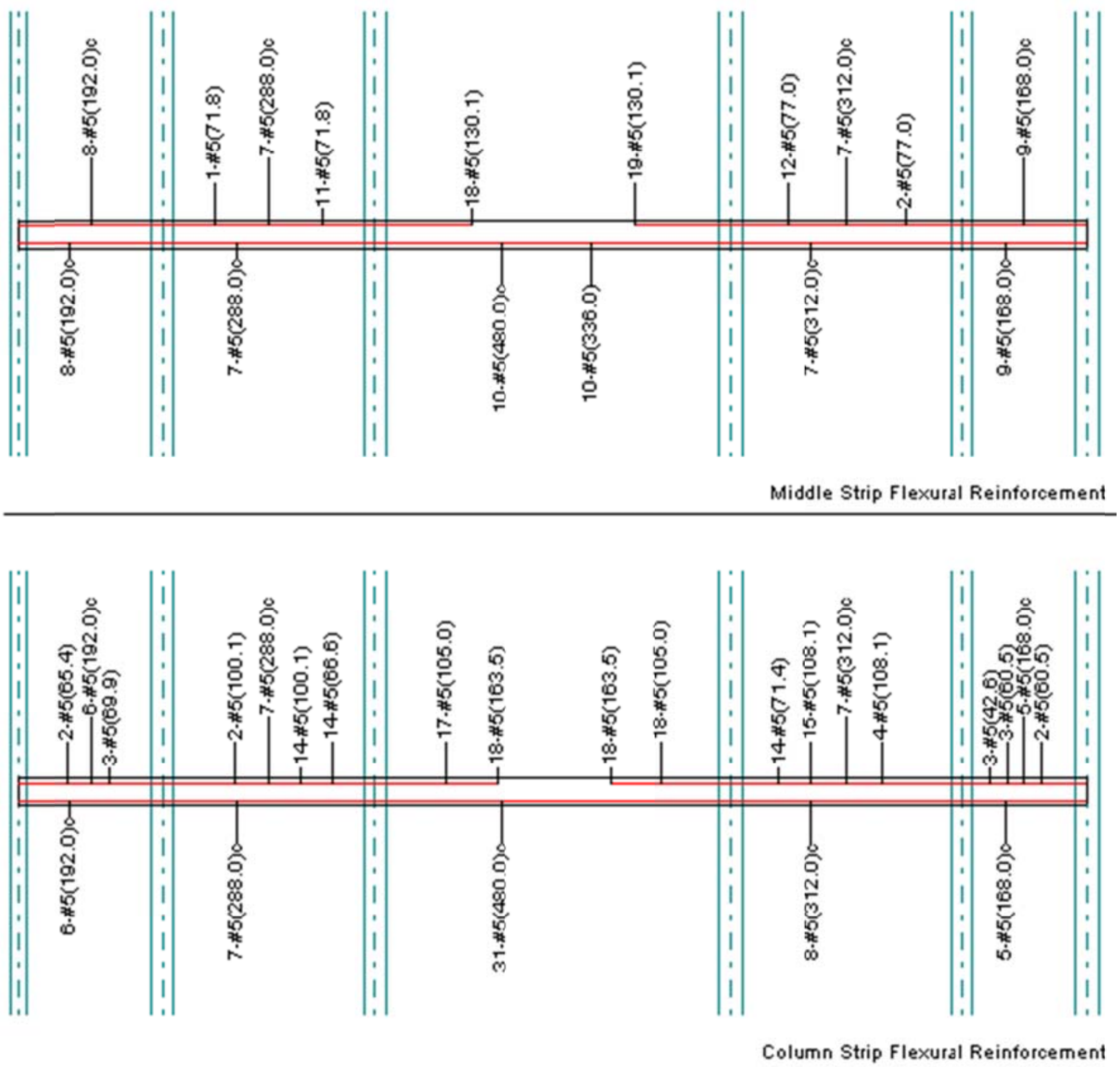


Figure 48: spSlab Column Line H Reinforcing Diagram

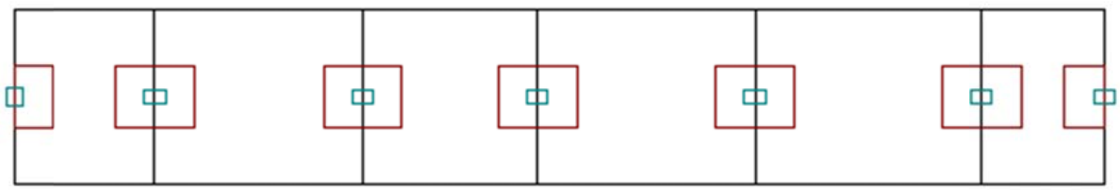


Figure 49: spSlab Model Column Line J: Top View

Figure 50: spSlab Column Line J Reinforcing Diagram

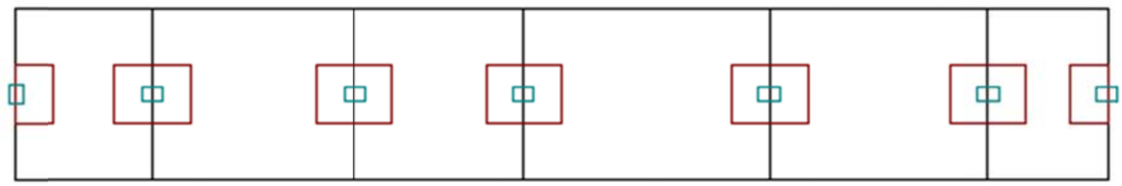


Figure 51: spSlab Model Column Line K: Top View

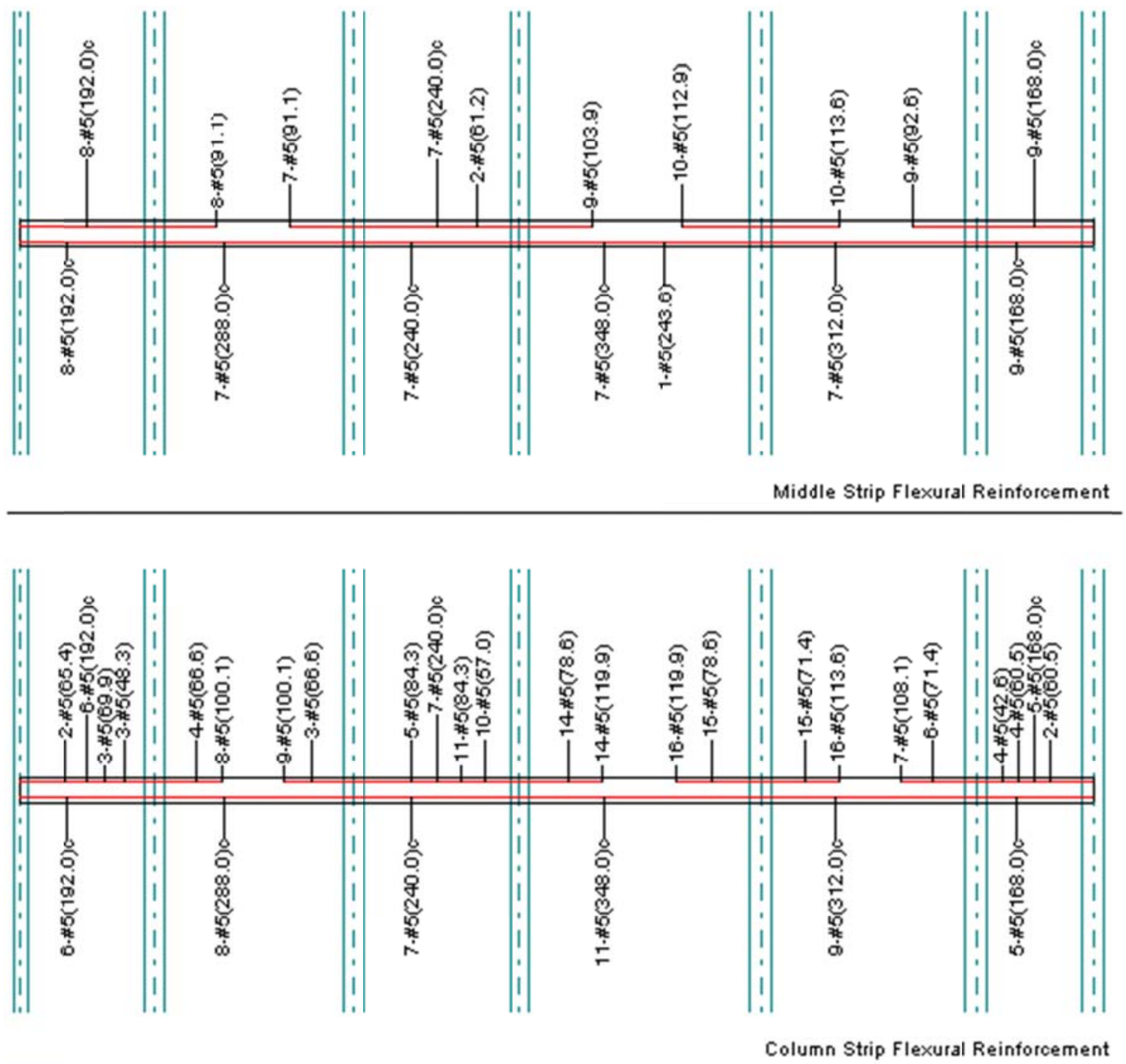


Figure 52: spSlab Column Line K Reinforcing Diagram

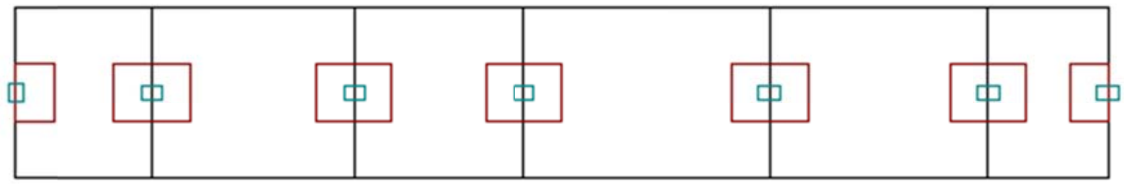


Figure 53: spSlab Model Column Line L: Top View

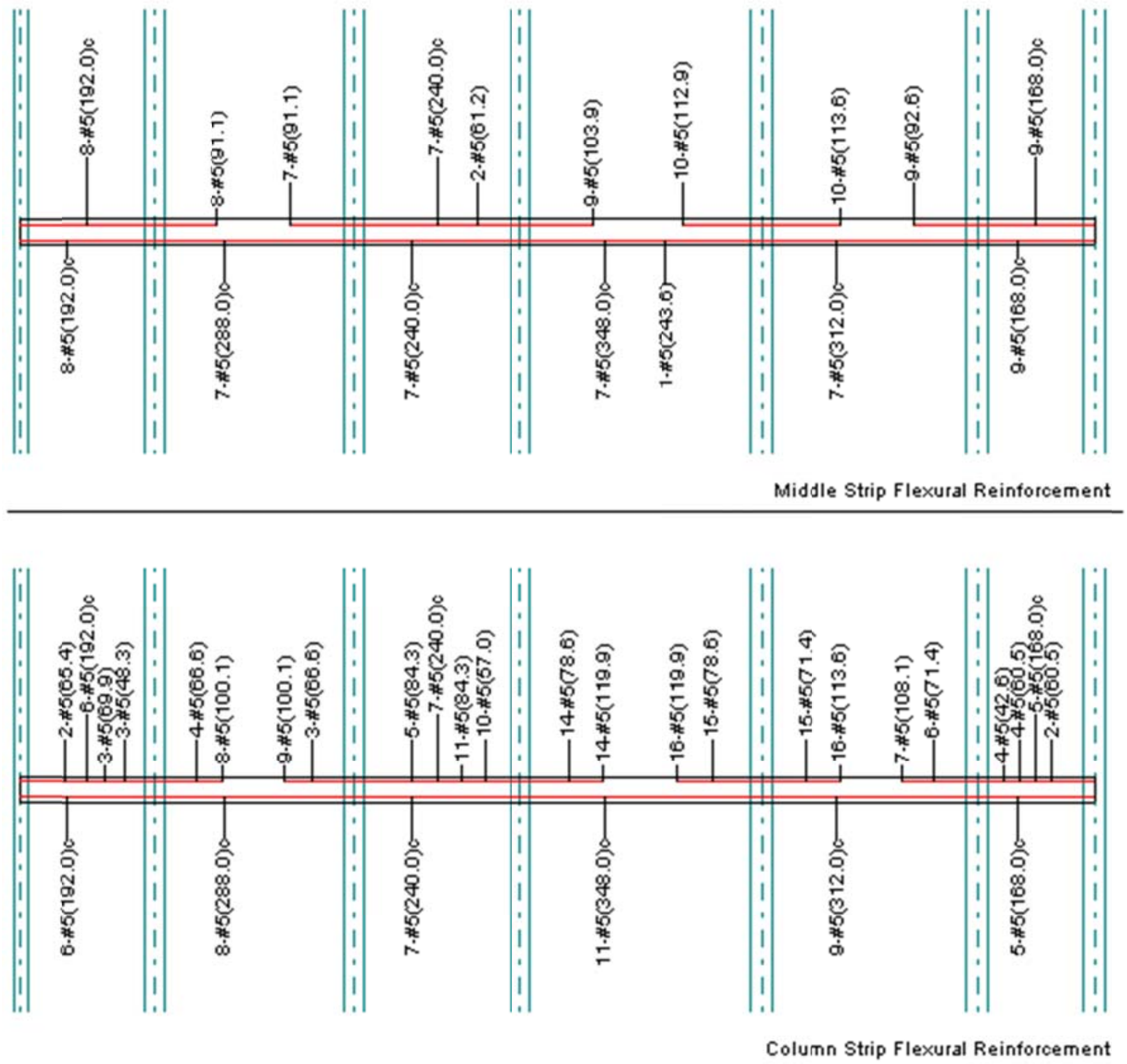


Figure 54: spSlab Column Line L Reinforcing Diagram

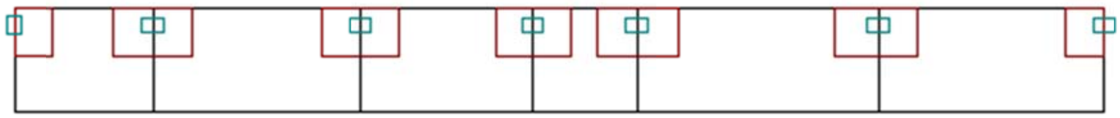


Figure 55: spSlab Model Column Line M: Top View

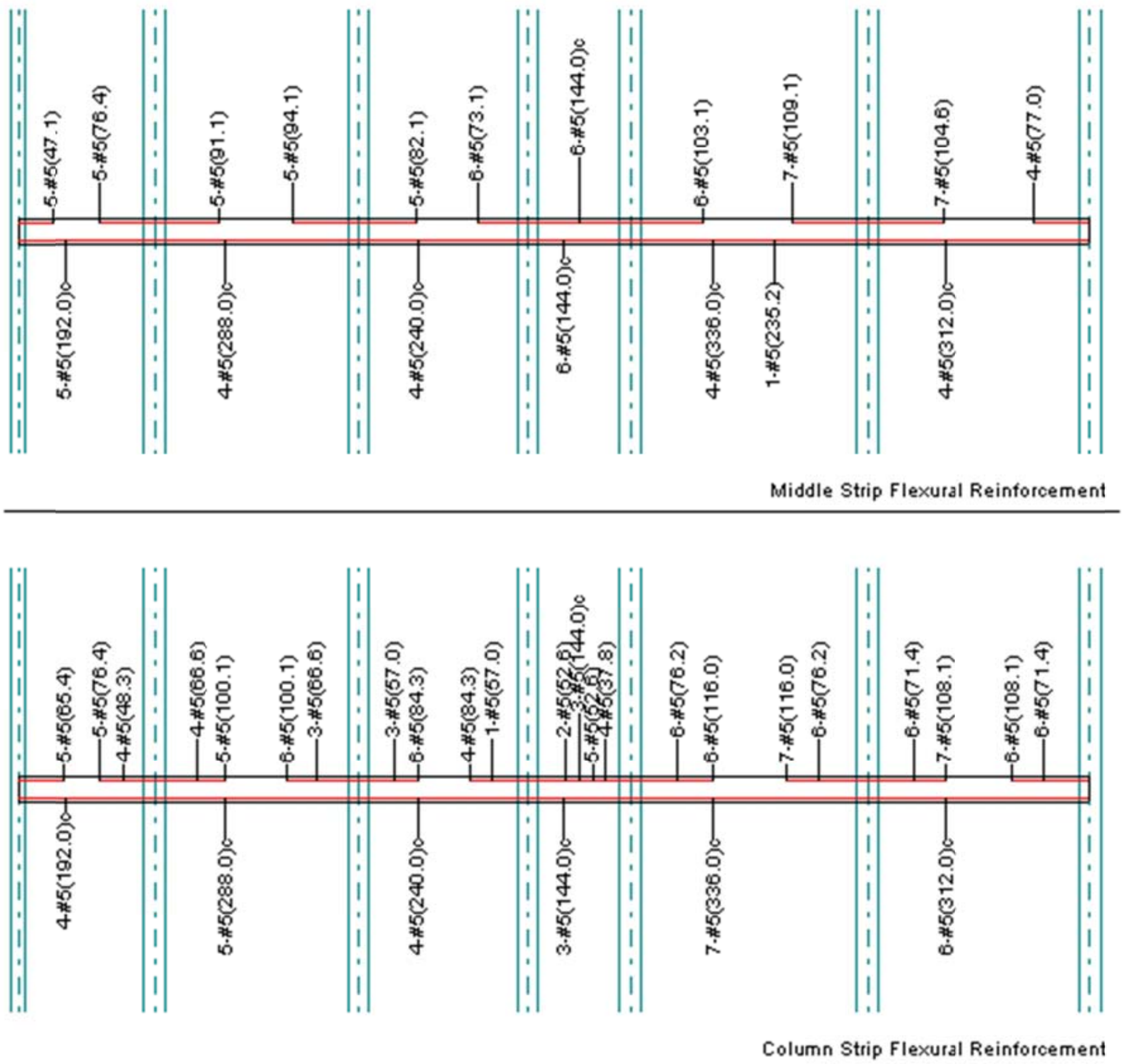


Figure 56: spSlab Column Line M Reinforcing Diagram

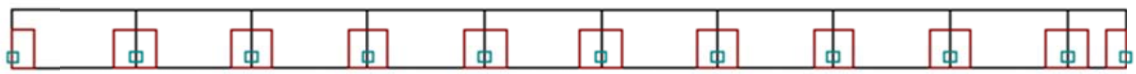


Figure 57: spSlab Model Column Line 1: Top View

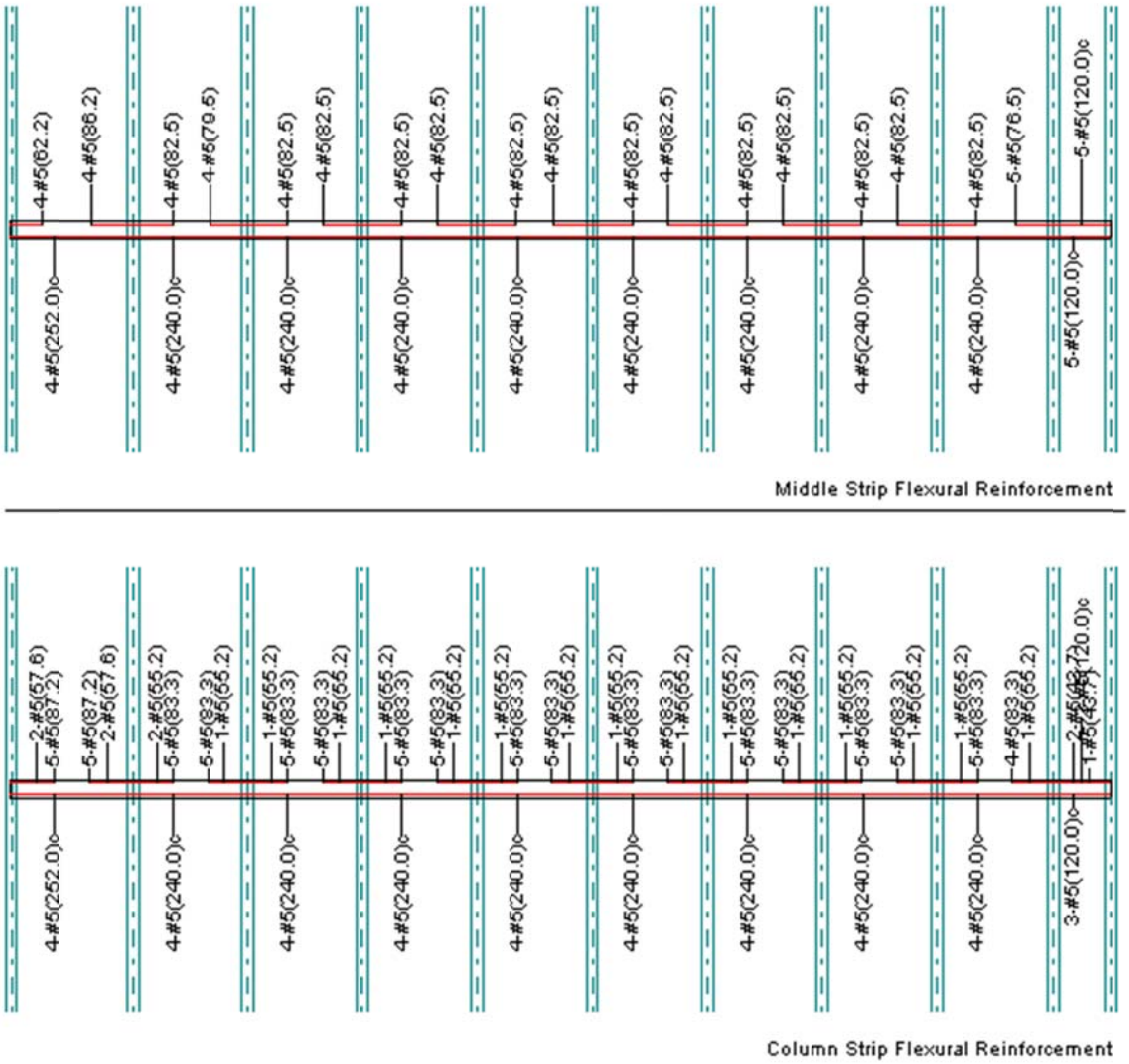


Figure 58: spSlab Column Line 1 Reinforcing Diagram

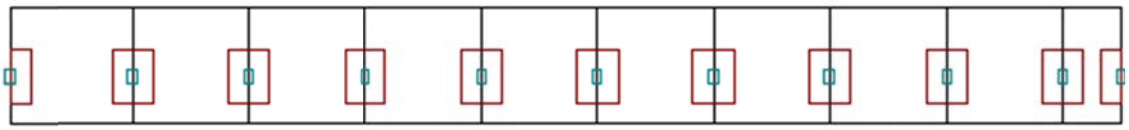


Figure 59: spSlab Model Column Line 2: Top View

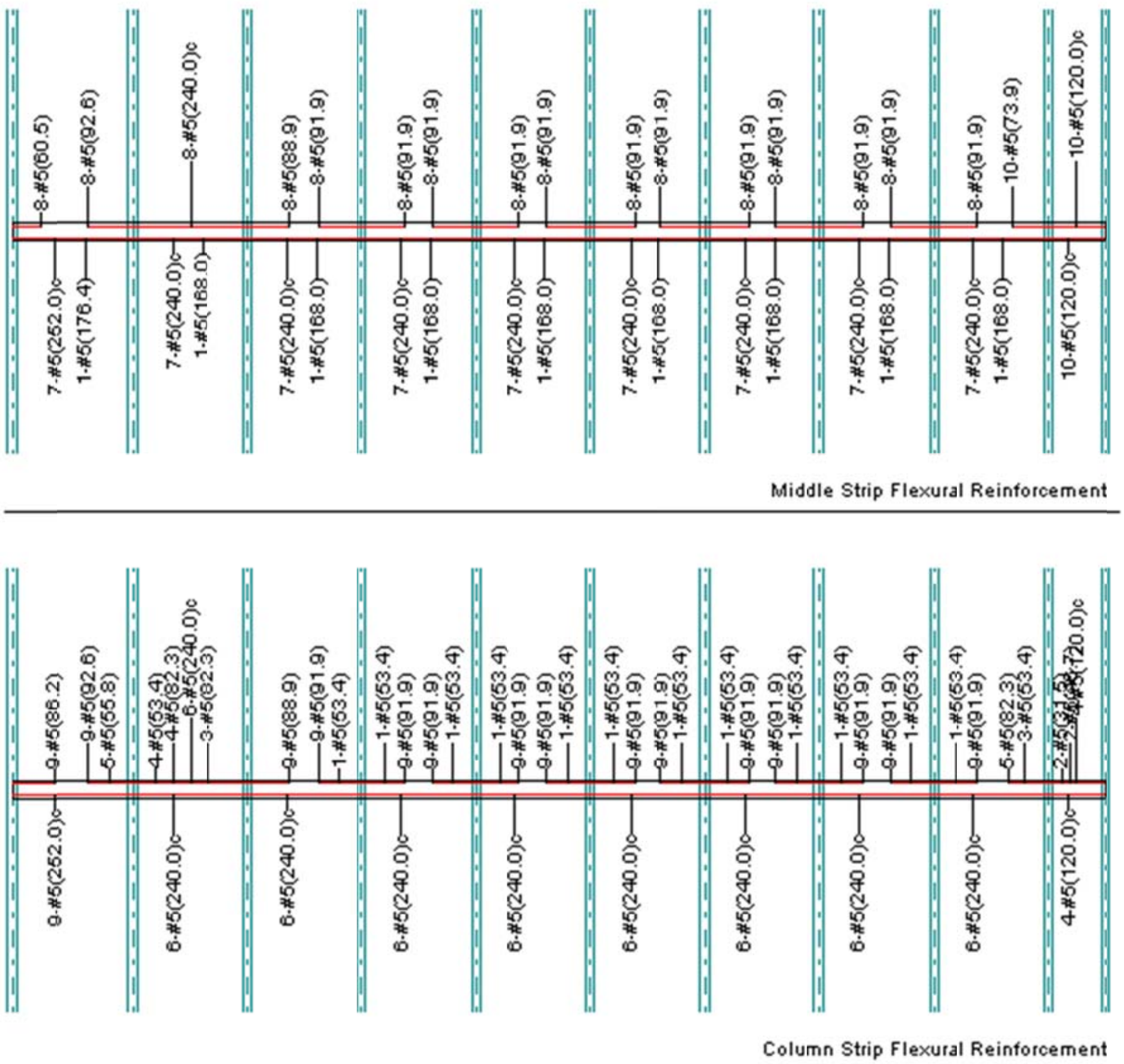


Figure 60: spSlab Column Line 2 Reinforcing Diagram

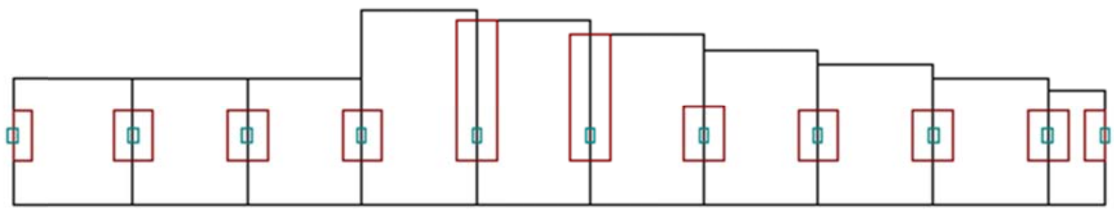


Figure 61: spSlab Model Column Line 3: Top View

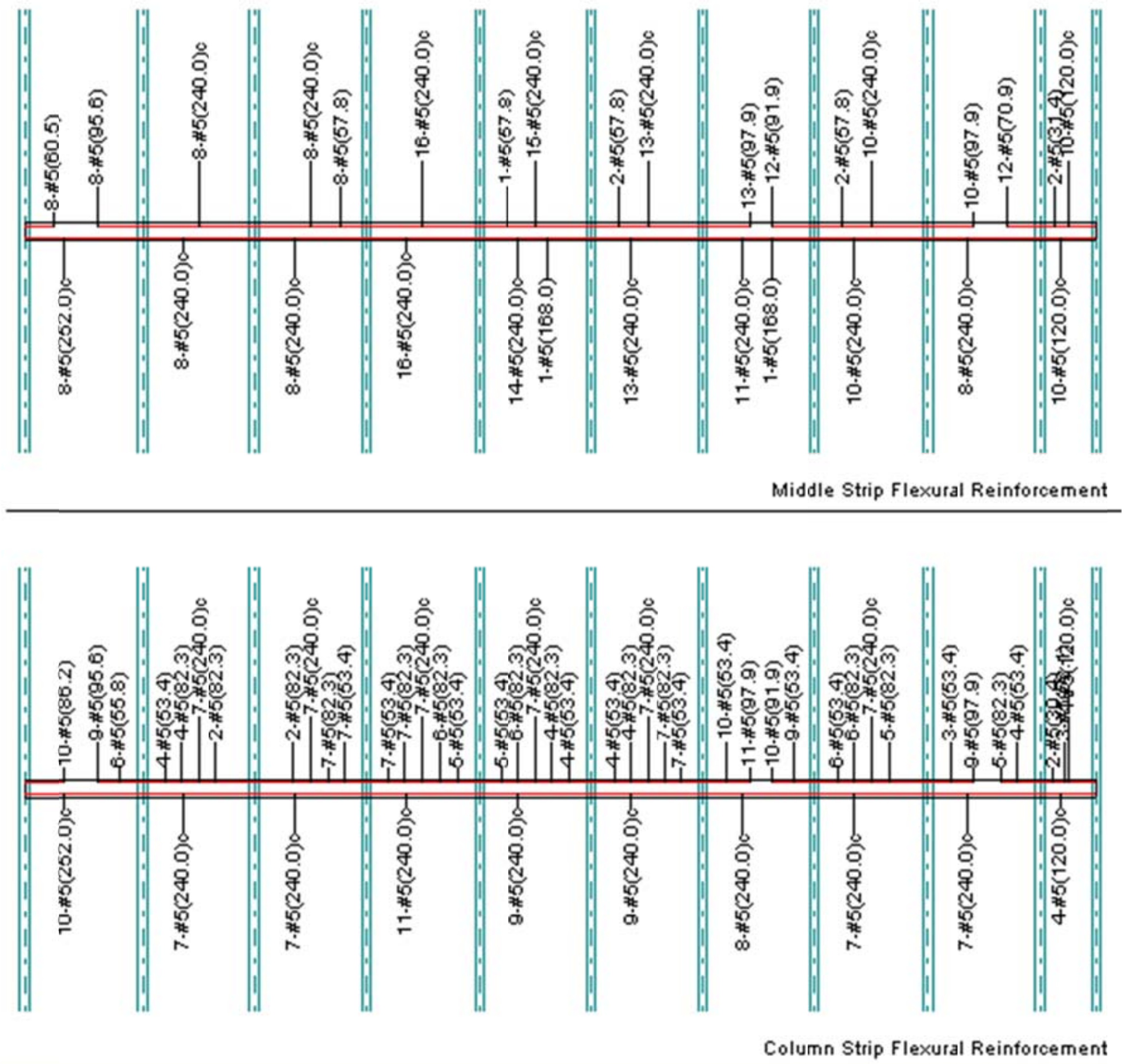


Figure 62: spSlab Column Line 3 Reinforcing Diagram

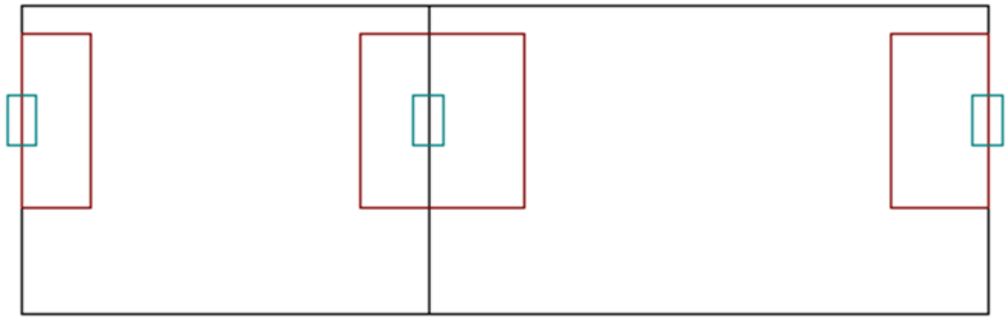


Figure 63: spSlab Model Column Line 4: Top View

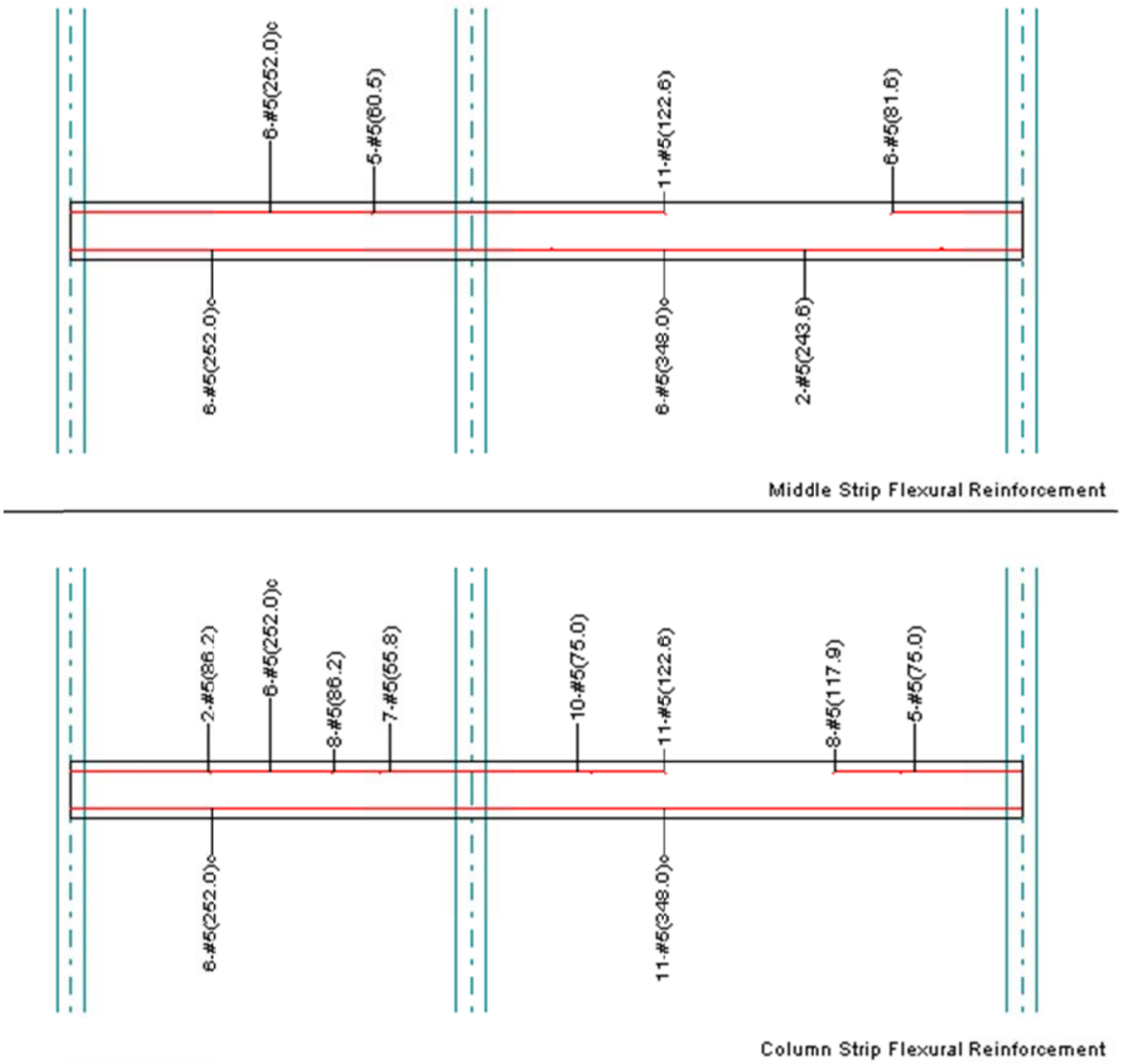


Figure 64: spSlab Column Line 4 Reinforcing Diagram

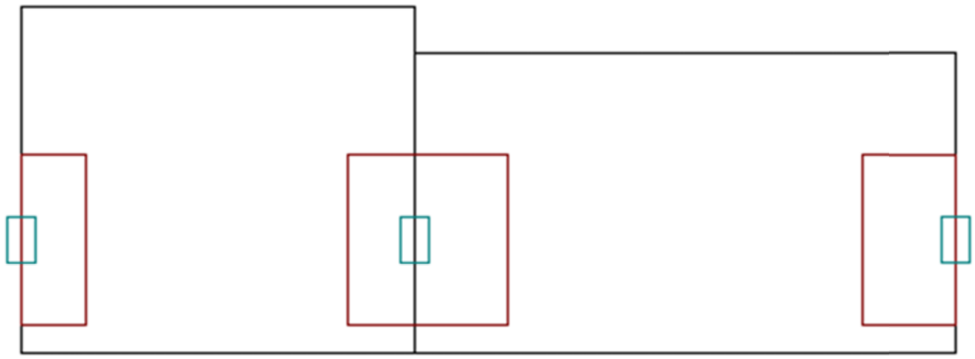


Figure 65: spSlab Model Column Line 4.5: Top View

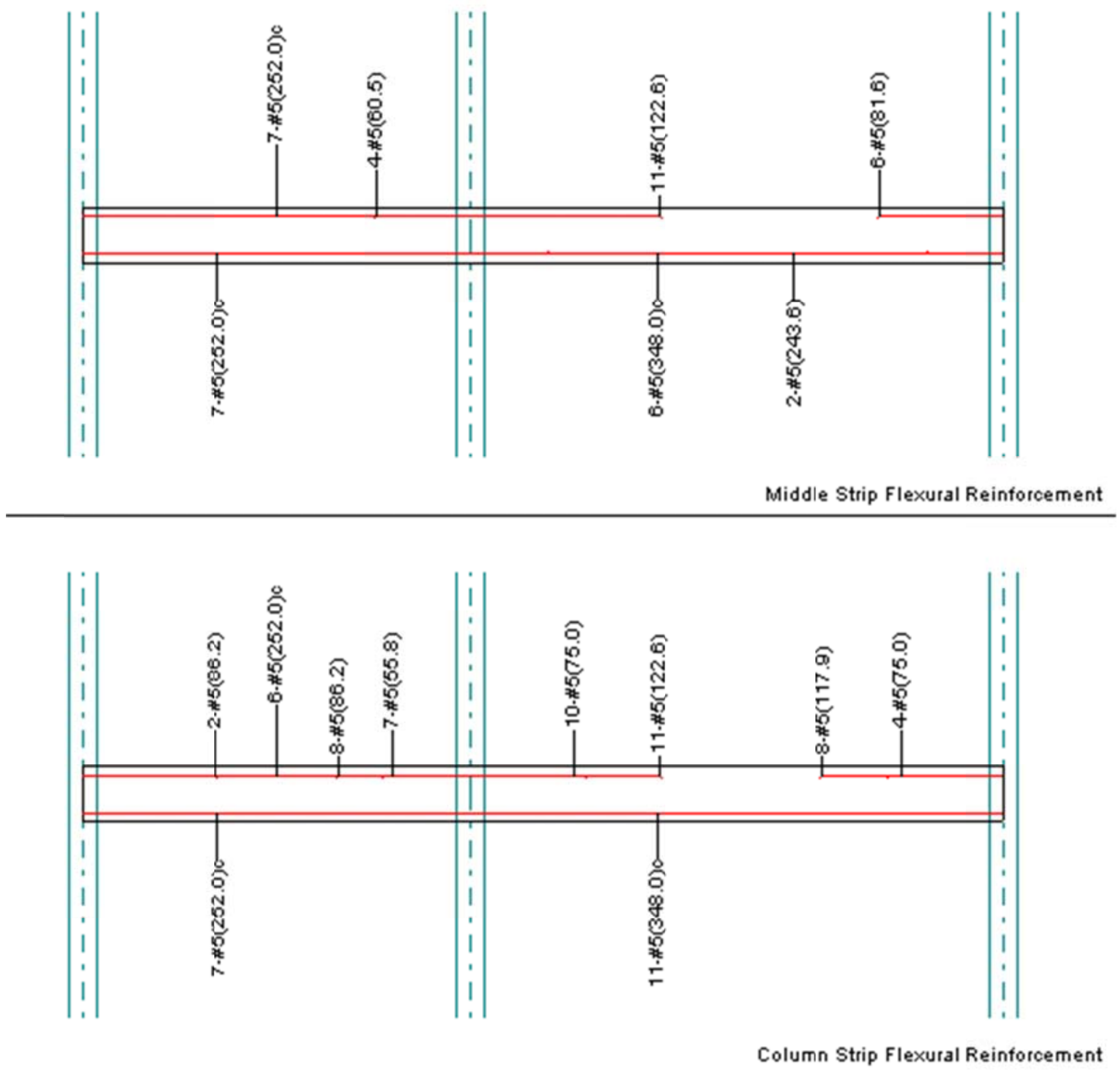


Figure 66: spSlab Column Line 4.5 Reinforcing Diagram

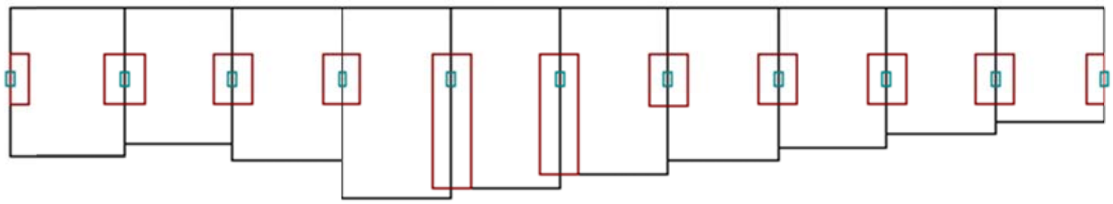


Figure 67: spSlab Model Column Line 7: Top View

Figure 68: spSlab Column Line 7 Reinforcing Diagram

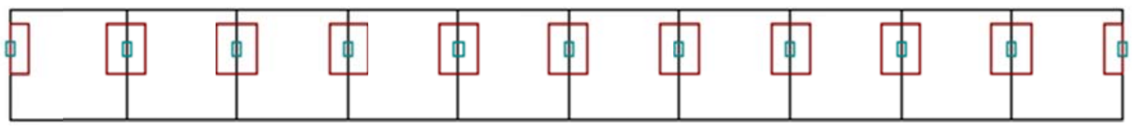


Figure 69: spSlab Model Column Line 8: Top View

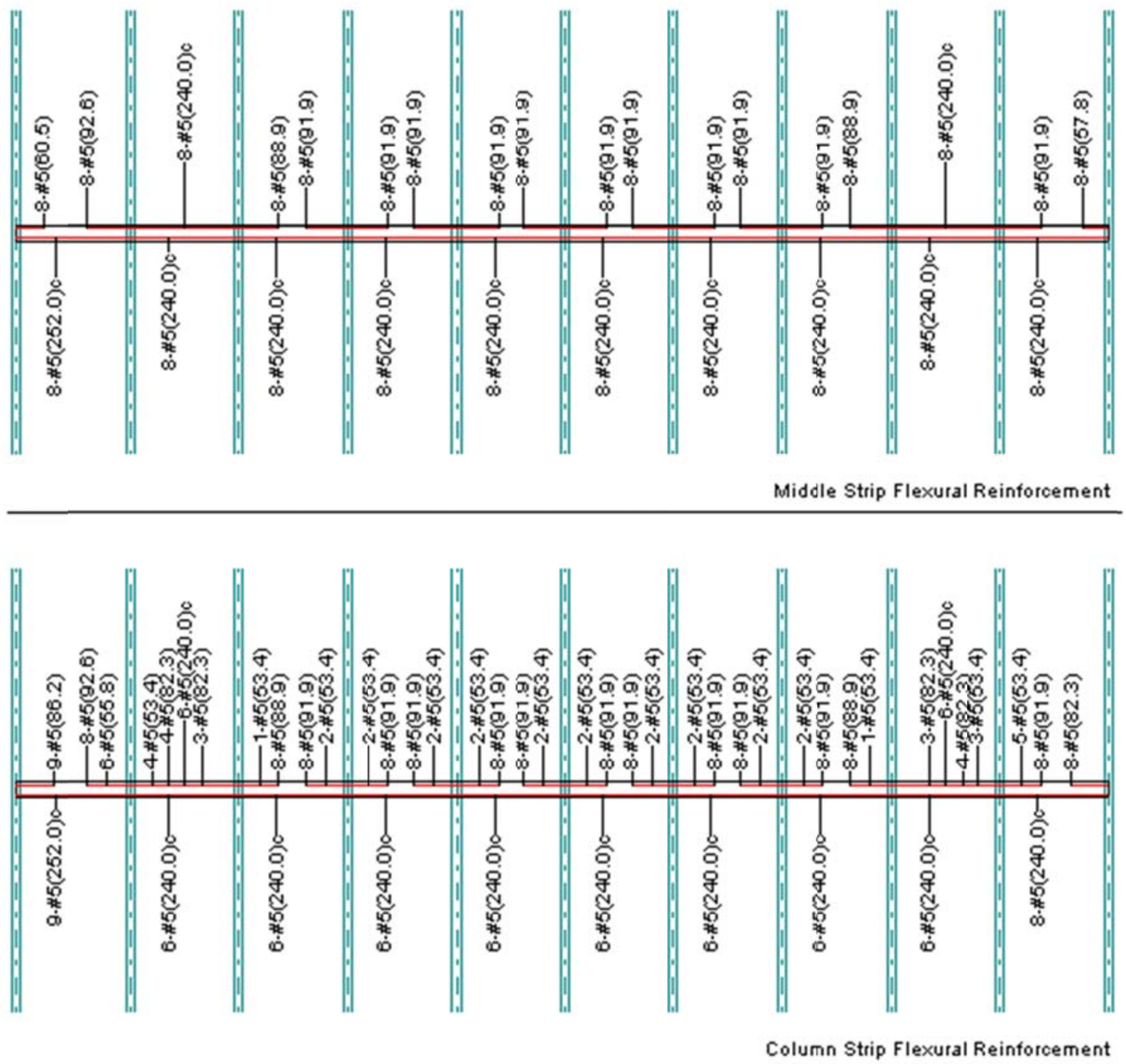


Figure 70: spSlab Column Line 8 Reinforcing Diagram

Figure 71: spSlab Model Column Line 9: Top View

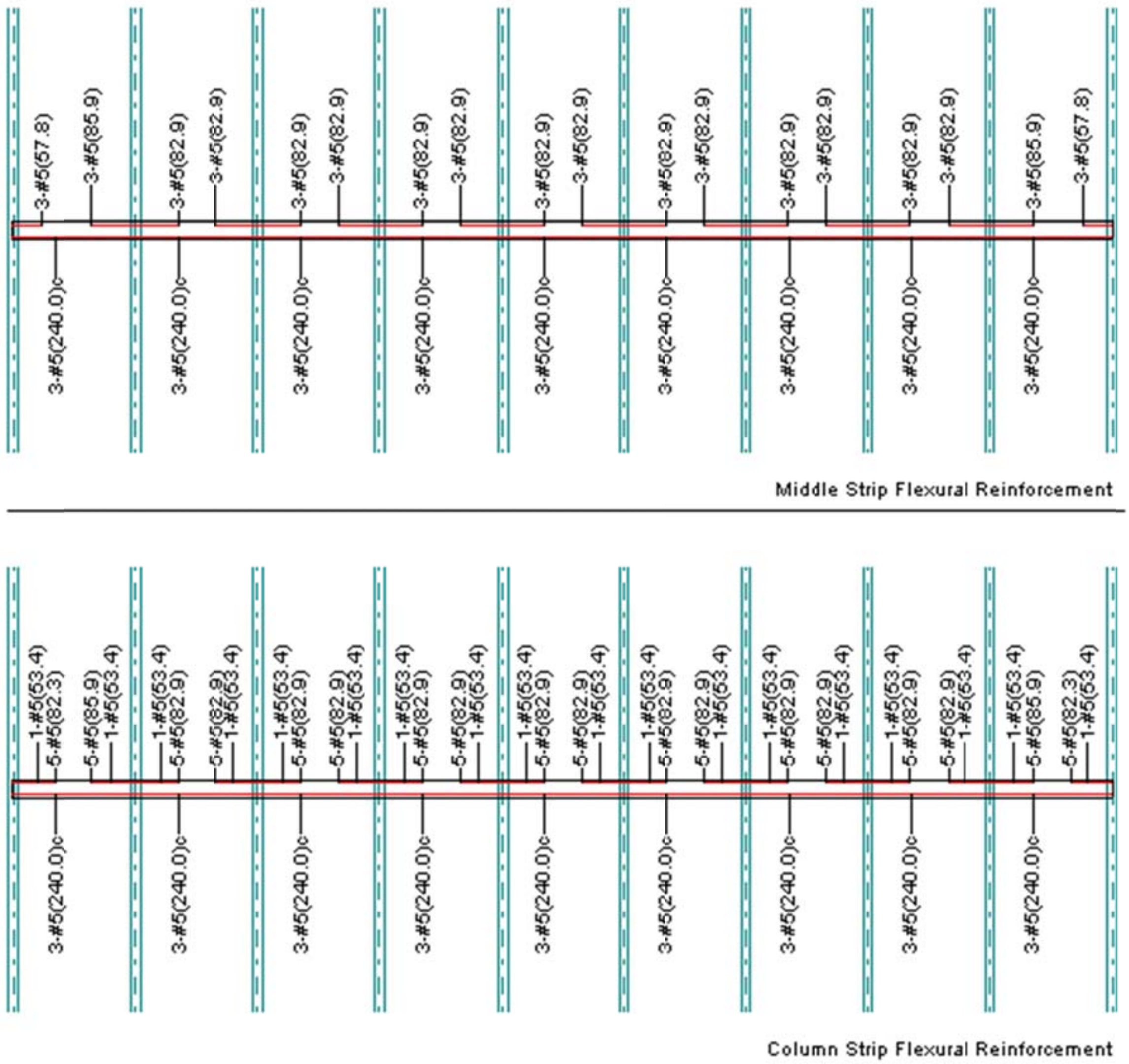


Figure 72: spSlab Column Line 9 Reinforcing Diagram

Appendix C: spBeam Models and Gravity Reinforcement Diagrams

Figure 73: spBeam Model Column Line B: Top View

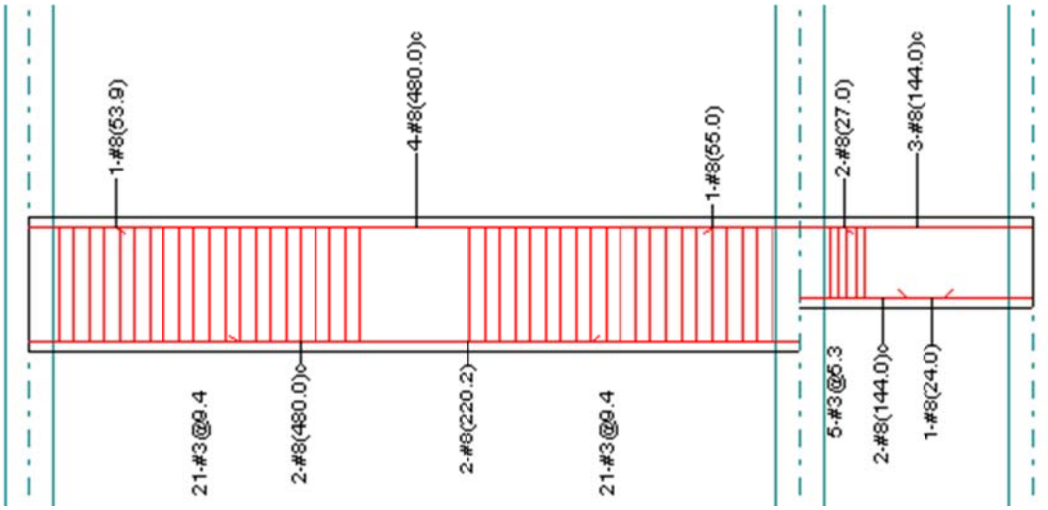


Figure 74: spBeam Column Line B Reinforcing Diagram

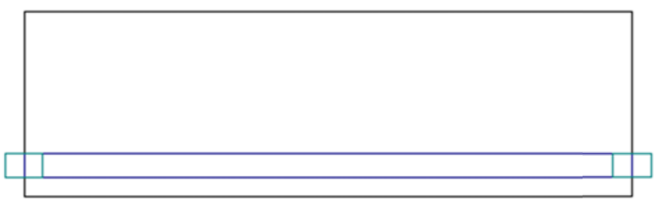


Figure 75: spBeam Model Column Line B.1: Top View

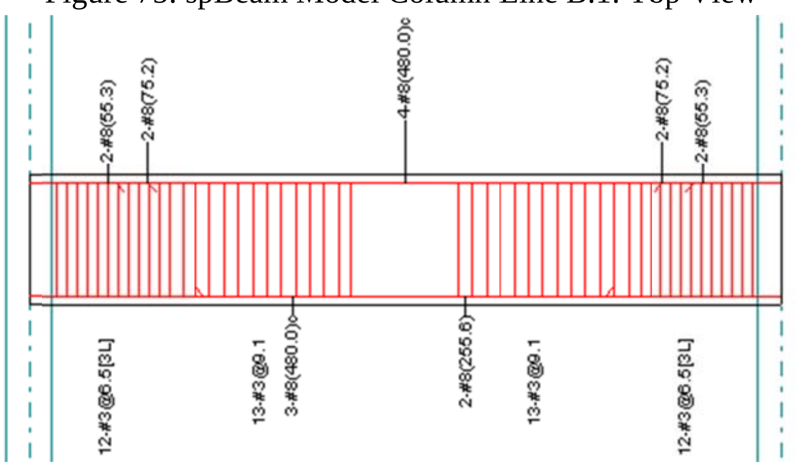


Figure 76: spBeam Column Line B.1 Reinforcing Diagram

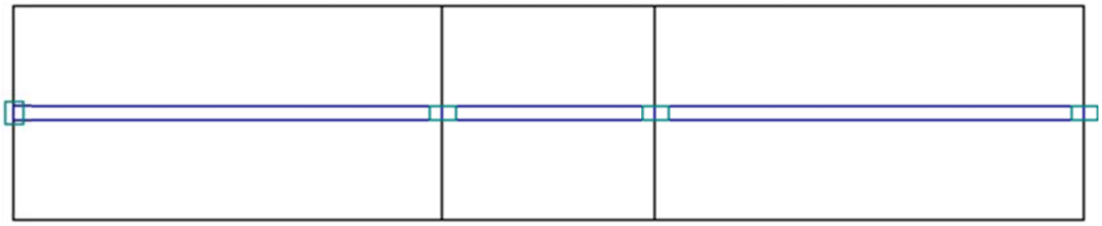


Figure 77: spBeam Model Column Line D: Top View

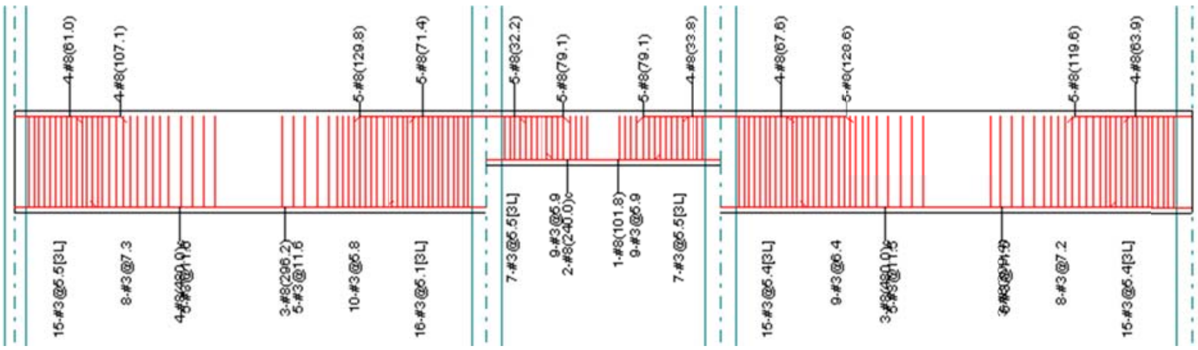


Figure 78: spBeam Column Line D Reinforcing Diagram

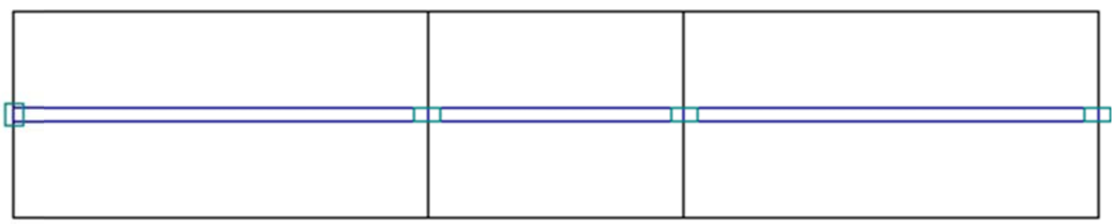


Figure 79: spBeam Model Column Line E: Top View

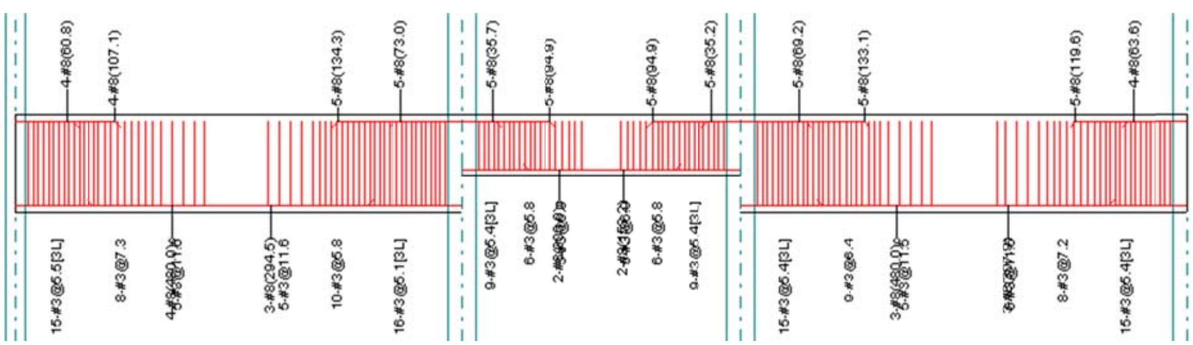


Figure 80: spBeam Column Line E Reinforcing Diagram

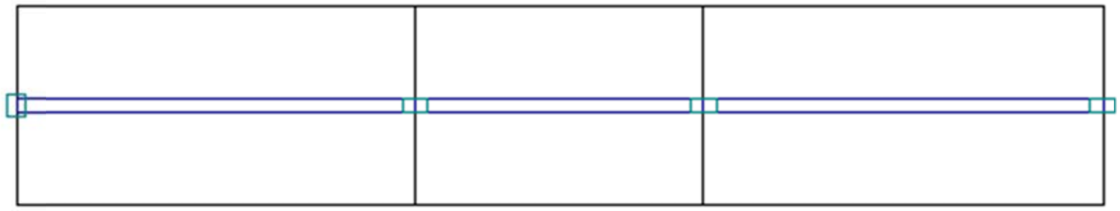


Figure 81: spBeam Model Column Line F: Top View

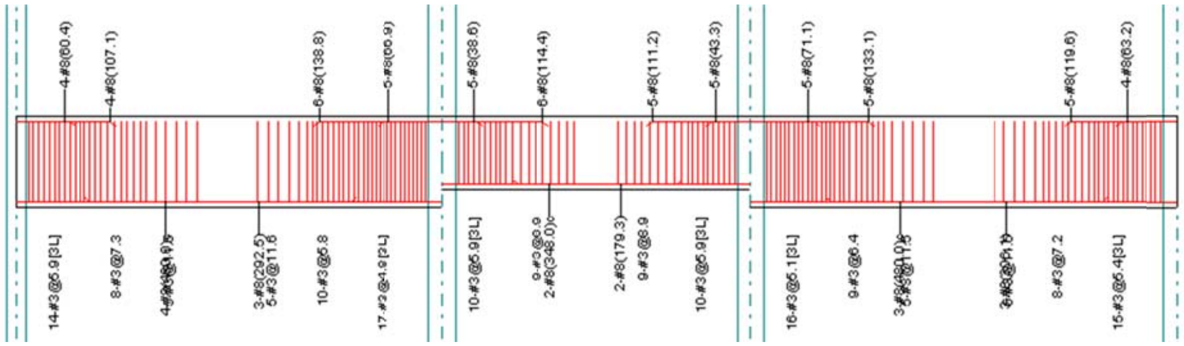


Figure 82: spBeam Column Line F Reinforcing Diagram

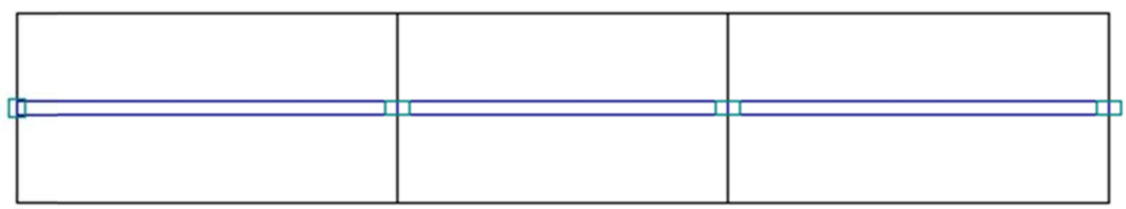


Figure 83: spBeam Model Column Line G: Top View

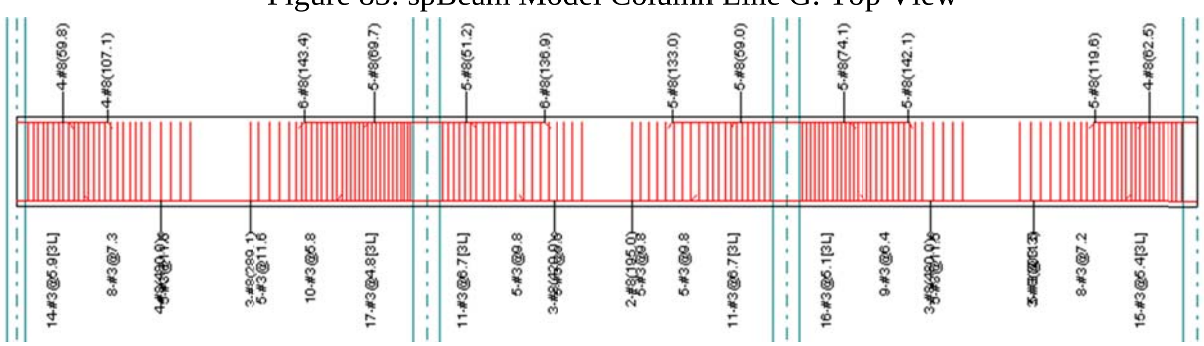


Figure 84: spBeam Column Line G Reinforcing Diagram

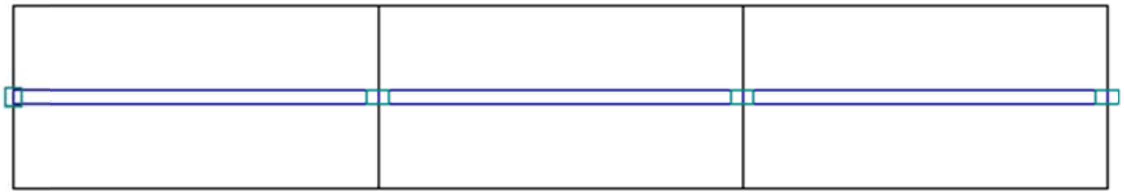


Figure 85: spBeam Model Column Line H: Top View

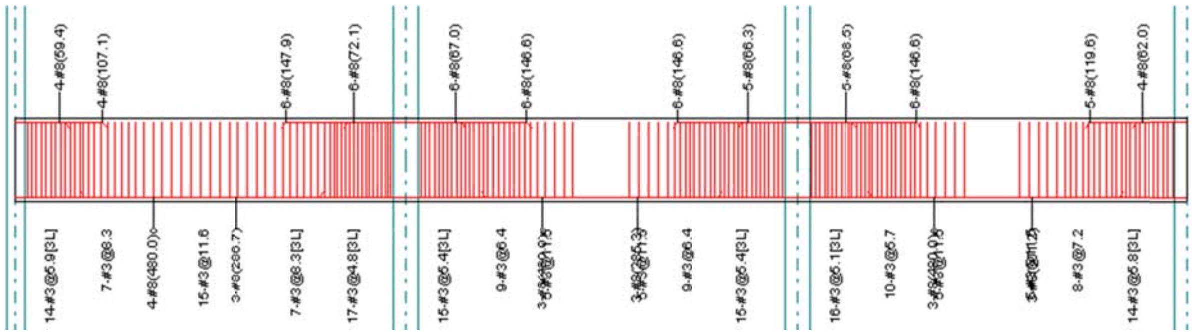


Figure 86: spBeam Column Line H Reinforcing Diagram

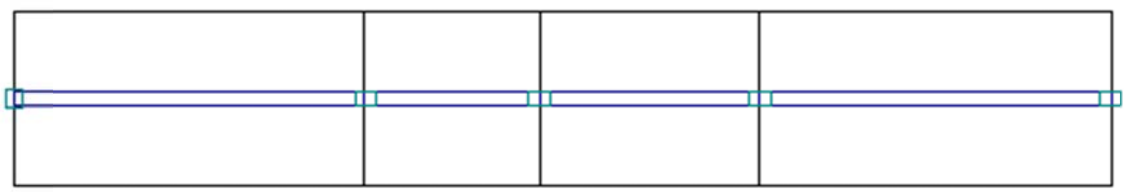


Figure 87: spBeam Model Column Line J: Top View

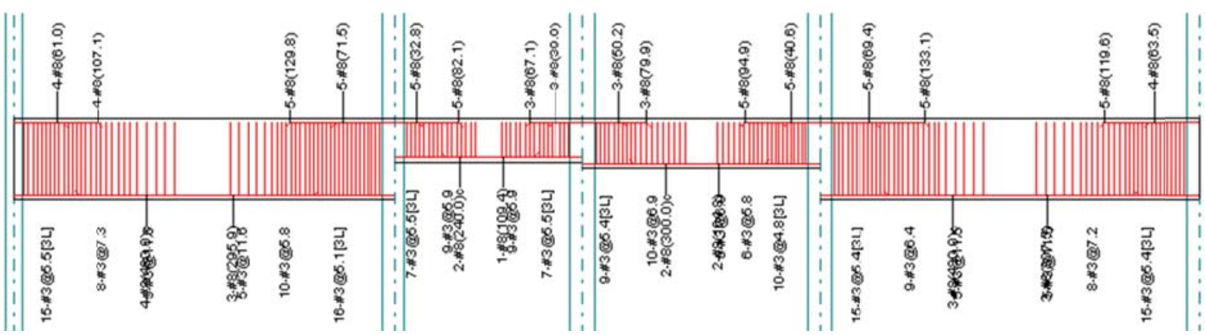


Figure 88: spBeam Column Line J Reinforcing Diagram

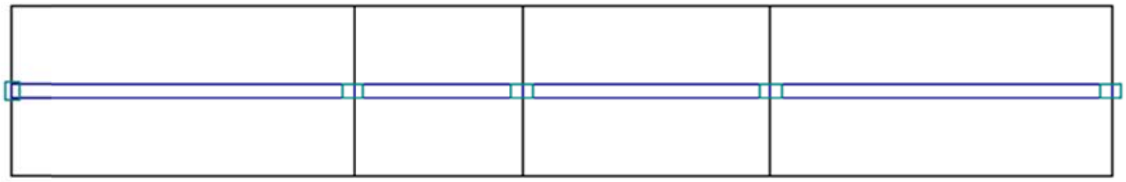


Figure 89: spBeam Model Column Line K: Top View

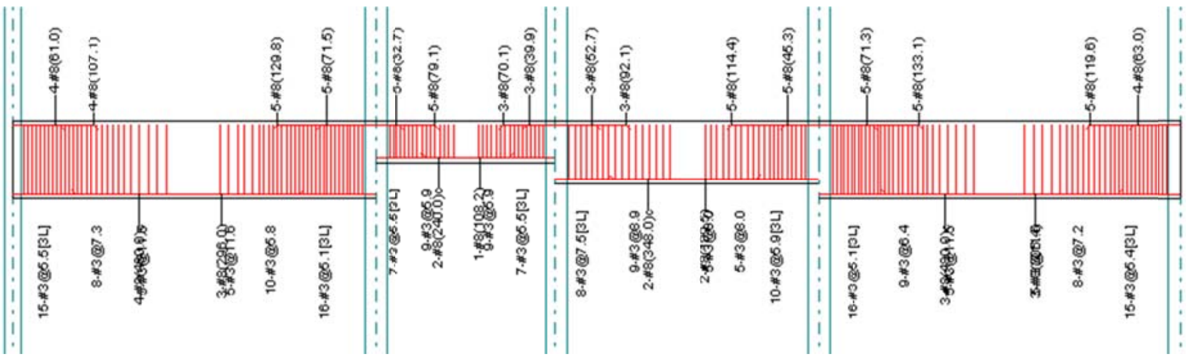


Figure 90: spBeam Column Line K Reinforcing Diagram

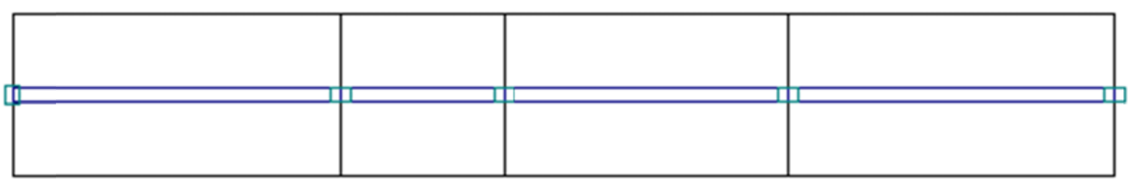


Figure 91: spBeam Model Column Line L: Top View

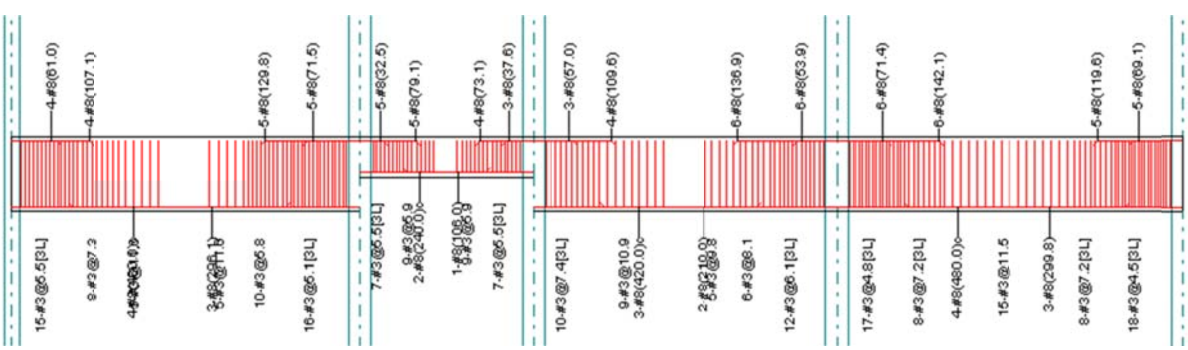


Figure 92: spBeam Column Line L Reinforcing Diagram

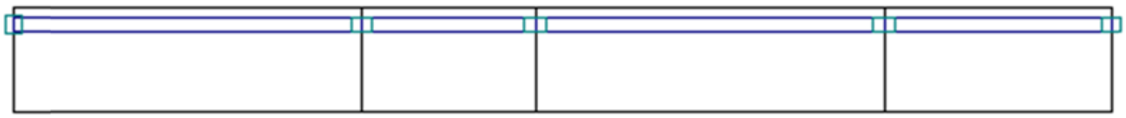


Figure 93: spBeam Model Column Line M: Top View

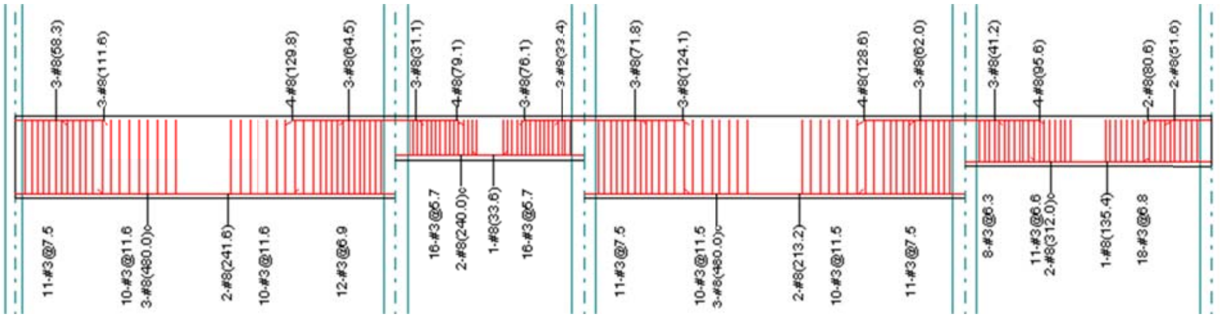


Figure 94: spBeam Column Line M Reinforcing Diagram

Figure 95: spBeam Model Column Line 1: Top View

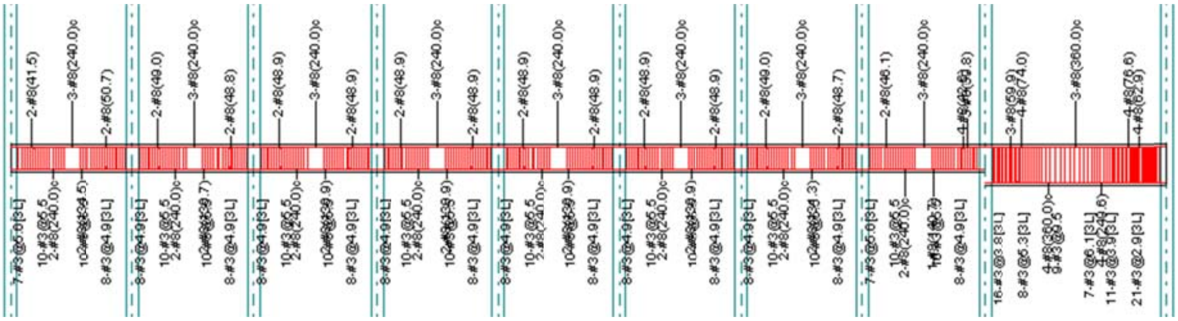


Figure 96: spBeam Column Line 1 Reinforcing Diagram

Figure 97: spBeam Model Column Line 8: Top View

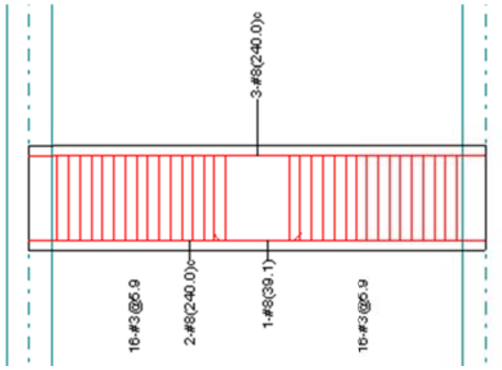


Figure 98: spBeam Column Line 8 Reinforcing Diagram

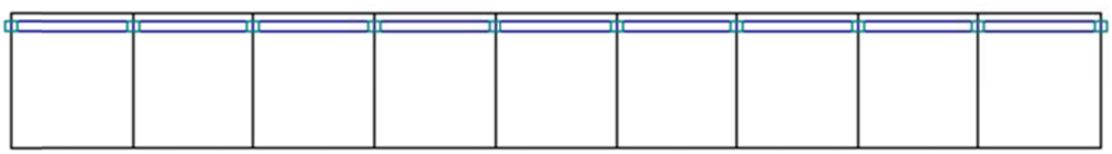


Figure 99: spBeam Model Column Line 9: Top View

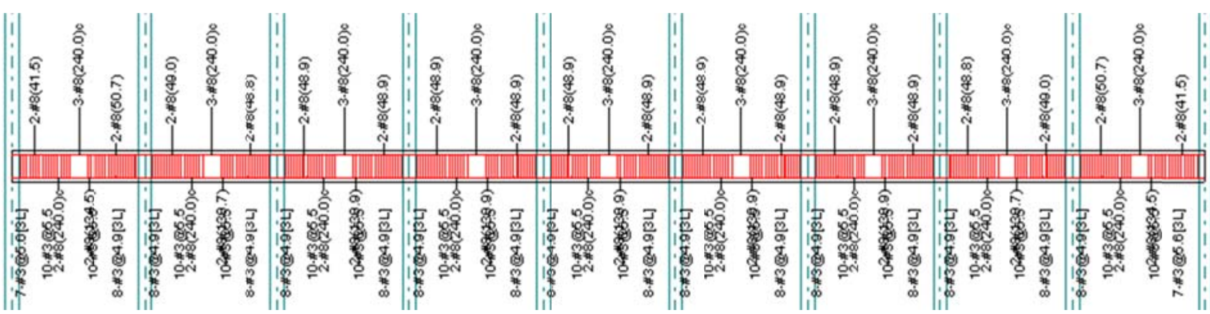


Figure 100: spBeam Column Line 9 Reinforcing Diagram

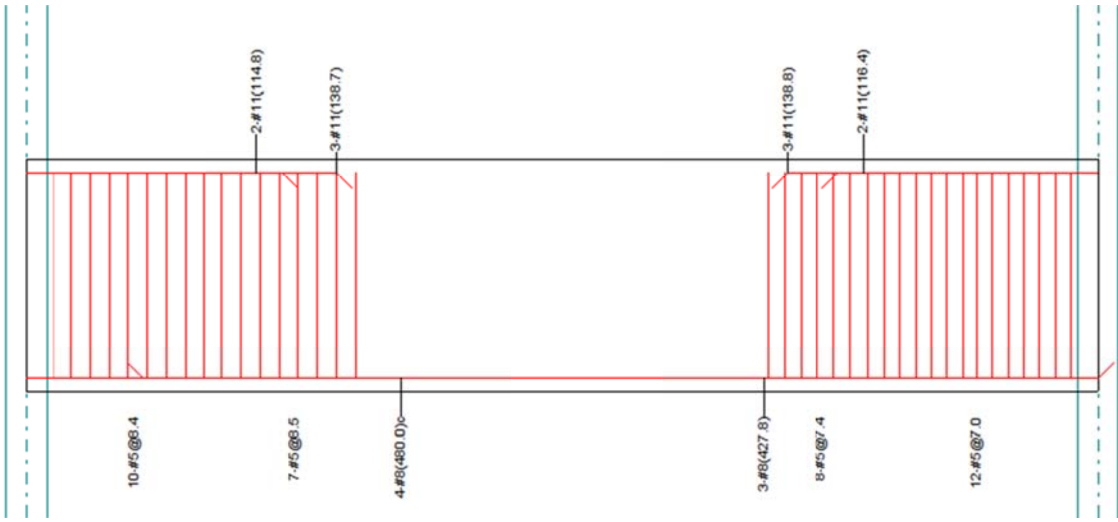


Figure 101: spBeam Transfer Girder 1 & 2 Reinforcing Diagram

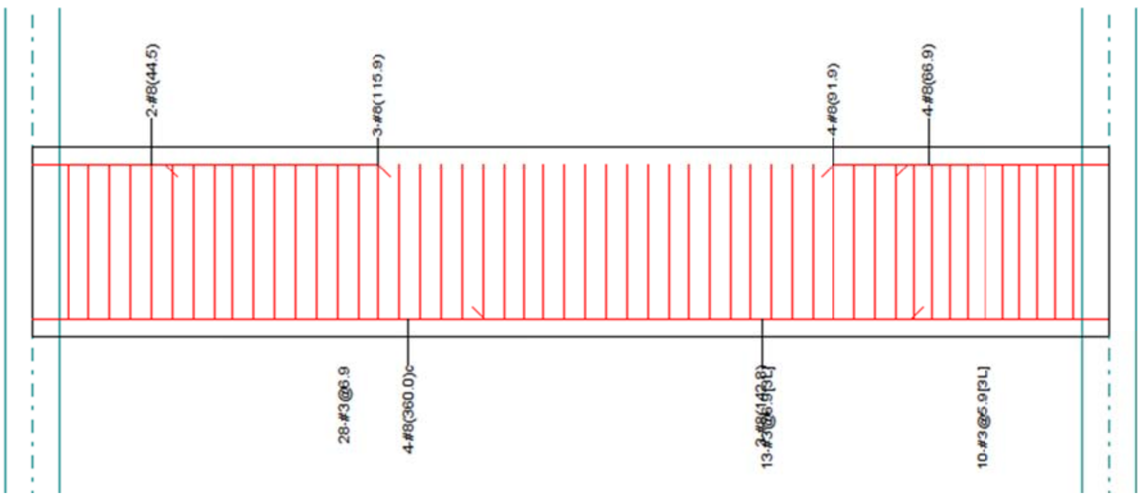


Figure 102: spBeam Transfer Girder 3 Reinforcing Diagram

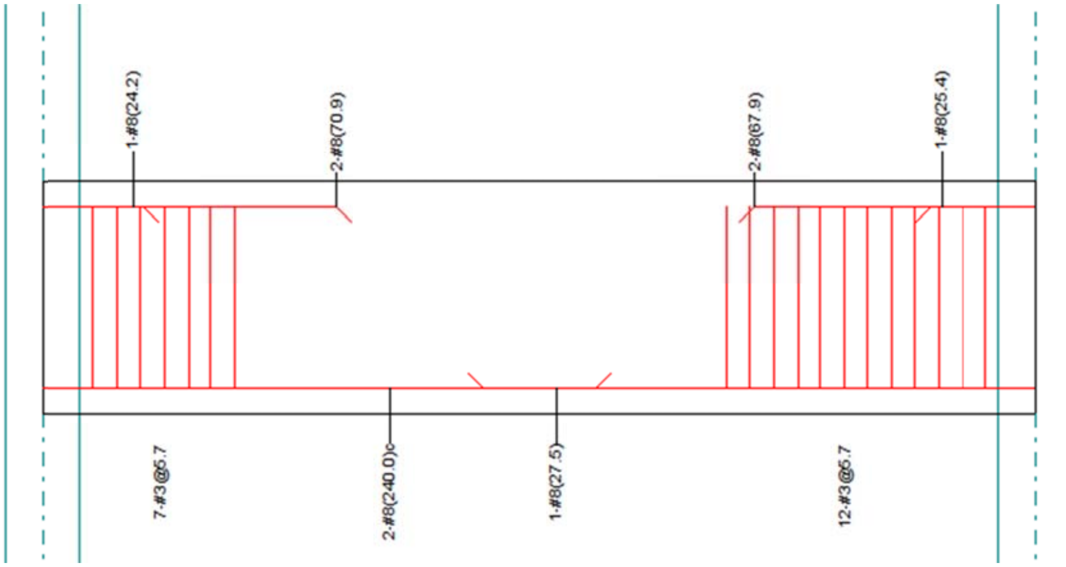


Figure 103: spBeam Transfer Girder 4 Reinforcing Diagram

Appendix D: spColumn Designs for Gravity Loads

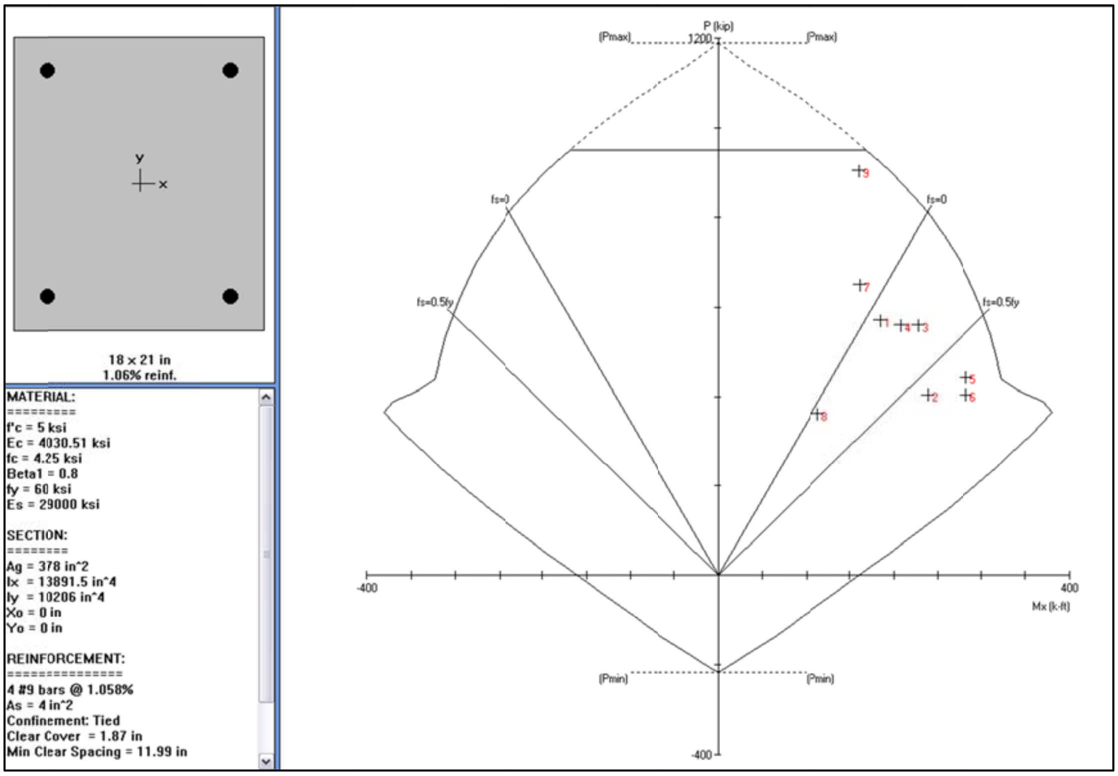


Figure 104: spColumn Typical Corner Column Design

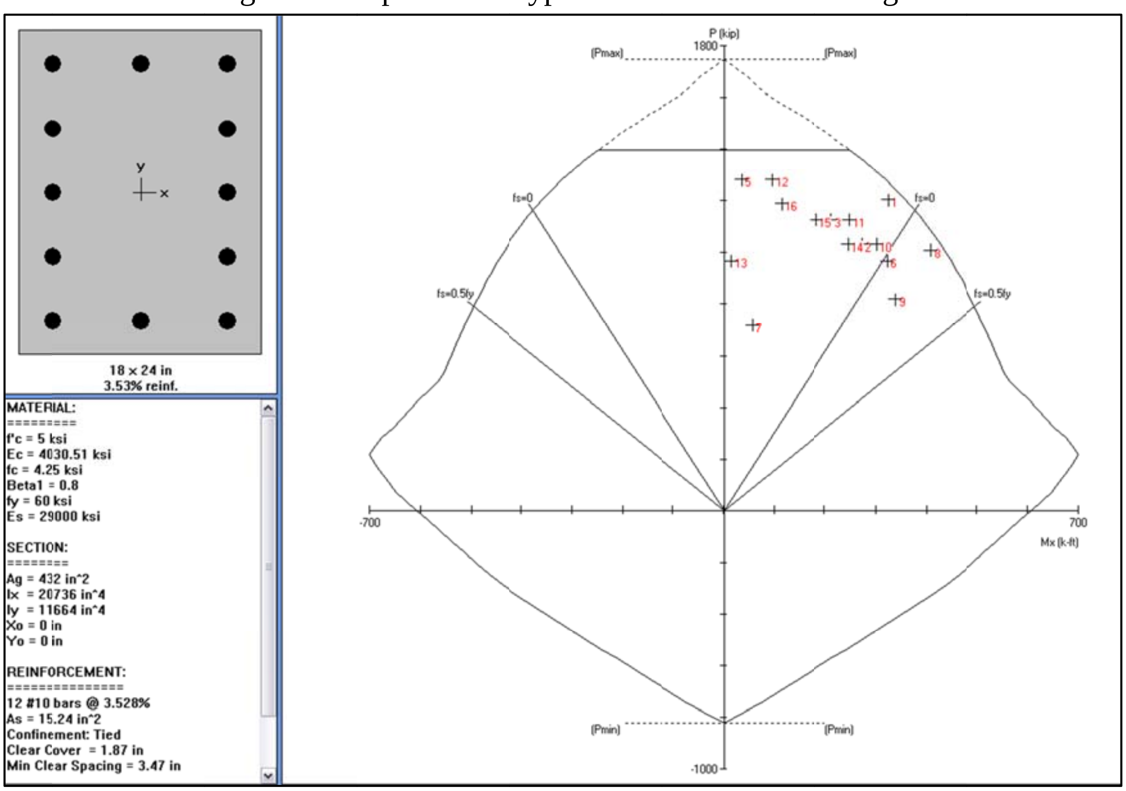


Figure 105: spColumn Typical Interior Column Design

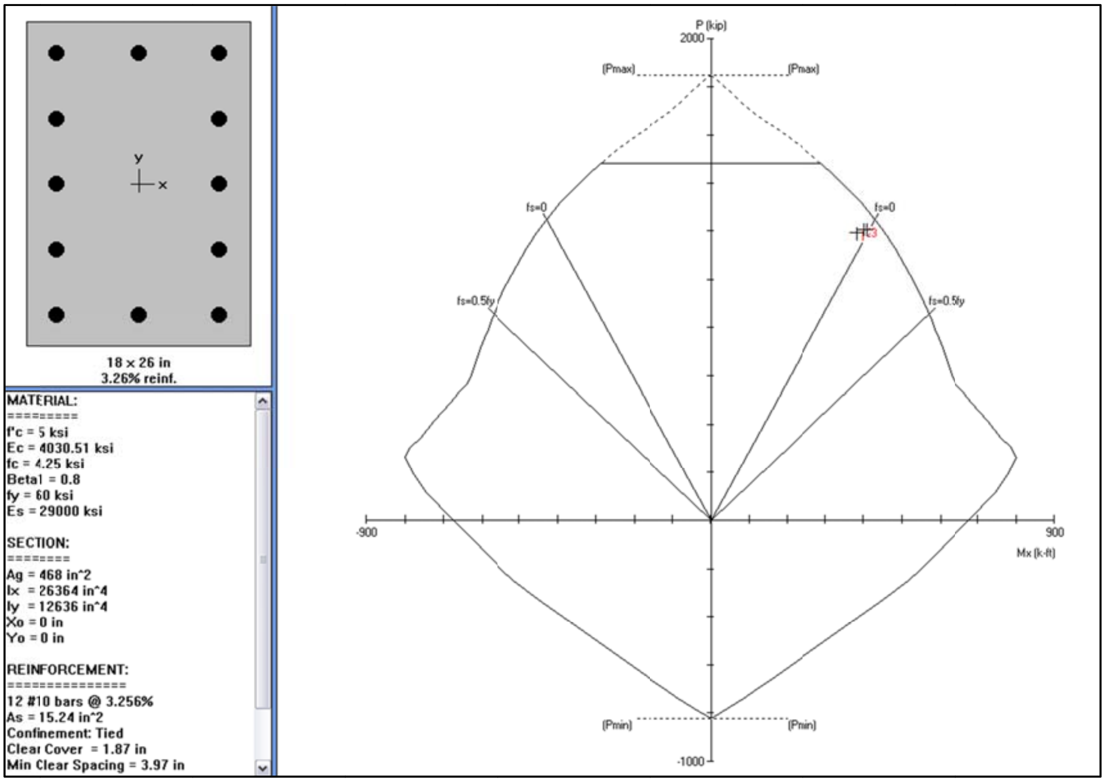


Figure 106: spColumn Big Interior Column Design (F.1-7, M.2-4, K.5-3)

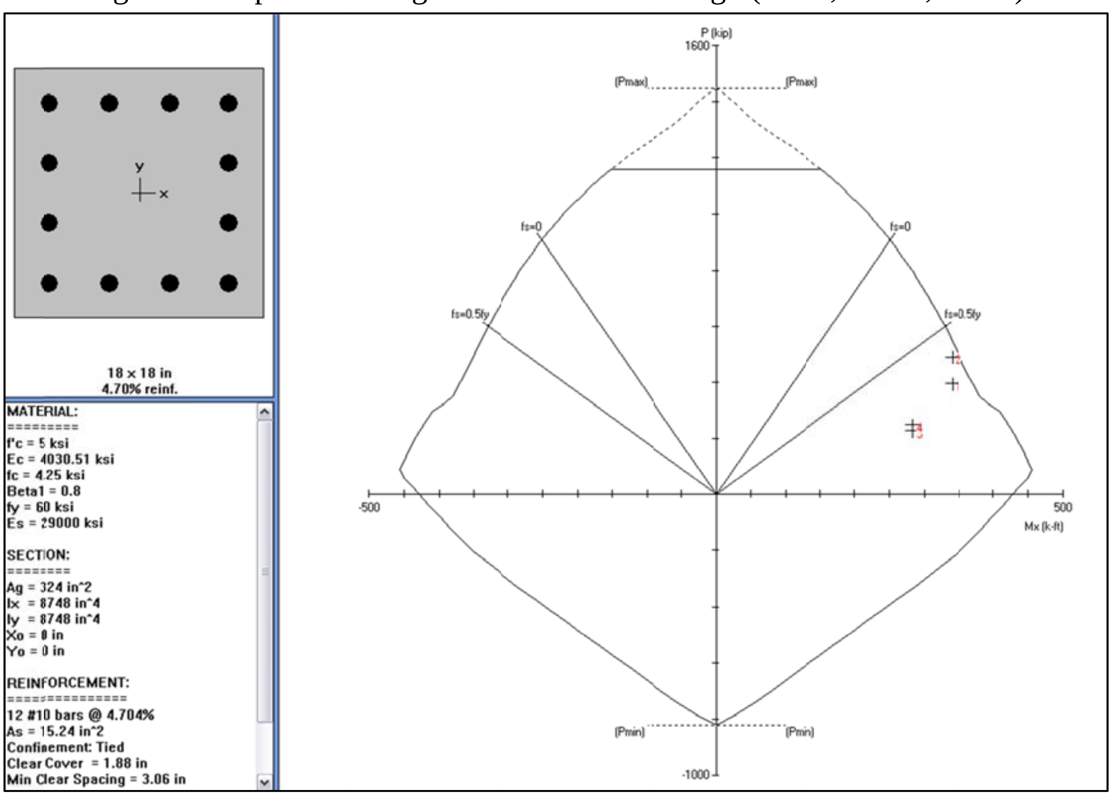


Figure 107: spColumn Typical Exterior Column Design – Above Splice at Level 4

Appendix E: spBeam Reinforcing Diagrams for Gravity and Lateral Loads

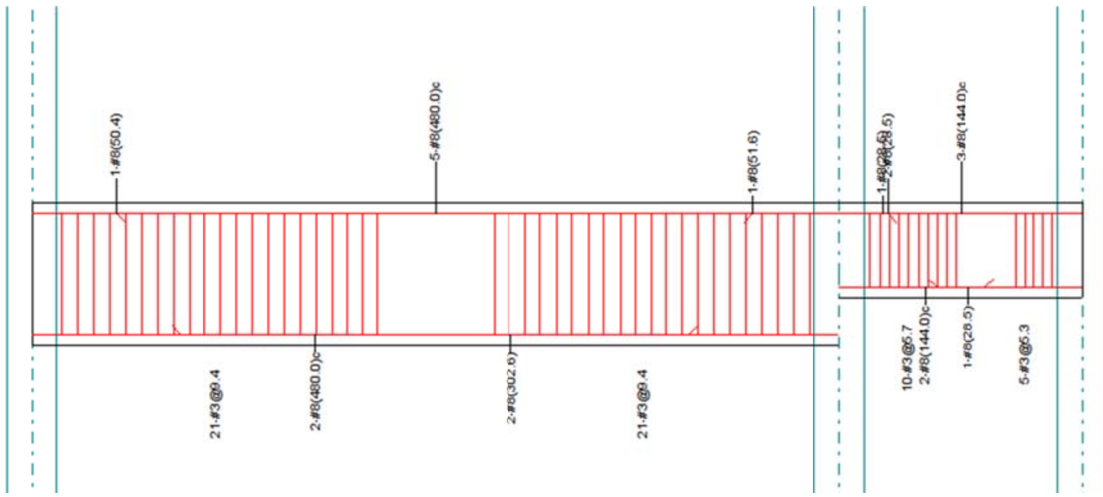


Figure 108: spBeam Column Line B Reinforcing Diagram (Lateral Loads)

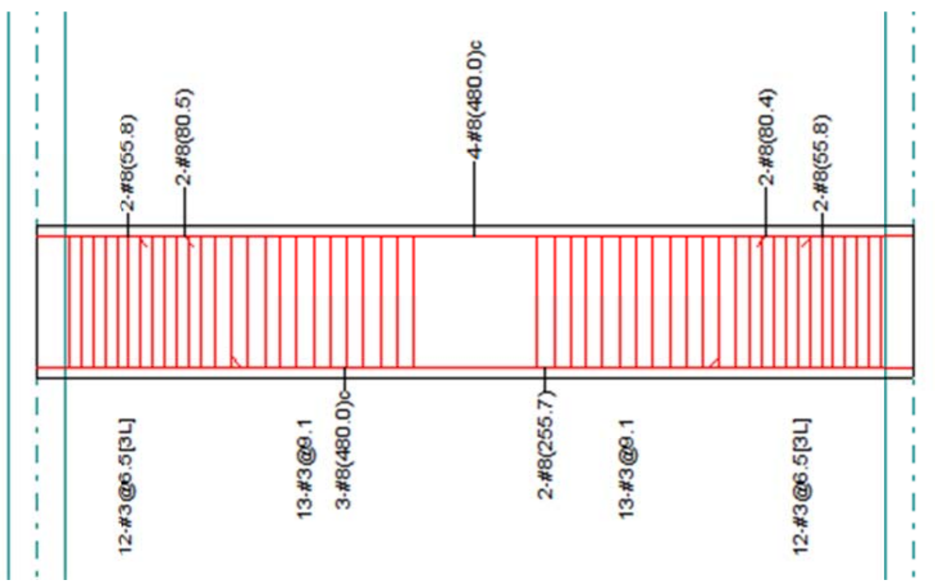


Figure 109: spBeam Column Line B.1 Reinforcing Diagram (Lateral Loads)

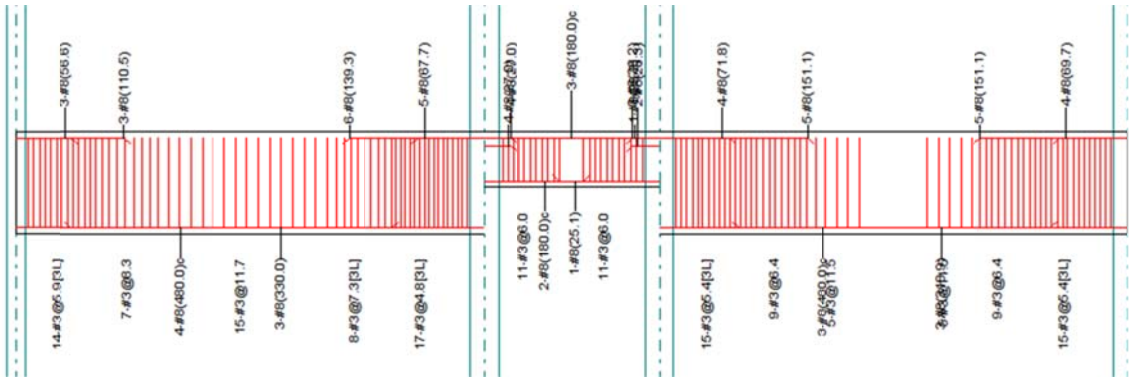


Figure 110: spBeam Column Line C Reinforcing Diagram (Lateral Loads)

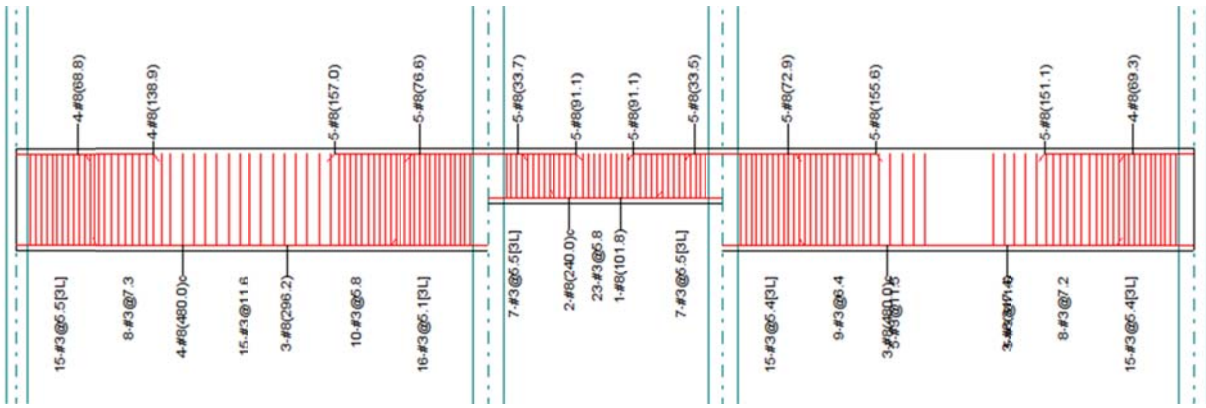


Figure 111: spBeam Column Line D Reinforcing Diagram (Lateral Loads)

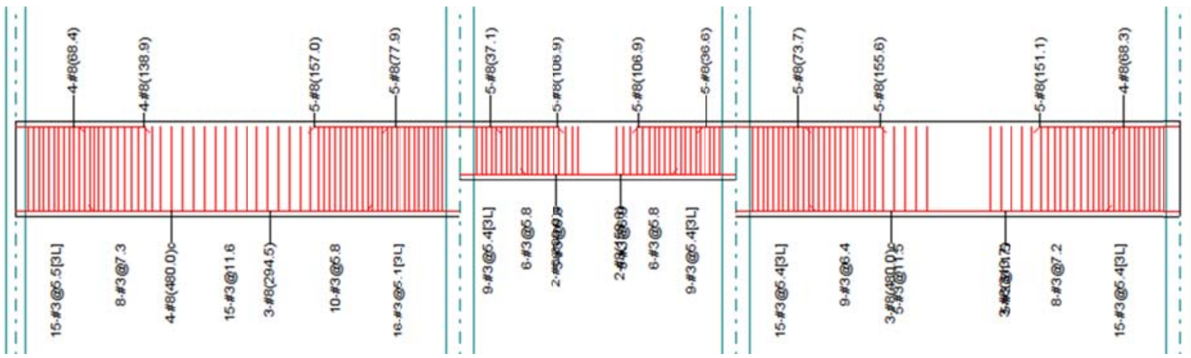


Figure 112: spBeam Column Line E Reinforcing Diagram (Lateral Loads)

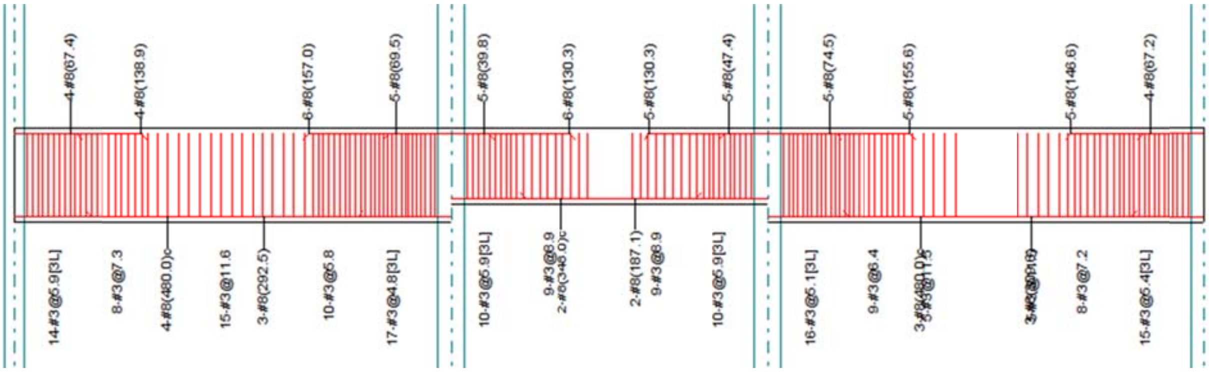


Figure 113: spBeam Column Line F Reinforcing Diagram (Lateral Loads)

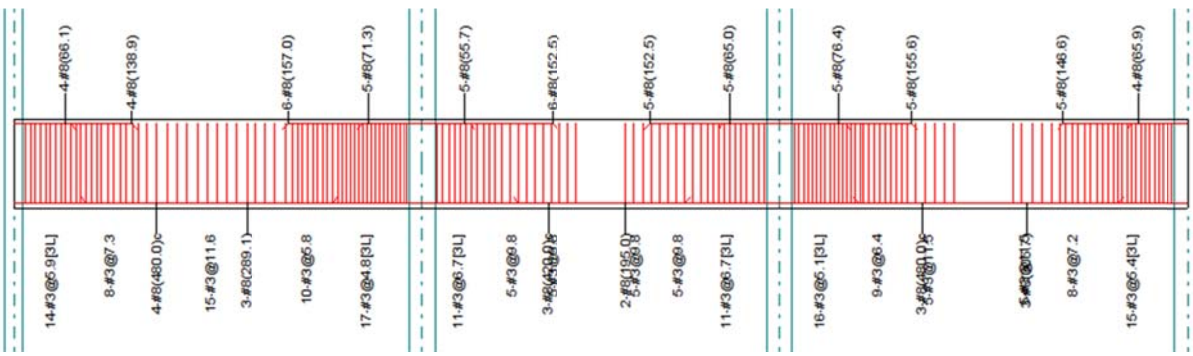


Figure 114: spBeam Column Line G Reinforcing Diagram (Lateral Loads)

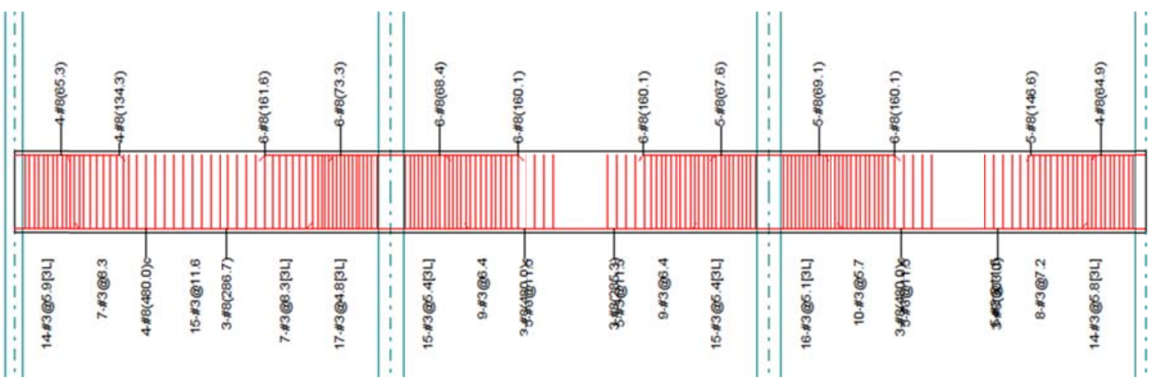


Figure 115: spBeam Column Line H Reinforcing Diagram (Lateral Loads)

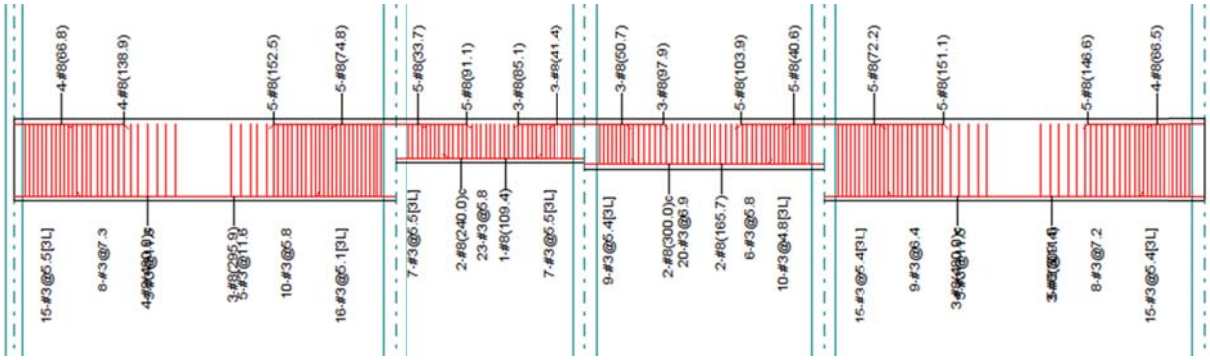


Figure 116: spBeam Column Line J Reinforcing Diagram (Lateral Loads)

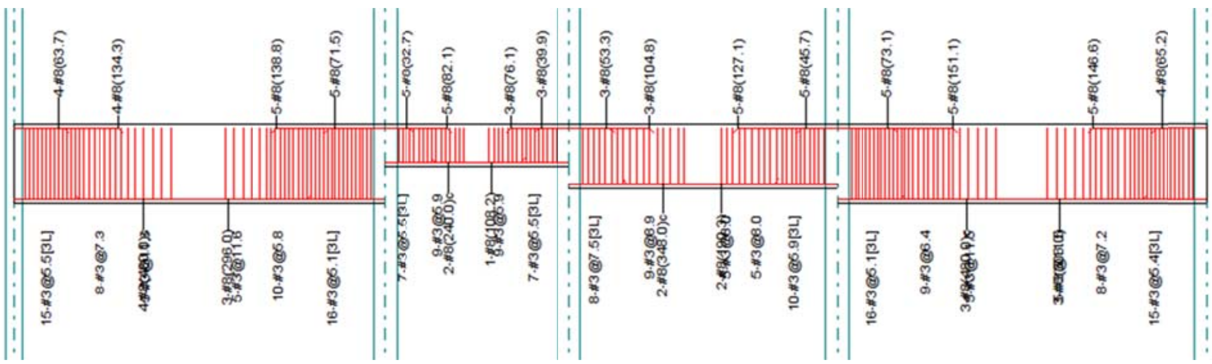


Figure 117: spBeam Column Line K Reinforcing Diagram (Lateral Loads)

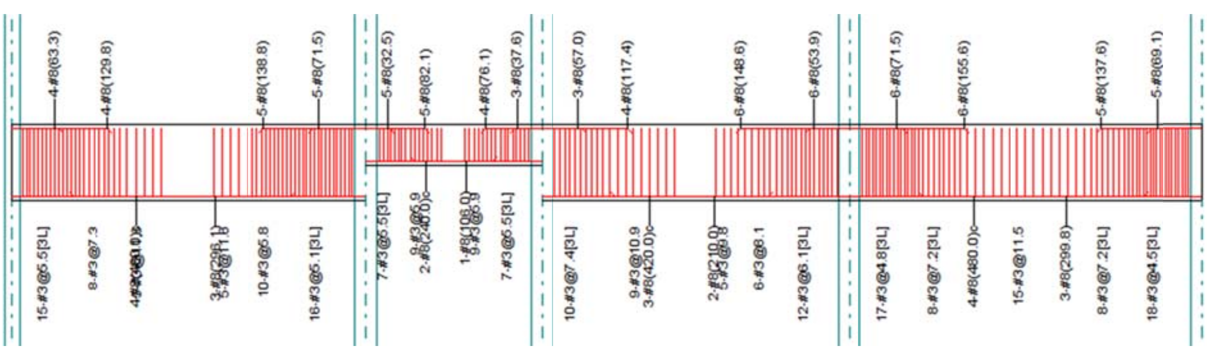


Figure 118: spBeam Column Line L Reinforcing Diagram (Lateral Loads)

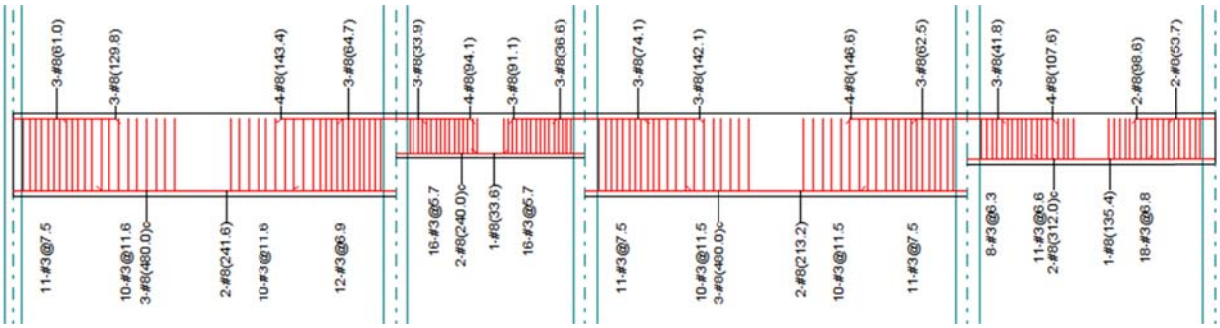


Figure 119: spBeam Column Line M Reinforcing Diagram (Lateral Loads)

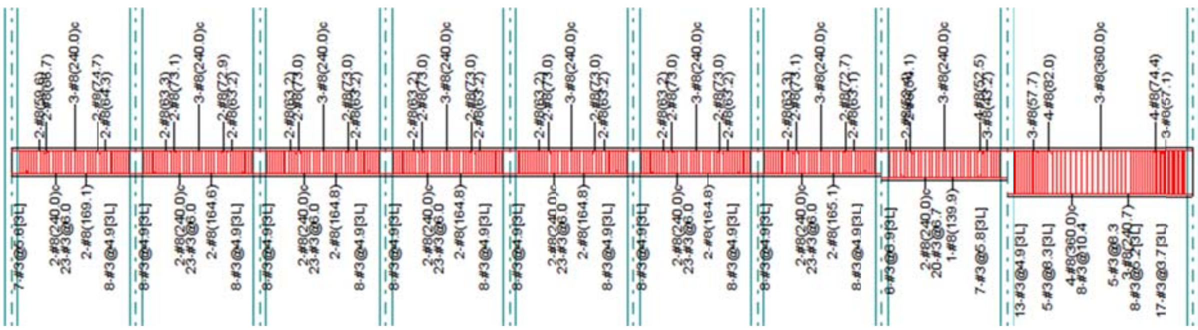


Figure 120: spBeam Column Line 1 Reinforcing Diagram (Lateral Loads)

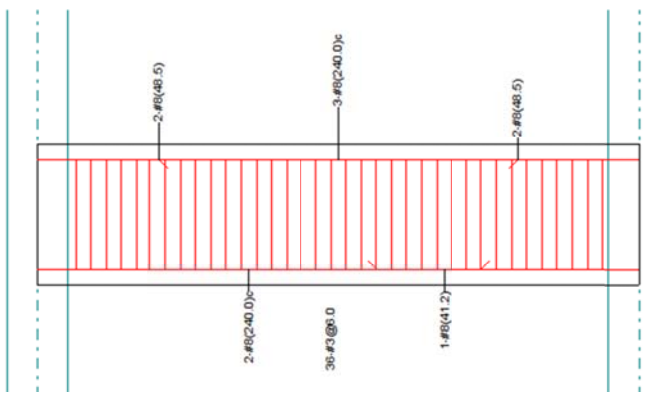


Figure 121: spBeam Column Line 8 Reinforcing Diagram (Lateral Loads)

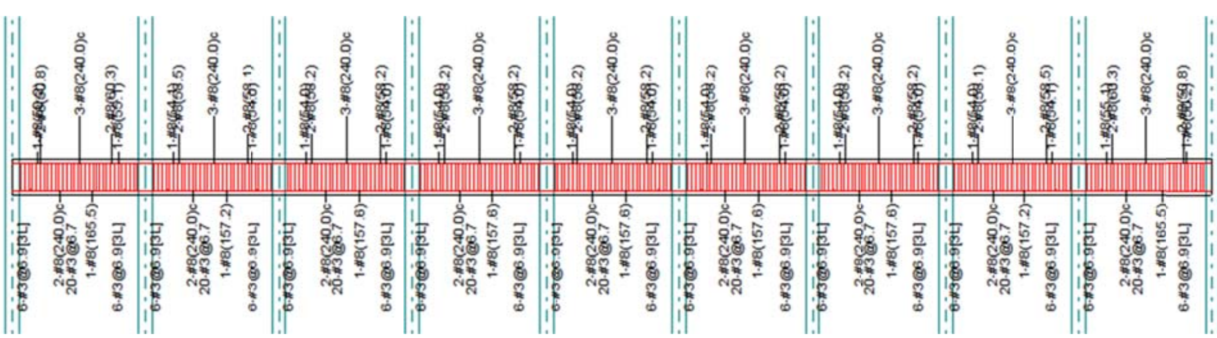


Figure 122: spBeam Column Line 9 Reinforcing Diagram (Lateral Loads)

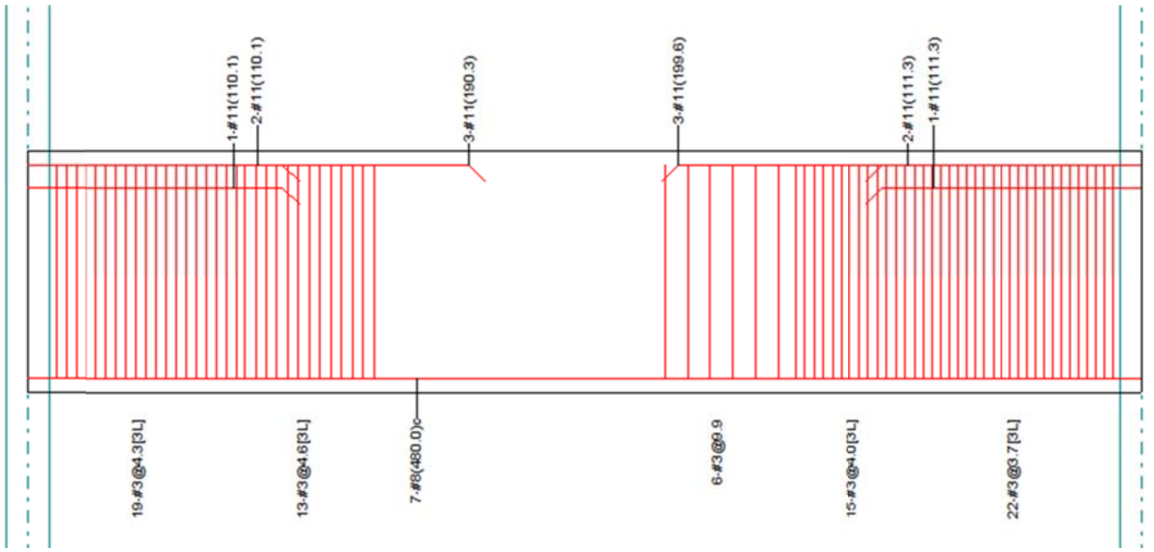


Figure 123: spBeam Transfer Girder 1 Reinforcing Diagram (Lateral Loads)

Appendix F: spColumn Designs for Gravity and Lateral Loads

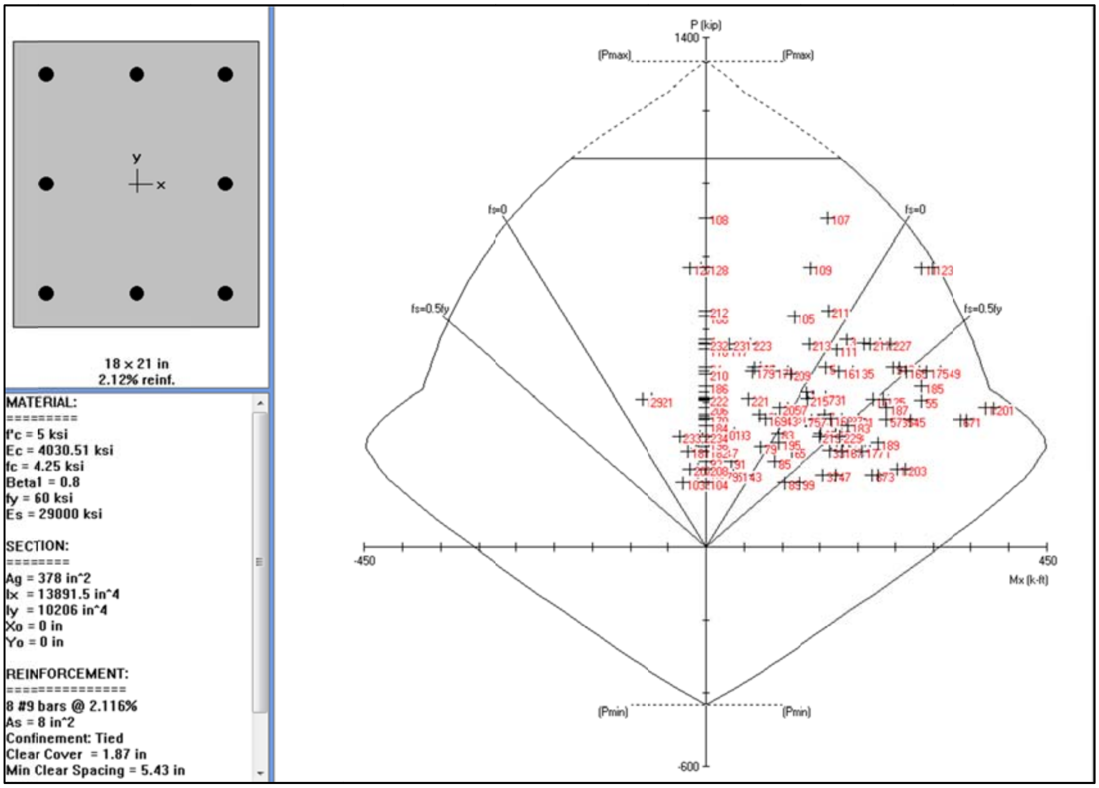


Figure 124: spColumn Typical Corner Column Design (Lateral Loads)

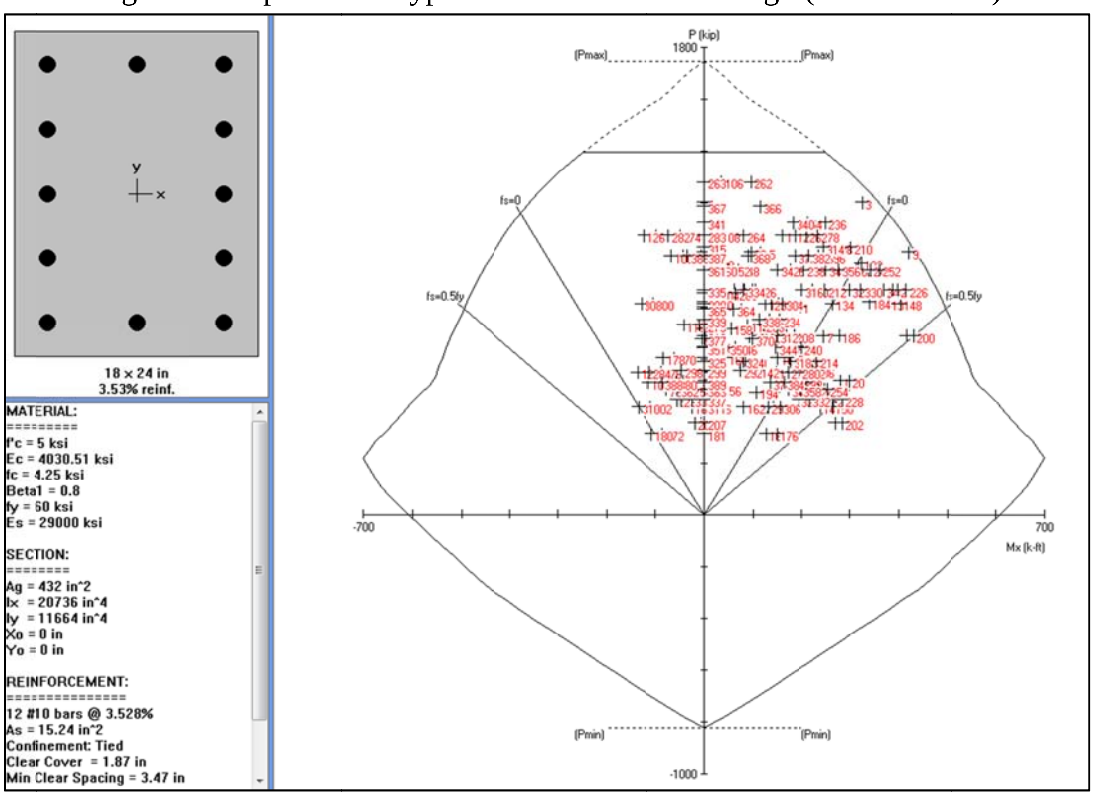


Figure 125: spColumn Typical Interior Column Design (Lateral Loads)